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ANALYSIS, DESIGN AND CONSTRUCTION OF AN INNOVATIVE GROUND IMPROVEMENT SOLUTION FOR A SEISMICALLY-QUALIFIED STRUCTURE

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Abstract: *This paper concerns the geotechnical design of a heavy duty reinforced concrete extension (30m by 40m in plan) to a large defence-related production facility, subject to stringent tolerances on dynamic and static behaviour and interaction with adjacent facilities.*

The ground conditions were especially challenging. Half the extension is located over an infilled dock with the dock wall retained in situ. Under the other half of the extension is Victorian fill. Analysis of the early stage design for the facility demonstrated excessive settlement and a liquefaction hazard in the dock fill materials. Conventional solutions such as piled foundations or excavation and soil replacement were not compliant with the key criteria. An alternative innovative solution was selected comprising the foundation raft supported on a firm-firm interlocking secant pile block, eliminating liquefaction, safety and settlement risks.

A series of challenges remained, which were successfully addressed by a series of modelling, design and testing innovations, including advanced non-linear time-domain seismic numerical analysis, a world-first pile pull-out test to prove the secant joint strength, robust concrete mix design and construction monitoring.

Introduction

BAE Systems are undertaking a programme of major facilities development at their Barrow-in-Furness site to support ongoing and future submarine build activities. This includes major development on the licensed site, where site licensees must demonstrate control over the design and construction for safety related Structures, Systems and Components (SSCs) in accordance with nuclear site License Conditions (LC's). This includes control over the design and construction of works in the vicinity of existing facilities.

The D58 facility is an annexe to the existing Devonshire Dock Hall (DDH). The facility has been designed to withstand extreme hazard loads, and designed to ensure that it does not have an adverse effect on the adjacent DDH structure.

As part of the boat build programme, boat modules are required to transfer from the DDH into D58. Modules are to be transferred via a transfer rail system built in to the floors of the adjacent facilities. Stringent tolerances are to be maintained on the vertical alignment of the rails, and as such the design of D58 is required to limit differential deflections between D58 and DDH to ensure that the facility can support required transfer operations.

The design of the new facility has been subject to stringent approval processes and external regulatory permissioning from the UK Office for Nuclear Regulation (ONR).

This paper focusses on the development of the novel ground improvement solution and seismic soil-structure interaction analysis. A detailed treatment of liquefaction assessment and static analysis will be the subject of future papers.

Site conditions

Site History

Building D58 is located on the site of a marine channel which was developed into the Devonshire Dock in the 1860s as shown in Figure 1. A section of the dock wall obliquely crosses the D58 footprint. The Devonshire Dock Hall (DDH) was built on the site of the dock in the mid-1980s, the dock being backfilled with hydraulically-placed fill, including under the "wet side" part of the

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footprint of D58. The dock wall was partially broken down during the backfilling project. The fill was improved with vibro-compaction and surcharging. As-built records showed that the D58 site was on the periphery of the surcharge fill and hence may not have benefited from the improvement techniques to the same degree as the DDH.

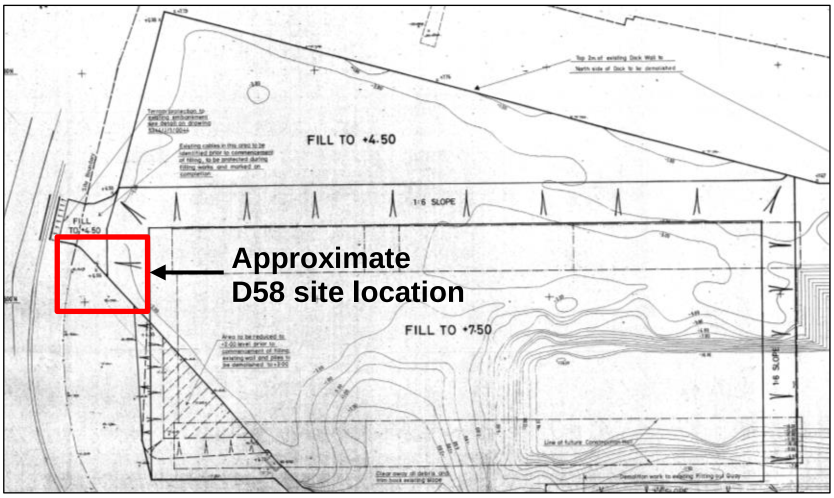

Figure 1. Devonshire Dock fill plan.

