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AGL CONSULTING
GEOTECHNICAL ENGINEERS

AN BORD PLEANÁLA
04 MAR 2024
LTR DATED ________ FROM Irish Life
Conor O'Donnell
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TECHNICAL NOTE

To: **Owen Bourke, Urban Solutions**

From: Conor O'Donnell

Re: **Project Metrolink - Phase IIa Building Damage Assessment for Cadenza Building on Earlsfort Terrace**

Date: 1/3/2024 **Ref: 24-115-TN001 (Rev.1)**

1.0 INTRODUCTION

This technical note presents the results of the Phase 2a assessment of building damage that could occur due to tunnelling-induced ground movements during the construction of the Metrolink Tunnel under the Irish Life Cadenza Building on Earlsfort Terrace in Dublin City Centre.

The assessment has been carried out in accordance with the criteria and methodology set out for a refined Phase 2a assessment of subsidence damage in Section 4 of the Building Damage Report ("the BDR") produced by Jacobs/IDOM, which is included as Appendix A5.17 of the Environmental Impact Assessment Report ("the EIAR").

The analysis has been carried out to assess the potential tunnelling settlements and associated building damage that could occur at the Cadenza building based on the specific ground conditions, tunnel geometry and building characteristics at the site.

The tunnel rises slightly from north to south across the width of the building. Therefore, analyses have been carried out at 3 No. locations across the width of the building: on the north side, adjacent to the Arthur Cox building at Ch. 18+995, within the interior of the building at Ch.19+010, and where the tunnel passes under the east façade at Ch. 19+025, where the building faces onto the Earl Court apartments.

At Ch. 18+995 and 19+010 the damage to the basement floor slab and the interior reinforced concrete structure of the building, respectively, have been assessed based on the settlements that could occur at the subgrade level for the basement floor slab and foundations.

At Ch. 19+025 the potential damage to the building façade has been assessed based on the tunnelling settlements that could occur at the toe level of the perimeter secant pile wall, which supports the façade and internal structure. The tunnel is shallowest on this side of the structure.

Analyses have been carried out for the design tunnel profile shown on the drawings in the EIAR, and for a raised and lowered profile within the proposed *modified* vertical Limits of Deviation, which are up to **1.0m** above and **10.0m** below the design profile.

2.0 STATEMENT OF EXPERTISE

Conor O'Donnell is the Senior Geotechnical Consultant and Managing Director of AGL Consulting with more than 25 years' experience as a Geotechnical Engineer in Ireland and the United States. He is a Chartered Engineer with Bachelors Degree in Civil, Structural and Environmental Engineering from Trinity College Dublin, and a Masters Degree in Geotechnical Engineering and Structural Mechanics from Cornell University, Ithaca, NY.

Prior to 2001, Conor worked for Mueser Rutledge Consulting Engineers, a specialist geotechnical engineering consulting firm in New York City. At MRCE, Conor specialised in the geotechnical design of foundations, excavation support systems and ground improvement schemes, including deep soil mixing, grouting and ground freezing for tunnelling projects. He worked on a number of landmark tunneling and underground mass transit projects, including the NATM tunnel for the MBTA South Boston Transitway under Russia Wharf, and Contract C09A4 of the Boston Central Artery Project, which involved tunnel jacking under the railway lines at the approach to South Station in Boston, Massachusetts.

At AGL Conor has been involved in a number of major civil, infrastructure and commercial building projects across Ireland at planning, detailed design and construction stage. He was the lead geotechnical consultant for the detailed ground investigation and preliminary design of the basement excavation and perimeter secant piling works for the Dublin Central development off O'Connell St. in Dublin City Centre. He prepared the hydrogeological impact assessment for the project and a report on ground movements related to basement excavation and dewatering. He was also the geotechnical specialist adviser to An Bord Pleanála for the oral hearing into the planning application for the onshore Corrib Gas Pipeline, which included a long microtunnel crossing of Sruwaddacon Bay in Co. Mayo.

Mr. O'Donnell has extensive experience in assessing ground movements related to tunnelling, micro-tunnelling and underground excavation works. His postgraduate studies and Masters thesis involved forensic analyses and numerical modelling of ground movements adjacent to deep excavations for a cut and cover section of the Boston Central Artery tunnel project. Related papers were subsequently published in the ASCE Journal of Geotechnical and Geoenvironmental Engineering which were co-authored by Prof. Tom O'Rourke, who is a leading international expert on ground movements and related building damage adjacent to excavations. In Ireland Mr. O'Donnell was the geotechnical consultant for a large-diameter micro-tunnelling section of the North Docklands Sewerage Scheme in Dublin Port. He also advised on the specification and scope of work for the ground investigation for the Blanchardstown Regional Drainage Scheme, which involved large diameter microtunnelling and deep caissons. Mr. O'Donnell carried out numerical modelling to assess the impact of microtunnelling behind the masonry abutments of Sarsfield Bridge for the Limerick Main Drainage project.

AGL Consulting have been involved in most of the large tunnelling projects in Ireland. We were geotechnical advisers to Dublin City Council on temporary works designs involving soil nailing for the cut and cover section of the Dublin Port Tunnel along the M1 in Swords. We have also developed a 3D model of ground and groundwater conditions along the alignment of the tunnel in AutoCAD Civil 3D for Transport Infrastructure Ireland, which collates all the available ground investigation information on the project. We recently used this ground model to assess the impact of multi-story apartment buildings on the tunnel at Hartfield Place in Swords by advanced 3D finite element modelling.

AGL were also temporary works designers for the casting basin of the immersed tube sections of the Limerick Tunnel, and for the launch shaft and reception pit for the Tunnel Boring Machine (TBM) used on the 4.0m dia. Corrib Gas Pipeline tunnel in Co. Mayo.

3.0 ANALYSIS PROFILES FOR CADENZA BUILDING

The proposed alignment and vertical profile of the tunnel are shown on the following drawings in Book 2 of the Railway Order (RO) Alignment Details (Area ML304 to ML307 - Balbutcher Lane to Ranelagh Road):

- ML1-JAI-ARD-ROUT_XX-DR-Y-03095: Metrolink General Arrangement – Hatch Street Lower to Grande Parade;
- ML1-JAI-ARD-ROUT_XX-DR-Y-01018: Metrolink Alignment – Long Section 18

Copies of the drawings are included in Appendix A. Figure 3-1 shows an excerpt from the general arrangement alignment drawing with the location of the Cadenza building highlighted. Figure 3-2 shows an excerpt from the long-section profile drawing which shows the design tunnel profile and top of rail level under the building. The chainage of the tunnel is not shown on the alignment drawings. Therefore, it has been determined from the long-section drawings, which identify where the tunnel passes under Adelaide Road (Ch. 19+045).

Based on these drawings, the tunnel crosses under the Cadenza Building for approximately 50m between Ch. 18+995 and 19+045, which is measured from north to south across the width of the building. The top of rail level in the tunnel rises by **0.55m** from **-10.28mOD** at Ch. 18+995 to **-9.73mOD** at Ch. 19+045. Ground level on Adelaide Rd. is at **+14.10mOD (Malin)**, which is **23.8m** above the rail level in the tunnel on this side of the building.

Figure 3-3 to Figure 3-8 show details of the building basement, foundations, sub-structure, perimeter secant pile wall and anti-flotation anchors that are relevant to the Building Damage Assessment. A copy of the drawings is included in Appendix A.

Figure 3-3 shows the proposed tunnel alignment superimposed on a plan drawing of the basement and perimeter secant pile wall of the building. The Building Damage Assessment has been carried out at 3 No. profiles across the width of the building, as shown in Figure 3-3:

- Section 1 @ Ch.18+995: North side of building, adjacent to Arthur Cox Building
- Section 2 @ Ch. 19+010: Interior of building
- Section 3 @ Ch. 19+025: Building east façade facing Earls Court apartments

Figure 3-3 shows a plan layout of the basement that identifies the location of the following significant components of the substructure and foundations along the alignment of the tunnel:

- The integral pad foundations for the interior columns;
- The perimeter secant pile wall around the perimeter of the basement;
- The anti-flotation anchors for the rainwater harvesting tank in the basement; and
- Structural stair core #02 on the east side of the building at basement level.

Along the north side of the basement the perimeter secant pile wall ties into a pre-existing secant pile wall that was constructed for the adjacent Arthur Cox Building, which has a toe level of +0.65mOD.

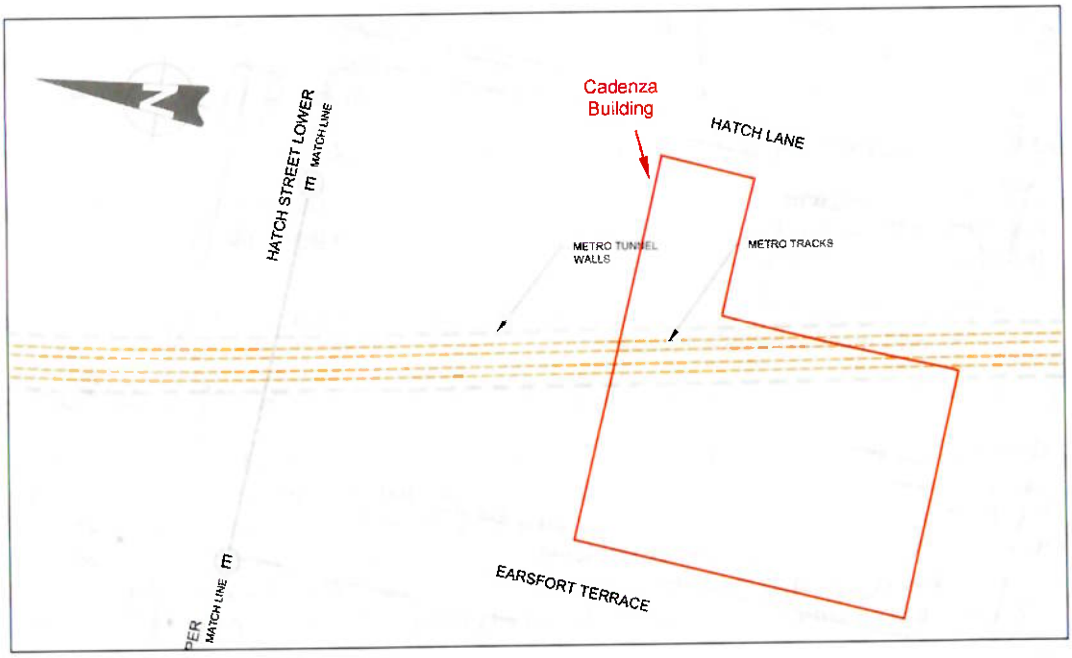

Figure 3-1 – Design tunnel alignment from RO Alignment Drawings

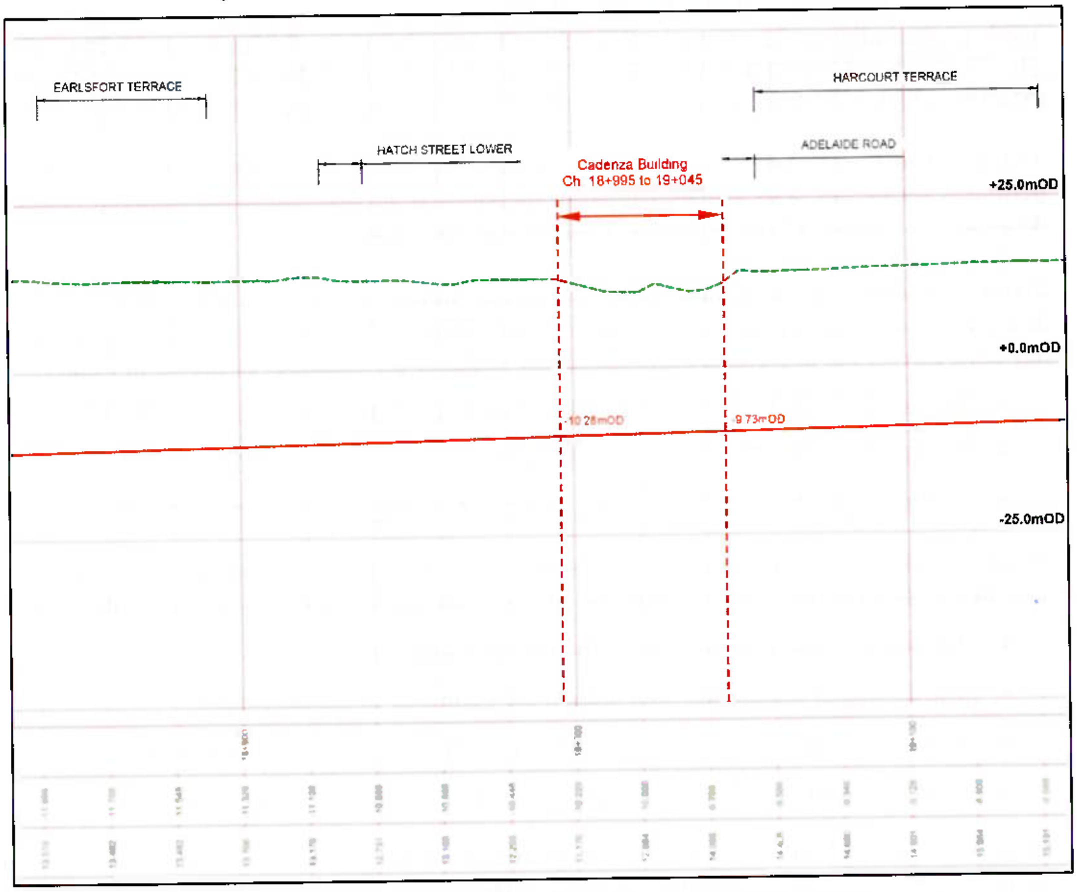

Figure 3-2 Design tunnel profile (top of rail level) from RO Long-Section Drawings

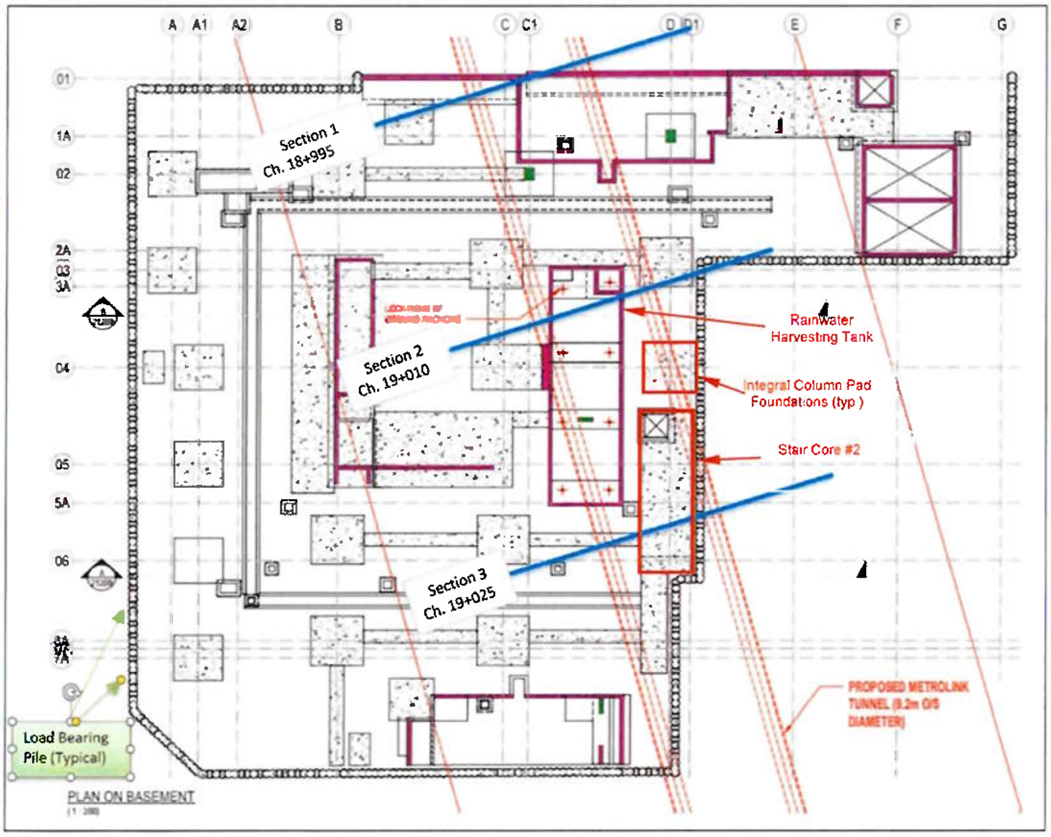

Figure 3-3 Tunnel alignment superimposed on plan of the basement and perimeter secant pile wall.

Figure 3-4 shows the layout of the perimeter secant pile wall. A profile of the wall on the east side of the building at Section 3 (Ch. 19+025) is shown on the design drawing in Figure 3-5 [Wall Type B-1]. Where the tunnel passes under the secant pile wall:

- The wall is comprised of 640mm diameter reinforced grade C30/37 concrete structural piles at 1.2m centres, socketed into the Limestone bedrock to a toe level of +2.50mOD;
- The space between the structural piles is filled with 900mm diameter unreinforced grade C8/10 concrete CFA (continuous flight auger) piles which were constructed to refusal at the top of rock at about +6.6mOD, which is approximately 4.1m above the toe of the structural piles;
- The location of the structural piles is highlighted by the coloured circles in Figure 3-4;
- The structural piles are supporting the façade and internal reinforced concrete structural frame of the building and have been designed for a characteristic (unfactored – SLS) load of **1,025kN**.

Where the structural piles are directly over the tunnel they will apply a concentrated load onto the rock over the tunnel bore.

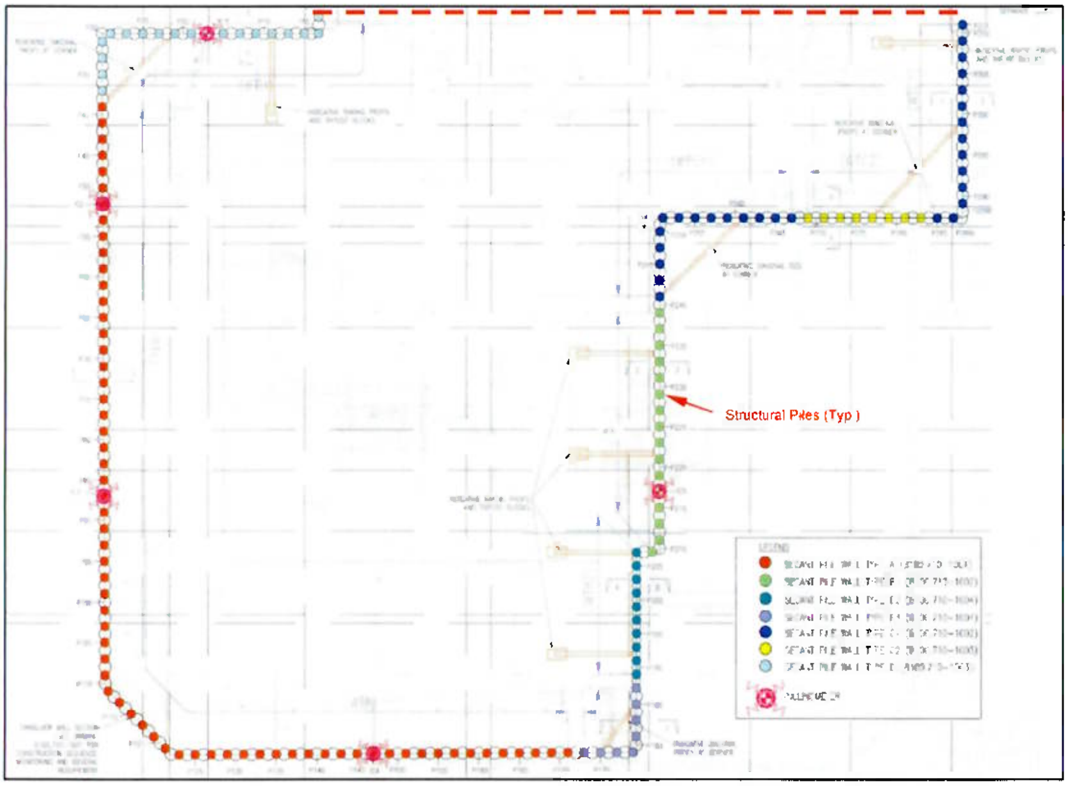

Figure 3-4 Plan of perimeter secant pile wall for the basement

Figure 3-3 identifies a typical pad foundation for the interior columns over the tunnel alignment. A profile of the foundation and basement floor slab is shown in Figure 3-6.

- The foundation is a 3.5m x 3.5m x 1.2m deep Grade C40/50 reinforced concrete pad that has been constructed as a thickened integral section of the 300mm thick basement floor slab;
- Formation level for the floor slab is at +6.00mOD.
- Formation level for the pad foundation is 1.2m lower at +4.80mOD.
- The pad foundations are supporting the internal concrete frame for the building, which has spans up to 15.0m with post-tensioned concrete floor slabs.
- The foundations have been designed to be supported on rock at an allowable bearing pressure up to 1,000 kPa.

Figure 3-3 also identifies the location of the rainwater harvesting tank in the basement of the building. The tank extends approximately 2m below formation level for the basement floor slab, and is up to 4.5m below groundwater level on the site (+8.42mOD). Therefore, the tank is supported by 8 No. anti-flotation anchors to prevent hydrostatic uplift of the tank when it is empty, as shown on Figure 3-7 (A.01 to A.08). There are also 2 No. anchors in another tank on the south side of the basement (A.09 & A.10).

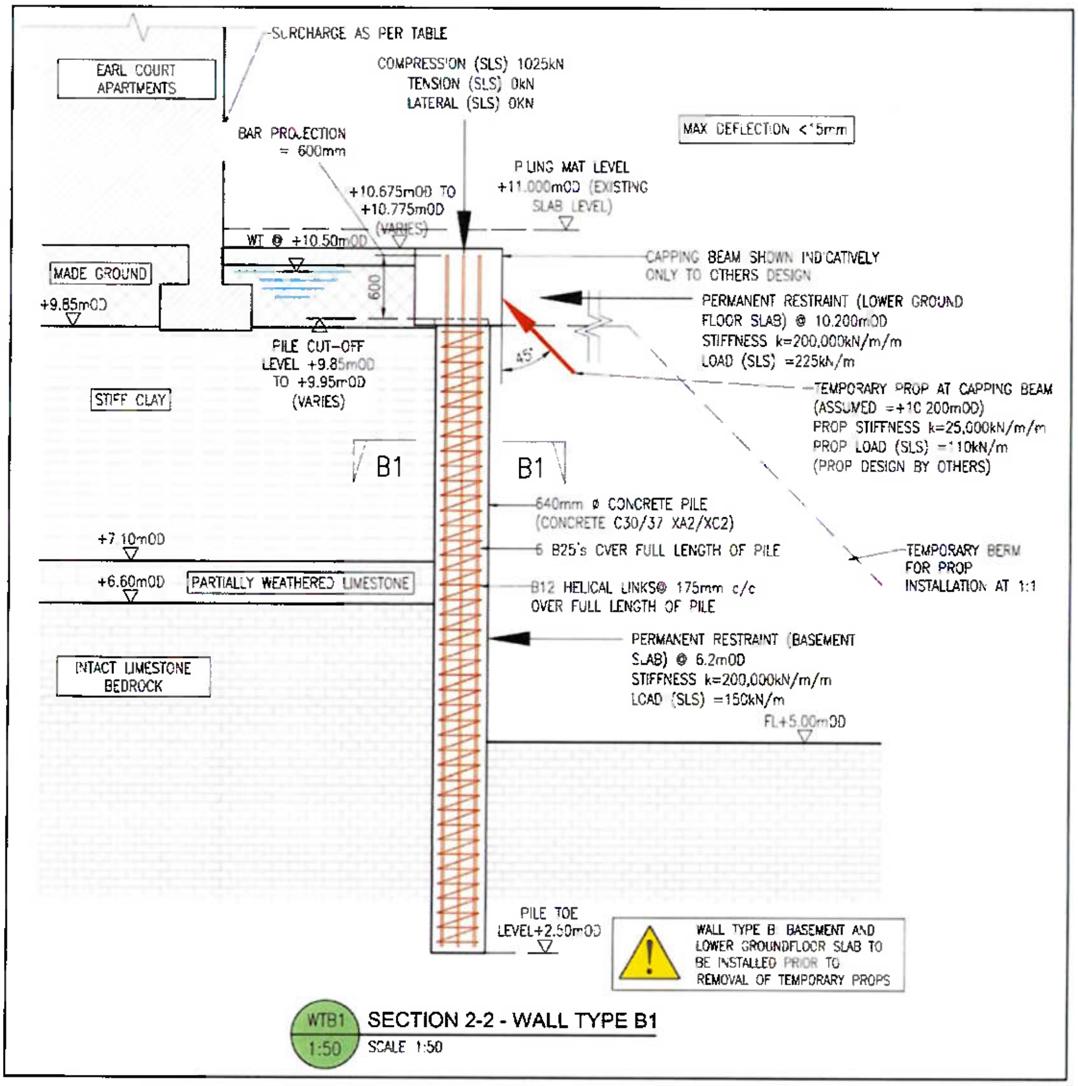

Figure 3-5 Profile of Secant Pile Wall along east side of building

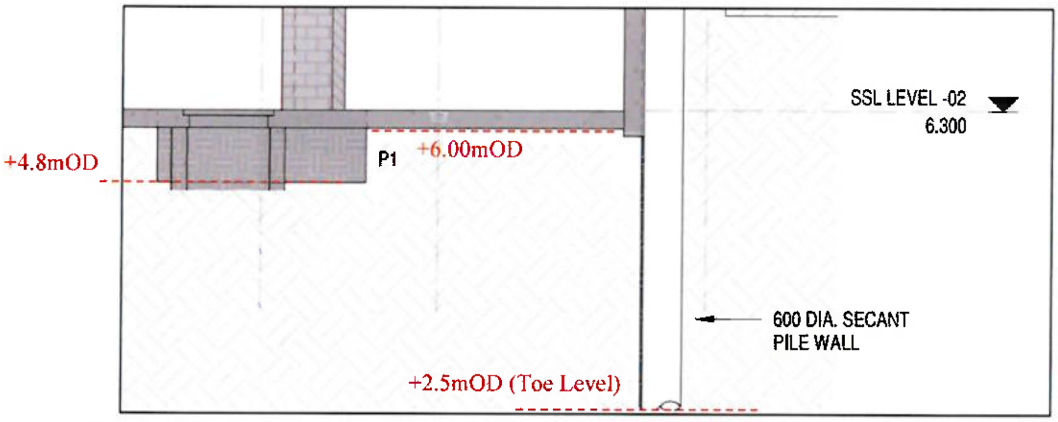

Figure 3-6 Profile of basement floor slab, 1.2m deep P1 pad foundations and 640mm diameter perimeter secant pile wall

Details of the anti-flotation anchors are shown on Figure 3-7.

- The anchors are comprised of 8.0m long 50mm diameter GEWI Plus Dywidag Steel Threadbars with double-corrosion protection grouted into a 140mm diameter drillhole.
- Formation level for the rainwater harvesting tank is at +3.95mOD. The anchors extend 8.0m below this level to a toe level of -4.05mOD.
- This is 0.9m below the crown of the TBM tunnel bore at this location (Ch. 19+020), as illustrated on Figure 3-7, which means that the TBM will hit the end of the anchors at the design profile and by up to 1.9m at the upward Limit of Deviation.
- The anchors have been designed for a characteristic (unfactored) uplift force of 750kN.

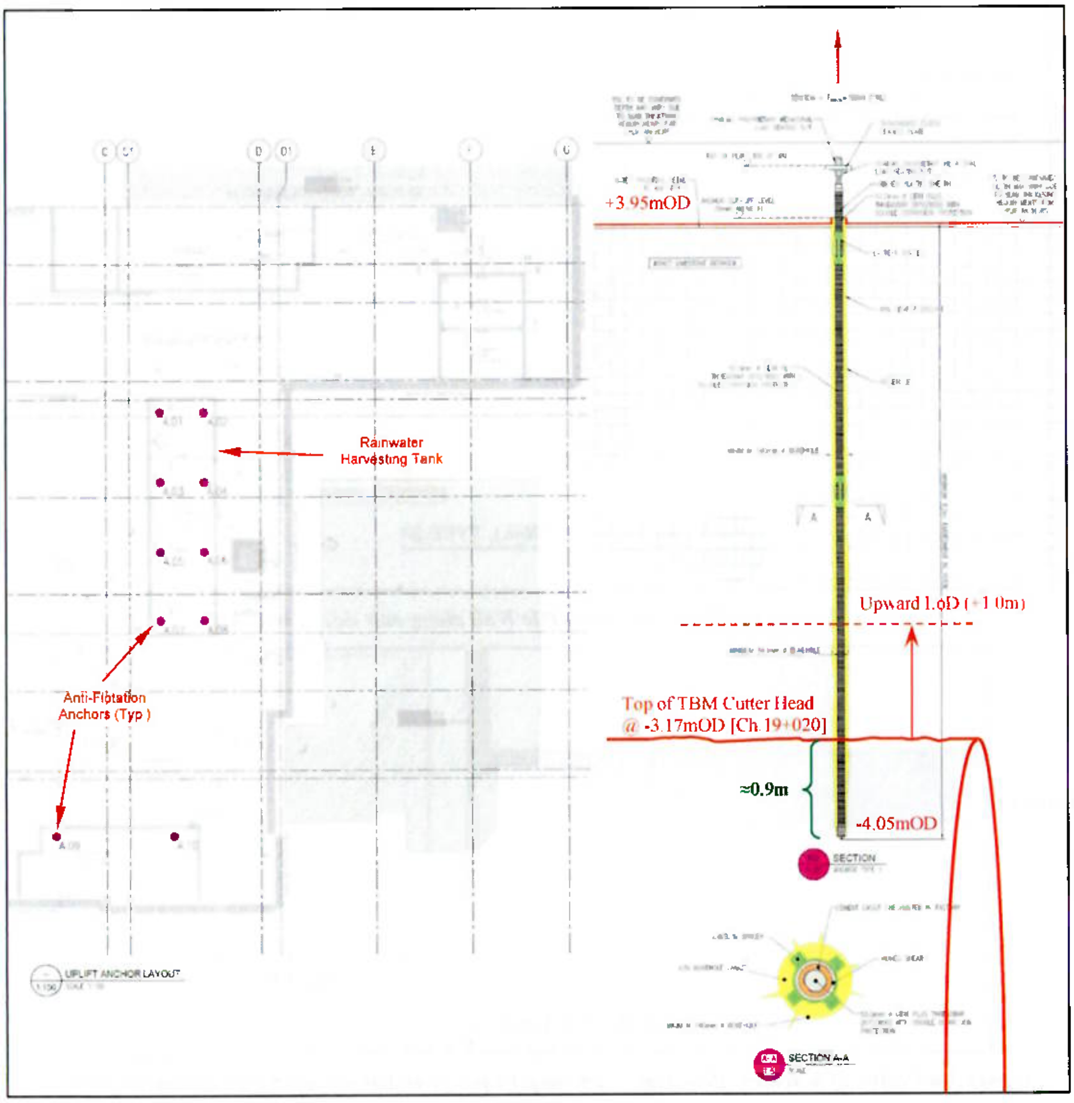

Figure 3-7 Profile of anti-flotation anchors for rainwater harvesting tank

Figure 3-8 shows a typical profile of the tunnel. The following is a summary of relevant information on the tunnel geometry from Section 2.1 of the BDR (copy in Appendix A):

- Tunnel Diameter – Internal = 8.50m
- Tunnel Diameter – External = 9.20m (350mm thick concrete lining segments)
- Diameter of TBM Cutter Head = 9.53m (165mm annular clearance outside segments)
- Height of tunnel axis above Top of Rail (ToR) level = **2.07m**

The calculation of tunnelling settlements for the Building Damage Assessment is based on the **9.53m** outer diameter of the TBM cutter head.

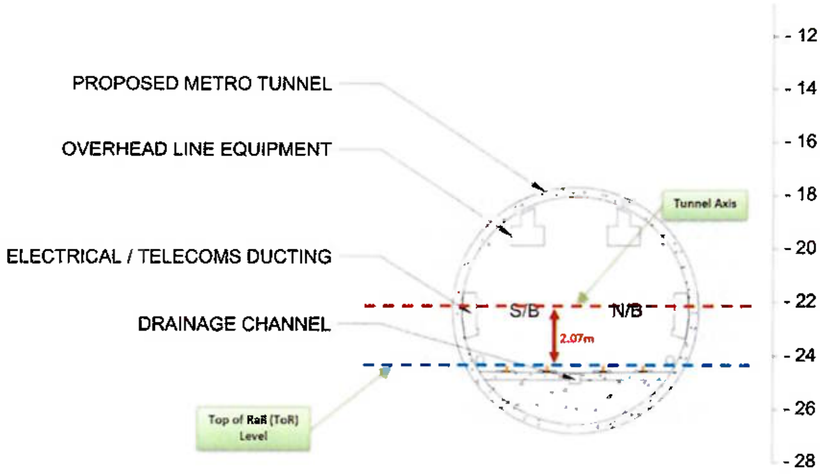

Figure 3-8 – Typical profile of the tunnel

The Limits of Deviation (LoD) on the vertical profile that are proposed in Article 6 (Deviation) of Part 2 of the Draft Railway Order [6.1(d)] are **5.0m upwards** and **10.0m downwards** from the design profile shown on the drawings. Raising or lowering the tunnel profile could have a significant impact on the building and has therefore been considered in our assessment.

During the Oral Hearing for the project, in response to submissions received during the statutory consultation process TII proposed to amend the upward LoD for the tunnel in the Railway Order to **1.0m**, due to concerns of potential increased impact on buildings if the original upward LoD were allowed (Ref. Expert Witness Statement of Ronan Hallissey p7). Therefore, we have used this amended value in our Building Damage Assessment.

In assessing the impact of lowering the tunnel profile we have considered a level that is **5.0m** below the proposed design level.

The LoD proposed in the Draft RO for the horizontal alignment is **±15.0m** from the design alignment. This has been considered qualitatively in our assessment insofar as it could impact the anti-flotation anchors and the extent of the building that would be impacted by the tunnel.

The Building Damage Assessment has been carried out for **9 No. cases** as follows:

- **Section 1 (Ch.18+995):** Below the basement slab and side walls on the north side of the building to assess the potential impact of tunnelling on the basement floor slab due to tunnelling induced settlements at slab subgrade level (+6.00mOD):
    - Case 3A: for the design tunnel profile
    - Case 3B: for a raised tunnel profile (**+1.0m** from design level*)*
    - Case 3C: for the lowered tunnel profile (**-5.0m** from design level).

- **Section 2 (Ch.19+010):** Below the interior of the building to assess the potential impact of tunnelling on the internal reinforced concrete structure due to tunnelling induced settlements at pad foundation level (+4.80mOD) *(see Note 1)*:
    - Case 2A: for the design tunnel profile
    - Case 2B: for a raised tunnel profile (+1.0m from design level)
    - Case 2C: for the lowered tunnel profile (-5.0m from design level).

  Note:

  1. *the interaction between the TBM and the anti-flotation anchors at this location has been considered separately. The building damage assessment considers the impact of tunnelling induced foundation settlements on the main structure.*

- **Section 3 (Ch.19+025):** Below the perimeter secant pile wall (Type B1) to assess the potential impact of tunnelling on the basement walls and building façade based on tunnelling settlements at pile toe level (+2.50mOD):
    - Case 1A: for the design tunnel profile
    - Case 1B: for a raised tunnel profile (**+1.0m** from design level)
    - Case 1C: for the lowered tunnel profile (**-5.0m** from design level).

