Kellystown Wind Farm Planning Support: FI Request Related to Vibration Effects from Blasting

Client	EDF Renewables Ireland Ltd.
Document Ref.	25068-R-0001-05
Project Title	Kellystown Wind Farm
Date	06/08/2025

GDG
GAVIN & DOHERTY
GEOSOLUTIONS

Project Title:	Kellystown Wind Farm
Report Title:	Kellystown Wind Farm Planning Support: FI Request Related to Vibration Effects from Blasting
Document Reference:	25068-R-0001-05

Client:	EDF Renewables Ireland Ltd.
Ultimate Client:	EDF Renewables Ireland Ltd.
Confidentiality	Client Confidential

REVISION HISTORY

Rev	Date	Reason for Issue	Originator	Checker	Reviewer	Approver
01	27/05/2025	For Client comments	AB	BvdM	TOS	TOS
02	04/06/2025	Final issue	AB	BvdM	TOS	TOS
03	13/06/2025	Revised to address Louth County Council comments	AB	BvdM	TOS	TOS
04	07/07/2025	Revised to address Jennings O'Donovan Ltd comments	AB	BvdM	TOS	TOS
05	06/08/2025	Revised to legal comments	AB	BvdM	TOS	TOS

DISCLAIMER

REVISION SUMMARY

Rev	Date	Section(s)	Detail of Change
02	04/06/2025	General, §2.1, §2.2	• Typos • Updated Figure 2-1 and Figure 2-2
03	13/06/2025	Executive Summary, Section §4.2, §6 and §7	• Text added
04	07/07/2025	Executive Summary, Section §1.3, §6, §7	• Text amended
05	06/08/2025	Executive Summary, Section §1, §3, §4, §5, §6, §7	• Text amended

GDG
GAVIN & DOHERTY
GEOSOLUTIONS

TABLE OF CONTENTS

Chapter **Page**

Executive Summary 6

1 Introduction 8

1.1 Background 8
1.2 WTG Description 9
1.3 Scope 9

2 Ground Summary 10

2.1 Desk Study 10
2.2 Site Investigation 12

3 WTG Foundation 14

4 Review of Standards, Codes and Literature 17

4.1 Review of Literature 17
4.2 Allowable Values of Groundborne Vibrations – Codes and Standards 18

5 WTG Foundation Assessment 21

5.1 WTG suppliers' requirements 21
5.2 Dynamic Displacement and Base Rotation 21
5.3 Moment Developed By Blasting-Induced Vibration 22
5.3.1 Key Assumptions 24
5.4 Concrete Vibration Damage Threshold 25

6 Conclusions 27

7 Measures to be Adopted at Detailed Design Stage 28

8 References 30

GDG
GAVIN & DOHERTY
GEOSOLUTIONS

LIST OF FIGURES

Figure 1-1: Satellite image with the location of Kellystown Wind farm (Google Earth Pro@), Ref. [5] 8
Figure 2-1: Superficial geology (derived from Ref. [9, 10]) 11
Figure 2-2: Bedrock Solid Geology including fault lines (derived from Ref. [9, 10]) 11
Figure 2-3 Ground investigation location of T2 (Ref. [10]) 13
Figure 3-1: Typical WTG foundation detail (Drawing no: 6918-PL-502 Proposed foundation, Ref. [12]) 15
Figure 3-2: Proposed WTG details plan and elevation (Drawing no: 6918-PL-501, Ref. [7]) 16
Figure 4-1: Example of blasted open pit quarry close to an existing wind farm in Spain (Gómez-Márquez et al., 2010 (Ref. [13]) 17
Figure 4-2: Typical WTG foundation with reinforcement (Champ Chardon Wind Farm, France - Foundation design: 5 Senvion MM100, Ref. [20]) 19
Figure 4-3: Comparison of various allowable PPV limits for structural damage 20
Figure 5-1: Limiting angular distortion (derived from Ref. [29, 30]) 22
Figure 5-2: Sketch of rotation in a gravity-base foundation 23

LIST OF TABLES

Table 2-1: Encountered Stratigraphy at T2 12
Table 5-1 Typical bending moment M developed under extreme load conditions 25
Table 5-2 TVA mass concrete damage criteria (Ref. [42, 24]) 26

EXECUTIVE SUMMARY

Gavin and Doherty Geosolutions Ltd. (GDG) was commissioned by EDF Renewables Ireland Ltd. to assess the potential impact of blasting-induced vibrations from the adjacent Kilsaran Quarry on planned Wind Turbine Generator (WTG) foundations at Kellystown Wind Farm, particularly focusing on WTG T2 due to its proximity to the quarry (circa 100-130m from T2, Ref. [1, 2]) which represents a worst-case scenario, based on the full extent of the 25-year planning permission for the Kilsaran Quarry (ref. [3, 4]).

The report responds to a Further Information (FI) Request from Louth County Council, which questioned the appropriateness of using a Peak Particle Velocity (PPV) limit of 100 mm/s—higher than the 50 mm/s cosmetic damage threshold recommended by BS 5228-2:2009 for reinforced structures. The applicant (EDF Renewables Ireland) has been requested to provide confirmation that a higher PPV value could be appropriate for these structures.

Key Findings:

- Site and Foundation Conditions: Ground investigations confirmed that WTG T2 will be founded on weathered non-rippable weak to moderate weak siltstone/greywacke bedrock with no groundwater encountered during site investigation. The foundation will be a 27.2m diameter concrete structure, cylindrical in shape, heavily reinforced and poured into a prepared excavation on the ground. The foundation will be designed to withstand significant static and dynamic operational loads.
- Standards and Literature Review: Vibration threshold PPV limits defined in most commonly referenced standards are primarily intended for conventional structures like residential, commercial and industrial buildings (light framed buildings with windows, masonry or concrete walls, brittle finishes e.g. plaster etc). WTG foundations are massive structures that are heavily reinforced to resist extreme operation loads, they are therefore inherently more resistant to vibration induced damage than framed buildings with brittle finishes. A comprehensive review of international standards and case studies supports the use of a 100 mm/s PPV limit for reinforced concrete structures like WTG foundations.
- WTG Manufacturer Confirmation: Several WTG suppliers have confirmed that a PPV of 100 mm/s poses no risk to the structural integrity or operation of the WTG, provided the foundation is adequately designed. Confirmation has been provided from 4 major manufacturers. Large multi megawatt WTGs are designed and certified in accordance with the same unified standards. It may therefore be concluded that all WTG models that could potentially be installed at Kellystown Wind Farm will not be impacted by a PPV of 100mm/s.
- Foundation Assessment: Simplified foundation assessment has demonstrated that:
  - Angular distortion and base inclination due to blasting are well below critical thresholds.

- The bending moment from blasting induced vibrations is of the same order of magnitude of the bending moment developed at top of foundation of similar-sized WTG under extreme operational loads (GDG previous experience).
- The concrete vibration damage threshold is above 100 mm/s, confirming structural safety.
- The WTG foundation can be designed to withstand the loads and displacement developed by blasting induced vibration. This is subject to further evaluation during the detailed foundation design phase.

Measures to be adopted at detailed design stage:

- Adopt 100 mm/s PPV Limit: This value is deemed safe and appropriate for blast design near the WTG.
- Monitoring and Mitigation: Consider implementing a real-time vibration monitoring regime (e.g., geophones, strain gauges) and conduct regular inspections for cracks or structural changes to ensure structural integrity and long-term performance.
- Numerical Modelling: Use finite element tools (e.g. PLAXIS) to further validate vibration impacts and soil-structure interaction effects.

Conclusions:

The report concludes that a PPV limit of 100 mm/s is technically justified and safe for the WTG foundations at Kellystown Wind Farm. This PPV limit can be incorporated into the detailed foundation design. The risk of structural damage from quarry blasting is minimal. Blasting will have no adverse effect on the long-term strength and performance of the WTG foundations.

1 INTRODUCTION

Gavin and Doherty Geosolutions Ltd. (GDG) was commissioned by EDF Renewables Ireland Ltd. (EDF) to assess the potential impact of blasting-induced vibrations from a nearby quarry on planned WTG foundations at Kellystown Wind Farm, Co. Louth, Ireland.

1.1 BACKGROUND

EDF submitted a planning application (planning reference 2460766) to Louth County Council for Kellystown Wind Farm (consisting of five wind WTGs, T1 to T5) in December 2024. The project is located approximately 5km southeast of Dunleer and is adjacent to Kilsaran Quarry, which is operated by Kilsaran Concrete (Figure 1-1).

