

GEOTECHNICAL DESIGN AND CONSTRUCTION METHODOLOGY OF A DEEP BASEMENT CUT NEXT TO SENSITIVE BUILDINGS

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ABSTRACT

The Maritime Apartment Building is a proposed 16 level building over 2 basement levels to be constructed on a steep site with neighbouring buildings on 3 sides. The primary challenge of this site is limiting movement at the western boundary, which is within 1 metre of a 5-storey sensitive historic building located at the crest of the slope. The basement structure retains approximately 13-metres and has been designed collaboratively by the structural and geotechnical engineers. Coffey has undertaken geotechnical analysis to assess the design soil loads and soil-pile interactions. To limit deflections the basement soldier piles are very stiff and are supported with a capping beam, diaphragm slab and associated wing wall. However, poor pile drill rig access to the top of the site and the requirement to install the perimeter soldier piles and pile cap before excavation presents constructability issues. By working closely with the main contractor a temporary works methodology was developed that was both constructible and ensured the stability of the site and neighbouring structures. The proposed solution comprises the construction of a geogrid reinforced ramp to the top of the site where a temporary platform founded on piles would be constructed to support the permanent pile drilling rig. A detailed monitoring plan was prepared as a key risk treatment during construction.

1 INTRODUCTION

Coffey was engaged by the developer as the geotechnical engineer of the Maritime Apartment Building Project, which comprises of 16 above ground levels over 2 underground basement levels. Our work involved the temporary support and construction sequencing design which is critical to the feasibility of the project. Coffey then produced a geotechnical monitoring plan. Prior to this Coffey undertook a review and engineering validation of the permanent geotechnical design of the basement structure.

The Maritime Apartment Building is proposed to be built on a narrow site in the Auckland CBD that currently contains a car park and an existing building (to be demolished) that abuts the road frontage. A retaining wall and steeply sloping ground is present to the rear of the site. The proposed apartment building will encompass the full site and will be bounded to the north, south, and west by neighbouring buildings. Figure 1 depicts the existing site layout.

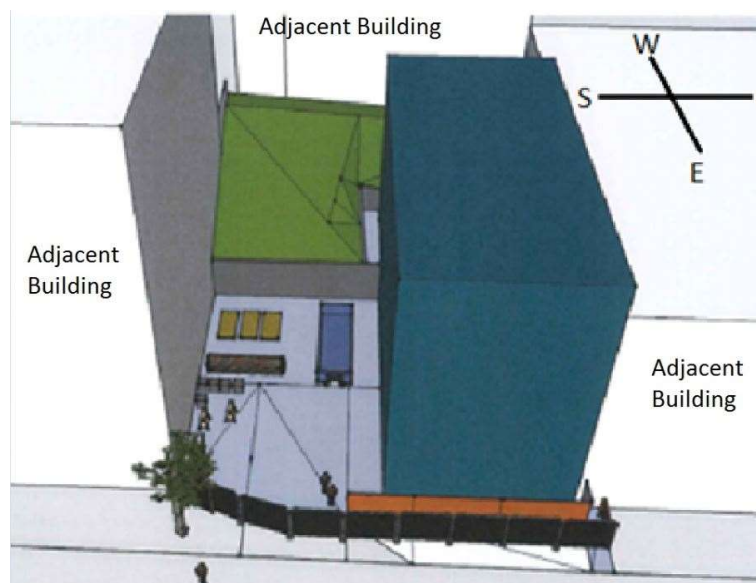


Figure 1: Existing Site Layout

The neighbouring buildings comprise 2 to 5 storey reinforced concrete structures founded on shallow pad footings that bear on soil comprising residual East Coast Bays Formation (ECBF). The significant cuts proposed to form the basement

structure present the potential to cause damaging displacements to the shallow foundations of the neighbouring buildings. Controlling deflection of the basement retaining structure of the Maritime Apartment Building is critical to limiting settlement of the neighbouring buildings, and a very stiff basement retention structure is therefore required. Careful construction sequencing and temporary works design is also necessary to ensure the neighbouring buildings are not damaged. The groundwater table is below the rock interface level and settlement due to groundwater drawdown is not considered an issue. A deflection criterion of 0.25% of the height between points of support has been adopted for SLS and 2.5% for has been adopted for ULS at the western wall in accordance with NZS1170. The above deflection criterion should limit maximum settlements beneath the neighbouring shallow foundations to less than 25mm, which was initially adopted as the maximum allowable settlement, with the structural engineer to assess the effect of this settlement on neighbouring structures.