Table 3-1 Profile geometry and levels at each analysis section for the design tunnel profile in the EIAR

	Section 1 Ch. 18+995	**Section 2 Ch. 19+010**	**Section 3 Ch. 19+025**
Top of Rail (ToR) Level (mOD)	-10.28	-10.12	-9.95
Tunnel Axis Level (mOD)	-8.21	-8.05	-7.88
Foundation Level (mOD)	+6.00 (Basement Slab Subgrade Level)	+4.80 (Foundation Subgrade Level)	+2.50 (Pile Toe Level)
Depth to Tunnel Axis from Foundation Level, z_o (m)	14.21	12.85	10.38
Tunnel Crown Level (TBM Cutter Head) (mOD)	-3.45	-3.29	-3.12
Clearance to Foundation Subgrade from Tunnel Crown (m)	9.45	8.09	5.62

4.0 GROUND CONDITIONS

4.1 Interpretation of Ground Conditions at the Cadenza Building in the EIAR

Figure 4-1shows the location of the boreholes that were used by Jacobs/IDOM to interpret the ground conditions along the tunnel in the vicinity of the Cadenza Buildling. This is from Figure 20.6 (Sheet 7 of 8) in Chapter 20 of the EIAR (Soils & Geology).

The corresponding interpreted geological cross-section at this location from Appendix A20.9 of the EIAR is shown in Figure 4-2. The outline of the Cadenza Building, basement and perimeter secant pile wall has been added to the profile over the tunnel.

The following is a summary of the relevant information on the assessment of the ground and groundwater conditions at the Cadenza Building in the EIAR:

- The most relevant boreholes used for the interpretation of the ground and groundwater conditions in the area are NBH221 and NBH93. Borehole NBH93 is located at the building, as shown on Figure 4-1.
- None of these boreholes are identified on the geological cross section in Figure 4-2 .
- None of the logs for the investigation points shown on the location plans or geological sections were included in the EIAR so it is not possible to verify the ground conditions interpreted by Jacobs/IDOM.
- The interpreted geological section in Figure 4-2 would indicate that the subgrade for the basement floor slab at +6.00mOD is in the glacial till deposits of Boulder Clay.
- The profile would also indicate that the toe of the perimeter secant pile wall, at +2.50mOD, is embedded into the Sand & Gravel layer at the base of the Glacial Till and does not penetrate into the underlying Weathered Rock or Limestone bedrock.
- The top of the Limestone Rock is shown to be at about 0.0mOD, which is approximately 6.0m below the basement subgrade level.
- The tunnel profile is shown to be in the Limestone bedrock under the building.
- Based on the interpreted geological section in Figure 4-2 Jacobs/IDOM have assumed the following ground loss parameters for the section of the tunnel under the site of the Cadenza Building in the Building Damage Report (BDR):
    - **1.50%** between Ch.18+980 and 19+100, where the tunnel is in rock but the cover of rock over the tunnel is <0.5D.
    - Trough Width Parameter, **K = 0.40**

  These parameters will be discussed in more detail in Section 5.2.
- A groundwater level of +11.0mOD (≈2.50mBGL) is shown in BH-RC01 at Ch. 18+800, approximately 200m north of the Cadenza Building.

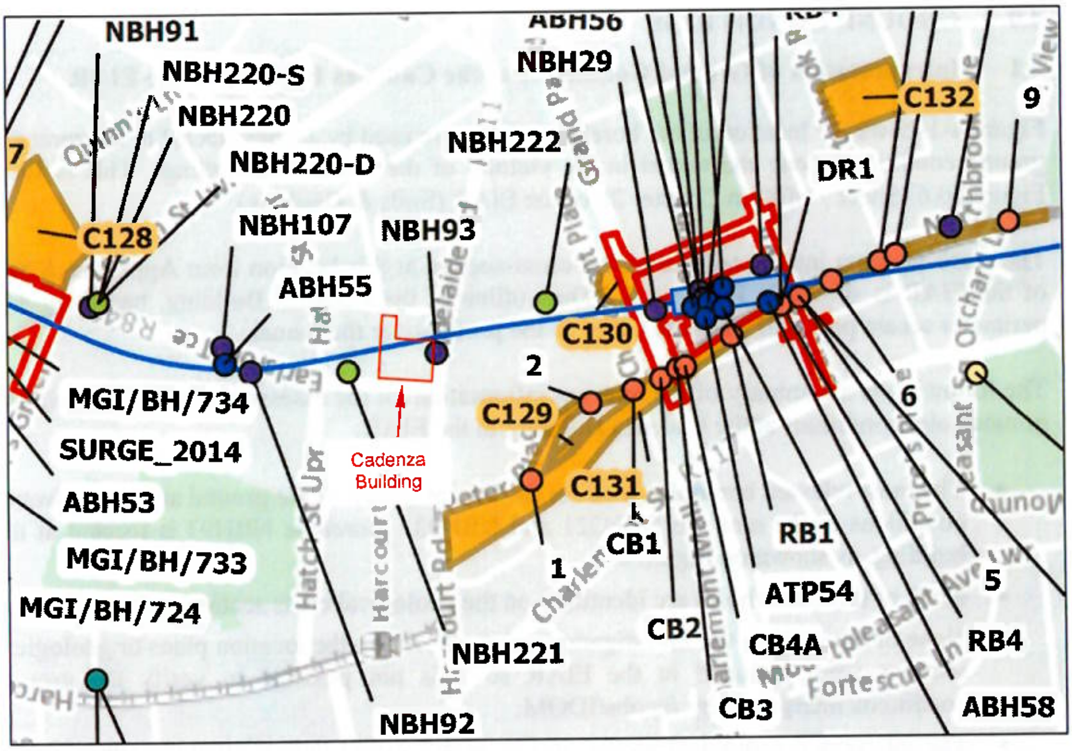

Figure 4-1 Site Investigation Location Plan [from Figure 20.6 (Sheet 7 of 8) in Chapter 20 of the EIAR]

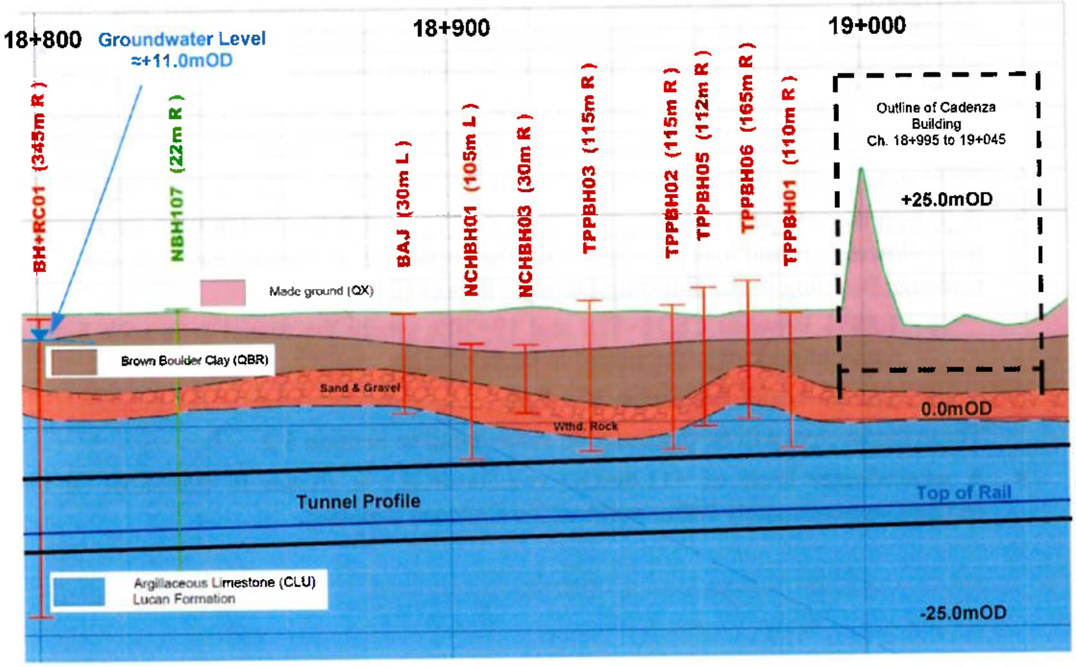

Figure 4-2 Geological Cross-Section [Sheet 26 of 28 from Appendix A20.9 to Chapter 20 of the EIAR]

4.2 Site Specific Site Investigation (SI) Data

Figure 4-3 shows the location plan for the site investigation that was carried out at the site of the Cadenza Building in 2018 for Urban Solutions. The investigation was comprised of 3 No. rotary coreholes (BH-1 to BH-3) at the locations shown on Figure 4-3. Copies of the logs are included in Appendix B.

The coreholes were advanced through Made Ground and stiff to very stiff Boulder Clay and 5m into the underlying Limstone bedrock. The top of the competent Limestone rock was encountered at +6.32mOD in RC-1 on the north side of the building and at the higher level of +7.60mOD in RC-3 on the south side of the building. Table 4-1 gives a summary of the rock levels in the coreholes.

Table 4-1 – Summary of rock levels in rotary coreholes

	Ground Level (mOD)	Top of Wthd. Rock (mOD)	Top of Competent Rock (mOD)	End of Corehole (mOD)
BH-1 (RC-Geobore S)	+12.22mOD	-	+6.32mOD	+1.52mOD
BH-2 (RC-Geobore S)	+11.24mOD	-	+7.34mOD	+2.64mOD
BH-3 (RC-Geobore S)	+10.80mOD	-	+7.60mOD	+2.70mOD

The rock is classified on the logs as medium strong, dark grey, fine-grained, medium to thinly bedded LIMESTONE interbedded with dark grey/black laminated Mudstone seams. This is consistent with the typical characteristics of the Calp Limestone of the Lucan Formation.

The rock is described as partially weathered to unweathered. Total core recovery (TCR) in the competent rock was 100% and the RQD (Rock Quality Designation) ranged from 20-58%. Bedding in the rock was gently dipping (0-15) and medium to closely or very closely spaced. The surfaces were planar smooth with some clay infill.

The rock strength tests indicate that the compressive strength of the rock ranges from 55-120MPa, which is strong to very strong.

The coreholes were terminated in competent rock at levels between +1.5mOD and +2.7mOD, which is below the basement floor slab (+4.8mOD) and generally below the toe level for the perimeter secant pile wall (+2.50mOD). However, they did not reach the design level of the tunnel, which is below -3.0 to -3.5mOD.

The highest groundwater level recorded in the piezometers ranged from +7.75mOD to +8.40mOD, which is 1.75-2.4m above subgrade level for the basement floor slab.

Figure 4-3 SI Location plan for 2018 Ground Investigation for Urban Solutions

4.3 Ground Model for Refined Phase 2a Building Damage Asssessment

The site investigation carried out on the site of the Cadenza Building in 2018 indicates that the top of rock is at a higher level than shown on the geological cross section produced by Jacobs/IDOM for the EIAR (Figure 4-2) i.e.:

- 2018 Ground Investigations: **+6.3mOD to +7.6mOD**
- Jacobs/IDOM Cross Section (Figure 4-2): **+0.0mOD**

Based on the 2018 investigation:

- The internal pad foundations and basement floor slab are supported on competent Limstone rock at +4.80mOD and +6.00mOD, respectively.
- The perimeter secant pile wall is embedded 3.5-4.0m into competent rock below the basement to a toe level of +2.5mOD.
- At the design profile the top of the TBM tunnel bore rises from -3.45mOD on the north side of the building to -3.1mOD on the south side. Therefore, the tunnel will be fully in rock and there will be 7.9 to 8.3m cover of rock between the foundations and the top of the tunnel.

This is the ground, building and design tunnel profile that has been adopted for the Building Damage Assessment in this report.

Note that, at the design profile there will be >0.5D cover of rock over the tunnel (i.e. >4.75m). This would mean that the lower bound volume loss of 0.75% used by Jacobs/IDOM for their Phase 2a BDA would apply across the full width of the building and not the upper bound value of 1.5% that they used for the site based on the incorrect top of rock level, as discussed in Section 4.1.

5.0 BUILDING DAMAGE ASSESSMENT METHODOLOGY

5.1 Building Risk Category and Damage Classification

The risk category and building damage for each building model and tunnel profile has been classified using the criteria set out in Table 5-1, which is presented as Table 4-4 in Section 4 of the Building Damage Report by Jacobs/IDOM (EIAR Appendix A5.17).

Table 5-1 – Criteria for Building Risk Category and Damage Classification (Table 4-4 in Building Damage Report by Jacobs/IDOM – Appendix A5.17 of the EIAR)

Building and Structure Damage Classification (after Burland et al (1977) and Boscarding and Cording (1989))					Approximately Equivalent Ground Settlements and Slopes (after Rankin 1988)	
Risk Category	Degree of Damage	Description of Typical Damage and Likely Forms of Repair for Typical Masonry Buildings	Approx. Crack Width (mm)	Limiting Max Tensile Strain (%)	Max Slope of Ground	Maximum Settlement of Building (mm)
0	Negligible	Hairline cracks	<0.1	Less than 0.05		
1	Very Slight	Fine cracks easily treated during normal redecoration. Perhaps isolated slight fracture in building Cracks in exterior brickwork visible upon close inspection	0.1 to 1	0.05 to 0.075	Less than 1:500	Less than 10
2	Slight	Cracks easily filled. Redecoration probably required. Several slight fractures inside building. Exterior cracks visible some re-pointing may be required for weather tightness. Doors and windows may stick slightly	1 to 5	0.075 to 0.15	1:500 to 1:200	10 to 50
3	Moderate	Cracks may require cutting out and patching. Recurrent cracks can me masked by suitable linings. Re-pointing and possibly replacement of a small amount of extent brickwork may be required. Doors and windows sticking. Utility services may be interrupted. Weather tightness often impaired	5 to 15 or a number of cracks greater than 3	0.15 to 0.3	1:200 to 1:50	50 to 75
4	Severe	Extensive repair involving removal and replacement of sections of walls, especially over doors and windows required. Windows and frames distorted. Floor slopes noticeably. Walls lean or bulge noticeably, some loss of bearing in beams. Utility services disrupted.	15 to 25 but also depends on number of cracks	Greater than 0.3	1:200 to 1:50	Greater than 75
5	Very Severe	Major repair required involving partial or complete reconstruction. Beams lose bearing, walls lean badly and require shoring. Windows broken by distortion Danger of instability	Greater than 25 but also depends on number of cracks	Greater than 0.3	Greater than 1:50	Greater than 75

For this assessment the Risk Category and potential Building Damage have been classified as a function of the max. building settlement, ground slope and limiting maximum tensile strain that could occur due to ground loss and settlement when tunnelling under the building.

The maximum building settlement and ground slope have been calculated from the estimated profile of vertical settlements over the tunnel. The limiting maximum tensile strain has been calculated as a function of the corresponding horizontal, bending and diagonal strains that could occur in the building.

The following sections give details of the calculations that were involved.

5.2 Settlements & Maximum Ground Slope

As described in Section 4.2.2 of the BDR, the shape of the settlement trough above the tunnel has been assumed to follow a Gaussian distribution curve centred over the centreline of the tunnel, as illustrated in Figure 5-1 and Figure 5-2.

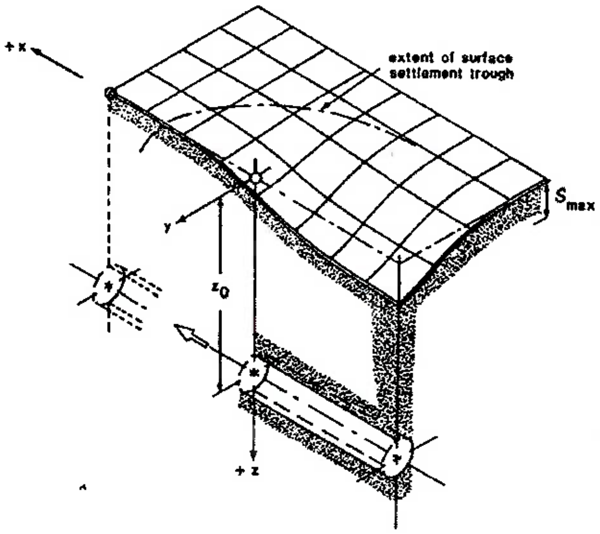

Figure 5-1 Profile of settlements over an advancing tunnel (Mair et al, 1996)

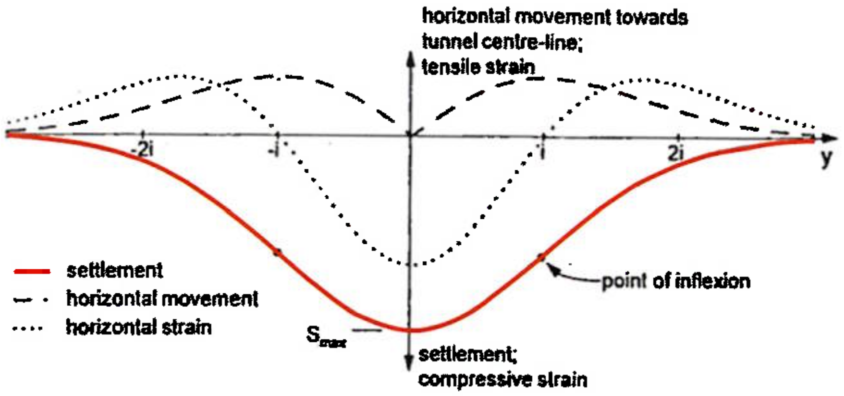

Figure 5-2 Profile of vertical and horizontal movements over the tunnel centreline and the corresponding horizontal strains (Figure 4-3 of the BDR in Appendix 5.17 of the EIAR)

The settlement (S_v) is defined as a function of the distance from the tunnel centreline, **y**, by the equation:

$$S_v = S_{max}\exp(-y^2/2i^2)$$

Where:

- S_{max} = maximum settlement over the centreline of the tunnel
- y = horizontal distance from the tunnel centreline
- **i** = the horizontal distance to the point of inflection on the settlement trough, which is defined as:

  $$i = Kz_0$$

  Where:

  - **K** = the trough width parameter
  - z_0 = depth to tunnel axis below ground level

The trough width parameter, **K**, is an empirical parameter that is defined by the ground conditions along and above the tunnel horizon.

At the Cadenza building the tunnel will be wholly in rock with at least half a tunnel diameter (>4.75m) of rock above the crown of the tunnel. Therefore, as described in Section 4.2.3 of the BDR, a value of **K=0.4** has been adopted for this Phase 2a assessment.

The pad foundations and basement floor slab of the Cadenza Building are supported on the rock at **+4.80mOD** and **+6.0mOD**, respectively, which is approximately 8.1-9.3m below ground level on Adelaide Road (+14.1mOD).

Also, the façade over the tunnel on the east side of the building is supported on 640mm diameter structural piles which are embedded into the rock to a toe level of **+2.50mOD**.

Therefore, for this Phase 2a assessment the depth to the tunnel axis, z_0, has been calculated relative to

- the level at the underside of the basement floor slab (Section 1: +6.0mOD);
- subgrade level for the pad foundations (Section 2 +4.80mOD); and
- the pile toe level (Section 3: +2.50mOD)

rather than street level to get a more representative assessment of the ground movements that could impact the building.

The maximum settlement, S_{max}, is calculated as a function of the volume of the settlement trough per metre length of tunnel, **Vs**, using the equation:

$$S_{max} = V_s/(i\sqrt{2\pi})$$

The volume of the settlement trough is assumed to be equal to the total volume of ground loss during tunnelling, i.e.:

$$V_s = V_L A$$

Where:

- **V_l** = the ground loss due to tunnelling expressed as a percentage of the cross-sectional area of the tunnel bore; and
- **A** = cross-sectional area of the tunnel = $\pi D^2/4$, where D is the outer diameter of the TBM tunnel bore.

The following ground loss parameters were assumed for the *refined* Phase 2a assessment methodology in Section 5.2.1 of the BDR:

- **V_l = 0.5%** where the tunnel is in rock and there is at least half a tunnel diameter (i.e. ≥0.5D) of rock cover above the crown; and
- **V_l = 1.0%** in mixed soil/rock strata with <0.5D cover, or in superficial material (clay/granular soil).

These are considered by Jacobs/IDOM to be compatible with the values experienced using the modern tunnelling equipment and control systems that are expected to be used on the Metrolink project.

The value of 0.5% for ground loss related to tunnelling in rock is also consistent with experience on the Dublin Port Tunnel (Gillarduzzi, 2014).

Therefore, we have adopted these values for the building damage assessment in this report rather than the more conservative values of 0.75%/1.50% used by Jacobs/IDOM for the Phase 1 and Phase 2a assessments in the BDR.

At the Cadenza building the tunnel will be in rock with at least half a tunnel diameter of rock cover. Therefore, the lower bound value of 0.5% should apply for calculating ground movements due to ground loss due to tunnelling.

However, these parameters are used to assess "greenfield" settlements that do not account for concentrated building loads from the secant pile wall or interior pad foundations. Therefore, we have also calculated settlements for a higher ground loss of 1.0% to calibrate the sensitivity of the analysis. The concentrated loads will be more significant where the depth of cover to the tunnel is <5.0m.

For this refined Phase 2a assessment, the tunnel diameter, D, has been taken as the diameter of the TBM cutter head (9.53m – BDR Section 2.1).

The calculated settlement trough profiles are included in Appendix C. Table 5.2 gives a summary of the upper and lower bound values of S_{max} for each analysis case.

The maximum ground slope across the settlement trough, $\mathbf{m_{max}}$, has been calculated using the following equation that was derived by differentiating the equation for S_v with respect to y at the point of inflection of the settlement trough, i.e. @y=i.

- $\mathbf{m_{max}} = [dS_v/dy \text{ @ } y=i] = \mathbf{(-S_{max}/i).e^{-0.5}}$

The corresponding values for each analysis case are also included in Table 5.2.

Table 5-2 – Max settlement (S_{max}) and maximum ground slope (m_{max}) for each analysis

Analysis	Profile Details	Depth to tunnel axis, z_o (m)	Cover to Fndn. Subgrade (m)	Lower Bound (V_l = 0.5%)		Upper Bound (V_l = 1.0%)	
				S_{max} (mm)	m_{max}	S_{max} (mm)	m_{max}
Case 1A	Ch. 18+995 Basement Floor Slab Design Vertical Alignment **Design Vertical Alignment**	14.21	9.45	25	0.30%	51	0.50%
Case 1B	Ch. 18+995 Basement Floor Slab Raised Vertical Alignment (+1.0m)	13.21	8.45	27	0.30%	54	0.60%
Case 1C	Ch. 18+995 Basement Floor Slab Lowered Vertical Alignment (-5.0m)	19.21	14.45	19	0.10%	37	0.30%
Case 2A	Ch. 19+010 (Centre) Internal Building RC Frame **Design Vertical Alignment**	12.85	8.09	28	0.30%	55	0.70%
Case 2B	Ch. 19+010 (Centre) Internal Building RC Frame Raised Vertical Alignment (+1.0m)	11.85	7.09	30	0.40%	60	0.80%
Case 2C	Ch. 19+010 (Centre) Internal Building RC Frame Lowered Vertical Alignment (-5.0m)	17.85	13.09	20	0.20%	40	0.30%
Case 3A	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade **Design Vertical Alignment**	10.38	5.62	34	0.50%	69	1.00%
Case 3B	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade Raised Vertical Alignment (+1.0m)	9.38	4.62	38	0.60%	76	1.20%
Case 3C	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade Lowered Vertical Alignment (-5.0m)	15.38	10.62	23	0.20%	46	0.50%

At the design tunnel profile and raised tunnel (+1.0m) profile the settlement trough extends up to 15.0m to either side of the tunnel centreline.

At the lowered tunnel profile (-5.0m) the settlement trough extends up to 20.0-25.0m to either side of the tunnel centreline.

Therefore, the settlements will not impact the western façade of the building, and should not significantly impact the entrance lobby at the southwest corner of the building.

5.3 Horizontal Movement (S_h) & Horizontal Ground & Building Strain (ε_h)

The horizontal movement of the ground within the settlement trough, S_h, has been calculated from the settlement profile using the following equation from Section 4.3.4 of the BDR:

$$S_h = (y/z_o)S_v = (y/z_o)S_{max}.\exp(-y^2/2i^2)$$

Figure 5-2 shows a typical profile of horizontal movement across the settlement trough. The resultant vectors of ground movement are directed towards the tunnel axis. The calculations for each analysis case are presented in Appendix C.

The horizontal ground strains, ε_h, were calculated using the following equation that was derived by differentiating the equation for horizontal ground movements with respect to y:

$$\varepsilon_h = dS_h/d_y = (S_{max}/z_o)[1-(y^2/i^2)].\exp(-y^2/2i^2)$$

Figure 5-2 shows a typical profile of horizontal ground strains across the settlement trough. The calculations for each analysis case are presented in Appendix C.

As described in Section 4.3.4 of the BDR, to assess the potential building damage it is assumed that the building behaves as an ideal beam of height **H** that deforms to the profile of the ground movements at the foundation level (i.e. at the tip of the secant pile wall or at basement or foundation subgrade level). This creates sagging and hogging zones of building movements, as illustrated in Figure 5-3, which are analysed separately to determine the maximum limiting tensile strain on the building.

Where the Metrolink tunnel passes under the Cadenza building the basement and interior reinforced concrete frame extend across the full width of the settlement trough. Therefore, the building response has been assessed over one half of the trough with the maximum settlement centred over the centreline of the tunnel.

The tunnel passes under the eastern façade at an acute angle of about 15 degrees. This will have the effect of "stretching out" the transverse settlement trough shown in Figure 5-2 so that, although the maximum settlement over the centreline of the tunnel will be the same, the slope of the trough will be flatter along the line of the secant pile wall. For this assessment we have conservatively assumed that the transverse profile will apply.

The extent of the hogging and sagging zones are as follows:

- Sagging Zone: y=0 (tunnel centreline) to y=i (point of inflection of settlement trough)
- Hogging Zone: y=i (point of inflection of settlement trough) to y = 2.5i (practical limit of settlement trough)

Therefore:

- Length of sagging zone, $\mathbf{L_s = i}$ (i.e. from y=0 to y=i), and
- Length of hogging zone, $\mathbf{L_h = 1.5i}$ (i.e. from y=i to y=2.5i).

The average horizontal strain in each zone has been calculated by subtracting the horizontal movement at either end by the corresponding length of the zone.

The calculations for each analysis case are presented in Appendix C and summarised on Table 5-3. Horizontal strain within the sagging zone is compressive (+ive), whereas horizontal strain in the hogging zone is tensile (-ive). Calculations are included for lower and upper bound displacements corresponding to the assumed volume loss parameters of 0.5% and 1.0% of the tunnel volume.

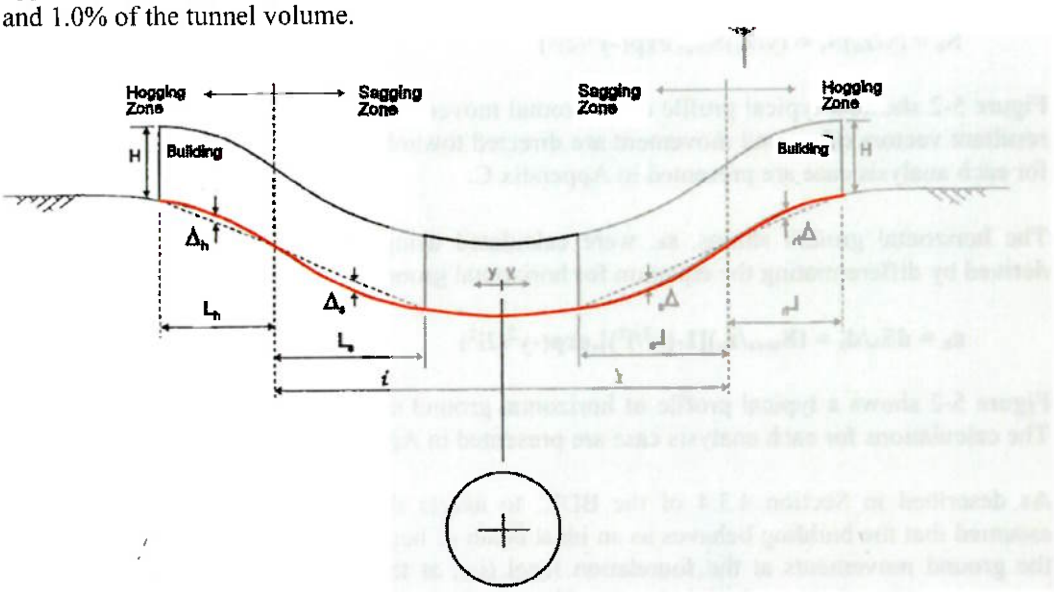

Figure 5-3 – Theoretical profile of building deformation (Figure 4-5 in the BDR)

Table 5-3 Horizontal movements (S_h) & horizontal building strains (ε_h)

Analysis	Details	Lower Bound (V_l = 0.5%)							Upper Bound (V_l = 1.0%)						
		S_h @y=0	S_h @y=i	S_h @y=2.5i	Sagging		Hogging		S_h @y=0	S_h @y=i	S_h @y=2.5i	Sagging		Hogging	
					L_s	ε_{hs}	L_h	ε_{hh}				L_s	ε_{hs}	L_h	ε_{hh}
		(mm)	(mm)	(mm)	(m)	[%]	(m)	[%]	(mm)	(mm)	(mm)	(m)	[%]	(m)	[%]
Case 1A	Ch. 18+995 Basement Floor Slab Design Vertical Alignment **Design Vertical Alignment**	0.0	6.1	1.1	5.63	0.11%	8.45	-0.06%	0.0	12.3	2.2	5.63	0.22%	8.45	-0.12%
Case 1B	Ch. 18+995 Basement Floor Slab Raised Vertical Alignment (+1.0m)	0.0	6.6	1.2	5.23	0.13%	7.85	-0.07%	0.0	13.2	2.4	5.23	0.25%	7.85	-0.14%
Case 1C	Ch. 18+995 Basement Floor Slab Lowered Vertical Alignment (-5.0m)	0.0	4.5	0.8	7.63	0.06%	11.45	-0.03%	0.0	9.0	1.6	7.63	0.12%	11.45	-0.06%
Case 2A	Ch. 19+010 (Centre) Internal Building RC Frame **Design Vertical Alignment**	0.0	6.7	1.2	5.14	0.13%	7.71	-0.07%	0.0	13.4	2.4	5.14	0.26%	7.71	-0.14%
Case 2B	Ch. 19+010 (Centre) Internal Building RC Frame Raised Vertical Alignment (+1.0m)	0.0	7.3	1.3	4.74	0.15%	7.11	-0.08%	0.0	14.6	2.6	4.74	0.31%	7.11	-0.17%
Case 2C	Ch. 19+010 (Centre) Internal Building RC Frame Lowered Vertical Alignment (-5.0m)	0.0	4.8	0.9	7.14	0.07%	10.71	-0.04%	0.0	9.7	1.8	7.14	0.14%	10.71	-0.07%
Case 3A	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade **Design Vertical Alignment**	0.0	8.3	1.5	4.15	0.20%	6.23	-0.11%	0.0	16.6	3.0	4.15	0.40%	6.23	-0.22%
Case 3B	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade Raised Vertical Alignment (+1.0m)	0.0	9.2	1.7	3.75	0.25%	5.63	-0.13%	0.0	18.4	3.3	3.75	0.49%	5.63	-0.27%
Case 3C	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade Lowered Vertical Alignment (-5.0m)	0.0	5.6	1.0	6.15	0.09%	9.23	-0.05%	0.0	11.2	2.0	6.15	0.18%	9.23	-0.10%

5.4 Bending Strain and Diagonal (Shear) Strains

As described in Section 4.3.4 of the BDR and as illustrated in Figure 5-3, by treating the building as an idealised beam with span L and height H that deforms to the profile of the settlement trough (as if deforming under a point load at the point of maximum settlement), the maximum bending strain, ε_b, and diagonal (shear) strain, ε_d, in the sagging and hogging zones of the building can be determined from the following equations:

$$\frac{\Delta}{L} = \left\{\frac{L}{12t} + \frac{3I\,E}{2tLH\,G}\right\}\varepsilon_b$$

$$\frac{\Delta}{L} = \left\{1 + \frac{HL^2G}{18IE}\right\}\varepsilon_d$$

Where:

- **Δ** is the maximum vertical displacement relative to a linear profile across the sagging (Δ_s) and hogging (Δ_s) zones (see Figure 5-3);
- **L** is the length of the building in the sagging ($L_s = i$) and hogging ($L_h = 1.5i$) zones of the settlement trough;
- **E** and **G** = Young's modulus and shear modulus of the building modelled as a beam of height H;
- **H** is the height of the building – from foundation subgrade level (or base of secant pile wall) to roof level (Case 2/Case 3), or the thickness of the basement floor slab (Case 1);
- **t** is the furthest distance from the neutral axis to the edge of the beam; and
- **I** is the moment of inertia of the beam.

The strains were calculated using the following parameters for E & G that were used for the design of the concrete in the building frame (E/G = 2.0):

- $E = 20 \times 10^6$ N/mm^2
- $G = 10 \times 10^6$ N/mm^2

In the **sagging** zone the neutral axis is assumed to be at the centre of the beam representing the building. Therefore:

- $t_s = H/2$
- $I_s = H^3/12$

Bending in this zone will cause **compressive** (+ive) **and tensile** (-ive) bending and diagonal strains (ε_{bs} & ε_{ds}).

In the **hogging** zone the neutral axis is assumed to be at the base of the beam representing the building due to the restraining effect of the foundations. Therefore:

- $t_h = H$
- $I_h = H^3/3$

Bending in this zone will cause **tensile** (-ive) bending and diagonal strains (ε_{bh} & ε_{dh}).

The calculations for each analysis case are presented in Appendix C and summarised on Table 5-4 and Table 5-5 for the sections of the building in the sagging and hogging zones, respectively.

Calculations are included for lower and upper bound displacements corresponding to the assumed volume loss parameters of 0.5% and 1.0% of the tunnel volume (V_l = 0.5% & 1.0%).

Strains within the sagging zone are shown as compressive (+ive) but can also be tensile. Strains in the hogging zone are tensile (-ive).