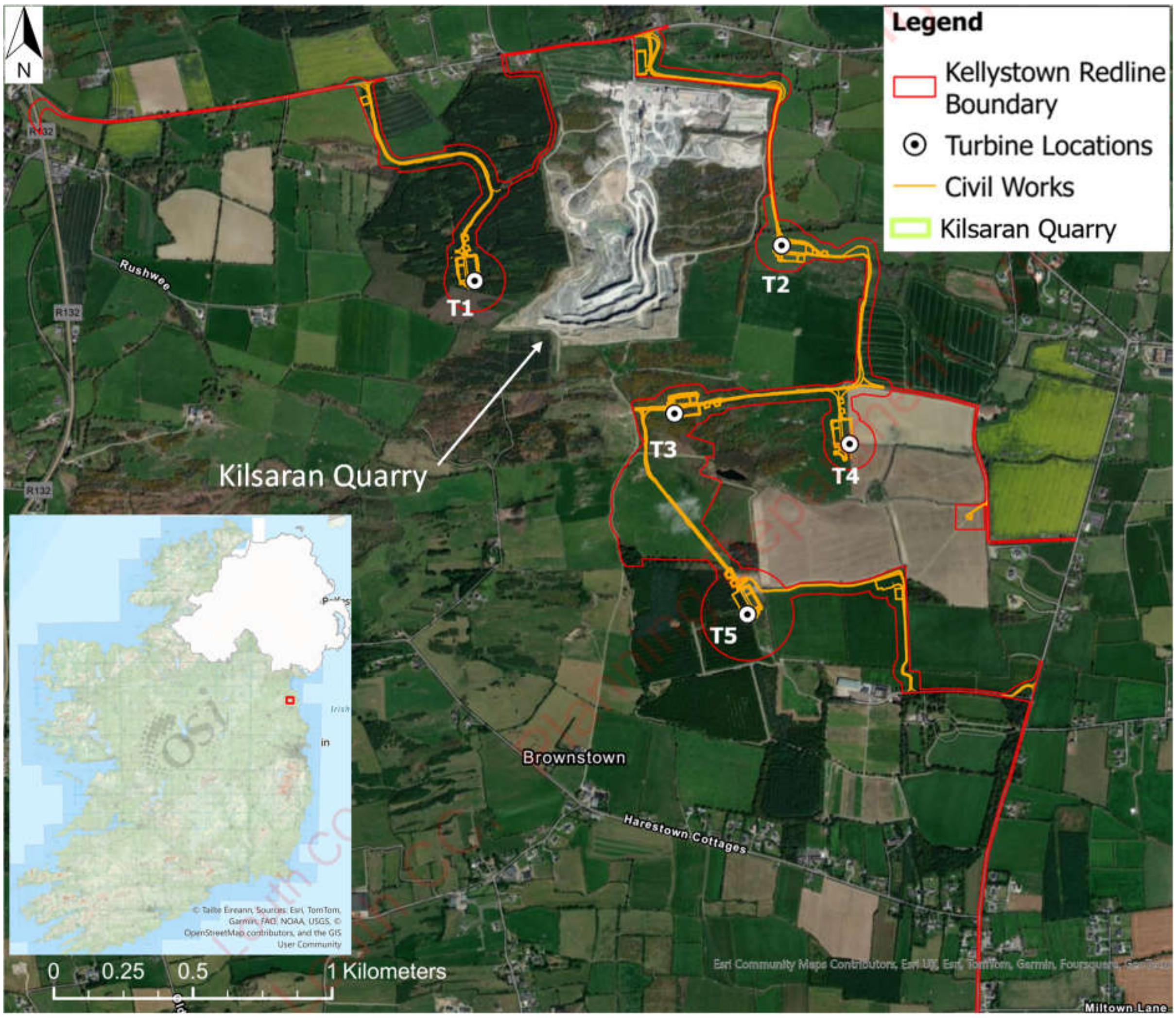

Figure 1-1: Satellite image with the location of Kellystown Wind farm (Google Earth Pro®), Ref. [5]

It is noted that EDF engaged WSP to carry out a vibration assessment, which is included in Appendix 13.4 of the Environmental Impact Assessment Report (EIAR). This report addresses the potential impact of vibrations from quarry blasting operations on the WTG foundations.

During the planning period, Kilsaran Concrete lodged an objection, raising concerns about the vibration assessment and the proximity of WTG T1, T2 and T3 to the quarry. Louth County Council have issued a Further Information (FI) Request in relation to the application. A section of the FI Request from Louth County Council (Ref. [6]) is related to vibration effects from blasting at Kilsaran Quarry and states the following:

"The "Ground Vibration and Air Overpressure Blast Monitoring" report undertaken by WSP sets a Peak Particle Velocity (PPV) threshold of 100mm/s as the cosmetic damage limit for the turbines. However, the standard BS 5228- 2 sets a cosmetic damage limit of 50mm/s for "reinforced or framed structures, industrial and heavy commercial buildings". The WSP report states that while BS 5228- 2:2009 suggests a 50 mm/s limit for cosmetic damage in reinforced structures, it is considered that the higher threshold of 100 mm/s which corresponds to the potential for minor damage is a more appropriate threshold.

However, no justification has been provided for this revised PPV threshold. It is further noted, using a PPV threshold of 50mm/s, that the estimated PPV for a 330kg MIC (Maximum Instantaneous Charge) at the quarry would be 67.9mm/s for T2, 48.9mm/s for T3 and 36.9mms for T1 which would indicate that T1, T2 and T3 may be located in an unsuitable position. The applicant is therefore requested to provide confirmation from the turbine manufacturer that a higher PPV value could be appropriate for these structures."

EDF requested GDG to provide technical support in response to the FI Request. This report reviews relevant codes, standards, public literature, WTG suppliers' requirements and simplified foundation assessment to evaluate if the ground-borne design vibration limit of 100mm/s PPV limit from blasting activities will have an adverse effect on the long-term strength and performance of the WTG foundations.

1.2 WTG DESCRIPTION

The planned wind farm consists of five WTGs (T1 to T5) with rotor diameter range of 149-163m, total tip height range of 179.5-180m and hub height range of 98-105m. Each WTG will be supported on a large foundation consisting of a circular, reinforced concrete footing (Ref. [7]). Based on the planning drawings, the WTG foundations will be shallow gravity footings, with a diameter of 27.2 metres (Ref. [7, 8]). Details about the geometry of the proposed WTGs are summarised in Section 3.

1.3 SCOPE

The objective of this report is to provide confirmation that a ground-borne design vibration limit of 100mm/s Peak Particle Velocity (PPV) from blasting activities in the quarry will have no adverse effect on the long-term strength and performance of the WTG foundations. This report mainly focuses on WTG T2 because a) it is the closest to blasting activities (quarry boundary is circa 100-130m from T2, Ref. [1, 2]), and b) it has the highest expected PPV value among the proposed WTGs, i.e. a contour-based estimate of 67.9 mm/s (Ref. [2]).

It is worth noting that the quarry face is currently approximately 415m from the centre point of turbine T2. The quarry boundary being approximately 100–130 metres from T2 represents a worst-case scenario, based on the full extent of the 25-year planning permission for the Kilsaran Quarry (ref. [3]). Furthermore, only a limited number of blasts will occur at this range (blasting is limited to once per month, ref. [3, 4]), as it marks the maximum extent of Kilsaran's permitted expansion, and blasting locations within the quarry vary, meaning this blasting location (i.e. 100-130m from T2) will not be used monthly.

The scope of this report is as follows:

- Review of Site Investigation (SI)
- Review of standards, codes, literature and typical supplier's requirements
- Simplified foundation assessment

2 GROUND SUMMARY

2.1 DESK STUDY

A desk study has been carried out for the Site (Ref. [9, 8, 10]) as shown in Figure 2-1. The key findings from the desk study are summarised below:

- Superficial soils present within the wind farm largely consist of thin glacial till soils overlying shallow, often outcropping greywacke rock or glacial tills (sands and gravels/boulder clay) derived from Lower Palaeozoic sandstones and shales.
- The GSI indicates that the bedrock on the site is thickly bedded calcareous greywacke from the Clogherhead Formation, which outcrops within the northern western portion of the site.
- Alluvium is present within the boundary (present in river valley bottoms), and some isolated areas of peat (with a depth less than 0.5m) are also indicated to be within the site boundary.
- There are several fault lines in the vicinity of the site (Figure 2-2). A series of three faults also bisect the wind farm site, trending in an approximately northeast –southwest direction potentially affecting the quality of the subsurface rock formation at WTG T2, T4 and T5. Faults have the potential to result in highly fractured and deeply weathered bedrock and / or the channelling of groundwater flow. Faults can often be associated with an increased hazard of ground movement. Ireland is currently located within a region of extremely low tectonic activity and well removed from regions of significant seismic activity. An assessment has been conducted concerning natural seismic activity (Ref. [10]), which has determined that any recorded ground movement can be expected to be negligible to the development proposed, where Peak Ground Acceleration can be expected to be in the order of 0.02g.

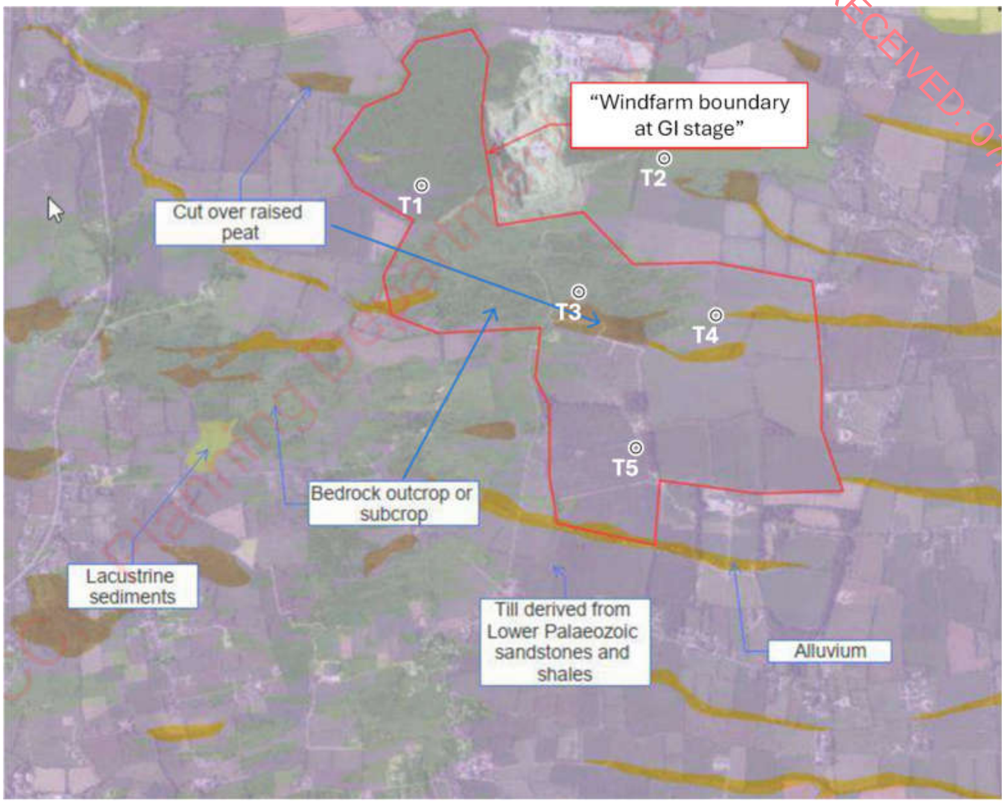

Figure 2-1: Superficial geology (derived from Ref. [9, 10])