Ground and groundwater conditions

The typical stratigraphy below D58 is illustrated in Figure 2. The upper part of the site stratigraphy consists of made ground from several phases of development, split by the buried dock wall. On the "wet side" of the wall, the ground investigation suggested two layers within the fill: a denser, more compacted granular fill which overlies a looser granular silty deposit; underlain sporadically by a thin layer of soft generally silty dock sediment. On the "dry side" of the dock variable made ground, presumed to be backfill to the dock wall, overlies the natural strata. The natural strata comprise a succession of Glacial Till units overlying bedrock of weathered mudstone.

Ground conditions were extensively investigated by a detailed intrusive ground investigation. This included various geophysics techniques and advanced laboratory testing carried out on undisturbed samples obtained from the investigation to establish the dynamic parameter values needed for soil-structure interaction analysis.

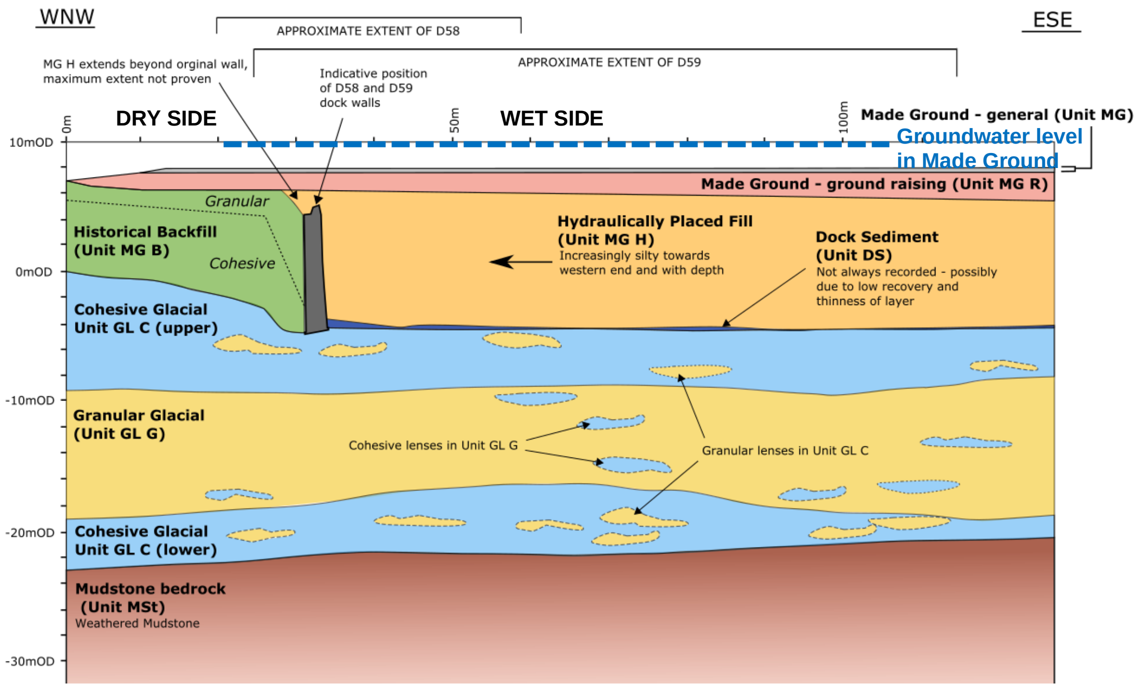

Figure 2. Indicative ground model section.

Foundation design

Performance criteria

The building foundation was subject to onerous seismic performance criteria of resistance to seismic hazard events with a return frequency of at least 10^{-4} per year with no cliff edge beyond the Design Basis; hazards include structural collapse and liquefaction.

The client required seismic design to be undertaken using Principia Mechanica Ltd (1981) piecewise linear spectra with an earthquake magnitude range at design basis of M = 5 to 6.

In addition, the following static performance criteria were established:

1. maximum differential settlement of 4 mm over 10 m in any direction,
2. strict relative deflection criteria between the D58 building and the adjacent DDH, and
3. limits on construction-induced deflection on the adjacent DDH.

(The details of the liquefaction assessment and static performance analysis will be the subject of future papers).

Foundation concept design

Three general foundation concepts were considered for Building D58. These were:

1. raft foundation,
2. deep foundation, and
3. raft foundation with ground improvement.

A **raft foundation** supported by the existing ground profile would experience substantial foundation deflections which significantly exceed the deflection criteria and risk failure of the foundation along the line of the dock wall. Therefore, this option was discounted.