Table 5-4 – Bending strains (ε_{bs}) and diagonal strains (ε_{ds}) in the sagging zone

Analysis	Details	Lower Bound (V_l = 0.5%)					Upper Bound (V_l = 1.0%)				
		H	Sagging Zone				H	Sagging Zone			
			L_s	Δ_s	ε_{bs}	ε_{ds}		L_s	Δ_s	ε_{bs}	ε_{ds}
		(m)	(m)	(mm)	Bending	Diagonal	(m)	(m)	(mm)	Bending	Diagonal
Case 1A	Ch. 18+995 Basement Floor Slab Design Vertical Alignment **Design Vertical Alignment**	0.30	5.63	2.0	0.01%	0.00%	0.30	5.63	4.1	0.02%	0.00%
Case 1B	Ch. 18+995 Basement Floor Slab Raised Vertical Alignment (+1.0m)	0.30	5.23	2.2	0.01%	0.00%	0.30	5.23	4.4	0.03%	0.00%
Case 1C	Ch. 18+995 Basement Floor Slab Lowered Vertical Alignment (-5.0m)	0.30	7.63	1.5	0.00%	0.00%	0.30	7.63	3	0.01%	0.00%
Case 2A	Ch. 19+010 (Centre) Internal Building RC Frame **Design Vertical Alignment**	36.65	5.14	2.2	0.01%	0.04%	36.65	5.14	4.5	0.02%	0.09%
Case 2B	Ch. 19+010 (Centre) Internal Building RC Frame Raised Vertical Alignment (+1.0m)	36.65	4.74	2.4	0.01%	0.05%	36.65	4.74	4.8	0.03%	0.10%
Case 2C	Ch. 19+010 (Centre) Internal Building RC Frame Lowered Vertical Alignment (-5.0m)	36.65	7.14	1.6	0.01%	0.02%	36.65	7.14	3.2	0.02%	0.04%
Case 3A	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade **Design Vertical Alignment**	31.03	4.15	2.8	0.02%	0.07%	31.03	4.15	5.5	0.04%	0.13%
Case 3B	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade Raised Vertical Alignment (+1.0m)	31.03	3.75	3.1	0.02%	0.08%	31.03	3.75	6.1	0.04%	0.16%
Case 3C	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade Lowered Vertical Alignment (-5.0m)	31.03	6.15	1.9	0.01%	0.03%	31.03	6.15	3.7	0.02%	0.06%

Table 5-5 – Bending strains (ε_{bh}) and diagonal strains (ε_{dh}) in the hogging zone

Analysis	Details	Lower Bound (V_l = 0.5%)					Upper Bound (V_l = 1.0%)				
		H	Hogging Zone				H	Hogging Zone			
			L_H	Δ_H	ε_{bh}	ε_{dh}		L_H	Δ_H	ε_{bh}	ε_{dh}
		(m)	(m)	(mm)	Bending	Diagonal	(m)	(m)	(mm)	Bending	Diagonal
Case 1A	Ch. 18+995 Basement Floor Slab Design Vertical Alignment **Design Vertical Alignment**	0.3	8.45	-2.7	-0.01%	0.00%	0.3	8.45	-5.5	-0.03%	0.00%
Case 1B	Ch. 18+995 Basement Floor Slab Raised Vertical Alignment (+1.0m)	0.3	7.85	-3.0	-0.02%	0.00%	0.3	7.85	-6.0	-0.03%	0.00%
Case 1C	Ch. 18+995 Basement Floor Slab Lowered Vertical Alignment (-5.0m)	0.3	11.45	-2.0	-0.01%	0.00%	0.3	11.45	-4.0	-0.01%	0.00%
Case 2A	Ch. 19+010 (Centre) Internal Building RC Frame **Design Vertical Alignment**	36.65	7.71	-3.0	-0.01%	-0.04%	36.65	7.71	-6.0	-0.02%	-0.08%
Case 2B	Ch. 19+010 (Centre) Internal Building RC Frame Raised Vertical Alignment (+1.0m)	36.65	7.11	-3.3	-0.01%	-0.05%	36.65	7.11	-6.5	-0.02%	-0.09%
Case 2C	Ch. 19+010 (Centre) Internal Building RC Frame Lowered Vertical Alignment (-5.0m)	36.65	10.71	-2.2	-0.01%	-0.02%	36.65	10.71	-4.3	-0.01%	-0.04%
Case 3A	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade **Design Vertical Alignment**	31.03	6.23	-3.7	-0.01%	-0.06%	31.03	6.23	-7.4	-0.02%	-0.12%
Case 3B	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade Raised Vertical Alignment (+1.0m)	31.03	5.63	-4.1	-0.01%	-0.07%	31.03	5.63	-8.2	-0.03%	-0.15%
Case 3C	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade Lowered Vertical Alignment (-5.0m)	31.03	9.23	-2.5	-0.01%	-0.03%	31.03	9.23	-5.0	-0.02%	-0.05%

5.5 Total Bending and Diagonal Strains & Maximum Combined Tensile Strain

As described in Section 4.3.4 of the BDR, the maximum combined tensile strain that is used to assess the potential building damage with Table 5-1 is determined by combining the total horizontal building strain (ε_h) from Section 5.3 with the bending strains (ε_b) and diagonal strains (ε_d) from Section 5.4 using the following equations:

$$\varepsilon_{bt} = \varepsilon_h + \varepsilon_b$$

$$\varepsilon_{dt} = 0.35\varepsilon_h + \left[(0.65\varepsilon_h)^2 + \varepsilon_d^2\right]^{0.5}$$

The maximum value of the combined tensile strain obtained from these equations is used in the assessment of the potential building damage category in Table 5-1.

The calculations for each analysis case are presented in the appendices and summarised on Table 5-6 and Table 5-7 for the sections of the building in the sagging and hogging zones, respectively. Calculations are included for lower and upper bound displacements corresponding to the assumed volume loss parameters of 0.5% and 1.0% of the tunnel volume (V_l = 0.5% & 1.0%). Compressive strains are shown as positive and tensile strains are shown as negative.

In the **sagging** zone the horizontal strains are compressive (+ive) but the bending and diagonal (shear) strains can be either compressive (+ive) or tensile (-ive) because the neutral axis is assumed to be at the centre of the beam (H/2). The positive horizontal strains in the sagging zone are significantly larger than the tensile bending and diagonal strains. Therefore:

- For the bending strains, the total bending strain (ε_{bt}) has been calculated as the sum of the compressive (+ive) horizontal strain ($\varepsilon_{h\ +ive}$) and bending strains ($\varepsilon_{b\ +ive}$);
- However, for the diagonal strains we have calculated the maximum *compressive* diagonal strain ($\varepsilon_{dt\ +ive}$) using positive values for both the horizontal and bending strains ($\varepsilon_{h\ +ive}$ & $\varepsilon_{d\ +ive}$).

In the **hogging** zone, where the building is more susceptible to damage, the horizontal strains, bending strains and diagonal (shear) strains are tensile (-ive) because the neutral axis is assumed to be at the base of the beam. Therefore:

- For the bending strains, the total bending strain (ε_{bt}) has been calculated as the sum of the tensile (-ive) horizontal ($\varepsilon_{h\ -ive}$) and bending strains ($\varepsilon_{b\ -ive}$);
- For the diagonal strains a representative resultant total diagonal strain ($\varepsilon_{dt\ -ive}$), has been calculated using the *absolute* values of the tensile horizontal and diagonal strains ($\varepsilon_{h\ -ive}$ & $\varepsilon_{d\ -ive}$) because of the square functions in the equation. However, the calculated resultant has been reported as a maximum *tensile* diagonal strain ($\varepsilon_{dt\ -ive}$) for consistency with the sign convention in this report.

Only negative tensile strains are considered in the building damage assessment.

The maximum tensile (-ive) bending or diagonal strain from the sagging or hogging zone (typically hogging) is used in the building damage assessment. The relevant values are summarised in Table 5-8.

Taable 5-6 – Total bending strains (ε_{bts}) and diagonal strains (ε_{dts}) in the sagging zone [Compressive]

Analysis	Profile Details	Combined Strains (Sagging Zone)							
		Lower Bound (V_l = 0.5%)				Upper Bound (V_l = 1.0%)			
		Horizontal	Bending	Combined (Bending)	Combined (Diagonal)	Horizontal	Bending	Combined (Bending)	Combined (Diagonal)
		ε_h (%)	ε_b (%)	ε_{bt} (%)	ε_{dt} (%)	ε_h (%)	ε_b (%)	ε_{bt} (%)	ε_{dt} (%)
Case 1A	Ch. 18+995 Basement Floor Slab Design Vertical Alignment **Design Vertical Alignment**	0.11%	0.01%	0.12%	0.11%	0.22%	0.02%	0.24%	0.22%
Case 1B	Ch. 18+995 Basement Floor Slab Raised Vertical Alignment (+1.0m)	0.13%	0.01%	0.14%	0.13%	0.25%	0.03%	0.28%	0.25%
Case 1C	Ch. 18+995 Basement Floor Slab Lowered Vertical Alignment (-5.0m)	0.06%	0.00%	0.06%	0.06%	0.12%	0.01%	0.13%	0.12%
Case 2A	Ch. 19+010 (Centre) Internal Building RC Frame **Design Vertical Alignment**	0.13%	0.01%	0.14%	0.14%	0.26%	0.02%	0.29%	0.28%
Case 2B	Ch. 19+010 (Centre) Internal Building RC Frame Raised Vertical Alignment (+1.0m)	0.15%	0.01%	0.17%	0.17%	0.31%	0.03%	0.33%	0.33%
Case 2C	Ch. 19+010 (Centre) Internal Building RC Frame Lowered Vertical Alignment (-5.0m)	0.07%	0.01%	0.08%	7.00%	0.14%	0.02%	0.15%	0.15%
Case 3A	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade **Design Vertical Alignment**	0.20%	0.02%	0.22%	0.22%	0.40%	0.04%	0.44%	0.43%
Case 3B	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade Raised Vertical Alignment (+1.0m)	0.25%	0.02%	0.26%	0.26%	0.49%	0.04%	0.53%	0.53%
Case 3C	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade Lowered Vertical Alignment (-5.0m)	0.09%	0.01%	0.10%	0.10%	0.18%	0.02%	0.21%	0.20%

Table 5-7 – Total bending strains (ε_{bth}) and diagonal strains (ε_{dth}) in the hogging zone [Tensile]

Analysis	Profile Details	Combined Strains (Hogging Zone)							
		Lower Bound (V_l = 0.5%)				Upper Bound (V_l = 1.0%)			
		Horizontal	Bending	Combined (Bending)	Combined (Diagonal)	Horizontal	Bending	Combined (Bending)	Combined (Diagonal)
		ε_h (%)	ε_b (%)	ε_{bt} (%)	ε_{dt} (%)	ε_h (%)	ε_b (%)	ε_{bt} (%)	ε_{dt} (%)
Case 1A	Ch. 18+995 Basement Floor Slab Design Vertical Alignment **Design Vertical Alignment**	-0.06%	-0.01%	**-0.07%**	-0.06%	-0.12%	-0.03%	**-15.00%**	-12.00%
Case 1B	Ch. 18+995 Basement Floor Slab Raised Vertical Alignment (+1.0m)	-0.07%	-0.02%	**-0.09%**	-0.07%	-0.14%	-0.03%	**-0.17%**	-0.14%
Case 1C	Ch. 18+995 Basement Floor Slab Lowered Vertical Alignment (-5.0m)	-0.03%	-0.01%	**-0.04%**	-0.03%	-0.06%	-0.01%	**-0.08%**	-0.06%
Case 2A	Ch. 19+010 (Centre) Internal Building RC Frame **Design Vertical Alignment**	-0.07%	-0.01%	-0.08%	**-0.09%**	-0.13%	-0.02%	-0.15%	**-0.16%**
Case 2B	Ch. 19+010 (Centre) Internal Building RC Frame Raised Vertical Alignment (+1.0m)	-0.08%	-0.01%	-0.09%	**-0.10%**	-0.17%	-0.02%	-0.19%	**-0.20%**
Case 2C	Ch. 19+010 (Centre) Internal Building RC Frame Lowered Vertical Alignment (-5.0m)	-0.04%	-0.01%	-0.04%	**-0.04%**	-0.07%	-0.01%	-0.09%	**-0.09%**
Case 3A	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade **Design Vertical Alignment**	-0.11%	-0.01%	-0.12%	**-0.13%**	-0.22%	-0.02%	-0.24%	**-0.26%**
Case 3B	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade Raised Vertical Alignment (+1.0m)	-0.13%	-0.01%	-0.15%	**-0.16%**	-0.27%	-0.03%	-0.29%	**-0.32%**
Case 3C	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade Lowered Vertical Alignment (-5.0m)	-0.05%	-0.01%	-0.06%	**-0.06%**	-0.10%	-0.02%	-0.12%	**-0.12%**

Table 5-8 – Maximum tensile strain ($\varepsilon_{t\text{-}max}$)

Analysis	Profile Details	Maximum Limiting Tensile Strains			
		Lower Bound (V_l = 0.5%)		Upper Bound (V_l = 1.0%)	
		$\varepsilon_{t\text{-}max}$ (%)	Zone	$\varepsilon_{t\text{-}max}$ (%)	Zone
Case 1A	Ch. 18+995 Basement Floor Slab Design Vertical Alignment **Design Vertical Alignment**	-0.07%	Hogging	-0.15%	Hogging
Case 1B	Ch. 18+995 Basement Floor Slab Raised Vertical Alignment (+1.0m)	-0.09%	Hogging	-0.17%	Hogging
Case 1C	Ch. 18+995 Basement Floor Slab Lowered Vertical Alignment (-5.0m)	-0.04%	Hogging	-0.08%	Hogging
Case 2A	Ch. 19+010 (Centre) Internal Building RC Frame **Design Vertical Alignment**	-0.09%	Hogging	-0.17%	Hogging
Case 2B	Ch. 19+010 (Centre) Internal Building RC Frame Raised Vertical Alignment (+1.0m)	-0.10%	Hogging	-0.20%	Hogging
Case 2C	Ch. 19+010 (Centre) Internal Building RC Frame Lowered Vertical Alignment (-5.0m)	-0.04%	Hogging	-0.09%	Hogging
Case 3A	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade **Design Vertical Alignment**	-0.13%	Hogging	-0.26%	Hogging
Case 3B	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade Raised Vertical Alignment (+1.0m)	-0.16%	Hogging	-0.32%	Hogging
Case 3C	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade Lowered Vertical Alignment (-5.0m)	-0.06%	Hogging	-0.12%	Hogging

5.6 Building Damage Assessment (BDA)

The results of the building damage assessment calculations are summarised in Table 5-9. The table gives the following values for each analysis case:

- Max limiting tensile strain, ε_{max} (%)
- Maximum ground slope on the settlement trough, m_{max} (%)
- Max settlement over the tunnel centreline, S_v (mm)

The risk category and degree of damage have been determined from the criteria in Table 5-1. The following is a summary of the analyses that were carried out:

- The BDA has been carried out for a lower and upper bound tunnel volume loss (V_l) of 0.5% & 1.0%, respectively. This corresponds to the parameters for the *Refined* Phase 2a assessment in the EIAR.
- At the Cadenza building the tunnel will be in rock with at least half a diameter cover (>4.75m) of rock below the building basement. Therefore, the lower bound estimates for a tunnel volume loss, **V_l, of 0.5%** should represent the conditions that could be achieved for modern TBM tunnelling in rock.
- The BDA methodology assumes that the building deforms and articulates to the profile of the greenfield settlement trough at foundation subgrade level. This is conservative as it does not account for re-distribution of stresses and ground movements as a result of the stiffness of the building.
- However, the methodology does not account for concentrated point loads such as those from the integral pad foundations in the basement floor slab, which are supporting the internal concrete building frame, or from the load-bearing piles in the perimeter secant pile wall, which are supporting combined loads from the building structure and external façade. Therefore, we have included calculations for an upper bound volume loss, **V_l, of 1.0%** to represent conditions that could potentially occur where there are concentrated loads over the tunnel and to calibrate the sensitivity of the analysis.
- The impact of concentrated loads from the building foundations in the basement floor slab or from load bearing piles in the perimeter secant pile wall is most acute where the level of the tunnel is high and close to the underside of the foundations (e.g. under the structural secant piles at the upper LoD). At deeper levels the concentrated loads become more dispersed through the rock so that they become less concentrated (e.g. at the lower LoD).
- The design vertical profile for the tunnel rises by **0.55m** from south to north across the width of the building so that it is shallowest on the south side.
- **Case 1** models the impact of the tunnel on the 300mm thick basement floor slab on the south side of the building (Ch. 18+995) based on the settlement profile at formation level for the slab (+6.00mOD).
- **Case 2** models the impact of the tunnel on the internal RC structure of the building (Ch. 19+010) based on the settlement profile at subgrade level for the integral pad foundations for the columns (+4.80mOD).

- **Case 3** models the impact of the tunnel on the secant pile wall and façade on the east side of the building at Ch. 19+025) based on the settlement profile at toe level for the 640mm diameter structural piles in the wall (+2.50mOD).
- **Case 1A, Case 2A and Case 3A** represent the analyses that have been carried out at the design vertical profile for the tunnel.
- **Case 1B, Case 2B and Case 3B** represent the analyses that have been carried out for a raised vertical profile of the tunnel within the ***amended*** upper Limit of Deviation proposed for the Draft Railway Order (+1.0m)
- **Case 1C, Case 2C and Case 3C** represent the analyses that have been carried out for a lowered vertical profile of the tunnel at 5.0m below the design level, which is within the lower limit of deviation proposed in the Draft Railway Order (-10.0m)
- For **Case 1** the pile toe level is only 4.9m above the crown of the tunnel at the design profile. At the maximum proposed vertical LoD the TBM will hit the toe of the piles. Therefore, for Case 1B we have only raised the tunnel profile by **3.9m** so that the crown of the tunnel bore is at least **1.0m** below the toe of the piles.

Based on the results of the assessment in Table 5-9:

Results for Design Tunnel Profile (Case 1A, 2A & 3A):

- **At the design tunnel profile**, the lower bound estimates of ground movements for a volume loss of **0.5%** result in the following Risk Categories:
  - Case 1A (RC Basement Floor Slab):
    - Risk Category = 2/1 (Slight to Very Slight Damage)
  - Case 2A (Internal RC Structure):
    - Risk Category = 2 (Slight Damage)
  - Case 3A (Perimeter Scant Pile Wall/East Building Façade):
    - Risk Category = 2 (Slight Damage)
- For the upper bound estimates of ground movements corresponding to a volume loss of **1.0%** at the design tunnel profile, the Risk Category raises to
  - Case 1A (RC Basement Floor Slab):
    - Risk Category = 2/3 (Slight to Moderate Damage)
  - Case 2A (Internal RC Structure):
    - Risk Category = 3 (Moderate Damage)
  - Case 3A (Perimeter Scant Pile Wall/East Building Façade):
    - Risk Category = 3 (Moderate Damage)
- There is there is **7.90-8.25m** cover between the subgrade for the internal pad foundations and the top of the TBM tunnel bore. The cover reduces to **5.60m** at the toe level of the structural piles in the secant pile wall along the building façade, which are embedded 3.5m into competent rock below basement subgrade level.
- The lower-bound results for the **0.5%** ground loss, should represent the conditions for tunnelling in the Limestone rock below the building. However, this does not take into account the concentrated loads from the pad foundations or structural piles, which are directly over the tunnel in places and carry loads up to 1,000kPa and 1025kN, respectively. Therefore, the results for a ground loss of **1.0%** could represent a

conservative upper bound value for the damage that could occur under the additional concentrated loads, particularly for the pile foundations supporting the façade, where there will be only 5.6m cover over the tunnel bore.

Results for Raised Tunnel Profile (Case 1B, 2B & 3B):

- **For the raised tunnel profile**, there is a slight ***increase*** in concentrated displacements and strains in the building so that the lower bound estimates of ground movements for a volume loss of **0.5%** result in the following Risk Categories:
    - Case 1B (RC Basement Floor Slab):
        - Risk Category = 2 (Slight to Very Slight Damage)
    - Case 2B (Internal RC Structure):
        - Risk Category = 2 (Slight Damage)
    - Case 3B (Perimeter Scant Pile Wall/East Building Façade):
        - Risk Category = 2/3 (Slight to Moderate Damage)
- At the upper bound estimates of ground movements corresponding to a volume loss of **1.0%** the risk category and degree of damage increases across the building as follows:
    - Case 1B (RC Basement Floor Slab):
        - Risk Category = 3 (Moderate Damage)
    - Case 2B (Internal RC Structure):
        - Risk Category = 3 (Moderate Damage)
    - Case 3B (Perimeter Scant Pile Wall/East Building Façade):
        - Risk Category = 3/4 (Moderate to Severe Damage)
- For the raised tunnel profile there is there is **6.90-7.25m** cover between the subgrade for the internal pad foundations and the top of the TBM tunnel bore. The cover reduces to **4.60m** at the toe level of the structural piles in the secant pile wall along the building façade, which are embedded 3.5m into competent rock below basement subgrade level.
- The lower-bound results for the **0.5%** ground loss, should represent the conditions for tunnelling in the Limestone rock below the building. However, this does not take into account the concentrated loads from the pad foundations or structural piles, which are directly over the tunnel in places and carry loads up to 1,000kPa and 1025kN, respectively. Therefore, the results for a ground loss of **1.0%** could represent a conservative upper bound value for the damage that could occur under the additional concentrated loads, particularly for the reduced depth of cover for the pad foundations and for the pile foundations supporting the façade, where there will be only 4.6m cover over the tunnel bore.

Results for Lowered Tunnel Profile (Case 1C, 2C & 3C):

- **For the lowered tunnel profile**, there is a significant ***reduction*** in concentrated displacements and strains in the building so that the lower bound estimates of ground movements for a volume loss of **0.5%** result in the following Risk Categories:
    - Case 1C (RC Basement Floor Slab):
        - Risk Category = 1 (Very Slight Damage)
    - Case 2C (Internal RC Structure):

        - Risk Category = 1 (Very Slight Damage)
    - Case 3C (Perimeter Scant Pile Wall/East Building Façade):
        - Risk Category = 1/2 (Very Slight to Slight Damage)
- At the upper bound estimates of ground movements corresponding to a volume loss of **1.0%** the risk category and degree of damage increases across the building as follows:
    - Case 1C (RC Basement Floor Slab):
        - Risk Category = 2 (Slight Damage)
    - Case 2C (Internal RC Structure):
        - Risk Category = 2 (Slight Damage)
    - Case 3C (Perimeter Scant Pile Wall/East Building Façade):
        - Risk Category = 2/3 (Slight to Moderate Damage)

- For the *lowered* tunnel profile there is there is **12.90-13.25m** cover between the subgrade for the internal pad foundations and the top of the TBM tunnel bore. The cover reduces to **10.60m** at the toe level of the structural piles in the secant pile wall along the building façade, which are embedded 3.5m into competent rock below basement subgrade level.
- At the lowered level, the concentrated loads from the internal pad foundations and perimeter secant pile wall will have a less significant impact on ground movements as the loads will become more distributed through the rock with depth. **Therefore, for Case 1C, 2C and 3C the *lower* bound ground loss of 0.5% should give a more representative assessment of the risk of building damage.**

Note that, as discussed previously, the refined Phase 2a building damage assessment is a preliminary semi-empirical estimate of the potential damage that could occur to the building due to tunnelling related ground movements. It does not account for the stiffness of the building, which can reduce and redistribute settlements across the tunnel profile. At the same time, it does not model the concentrated load from the building foundations which can have the opposite effect.

The more detailed Phase 3 analysis of the soil-structure response to tunnelling referred to in the EIAR would be required to give a more comprehensive and representative engineering assessment of the response of the building to tunnelling in the underlying rock.

It may also be possible to justify a lower range in ground loss parameters for the type of TBM that will be used for the project. However, this would need to be supported with representative case studies and more comprehensive information on the quality and strength of the rock under the building i.e. higher ground loss can be experienced in zones of weathered, highly fractured rock and currently the ground investigation information that we have does not reach the tunnel profile. Also, the type of TBM will not be mandated through the contract so it will be selected by the Contractor. Therefore, any analysis that is carried out at planning stage should consider a risk assessment of the full range of ground loss that could occur and not just the "best case scenario".

Table 5-9 – Summary of Building Damage Assessment

Analysis	Details	Depth to Tunnel Axis (z_o)/ Cover to Foundation Subgrade (m)	Lower Bound (V_l = 0.6%)					Upper Bound (V_l = 1.0%)				
			Lim. (Max) Tensile Strain	Max Ground Slope	Max Settlement	Risk Category	Degree of Damage	Lim.(Max) Tensile Strain	Max Ground Slope	Max Settlement	Risk Category	Degree of Damage
			ε_{tmax} (%)	m_{max} (%)	S_{max} (mm)			ε_{tmax} (%)	m_{max} (%)	S_{max} (mm)		
Design Tunnel Profile												
Case 1A	Ch. 18+995 Basement Floor Slab Design Vertical Alignment **Design Vertical Alignment**	z_o = 14.2m Cover= 9.5m	-0.07%	0.30%	25	2/1	Slight to Very Slight	-0.15%	0.50%	51	2/3	Slight to Moderate
Case 2A	Ch. 19+010 (Centre) Internal Building RC Frame **Design Vertical Alignment**	z_o = 12.9m Cover= 8.1m	-0.09%	0.30%	28	2	Slight	-0.17%	0.70%	55	3	Moderate
Case 3A	Ch. 19+025 (South Side) Secant Pile Wall/Bldg. Façade **Design Vertical Alignment**	z_o = 10.4m Cover= 5.6m	-0.13%	0.50%	34	2	Slight	-0.26%	1.00%	69	3	Moderate
Raised Tunnel Profile (Max. Proposed Vertical Deviation = + 1.0m)												
Case 1B	Ch. 18+995 Basement Floor Slab Raised Vertical Alignment (+1.0m)	z_o = 13.2m Cover= 8.5m	-0.09%	0.30%	27	2	Slight	-0.17%	0.60%	54	3	Moderate
Case 2B	Ch. 19+010 (Centre) Internal Building RC Frame Raised Vertical Alignment (+1.0m)	z_o = 11.9m Cover= 7.1m	-0.10%	0.40%	30	2	Slight	-0.20%	0.80%	60	3	Moderate
Case 3B	Ch. 19+025 (Centre) Secant Pile Wall/Bldg. Façade Raised Vertical Alignment (+1.0m)	z_o =9.4m Cover= 4.6m	-0.16%	0.60%	38	2/3	Slight to Moderate	-0.32%	1.20%	76	3/4	Moderate to Severe
Lowered Tunnel Profile (Max. Proposed Vertical Deviation = - 5.0m)												
Case 1C	Ch. 18+995 Basement Floor Slab Lowered Vertical Alignment (-5.0m)	z_o = 19.2m Cover= 14.5m	-0.04%	0.10%	19	1	Very Slight	-0.08%	0.30%	37	2	Slight
Case 2C	Ch. 19+010 (Centre) Internal Building RC Frame Lowered Vertical Alignment (-5.0m)	z_o = 17.9m Cover= 13.1m	-0.04%	0.20%	20	1	Very Slight	-0.09%	0.30%	40	2	Slight
Case 3C	Ch. 19+025 (Centre) Secant Pile Wall/Bldg. Façade Lowered Vertical Alignment (-5.0m)	z_o = 15.4m Cover= 10.6m	-0.06%	0.20%	23	1/2	Very Slight to Slight	-0.12%	0.50%	46	2/3	Slight to Moderate

Consideration should also be given to what is an acceptable risk category and degree of damage for the Cadenza building.

For example, at a Risk Category of 2 (Slight), which is the lower bound estimate of the damage that could occur to the building and façade at the design tunnel profile (Case 2A/3A), this corresponds to crack widths up to 1-5mm and the following damage criteria in Table 5-1:

Cracks easily filled. Redecoration probably required. Several slight fractures inside building. Exterior cracks visible some re-pointing may be required for weather tightness. Doors and windows may stick slightly

If the concentrated foundation loads have a significant impact on the ground loss during tunnelling then the damage Risk Category could rise to 3 (Moderate), which is the upper bound estimate of the damage that could occur to the building and façade at the design tunnel profile (Case 2A/3A). This corresponds to crack widths up to 5-15mm (or a number of cracks greater than 3mm) and the following damage criteria in Table 5-1:

Cracks may require cutting out and patching. Recurrent cracks can me masked by suitable linings. Re-pointing and possibly replacement of a small amount of extent brickwork may be required. Doors and windows sticking. Utility services may be interrupted. Weather tightness often impaired

This damage criteria was developed for masonry structures and it is arguable whether it is representative of the modern design and construction characteristics of the Cadenza Building, which would be more sensitive to damage than indicated by the criteria and corresponding risk categories in Table 5-1. i.e.:

- The basement uses a Rascor waterproofing system, which relies on the watertightness of the reinforced concrete basement floor slab and walls. Therefore, even minor cracking could lead to groundwater ingress;
- The internal reinforced concrete frame uses post-tensioned reinforced concrete floor slabs for large spans which would be more sensitive to differential settlement than conventional RC slabs;
- The tunnel passes under the one of the main structural cores for the building (Stair Core #2) and this incorporates a complex transfer structure at first floor level to accommodate a shift in the position of the core for the upper floors of the building;
- The building has a modern steel and glass façade which would be more sensitive to distortion due to differential settlement along the perimeter secant pile wall than a masonry or blockwork structure.

The settlement troughs that have been calculated for the tunnelling-induced ground loss for each case in Table 5-9 are included in the calculations in Appendix C.

At the design tunnel profile and for the raised tunnel profile (i.e. +1.0m) the settlement trough of the ground movements due to tunnelling extends up to 15.0m to either side of the tunnel centreline.

At the lowered tunnel profile (-5.0m) the settlement trough extends up to 20.0-25.0m to either side of the tunnel centreline, but with a lower maximum settlement and flatter slope.

Therefore, the settlements will not impact the western façade of the building and should not significantly impact the entrance lobby at the southwest corner of the building, which contains a particularly sensitive curved glass wall and terrazzo floor at ground floor level but is >25m from the centerline of the tunnel.

However, if the alignment of the tunnel under the building is moved horizontally by up to 15.0m to the west, which is within the horizontal Limits of Deviation proposed in the Draft Railway Order, then the sensitive structures and finishes of the entrance lobby could be brought into the zone of influence of the tunnelling induced settlements.

Significantly, as discussed in Section 3.0 and illustrated in Figure 3-7, at the design tunnel alignment the TBM cutter head will hit the bottom 0.9m of the anti-flotation anchors for the rainwater retention tank in the basement and this will increase to 1.9m if the tunnel is raised by 1.0m to the upper limit of the LoD.

The anti-flotation flotation anchors prevent hydrostatic uplift of the floor slab of the tank and are designed for a tensile force of 725kN. If the piles are hit by the TBM it will have a significant impact on the building, i.e.:

- The capacity of the anchors will be reduced, potentially causing heave, cracking and groundwater ingress in the rainwater harvesting tank;
- The TBM will also compromise the corrosion protection sleeve of the anchors, which will leave the steel bar exposed to corrosion and reduce the design life of the anchors;
- The steel bars will obstruct the cutter head of the TBM potentially getting wrapped around the cutting discs. This will necessitate an intervention below the building which will involve:
    - Stopping the TBM directly under the building foundations;
    - Cutting off the slurry circulation and backing up the TBM; and
    - Accessing to the front of the TBM to repair the damage to the cutter discs, possibly under compressed air.
- This could have a significant adverse impact on the ground movements and related damage to the building.

It would be recommended to lower the alignment of the tunnel by at least 5.0m to avoid hitting the anti-flotation anchors and to reduce the potential damage category to the building to Risk Category 1 (Very Slight) or lower. It would still be necessary to carry out a detailed Phase 3 analytical assessment of the ground movements and distortion of the building to confirm that the damage is withing acceptable thresholds that take account the specific design and construction characteristics of the building within the settlement trough.

The anti-flotation anchors under the rainwater harvesting tank could be avoided by moving the horizontal alignment of the tunnel 15.0m to the west. However, are additional anti-flotation anchors for another tank in the basement at the south end of the building that could be impacted, as shown on Figure 3-7. This would also adversely impact the façade and entrance lobby on the west side of the building, as discussed.

Moving the horizontal alignment of the tunnel 15.0m to the east, which is still within the maximum proposed horizontal LoD, would have the double benefit of avoiding the anti-flotation anchors and also significantly reducing the extent of the building that will be over the tunnel. Therefore, this should also be considered in conjunction with lowering the level of the tunnel to mitigate the likely significant adverse impacts of tunnelling under the building.

It should also be noted that changing the alignment and lowering the level of the tunnel are realistically the only effective mitigation measures that can be implemented to reduce the impact of tunnelling on the Cadenza Building. There is very limited potential for other methods of mitigation, such as compensation grouting, because the building is supported on rock.

5.7 Building Damage Assessment for Cadenza Building in the EIAR

The results of the building damage assessment calculations for the conditions assumed by Jacobs/IDOM in the Building Damage Report (BDR) in Appendix A5.17 of the EIAR are presented in Appendix D.

The assessment was carried out for the Davitt House building that occupied the site prior to the construction of the Cadenza Building.

This means that no building damage assessment has been carried out in the EIAR for the Cadenza Building. This is a significant omission because:

- The EIAR and Draft Railway Order (RO) were published in November 2022;
- The Davitt House Building was demolished in 2019; and
- Construction work started on the Cadenza Building in January 2020 and reached practical completion in October 2022.

This means that the Cadenza building would have been clearly under construction by the time that the building damage assessments were been carried out. Also, the building was completed prior to the issue of the EIAR and Draft RO. Therefore, the building damage assessment in the EIAR should have been carried out for the Cadenza building and not Davitt House, which had been demolished three years earlier.

This is a notable omission in the EIAR as it means that the report does not assess the likely significant impacts of tunnelling on the Cadenza building, despite the fact that it should have been evident that the assessment on the Davitt House was no longer applicable.

What makes it particularly notable is that the Phase 2a building damage assessment that Jacobs/IDOM carried out on Davitt House identified that the potential degree of damage to the building would be in Risk Category 2 [Slight] due to ground movements below the 2.5m deep foundations. In comparison, the foundations for the Cadenza Building, which has a double basement, are at a depth of over 9.0mBGL which is much closer to the crown of the tunnel and would result in a significantly higher level of distortion and damage to the building.

For the record, the following is a summary of the Phase 2a building damage assessment that was carried out by Jacobs/IDOM for Davitt House (see Appendix C) in the BDR:

- Davitt House has been identified in the BDR as a Case D "prominent" building that requires special consideration (Building B-147 @ Ch. 19+020 in Appendix B.1).
- The assessment has been carried out for a 4-storey building with a total height of 12m and a foundation depth of 2.5m below a ground level of 13.0mOD.
- Based on their interpreted geological profile for the site (see Figure 4-2) the Phase 2a building damage assessment should have been carried out using the upper bound ground loss of **1.50%** (ref: Table 5-1 of the BDR: Ch.18+980-19+100) and a trough width parameter, **K=0.4**. This is intended to represent tunnelling in rock with <0.5D cover of rock over the tunnel, where D is the tunnel diameter.

- We have checked the calculations, and our results would indicate that Jacobs/IDOM actually carried out the assessment for the lower bound ground loss of **0.75%**, which is intended to represent tunnelling in rock with >0.5D cover of rock over the tunnel.
- While this would be more representative of the actual conditions on the site, it is not consistent with the information presented in the Building Damage Report by Jacobs/IDOM, which incorrectly shows rock at a lower level below the building.
- The structural response of the building was analysed using the ratio E/G = 2.6, which the BDR states is representative of a masonry structure. [E= Young's Modulus, G = Shear Modulus].
- The results of the assessment are presented in Table 5-10, which gives the calculated values for:
  - Max limiting tensile strain, ε_{max} (%)
  - Maximum ground slope on the settlement trough, m_{max} (%)
  - Max settlement over the tunnel centreline, S_v (mm)
- Based on these results and the criteria in Table 5-1, the estimated degree of damage to the building due to tunnelling was in Risk Category 2 [Slight], which corresponds to crack widths up to 1-5mm and the following damage criteria in Table 5-1:

Cracks easily filled. Redecoration probably required. Several slight fractures inside building. Exterior cracks visible some re-pointing may be required for weather tightness. Doors and windows may stick slightly

- This was considered by Jacobs/IDOM to be an acceptable degree of damage to the building for the Phase 2a assessment so that no further analysis was required for the EIAR, despite the fact that Davitt House was categorised as a Case D "prominent building".

Table 5-10 – Summary of Building Damage Assessment for Davitt House (Building B-147) in the Jacobs/IDOM BDA Report in Appendix A5.17 of the EIAR

Analysis	Details	Depth to Tunnel Axis (z_o)/ Cover to Foundation Subgrade (m)	Lower Bound (V_l = 0.75%)					Upper Bound (V_l = 1.5%)				
			Lim. (Max) Tensile Strain	Max Ground Slope	Max Settlement	Risk Category	Degree of Damage	Lim.(Max) Tensile Strain	Max Ground Slope	Max Settlement	Risk Category	Degree of Damage
			ε_{tmax} (%)	m_{max} (%)	S_{max} (mm)			ε_{tmax} (%)	m_{max} (%)	S_{max} (mm)		
EIAR Results (Jacobs/ IDOM BDA)	Building B-147 Ch. 19+020 Davitt House **Design Vertical Alignment**	z_o = 18.44m Cover= 13.92m	-0.08%	0.28%	35	2	Slight	Not Used - See Note Below				

<u>Note</u>: Based on our calculations it appears that Jacobs/IDOM have carried out the assessment of the building damage to Davitt House using the lower bound ground loss of 0.75%, which is intended to represent tunnelling in rock with >0.5D cover of rock over the tunnel, where D is the tunnel diameter. While this is more representative of the actual conditions on the site, it is not consistent with the geological profile interpreted by Jacobs/IDOM for the site, or with the information presented in the Building Damage Report, which indicated that the assessment should be carried out using the upper bound ground loss of 1.5% for <0.5D cover of rock over the tunnel.

5.8 The Assessment of the Impact of Implementing the Limits of Deviation for the Tunnel Alignment on the Building Damage Assessment in the EIAR

The Phase 2a building damage assessments in the Jacobs/IDOM Building Damage Report (BDR) in Appendix A5.17 of the EIAR were carried out for the proposed ***design profile*** of the tunnel shown on the alignment drawings.

The following Limits of Deviation (LoD) have been proposed for the tunnel alignment in Article 6 (Deviation) of Part 2 of the Draft Railway Order [6.1(d)]:

- 5.0m upwards
- 10.0m downwards
- ±15.0m horizontally

During the Oral Hearing, in response to submissions received during the statutory consultation process, TII proposed to amend the upward LoD for the tunnel in the Railway Order to **1.0m**, due to concerns of potential increased impact on buildings if the original upward LoD were allowed (Ref. Expert Witness Statement of Ronan Hallissey p7).

