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LAND, SOILS AND GEOLOGY 7.0

7.0 LAND, SOILS AND GEOLOGY

7.1 Introduction

This chapter of the EIAR considers and assesses potential effects resulting from the Proposed Development; both the Mine Development and the further development of the Community Sports Complex.

The main elements of the Mine Development include the excavation at the Knocknacran West Open-Cast Mine, the restoration of the Knocknacran Open-Cast Mine and the continuation of use of the Knocknacran Processing Plant. It also includes the construction of a Cut-and-Cover Tunnel under R179 and a temporary diversion of the R179 during construction. The development requires the demolition of one residential house and three unoccupied houses and sheds.

Mining activities have been ongoing since 1988 at the adjacent Knocknacran Open-Cast Mine and since 2007 at the underground Drummond Mine. The gypsum mined from Knocknacran West is a replacement for the gypsum currently mined from Knocknacran which will be exhausted by 2027. Knocknacran Open-Cast Mine will undergo closure and restoration once Knocknacran West Open-Cast is operational, Drummond Mine is currently permitted to continue until 2032.

7.2 Legislative and Policy Context

This section addresses the legislation and guidance that has been considered when preparing this chapter, and key policy context relevant to land, soils and geology that has guided the focus of the assessment.

7.2.1 Legislation and Guidance

In addition to the Regulations that underpin the EIA process, this assessment has been made with cognisance to relevant guidance, advice and legislation, including, but not limited to:

- Department of the Environment, Climate and Communications - Policy Statement on Mineral Exploration and Mining Critical Raw Materials for the Circular Economy Transition, 2022;
- Directive 2014/52/EU of the European Parliament and of the Council;
- Planning Development Regulations, S.I. No. 600/2001 as amended;
- Gov.uk online guidance, Guidance on Land Contamination Risk Management (LCRM). Available at https://www.gov.uk/government/publications/land-contamination-risk-management-lcrm (2020). Uses a tiered approach to risk assessment, including preliminary risk assessment, generic quantitative risk assessment and detailed quantitative risk assessment;
- Irish Government. Guidelines for Planning Authorities and An Bord Pleanála on carrying out Environmental Impact Assessment (2018);
- European Union (Planning and Development) (Environmental Impact Assessment) Regulations 2018 (SI No. 296 of 2018) which amended the Planning and Development Act, 2000, and the Planning and Development Regulations, 2001. The 2014/52/EU Directive was transposed into Irish law through this Directive;

- European Commission. Environmental Impact Assessment of Projects: Guidance on the preparation of the Environmental Impact Assessment Report (2017);
- The EPA Guidelines on the Information to be Contained in Environmental Impact Assessment Reports (May 2022), which presents key topics of interest, high-level information on the interactions that should be considered in relation to EIA legislation, and overviews on the recommended approach to describing the baseline environment, completing impact assessments, describing effects, and addressing mitigation and monitoring;
- The EPA Advice Notes for Preparing Environmental Impact Statements (Draft, September 2017);
- CIRIA C741: Environmental Good Practice on Site (2015, Fourth Edition) in relation to source of impact and mitigation;
- Institute of Geologists of Ireland. Guidelines for the Preparation of Soils, Geology and Hydrogeology Chapters of Environmental Impact Statements (April 2013);
- European Union Directive 2011/92/EU as amended by Directive 2014/52/EU – these Directives required that certain private and public projects which are likely to have significant resultant environmental impacts are subject to a formalised Environmental Impact Assessment prior to their consent;
- The National Roads Authority Guidelines for the Creation, Implementation and Maintenance of an Environmental Operating Plan (2009) in relation to impact mitigation;
- The European Communities (Environmental Liability) Regulations 2008 (as amended);
- The National Roads Authority Guidelines on Procedures for Assessment and Treatment of Geology, Hydrology and Hydrogeology for National Road Schemes (2008) in relation to aspects to be considered and assessment approach (including relative receptor importance and cross discipline interactions);
- The Environmental Protection Agency Act 1992 and the Protection of the Environment Act 2003, as amended, which detail the requirements associated with general pollution control and activities that come under integrated pollution prevention and control;
- These Regulations (SI 547/2008) transpose EU Directive 2004/35/CE on environmental liability with regard to the prevention and remedying of environmental damage. The purpose of these Regulations is to establish a framework of environmental liability based on the 'polluter-pays' principle, to prevent and remedy environmental damage. The EPA is designated as the competent authority for all aspects of these Regulations;
- Geological Survey Ireland. Geological Heritage Guidelines for the Extractive Industry. January 2008; and
- Guidance for Pollution Prevention (GPPs) – these guidance documents provide environmental good practice guidance for the UK including for activities such as oil and chemical storage, works in or near water, works on construction sites, and dealing with spills and pollution incidents.

Relevant statutory instruments in the context of the protection of groundwater, surface water and geology:

- S.I. No. 272/2009 – European Communities Environmental Objectives (Surface Waters) Regulations 2009, as amended; and
- S.I. No. 9/2010 – European Communities Environmental Objectives (Groundwater) Regulations) 2010, as amended.

Relevant statutory instruments in the context of mining/quarrying include:

- Minerals Development Act 1940 (as amended); and
- Mines and Quarry Act 1965 (7 of 1965).

7.2.2 Relevant Planning Objectives

The Monaghan County Development Plan 2019 - 2025 acknowledges that there is an increasing demand for aggregates and that new areas for extraction of aggregates and minerals will be needed in the county. To address this the Council notes that planning policies should be carefully constructed to avoid adverse effects on aggregate resources and related extractive industries, and that the proposed plans should be developed in a sustainable manner not to cause adverse effects.

Monaghan County Development Plan 2019-2025

The Monaghan County Development Plan (Plan) provides an overall strategy for the proper planning and sustainable development of County Monaghan over the timescale of the Plan. Spatial planning through the development plan policies endeavours to achieve balance between the common good and the interests of those individuals.

Section 4.8 of the Plan acknowledges that the significant natural resources of the county make an important contribution to the economy, and it is important for these to be safeguarded for future use whilst also ensuring that the impacts on the environment and communities are acceptable.

Policy ERP1: To safeguard for future extraction all identified locations of major mineral deposits in the County;

Policy ERP2: To promote development involving the extraction of mineral reserves and their associated processes, where the Planning Authority is satisfied that any such development will be carried out in a sustainable manner that does not adversely impact on the environment or on other land uses. Consideration in this regard shall be given to the impact of the development on the local economy;

Section 15.25 of the plan identifies four policies to which the extractive industries (generally in the context of aggregate extraction) within the county are to have regard for;

Policy EIP1: To require all applications for extractive development to submit the following as part of the planning applications; a) Map detailing total site area, area of excavation, any ancillary proposed development and nearest dwelling and/or any other development within 1 km of the application site. b) Description of the aggregate to be extracted, method of extraction, any ancillary processes (crushing etc), equipment to be used, stockpiles, storage of soil and overburden and storage of waste materials. c) Total and annual tonnage of extracted aggregates expected life time of the extraction, maximum extent and depth of working and a phasing programme. d) Details of water courses, water-table depth and hydrological impacts, natural and cultural heritage impacts, traffic impact and waste management. e) Assessment of cumulative impact when taken with any other extractive operations in the vicinity. f) Likely environmental effects, proposed mitigation measures and restoration and after care proposals;

Policy EIP2: To prohibit extractive development within an area of primary or secondary amenity, Special Protection Area (SPA's), Special Area of Conservation (SAC's), Natural Heritage Area/pNHA (NHA's), Architectural Conservation Area (ACA's) or on or near protected structures unless in exceptional circumstances where the Planning Authority is satisfied that the need for the resource outweighs the environmental impact;

Policy EIP3: To restrict development proposals located in close proximity to existing extractive sites of significant resource potential where such developments would limit future exploitation; and

Policy EIP4: To restrict extractive developments that may have a detrimental impact on the natural or built environment or matters of acknowledged public importance including the use of public rights of way.

In their County Development Plan 2013 - 2019 one of Monaghan County Council's objectives was to identify sites of geological importance. A study conducted in 2013 as part of the Irish Geological Heritage Programme (the Geological Survey of Ireland (GSI) in conjunction with the Monaghan County Heritage Officer) identified 20 locally important sites in the county and classified them as County Geological Sites (CGS), one of which is CGS 10, Knocknacran Gypsum Mine.

Monaghan County Council policies relevant to this assessment of geology (refer to Section 6.12 of the 2019 – 2025 County Development Plan) include:

Policy GEP1: To promote awareness of and access to sites of geological interest in consultation with landowners (where appropriate) and on recommendations regarding safety with GSI;

Policy GEP2: Where a proposed development is likely to impact on the setting or integrity of a CGS listed in the Monaghan County Development Plan 2019 - 2025 the Geological Survey of Ireland hall be consulted;

Policy GEP3: To protect from inappropriate development and maintain the integrity and conservation value of those features in areas of geological interest that are listed in the plan, or any sites proposed by the Department of the Environment, Heritage and the Gaeltacht or Geological Survey of Ireland during the lifetime of the plan;

Policy GEP4: To contribute towards the appropriate protection and maintenance of the character, integrity and conservation value of the features or areas of geological interest; and

Policy GEP5: To promote CGS15 Rockorry - Cootehill ribbed Moraine and CGS16 Scotshouse – Redhills cross cutting ribbed moraines as unique landscapes as per the recommendations of the Geological Survey of Ireland.

7.3 Assessment Methodology and Significance Criteria

7.3.1 Technical Scope

The technical scope of this assessment is to consider the potential impacts and effects on land, soils and geology that can be reasonably foreseen as consequences of the normal construction, operation, and closure of the Proposed Development, where relevant. The assessment considers the potential sources of change resulting from Proposed Development activities detailed in the project description (Chapter 3.0).

The potential for loss of agricultural soils will be considered, as will the potential to impact geologically important sites and land quality. Associated secondary potential impacts of changes to land quality on human health are also considered. It should be noted that this assessment does not, however, constitute a contaminated land risk assessment, or detailed quantitative human health risk assessment.

The potential effects associated with hydrological and hydrogeological receptors are considered in Chapter 8.0 (Water). The effects of the Proposed Development on population and human health are addressed in Chapter 5.0 (Population & Human Health), although as noted above the potential effects of land quality on human health are considered in the current chapter. Any secondary effects on ecology or biodiversity due to changes in land quality or habitat removal are considered in Chapter 6.0 (Biodiversity).

7.3.2 Geographical and Temporal Scope

For the purposes of this assessment, the geographical Study Area has used a 2 km offset from the Application Site boundary as this incorporates both underground and above ground activities relating to both mining and non-mining related activities and is considered a conservative boundary to use, Figure 7.1. A 2 km offset from this boundary has been used in line with the IGI's "Guidelines for the Preparation of Soils, Geology and Hydrogeology Chapter of Environmental Impact Statements" (2013). The Proposed Development incorporates both the Mine Development areas and the Community Sports Complex area. Both the proposed Mine Development and Community Sports Complex lie within the Application Site (Figure 7.2).

The temporal scope of the assessment covers the construction, operation, closure, and restoration phases of the Proposed Development. Temporally, the construction phase for the Community Sports Complex is ca. 2 years, while the Mine Development is ca. 1 year, there will be overlap of 1 year between these development phases. The operational phase for the Mine Development is ca. 30-35 years, depending on market conditions while the Community Sports Complex is in operation in perpetuity. The closure phase of the Mine Development begins after the operational phase has ceased.

Once the Knocknacran West Open-Cast Mine is operational, the existing Knocknacran Open-Cast Mine will undergo restoration, as extraction will have ceased. The existing underground Drummond Mine is currently permitted until 2032.

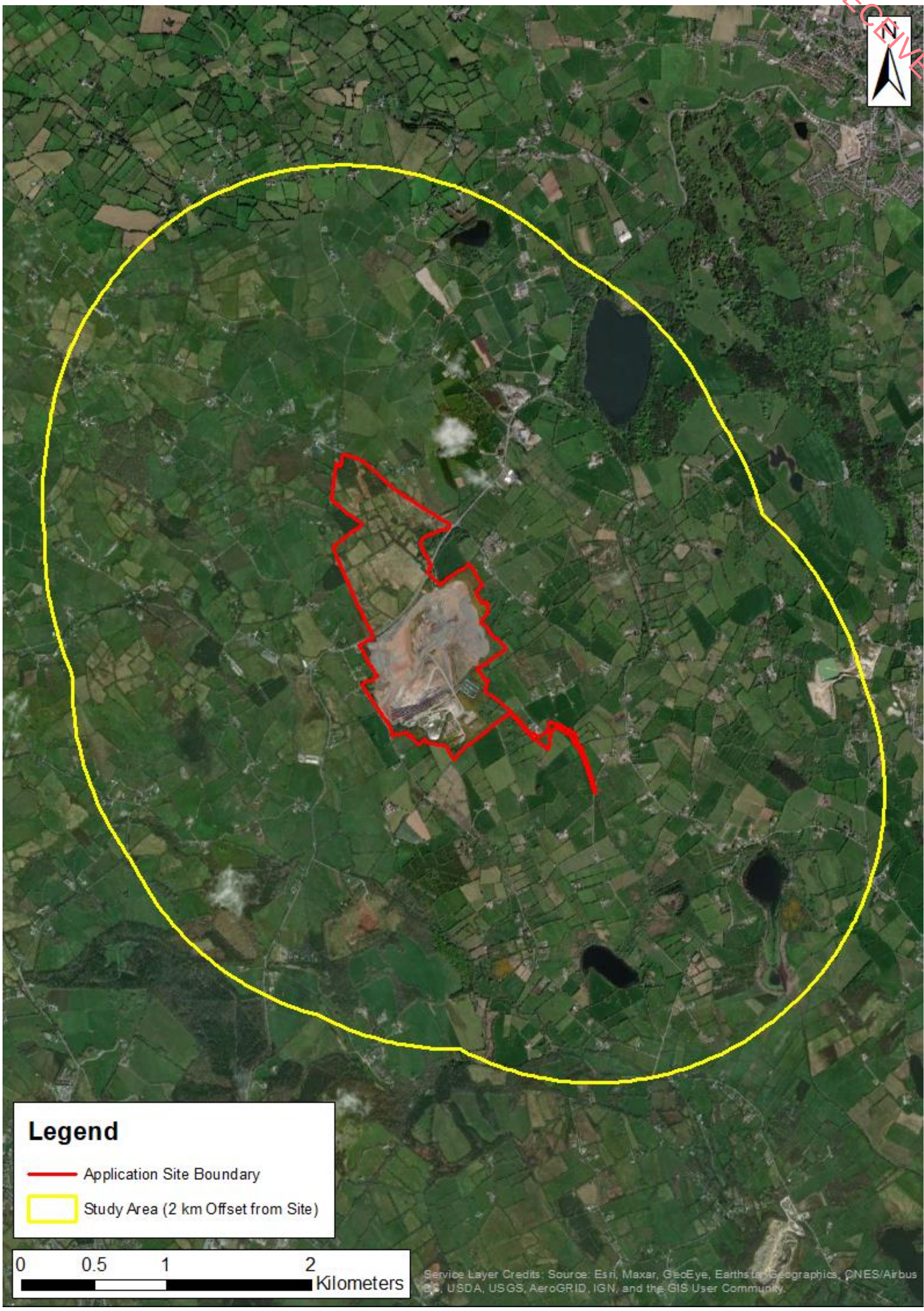

Figure 7.1: Site boundary and Study Area

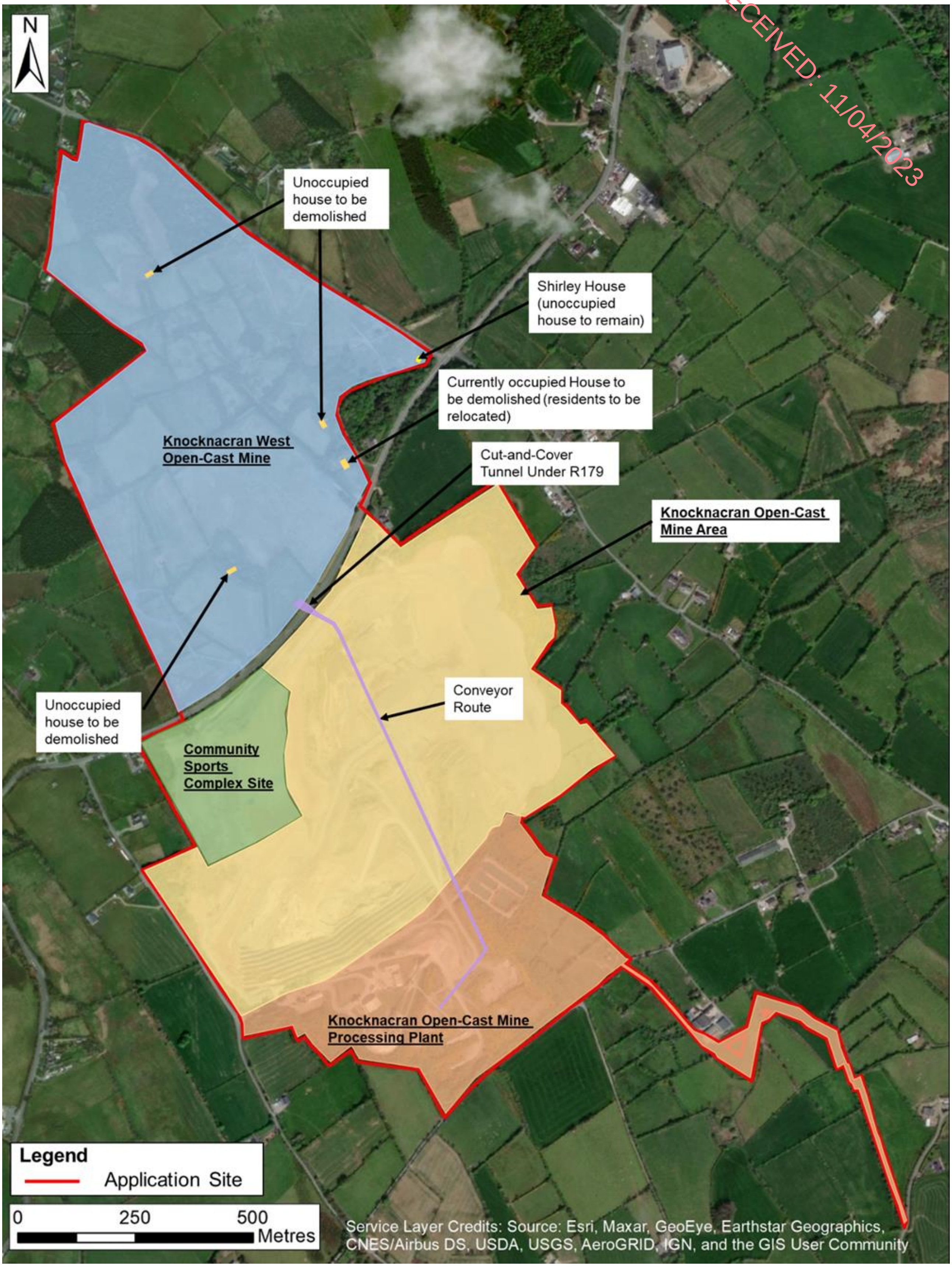

Figure 7.2: Extent of Proposed Development and activity areas within the Site

7.3.3 Qualitative Assessment Method

This section presents the method used to assess the impacts and effects of the Proposed Development on soils, land and geology, and to secondary associated human health receptors. It establishes the stages of the assessment, and the qualitative criteria used to assess impact magnitude and determine the level of effect significance.

The assessment of potential effects has been undertaken based on the EPA's Guidelines on the Information to be Contained in EIARs (EPA, 2022), using the qualitative assessment method outlined below, and is supported by the baseline condition information, and the proposed development design. A Construction Environmental Management Plan (CEMP) will be developed on foot of a grant of permission being received, following discussions and agreement with the Local Authority. Versions of this document will be further developed by the Contractor as the Project goes through the construction phase. The assessment follows a staged approach. A summary of the stages involved is included below:

1) Confirm baseline conditions – determine baseline and develop conceptual site model by consideration of available records and data sets, site reports and published information.
2) Confirm the key receptors and their value/importance.
3) Qualitatively characterise the magnitude of impacts on the receptors – describe what potential changes could occur to each receptor as a result of the Proposed Development, identify source-pathway receptor linkages, and assign the magnitudes of impact. This stage takes into account embedded design mitigation, good practice in construction environment management and pollution prevention.
4) Determine the initial effect significance of each potential impact on each sensitive receptor.
5) Consider the need for additional mitigation if it is considered necessary to reduce the initial magnitude of the impact and associated effect significance further.
6) Assess the residual impact magnitude and residual effect significance after all mitigation is applied.

Stages 1 and 2 have been completed using published literature and guidance and available information specific to the Proposed Development, which is presented in Chapter 3.0. For the identification of receptor value/importance that completes Stage 2, and for the description of impact magnitude (Stage 3), a common framework of assessment criteria and terminology has been used based on the EPA's Guidelines on the Information to be Contained in EIARs (EPA, 2022), with some modifications made to increase clarity. The descriptions for value (sensitivity) of receptors are provided in Table 7.1 and the descriptions for magnitude of impact are provided in Table 7.2.

The potential for an impact to occur at a receptor has been determined using the understanding of the baseline environment and its properties and consideration of whether there is a feasible linkage between a source of impact and each receptor (i.e. a conceptual site model). This follows the method of preliminary risk assessment that is widely presented in some of the guidance documents listed in Section 7.2.1.

Table 7.1: Environmental Value (Sensitivity) and Descriptions

Value (sensitivity) of receptor / resource	Typical description
High	High importance and rarity, national scale, and limited potential for substitution. For example: • Attribute has a high quality, significance or value on a Global/European/National designation; • Large volumes of nationally or locally important peat; • Well drained and highly fertile soils; • Proven economically extractable mineral resource; and • Human health.
Medium	Medium or high importance and rarity, regional scale, limited potential for substitution. For example: • Regionally important sites; • Sub-economic extractable mineral resource; and • Moderately drained and/or moderate fertility soils.
Low	Low or medium importance and rarity, local scale. For example: • Locally designated sites; • Uneconomically extractable mineral resource; and • Poorly drained and/or low fertility soils
Negligible	Very low importance and rarity, local scale.

Table 7.2: Magnitude of Impact and Typical Descriptions

Magnitude of impact (change)		Typical description
High	Adverse	Loss of resource and/or quality and integrity of resource; severe damage to key characteristics, features or elements. Significant harm to human health - death, disease, serious injury, genetic mutation, birth defects or the impairment of reproductive functions. Significant harm to buildings/infrastructure/plant - Structural failure, substantial damage or substantial interference with any right of occupation.
	Beneficial	Large scale or major improvement of resource quality; extensive restoration; major improvement of attribute quality.
Medium	Adverse	Loss of resource, but not adversely affecting the integrity; partial loss of/damage to key characteristics, features or elements.
	Beneficial	Benefit to, or addition of, key characteristics, features or elements; improvement of attribute quality.
Low	Adverse	Some measurable change in attributes, quality or vulnerability; minor loss of, or alteration to, one (maybe more) key characteristics, features or elements.
	Beneficial	Minor benefit to, or addition of, one (maybe more) key characteristics, features or elements; some beneficial impact on attribute or a reduced risk of negative impact occurring.
Negligible	Adverse	Very minor loss or alteration to one or more characteristics, features or elements.

	Beneficial	Very minor benefit to or positive addition of one or more characteristics, features or elements.

The assessment of magnitude of impact considers whether the change that causes the impact is positive or negative, and whether the impact is direct or indirect, short, medium, or long-term, temporary or permanent, and if it is reversible.

For the purposes of this assessment, a direct impact is one that occurs as a direct result of the Proposed Development and is likely to occur at or near the development itself. Indirect impacts (or secondary/tertiary impacts) are those where a direct impact on one receptor has another knock-on impact on one or more other related receptor(s) (e.g. the Proposed Development results in a change in land quality, which then has an indirect impact on human health). Indirect impacts can occur within the study area or away from the Proposed Development.

For the purposes of this assessment, the following definitions of duration have been used:

- Temporary – effect likely to last less than 1 year without intervention (i.e. less than the construction phase);
- Short term – effect likely to last 1 to 7 years without intervention;
- Medium term – effect likely to last 7 to 15 years without intervention;
- Long term – effect likely to last 15 to 60 years without intervention; and
- Permanent – effect likely to last over 60 years without intervention.

An irreversible impact is defined as a change to the baseline that would not reverse itself naturally. Such impacts will usually be long-term and irreversible, such as the removal of best and most versatile agricultural soils. A reversible impact is defined as a change to the baseline conditions that would reverse naturally once the source of the impact is exhausted or has stopped.

7.3.4 Significance Criteria

The approach followed to derive effects significance from receptor value and magnitude of impacts (Stage 4) is shown in Table 7.3. Where Table 7.3 includes two significance categories, reasoning is provided in the text if the lower of the two significance categories is selected. A description of the significance categories used is provided in Table 7.4.

Table 7.3: Significance Matrix

	Magnitude of Impact (Degree of Change)				
Environmental value (Sensitivity)		Negligible	Low	Medium	High
	High	Slight	Slight or moderate	Moderate or large	Profound

	Medium	Imperceptible or slight	Slight or moderate	Moderate	Large or profound
	Low	Imperceptible	Slight	Slight	Slight or moderate
	Negligible	Imperceptible	Imperceptible or slight	Imperceptible or slight	Slight

Table 7.4: Significance Categories and Typical Descriptions

Significance Category	Typical Description
Profound	An effect which obliterates sensitive characteristics.
Large	An effect which, by its character, magnitude, duration or intensity alters a significant proportion of a sensitive aspect of the environment.
Moderate	An effect that alters the character of the environment in a manner that is consistent with existing and emerging baseline trends.
Slight	An effect which causes noticeable changes in the character of the environment without affecting its sensitivities.
Imperceptible	An effect capable of measurement but without significant consequences.

Residual adverse effects of **Large or Profound significance are considered to be Significant** for the purposes of this assessment.

If required following the assessment of the level of effect significance, additional mitigation measures are presented that will be used to avoid, prevent or reduce the magnitude of the potential impact (Stage 5). The significance of the effect taking into account the additional mitigation is then assessed (Stage 6) to give the residual effect significance.

The effects of the Proposed Development are also considered cumulatively with those that could foreseeably result from other known developments in the assessment study area that are going through the planning process.

7.4 Baseline

This Section presents baseline information on land use, land quality, soils, and geology. Information about the water environment (including hydrogeology) is included in Chapter 8.0.

7.4.1 Topography

The area around the Site is dominated by a rolling (drumlin) topography with agriculture being the main activity. The highest point on the Site is ca. 70 m OD (Malin Datum) with the lowest being the floor of the Knocknacran open-cast mine at ca. 0 m OD (Malin Datum).

(Note that the mine operates on its own Mine Grid system, where all height values are to Poolbeg Datum + 1,000 m. Poolbeg is still used for historical reasons as the original drilling, mine plans etc. used Poolbeg. 1,000 m has been added to all elevations, so all elevations are positive (and do not have a negative sign) when mining takes place below 0 m OD. In comparison to Malin Datum, Poolbeg Datum is an additional + 2.6 m locally at the Site, i.e. Mine Grid system is + 1,002.6 m).

7.4.2 Land

According to the EPA 2022 Guidelines in referencing and describing 'land' it is clarified that the amended Directive introduces Land as a prescribed environmental factor; "Recital 9 gives context to this addition, showing that it relates to the issue of 'land take'. This change aligns the Directive with proceedings of the United Nations Conference on Sustainable Development (Rio de Janeiro, 2012) and with Commission strategy."

The Environment Directorate-General of the European Commission sets down policies in relation to myriad environmental factors including 'land' opening as follows;

"Land is a finite resource. It is subject to competing pressures from urbanisation, infrastructure, increased food, feed, fibre and fuel production and the provision of key ecosystem services.

But it's also a shrinking resource. Almost 1000 km^2 of agriculture or natural land disappears every year in the EU, as it is converted into artificial areas. More EU land is affected by degradation all the time, and ecosystem services are lost as a result.

This is a global problem. The EU contributes to land degradation in third countries, as we are a net "importer" of land embedded into imported products. Demand for areas to settle, grow food and biomass is rising around the world, and climate change is likely to impact on land demand, availability and degradation.

But the EU is taking action. The 2011 Road Map for Resource-Efficient Europe, part of Europe 2020 Strategy has the following aim: "By 2020, EU policies take into account their direct and indirect impact on land use in the EU and globally, and the rate of land take is on track with an aim to achieve no net land take by 2050"."

The surface working area, in which the Mine Development will take place is ca. 130.4 ha in size and includes the Knocknacran West Open-Cast Mine, Knocknacran Processing Plant, Knocknacran site and the discharge point to the River Bursk. The non-mining related surface works at the proposed Community Sports Complex site will take place in an area of ca. 8.6 ha.

7.4.3 Land Use

There are a number of historical maps available for the Study Area which have been considered in this assessment to characterise the land use, in addition to reconnaissance surveys carried out between 2018 and 2023, as follows:

- 6" historical map (1837-1842);
- 25'' OSI maps (1888-1913);
- 6" Cassini Map (1830s to 1930s);
- Historical Goggle Earth Imagery (between 2001 and 2023); and
- GSI's (2023) aggregate potential mapping online viewer (historical quarries layer).

A review of the 6" historical map (1837 - 1842) and the 25" inch map (1888 - 1913) shows the Study Area contained a mixture of scattered fields, sparse residential housing and farmsteads during these periods. Extractive industries are noted within the Study Area on the 6" historical map as follows:

- A brick kiln is noted within the east of the Knocknacran West site (Knocknacran East townland);
- A quarry is located to the north of the Knocknacran West site (Corduff townland);
- Two gravel pits are located adjacent to the church in Drumgoosat village (Drumgoosat townland);
- Two gravel pits are located west of Knocknacran West (Stranatona and Tonaneeve townlands);
- An area near to the north of Rahans Lough (east of Knocknacran site) notes gravel pits (Mason Lodge townland);
- A gravel pit is noted straddling the townlands of Mokeeran and Killygally, east of the Knocknacran site;
- A quarry is noted adjacent to the south of Descart Lough (Descart townland) to the south of the Knocknacran site; and
- A kiln (unknown type) is noted west of the Knocknacran West site (Lisnakeeny townland).

All of the pits and the quarry site occurred in areas noted to be sandstone or sandstone and shale bedrock.

The latter 25” 1888 – 1913 map notes the following within the Study Area:

- A smithy, west of the Knocknacran site (Drummond townland); and
- A disused quarry located to the north of the Knocknacran West site (Corduff townland).

The 6” Cassini map provides less detail than the 25” map for the Study Area, only the smithy is noted. Notably, these maps do not show any significant wooded areas near or within the Site and it is not until the 25” map and 6” Cassini map that a wooded area is noted within the Knocknacran West site in the eastern corner, as is still present today.

A review of the GSI’s aggregate potential mapper identifies one historical quarry within the Site boundary, noted to have occurred between 1975 and 1995 with the location pin over the current Plant Site on the Knocknacran Mine site. Within the wider Study Area the following are noted:

- Three historical pits are noted to have occurred adjacent to the church in Drumgoosat village (Drumgoosat townland), although no age is given these are likely to be synonymous with the pits noted in the 25” and 6” Cassini maps and which are most likely to have been extracting overburden sands or sandstone from the Kingscourt Sandstone Formation that occurs here;
- A further three pits are noted to the west of Knocknacran West, of unknown age in Tullylougherny, Toaneeve and Stranatona townlands. The Stranatona and Toaneeve pits appear to be synonymous with the gravel pits noted on the 6” 1837 – 1842 map;
- Four quarries are noted to the north of the Knocknacran West site in the townlands of Drumgoosat (noted as a Mid-Late 19th century sandstone quarry) and Corduff (one noted of three which quarries clay and shale for chimney pots);
- Three pits are noted to the east of the Knocknacran site, one straddles Mokeeran and Killgally townland (synonymous with the 6” 1837 – 1842 mapped gravel pit here) and two occur within the townland of Mason Lodge, north of Rahens Lough and are synonymous with the two noted on the 6” map from 1837 – 1842. A quarry (age 1975 – 1995) is also noted in the townland of Killgally;
- Three quarries (all aged 1975 – 1995, two described as brick pit at former Cormey mine) are noted to the southwest of Knocknacran, one in the townland of Enagh and two are in Cormey townland; and
- One quarry is noted to the southwest of the Knocknacran site in the townland of Mullantlavan, no age is noted with this quarry.

Figure 7.3 presents the locations of historical pits and quarries within the Study Area (GSI, 2023).

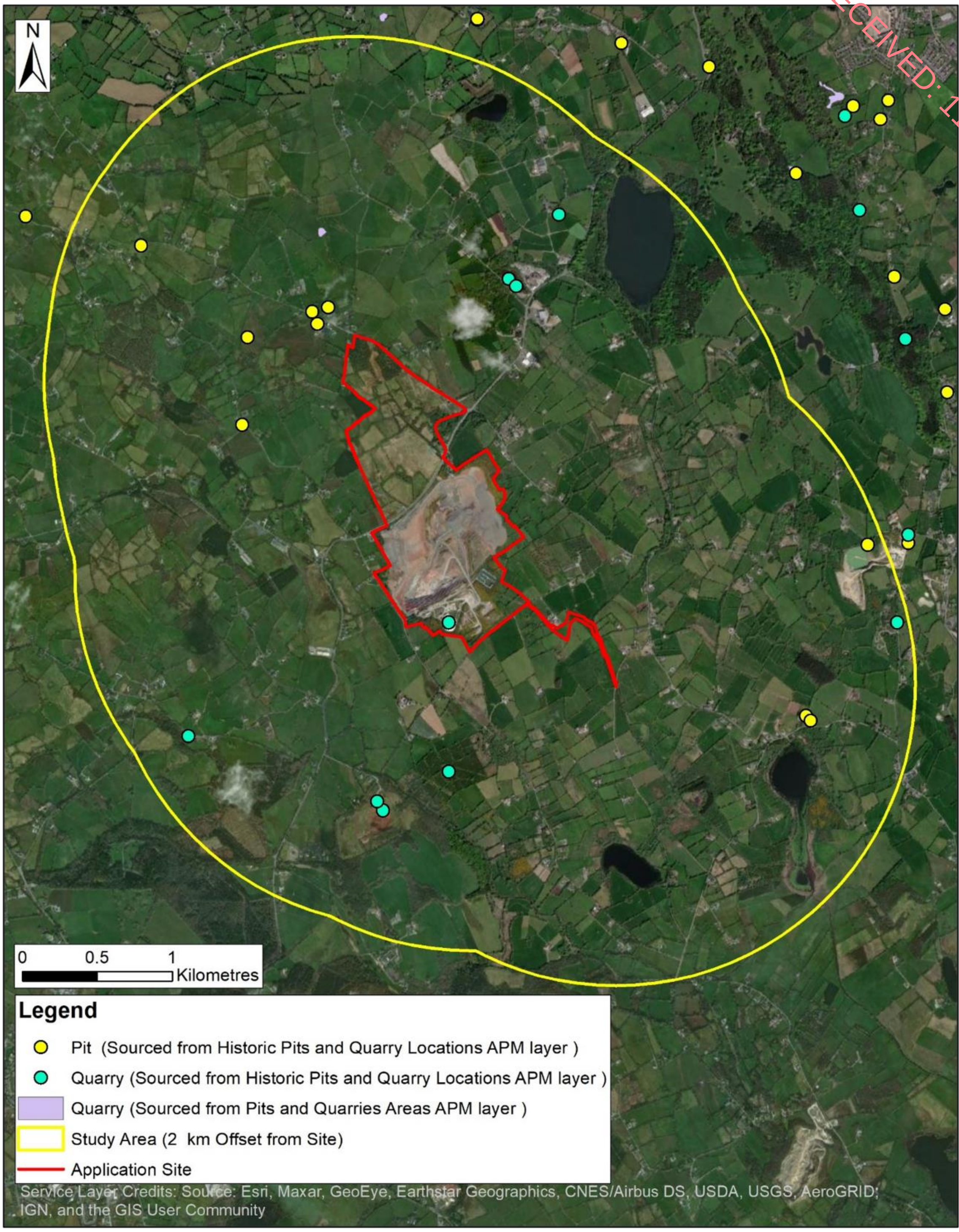

Figure 7.3: Historical Pits and Quarries within the Study Area (GSI, 2023)

A review of historical Google Earth imagery shows a series of aerial imagery taken between 2001 and the present day. An image from 1985 is included on Google Earth for the area, however, the resolution is very poor and there are no discernible features visible.

The first visible image on Google Earth is from May 2001, which shows the open-cast at Knocknacran is active at this time, as is the Plant Site. Large areas of scrubland are present to the south of the Plant Site and to the northeast of the open-cast and they appear to be primarily composed of gorse bushes as the areas exhibit a very strong yellow colour. The Plant Site is smaller than the current Plant Site is, notably the Drummond Mine conveyors and haul routes are missing as this mine was not yet open. The Knocknacran West site is largely agricultural fields, however the former GAA and Community Centre occupy part of the site during this time. To the northeast of the Knocknacran West site, an access route is visible in the area of the former Drumgoosat Mine's surface plant area located near the village of Drumgoosat. The wider Study Area is similar to the present day in that it is occupied by agricultural fields, residential housing and some local industrial and commercial facilities. To the east of the open-cast it appears that three residences are under construction.

The next available image from Google Earth is from June 2006, although this image covers only the Knocknacran Mine site and eastwards, part of the Knocknacran West site is obscured by cloud cover, or no image is available further west. It is apparent in the image that earthworks are taking place to put in the infrastructure on the Plant Site for Drummond Mine, the haul roads and conveyors are visible to the south. The Knocknacran open-cast has progressed eastwards since 2001. The wider Study Area (no visibility westwards of Drumgoosat village/Knocknacran West site) is still composed of agricultural fields and residential dwellings.

Imagery from April 2009 (Google Earth) is also partially complete for the Site and Study Area, the image has not been updated to east of the Knocknacran Open-Cast and Plant Site for this period. The Knocknacran West site shows minor changes since the May 2001 imagery including coniferous tree planting now occupying two fields to the west of the site and the access road to the area of the former Drumgoosat Mine's surface plant area is no longer visible, and the area is occupied by scrubland. The open-cast in Knocknacran has deepened since 2006 and it has also progressed closer to the present day R179 alignment and has removed the former R179 in the process. Areas in the Plant Site which were earthworks in 2006 are revegetated or occupied by infrastructure for Drummond Mine. The wider Study Area remains the same, albeit with the addition of two residences on the western side of the open-cast.

The April 2010 Google Earth image shows very little change within the Site, within the wider Study Area some farm buildings have expanded or have been upgraded overtime (e.g. expansion of the chicken shed to the east of the open-cast) and two additional houses are visible on the eastern side of the open-cast.

The April 2011 Google Earth image shows very little change since 2010, the Knocknacran Open-Cast has expanded in area to the northeast slightly and a residence is being constructed to the southeast of the open-cast.

By July 2013 (Google Earth) the Knocknacran Open-Cast is showing restoration along the western and northern areas. The Knocknacran West site and wider Study Area remain much the same. June 2014 imagery shows significant cloud cover and shadowing over the Site. March 2015 imagery is broadly similar to 2013.

August 2015 Google Earth imagery shows the western side of the open-cast is partly re-vegetating while extraction is still focussed on the eastern areas. Knocknacran West and the wider Study Area remain much the same as previous.

May, June and July 2018 Google Earth imagery shows extraction has moved to the southwest of the open-cast and restoration is occurring on the eastern side. Google Earth Imagery from February 2019 shows continued development of the southwest of the open-cast and restoration on the east. The Knocknacran West site shows visible evidence of subsidence (cracks, ridges, crownholes) around the former GAA and Community Centre and security fencing is visible around the area of subsidence. A trackway is visible to the west of the site which was used to construct a new groundwater monitoring well. The wider Study Area remains largely the same as in previous years, with some tree planting visible to the southeast of the open-cast.

September 2019 imagery (Google Earth) shows the former GAA and Community Centre buildings are in the process of being removed and the subsidence area is being remediated by infilling of subsidence features and clearance of hazards. Visually the area is a large site of unvegetated bare ground, some smaller crownholes are visible in the east, west and north of the site. The wider Study Area remains largely the same as previous years. By May 2020 (Google Earth) the Knocknacran West site is a mixture of recolonising remediated grounds, both at the former GAA/Community Centre site and to the east, west and north. Significant earthworks have been carried out on the Knocknacran Open-Cast site whereby the east and the northwest have been restored. A pond area to the northwest has been filled in and a large part of the site has been levelled as part of the restoration.

By March 2021 (Google Earth) most of the Knocknacran West site is revegetated, a crownhole is visible to the west, although this is an earlier crownhole that had not yet been filled in (it would be filled in late Summer 2021). A site area within Knocknacran Open-Cast has been levelled and shaped to the boundaries of the recently permitted Community Sports Complex site (Reg. Ref. 20/365). The wider Study Area remains similar to previous, a large chicken farm shed now abounds the Processing Plant area to the immediate east, recent earthworks are visible around the shed and revegetation has not yet re-occurred.

By June 2021 (Google Earth), the only two noticeable changes since March 2021 are green vegetation marking much of the Site area (except the existing Knocknacran Open-Cast) and the setting of the sports pitch on the Community Sports Complex site.

March 2022 (Google Earth) again identifies the changes within the Site area as being yellow/brown vegetation and further construction on the Community Sports Complex site with the changing rooms and associated buildings constructed onsite. The sports pitch is now grassed and vivid green in the imagery. Surrounding farmland in the Study Area is predominantly green compared to the unmanaged yellow/brown fields within the Knocknacran West site.

The Corine landcover classification (EPA, 2018) has also been considered in this assessment (Figure 7.4). The Knocknacran Open-Cast Mine and Process Plant Site are classed as 'mineral extraction sites', while the Knocknacran West Open-Cast Mine site and much of the wider Study Area are classed as 'pasture'. Water bodies, mixed forest and agricultural land with areas of natural vegetation also occur within the Study Area.

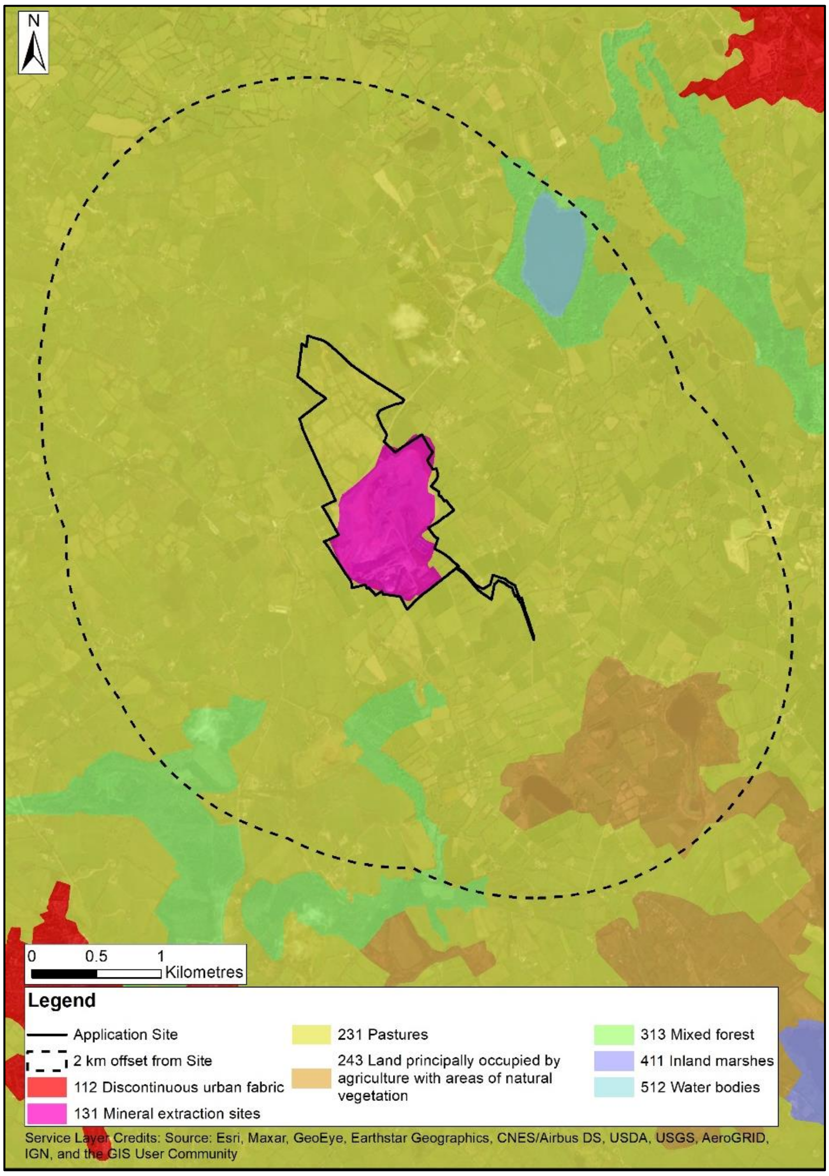

Figure 7.4: Corine Land Use mapping within the Study Area (EPA, 2018)

7.4.4 Soils

According to soil mapping compiled by the Irish Soil Information System (EPA & Teagasc 2007 – 2015), the areas immediately surrounding the existing Knocknacran Open-Cast and Processing Plant Site consist of ‘Urban’ ground, Figure 7.5. Areas to the east of the existing open-cast and Plant Site, including along the discharge pipeline, are of the Kilrush Association, which is a fine loamy drift with siliceous stones or river alluvium. A small central area within the existing open-cast is mapped as rock. While the mapping shows the existing open-cast mapped as ‘Urban’ ground, in reality it is currently exposed gypsum rock, mudstones or doleritic sands of the underlying bedrock units. The Knocknacran West site is exclusively mapped as being underlain by the Kilrush Association, as is much of the surrounding Study Area.

The southern section of the Study Area also consists of river alluvium, the Drumkeeran Association (a clayey drift with siliceous stones), rock, peat and water bodies. The eastern part of the Study Area consists of predominantly the Kilrush Association or the Elton Association which is a fine loamy drift with limestones.

There are lesser amounts of the Drumkeeran Association, water bodies, rock and river alluvium. To the north the Study Area is again predominantly composed of the Kilrush Association with areas of Elton, river alluvium, water bodies and the Ballylanders Association (a fine loamy soil over shale or slate bedrock). To the east the Study Area is predominantly composed of the Kilrush Association and the Ballylanders Association with areas of river alluvium.

Within the existing Knocknacran Mine site area and in particular the area of the existing and proposed Community Sports Complex site, it is noted that as this has been an area within an active open-cast mine, soils which do occur on this site may in some locations no longer be original soil. In some areas, such as the eastern side of the Community Sports Complex site, the soil profile is composed of backfilled soils which were emplaced as part of the phased restoration works. These soils are a mixture of weathered dolerites, glacial till and the Upper Mudstone and Middle Mudstone Members of the Kingscourt Gypsum Formation which occurs onsite as a soft red clay.

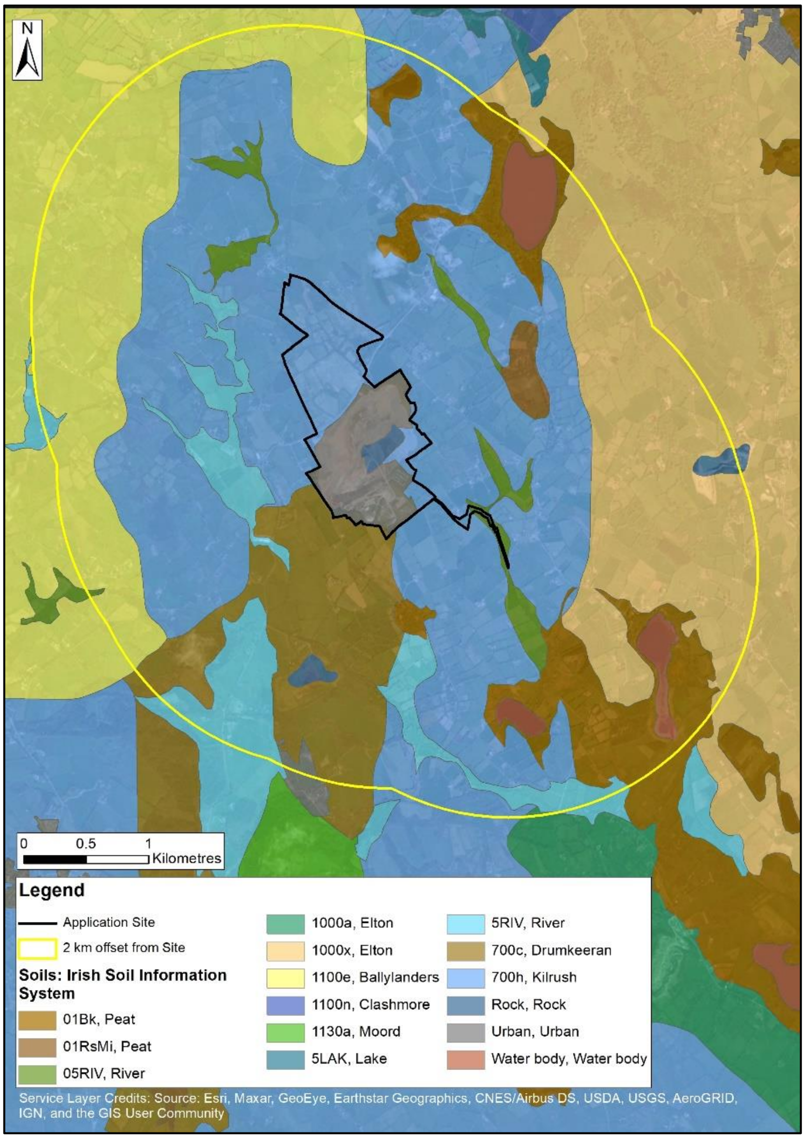

Figure 7.5: Soils Map (Irish Soil Information System mapping, EPA & Teagasc)

7.4.5 Subsoils

According to subsoil mapping compiled by Teagasc and the EPA, the areas immediately surrounding the existing Knocknacran Open-Cast and Processing Plant Site consists of Made Ground. Areas in the east of these sites are mapped as a mixture of bedrock at surface, sandstone and shale till (of Devonian/Carboniferous age) or gravelly undifferentiated alluvium. The Knocknacran West site consists primarily of sandstone and shale till with some undifferentiated alluvium and bedrock at surface near the village of Drumgoosat. Much of the wider Study Area is composed of the sandstone and shale till of Devonian/Carboniferous age with some areas of alluvium, peat, bedrock, lacustrine sediments, Made Ground or sandstone and shale sands and gravels of Lower Palaeozoic age.

The thickness of the superficial deposits is variable across the area. Thicker till layers are observed at the higher points of the terrain (drumlins), with overburden thickness reaching about 50 m. Away from the drumlins, the overburden can be as thin as 1 m, with areas of bedrock outcrop seen to the east of the site (i.e. there is no overburden present). The average overburden thickness is 13 m according to drill hole logs and the Geological Survey of Ireland (GSI) National Well Database.

Isolated deposits of peat occur to the east of the Site within topographical hollows (Figure 7.6). The depth of overburden across the Application Site where it has not been stripped or re-worked is typically variable in thickness, reflecting the nature of the drumlin landscape.

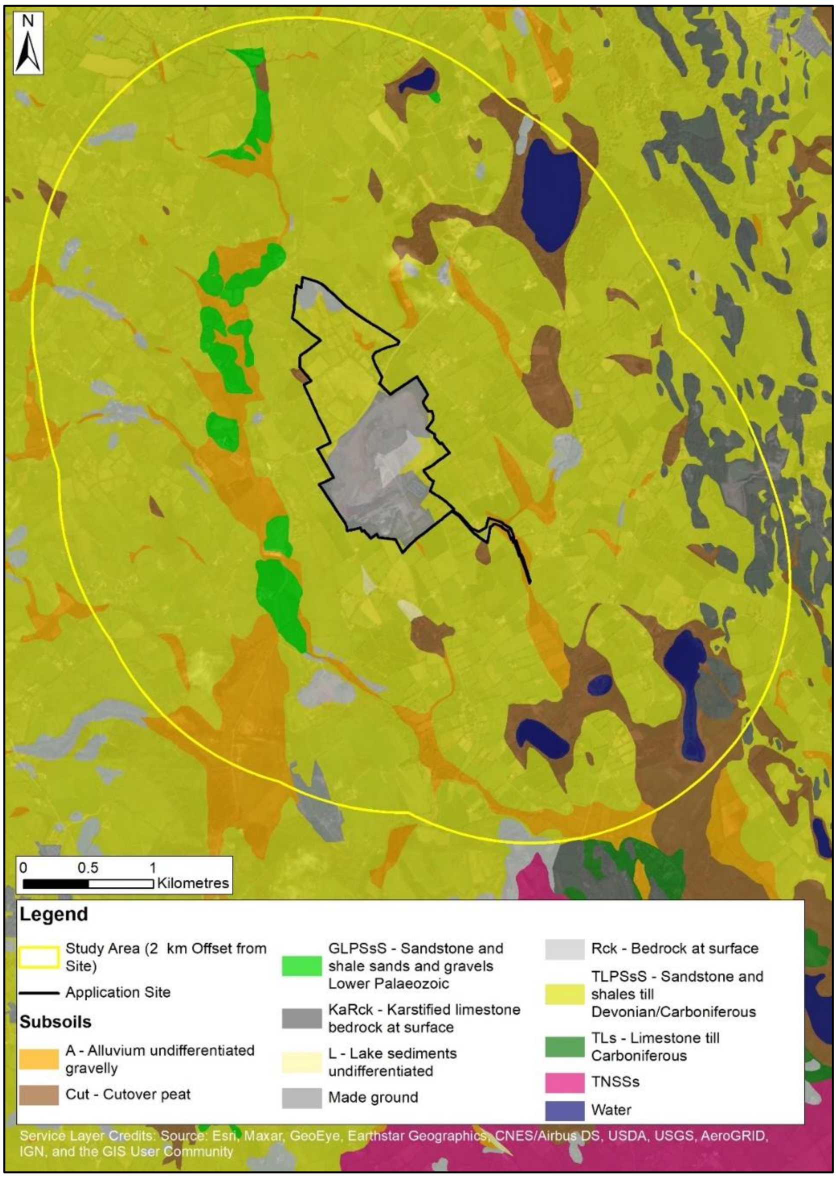

Figure 7.6: Subsoils Map (Teagasc and EPA)

Based on information from exploration boreholes (drilled by the Applicant since the 1940s) and groundwater monitoring boreholes (drilled by Minerex Environmental Limited) and a review of the National Well Database (GSI), the thickness of overburden within a ca. 3 km radius of the Application Site ranges between 1 and 50 m, with an average thickness of ca. 13 m. This variation in thickness of overburden is typical of drumlin landscapes where the localised topographic highs (drumlins) are associated with thick overburden deposits.

Within the existing Knocknacran Mine site area, the area of the existing mine and proposed Community Sports Complex site, it is noted that as this has been an area within an active open-cast mine, subsoils which do occur on this site may in some locations no longer be original subsoil.

In some areas, such as the eastern side of the Community Sports Complex site it is noted that although there never has been underground mining, the subsoil profile is composed of backfilled subsoils which were emplaced as part of the phased restoration works. These subsoils are a mixture of weathered dolerites, glacial till and the Upper Mudstone and Middle Mudstone Members of the Kingscourt Gypsum Formation which occurs onsite as a soft red clay.

A ground investigation carried out as part of the Community Sports Complex development (Reg. Ref.: 20/365) identified Made Ground overlying mudstone bedrock of the Kingscourt Gypsum Formation. The ground investigation was undertaken in accordance with guidelines set out in BS5930 Code of Practice for Site Investigations, 4th Edition (2015); UK Specification for Ground Investigation, 2nd Edition (2011); BS EN 1997-2 (2007) and BS EN ISO 22475-1 (2006).

Four boreholes were drilled to depths from between 12.5 m to 20.5 m below ground level (bgl). The locations of the boreholes are shown on Figure 7.7. The borehole logs are provided in Appendix 7.1.

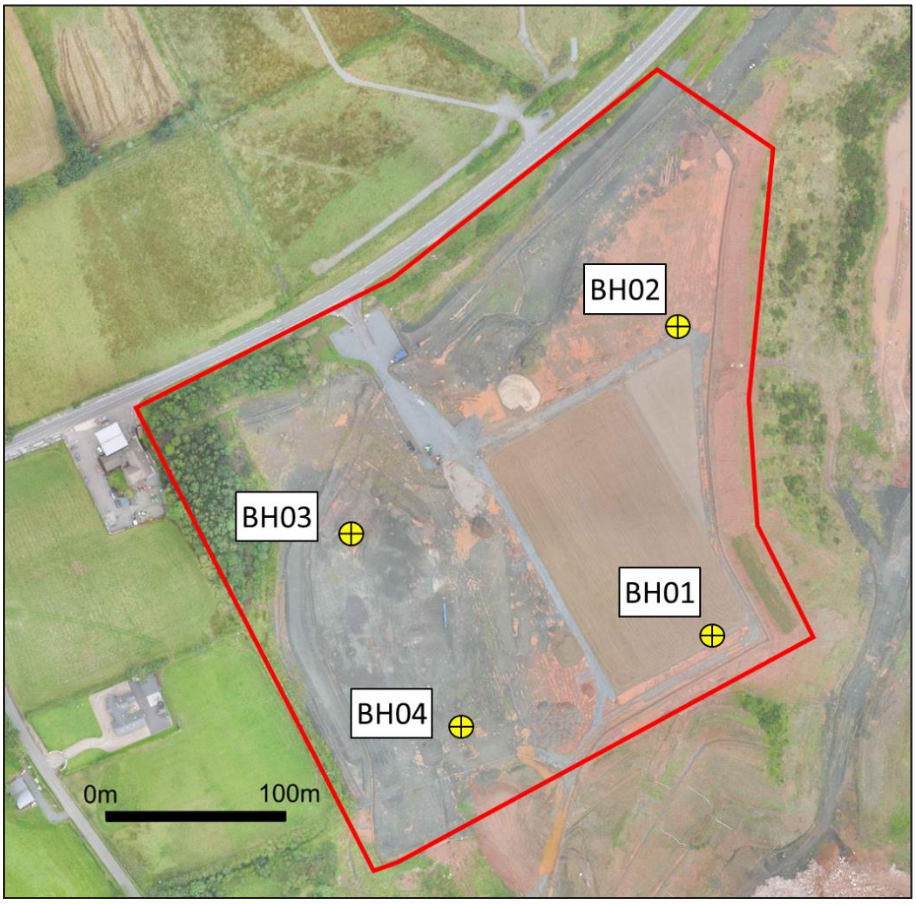

Figure 7.7: Location of the 2021 ground investigation boreholes at the Community Sports Complex site

The Made Ground underlying the area consists of a layer of engineered fill material which was emplaced during the progressive restoration of the Knocknacran Open-Cast Mine area. The fill material is primarily composed of layers of; soft to firm / firm, becoming stiff with increasing depth, reddish brown and brown, slightly gravelly, sandy, silty clay with occasional cobbles and occasional thin bands of silty, fine to medium sand and fine sub-rounded gravel. Bands of medium dense, dark greyish brown, very silty, fine to coarse sand and fine to medium sub-angular gravel also occurred, with occasional bands of grey to dark grey, very silty, slightly gravelly, fine to coarse sand. Firm, grey, gravelly, very sandy, clayey silt with bands of silty, gravelly, fine to coarse sand were also identified.

The bedrock underlying the site, encountered in BH03 and BH04 only, was composed of extremely weak to very weak, bright reddish brown and locally bluish grey, fine grained argillaceous, fissured, mudstone of the Kingscourt Gypsum Formation (Triassic in age).

No underground mine workings occur below the proposed Community Sports Complex Site.

7.4.6 Bedrock

The geology of the area has a strong north-south strike and is located in the Kingscourt Outlier, a half-graben structure some 1.2 km wide and 12 km long, formed of Carboniferous and Permo-Triassic rocks. The Kingscourt Fault forms the western boundary of the Kingscourt Outlier (Figure 7.8).

The sequence within this outlier predominantly consists of red-brown mudstones and sandy mudstones up to ca. 550 m in thickness, within which are two distinct gypsum / anhydrite units of Permian age in the lower part of the sequence (Figure 7.9). These deposits form a cap on the north-south trending Carboniferous outlier within the Lower Palaeozoic Longford Down Massif.

The Lower Seam Gypsum (bed/unit) comprises of gypsum and anhydrite which are grey in colour and varies in thickness of between ca. 20 and 35 m. The Upper Gypsum Seam (bed/unit) which tends to be red in colour is typically ca. 6 to ca. 10 m in thickness. The lithologies present a record of the deposition of sediments in arid deserts and temporary seas that were periodically dried out to precipitate thick evaporite sediments of gypsum. The Upper gypsum bed is separated from the Lower gypsum bed by a band of mudstone (ca. between ca. 6 to 12 m in thickness).

There is substantial evidence of post-depositional weathering or solution (karst) on the upper surfaces of the gypsum beds as seen in the west of the deposit exposed in the open-cast mine. However, no major cave systems have been encountered in either the current open-cast mine or adjacent underground workings.

The Kingscourt Gypsum Formation is underlain by undifferentiated micaceous shales, siltstones and sandstones, and occasional thin coal beds of Westphalian and Namurian (Carboniferous) age, which outcrop in small areas to the south and north of the Kingscourt Outlier (Figure 7.8). The Kingscourt Sandstone Formation is the youngest of the sequence and outcrops immediately to the east of the Kingscourt Fault (Figures 7.8 and 7.9).

Dolerite sills occur in the Permo-Triassic sequences at Kingscourt, with the principal intrusion in the Middle Mudstone between the two gypsum seams/beds. A secondary intrusion is generally restricted to the Lower Mudstone but is known to occasionally cross-cut the Lower Gypsum Seam in some areas. The sills are interpreted as having been hydrothermally altered as they were intruded, resulting in susceptibility to weathering and thereby acting as potential conduits for water where altered.

The area contains five primary stratigraphic units which are summarised below (from youngest to oldest):

Kingscourt Sandstone Formation: Outcrops to the east of the Kingscourt Fault and is the youngest formation of the sequence. This part of the sequence comprises a siltstone member (between 80 to 100 m in thickness), conformably overlain by Lower Triassic red-beds sandstone (up to 300 m thick), which typically comprises deep beds with parallel and cross lamination.

Kingscourt Gypsum Formation: Is a mudstone unit with two distinct mineralised beds. The provenance of the gypsum suggests deposition of sediments when arid deserts were occasionally encroached upon by the sea, which then evaporated to precipitate thick deposits of evaporite minerals. Figure 7.9 presents the stratigraphy of the formation which is typically divided into five units.

- ***Lower Mudstone Member*** is a transitional mudstone which grades up into the Lower Gypsum from 50% gypsum to good quality gypsum.

- ***Lower Gypsum Member*** and anhydrite bed is up to 35 m in thickness and is grey in colour. Above the transition zone with the Lower Mudstone, it comprises a thickly bedded, high quality white to grey nodular gypsum that has been the target of underground mining. This, in turn, transitions upwards into good quality, light brown laminated gypsum with rhythmic banding, which gradually changes to creamy pink or red further up the succession. Next are banded magnesium-rich gypsum layers which can be high in carbonates and show signs of being heavily leached by groundwater. Massive white gypsum is the upper- most section of the Lower Gypsum unit. Sub-outcrop of the Lower Gypsum Member underlies the Knocknacran open cast area, from the settlement ponds in the east to the extent of Drumgoosat underground workings in the north.
- ***Middle Mudstone Member*** is a band of mudstone that separates the upper and lower gypsum members. It varies between 6 and 12 m in thickness. The member consists of reddish, micaceous, plastic mudstones, with frequent green reduction spots and laminations near the base.
- ***Upper Gypsum Member*** is a massive, fine grained, grey-brown to red pure gypsum. It is typically red and is thinner than the lower bed, ranging between 6 and 10 m in thickness. Moving upward in the sequence from the massive red gypsum is inter-banded gypsum and red siltstone, coarse gypsum and finally massive gypsum containing very pure and fine grained grey or cream laminated mineral. The Upper Gypsum subcrop only underlies the western side of the Knocknacran open cast and is well exposed in this location.
- ***Upper Mudstone Member*** the Upper Gypsum is overlain by the Upper Mudstone, which is between 26 and 36 m in thickness.

Namurian Sandstones: The **Cabra Formation, Corratober Bridge Formation, Clontrain Formation and Carrickleck Formation** – underlying the Kingscourt Gypsum formation and outcropping to the east of eastern-most fault within the graben structure. The formations comprise Namurian-age (Carboniferous) sandstones and interbedded shales. These are poorly cemented and typically very weathered. This results in increased permeability.

Carrickleck Sandstone Member: The basal member of the sandstone sequence, it is distinguishable from the Carrickleck Formation as being buff-coloured ferruginous sandstone.

Milverton Group: Underlying the Carrickleck Sandstone Member and outcropping further to the east, the Milverton Group comprises Dinantian pure bedded limestone. The limestone within this group is extensively karstified with numerous features including caves, enclosed depressions, springs, swallow holes and turloughs.

Dolerite (Basalt) Sills: Are also present in the Kingscourt sequence. The sills have been described as being conduits for water, having been hydrothermally altered during intrusion, making them susceptible to weathering and incompetent in places. The primary intrusion is a fine grained homogeneous dolerite/basalt between the Upper and Lower Gypsum Units, in the Middle Mudstone (Figure 7.11 and Figure 7.12). The intrusion reaches a maximum thickness of 60 m. It has undergone extensive near-surface lateritic weathering and hydrothermal alteration, weathered to a fine grained sand in places. The dolerite sill chiefly occurs to the east of the orebody and thins out towards the west, with the dip of the gypsum beds. There is a secondary intrusion (ca. 8 to 10 m in thickness) that is typically confined to the Lower Mudstone.

In addition, the following two formations occur in close proximity to the area:

Castlerahan Formation: Outcrops to the west of the Kingscourt Fault. This Silurian aged massive quartzo-greywacke has been thrust upwards along the Kingscourt Fault to juxtapose the Permian Kingscourt Sandstone Formation.

Westphalian Shales: Outcrop to the north of the site are consisting of grey to black shale and carbonaceous or pyritous, thin bedded siltstones and fine grained sandstones. In addition, minor thin beds of coal may be present.

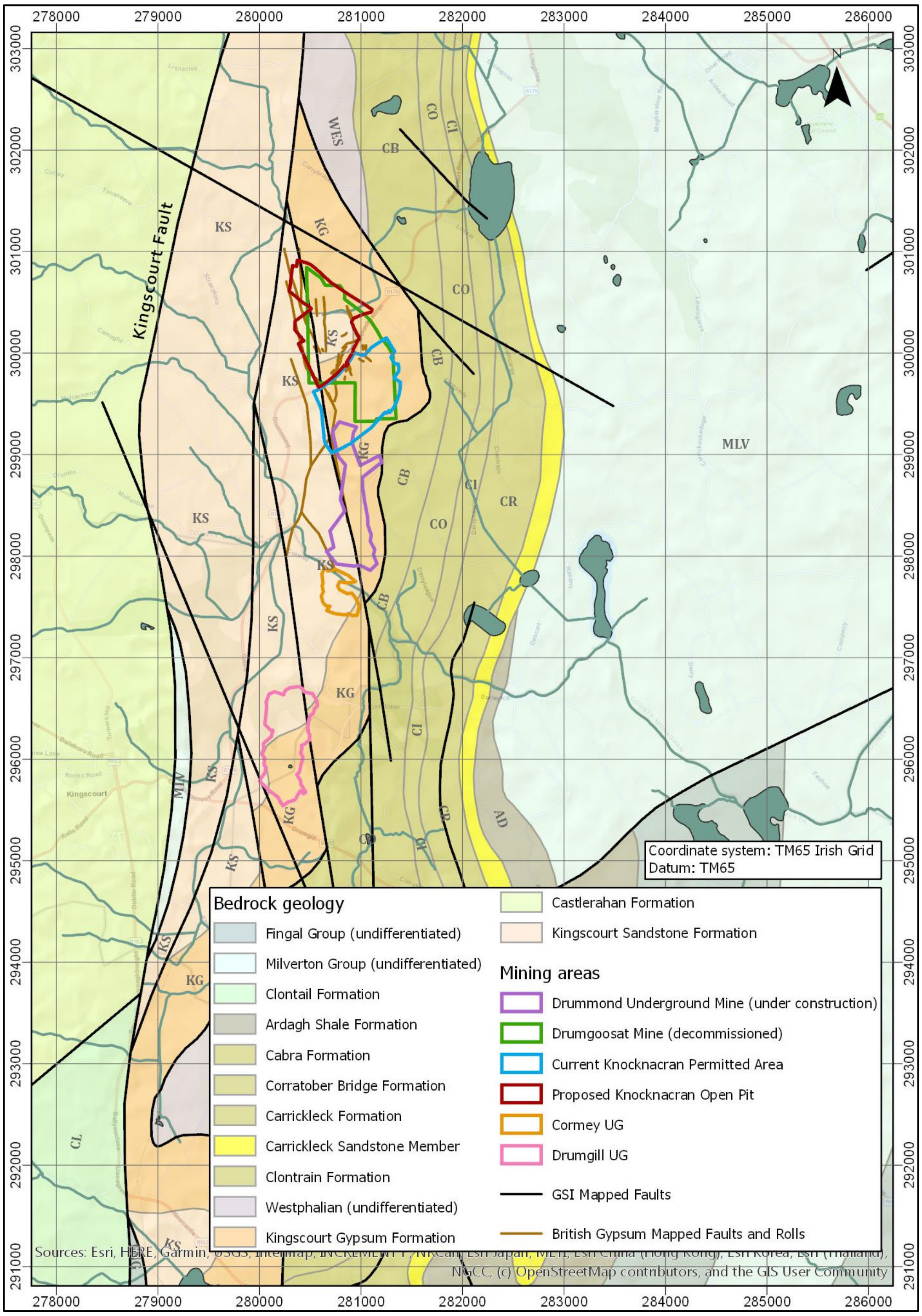

Figure 7.8: Bedrock Map (showing mining areas) with half-graben cross section

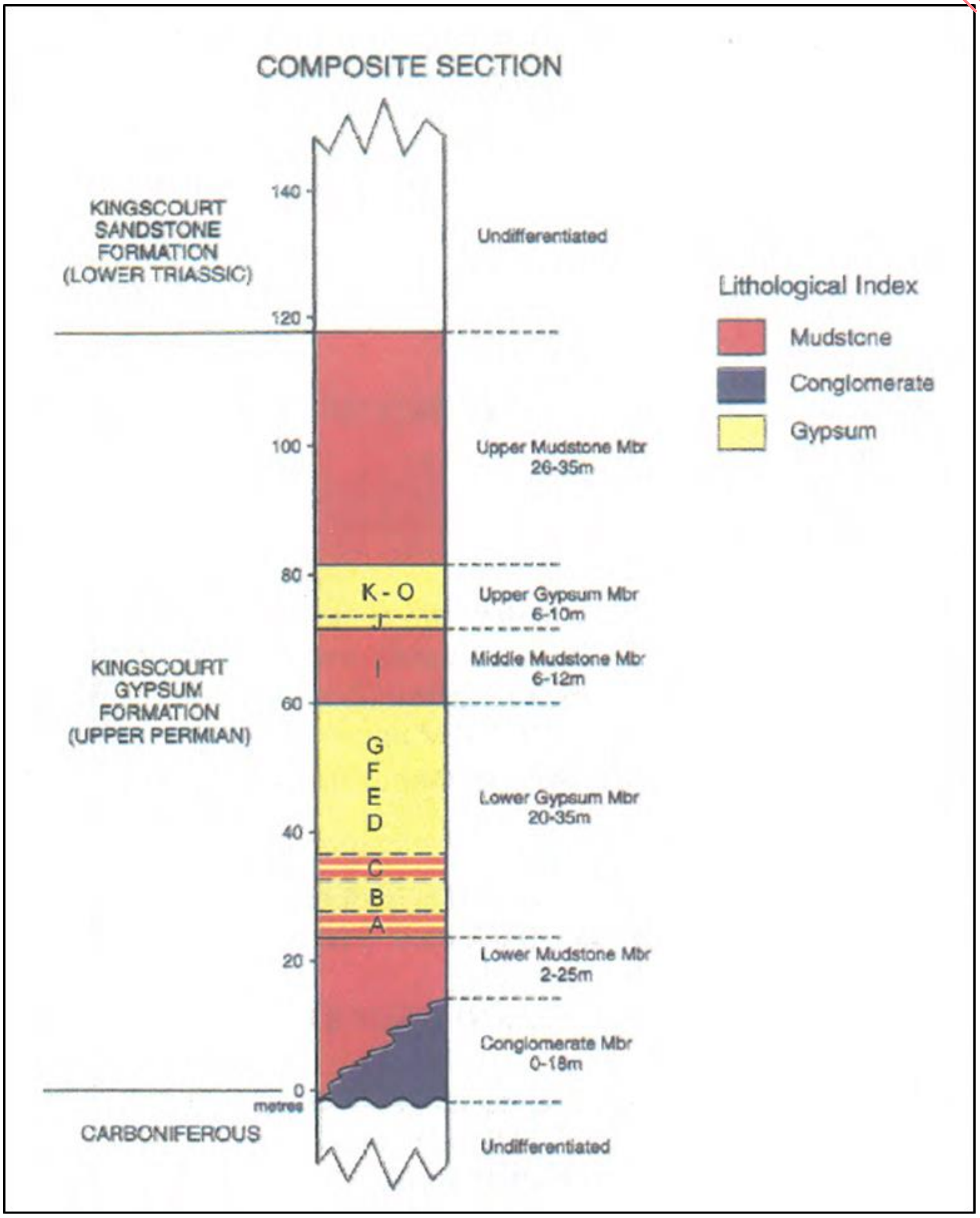

Figure 7.9: Kingscourt Gypsum Formation Stratigraphy (Gardiner & McArdle, 1992)

7.4.7 Structural Geology

The Permo-Triassic rocks occur in a series of open north-east trending folds, with strata dipping to the west, towards the Kingscourt Fault, the western boundary of the outlier, at an angle of between 10° and 30°. The Kingscourt Fault has a down-throw to the east of ca. 1.5 km. There are several other major north-south trending faults within the Permo-Triassic sequence, with opposed throws of up to ca. 150 m, forming graben like structures (Figure 7.8). In the Drumgoosat underground workings none of these structures have been found to be significant water bearing conduits. However, in the Drummond Mine, a fault referred to as the Drummond Mine Fault intersected a large inflow of water in June 2018 (Figure 7.10), with an initial maximum inflow estimate of ca. 450 m^3/hr (ca. 10,800 m^3/d). Over the flowing weeks this reduced to an average of ca. 170 m^3/hr (ca. 4,080 m^3/d), reducing further to an estimated seasonal range of between ca. 700 and ca. 2,400 m^3/d.

Structural mapping (GSI) of the gypsum deposits underlying the Knocknacran area indicate the presence of two major north-south trending faults to the west of the current open-pit mine (Figures 7.7 and 7.9).

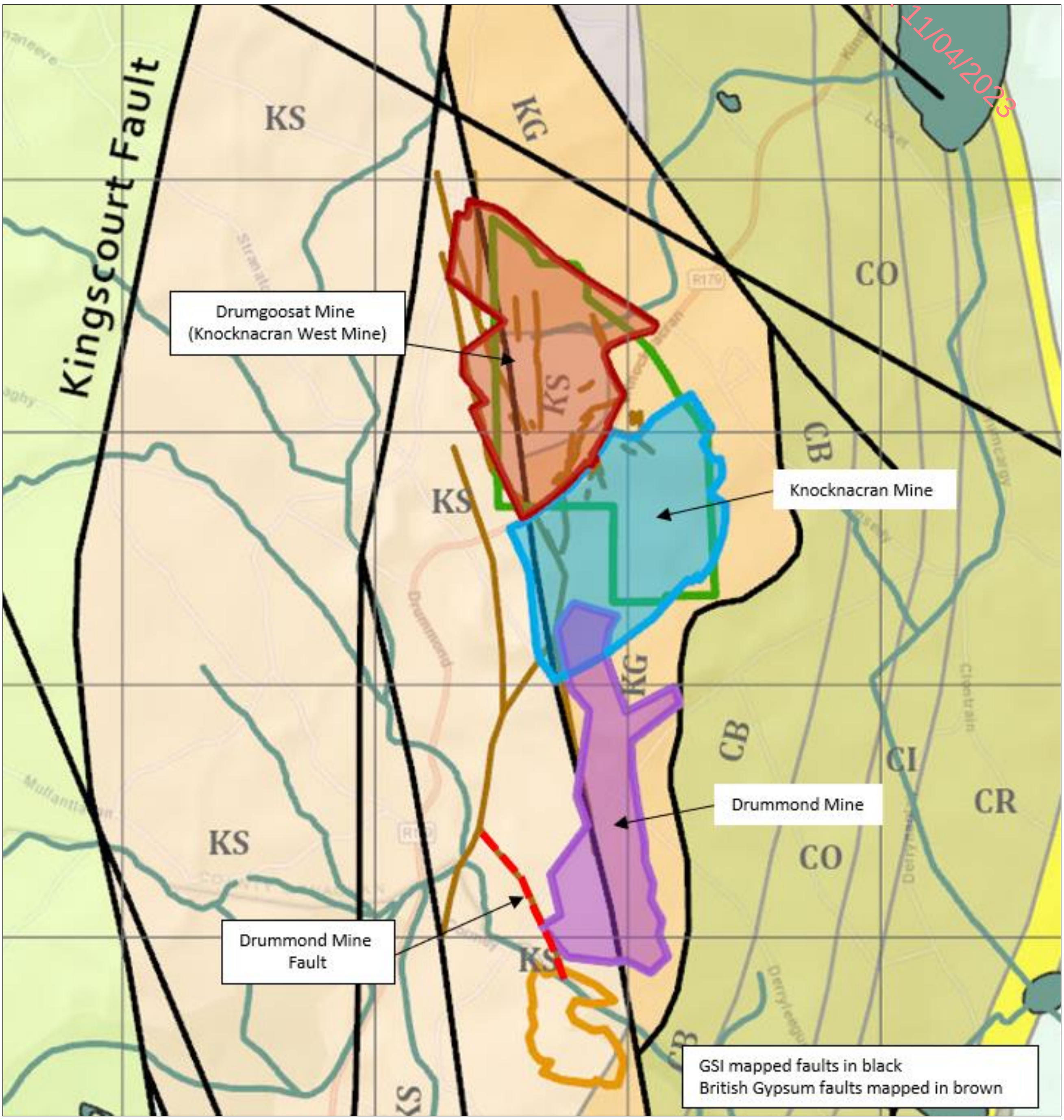

Figure 7.10: Plan showing the location of the Drummond Mine Fault

Both have a north-south trend and appear to extend at least as far as the Cormey workings to the south. One underlies the pit along its southwestern margin. The other occurs approximately 500 m to the west. The faults are believed to downthrow the Upper and Lower Gypsum Units by about 10 m and 30 m, respectively. Discontinuous groundwater levels have been identified between exploration holes on either side of the fault that underlies the south western edge of the pit. This suggests that the fault acts as a low permeability barrier to groundwater flow.

Roll and fault information for the Knocknacran West site also shows structures primarily following the north-south orientation although some rolls are orientated in a northeast-southwest direction.

Mapping data for the Knocknacran open-cast and a number of underground pillar faces has also identified minor faults that appear to form a dendritic pattern through the centre of the pit. Analysis of the data found that the major discontinuities in the gypsum are near vertical and strike north-south and east-west, with less dominant features striking northeast-southwest and northwest-southeast. The shaley units within the gypsum exhibit well developed bedding, with a regional bedding trend between 15° and 30° to the east.

7.4.8 Mine Geology

The gypsum deposits within the Kingscourt Outlier have been subject to extensive underground mining in the past. To the north and northwest of the Knocknacran open-cast, the decommissioned Drumgoosat underground mine extends to a maximum depth of ca. – 83 m OD.

Previously, underground workings were exposed in the floor of the Knocknacran open-cast mine where pillars of unmined gypsum were visible. Underground extraction occurred in both gypsum units, but was predominantly in the Lower Unit. A room and pillar mining method was employed, in which rooms, or tunnels, about 10 m wide and 6 m high were extracted, leaving pillars that were about 12 m square in plan dimension. Actual mining dimensions varied from these values due to the mining technology and the natural fracture spacing in the gypsum. By design, at least 1 m of gypsum was left in the roof and floor to isolate weak mudstone above and below the gypsum from the loads caused by excavation. Although investigation has confirmed that the upper surface of the gypsum seam is typically irregular as a result of local variability caused by variation in topography as the gypsum was formed and subsequent later dissolution by groundwater, there are locations where the roof or floor are thinner and where the overlying mudstone has been exposed in the mine workings.

As part of the water management plan for the Drummond Underground Mine, the Knocknacran Open-Cast Mine, the primary processing facilities at the site and the site's overall infrastructure; the underground workings at Drumgoosat were previously used to store mine water in times of low flow in the River Bursk.

The Drummond Underground Mine, in operation since 2005 (permitted to 2032), is located to the south and southwest of the Knocknacran open-cast and is currently at a maximum of ca. -115 m OD.

The base of the Knocknacran Open-Cast Mine currently extends to ca. 0 m OD and will be exhausted by 2027 (depending on markets conditions).

Within the area surrounding the Knocknacran Open-Cast Mine, the sub-outcrop of the Lower Gypsum Member underlies the entire open-cast area extending as far as the settlement ponds to the east and as far north as the extent of the former Drumgoosat underground workings. The sub-crop of the Upper Gypsum Member underlies the western part of the open-cast only and is well exposed in this area. Bands of dolerite are also exposed in a number of faces. Figure 7.11 presents a photograph of the open-cast showing previously exposed, overburden, Upper and Lower Gypsum Members, mudstone and dolerite.

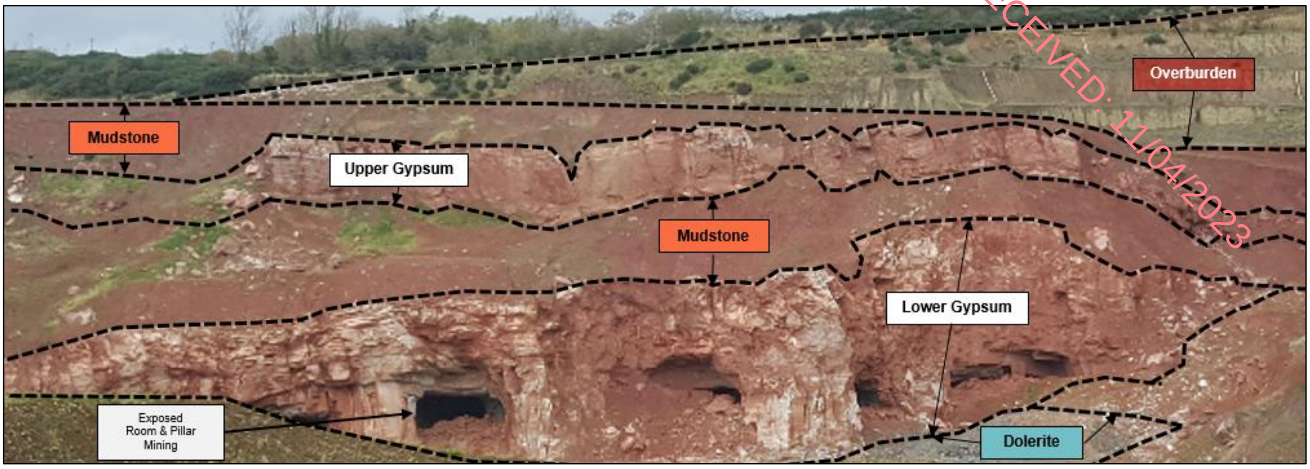

Figure 7.11: Photograph showing Geological Units at Knocknacran Open-pit Mine

A schematic log of the geological sequence associated with the Knocknacran Open-Cast Mine is presented in Figure 7.12.

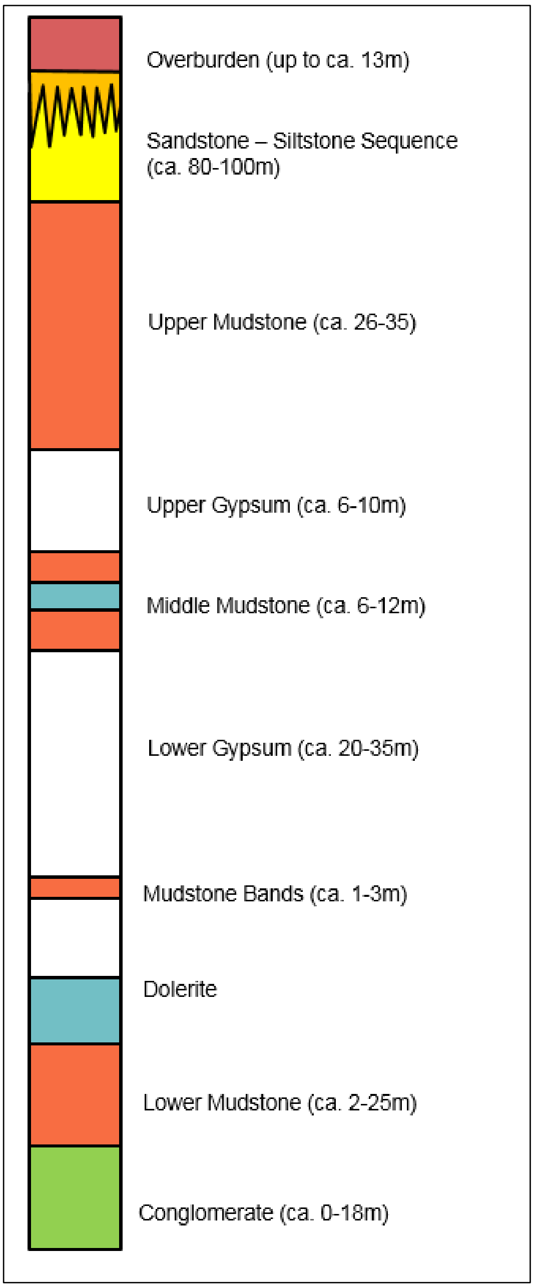

Figure 7.12: Schematic Geological Log (not to scale)

Figure 7.13 and Figure 7.14 present West - East and North - South cross-sections (respectively) through the proposed Knocknacran West Open-Cast Mine, showing the relationship between the gypsum and later dolerite intrusions. Figure 7.15 presents the plan for the section lines.

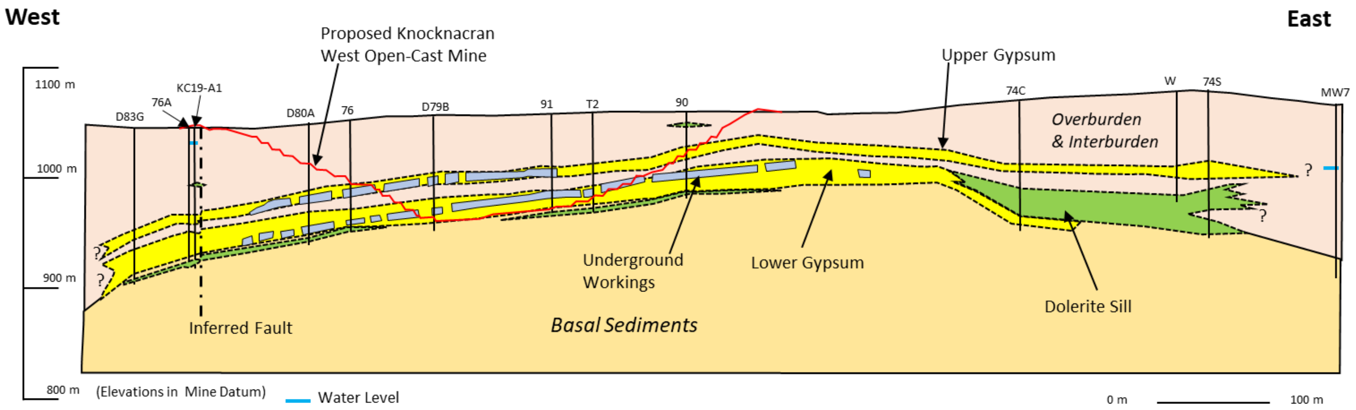

Figure 7.13: West - East Geological Cross-section through the proposed Knocknacran West Mine

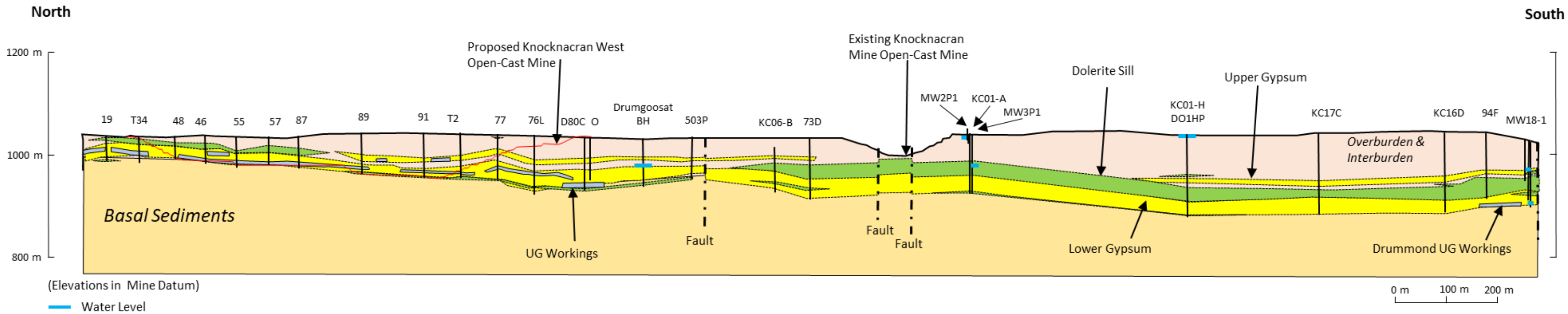

Figure 7.14: North - South Geological Cross-section through the proposed Knocknacran West Mine

Figure 7.15: Section line plan map

7.4.9 Palaeokarst

No naturally occurring palaeokarst features have been identified at the Site, Figure 7.16. A spring is identified by the GSI (2023) to the north of the Site in the townland of Drumgoosat, the legend on the spring indicates it was originally identified through a historical 6" map. A review of the available historical maps from the OSI (refer to those referenced in Section 7.4.3., above) does not identify any springs on the 6" historical map or

Cassini map, however, one is identified on the 25" 1888 – 1913 map series at this location. No other features are identified by the GSI within the Study Area.

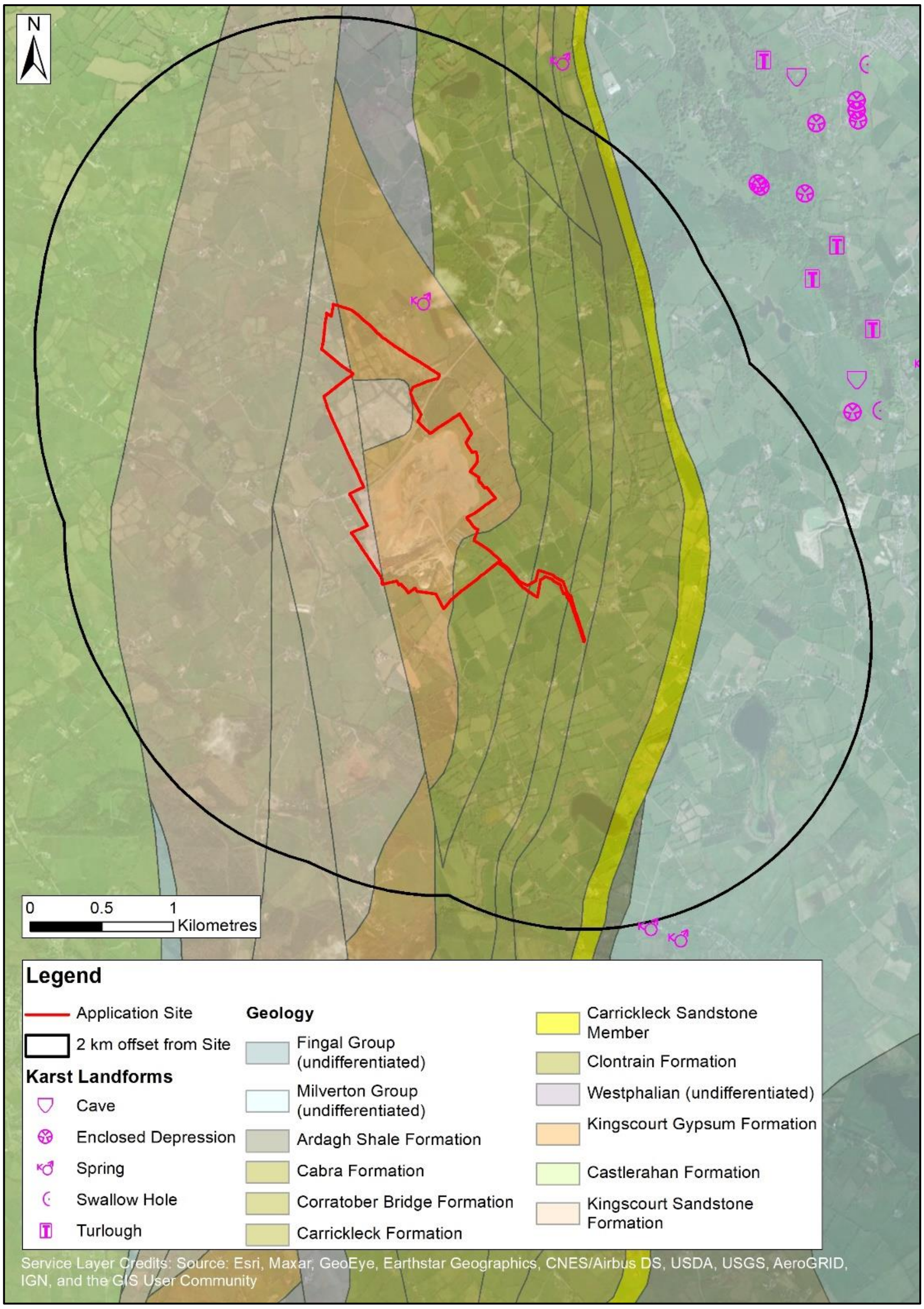

Figure 7.16: Karst features in the Study Area (GSI, 2023)

Limited development of karst had occurred in the Irish Midlands during the Cenozoic Era, primarily controlled by the presence of dominant NNW-SSE trending structural features in the limestone bedrock. However, by the Holocene epoch (ca. 12,000 years ago) the karstic environment had become clogged with sediment and there was no longer any active groundwater circulation. The karst features became "buried, inert and fossilised karst" termed 'palaeokarst' (Drew & Jones, 2000).

However, in the case of the Kingscourt Sandstone and Gypsum Formations, no limestones occur but rather mudstones (and associated sediments), gypsum and cross-cutting intrusions of dolerite, which when altered tend to act as conduits (and reservoirs) for water, which in turn can lead to the development of localised karst features (i.e. cavities) along the contact of the gypsum and surrounding/intruded dolerite bodies.

7.4.10 Geohazards

7.4.10.1 Subsidence

The Knocknacran West site has had several subsidence events over the years within the site which have been confined to areas over the Drumgoosat underground mine workings. Figure 7.17 shows the locations of known sinkholes (crownholes) over the Knocknacran West site. The sinkholes are numbered according to year of recording/occurrence (i.e. “DT16a” was recorded in 2016).

The stability of the Drumgoosat Mine has been studied by SRK Consulting (UK) Ltd. on behalf of the Exploration and Mining Division of the Department of Communications, Climate Action and Environment (EMD), which issued reports on their studies in 1999 and 2002 (Appendices 7.2 to 7.3 respectively). SRK initially employed empirical analyses and qualitative risk assessment to support its assessment of mine stability, and subsequently augmented these analyses with simple computer models of stress and deformation.

In the summer of 2018, a high volume of groundwater associated with a fault structure was intersected in the Drummond Underground Mine. As had been normal practice for many years, the water was pumped to the old Drumgoosat Mine workings for storage prior to discharge to the River Bursk during periods of high flow. The high volume of water encountered in Drummond led to a larger than usual volume of water being stored in Drumgoosat, reaching a greater height/level in the underground workings than had historically occurred. In September 2018, this resulted in a subsidence event taking place in a part of the mine below the former Magheracloone GAA/Community Centre, where gypsum had previously been extracted from rooms up to 12 m in height.

Following the subsidence event in September 2018, work was undertaken by SRK, on behalf of the Applicant, to assess the causes and current, and future, stability of the existing underground workings beneath the site (Appendix 7.4).

Wardell Armstrong International (WAI) reviewed the work completed by SRK from 1999 to 2018 on behalf of the Department of Communications, Climate Action and Environment (DCCAE). WAI undertook their own analyses of pillars below the September 2018 subsidence event and at several locations below the L4900 and R179 roadways (Appendix 7.5). They also used numerical stress and deformation modelling, applying similar approaches to SRK. In all critical respects, Wardell Armstrong concurred with SRK’s conclusion that the risk of future mine instability was very low. Where there was a minor difference in opinion related to the predicted stability of one pillar below the R179, a drilling investigation was requested and conducted, from which it was concluded that the pillar was stable. Wardell Armstrong considered that it is important to maintain the mine workings in a dewatered condition.

In addition, the R179 Kingscourt to Carrickmacross road was closed for a number of weeks until the risk from further land subsidence could be determined. It was concluded that loss in underground mine stability was localised and that further mine collapse is unlikely (Appendix 7.4).

Numerical stress modelling was employed in the investigation of this failure and concluded that three unique conditions at this location had interacted to result in the event and without any one of these three conditions, the event would not have occurred. The three unique conditions can be summarised as follows (Appendix 7.4):

1) 12 m high pillars occurred at this location compared to 6 m high pillars elsewhere;
2) Water levels rising and submerging the 12 m high pillars by the introduction of excess water being pumped into Drumgoosat at a level higher than had previously occurred and sufficient enough to submerge these particular pillars; and
3) A thin gypsum floor beam.

Following the September event, a small crownhole (sinkhole) failure occurred to the south of the L4900 and over old underground workings in December 2018. The L4900 was closed as a precautionary step. Using the results of updated laboratory strength tests, observations from earlier underground visits, new drilling investigations and laser scans of selected mine workings, SRK (2019) (Appendix 7.6) assessed the stability of the mine below the L4900. A total of 25 boreholes were drilled as part of the investigations along the L4900. They concluded that there was sufficient gypsum above the mine excavations to provide a very low risk of roof instability and that the pillars were sufficiently strong to provide a low risk of future subsidence. Recommendations for monitoring the future stability of the mine workings were provided (Appendix 7.6).

SGMI retained SRK to undertake a similarly detailed investigation of the conditions of the mine below the R179 by drilling and surveying further boreholes. The SRK report (Appendix 7.7) provides information on rockmass strength characteristics from borehole logging and laboratory data, which was used to form the basis for subsequent computer stability modelling. Finite element modelling analysis of defined cross-sections along the R179, coupled with a geotechnical assessment and interpretation of laser scans of the mine workings intersected by the boreholes was undertaken. Based on the investigations carried out, no high risk, unstable undermining areas were identified. The laser surveys and the geotechnical borehole logging have provided strong evidence that there has been virtually no deterioration in the mine conditions since the excavations were created. This provides confidence that the roof beams and pillars are still doing the job for which they were designed, which is to support the underground openings and prevent surface subsidence.

Since SRK's report, extensometers have been installed adjacent to the R179 to measure roof beam movement. A copy of the Trigger Action Response Plan (TARP) associated with the extensometers for the R179 is provided in Appendix 7.8. The R179 monitoring system consists of five multi-point borehole extensometers along with precise levelling points on the surface.

Appendix 7.9 provides a Trigger Action Response Plan (TARP) for the monitoring of gypsum roof beam stability at various locations along the L4900 road. The monitoring system comprises eight multi-point borehole extensometers along with precise levelling points located on the surface in the vicinity of the collar positions of the extensometers.

The purpose of the TARP for both the L4900 and the R179 is to provide an early warning of failure of the gypsum roof beams and the potential migration of instability to surface that may affect the stability of the roads and the safety of road users. The extensometers are connected to data loggers that automatically collect movement data which are transmitted wirelessly back to the Gyproc Survey Office as part of the early warning system.

SRK (July 2020) (Appendix 7.8) have noted that historically there have been no instability events associated with the underground mine within 50 m of the R179 and that the underground workings below both the R179 and L4900 have been in place and stable for at least 40 years.

Wardell Armstrong International were again retained by the regulatory authorities to provide an independent review of SRK's work in 2020 (Appendix 7.10). Their review agrees with the conclusions reached by SRK that the R179 continues to be safe to use. They also consider it prudent that the comprehensive monitoring programme in place since 2018, remains in place for the R179, thereby providing for an early warning of any potential underground instability.

It is recognised that there is substantial body of information available (more than would be normally available) about the site due to the extensive investigations taken in response to different subsidence concerns over many years. To aid clarity, a review was undertaken by Golder of these reports, their report provides its own analysis to support an opinion on the work of SRK and WAI. The conclusion of that review states the following:

"Predictions of underground mine stability below public roads (R179 and L4900) adjacent to the existing Knocknacran Open-Cast Mine and the proposed Knocknacran West Open-Cast Mine have been undertaken by SRK[1,2,3,4,11,13] and independently reviewed by Wardell Armstrong (WAI)[5,16,17]. Golder's review of their work concludes that their findings are reasonable. Where analytical methods could not be fully verified based on information presented in the available reports, independent checks confirmed the reasonableness of their conclusions. Predictions of mine roof stability are validated by cavity laser surveys showing minimal change in profile over many years. SRK, in various reports, recommends a programme of monitoring to identify symptoms of any change in stability of the mine workings and has presented TARPs for the two roadways (R179 and L4900). A regular monitoring program of this type is considered to be appropriate to manage the minor risk associated with the current and anticipated conditions. Maintaining the workings below the roads in a dewatered condition during future mining is considered to be prudent and the condition of the pillars and underground road intersection roofs should be reassessed prior to site remediation and mine closure."

The review report is provided in full in Appendix 7.11.

While the former Drumgoosat workings have historically been used to store water, this is no longer taking place. Currently, the workings are being dewatered by the Drumgoosat dewatering borehole pump located on the existing Knocknacran site to the south of the R179. The development proposes to relocate the borehole pump from the current Knocknacran site to an existing monitoring borehole located on the Knocknacran West Site. Further discussion on dewatering can be found in Chapter 8.0, Water.

Remediation of crownholes and fissures associated with subsidence events have taken place on the site. These remediation works were finished in 2020 (Appendix 3.1). The site of the former GAA grounds remains not in use, as does the wider site over the former Drumgoosat workings.

Regarding historic subsidence studies and occurrences in relation to Drummond Mine, SRK Consulting (UK) Ltd undertake annual audits of the underground mining operations at Drummond, specifically in relation to the avoidance and mitigation of surface subsidence to comply with Clause 7c of planning conditions Reg. Ref.: 03-578 since August 2006.

The most recent report (titled *'Drummond Mine Fifteenth Independent Review of Subsidence Monitoring Issues, December 2021'*, the 15th since the mine began operation) indicates that the mine is performing as designed, and is not giving rise to any material subsidence problems in the areas being monitored. A copy of the annual report is submitted to MCC each year.

In January 2023 a hole measuring 30 cm wide and 60 cm deep was discovered in a field adjacent to the L4900, on third party land. Ground surrounding the hole was found to be heavily disturbed and a review of aerial imagery since November 2022 shows that the hole was first visible in an early January drone image. The difference in the images from the end of November and the start of January indicates hedge trimming and surface working along the hedge line at the location of the surface hole during the month of December 2022. There is also evidence of a bush or similar located next to the surface hole prior to December 2022 and it is no longer present after the works. Records from the two extensometers near to the hole indicate there has been no movement of the roof beams in the mine workings under the L4900. It was concluded that the hole is a surface feature most likely caused by the ground disturbance associated with the works that involved hedge trimming and field maintenance during December 2022, and is not a subsidence feature and is not an indication of instability or of any impending subsidence event from the mine workings in the area.

Figure 7.17: Plan showing locations of historical known sinkholes (crownholes) over Knocknacran West which are numbered according to their chronology

7.4.10.2 Landslides

Landslides/mass movements typically occur due to erosion of features such as cliffs, or due to factors such as slope, saturation/drainage, vegetation, soil structure and loading/disturbance on sites with unconsolidated deposits such as peat. The Study Area is predominantly within an area of low landslide susceptibility (GSI, 2023). Some areas such as are noted as moderately low to moderately high landslide susceptibility within the existing open-cast area. A review of recent aerial imagery of the open-cast shows that these have been restored (i.e. material emplaced to lower slope angles and gradients) since the GSI data was published in 2016. It is considered that this area now has a lower landslide susceptibility (low). The GSI (2016) also notes that there have been no recorded landslide events within the Study Area.

7.4.10.3 Geotechnical Considerations for the Existing Open-Cast at Knocknacran

Following a review of the stratigraphy of the materials (overburden and interburden (mudstone and dolerite)) exposed by the excavated benches, a slope stability analysis of the Knocknacran Open-Cast found that the overall pit slope and individual bench slopes would achieve a minimum Factor of Safety (FoS) of 1.3 for the overall slope, and a minimum FoS of 1.1 for individual benches, for a worst-case scenario in which the bulk of the material to be excavated would consist of mudstone. Modelling showed that where dolerite was present greater values for FoS could be achieved. Section 7.6.5.9 and Appendix 7.12 provide details on the slope stability assessment of the proposed Knocknacran West Open-Cast.

7.4.10.4 Radon

The Radon Map for Ireland (EPA, 2023) indicates that there are variable levels of radon risk at the Site and within the study area. The centre of the Knocknacran Open-Cast indicates the radon risk would be 1 in 5 homes here are estimated to be above the radon reference level of 200 becquerel per cubic metre (Bq/m^3). Much of the wider Site and Study Area are expected to be between 1 in 10 and 1 in 20 homes above the reference level for radon. Given the radon risk is associated with homes (because they are indoor environments where the gas may accumulate) the higher risk associated with the existing Knocknacran Open-Cast does not apply here.

7.4.11 Geological Assets

There are no active quarries at or near the Proposed Development according to the extractive register on the GSI online viewer (GSI, 2023). However, the Site contains the existing Knocknacran Open-Cast Mine, and the Drummond Underground Mine is also adjacent. In the wider Study Area, Cormey Pit is operated by Breedon Bricks to the south and a limestone quarry is in operation to the east.

According to the mineral localities layer within the GSI (2023) online viewer, several mineral localities are noted within the Study Area including gypsum, clay, shale, dolomite, coal and marl (Figure 7.18).

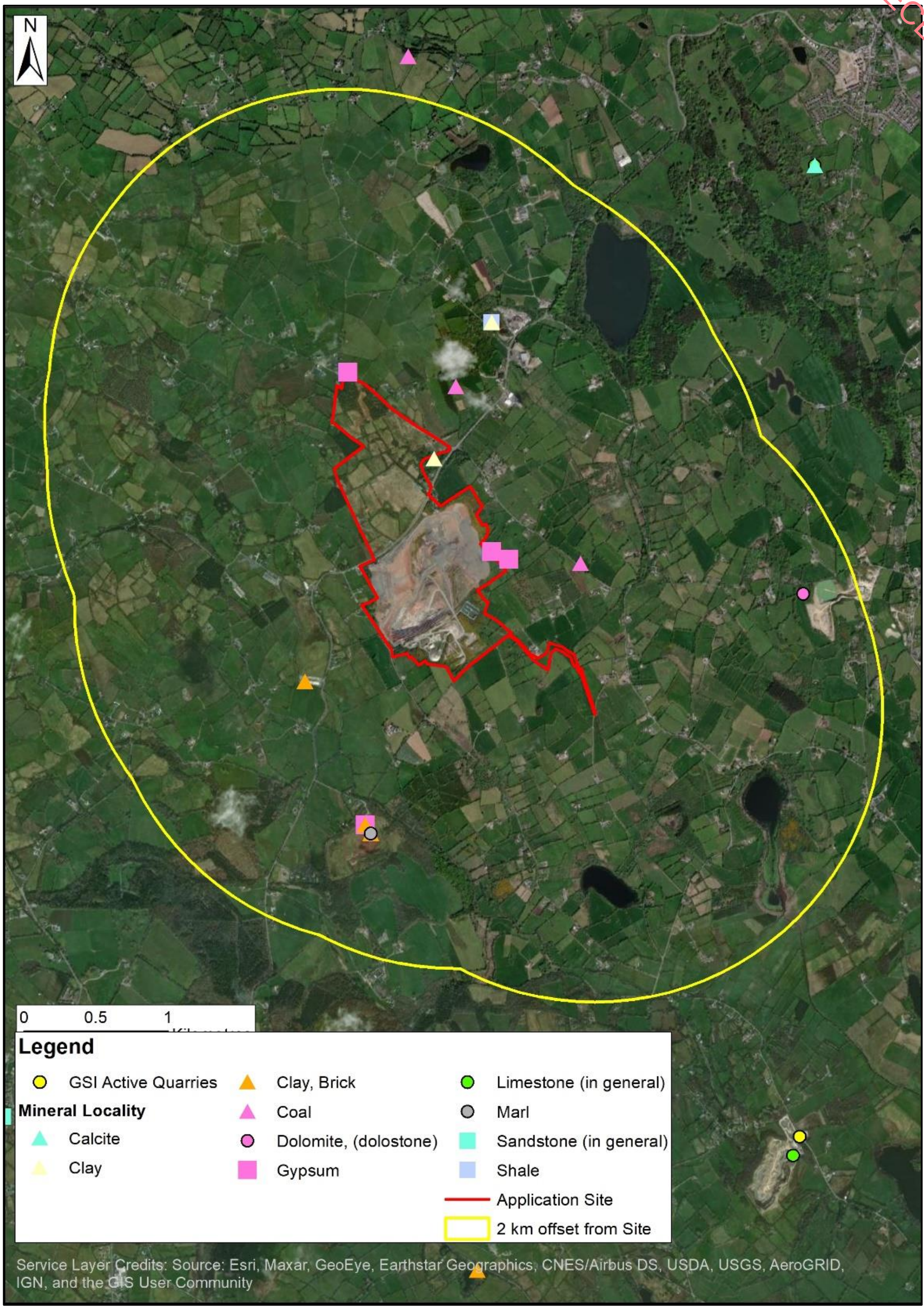

Figure 7.18: Mineral localities within the Study Area (GSI, 2023)

7.4.12 Geological Heritage

In their 2019 – 2025 county development plan, Monaghan County Council presents 5 policy objectives (GEP 1 to GEP 5) under their County Geological Sites Policy.

Following a county wide audit in 2013 under the Irish Geological Heritage Programme, the Geological Survey of Ireland (GSI) in conjunction with the Monaghan County Heritage Office did not identify any nationally geological important sites in the County. However, 20 locally important geological sites were identified and classified as County Geological Sites (CGS).

Within the Study Area, 2 geological audited heritage sites have been identified, Figure 7.19 below. One is MN010, the 'Knocknacran Gypsum Mine' and is the existing Knocknacran Open-Cast Mine site within the Site. The second is MN013, 'Mokeeran Quarry' and is located ca. 1.8 km east of the site and this is also an existing limestone quarry. The GSI notes that MN010 (Knocknacran Gypsum Mine) *'is a large open-cast gypsum mine, with numerous intersections into old underground mine workings. It is probably the largest man- made excavation in Ireland'.*

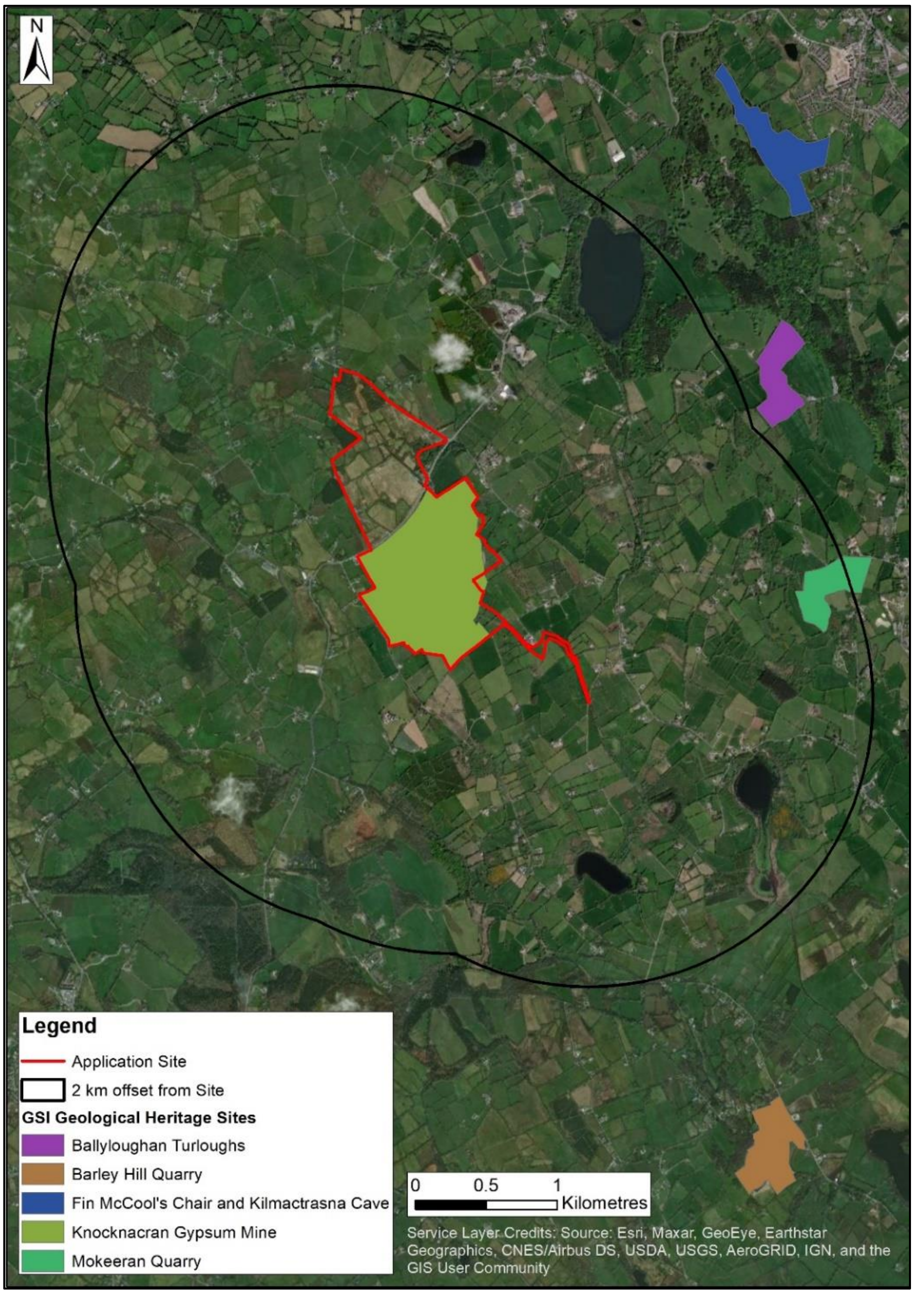

Figure 7.19: Geological Heritage sites within the Study Area (GSI, 2023)

7.4.13 Selection of Sensitive Receptors

Land (Land Use)

Consideration will be given to land within the Site boundary due to the Proposed Development. It is considered that the sensitivity of Land is 'Low'.

Soils & Subsoils

The proposed mining activities will require disturbance to natural soils and subsoils within the Knocknacran West site through the removal of these in the open-cast area and reuse in perimeter screening berms or in the restoration of mining areas.

The construction of the additional facilities at the Community Sports Complex site will require limited disturbance to natural soils and subsoils within the site to allow foundations to be laid for the additional facilities, however, this will be restricted to the shallow upper soil layer and any soil removed will remain onsite and reused in landscaping.

The superficial deposits at the Site and within the study area have no special designation, are not locally important, are not unique in the area and will remain onsite for use in the developments, it is considered that their sensitivity is 'Negligible'. However, consideration will be given further in the assessment to the potential impacts to soils and subsoils during the developments.

Bedrock

The bedrock within the Site and the Study Area is an uncommon bedrock, which is of 'High' importance at the mine site as an economic resource nationally. Consideration will be given to the bedrock as an asset under the topic of 'geological asset' below. It will also be considered as a potential receptor which may be impacted by any infiltration of leaks or spills into bedrock.

Geological Assets

The Proposed Development includes the proposed mining of a geological mineral resource at the Knocknacran West site. The sensitivity of this receptor is considered to be 'High' and this will be considered further in this assessment.

Palaeokarst

The impact and significance associated palaeokarst will be considered under 'Geohazards: Subsidence', below.

Geohazards: Landslides

There have been no known landslides within the Study Area and while an area of low to moderately high landscape susceptibility is noted within the existing open-cast by the GSI (2016), it is considered that restoration of this area since 2016 has lowered the susceptibility of such an event to a similar level (low) as the Study Area. Landslides will not be considered further in this assessment.

Geohazards: Subsidence

Receptors which will be considered in this assessment that have the potential to be impacted by a subsidence event include road users (human health receptor), mine workers (human health receptor) and the road network/infrastructure (e.g. ESB lines) over the workings. With regards to the potential for surface subsidence events to coincide with local residential receptors; it should be noted that these surface subsidence events may only occur where underground extraction has taken place directly below the property, and underground extraction has not taken place under such receptors. Underground workings extend under the majority of the Knocknacran West site, with some workings extending under the R179 and L4900.

Receptor sensitivity is considered 'High' for human beings and 'Medium' for built structures (including road and infrastructure).

Geohazards: Radon

Due to the nature of both the non-mining and mining related activities they will not be isolated to indoor activities (which would have poorer ventilation than outdoors). The activities occur predominantly outdoors. Therefore, radon will not be considered further within this assessment.

Geohazard: Geotechnical considerations for Knocknacran Open-Cast

The Proposed Development will involve the restoration of the open-cast at Knocknacran to near original ground levels thereby negating geotechnical considerations for an open-cast mine as it will no longer exist. However, there will be an open-cast mine with pit faces on the Knocknacran West site. Consideration will be given to workers and human receptors (human health receptors) further in this assessment and which are considered to have a 'High' sensitivity.

Geological Heritage

The existing open-cast at Knocknacran is considered a locally important geological site, and its sensitivity is considered to be 'Low'. It will be considered further in this assessment.

There is another locally important geological site within the Study Area (Mokeeran Quarry), however this is not within the Site boundary, there would be no potential impact on it, and it is not considered further in this assessment.

7.4.14 Sensitive Receptor Summary

Taking account of the above and the receptor classification method described in Section 7.3.3 the receptors carried forward in this assessment and their assigned importance are presented in Table 7.5.

Table 7.5: Soil, Land and Geology Receptors

Receptor	Sensitivity and Reasoning
Geological Asset	High (proven economically extractable mineral resource)
Human Health	High (human health receptor)

Built Structures	Medium (road network)
Soils and subsoils Bedrock (mudstone and dolerite units)	Negligible (no designation, no rarity, site importance)
Geological Heritage	Low (existing open-cast at Knocknacran, locally important geological site)
Land Quality	Low (local importance)

7.5 Key Characteristics of the Proposed Development

7.5.1 Construction Phase: Community Sports Complex

During this phase, the existing Community Sports Complex will be further developed. The initial phase of this development has been constructed (Reg. Ref.: 20/365), and the next phase will involve extending the Community Sports Complex with the construction of two further playing pitches, one with a perimeter running track, an all-weather pitch, a new club building, including a sports hall, a handball alley, changing rooms & toilets, a viewing gallery, a part-covered grandstand, additional parking and associated siteworks.

7.5.2 Construction Phase: Mine Development

During this phase:

- Screening berms will be constructed;
- Planting (including bolstering and retention of the existing perimeter hedgerow which sits in front of/is separate to the proposed planted screening berms) will be carried out;
- Perimeter fencing, will be installed;
- One residential house and three unoccupied houses and sheds on the Knocknacran West site will be demolished;
- A temporary diversion of the R179 will be constructed to maintain traffic flow while a Cut-and-Cover Tunnel is constructed; and
- A new vehicular entrance will be constructed to the existing mine site from the L4816.

7.5.3 Operational Phase: Community Sports Complex

During this phase, the Community Sports Complex will be in operation.

7.5.4 Operational Phase: Mine Development

The phased extraction of gypsum by open-cast mining methods at Knocknacran West from the closed (since 1989) Drumgoosat underground mine workings. In parallel, the Knocknacran Mine will be backfilled and remediated to near original ground levels. The proposed Mine Development amounts to the replacement of the loss of mining of gypsum at the Knocknacran Open-Cast Mine with the mining of gypsum at Knocknacran West Open-Cast Mine. Both mine sites are comparable in size and nature of operations.

During this phase:

- Overburden and Interburden will be stripped to expose the Gypsum Mineral at the new Knocknacran West Open cast mine. Gypsum will be extracted by open-cast mining methods;
- The stripping of the site will be undertaken in a series of campaigns at specific times and last for defined periods of time (typically < 6 months) over the life of the proposed Mine Development. The stripping earthworks will be undertaken by a specialist contractor following a tender process;
- The existing Knocknacran Mine will be restored to near original ground levels;
- The existing plant site will process and despatch the extracted gypsum;
- The existing Drumgoosat dewatering pump, will be relocated to an existing borehole on the Knocknacran West site to continue to provide dewatering; and
- The depth of mining will be to the base of the Lower gypsum, ca. - 53 m OD.

7.5.5 Restoration/Closure Phase: Community Sports Complex

There is no proposal to close the Community Sports Complex development, and this phase is therefore not applicable in this case.

7.5.6 Restoration/Closure Phase: Mine Development

During this phase:

- The new Knocknacran West site will be returned to grassland and a waterbody;
- The existing Knocknacran site will be returned to near original ground level;
- The existing Knocknacran Plant site will be partially dismantled whereby mine plant is removed; and
- In line with the current CRAMP it is presented that here that a suitable developer would be sought to utilise the general buildings existing on the existing site for a light industrial usage into the future. This would be subject to a future developer seeking the necessary permits for continuation of use and change of use from mining to a non-mining use.

7.6 Potential Effects

The main potential impacts and associated effects that will be considered in the assessment relate to the following:

- Activities or events that might impact soil or subsoil (e.g. leaks and spills from machinery or stored substances, or discharges, demolition of houses, soil importation for playing pitches);
- Loss of superficial deposits and bedrock;

- Impact on human health or built structures (road network) due to mining triggering a subsidence event; or
- Impact on human health (workers) from geotechnical instability leading to failure within the open-cast.

These are considered and assessed in the following sections.

7.6.1 Potential Effects: Construction Phase: Community Sports Complex

The construction of the Community Sports Complex will occur over a two year period and involve the initial construction of playing pitches estimated to take place over six to nine months. This will involve the removal of a shallow soil layer and the emplacement of suitably sourced clean soil. The soil used in pitch construction will be imported from a suitably approved supplier and material will comply with Article 27. The Community Sports Complex site contains insitu soils and subsoils on the western side, but the eastern side consists of a backfilled area of soils which were emplaced as part of the Knocknacran Mine site's phased restoration works, and the natural soil and subsoil profile no longer exists.

Construction will also occur to provide a facilities building, grandstand and ancillary works. Soil will be removed to enable the placing of foundations for these structures. Removed soil will be stored and reused onsite for landscaping purposes. The magnitude of the impact superficial deposits is considered to be Negligible (Adverse). The sensitivity of the receptor (soils and subsoils) is considered to be Negligible and the significance of the effect is considered to be Imperceptible.

The Community Sports Complex site is not located over underground mine workings, rather it is located on a part of the former mine site which has been partially backfilled or has been unmined on the western side and soils, subsoil and bedrock remain insitu. The risk of subsidence related to mining activities on the site is not considered to be applicable and has been scoped out of the assessment.

Fuel and other substance leaks or spills from stored substances or from machinery/equipment used during the construction of the Community Sports Complex could affect the chemistry of the soil during construction activities or could infiltrate to the groundwater through the bedrock. Material underlying the site is a mixture of natural and backfilled soils, subsoils and bedrock including tills, red clays of the Middle and Upper Mudstone Members and doleritic sands. Proposed construction activities would be undertaken by licenced contractors and regular maintenance of machinery/equipment would take place. Any leaks or spills would be small in scale and the underlying clays would hinder flow to bedrock. Therefore, the predicted potential impact on superficial deposits from potential fuel or other substance leaks is considered to be Negligible (Adverse). The sensitivity of the receptor is considered to be Negligible and the significance of the effect is considered to be Imperceptible.

7.6.2 Potential Effects: Construction Phase: Mine Development

Consideration in this construction section for mining activities will be related to the construction of a new mine entrance on the Knocknacran Mine site, the construction of the screening berms and the temporary diversion of the R179 and construction of the Cut-and-Cover tunnel connecting the Knocknacran West Mine site and the Knocknacran Mine site. The mining activities will also require the demolition of four houses, one of which is currently occupied and will be vacant prior to demolition works) and three unoccupied houses.

Impact of Construction of New Mine Entrance along L4816

To enable the development of the proposed mine entrance off the L4816, earthworks will occur and involve the removal of a shallow soil layer over an area of ca. 865 m^2 so that the road paving can be laid. Soil removed in the earthworks process will include at a minimum the topsoil and organic layers. Removed soil will be reused in landscaping around the new entrance and the former entrance. No soils will be exported offsite. Material brought to site for paving of the road will be sourced from an approved supplier to ensure the material is of suitable quality and free of potential contamination sources. An access point will be maintained to existing monitoring wells by the proposed entrance and routine monitoring of these wells will not be impacted by the proposed works, a layby has been accounted for in the design so access is maintained. The magnitude of the impact on superficial deposits is considered to be Negligible (Adverse). The sensitivity of the receptor is considered to be Negligible and the significance of the effect is considered to be Imperceptible.

Impact of Construction of Temporary Road Diversion

To enable the development of the temporary diversion of the R179 shallow soil will be removed to allow for the paving of the diversion road along an area of ca. 8,500 m^2. Soil removed will remain onsite to be used in screening berms or landscaping within the Site. Material brought to site for paving will be sourced from an approver supplier. The location of the existing extensometer network (TARP) along the existing R179 have been accounted for in the design of the diversion route to ensure that the boreholes remain in place and can be accessed throughout the works. Appendix 3.4 provides details of the proposed road diversion and Cut and Cover Tunnel. The diversion is temporary in nature and will be constructed in a 3 - 4 month period. The magnitude of the impact superficial deposits is considered to be Negligible (Adverse). The sensitivity of the receptor is considered to be Negligible and the significance of the effect is considered to be Imperceptible.

The construction of the Cut-and-Cover tunnel beneath the R179 will occur once the temporary road diversion has been constructed and cumulative impacts are not anticipated during the construction phase of these developments. The tunnel will require earthworks to remove the soil, subsoil and bedrock down to the depth of the base of the tunnel which is ca. 38 m OD. The bedrock unit the tunnel will be located in is the Upper Mudstone Member of the Kingscourt Gypsum Formation which is a soft clay at the location of the proposed tunnel. The tunnel construction area will be ca. 940 m^2. Material excavated will be reused onsite to either cover the tunnel once emplaced, or material will be stored in screening berms on the site. No material will be brought offsite and the materials for the construction of the tunnel will be sourced from suitable suppliers. Appendix 3.5 provides the Design Report for the proposed road diversion. The magnitude of the impact on the superficial deposits and bedrock is considered to be Negligible (Adverse). The sensitivity of the receptors are considered to be Negligible and the significance of the effect is considered to be Imperceptible.

Impact of Construction of the Cut-and-Cover Tunnel

The construction of the Cut-and-Cover tunnel beneath the R179 will occur once the temporary road diversion has been constructed and cumulative impacts are not anticipated during the construction phase of these developments. The tunnel will require earthworks to remove the soil, subsoil and bedrock down to the depth of the base of the tunnel which is ca. 38 m OD. The bedrock unit the tunnel will be located in is the Upper Mudstone Member of the Kingscourt Gypsum Formation which is a soft clay at the location of the proposed tunnel. The tunnel construction area will be ca. 940 m^2. Material excavated will be reused onsite to either cover the tunnel once emplaced, or material will be stored in screening berms on the site. No material will be brought offsite and the materials for the construction of the tunnel will be sourced from suitable suppliers. Appendix 3.5 provides the Design Report for the proposed Cut-and-Cover Tunnel. The magnitude of the impact on the superficial deposits and bedrock is considered to be Negligible (Adverse). The sensitivity of the receptors are considered to be Negligible and the significance of the effect is considered to be Imperceptible.

Consideration has been given to the potential interaction between the Cut-and-Cover Tunnel and the underlying mine workings. The Cut-and-Cover Tunnel will be excavated ca. 32 m above the underlying pillars. The pillars beneath the proposed tunnel are at least 19 m x 35 m and modelling showed that the excavation of the tunnel does not affect the stability of the underground workings. Modelling demonstrated that displacements of the room roofs (beams) due to the excavation of the tunnel are upwards, due to the elastic rebound of the rock after removal of material to form the tunnel cut. Appendix 7.13 provides full details of the stability modelling associated with the tunnel.

It is considered that the potential impact magnitude of the change in stability of the underground workings caused by the placement of the Cut-and-Cover tunnel is Negligible (Beneficial). In turn, this considers that the change in stability of the underground workings below the Cut-and-Cover tunnel and the resulting potential impact magnitude on built structures such as the R179/L4900 or people in the area during the operational life of the mine is Negligible (Beneficial). The sensitivity of the receptor is considered to be High and the significance of the effect is considered to be Slight.

Impact of Construction of Landscaping Features

Approximately 200,000 t of stripped material will be used to construct the perimeter screening berms around the proposed Knocknacran West Mine. The potential impact magnitude on soils and subsoils is considered in the context that these materials will not be removed from the site and they will be stored within screening berms and therefore they will not be lost or removed permanently from site. The potential impact magnitude on the superficial deposits is considered to be Negligible (Adverse). The sensitivity of the receptor is considered to be Negligible and the significance of the effect is considered to be Imperceptible.

Impact of Machinery Operating during Construction

Potential fuel and or leaks from machinery or items stored on site have the potential to impact the underlying soils, subsoils and bedrock. In the area of the diversion and tunnel recent works have been undertaken to characterise the soil for the purposes of an effluent treatment system for the future site office and welfare facility on the Knocknacran West site. As part of the work, trial holes were dug to test the nature of the shallow soils. The soils were poorly drained given the nature of the soil in the area (Appendix 3.2). Any spills or leaks would be contained and the natural subsurface material would inhibit rapid percolation at this location. Works for the proposed new mine entrance would be short in duration and with limited plant and machinery needed. Plant and machinery will be regularly maintained and inspected for leaks or spills and bunds will be placed onsite around items requiring a bund for both the tunnel, diversion and new entrance works. The magnitude of the impact on the superficial deposits and bedrock from potential leaks and spills is considered to be Negligible (Adverse). The sensitivity of the receptors are considered to be Negligible and the significance of the effect is considered to be Imperceptible.

Impact of the Demolition Works

Regarding the demolition of the four houses, the works will be undertaken by a licenced contractor. Prior to the demolition of the structures, checks of items which should be removed will be undertaken (such as stripping of electrical wiring). Demolition material will be segregated and material which can be reused onsite will be retained on site, other material will be removed from site by a suitably qualified and licenced operator to ensure no potential contamination of demolition material or surrounding soils and subsoils. The magnitude of the potential impacts from the demolition of the four house is considered to be Negligible (Adverse). The sensitivity of the receptor is considered to be Negligible and the significance of the effect is considered to be Imperceptible.

Impact of Construction Operations on Stability of Mine Workings
Consideration has been given to the potential risk of subsidence caused by personnel or plant on the mine site working above the former underground mine during construction. SRK carried out an assessment of the impact of construction vibration effects and conclude in their technical memorandum that ground vibrations initiated by equipment (during construction) are unlikely to cause any new subsidence on the Site.

Figure 7.20 (reproduced from the SRK report), highlights in orange the main types of equipment that would operate in an open-cast environment, large dozers, trucks and blasthole drills (jack hammers). Also included are various typical vibration thresholds – perception (0.5 mm/sec), damage to residential buildings (50 mm/sec) and damage to commercial buildings (100 mm/sec).

The potential impact magnitude on the workings is considered to be Negligible (Adverse). The sensitivity of the receptors are considered to be Negligible and the significance of the effect is considered to be Imperceptible.

Appendix 7.14 provides the SRK report.

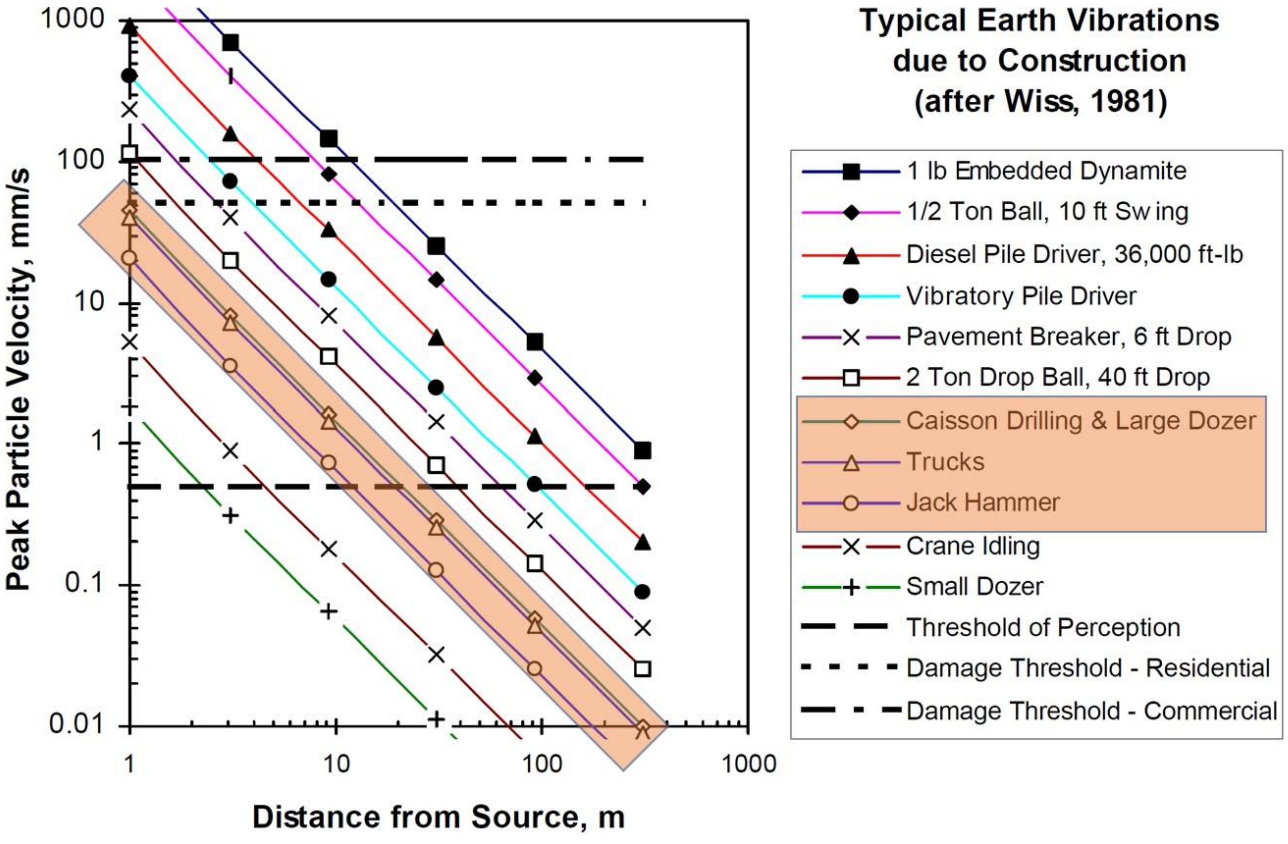

Figure 7.20: Vibration Attenuation Graph for Typical Construction Equipment

7.6.3 Potential Effects: Operational Phase: Community Sports Complex

Once the Community Sports Complex is constructed, there would be no further impacts on soils, subsoils and bedrock beneath the site and this has been scoped out of the assessment.

7.6.4 Potential Effects: Operational Phase: Mine Development

The extraction at the Knocknacran West Mine will require excavation of soil, subsoil and bedrock layers. Topsoil and subsoils are both finite resources, however, these soils will remain onsite to be used in the restoration of Knocknacran Mine and in the later restoration of Knocknacran West Mine. As previously stated, the bedrock units at this site occur as soft, principally unconsolidated sediments that can be removed by excavator and used for restoration purposes onsite. The removal of these materials from the Knocknacran West site will occur on a phased basis in stripping campaigns. Excavated soil and subsoil used to cover stripped bedrock material will be seeded with native grasses away from nearby watercourses and drainage ditches to prevent erosion and run-off.

During the operational phase, the Plant Site at Knocknacran will not produce further impacts on the underlying soils, subsoils and geology at the site beyond the emplacement of foundations for the proposed Knocknacran West conveyor system to connect to the existing processing system; all removed soils will remain onsite for reuse in berms or restoration. The magnitude of the potential impacts is considered to be Negligible (Adverse). The sensitivity of the receptors are considered to be Negligible and the significance of the effect is considered to be Imperceptible.

The restoration of Knocknacran Mine will allow the former open-cast at this site to be returned entirely to near original ground levels, rather than the currently permitted (Reg. Reg. 17/217) waterbody with some areas of near-original ground. The reuse of removed soils, subsoils and bedrock on the Knocknacran West site will also allow the northern open-cast area within this site to be restored to near original ground levels. During the excavation of the southern extraction area, excess material will be initially stored on the former northern extraction area (i.e. stockpiled). This will result in the short-term effect of the northern area being backfilled higher than original ground levels, until the southern extraction area is mined out and material can then be placed in the southern open-cast area.

The potential impact magnitude on soils, subsoils and bedrock is considered in the context that these materials will not be removed from the site and they will be stored for either future restoration or they will be immediately used in restoration areas, and therefore they will not be lost or removed from site. Consideration to the visual impact of the mine development is considered further in Chapter 13.0 (Landscape and Visual). The potential impacts on groundwater from excavation of the gypsum below the water-table are addressed in Chapter 8.0 (Water). The potential impact magnitude on the superficial deposits is considered to be Negligible (Adverse). The sensitivity of the receptors are considered to be Negligible and the significance of the effect is considered to be Imperceptible.

Restoring the Knocknacran Mine to near original ground levels will have an impact on the site as a locally important geological site (site code MN010), as the existing open-cast will be filled during the restoration process. Extensive records of the Knocknacran existing mine have been made over its lifetime. With gypsum being a partially water soluble mineral, the CRAMP for the site requires all mineral to be covered within a short time period after closure; it is therefore proposed not to retain any areas of exposed gypsum in the existing Knocknacran Open-Cast Mine.

As the southern extraction area of the proposed Knocknacran West Open-Cast will form a waterbody with a seasonally variable water level, some areas of the former open-cast could remain visible at times, although in limited capacity after restoration has occurred. Access requests from interested geological stakeholders (e.g. the Geological Survey of Ireland) would be facilitated during the extraction life of the Knocknacran West Mine to allow for geological features to be recorded prior to extraction and/or backfilling of the mine. The impact magnitude of the loss of Knocknacran Mine as a locally important geological site is considered to be

Negligible (Adverse) while the proposed development of the Knocknacran West Mine is considered to be Negligible (Beneficial), as it would allow for a compensatory important geological site to be recorded in the same geological setting. The sensitivity of the receptors are considered to be Negligible and the significance of the effect is considered to be Imperceptible.

The gypsum resource at Knocknacran West is a geological asset. This allows for the exploitation of the geological asset which has a predicted impact that is considered to be Medium (Beneficial). The sensitivity of the receptors are considered to be High and the significance of the effect is considered to be Large (beneficial).

Fuel and other substance leaks or spills from stored substances or from machinery/equipment used during the operational life of the mine could also affect the chemistry of the soil on the mine sites. However, proposed mining activities will be undertaken by trained and experienced miners and contractors and regular maintenance of machinery/equipment will take place on the Knocknacran Plant Site. Fuels and other substances would be stored onsite in bunded and secured areas and checks would occur regularly. Therefore, the predicted potential impact on superficial deposits and bedrock from potential fuel or other substance leaks is considered to be Negligible (Adverse). The sensitivity of the receptors are considered to be Negligible and the significance of the effect is considered to be Imperceptible.

During the operational life of the Knocknacran West Mine, the Plant Site will not have further impacts on the underlying soils, subsoils and geology at the site and is not considered further here.

Mining Studies

Mining involves the extraction of ore or mineral (in this case gypsum) creating a void where none existed before.

The mining method for the future recovery of gypsum from the proposed Knocknacran West Mine is by the open-cast (pit) mining method of drilling and blasting by benching. While the majority of the historical underground workings will be extracted with the gypsum as part of the proposed open-cast mine, underground workings will remain below the R179 Kingscourt to Carrickmacross road, and below the local road L4900, linking the R179 to Drumgoosat.

A potential failure mechanism was identified by SRK (July 2020, Appendices 7.8 and August 2019, Appendix 7.9) for both the R179 and L4900 after recent subsidence events over underground workings in the vicinity of these roads. This potential failure mechanism was considered independent of the Proposed Development, however, it is considered relevant in this assessment. It is considered by SRK (July 2020 and August 2019) that future instability would likely be progressive in nature and that it would be the same failure mechanism for either road. This potential failure mechanism requires the following sequence of events to occur in the underground workings:

1) The roof beam blocks or slabs of gypsum may start to detach from the roof resulting in a thinning of the roof beam at the point of detachment.
2) If this process continues to propagate through the roof beam, eventually the beam would become thin enough that the overlying weight of the overlying glacial drift, mudstone and dolerite would cause the roof beam to collapse and fail.
3) With no roof beam to support the overlying strata, this material would become free to flow or fall into the old mine workings. This is usually a slow process as these materials are (to a degree) self-supporting; and there could be a collapse causing the development of a crownhole on surface.

Given the historical subsidence events related to the underground workings and assessments of future stability, a number of concerns were explored by way of further modelling and assessment carried out by SRK. These are considered in the following subsections.

7.6.5.1 Impact of open-cast mining, and cut and cover tunnel on the existing mine workings and the R179 and L4900 roads

As noted in Appendix 7.13, the Cut-and-Cover Tunnel will be excavated ca. 32 m above the underlying pillars. The pillars beneath the proposed tunnel are at least 19 m x 35 m and modelling has shown that the excavation of the tunnel does not affect the stability of the underground workings. Modelling demonstrated that displacements of the room roofs (beams) due to the excavation of the tunnel are upwards, due to the elastic rebound of the rock after removal of material to form the tunnel cut.

The potential failure mechanism for a future subsidence event for workings under the road is that the roof beams thin and pressure from overlying sediments collapses the beam, resulting in accommodation space being created for overlying sediments to migrate downwards and voids at surface to open up. However, Appendix 7.13, indicates that in the case where the roof relaxes due to elastic rebound, this means that some of the overlying sediment weight is removed resulting in less weight and pressure on the roof from remaining overlying sediment. This allows the roof to rebound upwards slightly, it does not indicate that the roof has thinned nor is space created for sediment to flow downwards as the roof beam would not fail with less weight and pressure on it from overlying sediment.

As Knocknacran West Mine is progressed, where sections of the underground workings are exposed in the sidewalls (i.e. outside the extraction area and which will not be mined) they will be inspected and assessed by competent experts. If assessments deem further action is warranted to optimise geotechnical stability, particularly outside the Site boundary, it will be discussed with the relevant Authorities and plans will be emplaced. The potential mechanism for failure in both areas is considered to be progressive in nature and the TARP along the R179 and L4900 will remain in place as an early warning system to ensure additional mitigation could be enacted to offset an event.

Further consideration was given by SRK (Appendix 7.13) to the potential for dissolution of the workings remaining under the R179 after closure. The SRK report considered that the backfill design must ensure that surface water cannot be allowed to directly enter the mine workings.

It is considered that the change in stability of the underground workings and the resulting potential impact magnitude on built structures such as the R179/L4900 during the operational life of the mine is Negligible (adverse). The sensitivity of the receptor is considered to be High and the significance of the effect is considered to be Slight.

7.6.5.2 Long-Term Mine Stability due to the Mine Development

SRK have assessed the potential impact (or not) of the Knocknacran West Mine on the existing underground workings and on the surrounding roads, the R179 and L4900. In addition, they also looked at the potential impact of backfilling (remediating) the existing open-cast at Knocknacran Mine on the underground workings and the R179 and L4900. The analysis used updates of previous 2D finite element models which intersect both the road and mine slopes. The models were set up to calculate the additional deformation at the road surface generated following the excavation of the mine slopes adjacent to the roads. The analysis found that the excavation of Knocknacran West will have no impact on the stability of the workings below the R179 and L4900, and therefore, will not affect the roads themselves. The outcome of their initial analysis is provided in Appendix 7.13.

A further study was undertaken by SRK to assess the long-term mine stability of the existing underground mine workings (Appendix 7.15) due to the proposed Mine Development, in particular to consider the impact of excavating the Knocknacran West open-cast followed by the backfilling of the existing open-cast Knocknacran Mine on the stability of the underground workings beneath the R179. The impact of the existing void (at Knocknacran) and the backfilling operation planned within the existing Knocknacran Open-Cast Mine.

The SRK report (Appendix 7.15) considers two additional cross-sections which are orientated at right angles to the open-cast slopes at their deepest part, and which interest the R179.

Modelling was undertaken on the following mining sequence:

- Excavate the Drumgoosat Underground Mine;
- Excavate the Knocknacran Open-Cast Mine;
- Partially backfill the Knocknacran Mine (this represents the current conditions);
- Excavate the Knocknacran West Open-Cast Mine;
- Restore the Knocknacran Open-Cast Mine to near original ground levels; and
- Backfill the base of the Knocknacran West Open-Cast Mine.

The results show that the model is stable for all stages. Mining of the Knocknacran West Open-Cast Mine has a very small potential impact on roof or on surface deformation, in the range of -1 mm to + 2 mm.

Restoration of the Knocknacran Open-Cast Mine has a negligible potential impact on roof or on surface deformation, in the range of 0 mm to + 1 mm.

Restoration of Knocknacran Open-Cast Mine (occurring during the operational life of Knocknacran West) has no impact potential (0 mm) on either roof or on surface deformation.

It is considered that the change in long-term stability of the underground workings due to the proposed Mine Development and the resulting potential impact magnitude during the operational life of the mine is Negligible. The sensitivity of the receptor (built structures) is considered to be High and the significance of the effect is considered to be Slight.

7.6.5.3 Consideration of the Impact of the Historical Failure of Underlying Pillars

Consideration has been given to the failure of underlying pillars that has previously occurred within the area of the proposed open-cast mine to establish whether further movement is possible. This is included in a report commissioned by SRK (Appendix 7.16) The report should be read in conjunction with Appendix 7.15 on the long-term mine stability. In summary, the report (Appendix 7.16) notes that all of the finite element modelling work undertaken in 2021 and 2022 indicates that the pillars will remain stable during the extraction of gypsum from Knocknacran West. It also considers that gypsum pillars and roof beams that have been affected by subsidence will have been filled by broken rock. It is likely that because of the shape and size of the broken rock, there will be voids present between the rocks when they are excavated. The position and extent of these areas of broken rock is reasonably well defined from detailed surface surveying undertaken over many months.

SGMI has experience in working in areas with historical underground workings in the existing Knocknacran Open-Cast Mine area. In addition, mining above underground workings is undertaken worldwide, and safe practices have been developed, most notably in Australia, where guidelines for void management have been developed and tested in operating mines.

SGMI will build on their existing experience in mining at Knocknacran, and further refine the operational procedures used in Knocknacran to identify and mitigate risks from voids prior to the commencement of overburden/interburden stripping on the Knocknacran West site. This operating procedure and method of safe working will be updated and amended, as needed, throughout the life of the proposed development at Knocknacran West. A copy of the proposed operating procedure developed for the Knocknacran West Mine based on experience gained from the Knocknacran Mine is provided in Appendix 7.17.

It is considered that the potential impact magnitude of the proposed Mine Development interacting with the historical failed pillars during the operational life of the mine is Negligible. The sensitivity of the receptor (i.e. value of underground workings) is considered to be Negligible and the significance of the effect is considered to be Imperceptible.

7.5.5.4 Consideration of Roof Beam Stability in the Historical Underground Workings

A further report assesses the roof beam stability in the Drumgoosat underground workings (Appendix 7.18). This report notes that as overburden is removed during the normal process of excavating the open-cast mine, loading of the roof beams decreases. 3D finite element modelling was undertaken which looked at a range of room and four-way intersection spans, roof beam thicknesses, and open-cast excavation depths to the underground workings. The model simulations all indicated that roof beam stability remains stable irrespective of roof beam span, thickness or overburden loading.

The analysis demonstrates that as material is removed, loading on the Lower Gypsum rock mass that forms the roof beam is reduced. This results in elastic rebound of the rock mass, which in theory, should improve the stability of the workings.

However, when specifically looking at areas where the open-cast slope/toe, underground mine workings and orthogonal sub-vertical jointing (normally orientated approximately N-S and E-W) in the Gypsum, intersect, the creation of vertical prismoidal blocks that could drop out of the room beam occurs. Evidence for this is provided in Figure 7.20 below, which shows where prismoidal blocks have dropped from the roof beam where the Knocknacran Open-Cast Mine had intersected the old Drumgoosat underground mine workings along the western wall of the Knocknacran Mine adjacent to the R179.

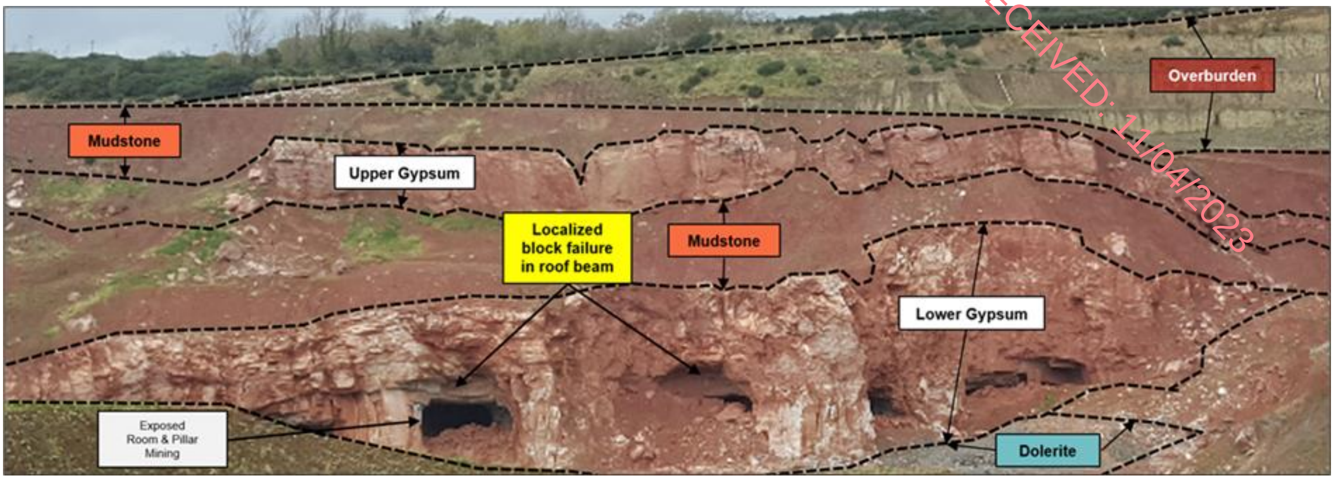

Figure 7.21: Roof beam showing drop out of prismoidal blocks in Knocknacran Open-Cast Mine

Based on the experience of dealing with these conditions at the existing Knocknacran Mine, an assessment has been made on the stability of the historical underground workings where they intersect with the toe of the proposed Knocknacran West Open-Cast Mine, in that the stability of the workings will be locally confined to the immediate roof beams of the workings.

Experience gained from mining in Knocknacran has led to the development of a proposed Standard Operating Procedure (SOP) for the safe extraction of Gypsum in such a scenario. The proposed SOP is provided in Appendix 7.17.

It is considered that the potential impact magnitude on roof beam stability due to the proposed Mine Development during the operational life of the mine is Negligible. The sensitivity of the receptor (roof beams) is considered to be Negligible and the significance of the effect is considered to be Impereceptible.

7.6.5.5 Potential for Dissolution of Gypsum and impact on stability of workings under the R179

Consideration was given by SRK to the potential for dissolution of the workings remaining (Appendix 7.13 refers). The SRK report advised that the backfill design must ensure that surface water cannot be allowed to directly enter the mine workings.

An additional report by Piteau Associates (*Hydrogeology Study of Drumgoosat Underground Workings, May 2021*), produced on the hydrogeochemistry of the Drumgoosat underground workings (at present) concluded that the actual saturation state of the water entering the mine workings would not cause dissolution of the gypsum in the mine workings (Appendix 8.8).

Figure 7.22 below shows the proposed methodology for the sealing of the historical underground mine workings in conjunction with the backfilling of the open-cast mine floor/base thereby ensuring that surface water cannot directly enter the underground mine workings. Underground workings exposed within the proposed open-cast mine will be sealed in the same way.

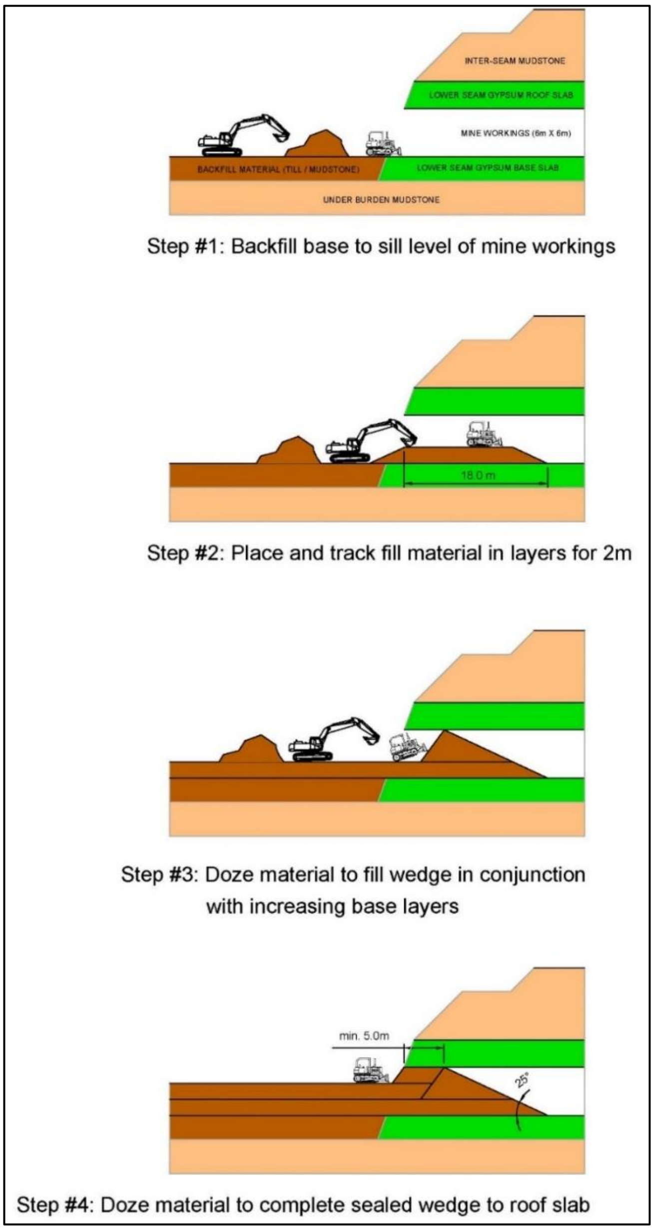

Figure 7.22: Methodology for Sealing of the Mine Workings at base of quarry slope

It is considered that the potential impact magnitude of gypsum dissolution causing stability issues due to the proposed Mine Development during the operational life of the mine is Negligible. The sensitivity of the receptor (built structure – R179) is considered to be High and the significance of the effect is considered to be Slight.

7.6.5.6 Impact of blasting on the extensometer network

During the life of the proposed mine, extensive monitoring of the historical Drumgoosat underground mine workings will continue to take place using the instrumentation installed along the R179 and L4900 to monitor the integrity of the pillars underlying these roads.

Appendix 7.13, considers the effect of blasting on the installed instrumentation (extensometers) in the monitoring boreholes along the L4900 and, considered the future extensometers along the R179 which were planned to be installed (and have since been) after the report had been written.

SRK also considered in this report that the impact of blasting on the extensometer network, and concluded that the instruments will remain unaffected by blasting.

Therefore, it is considered that the potential impact magnitude of blasting due to the proposed Mine Development on the extensometer network is Negligible. The sensitivity of the receptor (extensometer network) is considered to be Negligible and the significance of the effect is considered to be Imperceptible.

7.6.5.7 Potential Risk of Subsidence due to personnel, plant and vibration at Knocknacran West

Consideration has been given to the potential risk of subsidence caused by personnel or plant operating on the mine site. SRK conclude in their technical memorandum (Appendix 7.14) that ground vibrations initiated by equipment (during operations) and blasting (during operations) are unlikely to cause any new subsidence. on the Site.

An extract from the report states:

"Vibration is normally measured as peak particle velocity (PPV) in mm/second. PPV reduces or attenuates as the distance from the source of vibration increases. Relationship between PPV and distance from source for a number of types of moving plant has been developed by Wiss (1981)."

This relationship is shown in Figure 7.23, below:

"Highlighted in orange are the main types of equipment that would operate in an open-cast environment, large dozers, trucks and blasthole drills (jack hammers). Also included are various typical vibration thresholds – perception (0.5 mm/sec), damage to residential buildings (50 mm/sec) and damage to commercial buildings (100 mm/sec).

It can be seen that for the equipment that any vibration generated by them would become imperceptible from 10 m to 20 m away from the equipment. All equipment considered generates vibration which lies below the residential damage threshold at a distance of 1 m away."

As the closest residential receptor is ca. 100 m from any potential source of vibration associated with the movement of overburden and interburden, the analysis indicates that the level of vibration necessary to cause damage to residential property will be contained to within 1 m of the operating plant.

Regarding previous subsidence, as the floor of the open-cast mine is excavated towards the underground workings that have been subject to subsidence, the possibility of voids occurring in these areas will increase. An assessment of these areas will be addressed on an operational basis, bench by bench, with the working method forming part of the safe operating procedure for mining through voids and unstable ground, as is currently the case when mining in Knocknacran Open-Cast Mine above historic underground mine workings.

Appendix 7.17 provides a proposed Standard Operating Procedure (SOP) based on that currently used when mining in the existing Knocknacran Open-Cast Mine over previously mined out areas. This SOP takes into account pre-existing subsidence events and the risk for potential subsidence development during open-cast mining activities.

It is considered that the potential impact magnitude of the change in stability of the underground workings caused by the mine operation (specifically the plant vibration/extra working plant and blasting) is Negligible

(Adverse). In turn, this considers that the change in stability of the underground workings and the resulting potential impact magnitude on built structures and human health (such as workers) during the operational life of the mine is Negligible (Adverse). The sensitivity of the receptor is considered to be High and the significance of the effect is considered to be Slight.

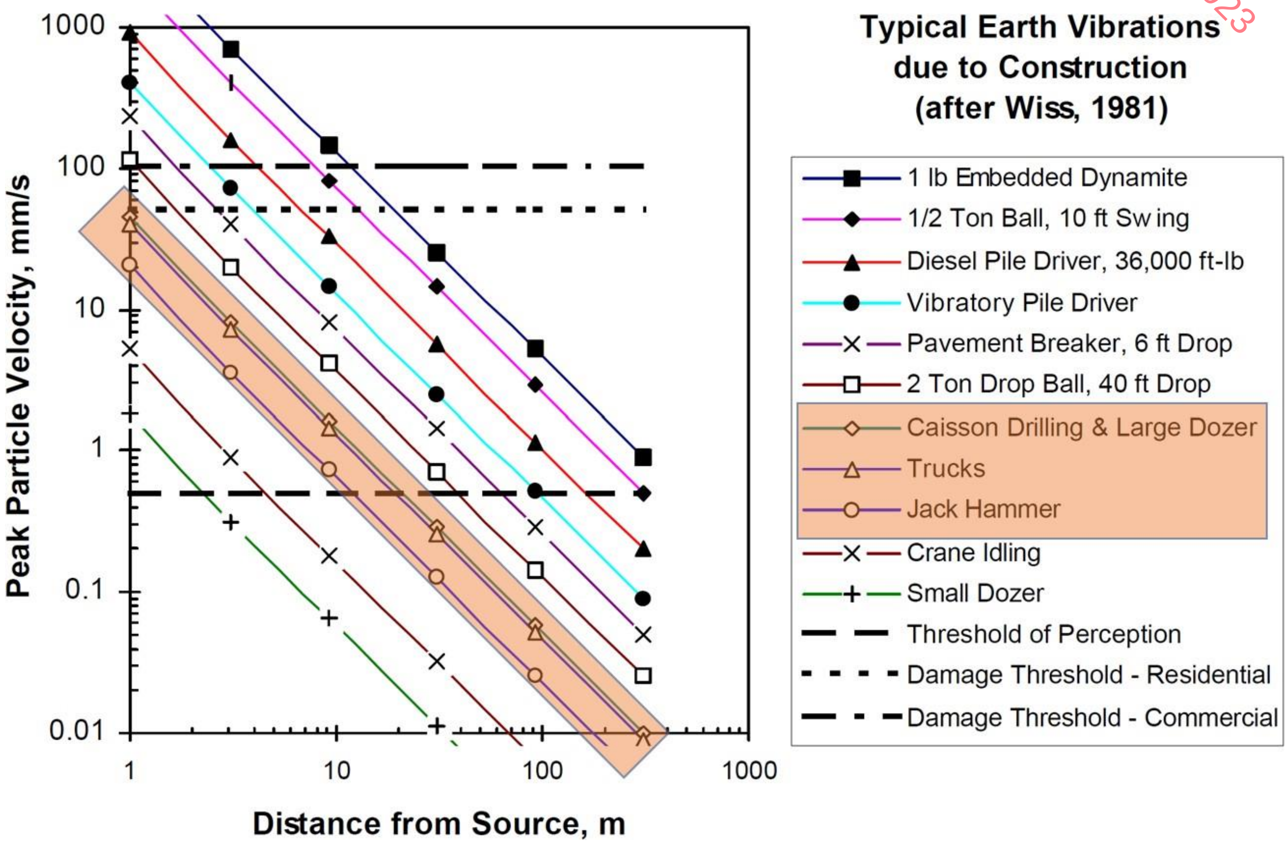

Figure 7.23: Vibration Attenuation Graph for Typical Construction Equipment

7.6.5.8 Operational Phase Backfill Under the Roads

During the operational phase of the development, there is a proposal to engage a reputable third-party engineering company (specialising in ground support) to assess the opportunity and practicalities of accessing the workings to carryout support work to ensure continued ground stability under the roadways where the mine workings occur. This will be presented within this section; however, this is not included within the impact assessment for significance of effects as the findings of such work are not yet known and their magnitude of change and significance cannot be concluded at this stage.

Studies on the mine workings that pass under the R179 and L4900 roads indicate that the workings are stable and expected to stay so, regardless of the Mine Development (Appendices 7.7, 7.8, 7.9 and 7.15).

As detailed in Section 7.6.5.1 studies carried out to assess the impact of the Mine Development have also indicated that the development will not adversely impact the stability of the mine workings and road.

Hydrogeological studies have concluded that the long-term stability of the workings (post closure of the Knocknacran West development) is not a concern (Chapter 8.0).

Notwithstanding the confidence in the future stability of the mine workings from the studies carried out, SGMI have taken the step to install extensometer sets at 13 locations under the L4900 (No. 8) and R179 (No. 5) adjacent to the mine workings. These instruments allow any movements (sub mm) in the roof beam of the mine workings to be measured live and in real time. The system also will send an alert in the event of readings being noted according to the TARP that remains under the supervision of mine management and local authorities (Appendix 7.8 and Appendix 7.9).

As the Knocknacran West Mine is developed the old Drumgoosat underground workings will become exposed, providing an opportunity to better investigate and inspect the condition and potential long-term stability of the workings. Prior to the completion and closure of each operational phase of the development, a reputable third-party engineering company (specialising in ground support) will be retained by SGMI to assess the opportunity and practicalities of accessing the workings to carryout backfilling of old mine tunnels to provide long-term assurance of ground stability under the roadways where the mine workings occur.

SGMI will undertake to implement backfilling to the underground workings under the L4900 and R179 roads where it is recommended following inspection by the specialist ground support engineer, so long as:

1) Inspection and subsequent placement of support can be carried out safely;
2) Risks of roof beam and pillar instability can be minimised; and
3) The need for ongoing long-term monitoring of the underground mine workings can be eliminated.

It is proposed that this network of extensometers is retained and maintained along with the associated TARP until an approved backfilling solution is agreed with the relevant authorities and implemented.

A proposed methodology (Appendix 7.19) for backfilling has been developed and is summarised below.

The locations of the underground workings for backfilling under the R179 and L4900 roads have been identified from historical mine survey records as shown in Figure 7.24.

On intersecting an opening to the historical Drumgoosat underground mine workings during the development of the Knocknacran Open-Cast Mine, SGMI will carry out the following actions, dependant on safe working conditions:

- Confirm location of mine opening(s) with respect to historical mine survey plans;
- Conduct an initial Geotechnical Assessment;
- Characterise mine opening(s) in terms of stability;
- Conduct ground support remediation works along the length of the tunnels to provide safe access to workings under the roads;
- Construct buttress walls to contain/support backfill materials;
- Following construction of buttress walls, rockfill will be placed as backfill in all 4-way-junctions under the R179 and L4900 to provide long-term stability of underground mine workings (Figure 7.25);
- A photographic record of the works will be made for each location; and

- A final topographical survey of the buttress locations will be completed prior to vacating the underground mine workings.

Where backfill cannot be placed safely, it is proposed that the extensometers are retained until agreement that they are no longer required or confidence in the future stability of the roads network has been superseded by an alternative means of managing the concerns regarding mine instability on road safety with the relevant authorities.

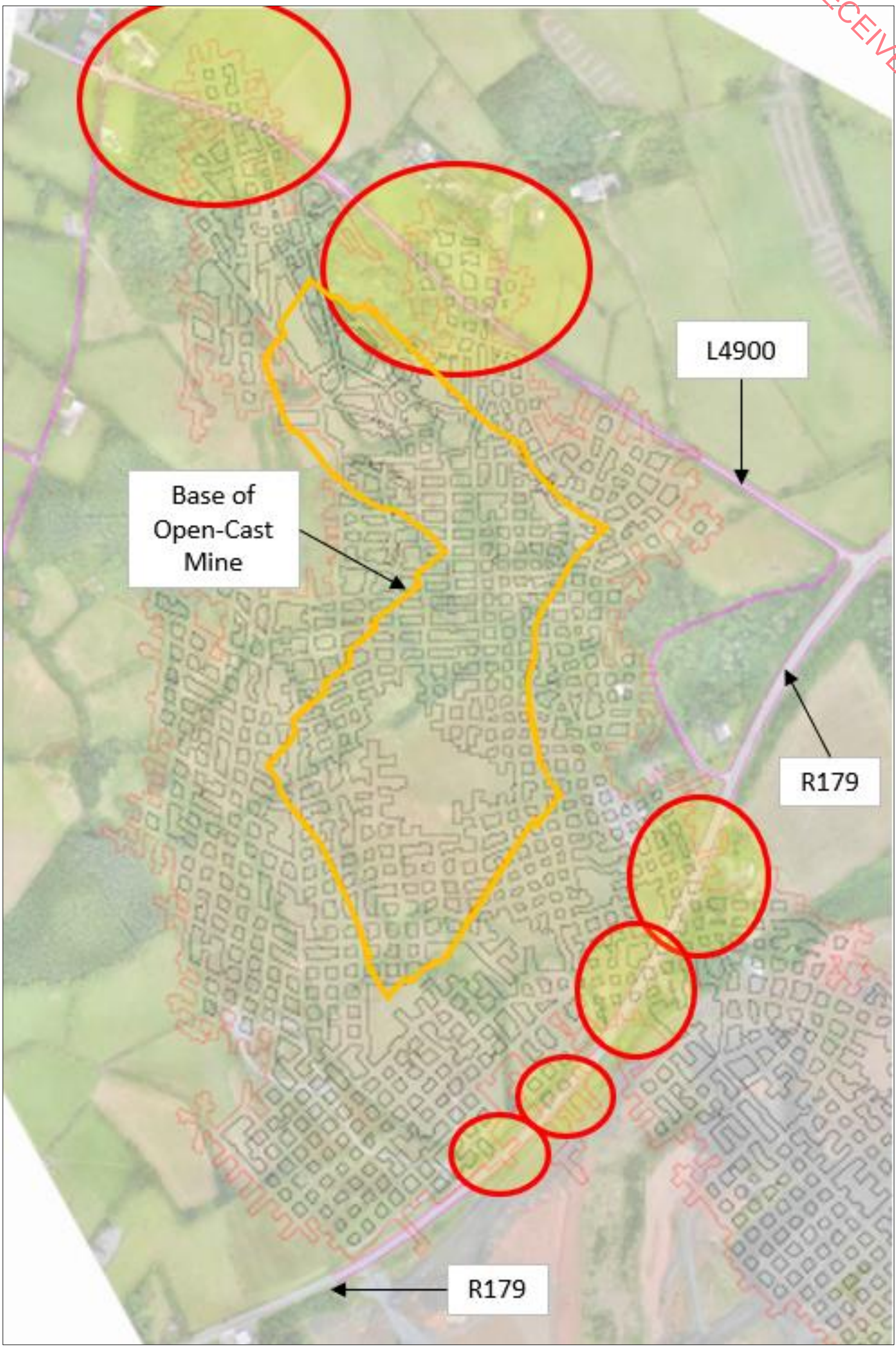

Figure 7.24: Areas for Backfilling under the R179 and L4900

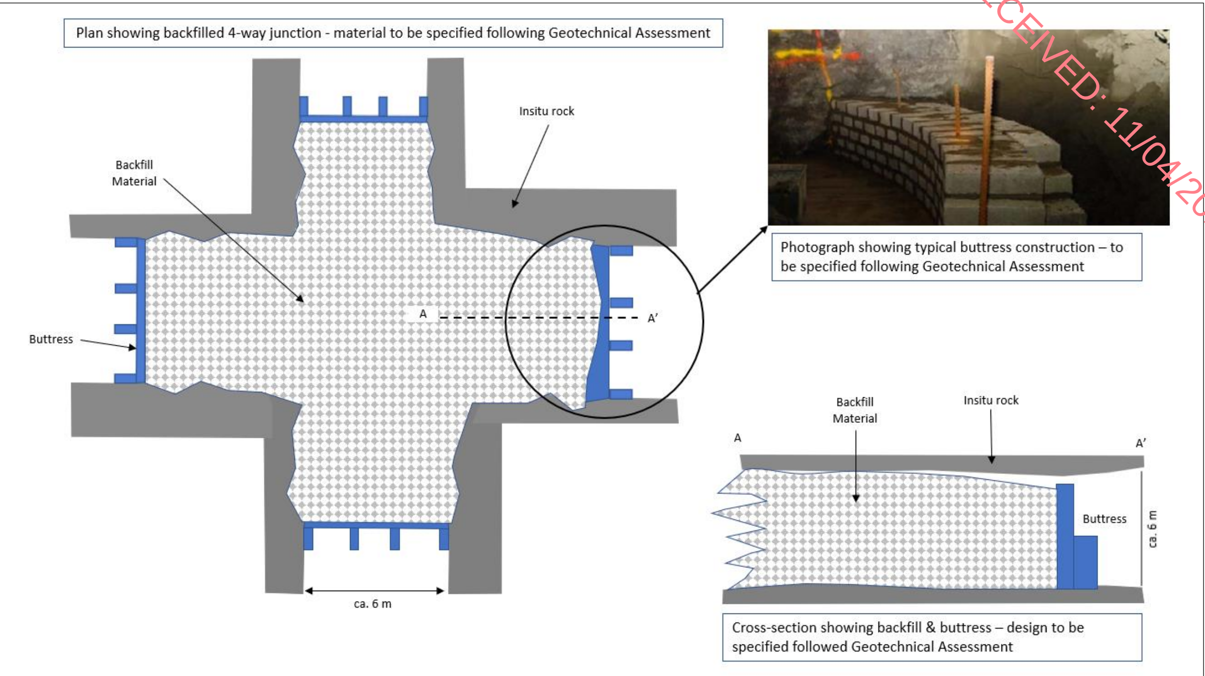

Figure 7.25: Schematic plan and cross-section for the backfilling of a 4-way junction

7.6.5.9 Knocknacran West - Slope Stability

The geotechnical parameters of the stratigraphy have been well established by previous works. During 2018 and 2019 Golder conducted additional borehole, sampling and laboratory testing to confirm the stratigraphy and material parameters associated with the proposed Knocknacran West Mine.

The *Knocknacran West Pit Slope Stability Assessment* (April 2023 – Appendix 7.12) report considers the Knocknacran West Open-Cast Mine slope stability design and presents the geotechnical analyses for the proposed open-cast. It considers that the phreatic surface within the pit footprint is within the underground workings in the Lower Gypsum and has conducted sensitivity analyses for cross-sections which align with monitoring well installations and recorded water elevations. Nine representative cross-sections around the perimeter of the proposed Knocknacran West Open-Cast Mine have been selected for stability analyses and have been assessed to meet the required design criteria and Factor of Safety (FoS).

The entirety of the open-cast mine proposed for Knocknacran West has been designed in accordance with the design criteria set out in Table 2 of the Pit Slope Stability Assessment (Appendix 7.12). The FoS reported are the minimum values recorded for a variety of slope scenarios.

The piezometric level used is below the level of the Lower Gypsum as the operational underground Drummond Mine located to the south provides dewatering for the proposed Knocknacran West Open-Cast Mine.

As the Knocknacran West Open-Cast Mine is developed in specific Phases, updated detailed design of the long-term perimeter slopes and the short-term internal slopes will be undertaken with additional geotechnical information acquired as the mine is developed. The detailed designs will be optimized to maintain the required FoS and thus may have shallower or steeper overall slope gradients depending on the nature of overburden materials present in a particular Phase footprint.

Base based on a review of historical data and recent slope stability analyses (using limit equilibrium modelling software SLOPE-W version 10.0.2.1001) carried out by Golder, the FoS varies from between 1.5 to 2.3 for the overall pit slope, and from 1.2 to 2.5 for the inter bench which meets the minimum recommended design criteria FoS values. The proposed Knocknacran West Open-cast Mine will be developed on a phased basis, which will require detailed design of the long-term perimeter slopes and the short-term internal slopes. These detailed designs will be optimized to extract the Lower Seam Gypsum and maintain the required FoS and thus may have shallower or steeper overall slope gradients depending on the nature of overburden materials present in that phase footprint (i.e., as information on specific rock mass quality becomes available).

Once mining operations commence at the Knocknacran West Mine, more detailed geotechnical data will be gathered from the open-cast pit walls/benches to reaffirm the overall slope stability of the excavation as it progresses throughout its lifetime, thus allowing for modifications to be made to the mining design as variations in ground condition are encountered.

Consideration has been given to the presence of underground mine workings which will be close to the maximum extent slopes within the slope stability report. From historical mine records the following dimensions are understood to be applicable for the underground mine workings at the former Drumgoosat Mine:

- Roof beam thickness = normally 3 m thick;
- Mine workings height = normally 6 m high;
- Mine workings width = normally 9 m wide; and
- Floor beam thickness = normally minimum of 1 m thick.

The mine workings have been inserted into the slope stability models for representative cross-sections; Cross-Section D-D' and Cross-Section G-G' (Appendix 7.12). The model for Section D-D' returned a minimum Factor of Safety (FoS) of 1.87. The model for Section G-G' returned a minimum FoS of 1.75, which is greater than that recorded in Figure 13 of the Pit Slope Stability Assessment. Both models returned minimum FoS values greater than the design criteria FoS of 1.5 (FoS of 1.87 for Section D-D' and FoS of 1.75 for Section G-G').

The report (Appendix 7.12) also includes an assessment of planar failures surfaces where there is a dip evident in the gypsum beds. The Cross-Sections for Section A-A', Section B-B' and Section F-F' have been run with the failure surface forced to occur along the top surface of the Upper Gypsum Seam. All models returned minimum FoS values greater than the design criteria FoS of 1.5, and greater than the global stability FoS values.

Consideration has also been given to the dumper haulage route extending into the base of the proposed open-cast mine from the north side of the Tunnel (south side slopes of the open-cast) and along the west side-slopes of the open-cast (the first cut and the potential stability associated with this). This assessment is detailed in Appendix 7.12. In summary it is considered that an edge bund will be constructed along the outer edge of the haul road in accordance with the 'Safe Quarry Guidelines 2020'. A two-way traffic haul route will require a minimum width of 12.8 m, and an overall design width of 17 m, which allows for a width of 4.2 m for an edge berm, thereby ensuring that the worst-case tyre loading will be centred at 4.5 m from the crest.

The potential impact magnitude of slope instability in Knocknacran West Mine is considered to be Negligible (Adverse) and impacts would be confined to the pit area as the design has incorporated a step-back from the nearby roads. Slope stability is not considered in this assessment for Knocknacran Mine as the proposal is to restore this area to near original ground and slopes will therefore be backfilled entirely. The sensitivity of the receptor is considered to be High and the significance of the effect is considered to be Slight.

7.6.5.10 Summary of Mine Stability Studies

It is considered that the potential impact magnitude of the change in stability of the underground workings caused by the proposed mining of Knocknacran West is Negligible (Adverse). For the Cut-and-Cover tunnel it is considered that the potential impact magnitude of the change in stability of the underground workings caused by the placement of the Cut-and-Cover tunnel is Negligible (Beneficial). The sensitivity of the receptor (workings) is considered to be Negligible and the significance of the effect is considered to be Imperceptible.

In turn, this considers that the change in stability of the underground workings and the resulting potential impact magnitude on built structures such as the R179/L4900 or people in the area during the operational life of the mine (during removal) is Negligible (Adverse). The sensitivity of the receptors are considered to be High and the significance of the effect is considered to be Slight (Adverse).

It is separately considered that the impact magnitude of removing the majority of the underground workings which have historically been associated with surface subsidence will have a Medium (Beneficial) impact magnitude on the Site and will create a permanent positive impact. The sensitivity of the receptors (built structure and human health) is considered to be High and the significance of the effect is considered to be Large (Beneficial).

7.6.5 Potential Effects: Closure/Restoration Phase: Community Sports Complex

No closure phase is proposed for the Community Sports Complex, therefore the potential impact and effect from this phase is not considered further. It is scoped out for consideration in this phase.

7.6.6 Potential Effects: Closure/Restoration Phase: Mine Development

For the mining development, consideration has been given again to the SRK stability assessment of the Knocknacran West Mine (Appendices 7.13 and 7.15). This assessment also considered the impact of backfilling the existing open-cast at Knocknacran Mine and the partial backfilling of the proposed open-cast at Knocknacran West Mine. Once backfilling is completed, groundwater and surface water could infiltrate the workings through the backfill. SRK considered in their report that the long-term stability of the underground workings could be impacted by one of two ways as follows:

1) Infiltration of water into cracks and joints within the gypsum that may result in blocks dislodging from the roof and pillar side walls. This will likely only occur to blocks that are already partly disconnected from the surrounding rock mass and are in a state of incipient collapse providing no material support to the mine elements in which they are located. The introduction of water completes a process that was in progress.

2) Dissolution of gypsum by flowing water through the mine workings.

Further consideration was given by SRK to the potential for dissolution of the workings remaining under the R179 after closure and it was estimated that saturated water would take between 6,000 and 7,000 years for a standard pillar of 12 m by 12 m to reduce to a dimension where it potentially becomes unstable. If water were able to enter the workings and it was undersaturated surface water with respect to gypsum, rather than saturated groundwater, that the rate of gypsum dissolution would be much more rapid. The SRK report advised that the backfill design in the mine sites must ensure that surface water cannot directly enter the mine workings through the use of low permeability backfill.

As detailed in Chapter 8.0 (Water) of this EIAR, the closure of the Knocknacran West Mine will see backfill (stripped low permeability mudstone from the site) placed against the southern and eastern walls of the open-cast adjacent to the R179 and L4900 (and along the northern and western walls where gypsum is exposed). This results in the insitu gypsum in the Upper and Lower Units surrounding the open cast (including beneath the roads) becoming hydraulically isolated from any active flow pathways. This will greatly reduce the potential for any possible gypsum dissolution which, in turn, will minimize the potential for any future settlement. The northern side of the existing Knocknacran open-cast (adjacent to the southern side of the R179) has already been backfilled in this manner to minimize the circulation of water beneath the R179.

Following cessation of mining and prior turning off the dewatering pumps, detailed analyses of the geotechnical integrity of the remaining pillars (and the underground workings as a whole) will be undertaken under the R179 and L4900.

Flooding of the underground workings by fresh surface water prior to cessation of mining and mine closure will be prevented as discussed above due to the potential for dissolution and softening of gypsum units. Flooding by saturated groundwater would have a lesser effect on the potential for dissolution of gypsum but would potentially have a similar effect on pillar integrity as the influence of fresh surface water over time.

An additional report by Piteau Associates (*Hydrogeology Study of Drumgoosat Underground Workings, May 2021*), produced on the hydrogeochemistry of the Drumgoosat underground workings concluded that the actual saturation state of the water entering the mine workings would not cause dissolution of the gypsum in the mine workings (Appendix 8.8).

It is considered that the potential impact magnitude of the change in stability of the remaining underground workings caused by the closure of the mine sites (including backfilling with low permeability mudstone along road sections) is Low (Adverse). The sensitivity of the receptors (human health and built structures) are considered to be High and the significance of the effect is considered to be Slight. It is separately considered that the impact magnitude of having removed the majority of the underground workings which have historically been associated with surface subsidence will have a Medium (Beneficial) impact magnitude on the Site and will create a permanent positive impact. The sensitivity of the receptors (built structure and human health) is considered to be High and the significance of the effect is considered to be Large (Beneficial).

Appendix 7.15 considers the long-term stability and subsidence risks posed to third partly lands beyond the open-cast void perimeter below which existing underground mine workings remain.

In relation to lands beyond the limit of the open-cast mine area, where Upper Seam Gypsum workings are identified as being shallow (within ca. 30 m of the surface) sealing the Upper Seam Gypsum workings exposed in the open-cast to inhibit water ingress into the workings will reduce the risk of the formation of crownholes where workings remain.

The remaining workings in the Upper Seam Gypsum, which adjoin the south of the site, will be sealed off to prevent ingress of water beneath these lands, with material (mudstone from site) of ca. 10^{-9} m^2 permeability. The Lower Seam Gypsum workings in this area will be sealed during restoration by the emplacement of backfill material.

The north of the Knocknacran West site will be fully restored to near original ground levels and direct water ingress to the underground workings will therefore be cut off from the waterbody.

The potential impact magnitude to lands adjoining the site (which contain underground workings) is considered to be Negligible (Adverse). The sensitivity of the receptor (land) is considered to be Low and the sensitivity is considered to be Imperceptible. .

During the operational phase of the development, it has been noted that there is a proposal to engage a reputable third-party engineering company (specialising in ground support) to assess the opportunity and practicalities of accessing the workings to carryout support work to ensure continued ground stability under the roadways where the mine workings occur. This has not been included in the impact assessment for the closure phase as the findings of such work are not yet known and their magnitude of change and significance cannot be concluded at this stage, however, it is noted that such investigations could lead to an opportunity to provide additional support.

7.7 Mitigation and Management

7.7.1 Mitigation and Management: Construction Phase: Community Sports Complex

7.7.1.1 Embedded Design Mitigation: Construction Phase: Community Sports Complex

- Construction phase activities at the Community Sports Complex will take place in accordance with the Construction Environmental Management Plan.

7.7.1.2 Additional Mitigation: Construction Phase: Community Sports Complex

- Works will be undertaken in line with any conditions set by MCC;
- All works will be undertaken in accordance with best practice and adhere to the following guidelines:
    - Inland Fisheries Ireland (2016). Guidelines on Protection of Fisheries During Construction Works in and Adjacent to Waters;
    - CIRIA (2009). Control of Water Pollution from Construction Sites – Guidance for Consultants and Contactors (C532);
    - NRA Guidelines (2006). NRA Guidelines for the Crossing of Watercourses during the Construction of National Road Scheme; and
    - Defra (Department for Environment, Food and Rural Affairs) (2009). Construction Code of Practice for the Sustainable Use of Soils on Construction Sites.
- Any plant will be regularly maintained and kept in good order on the proposed Community Sports Complex site; and
- Refuelling of mobile plant will take place from bunded fuel tanks.

7.7.2 Mitigation and Management: Construction Phase: Mine Development

7.7.2.1 Embedded Design Mitigation: Construction Phase: Mine Development

- Construction phase activities will take place in accordance with the Construction Environmental Management Plan;

- Fencing will be maintained at the Site to ensure that the risk of injury to the public and livestock is minimised;
- Re-handling of the topsoil will be kept to a minimum to preserve the integrity of the material;
- All plant on the Site be regularly maintained, and where plant is damaged or leaking, it will be fixed or replaced immediately, as part of ongoing management of the site;
- Refuelling of mobile plant will take place from bunded fuel tanks as required;
- Refuelling and the addition of hydraulic oils or lubricants to vehicles or generators will take place on-site only in designated areas; and
- Stockpiles will be evaluated and monitored and kept stable for safety and to minimise erosion.

7.7.2.2 Additional Mitigation: Construction Phase: Mine Development

Embedded design mitigation has already been outlined in Section 7.7.2.1 above, the following additional mitigation will be implemented onsite:

- Works will be undertaken in line with any conditions set by MCC;
- Earthworks will follow the embedded mitigation measures outlined above. All works will be undertaken in accordance with best practice and adhere to the following guidelines:
    - Inland Fisheries Ireland (2016). Guidelines on Protection of Fisheries During Construction Works in and Adjacent to Waters;
    - CIRIA (2009). Control of Water Pollution from Construction Sites – Guidance for Consultants and Contactors (C532);
    - NRA Guidelines (2006). NRA Guidelines for the Crossing of Watercourses during the Construction of National Road Scheme; and
    - Defra (Department for Environment, Food and Rural Affairs) (2009). Construction Code of Practice for the Sustainable Use of Soils on Construction Sites.
- On-going geotechnical monitoring by means of extensometers will continue along the R179 and L4900; and
- Any processing plant and / or mobile plant on the mine sites will be regularly maintained and kept in good working order.

7.7.3 Mitigation and Management: Operational Phase: Community Sports Complex

- Works will be undertaken in line with any conditions set by MCC;
- Any plant will be regularly maintained and kept in good order on the proposed Community Sports Complex site; and
- Refuelling of mobile plant will take place from bunded fuel thanks.

7.7.4 Mitigation and Management: Operational Phase: Mine Development

7.7.4.1 Embedded Mitigation: Operational Phase: Mine Development

- Site operations will be managed in accordance with relevant health and Safety legislation (Safety, Health & Welfare at Work Act (2005, as amended); and the Mines and Quarries Act (1965, as amended));
- Fencing will be maintained at the Site to ensure that the risk of injury to the public and livestock is minimised. The entrance gate is locked and controlled by the Site's management;
- The extraction of gypsum will take place using the mining industry standard method of cyclical drilling, blasting, loading, hauling and supporting;
- The removal of soils will be conducted in a phased basis to reduce the overall potential impact on the land use and underlying groundwater;
- Re-handling of the topsoil will be kept to a minimum to preserve the integrity of the material;
- All plant on the Site be regularly maintained, and where plant is damaged or leaking, it will be fixed or replaced immediately, as part of ongoing operational management of the site;
- Refuelling and the addition of hydraulic oils or lubricants to vehicles or generators will take place on-site only in designated areas.
- Existing groundwater wells will be continuously monitored on site during mining operations and for a period following cessation of mining (to be agreed with the relevant authorities);
- Blasting will take place at the Site using licenced and experienced operators. Site management give advance notification of blast events to nearby residents as is standard procedure for the existing mine;
- Geotechnical assessments will be conducted on a regular basis by an experienced and suitably qualified geotechnical engineer;
- The mine manager will ensure compliance with relevant safety and statutory legislation and best practices recommended by national legislation (and guidelines);
- Stockpiles will be evaluated and monitored and kept stable for safety and to minimise erosion;
- The designed intercept drainage system(s) and settling pond/filter system, for each stripping campaign, will be installed prior to stripping of material. The design will be updated throughout the stripping campaigns as the works progress. The design will be agreed with the relevant authorities prior to stripping;
- The contractor will organize the earthworks operations, whether in excavation or in restoration, so that all water shed onto the earthworks, or which enters the earthworks from any source is rapidly led away into a specifically designed intercept drainage system(s) and settling pond / filter system prior to discharge into the underlying mine workings, where it will enter the existing mine water management and treatment system;

- As the earthworks progress, the contractor will construct, maintain and revise, as necessary, all temporary ditches, sumps, pump lines and pumping units required for the effective disposal of all such water flows;

- The contractor will not commence main earthworks operations or continue with a section of main earthworks operations until a plan and programme of ditches, sumps, pump lines and pumping units has been agreed with SGMI's project manager;

- Depending on the area(s) to be stripped and restored, the contractor will construct a temporary de-silting settling pond / system at approximate location(s) to be agreed with SGMI's project manager prior to any stripping taking place;

- The contractor will construct surface water cut-off drains, ditches, swales and sumps, as required to ensure that the works are maintained free from standing water and to divert surface water and groundwater gathered to the drainage system via gravity and/or pumping. The cut-off drains will be a minimum of 600 mm deep and 400 mm wide at the base, and will have side slopes of no steeper than 1(V):2(H);

- The contractor's working surfaces in excavation and in filling will be sufficiently regular and will have such cross falls or longitudinal falls or both as are necessary to prevent standing water and to rapidly dispose of water run-off. The contractor's earthworks slopes, whether in cutting or in filling, will be trimmed to regular profile and compacted so as to prevent ponding water and to rapidly dispose of water run-off without scour;

- The contractor's temporary ditches and sumps will be located such that when backfilled they shall not have any adverse effects on the strengths or stability of the completed works. When temporary ditches and sumps are no longer required in a particular area of the site by reason of progress of the work, the contractor will promptly remove all temporary pump pipelines and pumping units. All softened deposits will be removed from the ditches and these areas backfilled with suitable material. Filling, compaction and field quality control will be as specified for the adjacent earthworks operations;

- The contractor's temporary sumps, pumping units and fuel or power supply will have adequate capacities for the pumping loads and will be maintained regularly to ensure efficient and reliable operation. The contractor will provide adequate supervision to ensure continuous operation whenever this is required to ensure rapid disposal of water run-off and will have adequate standby arrangements available to cope with pump or power failures;

- To avoid siltation of watercourses from crossing point locations, silt traps will be placed beside temporary crossing points with an associated buffer strip. The silt-traps will be maintained and cleaned regularly during the course of the works; and

- A maintenance schedule and operational procedure will be established by the contractor for silt and pollution control measures during the construction period. This will be undertaken in consultation with the relevant statutory authorities.

7.7.4.2 Additional Mitigation: Operational Phase: Mine Development

Embedded mitigation has already been outlined in Section 7.7.4.1 above, the following additional mitigation will be implemented onsite:

- Works will be undertaken in line with any conditions set by the IE licence;

- Geotechnical assessments will be conducted on a regular basis by an experienced and suitably qualified geotechnical engineer on the mine sites. The current slope angles are designed to ensure that the risk of slope failure is effectively eliminated by using a suitable safety factor;

- During mining of Knocknacran West, where underground workings are exposed (which would remain in situ) the opportunity and practicalities of accessing the workings to carryout support work to ensure continued ground stability under the roadways where the mine workings occur will be assessed;

- Earthworks will follow the embedded mitigation measures outlined above. All works will be undertaken in accordance with best practice and adhere to the following guidelines:

    - Inland Fisheries Ireland (2016). Guidelines on Protection of Fisheries During Construction Works in and Adjacent to Waters;
    - CIRIA (2009). Control of Water Pollution from Construction Sites – Guidance for Consultants and Contactors (C532);
    - NRA Guidelines (2006). NRA Guidelines for the Crossing of Watercourses during the Construction of National Road Scheme; and
    - Defra (Department for Environment, Food and Rural Affairs) (2009). Construction Code of Practice for the Sustainable Use of Soils on Construction Sites.

- On-going Geotechnical monitoring by means of extensometers will continue throughout the life of the mine along the R179 and L4900;

- The provision of adequate drainage along the upper benches of the proposed Knocknacran West Mine in the overburden will be employed as is the current arrangement in the existing Knocknacran Mine; and

- The qualified mine manager will ensure compliance with relevant safety and statutory legislation and best practices as set out in the HSA's 'Guidelines to the Safety, Health and Welfare at Work (Quarries) Regulations 2008', and other relevant statutory and industry guidelines from Government Departments and the EPA for the mine sites.

7.7.5 Mitigation and Management: Restoration/Closure Phase: Community Sports Complex

There is no proposed decommissioning of the Community Sports Complex and so this is not considered further.

7.7.6 Mitigation and Management: Restoration/Closure Phase: Mine Development

7.7.6.1 Embedded Design Mitigation: Restoration/Closure Phase: Mine Development

- Stockpiles will be evaluated and monitored and kept stable for safety and to minimise erosion;

- The designed intercept drainage system(s) and settling pond/filter system, for each stripping campaign, will be installed prior to stripping of material. The design will be updated throughout the stripping campaigns as the works progress. The design will be agreed with the relevant authorities prior to stripping;

- The contractor will organize the earthworks operations, whether in excavation or in restoration, so that all water shed onto the earthworks, or which enters the earthworks from any source is rapidly led away into a specifically designed intercept drainage system(s) and settling pond / filter system prior to discharge into the underlying mine workings, where it will enter the existing mine water management and treatment system;

- As the earthworks progress, the contractor will construct, maintain and revise, as necessary, all temporary ditches, sumps, pump lines and pumping units required for the effective disposal of all such water flows;

- The contractor will not commence main earthworks operations or continue with a section of main earthworks operations until a plan and programme of ditches, sumps, pump lines and pumping units has been agreed with SGMI's project manager;

- Depending on the area(s) to be stripped and restored, the contractor will construct a temporary de-silting settling pond / system at approximate location(s) to be agreed with SGMI's project manager prior to any stripping taking place;

- The contractor will construct surface water cut-off drains, ditches, swales and sumps, as required to ensure that the works are maintained free from standing water and to divert surface water and groundwater gathered to the drainage system via gravity and/or pumping. The cut-off drains will be a minimum of 600 mm deep and 400 mm wide at the base, and will have side slopes of no steeper than 1(V):2(H);

- The contractor's working surfaces in excavation and in filling will be sufficiently regular and will have such cross falls or longitudinal falls or both as are necessary to prevent standing water and to rapidly dispose of water run-off. The contractor's earthworks slopes, whether in cutting or in filling, will be trimmed to regular profile and compacted so as to prevent ponding water and to rapidly dispose of water run-off without scour;

- The contractor's temporary ditches and sumps will be located such that when backfilled they shall not have any adverse effects on the strengths or stability of the completed works. When temporary ditches and sumps are no longer required in a particular area of the site by reason of progress of the work, the contractor will promptly remove all temporary pump pipelines and pumping units. All softened deposits will be removed from the ditches and these areas backfilled with suitable material. Filling, compaction and field quality control will be as specified for the adjacent earthworks operations;

- The contractor's temporary sumps, pumping units and fuel or power supply will have adequate capacities for the pumping loads and will be maintained regularly to ensure efficient and reliable operation. The contractor will provide adequate supervision to ensure continuous operation whenever this is required to ensure rapid disposal of water run-off and will have adequate standby arrangements available to cope with pump or power failures;

- To avoid siltation of watercourses from crossing point locations, silt traps will be placed beside temporary crossing points with an associated buffer strip. The silt-traps will be maintained and cleaned regularly during the course of the works; and

- A maintenance schedule and operational procedure will be established by the contractor for silt and pollution control measures during the construction period. This will be undertaken in consultation with the relevant statutory authorities.

7.7.6.2 Additional Mitigation: Restoration/Closure Phase: Mine Development

Embedded mitigation has already been outlined in Section 7.7.6.1 above, the following additional mitigation will be implemented onsite:

- Works will be undertaken in line with any conditions set by the IE licence and CRAMP (a provisional CRAMP is provided in Appendix 3.3);

- Any processing plant and / or mobile plant on the mine sites will be regularly maintained and kept in good working order;

- Earthworks will follow the embedded mitigation and design measures outlined above. All works will be undertaken in accordance with best practice and adhere to the following guidelines:
    - Inland Fisheries Ireland (2016). Guidelines on Protection of Fisheries During Construction Works in and Adjacent to Waters.
    - CIRIA (2009). Control of Water Pollution from Construction Sites – Guidance for Consultants and Contactors (C532).
    - NRA Guidelines (2006). NRA Guidelines for the Crossing of Watercourses during the Construction of National Road Scheme.
    - Defra (Department for Environment, Food and Rural Affairs) (2009). Construction Code of Practice for the Sustainable Use of Soils on Construction Sites; and
    - On-going Geotechnical monitoring by means of extensometers will continue throughout the life of the mine along the R179 and L4900 and during closure.

- On-going Geotechnical monitoring by means of extensometers will continue along the R179 and L4900 during closure, unless otherwise agreed with the Regulatory Authority.

7.8 Monitoring

7.8.1 Monitoring: Construction Phase: Community Sports Complex

- The appointed Main Contractor will be required to produce a final Construction Management Plan (CMP), which will document appropriate procedures and responsible persons when working on the site; and

- Any monitoring associated with authorisation or consents (e.g., construction discharges) will be incorporated into the Main Contractor's CMP and will be adhered to.

7.8.2 Monitoring: Construction Phase: Mine Development

- Monitoring will be undertaken in line with any conditions set by MCC;

- The appointed Main Contractor will be required to produce a final CMP, which will document appropriate procedures and responsible persons when working on the site;

- Any monitoring associated with authorisation or consents (e.g., construction discharges) will be incorporated into the Main Contractor's CMP and will be adhered to;

- On-going Geotechnical monitoring by means of extensometers will continue along the R179 and L4900; and

- The Applicant will continue to maintain a Complaints Register. This register will record complaints in relation to the construction of phase the mine and associated infrastructure. In each entry the Applicant will record the date and time of the complaint, the name of the complainant (if provided), and will give details of the nature of the complaint. A record shall also be kept of any response made in the case of each complaint.

7.8.3 Monitoring: Operational Phase: Community Sports Complex

There is no proposed environmental monitoring of the Community Sports Complex and so this is not considered further.

7.8.4 Monitoring: Operational Phase: Mine Development

- Monitoring will be undertaken in line with any conditions set by the IE Licence;

- On-going Geotechnical monitoring by means of extensometers will continue along the R179 and L4900 and subsidence monitoring and regular stability surveys of the open-cast slopes (and benches) will be undertaken; and

- The Applicant will continue to maintain a Complaints Register. This register will record complaints in relation to the operation of the mine and associated infrastructure. In each entry the Applicant will record the date and time of the complaint, the name of the complainant (if provided), and will give details of the nature of the complaint. A record shall also be kept of any response made in the case of each complaint.

7.8.5 Monitoring: Restoration/Closure Phase: Community Sports Complex

There is no proposed decommissioning of the Community Sports Complex and so this is not considered further here.

7.8.6 Monitoring: Restoration/Closure Phase: Mine Development

- Monitoring will be undertaken in line with any conditions set by the IE Licence and CRAMP (a provisional CRAMP is provided in Appendix 3.3). The physical closure works will be followed by a period of monitoring, during which time the mining company must carry out monitoring and measurements to demonstrate that the closure works have been successful, and that all environmental metrics for the Site are stable. This will be controlled by the EPA through the IE Licencing procedure. Following this, it is envisaged that the former mining areas will transition to an aftercare period, which will be of reduced scope and intensity to the monitoring carried out during the closure works;

- Appendix 3.3 sets out details of the closure and aftercare vision for the Application Site, which will be developed in line with Saint-Gobain's Stakeholder Management Plan and the CRAMP will evolve through the life of the mine, taking community and statutory interests into account; and

- On-going Geotechnical monitoring by means of extensometers will continue along the R179 and L4900 during closure, unless otherwise agreed with the Regulatory Authority.

7.9 Residual Effects

7.9.1 Community Sports Complex

Once the identified mitigation measures, appropriate design standards and operational infrastructure management plans are adhered to, it is considered that any effects surrounding the Community Sports Complex will be **Not Significant.**

7.9.2 Mine Development

The materials to be extracted will be used as raw materials in the construction industry, which is considered an acceptable use of the resource. The extraction of gypsum on the Site is an important industrial mineral resource both locally, regionally and nationally. Removal of the underground workings beneath the site will improve the stability of the Site over the long-term and will also allow for an inspection of remaining exposed pillars outside the Site boundary.

Following the cessation of mining, the dewatering pumps will be turned off and the water-table will return to its pre-mining levels. Any voids or porous sediments will again become saturated. There is no expected residual impact due to dewatering operations once the mine has closed and the pre-mining situation reinstated.

Mitigation measures, as previously mentioned above, will be utilised to minimise the risk from mining related slope failure or subsidence through careful management and planning. Continuous monitoring will be undertaken of ground stability throughout the life of the proposed mine.

A consideration of the effects of the Mine Development on the geological environment during the construction, operational and closure phase has not identified a significant effect and it is considered here that any residual effects would also be **Not Significant.**

In the long-term, there will be no deleterious effects on the remaining bedrock and groundwater in the open-cast mine following restoration as set out in the CRAMP (Appendix 3.3).

7.10 Cumulative Effects

7.10.1 The Project – Community Sports Complex and Mine Development

The construction phases of the Community Sports Complex and the Mine Development occur simultaneously, however, no significant effects are identified for either and it is considered that there is no potential for cumulative effects on land, soils and geology between the two developments.

The construction phase of the Community Sports Complex overlaps with the first year of the operational life of the Mine Development, however, no significant effects are identified for either and it is considered that there is no potential for cumulative effects on land, soils and geology between the two developments.

The operational phase of the Community Sports Complex and mine development overlap, however, no significant effects are identified for either and it is considered that there is no potential for cumulative effects on land, soils and geology between the two developments.

The restoration phase of the mine development overlaps with the operational phase of the Community Sports Complex, however, no significant effects are identified for either and it is considered that there is no potential for cumulative effects on land, soils and geology between the two developments.

7.10.2 The Project and Other Offsite Projects

The proposed mining development will take place below the water-table. Mobile plant will use the existing refuelling facilities at the Plant Site garage for refuelling (static plant or tracked excavators will refuel over a drip tray with an absorbent mat).

Drummond Mine and the Project occur within the same gypsum deposit. Although the mine areas are hydraulically connected, both the Drummond Mine and the proposed mine development area (the former Drumgoosat Mine) have been dewatered for many decades. This limits potential groundwater movement between the areas.

The geology does not facilitate the movement of groundwater into the proposed development area and Drummond Mine due to its low permeability, except through faults and fractures. The proposed development area has been extensively mined in the past, intersecting any faults and fractures within the deposit. This means that the potential for additional water ingress from fractures and faults is negligible (refer to Conceptual Model in Section 8.4.9, Water Chapter 8.0 of the EIAR).

No water is currently, or is proposed to be, pumped between the areas.

Hydrogeologically the Drummond Mine and the proposed development are currently of low relevance to each other.

It is proposed that dewatering of Drummond will continue for as long as mining continues in the existing Knocknacran Open-Cast Mine. Once mining ceases in Knocknacran open-cast, backfill (material stripped from Knocknacran West) will be placed in the existing Knocknacran open-cast void space as the planned mining of Knocknacran West commences, leading to a reduction of the hydraulic connection between Knocknacran open-cast and Drummond Mine. The hydrogeology of the Drummond Mine will therefore become independent of the hydrogeology of the existing Knocknacran Open-Cast Mine and the proposed Knocknacran West Open-Cast Mine.

They will remain of low hydrogeological relevance to each other and will not have a cumulative effect.

Other extractive industries near to the Project include four operational quarries within a radius of 5 km of the Project. These are; (i) Cormey Clay Pit, Breedon Brick Ltd.'s open-cast clay quarry, located ca. 1.5 km south of the Site. (ii) an associated site located ca. 4 km south of the Site, (iii) Limestone Industries Ltd limestone quarry, located ca. 2 km west of the Site, and (iv) Roadstone Barley Hill open-cast quarry located ca. 4 km southeast of the Site. As these facilities are not within the immediate vicinity of the Site (ca. 1 km), there will be no cumulative effect on the soils and geology environment that could be attributed to the interaction of several extractive industries in close proximity to each other.

Losset ADN Materials Ltd. have a planning application under consideration (Reg. Ref. 22/254) and are located ca. 1 km to the north of the Project site. Based on a review of the current planning file data (to date 27th March 2023), this development does not appear to require any excavation into bedrock or into soils beyond foundational level. There will be no cumulative effect on the soils and geology environment due to this development.

The cumulative effects are deemed **Not Significant** between the Project and other offsite Projects.

7.11 'Do-Nothing' Scenario

In a 'Do Nothing' Scenario, the Knocknacran West Mine would not be mined out and by association, the former Drumgoosat underground workings would not be removed at all. The Knocknacran Mine site would close once the resource is extracted and in line with the existing closure plan (Reg. Ref. 17/217) resulting in a mixed use of agricultural lands and a waterbody onsite. The proposed Community Sports Complex would not be further expanded and would remain as currently constructed.

7.12 Difficulties Encountered

No particular difficulties were encountered in undertaking the assessment of land, soils and geology.

7.13 References

- Charles, P. and Edwards, P. (eds) (2015) Environmental good practice on site guide. Fourth edition. London: CIRIA (CIRIA, 741).
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APPENDIX 7.1

Community Sports Complex Site Borehole Logs – 2021

Borehole Log

GroundCheck — Site Investigation Specialists

Location ID: BH01	Page1/2			
Date Start: 27/04/2021	Location Type: Rotary open hole	Project ID: 21-2676	Project Name: Proposed Playing Pitch	Easting: 680748 Northing: 799485
Date Finish: 28/04/2021	Logged By: S. Thompson	Site Location: Drummond, Magheracloone, County Monaghan		Elevation:

Samples & In-situ Testing Type	Depth (m)	Results & Information	Wells	Water	Legend	Stratum Description	Scale	Depth (m)	Reduced Level (m)
						Firm, reddish brown and brown, slightly gravelly, sandy, silty CLAY with occasional cobbles and occasional thin bands of silty, fine to medium sand and fine sub-rounded gravel. Gravel is fine to medium sub-angular to sub-rounded. Sand is fine to medium. [MADE GROUND]			
D	0.50								
SPT	1.00	N=9 (2,2/2,2,2,3)					1.0		
D	1.00								
		Water Strike 1.80m		▼					
SPT	2.00	N=13 (2,2/3,3,3,4)					2.0		
D	2.00								
SPT	3.00	N=13 (2,2/3,3,3,4)					3.0		
D	3.00								
SPT	4.00	N=12 (2,2/2,2,3,5)					4.0		
D	4.00								
D	5.00						5.0		
SPT	5.50	N=12 (2,1/2,2,3,5)							
D	6.00						6.0		
SPT	7.00	N=14 (2,3/3,3,3,5)					7.0		
D	7.00								
D	8.00						8.0		
SPT	8.50	N=15 (2,2/3,3,4,5)							
D	9.00						9.0		
SPT	10.00	N=15 (2,3/3,3,4,5)					10.0		
D	10.00								

Continued on Next Page

Water Monitoring Depth	Date	Water Strikes Struck	Date	Flow	Shift Information Depth	Water	Remarks	Date Time	Depth Related Remarks Top	Base	Remarks	Backfill Top	Base	Type
		1.80m	28/04	Seepage	20.00			28/04/2021 16:30	0.00	1.20	Obstruction time - Hand dug inspection pit.			

Termination Reason: Borehole terminated at scheduled completion depth.	General Remarks:	AGS	Scale: 1:60

Borehole Log

GroundCheck SITE INVESTIGATION SPECIALISTS

Date Start: 27/04/2021	Location Type: Rotary open hole	Project ID: 21-2676	Project Name: Proposed Playing Pitch	Location ID: BH01 Page2/2
Date Finish: 28/04/2021	Logged By: S. Thompson	Site Location: Drummond, Magheracloone, County Monaghan		Easting: 680748 Northing: 799485 Elevation:

Samples & In-situ Testing Type	Depth (m)	Results & Information	Wells	Water	Legend	Stratum Description	Scale	Depth (m)	Reduced Level (m)
						Firm, reddish brown and brown, slightly gravelly, sandy, silty CLAY with occasional cobbles and occasional thin bands of silty, fine to medium sand and fine sub-rounded gravel. Gravel is fine to medium sub-angular to sub-rounded. Sand is fine to medium. [MADE GROUND]	11.0		
SPT	11.50	N=28 (3,2/3,10,8,7)							
							12.0		
SPT	13.00	N=12 (2,3/3,3,3,3)					13.0		
							14.0		
SPT	14.50	N=16 (2,3/3,3,4,6)							
							15.0		
SPT	16.00	N=30 (2,3/4,8,12,6)					16.0		
							17.0		
SPT	17.50	N=17 (2,3/3,4,4,6)							
							18.0		
SPT	19.00	N=21 (2,3/3,4,6,8)					19.0		
						End of Borehole at 20.00m	20.0	20.00	
							21.0		

Water Monitoring Depth	Date	Water Strikes Struck	Date	Flow	Shift Information Depth	Water	Remarks	Date Time	Depth Related Remarks Top	Base	Remarks	Backfill Top	Base	Type
					20.00			28/04/2021 16:30						

Termination Reason: Borehole terminated at scheduled completion depth.	General Remarks:	AGS	Scale: 1:60

Borehole Log

GroundCheck SITE INVESTIGATION SPECIALISTS

Location ID: **BH02**

Date Start:	Location Type:	Project ID:	Project Name:	Easting:	Northing:
27/04/2021	Rotary open hole	21-2676	Proposed Playing Pitch	680725	799659

Date Finish:	Logged By:	Site Location:	Elevation:
27/04/2021	S. Thompson	Drummond, Magheracloone, County Monaghan	

Type	Depth (m)	Results & Information
D	0.50	
SPT	1.00	N=8 (1,2/2,2,2,2)
D	1.00	
SPT	2.00	N=14 (1,2/3,3,3,5)
D	2.00	
SPT	3.00	N=18 (3,4/4,4,5,5)
D	3.00	
SPT	4.00	N=20 (3,4/5,4,5,6)
D	4.00	
D	5.00	
SPT	5.50	N=21 (4,5/4,5,6,6)
D	6.00	
SPT	7.00	N=25 (4,5/5,6,9,5)
D	7.00	
D	8.00	
SPT	8.50	N=12 (1,2/1,2,4,5)
D	9.00	
SPT	10.00	N=15 (2,3/3,3,4,5)
D	10.00	

Stratum Description	Depth (m)	Reduced Level (m)
Soft to firm, becoming firm, reddish brown and brown, slightly gravelly, slightly sandy, silty CLAY with occasional thin bands of silty, fine to coarse sand. Gravel is fine to medium sub-rounded. Sand is fine to medium. [MADE GROUND]	4.10	
Medium dense, dark greyish brown, very silty, fine to coarse SAND and fine to medium sub-angular GRAVEL. [MADE GROUND]	4.90	
Firm , locally stiff, reddish brown and brown, slightly gravelly, sandy, silty CLAY with occasional cobbles. Gravel is fine to medium sub-rounded, Sand is fine to medium. [MADE GROUND]		

Scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0

Continued on Next Page

Water Monitoring

Depth	Date

Water Strikes

Struck	Date	Flow

Shift Information

Depth	Water	Remarks	Date Time
20.50			27/04/2021 16:30

Depth Related Remarks

Top	Base	Remarks
0.00	1.20	Obstruction time - Hand dug inspection pit.

Backfill

Top	Base	Type

Termination Reason:
Borehole terminated at scheduled completion depth.

General Remarks:

AGS

Scale: 1:60

Borehole Log

GroundCheck SITE INVESTIGATION SPECIALISTS

Date Start: 27/04/2021	Location Type: Rotary open hole	Project ID: 21-2676	Project Name: Proposed Playing Pitch	Location ID: BH02 Page2/2
Date Finish: 27/04/2021	Logged By: S. Thompson	Site Location: Drummond, Magheracloone, County Monaghan		Easting: 680725 Northing: 799659 Elevation:

Type	Depth (m)	Results & Information	Wells	Water	Legend	Stratum Description	Scale	Depth (m)	Reduced Level (m)
D	11.00					Firm , locally stiff, reddish brown and brown, slightly gravelly, sandy, silty CLAY with occasional cobbles. Gravel is fine to medium sub-rounded, Sand is fine to medium. [MADE GROUND]	11.0		
SPT	11.50	N=43 (25 for 125mm/13,13,10,7)						11.40	
D	12.00					Stiff, greyish brown, gravelly, very sandy, clayey SILT with cobbles. Gravel is fine to coarse sub-angular to sub-rounded. Sand is fine to coarse. [MADE GROUND]	12.0		
SPT	13.00	N=24 (4,5/5,6,6,7)					13.0		
D	13.00							13.10	
D	14.00					Stiff, reddish brown, slightly gravelly, sandy, silty CLAY with occasional cobbles and occasional thin bands of silty, fine to coarse sand and fine to medium sub-angular gravel. Gravel is fine to medium sub-angular to sub-rounded. Sand is fine to medium. [MADE GROUND]	14.0		
SPT	14.50	N=34 (5,5/8,8,8,10)							
D	15.00						15.0		
SPT	16.00	N=23 (4,5/5,5,6,7)					16.0		
D	16.00								
D	17.00						17.0		
SPT	17.50	N=26 (3,4/6,6,7,7)							
D	18.00						18.0		
SPT	19.00	N=23 (4,4/4,5,6,8)					19.0		
D	19.00								
D	20.00						20.0		
SPT	20.50	N=30 (3,4/6,8,8,8)				End of Borehole at 20.50m		20.50	
							21.0		

Water Monitoring		Water Strikes			Shift Information				Depth Related Remarks			Backfill		
Depth	Date	Struck	Date	Flow	Depth	Water	Remarks	Date Time	Top	Base	Remarks	Top	Base	Type
					20.50			27/04/2021 16:30						

Termination Reason:
Borehole terminated at scheduled completion depth.

General Remarks:

AGS

Scale: 1:60

Borehole Log

GroundCheck SITE INVESTIGATION SPECIALISTS

Location ID: **BH03**

Date Start:	Location Type:	Project ID:	Project Name:	Easting:	Northing:
28/04/2021	Rotary open hole	21-2676	Proposed Playing Pitch	680546	799543

Date Finish:	Logged By:	Site Location:	Elevation:
28/04/2021	S. Thompson	Drummond, Magheracloone, County Monaghan	

Samples & In-situ Testing Type	Depth (m)	Results & Information	Wells	Water	Stratum Description	Depth (m)	Reduced Level (m)
					Grey, very silty, slightly gravelly, fine to coarse SAND. Gravel is fine to medium sub-angular. [MADE GROUND]	0.30	
D	0.50						
SPT D	1.00 1.00	N=6 (2,2/2,1,2,1)			Soft to firm, reddish brown and brown, slightly gravelly, very sandy, clayey SILT with occasional cobbles. Gravel is fine to medium sub-angular to sub-rounded, Sand is fine to medium, [MADE GROUND]	1.30	
					Grey MUDSTONE silt & sand & gravel [MADE GROUND?]	3.60	
SPT D	2.00 2.00	N=9 (2,2/2,3,2,2) Water Strike 2.00m		▼			
SPT D	3.00 3.00	N=10 (1,2/2,2,3,3)					
					Dark grey, slightly clayey, very silty, slightly gravelly, fine to coarse SAND. Gravel is fine to coarse sub-angular. [MADE GROUND]	4.10	
SPT D	4.00 4.00	N=24 (2,3/3,4,8,9)			Firm to stiff, reddish brown, gravelly, sandy, silty CLAY. Gravel is fine to coarse sub-rounded. Sand is fine to coarse. [MADE GROUND]	4.30	
					Extremely weak to very weak, bluish grey, fine grained argillaceous, fissured, MUDSTONE. [KINGSCOURT GYPSUM FORMATION]	9.80	
SPT D	5.00 5.00	50 (25 for 125mm/50 for 125mm)					
D	6.00						
SPT	6.50	50 (6,12/50 for 160mm)					
D	7.00						
SPT D	8.00 8.00	50 (8,17/50 for 150mm)					
D	9.00						
SPT	9.50	50 (10,15/50 for 155mm)					
D	10.00				Extremely weak to very weak, bright reddish brown, fine grained argillaceous, fissured, MUDSTONE. [KINGSCOURT GYPSUM FORMATION]		

Continued on Next Page

Water Monitoring Depth	Date	Water Strikes Struck	Date	Flow	Shift Information Depth	Water	Remarks	Date Time	Depth Related Remarks Top	Base	Remarks	Backfill Top	Base	Type
		2.00m	28/04	Moderate	12.00			28/04/2021 16:30	0.00	1.20	Obstruction time - Hand dug inspection pit.			

Termination Reason:
Borehole terminated at scheduled completion depth.

General Remarks:

AGS

Scale: 1:60

Borehole Log

GroundCheck SITE INVESTIGATION SPECIALISTS

Location ID: **BH03**

Page2/2

Date Start:	Location Type:	Project ID:	Project Name:	Easting:	Northing:
28/04/2021	Rotary open hole	21-2676	Proposed Playing Pitch	680546	799543

Date Finish:	Logged By:	Site Location:	Elevation:
28/04/2021	S. Thompson	Drummond, Magheracloone, County Monaghan	

Samples & In-situ Testing Type	Depth (m)	Results & Information	Wells	Water	Legend	Stratum Description	Scale	Depth (m)	Reduced Level (m)
SPT D	11.00 11.00	50 (25 for 125mm/50 for 125mm)				Extremely weak to very weak, bright reddish brown, fine grained argillaceous, fissured, MUDSTONE. [KINGSCOURT GYPSUM FORMATION]	11.0		
D	12.00					End of Borehole at 12.00m	12.0	12.00	
							13.0		
							14.0		
							15.0		
							16.0		
							17.0		
							18.0		
							19.0		
							20.0		
							21.0		

Water Monitoring Depth	Date	Water Strikes Struck	Date	Flow	Shift Information Depth	Water	Remarks	Date Time	Depth Related Remarks Top	Base	Remarks	Backfill Top	Base	Type
					12.00			28/04/2021 16:30						

Termination Reason:
Borehole terminated at scheduled completion depth.

General Remarks:

AGS

Scale: 1:60

GroundCheck
SITE INVESTIGATION SPECIALISTS

Borehole Log

Location ID: **BH04**

Page1/2

Date Start:	Location Type:	Project ID:	Project Name:	Easting:	Northing:
28/04/2021	Rotary open hole	21-2676	Proposed Playing Pitch	680605	799436

Date Finish:	Logged By:	Site Location:	Elevation:
28/04/2021	S. Thompson	Drummond, Magheracloone, County Monaghan	

Samples & In-situ Testing: Type	Depth (m)	Results & Information	Wells	Water	Legend	Stratum Description	Depth (m)	Reduced Level (m)
						Grey, silty, fine to coarse SAND and fine to coarse sub-angular GRAVEL [MADE GROUND]	0.30	
D	0.50					Firm, grey, gravelly, very sandy, clayey SILT with bands of silty, gravelly, fine to coarse sand. Gravel is fine to coarse sub-angular,. Sand is fine to coarse. [MADE GROUND]		
SPT D	1.00 1.00	N=14 (1,2/2,2,4,6)					1.80	
SPT D	2.00 2.00	N=12 (2,2/2,2,4,4)				Firm, becoming stiff, reddish brown and brown, slightly gravelly, sandy, silty CLAY with occasional thin bands of silty, fine to coarse sand and fine to medium sub-angular gravel. Gravel is fine to medium sub-angular to sub-rounded. Sand is fine to medium. [MADE GROUND]		
SPT D	3.00 3.00	N=13 (2,2/3,3,3,4)						
SPT D	4.00 4.00	N=19 (3,4/4,5,5,5)						
		Water Strike 4.50m		▼				
D	5.00							
SPT	5.50	N=23 (4,4/5,5,5,8)						
D	6.00							
SPT D	7.00 7.00	N=28 (4,5/6,7,7,8)						
D	8.00						8.30	
SPT	8.50	50 (12,13/50 for 200mm)				Extremely weak to very weak, bright reddish brown, fine grained argillaceous, fissured MUDSTONE [KINGSCOURT GYPSUM FORMATION]		
D	9.00							
SPT D	10.00 10.00	50 (25 for 125mm/50 for 150mm)						

Continued on Next Page

Water Monitoring: Depth	Date	Water Strikes: Struck	Date	Flow	Shift Information: Depth	Water	Remarks	Date Time	Depth Related Remarks: Top	Base	Remarks	Backfill: Top	Base	Type
		4.50m	28/04	Seepage	13.50			28/04/2021 16:30	0.00	1.20	Obstruction time - Hand dug inspection pit.			

Termination Reason:
Borehole terminated at scheduled completion depth.

General Remarks:

AGS

Scale: 1:60

Borehole Log

GroundCheck
SITE INVESTIGATION SPECIALISTS

Location ID: **BH04**

Page2/2

Date Start:	Location Type:	Project ID:	Project Name:	Easting:	Northing:
28/04/2021	Rotary open hole	21-2676	Proposed Playing Pitch	680605	799436

Date Finish:	Logged By:	Site Location:	Elevation:
28/04/2021	S. Thompson	Drummond, Magheracloone, County Monaghan	

Samples & In-situ Testing: Type	Depth (m)	Results & Information	Wells	Water	Legend	Stratum Description	Depth (m)	Reduced Level (m)
D	11.00					Extremely weak to very weak, bright reddish brown, fine grained argillaceous, fissured MUDSTONE [KINGSCOURT GYPSUM FORMATION]		
SPT	11.50	50 (25 for 50mm/50 for 50mm)						
D	12.00							
D	13.00						13.50	
						End of Borehole at 13.50m		

Scale: 11.0, 12.0, 13.0, 14.0, 15.0, 16.0, 17.0, 18.0, 19.0, 20.0, 21.0

Water Monitoring: Depth	Date	Water Strikes: Struck	Date	Flow	Shift Information: Depth	Water	Remarks	Date Time	Depth Related Remarks: Top	Base	Remarks	Backfill: Top	Base	Type
					13.50			28/04/2021 16:30						

Termination Reason:
Borehole terminated at scheduled completion depth.

General Remarks:

AGS

Scale: 1:60

APPENDIX 7.2

Subsidence at the former Underground Gypsum Mines (Drumgill & Drumgoosat) near Kingscourt, Co. Cavan, Ireland - SRK - May 1999

REPORT ON SUBSIDENCE AT FORMER UNDERGROUND GYPSUM MINES NEAR KINGSCOURT, CO. CAVAN, IRELAND

Prepared for:

Department of the Marine and Natural Resources,
Government of Ireland

Prepared by:

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Project U1225 **MAY 1999**

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Project U1225 **MAY 1999**

REPORT ON SUBSIDENCE AT FORMER UNDERGROUND GYPSUM MINES NEAR KINGSCOURT, CO. CAVAN, IRELAND

1 INTRODUCTION

1.1 General

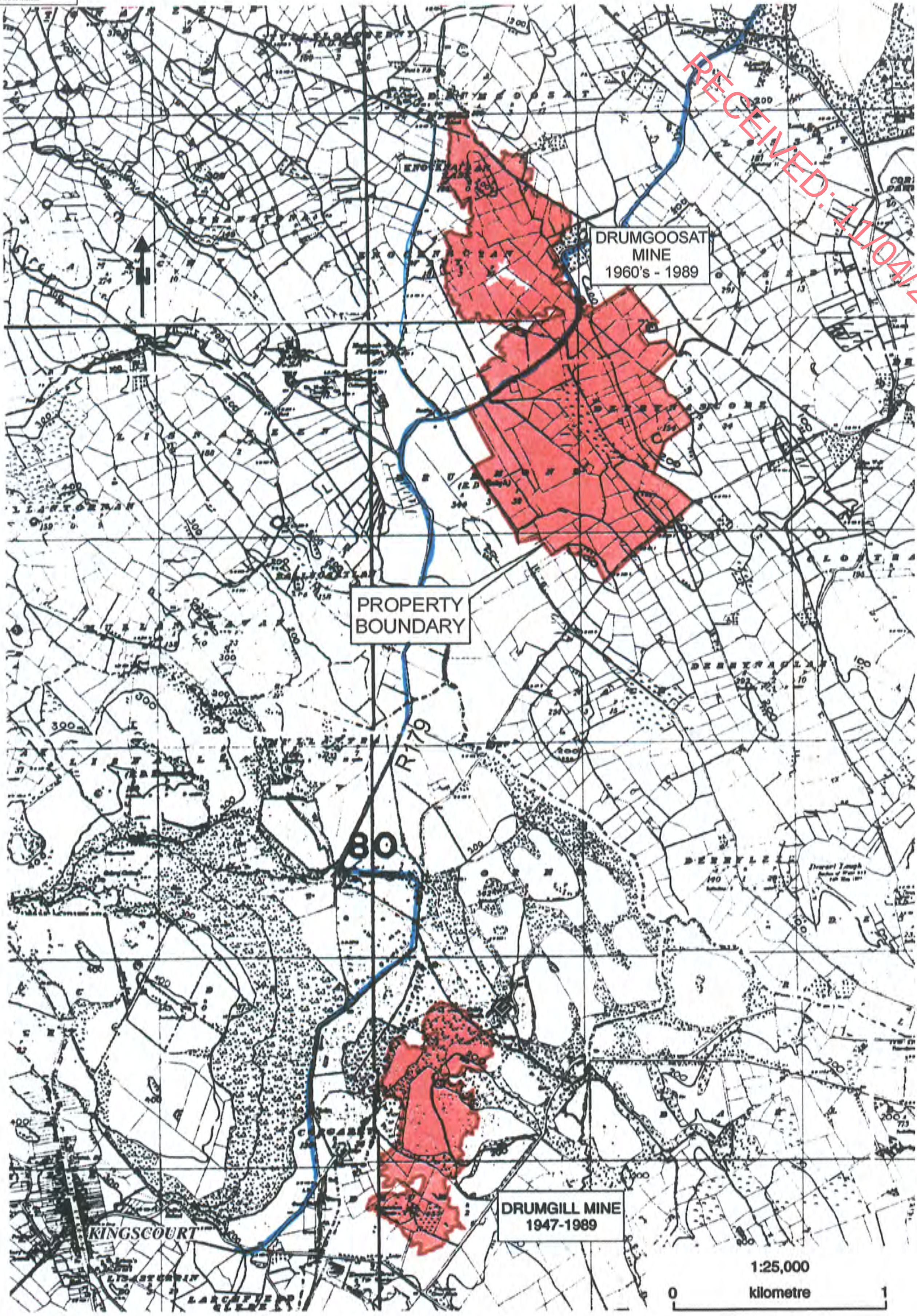
The Kingscourt Gypsum Deposit is located about 80km north of Dublin and straddles the border of Counties Cavan and Monaghan (Fig.1.1). Two separate mines, Drumgoosat and Drumgill, have been operated by Gypsum Industries plc and its predecessors since the late 1940's and 1950's respectively. The gypsum deposit, which is located 50 to 100m below ground surface, has been mined by shallow underground room and pillar methods. Underground mining at both mines was completed in 1989. The deposit is currently being exploited by open-pit methods at the southern end of the Drumgoosat Mine, in Knocknacran Quarry.

A number of mining-related surface subsidence features have occurred over the deposit since underground mining was completed. The Department of the Marine and Natural Resources of the Government of Ireland have appointed Steffen, Robertson and Kirsten (UK) Ltd (SRK) to undertake an independent review of the gypsum mine subsidence risk.

1.2 Terms of reference

The study was carried out under the following terms of reference provided by the Department of the Marine and Natural Resources:

"To provide to the Minister for the Marine and Natural Resources an independent assessment of the short and long term stability of the former gypsum mines near Kingscourt, County Cavan and to recommend any monitoring or remedial measures which might reasonably be required of Gypsum Industries plc."



DATE: 25/11/98	PROJ. No. U1225	GYPSUM MINE SUBSIDENCE	
SRK THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT		Location Plan	Figure 1.1

The study of the Drumgoosat Mine is designed to provide an assessment of its stability and the possible effects of subsidence on public roads and other infrastructure not owned by Gypsum Industries, lying above and within the area of influence of the underground mine workings. An assessment was also required of the potential risk of sudden unexpected subsidence occurring, which may cause damage and consequent risk to public safety.

The objective of the study of the Drumgill Mine is to assess the risk of subsidence and, depending upon the magnitude of the risk, provide advice on the requirement for more detailed investigation. No buildings or tarred roads overly Drumgill Mine.

1.3 Programme of Site Work

A visit was made to the mine between 7th and 9th October 1998 by Dr Ian Brackley and Mr Neil Marshall, SRK Principal Engineer and Senior Geotechnical Engineer, respectively. During the period of the visit SRK was given the opportunity to inspect the Knocknacran Quarry, the surface subsidence features at both Drumgoosat Mine and Drumgill Mine and as much of the underground workings at Drumgoosat Mine as was safely accessible.

During the open pit inspection, mapping of some of the exposed gypsum faces for the determination of rock characteristics was carried out, and the exposures on the sides of the quarry gave an excellent view of the geological succession. The upper surface of the gypsum had been exposed in one part of the quarry, showing clearly the great variation in its elevation, a very important consideration when assessing the stability of the roof of the underground workings. Also exposed in the side of the quarry were solution cavities in the upper gypsum, caused by dissolution of the rock by seepage water.

All underground inspections were carried out under the supervision of the Mine Manager of Gypsum Industries Ltd, Mr Barry McKeon. The inspections included typical areas of the underground workings, and visits to zones below the roads and the Community Centre. It was not possible to inspect the area directly below the existing subsidence, because of the accumulation of fallen rock, and because of the general instability of the roof, but the failed roof beams on the perimeter of the failure were examined.

1.4 Information available

Copies of the documents and plans listed in Table 1.1 were provided by Gypsum Industries Ltd and others for inspection and review. The most important document was the geotechnical report on subsidence by Dr. Lehane of Trinity College, Dublin, which included information on material properties, monitoring results and an analysis of the subsidence. In addition to these documents Mr McKeon of Gypsum Industries Ltd provided useful factual and anecdotal information on many aspects of the mine history and operation relevant to subsidence issues.

Table 1.1 : Information provided for the study

DOCUMENT DESCRIPTION	AUTHOR/SOURCE	DATE
Geotechnical Report on the Subsidence at Drumgoosat Mine, Co. Cavan.	Dr Barry Lehane, Trinity College, Dublin	April 1997
Update on Geotechnical Report on the Subsidence at Drumgoosat Mine, Co. Cavan.	Dr Barry Lehane, Trinity College, Dublin	June 1998
The geological setting of Permian gypsum and anhydrite deposits in the Kingscourt district, Counties Cavan, Meath and Monaghan.	P.R.R. Gardiner and P. McArdle [in Bowden, Earls, O'Connor & Pyne (eds.) 1992. The Irish Minerals Industry 1980-90. IAEG, pp 301-316.]	1992
Laboratory rock strength testing results	Gypsum Industries	1998
Photograph of Sinkhole, Rowntrees Field	Gypsum Industries	Jan 1997
Geological Description of Gypsum Sections	Gypsum Industries	
Knocknacran Quarry survey plan showing contours of exposed gypsum hangingwall	Gypsum Industries	Sept 1998
Geological cross sections through the southern area of Drumgoosat Mine	Gypsum Industries	1983
Location plan of Geological cross sections	Gypsum Industries	1983
Site Map and Drumgoosat Mine 1:2,500	Gypsum Industries	1983
Drumgoosat Mine, Lower Seam Workings 1:2,500	Gypsum Industries	October 1989
Drumgoosat Mine, Lower Seam Workings 1:1,250	Gypsum Industries	October 1989
Drumgoosat Mine, Upper Seam Workings 1:2,500	Gypsum Industries	October 1989
Drumgoosat Mine, Upper Seam Workings, 1:1,250	Gypsum Industries	October 1989
Drumgill No.2 Mine, Lower Seam Workings,1:2,500	Gypsum Industries	August 1989
Drumgill No.2 Mine, Lower Seam Workings,1:1,250	Gypsum Industries	September 1989
Drumgill No.2 Mine, Upper Seam Workings,1:2,500	Gypsum Industries	August 1989
Drumgill No.2 Mine, Upper Seam Workings,1:2,500	Gypsum Industries	September 1989
Plan of Land Ownership above Drumgoosat Gypsum Mine	Department of the Marine and Natural Resources	1998

The mine plans contain information about the level and geometry of the workings, but the position of the workings relative to the top and bottom of the gypsum is only known at several locations

2 THE MINES

Gypsum mining in the Kingscourt area has been in progress since the early 1920's, initially by quarrying. Gypsum quarrying began at Drumgoosat in 1958, and underground mining began in the 1960's, and began at the smaller Drumgill Mine in the 1940's. Mining at both mines was completed in 1989. Underground mining at both properties was carried out generally between 50m to 100m below ground level using room and pillar mining methods. Rooms were mined 10m wide and 5m to 6m high, and the average pillar size was approximately 12m x 12m. Using this layout a total area extraction of 75% was achieved. Volume extraction reached 33%. At Drumgoosat mine, rooms had a rectangular cross-section. At Drumgill, the rooms also had a rectangular cross section, except in some areas of the Lower Bed, where roof ground conditions were somewhat less favourable due to the presence of shaley partings. These rooms were mined with an arched roof. Room orientations varied slightly throughout the mine, but were generally parallel to major discontinuity sets and intersecting at right-angles. To the north and western limits of the underground mine the gypsum dips at an angle of up to 30°. The mining depth in these areas increased to up to 110m.

SRK was informed by the mine manager that the mining horizons were located so that there was at least 3m of gypsum in the floor and 3m of gypsum in the roof beam. There is significant variation in the elevation of the upper surface of the gypsum, as illustrated by the exposure of the top of the gypsum in Knocknacran Quarry, Figure 2.1. At one point there is a vertical step of more than four metres. Therefore, to ensure that the gypsum roof beam was at least 3m thick, probe drilling was undertaken at each room intersection. If the drilling showed that the roof beam was less than 3m thick, the mining elevation was lowered.

In order to extract the best quality gypsum, which was to be found in either the B or D sections of the gypsum, (see Section 7.2 for descriptions of these categories of gypsum), the mining elevation varied across the two sections. The gypsum roof beam is likely to be thinner where mining was carried out in the D section gypsum, because the D Section gypsum lies near the top of the unit, but the criterion of 3m minimum thickness is understood to have been maintained during mining. Figure 2.2 shows two examples of roof beams in D section gypsum. The upper photograph shows a massive beam of competent gypsum where underground workings have been exposed in the southern wall of Knocknacran Quarry. The lower photograph is of a roof beam in the underground workings, showing some evidence of failure as a result of parting along a shaley bedding plane.

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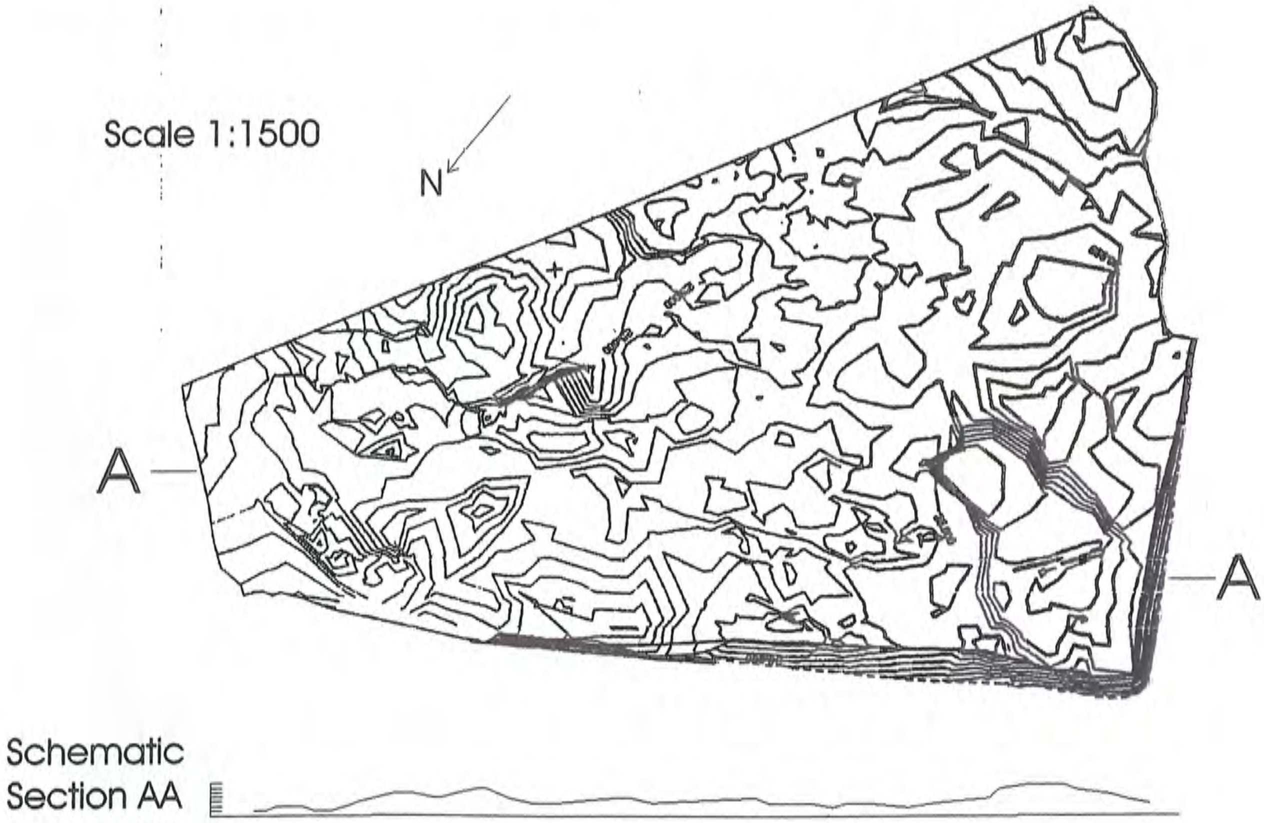
The exposed upper surface of the gypsum, looking north. The erratic dissolution of the gypsum by water seepage has resulted in sudden changes in elevation of up to five metres.



DATE: 20/10/98	PROJ. No. U1225	GYPSUM MINE SUBSIDENCE	
SRK THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT		The Upper Gypsum Surface in The Quarry	Figure 2.1

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Competent roof beam of greater than 3m thick, exposed in the south east wall of Knocknacran Quarry.

The gypsum roof beams are a minimum of 3 metres in thickness,
and show no signs of failure. This was the only beam noted during the
inspections which exhibited a partial slabbing failure on close bedding.

DATE: 20/10/98	PROJ. No. U1225	GYPSUM MINE SUBSIDENCE	
SRK THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT		**Examples of Roof Beams**	Figure 2.2

At both mines, upper and lower beds were mined concurrently. Mining layouts were designed to superimpose the pillars in each bed, to eliminate problems associated with pillars in the upper seam overlying voids in the lower bed.

3 GROUNDWATER

Mining at Drumgoosat and Drumgill was carried out below the original water table level, and water was pumped from the workings. Pumping has not continued at Drumgill after closure, but the ground water at Drumgoosat Mine is being maintained at 19m below Ordnance Datum by pumping from underground pump stations. This is predominantly a requirement for quarry mining, but also ensures that the flat lying areas of the mine remain dry. Areas in the west of the mine in the dipping areas are flooded.

There is geological evidence of groundwater flow on the footwall and hangingwall gypsum contact, particularly within the F Section gypsum There is, however, no record of general groundwater seepage from the roof of the mine openings.

A number of boreholes drilled from underground have intersected groundwater in the hangingwall at Drumgoosat, and some now act as conduits for flow into the workings

Drumgill mine is completely flooded, with the water level having risen over a period of eight years. Groundwater was observed to be overflowing from the pool at the mine portal entrance, Figure 3.1.

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There are a number of sinkholes above the shallow workings near the portal.

The Drumgill portal is completely submerged and small quantities of water are spilling to the natural stream. The workings have flooded over a period of about eight years

DATE: 20/10/98	PROJ. No. U1225	GYPSUM MINE SUBSIDENCE	
SRK — THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT		**Drumgill Mine Showing Adit Entrance & Sinkhole**	Figure 3.1

4 EXISTING SURFACE SUBSIDENCE

There are two types of subsidence at Drumgoosat mine:

- surface settlement caused by subsidence of an area of the underground mine
- small sinkholes (less than 10m in diameter) caused by the upward migration of a cavity resulting from the failure of a single roof beam by "doming"

At Drumgill Mine there are sinkholes resulting from the collapse of incompetent roof beams in the area of shallow workings, but no area of surface settlement.

4.1 Surface settlement

There is one area of active progressive subsidence. The area is situated at the north-eastern limit of the underground mine. The subsidence trough is about 140m in diameter. The centre of the trough is located 50m south of the minor road linking the main road, R179, to Drumgoosat village. Subsidence was first observed in the early 1980's, when a settlement monitoring network was installed. By June 1998, the centre of the trough had subsided by 1500mm, with subsidence of the road reaching 600mm. The main surface expression of the subsidence is a series of radial cracks across the road and a large crack through a wall on the roadside. A plan of the subsidence site showing the limit of the subsidence trough and a graph of cumulative subsidence with time are illustrated in Figure 4.1. The subsidence, which has been accelerating since the mid 1980's, was the subject of a detailed site investigation and geotechnical analysis carried out in 1997 by Dr Barry Lehane of Trinity College, Dublin. The report concluded that subsidence was initiated by bearing capacity failure in an area where the gypsum had been mined-out to expose the basal mudstone. Pillar failure resulted in an effective increase in roof beam span. Tensile failure occurred where the beam is supported by stiffer pillars and the roof beam is now settling en-masse. Dr Lehane has predicted that settlement will continue until the mine openings have closed, in approximately six years (ie., by 2005) either by failure of and bulking of the failed material in the mine void, or by complete settlement of the roof beam. He has also estimated that a total vertical settlement of the road of 1.2m will have occurred by the time subsidence has been completed.

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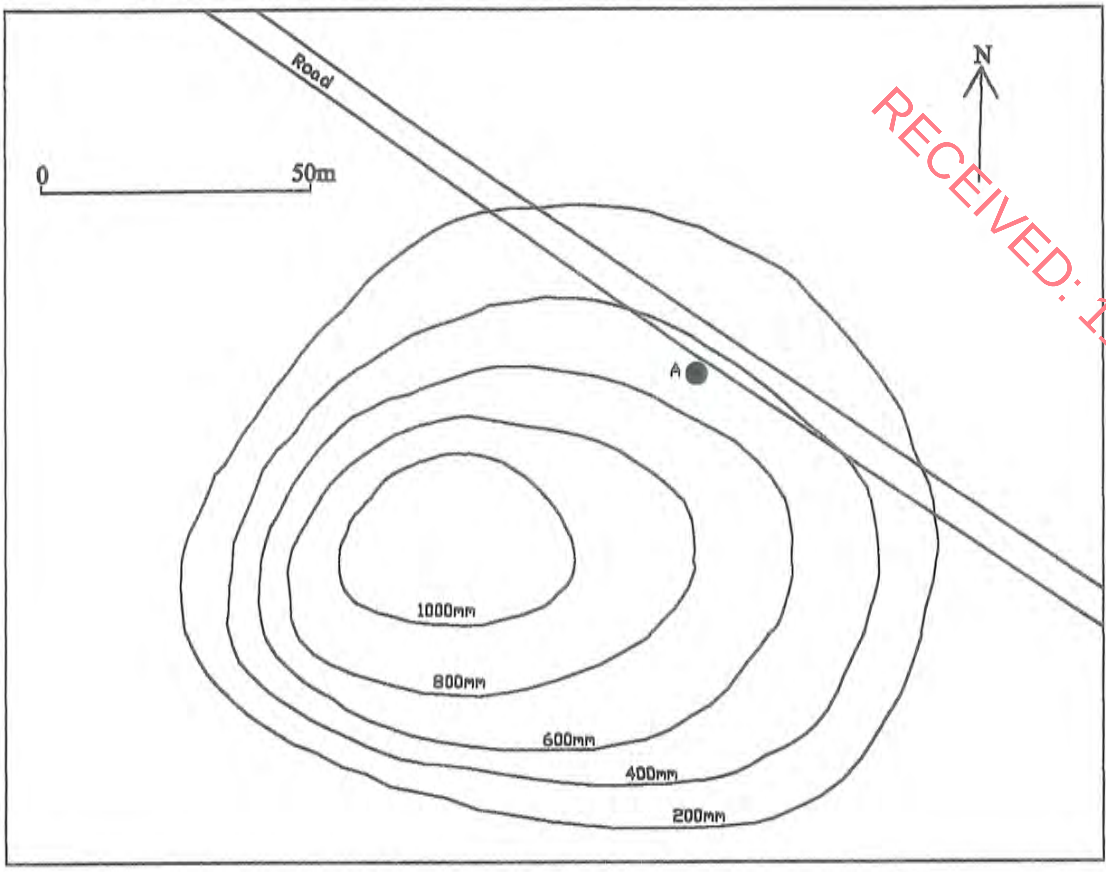
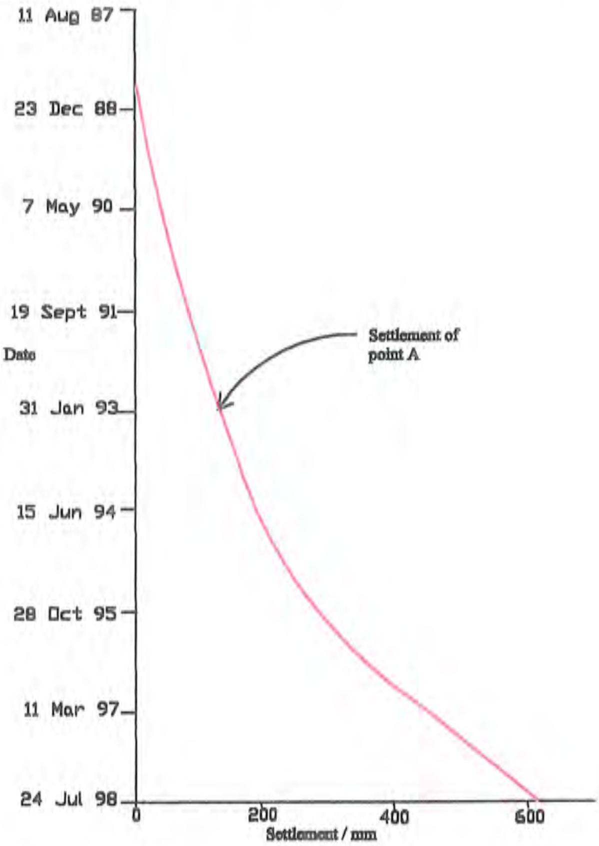


Subsidence Contours, March 1997z



Development of settlement with time (to March 1997)

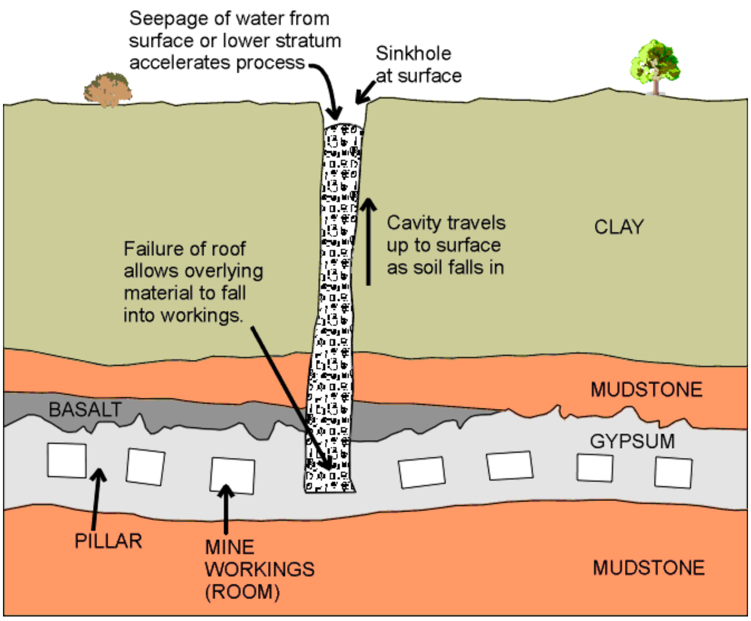
Date : 30/11/98	PROJ. No: U1225	GYPSUM MINE SUBSIDENCE	
SRK THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT		Existing Subsidence	Fig. 4.1

4.2 Sinkholes

Two small sinkholes have been recorded at Drumgoosat Mine. These have occurred below areas with 30m to 40m cover and appear to have been caused by progressive failure of doming features in the roof of the mine excavation. These domes are associated with solution cavities filled with mudstone or glacial till (Figure 4.2). When the dome forms as a cavity in the gypsum roof, the overlying weathered basalt or Middle Mudstone then flows through the void into the mine workings. If there is sufficient removal of weathered basalt or mudstone to undermine the overlying glacial till a sinkhole propagates to surface.

About 10 sinkholes, up to 50m in diameter and about 6m deep, have developed adjacent to the entrance of the Drumgill mine at the southern end of the mine property. The sinkholes (Figure 3.1) have occurred where mining was carried out closest to surface and where pillar size and shape appears to be relatively small and irregular. The sinkholes are consistent with extensive roof beam failure, where the roof beam is located in a zone of weathering, or is particularly thin. The mechanism of sinkhole formation is illustrated schematically below. The area of subsidence which occurred just north of the main road has been backfilled.

The Drumgill failures in shallow, weathered material involving two or more room widths, are more extensive than the doming failures observed in areas of deeper mining at Drumgoosat, which appear to be limited to a single room width.



Schematic section showing sinkhole formation
(sinkhole does not necessarily form completely vertically)

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Eroded upper surface of gypsum

Solution cavity infilled with glacial till

DATE: 30/11/98	PROJ. No. U1225	GYPSUM MINE SUBSIDENCE	
SRK THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT		Solution Cavity in Upper Gypsum	Figure 4.2

5 EXISTING SURFACE STRUCTURES

5.1 Land ownership

Figure 5.1 gives a plan of the Drumgoosat Mine area showing land ownership above the mine. The major proportion of the surface above Drumgoosat Mine is owned by Gypsum Industries Ltd. Two strips of land through the mining area and a number of small parcels of land at the periphery of the underground mining area are not owned by Gypsum Industries.

