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The design and construction of new quay walls and gates at the Port of Boston, UK

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ABSTRACT: The Environment Agency's £100m Boston Barrier Tidal Flood Defence Scheme will protect 14,300 properties from tidal flooding in Boston, UK. Delivered by a Design & Build Joint Venture between BAM Nuttall and Mott MacDonald, the scheme includes two major hydraulic structures, and 1.2km of new sheet piled quay wall with a retained height typically in excess of 10m. The paper describes the geotechnical design and construction of the new quay walls, including their anchorage systems which variously comprise continuous and tubular pile anchor walls, pile groups, and drilled anchors. These structures were significantly complicated by their interaction with the existing historic port structures, which are variably in a poor state of disrepair, as well as ongoing port operations. Use of a comprehensive BIM model was paramount for the development of innovative solutions to overcome these challenges. The design incorporated interpretation of advanced ground investigation laboratory and field tests, selection of design non-linear stiffness parameters, numerical soil-structure-interaction analysis, and validation of results based on site observations.

RÉSUMÉ : La barrière de protection de Boston contre les inondations causées par les marées est un projet de 100 millions de livres sterling pour l'Environment Agency. Ce projet protégera 14 300 propriétés contre les inondations liées aux marées à Boston au Royaume-Uni. Délivré par une coentreprise de conception et de construction composée de BAM Nuttall et Mott MacDonald, le projet comprend deux ouvrages hydrauliques majeurs et 1,2 km de nouveau quais en palplanches dont la hauteur de retenue est généralement supérieure à 10 m. Ce papier/cet article décrit la conception géotechnique et la construction des nouveaux quais, y compris les systèmes d'ancrage; notamment les rideaux de pieux tubulaires et continus, les groupes de pieux, et les ancrages forés. Ces structures ont été considérablement compliquées dues à leur interaction avec les structures portuaires historiques existantes – étaient qui sont dans un état de délabrement- ainsi que par les opérations portuaires en cours. L'utilisation d'un modèle BIM détaillé était primordiale pour le développement de solutions innovantes pour surmonter ces défis. La conception a intégré l'interprétation de tests avancés en laboratoire et sur le terrain de l'étude des sols, la sélection des paramètres de conception en termes de rigidité non-linéaires, l'analyse numérique des interactions sol-structures, et la validation des résultats en fonction des observations sur site.

KEYWORDS: Sheet piles and cofferdams, anchors and anchorages, finite-element modelling, ports, docks and harbours.

1 THE PROJECT

Being delivered by BMMJV, a Design & Build Joint Venture between BAM Nuttall and Mott MacDonald, the Environment Agency's £100m Boston Barrier Tidal Flood Defence Scheme will reduce the risk of tidal flooding to 14,300 properties from tidal flooding in Boston, UK.

Boston, Lincolnshire is located on the east coast of the United Kingdom, off The Wash, and has a history of tidal surges. Extensive flooding occurred in 1953, 1978, and most recently in December 2013, when over 800 properties and 55 streets were flooded. The project provide protection against tidal flooding with an annual exceedance probability of 0.33%, for the 100-year project design life.

The site is located at the tidal section of the River Witham, known as The Haven. The tidal barrier is strategically located upstream of the entrance to the Port of Boston (PoB) to mitigate the need to design the barrier structure for passage of cargo vessels (see Figure 1), and downstream of the entrance to the South Forty Foot drain to keep open the opportunity for future development as part of the Fens Waterways Link project.

1.1 *New works*

The main structures comprising the flood defence scheme are:

- Rising sector gate (RSG): 362 tonnes, 25m clear width and 11m high gate, to be raised during tidal surges and protect Boston from flooding;
- Twin wall bypass cofferdam structure 60m long by 35m wide to close the river path adjoining the RSG:
- Barrier control building: for operation and maintenance of barrier;
- Left Bank: new 590m long quay wall formed by 19m long sheet piles anchored variously to tubular piles, ground anchors and three new load relieving platforms for port crane operations;
- 830m long and 2m high new reinforced concrete flood wall within the live Port of Boston
- New knuckle structure at the junction of the river Witham and widened Wet Dock Entrance (WDE);
- Wet Dock Entrance: new 18m wide and 11.5m high vertical sector gate (VSG) to widen the existing wet dock entrance;
- Right Bank: new 525m flood defence wall formed by up to 19m long anchored and cantilevered sheet piles and earth embankments.

