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01 May 2026

Downtown Carpark Developments - Te Pūmanawa o Tāmaki

2 Lower Hobson Street, Auckland CBD

FAST TRACK COMMENTS

Millenium & Copthorne Hotels New Zealand Limited

Job No. AKL2025-0064 AG | Version 1



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Document version information	
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Review and Update History

Revision	Date	Comments
A	21 April 2026	Initial draft for internal review
B	22 April 2026	Draft for internal review
0	23 April 2026	Issue for client's comments
1A	30 April 2026	Draft for internal review
1	01 May 2026	Issue for Panel

EXECUTIVE SUMMARY

CMW Geosciences has undertaken an independent geotechnical review of the proposed Downtown Carpark Development, with a particular focus on potential effects on the adjacent M Social Hotel. The project involves a deep excavation for a 4-level basement in complex reclaimed ground conditions, immediately adjacent to sensitive structures and the property boundary of the MSocial Hotel.

While the applicant's investigations and addendum are sufficient for an initial high-level assessment, they do not adequately constrain and mitigate several critical geotechnical risks. If not robustly addressed, these unresolved risks have the potential to adversely influence ground movements, alter groundwater behaviour and compromise the M Social Hotel structures. Presented below are the main points which need to be addressed further:

- Ground model uncertainty: Limited site-specific testing and reliance on non-verifiable datasets introduce uncertainty in key parameters governing wall behaviour and ground movements. Additional site-specific investigation and parameter validation is required to support the current assessment and shall be conducted and refined at the detailed design stage..
- Retaining wall performance: Independent analysis indicates potentially greater lateral displacements than predicted by the applicant. Wall behaviour is highly sensitive to modelling assumptions, mesh discretisation and construction sequencing, requiring refined modelling, confirmed construction sequence and sensitivity analyses are required at detailed design stage to confirm the expected displacements and adjust monitoring and alert levels if necessary.
- Groundwater effects: The groundwater regime is complex and tidally influenced. The diaphragm wall may act as a barrier to groundwater flow, potentially inducing mounding and changes in pore pressures that could affect adjacent structures. A calibrated, site-specific groundwater model is required.
- Construction methodology risks: The presence of unmapped buried obstructions and the sensitivity of the system to construction sequencing introduce uncertainty in construction performance. Detailed methodology must be developed, confirmed and provided at detailed design stage and independently reviewed by Council and affected stakeholders' geotechnical counsel.
- Monitoring and contingency framework: Proposed trigger levels and monitoring approach are not sufficiently detailed. Continuous, real-time monitoring with building-specific thresholds and immediate response mechanisms are required.

In summary, the following key geotechnical risks relate specifically to potential effects on the MSocial Hotel and must be addressed through strengthened and enforceable consent conditions, which shall require:

- Completion of additional site investigations and formal validation of geotechnical parameters
- Refinement of numerical modelling, subject to independent peer review and verification by Council and affected stakeholders' geotechnical counsel
- Preparation and approval of a detailed, peer-reviewed construction methodology prior to commencement of works by Council and affected stakeholders' geotechnical counsel
- Implementation of continuous automated monitoring systems with clearly defined trigger levels and mandatory response actions
- Immediate notification procedures and structured stakeholder communication protocols in the event of trigger exceedance

It is recommended that these critical geotechnical risks are managed through binding consent conditions and detailed design requirements at the consent stage, rather than being deferred to the construction stage. This is necessary to ensure that potential adverse effects on the adjacent M Social Hotel structure are clearly defined, appropriately controlled, and effectively mitigated.

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1.0 INTRODUCTION

1.1 Scope of works

CMW Geosciences (CMW) was engaged by Millenium & Copthorne Hotels New Zealand Limited to carry out a geotechnical assessment of a site located at 2 Lower Hobson Street, Auckland CBD, to undertake an independent geotechnical review of the proposed Downtown Carpark redevelopment, with particular focus on potential effects of the proposed redevelopment on the adjacent M Social Hotel.

The scope of work and associated terms and conditions of our engagement were detailed in our services proposal letter referenced AKL2025-0064AA, Rev1 dated 17 April 2025.

1.2 Reviewed documents

This review considers the following supporting documents prepared by Tonkin & Taylor (T+T) for the Fast-track project:

- [1] Tonkin & Taylor, Downtown Car Park Development - Preliminary Detailed Site Investigation (Ref. 1016043.1000 v2 dated November 2025)
- [2] Tonkin & Taylor, Downtown Car Park Development Geotechnical and Groundwater Assessment Report (Ref. 1016043.2000 v3.1 dated November 2025).
- [3] Tonkin & Taylor, Downtown Carpark Site Development. Groundwater and Settlement Monitoring and Contingency Plan. T+T Ref: 1016043.2000 v5.1. dated November 2025.
- [4] Tonkin & Taylor, PC120 Landslide Hazard Risk Assessment (Ref. 1016043.2000 dated December 2025).
- [5] Tonkin & Taylor, Geotechnical Addendum Report (Ref. 1016043.2000 dated April 2026).

This review also considers the preliminary analysis conducted by CMW and presented in the following reports:

- [6] CMW Geotechnical Preliminary Assessment Report, ref. AKL2025-0064 AB Rev 1 dated 13 June 2025,
- [7] CMW Gap Analysis Letter, ref. AKL2025-0064 AE Rev 1 dated 19 February 2026.

2.0 TECHNICAL REVIEW

2.1 Ground Model and Parameter Uncertainty

The ground model remains subject to significant uncertainty due to the highly variable nature of reclamation fill, underlying marine sediments and the erosive surface of the ECBF rock. While the applicant has supplemented the dataset through additional permeability testing [5], key parameters such as stiffness, are still not robustly defined, with only a limited number of site-specific tests (notably three pressuremeter tests) were undertaken [2]. As a result, critical design parameters rely on a limited number of site-specific measurements and values derived from internal and non-public datasets, which are not independently verifiable, as acknowledged by the applicant's geotechnical counsel at the meeting held on 23 March 2026.

Given the sensitivity of retaining wall performance and ground movements to these parameters, reliance on a limited and non-statistically representative dataset introduces material risk. The applicant has indicated that further investigations and sensitivity analyses will be undertaken at detailed design stage, which confirms that the current design is based on preliminary assumptions and that parameter uncertainty remains material.

These uncertainties should be therefore addressed through the enforceable resource consent conditions and subsequent detailed design requirements for targeted on-site investigations and parameter validation at detailed design.

2.2 Diaphragm Wall Behaviour and Displacement Criteria

CMW preliminary assessments [6][7] identify a discrepancy with the retaining wall movements predicted by the applicant [2] indicating potentially larger lateral displacements. This difference reflects the sensitivity of the preliminary design model to input assumptions, including soil parameters, construction staging, and numerical discretisation, as acknowledged by the applicant's geotechnical counsel at the meeting held on 23 March 2026.

This divergence should be understood as an indication of uncertainty and sensitivity in predicted behaviour. In particular, the adopted WALLAP analysis and approach may limit the resolution of peak bending moments, shear forces and wall deflections, henceforth requiring confirmation through sensitivity assessment. Reliance on precedent project performance does not remove the need for site-specific validation, given differences in ground conditions, geometry, and proximity to adjacent structures.

Given the proximity of M Social structures, the uncertainty regarding its foundation system (noting that while some elements may be pile-supported, a proportion of the structure and associated services are understood to be supported on shallow foundations), and the limited embedment of the wall within the ECBF formation, displacement assessment should include parameters sensitivity and be assessed against upper-bound scenarios. Therefore, more robust numerical modelling should be undertaken at the detailed design stage, including demonstrated mesh, parameter and construction sequencing sensitivity analyses.

The construction methodology, particularly excavation sequencing and support installation, should also be developed in sufficient detail at detailed design stage to enable independent peer review as well as potential review or update of monitoring and/or consent conditions, prior to construction.

The contractor's methodology (including temporary works), once developed or refined, should likewise be submitted within a sufficient time frame (minimum 4 weeks prior) for review and acceptance by Council and affected stakeholders before implementation, given its potential to materially influence wall behaviour and associated risk to M Social structure.

Similarly, M Social representatives should be notified of any modification to design or construction methodology. These modifications should be submitted within a sufficient time frame (min 2 weeks) for review and acceptance by the affected stakeholders before implementation, given its potential to materially influence wall behaviour and associated risk to M Social structure.

2.3 Groundwater Effects

The groundwater regime at the site is complex and influenced by tidal interaction with the Waitematā Harbour, resulting in dynamic and spatially variable conditions. This tidal influence within the superficial groundwater system has been acknowledged by the applicant's geotechnical counsel at the meeting held on 23 March 2026. While the applicant's addendum indicates that some assumptions may be conservative and additional permeability tests have been conducted [5], key aspects, particularly large-scale groundwater behaviour, are insufficiently constrained.

For the detailed design stage to support assessment of the diaphragm wall effects, a calibrated and site-specific groundwater model, supported by additional large-scale testing, is required to assess drawdown, mounding, and uplift effects, which may have significant implications for neighbouring structures, including the M Social Hotel's underground elements (services and shallow foundations), and could result in associated ground settlement.

Diaphragm walls are recognised to act as partial barriers to groundwater flow, potentially altering hydraulic conductivity and inducing changes in groundwater levels and pore pressures that could affect adjacent properties[1]. Diaphragm wall should be structurally designed for forces from tidal mounding effects.

Continuous monitoring of both the shallow groundwater system (subject to tidal recharge) and the deeper regional groundwater within the ECBF formation are necessary to adequately characterise the magnitude and temporal variation of groundwater levels across the site.

2.4 Construction Methodology and Obstruction Risk

The site's history of reclamation and prior development introduces a high likelihood of un-mapped buried obstructions, including remnants of historical structures, piles, and seawalls. While these are considered manageable in principle, their presence may influence construction sequencing, wall alignment, and overall performance of the retaining system.

Variability in construction execution, in complex ground conditions, can materially affect ground movements and structural response.

Therefore, this geohazard should be explicitly accounted for in the detailed design and construction methodology. The contractor's methodology, once developed or refined, should be submitted with sufficient time for independent review, to ensure that associated risks are appropriately managed and potential impacts on the M Social structure are minimised.

2.5 Monitoring, Trigger Levels, and Contingency

The proposed monitoring framework is appropriate in principle; however, inconsistencies remain between predicted movements and the proposed alert and alarm thresholds [3]. In some instances, trigger levels approach predicted design movements, limiting their effectiveness as early warning indicators.

To strengthen the risk management framework, trigger levels should be defined relative to conservative or upper-bound predictions and supported by building-specific assessments that establish acceptable movement limits, monitoring requirements, and mitigation measures for each affected structure, including the M Social Hotel.

Monitoring shall be undertaken on a continuous, automated (near real-time) basis for all relevant parameters, including groundwater levels, diaphragm wall performance, ground movements and building deformation, throughout diaphragm wall construction, excavation and dewatering works. Monitoring frequencies shall be stage-dependent and proactively adjusted based on assessed risk, rather than solely in response to exceedances and shall only be reduced once stability has been demonstrated to the satisfaction of a suitably qualified and experienced professional (SQEP) and the Council.

Conservative trigger thresholds and supplementary monitoring measures shall be provided, including alternative monitoring approaches where access to adjacent properties is constrained. All monitoring data, including near real-time outputs, shall be made readily accessible to the Council and all affected external parties, through a shared digital platform or equivalent system, to enable independent review, timely oversight, and informed response to emerging risks.

[1] Hydraulic characterization of diaphragm walls for cut and cover tunnelling (2012) Engineering Geology, Volume 125, 27 January 2012, Pages 1-10

3.0 SUGGESTED CONDITION CHANGES

Table 1: Suggested condition changes

Geohazard Ref.	Consent Ref. / Page	Original Consent Wording (Extract)	Proposed Wording	Comment
2.1 – Ground Model and Parameter Uncertainty	40, 164–165 / p.14, 50–51	"The GSMCP must, as a minimum, include the following information: …"	The GSMCP shall include additional investigation where existing data is insufficient, and full justification of key geotechnical and groundwater parameters used in design.	Limited testing and reliance on non-verifiable datasets introduce uncertainty in design parameters.
2.1 – Ground Model and Parameter Uncertainty	158 / p.47	The design and construction of the basement retaining walls must be undertaken in accordance with the specifications contained in the following report:…	The design and construction of the basement retaining walls must be undertaken in accordance with the specifications contained in the detailed design geotechnical reports. Prior to detailed design certification, the consent holder shall undertake additional site-specific geotechnical investigations where required to adequately characterise ground conditions and validate key design parameters, including but not limited to soil and rock stiffness and strength. The detailed design shall incorporate these updated investigation results and include documented justification of all adopted geotechnical parameters, including the basis of derivation and consideration of parameter variability. Where parameters are derived from external datasets or engineering judgement, this shall be clearly identified and supported by appropriate sensitivity analyses. The final design shall demonstrate that retaining wall performance, including predicted deformations and stability, has been assessed using parameters that are representative of site-specific conditions and consider upper-bound scenarios. All such investigations, parameter derivations, and analyses shall be subject to independent peer review and submitted to Council for	Limited testing and reliance on non-verifiable datasets introduce uncertainty in design parameters.

Geohazard Ref.	Consent Ref. / Page	Original Consent Wording (Extract)	Proposed Wording	Comment
			certification prior to commencement of construction.	
2.1 – Ground Model and Parameter Uncertainty	Advice Notes 16 / p.62	*"16. Additional geotechnical investigations may be required for final design for Building Consent. "*	All requirements currently identified as Advice Notes shall be incorporated into enforceable consent conditions, with "may" replaced by "shall".	Ensures critical requirements addressing uncertainty are enforceable.
2.1 – Ground Model and Parameter Uncertainty	Advice Notes 17 / p.62	*"17. Geotechnical review of the detailed design drawings may be required to confirm their recommendations have been adopted for Building Consent."*	All requirements currently identified as Advice Notes shall be incorporated into enforceable consent conditions, with "may" replaced by "shall".	Ensures critical requirements addressing uncertainty are enforceable.
2.2 – Diaphragm Wall Behaviour and Displacement Criteria	173 / p.57	"Monitoring required…"	A minimum of three (3) inclinometers shall be installed along the diaphragm wall and monitored continuously until movements stabilise to SQEP and Council satisfaction.	Addresses uncertainty in predicted wall deflections and need for continuous verification.
2.2 – Diaphragm Wall Behaviour and Displacement Criteria	162–163 / p.49–50	"Alert and Alarm triggers… contingency actions…"	Alert exceedances shall trigger immediate review and mitigation. Alarm exceedances shall trigger immediate notification, contingency actions, and stop-work.	Ensures early intervention given modelling sensitivity and potential underestimation of movements.
2.2 – Diaphragm Wall Behaviour and Displacement Criteria	90 / p.27	"Construction methodology… based on geotechnical report…"	A detailed construction methodology shall be developed at detailed design stage, including excavation sequencing, temporary works, predicted deformations, and mitigation measures, and submitted for Council certification.	Sequencing and modelling sensitivity directly influence wall behaviour.
2.2 – Diaphragm Wall Behaviour and Displacement Criteria	90 / p.27	Same as above	The construction methodology shall be independently peer reviewed and verified against deformation predictions and monitoring requirements prior to commencement of works.	Provides independent validation of modelling assumptions.
2.2 – Diaphragm Wall Behaviour and Displacement Criteria	90 / p.27	Same as above	The contractor's methodology, once developed or refined, shall be submitted with sufficient lead time for review and acceptance prior to implementation.	Contractor-led changes may materially affect wall performance.

Geohazard Ref.	Consent Ref. / Page	Original Consent Wording (Extract)	Proposed Wording	Comment
2.3 – Groundwater Effects	40 (a-f)/ p.14	"GSMCP must include monitoring…"	Monitoring shall include continuous automated groundwater monitoring and stage-dependent monitoring frequencies during diaphragm wall construction, excavation and dewatering.	Addresses tidal influence and dynamic groundwater behaviour.
2.3 – Groundwater Effects	40(a–f) / p.14	"Monitoring data… triggers…"	Monitoring of groundwater shall be undertaken continuously using automated (real-time) systems, including both shallow (tidally influenced) and deeper ECBF groundwater systems.	Required to capture temporal and spatial groundwater variability.
2.3 – Groundwater Effects	40, 164–165 / p.14, 50–51	"GSMCP…"	A calibrated, site-specific groundwater model supported by additional testing shall be provided to assess drawdown, mounding, and uplift effects.	Diaphragm wall barrier effects and groundwater uncertainty require validation.
2.4 – Construction Methodology and Obstruction Risk	90 / p.27	"Construction methodology…"	The detailed design and construction methodology shall explicitly account for potential buried obstructions, including their influence on sequencing, wall alignment, and performance.	Unmapped obstructions may affect construction and deformation behaviour.
2.4 – Construction Methodology and Obstruction Risk	90 / p.27	Same as above	The contractor's methodology shall be submitted with sufficient lead time for independent review to confirm risks associated with obstructions are appropriately managed.	Ensures construction variability does not increase risk to adjacent structures.
2.5 – Monitoring, Trigger Levels and Contingency	40(d) / p.14	"Timing and frequency of monitoring…"	Monitoring frequency shall be stage-dependent, continuous during high-risk stages, and only reduced where stability is demonstrated to the satisfaction of a SQEP and Council.	Addresses risk of insufficient monitoring during critical phases.
2.5 – Monitoring, Trigger Levels and Contingency	40(d) / p.14	Same as above	Monitoring frequency shall be proactively adjusted based on risk and not solely in response to trigger exceedances.	Moves from reactive to proactive monitoring approach.
2.5 – Monitoring, Trigger Levels and Contingency	41 / p.14–15	"Pre-excavation building surveys…"	Building-specific assessments shall define acceptable movement limits, monitoring requirements, and mitigation measures for each structure, including M Social Hotel.	Ensures trigger levels are linked to building sensitivity.

Geohazard Ref.	Consent Ref. / Page	Original Consent Wording (Extract)	Proposed Wording	Comment
2.5 – Monitoring, Trigger Levels and Contingency	40(a–f), 176 / p.14, 58	"Monitoring results…"	All monitoring data shall be made available to Council and affected stakeholders, including near real-time access, and regular summary reports.	Ensures transparency and timely response capability.
2.5 – Monitoring, Trigger Levels and Contingency	173 / p.57	"Schedule E Retaining wall deflection stations: … at a minimum of once weekly"	Monitoring shall include continuous monitoring using automated (real-time) systems during diaphragm wall construction, excavation and dewatering.	Addresses uncertainty in predicted wall deflections and need for continuous verification.
2.5 – Monitoring, Trigger Levels and Contingency	173 / p.57	"Schedule E Inclinometers: … at a minimum of once weekly"	Monitoring shall include continuous monitoring using automated (real-time) systems during diaphragm wall construction, excavation and dewatering.	Addresses uncertainty in predicted wall deflections and need for continuous verification.
2.5 – Monitoring, Trigger Levels and Contingency	175 / p.58	"Notification to Council…"	Immediate notification to Council and all affected stakeholders (including M Social Hotel) upon detection of exceedance or incident.	Removes delay in response and improves risk management.
2.5 – Monitoring, Trigger Levels and Contingency	106 / p.30	"Alternative monitoring…"	Where access to adjacent properties is not available, conservative thresholds and supplementary monitoring shall be implemented.	Maintains protection where monitoring is constrained.

4.0 CLOSURE

This report has been prepared for use by Millenium & Copthorne Hotels New Zealand Limited in relation to the neighbouring Carpark Development Project, 2 Lower Hobson Street, Auckland CBD in accordance with the scope, proposed uses and limitations described in the report. Should you have further questions relating to the use of your report please do not hesitate to contact us.

Where a party other than Millenium & Copthorne Hotels New Zealand Limited or Auckland City Council seeks to rely upon or otherwise use this report, the consent of CMW should be sought prior to any such use. CMW can then advise whether the report and its contents are suitable for the intended use by the other party.

USING YOUR CMW GEOTECHNICAL REPORT

Geotechnical reporting relies on interpretation of facts and collected information using experience, professional judgement, and opinion. As such it generally has a level of uncertainty attached to it, which is often far less exact than other engineering design disciplines. The notes below provide general advice on what can be reasonably expected from your report and the inherent limitations of a geotechnical report.

Preparation of your report

Your geotechnical report has been written for your use on your project. The contents of your report may not meet the needs of others who may have different objectives or requirements. The report has been prepared using generally accepted Geotechnical Engineering and Engineering Geology practices and procedures. The opinions and conclusions reached in your report are made in accordance with these accepted principles. Specific items of geotechnical or geological importance are highlighted in the report.

In producing your report, we have relied on the information which is referenced or summarised in the report. If further information becomes available or the nature of your project changes, then the findings in this report may no longer be appropriate. In such cases the report must be reviewed, and any necessary changes must be made by us.

Your geotechnical report is based on your project's requirements

Your geotechnical report has been developed based on your specific project requirements and only applies to the site in this report. Project requirements could include the type of works being undertaken; project locality, size and configuration; the location of any structures on or around the site; the presence of underground utilities; proposed design methodology; the duration or design life of the works; and construction method and/or sequencing.

The information or advice in your geotechnical report should not be applied to any other project given the intrinsic differences between different projects and site locations. Similarly geotechnical information, data and conclusions from other sites and projects may not be relevant or appropriate for your project.

Interpretation of geotechnical data

Site investigations identify subsurface conditions at discrete locations. Additional geotechnical information (e.g. literature and external data source review, laboratory testing etc) are interpreted by Geologists or Engineers to provide an opinion about a site specific ground models, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist due to the variability of geological environments. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. Interpretation of factual data can be influenced by design and/or construction methods. Where these methods change review of the interpretation in the report may be required.

Subsurface conditions can change

Subsurface conditions are created by natural processes and then can be altered anthropically or over time. For example, groundwater levels can vary with time or activities adjacent to your site, fill may be placed on a site, or the consistency of near surface conditions might be susceptible to seasonal changes. The report is based on conditions which existed at the time of investigation. It is important to confirm whether conditions may have changed, particularly when large periods of time have elapsed since the investigations were performed.

Interpretation and use by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a geotechnical report. To help avoid misinterpretations, it is important to retain the assistance of CMW to work with other project design professionals who are affected by the contents of your report. CMW staff can explain the report implications to design professionals and then review design plans and specifications to see that they have correctly incorporated the findings of this report.

Your report's recommendations require confirmation during construction

Your report is based on site conditions as revealed through selective point sampling. Engineering judgement is then applied to assess how indicative of actual conditions throughout an area the point sampling might be. Any assumptions made cannot be substantiated until construction is complete. For this reason, you should retain geotechnical services throughout the construction stage, to identify variances from previous assumption, conduct additional tests if required and recommend solutions to problems encountered on site.

A Geotechnical Engineer, who is fully familiar with the site and the background information, can assess whether the report's recommendations remain valid and whether changes should be considered as the project develops. An unfamiliar party using this report increases the risk that the report will be misinterpreted.

Environmental matters are not covered

Unless specifically discussed in your report environmental matters are not covered by a CMW Geotechnical Report. Environmental matters might include the level of contaminants present of the site covered by this report, potential uses or treatment of contaminated materials or the disposal of contaminated materials. These matters can be complex and are often governed by specific legislation.

The personnel, equipment, and techniques used to perform an environmental study can differ significantly from those used in this report. For that reason, our report does not provide environmental recommendations. Unanticipated subsurface environmental problems can have large consequences for your site. If you have not obtained your own environmental information about the project site, ask your CMW contact about how to find environmental risk-management guidance.

CMW Geosciences
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APPENDIX A

CMW Geotechnical Preliminary Assessment Report

CMW Geosciences
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13 June 2025

Downtown Carpark Developments

2 Lower Hobson Street

GEOTECHNICAL PRELIMINARY ASSESSMENT REPORT

Millenium & Copthorne Hotels New Zealand Limited

Job No. AKL2025-0064AB | Version 1



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Version Control

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Review and Update History

Revision	Date	Comments
0	11 June 2025	DRAFT - Issued for Client review
1	13 June 2025	Issued for Client review

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1.0 INTRODUCTION

CMW Geosciences (CMW) was engaged by Millenium & Copthorne Hotels New Zealand Limited to carry out a geotechnical assessment of the potential impacts to the adjacent M Social Hotel of the proposed 20m excavation along the shared boundary located at 2 Lower Hobson Street.

The scope of work and associated terms and conditions of our engagement were detailed in our services proposal letter referenced AKL2025-0064AA, Rev. 0 dated 14 April 2025.

2.0 REFERENCE DOCUMENTS

The following documents have been reviewed during the assessment and preparation of this report. These are attached in Appendix B.

1. Downtown West – Downtown Car Park Redevelopment Geotechnical and Groundwater Assessment Report (Ref. 1016043.1000 v6 dated December 2024) by Tonkin & Taylor Ltd.
2. Geotechnical Investigation Report for Copthorne Harbour City – 196 Quay Street (Ref. 29493.1 dated December 2014) by Tonkin & Taylor Ltd.
3. Copthorne Harbourcity Refurbishment Structural Drawings (Ref. 1551 Rev. 4 dated 24 February 2015) by Structure Design

3.0 PROPOSED DEVELOPMENT

The proposed development at 2 Lower Hobson Street comprises a 20 m deep excavation for a basement of a new building development. Three sections of excavation protection have been designed as shown in Ref. no. 1 - Appendix G Figure 1 Geotechnical Layout Plan.

At Section 1, the shared boundary between the proposed development and the M Social Hotel, the excavation protection will comprise: 800mm thick reinforced concrete diaphragm wall, top-down construction of basement levels/ slabs, and temporary props at the top and bottom of the wall (Ref. no. 1 – Appendix G Section 1 – Top Down Construction)

M Social Hotel comprises a single structure built in the 1960's and have been redeveloped in 2015. The thirteen-storey structure is founded on reinforced concrete piles at approximately 13m depth (RL -12.5m). A portion of the structure (Grid D-14 to Grid D-17) having three levels of stairwell is extended to approximately 100mm from the property boundary as shown in Ref. no. 3 – Sheet no. S1-00.

4.0 FINDINGS AND RECOMMENDATIONS

4.1 Effects of the proposed excavation on the existing M Social Hotel

A 2D Finite Element Model was prepared utilising Bentley Plaxis 2D Advanced Version 2024.2.0.1144 to analyse the potential pile and ground displacement of the M Social Hotel during the construction of the basement of the neighbouring property. The following was taken into account in the model:

- Ground model and parameters based on Ref. no. 1 Table 4.7 (assessed by T+T) and Young's Modulus correlations for Hardening Soil Small parameters in Table 1 (assessed by CMW).

Table 4.7: Retention geotechnical design parameters

Design parameter		Units	Reclamation Fill / Marine Sediments	Pleistocene Sediments	Weathered ECBF	ECBF rock
Unit weight	γ	kN/m³	16.5	17.5	19	21
Effective friction angle	ϕ	deg	28	30	32	40
Effective cohesion	c'	kPa	3	5	7	100
Poisson's ratio	v'	-	0.3	0.3	0.3	0.25
Equivalent linear modulus of elasticity [1]	E'	MPa	6	20	40	400

Table 1: Plaxis HS Small Design Parameters

Unit Description	υ	E' (MPa)	E_{50} (MPa)	E_{oed} (MPa)	E_{ur} (MPa)	G_0 (MPa)	$\gamma 0.7$
Reclaimed fill / Marine Sediments	0.3	6	3.5	2.1	8.3	16800	1.9×10^{-4}
Pleistocene Sediments	0.3	20	13.3	7.8	31.4	71300	1.7×10^{-4}
Weathered ECBF	0.3	40	30.8	18.1	72.4	174300	1.5×10^{-4}
ECBF Rock	0.25	400	200	117.6	470.6	342500	1.4×10^{-4}

Notes:
ECBF = East Coast Bays Formation
υ = Poisson's ratio (based on typical published values for similar soil types)
E_{50} = secant modulus corresponding to 50% deviatoric stress at failure (1.3 – 2 times E')
E_{oed} = tangent oedometric modulus (1.3 – 2 times E50)
E_{ur} = unloading-reloading stiffness (4 times Eoed)
G_0 = maximum small-strain shear modulus (based on typical published values of shear wave velocity for similar soil types)
$\gamma_{0.7}$ = value of small strain (based on typical published values for similar soil types)

- Groundwater modelled at RL 3.4m (0.6m below ground level) to account for high tide.
- Wall stiffnesses for concrete (f'c = 40MPa) were reduced for long-term conditions as follows:
  - 50% reduction for existing pile and pile caps at M Social Hotel
  - 70% reduction for diaphragm wall and proposed basement slabs
- A 25-year return period earthquake (PGA = 0.05g) was modelled at the most critical excavation stage (Stage 6 in Figure 1)
- M Social Hotel axial pile loading (1,400kN per pile), layout and sizes based on Ref. no. 2

- Construction sequence, and wall properties based on Ref. no. 1 (Figure 1)

Figure 1: Retaining Wall Section 1 staging proposed by T+T

Section 1 – Top-down Diaphragm wall

1 Install 800 mm thick Diaphragm wall with a toe level at RL -20.3 m.
2 Excavate no deeper than RL 0.0 m and form 1(v):2(h) temporary berm to B01 FFL (RL +0.90 m).
3 Install 310UC118 temporary prop founding upon internal plunge piles at 5 m c/c spacing (or alternatively corner propping).
4 Excavate berm and continue excavation no deeper than RL 0.0 m and install 400 mm thick B01 level floor slab.
5 Excavate no deeper than RL -6.0 m and install 400 mm thick B03 level floor slab.
6 Excavate to no deeper than RL -10.7 m and form 1(v):1(h) temporary berm to basement excavation level (RL -16.3 m).
7 Install 450x9.53 temporary prop founding on the basement excavation floor (or leave berm in place until B04 slab construction).
8 Excavate berm and continue excavation to no deeper than RL -16.3 m and install B05 level slab.
9 Install B04 and B02 level slab.
10 Remove the lower and upper-level temporary prop and install GF slab.
11 Apply elevated groundwater design scenario.
12 Apply seismic design scenario.

The results of our analysis show that a maximum total displacement of about 55mm is estimated to occur throughout the excavation of the basement. The maximum displacement occurs at the 914mm diameter piles at Grids B-15 and C-15 and pushes down towards the bottom section of the retaining wall.

The estimated design actions on the diaphragm wall are as follows: Maximum Shear Force = 1230kN/m acting at depth RL -6m; Maximum Bending Moment = 1290kN-m/m acting at depth RL -12m. These are both occurring at the last stage of excavation.

The details of the analysis are presented in Appendix A.

Based on Ref. no. 1 - Appendix H, the combined ground settlement due to groundwater drawdown and mechanical settlement of M Social Building analysed by the designers is estimated to be about 12mm. This is significantly larger than the estimated displacement presented by T+T. The maximum differential settlement estimated was 1V: 1044H.

4.2 Further Detailed Design Considerations

Due to the findings in Section 4.1, the following clarifications/comments on the design is warranted:

- Potential groundwater drawdown has been modelled by T+T using GeoStudio SEEP/w for transient drained, permanent drained, and tanked basement conditions. Considering the proximity of the wall to the sea, the tidal influence needs to be considered in the wall design.
- The existing pile loads and expected pile movements of the M Social Hotel foundation piles has a potential influence on the retaining wall as evident in the 2D FEM. One side of the existing piles will effectively lose its lateral resistance and is expected to move towards the diaphragm wall. This pile movement needs to be mitigated in the design of the diaphragm wall and the excavation steps.
- A sensitivity analysis must be undertaken for different slab stiffnesses and deflections to account for potential variation in concrete strength and wall/ slab connections upon installation.

- An assessment of the rock mass behaviour must be presented in the detailed design.

It is indicated in the T+T report (Ref. no. 2) that a detailed analysis using FEM will be undertaken considering sensitivity of wall embedment for fault / joint at the base, water pressures, and variability of rock strength. It is recommended to incorporate the comments provided above in the model.

4.3 Further Construction Considerations

Additionally, the following must be considered in the construction:

- Ensure the tolerances of the diaphragm walls, the guide walls, crane boom etc. will not encroach the property boundary. This should be done considering the worst-case scenario for the hotel piles in terms of construction tolerances.
- The settlement of existing services must be monitored.
- Construction plant temporary works platform must be considered.
- A real-time monitoring of lateral displacements and early warning system is recommended to be established. It is recommended to increase the instrumentation points indicated in Ref. no. 1 - Appendix A Construction Monitoring and Instrumentation Plan as per Table 2.

Table 2: Required Monitoring Instrumentation

Type / Description	Minimum Monitoring Frequency	Trigger Levels	
		Alert	Alarm
3nos. Inclinometer with data logger located at both ends and centre of the retaining wall crest	Real-time, 24/7	50% of expected lateral deflection	100% of expected lateral deflection
Piezometer	Once per week or every 1m excavated depth		
Ground settlement pins behind retaining wall and at critical areas of the building, and buried services	Once per week or every 1m excavated depth	1:750 Differential vertical settlement	1:500 Differential vertical settlement
Tilt meter at the top of the building	Once per week or every 1m excavated depth	TBC	TBC

5.0 CLOSURE

This report has been prepared for use by Millenium & Copthorne Hotels New Zealand Limited in relation to the Downtown Carpark Developments, 2 Lower Hobson Street project in accordance with the scope, proposed uses and limitations described in the report. Should you have further questions relating to the use of your report please do not hesitate to contact us.

Where a party other than Millenium & Copthorne Hotels New Zealand Limited seeks to rely upon or otherwise use this report, the consent of CMW should be sought prior to any such use. CMW can then advise whether the report and its contents are suitable for the intended use by the other party.

USING YOUR CMW GEOTECHNICAL REPORT

Geotechnical reporting relies on interpretation of facts and collected information using experience, professional judgement, and opinion. As such it generally has a level of uncertainty attached to it, which is often far less exact than other engineering design disciplines. The notes below provide general advice on what can be reasonably expected from your report and the inherent limitations of a geotechnical report.

Preparation of your report

Your geotechnical report has been written for your use on your project. The contents of your report may not meet the needs of others who may have different objectives or requirements. The report has been prepared using generally accepted Geotechnical Engineering and Engineering Geology practices and procedures. The opinions and conclusions reached in your report are made in accordance with these accepted principles. Specific items of geotechnical or geological importance are highlighted in the report.

In producing your report, we have relied on the information which is referenced or summarised in the report. If further information becomes available or the nature of your project changes, then the findings in this report may no longer be appropriate. In such cases the report must be reviewed, and any necessary changes must be made by us.

Your geotechnical report is based on your project's requirements

Your geotechnical report has been developed based on your specific project requirements and only applies to the site in this report. Project requirements could include the type of works being undertaken; project locality, size and configuration; the location of any structures on or around the site; the presence of underground utilities; proposed design methodology; the duration or design life of the works; and construction method and/or sequencing.

The information or advice in your geotechnical report should not be applied to any other project given the intrinsic differences between different projects and site locations. Similarly geotechnical information, data and conclusions from other sites and projects may not be relevant or appropriate for your project.

Interpretation of geotechnical data

Site investigations identify subsurface conditions at discrete locations. Additional geotechnical information (e.g. literature and external data source review, laboratory testing etc) are interpreted by Geologists or Engineers to provide an opinion about a site specific ground models, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist due to the variability of geological environments. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. Interpretation of factual data can be influenced by design and/or construction methods. Where these methods change review of the interpretation in the report may be required.

Subsurface conditions can change

Subsurface conditions are created by natural processes and then can be altered anthropically or over time. For example, groundwater levels can vary with time or activities adjacent to your site, fill may be placed on a site, or the consistency of near surface conditions might be susceptible to seasonal changes. The report is based on conditions which existed at the time of investigation. It is important to confirm whether conditions may have changed, particularly when large periods of time have elapsed since the investigations were performed.

Interpretation and use by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a geotechnical report. To help avoid misinterpretations, it is important to retain the assistance of CMW to work with other project design professionals who are affected by the contents of your report. CMW staff can explain the report implications to design professionals and then review design plans and specifications to see that they have correctly incorporated the findings of this report.

Your report's recommendations require confirmation during construction

Your report is based on site conditions as revealed through selective point sampling. Engineering judgement is then applied to assess how indicative of actual conditions throughout an area the point sampling might be. Any assumptions made cannot be substantiated until construction is complete. For this reason, you should retain geotechnical services throughout the construction stage, to identify variances from previous assumption, conduct additional tests if required and recommend solutions to problems encountered on site.

A Geotechnical Engineer, who is fully familiar with the site and the background information, can assess whether the report's recommendations remain valid and whether changes should be considered as the project develops. An unfamiliar party using this report increases the risk that the report will be misinterpreted.

Environmental matters are not covered

Unless specifically discussed in your report environmental matters are not covered by a CMW Geotechnical Report. Environmental matters might include the level of contaminants present of the site covered by this report, potential uses or treatment of contaminated materials or the disposal of contaminated materials. These matters can be complex and are often governed by specific legislation.

The personnel, equipment, and techniques used to perform an environmental study can differ significantly from those used in this report. For that reason, our report does not provide environmental recommendations. Unanticipated subsurface environmental problems can have large consequences for your site. If you have not obtained your own environmental information about the project site, ask your CMW contact about how to find environmental risk-management guidance.