The western basement wall is the most challenging section having the largest retained height of approximately 13-metres with the footings of the neighbouring building here founding approximately 2-metres below ground level. Using permanent anchors as part of an alternative basement retention system would require permission from the adjoining land owners and therefore cannot be used. In addition to the challenging deflection criteria, the small site, and steep slopes at the rear of the site present constructability issues.

This paper presents the basement retention solution and the method of analysis, as well as, the challenges faced in developing the construction methodology to build the basement structure. Construction of the apartment building has not yet begun.

2 BASEMENT PERMANENT DESIGN AND GEOTECHNICAL VALIDATION ANALYSIS

The ground conditions at the site comprise residual ECBF grading to very weak ECBF rock at approximately 2-metres below ground level at the front (eastern) portion of the site and grading to ECBF rock at approximately 10-metres below ground level at the more elevated rear (western) portion of the site. Figure 2 below presents the geotechnical model.

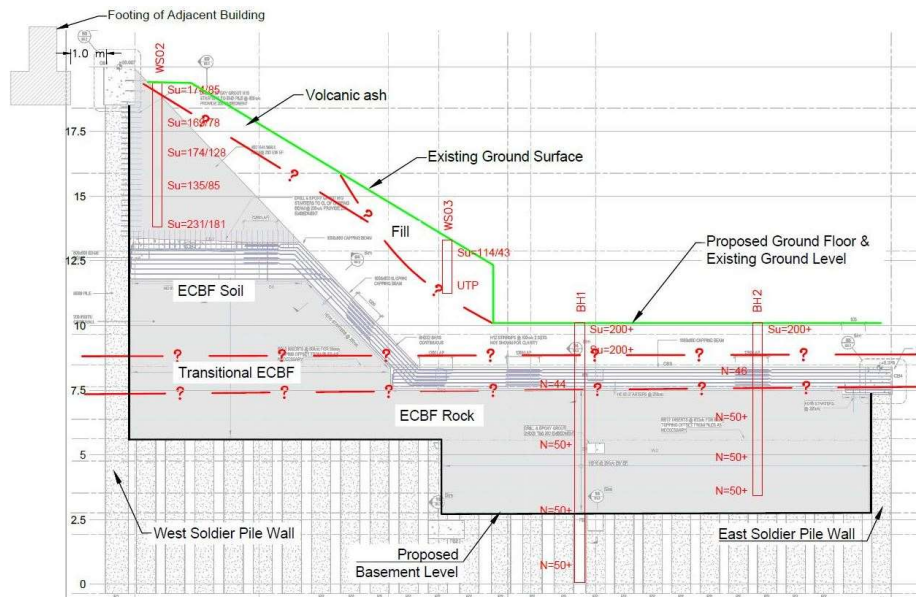


Figure 2: Geotechnical model

The western wall retains stiff to very stiff residual soils with rock exposed at the toe of the wall providing passive resistance. The large retained height, mostly comprised of moderate strength residual soils, induces relatively large forces due to soil pressure under static and seismic loading. Geotechnical parameters were assessed based on insitu tests and our experience in the local area, these are presented below in Table 1.

Table 1: Geotechnical Soil Parameters

Material	Effective Friction Angle Φ' [°]	Effective Cohesion c' [kPa]	Unit Weight γ [kN/m ³]	Drained Modulus E' [MPa]	Poisson's ratio ν
Weathered Ash	28	5	17.5	40	0.3
Residual ECBF	28	5	17.5	40	0.3
Transitional ECBF	32	7	18	60	0.3
ECBF Bedrock	40	50	21	210	0.25

The basement retention structure comprises bored concrete piles and the more elevated western wall includes a diaphragm slab approximately 6-metres below the top of the wall. The less elevated north, south, and eastern walls do not include a diaphragm slab but restraint is provided at the top of the wall from the capping beam. Figure 3 (below left) depicts the basement retaining structure as well as the location of the sections for analysis discussed in further detail below.