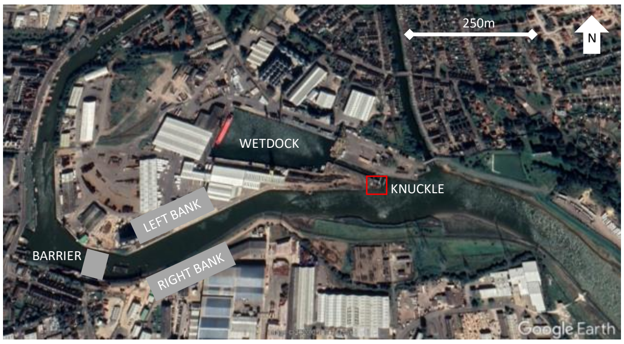

Figure 1. Site layout

Figure 1 above shows the site layout. At the time of preparation of this paper, the RSG, control building and bypass channel structure were complete, as well as the Right Bank. The remaining of the works will be completed in 2021/2022. Details of the Right Bank works are presented in Gelder et al (2020).

1.2 *Ground and groundwater conditions*

The ground conditions at the site comprises Jurassic aged Ampthill Formation at depth, overlain by very stiff Glacial Till and localised fluvioglacial deposits, which are in turn overlain by saltmarsh deposits including the Barroway Drove Beds. Above the natural deposits is extensive made ground comprising the pre-existing flood defence embankment on the Right Bank, and the PoB on the Left Bank. Groundwater is at 2-3m below ground level. The history of Port of Boston is summarised in Gelder et al (2020).

A 3D model of the site stratigraphy was developed in Leapfrog Works and integrated into the federated BIM model for the project. An extract of the model is shown in Figure 2.

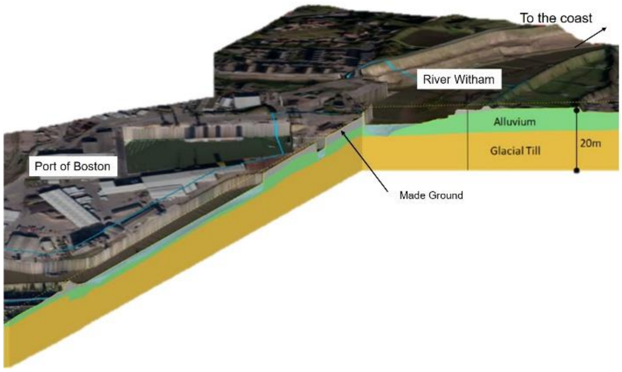

Figure 2. Annotated Leapfrog Works model

2 RISING SECTOR GATE

2.1 *Details*

The barrier is a 26m wide, 362 tonne rising sector gate housed in $6000m^3$ reinforced concrete structure, 35m x 33m in plan, 5m thick and 12m high walls, designed to stand 10.5m hydraulic head that will prevent tidal flooding of Boston during extreme tidal surge events. The design surge level was +7.55mAOD, which has an allowance for climate change induced sea level rise over the 100-year design life - see Pollard et al (2021).

The structure also needed to be constructed within the river, and therefore within a sheet pile cofferdam, which would be retained to form part of the permanent works.