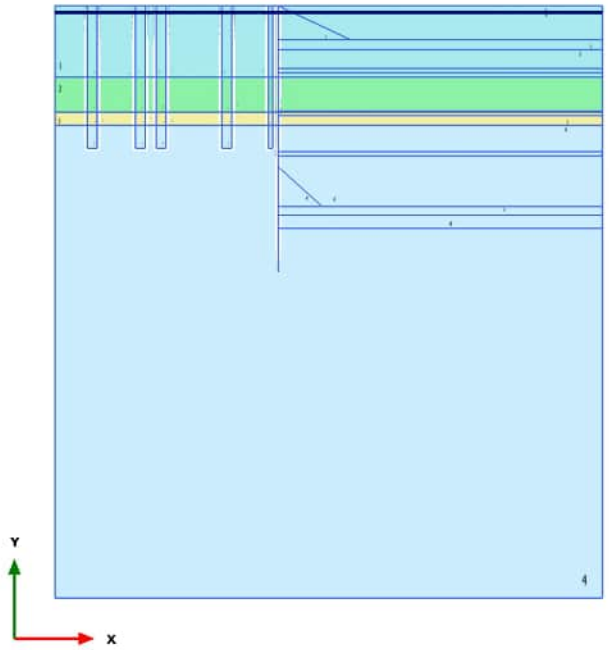
CMW Geosciences
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APPENDIX A
2D Finite Element Modelling Outputs

PLAXIS Report

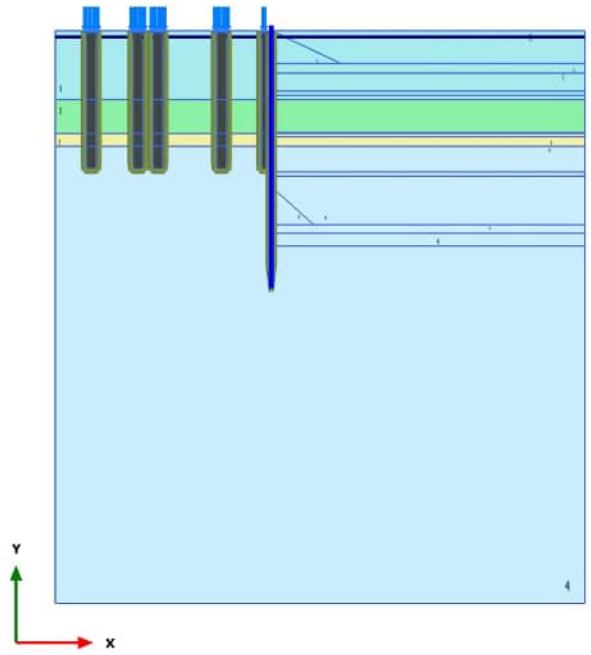
Filename	M Social Hotel_2.p2dx
Directory	C:\Users\JennaPallarca\CMW Geosciences Pty Ltd\Auckland Office - AKL2025-0064 Downtown Carpark Redevelopment\Office Technical\Plaxis\RevA\
Title	M Social Hotel
Model	Plane strain
Elements	15-Noded
PLAXIS Version	Version 2024.2.0.1144

1.1.1.1 Calculation results, Initial phase [InitialPhase] (0/0), Materials plot



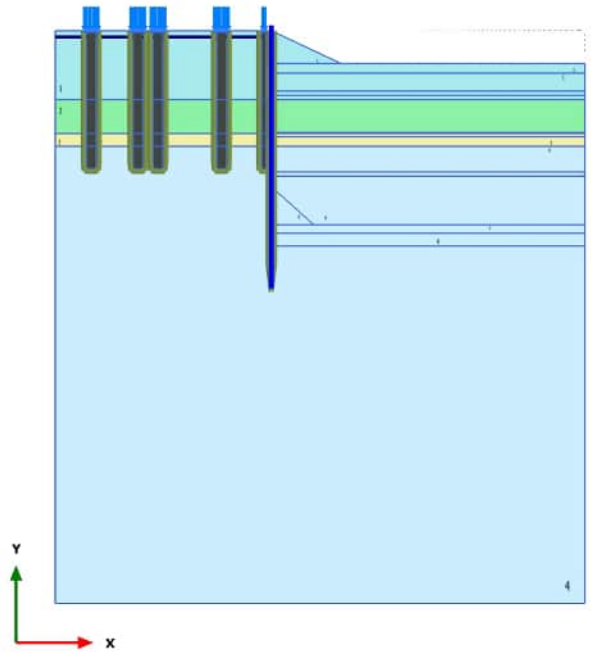
Materials plot

1.1.1.2 Calculation results, Phase_2 [Phase_2] (2/3), Materials plot



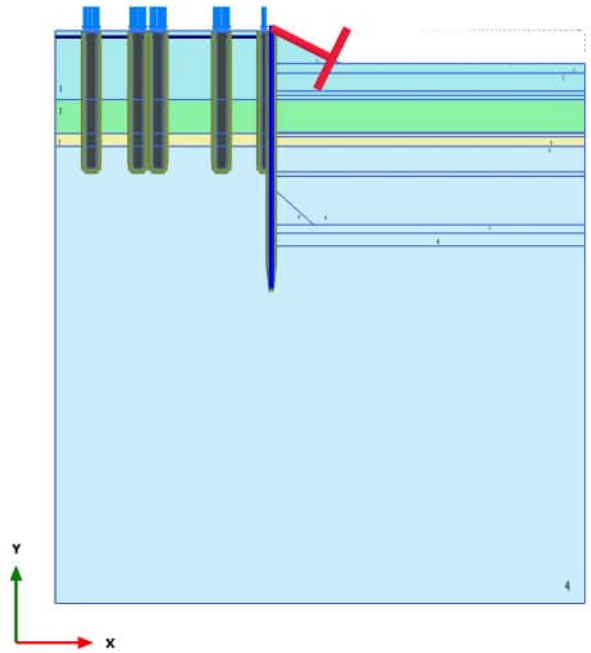
Materials plot

1.1.1.3 Calculation results, Phase_3 [Phase_3] (3/10), Materials plot



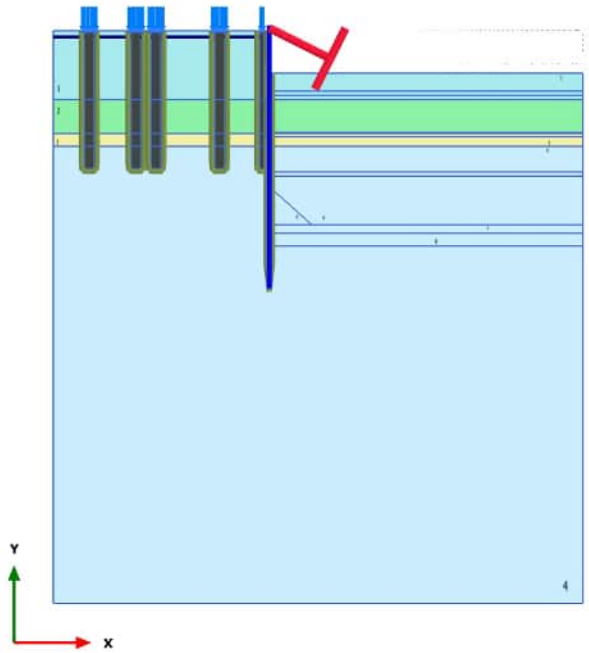
Materials plot

1.1.1.4 Calculation results, Phase_4 [Phase_4] (4/34), Materials plot



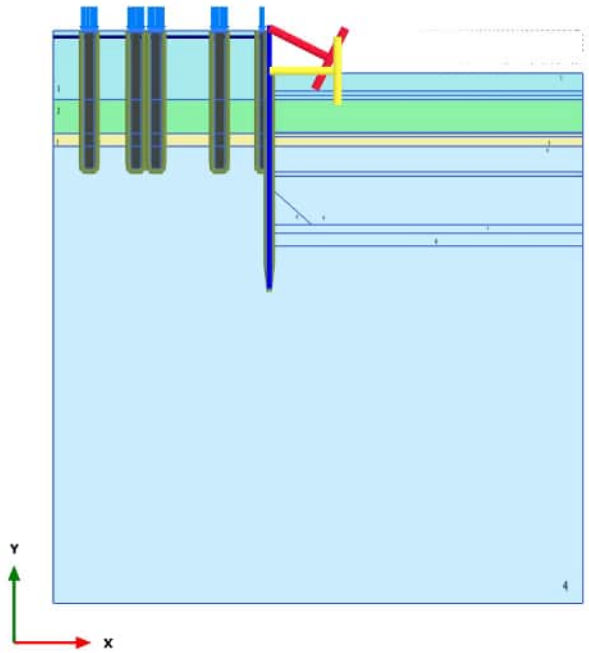
Materials plot

1.1.1.5 Calculation results, Phase_5 [Phase_5] (5/42), Materials plot



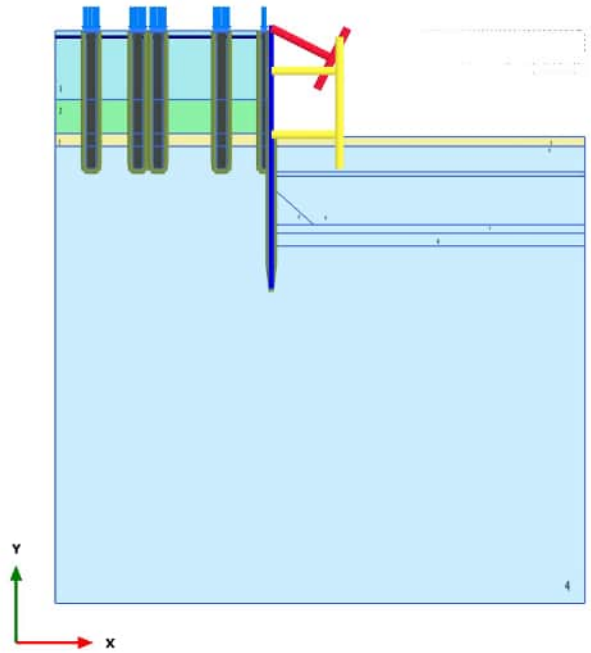
Materials plot

1.1.1.6 Calculation results, Phase_6 [Phase_6] (6/89), Materials plot



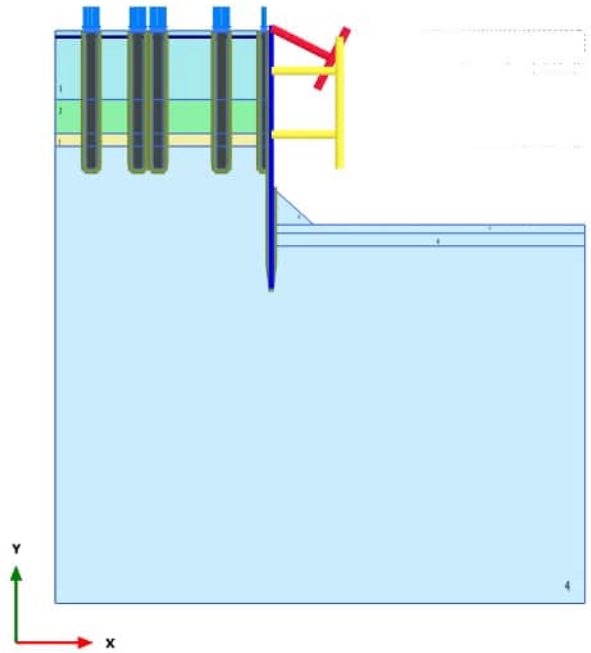
Materials plot

1.1.1.7 Calculation results, Phase_8 [Phase_8] (8/91), Materials plot



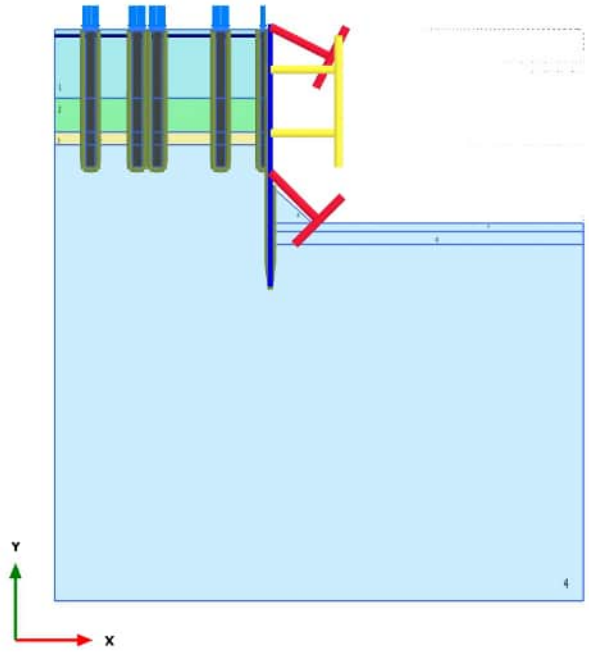
Materials plot

1.1.1.8 Calculation results, Phase_9 [Phase_9] (9/166), Materials plot



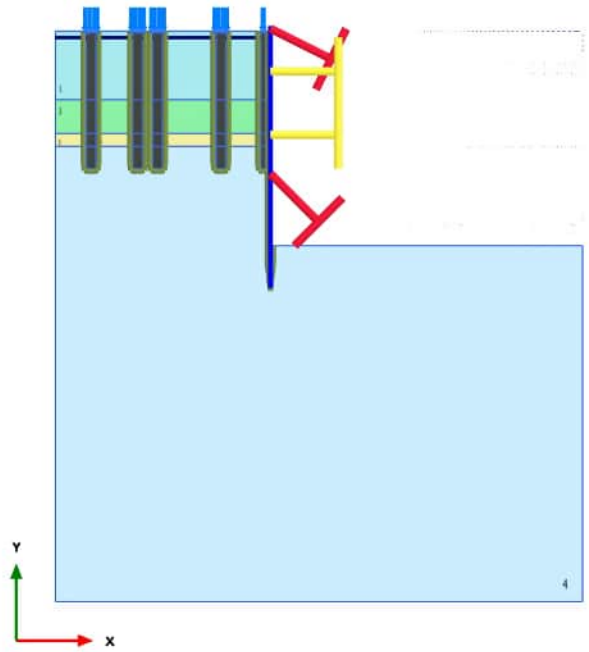
Materials plot

1.1.1.9 Calculation results, Phase_10 [Phase_10] (10/168), Materials plot



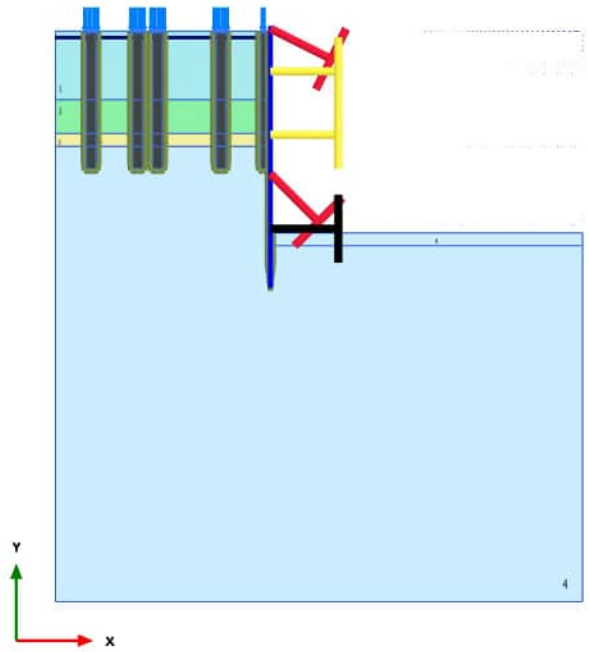
Materials plot

1.1.1.10 Calculation results, Phase_11 [Phase_11] (11/218), Materials plot



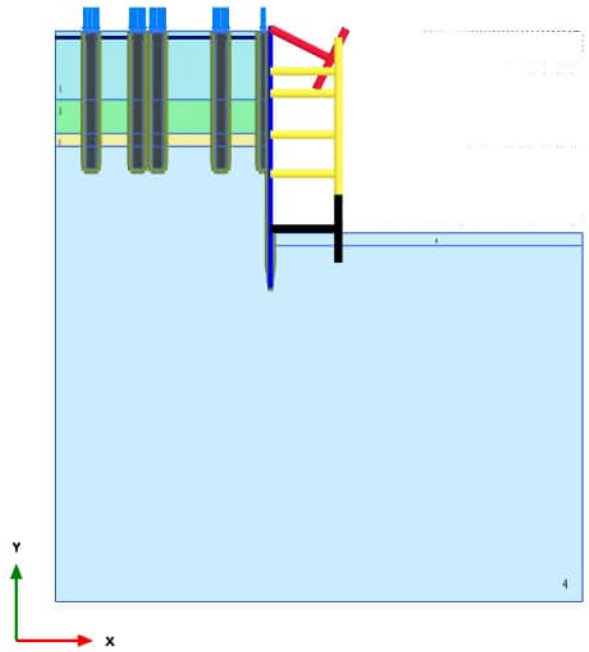
Materials plot

1.1.1.11 Calculation results, Phase_12 [Phase_12] (12/221), Materials plot



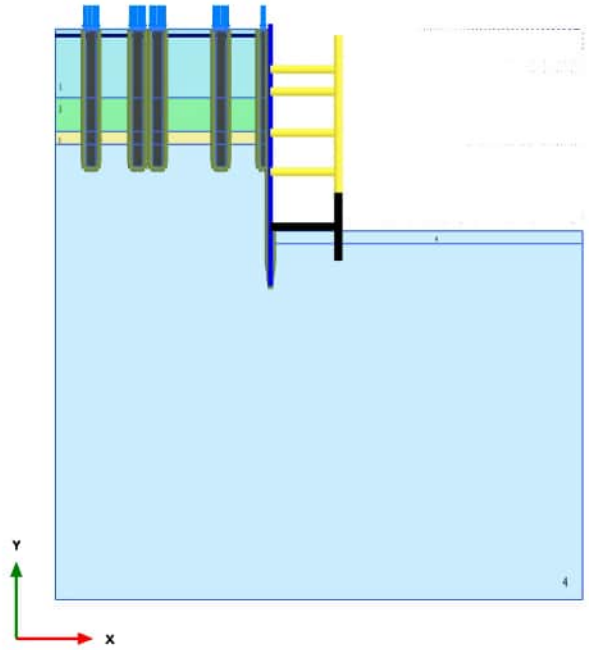
Materials plot

1.1.1.12 Calculation results, Phase_13 [Phase_13] (13/226), Materials plot



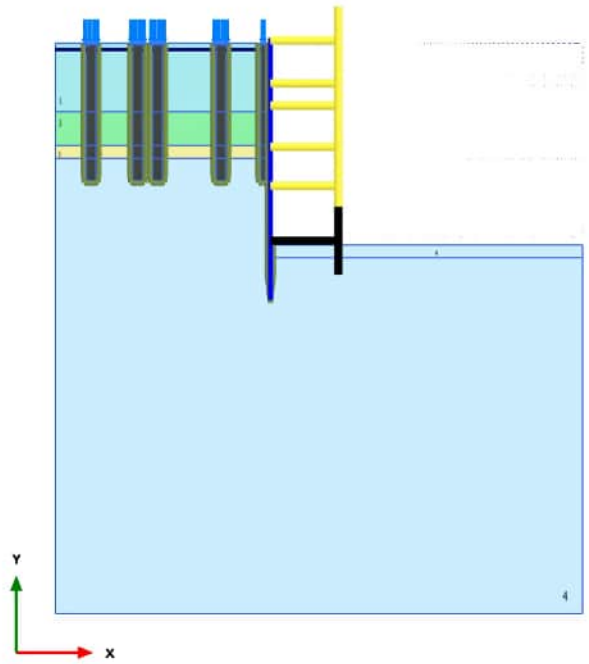
Materials plot

1.1.1.13 Calculation results, Phase_14 [Phase_14] (14/229), Materials plot



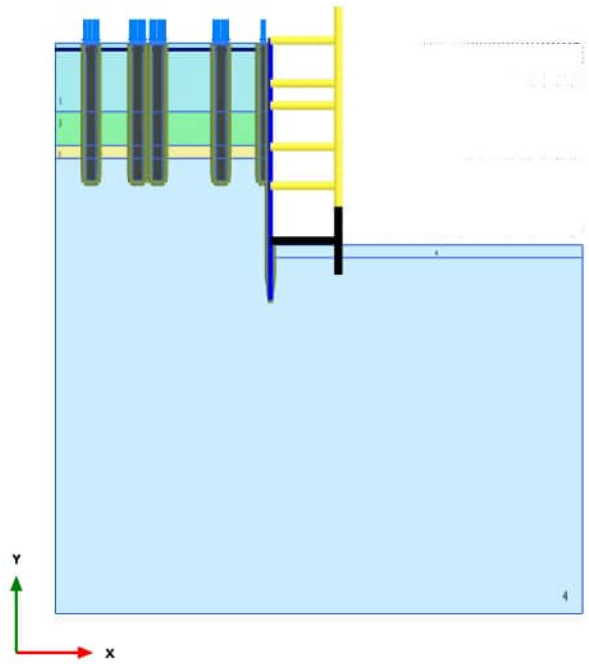
Materials plot

1.1.1.14 Calculation results, Phase_15 [Phase_15] (15/245), Materials plot



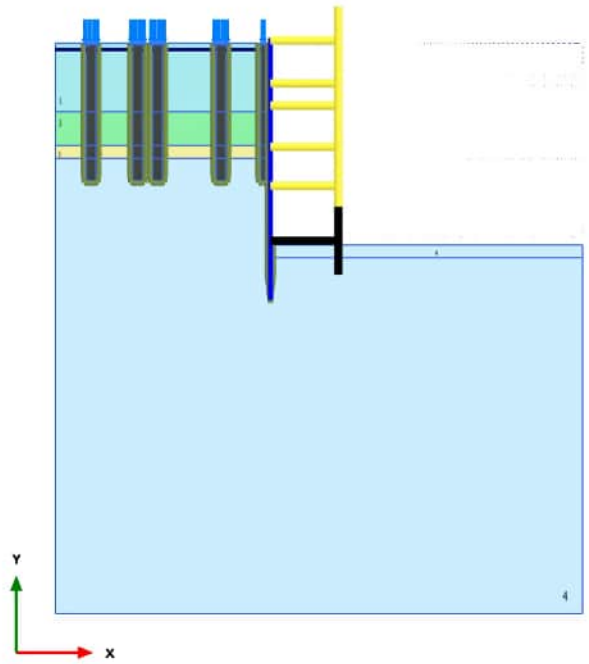
Materials plot

1.1.1.15 Calculation results, Phase_16 [Phase_16] (16/248), Materials plot



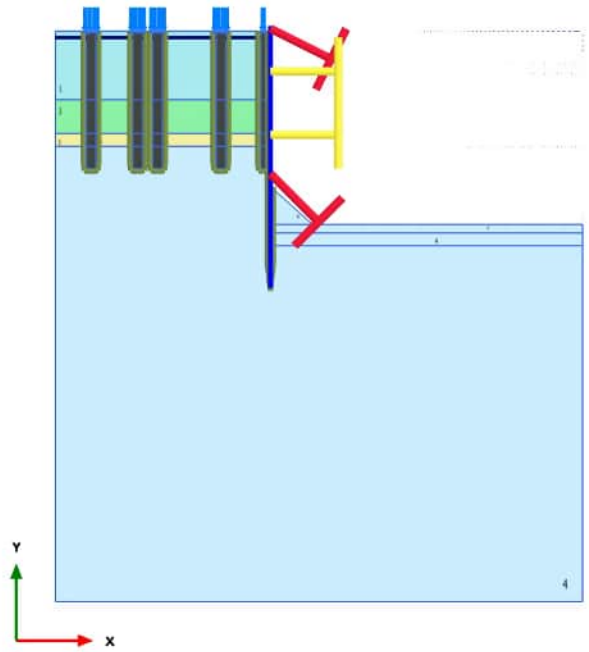
Materials plot

1.1.1.16 Calculation results, Phase_17 [Phase_17] (17/253), Materials plot



Materials plot

1.1.1.17 Calculation results, Phase_18 [Phase_18] (18/258), Materials plot



Materials plot

1.1.2.1.1 Materials - Soil and interfaces - HS small

Identification number		1	2	3	4
Identification		Reclamation Fill	Pleistocene Sediments	Weathered ECBF	ECBF Rock
Soil model		HS small	HS small	HS small	HS small
Drainage type		Undrained A	Undrained A	Undrained A	Undrained A
Colour					
Comments					
γ_unsat	kN/m³	16.50	17.50	19.00	21.00
γ_sat	kN/m³	18.50	19.50	21.00	21.00
e_init		0.5000	0.5000	0.5000	0.5000
n_init		0.3333	0.3333	0.3333	0.3333
E_50^ref	kN/m²	3500	13.30E3	30.80E3	200.0E3
E_oed^ref	kN/m²	2100	7800	18.10E3	117.6E3
E_ur^ref	kN/m²	8300	31.40E3	72.40E3	470.6E3
ν_ur		0.3000	0.3000	0.3000	0.2500
Use alternatives		False	False	False	False
C_c		0.1645	0.04428	0.01908	2.937E-3
C_s		0.05827	0.01634	7.539E-3	1.712E-3
e_init		0.5000	0.5000	0.5000	0.5000

Identification number		1	2	3	4
power (m)		0.5000	0.5000	0.5000	0.5000
p_ref	kN/m²	100.0	100.0	100.0	100.0
G_0^ref	kN/m²	16.80E3	71.30E3	174.3E3	342.5E3
γ_0.7		0.1900E-3	0.1700E-3	0.1500E-3	0.1400E-3
c'_ref	kN/m²	3.000	5.000	7.000	100.0
φ' (phi)	°	28.00	30.00	32.00	40.00
ψ (psi)	°	0.000	0.000	0.000	0.000
c'_inc	kN/m²/m	0.000	0.000	0.000	0.000
y_ref	m	0.000	0.000	0.000	0.000
Dilatancy cut-off		False	False	False	False
e_min		1.000E-9	1.000E-9	1.000E-9	1.000E-9
e_max		999.0	999.0	999.0	999.0
Tension cut-off		True	True	True	True
Tensile strength	kN/m²	0.000	0.000	0.000	0.000
Use defaults		True	True	True	True
K_0^nc		0.5305	0.5000	0.4701	0.3572
R_f		0.9000	0.9000	0.9000	0.9000
Determination		ν-undrained definition	ν-undrained definition	ν-undrained definition	ν-undrained definition
ν_u definition method		Direct	Direct	Direct	Direct
ν_u,equivalent (nu)		0.4950	0.4950	0.4950	0.4950

Identification number		1	2	3	4
Skempton B		0.9783	0.9783	0.9783	0.9833
K_w,ref/n	kN/m²	311.2E3	1.177E6	2.715E6	18.45E6
Classification type		Standard	Standard	Standard	Standard
Soil class (Standard)		Coarse	Fine	Coarse	Coarse
< 2 µm	%	10.00	46.00	10.00	10.00
2 µm - 50 µm	%	13.00	26.00	13.00	13.00
50 µm - 2 mm	%	77.00	28.00	77.00	77.00
Use defaults		False	False	False	False
k_x	m/day	0.000	0.000	0.000	0.000
k_y	m/day	0.000	0.000	0.000	0.000
Void ratio dependency		False	False	False	False
c_k		1000E12	1000E12	1000E12	1000E12
n_init		0.3333	0.3333	0.3333	0.3333
-ψ_unsat	m	10.00E3	10.00E3	10.00E3	10.00E3
c_s	kJ/t/K	0.000	0.000	0.000	0.000
λ_s	kW/m/K	0.000	0.000	0.000	0.000
ρ_s	t/m³	2.600	2.600	2.600	2.600
Thermal expansion type		Isotropic	Isotropic	Isotropic	Isotropic
α_sv	1/K	0.000	0.000	0.000	0.000
Phase change		False	False	False	False

Identification number		1	2	3	4
D_v	m²/day	0.000	0.000	0.000	0.000
f_Tv		0.000	0.000	0.000	0.000
Stiffness determination		Derived	Derived	Derived	Derived
Strength determination		Manual	Manual	Manual	Manual
R_inter		0.8000	0.8000	0.8000	0.8000
Consider gap closure		True	True	True	True
δ_inter	m	0.000	0.000	0.000	0.000
Cross permeability		Impermeable	Impermeable	Impermeable	Impermeable
Drainage conductivity, dk	m³/day/m	0.000	0.000	0.000	0.000
R_thermal	m² K/kW	0.000	0.000	0.000	0.000
K_0 determination		Automatic	Automatic	Automatic	Automatic
K_0,x		0.5305	0.5000	0.4701	0.3572
K_0,z		0.5305	0.5000	0.4701	0.3572
POP	kN/m²	0.000	0.000	0.000	0.000
OCR		1.000	1.000	1.000	1.000

1.1.2.1.2 Materials - Soil and interfaces - Linear Elastic

Identification number		5
Identification		Concrete 50%E
Soil model		Linear Elastic
Drainage type		Non-porous
Colour		
Comments		
γ_unsat	kN/m³	24.00
γ_sat	kN/m³	24.00
e_init		0.5000
n_init		0.3333
E_ref	kN/m²	13.70E6
ν (nu)		0.2000
G_ref	kN/m²	5.708E6
E_oed	kN/m²	15.22E6
E_inc	kN/m²/m	0.000
y_ref	m	0.000
V_s	m/s	1528
V_p	m/s	2494

Identification number		5
k_x	m/day	0.000
k_y	m/day	0.000
c_k		1000E12
n_init		0.3333
c_s	kJ/t/K	0.000
λ_s	kW/m/K	0.000
ρ_s	t/m³	2.600
Thermal expansion type		Isotropic
α_sv	1/K	0.000
Phase change		False
D_v	m²/day	0.000
f_Tv		0.000
Stiffness determination		Derived
Strength determination		Manual
R_inter		1.000
Consider gap closure		True
Cross permeability		Impermeable
Drainage conductivity, dk	m³/day/m	0.000
R_thermal	m² K/kW	0.000
K_0 determination		Automatic

Identification number	5
K_0,x	0.5000
K_0,z	0.5000

1.1.2.2 Materials - Anchors

Identification number		1	2	3
Identification		Strut	Slab b1-b4	Slab b5
Material type		Elastic	Elastic	Elastic
Colour				
Comments				
L_spacing	m	5.000	1.000	1.000
EA	kN	3.150E6	7.672E6	15.38E6
c	kJ/t/K	0.000	0.000	0.000
λ	kW/m/K	0.000	0.000	0.000
ρ	t/m³	0.000	0.000	0.000
α	1/K	0.000	0.000	0.000
A_eff,T	m²	0.000	0.000	0.000

1.1.2.3 Materials - Plates

Identification number		1	2	3
Identification		D-wall 70%	Pile cap 50%	D-wall
Material type		Elastic	Elastic	Elastic
Colour				
Comments				
w	kN/m/m	24.00	24.00	24.00
Prevent punching		False	False	False
Isotropic		True	True	True
EA_1	kN/m	15.34E6	6.850E6	15.34E6
EA_2	kN/m	15.34E6	6.850E6	15.34E6
E_1	kN/m²	19.18E6	13.70E6	19.18E6
E_2	kN/m²	19.18E6	13.70E6	19.18E6
EI	kN m²/m	818.3E3	142.7E3	818.3E3
ν (nu)		0.2000	0.2000	0.2000
d	m	0.8000	0.5000	0.8000
c	kJ/t/K	0.000	0.000	0.000
λ	kW/m/K	0.000	0.000	0.000
ρ	t/m³	0.000	0.000	0.000

Identification number		1	2	3
α	1/K	0.000	0.000	0.000
A_eff,T	m²	0.000	0.000	0.000

1.1.2.4 Materials - Embedded beams

Identification number		1	2
Identification		Hotel Piles P3	Hotel Piles P2
Material type		Elastic	Elastic
Colour			
Comments			
γ	kN/m³	24.00	24.00
L_spacing	m	10.00	8.000
Cross section type		Predefined	Predefined
Predefined cross section type		Solid circular beam	Solid circular beam
Diameter	m	0.4000	0.9140
A	m²	0.1257	0.6561
I	m⁴	1.257E-3	0.03426
E	kN/m²	27.40E6	27.40E6
Axial skin resistance		Layer dependent	Layer dependent
T_max	kN/m	65.00	325.0
Lateral resistance		Linear	Linear
T_lat, start, max	kN/m	0.000	0.000
T_lat, end, max	kN/m	800.0	800.0

Identification number		1	2
F_max	kN	775.0	3900
Default values		True	True
Axial stiffness factor		0.2236	0.4913
Lateral stiffness factor		0.2236	0.4913
Base stiffness factor		2.236	4.913

1.1.3.4 Calculation results, Fixed-end anchor, Phase_4 [Phase_4] (4/34), Table of fixed-end anchors

Structural element	Node [10^3]	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_1_1	25714	1	20.400	4.250	-1.472	-1.472	0.000	-26.565	2.236

1.1.3.5 Calculation results, Fixed-end anchor, Phase_5 [Phase_5] (5/42), Table of fixed-end anchors

Structural element	Node [10 ^{3}]	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_{z} [°]	Length [m]
FixedEndAnchor_1_1	25714	1	20.400	4.250	-311.218	-311.218	0.000	-26.565	2.236

1.1.3.6 Calculation results, Fixed-end anchor, Phase_6 [Phase_6] (6/89), Table of fixed-end anchors

Structural element	Node [10^3]	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_1_1	25714	1	20.400	4.250	-310.214	-311.218	0.000	-26.565	2.236
FixedEndAnchor_2_1	31403	1	20.400	0.200	-1.363	-1.363	0.000	0.000	1.000

1.1.3.7 Calculation results, Fixed-end anchor, Phase_8 [Phase_8] (8/91), Table of fixed-end anchors

Structural element	Node [10^3]	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_1_1	25714	1	20.400	4.250	395.917	-311.218	395.917	-26.565	2.236
FixedEndAnchor_2_1	31403	1	20.400	0.200	-456.906	-456.906	0.000	0.000	1.000
FixedEndAnchor_4_1	36587	1	20.400	-5.800	0.374	0.000	0.374	0.000	1.000

1.1.3.8 Calculation results, Fixed-end anchor, Phase_9 [Phase_9] (9/166), Table of fixed-end anchors

Structural element	Node [10^3]	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_1_1	25714	1	20.400	4.250	-248.351	-311.218	395.917	-26.565	2.236
FixedEndAnchor_2_1	31403	1	20.400	0.200	5.117	-456.906	5.117	0.000	1.000
FixedEndAnchor_4_1	36587	1	20.400	-5.800	-2033.628	-2033.628	0.374	0.000	1.000

1.1.3.9 Calculation results, Fixed-end anchor, Phase_10 [Phase_10] (10/168), Table of fixed-end anchors

Structural element	Node [10^3]	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_1_1	25714	1	20.400	4.250	-248.362	-311.218	395.917	-26.565	2.236
FixedEndAnchor_2_1	31403	1	20.400	0.200	5.131	-456.906	5.131	0.000	1.000
FixedEndAnchor_4_1	36587	1	20.400	-5.800	-2033.669	-2033.669	0.374	0.000	1.000
FixedEndAnchor_5_1	51138	1	20.400	-9.500	-0.725	-0.725	0.000	-45.000	1.414

1.1.3.10 Calculation results, Fixed-end anchor, Phase_11 [Phase_11] (11/218), Table of fixed-end anchors

Structural element	Node [10^3]	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_1_1	25714	1	20.400	4.250	-365.783	-365.783	395.917	-26.565	2.236
FixedEndAnchor_2_1	31403	1	20.400	0.200	88.648	-456.906	88.648	0.000	1.000
FixedEndAnchor_4_1	36587	1	20.400	-5.800	-2027.897	-2048.493	0.374	0.000	1.000
FixedEndAnchor_5_1	51138	1	20.400	-9.500	-7057.010	-7057.010	0.000	-45.000	1.414

1.1.3.11 Calculation results, Fixed-end anchor, Phase_12 [Phase_12] (12/221), Table of fixed-end anchors

Structural element	Node [10^3]	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_1_1	25714	1	20.400	4.250	-365.693	-365.783	395.917	-26.565	2.236
FixedEndAnchor_2_1	31403	1	20.400	0.200	87.198	-456.906	88.648	0.000	1.000
FixedEndAnchor_4_1	36587	1	20.400	-5.800	-2038.897	-2048.493	0.374	0.000	1.000
FixedEndAnchor_5_1	51138	1	20.400	-9.500	-7077.428	-7077.428	0.000	-45.000	1.414
FixedEndAnchor_6_1	56755	1	20.400	-14.700	19.954	0.000	19.954	0.000	1.000

1.1.3.12 Calculation results, Fixed-end anchor, Phase_13 [Phase_13] (13/226), Table of fixed-end anchors

Structural element	Node [10^3]	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_1_1	25714	1	20.400	4.250	-549.050	-549.050	395.917	-26.565	2.236
FixedEndAnchor_2_1	31403	1	20.400	0.200	169.866	-456.906	169.866	0.000	1.000
FixedEndAnchor_3_1	31374	1	20.400	-1.900	-80.427	-80.427	0.000	0.000	1.000
FixedEndAnchor_4_1	36587	1	20.400	-5.800	-2094.470	-2094.470	0.374	0.000	1.000
FixedEndAnchor_5_1	51138	1	20.400	-9.500	-1211.104	-1415.486	0.000	0.000	1.000
FixedEndAnchor_6_1	56755	1	20.400	-14.700	-138.971	-138.971	19.954	0.000	1.000

1.1.3.13 Calculation results, Fixed-end anchor, Phase_14 [Phase_14] (14/229), Table of fixed-end anchors

Structural element	Node [10^3]	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_2_1	31403	1	20.400	0.200	-114.091	-456.906	169.866	0.000	1.000
FixedEndAnchor_3_1	31374	1	20.400	-1.900	119.632	-80.427	119.632	0.000	1.000
FixedEndAnchor_4_1	36587	1	20.400	-5.800	-2108.831	-2108.831	0.374	0.000	1.000
FixedEndAnchor_5_1	51138	1	20.400	-9.500	-1219.692	-1415.486	0.000	0.000	1.000
FixedEndAnchor_6_1	56755	1	20.400	-14.700	-147.226	-147.226	19.954	0.000	1.000

1.1.3.14 Calculation results, Fixed-end anchor, Phase_15 [Phase_15] (15/245), Table of fixed-end anchors

Structural element	Node [10^3]	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_1_1	25714	1	20.400	4.250	-0.073	-109.810	79.183	0.000	2.000
FixedEndAnchor_2_1	31403	1	20.400	0.200	-114.062	-456.906	169.866	0.000	1.000
FixedEndAnchor_3_1	31374	1	20.400	-1.900	119.709	-80.427	119.709	0.000	1.000
FixedEndAnchor_4_1	36587	1	20.400	-5.800	-2108.807	-2108.850	0.374	0.000	1.000
FixedEndAnchor_5_1	51138	1	20.400	-9.500	-1220.299	-1415.486	0.000	0.000	1.000
FixedEndAnchor_6_1	56755	1	20.400	-14.700	-148.445	-148.445	19.954	0.000	1.000

1.1.3.15 Calculation results, Fixed-end anchor, Phase_16 [Phase_16] (16/248), Table of fixed-end anchors

Structural element	Node [10^3]	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_1_1	25714	1	20.400	4.250	-0.073	-109.810	79.183	0.000	2.000
FixedEndAnchor_2_1	31403	1	20.400	0.200	-114.062	-456.906	169.866	0.000	1.000
FixedEndAnchor_3_1	31374	1	20.400	-1.900	119.709	-80.427	119.709	0.000	1.000
FixedEndAnchor_4_1	36587	1	20.400	-5.800	-2108.805	-2108.850	0.374	0.000	1.000
FixedEndAnchor_5_1	51138	1	20.400	-9.500	-1220.299	-1415.486	0.000	0.000	1.000
FixedEndAnchor_6_1	56755	1	20.400	-14.700	-148.441	-148.445	19.954	0.000	1.000

1.1.3.16 Calculation results, Fixed-end anchor, Phase_17 [Phase_17] (17/253), Table of fixed-end anchors

Structural element	Node [10^3]	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_1_1	25714	1	20.400	4.250	6.150	-109.810	79.183	0.000	2.000
FixedEndAnchor_2_1	31403	1	20.400	0.200	-89.979	-456.906	169.866	0.000	1.000
FixedEndAnchor_3_1	31374	1	20.400	-1.900	160.982	-80.427	160.982	0.000	1.000
FixedEndAnchor_4_1	36587	1	20.400	-5.800	-2046.575	-2108.850	0.374	0.000	1.000
FixedEndAnchor_5_1	51138	1	20.400	-9.500	-1192.225	-1415.486	0.000	0.000	1.000
FixedEndAnchor_6_1	56755	1	20.400	-14.700	-66.087	-148.445	19.954	0.000	1.000

1.1.3.17 Calculation results, Fixed-end anchor, Phase_18 [Phase_18] (18/258), Table of fixed-end anchors

Structural element	Node [10^3]	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_1_1	25714	1	20.400	4.250	-236.061	-311.218	395.917	-26.565	2.236
FixedEndAnchor_2_1	31403	1	20.400	0.200	59.453	-456.906	59.453	0.000	1.000
FixedEndAnchor_4_1	36587	1	20.400	-5.800	-1944.213	-2033.628	0.374	0.000	1.000
FixedEndAnchor_5_1	51138	1	20.400	-9.500	172.982	0.000	172.982	-45.000	1.414

1.1.3.19 Calculation results, Fixed-end anchor, Phase_7 [Phase_7] (7/339), Table of fixed-end anchors

Structural element	Node [10^3]	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_1_1	25714	1	20.400	4.250	395.831	-311.218	395.831	-26.565	2.236
FixedEndAnchor_2_1	31403	1	20.400	0.200	-456.854	-456.854	0.000	0.000	1.000

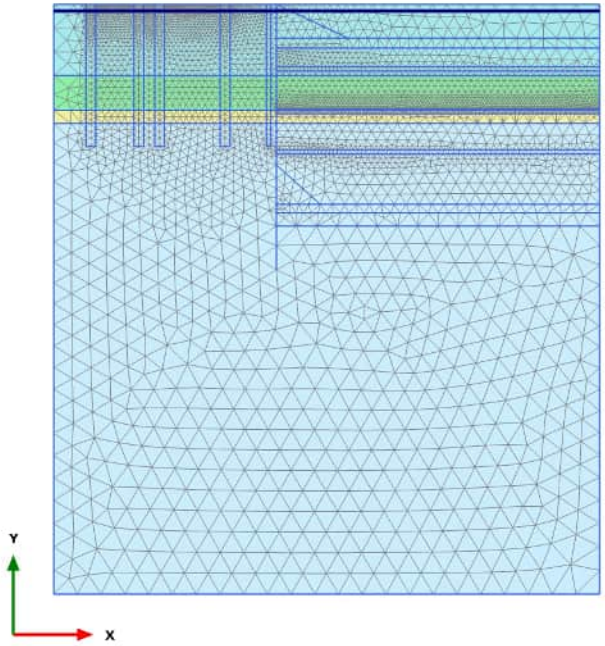
1.1.4 General information

General information	
Project	
Filename	M Social Hotel_2.p2dx
Directory	C:\Users\JennaPallarca\CMW Geosciences Pty Ltd\Auckland Office - AKL2025-0064 Downtown Carpark Redevelopment\Office Technical\Plaxis\RevA\
Title	M Social Hotel
General	
Model	Plane strain
Elements	15-Noded
Acceleration	
Gravity angle	-90.00 °
x-acceleration	0.000 m/s²
y-acceleration	0.000 m/s²
Earth gravity	9.810 m/s²
Mesh	
Nr of soil elements	9730
Nr of nodes	80199
Average element size	0.5929 m
Maximum element size	3.199 m
Minimum element size	0.05000 m

General information

Comments

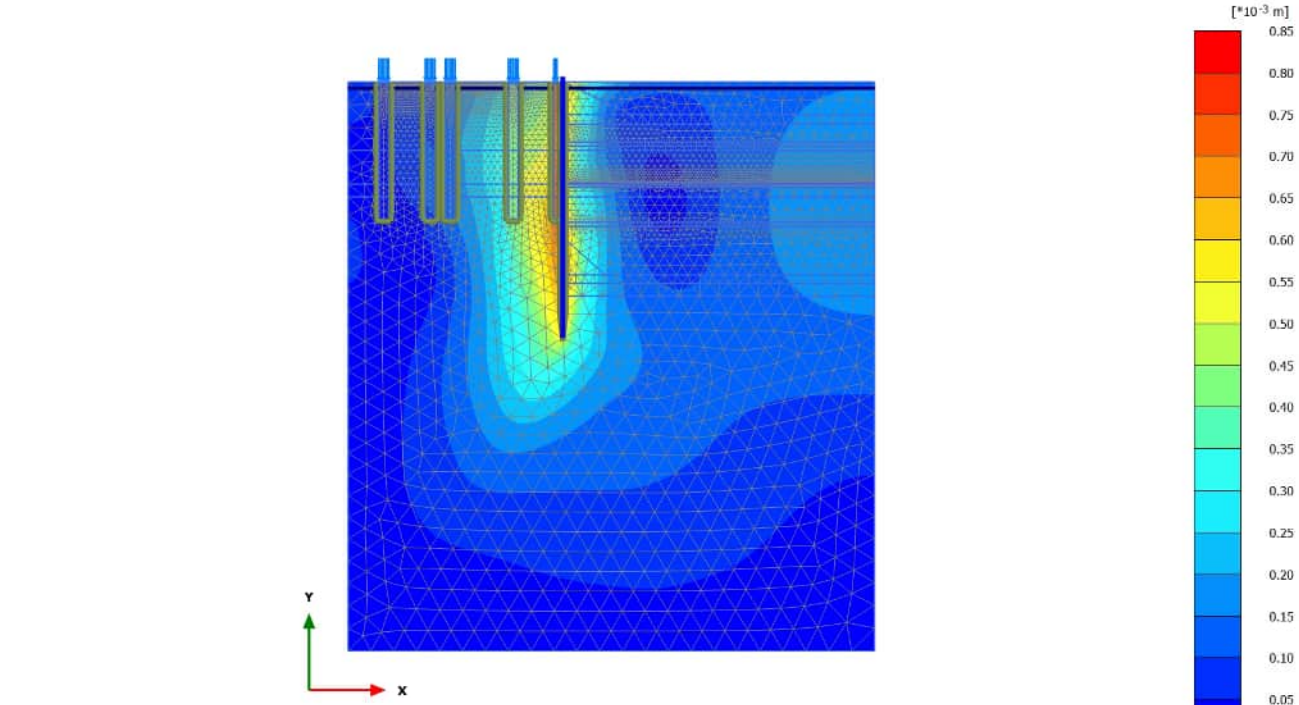
2.1.1.1.1 Calculation results, Initial phase [InitialPhase] (0/0), Total displacements |u|



Total displacements |u| (at true scale)