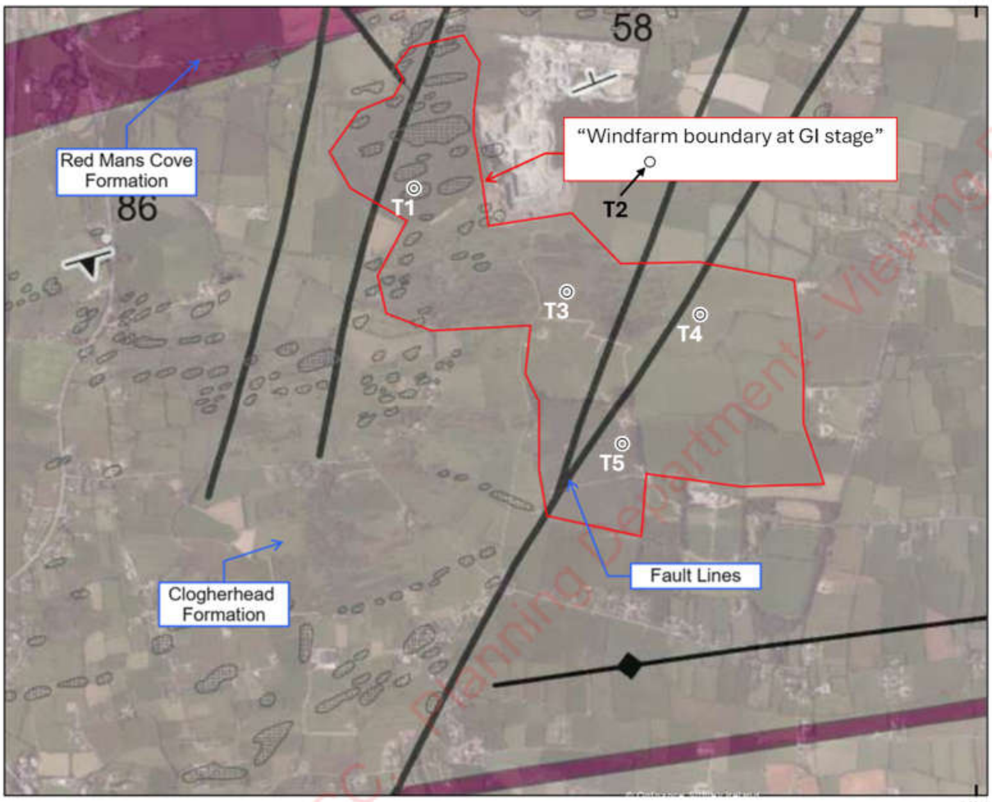

Figure 2-2: Bedrock Solid Geology including fault lines (derived from Ref. [9, 10])

2.2 SITE INVESTIGATION

A ground investigation was undertaken by Whiteford Ltd in April 2024 (Ref. [10]) and comprised of the following methods:

- Trial pitting (14 no.) - machine excavated
- Peat probing (5 no.)

The ground investigation location plan is presented in Figure 2-3. Three trial pits (TP-T02, TP-Spring1 and TP-Spring 2) to depths of between 0.4m and 1.5m BGL were carried out at the proposed location of T2.

Trial pits indicate that the ground conditions comprise:

- Topsoil, over
- Stiff cohesive glacial till, over
- Weathered rock.

The top of the weathered rock is typically found at depths of 0.1 to 1.3m bgl, and it is described on the logs as extremely weak to weak, dark grey, fine-grained, distinctly weathered siltstone/greywacke.

The trial pits were recorded as dry, however weak flow was recorded in 3 no. trial pits (TP-T04, TP-T05, and TP-D) at depths ranging between 1.9m BGL and 2.8m BGL. Table 2-1 summarises the expected stratigraphy at T2.

5no. Point Load tests were undertaken on rock cores obtained from trial pits TP-A, TP-B, TP-C and TP-T05 to assess the bedrock underlying the proposed development area.

Table 2-1: Encountered Stratigraphy at T2

WTG	Exploratory Holes	Topsoil	Glacial Till	Weathered Siltstone/ Greywacke	Groundwater (m bgl)
		Depth to bottom (m)			
2	TP-T02	0.1	-	0.7*	Not encountered
2	TP-SPRING1	0.1	-	0.4*	Not encountered
2	TP-SPRING2	0.2	1.3	1.5*	Not encountered

**End of trial pit. Base of strata not proven*

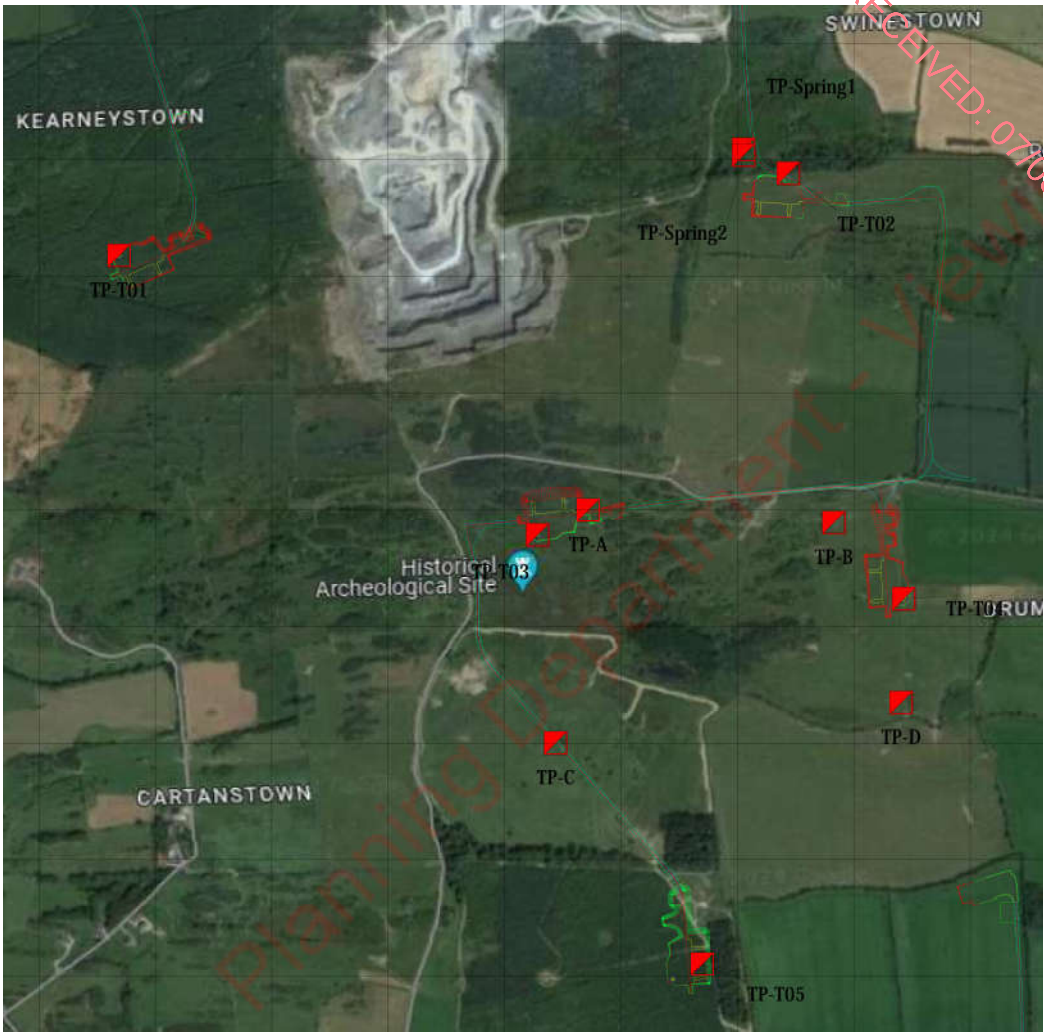

Figure 2-3 Ground investigation location of T2 (Ref. [10])

The results of the Point Load tests are summarised in GI report (Ref. [10]). The Point Load tests were completed with the load applied diametral to the rock samples and determined index strength $I_{s(50)}$ values ranging between 0.56MPa and 4.73MPa, with an average of 1.7MPa.

The rock samples can thus be described as being recovered from a strong rock. The Point Load test $I_{s(50)}$ values can be converted to equivalent UCS values using the UCS = K*$I_{s(50)}$. The K factor of 5-15 (average of 10) is estimated for siltstone/greywacke (Look 2007, Ref. [11]). The value of UCS (=10 x $I_{s(50)}$) ranges between 5.6 MPa and 47.3MPa with a representative design value of 10MPa, suggesting that the rock has a very low to high strength at the tested depths (Look 2007, Ref. [11]).

Based on the available sample density testing data summarised in GI report (Ref. [10]), one sample recovered in the bedrock recorded bulk densities of 2.4 Mg/m³, approximately and dry density of about 1.8 Mg/m³. The bulk density values fall between the typical range provided in Table 9.2 of Look, 2007 (Ref. [11]) of 22 kN/m^3 to 26 kN/m^3. The unit weight for the siltstone/greywacke ranges between 22 kN/m3 and 26 kN/m3, with an average of 24 kN/m^3.

Detailed drawings and GI findings are summarised in Ref. [10]

3 WTG FOUNDATION

As per the 6918-PL-502 WTG base drawings (Ref. [7, 12]), the base of the proposed WTGs across the site consists of a 27.2m diameter concrete structure, cylindrical in shape, which is reinforced and poured into a prepared excavation on the ground. It is noted that this is a planning drawing and that the final foundation dimensions will be subject to detailed design based on the specific WTG model. This report focuses on T2 due to its proximity to the quarry boundary (circa 100-130m from T2, Ref. [1, 2]). The proposed plan and cross section of the WTG foundation are shown Figure 3-1 below.

According to the GI report (Ref. [10]) and WTG details drawings (Ref. [7, 12]) infer that:

- The T2 base is to be situated on weathered non-rippable weak to moderate weak siltstone/greywacke bedrock (i.e. UCS range of 5-12.5MPa, Table 10.10 of Ref. [8]), and the formation level is 3m to 5m BGL (Ref. Ref. [7, 8]). Typically, any areas with soft or disturbed ground are identified and excavated, with the removed material replaced by well-compacted 6F2 material to establish a solid and reliable base for the WTG foundation.
- The proposed WTG will have a total tip height range of 179.5-180m, a hub height range of 98-105m, a rotor diameter range of 149-163m, and a tower base diameter range of 5.5-6m (Figure 3-2).
- The concrete foundation will be a shallow spread footing providing the necessary weight and stability to support the superstructure.
- The interface between the steel tower and the concrete foundation will consist of embedded anchor bolts.