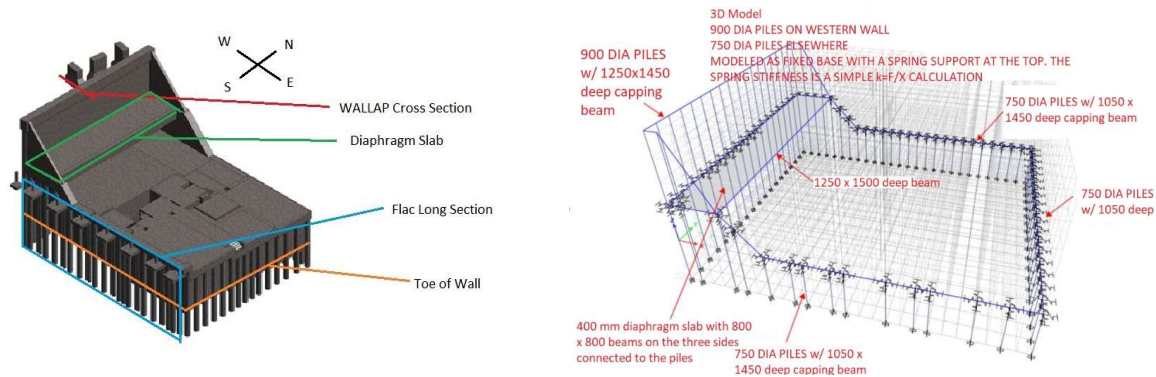


Figure 3: Basement structure (left) and structural analysis model (right)

Figure 3 (above right) is the model used in the structural analysis software. To validate the structural analysis, a 2-dimensional geotechnical soil and pile structural interaction analyses was carried out using WALLAP and FLAC to evaluate the retaining capacity of the western soldier pile wall and the forces on the supporting southern and western walls.

The basement was designed for the following serviceability limit state (SLS) and ultimate limit state (ULS) load combinations for gravity (G) and live load (Q) and seismic loading (E):

SLS: $G + 0.3Q + E_s$
 $G + Q$

ULS: $1.2G + 1.5Q$
 $G + 0.3Q + E_u$

The limiting design criteria for the wall is deflection. At the western wall where retained heights are the highest, the strict deflection criteria necessitates that the soldier pile wall is laterally restrained by the capping beam and the diaphragm slab.

Lateral forces from the top of the western piles are transferred to the capping beam, then to the wing walls, and then to the capping beam at the pile tops along the north, south and east of the site. Similarly, at diaphragm slab level lateral forces from the west wall are transferred to the other basement walls. Lateral restraint to the western pile wall is therefore ultimately provided by the lateral capacity of the piles along the north, south, and east of the site.

The retaining wall software WALLAP was used to calculate deflections of the soldier pile wall. In WALLAP, the lateral restraints of the props (i.e. the capping beam and diaphragm slab) are modelled as springs. Provided the capping beam

and diaphragm slab are of sufficient stiffness, the restraint stiffness is the sum of the stiffness contribution from the north, south, and east pile walls. Where piles are acting in plane with the direction of force (i.e. north and south walls, refer to Figure 4, below right) the piles can be thought of as a beam fixed at one end and with a moment restraint at the other end as shown below in Figure 4, below left.

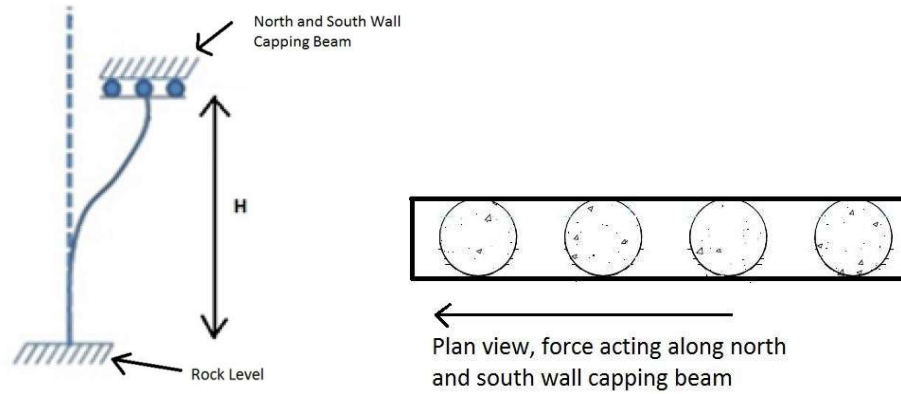


Figure 4. Deflection of the pile (left) and force acting along capping beam (right)

The stiffness of such a beam is given by equation 1 below where H is assumed to be the depth above the rock socket and E and I are the Young’s Modulus and second moment of area, respectively.

$$k = \frac{12EI}{H^3} \tag{1}$$

The stiffness contribution from the piles along the eastern basement wall, where the piles are perpendicular to the direction of loading and the soil provides passive resistance, have been assessed in the previous design phase using a software program such as All Pile based on the gradient derived when lateral load is plotted against the deflection at the top of pile. The stiffness provided by the eastern basement wall was assessed to be approximately 40% of the total pile group stiffness to the western wall.