Uniform value of 0.000 m

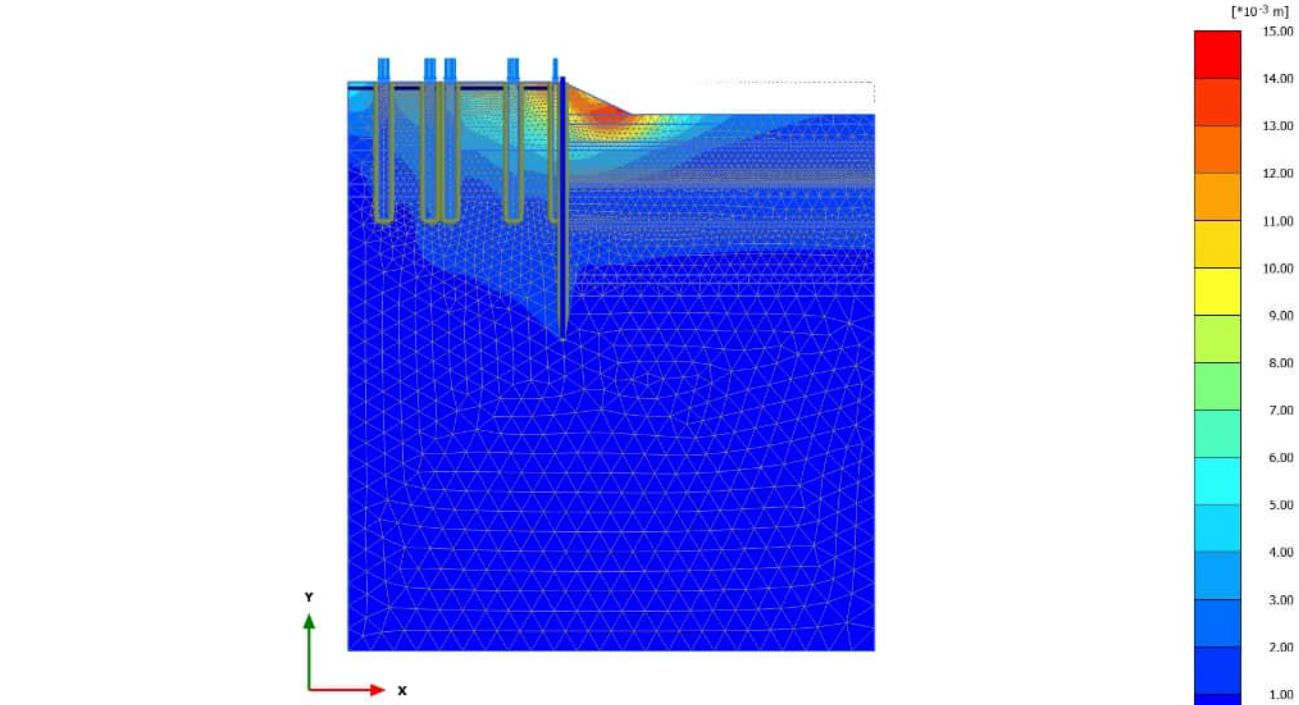
2.1.1.1.2 Calculation results, Phase_2 [Phase_2] (2/3), Total displacements |u|



Total displacements |u| (scaled up 2.00*10³ times)

Maximum value = 0.8233*10^{-3} m (Element 7256 at Node 44345)

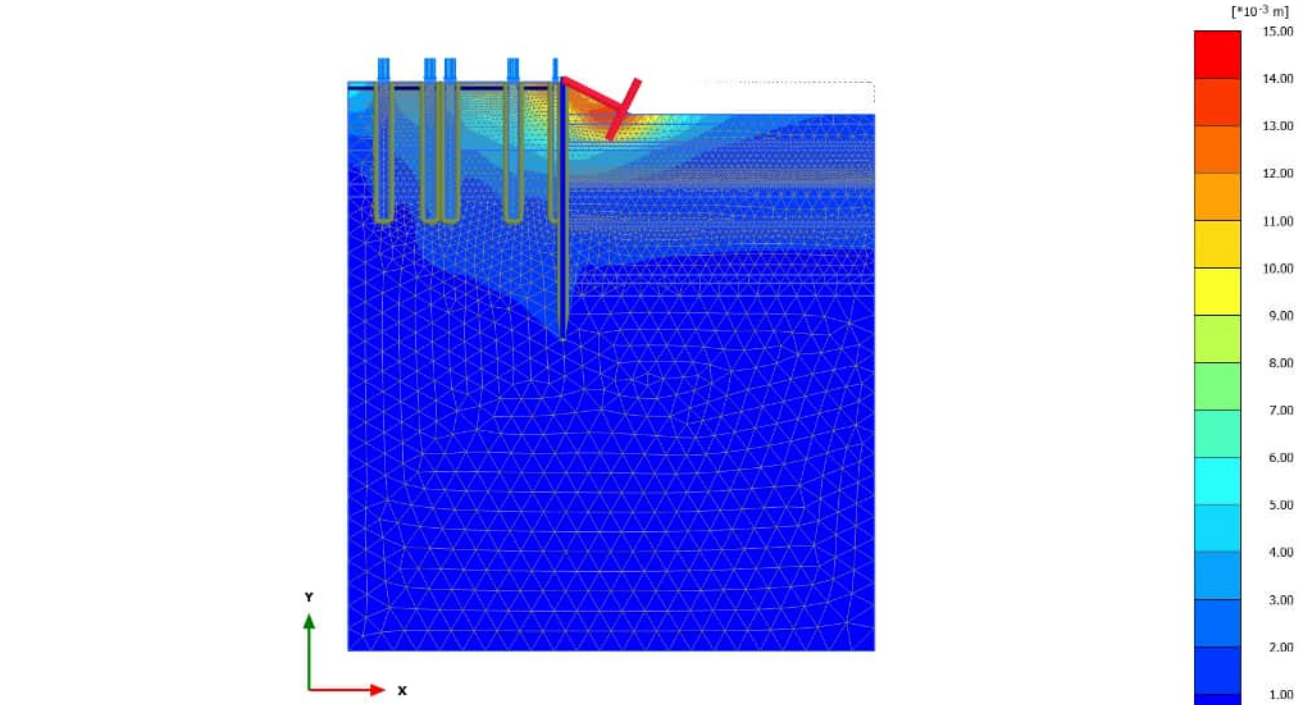
2.1.1.1.3 Calculation results, Phase_3 [Phase_3] (3/10), Total displacements |u|



Total displacements |u| (scaled up 200 times)

Maximum value = 0.01458 m (Element 150 at Node 17547)

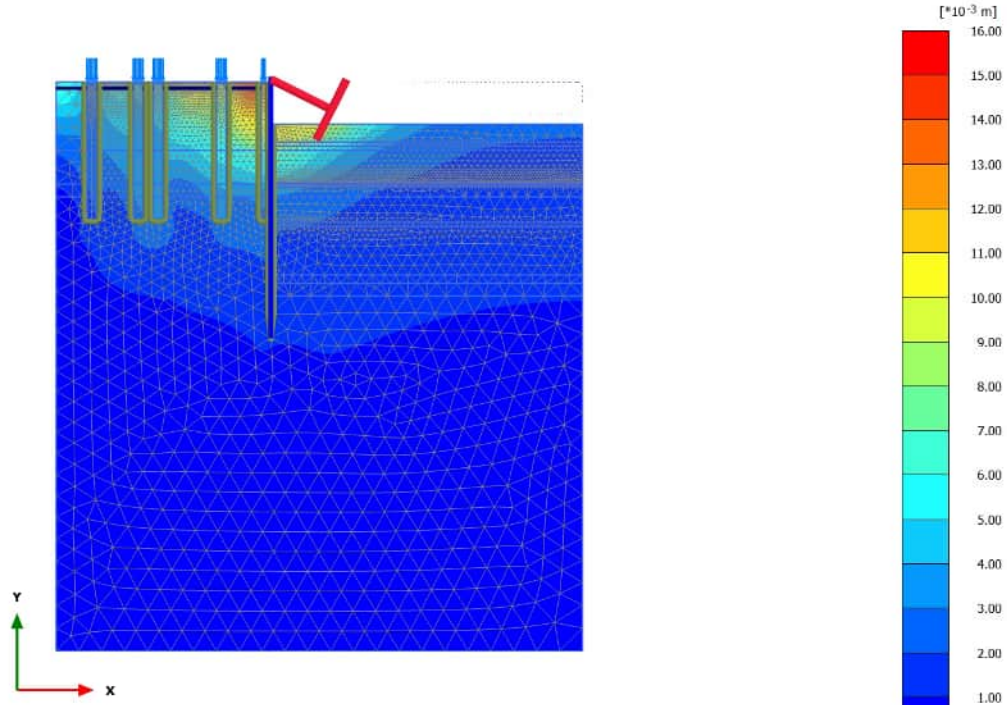
2.1.1.1.4 Calculation results, Phase_4 [Phase_4] (4/34), Total displacements |u|



Total displacements |u| (scaled up 200 times)

Maximum value = 0.01463 m (Element 150 at Node 17547)

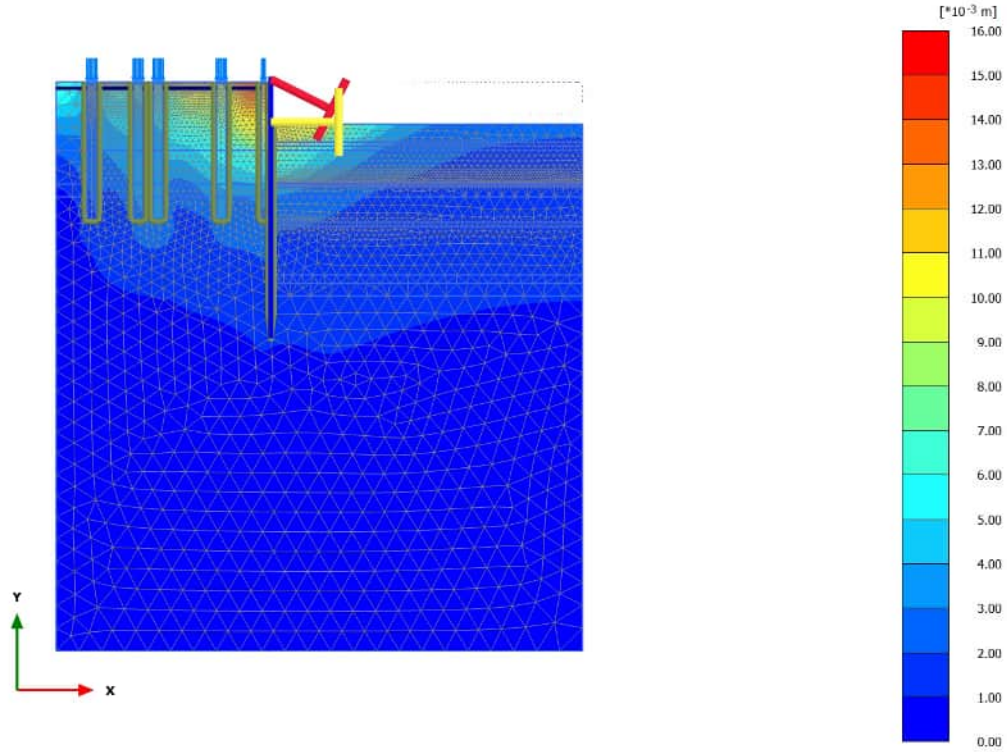
2.1.1.1.5 Calculation results, Phase_5 [Phase_5] (5/42), Total displacements |u|



Total displacements |u| (scaled up 200 times)

Maximum value = 0.01539 m (Element 2772 at Node 25811)

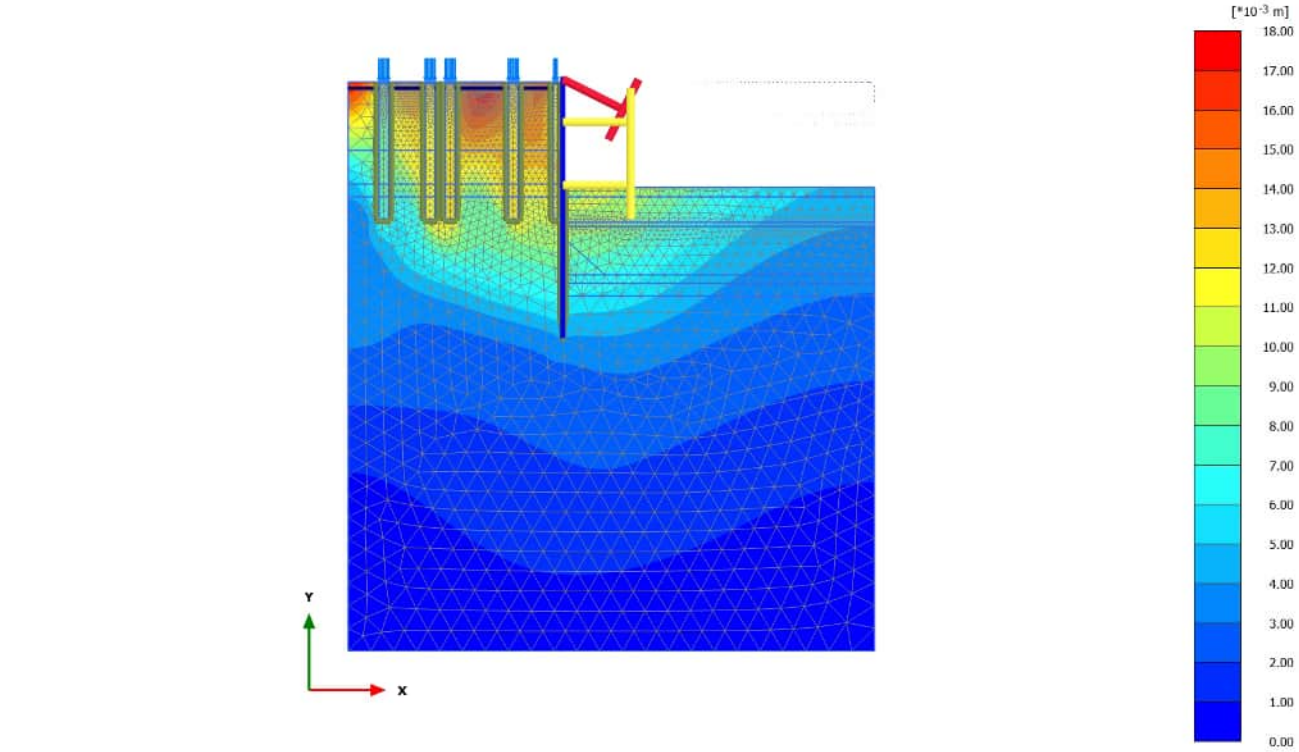
2.1.1.1.6 Calculation results, Phase_6 [Phase_6] (6/89), Total displacements |u|



Total displacements |u| (scaled up 200 times)

Maximum value = 0.01539 m (Element 2772 at Node 25811)

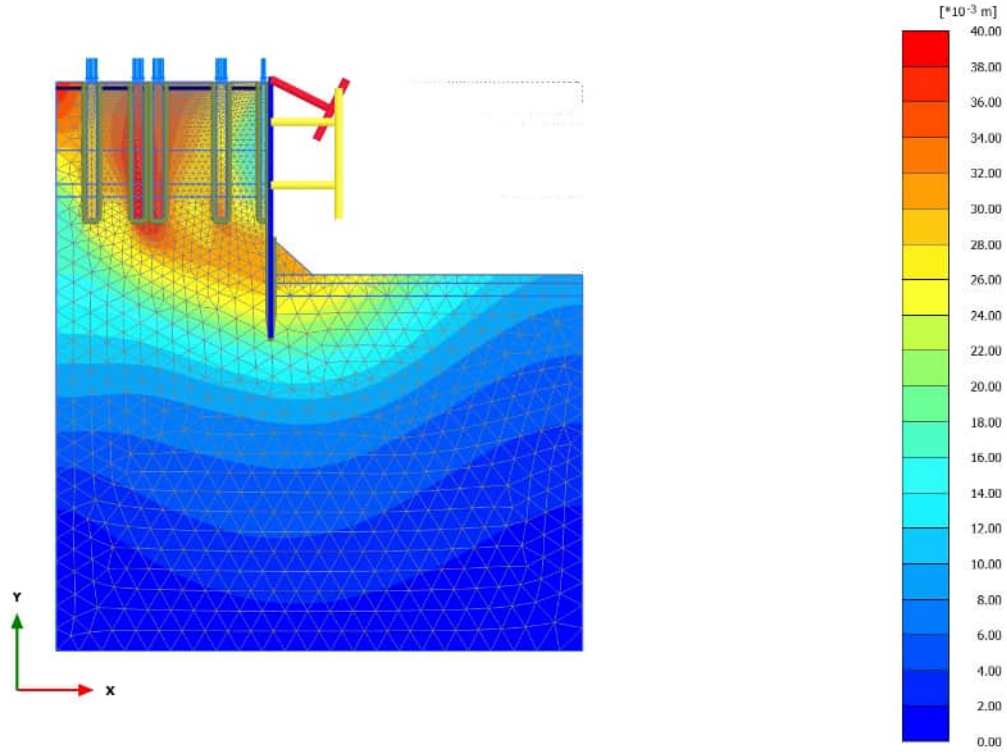
2.1.1.1.7 Calculation results, Phase_8 [Phase_8] (8/91), Total displacements |u|



Total displacements |u| (scaled up 100 times)

Maximum value = 0.01798 m (Element 2193 at Node 47354)

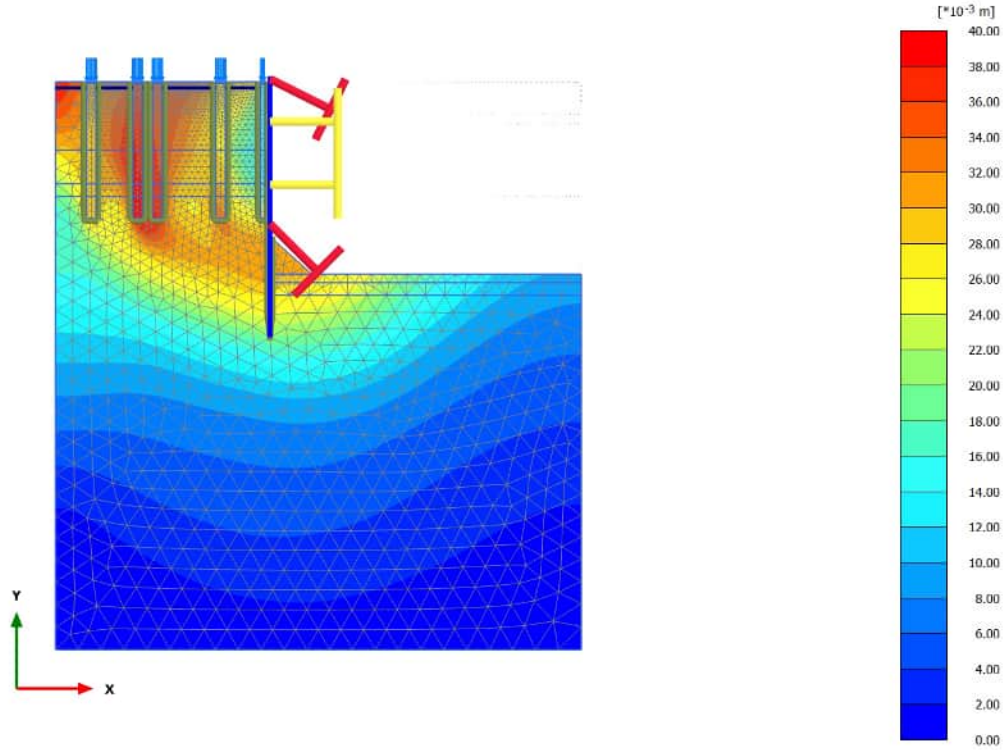
2.1.1.1.8 Calculation results, Phase_9 [Phase_9] (9/166), Total displacements |u|



Total displacements |u| (scaled up 50.0 times)

Maximum value = 0.03974 m (Element 7121 at Node 64675)

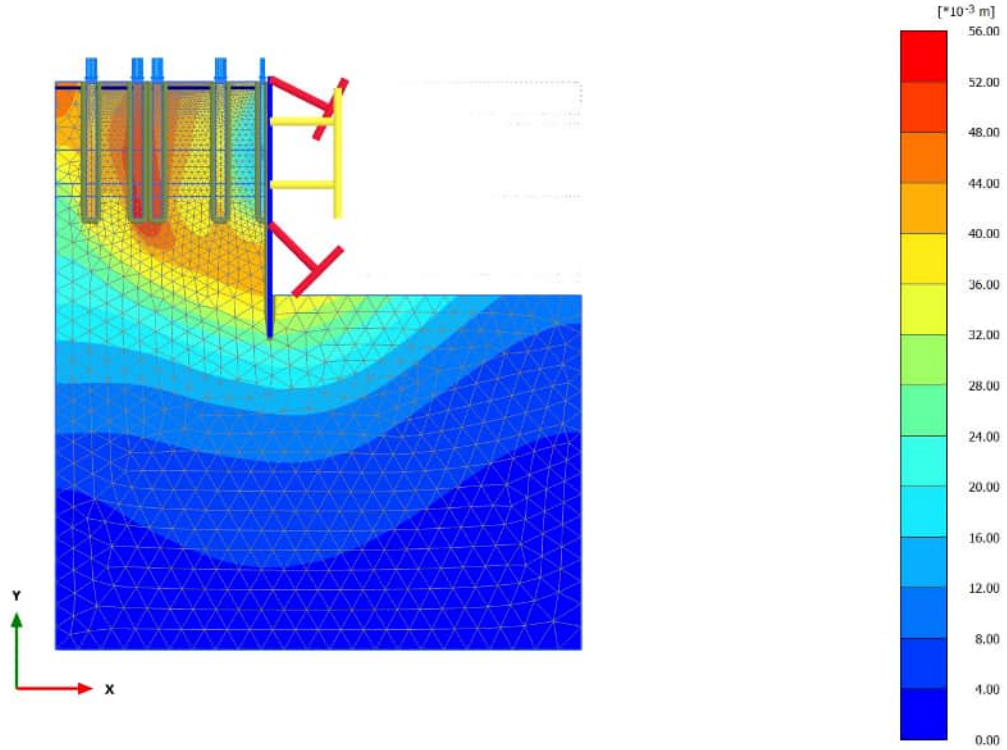
2.1.1.1.9 Calculation results, Phase_10 [Phase_10] (10/168), Total displacements |u|



Total displacements |u| (scaled up 50.0 times)

Maximum value = 0.03974 m (Element 7121 at Node 64675)

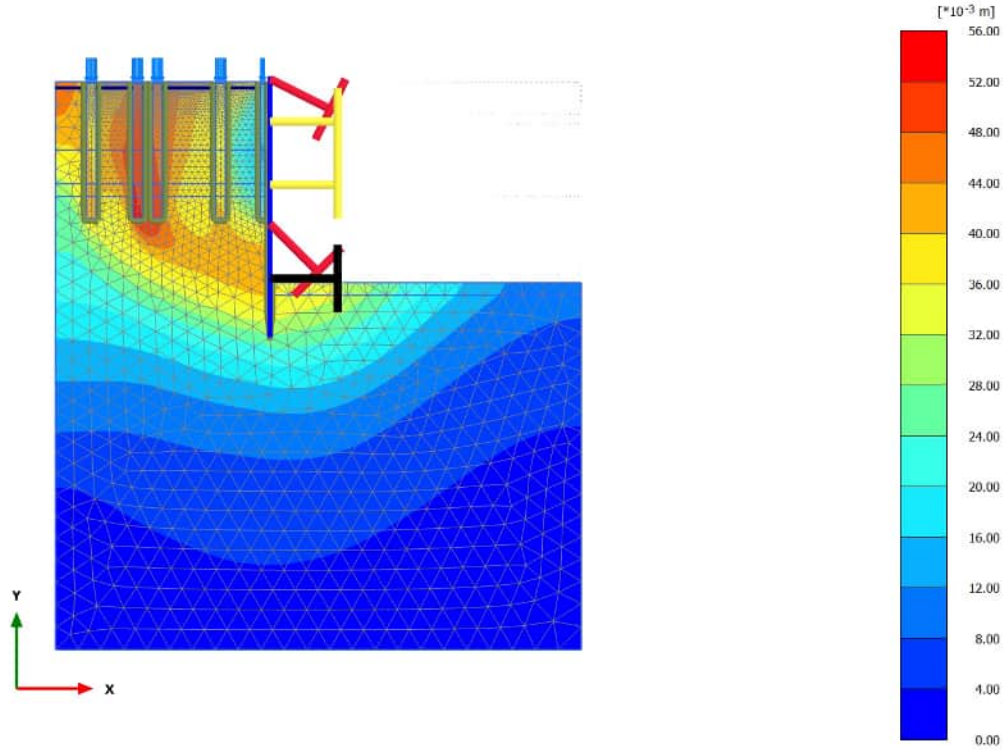
2.1.1.1.10 Calculation results, Phase_11 [Phase_11] (11/218), Total displacements |u|



Total displacements |u| (scaled up 50.0 times)

Maximum value = 0.05357 m (Element 7121 at Node 64675)

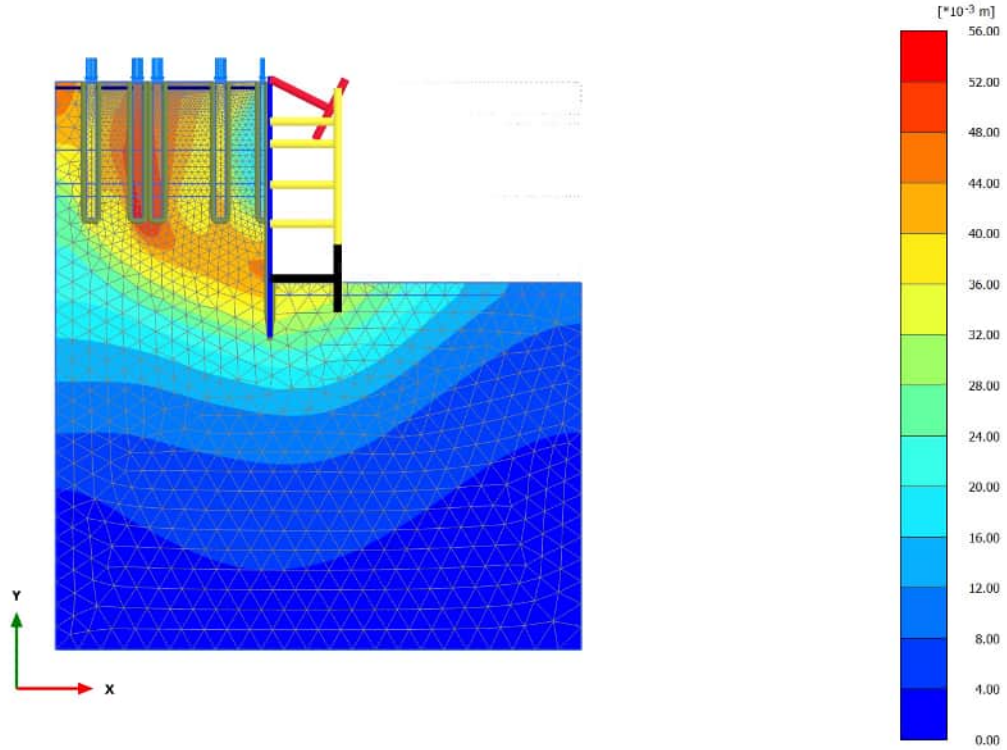
2.1.1.1.11 Calculation results, Phase_12 [Phase_12] (12/221), Total displacements |u|



Total displacements |u| (scaled up 50.0 times)

Maximum value = 0.05337 m (Element 7121 at Node 64675)

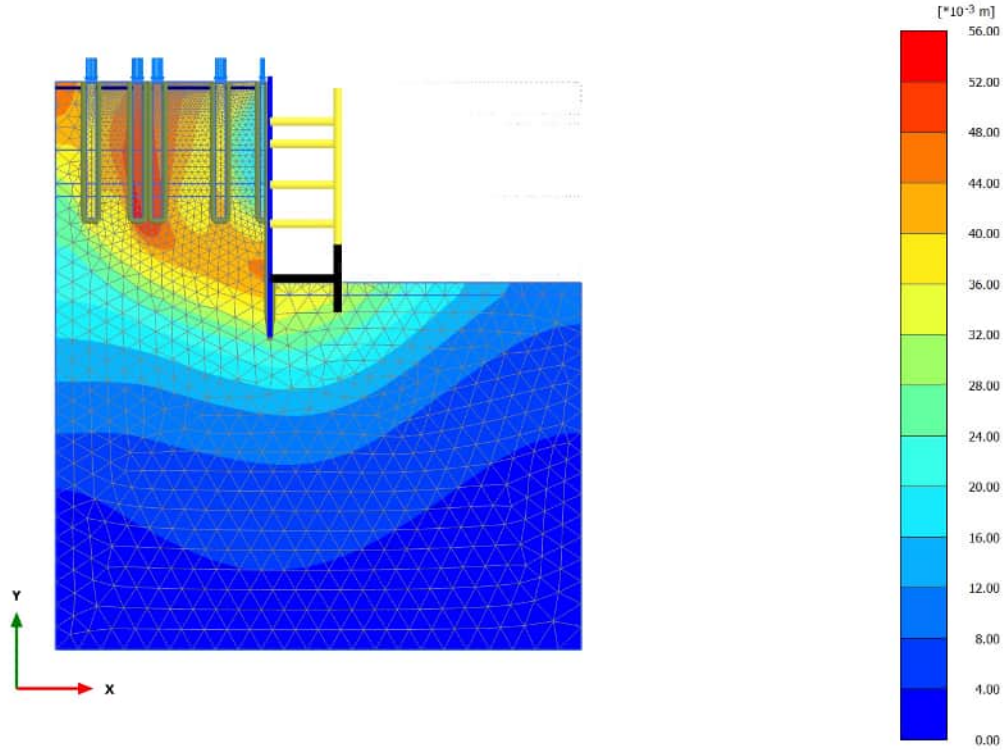
2.1.1.1.12 Calculation results, Phase_13 [Phase_13] (13/226), Total displacements |u|



Total displacements |u| (scaled up 50.0 times)

Maximum value = 0.05341 m (Element 7121 at Node 64675)

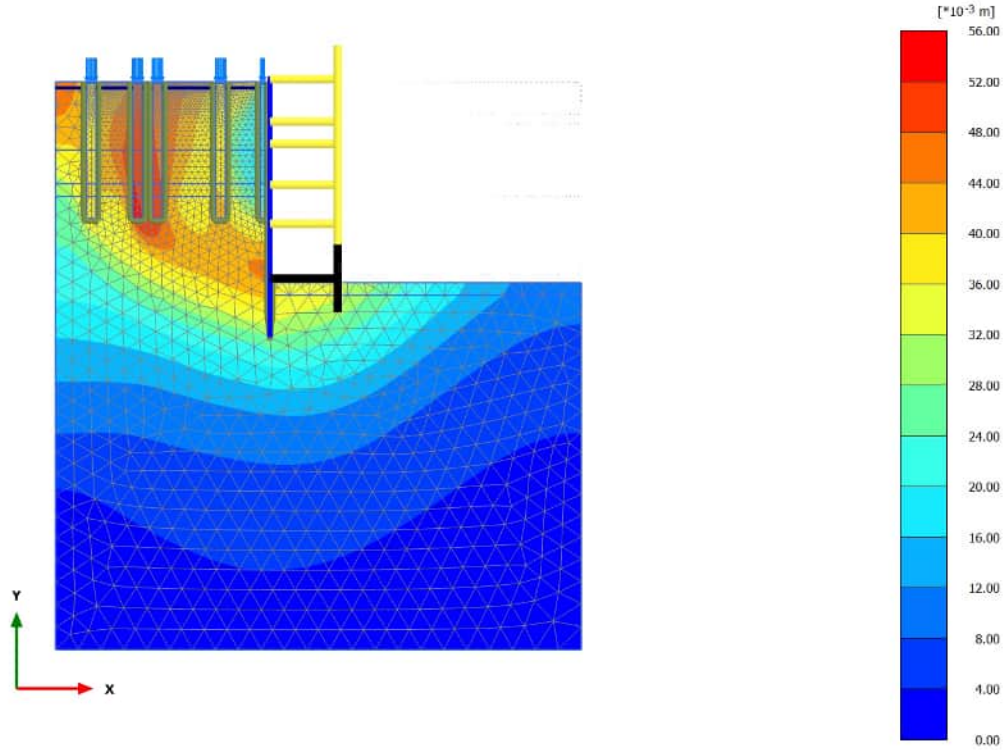
2.1.1.1.13 Calculation results, Phase_14 [Phase_14] (14/229), Total displacements |u|



Total displacements |u| (scaled up 50.0 times)

Maximum value = 0.05343 m (Element 7121 at Node 64675)

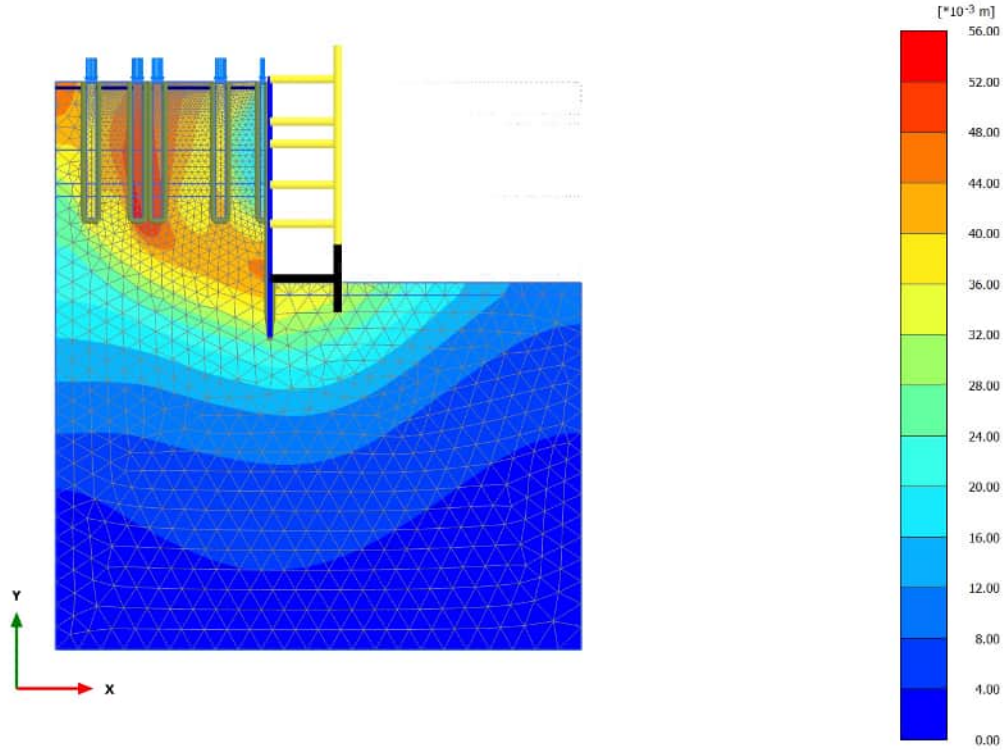
2.1.1.1.14 Calculation results, Phase_15 [Phase_15] (15/245), Total displacements |u|



Total displacements |u| (scaled up 50.0 times)

Maximum value = 0.05343 m (Element 7121 at Node 64675)

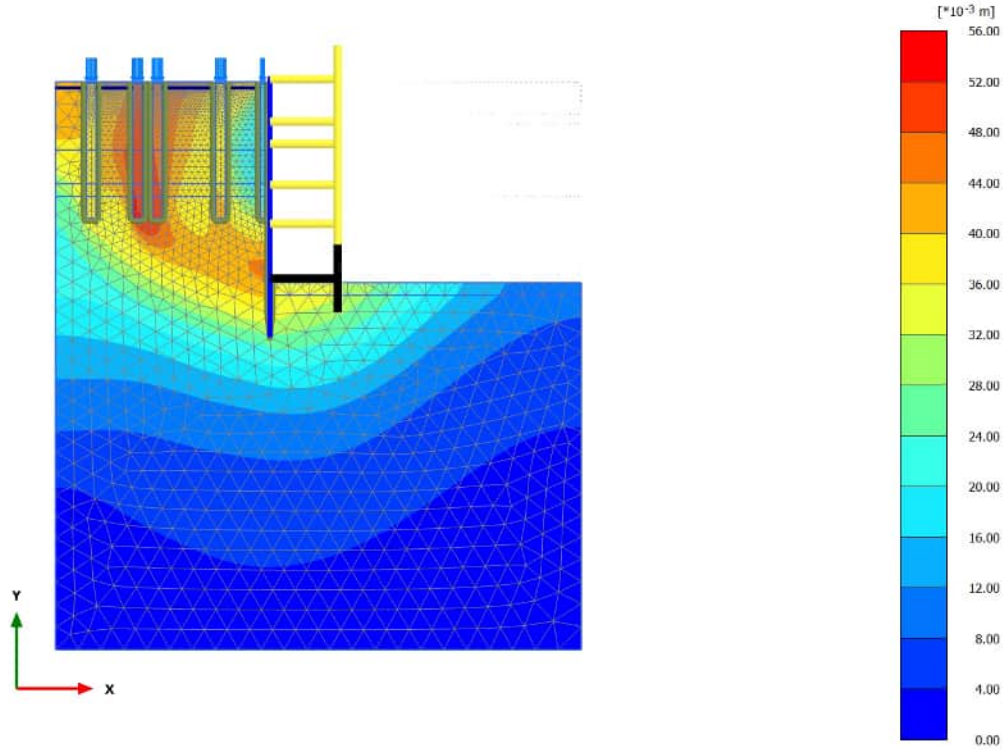
2.1.1.1.15 Calculation results, Phase_16 [Phase_16] (16/248), Total displacements |u|



Total displacements |u| (scaled up 50.0 times)

Maximum value = 0.05343 m (Element 7121 at Node 64675)

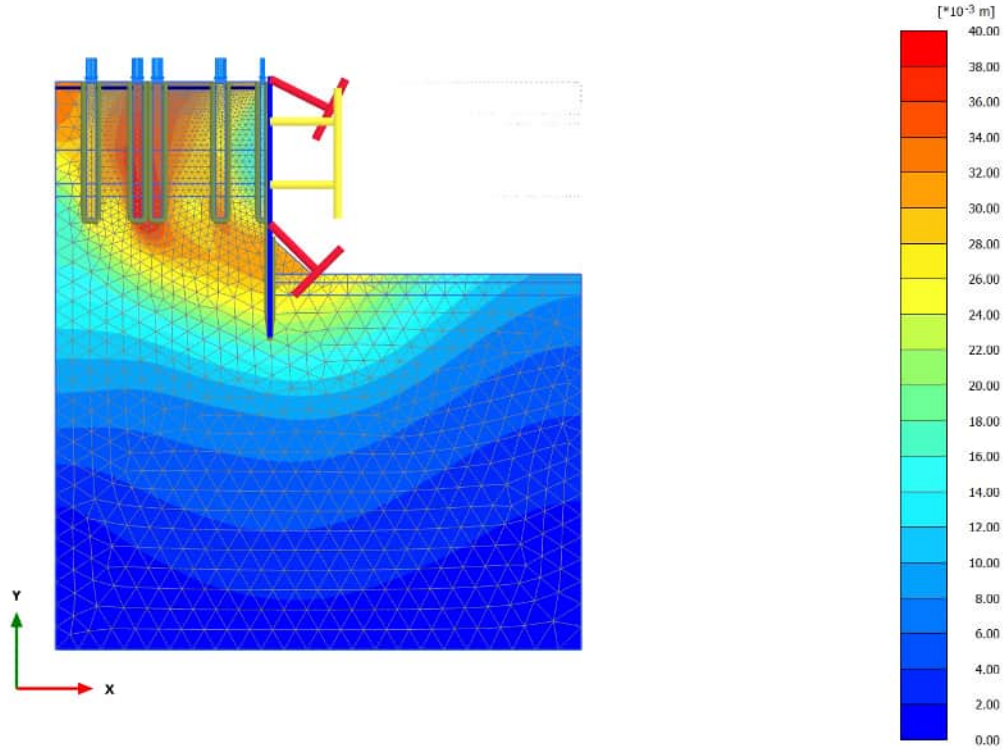
2.1.1.1.16 Calculation results, Phase_17 [Phase_17] (17/253), Total displacements |u|



Total displacements |u| (scaled up 50.0 times)

Maximum value = 0.05317 m (Element 7121 at Node 64675)

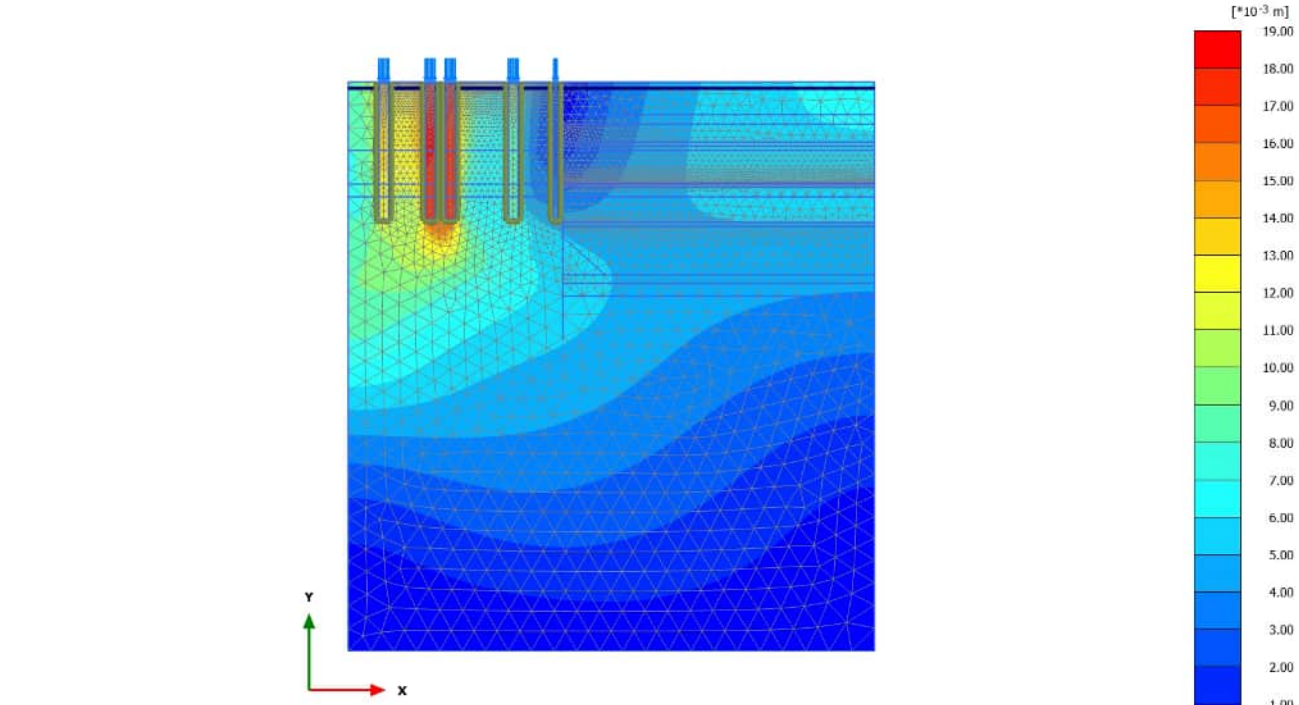
2.1.1.1.17 Calculation results, Phase_18 [Phase_18] (18/258), Total displacements |u|