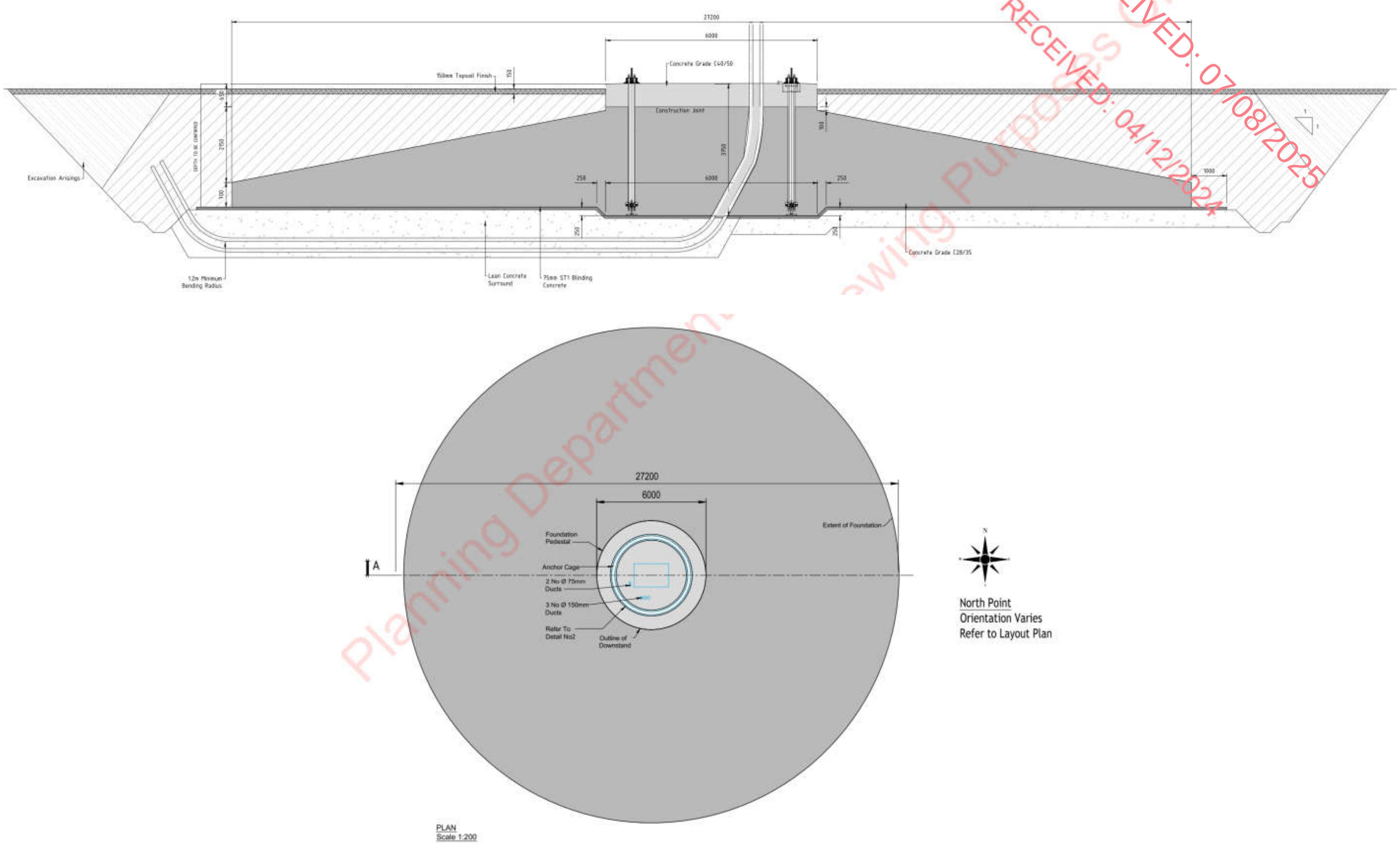

Figure 3-1: Typical WTG foundation detail (Drawing no: 6918-PL-502 Proposed foundation, Ref. [12])

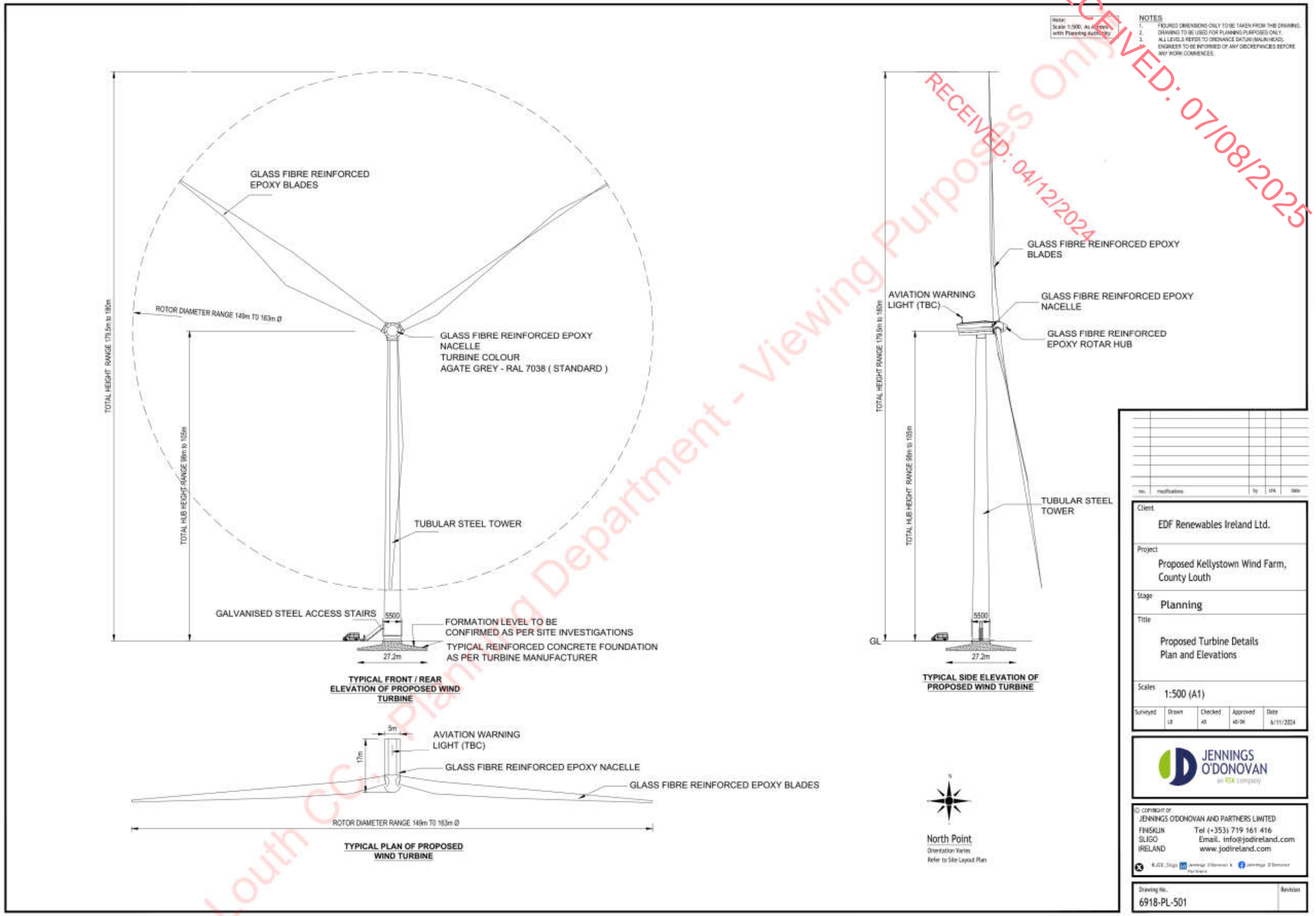

Figure 3-2: Proposed WTG details plan and elevation (Drawing no: 6918-PL-501, Ref. [7])

4 REVIEW OF STANDARDS, CODES AND LITERATURE

4.1 REVIEW OF LITERATURE

Ground vibrations can be generated either by natural phenomena or by human activities. This report evaluates the potential impact of Kilsaran quarry, an open-pit excavation (which uses blasting) located approximately 100-130m from T2, Ref. [1, 2]) from the nearest WTG T2. Similar scenarios have been previously investigated; for instance, Gómez-Márquez et al. (2010) [13] studied blasting in an open pit situated next to an existing wind farm located in NW Spain, as shown in Figure 4-1. According to the paper, the following conclusions were drawn:

- The prevailing registered ground frequencies range about 40-50 Hz.
- Spanish standard UNE 22-381-93 [14] with an allowable peak particle velocity of 53.3 mm/s is adopted.
- The safety distance of 200 m - recommended by the Mining Authority -, as a minimum distance between wind farms and mining exploitations to avoid vibration structural damage, is overestimated in this case.
- The minimum distance between the wind towers and the mine for the maximum charge weight per delay used - 168 kg of explosive - has been conservatively determined as 90 m. This distance could be even more reduced by diminishing the maximum charge weight per delay or changing the firing sequencing.
- It is worth noting that this distance is similar to the distance between the examined T2 from the blasting activities associated with Kilsaran quarry nearby (circa 100m).

Figure 4-1: Example of blasted open pit quarry close to an existing wind farm in Spain (Gómez-Márquez et al., 2010 (Ref. [13])

Norén-Cosgriff et al., 2020 (Ref. [15]) presented a study to test buildings constructed near a rock quarry in Norway. One building was of insitu concrete construction and the other of lightweight block masonry construction. Both buildings were founded on a layer of compacted gravel over rock. Blast tests produced vibrations up to 260 mm/s PPV in the frequency range of 50 to 130 Hz. No damage to either building was recorded.

Measurement methods for vibrations involve pre-blast inspections of existing structures, with specialists mapping existing cracks and stress zones to anticipate potential damage. European standards govern measurement equipment, typically focusing on velocity and frequency measurements, with additional considerations for acceleration in sensitive areas. Specific analyses are conducted for sensitive equipment, often guided by manufacturers' limits as a basis for assessment.

4.2 ALLOWABLE VALUES OF GROUNDBORNE VIBRATIONS – CODES AND STANDARDS

Construction activities such as blasting, piling, compaction, excavations, and construction traffic can produce vibrations of sufficient strength to cause damage to neighbouring buildings and structures. Therefore, many countries have national limit values for construction vibration in standards. However, structural damages assumed to originate from vibrations are seldom observed which may indicate that today's limit values are unnecessarily strict (see Ref. [15]). Allowable vibration values are usually given in terms of peak particle velocity (PPV) and are actually limiting values above which cracking of structures is more likely to be developed. Ground vibration guidelines are typically established for blasting sites to prevent damage to adjacent facilities or infrastructure. These guidelines are provided by various organizations or bodies. However, there are no specific standards that specify the vibration threshold for WTGs.