The obtained prop stiffness was input into WALLAP analysis. The bending moments, shear forces, and deflections for the western wall were calculated using WALLAP. Figure 5 depicts the model in WALLAP to the left and to the right depicts the WALLAP displacement output for the SLS Q + G case.

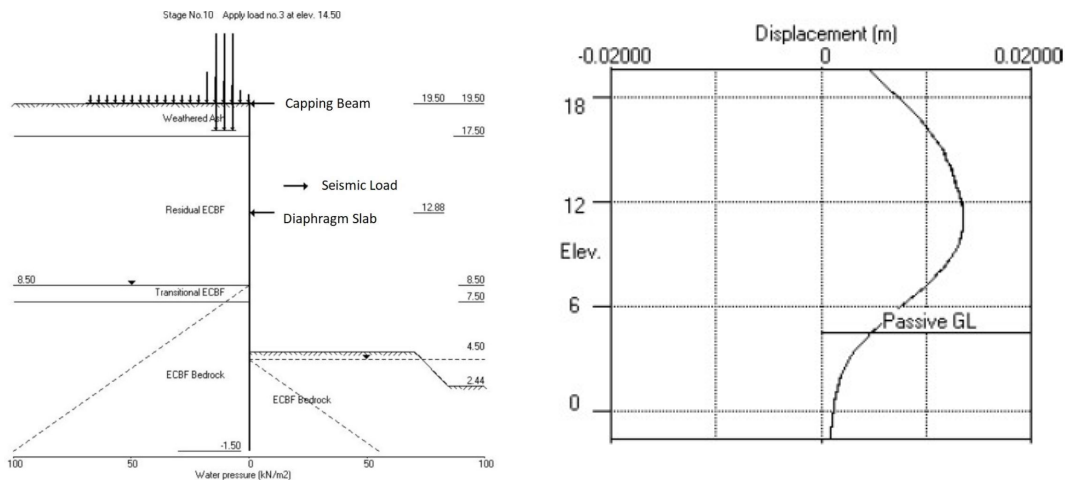


Figure 5: WALLAP model (left), WALLAP deflection output (right)

The resultant loads acting on the capping beam and diaphragm slab are also computed by the software WALLAP and were provided to the structural engineer to design these aspects of the structure, they were also applied to the other basements walls as discussed below.

While the forces acting on the western wall are solely due to earth pressures, the force acting on the north, south, and east pile walls are a combination of the earth pressure from the retained soils (including any surcharge and building loading transferred through the retained earth), and the longitudinal forces acting on the capping beam that result from the restraint forces (due to the diaphragm slab and capping beam) applied at the western wall.

Initially the north and south walls were analysed and designed using WALLAP, which was used to model a cross section perpendicular to the wall. However, significant forces act along the capping beam, longitudinally to the north and south walls. A basement soil (rock) and pile structural interaction analysis on the north and south walls in the longitudinal direction was undertaken using the geotechnical finite difference software FLAC. FLAC was used to more efficiently model and identify the design actions on the designed pile group as it supports the 13-metre high western wall and the adjacent building at the crest of the slope. It was found that bending moments in the longitudinal direction could be up to 5 times higher than in the perpendicular direction, making it clear that when piles are relied upon to provide restraint to another section of wall the resulting forces are critical in the design of that wall. Figure 6 below depicts the longitudinal bending moment along the southern wall. The action on the pile group was provided to the structural engineer.