No land ownership plan was available for Drumgill mine, but it is understood that most of the area is owned by Gypsum Industries.

5.2 Surface structures

Most of the area overlying the underground workings at Drumgoosat and Drumgill is agricultural land. The structures outside the Gypsum Industries property comprise:

- **The main Carrickmacross to Kingscourt road (R179)** passes south-west/north-east across the centre of Drumgoosat mine.

- **The road to Drumgoosat village** on the north eastern side of the mine crosses the edge of the underground mine workings at three points. The southerly point contains the area of active subsidence.

- **A Community Centre** and sports field are located on the eastern side of the Drumgoosat mine area.

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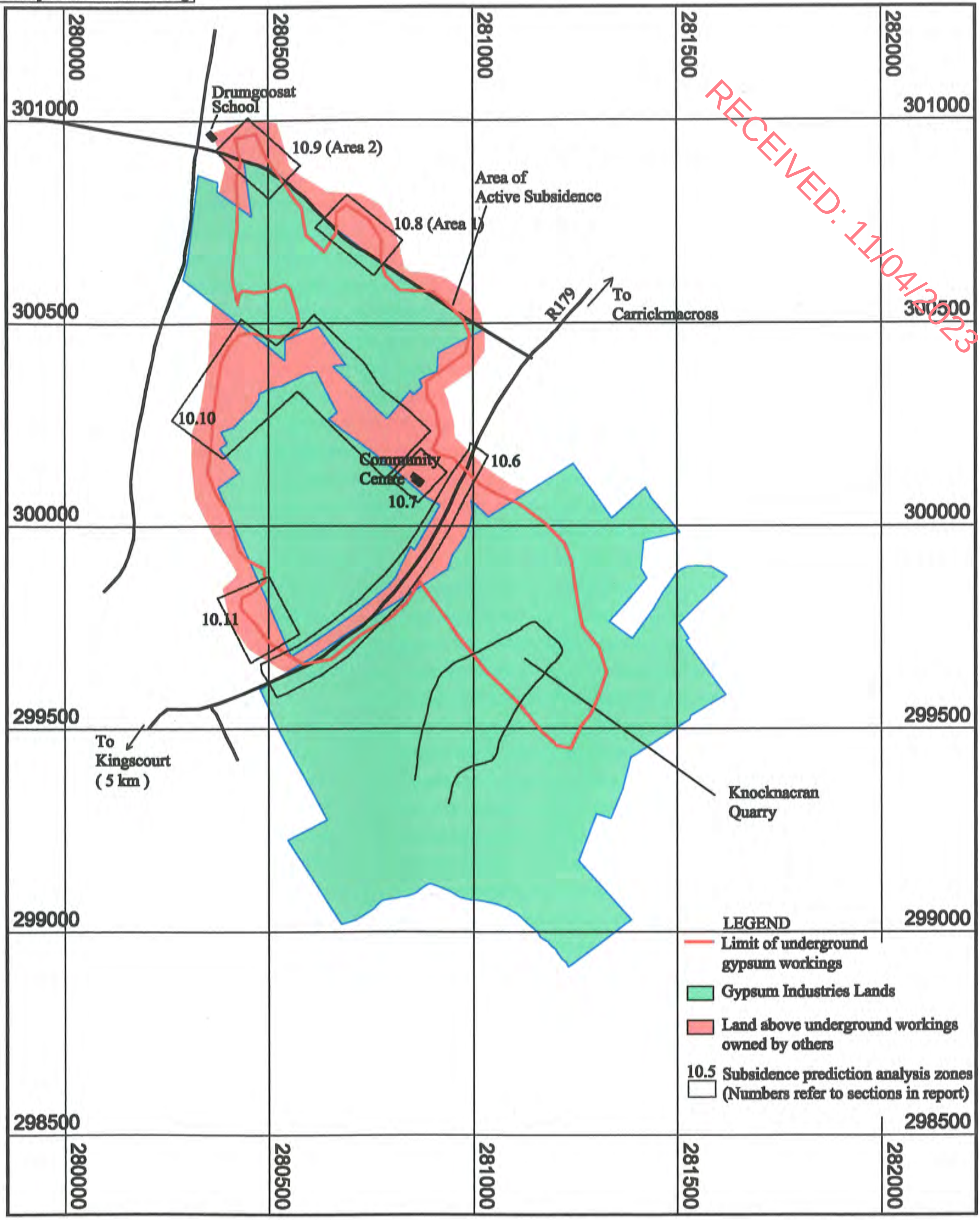

Date:21/10/98	PROJ. No: U1225	GYPSUM MINE SUBSIDENCE	Scale 1:12500
SRK THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT		Plan of Drumgoosat Mine Area	Fig. 5.1

6 GEOLOGY

6.1 General geology

The significant Permian deposits of gypsum in Ireland occur in a narrow north-south belt, up to 18m in thickness, 12km in length and with a maximum width of 1.2km, east of the town of Kingscourt, Co. Cavan. These deposits occur in the lower part of the Permian-Triassic sequence that forms the westerly cap to a large north-south trending Carboniferous outlier within the Lower Palaeozoic Longford Down Massif. The western margin of the outlier is truncated by a major north-south fault system.

6.2 Stratigraphy

The Permian succession consists of a conglomerate at the base, overlain by a mudstone sequence containing the two major gypsum units. The lower gypsum unit generally exceeds 20m thickness. The upper gypsum unit is typically 9m thick. The sequence passes conformably into Lower Triassic red-beds dominated by sandstone. The Permian-Triassic sequence is intruded by basaltic and doleritic sills and dykes of Tertiary age. The area is blanketed by glacial tills, boulder clays, of up to 30m thickness or more. An extensive example of the stratigraphy is exposed in Knocknacran Quarry (Appendix C).

A general stratigraphic column of the Kingscourt area is given in Figure 6.1. The gypsum units are sub-divided into fifteen stratigraphic horizons or Sections. The A Section lies at the base of the Lower Gypsum unit, the J Section lies at the base of the Upper Gypsum unit. The stratigraphy and brief descriptions of each section are given below. These descriptions have been extracted from Gardiner and McArdle (1992). Geotechnical descriptions of the more important units are given in Section 7.

Lower Gypsum Unit

A Section : *Mudstone*. The base of the lower unit. Is transitional in nature grading upwards from 50% gypsum to good quality gypsum. This section has poorly defined boundaries and can vary in thickness from 0m to 4.5m.

B Section : *Nodular gypsum*. Consisting of thickly bedded, good quality white to grey gypsum with thin bands and strings of shale. This is one of the main sections of gypsum to have been exploited by underground mining. This unit varies between 3m and 12m in thickness.

C Section : Mudstone. A shaley section varying from grey gypseous shale to white gypsum. In a few borehole cores the shale content is negligible but normally the section consists of 1.5m to 3m of very shaley gypsum. Where mining has been undertaken in the B section gypsum, C section gypsum forms the roof beam.

D Section : Laminated gypsum. Good quality, light brown laminated gypsum showing well defined rhythmic banding of wavy shale films. There is a change in colour at the top of the section with the material changing from grey to creamy pink or red. The unit varies between 4.6m and 9m in thickness and has also been extensively exploited by underground mining.

E Section : Banded magnesium rich gypsum. Cream to pink gypsum showing alternating bands up to 5cm thick of white and pink gypsum. This section is usually high in carbonates of lime and magnesium.

F Section : Banded magnesium rich gypsum. This is effectively the top of the E Section but is characterised as being heavily leached by circulating groundwater and consists of gypsum interbanded with thin layers of soft talcose clay. The E and F sections combined are typically 3m thick. The E and F sections comprise the roof beam in areas of the mine where D Section gypsum has been extracted.

G Section : *Massive white gypsum.* This unit consists of fine grained, white to creamy gypsum of high purity and is between 3m and 4.5m thick.

Interbeds

H Section : *The Main Sill.* A fine grained homogeneous basalt directly overlying the Lower Gypsum Unit. The unit has undergone intensive hydrothermal alteration and near surface extensive lateritic weathering. The sill is generally present over the east of the orebody, dying out towards the west where the gypsum units dip down. The basalts tend to be intensely weathered to a fine grained sand and can be up to 60m thick in places.

I Section : *Middle Mudstone Member.* Where the basalt sill is not present the interbed consists of the Middle Mudstone Member. This unit, which is 6m to 12m thick consists of reddish, micaceous, plastic mudstones, often with green reduction spots, and faintly laminated near the base.

Upper Gypsum Unit

J Section : *Massive, fine grained, grey-brown to red, pure gypsum.* The basal section of the upper gypsum unit is 1.6m to 3m thick and comprises good quality gypsum, pink to brown in colour with occasional blebs or strings of marl.

K, L, and M Sections : *Interbanded gypsum and red siltstone laminae.* These are minor units 0.3m to 1.9m thick. They are strongly laminated and contain fibrous veins of gypsum and coarse gypsum crystals. The unit forms the mining footwall of the Upper Bed.

N Section : *Coarse Gypsum.* This is the main upper bed unit exploited by underground mining. It varies between 0m and 4.6m thick. It contains coarsely crystalline, pink gypsum with a variable quantity of marl inclusions in thin irregular veinlets.

O Section : *Massive gypsum*. Contains very pure and fine grained grey or cream coloured laminated gypsum. The unit is 1.5m to 4.6m thick.

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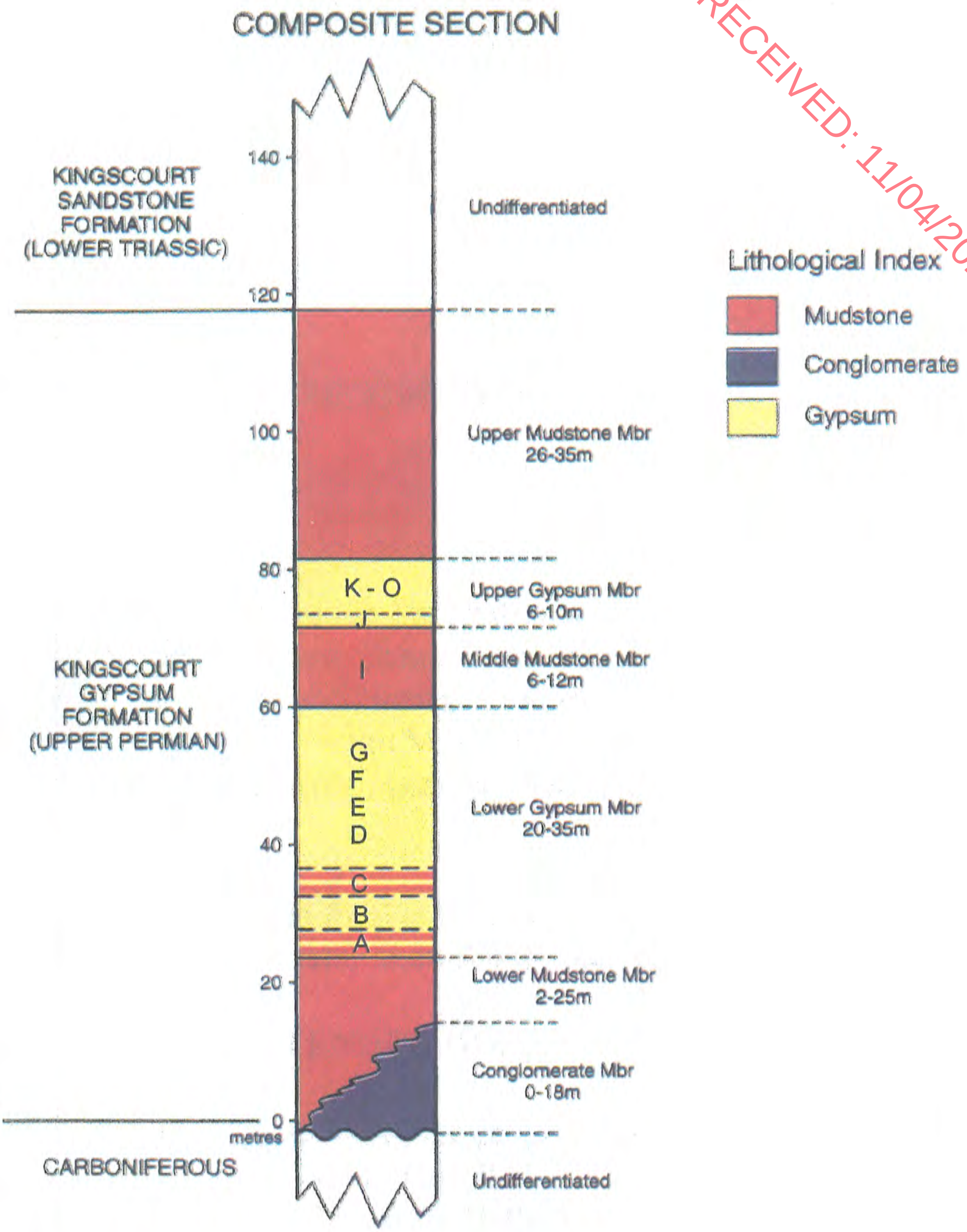

From Gardiner and McArdle (1992)

DATE: 27/11/98	PROJ. No. U1225	GYPSUM MINE SUBSIDENCE	
SRK THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT		Stratigraphy of the Kingscourt Area	Figure 6.1

7 ROCK MASS CHARACTERISATION

7.1 Derivation of Rock Mass Strength Properties

The ability of the mine structures (floor, pillars and roof beams) to withstand long term loading and deformation will be dependent upon the rock mass strength of the different lithological units forming these structures. The rock mass comprises both the intact rock material and the rock defects (joints, bedding planes, shear zones, faults etc) which cut through the rock material. The rock mass classification, together with reported laboratory strength testing results, has been used to provide an estimate of the strength of the main lithological units which form the mine structures.

The Mining Rock Mass Rating Classification (MRMR) scheme developed by Laubscher (1990) applies a rating to a number of parametric values which characterise a specific rock unit. The parametric values rated are –

- intact rock strength,
- fracture frequency or RQD and joint spacing,
- joint roughness, joint infill type and strength,
- groundwater condition.

The sum of the parametric ratings is known as the rock mass rating (RMR) and is a number in the range 0 to 100. The various parametric values required for input into the classification scheme were obtained from the following sources:

1. Laboratory testing data presented in Dr. Lehane's subsidence report dated April 1997;
2. Laboratory testing data provided by Gypsum Industries;
3. Mapping of quarry and underground pillar faces carried out by SRK geotechnical engineers.

The intact rock strength and RQD data together with underground and quarry mapping data sheets are in appendix A.

Adjustments can be applied to the RMR value to take into account the long term response of the rock mass to factors such as weathering, discontinuity orientation, blasting and mining induced stress changes. The adjusted value is known as the Mining Rock Mass Rating (MRMR). No adjustments were made in this case.

For each parameter a range of values has been selected to represent the likely upper and lower bound conditions prevailing within the gypsum rock mass. This range is one standard deviation either side of the mean value of each parameter. Engineering judgement has been used to correlate field strength assessments with laboratory strengths. The MRMR is used in conjunction with the intact rock strength to give a Design Rock Mass Strength (DRMS). The Design Rock Mass Cohesion (DRMC) is estimated from the relationship proposed by Stacey and Page (1986) which is,

$$DRMC = DRMS \times 0.16$$

The combined thickness of the B, C and D gypsum sections is estimated to be between 12m and 24m. It would be possible to locate the 6m high mining horizon within these sections while maintaining a 3m footwall and hangingwall gypsum cover without exposing any of the other gypsum sections. This scenario was broadly confirmed during inspection of the underground workings. The main lithological units which most probably control the global mine stability are the B, C and D Section gypsum sections. These sections comprise the floor, pillars and roof beam of the mine. Accordingly rock mass strength estimations have been made for these sections only. From the descriptions of the upper gypsum sections it is estimated that these units will have similar rock mass strength properties to those of the C section gypsum. For the back analysis of the active subsidence area the rock mass strength of the basal mudstone has also been estimated.

7.2 B and D Section Gypsum

These are generally massive, good quality gypsum beds containing thin bands of shale. Gypsum mining has largely been concentrated in these sections. An example of massive gypsum is shown in Figure 2.2.

Table 7.1 gives the derivation of the rock mass classification for B and D gypsum.

7.3 C Section Gypsum

This is a shaley gypsum about 1.5m to 3m in thickness. Where mining has been in the B Section gypsum, the C Section gypsum forms the roof beam. Its rock mass strength is generally lower than that of the B and D section gypsum, due largely to the percentage of lower strength shale contained within the unit. The rock mass classification data for this unit is tabulated in Table 7.2.

Table 7.1 : B and D Section Gypsum – Rock Mass Classification

Parameter	Values for Minimum rating	Values for Maximum rating
Rock Strength (MPa)	20	30
RQD (%)	65	95
Joint Spacing (m)	0.25	1.00
No of Joint Sets	3	
Fracture Frequency (per m)	4.0	1.0
Large Scale J'nt Expression	Curved	Wavy, unidirectional
Small Scale J'nt Expression	Smooth, undulating	Smooth, stepped
Joint Alteration	No alteration	
Joint Infill	No infill	
Water	Dry	Dry

Ratings	Minimum	Maximum
Rock Strength	3	4
Fracture Frequency	16	31
Joint Condition	26	34
RMR	**45**	**69**

MRMR	**45**	**69**
DRMS (MPa)	8	20
DRMC (MPa)	1.3	3.2

Table 7.2 : C Section Gypsum – Rock Mass Classification

Parameter	Values for Minimum rating	Values for Maximum rating
Strong Rock Strength (MPa)	20	30
Weak Rock Strength (MPa)	8	10
Percentage of Weak Rock	50	10
Intact Rock Strength (MPa)	10	24
RQD (%)	65	95
Joint Spacing (m)	0.10	0.50
No of Joint Sets	3	
Fracture Frequency (per m)	10.0	2.0
Large Scale J'nt Expression	Curved	Wavy, unidirectional
Small Scale J'nt Expression	Smooth, undulating	Smooth, stepped
Joint Alteration	No alteration	
Joint Infill	No infill	
Water	Dry	Dry

Ratings	Minimum	Maximum
Rock Strength	2	3
Fracture Frequency	10	21
Joint Condition	26	34
RMR	**38**	**58**

MRMR	**38**	**58**
DRMS (MPa)	4	13
DRMC (MPa)	0.6	2.1

7.4 Basal Mudstone

This is a closely-bedded rock with a relatively high dry strength that reduces to a stiff clay consistency when saturated in an unloaded condition. Table 7.3 gives the rock mass classification data. A photograph of the mudstone as exposed in the southern section of the Knocknacran Quarry is shown in Figure 7.1. In his report Dr Lehane states that the mudstone decreases in strength when wet, but he also concludes that failure was initiated by overloading of pillars rather than as a result of weakening of the mudstone by wetting. In order to confirm this conclusion the subsequent stability analyses have been carried out using a dry rock mass strength.

Table 7.3 : Basal Mudstone – Rock Mass Classification

Parameter	Values for Minimum rating	Values for Maximum rating
Intact Rock Strength (MPa)	4	6
RQD (%)	65	85
Joint Spacing (m)	0.10	0.20
No of Joint Sets	3	
Fracture Frequency (per m)	10.0	5.0
Large Scale Joint Expression	Sl undulating	Curved
Small Scale Joint Expression	Smooth undulating	Smooth, stepped
Joint Alteration	No alteration	
Joint Infill	No infill	
Water	Dry	Dry

Ratings	Minimum	Maximum
Rock Strength	1	2
Fracture Frequency	10	15
Joint Condition	24	31
RMR	**35**	**48**

MRMR	35	48
DRMS (MPa)	1.36	2.7
DRMC (MPa)	0.22	.43

7.5 Overburden Units

The overburden units comprise mudstones containing sills of weathered basalt or dolerite, overlain by glacial till (boulder clay). Glacial till exposed in the side wall of a sinkhole at Drumgill is illustrated in Figure 7.1. The strength of these units will generally not play an active role in the mine support, but will be of influence in sinkhole development.

Basal Mudstone overlain by A Section Gypsum

Glacial till exposed in side slope of sinkhole at Drumgill

DATE: 18/11/98	PROJ. Number U1225	GYPSUM MINE SUBSIDENCE	
SRK	THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT	**Photographs Showing Basal Mudstone and Glacial Till**	Figure 7.1

7.6 Rock Jointing and Bedding in the Gypsum

Mapping of a number of underground pillar faces and the south eastern face of Knocknacran Quarry was carried out to determine overall trends of major discontinuities in the gypsum. Details of the main discontinuities are illustrated in the stereographic projection in Figure 7.2 The major discontinuities are north-south and east-west striking near vertical sets with less dominant north-east south-west and north-west south-east striking near vertical sets. Bedding is well developed in the shaley units within the gypsum. The regional bedding trend is 15-30° to the east. Bedding data illustrated on the stereographic projection reflects local dips. The main discontinuity sets and the variable bedding are shown in Figure 7.3 which is a view of the south eastern wall of Knocknacran Quarry.

From observations made during the underground inspection visits, apart from the bedding which can give rise to slabbing type failure where the shaley C section forms the roof of the mine excavations, discontinuities do not appear to contribute significantly to roof beam instability. This is probably the result of the relative roughness of the discontinuity surfaces and horizontal clamping stresses acting through the roof beam, approximately normal to the discontinuity surfaces. These factors will have given rise to a relatively high frictional strength along the discontinuities. The vertical discontinuity sets do have some influence on pillar stability where they strike parallel to the pillar faces. In places, minor spalling of some pillar faces was observed to have taken place. Continued spalling may provide the mechanism for long term pillar failure.

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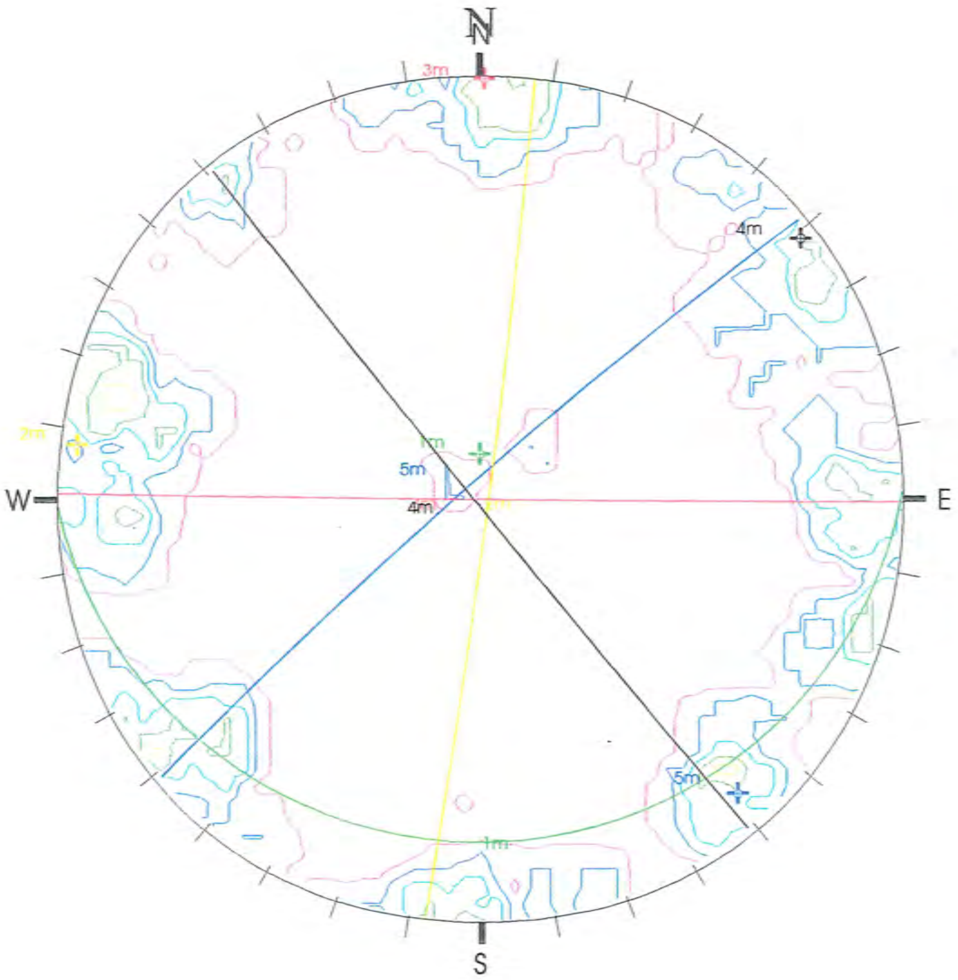


EQUAL ANGLE
LOWER HEMISPHERE

CONTOUR LEGEND

SCHMIDT POLE CONCENTRATIONS
% of total per
1.0 % area

Minimum Contour	=	1.5
Contour Interval	=	1.5
Max.Concentration	=	8.25

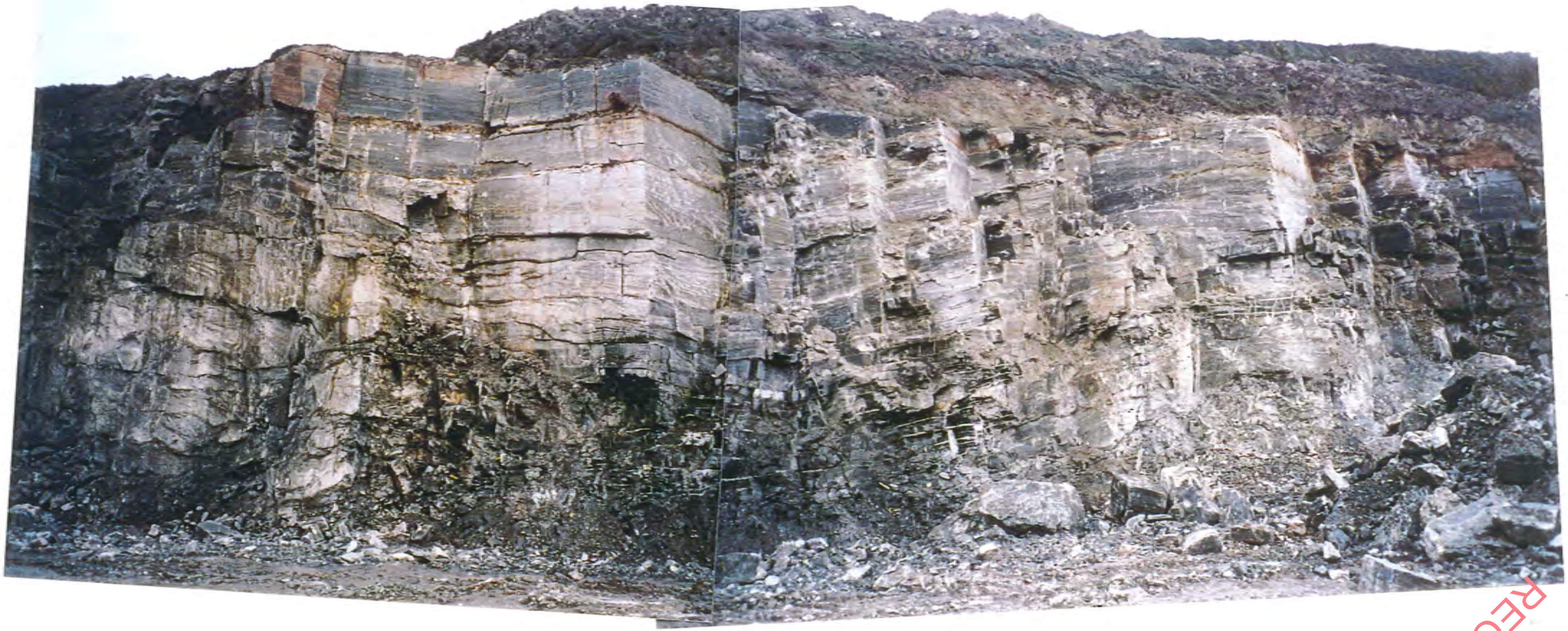
MAJOR PLANES

ORIENTATIONS

#	DIP/DIR.
1	m 12/179
2	[illegible]
3	m 89/181
4	m 89/231
5	m 86/319

97 Poles Plotted
97 Data Entries

DATE: 25/11/98	PROJ. Number U1225	GYPSUM MINE SUBSIDENCE	
SRK	THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT	**Stereographic Projection of Major Discontinuity Sets**	Figure 7.2



Showing the two major sub-vertical joint sets and the variable bedding

DATE: 21/10/98	PROJ. Number U1225	GYPSUM MINE SUBSIDENCE	
SRK	THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT	**Knocknacran Quarry - South East Wall**	Figure 7.3

8 BACK-ANALYSIS OF FAILURE IN AREA OF EXISTING SUBSIDENCE AT DRUMGOOSAT MINE

8.1 Analysis Methodology

Three failure mechanisms can result in surface subsidence. These are;

- pillar yield,
- pillar bearing capacity failure.
- roof beam failure,

Pillar yield occurs when the stress imposed on the pillar by the overlying mass of ground exceeds the strength of the rock mass in the pillar. Bearing capacity failure occurs when the strength of the rock mass below a pillar is not sufficient to support the weight of the pillar and the overburden stresses imposed upon it. Roof beam failure occurs as a result of beam deflection, caused by the weight of overburden, imposing tensile stresses in the beam that are greater than the tensile strength of the rock mass comprising the beam. Simple mathematical models, tributary area, bearing capacity and Voussoir arch models, are available to analyse each type of failure.

The approach used by SRK was firstly to back-analyse the failure of the workings in the area of the existing subsidence using each method. This was to model the failure mechanism proposed by Dr. Lehane and to confirm the mean rock mass strength parameters derived from the rock mass classification studies described in Section 7 of this report.

For tributary area and bearing capacity analyses spreadsheets were set up to calculate the factors of safety. For roof beam stability the Voussoir arch analysis method incorporated in the computer program CPILLAR was used.

During the underground visit it was not possible to examine the subsidence area as access was considered to be unsafe. SRK have therefore used the information provided by the Mine Manager and given in the Dr. Lehane’s report to develop the geotechnical model used in the back-analysis of that subsidence.

8.2 Factor of Safety and Reliability

When designing mine structures it is usual to consider the relationship between the capacity C (strength or resisting force) of a mine element (the mine element in this case being the pillar or roof beam) and the demand D (stress or disturbing force).

The factor of safety is defined as FoS = C/D and failure is assumed to occur when the FoS is less than 1, i.e. when the demand exceeds the capacity.

Reliability is defined as the probability that the factor of safety will be greater than 1 for a proscribed range of strength parameters.

The stability analyses produce a factor of safety against pillar failure and a reliability. If the variability of strength properties is constant, there is a unique relationship between the factor of safety and the reliability (Figure 8.1). The three categories of risk shown on Figure8.1 are discussed in Section 9.

Harr (1987) states that the expected long term reliability for civil engineering systems will be in the range 0.95 to 0.99. A factor of safety of 1.5 for pillar stability is used for mining purposes, but a higher factor of safety of 2.5 is required for barrier or abutment pillars, and a factor of safety of 3 is considered necessary for long-term stability. This factor of safety of 3 allows for some weathering and erosion of a pillar before its strength would be reduced to a level where crushing could occur. It will be seen from Fig.8.1 that a factor of safety of 3 is approximately equivalent to a reliability of 95% in this case, and a factor of safety of 1.5 is approximately equivalent to a reliability of 80%.

8.3 Rock Strengths Used in the Analyses

A summary of the rock strengths used for the back analysis, and derived from the rock mass classification data given in Section 7, are tabulated below. The strength range, which is used for the probability analysis, is a two standard deviation range extending from 1 standard deviation below the mean to 1 standard deviation above the mean. For both analyses the density of the gypsum and overburden was taken as 2.3 tonnes/m^3.

TABLE 8.1 : Rock Strengths Used in the Analysis

ROCK TYPE	MEAN ROCK MASS STRENGTH (MPa)	STANDARD DEVIATION (MPa)	STRENGTH RANGE (MPa)	COMMENTS
B & D Gypsum	14.0	6.0	8.0 – 20.0	Used for pillar strength
C Gypsum	8.5	4.5	4.0 – 13.0	Used for roof beam strength
Basal Mudstone	1.0	0.34	0.66 – 1.34	Pillar Foundation cohesion

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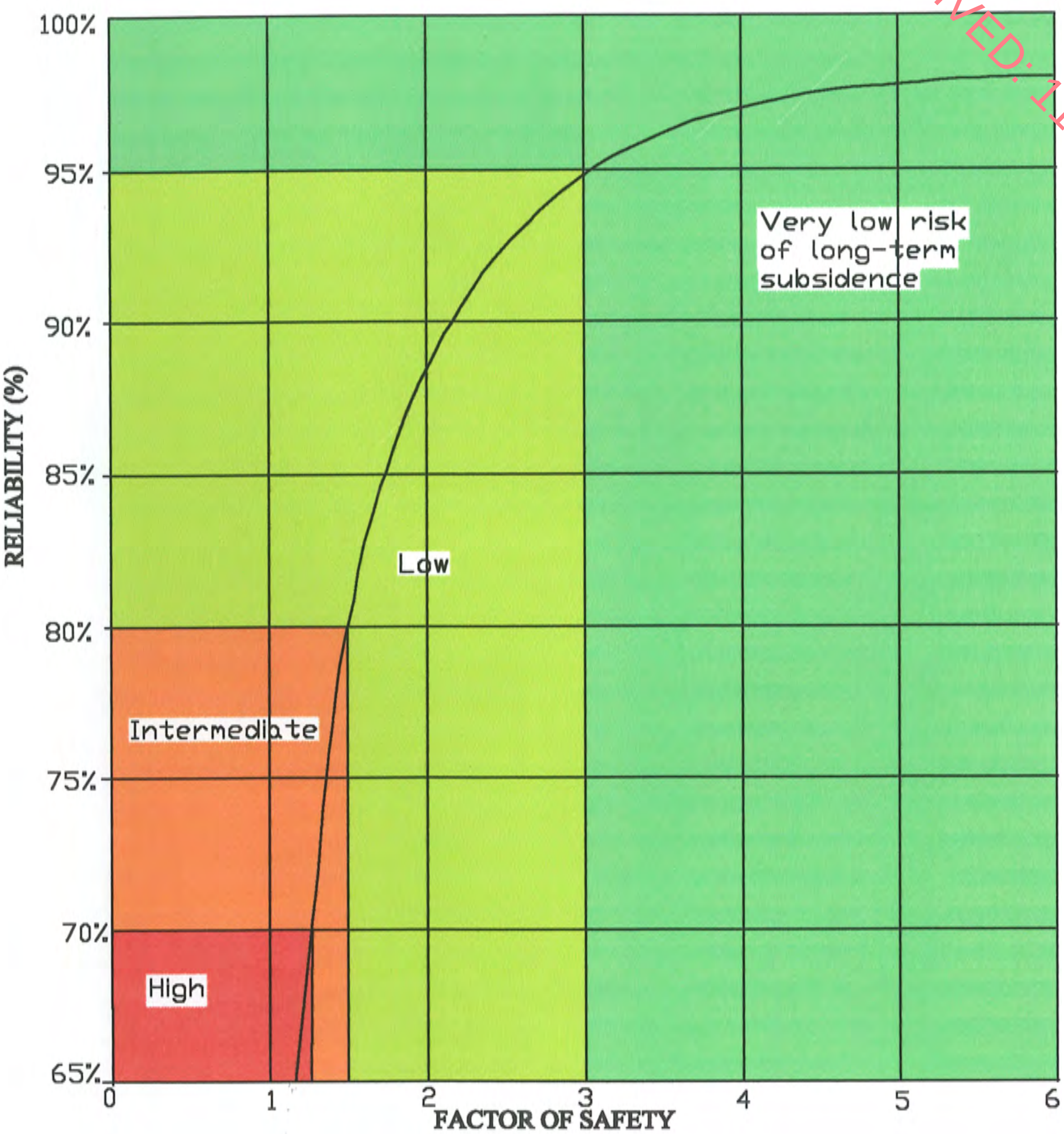

Date : 1/3/99	PROJ. No: U225	GYPSUM MINE SUBSIDENCE	
SRK	THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT	Relationship Between Reliability and Factor of Safety	Fig. 8.1

8.4 Pillar Yield and Bearing Capacity Analysis

The results of the pillar yield and bearing capacity failure analyses are summarised in Table 8.2. The detailed analysis spreadsheets are reproduced in Appendix B. The pillar identification labels are shown in a plan of the subsidence area, Figure 8.2.

Table 8.2 : Results of pillar yield and bearing capacity stability analysis

Pillar No.	Pillar Yield		Bearing Capacity	
	Mean FoS	Reliability	Mean FoS	Reliability
SINGLE PILLARS				
1	4.4	96%	1.6	88%
2	3.3	95%	1.8	91%
3	2.5	92%	1.4	81%
4	1.9	86%	1.2	66%
5	2.2	90%	1.4	81%
6	1.6	81%	1.1	56%
7	4.3	96%	2.0	94%
8	5.1	97%	2.3	96%
9	3.3	95%	1.7	90%
10	5.1	97%	2.2	95%
11	3.1	94%	1.7	89%
12	4.4	96%	2.0	93%
13	3.6	95%	2.0	94%
14	2.9	94%	1.6	87%
15	3.1	94%	1.7	90%
16	4.8	97%	2.2	95%
PILLAR GROUPS				
1 + 8	4.4	96%	1.4	80%
3 + 5	1.9	86%	1.0	45%
2 + 9	2.9	94%	1.3	76%
5 + 12	3.2	95%	1.4	81%

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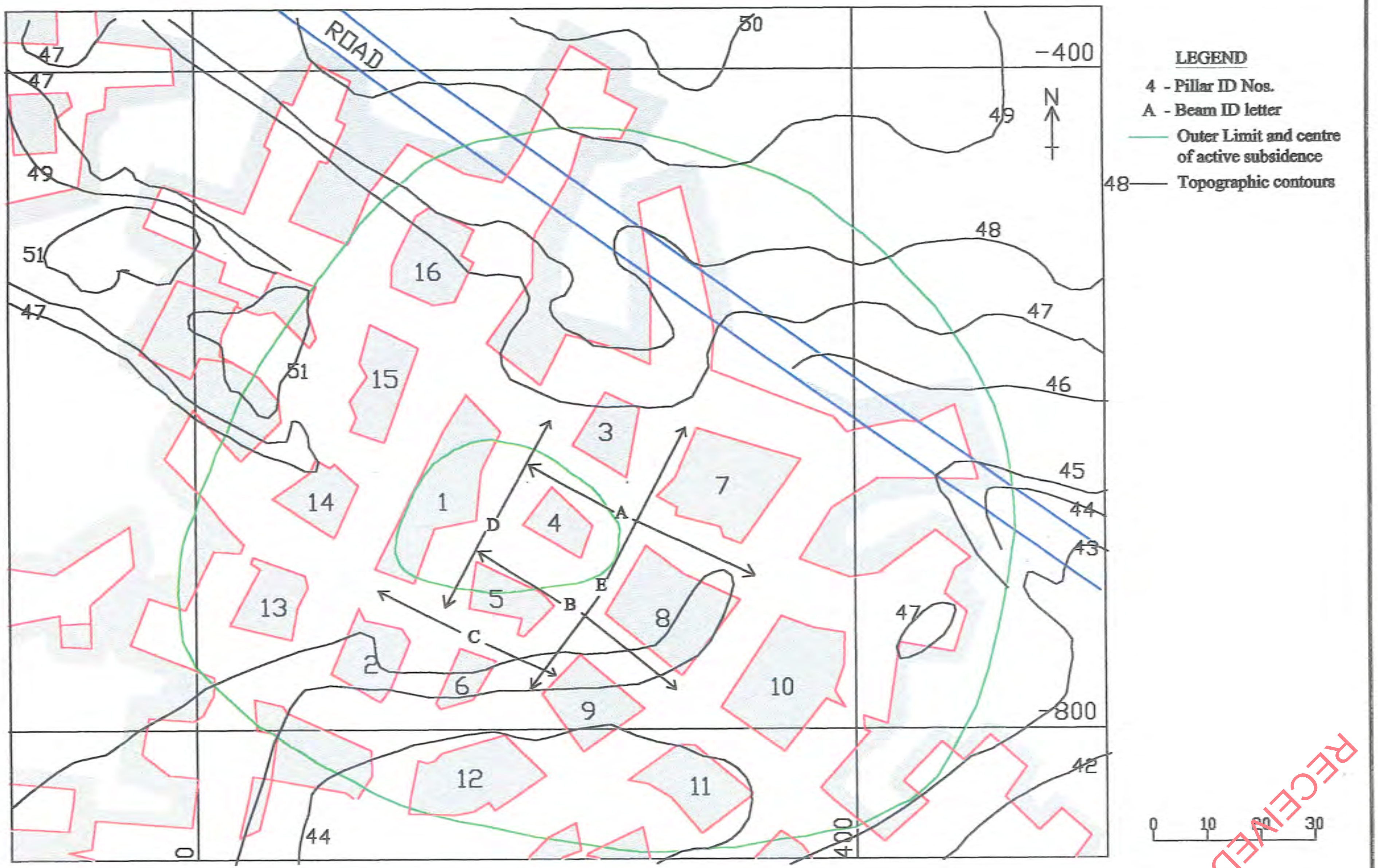


Date : 18/11/98 | PROJ. No: U1225

GYPSUM MINE SUBSIDENCE

Plan of Area of Active Subsidence

Fig.8.2

SRK

THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT

All of the pillars are stable against pillar yield. Pillar 4 and pillar 6 have the lowest factors of safety against bearing capacity failure, 1.1 and 1.0 respectively, making them the most potentially unstable. The analysis was extended to consider stability of pillar pairs surrounding the potentially unstable pillars. This analysis assumes the failure of the central pillar enclosed by the pair. Pillar 4 is surrounded by pillars 1, 8, 3 and 5. Pillar 6 is surrounded by pillars 2, 5, 9 and 12. The results of the analyses of pairs of pillars are also given in Table 8.2. The pillar combination (3 + 5) gives the lowest factor of safety for bearing capacity failure. The other pillar combinations have a high factor of safety against bearing capacity failure.

8.5 Roof Beam Stability

The rock mass strength of the C section gypsum has been used in the analysis of roof beam stability. The strength of the C section gypsum is lower than that of the B and D section materials thus the analysis provides a realistic lower bound assessment of roof stability.

A summary of the results of the CPILLAR roof beam stability analysis is given in Table 8.3. The results are presented as beam deflection, calculated as a percentage of beam thickness.

Table 8.3 : Results of Roof Beam Stability Analysis

Beam No.	Assumed Thickness (m)	Beam Deflection (% of thickness)	Probability of Failure
A	1	7.4%	Very high
	1.5	3.9%	Low
	2	1.3%	Low
	3	0.3%	Low
B	1	7.1%	Very high
	1.5	3.8%	Low
	2	1.2%	Low
	3	0.2%	Low
C	1	8.1%	Very high
	1.5	4.4%	Moderate
	2	1.4%	Low
	3	0.3%	Low
D	1	9.7%	Very high
	1.5	5.1%	Moderate
	2	1.6%	Low
	3	0.3%	Low
E	1	9.8%	Very high
	1.5	5.2%	Moderate
	2	1.6%	Low
	3	0.3%	Low

A qualitative estimate of failure probability is made as part of the programme output. In the CPILLAR program manual (Curran, et al 1996) it is stated that a beam deflection of 10% of beam thickness or greater indicates beam failure. A low probability of failure is indicated for a beam deflection of 4% or lower. This level of deflection has been taken as the upper limit for long term beam stability. The beam identification numbers are shown in Figure 8.2.

Because the actual roof beam thickness is generally only known to be at least 3m at room intersections (see Section 2), analyses have been undertaken for thicknesses from 1.5m to 3m, to examine the sensitivity of beam stability to beam thickness.

The results show that, assuming that all pillars remain intact, roof beams whose spans are constrained by the existing pillar layout below the focus of the subsidence are stable for the design minimum beam thickness of 3m. Beam thickness would have to reduce to less than 1.5m before the roof became unstable. It is inferred from these results that roof beam failure did not play a role in the initiation of subsidence. The plan distance between the pillars surrounding pillar 4 is approximately 33m by 30m. Results of a CPILLAR analysis of a 3m thick beam of these dimensions shows a deflection greater than 10% of the beam thickness. This indicates that on failure of the pillar the probability of failure of the overlying roof beam is high.

8.6 Conclusions

It is inferred from the results of this back analysis that the failure was initiated by bearing capacity failure of pillar 4. The roof beam overlying this pillar, assuming its thickness did not exceed 3m, then began failing in tension.

Simple analytical methods have been used to model the failure mechanism propounded by Dr. Lehane. The same methods of analysis and the same strength properties have been used for the prediction of subsidence in other areas of the mine. (The actual mechanism of a potential failure elsewhere will, of course, be different.)

9 RISK CRITERIA

Risk is a function of both the *probability* of failure occurring and of the consequences of such a failure, and the consequences are referred to as the *hazard*.

RISK = PROBABILITY x HAZARD

Thus, if the consequence of a pillar failure is subsidence resulting in damage to a structure (the hazard), the risk is of damage to that structure. If there is a probability of 10% that subsidence will occur, damaging a house, there is a 10% risk of damage to that house. The relationship can be expressed financially. If the estimated cost of repairs is £10 000, say, the financial risk is 10% of £10 000, or £1 000.

The probability of occurrence of subsidence can be calculated or qualitatively estimated. The potential consequences can be calculated or deduced, and these may include loss of human life, damage to human health, loss of property, loss of money and environmental degradation. The criteria for judging these hazards are not the same. The acceptable probability of occurrence of a human death, for example, will clearly be different to that for loss of property.

Various risk criteria have been proposed for assessing risk, and Table 9.1 gives those of Cole et al (1993), used for the assessment of and development of remediation strategies for subsidence above the Carrickfergus Salt Mine in Northern Ireland. They are similar to a number of criteria used internationally, by SRK and others, and are considered to be appropriate for Drumgoosat and Drumgill Mines.

Cole et al proposed indices for deriving a *relative risk* (Table 9.2). The *relative risk* $R_{R(I)}$ is defined by the following equation,

$R_{R(I)} = P[f]*I$,

where P[f] is the probability of failure, and
I is an empirical value related to the importance of land use, structures and services overlying the mine.

The relative risk removes the difficulty in quantifying hazards and, in fact, the index I is a hazard index.

TABLE 9.1: RISK CRITERIA (Cole et al, 1994)

RISK DESCRIPTION	ANNUAL RELATIVE RISK	ATTITUDE TO ANNUAL RISK
Very low	Greater than 1 in 700	Of little or no concern
Low	1 in 70 to 1 in 700	Cautious
Intermediate	1 in 20 to 1 in 70	Concerned to cautious
High	1 in 7 to 1 in 20	Concerned
Very high	1 in 2 to 1 in 7	Very concerned to concerned

TABLE 9.2: RELATIVE IMPORTANCE OF STRUCTURE, I_S

SURFACE STRUCTURES/USAGE	VALUE OF I
Public open space, farmland, tidal land.	0.3
Domestic houses (single family occupancy), secondary communications networks/roads and railways, small factories and small places of assembly.	1
Domestic multiple occupancy, places of assembly, medium to large factories and offices, main roads and railways.	3
Essential services, valuable and/or costly property.	10
Structures or services giving great danger if damaged.	30

Table 9.3 shows the relationship between the categories of risk, (very low, low, intermediate and high), used in this report, and the reliability numbers/factors of safety for Drumgoosat. These categories are in accordance with the criteria of Cole, with accepted practice in terms of the calculated reliability and factor of safety, and with SRK's assessment of the stability of the workings at Drumgoosat.

TABLE 9.3: RELATIONSHIP BETWEEN RISK, RELIABILITY AND FACTOR OF SAFETY USED IN THE SUBSIDENCE ASSESSMENT OF DRUMGOOSAT AND DRUMGILL MINES

OVERLYING STRUCTURES	PILLAR RELIABILITY	SAFETY FACTOR	RELATIVE RISK	DEGREE OF RISK	APPROPRIATE ATTITUDE TO RISK	ACTION
Farmland	80-100%	>1.5	Very low	Slight chance to unlikely	Little or no concern	Annual surface inspections
	70-80%	1.3 - 1.5	Low	Some risk	Cautious	Annual surface inspections
	<70%	<1.3	Intermediate	Some risk to risky	Cautious to concerned	Quarterly surface inspections
Roads and buildings	95-100%	>3.0	Very low	Slight chance to unlikely	Little or no concern	Quarterly surface inspections
	80-95%	1.5 - 3.0	Low	Some risk	Cautious	Monitoring of surface levels at six month intervals; quarterly surface inspections; initial underground inspection
	70-80%	1.3 - 1.5	Intermediate	Some risk to risky	Cautious to concerned	Monitoring of surface levels at quarterly intervals; quarterly surface inspections; initial underground inspection
	<70%	<1.3	High	Risky to very risky	Concerned to very concerned	Further investigation; monitoring of surface levels at quarterly intervals; quarterly surface inspections; annual underground inspections of individual key pillars

10 SUBSIDENCE PREDICTION ANALYSES (DRUMGOOSAT MINE)

In order to assess the subsidence risk above Drumgoosat Mine both historical (extrapolating from past experience at the mine) and probabilistic (using mine geometry and rock properties to predict future performance) analyses have been undertaken.

10.1 Historical Analysis

A prediction of future behaviour can be made from past performance. Drumgoosat Mine was operational from the early 1960's to the late 1980's. The total plan area of the mine is approximately 60ha. Based upon an area extraction ratio of 75% and an average pillar size of 12m by 12m then within the mine area there are approximately 1,000 pillars and 2,000 rooms. During the whole life of the mine there have been about 6 pillar failures which have given rise to surface subsidence. These failures have occurred in the area of active subsidence only, and the area of mine workings affected is about 1% of the total mine area. There have been about 10 occurrences of sinkholes propagating to surface above rooms. These have generally occurred over areas of the mine which are between 50m to 60m below surface which constitutes about 60% of the mine area. The total mine life was approximately 30 years, therefore the average age of any mine element (pillar or roof beam) is 15 years.

The risk of failure occurring anywhere on the mine during the course of one year can be calculated from the equation given below -

Annual Risk of Failure (R) = $(N_f*a)/(T_e*tp)$ (Cole et al, 1994)

where N_f = number of mine element failures over a given time period
a = fraction of the mine area within which the failures have occurred (this value is equal to 1 if failures have occurred randomly over the whole mine area)
T_e = total number of mine elements
t_p = time period, in years, over which the failures have occurred.

The annual risk of a pillar or room failure giving rise to surface subsidence can then be calculated as follows,

For pillars, R = (6*0.01)/(1,000*15) = 0.000004 or 1 in 250 000.
For rooms, R = (10*0.6)/(2,000*15) = 0.0002 or 1 in 5,000.

The risk values equate to an annual and mine life reliability of 100.00% and 99.99% for the mine pillars and an annual and mine life reliability of 99.98% and 99.40% for the mine rooms.

Taking into consideration the use of the land overlying the mine, using the appropriate I values given in Table 9.2, the annual relative risk of failure is given in Table 9.4.

TABLE 9.4: RELATIVE RISK BASED ON PREVIOUS PERFORMANCE

SURFACE USAGE	Buildings and Secondary Roads	Farmland
I VALUE	1	0.3
RISK OF PILLAR FAILURE	1 in 250,000	1 in 850,000
RISK OF ROOM FAILURE	1 in 5,000	1 in 17,000

Using the criterion proposed by Cole et al (1994) (Table 9.1) a risk or likelihood of failure of less than 1 in 700 is considered to be very low. On the basis of this analysis alone the chance of future mine failure occurring anywhere at Drumgoosat Mine which will lead to surface subsidence is estimated to be extremely low indeed and there should be very little concern.

10.2 Probabilistic Analysis

10.2.1 General

The historical analysis above assumes that the physical and mechanical conditions which have prevailed in the past will remain constant for the future life of the abandoned mine. This is not necessarily the case. Time-dependent physical effects resulting from pillar loading, beam deflection and weathering can give rise to a reduction in strength of the mine elements which may over increase the risk of failure in the long-term.

Probabilistic analyses for different failure modes have been undertaken. The mode of collapse at the area of existing subsidence is by failure of the mudstone below the pillars, which was not strong enough to support the weight applied by the pillars. The Mine Manager has stated that, to his knowledge, this was the only area where mining occurred at the base of the gypsum, because there was a rule that the floor should be underlain by a minimum thickness of three metres of gypsum. In those circumstances, the gypsum beneath the pillars provides a competent foundation with a high factor of safety, and a bearing capacity failure has a very low probability of occurrence. SRK could not independently confirm the thickness of the gypsum floor slab. This would require an extensive programme of probe drilling. In all other underground areas visited, however, there was no evidence of this type of instability, supporting the Mine Manager's statement.

The mode of potential failure elsewhere in the mine is postulated to be by failure of the pillar itself, followed by roof failure. Individual pillars have been examined for compressive failure. Where adjacent pillars have less than 95% reliability an analysis of the pillar group was carried out.

There are circumstances where roof beam failure has occurred without pillar failure (Section 4.2), and sinkholes have formed. The circumstances are of very shallow workings and poor roof conditions near the adit at Drumgill, and of doming and upward ravelling at Drumgoosat. The first mode will not occur outside the area presently affected, and the second mechanism is unpredictable, except by inspection of the individual roof beams. However, the historical analysis has provided an indication of the risk of sinkhole development occurring in the future.

10.2.2 The numerical analysis

Spreadsheets were developed to carry out the pillar yield analyses. Probabilistic analyses were undertaken using the Point Estimate Method (PEM) according to the following methodology.

The minimum and maximum rock mass strength values quoted in Table 8.1 were taken to lie one standard deviation either side of the mean strength value.

Factor of safety calculations were made for each strength value, and parameters defining a normal probability distribution of factor of safety were estimated for each pillar using the PEM. The relationship was interrogated to establish the probability of failure i.e., the probability that the factor of safety will be less than unity.

For those pillars with a long term reliability of less than 95%, which corresponds to an annual relative risk of failure of greater than 1 in 700, stability analyses of the unsupported beam overlying the failed pillar have been undertaken to give some indication of the possibility of failure propagating to surface.

10.3 Risk Assessment

The numerical probabilities of failure and reliabilities are derived using standard and accepted methods of analysis, but they are based on estimates of the properties of the rock mass using limited data. To date, as far as SRK are aware, pillar failure has only occurred in the area of active subsidence. Pillar reliability elsewhere on the mine, over a period probably exceeding 20 years, has been 100%. The reliability numbers generated are therefore indicators, rather than absolute values. A reliability of 95% does not imply a 1 in 20 probability of failure, but it does indicate a boundary selected as an indicator of future long-term stability. In practical terms, any time-dependent reduction in strength would lead to instability of these particular pillars before any others. Based on past mine performance it is estimated that long-term defines an abandoned mine life of at least another 50 years.

The criteria of Table 9.3 have been used to establish the degree of risk and the appropriate action recommended to be taken for each risk level. The table gives meaning to the numerical and qualitative descriptions of risk in terms of the attitude and response which is considered to be appropriate.

10.4 Dimensions and rock properties used in the analysis

10.4.1 Rock Properties

Rock properties used in the analyses (Table 8.1) are taken from the available information, from the inspections and mapping on site, and from the results of the back-analyses of the existing subsidence.

10.4.2 Mine Dimensions

Pillar and room sizes, underground geometry and mining depth have been taken from the mine plans. Checks of the dimensions of two pillars underground indicated that their dimensions on the mine plan are correct. The height of mining has been taken to be six metres.

10.4.3 Beam Thickness

The position of the mine within the gypsum is known only at a few locations. The mine manager has stated that the mine design required the thickness of gypsum above the roof and below the floor to be a minimum of three metres. Based on this requirement, and on the known strength of the gypsum, it has been assumed that apart from in the area of existing subsidence there is at least three metres of gypsum below the pillars, and that there is a very low probability of a bearing capacity failure beneath the pillars in those circumstances. For the analyses of roof beam stability, it has been assumed that the beam thickness is three metres. It is considered that this is a conservative assumption, because the thickness of gypsum above the roof is generally greater than three metres. There have been occasions, however, where the beam thickness was less than three metres, usually where doming occurred in the roof. It is possible that, where the thinner horizon of D Section gypsum has been mined, the roof beam can be locally thinner that 3m.

Sensitivity analyses using the program CPILLAR have been carried out for varying beam thickness and beam length using C Section gypsum strength parameters for a standard room width of 10m. A typical overburden thickness of 50m has been used. The results of the analysis are illustrated in Figure 10.1. This is a graph of roof beam thickness plotted against beam deflection, which is expressed as a percentage of beam thickness. Individual data points have been plotted on the graph. The data points have been enclosed by lower and upper bound curves representing a 10m by 10m and a 10m by 40m beam, respectively. 10m by 10m represents the normal room size, and 10m by 40m beam size represents a maximum room dimension. The sensitivity analysis shows that for long-term stability the roof beam should be at least 1.5m thick for the maximum room dimension reducing to 1.2m thick for a 10m by 10m beam. The minimum thickness of the combined E, F and G section gypsum members is at least 3m. Even if the roof of the mining horizon was at the top of the D section gypsum it is therefore considered unlikely that the roof beam will be less than 1.2m thick.

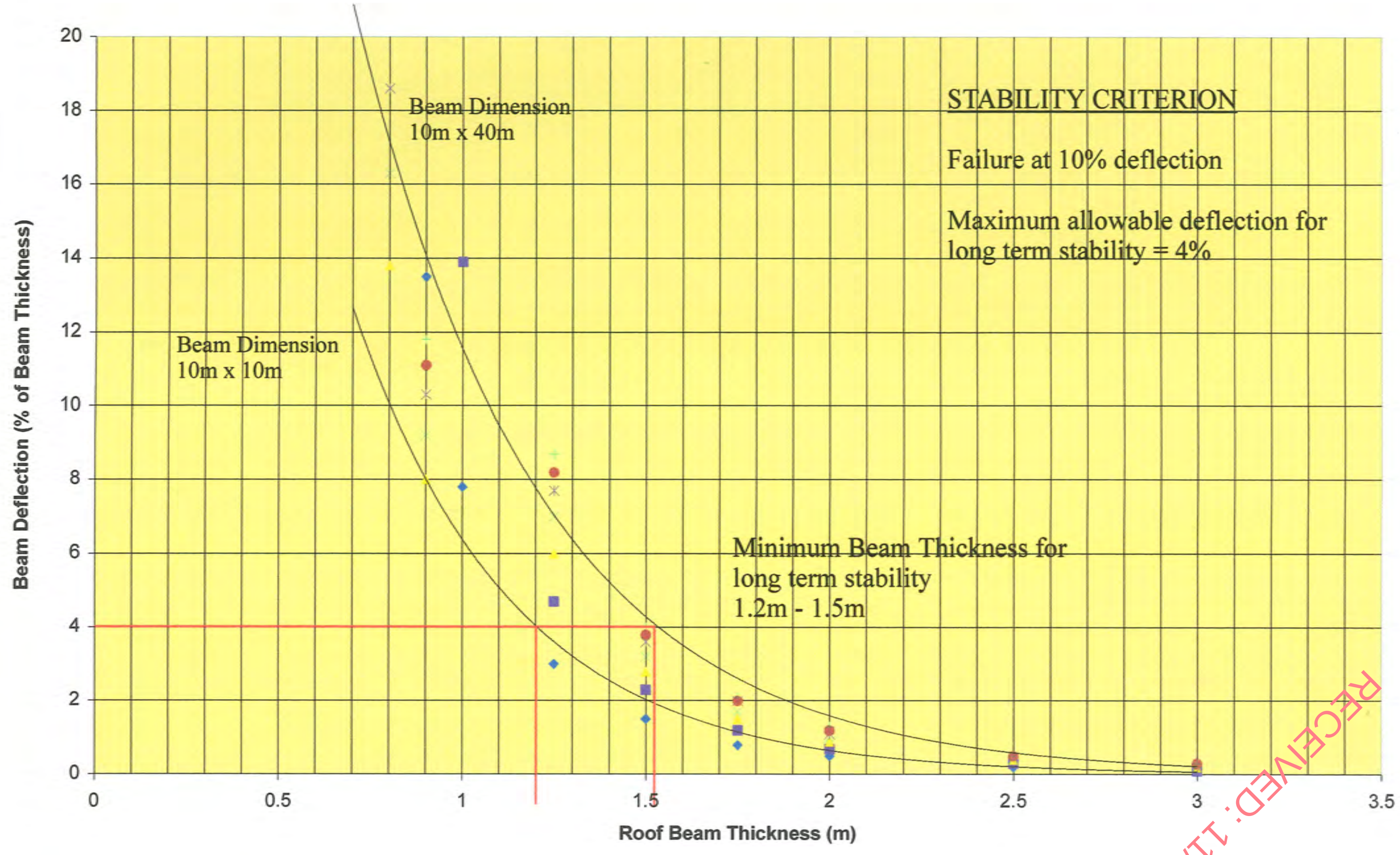
10.5 Areas for which analyses were carried out

Subsidence prediction has been restricted to those mine areas lying below land not owned by Gypsum Industries Ltd. These areas are –

a) land containing the R179 main road,
b) land containing the community centre and sports field,
c) two areas along the road leading to Drumgoosat Village, adjacent to the eastern limit of the mine,
d) the strip of land running north-west south-east through the northern section of the mine,
e) a small area of land at the south western mine limit.

The approximate location of these areas is illustrated in Figure 5.1. The location identifiers on the plan are the report section numbers in which the results of the subsidence prediction analyses are presented.

The stability of each of the areas has been analysed separately.



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Sensitivity of Roof Beam Stability to Thickness

Figure 10.1

10.6 The Main R179 Road

10.6.1 The extent of undermining

The location of this area is shown on Figure 5.1. A plan of the road showing the underlying mine layout is in Figure 10.2. Also shown in the Figure is a cross section along the road centre line with the position of the rooms and a schematic geological interpretation.

10.6.2 The results of the analyses

A zone extending 50m either side of the road was analysed. It is considered that any failures occurring beyond this distance will have a negligible effect on the road because of the presence of the mine abutment to the east of the road and large pillars to the west. This zone contained 28 discrete pillars. Pillars with large cross-sectional areas were not analysed. The results of the pillar yield stability analyses are tabulated below.

Ten pillars have a reliability of less than 95% with a ***low*** risk of failure. One pillar, number 21, has a ***high*** risk of failure by virtue of its low cross sectional area. It is however located adjacent to unmined ground and the risk of pillar failure leading to surface subsidence is therefore considered to be low. The roof beams overlying five of these pillars have medium reliability. All the other pillars analysed have an appropriate level of reliability for long term stability. Of the ten pillars a group of four (pillars 8, 23, 24, 25) lie adjacent to each other and can be analysed as pillar pairs and as groups of three pillars. Pillars 1 and 2 are adjacent to each other and can be analysed as a pillar pair. Table 10.1 shows the results of the pillar group analyses. All the pillar groups analysed have a reliability of less than 95%. The risk of failure for these groups is categorised as ***low***.

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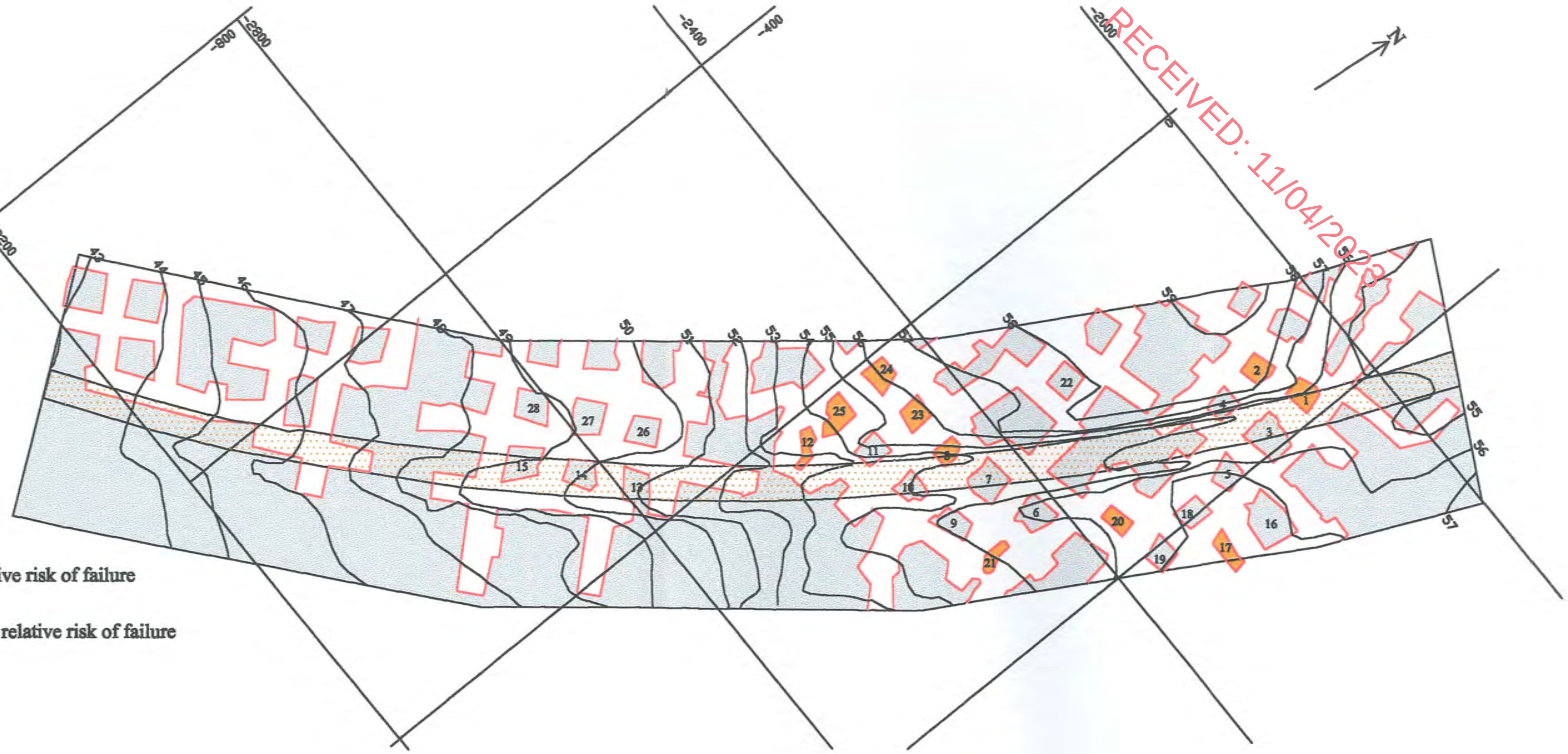


LEGEND

5 - Pillar identification numbers

Individual pillars with a low relative risk of failure

Individual pillars with a very low relative risk of failure

0 50m

Glacial till

Upper gypsum

Middle Mudstone

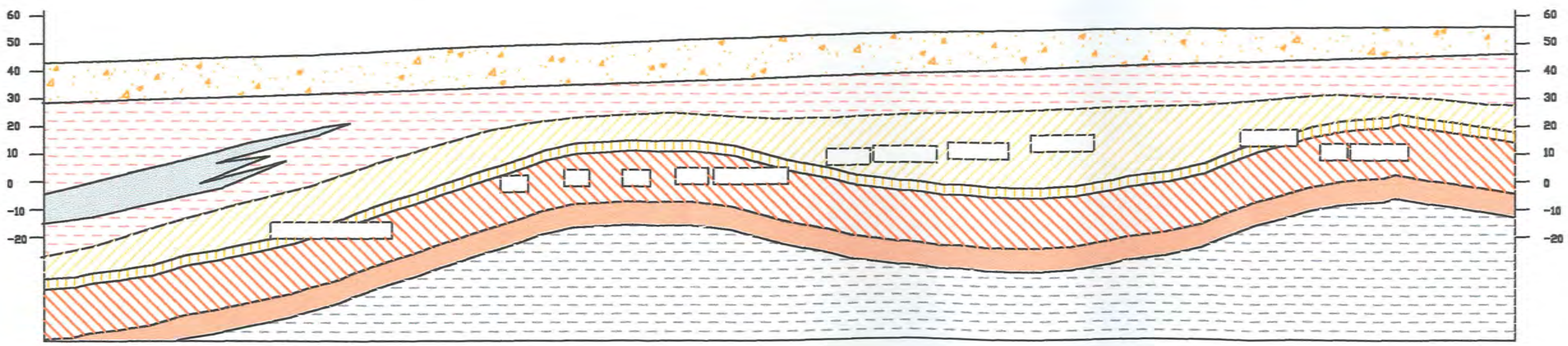
D,E,F,G section gypsum

C section gypsum

B section gypsum

A section gypsum

Basal Mudstone



(Note: Geology is schematic only)

Date : 18/11/98	PROJ. No: U1225	GYPSUM MINE SUBSIDENCE	
SRK	THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT	Plan and Section Through R179 Road Showing Mine Layout And Potential Long Term Unstable Areas	Fig. 10.2

Table 10.1 : R179 Road–Results of Pillar Yield and Roof Beam Stability Analyses

(a) Single pillars and beams

	Pillar Yield			Beam Analysis	
Pillar No.	Mean FoS	Reliability R[f]	Relative Risk of Failure	Beam Deflection (% of thickness)	Reliability R[f]
SINGLE PILLARS					
1	3.0	94%	Low	3%	High
2	2.6	92%	Low	5%	Medium
3	5.5	97%	Very Low	0%	High
4	3.6	95%	Very Low	0%	High
5	3.6	95%	Very Low	0%	High
6	4.7	97%	Very Low	0%	High
7	7.5	98%	Very Low	0%	High
8	2.1	89%	Low	3%	High
9	3.7	96%	Very Low	0%	High
10	3.2	95%	Very Low	0%	High
11	3.2	95%	Very Low	0%	High
12	1.8	85%	Low	2%	High
13	3.4	95%	Very Low	0%	High
14	3.2	95%	Very Low	0%	High
15	3.8	96%	Very Low	0%	High
16	6.9	98%	Very Low	0%	High
17	2.3	91%	Low	3%	High
18	4.8	97%	Very Low	0%	High
19	4.1	96%	Very Low	0%	High
20	2.7	93%	Low	4%	High
21	1.1	60%	High	5%	Medium
22	3.4	95%	Very Low	0%	High
23	2.1	89%	Low	7%	Medium
24	2.6	92%	Low	8%	Medium
25	2.6	93%	Low	6%	Medium
26	3.6	95%	Very Low	0%	High
27	4.3	96%	Very Low	0%	High
28	3.7	96%	Very Low	0%	High

(b) Pillar and beam groups

	Pillar Yield			Beam Analysis	
Pillar No.	Mean FoS	Reliability R[f]	Relative Risk of Failure	Beam Deflection (% of thickness)	Reliability R[f]
1 + 2	3.2	94%	Low	6%	Medium
8 + 23	2.2	90%	Low	12%	Low
23 + 24	2.2	90%	Low	5%	Medium
24 + 25	2.3	90%	Low	12%	Low
12 + 25	1.8	85%	Low	13%	Low
24+23+8	1.9	87%	Low	8%	Medium
24+25+12	1.9	87%	Low	9%	Medium
23+24+25	1.8	85%	Low	8%	Medium

NOTE: Grey shading identivies those pillars having a failure risk greater than *very low*

10.6.3 The significance of the results

No indications of failure were noted during the underground inspections. The consequence of failure of one or more of the ten pillars with a reliability of less than 95% would be the failure of the overlying roof beams, and the development of surface settlement. The settlement would probably be more limited than that experienced on the road to Drumgoosat village, because the small pillars under the R179 road are adjacent to some very large pillars. Should there be more than one pillar failure of the second group of pillars there will be a possibility that (delection) subsidence will occur close to the road. It is unlikely that there would be sudden rapid movements, but rather the gradual development of settlement similar to that on the road to Drumgoosat village. The risk of road subsidence and the risk to human life is considered ***very low,*** implying "some risk".

10.6.4 Proposed action

It is proposed that monitoring of surface levels along the undermined section of the road be carried out at six month intervals, together with quarterly surface inspections.

It is also proposed that a detailed geotechnical inspection of the smaller pillars in the vicinity of the undermined section of the road should be carried out. This inspection will allow a more precise assessment of the risk, and provide baseline information against which the future condition of the pillars could be judged if necessary.

10.7 The Community Centre

10.7.1 The extent of undermining

Thirty three pillars lie to the north, east and west and within an 80m radius of the Community Centre. To the south the centre is protected by a large pillar. The pillars analysed are shown in Figure 10.3. The roof of the mine workings lies approximately 60m below the ground surface. The Community Centre building overlies a large pillar having a surface area of over 1,000m^2. Although this pillar does not underlie the entire building, its effect is to ensure that the workings directly below the Centre will remain stable.

10.7.2 The results of the analyses

The results of the pillar stability analysis are tabulated below. Pillar 4, which has an ***intermediate*** risk of failure, and pillar 5, which has a ***low*** risk of failure, lie below the car park about 25m to the north-east of the Community Centre. The risk of failure of the two pillars combined is ***low***. The beam overlying the pillars has a medium long-term reliability.

Figure 10.3 : Community Centre

Table 10.3 : Community Centre-Results of Pillar Yield and Roof Beam Stability Analyses

	Pillar Yield			Beam Analysis	
Pillar No.	Mean FoS	Reliability R[f]	Relative Risk of Failure	Beam Deflection (% of thickness)	Reliability R[f]
SINGLE PILLARS					
1	4.7	97%	Very Low	0%	High
2	4.0	96%	Very Low	0%	High
3	3.4	95%	Very Low	0%	High
4	1.5	78%	Intermediate	3%	High
5	2.8	93%	Low	0%	High
6	3.9	96%	Very Low	0%	High
7	8.2	98%	Very Low	0%	High
8	4.1	96%	Very Low	0%	High
9	4.9	97%	Very Low	0%	High
10	2.5	92%	Low	5%	Medium
11	5.0	97%	Very Low	0%	Medium
12	3.4	95%	Very Low	0%	Medium
13	2.5	92%	Low	5%	Medium
14	4.4	96%	Very Low	0%	High
15	3.5	95%	Very Low	0%	High
16	1.5	79%	Intermediate	4%	Medium
17	2.5	92%	Low	4%	Medium
18	2.5	92%	Low	6%	Medium
19	2.4	91%	Low	5%	Medium
20	2.2	90%	Low	4%	Medium
21	1.9	86%	Low	4%	Medium
22	3.8	96%	Very Low	0%	High
23	2.1	89%	Low	4%	High
24	2.7	93%	Low	6%	Medium
25	3.7	96%	Very Low	0%	High
26	3.3	95%	Very Low	0%	High
27	2.9	94%	Low	6%	Medium
28	2.0	88%	Low	4%	High
29	3.4	95%	Very Low	0%	High
30	2.3	91%	Low	4%	High
31	4.0	96%	Very Low	0%	High
32	2.4	91%	Low	5%	Medium
33	2.9	94%	Low	8%	Medium
PILLAR GROUPS					
4 + 5	2.2	90%	Low	8%	Medium
23+24	2.8	93%	Low	8%	Medium
27+28	2.8	93%	Low	12%	Low
32+33	2.7	93%	Low	9%	Medium
27+28+32+33	2.9	94%	Low	12%	Low
10+13	2.2	90%	Low	9%	Medium
13+16	1.6	81%	Low	8%	Medium
16+19	1.5	79%	Intermediate	7%	Medium
17+20	1.9	86%	Low	8%	Medium
16+17 +19+20	1.9	87%	Low	11%	Low

NOTE: Grey shading identivies those pillars having a failure risk greater than *very low*

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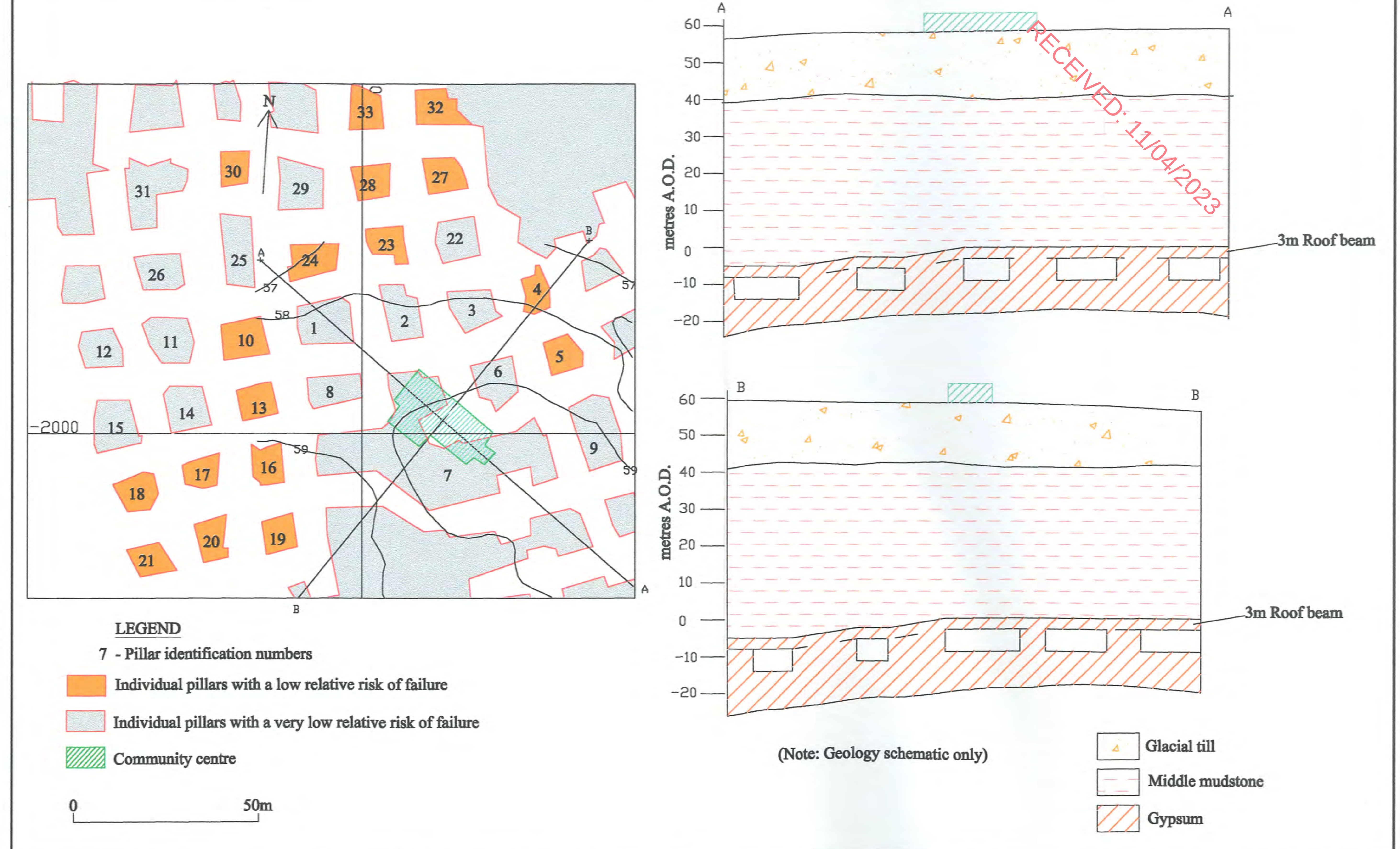

Date : 18/11/98	PROJ. No: U1225	GYPSUM MINE SUBSIDENCE	
SRK	THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT	Plan And Cross Sections Through The Community Centre Showing Mine Layout And Potential Long Term Unstable Areas	Fig. 10.3

There are also a number of pillars to the west of the Community Centre, on land owned by Gypsum Industries, whose reliability is less than 95%. Most of these have a l***ow*** risk of failure but pillar 16 has an ***intermediate*** risk of failure. The long-term reliability of the pillar groups analysed is less than 95%. Roof beams have medium reliability.

10.7.3 The significance of the results

A number of individual pillars and pillar groups surrounding the Community Centre have a ***low*** risk of failure, indicating that there will be "some risk" according to the criteria of Table 9.3, and a cautious attitude is therefore appropriate. The Community Centre building stands above a very large pillar, and the risk of sudden large movements affecting this structure is considered ***low***. The risk to human life is considered low.

10.7.4 Proposed action

It is proposed that a quarterly visual inspection of the Community Centre building and the land extending to a radius of 80m around the Community Centre are carried out. Level monitoring points should be established around the perimeter of the building, to be monitored at six month intervals.

It is also proposed that a detailed inspection of the smaller pillars in the vicinity of the Community Centre should be carried out. This inspection will allow a more precise assessment of the risk, and provide baseline information against which the future condition of the pillars could be judged if necessary.

10.8 Road to Drumgoosat Village (Area 1)

10.8.1 The extent of undermining

Details of the mine layout in this area are shown on Figure 10.4. The area is protected on the north, east and west sides by unmined ground and on the south side by larger pillars. Eighteen pillars which underlie the road and surrounding area have been analysed for stability.

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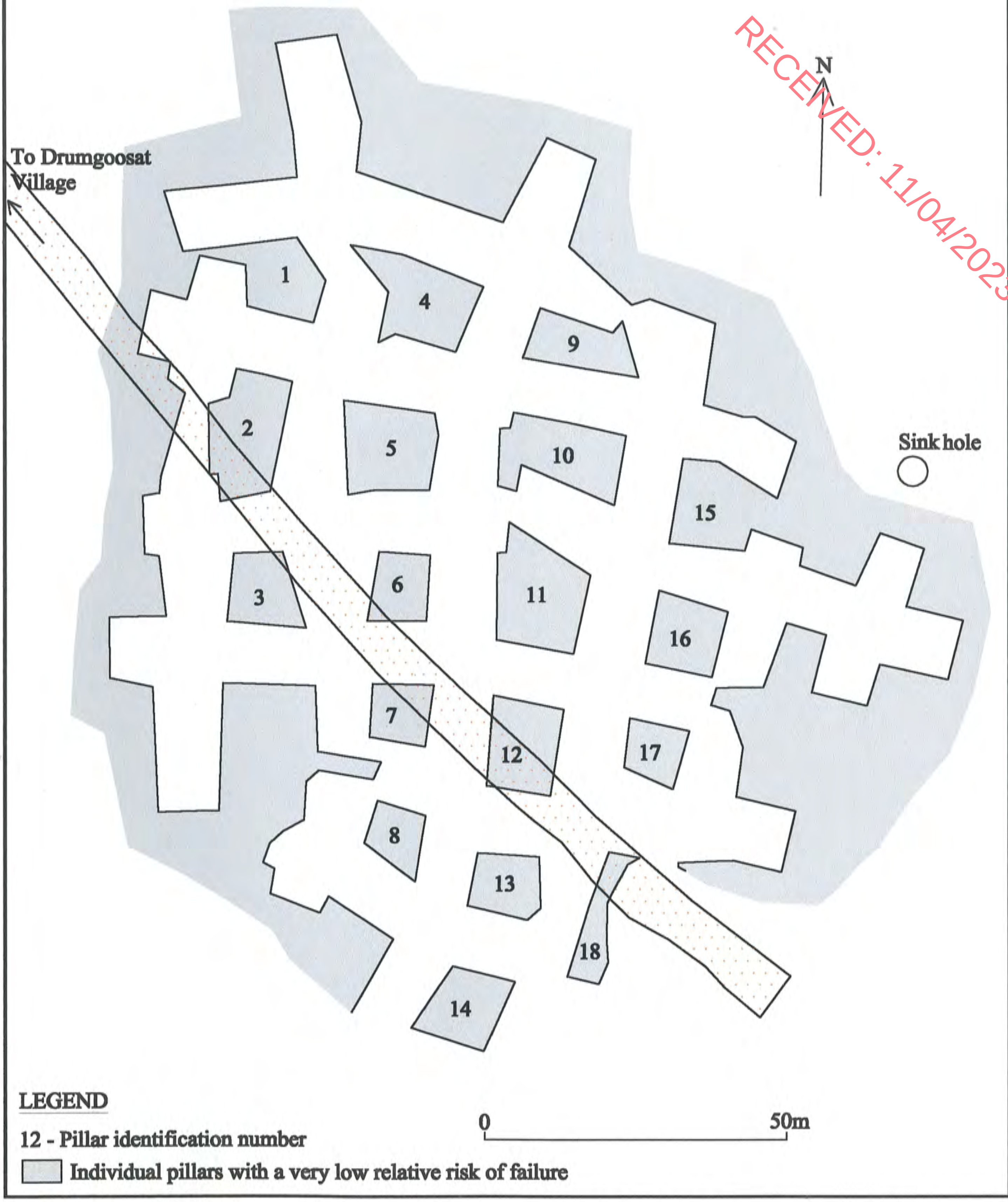

Date : 17/11/98	PROJ. No: U1225	GYPSUM MINE SUBSIDENCE	
SRK	THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT 1998	Mine Layout Below Drumgoosat Village Access Road (Area 1)	Fig. 10.4

10.8.2 The results of the analyses

The results of the analysis are given in Table 10.4.

Table 10.4 : Road to Drumgoosat Village (Area 1)
Results of Pillar Stability and Roof Beam Analyses

	Pillar Yield			Beam Analysis	
Pillar No.	Mean FoS	Reliability R[f]	Relative Risk of Failure	Beam Deflection (% of thickness)	Reliability
1	6.5	98%	Very Low	0%	High
2	7.6	98%	Very Low	0%	High
3	4.4	96%	Very Low	0%	High
4	5.6	97%	Very Low	0%	High
5	7.3	98%	Very Low	0%	High
6	4.1	96%	Very Low	0%	High
7	4.2	96%	Very Low	0%	High
8	3.2	95%	Very Low	0%	High
9	3.3	95%	Very Low	0%	High
10	20.1	99%	Very Low	0%	High
11	7.5	98%	Very Low	0%	High
12	6.0	97%	Very Low	0%	High
13	5.5	97%	Very Low	0%	High
14	6.4	98%	Very Low	0%	High
15	8.5	98%	Very Low	0%	High
16	6.6	98%	Very Low	0%	High
17	4.5	97%	Very Low	0%	High
18	3.7	96%	Very Low	0%	High

All of the pillars in this area satisfy the reliability criterion for long term stability and no long-term subsidence instability is predicted. It was considered unnecessary to carry out roof beam stability analyses for this area.

10.8.3 The significance of the results

The results indicate that the pillars should be stable in the long-term.

10.8.4 Proposed action

The potential relative risk of mine failure at this location may be considered ***very low***, and the proposed action comprises annual surface inspections only.

10.9 Road to Drumgoosat Village (Area 2)

10.9.1 Extent of undermining

Details of the mining layout in this area are shown on Figure 10.5. The area is protected on the north, east and west sides by unmined ground, and on the south side by larger pillars. Twelve pillars which underlie the road and surrounding area have been analysed for stability.

10.9.2 Results of the analyses

The results of the analysis are given in Table 10.5. All but one of the pillars have a high reliability and ***very low*** risk of failure.

TABLE 10.5 : Road to Drumgoosat Village (Area 2) – Results of Pillar Yield and Roof Beam Stability Analyses

	Pillar Yield			Beam Analysis	
Pillar No.	Mean FoS	Reliability R[f]	Relative Risk of Failure	Beam Deflection (% of thickness)	Reliability
1	4.0	96%	Very Low	0%	High
2	4.8	97%	Very Low	0%	High
3	10.4	98%	Very Low	0%	High
4	10.4	98%	Very Low	0%	High
5	8.0	98%	Very Low	0%	High
6	4.8	97%	Very Low	0%	High
7	10.2	98%	Very Low	0%	High
8	6.1	97%	Very Low	0%	High
9	8.1	98%	Very Low	0%	High
10	3.0	94%	Low	1%	High
11	5.4	97%	Very Low	0%	High
12	4.3	96%	Very Low	0%	High

NOTE: Grey shading identivies those pillars having a failure risk greater than *very low*

10.9.3 Significance of the results

The probability of long-term failure is considered to be ***very low***.

10.9.4 Proposed action

As for Section 1 of this road, the proposed action comprises annual surface inspections only.

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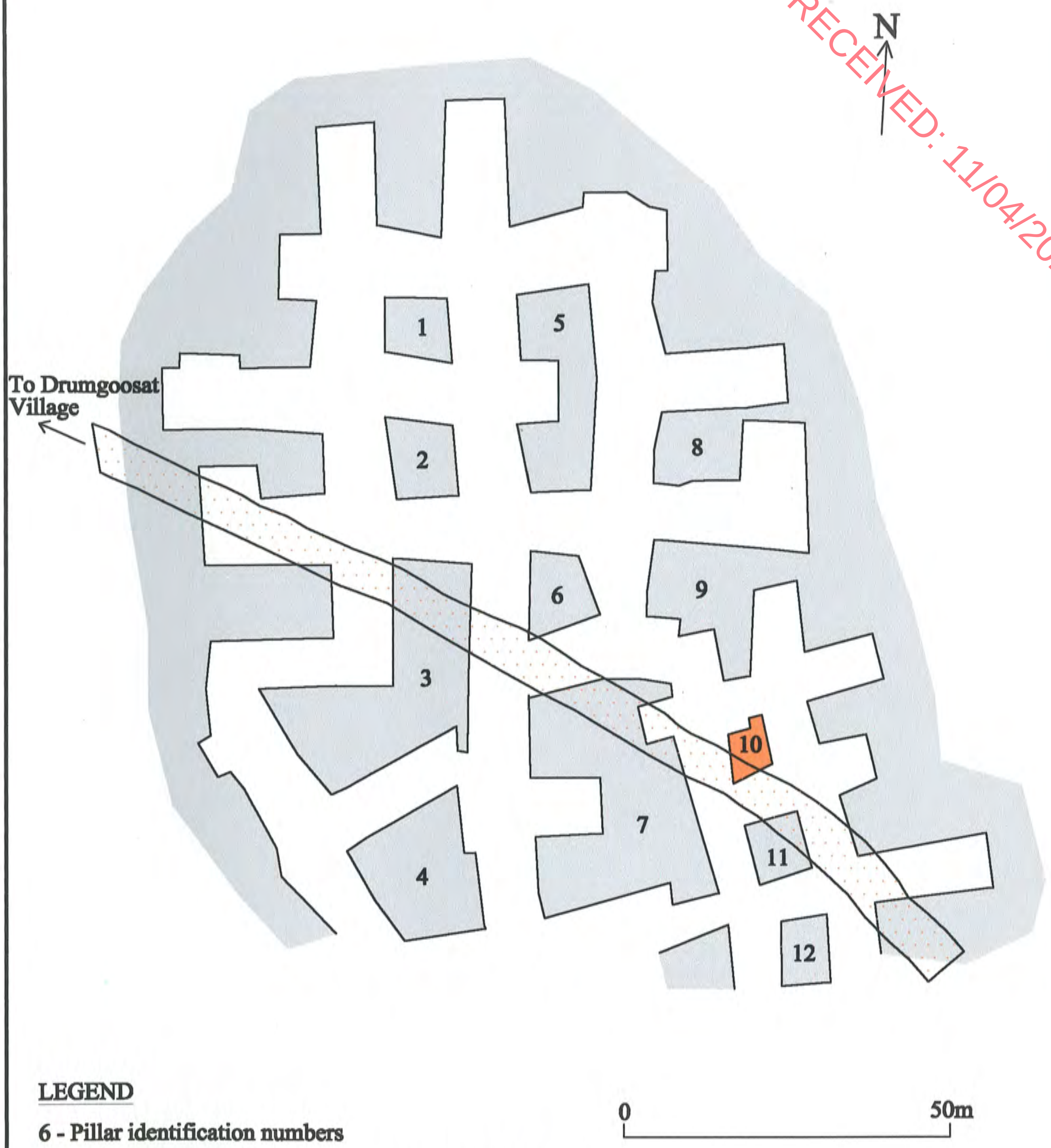

LEGEND

6 - Pillar identification numbers

Individual pillars with a low relative risk of failure

Individual pillars with a very low relative risk of failure

Date :17/11/98	PROJ. No: U1225	GYPSUM MINE SUBSIDENCE	
SRK	THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT	Mine Layout Below Drumgoosat Village Access Road (Area 2)	Fig. 10.5

10.10 Area North West of the Community Centre

10.10.1 Extent of undermining

This area consists of a strip of farmland extending from immediately north-west of the Community Centre to the north-western limit of the underground mine, Figure 10.6. Mining beneath most of this area has been carried out at a depth of 55m below surface. Towards the north-western limit of the mine, the orebody dips to the west at about 30°. In this area the mining depth increases to about 105m.

10.10.2 Results of the analyses

Results of the subsidence predictions are given in Table 10.6. Using the criteria given in Table 9.3 all but one of the pillars analysed lie in the ***very low*** risk category. (Note that because of the land use above the mine in this area the very low risk category encompasses pillars with reliability values in the range 80% to 100%.)

10.10.3 Significance of the results

The risk of subsidence of the overlying farmland is determined to be ***very low***.

10.10.4 Proposed action

The proposed action is annual visual surface inspections.

Table 10.6a) : Area North West of Community Centre – Results of Pillar Yield and Roof Beam Stability Analyses

	Pillar Yield			Beam Analysis	
Pillar No.	Mean FoS	Reliability R[f]	Relative Risk of Failure	Beam Deflection (% of thickness)	Reliability R[f]
1	4.4	96%	Very Low	0%	High
2	2.0	88%	Very Low	0%	High
3	3.8	96%	Very Low	0%	High
4	2.9	94%	Very Low	0%	High
5	2.6	92%	Very Low	0%	High
6	2.4	91%	Very Low	0%	High
7	3.3	95%	Very Low	0%	High
8	2.2	90%	Very Low	0%	High
9	3.8	96%	Very Low	0%	High
10	1.6	81%	Very Low	0%	High
11	1.9	86%	Very Low	0%	High
12	2.3	91%	Very Low	0%	High
13	1.8	85%	Very Low	0%	High
14	4.1	96%	Very Low	0%	High
15	2.0	88%	Very Low	0%	High
16	2.9	94%	Very Low	0%	High
17	2.9	94%	Very Low	0%	High
18	1.3	69%	Intermediate	4%	High
19	3.5	95%	Very Low	0%	High
20	3.3	95%	Very Low	0%	High
21	3.2	95%	Very Low	0%	High
22	4.6	97%	Very Low	0%	High
23	3.3	95%	Very Low	0%	High
24	3.9	96%	Very Low	0%	High
25	4.1	96%	Very Low	0%	High
26	5.5	97%	Very Low	0%	High
27	2.8	93%	Very Low	0%	High
28	3.7	96%	Very Low	0%	High
29	4.2	96%	Very Low	0%	High
30	6.6	98%	Very Low	0%	High
31	3.4	95%	Very Low	0%	High
32	2.8	93%	Very Low	0%	High
33	2.4	91%	Very Low	0%	High
34	3.5	95%	Very Low	0%	High
35	2.7	93%	Very Low	0%	High
36	2.9	94%	Very Low	0%	High
37	4.4	96%	Very Low	0%	High
38	4.6	97%	Very Low	0%	High
39	4.3	96%	Very Low	0%	High
40	5.8	97%	Very Low	0%	High

NOTE: Grey shading identivies those pillars having a failure risk greater than *very low*

Table 10.6b) : Area North West of Community Centre – Results of Pillar Yield and Roof Beam Stability Analyses

	Pillar Yield			Beam Analysis	
Pillar No.	Mean FoS	Reliability R[f]	Relative Risk of Failure	Beam Deflection (% of thickness)	Reliability R[f]
41	5.4	97%	Very Low	0%	High
42	7.6	98%	Very Low	0%	High
43	5.9	97%	Very Low	0%	High
44	5.0	97%	Very Low	0%	High
45	4.5	97%	Very Low	0%	High
46	4.8	97%	Very Low	0%	High
47	5.0	97%	Very Low	0%	High
48	4.1	96%	Very Low	0%	High
49	6.7	98%	Very Low	0%	High
50	5.0	97%	Very Low	0%	High
51	3.7	96%	Very Low	0%	High
52	1.7	82%	Very Low	0%	High
53	4.4	96%	Very Low	0%	High
54	4.7	97%	Very Low	0%	High
55	4.2	96%	Very Low	0%	High
56	3.5	95%	Very Low	0%	High
57	2.6	93%	Very Low	0%	High
58	5.8	97%	Very Low	0%	High
59	6.1	97%	Very Low	0%	High
60	6.0	97%	Very Low	0%	High
61	6.2	97%	Very Low	0%	High
62	4.2	96%	Very Low	0%	High
63	4.7	97%	Very Low	0%	High
64	3.9	96%	Very Low	0%	High
65	2.7	93%	Very Low	0%	High
66	2.1	89%	Very Low	0%	High
67	3.8	96%	Very Low	0%	High
68	2.9	94%	Very Low	0%	High
69	1.9	87%	Very Low	0%	High
70	2.5	92%	Very Low	0%	High
71	2.8	93%	Very Low	0%	High
72	3.6	95%	Very Low	0%	High
73	1.6	82%	Very Low	0%	High
74	2.1	88%	Very Low	0%	High
75	1.9	86%	Very Low	0%	High
76	2.1	89%	Very Low	0%	High
77	2.0	88%	Very Low	0%	High
78	2.9	94%	Very Low	0%	High
79	1.7	83%	Very Low	0%	High
80	1.7	83%	Very Low	0%	High
81	1.7	83%	Very Low	0%	High

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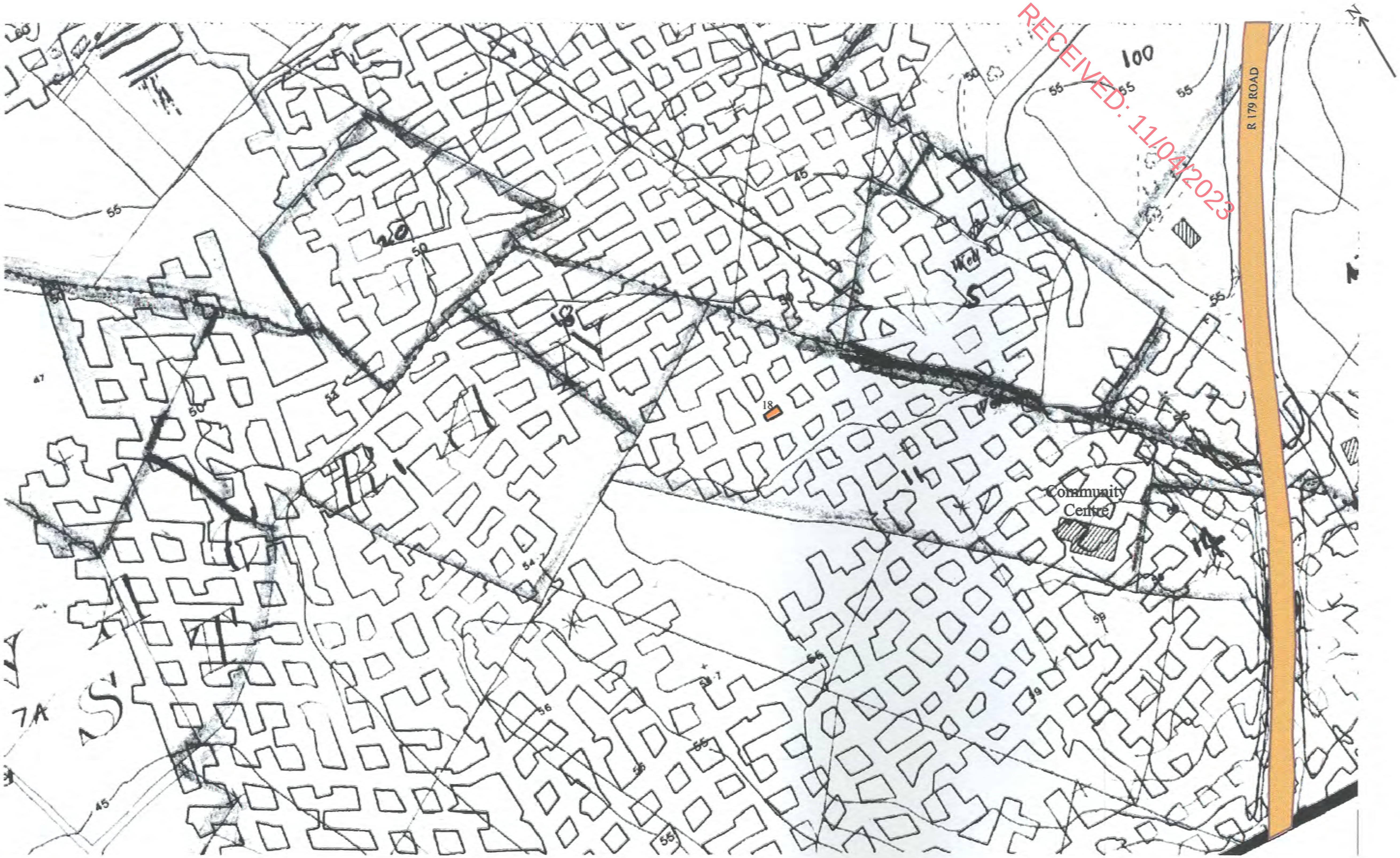

Individual pillars with a low relative risk of failure

Individual pillars with a very low relative risk of failure

DATE: 25/11/98	PROJ. No: U1225	GYPSUM MINE SUBSIDENCE	
SRK	THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT	Mine Layout in Area N.W. of Community Centre Showing Potential Long Term Unstable Areas	Figure 10.6

10.11 South-West Area

10.11.1 Extent of undermining

This small area is located on the south-western limit of the mine, Figure 10.7. The average mining depth is 105m.

10.11.2 Results of analyses

The results of the analysis are tabulated below.

Table 10.7 : South West Area of Mine – Results of Pillar Yield and Roof Beam Stability Analyses

	Pillar Yield			Beam Analysis	
Pillar No.	Mean FoS	Reliability R[f]	Relative Risk of Failure	Beam Deflection (% of thickness)	Reliability
1	2.2	90%	Very Low	0%	High
2	1.6	82%	Very Low	0%	High
3	2.4	91%	Very Low	0%	High
4	1.5	79%	Low	6%	Medium
5	1.8	84%	Very Low	0%	High
6	1.5	79%	Low	5%	Medium
7	1.7	82%	Very Low	0%	High

NOTE: Grey shading identivies those pillars having a failure risk greater than *very low*

10.11.3 Significance of the results

Apart from pillars 4 and 6 whose risk of failure is ***low*** all other pillars have a ***very low*** risk of failure. Because the land above these pillars is agricultural a very low failure risk is defined by pillars with a reliability greater than 80%.

10.11.4 Proposed action

The proposed action is annual surface visual inspections.

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Date: 25/11/98	PROJ. No: U1225	GYPSUM MINE SUBSIDENCE	
SRK THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT		Mine Layout in SW Corner of Mine Showing Potential Long Term Unstable Areas	Fig. 10.7

11 SUBSIDENCE POTENTIAL AT DRUMGILL MINE

Figure 11.1 is a plan of the Drumgill Mine area. The mine had an active life of about 40 years and covered a plan area of approximately 260,000m^2 . With an extraction ratio of 75% and an average pillar size of 12m x 12m, mining resulted in the creation of approximately 500 pillars and 1000 rooms. During the life of mine about 10 sinkholes developed within the shallow parts of the mine, where mining did not generally exceed 30m. This accounts for about 40% of the mine area. Assuming that the sinkholes developed from collapse of roof beams, the life of mine and annual risk of failure developing over the whole of the mine area can be calculated from the equation given in Section 10.1.

The annual risk of failure at Drumgill is
R = (10*0.4)/(1000*20) = 0.0002 or 1 in 5,000

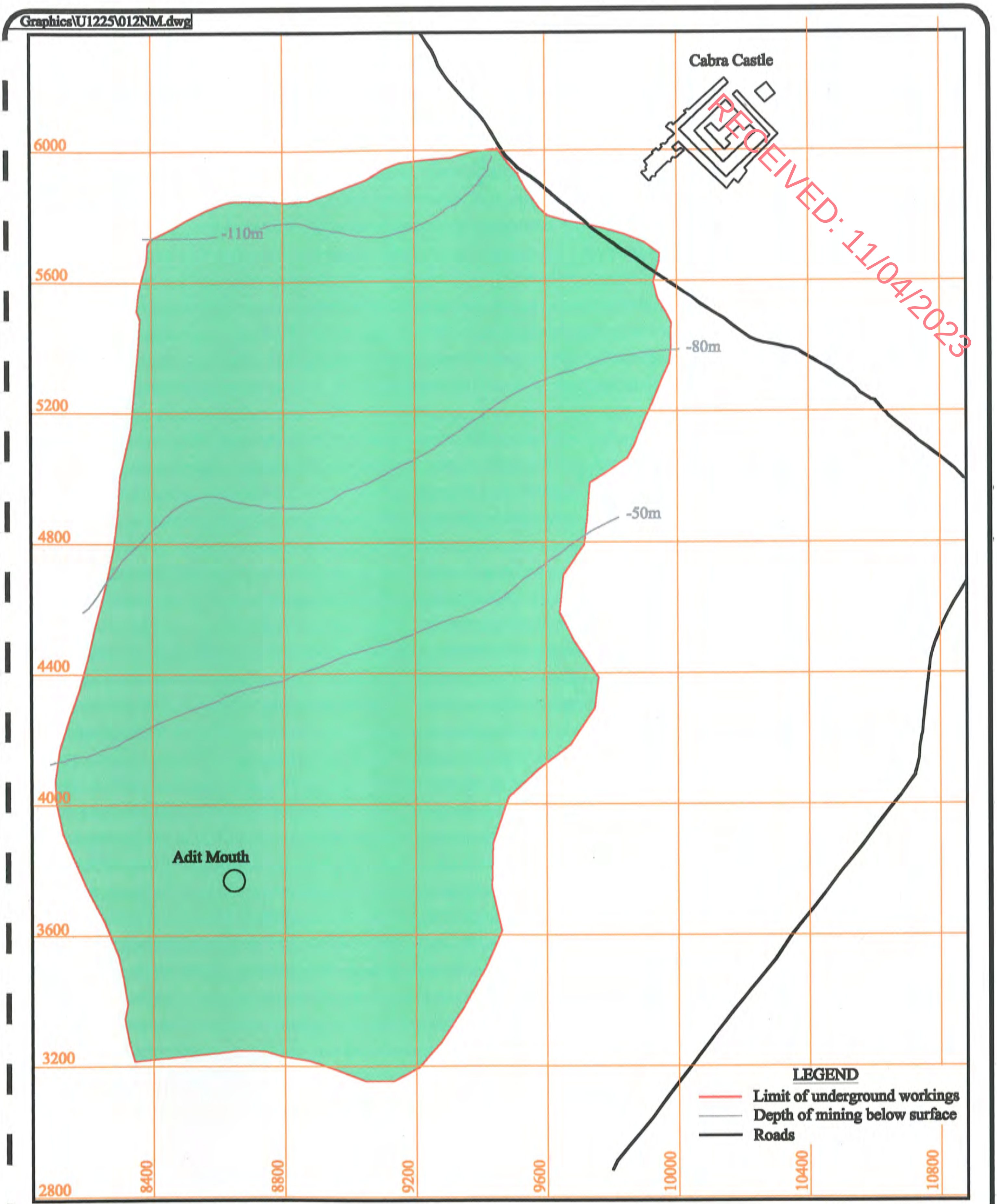
This risk of failure equates to an annual reliability of 99.98% or a life of mine reliability of 99.73%.

Because the land above the mine is generally agricultural the I_s value is 0.3 therefore the annual relative risk of failure reduces to 1 in 16,500. This constitutes a ***very low*** risk.

Because the mine is flooded, it was not possible to inspect the workings and make an independent assessment of the ground characteristics of the mine. It was not appropriate, therefore, to undertake a subsidence prediction assessment, to establish future potential for failure, to the degree of detail carried out for Drumgoosat Mine. The assessment has been limited to comments on stability based upon the depth of mining and room and pillar size and configuration, and on an extrapolation of the observations at Drumgoosat.

At the south of the mine, the depth of cover is approximately 30m. Pillars appear to be typically between 100m^2 and 140m^2 in area. Room spans are 10m. As the upper seam was mined at Drumgill, it is likely that the roof beam thickness did not greatly exceed the minimum of 3m (because the upper seam is relatively thin). Extrapolating from the results of the Drumgoosat analyses, it is likely that the mine will have a high reliability against pillar yield and roof beam stability.

At the north of the mine, the depth of cover is about 105m. The room and pillar layout is fairly regular. Pillar areas are on average 120m^2, with room spans being 10m. This mining environment is consistent with that on the western side of Drumgoosat Mine, where SRK consider that there is little risk of the occurrence of large scale subsidence.



Date: 27/11/98	PROJ. No: U1225	GYPSUM MINE SUBSIDENCE	Scale 1:15000
SRK THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT 1998		Plan of Drumgill Mine Area	Fig. 11.1

12 PREDICTION OF SINKHOLES

12.1 Mechanism of sinkhole formation

Sinkholes can form as a result of roof doming and intrusions of overlying boulder clay or mudstone. It is presumed that the clay has, in geological time, filled solution cavities, which are exposed by mining. The clay falls into the workings, leaving a dome shaped cavity on the roof. Often, nothing further occurs but, if the dome meets the top of the gypsum, clay, mudstone, dolerite or basalt can fall into the workings, starting the ravelling process, which can result in the appearance of a sinkhole at the ground surface. These sinkholes are usually of a small diameter. Two such features are known to have occurred above Drumgoosat Mine. Both have occurred where the overburden thickness is less than about 40m, and have had diameters generally less than 10m.

The formation of a sinkhole depends upon a local failure of a pillar or beam, or of a small group, the presence of stable adjacent pillars and beams, and an erodible overlying soil. It also depends on minimal "bulking" of the soil. If the workings are deep, and the loose soil falling into them occupies a larger volume than the compact undisturbed soil, this bulking can compensate for the volume of the mine void, and the sinkhole never reaches the surface.

If a local failure results in significant failure of adjacent pillars and beams, then a trough of surface settlement will occur rather than a sinkhole.

12.2 Prediction of sinkhole occurrence

It is not possible to predict the location of a sinkhole but, as they take time to propagate to the surface and as they must commence as a failure into the workings, they can be anticipated by inspections of the underground workings. Such inspections may not be feasible in the long-term. During the underground inspection for this study, examples of doming were observed, but there were no examples of the large-scale ravelling necessary for the development of a sinkhole.

In areas where there is a high incidence of sinkholes and a high risk, various methods of monitoring to give warning of sinkhole development are employed. These methods include telescopic benchmarks and geophysical surveys. They are often not very successful.

12.3 Reducing the occurrence of sinkholes

Sinkhole formation is often associated with seepage of water, and the probability of sinkhole formation can also be reduced by ensuring good surface drainage to reduce ponding of water in critical areas adjacent to roads or structures.

12.4 Risk of sinkhole occurrence

The available information indicates that there is a ***very low risk*** of the development of sinkholes at Drumgoosat Mine, where the thickness of overburden above a significant area of the mine, and particularly above the roads and Community Centre, is greater than 40m. At Drumgill Mine, additional sinkholes can be expected in the shallow workings in the vicinity of the mine entrance.

12.5 Proposed action

No additional measures are considered necessary for the monitoring of potential sinkhole formation, beyond those recommended in Section 10.

13 EFFECTS OF MINE FLOODING

13.1 Present water conditions

The deeper parts of Drumgoosat Mine and the whole of Drumgill Mine are flooded. The only known areas of subsidence associated with the flooded parts of the mines are the sinkholes near the entrance to Drumgill Mine (though the mine has only recently become fully flooded).

13.2 Possible future conditions

Drumgill will remain flooded. Drumgoosat may be allowed to flood at some later date, after mining in the quarry is complete. The water will presumably reach equilibrium at the same elevation as existed prior to mining. The wetting of the upper gypsum and the overlying materials may result in the development of sinkholes, particularly in places where doming has already occurred in the workings. At Drumgill, this has occurred, but only over very shallow workings near the mine entrance.

It is possible that flooding may reduce the strength of the rock or erode it. During underground inspections, several areas of water ingress were visited, but no signs of deterioration of the gypsum were noted.

13.3 Gypsum solubility and softening

Gypsum is sparingly soluble but, over an extended period, it can be dissolved in water. At Knocknacran Quarry, the presence of smoothed surfaces at the top of the gypsum and of occasional cavities in the upper gypsum indicate that this dissolution has occurred above the original water table, but perhaps over a very long time period.

The factors which control gypsum dissolution are:

- **The crystallinity of the gypsum.** Coarser crystals reduce the specific surface area of the gypsum, and reduce the rate of dissolution.

- **The origin, chemistry and mineralogy of the gypsum.** Gypsum of an evaporative origin, as that at Drumgoosat and Drumgill is, will contain chloride salts, which will dissolve quickly and accelerate the dissolution of the gypsum.

- **Water chemistry.** Water can only carry about 2000mg/l of sulphates and, if it is already a saturated solution, it cannot react chemically with the gypsum.

- **Water quantities.** The rate at which water flows through the gypsum controls the rate at which dissolution can occur, because of the limit on the sulphate that the water can carry.

  The factors controlling pillar stability during erosion by dissolution are the extent to which wetting reduces material strength within the pillar, and the reduction in pillar size. From observations of the behaviour of the gypsum in wet conditions underground, it is not anticipated that there will be any significant loss in strength of the rock mass, except in the very long-term (over fifty years). Regarding the reduction in pillar dimensions, it has been calculated that the pillar factor of safety reduces below 1.5 when the pillar is less than 8m in width. This represents a reduction in average pillar width of 4m. With relatively slow flows of groundwater through the workings, simple calculations indicate that the dissolution of the gypsum will not have a significant influence on pillar widths.

13.4 Proposed action

If the possibility of flooding the Drumgoosat Mine comes under consideration, the potential consequences should be technically reviewed, and suitable alterations to the monitoring programme devised.

14 CONCLUSIONS AND RECOMMENDATIONS

14.1 The existing subsidence

The back analysis reported here generally confirms the failure mechanism proposed by Dr Lehane. The subsidence under the Drumgoosat Village access road is due to failure of weak mudstone beneath the pillars, which has allowed the pillars to settle and the roof to collapse. SRK agrees with Dr Lehane that the settlement will continue for several years. If additional outer pillars begin to fail, it is possible that there may be an expansion of the affected area. No sudden movements are expected, but monitoring of levels should continue. Changes in the rate or pattern of settlement should be reviewed as indications of future behaviour. It is possible that, with further settlements of more than 500mm, more extensive road repairs will be required, involving limited reconstruction to restore the road elevation.

14.2 General conditions in the workings

The shallow workings of the Drumgoosat mine are accessible and, apart from the presence of areas of localised structurally controlled block failures in the roof of some of the workings, most of the accessible mine workings are safe to enter. The deeper workings of Drumgoosat Mine, below the floor elevation of Knocknacran Quarry, are flooded. The Drumgill mine is completely flooded. The visible condition of the rock in the accessible workings at Drumgoosat is competent and unweathered. Pillars and beams are generally stable, with high average factors of safety.

14.3 Subsidence Prediction

The possible modes of failure and subsidence in the Drumgoosat mine are by crushing of the pillars and failures of the roof beam. Where this is limited to one pillar or beam, the subsidence that may occur will be due to the migration of a small sinkhole to the surface. Such sinkholes have formed in the past. Where the failure involves several pillars and beams, the type of subsidence which has affected the road to Drumgoosat village could take place. Bearing capacity failure cannot occur because, apart from the area of existing subsidence, pillars are understood to stand on at least a 3m thickness of gypsum.

As far as is known, crushing of pillars has not yet occurred. In general, factors of safety against pillar failure are greater than 3 (reliability >95%), but a number of pillars have lower factors of safety. Where reliabilities of less than 95% have been calculated, it has been concluded that instability of a pillar in the long term (50 years plus) may occur. Instability could result from long term weathering and erosion processes. The failure of individual pillars is unlikely to produce significant subsidence at the surface, whereas the failure of a group of pillars could give rise to the magnitude of subsidence currently being measured above the Drumgoosat Village access road. Different responses are proposed, depending on the potential risk. The risk is a function not only of the probability of failure but also the consequences of failure. Thus a particular probability of pillar failure has a lesser risk if it occurs under farmland than if it occurs under a road.

14.3.1 **R179 Road**

Most pillars underlying the main R179 road have high reliabilities, but a number have reliabilities of less than 95%. It is considered that there is a low probability of the failure of one or more pillars in the long-term, and a ***low*** risk of settlements affecting the road.

It is recommended that the road be inspected at quarterly intervals, and that surface levels be monitored at six-monthly intervals. A detailed geotechnical inspection of the smaller pillars in the vicinity of the road prior to commencement of monitoring is recommended.

14.3.2 **Community Centre**

The Community Centre is over a very large stable pillar, and no movements are expected in the long-term which could endanger the structure. A number of pillar groups, lying about 40m to 50m to the west and north west of the community centre, have reliabilities of less than 95%. Should pillars in these groups fail there is a ***low*** risk that subsidence will occur above them. It is considered that the risk to the Centre is ***low***, requiring monitoring of surface levels at six month intervals and quarterly inspections of the building for cracks. A detailed geotechnical inspection the smaller pillars in the vicinity of the Centre is recommended prior to commencement of surface monitoring.

14.3.3 **Drumgoosat Village Access Road**

There is a ***very low*** risk of subsidence under the two areas of undermining of the Drumgoosat village road, beyond the area of existing subsidence. It is recommended that the road surface be inspected annually.

14.3.4 **Farmland**

Two areas of pillars with a possible long-term potential for subsidence were identified at Drumgoosat, in an area to the north-west of the Community Centre, and in an area in the south-west of the mine. The workings in these areas are deep, and it is the extra loading from the overlying material which results in failure potential. Because these areas lie under farmland they pose only a ***very low*** risk. An annual inspection is recommended.

14.3.5 **Drumgill Mine**

The workings are deep and the pillars are of standard size. No widespread subsidence is anticipated, but additional sinkholes are anticipated in the area of shallow workings near the adit. No monitoring is recommended, other than an annual inspection of the area, concentrating on the surface above the shallow workings.

14.4 Effects of Groundwater and Mine Flooding

At the time that pumping stops at Drumgoosat, if it stops, the monitoring programme should be revised, and a final underground inspection should be carried out by a geotechnical engineer, in response to the possibility that flooding may precipitate an acceleration of the weathering and deterioration of the mine workings.

It is considered that the slow process of dissolution of the gypsum by water will not effect the stability of the pillars over a nominal period of fifty years, because the pillars have high average factors of safety. It is recommended that an investigation of the gypsum properties be carried out to confirm this conclusion prior to a decision to flood the mine, and that an underground inspection be carried out if possible at that time.

Small quantities of water are being discharged from the adit of the Drumgill mine. It is proposed that the monitoring of water quality continue. The results will give an indication of underground conditions and of the acceptability of the discharged water quality.

14.5 Response to future subsidence

If measurements or observations indicate that movement is taking place under a road or the Community Centre, the immediate advice of a geotechnical engineer experienced in mining subsidence should be sought. It is not anticipated that access to the road or building would be immediately closed in the interim, however, unless the movements are large, or there is surface cracking, indicating an incipient sinkhole.

The response to minor movements would be the monitoring of levels and the prediction of future additional subsidence at that place. Moderate settlement damage to buildings could be controlled by strengthening the structure and/or underpinning and jacking. Road damage would be controlled by increased repairs and maintenance and, if necessary, the raising of the road-bed.

For and on behalf of SRK (UK) Ltd

Neil Marshall MSc
Senior Geotechnical Engineer

Ian Brackley PhD
Principal Engineer

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APPENDIX A

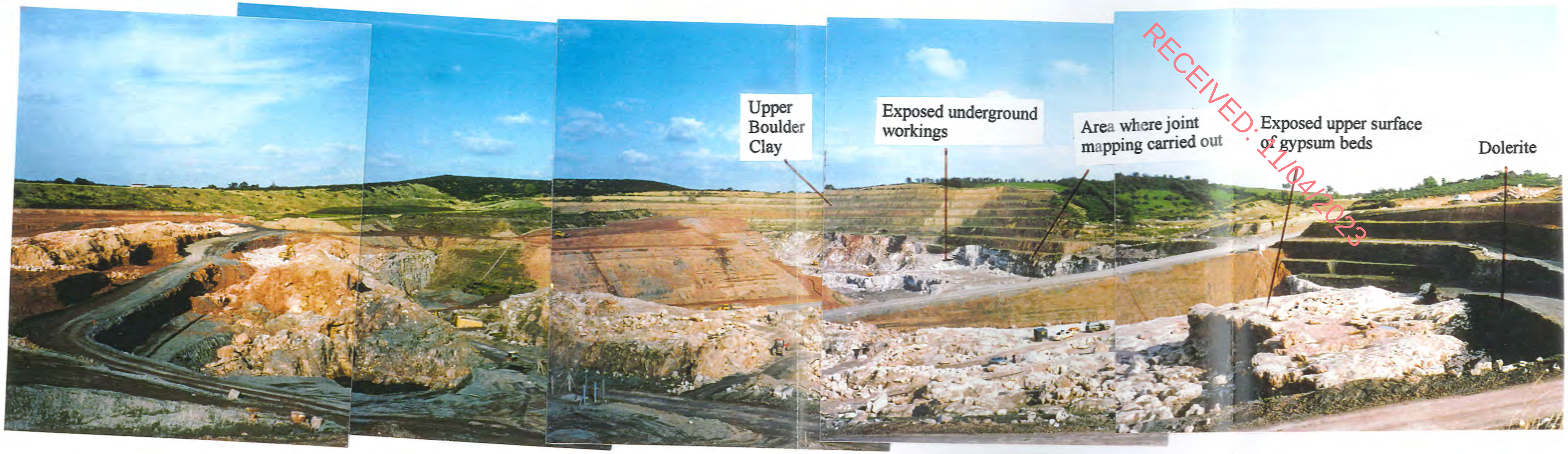
Field Sheets, Strength and RQD Data

APPENDIX B

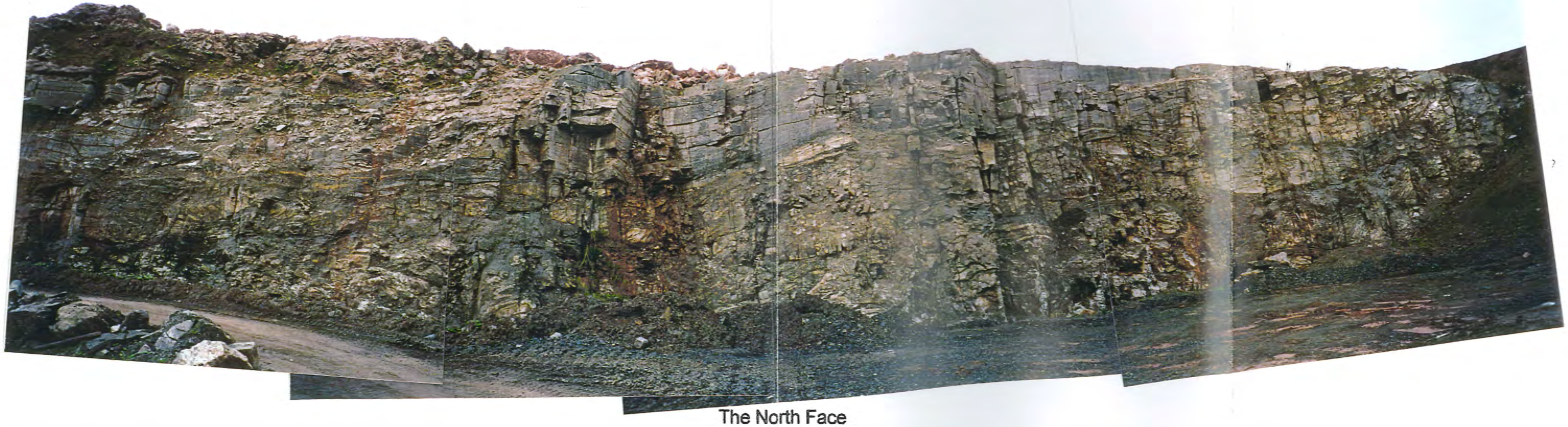
Stability Analysis Spreadsheets

APPENDIX C

Views of Knocknacran Quarry



View to the east



The North Face

DATE: 18/11/98	PROJ. No: U1225	GYPSUM MINE SUBSIDENCE	
SRK	THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT	THE KNOCKNACRAN QUARRY	Figure

APPENDIX 7.3
Check Survey and Geotechnical Inspections at Drumgoosat Disused Mines - SRK - March 2002

CHECK SURVEY AND GEOTECHNICAL INSPECTIONS AT DRUMGOOSAT DISUSED MINES, Co MONAGHAN

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U1598 **MARCH 2002**

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CHECK SURVEY AND GEOTECHNICAL INSPECTIONS AT DRUMGOOSAT DISUSED MINES, Co MONAGHAN

1 INTRODUCTION

1.1 General

The disused underground gypsum mines of Drumgoosat and Drumgill are situated near Kingscourt in Co. Cavan. These mines, owned by Gypsum Industries Ltd, were worked by shallow underground room and pillar mining methods until their closure in 1989. Current gypsum production is from Knocknacran Quarry located at the southern end of the former Drumgoosat Mine.

Following ground subsidence above the workings SRK (UK) Ltd were commissioned by the Department of the Marine and Natural Resources (DMNR) in September 1998 to undertake an independent subsidence risk assessment. This work was completed in January 1999 and following review by the DMNR a report was issued in May 1999. The report recommended detailed underground inspection and a programme of periodic measurement of surface levelling points and surface inspections in the vicinity of those areas of the mine that lay below public access land, roads and buildings. The frequency of monitoring was based on the level of risk of subsidence.

A public meeting was held at Kingscourt in December 1999 to present the findings of the study. Senior staff members of the DMNR and the main author of the SRK report attended this meeting together with members of the local authority, Gypsum Industries Ltd management and interested members of the public.

In a letter dated 17th September 2001 SRK were commissioned by the DMNR to undertake the underground component of the subsidence monitoring work and carry out check surveys of mine areas underlying specific areas of land not owned by Gypsum Industries.

1.2 Terms of Reference and Scope of Work

The terms of reference and general scope of work for the study are outlined in a letter from the Department of the Marine dated 19th February 2001, which is included as Appendix 1. The scope and overall objectives of the study are summarised below.

- To provide assurance that the mine plans for Drumgoosat are sufficiently accurate, i.e. +/- 0.5m, in the areas of two nearby properties owned by Mr Maxwell and Mr Martin, and shown in Figure 1.1 and 1.2.

- To implement the proposals for further underground work in Section 10 of SRK's 'Report on Subsidence at Former Underground Gypsum Mines near Kingscourt, Co. Cavan', dated May 1999.

The aims of the further underground work proposed by SRK in their May 1999 report were to:

1. Confirm the physical mine conditions and geotechnical properties of the rock mass forming the pillars and roof beams underlying the R179 road and Community Centre, which were identified in SRK's original study as being most at risk of failure.

2. Identify other specific geotechnical or physical conditions that may exist in and around other pillars or roof beams located within a 50m radius of the road and Community Centre that may increase the risk of subsidence occurring. Identification of such conditions would allow the surface survey targets to be appropriately sited.

The areas targeted for this additional work are shown in Figure 1.3.

1.3 Programme of Site Work

Gypsum Industries Ltd carried out initial work for the study when SRK, as part of their original proposal for the work, provided the collar locations of six cored boreholes. These holes were drilled in the proposed study areas during the period December 2000 to October 2001. The purpose of the holes was to carry survey control from surface into the underground workings and to probe the thickness and competence of the gypsum roof beam.

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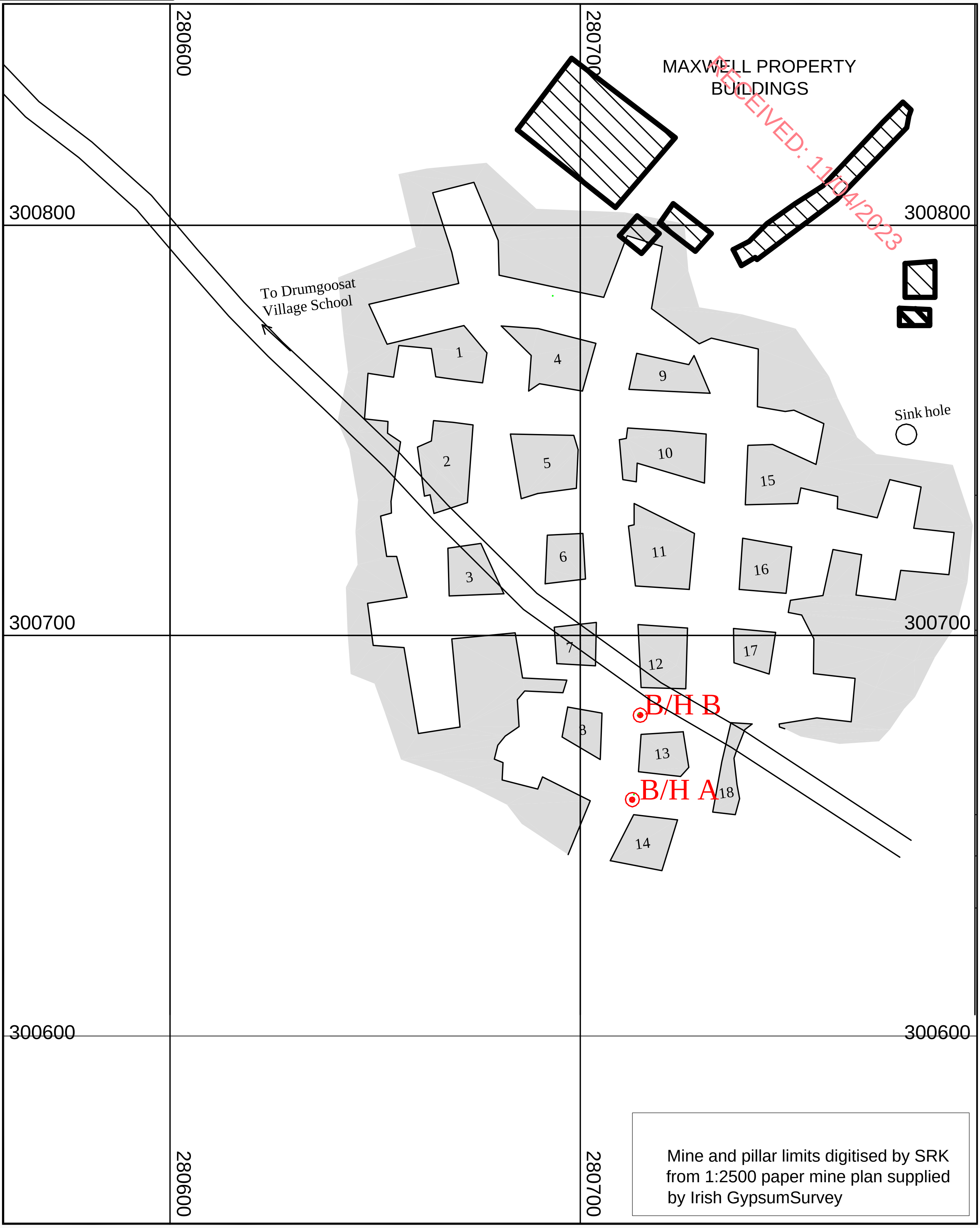

DEC 2001	PROJ. No: U1598	GYPSUM MINE SUBSIDENCE MONITORING	Scale 1:1250
SRK Consulting Engineers and Scientists		UNDERGROUND WORKINGS AND PROBE HOLE LOCATIONS NEAR THE MAXWELL PROPERTY	Fig 1.1

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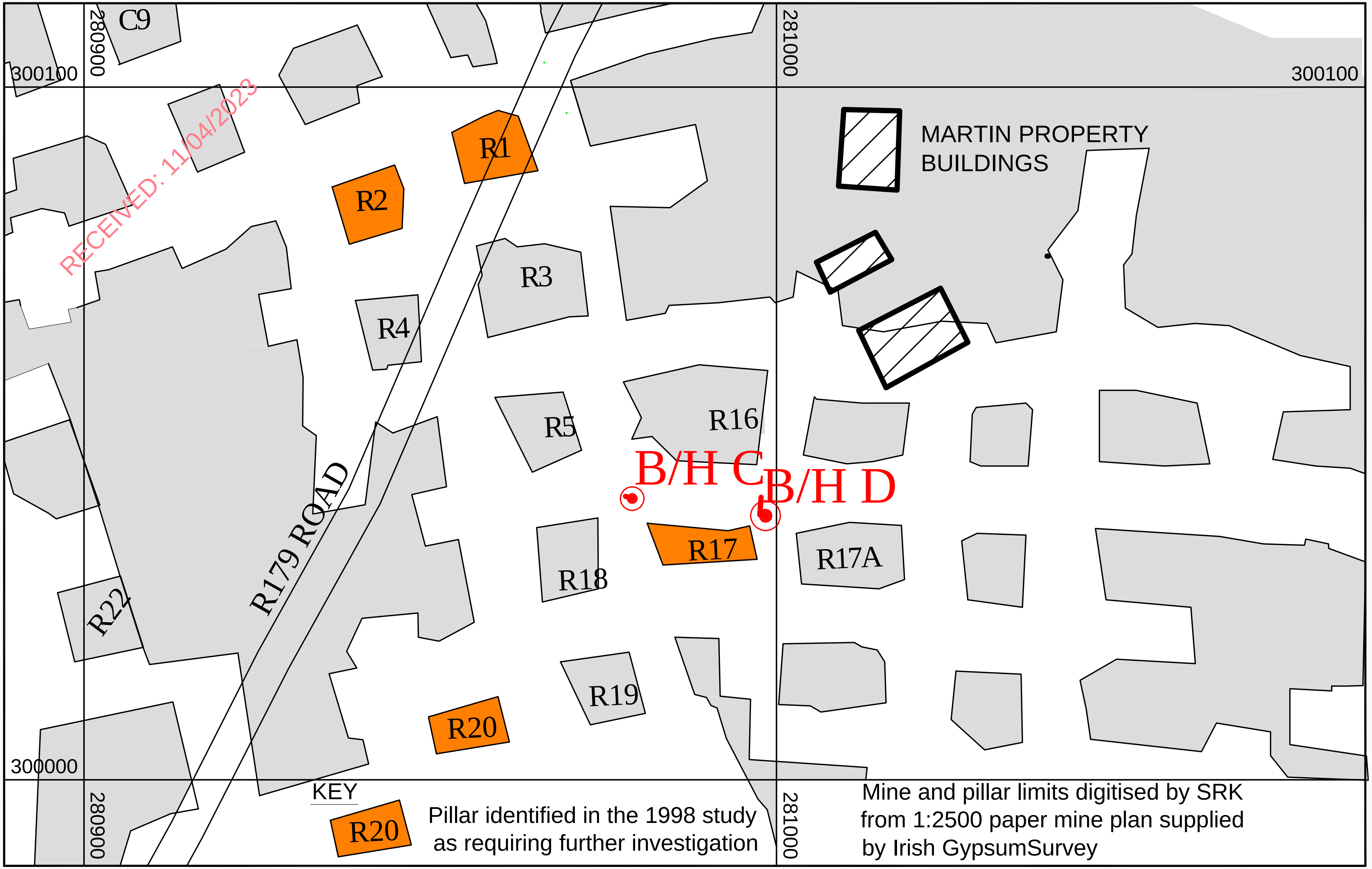

DEC 2001	PROJ. No: U1598	GYPSUM MINE SUBSIDENCE MONITORING	Scale 1:750
SRK Consulting Engineers and Scientists		UNDERGROUND WORKINGS AND PROBE HOLE LOCATIONS NEAR THE MARTIN PROPERTY	Fig 1.2

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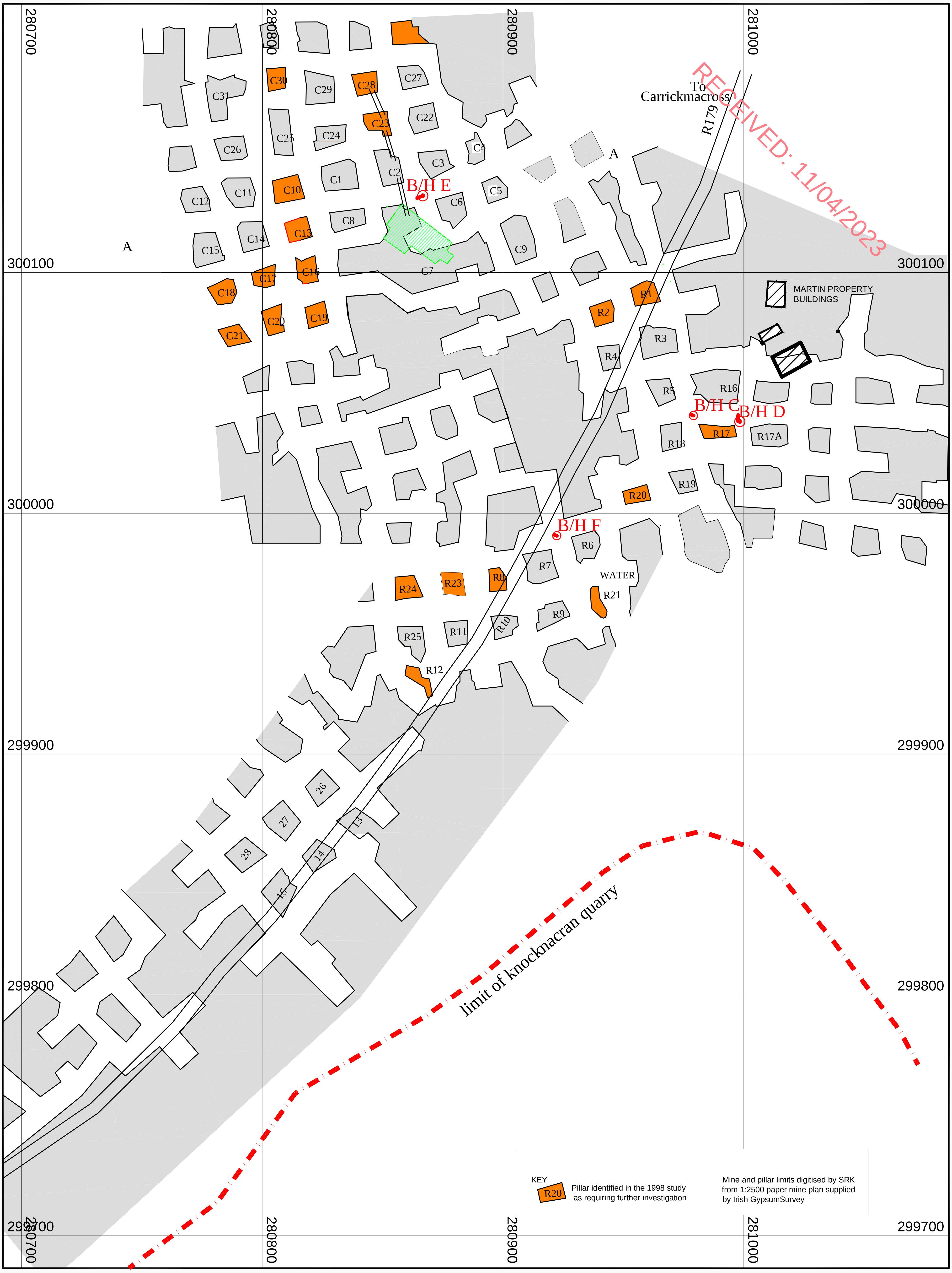


NOV 2001	PROJ. No: U1598	GYPSUM MINE SUBSIDENCE MONITORING	Scale 1:1500
SRK Consulting Engineers and Scientists		UNDERGROUND WORKINGS AND PROBE HOLE LOCATIONS NEAR THE R179 ROAD AND COMMUNITY CENTRE	Fig 1.3

A four-man team visited the site between the 4th and 10th November 2001. The team comprised an SRK principal and senior geotechnical engineer and a two man mine survey crew.

The site work began with an underground and surface tour conducted by Mr Dave Kent (Gypsum Industries Mine Manager) and Mr Andrew Ellis (Mine Surveyor). On surface, the collar locations of the six boreholes were inspected. The underground areas visited were confined to pillars within the vicinity of the break-through points of the boreholes. These areas were either accessed by vehicle or by foot. One borehole (Hole F) did not break through the roof. It was adjudged that the driller had intersected a cavity assuming that the hole had intersected the underground workings.

Following the review tour, SRK carried out check surveys of surface and underground points and carried out underground geotechnical mapping of pillars and core logging. The geological logs (described by Irish Gypsum) and the geotechnical core logs and underground mapping undertaken by SRK are located in Appendix C. Representative samples, (10 in total), of the core from the roof beam section were selected for laboratory testing at Trinity College, Dublin. The logging and testing results are discussed further in Section 3.

2 GEOTECHNICAL INSPECTIONS

2.1 Approach to the Underground Geotechnical Inspections

A quantitative approach was adopted for each individual pillar and a qualitative approach was adopted for the general underground conditions.

Each pillar inspected was identified and marked with survey paint. The pillars were mapped by inspecting faces in the order north, west, south and east walls. Due to the complexity of the underground workings and the ease of disorientation, it was felt that standardising the methodology of mapping would reduce the risk of error.

2.1.1 Quantitative Assessment of Pillar Conditions

Specific geotechnical data for each pillar were collected on scanline and cell mapping data sheets. Using a tape measure and survey staff, the dimensions of each pillar face was recorded. The effective width of each wall was then determined by measuring the distance of observed open discontinuities or cracks from each corner of the pillar. Kinematic and geotechnical data such as dip and dip direction, intact rock strength, roughness profiles and persistence of discontinuities were then collected for each wall of the pillar. This data is further described in Section 2.2.

2.1.2 Qualitative Assessment of Pillar Roof and Floor Conditions

The underground conditions are generally similar throughout, although there are localised areas of variable ground conditions. It is obvious that falls of ground have occurred from the roof and pillar sidewalls throughout the mine since it was abandoned in 1989. The size of rock fall events varies from frittering of small sugar cube size blocks in areas of the roof where there gypsum and shale are closely interbedded to large roof slabbing associated with well defined bedding planes and shaley partings. Spalling of pillar sidewalls also occur, which is associated with dilation of sub vertical joints orientated parallel to the pillar side walls. Photographs of the type of rock fall events are presented as Figure 2.1.

An area was identified to the east and south-east of the Community Centre having pillars double the height of the standard pillar. These were approximately 12m in height but may have been higher in places. The north side of this area exhibited floor heave. The location of these pillars is given later in Figure 2.3. A photograph of the floor heave is shown in Figure 2.2.

Figure 2.1: Types of Underground Rockfall Events

Figure 2.2: Floor Heave By Pillar C24

2.1.3 Data Collected

The geotechnical mapping of each pillar was divided on to two sheets, namely Rock Mass Description and Classification sheets to determine the Rock Mass Rating (RMR) of the pillar, and Scanline mapping sheets customised to determine the geometry and predominant structure of the pillar. The contents of the data sheets are summarised in Table 2.1 below.

Table 2.1: Listing of Geotechnical Data Collected

<table>
<tr><th>Rock Mass Description and Classification</th><th>Scanline Data</th></tr>
<tr><td colspan="2">Lithology
Intact material strength
Weathering
Groundwater Conditions
Roughness
Discontinuity Orientation
Alteration</td></tr>
<tr><td>Blasting Effects</td><td>Wall Width</td></tr>
<tr><td>Spacing</td><td>Effective Width</td></tr>
<tr><td>RMR Value</td><td>Persistence</td></tr>
<tr><td>Q Value</td><td></td></tr>
<tr><td>RQD Equivalent</td><td></td></tr>
</table>

2.2 Area Below R179 Road

2.2.1 Rock mass characteristics

The area contains shaley grey gypsum, which is very weak to weak in strength i.e. 1-25MPa. The rock is damp. Although generally fresh, there are localised areas of slight weathering confined to the sub surface of exposed rock and discontinuity surfaces. There are two dominant sub-vertical joint sets striking north–south and east–west. There average spacing is 0.1m and 0.15m respectively. The rock mass classification has been calculated between RMR 38-46 i.e. poor to fair quality rock mass. Detailed mapping of Pillars R1, R2, R17, R17a and R20 was undertaken. Because there was little difference in their rock mass characteristics detailed mapping was not undertaken for pillars R8, R12, R21, R23, R24 and R25. These pillars were visually assessed and photographed.

2.2.2 Conditions of pillars and roof

The condition of the roof is generally good. There have been small amounts of roof fall. This has been confined to slabbing along bedding partings and occasional frittering and doming, especially around Pillar R2. There has generally been some degradation of the pillar faces reducing the effective pillar size. Open discontinuities have been observed up to 4m from the pillar edge and in some instances throughout the pillar wall exposure. However the pillars and roof beams inspected in this area are stable.

2.2.3 Detailed Description of Selected Pillars

2.2.3.1 Pillar R1

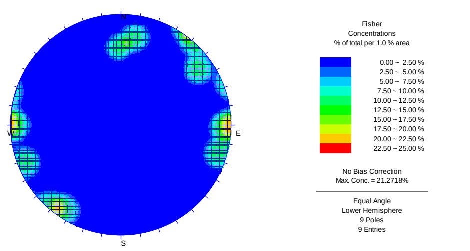
Pillar R1 is located directly below the R179 road. The geometry of the pillar and effective width is outlined in Table 2.2 below. Some joint dilation was observed. Photographs of the pillar walls are located in Appendix B as Plate B1.

Table 2.2 : Geometric Details of Pillar R1

Face	Total Width	Effective Width	Face Height
North	8.3m	5.3m	6m
East	7.6m	4.3m	6m
South	10.4m	6.4m	6m
West	6.2m	5.3m	6m

The rock mass is predominantly grey gypsum containing bedding planes and sub-vertical discontinuities. The sub-vertical discontinuity spacing ranges between 2cm to 200cm. Bedding plane partings are difficult to distinguish. The RMR is calculated at 46 i.e. FAIR quality rock mass. The geotechnical conditions of the pillar are described in the table below.

Table 2.3 : Geotechnical Properties and Stability of Pillar R1

<table>
<tr><td colspan="2"><u>Rock Mass Properties</u></td><td><u>Lower Hemisphere Stereo Net</u></td></tr>
<tr><td>Intact Strength</td><td>8-15 MPa</td><td rowspan="8">
</td></tr>
<tr><td>Density (g/cc)</td><td>2.0</td></tr>
<tr><td>Weathering</td><td>None</td></tr>
<tr><td>Blasting Effects</td><td>Moderate</td></tr>
<tr><td>Spacing E-W</td><td>1/m</td></tr>
<tr><td>Spacing N-S</td><td>1/m</td></tr>
<tr><td>Roughness</td><td>Planar Rough</td></tr>
<tr><td>Continuity</td><td>6m +</td></tr>
<tr><td>Stability Assessment</td><td colspan="2">No evidence of instability. Minor joint dilation on pillar faces</td></tr>
<tr><td colspan="2">Relative Risk of Failure (From Table 4.5)</td><td>Very Low</td></tr>
</table>

2.2.3.2 **Pillar R2**

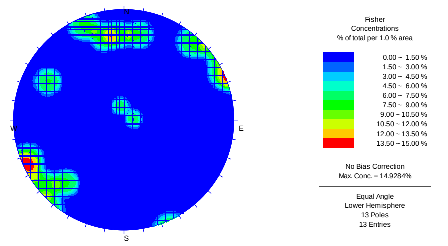
Pillar R2 is located approximately 10m northwest and 45m below the R179 road. The geometry of the pillar and effective width is outlined in Table 2.4 below. The pillar appears to be under some stress. There is evidence of slabbing and frittering from the roof as well as spalling from the sidewalls of the pillar. The spalling has taken place along the entire height of the pillar, which has reduced the effective width of the pillar by several metres and therefore increased the overall unsupported span.

Table 2.4 : Geometric Details of Pillar R2

Face	Total Width	Effective Width	Face Height
North	8.9m	4.2m	6m
East	9.5m	5.0m	6m
South	6.0m	3.3m	6m
West	8.8m	4.8m	6m

The rock mass is predominantly massive grey gypsum containing bedding plane and sub vertical discontinuities. The sub vertical discontinuity spacing ranges between 3cm to 150cm. The bedding plane partings however are difficult to distinguish. The RMR is calculated at 42 i.e. FAIR quality rock mass. The geotechnical conditions of the pillar are described in Table 2.5 below. The RQD has been estimated at be 50% accordingly.

Table 2.5 : Geotechnical Properties of Pillar R2

Rock Mass Properties		Lower Hemisphere Stereo Net
Intact Strength	8-15 Mpa	
Density (g/cc)	2.0	
Weathering	None/Slight	
Blasting Effects	Moderate	
Spacing E-W	5/m	
Spacing N-S	5/m	
Roughness	Planar Rough	
Continuity	5-6m +	
Stability Assessment	No evidence of instability. Minor joint dilation on pillar faces	
Relative Risk of Failure (From Table 4.5)	Low	

2.2.3.3 **Pillar R17**

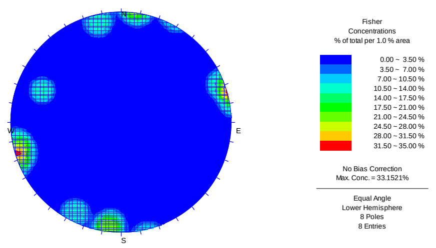
Pillar R17 is located approximately 40m southeast and 39m below the R179 road. The geometry of the pillar and effective width is outlined in Table 2.6 below. The pillar appears to be under stress with obvious open discontinuities or possible stress fractures. Photographs of the pillar walls are located in Appendix B as plate B3

Table 2.6 : Geometric Details of Pillar R17

Face	Total Width	Effective Width	Face Height
North	14.0m	10.0m	6m
East	5.0m	0.0m	6m
South	13.2m	5.7m	6m
West	8.0m	2.6m	6m

The rock mass is predominantly shaley grey gypsum containing bedding plane and sub vertical discontinuities. The sub vertical discontinuity spacing ranges between 2cm to 50cm. Bedding plane partings generally range between 15-30cm, giving a blocky appearance. The RMR is calculated at 38 i.e. POOR quality rock mass. The geotechnical conditions of the pillar are described in the Table 2.7 below.

Table 2.7 : Geotechnical Properties of Pillar R17

<table>
<tr><td colspan="2"><u>Rock Mass Properties</u></td><td><u>Lower Hemisphere Stereo Net</u></td></tr>
<tr><td>Intact Strength</td><td>8-15 MPa</td><td rowspan="8">
</td></tr>
<tr><td>Density (g/cc)</td><td>2.0</td></tr>
<tr><td>Weathering</td><td>None</td></tr>
<tr><td>Blasting Effects</td><td>Poor-Moderate</td></tr>
<tr><td>Spacing E-W</td><td>10/m</td></tr>
<tr><td>Spacing N-S</td><td>6.7/m</td></tr>
<tr><td>Roughness</td><td>Planar Rough</td></tr>
<tr><td>Continuity</td><td>3m +</td></tr>
<tr><td>Stability Assessment</td><td colspan="2">Minor rock falls from pillar faces Some joint dilation on pillar faces</td></tr>
<tr><td colspan="2">Relative Risk of Failure (From Table 4.5)</td><td>Low</td></tr>
</table>

2.2.3.4 Pillar R17a

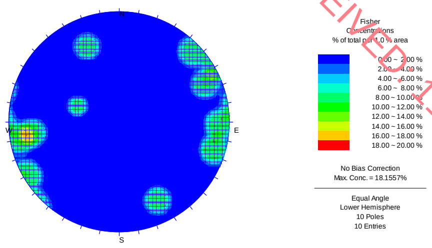
Pillar R17a is located approximately 55m southeast and 39m below the R179 road. The geometry of the pillar and effective width is outlined in Table 2.8 below. The pillar appears to be under stress with obvious open discontinuities or possible stress fractures. Photographs of the pillar walls are located in Appendix B as Plate B4.

Table 2.8 : Geometric Details of Pillar R17a

Face	Total Width	Effective Width	Face Height
North	15.5m	11.8m	6m
East	9.0m	6.5m	6m
South	13.1m	10.4m	6m
West	8.6m	4.1m	6m

The rock mass is predominantly massive grey gypsum containing bedding plane and sub vertical discontinuities. The sub vertical discontinuity spacing ranges between 2cm to 200cm. The bedding plane partings however are difficult to distinguish. The RMR is calculated at 46 i.e. FAIR quality rock mass. The geotechnical conditions of the pillar are described in Table 2.9 below.

Table 2.9 : Geotechnical Properties of Pillar R17a

Rock Mass Properties		Lower Hemisphere Stereo Net
Intact Strength	8-15 MPa	
Density (g/cc)	2.0	
Weathering	None	
Blasting Effects	Moderate	
Spacing E-W	6/m	
Spacing N-S	6/m	
Roughness	Planar Rough	
Continuity	5-6m +	
Stability Assessment	Sidewall failures. Joint dilation on pillar faces	
Relative Risk of Failure (From Table 4.5)	Very Low	

2.2.3.5 **Pillar R20**

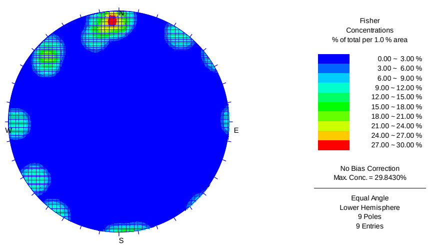
Pillar R20 is located approximately 30m southeast and 40m below the R179 road. The geometry of the pillar and effective width is outlined in Table 2.10 below. The pillar appears to be under minimal stress probably due to the cover provided by the large pillar to the northwest and to the south. Photographs of the pillar walls are located in Appendix B as Plate B5.

Table 2.10 : Geometric Details of Pillar R20

Face	Total Width	Effective Width	Face Height
North	11.7m	10.7m	6m
East	7.5m	6.0m	6m
South	11.8m	9.2m	6m
West	6.0m	2.3m	6m

The rock mass is predominantly grey gypsum containing bedding plane and sub vertical discontinuities. The sub vertical discontinuity spacing ranges between 2cm to 100cm. The bedding plane partings are difficult to distinguish. The RMR is calculated at 45 i.e. FAIR quality rock mass. The geotechnical conditions of the pillar are described in Table 2.11 below. There is a reduction in rock mass quality in small, localised zones due to the presence of solution cavities. The RQD has been estimated at between 36-60% accordingly.

Table 2.11 : Geotechnical Properties of Pillar R20

Rock Mass Properties		Lower Hemisphere Stereo Net
Intact Strength	8-15 MPa	
Density (g/cc)	2.0	
Weathering	None	
Blasting Effects	Moderate	
Spacing E-W	4/m	
Spacing N-S	1/m	
Roughness	Planar Rough	
Continuity	5-6m +	
Stability Assessment	No evidence of instability. Minor joint dilation on pillar faces	
Relative Risk of Failure (From Table 4.5)	Low	

Photographs of Pillars R21, R23, R24 and R25 are also included in Appendix B as Plates B6-B9. These pillars were not mapped but visually assessed. The scanline and cell mapping sheets for the above mentioned pillars are located in Appendix C.

2.3 Area to the west and northwest of the Community Centre

A visual assessment of the pillars in this area was undertaken. In addition to the pillars identified as a potential risk in the 1999 study the area of investigation was widened by the discovery of a group of 12m high pillars located some 70-80m west of the Community Centre. This section describes the condition of the pillars in this area.

2.3.1 General description of rock mass characteristics

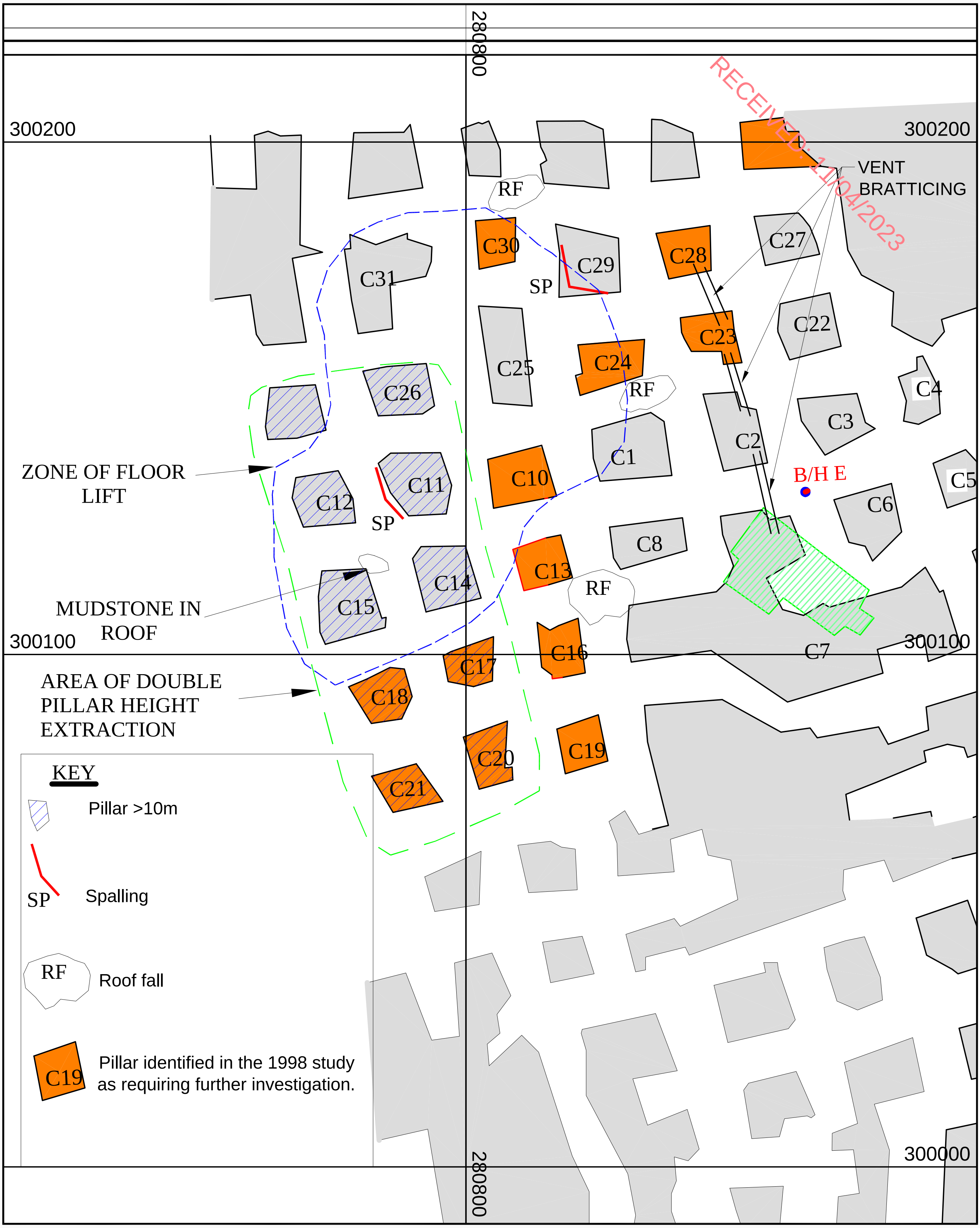
The lithology and rock mass characteristics to the west and northwest of Pillar C7 are similar to those that had previously been mapped and described below the R179 road at pillar locations R1, R2, R17, R17a and R20 some 170m to the southeast.

The area contains grey gypsum, which is very weak to weak in strength i.e. 1-25MPa. The rock is damp and although generally fresh there are localised areas of slight weathering confined to the sub surface of exposed rock and discontinuities. There are two dominant sub vertical joint sets striking north–south and east–west. The average spacing is 0.1m and 0.15m respectively. The rock mass classification has been estimated between RMR 30-45 i.e. poor to fair quality rock mass. Rock fall areas are associated with poorer quality rock mass zones. These areas appear to be associated with either well bedded shaley gypsum or sections of gypsum that have been affected by salt solution cavities, thus reducing the tensile capability of the rock mass. All the pillars were observed and some were photographed successfully.

2.3.2 Conditions of pillars, roof and floor

The condition of the pillars, roof and floor varies considerably in the area to the west and northwest of Pillar C7. This is probably mainly due to excessive extraction resulting in tall pillars, coupled with a varying roof beam thickness. The position of the pillars and location of ground conditions with respect to Pillar C7 is presented in Figure 2.3.

D:\GRAPHICS\U1598\001IG.dwg



DEC 2001	PROJ. No: U1598	GYPSUM MINE SUBSIDENCE MONITORING	Scale 1:1000
SRK Consulting Engineers and Scientists		LOCATION OF ROCKFALL EVENTS & HIGH PILLARS TO WEST AND NORTH WEST OF PILLAR 7	Fig 2.3

Roof Falls occur to the west and northwest of Pillar C7. The rock fall events have involved up to 1m of roof beam thickness approximately 4-6m in diameter. The events are located along the periphery of the floor heave zone and may be the result of increased tensional stress during settlement.

Spalling – was observed along the southwest ends of pillar numbers C11 and C29. Pillar C29 is located approximately 50m northwest from Pillar C7 and bisects the zone of floor heave. The spalling of the pillar is or seems to be the direct result of the floor heave event and increased compressional stress. Pillar C11 is located almost in the centre of the floor heave zone and 10m northeast from where mudstone is observed in the roof. The spalling was probably the result of increased compressional forces in this area. A relatively thin and irregular roof beam may have exacerbated these effects.

Floor Heave – was observed between 22m – 90m northwest of Pillar C7 (see Figure 2.2). The area is an irregular oval shape approximately 100m x 50m and 4340m^2 in area. The floor heave disturbance is generally 30cm in height but there are areas that have been affected up to 1m of floor convergence. The zone is spread evenly between six 6m pillars and six 12m pillars. This reason for the specific floor heave zone may be the result of the relationship between double and single pillar location with respect to roof beam and floor thickness and the position of large buttress pillars with respect to the area.

Double Height Pillars – were identified some 30m to the west from Pillar 7. Ten pillars were identified in total, covering an area of some 3550m^2. The condition of the pillars varied depending on the size of the pillar. The larger pillars are seen to be more intact and ground movements have not affected their integrity. The small pillars appear stressed with open joints and clear definition of joint planes resulting in a columnar appearance. Some pillars appeared to be stressed and were hour glass shaped much like concrete testing blocks. Photographs are located in Appendix B as Plate B10. These are however poor quality due to the ineffectiveness of the camera flash in the large open voids.

Mudstone Roof – Mudstone is observed in the roof, just to the northeast of double pillar C15. A photograph is located in Appendix B as Plate B11. Whilst the mining level is reasonably constant the excavation in to the lower mudstone proves the uneven nature of the contact between the Gypsum and Mudstone. This is also confirmed by exposure in the open pit. Whilst there has been some collapse the area seems to have stabilised, with perhaps only small quantities frittering from time to time.

Concluding Remarks – Whilst the conditions in the area to the west and northwest of Pillar 7 below the Community Centre are generally poorer than other areas observed the pillar and roof failures noted above are not fresh. It is the opinion of SRK that instability of the mine elements in this area has significantly reduced and may have ceased. The observed conditions are unlikely to deteriorate further unless there is a material change in physical condition of the gypsum. The area at present may be considered to be stable.

3 GEOTECHNICAL DRILLING

3.1 Core Logging

Six vertical holes were drilled around the study area. The positions of the holes are shown in Figures 1.1, 1.2 and 1.3. The core was geologically logged throughout its entire length and geotechnically logged along the roof beam section i.e. generally within gypsum lithology. A summary of the location of each borehole and the thickness of gypsum intersected in each hole is tabulated below.

Table 3.1 : Borehole Summary

Hole Id	Hole Location	Gypsum Thickness
A	Maxwell Property	7.9m
B		13.1m
C	Martin Property	15.5m
D		9.3m
E	Community Centre	8.0m
F	R179 Road	+14.2m

The detailed logging sheets are located in Appendix C. A summary of the geotechnical logging data is presented in Table 3.2.

3.2 Laboratory Testing

Laboratory testing of the core was undertaken at Trinity College, Dublin under the supervision of Dr Barry Lehane. Ten representative samples were selected from all of the holes drilled, two each from holes C, D, F & E and one each from holes A & B. The core samples were subjected to Brazilian tests to determine the tensile strength and unconfined compressive strength (UCS) tests to determine the compressive strength of the rock. The mean values for the tests are σ_t = 3.1MPa, and σ_c = 16.4MPa. The compressive strength results are lower than what was previously estimated. The maximum UCS value of the rock is tested at 23.9 MPa. The results classify the intact rock strength as <u>WEAK</u>.

A summary of the test results is presented in Tables 3.3 and 3.4 below.

Table 3.2 : Summary of Geotechnical Logging

Hole ID	Depth (m)	Lithology	TCR %	SCR %	RQD %	Number of Natural Fractures		
						Low Angle	Moderate Angle	High Angle
Hole A	7-22	Dolerite	3.5	0	0	-	-	-
	22-27	D-Section Gypsum	95	95	31	1	1	18
	27-29.9	Gypsum / Anhydrite	100	100	86	0	0	17
Hole B	16.9-28.6	Gypsum pink and grey	100	100	90	0	0	54
	28.6-30	Anhydrite, mudstone partings	100	100	89	0	0	8
	30-30.6	Mudstone	100	100	0	0	0	16
Hole C	0-10	Soil and Red Clay	-	-	-	-	-	-
	10-23.84	Mudstone interbedded Gypsum	100	95	80	0	0	12
	23.8-39.3	Grey Gypsum	100	100	85	2	2	67
Hole D	24.4-29.2	Red 7 Grey Mudstone	100	95	70	0	3	34
	29.2-35	Gypsum selenite, colour bands	100	95	78	1	0	38
	35-38.5	Gypsum becoming grey	100	100	86	1	3	22
Hole E	46.5-51.5	Mudstone Red plastic in parts	86	69	-	0	10	13
	51.5-53.3	Grey Gypsum	100	92	80	0	0	10
	53.3-59.5	White Gypsum	100	98	84	0	2	31
Hole F	24.1-25	Mudstone Red plastic in parts	100	100	60	0	0	8
	25-26.6	Grey Gypsum	100	100	100	0	0	5
	26.6-31	White Gypsum	100	96	88	0	1	10
	31-36.2	Grey Gypsum	100	97	89	0	1	16
	36.2-38.4	White Gypsum	100	100	95	0	1	10
	38.4-39.2	Grey Gypsum	100	100	68	0	0	6

Table 3.3 : Summary of Brazilian Test Results

Sample No	Depth	Hole ID	Tensile Strength MPa	Water Content %
SRK 001	52.91-53.48	E	2.37	5.1
SRK 002	58.03-58.38	E	1.83	5.2
SRK 003	30.13-30.68	F	1.57	6.5
SRK 004	35.84-36.33	F	6.33	2.3
SRK 005	35.75-36.07	D	2.0	5.3
SRK 006	36.0-36.2	D	2.92	2.9
SRK 007	37.5-37.88	C	2.44	4.5
SRK 009	28.1-28.4	B	4.44	4.4
SRK 010	29.1-29.3	A	3.85	4.4

Table 3.4 : Summary of Unconfined Compressive Strength Results

Sample No	Depth	Hole ID	UCS MPa	Density Mg/m^3	Water Content %	Type of Failure
SRK 001	52.91-53.48	E	15.2	2.31	4.3	Shear
SRK 002	58.03-58.38	E	12.7	2.30	4.7	Shear
SRK 003	30.13-30.68	F	10.6	2.26	7.3	Shear
SRK 004	35.84-36.33	F	20.0	2.33	2.1	Shear
SRK 005	35.75-36.07	D	13.3	2.29	5.3	Shear
SRK 006	36.0-36.2	D	20.1	2.44	2.1	Shear
SRK 007	37.5-37.88	C	16.0	2.31	4.5	Vertical Cleavage
SRK 009	28.1-28.4	B	23.9	2.30	4.9	Vertical Cleavage
SRK 010	29.1-29.3	A	15.6	2.31	4.2	Shear

4 DATA ANALYSIS

4.1 Rock Mass Characterisation

Two rock mass classification schemes, those developed by Laubscher (MRMR) and Bieniawski (RMR), were used to characterise the gypsum forming the roof beams and the pillars. The RMR is used in the Hoek-Brown empirical rock mass criterion to develop the Mohr Coulomb rock mass shear strength parameters of friction angle, cohesion and deformation modulus for input into the numerical modelling analysis described in Section 4.4. The MRMR was used to establish the design rock mass strength of the pillars to be able to update the pillar and beam analysis carried out in the original study for those critical pillars.

The MRMR rock mass classification system was described in the original report and is an adaptation of the RMR system. Parametric inputs are the same for both classification systems but there are slight differences in ratings given to each individual rock mass parameter.

As both schemes were originally developed for the rating of borehole core SRK has used the results of the geotechnical logging of the six diamond drill holes in the estimation of both RMR and MRMR.

The result of the rock mass classification of the borehole core is given in Tables 4.1 and 4.2.

4.2 Empirical Rock Mass Strength

4.2.1 Design Rock Mass Strength

The design rock mass strength, used to define the strength of the pillars and roof beam, is the intact rock strength reduced by a percentage defined by the sum of the MRMR joint condition and fracture frequency rating.

The intact rock strength used for the classification of the gypsum is that obtained from the laboratory test work carried out for this study and described in Section 3.

The mean and standard deviation of the gypsum design rock mass strength is tabulated below and is compared with the same values obtained during the original study. It will be recalled that in order to calculate a probability of failure using the point estimate method, pillar factors of safety must be calculated for pillar strength values lying one standard deviation either side of the mean value.

Table 4.1 : Rock Mass Classification of Gypsum Core Using Laubscher's MRMR

SRK

Windsor Court, 1/3 Windsor Place, Cardiff, UK, CF10 3BX
Tel: +44 (0)2920 348150 Fax: +44 (0)2920 348199

MRMR SUMMARY SHEET

Site:	Drumgoosat	Borehole No.:	Various	Project No:	U1598
Location:	Ireland	Data Sheet No.:	1	Date:	21-Nov-01
				Logged By:	Greg

			Rock Strength		Fracture Frequency			Joint Condition						RMR			ADJUSTMENTS				MRMR			RMS	DRMS
FROM	TO	Lithology	Value (Mpa)	Rating	Joint Sets	FF/m	Rating	Water	JA	JB	JC	JD	Rating	Rating	Class	Descr	Weath	Orient	Stress	Blast	Rating	Class	Descr	(MPa)	(MPa)
Borehole 00A																									
7.00	22.00	Dolerite	1	1	3	40	0	Dry	0.75	0.65	1.00	1.00	20	20	5A	Very Poor	1	1	1	1	20	5A	Very Poor	0	0
22.00	23.80	Gypsum	5	2	3	40	0	Dry	0.75	0.65	1.00	1.00	20	21	4B	Poor	1	1	1	1	21	4B	Poor	1	1
23.80	27.00	Gypsum	15	3	3	10	10	Dry	0.75	0.80	1.00	1.00	24	37	4A	Poor	1	1	1	1	37	4A	Poor	5	5
27.00	29.90	Gypsum	15	3	1	4	22	Dry	0.75	0.65	1.00	1.00	20	44	3B	Fair	1	1	1	1	44	3B	Fair	6	6
Borehole 00B																									
16.90	23.80	Gypsum	5	2	1	2	26	Dry	0.75	0.95	1.00	1.00	29	55	3A	Fair	1	1	1	1	55	3A	Fair	3	3
23.80	28.60	Gypsum	24	3	1	3	24	Dry	0.75	0.95	1.00	1.00	29	55	3A	Fair	1	1	1	1	55	3A	Fair	12	12
28.60	30.05	Anhydrite	24	3	1	3	23	Dry	0.75	0.65	1.00	1.00	20	46	3B	Fair	1	1	1	1	46	3B	Fair	10	10
30.05	30.60	Mudstone	25	4	1	16	12	Dry	0.75	0.60	1.00	1.00	18	34	4A	Poor	1	1	1	1	34	4A	Poor	8	8
Borehole 00C																									
22.00	23.84	Mudstone	0		1	4	22	Dry	0.75	0.95	1.00	1.00	29	50	3B	Fair	1	1	1	1	50	3B	Fair	0	0
23.84	39.35	Gypsum	16	3	3	3	20	Dry	0.75	0.95	1.00	1.00	29	51	3A	Fair	1	1	1	1	51	3A	Fair	8	8
Borehole 00D																									
24.40	29.16	Mudstone	1	1	2	5	18	Dry	0.75	0.95	1.00	1.00	29	48	3B	Fair	1	1	1	1	48	3B	Fair	0	0
29.16	35.00	Gypsum	13	3	2	4	19	Dry	0.75	0.65	1.00	1.00	20	42	3B	Fair	1	1	1	1	42	3B	Fair	5	5
35.00	38.52	Gypsum	20	3	3	4	16	Dry	0.75	0.95	1.00	1.00	29	48	3B	Fair	1	1	1	1	48	3B	Fair	9	9
Borehole 00E																									
46.50	51.50	Mudstone	0		2	4	19	Dry	0.75	0.65	1.00	1.00	20	39	4A	Poor	1	1	1	1	39	4A	Poor	0	0
51.50	53.32	Gypsum	15	3	1	3	23	Dry	0.75	0.95	1.00	1.00	29	54	3A	Fair	1	1	1	1	54	3A	Fair	8	8
53.32	59.55	Gypsum	13	3	2	3	21	Dry	0.75	0.95	1.00	1.00	29	52	3A	Fair	1	1	1	1	52	3A	Fair	6	6
Borehole 00F																									
24.10	25.00	Mudstone	0		1	6	20	Dry	0.75	0.65	1.00	1.00	20	40	4A	Poor	1	1	1	1	40	4A	Poor	0	0
25.00	26.62	Gypsum	11	2	1	2	27	Dry	0.75	0.95	1.00	1.00	29	58	3A	Fair	1	1	1	1	58	3A	Fair	6	6
26.62	31.00	Gypsum	11	2	2	1	26	Dry	0.75	0.65	1.00	1.00	20	48	3B	Fair	1	1	1	1	48	3B	Fair	5	5
31.00	36.22	Gypsum	20	3	2	2	24	Dry	0.75	0.95	1.00	1.00	29	56	3A	Fair	1	1	1	1	56	3A	Fair	11	11
36.22	38.44	Gypsum	20	3	2	3	22	Dry	0.75	0.95	1.00	1.00	29	53	3A	Fair	1	1	1	1	53	3A	Fair	10	10
38.44	39.20	Gypsum	20	3	1	4	21	Dry	0.75	0.60	1.00	1.00	18	42	3B	Fair	1	1	1	1	42	3B	Fair	8	8

Table 4.2 : Rock Mass Classification of Gypsum Core Using Bieniawski's RMR

RMR SUMMARY SHEET

Windsor Court, 1-3 Windsor Place, Cardiff, UK, CF10 3BX — Tel: +44 (0)2920 348150 Fax: +44 (0)2920 348199

Site: Drumgoosat — Borehole No: Various — Project No: U1598
Location: Ireland — Data Sheet No: — Date: 21-Nov-01
Logged By: Greg

FROM	TO	Lithology	Rock Strength Value (Mpa)	Rock Strength Rating	RQD Value (%)	RQD Rating	Joint Spacing (m)	Joint Spacing Rating	Persistence	Aperture	Roughness	Infill	Weathering	Condition of Joints Rating	Groundwater Value	Groundwater Rating	Orientation Value	Orientation Rating	RMR Rating	RMR Class	RMR Descr	GSI Rating
Borehole 00A																						
7.00	22.00	Dolerite	1		0		2.000	20	2	4	3	1		10	Dry	15			45	III	Fair	40
22.00	23.80	Gypsum	0		0		1.800	19	2	4	3	1		10	Dry	15			44	III	Fair	39
23.80	27.00	Gypsum	15	2	31	6	0.178	8	2	4	6	1	1	14	Dry	15			45	III	Fair	40
27.00	29.90	Gypsum	15	2	86	17	0.171	8	2	4	3	1	6	16	Dry	15			58	III	Fair	53
Borehole 00B																						
16.90	23.80	Gypsum	5	1	90	18	0.230	9	2	4	6	1	6	19	Dry	15			62	II	Good	57
23.80	28.60	Gypsum	24	3	91	18	0.200	9	2	4	6	1	6	19	Dry	15			64	II	Good	59
28.60	30.05	Anhydrite	24	3	89	18	0.191	8	2	4	3	1	6	16	Dry	15			60	III	Fair	55
30.05	30.60	Mudstone	25	3	0		0.034	5	2	4	1	1	6	14	Dry	15			37	IV	Poor	32
Borehole 00C																						
22.00	23.84	Mudstone	0		80	16	0.153	8	2	4	6	1	6	19	Dry	15			58	III	Fair	53
23.84	39.35	Gypsum	16	2	85	17	0.218	9	2	4	6	1	6	19	Dry	15			62	II	Good	57
Borehole 00D																						
24.40	29.16	Mudstone	1		70	14	0.129	8	2	4	6	1	6	19	Dry	15			56	III	Fair	51
29.16	35.00	Gypsum	13	2	78	16	0.150	9	2	4	3	1	6	16	Dry	15			57	III	Fair	52
35.00	38.52	Gypsum	20	2	86	17	0.136	8	2	4	6	1	6	19	Dry	15			61	II	Good	56
Borehole 00E																						
46.50	51.50	Mudstone	0		0		0.217	9	2	4	3	1	5	15	Dry	15			39	IV	Poor	34
51.50	53.32	Gypsum	15	2	80	16	0.182	8	2	4	6	1	6	19	Dry	15			60	III	Fair	55
53.32	59.85	Gypsum	13	2	84	17	0.189	8	2	4	6	1	6	19	Dry	15			61	II	Good	56
Borehole 00F																						
24.10	25.00	Mudstone	0		60	12	0.113	7	2	4	3	1	6	16	Dry	15			50	III	Fair	45
25.00	26.62	Gypsum	11	1	100	20	0.324	9	2	4	6	1	6	19	Dry	15			64	II	Good	59
26.62	31.00	Gypsum	11	1	88	18	0.388	10	2	4	3	1	6	16	Dry	15			60	III	Fair	55
31.00	36.22	Gypsum	20	2	89	18	0.307	9	2	4	6	1	6	19	Dry	15			63	II	Good	58
36.22	38.44	Gypsum	20	2	95	19	0.282	9	2	4	6	1	6	19	Dry	15			64	II	Good	59
38.44	39.20	Gypsum	20	2	68	14	0.127	8	2	4	1	1	6	14	Dry	15			53	III	Fair	48

Table 4.3 : Pillar Design Rock Mass Strength

	2001 Study	1999 Study
Mean Value	8 MPa	12 MPa
Standard Deviation	2 MPa	6 MPa
Coefficient of Variation	25%	50%
Mean – 1 Standard Deviation	5 MPa	8 MPa
Mean + 1 Standard Deviation	11 MPa	20 MPa

It can be seen that the overall pillar strength based on laboratory data is less than the values estimated in the 1999 study. This is due to the laboratory strength tests returning lower strength results than the field estimates of strength that were used in the original study. These laboratory results are however consistent with the results of other laboratory strength tests undertaken by Irish Gypsum in the past.

4.2.2 Rock Mass Shear Strength

For the geotechnical study the rock mass shear strength properties have been determined empirically using Hoek's Geologic Strength Index (GSI), which is obtained from Bieniawski's 1974 RMR classification scheme, and the Hoek-Brown empirical rock mass strength criteria[1]. The GSI was calculated from the geotechnical logging of the gypsum core. The RMR and GSI values for the core have been given in Table 4.2 above.

For the given GSI, intact rock strength and m_i value (an empirical constant for the intact rock, related to generic rock type) the Hoek-Brown criterion produces a non-linear strength envelope over the range of normal stresses of interest. A tangent to the non-linear envelope produces an instantaneous cohesion and friction angle for a given normal stress. From an analysis of the variability in the GSI calculated from the core logging typical values were chosen to define the rock mass strength of the gypsum. Additionally, estimates of the rock mass shear strength of the basal mudstone and overburden material have been made with reference to the core logging where applicable and to data given in SRK's 1999 report. The normal stress range used to calculate the cohesion and friction angle corresponds to the anticipated range of stresses generated by the overburden pressure.

The parameters used to define the rock mass strength of each unit and the instantaneous cohesion and friction angle over the given range of overburden stress are tabulated below.

[1] Hoek, E., Kaiser, P.K. and Bawden, W.F. (1995). Support of Underground Excavations in Hard Rock., Ch 8. pp84-97, AA Balkema, Rotterdam.

Table 4.4 : Rock Mass Shear Strength Parameters

Rock Unit	Overburden	Gypsum	Basal Mudstone
Intact Strength (MPa)	1	15	5
m_i	9	12	9
GSI	40	40	40
Material Density (t/m3)	2	2.3	2.0
Overburden Stress (MPa)	1.2	1.6	1.65
Cohesion (kPa)	60	43	130
Friction Angle (°)	19	200	30
Rock Mass Deformation Modulus (MPa)	560	4875	1260
Poissons Ratio	0.3	0.15	0.3
Rock Mass Bulk Modulus (MPa)	450	2320	1050
Rock Mass Shear Modulus (MPa)	215	2120	485

4.3 Pillar Stability Analysis

The pillar stability analysis presented in SRK's 1999 report has been updated for the sensitive pillars below the road and community centre using the revised pillar strength parameters. The results of the pillar analyses are tabulated below.

Table 4.5 : Road Area - Results of Revised Pillar Stability Analysis

	Pillar Yield			
Pillar No.	Mean FoS	Reliability R[f]	Relative Risk of Failure	Relative Risk of Failure
SINGLE PILLARS			2001	1999
R1	2.1	98%	Very Low	Low
R2	1.6	93%	Low	Low
R8	1.2	73%	Intermediate	Low
R12	1.1	63%	High	Low
R17	1.5	92%	Low	Low
R17a	2.3	99%	Very Low	
R20	1.4	87%	Low	Low
R21	0.9	31%	High	High
R23	1.5	90%	Low	Low
R24	1.5	91%	Low	Low
R25	1.9	97%	Very Low	Low
PILLAR GROUPS				
R1 + R2	2.1	98%	Very Low	Low
R8 + R23	1.4	87%	Low	Low
R23 + R24	1.4	88%	Low	Low
R24 + R25	1.5	90%	Low	Low
R12 + R25	1.2	78%	Intermediate	Low
R24+R23+R8	1.2	75%	Intermediate	Low
R24+R25+R12	1.2	77%	Intermediate	Low
R23+R24+R25	1.2	77%	Intermediate	Low

Table 4.6: Community Centre Area - Results of Revised Pillar Stability Analysis

	Pillar Yield			
Pillar No.	Mean FoS	Reliability R[f]	Relative Risk of Failure 2001	Relative Risk of Failure 1999
SINGLE PILLARS				
C10*	1.8	97%	Very Low	Low
C11*	1.5	91%	Low	Very Low
C12*	1.1	65%	High	Very Low
C13	1.7	95%	Very Low	Low
C14*	1.3	83%	Low	Very Low
C15*	1.4	87%	Low	Very Low
C16	1.3	85%	Low	Low
C17*	0.7	8%	High	Low
C18*	1.1	65%	High	Low
C19	1.2	75%	Intermediate	Low
C20*	0.8	13%	High	Low
C21*	1.4	74%	Intermediate	Low
C23	2.4	91%	Low	Low
C24	2.7	93%	Low	Low
C25	3.2	95%	Very Low	Very Low
C26*	1.7	83%	Low	Very Low
C28	2.4	91%	Low	Low
C29	3.6	95%	Very Low	Very Low
C30	2.0	88%	Low	Low
C31	3.5	95%	Very Low	Very Low
C32	2.7	93%	Low	Low
PILLAR GROUPS				
C23+C24	2.9	94%	Low	Low
C32+C33	3.1	94%	Low	Low
C10*+C13	2.7	93%	Low	Low
C13+C16	3.0	94%	Low	Low
C16+C19	2.7	93%	Low	Low
C17*+C20*	1.5	79%	Intermediate	Low
C16+C17* +C19+C20*	2.4	91%	Low	Low

Note: Pillars suffixed by an asterisk are 12m high

In the original analysis most of the pillars had a low or very low probability of failure. Those pillars that had a low probability of failure were targeted for further investigation during this study. Because of the reduced estimate of rock mass strength as established from the logging and testing of the borehole core, the reliability of some of the pillars has reduced slightly. Some of the pillars that were previously classified as having a very low risk of failure have now been classified as having a low risk of failure. Similarly some previously low risk pillars have been classified as being of intermediate or high risk.

Having had an opportunity to inspect most of the critical pillars SRK considers that the integrity of pillars having low and very low risk of instability do not in fact pose any significant threat to the future stability of the mine. However those pillars that have been downgraded to an intermediate or high risk of failure do deserve further consideration. They are dealt with in the following sections.

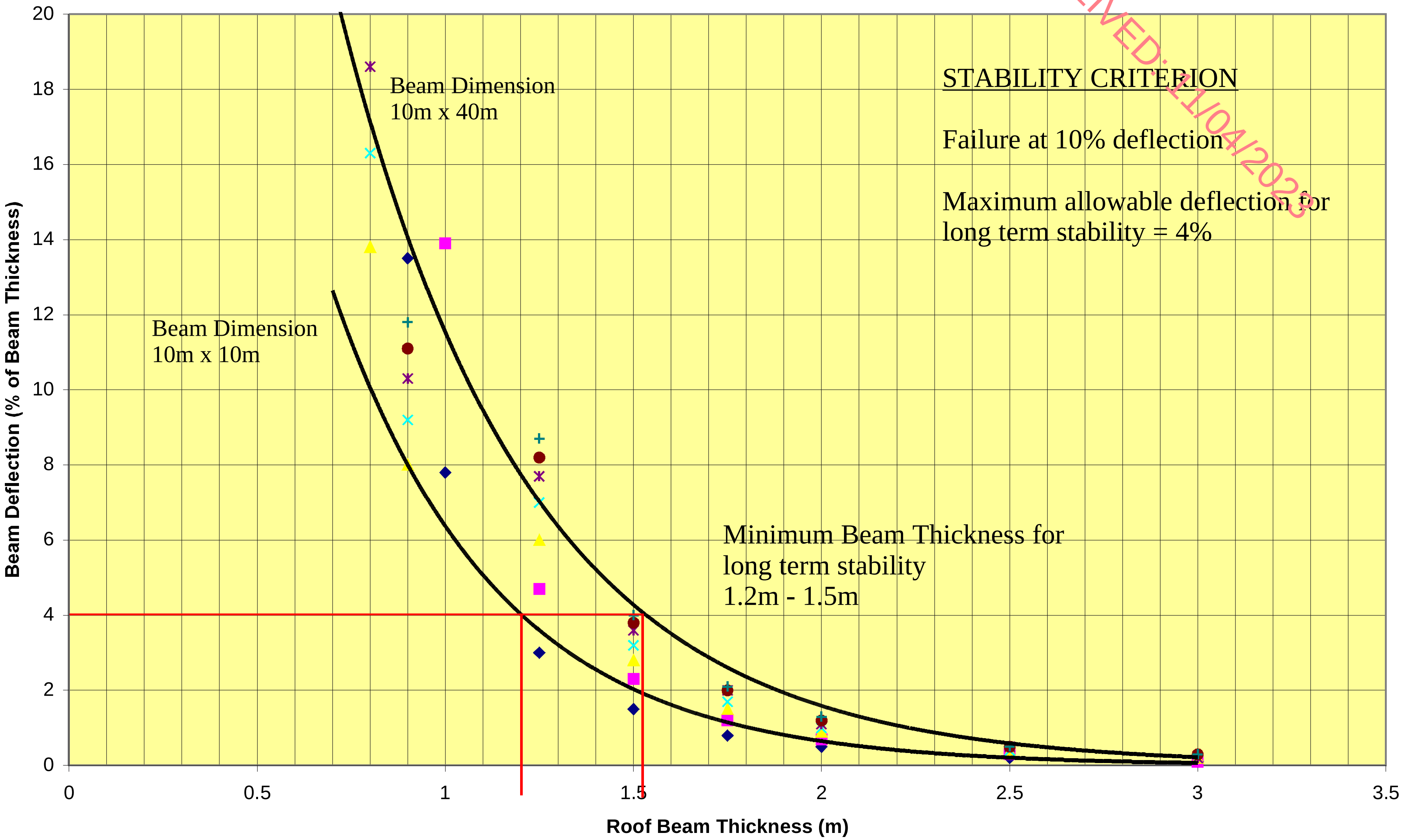
4.4 Roof Beam Stability Analysis

The original study considered the sensitivity of roof beam stability to roof beam thickness. A relationship between beam thickness and stability was developed and is shown in Figure 4.1. The relationship showed that for stability the roof beam should be no thinner than 1.5m and the analysis assumed that the average thickness of the roof beam was 3m. The analysis also showed that for the average excavation span roof beams greater than 3m in thickness would show little deflection. The thickness of gypsum intersected in the six probe holes varied from a minimum of 8m to in excess of 15m. The gypsum intersected demonstrated similar strength characteristics to that of the C gypsum used in the original analysis, which therefore remains valid. It can therefore be concluded that if the roof beam thickness remains in excess of 3m and up to the thickness intersected in the boreholes, then there is little probability of large scale roof beam failure that could give rise to surface subsidence.

4.5 Analysis of Pillar and Roof Beam Stability Using FLAC

An area of the underground mine between 50 and 80m to the north west of the community centre, underlying the football field, has been mined over a room height of 12m. The location of this area, which lies below land owned by Gypsum Industries is shown in Figure 2.3. In this area there is evidence of some pillar distress with slabbing of the pillar sidewall together with localised falls of ground from the roof and about 0.1 to 0.3m of floor heave. However the area appears to have remained largely stable since the area was mined possibly at least 20 years ago as none of the rockfalls appears to be fresh

The pillar stability analysis reported in Section 4.3 shows that the pillars should remain stable. In the long term (50 year +) it is possible that the pillars may deteriorate further increasing the risk of mine instability, which could cause the development of a localised surface subsidence feature. To confirm this SRK have used a more sophisticated analysis technique to investigate whether pillar failure could occur and lead to the development of a surface subsidence feature and whether such a subsidence feature could encroach on to the community centre.



December 2001	PROJ. Number U1598	GYPSUM MINE SUBSIDENCE	
SRK	THE QUEEN'S AWARD FOR EXPORT ACHIEVEMENT 1998	**Sensitivity of Roof Beam Stability to Beam Thickness**	Figure 4.1

The geotechnical numerical modelling code FLAC (Fast Lagrangian Analysis of Continua) has been used for this analysis. FLAC is a two-dimensional finite difference code, which realistically models the development of virgin stresses in a rock mass by the application of overburden loading. Various parts of the model can than be removed to simulate mining of underground openings. The code calculates the redistribution of stresses together with the accompanying strains and displacements in the rock mass created by the effects of mining. For the given rock mass strength parameters the code will establish areas where shear, tensile, compressive and plastic failure have occurred. By the use of history points the total displacement generated by mining can be calculated at any point in the model. The code uses an iterative procedure for analysis. Long-term stability of the model is reached when the forces in the model balance. If the model shows signs of failure than the model forces will remain unbalanced and the iteration process will continue indefinitely.

The pillar model for this analysis was created from one metre square elements. These elements were distorted to form the geological boundaries at the top and bottom of the gypsum and to form the underground rooms. The model was initialised with the strength parameters given in Table 4.4 and gravitational stresses were applied to the model. The underground rooms were then created and the model run until the forces within the model balanced out.

Figure 4.2 shows the location of the cross section of the mine analysed. Figure 4.3 shows the FLAC model used for the analysis, a 20m thickness of gypsum dipping at 5° to the west. The elevation of the mine workings was taken from the original mine survey plans. Exposure of Middle Mudstone in the roof of the excavations adjacent to the 12m pillars indicates a thinning out of the gypsum at these locations. Whilst the gypsum roof beam above the 6m high excavations is modelled as being about 8m thick, from the gypsum intersection in Borehole E, it has been reduced to between 1m and 1.5m in the roof above the 12m high pillars. The gypsum thickness in the floor was varied in the FLAC analysis to generate some floor heave in the model. Shown on the figure are the position of the Community Centre, borehole E and the locations on the ground surface where vertical displacement history points were placed. The history points were used to evaluate whether surface subsidence could occur as a result of the underground mining.

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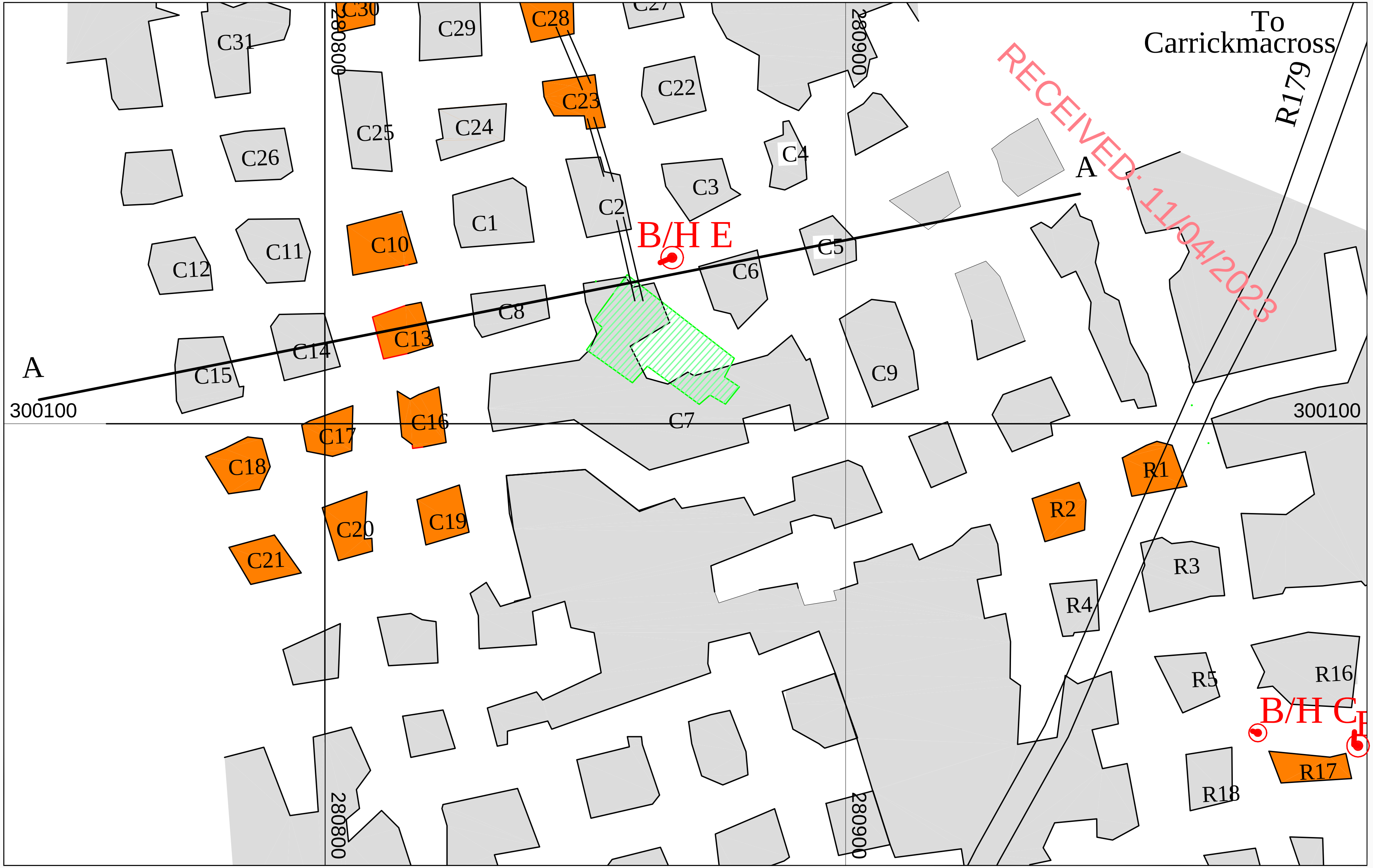

DEC 2001	PROJ. No: U1598	GYPSUM MINE SUBSIDENCE MONITORING	Scale 1:1000

SRK Consulting
Engineers and Scientists

PLAN POSITION OF FLAC MODEL

Fig 4.2

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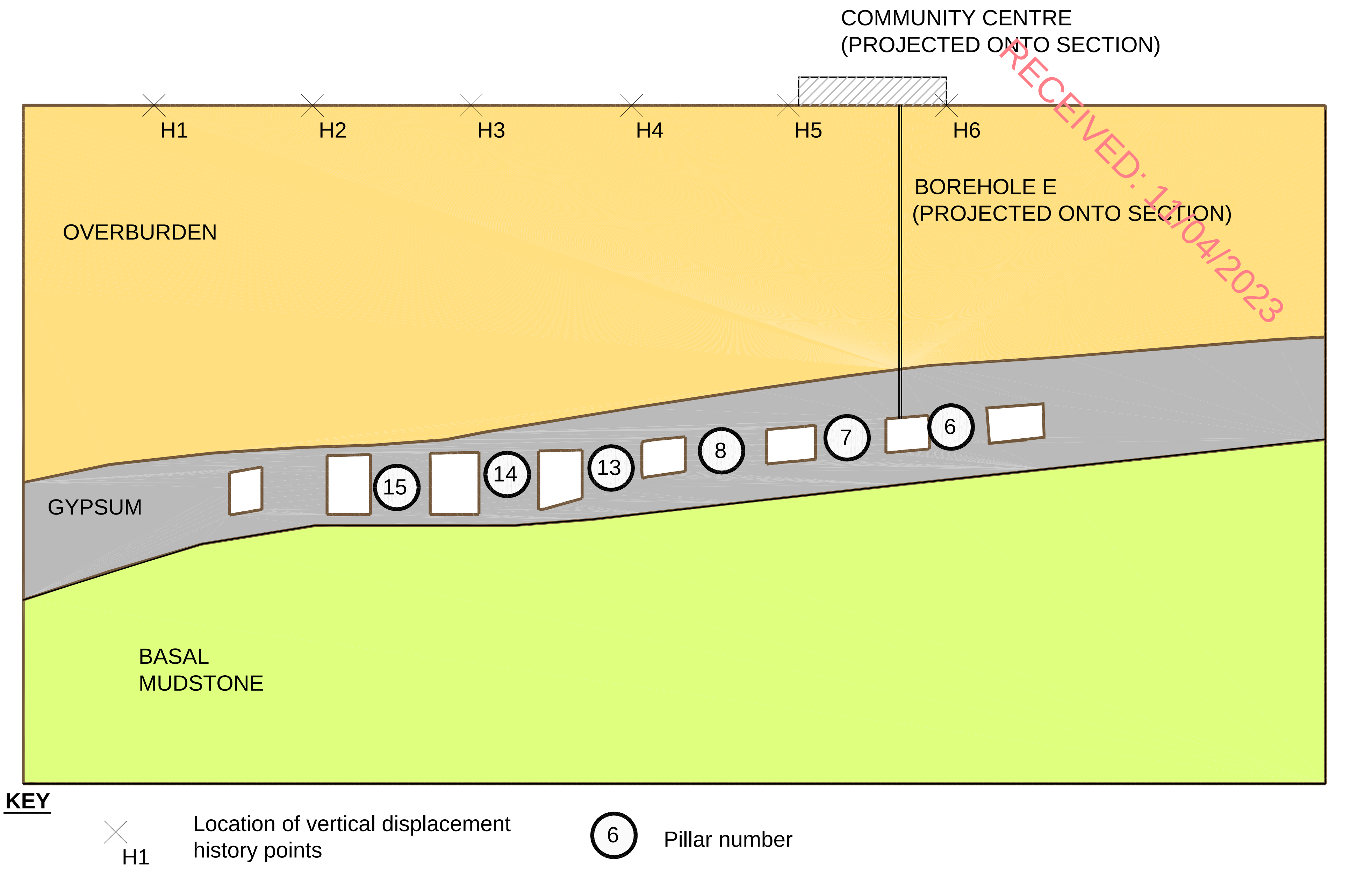

DEC 2001	PROJ. No: U1598	GYPSUM MINE SUBSIDENCE MONITORING	Scale 1:1000.
SRK Consulting Engineers and Scientists		MINE CROSS SECTION USED IN FLAC ANALYSIS	Fig 4.3

The results of the analysis are illustrated in Figure 4.4, Figure 4.5 and Figure 4.6. Figure 4.4 is a plasticity plot that shows the location of element within the model where mining induced stresses have exceeded the strength of the rock mass forming the model. Two forms of element failure are shown in the model. Tensile failure has occurred in the roof and floor of the 12m high excavation openings and shear failure has occurred in the pillar sidewalls and in the overburden. Shear failure is shown to be developing towards the ground surface along discrete zones on both the up-dip and down-dip sides of the 12m high opening. The shear zone on the down-dip side is more continuous than on the up-dip side. The development of such shear zones indicates the onset of subsidence. If these zones were to propagate to surface then a surface subsidence feature similar to that which occurred over the Drumgoosat village road could possibly develop. In this particular case however the FLAC model has achieved a state of stable equilibrium. The development of a subsurface subsidence feature for this particular underground geometry and rock mass parameters is unlikely to occur. It is of interest to note that if one projects the up-dip shear zone to surface along its angle of propagation it will not encroach on the Community Centre.

Figure 4.5 shows a close up view of the 12m high pillars and illustrates the roof, sidewall and floor movement around the excavation boundary by means of displacement vectors. These vectors are significantly magnified. The longest vector, which is in the roof, represents a roof displacement of 5.5cm. The upward movement of the floor is between 1 and 2cm.

Figure 4.6 shows the vertical displacements measured at each of the surface history points. Again the scale of the displacements has been greatly exaggerated. The maximum displacement of about 14mm occurs on surface immediately above the 12m high pillars. In the vicinity of the Community Centre vertical movement is between 4mm and 6mm giving a differential movement between each end of the building of 2mm. This magnitude of differential movement is unlikely to have significant structural effects on the Community Centre. There is unlikely to be any risk to users of the building. It is important to note that all the displacement graphs flatten out showing that a state of long-term equilibrium has been achieved.

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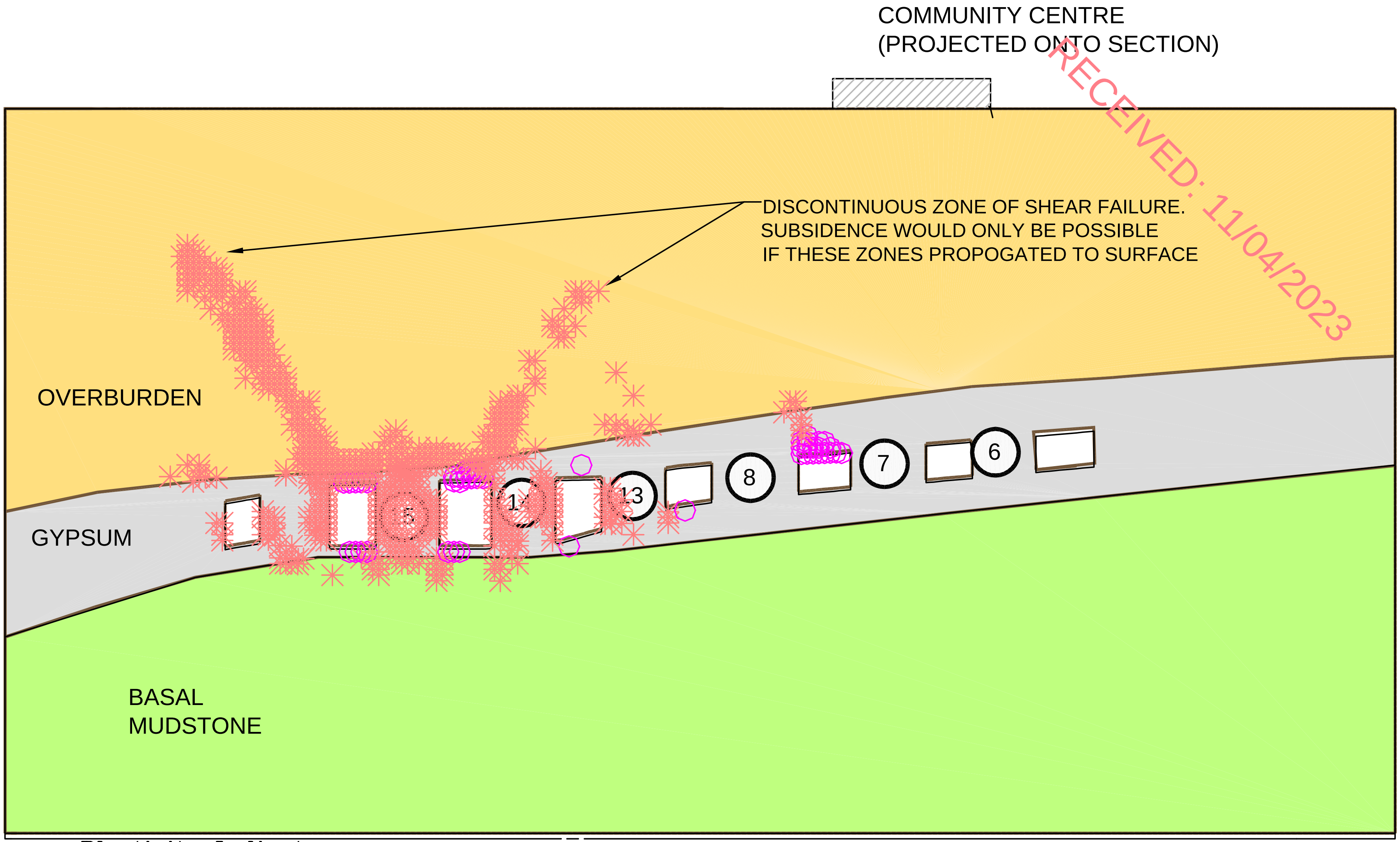

KEY Plasticity Indicator

* at yield in shear or vol.
o at yield in tension

6 Pillar number

DEC 2001	PROJ. No: U1598	GYPSUM MINE SUBSIDENCE MONITORING	Scale 1:1000.
SRK Consulting Engineers and Scientists		RESULTS OF FLAC MODELLING SHOWING LONG TERM STABILITY CONDITION	Fig 4.4

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JOB TITLE :

FLAC (Version 3.40)

LEGEND

27-Nov- 1 13:13
step 18
5.000E+01 <x< 1.200E+02
3.000E+01 <y< 1.000E+02

Boundary plot

0 20m

Displacement vectors
Max Vector = 5.5cm

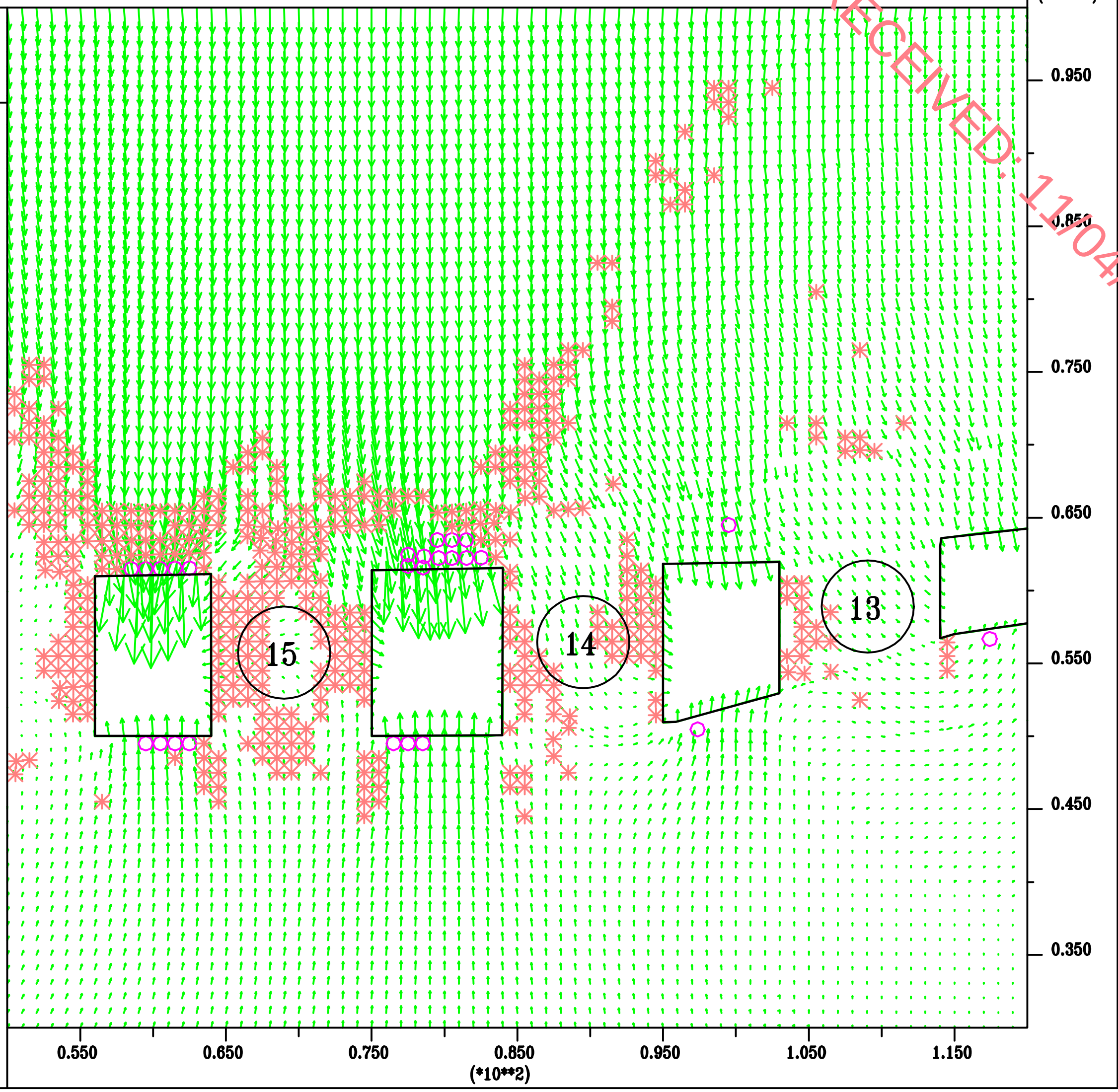
0 10cm

Plasticity Indicator
* at yield in shear or vol.
o at yield in tension

15 Pillar Number

Note:
The displacement vector scale is 200 times the model scale

SRK Consulting (UK) Ltd



DEC 2001	PROJ. No: U1598	GYPSUM MINE SUBSIDENCE MONITORING	
SRK Consulting Engineers and Scientists		DETAIL OF DISPLACEMENTS AND INSTABILITY AROUND 12m HIGH PILLARS	Fig 4.5

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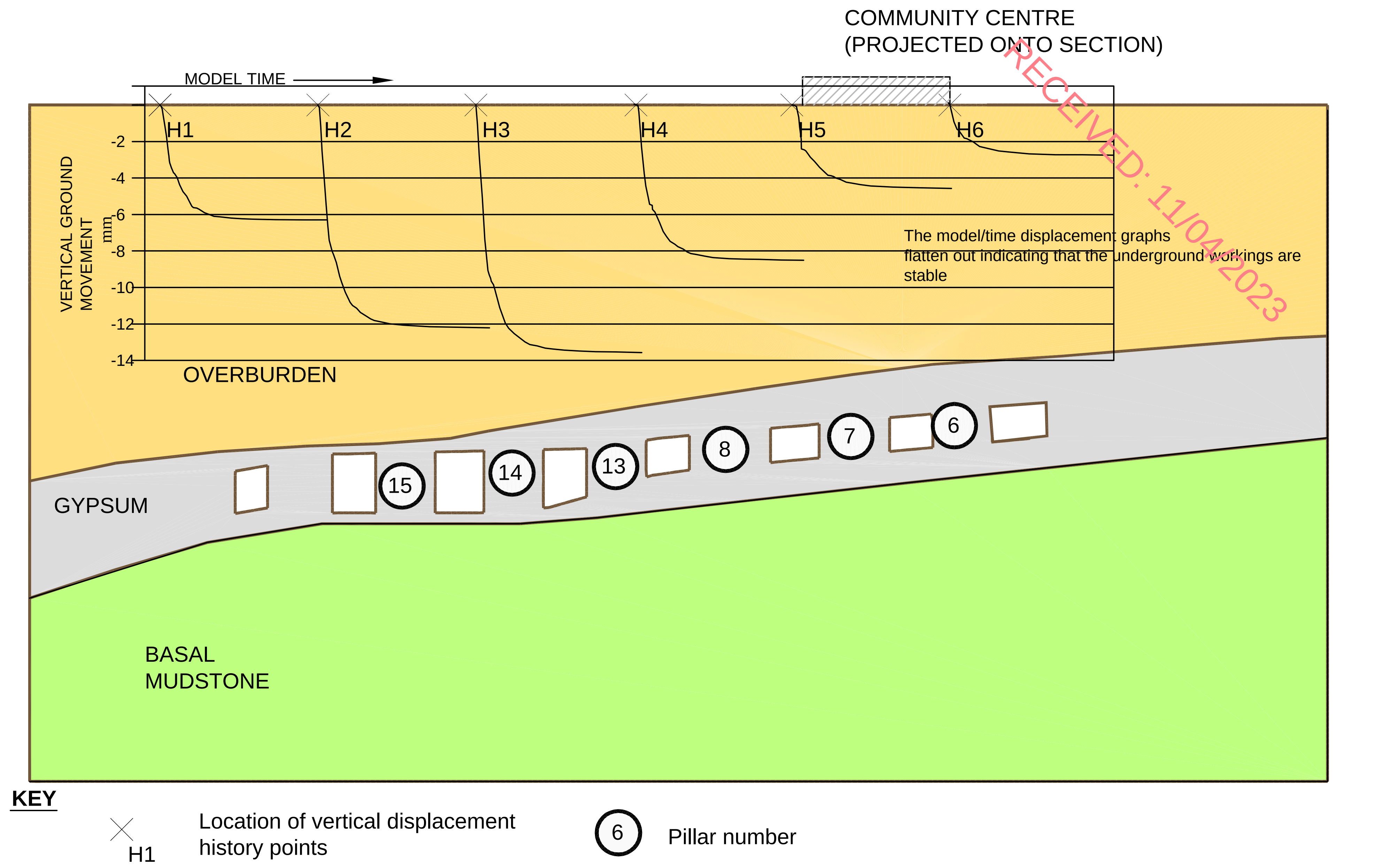


DEC 2001	PROJ. No: U1598	GYPSUM MINE SUBSIDENCE MONITORING	Scale 1:1000.
SRK Consulting Engineers and Scientists		SURFACE SUBSIDENCE PREDICTED BY FLAC MODEL	Fig 4.6

5 UNDERGROUND CHECK SURVEYS

5.1 General

The work undertaken by the survey crew involved –

1. Bringing independent survey control to the mine site from Ordnance Survey trig pillars.
2. Surveying the co-ordinates of the collars of the surface boreholes.
3. Surveying the underground holing points of the boreholes and calculating the co-ordinates of the holing points taking into consideration hole deviations determined from down the hole surveys undertaken by the drilling contractor.
4. Undertake surveys of the mine limits below the Maxwell and Martin properties as well as surveying the limit of the large pillar underlying the community centre using the holing points of the boreholes as survey control points.

5.2 Main Survey Control

Independent control was brought in to the site by means of a closed traverse from 2 OS trig pillars:

1. Drumcarrow TP, value E279592.786 N302305.157 Elevation unknown.
2. Coolreagh TP, value E288398.022 N303151.535 Elevation unknown.

The accuracy of the survey was as follows;

Linear misclosure = 15mm
Closing Error = 1 in 387,000

Local Scale Factor was not used in these calculations.

A common station was used on the main overburden mound, it was found that the difference between the mine co-ordinates and the co-ordinates calculated from the traverse were as follows;

E195mm N264mm.

5.3 Surface Borehole Check Position

Borehole positions were found to be as follows;

Borehole A - E280713.107 (.131) N300661.269 (.151)
Borehole B – E280714.196 (.268) N200680.915 (.864)
Borehole C – E280978.911 (.686) N300040.705 (.516)
Borehole D – E280998.272 (.025) N300038.680 (.486)
Borehole E – E280866.224 (.912) N300131.716 (.602)

Figures in brackets represent co-ordinates of boreholes as surveyed by the mine surveyor, as can be seen there appears to be a close correlation between the two sets of co-ordinates.

Although it was thought that borehole F had intersected the underground workings its holing point could not be seen. It appears that this borehole was stopped in a void some 10m above the underground workings.

5.4 Underground Borehole Positions.

The underground position of the boreholes was calculated from information on borehole deviations calculated from down-hole surveys undertaken by the contracted drilling company.

Using this information, the following positions for the boreholes have been calculated;

Borehole A - E280712.676 N300659.980
Borehole B – E280714.608 N200680.575
Borehole C – E280979.157 N300040.623
Borehole D – E280998.407 N300038.144
Borehole E – E280866.679 N300131.908

By comparing the collar and holing co-ordinates the deviation of the holes can be estimated. These are –

Borehole A - 1.359m to the north
Borehole B – 0.534m to the north-west
Borehole C – 0.259m to the west
Borehole D – 0.553m to the north
Borehole E – 0.494m to the west

5.5 Underground Traverse – Maxwell Property Area

The first traverse checked the working limit around boreholes A and B below the Maxwell property. This traverse closed to within an error of 1 in 11,300. As there were no other boreholes in the vicinity there was no additional checks to see if there was any rotation in the boreholes.

Plan Ref No LR-SRK-GMS-AREA 1 (Figure 5.1) shows the area-surveyed from these boreholes.

5.6 Underground Traverse – Martin Property Area

The second traverse, which checked the working limit below the Martin property, started on boreholes C and D and closed on to borehole E. This showed that there was a rotation of 2° 36' 45" between boreholes C and D. This rotation has been accounted for within the survey calculations.

Plan Ref No LR-SRK-GMS-AREA 2 (Figure 5.2) shows the area surveyed from this traverse.

5.7 Underground Traverse – Pillar Below Community Centre

The final traverse checked the limit around Pillar 7, the large pillar underlying the Community Centre. Part Plan Ref No LR-SRK-GMS-AREA 2, Figure 5.3, shows the results of this survey.

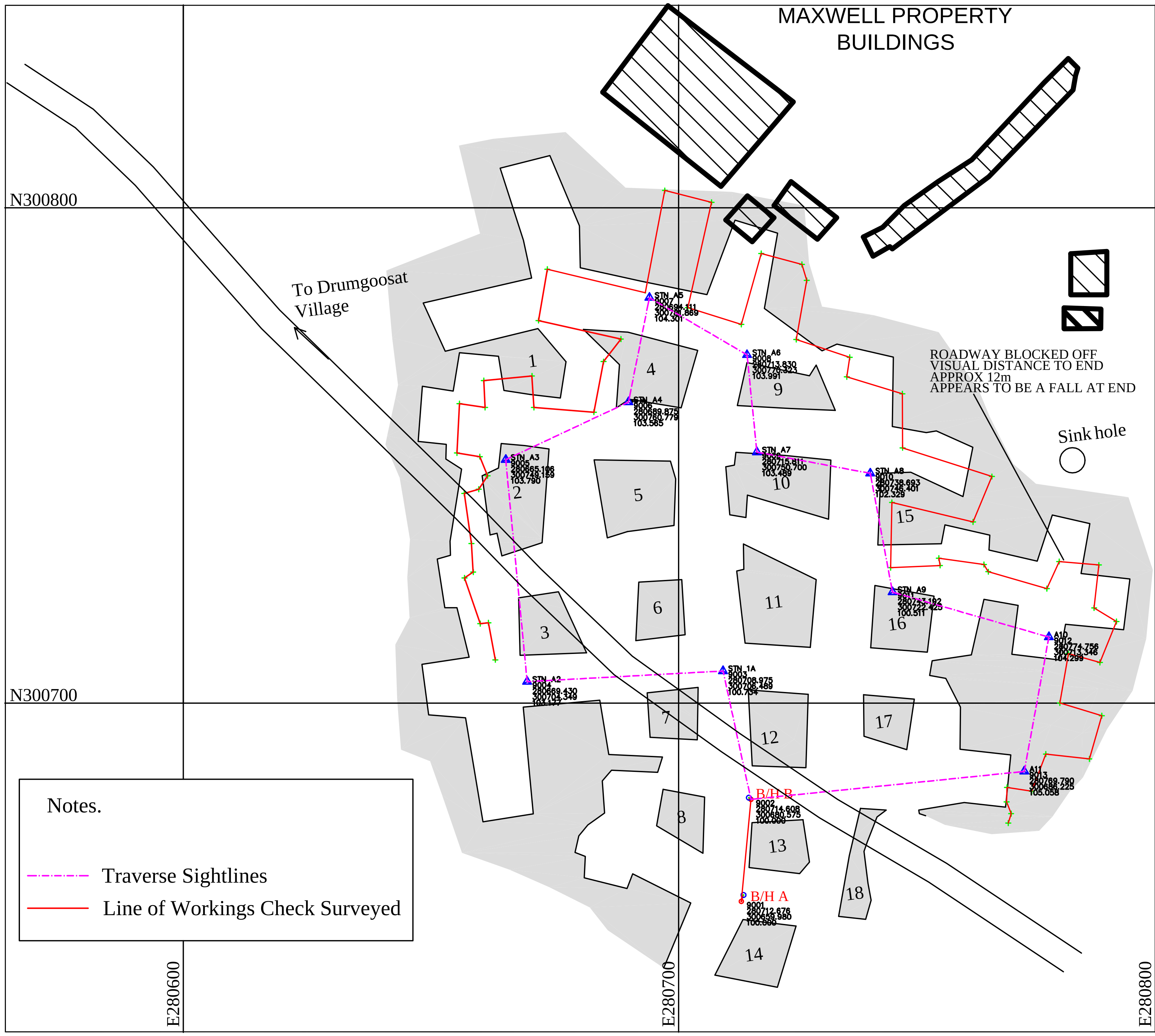
5.8 Conclusions of Check Surveys

From the results it is apparent that there are generally slight differences between the recorded workings on the original working plan and the workings surveyed during the period 5-9th of November 2001. This is to be expected particularly as the original mine limits were digitised from a 1:2500 paper copy of the original working plan.

There is one area on plan Ref No LR-SRK-GMS-AREA 1 (Figure 5.2) that shows one working out by approximately 14m. Close examination of the surface buildings may prove that the workings may have encroached closer than recorded.

It would be very hard to determine which plan is truly accurate, as the boreholes have been proved to deviate more than was expected. This would not have any effect upon the relative position of the boundaries.

Another factor that would highlight positional differences would be the angle of the cut faces. In most if not all, the sidewalls hang out by up to 3m. This therefore means that if the original survey was to the roof of the workings, then it is possible to have a 3m difference between the old and the new surveys, which was surveyed to the base of the pillars.



Plan Surveyed and Constructed By
LEE REYNOLDS & ASSOCIATES
25 Main Road
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Telephone 01639 701753
Mobile 07967 128964

On Behalf of

SRK Consulting
Windsor Court
1-3 Windsor Place
Cardiff
CF10 3BX

Job Title

U1598 - Irish Gypsum
Subsidence Risk Assessment

Drawing Title

MINE LIMIT BELOW
MAXWELL PROPERTY

Fig 5.1

Scale	1/1000		
Plan Constructed	October 2001	Drawn By	L.Reynolds

GRAPHICS\U1598\Fig5.1.dwg

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Job Title

U1598 - Irish Gypsum
Subsidence Risk Assessment

Drawing Title

CHECK SURVEY OF PILLAR BELOW COMMUNITY CENTRE

Fig 5.3

Scale 1/1000

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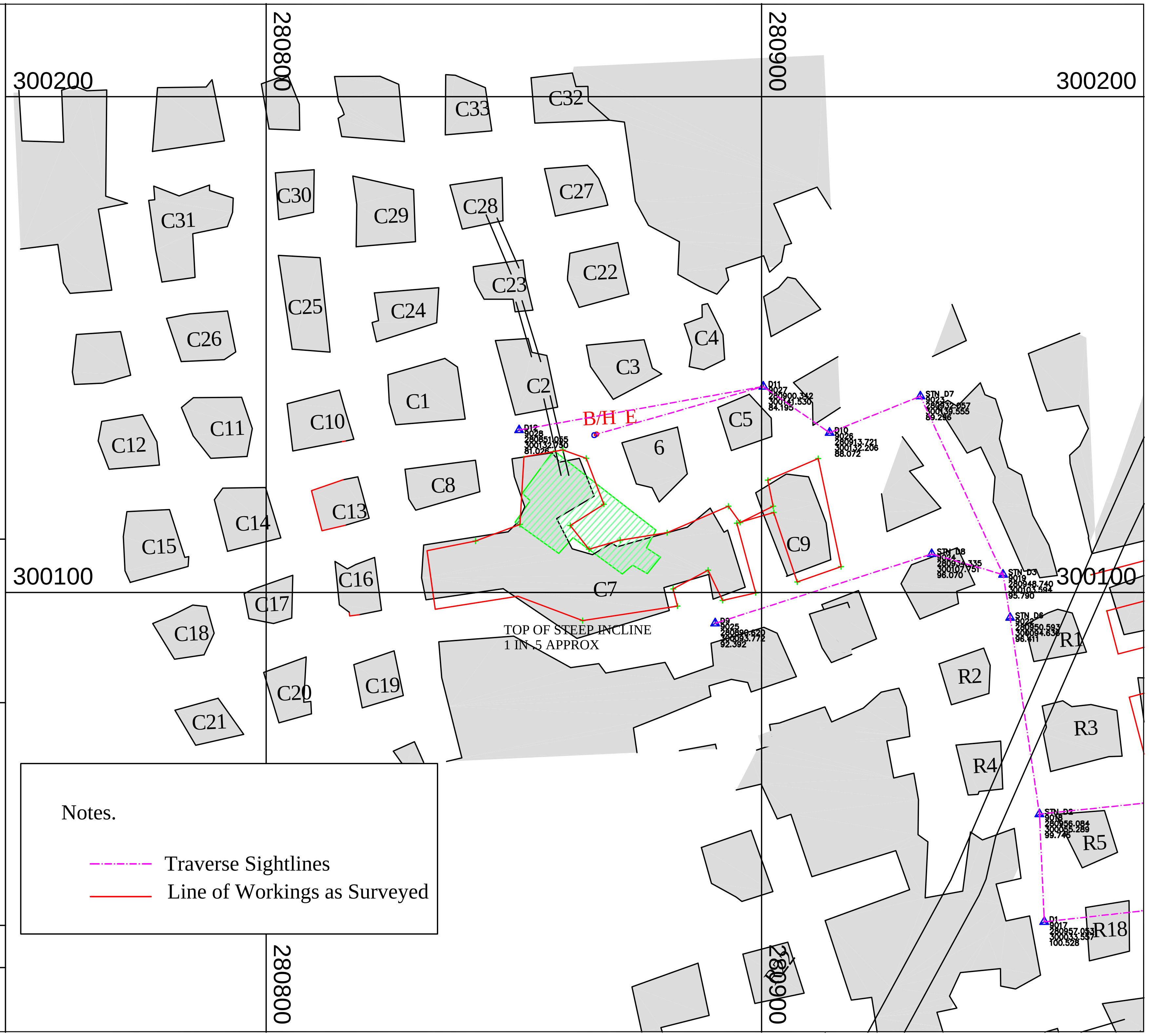

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Job Title

U1598 - Irish Gypsum
Subsidence Risk Assessment

Drawing Title

CHECK SURVEY OF PILLAR BELOW COMMUNITY CENTRE

Fig 5.3

Scale 1/1000

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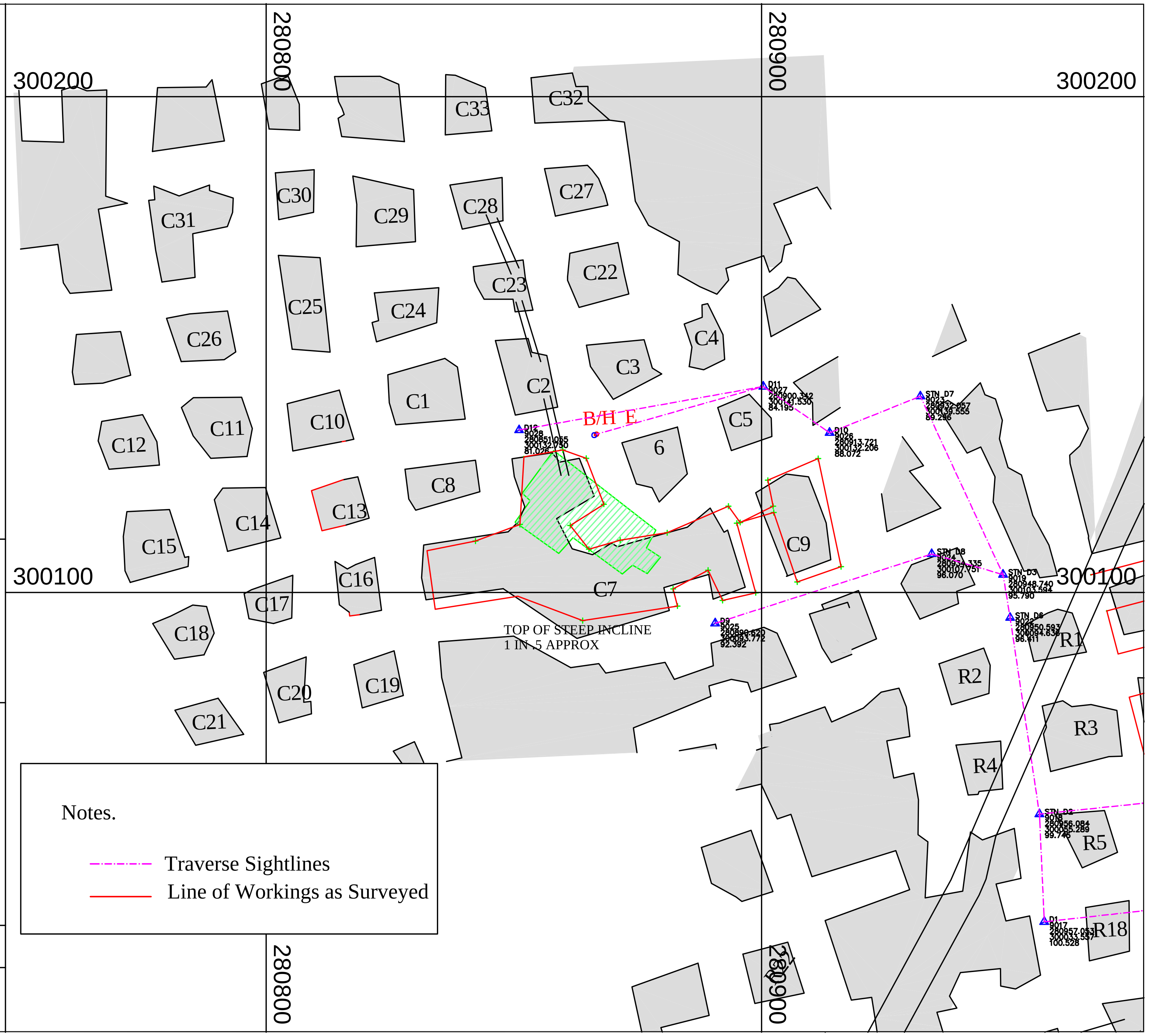

Notes.

Traverse Sightlines
Line of Workings as Surveyed

6 SUMMARY AND OVERALL CONCLUSIONS OF THE STUDY

6.1 Geotechnical Work

a) Detailed mapping of a number of the pillars identified as being of concern during the initial study was undertaken. All of the pillars identified as being of concern were visually inspected. The work has allowed SRK to update the geotechnical characteristics of the gypsum and to obtain a better understanding of the stability of the mine elements inspected.

b) The mine elements in most of the areas inspected were generally stable and appear to have been stable since mining was completed. Areas of discontinuity controlled instability have been noted around Pillars R17 and R17a, south east of the R179 road. The analysis has shown that the long term risk of failure of these pillars is low to very low.

c) A group of 12m high pillars has been identified some 50m to the west of the Community Centre and below land owned by Gypsum Industries. Elsewhere the pillars were less than 7m in height. The openings around the 12m high pillars are characterised by a limited amount of floor heave, by occasional roof falls and by sidewall spalling of the smaller diameter pillars. While these observations are consistent with potential development of mine failure there was no evidence of recent instability. This suggests that the mine elements have achieved a state of stable equilibrium. However because mudstone has been exposed in the roof adjacent to pillar C15 it is possible that with time a sinkhole could develop at this position.

d) The gypsum strength parameters determined from logging of the borehole core are slightly lower but less variable than those used in SRK's original analysis.

e) From the drilling it is concluded that, in general, the gypsum roof beam is more competent and thicker than the 3m assumed in the original analysis. This increases the estimated stability of the roof beam element and reduces the likelihood of roof beam failure. However in the area of the 12m pillars the gypsum roof beam is probably less than 3m thick.

f) Relating the actual pillar conditions observed underground to the relative risk of failure calculated from the probability analysis indicates that pillars with a low to very low risk of pillar failure show little sign of instability. Pillars analysed as having an intermediate or high relative risk of failure show varying degrees of instability. Future monitoring work should concentrate on the ground surface above these areas.

g) The more rigorous numerical modelling carried out using FLAC illustrated that it is unlikely that the 12m high pillars or roof beam will fail in the long term. The 1.5m thick roof beam above the workings, while showing some deflection, remains stable. This is consistent with the results of the roof beam sensitivity analysis. The pillars whilst showing some signs of distress also remain stable. This result is also consistent with the results of the pillar stability analysis. The model has predicted a maximum of 14mm of subsidence at surface. As the area appears to be stable it is possible that the subsidence has already occurred.

h) The areas identified from the analysis as being potentially at risk from long term mine element deterioration and surface subsidence are above pillars R8, R12 and R21 together with pillar groups R8, R23, R24 and R12, R25 and R24 adjacent to the road. 50m to 80m to the west of the Community Centre pillars C12 and C17 to C19 have been classified as pillars with an intermediate to high risk of failure.

6.2 Check Surveys

a) The check surveys in the vicinity of the Maxwell and Martin properties conclude that whilst there is some lateral discrepancy in the mine limit positions mining has not proceeded any further away from the Gypsum Industry ownership boundary than indicated on the original mine plans.

b) Check surveying of pillar C7, which directly underlies the Community Centre confirms that the original pillar survey is reasonably accurate.

7 RECOMMENDATIONS FOR FURTHER MONITORING

In SRK's 1999 study a relationship between risk, reliability, factor of safety and consequence of failure was proposed. The relationship was used to assign appropriate actions for subsidence monitoring based on SRK's understanding at that time of the underground geotechnical conditions and stability of the mine elements. This relationship is reproduced in Table 7.1 below.

The current study is a direct outcome of those original recommendations. As a result of this study SRK has now obtained more specific information on the underground geotechnical and mine conditions and, from the detailed mapping, observations and analyses carried out, has a better understanding of the gypsum mine stability below the R179 road and the area around the Community Centre. This has allowed the original risk relationship to be updated to reflect this increased understanding. The revised relationship is presented in Table 7.2 and forms the basis for the subsidence monitoring recommendations given below.

7.1 R179 Road

In the vicinity of the road, pillars R12 and R21 have been analysed as having a high risk of instability while pillar R8 has an intermediate risk of failure. Other pillars in the vicinity of the road have a low to very low risk of failure. From Table 7.2 the following monitoring actions are recommended.

Visual Monitoring

Visual inspections of the road and of the areas above pillars R8, R12 and R21 should be carried out every six months.

Survey Monitoring

Notwithstanding the fact that pillar R12 is bounded to the south-east by a large unmined buttress, in consideration of its small size and the fact that there is a large room under the road between pillars R12 and R11 it is recommended that a surface levelling point be installed above this room and surveyed every six months.

Because pillar R8 is surrounded by a stable pillar group and pillar R21 is some 30m away from the road it is considered that specific surface survey monitoring is not warranted for these areas.

Table 7.1: Relationship between Risk, Reliability and Factor of Safety used in the 1999 Subsidence Assessment of Drumgoosat and Drumgill Mines

OVERLYING STRUCTURES	PILLAR RELIABILITY	SAFETY FACTOR	RELATIVE RISK	DEGREE OF RISK	APPROPRIATE ATTITUDE TO RISK	ACTION
Farmland	80-100%	>1.5	Very low	Slight chance to unlikely	Little or no concern	Annual surface inspections
	70-80%	1.3 - 1.5	Low	Some risk	Cautious	Annual surface inspections
	<70%	<1.3	Intermediate	Some risk to risky	Cautious to concerned	Quarterly surface inspections
Roads and buildings	95-100%	>3.0	Very low	Slight chance to unlikely	Little or no concern	Quarterly surface inspections
	80-95%	1.5 - 3.0	Low	Some risk	Cautious	Monitoring of surface levels at six month intervals; quarterly surface inspections; initial underground inspection
	70-80%	1.3 - 1.5	Intermediate	Some risk to risky	Cautious to concerned	Monitoring of surface levels at quarterly intervals; quarterly surface inspections; initial underground inspection
	<70%	<1.3	High	Risky to very risky	Concerned to very concerned	Further investigation; monitoring of surface levels at quarterly intervals; quarterly surface inspections; annual underground inspections of individual key pillars

Table 7.2: Revised Relationship between Risk, Reliability and Factor of Safety for the 2001 Subsidence Assessment of Drumgoosat Mine

OVERLYING STRUCTURES	PILLAR RELIABILITY	SAFETY FACTOR	RELATIVE RISK	DEGREE OF RISK	APPROPRIATE ATTITUDE TO RISK	ACTION
Farmland	70-100%	>1.3	Low to Very low	Slight chance to unlikely	Little or no concern	Annual surface inspections
	<70%	<1.3	Intermediate to High	Some risk	Cautious	Six monthly surface inspections
Roads and buildings	80-100%	>1.5	Low to Very low	Slight chance to unlikely	Little or no concern	Six monthly surface inspections
	70-80%	1.3 - 1.5	Intermediate	Some risk	Cautious	Monitoring of surface levels at six month intervals; quarterly surface inspections
	<70%	<1.3	High	Some risk to risky	Cautious to concerned	Monitoring of surface levels at quarterly intervals; quarterly surface inspections

7.2 Community Centre

Areas identified as having an intermediate to high risk of failure lie between 50m and 80m to the west of the Community Centre and lie below open ground. The computer analyses have indicated that the potential for subsidence is low. From Table 7.2 the following monitoring actions are recommended.

Visual Monitoring

The area immediately surrounding the Community Centre and above pillars C12 and C17 to C21 should be visually inspected every six months for signs of subsidence.

Survey Monitoring

No specific survey monitoring is required at this time.

7.3 Review of Monitoring Recommendations

The results of the six monthly monitoring should be reviewed and advice sought from a geotechnical engineer if any unusual surface conditions are observed. The monitoring recommendations should be formally reviewed by a geotechnical engineer after 2 years and modified as required based on the monitoring results.

8 CONCLUDING REMARKS

In the underground areas below the Community Centre and R179 road that SRK inspected in detail, while there was some evidence of instability, there was no evidence of active or recent failure of the underground workings. Providing that there is no significant time related deterioration in the condition of the pillars and roof beams SRK considers that the long-term risk of large-scale subsidence affecting these areas is low.

Because Mr Maxwell's and Mr Martin's properties do not directly overly underground workings the risk of subsidence affecting these properties is considered to be low to very low.

For and On Behalf of Steffen, Robertson and Kirsten (UK) Ltd

Neil Marshall MSc MIMM CEng
Principal Geotechnical Engineer

Ian Gregson BSc FGS
Senior Geotechnical Engineer

Appendix A
Terms of Reference and Scope of Work

Appendix B

Pillar Photographs

Appendix C
Borehole Logging and Underground Mapping Data Sheets

SRK CONSULTING

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APPENDIX 7.4
Drumgoosat Subsidence Event - Technical Report - SRK - October 2018

DRUMGOOSAT SUBSIDENCE EVENT - TECHNICAL REPORT

Prepared For

Saint-Gobain Mining (Ireland) Ltd

Report Prepared by

srk consulting

SRK Consulting (UK) Limited

UK30238

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EXECUTIVE SUMMARY
DRUMGOOSAT SUBSIDENCE EVENT - TECHNICAL REPORT

1 REPORT SUMMARY

1.1 Background

SRK has carried out a back analysis to determine the cause of the subsidence event that occurred under the GAA playing fields and community centre on 23 September 2018. The back analysis was based on geotechnical characteristics of the mine rock mass developed by SRK during studies related to subsidence risk undertaken on behalf of the EMD between 1999 and 2004. The reports and data generated by these studies are included as an Appendix to this document.

Based on the outcome of the back analysis SRK has also carried out predictive analyses to consider whether the GAA subsidence event has impacted on other areas of the mine overlain by 3rd party surface infrastructure and what the potential risk of mine failure is at these locations based on SRK's current understanding of underground mine conditions.

1.2 Back Analysis

The back analysis was carried out using the finite element modelling software RS2, produced by Rocscience Inc. One west-east and one north south cross section cut through the centre of the subsidence depression were used in the back analysis. Sensitivity analyses were carried out by varying the strength of the gypsum pillars, the strength of the basal mudstone, the mine water condition and the pillar height. The back analysis concluded that:

1. Surface disturbance of the magnitude experienced would not have occurred in a dry mine in deteriorating strength conditions if this area was comprised of 6 m high pillars.

2. When the strength of these pillars was reduced to simulate long term time dependent deterioration pillar failure might have occurred but of a magnitude significantly lower than that experienced in this event. This indicates that the 12 m high pillars in a dry mine would not have given rise to surface subsidence.

3. The measured magnitude and extent of the disturbance zone reasonably accurately simulated by the finite element model for mine conditions where the water level in the mine has risen to the 993mL and the gypsum and basal mudstone strength have deteriorated by 10% to 20% below the minimum gypsum strength measured during SRK's underground mapping campaigns.

4. The introduction of excess water into previously dry sections of the mine which contain the 12 m high pillar group has reduced the strength of the gypsum pillars and underlying mudstone to such a degree that failure of the mine elements occurred, creating a major disturbance feature on surface. This situation has been exacerbated as in order to

create the 12m high pillars the floor of the rooms was excavated such that the gypsum floor beam was reduced in thickness making them unstable and subject to cracking.

The back analysis concluded that the GAA playing field failure was a result of the interaction of three unique conditions:

a) 12m high pillars,

b) Mine water levels increasing and submerging the pillars, and

c) A thin gypsum floor beam.

If any one of these conditions was missing, then a mine failure of this nature would not have occurred.

1.3 Predictive Analysis

- A series of predictive analyses were carried out on areas of the mine that are overlain by 3rd party surface infrastructure. These included the L4900 and R179 roads and properties close to the edge and below the mine owned by Mr Martin, Mr and Mrs Maxwell, Mrs Kiernan. Mr and Mrs Ward and Mr and Mrs Rafferty.
- Apart from Mr Rafferty's property, which lies within the GAA playing field exclusion zone, none of these areas have been affected by the GAA playing field subsidence and none of these areas contain all three conditions that were determined to have caused the GAA playing field subsidence.
- The pillar strength range used in the predictive analysis was determined from SRK's underground mapping campaign and the strength reduction required to cause the historical instability that has occurred below the eastern end of the L4900 road. This range defined an average and a weakened gypsum pillar strength which was used to assess the stability of the mine.
- The finite element modelling predicted that there is no mine pillar failure at either average or weakened strength along any of the cross sections analysed. Predicted surface movement is of the order of millimetres for both cases. This magnitude of movement is consistent with the results of surface survey levelling conducted by the mine that has been continuing along and adjacent to these infrastructures for the past 18 years or so.
- There are several small pillars adjacent to and underneath the R179 road. These were modelled and showed no signs of instability.
- Based on the results of this analysis SRK considers the risk of mine failure affecting these infrastructures is very low. This is consistent with the results of the subsidence risk assessments carried out by SRK between 1999 and 2004 and indicates that the level of risk has not increased over the last 14 years and has not been affected by the GAA playing field subsidence.

1.4 Recommendations

- The monitoring that has been in place for the last 18 years should continue, In the short term the frequency of monitoring should be increased to provide comfort that the GAA playing field failure is not influencing these areas. Monitoring frequency can be reduced over time if the results show no increase in subsidence.
- It will also be essential that the mine water level is not allowed to increase further. SGMI have committed to implement a mine dewatering programme in the short term.
- Deterioration in pillar strength will be higher the smaller the pillars are. Whilst the analysis has shown that the small pillars below the R179 road are stable it has been agreed that a borehole camera will be dropped into the underground workings close to the pillars to determine their current condition and confirm the results of these analyses.
- Whilst Mr Rafferty's property currently lies within the GAA playing field exclusion zone and there is visible damage to the house a structural engineer should carry out a survey of the property. SRK understands that this has already been done.

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DRUMGOOSAT SUBSIDENCE EVENT - TECHNICAL REPORT

1 INTRODUCTION

1.1 Background

SRK Consulting (UK) Limited ("SRK") is an associate company of the international group holding company, SRK Consulting (Global) Limited (the "SRK Group"). SRK has been requested by Saint-Gobain Mining (Ireland) Ltd ("SGMI", hereinafter also referred to as the "Company" or the "Client") to carry out an analysis of subsidence event at the Company's abandoned underground gypsum mine, Drumgoosat, located near Kingscourt, Ireland.

SRK has had geotechnical involvement with the SGMI underground mines since 1998. Between 1999 and 2005, SRK, on behalf of the Exploration and Mining Division of the Department of Communications, Climate Action and Environment ("EMD"), undertook specific investigations and analyses of areas of the mine underlying third-party surface infrastructure following concerns expressed by members of the local community on the stability of these areas.

The work undertaken by SRK in 1998-1999 comprised an independent subsidence risk assessment of the whole mine, with specific reference to the stability of the mine pillars under the area of the L4900 road that had been subject to subsidence in the late 1990s.

SRK was subsequently commissioned by the EMD to carry out more detailed subsidence risk assessments of the stability of the mine below:

- The Community Centre and playing fields, the R179 road, and properties owned by Mr Maxwell and Mr Martin (2001).
- The property owned by Mr Rafferty (2004).

This work concluded that the pillars and mine below these areas were strong enough to ensure long term stability. They were unlikely to collapse and cause any damage to surface infrastructure if there was no material change to the mine conditions and strength of the pillars at the time they were initially examined by SRK. SRK recommended a monitoring program which comprised strategically placed levelling points on surface over the areas studied. These were installed by SGM and have been surveyed on a regular basis. Copies of these reports are appended to this document (Appendix D) and listed below in Table 1-1.

Table 1-1: SRK Historical Drumgoosat Reports

Appendix	SRK Project Number	Date	Report Title
D1	U1225	May 1999	Report on Subsidence at Former Underground Gypsum Mines near Kingscourt, Co. Cavan, Ireland
D2	U1225	May 1999	Report on Subsidence at Former Underground Gypsum Mines near Kingscourt, Co. Cavan, Ireland – Executive Summary Report
D3	U1598	March 2002	Check Survey and Geotechnical Inspections at Drumgoosat Disused Mines, Co Monaghan
D4	U1598	March 2002	Check Survey and Geotechnical Inspections at Drumgoosat Disused Mines, Co Monaghan - Executive Summary Report
D5	U2171	January 2005	Underground Survey and Geotechnical Inspections Below Mr Rafferty's Properties at Drumgoosat Disused Mines, Co Monaghan
D6	U2771	May 2005	Drumgoosat Mine – Review of Surface Monitoring Data

Since 2005, SRK had no further involvement with the Drumgoosat Mine, but in 2006 SRK was commissioned by SGMI to undertake annual regulatory geotechnical audits of the operating Drummond Mine, the latest of which was undertaken in November 2017. The latest audit concluded that there were no areas of geotechnical concern in Drummond Mine.

1.2 Terms of Reference and Scope of Work

This report describes the results of the back analysis of the Drumgoosat Mine subsidence event that occurred on Sunday 23 September 2018. This document is a follow up to the preliminary assessment report that was issued on 26 September 2018 and describes the analysis work that has been undertaken to test the failure hypothesis described in the preliminary assessment report. This work has been carried out with reference to the underground geotechnical assessments carried out by SRK in 2001 with a follow up inspection in 2005.

It also includes a predictive analysis of the stability of the mine located below and adjacent to third party property and infrastructure, namely:

- The undermined areas of the R179 and L4900 roads.
- The areas below the properties belonging to:
    - Mr Martin;
    - Mr and Mrs Maxwell;
    - Mrs Kiernan;
    - Mr and Mrs Ward; and
    - Mr and Mrs Rafferty.

2 MINE CONDITIONS

2.1 Mine Water Level

The failure has occurred above a mine area that contains a group of 12 m high pillars. The hypothesis is that the pillar integrity has deteriorated over time. The failure catalyst has been the introduction to the mine of excess water being pumped out of the operating Drummond Mine. Whilst the Drumgoosat Mine has historically been a flooded mine, the water level has been controlled by pumping excess water such that the level of water in the mine is no higher than the 970m level ("970mL") (note that mine elevations add 1,000 m to natural ground elevation to eliminate negative elevations below surface). With the introduction of excess water from Drummond into Drumgoosat, the water level has increased to above the 990mL. At the time of the subsidence event, the mine water level stood at the 993mL. The base of the lowest 12 m high pillars is at the 980mL.

2.2 Geotechnical Conditions and Ground Movement

In 2001, SRK undertook detailed geotechnical inspections of the 12 m high pillars which were formed by floor cutting of the original 6 m high rooms. The condition of the 12 m high pillars was described by SRK as follows:

'The openings around these (12 m high) *pillars show signs of floor heave, roof falls and sidewall spalling of the smaller size pillars. While these types of instability are consistent with potential development of mine failure there was no evidence of fresh instability. This indicates that the mine elements have achieved a state of stable equilibrium.'*

The floor heave described was consistent with the pillar bedding down into the underlying basal mudstone.

SRK recommended that precise levelling survey points were installed on surface directly above the rooms surrounded by the 12 m high pillars. These were installed by the mine and SRK was invited back to carry out a further underground inspection and review the results of the surface monitoring in 2005.

In its 2005 report, SRK noted:

'The 12 m high pillars showed some signs of deterioration with evidence of tensile cracking and slabbing along the faces of the pillars. Some minor cracking of the floor beam was observed which looked relatively fresh. There was some minor exposure of mudstone in the roof beam. In general, the roof was in a stable condition. SRK considers that apart from the floor beam cracking there has been little further deterioration in this area since SRK's original inspection which was carried out in October 2001.'

Figure 2-1 is a graph of ground movement in mm/year versus time for monitoring point M1, which is located immediately above the central room of the 12 m high pillar group. The location of this monitoring point is shown in Figure 3-1. The graph shows that, apart from a period between 2004 and 2006, ground movement has been relatively constant at 10 mm/year or less. The onset of mine instability would be indicated by an increasing velocity with time, which is not indicated by the monitoring results.