A rigorous examination of international standards and regulations was conducted to ascertain an allowable range for PPV as a function of vibration frequency. Vibration threshold PPV limits defined in most commonly referenced standards are primarily intended for conventional structures like residential, commercial and industrial buildings (buildings with windows and glazing, masonry or concrete walls, brittle finishes e.g. plaster etc). Wind turbines, on the other hand, are engineered structures with very different dynamic characteristics. WTGs are constructed with flexible materials and damping systems and are subject to different vibration frequencies and amplitudes compared to buildings. Moreover, WTG foundations are massive reinforced structures engineered to withstand extreme dynamic loading from the WTG operation, due to wind and mechanical loads due to operation. Typically, WTG foundations are therefore designed with very substantial amounts of steel reinforcement (Figure 4-2) to resist the deign load effects. The limiting values of particle velocity summarised in literature correspond to observed direct damages from ground vibration (Ref. [19]) for reinforced or framed structures, such as industrial and commercial buildings, which are typically more vulnerable to vibrations than WTG structures. **As such, the vibration criteria for WTGs are not typically governed by the same PPV limits used for buildings.**

Figure 4-2: Typical WTG foundation with reinforcement (Champ Chardon Wind Farm, France - Foundation design: 5 Senvion MM100, Ref. [20])

Following an extensive review of the international standards and regulations summarised in ITAtech Report (Ref. [21]), a comprehensive understanding of PPV limits across various jurisdictions was obtained and is presented in Figure 4-3.

It is worth noting that, according to BS 5228-2: 2009 (Ref. [22]) minor damage is possible at vibration magnitudes which are greater than twice those given in Table B-2, and major damage to a building structure can occur at values greater than four times the tabulated values. Therefore, for WTGs considered as "Reinforced or framed structures Industrial and heavy commercial buildings", the following PPV limits are proposed depending on the level of damage allowed (shown in Figure 4-3):

- Cosmetic damage, defined as the formation of hairline cracks on drywall surfaces or the growth of existing cracks in plaster or drywall surfaces, may occur at 50 mm/s or above.
- Minor damage, which includes the formation of large cracks or loosening and falling of plaster or drywall surfaces, or cracks through bricks/concrete blocks, may occur at 100 mm/s.
- Major damage, involving significant structural compromise such as damage to structural elements, cracks in support columns, loosening of joints, and splaying of masonry cracks, may occur at 200mm/s.

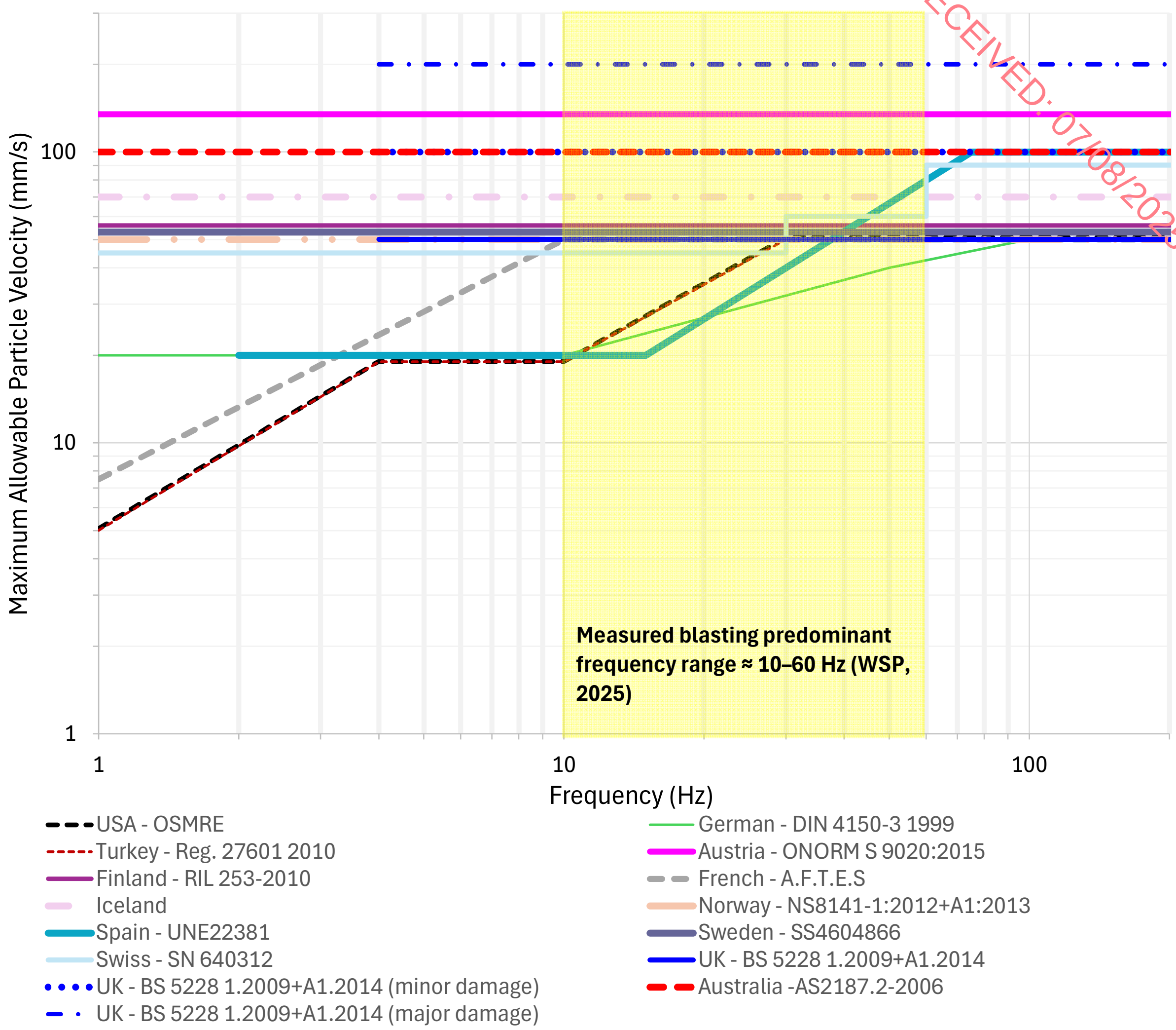

Figure 4-3: Comparison of various allowable PPV limits for structural damage

Based on ground vibration measurements summarised in WSP reports (Ref. [2, 23]) the predominant frequency range of 10-60 Hz with a representative frequency of 20Hz, and the allowable PPV limit ranges between 20-200mm/s. By incorporating these diverse standards, the resultant graph provides a holistic perspective on the permissible PPV levels concerning vibration frequency, which is crucial for evaluating the potential impact of blasting on WTG operations.

After an extensive review of blast vibration literature, it was identified that there are no vibration limits or guidance for carrying out quarry blasting operations adjacent to a wind farm consisting of a number of WTGs. However, it is worth noting that Terrock Ltd., an Australian consultancy completed a blast impact assessment for Willatook Wind farm PTY Ltd in April 2022 (Ref. [24]). They were engaged by Willatook Wind Farm Pty. Ltd. to assess the potential impacts of blasting at a proposed hard rock quarry at Old Dunmore Road, Orford, Victoria. They found that a ground vibration (PPV) limit of 100 mm/s (measured at WTG footings) is known to apply at other wind farms in the Western District of Victoria.

The managers of Golden Plains Wind Farm (under development near Rokewood, Victoria) have reported the 100 mm/s limit is based on advice from WTG manufacturers. While the strains imparted to WTGs from blast vibration are not known to have been locally assessed by structural design methodology, Terrock considered the 100 mm/s limit appropriate, considering the significant environmental loading WTGs are designed to withstand.

Terrock quoted AS2187.2-2006 Table J4.5(B) (Ref. [25]), which, for unoccupied structures of steel and concrete construction, recommends a PPV limit of 100 mm/s unless the owner agrees to a higher limit. Moreover, as reported in the Terrock report (Ref. [24])., Terrock is unaware of any instance of blasting operations at a nearby quarry causing adverse effects to wind farm infrastructure.

Based on the above, a ground vibration (PPV) limit of 100 mm/s (measured at WTG footings) is deemed acceptable and could be adopted for blast designs to avoid structural damage of WTGs.

5 WTG FOUNDATION ASSESSMENT

5.1 WTG SUPPLIERS' REQUIREMENTS

The FI Request related to vibration effects from blasting at Kilsaran Quarry stated that "*The applicant is therefore requested to provide confirmation from the turbine manufacturer that a higher PPV value could be appropriate for these structures.*" EDF has received confirmation from four major WTG suppliers (Ref. [26]), that a PPV of 100mm/s will have no adverse effect on the WTG. Large multi megawatt WTGs are designed and certified in accordance with the same unified standards. It may therefore be concluded that all WTG models that could potentially be installed at Kellystown Wind Farm will not be impacted by a PPV of 100mm/s.

WTG foundations are massive concrete structures with very substantial amounts of steel reinforcement (Figure 4-2) and blasting vibration are expected to have no adverse effect on the long-term strength and performance of the WTG foundation. To verify this, a simplified foundation assessment has been carried out and is presented in the following sections.

5.2 DYNAMIC DISPLACEMENT AND BASE ROTATION

WTG support towers are steel structures specifically engineered to withstand considerable loading resulting from extreme winds, and blade rotation. These towers, due to their height, possess a low natural frequency (typically <6 Hz, Ref. [27]), which contrasts with the higher frequencies (>10 Hz, Ref. [24]) typically associated with short-duration ground motions caused by blasting.

Consequently, the likelihood of dynamic resonance, which could potentially amplify the Peak Particle Velocity (PPV) levels up the towers, is minimal. The frequency from blasting vibration could range between 1Hz and 300Hz (see BS 7385, Ref. [28]), however, blast tests were carried out to assess the vibration level across the site. Ground vibration measurements summarised in the WSP report (Ref. [2, 23]) indicate that the predominant frequency of blasting vibrations at the site ranges between 10-60 Hz with a representative frequency of 20 Hz.

Therefore, a condition of structural resonance is likely to be ignored. The dynamic displacement caused by ground motion can be calculated using Equation 5-1.

$$s = \frac{PPV}{2\pi f}$$ Equation 5-1

Where s is the dynamic displacement, PPV is the Peak Particle Velocity and f is the frequency of blasting vibration.

Considering the PPV threshold of 100 mm/s threshold chosen for WTG structures by WSP (Ref. [2, 23]), a preliminary conservative assessment indicates a dynamic displacement at the base of WTG T2 between 0.27mm (assuming f=60Hz) and 1.6mm (assuming f=10Hz) founded on competent siltstone/greywacke rock.