Changing the alignment of the tunnel, and particularly raising the level, could have a significant impact on building damage. However, this is not considered in the Building Damage Report in the EIAR.

The Wider Effects Report (WER) in Appendix 5.19 of the EIAR assesses whether the power to deviate the tunnel alignment within the LoD would alter the predicted significant impacts assessed and reported in the EIAR.

The report gives a high-level qualitative assessment of the impact of implementing the LoD for relevant Chapters of the EAIR. However, the impact on building settlement and damage is not addressed for Chapter 5 of EIAR (Metrolink Construction Phase), which contains the Building Damage Report.

This is a very notable omission from the EIAR because it does not address the likely significant impacts to buildings along the route if the level of the tunnel is raised.

It also does not assess the *mitigating* effect that lowering the level of the tunnel will have on ground movements and building damage due to tunnelling. As the Cadenza building is supported on rock, this is the only realistic mitigation measure that can be used to reduce the level of building settlement, distortion and damage to within tolerable limits.

The WER concludes that raising the tunnel alignment will have "*no potential for significant additional impacts*" in relevant sections of the EIAR (e.g. soils & geology, Land Use, Major Accidents & Emergency etc.) and that "*there would be no change to the required mitigation measures, or to the residual impacts arising from the application of the mitigation measures set out in the EIAR.*"

Furthermore, the WER does not specify any constraints on the implementation of the vertical or horizontal LoD at the site of the Cadenza Building. This is very significant given the likelihood that, at the design profile, the tunnel will hit the anti-flotation anchors for the rainwater harvesting tank in the basement. Also, raising the level of the tunnel will increase the potential degree of damage to the building.

5.9 Oral Hearing – Additional Information on Implementing the LoD and the Building Damage Assessment in the EIAR

During the Oral Hearing Jacobs/IDOM issued a Technical Note that addresses the Impact on the Preliminary Design Building Damage Assessment Results due to the Imposition of the Limits of Deviation, under the following heading:

Updates Appendix 8 Impact on the Preliminary Design Building Damage Assessment Results due to Imposition of Limits of Deviation

A copy of the technical note is included in Appendix D. The technical note relates to the conclusions in the Building Damage Report (Appendix A5.17 of the EIAR). However, it is not issued as an addendum to the Wider Effects Report and the BDR and WER have not been updated.

I would have the following comments on the assessment in the technical note:

- Technically, this means that TII have now carried out an assessment of implementing the LoD on the Building Damage Assessment. However, I would consider that the assessment is very generic, does not properly assess the impacts and is not fit for purpose in the case of the Cadenza Building. i.e.
    - The assessment has been carried out on a generic project-wide basis for buildings that fall into Damage Category 1 (DC-1), Damage Category 2 (DC-2) and Damage Category 3 (DC-3) so there is still no specific assessment for the Cadenza Building.
    - The assessment for the raised tunnel profile has been carried out for the revised upward LoD of **+1.0m** but also at the maximum Lateral LoD of **15.0m**. It does not assess the impacts separately or state in which direction the lateral LoD is applied. This is particularly significant for the Cadenza Building where vertical and horizontal changes to the tunnel alignment can have different impacts on the anti-flotation anchors and building damage.
    - The technical note states that "*quantitative assessments have been carried out*" for buildings in the DC-2 category, which would include Davitt House on the site of the Cadenza Building. However, the methodology of the analyses is not stated and the results are not provided, so we can only assume that the methodology is the same as the Phase 2a assessment in the BDR.
    - The technical note concludes that
        - there will be no increase in the damage category level for any of the DC-2 buildings;
        - Different and/or additional impacts from the assessment are *below "Slight"* (i.e. below Risk Category 2). Therefore, there will be
            - no change to the required mitigation measures set out in the EIAR;
            - no change to the residual impacts arising from the application of these mitigation measures; and
            - no additional significant impacts.

Our assessment demonstrates that raising the tunnel level could potentially increase the impact on the building and façade to Risk Category 3 (Moderate), particularly if the concentrated loads from the pad foundations and perimeter secant piles are taken into account.

- The technical note generically concludes that lowering the vertical alignment could only improve on the damage potential in all cases. They have not properly assessed the *positive* impacts of lowering the tunnel profile as a mitigation measure.
- **Fundamentally, the methodology is still based on Risk Category 2 [Slight Damage] being considered an acceptable threshold of damage for the Phase 2A assessment in the Building Damage Report.**
- This generic methodology is not fit for purpose for the Cadenza Building because it does not reflect the lower limits of acceptable building distortion and damage that would apply to the building façade and basement waterproofing system, which are particularly sensitive to differential settlement and cracking.

6.0 SUMMARY

- The assessment of building damage due to tunnelling-induced ground movements is presented in the Building Damage Report (the "BDR") by Jacobs/IDOM in Appendix A5.17 of the EIAR.
- The assessment for the site of the Cadenza building was carried out for Davitt House, which previously occupied the site but was demolished three years prior to the issue of the EIAR and Draft Railway Order (RO) in November 2022.
- Construction work on the Cadenza building started in January 2020 and the building reached practical completion in October 2022, which was also prior to the issue of the EIAR and draft RO.
- **Therefore, it is a notable omission that the likely significant effects of tunnelling on the Cadenza building have not been assessed in the EIAR.**
- This is particularly notable because,
    - with a double basement and a foundation level that is up to 9m below ground level, the Cadenza Building would be classified as a Case B "special structure" which would require particular consideration in the BDR;
    - the deeper foundations of the Cadenza Building, which reduce the cover over the tunnel, would result in higher levels of building distortion and damage than those predicted for Davitt House, which had shallow foundations; and
    - the structural characteristics of the Cadenza Building would likely be more sensitive to ground movements with lower tolerances to cracking, building distortion and damage.
- The Building Damage Assessment for the Cadenza building in this report has been carried out using the *refined* Phase 2a parameters for ground loss of **0.5%** and **1.0%** that are presented in the BDR for tunnelling in rock and superficial material (i.e. soils), respectively. These are considered by Jacobs/IDOM to be compatible with the values experienced using the modern tunnelling equipment and control systems that are expected to be used on the Metrolink project. The value of 0.5% for ground loss related to tunnelling in rock is also consistent with experience on the Dublin Port Tunnel (Gillarduzzi, 2014).
- At the Cadenza building the tunnel will be in rock with >0.5D cover of rock under the building at the design profile, where D is the tunnel diameter (9.53m). Therefore, the lower bound value of 0.5% should generally apply. However, the upper bound ground loss of 1.0% assesses the sensitivity of the analysis and could potentially account for concentrated loads from the building foundations, depending on the level of the tunnel.
- The tunnel rises by 0.55m from north to south. Therefore, we have carried out our analyses at 3 No. representative profiles across the width of the building to assess the impact of the tunnelling on the basement floor slab, internal reinforced concrete frame, and the façade.
- We have accounted for the different foundation levels below the basement floor slab (+6.0mOD), internal pad foundations (+4.80mOD), and the load bearing structural piles in the perimeter secant pile wall which supports the façade (+2.50mOD).

- We have also assessed the adverse impact of raising the level of the tunnel by 1.0m and the *positive* impact of lowering it by up to 5.0m, which are within the *revised* Limits of Deviation proposed during the Oral Hearing for the Draft Railway Order (RO) (+1.0m upwards/-10m downwards).
- The potential impact of changing the horizontal alignment within the proposed Limits of Deviation (±15.0m) has been assessed qualitatively – specifically in relation to the extent of the building over the tunnel and the potential impact on the façade and the anti-flotation anchors for the rain in the basement.
- **At the design profile** shown on the RO alignment drawings, the potential damage to the building at the lower bound ground loss of **0.5%** has been assessed as **Slight (Risk Category 2)** for the internal RC structure and for the secant pile wall and façade on the east side of the building,
- This corresponds to crack widths up to 1-5mm and the following damage criteria in Table 5-1 which could still have unacceptable adverse impacts on the structure and basement waterproofing:

Cracks easily filled. Redecoration probably required. Several slight fractures inside building. Exterior cracks visible some re-pointing may be required for weather tightness. Doors and windows may stick slightly

- The lower-bound results for the **0.5%** ground loss, should represent the conditions for tunnelling in the Limestone rock below the building. However, it does not take into account the concentrated loads from the internal pad foundations or the load-bearing structural piles, which are directly over the tunnel in places.
- If the ground loss is increased to **1.0%** as a sensitivity analysis to account for the concentrated building foundation loads, then the damage level could potentially increase to **Moderate (Risk Category 3)** for the internal RC structure and for the secant pile wall and façade on the east side of the building,
- This corresponds to crack widths up to 5-15mm (or a number of cracks greater than 3mm) and the following damage criteria in Table 5-1, which could have significant unacceptable adverse impacts on the structure and basement waterproofing:

Cracks may require cutting out and patching. Recurrent cracks can me masked by suitable linings. Re-pointing and possibly replacement of a small amount of extent brickwork may be required. Doors and windows sticking. Utility services may be interrupted. Weather tightness often impaired

- **Significantly, at the design profile the TBM will hit the bottom 0.9m of the anti-flotation anchors for the rainwater retention tank in the basement of the building.** This will compromise the capacity and integrity of the anchors, potentially causing heave, cracking and groundwater ingress into the tank, which could flood the basement.

- If the TBM hits the anchors an intervention would likely be required to access the front of the tunnel and remove the obstruction. This would involve stopping the TBM directly below the building, turning off all the circulating slurry that controls ground movements, and then backing the machine up from the face to facilitate man access. This could have significant adverse impacts on ground movements and building damage.
- **If the tunnel is raised by 1.0m**, there will be a slight increase in the ground movements and building distortion, potentially increasing the level of damage to the perimeter secant pile wall and building façade to **Slight to Moderate (Risk Category 2/3)** at a ground loss of **0.5%** or **Moderate to Severe (Risk Category 3/4)** at an upper bound ground loss of **1.0%**, which could account for the concentrated loads under the toe of the structural piles which will be <5.0m above the crown of the tunnel at the higher level.
- Raising the level of the tunnel would also increase the depth of the anti-flotation anchors in the basement that could be hit by the tunnel, significantly increasing the risk of ground movements and building damage.
- **Lowering the level of the tunnel has significant *positive* impacts on building damage.** If the alignment is lowered by **5.0m**, the concentrated loads from the building foundations will have less impact on ground movements because the loads become less concentrated and more uniformly distributed into the ground. Therefore, the lower bound assessment for a ground loss of 0.5% may give a more representative assessment of the risk of building damage, which is **Very Slight (Risk Category 1)** for the basement floor slab and internal RC structure, and **Very Slight to Slight (Risk Category 1/2)** for the perimeter secant pile wall and east building façade.
- Risk Category 1 (Very Slight) corresponds to cracks up to 0.1-1mm wide and the following damage criteria in Table 5-1:

Fine cracks easily treated during normal redecoration. Perhaps isolated slight fracture in building
Cracks in exterior brickwork visible upon close inspection

- Lowering the tunnel by 5.0m would also bring the level of the TBM below the toe of the anti-flotation anchors for the rainwater harvesting tank in the basement.
- Therefore, the alignment of the tunnel should be lowered by at least 5.0m to avoid hitting the anti-flotation anchors and to reduce the potential damage category to the building to Risk Category 1 (Very Slight) or lower.
- Consideration would still need to be given to what is an acceptable risk category and degree of damage for the Cadenza building. Fundamentally, the building damage assessment methodology in the EIAR is based on Risk Category 2 (Slight Damage) being considered by Jacobs/IDOM to be an acceptable threshold of damage for all buildings along the route so that no further assessment is required.
- This generic methodology was developed for masonry structures and is not fit for purpose for the Cadenza Building because it does not reflect the lower limits of acceptable building distortion and damage that would apply to the building façade, internal RC structure and basement waterproofing system, which are particularly sensitive to differential settlement and cracking.

- It would still be necessary to carry out a detailed Phase 3 analytical assessment of the ground movements and distortion of the building to confirm that the damage is withing acceptable thresholds that take account the specific design and construction characteristics of the building.
- At the design tunnel alignment and for the raised alignment (+1.0m) the profile of settlements due to tunnelling extends up to **15.0m** to either side of the tunnel centreline. This distance increases to **20.0-25.0m** if the tunnel is lowered by 5.0m, but with a lower maximum settlement and flatter slope.
- This means that the settlements should not significantly impact the western façade of the building or the entrance lobby at the southwest corner, which contains a particularly sensitive curved glass wall and terrazzo floor at ground floor level but is >25m from the centerline of the tunnel.
- However, if the alignment of the tunnel is moved horizontally by up to **15.0m to the west**, which is within the horizontal Limits of Deviation proposed in the Draft Railway Order, then the sensitive structures and finishes of the entrance lobby could be brought into the zone of influence of the tunnelling induced settlements, which would not be recommended.
- Moving the alignment to the west also still potentially impacts the anti-flotation anchors on this side of the basement.
- Moving the horizontal alignment of the tunnel **15.0m to the east**, which is still within the maximum proposed horizontal LoD, would have significant *positive* impacts for the Cadenza Building i.e.:
  - It would significantly reduce the extent of the building over the tunnel; and
  - It would avoid all the anti-flotation anchors in the basement.
- Therefore, this should also be considered in conjunction with lowering the level of the tunnel to mitigate the likely significant adverse impacts of tunnelling under the building.
- The impact of implementing the LoD on building has not been assessed by Jacobs/IDOM in the Wider Effects Report (WER) in Appendix 5.19 of the EIAR, which is intended to assess whether the power to deviate the tunnel alignment within the LoD would alter the predicted significant impacts reported in the EIAR.
- **This is a very notable omission from the EIAR as changing the alignment of the tunnel will have likely significant impacts on building damage. Furthermore, lowering the level of the tunnel will have *positive* impacts on building damage, which has not been properly assessed in the EIAR.**
- As the Cadenza building is supported on rock, changing the alignment of the building would be the primary mitigation measure to reduce the level of building settlement, distortion and damage to within tolerable limits.
- During the Oral Hearing, Jacobs/IDOM prepared a technical note to assess the potential impact of implementing the LoD on the results of the building damage assessment in the BDR. However, in my opinion it does not properly assess the likely significant impacts, i.e.

- The assessment is very generic and still does not include a specific assessment for the Cadenza Building (i.e. it is still based on the results in the BDR for Davitt House);
- The methodology and results of the analyses are not presented in the technical note;
- It does not treat changes to the vertical and horizontal alignment separately, which is particularly relevant to the Cadenza building;
- It is still based on an acceptable threshold of building damage at Risk Category 2 (Slight), which is not fit for purpose fort he Cadenza Building as it does not reflect the lower limits of acceptable building distortion and damage that would apply to the building façade, internal structure, and basement waterproofing system;
- The positive impact of lowering the tunnel alignment has not been properly assessed.

7.0 RECOMMENDATIONS

Based on the results of the Refined Phase 2a BDA for the Cadenza Building we would recommend that:

- The level of the tunnel should be lowered by at least **5.0m** to avoid hitting the anti-flotation anchors and to reduce the effect of tunnelling related ground movements on the building.
- Consideration should also be given to moving the tunnel **horizontally to the east by 15.0m** to reduce the extent of the building that is over the tunnel and to move the tunnel away from the anti-flotation anchors.
- The Wider Effects Report should be revised to include a constraint on the application of the Limits of Deviation for the tunnel under the Arthur Cox Building so that there is no scope for deviation from the ***revised*** tunnel alignment due to the potential for significant adverse impacts on the building.
- A more detailed Phase 3 analytic building assessment should be carried out as part of the approval process to confirm that the degree of building distortion and damage due to tunnelling induced ground movements is within acceptable limits for all aspects of the structure.
- As part of the Phase 3 assessment TII and Jacobs/IDOM should liaise with the structural designers of the building façade, structure and basement waterproofing system to determine the acceptable threshold limits of building distortion, damage and ground movements due to tunnelling.
- The Phase 3 assessment should take account of the concentrated building foundation loads and should include a risk assessment to assess the effects of tunnelling induced ground movements over a range of ground loss of at least 0.25%-0.50% for tunnelling in rock, and not just the best-case scenario.
- The EIAR should properly assess the positive impact of lowering the tunnel alignment in the Phase 3 assessment, and should also assess appropriate design and construction mitigation measures that are relevant to the Cadenza Building. As the building is on rock, lowering the tunnel level will be the most effective mitigation measures to reduce the impact of tunnelling induced ground movements. There will be limited potential for compensation grouting or jacking.
- Consideration should also be given to specify the type of TBM that will be used, or to specify appropriate limits on building distortion or ground loss due to tunnelling.
- The EIAR should also include appropriate monitoring measures for the building to ensure that settlements and the resulting stresses and strains in the structure are within acceptable limits.

REFERENCES:

Gillarduzzi, Andrea, "*Investigating property damage along Dublin Port Tunnel alignment*", Proc. of the ICE, Forensic Engineering, Vol. 167, Issue FE3, August 2014.

Mair, R.J, & Taylor, R.N., "*Bored tunnelling in the urban environment*", Proc. of the 14th Intl. Conference on Soil Mechanics & Foundation Engineering, Hamburg, 1997.

Mair, R.J, Taylor, R.N., & Burland, J.B., "*Prediction of ground movements and assessment of risk of building damage due to bored tunnelling*", Geotechnical Aspects of Underground Construction in Soft Ground, International Symposium, London, 1996.

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<tr><td colspan="2">Revision No:</td><td>Date:</td><td>Notes</td></tr>
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<tr><td colspan="2">Rev.1</td><td>1/3/24</td><td>Minor Revisions – Appendix C Calculations Omitted.</td></tr>
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<tr><td>Made by:</td><td>Conor O'Donnell</td><td colspan="2">Conor O'Donnell</td></tr>
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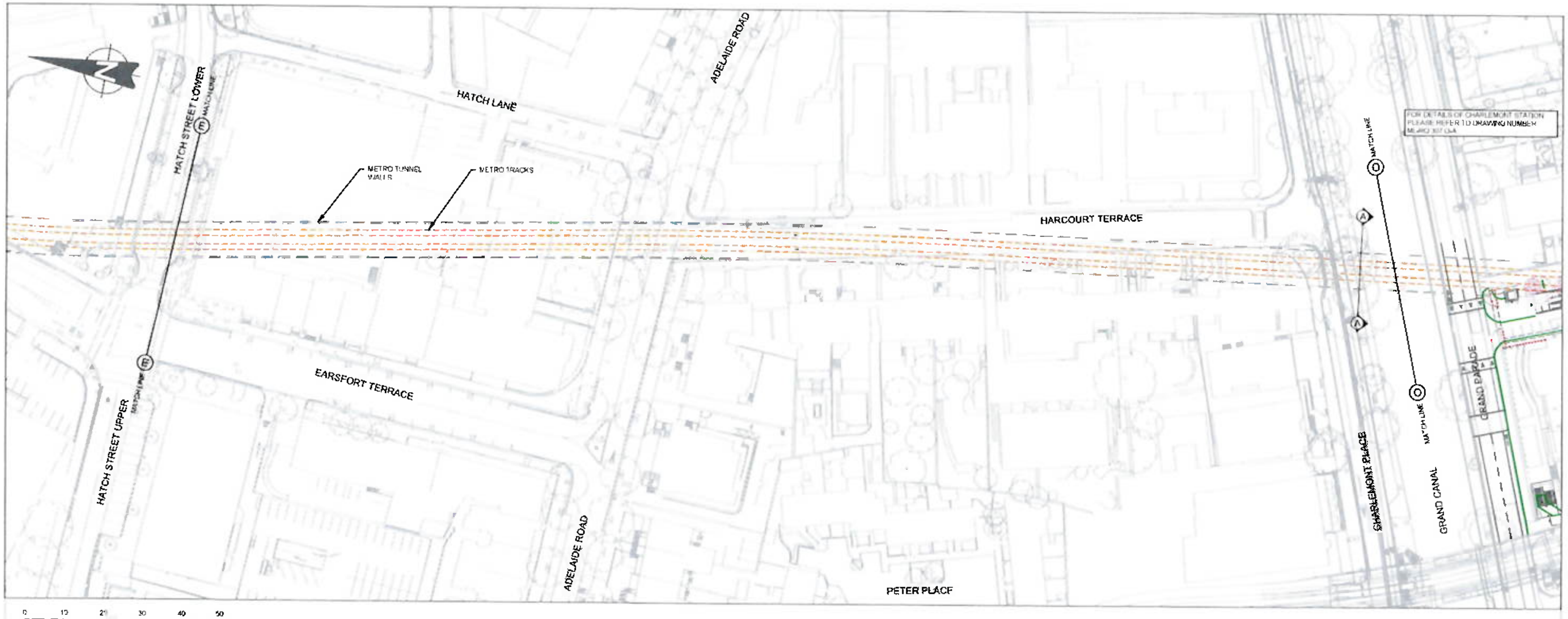
Appendix A
Drawings



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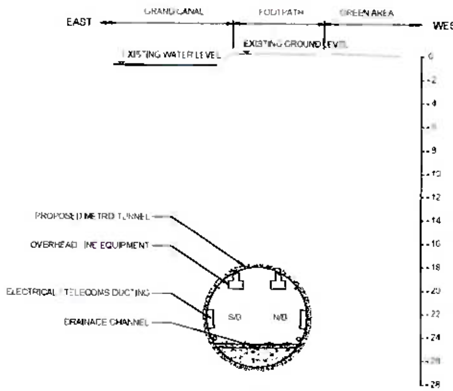
SCALE 1:1000 (A3)
METRES



SECTION A - A

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METRES

DRAWING LEGEND:

- EXISTING KERB
- PROPOSED EARTHWORK
- OVERHEAD LINES
- TRAFFIC FLOW
- METRO TUNNEL EXTENTS
- METRO RAIL TRACKS
- TREES TO BE REMOVED
- TREES TO BE RETAINED
- EXISTING KERB TO BE REMOVED
- PROPOSED KERB

NOTES:

1. FOR DETAILS OF STRUCTURES (INCLUDING STATIONS) SEE BOOK OF STRUCTURES DRAWINGS (E.G. DRAWINGS COMMENCING ML ST OR ML BR ETC.)
2. FOR DETAILS OF PROPERTY SEE BOOK OF PROPERTY DRAWINGS (DRAWINGS COMMENCING ML P)
3. FOR DETAILS OF UTILITIES SEE BOOK OF UTILITIES DRAWINGS (DRAWINGS COMMENCING ML UC) AND BOOK OF SURFACE WATER DRAWINGS (DRAWINGS COMMENCING ML UH)

REV.	DATE	Description	DRAWN BY	CHECKED BY	REVIEWED BY	APPROVED BY
P01	[illegible]	For Railway Order	[illegible]	[illegible]	[illegible]	NC
P03	[illegible]	For Railway Order	JR	[illegible]	[illegible]	NC
P04	[illegible]	For Railway Order	[illegible]	JR	[illegible]	PB
P05	17/08/22	For Railway Order	GB	MM	MM	PB

Client: TII, NTA

Project: METROLINK

Consultant: JACOBS, IDOM

Drawing Title: METROLINK - GENERAL ARRANGEMENT HATCH STREET LOWER TO GRANDE PARADE

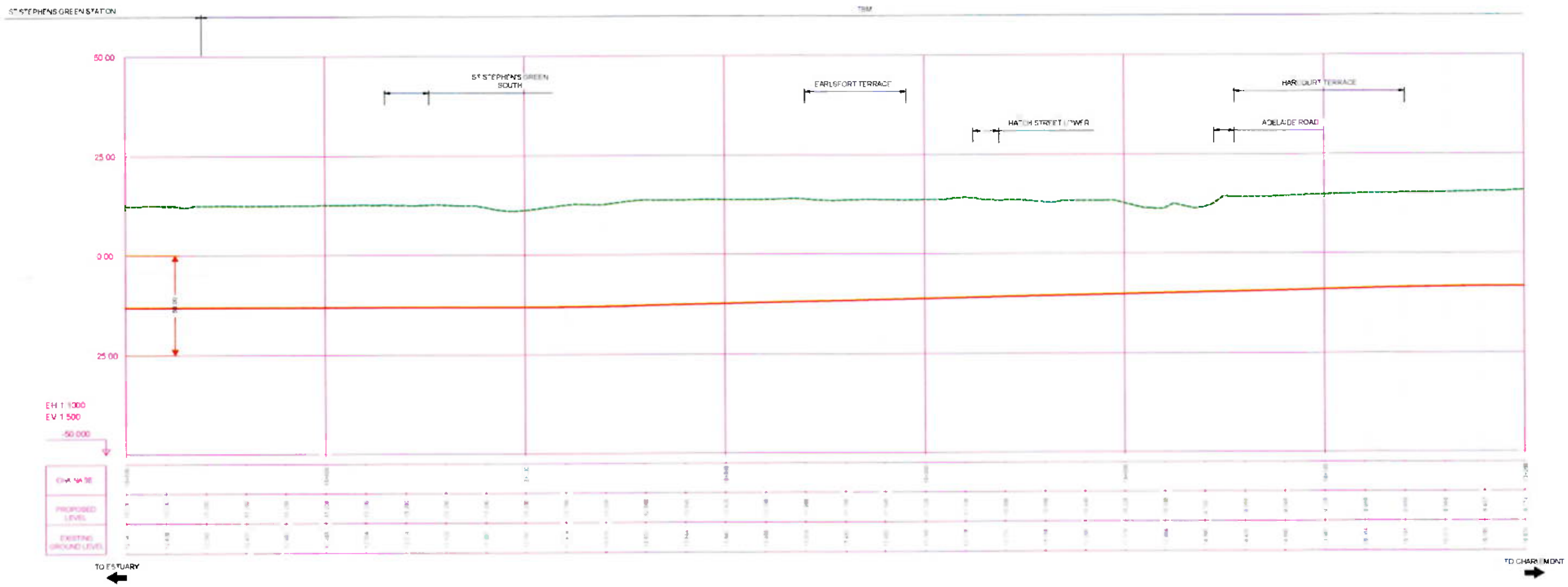
Drawing Status: S0

Drawing Nº: ML1-JAI-ARD-ROUT_XX-DR-Y-03095

Plan Drawing Nº: ML-RO 306 E-O

Scales Original Size A1: AS SHOWN

Sheet 1 of 1



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P02	[illegible]	For Railway Order	BJ	BS	AS	NC
P03	[illegible]	For Railway Order	BJ	JR	AS	FB

DRAWING LEGEND

RAIL LEVEL
EXISTING GROUND LEVEL

Client: TII, NTA

Project: METROLINK

Consultant: JACOBS, IDOM

Drawing Title: METROLINK - ALIGNMENT LONG SECTION 18

Drawing Status: S0

Drawing N°: ML1-JAI-ARD-ROUT_XX-DR-Y-01018

Plan Drawing N°: ML-LN O-C18

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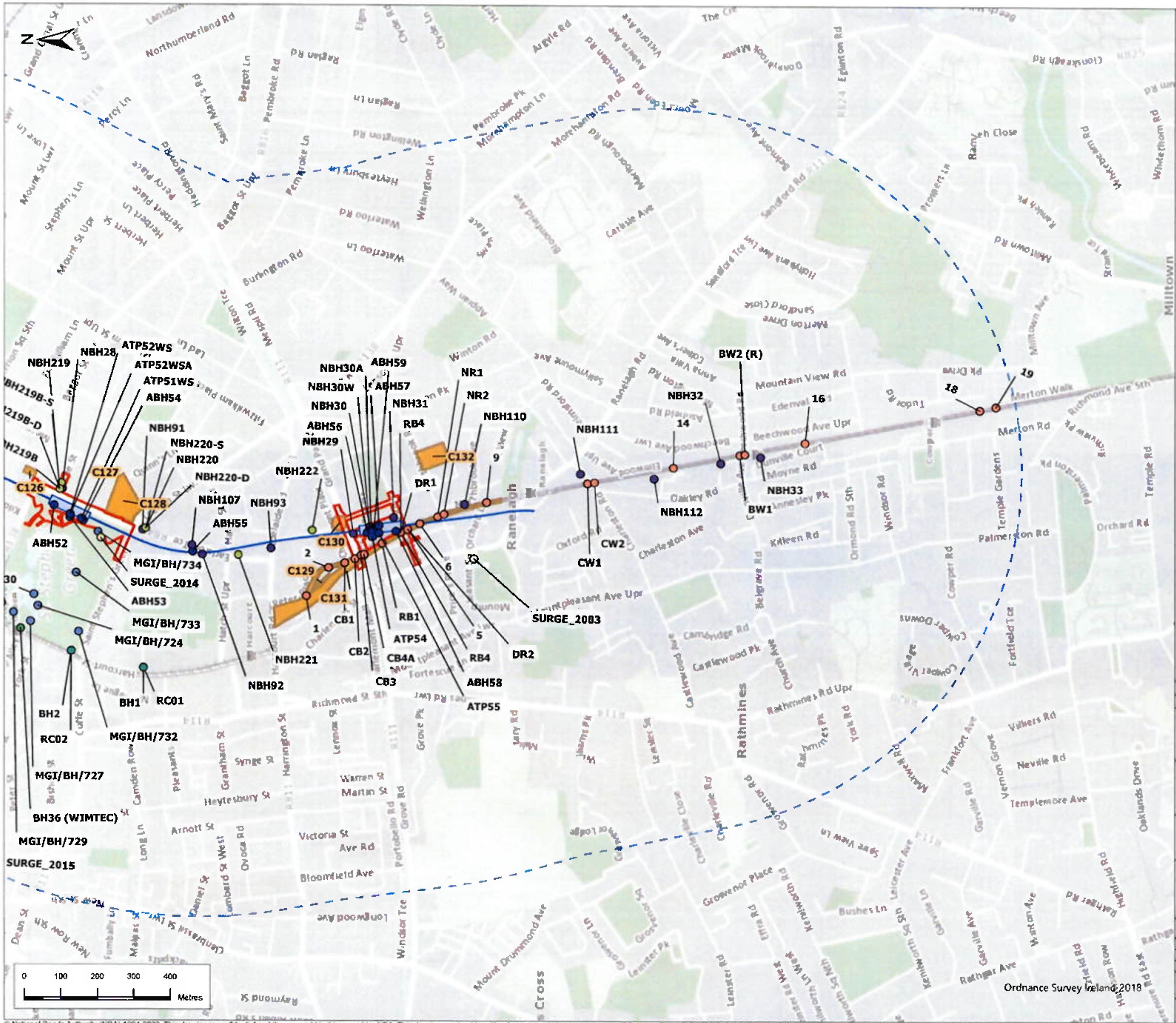
Sheet 18 of 19



Legend

Alignment
- Tunnel
- Station Locations
- Project Boundary

Geographic Split
- AZ4 Northwood to Charlemont

GI Locations
- Wimtec,2000
- IGSL,1995
- IGSL,2007
- Norwest Holst,2008
- GSI,2009
- Causeway Phase 1, 2019
- Causeway Phase 2, 2019
- Causeway Phase 5, 2021
- Potential land contamination sources within 250m of the Works Area and/or alignment

Rev	Date	Purpose of revision	Drawn	Check'd	Rev'd	Appr'd
P04	15/5/22	Final Issue	FND	RH	RH	NC

JACOBS IDOM

Client: TII

Project: METROLINK

Drawing Title: **Figure 20.6 Land Contamination Sheet 7 of 8**

Drawing Status: Final

Scale @ A3	1:10,000	DO NOT SCALE
Jacobs No.	32108800	
Client No.		
Drawing No.	ML1-JAI-EIA-ROUT_XX-DR-Y-21040	Rev P04

This drawing is not to be used in whole or in part other than for the intended purpose and project as defined on this drawing. Refer to the contract for full terms and conditions.