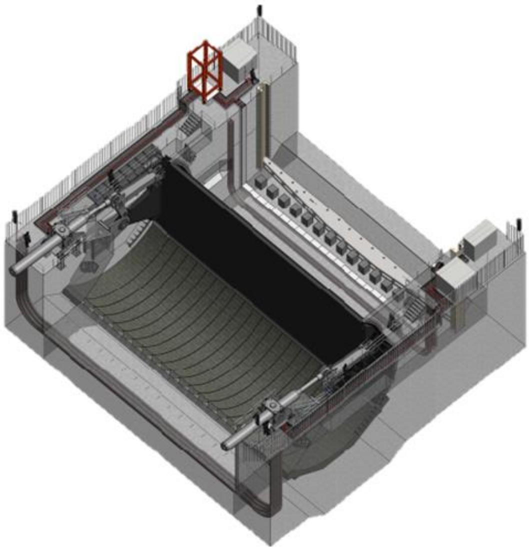

Figure 3. Detail of the rising sector gate structure

A schematic of the gate structure is shown in Figure 3 above.

2.2 *Foundation – soil structure interaction*

The barrier is founded 4m below existing (dredged) riverbed level, within the Glacial Till (Figure 4). The Glacial Till is a heavily overconsolidated clay with some flint gravels; it is very stiff, of low permeability, and has a high undrained shear strength, increasing with depth.

Figure 4. Formation

Stiffness degradation curves of secant shear modulus, G_{sec}, versus strain were derived considering pressuremeter and seismic CPT tests undertaken in the Glacial Till. A best estimate curve was then chosen for design (Figure 5).

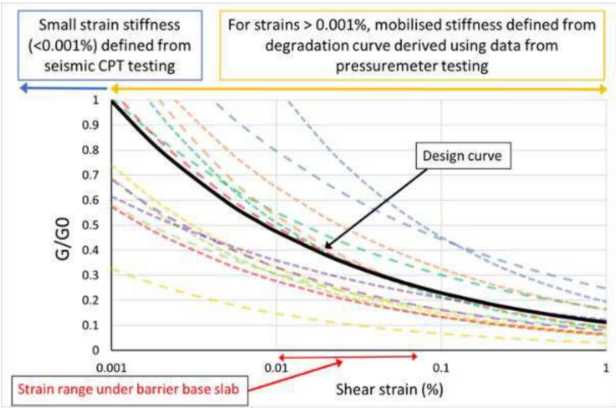

Figure 5. Stiffness degradation for Glacial Till

Design of the barrier structure was undertaken using a 3D FEA structural model using LUSAS, in conjunction with a geotechnical model of the formation using Oasys PDisp.

Output from the structural model was input to the geotechnical model and vice versa in an iterative process, until convergence between the two models was reached. The maximum settlement predicted for the structure is 15mm, with a maximum differential between the walls and central slab of 10mm. The non-linear ground stiffness model helped to reduce structural edge effects resulting from greater loading under the walls than under the central portion of the slab, thus facilitating a more efficient slab design.

The models took in to account the contribution of the structure's perimeter sheet piles in resisting uplift buoyant forces. This allowed a significant reduction of concrete needed to resist the uplift pressures. The model results were checked using a series of 2D PLAXIS models at typical sections through the barrier structure.

3 CONTROL BUILDING

A control building with critical equipment and tight settlement tolerances was required to operate the proposed flood barrier. According to the initial concept design, the structure was going to be supported by 70 No piles. However, following value engineering, shallow foundations sitting on the granular made ground were used instead. An instrumented large-scale test (Figure 6) was undertaken to confirm the settlement prediction for a flexible raft foundation. As shown on Figure 7, based on the results of the test, the predicted settlement was reduced from 20mm to 6mm, with no significant long-term settlement.

Figure 6. Large scale load test – completed height monitored for 10 days

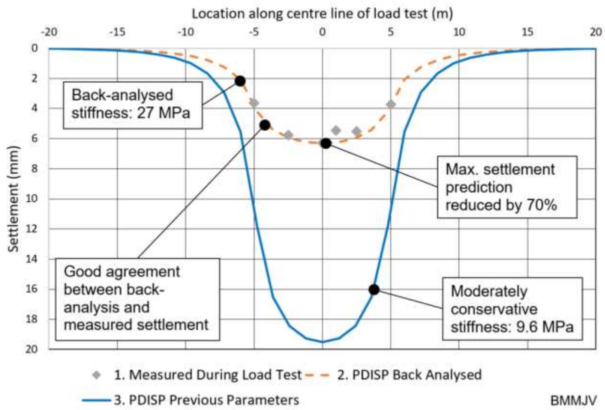

Figure 7. Measured and back-analysed settlement

4 BYPASS CHANNEL

During construction of the VSG, the river flow was diverted temporarily through a bypass channel which was then backfilled.