The maximum angular distortion β=s/D (where s is the dynamic displacement calculated from Equation 5-1 and D the WTG base diameter) assuming a WTG base diameter D=27.2m (Figure 3-1) is ranging between 0.01 x 10^{-3} (~1/100.000) to 0.059 x 10^{-3} (~1/17.000). **These values of maximum angular distortion β are much lower compared to the 1/750 limit where difficulties with machinery sensitive to settlements are to be potentially vulnerable (Ref. [29, 30]),** as shown in Figure 5-1.

It is important to note that ground vibration waves are elastic waves, and the ground returns to its original position after blast wavefronts pass. The area of permanent ground deformation from quarry blasting is limited to the fracture zone within a few metres each blast hole. Therefore, **it is expected that quarry-scale blasting presents no risk of permanent ground displacement or subsidence beyond the blast site.**

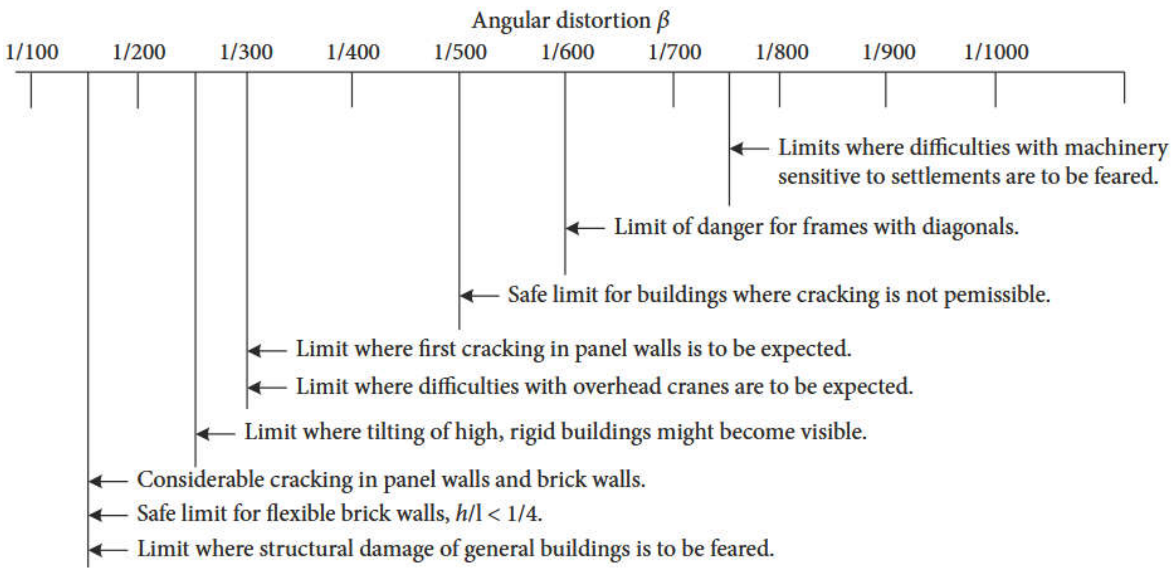

Figure 5-1: Limiting angular distortion (derived from Ref. [29, 30])

5.3 MOMENT DEVELOPED BY BLASTING-INDUCED VIBRATION

A simplified assessment of moment, M, developed by blasting-induced vibration is carried out. At very small angles of rotation θ of the WTG foundation, linear elasticity describes soil behaviour. The static ''effective'' stiffness $K_{R,elastic}$ of the M-θ response in rocking of circular-shaped foundations of radius, R (Figure 5-2) is calculated using Equation 5-2 (Ref. [31, 32, 33], and the moment loading M of a circular footing is calculated using Equation 5-3 and Equation 5-4.

$$K_{R,elastic} = \frac{8}{3}\frac{G \times R^3}{1-v} \qquad \text{Equation 5-2}$$

$$M = K_{R,elastic} \times \theta \qquad \text{Equation 5-3}$$

$$\theta = \tan^{-1}\left(\frac{s}{D}\right) \qquad \text{Equation 5-4}$$

Where $K_{R,elastic}$ is the static rotational stiffness, G is the soil shear modulus, R is the radius circular-shaped foundations, v is the Poisson ratio, and θ is the rotation of the footing.

The dynamic rotational spring stiffness $K_{R,dynamic}$ for embedded structures is calculated using Equation 5-5 (Ref. [34]). To account for embedment, the surface static spring value $K_{R,elastic}$ is multiplied by the embedment correction factor n_j. Finally, the result is multiplied by the dynamic coefficient α_j, in order to obtain the spring values for dynamic loads.

$$K_{R,dynamic} = K_{R,elastic} \times \alpha_j \times n_j \qquad \text{Equation 5-5}$$

Where $K_{R,elastic}$ is the static rotational stiffness, α_j is the dynamic correction factor, n_j is the embedment correction factor.

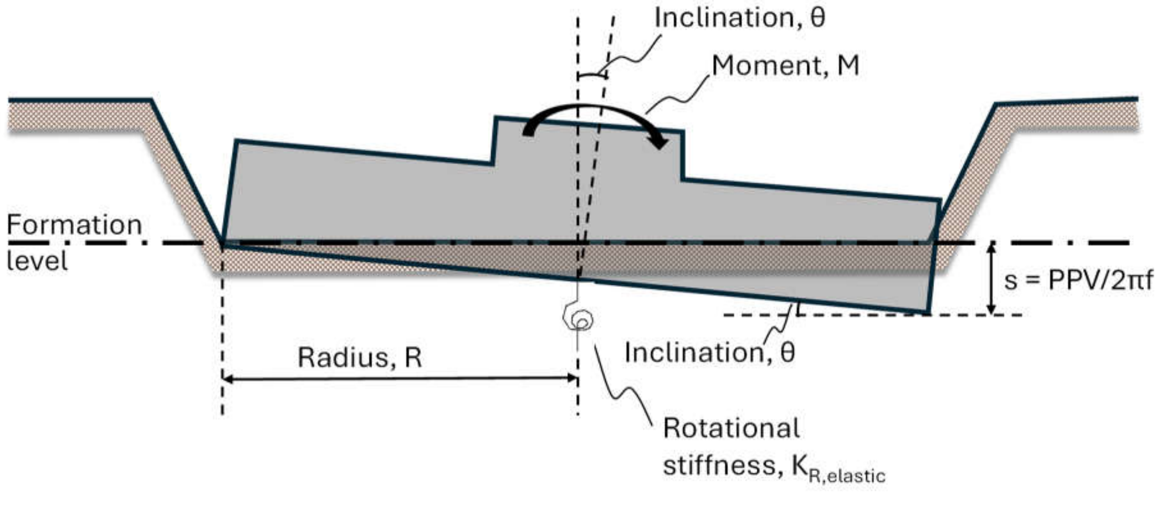

Figure 5-2: Sketch of rotation in a gravity-base foundation

The dynamic correction factors, α_j, is a function of the dimensionless frequency α_0 calculated using Equation 5-6.

$$\alpha_0 = \frac{\omega D}{V_s} = \frac{2\pi f_0 D}{V_s} \qquad \text{Equation 5-6}$$

Where f_0 is the structure's predominant frequency, D is the WTG base diameter, V_s is the small-strain shear wave velocity. It is noted that this frequency parameter α_0 is essentially unique for half-space conditions but may not be so in presence of a stiff stratum at shallow depth (Ref. [35, 34]).

5.3.1 KEY ASSUMPTIONS

The following assumptions were made to calculate the moment M developed by blasting-induced vibration:

- A rigid foundation always in contact with the soil is considered (Ref. [33]);
- Radius R of 13.6m is considered (Ref. [7]);
- Peak Particle Velocity (PPV) limit of 100mm/s is considered;
- The shear wave velocity of the rock $V_{s,30}$ at top 30m of 600m/s is considered (Site Class C i.e. 360m/s < $V_{s,30}$ ≤ 760m/s, Ref. [2]);
- Unit weight of 24kN/m^3 (section 2.2) is considered rock material based on laboratory measurements (Ref. [10]) and literature (Ref. [11]);
- Poisson ratio v of 0.25 (Ref. [11])
- Shear modulus G of 528MPa, approximately, is considered using the following two methods:
  - Method A: Using the empirical correlation between Young's modulus, E, and unconfined compressive strength (UCS) for rock as follows:

$$G = \frac{E}{2(1+v)} \quad \text{Equation 5-7}$$

$$E\ (MPa) = 146 \times UCS(MPa)\ (ref.[36]) \quad \text{Equation 5-8}$$

  - Method B: Using typical shear modulus reduction curves for rock (Ref. [37]) and maximum shear strain developed for a PPV limit of 100mm/s calculated as follows:

$$\gamma_{max} = \frac{PPV}{V_s}\ (Ref.\ [38]) \quad \text{Equation 5-9}$$

$$G_0 = \frac{\gamma}{g} V_s^2 \quad \text{Equation 5-10}$$

    Using the calculated small strain shear modulus G_0 and a maximum shear strain γ_{max}, the shear modulus G using Figure 2 of Ref. [37], is equal to $0.6 \times G_0$.

- Embedment correction factor n_j is ignored (i.e. n_j=1);
- Dynamic correction factors, α_j is ignored ($i.e.\ \alpha_j$=1) considering stiff stratum at shallow depth, Ref. [35, 34];

Based on the above dynamic rotational spring stiffness $K_{R,dynamic}$ of 4726.3 GNm/rad is calculated. The dynamic displacement s caused by ground motion can be calculated using Equation 5-1.

Considering the PPV threshold of 100 mm/s and a representative predominant frequency of blasting vibrations of 20Hz adopted by WSP (Ref. [2, 23]), a dynamic displacement s = 0.8mm, resulting in base inclination θ of 2.94x10^{-5} (in rad) or 0.0294mm/m. This results in a bending moment, M of 138 950 kNm, approximately.

Based on previous GDG experience on the foundation design for similar-sized WTGs, (i.e. rotor diameter range of 149-163m and hub height range of 98-105m hub height), the unfactored bending moment M developed at the top of foundation under extreme load conditions is summarised in Table 5-1. It is noted that these may not represent the exact loadings of the chosen WTG model but provide a useful indication of the order of magnitude of the design loads.