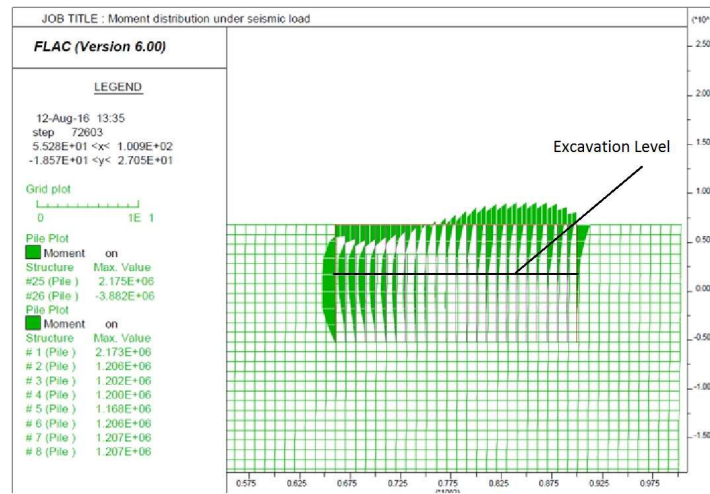


Figure 6: Bending moments in the longitudinal direction along the southern wall

Following the completion of the WALLAP, analysis settlements for the building to the west of the site were calculated using the volume lost method recommended by JE Bowles (1997). Volume lost per metre length can be assessed from the WALLAP displacement outputs. Settlements at the wall crest can be calculated using equation 2 below.

$$s_w = \frac{2V_s}{D} \quad (2)$$

Settlements are assumed to be at a maximum at the wall crest and decrease in a parabolic function as distance from the crest of the wall increases. s_w is the surface settlement at the edge of the wall excavation and is a function of V_s , which is the volume of soil in the WALLAP displacement zone; and D , the distance from the excavation over which ground loss occurs. This distance is a function of the wall height to dredge line, the excavation width and the internal angle of friction.

$$s_i = s_w \left(\frac{x}{D} \right)^2 \quad (3)$$

The ground loss settlement at a distance, x , from the wall crest can then be calculated as per equation 3, above.

The above formulas attempt to account for ground loss due to soil heave by considering the width of the excavation when computing 'D'. In this case the basement level will expose weak rock and heave may be negligible. An alternative settlement assessment due to wall displacement could be considered based on Figure 2.16 of CIRIA C580, which suggests that settlements at a distance 'D' from a propped wall can be taken as half of the wall deflection at depth 'D'. Figure 7 below illustrates the process. This approach yields a maximum settlement of 7mm, compared to 9mm based on the method recommended by JE Bowles.

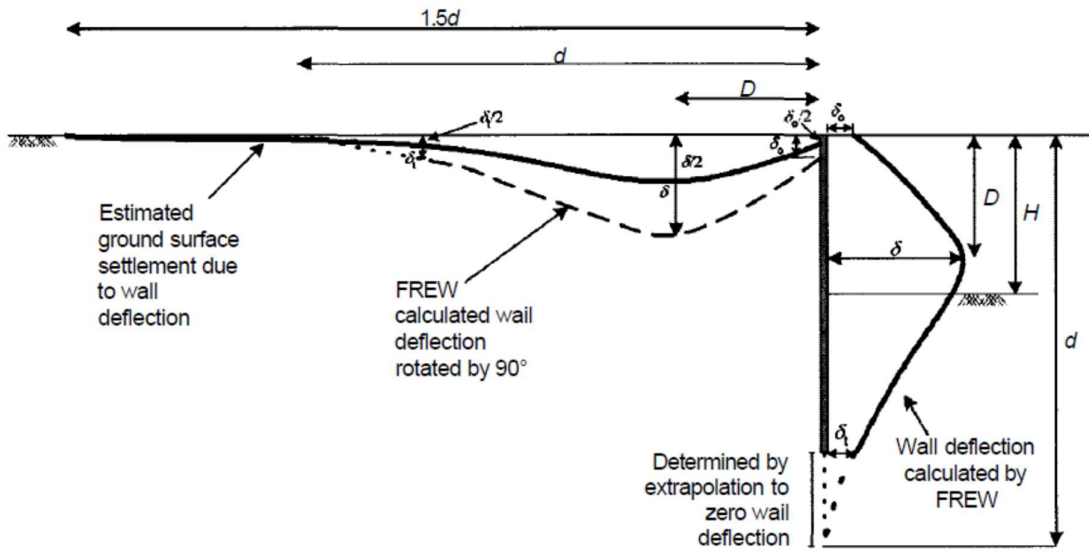


Figure 7: After CIRIA C580 for Estimation of Settlement due to Wall Deflection

By following the above procedure, settlements at the neighbouring property, as well as, deflection and bending moments were calculated at the western wall. Table 2 below presents a summary of these values. The outputs satisfy the design criteria and the structural capacity was checked by the structural engineer.