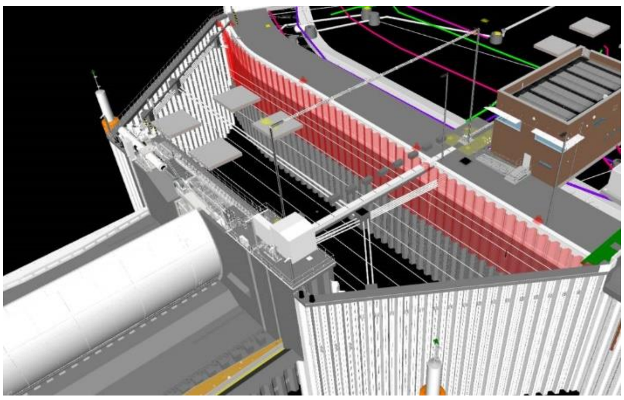

Figure 8. Bypass channel

A twin sheet pile wall cofferdam (see Figure 8) approximately 60m long and 25m wide at its centreline, with a trapezoidal shape was constructed and infilled with granular fill. AZ46-700N S430 steel sheet piles approximately 20m long were driven into Glacial Till to -14.5mAOD. Dywidag 75mm diameter ties were placed at approximately 2m below the top of the sheet piles at +4mAOD, spaced 2.8m and connected to a waling beam. The design was carried out using the computer program Wallap and its results verified using Flac 3D. Wall deflection monitoring indicated a maximum deflection in the order of 95mm during infilling, well below the predicted deflection of 130mm

5 LEFT BANK QUAY WORK

The Left Bank works included a sheet pile quay wall in front of the existing concrete quay wall, three load relieving platforms (LRPs) for loading and unloading cargo vessels by the Port of Boston, and a new 2m high flood protection wall. All new sheet piles were AZ42-700N S430 steel sheet piles 19m long driven to –13.5 mAOD into Glacial Till.

The choice of lateral support for the new quay wall was as follows:

- 1400mm discrete anchor piles at 4200mm spacing;
- Anchoring to the LRPs;
- Ground anchors at the existing LRP

The design ignores any contribution of the existing quay wall and assumes an accidental loss of a tie or anchor for all three anchoring support systems. Details are discussed below.

5.1 *Sheet piles anchored to 1400mm diameter pile*

The choice of discrete steel piles 1400mm diameter, 19mm thick spaced at 4.2m centres (3 times its diameter) was dictated by the presence of numerous buried structures associated with the existing quay wall anchoring system and numerous services. Isolated steel tubular piles instead of a continuous anchor sheet pile minimise the risk of hitting these obstructions. There is a degree of flexibility in the plan location of the anchor tube overcoming localised pinch points around existing buildings and known buried obstructions. The tubes were driven into Glacial Till to – 7.5m AOD.

Flexibility on plan location can also be exploited if unknown buried obstructions are encountered during the piling works. If the tube pile cannot be relocated, it is easier to pre-drill a tube pile location than a continuous anchor wall. The proposed design is illustrated on Figure 9 below.