There were significant technical challenges for reliable and robust design of **piles or other deep foundation** options under seismic loading cases, particularly given the ground conditions present where significantly non-linear soil behaviour and potential liquefaction were predicted. This would require large diameter piles or complex diaphragm or barrette arrangements which would require significant reinforcement and would be complex to construct. Furthermore, the complex kinematic and inertial interactions between the soil, buried wall, pile group and superstructure would make prediction of raft level response spectra more complex and less reliable than seismic soil-structure interaction (SSI) assessment of raft foundations. This is supported by the statement in IAEA NS-G-3.6 (2004):

> *"Owing to the complexity of the design, shallow foundations are usually considered first, the option of deep foundations being considered as a last resort."*

The outcome of the general foundation option selection process was the selection of a **raft foundation supported on improved ground**. Improving the ground en masse to increase its strength and stiffness reduces settlements and removes the risk of significant cyclic softening or liquefaction. It also removes buried structural elements that are difficult to inspect from the solution and enables a conventional approach to seismic SSI analysis and providing inputs to seismic structural analysis.

Improved ground option selection

Ten improved ground options were shortlisted under three broad headings of densification, reinforcement (or replacement) and, grouting and mixing.

Densification options were rejected either because they had achieved limited success previously on the site, were not appropriate given the site constraints, or had practical construction problems. Despite being previously subject to vibro-compaction and surcharging, the detailed ground investigation proved a significant thickness of unimproved loose material towards the base of the hydraulic fill.

Excavating and replacing the hydraulic fill and backfill to the buried dock wall was discounted due to the risk of generating unacceptable ground movements by forming an up to 14 m deep excavation, with associated dewatering, immediately adjacent to the DDH.

Of the reinforcement options, settlement-reducing piles were considered viable if they could be formed into a continuous block of material. Grouting and mixing options were also considered viable but only if mechanical or jetting techniques were used to ensure the extent of grouting was controlled.

The outcome of the improved ground option selection was the shortlisting of three options: jet grouting, soil mixing and secant piles. These options were compared, considering technical appropriateness, reliability of design process, foundation performance, ease of validation, constructability, sustainability, regulatory approval, cost, programme; and plant availability.

Bachy Solentache were consulted to provide independent contractor input into the constructability, cost and programme elements of the assessment. The results of Atkins', Bachy Solentache's and the Principal Contractor Morgan Sindall's combined assessment resulted in the selection of a **secant piled block** as the preferred ground improvement option. This innovative solution comprised the raft supported on a cuboid block of firm-firm interlocking piles. This replaced a 10 m depth of poor-quality backfill, eliminating liquefaction, safety and settlement risks.

Key benefits of the solution over other ground improvement options were:

- low carbon footprint concrete with predictable, controlled, well-defined properties;
- minimal risk of refusal or volumes of untreated ground; and
- comparable or lower costs;

Nonetheless a series of challenges remained:

- substantiating a novel solution to pass scrutiny of the industry regulator;
- demonstrating secant pile joints remain elastic under seismic loading;
- designing and testing a concrete mix with sufficient strength but delayed hardening for secant piling outside industry-norm criteria;
- achieving tight construction tolerances on position, verticality, and joint roughness; and
- ensuring and monitoring acceptable settlements of the existing facility.

Seismic SSI analysis

The seismic design of the building was undertaken as a two-step process:

1. A geotechnical SSI analysis was carried out using a 3D model of the ground and a reasonably (but not fully) detailed model of the building.
2. A fully detailed structural model with the soil represented as springs and dampers. Values for these were obtained from this geotechnical model as well as foundation input motions.

Two different geotechnical models were used as shown in Figure 3. A model with minimal structural mass was used to obtain the foundation input motions as these need to consider only the kinematic effects of SSI. By contrast the soil springs and damping values needed to consider the effects of building inertia and were obtained from analysis run with full structural mass.

The controlling standard for SSI analysis was ASCE 4. During the course of the project this standard was revised from ASCE 4-98 (1998) to ASCE 4-16 (2016). A detailed assessment of the revisions to the standard was undertaken to take due cognisance of changes in the standard.

SSI software was selected based on the following considerations:

- Some strata present on the site have low small-strain stiffness and the model needed to capture large-strain non-linear behaviour.
- The structure is poorly suited to two-dimensional analysis. The effects of shear walls in a relatively square structure, varying wall thicknesses, large openings and large live loads offset from the foundation centre would be difficult to capture in a 2D model.
- The foundation slab is underlain by the buried dock wall running at an oblique angle to the foundation edges. It splits the underlying strata into two distinct profiles that have different properties. These complex ground conditions will produce 3D effects which are reduced but not entirely mitigated by the ground improvement.