NOTES

KV= Vertical curve radio (m)
CV= Vertex level (m)
W= Difference between the exit slope minus the entrance slope (‰)
D = Sagitta: difference between CV and TOR level (m)

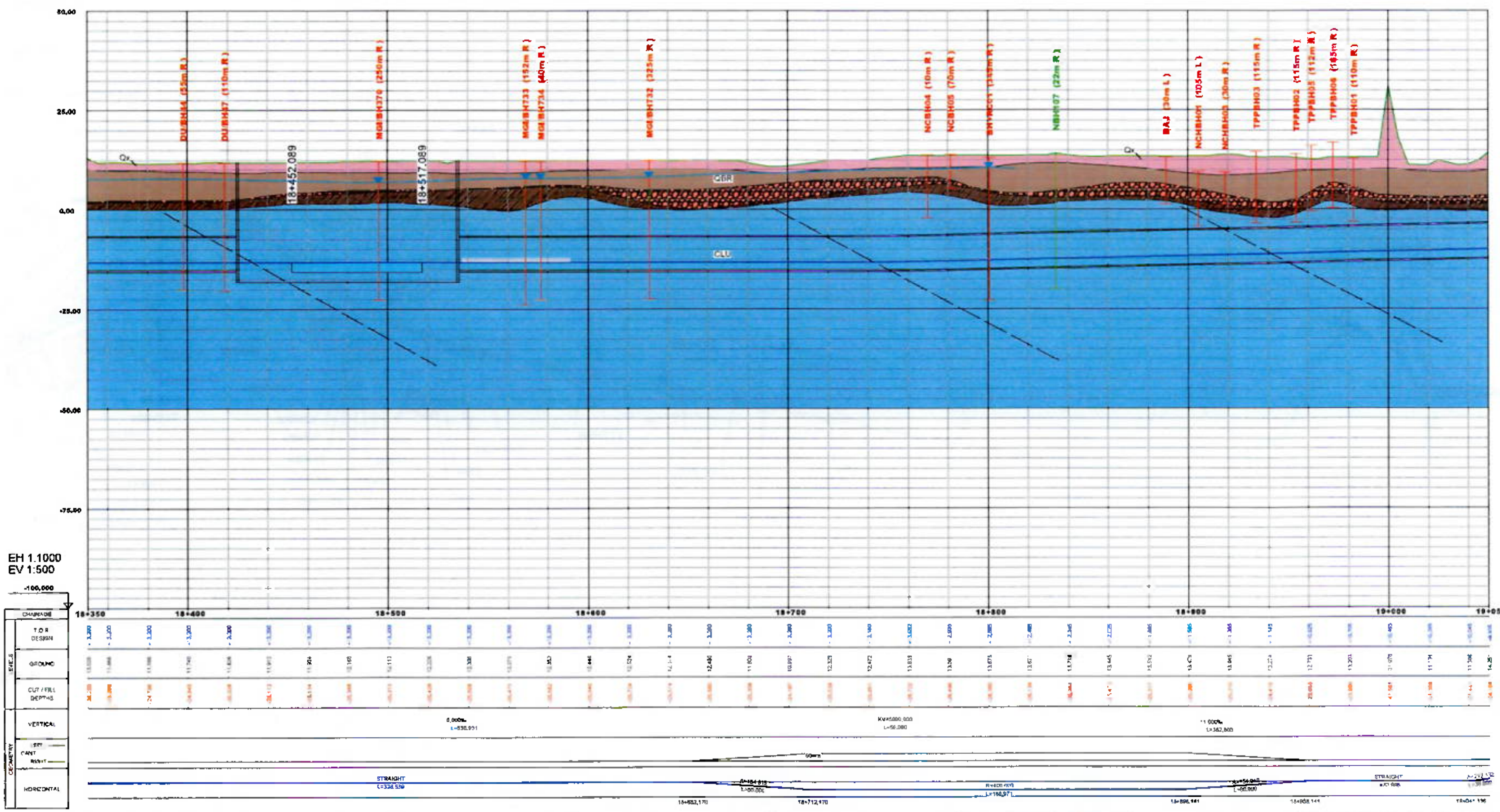
The distance from TOR to tunnel crown is 6.7 m

SYMBOLOGY

Cut and cover section

TOR Rail level (TOR)

STATION
ST. STEPHENS GREEN



Key Plan

REV.	DATE	Description	DRAWN BY	CHECKED BY	REVIEWED BY	APPROVED BY
P03	11/12/20	Final Preliminary Design	CT	FP	JB	NC
P02	03/04/20	Preliminary Design	CT	FP	JB	NC
P01	05/12/19	Preliminary Design	CT	FP	JB	NC

Client: TII NTA

Project: METROLINK

Consultant: JACOBS IDOM

Drawing Title: Ground Investigation Geological Long Section – Phase 1

Drawing Status: S4

Drawing N°: ML1-JAI-GEO-ROUT_XX-DR-Y-00013

Scales Original Size A1

Sheet 26 of 28

NOTES

KV= Vertical curve radio (m)
CV= Vertex level (m)
W= Difference between the exit slope minus the entrance slope (‰)
D = Saguitte, difference between CV and TOR level (m)

The distance from TOR to tunnel crown is 6.7 m

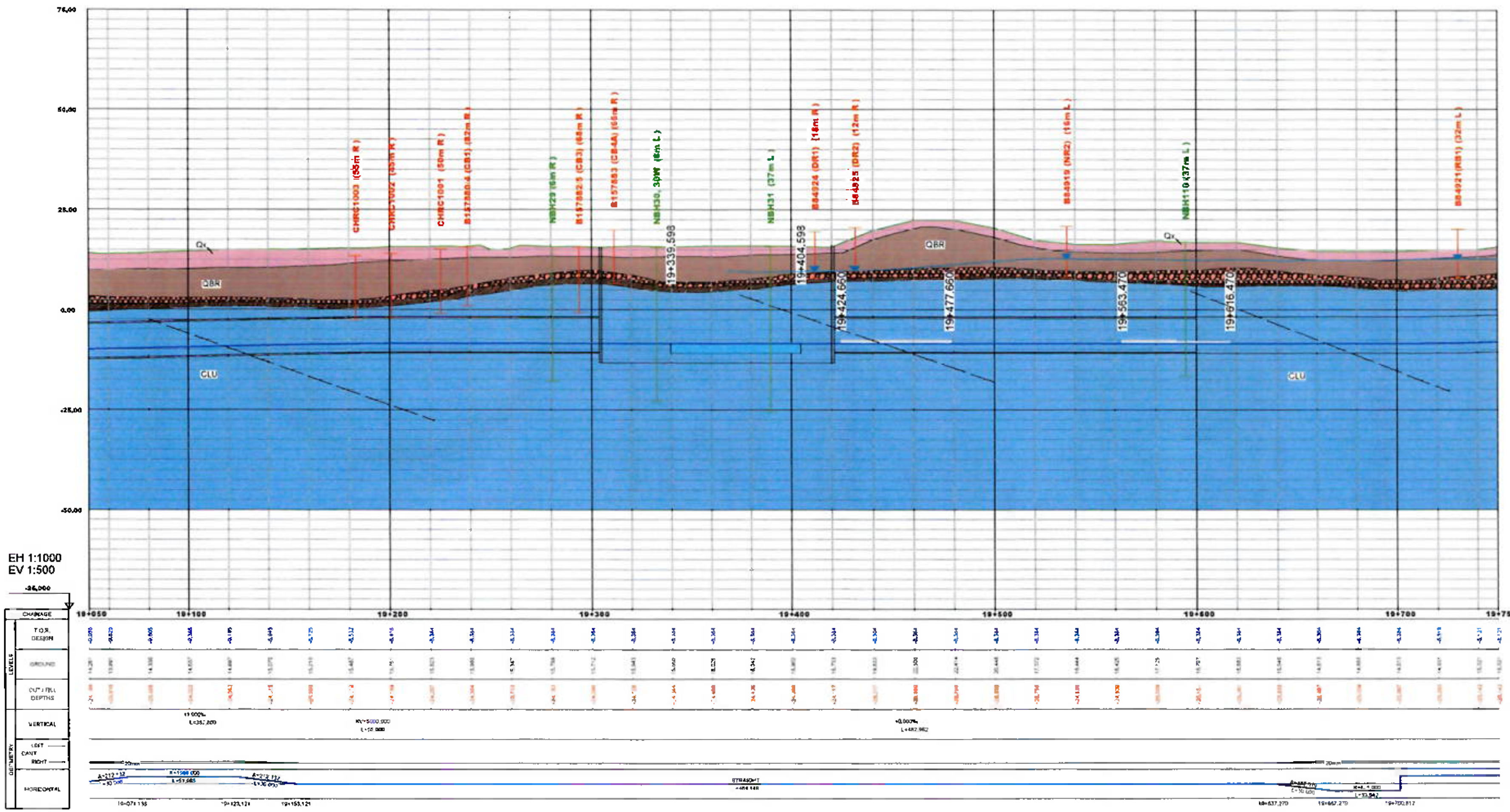
SYMBOLOGY

Cut and cover section

TOR — Rail level (TOR)

STATION CHARLEMONT

EH 1:1000
EV 1:500



Key Plan

REV.	DATE	Description	DRAWN BY	CHECKED BY	REVIEWED BY	APPROVED BY
P03	11/12/20	Final Preliminary Design	LT	FP	JB	NC
P02	03/04/20	Preliminary Design	LT	FP	JB	NC
P01	05/12/19	Preliminary Design	LT	FP	JB	NC

Client: TII, NTA

Project: METROLINK

Consultant: JACOBS, IDOM

Drawing Title: Ground Investigation Geological Long Section – Phase 1

Drawing Status: S4

Drawing N°: ML1-JAI-GEO-ROUT_XX-DR-Y-00013

Scales Original Size A1

Sheet 27 of 28

2. General Description of Underground Structures

2.1 TBM Tunnels

The MetroLink tunnel alignment will consist of a single bore bi-directional tunnel constructed by means of a tunnel boring machine (TBM) that will be specified and designed to enable tunnelling generated ground movements to be minimalised. The tunnels have been designed with an internal diameter of 8.5m, determined by the rolling stock kinematic clearance, and railway services requirement (See Figure 2-1).

From a review of the expected geology and hydrogeology along the tunnel alignment, the construction and logistics constraints, and the anticipated TBM operational procedures, it is considered likely that a variable density (VD) TBM or a Mix Shield TBM will be selected. It therefore follows that the main characteristics of the TBM required to meet the tunnel requirements will be as follows.

- Diameter of the cutter head: 9.53m
- Diameter of the frontal shield: 9.50m
- Diameter at the rear of the shield: 9.48m
- Shield length (approx.): 10.00m
- Diametrical gap (outer diameter of excavation): 0.33m
- Minimum radius of curvature: 300m (Note: minimum alignment curve radius is 350m).

The tunnel lining itself has been assessed for all relevant ground loading conditions, manufacturing loads (demoulding, storage, and transportation) and segment installation during ring-build. The main characteristics of the segments are:

- Typology: Universal ring.
- Thickness: 35 cm.
- Concrete class: C40/50
- Reinforcement: Steel bars class C + steel fibres (Model Code FRC 4e)
- Fire protection: Polypropylene fibres

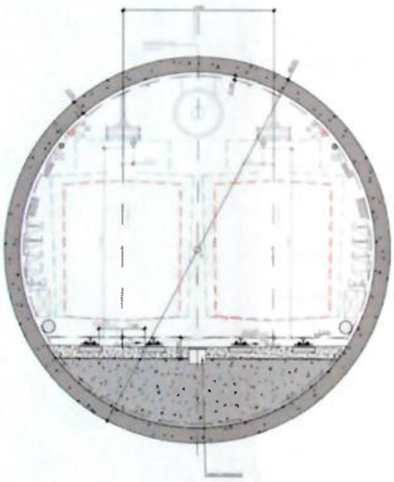
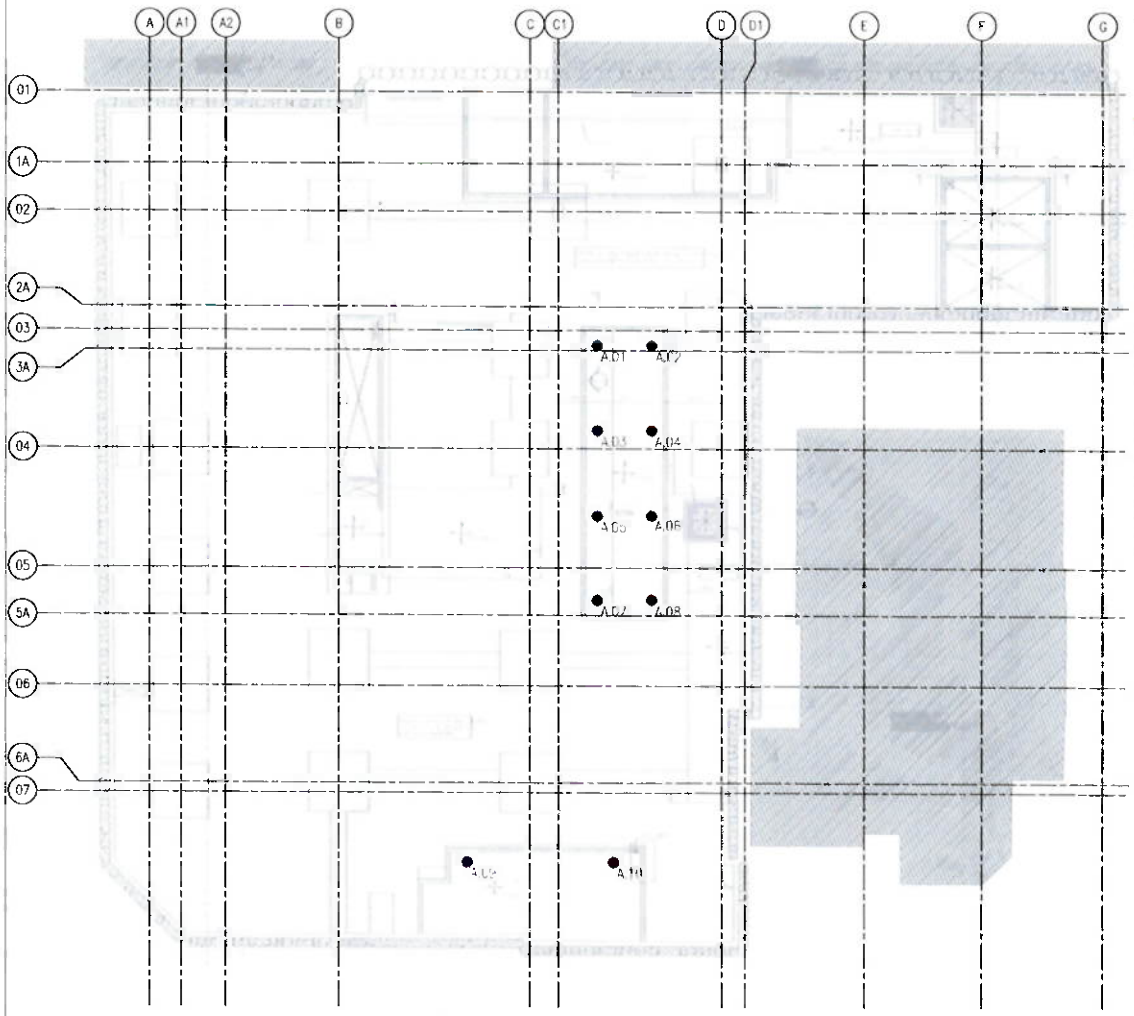


Figure 2-1: TBM Tunnel Geometry

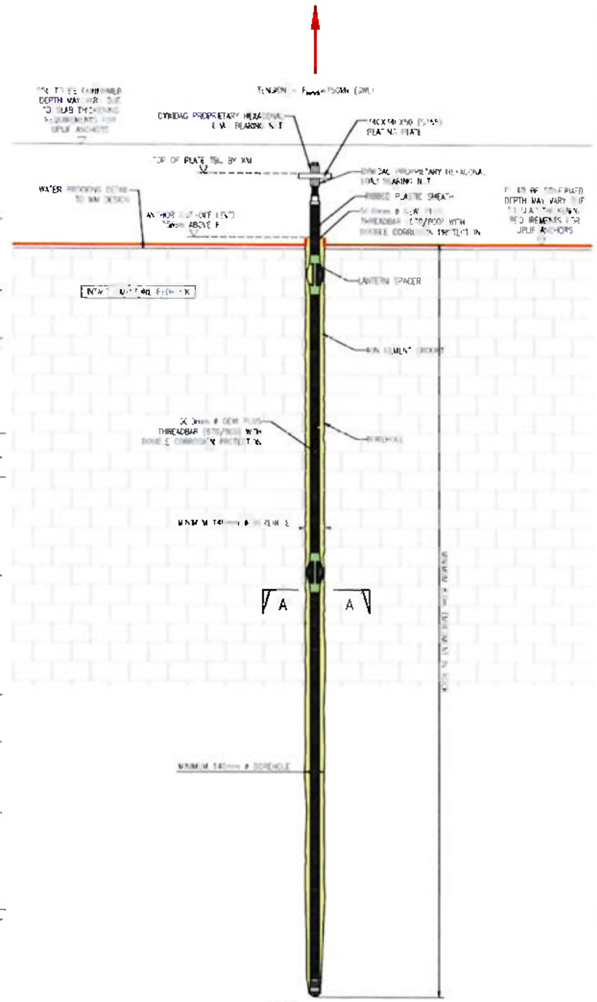
2.2 Non-TBM Underground Structures

Five galleries, comprising two ventilation and three emergency galleries will be required, connected to the main tunnels within the curtilage of Dublin Airport and at Albert College Park Intervention Shaft. These will enable

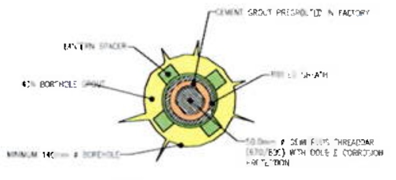
A1



UPLIFT ANCHOR LAYOUT
1:150 SCALE 1:150



SECTION
ANCHOR TYPE 1



SECTION A-A
SCALE

GENERAL NOTES

1. DO NOT SCALE OF THE DRAWING
2. ALL DIMENSION IN MILLIMETERS UNLESS OTHERWISE NOTED
3. ALL LEVELS ARE METERS [illegible] UNLESS OTHERWISE NOTED
4. [illegible] BY OTHERS SHOWN INDICATIVELY ONLY
5. ALL ANCHORS DESIGNED IN ACCORDANCE WITH BS 8081:2015 – CODE OF PRACTICE FOR GROUND ANCHORAGES
6. ANCHORS TO BE 140mm IN DIAMETER [illegible] TO SUPPORT THE PERMANENT TENSION LOAD OF [illegible] kN [illegible]
7. MAXIMUM [illegible] SIZE OF [illegible]
8. REFER TO TYPICAL [illegible] FOR THE DETAIL OF ANCHOR
9. TEMPORARY DEWATERING OF EXCAVATION IS THE RESPONSIBILITY OF THE PERMANENT WORKS CONTRACTOR
10. [illegible] HAS BEEN INTERPRETED [illegible]
11. A [illegible] TYPE ANCHOR IS DETAILED HERE WITH NO PRE-STRESS OR LOCK OFF LOAD REQUIRED
12. [illegible] TENSION TESTS ON 5% OF PRODUCTION ANCHORS [illegible] SUITABILITY TESTS TO A MAXIMUM LOAD [illegible] CARRIED OUT AS PER BS 8081 [illegible] METHOD [illegible] THIS TEST COMPRISES [illegible] LOAD CYCLES TO PROOF LOAD [illegible] LOAD STAGE IS ALSO TO BE CARRIED OUT WITH [illegible] MONITORING FOR [illegible] 250kN
13. THE CEMENT GROUT SHALL HAVE A MINIMUM [illegible] STRENGTH OF 40N/MM2. GROUT SHALL COMPRISE ORDINARY PORTLAND CEMENT TO EN 197-1 [illegible] WITH A MINIMUM 28-DAY [illegible] STRENGTH OF 40 N/MM2. THE GROUT MIX SHALL BE MIXED WITH 45 LITRES OF WATER FOR EVERY 100 KG OF CEMENT (0.45 W/C RATIO). GROUT CUBE [illegible] AND TESTING TO VERIFY THE CHARACTERISTIC 28-DAY COMPRESSIVE STRENGTH OF THE GROUT SHALL COMPRISE AT LEAST MINIMUM 2 NO. [illegible] GROUT TO BE TAKEN AND TESTED FOR EVERY 10 NO. ANCHORS, OR 2 NO. CUBES TAKEN PER DAY, WHICHEVER IS THE GREATER. AT LEAST 1 NO. CUBES TAKEN DURING THE CONTRACT ARE TO BE TESTED AFTER 7 DAYS
14. THE PERMANENT WATER PROOFING DETAIL AT THE UPLIFT ANCHOR PROJECTION INTO THE BASEMENT SLAB IS TO [illegible] REQUIREMENTS
15. [illegible] BE DESIGNED BY OTHERS [illegible] PUNCHING SHEAR AT ANCHOR LOCATION

Rev	Date	Description	By	Chk	App
01	09/04	ISSUED FOR REVIEW	AI	PG	MR
00	20/03	ISSUED FOR REVIEW	AI	PG	MR

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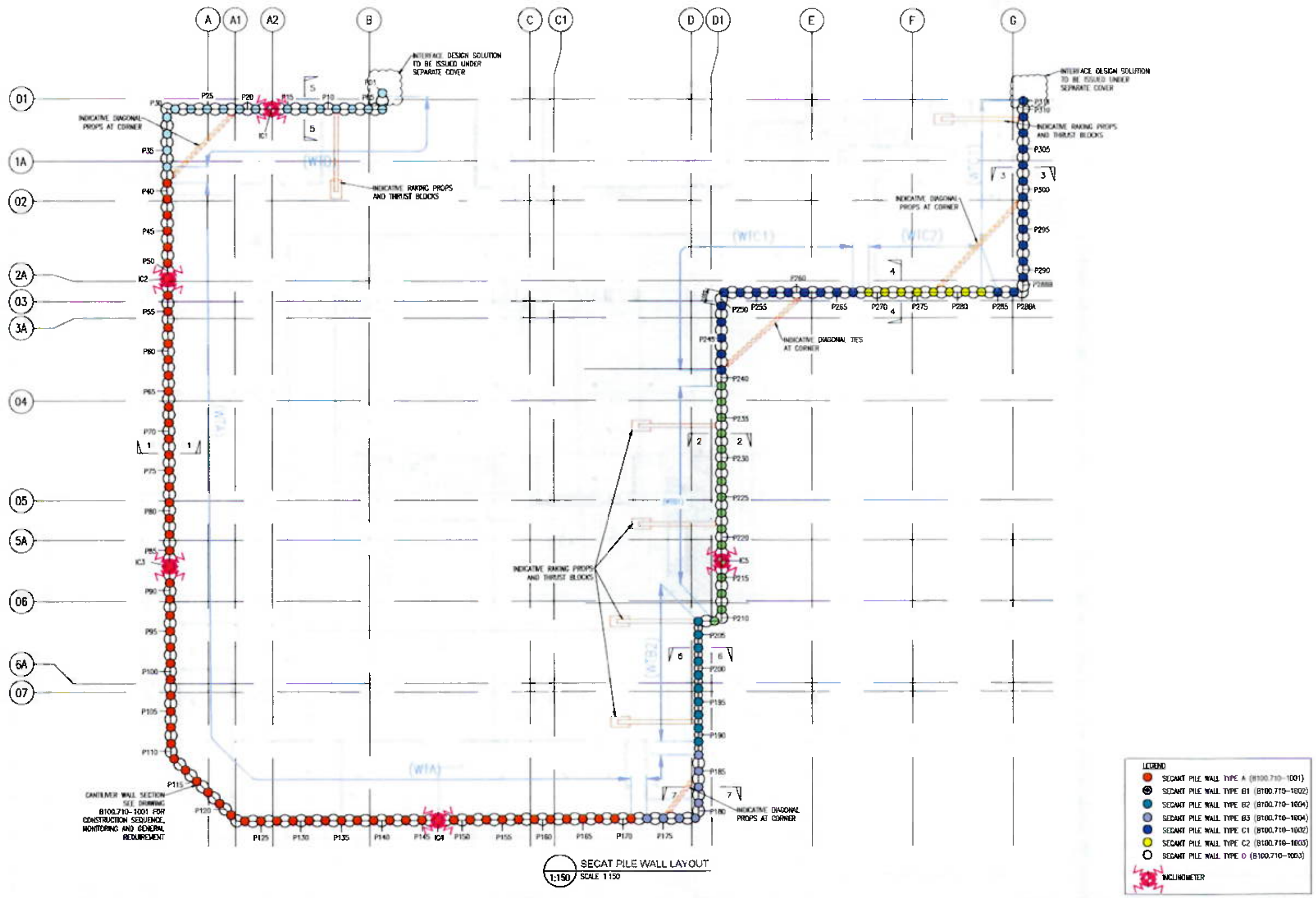
CLIENT
WALLS CONSTRUCTION Ltd.

PROJECT
No. 1 EARLSFORT TERRACE

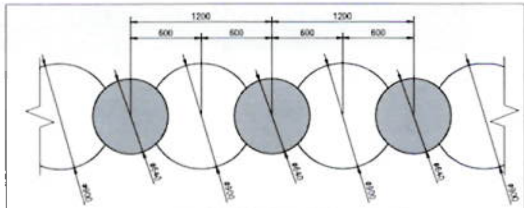
DRAWING TITLE
PLAN LAYOUT
UPLIFT ANCHOR SECTION & DETAILS

STATUS
FOR REVIEW

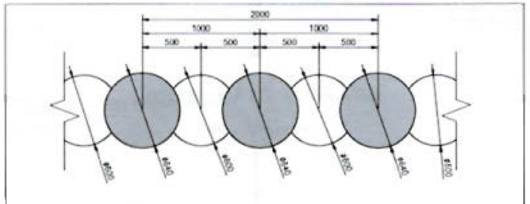
Date: 20/03/20	Scale: AS SHOWN	Drawn: AI	Chk: PG	App: MR
Project No: B100.710	Dwg. No: EARL-BLP-00-FN-DR-GT-0001 (BL Ref: B100.710-3000)			Rev: P02



SECAT PILE WALL LAYOUT
1:150 SCALE 1:150



SECANT PILE WALL TYPICAL SETOUT
1:20 SCALE



SECANT PILE WALL TYPICAL SETOUT (WTB2)
1:20 SCALE

⚠ BYRNELOOBY ARE NOT RESPONSIBLE FOR WALL SET-OUT. PILE SET OUT SHOULD ALLOW FOR PILE INSTALLATION TOLERANCE AND EXTENTS OF PILING TO BE CONFIRMED BY WATERMAN MOYLAN

⚠ ANY GROUNDWATER INGRESS THAT OCCURS THROUGH THE ROCK BENEATH THE SOFT SECANT PILES IS THE RESPONSIBILITY OF THE MAIN CONTRACTOR

GENERAL NOTES

GENERAL NOTES:

1. DO NOT SCALE FROM THIS DRAWING - USE FIGURED DIMENSIONS ONLY
2. UNLESS NOTED OTHERWISE, ALL DIMENSIONS ARE IN MILLIMETRES (mm) AND ALL LEVELS ARE STATED IN METERS (m OD)
3. ALL PILES ARE DESIGNED IN ACCORDANCE WITH IS EN 1997-1:2004 EUROCODE 7: GEOTECHNICAL DESIGN PART 1: GENERAL RULES AND THE IRISH NATIONAL ANNEX TO IS EN 1997-1:2004.
4. MINIMUM CONCRETE GRADE OF C30/37 AND EXPOSURE CLASS XA2/XC2 TO BE USED IN THE HARD SECANT PILES IN ACCORDANCE WITH IS EN 206-1:2013. MINIMUM CONCRETE GRADE OF C8/10 MIX TO BE USED IN SOFT PILES. STRICT QUALITY CONTROL MEASURES ARE TO BE IMPLEMENTED ON SITE FOR BOTH HARD AND SOFT SECANT PILES (I.E. CUBE TESTS AND SLUMP TESTS ETC). THE CONTRACTOR IS TO ENSURE SAME QUALITY CONTROL MEASURES ARE USED FOR THE SOFT PILES AS PER THE HARD PILES. THE SECANT PILES ARE DESIGNED FOR A DESIGN LIFE OF 100 YEARS.
5. PILES TO BE 640/900mm DIAMETER. ALL PILES REINFORCED AS PER TYPICAL PILE SECTIONS. UNLESS STATED OTHERWISE, MINIMUM COVER OF REINFORCEMENT TO BE 75mm.
6. PILE LENGTHS AND TOE DEPTHS AS PER TYPICAL SECTIONS PROVIDED.
7. REINFORCEMENT LAP LENGTHS ARE TO BE AS FOLLOWS:
   Ø25 = 1000mm
   Ø20 = 800mm.
8. FOR ALL PILE TYPES, CAGES TO EXTEND INTO PILE CAPS TO PROVIDE FULL TENSION LAP AS PER TYPICAL SECTIONS.
9. ALL PILES TO BE REINFORCED WITH HELICAL LINKS TO FULL LENGTH OF REINFORCEMENT CAGE AS PER TYPICAL SECTIONS.
10. MAXIMUM AGGREGATE SIZE TO BE 20mm.
11. PILE SETTING OUT DETAILS AND LAYOUTS ARE TO OTHERS DESIGN, BYRNELOOBY ARE NOT RESPONSIBLE FOR SETTING OUT OF PILES. PILES SHOWN INDICATIVELY ON DRAWINGS.
12. PILE CAPS TO OTHERS DESIGN.
13. PILE LENGTHS AND TOE DEPTHS AS PER TYPICAL SECTIONS PROVIDED
14. GROUND STRATIGRAPHY EXPECTED AS PER PILE SECTIONS WHICH HAVE BEEN BASED ON AVAILABLE SITE INVESTIGATION DATA PROVIDED TO BYRNELOOBY. ANY VARIATION IN THE GROUND CONDITIONS TO BE COMMUNICATED TO BYRNELOOBY IMMEDIATELY.
15. PILE BREAKDOWN TO BE CARRIED OUT IN A SAFE, CONTROLLED AND CAREFUL MANNER AFTER A MINIMUM OF 3 DAYS FOLLOWING PILE INSTALLATION.
16. THE CONTRACTOR SHALL REMOVE ALL OBSTRUCTIONS DUE TO THE EXISTING BASEMENT/FOUNDATION STRUCTURES TO FACILITATE THE PILE/TEMPORARY PROP INSTALLATION
17. DRAWING TO BE READ INCONJUNCTION WITH DESIGN REPORT B100.703-GEO-R001
18. THE INFLUENCE OF PREDICTED WALL MOVEMENTS ON NEARBY STRUCTURES TO BE ASSESSED BY WATERMAN MOYLAN
19. THE CONTRACTOR SHALL PROVIDE ADEQUATE MEASURES TO ENSURE EXCAVATION DEPTHS DO NOT EXCEED THE FORMATION LEVELS ILLUSTRATED IN THE DRAWING
20. SET OUT OF PILES TO ALLOW FOR 25MM TOLERANCE AND 1:100 RAKING TOLERANCE OVER RETAINED HEIGHT

Rev	Date	Description	By	Chk	App
02	07/01	GENERAL REVISION	AI	PC	MR
01	27/01	GENERAL REVISION	AI	PC	MR
00	07/01	FOR INFORMATION	AI	PC	MR

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CLIENT
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PROJECT
1 EARLSFORT TERRACE

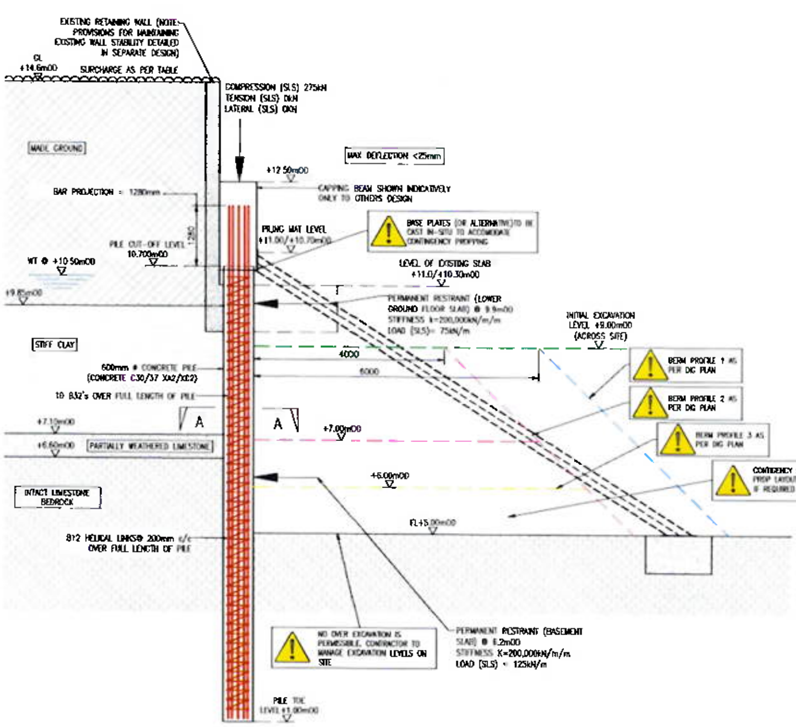
DRAWING TITLE
SECANT PILE WALL PLAN LAYOUT

STATUS
FOR INFORMATION

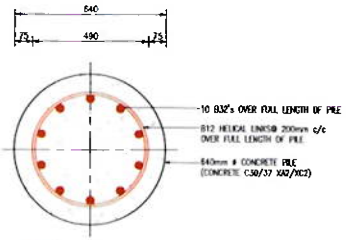
Date 07/01/20	Scale AS SHOWN	Drawn AI	Chk PC	Appr MR
Project No: B100.710	Drg. No: B100.710-1000			Rev: 02

A1

A1

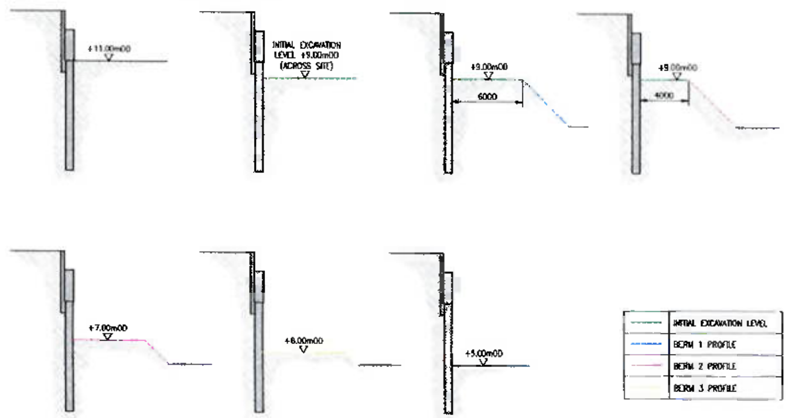


WTA 1:50 SECTION 1-1 - WALL TYPE A
SCALE 1:50



WTA 1:10 SECTION A-A
SCALE

EXCAVATION SEQUENCE



MOVEMENT MONITORING TRIGGER LEVELS

Trigger Levels for Wall Type A				
Formation Level, m OD (Dig Depth, m)	Berm Profile	Lateral Movement (mm)		
		Green	Amber	Red
9.0 (2.0)	1	5	5 - 8	>8
7.0 (4.0)	2	10	10 - 16	>16
6.0 (5.0)	3	20	20 - 28	>28
5.0 (6.0)	4 (Formation Level)	25	25 - 35	>35

Trigger Levels for Wall Type A		
Level	Reading	Action
Green	<20mm	Continue to monitor twice weekly
Amber	20-30mm	Increase monitoring to daily if excavation is completed. Install relief wells if excavation is not at formation level
Red	>35mm	Install contingency measures

BASIS OF DESIGN NOTES FOR PROPOSED CONSTRUCTION SEQUENCE

1. TO MINIMISE THE RISK THAT THE ALLOWABLE PILE MOVEMENTS ARE NOT EXCEEDED ADJACENT TO EARLSFORT TERRACE AND ADELAIDE ROAD, THE FOLLOWING MEASURES ARE PROPOSED AS A CONTINGENCY.
2. THE BYRNELOOBY DESIGN IS PROVIDED ON THE BASIS THAT CONTINGENCY MEASURES CAN BE RAPIDLY IMPLEMENTED SHOULD ALLOWABLE MOVEMENTS BE EXCEEDED, OR EXPECTED TO BE EXCEEDED.
3. A PROPOSED CONTINGENCY PLAN IS PRESENTED HERE. ALTERNATIVE REMEDIAL OPTIONS CAN ALSO BE DEVELOPED BY OTHERS.
4. BYRNELOOBY PROPOSAL CONSISTS ESSENTIALLY OF:
   - BASE PLATES WITH APPROPRIATE CONNECTIONS CAST IN-SITU WITH CAPPING BEAM AND THRUST BLOCKS, READY TO ACCEPT PROPS IF REQUIRED
   - PROPS AVAILABLE ON SITE, READY TO BE INSTALLED RAPIDLY, IF NEEDED
   - SUITABLE LABOUR, PLANT AND EQUIPMENT AVAILABLE TO INSTALL PROPS IF REQUIRED
   - APPROPRIATE CONTINUOUS MONITORING REGIME TO BE IMPLEMENTED - SEE NOTES BELOW
5. IT SHOULD BE NOTED THAT THERE IS THE POTENTIAL FOR MOVEMENTS IN THE REGION OF 40-50MM IN THIS SECTION. BYRNELOOBY HAVE OUTLINED CONTINGENCY MEASURES ON THIS DRAWING TO ENSURE MOVEMENTS DO NOT EXCEED THE MAX ALLOWABLE VALUE OF 30mm. SHOULD IT BE DECIDED TO BUILD THE BASEMENT WITHOUT THE CONTINGENCY MEASURES BEING READILY AVAILABLE TO PUT IN PLACE, THERE IS A RISK OF SIGNIFICANT MOVEMENT OCCURING.

WALL TYPE A CONSTRUCTION SEQUENCE

1. INSTALL PILING PLATFORM TO REQUIRED THICKNESS AND PLATFORM LEVEL.
2. CONSTRUCT GUIDEWALL TO INSTALL SECANT PILES (BASED ON WM/ARCHITECTS SETOUT).
3. INSTALL 900MM DIAMETER FIRM SECANT CFA PILES AT 1.20M C/C AS PER THE DRAWING SECTION DETAILS TO REFUSAL IN THE BEDROCK.
4. INSTALL 640MM DIAMETER DDCK/CFA SECANT HARD PILES AT 1.20M C/C AS PER THE DRAWING SECTION DETAILS.
5. EXCAVATE TO PILE CUT OFF LEVEL AND CONSTRUCT RC CAPPING BEAM. ENSURE BASE PLATES (OR ALTERNATIVE DETAILS) FOR CONTINGENCY PROPS ARE INSTALLED ON CAPPING BEAM
6. EXCAVATE TO FORMATION LEVEL IN THE AREAS ADJACENT TO WALL TYPE A, HOWEVER ENSURING THAT BERM PROFILE 1 IS LEFT IN PLACE AS PER THE SECTION DRAWING. RECORD WALL MOVEMENTS AND ENSURE THEY WITHIN PERMISSIBLE LIMITS AS SET OUT BELOW, OTHERWISE CARRY OUT CONTINGENCY MEASURES
7. INSTALL THRUST BLOCKS WITH BASE PLATES READY TO ACCEPT TEMPORARY PROPPING AT FORMATION LEVEL, AT THE BASE OF BERM PROFILE 1 (OR ALTERNATIVE DETAILS)
8. CONTINUE BERM EXCAVATION TO PROFILE 2 AS PER SECTION DRAWING, RECORDING MOVEMENTS AS EXCAVATION WORKS PROGRESS. PROVIDED MOVEMENTS ARE WITHIN LIMITS, CONTINUE TO STAGE 9, OTHERWISE CARRY OUT CONTINGENCY MEASURES
9. CONTINUE BERM EXCAVATION TO PROFILE 3 AS PER SECTION DRAWING, RECORDING MOVEMENTS AS EXCAVATION WORKS PROGRESS. PROVIDED MOVEMENTS ARE WITHIN LIMITS, CONTINUE TO STAGE 10. OTHERWISE CARRY OUT CONTINGENCY MEASURES.
10. SHOULD MOVEMENTS STAY WITHIN PERMISSIBLE LIMITS CONTINUE EXCAVATION TO FORMATION LEVEL CONTINUE TO MONITOR WALL MOVEMENTS
11. ONCE EXCAVATION IS COMPLETE MONITORING TO CONTINUE, WITH INCLINOMETRES TO BE MEASURED TWICE WEEKLY UNTIL BASEMENT SLAB IS CONSTRUCTED. AT ANY STAGE SHOULD RED TRIGGER LEVELS BE EXCEEDED, CONTINGENCY MEASURES TO BE IMPLEMENTED IMMEDIATELY
12. CONSTRUCT RC BASEMENT FOUNDATION SLAB
13. CONSTRUCT RC LINER WALL FROM TOP OF BASEMENT SLAB SSL TO UNDERSIDE OF LOWER GROUND FLOOR RC SLAB.
14. CONSTRUCT RC LOWER GROUND FLOOR SLAB
15. CONSTRUCT THE REMAINING SUPERSTUCTURE ABOVE LOWER GROUND FLOOR LEVEL.