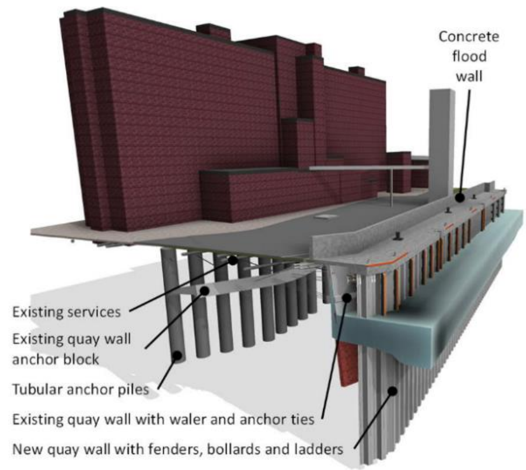

Figure 9. Discrete 1400mm steel tubular anchor piles

The AZ42-700N geotechnical design was undertaken using Plaxis 2D with the results checked using Wallap. The final design comprised 1420mm diameter piles 13m long and 13m behind the new sheet pile wall, with 75mm diameter connecting ties. Figure 10 shows part of the sheet wall during construction and Figure 11 the driving of a 1400mm diameter steel tube.

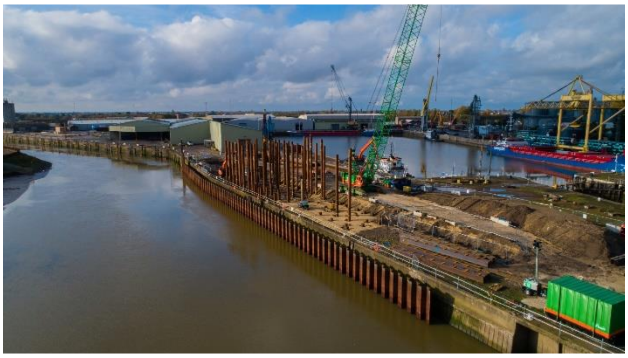

Figure 10. Left bank during construction

Figure 11. Driving 1400mm diameter piles with hydraulic impact hammer

Deflection monitoring during construction indicated deflections generally less than 30mm, well within the predicted values from Plaxis. A long-term deflection in the order of 100mm is predicted after corrosion and drained conditions.

5.2 *Drilled anchors*

In the central section of the left bank quay wall there is an existing crane pad to service the existing riverside berth. In addition, this is the location of the buried slipway from earlier in the history of the port which resulted in a piled anchor trestle being installed to support the current quay wall. The details based upon the available information are shown in Figure 12. For this reason, permanent Single Bore Multiple Anchors (SBMA) were used. For the anchor loads, the loads indicated by Wallap were checked using Plaxis 2D.

The anchors were installed using 140mm diameter rotary percussive flushed drilling techniques with full depth temporary casing, at every in-pan of the AZ42-700N sheet pile wall, at declination of 30° from the horizontal at 1.5m from the top of the wall (+4 mAOD). A total of 72 No. anchors, 37m long were installed, with four tendons of fixed length units between 3.5m and 5m long each, to carry design service load of 494 kN. Figure 13 shows an anchor installation.