Given the above points, the geotechnical analysis required a 3D numerical modelling package with the capability to model dynamic, non-linear soil behaviour at high strain levels. The software FLAC3D (Itasca Consulting Group, 2012a) is an explicit finite difference code with the required capabilities and was selected for the seismic modelling. The 3D numerical model geometry is shown in Figure 4.

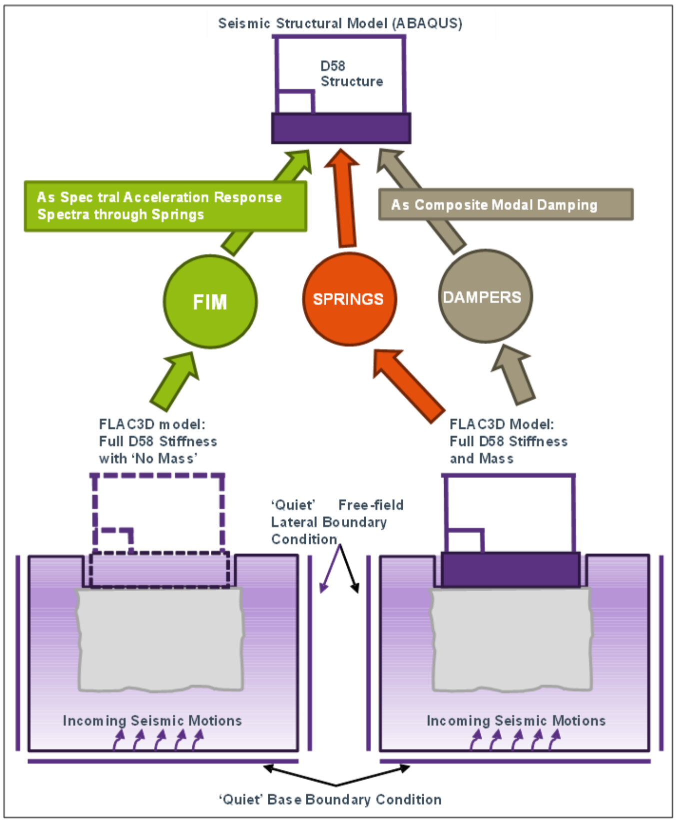

Figure 3. SSI methodology.

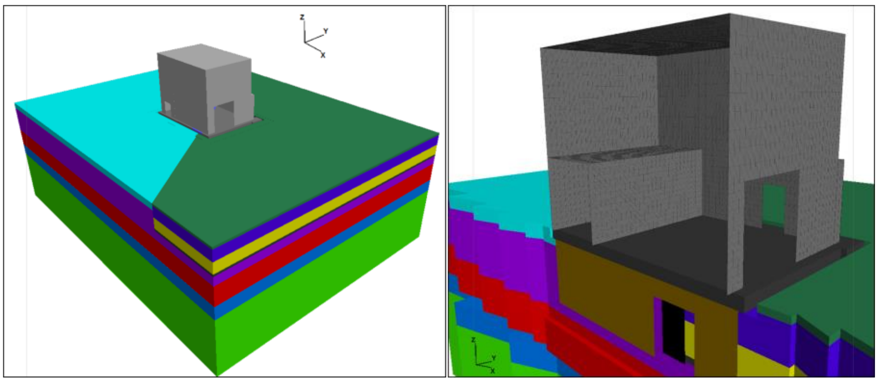

Figure 4. 3D numerical model geometry.

Non-linear soil behaviour

The dynamic stress-strain behaviour of the soil is determined from advanced laboratory testing of soil samples, benchmarked against published relationships. This behaviour is then represented by a suitable, idealised constitutive model. A non-linear, constitutive soil model was used. No yield criterion was specified, but non-linear stiffness degradation was employed using a versatile sigmoidal model (Itasca Consulting Group, 2012b) which defines both stiffness and damping curves by the following single equation:

$$M_s = y_0 + \frac{a}{1 + \exp(-(L - x_0)/b)} \tag{1}$$

where M_s is the normalised secant modulus, a, b, x_0 and y_0 are the calibration parameters, and $L = \log_{10}(\gamma)$, which is the logarithmic shear strain. The normalised secant modulus is given by G_{sec}/G_0, which is the ratio of secant to small-strain shear modulus. The bulk modulus does not vary with strain.

In addition to the hysteretic damping developed by a cyclic, non-linear numerical model, a low level of Rayleigh damping was applied to control high frequency noise during computation and to represent non-hysteretic small-strain material damping.