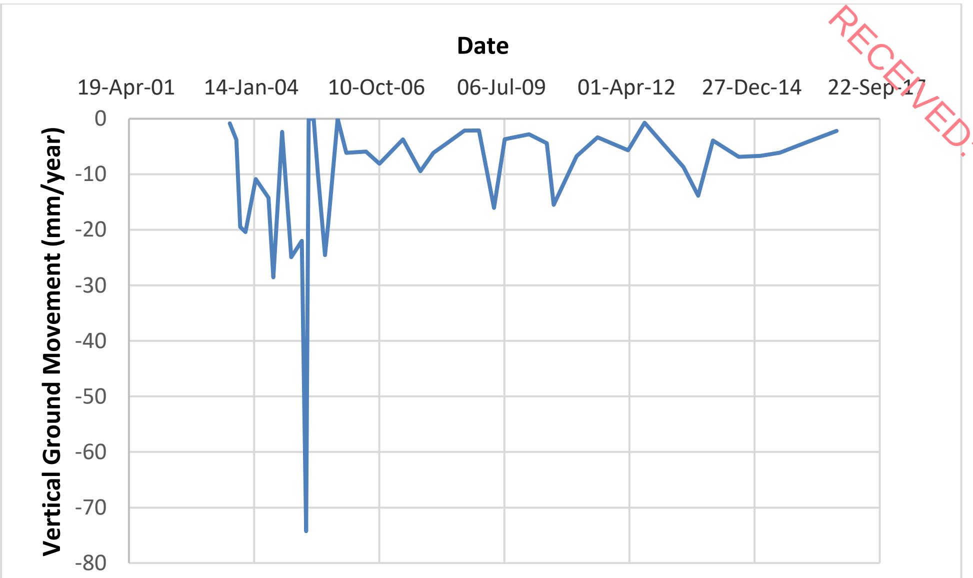

Figure 2-1: Vertical Movement on Surface in mm/year above 12 m high pillar group

2.3 Rock Strength

As part of its 2001 study, SRK commissioned a rock strength testing programme, a cored borehole drilling programme, and carried out a visual assessment of the 12 m high pillars along with geotechnical mapping of several accessible 6m high pillars. Using the results of this work, representative rock mass strength and deformation properties were developed for the gypsum, the mudstone, and the overlying overburden material. These strength parameters are shown in Table 2-1, were used for the original FLAC numerical modelling reported in 2001 and are now used as a starting point for the back analysis reported herein.

Table 2-1: Rock Mass Strength Parameters Used in Numerical Modelling

Rock Unit	Overburden	Gypsum	Mudstone
Intact Strength (MPa)	1	15	5
m_i	9	12	9
GSI	40	40	40
Material Density (t/m^3)	2.0	2.3	2.0
Cohesion (kPa)	60	200	130
Friction Angle (°)	19	43	30
Rock Mass Deformation Modulus (MPa)	560	4875	1260
Poisson's Ratio	0.3	0.15	0.3

An empirical rock mass strength criterion called the Hoek-Brown criterion was used to define the cohesion and friction angle of the rock masses. The Hoek-Brown criterion is an industry standard method for estimating the strength of bedded and jointed rock masses. The main inputs to this criterion are the intact rock strength and the Geologic Strength Index ("GSI"). The GSI is derived from an assessment of not only the rock strength, but also the impact that bedding, joints and weathering have on reducing the rock strength. The GSI number ranges from 100 to 0, with a value of 100 indicating an intact rock with no jointing and values approaching zero representing very broken rock with little strength.

The GSI value used for the original Drumgoosat rock mass numerical modelling was 40. This is a conservative value based on the lowest characteristic strength observed underground. This value is defined as a lower bound strength value, that is the value is highly unlikely to become lower than this unless there is a material change to underground conditions.

A GSI of 40 indicates the significant impact that bedding and jointing has on reducing the intact strength of the rock and accounts for the observed degradation of the pillars over time, from the time the pillars were created to the time they were inspected. Although jointing and bedding is not explicitly modelled in these analyses, they have been accounted for in the value of GSI used. A reduction in GSI value can also be used to simulate the impact that water has on reducing the rock mass strength.

Figure 2-2 shows the GSI Chart. The orange circle shows the GSI chosen by SRK as a starting point for numerical analysis and relates this value to the fracture and weathering condition of the gypsum in the pillars.

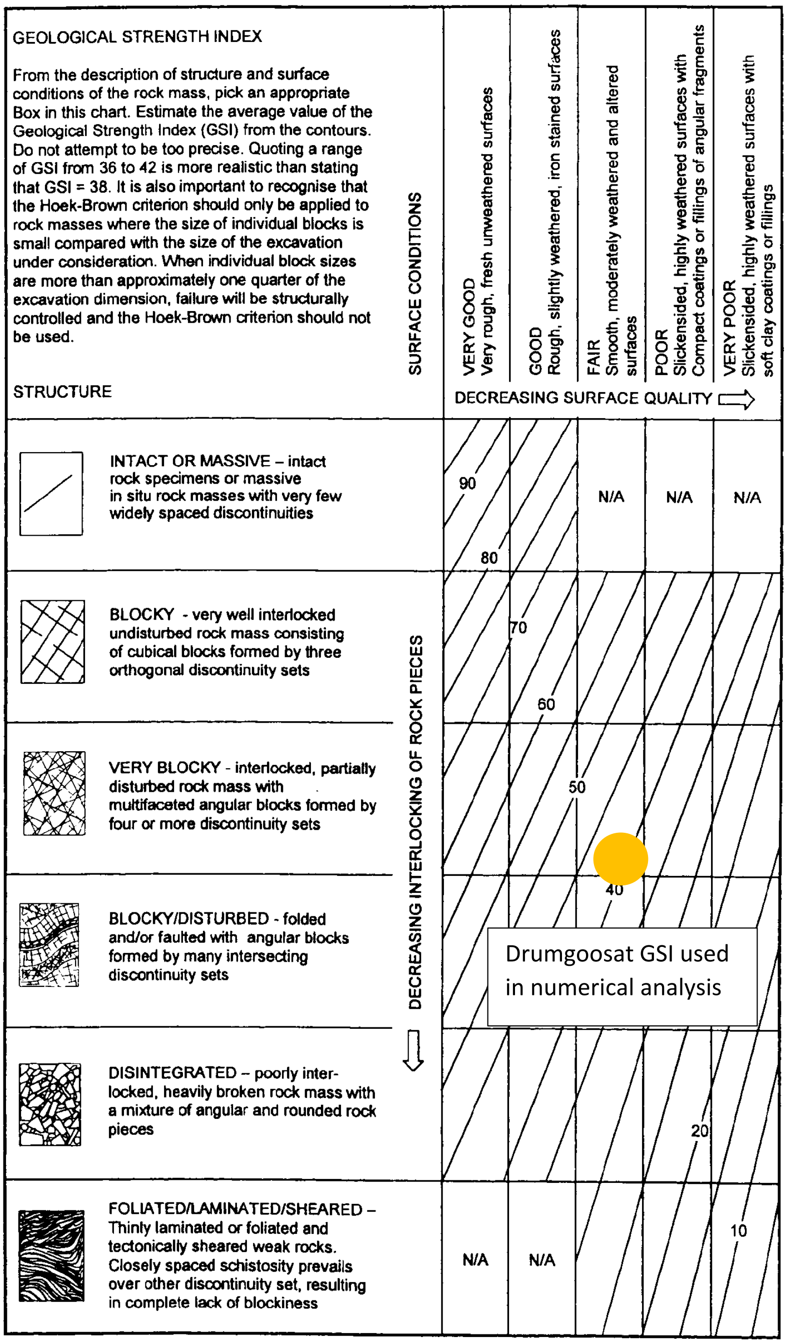

Figure 2-2: GSI Estimation Chart

As part of its various studies in Drumgoosat, in addition to making visual estimates of the gypsum engineering characteristics associated with the 12m high pillars, SRK has also generated GSI data from borehole logging and mapping of a selection of the 6m high pillars, details of which are provided in the historical reports in Appendix D.

A total of 27 gypsum GSI values have been generated from around the mine. The statistics from these values are presented in Table 2-2.

Table 2-2: Gypsum GSI Variability

Data Source	Average GSI	GSI Range
12m High Pillars	48	43 – 52
6m High Pillars	53	41 – 62
Borehole Core	53	45 - 73
GSI used in these Analyses	40	

The analyses reported herein have used the lowest GSI value that has been rounded down to the nearest decile.

Figure 2-3 show graphs of gypsum cohesion and friction angle values generated by the Hoek-Brown criterion for the range of GSI values presented in Table 2-2. On the graph are shown:

- the friction angle and cohesion for the GSI used in this report (orange line),
- the friction angle and cohesion for the average measured GSI of the 12m high pillars (grey line), and
- the friction angle and cohesion for the average measured GSI of the 6m high pillars (yellow line).

The graphs show that:

1. The cohesion of the gypsum forming the 12m high pillars using the average GSI of 48 is 21% higher than that used for the analyses reported herein.
2. The cohesion of the gypsum forming the 6m high pillars using the average GSI of 53 is 39% higher than that used for the analyses reported herein.
3. The friction angle of the gypsum forming the 12m high pillars using the average GSI of 48 is 7% higher than that used for the analyses reported herein.
4. The friction angle of the gypsum forming the 6m high pillars using the average GSI of 53 is 11% higher than that used for the analyses reported herein.

Based on this assessment SRK notes that the strength parameters used for the analyses are considered to represent a ‘worst-case’ strength scenario and are unlikely to reduce further providing there is no material change to the underground mine conditions.

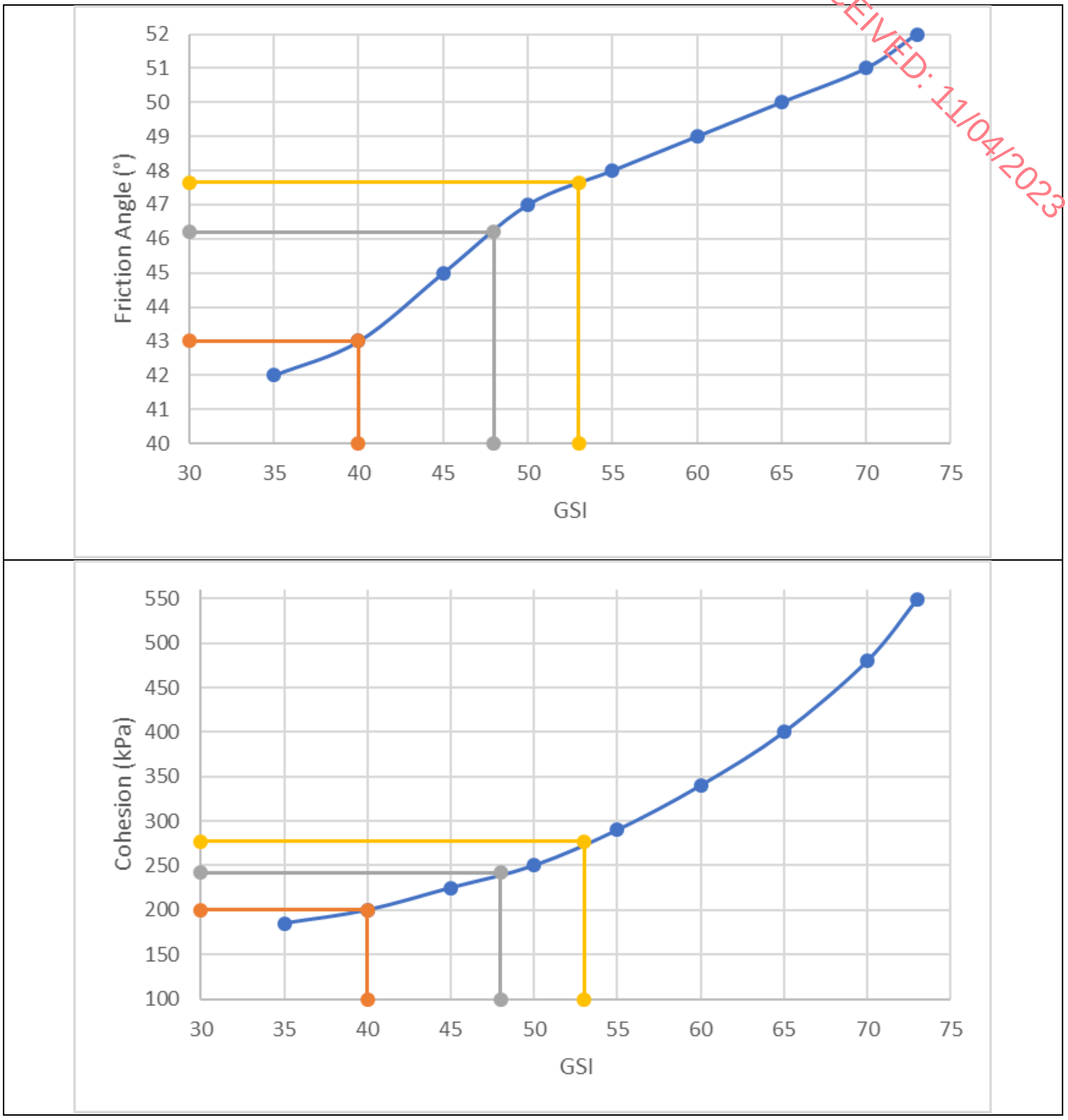

Figure 2-3: Friction Angle and Cohesion for Different GSI Values

3 SUBSIDENCE CHARACTERISTICS

The subsidence zone comprises a depression which is oval in shape (Figure 3-1), being longer in the E-W direction. The centre of the depression is located some 50 m to the west of the Magheracloone Community Centre. Surrounding the centre of the depression is a zone of compression where the material at surface has 'bunched' together. The ground then slopes upwards to a zone containing wide tension cracks. The limit of the disturbance zone is estimated to be slightly beyond the tension crack limit. A drone survey conducted on 26 September 2018 indicates that in the centre of the depression the ground level has dropped about 5 m below its original position.

Figure 3-1 is a drone image of the disturbance zone showing its extent, the tension cracks, and the location of the 12 m high pillar group. Figure 3-2 are photographs showing the major tension crack and the compression zone close to the position of maximum ground movement. Figure 3-3 is the same drone image superimposed with the position of all the pillars below the area.

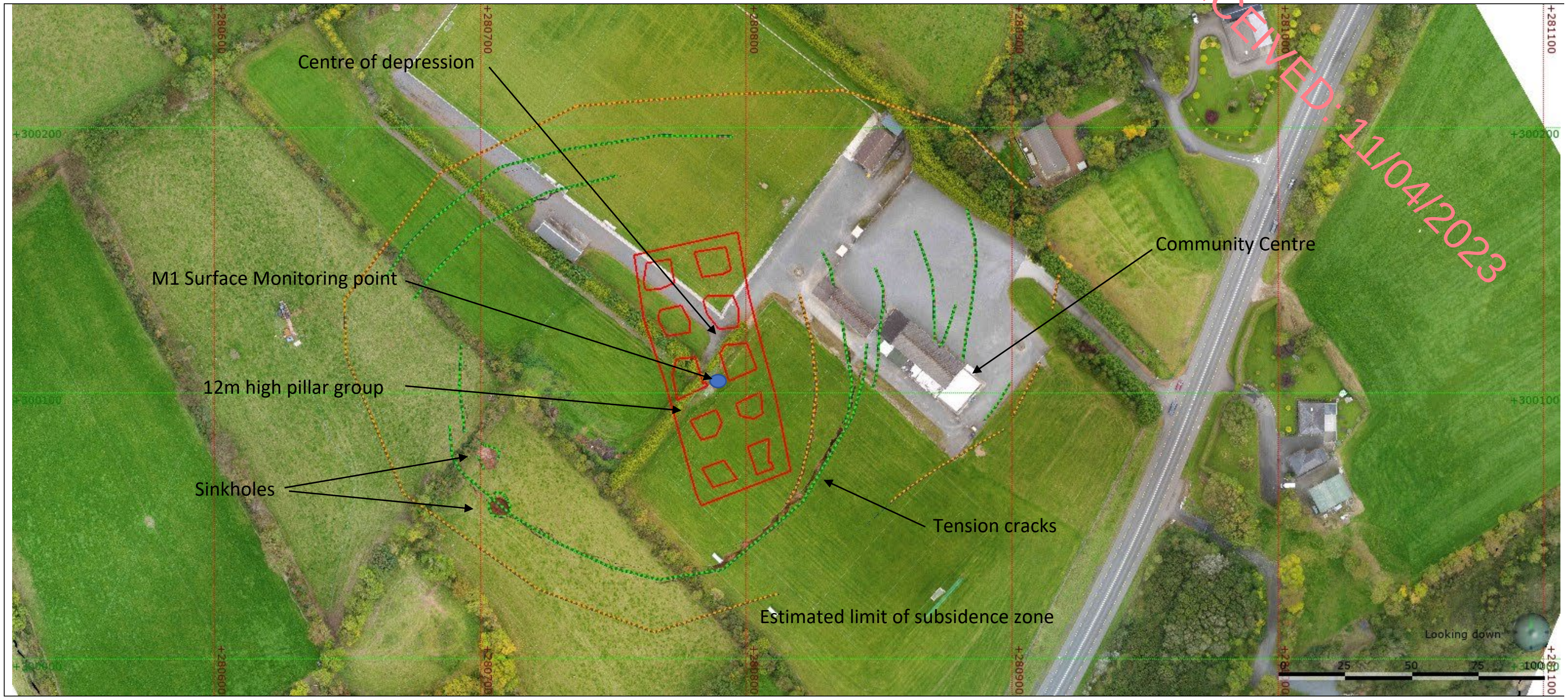

Figure 3-1: Extent of Disturbance Zone

Tension Cracks

Compression Zone

Figure 3-2: Photographs of Major Tension Crack and Compression Zone

Figure 3-3: Pillars below Community Centre and Playing Fields

4 NUMERICAL MODELLING

4.1 Analysis Profiles and Procedure

The 2D finite element code RS^2 Version 9, produced by Rocscience Inc, has been used for the simulation.

Finite element modelling (FEM) is a numerical method for solving problems of engineering. FEM subdivides the full problem domain into a finite number of small individual elements. The relatively simple governing (equilibrium and compatibility) partial differential equations (PDEs) are then solved approximately for each of these elements in terms of known and unknown forces and displacements. The FEM then assembles the individual approximate elemental solutions into a system of algebraic equations that is used to give solutions for all forces and displacements acting throughout the problem domain.

RS2 is a powerful 2D finite element program for soil and rock applications. RS2 can be used for a wide range of engineering projects including excavation design, slope stability, groundwater seepage, probabilistic analysis, consolidation, and dynamic analysis capabilities. Complex, multi-stage models can be easily created and quickly analysed for tunnels in weak or jointed rock, underground powerhouse caverns, open pit mines and slopes, embankments, and much more. Progressive failure and a variety of other problems can be addressed.

SRK has carried out 2D subsidence analyses for room and pillar mines around the world. Whilst the analysis of pillar stability is normally a 3D problem, 2D analyses are quick to run. Where SRK has followed up 2D analyses with 3D analyses of the same problem, the results of the 3D analyses generally result in lower subsidence values than the same problem run in 2D. 2D analysis therefore gives more conservative results than 3D analysis.

Two cross sections have been analysed, both through the middle of the 12 m high pillar group. One cross section is orientated approximately west-east, the other approximately north-south. Both cross-sections extend across the R179 road. The west-east cross section cuts through the Community Centre and mine area to the west where both the lower and upper gypsum seams were mined. The north-south cross section is located 50 m to the west of the Community Centre.

The location of the cross sections is shown in Figure 4-1. The finite element models are shown in Figure 4-2 and Figure 4-3. The position of the R179 road, the Community Centre, and the main features of the disturbance zone as measured from the drone survey are included on the cross-sections.

The back analysis focused on the west-east cross section where several sensitivity analyses were conducted:

1. The stability of the mine and surface was calculated for the historical mine water condition, that is with water at the 970m L. This is the condition that the mine was in prior to July 2018 and the condition that relates to the survey levelling data that has been collected since 2003. This scenario was run with the cohesion and friction angle of the lower gypsum and basal mudstone at lower bound strength as defined in Table 2-1. The lower gypsum and basal mudstone friction angle and cohesion were then reduced progressively to 70% of their lower bound strength to simulate degradation in the rock strength and thus to determine to what level the strength would need to

be reduced to generate the subsidence event experienced under dry mine conditions.

2. A water level was placed in the model at 993mL (the water level at the time the subsidence event occurred). The friction angle and cohesion of the lower gypsum and basal mudstone were reduced to simulate strength reduction due a combination of saturation of both the gypsum and the underlying basal mudstone and time dependent deterioration.

3. To verify that failure of the 12 m high pillars was indeed a contributing factor to the cause of the subsidence step 3) was repeated for a cross section where the 12 m high pillars were replaced by 6 m high pillars.

The analysis interpretation was checked against the position and extent of the surface subsidence features and the results of the historical surface monitoring.

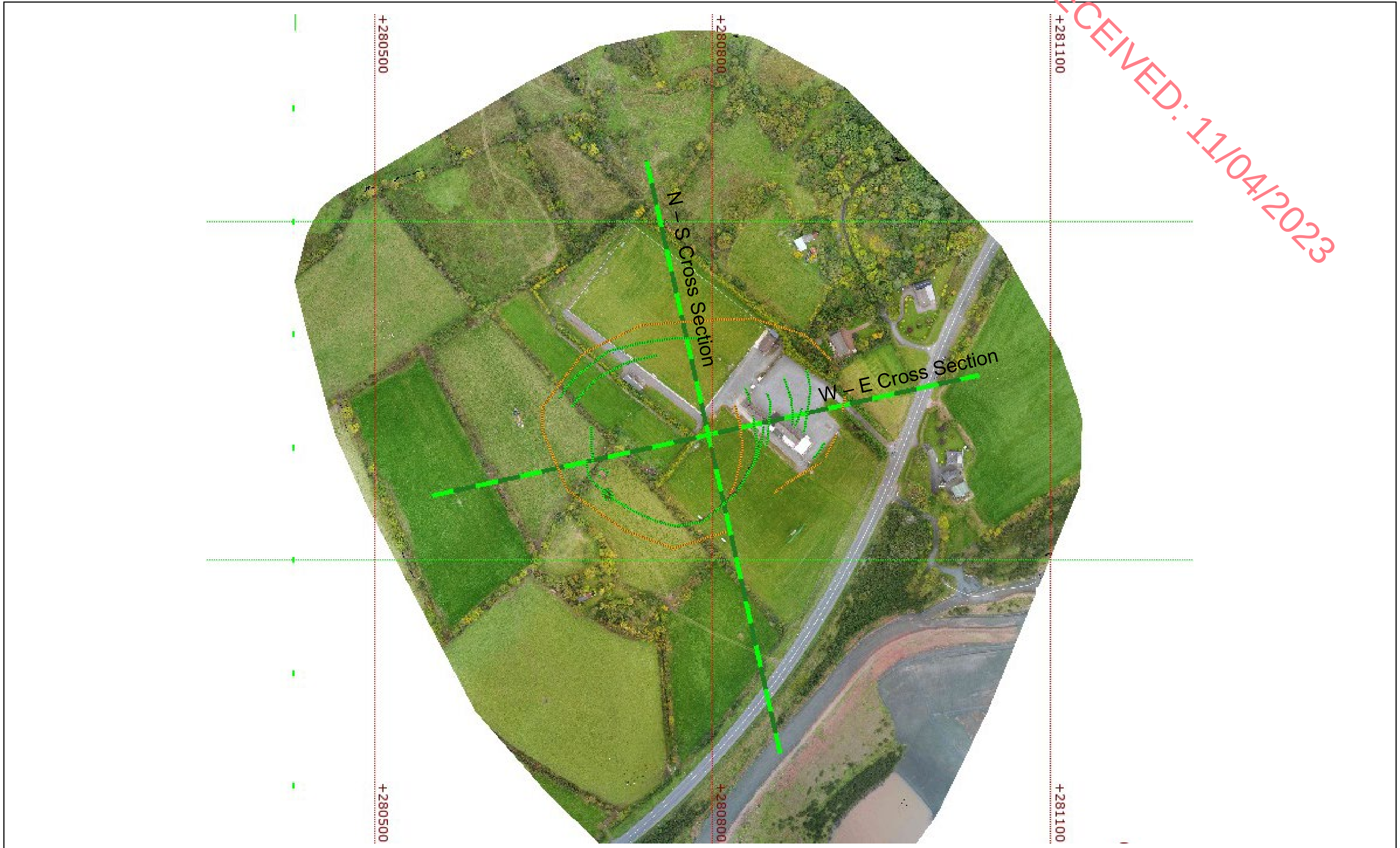

Figure 4-1: Location of Cross Sections

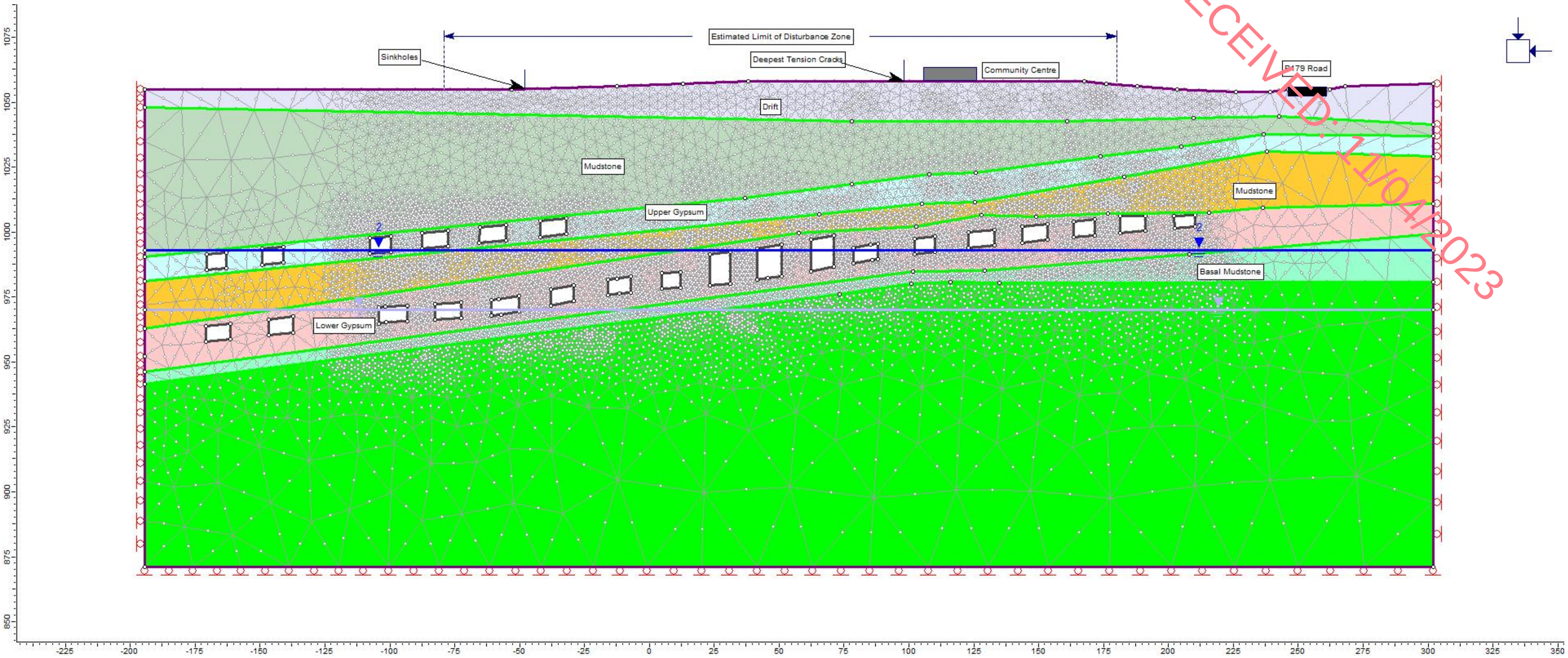

Figure 4-2: W - E Cross Section

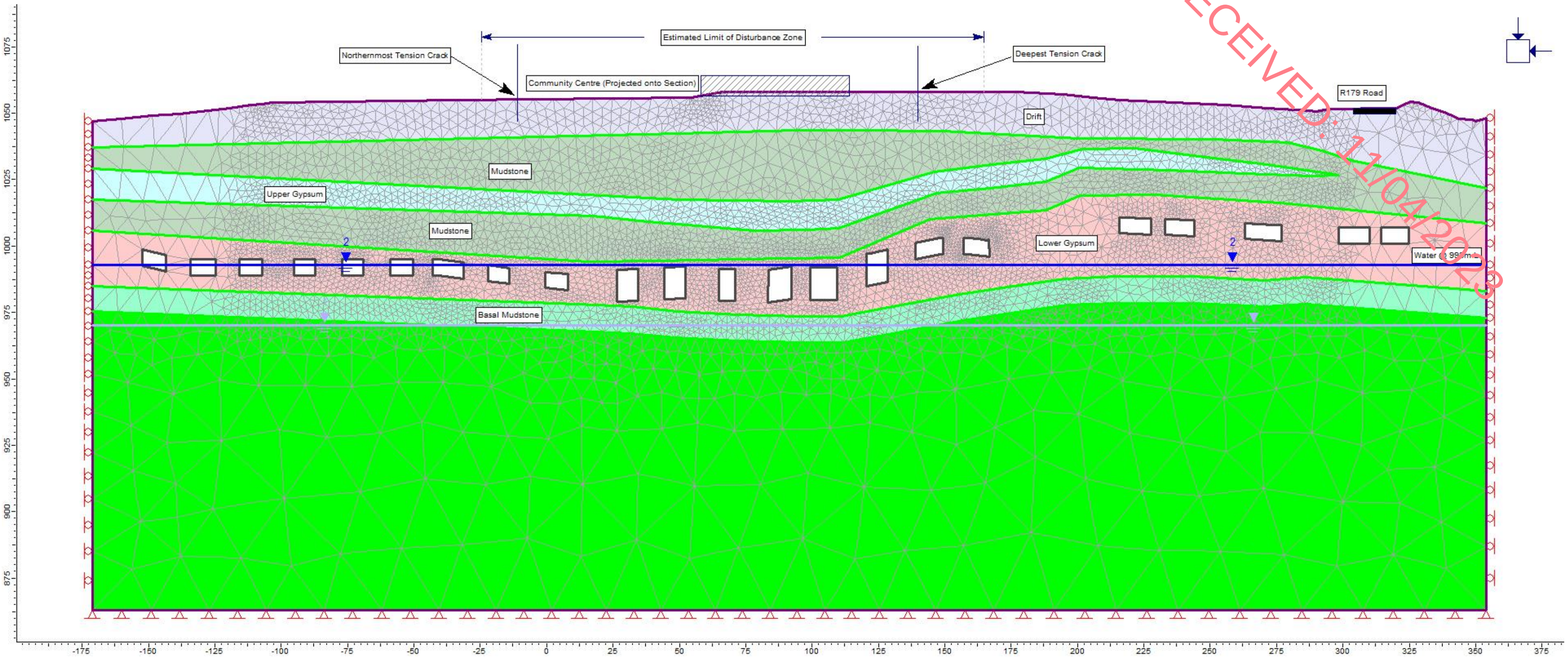

Figure 4-3: N – S Cross Section

4.2 W – E Section 'Dry' Mine Stability

The results of this analysis are presented in Table 4-1 as maximum vertical subsidence in the centre of the disturbance zone for different gypsum and basal mudstone strength conditions. Output from RS^2, showing contours of total displacement for all strength conditions, are presented in Appendix A.

The maximum displacement of the surface simulated by the model increases significantly when the rock strength is reduced by 20%; however, this displacement is significantly lower than that measured on surface, Figure 4-4.

Table 4-1: Predicted Surface Subsidence for Dry Mine Conditions

Condition	Predicted Surface Subsidence (mm)
Dry Mine - 100% Strength	22
Dry Mine - 90% Strength	46
Dry Mine - 85% Strength	98
Dry Mine - 80% Strength	2130
Dry Mine - 70% Strength	2130

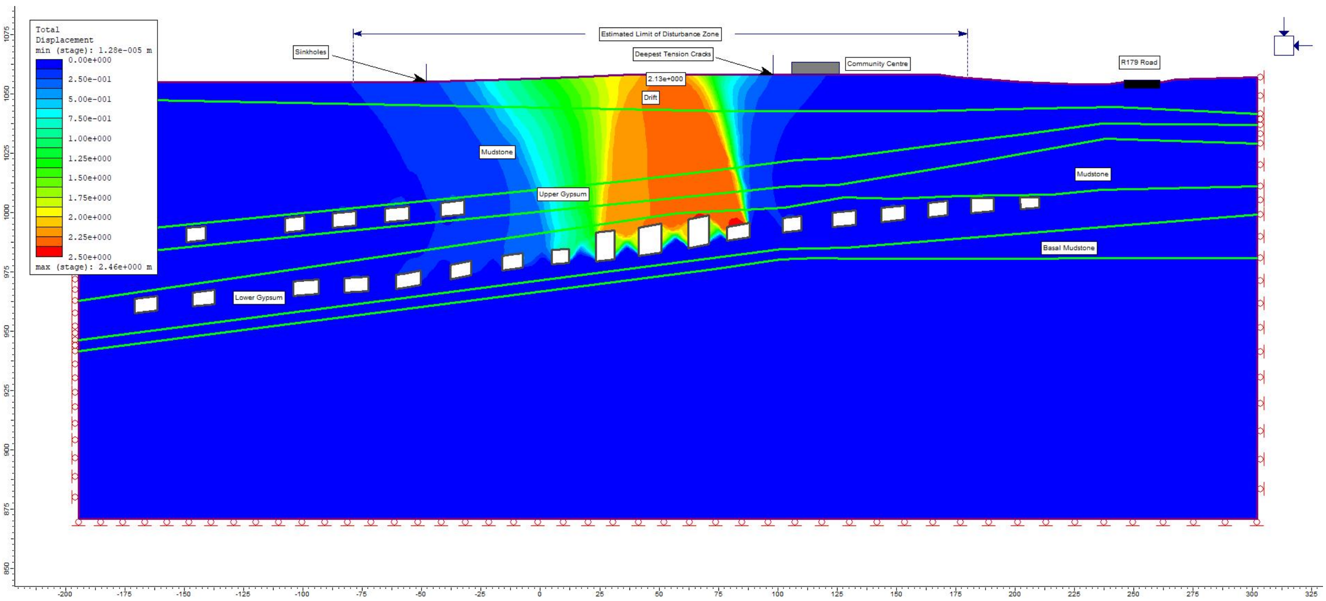

Figure 4-4: Contours of Total Displacement: Dry Mine, 80% of Lower Bound Strength

4.3 W – E Section Flooded Mine Conditions

The results of this analysis are presented in Table 4-2.

Table 4-2: Predicted Surface Subsidence for Flooded Mine Conditions

Condition	Predicted Surface Subsidence (mm)
Dry Mine - 100% Strength	22
Flooded Mine - 100% Strength	40
Flooded Mine - 90% Strength	50
Flooded Mine - 85% Strength	75
Flooded Mine - 80% Strength	6390
Flooded Mine - 70% Strength	6480

As the rock strength reduces from 85% to 80% of lower bound strength, there is a significant increase in surface displacement. The position of maximum surface displacement is consistent with that measured on surface (approximately 50 m from the Community Centre). The maximum modelled displacement is of a similar order of magnitude to the measured displacement. The eastern edge of the zone of maximum subsidence ties in almost exactly with the position of the deepest tension crack on surface (Figure 4-5). The western extent of the disturbance zone in the model, however, extends further west than estimated from surface where the disturbance zone appears to end at the position of the sinkholes. The large displacements in the core of the subsidence zone masks the deformation below and south of the Community Centre. The model does, however, simulate total surface displacement of 35 mm and higher within the car park area. This level of displacement is consistent with the observation of numerous continuous narrow cracks in this area. Output from RS^2, showing contours of total displacement for all strength conditions are presented in Appendix A.

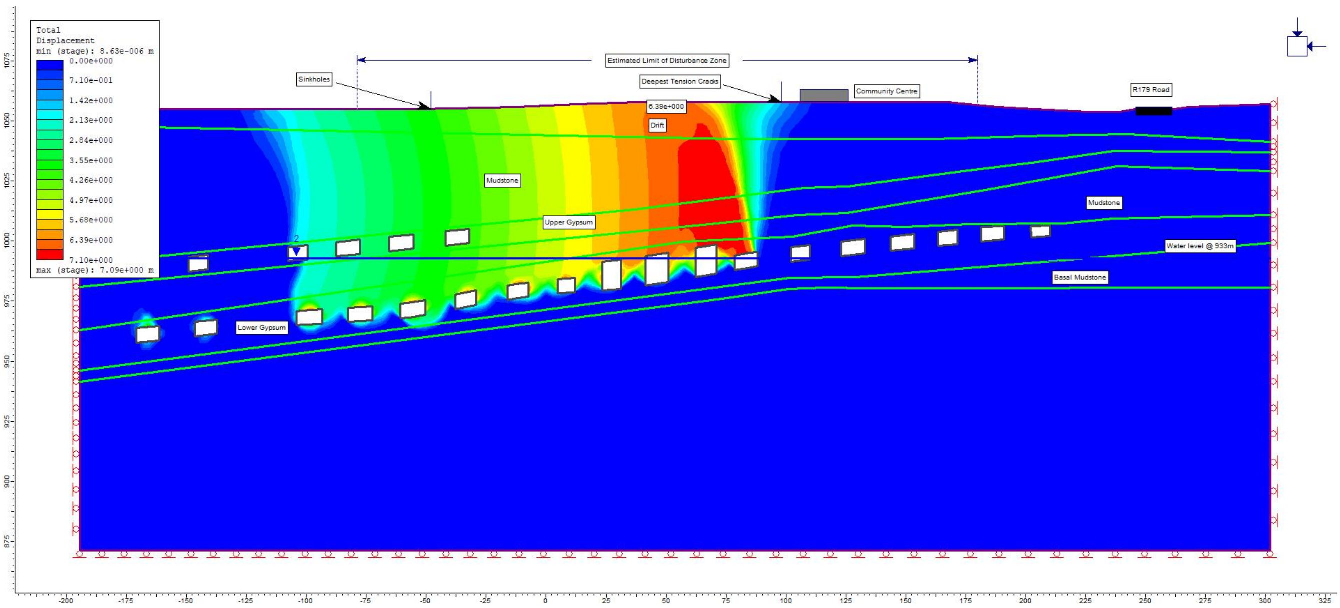

Figure 4-5: Contours of Total Displacement: Flooded Mine, 80% of Lower Bound Strength

4.4 W – E Section Flooded Mine, 6 m High Pillars

An additional model was run to test whether surface disturbance would have occurred if the pillars had been 6 m high rather than12 m high. The results of the maximum surface displacement at a point 50 m away from the Community Centre are shown in Table 4-3. Output from RS^2, showing contours of total displacement for all strength conditions are presented in Appendix A.

Table 4-3: Predicted Surface Subsidence for Flooded Mine Conditions, 6 m High Pillars

Condition	Predicted Surface Subsidence 50 m from Community Centre(mm)	Predicted Maximum Surface Subsidence (mm)
Flooded Mine - 100% Strength	4	8
Flooded Mine - 90% Strength	15	20
Flooded Mine - 85% Strength	25	35
Flooded Mine - 80% Strength	195	260
Flooded Mine - 70% Strength	4400	6600

As with the other models, there is a significant increase in surface displacement as the rock mass strength reduces from 85% to 80% of lower bound; however, the maximum model displacement occurs in the area that contains both upper and lower gypsum seam excavations (Figure 4-6). The results of this analysis indicate that it was the presence of and failure of the 12 m high pillars that has given rise to the subsidence event at the position at which it has occurred.

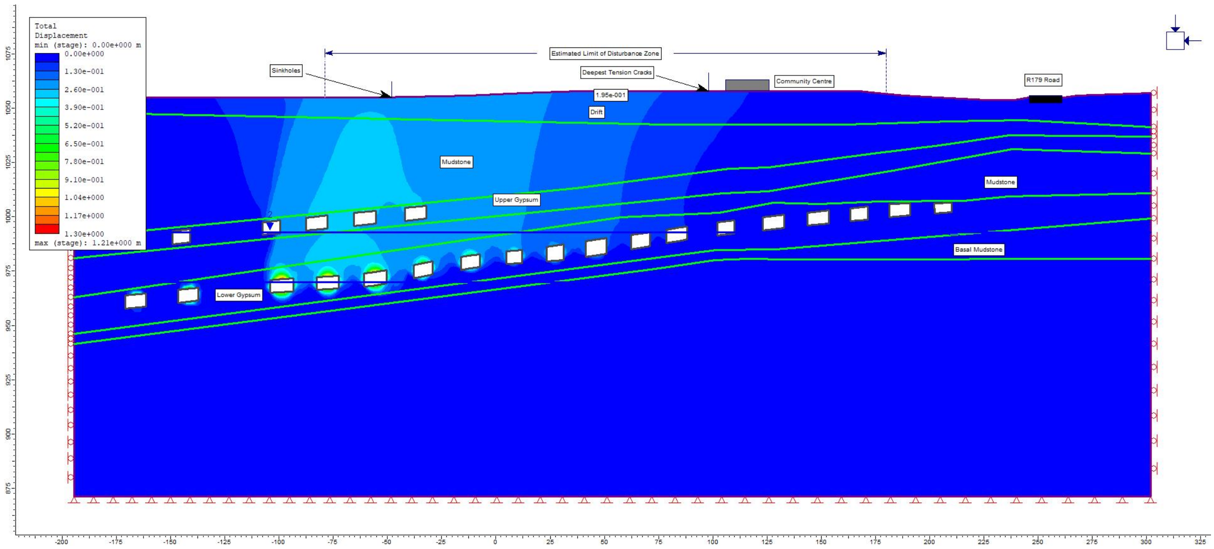

Figure 4-6: Contours of Total Displacement: 6 m Pillars, Flooded Mine, 80% of Lower Bound Strength

4.5 N – S Section Analysis, Flooded Mine Conditions

The N -S cross section cuts through a whole line of 12 m high pillars very close to the centre of the disturbance zone and about 50 m to the west of the Community Centre. The results of the maximum surface displacement at the centre of the disturbance zone are shown in Table 4-4. Output from RS^2, showing contours of total displacement for all strength conditions are presented in Appendix A.

Table 4-4: Predicted Surface Subsidence for Flooded Mine Conditions, N – S Section

Condition	Predicted Surface Subsidence (mm)
Dry Mine - 100% Strength	26
Flooded Mine - 100% Strength	57
Flooded Mine - 90% Strength	4140
Flooded Mine - 85% Strength	4140
Flooded Mine - 80% Strength	4140
Flooded Mine - 70% Strength	4230

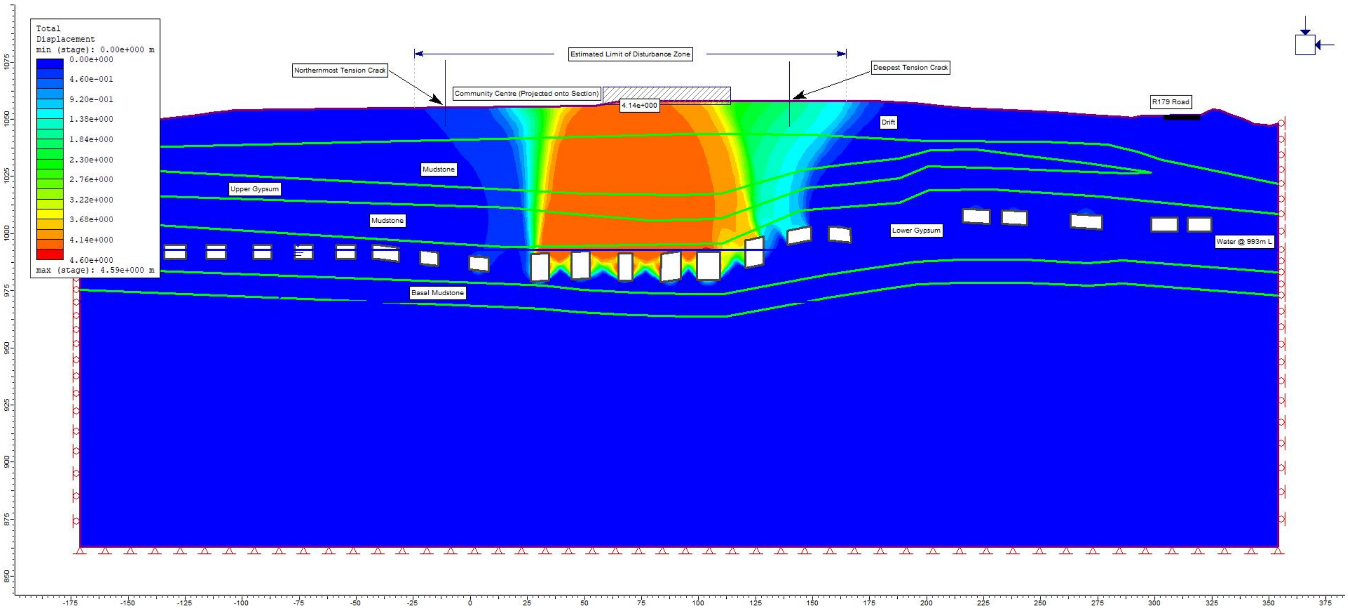

Figure 4-7: Contours of Total Displacement: N -S Section, Flooded Mine, 90% of Lower Bound Strength

Because there are a greater number of 12 m high pillars on this section line, only a 10% strength reduction is required to initiate subsidence along this section line. The magnitude of subsidence remains largely unchanged as the strength is further reduced. There is good correlation between the extent of the modelled subsidence zone and the extent measured on the ground surface. The position of the tension crack zone in this model is less well defined than in the W – E section model. The magnitude of subsidence at 4 m is close to the 5 m subsidence measured by the drone survey.

4.6 Back Analysis Summary and Conclusions

1. The historical surface survey monitoring, carried out from 2003, indicates that the ground above the 12 m high pillar group has demonstrated a regressive movement trend, that is reducing movement with time. This indicates the mine elements, pillars, roof beams, and floor beams, are generally in a state of stable equilibrium.

2. It is expected that with time the strength of the gypsum pillars and underlying basal mudstone has deteriorated. This has been simulated in the numerical models by reducing in percentage increments the friction angle and cohesion of the Lower Gypsum and Basal Mudstone and testing the model for stability.

3. Ongoing control of the water level in the mine by pumping has maintained the water level at around 970mL. This level is below the lowest elevation of the 12 m high pillar group; therefore the 12 m pillar group has historically been in a dry area of the mine.

4. The back analysis shows that the surface disturbance of the magnitude experienced would not have occurred in a flooded mine in deteriorating strength conditions if this area was comprised of 6 m high pillars.

5. When the strength of these pillars was reduced to simulate long term time dependent deterioration pillar failure might have occurred in a dry mine but of a magnitude significantly lower than that experienced in this event. This indicates that potential failure of 12 m high pillars in a dry mine would not have given rise to the level of surface subsidence generated by this event.

6. The back analysis has simulated the measured magnitude and extent of the disturbance zone reasonably accurately for mine conditions where the water level in the mine has risen to the 993mL and the gypsum and basal mudstone strength have deteriorated by 10% to 20% of their estimated lower bound value.

7. The back analysis therefore confirmed the preliminary assessment; that the introduction of excess water into previously dry sections of the mine which contain the 12 m high pillar group has reduced the strength of the gypsum pillars and underlying mudstone to such a degree that failure of the mine elements occurred, creating a major disturbance feature on surface.

5 DISTURBANCE ZONE PREDICTIVE ANALYSIS

The back analysis has demonstrated that 6 m high pillars are likely to be stable even when submerged in water. Potential extension to the disturbance zone is only likely to occur if the mine continues to be filled with water up to the maximum room elevation, submerging all the pillars. On the W-E cross section, this is the 1007mL. On the N-S cross section, this is the 1011mL. This will not happen as SGMI have committed to a programme of mine water level reduction. Therefore, this section presents the results of an unlikely 'what-if' scenario.

5.1 W – E Section: Impact of Complete Mine Flooding

The highest elevation of the mine on this cross section is the 1007mL. If the water level in the mine reaches this level, a further model analysis has been carried out to test whether there will be any W - E extension to the disturbance zone. The results of the analyses are shown in Table 5-1. Output from RS^2, showing contours of total displacement for all strength conditions are presented in Appendix B.

The results show that as the mine continues to flood the disturbance zone will continue to subside, but the limits of the zone of maximum disturbance will remain relatively unchanged. This is shown in Figure 5-1

Table 5-1: Predicted Surface Subsidence with Mine Submerged to 1007mL

Condition	Predicted Surface Subsidence - water level at 993mL (mm)	Predicted Surface Subsidence - water level at 1007mL (mm)
Dry Mine - 100% Strength	22	19
Flooded Mine - 100% Strength	40	115
Flooded Mine - 90% Strength	50	115
Flooded Mine - 85% Strength	75	10400
Flooded Mine - 80% Strength	6390	10400
Flooded Mine - 70% Strength	6480	10400

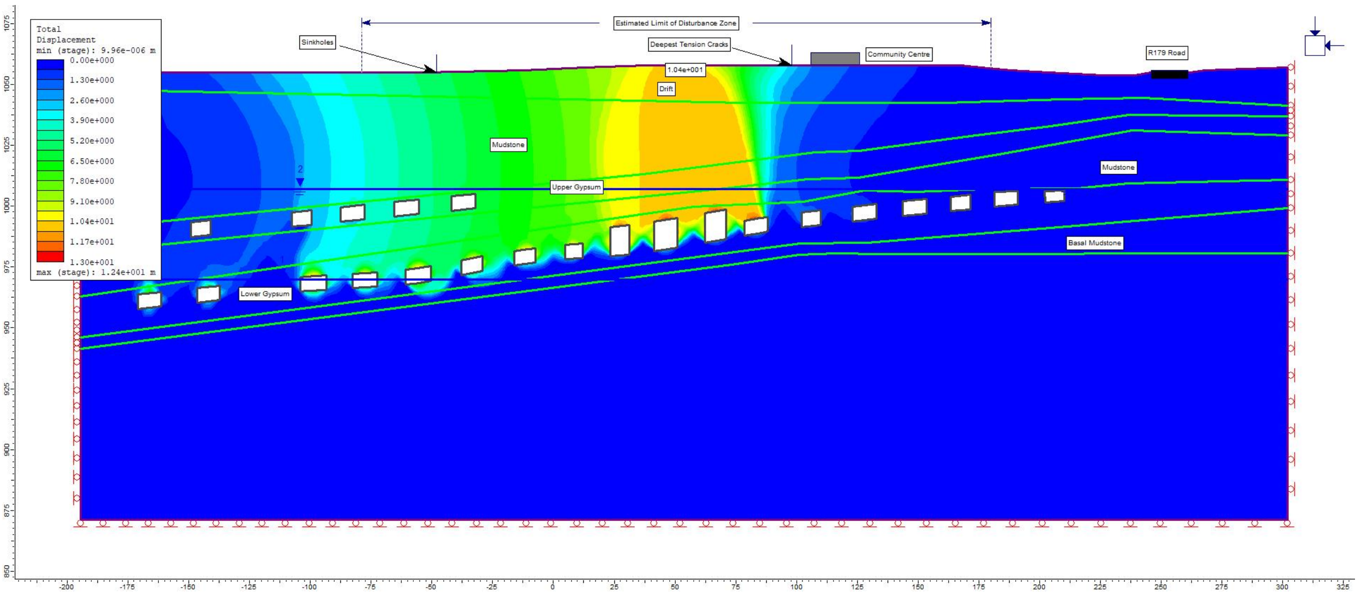

Figure 5-1: Contours of Total Displacement: W - E Section, Mine Submerged to 1007mL, 80% of Lower Bound Strength

Because of the greater vertical movement in the core of the disturbance zone, there could be further movement on surface to the east and west of the disturbance zone. In Figure 5-1, the deformation to the east of the Community Centre is masked by the high deformations in the core of the disturbance zone.

Figure 5-2 is a graph generated by the numerical model of total surface movement east of the Community Centre. The edge of the disturbance zone generated by flooding to the 993mL is about 40-50 m to the east of the Community Centre. If the mine were to be flooded to the 1007mL, it is possible that the disturbance zone would extend a further 50 m to the east. Note that even for this *worst-case scenario*, the R179 road remains outside the disturbance zone and its influence.

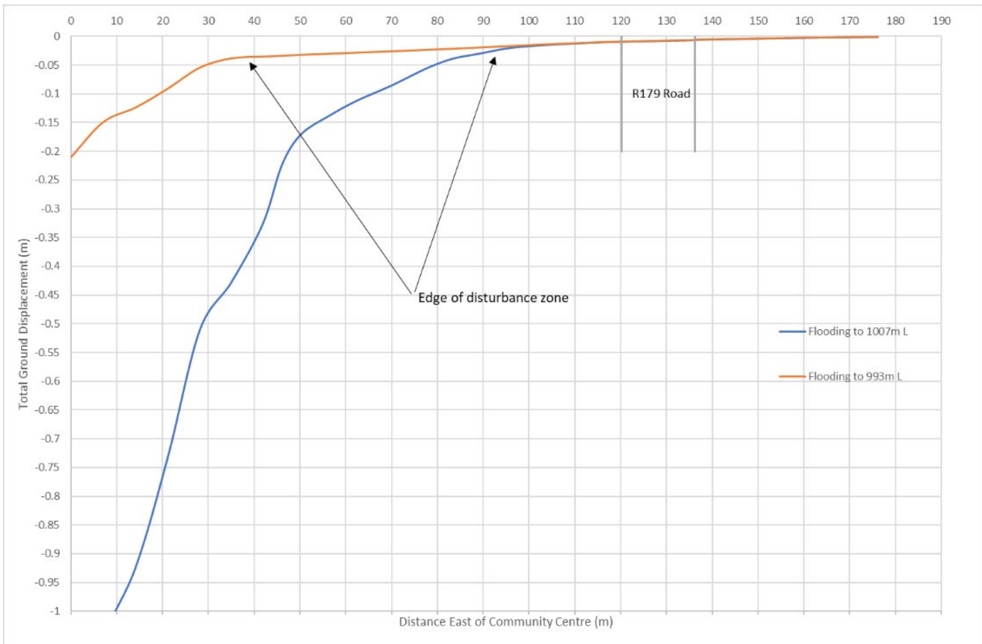

Figure 5-2: Graph of Total Surface Displacement East of the Community Centre

5.2 N – S Section, Impact of Complete Mine Flooding

The highest elevation of the mine on this cross section is the 1011mL. A further model analysis has been carried out to test whether there will be any extension to the disturbance zone particularly to the south towards the R179 road if the water level in the mine were to reach this level. The results of the analyses shown in Table 5-2. Output from RS^2, showing contours of total displacement for all strength conditions, are presented in Appendix B.

The results show that as the mine continues to flood the disturbance zone will continue to subside, but the limits of the zone of maximum disturbance will remain unchanged. There is no impact of flooding the mine on the 6 m high rooms and pillars to the south and north, which remain stable. This is shown in Figure 5-3.

Table 5-2: Predicted Surface Subsidence with Mine Submerged to 1007mL

Condition	Predicted Surface Subsidence water level at 993mL (mm)	Predicted Surface Subsidence water level at 1011mL (mm)
Dry Mine - 100% Strength	26	26
Flooded Mine - 100% Strength	57	240
Flooded Mine - 90% Strength	4140	12000
Flooded Mine - 85% Strength	4140	12000
Flooded Mine - 80% Strength	4140	12800
Flooded Mine - 70% Strength	4230	12800

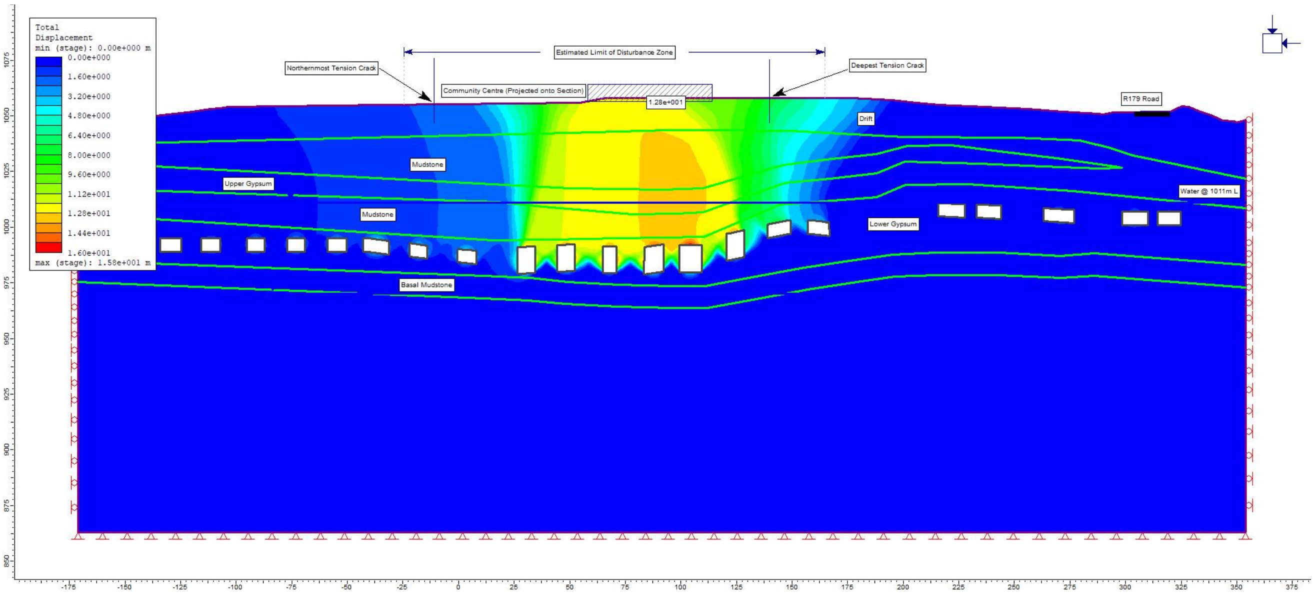

Figure 5-3: Contours of Total Displacement: N - S Section, Mine Flooded to 1011mL, 80% Strength

A graph of total surface displacement south of the Community Centre is presented in Figure 5-4. This compares the surface displacement resulting from mine flooding to the 993mL to the surface displacement resulting from flooding the mine to the 1011mL. This graph shows that there is no further southwards extension of the disturbance zone resulting from continued mine flooding.

Currently, the ground disturbance does not affect the integrity of the R179. Figure 5-3 shows that there are two rooms located 45 m below the R179 road. These rooms would become flooded if the mine water level increased to the 1011mL. The model predicts that about 5 cm of subsidence would occur at the position of the road on this cross section.

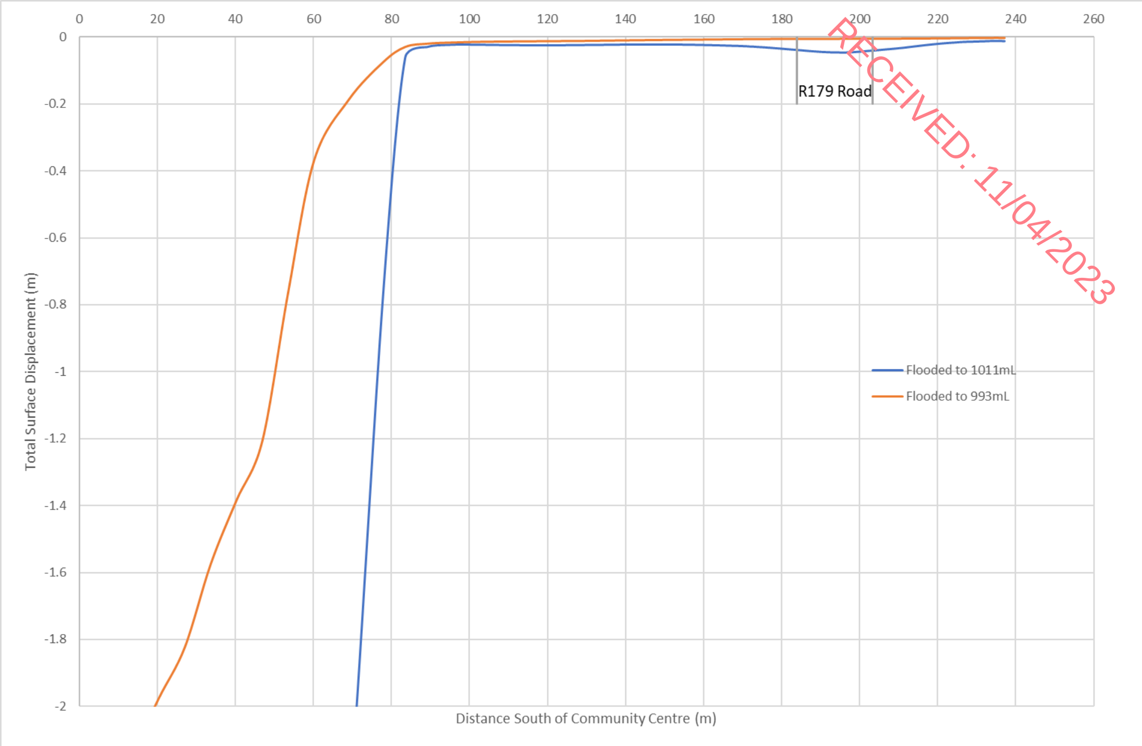

Figure 5-4: Graph of Total Surface Displacement South of the Community Centre

6 PREDICTIVE ANALYSIS

6.1 Areas for Analysis

Several areas outside of the main disturbance zone were identified for investigation. These were:

- The areas of the R179 and L4900 roads located over the mine workings.
- The areas below the properties belonging to:
  - Mr Martin;
  - Mr and Mrs Maxwell;
  - Mrs Kiernan;
  - Mr and Mrs Ward; and
  - Mr and Mrs Rafferty.

A total of 9 cross sections were generated for finite element modelling the results of which were presented in the draft report published on 15 October 2018 and which discussed at a meeting with Monaghan County Council on 17 October 2018. After the presentation three additional cross sections were included in the analysis:

- The L4900 Location 2 cross sections may contain, based on anecdotal information, two 6m high mining levels within the lower gypsum separated by an unknown thickness of lower gypsum beam. These are not indicated on any mine plans. To simulate this possibility SRK has analysed two cross sections at this location, one containing 6m high rooms and one containing 12m high rooms to simulate double level mining.
- Near the R179 road Location 2 cross section there are three small pillars below and adjacent to the road. In SRK's historical reports in Appendix D these pillars are identified as R8, R12 and R21. The original Location 2 cross section passed through Pillar R21. Two additional cross sections have been analysed, one north of the Location 2 cross section which passes through pillar R8, and one south of the location 2 cross-section which passes through pillar R12.

The location of the cross section cross sections is shown in Figure 6-1 and the finite element models for these cross sections are shown in Figure 6-2 to Figure 6-13. The lithology contacts have been provided by SGMI.

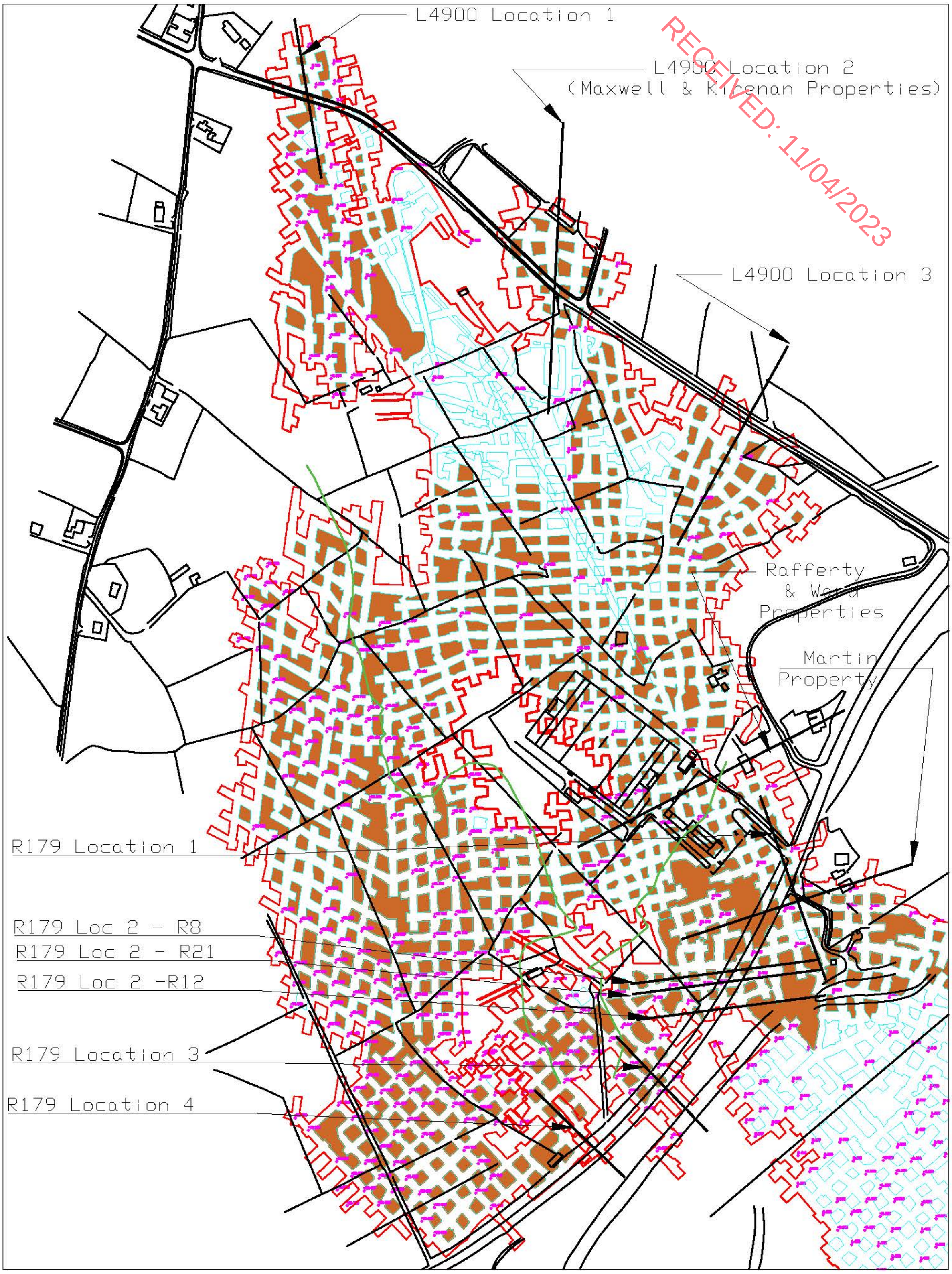

Figure 6-1: Location of Predictive Analysis Cross-Sections

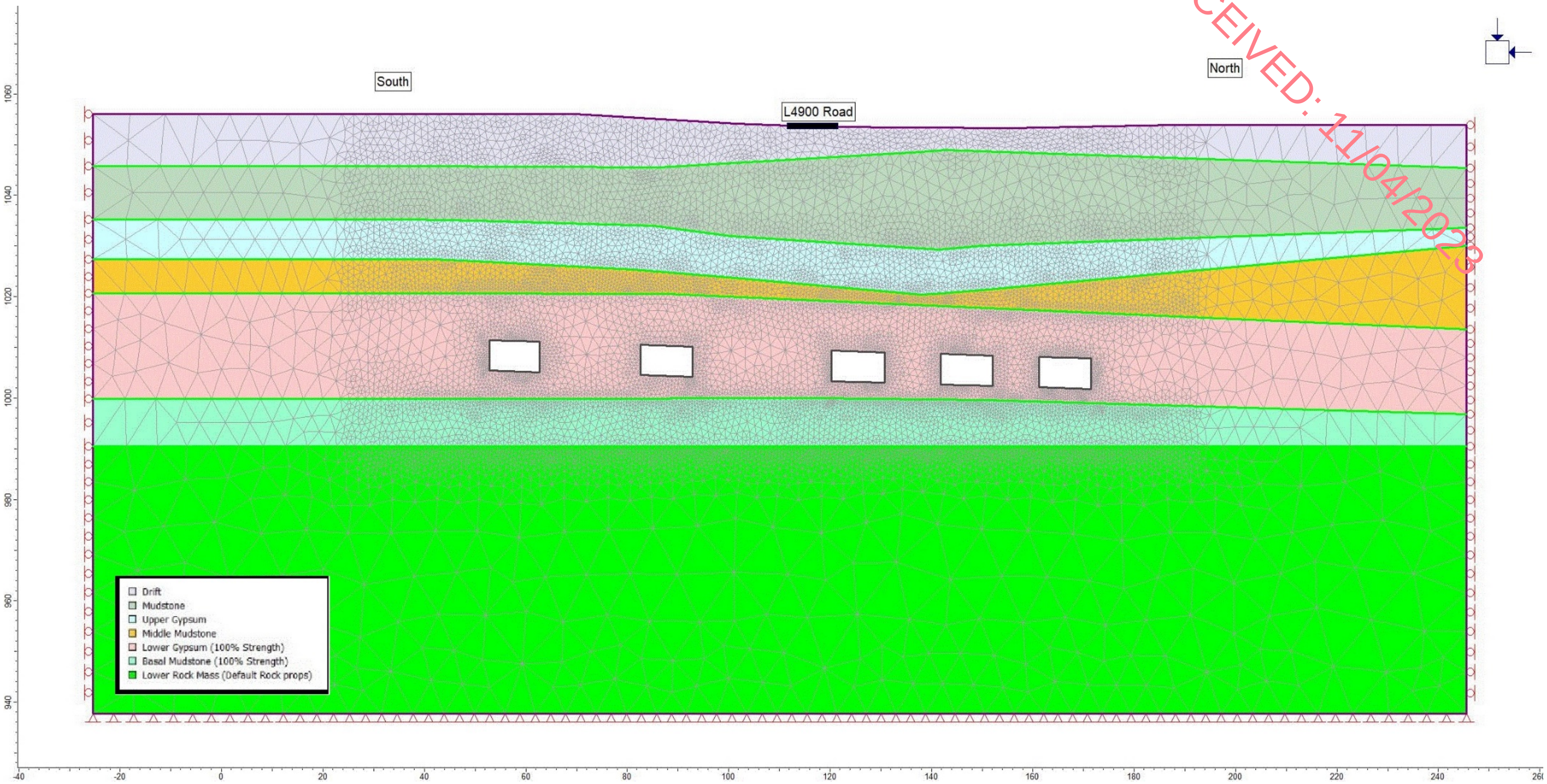

Figure 6-2: L4900 W Cross Section

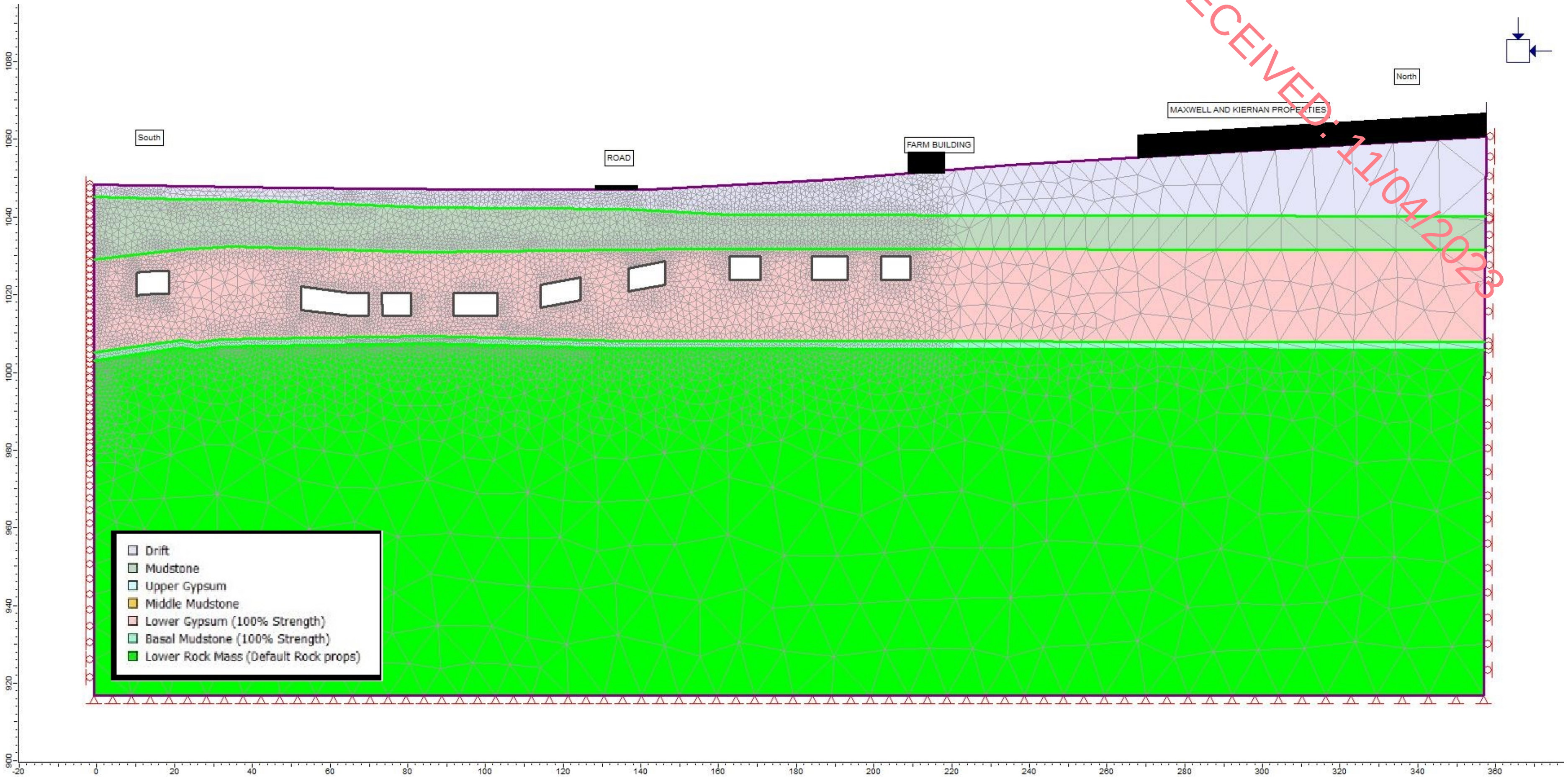

Figure 6-3: L4900 Central Cross section (Maxwell & Kiernan Properties)

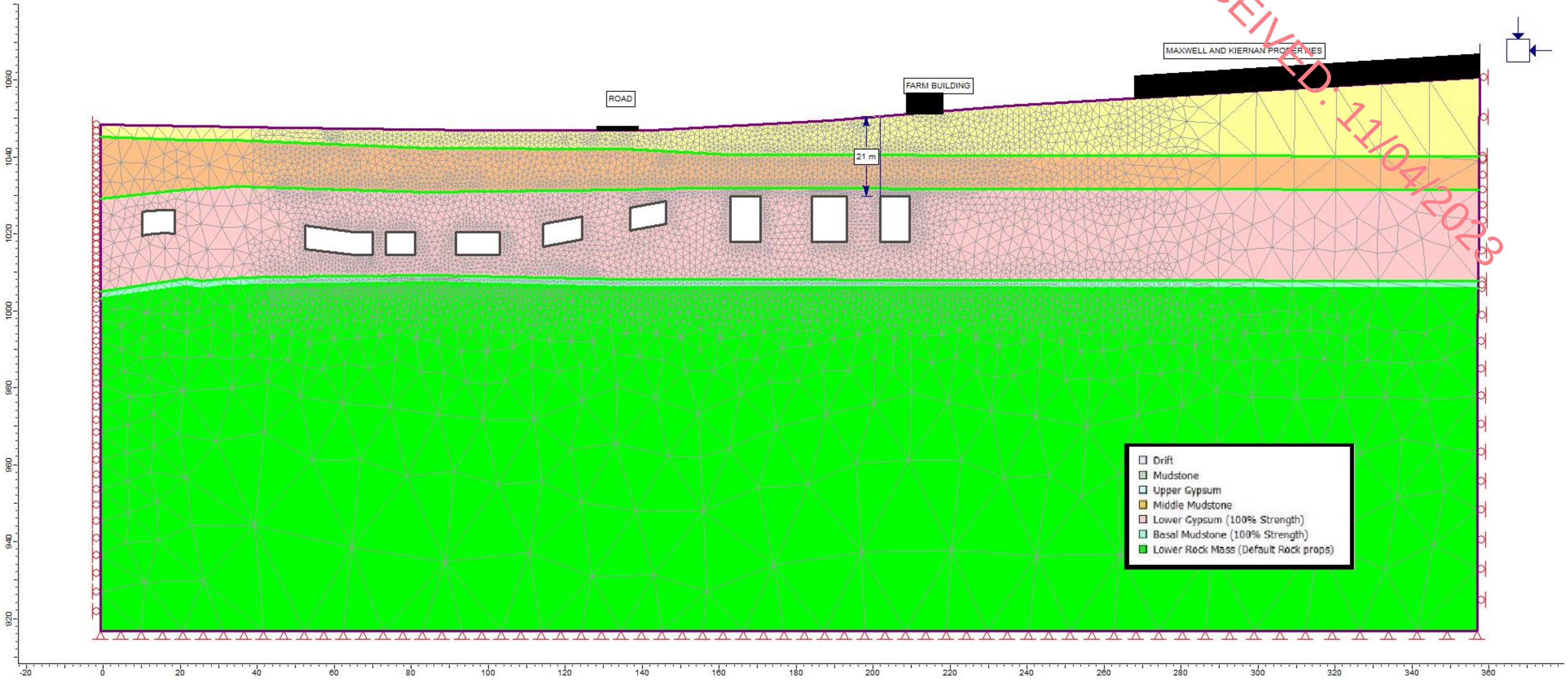

Figure 6-4: L4900 Central Cross section, 12m Room Model (Maxwell & Kiernan Properties)

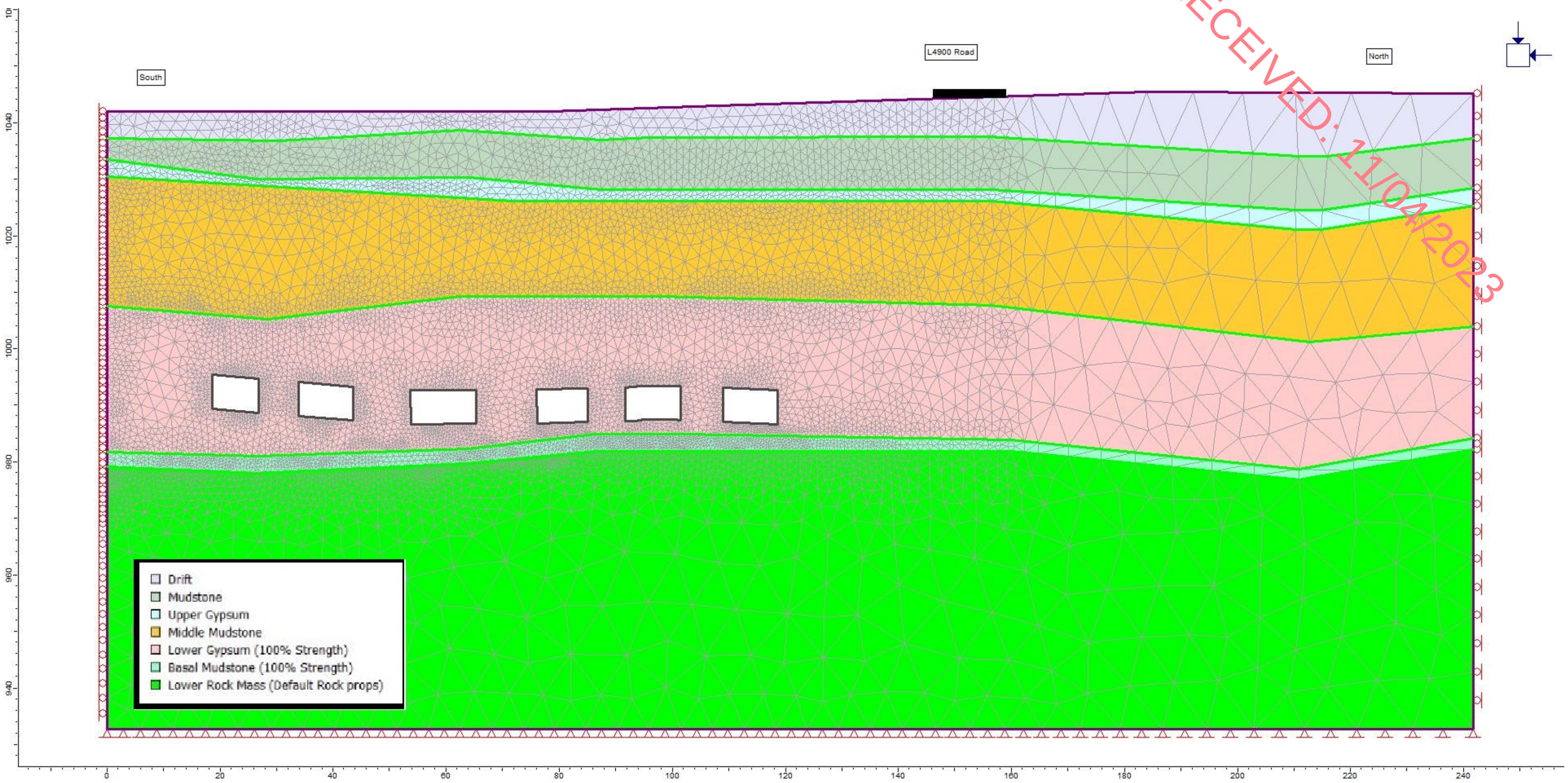

Figure 6-5: L4900 East Cross Section

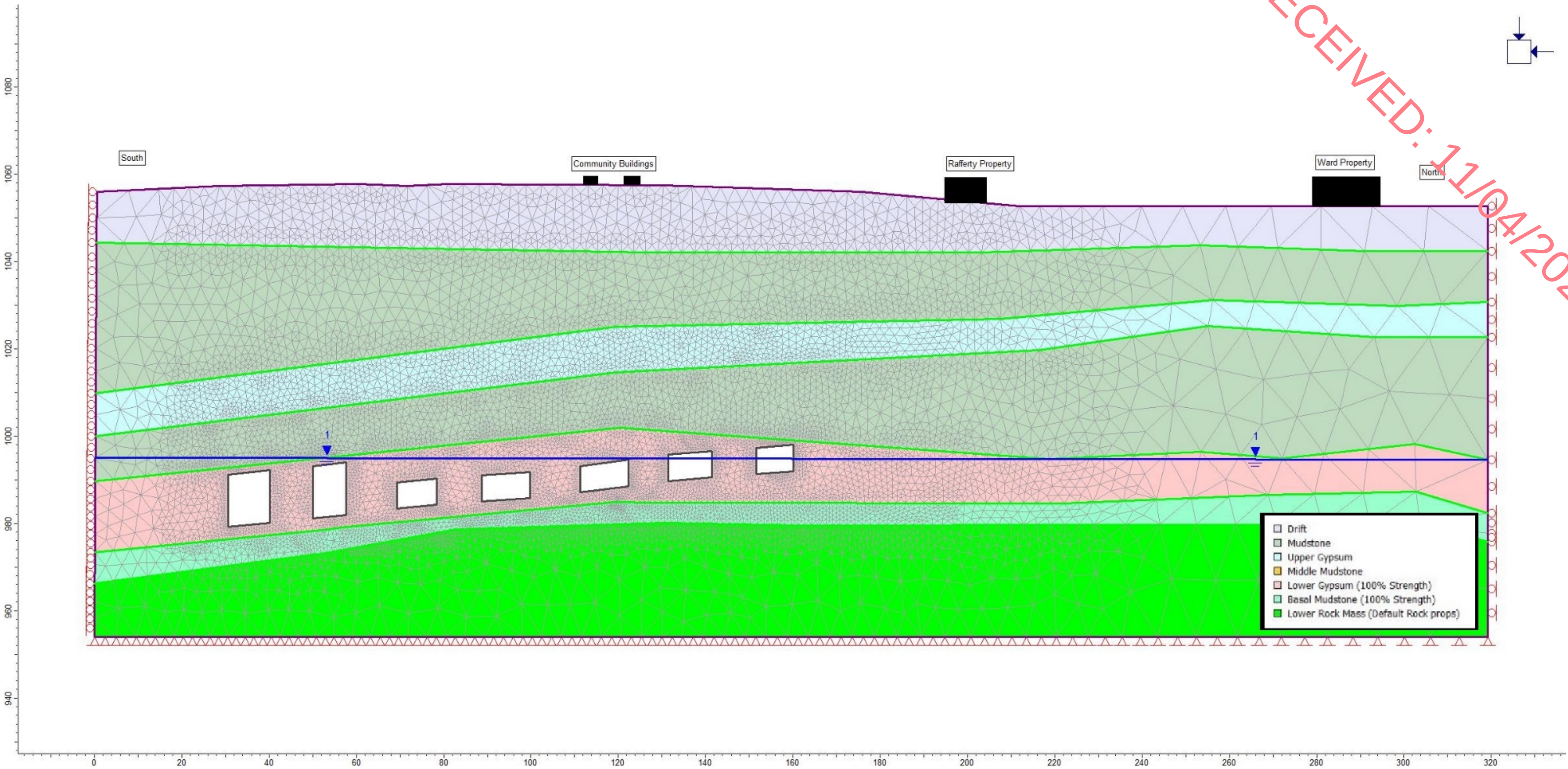

Figure 6-6: Rafferty & Ward Properties

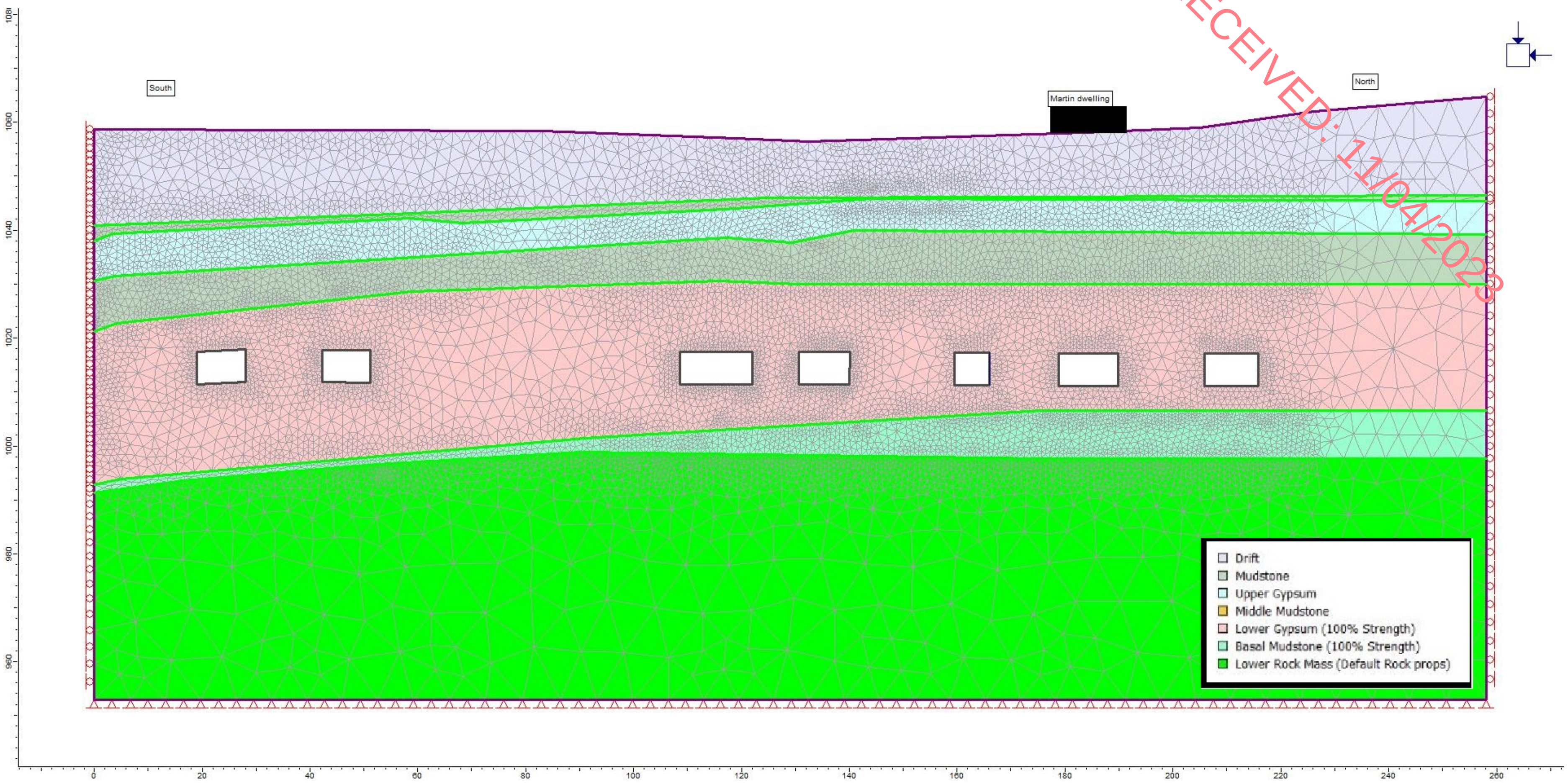

Figure 6-7: Martin Property

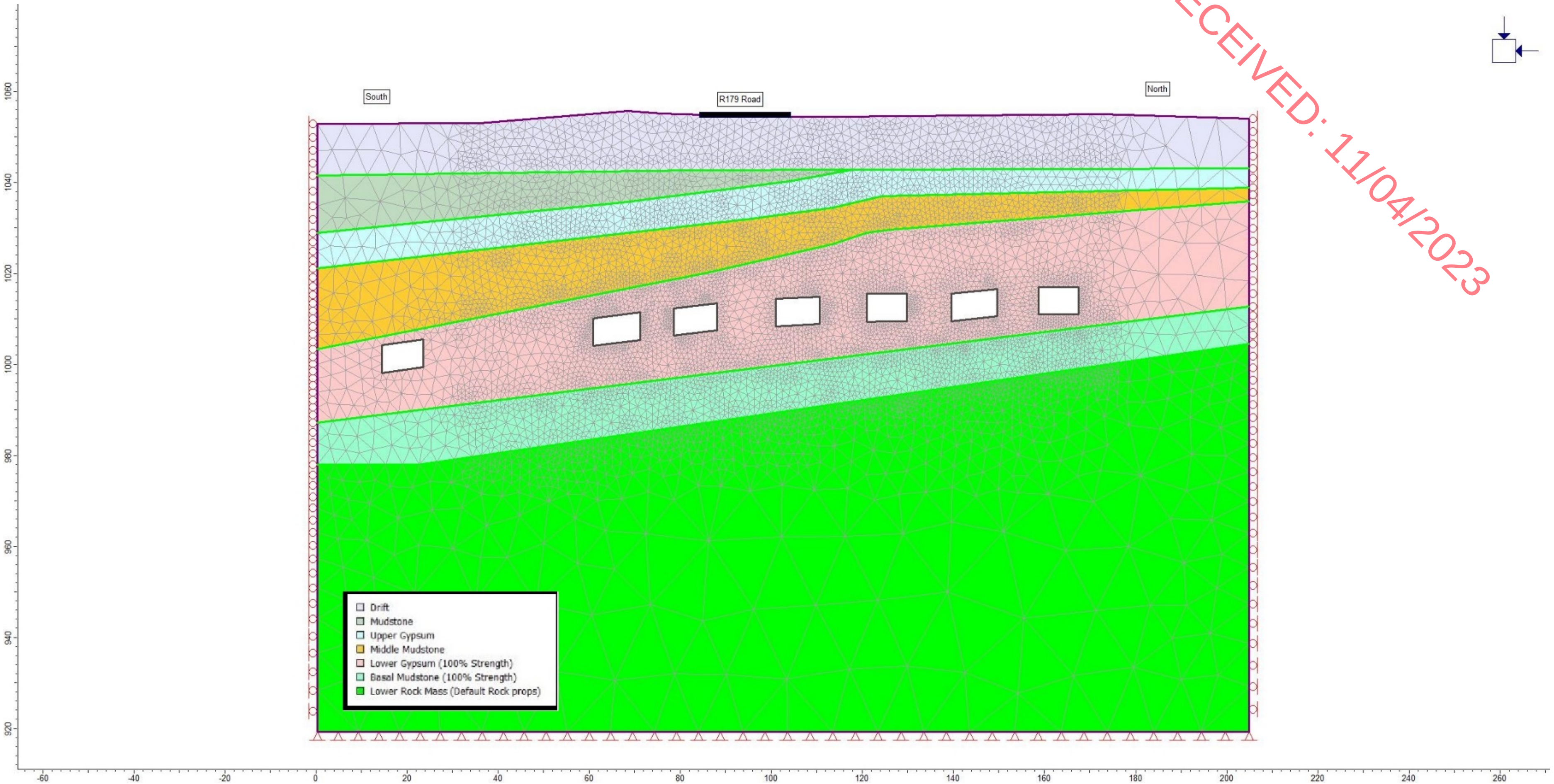

Figure 6-8: R179 Location 1 Cross Section

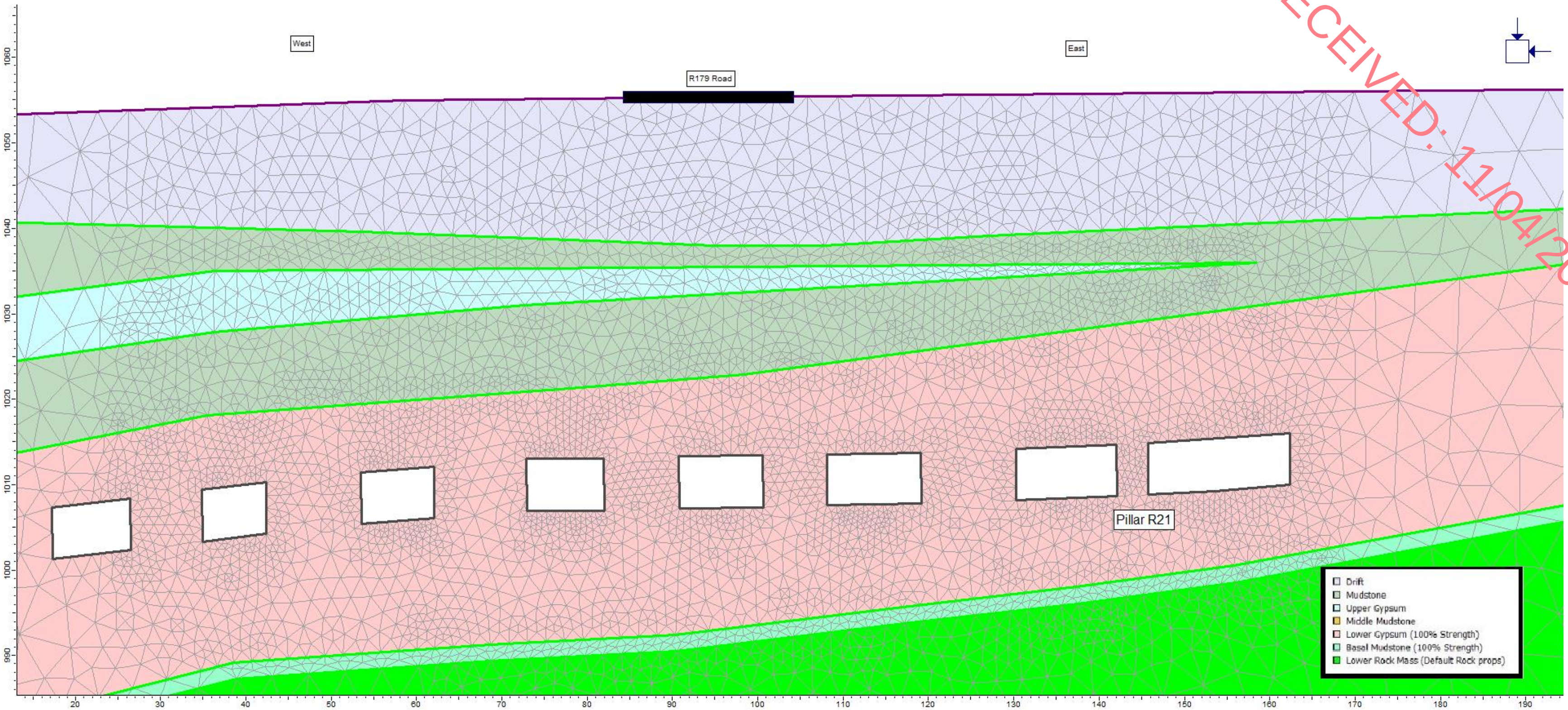

Figure 6-9: R179 Location 2 Cross Section Through Pillar R21 – Change image

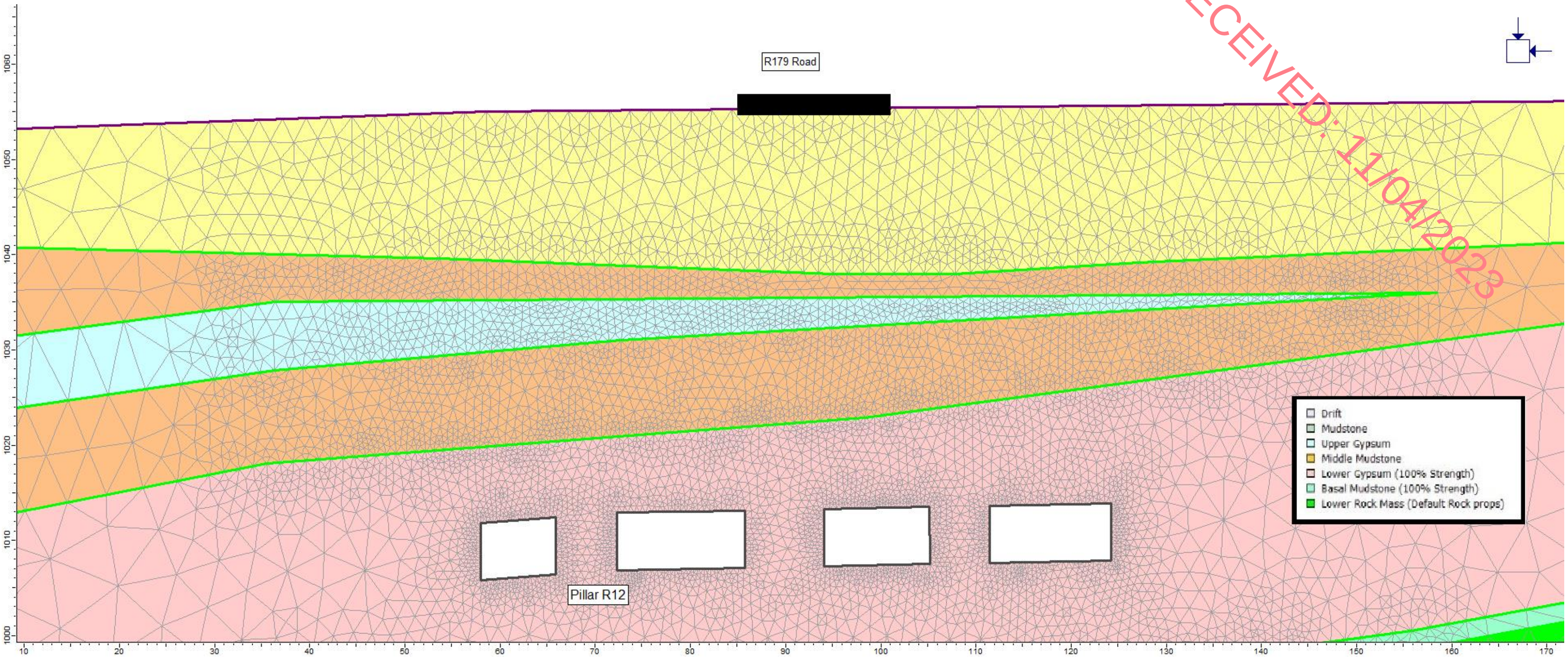

Figure 6-10: R179 Location 2 Cross Section Through Pillar R12

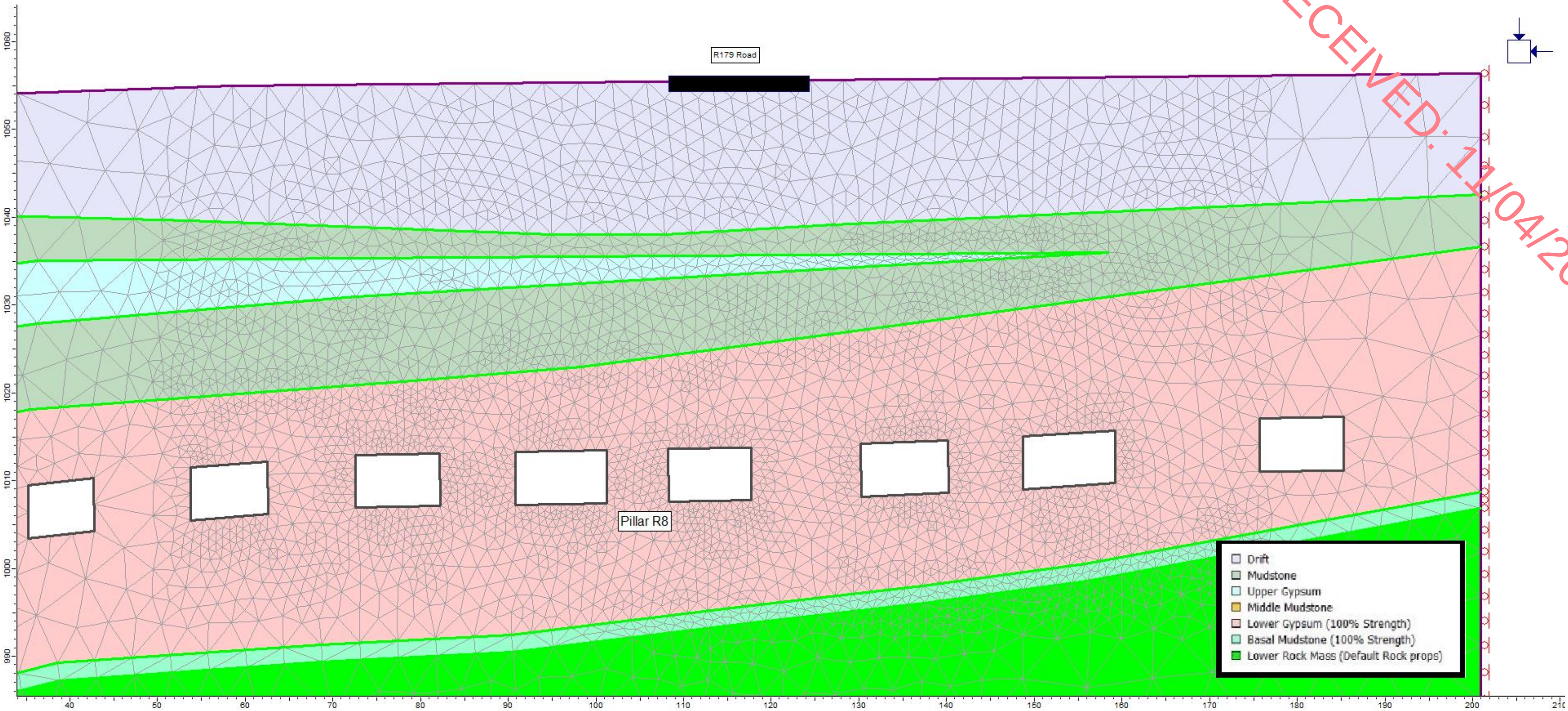

Figure 6-11: R179 Location 2 Cross Section Through Pillar R8

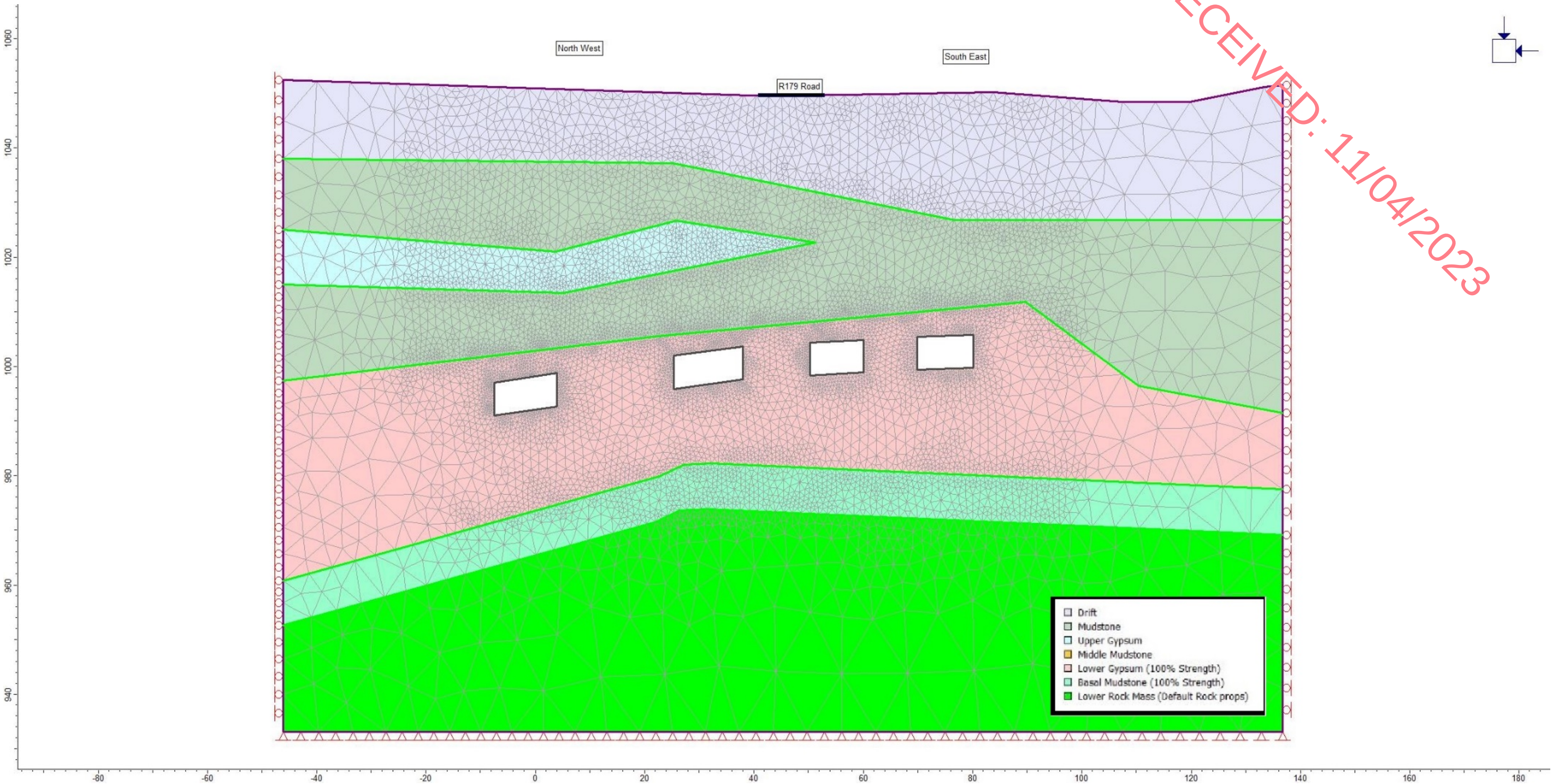

Figure 6-12: R179 Location 3 Cross Section

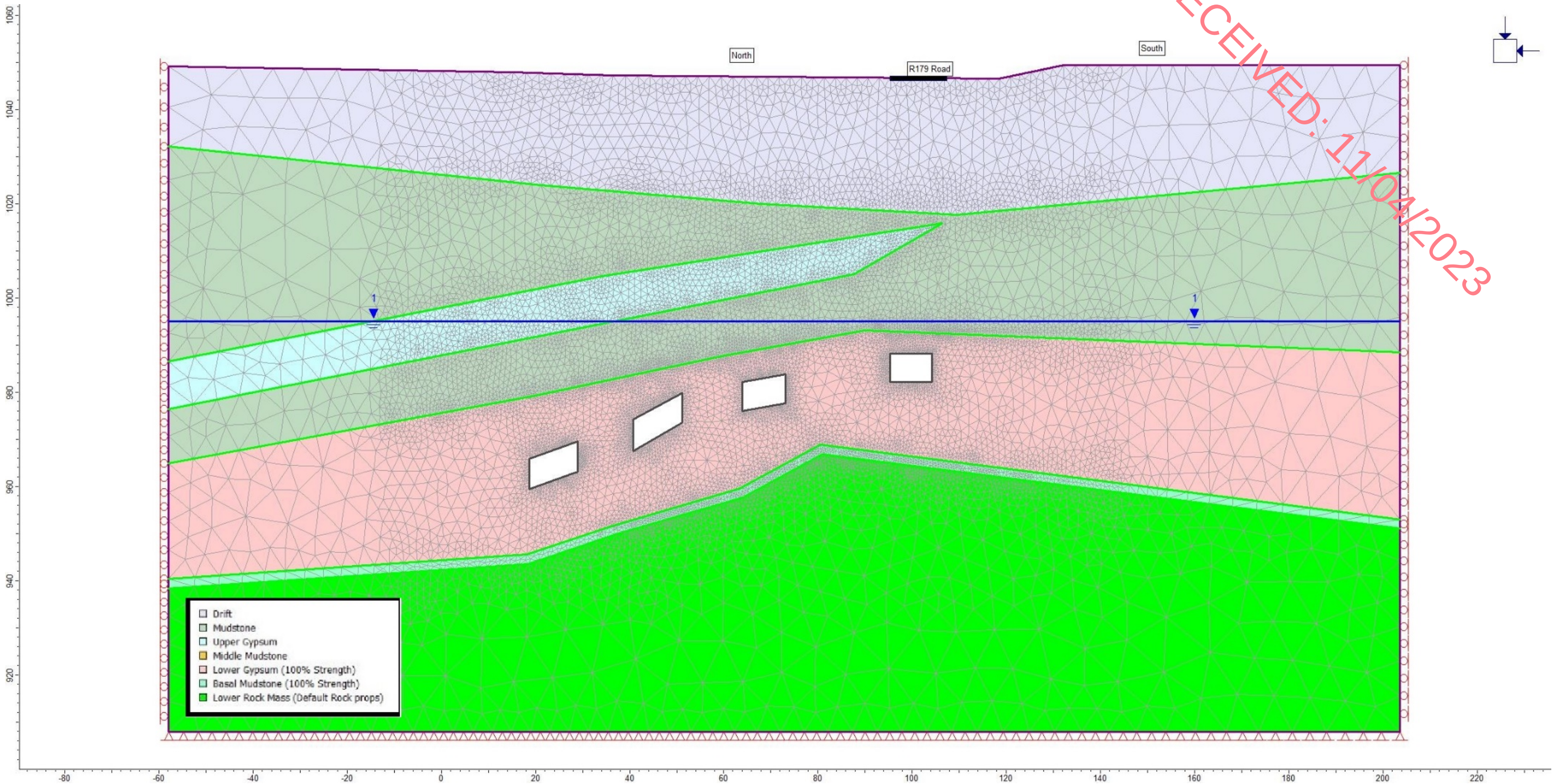

Figure 6-13: R179 Location 4 Cross Section

6.2 Predictive Analysis Calibration

Only one of the cross sections chosen for predictive analysis has been subject to historic subsidence. This is the cross section L4900 East. The road and land to the south of the road have undergone progressive subsidence since the late 1990's. The subsidence has been subject to detailed and ongoing assessment by Trinity College, Dublin. The cause of the subsidence was due to 6m high gypsum pillars bedding down into the underlying basal mudstone that had been weakened by the presence of water sitting on top of the mudstone. The gypsum floor beam in this area was either very thin or completely absent. Over the 20 or so years that this subsidence has been active the maximum surface displacement has been just over 1 metre located several metres to the south of the L4900 road.

Whilst SRK's work here is not to provide a definitive back analysis of this subsidence feature, to achieve a realistic simulation of the measured surface displacement along this cross-section line the following strength parameters of the lower gypsum and basal mudstone needed to be applied to the model:

- Basal Mudstone: Lower bound strength parameters had to be reduced by 50% to 60% below those given in Table 3-2. This simulated progressive strength reduction due to a combination of mudstone softening by the introduction of water and the pressure of the loading imparted on the mudstone by the pillars.
- Lower Gypsum: Lower bound strength parameters used for the 12m high pillars – GSI of 40, cohesion of 200KPa, friction angle of 43°. Note that the average GSI for the 6m high gypsum pillars is 53 (Table 3-2), 30% higher than that required to cause subsidence along this cross section.

For the predictive analysis of the areas of investigation an initial analysis was run using the gypsum cohesion and friction angle associated with the average GSI of the 6m pillars. The models were also run with gypsum pillar strength reduced to the friction angle and cohesion associated with lower bound GSI of 40. These strength values are presented in Table 6-1 below. Apart from on the L4900 East cross section basal mudstone strength was maintained at the lower bound value given in Table 6-2.

Table 6-1: Gypsum Pillar Strength Parameters Used in Predictive Analysis

Case	GSI	Cohesion (kPa)	Friction Angle (°)	Comments
Gypsum Average	53	277	48	From mapping of 6m high pillars
Gypsum Weakened	40	200	43	Strength required to simulate magnitude of measured surface subsidence along L4900 road.

Interpretation of the results has been carried out with reference to total model displacements for the following cases:

1. Expected rock mass strength conditions – the average pillar strength measured underground by SRK in 1999-2004.

2. Weakened rock mass conditions - the reduced pillar strength required to cause subsidence of a magnitude measured above the eastern end of the L4900 road.

3. Failure rock mass strength conditions – the percentage that the weakened rock mass condition would need to be reduced to cause complete pillar failure and significant (>1m) subsidence and surface.

The results of the predictive analyses, along with a discussion of the results, are presented in the following sections. Further details are presented in Appendix C.

6.3 Predictive Analysis Results

6.3.1 L4900 W Cross Section

Section Location and Description

This cross-section is located below the NW end of the L4900 road, about 120 m east of the Drumgoosat school road junction (Figure 6-2).

At this location, the mine rooms are about 44 m below ground. There is a large pillar immediately below the road. The pillars to the south of the road are large; however, there are two smaller pillars located on the north side of the road. The edge of the mine is about 50 m north of the road.

The geology interpretation indicates that the gypsum floor beam below the workings is between 2.5 m and 5 m thick. The top of the lower gypsum is at least 9 m above the mine roof.

This area of the mine is at a level of 1003 m, above the current maximum level of flood water in the mine at about 995mL. This area is therefore dry and has been analysed as such.

Expected Conditions

The analysis shows stable mine conditions at expected gypsum strength, with only 2-3mm of deformation at surface.

Weakened Conditions

There is no significant further deterioration at weakened gypsum strength, with 2-3mm of surface deformation being simulated by the FEM.

Failure Condition

When the gypsum pillar strength is reduced to 50% of weakened strength there is only about 9mm of subsidence on surface. This can be attributed to the large pillar separating the rooms on this cross section.

6.3.2 L4900 Central Cross Section (Maxwell and Kiernan Properties)

Section Location and Description

This cross-section is orientated approximately NS and is located below the central area of the L4900 road about 520 m NW of the R179 junction (Figure 6-3).

At this location, the mine rooms are about 25 m below ground. There are 7 pillars below and either side of the road. The Maxwell and Kiernan properties are located some 60 m to the north of the mine limit.

The geology interpretation indicates that north of the road, the rooms are towards the top of the gypsum seam. Here, the gypsum floor beam below the workings is about 15 m thick and the gypsum roof beam is about 2 to 3 m thick. South of the road, the rooms are located towards the base of the gypsum seam. Here, the floor beam is about 5 m thick and the roof beam is about 10 m thick.

The L4900 Location 2 cross sections contains, based on anecdotal information, may contain two 6m high mining levels within the lower gypsum separated by an unknown thickness of lower gypsum beam between the upper and lower levels. These are not indicated on any mine plans. To simulate this possibility SRK has analysed two cross sections at this location, one containing 6m high rooms and one containing 12m high rooms to simulate double level mining.

This area of the mine is at a level of 1020 m, above the current maximum level of flood water in the mine at about 995mL. This area is therefore dry and has been analysed as such.

Expected Condition

The analysis shows stable mine conditions at expected strength with zero surface deformation below the road and the properties of Mr Maxwell and Mr Kiernan for both the 6m high rooms as surveyed and the 12m high rooms if they were to be present.

Weakened Condition

When using weakened gypsum pillar strength there is 10mm surface deformation to the south of the road. There is zero surface deformation below the road and the properties of Mr Maxwell and Mr Kiernan for both the 6m high rooms and the 12m high rooms.

Failure Condition

Mine failure occurs when strength of the gypsum is reduced to 50% of weakened strength. However, the subsidence which is about 2200mm is located about 40m south of the L4900 road. There is little to no surface deformation below the properties of Mr Maxwell and Mr Kiernan for both the 6m high room and 12m high room cases.

6.3.3 L4900 East Cross Section

Section Location and Description

This cross section is located towards the SE of the L4900 road about 250 m NW of the R179 junction (Figure 6-5).

At this location, the mine rooms are about 50 m below ground. The edge of the mine is located immediately below the road. There are several small pillars to the south of the road. This area was subject to surface subsidence in the late 1990s, resulting from pillars punching through into the weaker basal mudstone.

The geology interpretation indicates that the gypsum floor beam below the workings is about 2 m thick. The top of the lower gypsum is at least 15 m above the mine roof.

This area of the mine is at a level of 990 m, just below the current maximum level of flood water in the mine at about 995mL. This area is potentially flooded and therefore has been analysed as both dry and flooded.

Expected Condition

There is no deformation at expected strength to report for this cross section as pillar strength has already been weakened by a historical subsidence event.

Weakened Condition

SRK notes that this area has been subject to subsidence in the late 1990s, with about 1.0 m of surface subsidence having been experienced by pillars bedding down in wet basal mudstone. To simulate this level of initial subsidence, the basal mudstone needs to be reduced in strength to 60% of its lower bound strength and the lower gypsum strength reduced to its weakened state.

Failure Condition

To be able to generate significant mine failure displacement along this cross-section the gypsum strength needs to be reduced to 80% of its weakened state. This generates an about 3 m of subsidence at surface.

6.3.4 Rafferty and Ward Property Cross Section

Section Location and Description

This cross section is orientated NE-SW and cuts through both properties and extends through the Community Centre and the failed 12 m high pillar group. Mr Rafferty's property is located immediately above the mine limit. Mr Ward's property is above unmined ground about 75 m NE of Mr Rafferty's property (Figure 6-6).

At this location, the mine rooms are about 60 m below ground.

The geology interpretation indicates that the gypsum floor beam below the workings is about 2.5 m thick. The top of the lower gypsum varies between 2 m and 8 m above the mine roof.

This area of the mine is at a level of 990 m, just below the current maximum level of flood water in the mine at about 995mL. This area is potentially flooded and therefore has been analysed as partly flooded.

a) Rafferty Property

Expected Condition

At expected 6m pillar strength surface deformation is 15mm under the Community Centre and 7mm under the Rafferty property.

Weakened Condition

At weakened 6m pillar strength the surface deformation is 190mm under the Community Centre and 22mm under the Rafferty property. These deformations are a function of the disturbance zone failure.

Failure Condition

Extensive 6m pillar failure occurs when strength is reduced to 50% of weakened strength, with about 4 m of surface deformation under the Community Centre. However, the deformation under the Rafferty property remains at 22 mm.

b) Ward Property

Expected Condition

At expected 6m pillar strength surface deformation is 15mm under the Community Centre and 0mm under the Ward property.

Weakened Condition

At weakened 6m pillar strength the surface deformation is 190mm under the Community Centre and 0mm under the Ward property. These deformations are a function of the disturbance zone failure.

Failure Condition

Extensive 6m pillar failure occurs when strength is reduced to 50% of weakened strength, with about 4 m of surface deformation under the Community Centre. However, the deformation under the Ward property remains at 0 mm (zero).

6.3.5 Martin Property Cross Section

Section Location and Description

This cross section is orientated NE-SW and passes over the Martin property, which is located above the mine limit about 50 m away from the R179 road. This road crosses the section line 150 m west of the section line end. The road is located immediately above a very large pillar. There are also large pillars to the south of the Martin property (Figure 6-7).

At this location, the mine rooms are about 40 m below ground and there are six moderate to large pillars on the cross-section.

The geology interpretation indicates that the gypsum floor beam below the workings is about 5 m or greater in thickness. The top of the lower gypsum is about 12 m above the mine workings.

This area of the mine is at a level of 1010 m, above the current maximum level of flood water in the mine at about 995mL. This area is therefore dry and has been analysed as such.

Expected Condition

The analysis shows stable mine conditions at expected gypsum strength with 2mm of subsidence on surface below the Martin property and the R179 road.

Weakened Condition

The analysis shows stable mine conditions at weakened gypsum strength with 4mm of subsidence on surface below the Martin property and the R179 road.

Failure Condition

Mine failure begins to occur when the gypsum strength is reduced to 50% of weakened, with about 1.08 m of surface deformation located between the road and the Martin property.

6.3.6 R179 Location 1 Cross Section

Section Location and Description

This cross section is orientated NS and is located at the position of the Community Centre entrance gate (Figure 6-8).

At this location, the mine rooms are about 40 m below ground and there are five moderate size pillars below and on either side of the road.

The geology interpretation indicates that the gypsum floor beam below the workings is about 7 m or greater in thickness. This thickness of floor beam effectively eliminates the possibility of pillars punching through into the underlying Basal Mudstone. The top of the lower gypsum is a minimum of 6 m above the mine workings.

This area of the mine is at a level of 1000 m, above the current maximum level of flood water in the mine at about 995mL. This area is therefore dry and has been analysed as such.

Expected Condition

The analysis shows stable mine conditions at expected gypsum strength 2mm of subsidence on surface.

Weakened Condition

At weakened strength the subsidence at surface does not increase and remains at 2mm.

Failure Condition

Mine failure begins to occur when the gypsum strength is reduced to 50% of weakened strength with about 2m of surface deformation located 20m east of the R179 road.

6.3.7 R179 Location 2 Cross Section Through Pillar R21

Section Location and Description

This cross section is located across the R179 road, 120 m SE of the location 1, where the road lies above and adjacent to several small area pillars (Figure 6-9).

At this location, the mine rooms are about 42 m below ground. The geology interpretation indicates that the gypsum floor beam below the workings is on average 14 m in thickness. The top of the lower gypsum is a minimum of 10 m above the mine workings.

This area of the mine is at a level of 1000 m, above the current maximum level of flood water in the mine at about 995mL. This area is therefore dry and has been analysed as such.

Expected Condition

The analysis shows stable mine conditions at expected gypsum strength with zero subsidence on surface below the R179 road.

Pillar R21 is stable at expected strength condition.

Weakened Condition

The analysis shows stable mine conditions at weakened gypsum strength with 4mm of subsidence on surface below the R179 road and 8mm above Pillar R21.

Pillar R21 is stable at weakened strength condition.

Failure Condition

Mine failure begins to occur when the gypsum strength is reduced to 50% of weakened strength with about 3.7m of surface deformation located 40m to the east of the road above Pillar R21.

6.3.8 R179 Location 2 Cross Section Through Pillar R12

Section Location and Description

This cross section is located across the R179 road, 120 m SE of the location 1, where the road lies above and adjacent to several small area pillars (Figure 6-10).

At this location, the mine rooms are about 42 m below ground. The geology interpretation indicates that the gypsum floor beam below the workings is on average 14 m in thickness. The top of the lower gypsum is a minimum of 10 m above the mine workings.

This area of the mine is at a level of 1000 m, above the current maximum level of flood water in the mine at about 995mL. This area is therefore dry and has been analysed as such.

Expected Condition

The analysis shows stable mine conditions at expected gypsum strength with 2mm of subsidence on surface below the R179 road and 2mm of subsidence north and south of the road.

The model indicates that pillar R12 is stable at expected strength condition.

Weakened Condition

The analysis shows stable mine conditions at weakened gypsum strength with 4mm of subsidence on surface below the R179 road and 4mm of subsidence north and south of the road.

The model indicates that pillar R12 is stable at weakened strength condition.

Failure Condition

Mine failure begins to occur when the gypsum strength is reduced to 50% of weakened strength with about 3m of surface deformation occurring below the R179 road.

6.3.9 R179 Location 2 Cross Section Through Pillar R8

Section Location and Description

This cross section is located across the R179 road, 120 m SE of the location 1, where the road lies above and adjacent to several small area pillars (Figure 6-11).

At this location, the mine rooms are about 42 m below ground. The geology interpretation indicates that the gypsum floor beam below the workings is on average 14 m in thickness. The top of the lower gypsum is a minimum of 10 m above the mine workings.

This area of the mine is at a level of 1000 m, above the current maximum level of flood water in the mine at about 995mL. This area is therefore dry and has been analysed as such.

Expected Condition

The analysis shows stable mine conditions at expected gypsum strength with 2mm of subsidence on surface below the R179 road.

The model indicates that pillar R8 is stable at expected strength condition.

Weakened Condition

The analysis shows stable mine conditions at weakened gypsum strength with 5mm of subsidence on surface below the.

The model indicates that pillar R8 is stable at weakened strength condition.

Failure Condition

Mine failure begins to occur when the gypsum strength is reduced to 50% of weakened strength with about 1.6m of surface deformation located on the R179 road above Pillar R8.

6.3.10 R179 Location 3 Cross Section

Section Location and Description

This cross section is orientated approximately NW-SE across the R179 road, 130 m SE of Location 2, where the road lies above several medium size pillars (Figure 6-12).

At this location, the mine rooms are about 45 m below ground and there are three moderate size pillars below and on either side of the road.

The geology interpretation indicates that the gypsum floor beam below the workings is generally greater than 14 m in thickness. The top of the lower gypsum is about 4 m above the mine workings.

Apart from the lowest room on the cross section, this area of the mine is at a level of 1000 m, above the current maximum level of flood water in the mine at about 995mL. This area is therefore dry and has been analysed as such.

Expected Condition

The analysis shows stable mine conditions at expected gypsum strength with 2mm of surface subsidence below the R179 road.

Weakened Condition

At weakened gypsum strength the analysis shows stable mine conditions with deformation below the R179 road of 6mm.

Failure Condition

At 50% of weakened strength there is no appreciable increase in surface movement. This is because of the presence of large stable pillars separating the rooms.

6.3.11 R179 Location 4 Cross Section

Section Location and Description

This cross-section is orientated NW-SE and is located across the R179 road, 130 m SE of the Location 3 cross section, where the road lies above several medium area pillars (Figure 6-13).

Immediately below the R179 road here is a 6 m high room at a depth of 56 m and the mine slopes steeply to the north. There are three large pillars on the cross-section line. The northernmost room on the cross section is 82 m below surface.

The geology interpretation indicates that the gypsum floor beam below the workings is generally greater than 14 m in thickness. The top of the lower gypsum is 4 m or higher above the mine workings.

This area of the mine is at a level of 960 m, which is below the current maximum level of flood water in the mine at about 995mL.

Expected Condition

The analysis shows stable mine conditions at expected strength with zero surface deformation below the R179 road.

Weakened Condition

At weakened strength the analysis shows stable mine conditions with surface deformation below the R179 road increasing to 18mm.

Failure Condition

Mine failure only occurs when the gypsum strength is reduced to 50% of peak with about 5.2m of surface deformation located about 60m to the east of the R179 road.

6.4 Discussion of Results

The back analysis has indicated that the subsidence event has been caused by a material change; that is, mine flooding in an area containing double height 12 m rooms, as opposed to the standard 6 m high rooms excavated throughout the rest of the mine. For the failure to occur, the strength of the rock needed to be reduced by between 10% and 20% below the weakened strength determined from SRK's 2001 underground mapping and investigation work and the gypsum floor beam must be very thin.

All the predictive cross-sections analysed contain 6 m high rooms and only three of the twelve cross sections analysed have been subject to the recent flooding event. The remaining nine areas has not been subject to material change.

At average and weakened strength, the FE models generated total displacements of the order of millimetres. The mine has been surveying level points along both the R179 and L4900 roads as well as around the Rafferty property for 15 years or so. The surface levelling data indicates settlement type movement of millimetres per year, which is typical for ground overlying a shallow underground mine. The rate of change of movement has been constant over the monitoring period, indicating that the mine below these areas is stable. The results of the FE modelling are therefore consistent with the measured surface movements. The FE modelling is considered to provide a reasonable simulation of current and potential future mine performance.

Room and pillar is a man-entry mining method and pillars are normally designed to a high factor of safety for man-entry, therefore, pillar strength degradation over time should be minimal providing the pillars have been excavated to design.

To create mine failure and significant disturbance at surface, the models indicate that a strength reduction of up to 30% of weakened strength in flooded mine conditions increasing to 50% of the weakened strength in dry mine conditions will be required for this to happen. For the lower gypsum, a 50% in weakened strength requires that the GSI is reduced from 40 to 20. A rock mass with a GSI of 20 will require the blocky gypsum rock mass to degrade to a disintegrated, heavily broken rock mass with little inherent strength, as shown in the GSI chart in Figure 7-13. In SRK's experience it would be extremely unlikely that gypsum would degrade to this degree in a 50 year abandoned mine life.

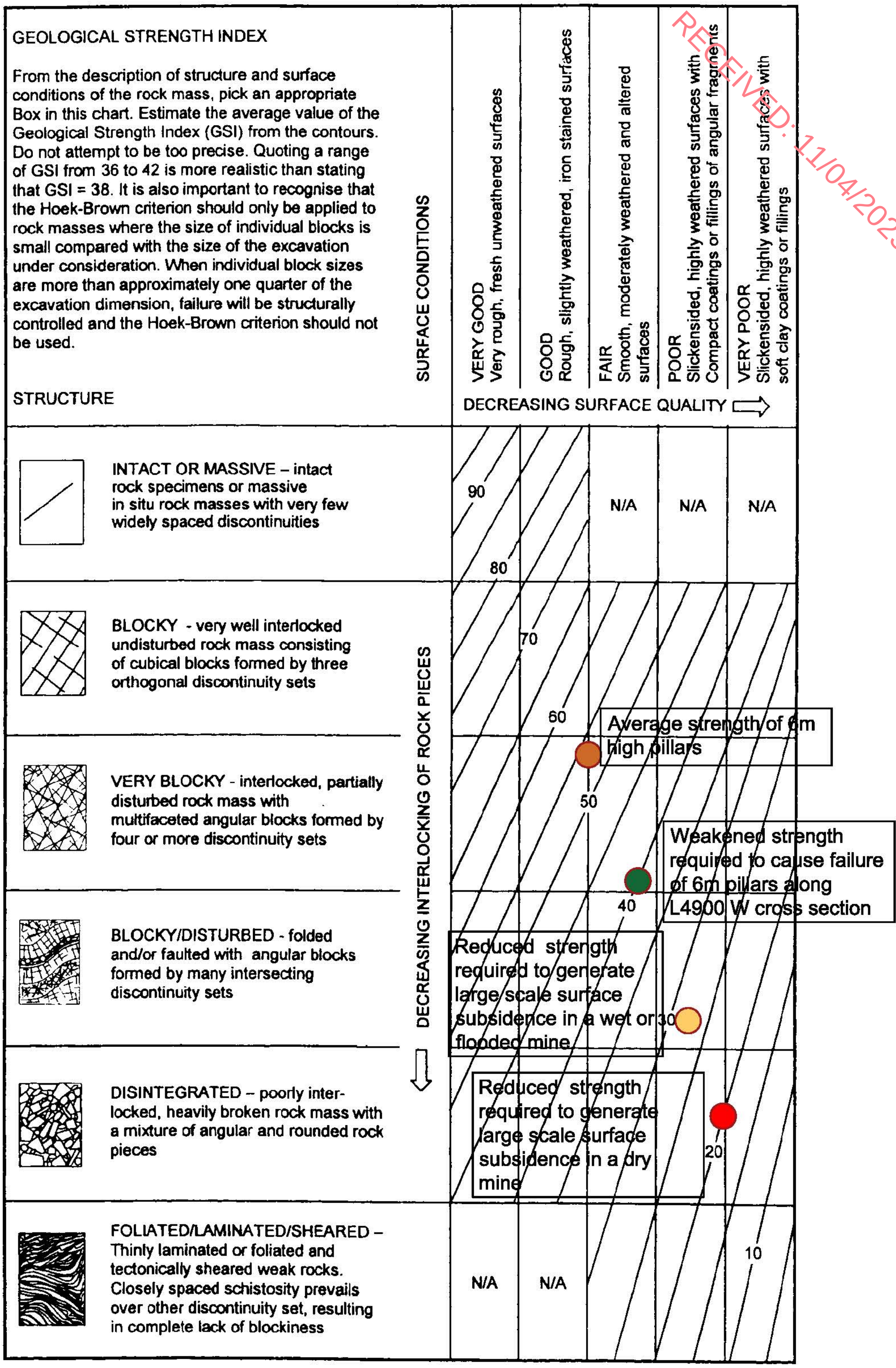

Figure 6-14: GSI Chart showing Measured Values and Reduced Values Required to Cause Mine Failure

6.5 Conclusions and Recommendations

6.5.1 General Conclusions

The predictive numerical modelling indicates that the mine is stable below the cross sections analysed for both the average and weakened gypsum strength cases. These results are consistent with the result of surface surveying that has been taking place along the roads and near buildings since the risks associated with these areas were first identified by SRK in their 1999 report to the EMD.

None of the areas analysed have been affected by the subsidence event that has taken place below the GAA playing fields. Providing the mine water level is not allowed to increase and submerge the pillars in the areas analysed it is highly unlikely that the gypsum pillars will deteriorate to an extent that will result in failure of the mine pillars.

It is SRK's opinion based on the results of the modelling that the level of risk of mine failure affecting the surface infrastructure analysed has not changed or worsened since SRK carried out its original studies between 1999 and 2004.

Table 6-2 summarises the results of both the back analyses and predictive analyses. The following sections describe the requirements for further action.

6.5.2 L4900 Road

The results of the predictive analyses indicate that there is very low risk of mine instability affecting the stability of the road through the west and central sections. Given subsidence has taken place over the last 20 or so years below the eastern end of the road further settlement is likely but there is a very low risk of significant new subsidence on the road. Surface survey monitoring of the whole road should be continued at a more frequent rate to provide comfort that subsidence is not increasing. The frequency of monitoring can be reduced when the rate of subsidence is shown to be not increasing.

6.5.3 Maxwell and Kiernan Property

These properties are not affected by the mine and in the unlikely event that pillar failure occurs will not be affected. No further action required.

6.5.4 Ward Property

The analysis shows that the property has not been affected by underground mining and will not be affected in the unlikely event that pillar failure occurs. No further action required.

6.5.5 Rafferty Property

This property lies within the safety zone around the Community Centre disturbance event. There are indications that it has been affected by this event. Although the surface deformations generated by the predictive modelling are small, differential ground movement may give rise to structural damage. A structural engineers advice should be sought as to whether the integrity of the building has been compromised by the ground movement.

6.5.6 Martin Property

The Martin property is located above the mine abutment. The analysis shows that the property has not been affected by underground mining and will not be affected in the unlikely event that pillar failure occurs. No further action required.

6.5.7 L179 Road

The thick gypsum floor beam under the mine workings that lie below the road eliminates the possibility of pillar foundation failure, thus improving the long-term stability of the mine workings. Based on the results of the modelling, it is considered that there is a very low risk of mine failure affecting the road.

Deterioration in pillar strength will be higher the smaller the pillars are. Whilst the analysis has shown that the small pillars below the R179 road are stable it has been agreed that a borehole camera will be dropped into the underground workings close to the pillars to determine their current condition and confirm the results of these analyses.

Table 6-2: Back Analysis and Predictive Analysis Results Summary

Type of Analysis	Section	Depth Below Surface (m)	Mine Level (m)	Floor Beam Thickness (m)	Roof Beam Thickness (m)	Criteria for Surface Subsidence that Must be Satisfied: 12m High Pillars?	Pillars Submerged?	Basal Mudstone close to Floor of Rooms?	Max. deformation at Average 6m Pillar Gypsum (GSI 53) and Basal Mudstone strength (mm)	Max. deformation at Weakened Gypsum (GSI 40) and Basal Mudstone strength (mm)	Strength Reduction below Weakened to Cause Significant (>1m) Surface Deformation (%)	Risk of Mine Failure	Comments
Failure Back Analysis	E - W Section Through Subsidence Zone	60	980	2 - 3	4 - 6	Yes	Yes	Yes	N/A	40	20	Failed	Area of surface Subsidence - All criteria satisfied
	N - S Section Through Subsidence Zone	67	980	2 - 3	4 - 6	Yes	Yes	Yes	N/A	57	10	Failed	Area of surface Subsidence - All criteria satisfied
	E - W Section Dry simulation	60	980	2 - 3	4 - 6	Yes	No	Yes	N/A	22	20	Very Low	Failure not indicated
	E- W Section 6m high pillars simulation	60	980	2 - 3	4 - 6	No	Yes	Yes	N/A	8	30	Very Low	Failure not indicated
Predictive Analysis - Roads & Properties	L4900 W	44	1003	2.5 - 5.0	+ 9	No	No	Yes	2	2	50	Very Low	Failure unlikely
	L4900 Central	25	1020	5	10	No	No	No	0	10	50	Very Low	Failure unlikely
	L4800 Central - 12m high rooms	25	1020	5	10	Not Confirmed	No	No	0	10	50	Very Low	12m high pillars, if present, indicated as being stable
	Maxwell and Kiernan Properties	25	1020	5	10	No	No	No	0	10	50	Very Low	Maxwell and Kiernan properties is not undermined and is outside the zone of influence of the mine
	L4900 E (This section subject to historical subsidence)	50	990	1 - 2	15	No	Floor wet	Yes	N/A	N/A	0	Very Low	Historic subsidence of 1m has occurred. Further significant subsidence unlikely
	Rafferty Property	60	990	2.5	2 - 8	No	Partly submerged	Yes	7	22	20	Very Low	Rafferty property lies within the zone of influence of the playing field subsidence zone and has been affected by the subsidence event
	Ward Property	60	990	2.5	2 - 8	No	Partly submerged	Yes	0	0	20	Very Low	Ward property is not undermined and is outside the zone of influence of the mine
	Martin Property	40	1010	+ 5	12	No	No	No	2	4	50	Very Low	Failure unlikely
	R179 Location 1	40	1000	+ 7	+ 6	No	No	No	2	2	50	Very Low	Failure unlikely
	R179 Location 2 - Pillar R21	42	1000	14	10	No	No	No	8	8	50	Very Low	Failure unlikely. Investigate integrity of the small pillars either side of the road.
	R179 Location 2 - Pillar R12	42	1000	14	10	No	No	No	4	4	50	Very Low	Failure unlikely. Investigate integrity of the small pillars either side of the road.
	R179 Location 2 - Pillar R8	42	1000	14	10	No	No	No	2	5	50	Very Low	Failure unlikely. Investigate integrity of the small pillars either side of the road.
	R179 Location 3	45	1000	+ 14	4	No	No	No	2	6	50	Very Low	Failure unlikely
	R179 Location 4	56	960	+ 14	+ 4	No	Yes	No	0	18	50	Very Low	Failure unlikely

For and on behalf of SRK Consulting (UK) Limited

Neil Marshall,	Richard Oldcorn,
Corporate Consultant - Geotechnical,	Managing Director,
Project Manager	**Project Director**
SRK Consulting (UK) Limited	SRK Consulting (UK) Limited

APPENDIX

A DISTURBANCE ZONE BACK ANALYSIS

APPENDIX

B DISTURBANCE ZONE PREDICTIVE ANALYSIS

APPENDIX

C PREDICTIVE ANALYSIS OF MINE AREAS BELOW 3RD PARTY INFRASTRUCTURE

APPENDIX

D DRUMGOOSAT MINE – SRK REPORTS

APPENDIX 7.5
Independent Review of Investigation into Collapse Workings at Drumgoosat - WAI - December 2018

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INVESTIGATION OF THE COLLAPSE OF WORKINGS AT DRUMGOOSAT
AN INDEPENDENT REVIEW OF THE WORKS COMPLETED BY SRK

DECEMBER 2018

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DEPARTMENT OF COMMUNICATIONS, CLIMATE ACTION AND ENVIRONMENT

INVESTIGATION OF THE COLLAPSE OF WORKINGS AT DRUMGOOSAT
AN INDEPENDENT REVIEW OF THE WORKS COMPLETED BY SRK

DECEMBER 2018

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EXECUTIVE SUMMARY

In June 2018 Drummond Mine intersected a fault in the underground workings. This resulted in significant water ingress which threatened the continued operation of the mine and had to be pumped out of the mine. Saint-Gobain Construction Products (Ireland) Limited (SG) were unable to discharge the water to the River Bursk through their normal discharge point as there was limited flow in the river, elevated sulphate, above discharge consent level, in the pumped water and a perceived risk of impact on the environment including the downstream water supply. SG therefore pumped the water to the closed Drumgoosat Mine, closed 1989, in line with their past practice of pumping limited water volumes to Drumgoosat during periods of low flow in the river.

On 24 September 2018 there was a collapse affecting facilities at the Magheracloone GAA club and the associated Community Centre when ground subsidence occurred.

The area of subsidence, plus a 50m exclusion zone, was set up to prevent public access. At the same time, it was known that the Drumgoosat workings encroached under the main R179 and L4900 roads and there was concern that whatever had affected the football pitch could affect the support of the roads. As a result, Monaghan County Council (MCC) closed both roads to the public.

SRK Consulting (SRK), a consultancy company working for SG, the operators of Drummond and Drumgoosat Mines, had previously undertaken work on the underground support of the area and were commissioned to investigate the major subsidence occurrence.

After reporting their conclusions to a meeting held at the MCC offices, attended by MCC, DCCAE and EPA, it was decided that Wardell Armstrong (WA) would check the works carried out by SRK in order to independently verify the conclusions reached by SRK.

WA carried out the independent check of the work by modelling the geotechnical aspects of the support system using FLAC software.

Whilst some of the aspects regarding the mechanism of collapse were questioned, the conclusions reached by WA agreed with those of SRK. These conclusions are basically

1. Pumping of water to Drumgoosat affected the stability of the pillars.
2. The collapse area is contained within the area of subsidence.
3. The mine workings under the roads R179 and L4900 are stable.

WA believe the subsidence occurrence was caused by collapse of an area where 12m high pillars had been formed during historical mining. The water being pumped from Drummond Mine had submerged the pillars and had affected the stability of those pillars by lubricating the cracks and fissures within the pillars causing them to collapse.

The 12m pillars collapsed causing a domino effect and putting dynamic forces on local 6m pillars which also collapsed. The subsidence area was then contained by larger static pillars which limited the collapse area.

WA's brief was to independently verify the geotechnical modelling carried out by SRK related to the main subsidence area and the roads infrastructure.

A section of the R179 was closed to the public, portions of which had been underworked by the room and pillar workings of the Drumgoosat Mine. WA therefore also checked the SRK work associated with the support stability of these sections of the road, and one area was specifically targeted for further investigation as support in this area constituted a 'slender' pillar (pillar 12).

The modelling did not predict any significant subsidence, although the result did not mirror the results of the monitoring of the area, which showed up to 30mm movement over approximately twenty years. A laser scanning tool was put down two boreholes, one either side of the pillar to physically check its condition. These scans showed the pillar to be intact with no stability issues.

However, because of it slender nature, WA recommends that this pillar be monitored in the future both by measuring surface movement above the pillar and by regular, possibly annual, laser scanning of the pillar integrity itself.

WA also carried out checks on the modelling of sections under the L4900 road which had also been closed. The modelling did not indicate any instability.

In conclusion WA were able to confirm the conclusions reached by SRK, that large scale movement of the surface outside of the main subsidence area was unlikely and that the support of the two roads, the R179 and the L4900, was intact and robust.
WA confirmed this to MCC and the roads were reopened to the public.

1 INTRODUCTION

In June 2018 Drummond Mine suffered an inflow of water which resulted in large quantities, up to 450m^3/hr having to be pumped out of the mine to prevent it from flooding.

Saint-Gobain Construction Products (Ireland) Limited (SG) were unable to discharge the water to the River Bursk through their normal discharge point, as there was limited flow in the river, elevated sulphate, above discharge consent level, in the pumped water and a perceived risk of impact on the environment including the downstream water supply. SG therefore pumped the water to the closed Drumgoosat Mine, closed 1989, in line with their past practice of pumping limited water volumes to Drumgoosat during periods of low flow in the river. Water was pumped into Drumgoosat from July 2018.

On 24 September 2018, a collapse of ground occurred at the Magheracloone GAA club and the associated Community Centre resulting in subsidence of circa 350m diameter and 5m deep at its deepest point, and structural damage to the Community Centre itself. One property adjacent to the Community Centre was also affected with minor cracking.

It was not known whether the area of collapse could enlarge and a 50m wide security area was imposed around the collapse.

Immediately after the collapse, SG had commissioned SRK Consulting (SRK), a consultancy company who had previously looked at the stability of the Drumgoosat area to investigate the causes of the collapse and to assess the potential for further issues, particularly in relation to the safety of nearby roads and property. Sections of two roads (the R179 and L4900) under which the Drumgoosat Mine workings partially extend, were closed as a precautionary measure by Monaghan County Council, until it could be demonstrated that the roads were safe for public use. Due to the urgency of the situation, SRK were required to report within 5 days of being commissioned. Initial investigations by SRK established that the collapse was associated with the water that had been pumped from Drummond Mine.

A meeting of officials was called on 10 October 2018 to discuss the SRK initial findings and the situation in general. Attendance at the meeting included SG, the operators of the closed Drumgoosat Mine, Monaghan County Council (MCC), within whose area the mining operation lies, representatives of the Department of Communications, Climate Action and Environment (DCCAE), Environmental Protection Agency (EPA), SRK Consulting (SRK), and Wardell Armstrong LLP (WA), consultants commissioned by the DCCAE to provide mines inspection and technical advice at licenced mines in Ireland.

1.1 SRK Investigations

SRK considered the situation using data and information from previous commissions which had been undertaken from 1999. The underground of Drumgoosat Mine is not accessible for inspection, therefore SRK used 'back analysis' of the previous data, an acceptable methodology, to model the surface collapse shown in Fig 1:1. Their report produced a conclusion of the cause of the collapse stating 'the back analysis concluded that the GAA playing field failure was a result of the interaction of three unique conditions:

a) 12m high pillars, [underground];
b) Mine water levels increasing and submerging the pillars; and
c) A thin gypsum floor beam.

Figure 1:1 Surface Subsidence & Location of 12m Pillars Below
(Source: SRK Oct 2018 Report)

Discussions at the meeting included comments from MCC that some within the local community questioned the independence of SG and their consultants SRK.

DCCAE, MCC and EPA agreed that in such circumstances the information available should be independently reviewed and decisions made, not on the results of the work carried out by SG and SRK but on the conclusion of the independent review of such works.

As a result, Wardell Armstrong was commissioned by DCCAE to carry out the independent review of selected works, agreed by MCC, EPA and DCCAE.

The reason for 'selected' works was twofold.

1. SRK had undertaken work which was reported in 1999, in 2002 and 2005 and the body of work was comprehensive. To carry out a full independent evaluation of all the work would involve a timescale which was unacceptable to MCC given the urgency of the situation and was not, in WA's opinion, necessary.

2. If the independent review of 'selected' works was undertaken, and the results found to be satisfactory, then in WA's experience, the quality of the SRK work could be endorsed for the remaining works not independently reviewed at that point.

For clarity, the identification of the works selected, including which and what number would be reviewed, was suggested independently by WA based on the observed collapse area and its potential zone of influence; it was not at the suggestion of SG or SRK.

Specifically, the 'selected' works included a section approximately W/E in orientation through the major collapse area and the community centre. In addition, sections developed by SRK, at Locations 2 and 4 shown on plan cutting through the R179 road at different points were also considered.

Figure 1:2 locations of sections through R179
(Source: SRK Oct 2018 Report)

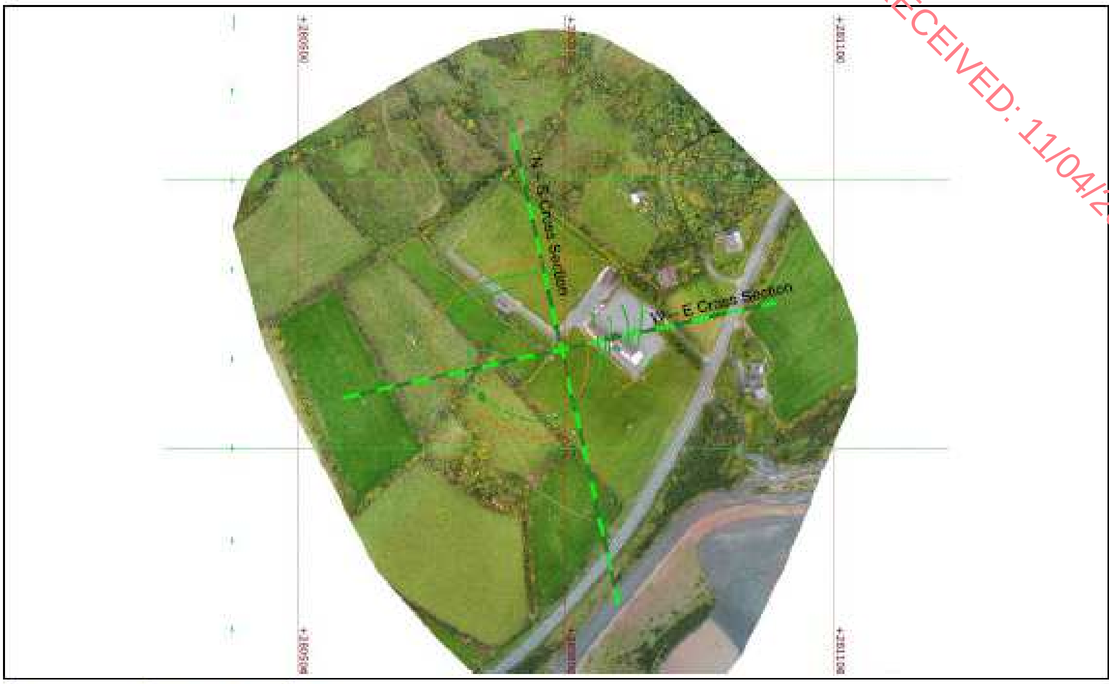

Figure 4-1: Location of Cross Sections

Figure 1:3 N-S & W-E Cross Sections Modelled by SRK
(Source: SRK Oct 2018 Report)

Other areas to review were also identified, such as section through a narrow pillar 12, located outside the collapse area but possibly affected by the pumped water and very close to the R179. This was quickly given priority over Locations 2 and 4 which had no obvious issues regarding changed conditions.

Subsequently the sections covering the smaller road, L4900, were also considered to enable that road to be reopened if safe.

Figure 1:4 Roads R179 & L4900

1.2 Further Background

As stated previously SG had been pumping water into the closed Drumgoosat mine from Drummond mine to alleviate flooding from an inflow that had occurred. The water level in

Drumgoosat had risen from its historic height of between +950 to +970m to +995.2m (and slightly rising).

The water level rose to submerge an area underground where 12m pillars had been formed. The water level is contained on Figure 1.5 and confirmed by drilling subsequent to the surface collapse.

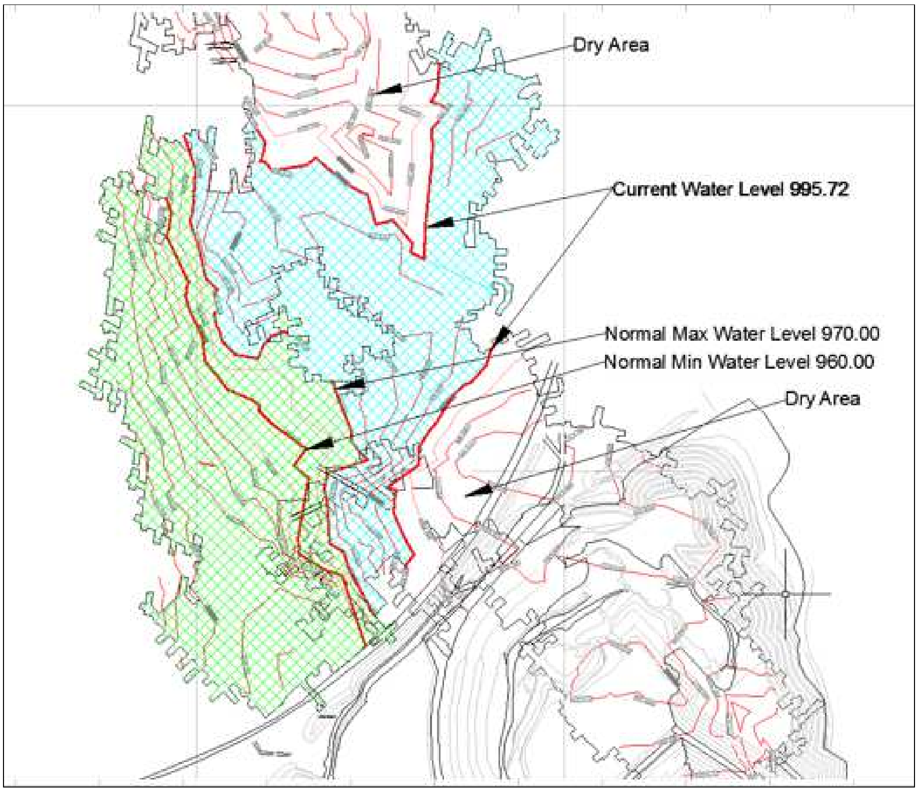

Figure 1:5 Water Level in Drumgoosat Before and After Pumping (Source: St Gobain Mining (Ireland) Ltd)

Previous inspections in 2005 by SRK had identified that the pillars or the ground around the pillars had settled giving rise to limited surface subsidence. However, the reported monitoring showed that the settlement of the surface above the pillars had reduced to 0.02mm per day (approx. 7mm per year). Settlement of the Community Centre was also monitored, the results showing a subsidence rate limited to less than 0.01mm per day and SRK considered the area stable.

However, SRK did consider the area to be an 'intermediate' to 'high risk' as the stable position could only be assumed if conditions affecting the area remained the same. As a result it was decided to monitor the surface for movement and stability has been demonstrated by precise levelling surveys undertaken by SG in recent years.

The deepest depression of the collapse of the surface is located directly over the 12m pillars leading to the conclusion that the pillars or the surrounding strata must have failed and since the condition of those pillars was considered stable before the occurrence then something appears to have changed that condition. The single factor that changed was flooding of the 12m pillar area with the water being pumped from Drummond mine.

1.3 Works Undertaken

WA used FLAC 2D modelling to check and verify the conclusions reached by SRK in relation to predicted ground movement associated with the GAA field, the Community Centre, the R179 and L4900 roads and specific properties on the edge of underground excavations to the North of the L4900 and East of the R179.

Also, since WA could not carry out original studies to determine the rock properties at Drumgoosat for use as parameters in their modelling WA have used those provided by SRK but verified the suitability of such parameters based on their own experience.

1.4 Results

WA considered the rock mass strength of the rock and pillars and have concluded that SRK have used parameters which are conservative. See Table 1:1.

Table 1:1 Rock Mass Strength Parameters used by SRK (source SRK Oct 2018 Report)

Table 2-1: Rock Mass Strength Parameters Used in Numerical Modelling

Rock Unit	Overburden	Gypsum	Mudstone
Intact Strength (MPa)	1	15	5
m_i	9	12	9
GSI	40	40	40
Material Density (t/m³)	2.0	2.3	2.0
Cohesion (kPa)	60	200	130
Friction Angle (°)	19	43	30
Rock Mass Deformation Modulus (MPa)	560	4875	1260
Poisson's Ratio	0.3	0.15	0.3

The parameters used are deemed acceptable and conservative, the assumed rock strength and nature being lower than actual 'in situ' measurements taken in various places at Drumgoosat and Drummond, See Table 1:2. WA has no reason to doubt these figures as they are typical of similar structures elsewhere, therefore WA considers the parameters used to be conservative and includes a contingency of at least 20%.

Table 1:2 GSI Actual Measurement
(Source SRK Oct 2018 Report)

Table 2-2: Gypsum GSI Variability

Data Source	Average GSI	GSI Range
12m High Pillars	48	43 – 52
6m High Pillars	53	41 – 62
Borehole Core	53	45 - 73
GSI used in these Analyses	40	

WA used cross sections provided by SRK for independent analysis. The W/E cross section provided by SRK follows a line through the surface collapse area and continues through the Community Centre. The section includes both the surveyed surface levels and the workings of the Upper and Lower Gypsum Seam. This was checked against the mining plan and showed to be correct and therefore the sections were deemed suitable for analysis.

The WA modelling of the area using FLAC did not initially mirror the extent of the surface collapse, although it did show that the 12m pillars and some 6m pillars to the west were in failure. The failure to the west was halted by a much larger pillar which was stable.

The FLAC software can only model the pillars to the point of failure and not beyond that unless different parameters are used e.g. a situation where the pillars are effectively not there. The model to the point of failure showed that at the point of collapse of the pillars the surface area affected was much smaller than that actually evident on the surface.

WA therefore considered a scenario resulting from the failure of the 12m and the 6m pillars to the west as predicted by FLAC. This included calculating the void space resulting from the collapse of the pillars and modelling the resultant effect on the surface. FLAC modelled the effect of the collapse of the resultant void and this scenario produced a model having a similar surface effect to that evident on the surface in terms of extent and magnitude. Figure 1:6 and Figure 1:7 show the FLAC sections modelled.

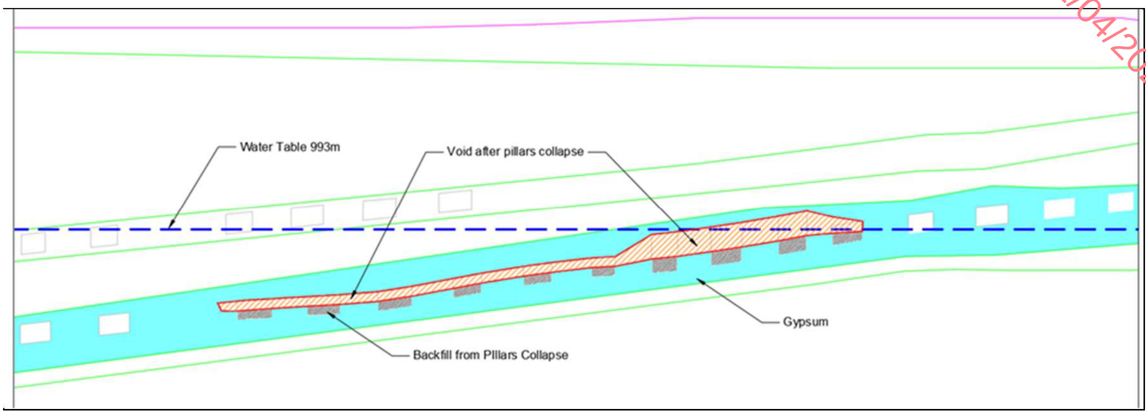

Figure 1:6 W-E Section Showing the Location of Pillars Collapse

WA therefore concluded that the water had changed the condition of the 12m pillars by lubricating the joints and fractures in the 12m pillars which had subsequently collapsed. This caused a domino effect, the dynamic failure causing the 6m pillars to the west to fail also, until contained by the larger (wider) pillar to the west of the subsidence area. The overall collapse was confined by larger static pillars surrounding the collapse area and providing adequate support.

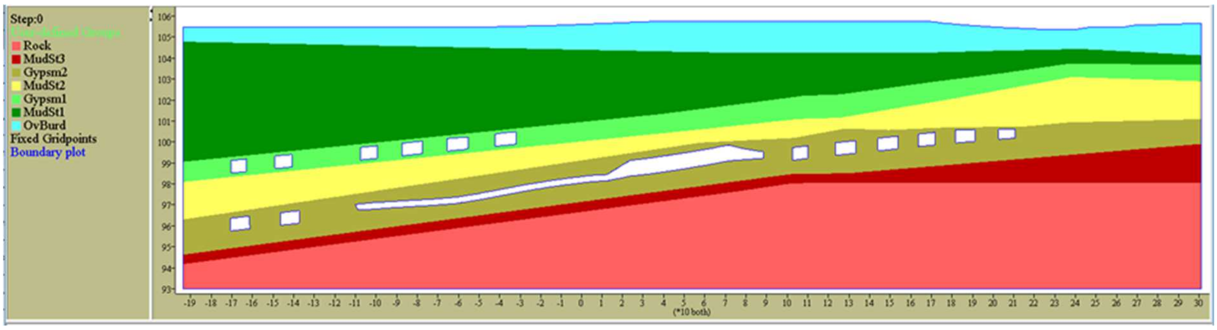

Figure 1:7 FLAC Model Layout of W-E Section Showing the Void after Pillar Collapse

The model also showed that the gradient of the seam resulted in the stress being thrust westward and not eastward and showed the line of maximum stress to the east to equate to the widest surface fractures evident on the surface.

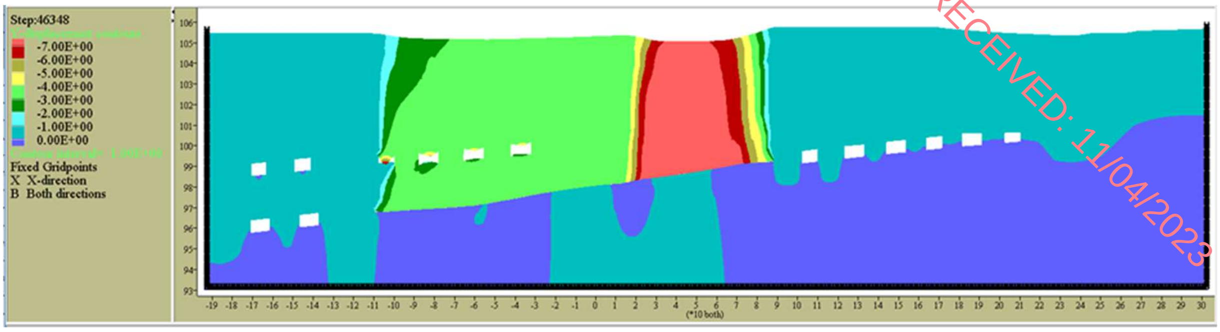

Figure 1:8 Ground Settlement Contours, W-E Section with 12m Pillars Collapse

The modelling, Figure 1:8, shows that the ground settlement is nearly vertical above the collapsed zone and that very little lateral surface lowering has taken place.

The modelling did show that some surface stress would be experienced further to the East of the collapsed area, with some tension cracks, caused by horizontal ground movement, appearing to the west of the R179 as shown in Figure 1:9 but with minimal vertical subsidence.

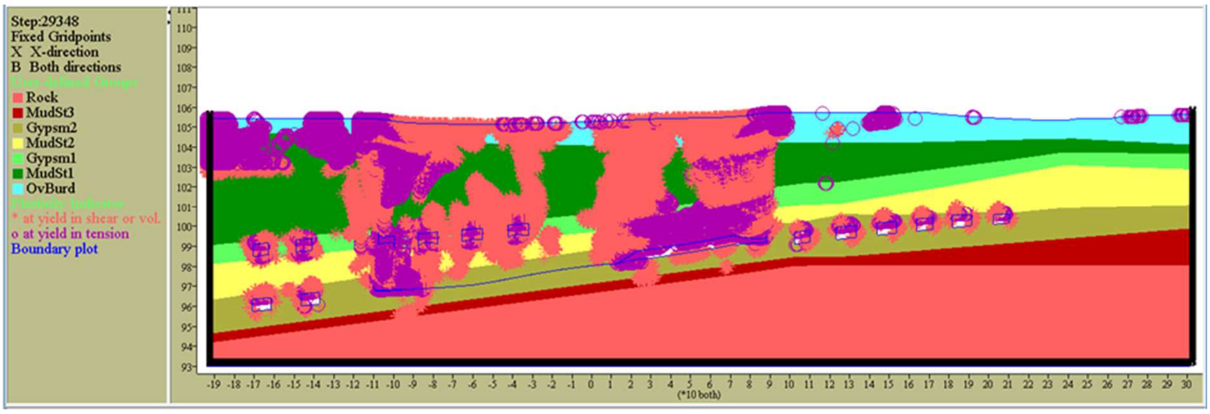

Figure 1:9 Ground Failure due to Ground Settlement, W-E Section with 12m Pillars Collapse

SRK using a different software had come to a slightly different conclusion regarding the cause of the collapse as they had assumed some effect of the water on the mudstone floor and some pillar failure i.e. three factors, but the different scenario has resulted in the same phenomenon at the surface.

SRK concluded that three conditions have to be present underground to create the problem at the surface:

a) 12m high pillars;

b) Mine water levels increasing and submerging the pillars; and
c) A thin gypsum floor beam (less than 2m).

The work WA has done would conclude that only a) and b) are required. WA have therefore looked at the areas where the water has been pumped to, Fig 1.5, and queried whether there are any other areas where 12m pillars occur within that area. SG stated that, as far as it is known there are no other locations where 12m pillars occur, however WA has no means of checking this.

WA can however state that the modelling indicates that the stability of 6m pillars, the normal mining height in this mine, is not affected by being submerged in water. The reason being that at 6m high the pillars do not appear to have cracks through the pillar itself, which affect its integrity, whereas the 12m pillars are classed as 'slender' leading to weaknesses in the pillar which can be affected by the water.

Therefore, WA would comment that, from the modelling evidence available:

1. it is very unlikely that any further surface movement, of the type encountered under the GAA playing field and adjacent at the Community Centre will occur;
2. that the area of major collapse has extended as far as it is likely to go, but
3. some tension does exist at the surface, created by the collapse which may continue to affect the surface outside the collapsed area with smaller settlement movement, millimetres, over a period of time.

The detailed ground surface subsidence curve shown in Figure 1:10 confirms that there is minimal vertical settlement outside of the current evidence of ground movement at the surface.

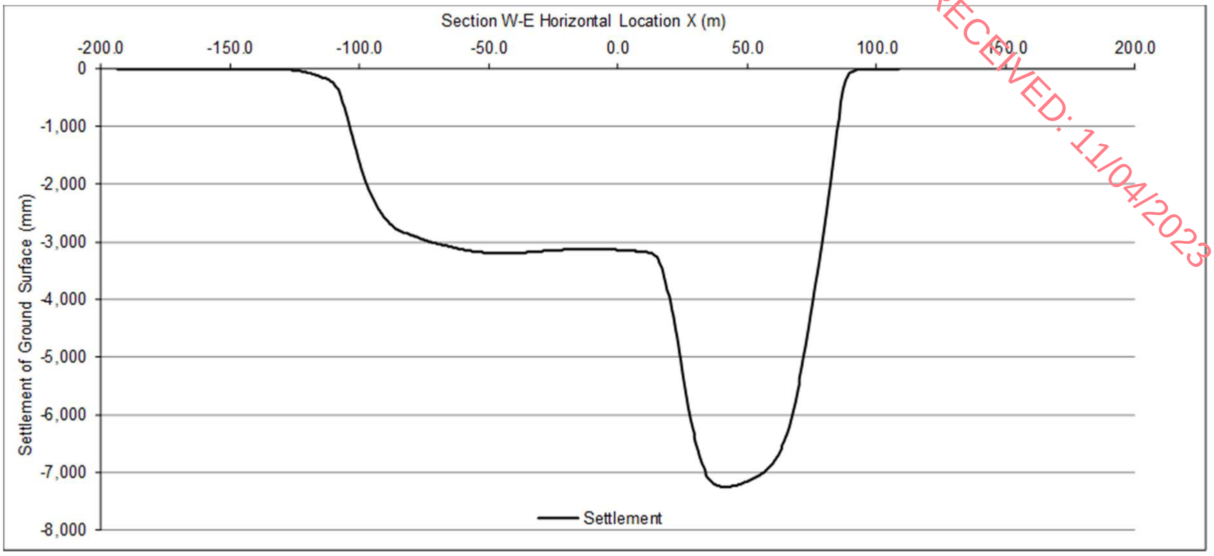

Figure 1:10 Ground Surface Settlement at Section W-E

As a result, Wardell Armstrong is able to state that its independent review of the situation has confirmed the SRK conclusions. Wardell Armstrong therefore believes that there will be very little further ground movement outside the area identified as the area is contained by larger pillars that have been shown to be stable, even in flooded conditions.

1.5 R179 Analysis

Drumgoosat workings undermine part of the R179 road, however the pillar configurations and size are such that the majority of the road is fully supported with only settlement over time of a few millimetres happening which is typical and expected over mine workings. FLAC analysis identifies the risk level to be very low.

However, whilst the majority of the pillars conform to the normal configuration, Pillar 12 appeared on the mining plan to be narrow, a ‘slender’ pillar, and WA were instructed to look particularly at the area where Pillar 12 supported the ground immediately adjacent to the R179. Unlike the 12m pillars, Pillar 12 (6m high but narrow), has not been submerged by the water from Drummond mine and has thick roof and floor gypsum, 9-10m and 14 -15m respectively. Water being pumped from Drummond has flowed past the pillar at floor level but there is no evidence that the pillar itself has been impacted, nor that water has remained on the floor in the vicinity.

WA received an SRK cross section through the pillar which, when modelled by FLAC, matched the SRK results. However, WA was not satisfied that the cross section truly represented the area properly, as the length of the section being considered did not take into account some underground working 'rooms' which WA considered could affect the stability of the area. WA created their own section, showing additional rooms to the west of Pillar 12. WA also used a pillar width which it considered was worst case, i.e. through its narrowest point.

In discussions SRK disagreed with the pillar width WA had used for Pillar 12, stating it was too narrow. WA had stated it was worst case, across the pillar and not on the section line. In addition, although the actual surface settlement measurements, ~ 30mm over 20 years were higher than predicted by SRK (4mm) they were lower than that predicted by WA, (0.9m), in pillar failure mode, indicating the modelling did not properly reflect the actual movement or the unknown condition of the pillar.

The analysis of the condition of the pillar was therefore deemed urgent and, since access to the underground location was not possible, two holes were drilled from the surface, either side of the pillar, to enable a laser scan of the pillar to be carried out by Geoterra. The clear images showed Pillar 12 to be intact and the thickness of the roof gypsum was confirmed at 9m.

WA have therefore concluded that although the modelling has not been able to mirror exactly the movement at the surface so far (~30mm), it does show that any settlement will be occurring very gradually, i.e. no sudden collapse, and the fact that the pillar has been shown not to have moved or degraded, is above the current water level in Drumgoosat, and has not been submerged by the water being pumped, indicates its condition has not changed. It can therefore be deduced that it is stable at this point.

However, WA would recommend that since there has been movement ~30mm over 20 years and that the pillar is a 'slender pillar', based on height to width ratio, monitoring should be continued and that the integrity of the pillar should be monitored on a regular basis.

Figures 1:11 to 1:13 show the section through Pillar 12 and the FLAC analysis of that section.

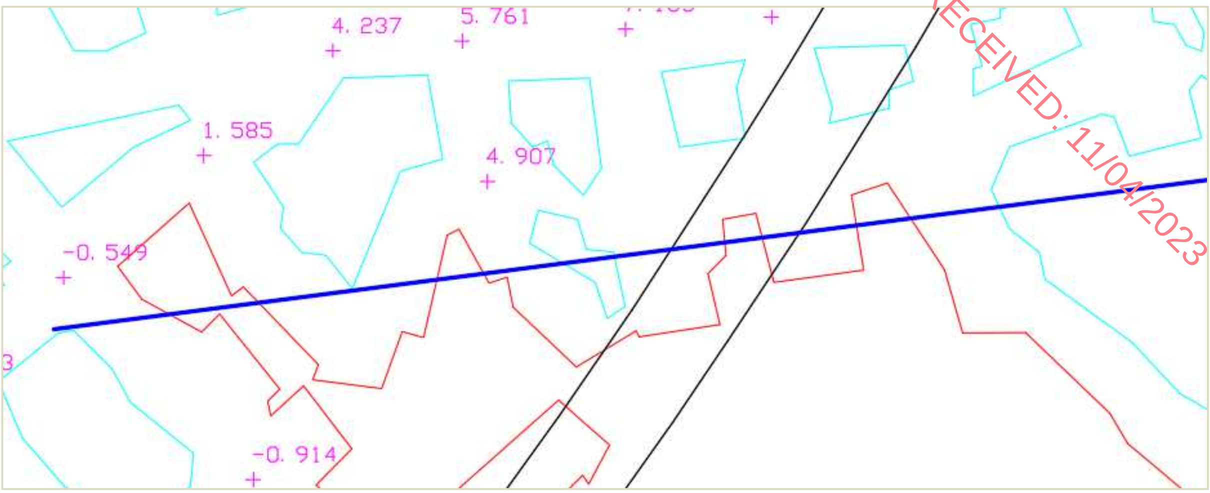

Figure 1:11 Cross Section Line across Pillar 12 and Road R179

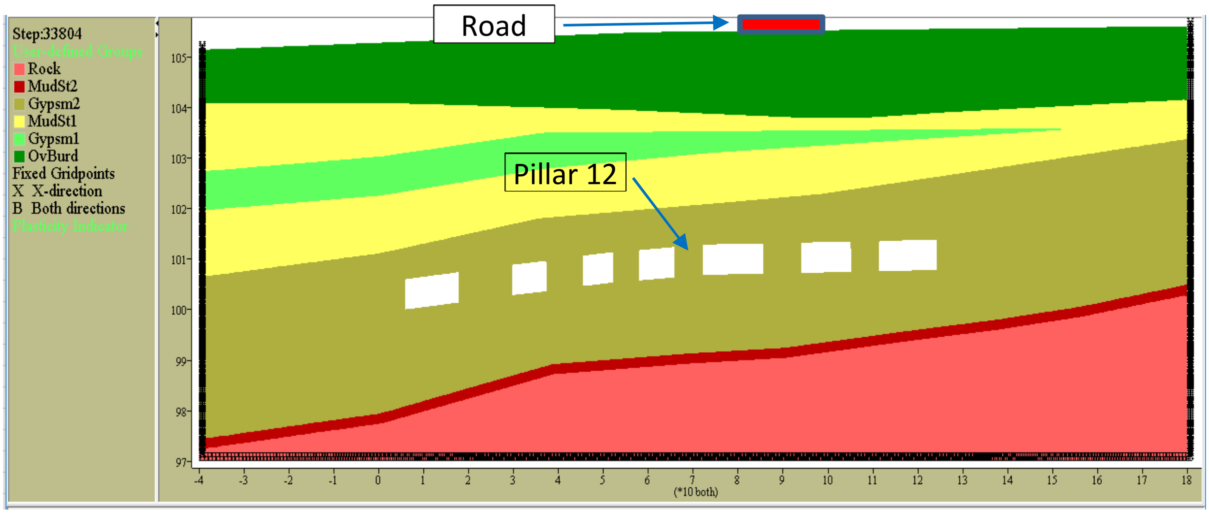

Figure 1:12 FLAC model layout of R179 Pillar 12 Section

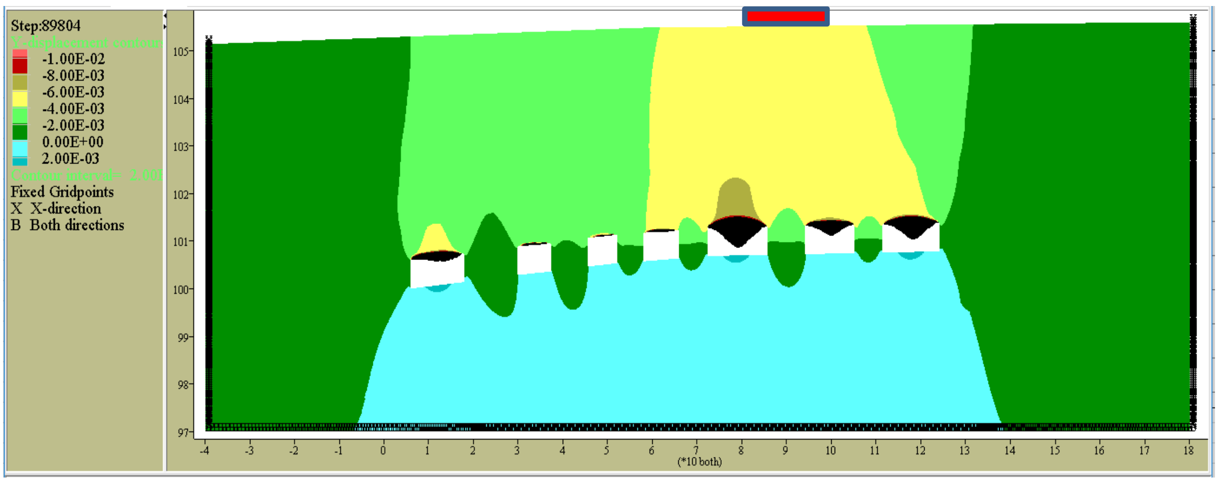

Figure 1:13 Ground Settlement Contours, R179 Pillar 12 Section

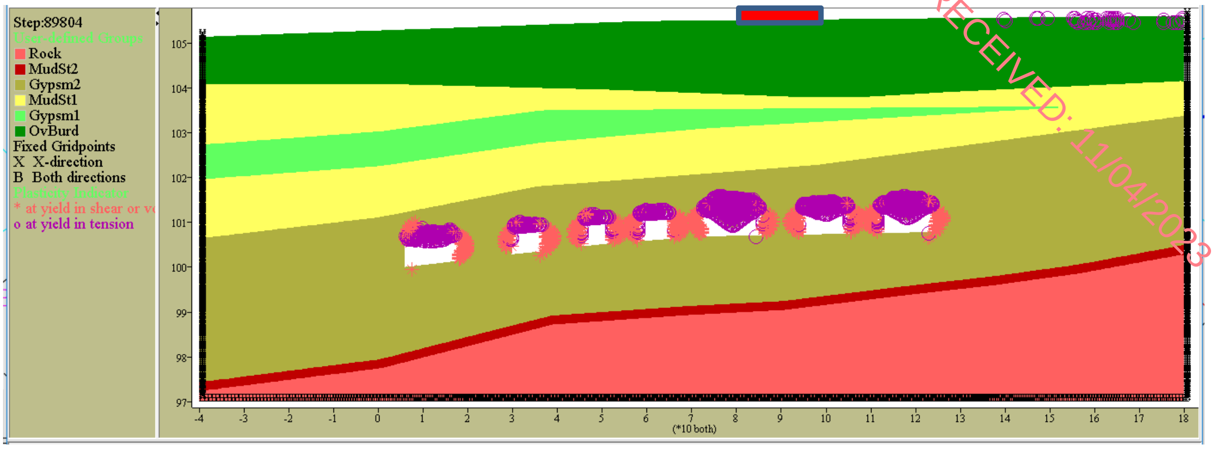

Figure 1:14 Localised Ground Failure Zones, R179 Pillar 12 Section

Figure 1.14 shows the modelling which indicates that at worst some localised roof failure may occur within a competent gypsum beam. However, this failure will not propagate towards the surface.

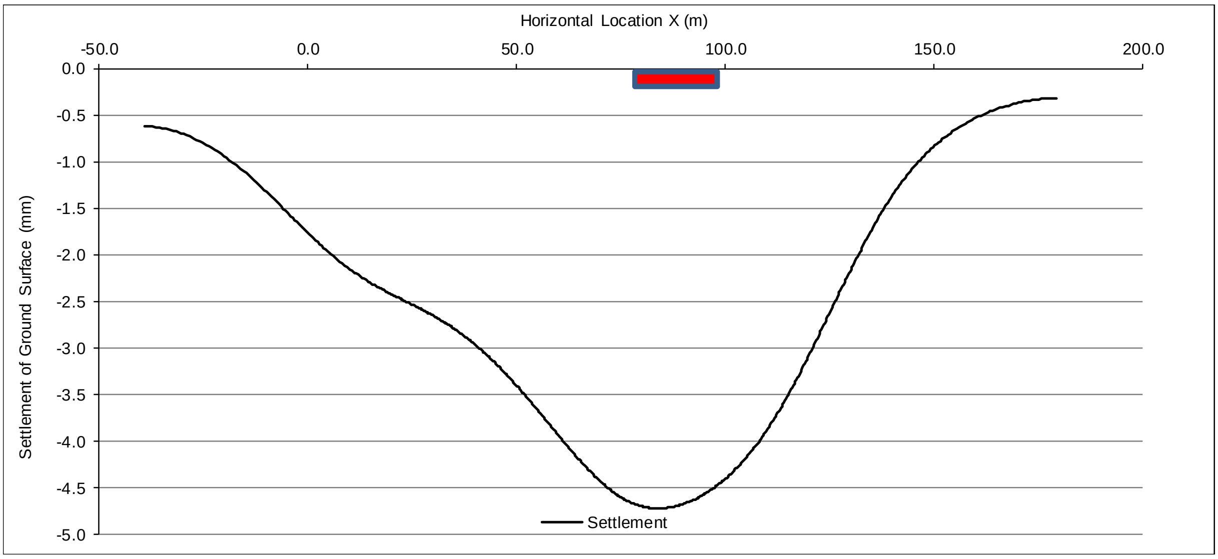

Figure 1:15 Ground Surface Settlement Under R179 at Pillar 12

The road surface subsidence is less than 5mm as indicated by Figure 1:15.

As stated the priority of modelling was given to those areas deemed most at risk of movement, either through the change in circumstance created by the water inflow or the location of a very slender pillar, Pillar 12.

Once this was completed WA also reviewed the modelling work carried out at Locations 2 and 4 across R179. Figures 1:16 to 1:18 show the FLAC analysis of Location 2.

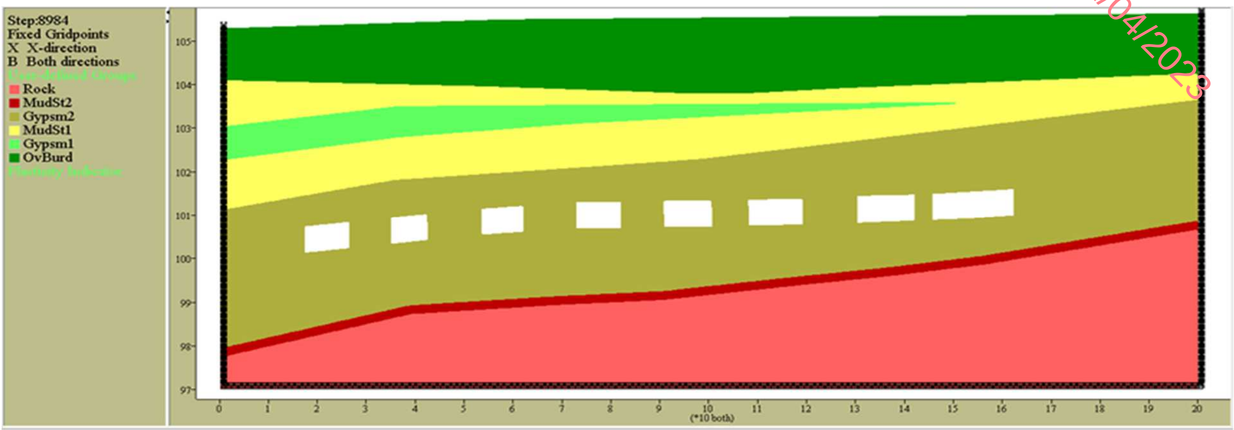

Figure 1:16 R179 Location 2 Cross Section through Pillar R21 – Strata Layout before deformation

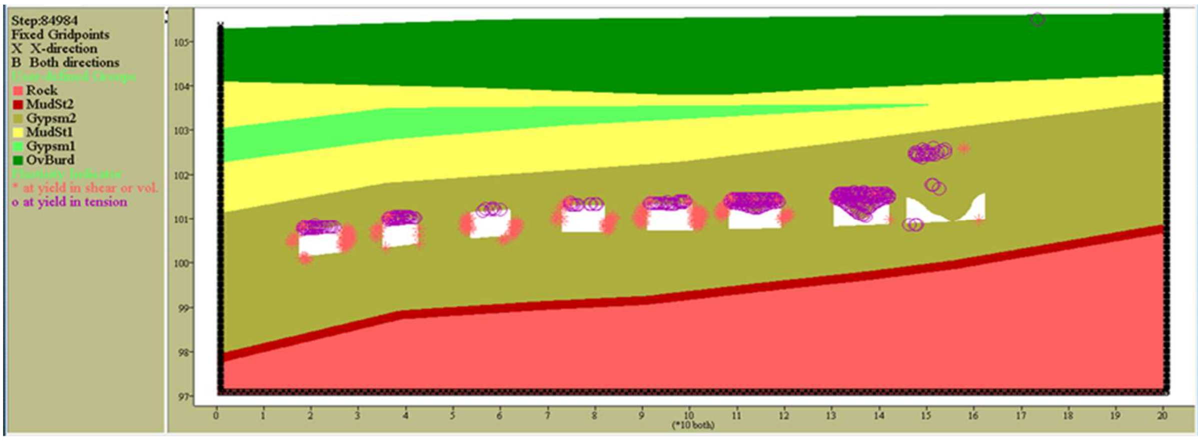

Figure 1:17 R179 Location 2 Cross Section through Pillar R21 – Strata Layout after deformation

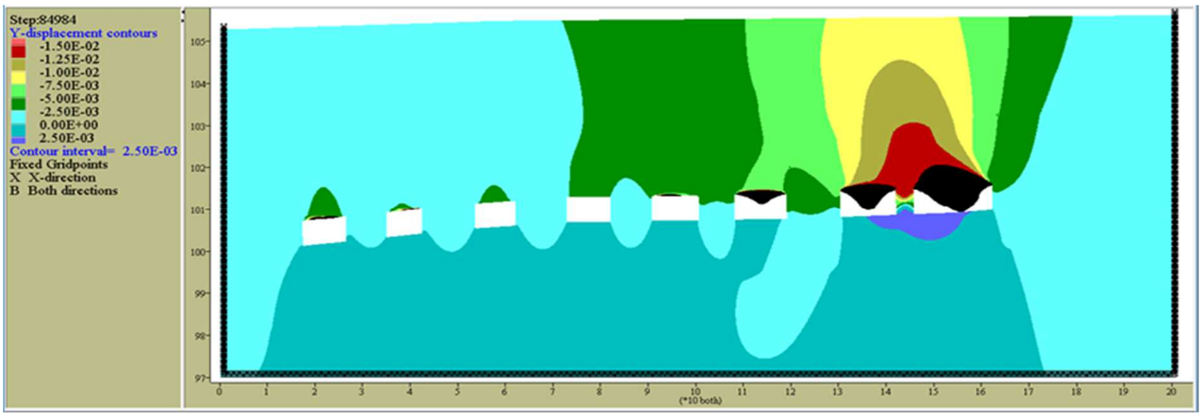

Figure 1:18 R179 Location 2 Cross Section through Pillar R21 – Ground Settlement Contours

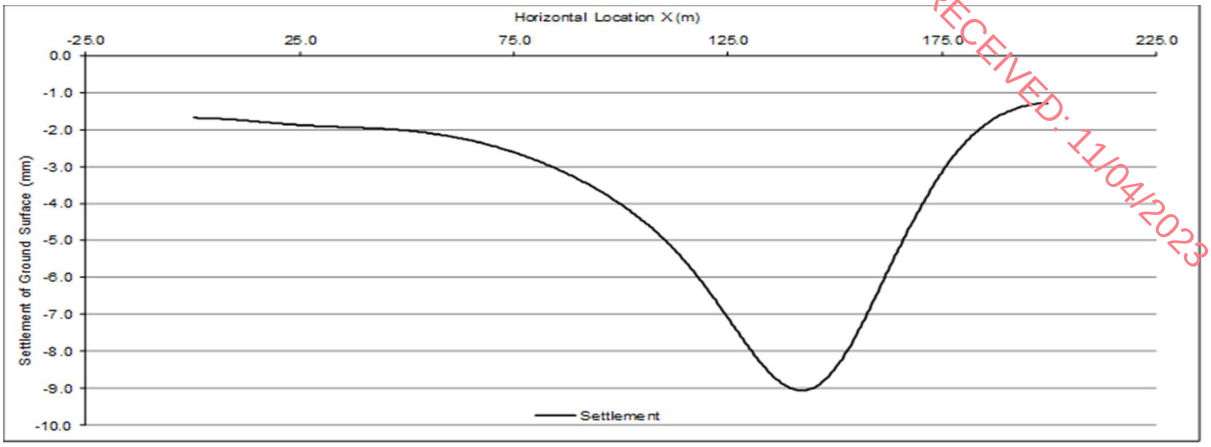

Figure 1:19 R179 Location 2 Cross Section Through Pillar R21 – Ground Settlement

Figure 1:19 indicates the maximum settlement under the road is 3mm (R179 at 75m horizontal location mark) and therefore considered safe.

Figures 1:20 to 1:22 shows the FLAC analysis of Location 4.

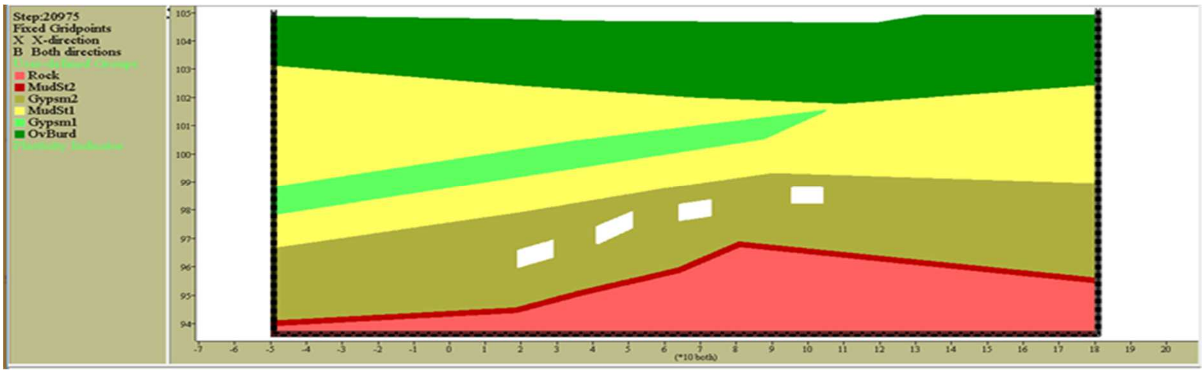

Figure 1:20 R179 Location 4 Cross Section – Strata Layout before deformation

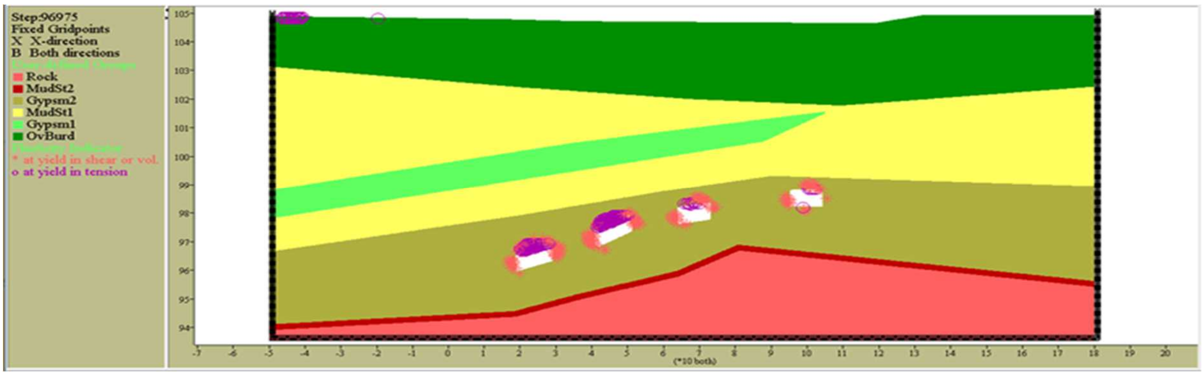

Figure 1:21 R179 Location 4 Cross Section – Strata Layout after deformation

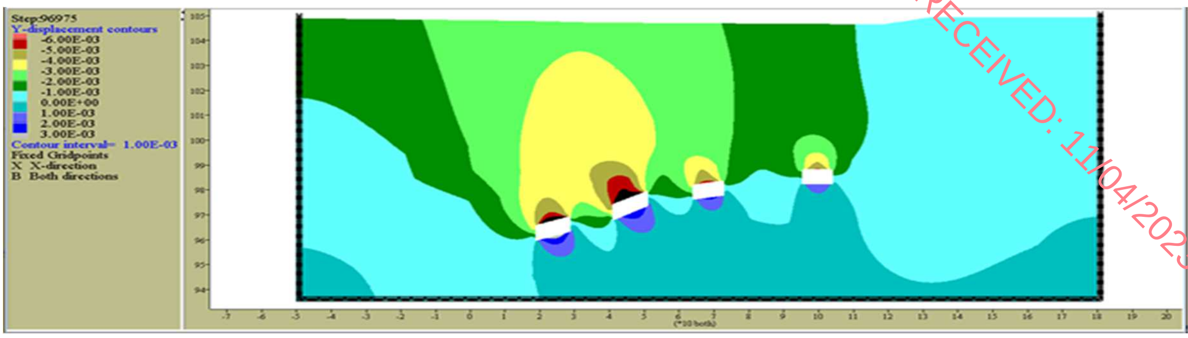

Figure 1:22 R179 Location 4 Cross Section – Ground Settlement Contours

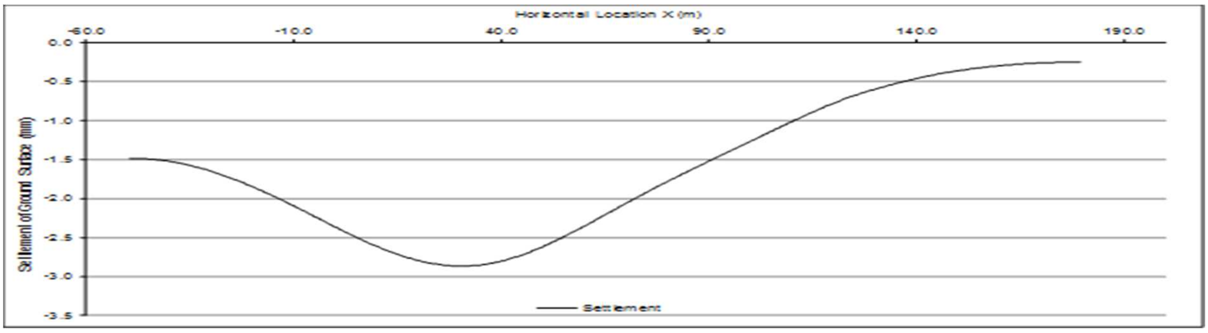

Figure 1:23 R179 Location 4 Cross Section– Ground Settlement

Figure 1:23 indicates the maximum settlement under the road is 1mm (R179 at 110m horizontal location mark) and therefore considered safe.

WA therefore confirmed that the R179 was safe to reopen on 26 October 2018.

1.6 L4900 Analysis

WA has also carried out another independent review of the works carried out by SRK related to the L4900. In a similar way to the other works, sections were provided by SRK and checked against mining plans to confirm their suitability.

The sections produced by SRK cover the West, Central and Eastern sections of the L4900 and are specifically targeted at the road and adjacent properties where stability concerns were raised. WA were satisfied that the sections were representative of the areas where the road has been undermined.

However, the FLAC analysis has found that currently the pillars remain stable with no major failure at any point. The modelling showed that the roof of the excavations may have localised roof failures underground, which is normal, but this will be limited to the immediate roof strata and will have minimal impact on the ground surface as there is an adequate thickness of gypsum in the roof.

The future ground settlement under the L4900 is predicted to be less than 3mm in all three West, Central and East sections.

WA therefore concluded that based on an independent analysis there was an extremely low risk of any major ground settlement, and that therefore the L4900 was considered safe for traffic. The road was reopened to the public on the 1 November 2018.

Figures 1:24 to1:26 show the FLAC analysis of the western section of the L4900.

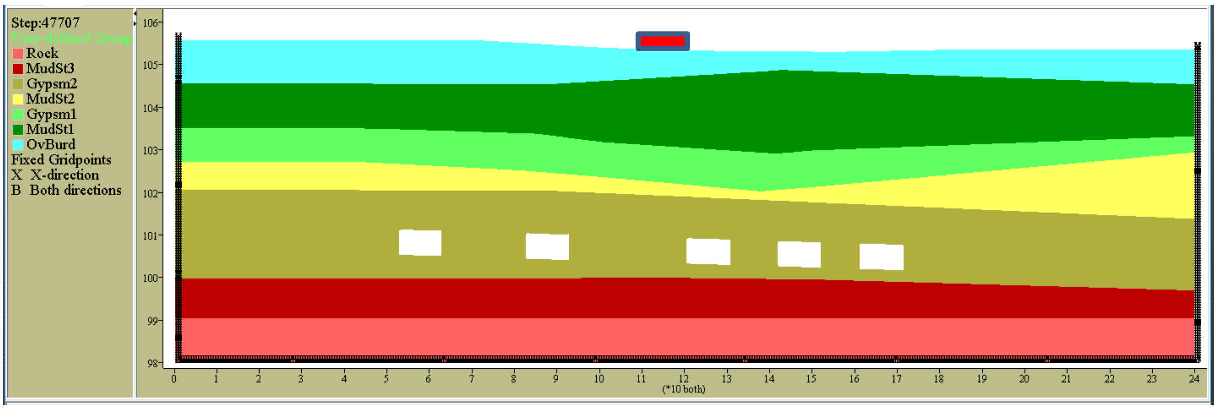

Figure 1:24 FLAC Model layout of Road L4900 West Section

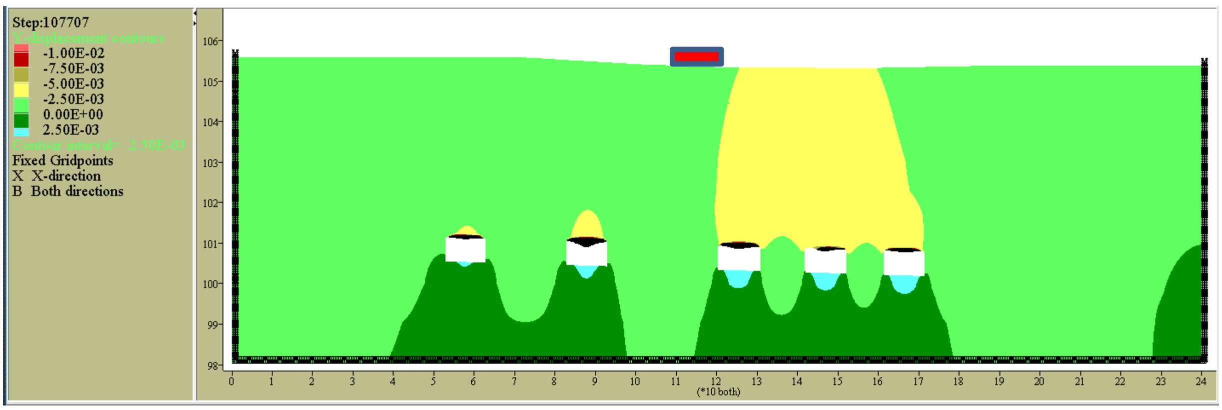

Figure 1:25 Ground Settlement Contours, Road L4900 West Section

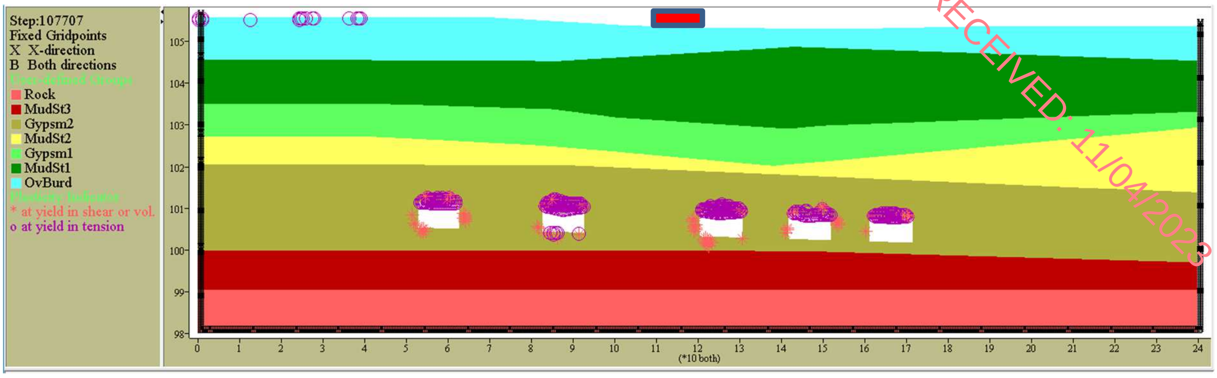

Figure 1:26 Localised Ground Failure, Road L4900 West Section

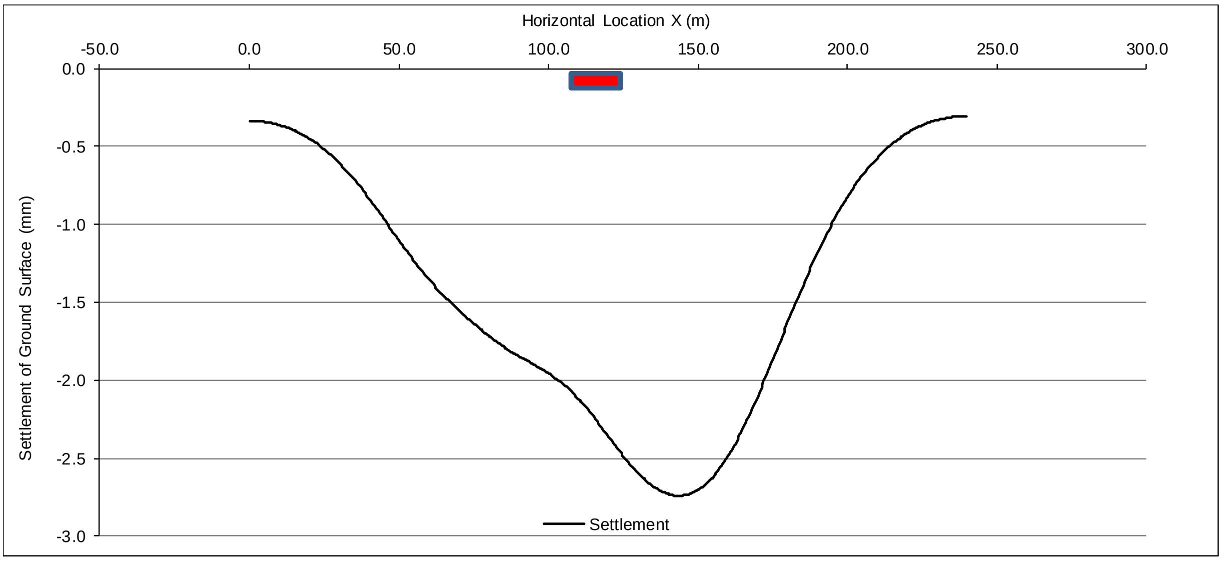

Figure 1:27 Ground Surface Settlement, Road L4900 West Section

Figure 1:27 shows a ground settlement of 2mm under the L4900.

Figures 1:28 to 1:30 shows the FLAC analysis of the Middle section of the L4900.

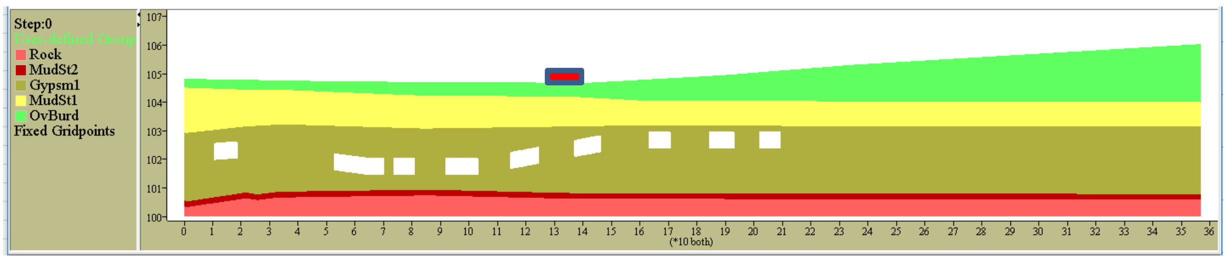

Figure 1:28 FLAC Model Layout of Road L4900 Middle Section, 6m high pillars

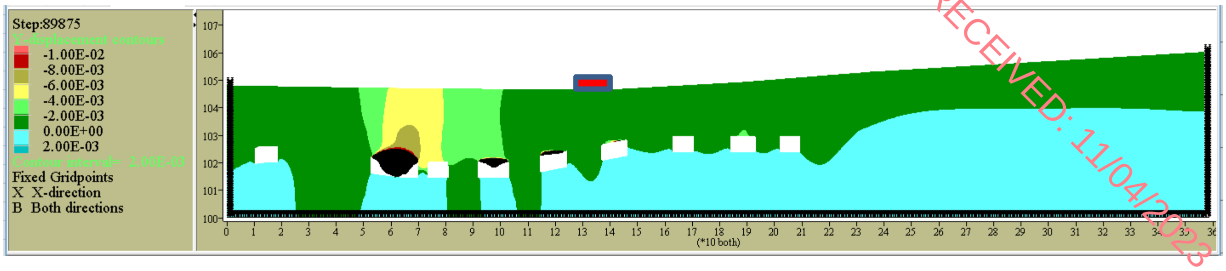

Figure 1:29 Ground Settlement Contours, Road L4900 Middle Section, 6m high pillars

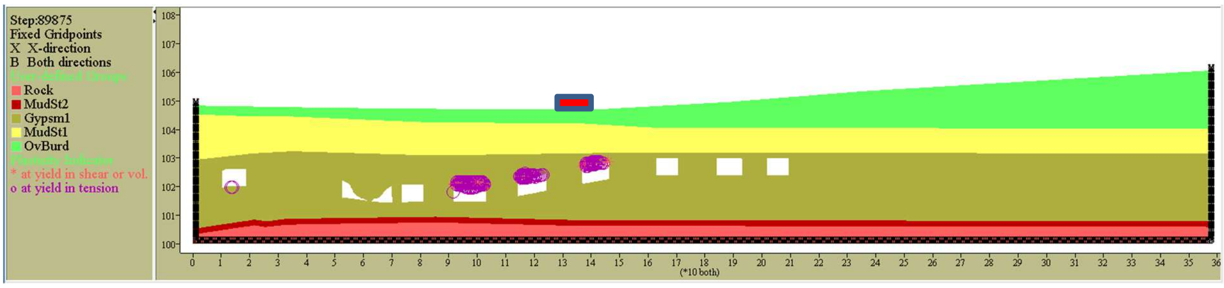

Figure 1:30 Localised Ground Failure, Road L4900 Middle Section, 6m high pillars

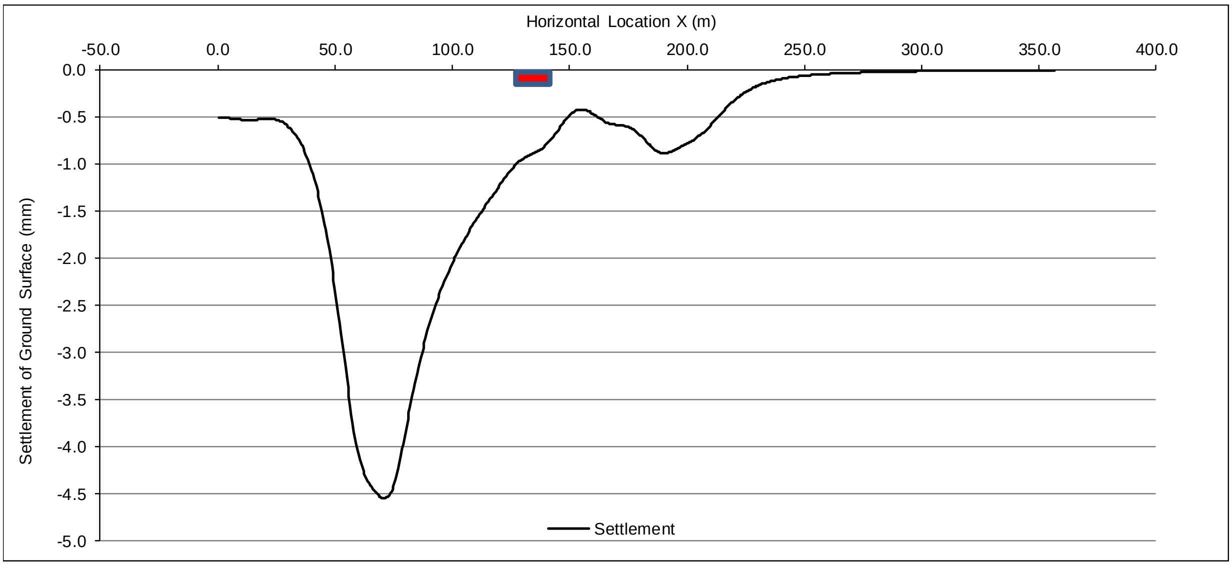

Figure 1:31 Ground Surface Settlement, Road L4900 central Section, 6m high pillars

Figure 1:31 shows the maximum settlement under the road as being less than 1mm.

SRK modelled the central section having 12m pillars to the north of the L4900. WA could not find evidence of either 12m pillars or the working of both seams in that area and did not therefore consider it relevant. However, for completeness, WA modelled the area using 6m and 12m high pillars and the model indicates stable conditions for both.

Figures 1:36 to 1:38 show the FLAC analysis for the eastern section of the L4900.

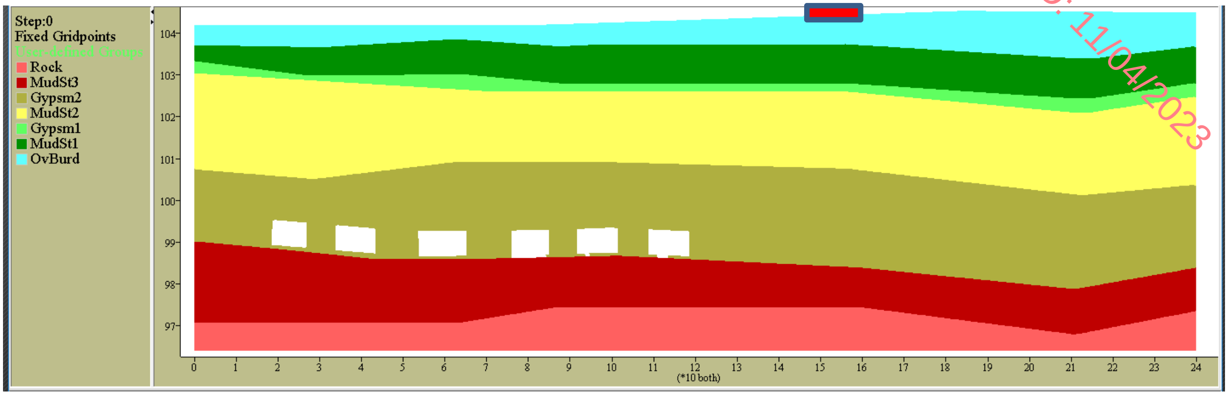

Figure 1:32 FLAC Model Layout of Road L4900 East Section

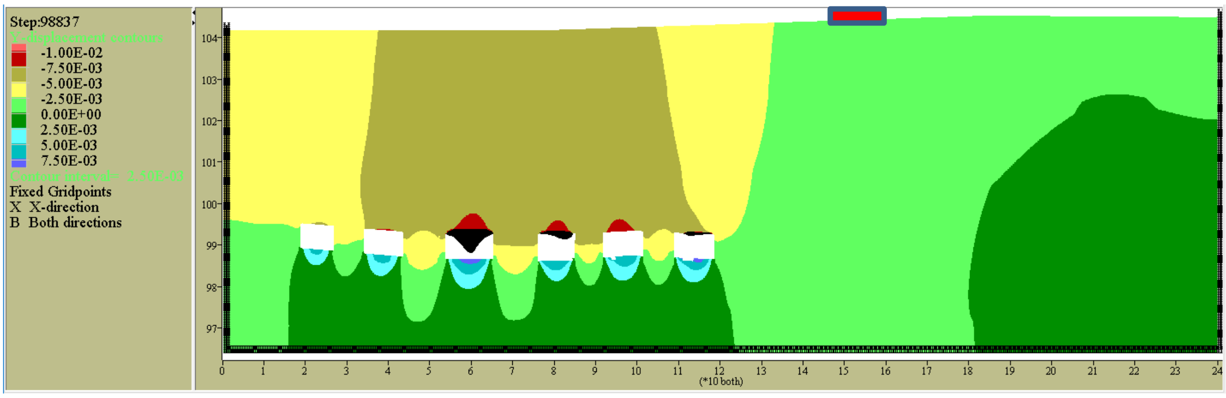

Figure 1:33 Ground Settlement Contours, Road L4900 East Section

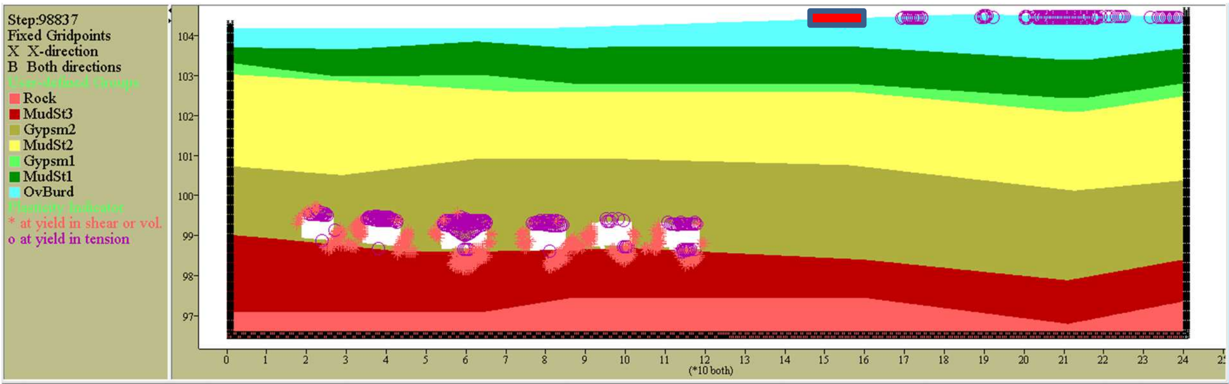

Figure 1:34 Localised Ground Failure, Road L4900 East Section

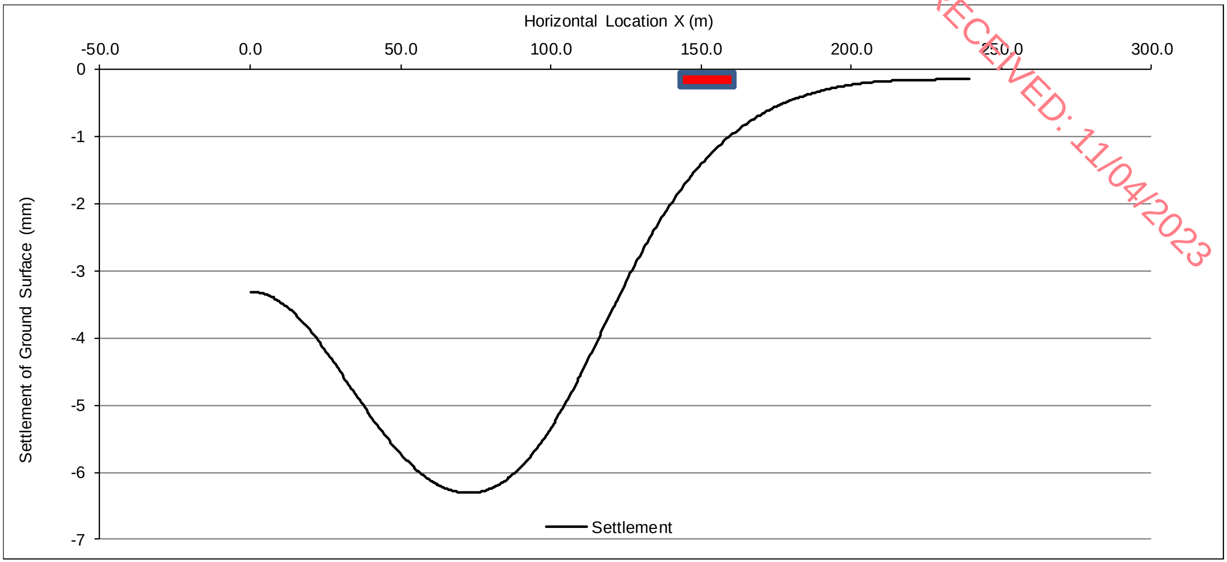

Figure 1:35 Ground Surface Settlement, Road L4900 East Section

Figure 1:39 shows the maximum settlement under the road of 1mm.

For all three sections, west, middle and east sections, the gypsum and basal mudstone have been assumed to have a GSI value of 40 and material properties have been taken from the SRK report. As previously stated the GSI value of 40 is considered to be conservative since the average gypsum GSI is about 53 at road L4900 area.

The maximum predicted road surface settlement at all three sections is less than 2.5mm and thus indicate a stable condition.

The modelling only indicates potential localised ground failure zones occurring in the immediate roof gypsum of the excavations with no signs of failure propagating to surface or under road L4900.

It is concluded that ground settlement along the road L4900 is less than 2.5mm with no risk of major road subsidence. The ground below the road appears to be in a stable condition with no sign of failure.

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APPENDIX 7.6
December 2018 Crown Hole - SRK - Apr 2019

DECEMBER 2018 CROWNHOLE

Prepared For

Gyproc Ltd

Report Prepared by

srk consulting

SRK Consulting (UK) Limited

30238

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Date:		April 2019
Project Number:		30238
SRK Project Director:	Richard Oldcorn	Managing Director
SRK Project Manager:	Neil Marshall	Corporate Consultant - Geotechnical
Client Legal Entity:		Saint-Gobain Mining (Ireland) Ltd
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EXECUTIVE SUMMARY
DECEMBER 2018 CROWNHOLE

1 INTRODUCTION

During a drone survey in early December 2018 a crownhole was identified. This was located on Gyproc land in a field located 35m to the south west of the L4900 road and about 340m north west of the L4900/R179 road junction.

In consultation with Monaghan County Council (MonCC) and the Exploration and Mining Division of the Department of Communications, Climate Action and Environment (EMD) the L4900 was closed and a detailed intrusive site investigation programme was developed and commissioned by Gyproc to investigate the likely cause and mechanism of the crownhole development as well as investigating ground conditions and the stability of all of the undermined areas below the L4900 road.

SRK Consulting (UK) Ltd (SRK) carried out the technical assessment of the September 2018 subsidence event that took place below the playing fields next to the Magherecloone Community Centre along with predictive stability analyses along R179 and L4900 roads. SRK has been commissioned by Gyproc to use the site investigation data collected to undertake analyses to:

1. Establish the likely cause of the crownhole development;
2. Carry out assessments to determine the stability and potential for failure of the rooms underlying the L4900;
3. Make recommendations to provide early warning of, manage or mitigate future subsidence risk along the line of the L4900 road.

2 WORK UNDERTAKEN

For the purpose of the site investigation and subsequent analysis work the road was divided into four zones.

- Zone A1 contains the location of the crownhole.
- Zone A2 is located in the area of historical subsidence along the L4900.
- Zone B3 is located NW of A1 close to the turnoff to the Maxwell and Kiernan properties.
- Zone B4 is located at the NW end of the L4900 road.

A total of 25 holes were drilled of which 23 are located within the four zones identified above. The selection of borehole locations was decided jointly by EMD, MonCC, EPA and Gyproc as the investigation programme progressed.

Most of the boreholes intersected underground workings. The UK company Geoterra was commissioned to carry out 3D laser scanning surveys (also known as Cloudscan surveys) down the holes that intersected the workings. This allowed a 3D image of the underground mine to be constructed which provided a very accurate picture of the stability condition of the underground workings. This study has made extensive use of these surveys to draw conclusions on the current condition of the underground workings.

The borehole core was geologically logged by British Gypsum geologists and geotechnically logged by an SRK geotechnical engineer. The intact strength of the core was estimated using a portable point load testing machine. A selection of borehole core was sent to a UK testing lab to allow calibration of the point load testing indices to intact rock strength measured in Mega pascals. This allowed SRK to develop a good understanding of the variability of the rock forming the gypsum roof beams.

Rock mass classification of the core was carried out. This is a measure of the fracture condition of the borehole core and allows the geological strength index (GSI) to be calculated. The GSI is an industry standard means of assessing the quality and strength of a rock mass made up of intact rock separated by natural fractures.

The intact strength and GSI of the gypsum and overlying dolerite was input to a 2-dimensional finite element package (RS2 produced by RocScience Inc of Canada) and eight cross sections across the L4900 road were analysed for underground room stability and surface deformation. The position of the eight cross sections analysed was agreed in conjunction with Gyproc, EMD, MonCC, EPA and EMD's consultants Wardell Armstrong prior to the start of the investigation.

The cloudscan surveys were processed and the mine openings that were surveyed and were inspected for evidence of roof instability.

The integration of all the data collected and the results of the analyses allowed SRK to formulate conclusions on the likely cause of the December 2018 crownhole, the stability condition of the mine below the L4900 road and the risk of possible future mine instability affecting the L4900 road.

3 CONCLUSIONS

3.1 General Condition of the Underground Workings

The cloudscan data revealed that even over 40 years after mining the condition of the rooms and pillars below the L4900 are very good. Outside of the area of historical subsidence and the immediate position of the crownhole the there is no evidence of any instability.

The strength and GSI values of the gypsum roof beams measured from the borehole core are consistent with the values generated by SRK during its underground mapping campaigns carried out between 1999 and 2005 and with the 'expected' strength conditions used in its predictive analyses of mine stability below the R179 and L4900 reported in October 2018. This validates the results of the previous predictive analyses and also indicates that there has been no degradation of the gypsum strength over the last 20 years.

3.2 Crownhole Development

The borehole drilling and laser scan surveys have indicated that the crownhole was formed as a result of localised thinning and breach of the gypsum roof beam above a section of the Lower Seam upper horizon workings. It is likely that the extent of the breach is very localised as the laser scan surveys show very good stable underground mine conditions immediately next to the area of the mine inflow.

The sandy weathered dolerite flowed into the mine workings and undercut the overlying glacial till. The glacial till is a cohesive material so collapse of this into the underlying void in the dolerite would likely have been slow. It is likely that the concrete slab observed near surface, which formed a roadway or laydown area for crushed gypsum stockpiles during the time that Drumgoosat was active, might have supported the ground surface overlying the crownhole until the crownhole void exceeded a critical width. The concrete slab then failed resulting in the appearance of the crownhole at surface. If this was the case, then the crownhole had probably developed over several months before it manifested itself on surface.

Because of the above SRK is of the opinion that the crownhole development is unrelated to the September 2018 subsidence event because the location of the event at about 380m distant from the crownhole and is too far away to have had any influence on the mine in the vicinity of the crownhole

The requirements for crownhole development can be summarised as follows:

1. Shallow mining depth, less than 50m
2. Presence of dolerite and glacial till which extend to surface immediately above the underground workings. For the avoidance of doubt any material with similar characteristics to dolerite and glacial till should be considered to behave in the same manner.
3. A very thin gypsum roof beam, typically less than 1 m.

If at least one of these factors is not present the likelihood of a crownhole developing above the mine is almost impossible.

3.3 Possibility of Future Crownhole Development Affecting the L4900 Road

Because of the conditions of the underground workings and the thickness of gypsum roof beam identified in the boreholes drilled for this investigation work along with the history of development of crownholes above Drumgoosat SRK considers that the risk of future crownhole development in the vicinity of the L4900 road is very low. However, because of the localised nature of conditions that lead to crownhole development there is still some risk that crownholes could occur above the workings that undermine the L4900 road. There are however ways to minimise the impact of the risk of crownhole development and these are described in the next Section.

3.4 L4900 Zone A1

The roof beam thickness below the road in this area, as determined from the borehole drilling, ranges from a minimum of 5 m to a maximum of 8.5m. The areas modelled and inspected from the laser surveys indicates stable mine conditions. Modelled surface deformations are consistent with those being measured on surface by Gyproc's on-going surface survey measuring programme. The mine workings are 30m below surface and the lower gypsum is overlain by a thick dolerite unit.

This zone contains two of the three criteria for crownhole development, depth of mining and overlying dolerite and glacial till/drift, therefore the potential for any future crownhole development is considered to be very low.

3.5 L4900 Zone A2

This area is located in the area of historical subsidence. The road is only undermined by single rooms in three places. The areas modelled indicates stable future mine conditions with a very low risk of mine instability. Modelled surface deformations are consistent with those being measured on surface by Gyproc's on-going surface survey measuring programme.

Two of the rooms have been surveyed, one by laser the other by sonar. The scans show the impact that the deformation of these rooms, floor heave and pillar damage, has on the historical subsidence on surface. Because of the floor heave the height of the workings has been reduced from 6m to about 3m. It is of interest to note that although these rooms are in an area of subsidence the roof of the workings is in good condition. This confirms that the interpretation of the subsidence was by punching of pillars through the floor into the underlying basal mudstone.

In this area the boreholes have intersected a gypsum roof beam with a thickness of between 12m and 13m. The depth of mining is between 55m and 60m below surface.

This zone contains only one of the three criteria for crownhole development, overlying dolerite and glacial till/drift, therefore the potential for any future crownhole development is considered to be extremely low.

3.6 L4900 Zone B3

The road in this area is undermined by a number of four-way intersections in the underground mine. The cross sections modelled show stable mine conditions. Modelled surface deformations are consistent with those being measured on surface by Gyproc's on-going surface survey measuring programme.

The laser scans show stable mine conditions in the rooms modelled and in the intersections that were laser scanned.

The underground workings are 30-35m below surface, the thickness of the gypsum roof beam is between 8 m and 10m and the lower gypsum is overlain by a thick dolerite unit.

This zone contains two of the three criteria for crownhole development, depth of mining and overlying dolerite and glacial till/drift, therefore the potential for any future crownhole development is considered to be very low.

3.7 L4900 Zone B4

This zone is located towards the western end of the L4900 road. There are a number of four-way intersections below the road. The cross sections modelled show stable mine conditions. Modelled surface deformations are consistent with those being measured on surface by Gyproc's on-going surface survey measuring programme.

The laser scans show stable mine conditions in the rooms modelled and in the two four way intersections that were laser scanned.

The depth of mining increase from 30m on the eastern end of the zone to 50m on the western end. The gypsum roof beam varies from 3.5 m to 13 m thick. The geology in this zone is different from the other three zones in that there is mudstone lying directly above the lower gypsum. Above this there is unmined Upper Gypsum seam. This adds a layer of protection above the mine preventing any possible mine instability reaching surface.

This zone contains only one of the three criteria for crownhole development, depth of mining. Furthermore because of the presence of the strong Upper Gypsum above the potential for any future crownhole development in this zone is considered to be exceptionally low.

3.8 Conclusion on Stability of L4900 Road

Based on the investigations carried out, the geotechnical analysis and interpretation of the cloudscan laser surveys no specific areas of concern have been noted in the areas of the mine that extend below the road.

The occurrence of the December 2018 crownhole has not increased the risk of future crownhole development or subsidence along the L4900 which continues to be very low. This is in line with the findings of the previous predictive analyses conducted by SRK in October 2018 which concluded that the L4900 is safe to use.

The laser surveys and the geotechnical borehole logging have provided strong evidence that, outside of the area of the crownhole and historical subsidence, there has been virtually no deterioration in the mine conditions since the excavations were created. This provides confidence that the roof beams and pillars are still doing the job for which they were designed, which is to support the underground openings and prevent surface subsidence.

The on-going interpretation of the surface levelling programme, the extensometers along with periodic underground laser scans will provide assurance that the mine below the road remains in a stable condition.

4 RECOMMENDATIONS

Whilst SRK considers the possibility of future subsidence occurring below the L4900 road to be generally very low to extremely low, in terms of relative risk of a possible future subsidence or crownhole event the four zones can be ranked from highest relative risk to lowest relative risk as follows:

Zone A1 – Very low risk of future subsidence or crownhole development. Contains two of the three criteria for crownhole development. Contains the December 2018 crownhole.

Zone B3 – Very low risk of future subsidence or crownhole development. Contains two of the three criteria for crownhole development.

Zone A2 – Area of historical subsidence. Extremely low risk of future subsidence or crownhole development. Contains only one of the three criteria for crownhole development.

Zone B4 – Extremely low risk to unlikely possibility of future subsidence or crownhole development Contains only one of the three criteria for crownhole development. Also has unmined upper gypsum overlying the underground workings making the possibility of any crownholes developing on surface highly unlikely.

The recommendations for action are presented below on a zone by zone basis and are commensurate with the relative risk identified above.

4.1 Zone A1

1. Since water is a major contributor to the development of crownholes carry out a ground surface drainage survey to ensure that any surface water cannot pond above and seep into the mine workings.

2. Carry out an investigation to determine whether there are underground water or sewage service lines crossing the area. Ensure that they are not leaking and make good if they are.

3. Continue surface level monitoring through the zone. Review the position of the monitoring points on surface and add additional points to ensure there is at least one monitoring point located above every room and one above every pillar along the road.

4. Install extensometers into boreholes KC19H1, KC19H3, KC19H4 and KC19H5 to monitor roof beam deflection adjacent to the road. Roof beam movement of 2% of roof beam thickness should be used as a trigger for further investigation.

5. Use borehole KC19H17 for future laser scans initially on a two-year frequency. Scanning frequency should be reviewed after each scan.

6. Those boreholes that will be kept open for future monitoring should be lined appropriately for use and an appropriate sealable and lockable collar constructed and installed which will prevent any water entering into the mine and prevent the open boreholes becoming a future initiator of mine instability. All boreholes not being used for a specific purpose should be grouted and closed.

4.2 Zone B3

1. Since water is a major contributor to the development of crownholes carry out a ground surface drainage survey to ensure that any surface water cannot pond above and seep into the mine workings.

2. Carry out an investigation to determine whether there are underground water or sewage service lines crossing the area. Ensure that they are not leaking and make good if they are.

3. Continue surface level monitoring through the zone. Review the position of the monitoring points on surface and add additional points to ensure there is at least one monitoring point located above every room and above every pillar along the road.

4. Install extensometers into borehole KC19H11 and KC19H21 to monitor roof beam deflection in the four-way intersections. Roof beam movement of 2% of roof beam thickness should be used as a trigger for further investigation.

5. Those boreholes that will be kept open for future monitoring should be lined appropriately for use and an appropriate sealable and lockable collar constructed and installed which will prevent any water entering into the mine and prevent the open boreholes becoming a future initiator of mine instability. All boreholes not being used for a specific purpose should be grouted and closed.

4.3 Zone A2

1. Since water is a major contributor to the development of crownholes carry out a ground surface drainage survey to ensure that any surface water cannot pond above and seep into the mine workings.

2. Carry out an investigation to determine whether there are underground water or sewage service lines crossing the area. Ensure that they are not leaking and make good if they are.

3. Continue surface level monitoring through the zone.

4. Ensure that all boreholes not being used for a specific purpose should be grouted and closed.

4.4 Zone B4

1. Continue surface level monitoring through the zone.

2. Install extensometers into borehole KC19H10 and KC19H15 to monitor roof beam deflection in the four- way intersections. Roof beam movement of 2% of roof beam thickness should be used as a trigger for further investigation.

3. Those boreholes that will be kept open for future monitoring should be lined appropriately for use and an appropriate sealable and lockable collar constructed and installed which will prevent any water entering into the mine and prevent the open boreholes becoming a future initiator of mine instability. All boreholes not being used for a specific purpose should be grouted and closed.

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L4900 CROWNHOLE

GEOTECHNICAL REPORT

1 INTRODUCTION AND PROBLEM DEFINITION

In September 2018 a major subsidence event occurred above the abandoned Drumgoosat shallow underground room and pillar mine owned by Gyproc Ltd. Pending the results of a detailed investigation both the R179 and L4900 roads which skirt the edge of the mine were temporarily closed. That investigation was completed by the end of October 2018. The results concluded that there was a low to very low relative risk of future subsidence affecting the areas of the R179 and L4900 that were undermined by the Drumgoosat workings. Both roads were re-opened in early November 2018.

Monitoring frequency of the ground surface overlying the mine was increased. This included flying regular drone surveys over the mine to monitor changes to the extent of the original subsidence. During a drone survey in early December 2018 a crownhole was identified. This was located on Gyproc land in a field located 35m to the south west of the L4900 road and about 340m north west of the L4900/R179 road junction.

In consultation with Monaghan County Council (MonCC) and the Exploration and Mining Division of the Department of Communications, Climate Action and Environment (EMD) the L4900 was closed and a detailed intrusive site investigation programme was developed and commissioned by Gyproc to investigate the likely cause and mechanism of the crownhole development as well as investigating ground conditions and the stability of all of the undermined areas below the L4900 road.

SRK Consulting (UK) Ltd (SRK) carried out the technical assessment of the September 2018 subsidence event that took place below the playing fields next to the Magherecloone Community Centre along with predictive stability analyses along R179 and L4900 roads. SRK has now been commissioned by Gyproc to use the site investigation data collected to undertake analyses to:

1. Establish the likely cause of the crownhole development;
2. Carry out assessments to determine the stability and potential for failure of the rooms underlying the L4900;
3. Make recommendations to provide early warning of, manage or mitigate future subsidence risk along the line of the L4900 road.

This report presents the detail of the work carried out.

2 GEOTECHNICAL INVESTIGATION WORK

For the purpose of the site investigation and subsequent analysis work the road was divided into four zones.

- Zone A1 contains the location of the crownhole.
- Zone A2 is located in the area of historical subsidence along the L4900.
- Zone B3 is located NW of A1 close to the turnoff to the Maxwell and Kiernan properties.
- Zone B4 is located at the NW end of the L4900 road.

A total of 25 holes were drilled of which 23 are located within the four zones identified above. The selection of borehole locations was decided jointly by the EMD, MonCC, EPA and Gyproc as the investigation programme progressed. Borehole KC19 H26 was drilled subsequent to SRK's geotechnically logging site visits. Whilst reference to this borehole is made in the report there is no geotechnical log for it.

Most of the boreholes intersected underground workings. The UK company Geoterra was commissioned to carry out 3D laser scanning surveys (also known as Cloudscan surveys) down the holes that intersected the workings. This allowed a 3D image of the underground mine to be constructed which provided a very accurate picture of the stability condition of the underground workings. This study has made extensive use of these surveys to draw conclusions on the current condition of the underground workings.

Further details of the work carried out are presented in the following sections.

2.1 Borehole Drilling

Borehole drilling was completed using a double-tubed wireline system. The first 20m of each borehole was open holed. Coring was then carried out through the dolerite and gypsum. Upon extraction the core was laid directly into wooden core boxes for logging. Detailed geological logging and core photographing was carried out by British Gypsum geologists. The borehole logs and photographs are presented in Appendix A. Table 2-1 shows the details of the boreholes drilled and Figure 2-1 shows the locations of the boreholes in plan view relative to the L4900 road.

Table 2-1: Borehole collar and survey data

Borehole ID	Easting	Northing	Elevation	Depth (m)	Dip Angle (°)	Azimuth (°)
KC18Q	280837.25	300607.98	1049.08	31	90	n/a
KC19H01	280848.71	300599.99	1049.03	52.7	90	n/a
KC19H02	280832.61	300617.68	1049.09	44.5	90	n/a
KC19H03	280822.53	300617.57	1048.92	44.6	90	n/a
KC19H04	280800.02	300638.97	1048.62	41.6	90	n/a
KC19H05	280779.78	300652.78	1048.69	43.2	90	n/a
KC19H07	280853.33	300551.53	1045.82	47.5	60.269	306.132
KC19H08	280804.28	300548.19	1045.64	38.4	44.672	7.176
KC19H09	280818.42	300639.62	1049.63	41	90	n/a
KC19 H10	280489.91	300888.19	1056.5	53.5	90	n/a
KC19 H11	280685.93	300724.09	1048.5	43.2	90	n/a
KC19 H12	280883.03	300576.75	1048.04	55.1	90	n/a
KC19 H13	280954.01	300532.63	1044.08	59.6	90	n/a
KC19 H14	280475.33	300893.36	1055.92	56.5	90	n/a
KC19 H15	280508.21	300880.26	1056.57	47.6	90	n/a
KC19 H16	280434.92	300905.75	1053.65	59.5	90	n/a
KC19 H17	280819.02	300599.83	1048.32	50.5	90	n/a
KC19 H18	280911.94	300558.17	1046.42	61.8	90	n/a
KC19 H20	280524.67	300867.5	1056.43	44.7	90	n/a
KC19 H21	280750.94	300672.47	1048.76	37	60.414	310.881
KC19 H22	280563.27	300821.71	1053.39	35.5	90	n/a
KC19 H23	280935.3	300165.34	1054.27	58.1	90	n/a
KC19 H26	280850.95	300558.53	1047.61	48.5	47.1	304.49

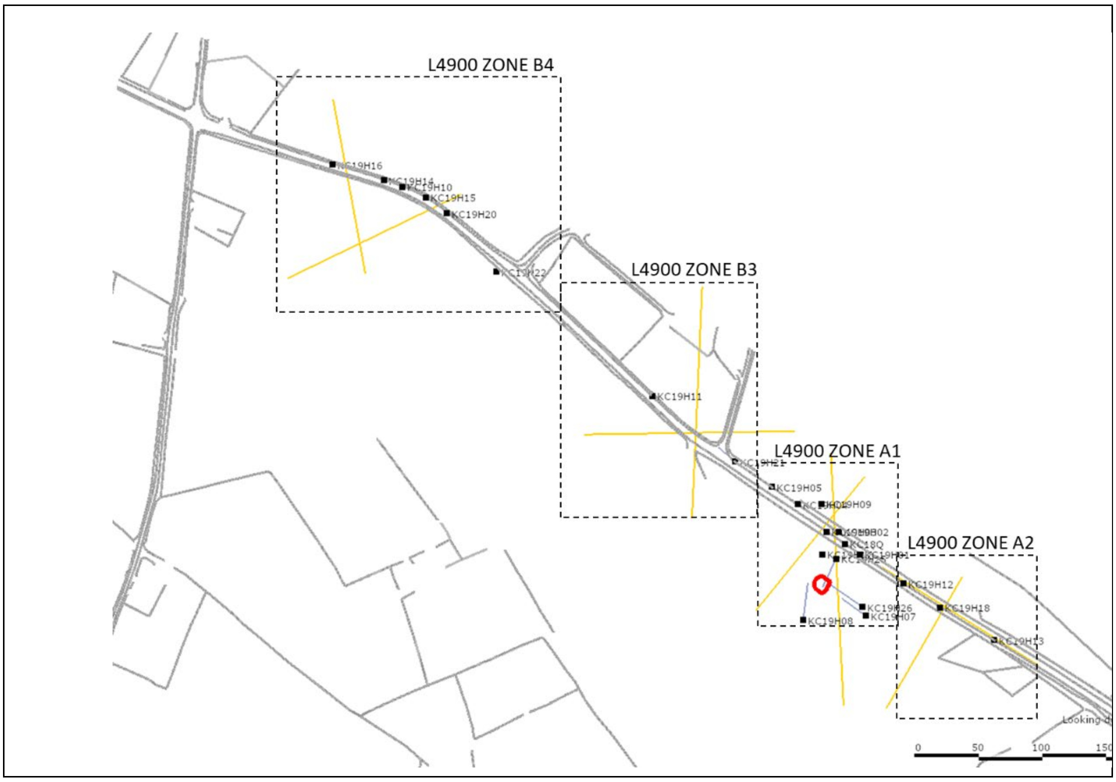

Figure 2-1: Locations of boreholes relative to the L4900 road and the crownhole.

2.2 Geotechnical Logging

The Rock Mass Rating system defined by Bieniawski (1989) (RMR^{89}), was used to characterise the rock materials.

The system assigns ratings to six parameters that are used to classify the rock mass. These parameters are the following:

1. Rock Quality Designation (RQD) rating
2. Intact Rock Strength (IRS) rating
3. Spacing rating
4. Joint Condition rating
5. Groundwater Conditions rating
6. Orientation of discontinuities

These parameters each contribute to the overall RMR value in the follow way:

Parameter	Rating
IRS	0-15
RQD	0-20
Spacing rating	0-20
Joint Condition rating	0-30
Groundwater Condition rating	0-15

The final parameter in the determination of RMR^{89}, Orientation of discontinuities (6), is an adjustment factor that is applied based on the orientation of the identified discontinuities relative to the slope, ranging from 0 for Very favourable, to -50 for Unfavourable.

Although the core was not orientated, the dip of the joints in the majority of boreholes was found to be perpendicular with the core axis, indicating near horizontal bedding planes (for vertical boreholes). This is confirmed by the cloudscan data where the joint sets are visible in-situ. As such, no additional adjustment was made to the RMR^{89} for the orientation of discontinuities. Furthermore, the Groundwater Conditions rating was set at 15 for all intervals as no groundwater data was available.

The resultant rock mass rating is given as a value between 0 and 100, with intervals within this range being assigned a 'class', as follows:

RMR value	Class
0-20	Very poor
21-40	Poor
41-60	Fair
61-80	Good
81-100	Very good

For modelling purposes, the weighted RMR^{89} is calculated and converted to GSI using the formula:

$$GSI = RMR'_{89} - 5\ (for\ RMR'_{89} > 23)$$

The results of the geotechnical logging and RMR^{89} system classification of the boreholes are presented in Appendix B. The GSI values used for modelling are shown in Table 4-1. The derivation of the gypsum and dolerite rock mass strength is presented in data sheets in Appendix C.

2.3 Rock Strength Testing

Intact rock strength (IRS) is an important input to determining rock mass strength for modelling. Most of the rock strength testing whilst core logging was carried out using a point load index testing machine (PLT). In order to convert PLT strength into IRS it is normal to carry out PLT and laboratory IRS strength tests on contiguous pieces of borehole core and then develop a factor that converts PLT into IRS. A selection of intact gypsum core was selected and shipped to a laboratory in the UK, KIWA CMT in Derby, for this purpose. Table 2-2 gives details of the laboratory testing that was performed on selected samples from the borehole core.

Table 2-2: Laboratory testing programme.

BHID	Sample ID	From (m)	To (m)	Length (m)	Lithology	Core Diameter	Test	Result (MPa)	Date sampled
KC19HC02	24452	26.49	27.16	0.67	Gypsum	NQ	UCS	30.2	14/02/2019
KC19HC03	24453	27.62	27.99	0.37	Gypsum	NQ	UCS	22.9	14/02/2019
KC19HC05	24454	22.26	22.64	0.38	Gypsum	NQ	UCS	22.5	14/02/2019
KC19HC01	24455	42.74	43.16	0.42	Gypsum	NQ	UCS	26.1	14/02/2019
KC19HC01	24456	38	38.6	0.6	Gypsum	NQ	UCS	18.9	14/02/2019
KC19HC04	24457	22.67	22.9	0.23	Gypsum	NQ	UCS	23.7	14/02/2019
KC19HC04	24458	29.55	29.84	0.29	Gypsum	NQ	UCS	26.4	14/02/2019
KC19HC10	24459	30.04	30.32	0.28	Gypsum	NQ	UCS	26.3	14/02/2019
KC19HC14	24460	37	37.35	0.35	Gypsum	HQ3	UCS	18.1	14/02/2019
KC19HC14	24461	38.24	38.52	0.28	Gypsum	HQ3	UCS	34.9	14/02/2019
KC19HC13	24465	48.32	48.82	0.5	Gypsum	NQ	UCS	22.1	14/02/2019

The laboratory data on UCS tests of the rock units was used along with the Point Load Test (PLT) data to inform the IRS rating assigned to each run of the rock units. PLT tests performed adjacent to UCS tests were used to calibrate appropriate Is50 conversion factors for the PLT results; the conversion factor used was 17. The data for this are shown in Table 2-3 and the linear regression line generated using this data is shown in Figure 2-2.

Table 2-3: UCS and PLT data used to determine the Is50 conversion factor

BHID	Core Depth	Core Depth	LITH	Test Type: Diametral (D); Axial (A); Lump (L)	Is(50) = Is*(De/50) ^0.45 MPa	UCS Result	Note
KC19H14	38.57	38.57	Gypsum	D	1.227	18.1	
KC19H14	36.88	36.88	Gypsum	D	0.531	18.1	
KC19H14	41.26	41.26	Gypsum	D	1.258	18.1	
KC19H14	41.26	41.26	Gypsum	D	1.621	18.1	
KC19H14	41.35	41.35	Gypsum	D	1.510	18.1	
KC19H14	41.26	41.26	Gypsum	A	1.236	18.1	
KC19H14	41.35	41.35	Gypsum	A	1.509	18.1	
KC19H14	41.40	41.40	Gypsum	A	1.197	18.1	
KC19H13	48.06	48.06	Gypsum	D	0.706	22.1	
KC19H13	48.09	48.09	Gypsum	A	1.283	22.1	
KC19H13	48.12	48.12	Gypsum	D	1.601	22.1	
KC19H13	48.12	48.12	Gypsum	D	0.801	22.1	
KC19H13	48.22	48.22	Gypsum	D	1.366	22.1	
KC19H13	48.22	48.22	Gypsum	A	1.323	22.1	
KC19H13	48.25	48.25	Gypsum	A	1.241	22.1	
KC19H13	48.28	48.28	Gypsum	A	1.904	22.1	
KC19H13	49.15	49.15	Gypsum	D	1.648	22.1	
KC19H13	49.15	49.15	Gypsum	D	1.366	22.1	
KC19H13	49.25	49.25	Gypsum	D	1.507	22.1	
KC19H13	49.25	49.25	Gypsum	A	1.526	22.1	
KC19H13	49.28	49.28	Gypsum	A	1.471	22.1	
KC19H10	29.50	29.50	Gypsum	D	1.130	26.3	
KC19H10	29.55	29.55	Gypsum	A	1.444	26.3	
KC19H10	29.65	29.65	Gypsum	D	1.319	26.3	
KC19H10	29.70	29.70	Gypsum	A	1.208	26.3	
KC19H10	29.75	29.75	Gypsum	D	2.873	26.3	Outlier - ommited
KC19H10	29.75	29.75	Gypsum	D	1.272	26.3	
KC19H10	47.27	47.27	Gypsum	D	1.554	26.3	
KC19H10	47.27	47.27	Gypsum	D	1.601	26.3	
KC19H10	47.34	47.34	Gypsum	D	1.695	26.3	
KC19H10	47.90	47.90	Gypsum	A	1.132	26.3	
KC19H04	29.40	29.40	Gypsum	D	1.319	26.4	
KC19H04	29.40	29.40	Gypsum	D	1.413	26.4	
KC19H04	29.40	29.40	Gypsum	A	1.170	26.4	
KC19H04	29.43	29.43	Gypsum	A	1.625	26.4	
KC19H04	29.47	29.47	Gypsum	D	0.754	26.4	
KC19H04	29.47	29.47	Gypsum	A	1.471	26.4	
KC19H04	29.51	29.51	Gypsum	A	1.773	26.4	
KC19H04	26.00	26.00	Gypsum	D	0.565	26.4	
KC19H04	26.00	26.00	Gypsum	D	0.942	26.4	
KC19H04	26.08	26.08	Gypsum	A	0.814	26.4	
KC19H04	26.13	26.13	Gypsum	A	0.801	26.4	

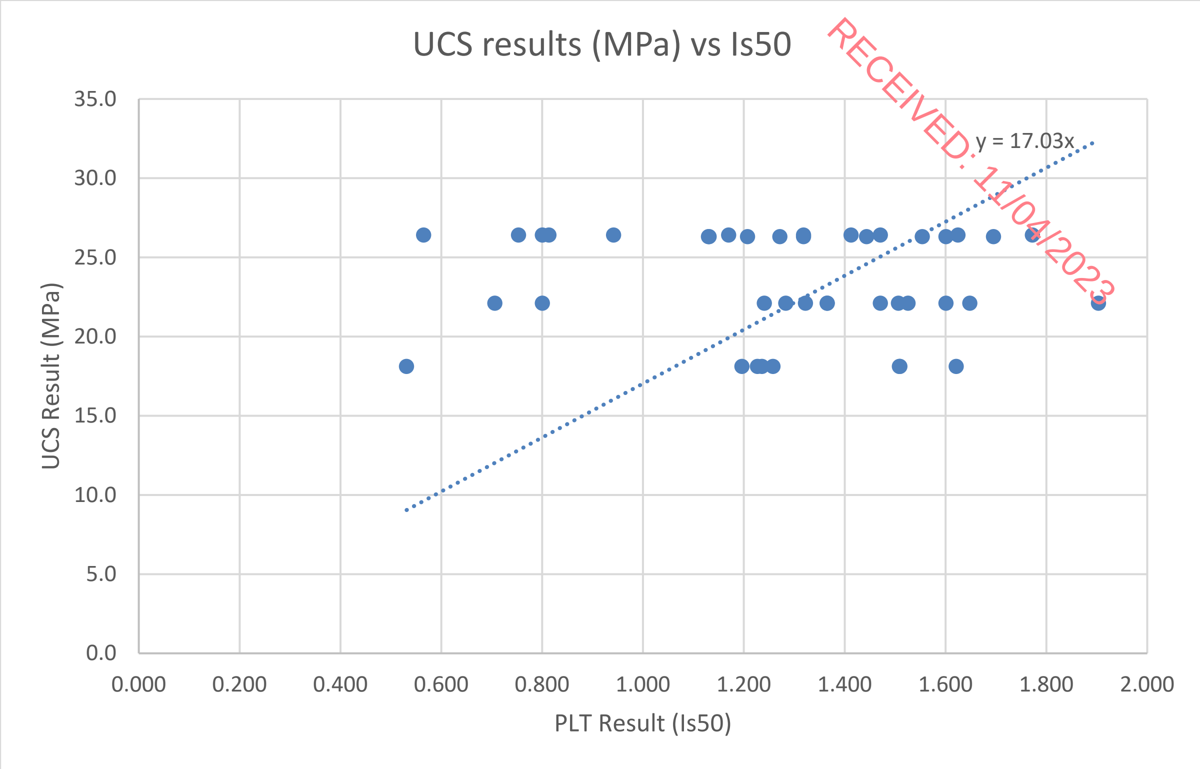

Figure 2-2: Linear regression line between UCS and PLT data

2.4 Borehole Laser Surveys

Borehole surveys were completed in 17 of the 23 boreholes drilled along the L4900 road; eight in Zone A1, two in Zone A2, two in Zone B3 and six in Zone B4. Fourteen scans were completed using the laser survey tool. Three scans utilised a sonar survey tool, one in Zone A2 (KC19H13) and two in Zone B4 (KC19H14, KC19H16), since the excavations into which these boreholes penetrated contained water to varying depths. Figure 2-3 shows the positions of all of the borehole scans carried out along the L4900 road. Also shown on the figure is the position of the approximate extent of the historical subsidence area. This comes from a 1999 drawing.