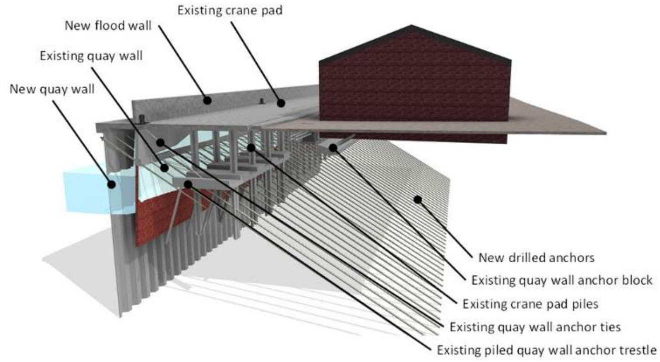

Figure 12. Ground anchors

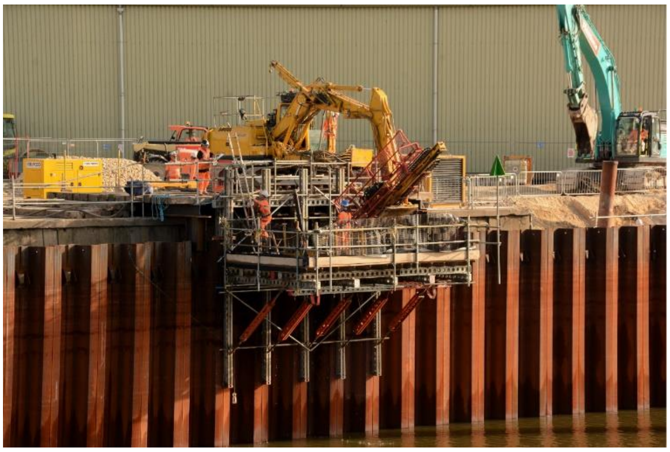

Figure 13. Ground anchor installation

All anchors were subject to either on-site acceptance tests (69 No), on-site suitability tests (4 No) to a proof load of 741 kN and a lock off load of 346 kN (70% of the service load) to avoid overstressing of the anchors in the long term following movements due to the existing wall no longer providing support, corrosion of the sheet piles and pore water pressure dissipation, when the drained soil mechanical properties apply.

5.3 *Load Relieving Platforms (LRP)*

The LRPs are required to support cranes operated by the Port. Foundations for the LRPs are provided by four rows of 508mm diameter, 16mm thick steel tubular piles 20m long driven to -14.5m AOD, with an available unfactored resistance of the order of 950 kN. Piles were driven with a hydraulic impact hammer. 1% of all piles were load tested. It was found that all 508mm diameter piles plugged during driving. Figure 10 shows an aerial view of LRP3 prior to full driving of the tubular piles and the left bank sheet piles in progress.

Lateral support to the new quay was provided by the three new LRPs connected to the first row of piles every 4.2m to the sheet pile wall (see Figure 14).

Calculations were undertaken to prove that the global stiffness of the LPRs was greater than that of the discrete 1400 mm diameter tubular pile anchor solution which it replaced. A combination of software packages was used for this: Repute, Piglet and ALP. Load distribution between piles was confirmed using Plaxis 2D.

Figure 14. Tie connection between AZ42-700N sheet piles and 508mm tubular piles

Lateral deflection was initially estimated to be less than that of the tubular pile system which it replaced, approximately 30mm at the end of construction and 70mm in the long term. However, during installation of LRP3 (furthest downstream towards the knuckle), a lateral deflection of the order of 230mm was recorded. Although back analysis using Plaxis 2D suggested that this movement may be caused by the breaking of the ties supporting the existing concrete sheet pile wall, it was concluded that the additional lateral movement was likely caused by cavity expansion of the ground during plugging of the tubular piles during driving. For the remaining LRP1 and 2, the driving sequence was modified so that the back rows of piles were driven first. The result of this was that the total deflection was less than 150mm. A capping beam has been constructed which allows up to 275mm lateral deflection of the sheet piles.

6 NEW KNUCKLE

The proposed works requires the widening of the existing entrance to the Wet Dock (see Figure 1), constructed in 1884, from 15m to 18m wide, and the replacement of the two lock gates with a single central vertical sector gate (VSG). As part of this work, it will be necessary to continue the AZ42-700N sheet pile wall around part of an existing knuckle (see Figure 15) cutting though the body of the structure to allow the widening of the wet dock entrance south of the existing masonry wall.

Figure 15. Southern elevation of the existing knuckle

At the time of writing this paper this part of the works has not been started and will be subject to a future paper.

Current design work has been undertaken using Plaxis 3D for the geotechnical and structural design of the replacement knuckle (see Figures 16 and 17).

The sheet piles will be AZ42-700N, S390 steel, 19m long, driven to – 13.5 mAOD. Eight radial tie rods will be provided at +4 mAOD, radiating from an octagon placed at the centre of the knuckle, using 75mm diameter Dywidag threaded bars S670/800. A cross beam will connect the left bank wall to the south wall of the WDE with corner braces as required.