Total displacements |u| (scaled up 50.0 times)

Maximum value = 0.03945 m (Element 7121 at Node 64675)

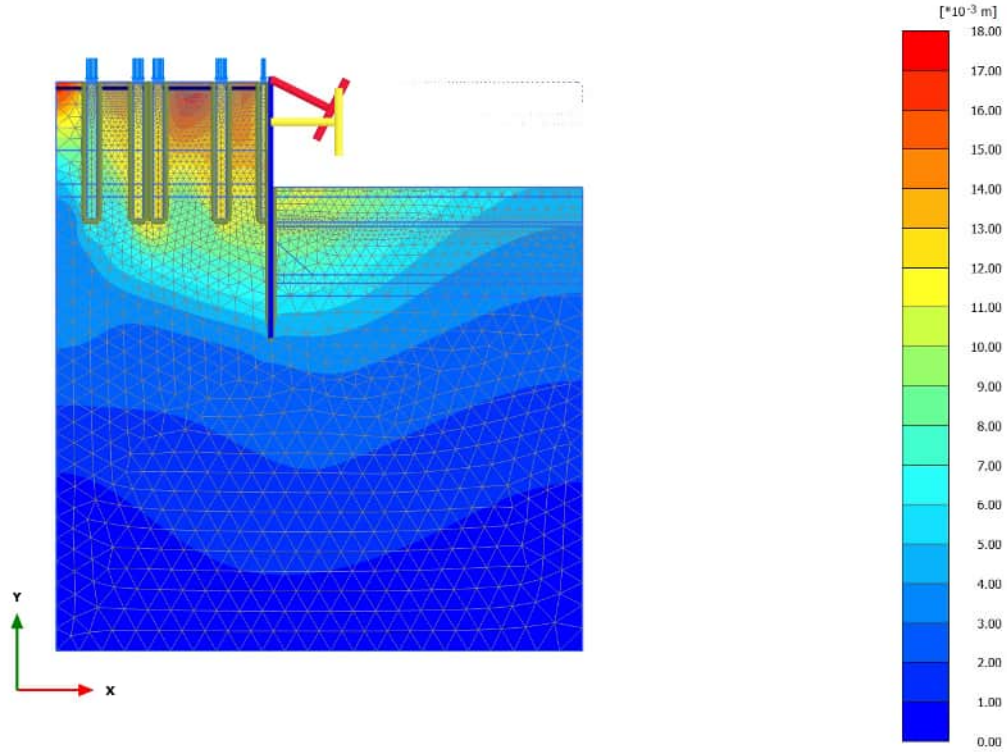
2.1.1.1.18 Calculation results, Phase_1 [Phase_1] (1/310), Total displacements |u|



Total displacements |u| (scaled up 100 times)

Maximum value = 0.01891 m (Element 1024 at Node 73114)

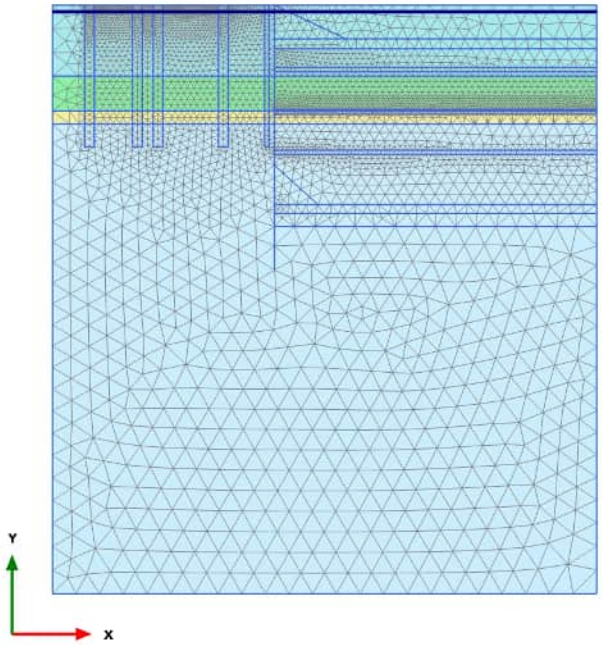
2.1.1.1.19 Calculation results, Phase_7 [Phase_7] (7/339), Total displacements |u|



Total displacements |u| (scaled up 100 times)

Maximum value = 0.01798 m (Element 2193 at Node 47354)

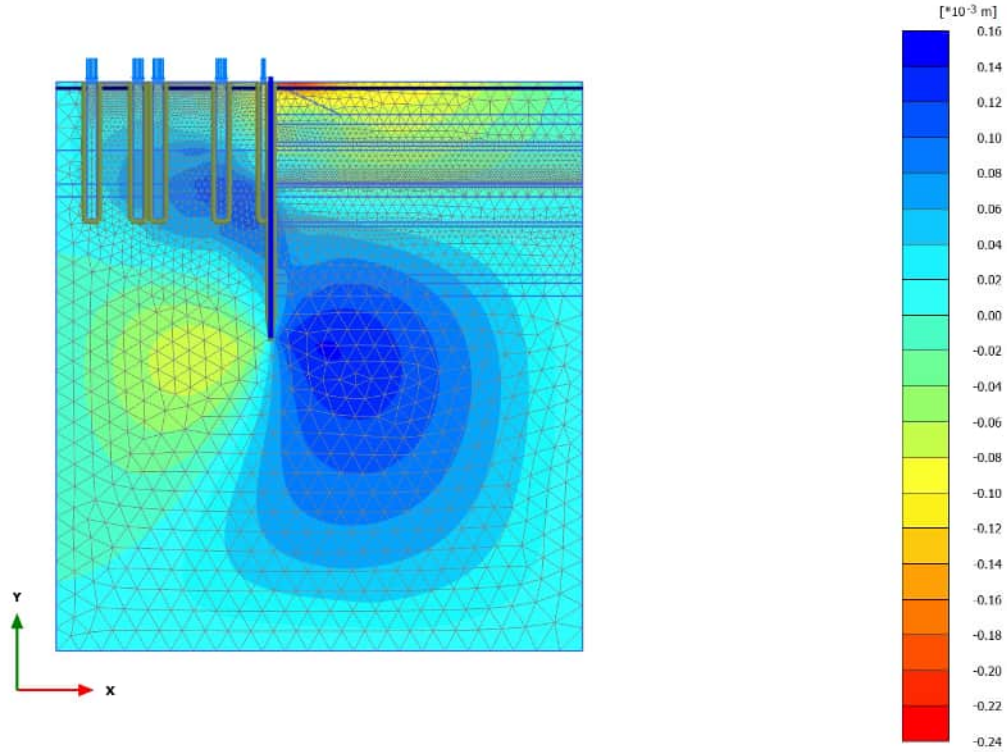
2.1.1.2.1 Calculation results, Initial phase [InitialPhase] (0/0), Total displacements u_x



Total displacements u_x (at true scale)

Uniform value of 0.000 m

2.1.1.2.2 Calculation results, Phase_2 [Phase_2] (2/3), Total displacements u_x

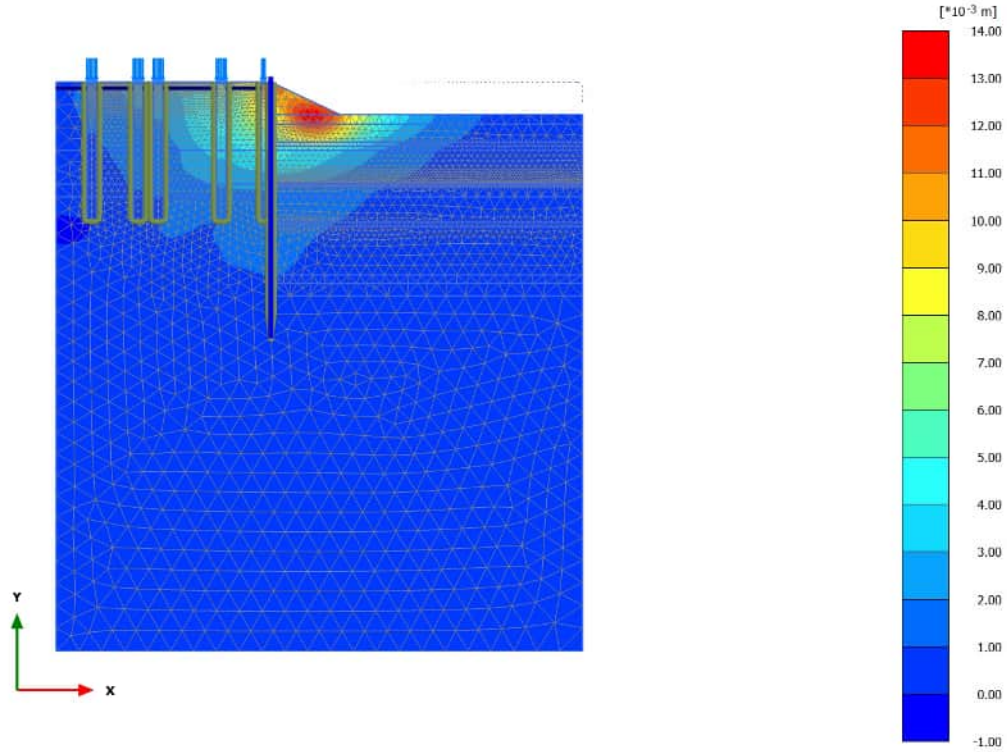


Total displacements u_x (scaled up 5.00*10³ times)

Maximum value = 0.1414*10^{-3} m (Element 9066 at Node 55495)

Minimum value = -0.2373*10^{-3} m (Element 137 at Node 23690)

2.1.1.2.3 Calculation results, Phase_3 [Phase_3] (3/10), Total displacements u_x

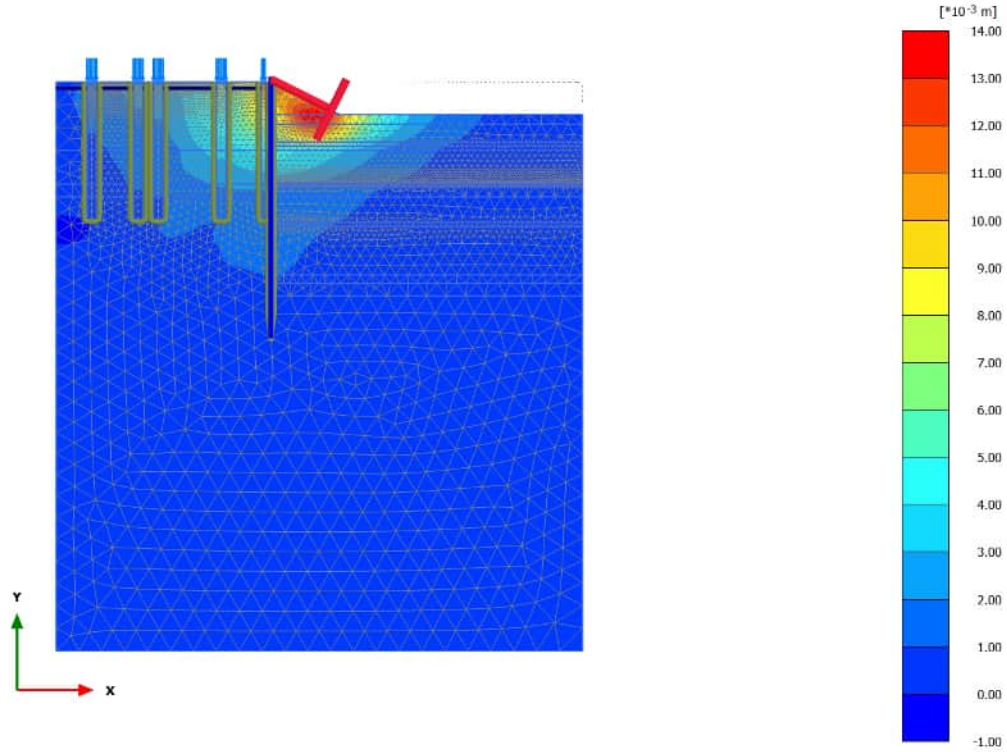


Total displacements u_x (scaled up 200 times)

Maximum value = 0.01365 m (Element 194 at Node 25703)

Minimum value = -0.09244*10-3 m (Element 9240 at Node 71354)

2.1.1.2.4 Calculation results, Phase_4 [Phase_4] (4/34), Total displacements u_x

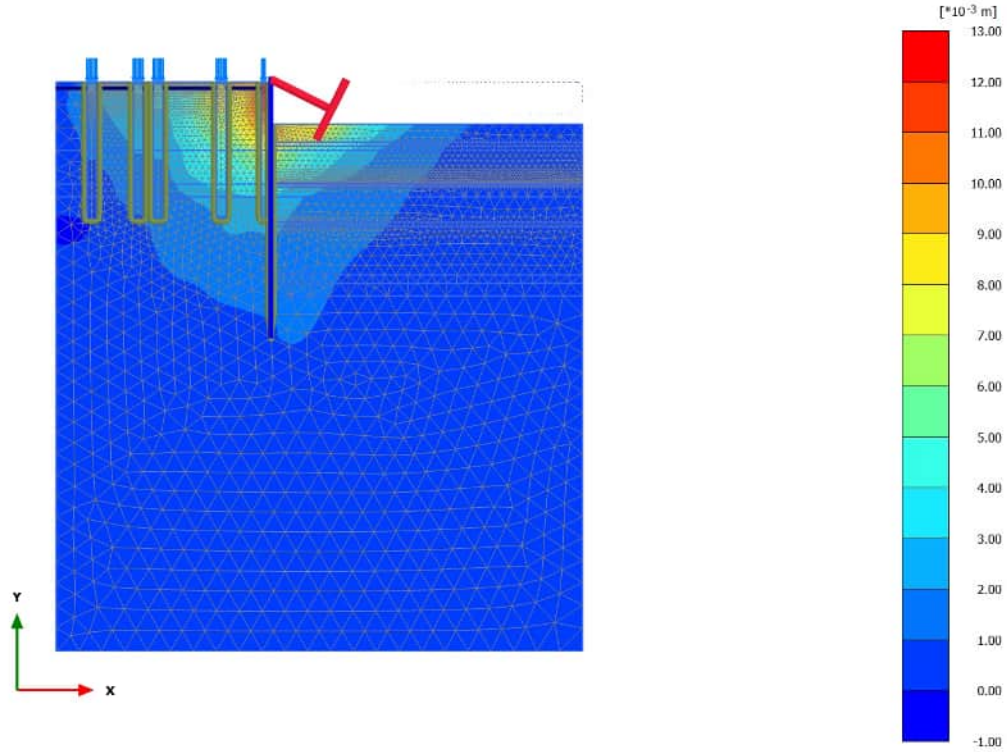


Total displacements u_x (scaled up 200 times)

Maximum value = 0.01388 m (Element 194 at Node 25703)

Minimum value = -0.09233*10^{-3} m (Element 9240 at Node 71354)

2.1.1.2.5 Calculation results, Phase_5 [Phase_5] (5/42), Total displacements u_x

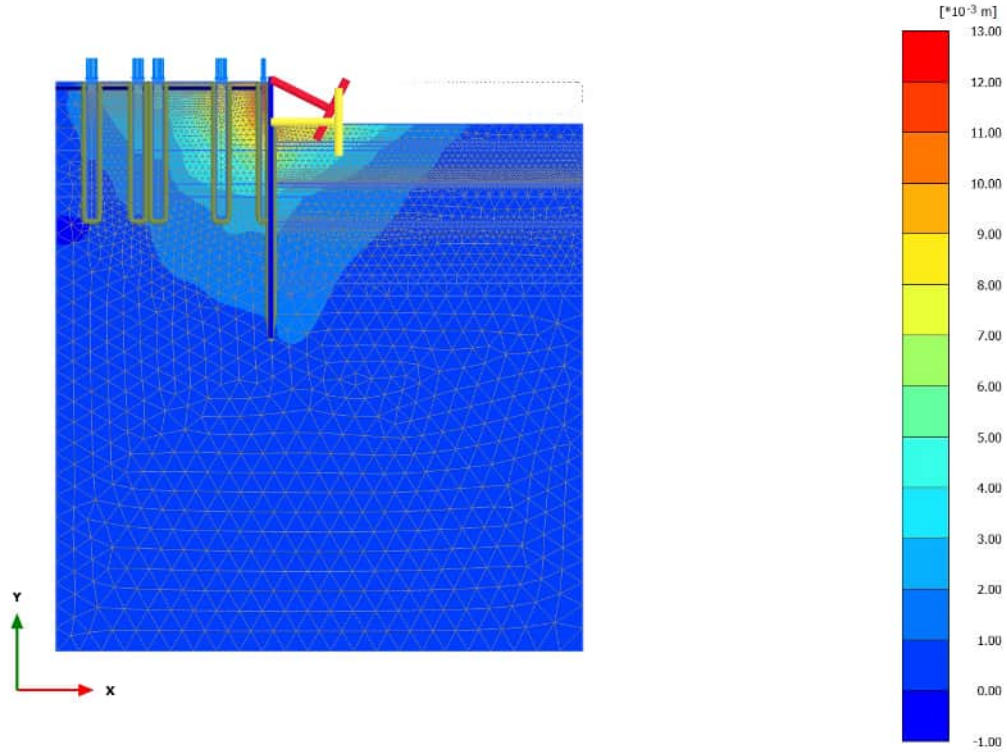


Total displacements u_x (scaled up 200 times)

Maximum value = 0.01283 m (Element 2996 at Node 25733)

Minimum value = -0.1233*10-3 m (Element 9240 at Node 71354)

2.1.1.2.6 Calculation results, Phase_6 [Phase_6] (6/89), Total displacements u_x

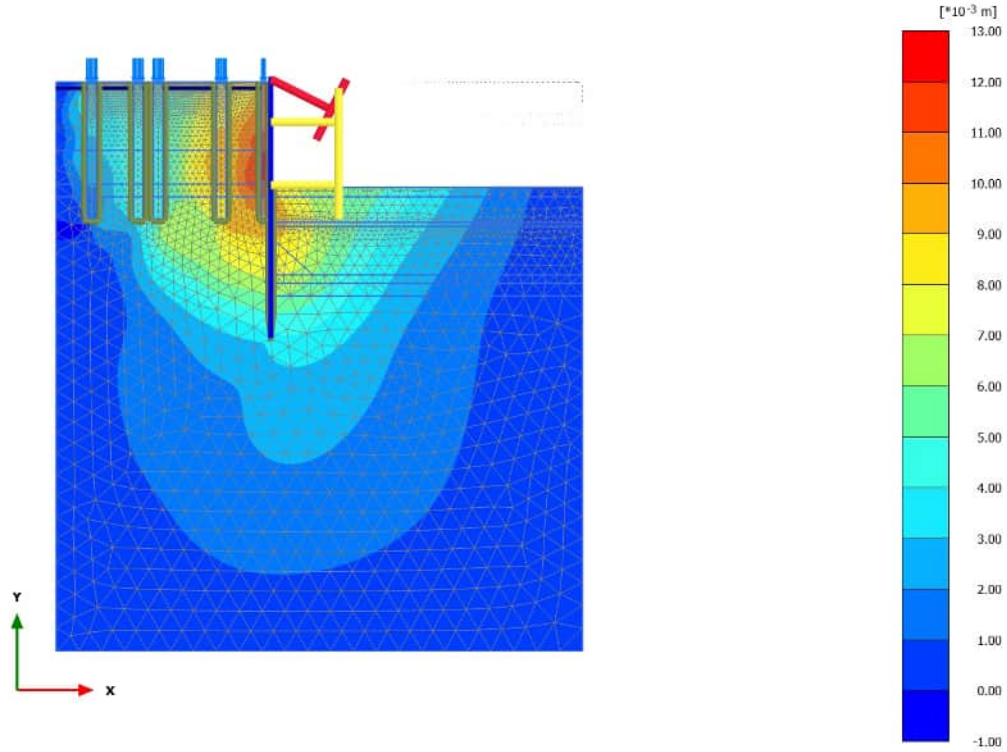


Total displacements u_x (scaled up 200 times)

Maximum value = 0.01283 m (Element 2996 at Node 25733)

Minimum value = $-0.1230*10^{-3}$ m (Element 9240 at Node 71354)

2.1.1.2.7 Calculation results, Phase_8 [Phase_8] (8/91), Total displacements u_x

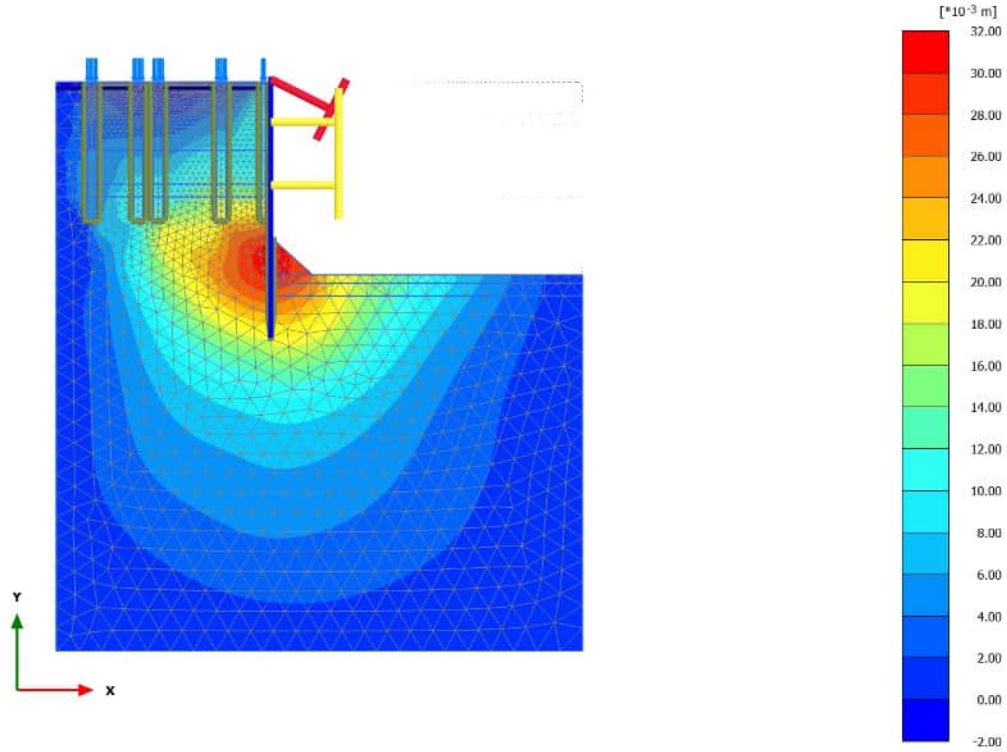


Total displacements u_x (scaled up 200 times)

Maximum value = 0.01240 m (Element 5957 at Node 35563)

Minimum value = -0.07566*10^{-3} m (Element 9240 at Node 71353)

2.1.1.2.8 Calculation results, Phase_9 [Phase_9] (9/166), Total displacements u_x

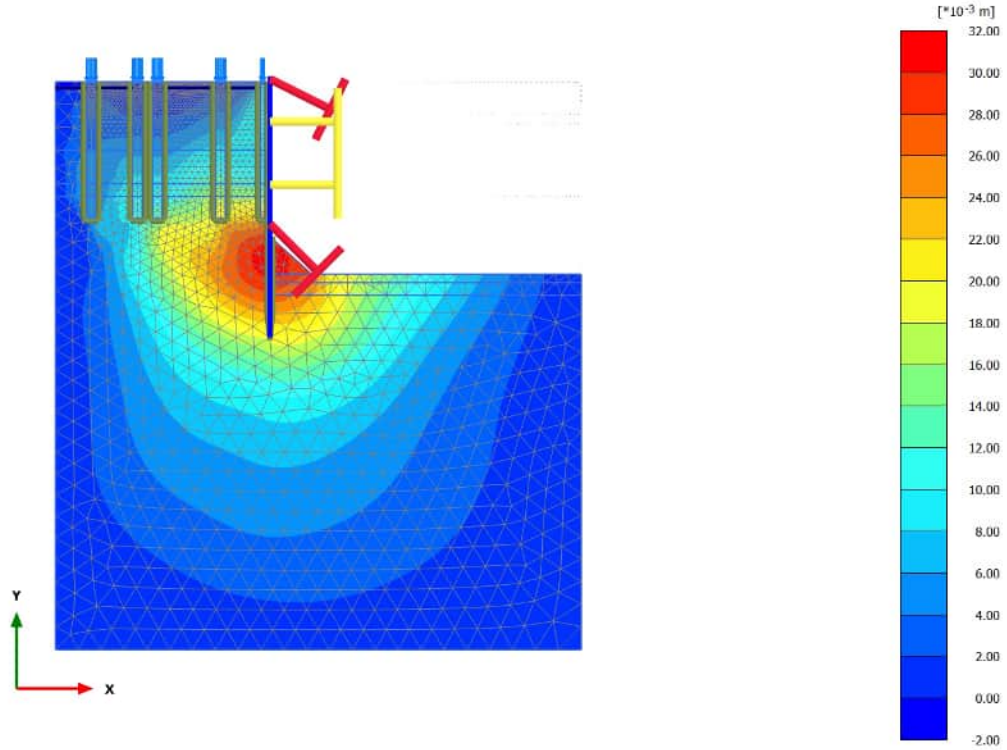


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.03110 m (Element 8108 at Node 55519)

Minimum value = -1.550*10-3 m (Element 2434 at Node 60188)

2.1.1.2.9 Calculation results, Phase_10 [Phase_10] (10/168), Total displacements u_x

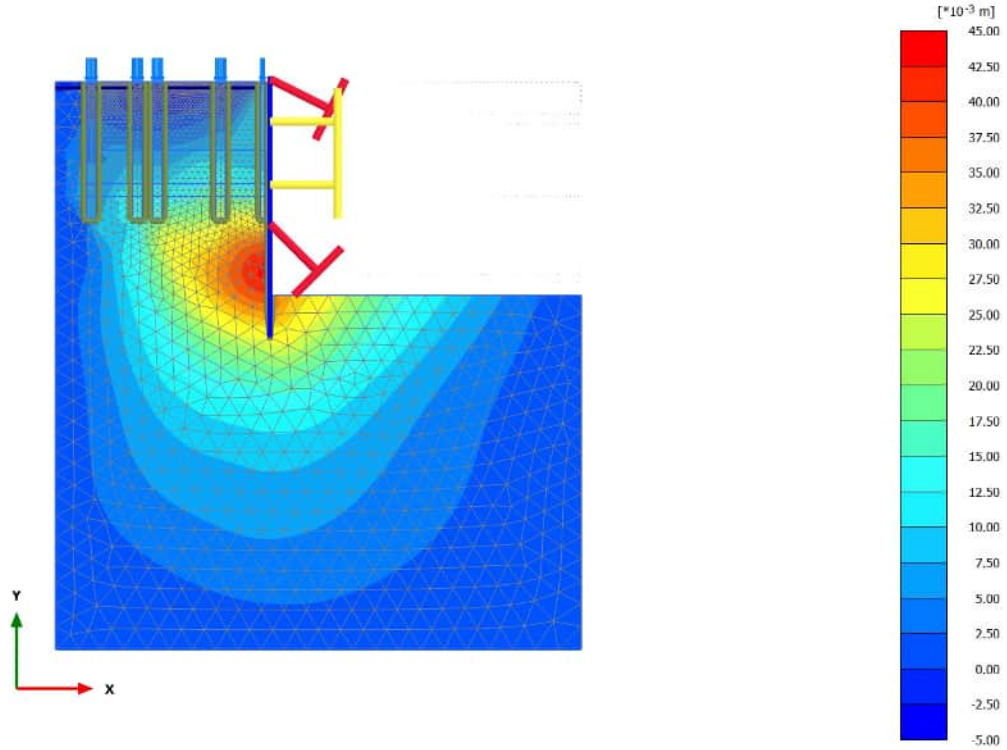


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.03110 m (Element 8108 at Node 55519)

Minimum value = -1.551*10-3 m (Element 2434 at Node 60188)

2.1.1.2.10 Calculation results, Phase_11 [Phase_11] (11/218), Total displacements u_x

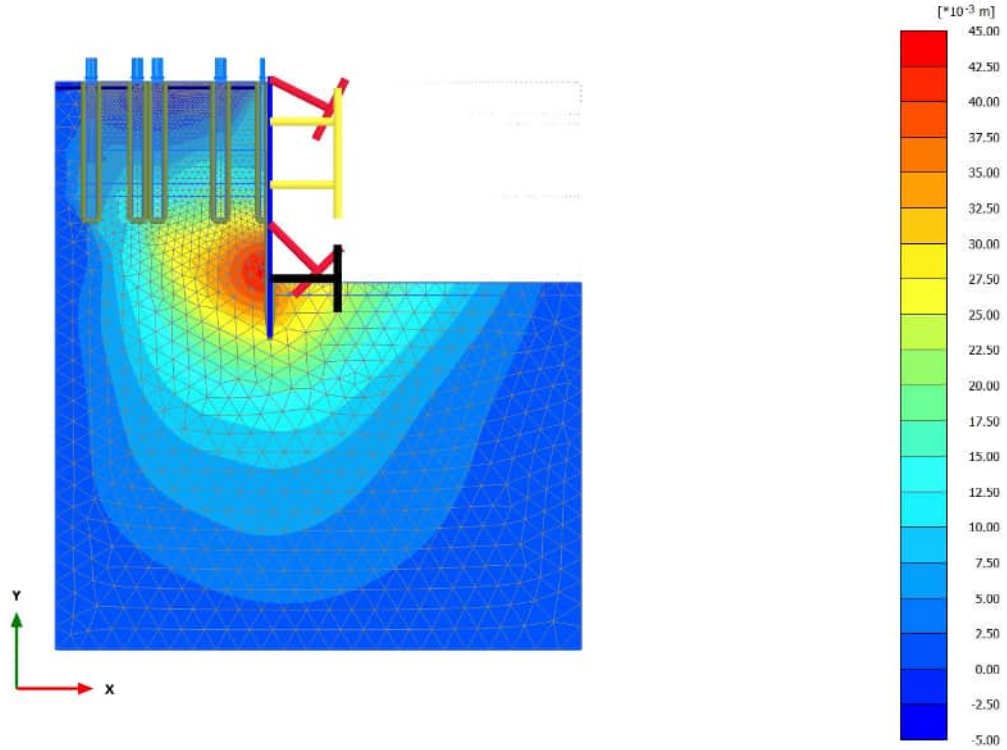


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.04331 m (Element 9374 at Node 57108)

Minimum value = $-4.828*10^{-3}$ m (Element 2414 at Node 35332)

2.1.1.2.11 Calculation results, Phase_12 [Phase_12] (12/221), Total displacements u_x

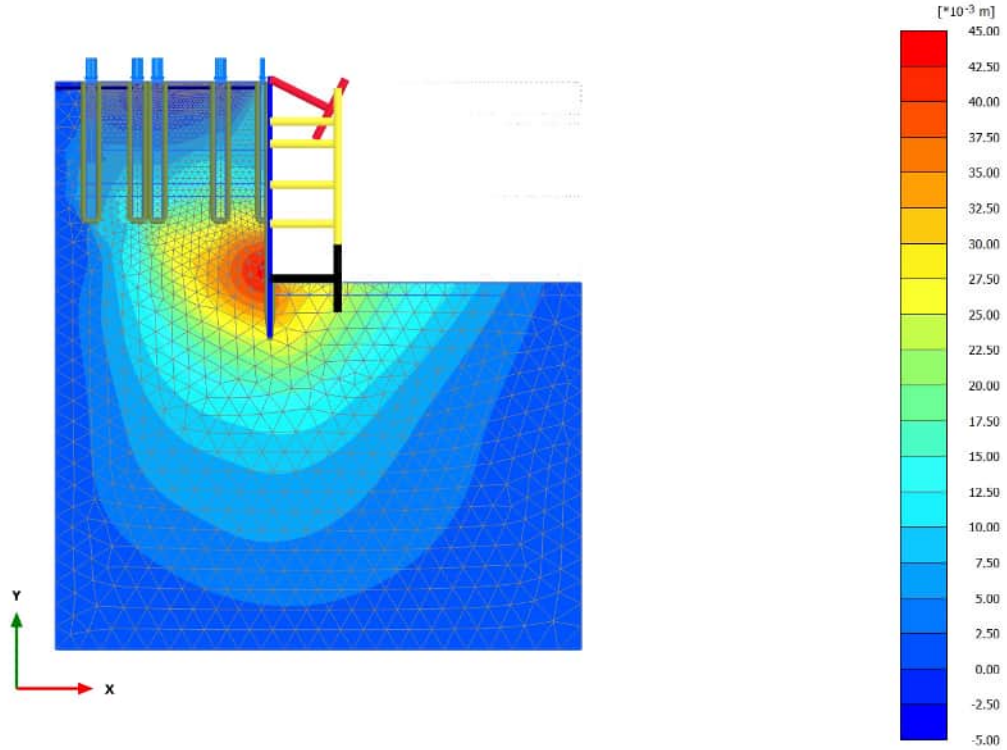


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.04333 m (Element 9374 at Node 57108)

Minimum value = $-4.804*10^{-3}$ m (Element 2414 at Node 35332)

2.1.1.2.12 Calculation results, Phase_13 [Phase_13] (13/226), Total displacements u_x

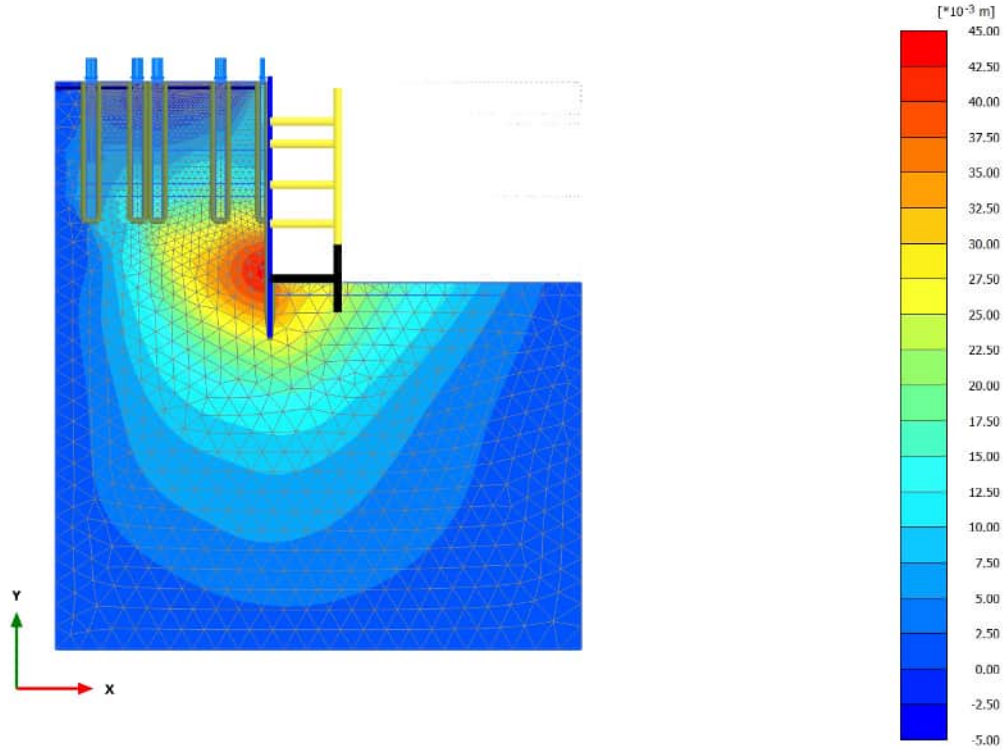


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.04341 m (Element 9374 at Node 57108)

Minimum value = $-4.636*10^{-3}$ m (Element 2435 at Node 59307)

2.1.1.2.13 Calculation results, Phase_14 [Phase_14] (14/229), Total displacements u_x

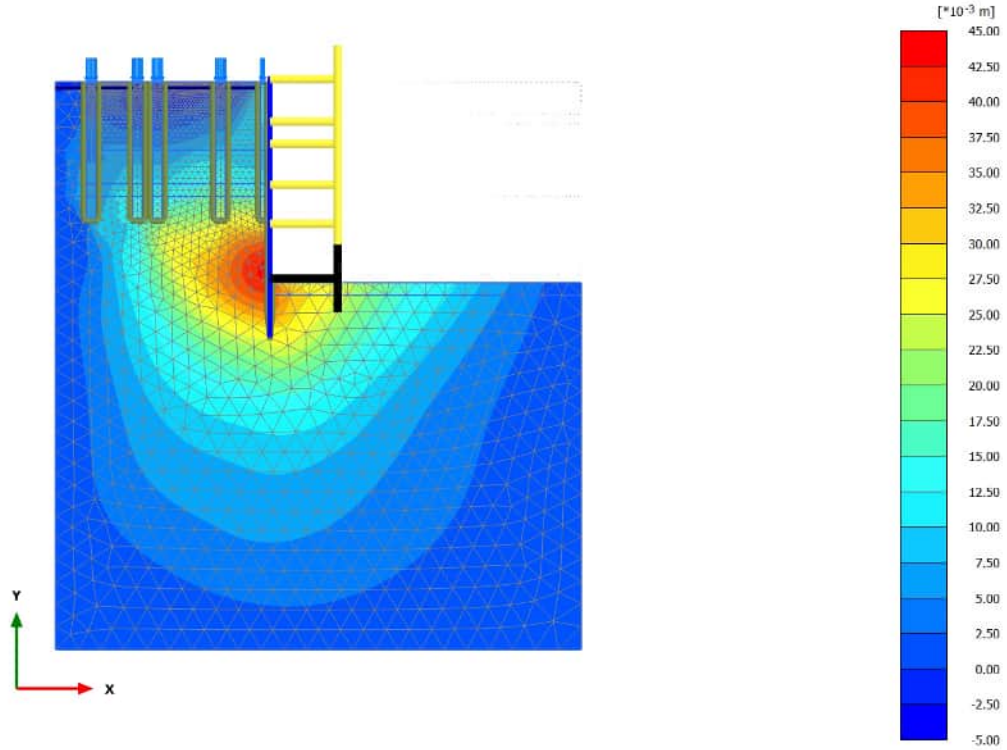


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.04341 m (Element 9374 at Node 57108)

Minimum value = $-4.416*10^{-3}$ m (Element 2435 at Node 59307)

2.1.1.2.14 Calculation results, Phase_15 [Phase_15] (15/245), Total displacements u_x

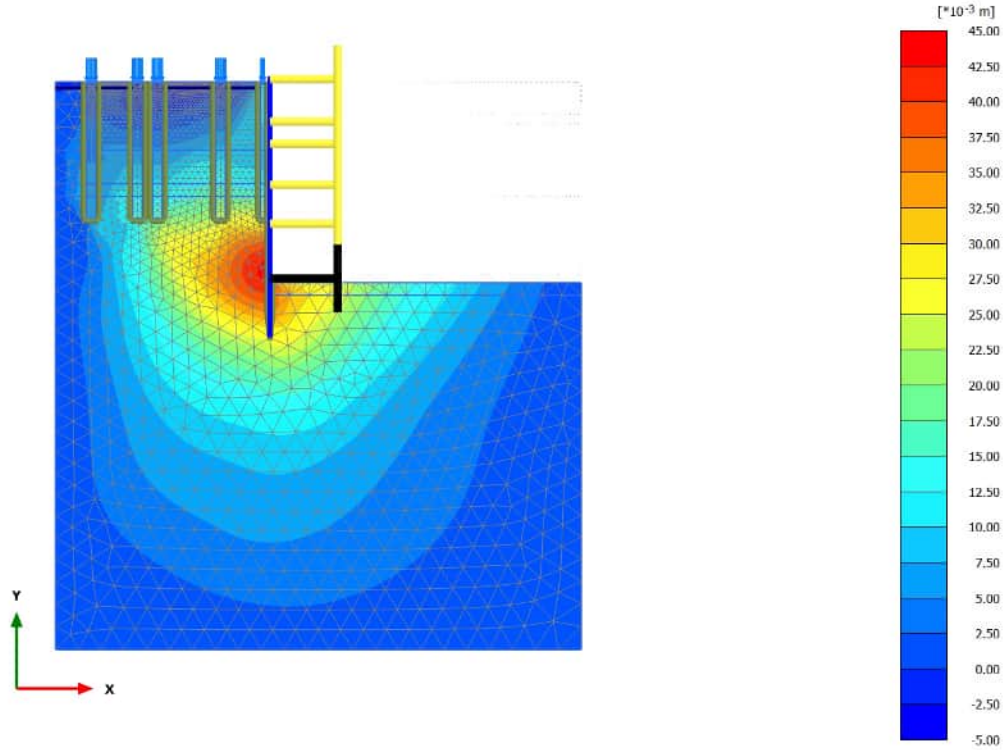


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.04341 m (Element 9374 at Node 57108)

Minimum value = $-4.419*10^{-3}$ m (Element 2435 at Node 59307)

2.1.1.2.15 Calculation results, Phase_16 [Phase_16] (16/248), Total displacements u_x

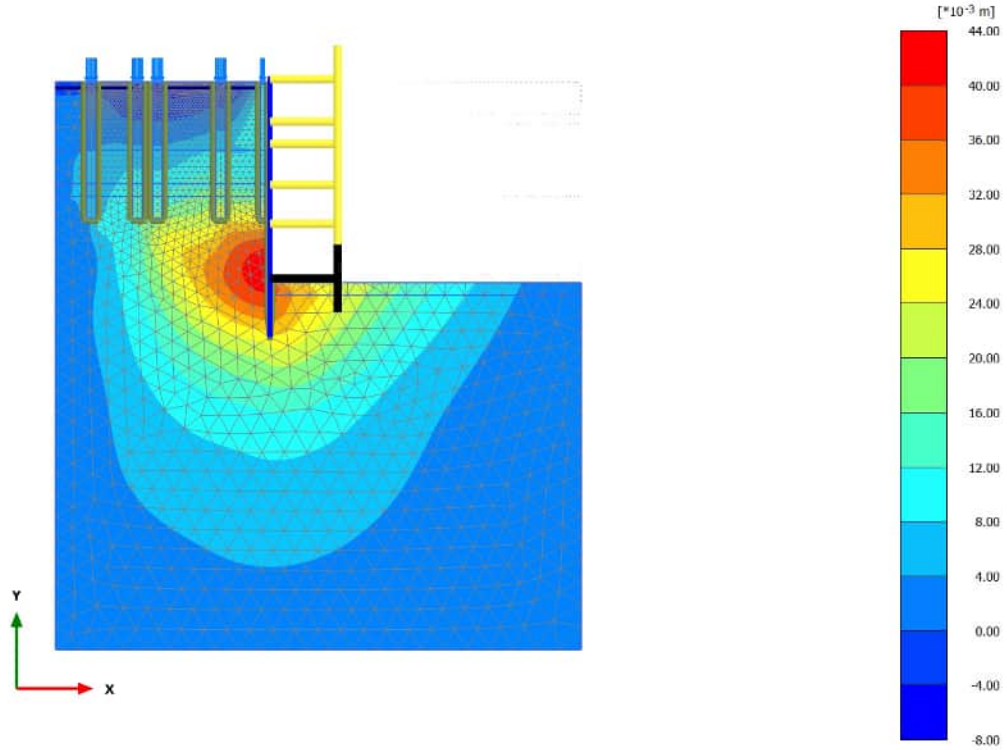


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.04341 m (Element 9374 at Node 57108)