SRK were provided with 3D point cloud data which it then processed to provide detailed 3D images of the underground workings. A description of the process SRK used to convert point cloud data into detailed 3D images of the underground workings is presented in Appendix D.

The following sections use the images to help inform its interpretation of underground stability conditions and to support the results of the 2D numerical modelling.

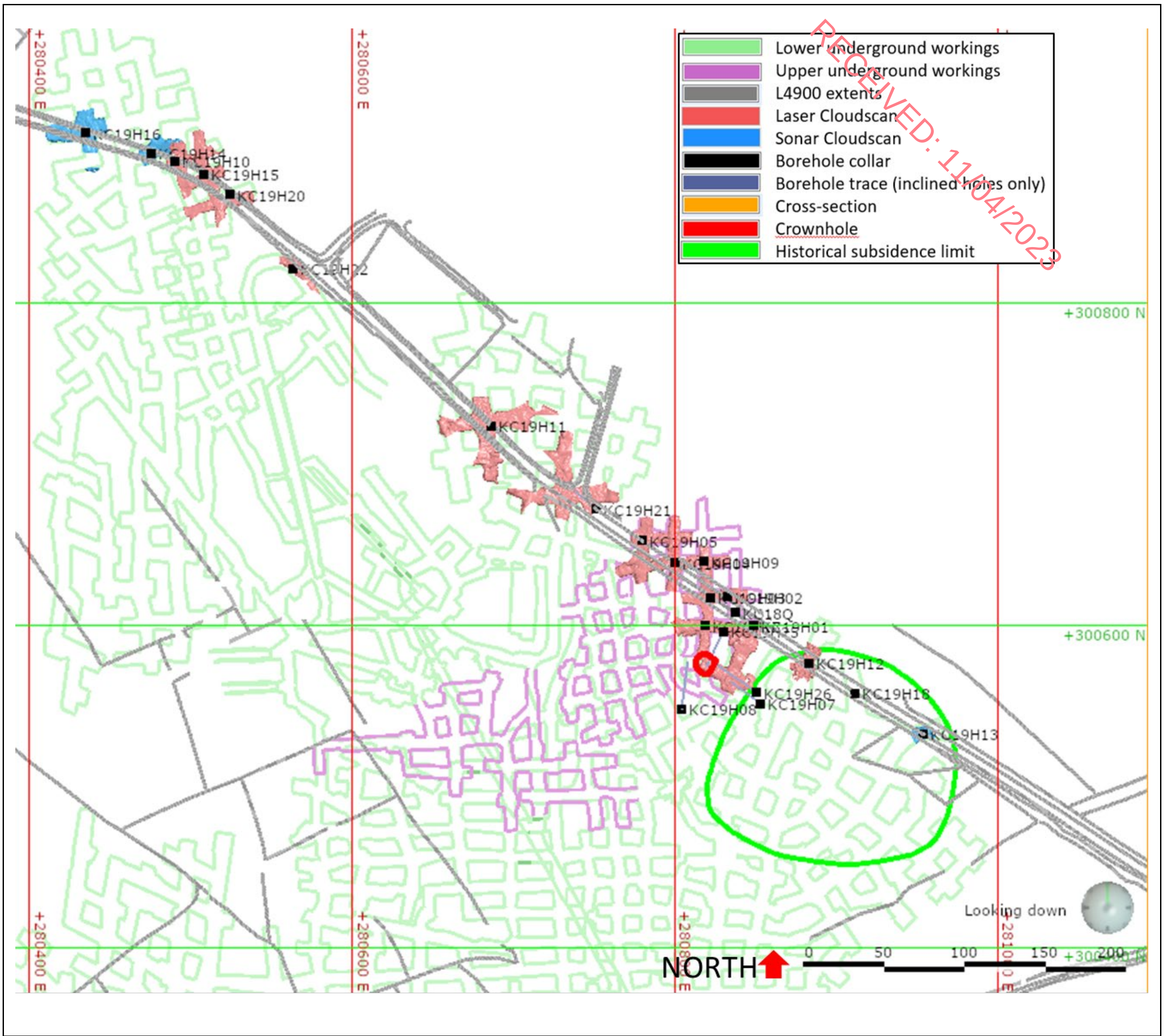

Figure 2-3: Location of Underground Borehole Surveys

3 DECEMBER 2018 CROWNHOLE

3.1 Historical Crownholes at Drumgoosat

Over the years, there has been a number of crownhole formations on Gypsum Industries land at Drumgoosat; they are therefore not an uncommon feature above shallow underground mine workings. At Drumgoosat Mine, 24 sinkholes or crownholes are recorded in the Gyproc crownhole database. These have appeared on the surface above the mine area where the mine is 50 m or shallower below surface. Over 70% of these occurred during the period of active mining at Drumgoosat, while the remaining 30% occurred since mining was completed. Where measured, the crownholes were 3 to 4 m in diameter and 1 to 2 m deep, on average. Figure 3-1 is a histogram showing crownhole occurrence. There are many years where crownholes were not recorded; however, the maximum number of annual occurrences happened during active mining.

Figure 3-2 shows the location of all historical crownholes above Drumgoosat Mine. These are shown as coloured balls with the colour representing the year in which the crownhole was formed. The red circle shows the L4900 crownhole.

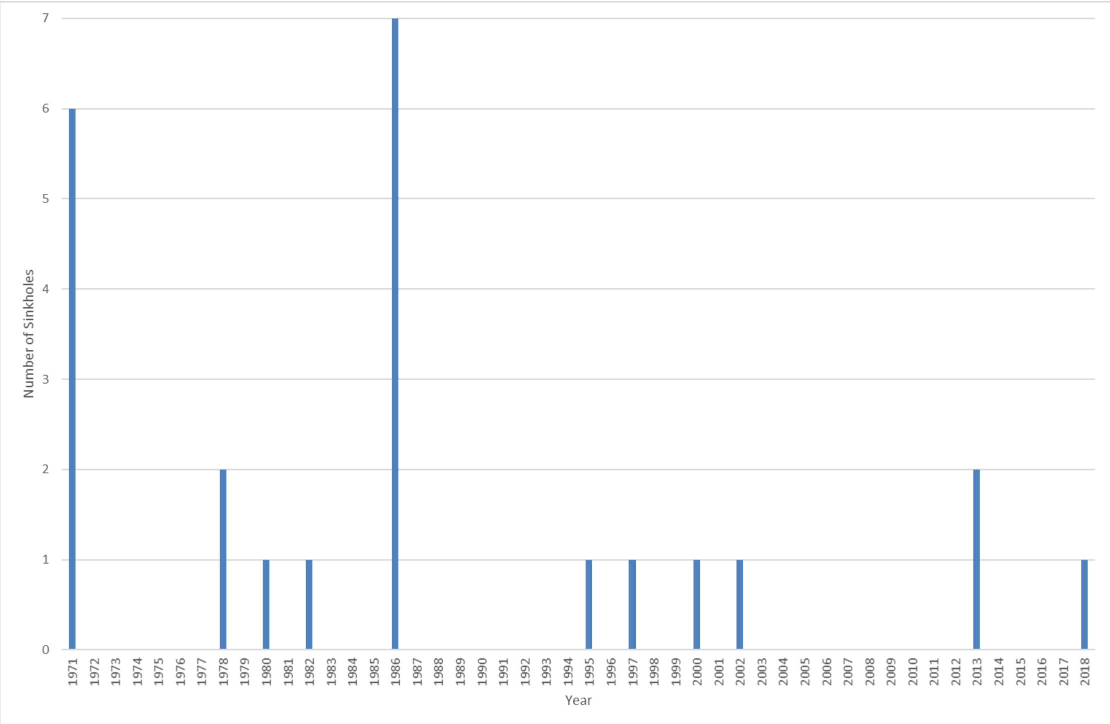

Figure 3-1: Histogram of Crownhole Occurrence

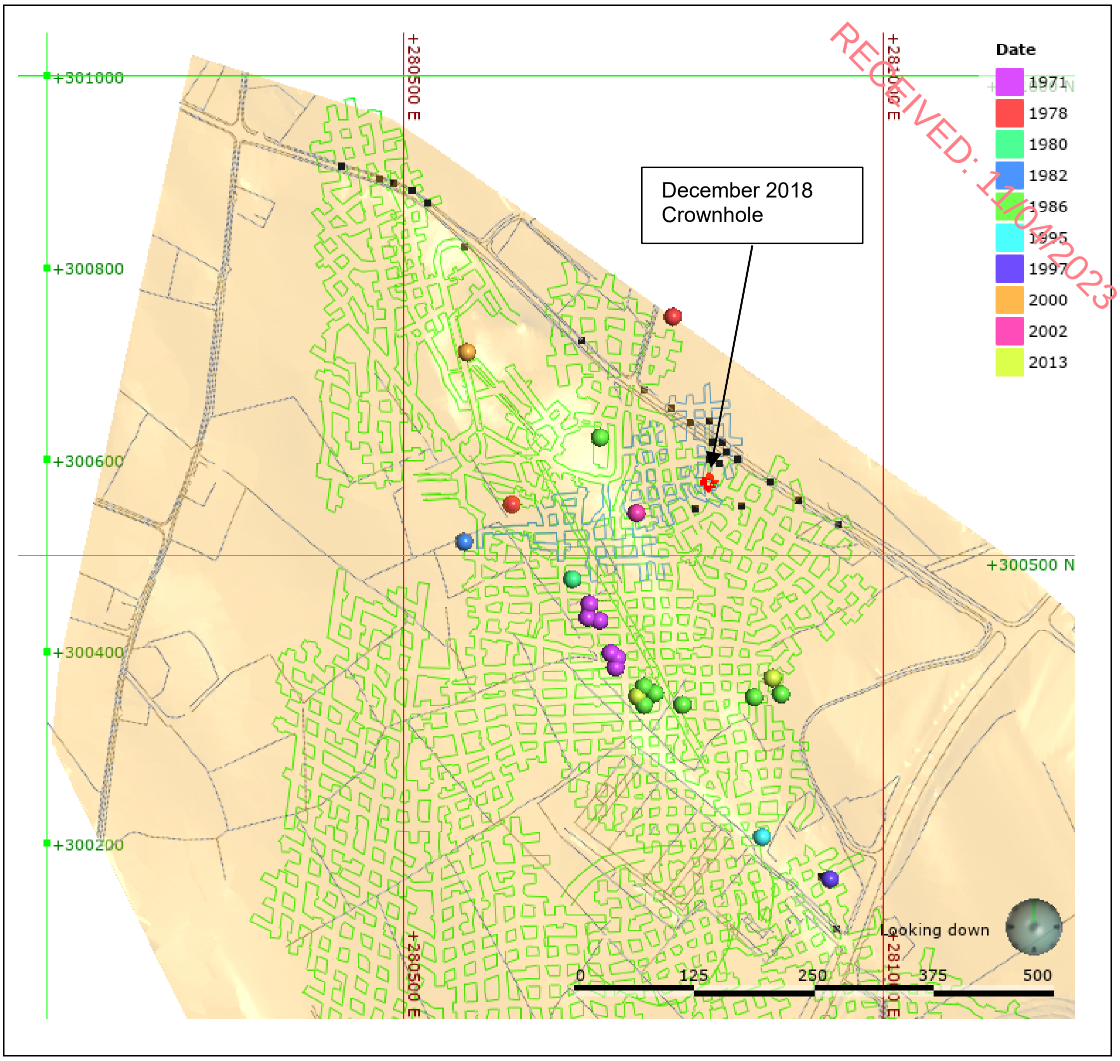

Figure 3-2: Historical Crownhole Location Plan

3.2 Mechanism of Crownhole Formation

The upper surface of the gypsum is very irregular as a result of weathering and erosion. Figure 3-3 is a low-resolution image of the top surface of the gypsum in Knocknacran Quarry taken by SRK in 1998. The irregular shape of the gypsum surface can be clearly seen containing dips and narrow crevasses. In order to maintain a stable underground mine, the gypsum roof beam must be of a minimum thickness.

Mining triggered crownhole formation is generally the result of roof failures over shallow workings, with the cavity travelling up to the surface as more of the overlying mudstone and clay till is dislodged. It is probable that the presence of cohesionless gravels of basalt or dolerite above the roof facilitates the formation of sinkholes, because it collapses and flows easily into the workings.

Figure 3-4 is a schematic cross section showing the formation of a crownhole. This drawing has been extracted from SRK's 1998 Drumgoosat report.

Figure 3-3: Irregular Gypsum Surface Exposed in Knocknacran Quarry

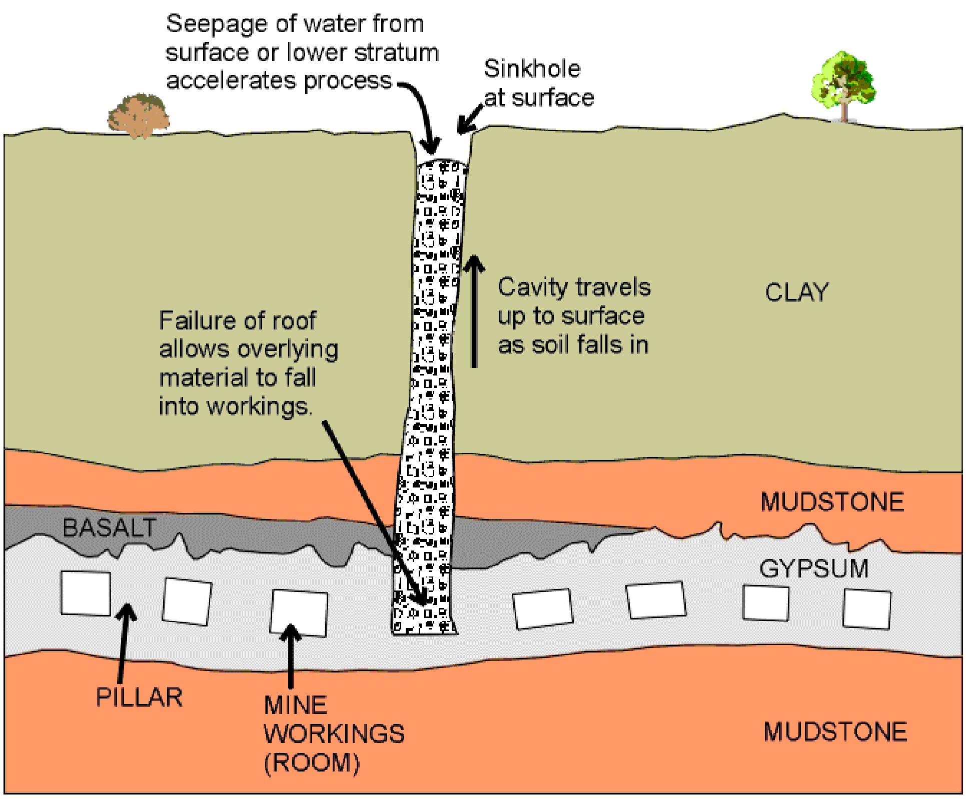

Figure 3-4: Schematic cross section showing crownhole formation

Given the general competence of the gypsum, there is a minimum roof beam thickness required for stability. In its 09 January 2019 technical note to Gyproc which provided a preliminary assessment of the stability of the rooms and pillars in the vicinity of the L4900 crownhole, SRK conducted sensitivity analyses to determine minimum roof beam thickness for stability. This analysis and its results are repeated below.

To check the stability of the roof beam, SRK has created a finite element numerical model using the lithological profile of KC18Q1. Below a 7.9 m thick roof beam, a generic room and pillar layout comprising 6 m high by 10 m wide rooms (the maximum roof span at four-way intersections) separated by 8 m wide pillars has been created. The model was run using the lower gypsum 'expected' strength parameters as defined in in SRK's October 2018 report (Ref: UK30238: Drumgoosat Subsidence Event – Technical Report); namely, a cohesion of 277 kPa and a friction angle of 48°. The 7.9 m thick roof beam thickness was progressively reduced and a graph of roof beam thickness against maximum beam deflection at the excavation roof mid-point was plotted, with beam deflection being represented as a percentage of beam thickness. This graph is shown in Figure 3-5.

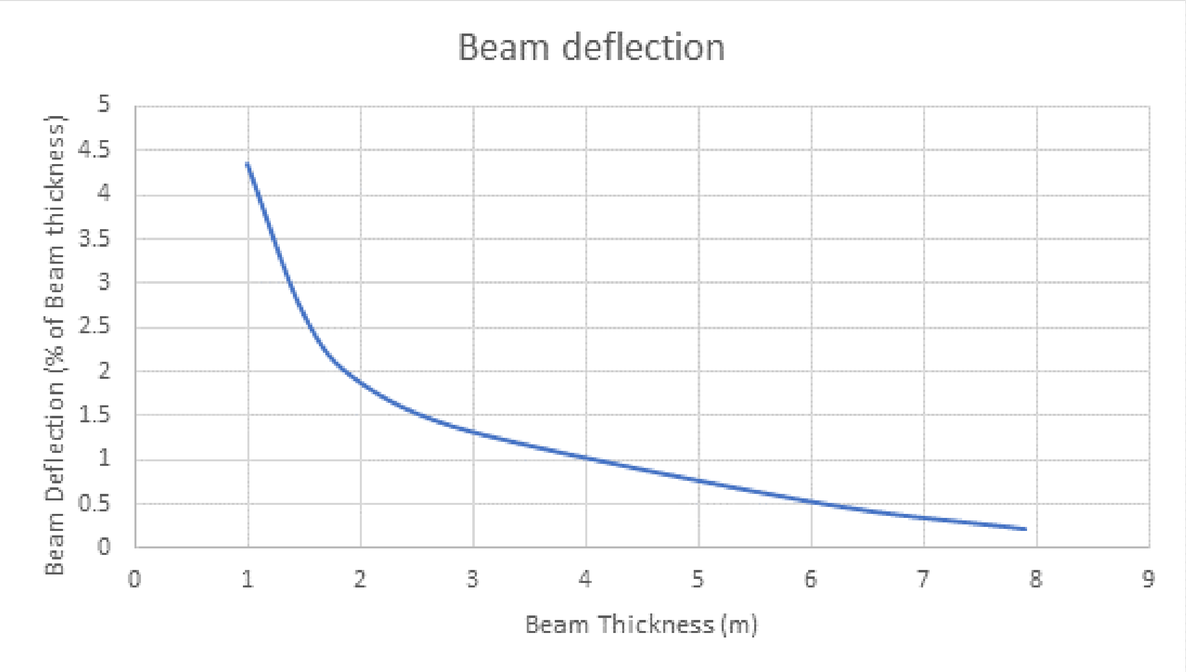

Figure 3-5: Relationship Between Beam Thickness and Beam Deflection

In SRK's 1998 report (Ref: U1225) it was stated that a beam deflection of 4% or less was consistent with long term beam stability. Based on the graph in Figure 3-5, the minimum beam thickness for long term stability should be 1.1 m or thicker for a 10 m roof span.

This is consistent with independent work carried out at horizontally bedded stone mines in the USA which indicated that roof beams of 4 ft (1.2 m) or thicker and with spans of 8 m to 16 m were stable. Roof beams less than 4 ft (1.2 m) thick generally failed. The results of this work are shown in the graph in Figure 3-6.

(*Ref: Information Circular 9526 - Pillar and Roof Span Design Guidelines for Underground Stone Mines. Gabriel S. Esterhuizen, Dennis R. Dolinar, John L. Ellenberger, and Leonard J. Prosser DEPARTMENT OF HEALTH AND HUMAN SERVICES, Centers for Disease Control and Prevention, National Institute for Occupational Safety and Health Office of Mine Safety and Health Research, Pittsburgh, PA • Spokane, WA – May 2011*)

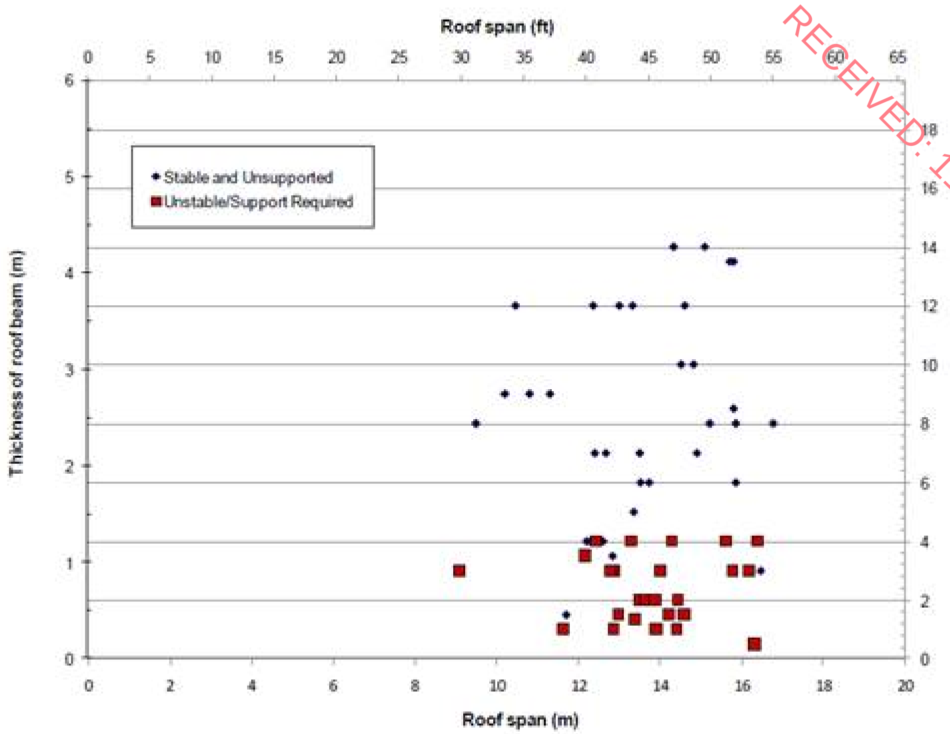

Figure 30. Chart showing the effect of the thickness of the roof beam on excavation stability in mines that did not experience horizontal stress related instability.

Figure 3-6: Relationship Between Beam Thickness, Span and Stability for Stone Mines in the USA

3.3 Description of December 2018 Crownhole

The L4900 crownhole identified by a drone survey carried out on 10 December 2018 is located about 35 m SW of the L4900 road and 340 m NW of the road junction with the R179. The crownhole, which is roughly circular in shape, has a diameter of about 9 m and is about 7 m deep. The top 0.5 to 0.75 m of the wall of the crownhole comprises made ground overlying a concrete slab which is underlain by about 5 cm of gravel. This appears to be a concrete road or hard standing. SRK understands that in the 1980s this area was used as a stockpiling and sorting area for the mine. The walls of the crownhole below the concrete comprise about 2-3 m of clayey glacial till sloping at an angle estimated to be at around 45-50°. From these images there is evidence of water in the crownhole void and the walls of the crownhole are wet. A location plan of the crownhole showing the position of the site investigation boreholes drilled in the area is presented in Figure 3-7. Figure 3-8 shows a number of photographs of the crownhole.

In order to investigate the cause of the crownhole, a number of cored boreholes were drilled around the crownhole positions. To ensure safety of the drill rig, these holes were collared a significant distance away from the crownhole and then drilled at an inclined angle towards the crownhole. The inclined holes KC19H7, KC19H8, and KC19H25, along with a vertical hole KC19H17, are shown in Figure 3-7. All these holes were geologically logged by British Gypsum geologists and geotechnically logged by SRK.

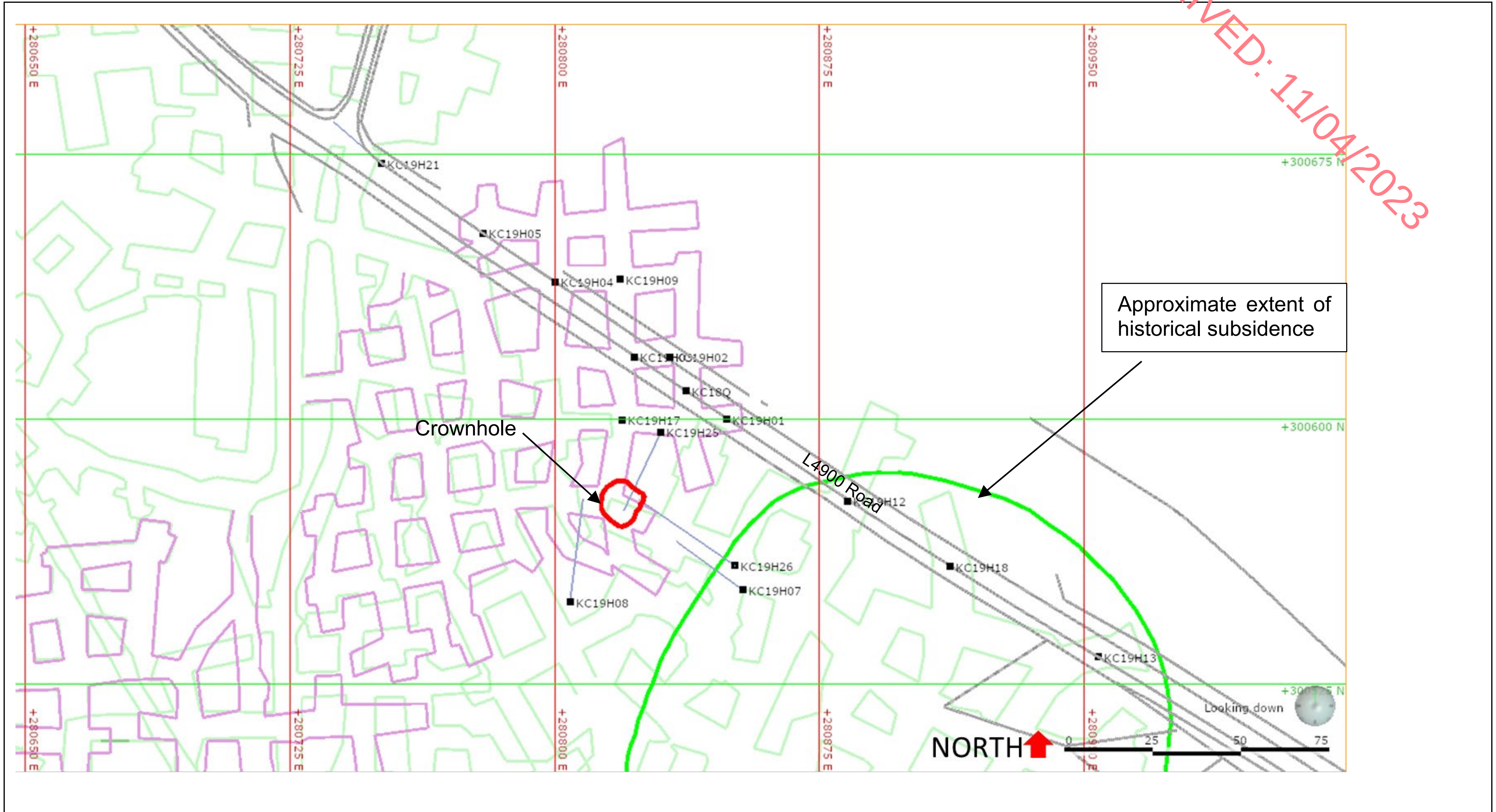

Figure 3-7: December 2018 Crownhole Location Plan

Note: The blue lines in the figure represent the position of the Lower Seam Upper Horizon workings. The green lines represent the position of the Lower Seam Lower Horizon Workings.

Figure 3-8: Pictures of the Crownhole

The full geological logs, core photographs and geotechnical logs are provided in Appendices A and B. An extract of the geotechnical log for Borehole KC19H08 is presented in Table 3-1.

The geotechnical log indicates no gypsum between the dolerite and the mine workings, whereas the geological log indicates a small amount of very broken gypsum at the base of the hole. The bottom 3 m core run (2.12 m vertical thickness) of the borehole is represented by only 10% core recovery or 90% core loss. The geology log describes this bottom, poor core recovery zone as '*Very weak mixed rock cobbles, loose angular to sub-rounded clasts of dolerite and gypsum with red-brown mud coating*'. The core photographs for this hole are shown in Figure 3-9.

Table 3-1: Borehole KC19H08 – Extract of Geotechnical Log

Interval Data				RMR Zone	Recovery Data			ROCK TYPE
BHID	FROM (m)	TO (m)	LENGTH (m)		REC. (m)	REC(%)	CORE LOSS	
KC-19H08	0.00	24.00	24.00	No Core	0.00	0	24.00	No Core
KC-19H08	24.00	26.40	2.40	Rock	1.10	46	1.30	Dolerite
KC-19H08	26.40	27.50	1.10	Rock	1.00	91	0.10	Dolerite
KC-19H08	27.50	29.40	1.90	Rock	1.10	58	0.80	Dolerite
KC-19H08	29.40	31.40	2.00	Rock	1.17	59	0.83	Dolerite
KC-19H08	31.40	32.40	1.00	Rock	0.35	35	0.65	Dolerite
KC-19H08	32.40	35.40	3.00	Rock	0.80	27	2.20	Dolerite
KC-19H08	35.40	38.40	3.00	Rock	0.30	10	2.70	Dolerite
KC-19H08	38.40	41.40	3.00	No Core - Mine Workings			3.00	

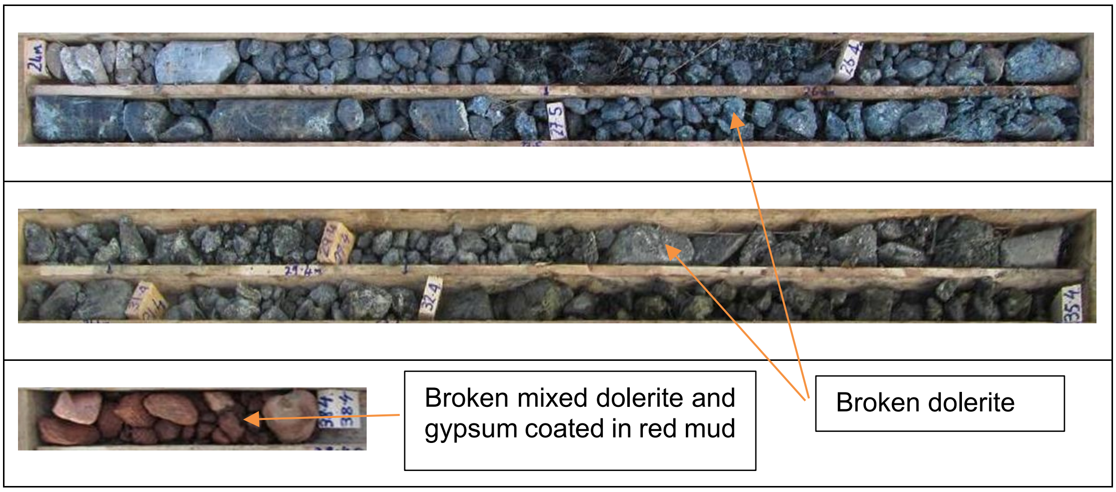

Figure 3-9: Borehole KC19H08: Core Photographs

The information from Borehole KC19H08 indicates that the gypsum roof beam, if present, is very thin and, together with the overlying dolerite, has been disturbed by the December 2018 crownhole development.

Borehole KC19H25 was an angled hole drilled to the south towards the crownhole. An extract of the geotechnical log for Borehole KC19H25 is presented in Table 3-2.

Table 3-2: Borehole KC19H25: Extract of Geotechnical Log

Interval Data					Recovery Data			
BHID	FROM (m)	TO (m)	LENGTH (m)	RMR Zone	REC. (m)	REC(%)	CORE LOSS	ROCK TYPE
KC-19H25	0.00	30.00	30.00	No Core	0.00	0	30.00	No Core
KC-19H25	30.00	30.60	0.60	Rock	0.24	40	0.36	Dolerite
KC-19H25	30.60	31.10	0.50	Rock	0.10	20	0.40	Dolerite
KC-19H25	31.10	31.50	0.40	Rock	0.21	53	0.19	Dolerite
KC-19H25	31.50	32.00	0.50	Rock	0.00	0	0.50	Dolerite
KC-19H25	32.00	32.50	0.50	Rock	0.14	28	0.36	Dolerite
KC-19H25	32.50	32.60	0.10	Rock	0.10	100	0.00	Dolerite
KC-19H25	32.60	33.00	0.40	Rock	0.40	100	0.00	Gypsum
KC-19H25	33.00	34.30	1.30	Rock	1.29	99	0.01	Gypsum
KC-19H25	34.30	35.00	0.70	Rock	0.70	100	0.00	Gypsum
KC-19H25	35.00	36.00	1.00	Rock	0.93	93	0.07	Gypsum
KC-19H25	36.00	37.50	1.50	Rock	1.41	94	0.09	Gypsum
KC-19H25	37.50	39.00	1.50	Rock	1.50	100	0.00	Gypsum
KC-19H25	39.00	40.60	1.60	Rock	1.60	100	0.00	Gypsum
KC-19H25	40.60	41.00	0.40	Rock	0.40	100	0.00	Gypsum
KC-19H25	41.00	42.50	1.50	No Core - Mine Workings			1.50	
KC-19H25	42.50	44.00	1.50	Rock	0.65	43	0.85	Gypsum
KC-19H25	44.00	45.60	1.60	Rock	1.50	94	0.10	Gypsum

The core from Borehole KC19H25, although drilled to intersect the L4900 crownhole from the opposite direction to that of KC19H08, displays completely different rock mass characteristics to that of KC19H08. There is a strong, competent roof beam above the mine workings which is about 11 m thick. The mine workings are shown to be less than 1.5 m thick; however, the geology log indicates that the 1.5 m of gypsum below the void is infill material; that is, material that has dropped into the mine void from above the roof. From 41.0 m to 44.15 m the geology log describes *'MINE WORKINGS (infilled). Mine roof reached at 41m, rods dropping through very soft material (NOT A VOID) to 42.5m but nothing recovered from 42.5m to 44m. Cobbles disks and stubs of core, mixture of lithologies in the following order: Dolerite, D+ Upper (laminated), D+ Mid (red/granitic), D+ (lower), D Upper, D+ Mid, then 15cm angular gravel at base'*. The core photographs for this borehole are shown in Figure 3-10.

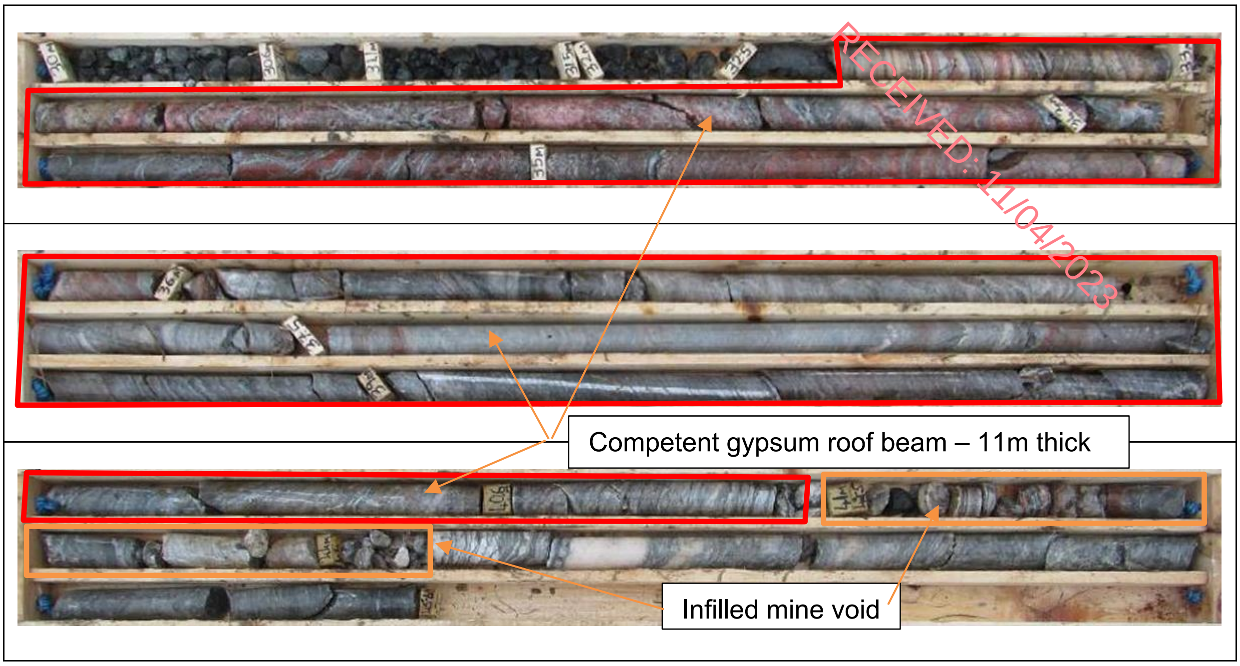

Figure 3-10: Borehole KC19H25: Core Photographs

3.4 Data Interpretation

Figure 3-11 is a composite S-N cross section through the middle of the position of the L4900 crownhole extending north to beyond the L4900 road. The cross section comprises a 50 m E-W slice and shows all boreholes drilled within that slice, along with the laser surveys of the lower seam upper horizon and lower seam lower horizon workings. The location and extent of the cross section is shown in Figure 3-12.

The drillhole traces are colour coded to identify the lithologies through which the drillholes were sunk (gypsum light blue, dolerite pink, etc), as well as the traces where the drillholes passed through the mine workings (identified in yellow along the drillhole traces). Lines connecting the top of the dolerite traces in the drillholes and the top of the gypsum traces have been added to define the dolerite thickness and gypsum roof beam thickness. The position of the crownhole and the L4900 road are also shown.

Figure 3-12 shows rendered images of the laser scan surveys undertaken of the rooms immediately below the surface expression of the crownhole. Laser scan View 1 is shown looking towards the NW towards the end of the room immediately below the crownhole. The survey image shows stable roof and wall conditions; however, the end of the room comprises a surface sloping at 30° into the void. This is interpreted as doleritic sand flow cone resulting from a breach in the roof beam. The angle of the slope is typical of the angle of repose of loose sand.

View 2 is at right angles to View 1 and, because the viewpoint is in the rock outside the mine, an ‘x-ray’ render has been used to allow the viewer to see into the mine through the rock. The roof beams are all stable. The 30° sloping doleritic sand flow cone is clearly visible on the right-hand side of the view and the domed feature at the top of the cone is probably where the breach of the gypsum roof beam has taken place.

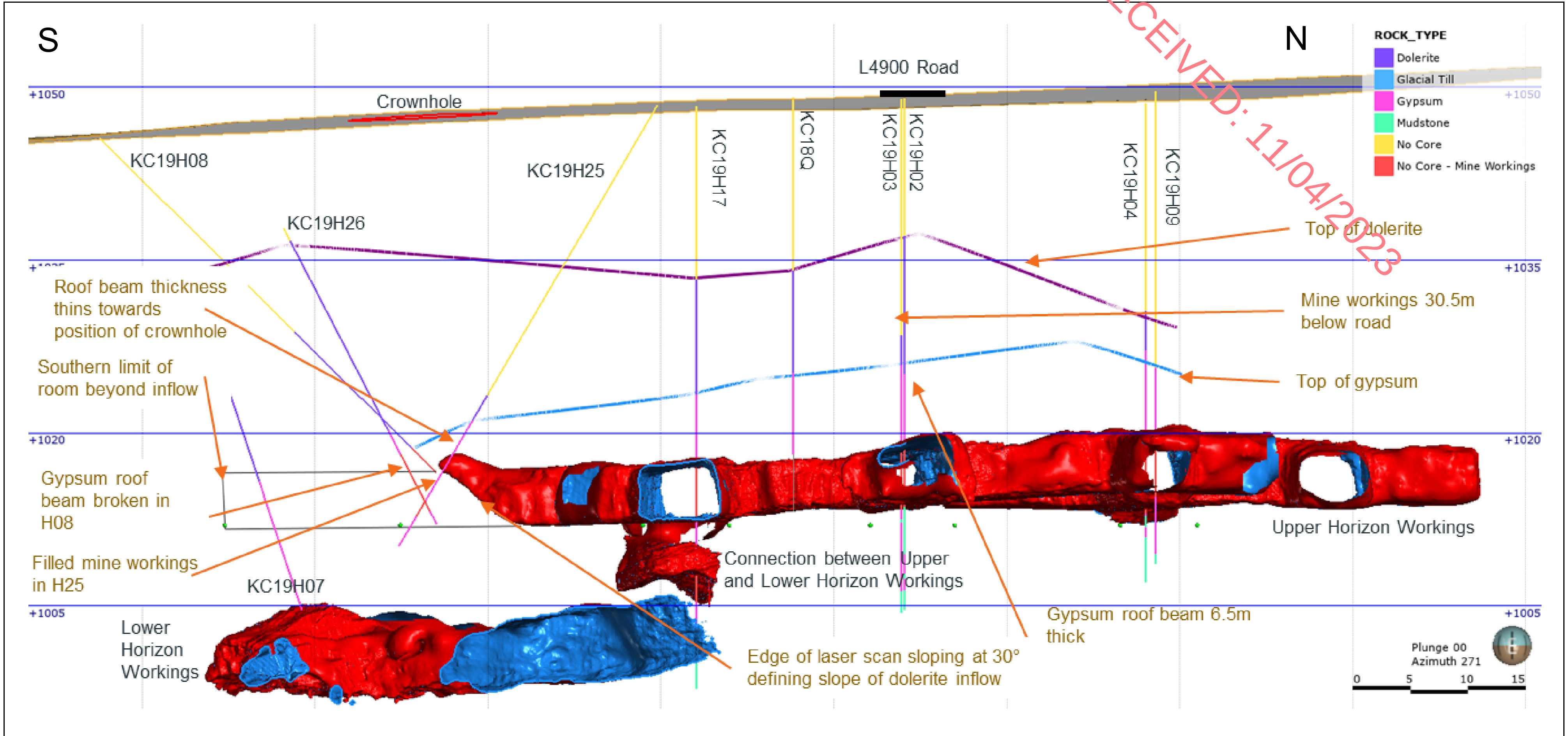

Figure 3-11: Composite S-N Cross Section through December 2018 Crownhole

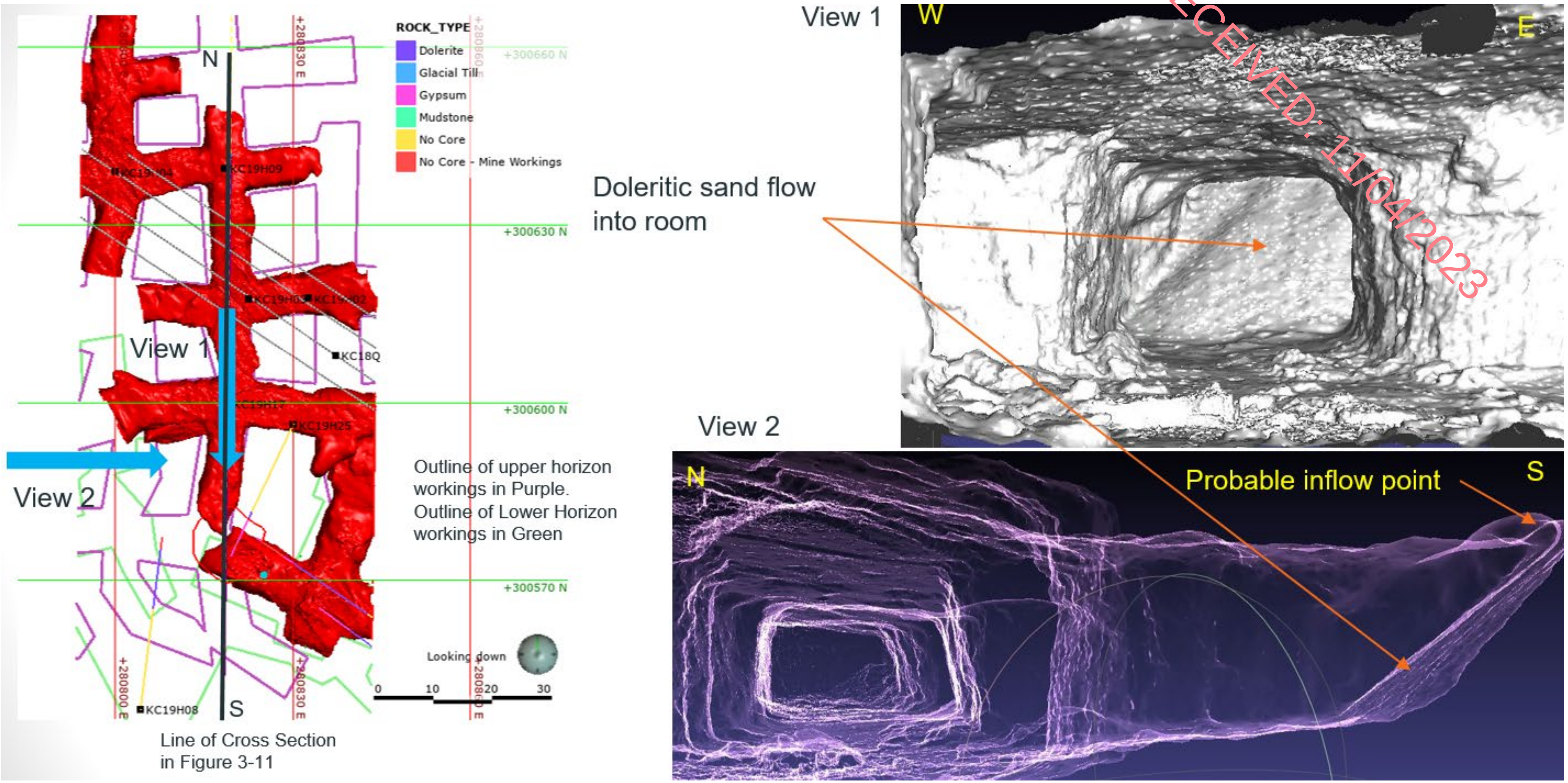

Figure 3-12: Laser Scan Surveys of Underground Workings Near L4900 Crownhole

3.5 December 2018 Crownhole Probable Failure Mechanism

SRK has used the information in the borehole record, its understanding of crownhole formation, and the 3D laser surveys of the mine voids in the vicinity of the crownhole to determine the probable failure mechanism of the L4900 crownhole.

1. Below the position of the December 2018 crownhole there was a significant thinning of the gypsum roof beam as indicated in Borehole K19H08, 17 and K18H26

2. It was difficult to obtain intact core pieces of the dolerite overlying the gypsum. There was 75% dolerite core loss in the Borehole K19H25 and 54% dolerite core loss in Borehole K19H08. What was recovered comprised cobble and gravel material, with the core loss material inferred as being doleritic sand. From this it is inferred that the dolerite comprised a generally free flowing sandy mass.

3. The dolerite sand entered the mine void through the breach in the gypsum roof beam forming the 30° angle of repose that can be seen in the laser scan survey. Once the stable 30° face had been formed, the dolerite plugged the breach preventing further movement of material into the underground void.

4. The flow of dolerite would have created a void undercutting the overlying glacial till which then would have started migrating into the doleritic void. Because of the cohesive nature of the glacial till as it fell into the doleritic void, it would have done so in blocks or large lumps. This would result in bulking of the till as it filled the void. Bulking means that the broken material would occupy a space larger in volume that it would have as an in-situ material.

5. The failure of the glacial till then migrated upwards towards surface.

6. The concrete road surface probably spanned the crownhole initially, preventing it from migrating to surface; however, over a short period of time, the walls of the glacial till void would continue to deteriorate. There is evidence of water seepage below surface in the crownhole void that would have accelerated this deterioration. This deterioration would be accompanied by a gradual enlargement of the void, removing support below the concrete slab which then failed in tension, allowing the crownhole to appear as a surface feature.

SRK notes that crownholes are generally very local in nature. Their manifestation underground is restricted to the area of the breach. Beyond the breach in the roof beam, the December 2018 crownhole formation has no impact on the stability of the underground workings. This is clearly shown in the laser surveys in Figure 3-12.

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4 L4900 UNDERMINING ANALYSIS

4.1 Areas for Investigation

Four specific areas of undermining were investigated by borehole drilling, laser scanning (red) and sonar scanning (blue). These are zones A1, A2, B3 and B4. These zones together with the cross sections for analysis are shown in Figure 4-1. The A and B zones coincide with the A and B zones used for the on-going road deformation survey reporting. Within each area two cross sections have been identified and agreed between SRK, Gyproc and the regulatory authorities EMD for finite element modelling. Further details of these areas are presented in the next chapter of the report.

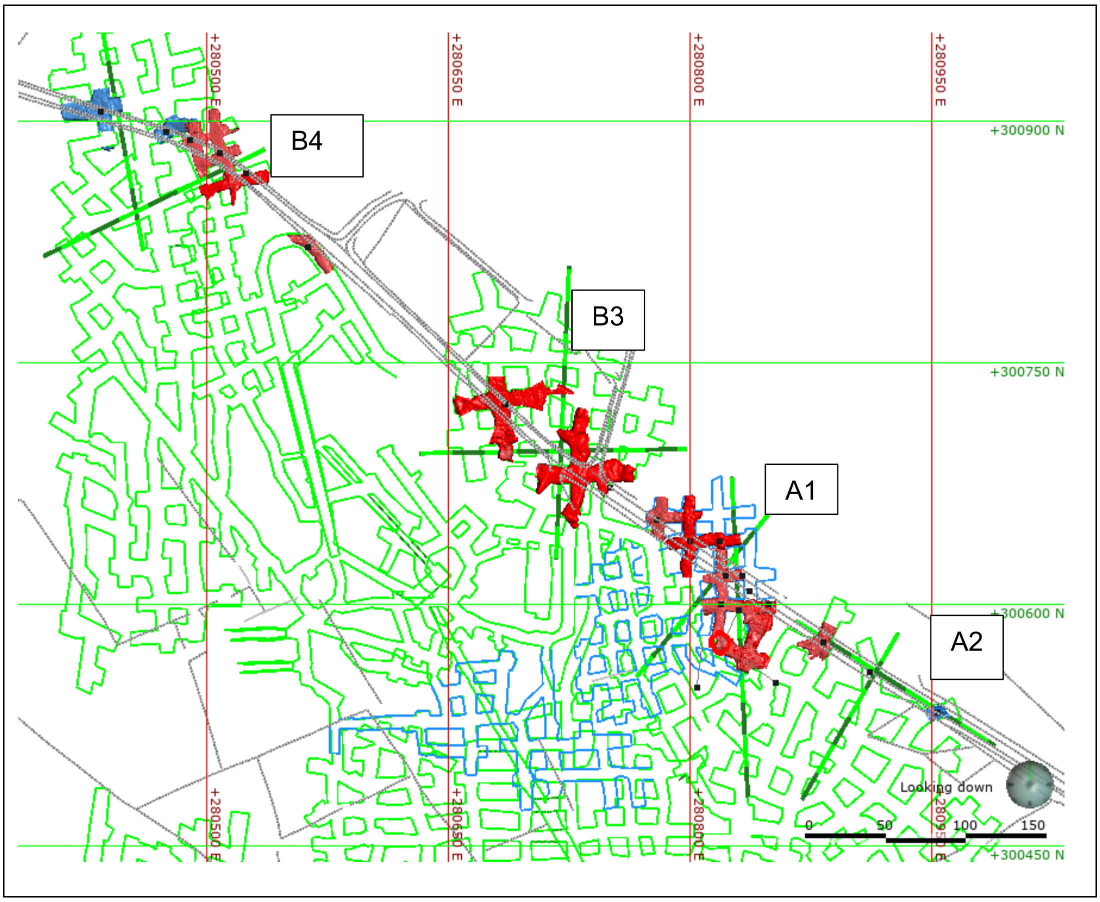


Figure 4-1: Location of Undermining Analysis Areas

4.2 Geotechnical Characteristics for Modelling

The empirical Hoek Brown strength criterion has been used to determine the cohesion and friction angle of the units for which borehole core has been recovered and which can be logged using Bieniawski's rock mass classification system. This is the dolerite and the gypsum. For the glacial till or drift overlying the dolerite, through which the boreholes were open holed, a typical cohesion and friction angle for firm clay has been used. Similarly, for the mudstone underlying the gypsum a typical cohesion and friction angle for stiff clay has been used. For each zone, the borehole logging and strength data using all of the boreholes in that zone was combined to provide average location specific parameters from which specific values of cohesion and friction angle were calculated in the Rocscience software RocData. In addition, the numerical model requires inputs of rock density, Young's modulus and Poisson's ratio.

Engineering judgement has been used to assign these parameters. A summary of the strength and deformation data used in the finite element numerical modelling is presented in Table 4-1.

Table 4-1: Modelling Input Parameters

ZONE	Lithology	IRS	RMR	GSI	Unit Weight (MN/m3)	c (MPa)	phi (°)	Young's Modulus (MPa)	Poisson's Ratio
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
A1	Glacial Till	N/A	N/A	N/A	0.018	0.2	20	100	0.3
	Dolerite	11	28	23	0.020	0.06	35	1000	0.3
	Gypsum	25	54	49	0.023	0.264	49.78	4875	0.15
	Mudstone	N/A	N/A	N/A	0.020	0.4	20	200	0.2
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
A2	Glacial Till	N/A	N/A	N/A	0.018	0.3	20	100	0.3
	Dolerite	17	29	24	0.020	0.06	35	1000	0.3
	Gypsum	21	59	54	0.023	0.287	49.7	4875	0.15
	Mudstone	N/A	N/A	N/A	0.020	0.4	20	200	0.2
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
B3	Glacial Till	N/A	N/A	N/A	0.018	0.3	20	100	0.3
	Dolerite	1	27	22	0.020	0.06	35	1000	0.3
	Gypsum	22	57	52	0.023	0.273	49.59	4875	0.15
	Mudstone	N/A	N/A	N/A	0.020	0.4	20	200	0.2
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
B4	Glacial Till	N/A	N/A	N/A	0.018	0.3	20	100	0.3
	Dolerite	15	29	24	0.020	0.06	35	1000	0.3
	Gypsum	25	57	52	0.023	0.298	50.39	4875	0.15
	Mudstone	N/A	N/A	N/A	0.020	0.4	20	200	0.2

The gypsum is a homogeneous material with only limited variability in terms of strength and RMR/GSI. The rock mass rating for the gypsum lies in upper Fair rock mass quality designation. The range of GSI values and intact rock strength (IRS) values for the gypsum determined from the detailed geotechnical borehole, and consequently the rock mass cohesion and friction angle derived from the GSI are consistent with the 'Expected' gypsum strength used by SRK for its predictive mine stability analyses carried out along both the L4900 and R179 roads and reported in October 2018. The borehole logging thus validates the strength estimate for the gypsum used previously and thus validates the outcome of the predictive analyses reported in October 2018. For comparison the 'Expected' gypsum strength values used in those analyses was:

- GSI – 53
- Cohesion – 0.277 MPa
- Friction Angle – 47°

5 L4900 ZONE A1

5.1 General Description of Undermining

Figure 5-1 shows a plan view of Zone A1 as modelled using Leapfrog Geo 3D modelling software. The figure shows both lower and upper underground working surveys, borehole collars and traces, the locations of cross-sections used for later RS2 modelling, the 3D cloudscan data, the extents of the L4900 road and the location of the crownhole.

The length of the undermined road in this zone is approximately 102 m, with a minimum depth of mining of 30 m (below drillhole KC19H09). The cloudscans are predominantly of the upper mine workings, where room height is approximately 6 m, based on the cloudscan data. Only the upper workings lie directly beneath the road; the lower workings all lie to the south west of the L4900.

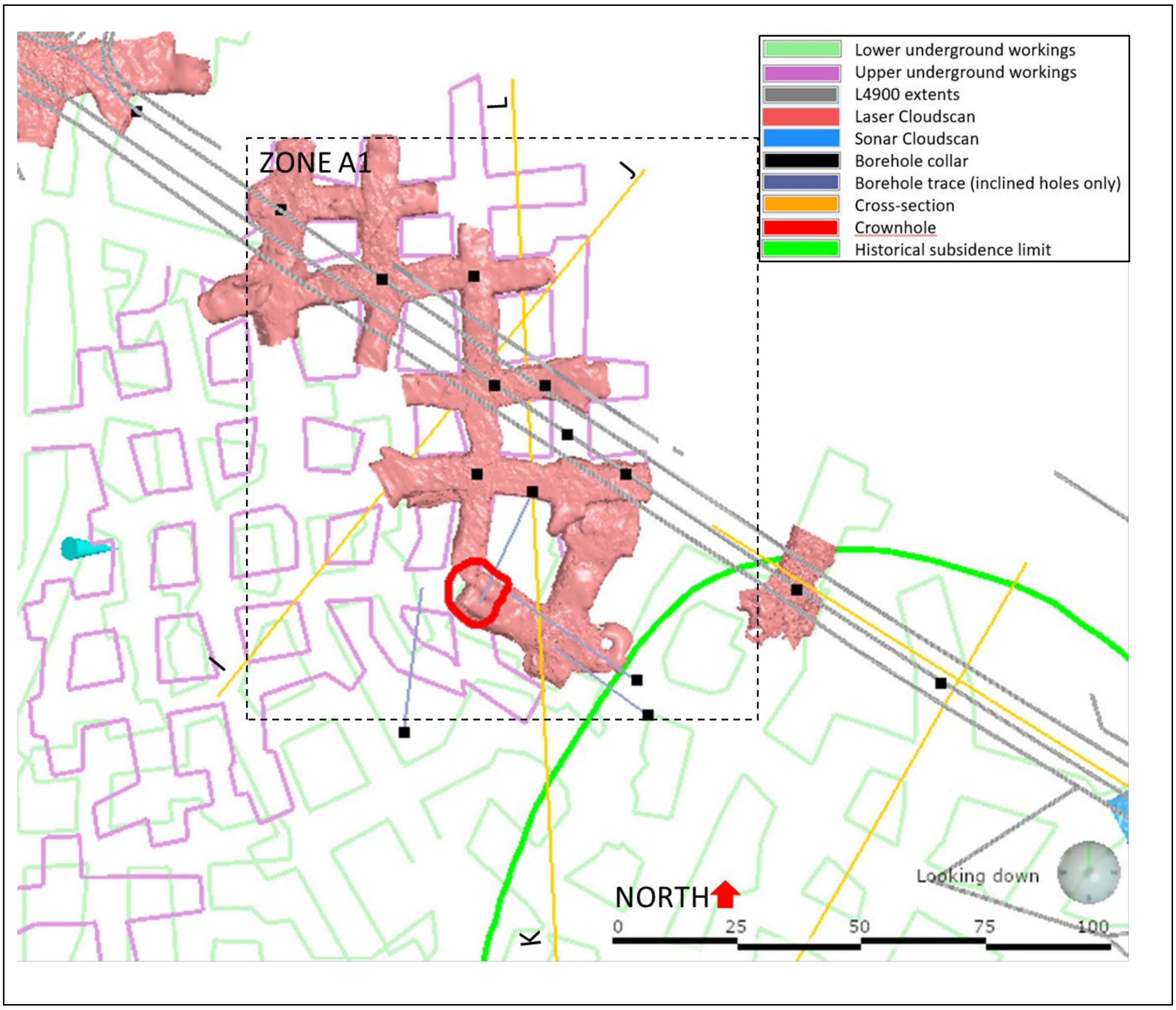


Figure 5-1: Plan view of L4900 Zone A1.

5.2 Summary of Investigation works

Table 5-1 shows details of the boreholes in L4900 Zone A1 and Figure 5-2 shows a plan view of their locations. The arrows and numbers in Figure 5-2 indicate the figure number and direction of viewing of the cloudscan images shown later in this section. Figure 5-3 is a cross section along the road showing detail of the boreholes and underground mining.

Table 5-1: Details of boreholes in L4900 Zone A1

BHID	Easting	Northing	Elevation	Max depth (m)	Dip (°)	Azimuth (°)	Cloudscan
KC18Q	280837.25	300607.98	1049.08	31	90	-	Yes
KC19H01	280848.71	300599.99	1049.03	52.7	90	-	Yes
KC19H02	280832.61	300617.68	1049.09	44.5	90	-	No
KC19H03	280822.53	300617.57	1048.92	44.6	90	-	Yes
KC19H04	280800.02	300638.97	1048.62	41.6	90	-	Yes
KC19H05	280779.78	300652.78	1048.69	43.2	90	-	Yes
KC19H07	280853.33	300551.53	1045.82	47.5	60	306	Yes
KC19H08	280804.28	300548.19	1045.64	41.4	45	7	No
KC19H09	280818.42	300639.62	1049.63	41	90	-	No
KC19H17	280819.02	300599.83	1048.32	50.5	90	-	Yes
KC19H25	280830.06	300596.29	1048.45	45.6	57	205	No

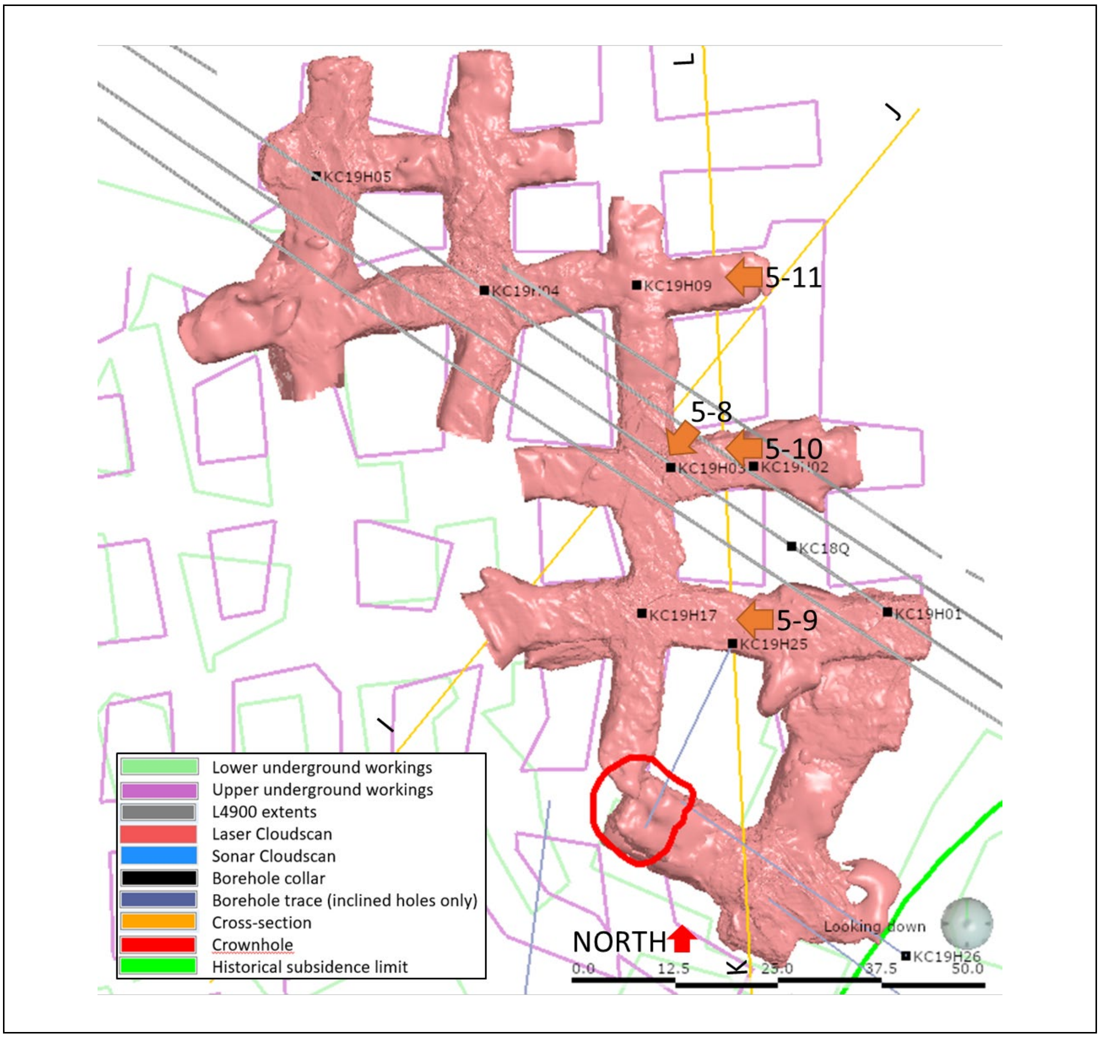


Figure 5-2: Plan view showing borehole locations in L4900 Zone A1

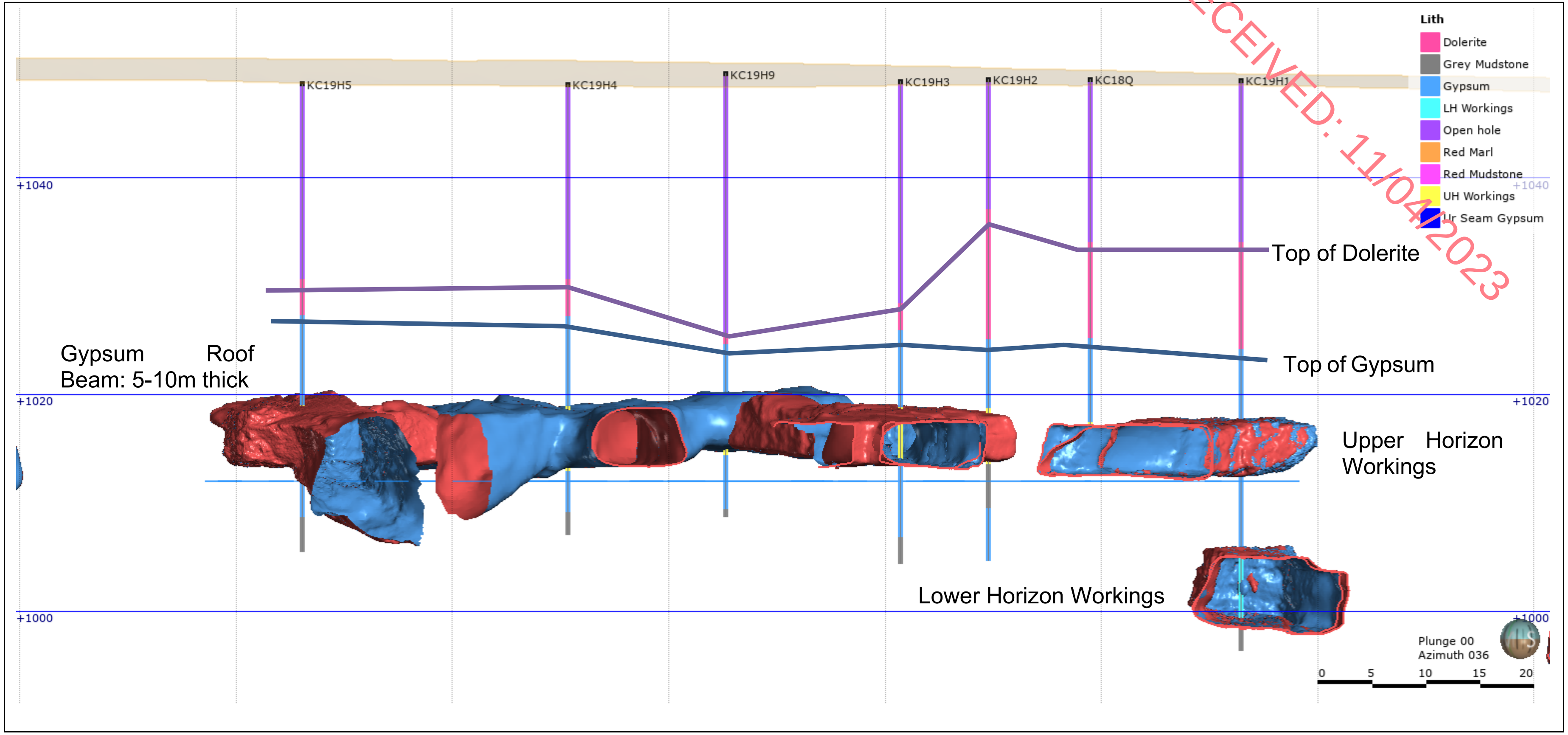


Figure 5-3: Zone A1 – NW-SE Section Along L4900 Road Showing Borehole Geology and Underground Mining

5.3 Finite Element Modelling

5.3.1 Model Geometry

The finite element models analysed are shown in Figure 5-4 and Figure 5-5. Each cross section shows the lithologies, the location of the L4900 road, the mine working, the depth to the mine workings below the road and the thickness of the gypsum roof beam above the workings closest to the L4900 road. At this location the workings lie approximately 30m below the road and the gypsum roof beam is between 6m and 7m thick. Note that the irregular shaped mine workings are based on the underground laser survey. The figures show that all of the workings immediately below the road on both of these cross sections have been laser surveyed. Therefore, their position and size are accurate. The regular shaped rooms have been positioned from the 2D mine survey and a nominal height of 6m applied.

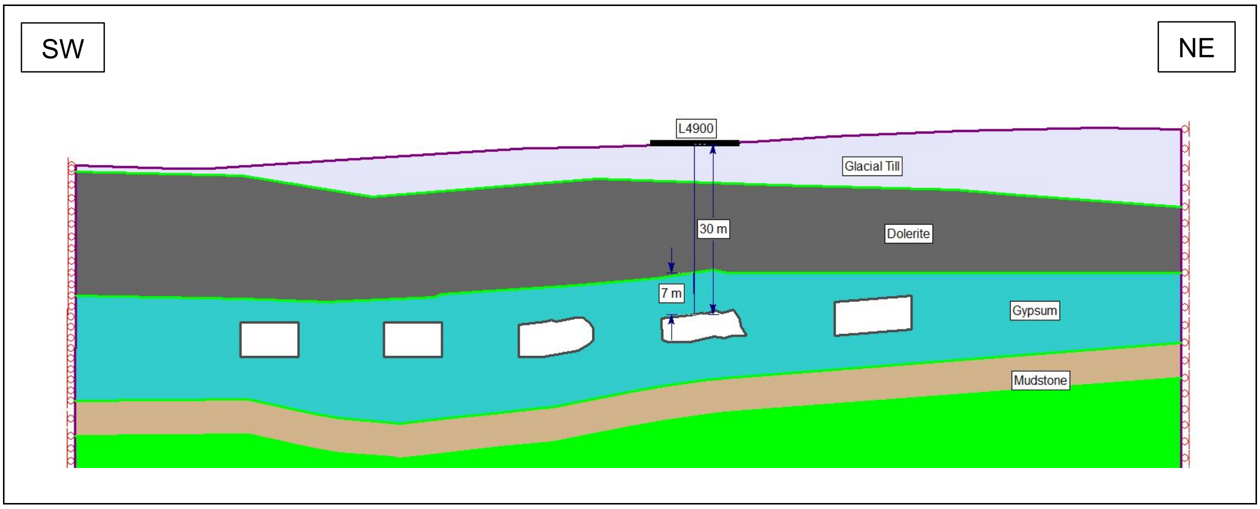


Figure 5-4: Cross Section I-J

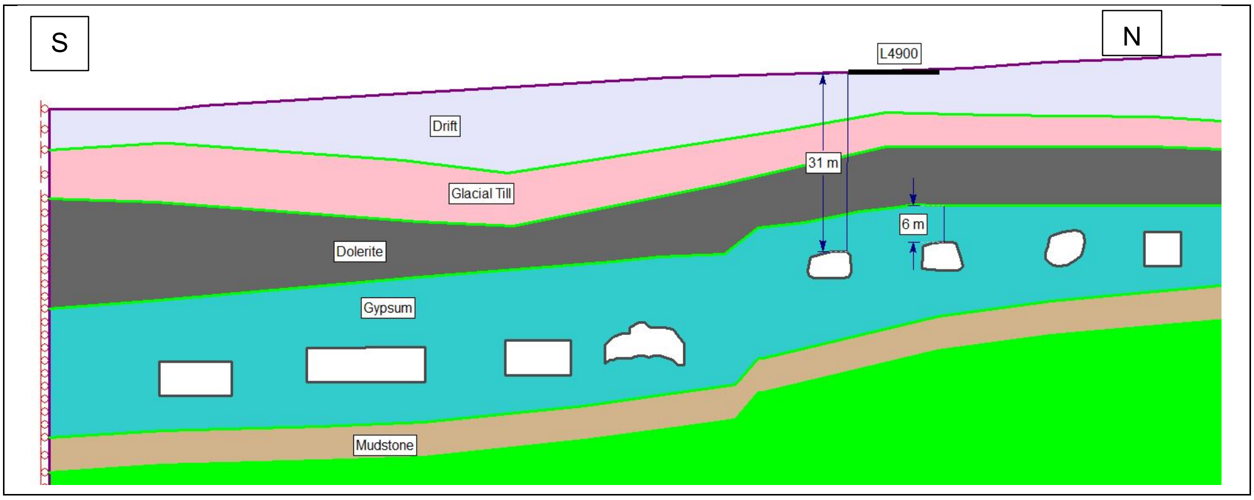


Figure 5-5: Cross Section K-L

5.3.2 Results

The results of the numerical modelling are reproduced below in Figure 5-6 and Figure 5-7. These are contour plots of total displacement in metres. As well as the contours of total displacement the plots show the predicted displacement close to surface below and adjacent to the L4900 road. Also shown is the maximum displacement at the underside of the roof beam above the workings which lie immediately below the road.

For cross section I-J (Figure 5-6) the numerical model defines stable pillar conditions. The maximum roof beam deflection is 0.037m which for the 7m thick gypsum roof beam gives a maximum deflection of 0.5% of roof beam thickness. Note that in Section 3.2 it was stated that roof beam deflection needed to be 4% or greater to give rise to roof beam failure. On surface, below and adjacent to the L4900 road, the maximum predicted surface deformation is 4mm.

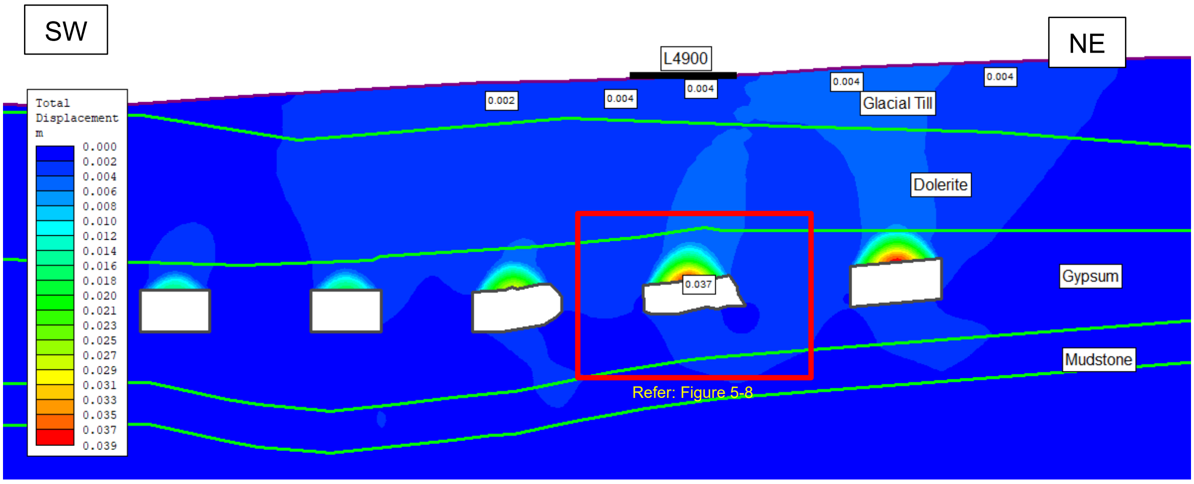


Figure 5-6: Cross Section I-J – Total Displacement

For cross section K-L (Figure 5-7) the numerical model defines stable pillar conditions. The maximum roof beam deflection occurs above a wide room (probably a 4-way intersection) located 75m to the south of the L4900 road. Here the maximum roof deflection is 0.11m which for the 11m thick gypsum roof beam above the room gives a maximum deflection of 1.0% of roof beam thickness. There is no modelled roof beam deflection above the room immediately below the L4900. On surface, below and adjacent to the L4900 road, the maximum predicted surface deformation is 4mm.

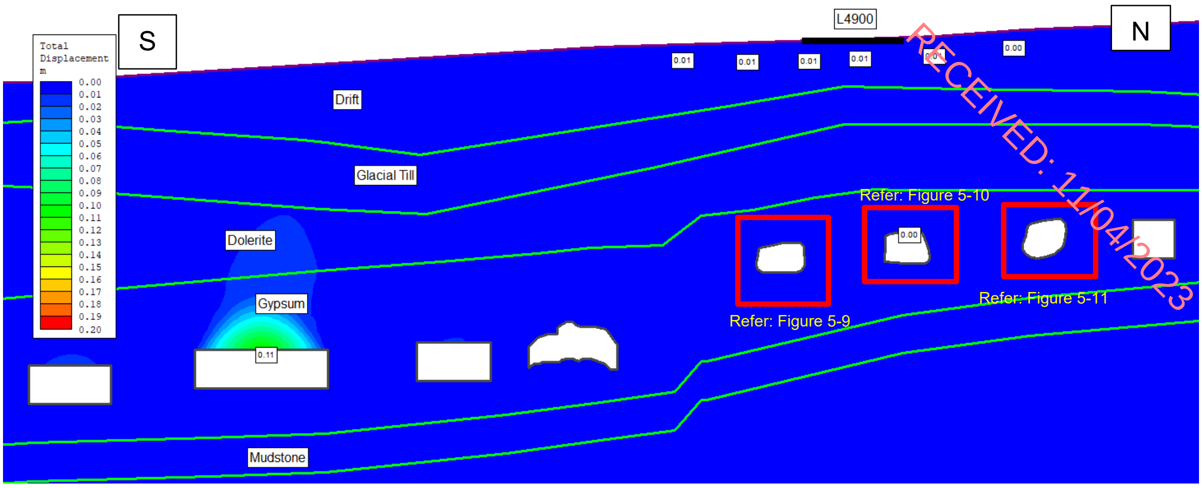


Figure 5-7: Cross Section K-L – Total Displacement

5.4 Cloudscan Surveys

In Figure 5-6 there is a red box surrounding the room below the L4900 road. This defines the limit of the laser scan survey of the 4-way intersection 30m below the L4900 road. This survey is shown in Figure 5-8.

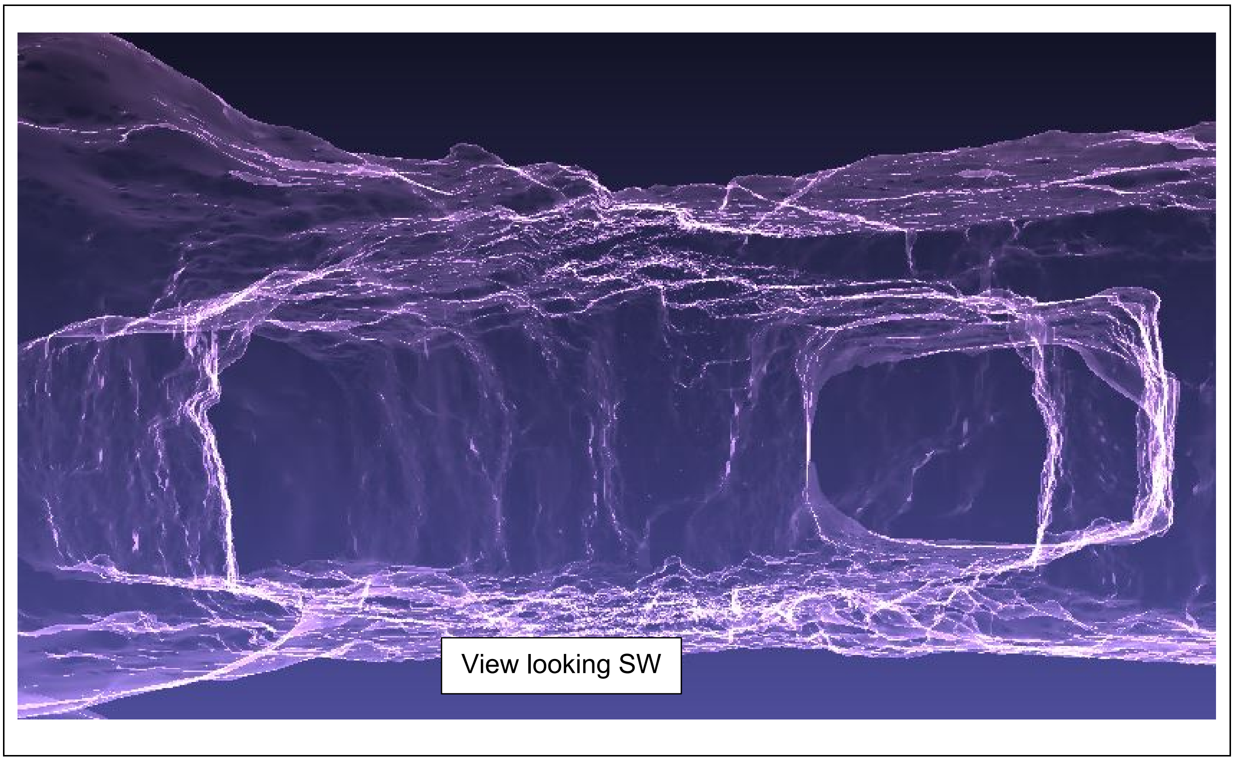


Figure 5-8: Laser scan Image of the 4-Way Intersection below the L4900 along Section I-J

The laser scan shows that the intersection is in good condition. There appears to be some minor slabbing of bedding and sub-vertical joint defined blocks with the debris visible on the floor. The break back height is estimated at only 20 to 30cm whereas the roof beam above this intersection is 7m thick.

Figure 5-9, Figure 5-10 and Figure 5-11 are images from the laser scan surveys of the rooms under the L4900 road. All three images show clean, stable roof beams with no indication of instability. This supports the results of the numerical modelling. The cones on the floor of the room in Figure 5-10 come from the boreholes drilled as this survey looks back towards the position of borehole KC19H03 from where the laser survey was carried out.

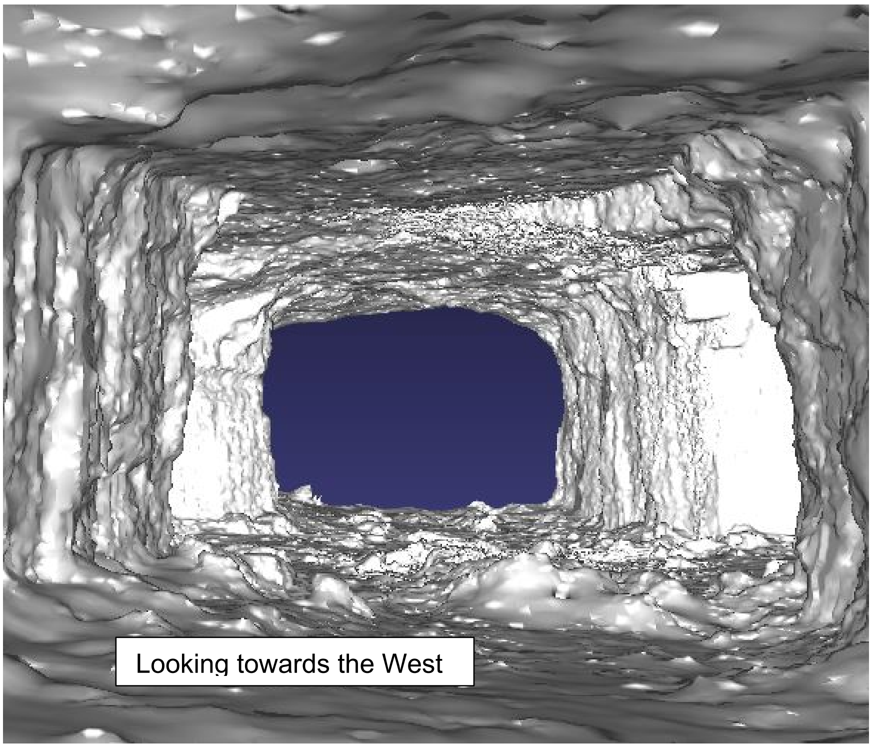


Figure 5-9: Laser Scan Image of the Left-Hand Room below the L4900 along Section K-L

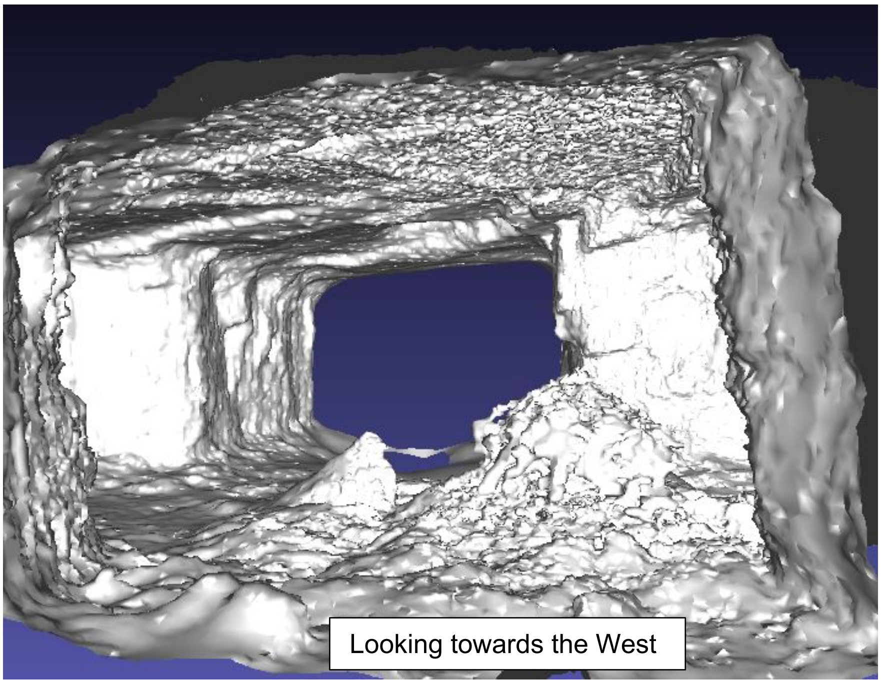


Figure 5-10: Laser Scan Image of the Central Room below the L4900 along Section K-L

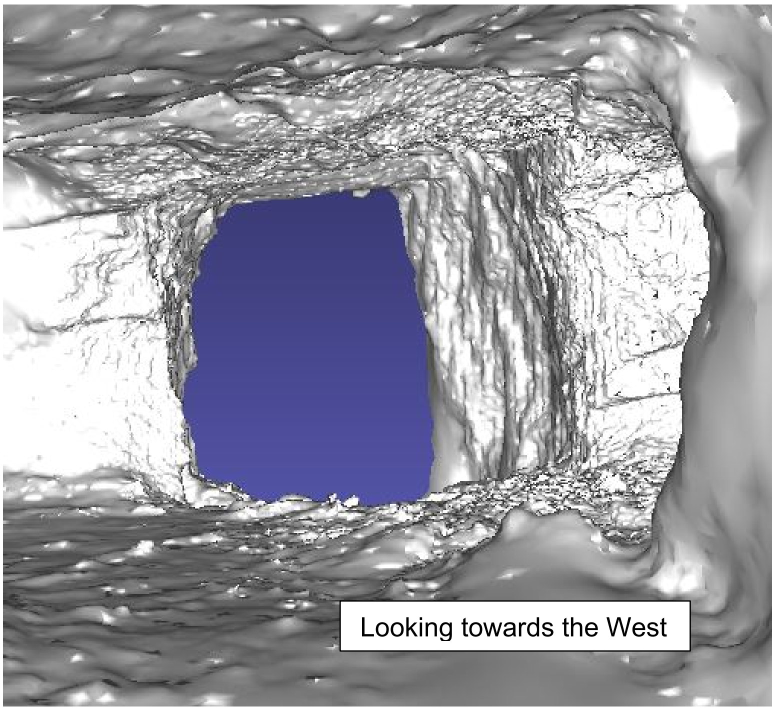


Figure 5-11: Laser Scan Image of the Right-Hand Room below the L4900 along Section K-L

6 L4900 ZONE A2

6.1 General Description of Undermining

Figure 6-1 shows a plan view of Zone A2 as modelled using Leapfrog Geo 3D modelling software. The figure shows both lower and upper underground working surveys, borehole collars and traces, the locations of cross-sections used for later RS2 modelling, the 3D cloudscan data, the extents of the L4900 road and the location of the crownhole. The green circle represents the approximate extent of the historical subsidence,

The length of the undermined road in this zone is approximately 100 m, with a minimum depth of mining of 48 m (below drillhole KC19H12). Only Lower Seam lower horizon mine workings are present below the road in this area. The cloudscans indicate that room height is about 4 m. Three rooms undermine the road at three locations in this area. There are no wide span 4-way intersection beneath the road. As measured from the three boreholes within this zone the total Lower Gypsum seam thickness is on average 18m. The gypsum roof beam thickness above the mine workings is 12m.

Note that this zone is located in the area of historical subsidence where about 1 m of surface deformation has been recorded over the last 20 years. Cross section M-N passes through the middle of the subsidence area.

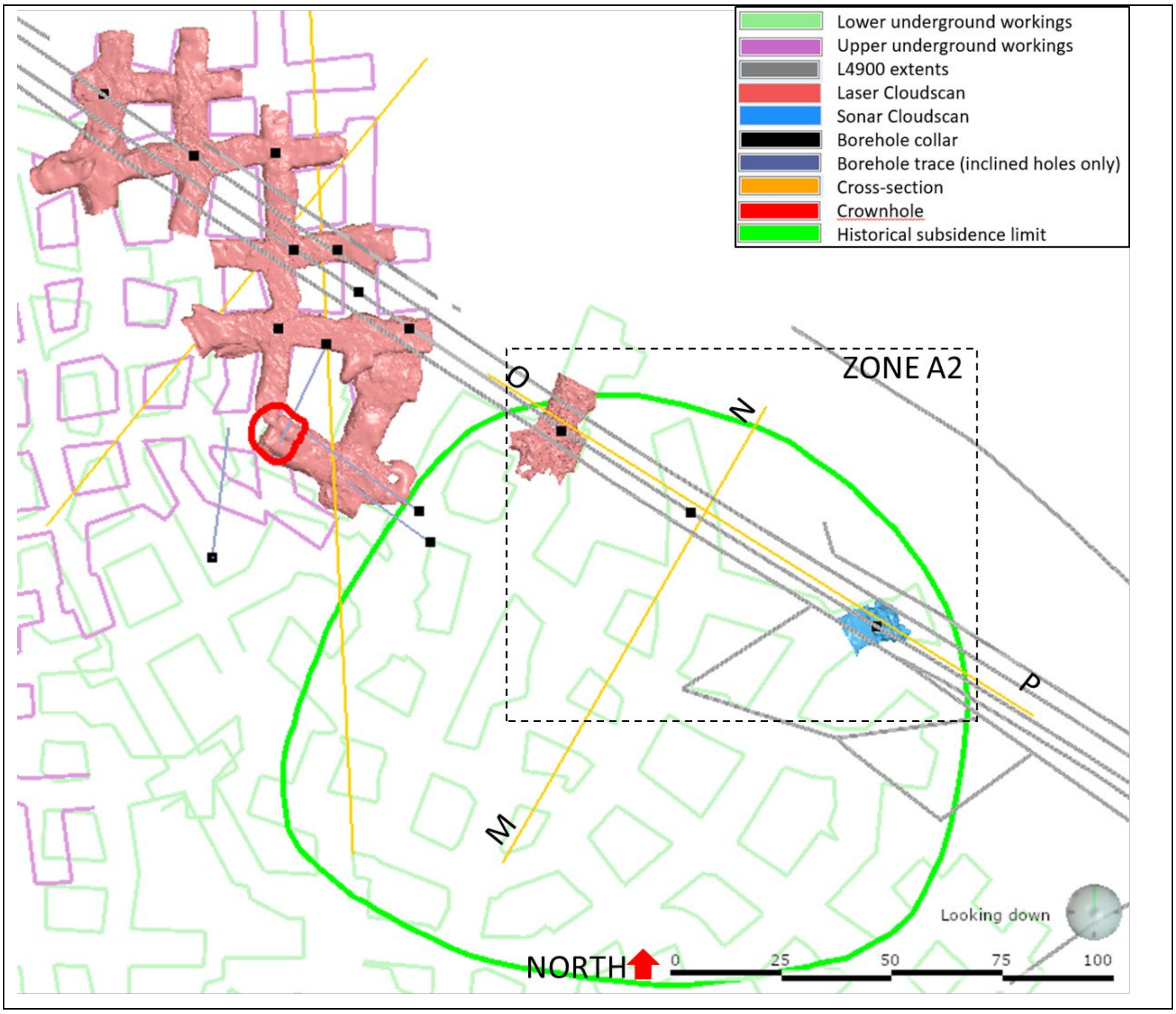


Figure 6-1: Plan view of L4900 Zone A2.

6.2 Summary of Investigation Works

Table 6-1 shows details of the boreholes in L4900 Zone A2 and

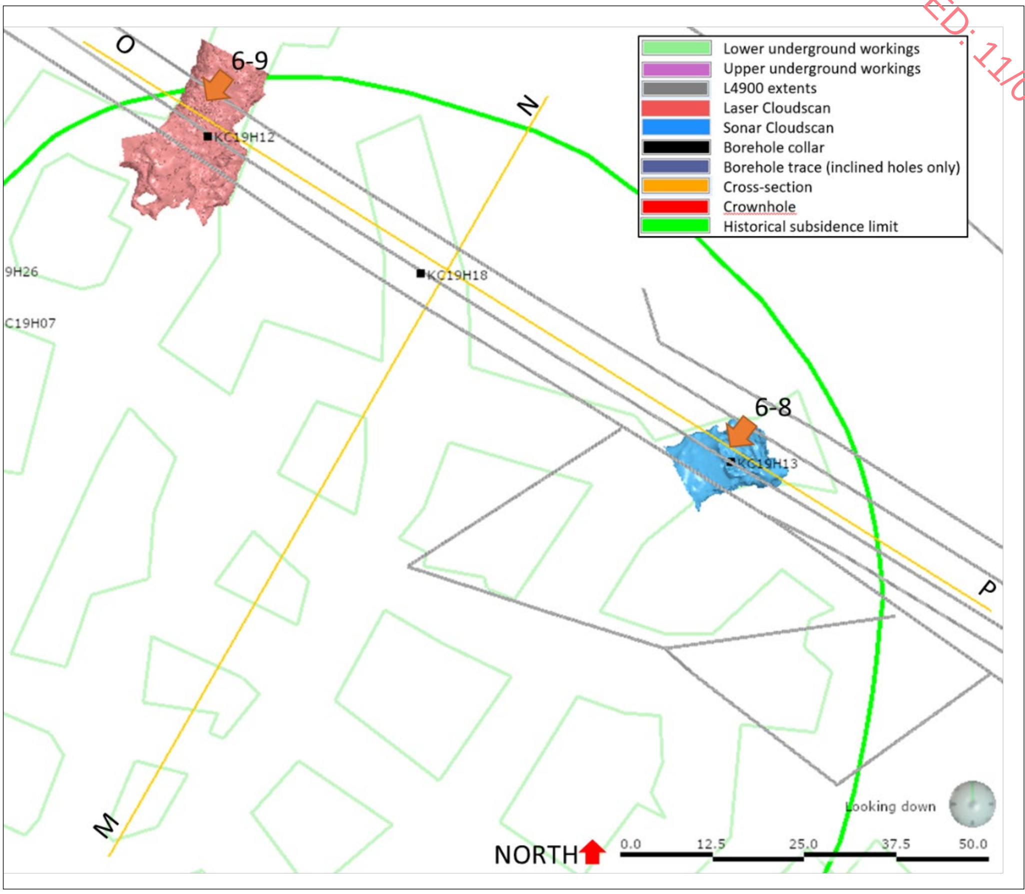


Figure 6-2 shows a plan view of their locations. The arrows and numbers in

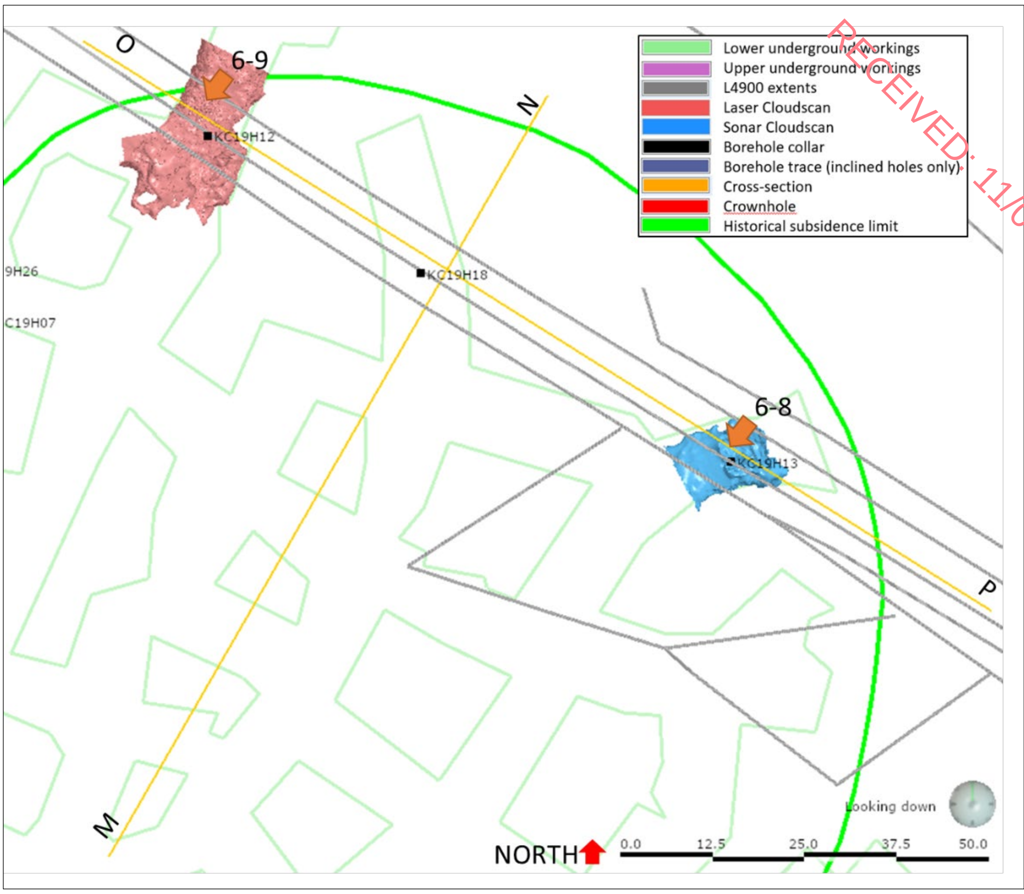


Figure 6-2 indicate the figure number and direction of viewing of the cloudscan images shown later in this section. Figure 6-3 is a cross section along the road showing detail of the boreholes and underground mining.

Table 6-1: Details of boreholes in L4900 Zone A2

BHID	Easting	Northing	Elevation	Max depth (m)	Dip (°)	Azimuth (°)	Cloudscan
KC19 H12	280883.03	300576.75	1048.04	55.1	90	n/a	Yes
KC19 H13	280954.01	300532.63	1044.08	59.6	90	n/a	Yes
KC19 H18	280911.94	300558.17	1046.42	61.8	90	n/a	No

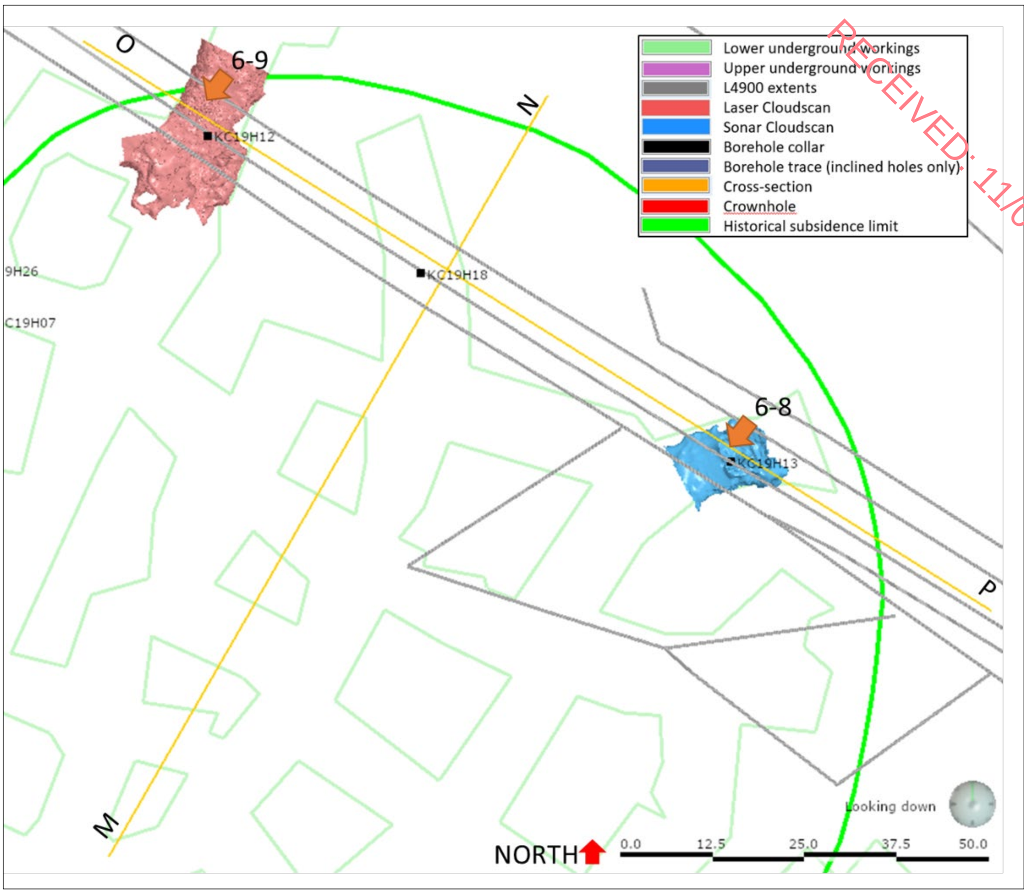


Figure 6-2: Plan view showing borehole locations in L4900 Zone A2

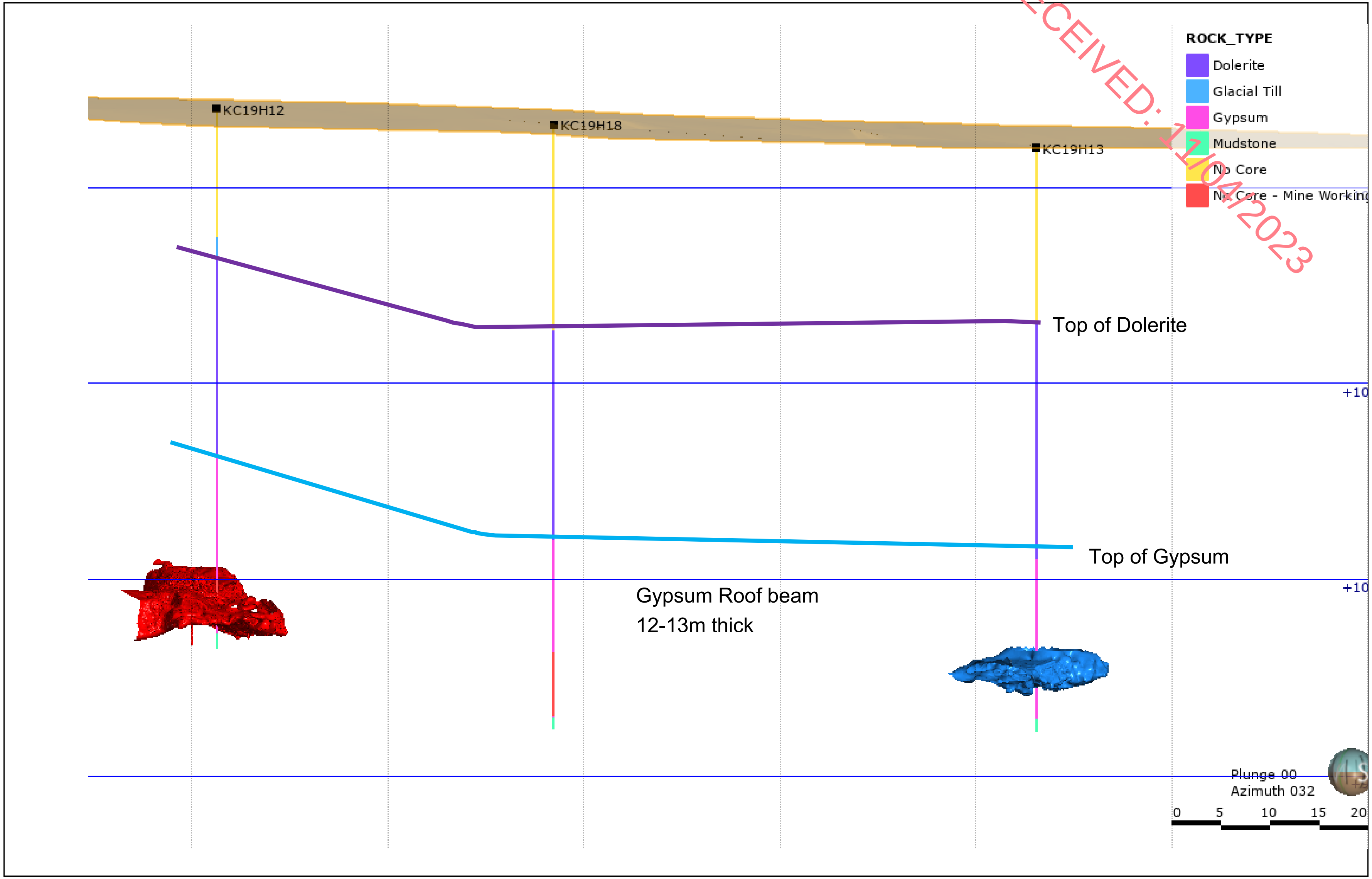


Figure 6-3: Zone A2 – NW-SE Section Along L4900 Road Showing Borehole Geology and Underground Mining

6.3 Finite Element Modelling

6.3.1 Model Geometry

The finite element models analysed are shown in Figure 6-4 and Figure 6-5. Each cross section shows the lithologies, the location of the L4900 road, the mine working, the depth to the mine workings below the road and the thickness of the gypsum roof beam above the workings closest to the L4900 road. Along cross section M-N, which runs through the area of historical subsidence, the depth of the mine workings below the L4900 road is 45m and the gypsum roof beam is 13m thick. There are no laser surveys in this area so the width of the pillars and rooms along with the height and shape of the rooms has been generated from the mine 2D survey. Cross section O-P follows the centre line of the L4900 road. The depth of mining varies between 48m and 61m with the gypsum roof beam thickness between 9m and 19m. Two laser scans are intersected by this cross section. The shape of the rooms as defined by the laser scan reflects the impact of the historical subsidence.

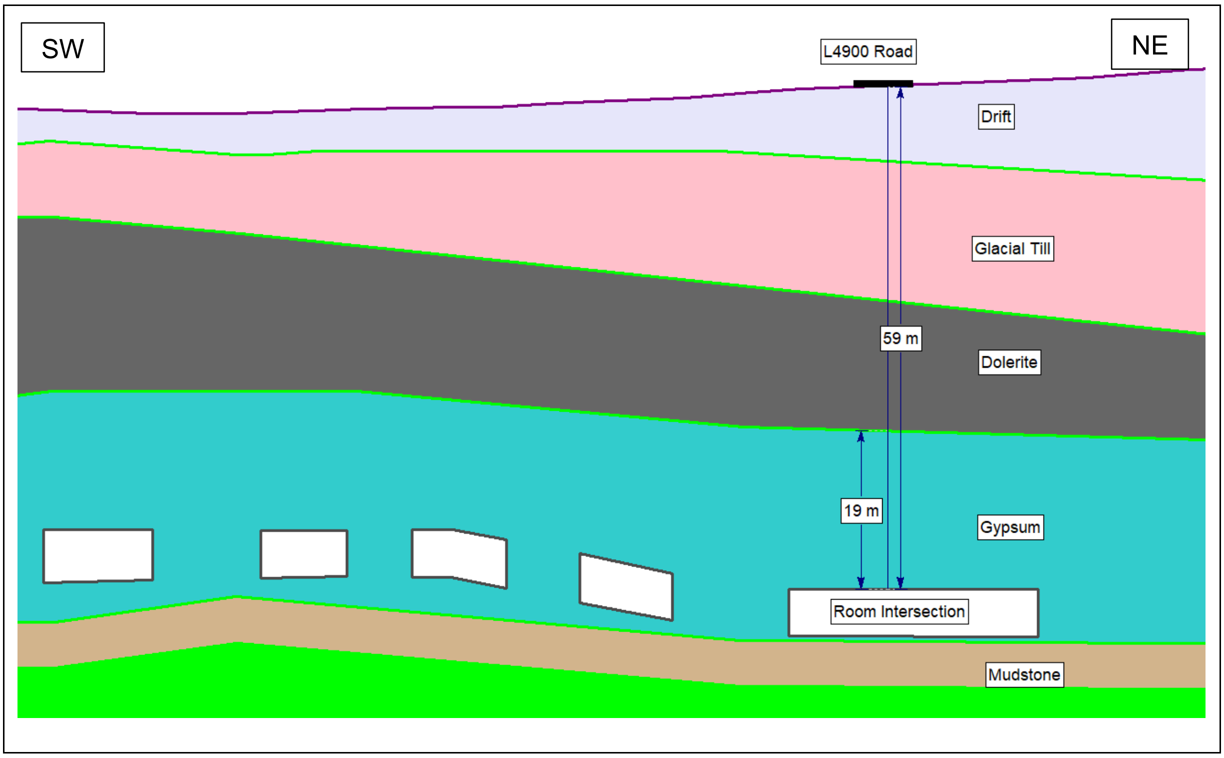


Figure 6-4: Cross Section M-N

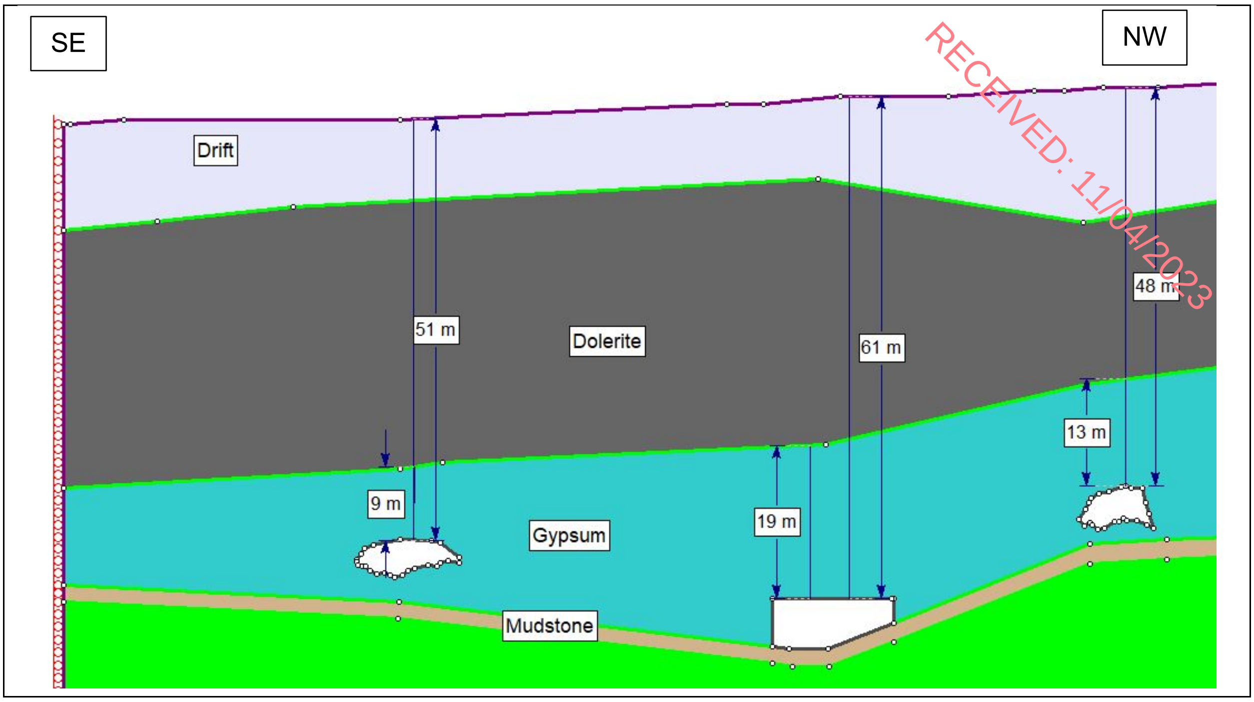


Figure 6-5: Cross Section P-O

6.3.2 Results

The results of the numerical modelling are reproduced below in Figure 6-6 and Figure 6-7. These are contour plots of total displacement in metres. As well as the contours of total displacement the plots show the predicted displacement close to surface below and adjacent to the L4900 road. Also shown is the maximum displacement at the underside of the roof beam above the workings which lie immediately below the road.

For cross section M-N (Figure 6-6) the numerical model defines stable pillar conditions. The maximum roof beam deflection is 0.28m which for the 13m thick gypsum roof beam gives a maximum deflection of 2.1% of roof beam thickness. On surface, below and adjacent to the L4900 road, the maximum predicted surface deformation is 28mm. Note that deformation on this cross section does not simulate the historical subsidence. For this the strength of the gypsum would need to be reduced. There is some indication in the core record for the three boreholes in this zone of reduced RQD within the roof beam area which is likely to be a function of the effect of ground deformation related to the historical subsidence.

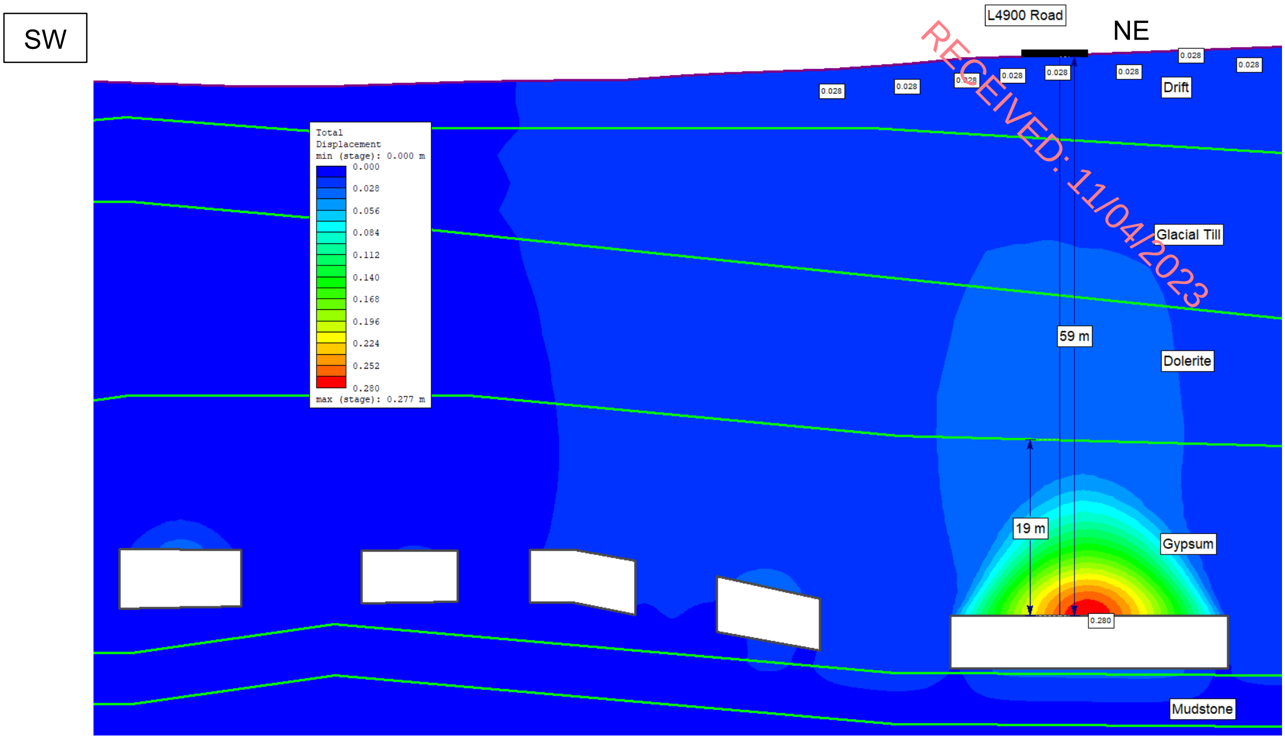


Figure 6-6: Cross Section M-N – Total Displacement

For cross section P-O (Figure 6-7) the numerical model defines stable pillar conditions. The maximum roof beam deflection occurs above the larger central excavation for which there is no laser survey. Here the maximum roof deflection is 0.014m which for the 19m thick gypsum roof beam above the room gives a maximum deflection of 0.07% of roof beam thickness. On surface along the L4900 road, the maximum surface deformation is between 1mm and 5mm.

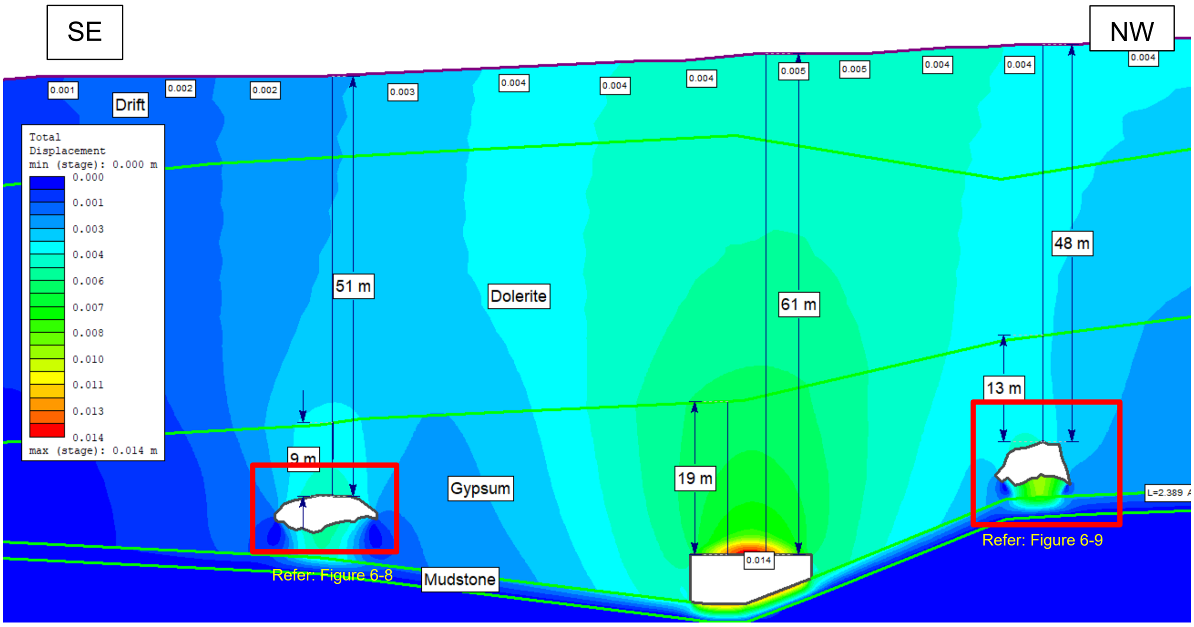


Figure 6-7: Cross Section P-O – Total Displacement

6.4 Cloudscan Surveys

There are no laser surveys along cross section M -N. There are two along cross section P-O. Figure 6-8 shows the 3D survey of the excavation on left-hand side of the cross section. The quality of the scan is not very good as the survey was done using a sonar probe because the excavation was partly water filled. The roof appears to be flat and in good condition. The protrusion is probably the borehole KC19H13. The floor is very irregular in shape and the height of the room is reduced to 3m. This is indicative of floor heave. A combination of pillars bedding down into the underlying mudstone creating floor heave in the rooms has been determined to be the main mechanism for the historical subsidence in this area.

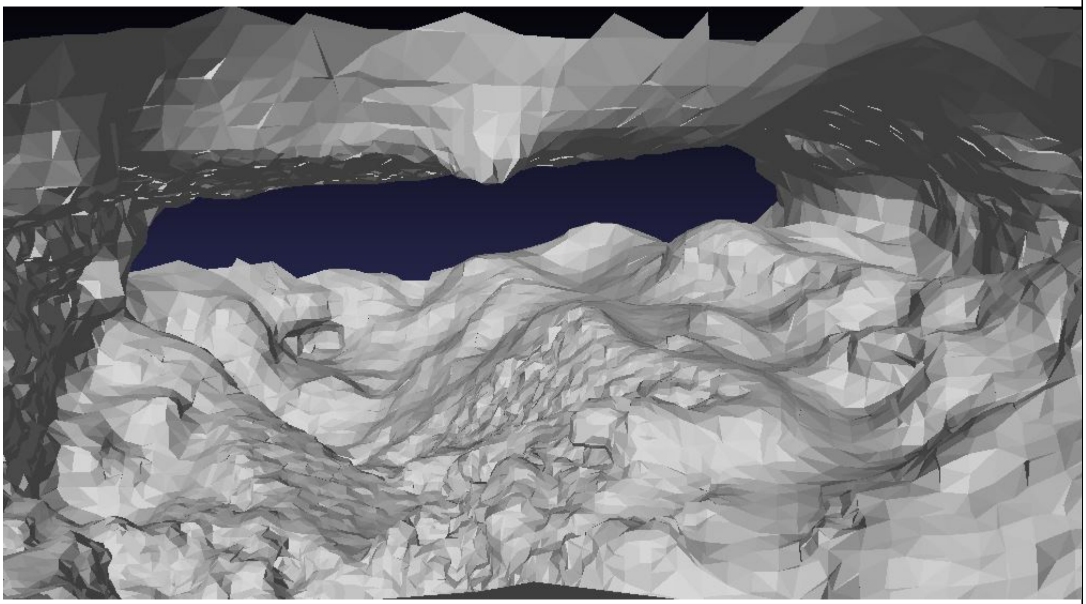


Figure 6-8: Sonar Scan Image of the Left-Hand Room below the L4900 along Section O-P

Figure 6-9 is the laser survey image of the mine workings intersected by borehole KC19H12. The image shows the roof to be in good condition. However, there is clear evidence of floor heave at the base of the excavation.

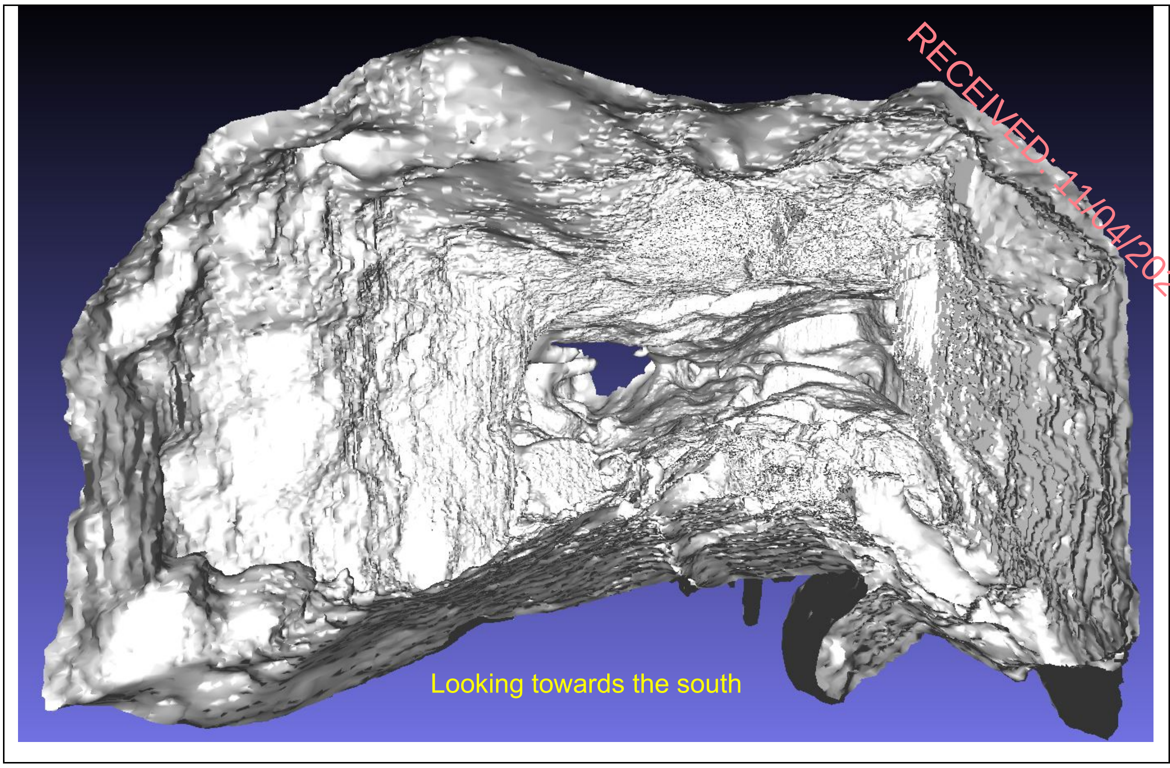


Figure 6-9: Laser Scan Image of the Right-Hand Room below the L4900 along Section O-P

7 L4900 ZONE B3

7.1 General Description of Undermining

Figure 7-1 shows a plan view of Zone B3 as modelled using Leapfrog Geo 3D modelling software. The figure shows both lower and upper underground working surveys, borehole collars and traces, the locations of cross-sections used for later RS2 modelling, the 3D cloudscan data, the extents of the L4900 road and the location of the crownhole.

The length of the undermined road in this zone is approximately 140 m, with a minimum depth of mining of 29 m (below drillhole KC19H11). Only Lower Seam lower horizon mine workings are present below the road in this area. The cloudscans indicate that room height is between 6m and 7m. There are three wide span 4-way intersections beneath the road, two of which have been laser surveyed. Two boreholes have been drilled in this zone and have intersected the Lower Gypsum seam with an average thickness of 20m. The gypsum roof beam thickness above the mine workings is 10m.

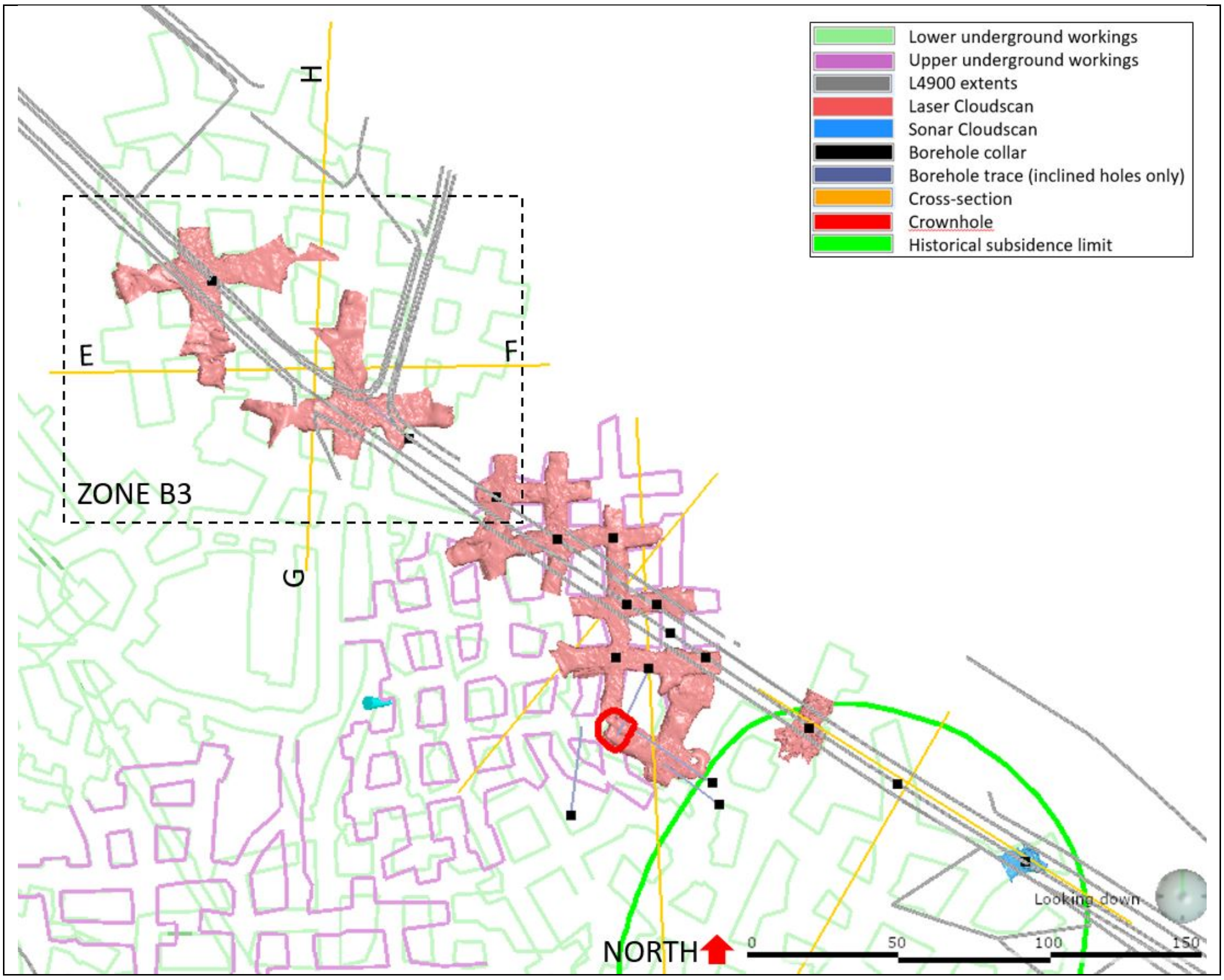


Figure 7-1: Plan view of L4900 Zone B3.

7.2 Summary of Investigation works

Table 7-1 shows details of the boreholes in L4900 Zone A2 and Figure 7-2 shows a plan view of their locations with the extent of the laser surveys. The arrows and numbers in Figure 7-2 indicate the figure number and direction of viewing of the cloudscan images shown later in this section. Figure 7-3 is a cross section along the road showing detail of the boreholes and underground mining.

Table 7-1: Details of boreholes in L4900 Zone B3

BHID	Easting	Northing	Elevation	Max depth (m)	Dip (°)	Azimuth (°)	Cloudscan
KC19 H11	280685.93	300724.09	1048.5	43.2	90	n/a	Yes
KC19 H21	280750.94	300672.47	1048.76	37	60.414	310.881	Yes

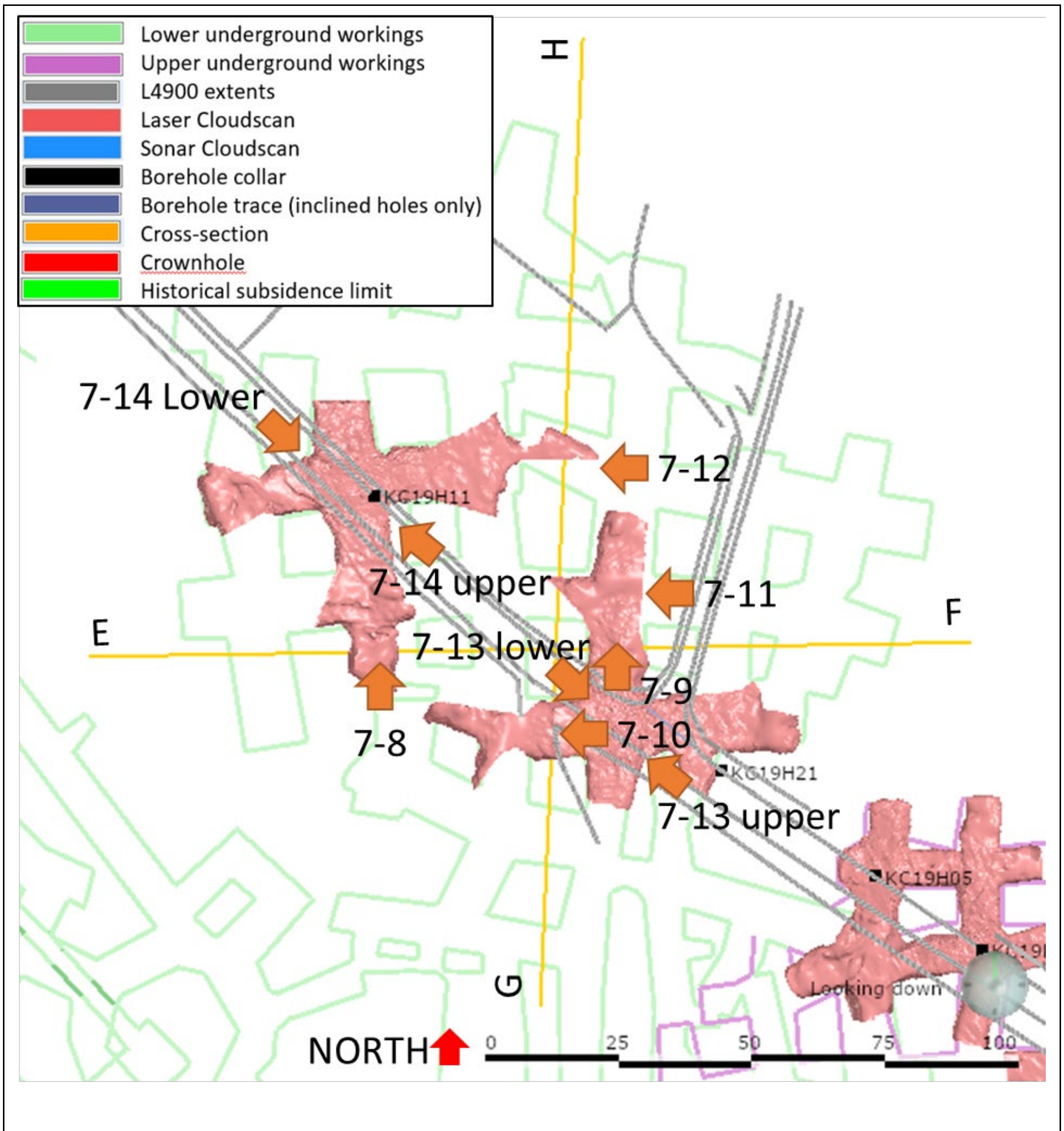


Figure 7-2: Plan view showing borehole locations in L4900 Zone B3

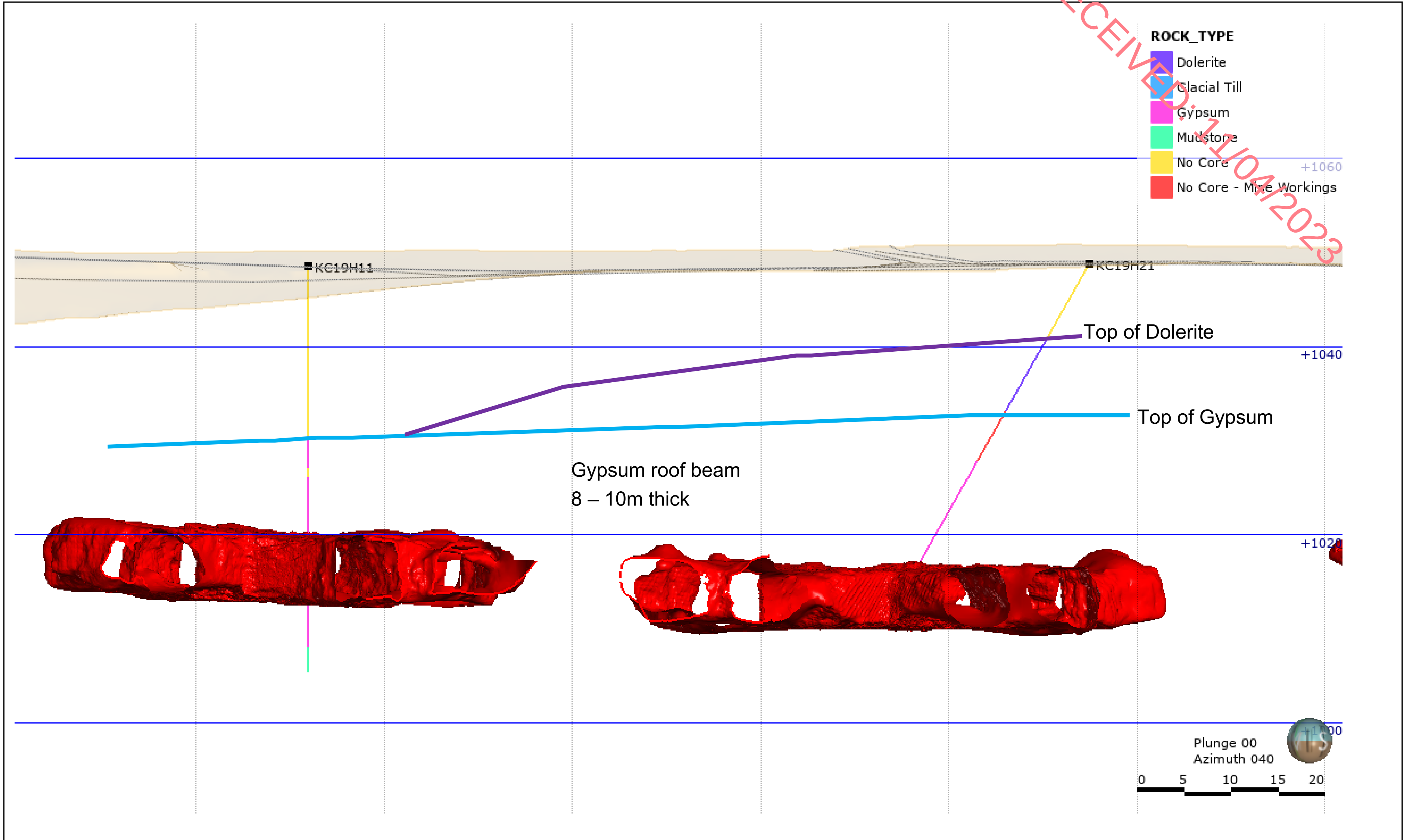


Figure 7-3: Zone B3 – NW-SE Section Along L4900 Road Showing Borehole Geology and Underground Mining

7.3 Finite Element Modelling

7.3.1 Model Geometry

The finite element models analysed are shown in Figure 7-4 and Figure 7-5. Each cross section shows the lithologies, the location of the L4900 road, the mine working, the depth to the mine workings below the road and the thickness of the gypsum roof beam above the workings closest to the L4900 road. There are three 4-way intersections located underneath the road.

Along cross section E-F the depth of the mine workings below the L4900 road is 29m and the gypsum roof beam is 9m thick. Two rooms along the cross section have been defined by laser survey which show that the room height is between 5m and 7m.

Along cross section G-H the depth of the mine workings below the L4900 road is 31m and the gypsum roof beam is 12m thick. Three laser scans are intersected by this cross section.

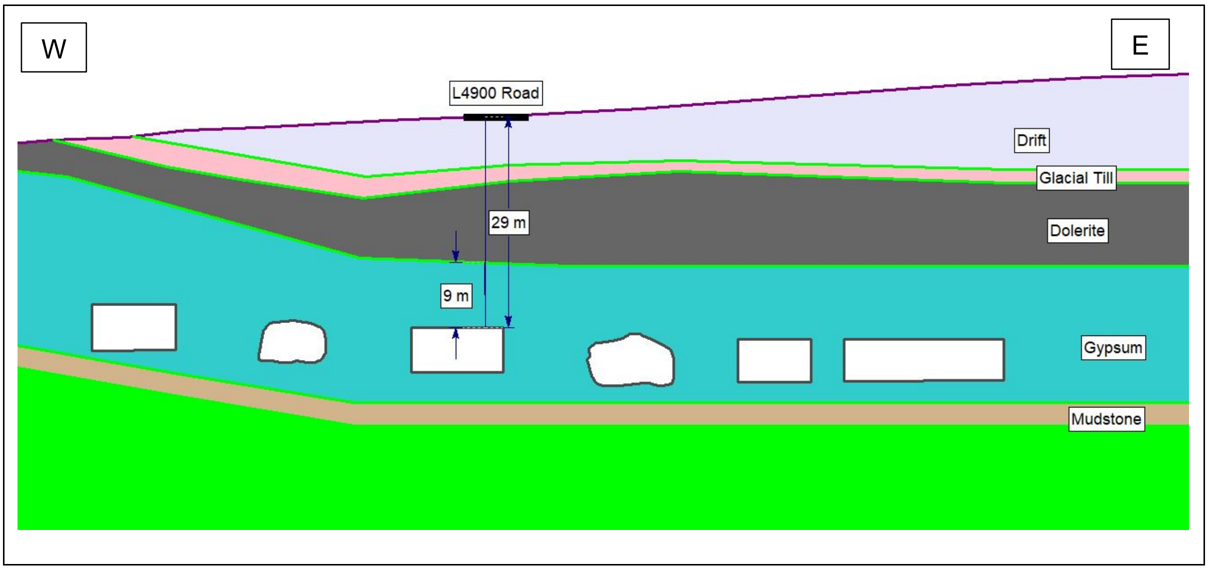


Figure 7-4: Cross Section E-F

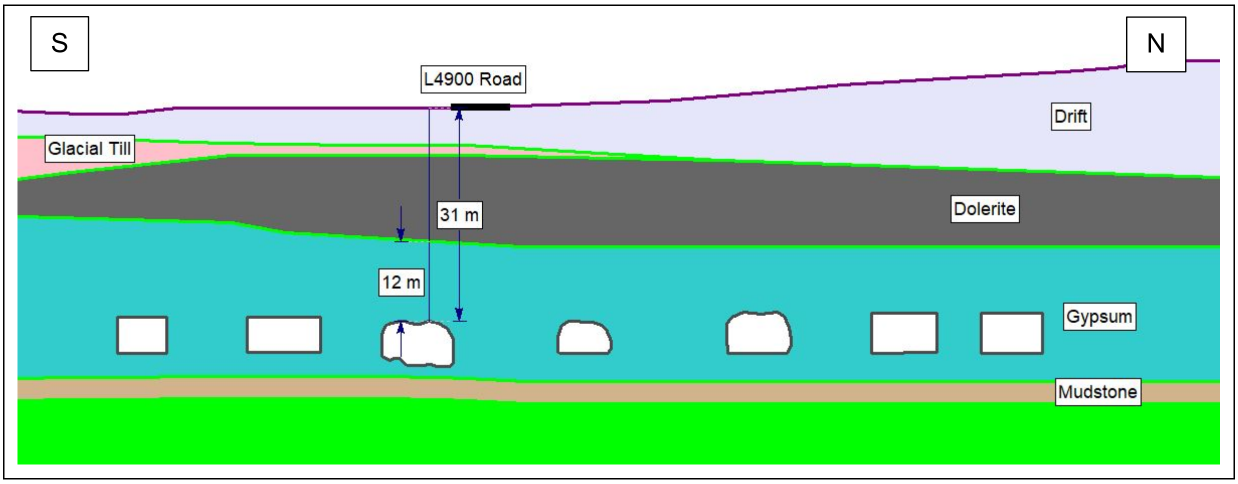


Figure 7-5: Cross Section G-H

7.3.2 Results

The results of the numerical modelling are reproduced below in Figure 7-6 and Figure 7-7. These are contour plots of total displacement in metres. As well as the contours of total displacement the plots show the predicted displacement close to surface below and adjacent to the L4900 road. Also shown is the maximum displacement at the underside of the roof beam above the workings which lie immediately below the road.

For cross section E-F (Figure 7-6) the numerical model defines stable pillar conditions. The maximum roof beam deflection for the room immediately below the L4900 road, for which there is no laser survey, is 0.034m which for the 9m thick gypsum roof beam gives a maximum deflection of 0.38% of roof beam thickness. The room intersection to the east of the road has a roof beam deflection of 0.162m which for a 9m thick roof beam equates to a maximum roof beam deflection of 1.8% of roof beam thickness. On surface, below and adjacent to the L4900 road, the maximum predicted surface deformation is 0mm increasing to 9mm below the room intersection.

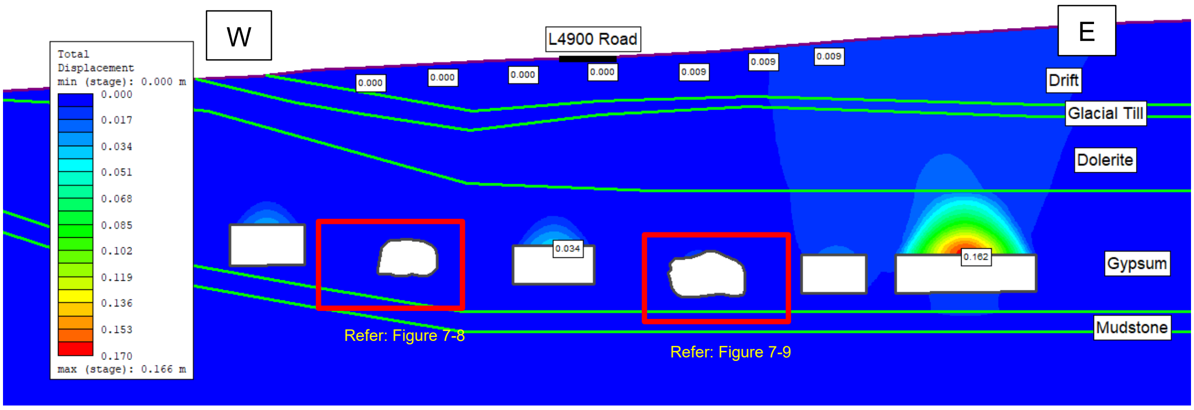


Figure 7-6: Cross Section E-F – Total Displacement

For cross section G-H (Figure 7-7) the numerical model defines stable pillar conditions. The maximum roof beam deflection occurs above the wider non-laser surveyed room to the south of the L4900 road. Here the maximum roof deflection is 0.004m which for the 12m thick gypsum roof beam above the room gives a maximum deflection of 0.03% of roof beam thickness. For the room with the laser surveyed profile below the road the roof beam deflection is 0.004m which for the 12m roof beam above the room gives a maximum roof beam deflection of 0.03% of roof beam thickness. On surface along the L4900 road, the maximum surface deformation is between 2mm and 4mm.

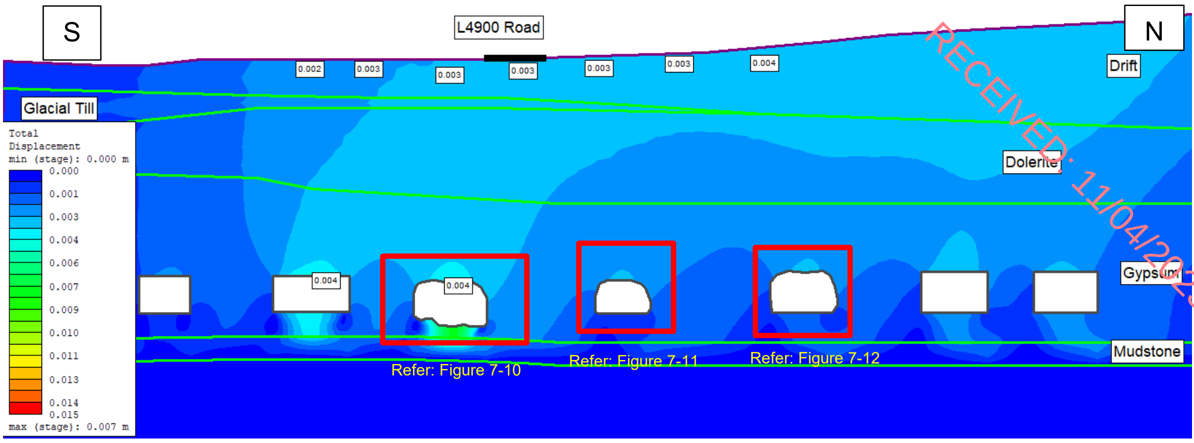


Figure 7-7: Cross Section G-H – Total Displacement

7.4 Cloudscan Surveys

In the cross section Figure 7-6 there are red boxes surrounding rooms whose profiles were drawn from the laser scan surveys. An image from the laser scan of the room on the west is shown in Figure 7-8. An image of the room on the east is shown in Figure 7-9. Both images show rooms which have good stability. There is no evidence of roof beam instability and the floors are generally clean and contain little debris. The odd shape of the end of the room in Figure 7-9 is due to the survey being truncated at an angle because this room was furthest away from the surveying borehole and the end of the room was hidden from the laser survey tool by a pillar.

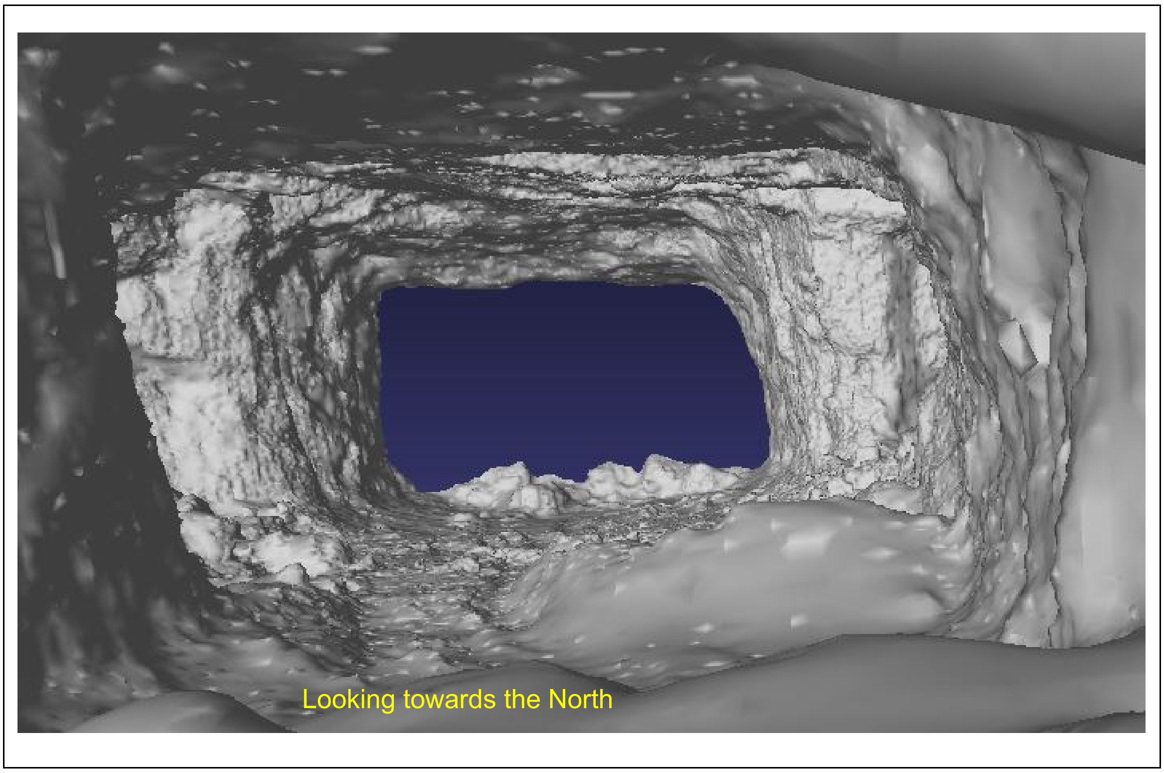


Figure 7-8: Laser Scan Image of the Western Room below the L4900 along Section E-F

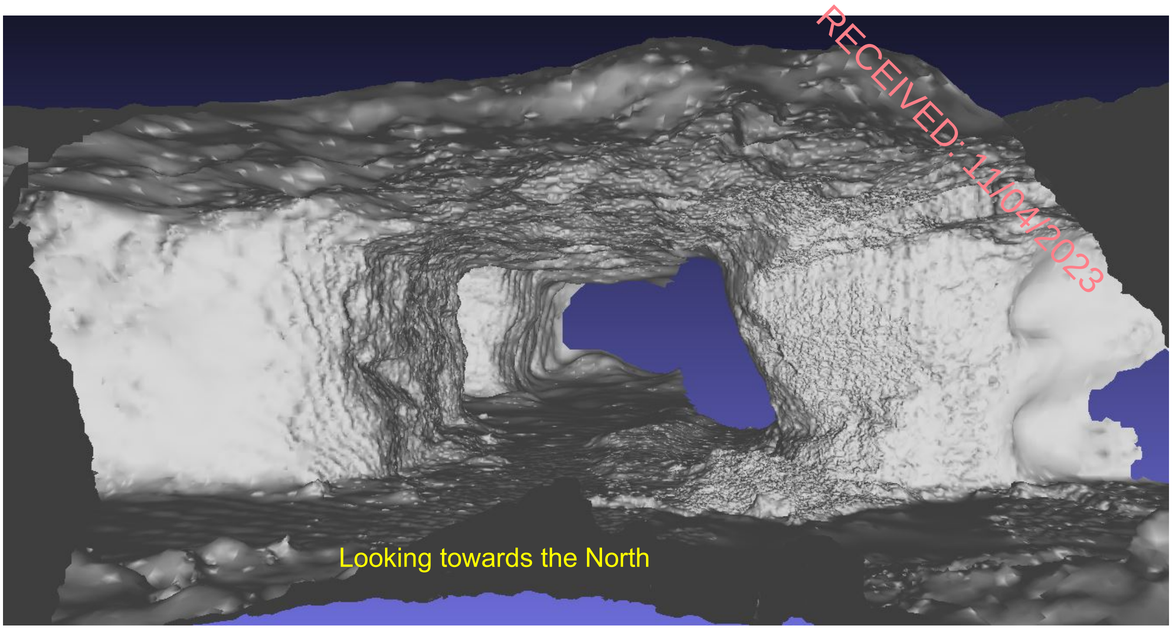


Figure 7-9: Laser Scan Image of the Eastern below the L4900 along Section E-F

Figure 7-10, Figure 7-11 and Figure 7-12 are images from the laser scan surveys of the rooms under the L4900 road along cross section E-F. These images are at the limit of the laser survey tool sight line, so the image quality and detail are not as good as the those shown in previous sections. However, all images show flat, clean stable roof beams. In Figure 7-10 the toe of a muck pile can be seen at the entrance to a room on the left. Because of the angular appearance of the muck pile surface this is probably waste material that has been pushed into the room.

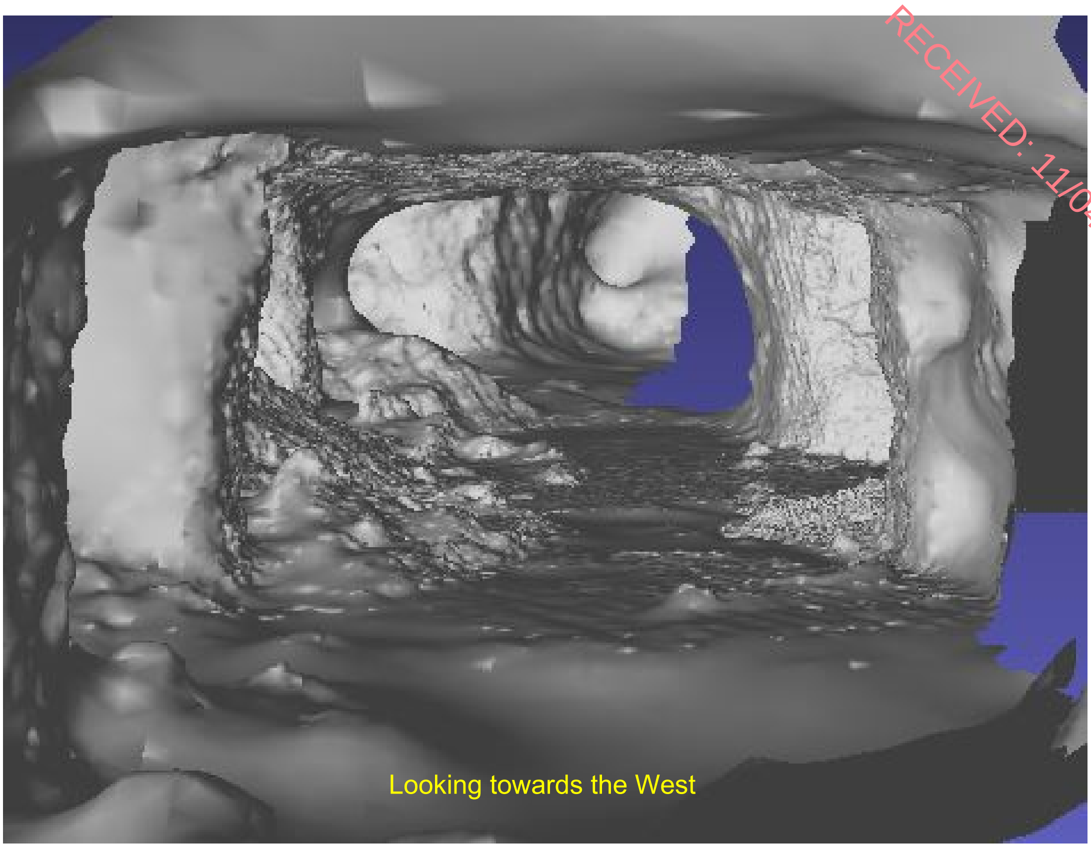


Figure 7-10: Laser Scan Image of the Southern Room below the L4900 along Section G-H

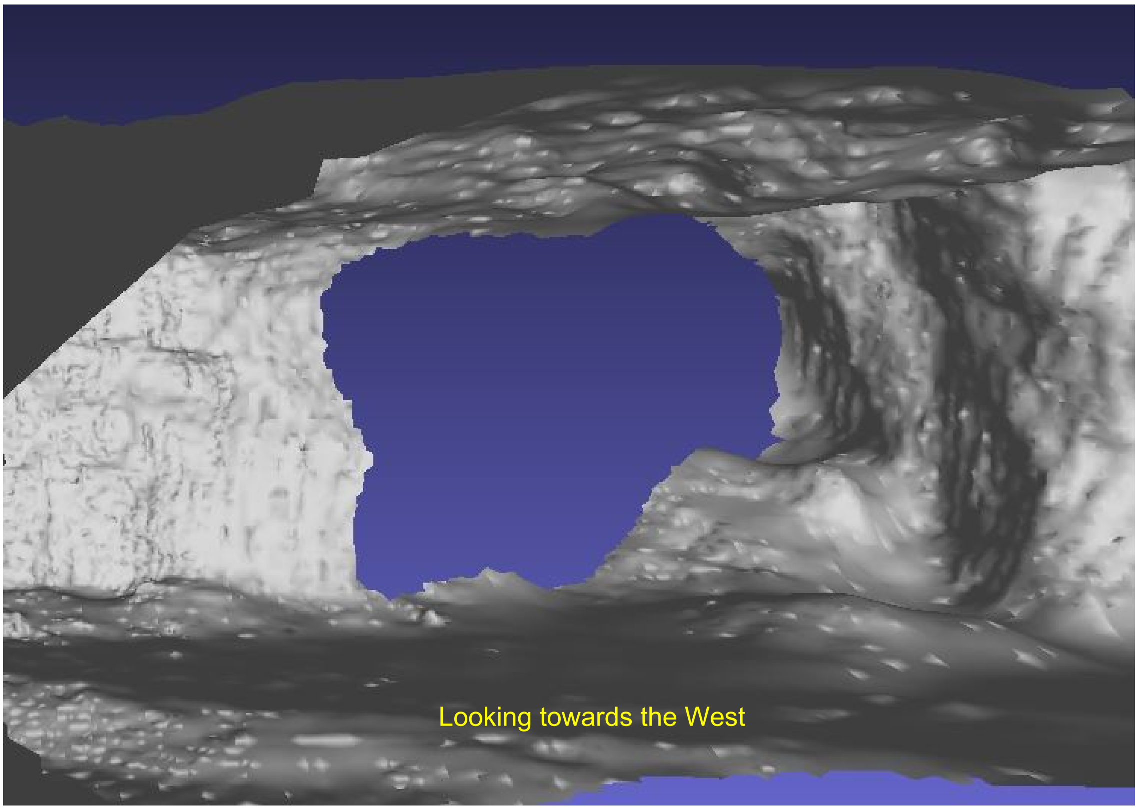


Figure 7-11: Laser Scan Image of the Central Room below the L4900 along Section G-H

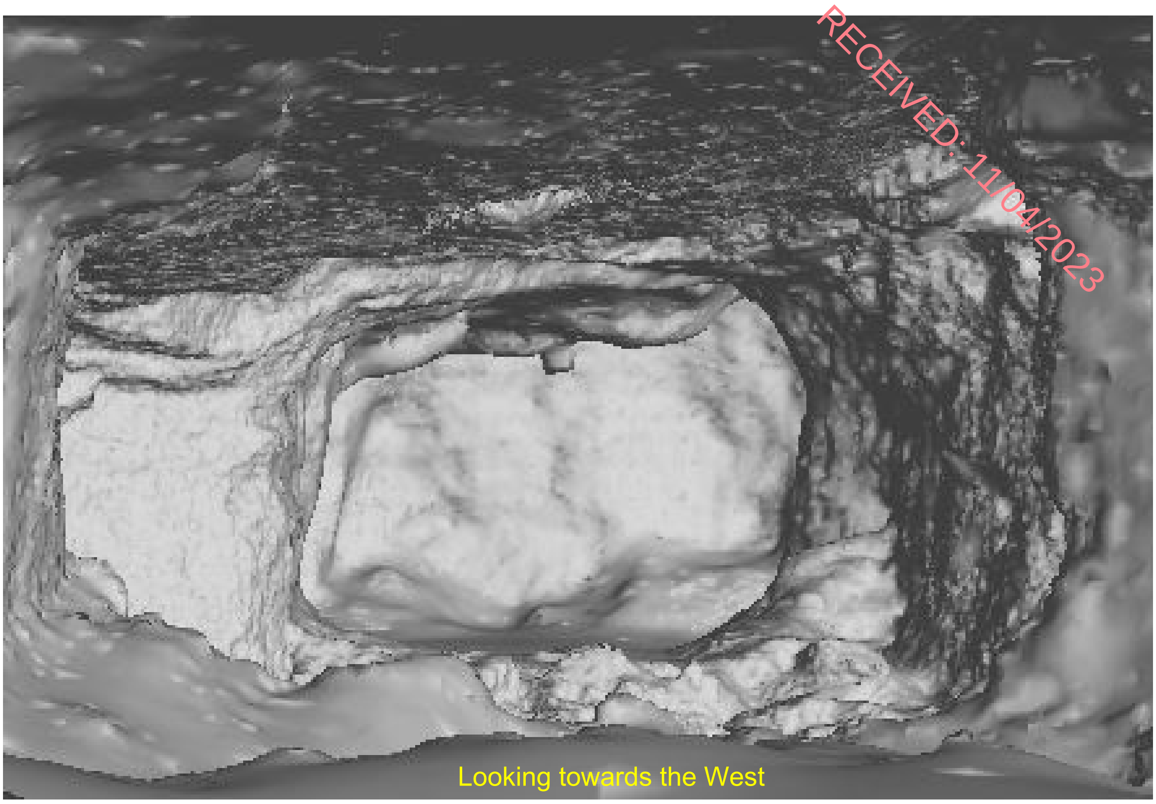
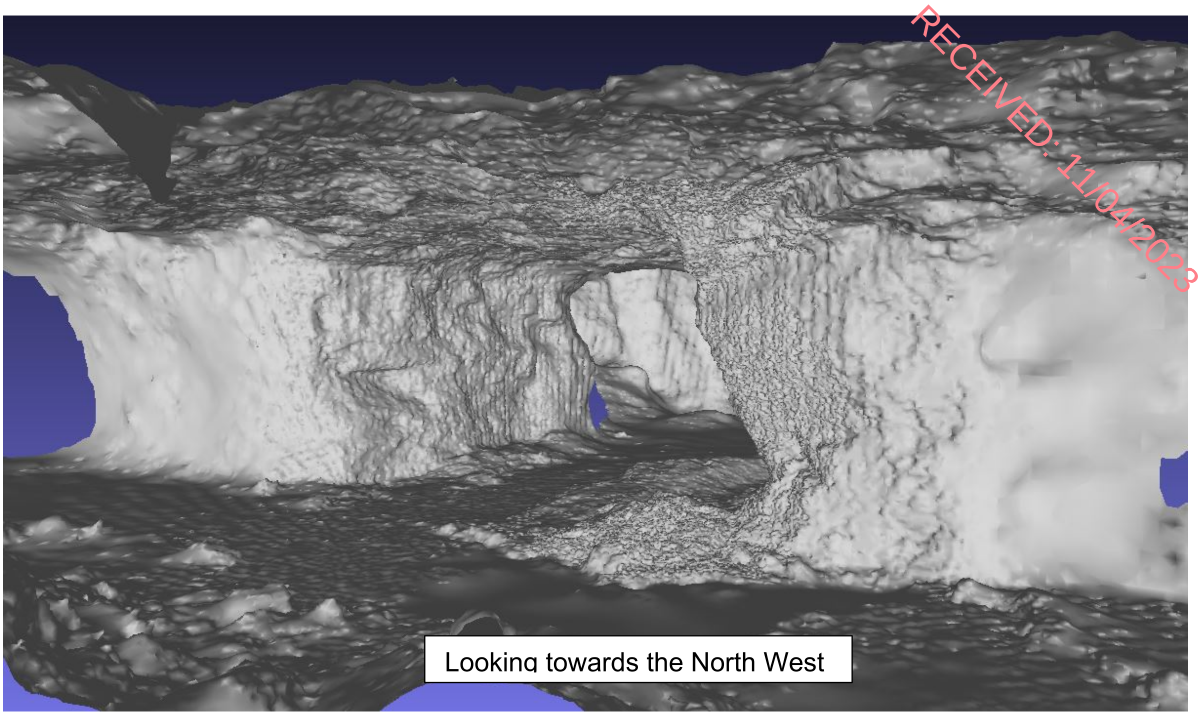


Figure 7-12: Laser Scan Image of the Northern Room below the L4900 along Section G-H

Although not captured on the finite element cross sections this area of undermining contains the widest four-way intersections below the road. Two of the intersections have been captured by the laser surveys from boreholes KC19H21 and KC19H11. The third, between the two boreholes has been masked by pillars obstructing the laser survey tool line of sight. Laser scan images of the two surveyed four-way intersections are shown in Figure 7-13 and Figure 7-14. Because one of these images are viewed from the outside of the laser scan with the viewing position being ‘in the rock’ these have been rendered as ‘x-ray’ images to allow the viewer to see into the rooms.

The four-way intersection surveyed all show very good roof condition with no evidence of roof slabbing or failure of any nature. There is some rock debris on the floor of the KC19H11 intersection. Some of this is likely to be related to drilling of this hole. The rest appears to be material pushed up by a front-end loader during floor clean up.



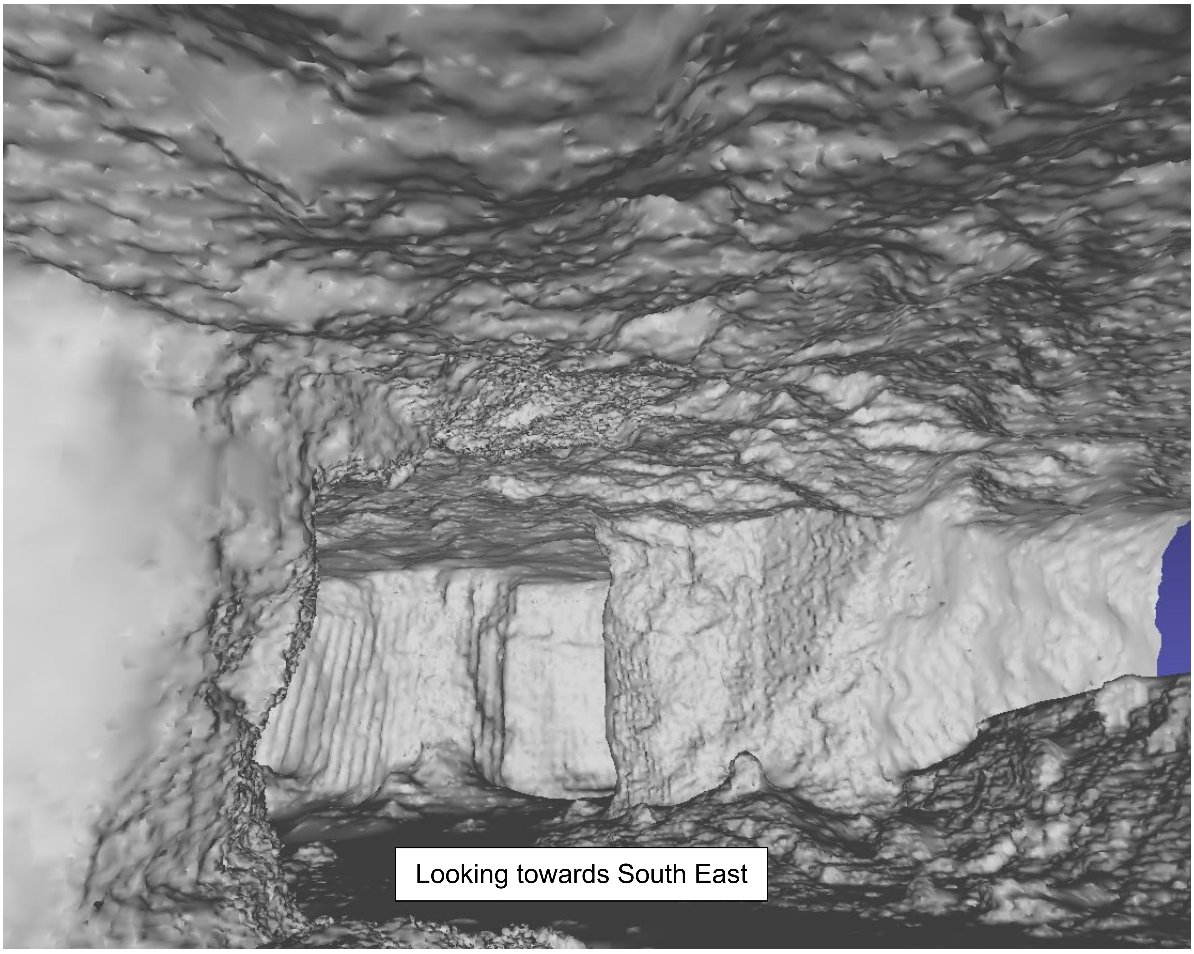


Figure 7-13: Laser Scan Image of the 4-Way Intersection NW of KC19H21

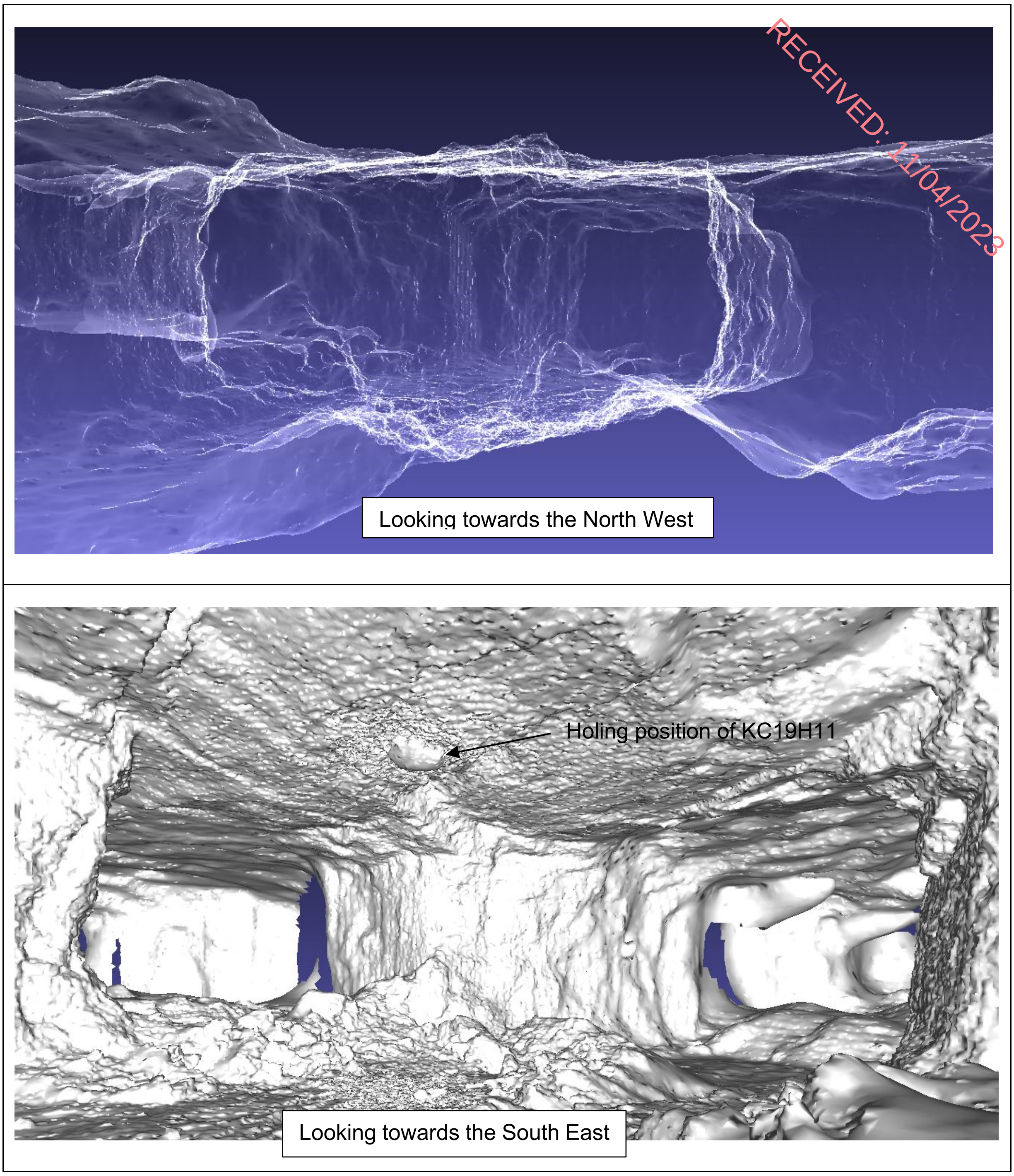


Figure 7-14: Laser scan Image of the 4-Way Intersection NW of KC19H21

8 L4900 ZONE B4

8.1 General Description of Undermining

Figure 8-1 shows a plan view of Zone B4 as modelled using Leapfrog Geo 3D modelling software. The figure shows both lower and upper underground working surveys, borehole collars and traces, the locations of cross-sections used for later RS2 modelling, the 3D cloudscan data, the extents of the L4900 road and the location of the crownhole.

The length of the undermined road in this zone is approximately 135 m, with a minimum depth of mining of 19 m where the access decline into the mine is located just on the south western edge of the road (below drillhole KC19H22). At the western end of the zone below KC19H16 the mining depth is greater at about 44m below surface. Only Lower Seam lower horizon mine workings are present below the road in this area. The cloudscans indicate that room height is about 6m. There are three wide span 4-way intersections beneath the road all of which have been laser surveyed. Six boreholes have been drilled in this zone and have intersected the Lower Gypsum seam with an average thickness of 20m. The average gypsum roof beam thickness above the mine workings is 7m but varies between 4m and 13m.

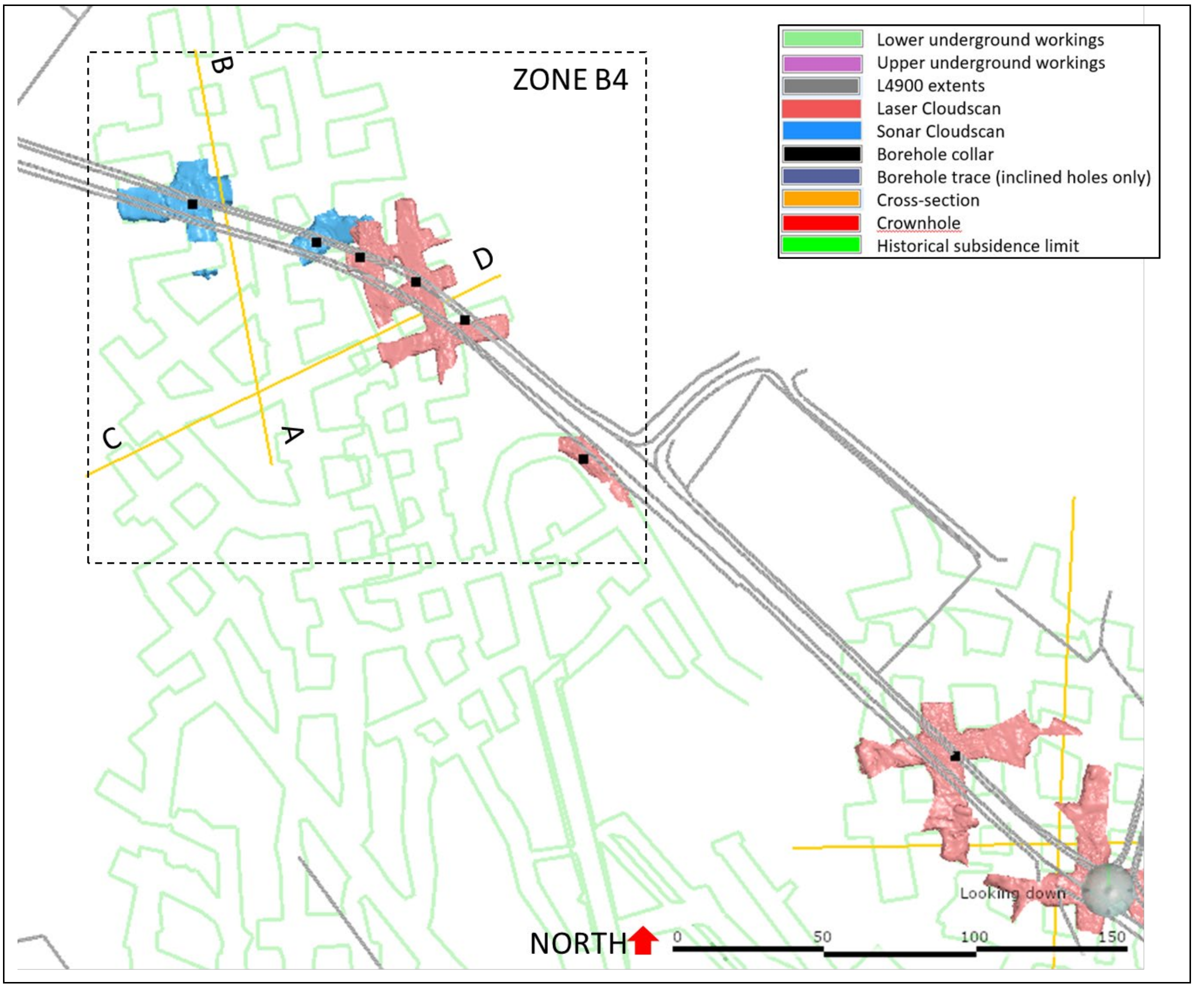


Figure 8-1: Plan view of L4900 Zone B3.

8.2 Summary of Investigation works

Table 8-1 shows details of the boreholes in L4900 Zone A2 and Figure 8-2 shows a plan view of their locations with the extent of the laser surveys. The arrows and numbers in Figure 8-2 indicate the figure number and direction of viewing of the cloudscan images shown later in this section. Figure 8-3 is a cross section along the road showing detail of the boreholes and underground mining.

Table 8-1: Details of boreholes in L4900 Zone B3

BHID	Easting	Northing	Elevation	Max depth (m)	Dip (°)	Azimuth (°)	Cloudscan
KC19 H10	280489.91	300888.19	1056.5	53.5	90	n/a	Yes
KC19 H14	280475.33	300893.36	1055.92	56.5	90	n/a	Yes
KC19 H15	280508.21	300880.26	1056.57	47.6	90	n/a	Yes
KC19 H16	280434.92	300905.75	1053.65	59.5	90	n/a	Yes
KC19 H20	280524.67	300867.5	1056.43	44.7	90	n/a	Yes
KC19 H22	280563.27	300821.71	1053.39	35.5	90	n/a	Yes

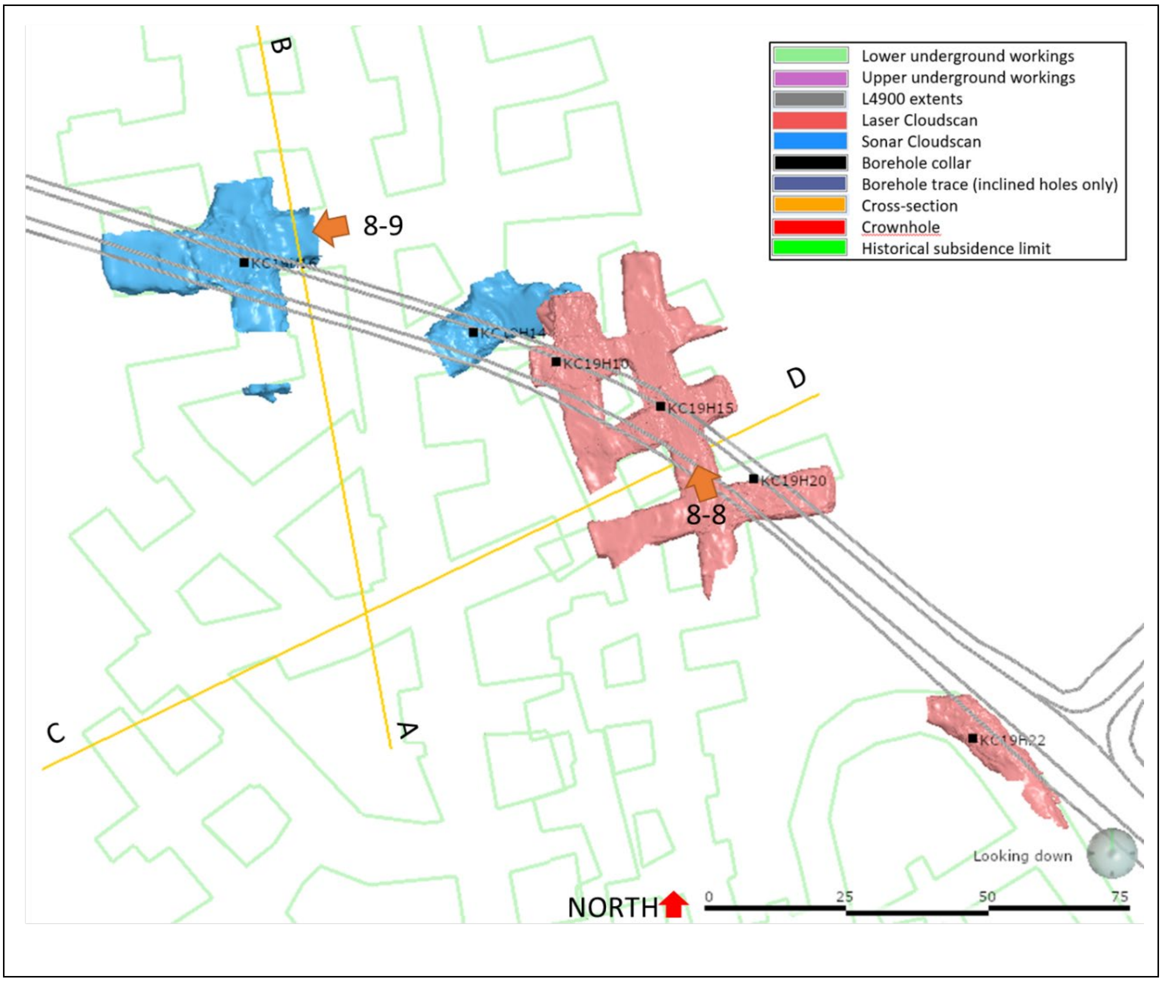


Figure 8-2: Plan view showing borehole locations in L4900 Zone B3

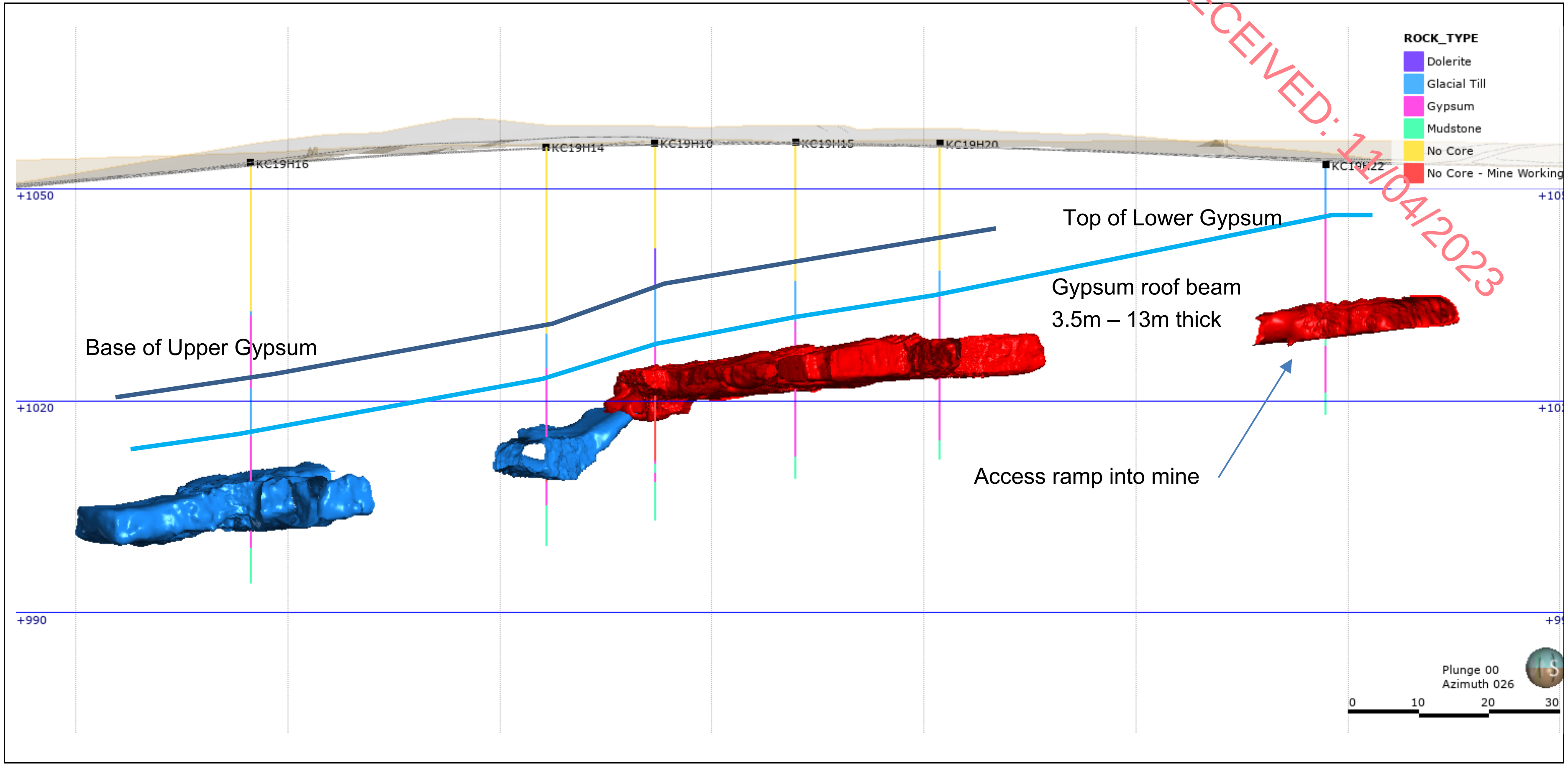


Figure 8-3: Zone B4 – W-E Section Along L4900 Road Showing Borehole Geology and Underground Mining

8.3 Finite Element Modelling

8.3.1 Model Geometry

The finite element models analysed are shown in Figure 8-4 and Figure 8-5. Each cross section shows the lithologies, the location of the L4900 road, the mine working, the depth to the mine workings below the road and the thickness of the gypsum roof beam above the workings closest to the L4900 road.

Along cross section C-D the depth of the mine workings below the L4900 road is 28m and the gypsum roof beam is 5m thick. The height of the mine workings is 6m.

Along cross section A-B the depth of the mine workings below the L4900 road is 43m with the gypsum roof beam thickness of 7m. The height of the mine workings is 6m.

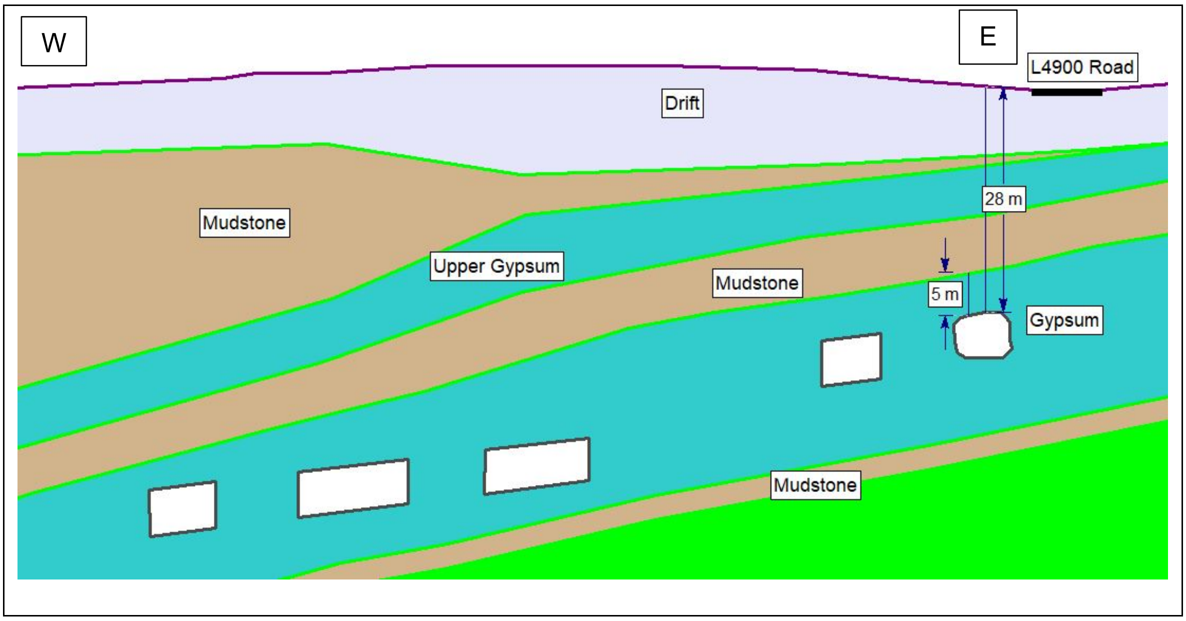


Figure 8-4: Cross Section C-D

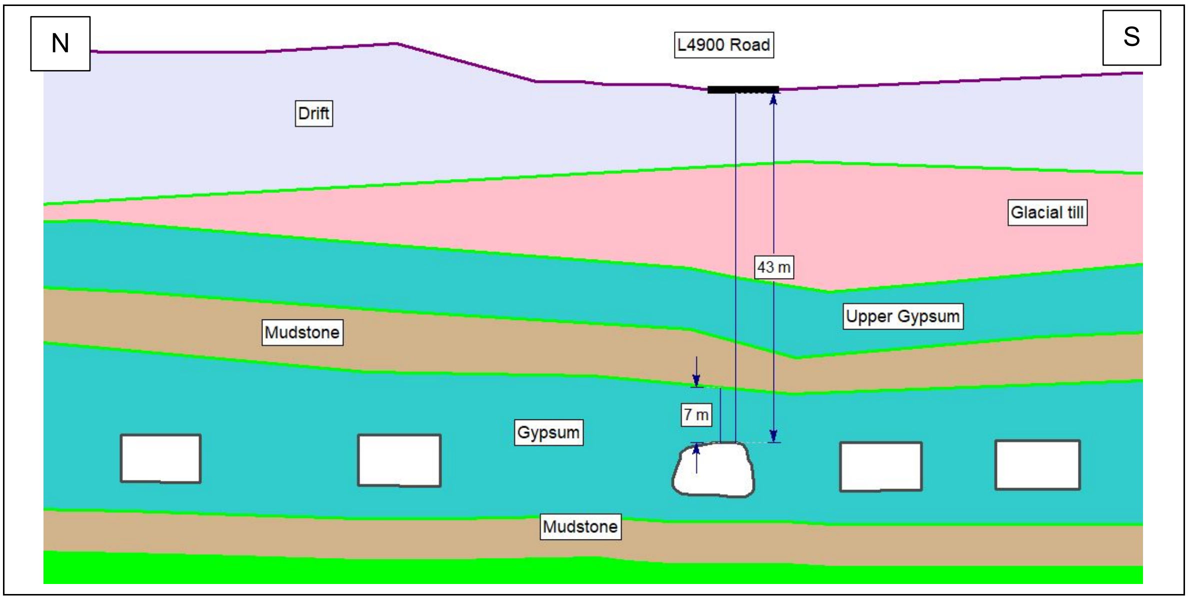


Figure 8-5: Cross Section A-B

8.3.2 Results

The results of the numerical modelling are reproduced below in Figure 6-6 and Figure 6-7. These are contour plots of total displacement in metres. As well as the contours of total displacement the plots show the predicted displacement close to surface below and adjacent to the L4900 road. Also shown is the maximum displacement at the underside of the roof beam above the workings which lie immediately below the road.

For cross section C-D (Figure 8-6) the numerical model defines stable pillar conditions. There is no modelled deformation above the rooms directly below the L4900 road. Roof deformation is restricted to the deeper, wider span rooms to the west of the road. Maximum beam deflection is 0.133m. The roof beam thickness here is about 8m and the maximum roof beam deflection is about 1.7% of roof beam thickness.

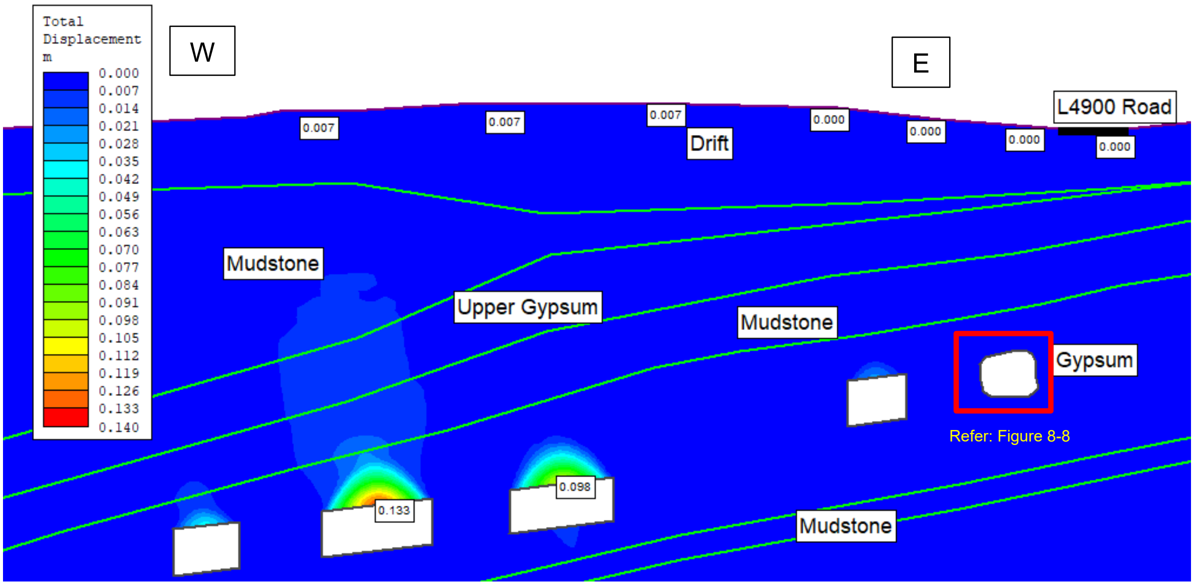


Figure 8-6: Cross Section C-D – Total Displacement

For cross section B-A (Figure 8-7) the numerical model defines stable pillar conditions. The maximum roof beam deflection occurs above the assumed flat roofed rooms where there is no laser survey. The maximum roof deflection in the room below the road is 0.004m which for the 6m thick gypsum roof beam above the room gives a maximum deflection of 0.07% of roof beam thickness. Modelled surface deformation below the road is 4mm.

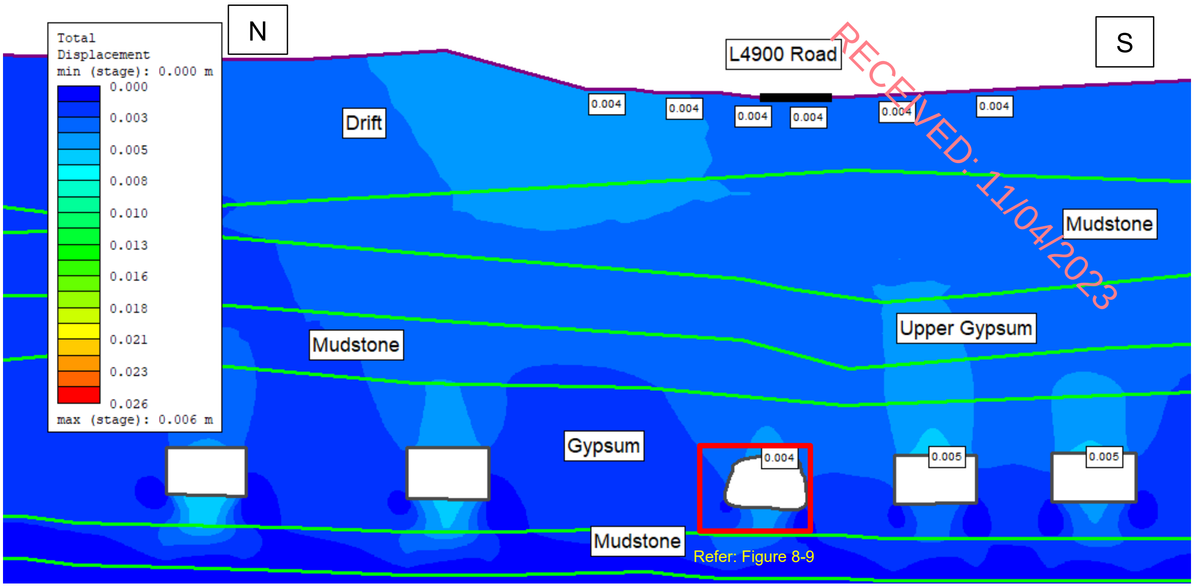


Figure 8-7: Cross Section B-A – Total Displacement

8.4 Cloudscan Surveys

One room only on each cross-section line was captured by the laser scanning survey. These are identified by the red boxes in Figure 8-6 and Figure 8-7.

Figure 8-8 shows the condition of the room under the L4900 road on cross section C-D. The roof has formed along shallow dipping bedding planes. There is no evidence of roof instability. The excavation floor is clean. The mushroom shaped feature has probably been formed from the laser beam reflecting back from water droplets dripping through borehole KC19H15.

Figure 8-9 shows the condition of the room under the L4900 road on cross section B-A. This survey was carried out using sonar rather than laser as the floor of the mine in this area was partially flooded. Sonar surveying does not provide the same level of detail as laser surveying. However, the roof appears to be in good condition with no evidence of instability or slabbing.

8.5 Access Ramp into Mine

This zone contains the access ramp into the mine. The ramp underlies the south western edge of the L4900. From borehole KC19H22, which intersected the ramp the roof is about 19m below the road and is overlain by almost 13m of competent gypsum. The access ramp is 6.5m wide by 5.0m high and has an arched roof. The arched roof profile improves stability for underground tunnels where long term access into a mine is required. The cloudscan survey indicates stable condition within the access ramp. Because of the limited size of the tunnel and the fact that there are no rooms and pillars in the vicinity it is considered that the presence of the access ramp 19m below the edge of the L4900 road does not pose a risk to the stability of the road.

8.6 12.5m High Mine Workings

Borehole KC19H10 intersected a 12.5m high mine working void overlain by 4.5m of gypsum roof beam. The cloudscan survey indicates stable roof conditions. Given that roof beam stability is a function of room width and not room height, the cloudscan survey indicates stable roof conditions and the Lower Gypsum is overlain by Upper Gypsum in this area the risk of potential instability of this area affecting the L4900 road is considered to be extremely low.

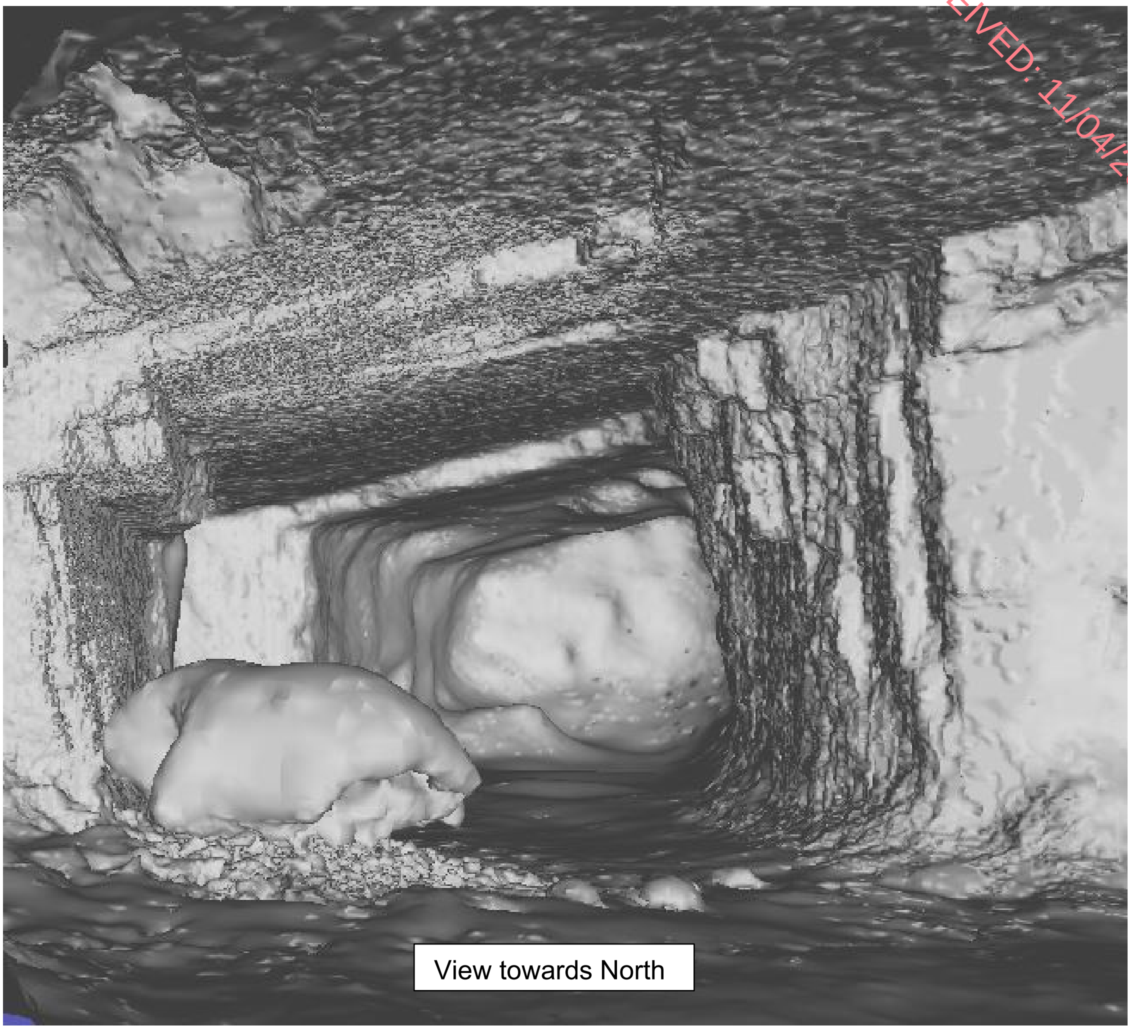


Figure 8-8: Laser Scan Image of Room below the L4900 along Section C-D

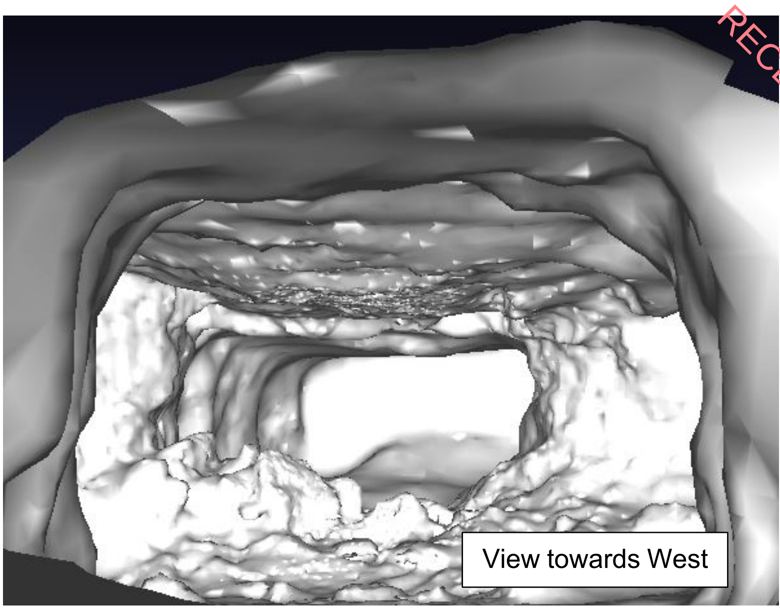


Figure 8-9: Sonar Scan Image of Room below the L4900 along Section B-A

9 CONCLUSIONS

9.1 Cause of Crownhole Development

The borehole drilling and laser scan surveys have indicated that the crownhole was formed as a result of localised thinning and breach of the gypsum roof beam above a section of the Lower Seam upper horizon workings. It is likely that the extent of the breach is very localised as the laser scan surveys show very good stable underground mine conditions immediately next to the area of the mine inflow.

The sandy weathered dolerite flowed into the mine workings and undercut the overlying glacial till. The glacial till is a cohesive material so collapse of this into the underlying void in the dolerite would likely have been slow. It is likely that the concrete slab observed near surface, which formed a roadway or laydown area for crushed gypsum stockpiles during the time that Drumgoosat was active, might have supported the ground surface overlying the crownhole until the crownhole void exceeded a critical width. The concrete slab then failed resulting in the appearance of the crownhole at surface. If this was the case, then the crownhole had probably developed over several months before it manifested itself on surface.

Because of the above SRK is of the opinion that the crownhole development is unrelated to the September 2018 subsidence event because the location of the event at about 380m distant from the crownhole is too far away to have had any influence on the mine in the vicinity of the crownhole

The requirements for crownhole development can be summarised as follows:

1. Shallow mining depth, less than 50m
2. Presence of dolerite and glacial till which extend to surface immediately above the underground workings. For the avoidance of doubt any material with similar characteristics to dolerite and glacial till should be considered to behave in the same manner.
3. A very thin gypsum roof beam, typically less than 1 m.

If at least one of these factors is not present the likelihood of a crownhole developing above the mine is almost impossible.

9.2 Possibility of Future Crownhole Development Affecting the L4900 Road

Because of the conditions of the underground workings and the thickness of gypsum roof beam identified in the boreholes drilled for this investigation work along with the history of development of crownholes above Drumgoosat SRK considers that the risk of future crownhole development in the vicinity of the L4900 road is very low. However, because of the localised nature of conditions that lead to crownhole development there is still some risk that crownholes could occur above the workings that undermine the L4900 road. There are however ways to minimise the impact of the risk of crownhole development and these are described in Section 10

9.3 General Geotechnical Conditions of the Gypsum Roof Beam

The general geotechnical condition of the gypsum roof beams intersected in the 23 boreholes drilled along and adjacent to the L4900 road indicate by the GSI number range to be a fair quality rock mass. The gypsum contains closely to moderately spaced bedding planes and widely spaced sub-vertical orthogonal joint planes. The gypsum has an intact rock strength in the range 15-30MPa and a geological strength index (GSI) in the range 50-60. Weighted average values of these parameters have been used to characterise the rock mass strength of the gypsum for finite element modelling and stability assessment.

The strength and GSI values measured from the borehole core are consistent with the values generated by SRK during its underground mapping campaigns carried out between 1999 and 2005 and with the 'expected' strength conditions used in its predictive analyses of mine stability below the R179 and L4900 reported in October 2018. This validates the results of the previous predictive analyses and also indicates that there has been no degradation of the gypsum strength over the last 20 years.

Apart from in the immediate vicinity of the crownhole the gypsum roof beam thickness intersected by the boreholes ranged from a minimum of 3.64m intersected in borehole KC19H15 which was drilled in Zone B4 to a maximum of 12.7m in borehole KC19H22 also drilled in Zone B4. The latter hole was drilled to intersect the access decline into the mine rather than the underground rooms.

9.4 General Stability Condition of Underground Workings

For a mine which is around 40 years old the condition of the underground rooms, as determined from the borehole laser surveys, is generally very good. There is little evidence of roof beam failure. The room floors are generally clean.

Details of the stability condition for the underground working modelled are presented in the previous sections. An assessment of all areas scanned by the borehole laser and sonar surveys has been carried out. Details are presented in Appendix E No areas of roof instability were noted in any of the cloudscans.

9.5 L4900 Zone A1

The roof beam thickness below the road in this area, as determined from the borehole drilling, ranges from a minimum of 5 m to a maximum of 8.5m. The areas modelled and inspected from the laser surveys indicates stable mine conditions. Modelled surface deformations are consistent with those being measured on surface by Gyproc's on-going surface survey measuring programme. The mine workings are 30m below surface and the lower gypsum is overlain by a thick dolerite unit.

This zone contains two of the three criteria for crownhole development, depth of mining and overlying dolerite and glacial till/drift, therefore the potential for any future crownhole development is considered to be very low.

9.6 L4900 Zone A2

This area is located in the area of historical subsidence. The road is only undermined by single rooms in three places. The areas modelled indicates stable mine conditions with a very low risk of mine instability. Modelled surface deformations are consistent with those being measured on surface by Gyproc's on-going surface survey measuring programme.

Two of the rooms have been surveyed, one by laser the other by sonar. The scans show the impact that the deformation of these rooms, floor heave and pillar damage, has on the historical subsidence on surface. Because of the floor heave the height of the workings has been reduced from 6m to about 3m. It is of interest to note that although these rooms are in an area of subsidence the roof of the workings is in good condition. This confirms that the interpretation of the subsidence was by punching of pillars through the floor into the underlying basal mudstone.

In this area the boreholes have intersected a gypsum roof beam with a thickness of between 12m and 13m. The depth of mining is between 55m and 60m below surface.

This zone contains only one of the three criteria for crownhole development, overlying dolerite and glacial till/drift, therefore the potential for any future crownhole development is considered to be extremely low.

9.7 L4900 Zone B3

The road in this area is undermined by a number of four-way intersections in the underground mine. The cross sections modelled show stable mine conditions. Modelled surface deformations are consistent with those being measured on surface by Gyproc's on-going surface survey measuring programme.

The laser scans show stable mine conditions in the rooms modelled and in the intersections that were laser scanned.

The underground workings are 30-35m below surface, the thickness of the gypsum roof beam is between 8 m and 10m and the lower gypsum is overlain by a thick dolerite unit.

This zone contains two of the three criteria for crownhole development, depth of mining and overlying dolerite and glacial till/drift, therefore the potential for any future crownhole development is considered to be very low.

9.8 L4900 Zone B4

This zone is located towards the western end of the L4900 road. There are a number of four-way intersections below the road. The cross sections modelled show stable mine conditions. Modelled surface deformations are consistent with those being measured on surface by Gyproc's on-going surface survey measuring programme.

The laser scans show stable mine conditions in the rooms modelled and in the two four-way intersections that were laser scanned.

The depth of mining increase from 30m on the eastern end of the zone to 50m on the western end. The gypsum roof beam varies from 3.5 m to 13 m thick. The geology in this zone is different from the other three zones in that there is mudstone lying directly above the lower gypsum. Above this there is unmined Upper Gypsum seam. This adds a layer of protection above the mine preventing any possible mine instability reaching surface.

This zone contains only one of the three criteria for crownhole development, depth of mining. Furthermore because of the presence of the strong Upper Gypsum above the potential for any future crownhole development in this zone is considered to be exceptionally low.

9.9 Conclusion on Stability of L4900 Road

Based on the investigations carried out, the geotechnical analysis and interpretation of the cloudscan laser surveys no specific areas of concern have been noted in the areas of the mine that extend below the road. The occurrence of the December 2018 crownhole has not increased the risk of future crownhole development or subsidence along the L4900 which continues to be very low. This is in line with the findings of the previous predictive analyses conducted by SRK in October 2018 which concluded that the L4900 is safe to use.

The laser surveys and the geotechnical borehole logging have provided strong evidence that, outside of the area of the crownhole and historical subsidence, there has been virtually no deterioration in the mine conditions since the excavations were created. This provides confidence that the roof beams and pillars are still doing the job for which they were designed, which is to support the underground openings and prevent surface subsidence.

The on-going interpretation of the surface levelling programme, the extensometers along with periodic underground laser scans will provide assurance that the mine below the road remains in a stable condition.

10 RECOMMENDATIONS

Whilst SRK considers the possibility of future subsidence occurring below the L4900 road to be generally very low to extremely low, in terms of relative risk of a possible future subsidence or crownhole event the four zones can be ranked from highest relative risk to lowest relative risk as follows:

Zone A1 – Very low risk of future subsidence or crownhole development. Contains two of the three criteria for crownhole development. Contains the December 2018 crownhole.

Zone B3 – Very low risk of future subsidence or crownhole development. Contains two of the three criteria for crownhole development.

Zone A2 – Area of historical subsidence. Extremely low risk of future subsidence or crownhole development. Contains only one of the three criteria for crownhole development.

Zone B4 – Extremely low risk to unlikely possibility of future subsidence or crownhole development Contains only one of the three criteria for crownhole development. Also has unmined upper gypsum overlying the underground workings making the possibility of any crownholes developing on surface highly unlikely.

The recommendations for action are presented below on a zone by zone basis and are commensurate with the relative risk identified above.

10.1 Zone A1

1. Since water is a major contributor to the development of crownholes carry out a ground surface drainage survey to ensure that any surface water cannot pond above and seep into the mine workings.

2. Carry out an investigation to determine whether there are underground water or sewage service lines crossing the area. Ensure that they are not leaking and make good if they are.

3. Continue surface level monitoring through the zone. Review the position of the monitoring points on surface and add additional points to ensure there is at least one monitoring point located above every room and one above every pillar along the road.

4. Install extensometers into boreholes KC19H1, KC19H3, KC19H4 and KC19H5 to monitor roof beam deflection adjacent to the road. Roof beam movement of 2% of roof beam thickness should be used as a trigger for further investigation.

5. Use borehole KC19H17 for future laser scans initially on a two-year frequency. Scanning frequency should be reviewed after each scan.

6. Those boreholes that will be kept open for future monitoring should be lined appropriately for use and an appropriate sealable and lockable collar constructed and installed which will prevent any water entering into the mine and prevent the open boreholes becoming a future initiator of mine instability. All boreholes not being used for a specific purpose should be grouted and closed.

10.2 Zone B3

1. Since water is a major contributor to the development of crownholes carry out a ground surface drainage survey to ensure that any surface water cannot pond above and seep into the mine workings.

2. Carry out an investigation to determine whether there are underground water or sewage service lines crossing the area. Ensure that they are not leaking and make good if they are.

3. Continue surface level monitoring through the zone. Review the position of the monitoring points on surface and add additional points to ensure there is at least one monitoring point located above every room and above every pillar along the road.

4. Install extensometers into borehole KC19H11 and KC19H21 to monitor roof beam deflection in the four-way intersections. Roof beam movement of 2% of roof beam thickness should be used as a trigger for further investigation.

5. Those boreholes that will be kept open for future monitoring should be lined appropriately for use and an appropriate sealable and lockable collar constructed and installed which will prevent any water entering into the mine and prevent the open boreholes becoming a future initiator of mine instability. All boreholes not being used for a specific purpose should be grouted and closed.

10.3 Zone A2

1. Since water is a major contributor to the development of crownholes carry out a ground surface drainage survey to ensure that any surface water cannot pond above and seep into the mine workings.

2. Carry out an investigation to determine whether there are underground water or sewage service lines crossing the area. Ensure that they are not leaking and make good if they are.

3. Continue surface level monitoring through the zone.

4. Ensure that all boreholes not being used for a specific purpose should be grouted and closed.

10.4 Zone B4

1. Continue surface level monitoring through the zone.

2. Install extensometers into borehole KC19H10 and KC19H15 to monitor roof beam deflection in the four- way intersections. Roof beam movement of 2% of roof beam thickness should be used as a trigger for further investigation.

3. Those boreholes that will be kept open for future monitoring should be lined appropriately for use and an appropriate sealable and lockable collar constructed and installed which will prevent any water entering into the mine and prevent the open boreholes becoming a future initiator of mine instability. All boreholes not being used for a specific purpose should be grouted and closed.

For and on behalf of SRK Consulting (UK) Limited

Neil Marshall, Corporate Consultant (Geotechnical Engineering), **Project Manager** SRK Consulting (UK) Limited	Richard Oldcorn, Managing Director Corporate Consultant (Due Diligence) **Project Director** SRK Consulting (UK) Limited

Glossary

Glossary Item	text inserted as example for definition of the term included in the Glossary as appropriate.

Abbreviations

IMMM	text inserted as example for definition of the term included in the Glossary as appropriate.

Units

Mt	Million metric tonnes

APPENDIX

A BRITISH GYPSUM GEOLOGY LOGS AND PHOTOGRAPHS

APPENDIX

B SRK GEOTECHNICAL LOGS

APPENDIX

C ROCK MASS STRENGTH CALCULATION DATA SHEETS

APPENDIX

D POINT CLOUD INTERPRETATION

APPENDIX

E ASSESSMENT OF CLOUDSCAN SURVEYS

APPENDIX 7.7
Drumgoosat Underground Mine - Investigation & Analysis of mine Stability below the R179 - SRK – April 2020

DRUMGOOSAT UNDERGROUND MINE - INVESTIGATION AND ANALYSIS OF MINE STABILITY BELOW THE R179 ROAD

Prepared For

Gyproc Ltd

Report Prepared by

srk consulting

SRK Consulting (UK) Limited
UK30787

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EXECUTIVE SUMMARY DRUMGOOSAT UNDERGROUND MINE - INVESTIGATION AND ANALYSIS OF MINE STABILITY BELOW THE R179 ROAD

1 INTRODUCTION

Drumgoosat Mine is a shallow underground mine that extracted gypsum using room and pillar mining methods. The mine stopped production in 1989. The main R179 road traverses the southern side of the mine with the mine passing below the road at a number of locations. The minor L4900 road traverses the north-eastern edge of the mine and the mine also passes below this road in a number of locations.

Following historical subsidence along the L4900 and the more recent occurrence of a crownhole adjacent to the L4900 road, the L4900 road was temporarily closed and an extensive ground investigation programme comprising the drilling of 25 cored boreholes was carried to determine the stability of the underground workings below the road and assess the potential for further instability. This investigation and the results of the analysis was presented in an SRK report titled '30238_December 2018 Crownhole Report_Final(V2)' dated April 2019.

Prior to this and following the September 2018 subsidence event, SRK carried out predictive stability analyses of the underground mine below the R179 road. The results of these analyses were presented in an SRK report titled '30238_Drumgoosat Subsidence Event Technical Report_Final(V4) dated October 2018.

Whilst the R179 has not been affected by any subsidence although a crownhole has occurred in a field adjacent to the road, because this road is a major road and carries heavier and more frequent traffic than the L4900 road, Gyproc decided to carry out a similarly detailed investigation of the conditions of the mine below this road by drilling and surveying a further 18 boreholes. This work therefore serves as an extension to the previous work reported by SRK.

2 WORK UNDERTAKEN

A total of 17 boreholes were drilled along the R179 road between late 2019 and early 2020, 15 of which were collared on the western side of the R179 road, two of which were collared on the eastern side. The selection of borehole locations was decided jointly by Monaghan County Council (MonCC), the Exploration and Mining Division of the Department of Communications, Climate Action and Environment (EMD), the Environmental Protection Agency (EPA) and Gyproc at the outset of the programme. The drilling strategy was to target four-way intersections under the road. For this, the boreholes were drilled from locations off the carriageway resulting in mostly inclined holes.

Geotechnical logging of all holes was performed by SRK's geotechnical consultants, except KC20-R17 and KC20-R18, which were drilled after SRK's geotechnical logging site visits. Whilst reference to these boreholes is made in the report, there are no geotechnical logs for them.

The absence of geotechnical logs is not considered to be significant for the R179 assessment. KC20-R17 and KC20-R18 were drilled between 50 m and 70 m away from the edge of the carriageway.

Two additional boreholes drilled during earlier drilling campaigns, but which are relevant to the current study, are also referred to in this report. Borehole KC18K-R8 was drilled in late 2018 to provide laser survey access to Pillar R12 (discussed later). Borehole KC19-H4 was drilled close to the R179 during the 2019 L4900 drilling campaign.

All the boreholes intersected underground workings. The UK company Geoterra Ltd was commissioned to carry out 3D laser scanning surveys (also known as laser scan surveys) down the boreholes that intersected the workings. This allowed a 3D image of the underground mine to be constructed which provided a very accurate picture of the stability condition of the underground workings. This study has made extensive use of these surveys to draw conclusions on the current condition of the underground workings.

For the purpose of analysis, the R179 road was divided into five zones. Figure ES1 shows the location of these zones and the boreholes drilled in them.

The borehole core was geologically logged by British Gypsum geologists and geotechnically logged by SRK geotechnical engineers. A selection of gypsum borehole core was sent to a UK testing laboratory to determine the intact strength of the gypsum forming the mine roof beam.

Rock mass classification of the core was carried out. This is a measure of the fracture condition of the borehole core and allows the geological strength index (GSI) to be calculated. The GSI is an industry standard means of assessing the quality and strength of a rock mass made up of intact rock separated by natural fractures.

The intact strength and GSI of the gypsum and overlying mudstone were input to a 2-dimensional finite element package (RS2 produced by RocScience Inc of Canada). Twelve cross sections across and parallel to the R179 road were analysed for underground room stability and surface deformation.

The cloudscan surveys were processed and the mine openings that were surveyed were inspected for evidence of roof and pillar instability.

The integration of all data collected, and the results of the analyses allowed SRK to formulate conclusions on the stability condition of the mine below the R179 road and the risk of possible future mine instability affecting the R179 road.

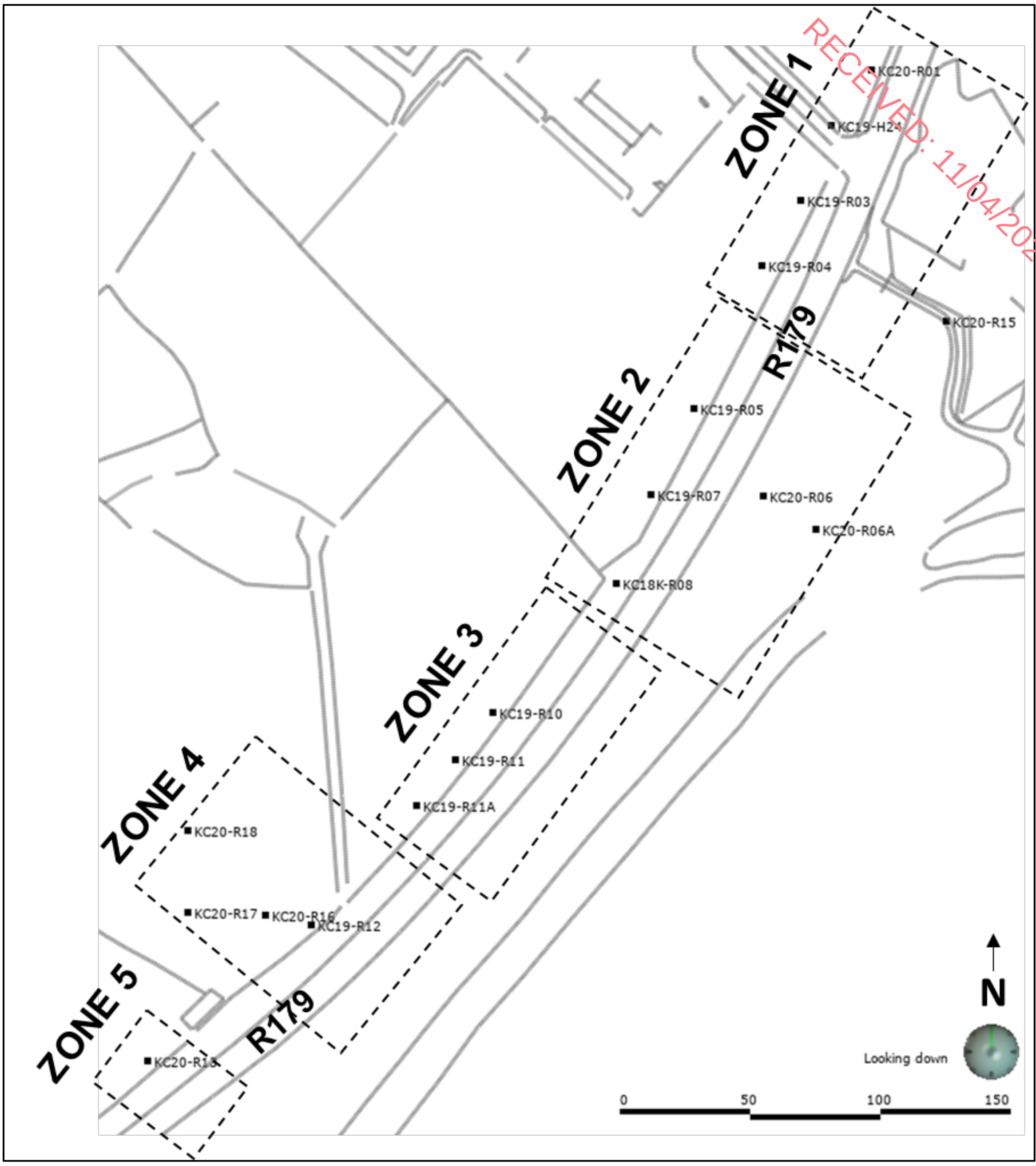

Figure ES 1: Borehole and Zone Locations

3 STUDY CONCLUSIONS

3.1 Geotechnical Characteristics

The investigation work undertaken has confirmed that the geotechnical conditions of the Lower Gypsum within which the underground workings below the R179 road are located are characterised as fair quality rock mass as indicated by the average GSI. The Lower Gypsum contains closely to moderately spaced bedding planes and widely spaced sub-vertical orthogonal joint planes and is interbedded with bands of weaker mudstone. The gypsum has an intact rock strength in the range 7-20 MPa (15-30 MPa for the L4900 study) and a geological strength index in the range 45-55 (50-60 for the L4900 study). The lower values of GSI is a consequence of the effect of the inclusion of weaker mudstone layers within the gypsum. The difference in strength is not considered material to the overall rock mass strength as the intact rock strength only contribute between 3% (for the 7–20 MPa range) and 5% (for the 15-30 MPa range) to the total value of GSI. The fracture and weathering condition of the rock mass is more important in defining rock mass strength.

The mine workings lie between 40 m and 90 m below the R179 road (30 m below the L4900). They are at their shallowest on the north eastern end of the road in Zone 1 and deepest at the south western end in Zone 5. The thickness of the roof beam above the workings below the road ranges between 5 m and 18 m (remembering that the minimum roof beam design thickness is 3 m). The floor beam below the workings is of a similar thickness range.

Below the R179 road the Lower Gypsum is overlain by mudstone which itself is overlain by Upper Gypsum of varying thickness.

The strength and GSI value ranges measured from the borehole core remain consistent with the values generated by SRK during its underground mapping campaigns carried out between 1999 and 2005 and with the ‘expected’ strength conditions used in its predictive analyses of mine stability below the R179 and L4900 reported in October 2018. This suggests that there has been little degradation of the Lower Gypsum rock mass strength over the last 20 years.

3.2 Stability Condition of Underground Workings

For a mine which is around 40 years old, the condition of the underground rooms, as determined from the borehole laser surveys, is generally reasonable. There is some evidence that bedding bounded slabs have fallen from the roof of the workings. The slab thickness as estimated from the laser scans is up to a maximum of 0.5 m. This is equivalent to 4.5% of roof beam thickness for the average roof beam thickness of 11 m identified by the borehole drilling. The slabbing is very localised and is found to be most prevalent in four-way intersections. This type of roof instability is typical of an underground mine whose rock mass contains well developed, open bedding planes. Slabbing of this nature normally occurs immediately above the workings where there is little rock mass confinement. Provided the gypsum roof beam is sufficiently thick, at least 3 m, propagation of slabbing deeper into the roof beam is generally prevented as a stable arch is formed above the mine opening.

The room floors contain evidence of debris. Some debris appears as rounded, hummocky mounds of material which appears to be waste material stowed in the workings as part of the mining operations. In some of the laser scan images tyre tracks are clearly visible on the floor. Other debris is angular, which suggests this material may have fallen from the roof or from the sidewalls of the pillars.

3.3 Finite Element Modelling

All the finite element modelling results indicate stable mine conditions. Limiting roof beam stability has been defined as a maximum deflection of 2% of roof beam thickness. None of the roof beams simulated exceeds the maximum deflection value. The deformation values indicate no instability of the pillars modelled. Simulated surface deformation is of the order of millimetres which is consistent with the magnitude of the actual deformation being measured by Gyproc by the surface levelling network located along the north western edge of the R179 road. The modelling is therefore considered to be a reasonable simulation of current actual mine stability condition.

The October 2018 predictive modelling used gypsum strength parameters estimated from historical underground mapping. Those predictive finite element analyses returned surface deformations below the R179 of between 0 mm and 8 mm. For the modelling reported herein, the surface deformation below the road varied from less than 1 mm to 2 mm. These are smaller but of the same order of magnitude as the earlier predictive modelling.

3.4 Crownhole Development Potential

The work carried out for the L4900 investigation and subsequent investigations into crownhole development at Drumgoosat identified a number of factors that were needed to create conditions amenable for the development of a crownhole. These are:

- A very thin (<1 m) or absent gypsum roof beam.
- Depth of mining less than 10 times the height of the mine openings. For an average mine height of 6 m, the potential for crownhole development would be greater where the depth of mining was less than 60 m below surface.
- Gypsum unit overlain by material that sufficiently weak to be able to flow or collapse into the mining void.
- The potential for crownhole development is heightened where underground workings have been flooded and the rock forming the roof beam has been weakened by wetting.

Below the R179, the mine workings in Zones 1, 2, and 3 are all less than 60 m below surface. The investigation has indicated that in the location where boreholes have been drilled, the thickness of the Lower Gypsum roof beam lies in the range 5 m to 18 m.

Below the R179 road in Zones 1, 3, 4, and 5, the Lower Gypsum is overlain by mudstone which in turn is overlain by unmined Upper Gypsum between less than 1 m and 10 m thick. In Zone 2, the Upper Gypsum is very thin or absent immediately below the road but thickens out to the north west of the road.

The underground workings in Zones 1, 2, and 3 are dry workings as they all lie above the level of maximum mine flooding. The underground workings below the road in Zones 4 and 5 have been permanently under water since mining at Drumgoosat ceased.

Based on this description, none of the Zones investigated contains all of the criteria required for crownhole development. Historically, the only crownhole that has occurred adjacent to the R179 is located north west of the road on northern end of Zone 1. The laser survey undertaken in borehole KC20-R01 has identified a chimney hole in the roof of one of the underground workings to the south east of the road; however, because there is a competent layer of upper gypsum above these workings, it is very unlikely that the chimney hole at this location will propagate into a crownhole at surface.

Based on the information generated by this investigation, the risk of crownholes developing along or adjacent to the R179 in the future is considered to be very low. There is a slightly greater, albeit small, risk in Zone 1 due to the historical occurrence of crownholes in the area. The monitoring measures that Gyproc have in place, surface levelling and visual inspections, are appropriate for managing the slightly greater risk.

3.5 Overall Conclusions on the Stability of the R179 Road

Historically, there has been no instance of mine induced stability along and adjacent to the R179. Based on the investigations carried out, the geotechnical analysis and interpretation of the cloudscan laser surveys, no high risk, unstable undermining areas have been identified.

The laser surveys and the geotechnical borehole logging have provided strong evidence that there has been virtually no deterioration in the mine conditions in the 40 years since the excavations were created. This provides confidence that the roof beams and pillars are still doing the job for which they were designed, which is to support the underground openings and prevent surface subsidence. On this basis, the R179 continues to be safe to use.

Gyproc has already initiated a number of measures to provide assurance and early warning of potential underground instability. These are:

- monitoring of a network of surface levelling points along and adjacent to the R179: and
- installation of extensometers movement in a number of the boreholes drilled as part of this investigation to measure roof beam movement.

These measures are described in Section 4.

4 RECOMMENDATIONS

4.1 Surface Monitoring

Following the September 2018 subsidence event, a network of levelling stations was installed adjacent to the R179 road. These are being surveyed by Gyproc on a regular basis. The levelling points are located above most of the areas investigated by drilling and laser surveys. Interpretation of the levelling data by SRK, which is being carried out on quarterly basis, shows very little increase in movement with time, indicating that there is currently no adverse underground mining. The movement magnitudes are associated with a Trigger Action Response Plan (TARP) developed for the L4900. The movements of the R179 levelling points fall within the Extremely Low Risk zone of the TARP, indicating that the movement is not significant. The monitoring should continue as defined in the TARP, should be reviewed, and appropriate actions taken if and when defined surface movement trigger levels are exceeded.

4.2 Borehole Extensometers

Gyproc has identified boreholes where it wished to install extensometers and with which SRK agrees. These are:

- KC19-R03
- KC19-R06
- KC18K-R08

- KC19-R11
- KC19-R12

Following the interpretation of the laser surveys, however, SRK recommends that the extensometer proposed for borehole KC19-R11 should be installed in borehole KC19-R10 instead. Monitoring and TARP protocols already set up for the borehole extensometers along the L4900 road should be adopted for the R179 boreholes.

4.3 3D Numerical Modelling of Pillar R12

Pillar R12 is a small pillar located adjacent to the north western side of the R179 carriage way. Because of its small size and its location, this pillar has been subject of specific analyses. Boreholes (KC18L-R9 and KC18K-R8) have been drilled either side of the pillar and laser scanning has been carried out down each of the boreholes on two separate occasions.

Whilst two dimensional stability analyses have been carried out along cross sections through the narrowest part of the pillar, which shows that the pillar is stable, the laser scans around the pillar are now sufficiently detailed to allow a 3D model of the pillar to be constructed. SRK recommends that a 3D analysis of the pillar is undertaken using the computer codes FLAC3D or 3DEC. The results of a 3D analysis would provide assurance of the stability of the pillar and the stability of the R179 road.

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DRUMGOOSAT UNDERGROUND MINE - INVESTIGATION AND ANALYSIS OF MINE STABILITY BELOW THE R179 ROAD

1 INTRODUCTION

1.1 Background

SRK Consulting (UK) Limited ("SRK") is an associate company of the international group holding company, SRK Consulting (Global) Limited (the "SRK Group"). SRK has been requested by Gyproc Ireland ("Gyproc", hereinafter also referred to as the "Company" or the "Client") to undertake geotechnical interpretation and analysis of the mine conditions and stability of those parts of the abandoned Drumgoosat Mine below the main R179 road that links Carrickmacross to Kingscourt in County Monaghan, Ireland.

Drumgoosat Mine is a shallow underground mine that extracted gypsum using room and pillar mining methods. The mine stopped production in 1989. The main R179 road traverses the southern side of the mine with the mine passing below (undermining) the road at a number of locations. The minor L4900 road traverses the north-eastern edge of the mine and the mine also passes below (undermines) this road in a number of locations.

Following historical subsidence along the L4900 and the more recent occurrence of a crownhole adjacent to the L4900 road, the L4900 road was temporarily closed and an extensive ground investigation programme comprising the drilling of 25 cored boreholes was carried to determine the stability of the underground workings below the L4900 road and assess the potential for further instability. This investigation and the results of the analysis was presented in an SRK report titled '30238_December 2018 Crownhole Report_Final(V2)' dated April 2019.

Prior to this and following the September 2018 subsidence event, SRK carried out predictive stability analyses of the underground mine below the R179 road. The results of these analyses were presented in an SRK report titled '30238_Drumgoosat Subsidence Event Technical Report_Final(V4) dated October 2018.

Whilst the R179 has not been affected by any subsidence although a crownhole has occurred in a field adjacent to the road, because this road is a major road and carries heavier and more frequent traffic than the L4900 road, Gyproc decided to carry out a similarly detailed investigation of the conditions of the mine below this road which comprised the drilling and surveying of a further 18 boreholes. This work therefore serves as an extension to the previous work reported by SRK.

1.2 Work Undertaken

SRK was commissioned by Gyproc to carry out a geotechnical assessment of the data collected. The scope of work was defined as:

- Geotechnical logging of boreholes and classification using Bieniawski and GSI classification systems.
- The selection of gypsum core samples for laboratory strength testing.
- Rock mass strength characterisation from the logging and laboratory data and development of characteristic strength values for subsequent computer stability modelling;
- Finite element modelling analysis of defined cross-sections along the R179 road and interpretation of the results;
- Geotechnical assessment and interpretation of laser scans of the mine workings reached via the boreholes; and.
- Preparation of draft and final reports, including presentation.

This report presents the detail of the work carried out.

2 GEOTECHNICAL INVESTIGATION WORK

A total of 17 boreholes were drilled along the R179 road between late 2019 and early 2020, 15 of which were collared on the western side of the R179 road, two of which were collared on the eastern side. The selection of borehole locations was decided jointly by Monaghan County Council (MonCC), the Exploration and Mining Division of the Department of Communications, Climate Action and Environment (EMD), the Environmental Protection Agency (EPA) and Gyproc at the outset of the programme.

The drilling strategy was to target four-way intersections under the road. For this, the boreholes were drilled from locations off the carriageway resulting in mostly inclined holes. Geotechnical logging of all holes was performed by SRK's geotechnical consultants, except for KC20-R17 and KC20-R18, which were drilled after SRK's geotechnical logging site visits. Whilst reference to these boreholes is made in the report, there are no geotechnical logs for them. The absence of geotechnical logs is not considered to be significant for the R179 assessment. Boreholes KC20-R17 and KC20-R18 were drilled between 50 m and 70 m away from the edge of the carriageway. Two additional boreholes drilled during earlier drilling campaigns, but which are relevant to the current study, are also referred to in this report. Borehole KC18K-R8 was drilled in late 2018 to provide laser survey access to Pillar R12 (discussed later). Borehole KC19-H4 was drilled close to the R179 during the 2019 L4900 drilling campaign.

All the boreholes intersected underground workings. The UK company Geoterra Ltd was commissioned to carry out 3D laser scanning surveys (also known as laser scan surveys) down the boreholes that intersected the workings. This allowed a 3D image of the underground mine to be constructed which provided a very accurate picture of the stability condition of the underground workings. This study has made extensive use of these surveys to draw conclusions on the current condition of the underground workings.

Further details of the geotechnical investigation work carried out are presented in the following sections.

2.1 Borehole Drilling

Borehole drilling was completed using a double-tubed wireline system. The top of each borehole was open holed through the overburden materials to depths that varied between 5 m and 14 m. Coring was then carried out through the underlying units of generally mudstone and gypsum until the mine workings were reached. Upon extraction, the core was laid directly into wooden core boxes for logging. Detailed geological logging and core photographing was carried out by British Gypsum geologists. The geological borehole logs and photographs are presented in Appendix A. For the purpose of analysis, the R179 road was divided into five zones. Table 2-1 shows the details of the boreholes drilled and Figure 2-1 shows the locations of the boreholes relative to the R179 road and the zonation of the boreholes used for analysis.

Table 2-1: Details of the boreholes drilled

BHID	Azimuth (°)	Dip (°)	Depth (m)	Easting	Northing	Elevation	Zone
KC20-R01	113	56	53	280966.51	300130.53	1054.37	1
KC19-R03	91	59	50	280939.51	300080.79	1057.97	1
KC19-R04	94	59	50	280924.73	300055.92	1058.58	1
KC19-R05	120	59	47	280898.69	300000.98	1056.85	2
KC20-R06	58	88	40	280925.24	299967.43	1055.00	2
KC20-R06A	332.9	89.7	35	280945.25	299954.79	1051.76	2
KC19-R07	100	63	46	280882.26	299967.86	1055.51	2
KC18K-R08	0	90	41.2	280868.83	299934.01	1053.38	2
KC19-R10	91	71	46	280821.8	299884.38	1049.57	3
KC19-R11	152	57	52	280807.19	299866.47	1048.92	3
KC19-R11A	155	60	51	280792.52	299848.7	1048.38	3
KC19-R12	175	75	63	280751.68	299803.16	1046.76	4
KC20-R13	98	79	80	280689.17	299751.17	1044.08	4
KC19-R14	104	90	93	280593.96	299676.8	1041.53	5
KC20-R15	8.9	58	46	280995.67	300034.34	1056.93	1
KC20-R16	149	89	91	280734.6	299806.65	1046.47	4
KC20-R17	59	90	100.6	280704.62	299808.11	1046.51	4
KC20-R18	187.7	89.7	106.6	280704.74	299839.32	1048.55	4
KC19-H24	137	71	50	280951.6	300109.5	1055.07	1

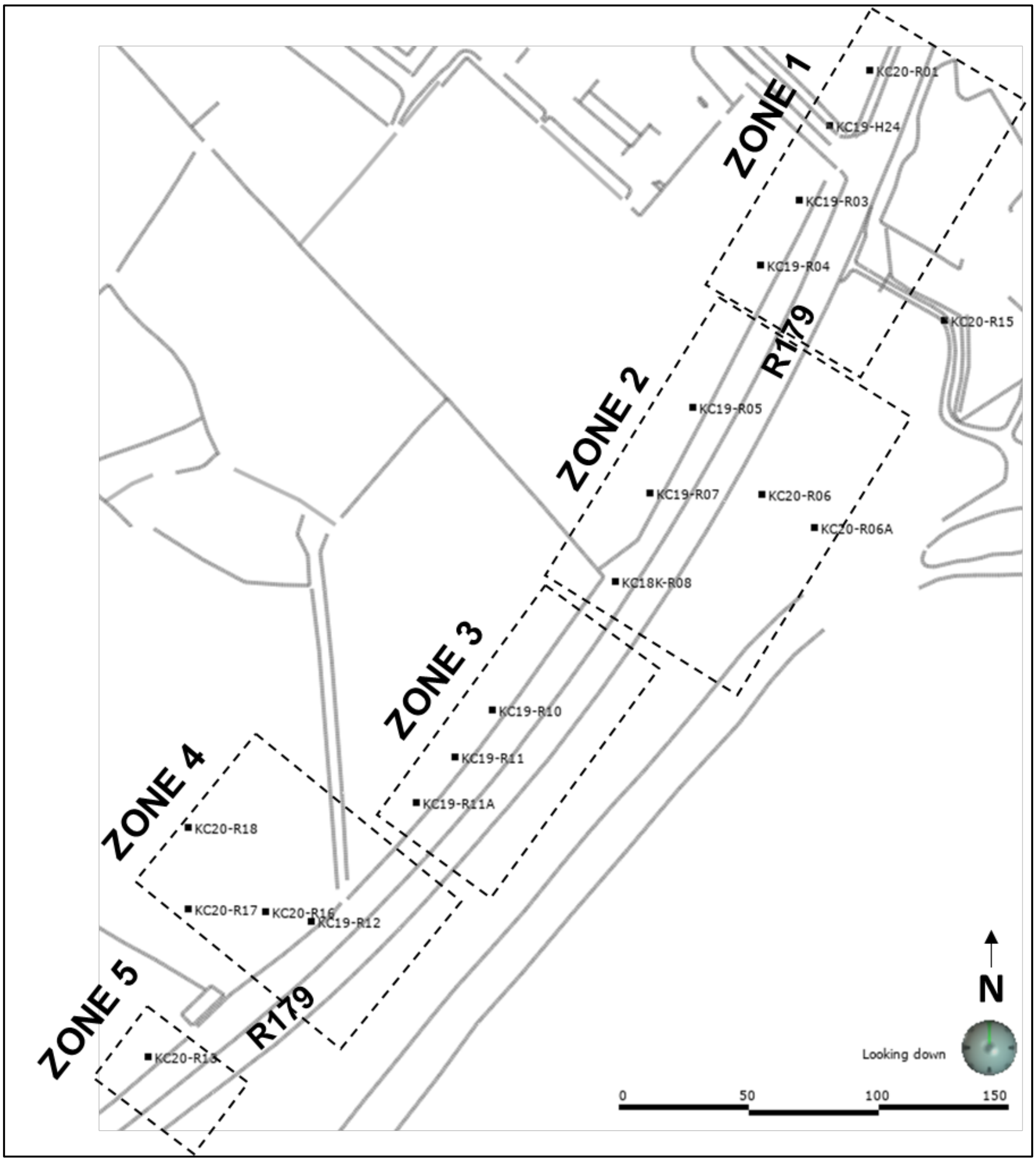

Figure 2-1: Borehole Positions relative to the R179 road

2.2 Geotechnical Logging

The Rock Mass Rating system defined by Bieniawski (1989) (RMR^{89}), was used to characterise the rock materials.

The system assigns ratings to six parameters that are used to classify the rock mass. These parameters are the following:

1. Rock Quality Designation (RQD) rating
2. Intact Rock Strength (IRS) rating
3. Spacing rating
4. Joint Condition rating
5. Groundwater Conditions rating
6. Orientation of discontinuities

These parameters each contribute to the overall RMR value in the follow way:

Parameter	Rating
IRS	0-15
RQD	0-20
Spacing rating	0-20
Joint Condition rating	0-30
Groundwater Condition rating	0-15

The final parameter in the determination of RMR^{89}, Orientation of discontinuities (6), is an adjustment factor that is applied based on the orientation of the identified discontinuities relative to the excavation surface and defines the ease or difficulty with which blocks that can fall or slide into the mine rooms. The adjustment factor ranges from 0 for Very favourable, to -50 for Unfavourable.

Although the core was not orientated, the dip of the joints in the majority of boreholes was found to be perpendicular with the core axis, indicating near horizontal bedding planes (for vertical boreholes). This is confirmed by the laser scan data where the joint sets are visible in situ. As such, no additional adjustment was made to the RMR^{89} for the orientation of discontinuities. The Groundwater Conditions rating was set at the maximum possible ratings of 15 for all intervals, which means each RMR considers the rock to be dry. Where groundwater data were available, piezometric surfaces have been added to the finite element models to account for the effects of varying water tables (see Finite Element Modelling in each section).

The resultant rock mass rating is given as a value between 0 and 100, with intervals within this range being assigned a ‘class’, as follows:

RMR value	Class
0-20	Very poor
21-40	Poor
41-60	Fair
61-80	Good
81-100	Very good

For modelling purposes, which requires rock mass ratings in the form of Geological Strength Index (GSI), the weighted RMR^{89} is calculated and converted to GSI using the formula taken from Hoek et al., 1995:

$$GSI = RMR'_{89} - 5\ (for\ RMR'_{89} > 23)$$

Logging of the core revealed interbedding between gypsum and mudstone in the Upper Gypsum seam in all Zones, and particularly high amounts in Zones 2 and 3. The strength of the mudstone beds was found to vary between extremely weak strength soil and very weak strength rock; for soil strength mudstone Bieniawski's RMR could not be applied (as this is only valid for rock materials). The percentages of mudstone making up the gypsum seams in each zone, and the average thickness of Upper Gypsum intersected by the boreholes drilled in each zone is shown in Table 2-2.

Table 2-2: Percentages of mudstone in gypsum seams in all zones

ZONE	Lithology	Average Top Depth Below Surface (m)	Average Thickness (m)	% Gypsum	% Mudstone
1	Upper Gypsum	15.40	4.25	84%	16%
	Lower Gypsum	30.08	-	93%	7%
2	Upper Gypsum	15.65	0.91	65%	35%
	Lower Gypsum	26.28	-	100%	0%
3	Upper Gypsum	15.33	8.17	61%	39%
	Lower Gypsum	35.32	-	100%	0%
4	Upper Gypsum	33.81	10.12	91%	9%
	Lower Gypsum	56.74	-	96%	4%
5	Upper Gypsum	58.53	10.27	89%	11%
	Lower Gypsum	81.43	-	100%	0%

To ensure that the presence of the weaker mudstone intervals in the gypsum units was accounted for in the determination of the strength of that unit, all mudstone intervals of soil strength existing within the gypsum units were given an RMR of 0 and included in the average RMR for the gypsum unit. Figure 2-2 shows a histogram of the RMR classifications for both the upper and lower gypsum seams, which includes the lower mudstone RMR values and zero values. Table 2-3 shows the effect that the inclusion of low and zero mudstone RMR values had on average RMR values across the upper and lower seams of gypsum.

The results of the geotechnical logging and RMR^{89} system classification of the boreholes are presented in Appendix B, which shows both the logged inputs and associated RMR ratings and calculation. The GSI values used for modelling are shown in Table 3-1.

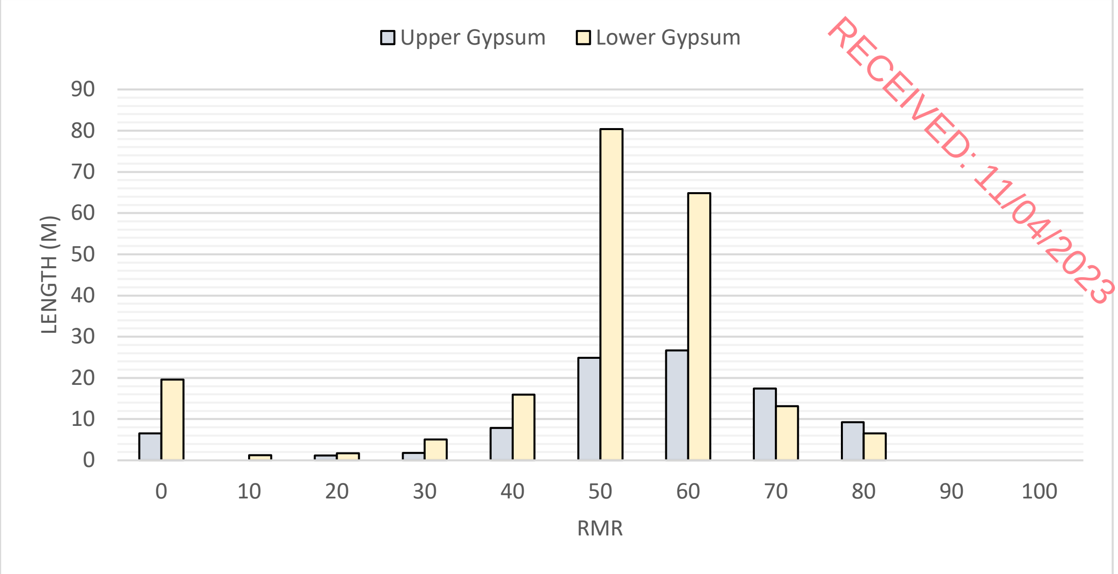

Figure 2-2: Histogram of RMR values for upper and lower gypsum seams

Table 2-3: Adjustments in RMR to account for mudstone/gypsum interbedding

Seam	% Mudstone	Average RMR without Mudstone included	Average RMR with Mudstone included
Upper Gypsum	11%	58	48
Lower Gypsum	5%	61	54

2.3 Rock Strength Testing

Intact rock strength (IRS) is an important input to determining rock mass strength for modelling. A selection of intact gypsum core was carefully packed to avoid damage and shipped to a laboratory in the UK, KIWA CMT in Derby, for uniaxial compressive strength (UCS) testing. Table 2-4 gives details of the UCS testing that was performed on selected samples from the borehole core and shows the UCS result for each sample in Mega Pascals (MPa).