The shear modulus degradation and damping curves were fitted to data from Resonant Column Tests (RCT) and Cyclic Triaxial Tests (CAUcyc) of intact and reconstituted soil samples as shown in Figure 5. The Cyclic Triaxial results were only used for values greater than 0.01% shear strain as the results for lower strain levels were not considered sufficiently reliable. The shear modulus data from Cyclic Triaxial Tests were normalised by the maximum shear modulus of the given cycle and scaled to the normalised Shear Modulus (G/G_0) from representative Resonant Column Tests at 0.01% shear strain. The sigmoidal hysteretic model parameters were then optimized to produce the best fit between both normalised shear modulus and damping.

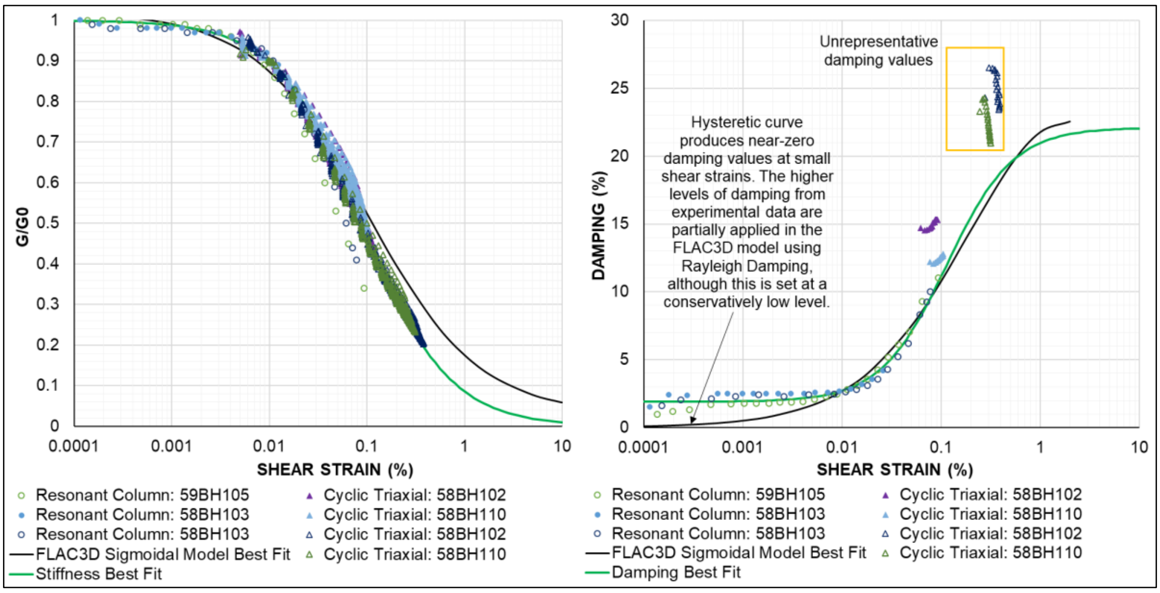

Figure 5. Strain-dependent stiffness and damping curves.

Model runs

To comply with the requirement of ASCE 4-98 (1998), three sets of independent time histories were used in combination with three sets of soil stiffness profiles: lower bound, best estimate and upper bound. As explained above, separate kinematic and inertial analysis needed to be run for the foundation input motions and the soil springs. Therefore, a total of 18 model runs were required as shown in Figure 6. In addition, three sinusoidal analyses were used to evaluate the soil damping.

Synthetic ground motion acceleration time histories were generated for use in the seismic analysis, targeting the PML design response spectrum, scaled to a peak ground acceleration of 0.25 g. The ground motion histories were checked against the requirements of ASCE 4-98 (1998) and ASCE/SEI 43-05 (2005) in order to closely match the design response spectrum. The acceleration time histories were also baseline-corrected to remove drift effects.

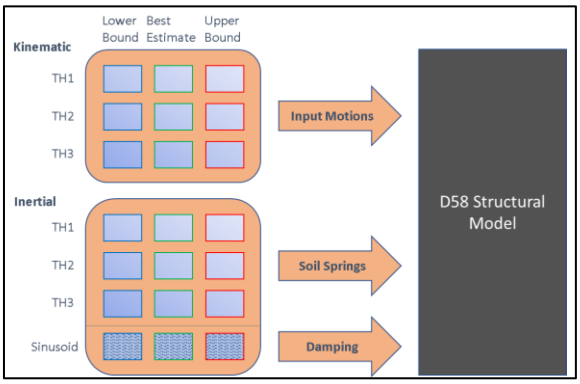

Figure 6. Model analysis runs diagram.