Table 2: WALLAP outputs at the Western Wall

Section	Maximum SLS deflection (mm)	Maximum SLS Settlement (mm)	Maximum ULS Bending Moment (kNm/m)	Maximum ULS Shear Force (kN/m)
Western Wall	14	9	474	502

3 CONSTRUCTION METHODOLOGY AND SEQUENCING

Once the design was completed, constructability became the primary and critical issues. Coffey closely collaborated with the contractor and had to consider the temporary stability, the permanent design, construction programme, available plant and materials and, economic feasibility. The following methodologies were agreed with the contractor to balance the competing requirements of the project.

3.1 CONSTRUCTION SEQUENCE

For the lateral restraints to act during the excavation, the basement is proposed to be constructed in a top down manner with the soldier piles installed, the capping beam poured, the wing walls installed, and then the excavation to diaphragm slab level undertaken, the diaphragm slab would then be poured and the bulk excavation completed.

The works are proposed to start with the partial demolition of the existing building. The rear of the building is retaining the adjacent slope and will be left in place. A temporary ramp is then proposed to be constructed to the top of the site to allow a drill rig to install the first piles along the western wall and part of the northern wall.

The temporary ramp comprises two back to back mass stabilised earth walls utilising wrap around sandbag facings and compacted GAP65 hardfill. Compared to an unsupported ramp, the advantage of an MSE wall is that an unsupported ramp would take up too much space and may spill over too closely to the neighbouring building, and induce settlements or earth pressures. Wrap around sandbag facing is easy and quick to construct. The option of using crushed concrete from the demolished existing building in place of the GAP65 hardfill was also explored, however, the expense and time to achieve a suitable product made this unpractical.

The piles along the south of the western wall and the southern wall cannot be accessed from the platform at the top of the ramp. Several options were considered to facilitate access to this area. These included, cutting a platform which would destabilise the slope prior to any piles being installed; a platform in part cut and part fill that would over steepen the slope and would be unsafe to sit a drill rig on; and the ramp could not be extended to the southern portion of the site as this area was needed for crane access and as a hard stand. In the end it was decided to construct a temporary suspended steel platform to the south of the ramp. The temporary platform will be supported on 6 bored and concreted piles 750 mm in diameter. These bored piles are arranged in three rows of two bored piles, spaced at approximately 4 metres centre to centre and are socketed into the rock. The first four piles for the temporary platform would be installed from the ramp and the deck would then be constructed. The remaining two piles could then be installed from the platform. Figure 8 below depicts the temporary ramp and platform.

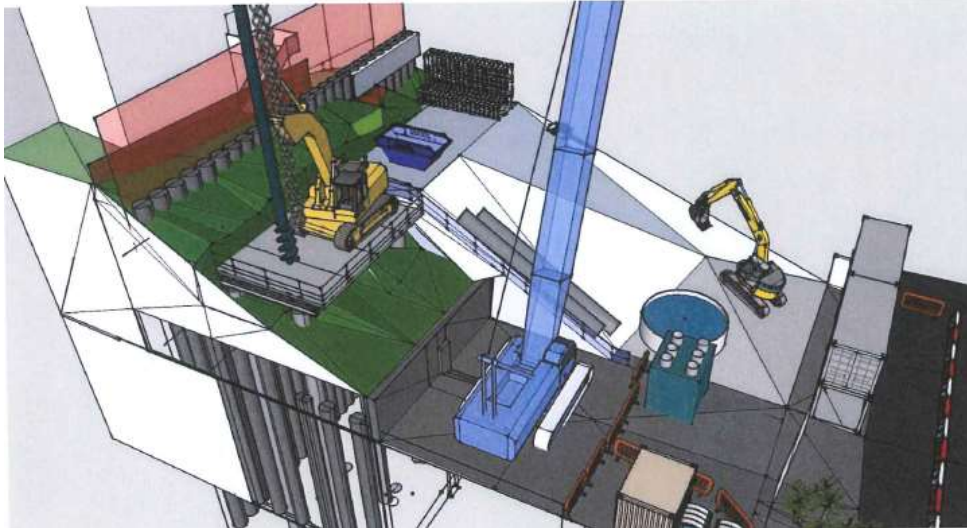


Figure 8: Construction of ramp and platform

Following construction of the temporary platform the piles along the southern portion of the western retaining wall and along the western portion of the southern retaining wall can be installed and the capping beam along the western wall poured. Piling can then commence around the perimeter of the lower lying portion of the site. Following the completion of piling in the elevated portion of site the ramp can be demolished.

Following completion of the piling work a small excavation at the northern and southern portion of the slope can be undertaken so the capping beam and wing walls can be constructed and connected with the capping beam around the rest of the site, as depicted in Figure 9 below.