Table 2-4: Laboratory UCS testing programme.

BHID	Depth from (m)	Depth to (m)	Length (mm)	Core Diameter (mm)	Lithology	UCS (MPa)	Corrected UCS (MPa)
KC19-R03	47.94	48.3	135	64	Gypsum	7	7
KC19-R04	46	46.3	130	64	Gypsum	20	20
KC19-R05	45.95	46.2	125	64	Gypsum	11	11
KC19-R07	45.53	45.8	125	64	Gypsum	13	13
KC19-R10	44.95	45.4	125	64	Gypsum	11	11
KC19-R11	51.1	51.4	130	64	Gypsum	7	6.9
KC19-R11A	49.7	50	125	64	Gypsum	15	15
KC19-R12	61.78	62.1	130	64	Gypsum	15	15
KC19-R14	90.97	91.2	130	64	Gypsum	11	11
KC19-R01	39.51	39.7	130	64	Gypsum	8	8.1
KC20-R06	38.54	38.8	130	64	Gypsum	10	10
KC20-R06A	34.45	34.7	130	64	Gypsum	9	8.7
KC20-R13	80.67	80.9	130	64	Gypsum	19	19
KC20-R15	43.25	43.6	130	64	Gypsum	7	7.5
KC20-R16	90.76	91	130	64	Gypsum	19	19

The IRS is an important part of RMR and the GSI calculation, which is estimated using a geological hammer during geotechnical logging. To ensure the accuracy of the IRS estimates made, the laboratory UCS test results were used to recalibrate the IRS estimates. The estimated IRS values for the intervals that were selected for UCS testing are plotted with the laboratory UCS test results in Figure 2-3. A trend line was calculated to determine the average difference between estimated IRS values and UCS test result values, which showed that IRS values were typically overestimated by 1.79 times. To allow for this overestimation, the estimate IRS values across all intervals were reduced by a factor of 1.79 to ensure accuracy of IRS estimate values based on UCS test results.

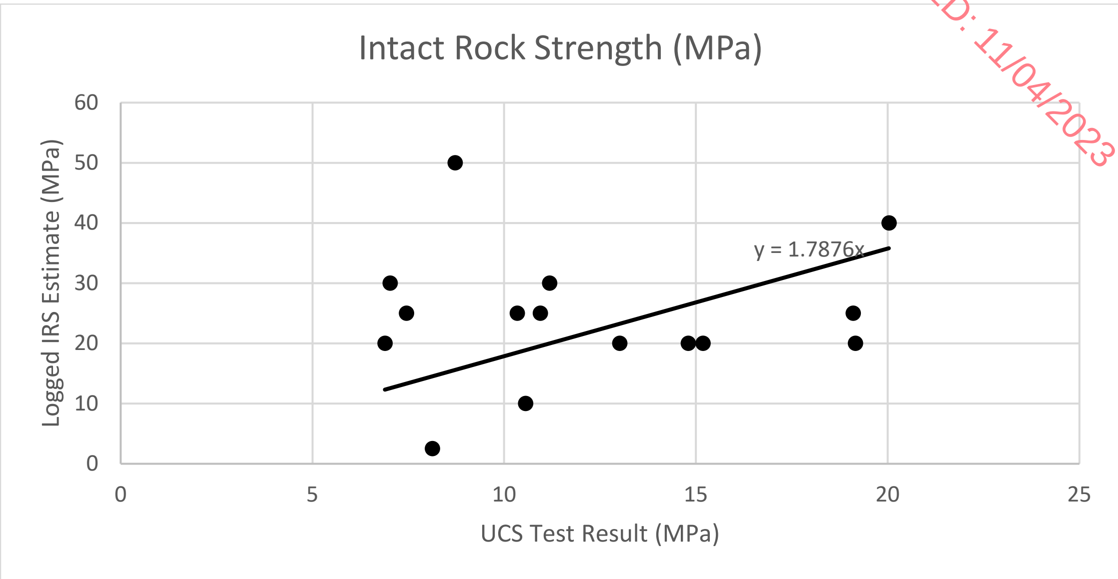

Figure 2-3: Estimated IRS plotted with UCS test results

2.4 Borehole Laser Surveys

Borehole laser scan surveys were completed in all but three of the boreholes drilled along the R179 road; six in Zone 1, five in Zone 2, three in Zone 3, two in Zone 4, and one in Zone 5. Figure 2-4 shows the positions of all the borehole scans carried out along the R179 road. Boreholes KC19-R12 and KC20-R16 in Zone 4 have not yet been scanned as they intersected dry airlocked workings. These holes have been closed to prevent flood water accumulating in them and wetting the roof of the excavations. Gyproc plans to reopen these holes and commission laser scans as soon as the mine water level has been lowered below the floor of the excavations intersected by the boreholes. Borehole KC20-R18, also in Zone 4, was the final hole to be drilled and is still awaiting laser surveying.

SRK was provided with 3D point cloud data which it then processed to provide detailed 3D images of the underground workings. A description of the process SRK used to convert point cloud data into detailed 3D images of the underground workings is presented in Appendix C. The surfaces defined from the laser scans have been presented in a grey-scale texture upon a dark blue background in the images in this report.

There is generally good correlation between the Gyproc mine survey plan and the 3D laser surveys in terms of the positioning and width of the rooms.

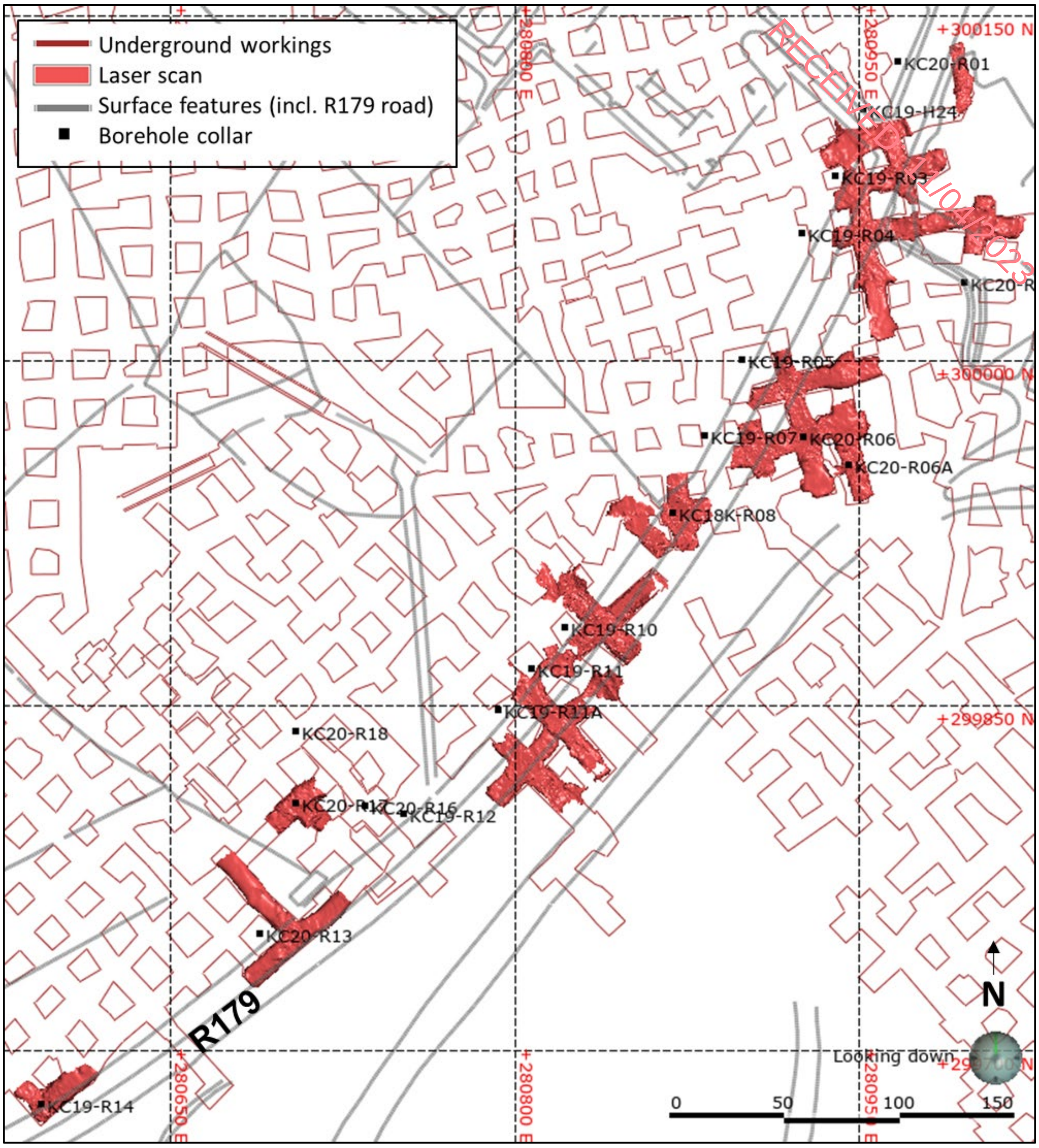

Figure 2-4: Locations of underground borehole surveys

3 R179 UNDERMINING ANALYSIS

3.1 Areas for Investigation

Five specific areas were investigated by borehole drilling and laser scanning, Zones 1 to 5, which are shown in Figure 3-1. The locations of the cross-sections used for undermining analysis of each of these areas are also shown, which have been identified and agreed between SRK and Gyproc for finite element modelling. Further details of these areas are presented in the next chapter of the report.

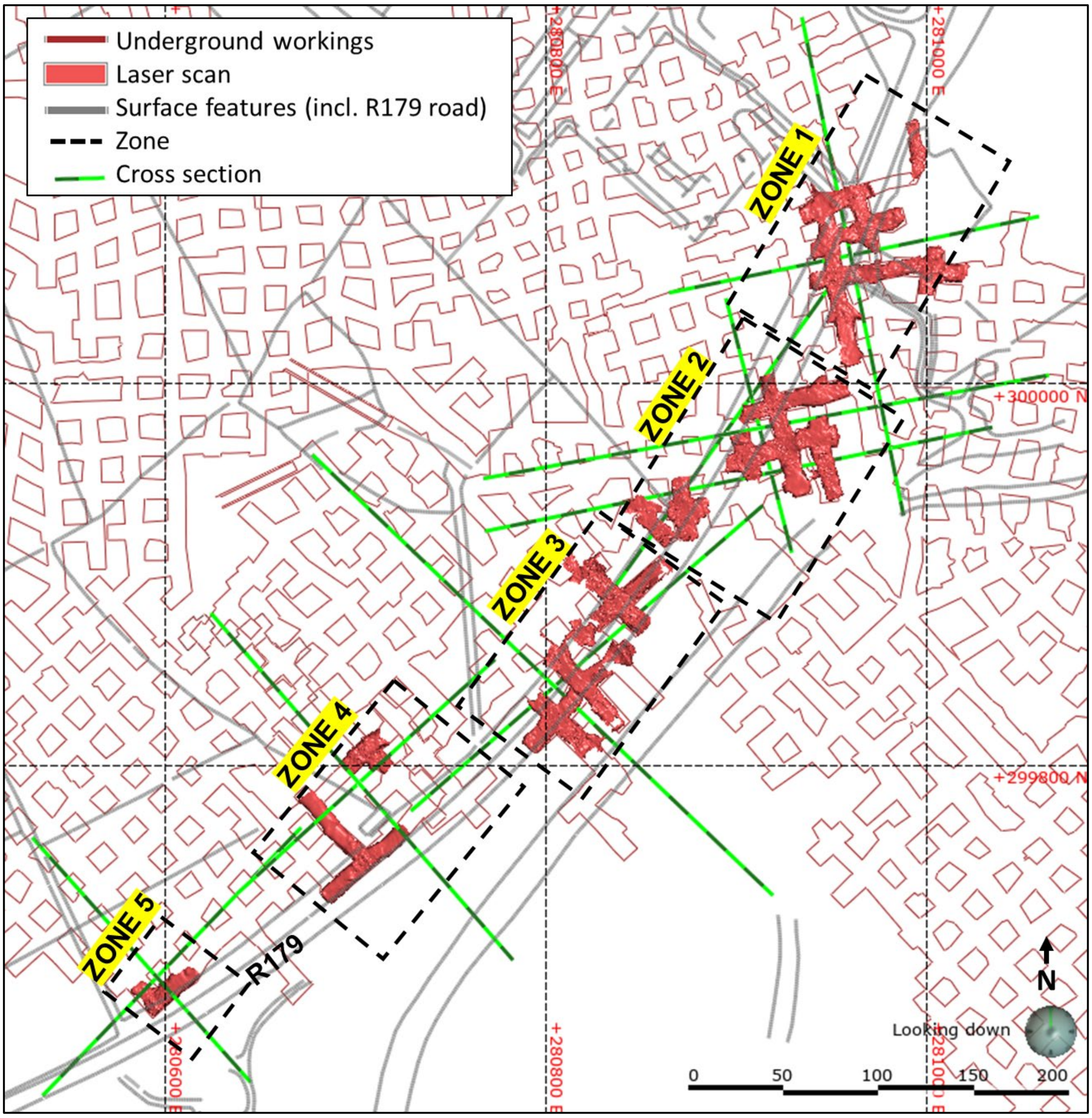

Figure 3-1: Location of Undermining Analysis Areas

3.2 Geotechnical Characteristics for Numerical Modelling

The empirical Hoek Brown strength criterion has been used to determine the cohesion values and friction angles of the units for which borehole core has been recovered and which could be logged using Bieniawski's RMR system (the upper and lower gypsum seams). For the glacial till or drift overlying the dolerite, through which the boreholes were open holed, a typical cohesion and friction angle for firm clay has been used. Similarly, for the mudstone units underlying and overlying the gypsum, a typical cohesion and friction angle for stiff clay has been used. For each zone, the borehole logging and strength data from all boreholes in that zone was combined to provide average location-specific parameters. The Rocscience software RocData was used to derive values of cohesion (c) and friction angles (phi) based on these location-specific parameters.

In addition to c and phi values, the numerical model requires inputs of rock density, Young's modulus, and Poisson's ratio. Engineering judgement has been used to assign these parameters. A summary of the strength and deformation data used in the finite element numerical modelling is presented in Table 3-1.

Table 3-1: Modelling Input Parameters

ZONE	Lithology	Average Top Depth Below Surface (m)	Average RMR	Average GSI	Unit Weight (MN/m^3)	c (MPa)	phi (°)	Young's Modulus (MPa)	Poisson's Ratio
	Drift	0	3	0	0.020	0.06	19	560	0.30
	Upper Mudstone	14.85	15	10	0.020	0.40	20	200	0.20
1	Upper Gypsum	15.40	47	42	0.023	0.09	48	4875	0.15
	Lower Mudstone	19.65	2	0	0.020	0.40	20	200	0.20
	Lower Gypsum	30.08	59	54	0.023	0.175	46.2	4875	0.15
	Basal Shale / Dolerite	42.73	N/A	N/A	0.025	10.5	35	20000	0.30
	Drift	0	6	1	0.020	0.06	19	560	0.30
	Upper Mudstone	N/A	N/A	N/A	0.020	0.40	20	200	0.20
2*	Upper Gypsum	15.65	30	25	0.023	0.09*	48*	4875	0.15
	Lower Mudstone	16.56	8	3	0.020	0.40	20	200	0.20
	Lower Gypsum	26.28	56	51	0.023	0.15	46.6	4875	0.15
	Basal Shale / Dolerite	41.92	N/A	N/A	0.025	10.5	35	20000	0.30
	Drift	0	3	0	0.020	0.06	19	560	0.30
	Upper Mudstone	11.24	8	3	0.020	0.40	20	200	0.20
3	Upper Gypsum	15.33	56	51	0.023	0.12	50.3	4875	0.15
	Lower Mudstone	23.49	0	0	0.020	0.40	20	200	0.20
	Lower Gypsum	35.32	54	49	0.023	0.162	44.0	4875	0.15
	Basal Shale / Dolerite	49.40	N/A	N/A	0.025	10.5	35	20000	0.30
	Drift	0	10	5	0.020	0.06	19	560	0.30
	Upper Mudstone	11.70	0	0	0.020	0.40	20	200	0.20
4	Upper Gypsum	33.81	60	55	0.023	0.26	48.9	4875	0.15
	Lower Mudstone	43.92	15	10	0.020	0.40	20	200	0.20
	Lower Gypsum	56.74	51	46	0.023	0.241	43.2	4875	0.15
	Basal Shale / Dolerite	77.80	N/A	N/A	0.025	10.5	35	20000	0.30
	Drift	0	0	0	0.020	0.06	19	560	0.30
	Upper Mudstone	8.70	0	0	0.020	0.40	20	200	0.20
5	Upper Gypsum	58.53	46	41	0.023	0.17	37.7	4875	0.15
	Lower Mudstone	68.80	0	0	0.020	0.40	20	200	0.20
	Lower Gypsum	81.43	59	54	0.023	0.279	39.1	4875	0.15
	Basal Shale / Dolerite	92.5	N/A	N/A	0.025	10.5	35	20000	0.30

**Less than 3 m of the Upper Gypsum and 0 m of the upper mudstone were intersected in the logging of the holes in Zone 2. As such, the same input parameters for the Upper Gypsum and Upper Mudstone in Zone 1 were used to define the strength of these units in Zone 2.*

The lower gypsum is a generally homogeneous material with only limited variability in terms of strength and RMR/GSI, although the lower gypsum seams in Zones 1 and 4 were found to have 7% and 4% mudstone, respectively. The rock mass rating for the lower gypsum seam lies in upper Fair rock mass quality designation. The range of GSI values and intact rock strength (IRS) values for the gypsum determined from the detailed geotechnical borehole, and consequently the rock mass cohesion and friction angle derived from the GSI are consistent with the 'Expected' gypsum strength used by SRK for its predictive mine stability analyses carried out along both the L4900 and R179 roads and reported in October 2018. The borehole logging thus validates the strength estimate for the gypsum used previously and thus validates the outcome of the predictive analyses reported in October 2018. The 'Expected' Lower Gypsum strength values used in October 2018 analyses are presented below with the average Lower Gypsum strength value from Zones 1 to 5 used in the current analysis shown in brackets.

- GSI – 53 (50)
- Cohesion - 0.277 MPa (0.200MPa)
- Friction Angle - 47° (44°)

3.3 Finite Element Modelling

Twelve cross sections have been cut through the area at a number of locations for numerical analysis. The geology and mining limits along each section line were provided by the British Gypsum Technical Department. Where the cross sections intersect the laser scans, the true shape of the underground excavations, excluding the material tipped on the floor of the workings, has been included in the cross sections. Where there is no laser survey, the underground rooms have been assumed to be rectangular with a height of 6 m.

The Rocscience computer program RS2 2019 has been used for the analyses. Interpretation of the results has been with reference to contour plots of total displacement. The criterion of roof beam deflection of 2% of roof beam thickness is used to define roof beam instability.

3.4 Mine Water Level

In order to remain consistent with the predictive analyses presented as part of its October 2018 report, SRK has used the following mine water levels in its finite element analyses:

- 993 mRL: the mine water level at the time that the September 2018 mine collapse event occurred;
- 970 mRL: the maximum level historically maintained by mine dewatering prior to mine flooding.

3.5 3D Laser Scan Interpretation

The 3D laser scans provide an excellent view of the conditions of the rooms and pillars surveyed. The interpretation of the laser scans involved examining the shape of the underground workings. Workings with regular shaped rectangular openings, 'smooth' walls and roof and minimal floor debris indicate stable workings, Figure 3-2. The most common type of mine instability is roof slabbing (fall of rock material due to separation of layers along planes) with associated fallen debris. Block falls, where they have occurred, would likely be associated with irregular wall and roof profiles, Figure 3-3. The angular appearance of debris on the floor indicates that the blocks of rock may have fallen from the roof or walls of the workings. More rounded debris piles indicate that material on the floor is a remnant of the mining process or material that has tipped onto the floor.

The thickness of the gypsum roof beam above the excavations was also considered in each laser scan analysis to put in context the effect that any identified areas of roof slabbing may have on the overall stability of the excavation and therefore any potential effect on the R179 road.

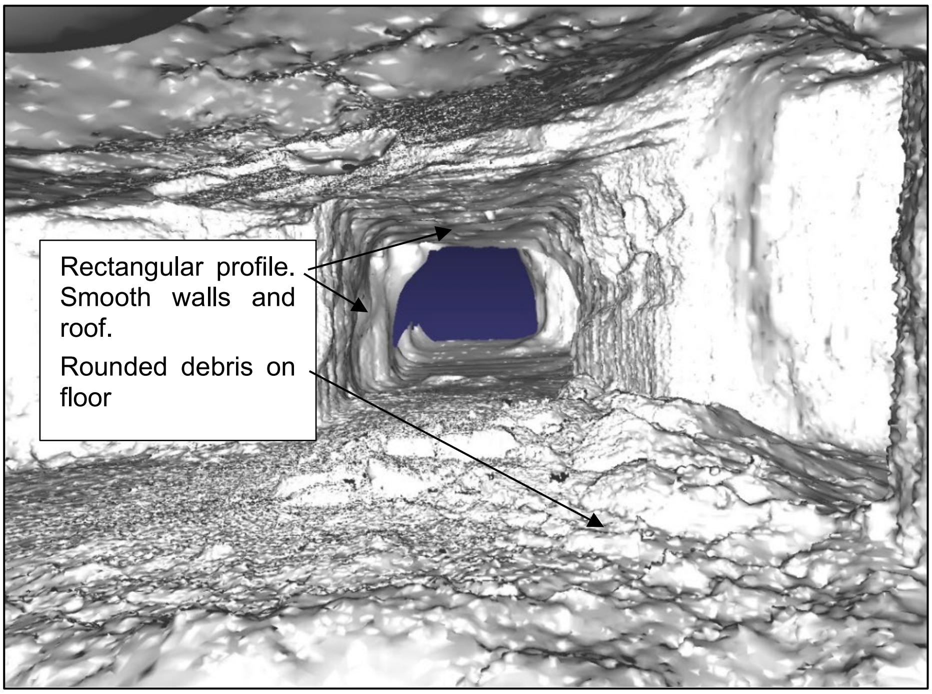

Figure 3-2: Example of Stable underground Workings

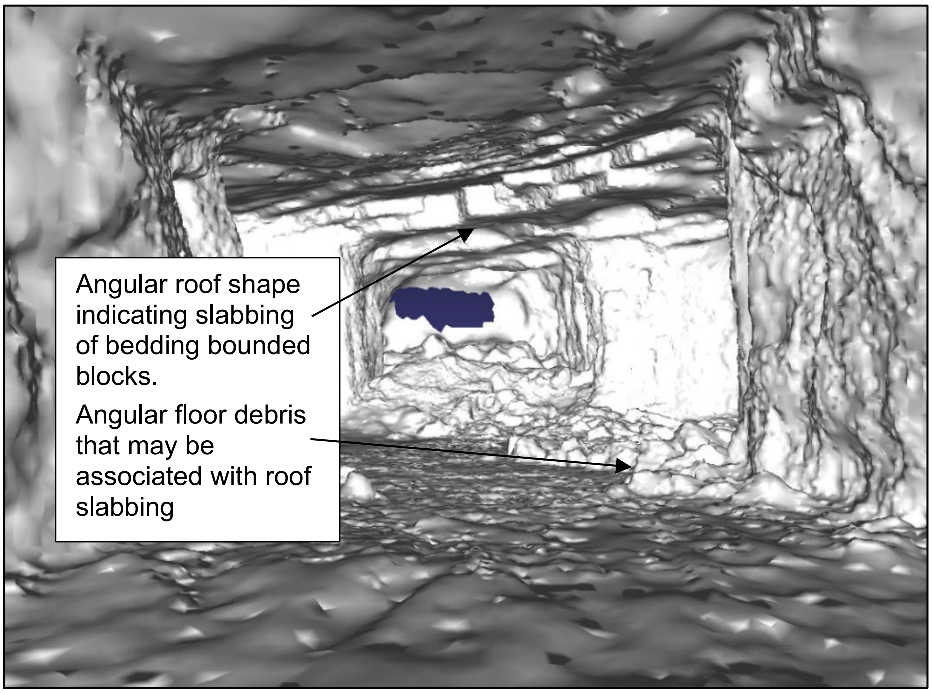

Figure 3-3: Example of Underground Workings Exhibiting Roof Slabbing

The laser scans were also imported into Leapfrog Geo® along with the mine plan survey maps for comparison. The overall alignment of the laser scans with the mine plan surveys is good, as can be seen in Figure 3-1 and in plan views of each individual zone in the following sections of this report. Additionally, the laser scans revealed that the actual underground excavations are generally narrower than the mine plan surveys indicate. This shows that in the areas surveyed the pillars are larger than shown on the mine plan.

The following sections use the laser scan images to help inform its interpretation of underground stability conditions and to support the results of the 2D numerical modelling. A more detailed analysis of the laser scans from each hole is given in Appendix D.

4 ZONE 1

4.1 General Description of Undermining

Figure 4-1 shows a plan view of Zone 1 as modelled using Leapfrog Geo 3D modelling software. The figure shows the underground workings survey, borehole collars and traces, the locations of cross-section used for later RS2 modelling, the 3D laser scan data, and the extents of the R179 road.

The length of the undermined road in this zone is approximately 75 m and the minimum depth of mine workings below surface that have been laser scanned is 40 m. The laser scans show that the room height of the underground workings is approximately 6 m. All excavations in Zone 1 lie above the maximum mine flooding elevation of 993 mRL.

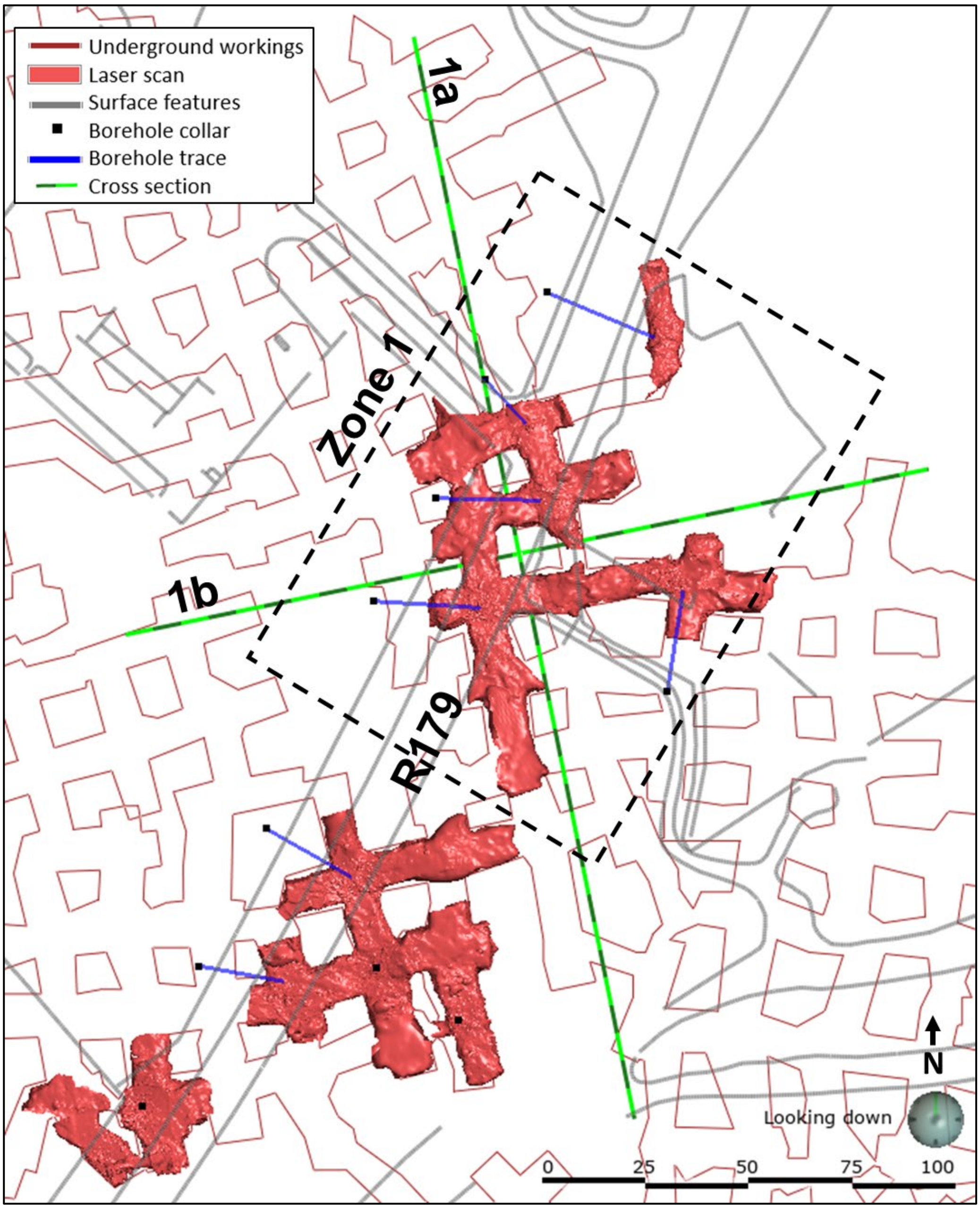

Figure 4-1: Plan view of Zone 1

4.2 Summary of Investigation Works

Table 4-1 shows details of the boreholes in Zone 1 and Figure 4-2 shows a plan view of their locations. The arrows and numbers in Figure 4-2 indicate the figure number and the direction of viewing of the laser scan images shown later in this section. Figure 4-3 is a cross-section along the R179 road showing detail of the boreholes and underground mining. For the purpose of this report, data from an additional hole, KC19-H24, which was drilled on a previous drilling programme, were also included in the analysis as it was deemed relevant.

Both upper and lower seams of gypsum are present in this zone, although the upper seam varies in thickness between 7 and < 1 m, with an average of 3.84 m. The thickness of the gypsum roof beam above the mine workings ranges between 9 m and 16 m, with an average of 11.75 m.

Table 4-1: Details of boreholes in Zone 1

BHID	Azimuth (°)	Dip (°)	Depth (m)	Easting	Northing	Elevation	Laser Scan?
KC20-R01	113	56	53	280966.51	300130.53	1054.37	Yes
KC19-R03	91	59	50	280939.51	300080.79	1057.97	Yes
KC19-R04	94	59	50	280924.73	300055.92	1058.58	Yes
KC20-R15	8.9	58	46	280995.67	300034.34	1056.93	Yes
KC19-H24	137	71	50	280951.6	300109.5	1055.07	Yes

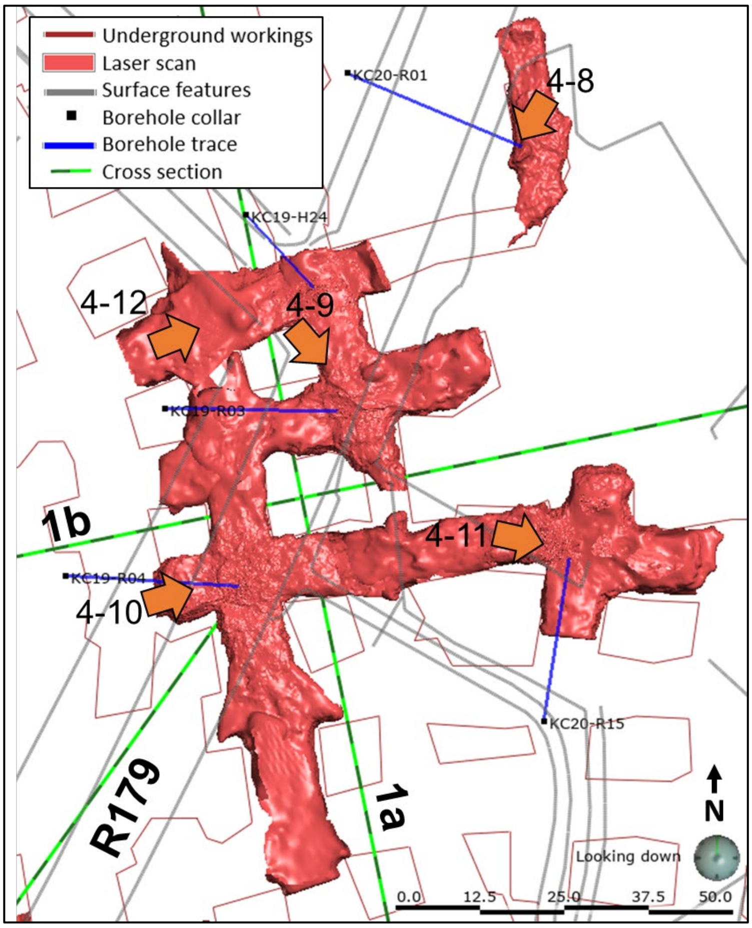

Figure 4-2: Zone 1 showing borehole collars and laser scan image locations

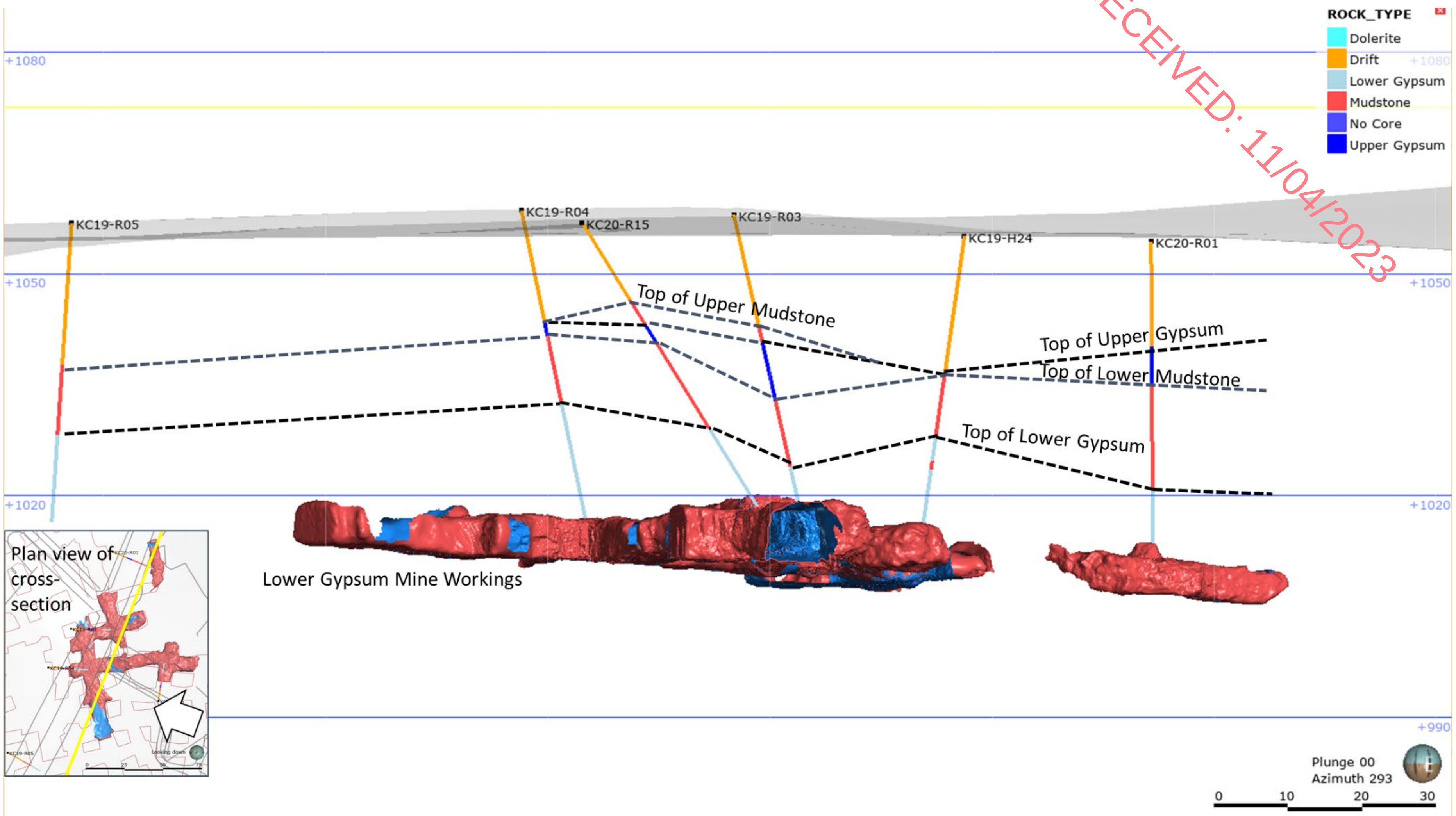

Figure 4-3: Zone 1: NE-SW section along R179 road showing borehole geology and underground mining

4.3 Finite Element Modelling

4.3.1 Model Geometry

The finite element models analysed are shown in Figure 4-4 and Figure 4-5. Each cross section shows the lithologies, the location of the R179 road, the mine working, the depth to the mine workings below the road and the thickness of the gypsum roof beam above the workings closest to the R179 road. At this location, the workings lie approximately 39 m below the road and the gypsum roof beam is about 9 m to 10 m thick. Note that the irregular shaped mine workings are based on the underground laser survey. The figures show that all the workings immediately below the road on both cross sections have been laser surveyed, therefore their position and size are accurate. The regular shaped rooms have been positioned from the 2D mine survey and a nominal height of 6 m applied.

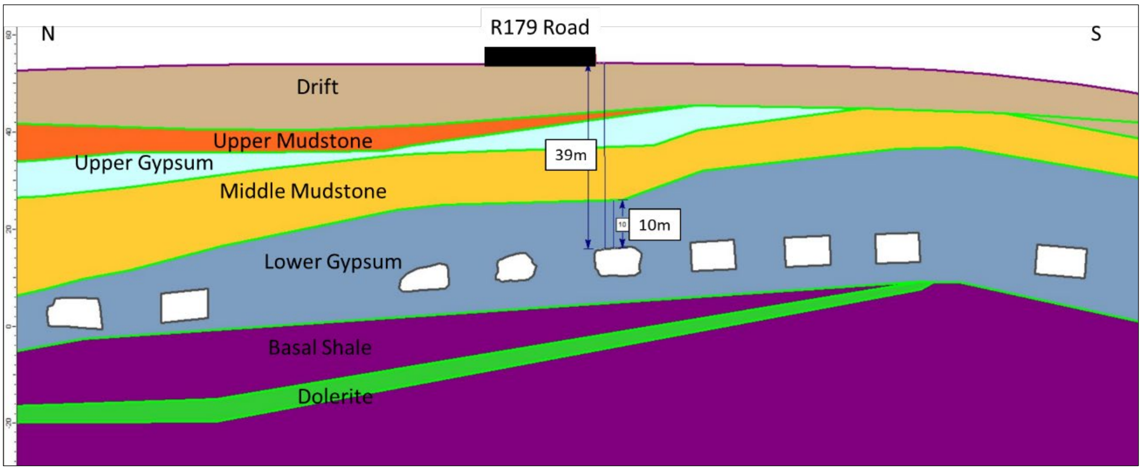

Figure 4-4: Cross Section 1a

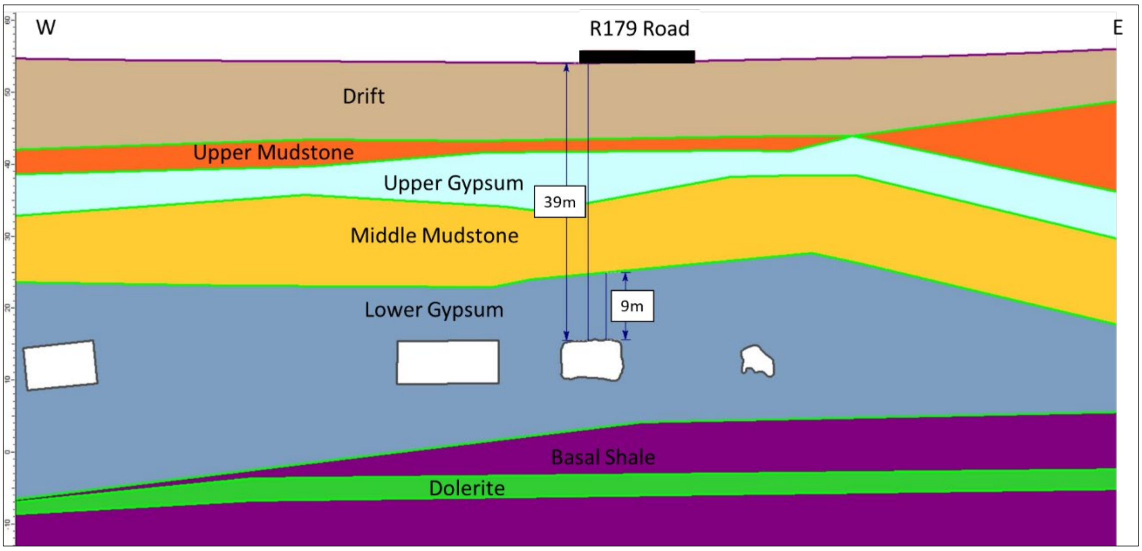

Figure 4-5: Cross Section 1b

4.3.2 Results

The results of the numerical modelling are reproduced in Figure 4-6 and Figure 4-7. These are contour plots of total displacement in metres. As well as the contours of total displacement, the plots show the predicted displacement close to surface below and adjacent to the R179 road. Also shown is the maximum displacement at the underside of the roof beam above the workings which lie immediately below the road.

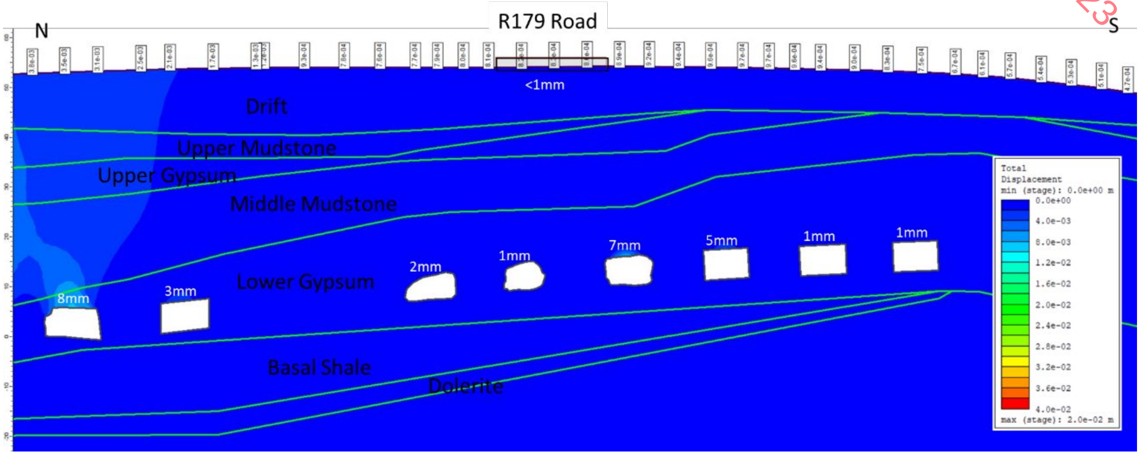

Figure 4-6: Cross Section 1a: Total Displacement

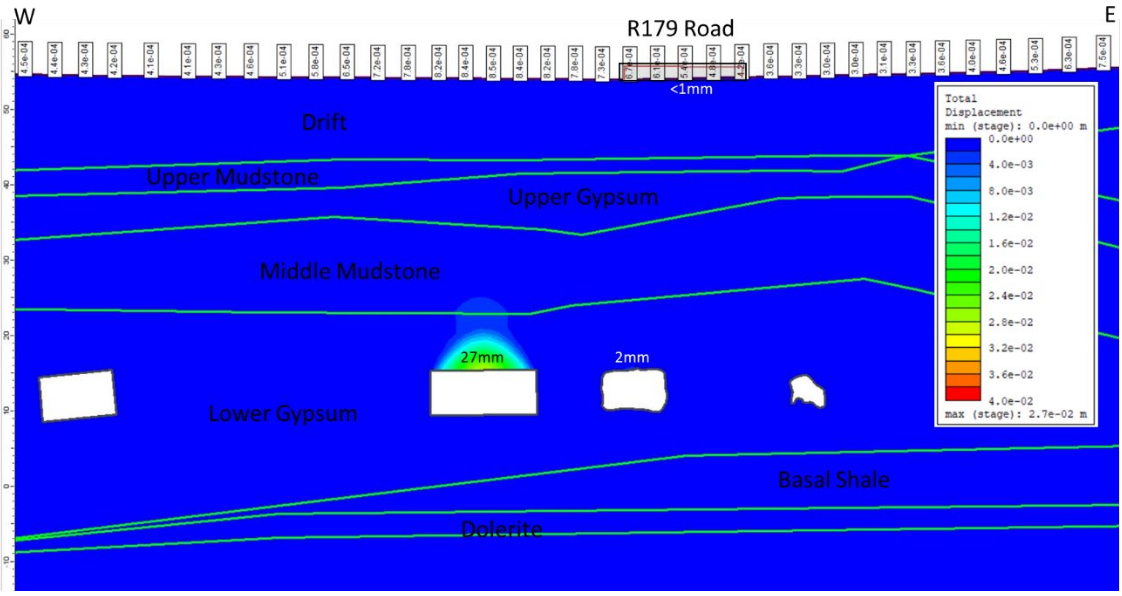

Figure 4-7: Cross Section 1b: Total Displacement

For both cross sections, the numerical model defines stable pillar conditions. For cross section 1a, the maximum roof beam deflection is 7 mm (0.007 m) which for the 10 m thick gypsum roof beam gives a maximum deflection of <0.1% of roof beam thickness. On surface, below and adjacent to the R179 road, the maximum simulated surface deformation is <1 mm. For cross section 1b, the maximum roof beam deflection is 27 mm (0.027 m) which for the 9 m thick gypsum roof beam gives a maximum deflection of 0.3% of roof beam thickness. On surface, below and adjacent to the R179 road, the maximum simulated surface deformation is <1 mm.

4.4 Laser Scan Surveys

Figure 4-8 through to Figure 4-11 show laser scan images from the boreholes in Zone 1; the exact locations of the views shown are indicated by the orange arrows in Figure 4-2.

Figure 4-8 shows the view looking south west from the laser scan of KC20-R01, where the depth to the mine workings is 44.03 m and the thickness of the gypsum roof beam is 10.52 m. The workings lie approximately 5 m to the east below the R179 road. A roof hole is clearly visible on the right sidewall and associated debris can be seen directly below this hole. The hole extends approximately 2.50 m into the roof, indicating that approximately 8.00 m of intact gypsum roof beam remains, which is likely to prevent further propagation of the hole to the upper beds. The remaining sidewalls and roof do not show any other obvious signs of instability.

Figure 4-9 shows the view looking south east from the laser scan of KC19-R03, where the depth to the mine workings is 43.25 m and the thickness of the gypsum roof beam is 9.20 m. The view is of an intersection that lies directly below the R179 road. There is some evidence of minor slabbing in the roof above the debris of approximately 0.4 m thickness, although this is unlikely to significantly reduce roof beam strength. Large amounts of debris are visible on the floor below the intersection; however, the hummocky shape of the debris and also its large volume suggest that it is soft material that has been tipped by a loader rather than material that has fallen from the roof.

Figure 4-10 shows the view looking east from the laser scan of KC19-R04, where the depth to the mine workings is 42.63 m and the thickness of the gypsum roof beam is 16.51 m. The view is of an intersection that lies directly below the R179 road. The roof and pillars are in good condition with no obvious signs of instability. There is no indication that roof failure is the cause of the debris on the floor in the centre back of the image (this appears to be fine waste material or a similar tipped material).

Figure 4-11 shows the view looking east from the laser scan of KC20-R15, where the depth to the mine workings is 38.37 m and the thickness of the gypsum roof beam is 10.53 m. The view is of an intersection that lies approximately 30 m to the east below the R179 road. Although there is some evidence of minor instability shown by the roof overbreak, debris on the floor on either side of the excavation and bedding planes are clearly evident in the roof, the change in roof height is likely due to the design of the excavation and not the failure; the north-south trending excavation in the back of the image is deeper than the east-west trending excavation in the foreground of the image, requiring a sloped excavation roof.

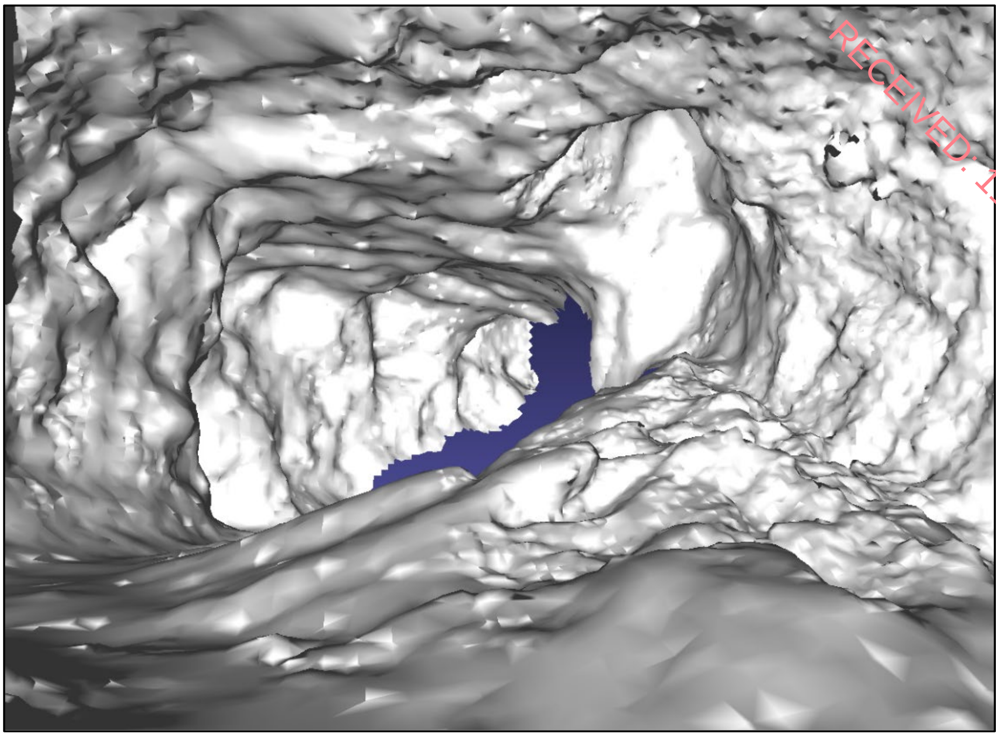

Figure 4-8: Laser scan image from KC20-R01 looking south west

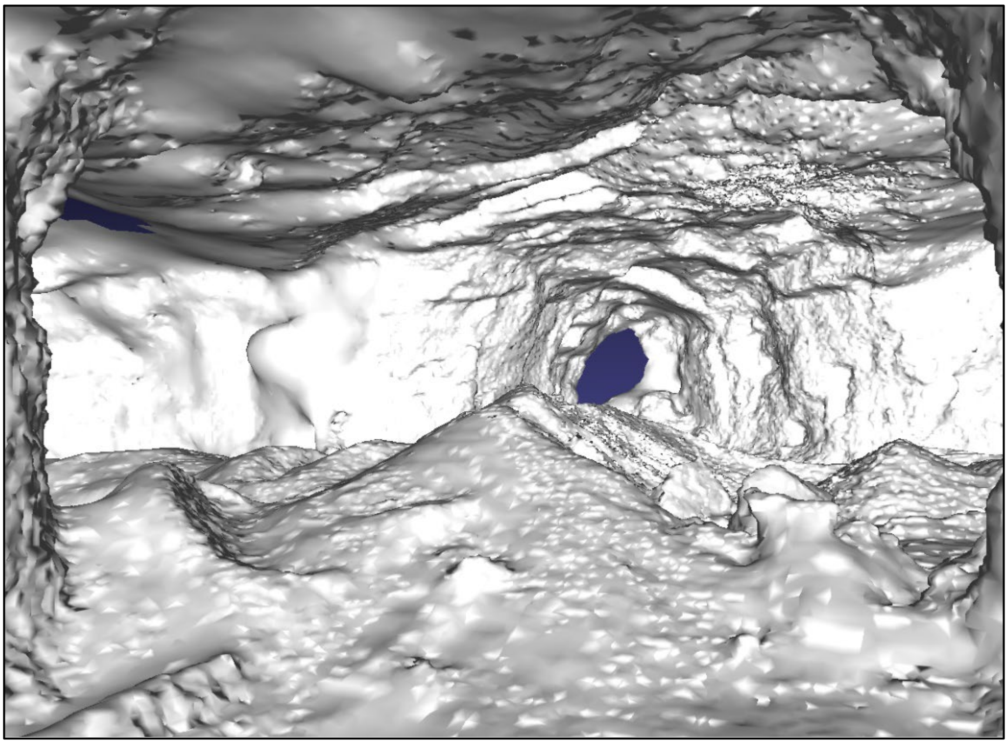

Figure 4-9: Laser scan image from KC19-R03 looking south east

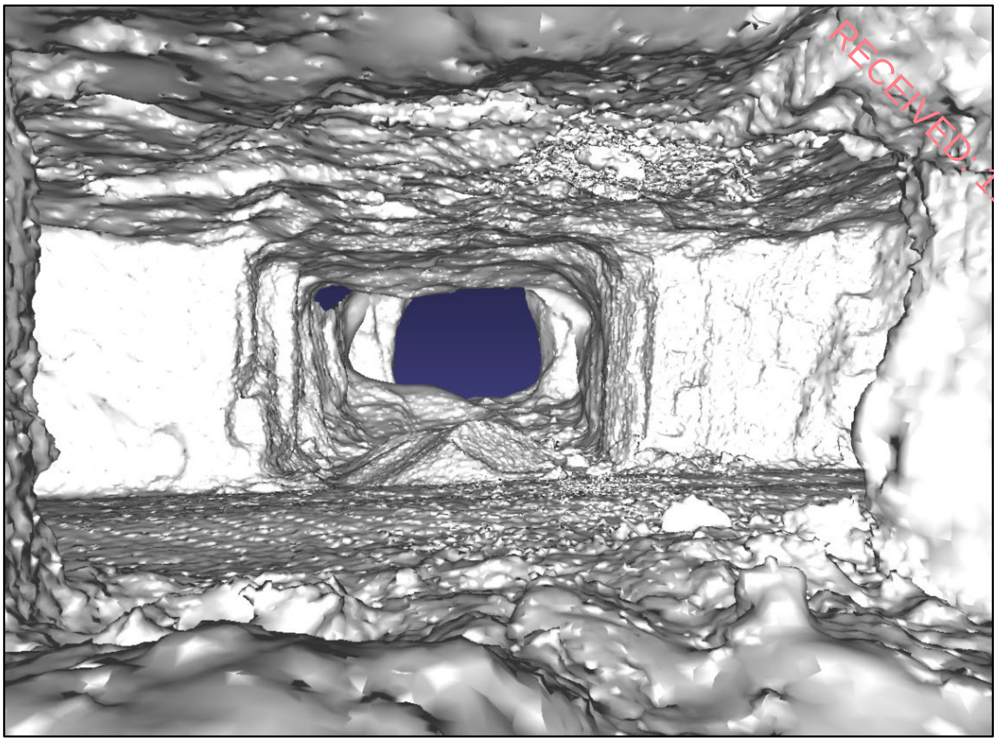

Figure 4-10: Laser scan image from KC19-R04 looking east

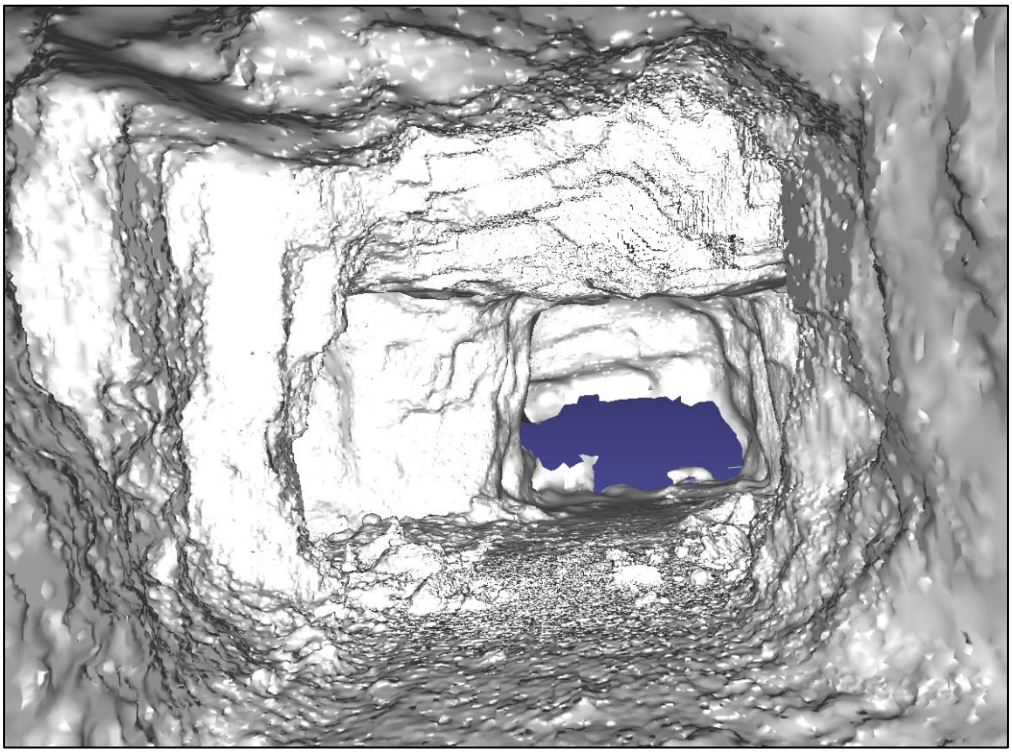

Figure 4-11: Laser scan image from KC19-R15 looking east

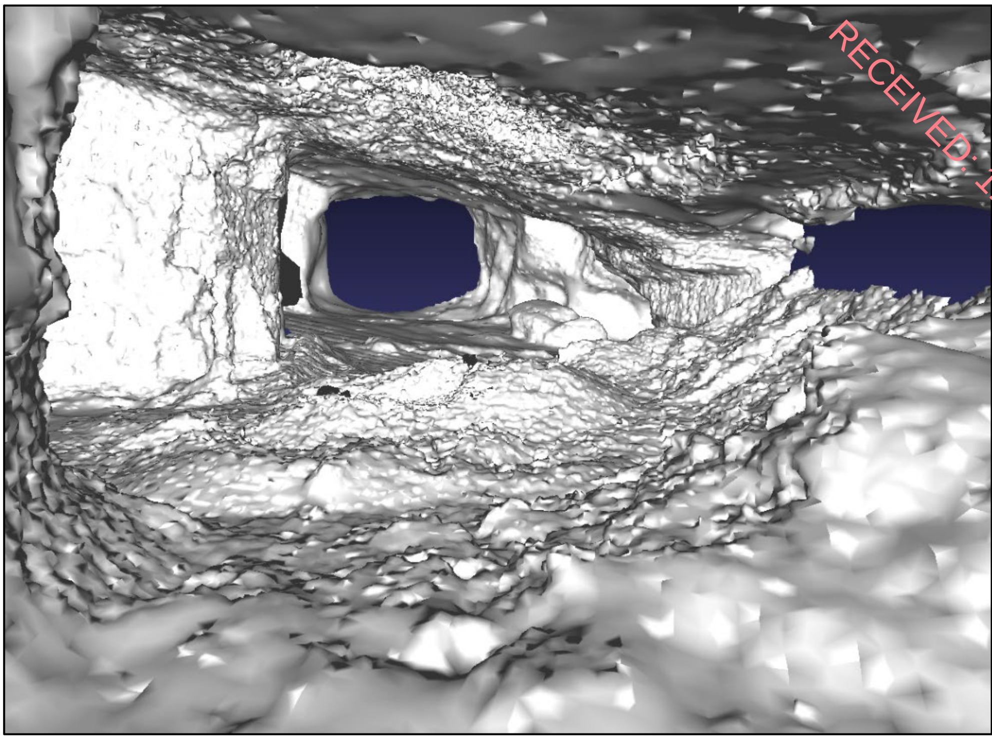

Figure 4-12: Laser scan image from KC19-H24 looking east

5 ZONE 2

5.1 General Description of Undermining

Figure 5-1 shows a plan view of Zone 2 as modelled using Leapfrog Geo 3D modelling software. The figure shows the underground workings survey, borehole collars and traces, the locations of cross-section used for later RS2 modelling, the 3D laser scan data, and the extents of the R179 road.

The length of the undermined road in this zone is approximately 120 m and the minimum depth of mine workings below surface that have been laser scanned is 37 m. The laser scans show that the room height of the underground workings is approximately 6 m, although there is one 11 m high room in the workings to the east between cross-section lines 2a and 2b (see Figure 5-1). All excavations in Zone 2 lie above the maximum mine flooding elevation of 993 mRL and are therefore workings that have been dry for the life of the mine .

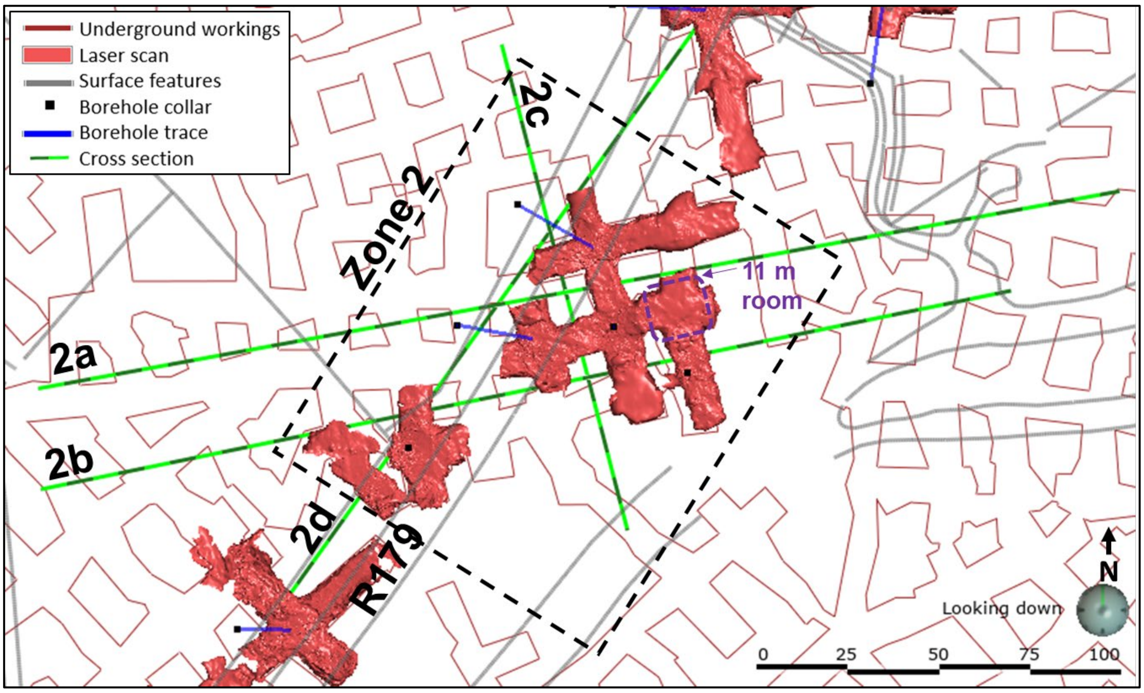

Figure 5-1: Plan view of Zone 2

5.2 Summary of Investigation Works

Table 5-1 shows details of the boreholes in Zone 2 and Figure 5-2 shows a plan view of their locations. The arrows and numbers in Figure 5-2 indicate the figure number and the direction of viewing of the laser scan images shown later in this section.

Both upper and lower seams of gypsum are present in this zone, although the upper seam was only present in KC19-R07 and KC18K-R08 and not encountered in KC19-R05, KC20-R06, or KC20-R06A to the north east. The thickness of the upper seam increases from 3.64 m in KC19-R07 to 4.28 m in KC18K-R08. The thickness of the gypsum roof beam above the mine workings ranges between 9 m and 14 m, with an average of 12 m. Figure 5-3 is a cross-section along the R179 road showing detail of the boreholes and underground mining.

Table 5-1: Details of the boreholes drilled in Zone 2

BHID	Azimuth (°)	Dip (°)	Depth (m)	Easting	Northing	Elevation	Laser Scan?
KC19-R05	120	59	47	280898.69	300000.98	1056.85	Yes
KC20-R06	58	88	40	280925.24	299967.43	1055	Yes
KC20-R06A	332.9	89.7	35	280945.25	299954.79	1051.76	Yes
KC19-R07	100	63	46	280882.26	299967.86	1055.51	Yes
KC18K-R08	0	90	41.2	280868.83	299934.01	1053.38	Yes

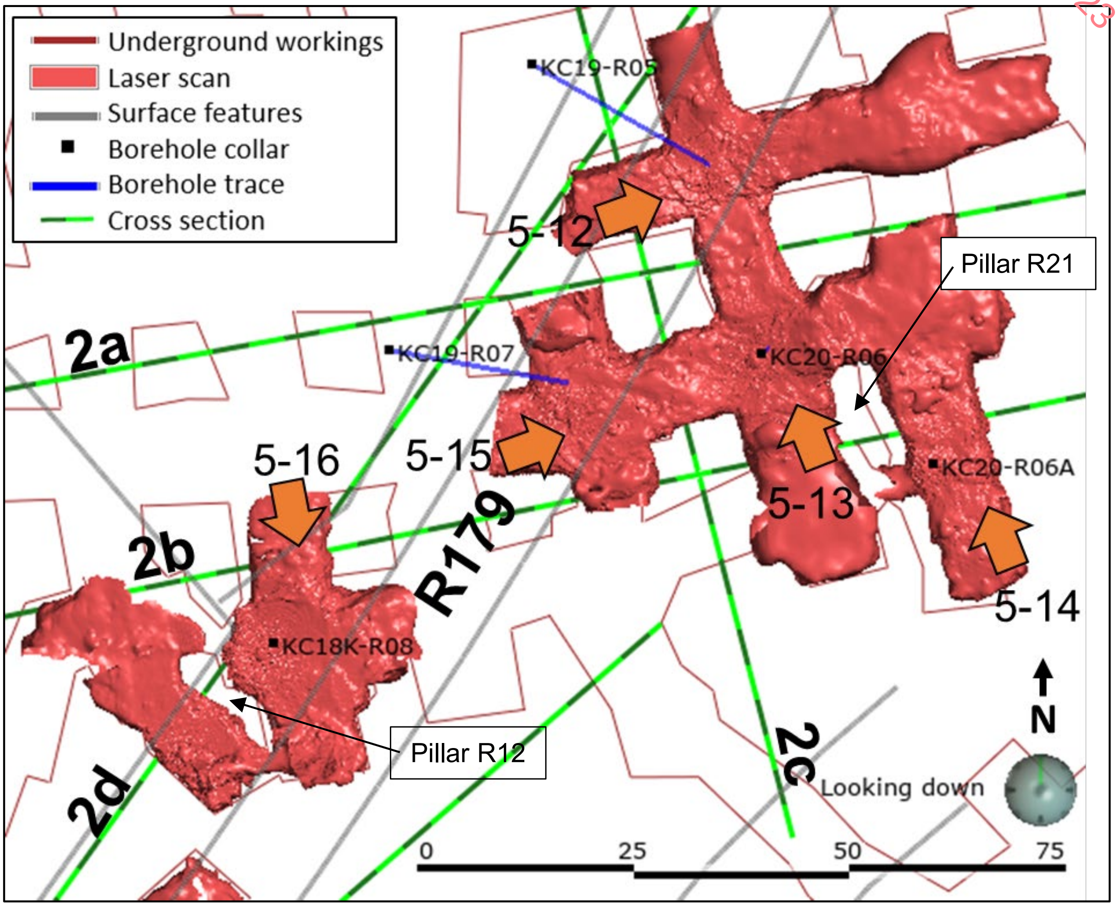

Figure 5-2: Zone 2 showing borehole collars and laser scan image locations

Some of the boreholes were drilled to investigate the size and shape of two small pillars. Pillar R12 is located adjacent to the north western edge of the R179. Pillar R21 is located about 20 m to the south east of the R179 carriageway. The location of both pillars and surrounding laser scans can be seen in Figure 5-2. It can be seen that the laser scan outline of Pillar R12 closely matches its shape on the mine survey plan. The laser scan of Pillar R21, however, shows that the pillar width is somewhat larger than is shown by the mine survey plan. Its plan area is between 30% and 40% greater than that shown by the mine plan.

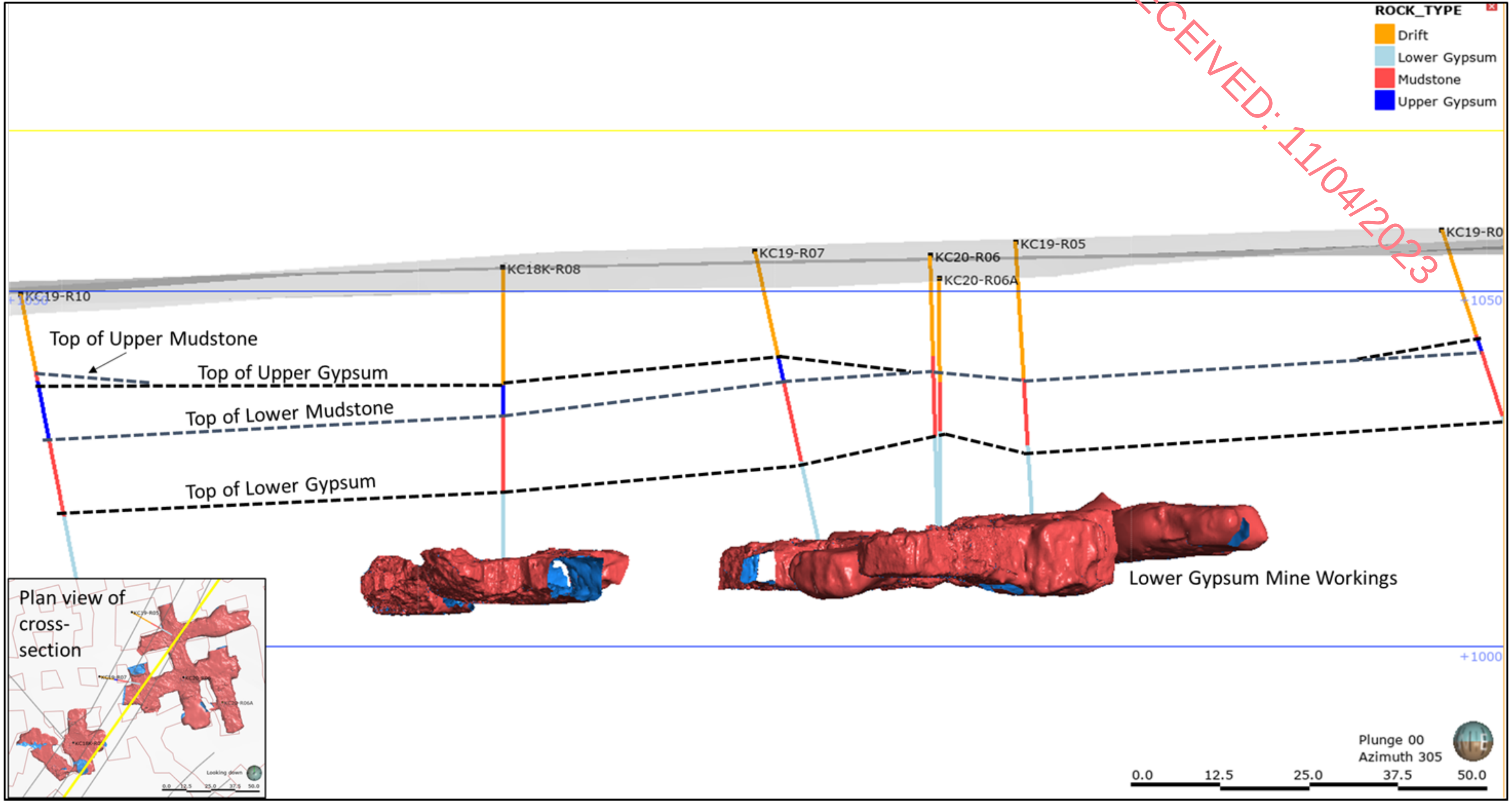

Figure 5-3: Zone 2: NE-SW section along R179 road showing borehole geology and underground mining

5.3 Finite Element Modelling

5.3.1 Model Geometry

The finite element models analysed for Zone 2 are shown in Figure 5-4, Figure 5-5, Figure 5-6, and Figure 5-7. Each cross section shows the lithologies, the location of the R179 road, the mine working, the depth to the mine workings below the road, and the thickness of the gypsum roof beam above the workings closest to the R179 road. At this location, the workings lie between 36 m and 38 m below the road and the gypsum roof beam is between 12 m and 13 m thick. Note that cross section 2d has been located to intersect Pillar R12 at its narrowest point. This cross section cuts the R179 road at an oblique angle, therefore the underground workings modelled are located between 5 m and 10 m to the west of the road. Note that the irregular shaped mine workings are based on the underground laser survey. Four cross sections are included as this zone contains the highest number of laser surveys.

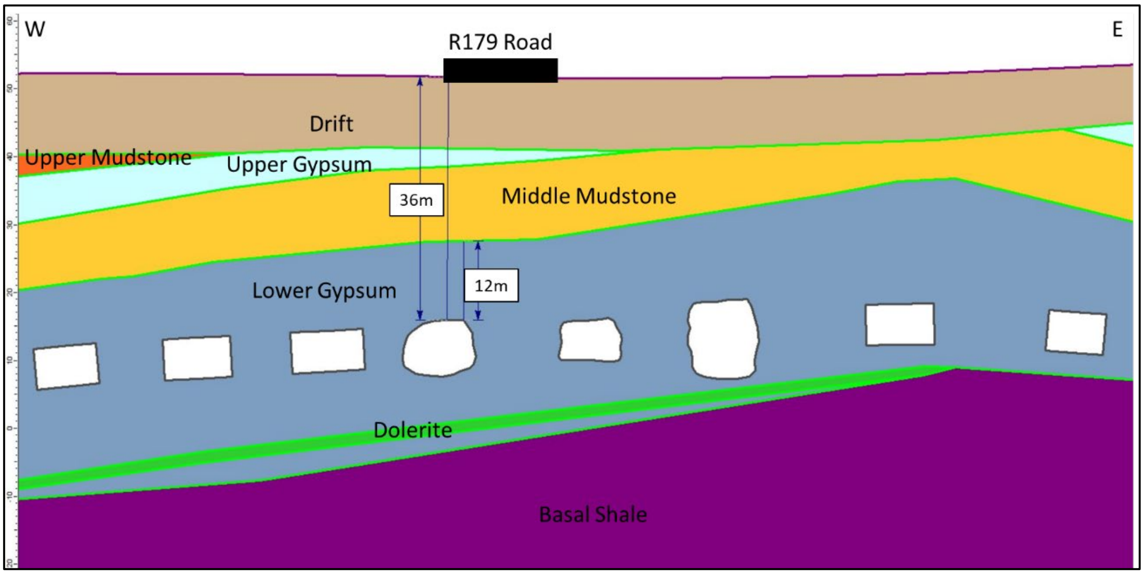

Figure 5-4: Cross Section 2a

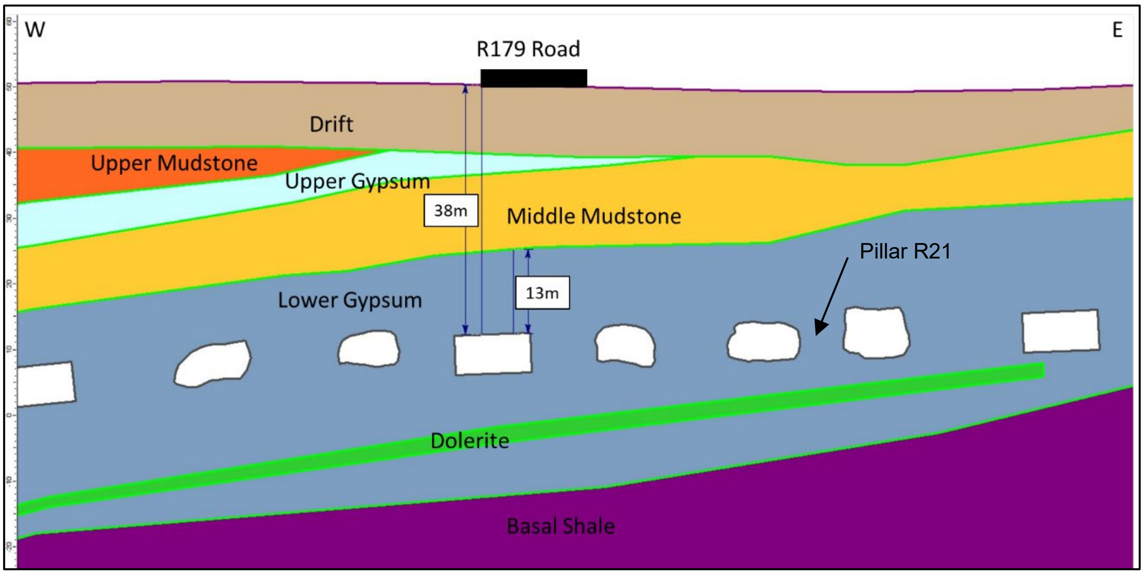

Figure 5-5: Cross Section 2b

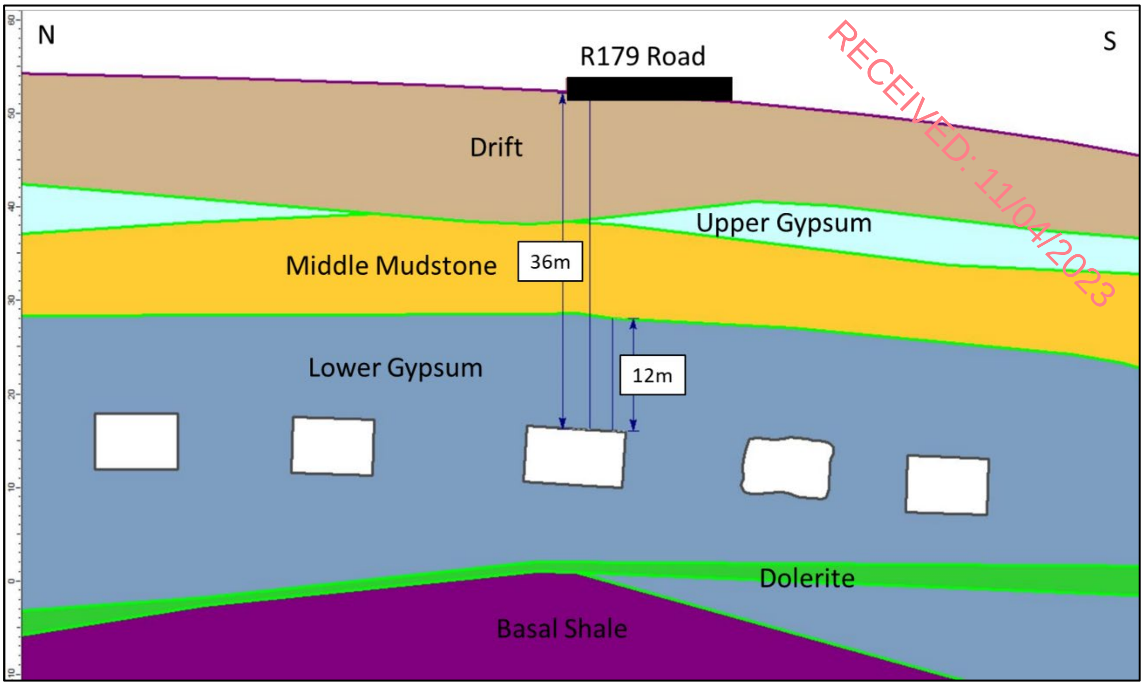

Figure 5-6: Cross Section 2c

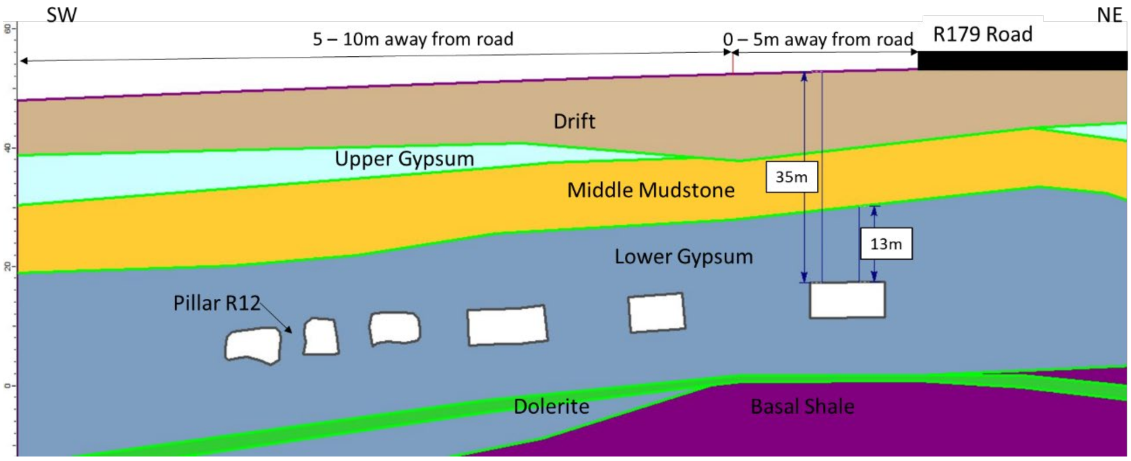

Figure 5-7: Cross Section 2d

5.3.2 Results

The results of the numerical modelling are reproduced in Figure 5-8, Figure 5-9, Figure 5-10, and Figure 5-11. These are contour plots of total displacement in metres. As well as the contours of total displacement the plots show the predicted displacement close to surface below and adjacent to the R179 road. Also shown is the maximum displacement at the underside of the roof beam above the workings which lie immediately below the road.

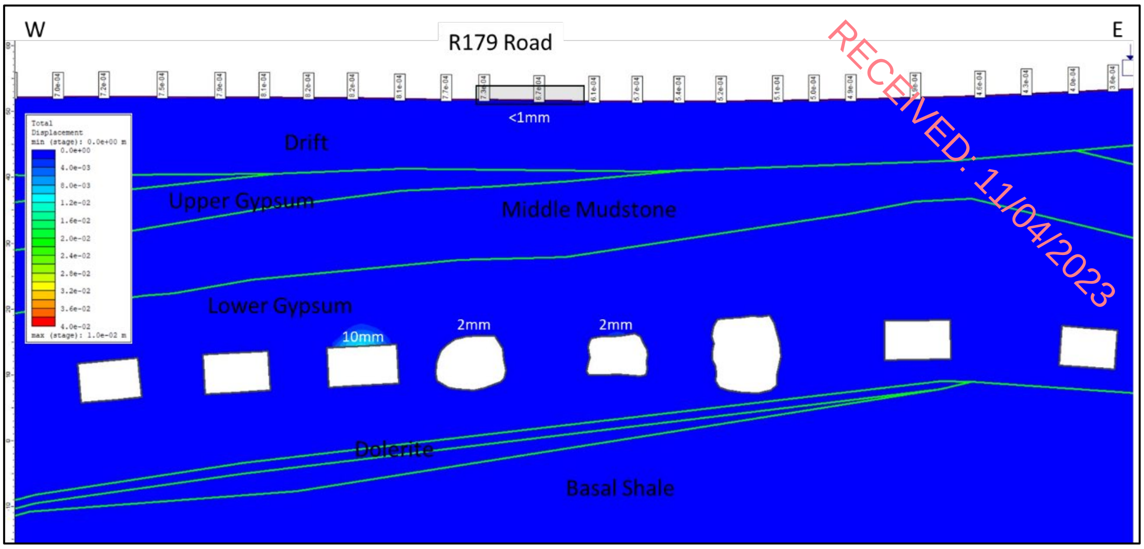

Figure 5-8: Cross Section 2a: Total Displacement

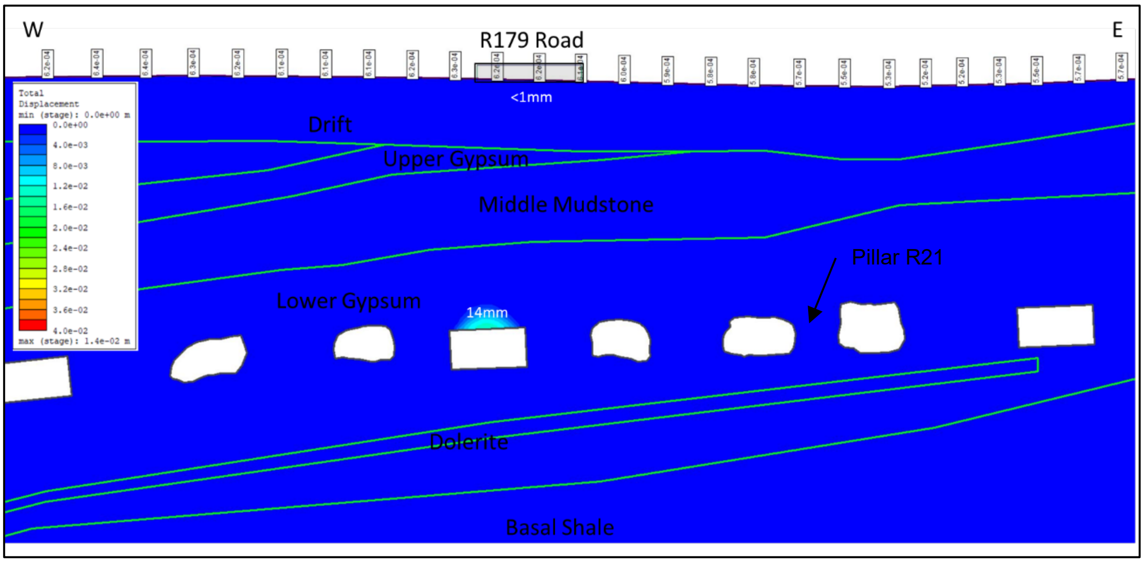

Figure 5-9: Cross Section 2b: Total Displacement

For cross sections 2a and 2b, the numerical model defines stable pillar conditions. For cross section 2a, the maximum roof beam deflection is 10 mm (0.01 m) which, for the 12 m thick gypsum roof beam, gives a maximum deflection of <0.1% of roof beam thickness. On surface, below and adjacent to the R179 road, the maximum simulated surface deformation is <1 mm. There is no indication of deformation or instability around the 11 m high four way intersection. There is no indication of deformation or instability of Pillar R21. For cross section 2b, the maximum roof beam deflection is 14 mm (0.014 m) which, for the 13 m thick gypsum roof beam, gives a maximum deflection of 0.1% of roof beam thickness. On surface, below and adjacent to the R179 road, the maximum simulated surface deformation is <1 mm.

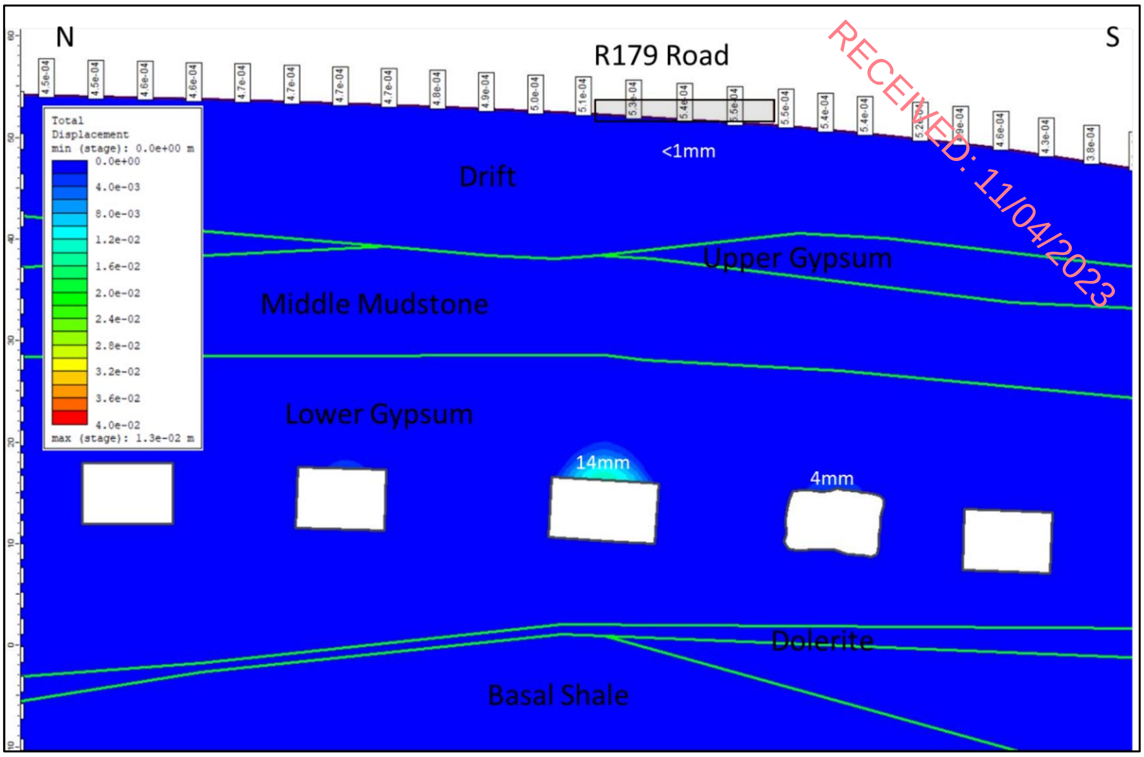

Figure 5-10: Cross Section 2c: Total Displacement

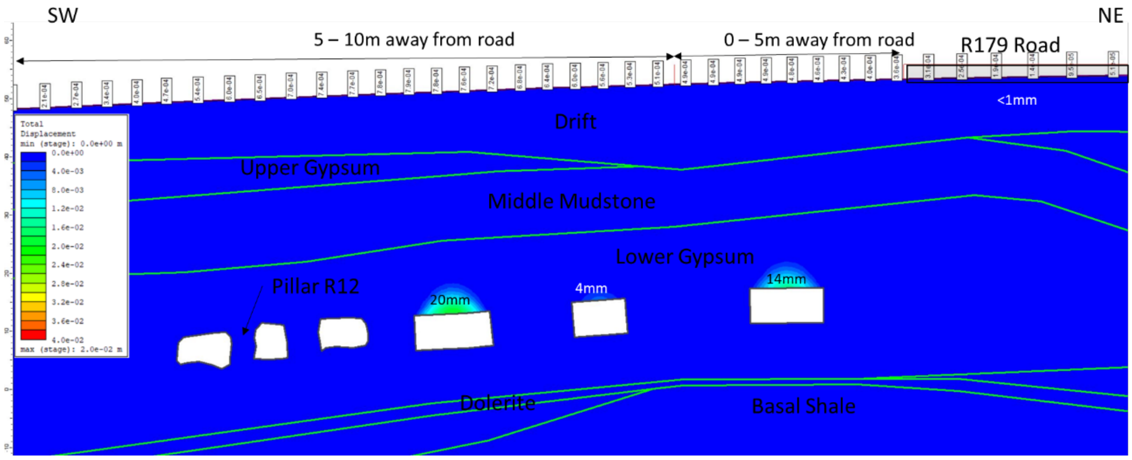

Figure 5-11: Cross Section 2d: Total Displacement

For cross sections 2c and 2d, the numerical model defines stable pillar conditions. For cross section 2c, the maximum roof beam deflection is 14 mm (0.014 m) which, for the 12 m thick gypsum roof beam, gives a maximum deflection of 0.1% of roof beam thickness. On surface, below and adjacent to the R179 road, the maximum simulated surface deformation is <1 mm. On cross section 2d, there is no indication of instability of Pillar R12. The maximum roof beam deflection is 20 mm (0.02 m) which, for the 13 m thick gypsum roof beam, gives a maximum deflection of 0.2% of roof beam thickness. On surface, below and adjacent to the R179 road, the maximum simulated surface deformation is <1 mm.

5.4 Laser Scan Surveys

Figure 5-12 through to Figure 5-16 show laser scan images from the boreholes in Zone 2; the exact locations of the views shown are indicated by the orange arrows in Figure 5-2.

Figure 5-12 shows the view looking east from the laser scan of KC19-R05, where the depth to the mine workings is 40.46 m and the thickness of the gypsum roof beam is 11.85 m. The view is of an intersection that lies directly below the R179 road. Although there is some debris on the excavation floor, there is no evidence that this is related a failure as both the roof and sidewalls/pillars appear in good condition with no signs of instability.

Figure 5-13 shows the view looking north east from the laser scan of KC20-R06, where the depth to the mine workings is 39.50 m and the thickness of the gypsum roof beam is 14.09 m. The view is of an intersection that lies approximately 5 m to the east below the R179 road. The sidewalls and roof are in good condition. The roof is dipping slightly with the bedding and some slabbing of approximately 0.3 m may have occurred, although this is not significant and unlikely to affect the roof strength. The debris directly below on the excavation floor may be a result of this slabbing.

Figure 5-14 shows the view looking north from the laser scan of KC19-R06A, where the depth to the mine workings is 35.00 m and the thickness of the gypsum roof beam is 13.46 m. The view is of an intersection that lies 25 m to the south east below the R179 road, although the excavation leading to the right is a dead end. Although the room in the centre of the four-way junction is between 10 m and 11 m high, the pillars surrounding it are all 5 m to 6 m high. The roof and sidewalls are in good condition with no evidence of instability.

Figure 5-15 shows the view looking east from the laser scan of KC20-R07, where the depth to the mine workings is 40.84 m and the thickness of the gypsum roof beam is 11.29 m. The view is of an intersection that lies directly below the R179 road. The roof and sidewalls are in good condition with no evidence of instability; the debris on the floor is likely to be a result of drilling.

Figure 5-16 shows the view looking south from the laser scan of KC20-R08, where the depth to the mine workings is 41.20 m and the thickness of the gypsum roof beam is 9.60 m. The view is of an intersection that lies below the R179 road, although the excavation at the back of the image is a dead end. The roof is in good condition with no signs of instability; however, there is evidence of debris near the sidewalls, at the base of Pillar R12. This could be due to sidewall slabbing, although it could equally be dumped or blasted material.

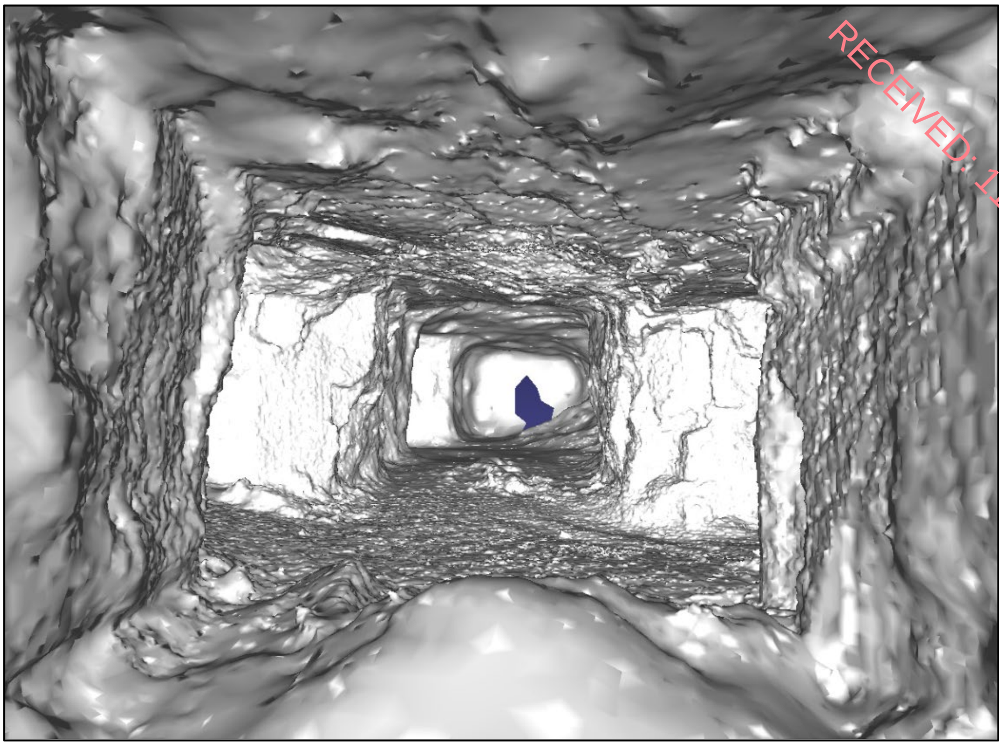

Figure 5-12: Laser scan image from KC19-R05 looking east

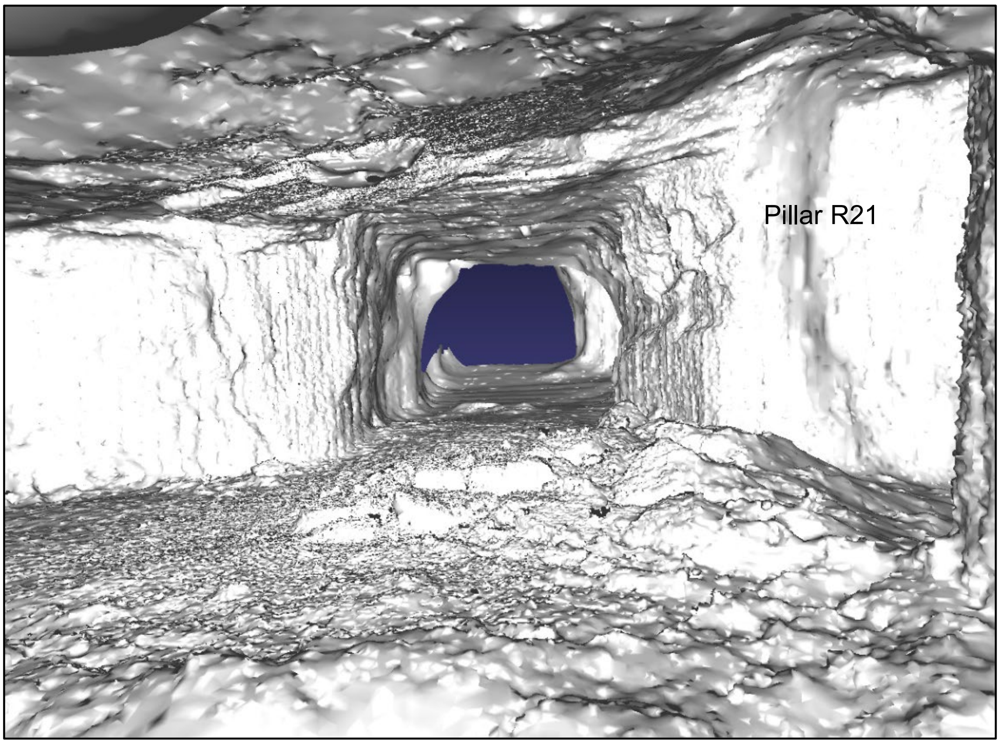

Figure 5-13: Laser scan image from KC20-R06 looking north east

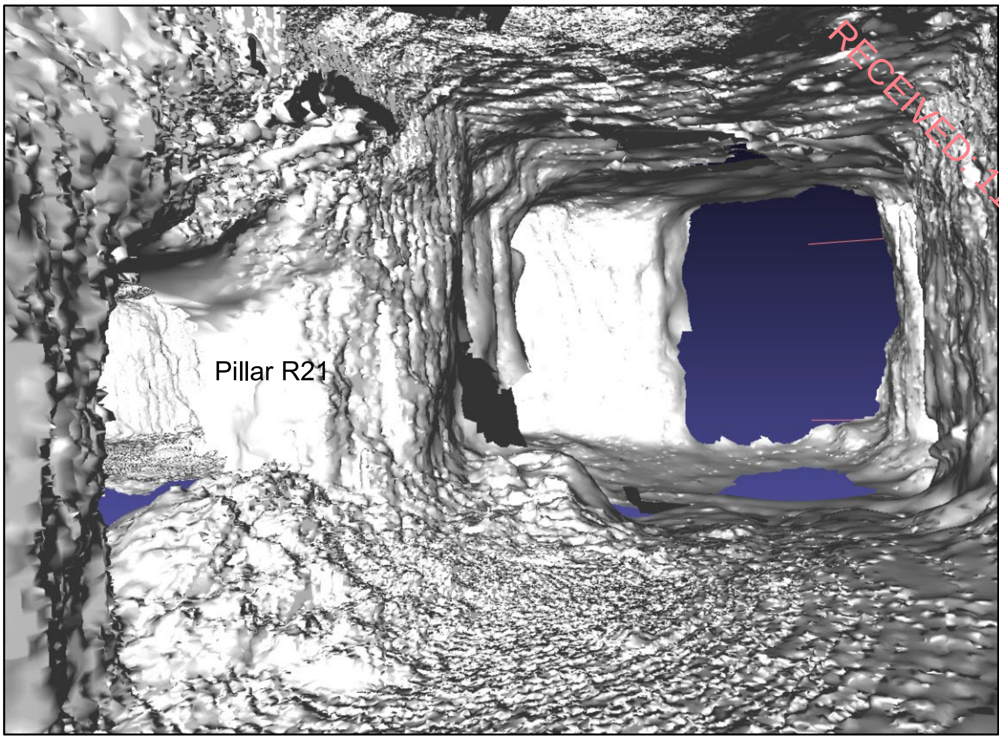

Figure 5-14: Laser scan image from KC20-R06A looking north

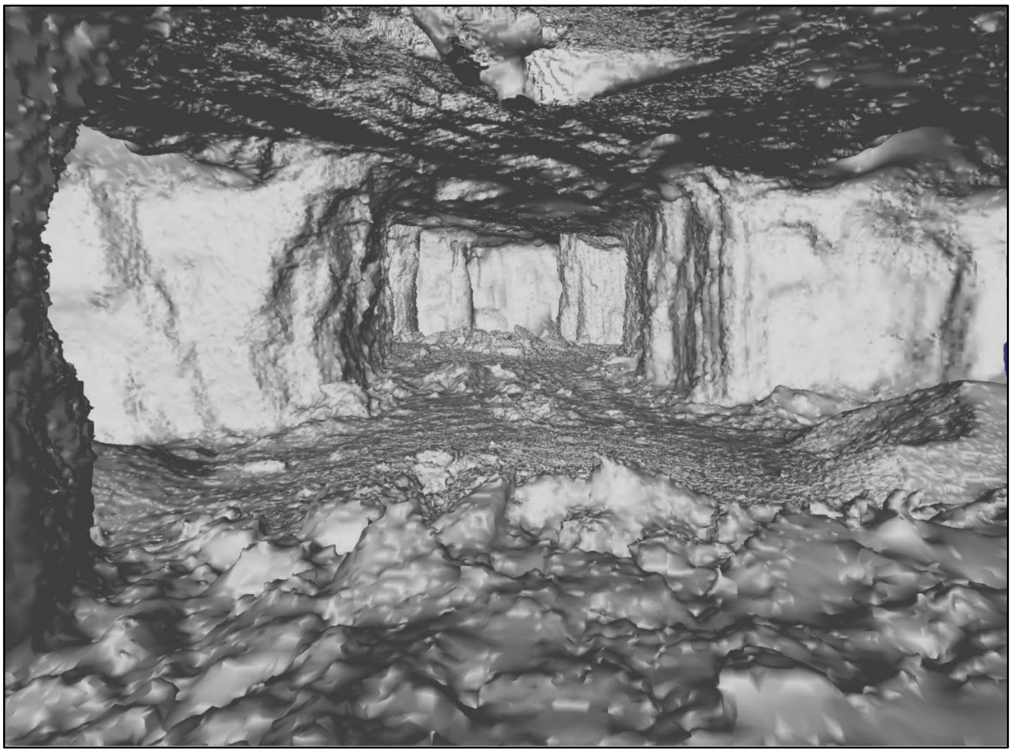

Figure 5-15: Laser scan image from KC20-R07 looking east

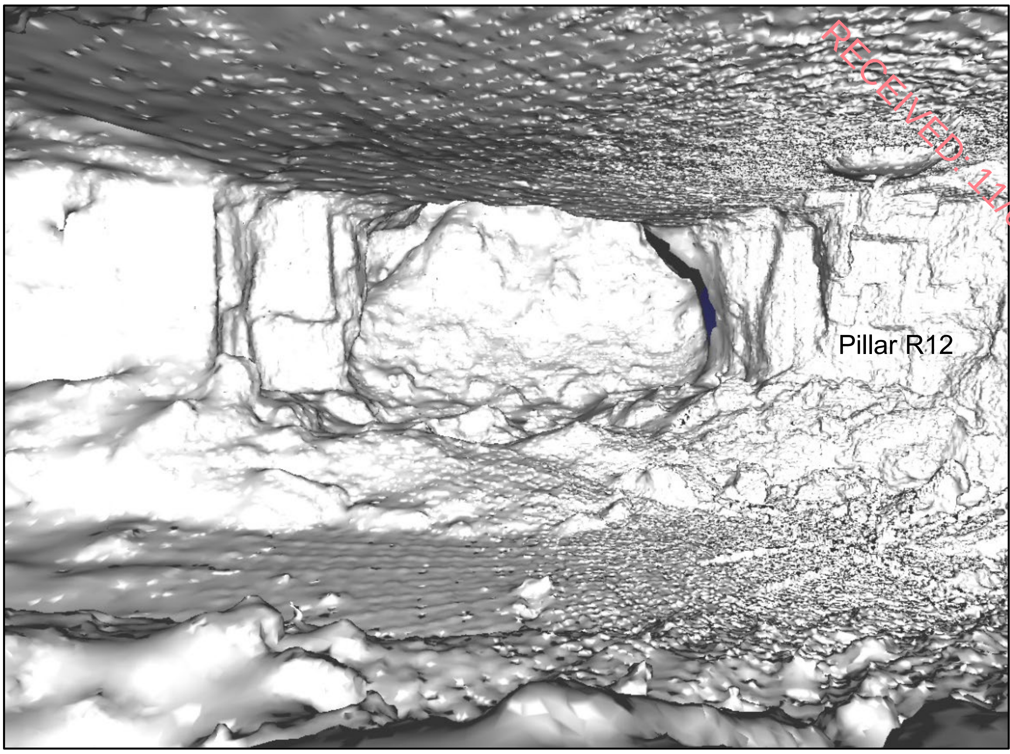

Figure 5-16: Laser scan image from KC20-R08 looking south

6 ZONE 3

6.1 General Description of Undermining

Figure 6-1 shows a plan view of Zone 3 as modelled using Leapfrog Geo 3D modelling software. The figure shows the underground workings survey, borehole collars and traces, the locations of cross-section used for later RS2 modelling, the 3D laser scan data, and the extents of the R179 road.

The length of the undermined road in this zone is approximately 120 m and the minimum depth of mine workings below surface that have been laser scanned is 43 m. The laser scans show that the room height of the underground workings is approximately 6 m. Some excavations in Zone 3 lie between the maximum mine flooding elevation of 993 mRL and the historical maximum mine water level of 970 mRL.

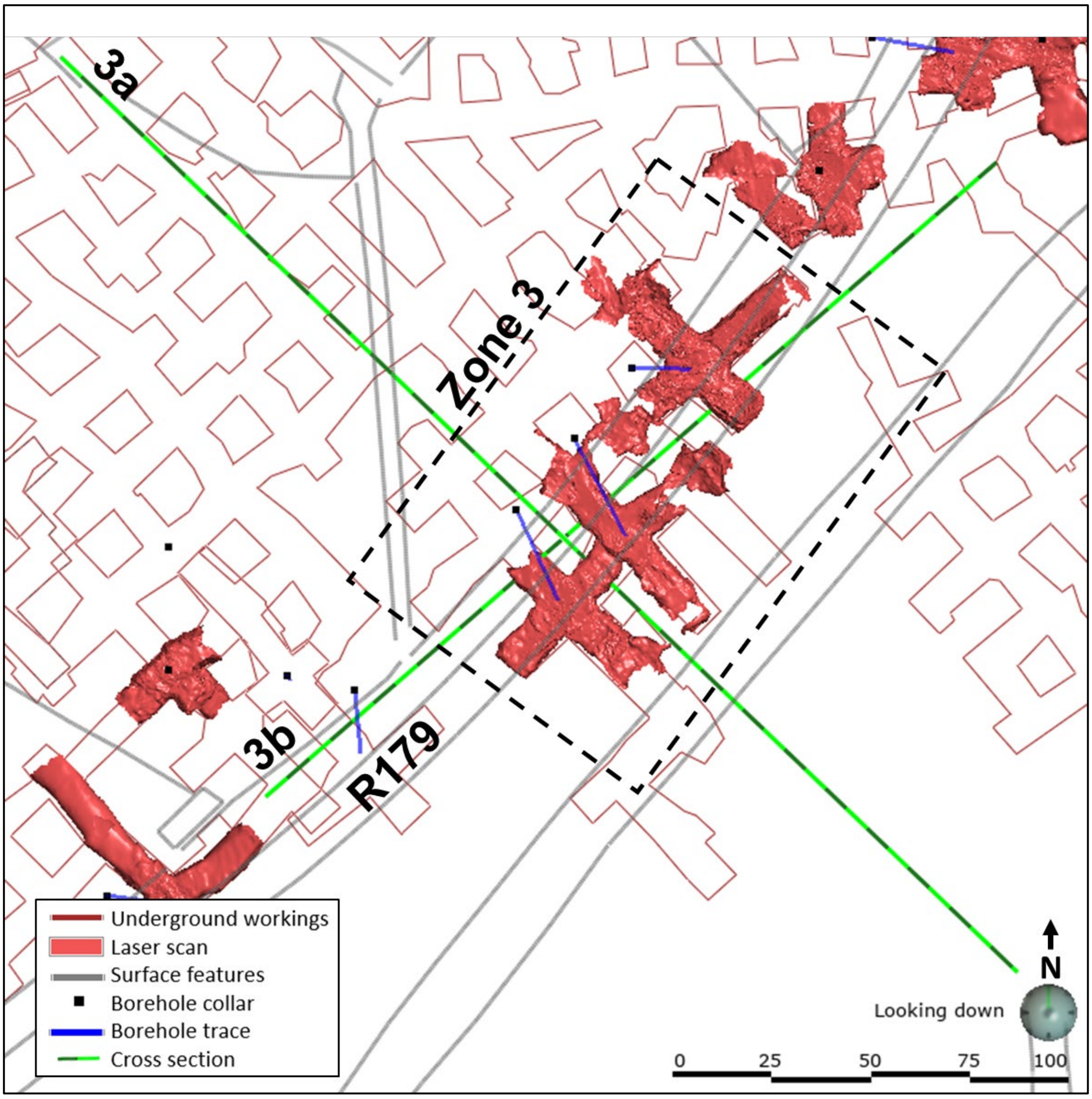

Figure 6-1: Plan view of Zone 3

6.2 Summary of Investigation Works

Table 6-1 shows details of the boreholes in Zone 3 and Figure 6-2 shows a plan view of their locations. The arrows and numbers in Figure 6-2 indicate the figure number and the direction of viewing of the laser scan images shown later in this section.

Both upper and lower seams of gypsum are present in this zone. The thickness of the upper seam is consistently 7 m to 8 m across the zone, but the thickness of the gypsum roof beam above the mine workings decreases towards the south west from 11 m to 5 m. Figure 6-3 is a cross-section along the R179 road showing detail of the boreholes and underground mining.

Table 6-1: Details of the boreholes drilled in Zone 3

BHID	Azimuth (°)	Dip (°)	Depth (m)	Easting	Northing	Elevation	Laser Scan?
KC19-R10	91	71	46	280821.8	299884.38	1049.57	Yes
KC19-R11	152	57	52	280807.19	299866.47	1048.92	Yes
KC19-R11A	155	60	51	280792.52	299848.7	1048.38	Yes

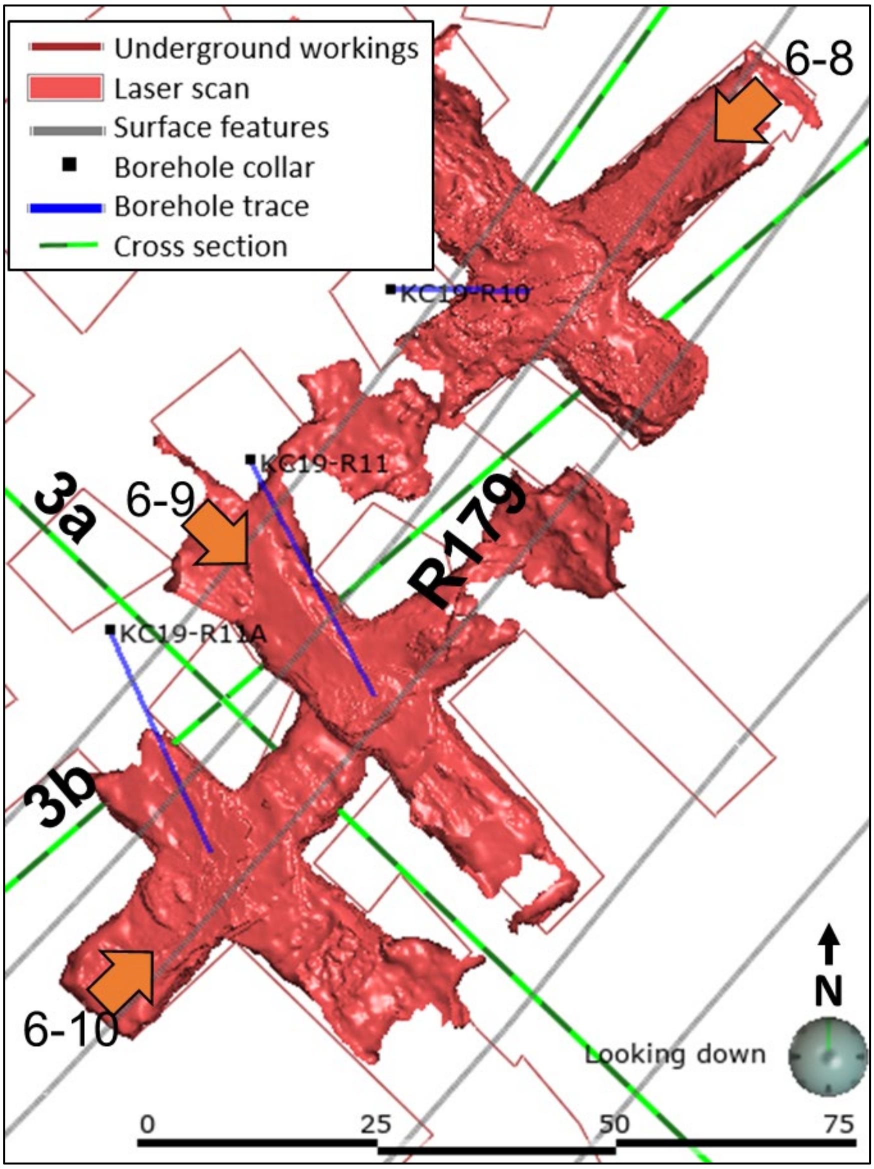

Figure 6-2: Zone 3 showing borehole collars and laser scan image locations

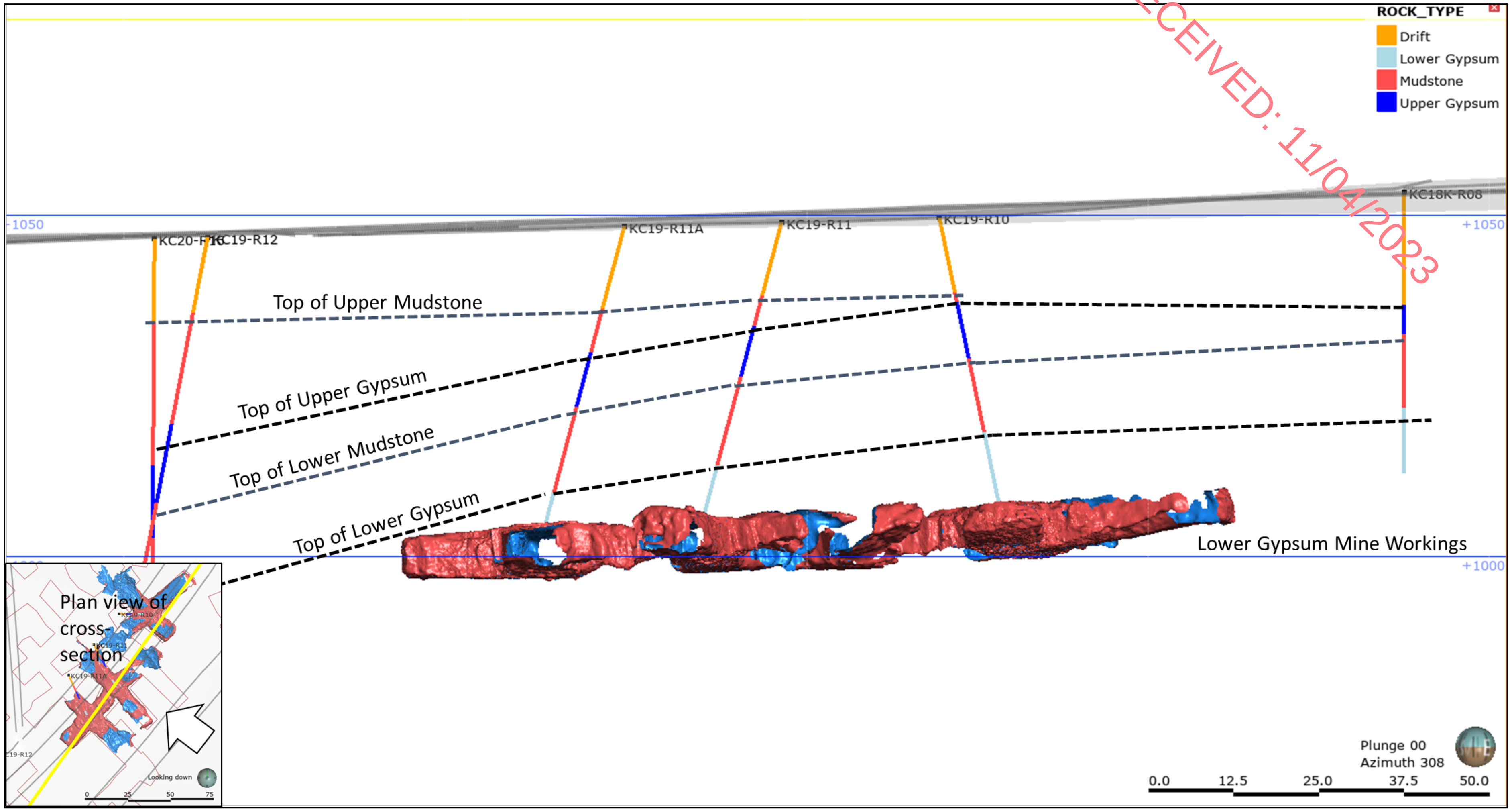

Figure 6-3: Zone 3: NE-SW section along R179 road showing borehole geology and underground mining

6.3 Finite Element Modelling

6.3.1 Model Geometry

The finite element models analysed for Zone 3 are shown in Figure 6-4and Figure 6-5 . Each cross section shows the lithologies, the location of the R179 road, the mine working, the depth to the mine workings below the road and the thickness of the gypsum roof beam above the workings closest to the R179 road. Note that cross section 3b cuts obliquely across the road, hence the increased roadway width. In addition to the high and low water levels an intermediate water level at 987mL. This represents the mine water level in January 2020. At this location the workings lie 39m below the road and the gypsum roof beam is between 8m and 12m thick. Note that the irregular shaped mine workings are based on the underground laser survey.

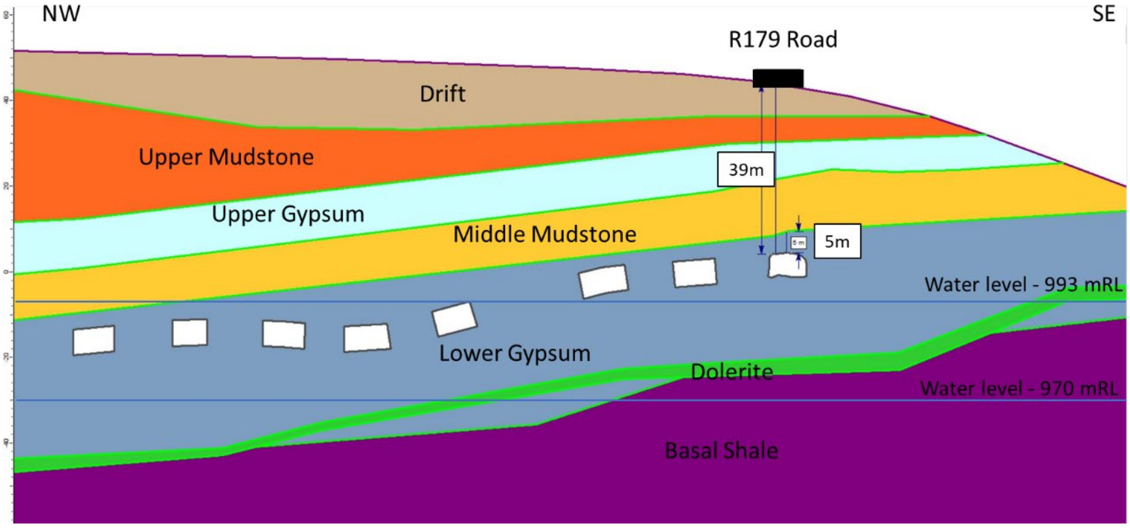

Figure 6-4: Cross Section 3a

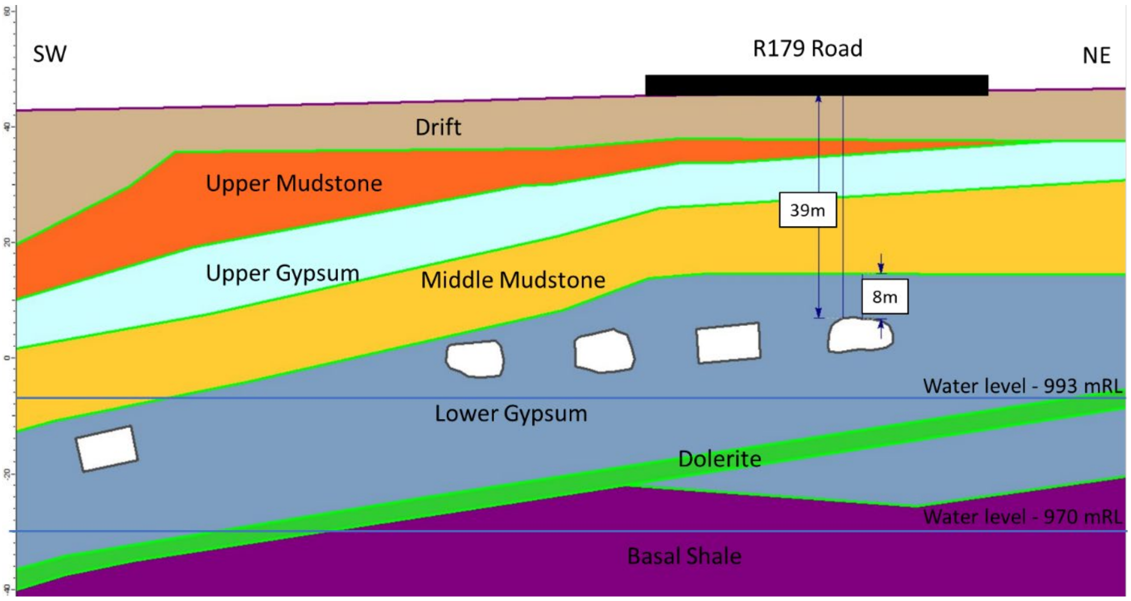

Figure 6-5: Cross Section 3b

6.3.2 Results

The results of the numerical modelling are reproduced in Figure 6-6 and Figure 6-7. These are contour plots of total displacement in metres. As well as the contours of total displacement, the plots show the predicted displacement close to surface below and adjacent to the R179 road. Also shown is the maximum displacement at the underside of the roof beam above the workings which lie immediately below the road. Displacements at points within the model are recorded in millimetres for ease of reference.

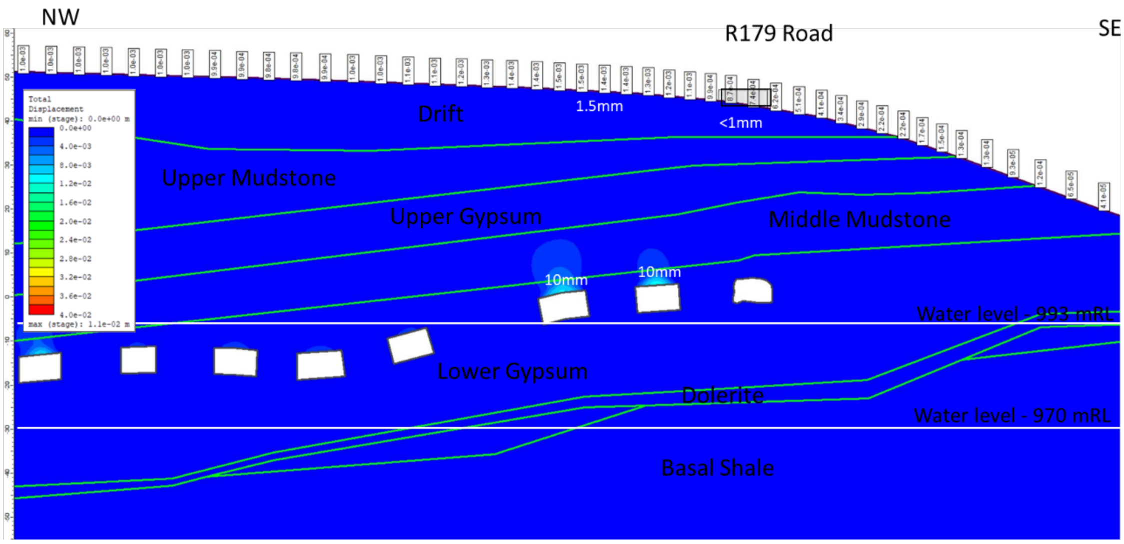

Figure 6-6: Cross Section 3a: Total Displacement

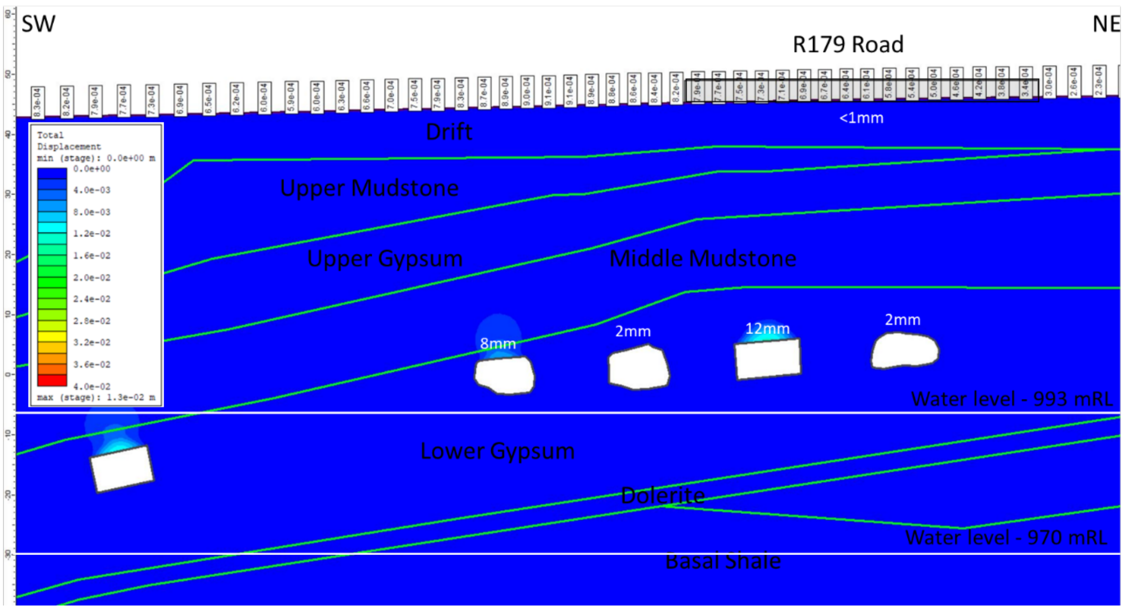

Figure 6-7: Cross Section 3b: Total Displacement

For cross sections 3a and 3b, the numerical model defines stable pillar conditions. For cross section 3a, the maximum roof beam deflection is 10 mm (0.01 m) which, for the 5 m thick gypsum roof beam, represents a maximum beam deflection of 0.2% of roof beam thickness. On surface, below and adjacent to the R179 road, the maximum simulated surface deformation is between 1 mm and 2 mm. For cross section 3b, the maximum roof beam deflection is 12 mm (0.012 m) which, for the 8 m thick gypsum roof beam, gives a maximum deflection of 0.15% of roof beam thickness. On surface, below and adjacent to the R179 road, the maximum simulated surface deformation is <1 mm.

As the underground workings in this zone have been subject to water level rise and fall, additional simulations were carried out, by placing phreatic surfaces in the model at the 993 mRL and the 970 mRL, to assess whether a mine water level change has had any impact on stability and surface deformation. The simulations indicated that there was no increase or decrease in deformation as the water level rose and fell.

6.4 Laser Scan Surveys

Figure 6-8 through to Figure 6-10 show laser scan images from the boreholes in Zone 3; the exact locations of the views shown are indicated by the orange arrows in Figure 6-2.

Figure 6-8 shows the view looking south west from the laser scan of KC19-R10, where the depth to the mine workings is 43.00 m and the thickness of the gypsum roof beam is 11.69 m. The view is of an intersection that lies directly below the R179 road. Blue 'holes' in the image indicate areas of roof and wall that were not captured by the laser instrument. There is approximately 2 m^3 of angular debris on the floor; the roof also shows signs of slabbing (clear bedding steps visible) which may be the source of the material. Only approximately 0.5 m thickness of the roof beam may have slabbed, meaning around 11 m of thickness remains, so the strength of the roof beam is unlikely to be significantly compromised.

Figure 6-9 shows the view looking south east from the laser scan of KC19-R11, where the depth to the mine workings is 43.19 m and the thickness of the gypsum roof beam is 7.64 m. The view is of an intersection that lies directly below the R179 road. The sidewalls and roof are in good condition; as there is no evidence of instability the debris on the floor is likely to be waste material that has been dumped on the floor of the workings.

Figure 6-10 shows the view looking north east from the laser scan of KC19-R11A, where the depth to the mine workings is 44.34 m and the thickness of the gypsum roof beam is 5.24 m. The view is of an intersection that lies directly below the R179 road, although view is taken from a dead end. Debris on the floor on the far centre of the image suggest possible instability, although there is no clear evidence of this in the immediate sidewalls and roof. Some localised bedding slabbing is visible on the roof in the centre of intersection of approximately 1 m thickness, but the amount of debris on the floor directly below suggests this is not a large volume and unlikely to compromise roof beam strength. Aside from this, the roof and sidewalls are in good condition.

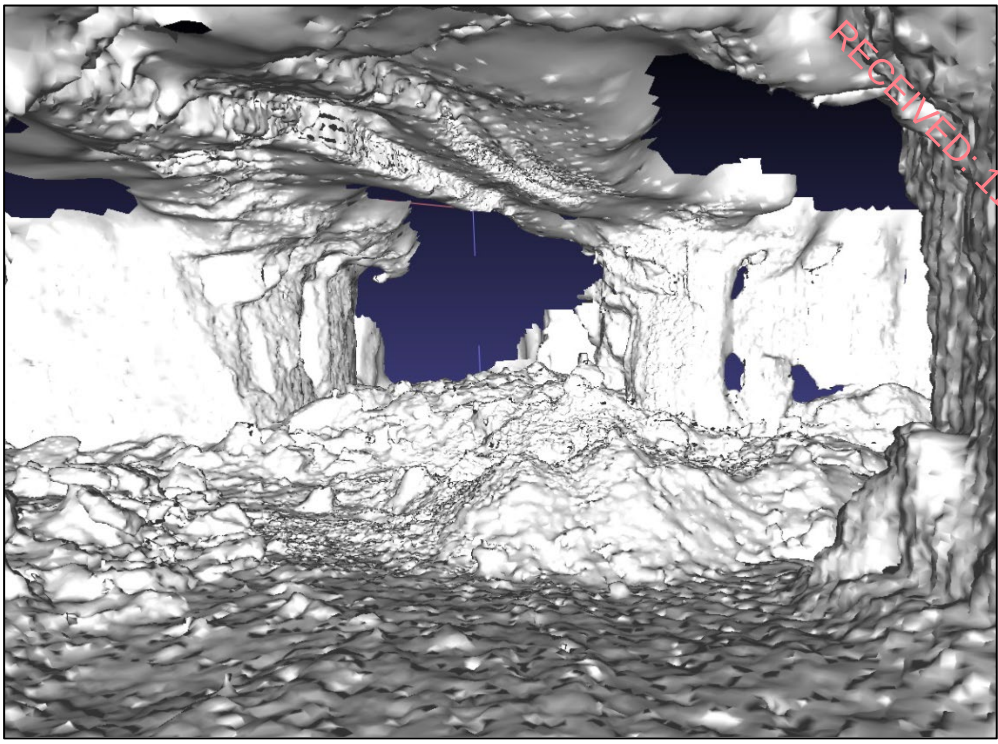

Figure 6-8: Laser scan image from KC19-R10 looking south west, directly below the R179

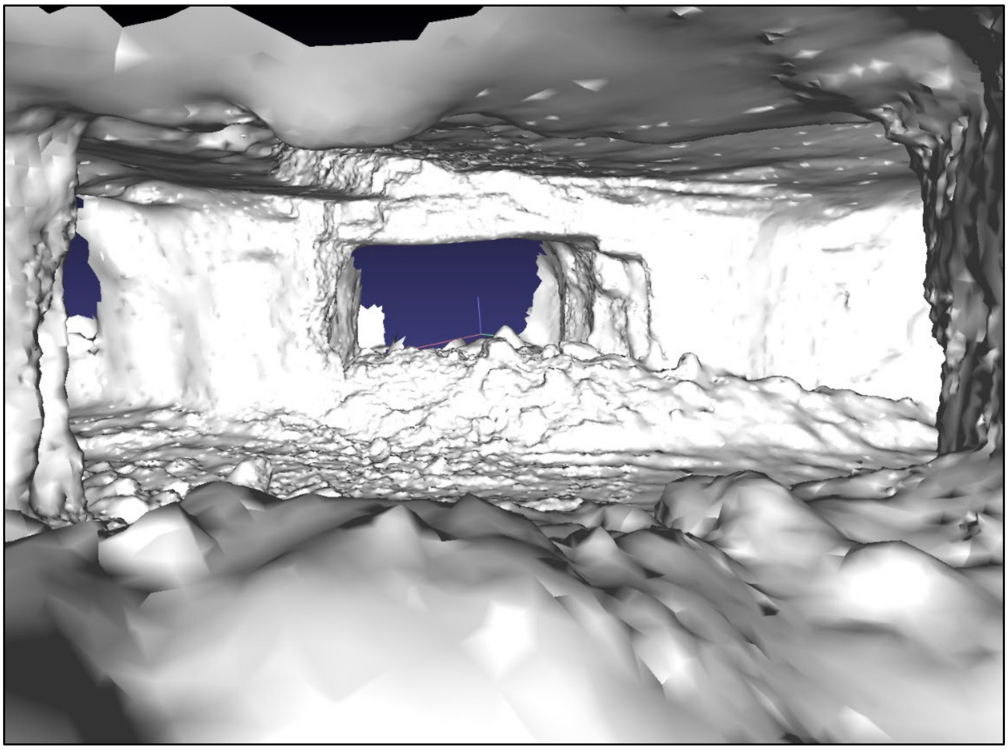

Figure 6-9: Laser scan image from KC19-R11 looking south east, directly below the R179

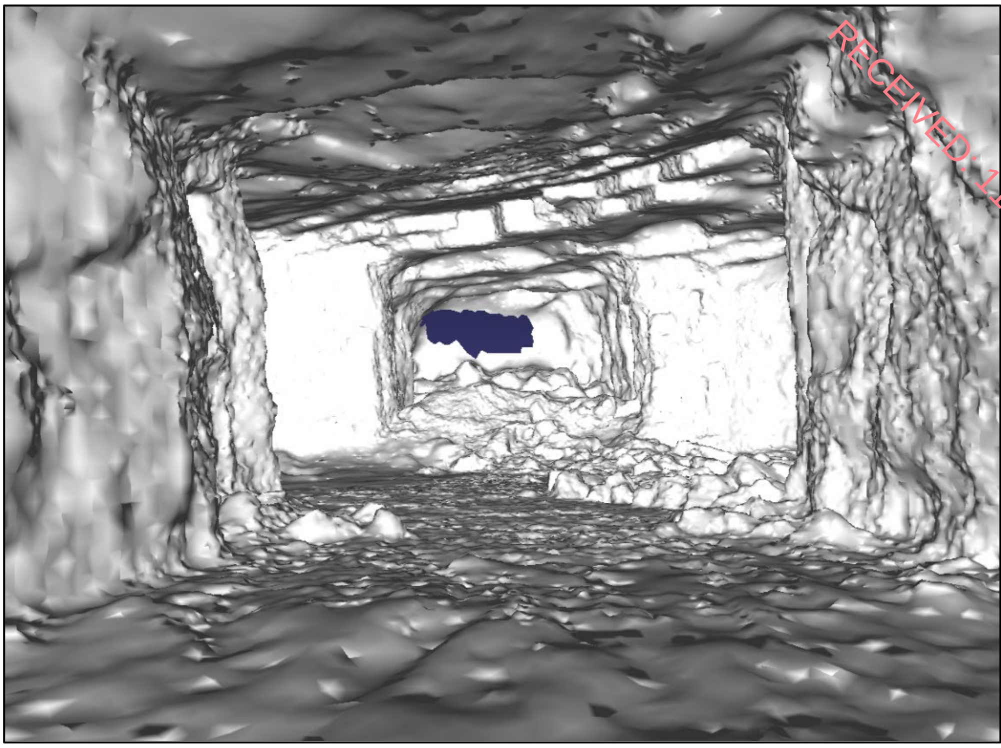

Figure 6-10: Laser scan image from KC19-R11A looking north east, directly below the R179

7 ZONE 4

7.1 General Description of Undermining

Figure 7-1 shows a plan view of Zone 4 as modelled using Leapfrog Geo 3D modelling software. The figure shows the underground workings survey, borehole collars and traces, the locations of cross-section used for later RS2 modelling, the 3D laser scan data, and the extents of the R179 road.

The length of the undermined road in this zone is approximately 110 m and the minimum depth of mine workings below surface that have been laser scanned is 77 m. The laser scans show that the room height of the underground workings is approximately 6 m. All excavations in Zone 4 (with the exception of the upper Lower Gypsum level) lie below the historical mine water level of 970 mRL.

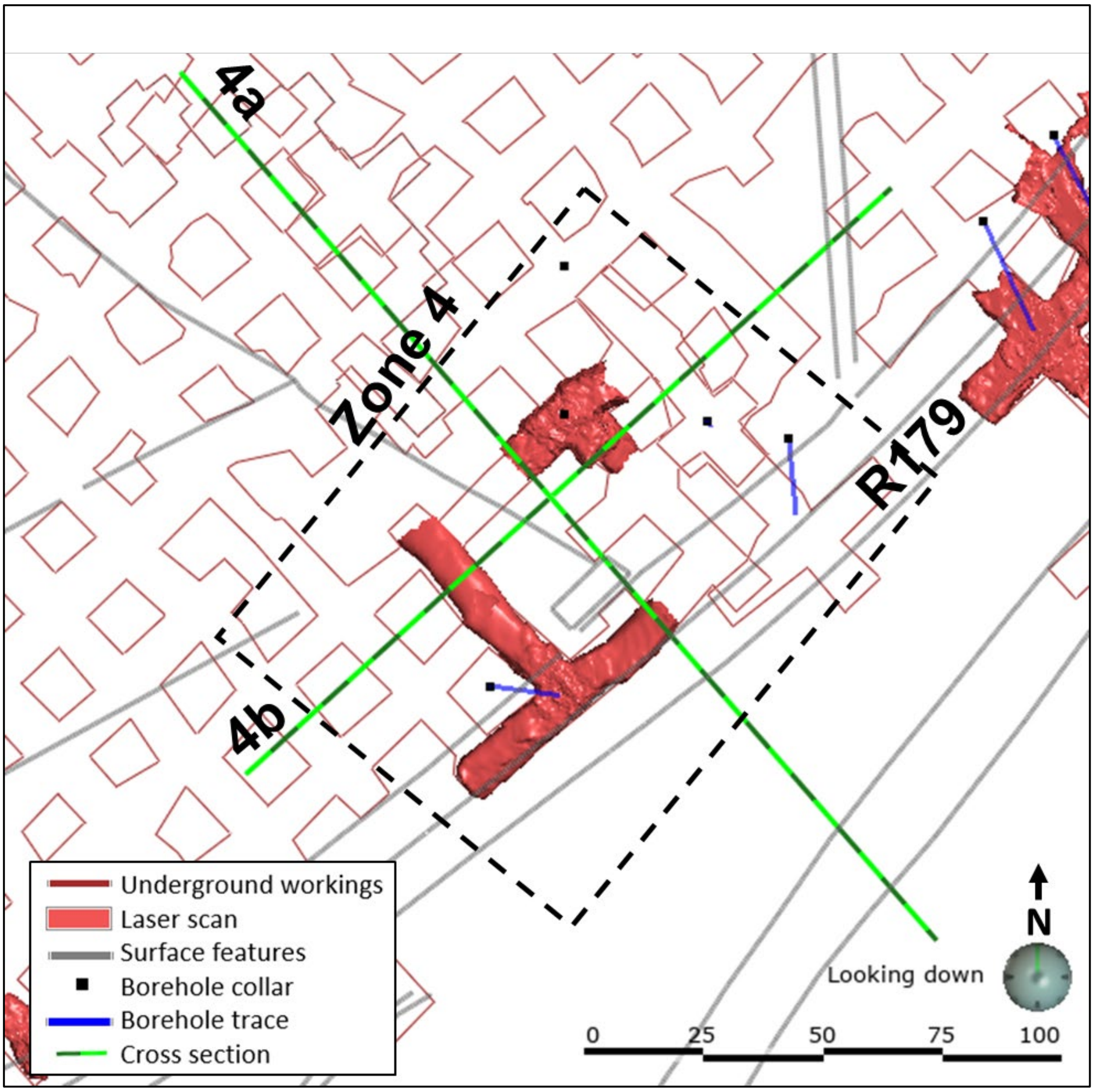

Figure 7-1: Plan view of Zone 4

7.2 Summary of Investigation Works

Table 7-1 shows details of the boreholes in Zone 1 and Figure 7-2 shows a plan view of their locations. The arrows and numbers in Figure 7-2 indicate the figure number and the direction of viewing of the laser scan images shown later in this section.

Both upper and lower seams of gypsum are present in this zone. The thickness of the upper seam is 9 m and the thickness of the gypsum roof beam above the mine workings is 16 m. Figure 7-3 is a cross-section along the R179 road showing detail of the boreholes and underground mining.

Table 7-1: Details of the boreholes drilled in Zone 4

BHID	Azimuth (°)	Dip (°)	Depth (m)	Easting	Northing	Elevation	Laser Scan?
KC19-R12	175	75	63	280751.68	299803.16	1046.76	No
KC20-R13	98	79	80	280689.17	299751.17	1044.08	Yes
KC20-R16	149	89	91	280734.6	299806.65	1046.47	No
KC20-R17	59	90	101	280704.62	299808.11	1046.51	Yes
KC20-R18	187.7	89.7	107	280704.74	299839.32	1048.55	No

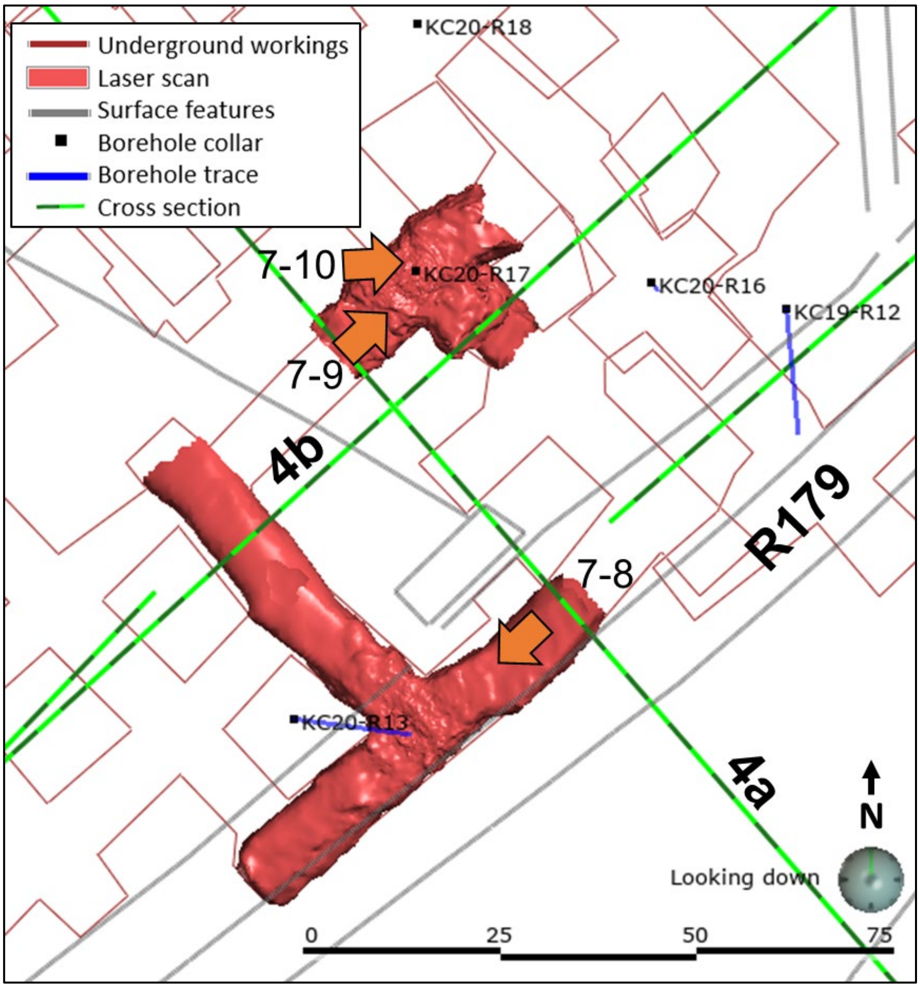

Figure 7-2: Zone 4 showing borehole collars and laser scan image locations

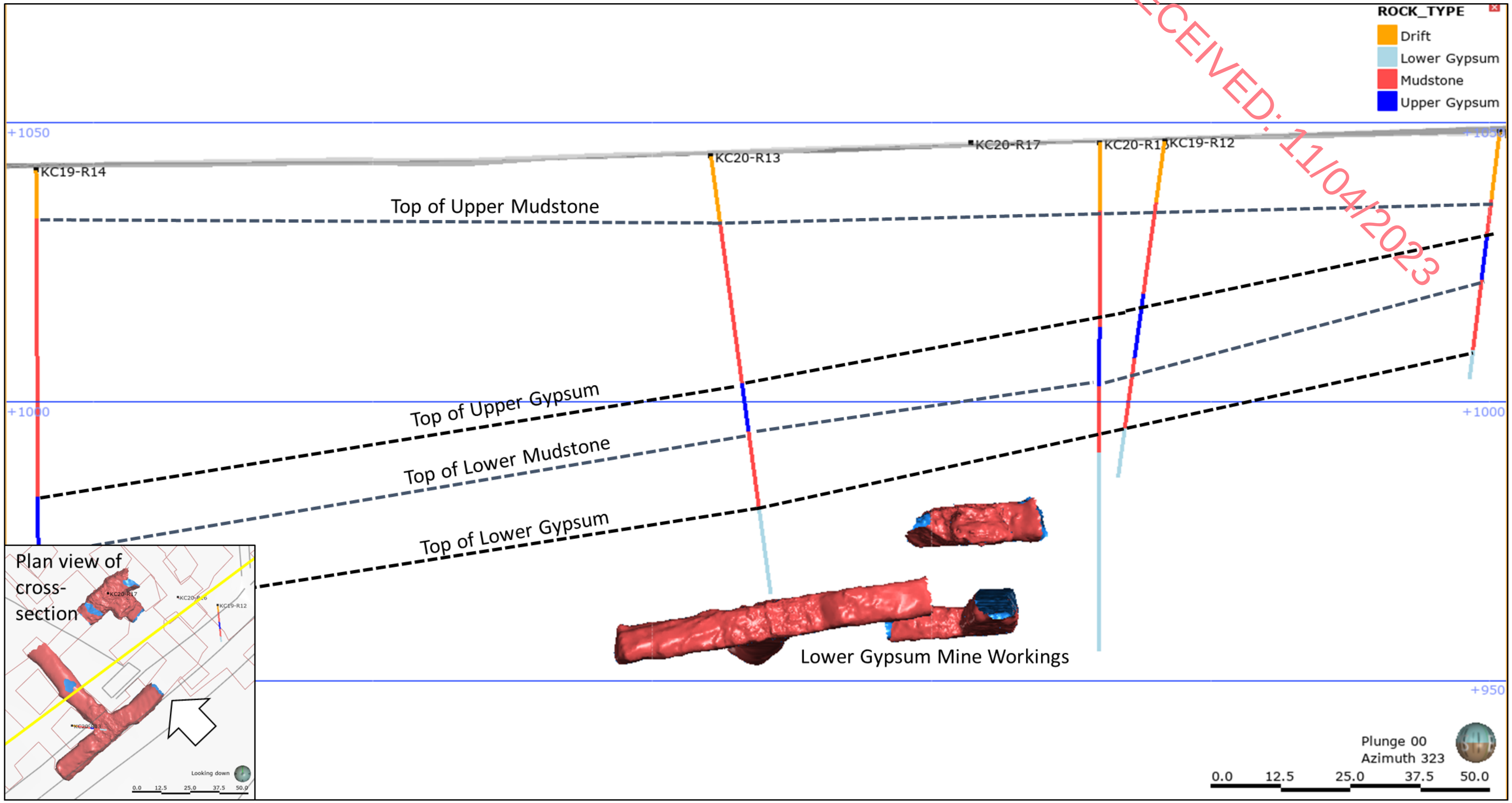

Figure 7-3: Zone 4: NE-SW section along R179 road showing borehole geology and underground mining

7.3 Finite Element Modelling

7.3.1 Model Geometry

The finite element models analysed for Zone 4 are shown in Figure 7-4 and Figure 7-5 . Each cross section shows the lithologies, the location of the R179 road, the mine working, the depth to the mine workings below the road, and the thickness of the gypsum roof beam above the workings closest to the R179 road. Note that cross section 4b is approximately parallel to the road and is positioned 38 m to the north west of the road edge. Again, three water levels are shown on the cross sections although almost all the excavations along these cross sections have remained permanently under water as the minimum mine water level is at 970 mRL.

At this location, the workings lie 74 m below the R179 road. Further to the north west the mining depth reduces to about 54 m. The cross sections indicate upper and lower mine workings within the Lower Gypsum unit. Only the lower workings occur directly below the road. The shallower upper workings are present about 30 m to the northwest of the road. Below the road the gypsum roof beam is 18 m. The roof beam above the upper workings is 3 m thick. Note that the irregular shaped mine workings are based on the underground laser survey.

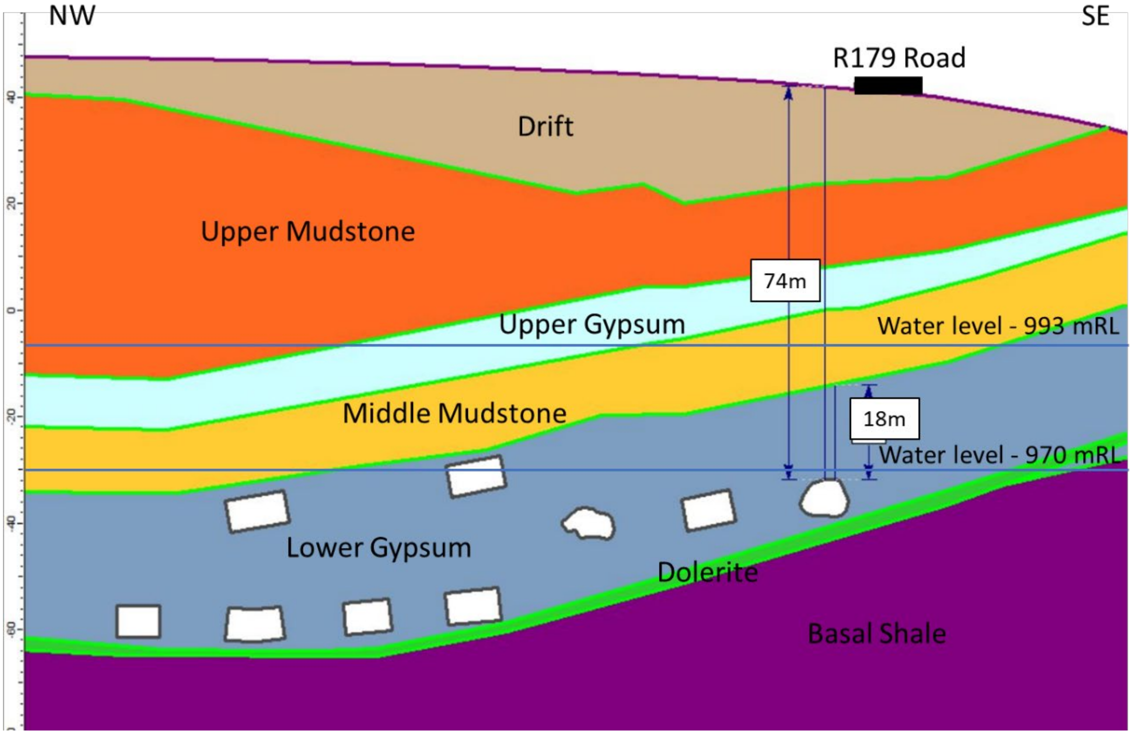

Figure 7-4: Cross Section 4a

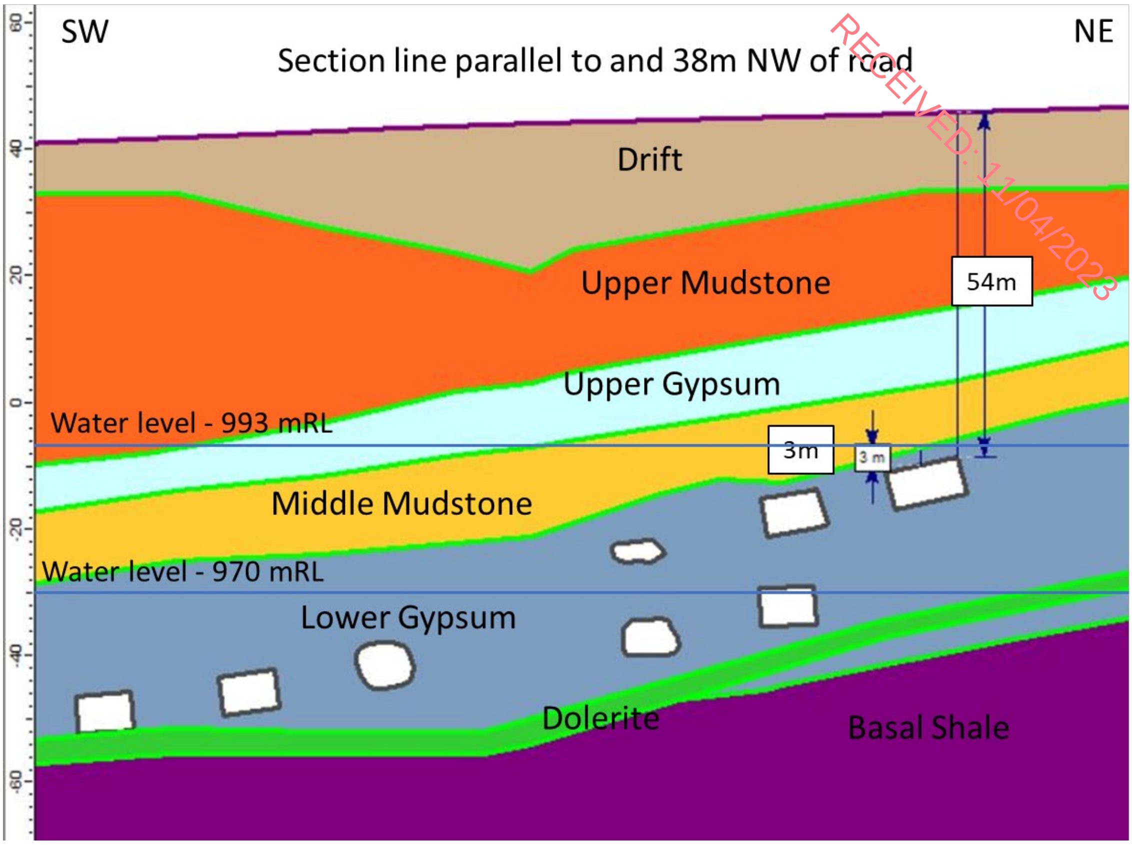

Figure 7-5: Cross Section 4b

7.3.2 Results

The results of the numerical modelling are reproduced in Figure 7-6 and Figure 7-7. These are contour plots of total displacement in metres. As well as the contours of total displacement, the plots show the predicted displacement close to surface below and adjacent to the R179 road. Also shown is the maximum displacement at the underside of the roof beam above the workings which lie immediately below the road. Displacements at points within the model are recorded in millimetres for ease of reference.

For cross sections 4a and 4b, the numerical model defines stable pillar conditions. For cross section 4a, the 3 m thick roof beam shows maximum roof beam deflection of 42 mm (0.042 m). This represents a maximum beam deflection of 1.4% of roof beam thickness. On the surface above the upper mine workings, the models simulate surface movement of about 6 mm. For the mine workings 74 m below the R179 on cross section 4a, the maximum roof beam deflection is 2 mm (0.002 m) which, for an 18 m roof beam thickness, represents negligible deformation. Surface deformation below the road is simulated as 2 mm; this is likely to be a function of the deformation in the upper workings.

For cross section 4b, the maximum roof beam deflection is 16 mm (0.016 m) which for the 3 m thick gypsum roof beam gives a maximum deflection of 0.5% of roof beam thickness. On surface, the maximum simulated surface deformation is 1-2 mm.

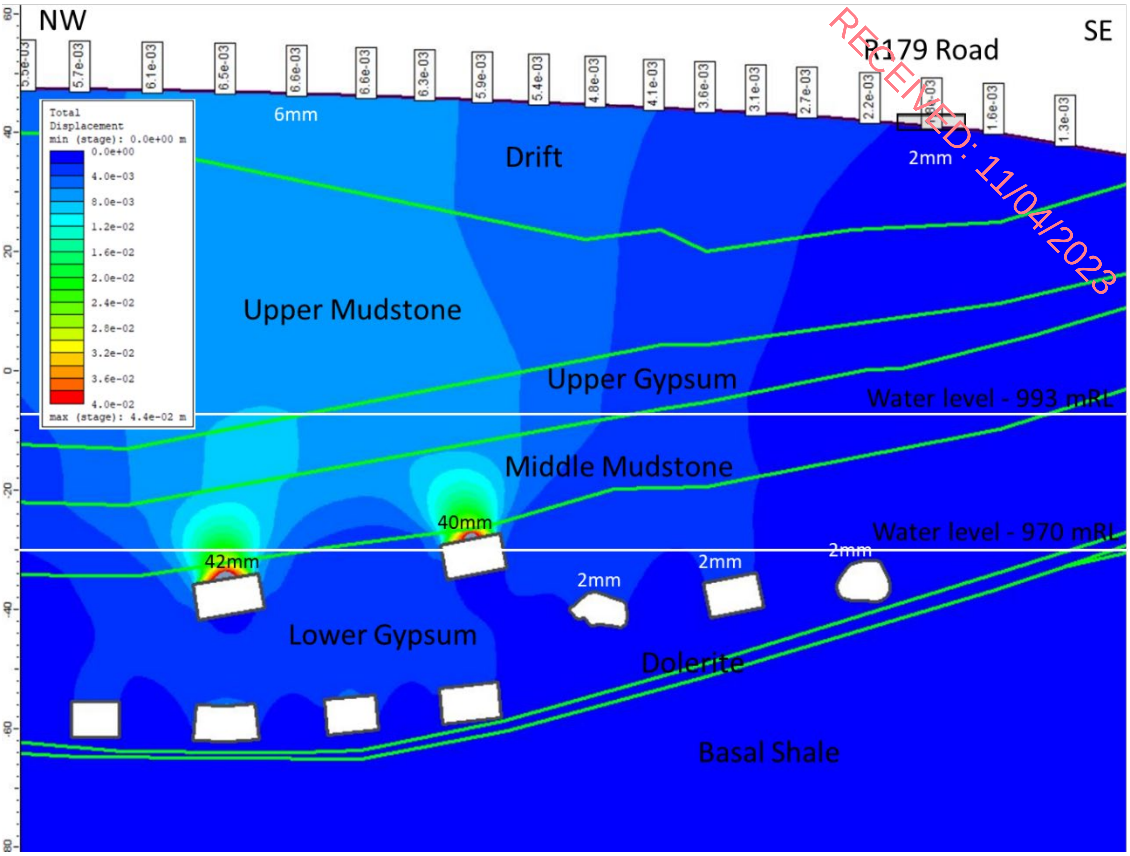

Figure 7-6: Cross Section 4a: Total Displacement

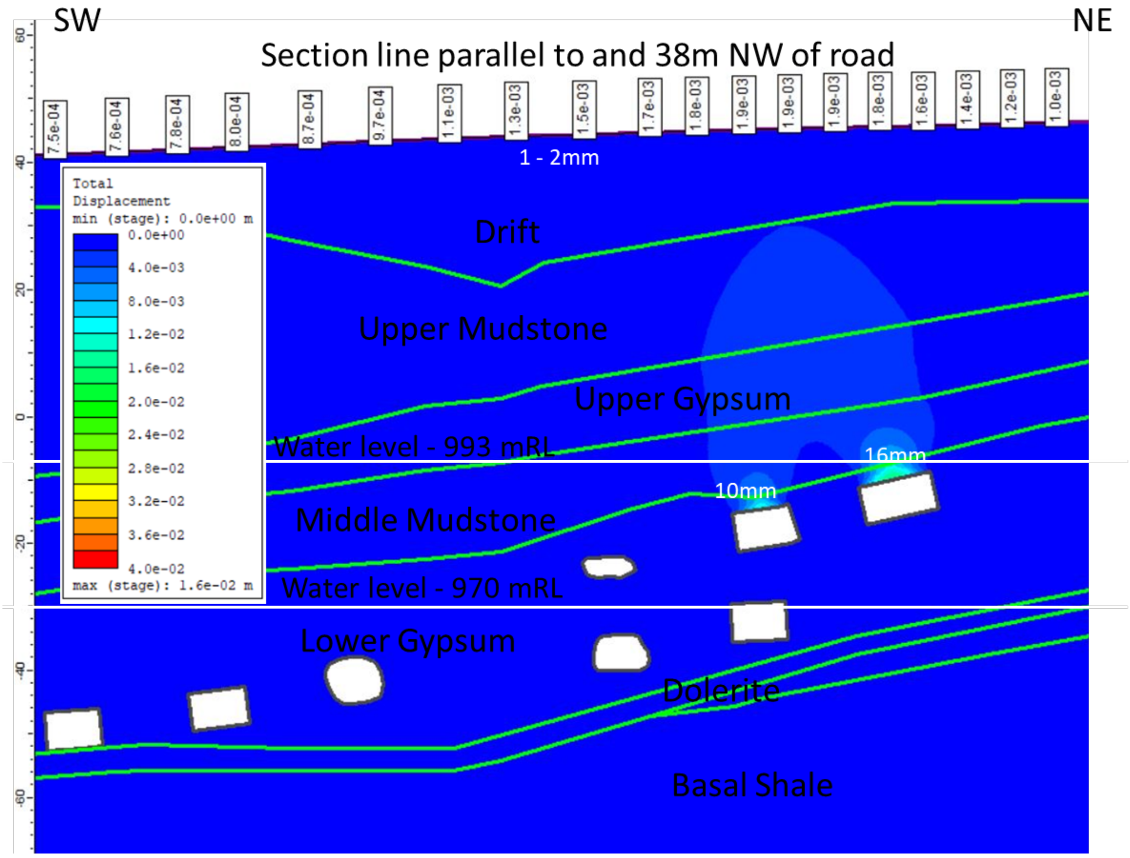

Figure 7-7: Cross Section 4b: Total Displacement

7.4 Laser Scan Surveys

Figure 7-8 through to Figure 7-10 show laser scan images from the boreholes in Zone 4; the exact locations of the views shown are indicated by the orange arrows in Figure 7-2.

Figure 7-8 shows the view looking south west from the laser scan of KC19-R13, where the depth to the mine workings is 78.58 m and the thickness of the gypsum roof beam is 15.59 m. The view is of an excavation that runs parallel below the north west side of the R179 road. The roof and sidewalls are in good condition with no indication of instability.

Figure 7-9 shows the view looking east from the laser scan of KC19-R17 (upper level), where the depth to the mine workings is 66.0 m. The view is of an excavation that lies approximately 35 m to the north west below the R179 road. There is no evidence of instability and the sidewalls and roof appear in good condition.

Figure 7-10 shows the view looking north east from the laser scan of KC19-R17 (lower level), where the depth to the mine workings is 86.00 m. The view is of a corner junction that lies approximately 35 m to the north west below the R179 road. The roof and sidewalls are in good condition with no indication of instability.

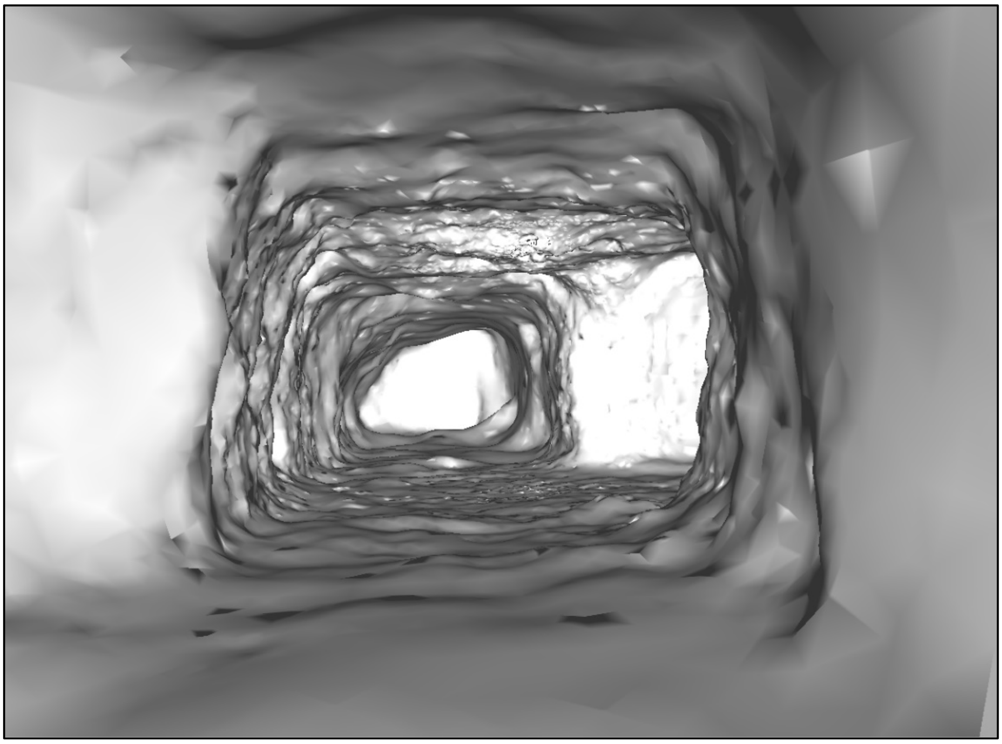

Figure 7-8: Laser scan image from KC19-R13 looking south west

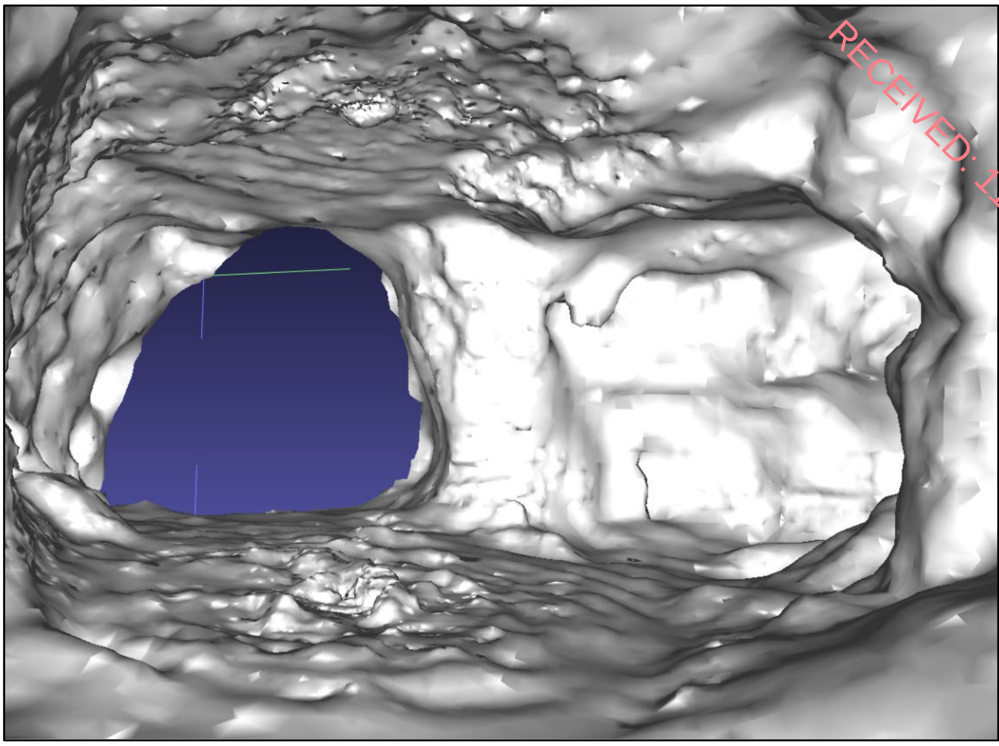

Figure 7-9: Laser scan image from KC19-R17 (Upper Level) looking east

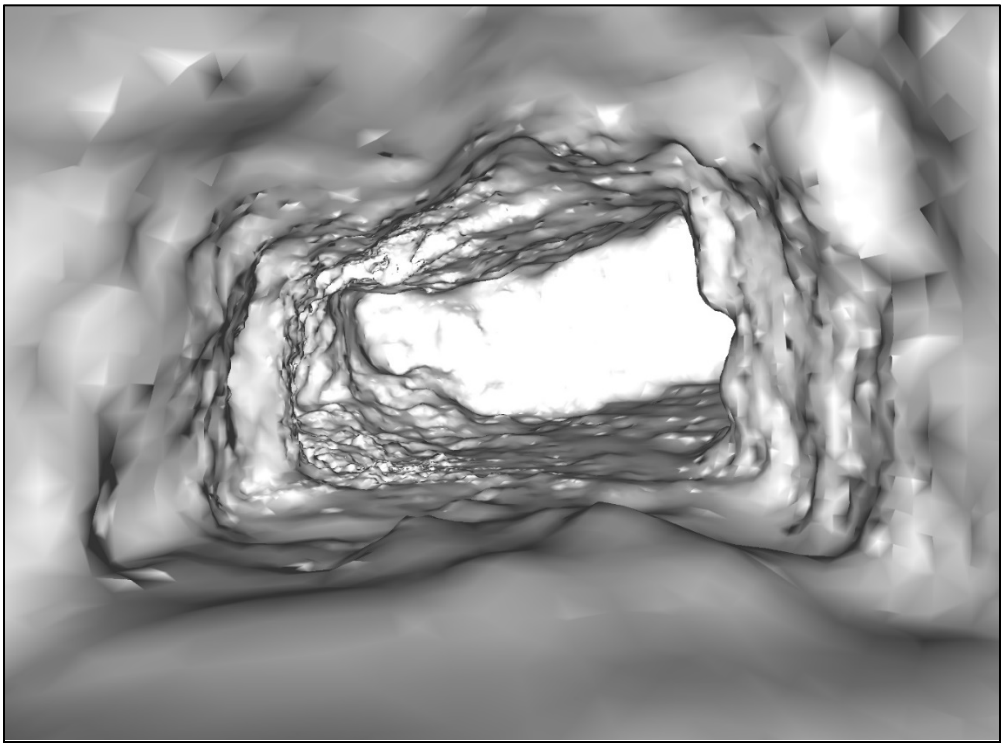

Figure 7-10: Laser scan image from KC19-R17 (Lower Level) looking north east

8 ZONE 5

8.1 General Description of Undermining

Figure 8-1 shows a plan view of Zone 5 as modelled using Leapfrog Geo 3D modelling software. The figure shows the underground workings survey, borehole collars and traces, the locations of cross-section used for later RS2 modelling, the 3D laser scan data, and the extents of the R179 road.

The is no direct undermining of the R179 in this zone apart from one 8 m wide drive extending to the east of this zone. The minimum depth of mine workings below surface that have been laser scanned is 90 m. The laser scans show that the room height of the underground workings is approximately 6 m. All excavations in Zone 5 lie below the historical mine water level of 970 mRL

Figure 8-1: Plan view of Zone 5

8.2 Summary of Investigation Works

Table 8-1 shows details of the boreholes in Zone 1 and Figure 8-2 shows a plan view of their locations. The arrows and numbers in Figure 8-2 indicate the figure number and the direction of viewing of the laser scan images shown later in this section.

Both upper and lower seams of gypsum are present in this zone. The thickness of the upper seam is 10 m and the thickness of the gypsum roof beam above the mine workings is 11 m. Figure 8-3 is a cross-section along the R179 road showing detail of the boreholes and underground mining.

Table 8-1: Details of the boreholes drilled in Zone 5

BHID	Azimuth (°)	Dip (°)	Depth (m)	Easting	Northing	Elevation	Laser Scan?
KC19-R14	104	90	93	280593.96	299676.8	1041.53	Yes

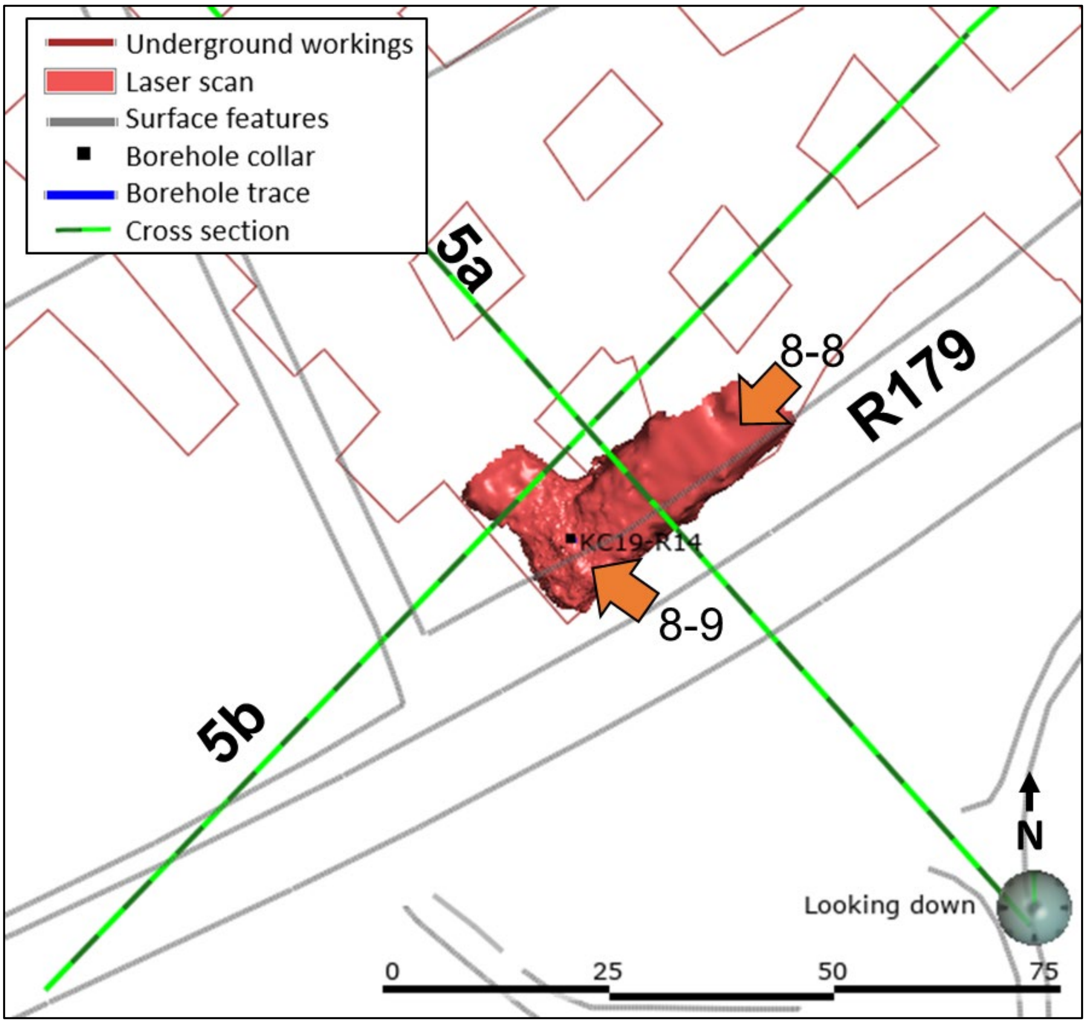

Figure 8-2: Zone 5 showing borehole collars and laser scan image locations

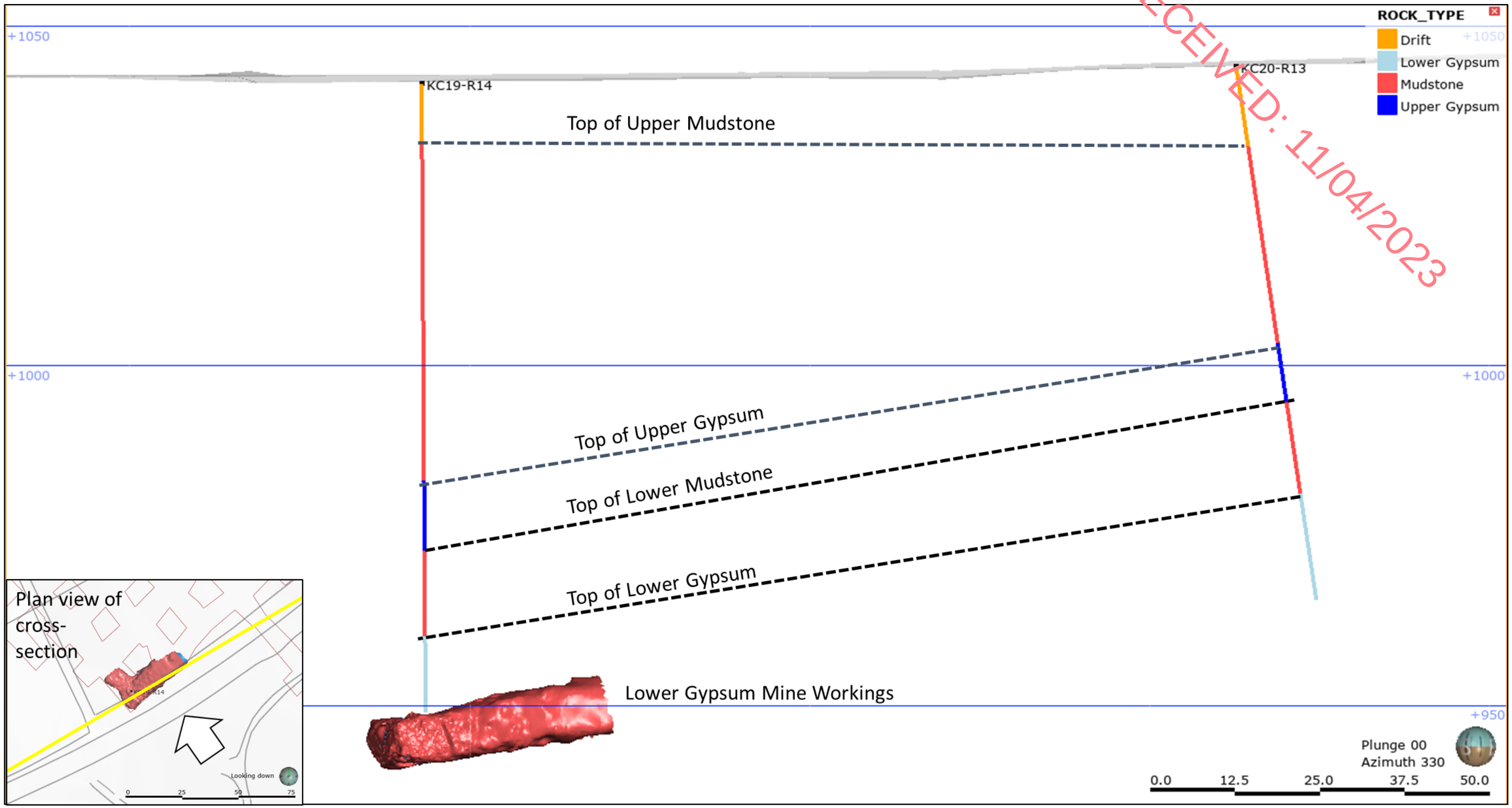

Figure 8-3: Zone 5: NE-SW section along R179 road showing borehole geology and underground mining

8.3 Finite Element Modelling

8.3.1 Model Geometry

The finite element models analysed for Zone 5 are shown in Figure 8-4 and Figure 8-5 . Each cross section shows the lithologies, the location of the R179 road, the mine working, the depth to the mine workings below the road, and the thickness of the gypsum roof beam above the workings closest to the R179 road. At this location the mine workings lie below the 970 mRL, the historical mine water level, indicating that they are permanently flooded. The workings lie between 89 m and 92 m below the road and the gypsum roof beam is between 10 m and 12 m thick. Note that the irregular shaped mine workings are based on the underground laser survey.

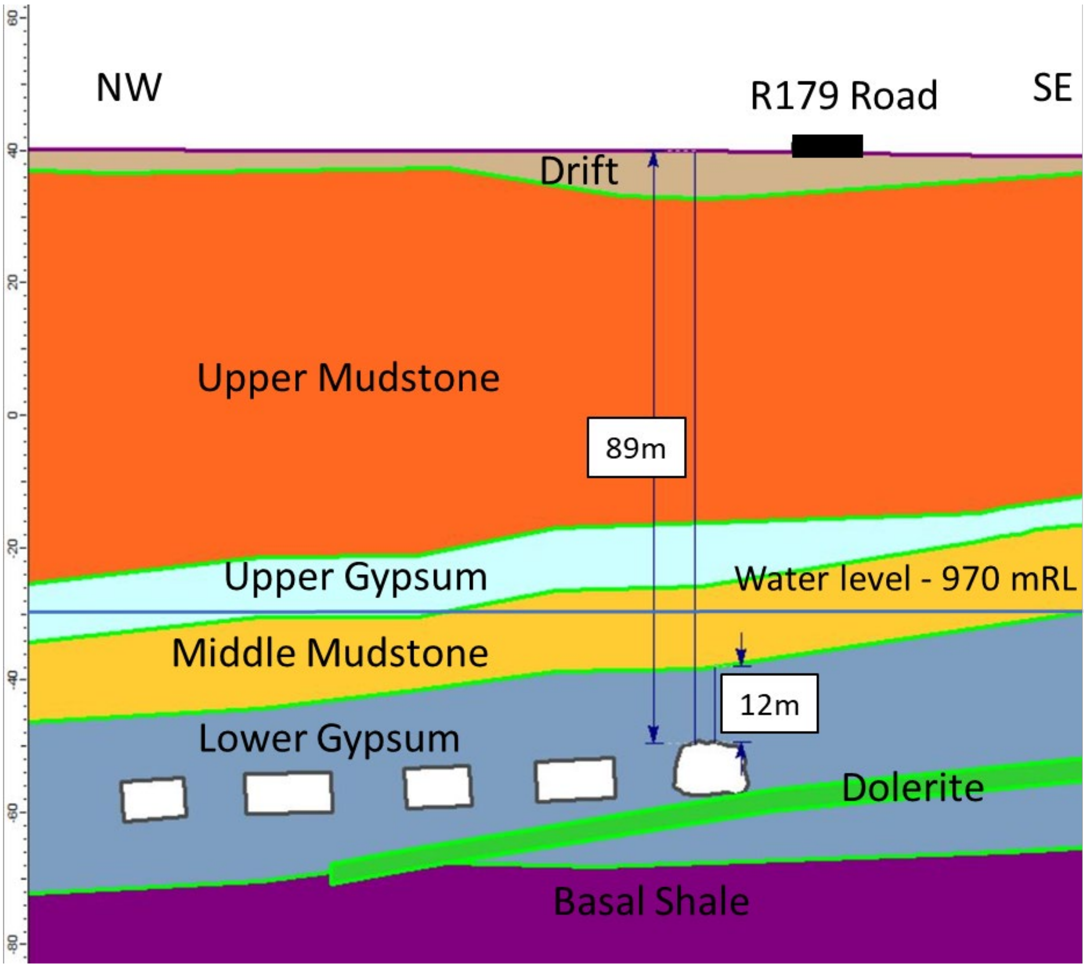

Figure 8-4: Cross Section 5a

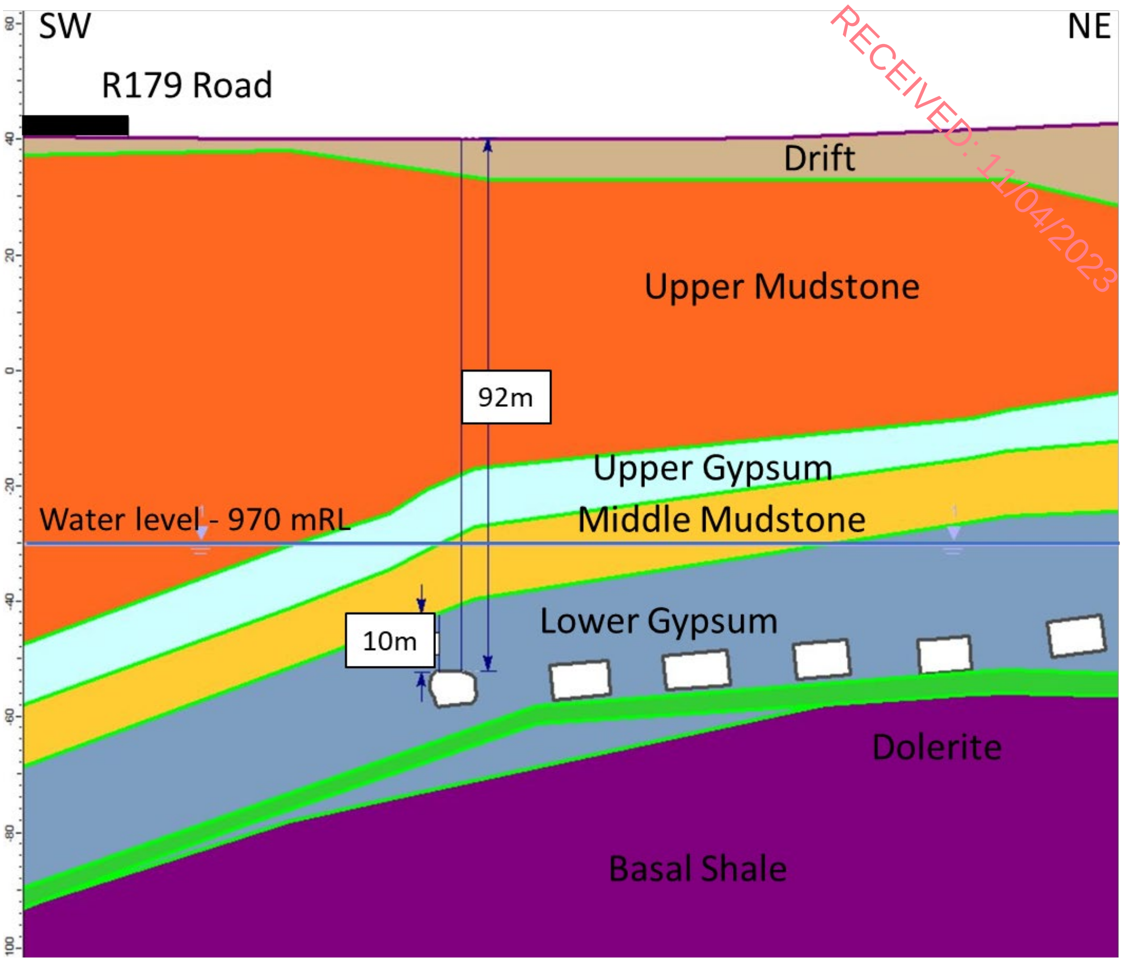

Figure 8-5: Cross Section 5b

8.3.2 Results

The results of the numerical modelling are reproduced in Figure 8-4 and Figure 8-5. These are contour plots of total displacement in metres. As well as the contours of total displacement the plots show the predicted displacement close to surface below and adjacent to the R179 road. Also shown is the maximum displacement at the underside of the roof beam above the workings which lie immediately below the road. Displacements at points within the model are recorded in millimetres for ease of reference.

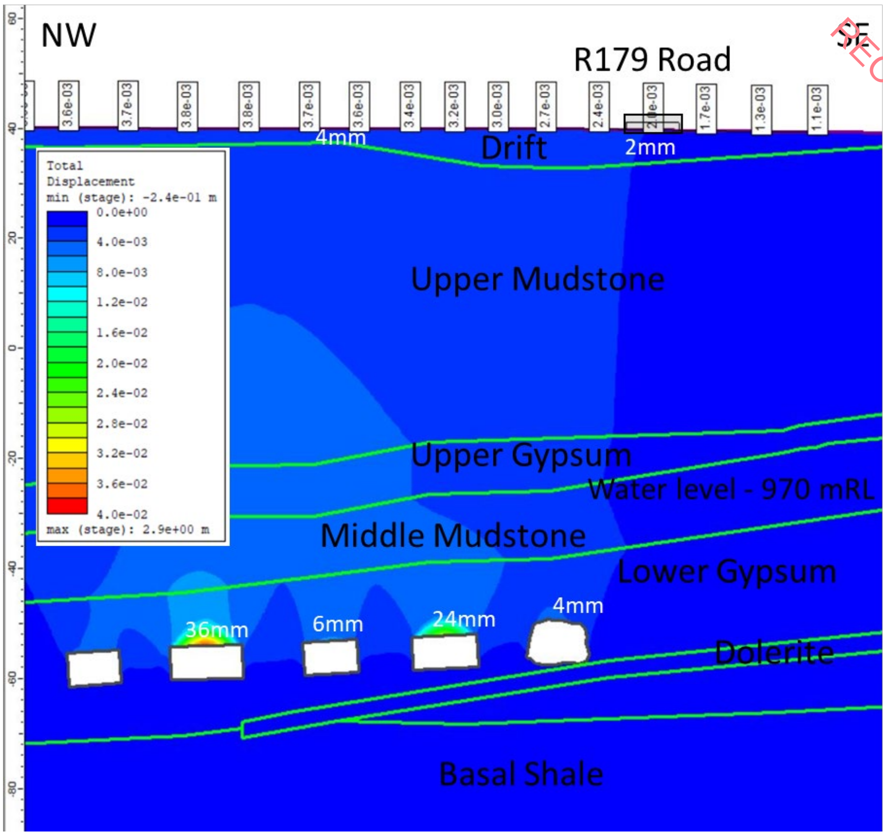

Figure 8-6: Cross Section 5a: Total Displacement

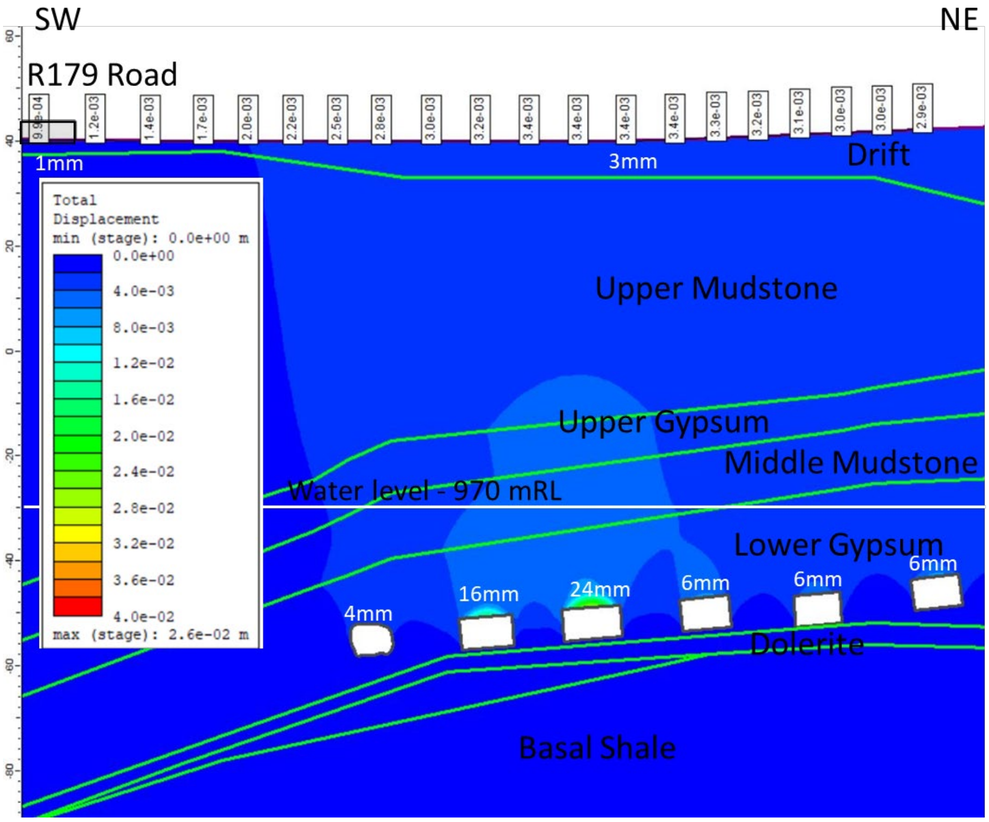

Figure 8-7: Cross Section 5b: Total Displacement

For cross sections 5a and 5b, the numerical model defines stable pillar conditions. For cross section 5a, the maximum roof beam deflection is 36 mm (0.036 m) which for the 12 m thick gypsum roof beam represents a maximum beam deflection of 0.3% of roof beam thickness. On surface, below and adjacent to the R179 road, the maximum simulated surface deformation is between2 mm and 4 mm. For cross section 5b, the maximum roof beam deflection is 24 mm (0.024 m) which for the 10 m thick gypsum roof beam gives a maximum deflection of 0.24% of roof beam thickness. On surface, below and adjacent to the R179 road, the maximum simulated surface deformation is between 1 and 3 mm.

8.4 Laser Scan Surveys

Figure 8-8 and Figure 8-9 show laser scan images from the boreholes in Zone 5; the exact locations of the views shown are indicated by the orange arrows in Figure 8-2.

Figure 8-8 shows the view looking south west from the laser scan of KC19-R14 and Figure 8-9 shows the view looking north west from the laser scan of KC19-R14. The depth to the mine workings is 92.50 m and the thickness of the gypsum roof beam is 11.16 m. Figure 8-8 shows that the roof and sidewalls are in good condition with no evidence of stability. Some of the material on the floor at the centre back of the image is likely to be loose material that has been pushed to the back end. Figure 8-9 shows that wedge formation may have occurred in the left sidewall; however, the roof and other sidewall otherwise look in good condition.

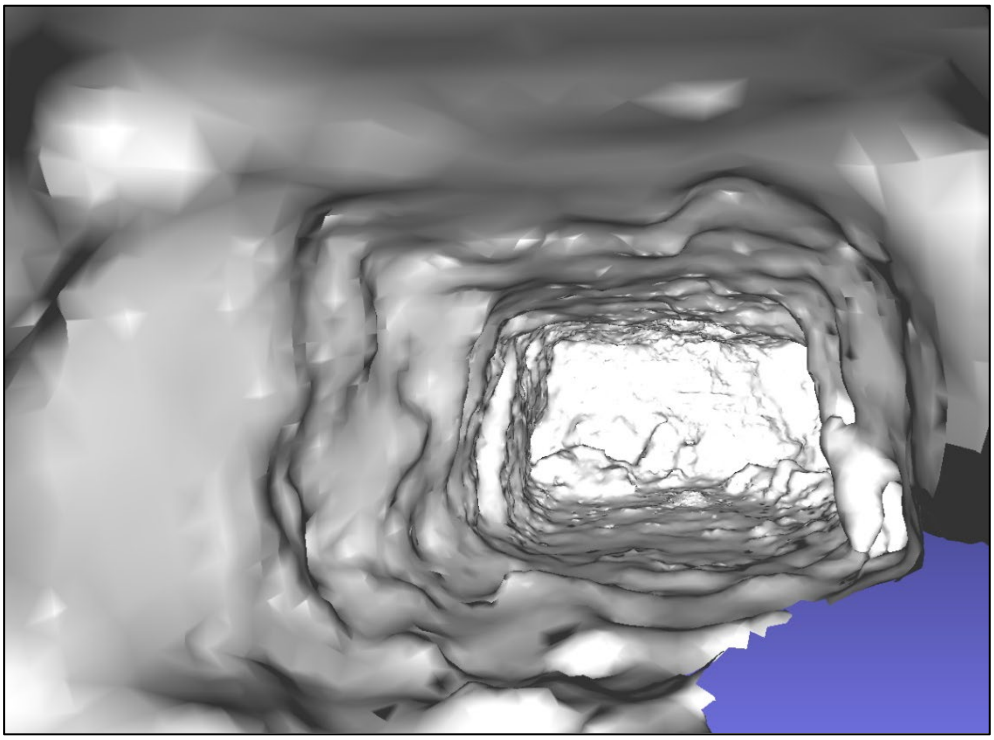

Figure 8-8: Laser scan image from KC19-R14 looking south west

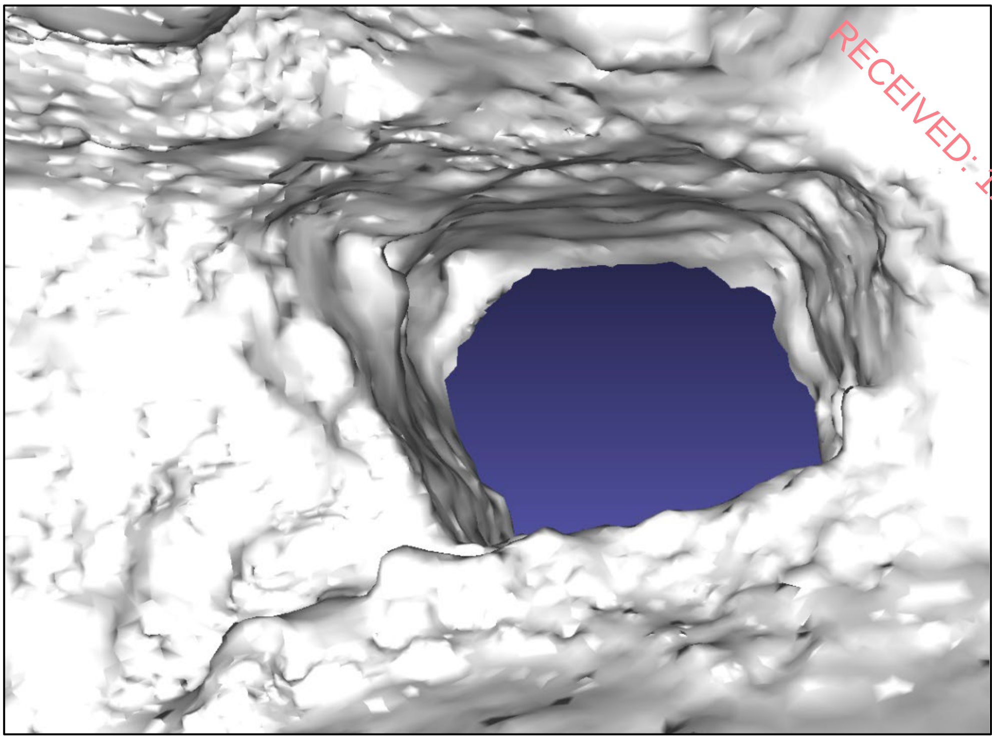

Figure 8-9: Laser scan image from KC19-R14 looking north west

9 CONCLUSIONS

The purpose of this study, comprising field investigations and analyses, was to assess rock mass conditions and the stability of those underground workings that lie below and adjacent to the R179 road. The outcome of the study is to establish if there is any risk of mine collapse that could impact the integrity of the road and pose a risk to traffic and road users. The conclusions of the study are presented in the following sections.

9.1 Geotechnical Characteristics

The investigation work undertaken has confirmed that the geotechnical conditions of the Lower Gypsum within which the underground workings below the R179 road are located are characterised as fair quality rock mass as indicated by the average GSI. The Lower Gypsum contains closely to moderately spaced bedding planes and widely spaced sub-vertical orthogonal joint planes and is interbedded with bands of weaker mudstone. The gypsum has an intact rock strength in the range 7-20 MPa (15-30 MPa for the L4900 study) and a geological strength index in the range 45-55 (50-60 for the L4900 study). The lower values of GSI is a consequence of the lower strength and the effect of the inclusion of weaker mudstone layers within the gypsum. The difference in strength is not considered material to the overall rock mass strength as the intact rock strength only contribute between 3% (for the 7–20 MPa range) and 5% (for the 15-30 MPa range) to the total value of GSI. The fracture and weathering condition of the rock mass is more important in defining rock mass strength.

The mine workings lie between 40 m and 90 m below the R179 road (30 m below the L4900). They are at their shallowest on the north eastern end of the road in Zone 1 and deepest at the south western end in Zone 5. The thickness of the roof beam above the workings below the road ranges between 5 m and 18 m (remembering that the minimum roof beam design thickness is 3 m). The floor beam below the workings is of a similar thickness range.

Below the R179 road the Lower Gypsum is overlain by mudstone which itself is overlain by Upper Gypsum of varying thickness.

The strength and GSI value ranges measured from the borehole core remain consistent with the values generated by SRK during its underground mapping campaigns carried out between 1999 and 2005 and with the ‘expected’ strength conditions used in its predictive analyses of mine stability below the R179 and L4900 reported in October 2018. This suggests that there has been little degradation of the Lower Gypsum rock mass strength over the last 20 years.

9.2 Stability Conditions of Underground Workings

For a mine which is around 40 years old, the condition of the underground rooms, as determined from the borehole laser surveys, is generally reasonable. There is some evidence that bedding bounded slabs have fallen from the roof of the workings. The slab thickness as estimated from the laser scans is up to a maximum of 0.5 m. This is equivalent to 4.5% of roof beam thickness for the average roof beam thickness of 11 m identified by the borehole drilling. The slabbing is very localised and is found to be most prevalent in four-way intersections. This type of roof instability is typical of an underground mine whose rock mass contains well developed, open bedding planes. Slabbing of this nature normally occurs immediately above the workings where there is little rock mass confinement. Provided the gypsum roof beam is sufficiently thick, at least 3 m, propagation of slabbing deeper into the roof beam is generally prevented as a stable arch is formed above the mine opening.

The room floors contain evidence of debris. Some of debris appears as rounded, hummocky mounds of material which appears to be waste material stowed in the workings as part of the mining operations. In some of the laser scan images tyre tracks are clearly visible on the floor. Other debris is angular which suggests this material may have fallen from the roof or from the sidewalls of the pillars. Specific areas of note are summarised in the next sections.

9.2.1 Zone 1, Laser Scan KC20-R01

A chimney hole is visible in the roof of the excavation with a pile of failed material below it. At this point, the depth to the mine workings is 44.03 m and the thickness of the gypsum roof beam intersected in borehole KC20-R01 is 10.52 m. Above this is 16 m of mudstone interbedded with gypsum. The mudstone is overlain by 5 m of Upper Gypsum. The chimney hole is located in a room which lies approximately 5 m to the east of the R179 road. The chimney hole extends approximately 2.50 m into the roof, indicating that approximately 8.00 m of intact gypsum roof beam remains. Geological logging of the hole indicates that the Lower Gypsum in the roof beam is interbedded with weaker mudstone. It is possible that the roof has exposed one of the mudstone interlayers, which has collapsed into the workings. Further propagation of the chimney hole is unlikely due to the presence of a thick competent layer of gypsum above.

9.2.2 Zone 2, Laser Scan KC19-R06A

In this borehole, the depth to the mine workings is 35 m and the thickness of the gypsum roof beam is 13.46 m. About 25 m to the south east of the R179 is a four-way intersection that is 10 m to 11 m high. The intersection roof and sidewalls are in good condition with no evidence of instability. The double height intersection has not resulted in the formation of any double height pillars in the area as the rooms entering the intersection area all standard height of 6 m.

9.2.3 Zone 3, Laser Scan KC19-R10

In this borehole, the depth to the mine workings is 43 m and the thickness of the gypsum roof beam is 11.69 m. This area lies immediately below the R179. It is located above the maximum flooding level so has remained dry throughout its life. There is a large amount of angular debris on the floor, which suggests instability; the roof also shows signs of slabbing. The borehole core photographs and core log suggests that locally the gypsum forming the roof beam contains closely spaced fractures and areas of mudstone filled cavities. It is probable that the roof slabbing is caused by this different geology.

The surface survey monitoring network along the R179 road has a levelling point (nB10) which is located almost vertically above this area of the mine. The results from the monitoring point, a graph of which is shown in Figure 9-1, indicate that the average movement at this point is 3 mm, which is not significant. Whilst the data suggest that historically there has been no stability issues related to this area, SRK considers that the area may require specific attention going forward as discussed in the recommendations section.

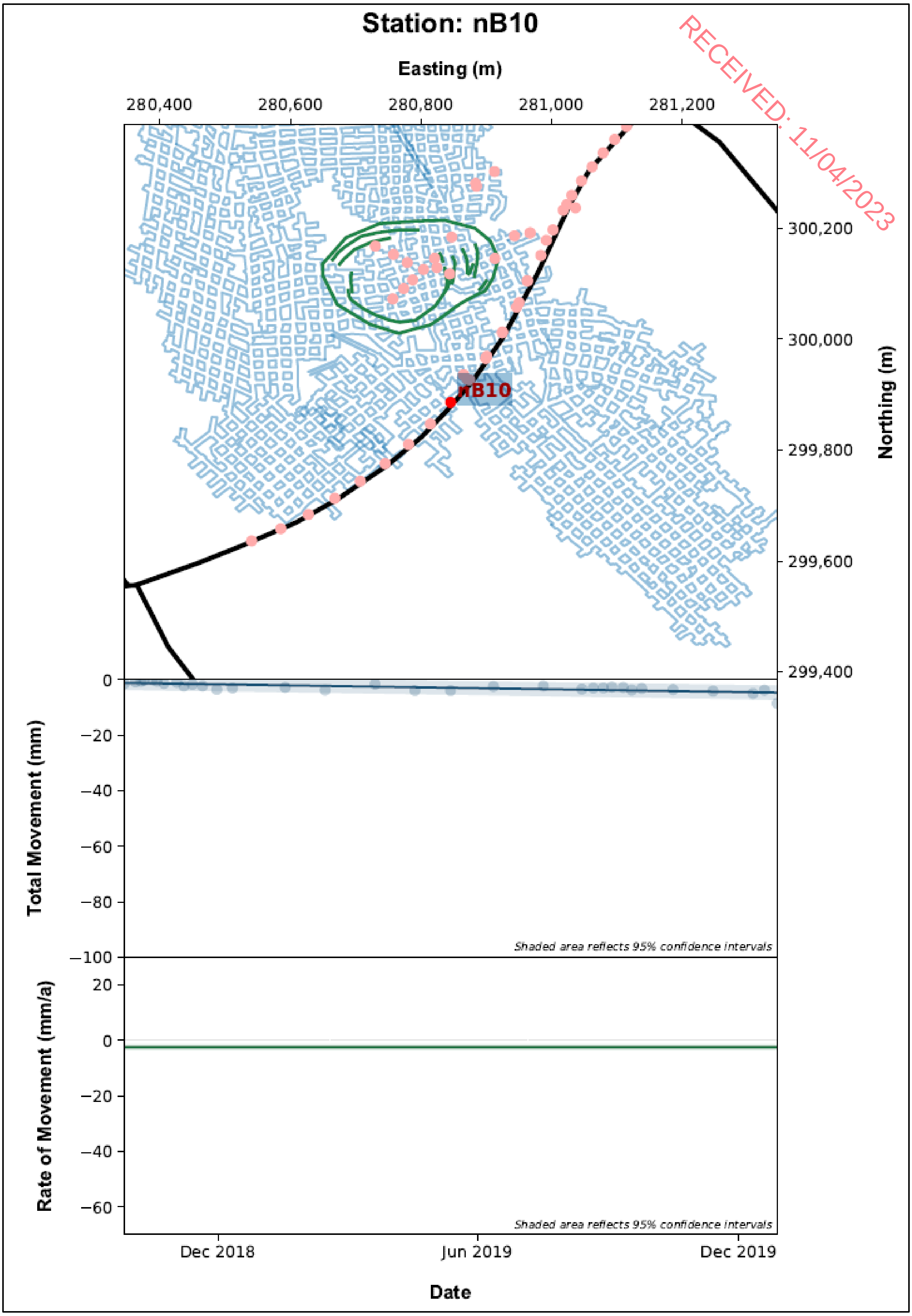

Figure 9-1: Surface Movement History: Station nB10

9.3 Finite Element Modelling Results

All the finite element modelling results indicate stable mine conditions. Limiting roof beam stability has been defined as a maximum deflection of 2% of roof beam thickness. None of the roof beams simulated exceeds the maximum deflection value. The deformation values indicate no instability of the pillars modelled. Simulated surface deformation is of the order of millimetres, which is consistent with the magnitude of the actual deformation being measured by Gyproc by the surface levelling network located along the north western edge of the R179 road. The modelling is therefore considered to be a reasonable simulation of current actual mine stability condition.

The October 2018 predictive modelling used gypsum strength parameters estimated from historical underground mapping. Those predictive finite element analyses returned surface deformations blow the R179 of between 0 mm and 8 mm. For the modelling reported herein, the surface deformation below the road varied from less than 1 mm to 2 mm. These are smaller but of the same order of magnitude as the earlier predictive modelling.

9.4 Crownhole Development Potential

The work carried out for the L4900 investigation and subsequent investigations into crownhole development at Drumgoosat identified a number of factors that were needed to create conditions amenable for the development of a crownhole. These are:

- A very thin (<1 m) or absent gypsum roof beam.
- Depth of mining less than 10 times the height of the mine openings. For an average mine height of 6 m, the potential for crownhole development would be greater where the depth of mining was less than 60 m below surface.
- Gypsum unit overlain by material that sufficiently weak to be able to flow or collapse into the mining void.
- The potential for crownhole development is heightened where underground workings have been flooded and the rock forming the roof beam has been weakened by wetting.

Below the R179, the mine workings in Zones 1, 2, and 3 are all less than 60 m below surface. The investigation has indicated that in the location where boreholes have been drilled, the thickness of the Lower Gypsum roof beam lies in the range 5 m to 18 m.

Below the R179 road in Zones 1, 3, 4, and 5, the Lower Gypsum is overlain by mudstone which in turn is overlain by unmined Upper Gypsum between less than 1 m and 10 m thick. In Zone 2, the Upper Gypsum is very thin or absent immediately below the road but thickens out to the north west of the road.

The underground workings in Zones 1, 2, and 3 are dry workings as they all lie above the level of maximum mine flooding. The underground workings below the road in Zones 4 and 5 have been permanently under water since mining at Drumgoosat ceased.

Based on this description, none of the Zones investigated contains all of the criteria required for crownhole development. Historically, the only crownhole that has occurred adjacent to the R179 is located north west of the road on northern end of Zone 1. The laser survey undertaken in borehole KC20-R01 has identified a chimney hole in the roof of one of the underground workings to the south east of the road; however, because there is a competent layer of upper gypsum above these workings, it is very unlikely that the chimney hole at this location will propagate into a crownhole at surface.

Based on the information generated by this investigation, the risk of crownholes developing along or adjacent to the R179 in the future is considered to be very low. There is a slightly greater, albeit small, risk in Zone 1 due to the historical occurrence of crownholes in the area. The monitoring measures that Gyproc have in place, surface levelling and visual inspections, are appropriate for managing the slightly greater risk.

9.5 Overall Conclusions on R179 Road Stability

Historically, there has been no instance of mine induced stability along and adjacent to the R179. Based on the investigations carried out, the geotechnical analysis, and interpretation of the cloudscan laser surveys, no high risk, unstable undermining areas have been identified.

The laser surveys and the geotechnical borehole logging have provided strong evidence that there has been virtually no deterioration in the mine conditions since the excavations were created. This provides confidence that the roof beams and pillars are still doing the job for which they were designed, which is to support the underground openings and prevent surface subsidence. On this basis, the R179 continues to be safe to use.

Gyproc has already initiated a number of measures to provide assurance and early warning of potential underground instability. These are:

- monitoring of a network of surface levelling points along and adjacent to the R179: and
- installation of extensometers to measure roof beam movement in a number of the boreholes drilled as part of this investigation.

These measures are described in Section 10.

10 RECOMMENDATIONS

10.1 Surface Monitoring

Following the September 2018 subsidence event, a network of levelling stations was installed adjacent to the R179 road. These are being surveyed by Gyproc on a regular basis. The levelling station network is shown in Figure 10-1.

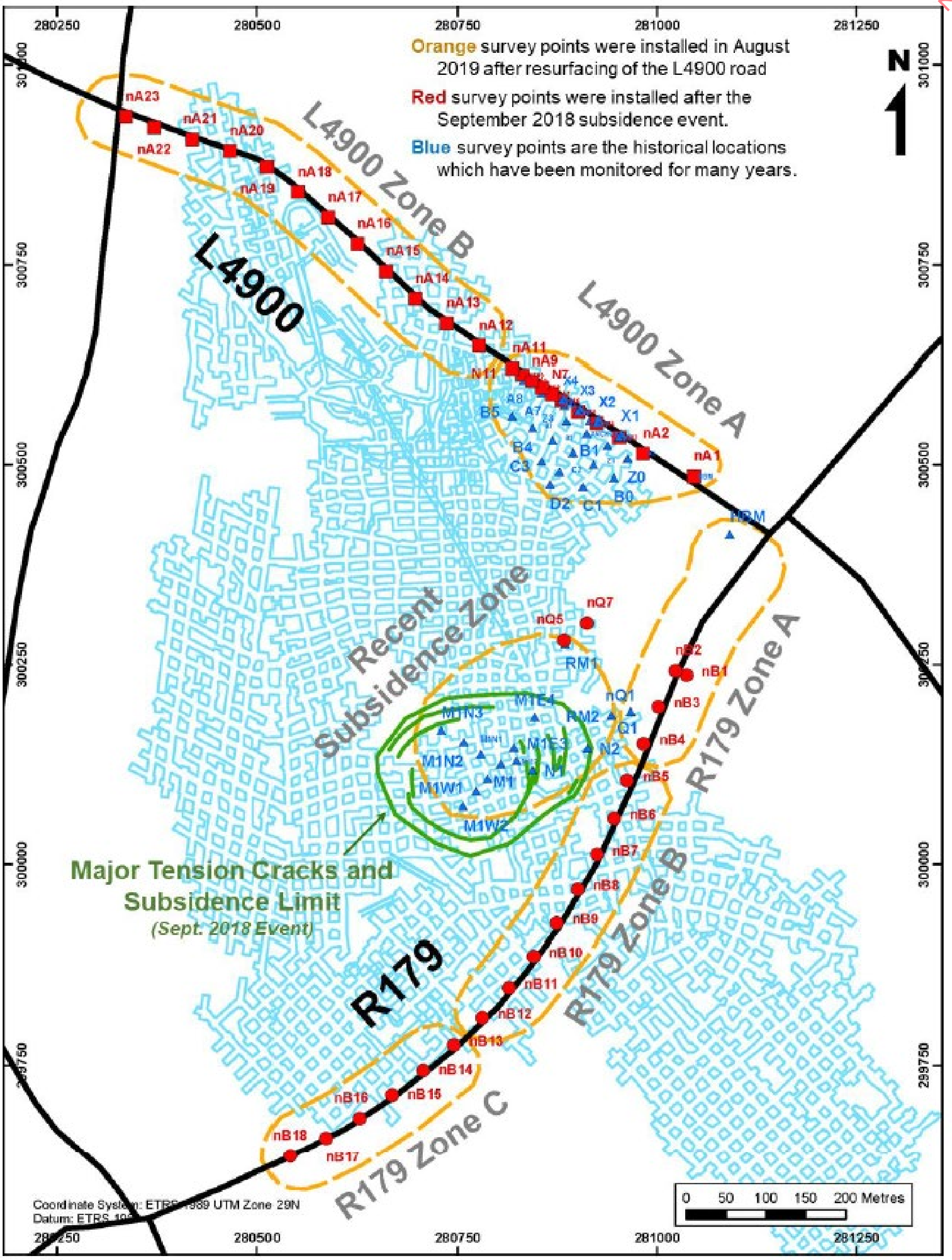

Figure 10-1: Surface Levelling Network

The levelling points are located above most of the areas investigated by drilling and laser surveys. Interpretation of the levelling data by SRK, which is being carried out on quarterly basis, shows very little increase in movement with time, indicating that there is currently no adverse underground mining. The movement magnitudes are associated with a Trigger Action Response Plan (TARP) developed for the L4900. The movements of the R179 levelling points fall within the Extremely Low Risk zone of the TARP, indicating that the movement is not significant. The monitoring should continue as defined in the TARP, should be reviewed, and appropriate actions taken if and when defined surface movement trigger levels are exceeded.

10.2 Borehole Extensometers

Gyproc has identified boreholes where it wished to install extensometers and with which SRK agrees. These are:

- KC19-R03
- KC19-R06
- KC18K-R08
- KC19-R11
- KC19-R12

The location of the boreholes is shown in the Gyproc plan in Figure 10-2. SRK had previously been requested to determine the position of the two extensometer anchor positions in each of these boreholes. Given the interpretation of the laser surveys, however, SRK recommends that the extensometer proposed for borehole KC20-R11 should be installed in borehole KC20-R10. The proposed extensometer boreholes with their anchor positions (including borehole KKC20-R10) are shown in the PowerPoint slides in Appendix E.

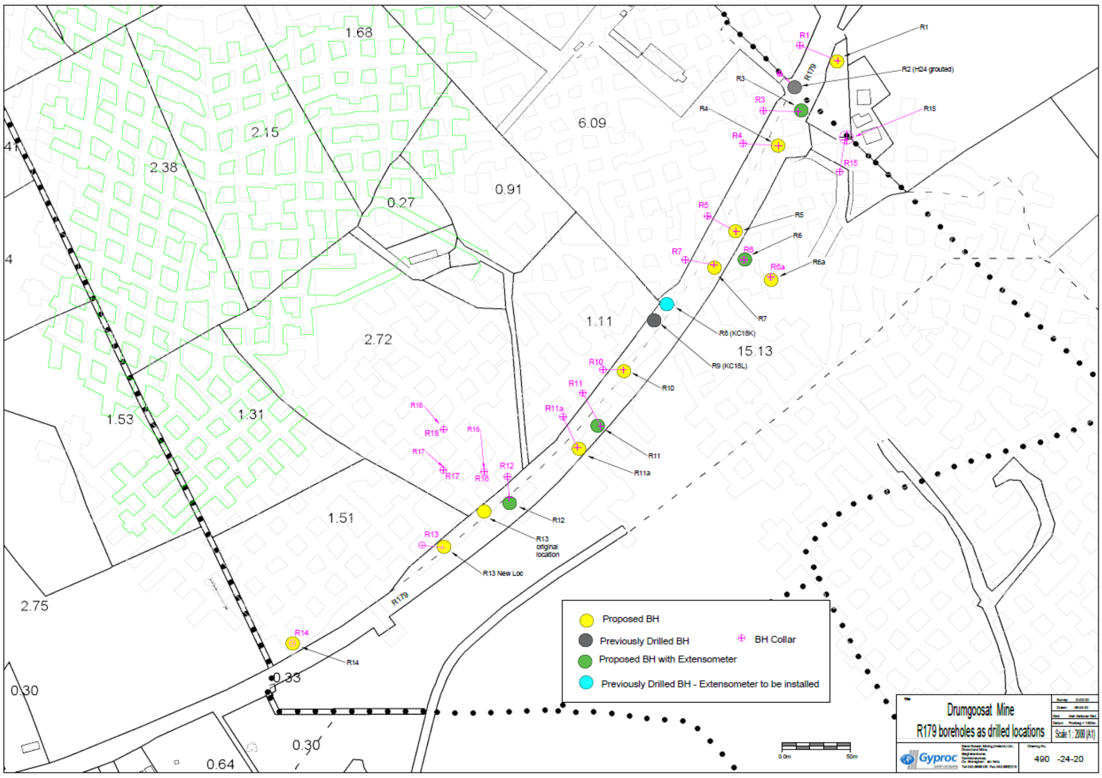

Figure 10-2: Borehole Extensometer Locations

10.3 3D Numerical Modelling of Pillar R12

Pillar R12 is a small pillar located adjacent to the north western side of the R179 carriage way. Because of its small size and its location, this pillar has been subject of specific analyses. Boreholes (KC18L-R9 and KC18K-R8) have been drilled either side of the pillar and laser scanning has been carried out down each of the boreholes on two separate occasions. The pillar along with the laser scans are shown in Figure 10-3.

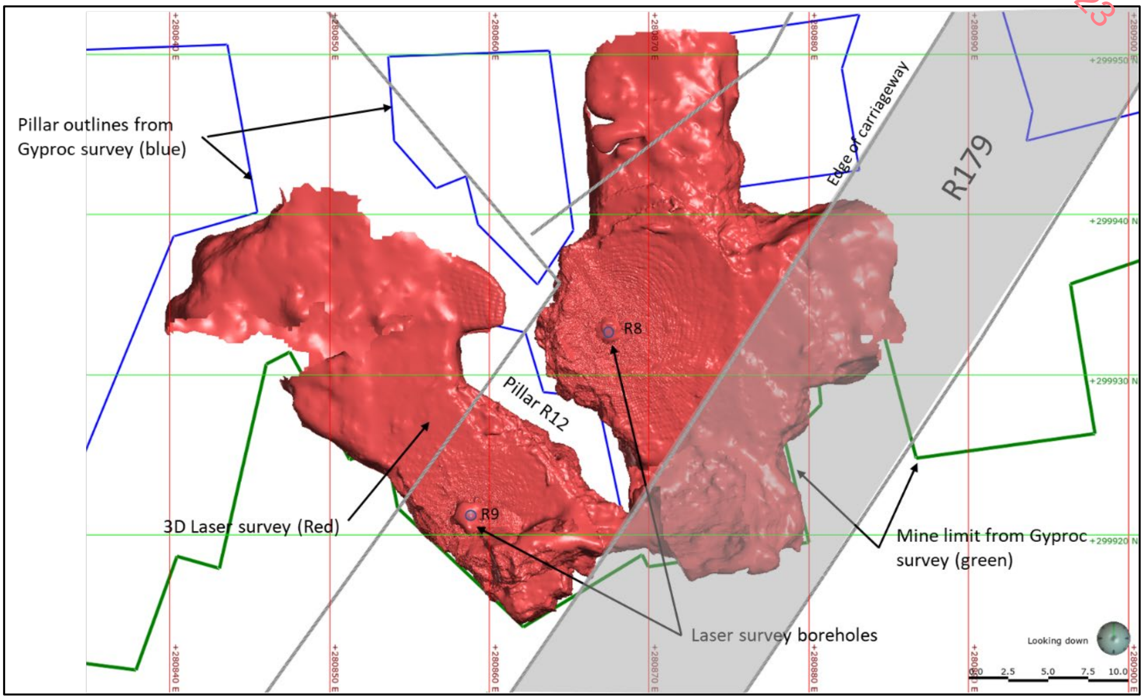

Figure 10-3: Pillar R12 and 3D Laser Scans

Whilst 2D stability analyses have been carried out along cross sections through the narrowest part of the pillar, which shows that the pillar is stable, the laser scans around the pillar are now sufficiently detailed to allow a 3D model of the pillar to be constructed. SRK recommends that a 3D analysis of the pillar is undertaken using the computer codes FLAC3D or 3DEC. The results of a 3D analysis would provide assurance of the stability of the pillar and the stability of the R179 road.

For and on behalf of SRK Consulting (UK) Limited

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APPENDIX

A GEOLOGICAL LOGS AND CORE PHOTOS

APPENDIX

B GEOTECHINCAL LOGS

APPENDIX

C PROCESSING 3D LASER SCANS

APPENDIX

D DOWNHOLE CLOUD SCAN ANALYSIS

APPENDIX

E BOREHOLE EXTENSOMETER ANCHOR POSITIONS.

APPENDIX 7.8
Drumgoosat Monitoring R179 Trigger Action Response Plan (TARP) - SRK
July 2020

DRUMGOOSAT R179 MONITORING TRIGGER ACTION RESPONSE PLAN (TARP)

Prepared For

Saint-Gobain Mining (Ireland) Ltd

Report Prepared by

srk consulting

SRK Consulting (UK) Limited

30787

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Date:		July 2020
Project Number:		30787
SRK Project Director:	Max Brown	Principal Consultant - Geotechnical
SRK Project Manager:	Neil Marshall	Corporate Consultant - Geotechnical
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DRUMGOOSAT R179 MONITORING

TRIGGER ACTION RESPONSE PLAN (TARP)

1 INTRODUCTION

The Trigger Action Response Plan (TARP) is a common tool in the mining industry. In particular, it is used for managing potentially critical situations from a mine safety point of view.

A TARP document sets out a certain set of conditions (or "triggers") and a set of actions which mine managers and supervisors must follow when those trigger events occur.

This document has been prepared by SRK Consulting (UK) Ltd for Gyproc Ltd and presents a TARP for the monitoring of gypsum roof beam stability at various locations where the R179 road has been undermined by the old Drumgoosat Mine underground workings.

The monitoring system comprises five multi-point borehole extensometers along with precise levelling points located on the surface in the vicinity of the collar positions of the extensometers.

2 MONITORING PURPOSE

The purpose of the monitoring system is to provide early warning of failure of the gypsum roof beams that lie at depth below the carriageway of the R179 road and potential migration of instability to surface that may affect the stability of the road and the safety of road users. SRK notes that historically there have been no instability events associated with the underground mine within 50m of the R179 carriageway.

3 POTENTIAL FAILURE MECHANISM

The underground workings below the R179 road have been in place and stable for at least 40 years. Extensive core drilling through the beams has confirmed the competence and thickness of the gypsum forming the roof beams. Stability analyses undertaken by SRK has indicated the roof beams to be currently stable. Future instability, should it occur, is likely to be progressive in nature and take the following form:

1. Immediately above the underground room the roof beam blocks or slabs of gypsum, isolated by flat bedding planes and vertical joint planes, may start detaching themselves from the roof resulting in a thinning of the roof beam at the point of detachment.
2. If this process continues to propagate through the roof beam eventually the roof beam becomes so thin the weight of the overlying drift, mudstone, and dolerite will cause the roof beam to collapse and fail.
3. With no roof beam to support the overlying drift, mudstone, and dolerite, this material becomes free to flow or fall into the mine workings. This tends to be a slow process as these materials are, to a degree, self-supporting; however, ultimately, there could be a collapse causing the development a crownhole on surface.

4 THE MONITORING SYSTEM

The monitoring system being installed by Gyproc comprises multi-point borehole extensometers (MPBX) installed in five of the boreholes drilled to investigate the condition of the underground workings below the R179. Each extensometer, which is manufactured and installed by RST Instruments Ltd, will have two anchors. Depending on the thickness of the roof beam being monitored, one anchor will be located in the roof beam about 1 m above the underground working; and the second will be located about 1 m below the top of the gypsum roof beam. These will be grouted in place with cement grout and will be connected with fibreglass rods to a reading head. The reading head will be located inside a manhole at surface. The extensometers will be connected to a data logger that will automatically collect anchor movement data at pre-programmed times and these will be transmitted wirelessly back to the Gyproc survey office for processing and evaluation.

A schematic showing the layout of the two anchor borehole extensometer installation is presented in Figure 4-1. The figure shows the upper and lower anchors cemented into the gypsum roof beam along with rods that link the anchors to the measuring head which is located in a manhole at surface. The data logger collects readings from the measuring heads of all five extensometers.

In addition to the in-ground monitoring, surface levelling stations will be used to determine whether underground movement is causing surface subsidence. Gyproc has a network of surface levelling stations along the side of the R179 road which were installed after the September 2018 subsidence event. The levelling stations closest to each extensometer collar position will form part of the monitoring system.

Data from the monitoring station and from the extensometers will provide the information to allow Gyproc to make informed decisions on the stability of the areas being monitored. The surface levelling stations are the ones which are located above the underground rooms being monitored. SRK notes that only one of the extensometer installations is not associated with an existing levelling point. Borehole KC20-R06 has been drilled on the eastern side of the carriageway whereas all of the levelling points are on the western side of the carriageway. A new levelling point located close to the monitoring anchor head position will be required. Details of the monitoring stations are presented in Table 4-1.

Plans and sections showing the location of each extensometer, the approximate position of the anchor points above the underground workings, and the position of the surface levelling stations adjacent to the extensometer heads are presented in Appendix A.

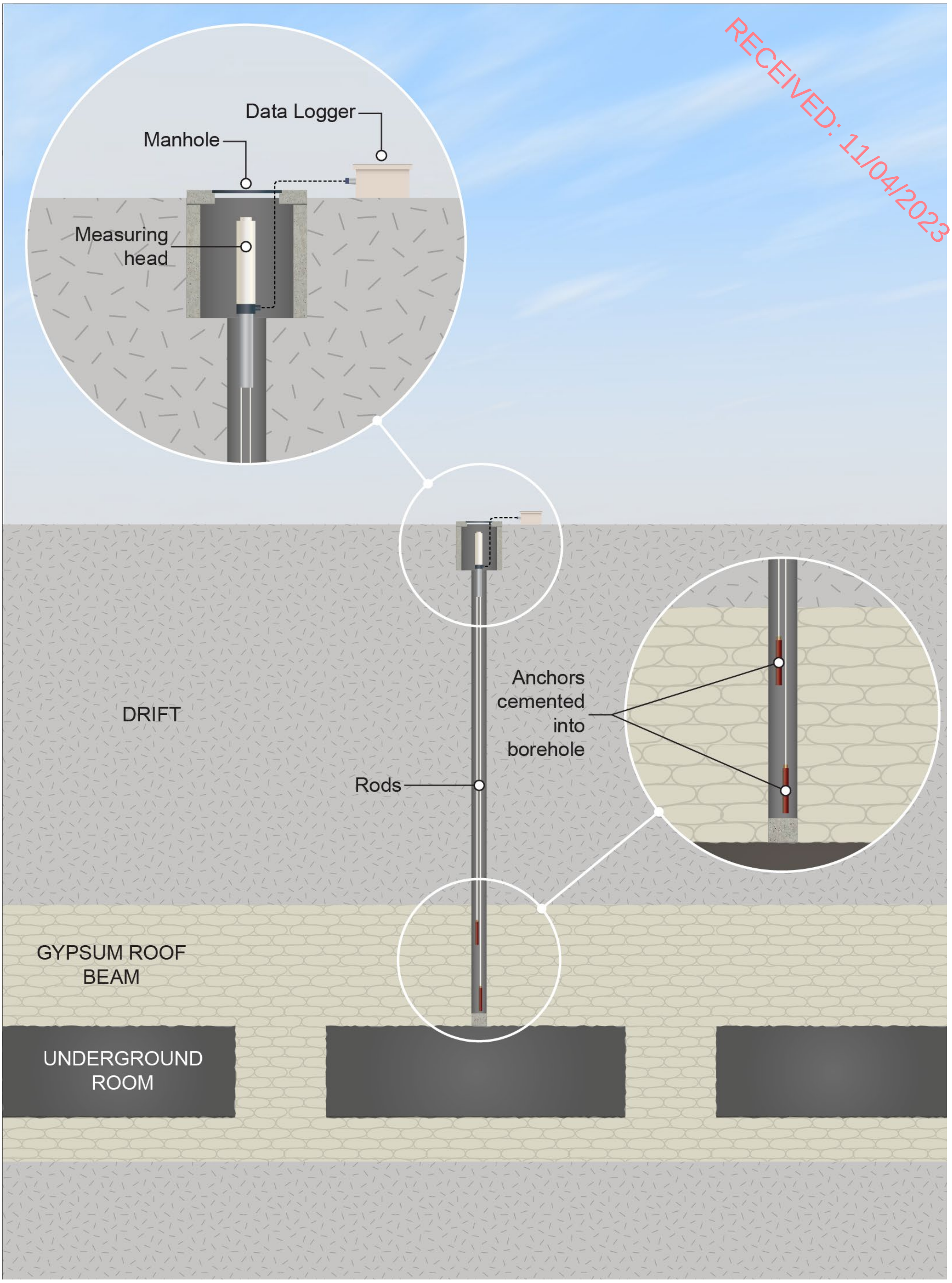


Figure 4-1: Schematic of the Two Anchor Borehole Extensometer Installation

Table 4-1: Monitoring System Details

	Anchor Head Position (m)			BH Dip	Roof Beam[1]		Anchor Point Position[2] (m)		Closest Levelling Stations	New Station Required	New Levelling Station Co-Ordinates	
Borehole	x	y	z	(°)	Depth (m)	Thickness (m)	Upper	Lower			x	y
KC20-R3	280939.5	300080.8	1057.97	59	34.34	8.58	42 (36)	48 (41)	nB5 and nB6	No		
KC20-R6	280925.2	299967.4	1055.00	88	25.12	13.89	27 (27)	37 (37)		Yes	280925.2	299967.4
KC18K-R08	280868.8	299934.0	1053.38	90	31.40	9.80	33 (33)	39 (39)	nM2 and nB9	No		
KC19-R10	280821.8	299884.3	1049.57	71	30.97	11.26	36 (34)	42 (40)	nB10	No		
KC19-R12[3]	280751.7	299803.2	1046.76	75	51.43	8.97	54.5 (53)	60 (58)	nB13	No		

Notes:

1. *Most of the boreholes are inclined as in order to intersect the mine workings below the road the boreholes had to be collared off the carriageway. True vertical depth to and thickness of the gypsum roof beam are shown in the table.*

2. *Anchor positions quoted as distance down the borehole. Vertical depth of anchor below surface shown in brackets.*

3. *Borehole R12 intersected an air pocket in an area of temporary flooded workings. This borehole has been sealed off to prevent water ingress to this area of the mine. The laser scan and extensometer installation in this borehole will occur as soon as the water level in the rest of the mine has been drawn down below the level of the workings intersected by this borehole.*

5 THE TARP

It should be noted here that a TARP is a live and evolving document. Whilst an initial set of triggers needs to be defined as a starting point, these are likely to change as monitoring data is collected and the stability of the underground workings becomes better understood. The TARP document, trigger action levels, and appropriate responses should be reviewed in conjunction with the response history at least once a year.

5.1 Extensometer Triggers

The extensometers have a maximum range of 50 mm. The expected failure mechanism of bedding and joint bounded blocks detaching themselves from the roof will be preceded by separation of the blocks along bedding planes. The extensometer movement range will be sufficient to identify when this may begin to happen. As there is no underground access, it becomes difficult to define appropriate movement trigger levels. Bigby, MacAndrew and Hurt (2010)[1] state that in active underground coal mines warning of roof instability is usually provided at roof deformations of between 10 mm and 25 mm depending on rock conditions, whist action is required at roof deformations of 25 mm to 50 mm. At Drumgoosat, there is no access to the underground workings, but as a starting point for the TARP, these ranges of deformation have been used to define a range of trigger actions as shown in Table 5-1.

5.2 Surface Levelling Station Triggers

The triggers for the surface levelling stations have been defined using the levelling data collected from the stations installed and monitored since 1988. The stations that have been monitored since 1988 collected information related to the historical subsidence that occurred along the L4900 road in late 1999. It illustrates the full response of the ground surface to a subsidence event from a pre-event scenario, through a time of increasing subsidence activity to post event stabilisation.

Figure 5-1 shows movement and rates of movement expressed in millimetres per year (mm/a) for Station A2 which was installed in 1988 and for Station LA4 which replaced Station A2 in late 2018 after this station was destroyed.

The graph in Figure 5-1 shows that as mine instability developed, there was a gradual increase in rate of movement from a background level of 25 mm/a or less to 75 mm/a. As the ground subsidence developed, the rate of movement in about 1997 increased to a maximum of about 135 mm/a. After this time, the rate of subsidence decreased gradually until by about 2002 the subsidence rate had reduced to the background rate of 25 mm/a, or less. The rate of surface movement currently is of the order of 2mm/a and this has remained constant for a number of years.

Based on this interpretation the rate of movement values of 25 mm/a and 75 mm/a have been used as the initial surface movement trigger points for the TARP, which is shown in Table 5.1.

[1] *Bigby, D, MacAndrew, K and Hurt, K, Innovations in mine roadway stability monitoring using dual height and remote reading electronic telltales, in Aziz, N (ed), 10th Underground Coal Operators' Conference, University of Wollongong & the Australasian Institute of Mining and Metallurgy, 2010*

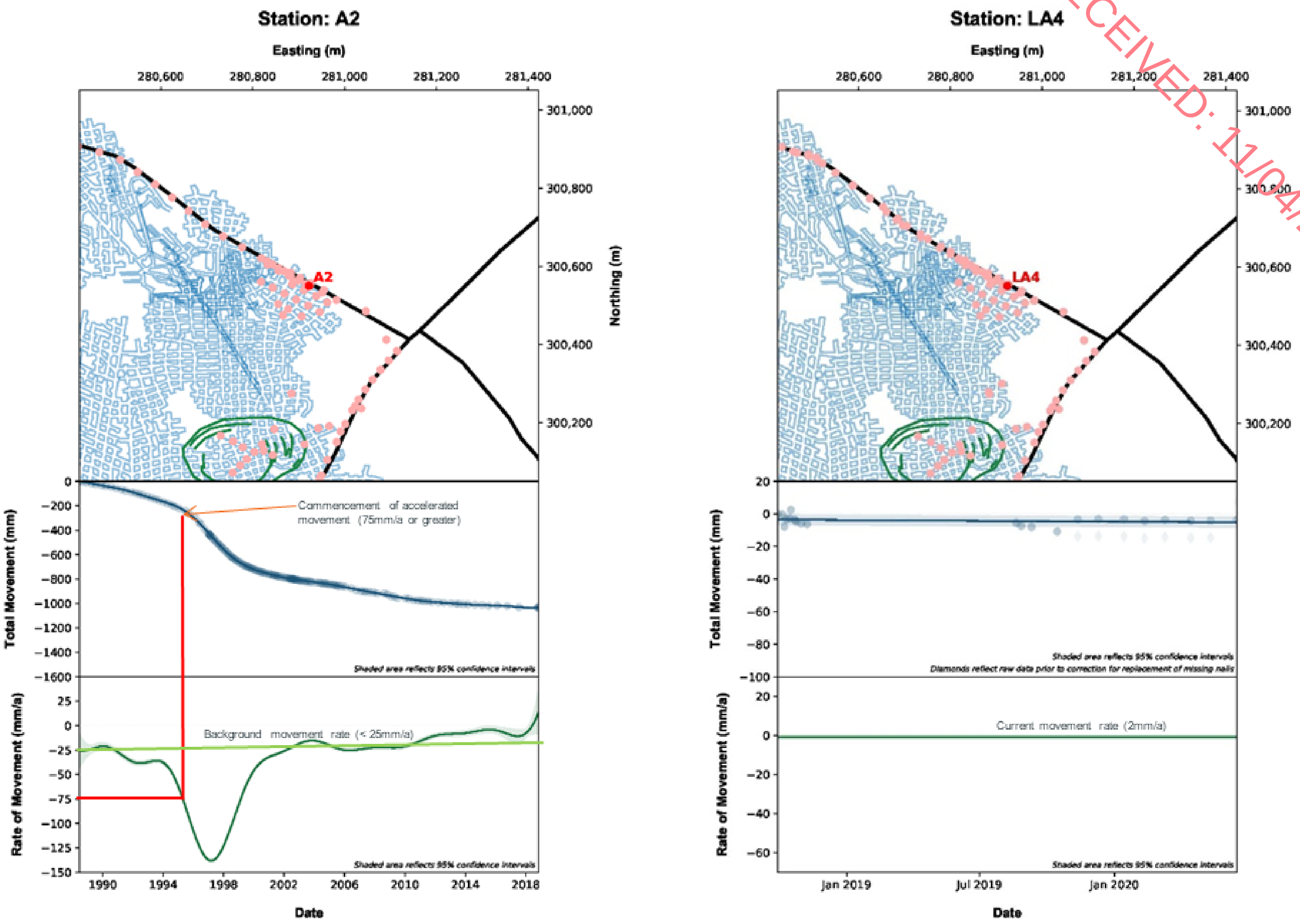


Figure 5-1: Surface Survey Monitoring Data Used to Define Surface Movement Triggers

Table 5-1: Drumgoosat Mine – R179 Road Stability Monitoring TARP

	Extremely Low Risk	Very Low Risk	Low Risk	Moderate Risk	High Risk
Surface Levelling Point	<25mm/a	<25mm/a	<25mm/a	25 - 75mm/a	75 - 135mm/a
Upper Roof Beam Anchor	<10mm	<10mm	10mm - 40mm	> 40mm	Extensometer Range Exceeded
Lower Roof Beam Anchor	<10mm	10mm - 40mm	10mm - 40mm	> 40mm	Extensometer Range Exceeded
Monitoring Frequency					
Surface Monitoring Points	Monthly	Monthly	Monthly	Fortnightly	Weekly
Extensometer Reading	Every 12 hours	Every 6 hours	Every 3 hours	Every 1 hour	
Reporting	Monthly	Fortnightly	Fortnightly	Weekly	NA
Status Change	Extremely Low to Very Low				
		Very Low to Low			
			Low To Moderate		
				Moderate to High	
Trigger Alarm	10mm (lower anchor)				
		10mm (lower and upper anchor)			
			40mm (lower anchor)		
				40mm (lower and upper anchor)	
Speed of Response	Next working day				
Monitoring Frequency	Reviewed and adjusted on the next working day				
Notifications	MCC and EMD advised that Risk Status has changed				
		MCC and EMD advised that Risk Status has changed			
			MCC and EMD advised that Risk Status has changed, further investigations launched		
				MCC and EMD advised that Risk Status has changed, further investigations launched	
Further Investigations	None	None	None	Carry out an underground laser scanning survey to confirm the integrity of the mine workings and roof beam	Consider road closure. Conduct more detailed sub-surface investigation of ground movement.
Interpretation	Normal conditions. Stable roof beam	Some deformation of roof beam immediately above room. Deformation has not propagated through the full roof beam thickness.	Deformation occurring through full roof beam thickness. Roof beam integrity has not been compromised.	Deformation occurring through full roof beam thickness. Roof beam integrity is beginning to be compromised resulting in increased surface movement.	Extensometer range has been exceeded indicating that roof beam might have failed. High surface deformation indicates that roof beam no longer provides support to the overlying material and that development of a subsidence feature on surface could occur.

5.3 Monitoring and Reporting Frequency

5.3.1 Extensometer Data Logger

The extensometer data logger will collect movement data from each anchor once every hour.

The data logger is programmed to calculate the average of the readings from each anchor over a defined period and report these. The time period will change depending on the Risk range in which the anchor movements lie, as shown in Table 5-1. These periods are:

- Every 12 hours when readings fall in the Extremely Low Risk range, increasing to:
- Every 6 hours when readings fall in the Very Low Risk Range, increasing to:
- Every 3 hours when readings fall in the Low Risk Range, increasing to:
- Every 1 hour when readings fall in the Moderate and High Risk Range.

The purpose of averaging the readings is to remove “noise” and unnecessary alarms from the system.

At each reporting frequency interval, the logger will record the arithmetic mean position of the extensometer compared to its datum since the previous report was made. A record will be generated of the mean deviation from datum of the extensometer over the previous time as defined above.

5.3.2 Data Management, Review, and Reporting

The extensometer data logger will store the data for a number of weeks. The Mine Surveyor will connect to the logger twice per week and make a copy of the logger information onto an office-based computer. At the monitoring frequency defined in the TARP and depending on the current risk category status, the Mine Surveyor will review the extensometer data and make a report to the Mine Manager which will be reviewed, any necessary actions identified, counter signed, and filed.

5.3.3 Alarms

When extensometer movement exceeds the trigger level for each risk state, a number of alarms will be configured for each extensometer rod. These alarms will allow the data logger to assess if the upper or lower extensometer has moved from datum by more than 10 mm or by more than 40 mm. In either event, this will cause an automatic alarm to be generated for any of the extensometers. These alarms will be sent by SMS message to four specific mobile phones for action to be taken: Mine Manager, Mine Production Manager, Mine Maintenance Manager, and Mine Surveyor.

The event for which the alarms have triggered are located deep underground and indicate a small amount of movement in the roof beam – not roof beam failure. When an alarm is triggered, the surface monitoring point associated with the extensometer should be surveyed. There is only a risk of a surface event if the surface levelling stations associated with the extensometer for which the alarm has been triggered also show excessive movement.

5.4 TARP Review

It is recommended that six months after the monitoring system becomes live the trigger actions are reviewed and the TARP updated.

Future reviews and TARP updates should be scheduled at least annually.