Secant Pile Wall Surcharge loads							
Wall Type	Adjacent Road Level (m OD)	Piling Platform Level (m OD)	SLS Ground Surcharge (kPa)		SLS Line Load from Adjacent Wall (kN/m)		Additional Moment from Eccentric load due to Superstructure (kNm/m)
			DL	LL	DL	LL	
A	14.7	11.0	0	10	-	-	200
B1	n/a	11.0	55	25	230	40	-
B2	n/a	11.0	55	25	230	40	300
B3	n/a	11.0	55	25	230	40	-
C1	12.7	12.7	0	10	-	-	-
C2	12.7	12.7	0	10	-	-	-
D	n/a	11.4	55	25	160	65	500

GENERAL NOTES

GENERAL NOTES:

1. DO NOT SCALE FROM THIS DRAWING - USE FIGURED DIMENSIONS ONLY
2. UNLESS NOTED OTHERWISE, ALL DIMENSIONS ARE IN MILLIMETRES (mm) AND ALL LEVELS ARE STATED IN METERS (m OD)
3. ALL PILES ARE DESIGNED IN ACCORDANCE WITH IS EN 1997-1:2004 EUROCODE 7: GEOTECHNICAL DESIGN PART 1 GENERAL RULES AND THE IRISH NATIONAL ANNEX TO IS EN 1997-1:2004.
4. MINIMUM CONCRETE GRADE OF C30/37 AND EXPOSURE CLASS XA2/XC2 TO BE USED IN THE HARD SECANT PILES IN ACCORDANCE WITH IS EN 206-1:2013. MINIMUM CONCRETE GRADE OF C8/10 MIX TO BE USED IN SOFT PILES. STRICT QUALITY CONTROL MEASURES ARE TO BE IMPLEMENTED ON SITE FOR BOTH HARD AND SOFT SECANT PILES (I.E. CUBE TESTS AND SLUMP TESTS ETC). THE CONTRACTOR IS TO ENSURE SAME QUALITY CONTROL MEASURES ARE USED FOR THE SOFT PILES AS PER THE HARD PILES. THE SECANT PILES ARE DESIGNED FOR A DESIGN LIFE OF 100 YEARS.
5. PILES TO BE 640/900mm DIAMETER ALL PILES REINFORCED AS PER TYPICAL PILE SECTIONS. UNLESS STATED OTHERWISE, MINIMUM COVER OF REINFORCEMENT TO BE 75mm.
6. PILE LENGTHS AND TOE DEPTHS AS PER TYPICAL SECTIONS PROVIDED.
7. REINFORCEMENT LAP LENGTHS ARE TO BE AS FOLLOWS:
   B25 = 1000mm.
   B20 = 800mm.
8. FOR ALL PILE TYPES, CAGES TO EXTEND INTO PILE CAPS TO PROVIDE FULL TENSION LAP AS PER TYPICAL SECTIONS.
9. ALL PILES TO BE REINFORCED WITH HELICAL LINKS TO FULL LENGTH OF REINFORCEMENT CAGE AS PER TYPICAL SECTIONS.
10. MAXIMUM AGGREGATE SIZE TO BE 20mm.
11. PILE SETTING OUT DETAILS AND LAYOUTS ARE TO OTHERS DESIGN, BYRNELOOBY ARE NOT RESPONSIBLE FOR SETTING OUT OF PILES. PILES SHOWN INDICATIVELY ON DRAWINGS.
12. PILE CAPS TO OTHERS DESIGN.
13. PILE LENGTHS AND TOE DEPTHS AS PER TYPICAL SECTIONS PROVIDED
14. GROUND STRATIGRAPHY EXPECTED AS PER PILE SECTIONS WHICH HAVE BEEN BASED ON AVAILABLE SITE INVESTIGATION DATA PROVIDED TO BYRNELOOBY. ANY VARIATION IN THE GROUND CONDITIONS TO BE COMMUNICATED TO BYRNELOOBY IMMEDIATELY.
15. PILE BREAKDOWN TO BE CARRIED OUT IN A SAFE, CONTROLLED AND CAREFUL MANNER AFTER A MINIMUM OF 3 DAYS FOLLOWING PILE INSTALLATION.
16. THE CONTRACTOR SHALL REMOVE ALL OBSTRUCTIONS DUE TO THE EXISTING BASEMENT/FOUNDATION STRUCTURES TO FACILITATE THE PILE/TEMPORARY PROP INSTALLATION
17. DRAWING TO BE READ INCONJUCTION WITH DESIGN REPORT B100.703-GED-R001
18. THE INFLUENCE OF PREDICTED WALL MOVEMENTS ON NEARBY STRUCTURES TO BE ASSESSED BY WATERMAN MOYLAN
19. THE CONTRACTOR SHALL PROVIDE ADEQUATE MEASURES TO ENSURE EXCAVATION DEPTHS DO NOT EXCEED THE FORMATION LEVELS ILLUSTRATED IN THE DRAWING
20. SET OUT OF PILES TO ALLOW FOR 25MM TOERLANCE AND 1:100 RAKING TOLERANCE OVER RETAINED HEIGHT

Rev	Date	Description	By	Chk	App
03	25/02	GENERAL REVISION	AI	PC	MR
02	05/02	GENERAL REVISION	AI	PC	MR
01	27/01	FOR INFORMATION	AI	PC	MR
00	07/01	FOR INFORMATION	AI	PC	MR

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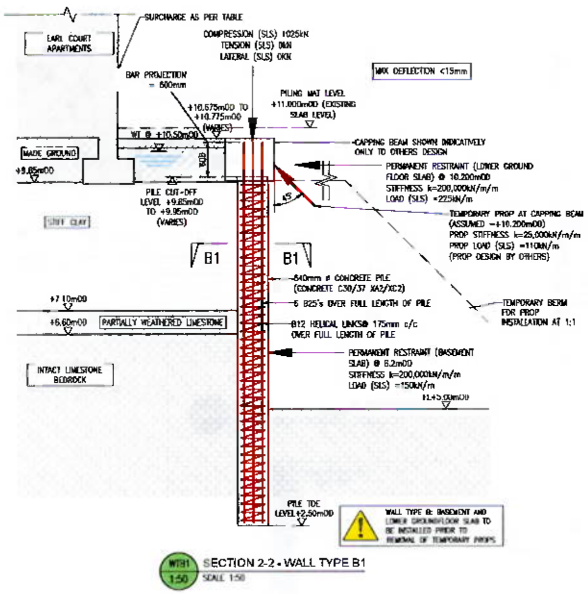
PROJECT
1 EARLSFORT TERRACE

DRAWING TITLE
SECTIONS AND DETAILS
WALL TYPE A

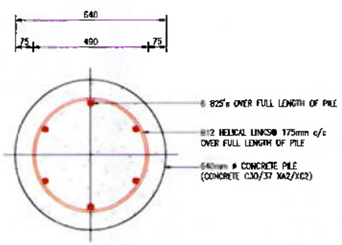
STATUS
FOR INFORMATION

Date: 07/01/20	Scale: AS SHOWN	Drawn: AI	Chk: PC	App: MR
Project No: B100.710	Drg. No: B100.710-1001			Rev: 03



SECTION 2-2 - WALL TYPE B1

SCALE 1:50



SECTION B1-B1

SCALE

CONSTRUCTION SEQUENCE WALL TYPE B1 AND B2

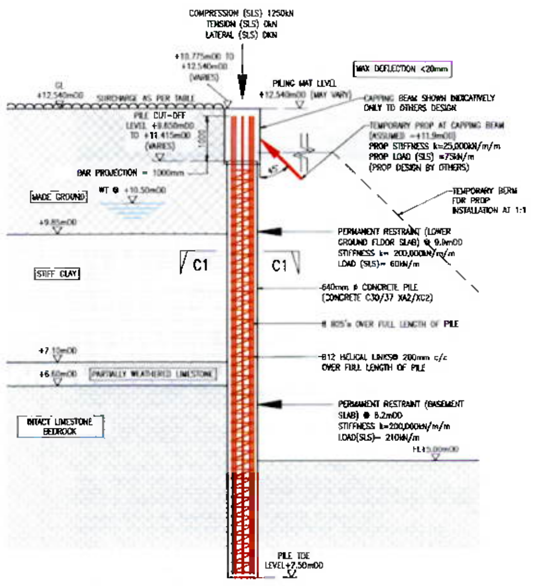
1. INSTALL PILING PLATFORM TO REQUIRED THICKNESS AND PLATFORM LEVEL.
2. CONSTRUCT GUIDEWALL TO INSTALL SECANT PILES (BASED ON WM/ARCHITECTS SETOUT).
3. INSTALL 600/900mm DIAMETER FIRM SECANT CFA PILES AT 1.00/1.20m c/c AS PER THE DRAWING SECTION DETAILS TO REFUSAL IN THE BEDROCK.
4. INSTALL 640MM DIAMETER DOCK/CFA SECANT HARD PILES AT 1.00/1.20m c/c AS PER THE DRAWING SECTION DETAILS.
5. EXCAVATE TO PILE CUT OFF LEVEL AND CONSTRUCT RC CAPPING BEAM
6. EXCAVATE A SOIL BERM DIRECTLY IN FRONT OF THE SECANT PILE WALL WITH THE FOLLOWING DIMENSIONS; TOP OF BERM TO BE APPROX. 0.5M BELOW BOTTOM OF CAPPING BEAM, BENCH WIDTH OF MINIMUM 4.0m, BATTERED SLOPES AT 45 DEGREES DOWN TO FORMATION LEVEL.
7. EXCAVATE A SLIT TRENCH IN THE SOIL BERM IN A HIT AND MISS SEQUENCE AND CONSTRUCT RC THRUST BLOCKS.
8. INSTALL TEMPORARY RAKING PROPS TO SUPPORT SECANT PILE WALL AT CAPPING BEAM LEVEL IN A HIT AND MISS SEQUENCE.
9. EXCAVATE TOP OF SOIL BERM DOWN TO BASEMENT FORMATION LEVEL – MAIN CONTRACTOR SHALL EMPLOY SAFE WORKING MEASURES TO ENSURE BASEMENT FORMATION LEVELS ILLUSTRATED ON DRAWINGS ARE NOT EXCEEDED – NO OVER EXCAVATION BELOW THE FORMATION LEVELS IS PERMISSIBLE.
10. CONSTRUCT RC BASEMENT FOUNDATION SLAB
11. CONSTRUCT RC LINER WALL FROM TOP OF BASEMENT SLAB SSL TO UNDERSIDE OF LOWER GROUND FLOOR RC SLAB.
12. CONSTRUCT RC LOWER GROUND FLOOR SLAB
13. REMOVE SUPPORT FROM TEMPORARY RAKING PROPS.
14. CONSTRUCT THE REMAINING SUPERSTUCTURE ABOVE LOWER GROUND FLOOR LEVEL.

CONSTRUCTION SEQUENCE WALL TYPE C1 AND C2

1. INSTALL PILING PLATFORM TO REQUIRED THICKNESS AND PLATFORM LEVEL.
2. CONSTRUCT GUIDEWALL TO INSTALL SECANT PILES (BASED ON WM/ARCHITECTS SETOUT).
3. INSTALL 900mm DIAMETER FIRM SECANT CFA PILES AT 1.20m c/c AS PER THE DRAWING SECTION DETAILS TO REFUSAL IN THE BEDROCK.
4. INSTALL 640MM DIAMETER DOCK/CFA SECANT HARD PILES AT 1.20M C/C AS PER THE DRAWING SECTION DETAILS.
5. EXCAVATE TO PILE CUT OFF LEVEL AND CONSTRUCT RC CAPPING BEAM
6. EXCAVATE A SOIL BERM DIRECTLY IN FRONT OF THE SECANT PILE WALL WITH THE FOLLOWING DIMENSIONS; TOP OF BERM TO BE APPROX. 0.5M BELOW BOTTOM OF CAPPING BEAM, BENCH WIDTH OF MINIMUM 4.0m, BATTERED SLOPES AT 45 DEGREES DOWN TO FORMATION LEVEL.
7. EXCAVATE A SLIT TRENCH IN THE SOIL BERM IN A HIT AND MISS SEQUENCE AND CONSTRUCT RC THRUST BLOCKS.
8. INSTALL TEMPORARY RAKING PROPS TO SUPPORT SECANT PILE WALL AT CAPPING BEAM LEVEL IN A HIT AND MISS SEQUENCE.
9. EXCAVATE TOP OF SOIL BERM DOWN TO BASEMENT FORMATION LEVEL – MAIN CONTRACTOR SHALL EMPLOY SAFE WORKING MEASURES TO ENSURE BASEMENT FORMATION LEVELS ILLUSTRATED ON DRAWINGS ARE NOT EXCEEDED – NO OVER EXCAVATION BELOW THE FORMATION LEVELS IS PERMISSIBLE.
10. CONSTRUCT RC BASEMENT FOUNDATION SLAB
11. REMOVE SUPPORT FROM TEMPORARY RAKING PROPS.
12. CONSTRUCT RC LINER WALL FROM TOP OF BASEMENT SLAB SSL TO UNDERSIDE OF LOWER GROUND FLOOR RC SLAB.
13. CONSTRUCT RC LOWER GROUND FLOOR SLAB
14. CONSTRUCT THE REMAINING SUPERSTUCTURE ABOVE LOWER GROUND FLOOR LEVEL.

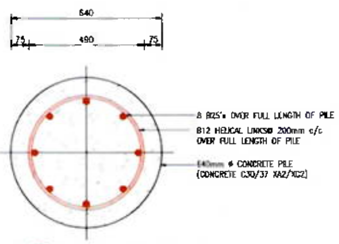
Secant Pile Wall Surcharge loads

Wall Type	Adjacent Road Level (m OD)	Piling Platform Level (m OD)	SLS Ground Surcharge (kPa)		SLS Line Load from Adjacent Wall (kN/m)		Additional Moment from Eccentric load due to Superstructure (kNm/m)
			[illegible]	[illegible]	[illegible]	[illegible]	
A	14.7	11.0	0	10	-	-	200
B1	n/a	11.0	55	25	230	40	-
B2	n/a	11.0	55	25	230	40	300
B3	n/a	11.0	55	25	230	40	-
C1	12.7	12.7	0	10	-	-	-
C2	12.7	12.7	0	10	-	-	-
D	n/a	11.4	55	25	160	65	500



SECTION 3-3 - WALL TYPE C1

SCALE 1:50



SECTION C1-C1

SCALE

GENERAL NOTES

GENERAL NOTES:

1. DO NOT SCALE FROM THIS DRAWING – USE FIGURED DIMENSIONS ONLY
2. UNLESS NOTED OTHERWISE, ALL DIMENSIONS ARE IN MILLIMETRES (mm) AND ALL LEVELS ARE STATED IN METERS (m OD)
3. ALL PILES ARE DESIGNED IN ACCORDANCE WITH IS EN 1997-1:2004 EUROCODE 7, GEOTECHNICAL DESIGN PART 1: GENERAL RULES AND THE IRISH NATIONAL ANNEX TO IS EN 1997-1:2004.
4. MINIMUM CONCRETE GRADE OF C30/37 AND EXPOSURE CLASS XA2/XC2 TO BE USED IN THE HARD SECANT PILES IN ACCORDANCE WITH IS EN 206-1:2013. MINIMUM CONCRETE GRADE OF C8/10 MIX TO BE USED IN SOFT PILES. STRICT QUALITY CONTROL MEASURES ARE TO BE IMPLEMENTED ON SITE FOR BOTH HARD AND SOFT SECANT PILES (I.E. CUBE TESTS AND SLUMP TESTS ETC.). THE CONTRACTOR IS TO ENSURE SAME QUALITY CONTROL MEASURES ARE USED FOR THE SOFT PILES AS PER THE HARD PILES. THE SECANT PILES ARE DESIGNED FOR A DESIGN LIFE OF 100 YEARS.
5. PILES TO BE 640/900mm DIAMETER. ALL PILES REINFORCED AS PER TYPICAL PILE SECTIONS. UNLESS STATED OTHERWISE, MINIMUM COVER OF REINFORCEMENT TO BE 75mm.
6. PILE LENGTHS AND TOE DEPTHS AS PER TYPICAL SECTIONS PROVIDED.
7. REINFORCEMENT LAP LENGTHS ARE TO BE AS FOLLOWS:
   B25 = 1000mm
   B20 = 800mm
8. FOR ALL PILE TYPES, CAGES TO EXTEND INTO PILE CAPS TO PROVIDE FULL TENSION LAP AS PER TYPICAL SECTIONS.
9. ALL PILES TO BE REINFORCED WITH HELICAL LINKS TO FULL LENGTH OF REINFORCEMENT CAGE AS PER TYPICAL SECTIONS.
10. MAXIMUM AGGREGATE SIZE TO BE 20mm.
11. PILE SETTING OUT DETAILS AND LAYOUTS ARE TO OTHERS DESIGN. BYRNELOOBY ARE NOT RESPONSIBLE FOR SETTING OUT OF PILES. PILES SHOWN INDICATIVELY ON DRAWINGS.
12. PILE CAPS TO OTHERS DESIGN.
13. PILE LENGTHS AND TOE DEPTHS AS PER TYPICAL SECTIONS PROVIDED
14. GROUND STRATIGRAPHY EXPECTED AS PER PILE SECTIONS WHICH HAVE BEEN BASED ON AVAILABLE SITE INVESTIGATION DATA PROVIDED TO BYRNELOOBY. ANY VARIATION IN THE GROUND CONDITIONS TO BE COMMUNICATED TO BYRNELOOBY IMMEDIATELY.
15. PILE BREAKDOWN TO BE CARRIED OUT IN A SAFE, CONTROLLED AND CAREFUL MANNER AFTER A MINIMUM OF 3 DAYS FOLLOWING PILE INSTALLATION.
16. THE CONTRACTOR SHALL REMOVE ALL OBSTRUCTIONS DUE TO THE EXISTING BASEMENT/FOUNDATION STRUCTURES TO FACILITATE THE PILE/TEMPORARY PROP INSTALLATION
17. DRAWING TO BE READ INCONJUCTION WITH DESIGN REPORT B100.710-GEO-R001
18. THE INFLUENCE OF PREDICTED WALL MOVEMENTS ON NEARBY STRUCTURES TO BE ASSESSED BY WATERMAN MOYLAN
19. THE CONTRACTOR SHALL PROVIDE ADEQUATE MEASURES TO ENSURE EXCAVATION DEPTHS DO NOT EXCEED THE FORMATION LEVELS ILLUSTRATED IN THE DRAWING
20. SET OUT OF PILES TO ALLOW FOR 25MM TOLERANCE AND 1:100 RAKING TOLERANCE OVER RETAINED HEIGHT

Rev	Date	Description	By	Chk	App
03	25/08	GENERAL REVISION	AI	PC	MF
02	05/08	GENERAL REVISION	AI	PC	MF
01	27/01	FOR INFORMATION	AI	PC	MF
00	07/01	FOR INFORMATION	AI	PC	MF

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PROJECT
1 EARLSFORT TERRACE

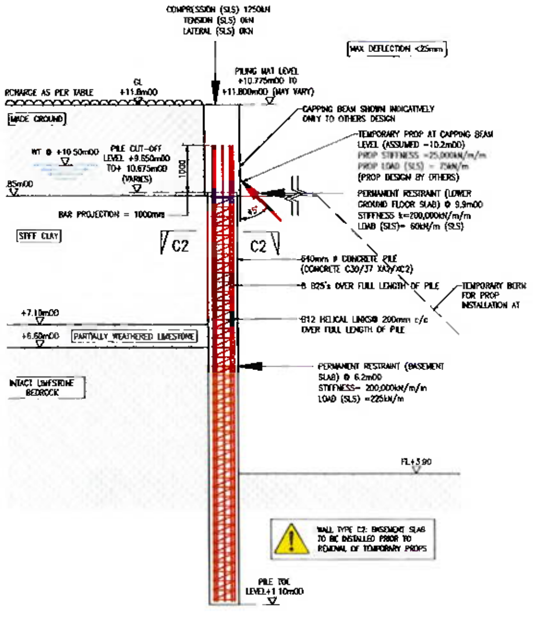
DRAWING TITLE
SECTIONS AND DETAILS
WALL TYPE B/C1

STATUS
FOR INFORMATION

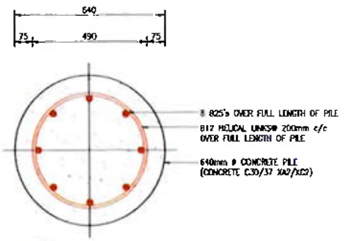
Date: 07/01/20	Scale: AS SHOWN	Drawn: AI	Chk: PC	Appr: MF
Project No: B100.710	Dwg No: B100.710-1002			Rev: 03

A1

A1



WTC2 1:50 SECTION 4-4 - WALL TYPE C2
SCALE 1:50



WTC2 1:10 SECTION C2-C2
SCALE

CONSTRUCTION SEQUENCE WALL TYPE C1 AND C2

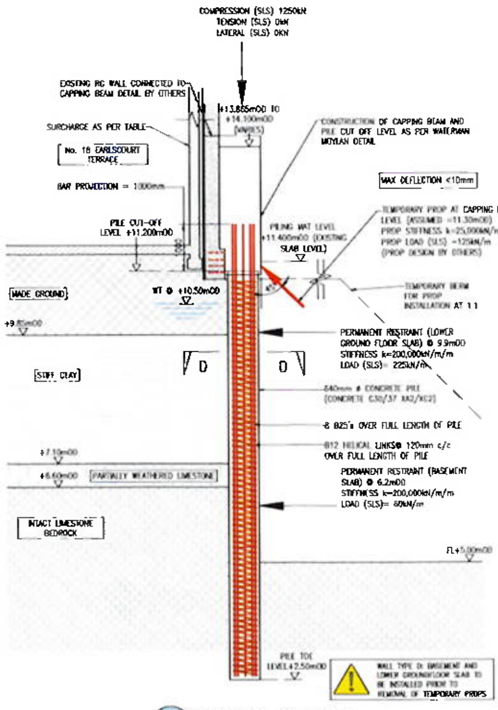
1. INSTALL PILING PLATFORM TO REQUIRED THICKNESS AND PLATFORM LEVEL.
2. CONSTRUCT GUIDEWALL TO INSTALL SECANT PILES (BASED ON WM/ARCHITECTS SETOUT).
3. INSTALL 900mmDIAMETER FIRM SECANT CFA PILES AT 1.20m c/c AS PER THE DRAWING SECTION DETAILS TO REFUSAL IN THE BEDROCK.
4. INSTALL 640MM DIAMETER ODEX/CFA SECANT HARD PILES AT 1.20M C/C AS PER THE DRAWING SECTION DETAILS.
5. EXCAVATE TO PILE CUT OFF LEVEL AND CONSTRUCT RC CAPPING BEAM
6. EXCAVATE A SOIL BERM DIRECTLY IN FRONT OF THE SECANT PILE WALL WITH THE FOLLOWING DIMENSIONS; TOP OF BERM TO BE APPROX. 0.5M BELOW BOTTOM OF CAPPING BEAM, BENCH WIDTH OF MINIMUM 4.0m, BATTERED SLOPES AT 45 DEGREES DOWN TO FORMATION LEVEL.
7. EXCAVATE A SLIT TRENCH IN THE SOIL BERM IN A HIT AND MISS SEQUENCE AND CONSTRUCT RC THRUST BLOCKS.
8. INSTALL TEMPORARY RAKING PROPS TO SUPPORT SECANT PILE WALL AT CAPPING BEAM LEVEL IN A HIT AND MISS SEQUENCE.
9. EXCAVATE TOP OF SOIL BERM DOWN TO BASEMENT FORMATION LEVEL – MAIN CONTRACTOR SHALL EMPLOY SAFE WORKING MEASURES TO ENSURE BASEMENT FORMATION LEVELS ILLUSTRATED ON DRAWINGS ARE NOT EXCEEDED – NO OVER EXCAVATION BELOW THE FORMATION LEVELS IS PERMISSIBLE
10. CONSTRUCT RC BASEMENT FOUNDATION SLAB
11. REMOVE SUPPORT FROM TEMPORARY RAKING PROPS.
12. CONSTRUCT RC LINER WALL FROM TOP OF BASEMENT SLAB SSL TO UNDERSIDE OF LOWER GROUND FLOOR RC SLAB.
13. CONSTRUCT RC LOWER GROUND FLOOR SLAB
14. CONSTRUCT THE REMAINING SUPERSTUCTURE ABOVE LOWER GROUND FLOOR LEVEL

CONSTRUCTION SEQUENCE WALL TYPE D

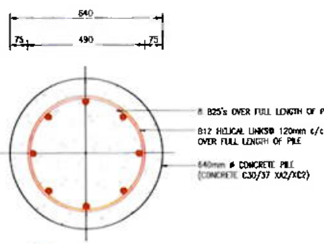
1. INSTALL PILING PLATFORM TO REQUIRED THICKNESS AND PLATFORM LEVEL.
2. CONSTRUCT GUIDEWALL TO INSTALL SECANT PILES (BASED ON WM/ARCHITECTS SETOUT).
3. INSTALL 900MM DIAMETER FIRM SECANT CFA PILES AT 1.20M C/C AS PER THE DRAWING SECTION DETAILS TO REFUSAL IN THE BEDROCK.
4. INSTALL 840MM DIAMETER ODEX/CFA SECANT HARD PILES AT 1.20M C/C AS PER THE DRAWING SECTION DETAILS.
5. EXCAVATE TO PILE CUT OFF LEVEL AND CONSTRUCT RC CAPPING BEAM
6. EXCAVATE A SOIL BERM DIRECTLY IN FRONT OF THE SECANT PILE WALL WITH THE FOLLOWING DIMENSIONS; TOP OF BERM TO BE APPROX. 0.5M BELOW BOTTOM OF CAPPING BEAM, BENCH WIDTH OF MINIMUM 4.0m, BATTERED SLOPES AT 45 DEGREES DOWN TO FORMATION LEVEL.
7. EXCAVATE A SLIT TRENCH IN THE SOIL BERM IN A HIT AND MISS SEQUENCE AND CONSTRUCT RC THRUST BLOCKS.
8. INSTALL TEMPORARY RAKING PROPS TO SUPPORT SECANT PILE WALL AT CAPPING BEAM LEVEL IN A HIT AND MISS SEQUENCE.
9. EXCAVATE TOP OF SOIL BERM DOWN TO BASEMENT FORMATION LEVEL – MAIN CONTRACTOR SHALL EMPLOY SAFE WORKING MEASURES TO ENSURE BASEMENT FORMATION LEVELS ILLUSTRATED ON DRAWINGS ARE NOT EXCEEDED – NO OVER EXCAVATION BELOW THE FORMATION LEVELS IS PERMISSIBLE.
10. CONSTRUCT RC BASEMENT FOUNDATION SLAB
11. CONSTRUCT RC LINER WALL FROM TOP OF BASEMENT SLAB SSL TO UNDERSIDE OF LOWER GROUND FLOOR RC SLAB.
12. CONSTRUCT RC LOWER GROUND FLOOR SLAB
13. REMOVE SUPPORT FROM TEMPORARY RAKING PROPS.
14. CONSTRUCT THE REMAINING SUPERSTRUCTURE ABOVE LOWER GROUND FLOOR LEVEL.

Secant Pile Wall Surcharge loads

Wall Type	Adjacent Road Level (m OD)	Piling Platform Level (m OD)	SLS Ground Surcharge (kPa) DI	SLS Ground Surcharge (kPa) II	SLS Line Load from Adjacent Wall (kN/m) DI	SLS Line Load from Adjacent Wall (kN/m) II	Additional Moment from Eccentric load due to Superstructure (kNm/m)
A	14.7	11.0	0	10			200
B1	n/a	11.0	55	25	230	40	-
B2	n/a	11.0	55	25	230	40	300
B3	n/a	11.0	55	25	230	40	-
C1	12.7	12.7	0	10	-	-	-
C2	12.7	12.7	0	10	-	-	-
D	n/a	11.4	55	25	160	65	500



WTD 1:50 SECTION 5-5 - WALL TYPE D
SCALE 1:50



WTD 1:10 SECTION D-D
SCALE

GENERAL NOTES

GENERAL NOTES:

1. DO NOT SCALE FROM THIS DRAWING – USE FIGURED DIMENSIONS ONLY
2. UNLESS NOTED OTHERWISE, ALL DIMENSIONS ARE IN MILLIMETRES (mm) AND ALL LEVELS ARE STATED IN METERS (m OD)
3. ALL PILES ARE DESIGNED IN ACCORDANCE WITH IS EN 1997-1:2004 EUROCODE 7; GEOTECHNICAL DESIGN – PART 1 GENERAL RULES AND THE IRISH NATIONAL ANNEX TO IS EN 1997-1:2004.
4. MINIMUM CONCRETE GRADE OF C30/37 AND EXPOSURE CLASS XA2/XC2 TO BE USED IN THE HARD SECANT PILES IN ACCORDANCE WITH IS EN 206-1:2013. MINIMUM CONCRETE GRADE OF C8/10 MIX TO BE USED IN SOFT PILES. STRICT QUALITY CONTROL MEASURES ARE TO BE IMPLEMENTED ON SITE FOR BOTH HARD AND SOFT SECANT PILES (I.E. CUBE TESTS AND SLUMP TESTS ETC.) THE CONTRACTOR IS TO ENSURE SAME QUALITY CONTROL MEASURES ARE USED FOR THE SOFT PILES AS PER THE HARD PILES. THE SECANT PILES ARE DESIGNED FOR A DESIGN LIFE OF 100 YEARS.
5. PILES TO BE 640/900mm DIAMETER. ALL PILES REINFORCED AS PER TYPICAL PILE SECTIONS. UNLESS STATED OTHERWISE, MINIMUM COVER OF REINFORCEMENT TO BE 75mm.
6. PILE LENGTHS AND TOE DEPTHS AS PER TYPICAL SECTIONS PROVIDED.
7. REINFORCEMENT LAP LENGTHS ARE TO BE AS FOLLOWS:
   B25 = 1000mm
   B20 = 800mm.
8. FOR ALL PILE TYPES, CAGES TO EXTEND INTO PILE CAPS TO PROVIDE FULL TENSION LAP AS PER TYPICAL SECTIONS.
9. ALL PILES TO BE REINFORCED WITH HELICAL LINKS TO FULL LENGTH OF REINFORCEMENT CAGE AS PER TYPICAL SECTIONS.
10. MAXIMUM AGGREGATE SIZE TO BE 20mm.
11. PILE SETTING OUT DETAILS AND LAYOUTS ARE TO OTHERS DESIGN. BYRNELOOBY ARE NOT RESPONSIBLE FOR SETTING OUT OF PILES. PILES SHOWN INDICATIVELY ON DRAWINGS.
12. PILE CAPS TO OTHERS DESIGN.
13. PILE LENGTHS AND TOE DEPTHS AS PER TYPICAL SECTIONS PROVIDED
14. GROUND STRATIGRAPHY EXPECTED AS PER PILE SECTIONS WHICH HAVE BEEN BASED ON AVAILABLE SITE INVESTIGATION DATA PROVIDED TO BYRNELOOBY. ANY VARIATION IN THE GROUND CONDITIONS TO BE COMMUNICATED TO BYRNELOOBY IMMEDIATELY.
15. PILE BREAKDOWN TO BE CARRIED OUT IN A SAFE, CONTROLLED AND CAREFUL MANNER AFTER A MINIMUM OF 3 DAYS FOLLOWING PILE INSTALLATION.
16. THE CONTRACTOR SHALL REMOVE ALL OBSTRUCTIONS DUE TO THE EXISTING BASEMENT/FOUNDATION STRUCTURES TO FACILITATE THE PILE/TEMPORARY PROP INSTALLATION
17. DRAWING TO BE READ INCONJUCTION WITH DESIGN REPORT B100.703-GEO-R001
18. THE INFLUENCE OF PREDICTED WALL MOVEMENTS ON NEARBY STRUCTURES TO BE ASSESSED BY WATERMAN MOYLAN
19. THE CONTRACTOR SHALL PROVIDE ADEQUATE MEASURES TO ENSURE EXCAVATION DEPTHS DO NOT EXCEED THE FORMATION LEVELS ILLUSTRATED IN THE DRAWING
20. SET OUT OF PILES TO ALLOW FOR 25MM TOLERANCE AND 1:100 RAKING TOLERANCE OVER RETAINED HEIGHT

Rev	Date	Description	By	Chk	App
03	25/07	GENERAL REVISION	AI	PG	MR
02	05/02	GENERAL REVISION	AI	PC	MR
01	27/01	FOR INFORMATION	AI	PC	MR
00	07/01	FOR INFORMATION	AI	PG	MR

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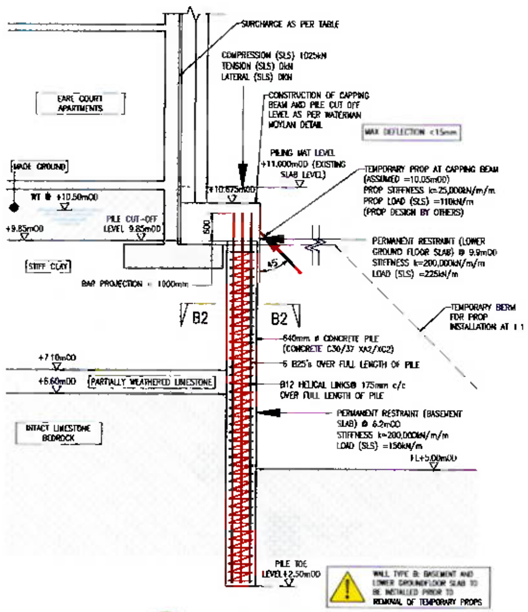
PROJECT
1 EARLSFORT TERRACE

DRAWING TITLE
SECTIONS AND DETAILS
WALL TYPE C2/D

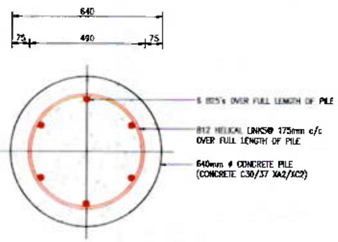
STATUS
FOR INFORMATION

Date	Scale: AS SHOWN	Drawn: AI	Chk: PG	App: MR
Project No: B100.710	Dwg. No: B100.710-1003			Rev: 03

A1



WTB2 1:50 SECTION 6-6 - WALL TYPE B2
SCALE 1:50



WTB2 1:10 SECTION B2-B2
SCALE

CONSTRUCTION SEQUENCE WALL TYPE B1 AND B2

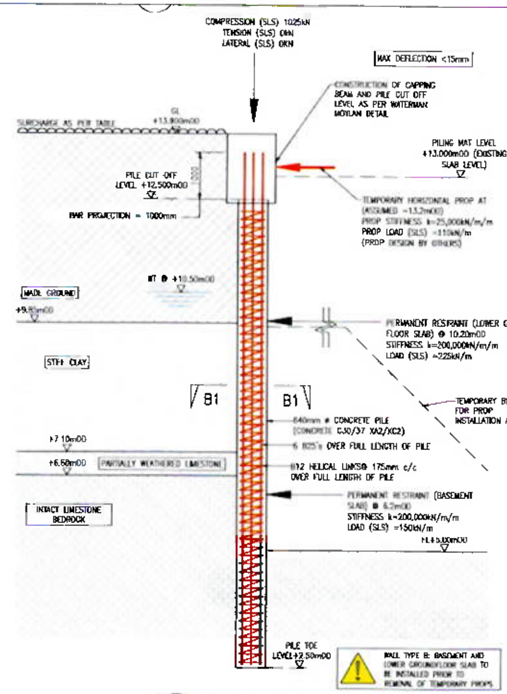
1. INSTALL PILING PLATFORM TO REQUIRED THICKNESS AND PLATFORM LEVEL.
2. CONSTRUCT GUIDEWALL TO INSTALL SECANT PILES (BASED ON WM/ARCHITECTS SETOUT).
3. INSTALL 800/900mm DIAMETER FIRM SECANT CFA PILES AT 1.00/1.20m c/c AS PER THE DRAWING SECTION DETAILS TO REFUSAL IN THE BEDROCK.
4. INSTALL 640MM DIAMETER DOCX/CFA SECANT HARD PILES AT 1.00/1.20m c/c AS PER THE DRAWING SECTION DETAILS.
5. EXCAVATE TO PILE CUT OFF LEVEL AND CONSTRUCT RC CAPPING BEAM
6. EXCAVATE A SOIL BERM DIRECTLY IN FRONT OF THE SECANT PILE WALL WITH THE FOLLOWING DIMENSIONS, TOP OF BERM TO BE APPROX. 0.5M BELOW BOTTOM OF CAPPING BEAM, BENCH WIDTH OF MINIMUM 4.0m, BATTERED SLOPES AT 45 DEGREES DOWN TO FORMATION LEVEL.
7. EXCAVATE A SLIT TRENCH IN THE SOIL BERM IN A HIT AND MISS SEQUENCE AND CONSTRUCT RC THRUST BLOCKS.
8. INSTALL TEMPORARY RAKING PROPS TO SUPPORT SECANT PILE WALL AT CAPPING BEAM LEVEL IN A HIT AND MISS SEQUENCE.
9. EXCAVATE TOP OF SOIL BERM DOWN TO BASEMENT FORMATION LEVEL - MAIN CONTRACTOR SHALL EMPLOY SAFE WORKING MEASURES TO ENSURE BASEMENT FORMATION LEVELS ILLUSTRATED ON DRAWINGS ARE NOT EXCEEDED - NO OVER EXCAVATION BELOW THE FORMATION LEVELS IS PERMISSIBLE.
10. CONSTRUCT RC BASEMENT FOUNDATION SLAB
11. CONSTRUCT RC LINER WALL FROM TOP OF BASEMENT SLAB SSL TO UNDERSIDE OF LOWER GROUND FLOOR RC SLAB.
12. CONSTRUCT RC LOWER GROUND FLOOR SLAB
13. REMOVE SUPPORT FROM TEMPORARY RAKING PROPS.
14. CONSTRUCT THE REMAINING SUPERSTRUCTURE ABOVE LOWER GROUND FLOOR LEVEL.