Figure 9: Construction of perimeter pile wall and capping beam

Once the capping beams and wing walls have cured the excavation to diaphragm slab level can commence and the diaphragm slab can then be constructed. Following curing of the diaphragm slab the remaining dig to finished level can be undertaken. Excavations are undertaken in 3 metre sections to allow the trailing shotcrete crew to work. Once at basement level the building construction begins in a bottom up fashion. Figure 10 below depicts the excavation prior to pouring the diaphragm slab.



Figure 10: Excavation to Diaphragm Slab level

3.2 TEMPORARY MONITORING PLAN

Coffey developed a monitoring plan to record the lateral and vertical displacements on all three adjacent buildings, the existing retaining wall, the proposed MSE wall and the proposed piling bridge. Vibration monitoring points have also been specified adjacent to the existing buildings. The monitoring plan for the permanent structure and groundwater levels has been completed by another consultant. The survey monitoring points are shown on Figure 11 below. Alert and alarm levels have been specified based on the design deflection/settlement levels. If the alert levels are exceeded construction will stop and contingency measures include stabilising slopes with berms or removing the ramp if settlements are excessive.

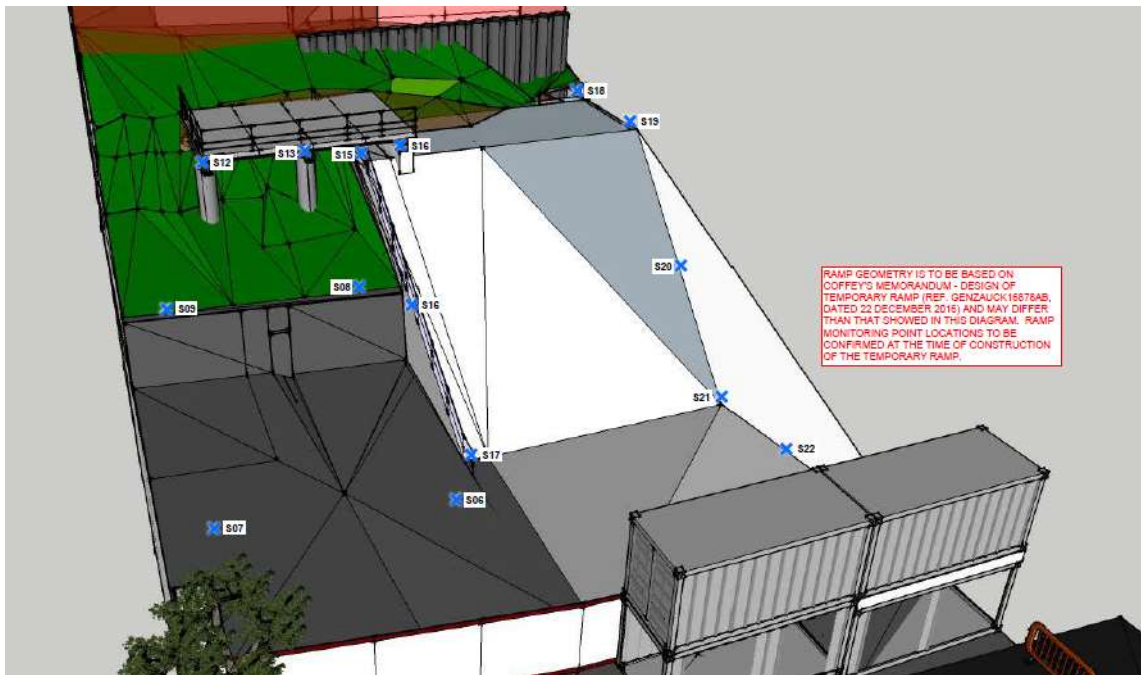


Figure 11: Monitoring point locations

3.3 DESIGN OF TEMPORARY RAMP

The retaining wall design software MSEW has been used to assess design actions and size the geogrid length and required tensile strength of the geogrid. Because the walls are back to back an active soil wedge behind either of the walls cannot develop and earth pressures acting on either wall are therefore relatively small.

The main concern with building the ramp is that settlements due to the weight of the ramp would affect the neighbouring building. We have undertaken deformation analysis using the software Plaxis 2D to assess the impact that the 6.5-metre-high ramp would have on the foundations of the adjacent northern building. This is shown on Figure 12. It was found that as long as the ramp is situated at least 2-metres from the existing neighbouring building then settlements due to the ramp would be negligible.