For and on behalf of SRK Consulting (UK) Limited

[Name Surname], [Designation & Specialisation], **Project Manager** SRK Consulting (UK) Limited	[Name Surname], [Designation & Specialisation], **Project Director** SRK Consulting (UK) Limited

APPENDIX

A MONITORING NETWORK DETAILS

RECEIVED: 11/04/2023

Appendix A

Monitoring Network Details

Presenter:

Location: Location of the presentation

srk consulting

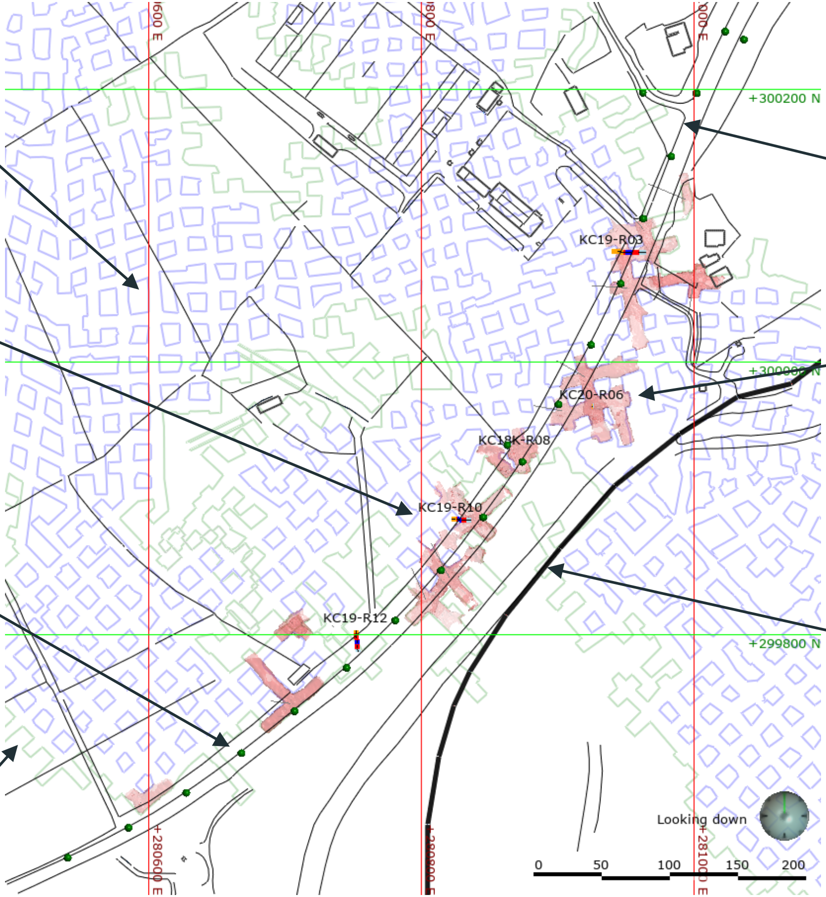
Extensometer and Surface Levelling Location Plan

Underground Mine Pillars (purple)

Extensometer borehole

Surface levelling stations (green)

Underground Mine Boundary (green)



R179 road

3D underground laser surveys (red)

Edge of Knocknacran Quarry

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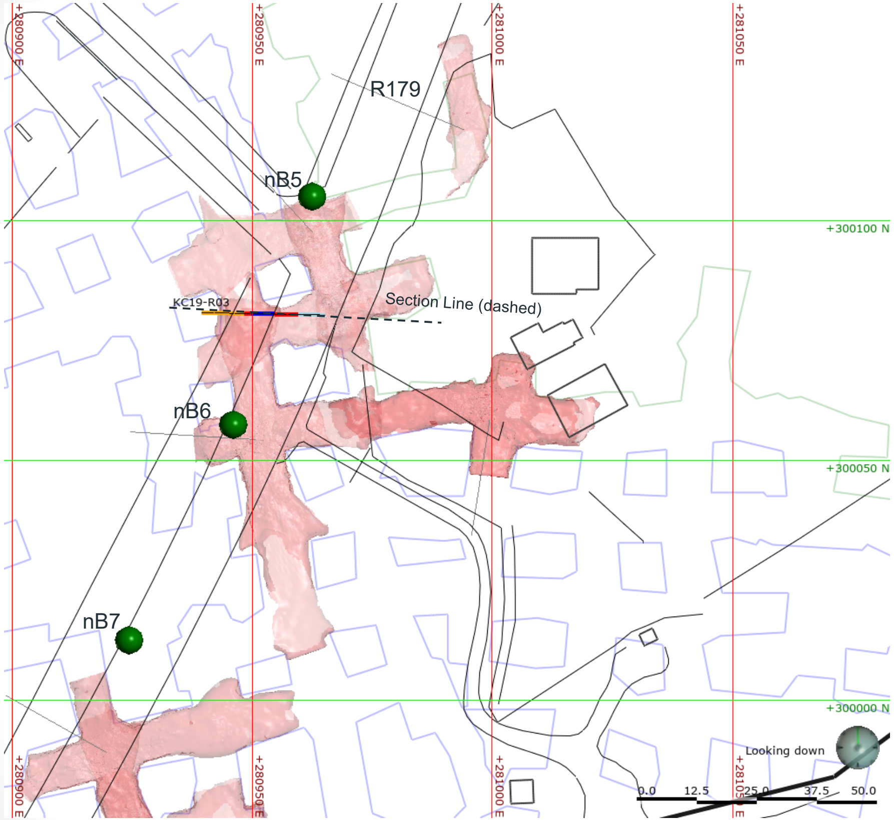
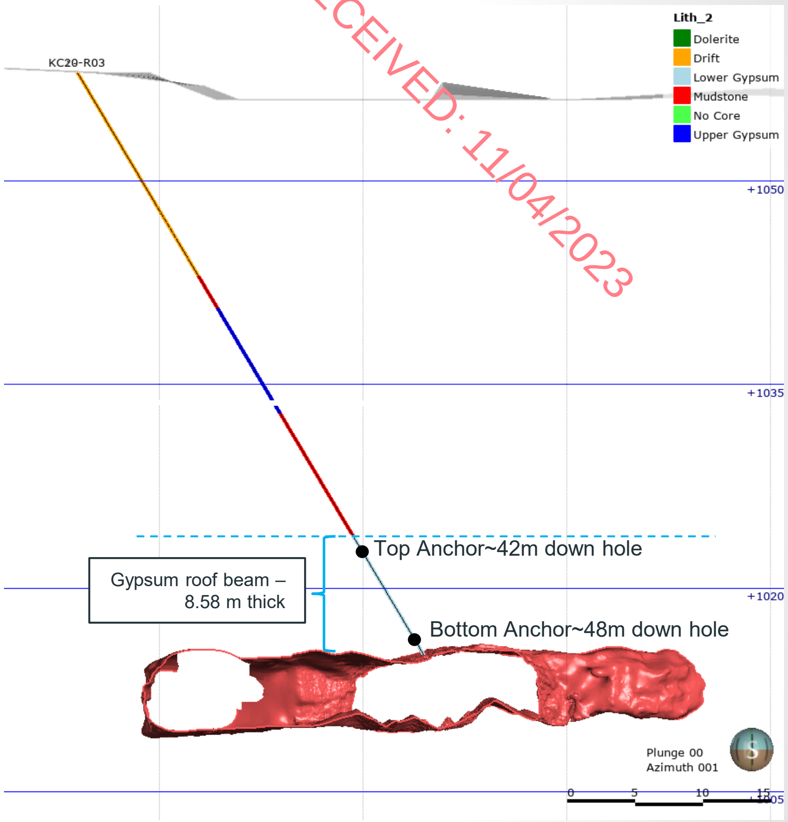
KC20-R03

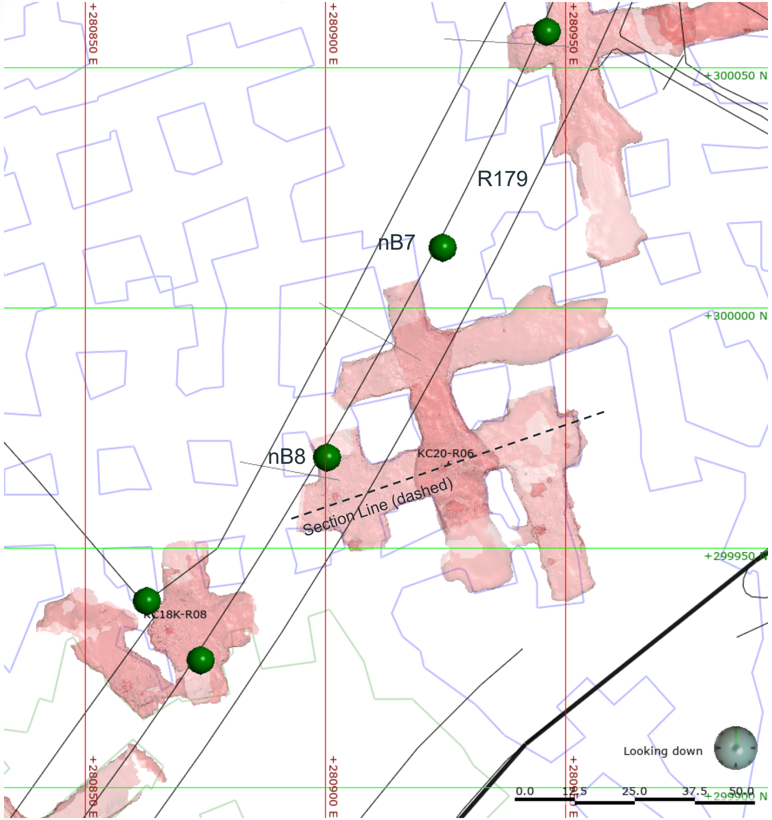


W – E Section along Line of Borehole

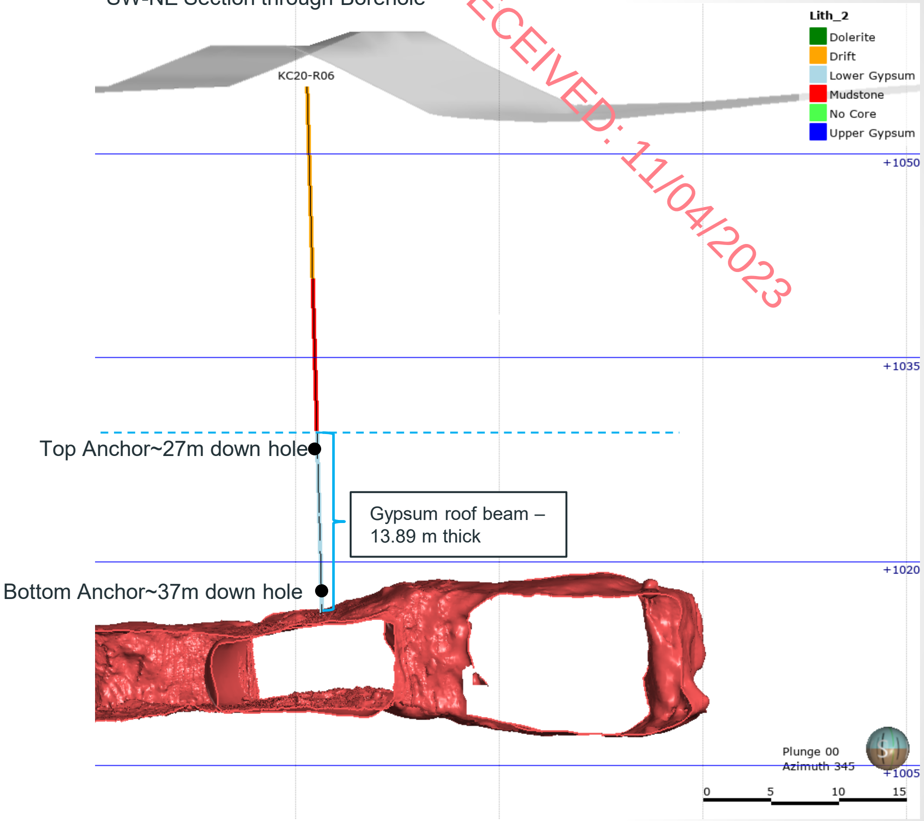


srk consulting

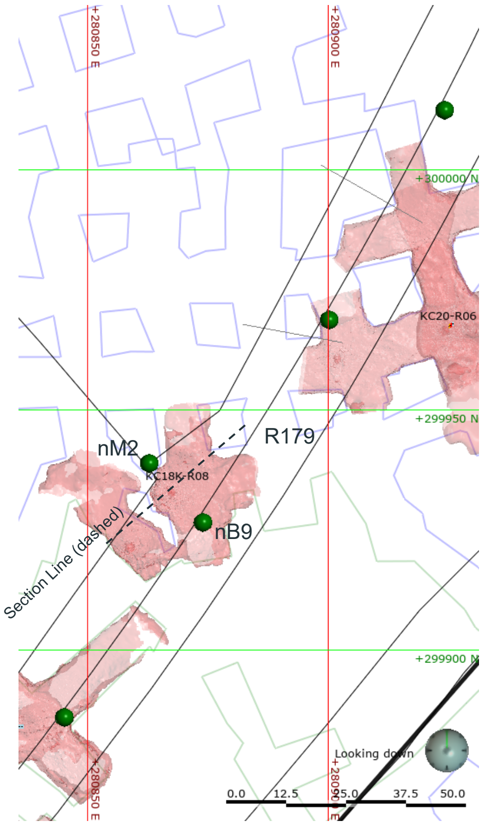
KC20-R06



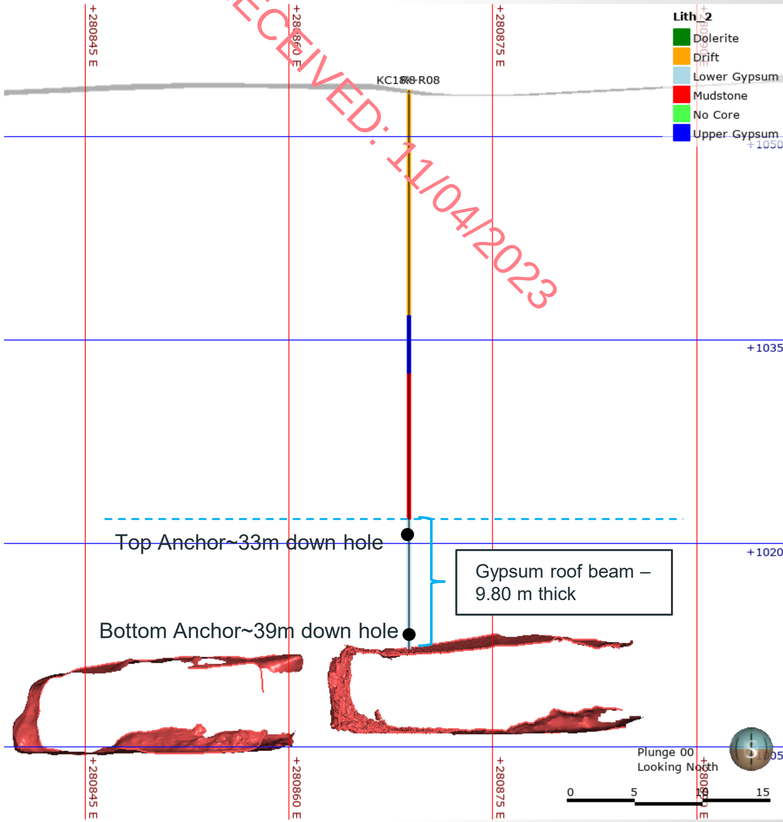
SW-NE Section through Borehole



KC18-R08



SW – NE Section through Borehole



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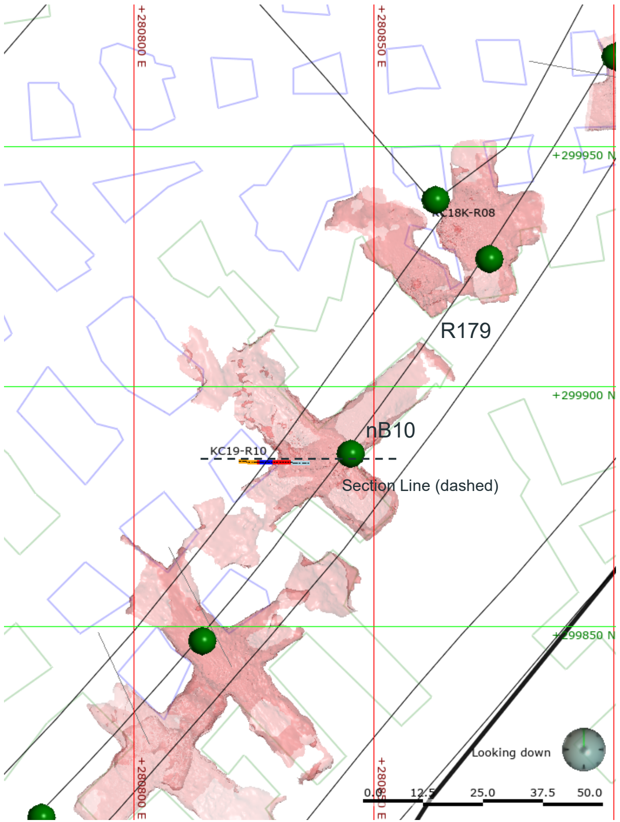

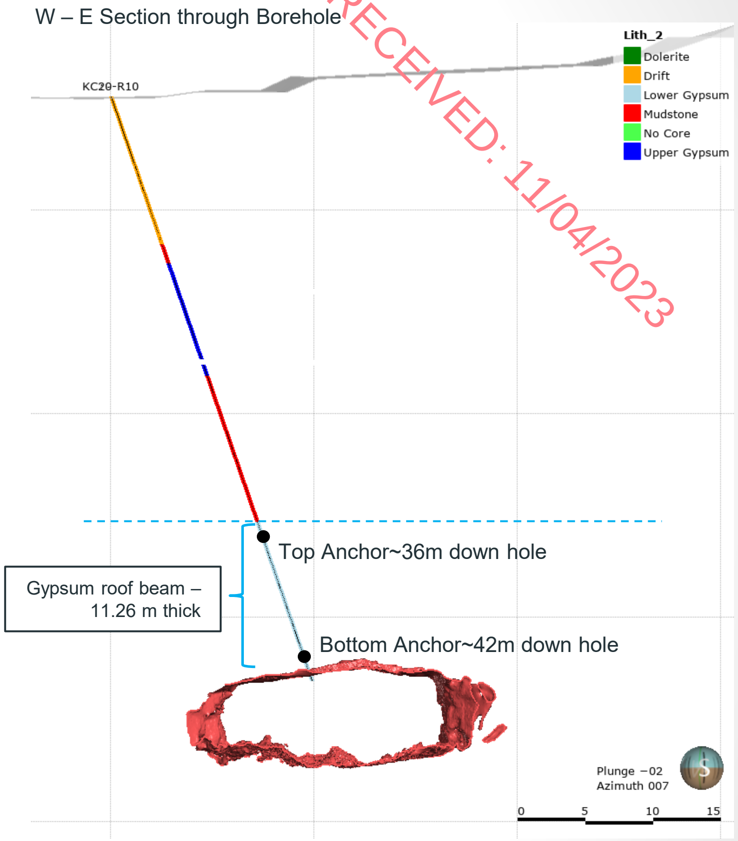

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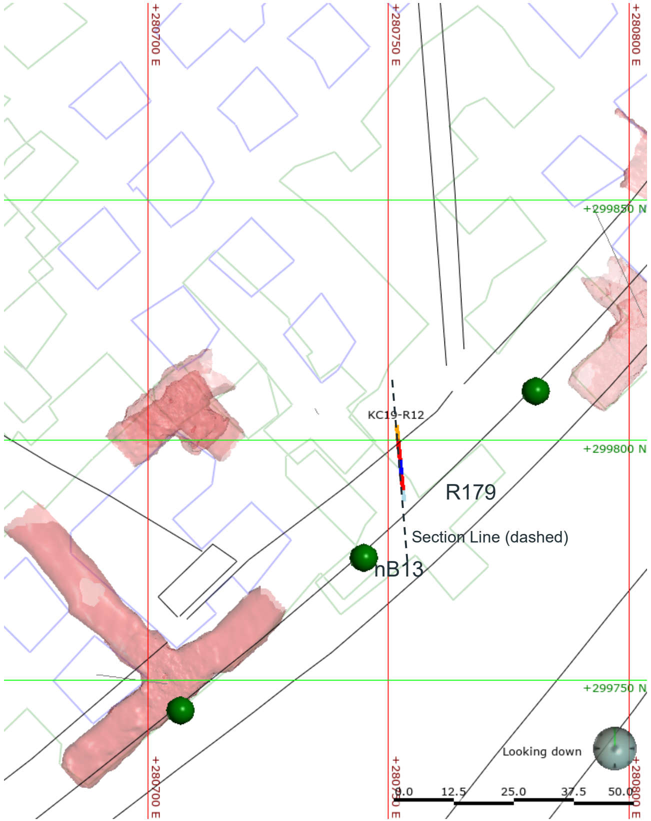
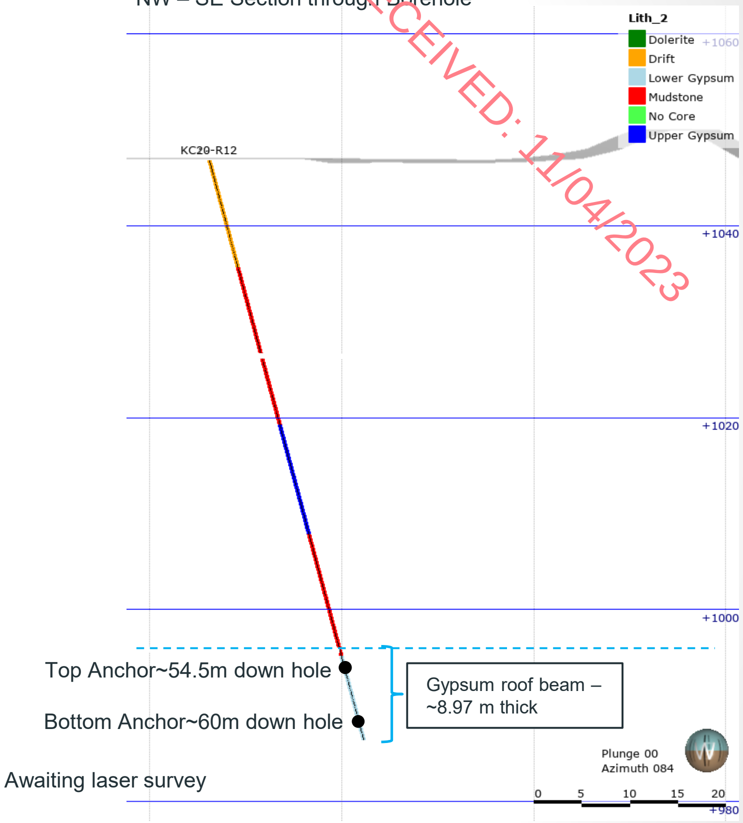
KC20-R12



NW – SE Section through Borehole



SRK Office Locations

>1,400 Professionals, 45 offices, 20 countries, 6 continents

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APPENDIX 7.9
Drumgoosat Monitoring L4900 Trigger Action Response Plan (TARP) - SRK August 2019

DRUMGOOSAT L4900 MONITORING TRIGGER ACTION RESPONSE PLAN (TARP)

Prepared For

Saint-Gobain Mining (Ireland) Ltd

Report Prepared by

srk consulting

SRK Consulting (UK) Limited

30238

SRK Legal Entity:		SRK Consulting (UK) Limited
SRK Address:		5th Floor Churchill House 17 Churchill Way Cardiff, CF10 2HH Wales, United Kingdom.
Date:		August 2019
Project Number:		30238
SRK Project Director:	Richard Oldcorn	Managing Director
SRK Project Manager:	Neil Marshall	Corporate Consultant - Geotechnical
Client Legal Entity:		Saint-Gobain Mining (Ireland) Ltd
Client Address:		Kingscourt, County Cavan A82 PF99, Ireland.

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DRUMGOOSAT L4900 MONITORING TRIGGER ACTION RESPONSE PLAN (TARP)

1 INTRODUCTION

The Trigger Action Response Plan (TARP) is a common tool in the mining industry. In particular, it is used for managing potentially critical situations from a mine safety point of view.

A TARP document sets out a certain set of conditions (or "triggers") and a set of actions which mine managers and supervisors must follow when those trigger events occur.

This document has been prepared by SRK Consulting (UK) Ltd for Gyproc Ltd and presents a TARP for the monitoring of gypsum roof beam stability at various locations where the L4900 road has been undermined by the old Drumgoosat Mine underground workings.

The monitoring system comprises eight multi-point borehole extensometers along with precise levelling points located on the surface in the vicinity of the collar positions of the extensometers.

2 MONITORING PURPOSE

The purpose of the monitoring system is to provide early warning of failure of the gypsum roof beams that lie at depth below the carriageway of the L4900 road and potential migration of instability to surface that may affect the stability of the road and the safety of road users.

3 POTENTIAL FAILURE MECHANISM

The underground workings below the L4900 road have been in place and stable for at least 40 years. Extensive core drilling through the beams has confirmed the competence and thickness of the gypsum forming the roof beams. Stability analyses undertaken by SRK and verified by Wardell Armstrong has indicated the roof beams to be currently stable. Future instability, should it occur, is likely to be progressive in nature and take the following form:

1. Immediately above the underground room the roof beam blocks or slabs of gypsum, isolated by flat bedding planes and vertical joint planes, may start detaching themselves from the roof resulting in a thinning of the roof beam at the point of detachment.
2. If this process continues to propagate through the roof beam eventually the roof beam becomes so thin the weight of the overlying drift, mudstone, and dolerite will cause the roof beam to collapse and fail.
3. With no roof beam to support the overlying drift, mudstone, and dolerite, this material becomes free to flow or fall into the mine workings. This tends to be a slow process as these materials are, to a degree, self-supporting; however, ultimately, there could be a collapse causing the development a crownhole on surface.

4 THE MONITORING SYSTEM

The monitoring system being installed by Gyproc comprises multi-point borehole extensometers (MPBX) installed in eight of the boreholes drilled to investigate the condition of the underground workings below the L4900. Each extensometer, which is manufactured and installed by RST Instruments Ltd, will have two anchors. Depending on the thickness of the roof beam being monitored, one anchor will be located in the roof beam about 1 m above the underground working; and the second will be located about 1 m below the top of the gypsum roof beam. These will be grouted in place with cement grout and will be connected with fibreglass rods to a reading head. The reading head will be located inside a manhole at surface. The extensometers will be connected to a data logger that will automatically collect anchor movement data at pre-programmed times and these will be transmitted wirelessly back to the Gyproc survey office for processing and evaluation.

A schematic showing the layout of the two anchor borehole extensometer installation is presented in Figure 4-1. The figure shows the upper and lower anchors cemented into the gypsum roof beam along with rods that link the anchors to the measuring head which is located in a manhole at surface. The data logger collects readings from the measuring heads of all eight extensometers.

In addition to the in-ground monitoring, surface levelling stations will be used to determine whether underground movement is causing surface subsidence. Gyproc has a network of surface levelling stations along the side of the L4900 road. Some of the stations were installed recently, after the September 2018 subsidence event, while some have been in place since the late 1980s. The levelling stations closest to each extensometer collar position will form part of the monitoring system.

Data from the monitoring station and from the extensometers will provide the information to allow Gyproc to make informed decisions on the stability of the areas being monitored. SRK notes that some of the extensometer installations are too far away from existing levelling stations, so a number of new stations will need to be installed. Details of the monitoring stations are presented in Table 4-1.

Plans and sections showing the location of each extensometer, the approximate position of the anchor points above the underground workings, and the position of the surface levelling stations are presented in Appendix A.

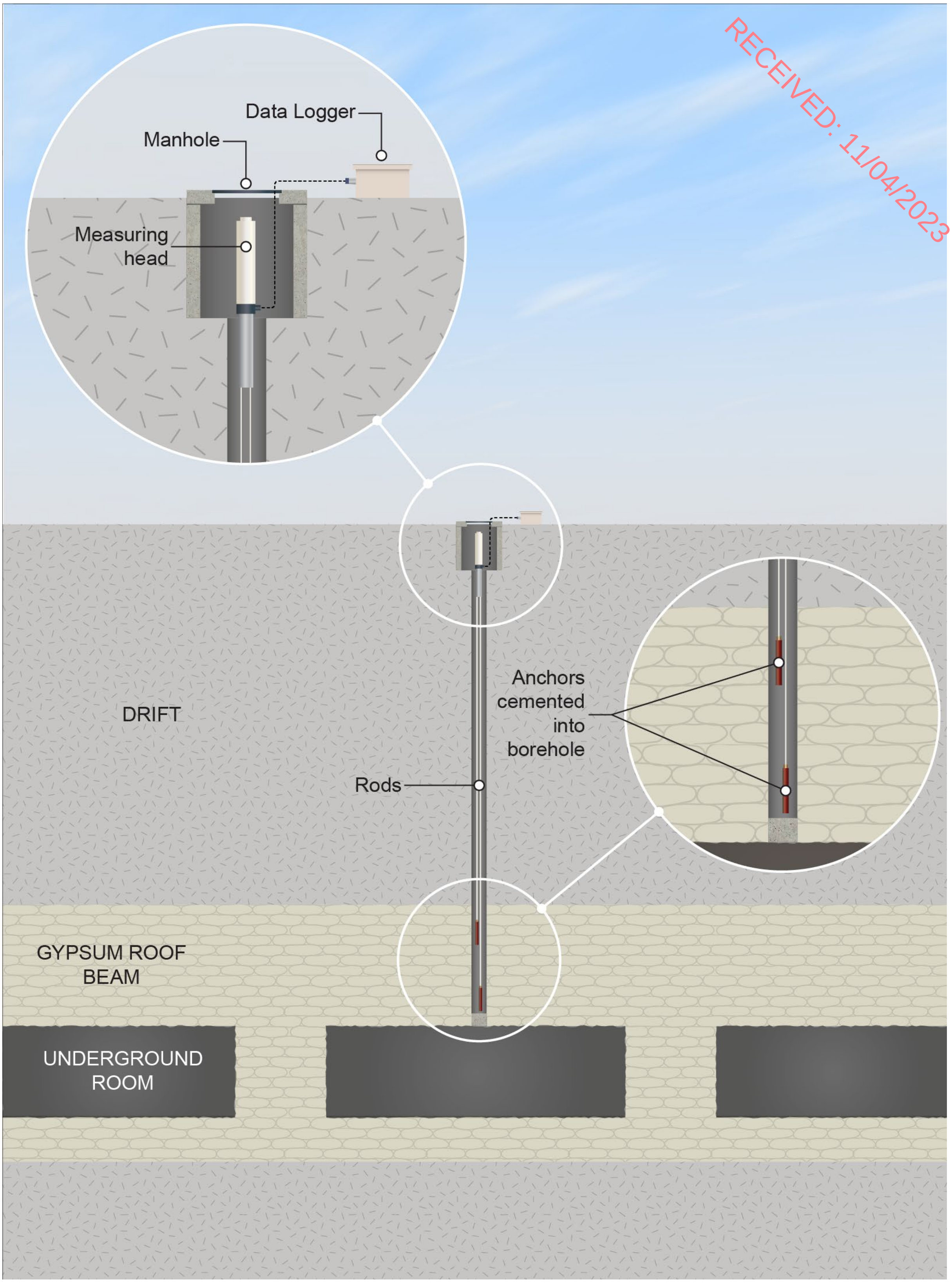

Figure 4-1: Schematic of the Two Anchor Borehole Extensometer Installation

Table 4-1: Monitoring System Details

	Anchor Head Position (m)			Roof Beam		Anchor Point Depth (m)		Closest Levelling Stations	New Station Required	New Levelling Station Co-Ordinates	
Borehole	x	y	z	Depth (m)	Thickness (m)	Upper	Lower			x	y
KC19-H1	280848.7	300599.99	1049.03	26.50	5.00	28.50	30.00	nA9	No		
KC19-H3	280822.5	300617.57	1048.92	23.05	7.05	25.00	28.50	nA11	No		
KC19-H4	280800.02	300638.97	1048.62	21.50	8.50	23.50	28.50		Yes	280799.711	300632.7616
KC19-H5	280779.8	300652.78	1048.69	21.40	8.30	23.50	28.00	nA12	No		
KC19-H10	280489.9	300888.19	1056.5	28.15	4.35	30.00	31.00		Yes	280492.3171	300881.4339
KC19-H11	280685.9	300724.09	1048.5	20.60	8.20	22.50	27.00		Yes	280681.9544	300721.3095
KC19-H15	280508.2	300880.26	1056.57	25.66	3.64	27.50	28.00	nA19	No		
KC19-H21	280750.9	300672.47	1076.76	20.87	11.30	26.00	33.50	nA13	No		

5 THE TARP

It should be noted here that a TARP is a live and evolving document. Whilst an initial set of triggers needs to be defined as a starting point, these are likely to change as monitoring data is collected and the stability of the underground workings becomes better understood. The TARP document, trigger action levels, and appropriate responses should be reviewed in conjunction with the response history at least once a year.

5.1 Extensometer Triggers

The extensometers have a maximum range of 50 mm. The expected failure mechanism of bedding and joint bounded blocks detaching themselves from the roof will be preceded by separation of the blocks along bedding planes. The extensometer movement range will be sufficient to identify when this may begin to happen. As there is no underground access, it becomes difficult to define appropriate movement trigger levels. Bigby, MacAndrew and Hurt (2010)[1] state that in active underground coal mines warning of roof instability is usually provided at roof deformations of between 10 mm and 25 mm depending on rock conditions, whist action is required at roof deformations of 25 mm to 50 mm. At Drumgoosat, there is no access to the underground workings, but as a starting point for the TARP, these ranges of deformation have been used to define a range of trigger actions as shown in Table 5-1.

5.2 Surface Levelling Station Triggers

The triggers for the surface levelling stations were defined using the levelling data collected from the recently installed stations and those installed and monitored since 1988. The stations that have been monitored since 1988 collected information related to the historical subsidence that occurred along the L4900 road in late 1999.

Figure 5-1 shows movement and rates of movement expressed in millimetres per year (mm/a) for two levelling stations, Station nA9 installed in September 2018 and Station A2 installed in 1988.

Station nA9 is close to extensometer KC19-H1. Whilst there has been gradual surface creep since the station was installed, the rate of movement has been constant at around 25 mm/a. Other recently installed stations in the vicinity show rates of movement of less than 25 mm/a.

Station A2 has monitored the ongoing historical subsidence. The graph in Figure 5-1 shows that as mine instability developed, there was a gradual increase in rate of movement from 25 mm/a to 75 mm/a. As the ground subsidence developed, the rate of movement in about 1997 increased to a maximum of about 135 mm/a. After this time, the rate of subsidence decreased gradually until by about 2002 the subsidence rate had reduced to the background rate of 25 mm/a, or less. The rate of movement values of 25 mm/a and 75 mm/a have been used as the initial trigger points for the TARP, which is shown in Table 5.1.

[1] *Bigby, D, MacAndrew, K and Hurt, K, Innovations in mine roadway stability monitoring using dual height and remote reading electronic telltales, in Aziz, N (ed), 10th Underground Coal Operators' Conference, University of Wollongong & the Australasian Institute of Mining and Metallurgy, 2010*

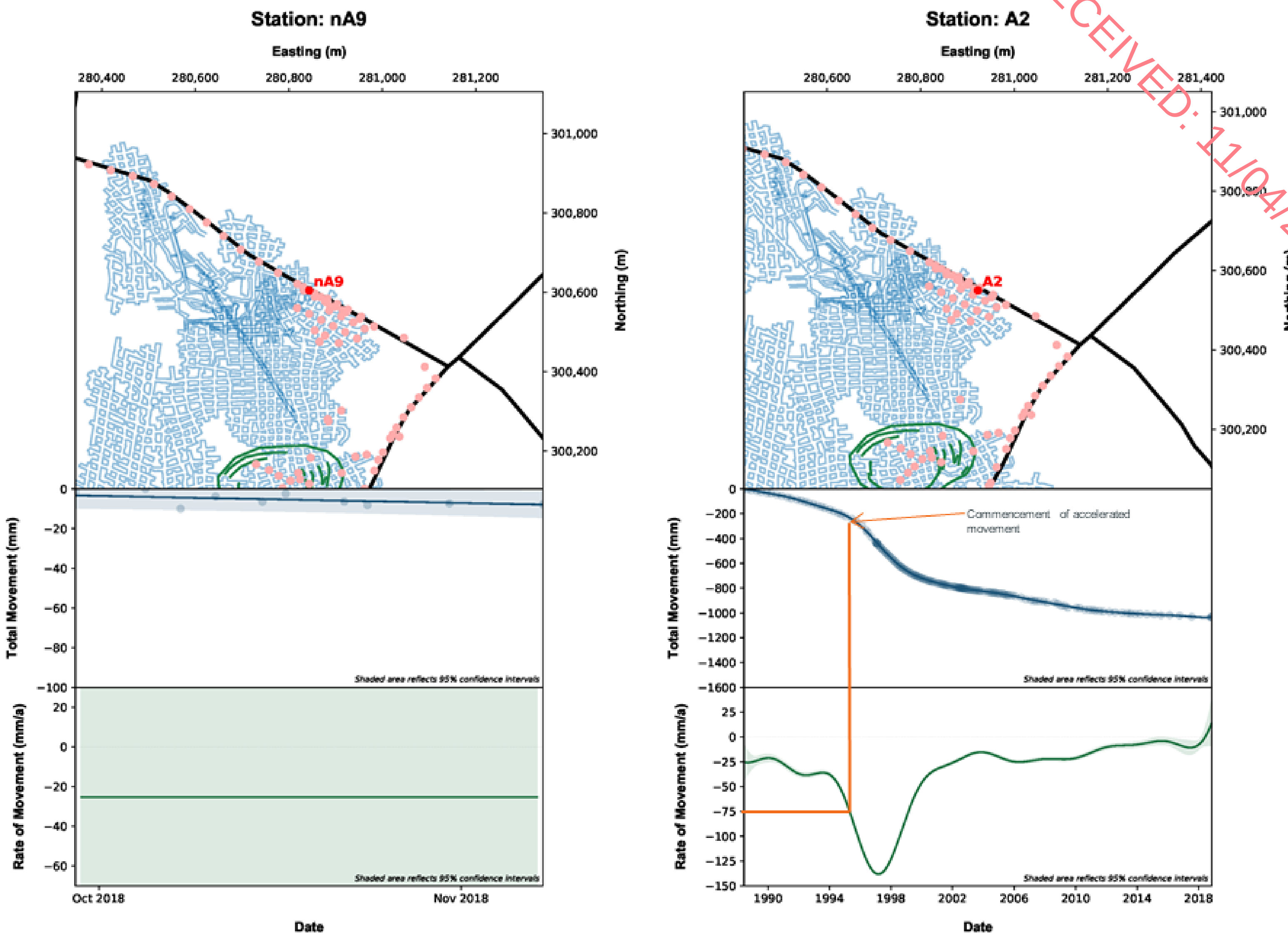

Figure 5-1: Surface Survey Monitoring Data

Table 5-1: Drumgoosat Mine – L4900 Road Stability Monitoring TARP

	Extremely Low Risk	Very Low Risk	Low Risk	Moderate Risk	High Risk
Surface Levelling Point	<25mm/a	<25mm/a	<25mm/a	25 - 75mm/a	75 - 135mm/a
Upper Roof Beam Anchor	<10mm	<10mm	10mm - 40mm	> 40mm	Extensometer Range Exceeded
Lower Roof Beam Anchor	<10mm	10mm - 40mm	10mm - 40mm	> 40mm	Extensometer Range Exceeded
Monitoring Frequency					
Surface Monitoring Points	Monthly	Monthly	Monthly	Fortnightly	Weekly
Extensometer Reading	Every 12 hours	Every 6 hours	Every 3 hours	Every 1 hour	
Reporting	Monthly	Fortnightly	Fortnightly	Weekly	NA
Status Change	Extremely Low to Very Low				
		Very Low to Low			
			Low To Moderate		
				Moderate to High	
Trigger Alarm	10mm (lower anchor)				
		10mm (lower and upper anchor)			
			40mm (lower anchor)		
				40mm (lower and upper anchor)	
Speed of Response	Next working day				
Monitoring Frequency	Reviewed and adjusted on the next working day				
Notifications	MCC and EMD advised that Risk Status has changed				
		MCC and EMD advised that Risk Status has changed			
			MCC and EMD advised that Risk Status has changed, further investigations launched		
				MCC and EMD advised that Risk Status has changed, further investigations launched	
Further Investigations	None	None	None	Carry out an underground laser scanning survey to confirm the integrity of the mine workings and roof beam	Consider road closure. Conduct more detailed sub-surface investigation of ground movement.
Interpretation	Normal conditions. Stable roof beam	Some deformation of roof beam immediately above room. Deformation has not propagated through the full roof beam thickness.	Deformation occurring through full roof beam thickness. Roof beam integrity has not been compromised.	Deformation occurring through full roof beam thickness. Roof beam integrity is beginning to be compromised resulting in increased surface movement.	Extensometer range has been exceeded indicating that roof beam might have failed. High surface deformation indicates that roof beam no longer provides support to the overlying material and that development of a subsidence feature on surface could occur.

5.3 Monitoring and Reporting Frequency

5.3.1 Extensometer Data Logger

The extensometer data logger will collect movement data from each anchor once every hour.

The data logger is programmed to calculate the average of the readings from each anchor over a defined period and report these. The time period will change depending on the Risk range in which the anchor movements lie, as shown in Table 5-1. These periods are:

- Every 12 hours when readings fall in the Extremely Low Risk range, increasing to:
- Every 6 hours when readings fall in the Very Low Risk Range, increasing to:
- Every 3 hours when readings fall in the Low Risk Range, increasing to:
- Every 1 hour when readings fall in the Moderate and High Risk Range.

The purpose of averaging the readings is to remove “noise” and unnecessary alarms from the system.

At each reporting frequency interval, the logger will record the arithmetic mean position of the extensometer compared to its datum since the previous report was made. A record will be generated of the mean deviation from datum of the extensometer over the previous time as defined above.

5.3.2 Data Management, Review, and Reporting

The extensometer data logger will store the data for a number of weeks. The Mine Surveyor will connect to the logger twice per week and make a copy of the logger information onto an office-based computer. At the monitoring frequency defined in the TARP and depending on the current risk category status, the Mine Surveyor will review the extensometer data and make a report to the Mine Manager which will be reviewed, any necessary actions identified, counter signed, and filed.

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5.4 TARP Review

It is recommended that six months after the monitoring system becomes live the trigger actions are reviewed and the TARP updated.

Future reviews and TARP updates should be scheduled at least annually.

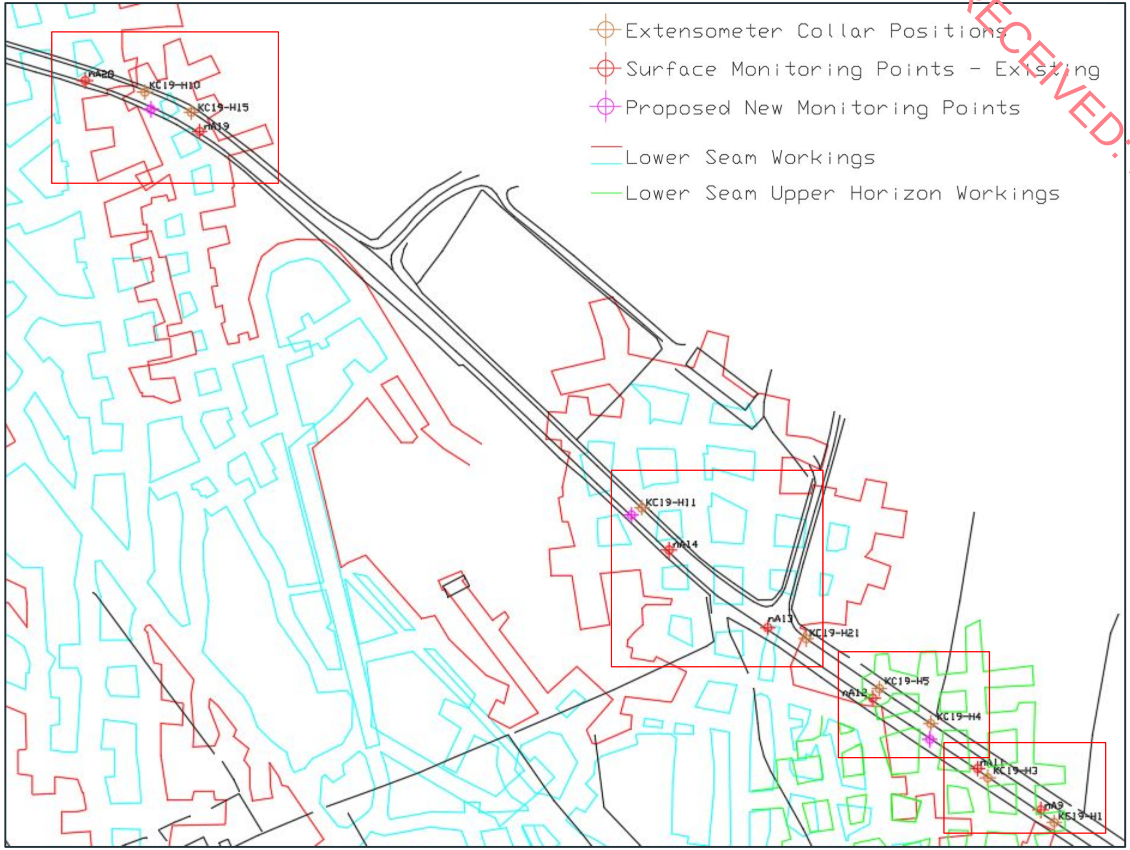
For and on behalf of SRK Consulting (UK) Limited

Neil Marshall, Corporate Consultant - Geotechnical, **Project Manager** SRK Consulting (UK) Limited	Richard Oldcorn, Managing Director, **Project Director** SRK Consulting (UK) Limited

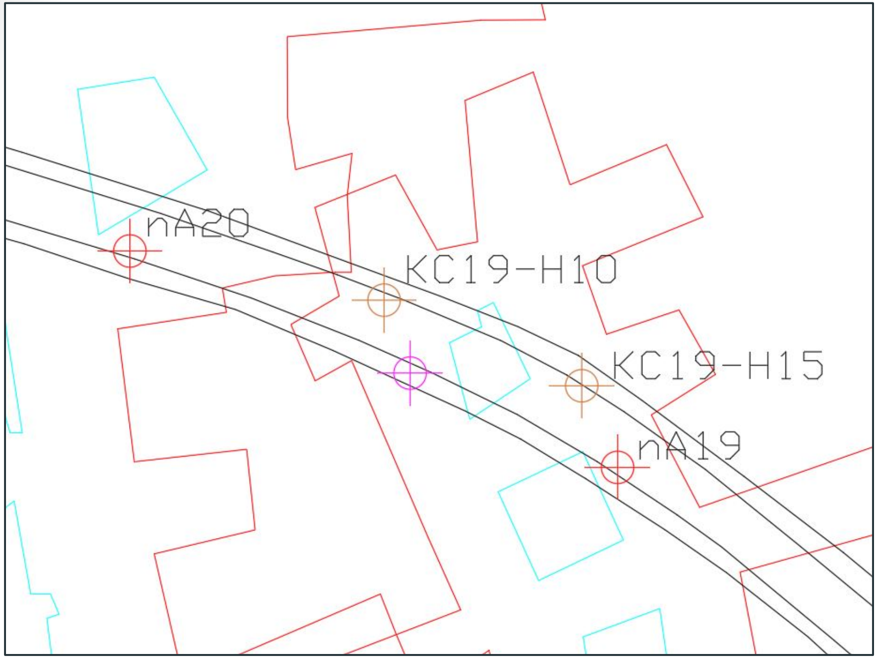
APPENDIX

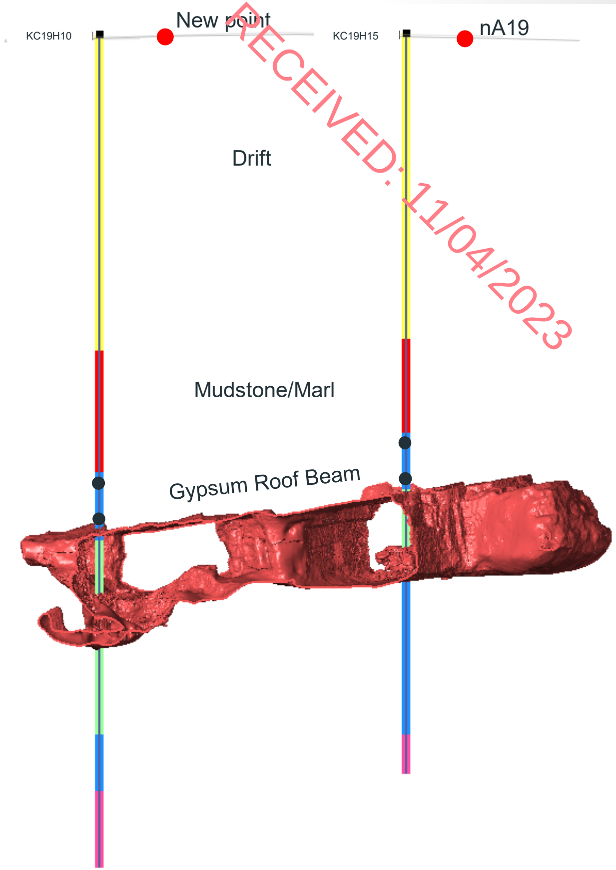
A MONITORING NETWORK DETAILS

Monitoring Location Plan



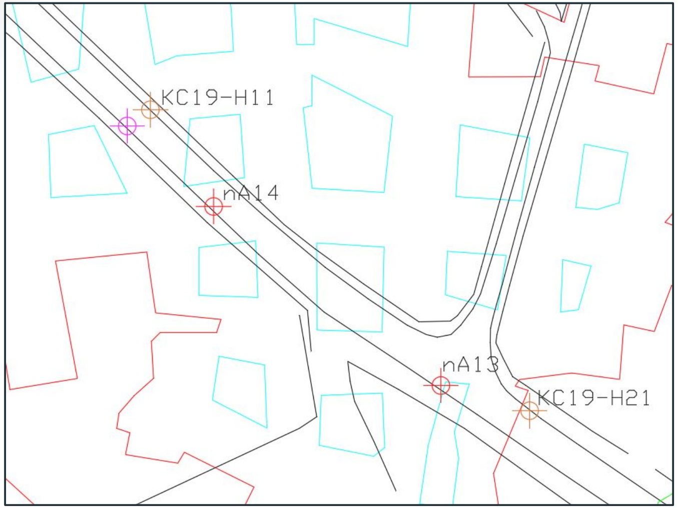
H10 – H15

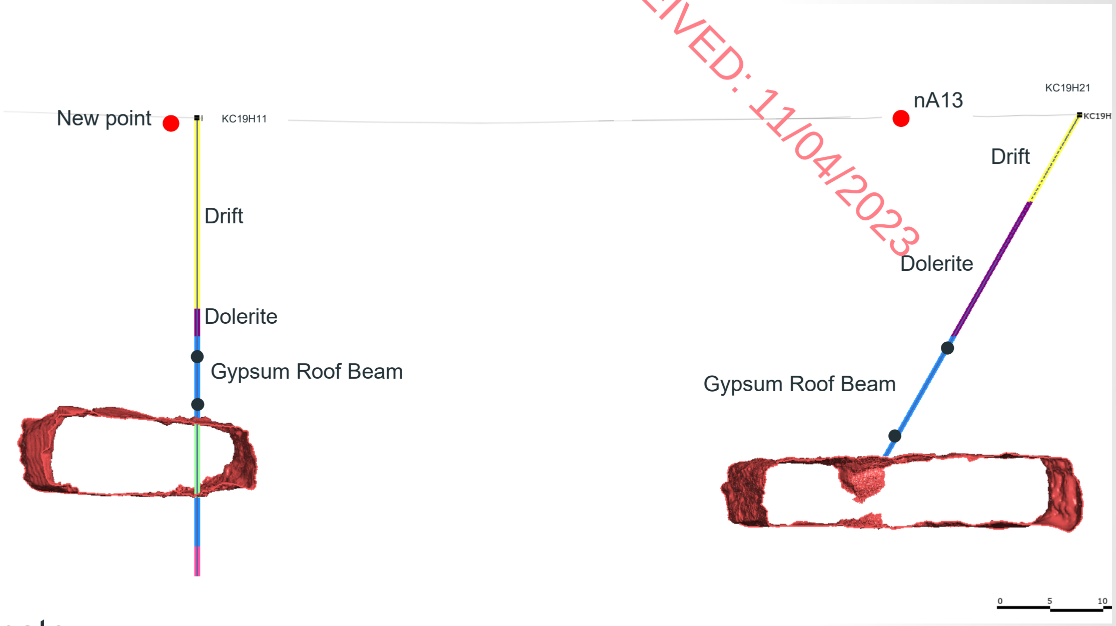




Black circles on boreholes indicate approximate position of extensometer anchor points

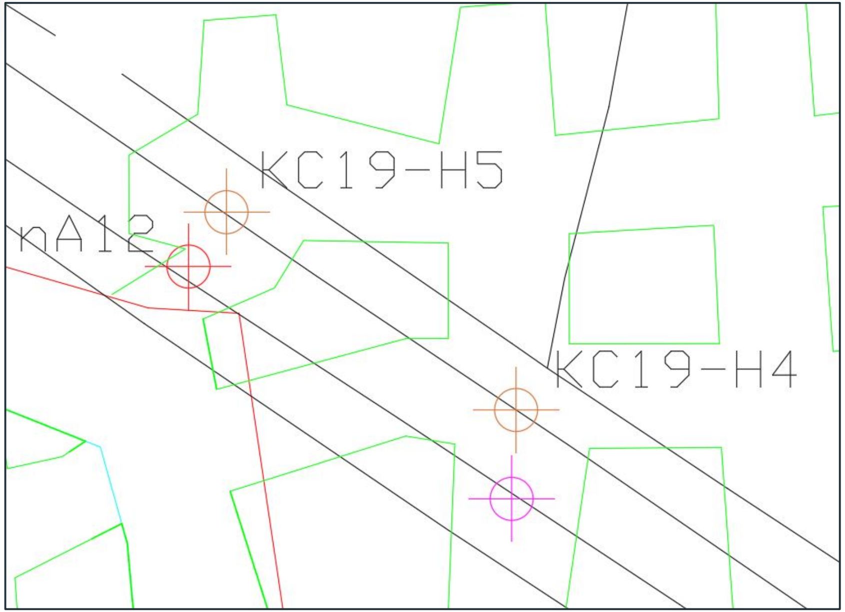
H11-H21

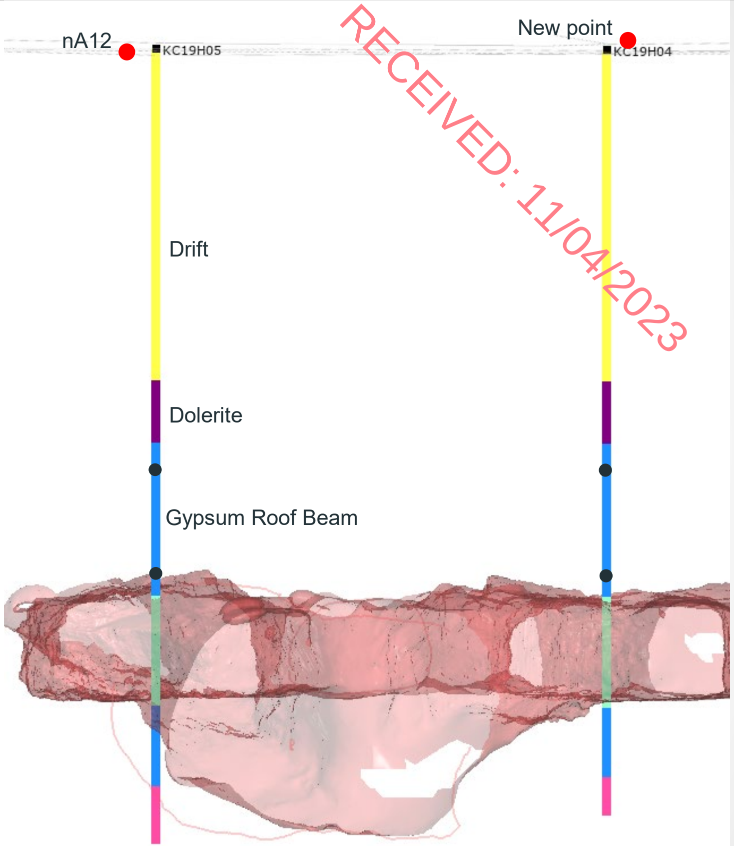




Black circles on boreholes indicate approximate position of extensometer anchor points

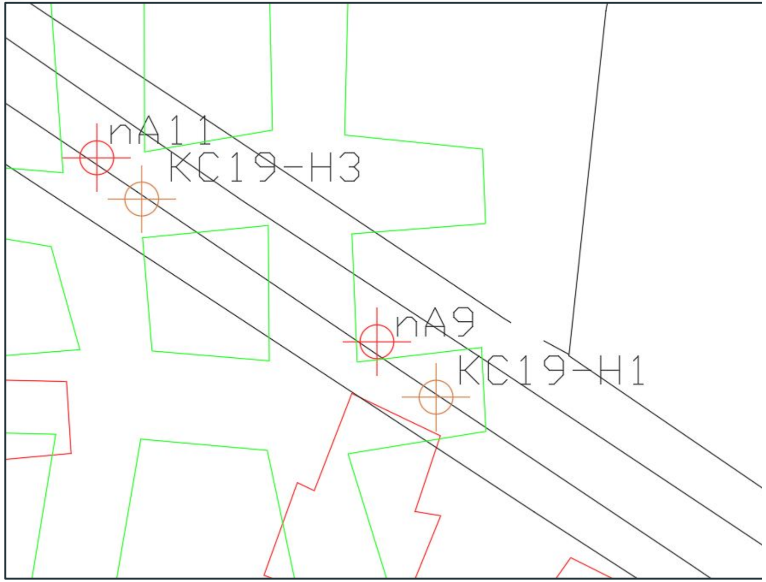
H4-H5





Black circles on boreholes indicate approximate position of extensometer anchor points

H1-H3



Black circles on boreholes indicate approximate position of extensometer anchor points

APPENDIX 7.10
Independent Review of the Stability Report on the Drumgoosat Underground Mine Workings below and adjacent to the R179 Carrickmacross to Kingscourt Road, Co. Monaghan - August 2021

wardell-armstrong.com

ENERGY AND CLIMATE CHANGE
ENVIRONMENT AND SUSTAINABILITY
INFRASTRUCTURE AND UTILITIES
LAND AND PROPERTY
MINING AND MINERAL PROCESSING
MINERAL ESTATES
WASTE RESOURCE MANAGEMENT

THE DEPARTMENT OF THE ENVIRONMENT, CLIMATE AND COMMUNICATIONS

INDEPENDENT REVIEW OF THE STABILITY REPORT ON THE DRUMGOOSAT UNDERGROUND MINE WORKINGS BELOW AND ADJACENT TO THE R179 CARRICKMACROSS TO KINGSCOURT ROAD, CO, MONAGHAN

ADDENDUM TO WARDELL ARMSTRONG INTERNATIONAL INTERIM REPORT (FEBRUARY 2021)

AUGUST 2021

wardell armstrong

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DATE ISSUED:	**AUGUST 2021**
JOB NUMBER:	**ZS611396**
VERSION:	**V6.0**
REPORT NUMBER:	**R001**
STATUS:	**FINAL**

THE DEPARTMENT OF THE ENVIRONMENT, CLIMATE AND COMMUNICATIONS

INDEPENDENT REVIEW OF THE STABILITY REPORT ON THE DRUMGOOSAT UNDERGROUND MINE WORKINGS BELOW AND ADJACENT TO THE R179 CARRICKMACROSS TO KINGSCOURT ROAD, CO, MONAGHAN

ADDENDUM TO WARDELL ARMSTRONG INTERNATIONAL INTERIM REPORT (FEBRUARY 2021)

AUGUST 2021

PREPARED AND APPROVED BY:

Robin Dean	Technical Director	

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FIGURES

APPENDIX

Appendix 1 Interim Report ZS611396-RPT-V3.0, dated February 2021

DRAWINGS

ZS611396-001-A Site Layout Plan
ZS611396-002-A Assessment Zones and Cross Section Locations
ZS611396-003-A Underground Workings Extent Assessment

EXECUTIVE SUMMARY

Wardell Armstrong International (WAI) has carried out an independent review of works undertaken by SRK Consulting (UK) Ltd (on commission from Gyproc Ltd) to investigate the stability of the Drumgoosat mine workings beneath the R179 Carrickmacross to Kingscourt Road in Co. Monaghan. WAI produced a draft interim report in June 2020 of that review. The draft report was used by the regulatory authorities to frame a number of additional queries that related to the stability of the R179. A total of 29 follow-up questions were referred to Gyproc for more detailed responses. WAI subsequently issued a Final Interim report in Feb 2021 whilst work continued on this Addendum addressing queries not fully answered within the SRK report.

Further works were carried out by Gyproc and SRK and finalised responses to the 29 questions were received in December 2020. WAI carried out a review of these responses to ensure all questions had been fully answered as far as could be reasonably expected, and that such answers were acceptable. WAI's considerations on the SRK/Gyproc follow-up responses are presented as this separate addendum to the interim report.

The WAI interim report identified that the investigation and study work related to 'Zone 1' did not give a full picture of the complicated geological structure that affects the area. Many of the regulatory authorities follow up questions related to obtaining a more comprehensive picture of this area to substantiate the overall conclusion reached by SRK-Gyproc that the R179 continues to be safe to use.

The Gyproc-SRK follow up responses have provided documents "R179 Zone 1 - Additional Assessment(V2)" and "R179 Zone 1 - Additional Assessment(V2)- Amended Dec 2020" and the plan "DCCAE Plan Final" which provide a much better overview of the wider Zone 1 area. The referenced documents are a more complete assessment and cover the broader aspects questioned within Zone 1 in relation to geological anomalies, Upper and Lower Seam gypsum thicknesses, historic crownholes and localised doming in the roof of some mine workings.

In addition, Zone 4 was an area where flooding of the workings created areas of high pressure air pockets that initially precluded further investigation of the mine voids by laser scanning. Accordingly, verification of the SRK conclusions reached by FLAC modelling was not initially possible. Pumping from Drumgoosat continued and the water level was reduced allowing the laser scanning to eventually take place in May 2021. Further works were also undertaken by Gyproc to clarify the mining plans in the area, which appeared to show overlapping workings. Gyproc has produced an interpretation of the workings which is accepted by WAI. It is a logical interpretation of the way in which the mine was worked in the Upper Horizon of the Lower Seam which was then undermined by workings in the Lower Horizon.

WAI have reviewed the laser scanning results of the area which does not show any cause for concern, the pillars appearing to be in a stable condition.

WAI has concluded that Gyproc and SRK in their original report, their answers to the 29 follow-up questions, and the additional work associated with the double level workings and airlocked areas in

Zone 4 have addressed all issues in a full and acceptable manner and that the answers provided close out the issues raised by the original report as far as is practically possible.

As a result, WAI agrees with the conclusions reached by Gyproc and SRK that the R179 continues to be safe to use. However, WAI consider it prudent for a comprehensive monitoring programme (and associated Trigger Action Response Plan, TARP) to remain in place for the R179 that provides for early warning of any potential underground instability. An acceptable monitoring scheme has been implemented by Gyproc since late 2018.

1 ADDENDUM TO THE INTERIM REPORT

1.1 Introduction

Following the issue of the draft WAI Interim report ('Independent Review of the Stability Report on the Drumgoosat Underground Mine Workings Below and Adjacent to the R179 Carrickmacross to Kingscourt Road, Co. Monaghan') in June 2020, a number of virtual meetings were held in July 2020 between Monaghan County Council and its advisors, the Environmental Protection Agency (EPA), the Department of the Environment, Climate and Communications (DECC) and Wardell Armstrong International Ltd (WAI). The regulatory authorities identified a number of queries relating to the stability of the R179 on which they sought additional information. At the end of July 2020, a list of 29 follow-up questions were forwarded on to Gyproc for a response. Gyproc issued responses to some of these questions within a few weeks and undertook additional works to address the remaining queries. A final response to all queries was issued by Gyproc on 2 October 2020. A final Interim Report was issued by WAI in February 2021, with any queries not fully dealt with within the report put into this separate Addendum.

This addendum to the Interim Report incorporates the questions raised by the regulatory authorities and their advisors, the interim and final responses to the questions provided by Gyproc and WAI's comments on the responses.

Based on all of the information submitted by Gyproc/SRK, this addendum additionally includes final conclusions reached by WAI in relation to the overall stability of the R179.

1.2 Questions and Responses Related to Interim Report

1) Ref: KC- 20 R01, SRK are requested to validate the extent of roof degradation at this location.

Interim Answer: Gyproc propose that a review of all data gathered related to the area around Hole R01, the 97A crownhole and the Roof Domes previously reported is carried out and submitted to the authorities as part of the final response planned to be issued in early September. This will address subjects raised in Q1, Q5, Q6, Q10 and Q12.

Final Answer: A report that summarises work that has been carried out between 1997 and the present whose purpose has been to investigate and analyse specific features that have occurred close to the R179 and general mine stability in the area of Zone 1 has been completed and is submitted as an attachment to these answers. Attachment Ref "R179 Zone 1 - Additional Assessment(V2).pdf"

WAI comment:

The referenced document is a more complete assessment of Zone 1 which covers the requested information and indeed covers the broader aspects questioned within the zone to do with geological anomalies , Upper and Lower Seam gypsum thicknesses, historic crownholes and localised roof doming.

A series of LSS plans and a PDF composite plan of the area has been subsequently submitted which shows all related information on one plan and which enables the stability of the Zone to be better evaluated.

WAI note that there is a geological feature running NNW/SSE to the NE of KC19H23 which if continued would intersect with the feature running WSW/ENE. The latter feature is spatially associated with a historic crownhole occurrence (*DT97a). The intersection point is under the R179 and whilst there are no mine workings in that area, there is a risk that a water conduit through the strata could occur at this point and a sink hole could potentially develop. This area must be monitored to ensure that no water inflows from the surface are occurring. This should be done at the same time as the subsidence monitoring, which is already being undertaken in the area.*

WAI concur with the SRK conclusion that the R179 is currently stable in Zone 1, but the agreed monitoring must be continued to verify that that status is maintained.

2) Mudstone modelling mechanisms need to be clarified to determine their impact and importance on crown hole formation.

Interim and Final Answer: Roof beam failure is a precondition to mining induced crownholes. No protection from crownhole formation is attributed to mudstone in the modelling.

WAI Comment
There is evidence within Zone 1 that whilst there is an acceptable thickness of Lower Gypsum seam roof, the gypsum may be inferior due to mudstone partings within the gypsum seam. It is not apparent that this has been taken into account, and in particular there are areas where geological anomalies are present and this combined with the mudstone within the Gypsum roof beam could result in some locations being more vulnerable than others to sinkhole or crownhole formation e.g. where there is no Upper Gypsum. However, the presence of the mudstone partings are offset by the considerable thickness of the Lower Gypsum roof and WA does not consider therefore that this compromises the stability of the R179. The monitoring of the area must be continued to verify that that status is maintained.

3) Pillar 12: A comparison of previous scans at the various locations taken at intervals should be examined to see if any changes or deterioration has occurred.

Interim and Final Answer: A comparison of both laser surveys completed for this area (December 2018 and August 2019) was carried out by SRK at the request of Gyproc in November 2019 in the form of a 3D cloud comparison pdf document. This pdf document is attached to this response and the narrative provided on the pdf is repeated below.

'In (the 3D pdf document) both the December 2018 and August 2019 point clouds have been combined in the software 'Cloud Compare' to create a single point cloud. Red points represent those where there is no difference between the two point clouds. Yellow points indicate some difference between the two point clouds.

If you spin the model around you can see some yellow points in the rooms below where the laser survey tool entered the underground workings, which is an artefact of the presence of the survey tool. There are also some yellow points towards the limits of the scans where the point cloud is less dense.

In the area of Pillar 12 where the point cloud is densest, all of the points are red indicating that there has been no change in the pillar or roof shape between the two laser surveys.'

The monitoring schedule that Gyproc have in place sees Pillar 12 being laser scanned at a nominal interval of 12 months until 2023 when the frequency will be reviewed. The condition of the pillar will in future be reported on in the routine R179 stability reports that are issued to the authorities. These reports are currently issued on a three monthly basis. Attachment Ref "3D_PDF_CloudComparison_RedYellow.pdf"

WAI Comment
WAI accept the comments and concur with the comment regarding continued monitoring.

4) Boreholes should be kept free to continue laser scanning. 3D scanning would give reassurance. What timeframe would this take to set up correctly? Please provide mapping of all boreholes and scans to date.

Interim and Final Answer: All recommendations with respect to ongoing laser scanning that were proposed in the various technical reports that have been published by Gyproc and EMD since October 2018, when laser scanning was first deployed, have been implemented as part of a routine laser scanning program by Gyproc.

The scanning locations are described in the following reports which have been previously issued to the Authorities.

- SRK – Oct 2018 – Drumgoosat Subsidence Event Technical Report – reviewed by WA/EMD
- SRK - April 2019 – December 2018 Crownhole – reviewed by WA/EMD
- SRK – May 2019 – External Memorandum – Review of mine conditions in the Vicinity of the 1997 Crownhole Event – reviewed by WA/EMD.
- SRK – Aug 2019 – Drumgoosat L4900 Trigger Action and Response Plan
- SRK – April 2020 – Investigation and Analysis of Mine Stability below the R179 Road – Review by WA / EMD in progress.

The scanning program currently being implemented by Gyproc, goes beyond the recommendations of these reports, is described in the attached document. Attachment Ref: "L4900 R179 Borehole Extensometer and Scanning Status 21.08.2020"

WAI Comment
Monitoring programme is acceptable.

5) **Zone 1, Location Hole R1. A geological anomaly running approx. N/S can be identified and appears to cross the L4900. Some evaluation of this anomaly using the monitoring along the L4900 and R179 may be useful. This monitoring could be an extensometer in the upper seam which would flag any movements in the lower seam.**

Interim and Final Answer: The stability of the L4900 was studied in detail and a program of ongoing assurance by means of instrument and surface monitoring was recommended and put in place. This was fully documented in the SRK report investigating the December 2018 Crownhole that was published in April 2019 and which was reviewed by EMD and WA.

Gyproc propose that a review of all data related to the area around Hole R01, the 97A Crownhole and the Roof Domes previously reported on in this area is carried out and submitted to the authorities as part of the final response planned to be issued in early September. This will address subjects raised in Q1, Q5, Q6, Q10 and Q12.
Should the Authorities wish to propose a specific location for the installation of an extensometer Gyproc will consider positively such a proposal with its advisors.

WAI Comment
WAI had previously considered that the roof feature at R01 may have been a result of the geological anomaly mentioned in the question. WAI had therefore recommended consideration of an extensometer in the strata above the 'roof feature' identified at R01. However, the explanation of the formation of that feature is in line with it being caused by the drilling of the borehole itself.

In addition, the 'DCCAE Plan' indicates that the thickness of Upper Gypsum over that feature is above 5m which would preclude the formation of a crown hole in that area. Therefore, WAI are satisfied that an additional extensometer is not required. Also at See Comment on Q1.

6) **It is not mentioned in the report but within Zone 1 there was a crown hole developed 97a. Given the proximity to the R179 and the occurrence of a chimney on the other side of the R179 some comment is required on the area 25m either side of the road.**

Interim and Final Answer: Previously Gyproc submitted a report on this specific area to the authorities, "SRK – May 2019 – External Memorandum – Review of mine conditions in the Vicinity of the 1997 Crownhole Event" which was reviewed by WA/EMD. This report found the area to be stable. This report is attached to this reply.

Gyproc propose that a review of all data related to the area around Hole R01, the 97A Crownhole and the Roof Domes previously reported on in is carried out and submitted to the authorities as part of the final response planned to be issued in early September. This will address subjects raised in Q1, Q5, Q6, Q10 and Q12. Attachment Ref: "30238_1997 Crownhole Area Assessment(V2)"

WAI Comment
See Comment on Q1

7) More information is required on the numerical model adopted. For example:

Interim and Final Answer: The R179 Investigation Report was deliberately scoped to be understandable by an informed member of the public and not just by a technical specialist. This is the approach that has been followed in all the reports published to date. This has meant that overly technical language in the descriptions of standard assessment approaches was avoided where it didn't add value. While the technical language is simplified, in all cases appropriate technical assessment standards have been applied. Gyproc have received the following advice with respect to the specific questions raised on numerical modelling from SRK.

WAI Comment
Accepted

7a. Should SRK not be reporting the rock classification as modified RMR89'?

Interim and Final Answer: SRK is not familiar with the term RMR89' but understand it to be the RMR89 rating with no joint orientation adjustment and ground water condition rating set to dry. This is the default RMR rating used to convert to GSI for estimating rock mass strength. In all of its studies and report for Drumgoosat SRK has only used RMR to calculate GSI. We clearly state in our report that RMR with dry groundwater conditions is used to calculate GSI. The report has been prepared so that it can be read and understood by non-technical readers. Where not relevant to the analysis, which is the case for joint orientation adjustments, for the purpose of clarity SRK has not discussed RMR joint orientation adjustments.

WAI Comment
Accepted

7b. How was UCS corrected?

Interim and Final Answer: The UCS correction was made by the testing laboratory. The lab report states 'UCS Values have been corrected for the tested height of the core samples (nominally 2:1 height to diameter)'. No further details were provided by the lab but the UCS corrections reported are minimal.

WAI Comment
Accepted

7c. Were all the samples submitted for strength testing, lower seam gypsum?

Interim and Final Answer: All samples submitted for strength testing were collected from the gypsum forming the roof beam of the excavations intersected by the boreholes.
WAI Comment
Accepted

7d. Why were no point loading tests undertaken?

Interim and Final Answer: A good understanding of the variability of the gypsum strength was developed from the point load strength testing of the L4900 boreholes. Point load testing of this round of borehole drilling was not considered necessary.

WAI Comment
Accepted

7e. Can SRK justify the parameters used in their numerical modelling with a better correlation to reflect the GSI changes?

Interim and Final Answer: These parameters are in good agreement with the parameters used in previous analyses.

WAI Comment
Accepted

7f. Has the model assessed effective, effective with pore water pressure variation, or total stress properties of the individual materials?

Interim and Final Answer: Where a groundwater surface is included in the models increase in pore water pressure and its impact on total stress is automatically accounted for.

WAI Comment
Accepted

7g Can they provide discussion on how they believe the material is behaving: i.e. linear-elastic, anisotropic elastic, or elastic-plastic or indeed the time setting of the predicted movements.

Interim and Final Answer: The material behaviour used in the modelling was one of an elastic, perfectly plastic material. The modelling is time independent.

WAI Comment
Accepted

8) SRK are requested to link the stability report to what is being monitored on the road level monitoring. Is the displacement what would be expected? The modelling shows what should have happened immediately after the workings. If movement is continuing why is this occurring now and not 40 years ago?

Interim Answer: A detailed response on FE modelling will be submitted to the authorities as part of the final response planned to be issued in early September.

Final Answer: In general, the Finite Element (FE) analysis can only model the instantaneous elasto-plastic response of the system to changes made to that system. Such system changes could, for example, involve things such as the removal of mining excavations (the creation of voids), a reduction in pillar sizes or some specified reduction in material strength of pillars or footings, etc. The FE analysis will compute the instantaneous response to any such changes made to the model: the calculations do not include the kind of time-dependent response that is being monitored along the roadway. To model such time dependent behaviour explicitly would require the use of a creep constitutive model in the analysis. However, such an approach is not recommended as calibrating a creep model to current measurements and then undertaking creep simulations would simply produce the results that the model has been "programmed" to produce and would provide no new, useful, information.

As previously stated, the FE analysis that was undertaken produced results for surface deformations that are consistent with those measured by Gyproc. This would indicate that the model is a reasonable representation of reality in this instance and the model indicates that the system of mine excavations and pillars beneath the road is stable in its current configuration. This is probably as much as can reasonably be expected from FE modelling at this time. However, further FE modelling could play a useful role if something unusual is observed in the ongoing monitoring data or something else changes.

WAI Comment
Accepted, WAI agree FE modelling has reached its limit of usefulness in this instance unless conditions change within the mine.

9) The origin of the scan data needs to be clarified- was this laser or sonar scanning?
Interim and Final Answer: The attached schedule identifies which locations were laser scanned and which locations were sonar scanned. Attachment Ref "L4900 R179 Borehole Extensometer and Scanning Status 24.08.2020"

WAI Comment
Accepted. Scan data clarified in document.

10) Clarification is required on how SRK interpreted the drop out of the cavity of the chimney in Zone 1.

Interim Answer*:* SRK have advised that by rotating the 3D image of the laser survey, the full extent of the size and shape of the roof hole was inspected. It is similar in shape and size to other features that SRK inspected in its historical underground visits.

Gyproc propose that a review of all data related to the area around Hole R01, the 97A Crownhole and the Roof Domes previously reported on in this area is carried out and submitted to the authorities as part of the final response planned to be issued in early September. This will address subjects raised in Q1, Q5, Q6, Q10 and Q12.

Final Answer: A report that summarises work that has been carried out between 1997 and the present whose purpose has been to investigate and analyse specific features that have occurred close to the R179 and general mine stability in the area of Zone 1 has been completed and is submitted as an attachment to these answers. Attachment Ref "R179 Zone 1 - Additional Assessment(V2).pdf"

WAI Comment

See Comment on Q1

11) Zone 3: Explanation required re the blind spots (blue visible where no data available in roof; see inset of figure 6.3) at these locations and what is happening at this location considering the volume of material that is lying on the ground.

Interim and Final Answer: The laser scan is line of sight. The blind spots in the laser scan referred to are just that, areas of the roof and sidewalls of the rooms that the laser scanner could not see.

The computer software (called Leapfrog) used by SRK for visualisation of 3D objects colours by default the outside of a 3D object red and the inside of the object, blue. The red area of the laser scan shows the roof of the underground tunnels. The blue area shows the floor of the tunnels in areas where the laser scanner has not been able to survey the roof. The blue area does not indicate the presence of water in the tunnels.

WAI Comment

The report reference '30787 R10 and R11 laser survey assessment (V2)' provides sufficient explanatory details and that explanation is accepted by WA.

12) Previous work by Gyproc has identified a number of criteria that are important in the formation of crown holes. However, the mechanism for the formation of 'chimneys' in the roof of workings, which on rare occasions may propagate to the surface as crown holes (e.g. 97a), remains poorly understood.

The formational mechanism for chimneys requires further evaluation, as well as the factors that enable chimneys to reach the surface.

Final Answer: Sinkhole and crownhole formation are quite common above abandoned shallow underground mine workings particularly above evaporite and coal mines. The occurrence and impact of legacy mining features such as these is not uncommon in the UK.

In the 2019-20 reporting period alone, The Coal Authority (https://www.gov.uk/government/organisations/the-coal-authority) assessed 352 subsidence damage claims and delivered over a quarter of a million mining reports, many of which were related to legacy mining issues. Network Rail employs a permanent group of mining specialists (around 5 to 10 people) who spend a significant amount of their time dealing with the impact of legacy underground mining on Network Rail track and infrastructure. They have over 5000 known shallow mining hazards near the railway on record (https://www.networkrail.co.uk/wp-content/uploads/2019/06/Mining-Mining-Ground-Investigations.pdf). Monitoring and site investigation are a key component of the work of both The Coal Authority and Network Rail in meeting this challenge.

Mainland UK is particularly influenced by coal mining in the Midlands and the north and by tin mining in the south-west. Underground legacy mining hazards in the UK date back to pre-Roman times (e.g. flint bell pits in chalk) but became increasingly more numerous from the date of the industrial revolution onwards.

Legacy underground mining and the impact of surface damage is a challenge that faces all countries in the world where there is a history of mining. There is a large body of literature relating to the issue of legacy underground mining, along with the development of techniques to predict and manage the impact of surface subsidence specifically in the UK, the USA and South Africa (see for example, http://www.scielo.org.za/scielo.php?script=sci_arttext&pid=S2225-62532018000700014&lng=en&nrm=iso&tlng=en) along with many other countries that have a history of underground mineral exploitation.

12a. Is it possible for example that the concentration of chimney features south of Rafferty's is linked to an area of gypsum dissolution? Drill hole logs of several holes in this region show evidence of dissolution of gypsum and infilling of the cavity by clays/muds. Could NNW-oriented structures control the spatial extent of these dissolution zones?

12b.Do similar geological conditions occur elsewhere at Drumgoosatt that could give rise to instability?

Interim Answer: Gyproc propose that a review of all data related to the area around Hole R01, the 97A Crownhole and the Roof Domes previously reported on in this area is carried out and submitted to the authorities as part of the final response planned to be issued in early September. This will address subjects raised in Q1, Q5, Q6, Q10 and Q12.

Final Answer: Zone 1 is the only area beneath the public roads where this geological condition has been observed. A report that summarises work that has been carried out between 1997 and the present whose purpose has been to investigate and analyse specific features that have occurred close to the R179 and general mine stability in the area of Zone 1 has been completed and is submitted as an attachment to these answers. This report discussed dissolution features, chimneys, domes, orientation of structures.

Attachment Ref "R179 Zone 1 - Additional Assessment(V2).pdf"

WAI Comment
See Comment on Q1

13) The occurrence of the double layer of workings beneath the R179 should be referenced in the report.

Interim and Final Answers: Previous correspondence on the possibility of double horizon workings was issued to the Authorities on 6th February 2020. A copy of this is appended. We request advice on any specific location that the authorities would like a more detailed commentary on. Attachment Ref: "Airlock Investigation 06_02_2020"

Gyproc and SRK subsequently carried out further works and issued additional reports related to this area in Q2 2021 as follows:

- Interpretation of Workings in the "airlock area" Drumgoosat Mine, Revised 26.7.21 (including report "Update to Authorities on R179 Drilling program – 6.2.2020).
- SRK R179 Investigation – Zone 4 Addendum Report, July 2021.
- Follow up email related to clarification of the pillar height adjacent to the declines 04.8.21

WAI Comment

The SRK report was issued without reference to the double layer workings that are evident in the Lower Seam and identified in Zone 4. A re-evaluation of the mining plans was undertaken by Gyproc in early 2021 in that area and an interpretation of the mining plans supplied to WAI in July 2021. Areas of working in the Upper horizon of the Lower Seam appear to overlap workings in the Lower horizon, the upper and lower horizons connected by a decline and an air raise. The decline resulted in a higher pillar side where the decline went into the floor of the Upper horizon and into the roof of the Lower horizon but there were no complete pillars that could be considered double height. The ventilation raise connects the Upper and Lower Horizons, but is not connected to the surface, although it is located under the edge of the R179, scanning appears to show the area is stable. Whilst double layer workings are present there is very little within the accepted zone of influence of the R179. In addition there is an Upper Gypsum across the whole area to a considerable thickness, at least 10m, above the double working and ventilation raise area. Based on the reports and information provided WAI consider that in this area the pillars are stable and risk to the R179 is extremely low

14) The Authorities request a timeframe and plan for when the boreholes that intersected air pockets will be laser scanned.

Interim and Final Answer: It is forecasted that water levels will recede to a level that will allow scanning by March 2021. The exact timing is dependent on a number of factors not all of which are within the company's control. A detailed plan for the completion of the work will be made closer to the time.

WAI comment

The area was laser scanned in Q2 2021 See comments in Section 1.3

15) The laser scan for drill hole R15 (located in Zone 1) is outstanding and should be submitted as soon as possible.

Interim and Final Answer: Hole R15 was not drilled as part of the R179 investigation study proposed to the authorities. Hole R15 was added to the drilling program to confirm the layout of the workings at this point for reasons not related to the R179 study. The borehole laser scan confirmed the layout of workings to be as assumed and the geology revealed by the borehole log did not reveal any concerning geology. The layout of the workings is included in the relevant section of Appendix D of the report. The laser scan data file for hole R15 will be submitted to EMD and MCC on 25th August 2020 and the borehole log is appended.
Attachment Ref: "KC20 R15 Report"

WAI Comment
The laser scan showed no evidence of instability although the change in the roof level is noted.

16) An explanation of Figure 4.8 is required and an interpretation of the large volume of material on the ground is sought.

Interim Answer: Appendix D of the report provides more detailed explanations of each of the cloud scans. SRK will prepare a more detailed response combining some of the commentary from the appendix, this will be included in the final response, planned to be submitted in early September.

Final Answer: Figure 4-8 is laser scan image taken from borehole KC20-R01. This image is replicated in Slide 7 of Appendix D. Slide 6 is a similar image taken for a slightly different location. The information in Appendix D describes the floor debris as originating from the hole in the roof.

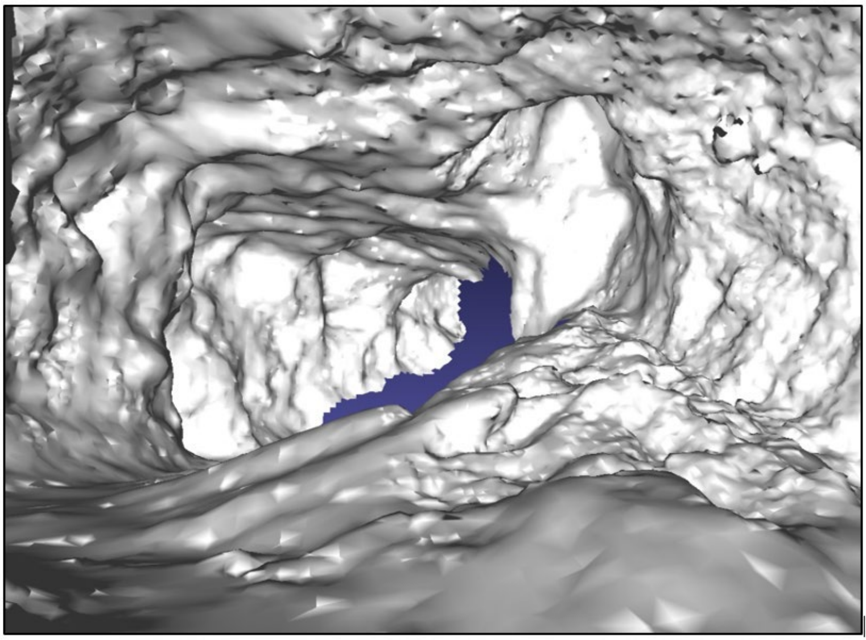


Figure 4-8: Laser scan image from KC20-R01 looking south west

Figure 1: (Figure 4-8) Laser Scan Image

WAI comment
WAI consider that there appears to be more material on the floor than just from roof void (even considering bulking). However, the shape of the pile could indicate that it may additionally contain

mining waste that was left on the floor of the mine and which was not mucked out by the operator. WA also note that the thickness of Upper Gypsum above this point is above 5m indicating that any feature in the Lower Seam should not surface.

17) Zone 3 - Request the Consultants view on the increased prevalence of slabbing in this area.

Interim Answer: This is under consideration and will be responded to in the final response, planned to be submitted in early September.

Final Answer: *An old geological plan shows the workings to have a 'D' section roof in this area. The laser scans images are typical of the 'D' horizon. The laser scans showing the beds dipping to the north west indicate that as mining continued in these headings to the south east, the seam rolls upwards which brings the mine roof into 'B' horizon. This can also be seen on the geological plans. The interpretation is that the workings encountered a downwards roll, bringing 'D' horizon into the roof, at which point working stopped.*

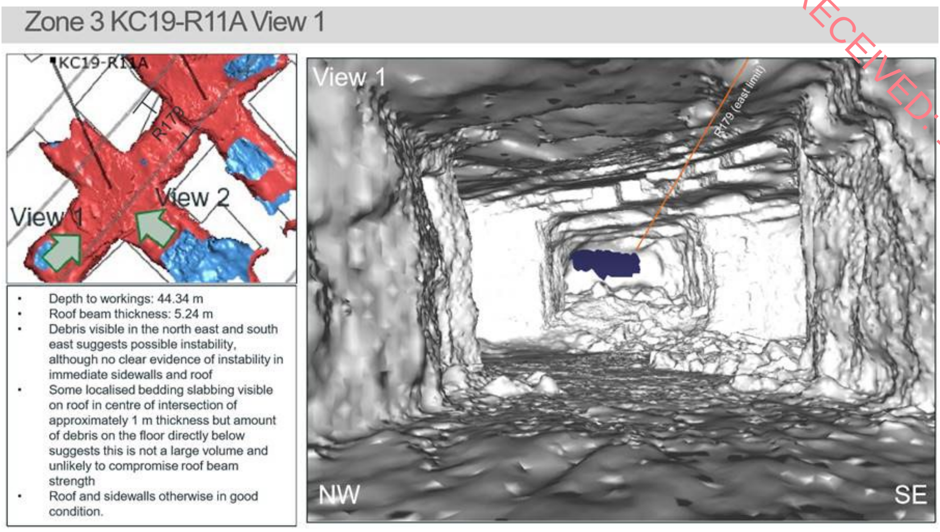


Figure 2: Geological Plan and Laser Scan

The seam structure was changing over a relatively short distance and the miners would need to change mining horizon to control quality and maintain a safe roof. The steps in the roof are evidence of this.

WAI Comment
Accepted

18) Clarification to be inserted into the report that it was commissioned by Gyproc at the request of the various authority agencies - MCC, DCCAE & EPA.

Interim and Final Answer: The report has been finalised and as such it is not possible to be re-issued. The study of the R179 and subsequent report was scoped and proposed to the authorities by Gyproc in correspondence of 9th September 2019. The final scope was concluded by correspondence between the company and the authorities on the 4th October 2019.

19) Clarification is required on the various depths of mine activity below the road mentioned in the report. Are these depths or the lengths of boreholes? The report should be reviewed to ensure all references to depths are accurate and consistent.

Interim and Final Answer: Depths and thicknesses relate to true depth and thickness. Downhole depths and lengths are only referred to in relation to the borehole drilling and logs. Note that the graphics for each borehole shown in Appendix A give both downhole and vertical depths.

WAI Comment
Accepted

20) (original # 15) Explanation required as to why air pockets exist in Zone 4, if the area is permanently flooded. When will the air pockets cease as the water levels is reduced, and can the pressurised air be released to allow investigations (linked to no. 21).

Interim Answer: The precautionary approach to all work and consideration of the Drumgoosat mine in recent investigations had led to the assumption that the rock body would be somewhat permeable to air and that rising water levels would have driven out any air from the workings through the rock body resulting in flooded workings. The existence of the air pockets some considerable time after the water levels in the area were increased points to the rock and surrounding geology being less fractured and less permeable to air than presumed. A series of sectional drawings will be prepared and submitted as part of the final reply explaining why the levels and direction of approach of the rising water levels will have caused air pockets to form in the first instance.

Final Answer :Attachment Ref: "Q20 - R179 - Airlock headings.pdf"

WAI Comment
Accepted but further works were completed in Q2 2021 related to the airlocked area. See Section 1.3

21) The Authorities request a contour of the current water levels in this area.

Interim and Final Answer: A contour of current water levels is appended Attachment Ref "490-24-20 Drumgoosat Mine-Water Level Zone 4."

WAI Comment
Accepted

22) What is the risk assessment for each of the zones under the R179? Low? Very low? Moderate?

Interim and Final Answer: SRK advise that the risk is considered Very Low.

WAI Comment
Based on the information provided WAI agree with this assessment for the length of the R179.
However, there are a number of factors connected with Zone 1, in terms of thickness or presence of Upper Gypsum, strength of the upper section of the Lower Seam due to mudstone partings and geological anomalies in the area which have been put into context as part of the additional work undertaken by Gyproc. The work has built up as comprehensive picture of the area as is possible which has then been professionally interpreted. WAI agrees with the interpretation reached by Gyproc and SRK, however the interpretation requires the continued monitoring of Zone 1 to quickly identify any changes to that status.

23) Please confirm that all available geological logs, geotechnical logs (excluding R17 & R18) and laser scans were considered as part of the SRK analysis.

Interim and Final Answer: SRK confirm this.

WAI Comment
Accepted

24) Please provide the following details on the drilling: drilling contractor, drill hole size, drilling flush details, who undertook supervision of drilling, what downhole surveys were undertaken to confirm accuracy of the borehole inclination.

Interim and Final Answer: A detailed schedule is appended. Attachment Ref: "L4900 R179 Borehole Extensometer and Scanning Status 21.08.2020."

WAI Comment
Accepted

25) When a wider overview is taken of the area and the roof is considered across all the scans, rather than in isolated locations, potential linear features are evident in the roof strata. These potential features might have had an impact on roof stability and degradation, or even exert a control on the location of crown holes. What is SRK's view on these linear features that appear to be oriented approximately NNW-SSE?

Interim Answer: A response will be submitted in early September.

Final Answer: The NNW-SSE trending linear features observed in the roof of the underground workings are one of a set of two sub-vertical to vertical joints, one trending NNW-SSE and the other trending WSW-ENE, that are pervasive throughout the mine and are typical structures within this type of rock mass. They are tensional and dilational features and occur in response to rock movement as a result of folding or rolling of the gypsum bed and generally lie parallel and at right angles to the fold axis orientation. A stereographic plot of structures mapped during SRK's 2001 underground inspections in the vicinity of Zone 1 are shown in Figure 7 of the attached document. Interaction of these features with horizontal bedding planes can result in blocks of rock that can fall from the roof. This is a normal occurrence in underground mines. They are unlikely to control the location or occurrence of crown holes. Attachment Ref "R179 Zone 1 - Additional Assessment(V2).pdf"

WAI Comment
WAI concur with this response except where block failure results in loss of the roof beam where only thin roof beam is present, or the quality of the roof beam is compromised by the presence of mudstone partings. Thin or loss of roof beam is identified as key mechanism of crown hole formation (in conjunction with other parameters). However in Zone 1, where there are more prevalent mudstone partings within gypsum units, the gypsum roof beam thickness is significant (e.g. >9m thick) and therefore this is will offset the impact of any mudstone partings .

26) (original # 21) Can SRK offer a suggestion as to why one of the rooms in Zone 2 is 11m high?

Interim Answer: A response will be submitted in early September.

Final Answer: The area noted as having 11m high workings has been identified as being an area along the main haulage in the mine. The 11m high workings are as a result of the floor being deliberately excavated at a very localised point, potentially to form a temporary sump. This is visible in the cross sectional views taken from the laser scanning data from boreholes R6, R6A and R7. Note the roof of the mine working is relatively uniform and lies on an even plane indicating it is not roof failure or excavation into the roof beam. Attachment Ref "Zone 2 – 11m high Workings.pdf"

WAI Comment
Accepted

27) Please show high, intermediate and low (historical) water levels on cross sections for Zone 3 and 4.

Interim and Final Answer: Drawings as requested are attached. Attachment Ref "490-24-20 Drumgoosat Mine-Zone 3 section" and "490-24-20 Drumgoosat Mine-Zone 4 section"

WAI Comment
Accepted

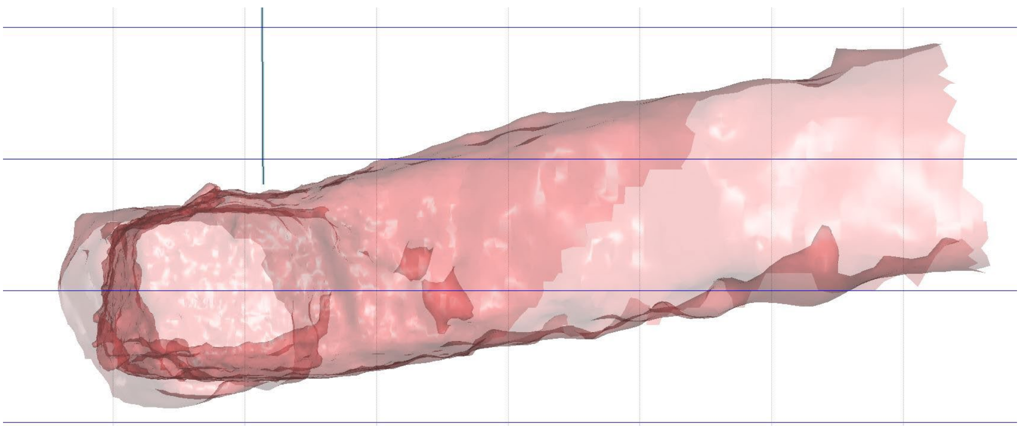
28) In some of the cross sections in Zone 4 and 5, dolerite occurs in the immediate floor of some workings. What is the condition of this dolerite? Is it competent or weathered and friable?

Interim and Answer: Only Borehole R16 intersected dolerite and it was found to be competent.

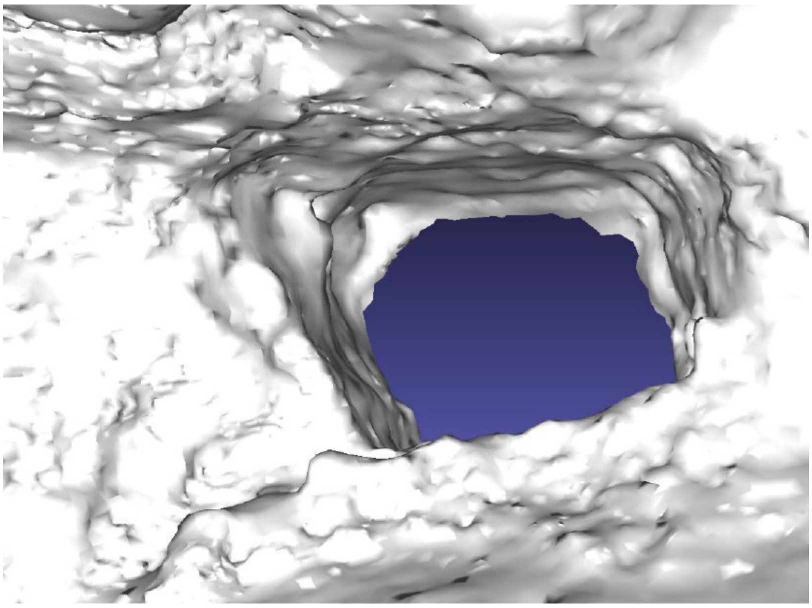
WAI Comment
Accepted

29) What are the stability implications for the R179 if the sidewall wedge formation evident in figure 8.9 failed?

Interim and Final Answer: The room referenced in Figure 8-9 is located on a dipping part of the orebody. The figure below shows the total extent of the R14 sonar scan viewed towards the NW. The box in the figure shows the approximate position of Figure 8-9.



The original Figure 8-9 is slightly rotated from the horizontal indicating a floor slope slightly steeper than it should be. An updated horizontal image of Figure 8-9 is presented below. The left hand wall of the tunnel is slightly non-vertical, but it is likely that it has formed parallel to a structure sitting at right angles to the seam dip. This is likely to be a very stable wall. Additionally, this wall is located at the mine limit. There is no further mining to the left (SW) of this tunnel.



Original Figure 8-9

Figure 8-9 updated and rotated to horizontal

Horizontal Line

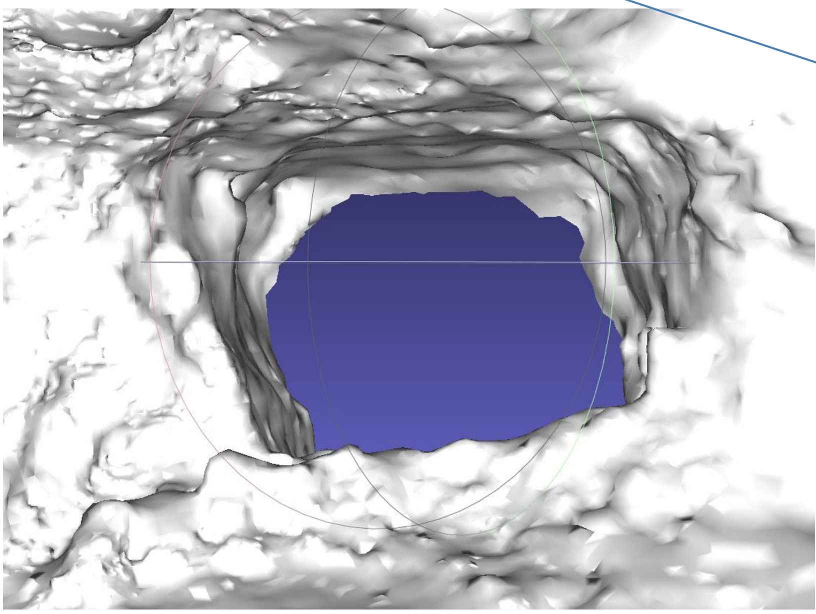


Figure 3: (Figure 8-9) Original and Updated

WAI Comment

Accepted

1.3 Follow-up Works Related to Airlocked Area in Zone 4

Context

During the original R179 drilling and laser scanning programme undertaken in early 2020 an area of workings in Zone 4 was identified as being airlocked. The laser scanning of two holes in this area could not be completed due to the risk created by the high air pressure within these holes. It was agreed that these holes would be laser-scanned in Spring 2021, once pumping had significantly reduced the water level within the Zone 4 mine workings. The laser scanning was completed in May 2021 and additional interpretation of the layout of mine workings in the Zone 4 area was finalised in late July 2021. Several reports related to the airlocked area were produced by Gyproc and SRK which were considered by WAI.

The following reports were supplied to WAI and reviewed:

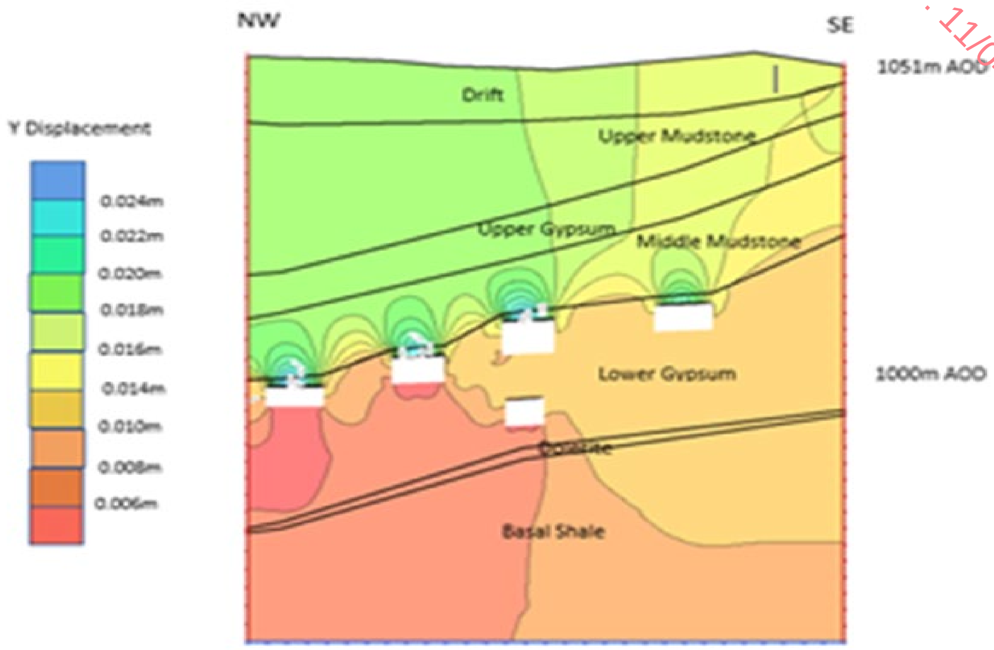
- SRK External Memorandum – KC20-R16 Dome Interpretation, 02 June 2021.
- Interpretation of Workings in the "airlock area" Drumgoosat Mine, Revised 26.7.21 (including report "Update to Authorities on R179 Drilling program – 6.2.2020).
- KC20 R16 drillhole log.
- SRK R179 Investigation – Zone 4 Addendum Report, July 2021.

In addition, WAI reviewed again, the following data related to the Zone 4 area which was contained within its records:

- KC20 – R12 drillhole log
- WAI FLAC stability assessment cross sections
- WAI assessment of extent of upper and lower workings within the Lower Gypsum Seam.

WAI Comment

WAI confirm that it agrees with the SRK assessment of the dome feature observed with the upper workings of the lower seam adjacent to the R179, in that it is likely to have formed at the time or soon after mining the area, and that the dome propagation is likely to have stabilised as a natural arch feature has developed. In the event that further deterioration occurs the approximately 10m thickness of gypsum in the upper seam will arrest any void migration. The strata configuration and working placement is comparable to the WA4 model shown below which indicates that the workings do not interact.



With regard to the assessment of the extent of the double levels workings, this appears to be a reasonable assessment of the extent of the two levels and matches with the assessment WAI completed as part of the initial review. It is noted that the ventilation raise is identified as a new feature but it is a part of the mine development between the Upper horizon and Lower horizon of the Lower Seam, and is overlain by a thick Upper Gypsum seam. The laser scanning of the area indicates it is stable.

While WAI broadly agree with the findings of the updated (July 2021) SRK report, there are a small number of keys points with which WAI does not concur. The July 2021 report has summarised the earlier SRK report on the dome feature and states that the roof of the workings with the dome feature are approx. 65m below surface and as such below the limit of where a crownhole could propagate to surface. However, the presented cross sections and the R16 log quite clearly show that the workings are 54 – 58m below ground level and the roof of the dome is some 5m above this, at a depth of around 49m – 53 m below ground level. Borehole R16 records indicate approximately 12m of drift. The depth below ground for crownhole propagation should only consider solid rock, therefore the dome feature is only approximately 40m below "ground level" and well within the limit of where a crownhole could propagate to surface. However, WAI agree with SRK that the feature appears to have reached a natural stable state and in the unlikely event that further deterioration occurs, the 10m thick Upper Gypsum layer would prevent propagation to the surface.

WAI agree with the assessment of the laser scans that with the exception of the dome feature the walls and roof appear to be in good condition with no obvious signs of instability. However, it is also noted that laser scanning has not been completed of the lower workings at R16 although they have been intercepted. WAI assumes this is due to difficulty in getting the survey equipment to the lower workings through the upper workings. It would be beneficial to confirm that the lower workings at this location are as anticipated.

2 CONCLUSIONS

WAI carried out a review of the original works of SRK in relation to the stability of the R179 and also a review of the responses to questions raised by WAI's review.

WAI have also carried out a review of the additional reports provided by Gyproc and SRK in Q2 2021 related to the 'airlocked' area and double level workings in Zone 4

WAI concludes that Gyproc have undertaken works to enable the queries related to the initial SRK report to be answered in a full and acceptable manner and have had SRK carry out further works to enable the questions to be answered where required.

WAI considers the answers provided close out those issues raised by the original report as far as is practically possible and that R179 is safe to use.

Much work has been done to assess and analyse Zone 1 and its surrounds. The documents "R179 Zone 1 - Additional Assessment(V2)" and "R179 Zone 1 - Additional Assessment(V2)-Amended Dec 2020" and the plan "DCCAE Plan Final" provide an overview of the wider area. The referenced documents are a more complete assessment and indeed cover the broader aspects questioned within the zone to do with geological anomalies, Upper and Lower Seam gypsum thicknesses, historic crownholes and localised doming.

In addition, Zone 4 was an area where flooding of the workings created areas of high pressure air pockets that initially precluded further investigation of the mine voids by laser scanning. Verification of the SRK conclusions reached by FLAC modelling was not initially possible. Further works were undertaken once the water level had dropped and the area depressurised. WAI agree with the interpretation of the works and the conclusions reached, that the risk to the R179 can be considered extremely low.

WAI conclude that whilst the current interpretation of the works by Gyproc and SRK is supported and the R179 is considered safe for use, the interpretation of Zone 1 could change depending on the monitoring results in the future. To this end, WAI are in agreement with the monitoring proposed to be undertaken by Gyproc with regard to its type, frequency and TARP. The results of the monitoring should be regularly reviewed and analysed to ensure that the conditions of mine voids below the R179 remain stable.

APPENDIX 1

Interim Report ZS611396-RPT-V3.0, dated February 2021

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ENERGY AND CLIMATE CHANGE
ENVIRONMENT AND SUSTAINABILITY
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MINING AND MINERAL PROCESSING
MINERAL ESTATES
WASTE RESOURCE MANAGEMENT

THE DEPARTMENT OF THE ENVIRONMENT, CLIMATE AND COMMUNICATIONS

DRUMGOOSAT UNDERGROUND MINE

INDEPENDENT REVIEW OF THE STABILITY REPORT ON THE DRUMGOOSAT UNDERGROUND MINE WORKINGS BELOW AND ADJACENT TO THE R179 CARRICKMACROSS TO KINGSCOURT ROAD, CO. MONAGHAN – INTERIM REPORT

February 2021

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THE DEPARTMENT OF THE ENVIRONMENT, CLIMATE AND COMMUNICATIONS

DRUMGOOSAT UNDERGROUND MINE

– INTERIM REPORT

February 2021

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1 INTRODUCTION

Wardell Armstrong International (WAI) has been appointed by The Department of the Environment, Climate and Communications, (DECC or the Client) to review and validate the findings of SRK's stability report on the Drumgoosat mine workings that are present below and adjacent to the R179 road in Co. Monaghan. The SRK report reviewed is referenced:

- Drumgoosat Underground Mine – investigation and analysis of mine stability below the R179 Road, prepared for Gyproc Ltd, by SRK Consulting (UK) Limited, referenced: 30787_R179 Road Investigation Final(V2) April 2020.

In detail the WAI scope of works comprised four main aspects:

- Review of all existing non-confidential documentation available on the project;
- Review and validation of the April 2020 SRK report to confirm the overall conclusions reached by SRK on the stability of the R179 and the underlying mine workings are correct. In particular this will involve confirming that all underlying assumptions and inputs are valid in the following key areas:
    - the geotechnical investigation work, geotechnical logging and rock strength testing;
    - the finite element (FE) modelling of 12 cross-sections that traverse the vicinity of R179; and
    - the 3d laser scanning to confirm interpretations made by SRK.
- Complete FE modelling of 4 cross sections across the R179 at locations agreed with DECC using FLAC 3D, Rockscience RS2 or similar software to check and verify the findings of SRK with particular consideration to surface displacement and excavation roof beam and pillar stability.
- Advise on the adequacy of SGMI/Gyroc's proposed monitoring measures and Trigger Action Response Plan (TARP).

This report summarises the review of the SRK report taking into account the historic data available on the project including previous SRK stability assessments and modelling, borehole logs, mine water records, surface monitoring data and previous WAI reviews and models. In addition, the findings of the finite element stability analysis undertaken by WAI of the four additional sections lines along the R179 are presented. For ease of reference this report is structured in the same ways as the SRK report.

2 REPORT REVIEW

2.1 Background

SRK have an extensive and long history with the Drumgoosat mine, ranging from underground inspections and surveying of the mine workings through to the recent stability assessments and analysis. They are well placed to understand the history and configuration of the mine.

They present a concise summary of the complex history of the site. An overall site location plan is shown on WAI Drawing ZS611396-001.

2.2 Work Undertaken

The scope of works is comprehensive and covers all the areas typically necessary for a geotechnical stability assessment. It should be noted that the SRK brief was to undertake a stability assessment and not a risk assessment, which is an important distinction to be aware of.

It is noted that the works were undertaken in a short time frame and that Covid-19 lockdown restrictions occurred during the core logging works preventing all of the core logging being completed.

3 GEOTECHNICAL INVESTIGATION WORK

3.1 Geotechnical Investigation Design and Execution

The report provides a brief summary of the rationale and design inputs on the geotechnical investigation (GI) with 17 boreholes drilled between late 2019 and early 2020. Exact dates have not been provided for the 2019/2020 drill campaign and two boreholes are included from earlier drilling campaigns. However, it is not easily determinable from the text and tables whether all borehole records (geological log, geotechnical log and downhole scan) were available to SRK at the time of the geotechnical assessment.

Of minor note, within the text, borehole KC19-H24 is wrongly referred to as KC19-H4, this is considered to be a minor typographical error as all other references refer to the correct borehole nomenclature.

The borehole data available for the SRK report is summarised in Table 3.1 below from which is it possible to see that although a total of 19 boreholes are referenced (17 boreholes drilled in 2019/2020 plus two from earlier campaigns), only 12 boreholes have a full suite of logs and records.

3.2 Borehole Drilling

The boreholes were drilled by double-tubed wireline system which is a standard method of drilling for core recovery. However, there are no details on the drilling flush used, the contractor who undertook the drilling, who undertook the supervision and what level of supervision was provided. While not vital to the assessment this information provides a more comprehensive summary of the works undertaken.

It is not clear who was responsible for setting out and confirming the location of each borehole. Furthermore, it is noted that the majority of the boreholes are inclined to target the 4-way junctions under the roadway, but no details are provided on what checks or downhole surveys were undertaken to confirm the accuracy of the borehole inclination, although it is noted that the scan survey of the workings identifies the final location of the boreholes.

Table 3.1: Geotechnical Investigation Summary

BH ID	Azimuth (o)	Dip (o)	Depth (m)(length)	Depth (m bgl)	Depth (elevation)	Easting	Northing	Elevation	Zone	Geo log + photo	Geotech log	Laser Scan	Comment
KC20-R01	113	56	53 (53.3)	44.19	1010.18	280966.51	300130.53	1054.37	1	x	x	x	
KC19-R03	91	59	50 (50.3)	43.12	1014.85	280939.51	3000080.8	1057.97	1	x	x	x	
KC19-R04	94	59	50	42.86	1015.72	280924.73	300055.92	1058.58	1	x	x	x	
KC19-R05	120	59	47 (47.2)	40.46	1016.39	280898.69	300000.98	1056.85	2	x	x	x	
KC20-R06	58	88	40 (39.5)	39.48	1015.52	280925.24	299967.43	1055.00	2	x	x	x	
KC20-R06A	332.9	89.7	35	35.00	1016.76	280945.25	299954.79	1051.76	2	x	x	x	
KC19-R07	100	63	46	40.99	1014.52	280885.26	299967.86	1055.51	2	x	x	x	
KC18K-R08	0	90	41.2	41.20	1012.18	280868.83	299934.01	1053.38	2	x		x	Graphic log only included in report, geological log provided by DECC to WAI after issue of SRK final report.
KC19-R10	91	71	46 (45.5)	43.02	1006.55	280821.80	299884.38	1049.57	3	x	x	x	
KC19-R11	152	57	52 (51.5)	43.19	1005.73	280807.19	299866.47	1048.92	3	x	x	x	
KC19-R11A	155	60	51 (51.2)	44.34	1004.04	280792.52	299848.70	1048.38	3	x	x	x	
KC19-R12	175	75	63 (62.2)	60.08	986.68	280751.68	299803.16	1046.76	4	x	x		No scan completed as airlock encountered - additional explanation provided under separate cover
KC20-R13	98	79	80	78.53	965.55	280689.17	299751.17	1044.08	4	x	x	x	
KC19-R14	104	90	93 (92.5)	92.50	949.03	280593.96	299676.80	1041.53	5	x	x	x	
KC20-R15	8.9	58	46	39.01	1017.92	280995.67	300034.34	1056.93	1	x	x		Geological log only available after issue of SRK report. Scan reported to have been completed but no record included in assessment report.
KC20-R16	149	89	91	90.99	955.48	280734.60	299806.65	1046.47	4	x	x		Crucial to understanding upper and lower workings relationship. Geological log not included in SRK report, provided by DECC to WAI. No scan completed due to airlock encountered. Additional explanation provided under separate cover.
KC20-R17	59	90	100.6	100.60	945.91	280704.62	299808.11	1046.51	4	x		x	Geological log not included in SRK report, provided by DECC to WAI. No geotechnical log available, scan covers upper and lower workings.
KC20-R18	187.7	89.7	106.6	106.60	941.95	280704.74	299839.32	1048.55	4	x			Geological log not included in SRK report, provided by DECC to WAI. No geotechnical log or scan available.
KC19-H24	137	71	50 (44.9)	42.45	1012.62	280951.60	300109.5	1055.07	1	x		x	Geological log not included in SRK report, provided by DECC to WAI. No geotechnical log available.

Note: (92.5) indicates correct depth recorded on logs. Depth report in summary table appears to reflect rounding of the actual depths. Minor variation and has not been reflected in the modelling.
X indicates log issued to WAI as part of the review but not included in the SRK report.

For ease of analysis the R179 has been divided into 5 zones, zone 1 – 5 moving from north to south. Each zone is not uniform in area but is grouped around clusters of boreholes and pillars in the underground mine workings. The zone areas are shown in Figure 3-1 below which is extracted from the SRK report.

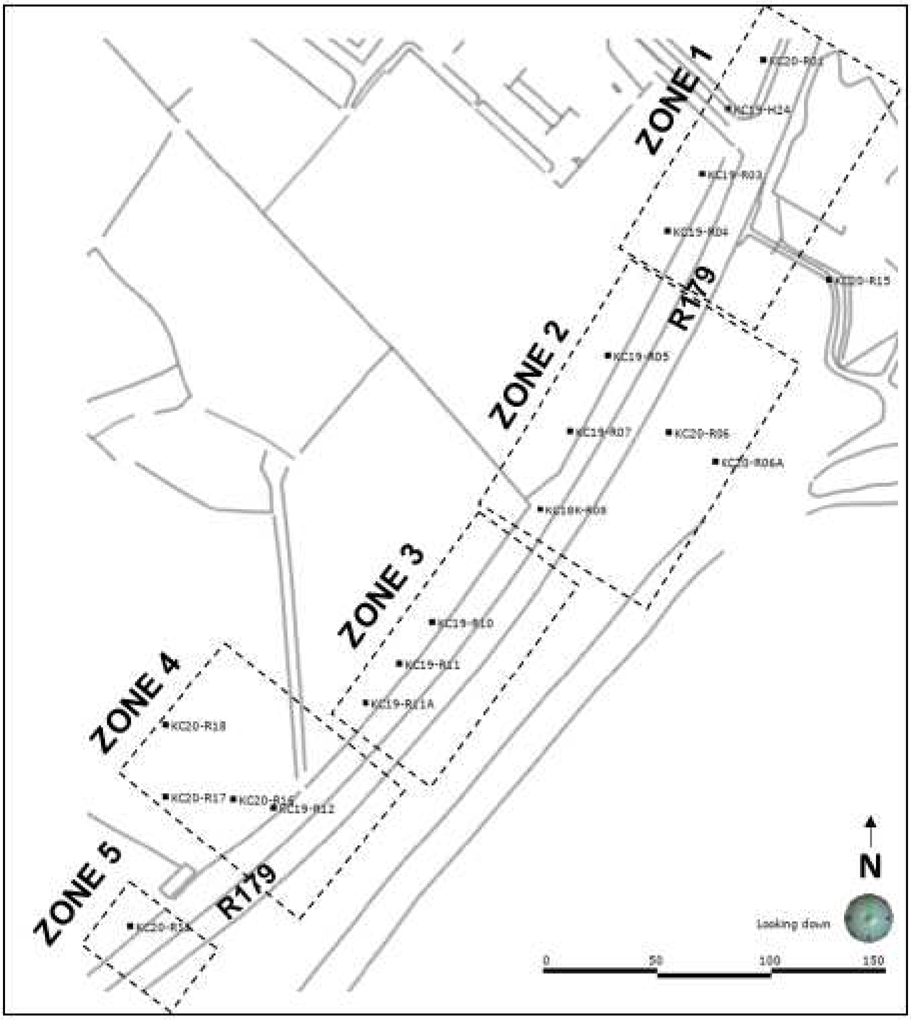

Figure 3-1: Extract from SRK report Figure 2-1 showing borehole location and assessment zones relative to the R179.

As a general comment it is noted that there is not a figure included in the report which shows all the aspects of the GI assessment on a single plan: road alignment, assessment zones, borehole locations and traces, mine workings and cross section locations. This has made reviewing the data and assessment more difficult and somewhat problematic; however, this is a minor comment. A combined layout is shown on WAI Drawing ZS611396-002.

Zone 1 is reported to include borehole KC20-R15, however the zone extent shown on the plans appears to exclude this borehole. This is further supported by the laser scans not including KC20-R15 although it was reported to have been scanned and SRK not having access to the geological log. It is therefore not clear if borehole KC20-R15 was included in the assessment of Zone 1. This is discussed further in the modelling section.

It is noted that the large spacing between Zone 4 and Zone 5 is due to the absence of mine workings beneath the R179 in this area with the exception of one single driveway.

The zone configuration is considered sensible and appropriate for the assessment. Based on the data included in the SRK report the zones assessments have been completed using the data sets as summarised in Table 3.2. A complete data set includes a geological log, a geotechnical log and a laser scan.

Table 3.2: Summary of data sets used in each assessment zone

Zone	No. Drilled boreholes	No. of complete data sets (geo log, geotechnical log and laser scan)	Comment
1	5	3	KC20-R15 no scan or geological log. KC19-H24 no detailed geological or geotechnical log.
2	5	4	KC18-R08 no geological and geotechnical log
3	3	3	No data missing
4	5	1	Only KC20-R13 complete
5	1	1	No data missing
Total	**19**	**12**	

It is noted that the settlement monitoring undertaken along the R179 does not appear to have been included in the assessment until the 'Conclusion' section.

3.3 Geotechnical Logging

Although this section is presented as detailing the geotechnical logging it refers to the Rock Mass Rating system used to determine the rock mass parameters. This is not a logging system but rather a classification system. It is stated elsewhere in the report that the geological logging was undertaken by Gyproc Geologists and the geotechnical logging by SRK engineers. This is not made clear in this section.

The geological logging, although not stated, appears to have been broadly undertaken in accordance with standard logging codes such as BS EN ISO 14689:2018 Geotechnical investigation and testing – identification and classification of rock – Part 1: Identification and description.

The core photography has greatly improved from the logging completed in 2018 with clear detailed photographs including depth markers, scale, colour charts and borehole details on each photograph. Each photograph has been standardised and all core has been photographed when a completed geological log has been included in the appendix.

While some attempt has been made to correlate the core logging parameters such as core run, total core recovery (TCR), solid core recovery (SCR) and rock quality designation (RQD) with the strata geological description and depth, discrepancies remain which on closer inspection make the logs difficult to interpret or simply wrong in some instances.

It is noted that the TCR has not been either recorded correctly or adjusted for drilling breaks and or missing core, as throughout the logs, values of greater than 100% are reported for the total core recovered. While this is not unusual in one core run, the next core run is typically less than 100% and as such the core records are adjusted to reflect this discrepancy. In one instance a combined extra TRC of 43% is recorded over 7 consecutive core runs (KC20-R01), an extract from the log is shown in Figure 3-2. While on an individual basis this error may not be significant, it raises questions on the true thickness of units recorded and depths below ground and the overall accuracy of the core logging. This is further supported by RQD of greater than 100% reported, something that is not possible. This indicates that the correct core logging procedure is not understood or being implemented by the Gyproc Geologist. Furthermore, as can be seen in Figure 3-2 there are discrepancies between the core run depths and the strata depths and thicknesses. This makes interpreting the logs and core data nearly impossible in the current format.

SAINT-GOBAIN

BOREHOLE NUMBER: KC20 - R1

Core Run From	Core Run To	Core Run (m)	TCR (m)	TCR (%)	SCR (m)	SCR (%)	RQD (m)	RQD (%)	Depth (to)	Thickness (m)
26.00	29.00	3.00	3.20	107	3.00	100	3.00	100	35.84	10.94
29.00	32.00	3.00	3.27	109	2.69	90	2.64	88	36.10	0.26
32.00	35.00	3.00	3.19	106	3.19	106	3.19	106	37.65	1.55
35.00	38.00	3.00	3.14	105	3.14	105	3.14	105	38.00	0.35
38.00	41.00	3.00	3.20	107	2.50	83	1.50	50	38.44	0.44
									38.66	0.22
									39.01	0.35
									39.41	0.40
									39.56	0.15
									39.73	0.17
									39.78	0.05
									40.14	0.36
									40.35	0.21

Figure 3-2: Extract from geological log for borehole KC20-R01 included in Appendix A of SRK report. Values of TCR greater than 100% highlighted in yellow. The text highlighted in green indicates areas where the core run does not match the strata depths and thicknesses recorded

It is not clear what geotechnical logging system has been used as the ratings applied, while following the parameters of RMR_{89} do not follow the rating divisions (see extract in Figure 3-2 for examples). The RMR input parameters are discussed in more detail below. RQD, and TCR appears correct, and the logging grouping follows standard international procedures. Therefore, while an RMR value is indicated it is not possible from the logging to establish if this is correct. However, the RMR are generally for fair to good rock which are overall relatively conservative. A review of the core photos appears generally consistent with the strata recovered as far as it is practicable to identify from core photographs.

RMR_{89} is an industry wide recognised rock mass classification system with set input parameters and ratings. SRK list the input parameters and ratings as follows:

- Rock Quality Designation (RQD) 0 – 20
- Intact Rock Strength (IRS) 0 – 15
- Spacing Rating 0 – 20
- Joint Condition Rating 0 – 30
- Groundwater Condition Rating 0 – 15
- Orientation -50 to 0.

While the majority of these input parameters and ratings are correct there are some minor discrepancies from the published RMR guidance such as RQD and joint spacing. However overall, these differences are not considered to impact of the final RMR assessment.

SRK state that dry ground water conditions have been assumed in all strata and as such a rating of 15 has been applied throughout and that the water level observed within the mine is then accounted for during the modelling with the addition of a piezometric level. This has been done to allow the Geological Strength Index (GSI) to be calculated quickly from RMR_{89}.

To calculate GSI, a RMR_{89} assessment has the groundwater rating set to 15 and the joint orientation to zero. This results in a modified RMR_{89} value reported as RMR_{89}'. In effect SRK are reporting RMR_{89}' rather than RMR_{89}. This should be made clear in the reporting but does not impact on the assessment findings.

While it is correct that the joint orientation is set to zero to determine GSI, SRK state that the majority of the discontinuities were found to be perpendicular to the core axis, indicating near horizontal bedding planes (for vertical boreholes). The majority of the boreholes for the R179 investigation are inclined. This therefore means that the joints and bedding are dipping, as can be seen in the roof strata on several of the scans where a stepped profile is observed. However, when calculating the GSI the joint orientation is set to zero, so as such SRK have calculated RMR_{89}' and should clearly state this.

SRK have accounted for the weaker mudstone layers within the gypsum by applying a weighting for the relative proportion of mudstone recorded.

The observations and discrepancies noted here in applying RMR_{89} ratings are the same as identified in previous SRK reviews. Standard procedures have not been adhered to, which make reproducing the values difficult, however, the overall ratings achieved are considered conservative and in line with the RMR obtained in previous stability assessments and therefore considered acceptable for the ground conditions observed here.

3.4 Rock Strength Testing

The strength of the gypsum has been determined by a combination of laboratory testing and field measurements. A total of 17 samples were sent for testing in the UK, one sample per borehole. All samples are of gypsum and SRK have not identified if they are all of the Lower seam or if representative samples of the Upper seam have also been included.

The Uniaxial Compressive Stress (UCS) results are presented in Table 2-4 of the SRK report. All UCS values are reported and then a corrected UCS value is also reported. It is not clear what this correction value is and why it is required but it appears to show how the UCS value has been rounded up.

SRK have then attempted to verify the field strength observations by comparison between the field recorded Intact Rock Strength (IRS) with the laboratory obtained UCS values. IRS is obtained by repeatedly hitting a section of core with a geological hammer and counting the number of blows required to achieve a specific effect, e.g. chip, break etc against a standard published table. This procedure relies on the field engineer(s) undertaking the procedure in the exact same way each time to achieve repeatable reliable results.

SRK have then compared the IRS field observations with the laboratory acquired strength results to apply a correction factor to the field results. SRK suggest that a conversion factor of 1.79 is required. However, the graph presented in the report only shows a weak correlation and other conversion factors could equally be assumed to be correct. SRK have not made use of previous UCS and Point Load Testing (PLT) undertaken as part of the L4900 assessment. Inclusion of this test data would have increased the data set, although it is acknowledged that this is from a different section of the mine but would give an indication of the range of strengths within the gypsum over a wider area.

As mentioned previously the RMR system uses input parameters of UCS and Point Load Test. PLT's are quick simple tests which can be undertaken either in the field if the correct equipment is available, or in larger numbers in the laboratory. The PLT are then used in conjunction with the more expensive and accurate UCS tests to determine the ranges in rock strength throughout the strata. SRK previously undertook PLT and UCS testing with subsequent analysis of the PLT/UCS conversion. It is not clear why PLT have not been utilised in this campaign. The UCS testing in 2018 typically recorded gypsum strengths of 18.1 MPa to 26.4 MPa, the gypsum strengths recorded here a typically lower and, as such, the assessment is more conservative.

In summary SRK could have used their historic data set to increase the verified strength parameters across a wider area. However, as the strength values used in the analysis are on the slightly low side it is considered that a conservative assessment has been used.

3.5 Borehole Laser Surveys

SRK report that laser scan surveys were completed in all but three of the drilled boreholes along the R179 with six boreholes in Zone 1, five in Zone 2, three in Zone 3, two in Zone 4 and one in Zone 5. However, this tally does not match with the number of boreholes drilled in each zone or the number of scans included in Appendix D as summarised in Table 3.3 below. This means that less data has been available to complete the assessment than SRK indicate.

Table 3.3 : Summary of laser scan surveys

Zone	SRK Reported	Scans included in Appendix D	No of boreholes in Zone	Missing Data
1	6	4	5	KC20-R15
2	5	5	5	None
3	3	3	3	None
4	2	4 listed but only 2 produced	5	2 boreholes have hit airlock workings and scanning not undertaken
5	1	1	1	none
Total	**17**	**13**	**19**	

SRK does not make it clear in the report that the scans in Zones 1 – 3 were carried out in dry conditions above the mine water level, while the lower workings of Zone 4 and all of the workings in Zone 5 were undertaken below the mine water level. There is no indication of any variation in survey technique, accuracy or interpretation of the data.

The overall information provided on the laser scan technique is limited and does not identify the change in scanning techniques for the flooded areas. However, the scan data included in the appendix is detailed and extensive.

4 R179 UNDERMINING ANALYSIS

4.1 Areas of inspection

The assessment zones are again identified in the report and in addition the location of the 12 section lines used in the undermining analysis are also highlighted. The section line location was selected and agreed between SRK and Gyproc. However, it is not made clear that several of these section lines were analysed as part of the 2018 analysis. The zones, section line locations and previous section line analyses are shown on WAI Drawing ZS611396-002. As can be seen on the drawing, the following 2020 proposed scan lines were initially modelled in 2018:

- Zone 1 scan line 1a is the same as SRK R179 -2018 Loc 1.
- Zone 2 scan line 2a is the same as SRK R179-2018 Loc 2b- R8.
- Zone 2 scan line 2b is the same as SRK R179-2018 Loc 2 – R21.
- Zone 3 scan line 3a is positioned one pillar south to the SRK R179-2018 Loc 3, the room and pillar configuration is almost identical between the two positions.
- Zone 4 scan line 4a is positioned one pillar south of the SRK R179-2018 Loc 4 position. It should be noted that the WAI scan line WA4 is the same position as SRK R179-2018 Loc 4.

This alignment of previous scan line analysis with the 2020 analysis allows for comparison and checking of the modelling, parameters used, and assumptions made. SRK make brief reference to this previous analysis in later sections.

4.2 Geotechnical Characteristics for Numerical Modelling

The geotechnical parameters used in the numerical modelling are based on a combination of calculated, logged, tested and engineering experience of similar materials. This is a standard way of determining material properties for a range of strata which has not all been logged and tested. The software used to determine cohesion and friction angle is an industry standard.

SRK report that they have determined values for the glacial till or drift overlying the dolerite. However, in the summary table they have only presented values for drift, while glacial till was recorded on the logs, and dolerite was not encountered overlying the gypsum in this area. However, the input parameters are similar or the same as the values used in 2018 as shown in Figure 4-1 and Figure 4-2 below and are considered reasonable.

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ZONE	Lithology	IRS	RMR	GSI	Unit Weight (MN/m3)	c (MPa)	phi (°)	Young's Modulus (MPa)	Poisson's Ratio
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
A1	Glacial Till	N/A	N/A	N/A	0.018	0.2	20	100	0.3
	Dolerite	11	28	23	0.020	0.06	35	1000	0.3
	Gypsum	25	54	49	0.023	0.264	49.78	4875	0.15
	Mudstone	N/A	N/A	N/A	0.020	0.4	20	200	0.2
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
A2	Glacial Till	N/A	N/A	N/A	0.018	0.3	20	100	0.3
	Dolerite	17	29	24	0.020	0.06	35	1000	0.3
	Gypsum	21	59	54	0.023	0.287	49.7	4875	0.15
	Mudstone	N/A	N/A	N/A	0.020	0.4	20	200	0.2
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
B3	Glacial Till	N/A	N/A	N/A	0.018	0.3	20	100	0.3
	Dolerite	1	27	22	0.020	0.06	35	1000	0.3
	Gypsum	22	57	52	0.023	0.273	49.59	4875	0.15
	Mudstone	N/A	N/A	N/A	0.020	0.4	20	200	0.2
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
B4	Glacial Till	N/A	N/A	N/A	0.018	0.3	20	100	0.3
	Dolerite	15	29	24	0.020	0.06	35	1000	0.3
	Gypsum	25	57	52	0.023	0.298	50.39	4875	0.15
	Mudstone	N/A	N/A	N/A	0.020	0.4	20	200	0.2

Figure 4-1: SRK parameters utilised in 2018 assessment

Table 3-1: Modelling Input Parameters

ZONE	Lithology	Average Top Depth Below Surface (m)	Average RMR	Average GSI	Unit Weight (MN/m³)	c (MPa)	phi (°)	Young's Modulus (MPa)	Poisson's Ratio
	Drift	0	3	0	0.020	0.06	19	560	0.30
	Upper Mudstone	14.85	15	10	0.020	0.40	20	200	0.20
1	Upper Gypsum	15.40	47	42	0.023	0.09	48	4875	0.15
	Lower Mudstone	19.65	2	0	0.020	0.40	20	200	0.20
	Lower Gypsum	30.08	59	54	0.023	0.175	46.2	4875	0.15
	Basal Shale / Dolerite	42.73	N/A	N/A	0.025	10.5	35	20000	0.30
	Drift	0	6	1	0.020	0.06	19	560	0.30
	Upper Mudstone	N/A	N/A	N/A	0.020	0.40	20	200	0.20
2*	Upper Gypsum	15.65	30	25	0.023	0.09*	48*	4875	0.15
	Lower Mudstone	16.56	8	3	0.020	0.40	20	200	0.20
	Lower Gypsum	26.28	56	51	0.023	0.15	46.6	4875	0.15
	Basal Shale / Dolerite	41.92	N/A	N/A	0.025	10.5	35	20000	0.30
	Drift	0	3	0	0.020	0.06	19	560	0.30
	Upper Mudstone	11.24	8	3	0.020	0.40	20	200	0.20
3	Upper Gypsum	15.33	56	51	0.023	0.12	50.3	4875	0.15
	Lower Mudstone	23.49	0	0	0.020	0.40	20	200	0.20
	Lower Gypsum	35.32	54	49	0.023	0.162	44.0	4875	0.15
	Basal Shale / Dolerite	49.40	N/A	N/A	0.025	10.5	35	20000	0.30
	Drift	0	10	5	0.020	0.06	19	560	0.30
	Upper Mudstone	11.70	0	0	0.020	0.40	20	200	0.20
4	Upper Gypsum	33.81	60	55	0.023	0.26	48.9	4875	0.15
	Lower Mudstone	43.92	15	10	0.020	0.40	20	200	0.20
	Lower Gypsum	56.74	51	46	0.023	0.241	43.2	4875	0.15
	Basal Shale / Dolerite	77.80	N/A	N/A	0.025	10.5	35	20000	0.30
	Drift	0	0	0	0.020	0.06	19	560	0.30
	Upper Mudstone	8.70	0	0	0.020	0.40	20	200	0.20
5	Upper Gypsum	58.53	46	41	0.023	0.17	37.7	4875	0.15
	Lower Mudstone	68.80	0	0	0.020	0.40	20	200	0.20
	Lower Gypsum	81.43	59	54	0.023	0.279	39.1	4875	0.15
	Basal Shale / Dolerite	92.5	N/A	N/A	0.025	10.5	35	20000	0.30

*Less than 3 m of the Upper Gypsum and 0 m of the upper mudstone were intersected in the logging of the holes in Zone 2. As such, the same input parameters for the Upper Gypsum and Upper Mudstone in Zone 1 were used to define the strength of these units in Zone 2.

Figure 4-2: SRK modelling input parameters 2020

4.3 Finite Element Modelling

The twelve cross sections have been developed by the British Gypsum Technical Department to define the geological and mining boundaries. Where the section lines intercepted a borehole, the log has been used to validate the model and provide the actual mine void. Where no mining void scan was available a uniform 6m room has been assumed.

It should be noted that none of the boreholes penetrate the strata below the mine workings with the exception of where double layer workings were intercepted. Furthermore, from the boreholes provided it is not possible to recreate the models presented which have been prepared using SRK's extensive knowledge of the area from previous works. This has been applied to achieve the full extent of the models. Some discrepancies have been noted between the borehole logs and sections lines, but this could reflect differences in collar levels and the topographical survey and are not considered to have a significant impact on the model results. Comparison of the 2018 section lines with the 2020 revised models clearly shows where the models have been refined following the further geotechnical investigations.

As a general comment on the section models, they do not appear to have been created in relation to the surface elevations but formed rather from a nominal elevation or possibly a mine datum, this makes checking with the borehole logs and the scan models difficult and inaccurate as shown in Figure 4-3 but does not create a material difference in the modelling or assessment results.

The software used to model the FE assessment is standard industry recognised software. The roof beam deflection of 2% of roof beam thickness has been used and agreed in previous studies. However, there is no discussion of the actual numerical model adopted, i.e. whether the model assesses effective, effective with pore water pressure variation, or total stress properties of the individual materials. Nor is there any discussion as to how these materials are considered to behave, i.e. whether linear-elastic, anisotropic elastic, or elastic-plastic or indeed the time setting of the predicted movements. There is no discussion of how the FEA relates to the possible failure modes or deflections as the use of FEA cannot account for issues such as joints, or structural changes in the materials caused by previous movements, faulting, or other irregularities.

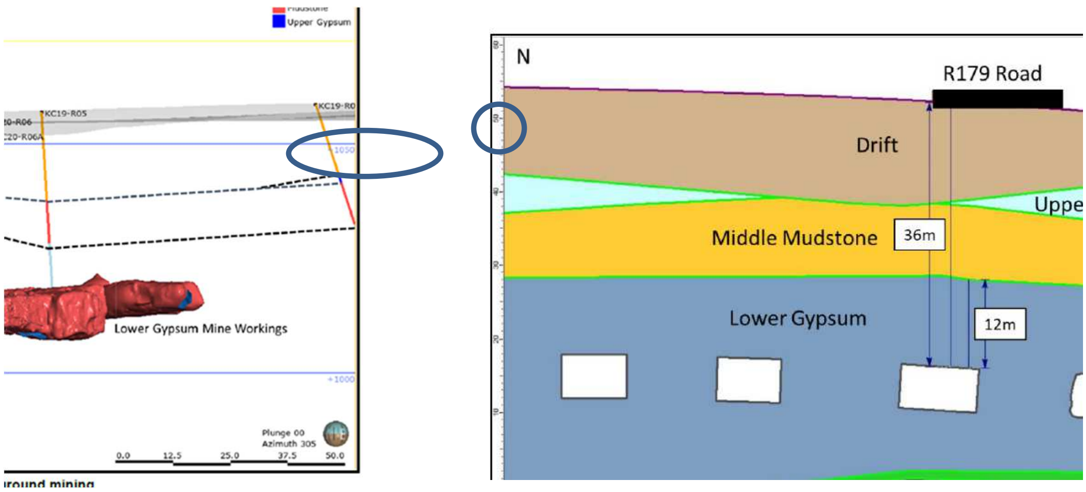


Figure 4-3: Extract from scale cross sections and model cross sections and different elevations

WAI finite element modelling has been undertaken at four locations as shown on Drawing No ZS611398-002. The analysis has been undertaken using propriety software Geostudios SIGMA/W. The model was assessed pre mining in the first instance followed by insertion of the mining voids to determine the settlement and deflection.

4.4 3D Laser Scan Interpretation

The typical characteristics of smooth stable roof and floor profiles and blocky strata potentially showing roof instability appear valid and reasonable.

In the next section the scan data and FE modelling is presented and summarised for each Zone.

5 ZONE 1

5.1 General Description of Undermining

SRK now show the extent of Zone 1 to include KC20-R15 which was excluded in the initial Zone area. Additionally, a scan from KC20-R15 is now shown on the overall plan, however it is not included in Appendix D so it cannot be interrogated in detail.

From the borehole logs the shallowest mine workings in the area are 38.37m below ground level not 40m and the cross sections record 39m, however this is recorded at KC20-R15 and is located east of the R179 alignment and the difference between 39m and 40m is not significant.

5.2 Summary of Investigation Works

The summary of seam thicknesses is supported by the borehole data. For clarity the length, rather than the depth, of the borehole should be reported in Table 3.1 because as the boreholes are inclined, the length down the borehole is greater than the depth below ground level.

There is still confusion around the data supplied for KC20-R15, however it does appear that a scan is available but has been omitted from the Appendix D detailed data.

5.3 Results

5.3.1 FE Modelling

The two cross sections have been modelled, orientated N-S and W-E, with results of surface deformation and roof beam displacement. The maximum roof beam displacement is recorded as 7mm below the road, which is correct but not quite the whole picture. There is an area where displacement is greater, and the roof beam is considerably thinner as shown in Figure 5-1 below. While it is difficult to read the scale from the presented cross section, it appears the roof beam is approximately 3m thick in this area, this equates to a roof beam deflection of approximately 0.3% which is still well below the trigger level of 2% but comparable with the maximum deflection recorded on the W-E section. However, it is noted that this area is a significant distance from the R179 and will not impact on road stability.

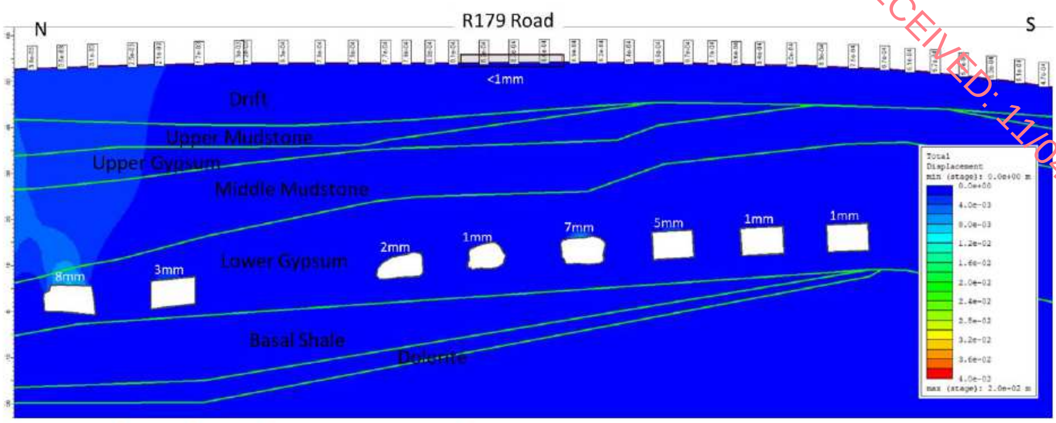


Figure 4-6: Cross Section 1a: Total Displacement

Figure 5-1: SRK 2020 cross section 1a through zone 1 showing 8mm displacement with a thin roof beam north of the R179.

The settlement results appear to be of the magnitude that would be expected and appear similar to the results previously undertaken and of our own assessment which has been undertaken on a simplified geological model derived solely from the closest boreholes, as shown in Figure 5-2 below. In addition to this, the WAI analysis has also investigated stress distribution which show localised stress concentrations at roof and wall junctions of the workings. Although these are as expected and do not show failures, they do appear to coincide with localised features observed in the 3D scans.

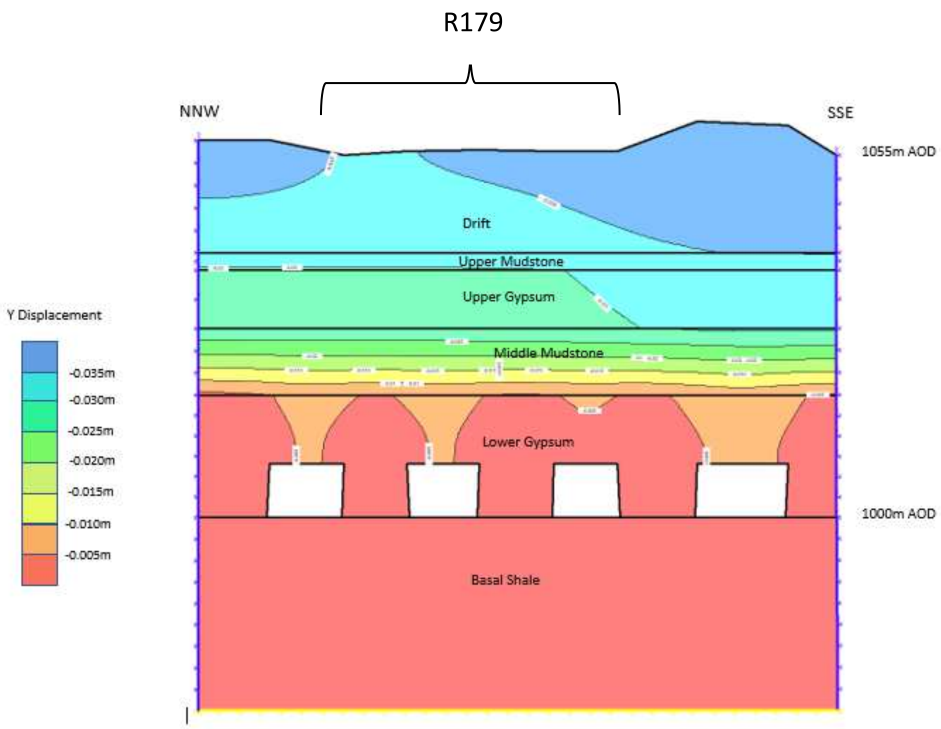


Figure 5-2: Extract of WA1 section line, Zone 1 showing Y displacement

5.3.2 Laser Scan

Four images are referenced within the SRK report, although 5 are presented. They are views from KC20-R01, KC19-R03, KC19-R04, KC19-R15 and KC19-H24. Additional views and orientations are included in Appendix D.

A clear roof beam failure has been identified in SRK image 4-8, reported to extend approximately 2.5m into the roof beam. There is visible material on the floor which suggests the roof failure extends higher than the SRK interpretation of the scan. However, the roof beam is recorded as approximately 10.5m thick in this area indicating that a significant thickness of roof beam remains. This failure is not predicted by the FE analyses undertaken by WAI or SRK, and hence suggests a variability in localised areas that the FE method of assessment cannot predict.

In addition to the roof beam failure at KC20-R01 small areas of slabbing are noted in other areas of Zone 1 but overall SRK consider the area to be stable. WA would concur with this assessment.

When a wider overview is taken of the area and the roof is considered across all the scans rather than as isolated locations it is possible to detect potential linear features in the roof strata. While they are currently not showing signs of significant instability these features should be reviewed in a holistic manner to ascertain their significance, if any, or if they are a structurally controlled feature which could impact on roof stability and degradation. The possible linear features are highlighted in Figure 5-3 below.

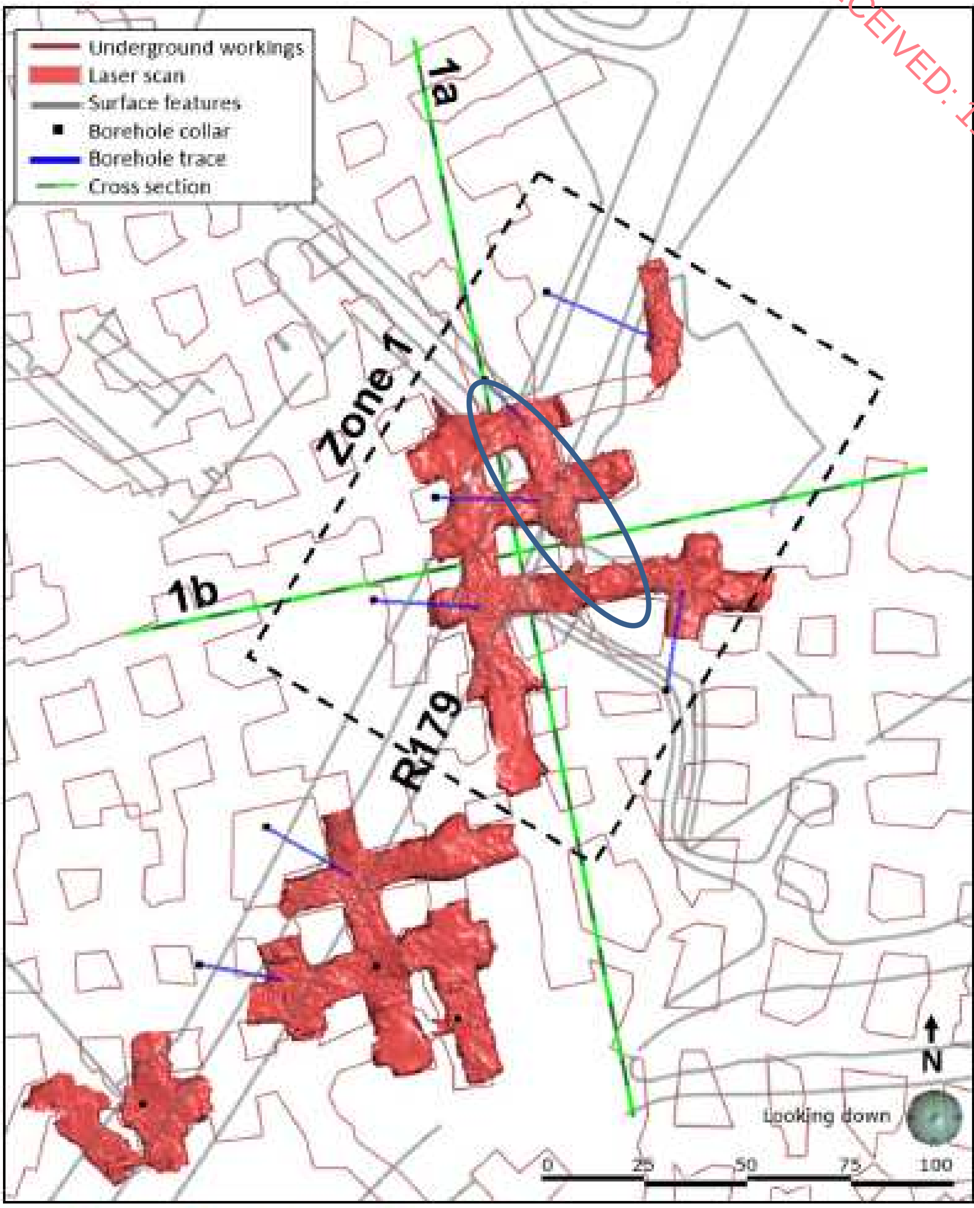


Figure 4-1: Plan view of Zone 1

Figure 5-3: Zone 1 scan survey overview with linear zone of roof anomalies highlighted

The SRK report does not provided an overall assessment of the potential risk or stability of Zone 1, but rather treats it as an isolated area without consideration of the receptor the R179. A standard risk assessment of likelihood, impact and consequence of occurring has not been completed as this was not part of SRK original brief. No reference has been made to the tolerance or stability requirement of the R179. The potential risk and likelihood of occurrence is not identified or assessed nor the potential impact on the R179 beyond the FE modelling which is not the controlling failure mechanism in this area, as shown by the onset of crown hole failure in the roof beam, and has also not been referenced to the settlement tolerances of the road. In addition, there has been an historic crownhole

DT97a within 25m of the R179 which is not commented on in the report. WAI considers this should have been addressed as it could be pertinent to the formation of crownholes around the R179.

WAI would summarise as follows: From the data presented it appears overall Zone 1 has an area of instability which is unrelated to pillar stability or flexing of the roof beam as assessed by FE modelling. While the majority of the roof and pillars appear stable there are signs of localised roof slabbing. There is on average 10m of roof beam above the workings, although the upper part of the Lower gypsum is of poorer quality, and it is therefore unlikely that the hole or roof instability will propagate into the overlying weak mudstones. The likelihood of such an occurrence is therefore very low. However, the area should continue to be monitored to confirm no further instability occurs.

6 ZONE 2

6.1 General Description of Undermining

The general description conforms with the plans and borehole logs. It should be noted that in addition to the 11m high room, Pillar R12 and Pillar R21, Zone 2 has some of the smallest pillars-largest room spans immediately to the west of the road, as shown in Figure 6-1. This area has not been covered by the borehole investigation or scanning but has been assessed as part of the FE modelling.

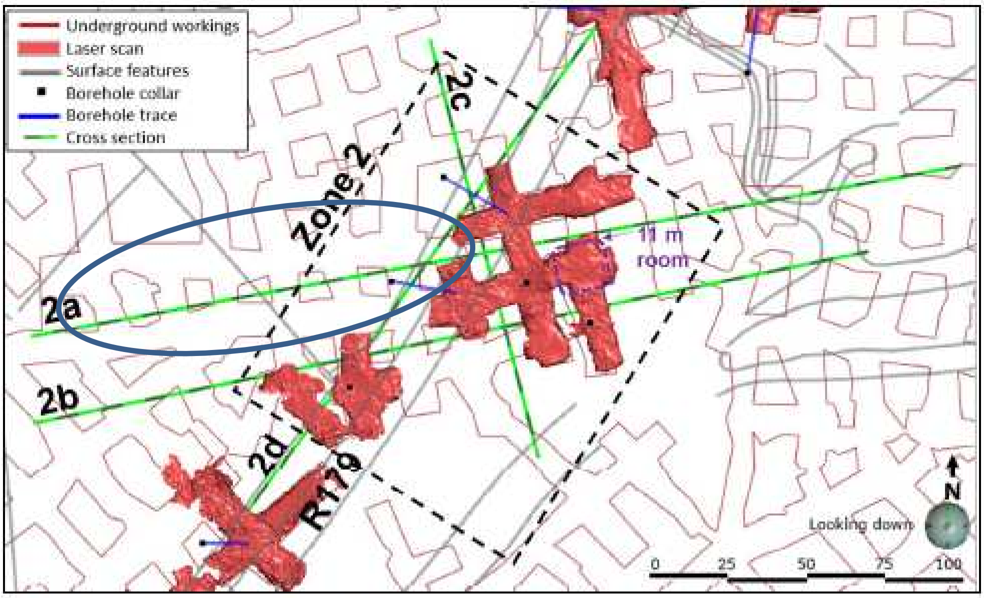


Figure 5-1: Plan view of Zone 2

Figure 6-1: Zone 2 with area of small pillars highlighted west of the R179

6.2 Summary of investigation works

WAI consider the summary correct, although as mentioned previously the summary tables record the borehole length and not depth as labelled. As reported, it would be more accurate to report depth rather than length.

Zone 2 comprises 5 boreholes, four of which have complete data sets. KC18k-R08 was drilled in an earlier campaign and only a scan survey is available. All other aspects are as reported.

6.3 Finite Element Modelling

6.3.1 Model Geometry

SRK do not report that section line 2b is nearly identical to the section line assessed in 2018 when Pillar R21 was first investigated. The alignment is slightly different but the majority of the same pillars and rooms are passed through.

From comparison of the 2018 section line loc 2 -R21, (Figure 6-2) and Section Line 2B 2020 (Figure 6-3) it is possible to see the refinement in the model with the additional data, the upper Gypsum seam is now considered to pinch out sooner, but that the models are not that dissimilar.

It should also be noted that previous modelling has been undertaken on Pillar R12 in 2018, an extract is shown in Figure 6-4. The cross-section orientation in 2018 was perpendicular to the R179 rather than the 2020 orientation which is oblique as shown in Figure 6-5. As such the mine layout differs considerably as different room and pillars are intercepted. Comparing the two cross sections highlights how pillars which may be of concern can appear to be further away from the R179 by projection of different cross sections.

The WAI simplified model, again based solely on the borehole data, indicates similar low magnitudes of deflection to those reported by SRK. Roof deflections are low indicating low strains within the roofs. An extract of WA2 Zone 2 is shown in Figure 6-6 below.

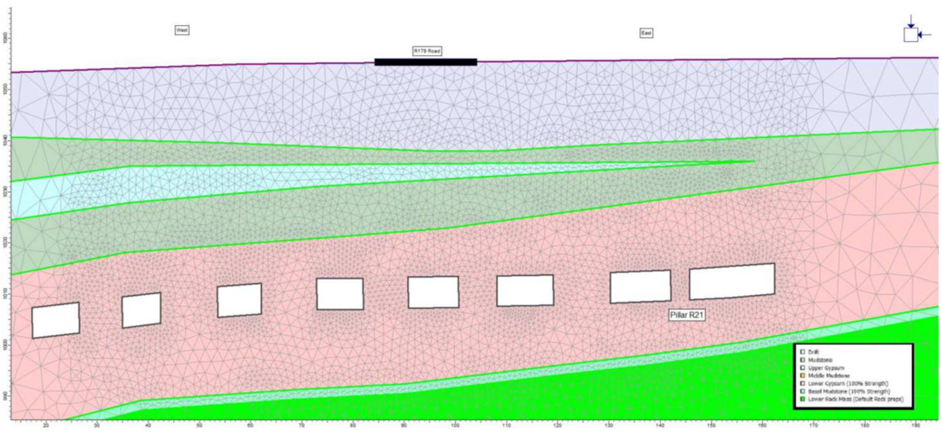


Figure 6-2: SRK 2018 assessment of R179 Location 2 cross section through Pillar R21

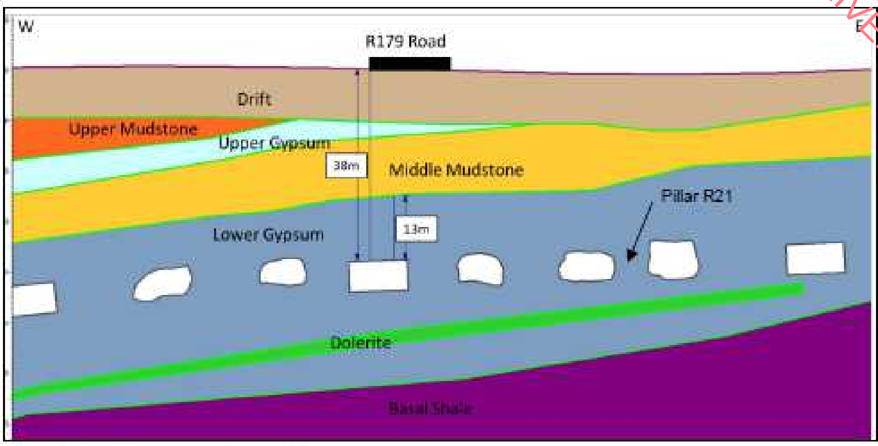


Figure 5-5: Cross Section 2b

Figure 6-3: SRK 2020 Cross section 2b through pillar R21

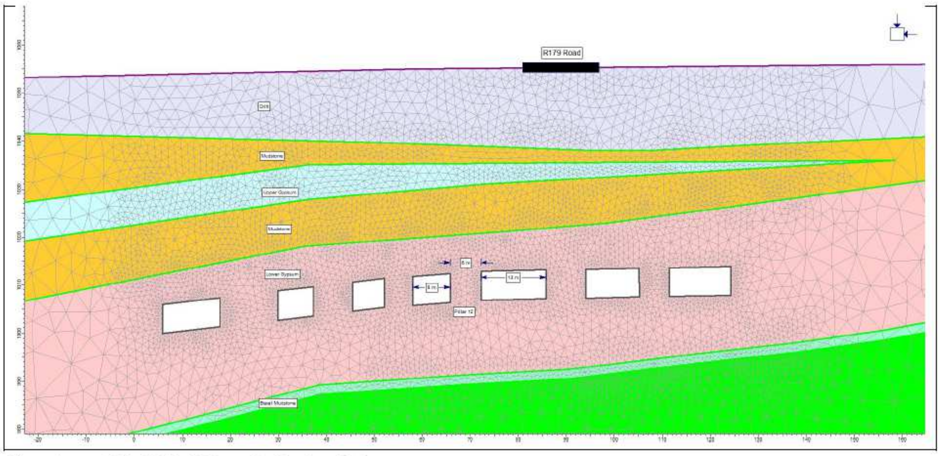


Figure 5: Original Pillar 12 Cross Section from 2D Survey

Figure 6-4: Extract from SRK External Memorandum ref 30238 Pillar 12 cloud scan December 2018

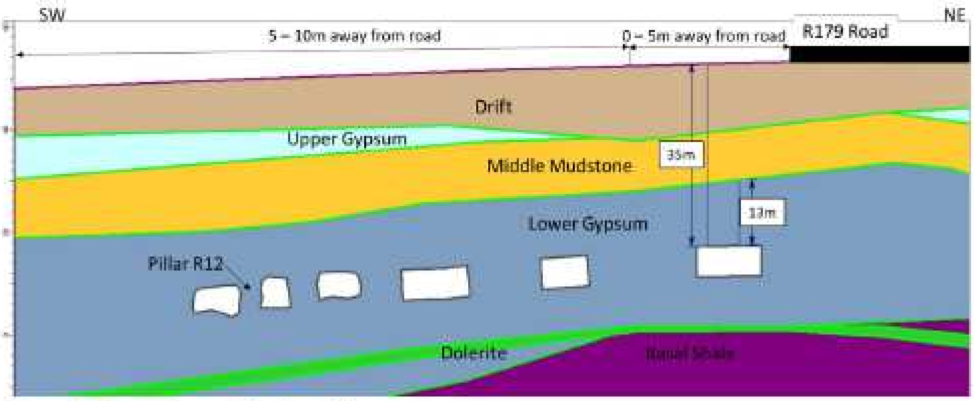


Figure 5-7: Cross Section 2d

Figure 6-5: Extract from SRK 2020 report, Section line 2d. Note cross section orientation is oblique to the R179 rather than perpendicular as assessed in 2018

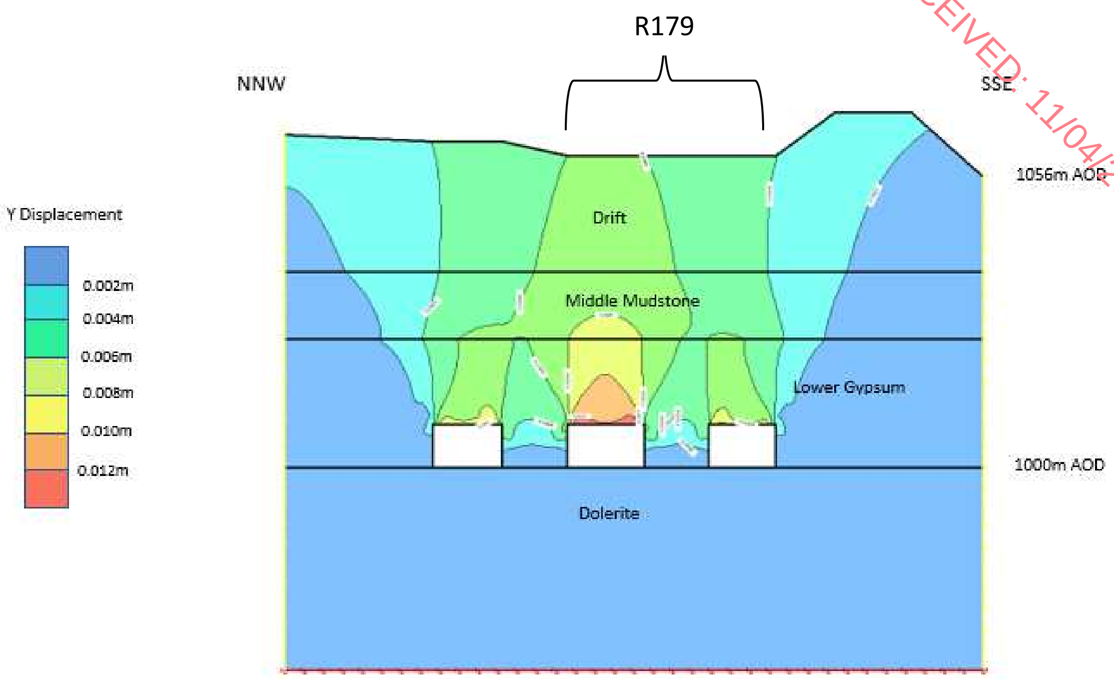


Figure 6-6: Extract from WA2, Zone 2 showing Y displacement

6.4 Laser Scan Survey

Generally, WAI concur with the findings of the SRK laser scan and agree that that the pile of debris below KC20-R06 seems too large to be solely from drilling and must include slabbing from the roof. However, Zone 2 appears to have only minor slabbing with the pillars and roof generally in good condition. As with Zone 1, SRK do not appear to have completed an overview assessment but have considered each scan area and view separately.

Looking at the area around KC19-R07 another possible linear feature is visible, some small scale degradation may have fallen from this feature, but it is not clear. Overall it does not appear to be impacting on the overall roof stability, however in the longer term, jointing or linear features have the potential to adversely impact on roof stability. The feature is shown in Figure 6-6 and is not considered to be a significant risk but rather builds up the overall picture of the mine and potential controlling features.

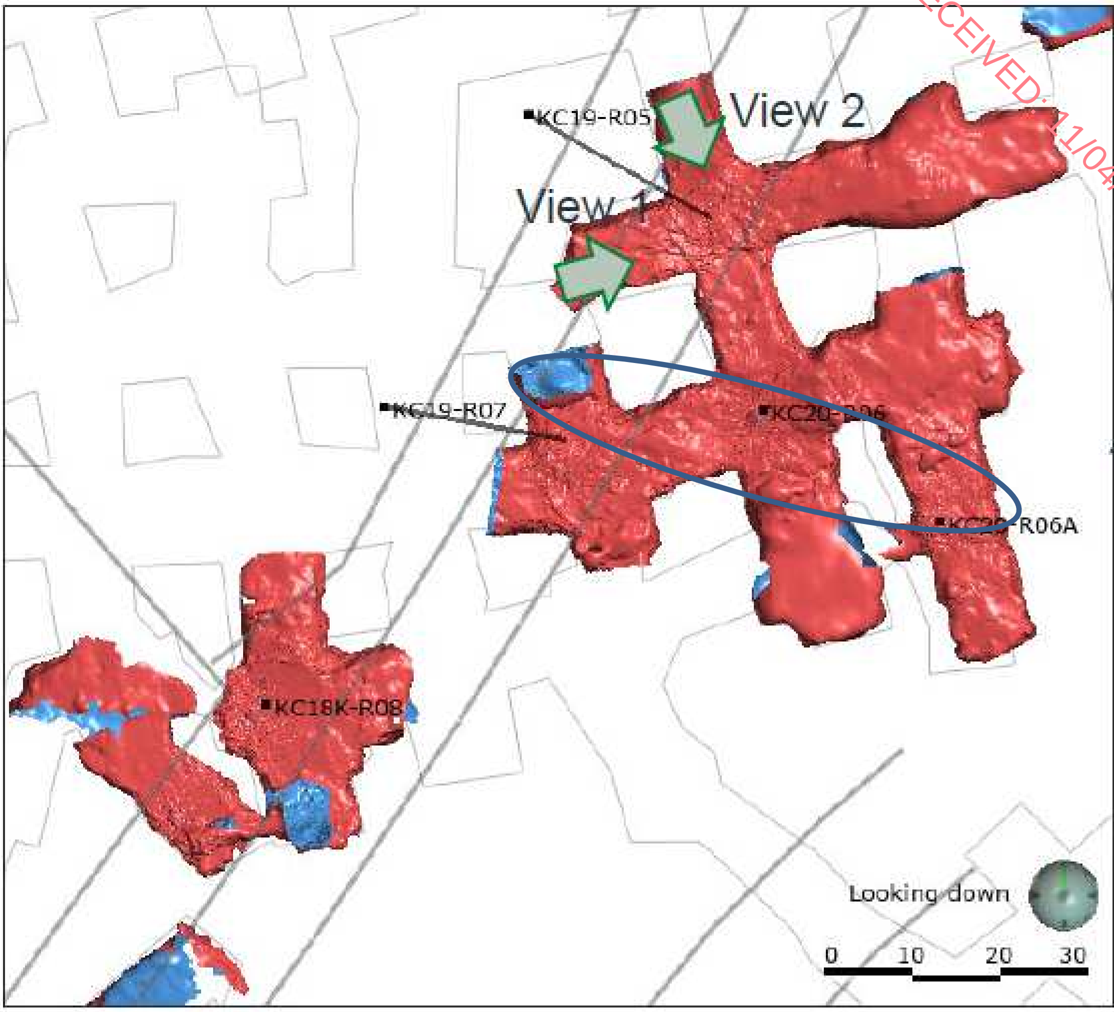


Figure 6-7: Zone 2 overview with possible linear feature identified

7 ZONE 3

7.1 General Description of Undermining

Zone 3 is the most uniform room and pillar arrangement. The general description accurately summarises the zone and identifies that the area has been previously flooded but is now dry. This transition from wet to dry has the potential to be the most destabilising conditions.

The gypsum seams are starting to dip more steeply at the southern end of the zone with the roof recorded at 43m below ground level.

7.2 Summary of Investigation Works

Zone 3 only comprises 3 boreholes but each borehole has a complete data set of geological logs, geotechnical logs, scan survey and laboratory testing.

7.3 Finite Element Modelling

7.3.1 Model Geometry

Zone 3 is the transition zone between the dry workings, workings which were flooded as a result of the water pumped from Drummond Mine and workings which are still flooded. The SRK report indicates that the maximum flood level, the historic water level and the current January 2020 water levels are shown on the cross sections however the January 2020 level has not been included in the Figures in the report.

The report description states that the roof beam below the road is 39m below ground level with a beam thickness of between 8m to 12m, however the cross sections record a beam thickness of between 5m and 8m which is supported by review of the closest boreholes (KC19-R11a and KC19-R11).

Section line 3a is one pillar south of the SRK section line Location 3 completed in 2018 R179 assessment, an extract is shown in Figure 7-1. It can be seen that the strata model has been updated and modified with the greater geotechnical investigation data to show the upper gypsum seam across the entire area. Also the surface topography has been modified to represent the open pit located immediately east of the R179 see Figure 7-2. This will generate a more accurate model of the ground conditions.

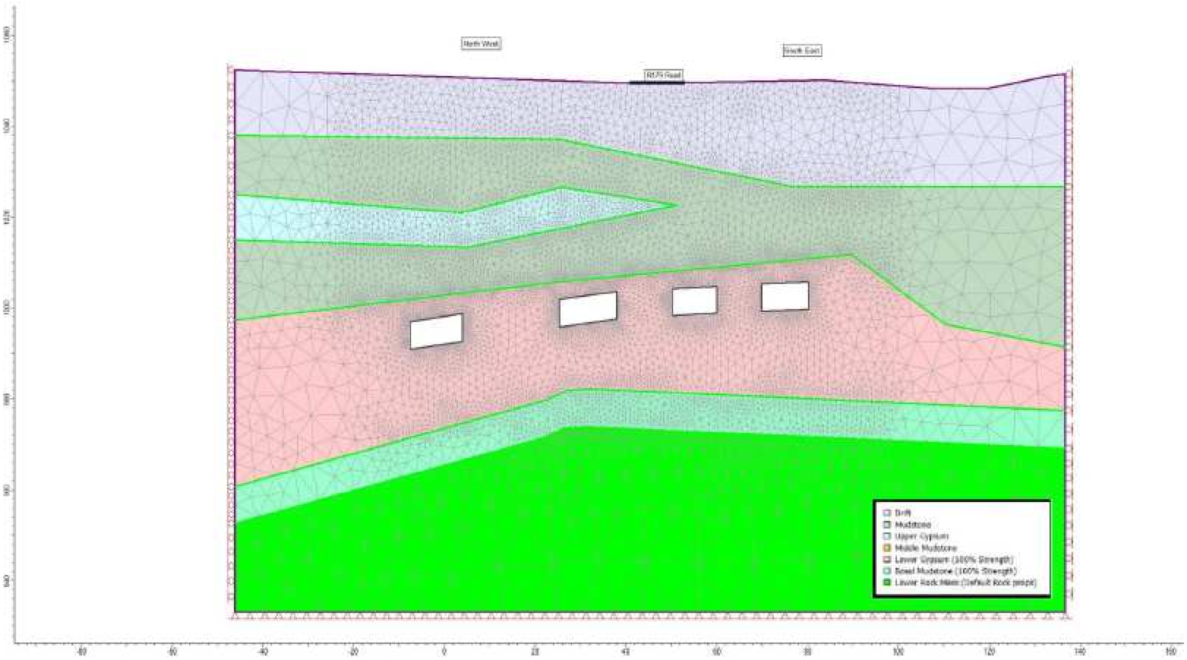


Figure 7-1: Extract from SRK 2018 R179 stability assessment report, Location 3 cross section

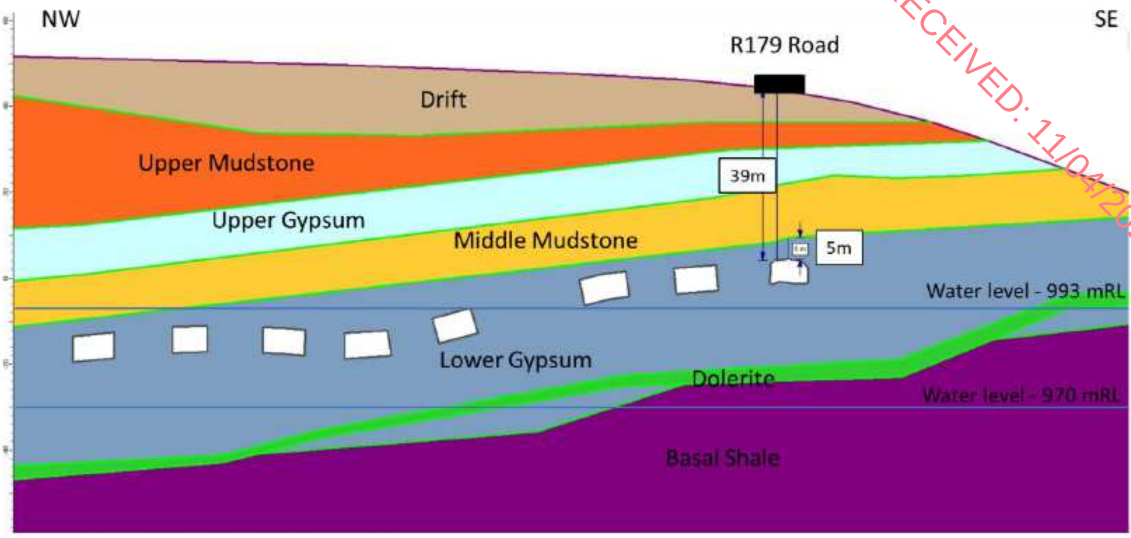


Figure 6-4: Cross Section 3a

Figure 7-2: Extract from SRK 2020 R179 stability report with cross section updated with latest ground investigation results

7.3.2 Results

The results appear acceptable.

7.4 Laser Scan Survey

The laser scan for Zone 3 has the most "holes" or missing data of any of the zones (see Figure 7-3). No explanation or rational is given for these data gaps. This leads to the potential that signs of instability may have been missed as entire roof sections have not been captured. Overall the general appearance of the roof appears to show the most slabbing and small scale degradation of any of the zones surveyed with considerable amounts of debris on the floor. This is as would be expected with a zone which has transitioned between flooded and dry. Borehole KC19-R10 is located within the area which has the most data gaps and debris, records a gypsum roof beam of 11.96m, which indicates that a significant thickness of roof beam is present even with the slabbing and deterioration observed. However, this area should be monitored to confirm no significant instability is occurring.

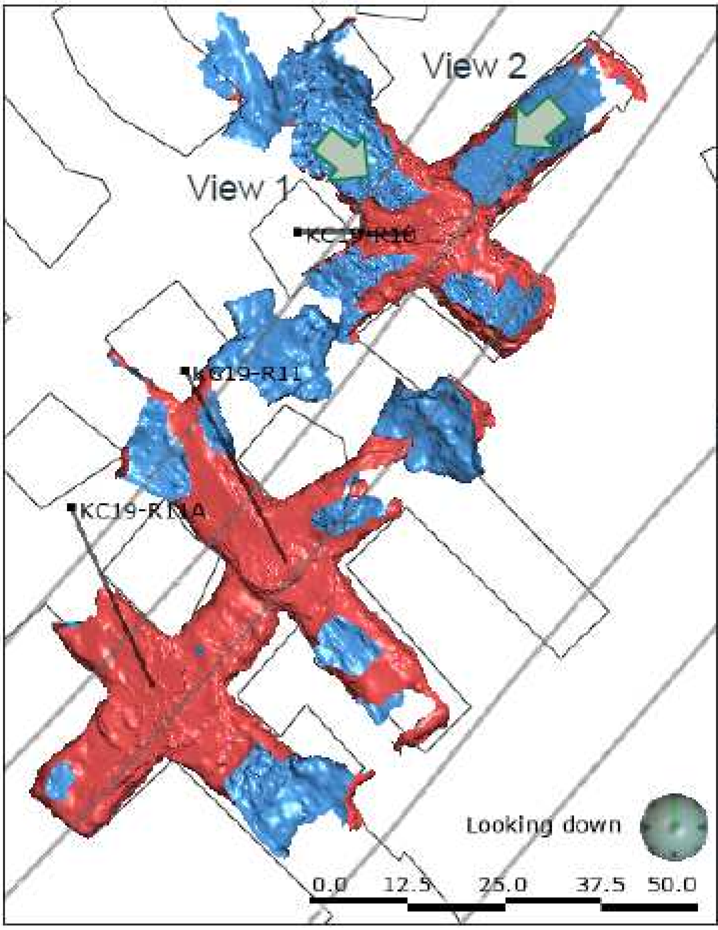


Figure 7-3: Extract from SRK 2020 Appendix D, Zone 3 overview

8 ZONE 4

8.1 General Description of Undermining

From the general description and summary of the investigation works it is not made clear that the lower seam has two layers of workings in this area. The extent of the workings which overlap are shown on WAI Drawing ZT611396-003. From this drawing it can be seen that the majority of the double layer working are located to the west of the R179 and only one limited area of double layer workings appear to be present below the R179.

8.2 Summary of Investigation Works

Although 5 boreholes have been drilled in this Zone, only one has a complete data set, all other boreholes are incomplete and scan surveys have not been undertaken in KC20-R16 and KC19-R12 due to airlocked workings being encountered and concerns of causing flooding within workings which are currently dry.

The borehole trace of KC20-R17 is missing from Figure 7-3 and should be added for completeness.

8.3 Finite Element Modelling

8.3.1 Model Geometry

Cross section 4a is one pillar south of the SRK 2018 R179 Location 4 assessment as shown in Figure 8-1. It can be seen that the model has been significantly updated with the additional geotechnical investigation data and now shows the double layer working Figure 8-2.

The model is reported to show three water levels, maximum, current and historical level. This is not the case as the current water level is not included. It is noted that the majority of the workings are below water level, however the upper workings intercepted by KC20-R12 and KC20-R16 encountered airlocked workings.

The model description states that no double layer works are beneath the R179, however the room immediately north of section line 4a appears to have a small area of double layer workings with a void directly below the R179, see WAI Drawing ZT611396-003.

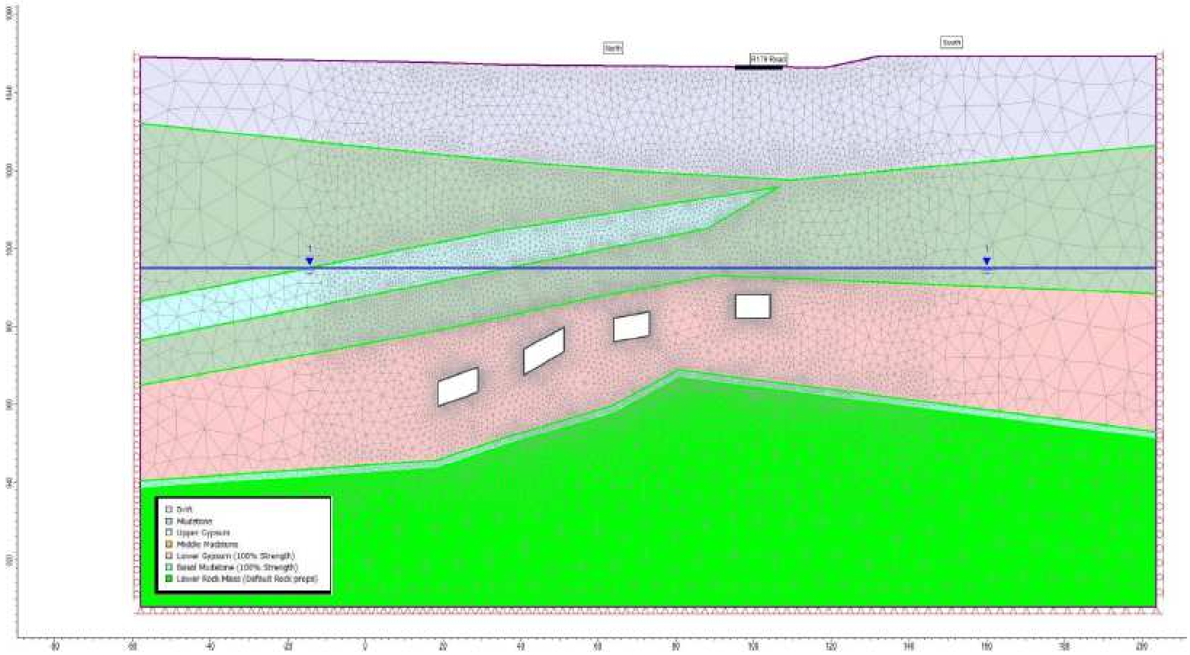


Figure 8-1: SRK 2018 R179 Location 4 cross section

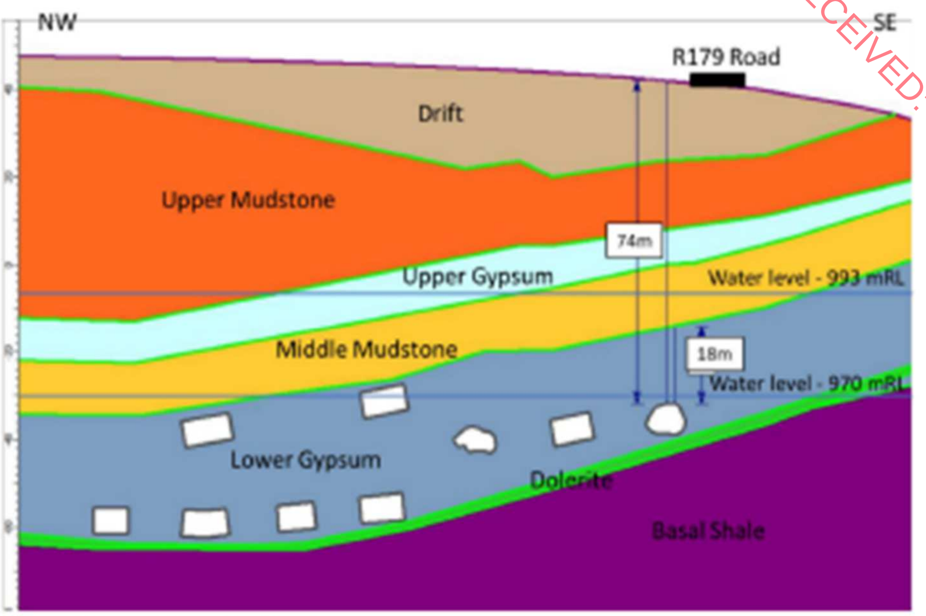


Figure 7-4: Cross Section 4a

Figure 8-2: SRK2020 R179 Location 4a cross section with updated ground and mining model

8.3.2 Results

The greatest roof beam deflection is recorded in cross section 4a at 1.4%. This is still below the trigger level of 2% and furthermore, the deflection is located to the NW of the R179. SRK do not indicate the risk or impact of the results on the R179, which is likely to be low as the workings are located to the NW of the R179.

The results of the WAI analysis suggest that there may be significantly more settlement and strain than suggested by SRK's model, though this may be due to the WA section intersecting the rooms diagonally, thus identifying the most onerous roof span. The maximum settlement predicted by WAI is 35mm, which would require a roof beam of at least 1.75m. Extracts from WAI modelling are shown in Figure 8-3 and Figure 8-4 below.

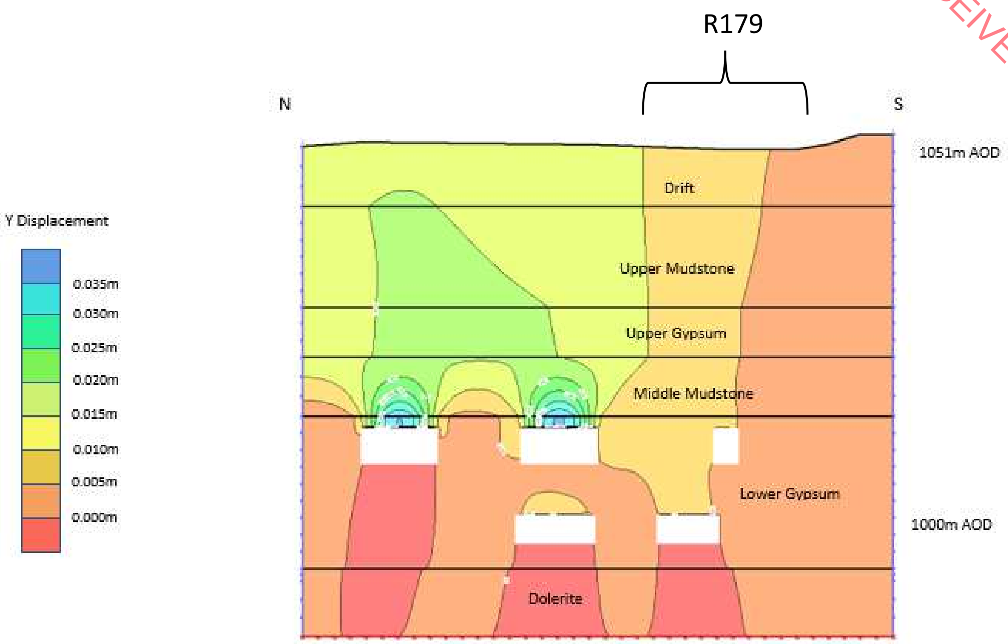


Figure 8-3: Extract from WA3, Zone 4 showing Y displacement (note cross section bisects road on the diagonal)

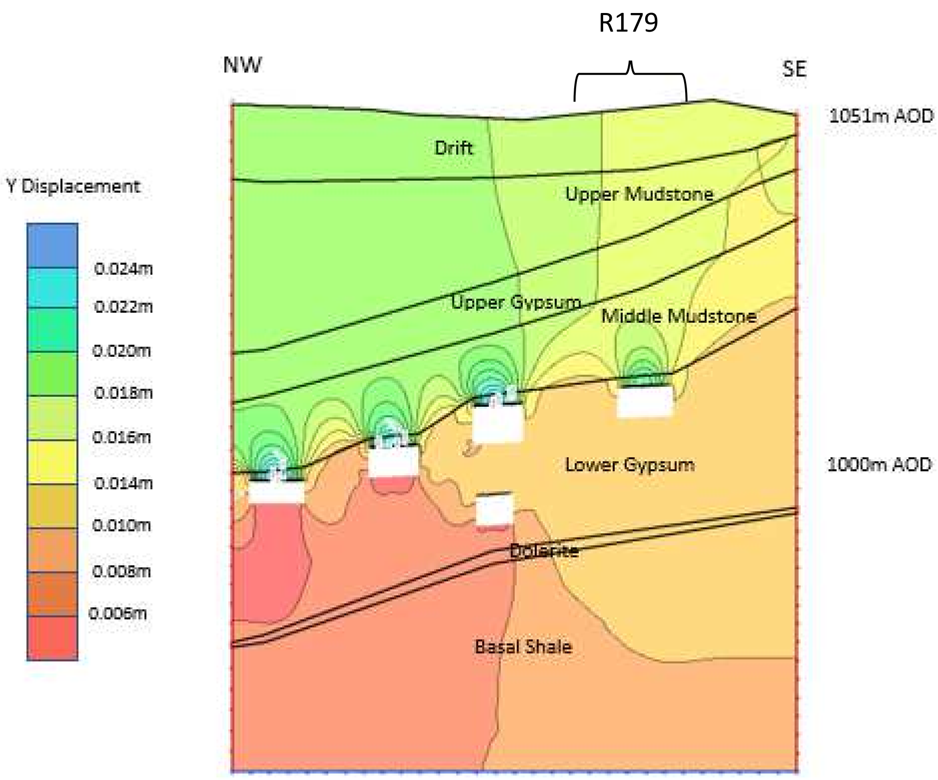


Figure 8-4: Extract from WA4, Zone 4 showing Y displacement (approximate position of R179 indicated)

8.4 Laser Scan Survey

No mention is made that the scan assessment was completed underwater, or how this may have impacted on the method of working and result interpretation, if at all. It is assumed that a sonar scan was completed but this should be confirmed.

One section of upper and lower workings have been scanned (KC20-0R17) and results are presented for both levels of workings.

With the exception of Zone 5 this is the smallest extent of workings scanned due to two of the boreholes being inaccessible.

SRK's assessment that the roof and pillars appear in stable good condition is supported by WAI's findings.

9 ZONE 5

9.1 General Description of Undermining

The description is accurate, Zone 5 has the least workings beneath the R179 and are the deepest and as such currently flooded.

9.2 Summary of Investigation works

Only one borehole was undertaken in Zone 5 and a complete data set is available.

9.3 Finite Element Modelling

9.3.1 Model Geometry

The model geometries and descriptions are accurate based on the data available although it should be noted that a dolerite seam appears to form the floor of some of the works. From previous studies it is known that this dolerite can be very weathered and weak and has the potential to influence the stress regime around the rooms and pillar boundaries. SRK do not make any reference to the possibility of the weaker zone within the workings.

9.3.2 Results

The results appear valid for the present configuration, however it is not known if any adjustment in rock quality has been applied for the strata which has been submerged for a significant period of time.

9.4 Laser Scan Surveys

Due to the workings being flooded the images obtained are of a lower resolution and quality. No reference is made to this in the text or explanation on any changes in procedure and analysis to reflect the workings being flooded.

A possible wedge formation has been identified in the southwest wall, however no further assessment is made of this feature of the potential impact and likelihood of failure.

It should be noted that the exit point of KC19-R14 appears to have formed a hole within the roof strata which is larger than observed at other locations. The debris identified on the floor is attributed to material being pushed into the end, however the blocks appear angular which SRK have identified as being fallen blocks within other Zones where the image quality is better. However, overall the SRK conclusion that the roof and pillars are in good condition appears valid.

10 SRK CONCLUSIONS

SRK provide a detailed summary of the works undertaken, however they also introduce new aspects, such as the surface monitoring data and previous 2018 modelling, which has not been discussed in the main section of the report .

Based on SRK crownhole criteria for formation, a roof failure should not have occurred in Zone 1. SRK do not provide any rational explanation or mechanism of formation of the chimney hole observed in Zone 1. While it is unlikely due to the thickness of gypsum in this area that a crown hole will propagate to the surface, SRK have not adequately assessed the mechanism of failure for this feature. In addition, there is no mention of other historic crownhole occurrences in the zone. One DT97a being in close proximity to the road. WAI believes this should have been considered.

The crownhole development potential states that Zone 3 is dry as it lies above the level of the maximum flood. This is not correct as portions of the lowest workings are within the flood level as discussed in the text.

SRK only identify the potentially slightly increased risk of crownhole formation along the R179 in Zone 1 in the conclusions, however based on their criteria a crown hole should not have been possible. This indicates that the crown hole conditions as defined by SRK are missing an aspect of the mechanism of failure. As previously stated no mention of the conditions prevalent at crown hole DT97a are commented on.

SRK conclude that there have been no instances of mine induced instability along and adjacent to the R179. This is somewhat misleading as subsidence has been recorded in an area next to the R179 as shown on Figure 10-1 of SRK report. This interpretation depends on what width is considered “adjacent” to the R179.

SRK conclude that virtually no deterioration in the mine conditions since the excavations were created have been observed. Whilst no significant deterioration has been observed, there are multiple areas of slabbing and roof failures in Zones 1 and 3. However, WAI agree that the majority of the mine is showing only minor deterioration.

WAI concur with the view that the R179 continues to be safe to use with the continued monitoring. However further assessment of chimney and crown hole formation mechanisms in Zone 1 is required to fully understand the potential roof failure mechanisms and the wider scale stability.

11 SRK RECOMMENDATIONS

11.1 Surface Monitoring

The surface monitoring and TARP should have been discussed sooner in reference to the FE results but WAI do not disagree with the comments made.

As a general point it would be beneficial to show how the surface monitoring points relate to the stability assessment Zones 1 – 4.

11.2 Borehole Extensometers

WAI agree with the overall number and location of the extensometers provided that the SRK recommendation to install an extensometer in KC20-R10 is adopted. It is noted that the details of KC20-R10 is not included in Appendix E as the report states.

In addition, WAI recommend that extensometer and surface settlement monitoring is installed to target the chimney hole observed in Zone 1.

11.3 3D numerical modelling of Pillar R12

WAI agree that sufficient data has now been obtained to undertake 3D modelling of pillar R12 to validated and refine the 2D analysis. While this will provide greater confidence in this individual pillar's continued stability, it will not provide any further clarification on the roof beam failures identified in other zones or greater understanding of the failure mechanisms not related to beam deflection and pillar stresses. These failure mechanisms cannot be identified and modelled by FE analysis as they appear to be joint or discontinuity controlled.

12 REVIEW CONCLUSIONS

12.1 Conclusions

Overall SRK have presented the data in a way which has made it difficult for the reviewer to build up an accurate picture of the site conditions and available data. It is not easy to ascertain whether all borehole locations have a full data set of geological log, geotechnical log, scan survey and laboratory testing. SRK do not appear to have utilised previous drilling and investigation campaigns to build up a comprehensive data set. However, having sought clarifications, WAI consider that there is sufficient data to make a valid assessment and also note the difficulties of completing the site investigation works due to travel restrictions enforced internationally in March 2020.

The geological logging completed by Gyproc has improved since the 2018 campaign, although there have been some errors and misunderstanding of logging procedures. The core photography has greatly improved and is now of a good standard.

The geotechnical logging completed by SRK has followed international standards although details have not been provided on the logging codes followed and the RMR rating system has not been strictly followed but the logging is of a good standard. The rock mass rating is quote as being RMR_{89} when it is in fact RMR_{89}', this is a small but significant rating difference and should be reported as such, however, overall the impact is negligible on the results.

The overall rock mass rating and geotechnical parameters utilised in the assessment are on the conservative side and as such considered acceptable.

SRK could have provided full details on the FE modelling undertaken as it has been difficult to recreate the cross sections as shown without considerable extra information and data which has not been presented in the report. Several of the section lines have either been previously modelled or are adjacent to previously modelled sections from the 2018 R179 assessment.

The scan survey assessment focuses on individual areas and provides a detailed assessment, although WAI believes that some alternative explanations could be taken for some of the features identified.

SRK have produced a report, as commissioned, on the stability of the R179, and not provided an overview of the wider stability of the area. WAI believe this has potentially missed some large scale linear features which impact on stability longer term. Interpretation of the area as a whole would build up an understanding of the mine and the factors influencing stability.

The lack of scan data in Zone 3 and the possible implications on the understanding of the stability in this area are not fully addressed in the report.

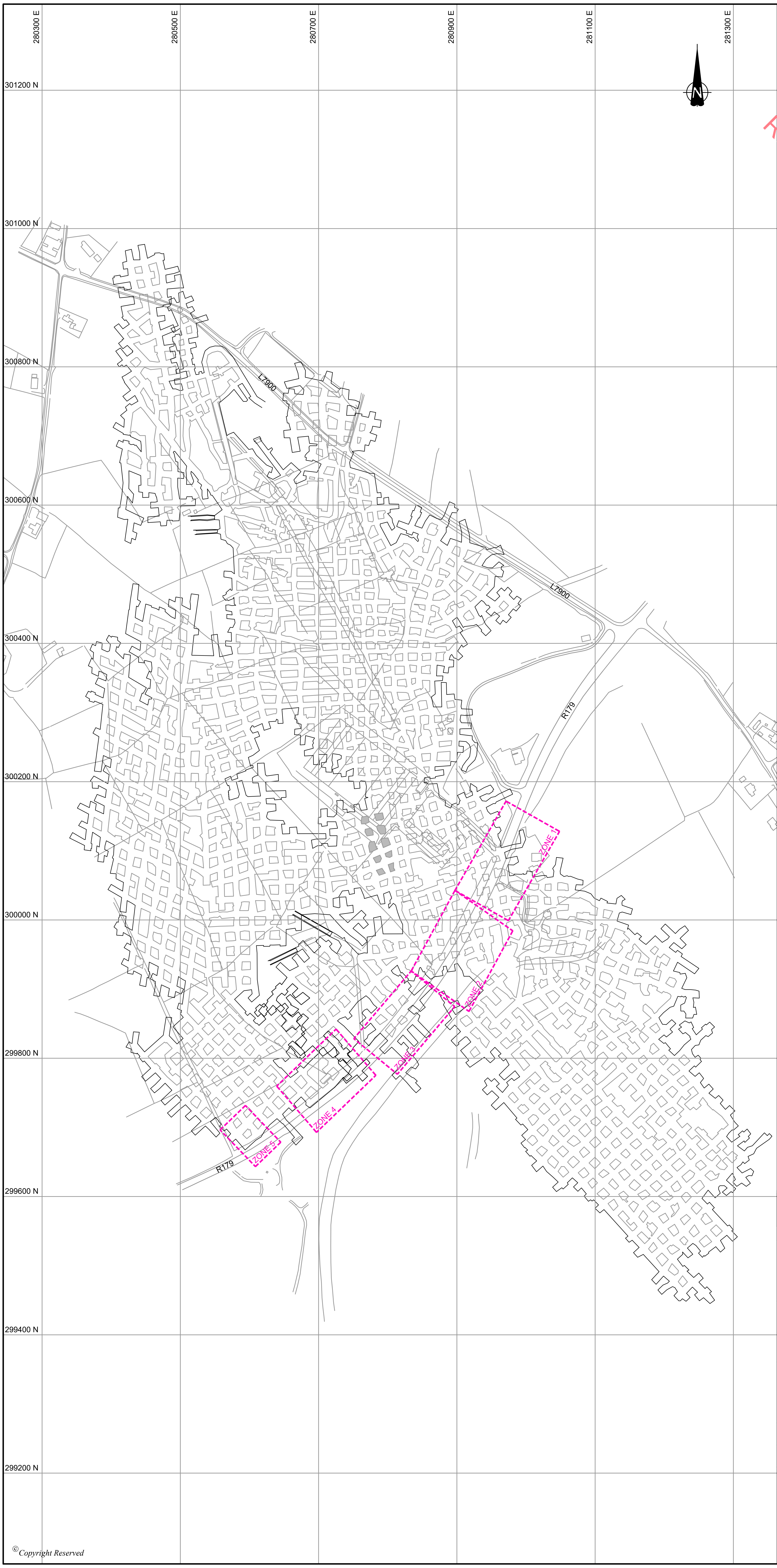
In addition, WAI considers Zone 1 should be reviewed further as there is historic evidence of a crownhole D97a surfacing close to the R179, the thickness of upper gypsum thins in the area, and there are geological anomalies within the area. Further analysis of the longer term stability of Zone 1 should be undertaken, particularly in relation to the parameters for crown hole formation determined in previous reports. This is not currently dealt with within the report.

The FE modelling results are broadly comparable to the results obtained in the WAI analysis, however it is considered that FE modelling demonstrating pillar and roof beam stability do not address all of the potential failure mechanisms observed at the mine such as chimney and crown hole formation and the report only briefly assess the potential risk and impact of these features.

WAI conclude that SRK's overall conclusion that the R179 continues to be safe to use is valid. The current monitoring scheme must be maintained and enhanced, if necessary, to ensure any changes to its safe condition are identified.

WAI agree with the suggested revised extensometer locations but would add that additional monitoring is required adjacent to and within the roof beam failure identified in Zone 1.

DRAWINGS



DO NOT SCALE FROM THIS DRAWING

KEY:

SKR ASSESSMENT ZONES

MINE PILLARS BASED ON DATA SUPPLIED BY GYPROC LTD

RECEIVED: 11/04/2023

REVISION	DETAILS	DATE	DRAWN	CHK'D	APP'D
A	FIRST ISSUE	09/07/20	LD	NL	AS

CLIENT

DCCAE

PROJECT

INDEPENDENT REVIEW OF THE DRUMGOOSAT STABILITY ASSESSMENT OF THE R179

DRAWING TITLE

SITE LAYOUT PLAN

DRG No.		REV
ZS11396-001		A
DRG SIZE A2	SCALE 1:4000	DATE JUN 20
DRAWN BY LD	CHECKED BY NL	APPROVED BY AS

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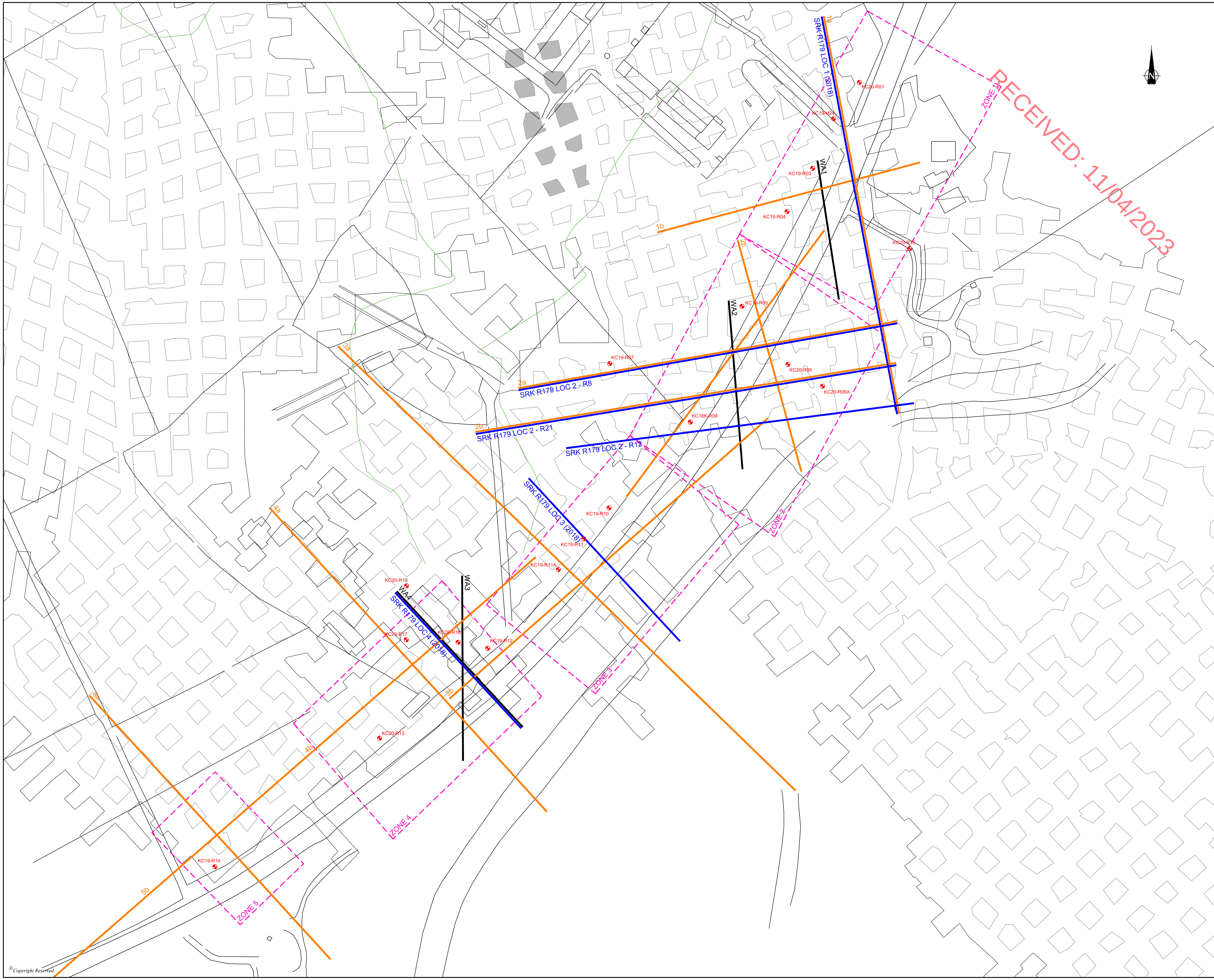

KEY:

- SRK 2018 R179
- SRK CROSS SECTION
- WA PROPOSED CROSS SECTION (1-4)
- SKR ASSESSMENT ZONES
- MINE PILLARS BASED ON DATA SUPPLIED BY GYPROC LTD

REVISION	DETAILS	DATE	DRAWN	CHK'D	APP'D
A	FIRST ISSUE	09/07/20	LD	NL	AS

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PROJECT

INDEPENDENT REVIEW OF THE DRUMGOOSAT STABILITY ASSESSMENT OF THE R179

DRAWING TITLE

ASSESSMENT ZONES AND CROSS SECTION LOCATIONS

DRG No.		REV
ZS11396-002		A
DRG SIZE: A1	SCALE: 1:1000	DATE: JUN 20
DRAWN BY: LD	CHECKED BY: NL	APPROVED BY: AS

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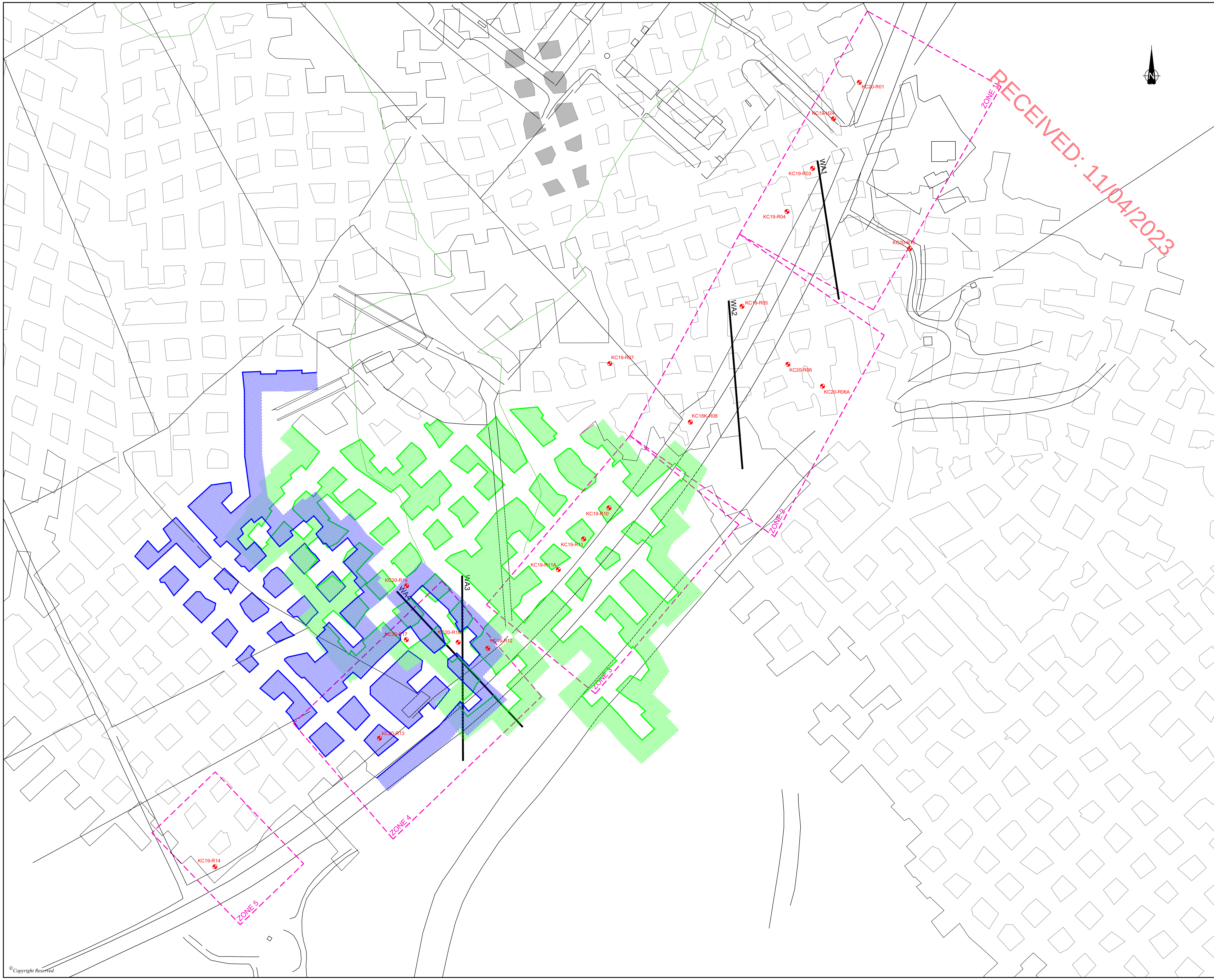

DO NOT SCALE FROM THIS DRAWING

KEY:

- SW LIMIT OF UPPER WORKING
- SW LIMIT OF LOWER WORKING
- SKR ASSESSMENT ZONES
- MINE PILLARS BASED ON DATA SUPPLIED BY GYPROC LTD

REVISION	DETAILS	DATE	DRAWN	CHK'D	APP'D
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INDEPENDENT REVIEW OF THE DRUMGOOSAT STABILITY ASSESSMENT OF THE R179

DRAWING TITLE

UNDERGROUND WORKINGS EXTENT ASSESSMENT

DRG No.		REV
ZS11396-003		A
DRG SIZE: A1	SCALE: 1:1000	DATE: JUN 20
DRAWN BY: LD	CHECKED BY: NL	APPROVED BY: AS

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APPENDIX 7.11
Review of Geotechnical Reports on Ground Stability related to the R179 and L4900 Roads overlying the Knocknacran West (Drumgoosat) Gypsum Deposit, Co. Monaghan - Golder - September 2021

GOLDER

TECHNICAL MEMORANDUM

RECEIVED: 11/04/2023

DATE 16 September 2021

Project No. 19121210 1023

TO Golder Associates Ireland

CC

FROM Richard Beddoes

EMAIL rbeddoes@golder.com

REVIEW OF GEOTECHNICAL REPORTS ON GROUND STABILITY RELATED TO THE R179 AND L4900 ROADS OVERLYING THE KNOCKNACRAN WEST (DRUMGOOSAT) GYPSUM DEPOSIT, CO. MONAGHAN

1.0 INTRODUCTION

Golder Associates Ireland Ltd (Golder) is assisting Saint-Gobain Mining Ireland Ltd (SGMI) with design and permitting of the Knocknacran West Open-Cast Mine at its gypsum operations in Co. Monaghan. Historically there were underground operations on the same property, operating from the 1940s to 1989. These workings underlay the entire area planned for open-cast mining as well as public roads (R179 and L4900) passing through or adjacent to lands owned by SGMI. The long-term stability of the roads is of great importance to the project stakeholders.

Several studies relevant to the stability of the underground workings have been completed between the late 1990's and today, precipitated by several episodes of instability as well as a general concern for good governance on the part of SGMI and its predecessors as site operators. This memo provides a desktop review the relevant reports, summarizes their conclusions, presents independent testing of those conclusions and provides a discussion on their reasonableness. This memo also describes the likely impacts of adjacent open-cast mining on the remaining underground workings - no independent investigations or inspection of the site has been conducted for this memo and the accuracy of data and calculations using data provided in previous studies must therefore be assumed.

The primary sources relied upon for this review are reports by SRK Consulting (UK) Ltd[1,2,3,4] and Wardell Armstrong LLP[5]. Other relevant documents are authored by Atkins[6], Arup and Golder[7]. References to relevant documents are provided at the end of this memo.

2.0 HISTORY OF MINING AND MINE INSTABILITY

The Drumgoosat Mine near Knocknacran began around the 1940s with underground mining in one of two gypsum seams that underlies glacial drift, mudstone, and weathered dolerite sills at a depth of approximately 20 m to 100 m below ground level (bgl). The geological history of the area is described in the documents referenced at the end of this memo. Although the majority of mining was conducted in the thicker lower gypsum seam, the upper gypsum was also mined extensively. The thickness of each seam varies substantially, where the lower seam is typically 20 m in thickness and the upper seam around half that, with local variability caused by variation in topography as the gypsum was formed and subsequent later dissolution by groundwater. These upper and lower gypsum seams are identified in a drawing prepared by Erkina Surveys[8] and shown in Figure 1.

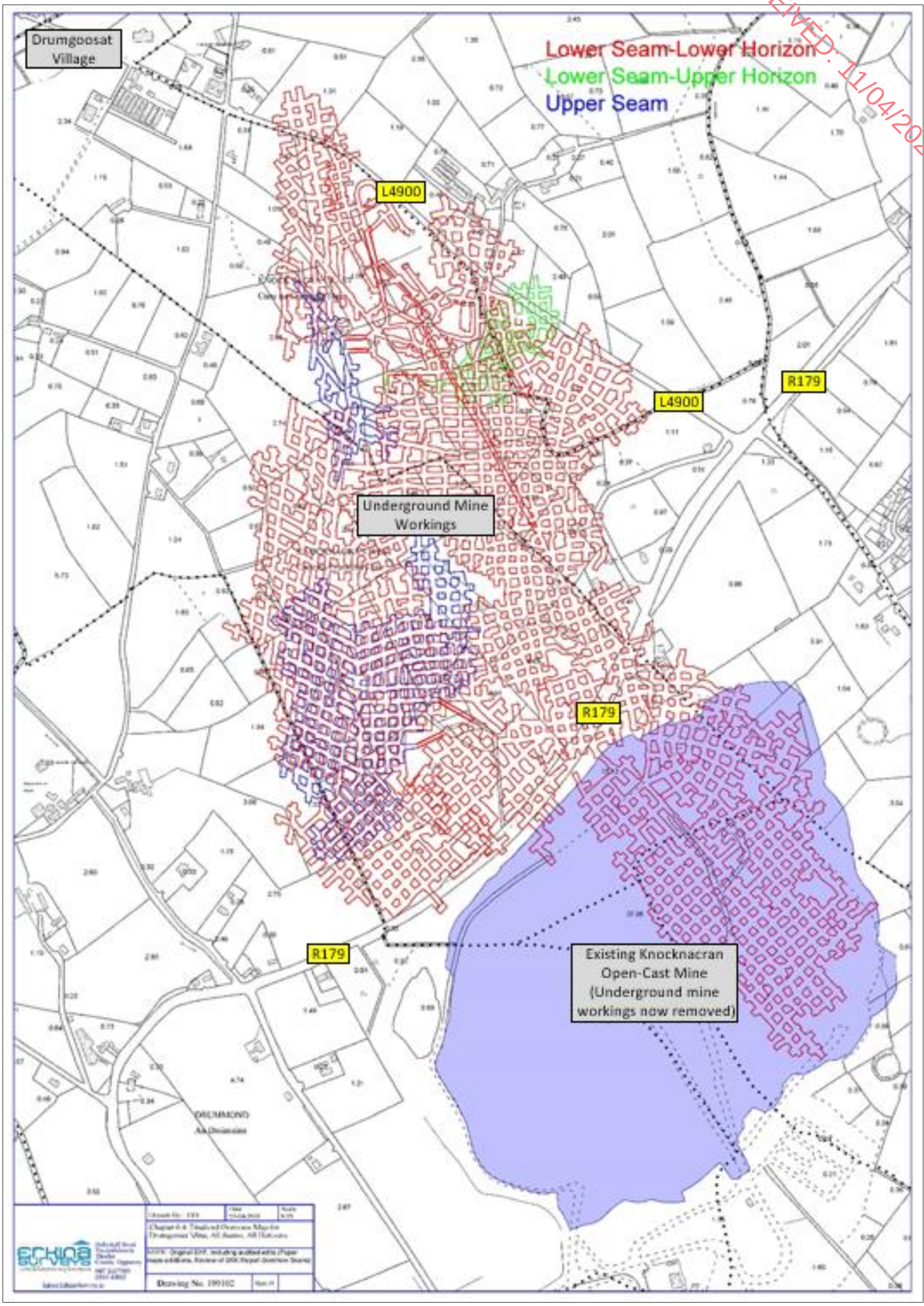

Figure 1: Mining in Upper and Lower Gypsum Seams at Drumgoosat Mine (Drawing Prepared by Erkina Surveys[8])

Mining was undertaken using the room-and-pillar method and, in approximate terms, pillars were 12 m square, and rooms were 10 m wide and 6 m high. Actual dimensions varied, partly due to the widely spaced orthogonal vertical jointing present in the gypsum which likely controlled the way in which the gypsum broke when blasted. This jointing is clearly visible in scans recently completed in boreholes by SRK[13]. In the majority of the mine, it has been understood, based on discussions between SRK and the mine operators as well as recent drilling investigations, that a substantial thickness (typically at least 3 m but often much thicker) of gypsum was left in the roof and floor of the mine workings.

Although mining was conducted with the intent of leaving gypsum in the roof and floor of the workings to control the potential for instability resulting from exposure of weak mudstone that forms the hangingwall and footwall of the gypsum seams, this did not always happen in practice. Sometimes mining came close to the overlying and underlying mudstone probably due to difficulties in predicting local variability of the geology. Additionally, it was identified in one area that mining in the lower gypsum was increased to a ca. 12 m room height. These local variations in mining practices are relevant to the incidents of instability that have occurred.

Surface deformations above mining operations similar to Drumgoosat can occur as a result of two primary reasons:

1. Pillars deform excessively because either:
    a. They do not have sufficient capacity to support the overlying rock strata without deforming and sometimes failing catastrophically (i.e. they are too small in plan, too high or their strength has decreased due to the effects of time or moisture such that they are unable to support the load applied to them); or
    b. The rocks below or above are very weak and allow the pillars to push into the roof or floor (sometimes called punching failure) or cause them to expand laterally, thus reducing their strength.
2. The roof fails because either:
    a. The remaining gypsum in the roof is insufficiently thick to support the very weak rock above (sometimes referred to as a beam failure); or
    b. A geological anomaly, typically a fault or dissolution channel caused by groundwater flow, is encountered unexpectedly. The existing Knocknacran open-cast mine has exposed features of this type.

Each of these mechanisms has historically occurred at Drumgoosat, with 1a creating surface collapse, deep subsidence cracks and extensive damage, 1b causing a broad gentle trough, and 2a or b causing small, discrete circular sinkholes (also referred to as crownholes). The various studies reviewed in this report aim to interpret the mechanism causing these observed surface deformations and predict whether similar events may occur in the future.

3.0 STUDIES OF STABILITY OF DRUMGOOSAT MINE

3.1 SRK 1999[1]

In 1999, SRK was commissioned by the Department of Marine and Natural Resources (DMNR) to report on observed subsidence above Drumgoosat Mine, in an area west of the L4900 access road to Drumgoosat Village, and provide an opinion on future instability above the mine as a whole. SRK was able to inspect parts of the underground mine. Several theoretical studies were undertaken to assess the capacity of pillars and the stability of roof beams in the mined rooms. Studies used relatively simple but common analytical or empirical methods, relying partially on earlier work by Dr. Barry Lehane at Trinity College[9,10]. These studies supported the interpretation that the observed subsidence was caused by pillars in the upper gypsum seam punching into a weak floor because the pillars were assessed to have sufficient strength to support the applied loads. Estimates were made of the Factor of Safety (capacity÷load) and reliability of pillars below public infrastructure, and from this work, it was concluded that there were a few pillars below the then existing Community Sports Centre, and the R179 and L4900 roadways that merited further study and regular monitoring. The report commented on risk factors due to groundwater inflow to and partial flooding of the mine, which was historically used for temporary mine water storage for the ongoing operations. Risk factors specifically included dissolution and softening of gypsum, loss of pillar dimensions due to erosion, and the possible inducement of sinkholes in locations where the gypsum roof beam was compromised or missing.

3.2 SRK 2002[2]

Further work on the assessment of pillar stability for the DMNR included additional underground inspections and surveys, geotechnical logging of new drill holes, rock testing, and additional stability assessments which included computer-based stress-deformation modelling. At this time, the existence of ca. 12 m high rooms and pillars in a small area below the Community Sports Centre was recognized and minor roof failures were noted. Underground workings were observed to be in good condition and showed no signs of instability with the exception of an area of floor heave around pillars to the east of the area with ca. 12 m high mining. A number of pillars in this area were identified as being at potential risk of instability, although numerical modelling was interpreted to indicate long-term stability. Below the R179 road, two pillars were estimated to be at risk of instability and regular monitoring was recommended. In general, the assessed risk of pillar instability increased compared to the 1999 work[1], in part because the unconfined compressive strength of samples recovered from drill holes was lower than assumed in the earlier work and the methods used to assess rock quality resulted in conservative values of rock mass strength. Again, a programme of regular monitoring and inspection was recommended.

3.3 SRK 2018[3]

In 2018, a large depression formed west of the Community Sports Centre with a series of open cracks forming circles in the field around it causing damage to the buildings on site. SRK was engaged by Saint-Gobain to investigate the subsidence event which was attributed to the failure of the area of the ca. 12 m high rooms and pillars in the lower gypsum mine workings. Extensive engineering analysis was undertaken to reach a conclusion that collapse would occur if the strength of gypsum in the pillars was reduced to 80% of its estimated lower bound value. That lower bound value was derived from drill hole core and historical underground mapping. It was also reported that prediction of failure required that gypsum in the floor below the pillars to be very thin. Simulation of the subsidence observed west of the L4900 (mentioned earlier) was also undertaken in order to estimate the further degraded strength of gypsum and mudstone that may cause such deformations. The rock strength parameters inferred from this work were then applied to simulations of several locations along the L4900 and R179 roads. These analyses concluded that the analysed sections would be stable if the strength of gypsum in the pillars was greater than 50% of the degraded value under dry conditions, or greater than 70% under flooded conditions. Further investigation to confirm the current condition of works was recommended.

3.4 Wardell Armstrong 2018[5]

Wardell Armstrong International (WAI) was retained by the Irish Department of Communications, Climate Action and Environment (DCCAE), to review the work of SRK. They considered the rock strength parameters reported by SRK to be conservative however, no further investigation was conducted. They also conducted their own numerical simulations using a similar approach to SRK but with a different software, and concluded that the area of ca. 12 m high pillars would have collapsed if joints in the gypsum were lubricated by water when the area was flooded. This scenario was interpreted to be the case during the summer of 2018 when the mine was being used for water storage as a result of limitations on water discharge from the nearby Drummond Mine. Like SRK, WAI analysed several critical sections below the L4900 and R179 and concluded that the pillars on these sections were stable.

WAI also subsequently reviewed SRK's 2020 work very thoroughly, providing confidence that the interpretations of drill hole data, excavation scans, geological models, etc. are sound.

3.5 SRK 2019[4]

SRK performed a further study in 2019 after discovery of a small crownhole, approximately 9 m diameter and 7 m deep, south of the L4900 and approximately 380 m from its junction with the R179. Extensive investigations were conducted along and adjacent to the L4900, including 25 drill holes and laser scanning of the old workings. This work produced a useful database of new strength tests on gypsum which can be used to update earlier work. The average strength was found to be around 30% higher than earlier testing. In general, all workings, except those in the known area of subsidence, were found to be in good condition with little evidence of deterioration since mining. The gypsum roof beam was found to be greater than 3 m thick everywhere except at the site of the new crownhole. At the point of failure, a local thinning of the gypsum, together with mobilization of overlying uncemented dolerite by inflowing water, had likely combined to form a crownhole that was deeper than the height of the mine workings. Taken as a whole, this work supported the conclusions of the earlier studies by confirming that workings are stable as previously predicted. Although other local roof failures could not be ruled out, the investigations indicated that this was very unlikely along the roadway.

This study was the first of several undertaken by SRK for Saint-Gobain related to the assessment, prediction and monitoring of stability of historical mine workings below public infrastructure.

3.6 SRK 2020[13]

Further work was recommended to confirm the stability of workings below the R179 roadway. An extensive drilling investigation which comprised of 17 geotechnically logged drill holes was conducted, from which cavity scans were conducted. This investigation provided a comprehensive view of the condition of the workings, demonstrating that only minor deterioration appeared to have occurred over the time since mining had ceased. The thickness of the gypsum roof was confirmed at all key intersections and showed no areas of concern. Stability analyses were completed, employing updated gypsum strength based on laboratory tests from the new holes, and recognizing locally significant mudstone interbedding in the gypsum. It was concluded that pillars are expected to remain stable and that there is a sufficient roof beam at all locations below the road. This provides confidence that intersection failure is very unlikely to lead to crown hole development. The work was intensively reviewed by WAI who performed independent stability assessments and concurred with all important conclusions while raising a number of questions and requesting clarification of a number of technical points. Subsequent further investigation was conducted along the R179 by SRK and reviewed by WAI (WAI[16], WAI[17]) leading to their acceptance of all significant technical conclusions regarding historic mine stability and the ongoing safe use of R179. An important outcome of the work was agreement among the various stakeholders on an appropriate monitoring plan.

Summary Comments on Previous Technical Reports

Work completed by SRK[1,2,3,4,13] and WAI[5,16,17] has been thorough and has used standard engineering approaches for investigation and analysis. Their approaches are very similar in instances where the two groups performed engineering analyses independently and their conclusions align. Recent investigations by SRK made extensive use of laser scanning of excavations from boreholes and this gave confidence that excavations below public infrastructure have deteriorated quite minimally since they were first mined and that analyses of their stability are justified in concluding that they are stable. In most major respects, then, the work by SRK appears sound and has been supported by WAI independent review.

In Golder's review of the work, two questions of data interpretation and technical approach arose. The significance of these questions was investigated by simple analyses in order to conclude whether or not they would impact any conclusions made by SRK and WAI. The first area relates to the derivation of rock mass strength parameters used for analysis as it is possible that these were overly conservative. Specifically, the value of Geological Strength Index (GSI) was derived from Rock Mass Rating (RMR) rather than being assessed simply on the basis of the number of joint sets and the joint condition in the rock mass, leading to conservative values for relatively weak rocks. There is nothing wrong with using conservative values to confirm the stability of existing mine workings, but it is possible that this leads to erroneous interpretation of the causes of failure events when used for back-analysis. In Golder's view (recognizing that this is a theoretical opinion without the benefit of a mine inspection), the GSI of a rock mass with three sets of joints with relatively fresh surfaces is likely to be higher than 40 to 52 as reported. Later work (SRK[13]) slightly modifies the values of GSI, again taking a somewhat conservative approach to incorporate the impact of mudstone interbedding within the gypsum below R179.

The second topic relates to the numerical modelling. Both SRK and Wardell Armstrong employ 2-dimensional numerical models to analyse a 3-dimensional situation. Thus, pillars that are square in plan are modelled as being infinitely long. Rooms are modelled as infinitely long tunnels rather than a network of rooms in two directions. The implication of this is that the load on pillars in the model is significantly less than its actual load This can be accounted for in various ways, but the reports do not explain how this was done. Graphical output is provided in terms of deformations rather than stresses, so it is not possible to confirm if the pillars are modelled under the expected stress. There are other more minor details of the analyses that also need to be known in order to confirm that the results are accurate. Rather than pursue a more detailed interrogation of the numerical modelling, a simple empirical analysis was undertaken independently to conclude on the reasonableness of the conclusions with respect to pillar stability.

4.0 INDEPENDENT ASSESSMENT OF PILLAR STABILITY

A very simple approach was taken to assess the likely stability of pillars below the public roads. This approach relies on an empirical equation, originally based on numerical analysis, which estimates the average principal stress difference at the mid-height of a pillar from its geometry and depth below surface (Equation 1[12]). This is compared to the rock mass unconfined compressive strength to estimate the state of stability (Equation 2[12]). The equations are shown below.

$$\sigma_q = \sigma_y \left(1 - \exp\left(-2.1 \times \frac{H^{0.6}}{W^{0.55}}\right)\right) \qquad \text{Equation } 1^{12}$$

$$Allowable\ \sigma_q \leq UCS\left(0.6 + \left(\frac{0.4}{3}\right)\left(\frac{W}{H} - 1\right)\right) \quad for\ 1 \leq \frac{W}{H} \leq 4 \qquad \text{Equation } 2^{12}$$

where:

σ_q = Average pillar deviatoric stress at mid-height (MPa)

σ_y = Average vertical stress on pillar (MPa)

UCS = Unconfined Compressive Strength

H = Height of pillar (m)

W = Width of pillar (m)

The rock mass unconfined compressive stress was estimated by back analysis of the pillar failure near the Community Sports Centre, then validated by comparing to the value estimated from laboratory UCS tests and GSI. This rock mass compressive strength was then compared to the predicted stress in groups of pillars below the roadways. Groups of pillars aggregated for analysis are shown in Figure 2. In each group, only the smaller pillars with approximately square shape along with their adjacent rooms were included in the averages shown in Table 1.

Table 1: Average Pillar and Room Dimensions by Pillar Group

Area	Room Width (m)	Cross Cut Width (m)	Pillar Length (m)	Pillar Width (m)	Pillar Height (m)	Depth from Surface (m)
A	9.5	9.2	13.0	11.3	6.0	60
B	9.9	10.7	11.8	10.9	6.0	45
C	9.7	9.8	11.2	9.7	6.0	42
D	9.0	9.2	11.4	8.2	6.0	40
E	9.9	9.5	15.4	12.5	6.0	50
F	10.5	10.1	11.9	10.7	6.0	21
G	8.9	8.9	9.7	8.3	6.0	45
Failed	8.9	8.6	11.0	9.3	12.0	65

From analysis of the failed area, in which pillars are assumed to have a factor of safety equal to 1.0, a rock mass UCS of 4,160 kPa was estimated. This is likely to be a conservative (low) strength because it does not consider the strength-reducing impact of a thin gypsum beam in the floor, which SRK concluded was also required for failure. Based on the UCS tests on core reported by SRK and assuming GSI=65, the rock mass global strength appropriate to pillar analysis is 4,500 kPa, somewhat higher than that estimated from the back analysis. The back-analysed value of 4,160 kPa was then used to estimate the factor of safety against failure for each of the pillar groups identified below the roadways. The groups of pillars assessed are shown in Figure 2 and their Factor of Safety ranges from 1.4 to 3.6, as summarized in Table 2.

Table 2: Empirical Pillar Load and Capacity Results

Area	Calculated Deviator Stress (kPa)	Allowable Deviator Stress (kPa)	Deviator Stress at Failure (kPa)	Factor of Safety Against Failure
A	2,969	2,985	4,160	1.4
B	2,598	2,948	4,160	1.6
C	2,560	2,837	4,160	1.6
D	2,545	2,699	4,160	1.6
E	2,229	3,096	4,160	1.9
F	1,225	2,930	4,160	3.4
G	2,995	2,708	4,160	1.4
Failed Pillars	4,160	<2,500	4,160	1.0

The outcome of this simple check analysis is that it is reasonable for SRK and WAI to conclude that pillars beneath the public roadways will remain stable in the future under the conditions they have assumed.

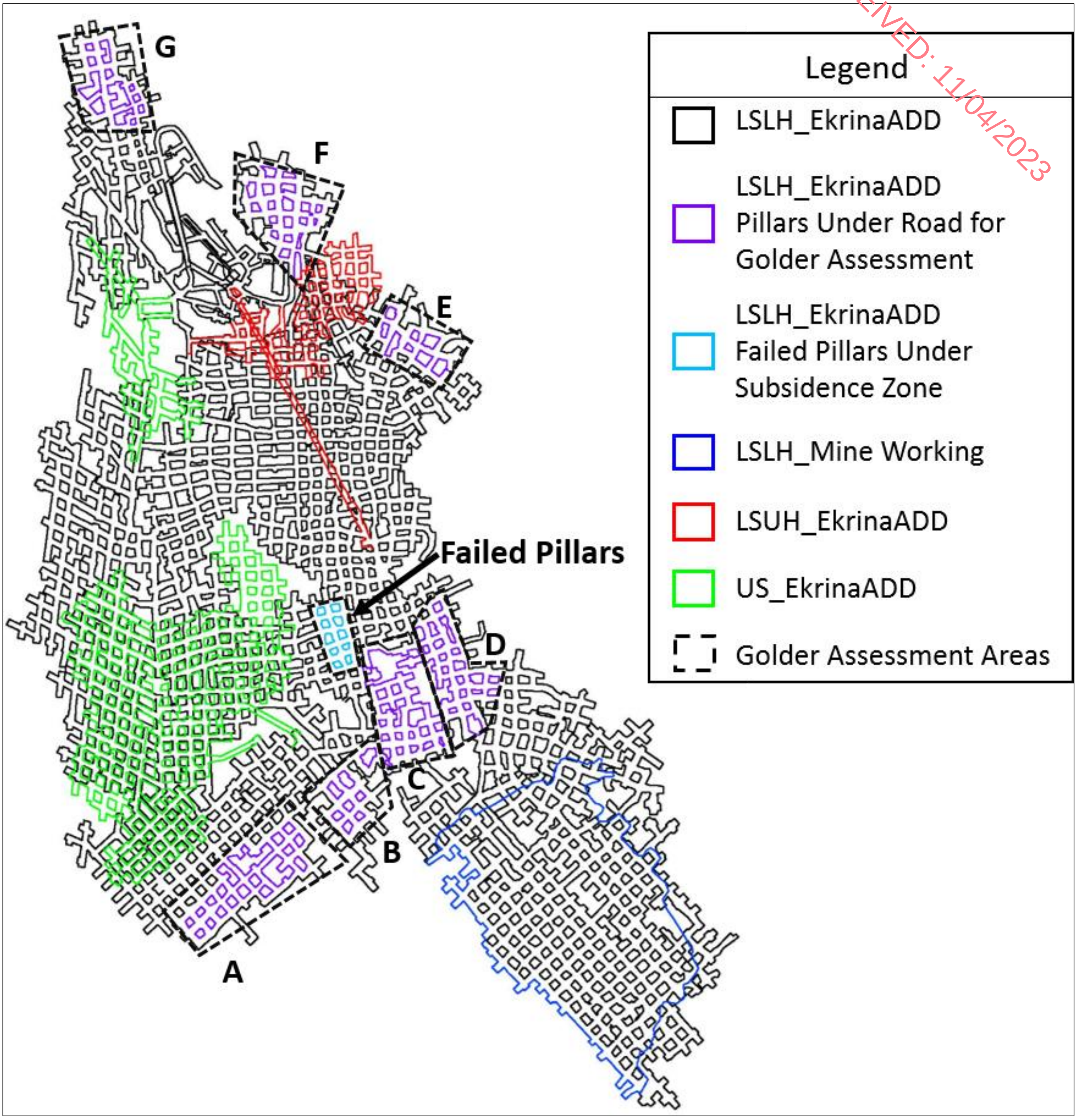

Figure 2: Groups of Pillars at Drumgoosat Mine used for Empirical Analyses

5.0 MONITORING PLANS

SRK presents monitoring plans for the Drumgoosat L4900 and R179 areas where historical workings will remain below public roadways (SRK[14], SRK[15]). Both plans employ a combination of surface level surveying and multi-point borehole extensometers (MPBX) with two anchor nodes installed in the roof of key intersections. The overall quantity of instrumentation and proposed anchor placement are appropriate. All intersections are currently considered stable based on cavity surveys, as well as WAI's detailed review. It is also mentioned that some existing boreholes will remain open to allow future cavity surveys along R179, which is appropriate.

The suggested monitoring frequency is considered to be very conservative and can perhaps be reduced with experience and following early data analysis. The trigger levels are a reasonable starting point but will need to be reviewed and updated as data is collected. It will be important to review rates of deformation (i.e., mm/time period) rather than just magnitudes in both the level surveys and MPBX data since this is often a better indicator of progress towards failure. TARPs could consider movements of individual anchor nodes, not only movements of the pair as appears to be suggested. As surface level surveying progresses and the potential to decrease monitoring frequency is considered, the use of InSAR satellite data may become a good option and provide more complete coverage at a reasonable cost. Overall though, the proposed TARPs are considered to be sound; they are suited to the level of risk and can be adjusted once they are implemented, as SRK recommends.

6.0 IMPACTS OF OPEN-CAST MINING AND WATER MANAGEMENT

Open-cast mining will occur adjacent to both the L4900 and R179. The mine slopes have been designed using standard approaches by Atkins[6] and Golder[7] employing guidelines in place at the time of design. The Knocknacran West Open-Cast Mine has a preliminary design by Golder[7] which uses standard engineering methods and achieves a factor of safety of 1.5 for overall slope instability. This factor of safety is appropriate and will be refined as information on site specific rock mass quality becomes available during planned mining operations.

Open-cast mining is not expected to impact the stability of the underground mine workings since this removes rather than adds load to the supporting pillars. The new Knocknacran West pit is expected to have no detrimental effects on underground mine stability. SRK[11] has recently conducted analyses to quantify these effects and confirmed that these will be negligible. Since we have confirmed the conclusions of earlier analyses through independent checks, and recognizing that no additional loads are applied by mining the pits, we consider SRK's opinion to be justified.

Water management in the underground workings is important to their stability. SRK[11] commented on this is their recent report. The impacts of water infiltration can include minor weakening of the rock mass, changes in stress conditions and dissolution of soluble gypsum. While water which is already saturated with respect to gypsum may have only minor impacts, the actual chemistry of groundwater after mine closure can vary and SRK highlights this uncertainty. While flooding with gypsum saturated water at closure is considered by SRK to be acceptable they note the importance of managing fresh surface water infiltration during mine operation. As part of the water management plan for the proposed mine, water will not be allowed to flow or pond around pillars below the roads during operation and diligent efforts will be made to manage infiltration from perimeter ditches around the pit crests.

At the time of site closure, it will be important to assess the condition of pillars, predict their stability under flooded conditions following closure and assess the feasibility of maintaining low flow, saturated conditions. Backfilling of excavations below the roadway may be advisable at closure and this can be assessed during operations.

A Cut-and-Cover Tunnel is planned below the R179 to link the new pit to existing mine infrastructure south of the road. SRK[11] found the impact of the tunnel on the stability of the underground mine workings would be negligible and, based on the planned location and geometry, their conclusion is very reasonable.

7.0 CONCLUSIONS

Predictions of underground mine stability below public roads (R179 and L4900) adjacent to the existing Knocknacran Open-Cast Mine and the proposed Knocknacran West Open-Cast Mine have been undertaken by SRK[1,2,3,4,11,13] and independently reviewed by Wardell Armstrong (WAI)[5,16,17]. Golder's review of their work concludes that their findings are reasonable. Where analytical methods could not be fully verified based on information presented in the available reports, independent checks confirmed the reasonableness of their conclusions. Predictions of mine roof stability are validated by cavity laser surveys showing minimal change in profile over many years. SRK, in various reports, recommends a programme of monitoring to identify symptoms of any change in stability of the mine workings and has presented TARPs for the two roadways (R179 and L4900). A regular monitoring program of this type is considered to be appropriate to manage the minor risk associated with the current and anticipated conditions. Maintaining the workings below the roads in a dewatered condition during future mining is considered to be prudent and the condition of the pillars and underground road intersection roofs should be reassessed prior to site remediation and mine closure.

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Principal

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APPENDIX 7.12
Knocknacran West Pit Slope Stability Assessment – Golder - September 2019

REPORT

Knocknacran West
Pit Slope Stability Assessment

Submitted to:

Saint-Gobain Mining (Ireland) Limited
Mine Office
Knocknacran
Magheracloone
Co. Monaghan
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Distribution List

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1.0 INTRODUCTION

Golder Associated Ltd (Golder) was commissioned by Saint-Gobain Mining (Ireland) Ltd (SGMI) to undertake a geotechnical assessment for the proposed Knocknacran West Mine (open-cast / pit) (the 'Site').

2.0 SITE DESCRIPTION

The existing Knocknacran Pit is a large open-cast gypsum mine located some ca. 7 km from Carrickmacross, Co. Monaghan and ca. 7 km from Kingscourt, Co. Cavan. The setting is rural with surrounding land use being mainly agricultural with low density residential housing.

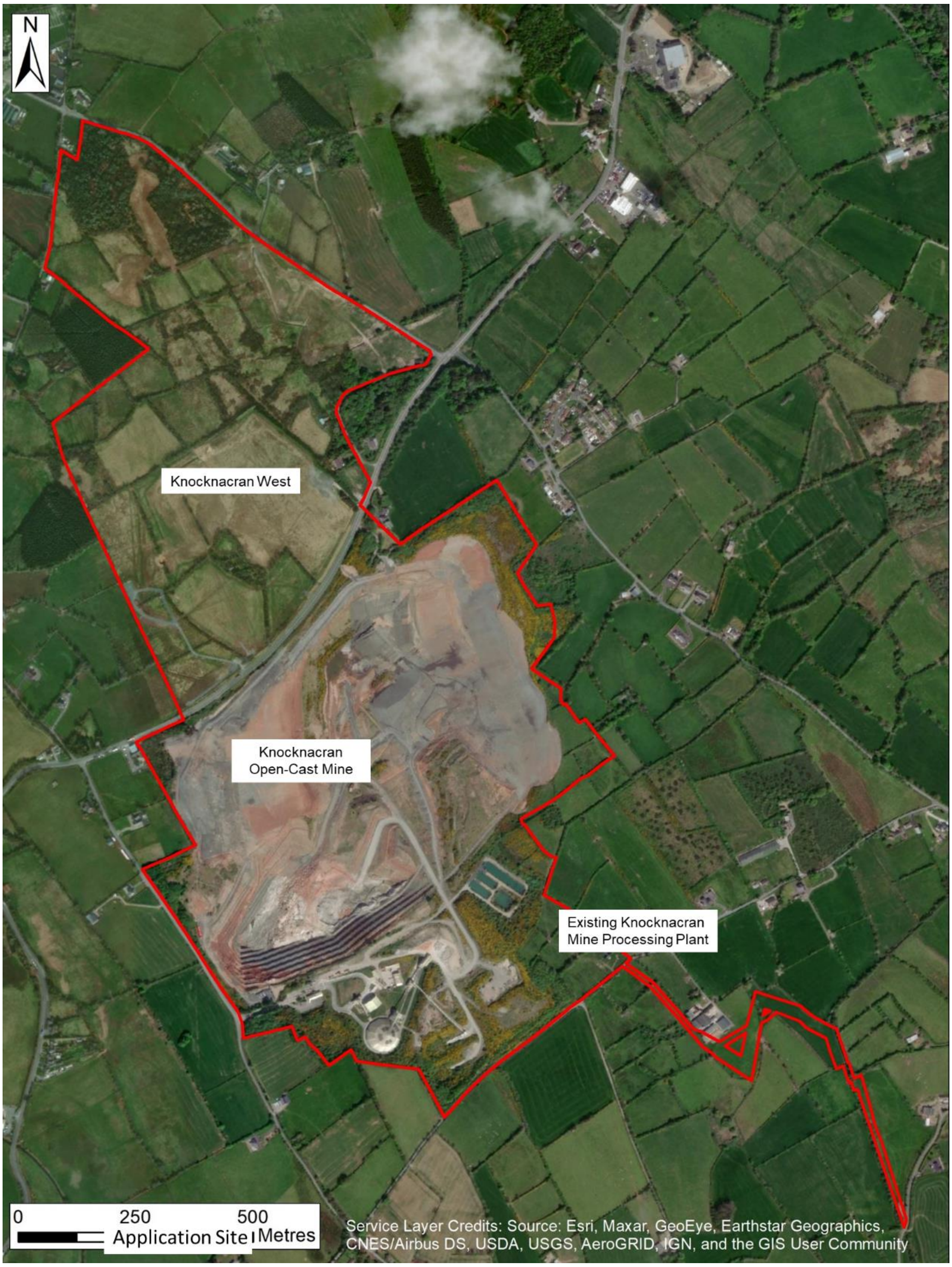

Figure 1: Location Map of the proposed Development

The Knocknacran West deposit is a continuation of the existing deposit currently being mined from the Knocknacran open-cast but is separated on the surface by a regional road (R179). The proposed development is located in the townlands of Knocknacran West and Magheracloone, Co. Monaghan. The Site is bounded to the south by the R179, a Regional Route which runs between Carrickmacross, Co. Monaghan and Kingscourt, Co. Cavan and to east by the L4900, a Local Route. Figure 1 provides the location of the Proposed Development.

3.0 PROPOSED DEVELOPMENT

The old workings at the former Drumgoosat Underground Mine (closed in 1989) exist under the proposed Knocknacran West Open-Cast Mine. SGMI proposes to extract the remaining pillars, overlying roof beams, underlying floor beams and previously un-mined areas from both the Upper and the Lower Gypsum Units using open pit mining methods.

The proposed area for open pit extraction at Knocknacran West is ca. 54.3 ha, with a maximum depth of extraction of ca. 80 m from the current ground elevation to the base of the Lower Gypsum Unit.

It is proposed to continue to use the existing processing facility on the existing Knocknacran Mine site for the processing of the extracted gypsum from Knocknacran West Mine. Transport of the extracted gypsum will be via a proposed Cut-and-Cover Tunnel constructed under the main Carrickmacross to Kingscourt regional road (R179).

The gypsum will be transported by a combination of haulage truck and covered conveyor, depending on operational demands. The tunnel will also be used for the transport of overburden and interburden (by dump truck) to the current Knocknacran open-cast mine for use in restoration.

4.0 BACKGROUND

4.1 Previous Studies

A summary of previous relevant reports produced on the Knocknacran deposit are provided in Table 1. This Table is followed by a description of the design changes to the Knocknacran pit since Geoffrey Walton's initial design.

Table 1: Summary of previous relevant reports

Works	Author
Original Design of the Knocknacran Pit	(Geoffrey Walton, 1982)
Geotechnical Assessment Knocknacran Open Pit Mine Ireland	(Golder, 2003)
Design of Knocknacran Pit extension to northern boundary.	(Atkins, 2006)
Design of Knocknacran Pit extension to south-eastern boundary	(Golder, 2017)
Drumgoosat Subsidence event – Technical report	(SRK, 2018)
Investigation of the collapse of working at Drumgoosat – An independent review of the works completed by SRK	(Wardell Armstrong, 2018)
Knocknacran Open Pit Geotechnical Assessment	(Golder, 2019)

4.2 Existing Knocknacran Open-Cast Mine

Geoffrey Walton Consulting Mining and Engineering Geologists (Geoffrey Walton, 1982) completed the original design for the current Knocknacran Pit. A comprehensive site investigation and laboratory testing programme was undertaken to characterise the overburden soils and geology. An interpretive geotechnical report was prepared, and design criteria and analyses are provided for the proposed open-cast mine.

Atkins Consulting Engineers (Atkins, 2006) completed a design to extend the Knocknacran pit to the northern site boundary. It is understood that Atkins conducted no additional laboratory testing of materials but based their assessment of material strengths on observations made during the construction supervision and the previous assessments. It is noted that the Knocknacran pit was developed with significantly steeper overall slopes that those recommended in the original design reports i.e., 1(H):1.5(V) versus 1(H):2.0(V).

The individual batter slopes excavated were as per the original design i.e., 1(H):0.5(V), although the individual bench widths were increased from 4.5 m to 6.0 m and the wider bench at every 12 m height interval was reduced from 13.5 m to 6.0 m (i.e., all benches were excavated at 6 m intervals in height, 6 m in width with batter slopes to 1(H):0.5(V)). The design appears to have been successful through the life of the Knocknacran pit for all overburden materials (mudstone and doleritic sands and gravels) and although the target factors of safety (FoS) are less than current standard criteria, the pit slopes only experienced localized bench failures 5 years after the north and north-east pit extension (2011-2012).

Golder submitted a planning application to extend the current pit to the south-western site boundary in early 2017 and additional investigation was conducted in the footprint of the proposed extension, comprising boreholes, sampling and laboratory testing. The interpreted data for the tested soil materials correlated with the previous assessments. It is understood that there have been no significant slope stability issues during the life of Knocknacran pit and the exposed north and north-east benches of the pit, comprising of overburden and mudstone, were observed by Golder during 2017-2018 to be in a stable, although deteriorating, condition since their excavation in 2011-2012.

The south-western extension was split into two development Phases (Phase 1 in 2017-18 and Phase 2 in 2019). The individual bench slopes and the overall slope were moderated for the Phase 1 and Phase 2 Pit designs. The bench slopes were reduced to 1(H):0.67(V) and the overall slope was reduced to 1(H):1.67(V) in order to achieve a minimum FoS of 1.3 for the overall slope and a minimum FoS of 1.1 for individual benches, for a worst-case scenario in which the bulk of the overburden to be excavated would comprise of mudstone.

The development of the Phase 1 Pit has subsequently shown the bulk of the excavation to be doleritic sands and gravels, which has improved geotechnical parameters than the mudstone and thus provides greater values for FoS for the overall slope and individual benches i.e., FoS > 1.5 for both (Golder, 2019).

4.3 Proposed Knocknacran West Open-Cast Mine

The Knocknacran West development is analogous in nature and in overburden material parameters to those present at the existing Knocknacran pit.

The design criteria for the factor of safety for slope stability of open pits has become more conservative since the original design of the Knocknacran pit, hence the gradient of the overall pit slope and the individual batter slopes will be lessened for the Knocknacran West pit.

5.0 SCOPE OF ASSESSMENT

This document presents the geotechnical analyses for the proposed Knocknacran West open-cast mine. The geotechnical parameters of the stratigraphy have been well established in previous reports (Table 1). Golder conducted additional ground investigations during 2018 and 2019 to confirm the stratigraphy and material parameters. Appendix A provides the borehole logs and photographs of the core from the 2018 and 2019 investigations.

This document reviews the stability through 9 no. representative cross-sections selected around the perimeter of the proposed Knocknacran West pit (Sections A-A' to I-I'). The locations of the cross-sections are shown in Figure 2, with the cross-sections provided in Appendix B.

These cross-sections have been created using a combination of logs from previous boreholes within the footprint of the pit and logs from the Golder 2018-2019 ground investigation programme. The cross-sections are developed from existing ground surface to the top of the Lower Gypsum Unit.

The Knocknacran West pit will be developed in specific Phases which will require detailed design of the long-term perimeter slopes and the short-term internal slopes. These detailed designs will be optimized to extract the Lower Gypsum and maintain the required FoS and thus may have shallower or steeper overall slope gradients depending on the nature of overburden materials present in that Phase footprint.

This document presents a preliminary design for the proposed Knocknacran West pit to support the Planning Application and the Environmental Impact Assessment Report (EIAR) for the development. Representative long-term cross-sections around the perimeter of the proposed pit are selected for stability analyses and have been preliminarily designed to meet the design criteria for FoS (Section 6.0 below). Further design cases will be considered at the detailed design stage for each Phase.

The phreatic surface within the pit footprint is assumed to be within the underground workings in the Lower Gypsum and sensitivity analyses have been conducted for three (3) cross-sections which align with monitoring well installations and recorded water elevations. Further design cases will be considered at the detailed design stage of the individual Phases.

6.0 DESIGN CRITERIA

Table 2 presents the design criteria for the minimum factor of safety (FoS) proposed for the slope stability assessment of the Knocknacran West open-cast mine.

Table 2: Pit Slope Stability Factor of Safety

Slope Scale	Applicability	Factor of Safety Criteria
		General Acceptance Criteria [1]
Overall Slope	Entry and Exit across any portion of the slope	1.5
Bench	Localised failures between benches	1.2

Note: General Practice Factors of Safety based on Sullivan (Sullivan, 2006) and Adams (Adams, 2015)

7.0 GROUND MODELS

The ground models for each of the 9 no. cross-sections were created using a combination of the previous borehole logs and the Golder 2018-2019 ground investigations. The 2018-19 investigation data (KC18 series) are logged to greater detail (i.e., full logs to BS 5930) with more accurate ground elevation data, and so more gravitas was given to these boreholes when defining the model. The boreholes used for each of the sections are listed in Table 3 and are shown on Figure 2 below.