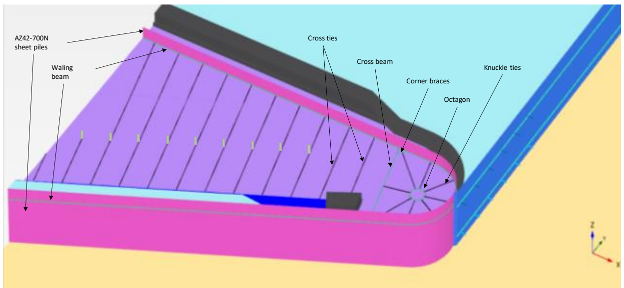

Figure 16. Geometry of the knuckle modelled in Plaxis 3D

7 WIDENED NEW WET DOCK ENTRANCE (WDE)

7.1 *Vertical Sector Gate (VSG)*

A replacement vertical gate will be constructed centrally within the widened wet dock entrance (WDE). A 3D model of the structure is shown in Figure 17 below. The soil-structural interaction analysis was carried out in a similar way to the main Barrier, using LUSAS and PDISP. The contribution of the perimeter sheet pile to resist downwards from the structure or upwards loads from water uplift was not included in this case.

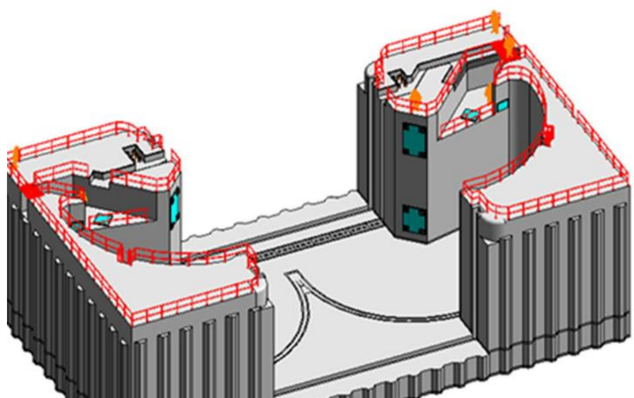

Figure 17. Model of Wet Dock structure

7.2 *Discrete anchored walls*

The widening of the WDE will require the replacement of the existing quay walls. The existing structures consists of two gravity quay walls retaining about 9.75m of made ground and alluvial soil, connected at the base by an inverted arch shaped concrete and masonry base.

For the majority of their length, the new quay walls will be AZ42-700N sheet piles 19.3m long with 9.55m of embedment. At +4mAOD (1.8m bgl), tied onto anchor piles consisting of steel tubes 1400mm diameter, 19mm thick at 4.2m spacing c/c installed 15m behind the sheet pile walls and driven into glacial till to -7.5mAOD (see Figure 17).

The new quay wall to the north will be installed directly in front of the existing gravity structure, while to the south the new wall will be 1.4m distant from the back of the existing structure.

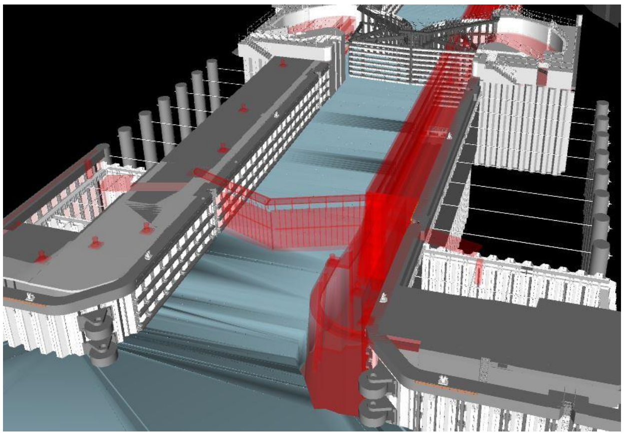

Figure 17. View of the upstream half of the WDE with existing (red) and proposed structures

The geotechnical design of the retaining walls was carried out using the software Wallap. However, downstream of the gate structure, the anchor piles connected to the WDE walls are also connected to the new quay walls of the left bank, with the ties coming from the two quay walls installed at two different levels. Plaxis 3D was used to assess the interaction between the two walls and at this location, with the results validated with Plaxis 2D. This assessment did also provide validation for the Wallap analyses.