The time-based models allowed shear strains to be evaluated in every zone at every time step over the course of the analysis, with soil stiffness and damping behaviour constantly updating. Figure 7 shows a snapshot of the analysis in progress, with soil zones coloured by shear strain.

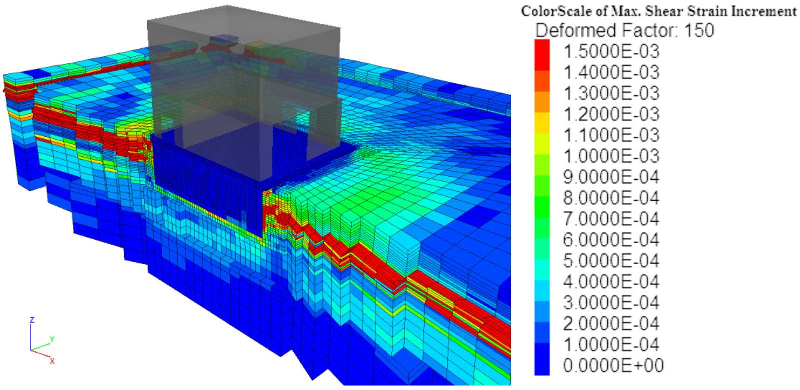

Figure 7. Model analysis showing shear strain development.

Model boundaries

The model boundaries were set sufficiently far from the building that they did not significantly affect the building response. This was verified using a series of sensitivity checks on the lateral extent of the model. The base depth was verified by comparing maximum shear strain profiles for free field and inertial analysis models, confirming that the shear strain at depth was not significantly changed.

At the base of the model a ‘quiet’ boundary condition was applied to absorb reflected waves. In reality, these waves radiate downwards beyond the model base and dissipate into the underlying ground. The lateral edges of the model were coupled to free field boundaries, also via viscous dashpots to provide ‘quiet’ lateral boundaries. The free-field boundary enforces free-field motions at the edges of the mesh and thereby supplies the conditions of an infinite model extent. Secondary outward propagating waves originating from within the mesh are absorbed such that the lateral boundaries retain their non-reflecting properties.

SSI results

The foundation input motions for all time history sets and soil profiles were combined and were found to marginally exceed the PML design response spectra as shown in Figure 8a. A series of modified response spectra envelopes were produced for different damping levels for use in the structural analysis as shown in Figure 8b.

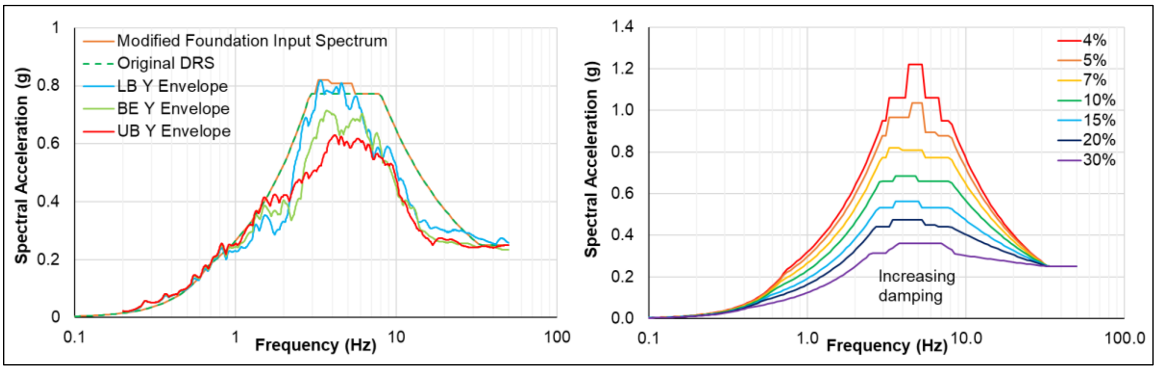

Figure 8. Foundation input motions (a) Combining results for LB, BE and UB soil profiles and (b) Results for different damping levels.

Seismic soil springs were for use in the detailed structural analysis were determined by pseudo-static analyses undertaken on models with degraded soil stiffness values. Separate degraded models for the BE, LB and UB ground profiles were generated from the results of the inertial analyses. This method allows a detailed soil spring distribution to be assessed for each direction, as illustrated in Figure 9.