Minimum value = -4.419*10-3 m (Element 2435 at Node 59307)

2.1.1.2.16 Calculation results, Phase_17 [Phase_17] (17/253), Total displacements u_x

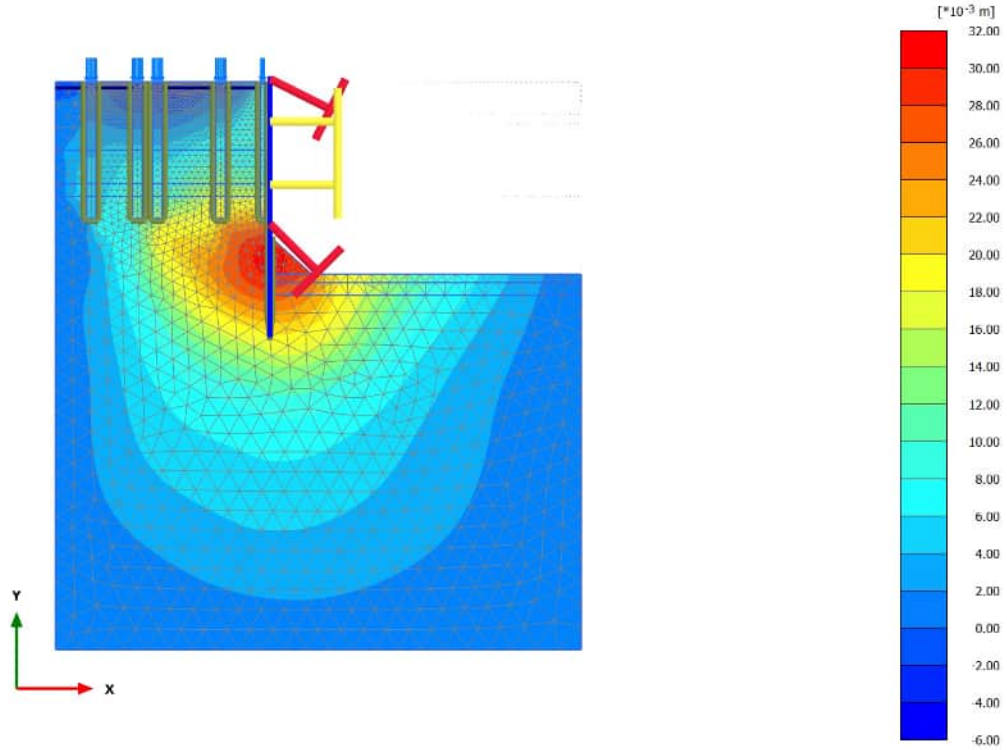


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.04342 m (Element 9374 at Node 57108)

Minimum value = -6.668*10-3 m (Element 989 at Node 73110)

2.1.1.2.17 Calculation results, Phase_18 [Phase_18] (18/258), Total displacements u_x

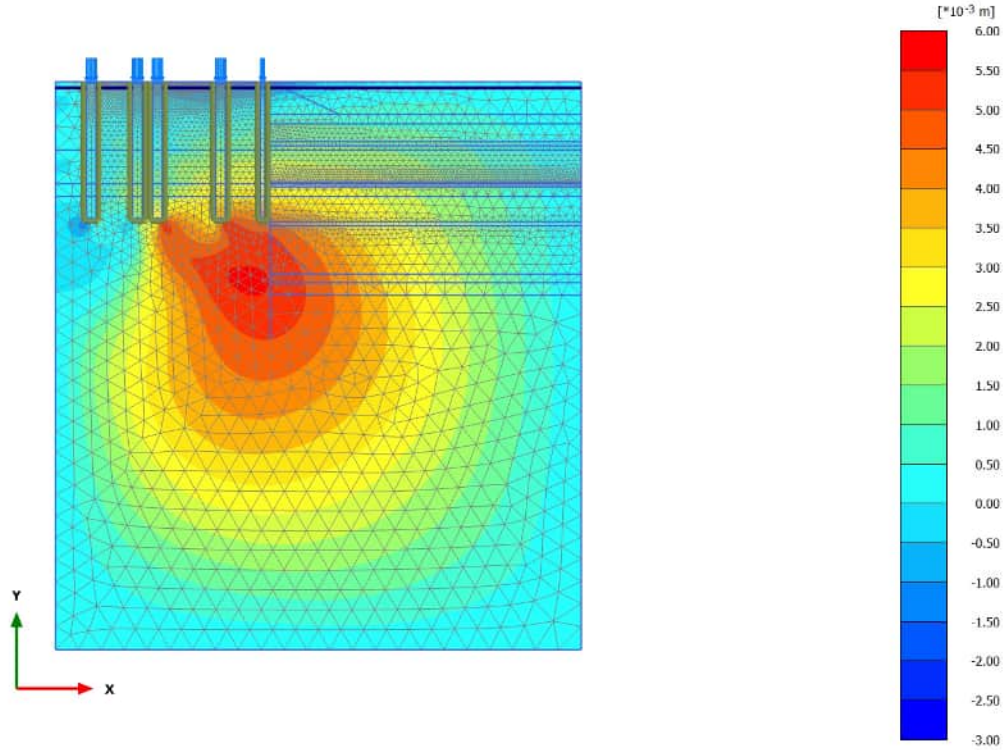


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.03095 m (Element 8108 at Node 55519)

Minimum value = $-4.156*10^{-3}$ m (Element 2434 at Node 60188)

2.1.1.2.18 Calculation results, Phase_1 [Phase_1] (1/310), Total displacements u_x

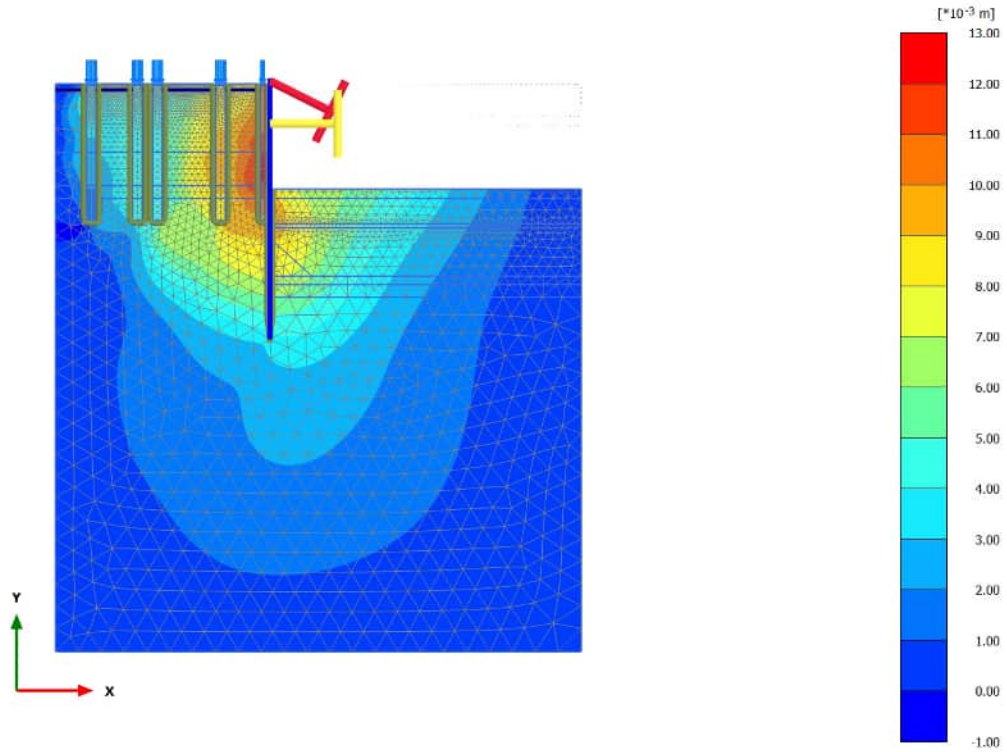


Total displacements u_x (scaled up 200 times)

Maximum value = 5.577*10-3 m (Element 9219 at Node 59585)

Minimum value = -2.901*10-3 m (Element 2406 at Node 54369)

2.1.1.2.19 Calculation results, Phase_7 [Phase_7] (7/339), Total displacements u_x

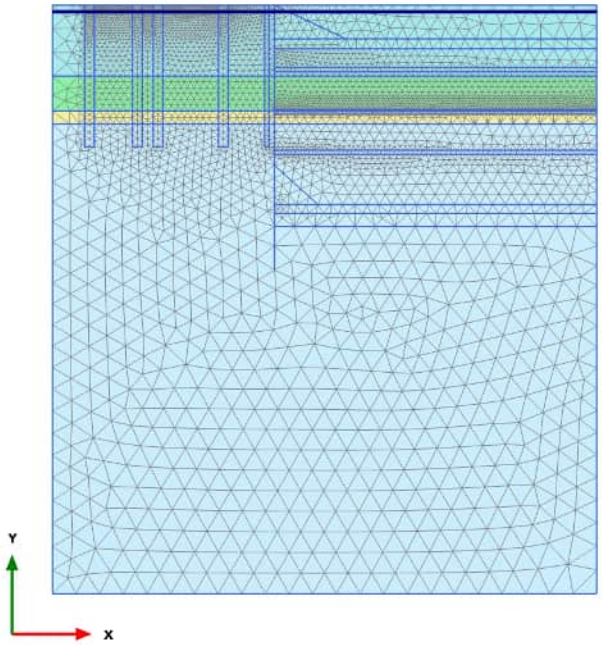


Total displacements u_x (scaled up 200 times)

Maximum value = 0.01240 m (Element 5957 at Node 35563)

Minimum value = -0.07591*10-3 m (Element 9240 at Node 71353)

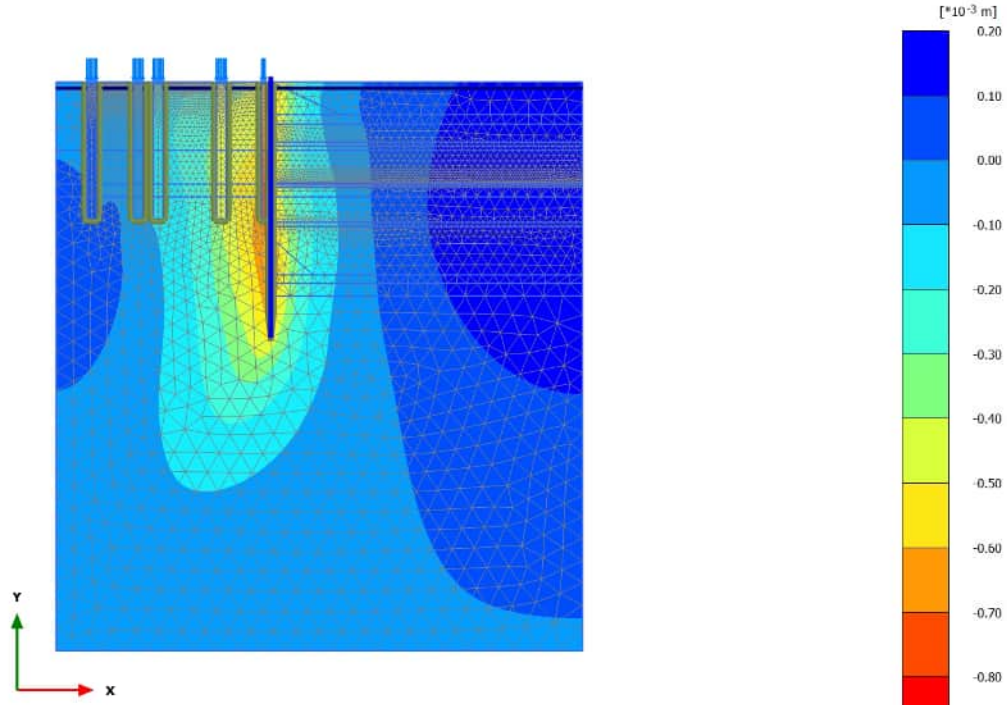
2.1.1.3.1 Calculation results, Initial phase [InitialPhase] (0/0), Total displacements u_y



Total displacements u_y (at true scale)

Uniform value of 0.000 m

2.1.1.3.2 Calculation results, Phase_2 [Phase_2] (2/3), Total displacements u_y

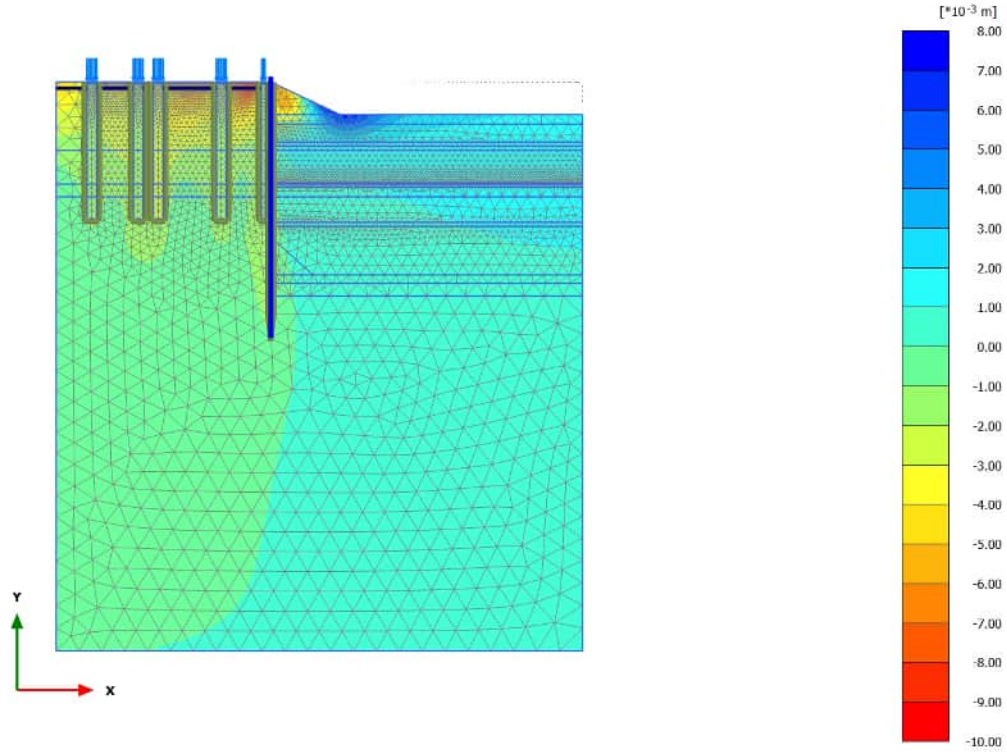


Total displacements u_y (scaled up 2.00*10³ times)

Maximum value = 0.1864*10^{-3} m (Element 7175 at Node 14729)

Minimum value = -0.8188*10^{-3} m (Element 7256 at Node 44345)

2.1.1.3.3 Calculation results, Phase_3 [Phase_3] (3/10), Total displacements u_y

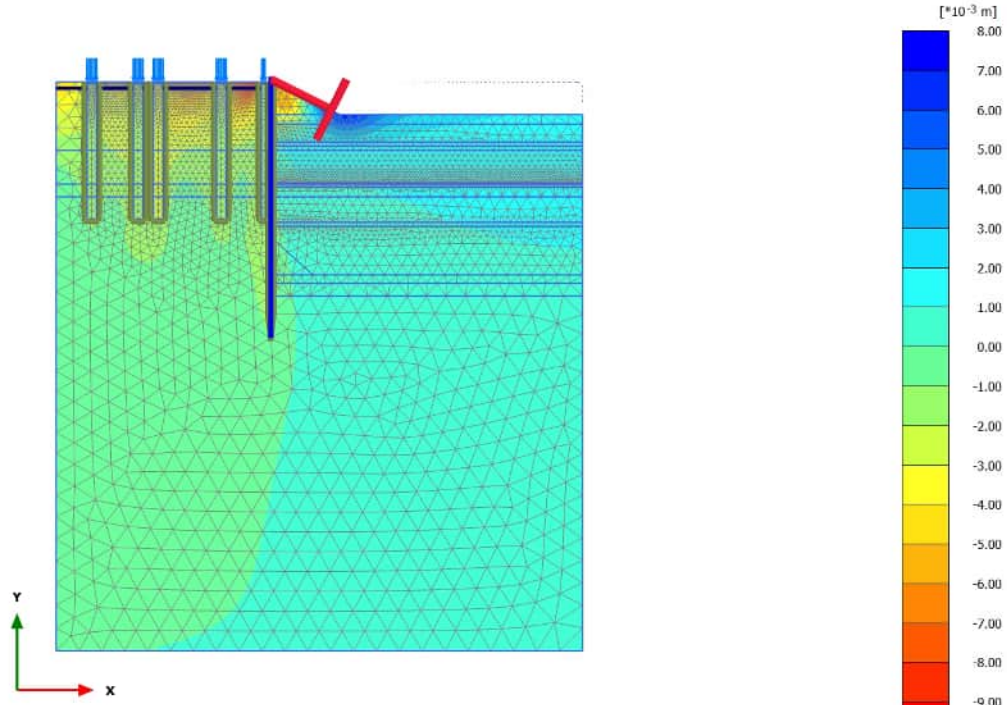


Total displacements u_y (scaled up 100 times)

Maximum value = $7.468*10^{-3}$ m (Element 149 at Node 17541)

Minimum value = $-9.474*10^{-3}$ m (Element 2772 at Node 25811)

2.1.1.3.4 Calculation results, Phase_4 [Phase_4] (4/34), Total displacements u_y

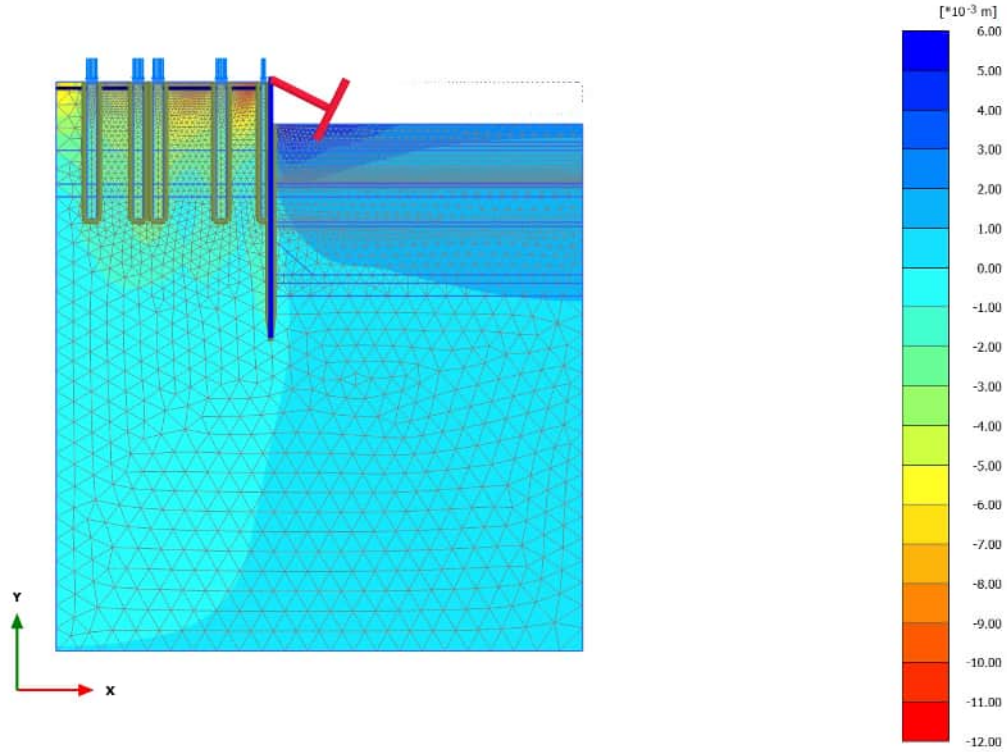


Total displacements u_y (scaled up 100 times)

Maximum value = $7.491*10^{-3}$ m (Element 149 at Node 17541)

Minimum value = $-9.562*10^{-3}$ m (Element 2772 at Node 25811)

2.1.1.3.5 Calculation results, Phase_5 [Phase_5] (5/42), Total displacements u_y

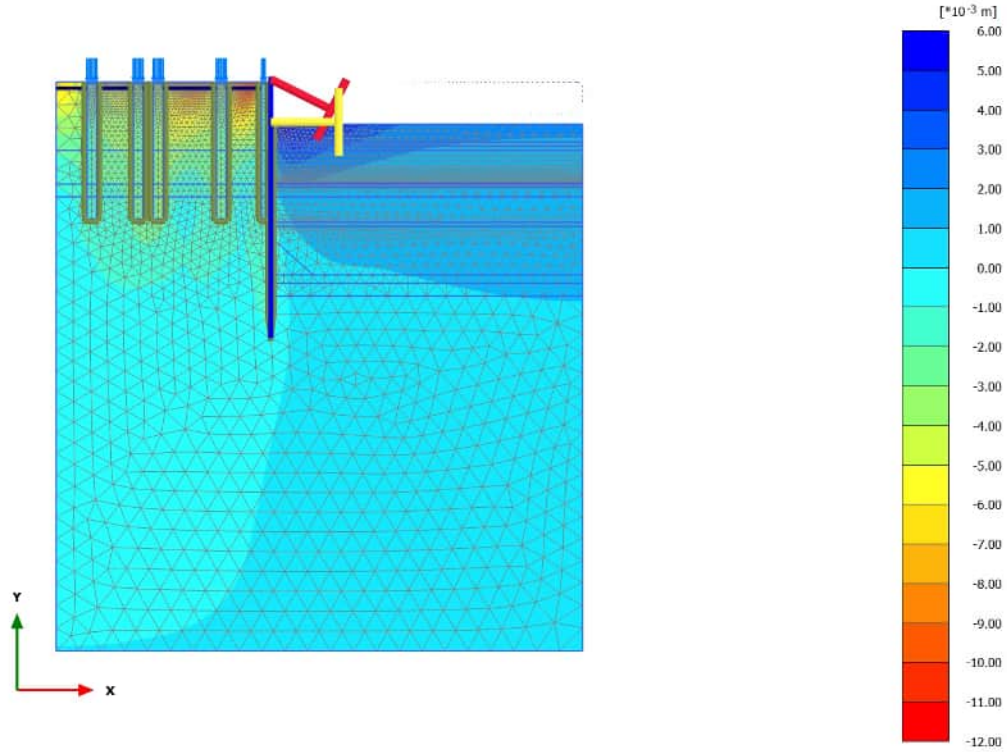


Total displacements u_y (scaled up 100 times)

Maximum value = $5.425*10^{-3}$ m (Element 3366 at Node 25474)

Minimum value = -0.01106 m (Element 2772 at Node 25811)

2.1.1.3.6 Calculation results, Phase_6 [Phase_6] (6/89), Total displacements u_y

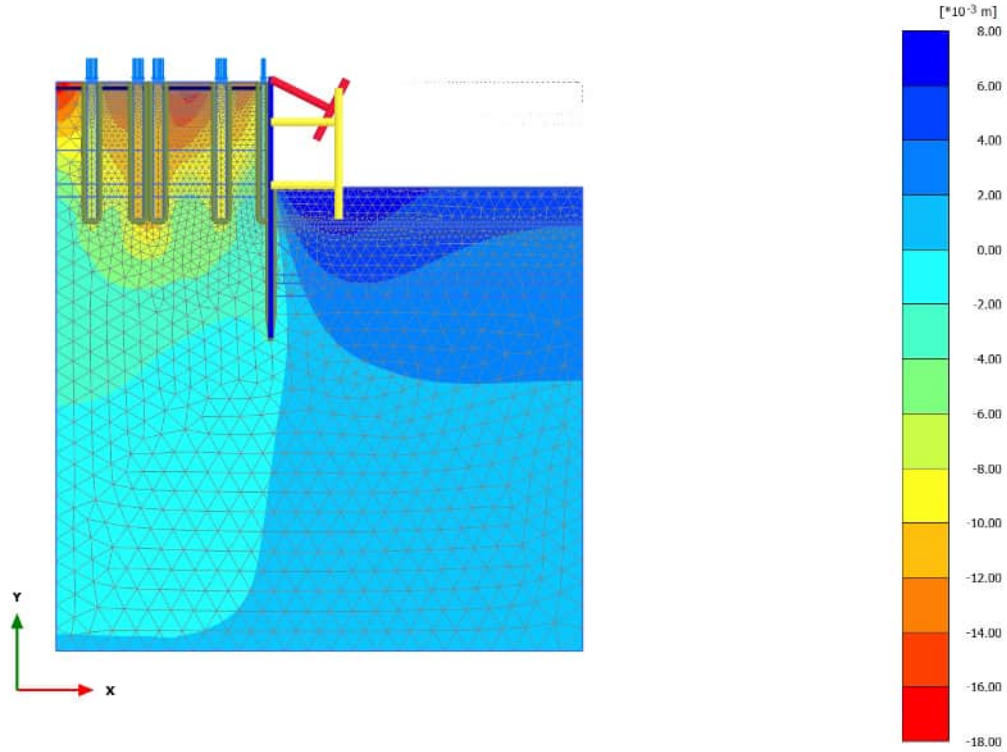


Total displacements u_y (scaled up 100 times)

Maximum value = $5.446*10^{-3}$ m (Element 3366 at Node 25482)

Minimum value = -0.01106 m (Element 2772 at Node 25811)

2.1.1.3.7 Calculation results, Phase_8 [Phase_8] (8/91), Total displacements u_y

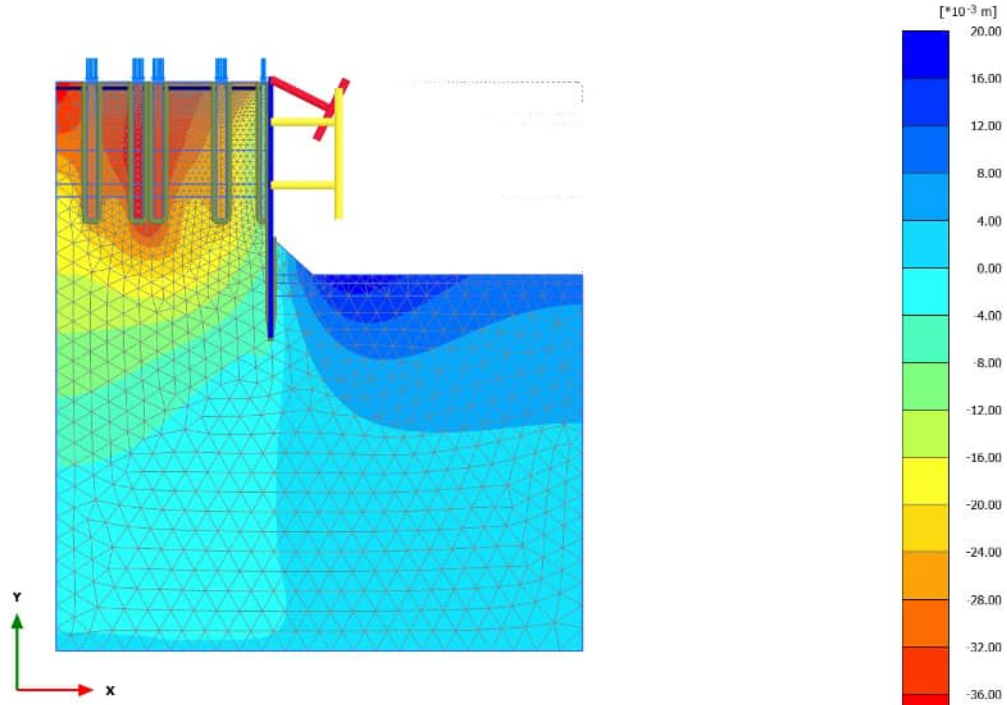


Total displacements u_y (scaled up 50.0 times)

Maximum value = $7.782*10^{-3}$ m (Element 7054 at Node 31990)

Minimum value = -0.01701 m (Element 2193 at Node 47354)

2.1.1.3.8 Calculation results, Phase_9 [Phase_9] (9/166), Total displacements u_y

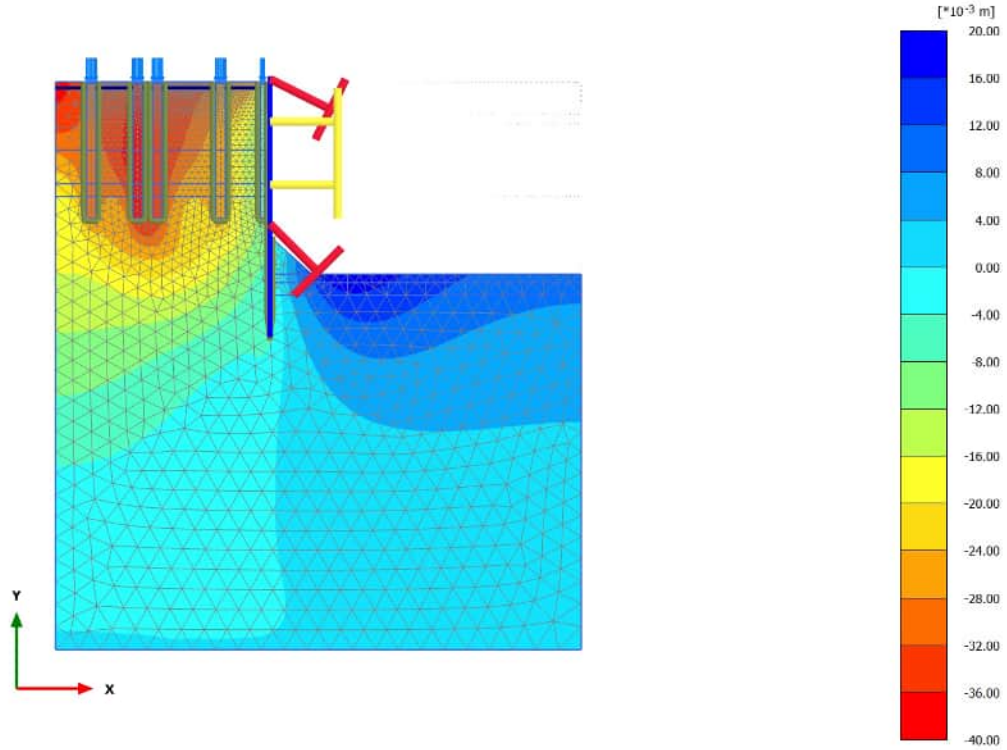


Total displacements u_y (scaled up 50.0 times)

Maximum value = 0.01845 m (Element 8148 at Node 51826)

Minimum value = -0.03793 m (Element 5681 at Node 67965)

2.1.1.3.9 Calculation results, Phase_10 [Phase_10] (10/168), Total displacements u_y

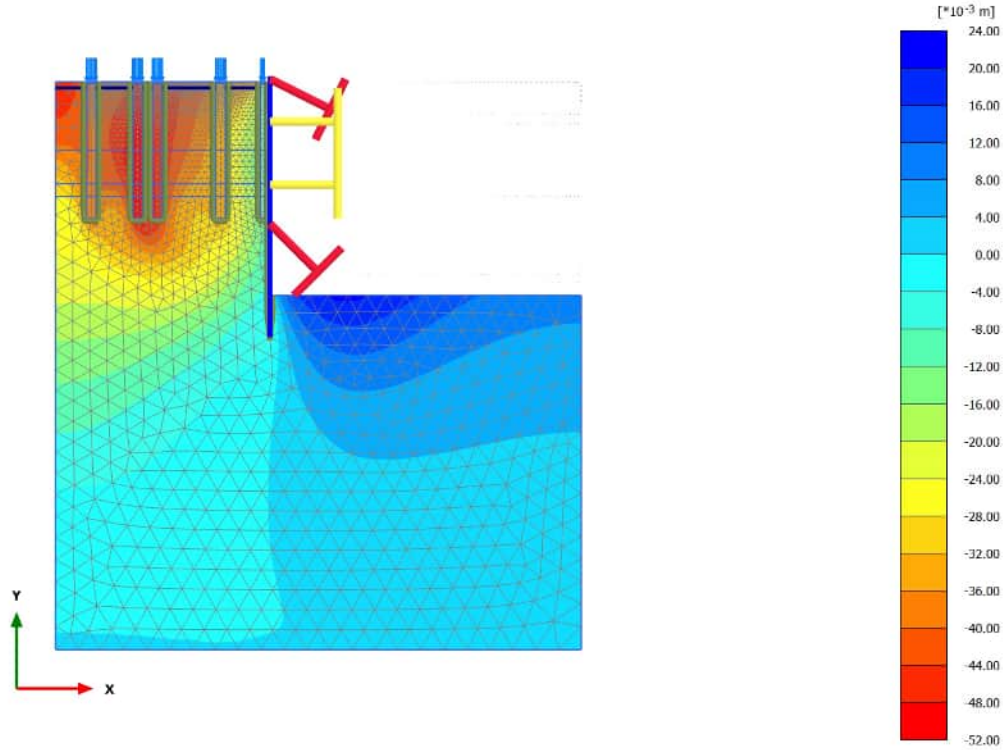


Total displacements u_y (scaled up 50.0 times)

Maximum value = 0.01845 m (Element 8148 at Node 51826)

Minimum value = -0.03793 m (Element 5681 at Node 67965)

2.1.1.3.10 Calculation results, Phase_11 [Phase_11] (11/218), Total displacements u_y

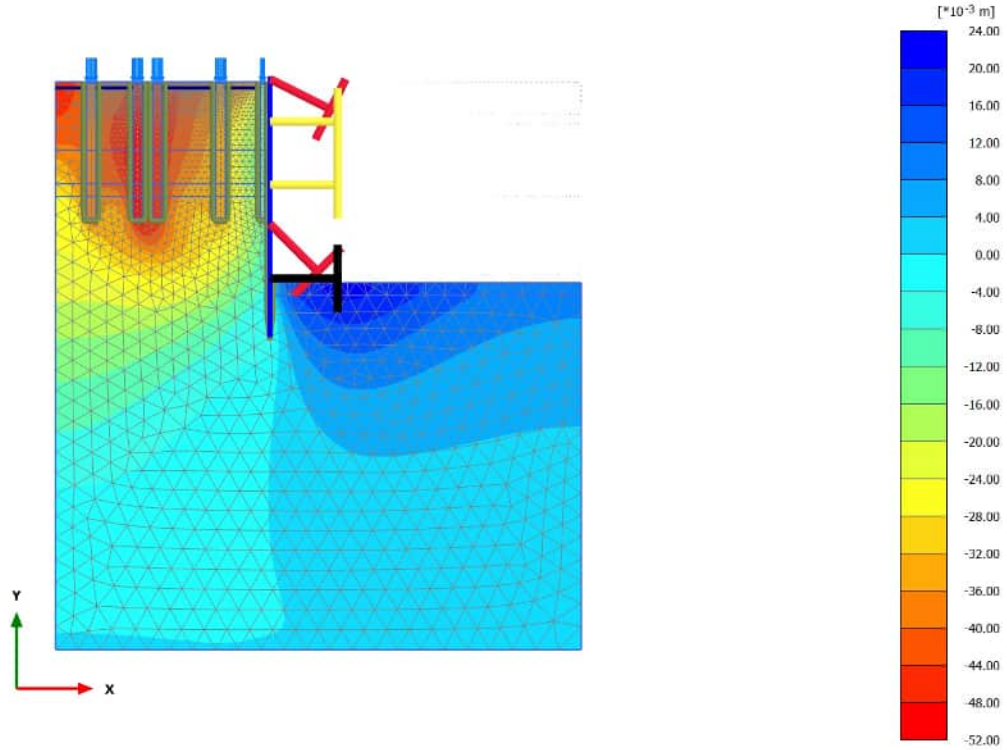


Total displacements u_y (scaled up 20.0 times)

Maximum value = 0.02117 m (Element 8703 at Node 53509)

Minimum value = -0.05134 m (Element 5699 at Node 66374)

2.1.1.3.11 Calculation results, Phase_12 [Phase_12] (12/221), Total displacements u_y

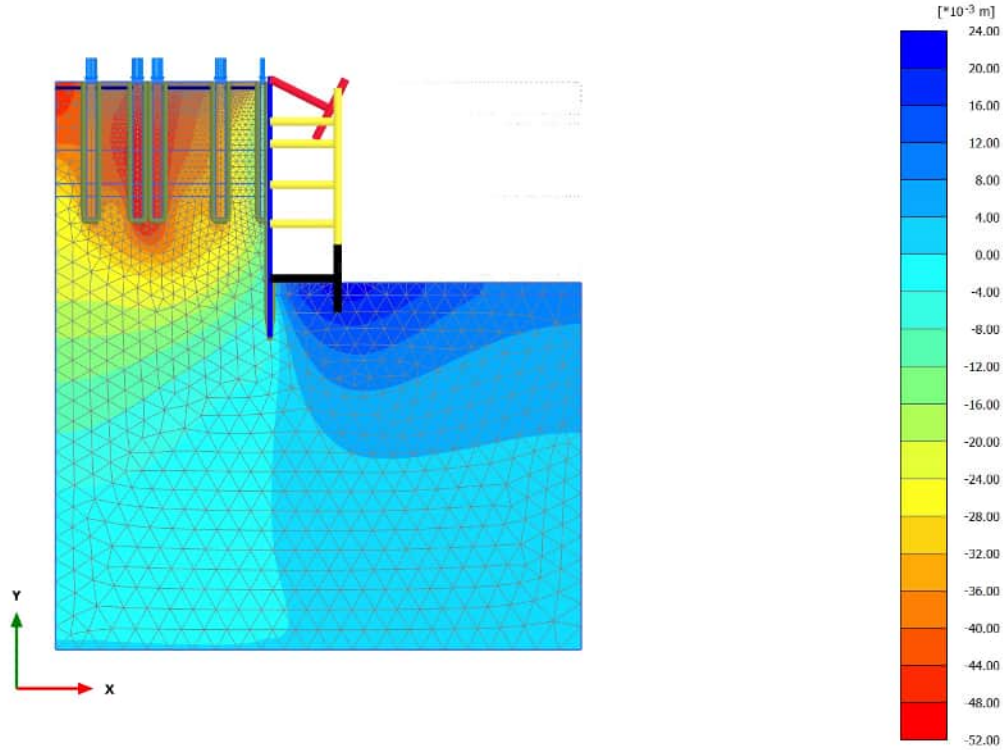


Total displacements u_y (scaled up 20.0 times)

Maximum value = 0.02153 m (Element 8229 at Node 51863)

Minimum value = -0.05113 m (Element 5681 at Node 67297)

2.1.1.3.12 Calculation results, Phase_13 [Phase_13] (13/226), Total displacements u_y

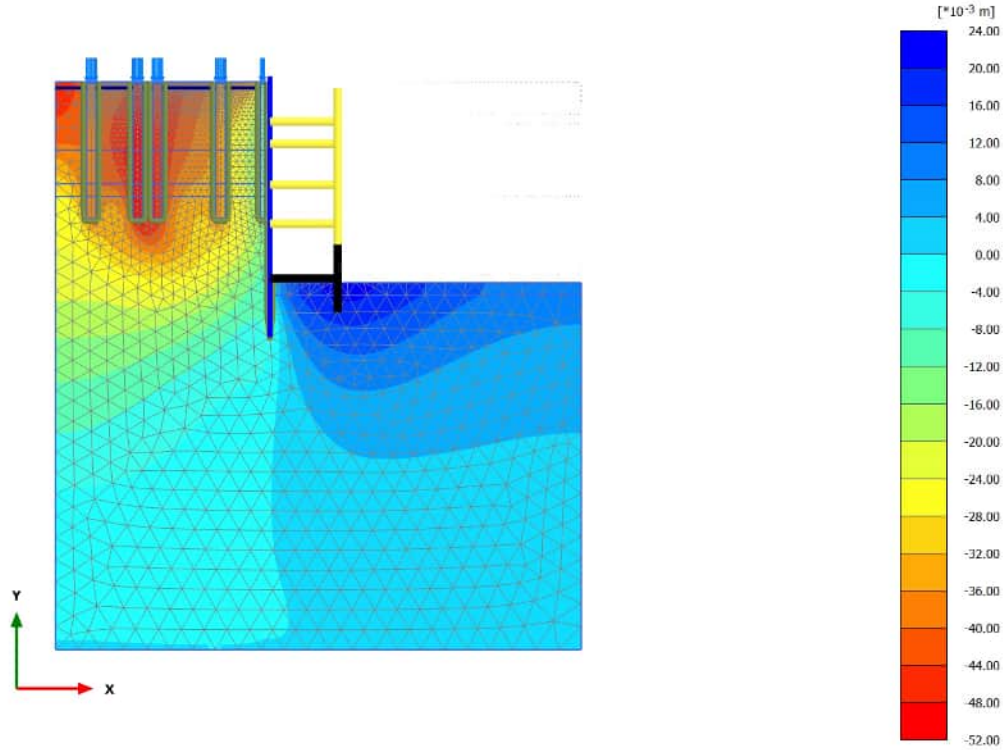


Total displacements u_y (scaled up 20.0 times)

Maximum value = 0.02171 m (Element 8229 at Node 51864)

Minimum value = -0.05118 m (Element 5681 at Node 67300)