7.3 *Cellular structures*

Due to space restrictions and difficulties in fitting the anchor piles required to stabilise the quay walls, at the western end of the WDE two cellular cofferdams will be constructed instead, one at each corner between the wet dock and the WDE. These will be constructed with AZ42-700N sheet piles 19.3m long, the same as the majority of the quay walls, with cross-ties connecting the opposite faces of the cofferdams to provide stiffness to the structure and allow it to behave as a box.

The cofferdams were verified against global stability failure mechanisms (overturning, sliding and bearing) and internal stability failure (vertical shear, horizontal shear, sheeting pull-out, sheeting penetration, sleeping between sheeting and fill) manually and using Plaxis 3D.

7.4 *Long term floor drainage*

The base of the WDE is subjected to uplift water pressure, driven by the shallow groundwater table. The base of the existing structure is an inverted arch in unreinforced concrete, connected to the massive gravity quay walls; therefore, the uplift force is transferred to the walls and the weight of the entire structure contributes to it.

The new WDE will have a flat slab with finished level at -3.95mAOD. The construction of a system capable of resisting the uplift pressures by self-weight would have required the complete removal of the existing slab and deepening the excavation to about -6mAOD, an extremely onerous conditions for the design of the quay walls.

The proposed design against uplift pressure consists of a permanent passive pressure relief system. During the construction of the walls, the uplift pressures will be dealt with by temporarily infilling the existing WDE. Following installation of the new quay walls and their connection to the anchor piles, the infill will be removed and the top 650mm of the existing base slab will be broken down. An array of pressure relief wells, 150mm diameter, 7m deep and 5m spacing c/c will be installed form the top of the remaining portion of the existing slab. A 150mm deep drainage blanket will be laid on the existing slab and the new 500mm thick RC base slab will be cast on top of it. Riser pipes connected to the drainage blanket will extend 1.2m above the top of the new slab, to prevent the risk of silt transported from the river clogging up the pressure relief system. Flap valves will be installed on top of the raiser pipes to prevent water from the WDE entering into the drainage system during high water events.

The design of the drainage system was carried out with SEEP/W seepage analyses, assessing the water pressure underneath the base slab (Figure 18). The redundancy of the system was tested analysing a series of scenarios including failure of up to 50% of the risers and reduction of permeability of the drainage system during its design life.

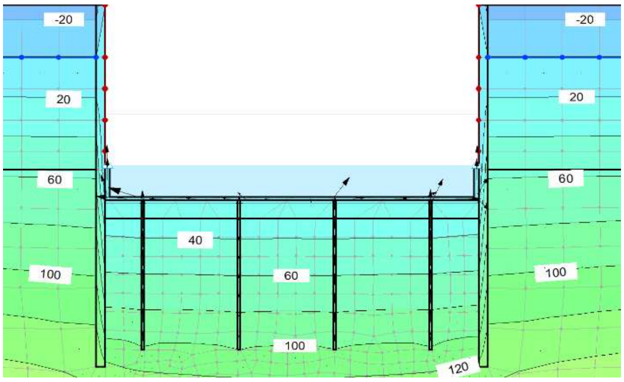

Figure 18. Water pressure distribution following installation of the pore pressure relief system

The shear and bending moment in the base slab caused by the differential heave pressures which will be develop beneath it, due to the different reduction of effective stress across the footprint of the excavation, has been estimated with Plaxis 3D FE analyses.

8 CONCLUSIONS

The design and construction of the Boston Barrier project has been accomplished successfully by a multidisciplinary team via the 3D BIM model of the structures. This enabled the team to visualise the task in hand and permitted the integration of temporary and permanent works. The availability of high-quality ground investigation allowed the use of appropriate non-linear parameters for the design of complex structures with many load cases with detailed soil-structure interaction.

9 ACKNOWLEDGMENTS

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10 REFERENCES

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