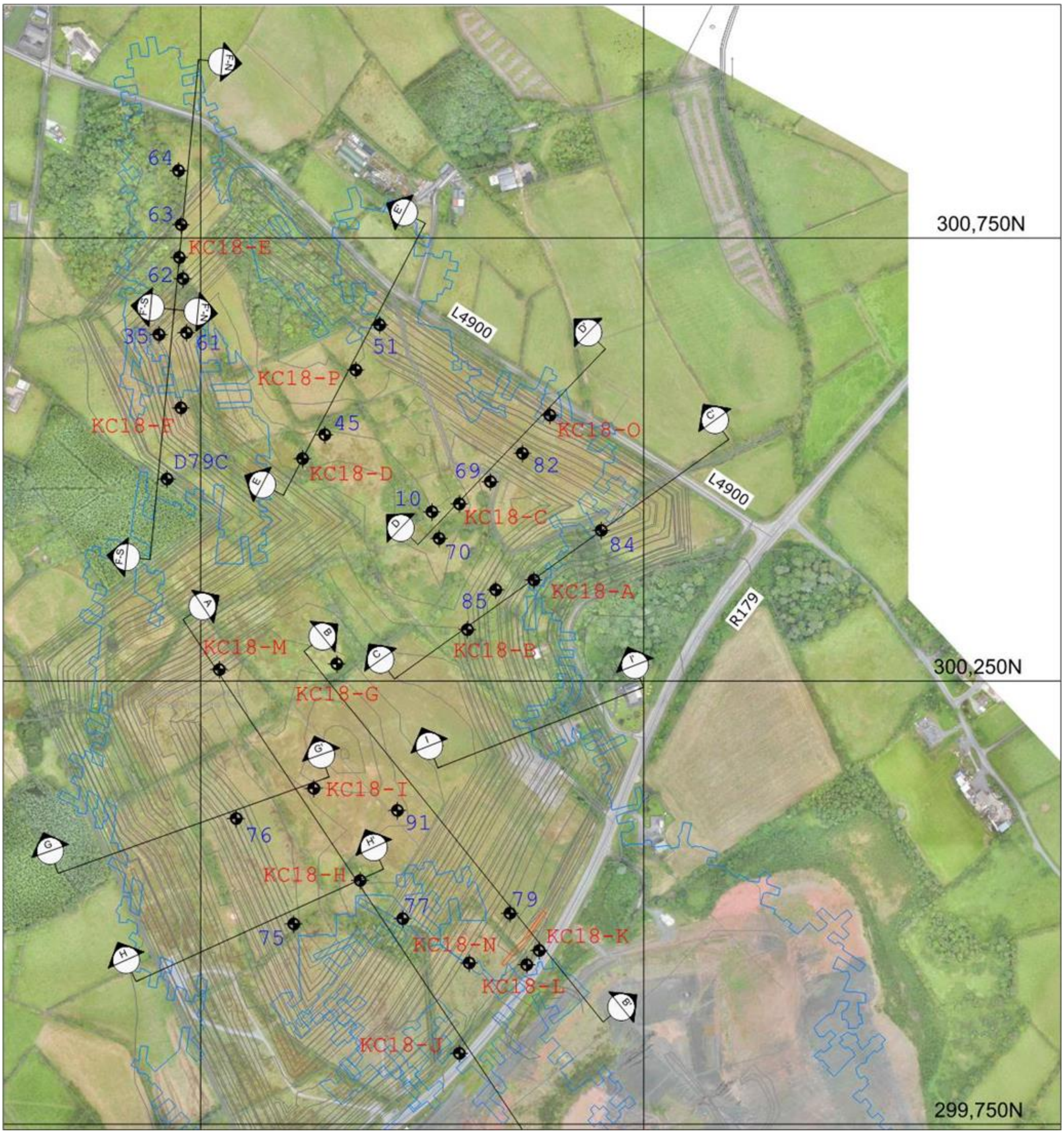

Figure 2: Site Plan presenting the Cross-Section Lines and Boreholes used for each ground model

Table 3: Ground Model Inputs

Cross Section	Boreholes Used	Location
Section A-A’	KC18-J, 77 and KC18-H	Southern side of Proposed Pit
Section B-B’	KC18-L, KC18-K, 79 and 91	
Section C-C’	KC18-A, KC18-B, 84 and 85	Eastern Side of Proposed Pit
Section D-D’	KC18-O, KC18-C, 69 and 82	
Section E-E’	KC18-D, KC18-P, 45 and 51	
Section F-F’	KC18-F, KC18-E, 63 and 64	
Section G-G’	KC18-I, 91, 76	Western side of Proposed Pit
Section H-H’	KC18-H and 75	
Section I-I’	KC18-B	South-east corner of Proposed Pit

7.1 Stratigraphy

The site is underlain by the Kingscourt Gypsum Formation of Permian age which comprises of conglomerate, sandstones, mudstones and gypsum with intruded dolerites. The recent geology of the region comprises of glacial tills and landforms. A typical stratigraphic section is shown in Figure 3 below (north corner of the Knocknacran Pit). The Upper Gypsum Unit can be absent at discrete locations and the mudstone is then typically replaced by weathered dolerite, typically in the form of ‘sands and gravels’

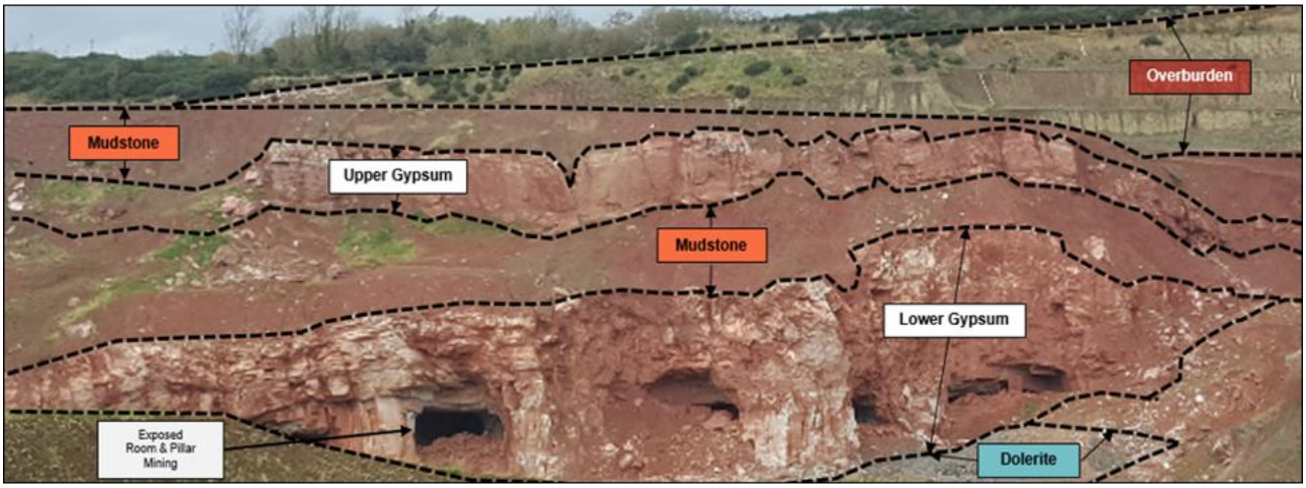

Figure 3: Typical Stratigraphy Section (north corner of Knocknacran Pit)

Note: The coordinated system in use on Site is Irish National Grid (ING) and elevations are taken to Knocknacran Mine Datum (Malin Head + 1,002.6 m) (i.e. 50 mOD is equivalent to 1,052.6 mMD).

Stratigraphy Summary for Sections A-A’ to I-I’

Section A-A’

Section A-A’ is situated to the south of the proposed pit. The cross-section runs from South to North using boreholes KC18-J, 77 and KC18-H and the ground elevation is ca. 1050 mMD.

Table 4: Section A-A' Stratigraphy

Unit	Elevation @ Top of Unit	Thickness of Unit	Comments
Till	1050 mMD (Surface)	11.5 m	
Overburden Mudstone	1038.5 mMD	1.5 m	
Upper Gypsum	1037 mMD	6.0 m	Beds have an apparent dip of 6.5° to the North.
Interburden Mudstone	1031 mMD	11.5 m	
Lower Gypsum	1019.5 mMD	-	

KC18-J was drilled at the crest of the proposed pit. The upper 13 m of the borehole was logged as "No returns, inferred Mudstone". The lack of information on the deposit and the presence of Till in the two other boreholes along the Section line has led to the upper 11. 5m being assigned the Till geotechnical parameters.

Section B-B'

Section B-B' is situated to the south of the proposed pit. The section runs from South to North using boreholes KC18-L, KC18-K, 79 and 91 and the ground elevation is ca. 1056 mMD.

Table 5: Section B-B' Stratigraphy

Unit	Elevation @ Top of Unit	Thickness of Unit	Comments
Till	1056 mMD (Surface)	10.0 m	
Overburden Mudstone	1046 mMD	10.0 m	
Upper Gypsum	1036 mMD	5.0 m	Beds have an initial apparent dip of 3.5° to the North. This increases to nearly 20° approximately 75 m from the crest.
Interburden Mudstone	1031 mMD	11.0 m	
Lower Gypsum	1020 mMD	-	

KC18-K and KC18-L were both drilled at the crest of the proposed pit. The upper 16.6 m and 16.7 m respectively were logged as "No returns, inferred Mudstone". The lack of information on the deposit and the presence of Till in the two other boreholes along the section line has led to the upper 10m being assigned the Till geotechnical parameters.

Section C-C'

Section C-C' is situated to the east of the proposed pit. The section runs from East to West using boreholes KC18-A, KC18-B, 83 and 84 and the ground elevation is ca.1046 mMD.

Table 6: Section C-C' Stratigraphy

Unit	Elevation @ Top of Unit	Thickness of Unit	Comments
Overburden Mudstone	1046 mMD (Surface)	10.0 m	
Doleritic Sands and Gravels	1036 mMD	46.0 m	The dolerite is shown to pinch out with distance from the pit crest i.e., to the west
Lower Gypsum	990 mMD	-	

Section D-D’

Section D-D’ is situated to the east of the proposed pit. The section runs from East to West using boreholes KC18-O, KC18-C, 69 and 82 and the ground elevation is ca. 1047 mMD.

Table 7: Section D-D’ Stratigraphy

Unit	Elevation @ Top of Unit	Thickness of Unit	Comments
Till	1047 mMD (Surface)	3.0 m	
Overburden Mudstone	1044 mMD	11.0 m	
Doleritic Sands and Gravels	1033 mMD	18.0 m	
Lower Gypsum	1015 mMD	-	

Section E-E’

Section E-E’ is situated to the east of the proposed pit. The section runs from East to West using boreholes KC18-D, KC18-P, 45 and 51 and the ground elevation is ca. 1049 mMD.

Table 8: Section E-E’ Stratigraphy

Unit	Elevation @ Top of Unit	Thickness of Unit	Comments
Till	1049 mMD (Surface)	1.0 m	
Overburden Mudstone	1048 mMD	5.0 m	
Doleritic Sands and Gravels	1043 mMD	8.0 m	
Lower Gypsum	1035 mMD	-	

Section F-F’ (northern face and southern face)

Section F-F’ is situated to the east of the proposed pit. The section runs from East to West using boreholes KC18-F, KC18-E, 63 and 64 and the ground elevation is ca. 1053 mMD north and ca. 1062 mMD south. This stratigraphy is summarized in Table 9 and Table 10 below.

Table 9: Section F-F’ Stratigraphy (Northern Face)

Unit	Elevation @ Top of Unit	Thickness of Unit	Comments
Till	1053 mMD (Surface)	7.0 m	
Overburden Mudstone	1046 mMD	6.0 m	All units are dipping approximately 4° to the south.
Upper Gypsum	1040 mMD	9.0 m	
Interburden Mudstone	1031 mMD	7.0 m	
Lower Gypsum	1024 mMD	-	

Table 10: Section F-F' Stratigraphy (Southern Face)

Unit	Elevation @ Top of Unit	Thickness of Unit	Comments
Till	1062 mMD (Surface)	6.0 m	
Overburden Mudstone	1056 mMD	48.0 m	Mudstone has increase in thickness from approximately 6m to 48m.
Upper Gypsum	1008 mMD	9.0 m	All units are dipping approximately 8° to the south.
Interburden Mudstone	999 mMD	4.5m	
Lower Gypsum	984.5 mMD	-	

Section G-G'

Section G-G' is situated to the west of the proposed pit. The section runs from West to East using boreholes KC18-I, 91, 76 and the ground elevation is ca. 1044 mMD.

Table 11: Section G-G' Stratigraphy

Unit	Elevation @ Top of Unit	Thickness of Unit	Comments
Till	1044 mMD (Surface)	15.5 m	
Overburden Mudstone	1028.5 mMD	33.5 m	All units have a minor dip to the west.
Upper Gypsum	995 mMD	11.5 m	
Interburden Mudstone	983.5 mMD	30.5 m	
Lower Gypsum	953 mMD	-	

Section H-H'

Section H-H' is situated to the west of the proposed pit. The section runs from West to East using boreholes KC18-H and 75 and the ground elevation is ca. 1043 mMD.

Table 12: Section H-H' Stratigraphy

Unit	Elevation @ Top of Unit	Thickness of Unit	Comments
Till	1043 mMD (Surface)	6.5 m	
Overburden Mudstone	1036.5 mMD	56.5 m	All units have approximately a 5.5° dip to the west.
Upper Gypsum	980 mMD	7.0 m	
Interburden Mudstone	973 mMD	14.5 m	
Lower Gypsum	958.5 mMD	-	

Section I-I'

Section I-I' is situated in the south-east corner of the proposed pit. The section runs from West to East using boreholes KC18-B, and the ground elevation is ca. 1044 mMD.

Table 13: Section I-I' Stratigraphy

Unit	Elevation @ Top of Unit	Thickness of Unit	Comments
Till	1044 mMD (Surface)	15.5 m	
Overburden Mudstone	1028.5 mMD	33.5 m	All units have approximately a 5.5° dip to the west.
Upper Gypsum	995 mMD	11.5 m	
Interburden Mudstone	983.5 mMD	30.5 m	
Lower Gypsum	953 mMD	-	

7.2 Ground Water Level

Following the subsidence event which occurred in September 2018, the Drumgoosat underground workings are being progressively dewatered at a rate of ca. 55 m^3/day. The water elevation in the workings reported on the 21 October 2021 was 976.05 mMD which indicates a decrease of ca. 19 m since the end of September 2018, when the water level was at ca. 995 mMD).

The Golder 2018-2019 ground investigation programme proposed three (3) no. long-term groundwater monitoring wells located at the northern, eastern and western extents of the new mining area, i.e., at locations KC 19-A, B and C (Figure 4). The monitoring wells were installed at various horizons at each location to intersect the underlying gypsum units (Table 14).

Table 14: Monitoring Well Data

Design ID	Well ID	Installed Depth (m)	Target Horizon	Reading July 2021 (mMD)
Location A				
WM-A1	KC19-A1	78.0	Base of Upper Gypsum	1035.03
WM-A2	KC19-A2	126.0	Base of Lower Gypsum	1033.43
Location B				
WM-B1	KC19-B1	61.0	Base of Upper Gypsum	1029.71
WM-B2	KC19-B2	97.5	Base of Lower Gypsum	1028.65
Location C				
WM-C1	KC19-C1	69.0	Base of Upper Gypsum	994.54

Figure 4: Groundwater Monitoring Borehole Locations

The footprint of the proposed Knocknacran West Open-Cast Mine is largely dewatered by the underground workings and by the adjacent Knocknacran pit.

Where mudstones are present, groundwater is confined by low hydraulic conductivity material and an upper water elevation exists at approximately 1030 mMD locally. Where weathered doleritic material is present, the groundwater is drained to lower elevations by the higher conductivity material to approx. 996 mMD (the groundwater elevations used here were recorded between June and October 2019).

7.3 Material Parameters

The geotechnical parameters of the stratigraphy have been well established in previous reports (Table 1). Golder conducted additional borehole, sampling and laboratory testing during 2018 and 2019 to confirm the stratigraphy and material parameters. Table 15 below summarises the strength parameters of the various materials that were interpreted and utilized in the analyses.

Table 15: Material Strength Parameters

Material	Unit Weight (kN/m^3)	Cohesion' (kPa)	Phi' (°)	Hoek-Brown Strength Function			
				σ_{ci} (MPa)	m_i	GSI	D
Fill Material	22	0	36	-	-	-	-
Till (Boulder Clay)	21	0	29	-	-	-	-
Weathered Doleritic Material (sands & gravels)	20	7	39	-	-	-	-
Weathered Doleritic Material (sands & gravels)	20	-	-	5.5	19	25	0.7
Overburden Mudstone	23	25	27	-	-	-	-
Gypsum	23	300	30	-	-	-	-
Interburden Mudstone	22	9.5	30				
Underburden Mudstone	23	0	27	-	-	-	-

The unit weight, cohesion and internal friction values were estimated based on Golder interpretation of the laboratory testing following site investigations in 2017 at Knocknacran and in 2019 at Knocknacran West, and review of the previous laboratory testing, interpretation and reporting of the same geological units by Geoffrey Walton (Geoffrey Walton, 1982), Atkins (Atkins, 2006) and SRK (SRK, 2018) (SRK, 2019).

The shear stress / normal stress strength function was estimated based on the Hoek-Brown model. The Hoek-Brown model is a nonlinear shear strength model for rock and is an industry standard method for estimating the strength of rock masses.

Four input parameters are required to compute the shear strength versus normal stress curve, namely:

- σ_{ci} = the uniaxial compressive strength of the intact rock;
- m_i = a property of the intact rock;
- GSI = Geological Strength Index (0 – 100); and
- D = rock mass disturbance factor (0 – 1).

The value used for the uniaxial compressive strength of the intact rock (σ_{ci}) is based on laboratory tests completed on rock in the same geological unit in the area as well as our interpretation of the information provided by the SRK geotechnical reports (SRK, 2018).

8.0 DESIGN CASE

The FoS design criteria are presented in Section 6.0. The following design conditions have been outlined for these analyses:

- The long-term (drained) case has been assessed for all sections. This case is selected as the proposed pit is dewatered;
- The short-term (undrained) case has been assessed for the three (3) no. sections located adjacent to the installed monitoring wells. A piezometric line gradient of 3(H):1(V) is modelled from the upstream toe of the pit excavation to the recorded water elevations;
- No pseudo static design case is considered from either earthquake or blasting influence;
- No loading has been applied to the benches as there will be no haulage or access;
- A traffic load of 10 kPa was applied over a 25 m width 5 m from the downstream toe of the safety berm for sections where the R179 and L4900 roads are present. This is following Eurocode 7 Guidance;
- A minimum slip surface depth of 1.5 m was applied to remove negligible superficial slips; and
- Slope runs were carried out locally for each unit to access the stability of the benches as well as for the overall slope stability.

8.1 Pit Slope Geometry

The pit slope geometry is defined by the stratigraphy of the pit. Each bench is 6.0m in the width and the bench elevations correspond around the perimeter of the pit. They are situated at elevations 1052 mMD, 1048 mMD, 1042 mMD, 1036 mMD and are on the same intervals until the top of the Lower Gypsum Unit.

The inter-bench slopes are each 6.0 m high. Two inter-bench slope angles have been utilized for the design to account for the varying material parameters and to maintain a minimum overall slope of 2(H):1(V) or 26.6° and meet the design criteria for FoS.

- Till – A slope of 2(H):1(V)or 26.6° ; and
- Remaining lithologies – A slope of 1(H):1V) or 45°.

A graphical representation of the pit slope geometry is presented in Figure 5 below.

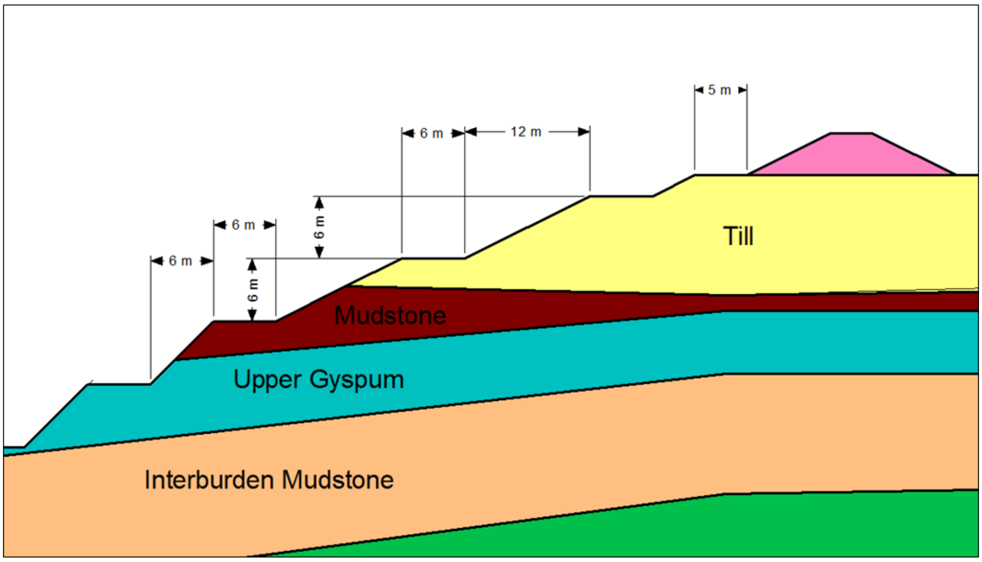

Figure 5: Representative section showing the Pit Slope Geometry

9.0 STABILITY ANALYSES RESULTS

9.1 Pit Slope Analyses

The slope stability analyses were carried out using the limit equilibrium modelling software SLOPE-W version 10.0.2.1001. The analytical method used was Morgenstern and Price method of slices, which satisfies both force and moment equilibrium. The following slope stability cases were analysed:

- An inter bench slip surface for the different strata; and
- An overall slope slip surface.

The piezometric level is below the level of the Lower Gypsum as the operational underground Drummond Mine located to the south provides dewatering for the proposed Knocknacran West Open-Cast Mine; the water elevation recorded in the underground workings on 21 October 2021 was 976.05 mMD.

The results of the stability analyses are presented in Table 16 below and the stability analyses are shown in Figure 6 to Figure 17 below. For Section I-I', Figure 16 shows the upper slope case which returned a minimum factor of safety (FoS) of 2.01, and Figure 17 the shallow failure in till which returned a minimum factor of safety (FoS) of 2.67. In summary:

- The FoS varies from 1.5 to 2.3 for the overall slope and from 1.2 to 2.5 for the inter bench which meets the minimum recommended values shown in Table 2; and
- The weathered doleritic 'sands and gravels' have been assessed as both a soil material with cohesion and an angle of internal and as a rock material using the Hoek-Brown Strength Function, and the results are generally similar for each slip surface.

Table 16: Stability Analyses Summary

Section	Factor of Safety (FoS)	
	Local	Overall
Section A-A'		
Till	1.2 (Inter bench)	**1.9** (Figure 6**)**
Interburden Mudstone	1.7	
Section B-B'		
Till	1.2 (Inter bench)	**1.6** (Figure 7**)**
Overburden Mudstone	1.8	
Interburden Mudstone	1.4	
Section C-C'		
Overburden Mudstone	2.1	**1.6** (Figure 8**)**
Doleritic Sands and Gravels	1.5	

Section	Factor of Safety (FoS)	
	Local	Overall
Section D-D'		
Till	1.4 (Inter bench)	**1.7** (Figure 9**)**
Overburden Mudstone	2.2	
Doleritic Sands and Gravels	1.5	
Section E-E'		
Overburden Mudstone	1.9	**1.8** **(**Figure 10**)**
Doleritic Sands and Gravels	1.3 (Inter Bench)	
Section F-F': Northern Face		
Till	1.2 (Inter bench)	**2.2** **(**Figure 11**)**
Overburden Mudstone	2.5	
Interburden Mudstone	1.4	
Section F-F' - Southern Face		
Till	1.5	**1.5** **(**Figure 12**)**
Overburden Mudstone	1.5	
Interburden Mudstone	1.4	
Section G-G'		
Till	1.2 (Inter bench)	**1.5** **(**Figure 13**)**
Overburden Mudstone	1.5	
Interburden Mudstone	1.4	
Section H-H'		
Till	1.5	**1.5 *** **(**Figure 14**)**
Overburden Mudstone	1.5	
Interburden Mudstone	1.3	
Section I-I'		
Till	2.7	**1.7** (Figure 15)
Overburden Mudstone	2.0	

* Gradient of highest overburden bench (typically Till) slackened to 1(V):2(H) to attain the required FoS of 1.5.

A piezometric line gradient of 3(H):1(V) is modelled from the upstream toe of the proposed pit excavation to the recorded groundwater elevations for the three (3) no. cross-sections aligned with the installed monitoring wells to assess the impact on the slope stability factor of safety i.e., Section C-C’, Section F-F’ and Section G-G’.

The results of the stability analyses are presented in Table 17 below and the stability analyses are shown in Figure 18 to Figure 21 below.

Table 17: Stability Analyses with Phreatic Gradient of 3(H):1(V)

Section	Factor of Safety (FoS)	
	Local	**Overall**
Section C-C’		
Doleritic Sand and Gravels	1.6	**1.8** (Figure 18)
Section F-F’ Southern Face		
Overburden Mudstone	1.5	**1.4** (Figure 19)
Interburden Mudstone	1.4	
Section F-F’ Northern Face		
Interburden Mudstone	1.3	**2.0** (Figure 20)
Section G-G’		
Overburden Mudstone	1.5	**1.4** (Figure 21)
Interburden Mudstone	1.4	

Note: The FoS returned for Section F-F’ and G-G’ is less than the design criteria FoS.

The cases assessed and summarized in Table 17 are for a worst-case scenario assuming fully saturated materials below the phreatic surface and a 3(H):1(H) gradient from the downstream toe of the pit slope.

The phreatic surface recorded within the mudstones at Section F-F’ and G-G’ are understood to be perched water-tables and do not represent fully saturated materials below.

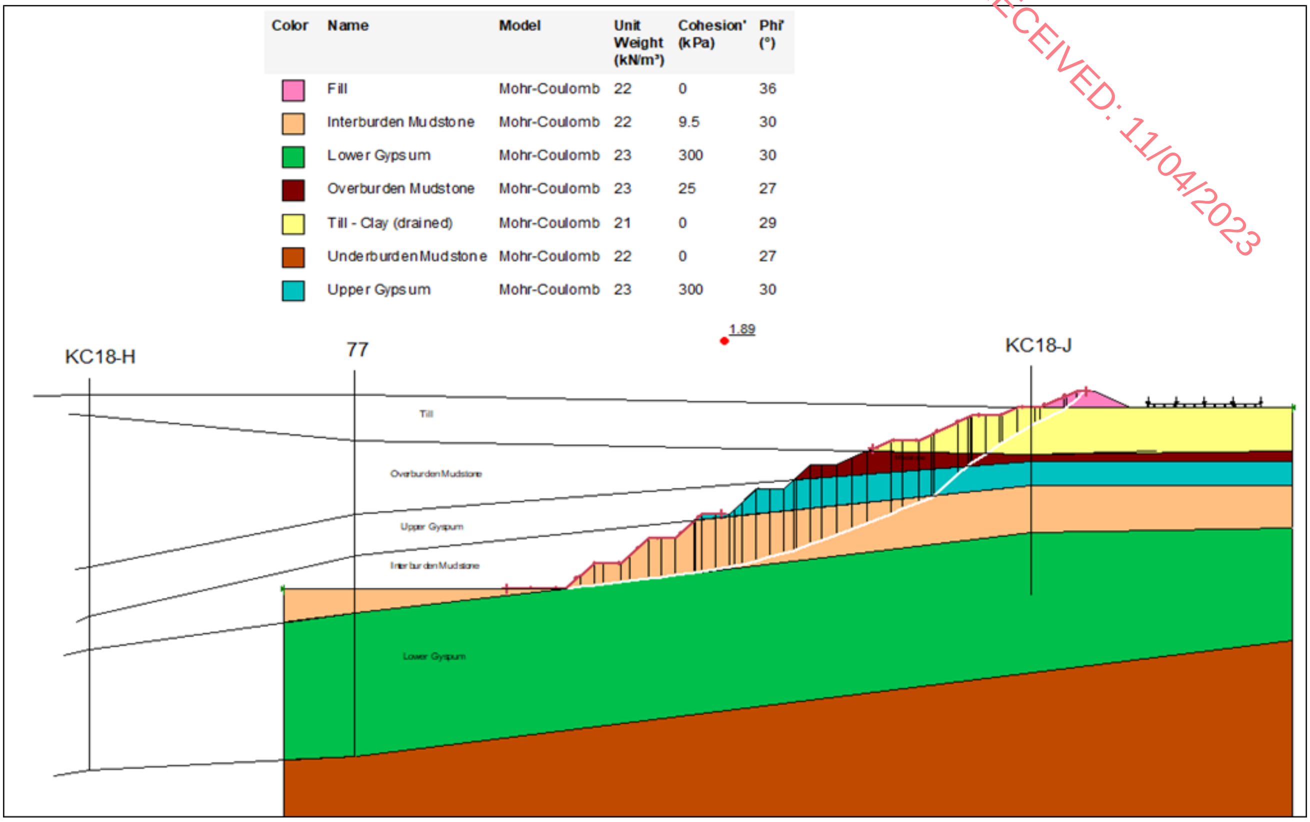

Figure 6: Section A-A' Overall Slope Stability

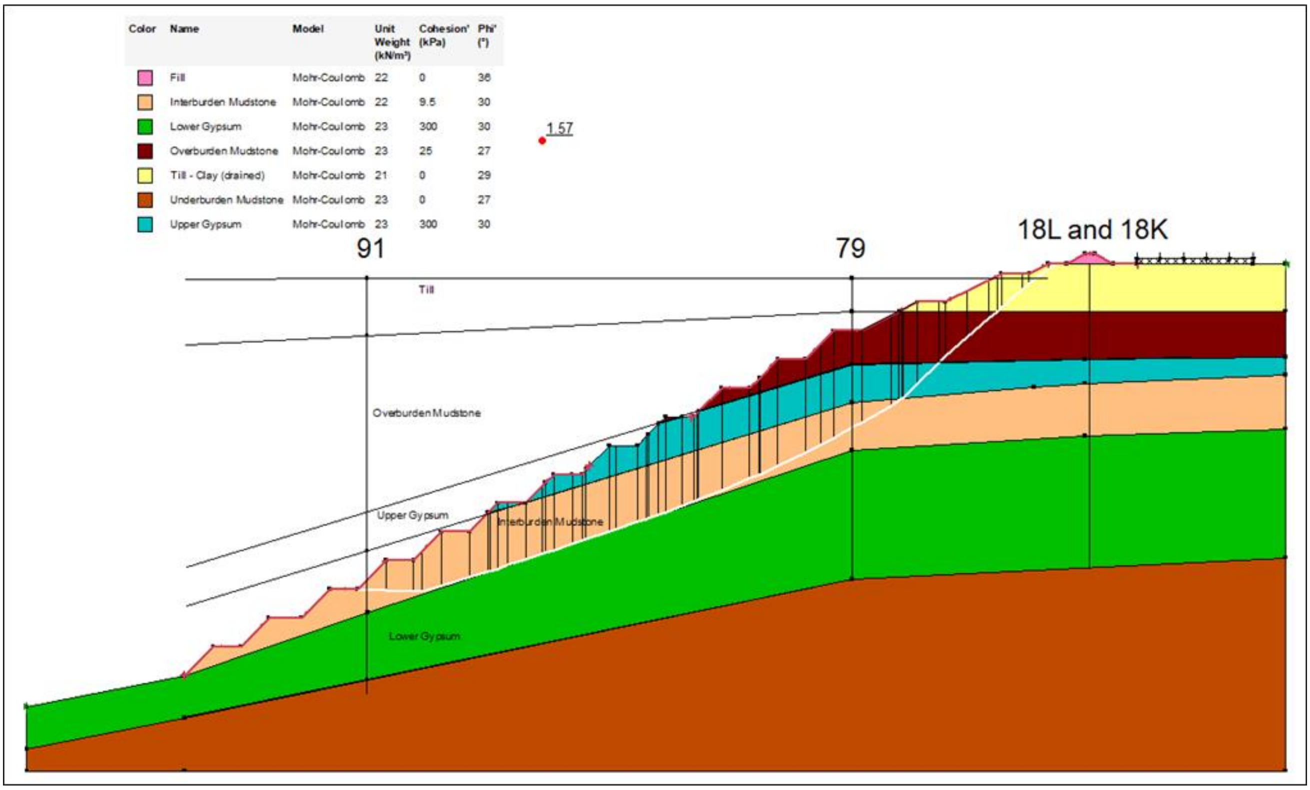

Figure 7: Section B-B' Overall Slope Stability

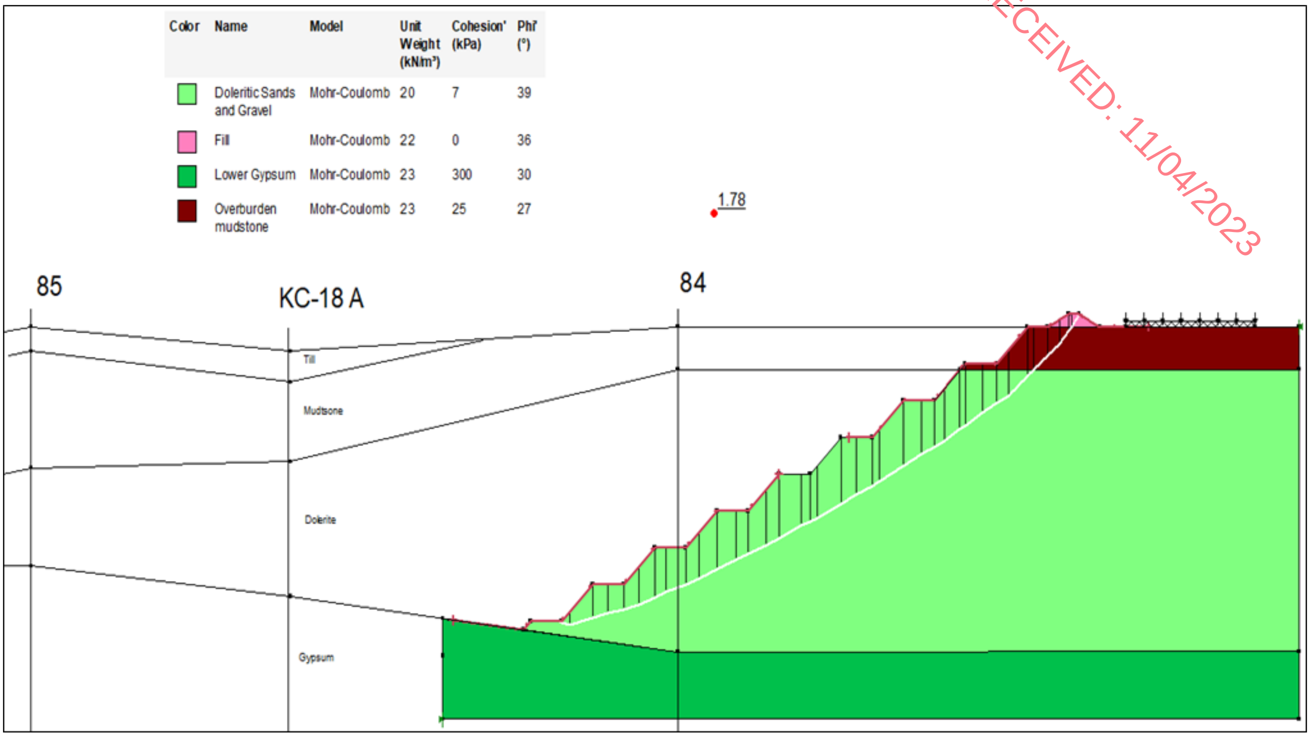

Figure 8: Figure 8: Section C-C' Overall Slope Stability

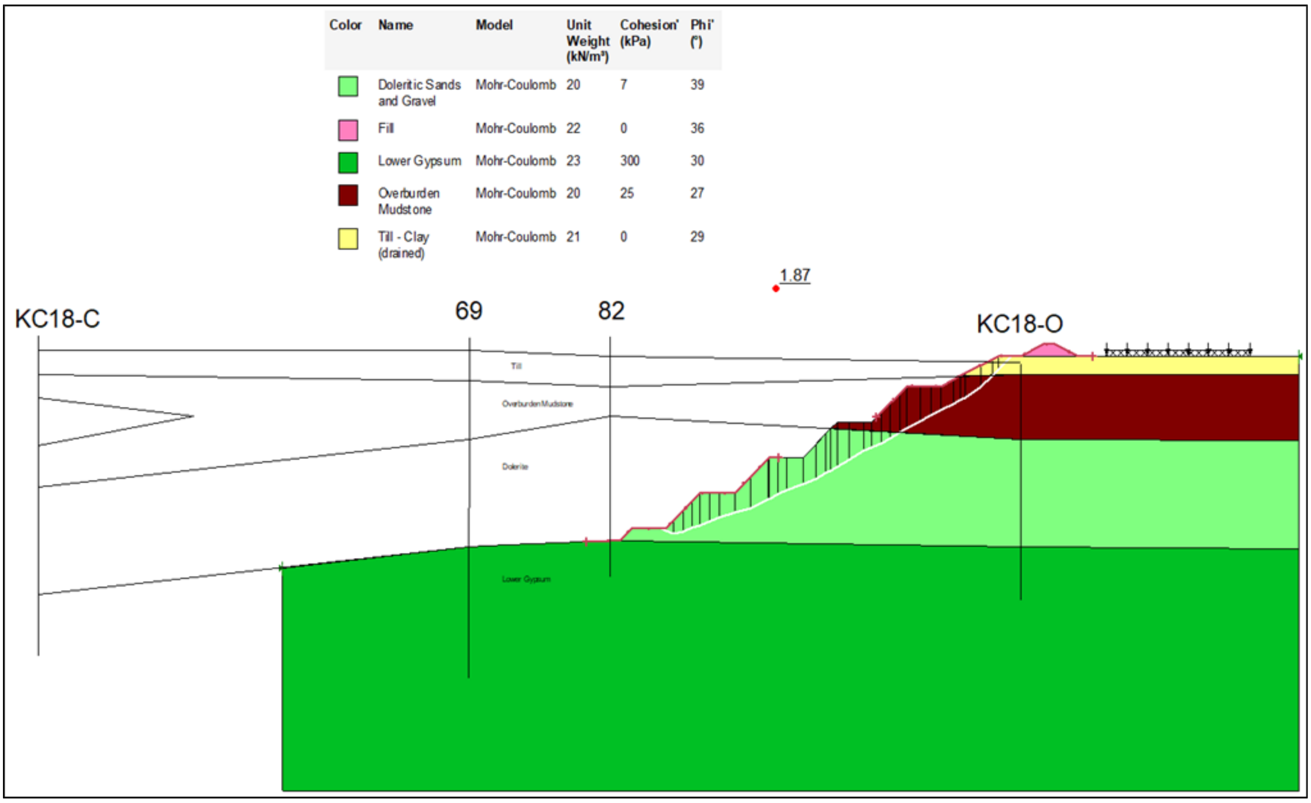

Figure 9: Section D-D' Overall Slope Stability

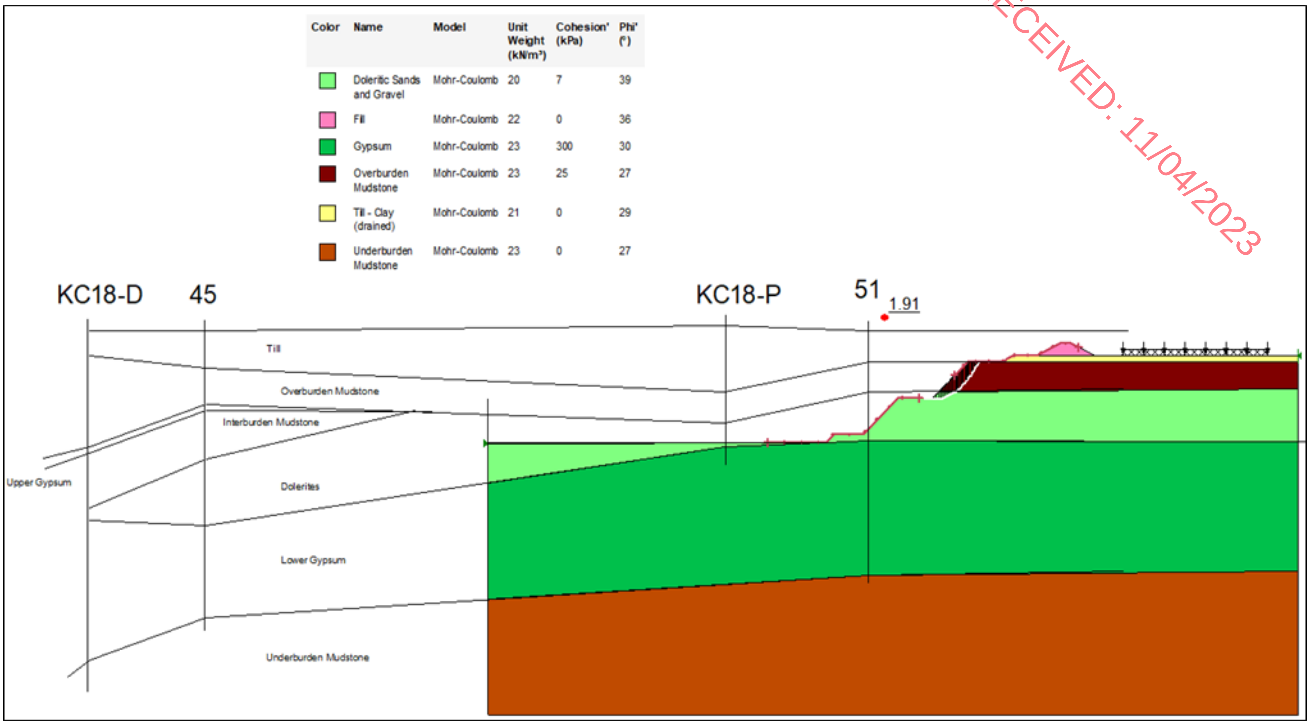

Figure 10: Section E-E' Overall Slope Stability

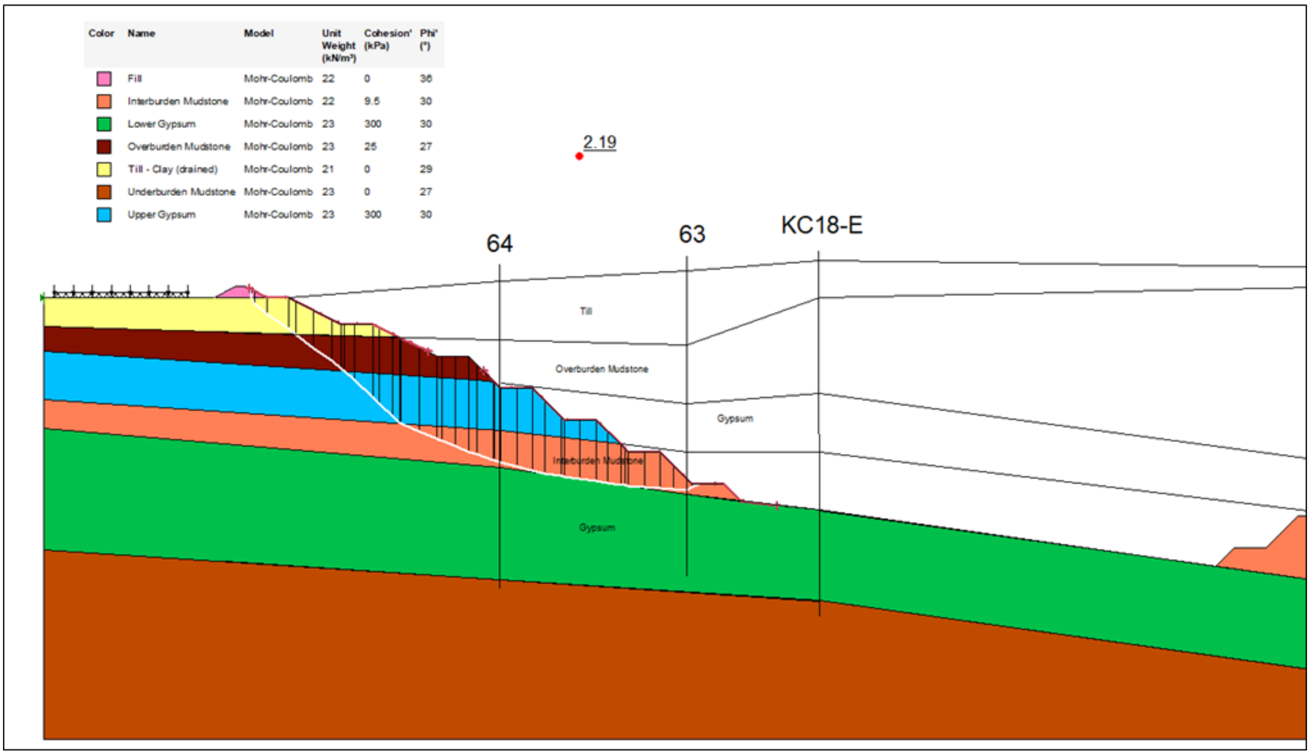

Figure 11: Section F-F' (North) Overall Slope Stability

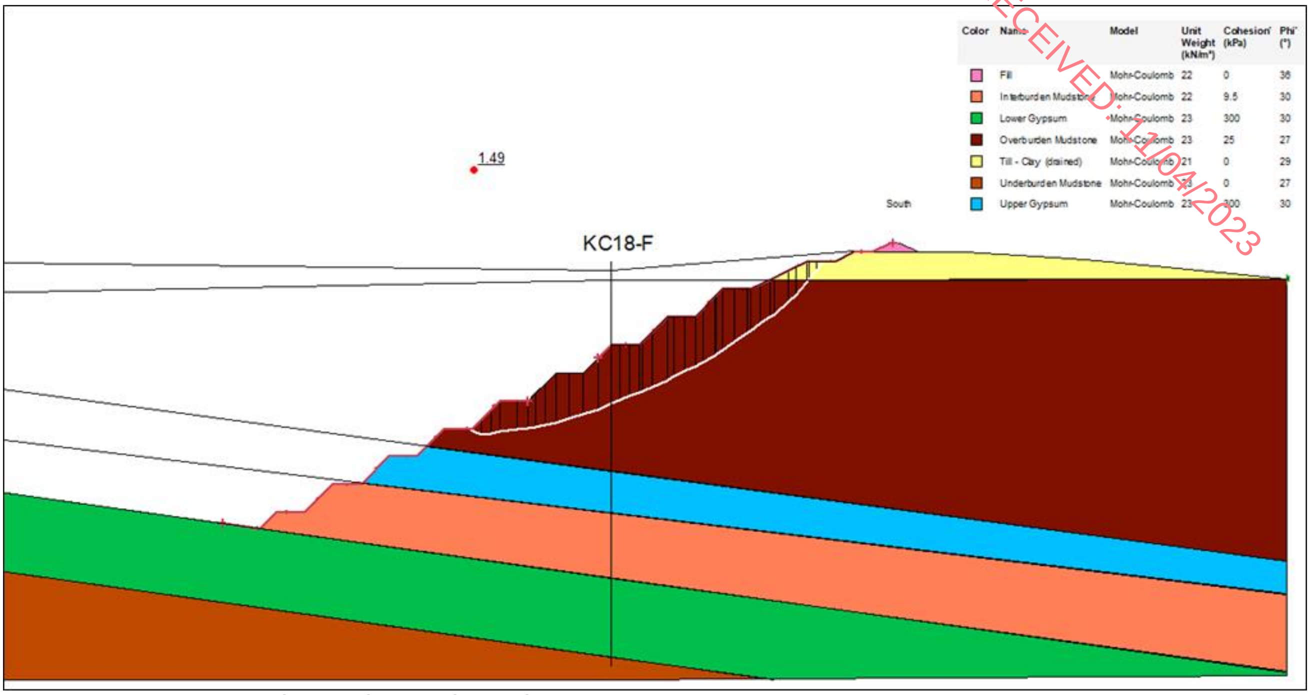

Figure 12: Section F-F' (South) Overall Slope Stability

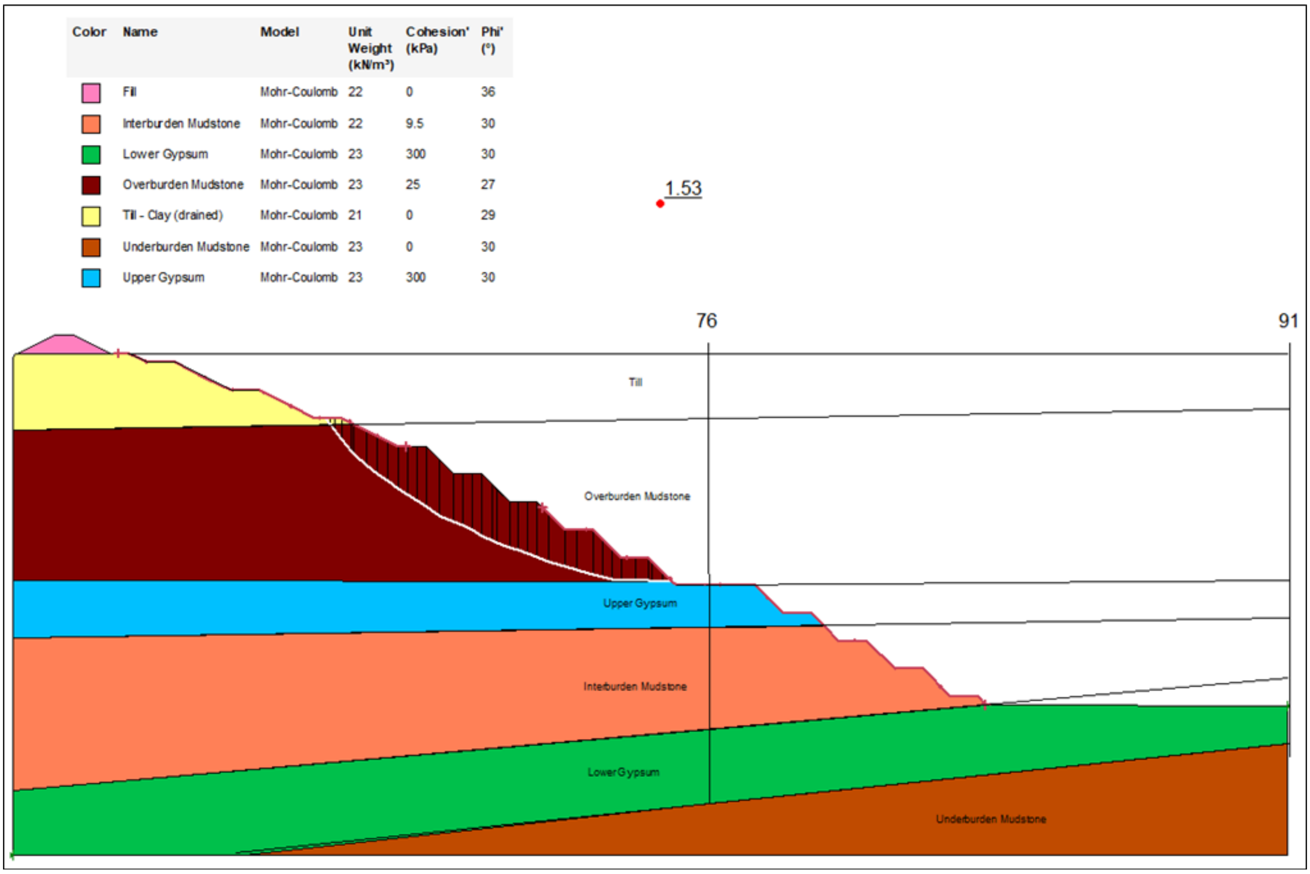

Figure 13: Section G-G' Overall Slope Stability

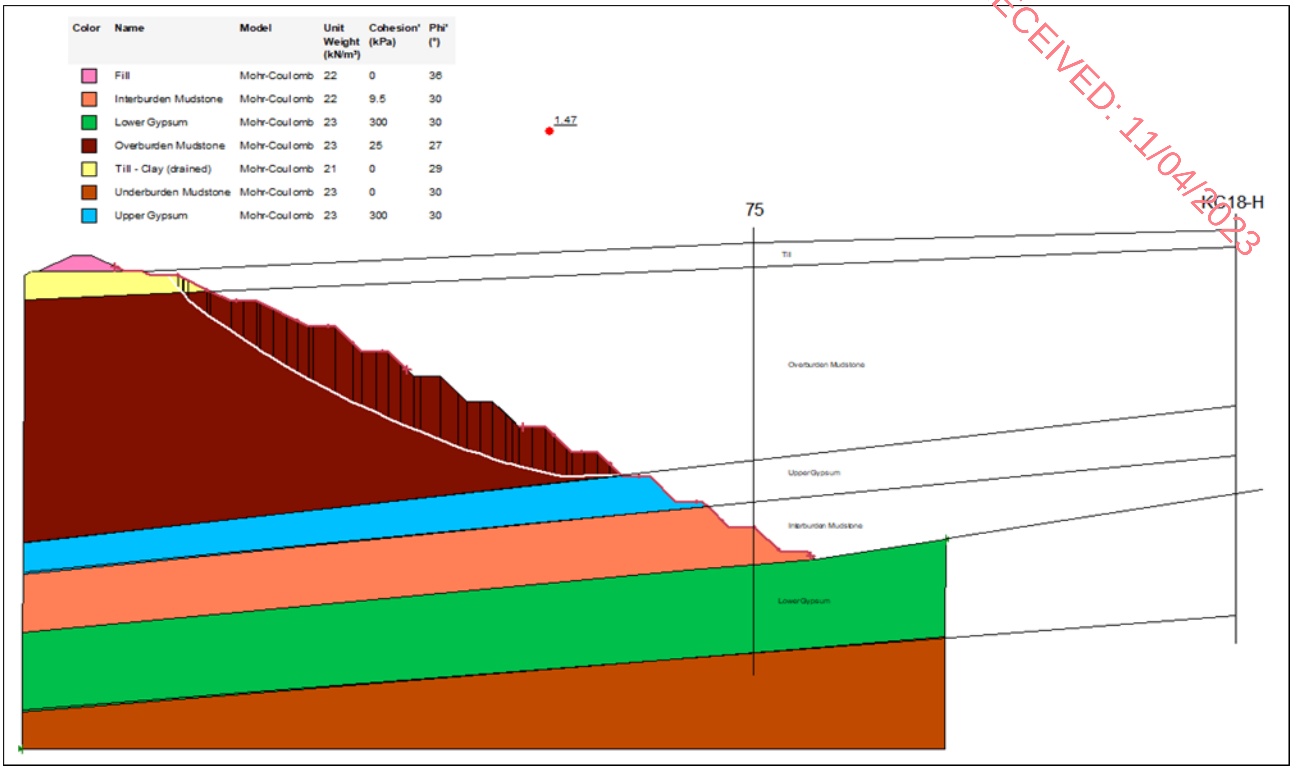

Figure 14: Section H-H' Overall Slope Stability

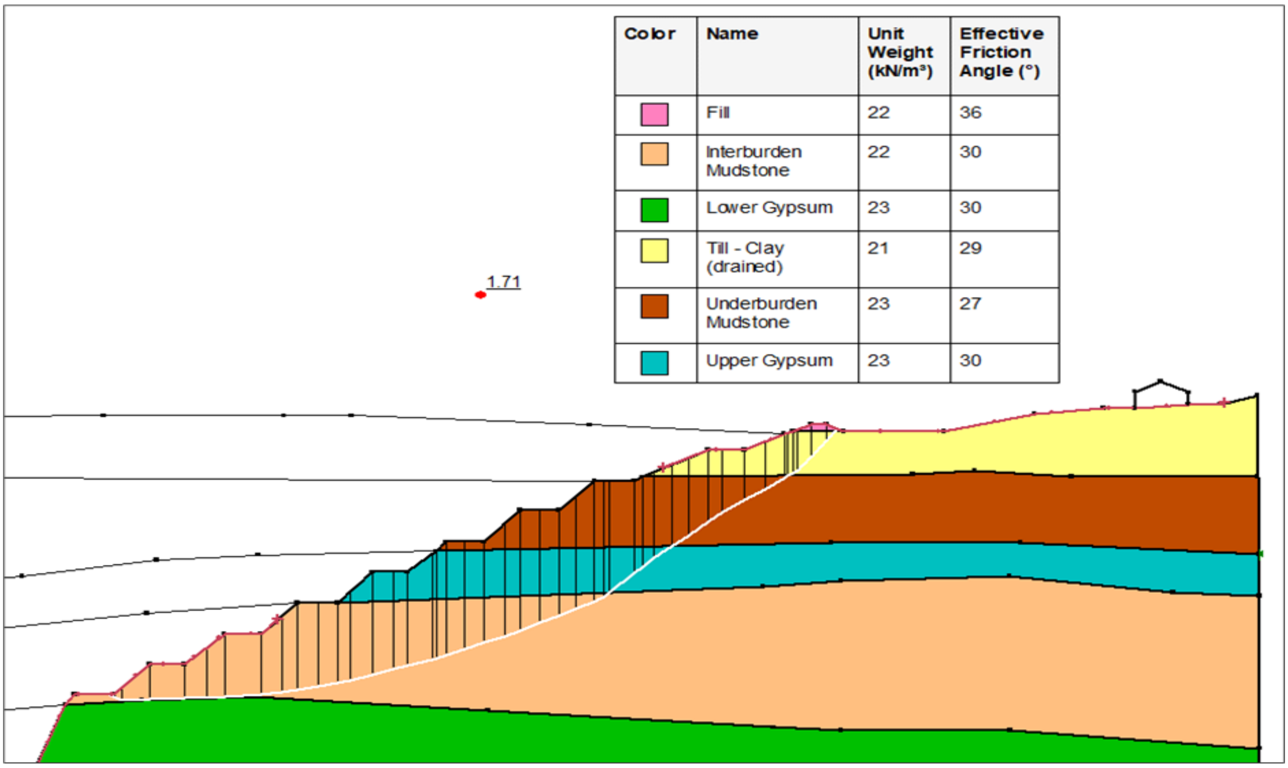

Figure 15: Section I-I' Overall Slope Stability

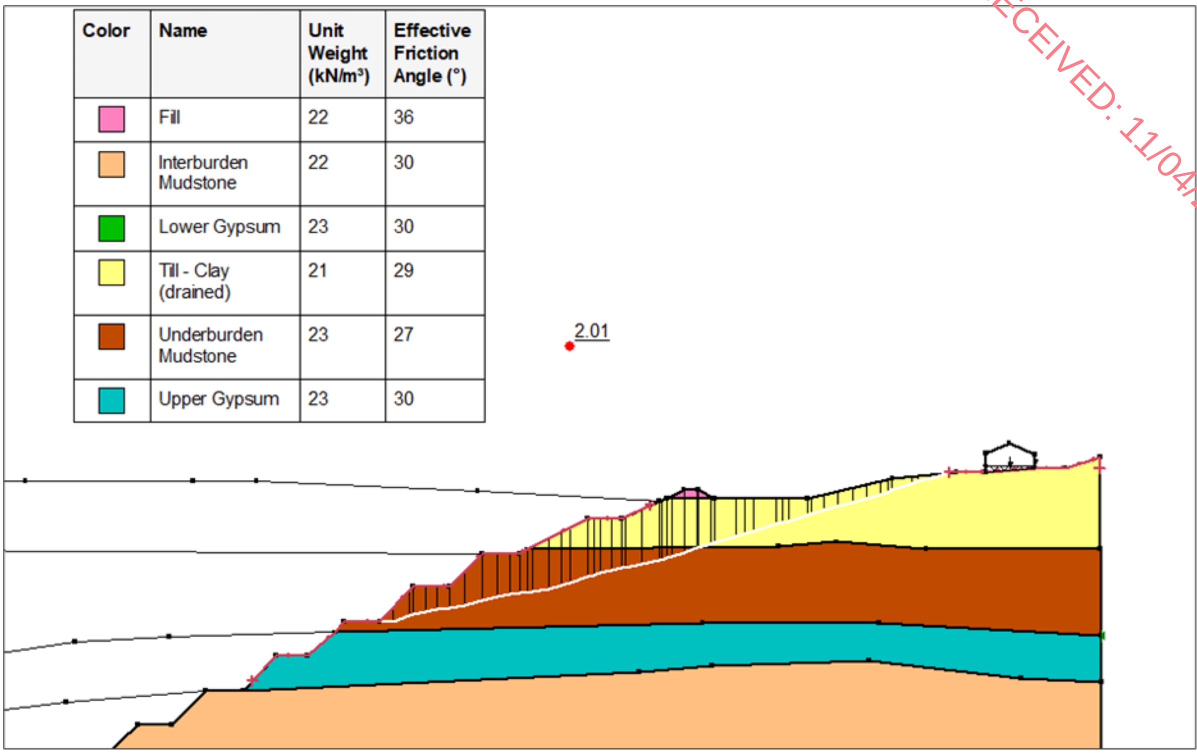

Figure 16: Section I-I' Upper Slope Stability

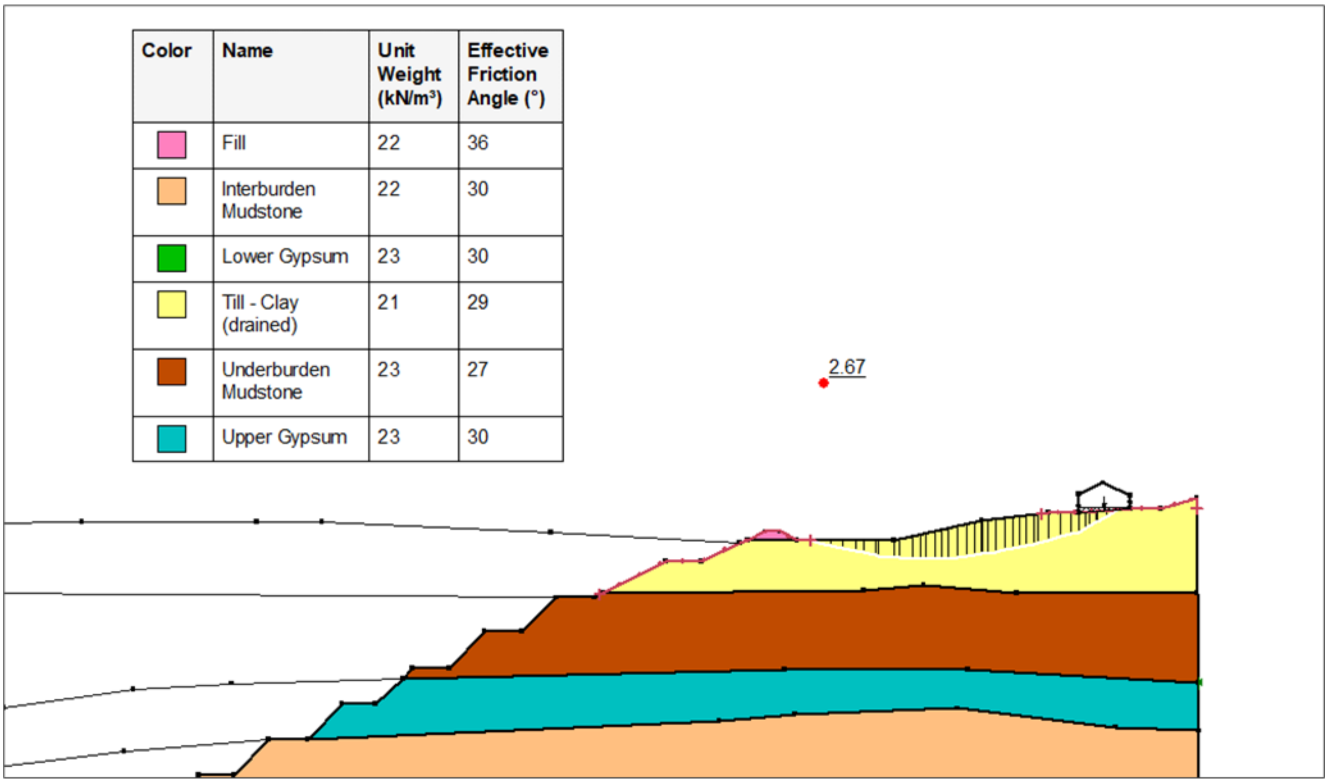

Figure 17: Section I-I' Till Stability

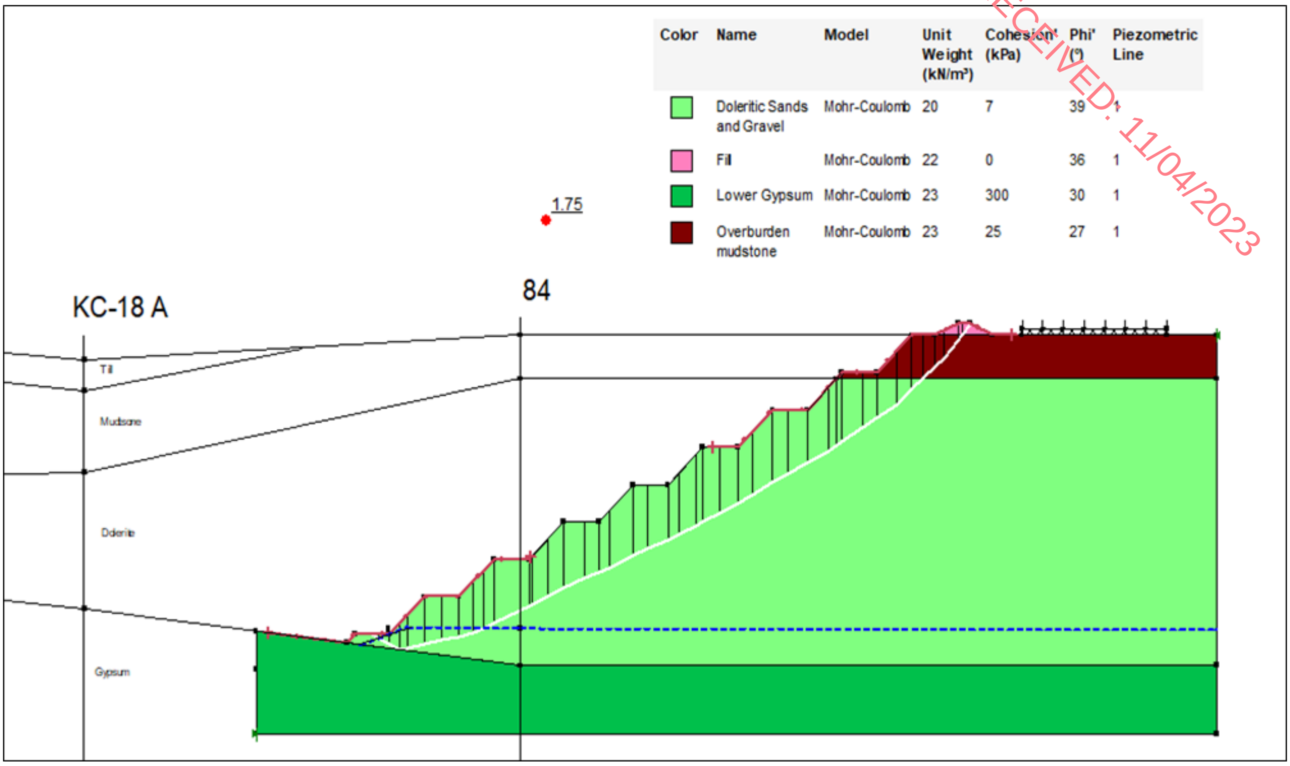

Figure 18: Section C-C' Overall Slope Stability with Phreatic Surface at 3(H):1(V)

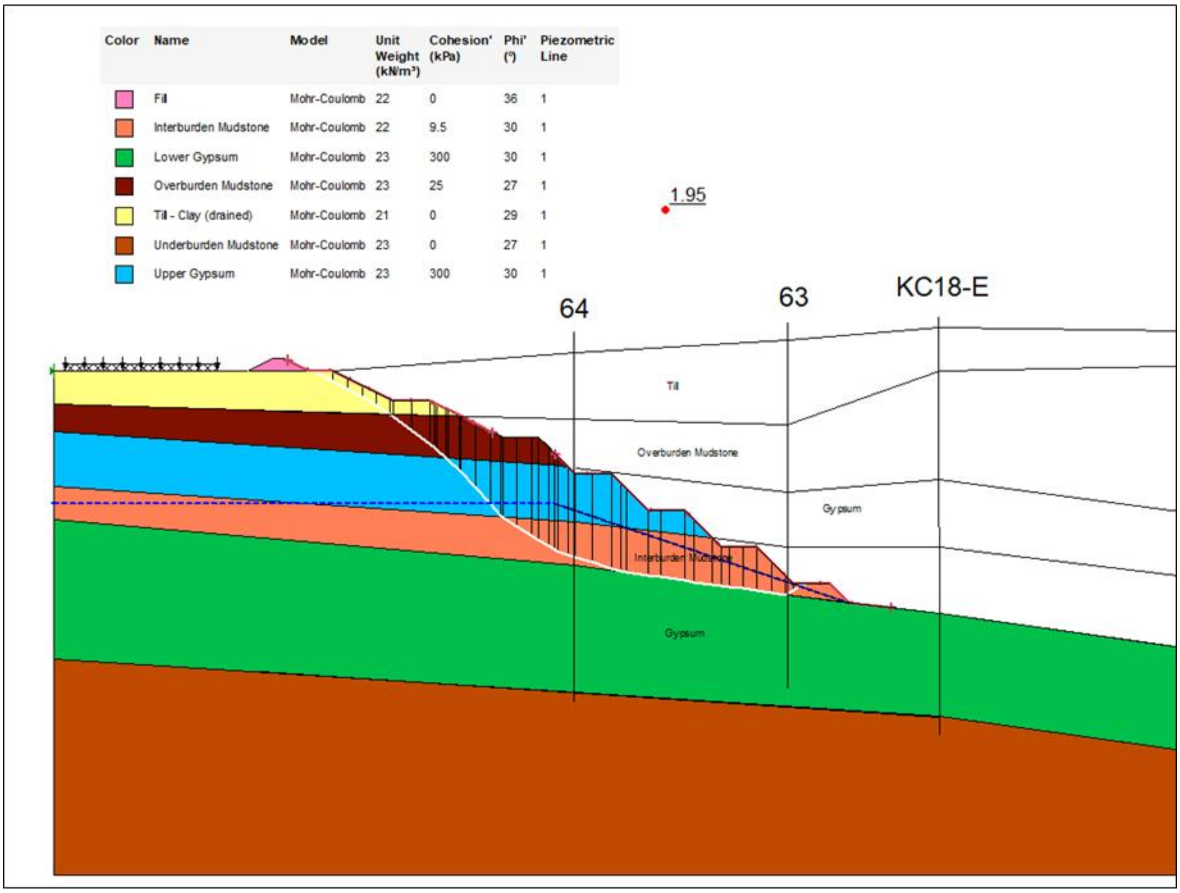

Figure 19: Section F-F' (North) Overall Slope Stability with Phreatic Surface at 3(H):1(V)

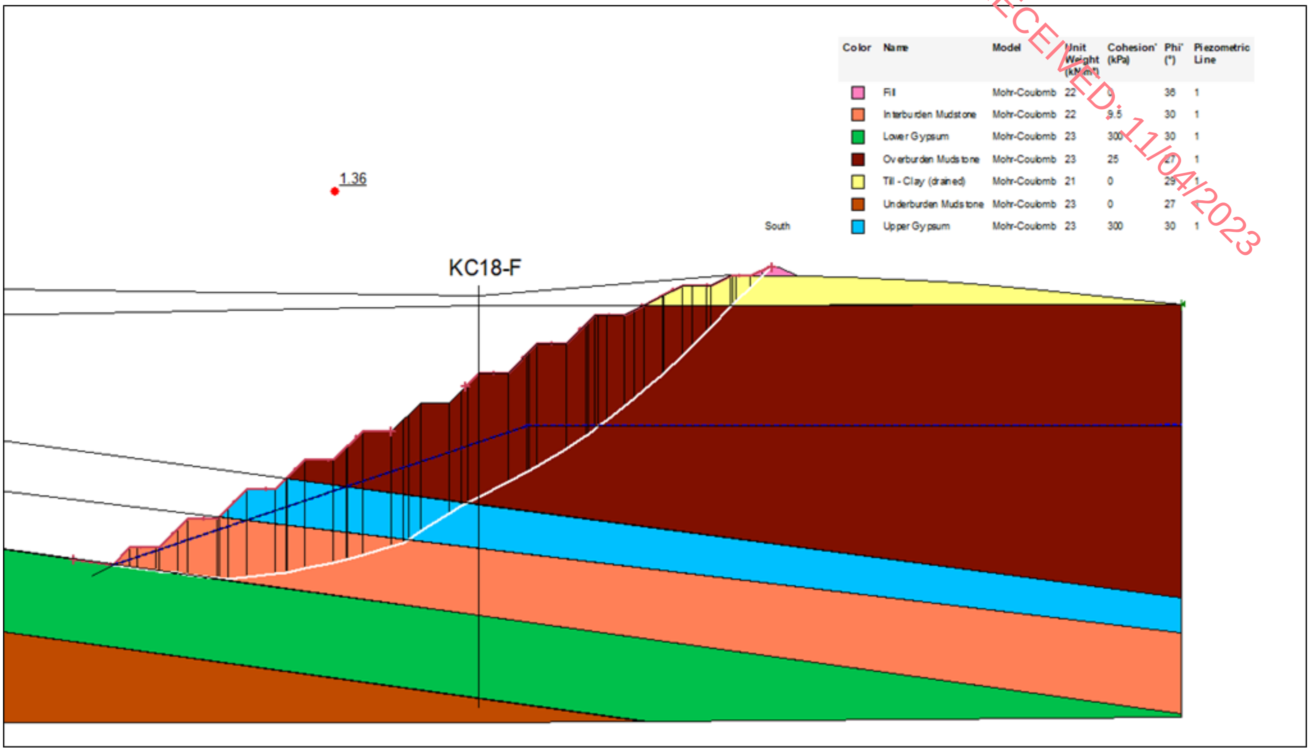

Figure 20: Section F-F' (South) Overall Slope Stability with Phreatic Surface at 3(H):1(V)

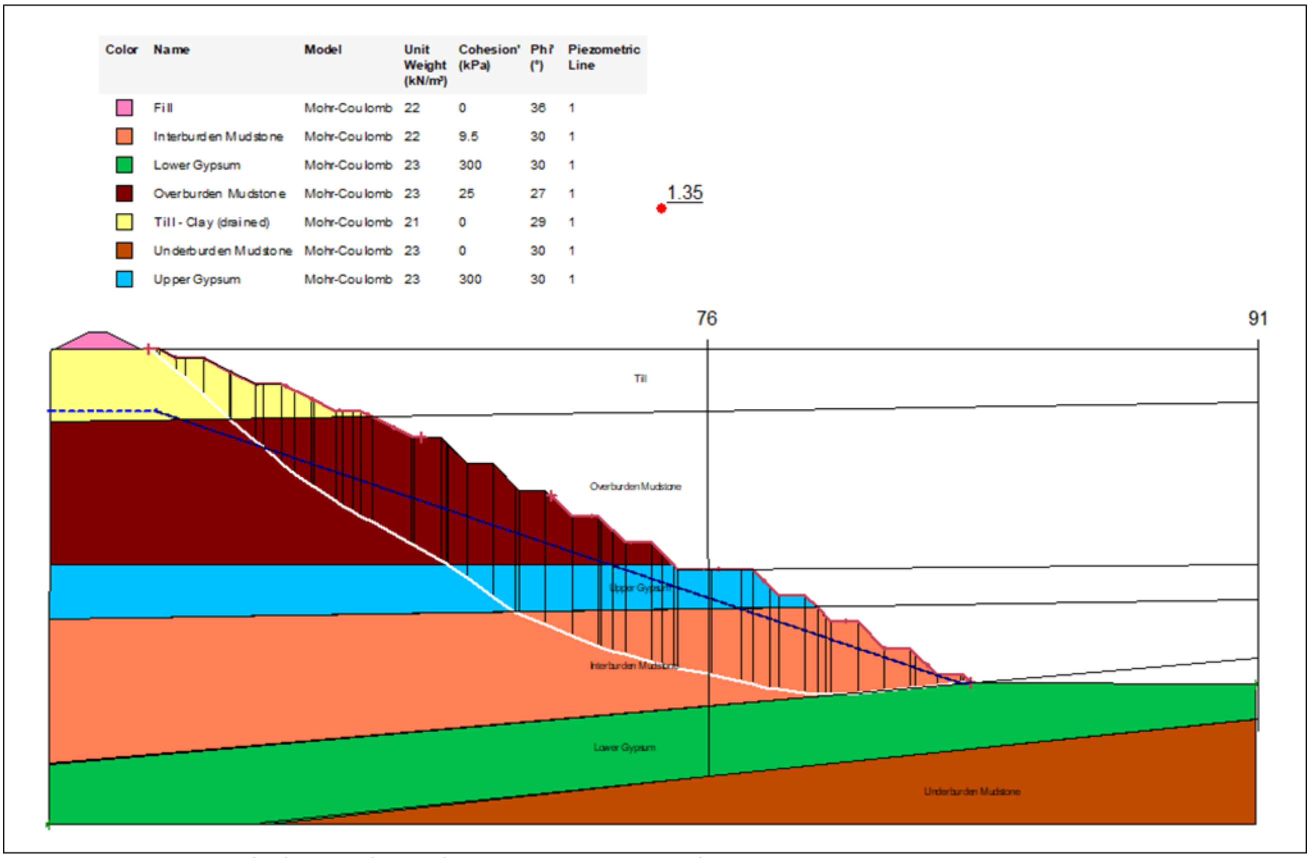

Figure 21: Section G-G' Overall Slope Stability with Phreatic Surface at 3(H):1(V)

9.2 Pit Slope Analysis - Presence of Underground Workings

The following dimensions are understood to be applicable for the underground mine workings at the former Drumgoosat Mine:

- Roof beam thickness = normally 3 m thick;
- Mine workings height = normally 6 m high;
- Mine workings width = normally 9 m wide; and
- Floor beam thickness = normally minimum of 1 m thick.

The mine workings have been inserted into the slope stability models for two representative sections; Section D-D' (Figure 22) and Section G-G' (Figure 23). Both these models have been run in accordance with the methodology used above. Figure 22 shows that the model for Section D-D' returned a minimum FoS of 1.87, which is the same as recorded in Figure 9 of the Pit Slope Stability Assessment. Figure 23 shows that the model for Section G-G' returned a minimum FoS of 1.75, which is greater than that recorded in Figure 13 of the Pit Slope Stability Assessment. Both models returned minimum FoS values greater than the design criteria FoS of 1.5.

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Doleritic Sands and Gravel	Mohr-Coulomb	20	7	39
	Fill	Mohr-Coulomb	22	0	36
	Lower Gypsum	Mohr-Coulomb	23	300	30
	Mined Area	Mohr-Coulomb	0.1	0	0
	Overburden Mudstone	Mohr-Coulomb	20	25	27
	Till - Clay (drained)	Mohr-Coulomb	21	0	29

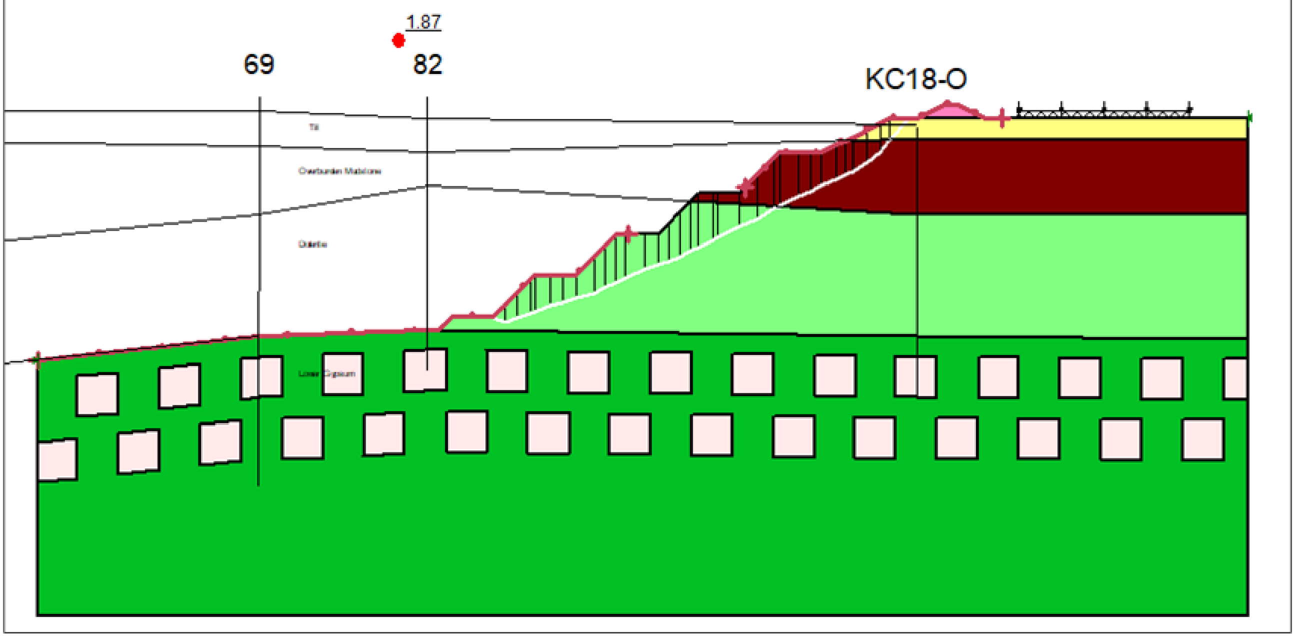

Figure 22: Section D-D', global case with mine workings (FoS = 1.87)

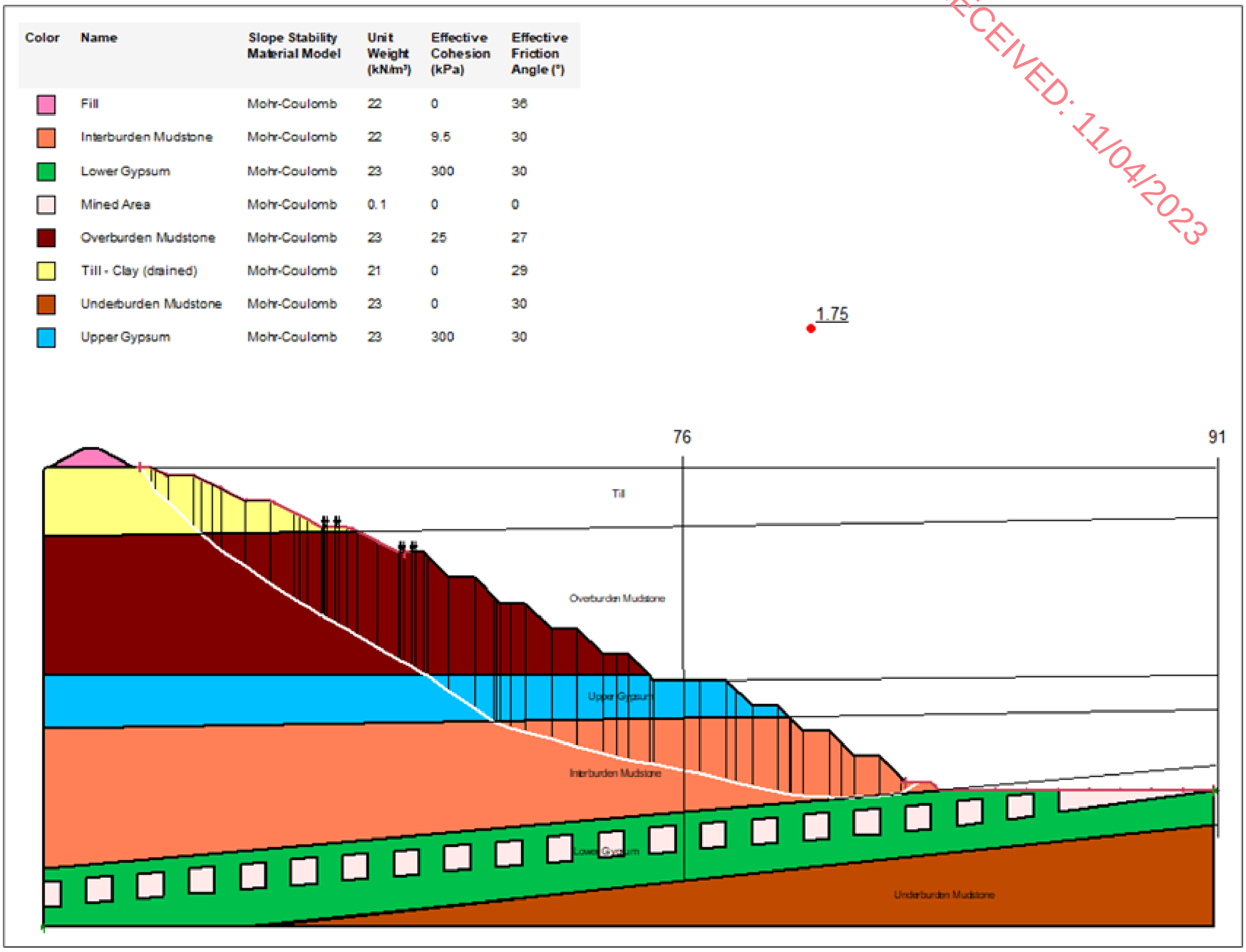

Figure 23: Section G-G’, global case with mine workings (FoS = 1.75)

9.3 Pit Slope Analysis - Assessment of Planar Failure Surfaces where there is a Dip Evident in Gypsum Beds

Potential planar slip failures may be of concern in terms of pit wall stability, where the gypsum beds dip ‘out-of’ the face.

Section A-A’, Section B-B’ and Section F-F’ (north) have been analysed further in accordance with the methodology presented above, with the failure surface forced to occur along the top surface of the Upper Gypsum Seam.

Figure 25 presents the model for Section A-A’, which returned a minimum FoS of 2.17, which is greater than the global stability (FoS = 1.89) recorded in Figure 6 above. Figure 26 presents the model for Section B-B’, which returns a minimum FoS of 1.82, which is greater than the global stability (FoS = 1.57) recorded in Figure 7 above; and Figure 27 presents the model for Section F-F’ North which returned a minimum FoS of 2.23, which is greater than the global stability (FoS = 2.19) recorded in Figure 11 above.

All models returned minimum FoS values greater than the design criteria FoS of 1.5, and greater than the global stability FoS values presented in Table 16 above.

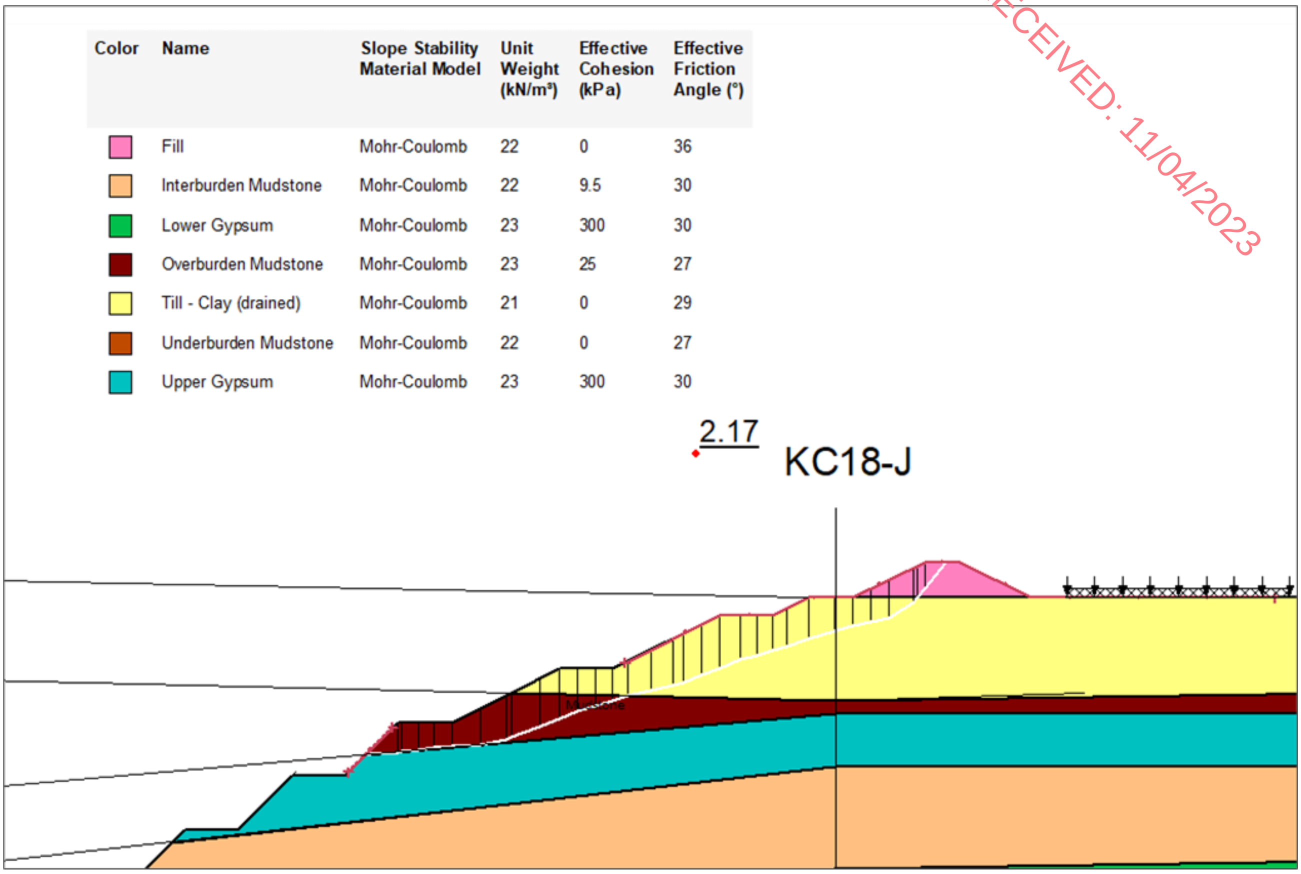

Figure 24: Section A-A', upper slope case (FoS = 2.17)

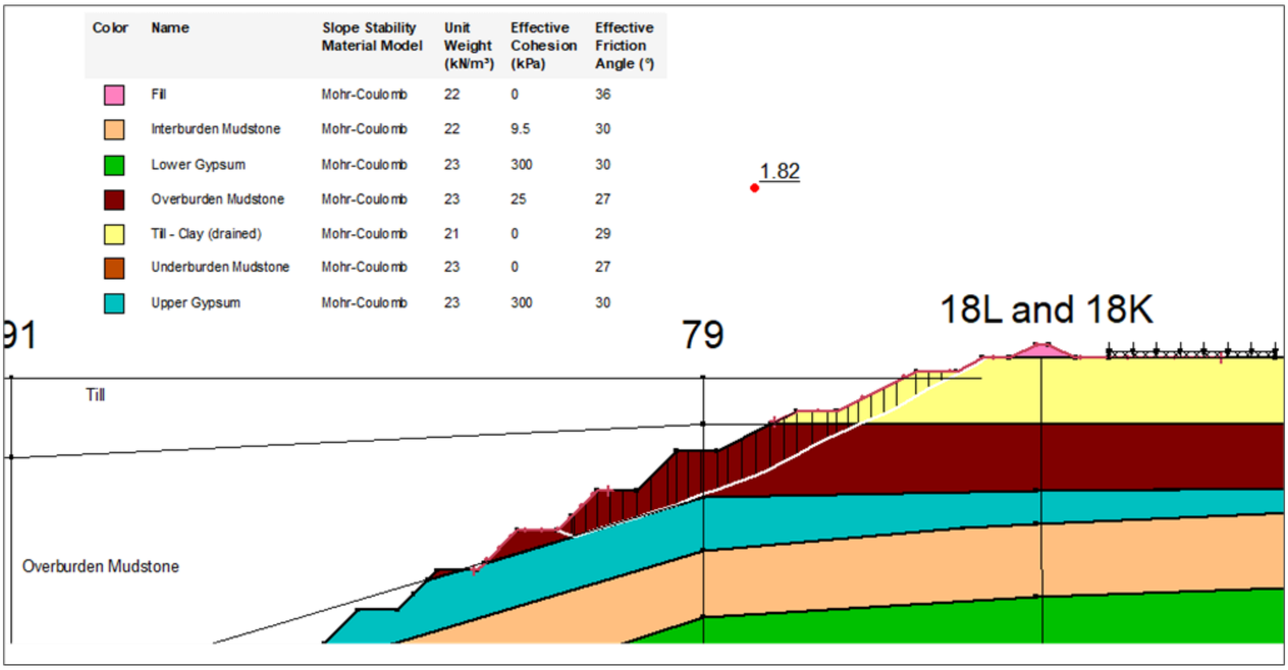

Figure 25: Section B-B', upper slope case (FoS = 1.82)

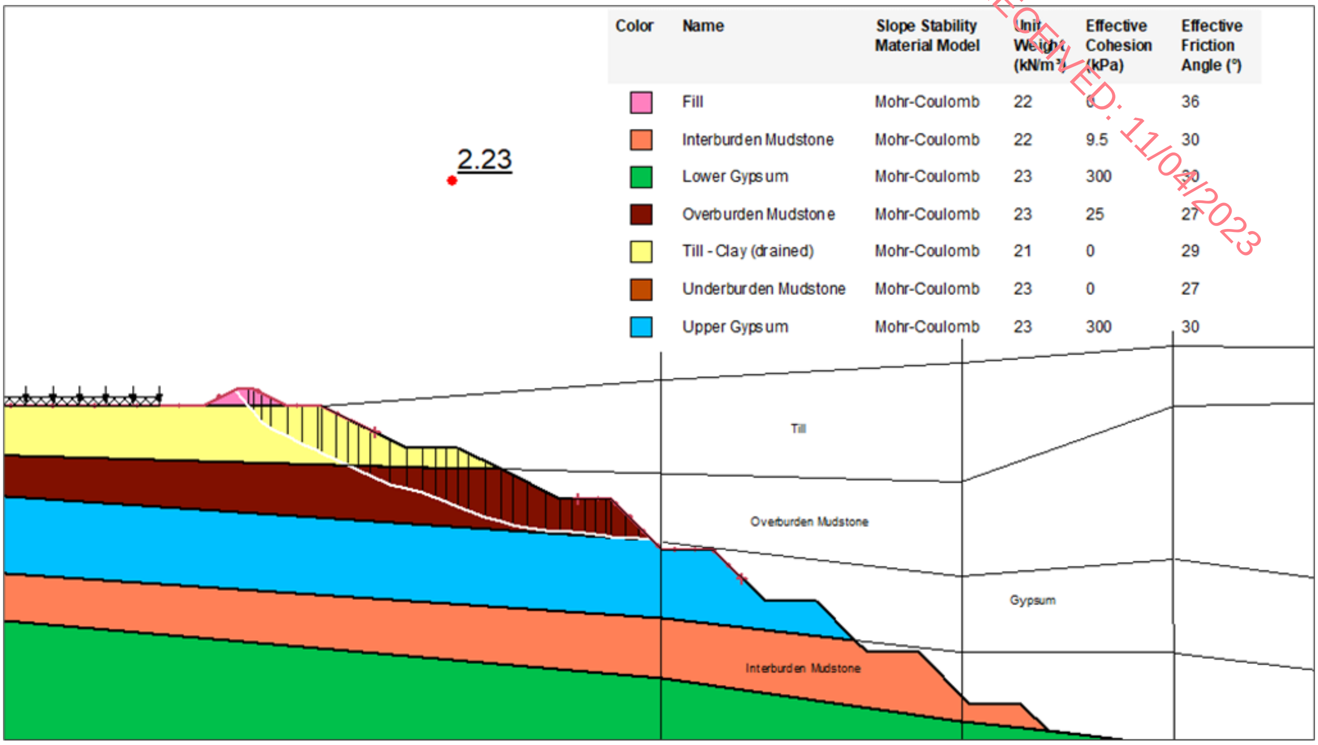

Figure 26: Section F-F' North, upper slope case (FoS = 2.23)

9.4 Pit Slope Analyses - Truck Haulage Route Initial Phase of Mining

This analysis relates to the haulage route extending into the base of the proposed open-cast mine from the north side of the Tunnel (south side slopes of the open-cast) and along the west side-slopes of the proposed open-cast mine.

A section was taken along Section G-G', across the western face of the proposed open-cast mine and the main haulage road, Figure 2 above. This section line was assessed for two scenarios:

- 6 m wide haul road on the till layer, with a 2(H):1(V) slope in the till above, and a 2(H):1(V) slope in the overburden mudstone below; and
- 6 m wide haul road on the overburden mudstone layer, with a 2(H):1(V) slope in the overburden mudstone/till above, and 1(H):1(V) slope in the overburden mudstone below.

The haul road design has a design width of 17 m but the scenarios above are assessed for plant trafficking in the outer lane (next to the inner pit) with a tyre centred 1.5 m from the edge of the haul road.

Haul truck traffic using by a CAT D40 dumper was assessed based on a total static loaded weight of approximately 68,000 kg. The stability was analysed by selecting a surcharge load acting over an area for the width of each tyre.

The two scenarios for Section G-G' have been run in accordance with the methodology presented above.

Figure 27 presents the model returned for Section G-G', with the dumper trafficking on a 6 m wide haul road with a 2(H):1(V) slope in the overburden mudstone below. A Factor of Safety (FoS) of 2.36 was returned.

Figure 28 presents the model returned for Section G-G', with the dumper trafficking on a 6 m wide haul road with a 1(H):1(V) slope in the overburden mudstone below. A FoS of 1.73 was returned.

As can be expected, the haul road with the steeper slope resulted in a lesser FoS, but still above the design criteria FoS of 1.5.

Sensitivity analysis for a haul road constructed on till with a 2(H):1(V) slope in the till below, resulted in a FoS of 1.2 for the scenario with a tyre centred at 1.5 m from the edge. A greater setback from the edge is required to achieve the target FoS of 1.5; with a minimum of 4 m being required.

An edge bund will be constructed along the outer edge of the haul road in accordance with the 'Safe Quarry Guidelines 2020'. A two-way traffic haul route will require a minimum width of 12.8 m, and an overall design width of 17 m, which allows for a width of 4.2 m for an edge berm, thereby ensuring that the worst-case tyre loading will be centred at 4.5 m from the crest.

Figure 29 presents the model returned for Section A-A' with the bund in place and a tyre loading centred at 4.5 m from the crest of the haul road (in till). A FoS of 1.70 is returned.

Reports commissioned by SRK on overall long-term mine stability (Appendix C) and roof beam stability (Appendix D) provide reassurance on the stability of underground workings throughout the life of mining at Knocknacran West. In addition, a Standard Operating Procedure (SOP) will be put in place for mining in the vicinity of suspected voids and unstable ground. A draft procedure is provided in Appendix E.

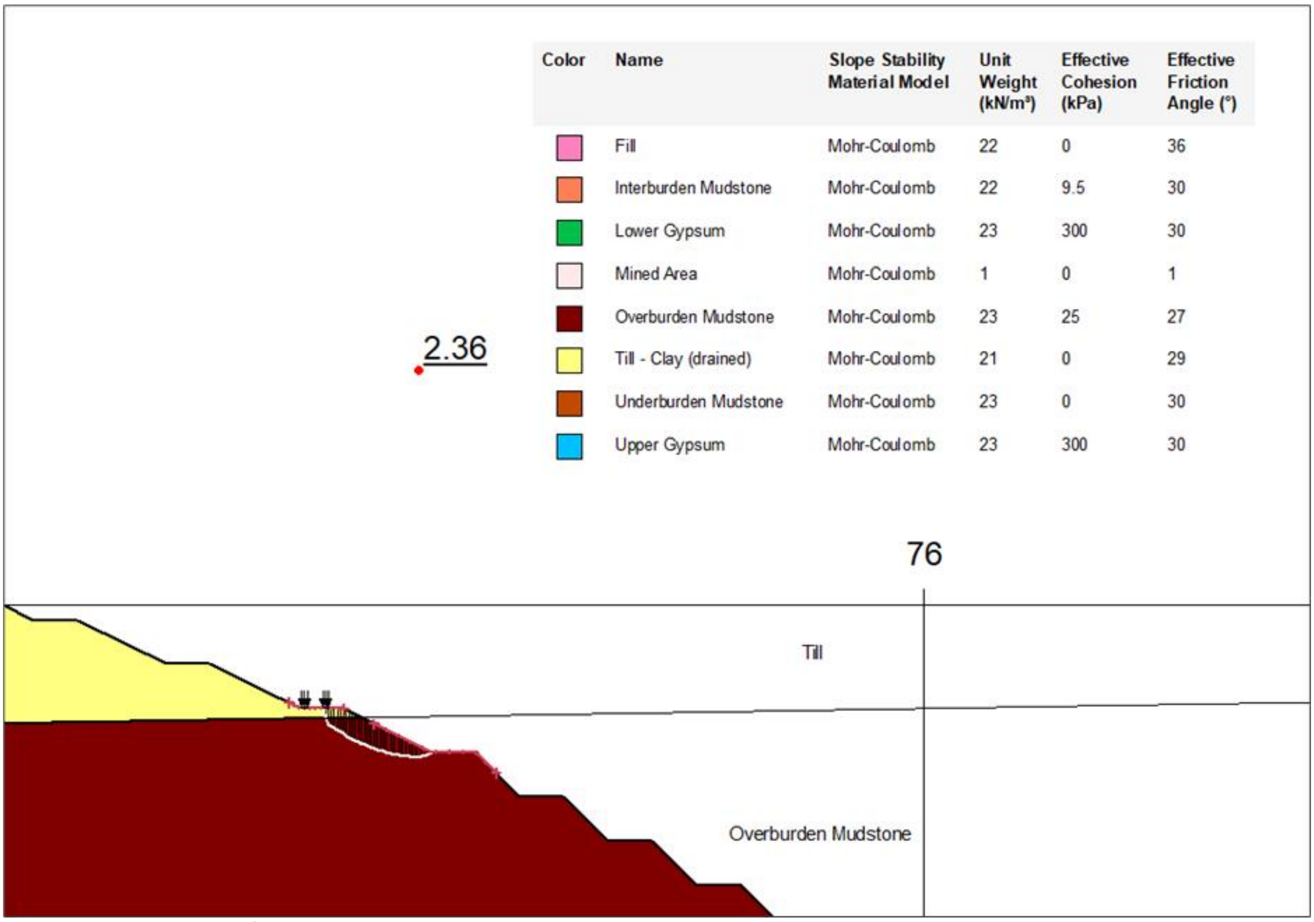

Figure 27: Section G-G', haul road on overburden mudstone, with 2(H):1(V) slope below (FoS = 2.36)

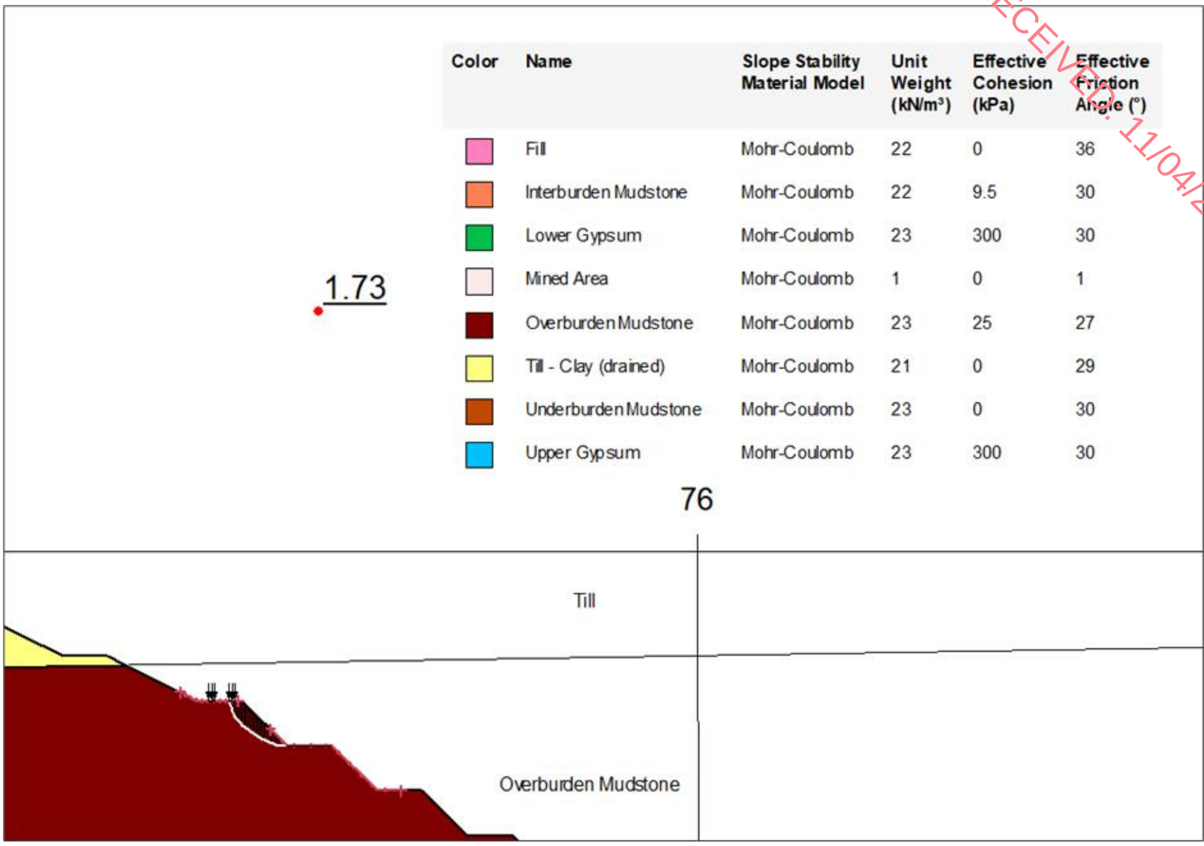

Figure 28: Section G-G', haul road on overburden mudstone with 1(H):1(V) slope below (FoS = 1.73)

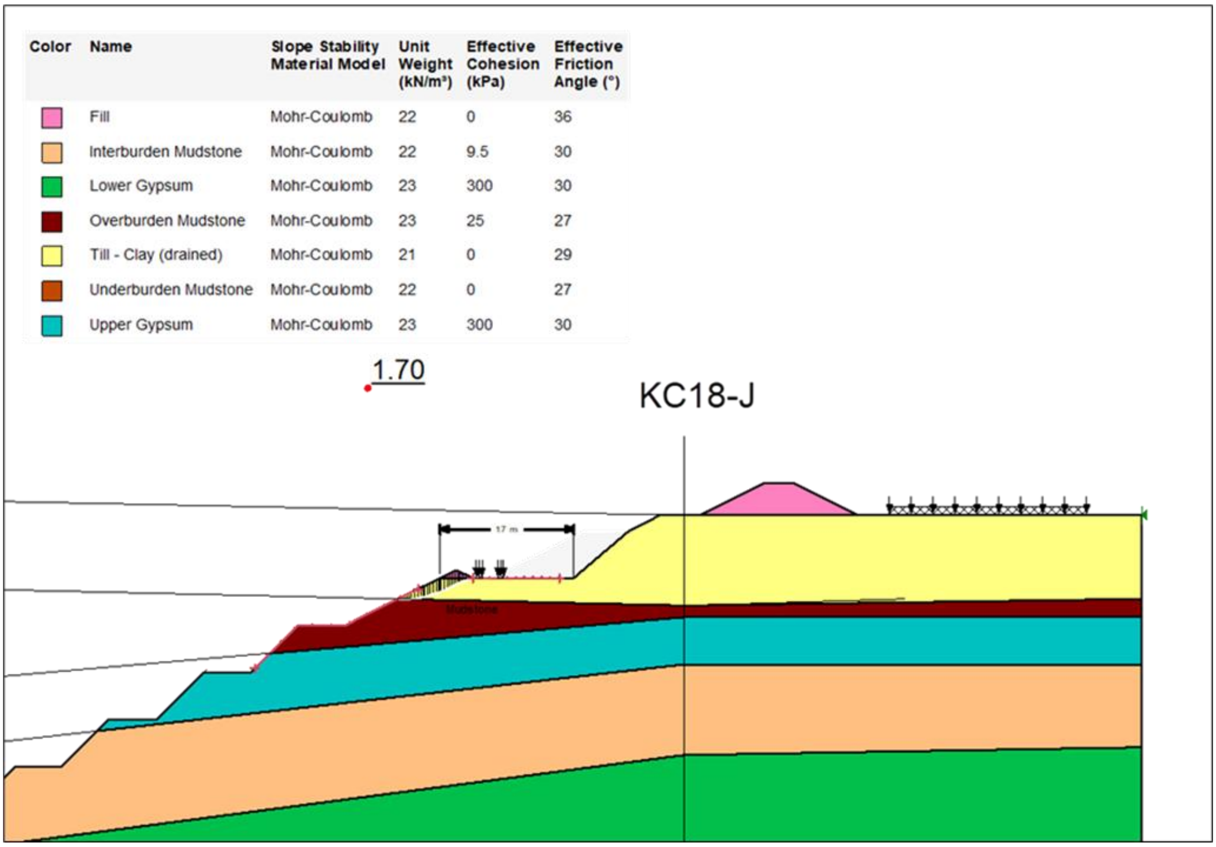

Figure 29: Section A-A', haul road on till, with 2(H):1(V) slope below (FoS = 1.70)

10.0 RISKS

The following risks may be associated with the proposed open-cast mine development:

- Weaknesses in the Lower Gypsum roof beam from the underground mining phase:
  1) Risk of failures through the layer; and
  2) Risk of sinkholes and instability to construction team.
- Risk of rising water levels from rain fall events, initial design assumes underground workings are practically fully dewatered prior to mining in Knocknacran West Open-Cast Mine;
- Upper Gypsum is more prevalent than expected and may halt excavation phases of the pit if they require blasting during contractor led stripping campaigns; and
- Planar slip failures from along the south-eastern pit wall and its northern corner. The dip of the gypsum beds out of the face may be of concern for wall stability i.e., Section A-A', Section B-B' and Section F-F' (north) and will require further analyses at detailed design stage.

11.0 CONCLUSIONS

Nine (9) no. representative long-term cross-sections around the perimeter of the proposed Knocknacran West Open-Cast Mine have been selected for stability analyses and have been assessed to meet the required design criteria for FoS.

These cross-sections have been created using a combination of logs from previous boreholes within the footprint of the pit and logs from the Golder 2018-2019 ground investigation programme. The cross-sections are developed from existing ground surface to the top of the Lower Gypsum unit.

The following slope stability cases were analysed:

- An inter bench slip surface for the different strata; and
- An overall slope slip surface.

The piezometric level is below the level of the Lower Gypsum as the operational underground Drummond Mine located to the south provides dewatering for the proposed Knocknacran West Open-Cast Mine

The results of the stability analyses are presented in Table 16 and Table 17 and the stability analyses are shown in Figure 6 to Figure 21. In summary:

- The FoS varies from 1.5 to 2.3 for the overall slope, and from 1.2 to 2.5 for the inter-bench which meets the minimum recommended design criteria FoS values.

The Knocknacran West Open-Cast Mine will be developed in specific Phases which will require detailed design of the long-term perimeter slopes and the short-term internal slopes.

These detailed designs will be optimized to extract the Lower Gypsum and maintain the required FoS and thus may have shallower or steeper overall slope gradients depending on the nature of overburden materials present in that Phase footprint.

12.0 REFERENCES

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Signature Page

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APPENDIX A

Borehole Logs and Photographs for Core

GOLDER	**Client :** Saint Gobin	**Hole No.** **KC 18-A**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 19/06/2018 **Completed :** 21/06/2018 **Logged by :** E. Sweeting	**Ground Level (mAOD) :** 1043.00 **Co-ordinates :** E 280876.0 N 300364.0

27

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
		0.00		0	0	1039.69		(3.00) 3.00	Inferred TILL
		3.00		58	0				GYPSUM (Upper) Pale orange/brown. Mottled sediments at base of lithology. Contains bands of mudstone - mostly washed away be drilling process.
		6.00		15	11	1034.69		(5.00) 8.00	
		8.00		24	15	1032.79		(1.90) 9.90	Very weak, thickly bedded, red, silty MUDSTONE, highly weathered
						1032.69		10.00	GYPSUM (Upper) Pale orange/brown. Mottled sediments at base of lithology. Contains bands of mudstone - mostly washed away be drilling process.
		10.00		24	21				Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered
		13.00		23	7			(11.52)	
		16.00		16	21				
		19.00							

33

Remarks :

Checked By:	
Scale 1:100	
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GOLDER	Client : Saint Gobin	Hole No. KC 18-A
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 19/06/2018 **Completed :** 21/06/2018 **Logged by :** E. Sweeting	**Ground Level (mAOD) :** 1043.00 **Co-ordinates :** E 280876.0 N 300364.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	TCR (%)	SCR (%)	RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
			12 50	22	32	1021.17		21.52	Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered
		21.52		10	0				Weak to moderately strong, grey, medium to coarse, decomposed DOLERITE, highly weathered
		22.00		49	16				
		25.00		9	4				
		28.00		4	0			(15.48)	
		31.00		0	0				
		34.00		0	0	1005.69		37.00	
		37.00		1	0			(6.00)	Weak, dark grey, medium sandy DOLERITE, highly to moderately weathered

Remarks :

Checked By:
Scale 1:100
GAUK - RC April 2008

GOLDER	Client : Saint Gobin	Hole No. **KC 18-A**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods : 100	**Contractor :** Irish Drilling Ltd **Date Started :** 19/06/2018 **Completed :** 21/06/2018 **Logged by :** E. Sweeting	**Ground Level (mAOD) :** 1043.00 **Co-ordinates :** E 280876.0 N 300364.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
		40.00	100	0	0	999.69		43.00	Weak, dark grey, medium sandy DOLERITE, highly to moderately weathered
		43.00	100	88	70			(25.46)	GYPSUM (Lower) Occasional beds of compacted Mudstones present.
		46.00		88	87				
		49.00		95	84				
		52.00		94	70				
		55.00		87	86				
		58.00		96	97				

Remarks :

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Scale 1:100

GAUK - RC
April 2008

GOLDER

Client : Saint Gobin	Hole No. **KC 18-A**

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 19/06/2018 **Completed :** 21/06/2018

Logged by : E. Sweeting

Ground Level (mAOD) : 1043.00

Co-ordinates : E 280876.0 N 300364.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
									GYPSUM (Lower) Occasional beds of compacted Mudstones present.
		61.00		95	72				
		64.00		93	85				
		67.00		94	86	974.23		68.46	
		68.46		20	7	972.69		(1.54) 70.00	Moderately strong, grey, fine, SILTSTONE, moderately weathered
									End of Hole at 70.00m

Remarks :

Checked By:

Scale 1:100

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GOLDER

Client : Saint Gobin

Hole No. **KC 18-B**

Site : Drumgoosat, Co. Monahan 53

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 22/06/2018 **Completed :** 27/06/2018

Logged by : E. Sweeting

Ground Level (mAOD) : 1048.00

Co-ordinates : E 280801.0 N 300308.0

CORING 77 | STRATA RECORD

WATER/ PROGRESS	INSTALLATION /BACKFILL	Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
		0.00		0.00	0.00	1044.11		(4.00) 4.00	Inferred TILL contains cobbles
		2.00		0.00	0.00				
		4.00		69.00	70.00	1040.11		(4.00) 8.00	Very weak, thickly bedded, red-brown, clayey MUDSTONE, highly weathered
		7.00		30.15	0.00				
		8.36		84.76	75.00	1036.11		(4.00) 12.00	GYPSUM (Upper)
		10.00		82	76	1035.61		12.50	Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered
						1034.61		(1.00) 13.50	GYPSUM (Upper)
		13.00		30	26	1033.11		(1.50) 15.00	Weak, thickly bedded, brown, clayey MUDSTONE, highly to moderately weathered
						1032.11		(1.00) 16.00	GYPSUM (Upper)
		16.00		60.33	30.67			(6.50)	Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered
		19.00							

Remarks :

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Scale 1:100

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GOLDER

43

Client : Saint Gobin		**Hole No.** **KC 18-B**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 22/06/2018 **Completed :** 27/06/2018 **Logged by :** E. Sweeting	**Ground Level (mAOD) :** 1048.00 **Co-ordinates :** E 280801.0 N 300308.0

27

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
			14	42	30	1025.61		22.50	Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered
		22.00		4	4				Moderatey strong, grey, medium to coarse DOLERITE, highly to moderately weathered
		25.00		2.67	0.00				
		28.00		0.00	0.00			(11.50)	
		31.00		28.67	24.67	1014.11		34.00	
		34.00		13.67	10.00	1011.11		(3.00) 37.00	Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered
		37.00		8.67	7.67			(4.00)	Moderatey strong, dark grey, medium sandy DOLERITE, moderately weathered

Remarks :

Checked By:

Scale 1:100

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Client : Saint Gobin	Hole No. KC 18-B

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 22/06/2018 **Completed :** 27/06/2018

Logged by : E. Sweeting

Ground Level (mAOD) : 1048.00

Co-ordinates : E 280801.0 N 300308.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
		40.00		20	0	1007.11		41.00	Moderatey strong, dark grey, medium sandy DOLERITE, moderately weathered
		43.00		78	68			(8.00)	GYPSUM (Lower)
		46.00		88	73	999.11		49.00	
									End of Hole at 49.00m

Remarks :

Checked By:

Scale 1:100

GAUK - RC
April 2008

18.33

GOLDER	Client : Saint Gobin	Hole No. KC 18-C
Site : Drumgoosat, Co. Monahan 20.00	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 26/06/2018 **Completed :** 28/06/2018 **Logged by :** E. Sweeting	**Ground Level (mAOD) :** 1042.00 **Co-ordinates :** E 280792.0 N 300450.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING 13.33				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
		0.00	13.33	0.00	0.00	1038.21		(4.00) 4.00	Inferred TILL
		4.00		14.67	10.67	1035.11		(3.10) 7.10	Very weak, thickly bedded, red-brown, clayey MUDSTONE, highly weathered
		7.00		66.67	66.67			(14.09)	Very weak, grey, medium to coarse, decomposed DOLERITE, highly weathered
		10.00		0.00	0.00				
		13.00		0.00	13.33				
		16.00		0.00	0.00				
		19.00							

Remarks :

Checked By:

Scale 1:100

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GOLDER	Client : Saint Gobin	Hole No. **KC 18-C**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 26/06/2018 **Completed :** 28/06/2018 **Logged by :** E. Sweeting	**Ground Level (mAOD) :** 1042.00 **Co-ordinates :** E 280792.0 N 300450.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	TCR (%)	SCR (%)	RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
			100.00	23.67	20.00	1021.02		21.19	Very weak, grey, medium to coarse, decomposed DOLERITE, highly weathered
		22.00		78.00	64.67	1017.21		(3.81) 25.00	GYPSUM (Lower) Contains bands of mudstone - mostly washed away be drilling process.
		25.00		63.67	33.33	1015.86		(1.35) 26.35	GYPSUM (Lower) - Contains red/brown interbeds of mudsone
								(18.50)	GYPSUM (Lower) Occasional beds of compacted Mudstones present.
		28.00		84.67	82.67				
		31.00		81.00	66.33				
		34.00		90.33	83.00				
		37.00		91.00	92.67				

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

Client :	Hole No.
Saint Gobin	**KC 18-C**

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 26/06/2018 **Completed :** 28/06/2018

Logged by : E. Sweeting

Ground Level (mAOD) : 1042.00

Co-ordinates : E 280792.0 N 300450.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
		40.00		96.33	97.00	997.36		44.85	GYPSUM (Lower) Occasional beds of compacted Mudstones present.
		43.00		96.00	80.00				
						997.21		45.00	Moderately strong, grey, SILTSTONE, moderately weathered
									End of Hole at 45.00m

Remarks :

Checked By:

Scale 1:100

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GOLDER

68

Client :	Hole No.
Saint Gobin	**KC 18-D**

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 06/07/2018 **Completed :** 10/07/2018

Logged by : E. Sweeting

Ground Level (mAOD) : 1053.00

Co-ordinates : E 280615.0 N 300501.0

20

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	CORING: TCR (%)	CORING: SCR (%)	CORING: RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
		0.00	100	0	0	1051.23		(2.00) 2.00	Inferred TILL
						1049.41		(1.82) 3.82	Inferred TILL contains cobbles
		4.00		16	16				
		7.00		42	42	1043.23		(6.18) 10.00	Very weak, thickly bedded, red-brown, clayey MUDSTONE, highly weathered
		10.00		74	71				Very weak, thickly bedded, brown, clayey MUDSTONE, highly to moderately weathered
		13.00		37	19				
		16.00		29	18	1034.51		(8.72) 18.72	
		18.72		75	75				
		19.00		100	100	1033.61		(0.90) 19.62	GYPSUM (Upper)
		19.62							

(see next page)

Remarks :

Checked By:

Scale 1:100

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GOLDER	Client : Saint Gobin	Hole No. **KC 18-D**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 06/07/2018 **Completed :** 10/07/2018 **Logged by :** E. Sweeting	**Ground Level (mAOD) :** 1053.00 **Co-ordinates :** E 280615.0 N 300501.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	TCR (%)	SCR (%)	RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
			94	47	26				Weak, thickly bedded, brown, clayey MUDSTONE, highly to moderately weathered
		22.00	104	47	28			(8.64)	
		25.00		35	16				
		28.00		100	100	1024.97		28.26	
		28.20		5	0			(2.41)	Weak to moderately strong, grey, medium to coarse, decomposed DOLERITE, highly weathered
		30.67		100	100	1022.56		30.67	GYPSUM (Lower)
		31.00		78	67				
		34.00		98	90				
		37.00		88	94				
		38.81		91	66				

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER	Client : Saint Gobin	Hole No. **KC 18-D**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods : 100	**Contractor :** Irish Drilling Ltd **Date Started :** 06/07/2018 **Completed :** 10/07/2018 **Logged by :** E. Sweeting	**Ground Level (mAOD) :** 1053.00 **Co-ordinates :** E 280615.0 N 300501.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
		40.00		94	26			(24.18)	GYPSUM (Lower)
		43.00		83	57				
		46.00		93	90				
		49.00		100	80				
		52.00		100	80	998.38		54.85	
						998.23		55.00	Moderately strong, grey, fine, SILTSTONE, moderately weathered
									End of Hole at 55.00m

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER	Client : Saint Gobin	Hole No. **KC 18-E**
Site : Drumgoosat, Co. Monahan 100	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods : 48	**Contractor :** Irish Drilling Ltd **Date Started :** 11/07/2018 **Completed :** 13/07/2018 **Logged by :** E. Sweeting/A.Crowley	**Ground Level (mAOD) :** 1060.00 **Co-ordinates :** E 280476.0 N 300728.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	TCR (%)	SCR (%)	RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
			51					(3.00)	No Return
						1057.28		3.00	
			56					(3.65)	Inferred TILL contains cobbles
			100						
						1053.63		6.65	
		6.65	33	100	100				Very weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered
		7.00	100	45	46				
		10.00		37	31				
		13.00		38	23				
								(18.13)	
		16.00		63	18				
		16.76		7	0				
		18.68		100	100				
		19.00							

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER	Client : Saint Gobin	Hole No. **KC 18-E**
67		
Site : Drumgoosat, Co. Monahan 100	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods : 100	**Contractor :** Irish Drilling Ltd **Date Started :** 11/07/2018 **Completed :** 13/07/2018 **Logged by :** E. Sweeting/A.Crowley	**Ground Level (mAOD) :** 1060.00 **Co-ordinates :** E 280476.0 N 300728.0

CORING

37

WATER/ PROGRESS	INSTALLATION /BACKFILL	Top of Core Run	TCR (%)	SCR (%)	RQD (%)
			51	4	0
		21.34		77	47
		22.00		23	8
		24.78		100	100
		25.00		90	92
		28.00		92	84
		31.00		93	98
		32.63		43	0
		32.93		88	76
		34.00		89	94
		35.36		30	18
		37.00		23	0

STRATA RECORD

Level (mAOD)	Legend	Depth (Thickness) m	Description
1035.50		24.78	Very weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered
1027.63		(7.87) 32.65	GYPSUM (Upper)
1027.35		32.93	Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered
1024.92		(2.43) 35.36	GYPSUM (Upper)
			Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered

Remarks :

Checked By:
Scale 1:100
GAUK - RC April 2008

GOLDER

Client : Saint Gobin	Hole No. **KC 18-E**

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

100

Contractor : Irish Drilling Ltd

Date Started : 11/07/2018 **Completed :** 13/07/2018

Logged by : E. Sweeting/A.Crowley

Ground Level (mAOD) : 1060.00

Co-ordinates : E 280476.0 N 300728.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	CORING: TCR (%)	CORING: SCR (%)	CORING: RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
		40.00	100					(11.24)	Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered
		43.00	100	69	39				
		46.00	100	100	89	1013.68		46.60	
									GYPSUM (Lower)
		49.00		99	76				
		52.00		93	54				
		55.00		100	95			(16.90)	
		58.00		100	94				

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

Client : Saint Gobin	Hole No. **KC 18-E**

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 11/07/2018 **Completed :** 13/07/2018

Logged by : E. Sweeting/A.Crowley

Ground Level (mAOD) : 1060.00

Co-ordinates : E 280476.0 N 300728.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	TCR (%)	SCR (%)	RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
									GYPSUM (Lower)
		61.00		100	66	996.78		63.50	
		64.00		83	22	993.28		(3.50) 67.00	Moderately strong, grey, finely bedded SILTSTONE, moderately weathered
									End of Hole at 67.00m

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER	Client : Saint Gobin	Hole No. **KC 18-F**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 03/09/2018 **Completed :** 06/09/2018 **Logged by :** A.Crowley	**Ground Level (mAOD) :** 1055.00 **Co-ordinates :** E 280680.0 N 300025.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
						1056.41		(2.00) 2.00	Inferred TILL contains cobbles
						1039.41		(17.00) 19.00	Very weak, thickly bedded, red-brown, fine, clayey MUDSTONE, highly weathered
		19.00							Very weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered @31 to 37 m Contains fine <20cm Gypsum lenses.

Remarks :

Checked By:
Scale 1:100
GAUK - RC April 2008

GOLDER

63

Client : Saint Gobin	Hole No. KC 18-F

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 03/09/2018 **Completed :** 06/09/2018

Logged by : A.Crowley

Ground Level (mAOD) : 1055.00

Co-ordinates : E 280680.0 N 300025.0

60

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	CORING: TCR (%)	CORING: SCR (%)	CORING: RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
			63						Very weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered @31 to 37 m Contains fine <20cm Gypsum lenses.
		22.00							
		25.00							
		28.00						(23.70)	
		31.00							
		34.00							
		37.00							

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

Client :	Hole No.
Saint Gobin	**KC 18-F**

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

57

Contractor : Irish Drilling Ltd

Date Started : 03/09/2018 **Completed :** 06/09/2018

Logged by : A.Crowley

Ground Level (mAOD) : 1055.00

Co-ordinates : E 280680.0 N 300025.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
		40.00	50			1015.71		42.70	Very weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered @31 to 37 m Contains fine <20cm Gypsum lenses.
		43.00	80						
		46.00				1006.41		(9.30) 52.00	GYPSUM (Upper) Grey, slightly weathered
		49.00							
		52.00						(14.00)	Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered 30cm grey mudstone along upper boundary with Gypsum
		55.00							
		58.00							

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

Client : Saint Gobin		**Hole No.** KC 18-F
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 03/09/2018 **Completed :** 06/09/2018 **Logged by :** A.Crowley	**Ground Level (mAOD) :** 1055.00 **Co-ordinates :** E 280680.0 N 300025.0

97

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
			100						Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered 30cm grey mudstone along upper boundary with Gypsum
		61.00	100						
		64.00				992.41		66.00	
								(2.00)	GYPSUM (Lower) Occasional beds of compacted Mudstones present.
		67.00				990.41		68.00	
								(2.00)	GYPSUM (Lower) Contains bands of mudstone - mostly washed away be drilling process.
						988.41		70.00	
		70.00							GYPSUM (Lower)
		73.00							
		76.00						(12.50)	
		79.00							

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

Client : Saint Gobin	Hole No. KC 18-F

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 03/09/2018 **Completed :** 06/09/2018

Logged by : A.Crowley

Ground Level (mAOD) : 1055.00

Co-ordinates : E 280680.0 N 300025.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
						975.91		82.50	GYPSUM (Lower)
		82.00				973.41		(2.50) 85.00	Moderately strong, grey, fine, SILTSTONE, moderately weathered
									End of Hole at 85.00m

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

Client : Saint Gobin	Hole No. KC 18-G

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 07/09/2018 **Completed :** 12/09/2018

Logged by : A.Crowley

Ground Level (mAOD) : 1054.00

Co-ordinates : E 280627.0 N 300129.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	CORING: TCR (%)	CORING: SCR (%)	CORING: RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
								(4.50)	Inferred TILL contains cobbles
						1049.80		4.50	
			90						Very weak, thickly bedded, red-brown, clayey MUDSTONE, highly weathered
			43					(12.00)	
		12.00	47						
		13.00							
		16.00				1037.80		16.50	Very weak, thickly bedded, brown, clayey MUDSTONE, highly to moderately weathered
		19.00							

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

83

Client : Saint Gobin	Hole No. **KC 18-G**

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 07/09/2018 **Completed :** 12/09/2018

Logged by : A.Crowley

Ground Level (mAOD) : 1054.00

Co-ordinates : E 280627.0 N 300129.0

RECEIVED: 11/04/2023

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	CORING: TCR (%)	CORING: SCR (%)	CORING: RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
			97						
			67					(12.50)	Very weak, thickly bedded, brown, clayey MUDSTONE, highly to moderately weathered
		22.00							
		25.00							
		28.00				1025.30		29.00	
		31.00						(9.50)	Very weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered Occasional Gypsum lenses
		34.00							
		37.00				1015.80		38.50	
								(3.00)	Weak, thickly bedded, brown, clayey MUDSTONE, highly to moderately weathered

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

Client : Saint Gobin

Hole No. **KC 18-G**

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

100

Contractor : Irish Drilling Ltd

Date Started : 07/09/2018 **Completed :** 12/09/2018

Logged by : A.Crowley

Ground Level (mAOD) : 1054.00

Co-ordinates : E 280627.0 N 300129.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	CORING: TCR (%)	CORING: SCR (%)	CORING: RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
		40.00	87			1012.80		41.50	Weak, thickly bedded, brown, clayey MUDSTONE, highly to moderately weathered
		43.00	83						
		46.00							
		49.00				1003.90		(8.90) 50.40	GYPSUM (Upper)
						1002.30		(1.60) 52.00	GYPSUM (Upper) Contains moderatey weak, thickly bedded, dark red to brown, fine, clayey mudstone, highly to moderately weathered
		52.00				999.30		(3.00) 55.00	GYPSUM (Upper)
		55.00							Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered Occasional grey mudstone bands and <0.5m gypsum lenses
		58.00							

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

67

GOLDER	Client : Saint Gobin	Hole No. **KC 18-G**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 07/09/2018 **Completed :** 12/09/2018 **Logged by :** A.Crowley	**Ground Level (mAOD) :** 1054.00 **Co-ordinates :** E 280627.0 N 300129.0

90

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	CORING: TCR (%)	CORING: SCR (%)	CORING: RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
			100						Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered Occasional grey mudstone bands and <0.5m gypsum lenses
		61.00	100					(14.00)	
		64.00							
		67.00				985.30		69.00	
								(2.00)	Weak, grey, medium to coarse, decomposed DOLERITE, highly weathered
		70.00				983.30		71.00	GYPSUM (Lower)
		73.00							
		76.00							
		79.00						(16.00)	

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

Client :	Hole No.
Saint Gobin	KC 18-G

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 07/09/2018 **Completed :** 12/09/2018

Logged by : A.Crowley

Ground Level (mAOD) : 1054.00

Co-ordinates : E 280627.0 N 300129.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	CORING: TCR (%)	CORING: SCR (%)	CORING: RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
									GYPSUM (Lower)
		82.00							
		85.00				967.30		87.00	
		87.00				966.30		(1.00) 88.00	Weak to moderately strong, grey, medium to coarse, decomposed DOLERITE, highly weathered
									End of Hole at 88.00m

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

Client :	Hole No.
Saint Gobin	**KC 18-H**

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 13/09/2018 **Completed :** 19/09/2018

Logged by : A.Crowley

Ground Level (mAOD) : 1053.00

Co-ordinates : E 280521.0 N 300264.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
						1048.78		(20.00)	Inferred TILL
									Very weak, thickly bedded, red-brown, clayey MUDSTONE, highly weathered. Minor silt lenses

Remarks :

Checked By:
Scale 1:100
GAUK - RC April 2008

GOLDER

Client : Saint Gobin	Hole No. KC 18-H
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision
Project No : 18104447	

67

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 13/09/2018 **Completed :** 19/09/2018

Logged by : A.Crowley

Ground Level (mAOD) : 1053.00

Co-ordinates : E 280521.0 N 300264.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	TCR (%)	SCR (%)	RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
			60						Very weak, thickly bedded, red-brown, clayey MUDSTONE, highly weathered. Minor silt lenses
		21.00							
		22.00	83						
		25.00				1028.78		26.00	
									Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered. Occasional Gypsum lenses
		28.00						(8.00)	
		31.00				1020.78		34.00	
		34.00							Very weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered
		37.00						(9.70)	

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

Client : Saint Gobin	Hole No. **KC 18-H**

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

100

Contractor : Irish Drilling Ltd

Date Started : 13/09/2018 **Completed :** 19/09/2018

Logged by : A.Crowley

Ground Level (mAOD) : 1053.00

Co-ordinates : E 280521.0 N 300264.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
		40.00	100			1011.08		43.70	Very weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered
		43.00	83						
		46.00						(7.30)	GYPSUM (Upper)
		49.00				1003.78		51.00	
						1003.18		51.60	Weak, thickly bedded, brown, clayey MUDSTONE, highly to moderately weathered
		52.00				999.78		(3.40) 55.00	GYPSUM (Upper)
		55.00							Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered Occasional Gypsum lenses
		58.00						(7.80)	

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

100

GOLDER	Client : Saint Gobin	Hole No. **KC 18-H**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 13/09/2018 **Completed :** 19/09/2018 **Logged by :** A.Crowley	**Ground Level (mAOD) :** 1053.00 **Co-ordinates :** E 280521.0 N 300264.0

100

RECEIVED: 11/04/2023

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
			100			991.98		62.80	Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered Occasional Gypsum lenses
		61.00	100						
		64.00							GYPSUM (Lower)
		67.00							
		70.00							
		73.00							
		76.00						(30.40)	
		79.00							

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

83

Client : Saint Gobin	**Hole No.** **KC 18-H**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision
Project No : 18104447	
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 13/09/2018 **Completed :** 19/09/2018 **Logged by :** A.Crowley
Ground Level (mAOD) : 1053.00 **Co-ordinates :** E 280521.0 N 300264.0	

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING Top of Core Run	TCR (%)	SCR (%)	RQD (%)	STRATA RECORD Level (mAOD)	Legend	Depth (Thickness) m	Description
			50						GYPSUM (Lower)
		82.00							
		85.00							
		88.00							
		91.00				961.58		93.20	
		94.00						(4.80)	Weak, dark grey, medium sandy DOLERITE, highly to moderately weathered
		97.00				956.78		98.00	
									End of Hole at 98.00m

67

Remarks :

Checked By:	
Scale 1:100	
GAUK - RC April 2008	

GOLDER	Client : Saint Gobin	Hole No. **KC 18-I**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** **Completed :** **Logged by :**	**Ground Level (mAOD) :** **Co-ordinates :** E N

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
						1052.50			NOT COMPLETE DUE TO SINK HOLE
									End of Hole at 2.00m

Remarks :	**Checked By:**
	Scale 1:100
	GAUK - RC April 2008

Client :	Hole No.
Saint Gobin	KC 18-J

GOLDER

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 01/11/2011 **Completed :** 13/11/2018

Logged by : A.Crowley

Ground Level (mAOD) : 1048.00

Co-ordinates : E 280792.0 N 299830.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
			50 95			1038.47		(12.00) 12.00	No Returns, inferred Mudsone
		12.00	98	20	60	1036.97		(1.50) 13.50	Very weak, thickly bedded, red-brown, clayey MUDSTONE, highly weathered
		13.00		65	33				GYPSUM (Upper)
		16.00		76	33	1030.97		(6.00) 19.50	
		19.00							Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

32

Client : Saint Gobin

Hole No. **KC 18-J**

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 01/11/2011 **Completed :** 13/11/2018

Logged by : A.Crowley

Ground Level (mAOD) : 1048.00

Co-ordinates : E 280792.0 N 299830.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	CORING: TCR (%)	CORING: SCR (%)	CORING: RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
			98						
			91	35	17				Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered
		22.00		40	23				
		25.00		43	19			(11.50)	
		28.00		54	32	1019.47		31.00	
		31.00		30	26				GYPSUM (Lower)
		34.00		70	42				
		37.00		61	13			(14.50)	

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER	Client : Saint Gobin	Hole No. **KC 18-J**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 01/11/2011 **Completed :** 13/11/2018 **Logged by :** A.Crowley	**Ground Level (mAOD) :** 1048.00 **Co-ordinates :** E 280792.0 N 299830.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
		40.00		83	47				GYPSUM (Lower)
		43.00		85	52	1004.97		45.50	
									End of Hole at 45.50m

Remarks :

Checked By:
Scale 1:100
GAUK - RC April 2008

GOLDER	Client : Saint Gobin	Hole No. KC 18-K
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 01/11/2011 **Completed :** 13/11/2018 **Logged by :** A.Crowley	**Ground Level (mAOD) :** 1053.00 **Co-ordinates :** E 280882.0 N 299946.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
			100			1036.78		(16.60) 16.60	No Returns, Inferrered Mudstone
		16.60		43	43	1035.28		(1.50) 18.10	GYPSUM (Upper)
		17.60	93	72	67	1034.88		18.50	Very weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered
		19.10						(2.50)	GYPSUM (Upper)

Remarks :

Checked By:
Scale 1:100
GAUK - RC April 2008

GOLDER	Client : Saint Gobin	Hole No. KC 18-K
Site : Drumgoosat, Co. Monahan	Project : Saint Gobian Drumgoosat Geotechnial Supervision	Project No : 18104447
Equipment & Methods :	Contractor : Irish Drilling Ltd Date Started : 01/11/2011 Completed : 13/11/2018 Logged by : A.Crowley	Ground Level (mAOD) : 1053.00 Co-ordinates : E 280882.0 N 299946.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
				88	62	1032.38		21.00	GYPSUM (Upper)
		20.60		47	17				Weak, thickly bedded, dark red to brown,clayey MUDSTONE, highly to moderately weathered
		22.10		0	0				
		23.60		47	30				
		25.10		0	0			(10.70)	
		26.60		0	0				
		28.10		0	0				
		29.60		87	75				
		31.10		93	93	1021.68		31.70	GYPSUM (Lower)
								(9.50)	

Remarks :	Checked By:
	Scale 1:100
	GAUK - RC April 2008

GOLDER

Client : Saint Gobin	Hole No. **KC 18-K**

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 01/11/2011 **Completed :** 13/11/2018

Logged by : A.Crowley

Ground Level (mAOD) : 1053.00

Co-ordinates : E 280882.0 N 299946.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	TCR (%)	SCR (%)	RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
						1012.18			GYPSUM (Lower)
									End of Hole at 41.20m

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

Client : Saint Gobin	Hole No. KC 18-L

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 01/11/2011 **Completed :** 13/11/2018

Logged by : A.Crowley

Ground Level (mAOD) : 1053.00

Co-ordinates : E 280868.0 N 299930.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	TCR (%)	SCR (%)	RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
								(16.70)	No Returns, Inferred Mudstone
			96			1036.08		16.70	
		16.70	69					(1.30)	GYPSUM (Upper)
		17.10		93 / 51	82 / 17	1034.78		18.00	
		17.60	100	89	45			(1.30)	Very weak, thickly bedded, brown, clayey MUDSTONE, highly to moderately weathered
						1033.48		19.30	
									GYPSUM (Upper)

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

	Client : Saint Gobin	**Hole No.** **KC 18-L**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods : 100	**Contractor :** Irish Drilling Ltd **Date Started :** 01/11/2011 **Completed :** 13/11/2018 **Logged by :** A.Crowley	**Ground Level (mAOD) :** 1053.00 **Co-ordinates :** E 280868.0 N 299930.0

RECEIVED: 11/04/2023

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
								(2.00)	GYPSUM (Upper)
		20.60		86	75	1031.48		21.30	
		22.10		40	17				Extremely weak to very weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered
		23.60		21	21				
		25.10		69	50			(10.50)	
		26.70		86	64				
		28.10		19	8				
		29.60		80	87				
		31.10		89	71	1020.98		31.80	GYPSUM (Lower)
		32.60		87	87				
								(9.80)	

Remarks :

Checked By:
Scale 1:100
GAUK - RC April 2008

GOLDER

Client : Saint Gobin		**Hole No.** **KC 18-L**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 01/11/2011 **Completed :** 13/11/2018 **Logged by :** A.Crowley	**Ground Level (mAOD) :** 1053.00 **Co-ordinates :** E 280868.0 N 299930.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
						1011.18			GYPSUM (Lower)
									End of Hole at 41.60m

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

Client :	Hole No.
Saint Gobin	**KC 18-M**

Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 01/12/2018 **Completed :** 01/12/2018 **Logged by :** E. Sweeting	**Ground Level (mAOD) :** 1053.00 **Co-ordinates :** E 280521.0 N 300263.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
								(24.00)	No Returns, Inferred Mudstone

Remarks :

Checked By:
Scale 1:100
GAUK - RC April 2008

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GOLDER	**Client :** Saint Gobin	**Hole No.** **KC 18-M**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 01/12/2018 **Completed :** 01/12/2018 **Logged by :** E. Sweeting	**Ground Level (mAOD) :** 1053.00 **Co-ordinates :** E 280521.0 N 300263.0

84

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	CORING: TCR (%)	CORING: SCR (%)	CORING: RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
			93 91			1029.40		24.00	No Returns, Inferred Mudstone
		24.00	56	31	23				Very weak to weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered occasional gypsum bands <10mm thick.
		27.00		47	15				
		30.00		71	29				
		33.00		82	45				
		36.00		45	23			(26.42)	
		39.00							

Remarks :

Checked By:
Scale 1:100
GAUK - RC April 2008

93

GOLDER

96

Client : Saint Gobin

Hole No. **KC 18-M**

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 01/12/2018 **Completed :** 01/12/2018

Logged by : E. Sweeting

Ground Level (mAOD) : 1053.00

Co-ordinates : E 280521.0 N 300263.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
			95						
			97	54	19				Very weak to weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered occasional gypsum bands <10mm thick.
		42.00		77	43				
		45.00		62	31				
		48.00		55	27	1002.98		50.42	
		50.42		88	83				GYPSUM (Upper) Abundant mudstole banding along upper and lower contacts
		51.00		92	91				
		54.00		94	92			(9.58)	
		57.00		96	71	993.40		60.00	

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

Client : Saint Gobin	Hole No. KC 18-M

Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods : 100	**Contractor :** Irish Drilling Ltd **Date Started :** 01/12/2018 **Completed :** 01/12/2018 **Logged by :** E. Sweeting	**Ground Level (mAOD) :** 1053.00 **Co-ordinates :** E 280521.0 N 300263.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
		60.00	100	65	44				Weak, thickly bedded, dark red to brown, clayey MUDSTONE, highly to moderately weathered occasional gypsum bands <10mm thick.
		63.00	100	61	32			(9.00)	
		66.00		94	78	984.40		69.00	
		69.00		100	92				GYPSUM (Lower) Beds of compacted Mudstones present especially at base of lithology.
		72.00		100	77				
		75.00		100	88			(18.00)	
		78.00		100	94				

Remarks :

Checked By:
Scale 1:100
GAUK - RC April 2008

GOLDER

Client : Saint Gobin

Hole No. **KC 18-M**

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 01/12/2018 **Completed :** 01/12/2018

Logged by : E. Sweeting

Ground Level (mAOD) : 1053.00

Co-ordinates : E 280521.0 N 300263.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	CORING: TCR (%)	CORING: SCR (%)	CORING: RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
									GYPSUM (Lower) Beds of compacted Mudstones present especially at base of lithology.
		81.00		100	88				
		84.00		100	64	966.40		87.00	
		87.00		100	20	963.40		(3.00) 90.00	Weak to medium strong, grey, fine, SILTSTONE, moderately weathered
									End of Hole at 90.00m

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

Client : Saint Gobin

Hole No. **KC 18-N**

Site : Drumgoosat, Co. Monahan 92

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods : 100

Contractor : Irish Drilling Ltd

Date Started : 01/12/2018 **Completed :** 01/12/2018

Logged by : E. Sweeting

Ground Level (mAOD) : 1052.00

Co-ordinates : E 280803.0 N 299932.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	TCR (%) 76	SCR (%)	RQD (%)
		0.00		0	0
		1.50	84	0	0
		3.00		0	0
		4.50		0	0
		6.00		0	0
		7.50		0	0
		9.00		0	0
		10.50		0	0
		12.00		0	0
		13.50		0	0
		15.00		43	13
		18.00		66	31
		19.35			

STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
1048.92		(1.50) 1.50	Inferred TILL contains cobbles
1038.42		(10.50) 12.00	Very weak, red, fine, SILTSTONE, highly weathered.
1031.07		(7.35) 19.35	Very weak, thickly bedded, red-brown, fine, clayey MUDSTONE, highly weathered
			GYPSUM (Upper)

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER

Client : Saint Gobin	Hole No. KC 18-N

Site : Drumgoosat, Co. Monahan

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods :

Contractor : Irish Drilling Ltd

Date Started : 01/12/2018 **Completed :** 01/12/2018

Logged by : E. Sweeting

Ground Level (mAOD) : 1052.00

Co-ordinates : E 280803.0 N 299932.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
				96	78	1029.42		(1.65) 21.00	GYPSUM (Upper)
									End of Hole at 21.00m

Remarks :

Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER	**Client :** Saint Gobin	**Hole No.** **KC 18-O**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods : 83 38	**Contractor :** Irish Drilling Ltd **Date Started :** 01/12/2018 **Completed :** 01/12/2018 **Logged by :** E. Sweeting	**Ground Level (mAOD) :** 1046.00 **Co-ordinates :** E 280894.0 N 300550.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	CORING: TCR (%)	CORING: SCR (%)	CORING: RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
		0.00	29	0	0	1044.16		(3.00) 3.00	Inferred TILL contains cobbles
		3.00	50	10	5	1041.16		(3.00) 6.00	Very weak, red, fine, SILTSTONE, highly weathered No solid return
		6.00		30	12	1038.16		(3.00) 9.00	Very weak, thickly bedded, red, fine, silty MUDSTONE, highly weathered
		9.00		57	37	1033.66		(4.50) 13.50	Very weak, thickly bedded, red-brown, fine, clayey MUDSTONE, highly weathered Occasional thin, <10mm, grey bands
		12.00		0	0				
		13.50		0	0				Weak, grey, medium to coarse, Decomposed DOLERITE, highly weathered Weak sands mostly washed away by drillling, remianing rubble are of medium hardness.
		15.00		0	0				
		18.00		0	0				

Remarks :

Checked By:	
Scale 1:100	
GAUK - RC April 2008	

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GOLDER

Client : Saint Gobin

Hole No. **KC 18-O**

Site : Drumgoosat, Co. Monahan 29

Project : Saint Gobian Drumgoosat Geotechnial Supervision

Project No : 18104447

Equipment & Methods : 100

Contractor : Irish Drilling Ltd

Date Started : 01/12/2018 **Completed :** 01/12/2018

Logged by : E. Sweeting

Ground Level (mAOD) : 1046.00

Co-ordinates : E 280894.0 N 300550.0

RECEIVED: 11/04/2023

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING: Top of Core Run	TCR (%)	SCR (%)	RQD (%)	STRATA RECORD: Level (mAOD)	Legend	Depth (Thickness) m	Description
									Weak, grey, medium to coarse, Decomposed DOLERITE, highly weathered Weak sands mostly washed away by drillling, remianing rubble are of medium hardness.
		21.00		0	0			(18.52)	
		24.00		0	0				
		27.00		0	0				
		30.00		8	8	1015.14		32.02	
		32.02		100	28	1014.16		(0.98) 33.00	GYPSUM (Lower)
									End of Hole at 33.00m

Remarks :

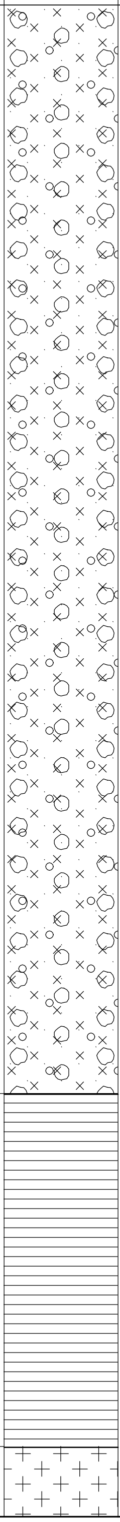
Checked By:

Scale 1:100

GAUK - RC April 2008

GOLDER	Client : Saint Gobin	Hole No. **KC 18-P**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods : 4	**Contractor :** Irish Drilling Ltd **Date Started :** 01/12/2018 **Completed :** 01/12/2018 **Logged by :** E. Sweeting	**Ground Level (mAOD) :** 1054.00 **Co-ordinates :** E 280675.0 N 300601.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
		0.00	100 100	0	0				Brown-grey, sandy gravelly SILT (TILL)
		3.00		0	0				
		6.00		0	0			(14.40)	
		9.00		0	0				
		12.00		0	0	1043.06		14.40	
		14.40		100	75				Weak, thickly bedded, red-brown, fine, clayey MUDSTONE, highly weathered grey banding present, occasional <50mm gypsum beds
		15.00		95	45			(4.67)	
		18.00		100	68	1038.39		19.07	
		19.07							Weak to medium strong, grey, medium to coarse DOLERITE, highly to moderately weathered Sandy matrix mostly washed away during drilling



Remarks :

Checked By:
Scale 1:100
GAUK - RC April 2008

GOLDER	Client : Saint Gobin	Hole No. **KC 18-P**
Site : Drumgoosat, Co. Monahan	**Project :** Saint Gobian Drumgoosat Geotechnial Supervision	**Project No :** 18104447
Equipment & Methods :	**Contractor :** Irish Drilling Ltd **Date Started :** 01/12/2018 **Completed :** 01/12/2018 **Logged by :** E. Sweeting	**Ground Level (mAOD) :** 1054.00 **Co-ordinates :** E 280675.0 N 300601.0

WATER/ PROGRESS	INSTALLATION /BACKFILL	CORING				STRATA RECORD			
		Top of Core Run	TCR (%)	SCR (%)	RQD (%)	Level (mAOD)	Legend	Depth (Thickness) m	Description
				4	0				Weak to medium strong, grey, medium to coarse DOLERITE, highly to moderately weathered Sandy matrix mostly washed away during drilling
		21.00		3	0	1034.46		(3.93) 23.00	
		23.00		100	83	1033.46		(1.00) 24.00	GYPSUM (Lower)
									End of Hole at 24.00m

Remarks :

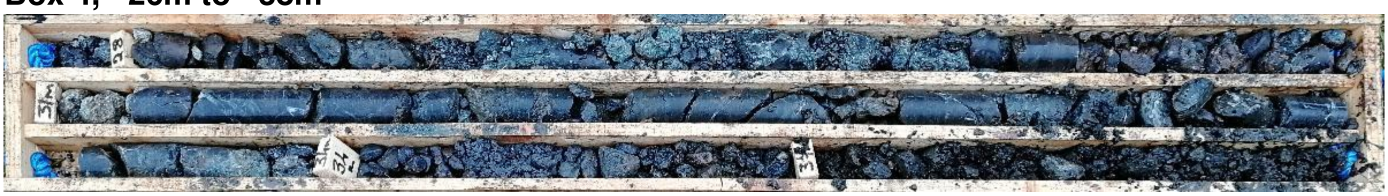
Checked By:	
Scale 1:100	
GAUK - RC April 2008	

Box 1, 3m to ~11.5m

Box 2, ~11.5m to 22m

Box 3, 22m to ~26m

Box 4, ~26m to ~38m



Box 5, ~38m to 44.5m

Box 6, 44.5m to 49m

Box 7, 49m to 53m

Box 8, 53m to 57.5m

Box 9, 57.5m to 62m

Box 10, 62m to 66m

Box 11, 66m to 70m (End Of Hole)

Box 1, 2m to ~9m

Box 2, ~9m to ~15m

Box 3, ~15m to ~21m

Box 4, ~21m to ~36m

Box 5, ~36m to 43.5m

Box 6, 43.5m to ~47.5m

Box 7, ~47.5m to 52m

Box 1, 4m to ~20.5m

Box 2, ~20.5m to 25.5m

Box 3, 25.5m to 29m

Box 4, 29m to 33.5m

Box 5, 33.5m to 38m

Box 6, 38m to 42m

Box 7, 42m to 45m

Box 1, 3.5m to 11.5m

Box 2, 11.5m to 19.5m

Box 3, 19.5m to 25m

Box 4, 25m to 31.5m

Box 5, 31.5m to 35.5m

Box 6, 35.5m to 40.5m

Box 7, 40.5m to 44.5m

Box 8, 44.5m to 48.5m

Box 9, 48.5m to 53m

Box 10, 53m to 55m (End Of Hole)

Box 1, 3m to 13m

Box 2, 13m to 19m

Box 3, 19m to 26.5m

Box 4, 26.5m to 30.5m

Box 5, 30.5m to 35m

Box 6, 35m to 42m

Box 7, 42m to 47 m

Box 8, 47m to 50.5m

Box 9, 50.5m to 55m

Box 10, 55m to 59m

Box 11, 59m to 63m

Box 12, 63m to 67m (End Of Hole)

Box 1, 21m to 26m

Box 2, 26m to 33m

Box 3, 33m to 40.5m

Box 4, 40.5m to 44.5m

Box 5, 44.5m to 49.5m

Box 6, 49.5m to 55m

Box 7, 55m to 62m

Box 8, 62m to 68m

Box 9, 68m to 72m

Box 10, 72m to 76.5m

Box 11, 76.5m to 80.5m

Box 12, 80.5m to 85m (End Of Hole)

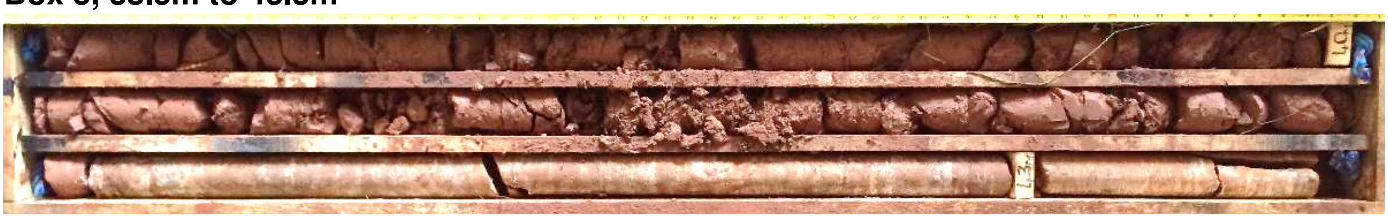
Box 1, 12m to 21m

Box 2, 21m to 28m

Box 3, 28m to 33.5m

Box 4, 33.5m to 38.5m

Box 5, 38.5m to 43.5m



Box 6, 43.5m to 47.5m

Box 7, 47.5m to 52m

Box 8, 52m to 56.5m

Box 9, 56.5m to 62m

Box 10, 62m to 67.5m

Box 11, 67.5m to 73m

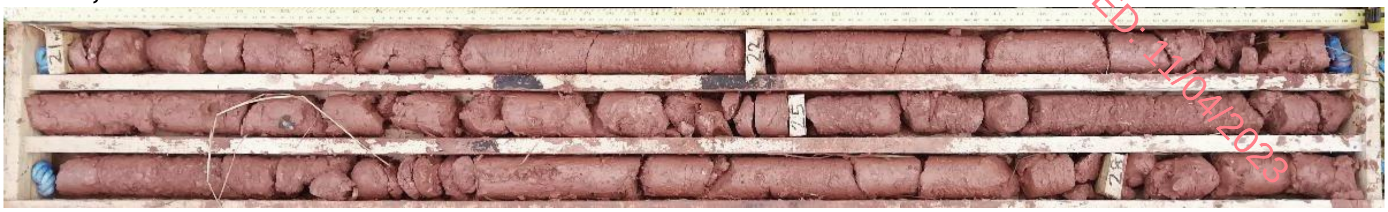
Box 12, 73m to77

Box 13, 77m to 81m

Box 14, 81m to 86m

Box 15, 86m to 88m (End Of Hole)

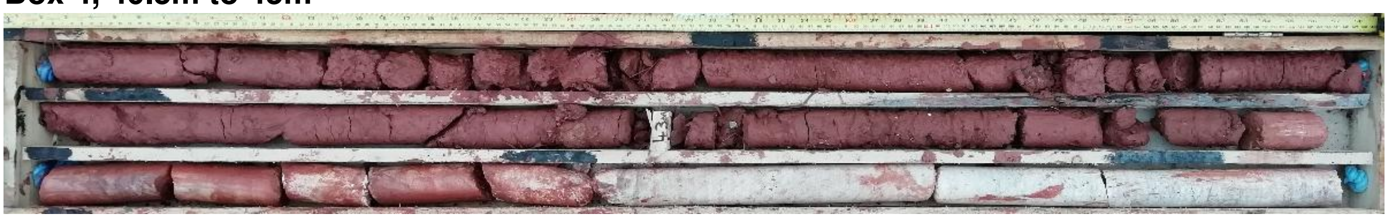
Box 1, 21m to 28.5m



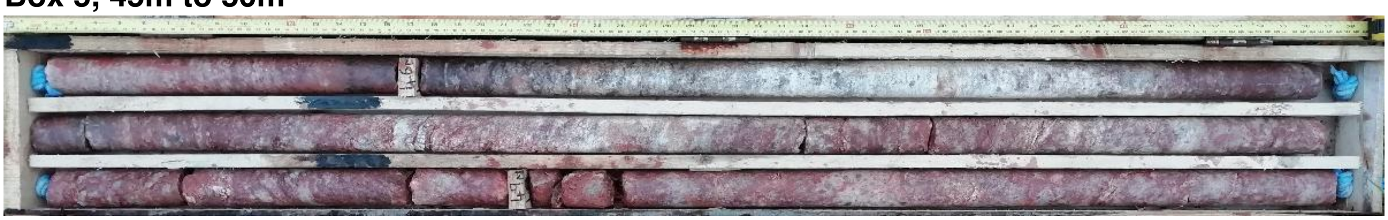
Box 2, 28.5m to ~35m

Box 3, ~35m to 40.5m

Box 4, 40.5m to 45m



Box 5, 45m to 50m

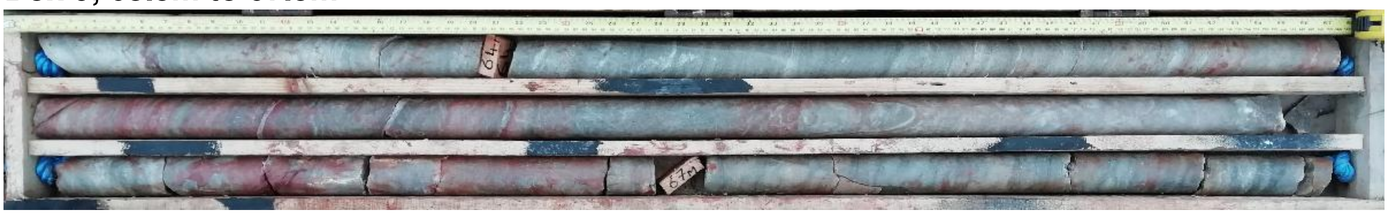


Box 6, 50m to 54m

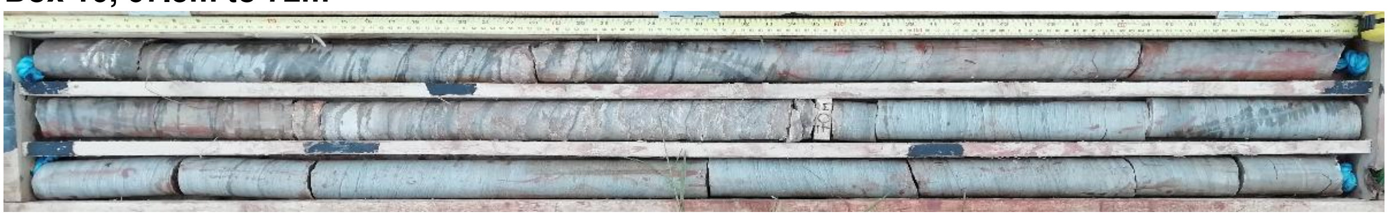
Box 7, 54m to 59m

Box 8, 59m to 63.5m

Box 9, 63.5m to 67.5m



Box 10, 67.5m to 72m

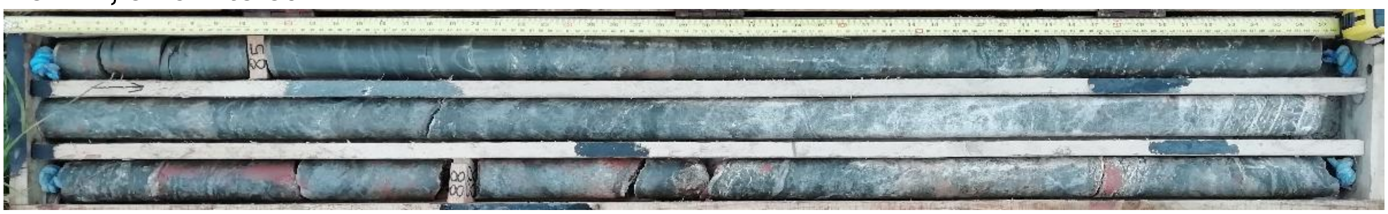


Box 11, 72m to 75.5m

Box 12, 75.5m to 80.5m

Box 13, 80.5m to 84.5m

Box 14, 84.5m to 89m



Box 15, 89m to 93.5m

Box 16, 93.5m to 98m (End Of Hole)

Box 1, 12m to 17.2m

Box 2, 17.2m to 22.5m

Box 3, 22.5m to 28.5m

Box 4, 28.5m to 35.5m

Box 5, 35.5m to 39.9m

Box 6, 39.9m to 44m

Box 7, 44m to 45.5m (End Of Hole)

Box 1, 16.6m to ~19.5m

Box 2, ~19.5m to 22.1m

Box 3, 22.1m to 25.1m

Box 4, 25.1m to 28.1m

Box 5, 28.1m to 31.1m

Box 6, 31.1m to 34.1m

Box 7, 34.1m to 37.1m

Box 8, 37.1m to 40.1m

Box 9, 40.1m to 41.2m (End Of Hole)

Box 1, 16.7m to ~19.5m

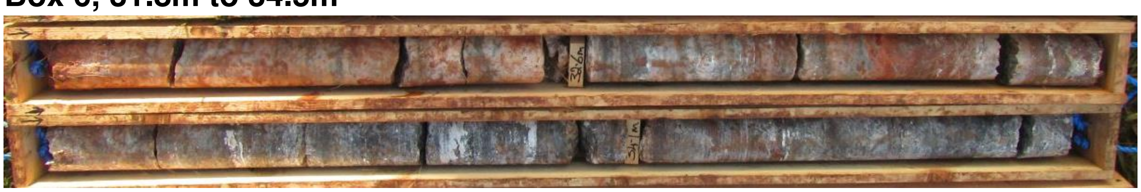
Box 2, ~19.5m to ~22.4m

Box 3, ~22.4m to 26.8m

Box 4, 26.8m to 29.5m

Box 5, 29.5m to 31.8m

Box 6, 31.8m to 34.5m



Box 7, 34.5m to 37.5m

Box 8, 37.5m to 40.1m

Box 9, 40.1m to 41.2m (End Of Hole)

Box 1, 24m to 28.5m

Box 2, 28.5m to 33m

Box 3, 33m to 37.5m

Box 4, 37.5m to 41.5m

Box 5, 41.5m to 46m

Box 6, 46m to 50.5m

Box 7, 50.5m to 54.5m

Box 8, 54.5m to 58.3m

Box 9, 58.3m to 63.5m

Box 10, 63.5m to 67.5m

Box 11, 67.5m to 71.8m

Box 12, 71.8m to 76m

Box 13, 76m to 80m

Box 14, 80m to 84.5m

Box 15, 84.5m to 89m

Box 16, 89m to 90m (End Of Hole)

Box 1, 0m to 7.5m

Box 2, 7.5m to 12.8m

Box 3, 12.8m to 17m

Box 4, 17m to 21m (End Of Hole)

Box 1, 0m to 9m

Box 2, 9m to 14.5m

Box 3, 14.5m to 25.5m

Box 4, 25.5m to 33m (End Of Hole)

Box 1, 0m to 10.5m

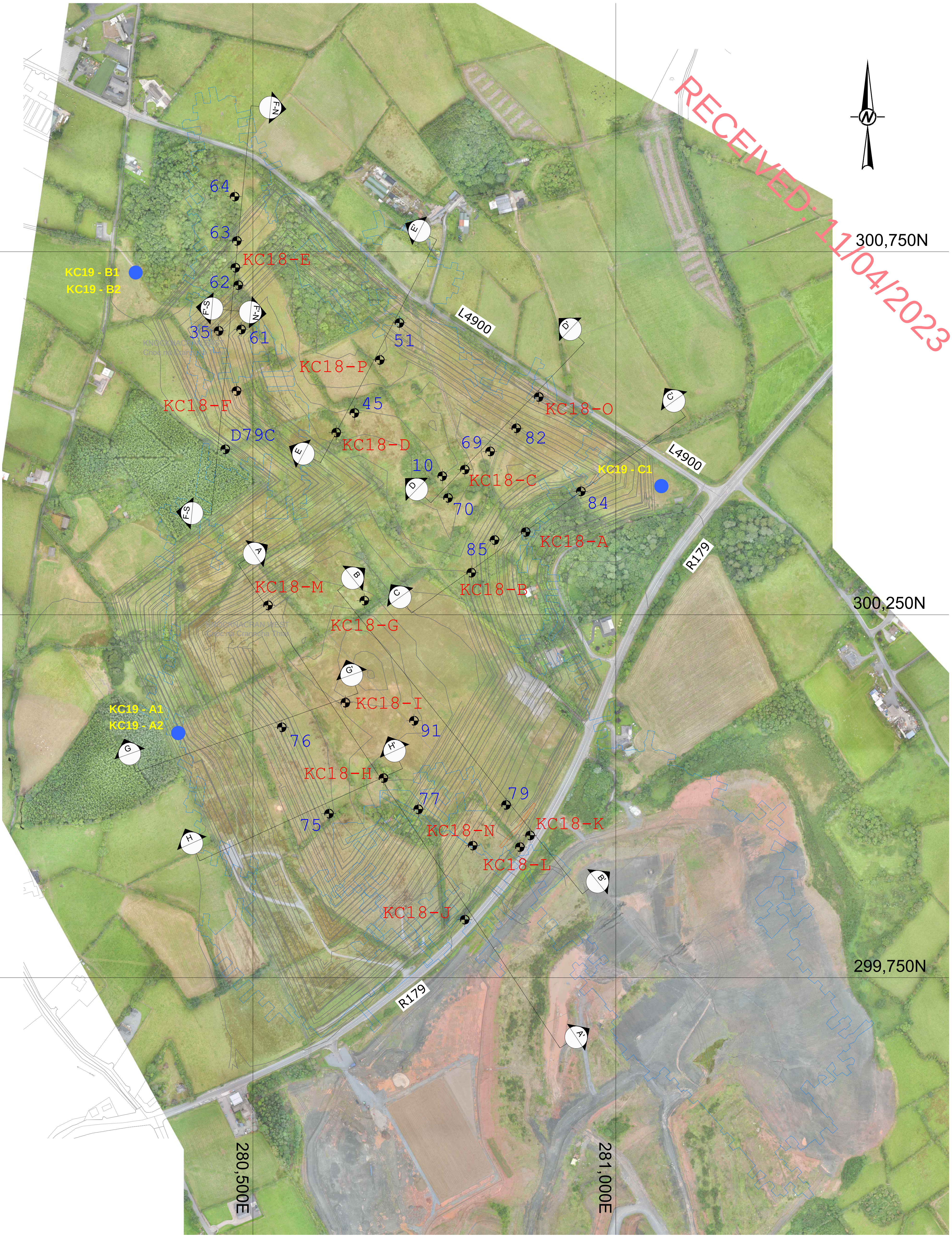
Box 2, 10.5m to 18m

Box 3, 18m to 24m (End Of Hole)

APPENDIX B

Cross-sections

Path: \\naa1-s-main01\Company\PROJECTS\2019\19121210 - Drumgoosat EIAR\5. Technical Work\GRAPHICS\30. Slope Stability Report DWG\ | File Name: 04 - Monitoring Wells - 20 Jan 2022.dwg



LEGEND:	
KC18-C	GOLDER 2018 BOREHOLES
	OUTLINE OF UNDERGROUND WORKINGS IN THE LOWER SEAM GYPSUM
	PIT DESIGN

NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID

ELEVATION ARE IN mMD (MALIN HEAD Mod + 1002.6 m)

DIMENSIONS ARE IN METRES.

DRONE BACKGROUND IMAGE AS SUPPLIED BY SGMI (March 2018)

OSI Licence No.: CYAL50192220

CLIENT
SAINT GOBAIN MINING (IRELAND) LIMITED

PROJECT
KNOCKNACRAN WEST PIT

CONSULTANT
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2022-Jan
PREPARED	BK
DESIGN	POB
REVIEW	BB
APPROVED	BB

TITLE
SECTION AND MONITORING WELL LOCATION PLAN

PROJECT No.	DRAWING No.	Rev.	SCALE
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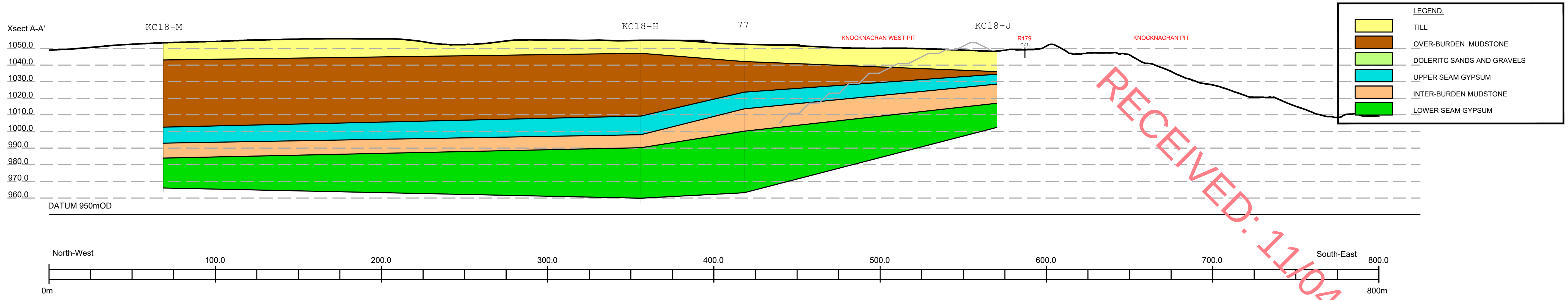

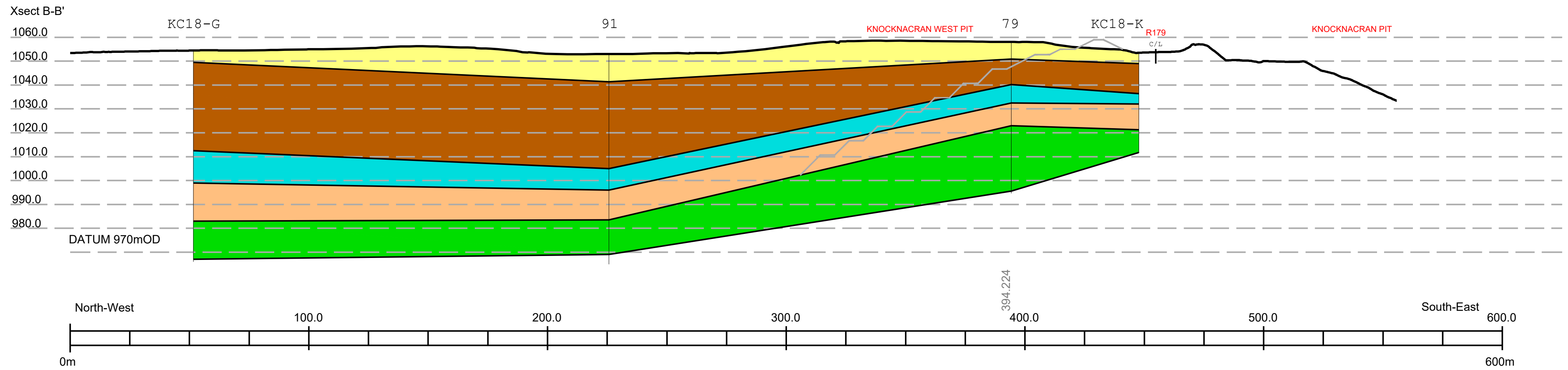

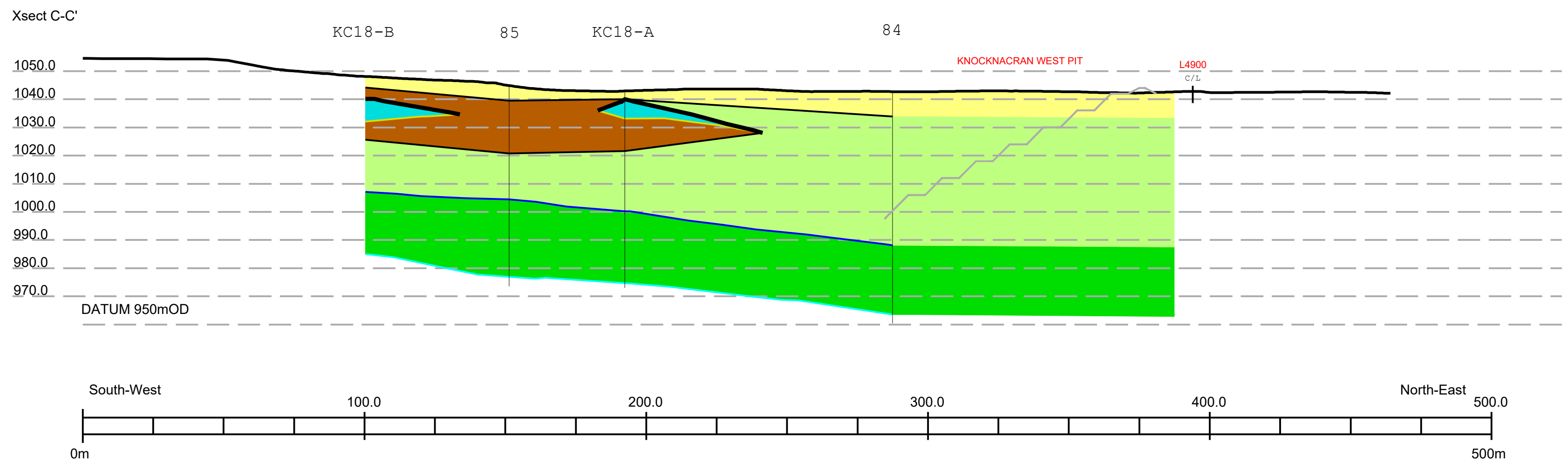

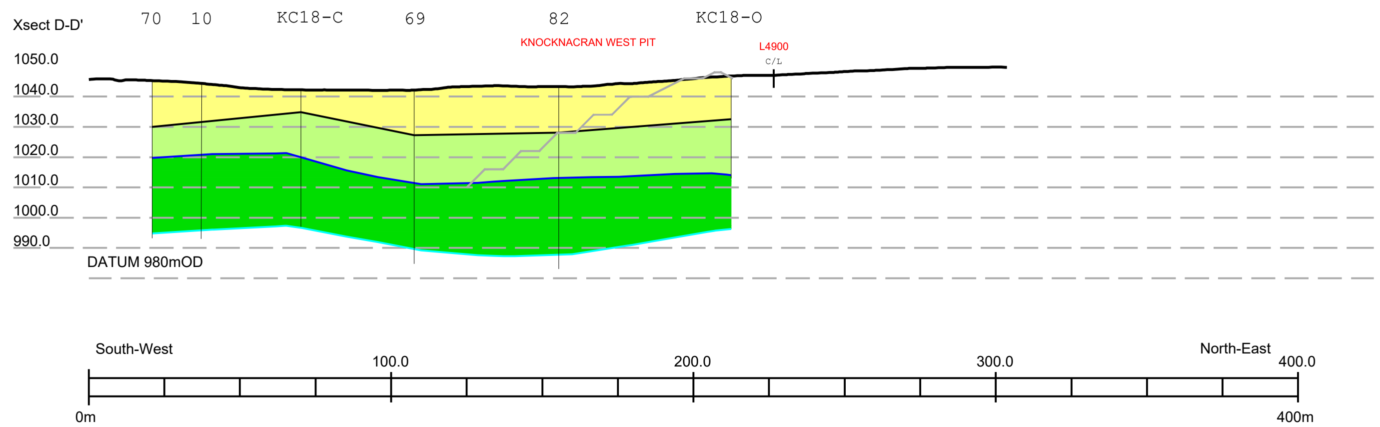

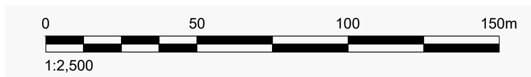

CLIENT	PROJECT
SAINT GOBAIN MINING (IRELAND) LIMITED	KNOCKRACRAN WEST PIT

CONSULTANT

GOLDER

YYYY-MM-DD	2019-Oct-25
PREPARED	BK
DESIGN	POB
REVIEW	BB
APPROVED	BB

TITLE

CROSS SECTIONS A-A' to D-D'

PROJECT No.	DRAWING No.	Rev.	SCALE
19121210	02	A	1:2500 A3

Path: C:\Users\bkeenan\Documents\Golder\Saint-Gobain_Gyproc Ireland\Drumgoosat\13. Bh Xsect Dwgs - 27 June 2019\ | File Name: 01 to 03 - Cross Sections - 25 Oct 2019.dwg

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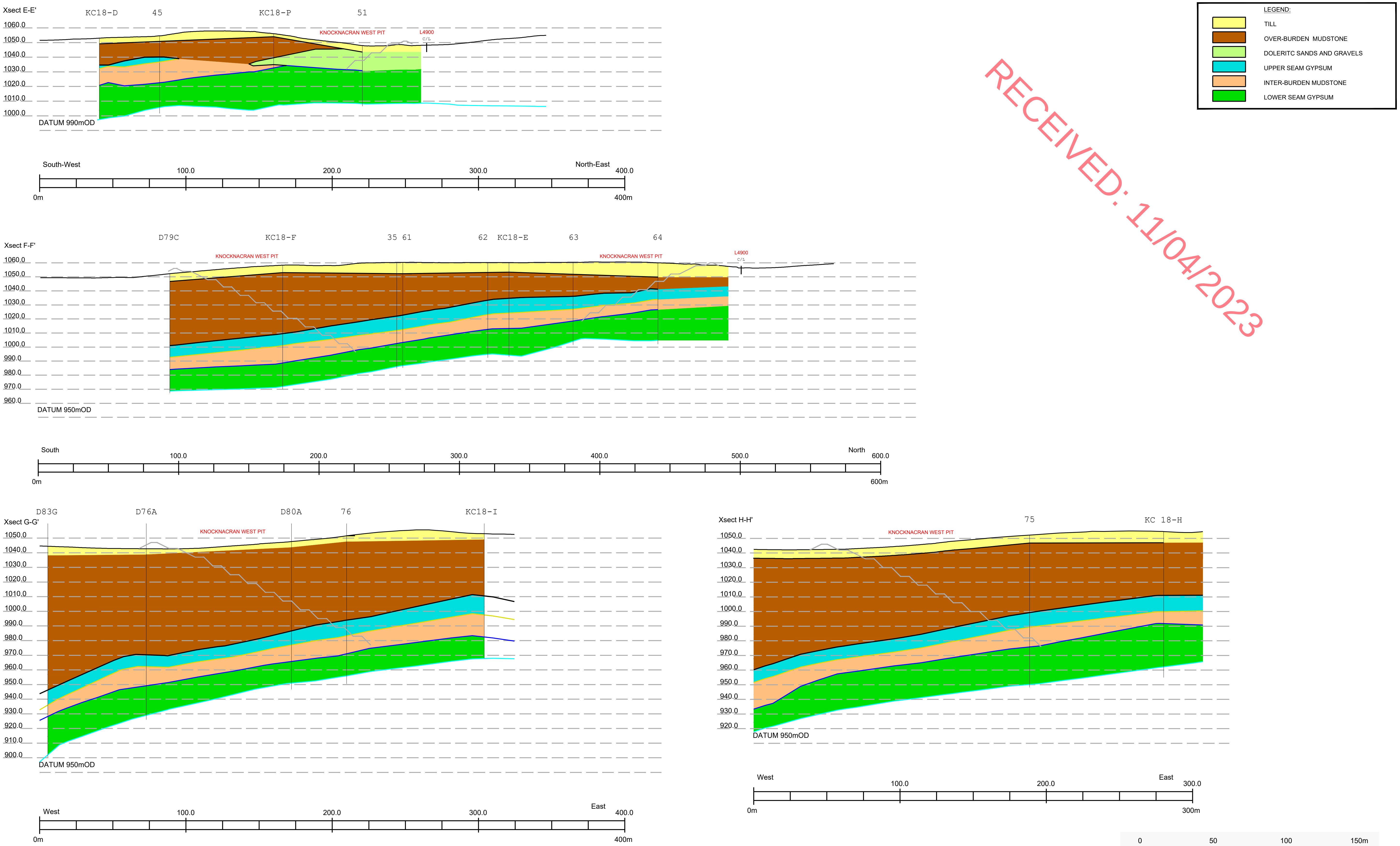

RECEIVED: 11/04/2023

CLIENT

SAINT GOBAIN MINING (IRELAND) LIMITED

PROJECT

KNOCKNACRAN WEST PIT

CONSULTANT

GOLDER

YYYY-MM-DD	2019-Oct-25
PREPARED	BK
DESIGN	POB
REVIEW	BB
APPROVED	BB

TITLE

CROSS SECTIONS E-E' TO H-H'

PROJECT No.	DRAWING No.	Rev.	SCALE
19121210	03	A	1:2500 A3

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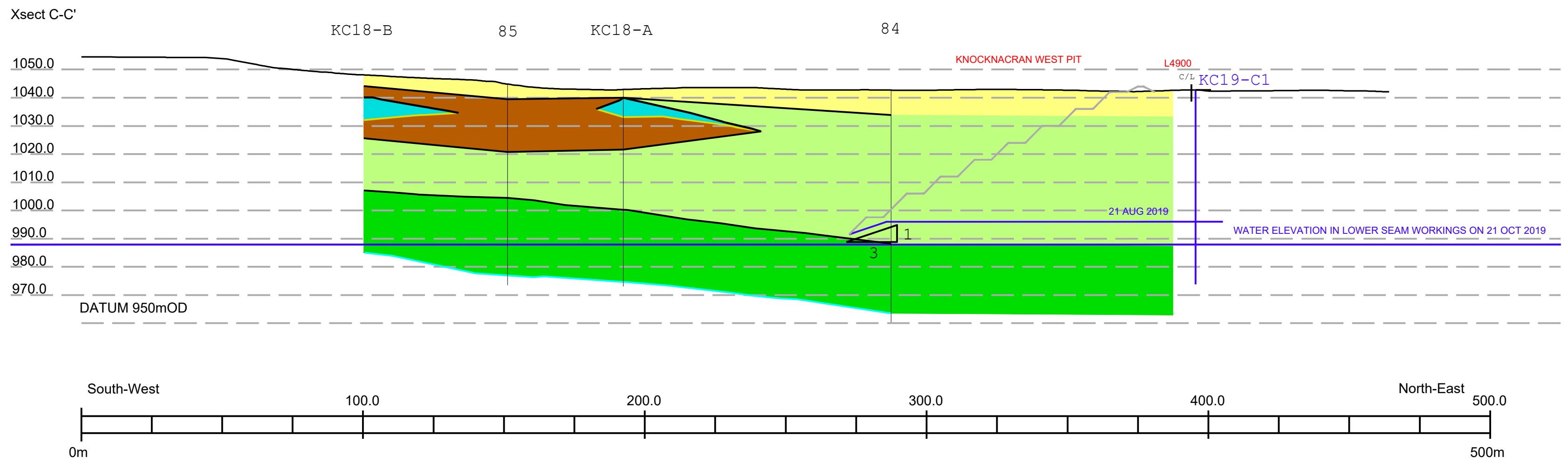
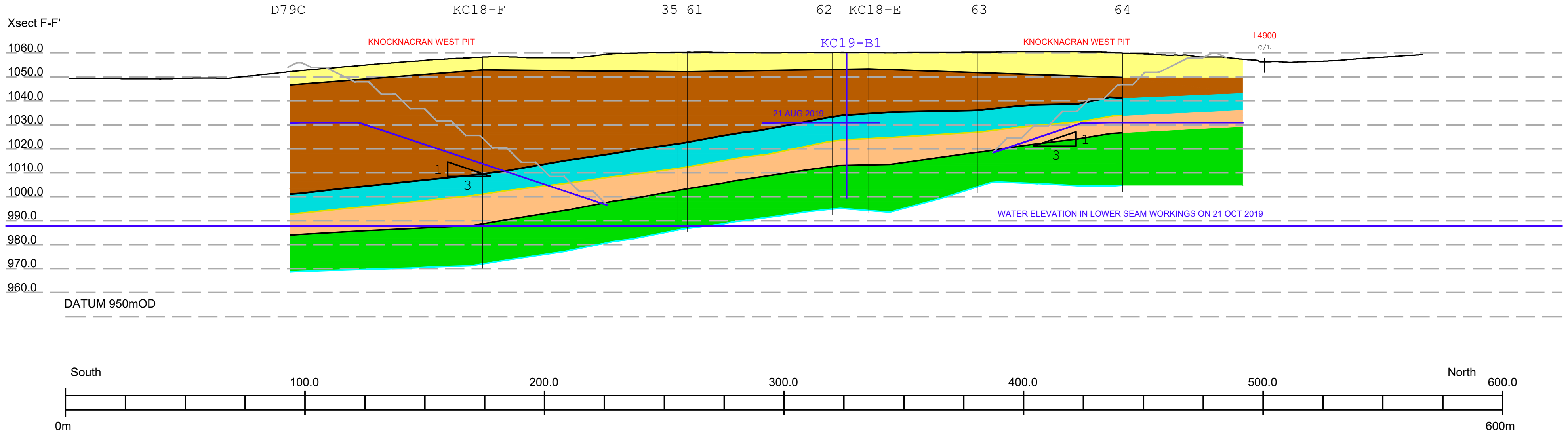
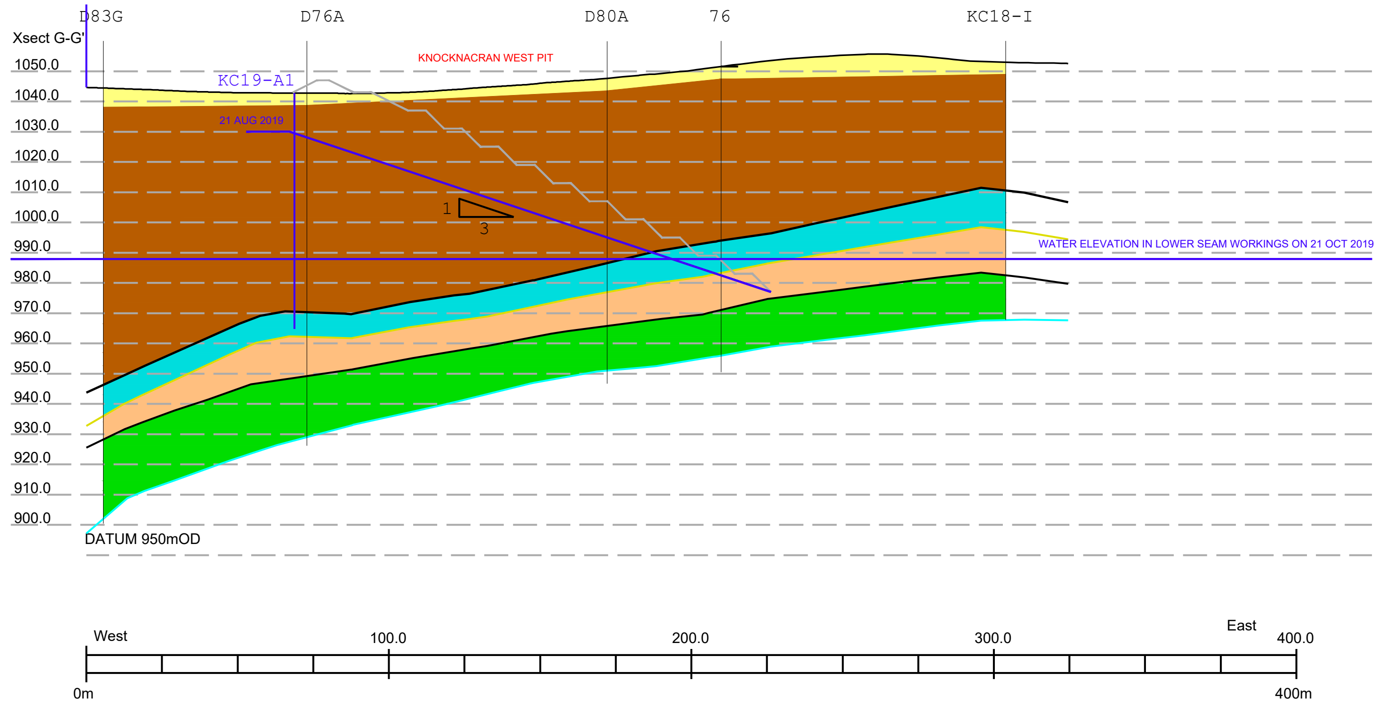
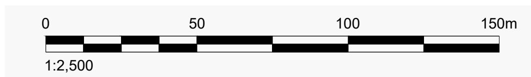






LEGEND:

- TILL
- OVER-BURDEN MUDSTONE
- DOLERITC SANDS AND GRAVELS
- UPPER SEAM GYPSUM
- INTER-BURDEN MUDSTONE
- LOWER SEAM GYPSUM



CLIENT	PROJECT
SAINT GOBAIN MINING (IRELAND) LIMITED	KNOCKNACRAN WEST PIT

CONSULTANT			TITLE
GOLDER	YYYY-MM-DD	2019-Oct-25	CROSS SECTIONS C-C', F-F' AND G-G' WITH MONITORING WELLS
	PREPARED	BK	
	DESIGN	POB	
	REVIEW	BB	
	APPROVED	BB	

PROJECT No.	DRAWING No.	Rev.	SCALE
19121210	05	A	1:2500 A3

golder.com

APPENDIX C

Long Term Mine Stability

SRK Consulting (UK) Limited
5th Floor Churchill House
17 Churchill Way
Cardiff CF10 2HH
Wales, United Kingdom
E-mail: enquiries@srk.co.uk
URL: www.srk.com
Tel: +44 (0) 2920 348 150

RECEIVED: 11/04/2023

External Memorandum

To:	Pat O'Connor	From:	Neil Marshall
Company:	Saint-Gobain Mining Ireland	Project Number:	UK31696
Copied to:	Benson Plunkett	Project Title:	Mine Stability
File Ref:	31696_Report 2 Mine Stability_Draft(V4).docx	Date:	28 July, 2022
Subject:	**Long Term Mine Stability**		

1 RFI POINTS

The following RFI points have been combined within this document as they largely relate to mine subsidence outside the final quarry boundary and long term mine stability.

Subsidence Risk (RFI Ref: Points 14)

Major Accidents chapter identifies subsidence as a major accident risk, controlled through ongoing monitoring during the life of the project. However, further detail is requested on the continued risk, if any, posed to lands beyond the scope of this open cast mine, where subsurface mine shafts may be flooded post restoration of the proposed site.

Long Term Stability/Stabilisation of Workings Below Road (RFI Ref: Point 20.q)

Permanent Solution to existing mine workings that go under the existing public road network: The applicant has not clearly demonstrated how they propose to address the issue of future road subsidence on the public road network where previous mine workings exist. The applicant must submit comprehensive proposals, including design reports, drawings, and other appropriate design details that demonstrate how the applicant proposes incorporating a permanent solution to the mine workings that go under the public roads as part of their open cast works.

Finite Element Modelling Update (RFI Ref: Points 22.i.a and b)

The Finite Element (FE) models should be updated to include the existing open pit void and proposed backfilling. In particular the current section lines across the R179 should consider the impact of the existing void and the backfilling operation planned within the existing Knocknacran Open Cast Mine.

2 RESPONSE

2.1 Proposed Approach to the Query Response

SRK has undertaken numerous 2D finite element modelling studies looking at the stability of the underground workings below the R179 and L4900 roads. We have also undertaken modelling which included the impact of the proposed Knocknacran West Quarry which was carried out for the consultants that prepared the planning application. This work was carried out in 2019 and updated in 2021 when the quarry design was modified. Whilst all of the finite element simulations carried out by SRK indicated that the underground workings would remain stable during the full history of excavation of Drumgoosat underground followed by the excavation of Knocknacran West quarry. Now that the final design of the Knocknacran West Quarry has been developed the numerical modelling has, as requested, been updated to include the quarry mining and backfilling of both East and West quarries. The results of this modelling will be used to provide insight into the long term stability and subsidence risk associated with the mine elements that remain below the R179 and of the L4900 during quarry excavation and subsequent flooding. We also consider any additional subsidence risks posed to third partly land beyond the quarry perimeter and below which underground mine voids remain. Figure 1 shows the final landform after backfilling of Knocknacran East Quarry, partial backfilling of Knocknacran West Quarry and formation of the quarry lake. The layers forming the figure have been made transparent so that the extent of underground rooms and pillars remaining below the quarry slope face and beyond the quarry boundary can be seen. Analyses of the five cross sections around the boundary of Knocknacran West Quarry are discussed in the following sections.

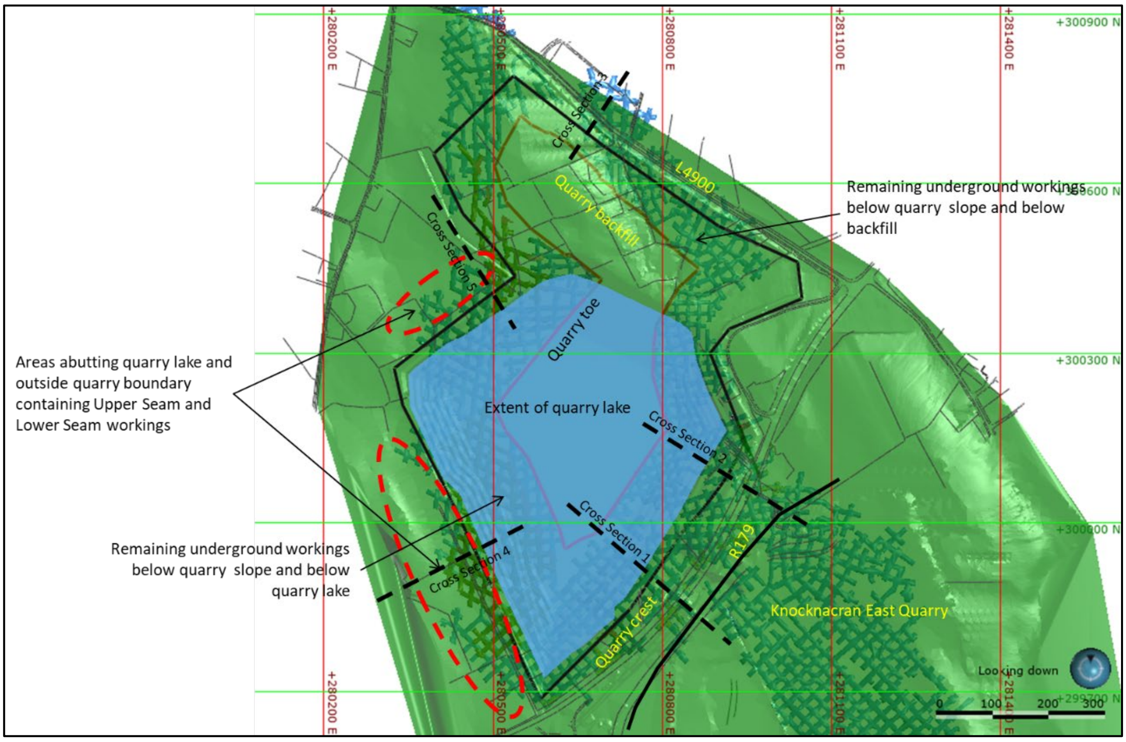


Figure 1: Final Quarry Landform showing Areas considered for Analysis

2.2 Finite Element Modelling – R179

The original request was to update the historical finite element modelling that had previously been reported to include the impact of the quarry voids. The orientation of the cross sections modelled were generated to assess the stability of specific underground excavations of concern. In order to properly integrate the quarry excavations into the modelling it becomes necessary to create new cross-sections orientated at right angles to the quarry slopes. Two cross sections have therefore been created that intersect the R179 and are orientated at right angles to the Knocknacran West quarry slope at their deepest points. Their location is shown in Figure 2. Details of each cross section showing the sequence of excavation and backfilling simulated are appended to this document.

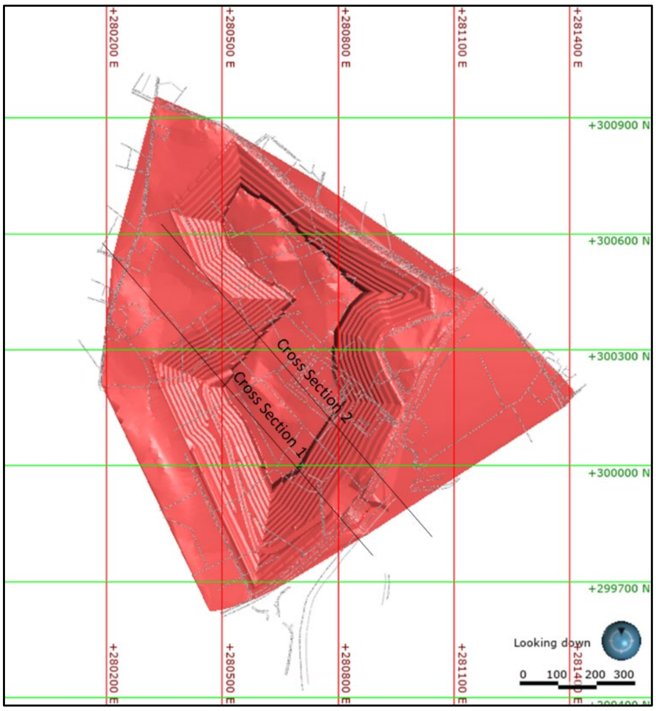


Figure 2: Location of Finite Element Modelling Cross Sections

The modelling was carried out in the following sequence:

1. Excavate the Drumgoosat mine.
2. Excavate the Knocknacran East Quarry.
3. Partially backfill the Knocknacran East Quarry. This is the current state of the landform southeast of the R179 road.
4. Excavate the Knocknacran West Quarry.
5. Backfill the base of the Knocknacran West Quarry and fill the Knocknacran East Quarry void to close to original ground level contours.

2.2.1 Cross Section 1 Results

Figure 3 is a contour plot of the model showing total vertical simulated displacement at the current state of the landform, i.e. Knocknacran East Quarry partly backfilled and Knocknacran West yet to be mined.

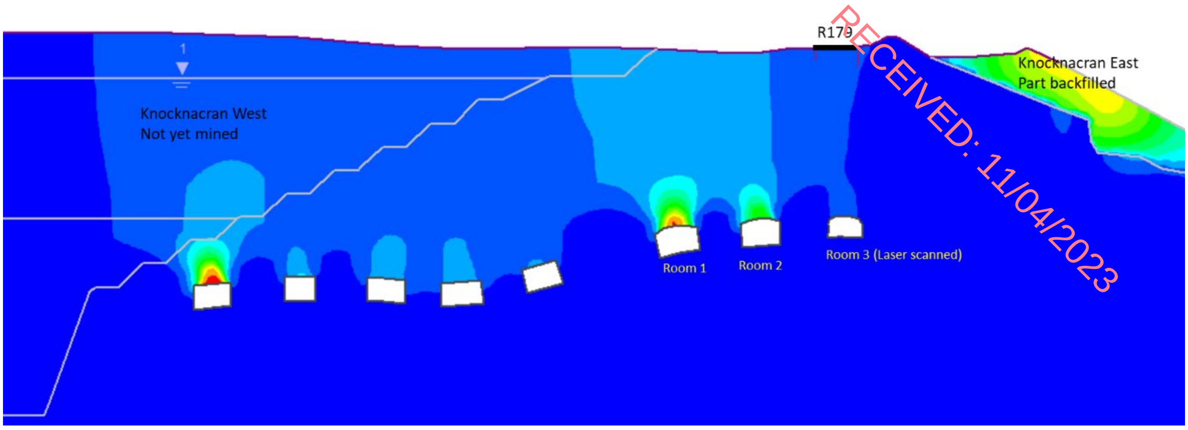


Figure 3: Cross Section 1 – Contours of Vertical Displacement, current State of the Landform

For the purpose of this assessment movement of the roof of the three rooms which underly the R179, Room 1, Room 2 and the laser scanned Room 3 as well as the vertical movement on the R179 is being considered. The roofs of Rooms 1 and 2 have also been slightly arched so that they take on the approximate roof shape of the laser scanned room.

Table 1 presents the results of the analyses. The initial roof deformation following mining of the rooms is quoted in millimetres. The additional simulated vertical deformation between each additional mining stage is also shown.

For all mining stages the model is stable. The creation of the Drumgoosat rooms generates an initial elastic roof beam deflection of 27 mm for Room 1 reducing to 4 mm for Room 3. The simulated surface deformation in the vicinity of the R179 is also 4 mm. East Quarry excavation and backfilling has negligible additional impact on roof deformation or on surface. Mining of the Knocknacran West Quarry has a very small impact on roof deformation and on surface. The room closest to the quarry slope indicates negative deformation change, or rebound, probably due to removal of overburden load. Final backfilling of Knocknacran West has no impact on roof deformation or on the surface near the R179.

Table 1: Cross Section 1 – Vertical Displacement Change by Mining Stage

	Simulated Vertical Ground Displacement (mm)	Additional Simulated Vertical Ground Displacement between Mining Stages (mm)			
	Underground Mined	East Quarry Mined	East Quarry Part Backfilled	West Quarry Mined	Final Backfill
Room 1	27	1	0	-1	0
Room 2	17	1	0	2	0
Room 3 (laser scanned)	4	0	0	2	0
Surface R179	4	0	1	1	0

2.2.2 Cross Section 2 Results

Figure 4 is a contour plot of the model showing total vertical simulated displacement at the current state of the landform, i.e. Knocknacran East Quarry partly backfilled and Knocknacran West yet to be mined.

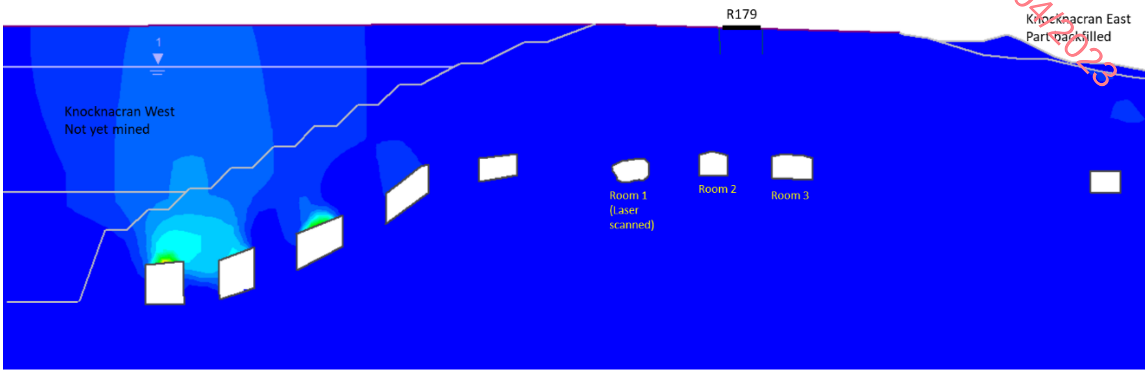


Figure 4: Cross Section 2 – Contours of Vertical Displacement, current State of the Landform

As with cross section 1 the roofs of Rooms 2 and 3 have been slightly arched to provide for a more realistic room profile in line with the shape of the laser scanned Room 1. It should be noted that this cross section intersects the 12 m high rooms and pillars that collapsed during the September 2018 event. These are located immediately behind the position of the Knocknacran West Quarry slope. Whilst no specific modification has been made to the strength of these pillars the rock mass above them is showing significant simulated deformation.

Table 2 presents the results of the analyses. The initial roof deformation following mining of the rooms is quoted in millimetres. The additional simulated vertical deformation between each additional mining stage is also shown.

Table 2: Cross Section 2 – Vertical Displacement Change by Mining Stage

	Simulated Vertical Ground Displacement (mm)	Additional Simulated Vertical Ground Displacement between Mining Stages (mm)			
	Underground Mined	East Quarry Mined	East Quarry Part Backfilled	West Quarry Mined	Final Backfill
Room 1 (laser scanned)	2.8	0.0	0.0	-0.4	0.0
Room 2	1.9	0.0	0.3	0.1	0.0
Room 3	2.8	-0.1	0.0	0.0	0.0
Surface R179	1.3	0.1	0.0	0.3	0.0

The initial deformation and that between mining stages is very small compared to that simulated for Cross Section 1, fractions of a millimetre in most cases. However the deformation between mining stages does follow the same trend as for Cross Section 1. The lower deformation values are due to the roof beam interpreted as being much thicker along this section line, 7 m to 12 m thick, than on section line 1, 3 m to 5 m thick.

2.2.3 Impact of Quarry Flooding on Underground Mine Stability

The final quarry landform will be developed into a quarry lake with the quarry containing water to a depth of between 36 m and 38 m. The Knocknacran West Pit Lake Model and Restoration Plan document prepared by Piteau Associates (Ref: 4238_Gyproc_LR05, dated 16 December 2021) indicates that there is likely to be limited groundwater generated from within the underground workings that remain in-situ after quarry mining has been completed. They indicate that the quarry backfill will be largely impermeable as it will be generated from mudstone interburden. They also state that it is expected that the underground workings will be gradually submerged as the water in the pit lake rises but no significant head of water will develop above the underground workings, suggesting that water will be constrained within the Lower Gypsum.

An additional analysis was undertaken to simulate a groundwater table at the top of the Lower Gypsum unit with underground rooms dry to determine what, if any, additional deformation of the underground workings may occur as the Lower Gypsum develops water pressure before the mine becomes flooded. The results are shown in Table 3 for Cross Section 1 and in Table 4 for Cross Section 2. The results are presented as a change in deformation from the Final Backfill case.

The results indicate that the increase in water pressure around the underground workings results in greater deflection of the roof beams. The model does not indicate that failure of the roof beam of the workings located below the R179 has occurred. Furthermore the increased roof beam deflection is not fully transmitted to surface because of the presence of unmined Upper Seam gypsum which provides some reinforcement to the rock mass.

Table 3: Cross Section 1 – Vertical Displacement Change Induced by Flooding

	Additional Simulated Vertical Ground Displacement between Mining Stages (mm)
	Quarry flooded
Room 1	195
Room 2	197
Room 3 (laser scanned)	389
Surface R179	11

Table 4: Cross Section 2 – Vertical Displacement Change Induced by Flooding

	Additional Simulated Vertical Ground Displacement between Mining Stages (mm)
	Quarry flooded
Room 1 (laser scanned)	1448
Room 2	708
Room 3	2917
Surface R179	127

The greatest deformation occurs above the rooms that are immediately behind the quarry slope and below the quarry lake.

2.2.4 Discussion – R179 Subsidence Potential and Mitigation

The results of the additional numerical modelling indicate that the mining voids that remain below and in the vicinity of the R179 are unlikely to be adversely affected by the activities of mining the Knocknacran West Quarry and backfilling of both East and West Quarries. The simulations do indicate however that there could be an elevated risk of roof deformation and possibly roof collapse as the underground rooms fill with water. This may occur as the water level in the quarry lake rises to its long term final level. No pillar failure in indicated in the simulations as the pillars below the road are only 6 m high and they are underlain by a significant thickness of competent lower gypsum unlike the mine conditions in the area of the September 2018 event where pillars were 12m high with a foundation of weak mudstone. Roof deformation is higher along Cross Section 2 than along Cross Section 1 probably because the underground workings are closer to surface along Cross Section 2.

The possible impact on surface should roof beam collapse occur could range from no impact, to increased levels of measured surface subsidence to the occurrence of crownholes above four way intersections. Almost all of the four way intersections that are present below the R179 have been accessed by boreholes and laser surveyed on several occasions. They are all currently in good condition showing no indication of significant rock fall or instability. The laser surveyed areas in relation to the R179 and the mine layout adjacent to the road is illustrated in Figure 5.

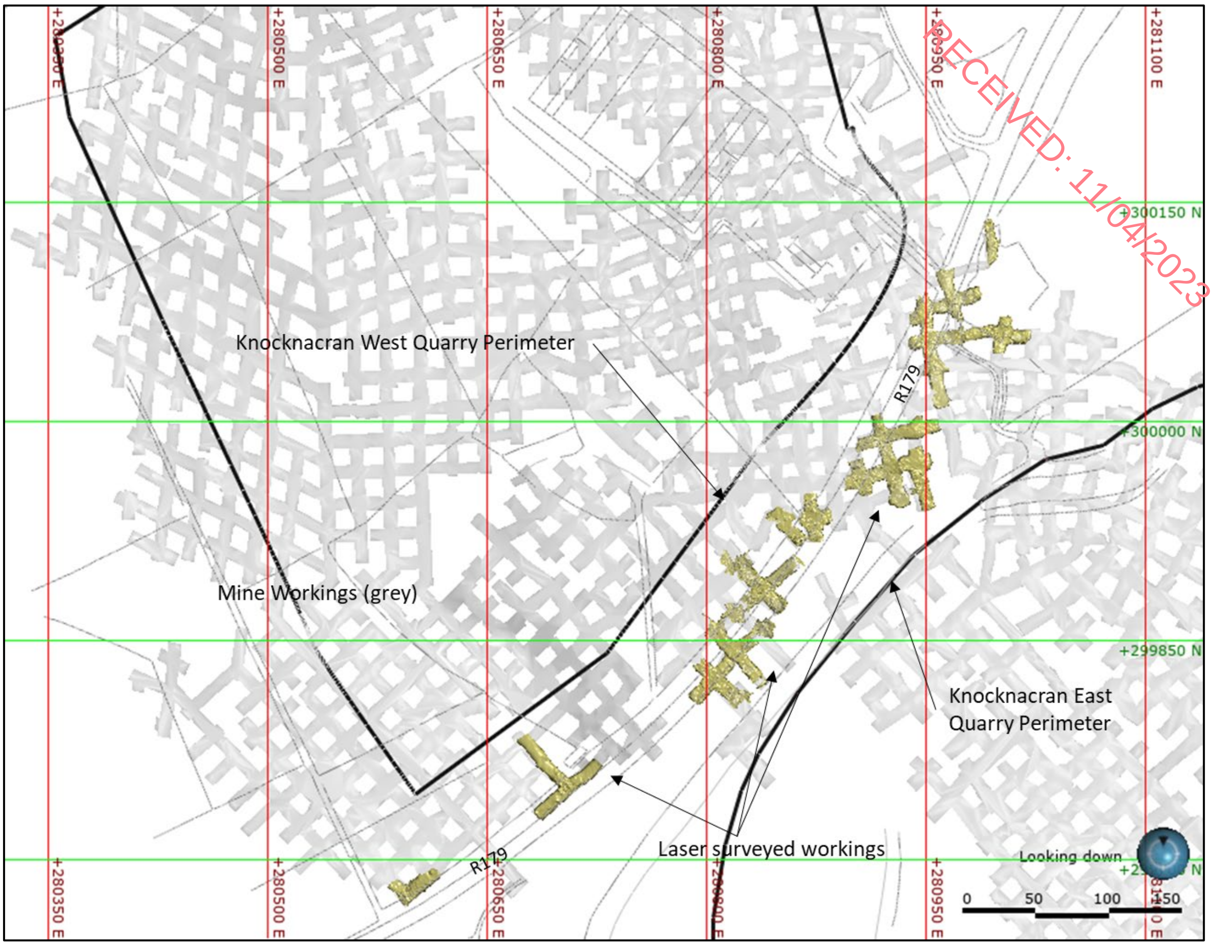


Figure 5: Laser Surveyed and Mine Workings below R179

A number of methods are available that can be used to manage any risks associated with changes to the stability of the underground workings as the quarry lake is formed. These are:

1. Continue the periodic surface levelling to provide early warning of any changes in movement magnitude along the road.
2. Continue monitoring the extensometers that have been installed within the roof beams at various location under the road.
3. Undertake sonar surveys as the underground workings become flooded to establish whether any instability develops.
4. If safe access into the underground workings can be established when rooms are exposed in the quarry slope consideration could be given to either:
    a. Constructing block walls in the workings and place rockfill material sourced from local quarries behind the walls to minimise infiltration of water into the underground workings, or

b. Packing the underground workings, specifically the four way intersections below the road, with waste rock to inhibit any potential roof movement as the quarry and underground workings are flooded.

2.3 Numerical Modelling – L4900

As shown in Figure 1 the quarry adjacent to the L4900 will be backfilled before the remaining quarry void is flooded. The underground workings should therefore be protected from water ingress by the presence of this backfill barrier. A finite element model has been crated along cross section 3. Its location is shown superimposed on the quarry design in Figure 6.

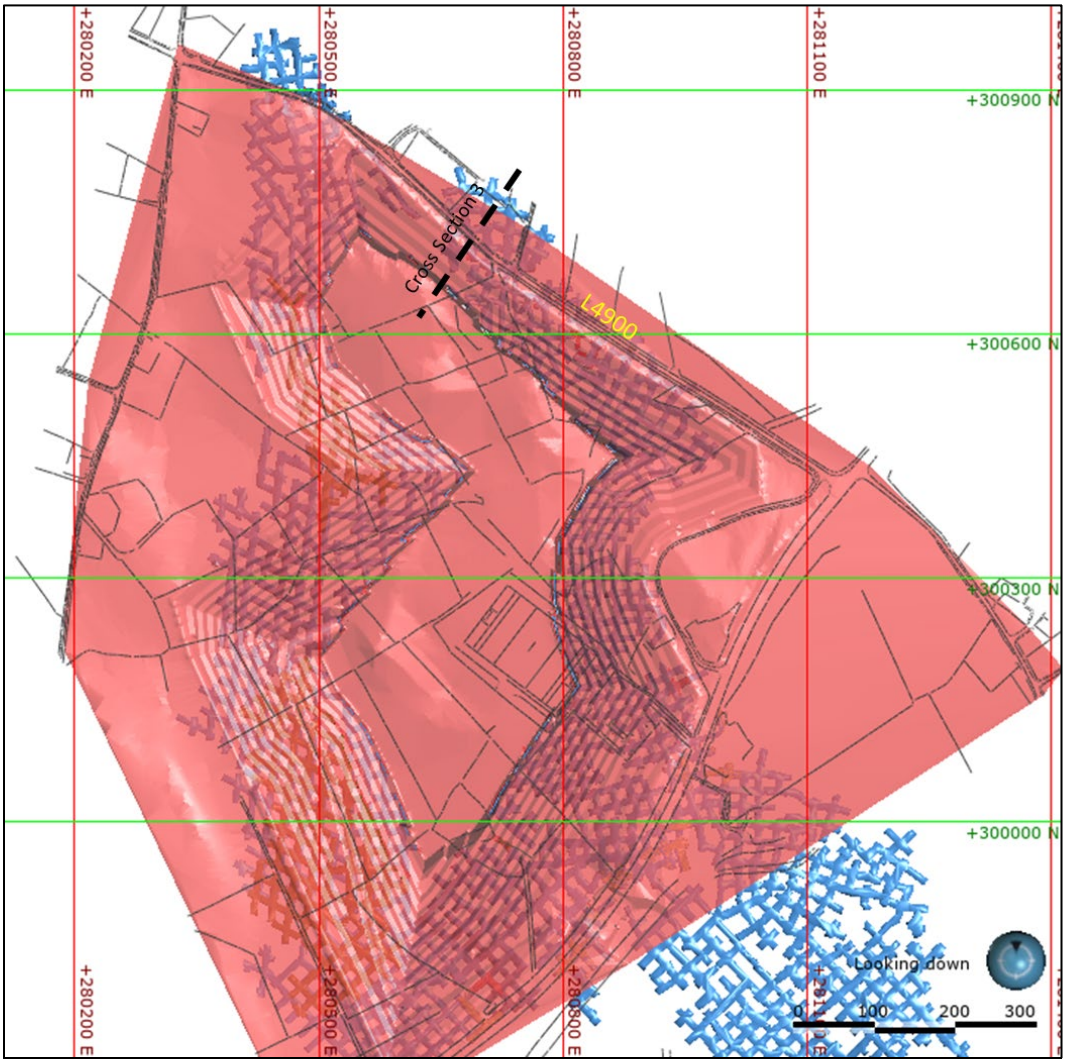


Figure 6: Location of Cross Section 3 across L4900

Figure 7 is a contour plot of vertical displacement after quarry excavation above the rooms below and adjacent to the L4900. Note that Room 3 contains the roof beam extensometer H11. In the three years that the instrument has been installed it has recorded roof movement of 0.03mm.

Table 5 shows the results of incremental roof beam deformation for each mining stage. It can be seen that quarry mining and backfilling induces little additional deformation to the roof beams or the ground surface around the L4900.

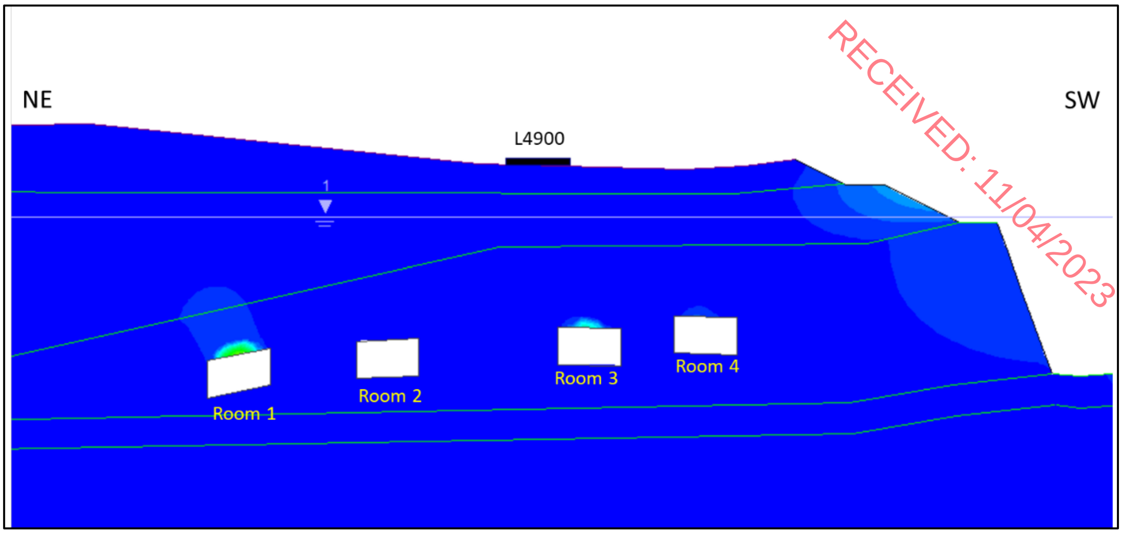


Figure 7: Cross Section 3 – Contours of Vertical Displacement after Quarry Excavation

After quarry flooding and assuming the lake water can flow through the backfill then additional forces can develop within the gypsum surrounding the underground workings to cause the roof beam to collapse. As with the flooding of the workings below the R179 no pillar failure is generated and the roof beam deformation does not propagate to surface.

In order to inhibit flooding of the quarry workings below the L4900 it is recommended that the mine rooms that are exposed in the quarry face are sealed prior to the commencement of quarry backfilling.

Table 5: Cross Section 3 – Vertical Displacement Change by Mining Stage

	Simulated Vertical Displacement (mm)	Additional Simulated Vertical Ground Displacement between Mining Stages (mm)		
	Underground Mined	Quarry Mined	Quarry Backfilled	Quarry Flooded
Room 1	18	13	2	2767
Room 2	2	0	0	2298
Room 3 (containing extensometer H11)	9	8	0	1943
Room 4	2	1	3	1704
L4900	0.8	0.2	0	21

2.4 Subsidence Risk South and West of the Quarry Perimeter

Having analysed mine instability below the R179 and L4900 roads which border the east and north perimeter of the quarry there are small areas to the south and west of the quarry perimeter that contain Upper Seam and Lower Seam workings. These areas have been highlighted in Figure 1. Cross sections 4 and 5 are illustrated and discussed below. In order to assess potential future subsidence risk we have considered the position of the workings in relation to various mine water levels and the potential response to the change in mine water levels that has been experienced above the mine workings over the last three years.

2.4.1 Cross Section 4

Details of Cross Section 4 which is located beyond the southwestern limit of the quarry are shown in Figure 8. This area contains the deepest areas of Upper and Lower Gypsum seam mining. The top of the Lower Seam is 110 m below surface and the top of the Upper Seam is 80 m below surface. These is beyond the maximum depth where potential crownholes could propagate to surface which has been determined to be about 60 m, 10 times the height of the mine openings. The lower seam workings have been permanently flooded since mining ceased at Drumgoosat as they lie below long term permanent mine water level of 970 m. The Upper Seam mine workings will have been largely dry until the mine flooding which caused the September 2018 event when they will have become completely flooded. Since then the Upper Workings have been subject to dewatering. The Upper Seam workings will be come completely submerged when the quarry lake is formed.

The Upper Seam workings extract a relatively thin seam of gypsum meaning that the roof beam in some areas is likely to be thin. Collapse of four way intersections has occurred since September 2018 above the Upper Seam workings further northeast along this section line in an area where the workings are closer to surface. There could be potential for future instability of rooms within the Upper Seam as the workings become permanently flooded. However because of the depth of the workings in this area the potential that any instability could manifest itself as a crownhole at surface is considered to be low.

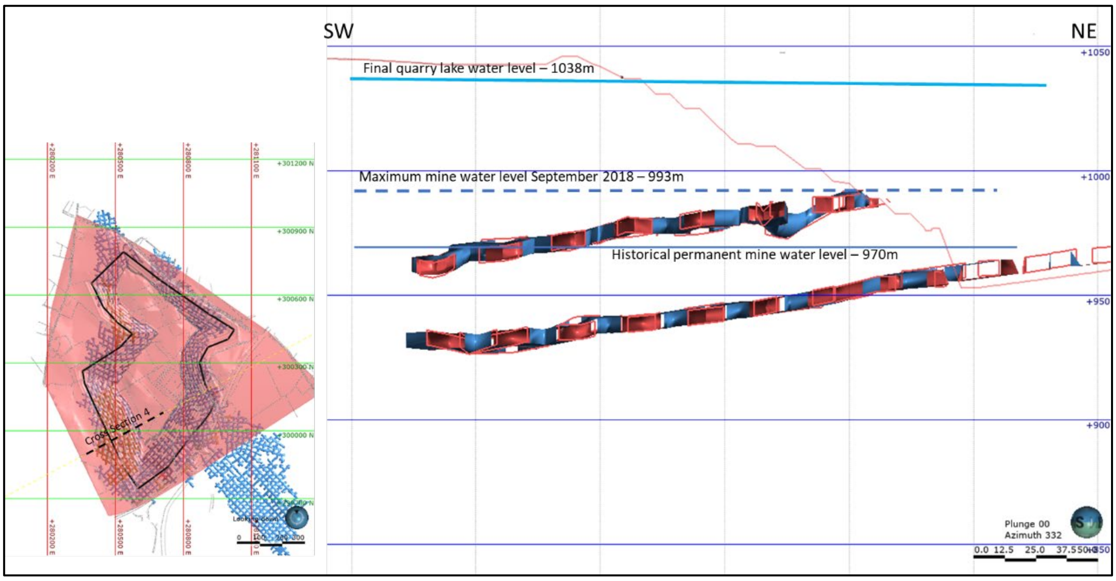


Figure 8: Cross Section 4

2.4.2 Cross Section 5

Details of Cross Section 5 which is located beyond the western limit of the quarry are shown in Figure 9.

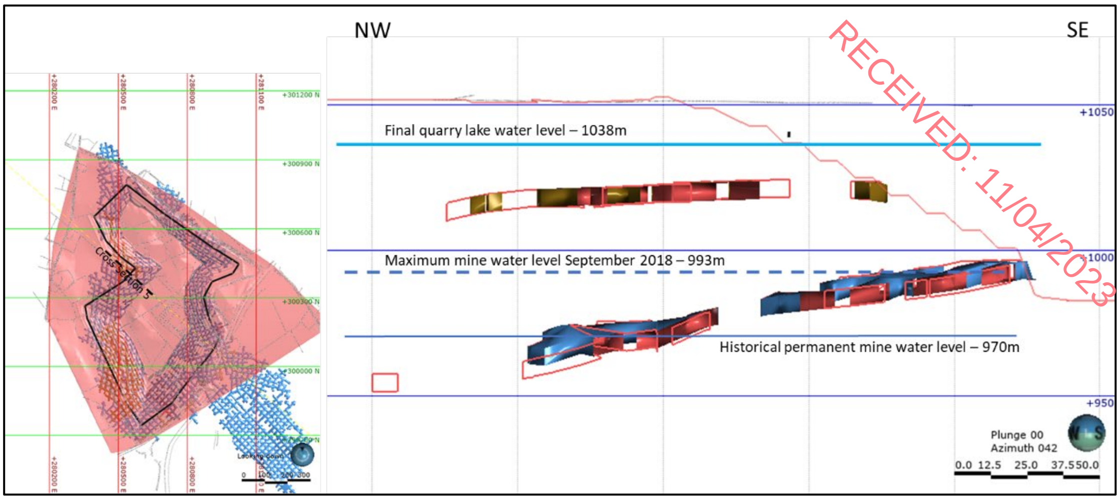


Figure 9: Cross Section 5

The top of the Lower Seam is 80 m below surface and the top of the Upper Seam is 30 m below surface. Both Lower and Upper Seam workings are located above the highest permanent mine water level of 970 m. The Lower Seam workings will have been largely dry since underground mining ceased until the mine flooding which caused the September 2018 event when they will have become completely flooded. The Upper Seam Workings lie above the level of temporary mine flooding and have been permanently dry but will become flooded when the quarry lake is formed. The Upper Seam workings extract a relatively thin seam of gypsum meaning that the roof beam in some areas is likely to be thin. There has been one recorded instance of the development of a crown hole in this area above the Upper Seam workings where the workings are about 35 m below surface. Because the Upper seam workings are shallow in this area there is potential for the development of new crownholes as the workings are flooded when the quarry lake is developed. It will be necessary to seal any Upper Seam rooms exposed in the quarry face to inhibit water ingress into the workings and reduce the risk of the formation of crownholes in this area.

3 DISCUSSION – MINING THROUGH FAILED PILLARS

There are a number of areas of subsidence through which quarry mining will take place. Subsidence has been the result of pillar failure of the Lower Gypsum in the area of the 2018 event and a combination of pillar and roof beam failure of the Upper Gypsum in the more recent western extension of the subsidence. It is anticipated that the ground within the subsidence areas will have moved and will therefore be of lower strength than the equivalent in-situ material. The overburden sitting above these failed pillar areas which is predominantly drift and mudstone will probably not pose a risk to excavation through as it is a relatively weak material and will have broken and re-compacted to accommodate movement of the rock mass. Gypsum pillars and roof beams that will have been affected by the subsidence will have broken up into large bedding and joint bounded slabs. The underground workings in these areas will have been filled by broken rock but it is likely that because of the shape and size of the slabs there will still be voids between them which may pose a hazard when trying to excavate through them. Position and extent of these unstable areas is reasonable well defined. In order to excavate safely through these area safe working methods will be developed by the operator and its contactors.

For and on behalf of SRK Consulting (UK) Limited

Neil Marshall,
Corporate Consultant - Geotechnical,
Project Manager
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APPENDIX D

Roof Beam Stability and Kinematics

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External Memorandum

To:	Pat O'Connor	From:	Neil Marshall
Company:	Saint-Gobain Mining Ireland	Project Number:	UK31696
Copied to:	Benson Plunkett	Project Title:	Roof Beam Assessment
File Ref:	31696_Report_3_Roof beam Stability_Draft(V2).docx	Date:	22 July, 2022
Subject:	**Roof Beam Stability and Kinematics**		

1 RFI POINTS

The following RFI points have been combined within this document as they relate to stability of mine elements.

Roof Beam Stability (RFI Ref: Points 22.i.c and d)

The roof beam stability assessment should be updated to include assessment of safe unsupported spans for the proposed maximum slope configurations. Roof beam instability needs additional consideration as confined pressure is released from the removal of overburden, interburden and upper seam gypsum. It is recommended that the rock mass should be characterised, and the maximum unsupported span determined for the reduced overburden loads to show that the roof beam will be stable.

Kinematic Pillar Failure (RFI Ref: Point 22.ii.b)

Pillar failure through rock mass has been calculated but the kinematic failures mechanism does not appear to have been considered. Additional planar failure mechanisms should be analysed to determine the potential for joint and bedding plane failures and details shall be submitted accordingly.

2 RESPONSE

2.1 Response Clarification

In a meeting with GSRO and Wardell Armstrong (WA) on 24 June 2022 WA clarified that kinematic pillar failure mechanisms and safe unsupported roof beam spans related to the interaction of these mine elements where they are exposed in the quarry faces and the potential for kinematic instability to impact the stability of the quarry slopes. Quarry slope design was the responsibility of Golder Associates (Golder) and therefore outside the scope of work that SRK has historically been involved in on behalf of Saint-Gobain Mining Ireland. Golder will therefore address these elements of the RFI's.

SRK's contribution to this response will focus on the roof beam stability of the underground workings within the footprint of the quarry as it is impacted by quarrying and the consequent reduction of overburden loading.

2.2 Roof Beam Stability in Response to Overburden Unloading

As overburden is removed from above the underground mine as a function of quarry excavation, loading of the roof beam decreases and stability should improve to some degree. To simulate this SRK has carried out 3D finite element modelling using the Rocscience computer program RS3. The underground survey of the Lower Seam workings, the Lower Seam geology model and the Knocknacran West Quarry design have been interrogated to determine the range of room and four way intersection spans, the range of roof beam thickness and the range of quarry excavation depth to the underground workings. These ranges are:

- Room spans – 10 m to 12 m.
- Intersection spans – 14 m to 17 m.
- Roof beam thickness – 3 m to 12 m.
- Depth to workings - 100 m.

These ranges have been simulated by the construction of two RS3 mine geometry models, one comprising 10 m square pillars separated by 10 m wide rooms and another comprising 10 m square pillars separated by 12 m wide rooms. The 10 m wide rooms result in 14 m wide four way intersection spans. The 12 m wide rooms result in 17 m wide intersection spans. For each mine geometry model three additional models were constructed containing 3 m, 6 m and 12 m thick roof beams. Each of these models comprised a 100 m thickness of overburden which was progressively removed in 25 m slices to expose the top of the roof beam thus simulating quarry excavation. At each stage of overburden removal the maximum deflection of the underside of the roof beam above the mining room and wider span four way intersection was interrogated.

An annotated perspective view of the RS3 model along with plan views of the two mining layouts is shown in Figure 1. The output of the modelling is presented as graphs of beam span versus maximum roof beam deflection in Figure 2 for the 3 m thick roof beam, Figure 3 for the 6 m thick roof beam and Figure 4 for the 12 m thick roof beam. The strength of the rock units was represented by the Generalised Hoek-Brown constitutive model, with input parameters as shown in Table 1. The value of GSI defines the fracture or jointing condition of the rock mass with lower GSI values representing rock containing a relatively greater number of joints or fractures than rock characterised by higher GSI values. The strength of the overburden is a composite of Upper Gypsum, mudstone and drift.

All the simulations converged to a solution indicating that the roof beams remained stable irrespective of roof beam span, thickness or overburden loading.

Table 1: RS3 Model Strength Parameters

	Gypsum Roof Beam	Gypsum Pillars	Overburden
Unit Weight (MN/m^3)	0.023	0.023	0.02
UCS (MPa)	15	20	10
GSI	55	75	40
mi	8	8	7
D	0	0	0
Poisson's Ratio	0.15	0.15	0.3

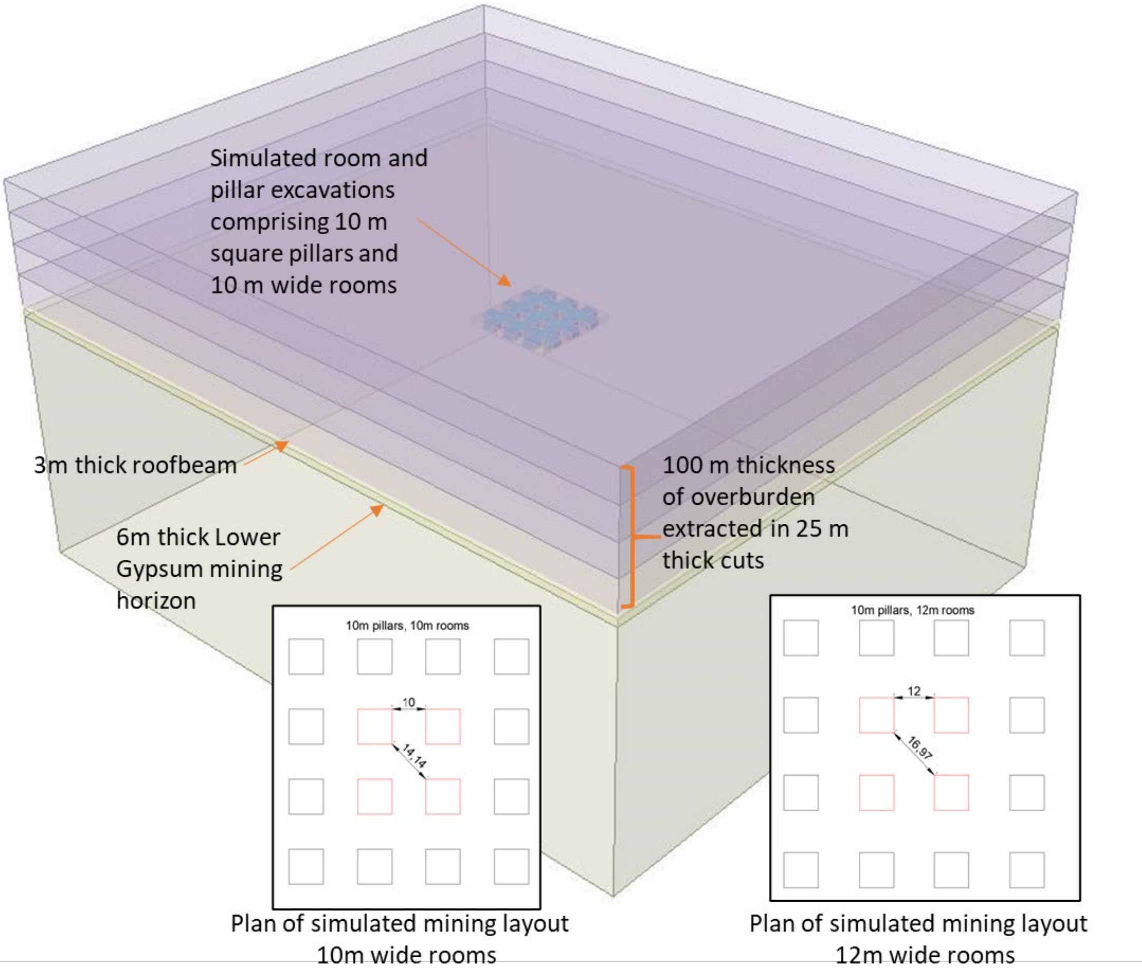

Figure 1: RS3 Roof Beam Stability Assessment Model

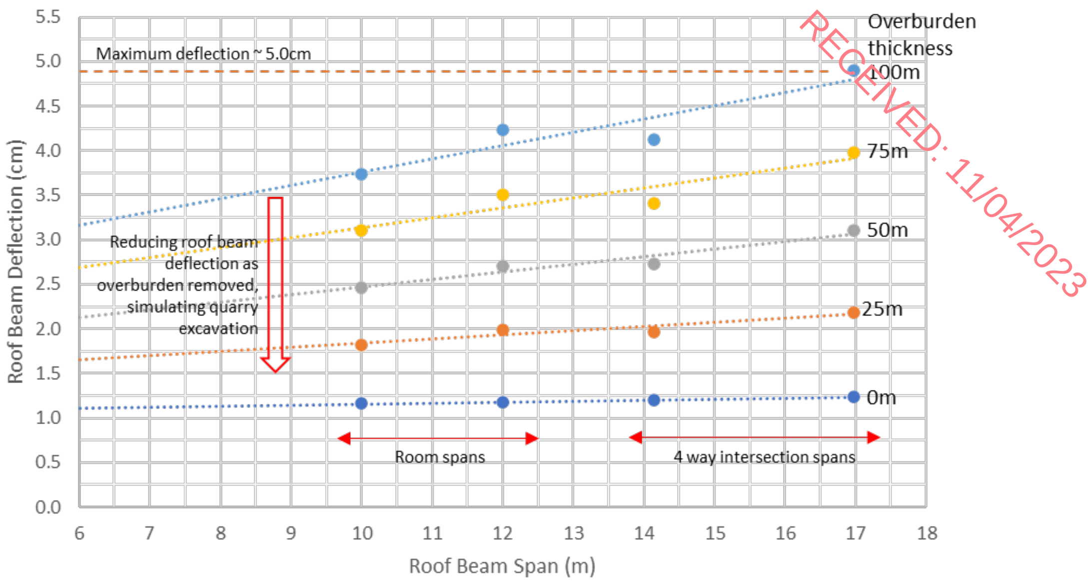

Figure 2: Roof Beam Deflection Results for a 3 m Thick Roof Beam

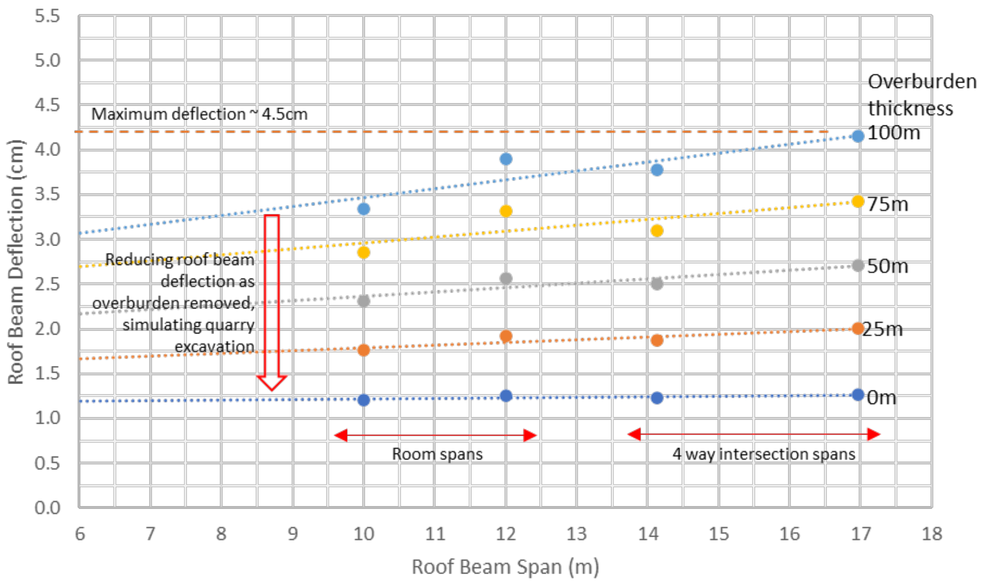

Figure 3: Roof Beam Deflection Results for a 6 m Thick Roof Beam

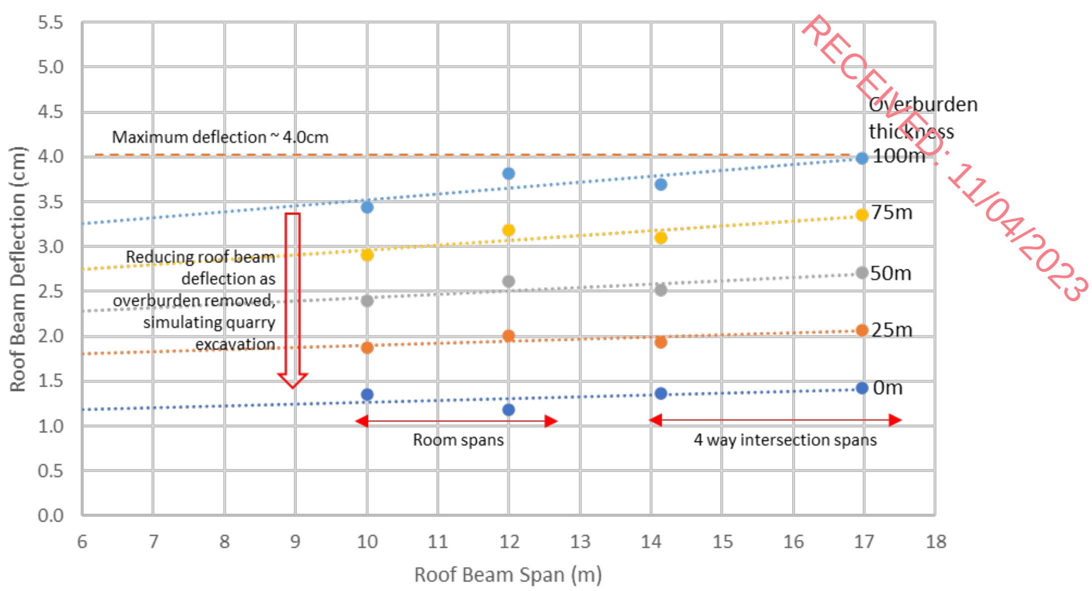

Figure 4: Roof Beam Deflection Results for a 12 m Thick Roof Beam

All of the graphs are of a similar form with roof beam deflection reducing as the overburden is removed. The maximum roof beam deflection also reduces as the roof beam becomes thicker. The maximum simulated deflection for the 3 m thick roof beam is 5 cm. The maximum simulated deflection reduces to 4 cm for the 12 m thick roof beam.

2.3 Discussion and Conclusions

This analysis has demonstrated that as overburden is removed from above the mine workings loading on the Lower Gypsum rock mass that forms the roof beam above the underground workings reduces. This results in elastic rebound of the rock mass which should, in theory, improve the stability of the roof beams above the workings.

What this modelling is not able to simulate is the effect that unloading may have on discrete joints and bedding planes within the beam. Unloading may relieve the stress acting across these features which may in turn reduce their frictional strength. This could allow joint bounded blocks to slip and fall into the underground workings locally reducing the roof beam thickness.

Standard operating procedures when mining above and through underground room and pillar mines in a quarry is to blast and collapse the roof beam to fill the room thus eliminating the risk associated with the presence of the underground workings. As part of the safe working methods for collapsing the roof beam a minimum pit floor pillar thickness will be defined to allow quarry equipment to operate safely above the underground voids along with appropriate personnel and equipment access, drilling and blasting strategies, barricading procedures and general reporting protocols as is currently the case when recovering gypsum from Knocknacran Open-Cast Mine, above the Drummond underground mine workings.

For and on behalf of SRK Consulting (UK) Limited

Neil Marshall, Corporate Consultant - Geotechnical, **Project Manager** SRK Consulting (UK) Limited	Max Brown, Principal Consultant - Geotechnical, **Project Director** SRK Consulting (UK) Limited

APPENDIX E

Procedure for mining in the vicinity of suspected voids & unstable ground - underground mine workings - (DRAFT)

Procedure for mining in the vicinity of suspected voids & unstable ground (underground mine workings) (DRAFT)

Subject	**Proposed Safe Operation Procedure**			
Owner	**Benson Plunkett**	**Doc. No.**		
Compiled by	**B Plunkett / T O'Reilly / Andrew Ellis**	**Rev: 0**	**Knocknacran West**	✓
Date of Issue:	**04/08/2022**			

Introduction

This draft Safe Operating Procedure (SOP) is based on the current practices in place for the existing Knocknacran Open-Cast Mine for the safe overburden / interburden removal and mining of Gypsum.

The SOP provides details on the work practices for the safe removal of overburden, interburden and Gypsum from locations above known or suspected underground mine workings, including areas of previous subsidence.

The SOP is based on practices developed and refined over many years (1988 – 2022) of experience gained by Saint-Gobain Mining Ireland (SGMI) from mining at Knocknacran.

Objectives

The objectives of the SOP are:

1. To ensure the safety of personnel at all times when working above underground mine workings and areas of subsidence.

2. To provide clear direction to all personnel regarding responsibilities and procedures to be followed when working over underground workings and areas of subsidence.

3. To reduce the likelihood of a serious incident occurring in relation to working over underground mine workings and areas of subsidence.

Principles of Working over Underground Mine Workings and Areas of Subsidence

Work From Safe ground - The main principle for working over underground mine workings and areas of subsidence is to 'work out' from 'known safe ground' based on confirmatory mine survey plans provided by Mine Records and signed off by Mine Management and the contractor Project Management team.

Clear Planning - Records of all plans will be kept in the Mine Survey Office.

Good Communication - Copies of plans for active mining areas will be pinned to a notice board in the mine site office at the Knocknacran West site for inspection by all staff working on the removal of overburden/interburden and extraction of Gypsum. Such plans will also be made available to contactors who are involved in stripping campaigns for overburden/interburden removal.

Regular Review - The plans will be updated periodically by the Mine Surveyor as new/additional information becomes available during stripping of materials and extraction of Gypsum. The size (area and depth) of overburden/interburden to be stripped will be agreed with Mine Management and the contactor prior to any works been carried out. Following blasting and removal of Gypsum after each blast, the area in question will be surveyed by the Mine Surveyor. Plans will be updated and recorded in the Mine Surveyor's Office.

Extensive studies have taken place over the last number of years in the areas of geology, mine design, ground stability (and condition), subsidence, and hydrogeology; including specific studies on the underground workings, including areas of interest in terms pillar size, room dimensions, roof and floor beam spans, all of which will be utilised to assess risk prior to the removal of overburden/interburden, and the extraction of Gypsum throughout the life of the mine.

Main Stages of Mining Gypsum from an Open-Cast Mine

There are two main stages of mining Gypsum from an open-cast mine:

1. Removal of overburden and interburden (i.e., soils and glacial overburden, mudstone and dolerite) by stripping using a combination of bulldozers, excavators and trucks.

2. Drilling and blasting of the Gypsum Seams, and subsequent hauling of Gypsum to the primary crusher.

Removal of Overburden/Interburden, Mudstone and Dolerite by Stripping

The removal of overburden and interburden (mudstone and dolerite) at any specific location will be dictated by the following:

- Design of open-cast bench positions over mined and un-mined areas.
- The location of Upper and Lower Gypsum Seam sub-outcrops.
- How subsidence features identified have affected Gypsum Seams at different working levels.
- Extraction of Upper Seam Gypsum (from previously mined & un-mined areas).
- Extraction of Lower Seam Gypsum (from previously mined & un-mined areas).
- Roof beam thicknesses of both Upper and Lower Seams - (old mine plates/plans & geotechnical reports will provide information).

From historical mine survey records, a 3D model for floor levels in both Upper and Lower Gypsum Seams from Drumgoosat will be developed - these will be used to generate a Roof Elevation Model for the historical mine workings in conjunction with information from historical borehole logs.

Before Overburden/Interburden, Mudstone and Dolerite Removal a Risk Map will be created based on:

- Locations/plans of known subsidence and sinkholes.
- Identification of roadway junctions with large roof spans - an example of this type of mapping already developed for the Upper Seam workings.
- Identification of areas that might be hazardous due to roof beam thickness.
- Identification of areas with two working horizons in a single Gypsum bed/seam.
- Identification of areas with mine workings over 6.0 m in height.
- Historical mine records to locate areas where water intersections were previously recorded underground, or anomalies such as un-mined areas or small pillars.

Method to Access and Excavate Overburden and Interburden

The Risk Map(s) produced will be used to determine the excavation method/approach to be used – it will at a minimum include:

- Definition of access routes to excavation area – i.e., consideration of size, weight and nature of equipment travelling over pillars and roofs between pillars, avoidance of travel over underground 4-way junctions as defined by the Risk Map.

- Definition of position of equipment during excavation of overburden/interburden - consideration of size, weight and nature of equipment, approach from areas of insitu rock (areas where no underground mining has taken place).

- Identification and treatment of suspected cavities - if a cavity is exposed or suspected – test holes will be drilled to determine its extent.

Production Drilling for a Blast

When overburden, interburden, mudstone and dolerite are removed, and Gypsum is exposed, the following procedures will take place before drilling and blasting is carried out:

- Design access route to drill test holes and blasting areas including consideration of size, weight and nature of equipment travelling over pillars and roadways between pillars - avoiding roadway junctions, and areas of subsidence.

- Generate a thickness model of the Gypsum Seam based on an exposed and updated survey of the top of Gypsum v's previously known/surveyed roof levels.

- Drill test holes from pillars/unmined ground to determine the actual roof beam thickness when no projected/inferred information is available.

- Update roof beam thickness model with test hole information.

- Carefully mark-out route ways to and from blasting areas to avoid areas of risk, such as underlying 4-way junctions and areas of previous subsidence.

- Where a roof beam is not suitable for working/travelling over - blast to be drilled from surrounding pillars.

The above procedures are based on current working practices that have been in use successfully for many years when extracting Gypsum from the current operating Knocknacran Open-Cast Mine.

Risk Analysis

SGMI understands that minimising risk is a key component of any mining activity, and continually considers operating risk throughout all its activities.

Mitigation strategies are incorporated (and reviewed and revised) into mine design and mine operational work practices on an ongoing basis.

Presented below is a risk assessment based approach proposed for the safe removal of overburden/interburden and extraction of Gypsum at Knocknacran West. The risk assessment process allows for the identification, qualitative assessment, and development of treatment strategies/actions for key mining related risks.

In order to adequately communicate and rank the consequence of perceived risks, SGMI will utilise a quantitative assessment when evaluating the Likelihood and Impact (scored from 1 to 5), as shown below, rating the activities and the impact for not being addressed.

Likelihood		
Score	**Description**	**Guidelines**
5	Almost Certain	This is a significant threat that could occur at any time. Immediate remedial action is required to remove or reduce the risk.
4	Likely	The threat exists, and it indicates high probability. Action is required to reduce this risk.
3	Possible	The threat exists but the history or expectation of this type of situation indicates occurrence is moderately probable. Action could be taken to reduce this risk.
2	Unlikely	A slight threat is perceived from this source, but the situation is unlikely to occur.
1	Rare	No perceived threat exists from this source. No action is required to reduce the risk.

Impact		
Score	**Description**	**Guidelines**
5	Severe	Major risk, injury to personnel and/plant, resulting in severe damage and therefore re-design.
4	Significant	Substantial risk, injury to personnel and/plant, resulting in damage or re-design.
3	Moderate	Notable risk, injury to personnel and/plant, causing down time of operations or similar.
2	Low	Minor risk, injury to personnel and/plant, some impact on daily operation.
1	Negligible	So minor as to be regarded as having no consequence – minimal impact on daily operation.

A matrix will be generated using the product of the scores assigned against likelihood and impact resulting in a Risk Rating which translates to a qualitative risk matrix, as shown below, this allows a rating of the relative risks of different combinations of likelihood and severity.