CONSTRUCTION SEQUENCE WALL TYPE B3

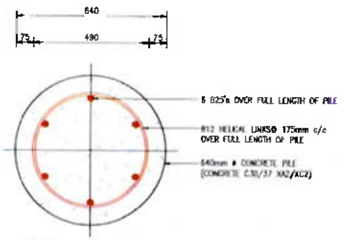
1. INSTALL PILING PLATFORM TO REQUIRED THICKNESS AND PLATFORM LEVEL.
2. CONSTRUCT GUIDEWALL TO INSTALL SECANT PILES (BASED ON WM/ARCHITECTS SETOUT).
3. INSTALL 900mm DIAMETER FIRM SECANT CFA PILES AT 1.20m c/c AS PER THE DRAWING SECTION DETAILS TO REFUSAL IN THE BEDROCK.
4. INSTALL 640MM DIAMETER DOCX/CFA SECANT HARD PILES AT 1.20m c/c AS PER THE DRAWING SECTION DETAILS.
5. EXCAVATE TO PILE CUT OFF LEVEL AND CONSTRUCT RC CAPPING BEAM
6. EXCAVATE A SOIL BERM DIRECTLY IN FRONT OF THE SECANT PILE WALL WITH THE FOLLOWING DIMENSIONS, TOP OF BERM TO BE APPROX. 0.5M BELOW BOTTOM OF CAPPING BEAM, BENCH WIDTH OF MINIMUM 4.0m, BATTERED SLOPES AT 45 DEGREES DOWN TO FORMATION LEVEL.
7. EXCAVATE TO APPROX 9.5mOD AND INSTALL HORIZONTAL PROP
8. EXCAVATE TOP OF SOIL BERM DOWN TO BASEMENT FORMATION LEVEL - MAIN CONTRACTOR SHALL EMPLOY SAFE WORKING MEASURES TO ENSURE BASEMENT FORMATION LEVELS ILLUSTRATED ON DRAWINGS ARE NOT EXCEEDED - NO OVER EXCAVATION BELOW THE FORMATION LEVELS IS PERMISSIBLE.
9. CONSTRUCT RC BASEMENT FOUNDATION SLAB
10. CONSTRUCT RC LINER WALL FROM TOP OF BASEMENT SLAB SSL TO UNDERSIDE OF LOWER GROUND FLOOR RC SLAB.
11. CONSTRUCT RC LOWER GROUND FLOOR SLAB
12. REMOVE SUPPORT FROM TEMPORARY RAKING PROPS.
13. CONSTRUCT THE REMAINING SUPERSTRUCTURE ABOVE LOWER GROUND FLOOR LEVEL.

Secant Pile Wall Surcharge Loads

Wall Type	Adjacent Road level (m OD)	Piling Platform Level (m OD)	SLS Ground Surcharge (kPa)		SLS Line load from Adjacent Wall (kN/m)		Additional Moment from Eccentric load due to Superstructure (kNm/m)
			[illegible]	[illegible]	[illegible]	[illegible]	
A	14.7	11.6	0	10	-	-	200
B1	n/a	11.6	55	25	230	40	-
B2	n/a	11.6	55	25	230	40	300
B3	n/a	11.6	55	25	230	40	-
C1	12.7	12.7	0	10	-	-	-
C2	12.7	12.7	0	10	-	-	-
D	n/a	11.4	55	25	160	65	300



WTB3 1:50 SECTION 7-7 - WALL TYPE B3
SCALE 1:50



WTB3 1:10 SECTION B3-B3
SCALE

GENERAL NOTES

GENERAL NOTES:

1. DO NOT SCALE FROM THIS DRAWING - USE FIGURED DIMENSIONS ONLY
2. UNLESS NOTED OTHERWISE, ALL DIMENSIONS ARE IN MILLIMETRES (mm) AND ALL LEVELS ARE STATED IN METERS (m OD)
3. ALL PILES ARE DESIGNED IN ACCORDANCE WITH IS EN 1997 1:2004 EUROCODE 7: GEOTECHNICAL DESIGN - PART 1: GENERAL RULES AND THE IRISH NATIONAL ANNEX TO IS EN 1997 1:2004.
4. MINIMUM CONCRETE GRADE OF C30/37 AND EXPOSURE CLASS XA2/XC2 TO BE USED IN THE HARD SECANT PILES IN ACCORDANCE WITH IS EN 206-1:2013. MINIMUM CONCRETE GRADE OF C8/10 MIX TO BE USED IN SOFT PILES. STRICT QUALITY CONTROL MEASURES ARE TO BE IMPLEMENTED ON SITE FOR BOTH HARD AND SOFT SECANT PILES (I.E. CUBE TESTS AND SLUMP TESTS ETC.). THE CONTRACTOR IS TO ENSURE SAME QUALITY CONTROL MEASURES ARE USED FOR THE SOFT PILES AS PER THE HARD PILES. THE SECANT PILES ARE DESIGNED FOR A DESIGN LIFE OF 100 YEARS.
5. PILES TO BE 640/900mm DIAMETER. ALL PILES REINFORCED AS PER TYPICAL PILE SECTIONS. UNLESS STATED OTHERWISE, MINIMUM COVER OF REINFORCEMENT TO BE 75mm.
6. PILE LENGTHS AND TOE DEPTHS AS PER TYPICAL SECTIONS PROVIDED.
7. REINFORCEMENT LAP LENGTHS ARE TO BE AS FOLLOWS:
   B25 = 1000mm
   B20 = 800mm.
8. FOR ALL PILE TYPES, CAGES TO EXTEND INTO PILE CAPS TO PROVIDE FULL TENSION LAP AS PER TYPICAL SECTIONS.
9. ALL PILES TO BE REINFORCED WITH HELICAL LINKS TO FULL LENGTH OF REINFORCEMENT CAGE AS PER TYPICAL SECTIONS.
10. MAXIMUM AGGREGATE SIZE TO BE 20mm.
11. PILE SETTING OUT DETAILS AND LAYOUTS ARE TO OTHERS DESIGN. BYRNELOOBY ARE NOT RESPONSIBLE FOR SETTING OUT OF PILES. PILES SHOWN INDICATIVELY ON DRAWINGS.
12. PILE CAPS TO OTHERS DESIGN.
13. PILE LENGTHS AND TOE DEPTHS AS PER TYPICAL SECTIONS PROVIDED
14. GROUND STRATIGRAPHY EXPECTED AS PER PILE SECTIONS WHICH HAVE BEEN BASED ON AVAILABLE SITE INVESTIGATION DATA PROVIDED TO BYRNELOOBY. ANY VARIATION IN THE GROUND CONDITIONS TO BE COMMUNICATED TO BYRNELOOBY IMMEDIATELY.
15. PILE BREAKDOWN TO BE CARRIED OUT IN A SAFE, CONTROLLED AND CAREFUL MANNER AFTER A MINIMUM OF 3 DAYS FOLLOWING PILE INSTALLATION.
16. THE CONTRACTOR SHALL REMOVE ALL OBSTRUCTIONS DUE TO THE EXISTING BASEMENT/FOUNDATION STRUCTURES TO FACILITATE THE PILE/TEMPORARY PROP INSTALLATION
17. DRAWING TO BE READ INCONJUNCTION WITH DESIGN REPORT B100.703-GEO-R001
18. THE INFLUENCE OF PREDICTED WALL MOVEMENTS ON NEARBY STRUCTURES TO BE ASSESSED BY WATERMAN MOYLAN
19. THE CONTRACTOR SHALL PROVIDE ADEQUATE MEASURES TO ENSURE EXCAVATION DEPTHS DO NOT EXCEED THE FORMATION LEVELS ILLUSTRATED IN THE DRAWING
20. SET OUT OF PILES TO ALLOW FOR 25MM TOLERANCE AND 1:100 RAKING TOLERANCE OVER RETAINED HEIGHT

Rev	Date	Description	By	Chk	App
02	25/02	GENERAL REVISION	AI	PC	MR
01	05/02	GENERAL REVISION	AI	PC	MR
01	27/01	FOR INFORMATION	AI	PC	MR

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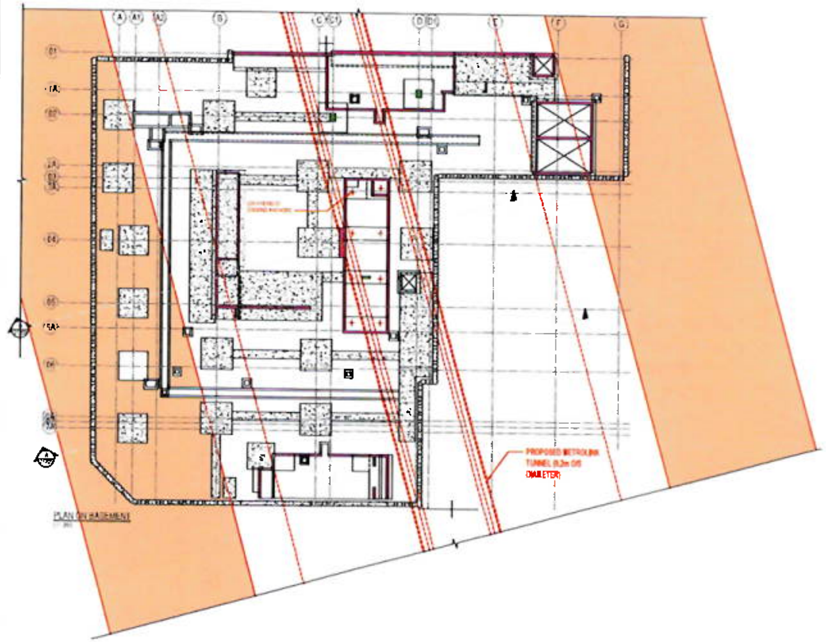
PROJECT
1 EARLSFORT TERRACE

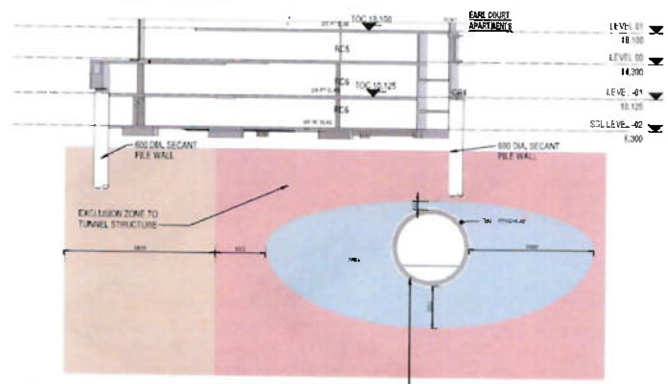
DRAWING TITLE
SECTIONS AND DETAILS
WALL TYPE B/C1

STATUS
FOR INFORMATION

Date: 27/01/20	Scale: AS SHOWN	Drawn: AI	Chk: PC	App: MR
Project No: B100.710	Dwg. No: B100.710-1004			Rev: 02



PLAN ON BASEMENT



SECTION A-A
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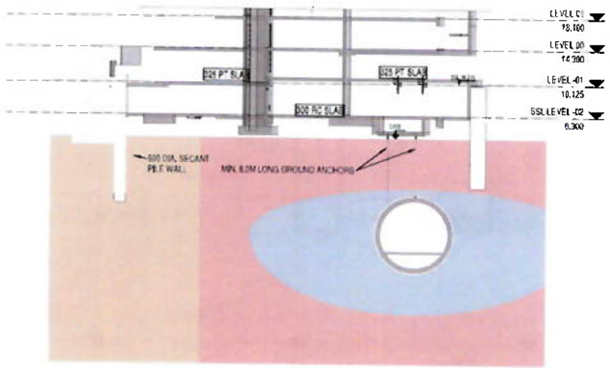
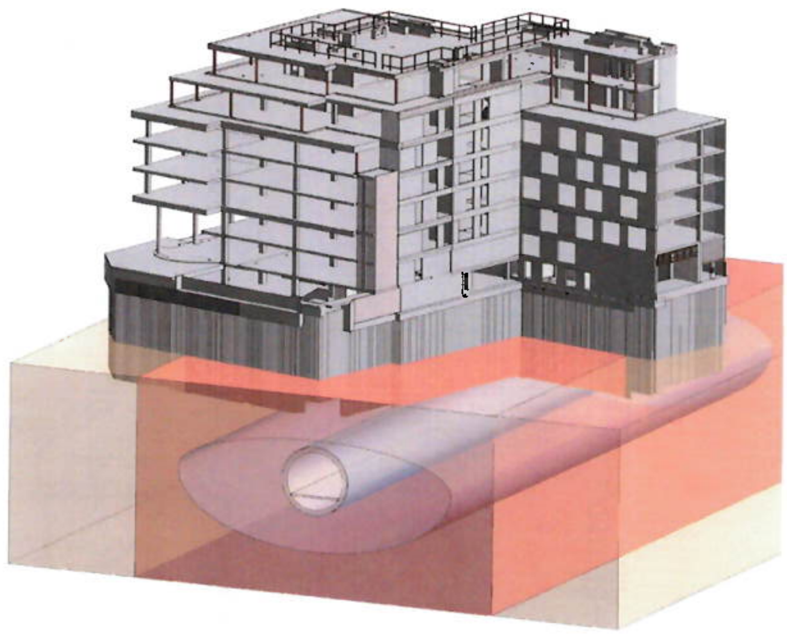


SECTION B-B
(1:200)



ISOMETRIC VIEW
(NTS)

GENERAL NOTES

EARLSFORT TERRACE

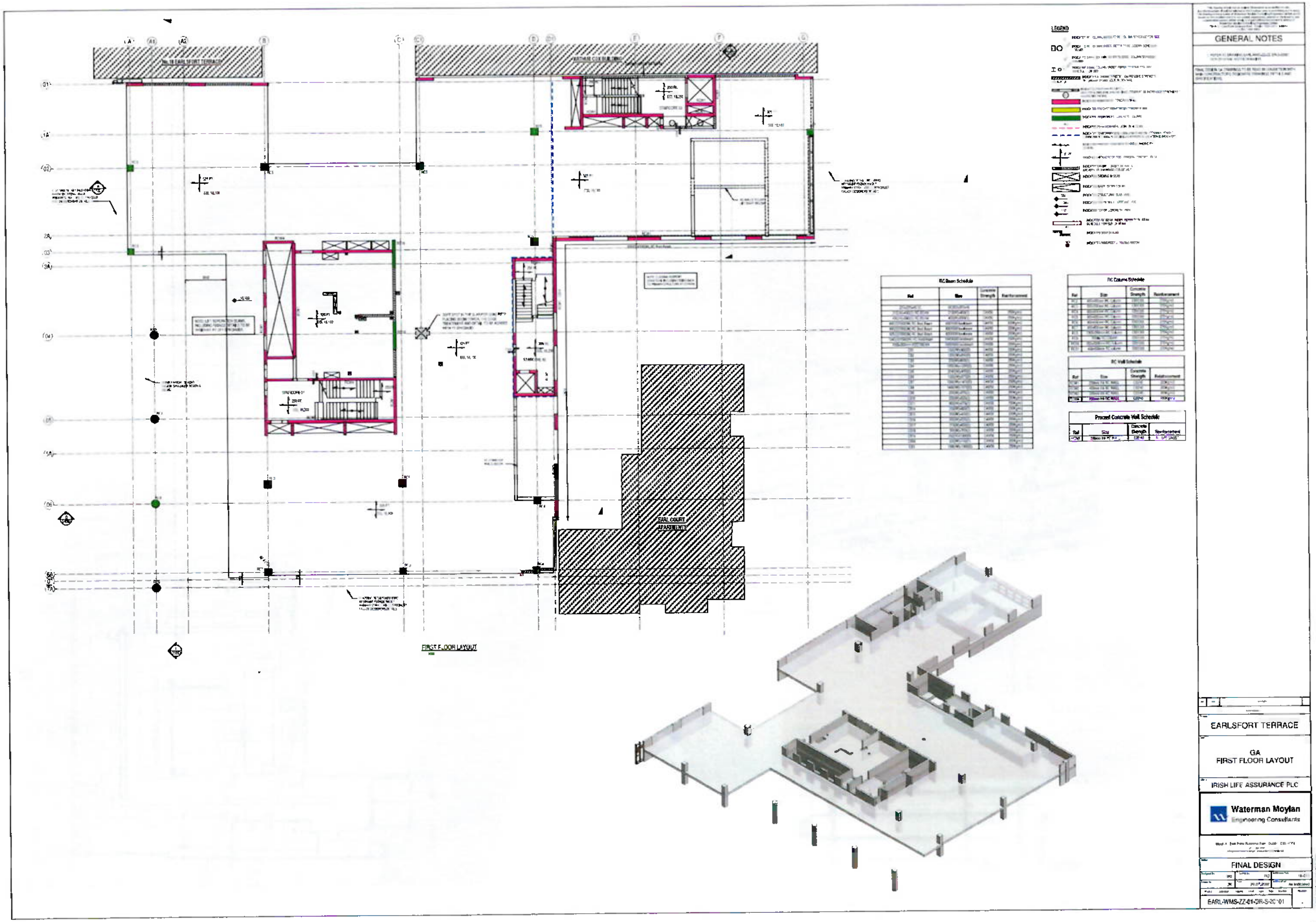
GA
CADENZA - PROPOSED
METROLINK INTERFACE

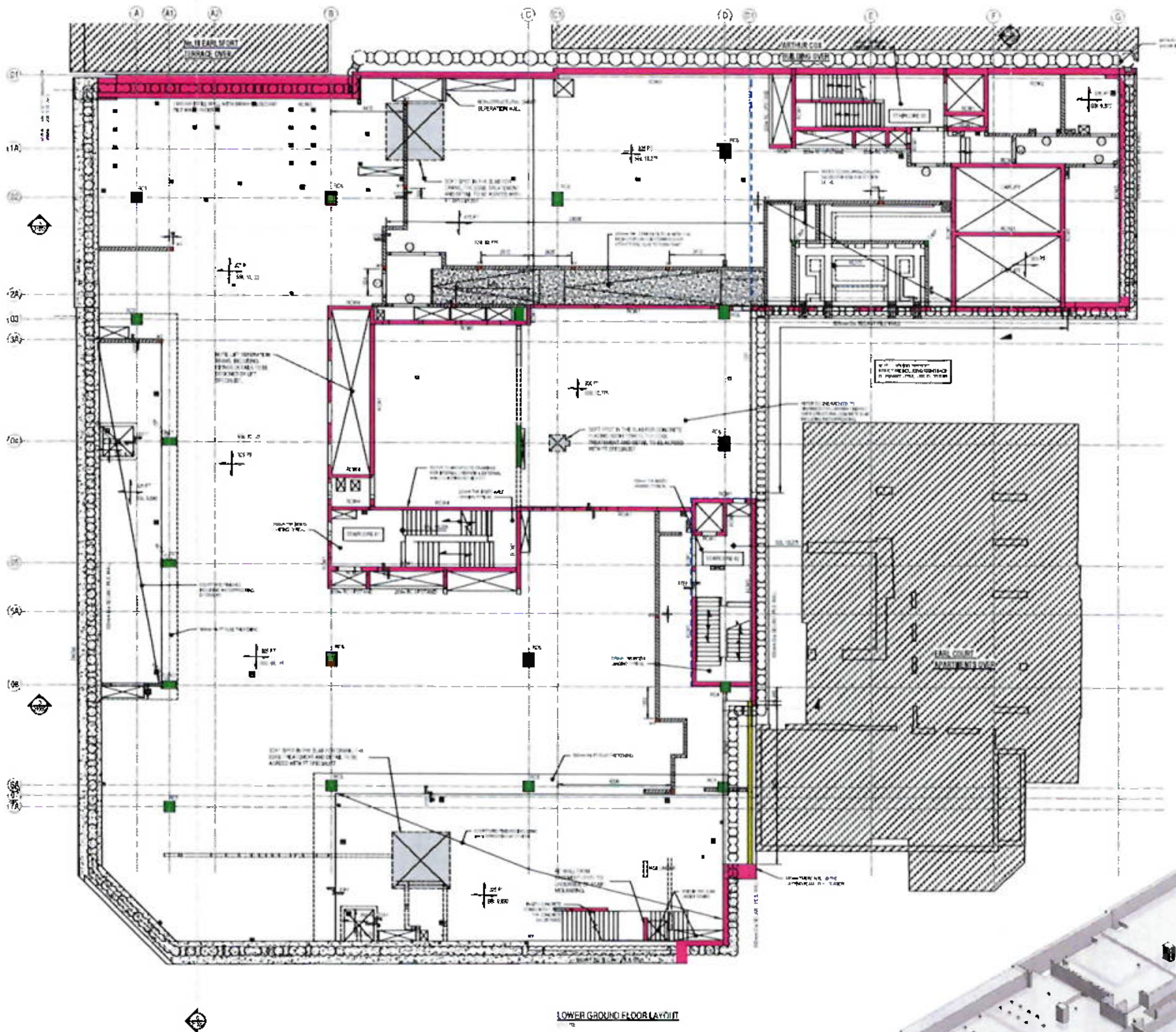
IRISH LIFE ASSURANCE PLC

Waterman Moylan
Engineering Consultants

FOR INFORMATION
DRAFT

EARL-WMS-ZZ-ZZ-DR-S-21200 | A


LEGEND
RC Beam Schedule
RC Column Schedule
RC Wall Schedule
Precast Concrete Wall Schedule
EARL COURT APARTMENTS
FIRST FLOOR LAYOUT
GENERAL NOTES
EARLSFORT TERRACE
GA
FIRST FLOOR LAYOUT
IRISH LIFE ASSURANCE PLC
Waterman Moylan
Engineering Consultants
FINAL DESIGN
EARL-WMS-ZZ-01-DR-S-20101



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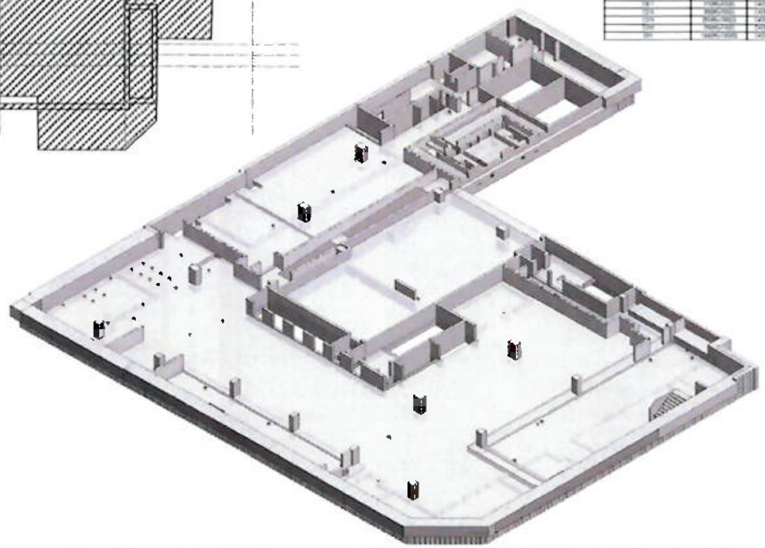


GENERAL NOTES

EARLSFORT TERRACE

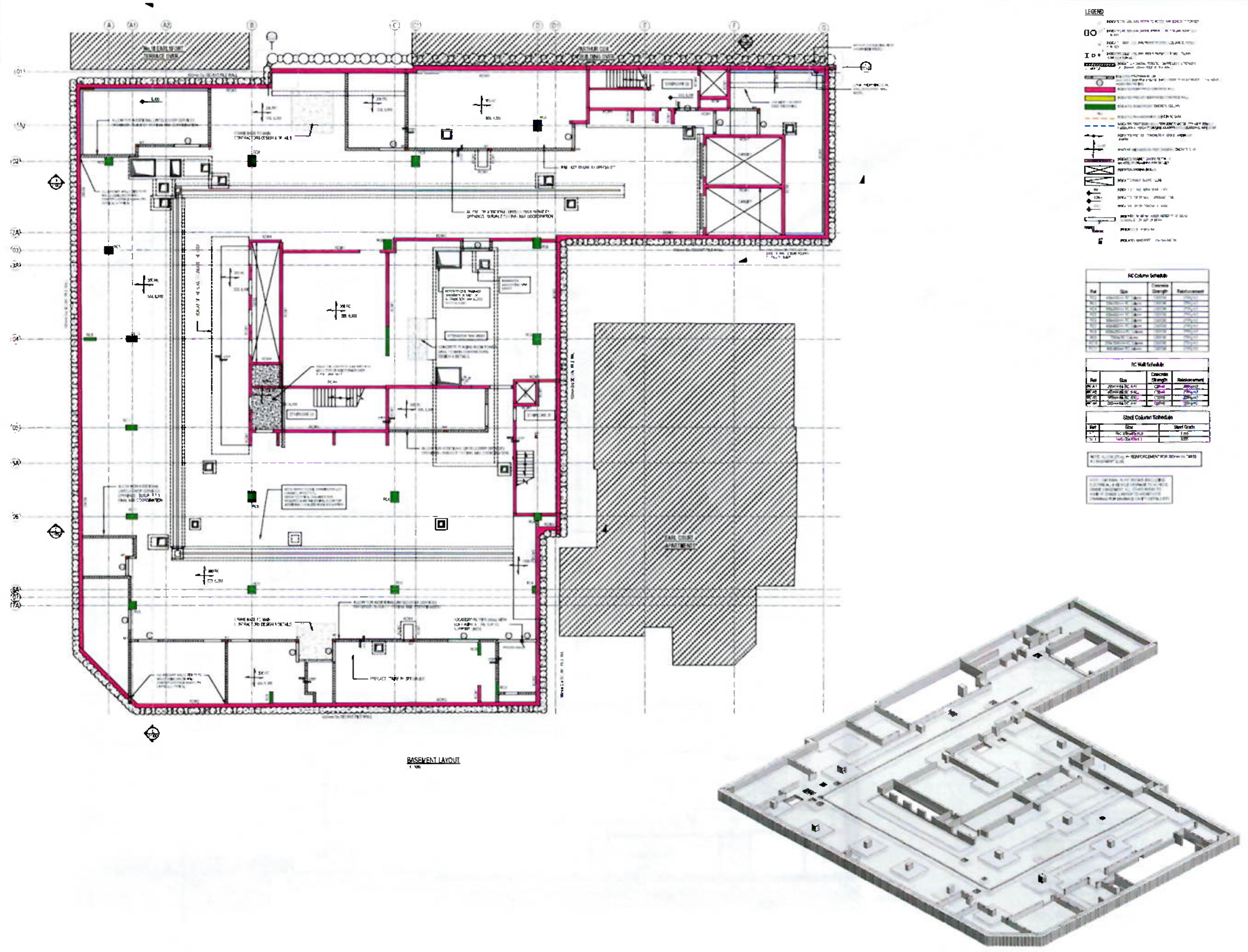
GA
LOWER GROUND FLOOR
LAYOUT

IRISH LIFE ASSURANCE PLC

Waterman Moylan
Engineering Consultants

FINAL DESIGN

EARL-WMS-ZZ-B1-DR-S-20090



GENERAL NOTES

EARLSFORT TERRACE

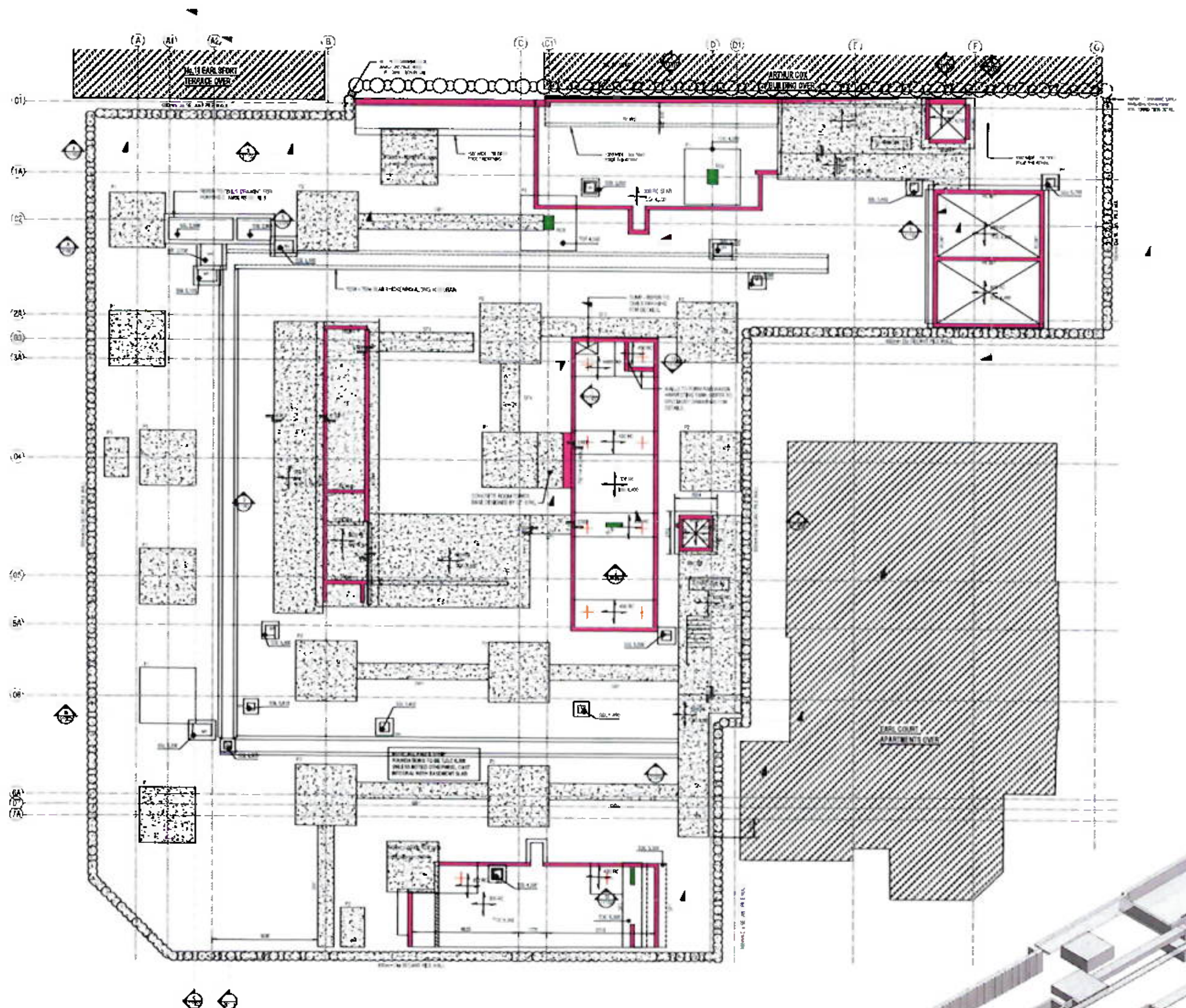
GA
BASEMENT LAYOUT

IRISH LIFE ASSURANCE PLC

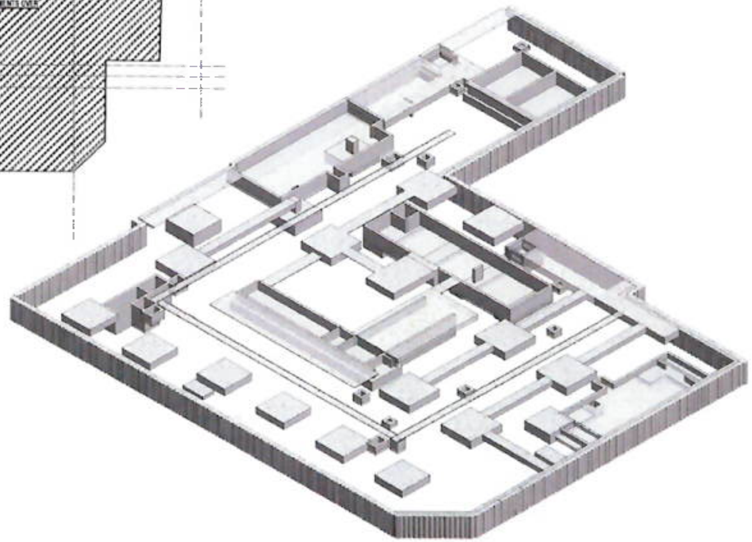
Waterman Moylan
Engineering Consultants

FINAL DESIGN

EARL-WMS-ZZ-B2-DR-S-20098



FOUNDATION LAYOUT



GENERAL NOTES

EARLSFORT TERRACE

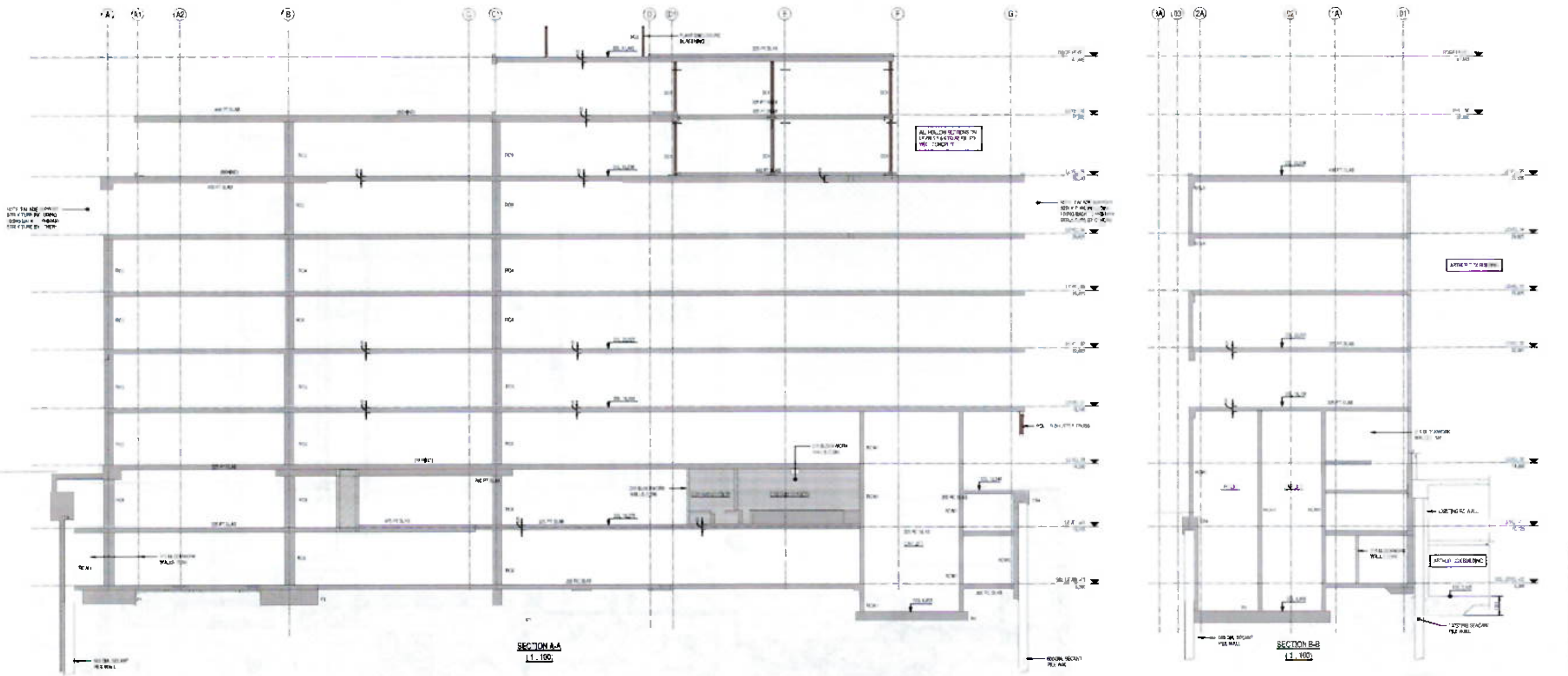
GA
FOUNDATION LAYOUT

IRISH LIFE ASSURANCE PLC

Waterman Moylan
Engineering Consultants

FINAL DESIGN

EARL-WMS-ZZ-B3-DR-S-11100



RC Beam Schedule

Ref	Size	Concrete Strength	Reinforcement
[illegible]	[illegible]	[illegible]	[illegible]