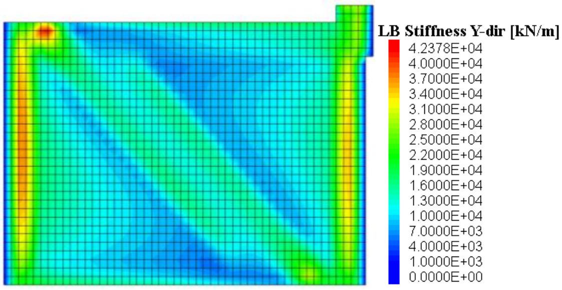

Figure 9. Example of dynamic soil spring distribution.

Damping values were obtained by applying a sinusoidal input motion in a given direction and measuring the attenuation decay rate of the building motion after abruptly halting the input motion.

Detailed design of improved ground

The details design of the firm-firm interlocking piles commenced with a value engineering exercise to reduce the number and length of the piles. 'Honeycomb' and 'Grid' layout options were considered to reduce the number of piles compared to the fully secanted base case as shown in Figure 10. The honeycomb layout was readily constructable without guide walls and provided an achievable piling sequencing. The layout contains significant redundancy as each pile interlocks with four others and its greater inherent stability under lateral loads led to its adoption in preference to the grid arrangement.

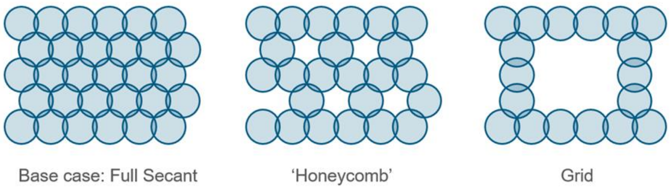

Figure 10. Interlocking piles value engineering options.

The critical design consideration for the interlocking piles was its resistance to complementary shear under seismic loading. Figure 11 indicates the vertical shear load applied through the pile interlocks as a result of complementary shear action generated by horizontal seismic shear action.

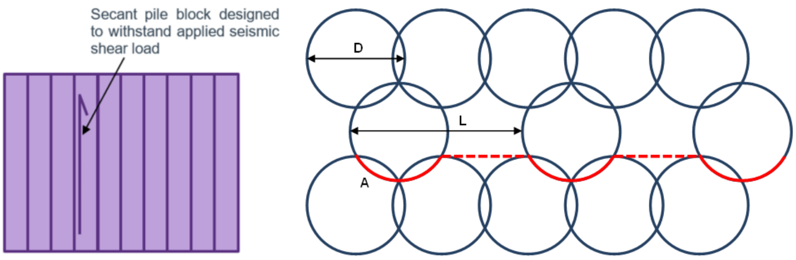

Figure 11. Illustration of (a) shear load applied through pile interlock (b) section long which the shear load is applied

Given that the secant piles were cast at different times, the allowable shear stress on the secant interlock areas were conservatively assessed as a cold joint following the methodology described in Eurocode 2 (BS EN 1992-1-1) using the following equation:

$$v_{\mathrm{Rdi}} = c \cdot f_{\mathrm{ctd}} + \mu \cdot \sigma_{\mathrm{n}} + \rho \cdot f_{\mathrm{yd}}(\mu \sin\alpha + \cos\alpha) \leq 0.5 \cdot \nu \cdot f_{cd} \qquad (2)$$

Where: c, μ – are factors which depend on the roughness of the interface;

f_{ctd} – design value of concrete tensile strength;

σ_n – stress per unit area normal to the interface that acts simultaneous to the shear force;

f_{yd} – design yield strength of reinforcement (term equates to zero as there is no reinforcement).

Based on the shear stress capacity derived from the above equation, the pile interlock strength can be calculated using the area of interlock. However, this area will vary from its nominal value according to the geometry achieved during construction, as shown in Figure 12.

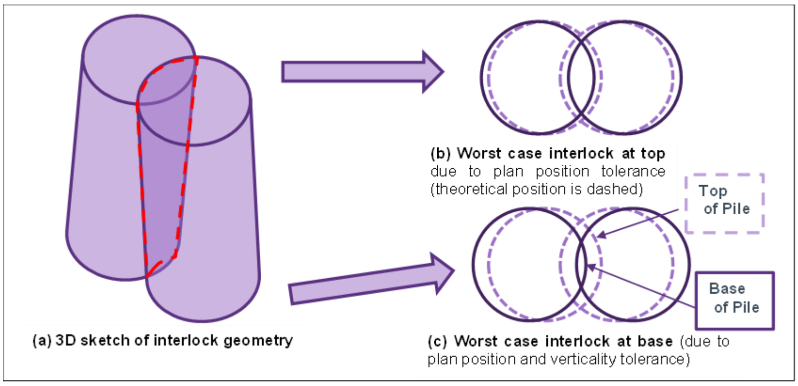

Figure 12. Illustration of pile interlock geometry.