Table 5-1 Typical bending moment M developed under extreme load conditions

Size of WTG	Extreme Moment (unfactored) at top of foundation (kNm)
6.7MW 119m Hub H_t 163m rotor diameter	156 576
125m Hub H_t 149m rotor diameter	153 980 (includes seismic)
100m Hub H_t 163m rotor diameter	138 267
105m Hub H_t 149m rotor diameter	119 807
101m Hub H_t 158m rotor diameter	93 493
6MW 110m Hub H_t 150m rotor diameter	162 000

It is noted that base inclination θ of 0.0294mm/m is less than the 3mm/m (approximately 0.17°) limit set out by WSP for the WTG design (Ref. [23]). Moreover, the value of bending moment M, calculated by the aforementioned simplified method, is of the same order of magnitude with typical bending moments developed under extreme load conditions at the top of foundation of similar-sized WTG (Table 5-1). **Therefore, it is concluded that the WTG foundation can be designed to withstand the loads and displacements caused by blasting induced vibration.**

5.4 CONCRETE VIBRATION DAMAGE THRESHOLD

Tripathy and Gupta, 2015 (Ref. [39]) suggested the concrete vibration damage threshold should account for the density (ρ) and tensile strength of the concrete (f_t'), tests and real applications, resulting in the proposed safe vibration limit for concrete of:

$$PPV_{conc} = \frac{f_t'}{\rho V_c} \times 10^6$$ Equation 5-11

Where PPV_{conc} is in mm/s, $f_{t'}$ is in MPa, ρ is in kg/m3 and V_c is in km/s.

Assuming ρ=2500kg/m^3, V_c=3-4.5km/s (ref. [40, 41]) and f_t' =2.8MPa for C28/35, a safe vibration limit for concrete PPV_{conc} range of 248 - 373mm/s.

Moreover, Tripathy and Gupta, 2015 (Tripathy and Gupta, 2015 (Ref. [39]) also conducted field experiments with the goal of developing frequency-dependent safe blasting vibration thresholds for mass concrete. Large blocks (1200x600x1800 mm) of 16 MPa concrete were cast partially embedded into hard rock, cured for 28 days, and then charges were detonated at different distances. Tripathy and Gupta (2015) suggested the following damage thresholds expressions for vibration frequencies in the range of 40-500 Hz.

$$PPV_{conc} = \begin{cases} 17.6\omega^{0,355}\ for\ minor\ damage \\ 32.8\omega^{0,355}\ for\ major\ damage \end{cases}$$ Equation 5-12

Considering a representative predominant frequency of blasting vibrations of 20Hz (WSP report Ref. [2, 23]), a safe vibration limit for concrete PPV_{conc} range of circa 100 mm/s for minor damage to 182mm/s for major damage.

The effect of blast vibration on newly poured concrete has been well-researched by the Tennessee Valley Authority (TVA), which applies the vibration limits shown in Table 5-2 to projects requiring mass concrete pouring (Ref. [42, 24]). Considering WTG T2 is circa 100-130m from the quarry boundary (Ref. [1, 2]), a distance factor of 0.6 is applied to assess the effective PPV safe vibration limit for concrete. The TVA ground vibration limits used to protect curing concrete are high, and after 1-3 days is 135mm/s.

Table 5-2 TVA mass concrete damage criteria (Ref. [42, 24])

Concrete Age from Batching	Allowable PPV (mm/s)	Effective PPV (mm/s)	Distance Factor (DF)	
0-4 hrs	100 x DF	60	DF	Distance (m)
4 hrs – 1 day	150 x DF	90	1.0	0-15
1-3 days	225 x DF	135	0.8	15-46
3-7 days	300 x DF	180	0.7	46-76
7-10 days	375 x DF	225	0.6	> 76
>10 days	500 x DF	300	-	-

Based on the above, a safe vibration limit for concrete (PPV_{conc}) is above the recommended limit of 100 mm/s adopted by WSP (Ref. [2, 23]) and recommended from AS2187.2-2006 (Ref. [25]) for unoccupied concrete and steel structures.

6 CONCLUSIONS

Gavin and Doherty Geosolutions Ltd. (GDG) was requested by EDF Renewables Ireland Ltd. (applicant) to provide technical support to the FI Request from Louth County Council (Ref. [6]) related to vibration effects from blasting at Kilsaran Quarry. EDF Renewables were requested to provide confirmation from the WTG manufacturer that a Peak Particle Velocity (PPV) limit of 100mm/s - used in "Ground Vibration and Air Overpressure" assessment carried out by WSP (Ref. [2]) and questioned by quarry (Ref. [1]) - as the cosmetic damage limit could be appropriate for structures T1-T3 given their proximity to the adjacent quarry. The objective of this report is to provide confirmation that ground-borne design vibration PPV limit of 100mm/s from blasting activities in the quarry will have no adverse effect on the long-term strength and performance of the WTG foundation.

The assessment conducted by GDG provides insights into establishing a PPV vibration limit for proposed WTG T2 (closest to quarry boundary, Ref. [1, 2]), in anticipation of potential blasting activities associated with nearby quarrying activities. To evaluate the potential impact of blasting-induced vibrations on existing WTG operations located circa 100m from the quarry face (worst case future scenario as discussed in section 1.3), a thorough review (available at time of this report) of site investigations, WTG foundation designs, ground vibration assessment reports, planning drawings, specifications, guidelines, regulations and code provisions (at the international, regional and national level) was carried out.

The comprehensive analysis of international standards and regulations has yielded a range of acceptable peak particle velocity (PPV) limits as a function of frequency of vibration as shown in Figure 4-3. After an extensive review of blast vibration literature, it was identified that there are no vibration limits or guidance for carrying out quarry blasting operations adjacent to a wind farm consisting of a number of WTGs. However, Terrock Ltd quoted AS2187.2-2006 (Ref. [25]) which for unoccupied structures of steel and concrete construction, recommends a PPV limits of 100 mm/s.

Vibration threshold PPV limits defined in most referenced standards are primarily intended for conventional structures like residential, commercial and industrial buildings (light framed buildings with windows, masonry or concrete walls, brittle finishes e.g. plaster etc). WTG foundations are massive structures that are heavily reinforced to resist extreme operation loads, they are therefore inherently more resistant to vibration induced damage than framed buildings with brittle finishes.

Based on this comprehensive review, a ground vibration (PPV) limit of 100 mm/s (measured at WTG footings) is deemed acceptable and could be adopted for blast designs to avoid structural damage of WTGs.

Following communication between EDF and several WTG suppliers (Ref. [26]), it has been confirmed that a PPV of 100mm/s will not have an adverse effect on the WTG.

WTG foundations are massive concrete structures with substantial amounts of steel reinforcement and blasting vibration are expected to have no adverse effect on the long-term strength and performance of the WTG foundation. A **simplified foundation assessment was carried out** and key findings are summarised below:

- Maximum angular distortion β (i.e. 0.01 x 10-3 to 0.059 x 10-3) is much lower compared to the 1/750 limit where difficulties with machinery sensitive to settlements are to be potentially vulnerable (Ref. [29, 30]);
- It is expected that quarry-scale blasting presents no risk of permanent ground displacement or subsidence beyond the blast site;
- Estimated base inclination θ of 0.0294mm/m developed by blasting-induced vibration is less than the 3mm/m (approximately 0.17°) limit set out by WSP for the WTG design (Ref. [23]);
- Estimated bending moment M developed by blasting induced vibration is of the same order of magnitude and typical bending moments developed under extreme load conditions at the top of foundation of similar-sized WTG (GDG previous experience); and
- A safe concrete vibration damage threshold of 100mm/s will be incorporated into the development.
- The performance of the WTG foundation can be designed to withstand the loads and displacements developed by blasting-induced vibration.
- Furthermore, only a limited number of blasts would occur at this range (only to blast once per month, ref. [3 [4]]), as it marks the maximum extent of Kilsaran's permitted expansion, and blasting locations within the quarry vary, meaning this blasting point (i.e. 100-130m from T2) may not be used monthly.

Based on the above, a vibration PPV limit of 100mm/s from blasting activities in the quarry is appropriate for the WTG foundations. The loads and stresses associated with a PPV limit of 100mm/s can be incorporated into a standard foundation at detailed design stage. To ensure the long-term strength and performance of the WTG foundation as well as the stability and fatigue resistance of the WTG, monitoring and mitigation measures are summarised in Section 7 and will be implemented.

7 MEASURES TO BE ADOPTED AT DETAILED DESIGN STAGE

The distance between blasting operations and WTG T2 will be approximately 100m (representing a worst-case future scenario based on the full extent of the 25-year planning permission for the Kilsaran Quarry, ref. [3, 4]), therefore monitoring and mitigation measures are required, some of the potential methods are discussed below. This is not a comprehensive set of requirements, the developer will implement these as a minimum.

Microcracking in concrete and the response of WTGs and their components to blast-induced ground vibrations can be assessed by installing sensors (strain gauges, geophones, accelerometers, etc.), with alerts to notify if PPV exceeds thresholds (e.g. 90% of threshold limit). These can be placed at strategic locations on the structures and different vibration limits could be applied depending on the location of measurements based on structural design. Monitoring for cracks in a wind turbine foundation is possible for ensuring structural integrity and long-term performance. An overview of potential crack monitoring techniques is summarised below:

- Visual Inspections of the visible parts of the foundation
  - Regular site inspections using checklists and photographic documentation.
  - Focus on joints, anchor bolt areas, and high-stress zones.
- Crack Gauges (Tell-Tales)
  - Simple mechanical devices fixed across a crack to measure relative movement.
  - Useful for tracking crack width changes over time.
- Digital Crack Monitors
  - Use displacement sensors or LVDTs (Linear Variable Differential Transformers).
  - Provide continuous or periodic digital readings.
- Strain Gauges or Fibre Optic Sensors
  - Embedded in or attached to concrete to detect micro-strains that may indicate crack initiation.
  - Fibre optic systems can monitor large areas and provide real-time alerts.

A proposed monitoring schedule is summarised below:

- Routine inspections monthly or quarterly, depending on site conditions.
- Detailed inspection on an annual basis.

Prediction of particle displacement, velocity, acceleration, pressure and other parameters in the earth media resulting from an explosive detonation is a complicated and difficult task. To predict the vibrations generated by a production blast, empirical ground vibration predictions models are commonly used in practise (as such used by WSP, Ref. [2, 23] and AGL, Ref. [1]) following two broad approaches a) waveform superposition techniques and b) charge weight scaling laws (Ref. [43]).