RC Column Schedule

Ref	Size	Concrete Strength	Reinforcement
[illegible]	[illegible]	[illegible]	[illegible]

Foundation Schedule

Ref	Size	Concrete Strength	Reinforcement Allowance
[illegible]	[illegible]	[illegible]	[illegible]

RC Wall Schedule

Ref	Size	Concrete Strength	Reinforcement
[illegible]	[illegible]	[illegible]	[illegible]

Steel Column Schedule

Ref	Size	Steel Grade
[illegible]	[illegible]	[illegible]

Precast Concrete Wall Schedule

Ref	Size	Concrete Strength	Reinforcement
[illegible]	[illegible]	[illegible]	[illegible]

GENERAL NOTES

EARLSFORT TERRACE

GA
BUILDING SECTION
SHEET 01

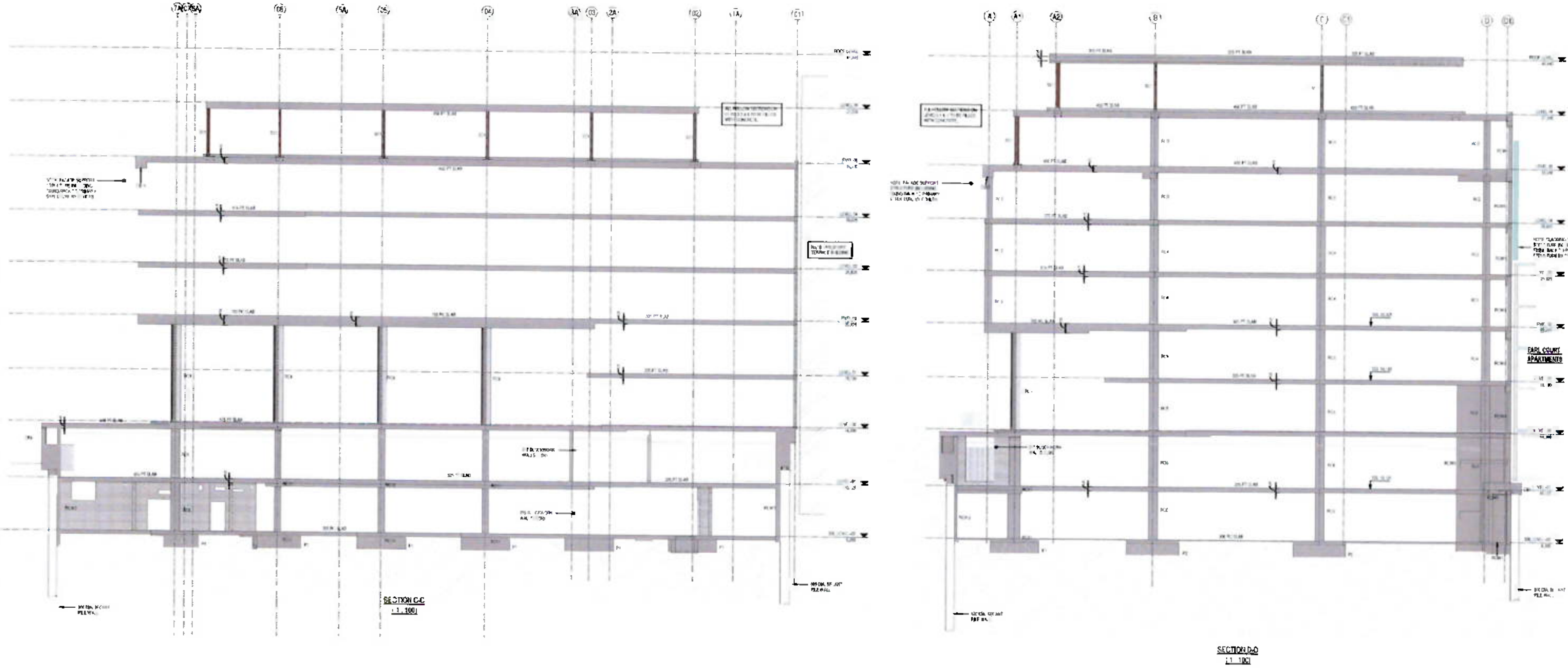
IRISH LIFE ASSURANCE PLC

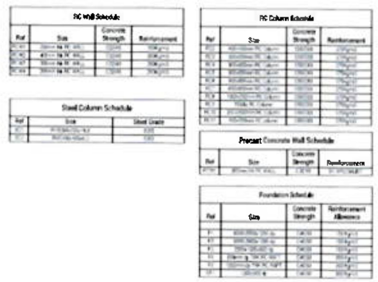
Waterman Moylan
Engineering Consultants

FINAL DESIGN

EARL-WMS-ZZ-ZZ-DR-S-21001

GENERAL NOTES





EARLSFORT TERRACE

GA
BUILDING SECTION
SHEET 02

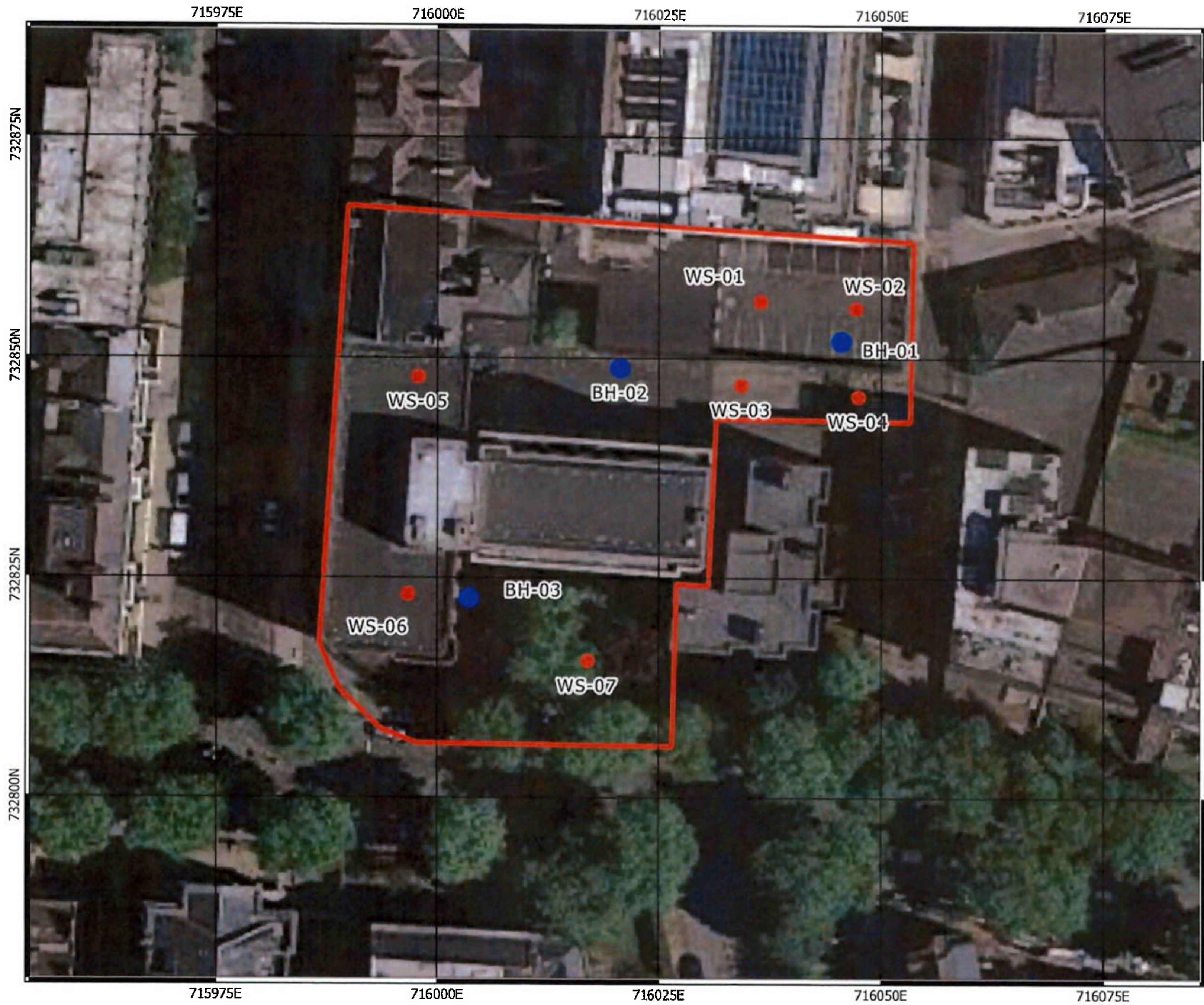
IRISH LIFE ASSURANCE PLC

Waterman Moylan
Engineering Consultants

FINAL DESIGN

EAR-WMS-ZZ-ZZ-DR-S-21002

Appendix B
SI Data for Cadenza Building



Site Boundary
Window Samples
Boreholes

Client:

Waterman Moylan
Engineering Consultants

Project Code:
8232-11-18

Project Title:
Davitt House

Drawing Title:
Figure 2 SI Points

GROUND INVESTIGATIONS IRELAND

Ground Investigations Ireland Ltd.
Catherinstown House,
Hazelhatch Road,
Newcastle, Co. Dublin
www.gii.ie 01-6015175/5176

0 5 10 15 20 m

Drawn By:	Date:
BS	13/12/2018

Ground Investigations Ireland Ltd www.gii.ie		Site Davitt House, Earlsfort Terrace	Borehole Number **BH01**
Machine : Beretta T44 **Flush** : Polymer **Core Dia**: 100 mm **Method** : Rotary Cored	**Casing Diameter** 100mm to 10.70m	**Ground Level (mOD)** 12.22	**Client** Urban Solutions	**Job Number** 8149-10-18
	Location	**Dates** 19/10/2018	**Project Contractor** Gound Investigations Ireland	**Sheet** 1/2

Depth (m)	TCR	SCR	RQD	FI	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water	Instr
						12.14	0.08	TARMAC			
							(0.42)	Grey fine to coarse angular to sub-angular Crushed Rock FILL			
						11.72	0.50	Poor Recovery Driller notes cobbles boulders and building rubble			
	0	0	0								
1.20											
							(2.20)				
1.80-2.25					1,2/3,3,4,4 SPT(C) N=14						
	25	0	0								
						9.52	2.70	MADE GROUND: Brown slightly sandy slightly gravelly Clay with rare fragments of redbrick and a strong hydrocarbon smell			
							(0.50)				
						9.02	3.20	Possible MADE GROUND dark grey slightly sandy slightly gravelly CLAY (Firm to Stiff)			
3.20 3.20-3.65					3,2/3,3,4,4 SPT(C) N=14						
	57	0	0				(1.50)				
4.70 4.70-5.15					4,3/4,4,3,4 SPT(C) N=15	7.52	4.70	Stiff dark grey slightly sandy slightly gravelly CLAY			
							(0.80)				
	90	13	10			6.72	5.50	Very stiff dark grey slightly sandy gravelly CLAY			
							(0.40)				
5.85						6.32	5.90	Medium strong medium bedded dark grey fine grained LIMESTONE with thin mudstone laminations. Partially weathered to unweathered F1: 0-15 Degrees. medium to close spacing, planar smooth with some clay smearing. F2: 70-8-Degrees, undulating rough, quartz crystal growth along fracture(vein).			
6.20											
				9			(1.20)				
	100	70	20								
7.10						5.12	7.10	Medium strong thinly bedded to thickly laminated dark grey fine grained LIMESTONE interbedded with weak thinly laminated black calcareous MUDSTONE. Partially weathered to unweathered F1: 0-15 Degrees, very closely spaced, planar smooth with some clay infill.			
				18			(0.60)				
7.70						4.52	7.70	Medium strong thinly bedded to thickly laminated dark grey fine grained LIMESTONE interbedded with weak thinly laminated black calcareous MUDSTONE. Partially weathered to unweathered F1: 0-15 Degrees, close to very clsoely spaced, planar smooth with some clay infilling and smearing.			
	100	93	57								
				9			(3.00)				
9.20											
	100	93	57								

Remarks
No Groundwater encountered.
Standpipe installed, slotted from 10.70m to 1.0m with gravel surround, sealed from 1.0m to GL with a bentonite surround and flush cover.
Borehole backfilled upon completion

Scale (approx)	Logged By
1:50	Tmcl

Figure No. 8149-10-18.BH01

Ground Investigations Ireland Ltd www.gii.ie	**Site** Davitt House, Earlsfort Terrace	**Borehole Number** **BH01**	
Machine : Beretta T44 **Flush** : Polymer **Core Dia**: 100 mm **Method** : Rotary Cored	**Casing Diameter** 100mm to 10.70m	**Ground Level (mOD)** 12.22	**Client** Urban Solutions
	Location	**Dates** 19/10/2018	**Project Contractor** Gound Investigations Ireland
Job Number 8149-10-18	**Sheet** 2/2		

Depth (m)	TCR	SCR	RQD	FI	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water	Instr
10.70						1.52	10.70	Complete at 10.70m			

Remarks	**Scale (approx)** 1:50	**Logged By** Tmcl
	Figure No. 8149-10-18.BH01	

Ground Investigations Ireland Ltd www.gii.ie			Site Davitt House, Earlsfort Terrace	Borehole Number **BH02**
Machine : Beretta T44 **Flush** : Polymer **Core Dia:** 100 mm **Method** : Rotary Cored	**Casing Diameter** 100mm to 10.70m	**Ground Level (mOD)** 11.24	**Client** Urban Solutions	**Job Number** 8149-10-18
	Location	**Dates** 18/10/2018	**Project Contractor** Gound Investigations Ireland	**Sheet** 1/1

Depth (m)	TCR	SCR	RQD	FI	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water
						11.19	0.05	TARMAC		
							(0.35)			
0.40						10.84	0.40	Grey fine to coarse angular to sub-angular Crushed Rock FILL		
	100	0	0				(0.95)	Reinforced Concrete		
1.50-1.70					12,14/50 SPT(C) 50/50	9.89	1.35	Very stiff dark grey slightly sandy slightly gravelly CLAY with rare cobbles		
1.80										
2.50-2.53	86	0	0		25/50 SPT(C) 25*/20 50/10		(2.55)			
3.20										
	100	83	83							
3.90				NI		7.34	3.90	Medium strong thinly bedded grey fine grained LIMESTONE interbedded with weak thinly bedded to thickly laminated black MUDSTONE. Partially weathered to unweathered The sequence is predominatley non - intact recovery indicates: One set of fractures F1: 0-10 Degress, medium spacing, planar rough with some clay smearing		
							(0.55)			
4.40 4.45						6.79	4.45	Medium strong thinly bedded grey fine grained LIMESTONE interbedded with weak thinly bedded to thickly laminated black MUDSTONE. Partially weathered to unweathered F1: 5-15 Degrees, closley spaced, planar smooth with some clay infilling		
	100	93	33	14			(2.10)			
5.90										
6.55	100	75	23	NI		4.69	6.55	Medium strong thinly bedded grey fine grained LIMESTONE interbedded with weak thinly bedded to thickly laminated black MUDSTONE. Partially weathered to unweathered Non-Intact Zone between 6.55m to 7.25m BGL F1: 0-10 Degrees, medium to close spacing, planar smooth with some clay smearing		
7.10 7.25										
	100	90	47	12			(2.05)			
8.60						2.64	8.60	Complete at 8.60m		

Remarks
No Groundwater encountered.
Borehole backfilled upon completion

Scale (approx)	Logged By
1:50	Tmcl

Figure No.
8149-10-18.BH02

Ground Investigations Ireland Ltd www.gii.ie		Site Davitt House, Earlsfort Terrace	Borehole Number **BH03**
Machine : Beretta T44 **Flush :** Polymer **Core Dia:** 100 mm **Method :** Rotary Cored	**Casing Diameter** 100mm to 10.70m **Location**	**Ground Level (mOD)** 10.80 **Dates** 09/10/2018- 17/10/2018	**Client** Urban Solutions **Project Contractor** Gound Investigations Ireland
			Job Number 8149-10-18 **Sheet** 1/1

Depth (m)	TCR	SCR	RQD	FI	Field Records	Level (mOD)	Depth (m) (Thickness)	Description
						10.70	0.10	TARMAC
							(0.40)	Grey fine to coarse angular to sub-angular Crushed Rock FILL
0.50	90	0	0			10.30 10.20	0.50 0.60	Dark grey slightly sandy very clayey angular to sub-rounded fine to coarse GRAVEL.
								Very stiff dark grey slightly sandy gravelly CLAY.
1.50 1.50-1.70	100	0	0		10,14/50 SPT(C) 50/50		(2.60)	
3.00 3.00-3.03 3.20 3.40	97	83	43	NI	25/50 SPT(C) 25*/20 50/10	7.60	3.20 (0.90)	Medium strong thinly bedded dark grey fine grained LIMESTONE with thin mudstone laminations. Partially weathered to unweathered. F1: 0-10 Degrees, medium to closely spaced, planar smooth with some clay infill. F2: 70-80 Degrees, undulating rough with crysatl growth along fracture surface(Vein).
				9		6.70	4.10	Medium strong thickly to thinly bedded dark grey fine grained LIMESTONE interbedded with weak thinly laminated black MUDSTONE. Partially weathered to unweathered.
4.50	100	70	40					
5.50				NI				Non-Intact Zone between 5.50m to 5.90m BGL
5.90 6.00	100	97	40	9			(4.00)	F1: 0-10 Degrees, medium to closely spaced, planar smooth with some clay infill. F2: 70-80 Degrees, Planar smooth with some clay smearing.
7.50	100	100	58					
8.10						2.70	8.10	Complete at 8.60m

Remarks	Scale (approx)	Logged By
No Groundwater encountered. Standpipe installed, slotted from 8.10m to 3.0m with gravel surround, sealed from 3.0m to GL with a bentonite surround and flush cover. No groundwater encountered	1:50	Tmcl
	Figure No. 8149-10-18.BH03	

Earlsfort Terrace – Geobore-S Rotary Core Photographs

BH01 0.0m – 3.20m (Box1,2,3)

BH01 3.20m – 7.70m (Box 4,5,6)

BH02 0.0m – 4.40m (Box 1,2,3)

BH02 4.40m – 8.60m (Box 4,5,6)

BH03 0.0m – 4.50m (Box 1,2,3)

BH03 4.50m – 8.10m (Box 4,5,6)

Appendix C

Building Damage Assessment Refined Phase 2a Calculations (Not Included)

Appendix D

Building Damage Assessment EIAR Phase 2a Assessment Methodology

&

Technical Note from Oral Hearing on Impact of Implementing the LoD on the Building Damage Assessment in the EIAR

Appendix B.1: Buildings Identified from Building Survey

BUILDING CODE	BUILDING DESCRIPTION			BUILDING LOCATION			BUILDING INFORMATION			
	NAME	CONSIDERATION	CATEGORY	Chainage	Dmin (m)	Dmax (m)	Height (m)	Nº Floors	Length (m)	Depth (m)
B-147	Davitt House	Hotel	Prominent Building	19+020	0.00	26.02	12.0	4	26.02	-2.50

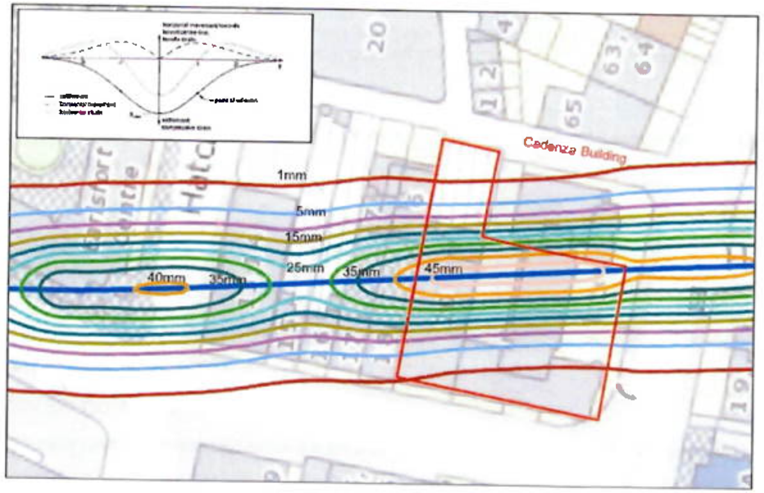
Table 5-1: Details of Ground Conditions, V_L and K Values by Chainage

Start Chainage	End Chainage	Length (m)	Excavated Material	Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Vs (%)	K
18900	18960	60	CLU	CLU (20%)	Sands and Gravels (20%)	QBR (30%)	Qx (30%)		1.5	0.3
18960	18980	20	CLU	CLU (40%)	Sands and Gravels (30%)	QBR (10%)	Qx (20%)		0.75	0.4
18980	19100	120	CLU	CLU (20%)	Sands and Gravels (20%)	QBR (30%)	Qx (30%)		1.5	0.4

- **Phase 1** – the assessment of the greenfield settlement contours using generic ground parameters and the identification of buildings that are
    - a) enclosed by the 10mm contour or with a ground settlement slope > 1:500 and
    - b) those buildings enclosed by the 1mm contour subject to 'special' considerations.
- **Phase 2** – all the buildings identified in Phase 1 are assessed using the greenfield ground movement profile making credible foundation assumptions and are classified into Damage Categories 0 – 5; those buildings placed in Damage Category 3 or above, and those subject to 'special' considerations (see below) are carried through to Phase 3.
- **Phase 3** – each identified building is considered individually to determine its behaviour using detailed information and assessment methods; this may include a refined ground model, detailed structural surveys, refined construction methodology and use of sophisticated soil-structure interaction analysis such as finite element analysis.

In the context of building damage assessment, 'special' considerations refer to buildings (hereafter referred as 'special' buildings) in proximity of the excavation, with deep basements, or those identified as designated Protected Structures, or sensitive buildings as defined below:

- **Case A:** it is on shallow foundation and is within a distance from a retained cutting, shaft, or box equal to the excavated depth of superficial deposits or 50% of the total excavation depth, whichever is the greater. In this context, superficial deposits are taken to be soils above the rockhead level.
- **Case B:** it has a foundation level deeper than 4m, or (in the case of a bored tunnel) greater than 20% of the depth to tunnel axis.
- **Case C:** it is a Protected Structure
- **Case D:** any 'prominent' or 'sensitive' buildings that might need further assessment to determine whether any protective works required.



4.3 Phase 2 Assessment

4.3.1 General

The Phase 2 assessment is split into two sub-phases, namely Phase 2a and Phase 2b as follows:

a) Phase 2a is undertaken as part of the Preliminary Design. This sub-phase initially adopts the same conservative assumptions used to predict the Phase 1 greenfield ground movements; refined assumptions are sometimes made to assess the sensitivity of the initial assessment results.

b) Phase 2b is a confirmatory/refined analysis undertaken by the detailed designer of the D&B Contractor. This sub-phase usually adopts tighter volume loss parameters and utilises a more refined construction methodology since the D&B contractor will now be progressing the development of the detail design and finalising his construction methodology and planning.

4.5 Phase 3 Assessment

All buildings that have been classified at the end of the Phase 2b assessment as Damage Category 3 (Moderate) or above (or where there exists any uncertainty after the Phase 2b assessment) will be the subject of a Phase 3 assessment by the D&B Contractor. Furthermore, all 'special' buildings (refer to Section 4.1), which have been the subject of a Phase 2a/2b assessment, but which do not qualify for further assessment (Damage Category 2 or below) will also be the subject of a Phase 3 assessment.

For the Phase 3 assessment, each building will be subject to detailed assessment on an individual basis. Both the strains developing within the building, and the applicability of the classification of risk categories will be reviewed in terms of their relevance for the buildings undergoing Phase 3 assessment. The purpose of the Phase 3 assessment is to ensure that any uncertainty or risk that might lead to damage is minimised.

A detailed survey will be carried out as part of the Phase 3 assessment to provide the necessary additional information to inform this detailed analysis of how the individual elements of the building would be affected by the predicted ground movements. The method and extent of the detailed analysis will be determined on a case-by-case basis and may include a more sophisticated semi-empirical or a detailed soil-structure interaction using finite element modelling methods. As part of this analysis, the detailed design and construction methodology, including the stiffness of the wall and propping system, together with the beneficial effects of the overall structural stiffness of the building will be taken account of. The overall structural stiffness of the building will limit the deformation of the building to the greenfield settlement profile and thus reduce the maximum tensile strains experienced by the building. It is therefore likely that the Phase 3 assessment will yield further improvement to the damage category determined by the Phase 2b assessment.

The ultimate output of the Phase 3 analysis will be to minimise risk and uncertainty and finalise any necessary protection works required to mitigate the impact of construction generated ground movements. This may include further refinement or modification by D&B Contractor of TBM drive parameters and control measures.

5.2 Phase 2a Building Assessment Results

5.2.1 Representative Buildings

Initial Phase 2a Assessment

The initial Phase 2a assessment results for the 'representative' buildings are given in Table 5-2 together with the key relevant building information. The actual location of the building and the worst-case orientation line that passes through the footprint of the buildings (i.e., close to being orthogonal to the settlement contour) have been determined from the OS Map.

The initial Phase 2a assessment shows that the following nine buildings fall within Damage Category 3:

B39, B76, B77, B142, B175, B176, B177, B178 & B179.

Refined Phase 2a Assessment

Considering the nine buildings which fell within Damage Category 3 at the end of the initial Phase 2a assessment, a refined Phase 2a assessment has been carried out with tighter volume loss values considering the advances in tunnelling equipment and control due to the capability of the TBM that will be used, and the Damage Category of all the buildings reassessed. In the refined Phase 2a assessment, the volume loss values have been taken as two-thirds of the corresponding values adopted for the initial Phase 2a as follows:

- Superficial material (clay or granular material): $V_l = 1.0\%$
- Rock strata: $V_l = 0.5\%$

In the case of a mixed strata:

- If the tunnel is wholly in rock and there is at least half-a-tunnel diameter rock cover above the crown, then $V_l = 0.5\%$;
- Else $V_l = 1.0\%$.

These volume loss values are compatible with those experienced using modern tunnelling equipment and control systems from variable density TBMs which it is anticipated will be employed for this project.

For the non-TBM construction, current methodologies with instrumentation and monitoring from the surface providing information to inform the control at the face also improve the losses that can be anticipated and allows the volume loss values to be taken as 50% more than that of the corresponding TBM volume loss values.

These values are moderatly conservate when comparing against the published data in CIRIA PR 30 for stiff fissured clay and glacial deposits.

The refined Phase 2a assessment results show that all the 'representative' buildings fall within Damage Category 2 or below.

Table 5-2: Result of Phase 2a Building Damage Assessment – Representative Buildings

Ref	Chainage	Description	Height (m)	Number of Floors	Length (m)	Depth of basement (m)	Initial Phase 2a Assessment Damage Category	Refined Phase 2a Assessment Damage Category	RPS, NIAH, RMP or other heritage (Y/N/unknown)	Continue to next assessment phase? (Y/N)	Comments
B-147	19020	Davitt House	12 0	4	26 0	-2 5	2 (Slight)	2 (Slight)	N	N	Damage category 2 or below

Table F1: Building Damage Assessment Results for 'Representative' and 'Additional' Buildings - Critical Segments within Each Building (Rev 1)

Specific Building	Parameter	Critical Segment	Start [m]	End [m]	Curvature	Max Slope	Max Settlement [mm]	Max Tensile Strain [%]	Min Radius of Curvature (Hogging) [m]	Min Radius of Curvature (Sagging) [m]	Damage Category
B-147	Max Slope	1	0.63901	18.749	Hogging	0.0027837	21.409	0.082142	3797.9	-	2 (Slight)
	Max Settlement	2	18.749	34.225	Sagging	0.0027837	35.374	0.051998	-	1695.3	1 (Very Slight)
	Max Tensile Strain	1	0.63901	18.749	Hogging	0.0027837	21.409	0.082142	3797.9	-	2 (Slight)
	Min Radius of Curvature (Hogging)	1	0.63901	18.749	Hogging	0.0027837	21.409	0.082142	3797.9	-	2 (Slight)
	Min Radius of Curvature (Sagging)	2	18.749	34.225	Sagging	0.0027837	35.374	0.051998	-	1695.3	1 (Very Slight)

IN THE MATTER OF AN APPLICATION TO
AN BORD PLEANALA

For Approval of the Railway (Metrolink – Estuary to Charlemont via Dublin Airport) Order [2022]

ABP-314724-22

ORAL HEARING

STATEMENT OF EVIDENCE

on

(i) Amendments to the Railway Order and Schedules / drawings and modifications to the scheme

(ii) Errata

(iii) Agreements presented to the Oral Hearing

(iv) Updates to the EIAR

By

Ronan Hallissey

19 February 2024

5.1.2 Chapter 5 MetroLink Construction Phase

(a) The proposed reduction in the vertical upward limits of deviation.

Chapter 5 of the EIAR presents details of the construction methodology and programme for the proposed project. TII wish to make a single amendment to this chapter in regard to the proposed Limit of Deviations

Limits of Deviations

In the Draft Railway Order for MetroLink Limits of Deviation (LODs) are proposed and these LODs are the same as those approved by the Board for "Old Metro North" and "Dart Underground". (Refer to Tables below)

Project Element	Vertically (upwards) (m)	Vertically (downwards) (m)	Horizontally (in all directions from centre line) (m)
Surface works (not impacting on public roadways)	2	2	5
Surface works (impacting on public roadways)	1	1	2.5
Tunnel Alignment	5	10	15

Project Element	Vertically (upwards) (m)	Vertically (downwards) (m)	Horizontally (in all directions from centre line) (m)
Retained Cut and Cut and Cover Alignment	1	2	2.5
Station Box Locations	5	10	2

However a number of submissions received from the statutory consultation process raised concerns with regard to the LODs, particularly those that allowed for movement upwards as it was identified that there was potential for increased impact on buildings should the LOD upwards be allowed.

In response to this, TII proposed to modify the proposed LOD to restrict any potential deviation upwards to just 1m.

These new limits will further reduce potential impacts above the alignment, specifically on:

a) Settlement Effects;

b) Groundborne Noise & Vibration;

c) Effects on future site development potential.

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Technical Note

Project:	Dublin MetroLink		
Doc No:	ML1-JAI-GEO-ROUT_XX-RP-Y-00034		
Subject:	Impact on the Preliminary Design Building Damage Assessment Results due to Imposition of Limits of Deviation		
Revision No.	P01		
Prepared by:	Alberto Jaen-Toribio	**Date:**	10.11.23
Checked by:	Mahee Maheetharan	**Date:**	10.11.23
Reviewed by:	Mahee Maheetharan	**Date:**	10.11.23
Approved by:	Paul Brown	**Date:**	10.11.23

1. Background and Purpose

The building damage assessment work carried out and reported in ML1-JAI-GEO-ROUT_XX-RP-Y-00034 P03 [Ref. 1] is based on the draft Railway Order (RO). The Draft RO includes for Limits of Deviation (LOD) for proposed MetroLink infrastructure that can be availed of, if practicable. This Technical Note assesses the potential impact on the Preliminary Design building damage assessment work should the LOD set out by the Draft RO (Article 6) be availed of.

2. Basis of Assessment

The impact of settlement has been assessed assuming that the LOD set out by the Draft RO Article 6 might be availed of, except for the tunnel vertical alignment, which it is assumed will only be moved upwards by 1m from that shown by the RO application.

3. Assessment Approach and Findings

A review of buildings within Damage Category 1 or below and those buildings not currently impacted by the RO design alignment has been undertaken to ascertain sensitivity to change due to alignment alterations within the horizontal LOD set out by the Draft RO. Based on the analysis of selected worst-case buildings, it has been concluded that any buildings assessed as falling into Damage Category-1 (DC-1) or below based on the Draft RO are unlikely to fall to above DC-2 level due to the imposition of the horizontal LOD or vertical LOD limited to 1m.

Further, the damage category of buildings in the vicinity of the proposed station boxes is unlikely to be affected due to the restriction in the LOD for station boxes; i.e. maximum of 2m. It is also concluded that the buildings currently outside the 10mm green-field contour line based on the RO design alignment are unlikely to be impacted above DC-2 level due to the imposition of the LOD, hence there are no significant impacts predicted.

For the buildings away from the proposed station boxes and showing DC-2 level based on the RO application tunnel alignment, quantitative assessments have been carried out with the tunnel horizontal alignment at the extremity of the LOD together with a vertical upwards LOD of 1m; this exercise showed that there will be no increase in the damage category level.

Further, for the buildings currently falling into DC-3 level, it has been confirmed by inspection that they are already at their worst possible position in relation to the RO application tunnel alignment and therefore the imposition of horizontal LOD with an upwards vertical LOD limited to 1m is unlikely to have any adverse impact.

In all cases, lowering of the vertical alignment could only improves on the damage potential.

4. Conclusions

The building damage assessment carried out and reported in ML1-JAI-GEO-ROUT_XX-RP-Y-00034 P03 [Ref. 1] is based on the RO application tunnel alignment.

The analysis carried out in this TN has concluded that there will be no additional buildings that would qualify for Phase-3 Assessment to that reported by the EIAR should the LOD set out by the Draft RO be availed, including the vertical upwards tunnel LOD limited to 1m.

Potentially different and/or additional impacts (below "Slight") associated with possible deviations to the route within the LOD have been identified. Based on this analysis, it is concluded that there would be no change to the required mitigation measures or residual impacts arising from the application of the mitigation measures set out in the EIAR and no additional significant impacts.

5. References

[1] Jacobs Engineering Ireland Limited (2022), Damage Assessment Report of Buildings and Other Assets ML1-JAI-GEO-ROUT_XX-RP-Y-00034 P03 (Dated 22/06/2022)