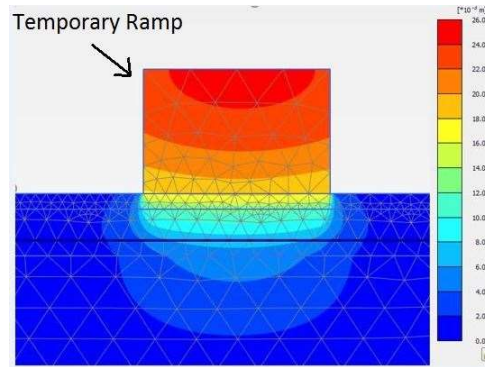


Figure 12: Plaxis Deformation Analysis

3.4 DESIGN OF TEMPORARY PLATFORM

The temporary platform is supported on 6 bored and concreted piles supporting steel beams and a concrete deck. The Coffey in house pile modelling software Combined Load Analysis of Piles (CLAP) has been used to analyse the proposed temporary platform. CLAP allows the analysis of axially and laterally loaded pile groups in two horizontal directions. In this case the pile group must resist axial loading due to the piling rig and horizontal loading due to seismic action.

Maximum bending moments as well as displacements and embedment depths have been assessed using this program. A check of our CLAP results was also undertaken using the program WALLAP. Typical output from CLAP can be imported into excel and are reproduced below in Figure 13.

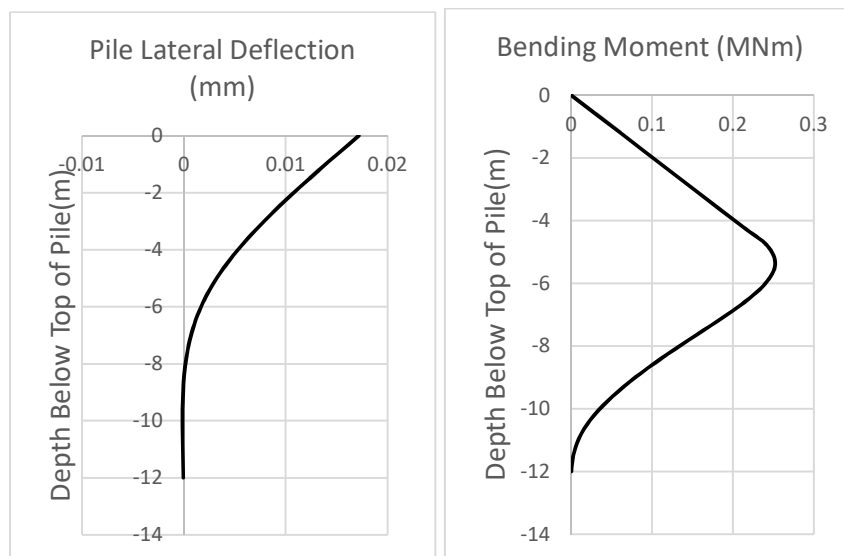


Figure 13: Clap deflection (left), bending moment (right)

4 CONCLUSION

A very stiff basement retention structure was designed without the utilisation of anchors or tiebacks to limit settlements at the neighbouring buildings on a steep site with limited working area. The design utilises the lateral force transfer from the more elevated western wall to the less elevated soldier pile walls to the east using capping beams and a diaphragm

slab. To ensure deflections were controlled careful construction sequencing was developed in consultation with the main contractor, temporary works including a ramp and suspended platform were then designed to facilitate the construction. A monitoring plan was developed to ensure ground movement does not exceed anticipated levels. By undertaking careful design of the basement retention structure strict deflection criteria has been met and by working closely with the main contractor a construction methodology has been developed that is expected to allow the basement retention structure to be constructed on a very steep site with limited room.

References

- Australian/New Zealand Standard (2012). Structural design actions Part 0: General Principles 1170.0.
- Gaba, A. R, Simpson, B. Powrie, W. Beadman, D, R (2003). Embedded retaining walls – guidance for economic design, CIRIA C580.
- Geosolve Limited (2017) WALLAP – Propped, Anchored and Cantilever Retaining Wall Analysis Program, Ver 6.05.
- Itasca Consulting Group, Inc. (2016) FLAC — Fast Lagrangian Analysis of Continua, Ver. 8.0. Minneapolis: Itasca
- Joseph, E. B. (1997). Foundation Analysis and Design, fifth edition. McGraw-Hill Book Co – Singapore.
- Plaxis v2017, Delft, The Netherlands