2.1.1.3.13 Calculation results, Phase_14 [Phase_14] (14/229), Total displacements u_y

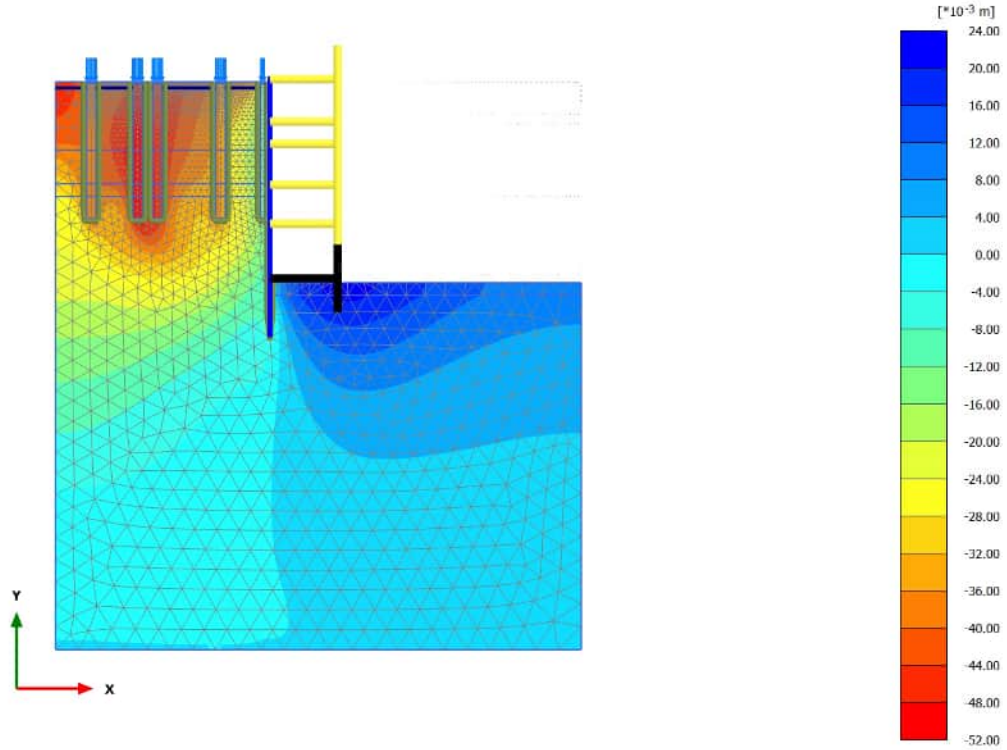


Total displacements u_y (scaled up 20.0 times)

Maximum value = 0.02174 m (Element 8229 at Node 51864)

Minimum value = -0.05119 m (Element 5681 at Node 67297)

2.1.1.3.14 Calculation results, Phase_15 [Phase_15] (15/245), Total displacements u_y

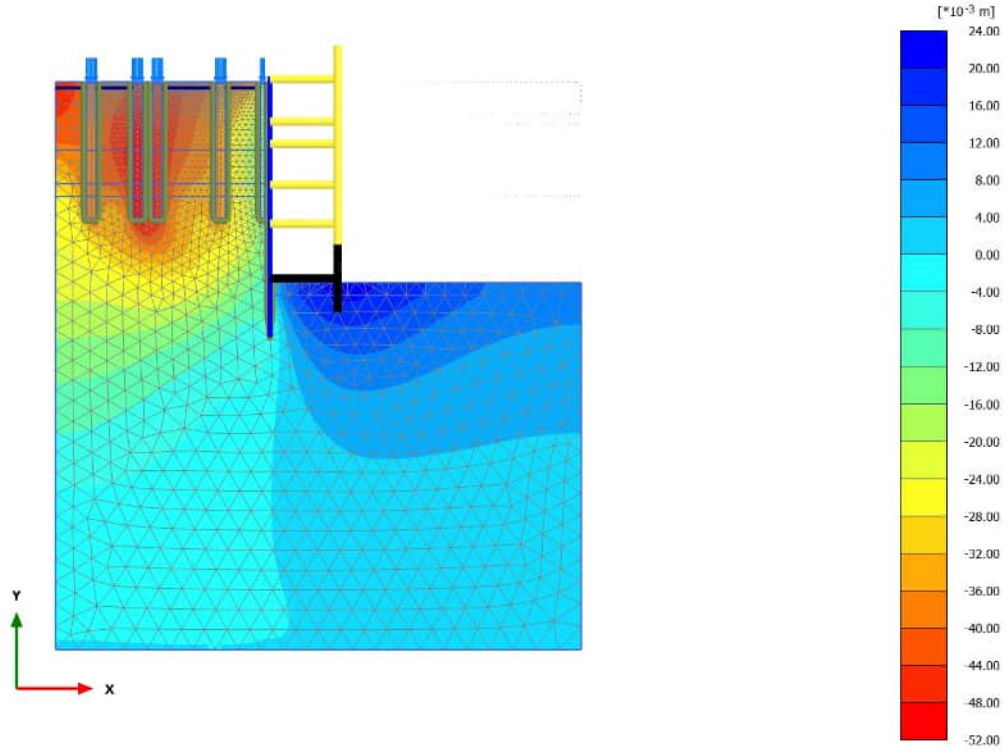


Total displacements u_y (scaled up 20.0 times)

Maximum value = 0.02174 m (Element 8229 at Node 51864)

Minimum value = -0.05119 m (Element 5681 at Node 67297)

2.1.1.3.15 Calculation results, Phase_16 [Phase_16] (16/248), Total displacements u_y

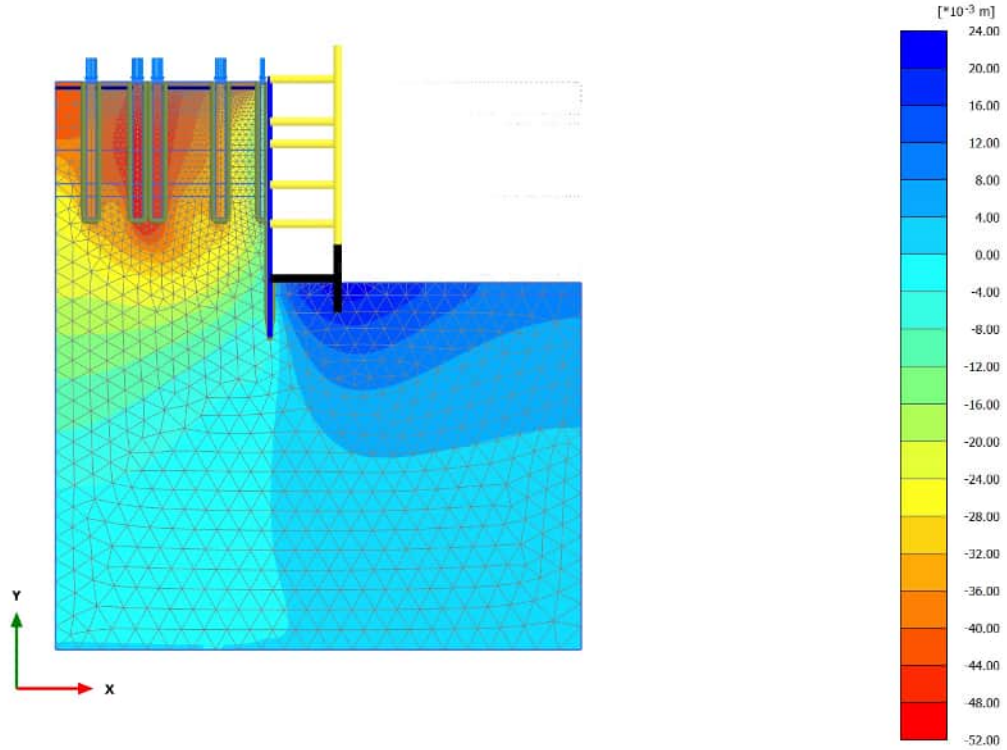


Total displacements u_y (scaled up 20.0 times)

Maximum value = 0.02174 m (Element 8229 at Node 51864)

Minimum value = -0.05119 m (Element 5681 at Node 67297)

2.1.1.3.16 Calculation results, Phase_17 [Phase_17] (17/253), Total displacements u_y

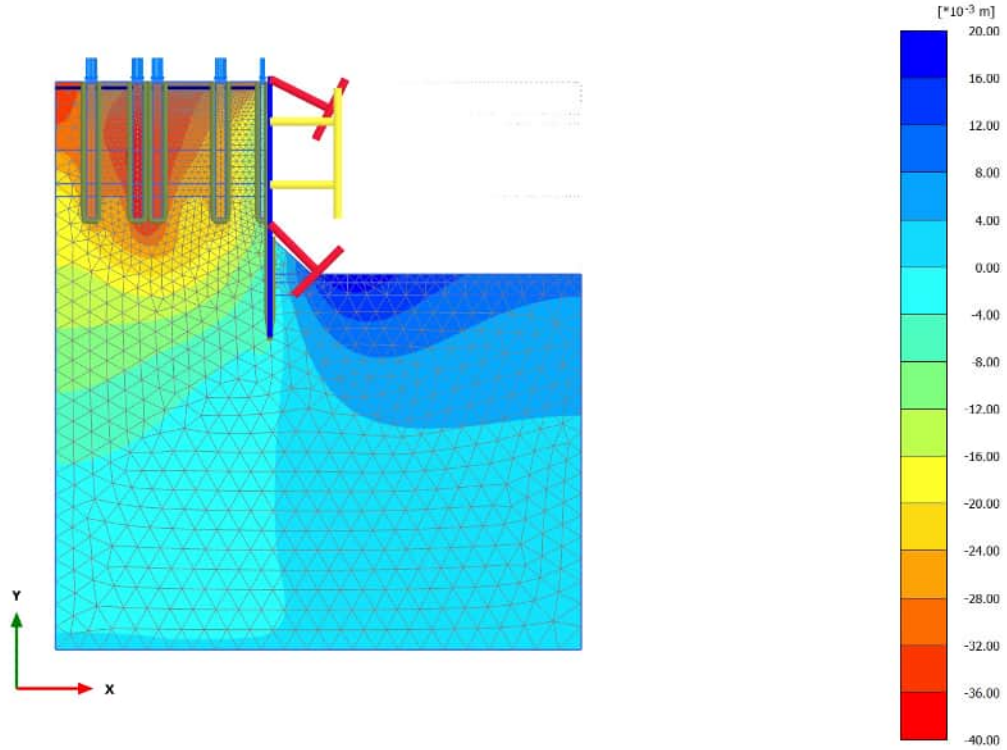


Total displacements u_y (scaled up 20.0 times)

Maximum value = 0.02168 m (Element 8229 at Node 51863)

Minimum value = -0.05100 m (Element 1024 at Node 73114)

2.1.1.3.17 Calculation results, Phase_18 [Phase_18] (18/258), Total displacements u_y

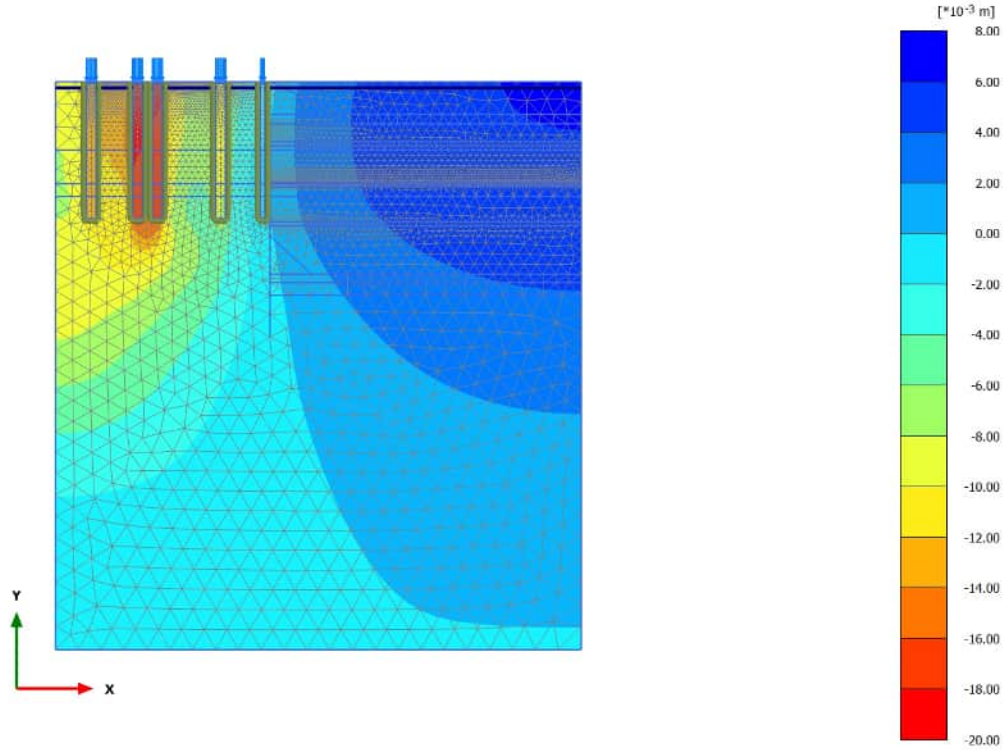


Total displacements u_y (scaled up 50.0 times)

Maximum value = 0.01839 m (Element 8148 at Node 51826)

Minimum value = -0.03775 m (Element 1024 at Node 73114)

2.1.1.3.18 Calculation results, Phase_1 [Phase_1] (1/310), Total displacements u_y

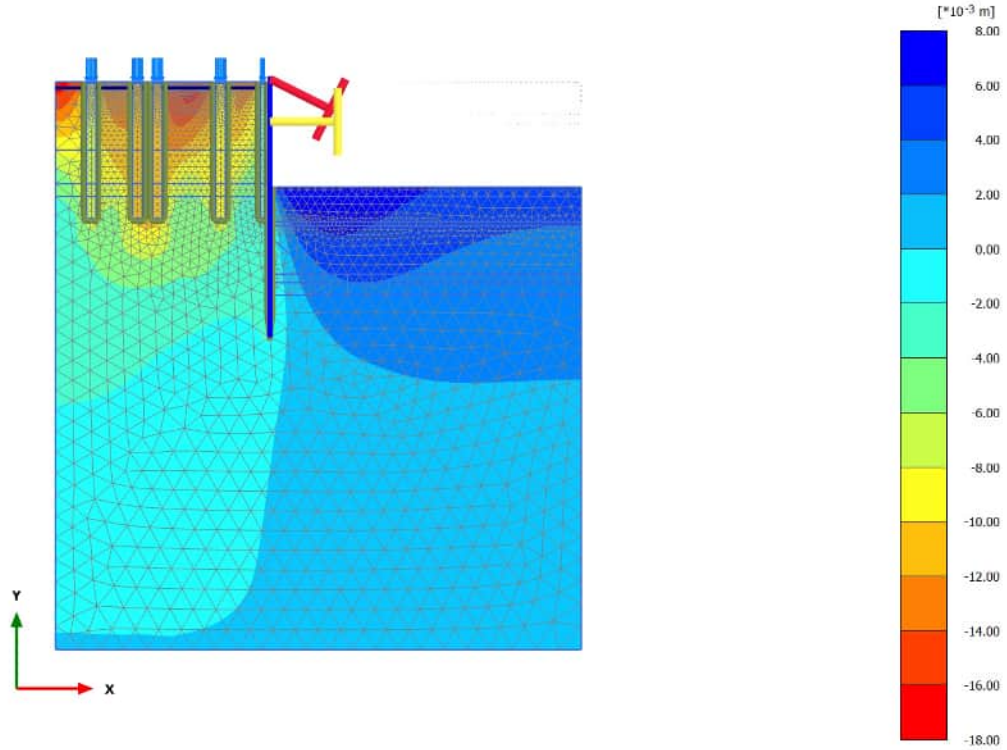


Total displacements u_y (scaled up 50.0 times)

Maximum value = $6.248*10^{-3}$ m (Element 1 at Node 10)

Minimum value = -0.01890 m (Element 1024 at Node 73114)

2.1.1.3.19 Calculation results, Phase_7 [Phase_7] (7/339), Total displacements u_y

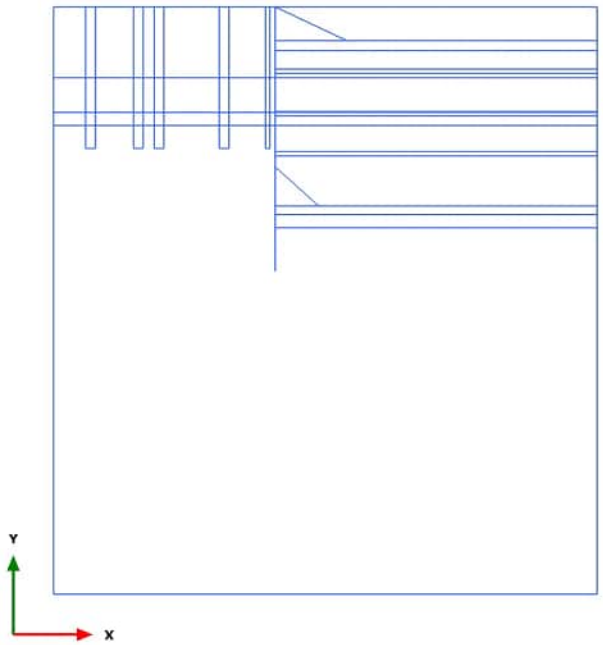


Total displacements u_y (scaled up 50.0 times)

Maximum value = $7.783*10^{-3}$ m (Element 7054 at Node 31990)

Minimum value = -0.01701 m (Element 2193 at Node 47354)

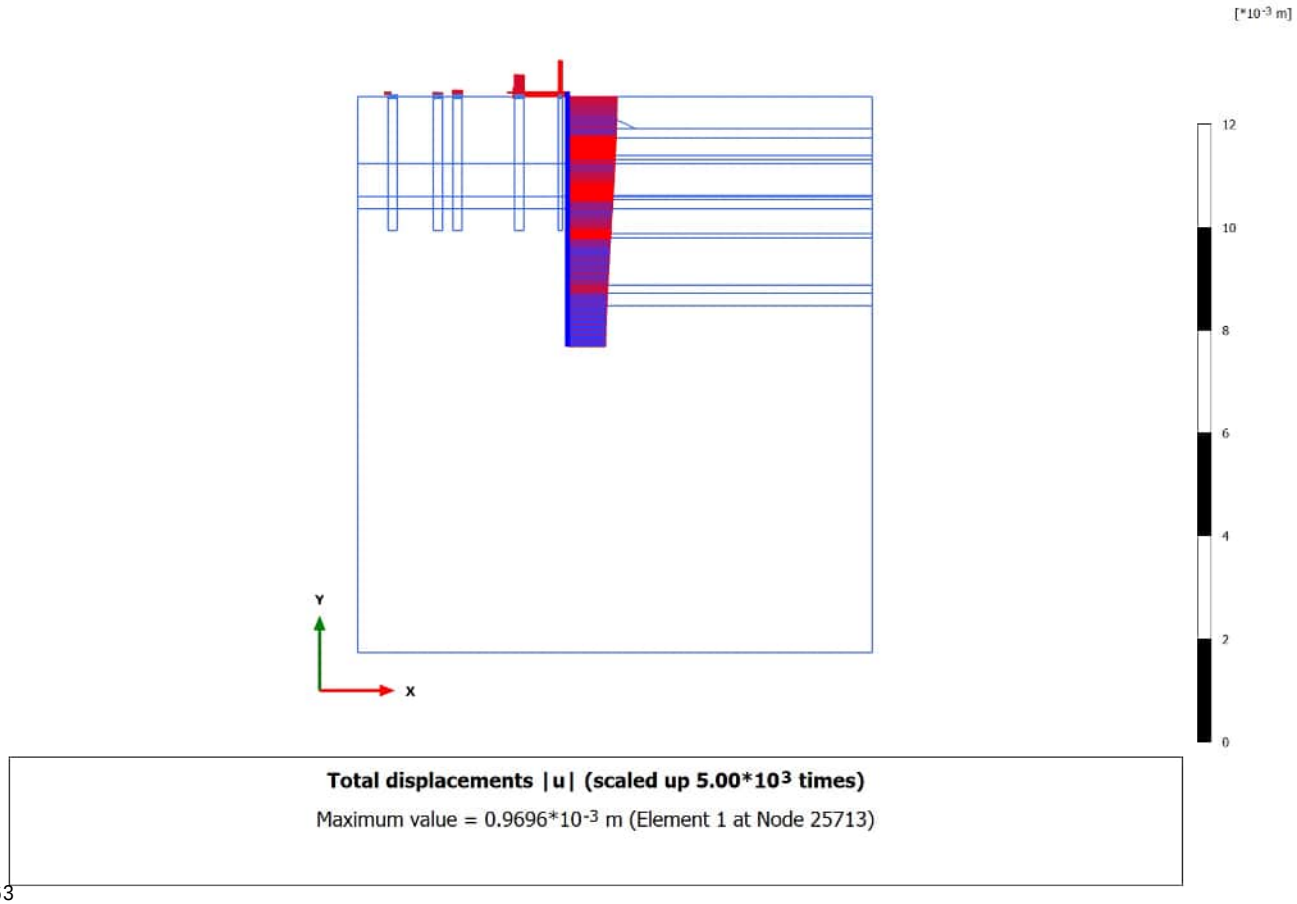
3.1.1.1.1.1 Calculation results, Plate, Initial phase [InitialPhase] (0/0), Total displacements |u|



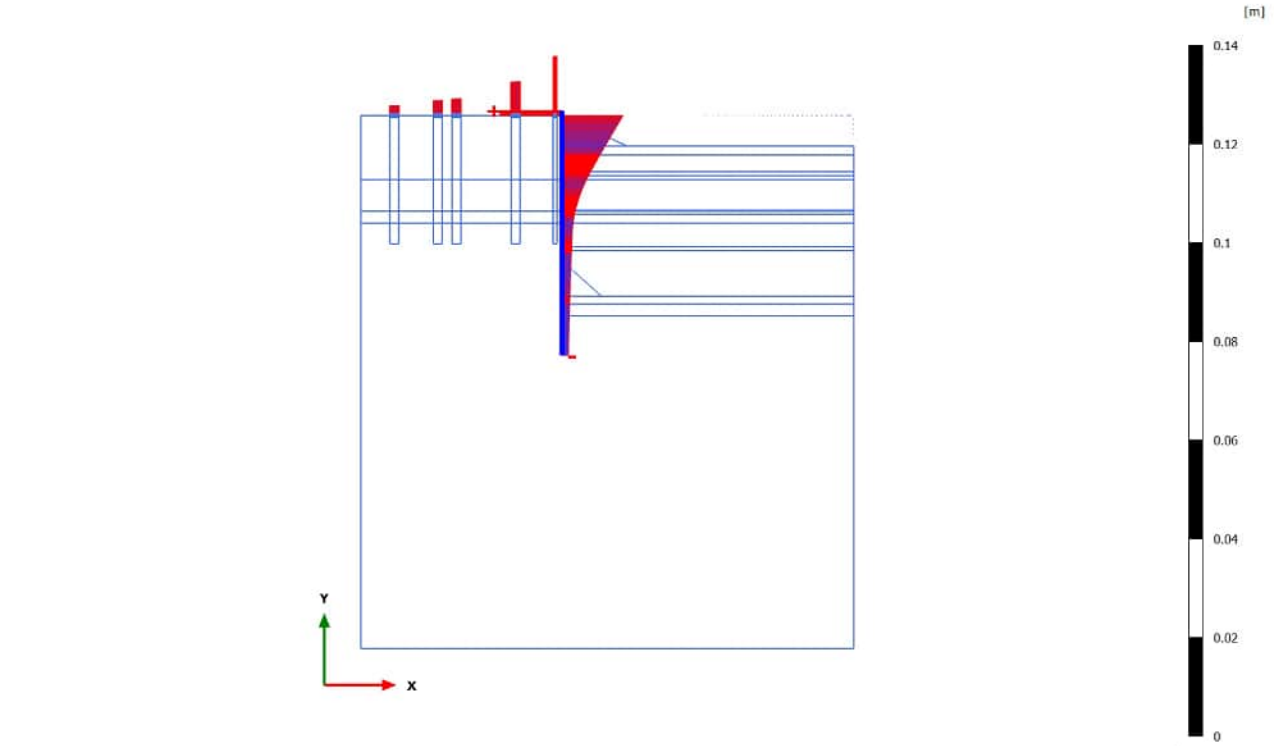
Total displacements |u| (at true scale)

No results

3.1.1.1.1.2 Calculation results, Plate, Phase_2 [Phase_2] (2/3), Total displacements |u|



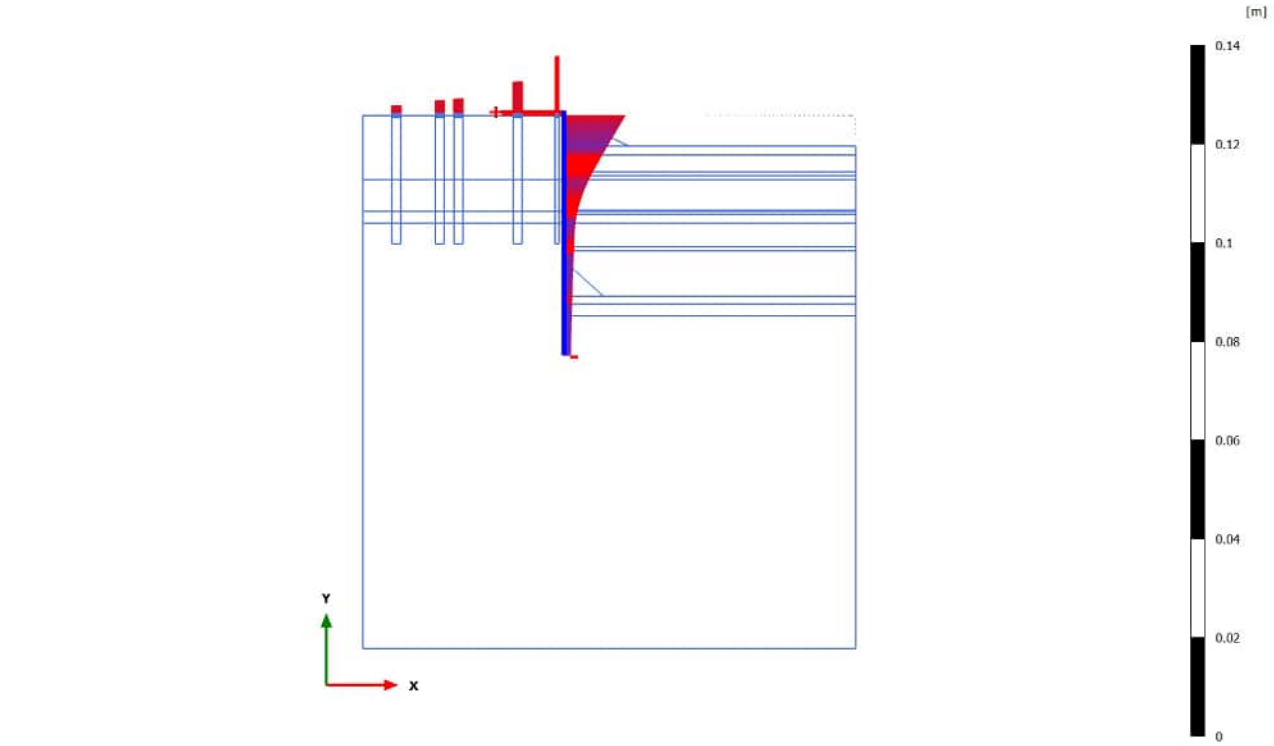
3.1.1.1.1.3 Calculation results, Plate, Phase_3 [Phase_3] (3/10), Total displacements |u|



Total displacements |u| (scaled up 500 times)

Maximum value = 0.01301 m (Element 1 at Node 25713)

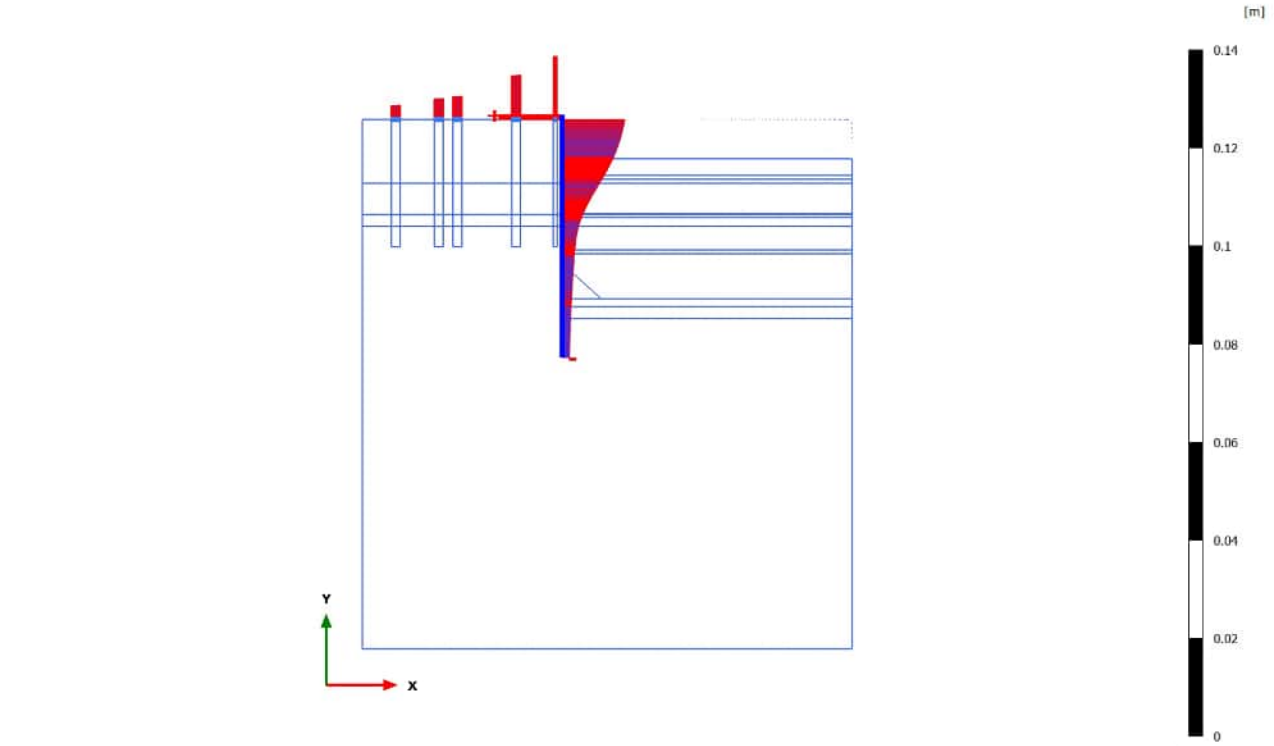
3.1.1.1.1.4 Calculation results, Plate, Phase_4 [Phase_4] (4/34), Total displacements |u|



Total displacements |u| (scaled up 500 times)

Maximum value = 0.01301 m (Element 1 at Node 25713)

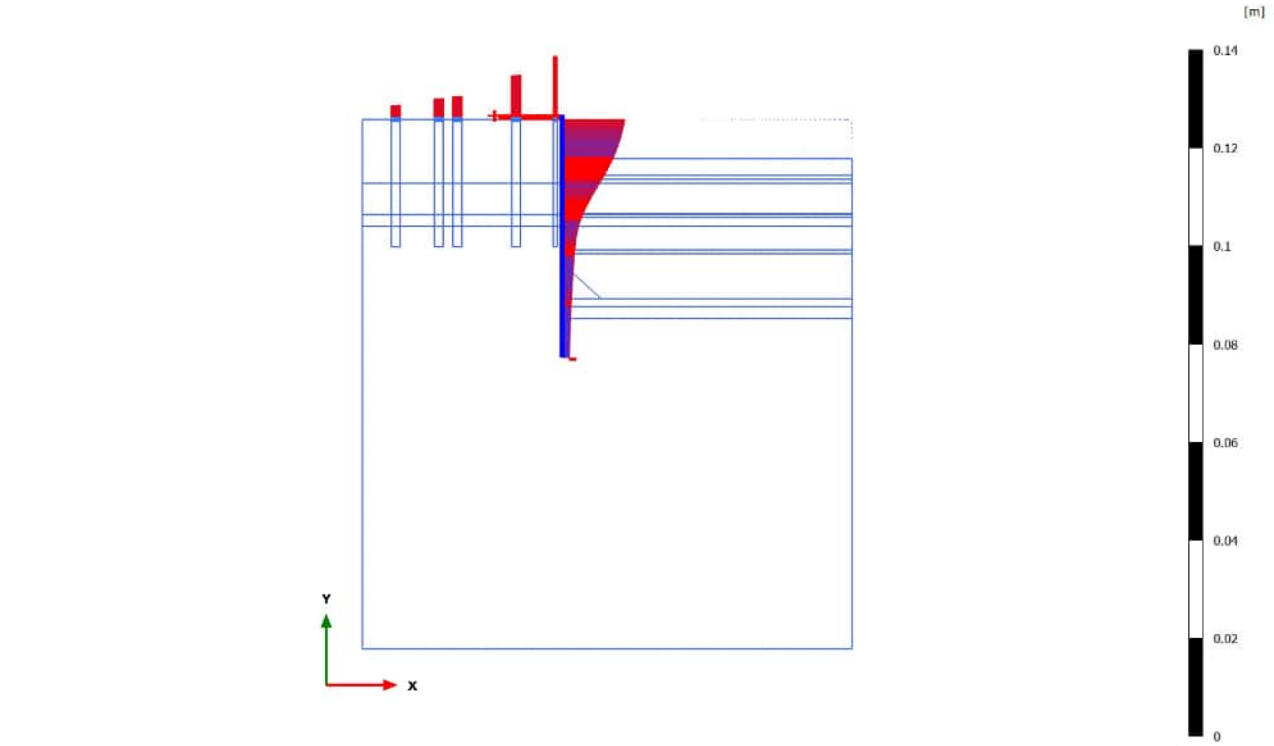
3.1.1.1.1.5 Calculation results, Plate, Phase_5 [Phase_5] (5/42), Total displacements |u|



Total displacements |u| (scaled up 500 times)

Maximum value = 0.01309 m (Element 1 at Node 25713)

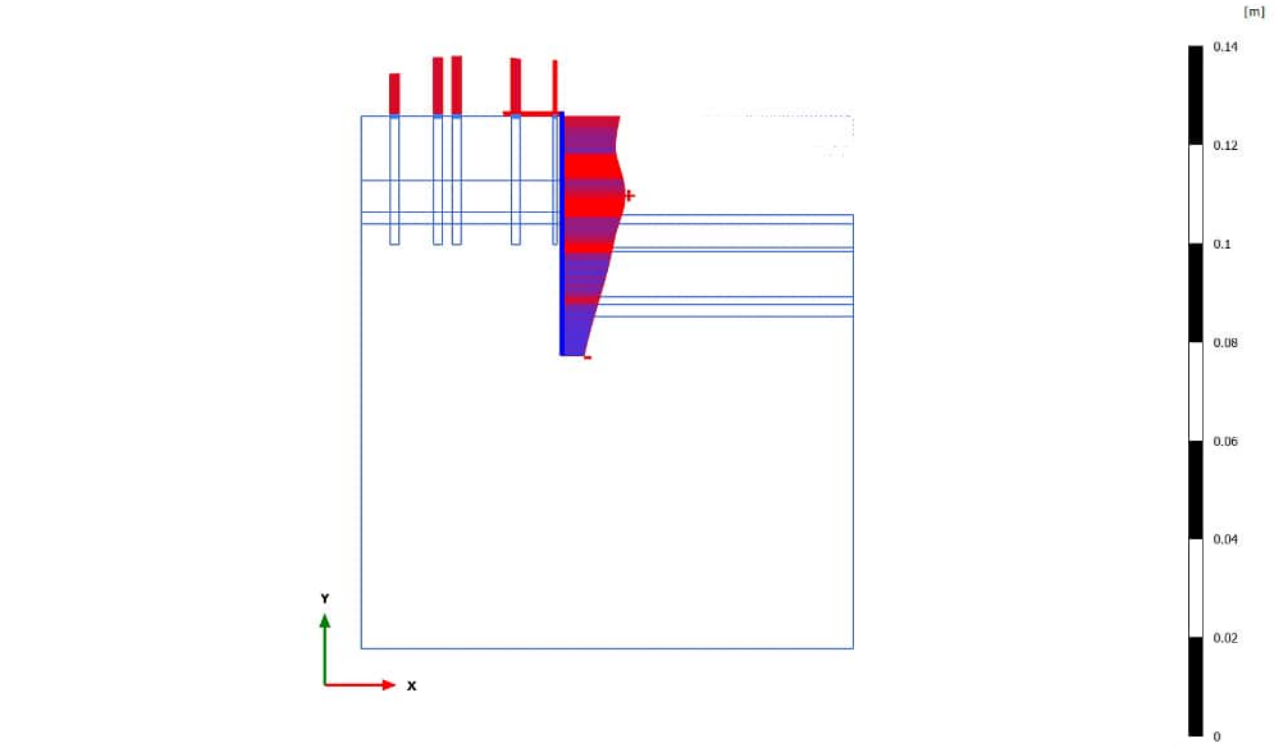
3.1.1.1.1.6 Calculation results, Plate, Phase_6 [Phase_6] (6/89), Total displacements |u|



Total displacements |u| (scaled up 500 times)

Maximum value = 0.01309 m (Element 1 at Node 25713)

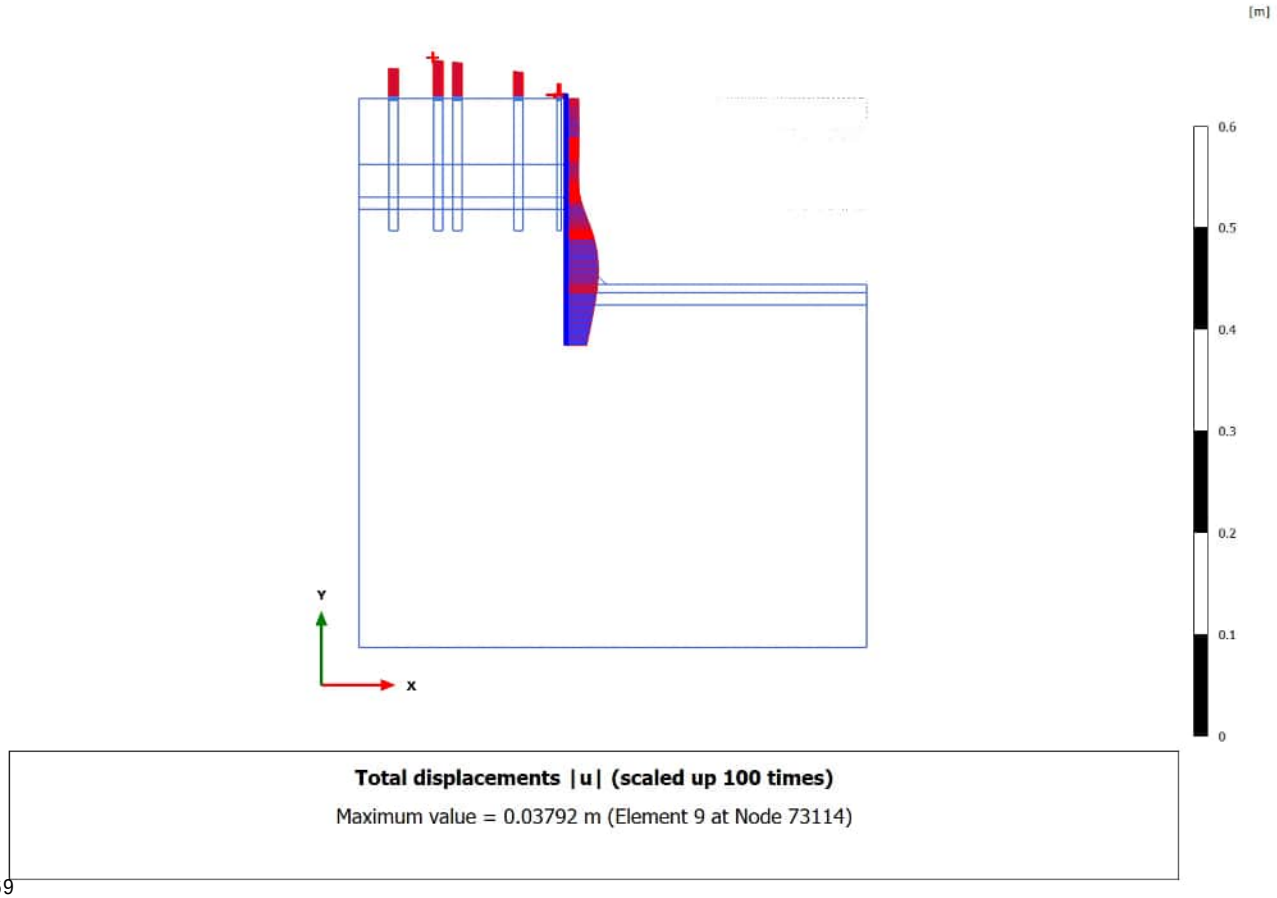
3.1.1.1.1.7 Calculation results, Plate, Phase_8 [Phase_8] (8/91), Total displacements |u|