A conservative probabilistic assessment of interlock arc length was used to assess overall interlock area for the IG block. Seismic shear demand was compared to the shear capacity and a utilisation factor of 0.95 was determined for a conservative estimate of pile interlock area.

Pile testing

The secant pile design included novel aspects, particularly the interlocks which needed to support a shear load. Also, the utilisation factor was close to one and was based on assumed values for the factors c and μ. Therefore, it was considered necessary to undertake a full-scale pile test to confirm the pile interlock shear capacity. The test rig is shown in Figure 13 and was designed to pull out a test pile from surrounding secanted piles whilst load and deflections were monitored.

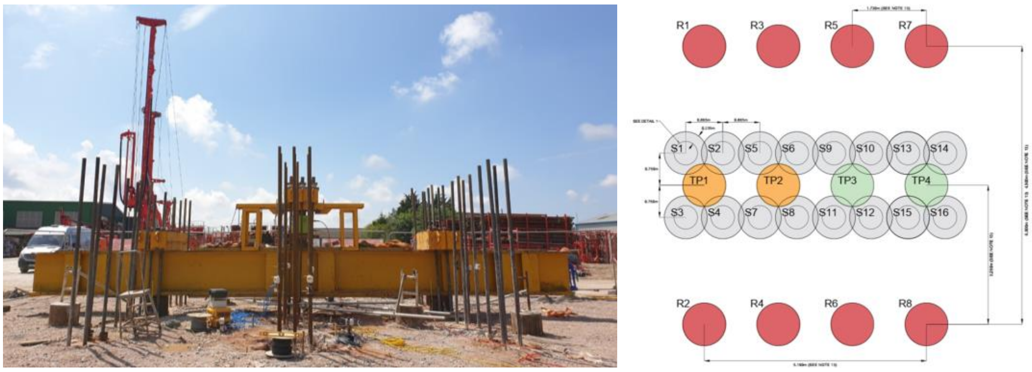

Figure 13. Pile test rig and layout.

The test demonstrated that the shear strength was acceptable, although based on the test data the utilisation increased to 0.97. Therefore, given the variable geometry and its effect on shear capacity, careful monitoring and control of the construction tolerances was essential.

Construction

Following the pull-out pile test, on-site trial piles were undertaken to verify the planned construction techniques. A total of 1651 piles were installed in the main works. Pile tolerances were 1 in ≥ 150 vertical and < 50mm position. The resulting secanted piled block is shown in Figure 14.

The as-built pile positional and verticality data was recorded for each pile during installation. Verticality measurements were obtained using two independent systems: a rig-based PRAD monitoring system and an Electronic Distance Measuring (EDM) surveying technique. The pile installation positional data was assessed to confirm whether or not the target secant pile intersection arc lengths and toe overlaps had been achieved using a 3D digital model. This enabled a rapid turnaround from receiving as-built data to specifying additional piles where the required capacity was not achieved.

Thermal integrity profiling testing was undertaken using fibre optic cables attached to a steel cage inserted to the full depth of the pile to measure changes in temperature during concrete curing. These measurements were interpreted to assess whether there were any significant soil inclusions in the pile concrete. All piles tested demonstrated that pile integrity was satisfactory.

Figure 14. Exposed piles during construction.

Conclusions

The design of D58 was required to meet stringent design criteria, with the design solution incorporating a novel secant pile ground improvement works solution which has been shown to prevent seismic amplification as well as addressing liquefaction concerns, and limit differential settlements and deflections between D58 and the adjacent DDH structure.

The ground improvement works design was effective in delivering a solution that supported the seismic qualification of the facility, including a demonstration that the design will perform in an essentially elastic manner under design basis loads with a demonstrable margin for events beyond the design basis.

Site testing of the secant pile arrangement in advance of construction was essential in underpinning the robustness of the design, and validating the parameters used in design. This supported the design in achieving acceptance through a comprehensive process of design review, approvals and regulatory permissioning to enable construction works to begin on site.

Acknowledgments

A collaborative approach between BAE Systems, Atkins, Morgan Sindall, and Bachy Soletanche successfully delivered a high-quality innovative foundation solution which met an extensive range of performance criteria in highly variable and poor ground conditions. The authors would like to acknowledge the work undertaken by the following colleagues over the course of the project:

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