These empirical formulas typically represent the earth medium as a homogeneous, isotropic mass and thus characterize the blast response within the medium using basic measures of linear elasticity (Ref. [44]). To capture natural soils and rock non-linear behaviour due to the large strains associated with a ground shock loading event, as well as the Soil-Structure-Interaction (SSI) effects which are expected to modify the vibratory wave characteristics propagating through the structure base, numerical modelling using finite elements (FE) software (e.g. PLAXIS 2D / 3D) to evaluate ground response to blast loading could be used to:

- evaluate the actions and displacements of WTG foundation induced by blasting; and
- further inform the findings (e.g. maximum PPV limit of 67.9mm/s for WTG T2) predicted by WSP (Ref. [2, 23]).

8 REFERENCES

[1] AGL Consulting (2025), "Rock Blasting Assessment Report - Impact Assessment of Rock Blasting at Kilsaran Quarry Gallstown", Doc. Ref. No. 24-240-R01, Rev 1, January 2025.

[2] WSP (2024), "Ground Vibration and Air Overpressure Blast Monitoring for the proposed Kellystown Wind Farm, Country Louth", Doc. Ref. No. 70123789.P04, dated 09 October 2024.

[3] Email from EDF to GDG, subject: "FW: Kellystown Wind Farm - Planning Support: FI Request Related to Vibration Effects from Blasting - GDG Rev 3", 04/07/2025 @10:58, [Online].

[4] LCC (2023), "Planning and Development Act, 2000 (as amended) Notification of Grant of Permission", Register Ref. No. 22190, 27/11/2023, [Online].

[5] Jennings O'Donovan (RSK company), "Kellystown Wind Farm Final layout", EIAR stage, dated 12/11/2024, Rev. 1.

[6] Further Information (FI) Request from Louth County Council to EDF Renewables Ireland, "Vibration effects from blasting at Gallstown Quarry", Dated 06/02/2025.

[7] Jennings O'Donovan (RSK company), "Proposed Turbine Details Plan and Elevation", Planning stage, Drawing Ref. No. 6918-PL-501, dated 6/11/2024.

[8] Jennings O'Donovan & Partners Limited (2024), "EIAR Chapter 10 Soils and Geology", November 2024.

[9] Whiteford Geoservices Ltd (2023), "Kellystown Wind Farm, Drogheda, Co. Louth, Ireland - Desktop Study and Walkover Survey for Preliminary Determination of Ground Conditions", Report No. 2200-23A Draft, 4th April 2023.

[10] Whiteford Geoservices Ltd (2024), "Kellystown Wind Farm, Drogheda, Co. Louth, Ireland - Preliminary Ground Investigation", Report No. 2200-23B, Rev01, 4th April 2024.

[11] Look (2007), "Handbook of Geotechnical Investigation and Design Tables", pp. 356.

[12] Jennings O'Donovan (RSK company), "Proposed Foundations General Arrangement", Planning stage, Drawing Ref. No. 6918-PL-502, dated 6/11/2024.

[13] Gómez-Márquez, I., García-Bastante, F., & Alejano, L. R. (January 2010). Blast vibration study to analyse the response of a wind tower close to an open pit mine. Rock Fragmentation by Blasting: Proc. 9th Int. Symp. on Rock Fragmentation by Blasting, Spain.

[14] UNE 22381:1993-04. Control de vibraciones producidas por voladuras [Control of vibrations produced by blasting] [Standard].

[15] Norén-Cosgriff, K. M., Ramstad, N., Neby, A., & Madshus, C. (2020). Building damage due to vibration from rock blasting. Soil Dynamics and Earthquake Engineering, 138, 106331..

[16] Svinkin, M.R., (2008), "Soil and structure vibration from construction and industrial sources", 6th International Conference on Case Histories in Geotechnical Enginerring, Arlington, August 11-16, pp 1-14 .

[17] Dowding CH. Construction vibrations. Upper Saddle River, NJ: Prentice Hall, 1996.

[18] Ambraseys, N.N. and A.J. Hedron, Jr. [1968]. “Dynamic behavior of rock masses”, in Rock Mechanics in Engineering Practice, (Stagg and Zienkiewicz, eds.), Chap. 7, John Wiley & Sons, New York.

[19] Athanasopoulos, G., & Pelekis, P. (2000). Ground vibrations from sheetpile driving in urban environment: Measurements, analysis and effects on buildings and occupants. Soil Dynamics and Earthquake Engineering, 19(5), 371-387. doi: 10.1016/S0267-7261(00)00.

[20] Champ Chardon Wind Farm, France - Foundation design: 5 Senvion MM100 (Source: https://gp-renewables.energy/meet-gp-wind/#projects-3).

[21] ITAtech (2016), "Vibration Control in Urban Drill and Blast Tunneling", ITAtech repor No 8/April 2016, ISBN: 978-2-9701013-7-6..

[22] British Standards Institution (BSI). (2014+A1:2014). BS 5228-2:2009 Code of practice for noise and vibration control on construction and open sites: Vibration. [ISBN 978-0-580-77750-9].

[23] WSP (2025), "Kellystown Wind Farm – Quarry Blasting Vibrations", Technical Memo, dated 18 April 2025.

[24] Terrock PTY LTD (2022), "Willatook wind farm PTY LTD - Blast impact assessement for the proposed Willatook wind farm quarry (WA007920)", .

[25] Standards Australia (2006). AS2187.2 Explosives: Storage and Use.

[26] Email sent from EDF to GDG, "FW: Kellystown Wind Farm - Planning Support: FI Request Related to Vibration Effects from Blasting - GDG Rev 3 (UPDATE TO REV4 : 25068-R-001-04)" on 30/07/2025 @ 13.35.

[27] Wang, S., Huang, Y., Li, L., & Liu, C. (2017). Dynamic analysis of wind turbines including nacelle–tower–foundation interaction for condition of incomplete structural parameters. Advances in Mechanical Engineering, 9(3), doi: 10.1177/1687814017692940.

[28] British Standards Institution (BSI). (1990). BS 7385-1:1990 ISO 4866:1990 Evaluation and measurement for vibration in buildings — Part 1: Guide for measurement of vibrations and evaluation of their effects on buildings.

[29] Zhang, X., & Zhan, Z. (2021). Evaluation of Risk of Building Damage due to Deep Excavations via Numerical Modelling. Advances in Civil Engineering, 2021, 1-16.

[30] L. Bjerrum, "Discussion on compressibility of soils," in Proceedings of the 1963 European Conference on Soil Mechanics and Foundation Engineering, vol. 2, pp. 26-27, Wiesbaden, Germany, September 1963.

[31] Gazetas, G., Anastasopoulos, I., Adamidis, O., & Kontoroupi, T. (2013). Nonlinear rocking stiffness of foundations. Soil Dynamics and Earthquake Engineering, 47, 83-91.

[32] Berthelot, P., Lamadon, T., Durand, D., Glandy, M., Aguado, P., Carpinteiro, L., ... & Puech, A. (2011). Recommendations for the design, calculation, installation and inspection of wind-turbine foundations-Final Version 1.1.

[33] Toña, R. J., Rojo, J. C., & Chandro, E. R. (2024). Foundations of onshore Wind turbines: current situation and trends. Informes de la Construcción, 76(573), 6443-6443.

[34] NIST, G. (2012). Soil-structure interaction for building structures. National Earthquake Hazards Reduction Program (NEHRP), 12.

[35] Anoyatis, G., and Mylonakis, G., 2012, "Dynamic winkler modulus for axially loaded piles," Geotechnique, Vol. 62, pp. 521-536.

[36] Vasarhelyi, B. (2003). Some observations regarding the strength and deformability of sandstones in dry and saturated conditions. Bulletin of Engineering Geology and the Environment, 62, 245-249.

[37] Anbazhagan, P., Prabhakaran, A., Madhura, H., Moustafa, S. S., & Al-Arifi, N. S. (2017). Selection of representative shear modulus reduction and damping curves for rock, gravel and sand sites from the KiK-Net downhole array. Natural Hazards, 88, 1741-1768.

[38] Chandra, J., Guéguen, P., & Bonilla, L. F. (2014, August). Application of PGV/VS proxy to assess nonlinear soil response–from dynamic centrifuge testing to Japanese K-NET and KiK-net data. 2nd ECEES, Instanbul, Aug. (pp. 25-29).

[39] Tripathy, G.R. and Gupta, I.D. (2015). "Safety criteria against blast vibrations for structures in green and ully cured concrete," Journal of Structural Engineering, Vol. 41, No. 5, pp. 468-474.

[40] Siorikis, V. G., Antonopoulos, C. P., Hatzigeorgiou, G. D., & Pelekis, P. (2024). Comparative Study of UPV and IE Results on Concrete Cores from Existing Structures. Civil Engineering Journal, 10(9), 2804-2819.

[41] Hussien, M., & Morgano, M. (1997). Structural integrity evaluation of concrete piles from stress wave measurements. NDT & E International, 30(4), 258-258.

[42] Oriard, L.L. (2002) Explosives Engineering, Construction Vibrations and Geotechnology, International.

[43] Ainalis, D., Kaufmann, O., Tshibangu, J. P., Verlinden, O., & Kouroussis, G. (2017). Modelling the source of blasting for the numerical simulation of blast-induced ground vibrations: a review. Rock mechanics and rock engineering, 50(1), 171-193.

[44] Castro, J. S., Bryson, L. S., Gamber, N. K., & Lusk, B. T. (2011). Numerical Modeling of Subsurface Blasts. In Pan-Am CGS, Geotechnical Conference.

GLOBAL PROJECT REACH

Offices

Dublin (Head Office)
Gavin & Doherty Geosolutions
Unit A2, Nutgrove Office Park
Rathfarnham
Dublin 14, D14 X627
Phone: +353 1 207 1000

Cork
Gavin & Doherty Geosolutions
First Floor, 12 South Mall
Cork
T12 RD43

London
Gavin & Doherty Geosolutions (UK) Limited
85 Great Portland Street, First Floor
London
W1W 7LT

Utrecht
Gavin & Doherty Geosolutions
WTC Utrecht, Stadsplateau 7
3521 AZ Utrecht
The Netherlands

Belfast
Gavin & Doherty Geosolutions (UK) Limited
Scottish Provident Building
7 Donegall Square West
Belfast
BT1 6JH

Edinburgh
Gavin & Doherty Geosolutions (UK) Limited
22 Northumberland Street SW Lane
Edinburgh
EH3 6JD

Rhode Island
Gavin & Doherty Geosolutions Inc.
225 Dyer St, 2nd Floor
Providence, RI 02903
USA

Website: www.gdgeo.com
Email: info@gdgeo.com

A Venterra Group Plc Member Company