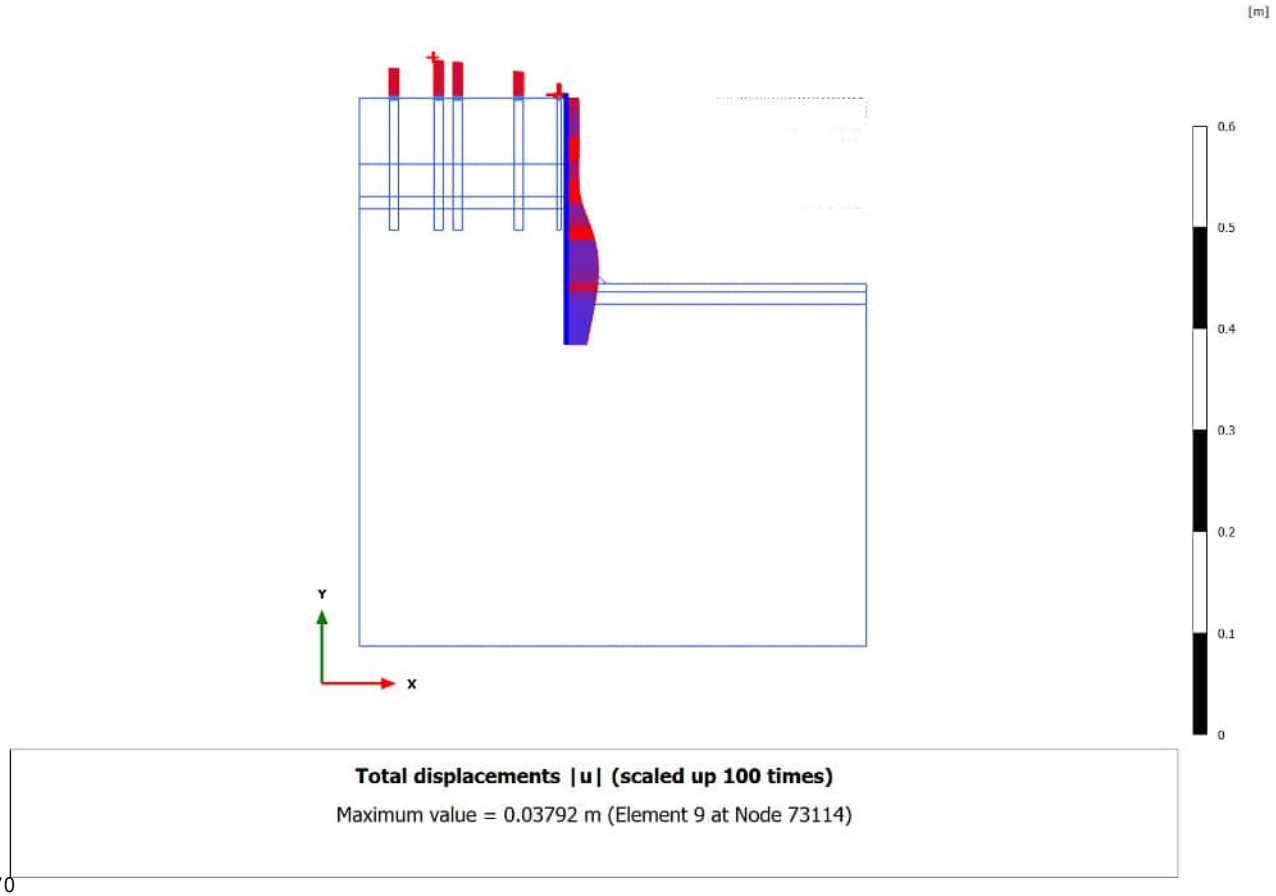
Total displacements |u| (scaled up 500 times)

Maximum value = 0.01284 m (Element 48 at Node 32363)

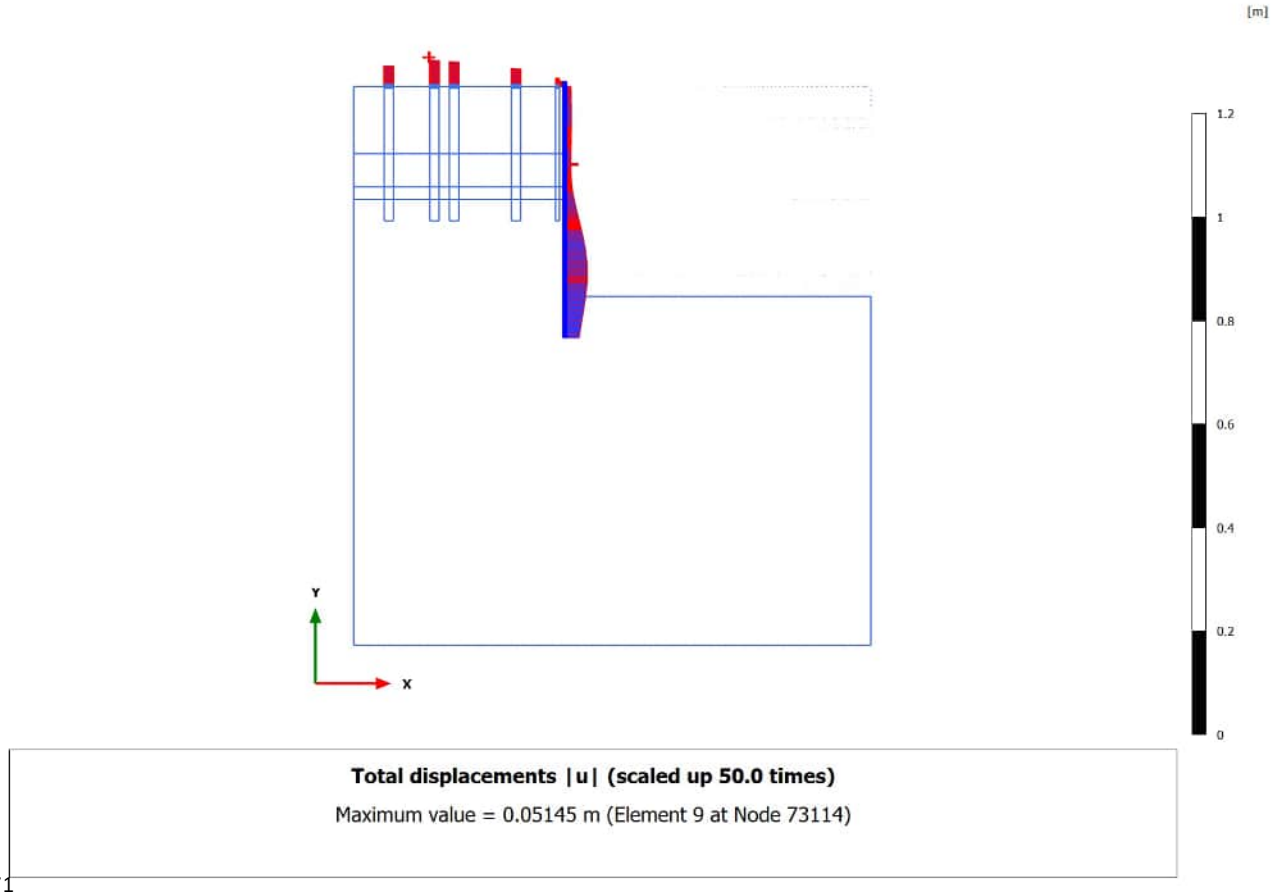
3.1.1.1.1.8 Calculation results, Plate, Phase_9 [Phase_9] (9/166), Total displacements |u|



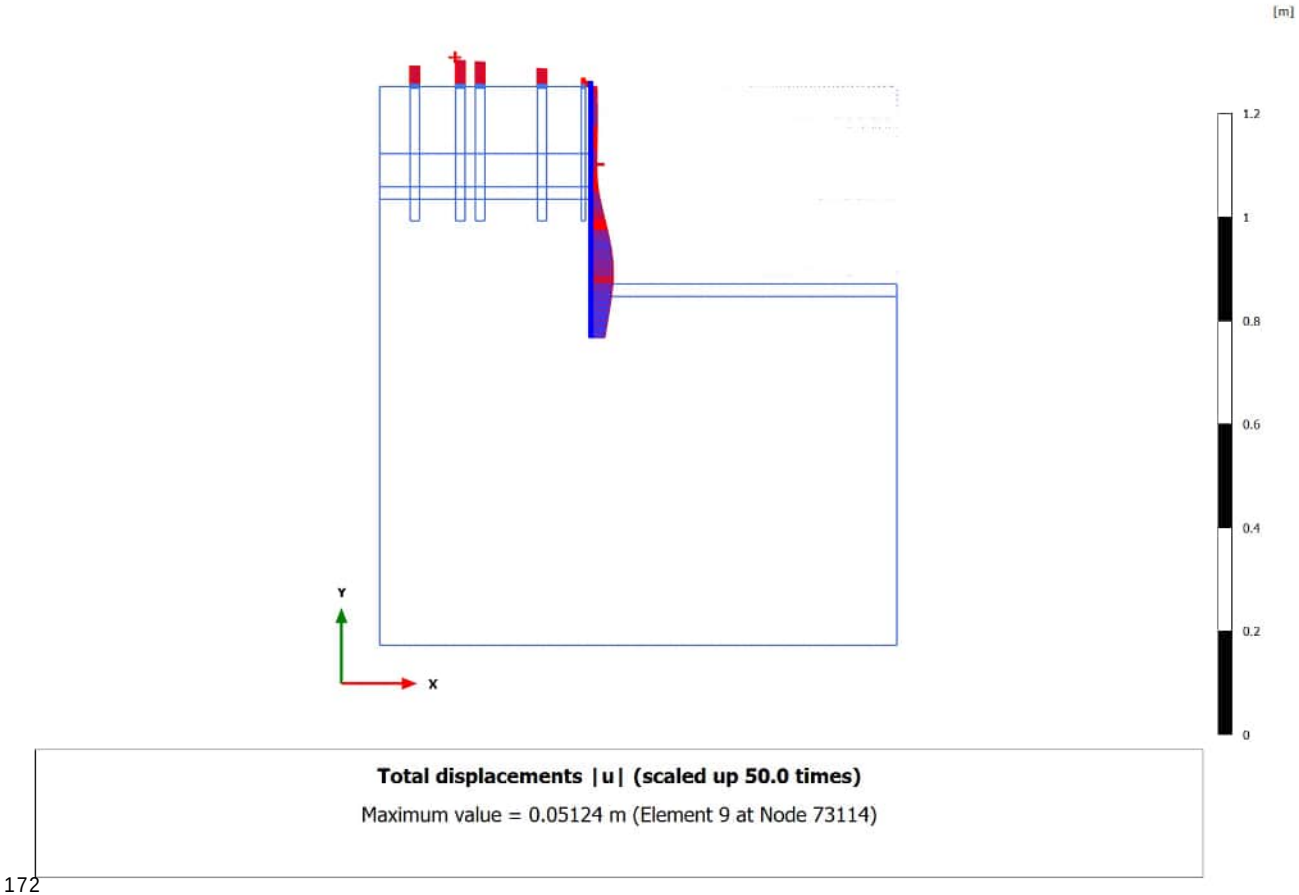
3.1.1.1.1.9 Calculation results, Plate, Phase_10 [Phase_10] (10/168), Total displacements |u|



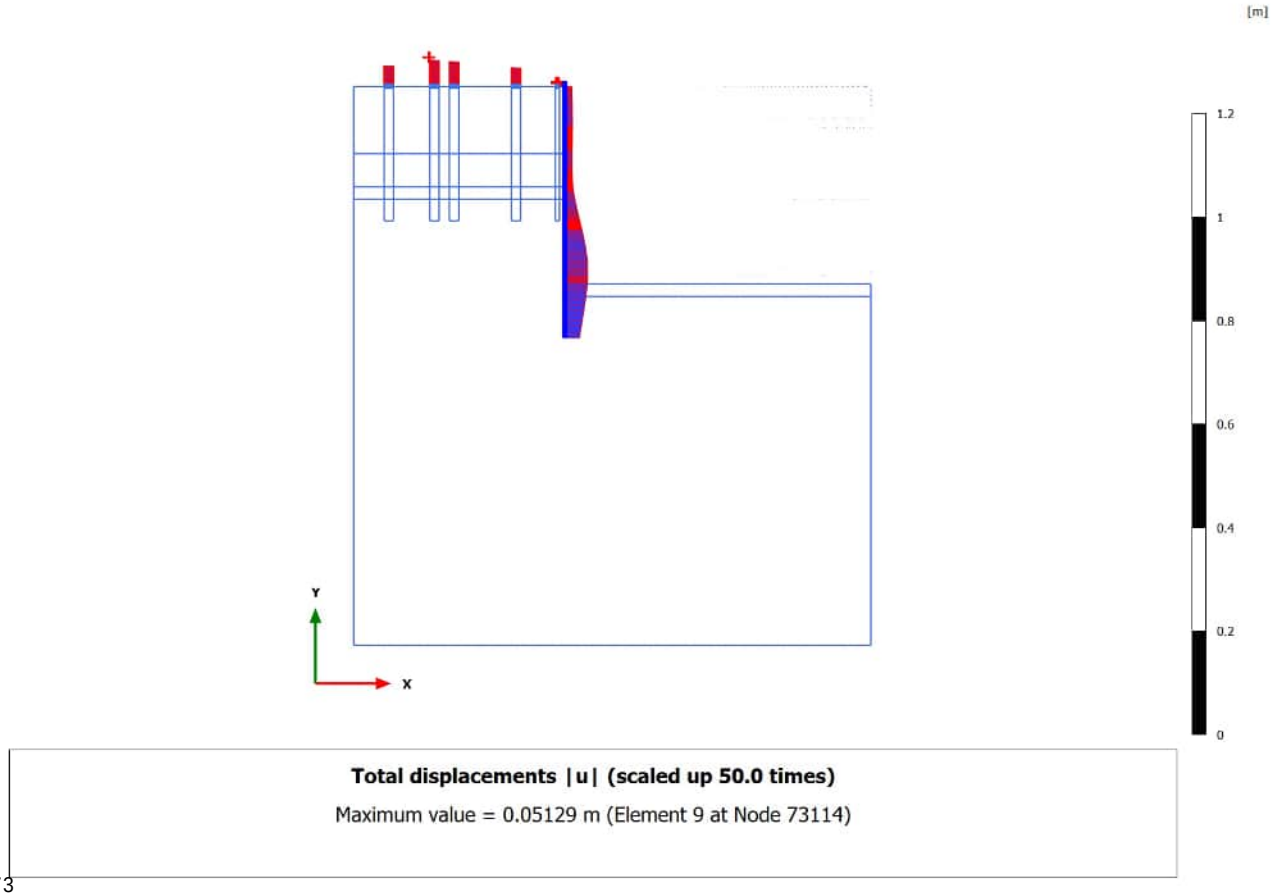
3.1.1.1.1.10 Calculation results, Plate, Phase_11 [Phase_11] (11/218), Total displacements |u|



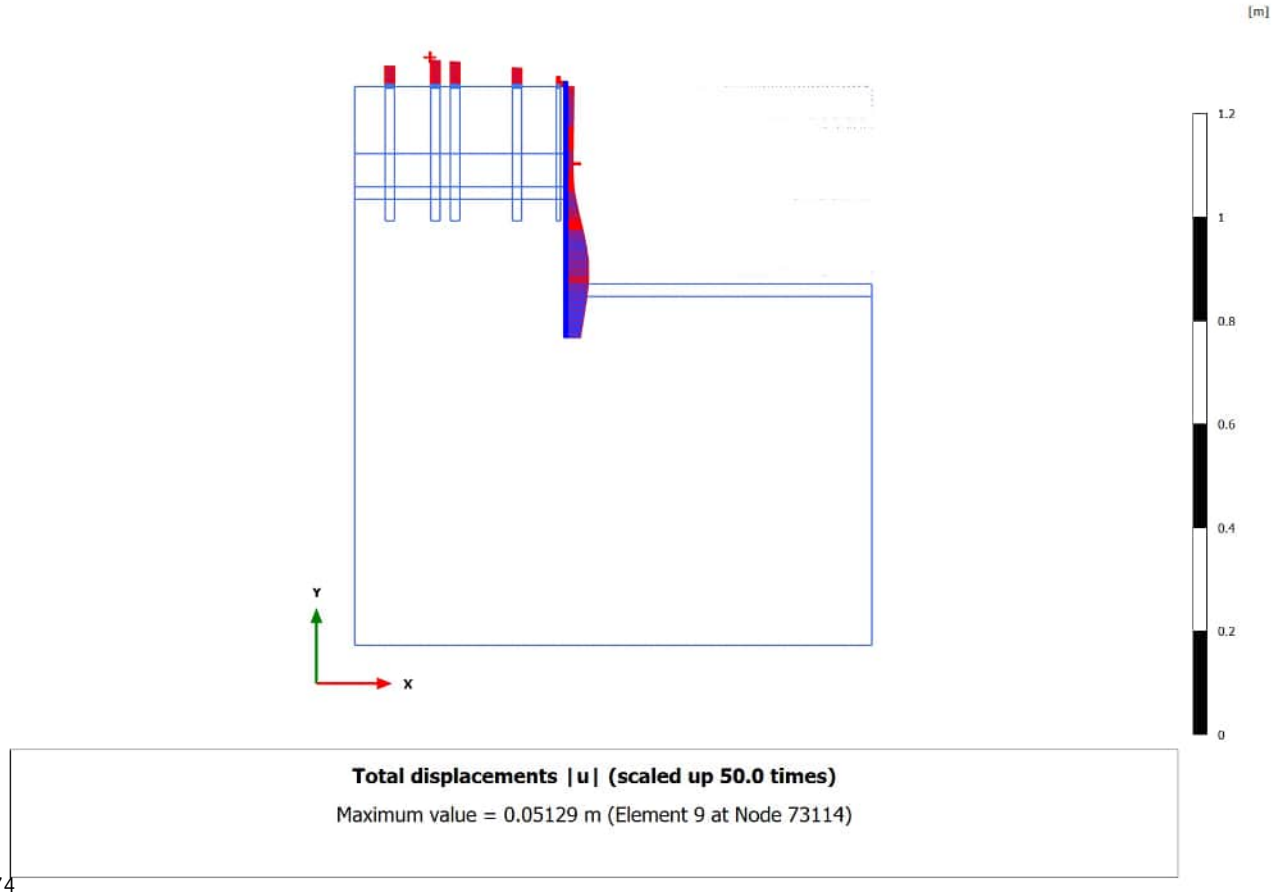
3.1.1.1.1.11 Calculation results, Plate, Phase_12 [Phase_12] (12/221), Total displacements |u|



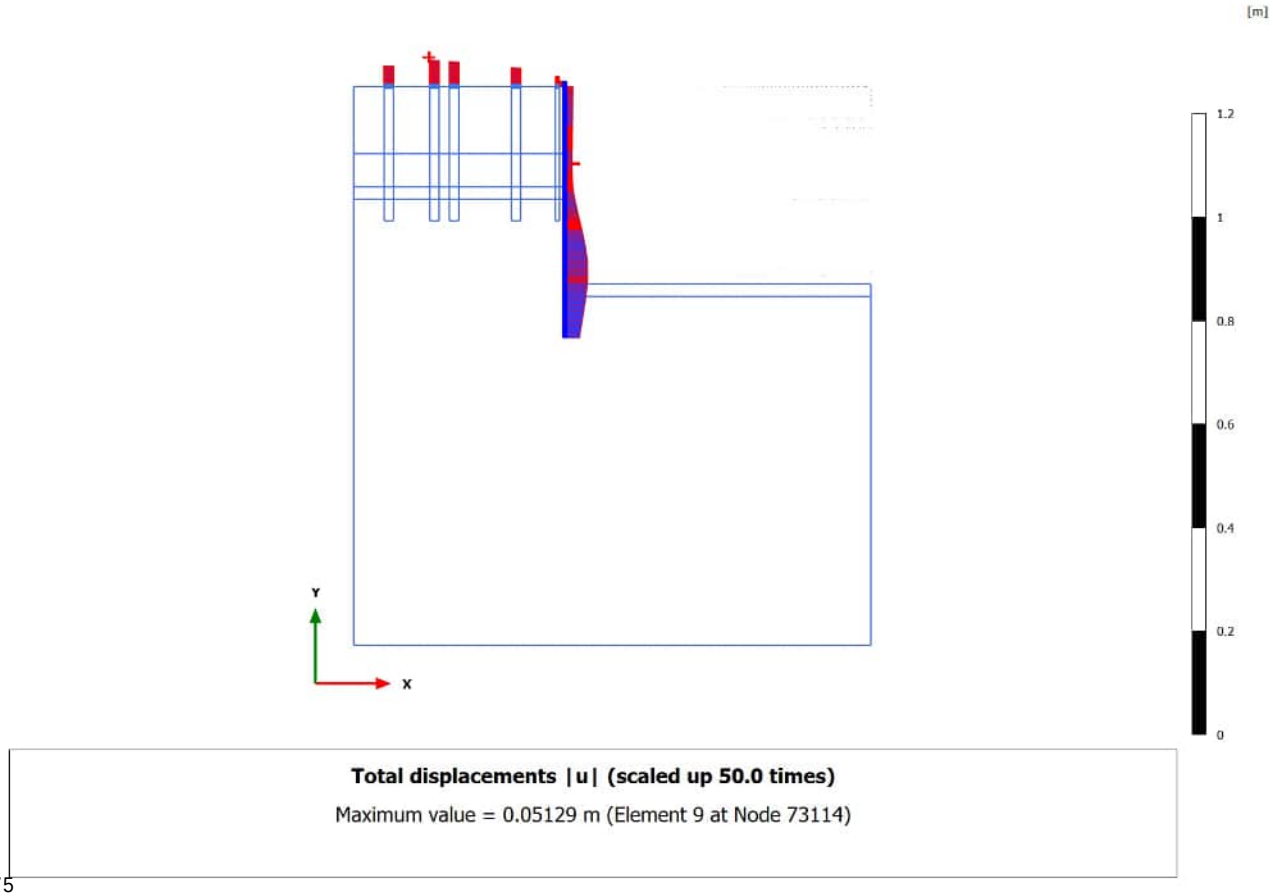
3.1.1.1.1.12 Calculation results, Plate, Phase_13 [Phase_13] (13/226), Total displacements |u|



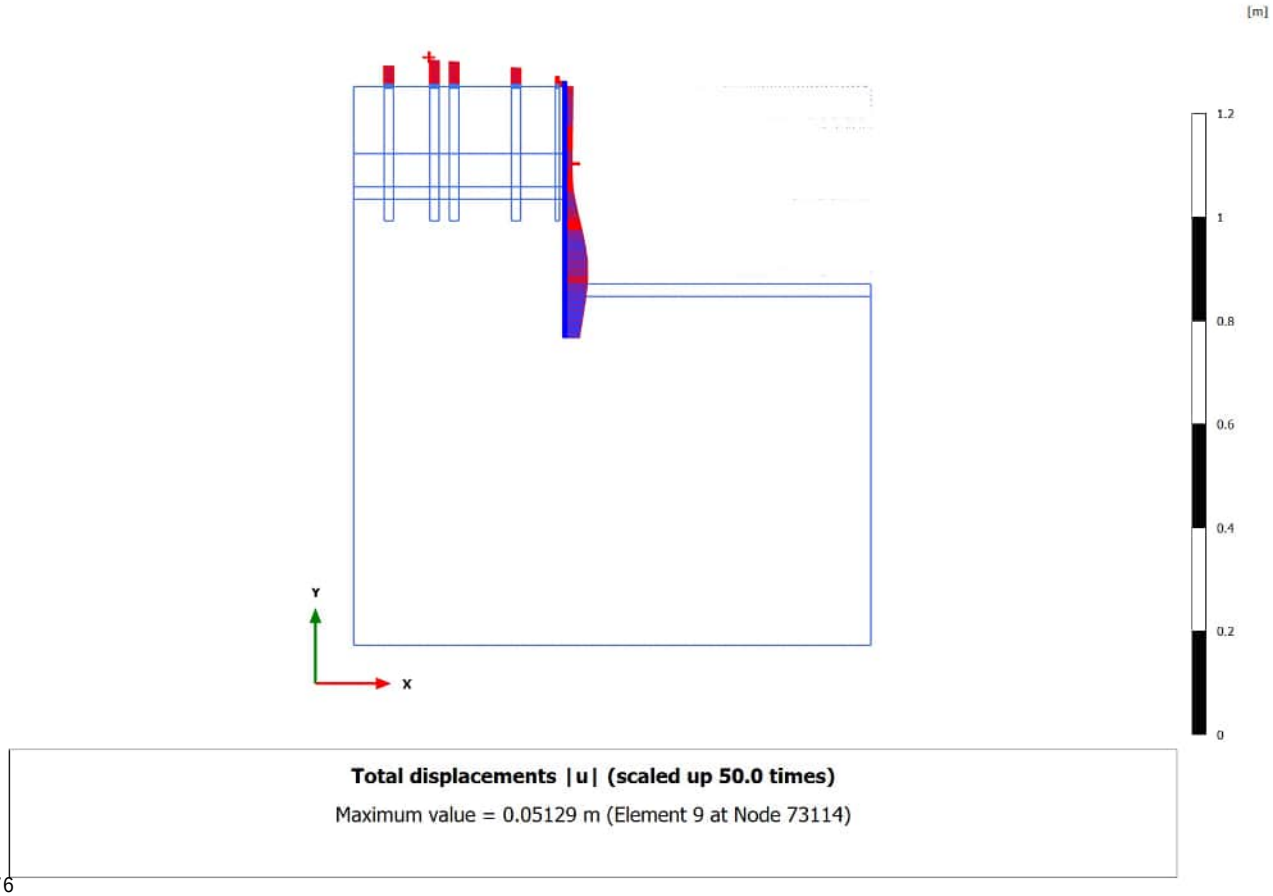
3.1.1.1.1.13 Calculation results, Plate, Phase_14 [Phase_14] (14/229), Total displacements |u|



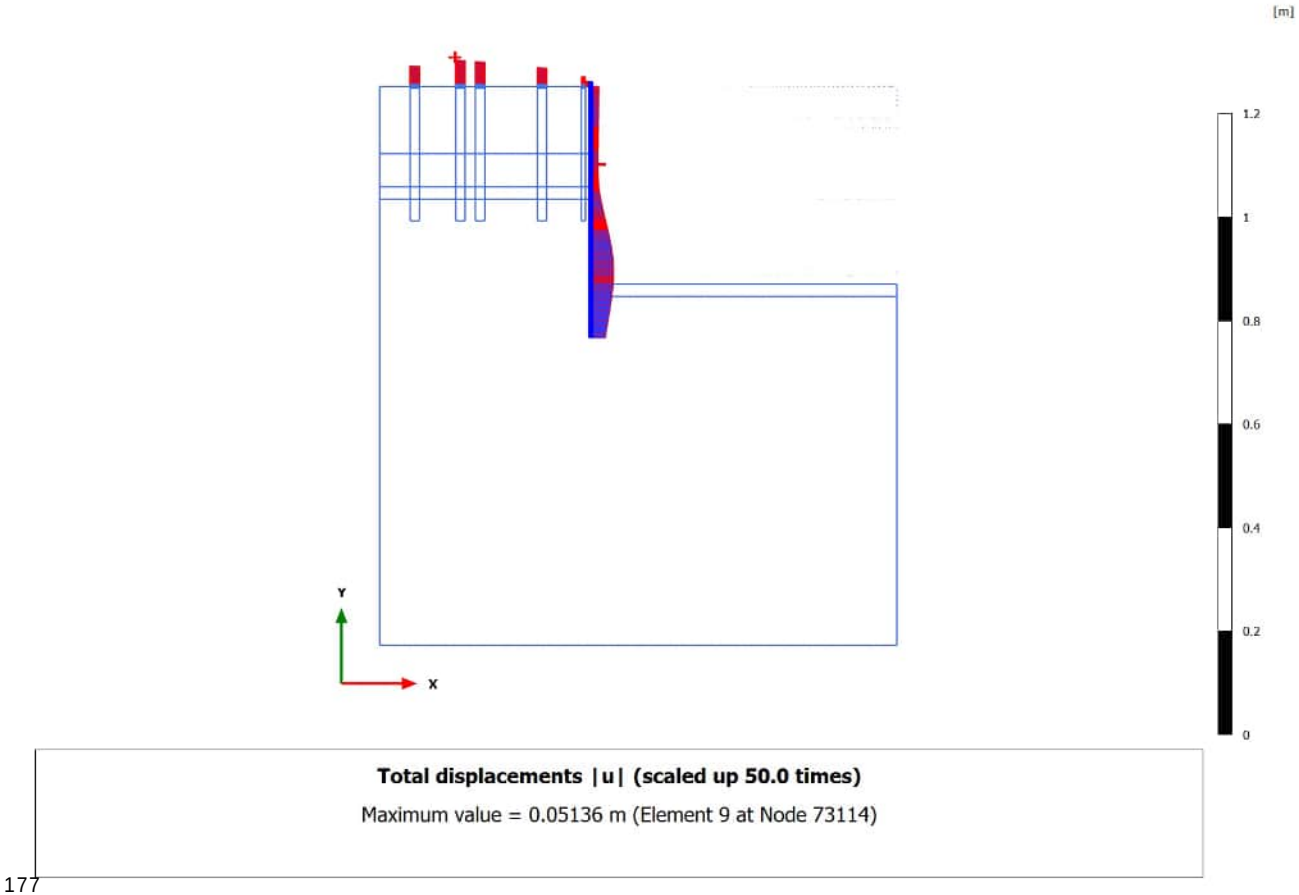
3.1.1.1.1.14 Calculation results, Plate, Phase_15 [Phase_15] (15/245), Total displacements |u|



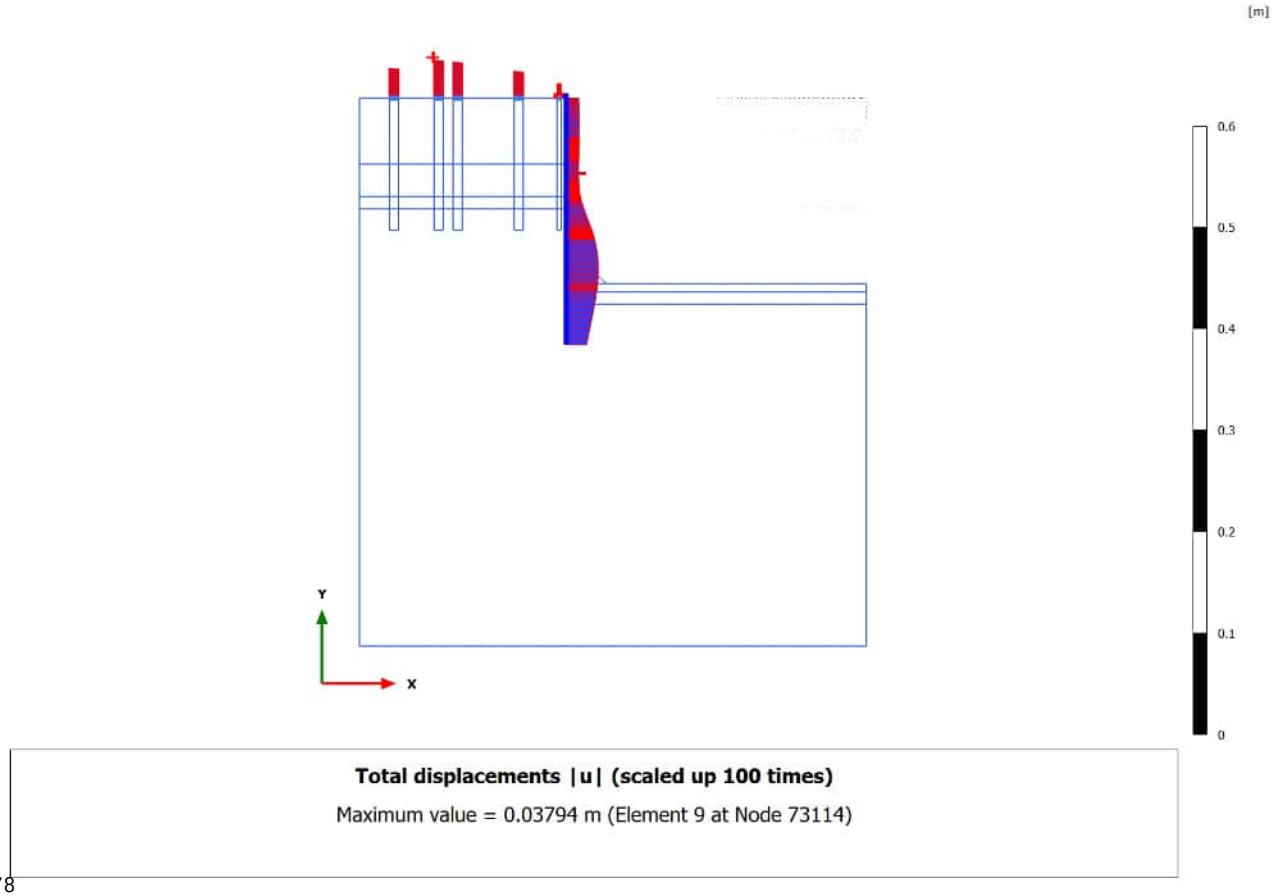
3.1.1.1.1.15 Calculation results, Plate, Phase_16 [Phase_16] (16/248), Total displacements |u|



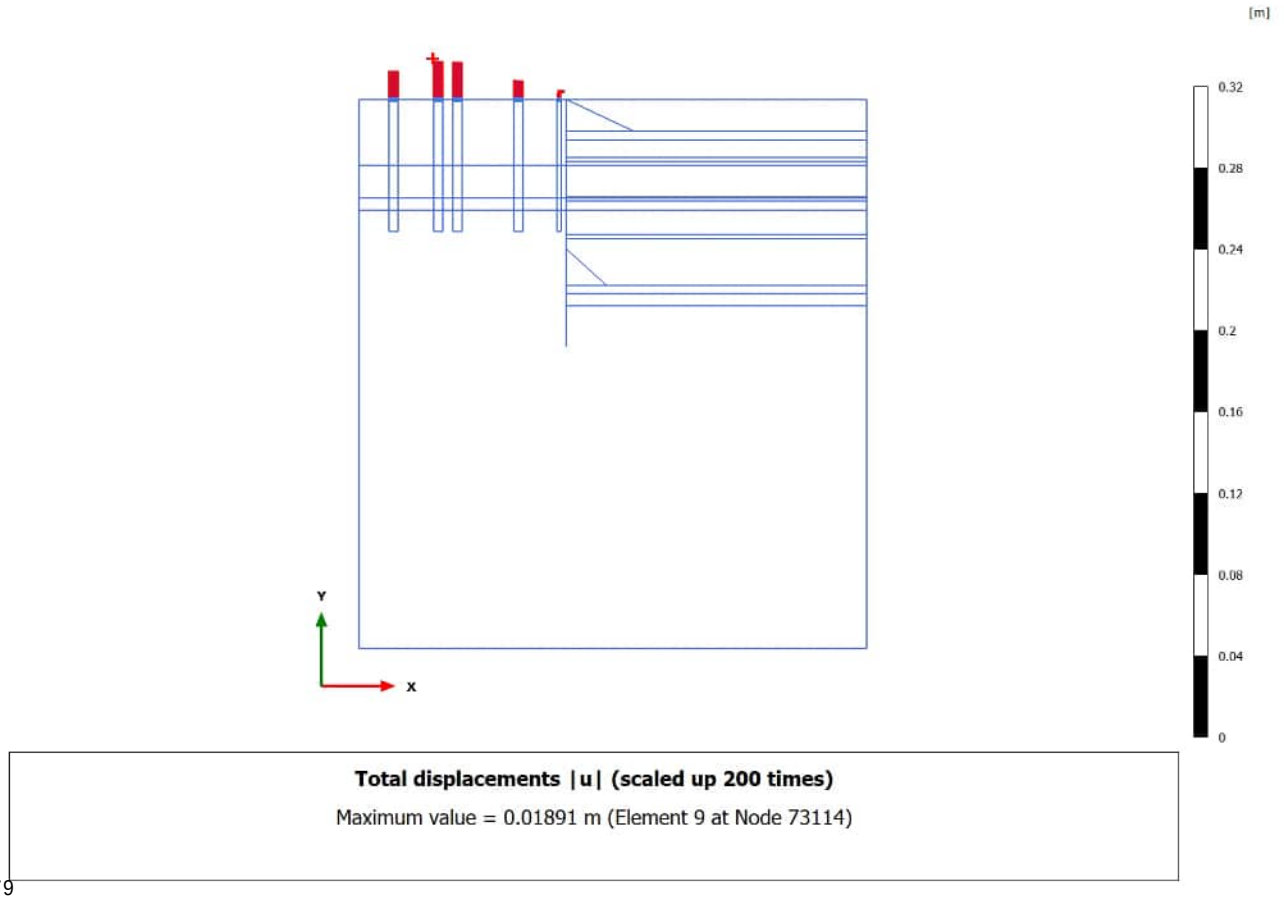
3.1.1.1.1.16 Calculation results, Plate, Phase_17 [Phase_17] (17/253), Total displacements |u|



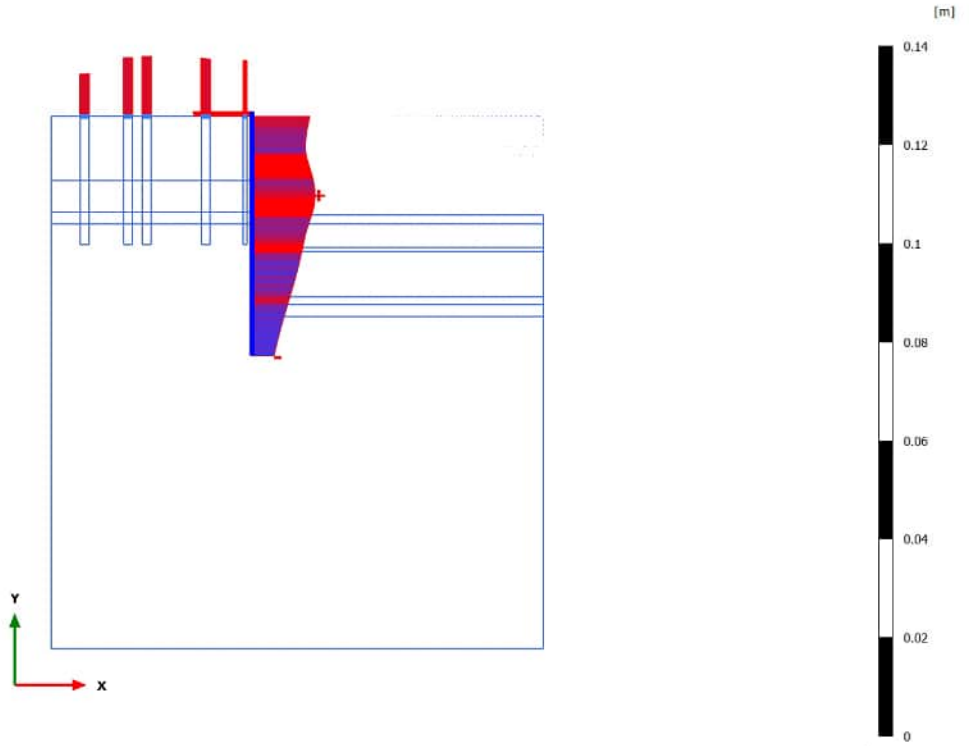
3.1.1.1.1.17 Calculation results, Plate, Phase_18 [Phase_18] (18/258), Total displacements |u|



3.1.1.1.1.18 Calculation results, Plate, Phase_1 [Phase_1] (1/310), Total displacements |u|



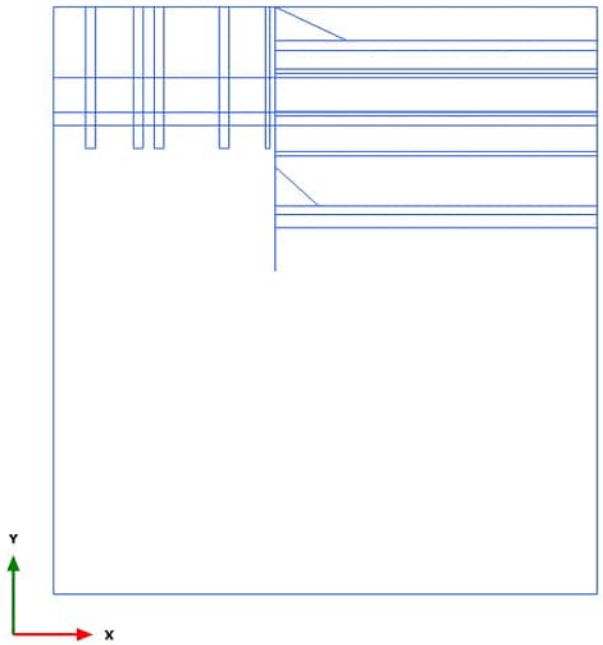
3.1.1.1.1.19 Calculation results, Plate, Phase_7 [Phase_7] (7/339), Total displacements |u|



Total displacements |u| (scaled up 500 times)

Maximum value = 0.01284 m (Element 48 at Node 32363)

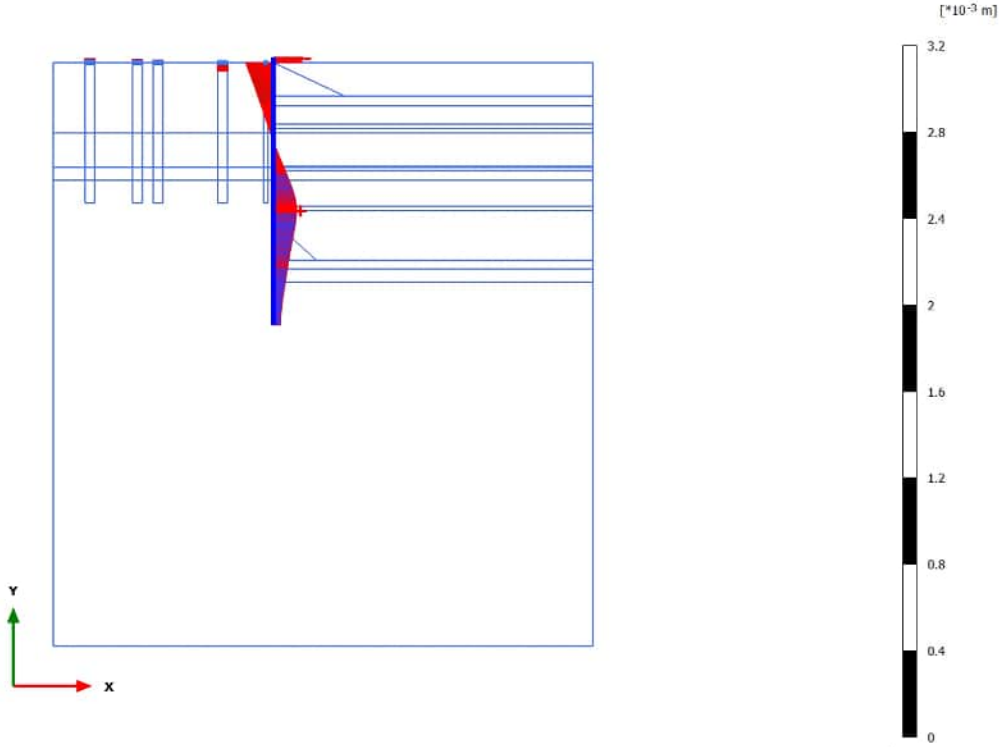
3.1.1.1.2.1 Calculation results, Plate, Initial phase [InitialPhase] (0/0), Total displacements u_x



Total displacements u_x (at true scale)