			Likelihood				
			Very Low (Rare)	Low (Unlikely)	Medium (Possible)	High (Likely)	Very High (Almost Certain)
		Score	1	2	3	4	5
Impact	Very High (Severe)	5	Moderate	Moderate	High	Very High	Very High
	High (Significant)	4	Moderate	Moderate	High	High	Very High
	Medium (Moderate)	3	Low	Moderate	Moderate	High	High
	Marginal (Low)	2	Very Low	Low	Moderate	Moderate	High
	Very Low (Negligible)	1	Very Low	Very Low	Low	Moderate	Moderate

Risk Mitigation

In order to effectively reduce the Impact and/or Likelihood of risks associated with mining activities at Knocknacran West, SGMI will adopt (and further develop) the mitigation measures currently in place at Knocknacran. Following application of such measures the residual risk is quantified by a description of the effectiveness of the controls applied, as presented below.

Control Effectiveness	
Rating	**Description**
Satisfactory	The control environment is operating effectively, providing a reasonable level of assurance that objectives are being achieved.
Some Weakness	The control environment has some weaknesses/inefficiencies. Although these are not considered to present a serious risk exposure, improvements are required to provide reasonable assurance that objectives will be achieved.
Weak	The control environment is not at an acceptable standard, as many weaknesses/inefficiencies exist. Reasonable assurance does not exist that objectives will be achieved.

The risk matrix for the removal of overburden, interburden and Gypsum from locations above or adjacent to known or suspected underground mine workings and areas of subsidence, with risk mitigation and control effectiveness, is presented in the Table below.

Table 1: Risk Matrix for the removal of overburden, interburden and Gypsum from locations above or adjacent to known or suspected underground mine workings and areas of subsidence

Risk №	Risk Issue	Impact		Likelihood		Risk Rating		Mitigation or Control	Control Effectiveness
1	Working above a pillar	1	Negligible	5	Almost Certain	5	Moderate	Updated mine survey records and risk map. Test drill edge of pillars to check for void space. Revise Risk Map.	Satisfactory
2	Working above a roadway	2	Low	5	Almost Certain	10	High	Test drilling from safe ground to confirm size, shape & depth of underlying roadway and roof beam to provide an updated mine survey plan. Update roof beam thickness model with test hole information. Revise Risk Map.	Satisfactory
3	Working above a junction	3	Moderate	5	Almost Certain	15	High	Test drilling from safe ground to confirm size, shape & depth of underlying junction and roof beam to provide an updated mine survey plan. Update roof beam thickness model with test hole information. Revise Risk Map.	Satisfactory
4	Working above a 4-way junction	4	Significant	5	Almost Certain	20	Very High	Test drilling from safe ground to confirm size, shape & depth of underlying 4-way junction and roof beam to provide an updated mine survey plan. Update roof beam thickness model with test hole information. Revise Risk Map.	Satisfactory
5	Working above or adjacent to fallen/collapsed ground	5	Severe	5	Almost Certain	25	Very High	Test drilling from safe ground to confirm size, shape & depth of fallen/collapsed ground to provide an updated mine survey plan. Revise Risk Map.	Satisfactory
6	Construction access routes for stripping material, and drilling and hauling of Gypsum	2	Low	5	Almost Certain	10	High	Test drilling from safe ground to confirm size, shape & depth of underlying roadways/junctions and areas of fallen/collapsed ground, and roof beam to provide a safe access to/from stripping area and Gypsum production faces. Update mine survey plan and roof beam thickness model with new information to revise Risk Map.	Satisfactory

GOLDER

golder.com

APPENDIX 7.13
Knocknacran West Mine Assessment, Ireland - SRK - November 2019

KNOCKNACKRAN WEST MINE ASSESSMENT, IRELAND

Prepared For

Gyproc Ireland

Report Prepared by

srk consulting

SRK Consulting (UK) Limited

UK30767

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Date:		November, 2019
Project Number:		UK30767
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KNOCKNACKRAN WEST MINE ASSESSMENT, IRELAND

1 INTRODUCTION

SRK has been requested by Gyproc Ireland ("Gyproc") to assess the impact of the proposed Knocknacran West Open Pit mine and of the cut-and-cover conveyor tunnel on the existing underground workings and on the surrounding roads, the R179 and L4900.

1.1 Terms of Reference

SRK has been supplied with the tasks that follow.

1. What impact will (or not) the planned mine have on the UG workings and therefore the L4900 and R179 over its life.
2. What impact will (or not) the proposed cut & cover tunnel have on the UG workings beneath the tunnel and therefore the R179 above.
3. What impact will (or not) backfilling the existing Knocknacran pit have on the UG workings and the roads if any.
4. Provide commentary of the effect of blasting on the installed instrumentation in the BHs along the L4900.

1.2 Summary of Analyses

The following sections describe the work undertaken and the results of the analyses. In summary the results indicate that the excavation of the proposed Knocknacran West mine will have no impact on the stability of the underground workings below the R179 and L4900.

2 IMPACT OF MINE EXCAVATION ON THE UNDERGROUND WORKINGS

In previous studies SRK carried out numerous 2D finite element analyses on the interaction between existing underground workings and surface infrastructure. For this project, SRK has updated those 2D finite element models which intersect both the road and mine slopes. The position of these cross sections is shown in Figure 2-1.

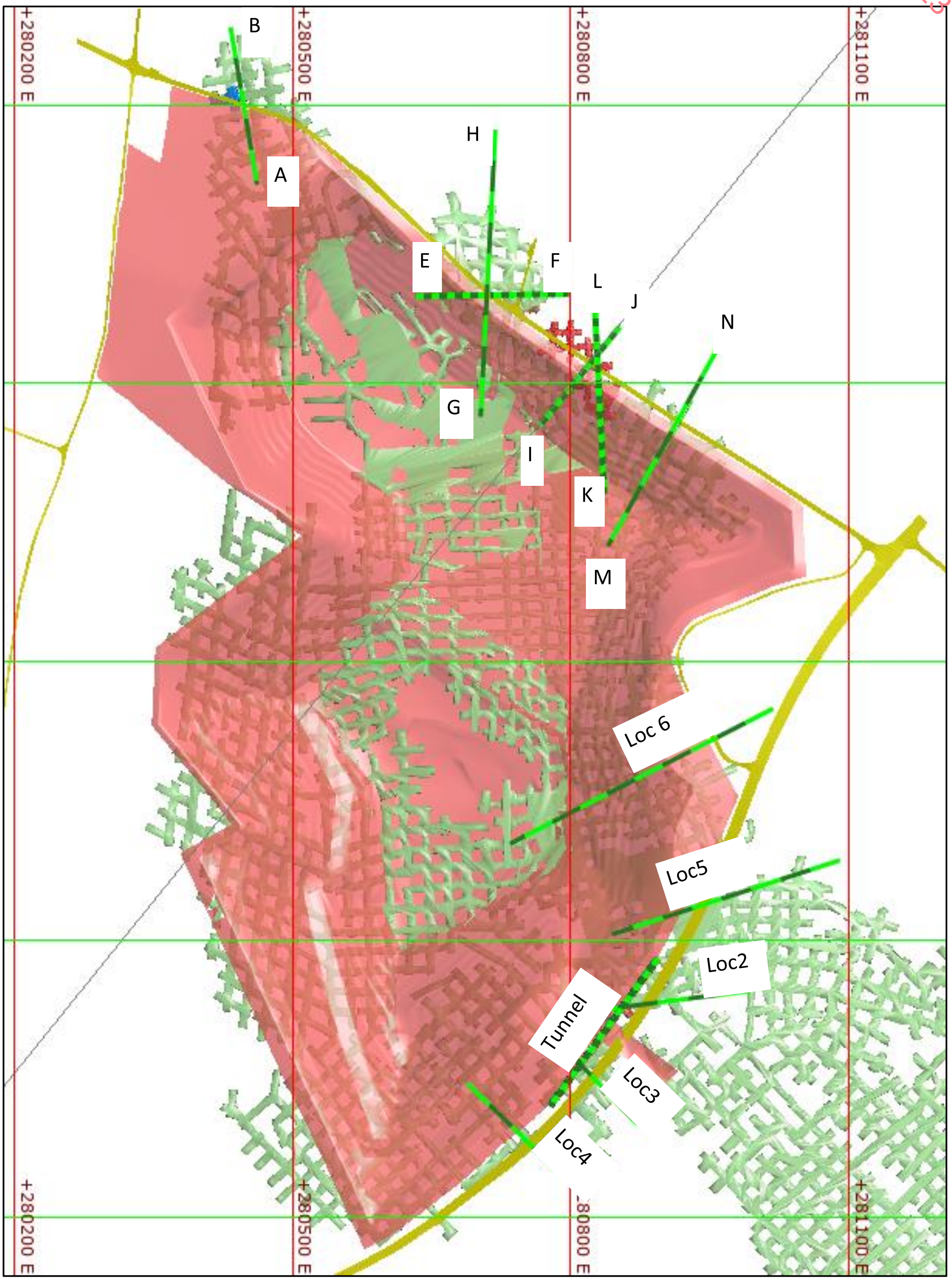


Figure 2-1: Plan of Proposed Mine Showing Section Lines for Analysis

In order to assess the impact of the mine on the underground workings and thus on the overlying L4900 and R179 roads, models are set up to calculate the additional deformation at the road surface generated following the excavation of the mine slopes adjacent to the roads.

Displacements for each cross section are reported in Table 2-1 and shown in Figure 2-2: through Figure 2-12:. Mining of the pit slopes does not influence the stability of the underground workings (especially those directly underneath the roads) and therefore does not affect the roads.

Details of the models are shown in Appendix A.

Table 2-1: Additional displacements (mm) due to open pit mining, for each cross section.

Road	Section	Displacement (mm)
L4900	A1_I-J	3.04
	A1_K-L	1.99
	A2_M-N	2.2
	B3_E-F	1.33
	B3_G-H	2.43
	B4_A-B	2.81
R179	Loc2	0.50
	Loc3	5.5
	Loc4	7.15
	Loc5	4.65
	Loc6	1.60

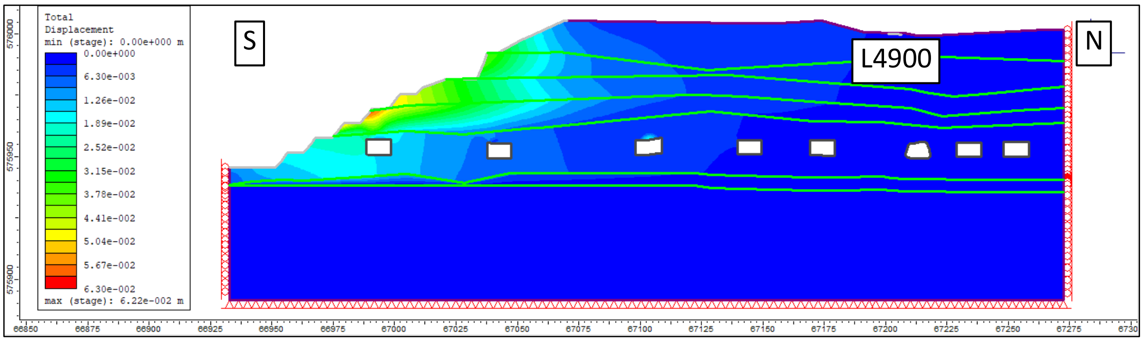


Figure 2-2: Section A-B. Additional displacements due to open pit mining.

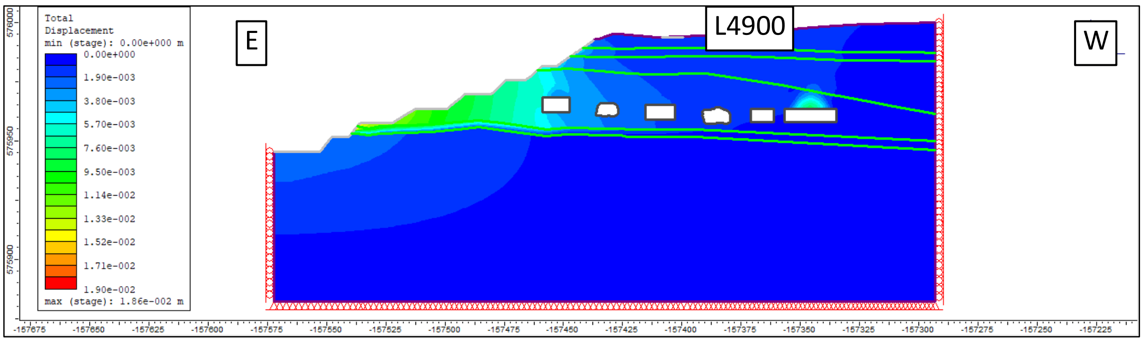


Figure 2-3: Section E-F. Additional displacements due to open pit mining.

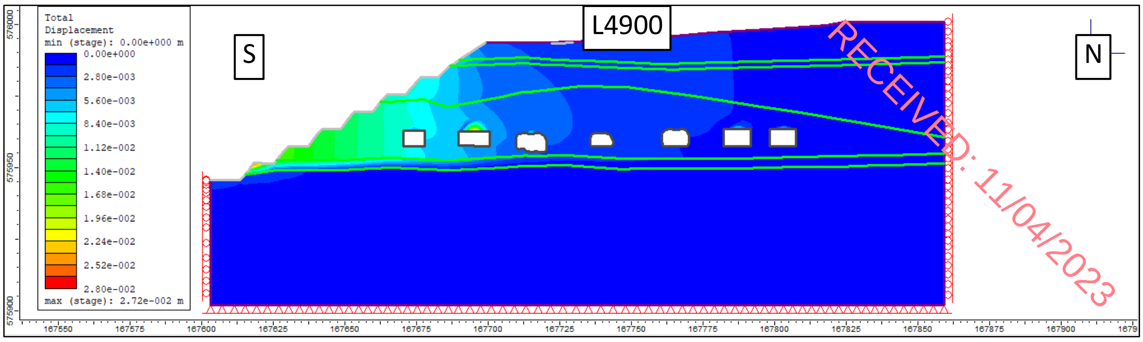


Figure 2-4: Section G-H. Additional displacements due to open pit mining.

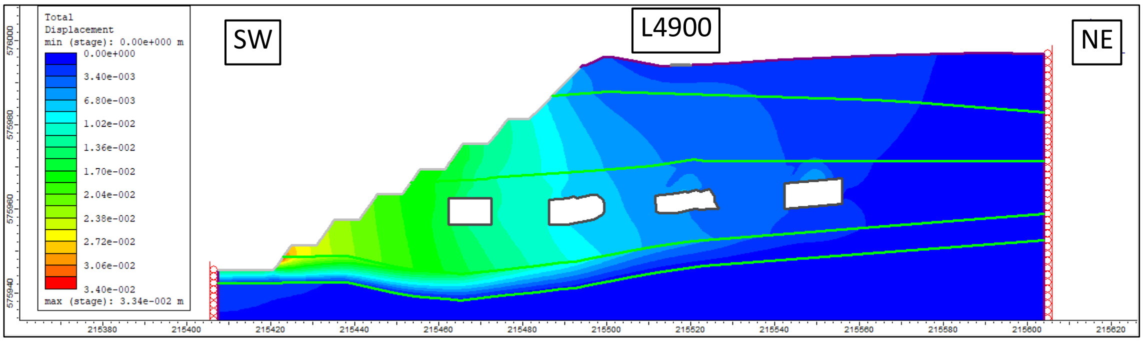


Figure 2-5: Section I-J. Additional displacements due to open pit mining.

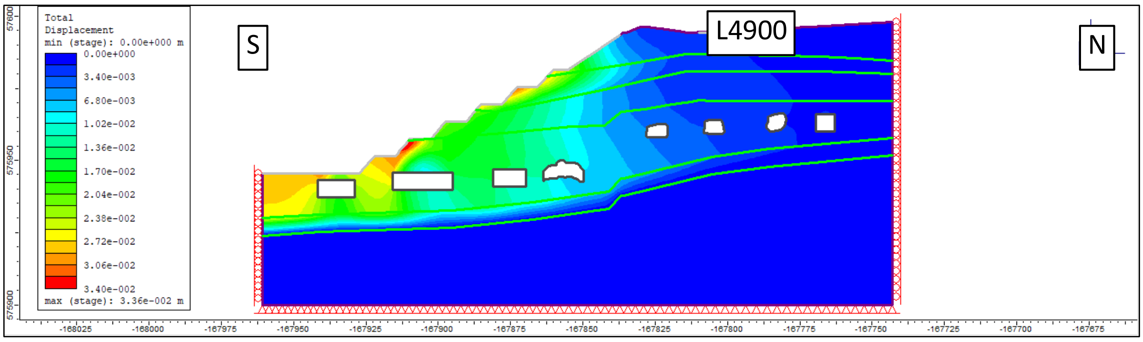


Figure 2-6: Section K-L. Additional displacements due to open pit mining.

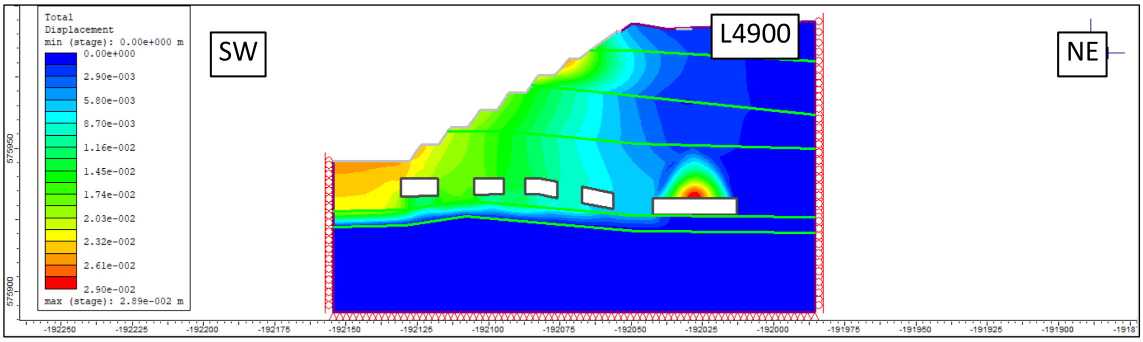


Figure 2-7: Section M-N. Additional displacements due to open pit mining.

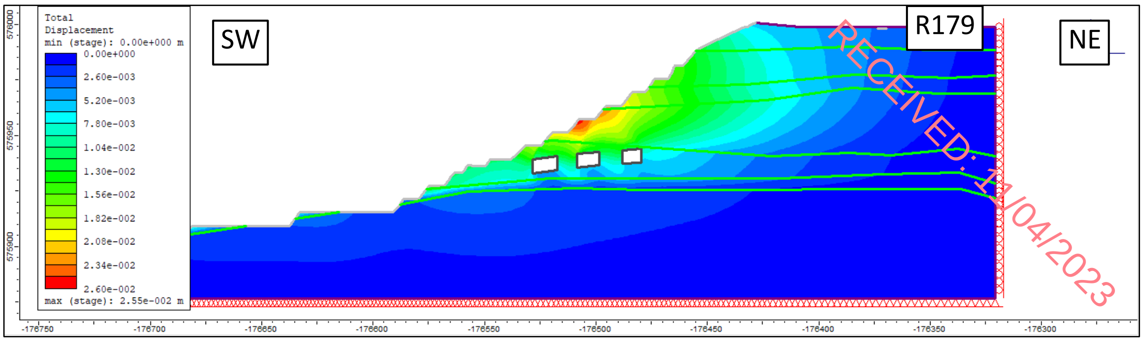


Figure 2-8: Section Loc6. Additional displacements due to open pit mining.

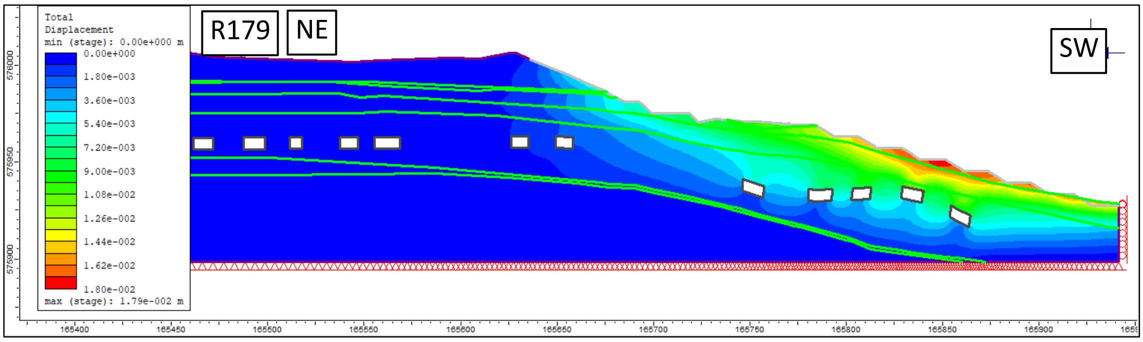


Figure 2-9: Section Loc5. Additional displacements due to open pit mining.

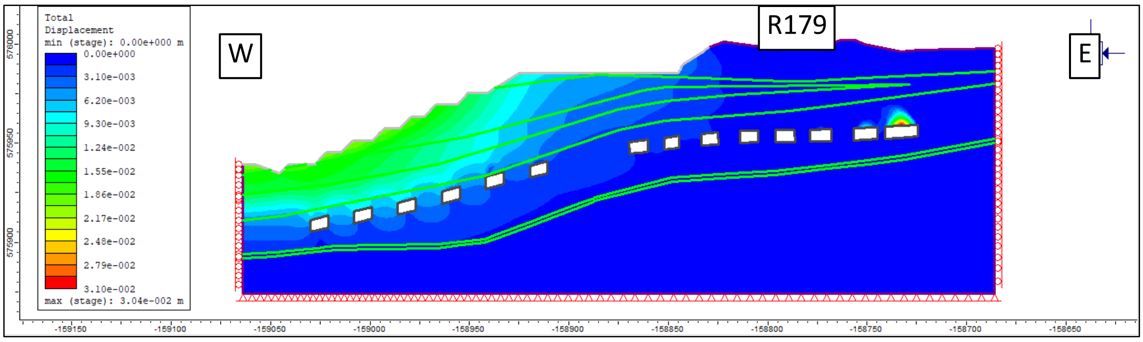


Figure 2-10: Section Loc2. Additional displacements due to open pit mining.

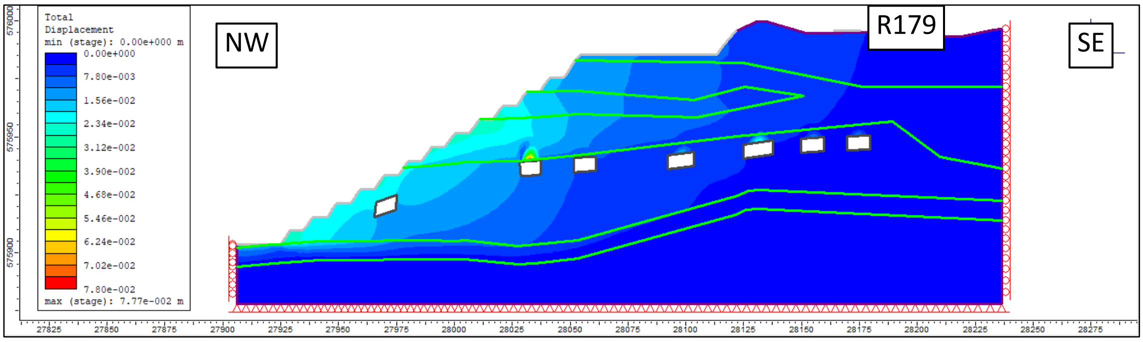


Figure 2-11: Section Loc3. Additional displacements due to open pit mining.

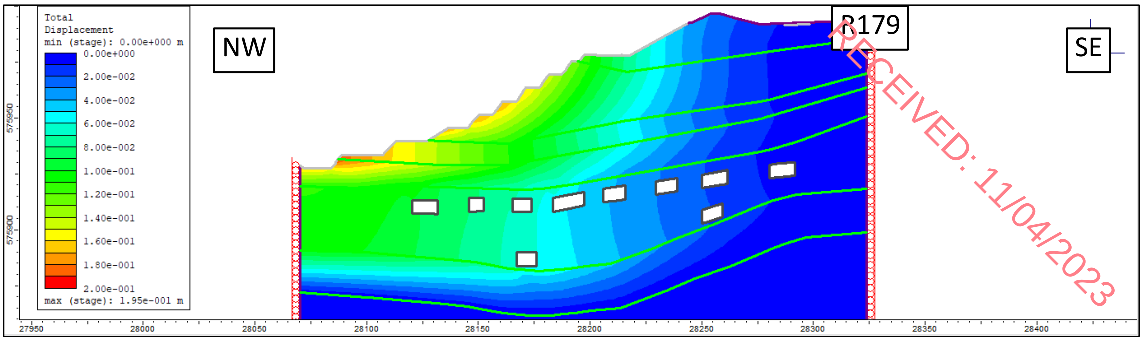


Figure 2-12: Section Loc4. Additional displacements due to open pit mining.

3 IMPACT OF CUT AND COVER TUNNEL ON THE UNDERGROUND WORKINGS BELOW R179

The tunnel is to be excavated 32m above pillars which are at least 19m×35m (Figure 3-1). These pillars are deemed large enough that the tunnel should not affect them.

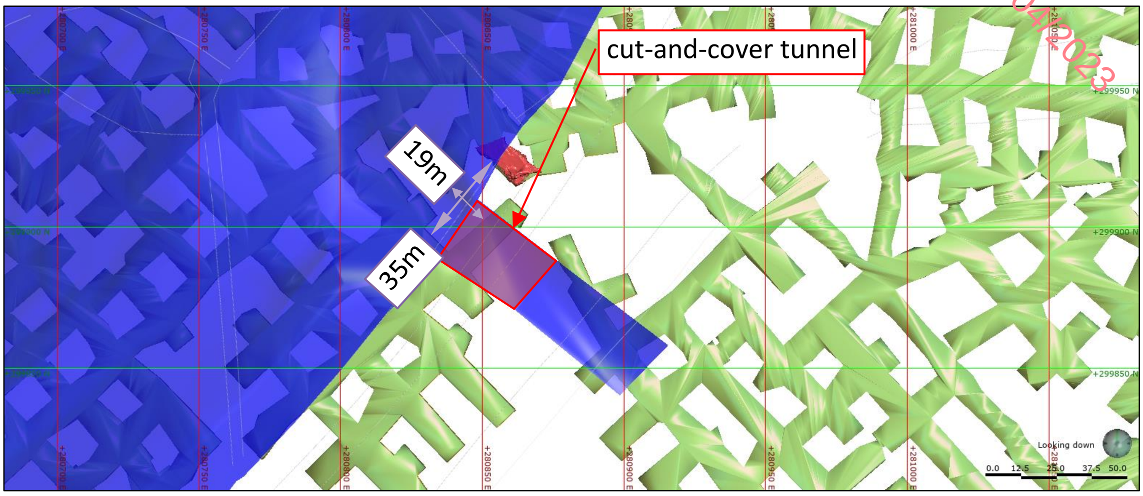


Figure 3-1: Position of the tunnel relative to underground workings (plan view).

As shown in Figure 3-2: through Figure 3-4:, excavation of the cut and cover tunnel does not affect the stability of the underground workings. Displacements of the room roofs due to excavation of the tunnel are shown in Table 3-1 (see Figure 3-4: for reference). These movements are upwards. This is due to elastic rebound of the rock after removal of material to from the tunnel cut.

Table 3-1: Displacements (mm) of room roofs near the cut-and-cover tunnel.

Room 1	Room 2	Room 3	Room 4	Room 5	Room 6
0.2	0.3	0.7	1.0	0.7	0.5

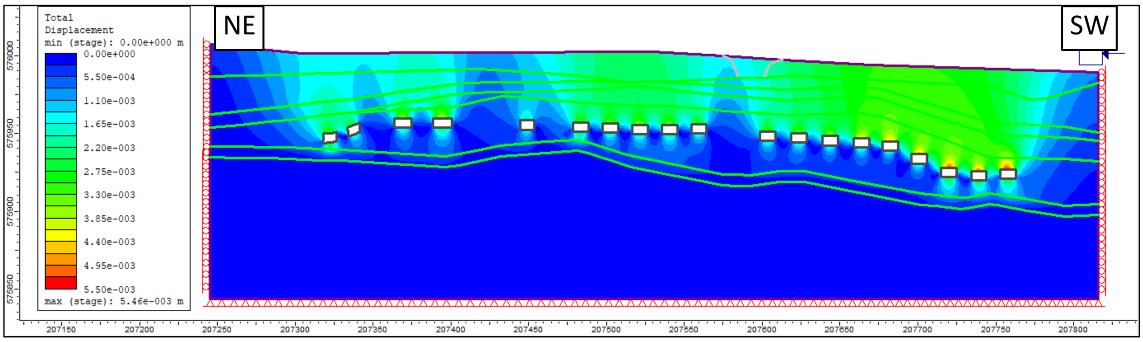


Figure 3-2: Section along R179. Displacements (mm) after mining of underground workings.

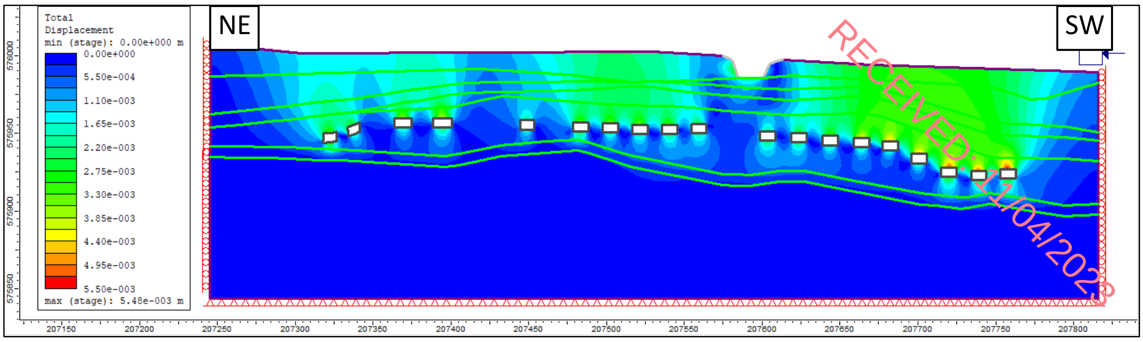


Figure 3-3: Section along R179. Displacements (mm) after excavation of tunnel.

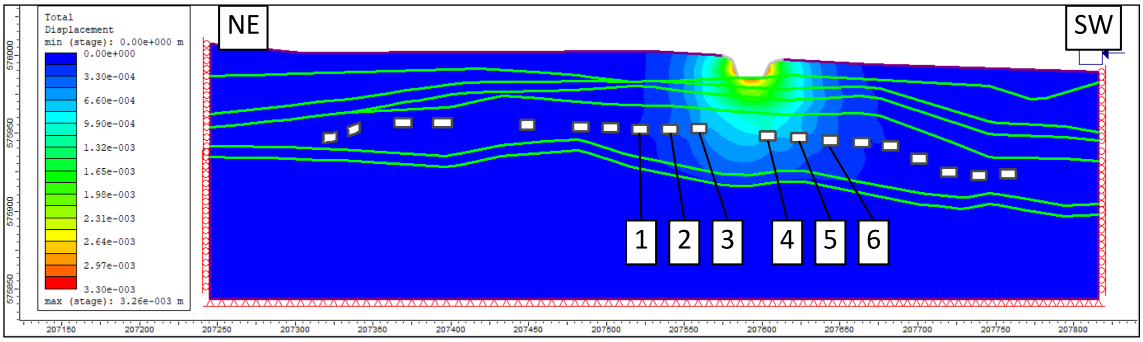


Figure 3-4: Section along R179. Additional displacements (mm) after excavation of tunnel.

4 IMPACT OF BACKFILLING

During open pit mining the underground workings will be fully drained. The existing Knocknacran mine will be backfilled to the original pre-mining ground surface. The proposed new open pit mine will also be partially backfilled which will result in development of a pit lake. The underground workings left in the unmined area between the two quarries and below the R179 road will ultimately become filled with groundwater. The potential sources of groundwater will be infiltration though the backfill in both open pit mines and ground water seepage from surface. Figure 4-1: is a cross section through both mines showing the relationship between the backfill, the remaining underground workings and the R179 road noting that:

- the present-day topography is in red,
- the planned west pit is in blue,
- the underground workings are in green and orange,
- the backfill profile is in green.

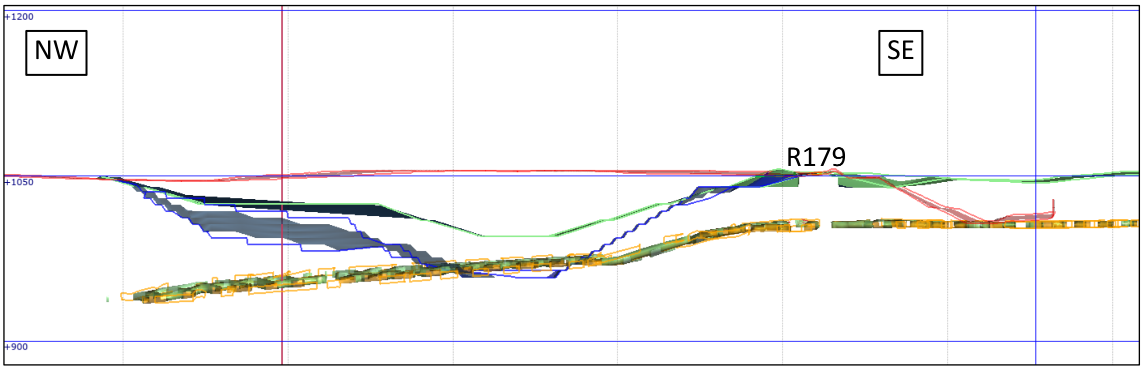


Figure 4-1: NW-SE cross section through the conveyor tunnel.

The long-term stability of the underground workings will be impacted in the following ways:

1. Infiltration of water into cracks and joints within the gypsum that may result in blocks dislodging from the roof and pillar side walls. This will likely only occur to blocks that are already partly disconnected from the surrounding rock mass and are in a state of incipient collapse providing no material support to the mine elements in which they are located. The introduction of water completes a process that was in progress.

2. Dissolution of gypsum by flowing water through the mine workings.

SRK has considered the potential for dissolution of the mine workings that will be left under the R179 road, and between the two pits, after closure (see Figure 4-1). Two distinct scenarios arise in this case:

- flow of water saturated with respect to gypsum,
- flow of undersaturated or “fresh” water, for example surface water runoff or direct precipitation to the pit lake

Estimating the likelihood of each is beyond the scope of this report. However, since the former scenario is much more favourable for long-term stability than the latter (see below), mitigation measures are proposed to minimise fresh water flow.

A high-level geochemical calculation was made to better understand the likely rate of gypsum dissolution as water flows through the mine workings under predicted post-closure groundwater flow conditions. It was initially assumed that the chemistry of groundwater flowing into the mine workings was similar to that which has previously been measured in the area, namely saturated with respect to gypsum. The post-closure groundwater flow gradient was assumed to be the north-east, towards Lough Fea, and following topography. The calculation (made using PHREEQC geochemical modelling software) shows that an average of around 7 cm of gypsum would dissolve from the exposed surface areas of the mine workings every 100 years. Dissolution of gypsum at this rate would require an elapsed time of between 6,000 and 7,000 years for a standard pillar of size 12m by 12m to be reduced in size to a dimension where it becomes potentially unstable.

However, SRK notes that if the chemistry of the water entering the mine were to be that of fresh surface water, and therefore strongly undersaturated with respect to gypsum, then the rate of mine dissolution would be very much greater. For this reason, it is imperative that the backfill design and placement for both mines ensures that surface water cannot directly enter the mine. For example, the backfill material that will be used to line the proposed Knocknacran West Open Pit should be of sufficiently low permeability so as to prevent infiltration of fresh (undersaturated) surface water from the pit lake, through the backfill and into the mine workings.

5 EFFECT OF BLASTING ON MPBX

As shown in Figure 5-1: (taken from section M-N), the extensometer reference heads are roughly 90m from the nearest blast. Additionally, Road R179 is approximately 130m from the nearest blast in the upper gypsum (on account that plans are in place to install extensometers under R179). The extensometer anchors are sitting within the gypsum roof beam (1m above the room and 1m below the upper gypsum contact), at distances of 80m-85m.

As advised by RST Instruments Ltd, the extensometer manufacturers, both the extensometers and the grouted anchors will be unaffected by blasting. Any measurement from the MPBX sensors taken while blasting is being carried out, however, should be ignored due to the accelerations affecting the sensor. Moreover, to ensure long-term performance of the system sensors should be recalibrated annually (or if there is a discontinuity in the readings).

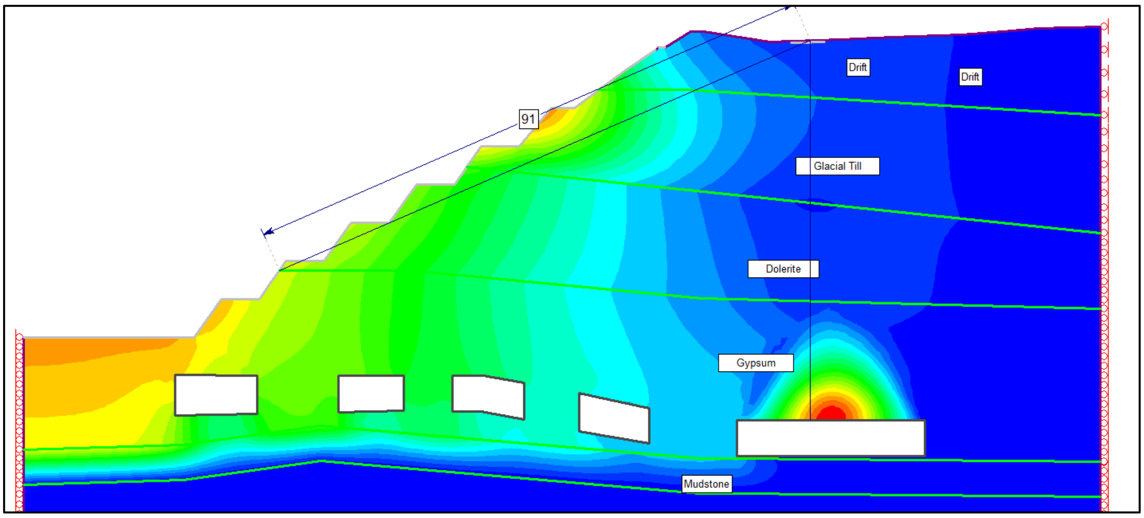


Figure 5-1: Distance between blast in MPBX anchor points.

For and on behalf of SRK Consulting (UK) Limited

Neil Marshall,
Corporate Consultant - Geotechnical,
Project Manager
SRK Consulting (UK) Limited

Iñaki Garcia,
Geotechnical Consultant,
SRK Consulting (UK) Limited

APPENDIX

A NUMERICAL MODELLING

Material Parameters

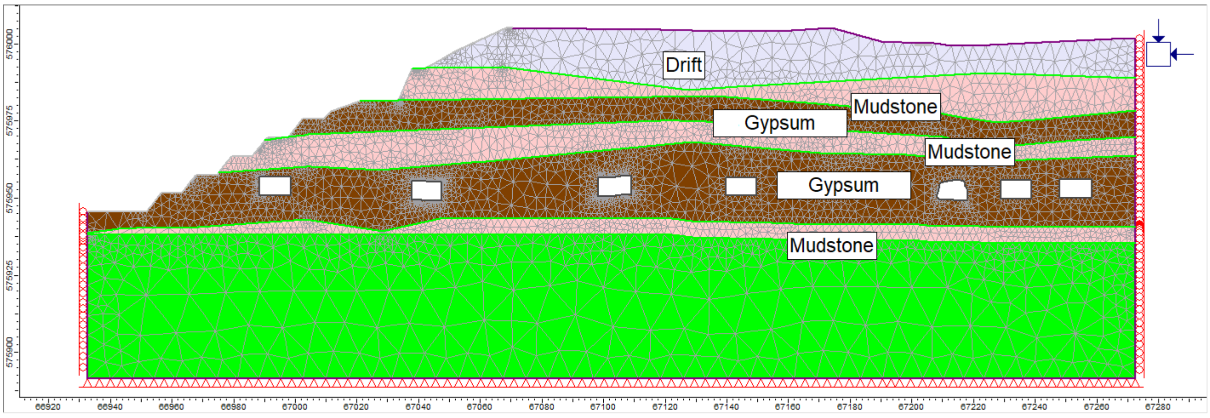
ZONE	Lithology	IRS	RMR	GSI	Unit Weight (MN/m3)	c (MPa)	phi (°)	Young's Modulus (MPa)	Poisson's Ratio
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
A1	Glacial Till	N/A	N/A	N/A	0.018	0.2	20	100	0.3
	Dolerite	11	28	23	0.020	0.06	35	1000	0.3
	Gypsum	25	54	49	0.023	0.264	49.78	4875	0.15
	Mudstone	N/A	N/A	N/A	0.020	0.4	20	200	0.2
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
A2	Glacial Till	N/A	N/A	N/A	0.018	0.3	20	100	0.3
	Dolerite	17	29	24	0.020	0.06	35	1000	0.3
	Gypsum	21	59	54	0.023	0.287	49.7	4875	0.15
	Mudstone	N/A	N/A	N/A	0.020	0.4	20	200	0.2
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
B3	Glacial Till	N/A	N/A	N/A	0.018	0.3	20	100	0.3
	Dolerite	1	27	22	0.020	0.06	35	1000	0.3
	Gypsum	22	57	52	0.023	0.273	49.59	4875	0.15
	Mudstone	N/A	N/A	N/A	0.020	0.4	20	200	0.2
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
B4	Glacial Till	N/A	N/A	N/A	0.018	0.3	20	100	0.3
	Dolerite	15	29	24	0.020	0.06	35	1000	0.3
	Gypsum	25	57	52	0.023	0.298	50.39	4875	0.15
	Mudstone	N/A	N/A	N/A	0.020	0.4	20	200	0.2
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
Tunnel	Mudstone	N/A	N/A	N/A	0.020	0.13	30	1260	0.3
	Upper Gypsum	N/A	N/A	N/A	0.023	0.25	43	4875	0.15
	Lower Gypsum	N/A	N/A	N/A	0.023	0.21	43	4875	0.15
	Basal Mudstone	N/A	N/A	N/A	0.020	0.13	30	1260	0.3
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
Loc2	Mudstone	N/A	N/A	N/A	0.020	0.13	30	1260	0.3
	Upper Gypsum	N/A	N/A	N/A	0.023	0.25	43	4875	0.15
	Lower Gypsum	N/A	N/A	N/A	0.023	0.21	43	4875	0.15
	Basal Mudstone	N/A	N/A	N/A	0.020	0.13	30	1260	0.3
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
Loc3	Mudstone	N/A	N/A	N/A	0.020	0.13	30	1260	0.3
	Upper Gypsum	N/A	N/A	N/A	0.023	0.25	43	4875	0.15
	Lower Gypsum	N/A	N/A	N/A	0.023	0.21	43	4875	0.15
	Basal Mudstone	N/A	N/A	N/A	0.020	0.13	30	1260	0.3
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
Loc4	Gypsum	N/A	N/A	N/A	0.023	0.29	49.7	4875	0.15
	Mudstone	N/A	N/A	N/A	0.020	0.40	20	200	0.2
	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
Loc5	Mudstone	N/A	N/A	N/A	0.020	0.13	30	1260	0.3
	Upper Gypsum	N/A	N/A	N/A	0.023	0.25	43	4875	0.15

	Lower Gypsum	N/A	N/A	N/A	0.023	0.21	43	4875	0.15
	Basal Mudstone	N/A	N/A	N/A	0.020	0.13	30	1260	0.3
Loc6	Drift	N/A	N/A	N/A	0.020	0.06	19	560	0.3
	Mudstone	N/A	N/A	N/A	0.020	0.13	30	1260	0.3
	Upper Gypsum	N/A	N/A	N/A	0.023	0.25	43	4875	0.15
	Lower Gypsum	N/A	N/A	N/A	0.023	0.21	43	4875	0.15
	Basal Mudstone	N/A	N/A	N/A	0.020	0.13	30	1260	0.3

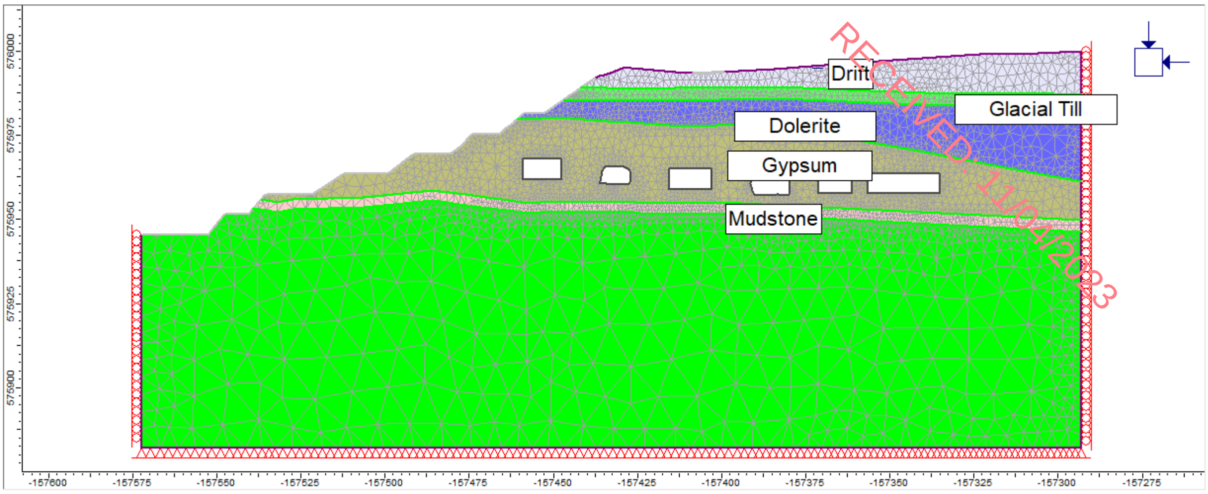
Geometry and Materials

No. of elements.

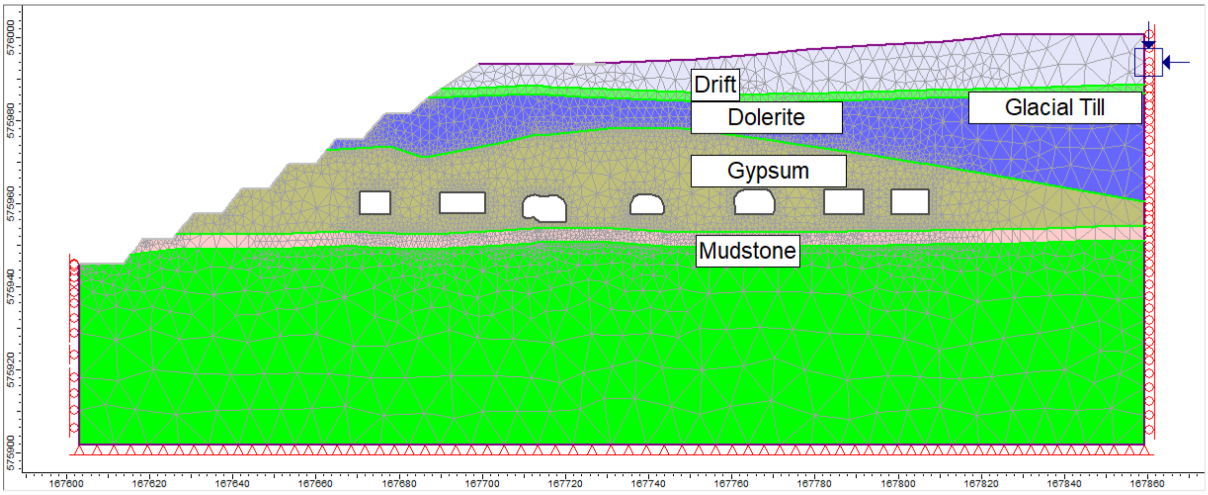
Model	No. of elements
A1_I-J	21540
A1_K-L	7287
A2_M-N	6783
B3_E-F	9496
B3_G-H	10044
B4_A-B	12263
Tunnel	5982
Loc2	11596
Loc3	9731
Loc4	7596
Loc5	9756
Loc6	17914



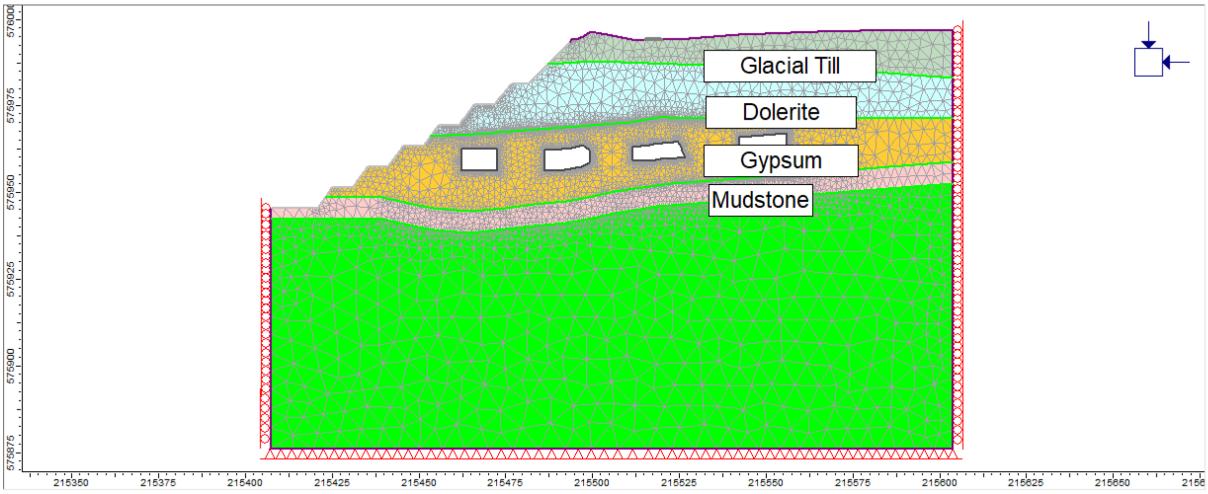
A-B. Mesh and materials.



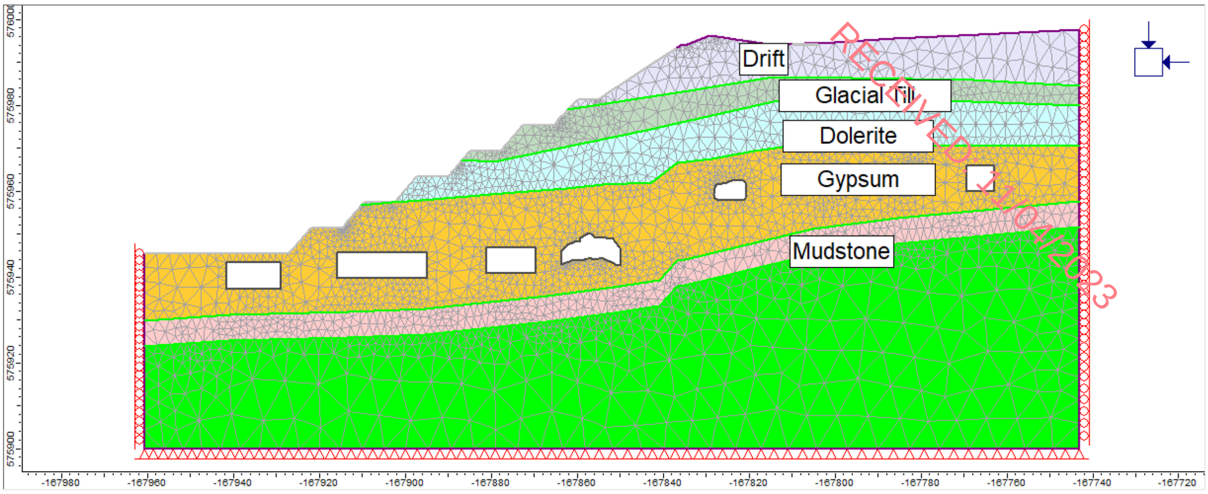
E-F. Mesh and materials.



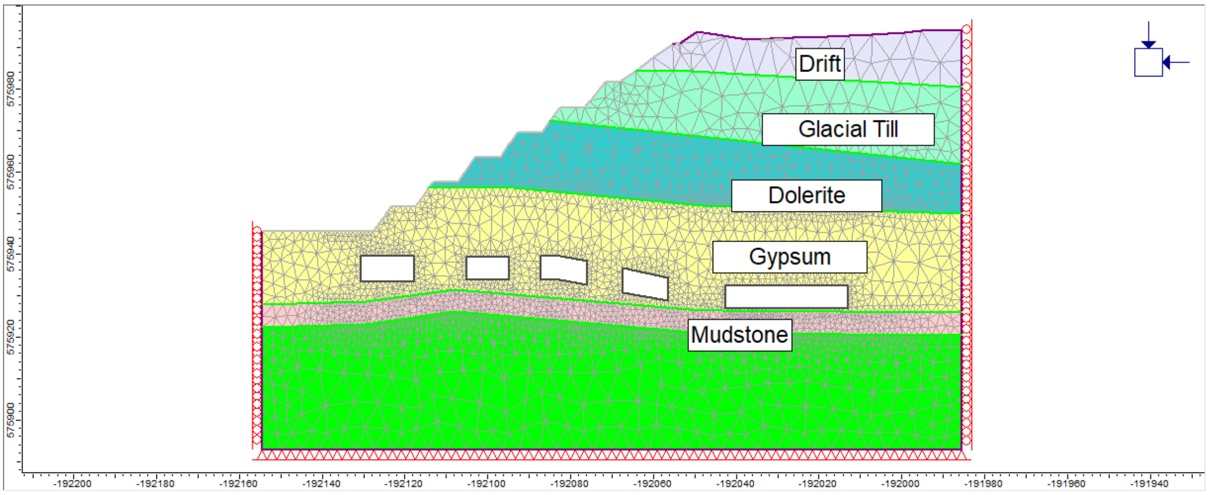
G-H. Mesh and materials.



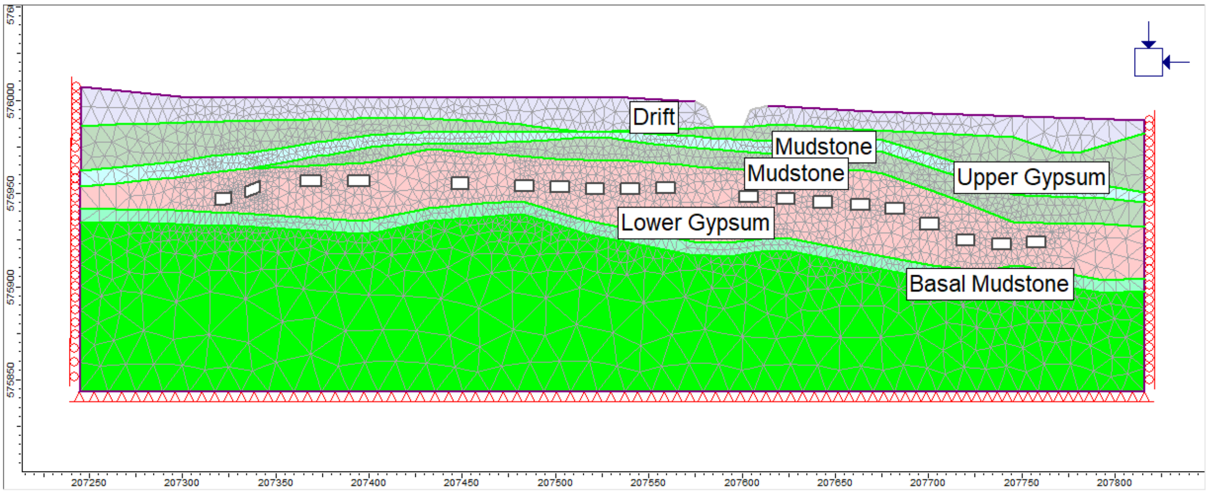
I-J. Mesh and materials.



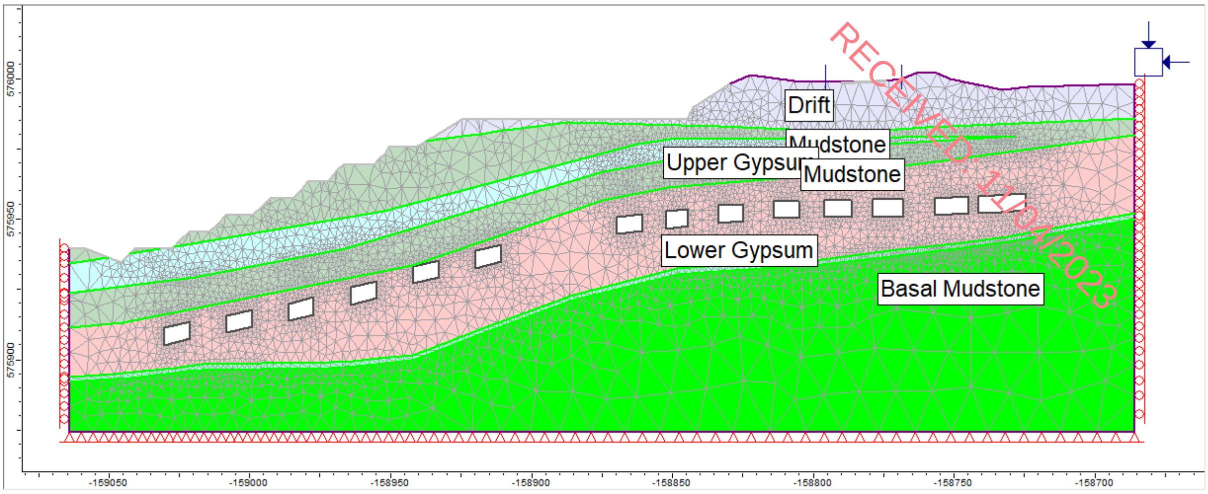
K-L. Mesh and materials.



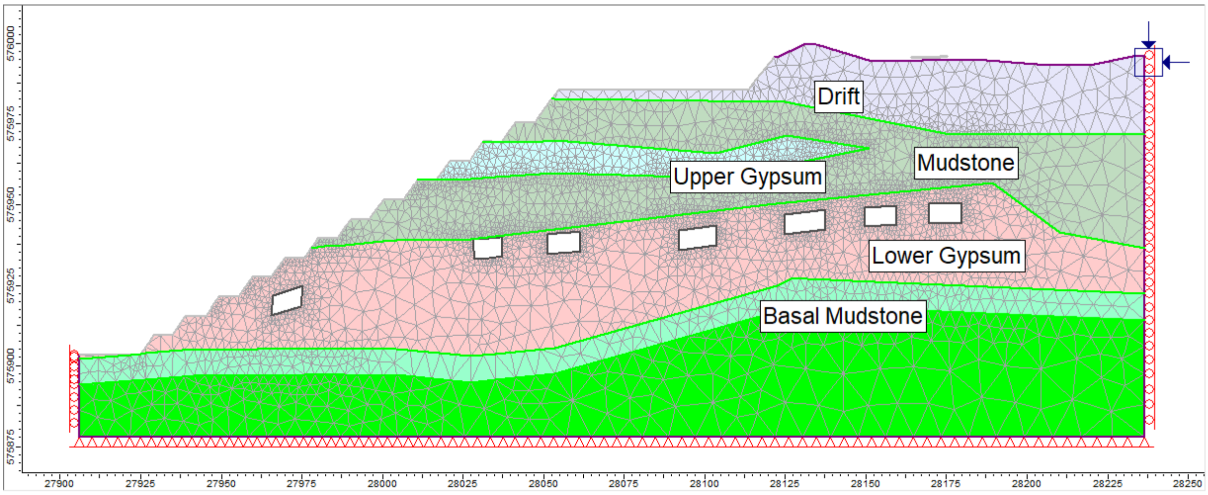
M-N. Mesh and materials.



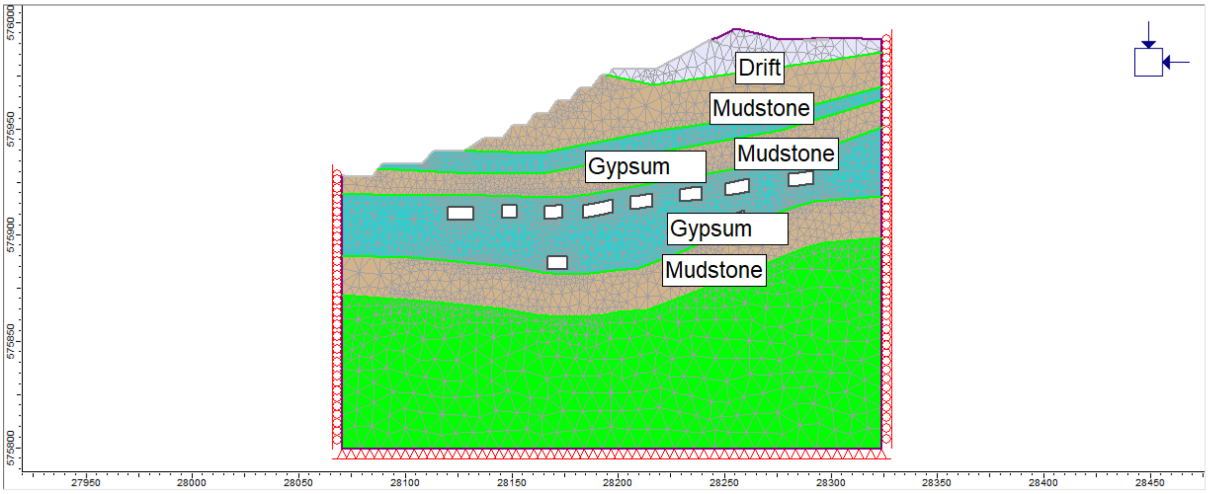
Tunnel. Mesh and materials.



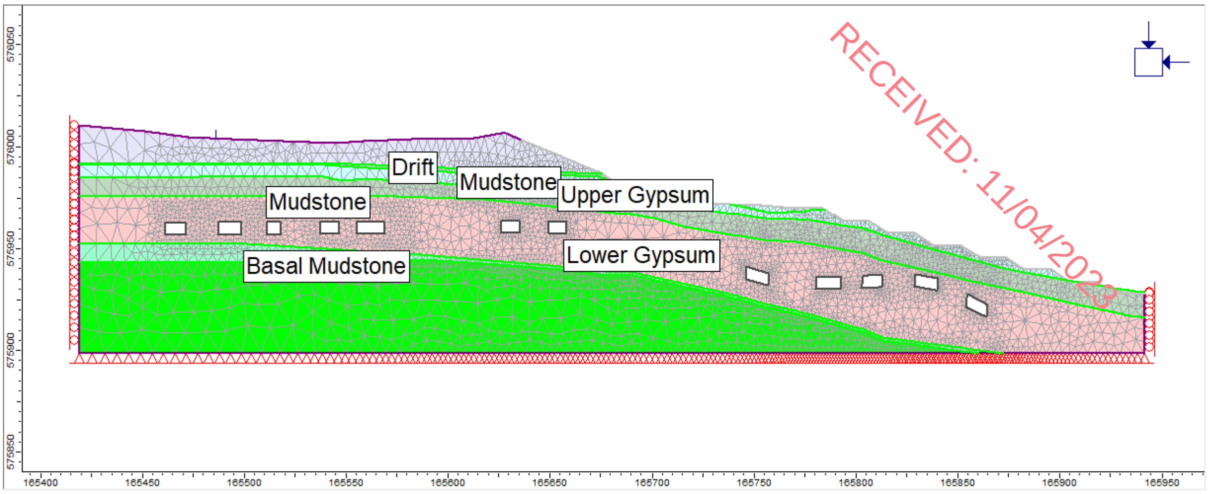
Loc2. Mesh and materials.



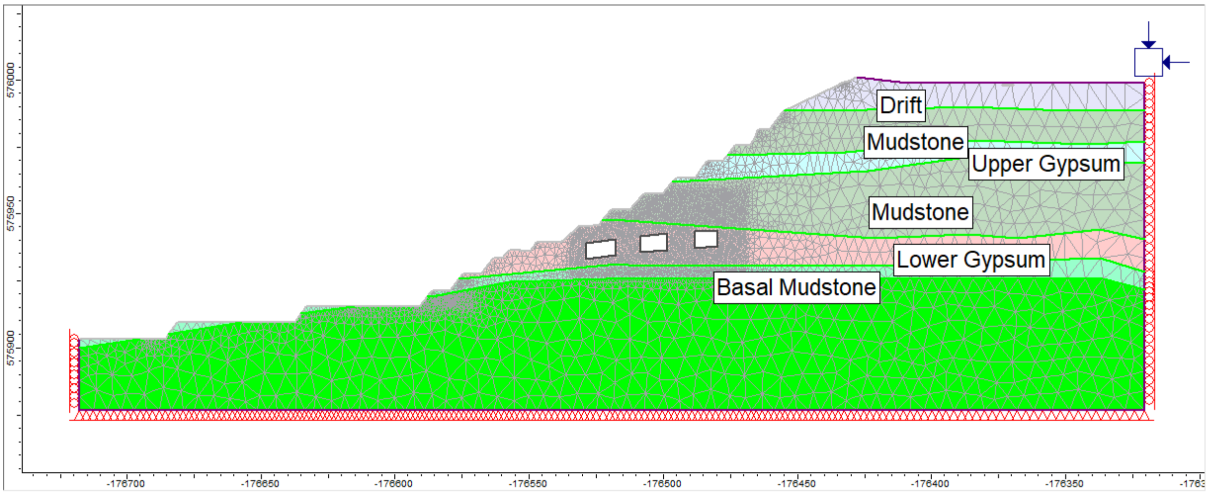
Loc3. Mesh and materials.



Loc4. Mesh and materials.



Loc5. Mesh and materials.



Loc6. Mesh and materials.

APPENDIX 7.14
Impact of Construction and Mining Vibration - SRK - July 2022

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External Memorandum

To:	Pat O'Connor	From:	Neil Marshall
Company:	Saint-Gobain Mining Ireland	Project Number:	UK31696
Copied to:	Benson Plunkett	Project Title:	Knocknacran West - Vibration Impacts
File Ref:	31696_Report_1_Vibration_Draft(V3).docx	Date:	12 July, 2022
Subject:	**Impact of Construction and Mining Vibration**		

1 RFI POINTS

The following RFI points have been combined as they largely reference similar quarrying impacts.

RFI Ref: Point 6.c – Construction and Mining Risks

It is considered that risks relating to the construction and mining operations at the Knocknacran West open cast mine have not been adequately considered. There will potentially be an increase in personnel, plant and temporary buildings/compound, stockpiles etc. during the proposed development. Consideration of the risks of subsidence events/sinkholes during or caused by construction and mining activities and the cumulative impacts are not adequately assessed or reported. The potential risks of subsidence caused by an increase in personnel, plant, temporary buildings/compounds and stockpiles should be comprehensively addressed in Chapters 5, 14 & 15 of the EIAR.

RFI Ref: Points 10.g.i to 10.g.iii, 10.f – Mining Vibration

Clarity is requested in relation to the assessment of the impact of under-lying mining tunnels in relation to vibration transmission, and to the impacts of vibration impacting the stability of these tunnels. Specifically the applicant is requested to provide details of the following:

- How has the stability of the under-lining mining tunnels been assessed?
- What mitigation measures will be in implemented in this regard.
- What is the risk of instability arising in the mining tunnels due to vibration - provide an assessment of this.

Within the vibration chapter, consideration is requested on the large—scale movement of soils, over/interburden and the potential for vibration impacts. Furthermore, it should be clarified, what limits will be applied in this instance and details of the monitoring and reporting carried out in respect of non-gypsum blasting, i.e. blasting within the inter burden layer.

2 RESPONSE

2.1 Potential Ground Vibration Sources and Receptors

By far the largest cause of vibration within and adjacent to the quarry will be blasting. Heavy equipment – bulldozers, excavators, haul trucks etc – will create ground vibrations but these will be localised.

The ground vibration receptors within the Knocknacran West quarry environs will be:

1. The underground workings outside the quarry boundary and that will remain below the R179 and L4900 roads, and

2. The ground within the quarry perimeter that has been affected by the various subsidence events and crown hole occurrences that have manifested since and before the major event that occurred in September 2018.

Figure 1 below shows the relationship of the quarry to the subsidence areas, mine area and roads.

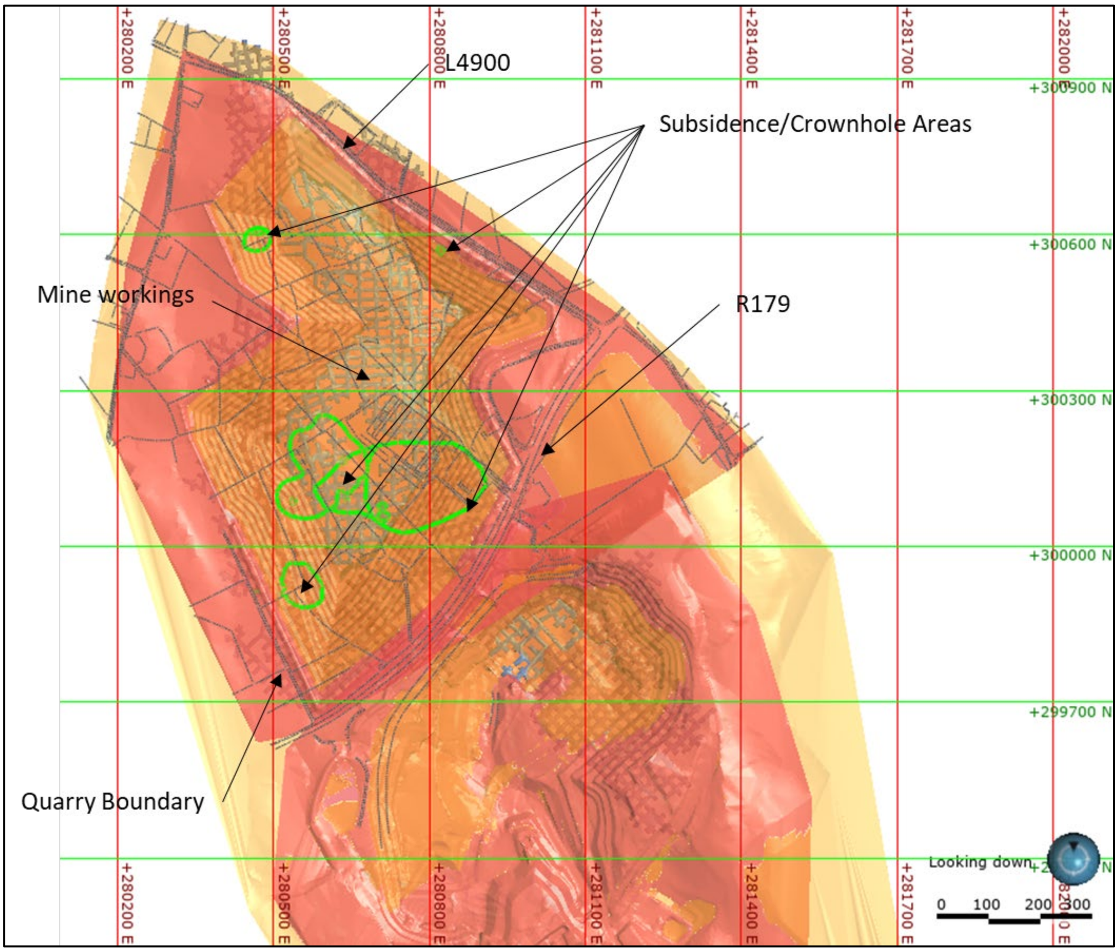

Figure 1: Plan of Proposed Knocknacran West Quarry Showing Interaction with Drumgoosat Mine Workings and Subsidence Locations

2.2 Construction and Mining Risks

The potential risk of subsidence caused by an increase in personnel, plant, temporary buildings/compounds and stockpiles is considered to be extremely low. Temporary buildings, compounds and stockpiles will likely be there for the duration of the quarrying operations. They will therefore be located away from the footprint of the quarry in areas where underground mining has not taken place so their impact in subsidence or more possibly the effect of the presence of the underground mine on surface structures will be negated. Effects of the movement of personnel and plant over the site is addressed in Section 2.3, below.

2.3 Construction Ground Vibration

Ground vibration is normally measured as peak particle velocity (PPV) in mm/second. PPV reduces or attenuates as the distance from the source of vibration increases. Relationship between PPV and distance from source for a number of types of construction plant has been developed by Wiss (1981)[1]. This relationship is shown in Figure 2.

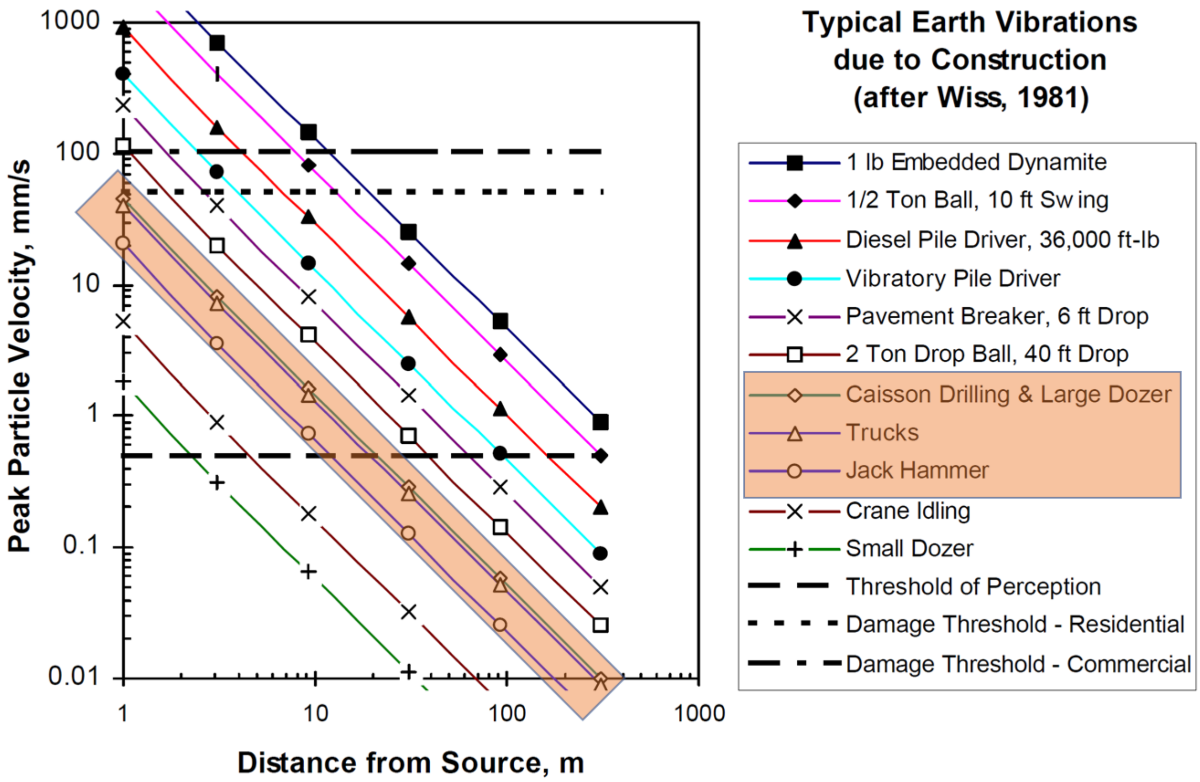

Figure 2: Vibration Attenuation Graph for Typical Construction Equipment

[1] Wiss, J. F. (1981), "Construction Vibrations: State of the Art," Journal of the Geotechnical Division, ASCE, v. 107, no. GT2, Proc. Paper. 16030, Feb. 1981, pp. 167-181.

Highlighted in orange are the main types of equipment that would operate in a quarry environment, large dozers, trucks and blasthole drills (jack hammers). Also included are various typical vibration thresholds – perception (0.5 mm/sec), damage to residential buildings (50 mm/sec) and damage to commercial buildings (100 mm/sec). It can be seen that for all of these equipment any vibration generated by them would become imperceptible from 10 m to 20 m away from the equipment. All equipment considered generates vibration which lies below the residential damage threshold at a distance of 1 m away.

With regards vibration damage criteria for underground workings sources in the literature quote the following:

- Underground hard rock caverns, 15 m to 18 m span – Limiting PPV 70 – 100 mm/sec[2]
- Damage threshold for underground works – Limiting PPV – 305 mm/sec[3]

Other vibration standards are quoted in the literature but those defined above represent the lower and upper end member of the range. The lower end member, from a Swedish vibration standard, refers to underground civil engineering structures excavated below built up areas. The lower bound standard will be used to estimate the impact of vibrations on Knocknacran Quarry.

Reference to Figure 2 shows that the lower bound limiting PPV necessary to cause damage to underground workings lies between the residential and commercial building damage threshold. None of the typical quarry equipment will produce vibrations of sufficient intensity to reach the limiting PPV threshold. The shallowest underground workings are between 25 m and 30 m below surface, where the lower seam upper horizon is located in the northern part of the quarry. At this depth the equipment vibration will be sufficiently attenuated that vibrations will be almost imperceptible. On this basis vibrations from heavy equipment are highly unlikely to cause damage to the gypsum roof beams even if equipment is working directly on top of the gypsum. Any damage that may result when working on top of the gypsum roof beams will be related to the structural integrity of the gypsum defined by its thickness, presence of planes of weakness within the gypsum and the weight of the equipment operating on the gypsum. The risks and risk mitigation associated with this situation will be addressed by method statements and standard operating procedures related to mining above and through underground workings. This is discussed elsewhere.

[2] Persson P A, Holmberg R, Lande G and Larsson B (1980). Underground blasting in a city. Subsurface space: Proc Rockstore ‘80, Stockholm, ed. M Bergman, Vol 1, pp 199-206. Pergamon Press, Oxford.

[3] Siskind, D. E. (2000). Vibrations from Blasting, International Society of Explosives Engineers, Cleveland, OH

2.4 Blasting Vibrations

Saint-Gobain Mining Ireland (SGMI) has maintained a database of blast monitoring information with over 450 blast records compiled over a 10 year period from 2012. The database comprises maximum PPV, distance from blast and maximum instantaneous charge weight. Within the database over 25% of the monitoring data did not register any vibrations because of the blast being too far away from the monitoring point and/or the maximum instantaneous charge weight being too small.

To establish the ground attenuation characteristics of a rock mass use is made of the relationship between PPV, charge weight and distance from charge. An empirical relationship between these parameters has the form –

$$PPV_t = A(D/\sqrt{Q})^b$$

where PPV_t = theoretical peak particle velocity
D = distance from the blast to the point at which the PPV is measured (m)
Q = weight of the charged detonated or maximum instantaneous charge (kg).

The parameter ($D/\sqrt{Q}$) is known as the scaled distance.

A and b are empirical constants which are established from actual blast measurements and are site specific. The SGMI blasting database has been used to establish these parameters for Knocknacran. The scaled distance and PPV for each monitoring record is plotted on a logarithmic scale graph. The equation of a straight line graph of the trendline through the points defines the empirical constants A and b. The resulting equation can then be used to calculate the theoretical PPV at a point generated by a blast at any distance and of any maximum instantaneous charge. This relationship can be used for quarry blast design to ensure that PPV thresholds are not exceeded in the vicinity of sensitive structures.

The scaled distance vs PPV graph for the SGMI database is presented in Figure 3. The best fit trend line (in yellow) through the datapoints has the equation:

$$PPV_{(t)} = 178.52(D/\sqrt{Q})^{-1.165}$$

Because there is quite a large variability in the data an upper bound trend line (in red) below which 95% of the monitoring points lie has been generated with the equation:

$$PPV_{(95)} = 350.0(D/\sqrt{Q})^{-1.165}$$

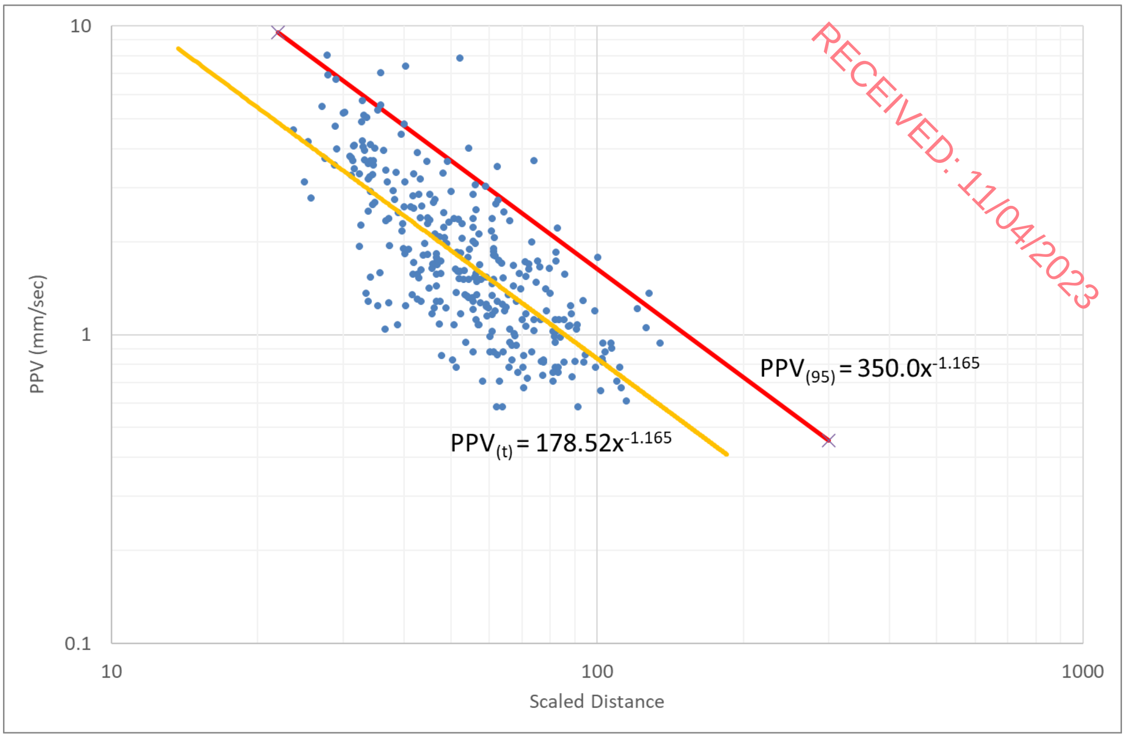

Figure 3: Scaled Distance/PPV Graph

To illustrate, Table 1 shows average and upper bound calculated PPV values at different distances from the blast and different charge weights detonated. For example the maximum theoretical peak particle velocity 100 m from a blast where the maximum instantaneous charge is 200 kg is estimated to be 18.3 mm/sec using the best fit equation. This increases to 35.8 mm/sec using the upper bound equation, still well below the lowest damage criterion, for residential buildings, of 50 mm/sec.

Table 1: Theoretical PPV Estimates

			Maximum Theoretical PPV (mm/sec)	
Charge Weight (kg)	Distance (m)	Scaled Distance	$PPV_{(t)}$	$PPV_{(95)}$
200	100	7.1	18.3	35.8
200	500	35.4	2.8	5.5
50	100	14.1	8.2	16.0
50	200	28.3	3.6	7.1

At Knocknacran East the average instantaneous charge weight that has been fired is 84 kg and the maximum has been 160 kg. From the data points in Figure 3 for this range of charge weights it can be seen that the maximum measured PPV has not exceeded 10 mm/sec. This is significantly below the vibration level required to cause damage to surface structures (50 mm/sec for residential buildings) or underground openings (70 mm/sec for hard rock caverns). In Knocknacran East SGMI has been careful in managing its blasts to ensure that vibrations are kept to a minimum. It is expected that this will continue in Knocknacran West. Blasting will be carried out only through the Upper and Lower Gypsum seams, with all other materials being free dug, and blasts will be designed and initiated to minimise vibrations in the area of the underground workings that will remain in-situ after quarrying operations have ceased. Based on these analyses, blasting at the Knocknacran West quarry is unlikely to have any significant effect on the stability of the existing underground excavations (mining tunnels).

2.5 Ground Subsidence

As shown in Figure 1 there are a number of areas within the proposed quarry limit that have been subject to subsidence due to pillar collapse or crownhole formation. These areas have been surveyed on surface and their extent has been well defined. The large depression and associated tension cracks that resulted from the collapse of the 12 m high pillars in September 2018 has been successfully remediated with tension cracks being filled in, buildings removed and the ground surface contoured. SGMI and its contractors will develop safe working procedures for operating above and through this unstable area. For quarrying, similar procedures will be adopted for working above and through the other subsidence zones on the site.

3 CONCLUDING REMARKS

The discussions above on heavy equipment and blasting initiated ground vibrations indicate that it is unlikely that these sources will generate sufficient vibrations to initiate any new subsidence. However as the quarry floor is excavated closer towards the mine areas that have been subject to subsidence the possibility of voids occurring in these areas could increase particularly within the gypsum units. An assessment of these areas will be addressed on an operational basis, bench by bench, with the working method forming part of the safe operating procedure for mining through voids and unstable ground described in the appropriate SOP's under development by SGMI.

For and on behalf of SRK Consulting (UK) Limited

Neil Marshall, Corporate Consultant - Geotechnical, **Project Manager** SRK Consulting (UK) Limited	Max Brown, Principal Consultant - Geotechnical, **Project Director** SRK Consulting (UK) Limited

APPENDIX 7.15
Long Term Mine Stability - SRK - July - 2022

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External Memorandum

To:	Pat O'Connor	From:	Neil Marshall
Company:	Saint-Gobain Mining Ireland	Project Number:	UK31696
Copied to:	Benson Plunkett	Project Title:	Mine Stability
File Ref:	31696_Report 2 Mine Stability_Draft(V4).docx	Date:	28 July, 2022
Subject:	**Long Term Mine Stability**		

1 RFI POINTS

The following RFI points have been combined within this document as they largely relate to mine subsidence outside the final quarry boundary and long term mine stability.

Subsidence Risk (RFI Ref: Points 14)

Major Accidents chapter identifies subsidence as a major accident risk, controlled through ongoing monitoring during the life of the project. However, further detail is requested on the continued risk, if any, posed to lands beyond the scope of this open cast mine, where subsurface mine shafts may be flooded post restoration of the proposed site.

Long Term Stability/Stabilisation of Workings Below Road (RFI Ref: Point 20.q)

Permanent Solution to existing mine workings that go under the existing public road network: The applicant has not clearly demonstrated how they propose to address the issue of future road subsidence on the public road network where previous mine workings exist. The applicant must submit comprehensive proposals, including design reports, drawings, and other appropriate design details that demonstrate how the applicant proposes incorporating a permanent solution to the mine workings that go under the public roads as part of their open cast works.

Finite Element Modelling Update (RFI Ref: Points 22.i.a and b)

The Finite Element (FE) models should be updated to include the existing open pit void and proposed backfilling. In particular the current section lines across the R179 should consider the impact of the existing void and the backfilling operation planned within the existing Knocknacran Open Cast Mine.

2 RESPONSE

2.1 Proposed Approach to the Query Response

SRK has undertaken numerous 2D finite element modelling studies looking at the stability of the underground workings below the R179 and L4900 roads. We have also undertaken modelling which included the impact of the proposed Knocknacran West Quarry which was carried out for the consultants that prepared the planning application. This work was carried out in 2019 and updated in 2021 when the quarry design was modified. Whilst all of the finite element simulations carried out by SRK indicated that the underground workings would remain stable during the full history of excavation of Drumgoosat underground followed by the excavation of Knocknacran West quarry. Now that the final design of the Knocknacran West Quarry has been developed the numerical modelling has, as requested, been updated to include the quarry mining and backfilling of both East and West quarries. The results of this modelling will be used to provide insight into the long term stability and subsidence risk associated with the mine elements that remain below the R179 and of the L4900 during quarry excavation and subsequent flooding. We also consider any additional subsidence risks posed to third partly land beyond the quarry perimeter and below which underground mine voids remain. Figure 1 shows the final landform after backfilling of Knocknacran East Quarry, partial backfilling of Knocknacran West Quarry and formation of the quarry lake. The layers forming the figure have been made transparent so that the extent of underground rooms and pillars remaining below the quarry slope face and beyond the quarry boundary can be seen. Analyses of the five cross sections around the boundary of Knocknacran West Quarry are discussed in the following sections.

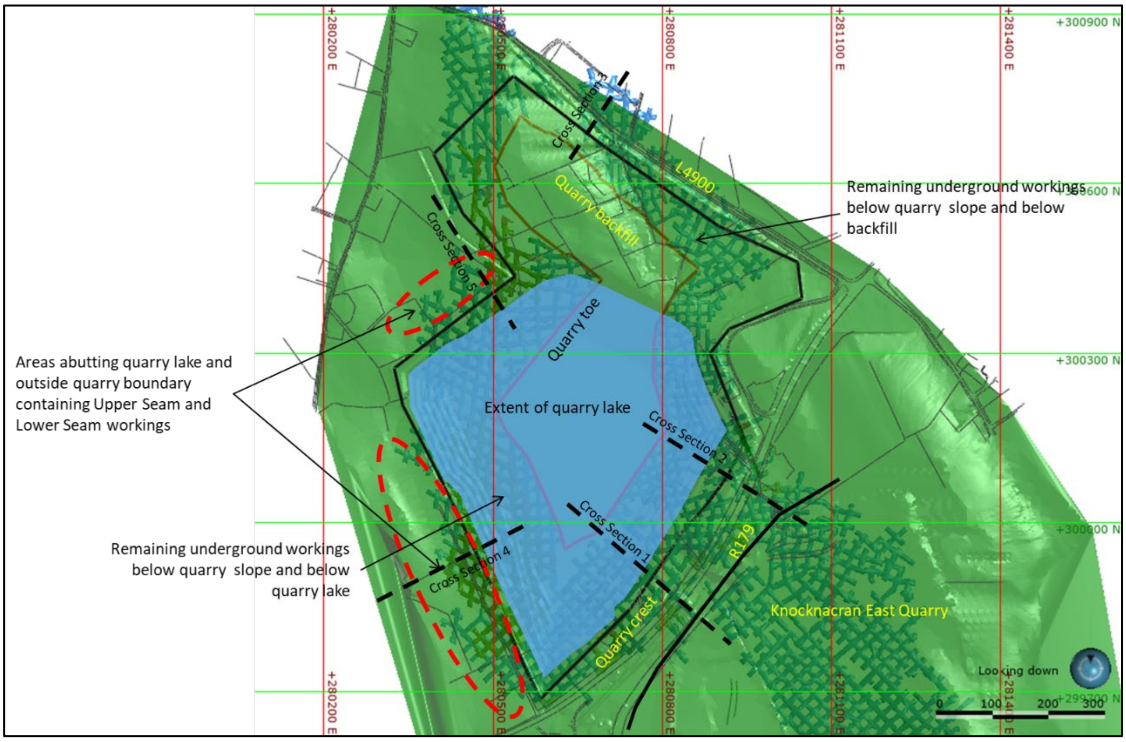

Figure 1: Final Quarry Landform showing Areas considered for Analysis

2.2 Finite Element Modelling – R179

The original request was to update the historical finite element modelling that had previously been reported to include the impact of the quarry voids. The orientation of the cross sections modelled were generated to assess the stability of specific underground excavations of concern. In order to properly integrate the quarry excavations into the modelling it becomes necessary to create new cross-sections orientated at right angles to the quarry slopes. Two cross sections have therefore been created that intersect the R179 and are orientated at right angles to the Knocknacran West quarry slope at their deepest points. Their location is shown in Figure 2. Details of each cross section showing the sequence of excavation and backfilling simulated are appended to this document.

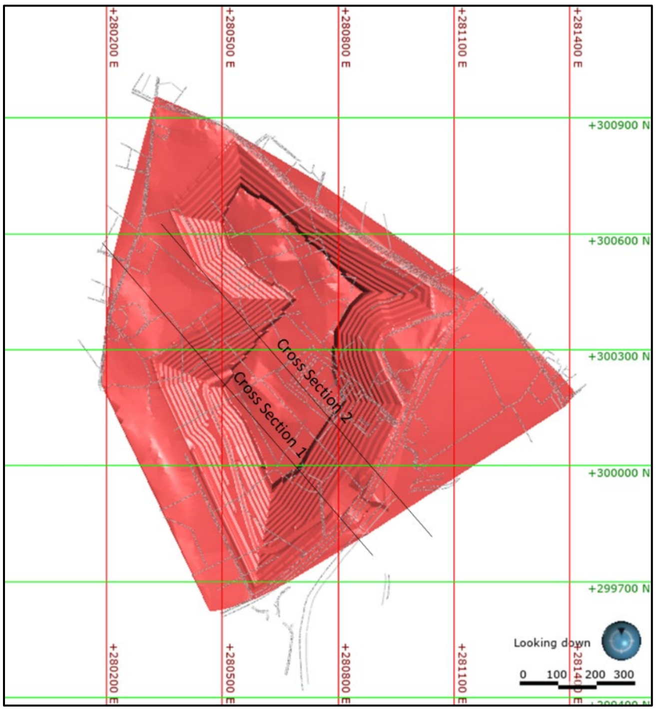

Figure 2: Location of Finite Element Modelling Cross Sections

The modelling was carried out in the following sequence:

1. Excavate the Drumgoosat mine.
2. Excavate the Knocknacran East Quarry.
3. Partially backfill the Knocknacran East Quarry. This is the current state of the landform southeast of the R179 road.
4. Excavate the Knocknacran West Quarry.
5. Backfill the base of the Knocknacran West Quarry and fill the Knocknacran East Quarry void to close to original ground level contours.

2.2.1 Cross Section 1 Results

Figure 3 is a contour plot of the model showing total vertical simulated displacement at the current state of the landform, i.e. Knocknacran East Quarry partly backfilled and Knocknacran West yet to be mined.

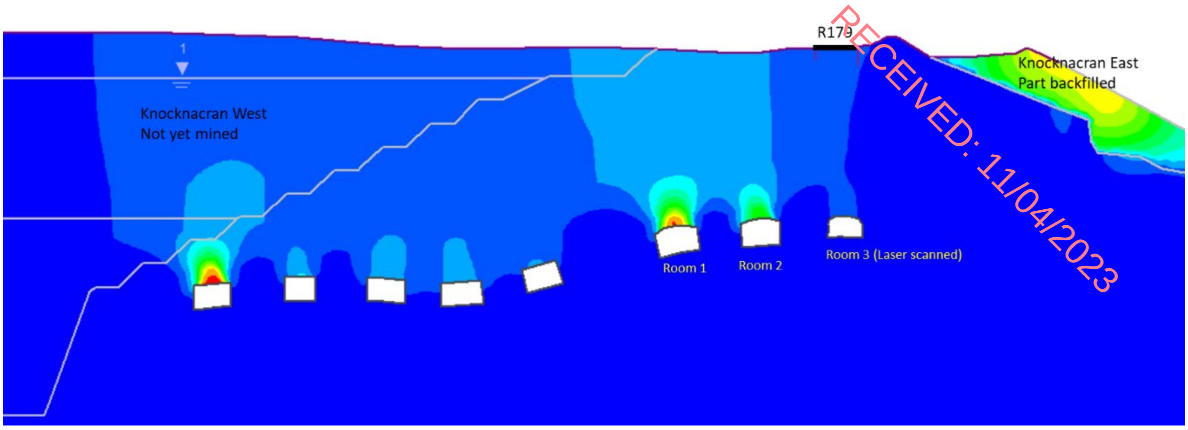

Figure 3: Cross Section 1 – Contours of Vertical Displacement, current State of the Landform

For the purpose of this assessment movement of the roof of the three rooms which underly the R179, Room 1, Room 2 and the laser scanned Room 3 as well as the vertical movement on the R179 is being considered. The roofs of Rooms 1 and 2 have also been slightly arched so that they take on the approximate roof shape of the laser scanned room.

Table 1 presents the results of the analyses. The initial roof deformation following mining of the rooms is quoted in millimetres. The additional simulated vertical deformation between each additional mining stage is also shown.

For all mining stages the model is stable. The creation of the Drumgoosat rooms generates an initial elastic roof beam deflection of 27 mm for Room 1 reducing to 4 mm for Room 3. The simulated surface deformation in the vicinity of the R179 is also 4 mm. East Quarry excavation and backfilling has negligible additional impact on roof deformation or on surface. Mining of the Knocknacran West Quarry has a very small impact on roof deformation and on surface. The room closest to the quarry slope indicates negative deformation change, or rebound, probably due to removal of overburden load. Final backfilling of Knocknacran West has no impact on roof deformation or on the surface near the R179.

Table 1: Cross Section 1 – Vertical Displacement Change by Mining Stage

	Simulated Vertical Ground Displacement (mm)	Additional Simulated Vertical Ground Displacement between Mining Stages (mm)			
	Underground Mined	East Quarry Mined	East Quarry Part Backfilled	West Quarry Mined	Final Backfill
Room 1	27	1	0	-1	0
Room 2	17	1	0	2	0
Room 3 (laser scanned)	4	0	0	2	0
Surface R179	4	0	1	1	0

2.2.2 Cross Section 2 Results

Figure 4 is a contour plot of the model showing total vertical simulated displacement at the current state of the landform, i.e. Knocknacran East Quarry partly backfilled and Knocknacran West yet to be mined.

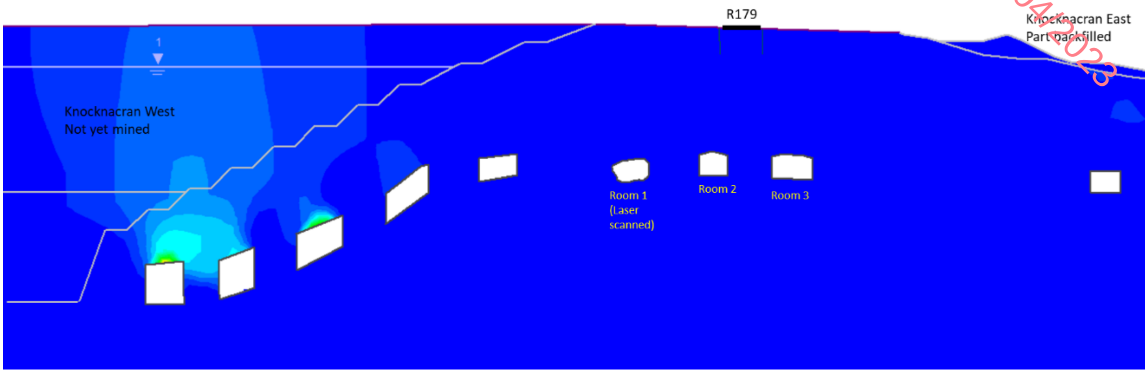

Figure 4: Cross Section 2 – Contours of Vertical Displacement, current State of the Landform

As with cross section 1 the roofs of Rooms 2 and 3 have been slightly arched to provide for a more realistic room profile in line with the shape of the laser scanned Room 1. It should be noted that this cross section intersects the 12 m high rooms and pillars that collapsed during the September 2018 event. These are located immediately behind the position of the Knocknacran West Quarry slope. Whilst no specific modification has been made to the strength of these pillars the rock mass above them is showing significant simulated deformation.

Table 2 presents the results of the analyses. The initial roof deformation following mining of the rooms is quoted in millimetres. The additional simulated vertical deformation between each additional mining stage is also shown.

Table 2: Cross Section 2 – Vertical Displacement Change by Mining Stage

	Simulated Vertical Ground Displacement (mm)	Additional Simulated Vertical Ground Displacement between Mining Stages (mm)			
	Underground Mined	East Quarry Mined	East Quarry Part Backfilled	West Quarry Mined	Final Backfill
Room 1 (laser scanned)	2.8	0.0	0.0	-0.4	0.0
Room 2	1.9	0.0	0.3	0.1	0.0
Room 3	2.8	-0.1	0.0	0.0	0.0
Surface R179	1.3	0.1	0.0	0.3	0.0

The initial deformation and that between mining stages is very small compared to that simulated for Cross Section 1, fractions of a millimetre in most cases. However the deformation between mining stages does follow the same trend as for Cross Section 1. The lower deformation values are due to the roof beam interpreted as being much thicker along this section line, 7 m to 12 m thick, than on section line 1, 3 m to 5 m thick.

2.2.3 Impact of Quarry Flooding on Underground Mine Stability

The final quarry landform will be developed into a quarry lake with the quarry containing water to a depth of between 36 m and 38 m. The Knocknacran West Pit Lake Model and Restoration Plan document prepared by Piteau Associates (Ref: 4238_Gyproc_LR05, dated 16 December 2021) indicates that there is likely to be limited groundwater generated from within the underground workings that remain in-situ after quarry mining has been completed. They indicate that the quarry backfill will be largely impermeable as it will be generated from mudstone interburden. They also state that it is expected that the underground workings will be gradually submerged as the water in the pit lake rises but no significant head of water will develop above the underground workings, suggesting that water will be constrained within the Lower Gypsum.

An additional analysis was undertaken to simulate a groundwater table at the top of the Lower Gypsum unit with underground rooms dry to determine what, if any, additional deformation of the underground workings may occur as the Lower Gypsum develops water pressure before the mine becomes flooded. The results are shown in Table 3 for Cross Section 1 and in Table 4 for Cross Section 2. The results are presented as a change in deformation from the Final Backfill case.

The results indicate that the increase in water pressure around the underground workings results in greater deflection of the roof beams. The model does not indicate that failure of the roof beam of the workings located below the R179 has occurred. Furthermore the increased roof beam deflection is not fully transmitted to surface because of the presence of unmined Upper Seam gypsum which provides some reinforcement to the rock mass.

Table 3: Cross Section 1 – Vertical Displacement Change Induced by Flooding

	Additional Simulated Vertical Ground Displacement between Mining Stages (mm)
	Quarry flooded
Room 1	195
Room 2	197
Room 3 (laser scanned)	389
Surface R179	11

Table 4: Cross Section 2 – Vertical Displacement Change Induced by Flooding

	Additional Simulated Vertical Ground Displacement between Mining Stages (mm)
	Quarry flooded
Room 1 (laser scanned)	1448
Room 2	708
Room 3	2917
Surface R179	127

The greatest deformation occurs above the rooms that are immediately behind the quarry slope and below the quarry lake.

2.2.4 Discussion – R179 Subsidence Potential and Mitigation

The results of the additional numerical modelling indicate that the mining voids that remain below and in the vicinity of the R179 are unlikely to be adversely affected by the activities of mining the Knocknacran West Quarry and backfilling of both East and West Quarries. The simulations do indicate however that there could be an elevated risk of roof deformation and possibly roof collapse as the underground rooms fill with water. This may occur as the water level in the quarry lake rises to its long term final level. No pillar failure in indicated in the simulations as the pillars below the road are only 6 m high and they are underlain by a significant thickness of competent lower gypsum unlike the mine conditions in the area of the September 2018 event where pillars were 12m high with a foundation of weak mudstone. Roof deformation is higher along Cross Section 2 than along Cross Section 1 probably because the underground workings are closer to surface along Cross Section 2.

The possible impact on surface should roof beam collapse occur could range from no impact, to increased levels of measured surface subsidence to the occurrence of crownholes above four way intersections. Almost all of the four way intersections that are present below the R179 have been accessed by boreholes and laser surveyed on several occasions. They are all currently in good condition showing no indication of significant rock fall or instability. The laser surveyed areas in relation to the R179 and the mine layout adjacent to the road is illustrated in Figure 5.

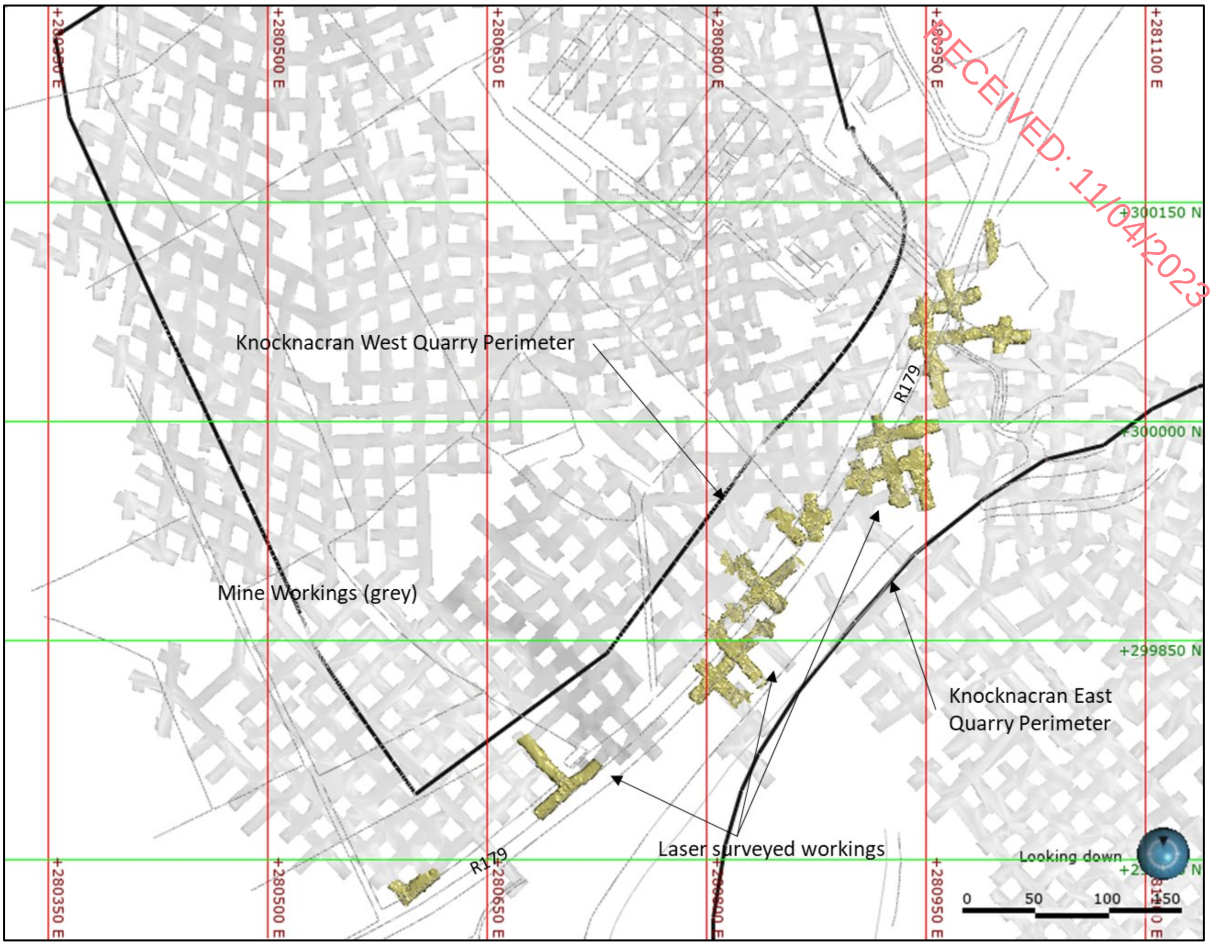

Figure 5: Laser Surveyed and Mine Workings below R179

A number of methods are available that can be used to manage any risks associated with changes to the stability of the underground workings as the quarry lake is formed. These are:

1. Continue the periodic surface levelling to provide early warning of any changes in movement magnitude along the road.
2. Continue monitoring the extensometers that have been installed within the roof beams at various location under the road.
3. Undertake sonar surveys as the underground workings become flooded to establish whether any instability develops.
4. If safe access into the underground workings can be established when rooms are exposed in the quarry slope consideration could be given to either:
    a. Constructing block walls in the workings and place rockfill material sourced from local quarries behind the walls to minimise infiltration of water into the underground workings, or

b. Packing the underground workings, specifically the four way intersections below the road, with waste rock to inhibit any potential roof movement as the quarry and underground workings are flooded.

2.3 Numerical Modelling – L4900

As shown in Figure 1 the quarry adjacent to the L4900 will be backfilled before the remaining quarry void is flooded. The underground workings should therefore be protected from water ingress by the presence of this backfill barrier. A finite element model has been crated along cross section 3. Its location is shown superimposed on the quarry design in Figure 6.

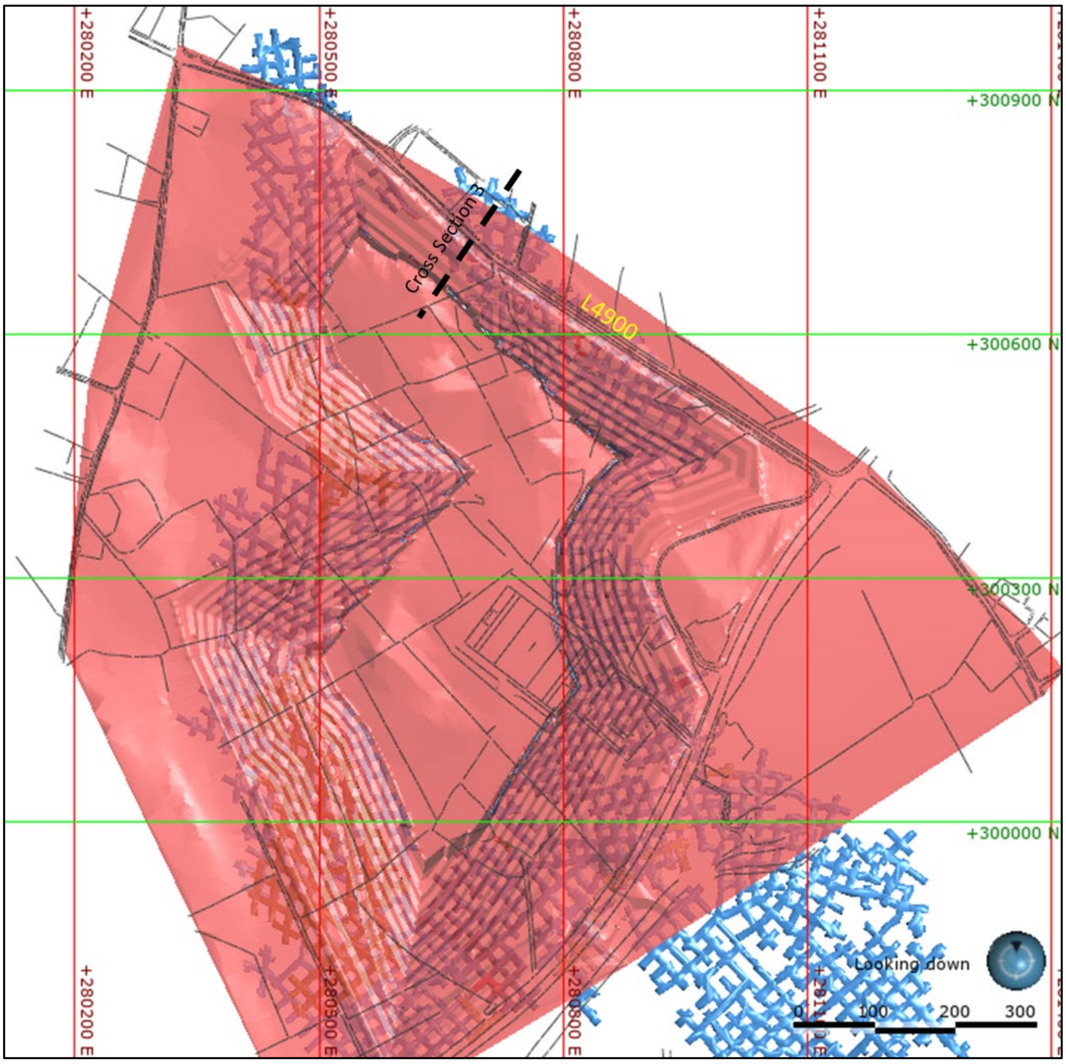

Figure 6: Location of Cross Section 3 across L4900

Figure 7 is a contour plot of vertical displacement after quarry excavation above the rooms below and adjacent to the L4900. Note that Room 3 contains the roof beam extensometer H11. In the three years that the instrument has been installed it has recorded roof movement of 0.03mm.

Table 5 shows the results of incremental roof beam deformation for each mining stage. It can be seen that quarry mining and backfilling induces little additional deformation to the roof beams or the ground surface around the L4900.

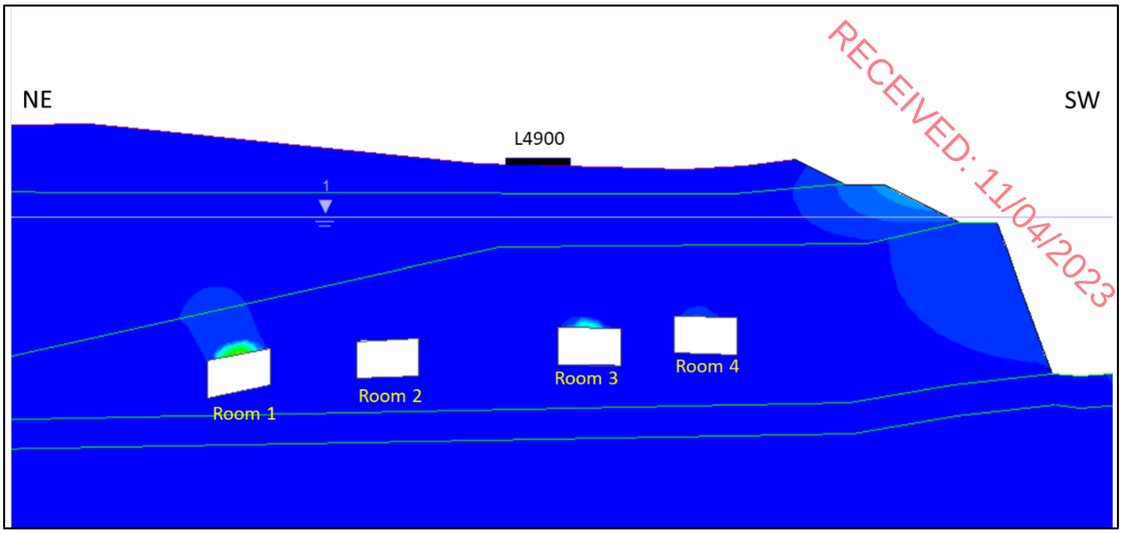

Figure 7: Cross Section 3 – Contours of Vertical Displacement after Quarry Excavation

After quarry flooding and assuming the lake water can flow through the backfill then additional forces can develop within the gypsum surrounding the underground workings to cause the roof beam to collapse. As with the flooding of the workings below the R179 no pillar failure is generated and the roof beam deformation does not propagate to surface.

In order to inhibit flooding of the quarry workings below the L4900 it is recommended that the mine rooms that are exposed in the quarry face are sealed prior to the commencement of quarry backfilling.

Table 5: Cross Section 3 – Vertical Displacement Change by Mining Stage

	Simulated Vertical Displacement (mm)	Additional Simulated Vertical Ground Displacement between Mining Stages (mm)		
	Underground Mined	Quarry Mined	Quarry Backfilled	Quarry Flooded
Room 1	18	13	2	2767
Room 2	2	0	0	2298
Room 3 (containing extensometer H11)	9	8	0	1943
Room 4	2	1	3	1704
L4900	0.8	0.2	0	21

2.4 Subsidence Risk South and West of the Quarry Perimeter

Having analysed mine instability below the R179 and L4900 roads which border the east and north perimeter of the quarry there are small areas to the south and west of the quarry perimeter that contain Upper Seam and Lower Seam workings. These areas have been highlighted in Figure 1. Cross sections 4 and 5 are illustrated and discussed below. In order to assess potential future subsidence risk we have considered the position of the workings in relation to various mine water levels and the potential response to the change in mine water levels that has been experienced above the mine workings over the last three years.

2.4.1 Cross Section 4

Details of Cross Section 4 which is located beyond the southwestern limit of the quarry are shown in Figure 8. This area contains the deepest areas of Upper and Lower Gypsum seam mining. The top of the Lower Seam is 110 m below surface and the top of the Upper Seam is 80 m below surface. These is beyond the maximum depth where potential crownholes could propagate to surface which has been determined to be about 60 m, 10 times the height of the mine openings. The lower seam workings have been permanently flooded since mining ceased at Drumgoosat as they lie below long term permanent mine water level of 970 m. The Upper Seam mine workings will have been largely dry until the mine flooding which caused the September 2018 event when they will have become completely flooded. Since then the Upper Workings have been subject to dewatering. The Upper Seam workings will be come completely submerged when the quarry lake is formed.

The Upper Seam workings extract a relatively thin seam of gypsum meaning that the roof beam in some areas is likely to be thin. Collapse of four way intersections has occurred since September 2018 above the Upper Seam workings further northeast along this section line in an area where the workings are closer to surface. There could be potential for future instability of rooms within the Upper Seam as the workings become permanently flooded. However because of the depth of the workings in this area the potential that any instability could manifest itself as a crownhole at surface is considered to be low.

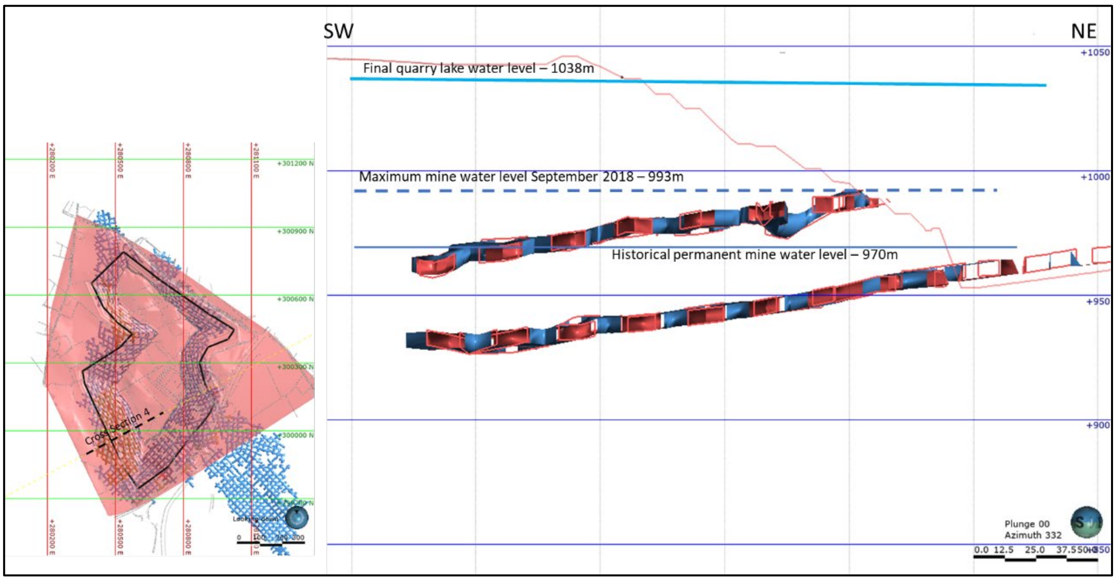

Figure 8: Cross Section 4

2.4.2 Cross Section 5

Details of Cross Section 5 which is located beyond the western limit of the quarry are shown in Figure 9.

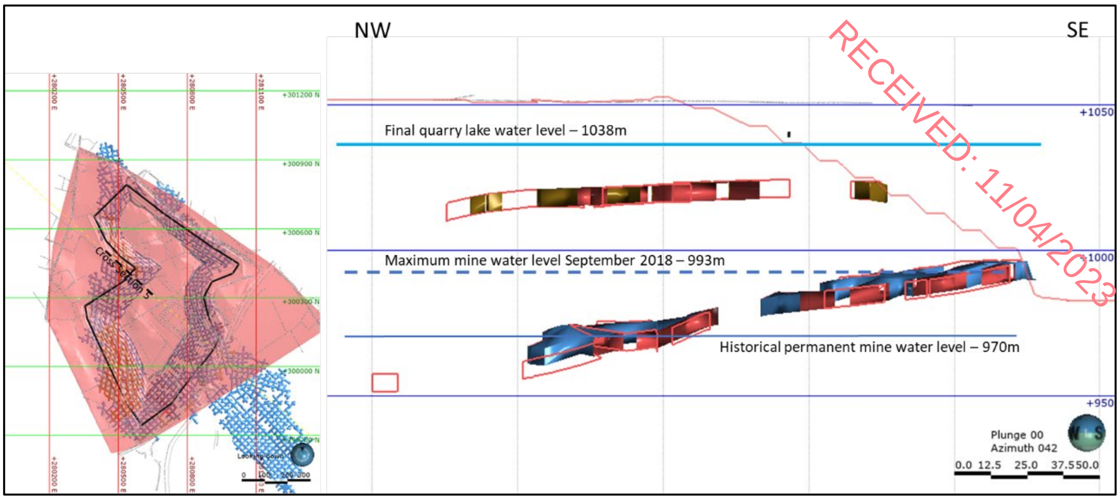

Figure 9: Cross Section 5

The top of the Lower Seam is 80 m below surface and the top of the Upper Seam is 30 m below surface. Both Lower and Upper Seam workings are located above the highest permanent mine water level of 970 m. The Lower Seam workings will have been largely dry since underground mining ceased until the mine flooding which caused the September 2018 event when they will have become completely flooded. The Upper Seam Workings lie above the level of temporary mine flooding and have been permanently dry but will become flooded when the quarry lake is formed. The Upper Seam workings extract a relatively thin seam of gypsum meaning that the roof beam in some areas is likely to be thin. There has been one recorded instance of the development of a crown hole in this area above the Upper Seam workings where the workings are about 35 m below surface. Because the Upper seam workings are shallow in this area there is potential for the development of new crownholes as the workings are flooded when the quarry lake is developed. It will be necessary to seal any Upper Seam rooms exposed in the quarry face to inhibit water ingress into the workings and reduce the risk of the formation of crownholes in this area.

3 DISCUSSION – MINING THROUGH FAILED PILLARS

There are a number of areas of subsidence through which quarry mining will take place. Subsidence has been the result of pillar failure of the Lower Gypsum in the area of the 2018 event and a combination of pillar and roof beam failure of the Upper Gypsum in the more recent western extension of the subsidence. It is anticipated that the ground within the subsidence areas will have moved and will therefore be of lower strength than the equivalent in-situ material. The overburden sitting above these failed pillar areas which is predominantly drift and mudstone will probably not pose a risk to excavation through as it is a relatively weak material and will have broken and re-compacted to accommodate movement of the rock mass. Gypsum pillars and roof beams that will have been affected by the subsidence will have broken up into large bedding and joint bounded slabs. The underground workings in these areas will have been filled by broken rock but it is likely that because of the shape and size of the slabs there will still be voids between them which may pose a hazard when trying to excavate through them. Position and extent of these unstable areas is reasonable well defined. In order to excavate safely through these area safe working methods will be developed by the operator and its contactors.

For and on behalf of SRK Consulting (UK) Limited

Neil Marshall, Corporate Consultant - Geotechnical, **Project Manager** SRK Consulting (UK) Limited	Max Brown, Principal Consultant - Geotechnical, **Project Director** SRK Consulting (UK) Limited

APPENDIX 7.16
Quarrying through Voids - SRK - July 2022

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External Memorandum

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Company:	Saint-Gobain Mining Ireland	Project Number:	UK31696
Copied to:	Benson Plunkett	Project Title:	Quarrying through Voids
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Subject:	**Quarrying through Voids**		

1 RFI POINTS

The following RFI points are being addressed in this technical note:

Mining Through Failed Pillars (RFI Ref: Point 22.iii.a.i)

1. The operational methodology and stability assessment should take into consideration that failure of underlying pillars has previously occurred within the area of the proposed open cast mine to establish whether further movement is possible.

Quarrying through Underground Voids (RFI Ref: Point 22.iii.a.ii)

1. Consideration of excavating above voids must be included in developing safe working practices during operations.

2 SRK RESPONSE

There are a number of areas of subsidence through which quarry mining will take place. Subsidence has been the result of pillar failure of the Lower Gypsum in the area of the 2018 event and a combination of pillar and roof beam failure of the Upper Gypsum in the more recent western extension of the subsidence. It is anticipated that the ground within the subsidence areas will have moved and will therefore be of lower strength than the equivalent in-situ material. The overburden sitting above these failed pillar areas which is predominantly drift and mudstone will probably not pose a risk to excavation through as it is a relatively weak material and will have broken and re-compacted to accommodate movement of the rock mass. Gypsum pillars and roof beams that will have been affected by the subsidence will have broken up into large bedding and joint bounded slabs. The underground workings in these areas will have been filled by broken rock but it is likely that because of the shape and size of the slabs there will still be voids between them which may pose a hazard when trying to excavate through them. Position and extent of these unstable areas is reasonable well defined. In order to excavate safely through these area safe working methods will be developed by the operator and its contactors.

As acknowledged in the full RFI query Saint-Gobain Mining Ireland (SGMI) has experience of working in areas above historical mine workings at the existing Knocknacran East Quarry. However in Knocknacran West there are more extensive underground workings as well as Upper Seam and Lower Seam workings along with Lower Seam Upper Horizon workings. The extent of the underground workings within the quarry limit is shown in Figure 1. It can be seen that in general the rooms and pillars of the upper and lower workings have been superimposed on one another with pillars lying above pillars and rooms above rooms. The potential for further subsidence during and after completion of quarry excavation is addressed in the stability analysis response included in the SRK document 31696 Report 2 Mine Stability.

Quarry or open pit mining through mineral deposits that have previously been mined by underground methods is not uncommon throughout the world. There is a significant history of open pit mining through underground voids, particularly in Australia, and guidelines for void

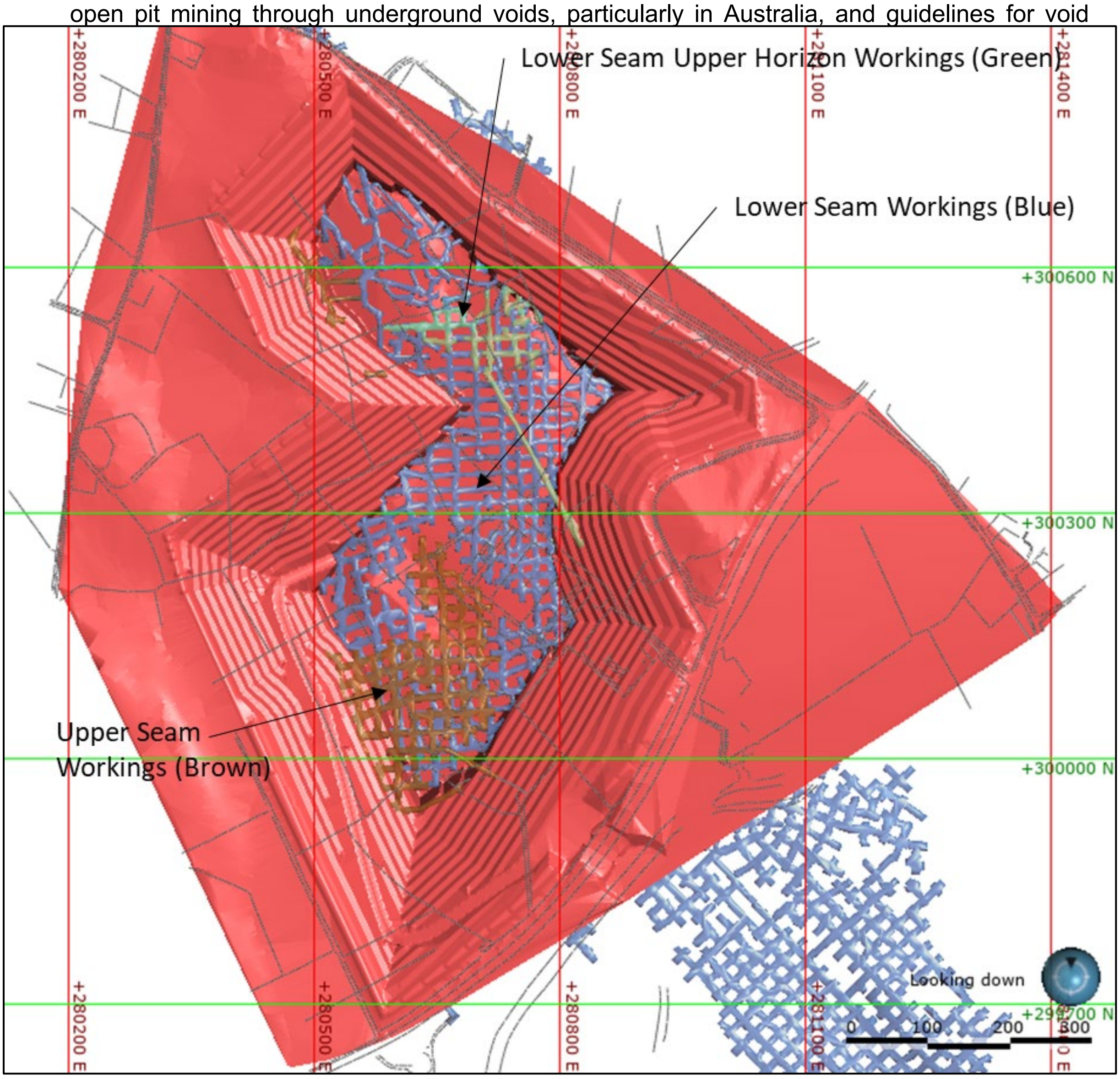

Figure 1: Extent of Underground Workings within Knocknacran West Quarry

Guidelines for mining through underground workings have been developed by the Mines Occupational Safety and Health Advisory Board (MOSHAB)[1] of the Government of Western Australia. The guidelines include hazard identification, risk analysis, control and mitigation.

Figure 2 is a graphic which summarises the void management process from identification through analysis to risk mitigation and has been reproduced from a paper by Barr et al (2018)[2] which details the currently available and tested risk management strategies for mining through voids.

Copies of the MOSHAB guidelines and the paper by Barr are appended to this technical note for information.

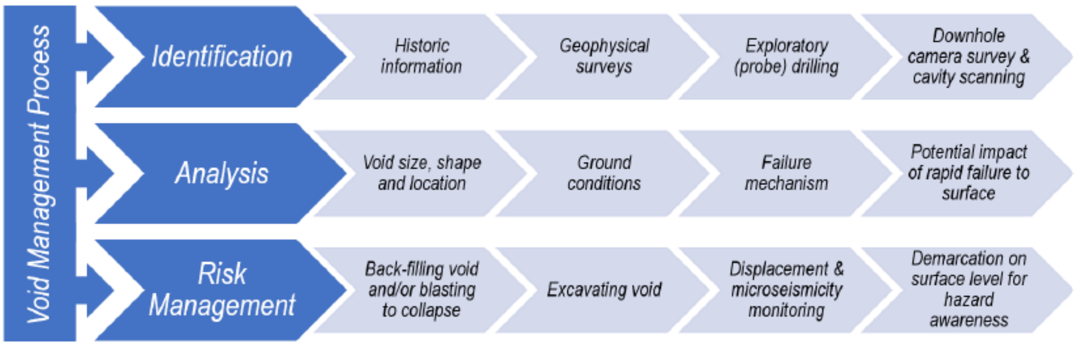

Figure 2: General Void Management Process

SGMI will need to develop an appropriate operational strategy, building on the success of the strategies used at Knocknacran East, to identify and mitigate the risks posed by the uncertainties identified above. SGMI's void management strategy currently in place for Knocknacran East is described later.

With regards to conditions at Knocknacran West the plan location of the underground workings is generally well defined. The main uncertainties that exist are:

1. The position of the roof of the workings;
2. The thickness of the gypsum above the workings. The upper surface of the gypsum is karstic in nature and in some instances the thickness of the gypsum can change significantly over short distances;
3. The presence, condition and extent of disturbed ground (both overburden and gypsum) within the areas that have been affected by recent subsidence events; and
4. The potential occurrence of chimney holes above the workings.

Some of the measures that could be implemented are listed below. They are not exhaustive but need to be developed into standard operating procedures and safe working methods by SGMI prior to commencement of waste stripping. The procedures will need modification and updating

[1] *MOSHAB (2000): Open Pit Mining through Underground Workings – Guideline.*
[2] *Barr. N, du Plessis. P, Nicoll. S, Welideniya. S, Ryan. C and McAllister. P (2018): Risk Management Strategies for Open Pit Mining Through Historic Underground Workings, ARMS10, 10th Asian Rock Mechanics Symposium, Singapore, November 2018.*

from time to time as experience in their implementation is gained.

1. The position of the underground workings and the extent of the disturbed ground should be pegged out on each bench. The methods of working within these areas needs to be defined and modified as the quarry floor approaches close to the underground workings.

2. Due to a good density contrast geophysical methods may be the best way to determine the thickness variability of the gypsum roof beams.

3. Geophysics could be combined with localised probe drilling into the disturbed ground to assess the competence of these areas particularly as the gypsum seam is approached.

4. Access control will need to be enforced. There is still a lot of solid ground between the gypsum rooms and temporary tramming routes within the quarry. These should align with and be located above pillars. According to the mine layout in Figure 1 these should be orientated north-south or east-west.

For and on behalf of SRK Consulting (UK) Limited

Neil Marshall, Corporate Consultant - Geotechnical, **Project Manager** SRK Consulting (UK) Limited	Max Brown, Principal Consultant - Geotechnical, **Project Director** SRK Consulting (UK) Limited

OPEN PIT MINING THROUGH UNDERGROUND WORKINGS

GUIDELINE

Department of Industry and Resources

MOSHAB
Mines Occupational Safety and Health Advisory Board

FOREWORD

This Mines Occupational Safety and Health Advisory Board (MOSHAB) Guideline offers advice on the issues that should be addressed when open pit mines are excavated through abandoned underground workings, or in close proximity to current underground workings.

Comments on, and suggestions for, improvements to the Guideline are encouraged. This Guideline will be revised as appropriate.

Comments should be sent to:

State Mining Engineer
Safety Health and Environment Division
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ISBN 0 7309 7814 1

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1.0 INTRODUCTION

A number of open pit mines in Western Australia (WA) are mining orebodies that have previously been mined by underground methods. There are hazards with high risk potential which develop where open pit mines approach and then progressively mine through underground workings.

These hazards in the open pit include:

- sudden and unexpected collapse of the open pit floor and/or open pit walls;
- the loss of people and/or equipment into unfilled or partially filled underground workings;
- loss of explosives from charged blast holes that have broken through into the underground workings;
- overcharging of blastholes where explosives have filled cavities connected to the blasthole;
- risk of ejecta (flyrock, etc) from cavities close to the pit floor and adjacent blast holes, particularly when explosives have entered the cavity from the blasthole during charging and the loss is not detected.

In general, the above hazards are significantly increased when the underground workings have not been backfilled with waste rock, sand fill, etc. As these hazards are not generally evident during normal open pit mining operations it is necessary to take additional measures to better define their nature and extent. Some of these measures are discussed in **section 5**. Once the relevant hazards have been adequately defined the mine operator should put in place a range of controls to mine safely through the underground workings. A number of these controls are discussed in **section 6**.

In addition to the above hazards, when open pit mines approach currently operating underground mines, the potential hazards may include:

- flooding of the underground workings;
- instability of the open pit walls and surrounding surface areas;
- adverse effects on the underground mine ventilation system.

This guideline primarily addresses hazards associated with open pit mining through abandoned underground mine workings. Some of the additional hazards associated with open pit mining through currently operating underground mine workings are summarized in **Appendix A**.

2.0 LEGISLATIVE REQUIREMENTS (WA)

The **Mines Safety and Inspection Regulations 1995** includes a provision (**Regulation 13.8**) relating to surface mining operations where mining is being conducted through or in proximity to underground mine workings.

Geotechnical considerations 13.8. (3) Each responsible person at a mine must ensure that appropriate precautions are taken and written safe working procedures are followed if open pits are excavated through abandoned underground workings, or in close proximity to current underground workings. Penalty: See regulation 17.1.

3.0 HAZARD IDENTIFICATION

Knowledge of the previous mining history of the area to be mined will be of primary importance in determining the likelihood of abandoned underground workings being present below the open pit. A thorough review of previous mine plans is essential prior to any open pit development. The validity of old underground mine plans should be checked diligently, particularly if they are abstracted or copied from originals.

A review of underground workings should be part of the design and planning of the open pit to ensure, as far as reasonably practicable, that:

- all known underground workings are marked clearly on all working mine plans and the plans rechecked;
- there is a recognition that the rock mass surrounding the underground workings may be highly variable in strength and potentially unstable;
- a three dimensional model of underground workings is developed and used in all mine design, planning and scheduling.

It is essential that all plans are updated following all phases of exploration to ensure that the revised outlines of the actual extent and shape of underground workings are recorded.

A further aspect requires a cautious approach. Tributors may have carried out further work in old gold mines, which may not have been recorded on the closure plans lodged by the mine before tribute mining took place.

Where it is likely that underground workings could be of large dimensions and extended in length and depth, or where no previous plans are available, it may be necessary to carry out specific investigations to confirm the location of the workings. Some of the methods that may be used for this purpose are briefly discussed in **section 5**.

4.0 MAKING THE HAZARD VISIBLE

All areas of a working bench or flitch that are likely to be underlain by underground workings must be clearly marked and access to this area must be controlled by a specific set of procedures. These procedures should specify the personnel responsible for monitoring and marking out the hazardous areas. Every bench or flitch should be clearly marked with the projected excavation outline as mining progresses downward through the underground workings.

The marking of areas potentially underlain by underground workings must involve a clear method of identifying the potential hazard. If coloured flagging tape is used, a specific colour – preferably visible in both day and night conditions - should be used solely for this purpose. Steps should be taken to ensure that hazardous areas are adequately marked at all times. Damaged or displaced flagging tape should be immediately replaced. All employees must be informed as to the purpose of the marking or flagging tape.

Care should be exercised in the location of the marked areas. Allowance should be made for the uncertainty in the precise position of the underground workings and any potentially unstable ground surrounding the underground workings. In short, an extra margin of safety should be allowed in the separation of permissible work areas from suspect zones.

5.0 RISK ANALYSIS

5.1 Determining the extent of underground workings

A number of detection methods are available which may be used to confirm the lateral extent and shape of underground workings prior to mining, including:

- probe drilling[1];
- geophysical techniques – including seismic, resistivity, conductivity, and gravity methods;
- ground probing radar;
- laser based electronic distance measurement (EDM) surveying methods;
- closed-circuit TV cameras lowered through probe holes;
- where practicable, actual physical inspection and survey.

Probe drilling is the most widely applied technique to delineate the detailed geometry of underground workings in WA. Remote sensing techniques (ie geophysical techniques and ground probing radar) have been used with varying degrees of success, with individual techniques having limitations depending on the nature of the local geological conditions. Ground probing radar has been used with limited success to detect voids below open pits in WA. Remote sensing techniques are not universally applicable and even when successful, require some level of confirmatory drilling.

5.2 Probe drilling procedures

Site specific written probe drilling procedures are essential. These procedures should specify the following:

- the type of drilling rig to be used and the provision and use of any special safety precautions, eg safety lines, remote controls, communications procedures, refuelling procedures, maintenance;
- the training requirements for persons operating drill rigs used for probe drilling purposes;

[1] Probe drilling should be carried out only on ground determined to be secure. Information obtained from exploration drilling, or grade control and blasthole drilling, may be of assistance when determining the shape and extent of underground workings.

- the procedures to be followed when working or drilling within a marked area that may be underlain by underground workings; the person responsible for approving entry to a marked area should be identified;
- the procedures to be followed when marking out the proposed probe drilling pattern;
- the sequence in which drilling should proceed – drilling operations should proceed from known safe ground towards the anticipated underground workings, see **Figure 1**;
- any equipment, eg tapes, inclinometers, etc required for the drilling activities;
- the capability to drill steeply dipping holes to determine a floor pillar thickness (measured vertically) is generally available; however, it may be necessary to drill shallow dipping holes to determine a rib pillar width (measured horizontally) in the walls of an open pit;
- the requirement for and method of completing any logging sheet to record the result of the drilling operations; eg driller, hole number, void depth, void size, descriptions of the ground conditions encountered, types of material encountered, drilling difficulties;
- the procedures to be followed when:
  - (a) difficulties occur in completing the hole to its planned depth;
  - (b) workings (either open or filled) are intersected during the drilling of a hole;
  - (c) workings are intersected at or less than the minimum stable floor pillar thickness or width (as determined in section 6);
  - (d) other potential hazards are intersected, eg gases, water, etc;
- the minimum and maximum probe hole drilling depth, angle and spacing; these should ensure that workings of hazardous magnitude do not remain undetected, eg ore passes, etc;
- the procedures to be followed when other personnel are working adjacent to any marked area when drilling is in progress, eg surveyors, samplers, maintenance crew, etc;
- special requirements or restrictions on carrying out maintenance work on the drilling equipment within a marked area; (ordinarily no maintenance work should be carried out within a marked area).

6.0 RISK CONTROL

6.1 Introduction

The control measures that are available to eliminate or minimise the risk of unexpected pit floor and/or wall collapse include:

- leaving a pillar of adequate dimensions between the current working bench or flitch and the underground workings;
- placing fill materials into the underground workings;
- restricting work to areas clear of the suspect location, with an adequate margin of safety;
- blasting waste rock in the pit floor into voids, followed by further back filling to stabilize the area.

6.2 Determination of adequate pillar dimensions

In all open pit mines where there is a risk of intersecting underground mine workings, appropriate studies must be carried out to determine the minimum stable pit floor pillar and/or rib pillar dimensions. The minimum pit floor pillar thickness is defined as the minimum rock cover, measured vertically, above the highest point of the underground workings which provides an acceptable factor of safety against floor pillar failure during all mining activities. The minimum rib pillar width is defined as the minimum rock and/ or soil barrier, measured horizontally, between open pit walls and adjacent underground workings which provides an acceptable factor of safety against wall failure.

The overall dimensions of the pillar, ie length, width (measured horizontally) and thickness (measured vertically) should be taken into account in any analysis of stable pillar thickness. Consideration should also be given to the appropriate factor of safety when selecting pillar dimensions. The factor of safety selected should be commensurate with the level of the risk posed by the extent of the underground workings and the nature of the rock mass.

The determination of the stable pit floor pillar thickness and/ or rib pillar width should be the result of a geotechnical engineering assessment in which specific attention is paid to:

- orebody geometry, particularly orebody dip and orebody width;

- the likely modes of failure of the stope crown pillar or floor pillar, whether controlled by, or independent of, geological structure;
- the likely modes of failure for the immediate hangingwall and footwall rocks whether controlled by, or independent of, geological structure;
- the potential accumulation of water in the open pit due to localised ponding via surface runoff from the surrounding catchment area and/or incident rainfall within the open pit perimeter;
- the loads imposed by equipment or stockpiles on the floor pillar;
- rock mass strength and/or general competence of pillar and wall rocks;
- "worst-case" geotechnical conditions with particular emphasis on structural geological features (planes of weakness), groundwater, variations in rock strength and their likely impact on the stability of the pit floor or rib pillars;
- the influence of open pit blasting on the integrity of the pillars;
- the relationship of pillar thickness to the width and strike length of stoped areas.

The adopted stable pillar thickness or stable pillar width will vary both within an individual site and from site to site, to reflect the extent of the hazard, the variation in controls on pillar stability, the range of geotechnical conditions, together with the extent and dimensions of stoping. The planned cut-back of an open pit wall may produce a situation where the stable rib pillar width that previously existed is reduced to unacceptable dimensions.

6.3 Open pit mining issues

Conventional open pit mining methods may need to be modified when mining above or through abandoned underground workings, when:

- mining through floor pillars smaller than the minimum stable thickness (the use of remote control of drilling and explosive charging operations may be required);
- backfilling narrow stopes (experience has shown that narrower stopes are potentially more difficult to backfill due to material "hanging up" or bridging across the stope walls);
- backfilling large stopes (backfill should not be relied upon as the sole means of providing safe working conditions);

- considering the use of mass blasting methods[2];
- mining through pillars and stopes has the potential to destabilise open pit walls[3]. This may have adverse consequences for mine infrastructure within or adjacent to the pit.
- maintaining the minimum safe working width on either side of stoped areas, particularly in the lower sections of narrow pits where mining widths may be restricted;
- controlling access to, and movement on, each bench or flitch, particularly where previous stoping may be continuous along strike.

6.4 Safe working procedures

A flow chart illustrating the key activities that need to be considered when an open pit is mining through abandoned underground workings is shown in **Figure 2**. This chart should be used as a basis for framing site-specific procedures, and the review and updating of all mining plans to ensure that an accurate model of the geometry of underground workings is maintained at all times.

Before commencing any open pit mining near or through abandoned underground workings an appropriate set of safe working procedures should be established that address a range of issues, including:

- probe drilling procedures;
- marking out the extent of the underground workings;
- drilling and blasting;
- plant and equipment movement;
- placement of fill materials in unfilled workings;
- rock stability monitoring;
- daylight and night operations;
- plant and equipment specifications;

[2] Such use should be reviewed on a case by case basis having regard to the stability of the surrounding rock mass, adjacent open pit walls, the potential hazards associated with charging of blast holes in the vicinity of underground workings and the requirement to monitor explosive quantities loaded into each blast hole.

[3] Experience suggests that stope hangingwall instability may be more extensive with the potential to undercut open pit walls, particularly in large unfilled stopes.

- personnel movement;
- regular communication of information and discussion of issues of concern with all those involved.

These safe working procedures should be progressively reviewed as the open pit depth increases.

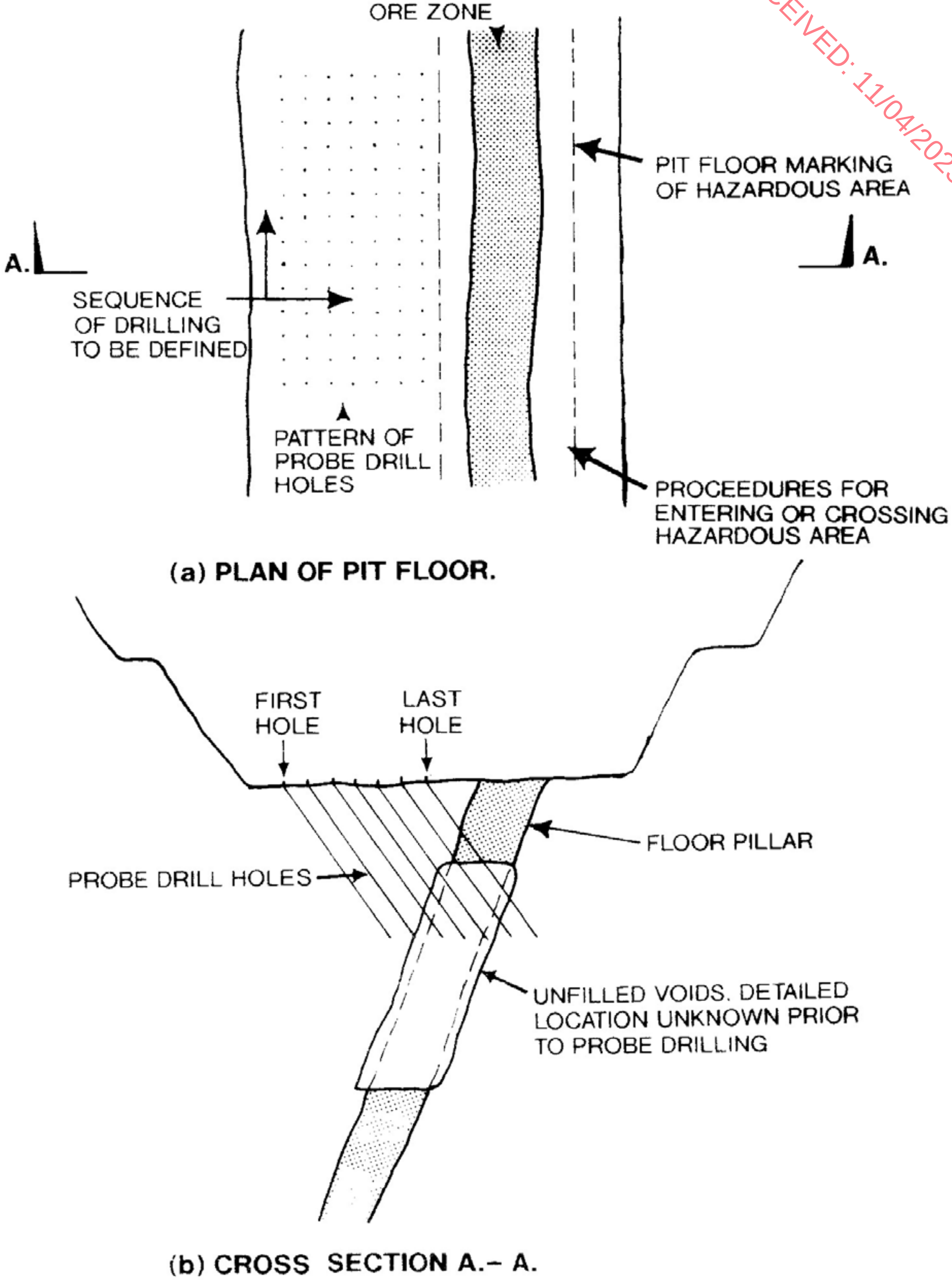

Figure 1. Probe hole drilling to locate underground workings

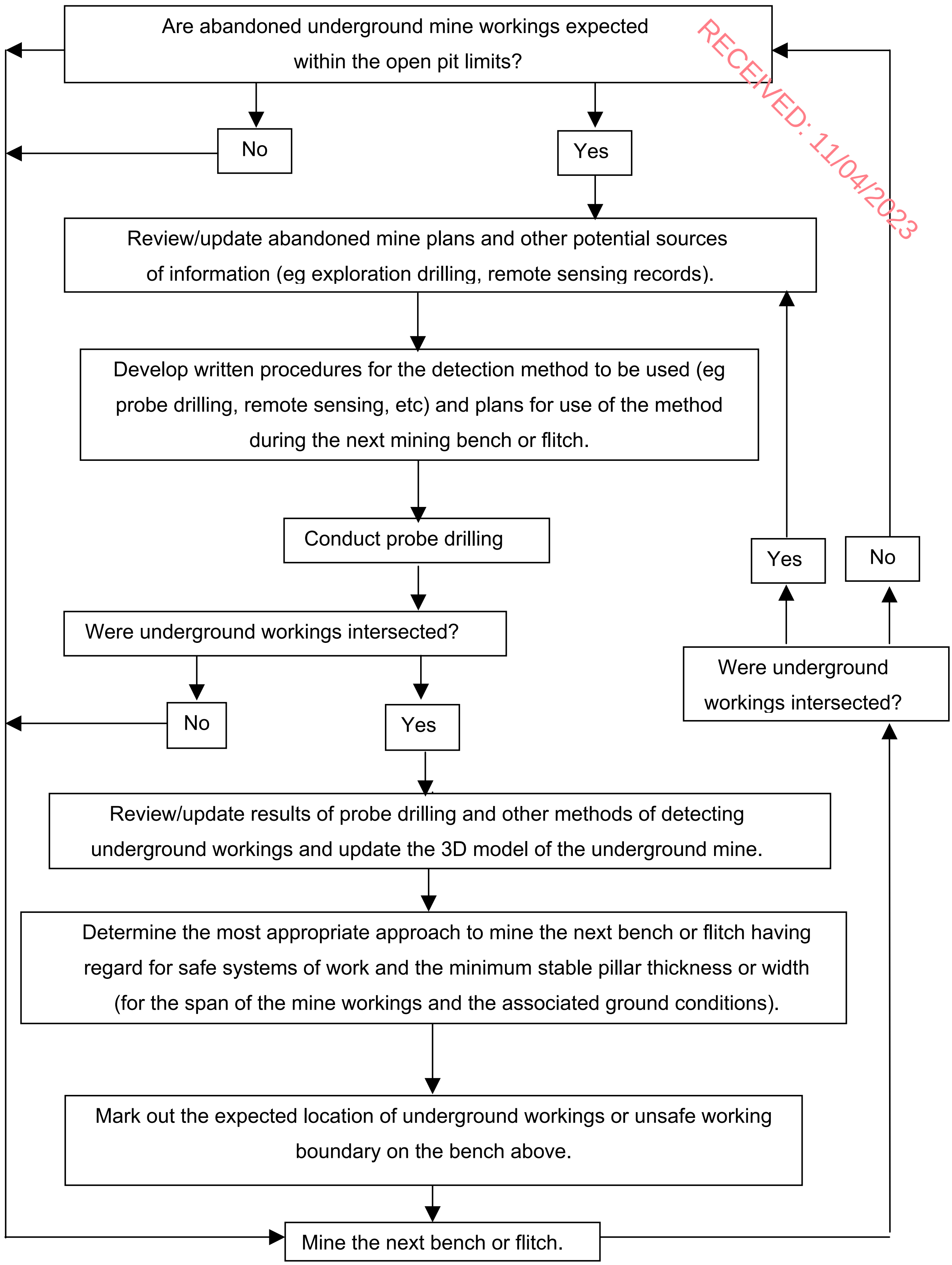

Figure 2. Flow chart for each bench or flitch, as appropriate, showing the key activities in open pit mining through abandoned underground workings

APPENDIX A - HAZARDS ASSOCIATED WITH OPEN PIT MINING THROUGH CURRENT UNDERGROUND WORKINGS

Currently operating underground mines may face a number of potential hazards when open pit mining is conducted through underground workings associated with the underground mine.

The location and extent of the current underground mine workings should be known with a much greater level of confidence than is the case with abandoned underground mine workings. The use of current underground mine surveying methods and equipment should largely eliminate any uncertainty as to the location of current underground mine workings. The presence of large unfilled stope voids may result in large scale collapse of the surrounding rock mass into the stope void. When this occurs the extent and location of the boundaries of the underground workings (eg walls, backs, etc) will obviously change.

The hazards associated with open pit mining through current underground mine workings include:

- flooding of the underground mine workings from large water and/or mud inrushes via the open pit and surrounding catchment areas;
- flooding of filled stopes, by accumulated drainage or by inrush, containing uncemented or inadequately cemented fill materials, that may become saturated, causing bulkheads to fail under hydrostatic pressure, resulting in a fill or mud rush in the mine;
- instability of the open pit walls and the surrounding surface areas, including any mine infrastructure (ventilation fans, shafts, headframes, winders, buildings, mobile equipment, any underground mine excavations in close proximity to the open pit, rising mains, electric cables in bore holes, fill passes, bore holes used for delivery of fill materials, etc);
- adverse effects on the underground mine ventilation system (short circuiting, ingress of open pit blasting fumes and dust, rock falls in large open stope voids creating dust which is drawn into the main intake airways, etc);

- potential for collapse of large unfilled stope voids that may cause a significant change in the underground mine geometry;
- deficiencies in co-ordination, communication and control of mining activities between the open pit and underground mines.

Each of these hazards must be adequately investigated and controlled by appropriate means according to the identified risk.

APPENDIX B - LEGISLATIVE FRAMEWORK

The *Mines Safety and Inspection Act 1994* sets objectives to promote and improve occupational safety and health standards. The Act sets out broad duties and is supported by more detailed requirements in the *Mines Safety and Inspection Regulations 1995*. A range of guidance material, including Guidelines, further supports the legislation. The legislative framework is set out in **Figure 3**.

Guidance material includes explanatory documents that provide more detailed information on the requirements of the legislation and include codes of practice and guidelines.

Guidelines contain practical information on how to comply with legislative requirements. They describe safe work practices that can be used to reduce the risk or work-related injury and disease and may also contain explanatory information.

Figure 3: Legislative framework

MINES SAFETY AND INSPECTION ACT	←	Major provisions: • The General Duties • Management of Mines • Enforcement of Act and Regulations • Resolution of issues • Safety and Health Representatives • Safety and Health Committees
Supported By		
MINES SAFETY AND INSPECTION REGULATIONS	←	The Mines Safety and Inspection Regulations set minimum requirements for specific hazards and work practices, including reference to National Standards, developed by the National Occupational Safety and Health Commission, and Australian Standards developed by Standards Australia.
And		
GUIDANCE MATERIAL	←	• Codes of Practice approved for Western Australian mines in accordance with Section 93 of the Act. • Guidelines produced by the Mines Occupational Safety and Health Advisory Board. • National Codes of Practice and National Standards developed by the National Occupational Health and Safety Commission. • Australian Standards developed by Standards Australia.

The information included in a Guideline may not represent the only acceptable means of achieving the standard referred to. There may be other ways of setting up a safe system of work and, providing the risk of injury or disease is reduced as far as practicable, the alternatives should be acceptable.

Risk Management Strategies for Open Pit Mining Through Historic Underground Workings

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Abstract

Underground mining in gold, base metals and coal deposits has occurred for centuries, and even millennia in some parts of the world. The mechanization of mining and economic appeal of open pit mining in the last 30 years has facilitated the conversion of many historic underground mines into large open pits. Historic underground workings or voids below the working floor of an open pit and voids behind or intersected by pit slopes have the potential to collapse and induce uncontrolled ground movement which may cause harm to personnel or equipment as well as impact upon the economic profitability of the operation. Identification and demarcation of controlled exclusion zones from potential voids beneath the working floor is common practice to prevent personnel and equipment entering at-risk areas. However, the way these potential void risk areas are determined and managed vary significantly from operation to operation. Risk management strategies for working in close proximity to historic underground workings are dependent on several factors including ground conditions, historic underground mining method and size of voids (open or filled stopes, room and pillar, caving, horizontal and vertical developments) and void status (open or filled, surveyed or estimated size). Blasting to collapse void areas and controlled excavation practice are paramount for safe mining. This paper discusses various approaches of managing risk in open pits with historic underground workings below from gold and base metals deposits in Australia and Africa.

Keywords: Voids, Open Pit Mining, Underground Mining, Risk Management.

1. Introduction

Underground mining for the last hundred to thousand plus years around the world has left behind voids of different size and geometry. The voids are also located in ground conditions of varying strength, geological structure, weathering and alteration, which may change over time. Near-surface void collapses have caused problems in open pit mining operations for many years, although this problem may also present a risk to the public in urban and rural settings.

Reliably identifying the presence of near-surface voids is paramount when commencing open pit mining in a region with history of underground mining. Although the underground mining method may have been various forms of stoping, room and pillar, or caving, the first aspect of almost all void management processes is identification as shown in Fig. 1.

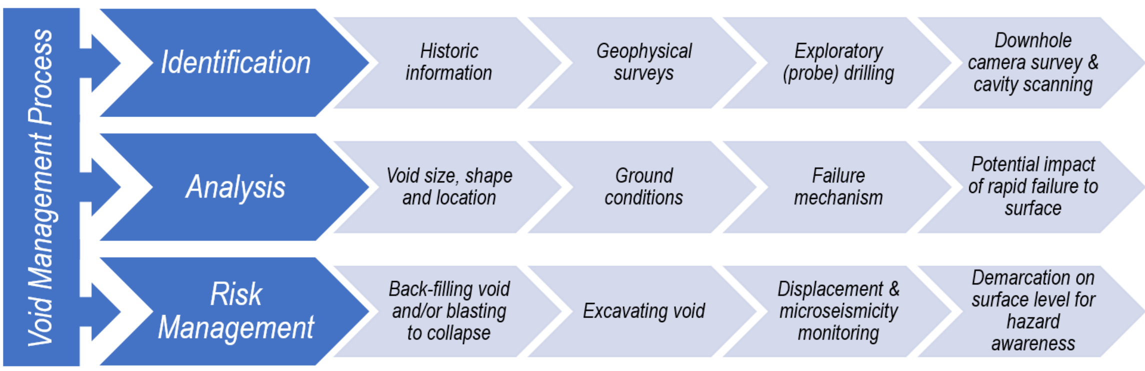

Fig. 1. General Void Management Process: Identification, Analysis and Risk Management

Establishing the competency and adequacy of the rock cover between the void and the risk to personnel and equipment at the surface is key to preventing cave-ins and other uncontrolled or unwanted ground movement (Carter, 2014). Empirical methods, rules-of-thumb and numerical simulations can be used to assist in predicting the potential impact of rapid failure or caving to the surface. Common risk management strategies for voids (after identification and assessment) typically involves demarcation of the void risk on the surface level in combination with any or several other measures including back-filling, blasting to collapse, displacement monitoring or additional identification and analysis measures.

Void management is an iterative process whereby risks are re-evaluated as mining progresses in an open pit or as circumstances change. For example, the void management process would typically be reviewed following high intensity or prolonged rainfall events, seismicity or prolonged periods of inactivity in the mine. These processes have significantly evolved and improved over time, largely through several different mishaps where although no personnel were injured and the equipment damage was minor, the potential consequences were quite severe. Examples shown in Fig. 2: 'A' resulted from back-fill mobilization within a stope network after heavy rainfall; 'B', 'C' and 'D' had inadequate rock cover between the void and the pit floor since the voids were not sufficiently identified; and 'E' was a result of void propagation to the surface over several years from unravelling in weak material, periodically exacerbated by rainfall.

Fig. 2. Void management mishaps and learnings from various undisclosed mines.

Mining legislation in most countries requires a structured plan for mining through underground workings that present a risk both from a geotechnical and planning perspective (MOSHAB, 1999). This paper discusses various approaches of managing void risk in open pit mines with historic underground workings and aims to provide a general framework of best practice options for engineers to implement in this field to reduce the likelihood of further mishaps such as those shown in Fig. 2.

2. Void Identification and Confirmation

Identifying voids prior to commencing open pit mining is essential for proactive risk management and reducing the likelihood of mining schedule delays and mishaps from unexpected voids.

Since open pit mining induces significant stress changes on the ground, it is likely that in some areas, concentrated stresses, or equally, the loss of confining stresses, can induce localized ground collapses. External influencing factors (e.g. high rainfall, seismicity, etc.) can also contribute to changing conditions. As a result, it is essential that any void identification process is iterative, as open pit mining progresses downward and as external influencing factors are realized. That is, voids should be confirmed on a bench by bench basis (or similar), and after external influencing factors.

This aspect of void identification and confirmation is both costly and time consuming. In open pit mines with significant void risks, the mine plan or schedule is frequently adapted or adjusted for changing conditions with respect to voids.

2.1 Historic information

Historic information pertaining to underground mining activities in an area with a planned open pit usually forms the basis for further investigations. The amount, detail and quality of data may vary significantly from site to site, and even within a single site. For example, in a mine operating for over 50 years, records for the most recent 20 years of mining most likely contain detailed surveys or laser scans of the underground workings and digital records of mining activity including ground conditions, support installation, backfill information, etc. However, in older areas, this information is likely to be paper-based, or non-existent.

Where available, historic plans or surveys of underground workings provide an understanding into the potential void risks that may be presented in a future open pit. In large underground mines, detailed records pertaining room and pillar, stoping and caving operations are sometimes available, and almost always, very useful. These records usually provide insight into the ground conditions encountered and the type of backfilling, if any, was completed.

In many cases where underground mining occurred before the 'digital era' and records are still available in good condition, the transformation of drafted paper-copy mine plans into three-dimensional models is possible.

In underground mines where historic workings of interest remain safely accessible, a campaign of geotechnical engineer assessment and surveying (usually with a laser scanning) is possible. However, this is typically very time consuming.

2.2 Geophysical surveys

Various forms of geophysical surveys from electrical resistivity to ground penetration radar can be implemented before mining commences, or at several stages during mining to identify voids or even backfilled voids. The effectiveness of geophysical surveys is highly dependent on the ground conditions and the resolution of the surveys.

In regions where historic, undocumented and artisanal mining has occurred, geophysical surveys are particularly useful. However, these are best implemented in conjunction with drilling.

2.3 Exploratory and routine probe and blast pattern drilling

Probe drilling and precondition blasting are an active practice in the void management process at most operations with historic underground workings. The extent of probe drilling is dependent on the level of confidence in the spatial data (i.e. location, size and geometry of underground workings), age of the underground workings, post blasting to verify collapse of voids or whether any form of backfilling has taken place during or after the mining of the underground workings.

The absence of spatial data in void areas take on an exploratory approach to identify areas of potential voids by probe drilling in a grid formation below the mining floor (generally 2 – 3 mine benches below the active mine floor). Any void intersection during this probe drill program will see the probe drill grid constrict within the area to improve identification of the void geometry and size. Alternatively, downhole cavity scanning can be utilized.

Underground workings with existing spatial data are also probe drilled to verify the accuracy in terms of spatial compliance of underground workings as well as identification of backfill material and collapse of voids primarily caused from ground support or rock mass deterioration over time as described in Fig. 3. Probe drilling is also used for verification of blasting results following blasts that are designed to collapse underground workings that may propagate to the surface in the event of rapid failure.

Probe drilling is typically carried out using percussion drills 'borrowed' from the drill and blast section of a mine production team. Probe drilling is often heavily reliant on experienced drillers for identifying changes in ground conditions by observing fluctuations in air pressure, changes in drill cutting returns, low resistance against the drill string or the loss of water returns. Results from drilling are compared against available spatial and historical information:

- Confirm minimum rock cover above voids and the integrity of pillars between voids.
- Identify any voids where collapse has or may be occurring or identify any backfill used.

The results are also used to position the drill for further drilling in a known safe location. In some cases where the ground condition below the surface is deemed high risk during verification (e.g.

identification of voids with insufficient minimum rock cover), the drilling program will cease, and continue from another known safe location, possibly with extreme drill angles, which may not be suitable for some equipment.

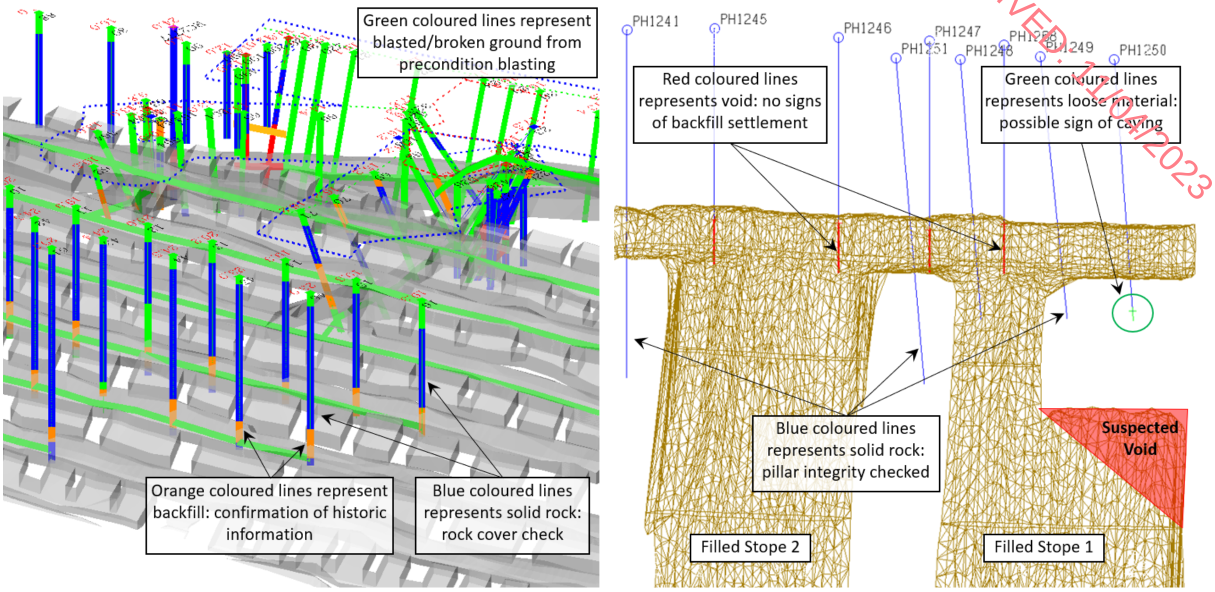

Fig. 3. Probe drilling from the mine floor. Left: Confirmation of rock cover, backfill and effects of precondition blasting in development drives and small stopes (volume ≈ 50m^3 each); Right: Confirmation of backfill and pillar integrity identifying possible signs of caving above a suspected void within large filled stopes (volume > 5,000 m^3 each).

2.4 Downhole camera surveys and cavity scanning

For more detail and accuracy than driller notes, probe drilling can be combined with downhole camera surveys and cavity scanning using borehole-deployable laser scanners such as C-ALS.

Downhole cameras are frequently used both to investigate unknown voids intersected by probe drilling as well as the integrity of known voids as shown in Fig. 4.

Cavity scanning can be used to confirm the size and shape of voids, such as the suspected void from Fig. 3. As illustrated in Fig. 4, cavity scans provide a 3D model of a void; however, scans are often limited by subsurface line-of-sight obstructions. Cavity scanning can also be used for confirming the effectiveness of void backfilling and to confirm historic laser scans or surveys.

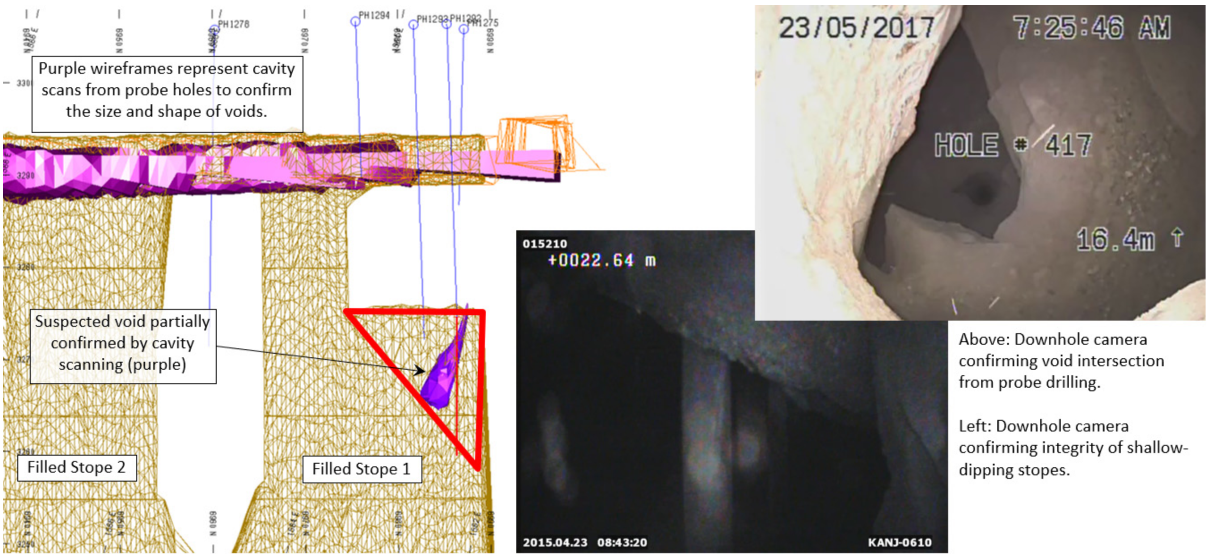

Fig. 4. Left: Cavity scanning to confirm void shapes and sizes. Note that sometimes only partial scans are possible due to line-of-sight obstructions. Right: Downhole camera photograph examples.

3. Void Analysis and Risk Assessment

The analysis and risk assessment of void hazards can take many forms depending on the type and confidence of available data and can vary in complexity from simple empirical estimates to complex numerical modelling.

3.1 Geometry and location

In order to undertake any type of analysis or risk assessment of a void, it is essential to understand its current geometry and location. The geometry includes the size and shape of the void as well as any possible interconnectivity that could facilitate progressive collapse.

In the event that some form of collapse, caving or failure has recently occurred, or is occurring, it is imperative that up-to-date information on void geometry & location is available to form the basis of an analysis or risk assessment. This is a key factor behind utilizing recurring probe drilling and cavity scanning as an open pit progresses vertically downward through historic underground workings.

3.2 Ground conditions

In any underground mining operation, ground conditions combined with nature of the orebody influence the size and shape of the voids as well as the mining method. Equally, ground conditions have an influence on slope stability and the interaction between and open pit and historic underground workings. By way of example, the behavior of a strong, massive rock mass would be vastly different to that of a weak, highly fractured rock mass.

In cut and fill stoping operations where primary and sometimes secondary stopes are backfilled with a cemented or uncemented fill, the original, stiff host rock (usually the orebody) is replaced by a much more ductile and weaker backfill. In situations with high intensity or prolonged rainfall, the backfill may be susceptible to mobilization under saturated and loaded conditions. Effectively, backfill is an introduced ground type with semi-engineered characteristics. However, in many cases and particularly in older underground mines, the backfill characteristics are unknown and may change over time.

3.3 Failure mechanism

Failure mechanisms involving historic underground workings propagating to the surface can take many forms. They are dependent on:

- Ground conditions.
- Size and geometry of underground workings.
- Initial (virgin) and changing stress regime.
- External factors (e.g. rainfall, seismicity, blasting, etc).

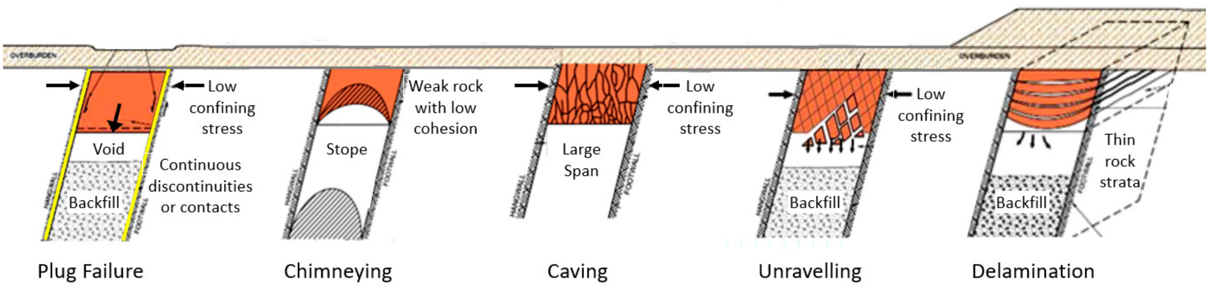

Fig. 5. Crown pillar failure mechanisms (simplified from Carter, 2014).

Common crown pillar failure mechanisms illustrated in Fig. 5 comprise:

- Plug failure typically requires steeply dipping, well-defined continuous discontinuities or geological contacts. At low confining stresses, or as horizontal stresses are reduced with overburden removal during open pit mining, effective friction on the discontinuities reduces to initiate downward sliding of a block or plug.
- Chimneying can occur in weak rocks with low cohesion.
- Caving typically requires large spans and low confining stresses to propagate. It can involve persistent discontinuities or general disturbance of the rock mass.
- Unravelling requires a blocky rock mass, typically with at least three joint sets, which under high confining stresses would be reasonably stable. However, at low confinement, wedges form and fall into the void.

- Delamination may occur in thin or laminated rock strata by allowing separation between the layers (i.e. bedding planes, foliation or continuous joints).

Crown pillar collapses or failures in active and abandoned mines have been examined since the late 1980's (Bell et al. 1988; Carter, 1989; Carter, 2014). The purpose of these studies has been to develop and improve the understanding around the management of void risks in open pit mines as well as the public arena. In order to design new crown pillar layouts, or assess the stability of historic voids and their crown pillars, three basic approaches can be used:

1. Empirical methods:
    - Rules-of-thumb: some described in Table 1.
    - Rock mass quality and the scaled-span method (Carter, 1992; Carter, 2014).
2. Structural analysis and cavability assessments.
3. Numerical modelling.

3.4 Potential impact of rapid failure to surface

In an open pit with underground workings below, the key risk or unwanted event is rapid failure of crown pillars propagating to the surface and impacting on personnel and equipment (Fig. 2).

Miners and engineers in the 20th century used benchmarked rules-of-thumb for surface crown pillar, typically relating the minimum height of the rock cover above a void with the maximum void span or width in a ratio as outlined in Table 1. Most of the rules-of-thumb are very subjective and based on experience of the miners; although more recent ones include rock mass quality assessments such as Q (Barton et al. 1974).

In Australia, the '2:1 rule' whereby the height of the rock cover above a void should be at least twice the maximum void span or width is frequently used as a guide for void management. However, its applicability is seldom verified against possible failure mechanisms based on the site-specific ground conditions.

The use of minimum rock cover to span ratios should be verified considering the ground conditions and possible failure mechanisms as illustrated in Fig. 6. Numerical models can be used to simulate the effects of reducing confinement and destressing the rock mass as an open pit progresses.

Table 1 Rock Cover vs Span Rules-of-Thumb

Min. Rock Cover to Span Ratio (RC:S)	Ground Conditions	Era	Region	Basis
1:1	'Good rock'	Pre-2000	Canada	Experience / benchmarking of
3:1	'Poor rock'	Pre-2000	Canada	miners (Carter, 2014).
2:1	Not specified	2000-Now	Australia	Void management plans.
(1.5-3):1	Not specified	1980s	South Africa	Case records (Bell et al. 1988).
5:1	Oxide/Soil	2016	Ivory Coast	Numerical modelling – Fig. 6
$1.55Q^{-0.62}$	Poor to good rock	After 2000	Canada	Case records: the Q rock mass quality index (Carter, 2014)

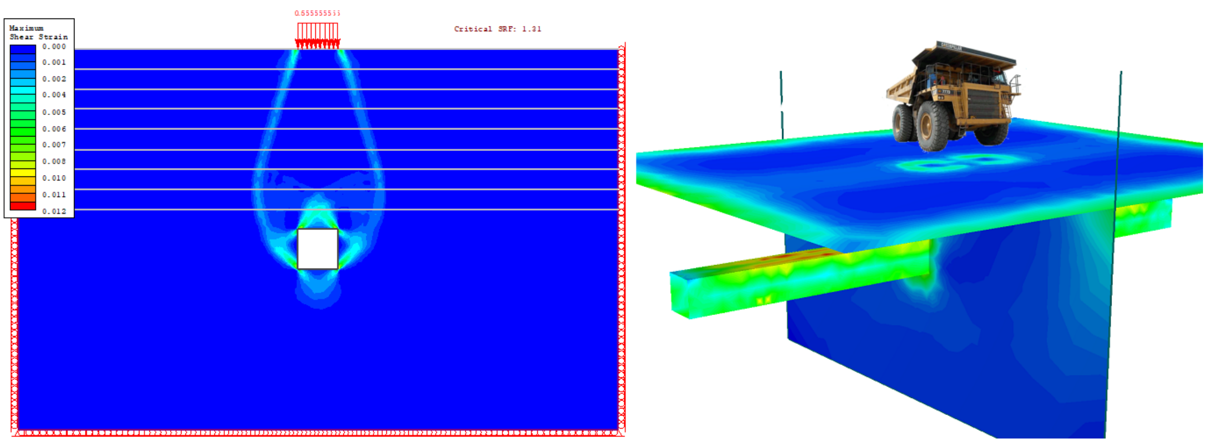

Fig. 6. 2D & 3D finite element models used to determine the risk of unsupported artisanal workings collapsing as excavations progress downward with surcharge loading from heavy mining equipment to derive the 5:1 rock cover vs. span ratio applicable to oxide / residual soils, Ivory Coast.

Once progressive failure mechanisms such as caving or unravelling initiate, geometric cavity failure analysis (GCFA) can be used to predict whether the failure will propagate to the surface and what impact it may have in terms of subsidence depth. Geometric cavity failure analysis is a simple and crude geometric analysis that considers the height from the surface to the base of a void and material swell factors during progressive failure as described in Fig. 7. It does not consider rock mass quality, nor is it applicable to plug failures. Geometric cavity failure analysis has been effectively used to estimate subsidence at the surface. It is important to note that in most instances, engineers and mine personnel are completely unaware that the crown of a stope or drive is progressively failing until the area is probe drilled or subsidence is visible at the surface.

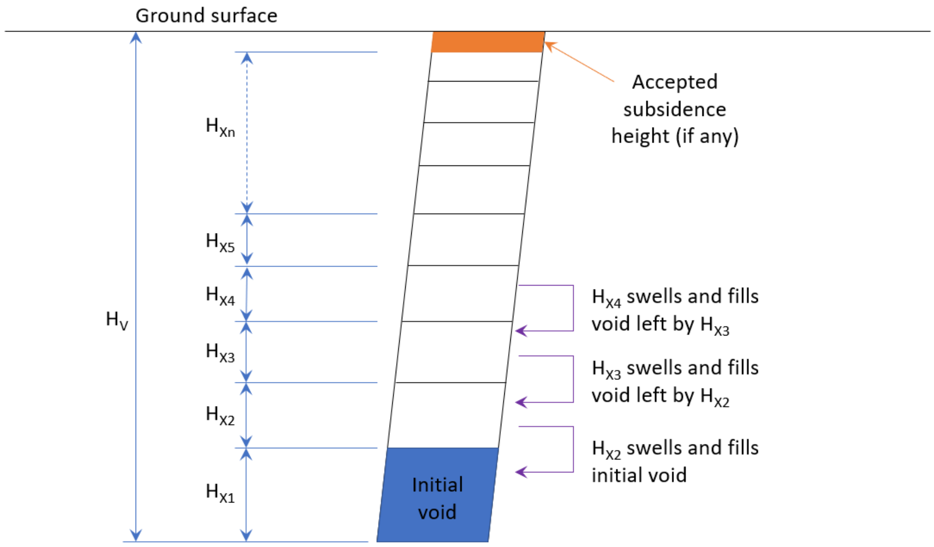

Fig. 7. Geometric cavity failure analysis for progressive failures.

3.5 Voids impact on pit slopes

Voids located behind or below pit slopes have the potential to cause significant impact to stability. Assessment of potential impact is possible with the use of numerical modelling (Svartsjaern et al. 2015; Karakus et al. 2017). However, it is critical to have a sound understanding of ground conditions to anticipate potential behavior during various stages of open pit excavations. By way of example, Fig. 8 (left) presents a network of small backfilled stopes and drives in relatively poor ground conditions intersected by a 250m high pit slope at the base. In Fig. 8 (right), very good conditions enabled the development of a 200m high stope / cave that remains open with a void volume of approximately 1,000,000m^3 that is located 100m below the current pit floor with future mining expected to reduce the rock cover to less than 50m.

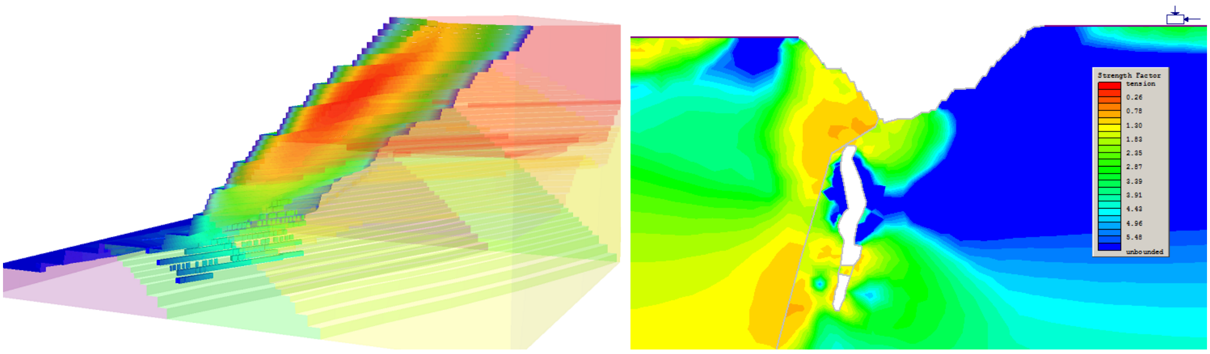

Fig. 8. 2D & 3D finite element models for various scale void problems and pit slopes, Australia.

4. Void Risk Management

Void risk management strategies are highly dependent on the scale of the historic underground workings. Simple rules-of-thumb such as those presented in Table 1 are not applicable or impractical to use for large voids developed through large stopes, sub-level or block caves. Equally, risk management strategies applied to small voids from room and pillar mining or small stopes, are generally not applicable or can be impractical for large voids.

4.1 Back-filling voids

Probe drilling and back-filling voids can be done concurrently when required. Probe drilling and cavity scans assist with determining the amount of fill required and the optimal location of fill-holes.

Back-filling of voids is typically undertaken for larger voids and spans where the crown failure may unexpectedly or rapidly initiate and propagate to the surface. Back-filling typically involves drilling larger diameter probe holes and either:

- Pumping a hydraulic, cemented fill where void connectivity is limited or has been blocked through engineered barricades.
- Dumping a muck pile of cohesionless, rock fill typically comprising gravels and particle sizes no greater than say 50mm in diameter. Under self-weight, the muck pile flows into the large diameter probe hole. Several muck piles are used until the void is as full as possible, considering the angle of repose of the fill.

Downhole camera surveys and cavity scanning can be used throughout the back-filling process to verify the size of the remaining void. Very large voids such as the one illustrated in Fig. 8 (right) would require several stages of back-filling, which is both time consuming and costly. As a result, in this type of situation, back-filling would likely be undertaken sequentially or in stages. This would both minimize the risk of void propagation to the surface by reducing enough of the void volume as rock cover is reduced, and allow for open pit mining to continue, although usually much slower than if there were no void.

It is important to note that back-filling voids may introduce additional risks if the underground mine is operating in parallel. Air-blast and inrush potential exists, particularly when dealing with a highly connected void network with numerous vertical workings such as vent rises, ore passes or other shafts.

4.2 Blasting to collapse voids

Collapsing voids with blasting practices has been an effective form of risk mitigation in void management for several years. It is frequently used in favor of back-filling for smaller voids.

Following the identification of voids below the mine floor after probe drilling, precondition blasting designs are integrated within production blast designs to collapse underground workings 1 to 2 mine benches below the production mining bench (Fig. 9). This allows adequate cover to safely mine the upper bench while maintaining safe cover above the collapsed voids in the event of ineffective precondition blasting results. Targeting underground workings well below the active mining bench provides both vertical distance (rock cover) and time for subsequent probe drilling verification following the completion of excavation on the production mining bench.

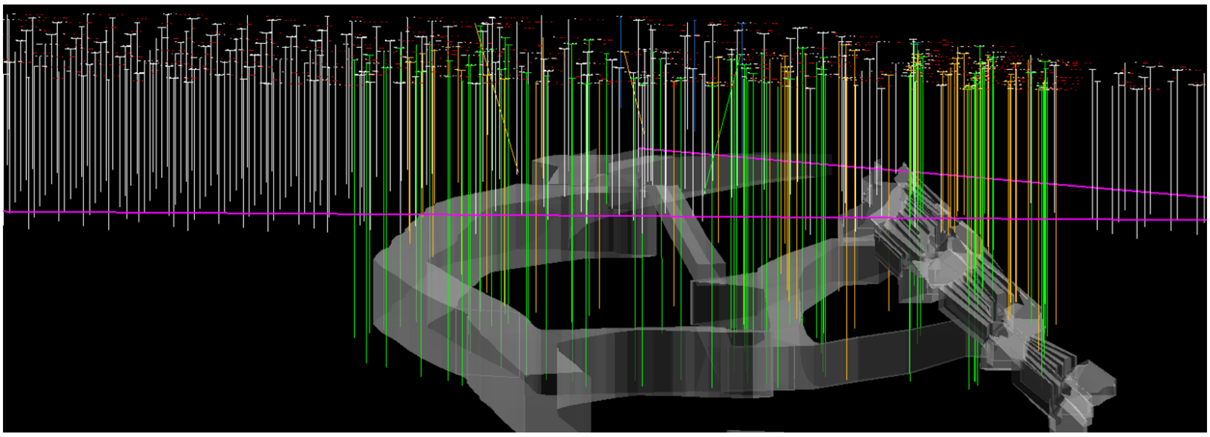

Fig. 9. Example of the complexity of blast patterns when collapsing voids (white lines represent the standard blast pattern for the bench while green and orange lines represent preconditioning holes targeting the backs and ribs, respectively).

Preconditioning holes generally follow a diamond grid pattern over the drives (tunnels), declines and other small underground workings targeting the "backs" (roof) as well as the "ribs" (walls) of the drives and declines to effectively blast the top and side of the tunnel, eliminating any means of support for broken/blasted rock to support blasted material and create cavities within the tunnel.

Blasting to collapse larger underground workings such as caverns or stopes is dependent on their size (i.e. span and length). In the case of inclined stopes, the hanging wall of the stope is often blasted down to the nearest production drive, using the production drive below the stope to act as a barrier to prevent broken material from the blast propagating to lower levels if connectivity exists.

4.3 Excavating voids

Excavating voids or blasted-void remnants is a critical risk mitigation method whereby the crown of a drive, stope or other void is removed. Typically using surveying techniques or GPS machine guidance, large excavators or shovels with roll-over protection systems 'excavate voids' is used to prove or confirm that the void was successful backfilled, blasted and that no remnant void is present.

4.4 Displacement monitoring

In circumstances where the result of crown pillar failure and the risk of void propagation to the surface that presents an immediate threat, subsurface displacement monitoring systems can be used to provide forewarning of collapse.

Most subsurface displacement monitoring systems require planning and involve drilling for downhole installation. These may comprise, but are not limited to:

- Deep hole wire extensometers anchored above a crown to monitor extension prior to, or during crown pillar failure; Fig. 10 provides an example illustrating stable conditions.
- Inclinometers and shape accel arrays can be used to identify shearing and displacement sub-perpendicular to boreholes. These can be used to monitor displacements and failure of large stopes or caves using vertical boreholes in close proximity to the ribs or walls. Alternatively, inclined boreholes can be used above crown pillars, although only partial deformation will be measured.
- Time-domain reflectometers (TDR) or a series of networked SMART markers can also be used to monitor deformation and are particularly useful for progressive crown pillar failures or to monitor the progress of sub-level and block caving mining operations (Nicoll et al. 2017).
- Open boreholes that can be used for repeat downhole camera surveys and cavity scanning to measure progressive crown pillar failure (Lowther et al. 2016).

When dealing with voids in proximity to pit slopes, surface deformation monitoring systems are implemented alongside subsurface monitoring. These may include a combination of survey prisms and automatic total stations, real and synthetic aperture radar, laser scanning and surface extensometers (Villegas & Nordlund, 2012; Bar et al. 2016; Baczynski & Bar, 2017).

Subsurface and surface displacement monitoring systems can be fitted with telemetry for near real-time data capture. Alarms can be automatically triggered to mine dispatch offices to warn personnel or linked to audio-visual devices in the field.

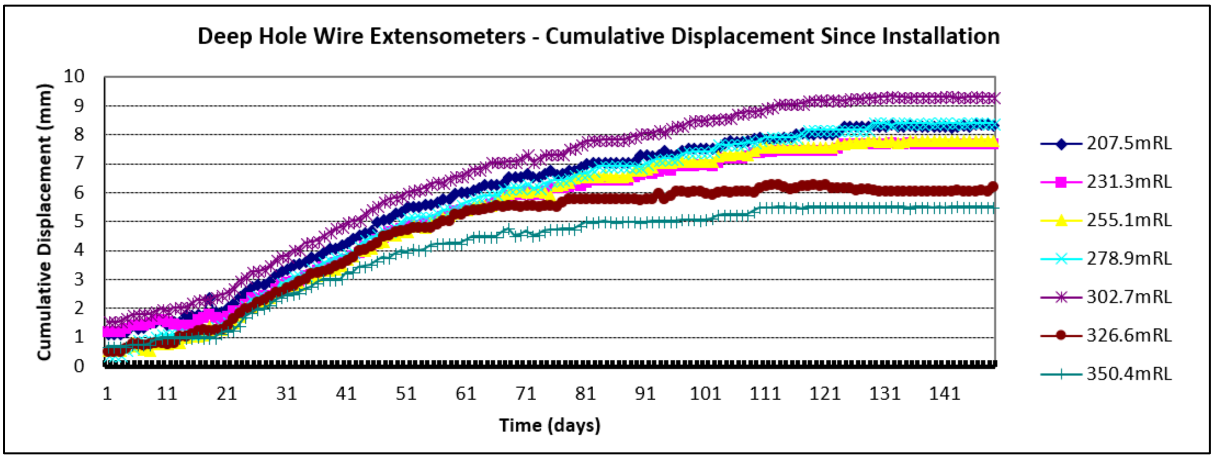

Fig. 10. Deep hole wire extensometers showing a stabilizing trend in a crown pillar some 100+ days after installation. Initial displacement likely to be instrument headframe settlement.

4.5 Microseismicity

Monitoring for microseismicity using triaxial geophones has become more commonly used in large underground mines in the last decade. Most, if not all modern sub-level and block caving operations utilize a robust network of microseismic sensors to accurately locate microseismic events which may be related to rock fracturing as a result of stress changes (Pfitzner, 2003; Dixon et al. 2010; Nicoll et al. 2017). This technology is typically used to identify areas of higher risk for rock bursts and the propagation of the seismogenic zone or front as caves propagate as shown in Fig 11.

In the same context microseismic monitoring has been applied for monitoring rock fracturing around complex network of smaller voids, although a denser array of triaxial geophones is usually required. As such it is generally used as a secondary control measure alongside some form of displacement monitoring.

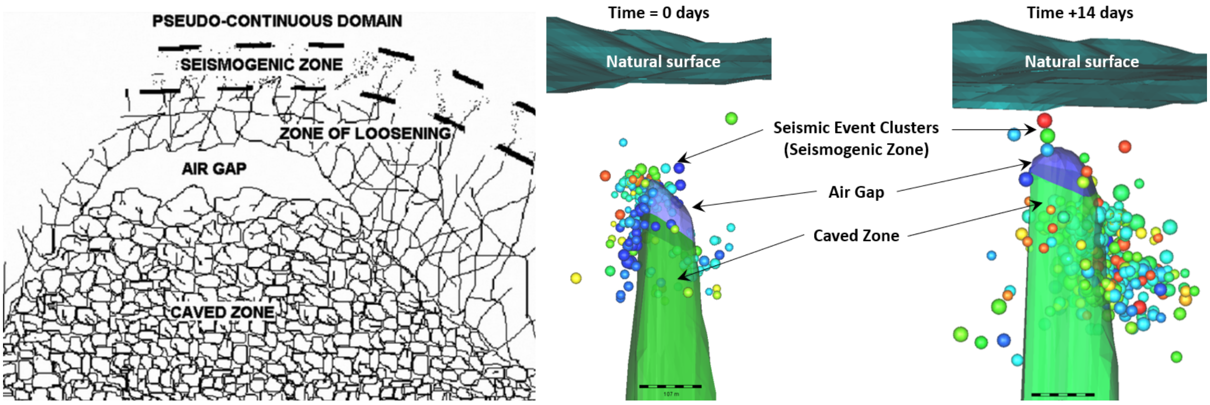

Fig. 11. Left: Caving model (Duplancic, 2001); Right: microseismic events and open hole monitoring used at an undisclosed block caving mine to illustrate cave propagation toward the surface.

4.6 Hazard awareness

Hazard awareness is a critical control measure in the void management process. The goal of hazard awareness is to transfer knowledge and understanding of the risks and control measures to the general workforce operating in the open pit, and may consist of:

- All personnel working in an open pit with void hazards undergo an induction for an overall understanding on voids and the risk management process used.
- Void maps illustrating the location and type of void hazards (Fig. 12). GPS machine guidance or collision avoidance systems can also display void & other geotechnical hazard areas.
- Demarcation of void hazards on the surface (i.e. on the pit floor).

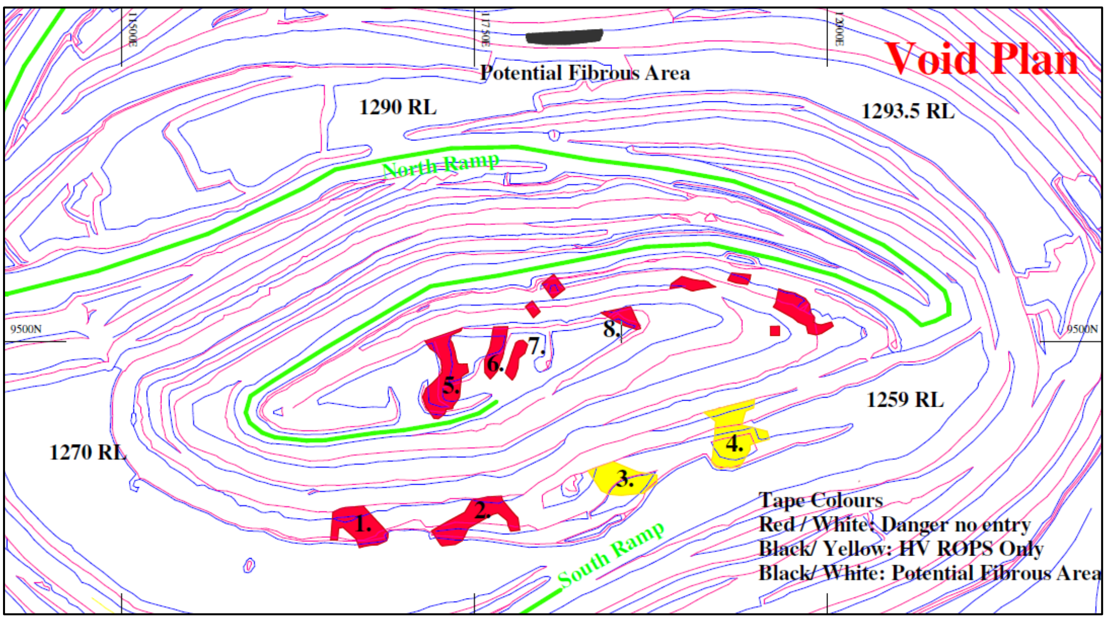

Fig. 12. Example of a Daily Void Map (red zones represent red & white tape demarcated high risk areas; yellow zones represent black & yellow tape demarcated low-moderate risk areas).

4.7 Demarcation on surface level

Unlike the voids presented in Fig. 13, most voids or historic underground workings remain a 'hidden hazard' until they are backfilled, blasted and/or excavated. That is, personnel working in the open pit rarely see a physical void on the pit floor. As such, demarcation of void hazards on the surface is a critical visual control measure to ensure that personnel do not traverse across them.

Void hazard demarcations are generally planned well ahead of time or mining in 4-6m vertical 'flitches' (intervals) to assist mining and drill and blast engineers to plan and sequence excavations. Typically, void demarcations are planned at several benches, say >50m vertically in ahead of mining.

Fig. 13. Known voids at surface demarcated with red & white danger tape to prevent access. Note: open voids such as these are rarely seen on the surface.

Void demarcation areas are generally offset a designated distance from the void itself (in plan view). These offsets are generally dependent on ground conditions and rock cover. Void demarcation on the pit floor requires a surveyor to identify and mark out the exclusion zone on the surface. Typically, at most mines, two types of void demarcation exist:

- Red & white: high risk void present beneath. No personnel or equipment can traverse an area demarcated as a red and white; these are typically excluded using a physical bund.
- Black & yellow: low to moderate risk void, back-filled or blasted void area. Only heavy mining equipment with roll-over protection systems may traverse areas demarcated black and yellow (i.e. no personnel on foot or in light vehicles).

The use of only two demarcations is often adopted to avoid confusion for personnel working the pit; however, in some cases, additional 'no stopping' signage is used (e.g. back-filled or blasted void areas on haul roads).

Void demarcations are checked several times per day by void officers and pit supervisors to ensure their integrity as they may be damaged by mining equipment, wind or rainfall.

5. Discussion

Voids are in most cases an 'unseen hazard', and in the best possible outcome during the operating life of an open pit, remain unseen or do not present any significant risk to personnel on the surface. A key drawback of the 'invisible nature' of void hazards and risks is that both the workforce and management teams in an open pit can be either unfamiliar with the potential risks or become complacent when risks are managed well. Void management technicians and geotechnical engineers are often faced with the challenge of convincing personnel that the hazard actually exists.

Several aspects of void management in open pit operations with historic underground workings have been discussed to provide general framework for void identification, analysis and risk management (Fig. 1). However, it remains essential that since every orebody, every underground mine and every open pit is unique, that the void management strategy is developed thoroughly to the site-specific ground conditions and void risks. Some key points for consideration:

- The use of generalized rules-of-thumb such as '2:1 rule' may provide some guidance; however, may not be suited to all ground conditions. It is critical that engineers using these rules understand their intended use and along with their limitations.

- Displacement and microseismicity monitoring systems need to be tailored to the specific problem they are trying to address (i.e. a monitoring strategy for a stope network would be quite different to that of a large block caving operation).

With considerable advances in computing capability and monitoring technology in the last decade, and likely advances in the next decade, further improvements to the understanding of ground conditions and failure mechanism prediction and early warning systems will become available. By way of example, the use of drones or UAV and laser scanning technology is becoming more prominent and is expected to have a significant impact on monitoring in the near future.

Practitioners are encouraged to remain familiar with the new technology and find ways to integrate them alongside conventional methods when dealing with underground workings in active open pits.

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APPENDIX 7.17
Procedure for mining in the vicinity of suspected voids & unstable ground (underground mine workings) - SGMI - August 2022

Procedure for mining in the vicinity of suspected voids & unstable ground (underground mine workings) (DRAFT)

Subject	**Proposed Safe Operation Procedure**			
Owner	**Benson Plunkett**	**Doc. No.**		
Compiled by	**B Plunkett/ T O'Reilly/ Andrew Ellis**	**Rev: 0**	**Knocknacran West**	✓
Date of Issue:	**04/08/2022**			

Introduction

This proposed Safe Operation Procedure (SOP) is based on the SOP currently in place for the existing Knocknacran Open-Cast Mine.

The SOP provides details on the work practices for the safe removal of overburden, interburden and Gypsum from locations above known or suspected underground mine workings, including areas of subsidence.

The SOP is based on practices developed and refined over many years of experience gained by Saint-Gobain Mining Ireland (SGMI) from mining at Knocknacran.

Objectives

The objectives of the SOP are:

1. To ensure the safety of personnel at all times when working above underground mine workings and areas of subsidence.

2. To provide clear direction to all personnel regarding responsibilities and procedures to be followed when working over underground workings and areas of subsidence.

3. To reduce the likelihood of a serious incident occurring in relation to working over underground mine workings and areas of subsidence.

Principles of Working over Underground Mine Workings and Areas of Subsidence

The main principle for working over underground mine workings and areas of subsidence is to 'work out' from 'known safe ground' based on confirmatory mine survey plans provided by Mine Records and signed off by Mine Management and the contractor Project Management team.

Records of all signed-off plans will be kept in the Mine Survey Office.

Copies of plans for active mining areas will be pinned to a notice bord in the mine site office at the Knocknacran West site for inspection by all staff working on the removal of overburden/interburden and extraction of Gypsum. Such plans will also be made available to contactors who are involved in stripping campaigns for overburden/interburden removal.

The plans will be updated periodically by the Mine Surveyor as new/additional information becomes available during stripping of materials and extraction of Gypsum. The size (area and depth) of overburden/interburden to be stripped will be agreed with Mine Management and contactor prior to any works been carried out.

Following blasting and removal of Gypsum after each blast, the area in question will be surveyed by the Mine Surveyor. Plans will be updated and recorded in the mine surveying office.

Figure 1 below provides a plan of the Upper Seam mine workings from the historic Drumgoosat Underground Mine, with the junctions of the workings ranked by area. Also shown on the figure are the known limits of the various subsidence events associated with the Drumgoosat mine. This plan will be used to form the basis of a series of Risk Maps for the stripping of overburden/interburden and extraction of Gypsum from the Knocknacran West site.

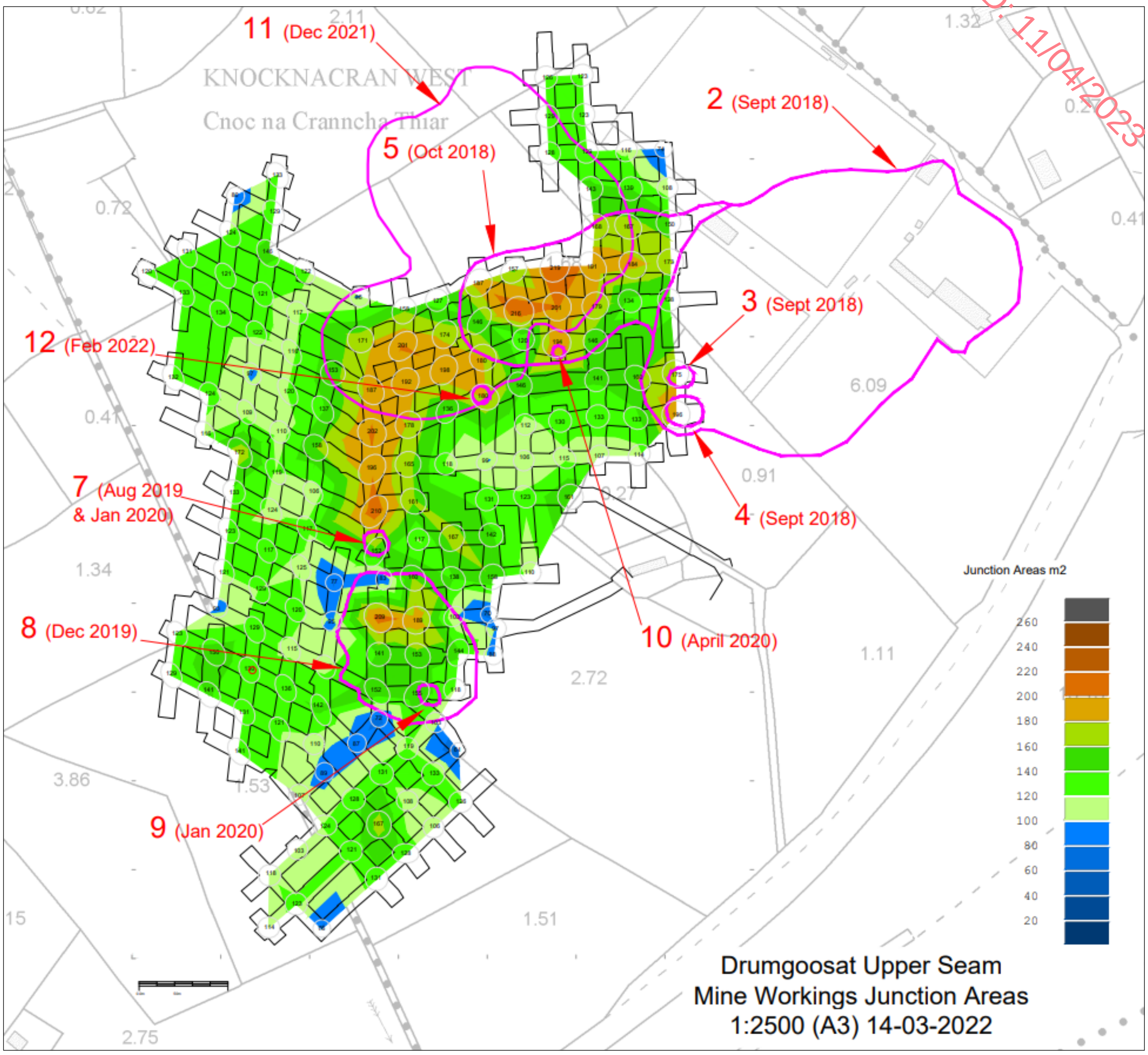

Figure 1: Plan of the Upper Seam mine workings with junction areas, and recorded subsidence events

Main Stages of Mining Gypsum from an Open-Cast Mine

There are two main stages of mining Gypsum from an open-cast mine:

1. Removal of overburden and interburden (i.e., soils and glacial overburden, mudstone and dolerite) by stripping using a combination of bulldozers, excavators and trucks.

2. Drilling and blasting of the Gypsum Seams.

Removal of Overburden/Interburden, Mudstone and Dolerite by Stripping

The removal of overburden and interburden (mudstone and dolerite) will be dictated by the following:

- Design of open-cast bench positions over mined and un-mined areas.
- The location of Upper and Lower Gypsum Seam sub-outcrops.
- How subsidence features identified have affected Gypsum Seams at different working levels.
- Extraction of Upper Seam Gypsum (from previously mined & un-mined areas).

- Extraction of Lower Seam Gypsum (from previously mined & un-mined areas).
- Roof beam thicknesses of both Upper and Lower Seams - (old mine plates/plans & geotechnical reports will provide information).

From historical mine survey records, a 3D model for floor levels in both Upper and Lower Gypsum Seams from Drumgoosat will be developed - these will be used to generate a Roof Elevation Model for the historical mine workings in conjunction with information from historical borehole logs.

Before Overburden/Interburden, Mudstone and Dolerite Removal a Risk Map will be created based on:

- Locations/plans of known subsidence and sinkholes (from Figure 1 above for Upper Seam Gypsum).
- Identification of roadway junctions with large roof spans as shown on Figure 1 above.
- Identification of areas that might be hazardous - based on roof beam thickness – for example mine workings in the Upper Seam are likely to have roof a thickness of ca. 3.0 m (based on a full seam thickness of ca. 10 m; a working height of ca. 5.0 m; and a floor beam thickness of ca. 2.0 m).
- Identification of areas with two working horizons in a single Gypsum bed/seam.
- Identification of areas with mine workings over 6.0 m in height.
- Historical mine records to locate areas where water intersections were previously recorded underground, or anomalies such as un-mined areas or small pillars.

Excavation Method

The Risk Map(s) produced will be used to determine the excavation method/approach to be used – for instance:

- Excavation of overburden/interburden - approach from areas of insitu rock (areas where no underground mining has taken place).
- Definition of access routes to excavation area – i.e., travelling over pillars and roofs between pillars – not to travel over underground roadway junctions as defined by the Risk Map (e.g., Figure 1 above).
- If a cavity is exposed or suspected – test holes will be drilled to determine its extent.

Production Drilling for a Blast

When overburden, interburden, mudstone and dolerite are removed, and Gypsum is exposed, the following procedures will take place before drilling and blasting is carried out:

- Generate a thickness model of the Gypsum Seam based on an exposed and updated survey of the top of Gypsum v's previously known/surveyed roof levels.
- Drill test holes from pillars/unmined ground to determine the actual roof beam thickness when no projected/inferred information is available.
- Update roof beam thickness model with test hole information.
- Design access route to drill test holes and blasting areas by travelling over pillars and roadways between pillars - avoiding roadway junctions, and areas of subsidence (see Figure 1 as an example).
- Where a roof beam is not suitable for working/travelling over - blast to be drilled from surrounding pillars (as is currently the procedure at Knocknacran open-cast).
- Carefully mark-out routeways to and from blasting areas to avoid areas of risk, such as underlying 4-way junctions and areas with subsidence.

The above procedures are based on current working practices used when extracting Gypsum from the current operating Knocknacran Open-Cast Mine.

Risk Analysis

SGMI understands that minimising risk is a key component of any mining activity, and continually considers operating risk throughout all its activities.

Mitigation strategies are incorporated (reviewed and revised) into mine design and mine operational work practices where possible.

Presented below is a risk assessment based approach proposed for the safe removal of overburden/interburden and extraction of Gypsum at Knocknacran West. The risk assessment process allows for the identification, qualitative assessment, and development of treatment strategies/actions for key mining related risks.

In order to adequately communicate and rank the consequence of perceived risks, SGMI will utilise a quantitative assessment when evaluating the Likelihood and Impact (scored from 1 to 5), as shown below, rating the activities and the impact for not being addressed.

Likelihood		
Score	**Description**	**Guidelines**
5	Almost Certain	This is a significant threat that could occur at any time. Immediate remedial action is required to remove or reduce the risk.
4	Likely	The threat exists, and it indicates high probability. Action is required to reduce this risk.
3	Possible	The threat exists but the history or expectation of this type of situation indicates occurrence is moderately probable. Action could be taken to reduce this risk.
2	Unlikely	A slight threat is perceived from this source, but the situation is unlikely to occur.
1	Rare	No perceived threat exists from this source. No action is required to reduce the risk.

Impact		
Score	**Description**	**Guidelines**
5	Severe	Major risk, injury to personnel and/plant, resulting in severe damage and therefore re-design.
4	Significant	Substantial risk, injury to personnel and/plant, resulting in damage or re-design.
3	Moderate	Notable risk, injury to personnel and/plant, causing down time of operations or similar.
2	Low	Minor risk, injury to personnel and/plant, some impact on daily operation.
1	Negligible	So minor as to be regarded as having no consequence – minimal impact on daily operation.

Short descriptions of the risk tiers are included below:

Tier 1: (Very High Risk): These are risks that would impact on the project and recovery of Gypsum. Mitigation may not necessarily remove or decrease the risk depending on ground conditions. The risk will be continually assessed before work commences

Tier 2: (High Risk): These are risks that may affect the project or may affect the project schedule. They will require definitive further work/analyses to mitigate.

Tier 3: (Moderate to Low Risks): These are minor risks that can be mitigated easily or those that are deemed as not likely to not affect the safe recovery of Gypsum.

Tier 4: (Very Low Risks): These are negligible risks which do not have any impact and are highly unlikely.

The product of the resultant scores will be assigned Likelihood and Impact results in a Risk Rating which translates to a qualitative risk matrix, as shown below.

			Likelihood				
			Very Low (Rare)	Low (Unlikely)	Medium (Possible)	High (Likely)	Very High (Almost Certain)
		Score	1	2	3	4	5
Impact	Very High (Severe)	5	Moderate	Moderate	High	Very High	Very High
	High (Significant)	4	Moderate	Moderate	High	High	Very High
	Medium (Moderate)	3	Low	Moderate	Moderate	High	High
	Marginal (Low)	2	Very Low	Low	Moderate	Moderate	High
	Very Low (Negligible)	1	Very Low	Very Low	Low	Moderate	Moderate

Risk Mitigation

In order to effectively reduce the Impact and/or Likelihood of risks associated with mining activities at Knocknacran West, SGMI will adopt (and further develop) the mitigation measures currently in place at Knocknacran. Following application of such measures the residual risk is quantified by a description of the effectiveness of the controls applied, as presented below.

Control Effectiveness	
Rating	**Description**
Satisfactory	The control environment is operating effectively, providing a reasonable level of assurance that objectives are being achieved.
Some Weakness	The control environment has some weaknesses/inefficiencies. Although these are not considered to present a serious risk exposure, improvements are required to provide reasonable assurance that objectives will be achieved.
Weak	The control environment is not at an acceptable standard, as many weaknesses/inefficiencies exist. Reasonable assurance does not exist that objectives will be achieved.

The risk matrix for the removal of overburden, interburden and Gypsum from locations above or adjacent to known or suspected underground mine workings and areas of subsidence, with risk mitigation and control effectiveness, is presented in the Table below.

Table 1: Risk Matrix for the removal of overburden, interburden and Gypsum from locations above or adjacent to known or suspected underground mine workings and areas of subsidence

Risk №	Risk Issue	Impact		Likelihood		Risk Rating		Mitigation or Control	Control Effectiveness
1	Working above a pillar	1	Negligible	5	Almost Certain	5	Moderate	Updated mine survey records and risk map. Test drill edge of pillars to check for void space. Revise Risk Map.	Satisfactory
2	Working above a roadway	2	Low	5	Almost Certain	10	High	Test drilling from safe ground to confirm size, shape & depth of underlying roadway and roof beam to provide an updated mine survey plan. Update roof beam thickness model with test hole information. Revise Risk Map.	Satisfactory
3	Working above a junction	3	Moderate	5	Almost Certain	15	High	Test drilling from safe ground to confirm size, shape & depth of underlying junction and roof beam to provide an updated mine survey plan. Update roof beam thickness model with test hole information. Revise Risk Map.	Satisfactory
4	Working above a 4-way junction	4	Significant	5	Almost Certain	20	Very High	Test drilling from safe ground to confirm size, shape & depth of underlying 4-way junction and roof beam to provide an updated mine survey plan. Update roof beam thickness model with test hole information. Revise Risk Map.	Satisfactory
5	Working above or adjacent to fallen/collapsed ground	5	Severe	5	Almost Certain	25	Very High	Test drilling from safe ground to confirm size, shape & depth of fallen/collapsed ground to provide an updated mine survey plan. Revise Risk Map.	Satisfactory
6	Construction access routes for stripping material, and drilling and hauling of Gypsum	2	Low	5	Almost Certain	10	High	Test drilling from safe ground to confirm size, shape & depth of underlying roadways/junctions and areas of fallen/collapsed ground, and roof beam to provide a safe access to/from stripping area and Gypsum production faces. Update mine survey plan and roof beam thickness model with new information to revise Risk Map.	Satisfactory

APPENDIX 7.18
Roof Beam Stability and Kinematics - SRK - July 2022

SRK Consulting (UK) Limited
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E-mail: enquiries@srk.co.uk
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External Memorandum

To:	Pat O'Connor	From:	Neil Marshall
Company:	Saint-Gobain Mining Ireland	Project Number:	UK31696
Copied to:	Benson Plunkett	Project Title:	Roof Beam Assessment
File Ref:	31696_Report_3_Roof beam Stability_Draft(V2).docx	Date:	22 July, 2022
Subject:	**Roof Beam Stability and Kinematics**		

1 RFI POINTS

The following RFI points have been combined within this document as they relate to stability of mine elements.

Roof Beam Stability (RFI Ref: Points 22.i.c and d)

The roof beam stability assessment should be updated to include assessment of safe unsupported spans for the proposed maximum slope configurations. Roof beam instability needs additional consideration as confined pressure is released from the removal of overburden, interburden and upper seam gypsum. It is recommended that the rock mass should be characterised, and the maximum unsupported span determined for the reduced overburden loads to show that the roof beam will be stable.

Kinematic Pillar Failure (RFI Ref: Point 22.ii.b)

Pillar failure through rock mass has been calculated but the kinematic failures mechanism does not appear to have been considered. Additional planar failure mechanisms should be analysed to determine the potential for joint and bedding plane failures and details shall be submitted accordingly.

2 RESPONSE

2.1 Response Clarification

In a meeting with GSRO and Wardell Armstrong (WA) on 24 June 2022 WA clarified that kinematic pillar failure mechanisms and safe unsupported roof beam spans related to the interaction of these mine elements where they are exposed in the quarry faces and the potential for kinematic instability to impact the stability of the quarry slopes. Quarry slope design was the responsibility of Golder Associates (Golder) and therefore outside the scope of work that SRK has historically been involved in on behalf of Saint-Gobain Mining Ireland. Golder will therefore address these elements of the RFI's.

SRK's contribution to this response will focus on the roof beam stability of the underground workings within the footprint of the quarry as it is impacted by quarrying and the consequent reduction of overburden loading.

2.2 Roof Beam Stability in Response to Overburden Unloading

As overburden is removed from above the underground mine as a function of quarry excavation, loading of the roof beam decreases and stability should improve to some degree. To simulate this SRK has carried out 3D finite element modelling using the Rocscience computer program RS3. The underground survey of the Lower Seam workings, the Lower Seam geology model and the Knocknacran West Quarry design have been interrogated to determine the range of room and four way intersection spans, the range of roof beam thickness and the range of quarry excavation depth to the underground workings. These ranges are:

- Room spans – 10 m to 12 m.
- Intersection spans – 14 m to 17 m.
- Roof beam thickness – 3 m to 12 m.
- Depth to workings - 100 m.

These ranges have been simulated by the construction of two RS3 mine geometry models, one comprising 10 m square pillars separated by 10 m wide rooms and another comprising 10 m square pillars separated by 12 m wide rooms. The 10 m wide rooms result in 14 m wide four way intersection spans. The 12 m wide rooms result in 17 m wide intersection spans. For each mine geometry model three additional models were constructed containing 3 m, 6 m and 12 m thick roof beams. Each of these models comprised a 100 m thickness of overburden which was progressively removed in 25 m slices to expose the top of the roof beam thus simulating quarry excavation. At each stage of overburden removal the maximum deflection of the underside of the roof beam above the mining room and wider span four way intersection was interrogated.

An annotated perspective view of the RS3 model along with plan views of the two mining layouts is shown in Figure 1. The output of the modelling is presented as graphs of beam span versus maximum roof beam deflection in Figure 2 for the 3 m thick roof beam, Figure 3 for the 6 m thick roof beam and Figure 4 for the 12 m thick roof beam. The strength of the rock units was represented by the Generalised Hoek-Brown constitutive model, with input parameters as shown in Table 1. The value of GSI defines the fracture or jointing condition of the rock mass with lower GSI values representing rock containing a relatively greater number of joints or fractures than rock characterised by higher GSI values. The strength of the overburden is a composite of Upper Gypsum, mudstone and drift.

All the simulations converged to a solution indicating that the roof beams remained stable irrespective of roof beam span, thickness or overburden loading.

Table 1: RS3 Model Strength Parameters

	Gypsum Roof Beam	Gypsum Pillars	Overburden
Unit Weight (MN/m^3)	0.023	0.023	0.02
UCS (MPa)	15	20	10
GSI	55	75	40
mi	8	8	7
D	0	0	0
Poisson's Ratio	0.15	0.15	0.3

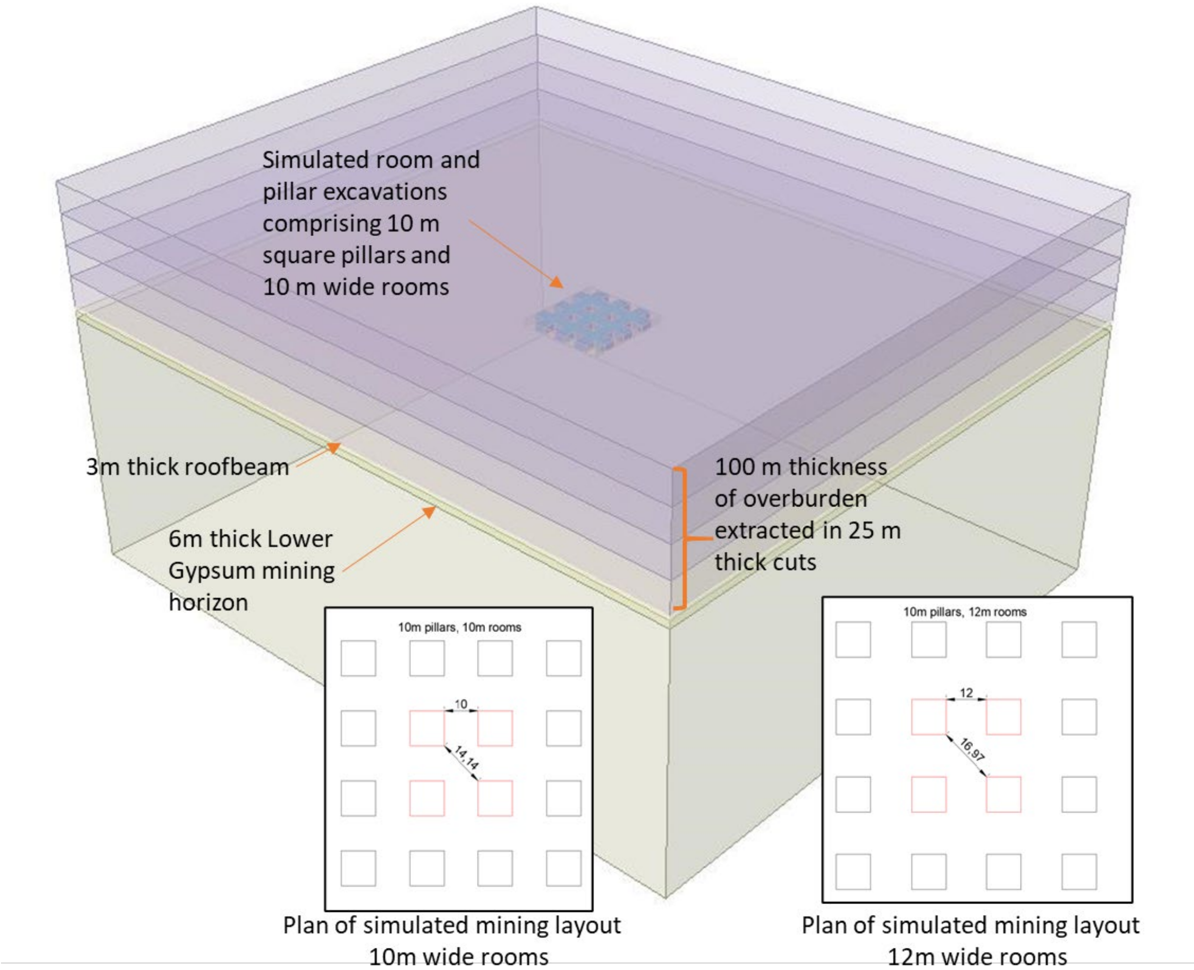

Figure 1: RS3 Roof Beam Stability Assessment Model

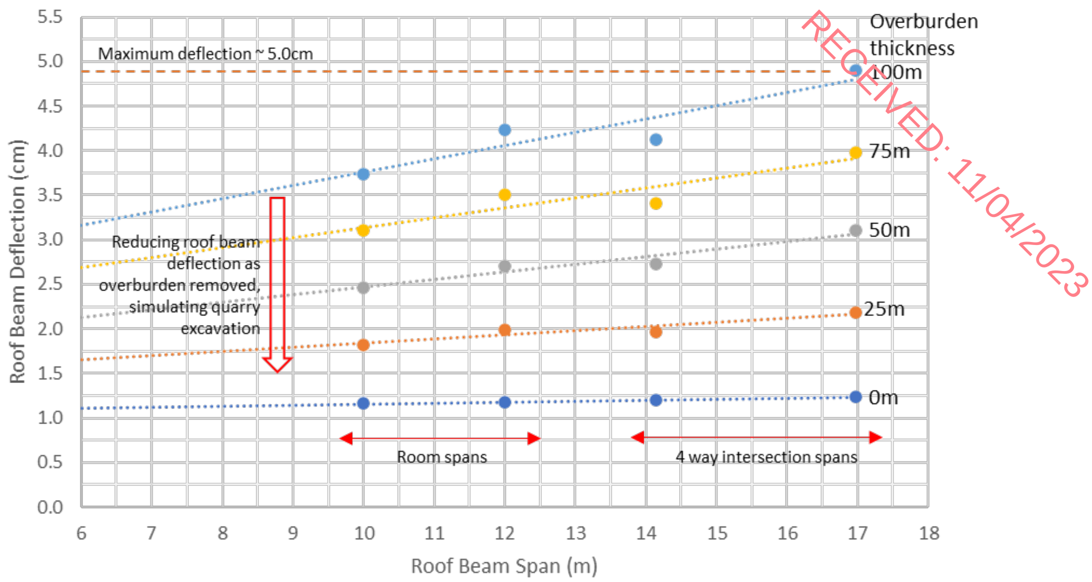

Figure 2: Roof Beam Deflection Results for a 3 m Thick Roof Beam

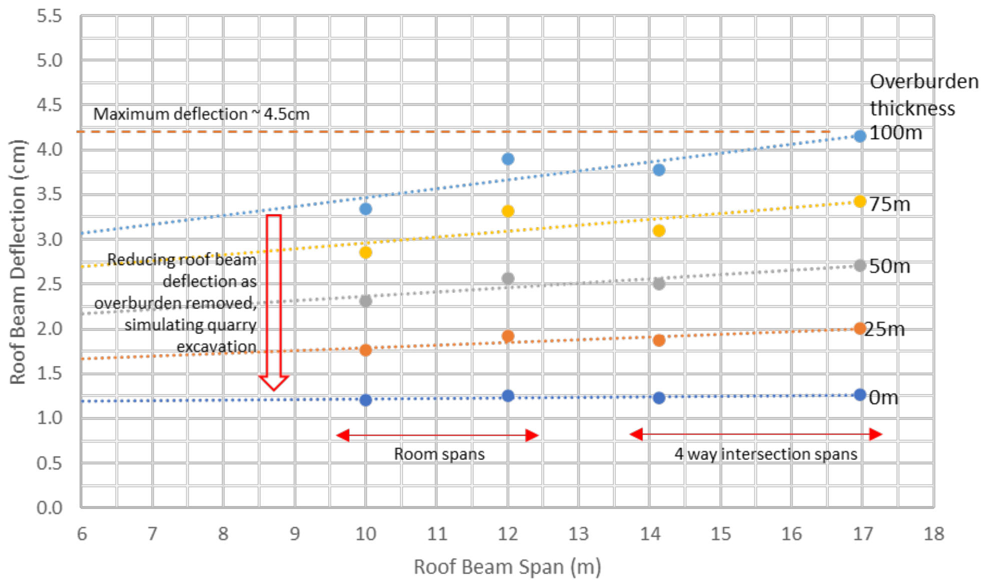

Figure 3: Roof Beam Deflection Results for a 6 m Thick Roof Beam

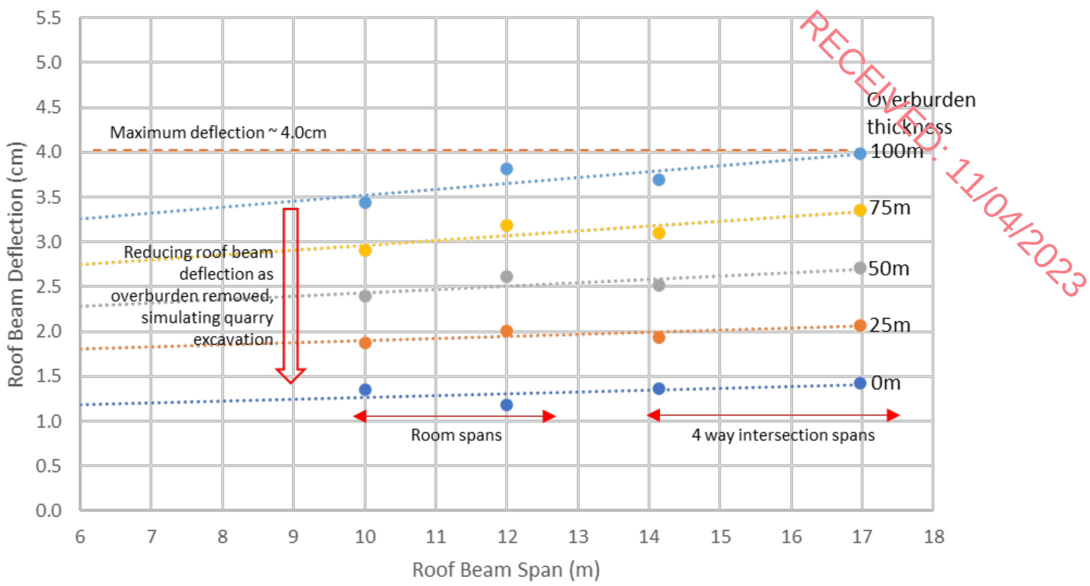

Figure 4: Roof Beam Deflection Results for a 12 m Thick Roof Beam

All of the graphs are of a similar form with roof beam deflection reducing as the overburden is removed. The maximum roof beam deflection also reduces as the roof beam becomes thicker. The maximum simulated deflection for the 3 m thick roof beam is 5 cm. The maximum simulated deflection reduces to 4 cm for the 12 m thick roof beam.

2.3 Discussion and Conclusions

This analysis has demonstrated that as overburden is removed from above the mine workings loading on the Lower Gypsum rock mass that forms the roof beam above the underground workings reduces. This results in elastic rebound of the rock mass which should, in theory, improve the stability of the roof beams above the workings.

What this modelling is not able to simulate is the effect that unloading may have on discrete joints and bedding planes within the beam. Unloading may relieve the stress acting across these features which may in turn reduce their frictional strength. This could allow joint bounded blocks to slip and fall into the underground workings locally reducing the roof beam thickness.

Standard operating procedures when mining above and through underground room and pillar mines in a quarry is to blast and collapse the roof beam to fill the room thus eliminating the risk associated with the presence of the underground workings. As part of the safe working methods for collapsing the roof beam a minimum pit floor pillar thickness will be defined to allow quarry equipment to operate safely above the underground voids along with appropriate personnel and equipment access, drilling and blasting strategies, barricading procedures and general reporting protocols as is currently the case when recovering gypsum from Knocknacran Open-Cast Mine, above the Drummond underground mine workings.

For and on behalf of SRK Consulting (UK) Limited

Neil Marshall,
Corporate Consultant - Geotechnical,
Project Manager
SRK Consulting (UK) Limited

Max Brown,
Principal Consultant - Geotechnical,
Project Director
SRK Consulting (UK) Limited

APPENDIX 7.19
Permanent Solution to Existing Mine workings that go under the Existing Public Road Network - SLR - September 2022

Memorandum

RECEIVED: 11/04/2023

To:	Benson Plunkett	**At:**	Saint-Gobain Mining Ltd
From:	Xander Gwynn	**At:**	SLR Consulting Ltd
Date:	14th September 2022	**Ref:**	
Subject:	**Permanent solution to existing mine workings that go under the existing public road network**		

SLR Consulting (Ireland) Ltd (SLR) has been engaged by Saint-Gobain Mining Ireland Ltd (SGMI), to respond to a request for further information (RFI), Reg. Ref. 22/34, issued by Monaghan County Council (MCC). In particular, with reference to RFI Item 20. q, presented below:

'Permanent Solution to existing mine workings that go under the existing public road network: The applicant has not clearly demonstrated how they propose to address the issue of future road subsidence on the public road network where previous mine workings exist. The applicant must submit comprehensive proposals, including design reports, drawings, and other appropriate design details that demonstrate how the applicant proposes incorporating a permanent solution to the mine workings that go under the public roads as part of their open cast works.'

SGMI proposes to backfill existing mine workings that go under the R179 and L4900 public roads adjacent to the Application Site, and in doing so, provide a permanent solution to the issue of future road subsidence on the public road network where previous mine workings exist.

The locations of the underground workings for backfilling under the R179 and L4900 roads have been identified from mine survey records as shown in Figure 1.

Proposed Methodology for Backfilling under R179 & L4900

On intersecting an opening to the historical Drumgoosat underground mine workings during the development of the Knocknacran Open-Cast Mine, SLR recommend that the following actions be undertaken, dependant on safe working conditions:

- Confirm location of mine opening(s) with respect to historical mine survey plans.
- Conduct an initial Geotechnical Assessment by a competent Geotechnical Engineer on the mine opening(s) uncovered from historical gypsum mining where they intersect with the new open-cast mine excavation.
- Characterise mine opening(s) in terms of stability based on rockmass integrity using the Barton Q or RMR (Rock Mass Rating) systems.
- Based on the Geotechnical Assessment, carryout remediation of 'tunnel' entrances to allow safe access for further Geotechnical Assessment of the access tunnels to the workings under

the public roads. Possible access routes the mine workings under the R179 and L4900 are shown in Figure 2 to 5.

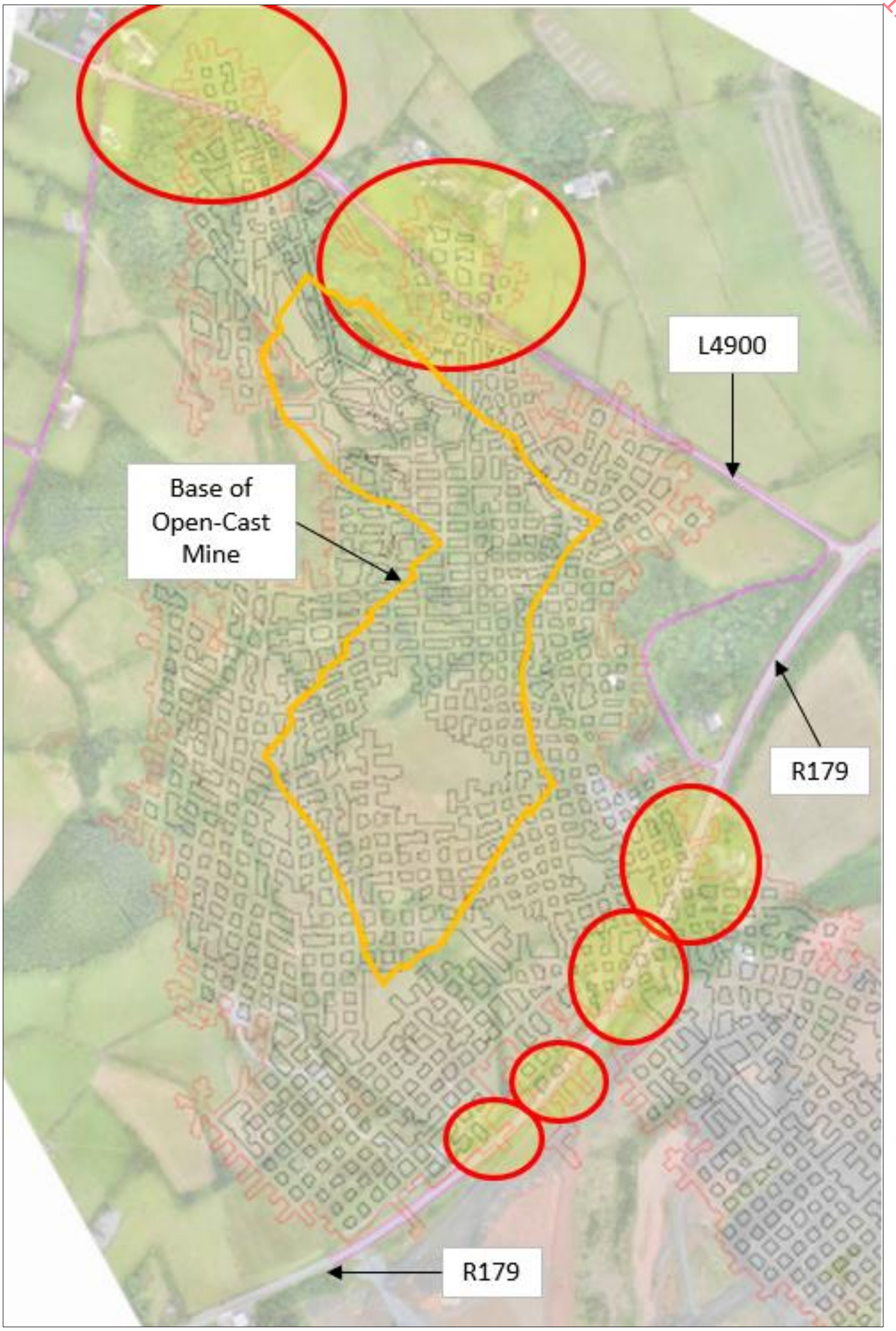

Figure 1: Areas for Backfilling under the R179 and L4900

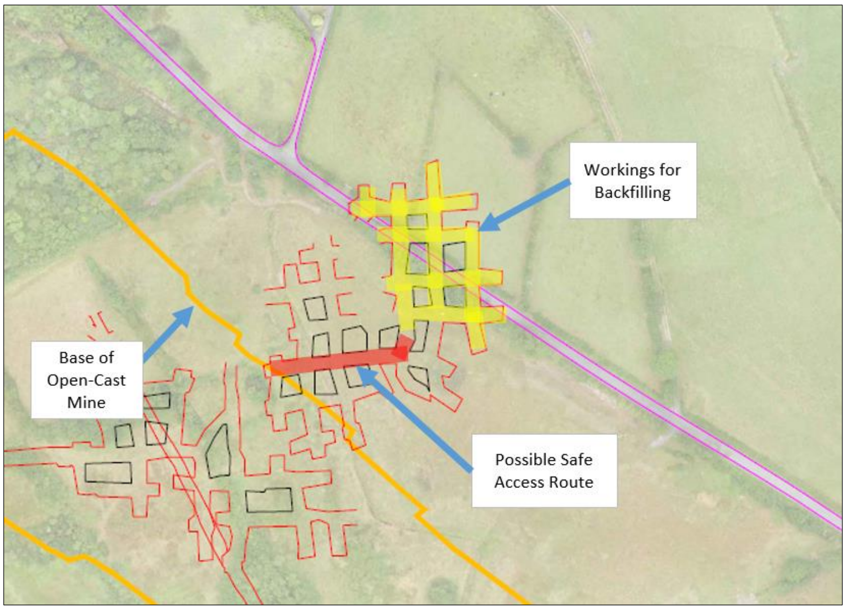

Figure 2: Possible access to the workings under the L4900 - Lower Seam, Upper Horizon

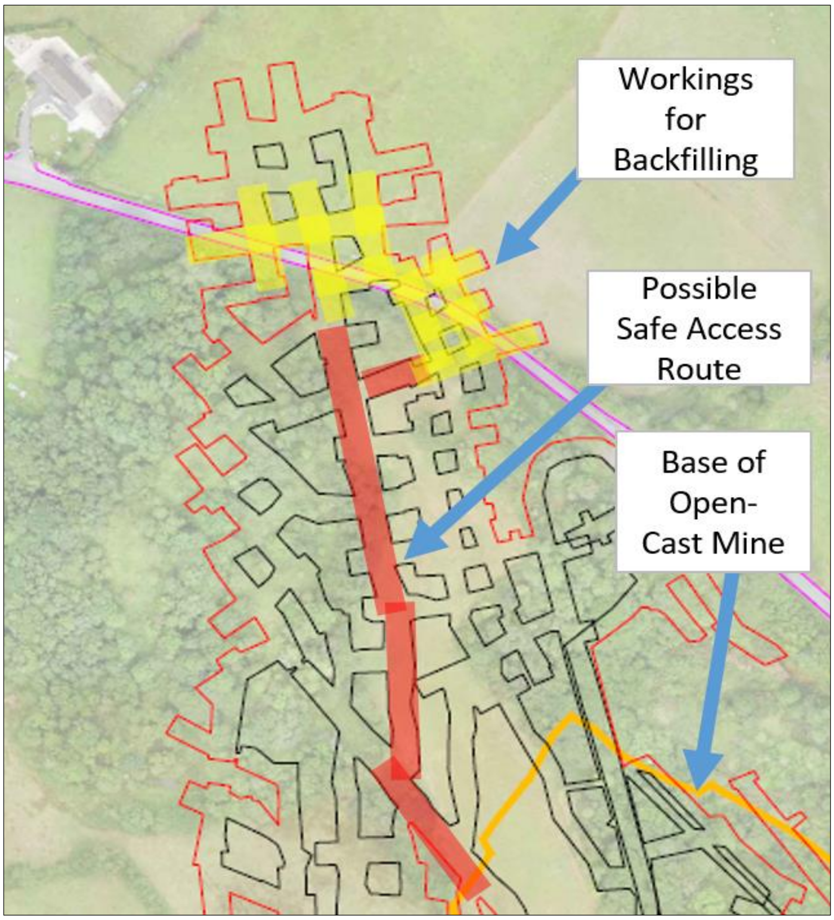

Figure 3: Possible access to the workings under the L4900 - Lower Seam, Upper Horizon

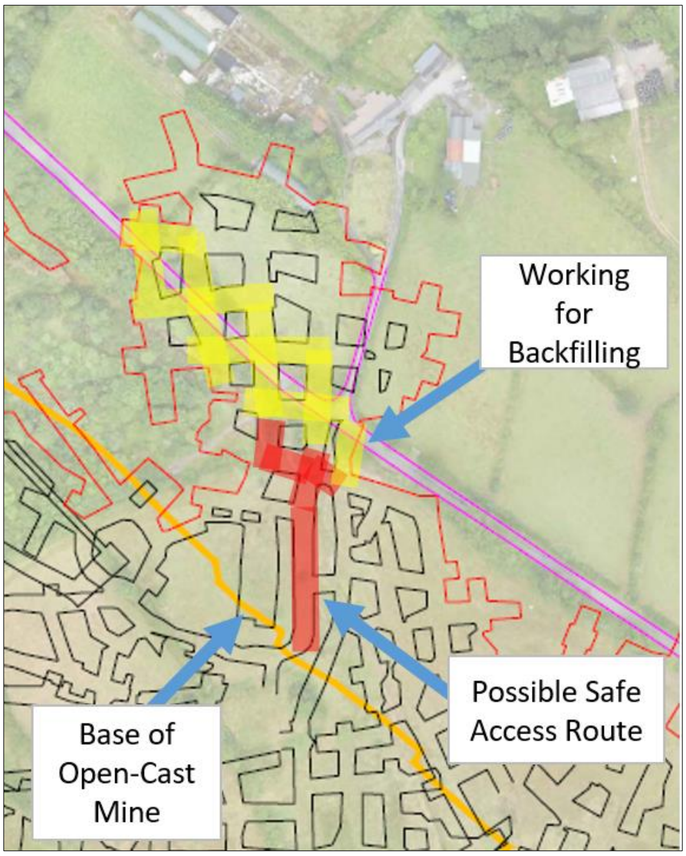

Figure 4: Possible access to the workings under the L4900 - Lower Seam, Upper Horizon

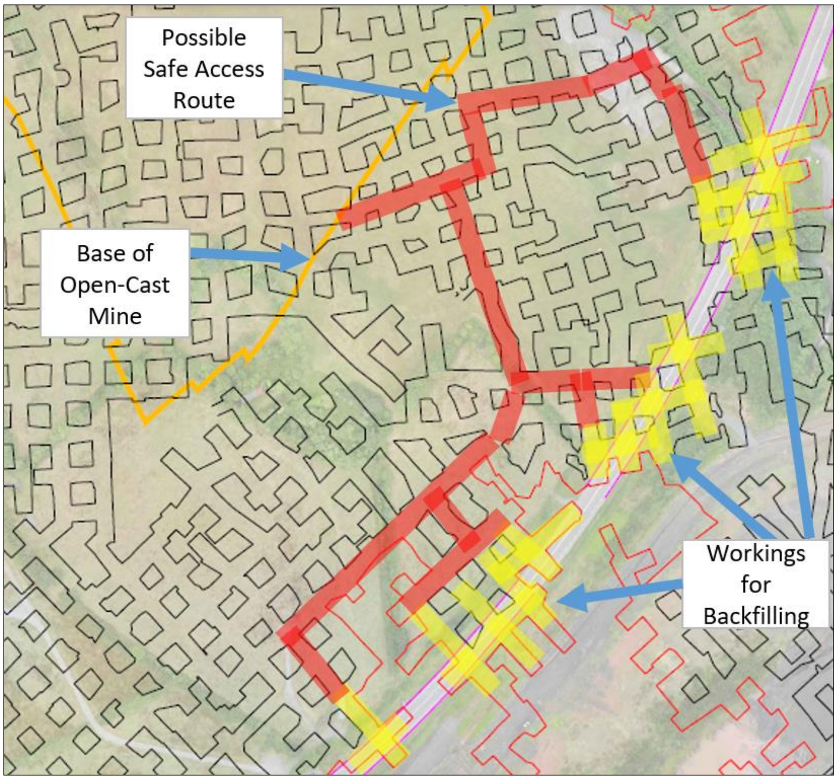

Figure 5: Possible access to the workings under the R179

- Lower Seam, Upper Horizon

- Based on the outcome of the further Geotechnical Assessment(s), conduct ground support remediation works along the length of the tunnels to provide safe access to workings under the roads. The final routes to the areas for backfilling and the areas for backfilling themselves will be confirmed following completion of Geotechnical Assessment(s) and any subsequent remediation works required to make the access tunnels to the areas under the public roads safe.

- Remedial work(s) along the access tunnels to the areas under the R179 and L4900 (and the areas for backfilling) will include a combination of the following, depending on the ground conditions encountered:
    - Scaling (both mechanical and by scaling-bar);
    - Rockbolting without mesh;
    - Installation of mesh with rockbolting.

  Figures 6 and 7 present photographs of example rock bolt and mesh installation, and a tunnel with installed rock bolts and mesh, respectively.

Figure 6: Rock bolt and mesh installation

Figure 7: Example of installed rock bolt and mesh

- Once access to the workings under the roads is made safe and secure, the access tunnels and locations recommended for backfilling will be surveyed (including all 4-way-junctions under the R179 and L4900).

- Buttress walls will then be constructed.

- Buttress wall dimensions and specifications will be determined based on the recommendations of the Geotechnical (and physical) Assessment(s) of the workings. Figure 8 provides a conceptual schematic plan and cross-section for the backfilling of a 4-way junction in the underground workings.

- The buttress walls will be constructed based on recommendations from the Geotechnical Assessments carried out, and will be designed to ensure that backfill will not 'run' or move after it has been emplaced.

- Following construction of buttress walls, rockfill will be placed as backfill in all 4-way-junctions under the R179 and L4900 to provide long-term stability of underground mine workings. Backfill will be in the form of 6" down or similar material, sourced from local quarries (as recommended following the Geotechnical Assessment);
    - Fill material will be placed in lifts and pushed against the walls of the underground workings, pillars and buttresses.
    - Fill will be compacted as it is placed.
    - A final buttress wall will be put in place to contain the backfill material.
    - As "tight" a fill as possible will be achieved. Due to the undulating nature of the roof and the material used for backfilling, there will be small gaps between the backfill and roof. If the roof were to move, it would only be into this small space and very limited movement will be translated into the strata above.

- A photographic record of the works will be made for each location.

- A final topographical survey of the buttress locations will be completed prior to vacating the underground mine workings.

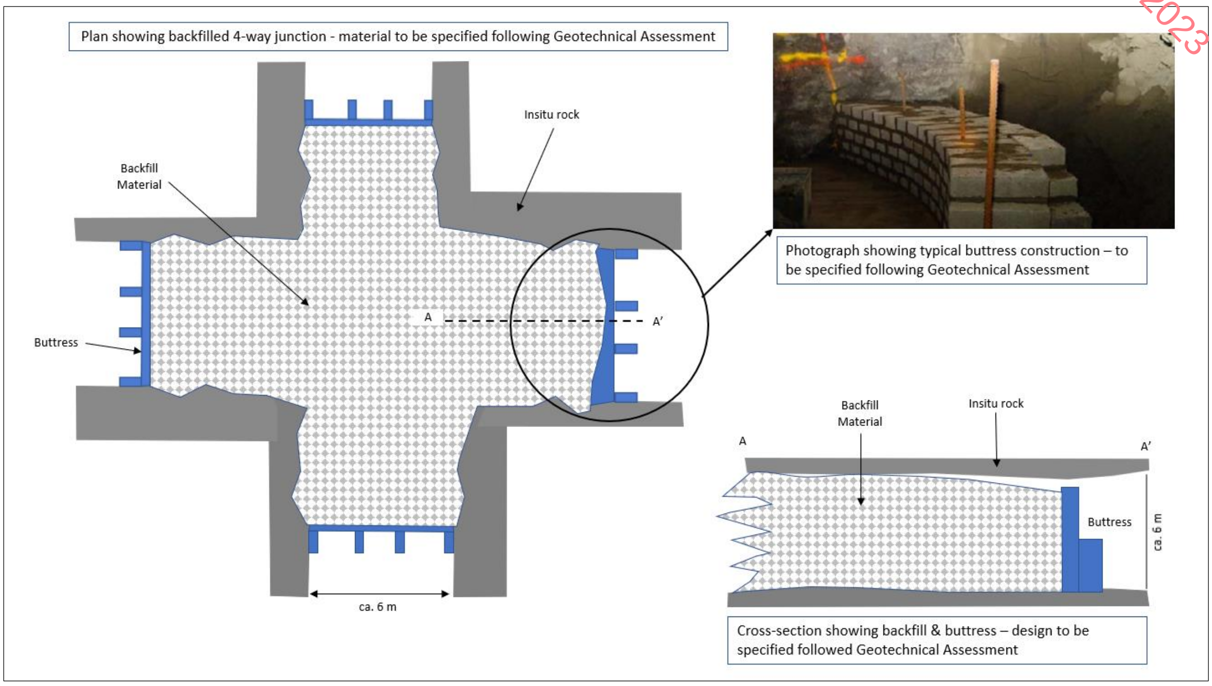

Figure 8: Schematic plan and cross-section for the backfilling of a 4-way junction

Existing ground control monitoring systems of in situ extensometers and surface level monitoring will be maintained and used to monitor underground mine workings under the R179 and L4900 (using the existing TARP (Trigger Action Response Plan)) for a period to be agreed with the Authorities.

Geotechnical Assessments will be carried out by a competent Geotechnical Engineer. Geotechnical Assessment reports will be submitted to the Authorities (including the GSRO) for their agreement prior to any works being carried out.