No results

3.1.1.1.2.2 Calculation results, Plate, Phase_2 [Phase_2] (2/3), Total displacements u_x

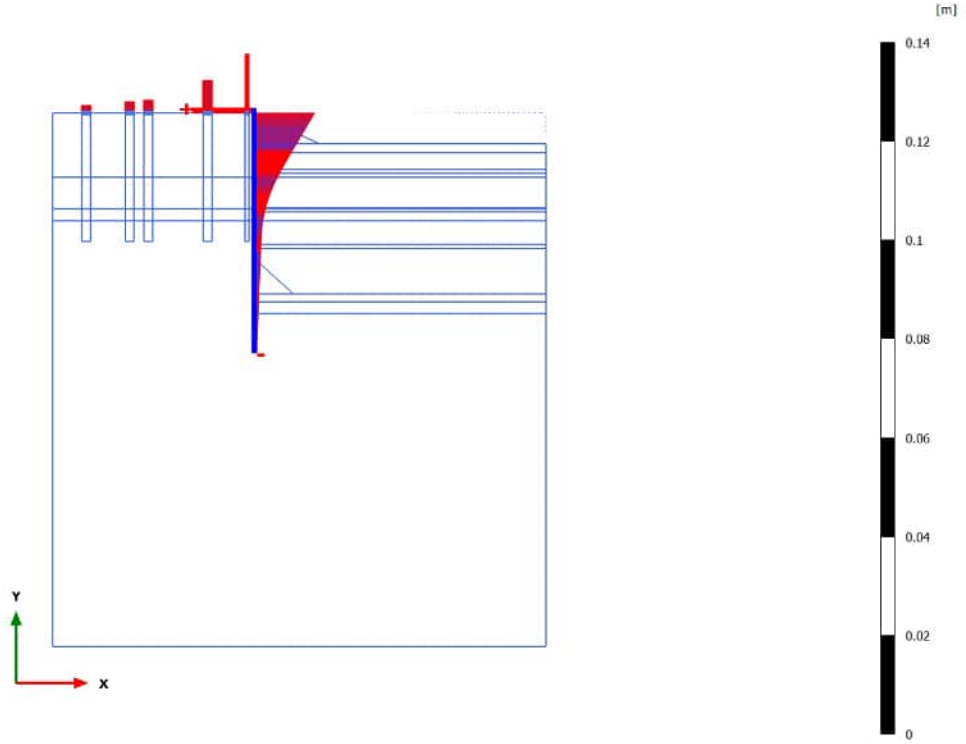


Total displacements u_x (scaled up 20.0*10³ times)

Maximum value = 0.1084*10-3 m (Element 67 at Node 51140)

Minimum value = -0.1387*10-3 m (Element 1 at Node 25713)

3.1.1.1.2.3 Calculation results, Plate, Phase_3 [Phase_3] (3/10), Total displacements u_x

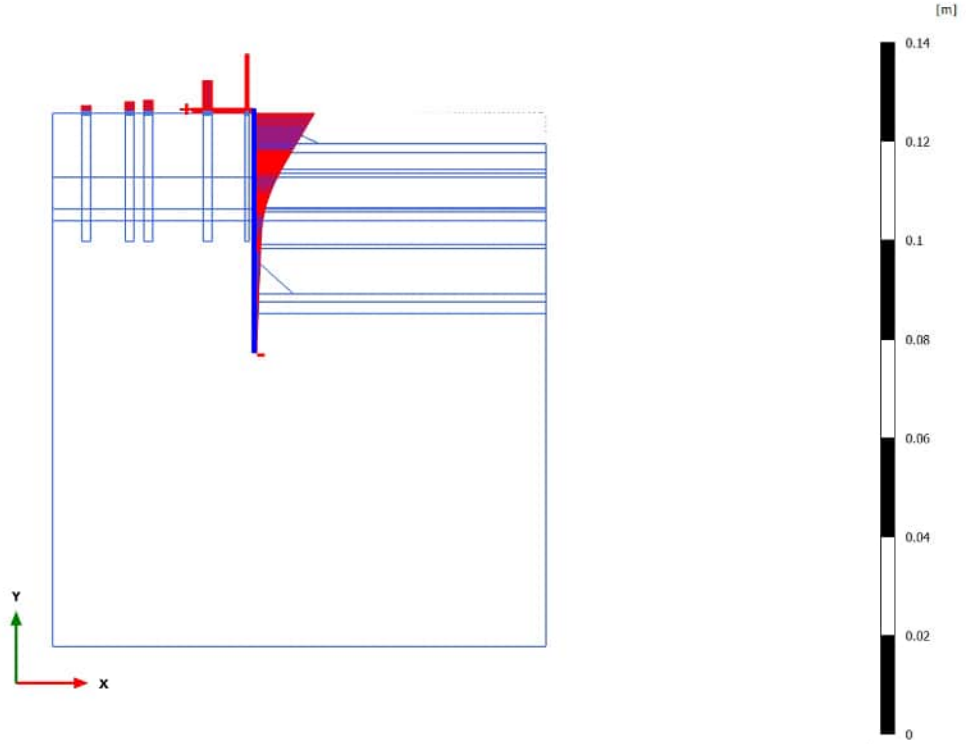


Total displacements u_x (scaled up 500 times)

Maximum value = 0.01293 m (Element 1 at Node 25713)

Minimum value = $0.6162*10^{-3}$ m (Element 78 at Node 58394)

3.1.1.1.2.4 Calculation results, Plate, Phase_4 [Phase_4] (4/34), Total displacements u_x

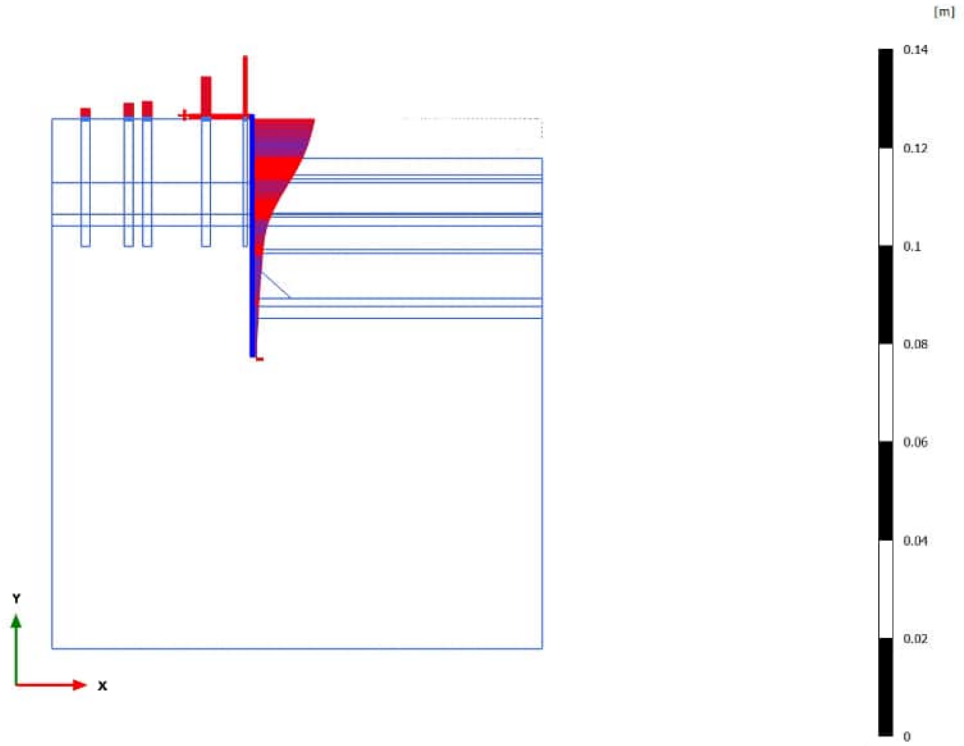


Total displacements u_x (scaled up 500 times)

Maximum value = 0.01293 m (Element 1 at Node 25713)

Minimum value = $0.6166*10^{-3}$ m (Element 78 at Node 58394)

3.1.1.1.2.5 Calculation results, Plate, Phase_5 [Phase_5] (5/42), Total displacements u_x

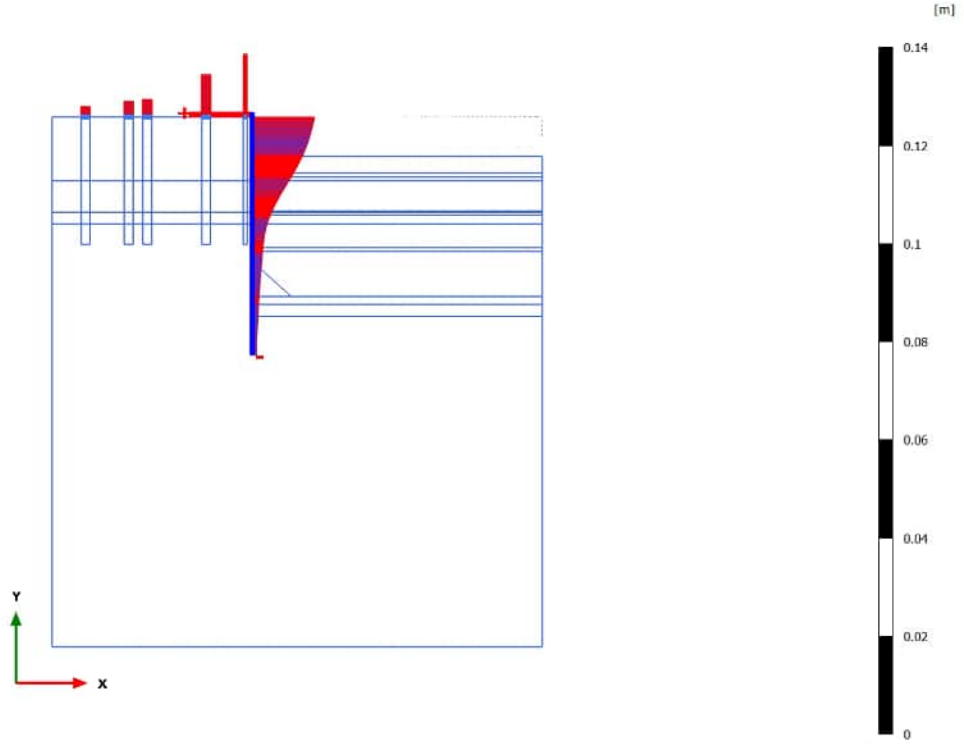


Total displacements u_x (scaled up 500 times)

Maximum value = 0.01301 m (Element 1 at Node 25713)

Minimum value = $0.8594*10^{-3}$ m (Element 78 at Node 58394)

3.1.1.1.2.6 Calculation results, Plate, Phase_6 [Phase_6] (6/89), Total displacements u_x

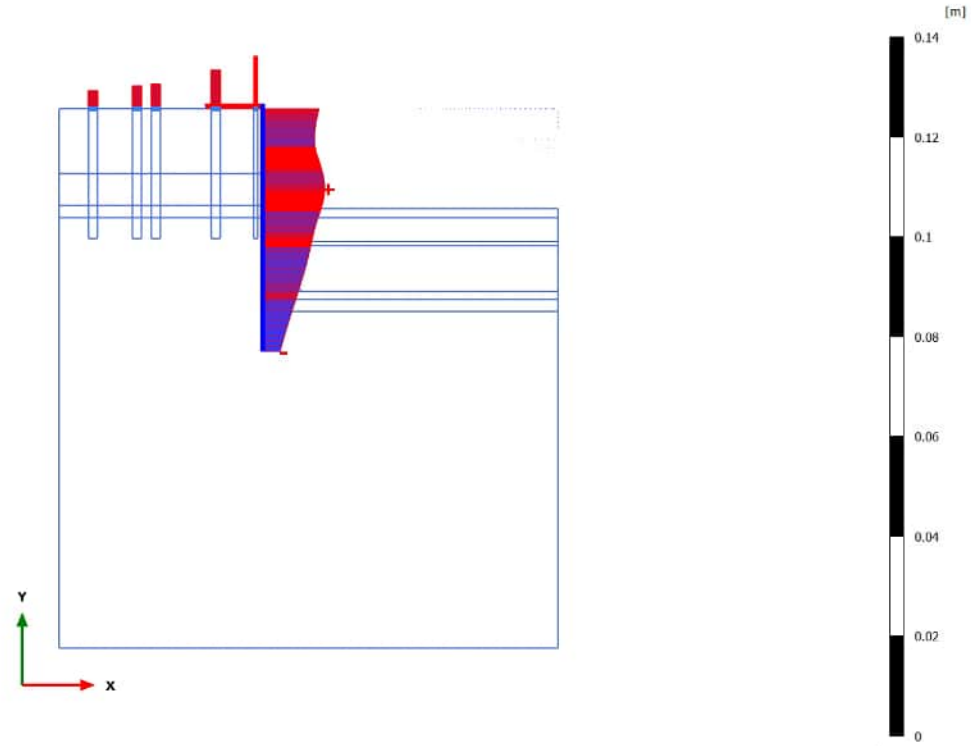


Total displacements u_x (scaled up 500 times)

Maximum value = 0.01301 m (Element 1 at Node 25713)

Minimum value = $0.8597*10^{-3}$ m (Element 78 at Node 58394)

3.1.1.1.2.7 Calculation results, Plate, Phase_8 [Phase_8] (8/91), Total displacements u_x

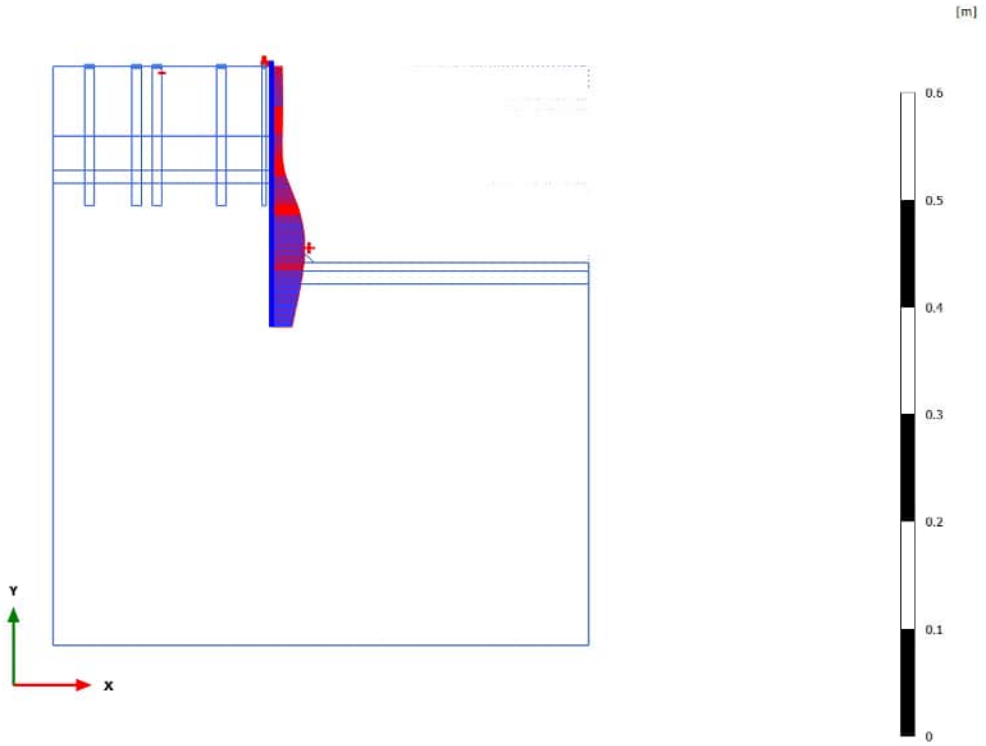


Total displacements u_x (scaled up 500 times)

Maximum value = 0.01246 m (Element 48 at Node 32363)

Minimum value = $3.513*10^{-3}$ m (Element 78 at Node 58394)

3.1.1.1.2.8 Calculation results, Plate, Phase_9 [Phase_9] (9/166), Total displacements u_x

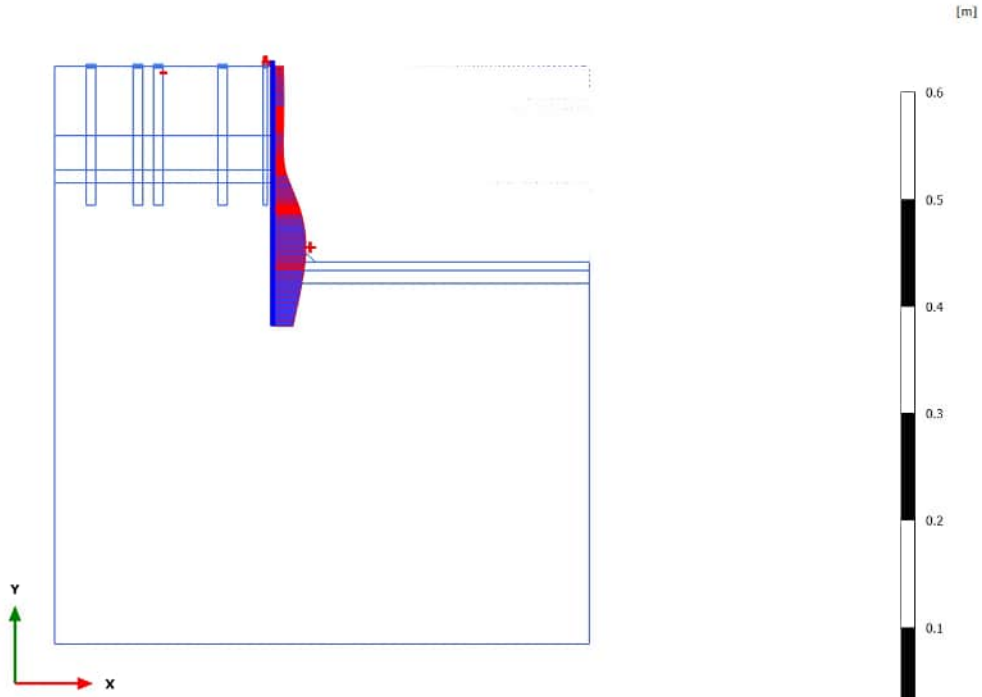


Total displacements u_x (scaled up 100 times)

Maximum value = 0.03110 m (Element 71 at Node 55523)

Minimum value = $-1.247*10^{-3}$ m (Element 19 at Node 61424)

3.1.1.1.2.9 Calculation results, Plate, Phase_10 [Phase_10] (10/168), Total displacements u_x

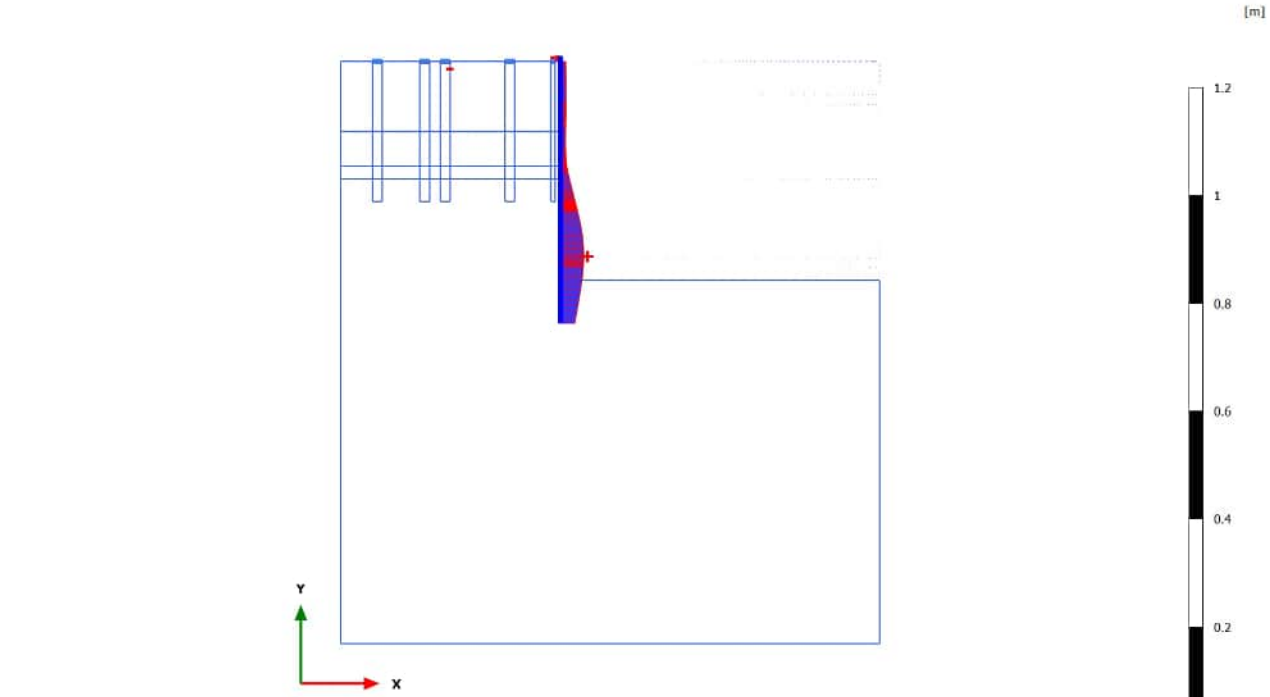


Total displacements u_x (scaled up 100 times)

Maximum value = 0.03110 m (Element 71 at Node 55523)

Minimum value = $-1.247*10^{-3}$ m (Element 19 at Node 61424)

3.1.1.1.2.10 Calculation results, Plate, Phase_11 [Phase_11] (11/218), Total displacements u_x

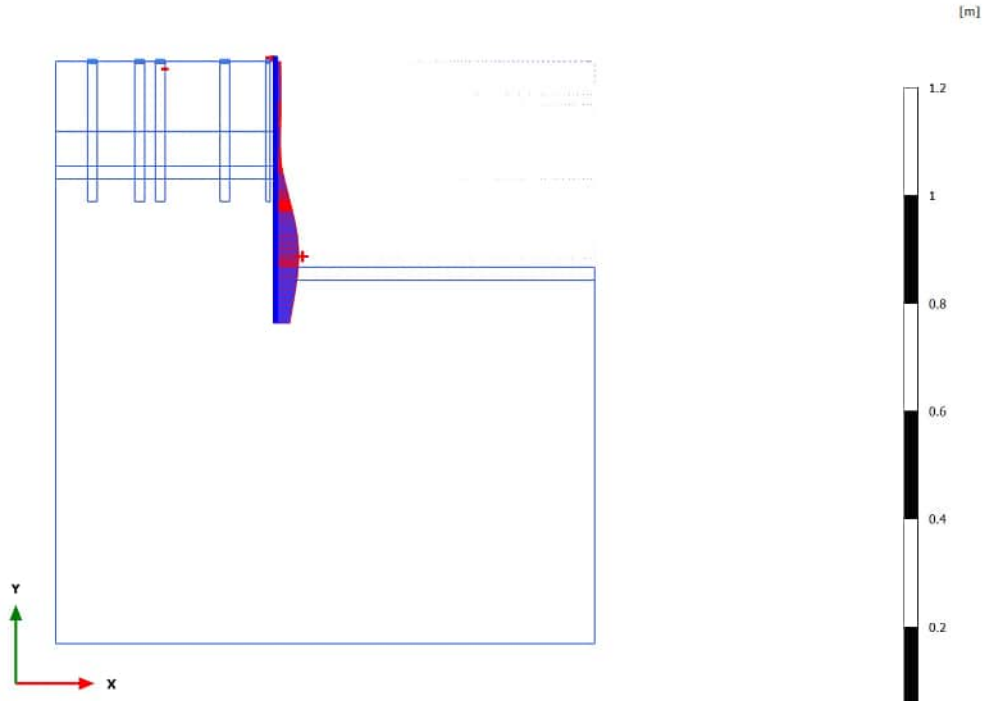


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.04333 m (Element 73 at Node 55747)

Minimum value = $-4.360*10^{-3}$ m (Element 19 at Node 61424)

3.1.1.1.2.11 Calculation results, Plate, Phase_12 [Phase_12] (12/221), Total displacements u_x

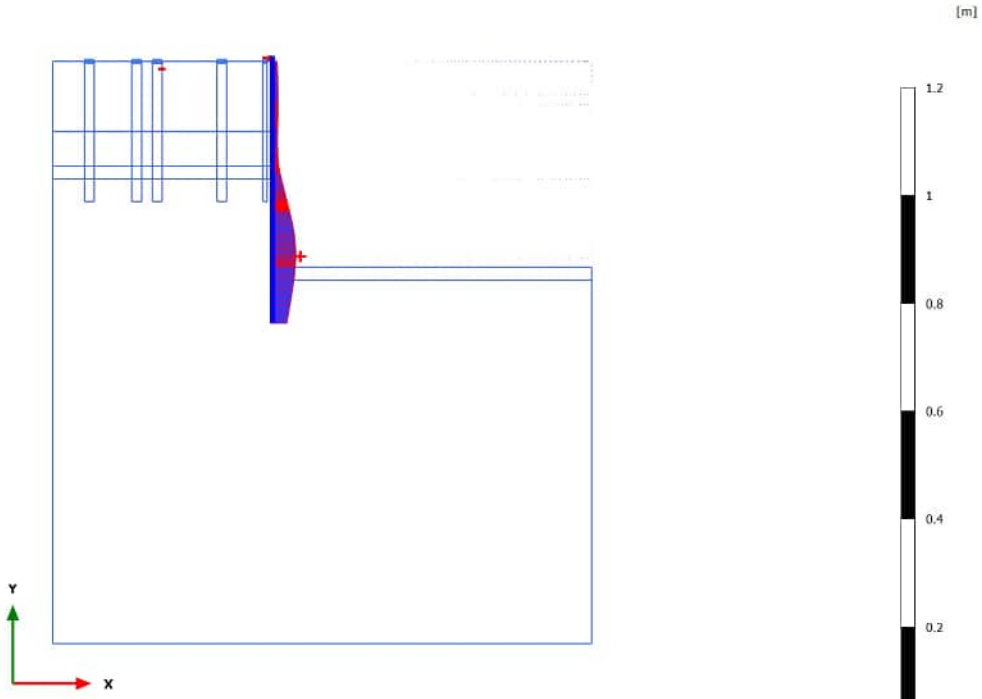


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.04335 m (Element 73 at Node 55747)

Minimum value = $-4.343*10^{-3}$ m (Element 19 at Node 61424)

3.1.1.1.2.12 Calculation results, Plate, Phase_13 [Phase_13] (13/226), Total displacements u_x

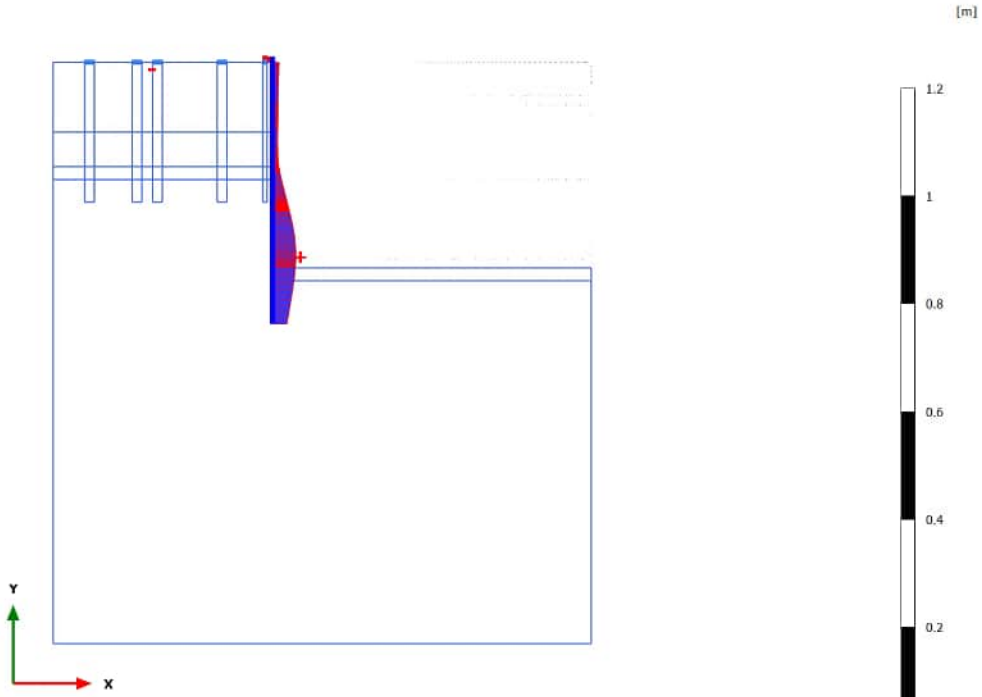


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.04343 m (Element 73 at Node 55747)

Minimum value = $-4.263*10^{-3}$ m (Element 19 at Node 61426)

3.1.1.1.2.13 Calculation results, Plate, Phase_14 [Phase_14] (14/229), Total displacements u_x

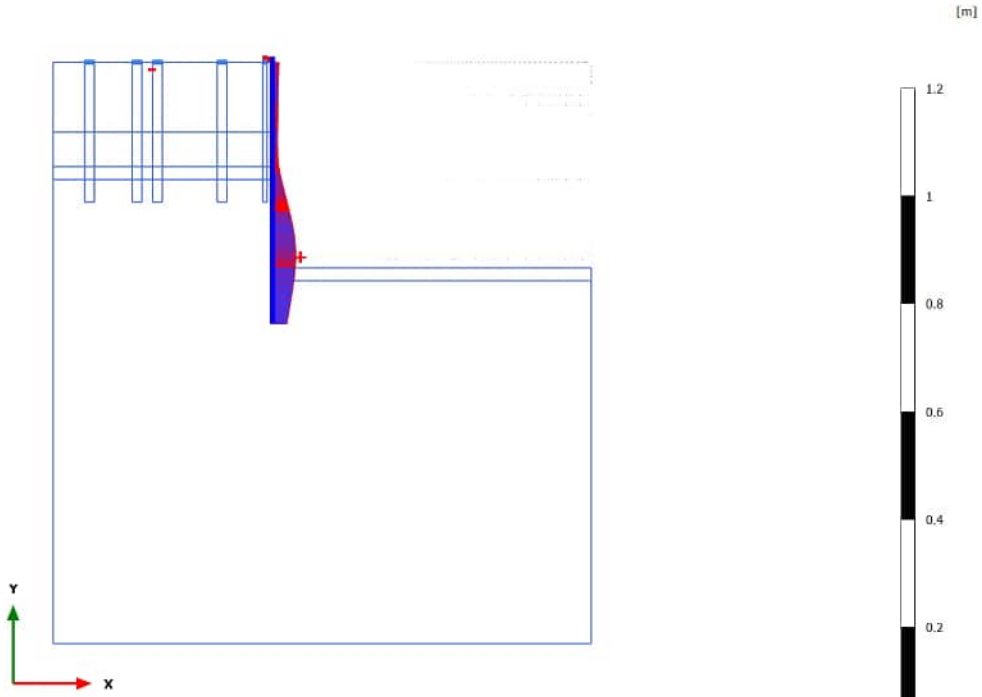


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.04343 m (Element 73 at Node 55747)

Minimum value = $-4.066*10^{-3}$ m (Element 15 at Node 66225)

3.1.1.1.2.14 Calculation results, Plate, Phase_15 [Phase_15] (15/245), Total displacements u_x

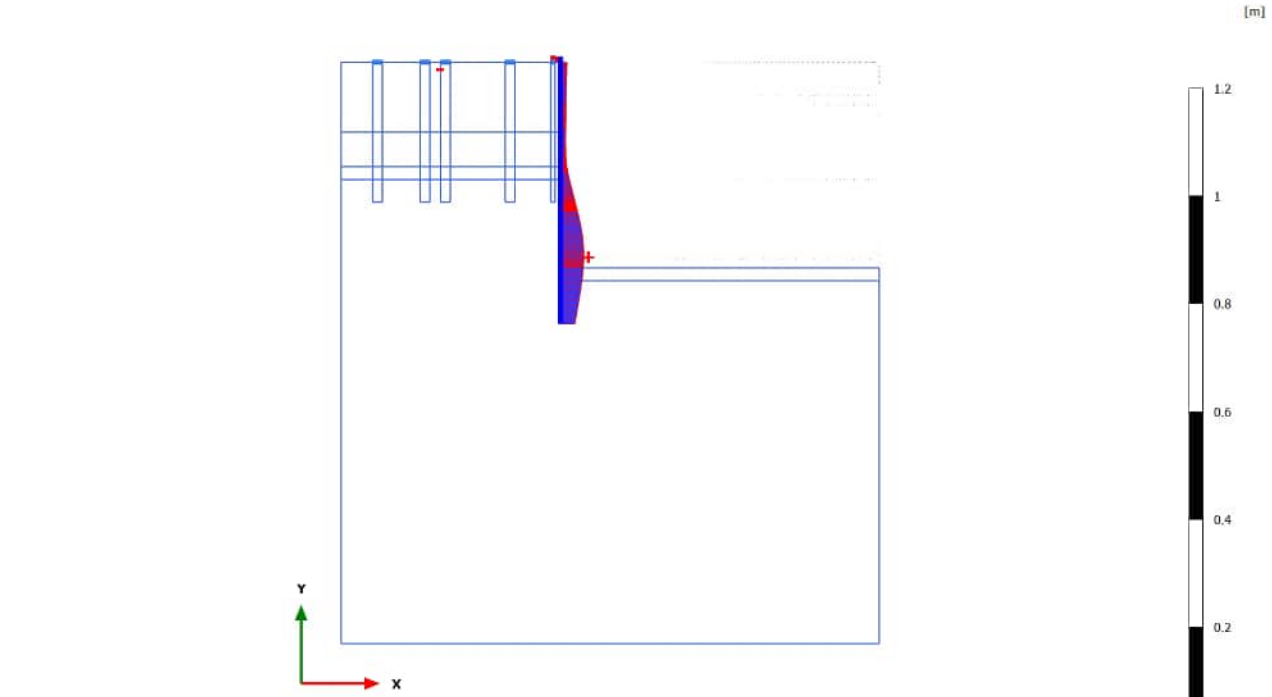


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.04343 m (Element 73 at Node 55747)

Minimum value = $-4.068*10^{-3}$ m (Element 15 at Node 66225)

3.1.1.1.2.15 Calculation results, Plate, Phase_16 [Phase_16] (16/248), Total displacements u_x

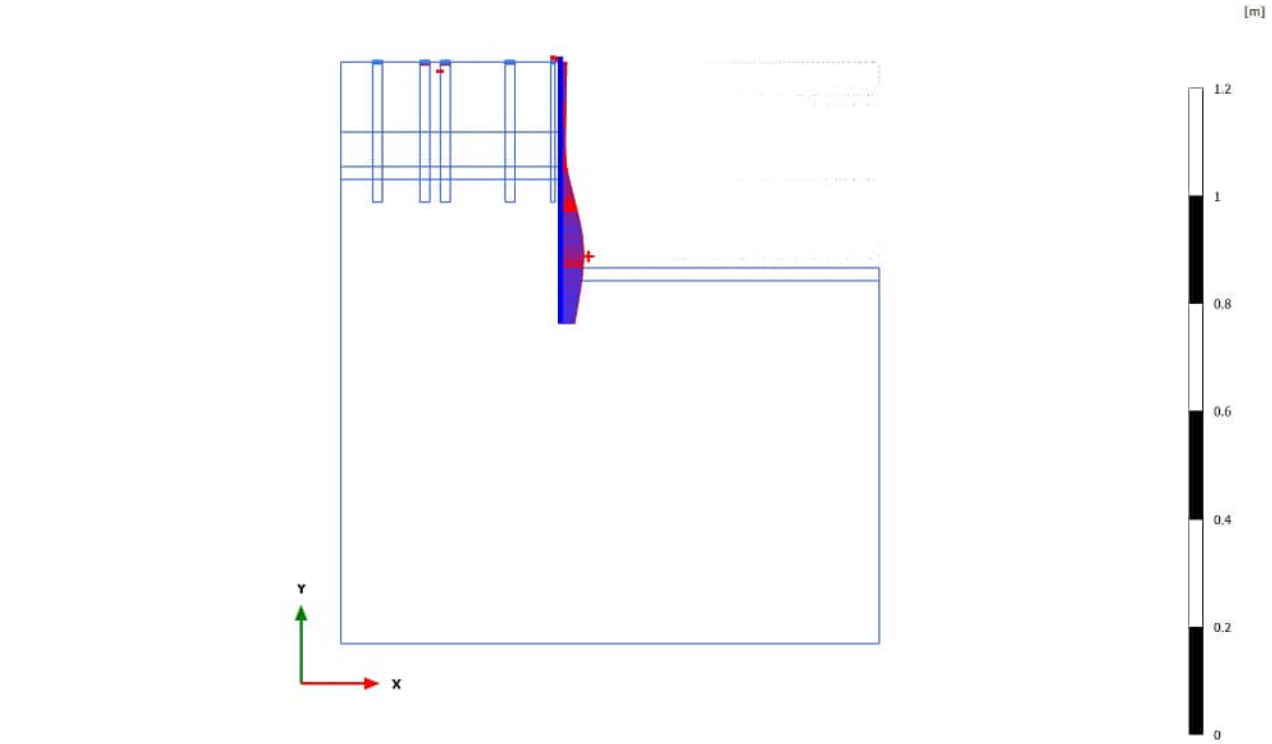


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.04343 m (Element 73 at Node 55747)

Minimum value = $-4.068*10^{-3}$ m (Element 15 at Node 66225)

3.1.1.1.2.16 Calculation results, Plate, Phase_17 [Phase_17] (17/253), Total displacements u_x

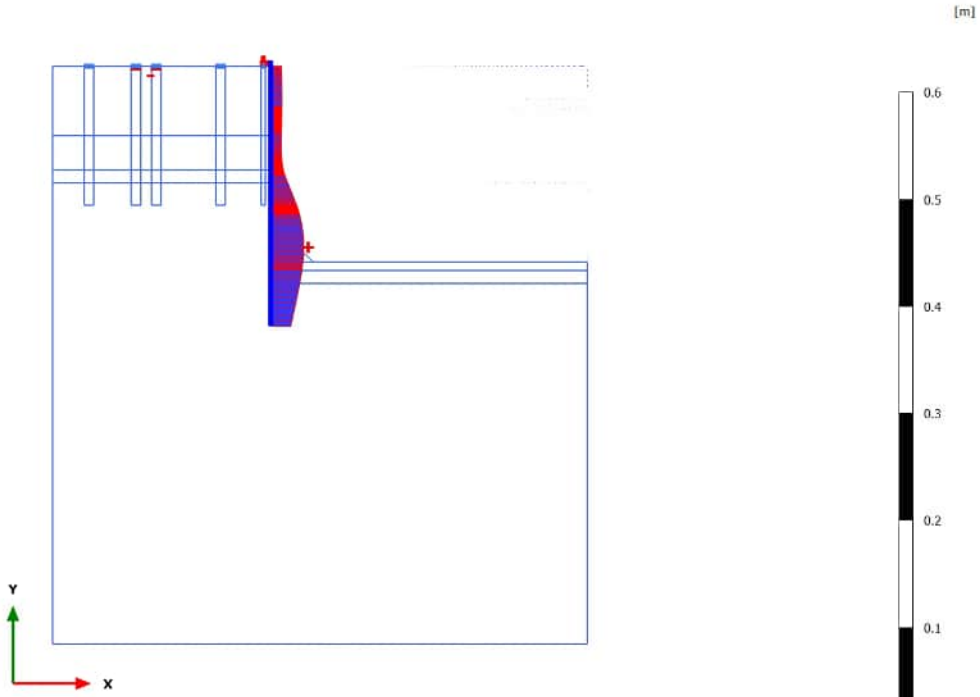


Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.04344 m (Element 73 at Node 55747)

Minimum value = $-6.281*10^{-3}$ m (Element 15 at Node 66225)

3.1.1.1.2.17 Calculation results, Plate, Phase_18 [Phase_18] (18/258), Total displacements u_x

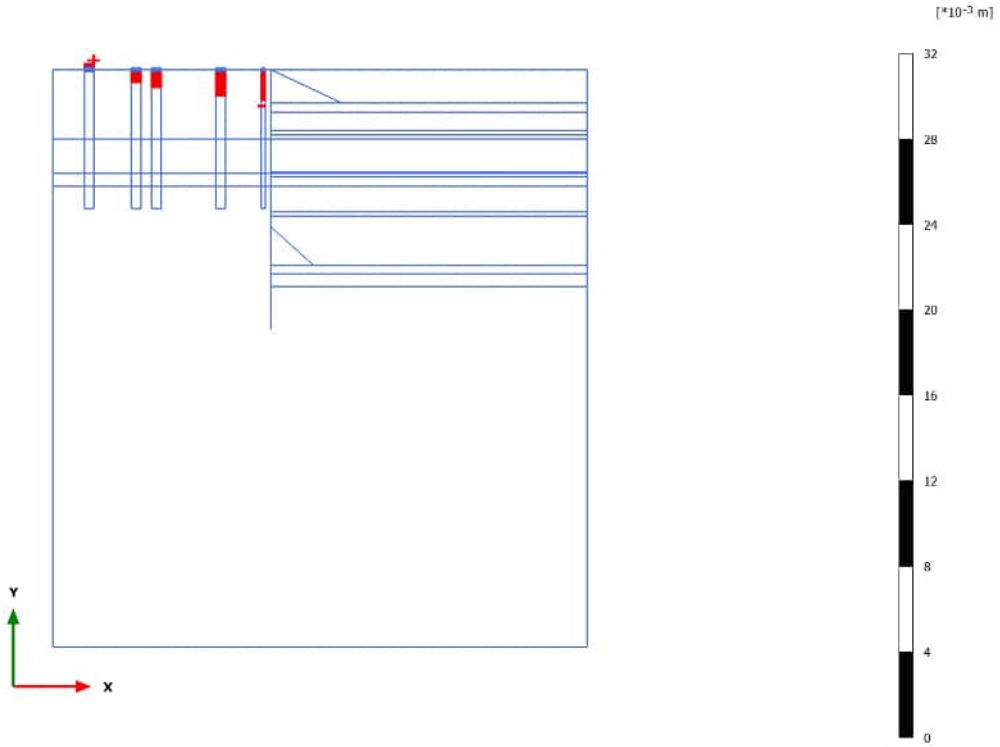


Total displacements u_x (scaled up 100 times)

Maximum value = 0.03095 m (Element 71 at Node 55523)

Minimum value = $-3.830*10^{-3}$ m (Element 15 at Node 66225)

3.1.1.1.2.18 Calculation results, Plate, Phase_1 [Phase_1] (1/310), Total displacements u_x

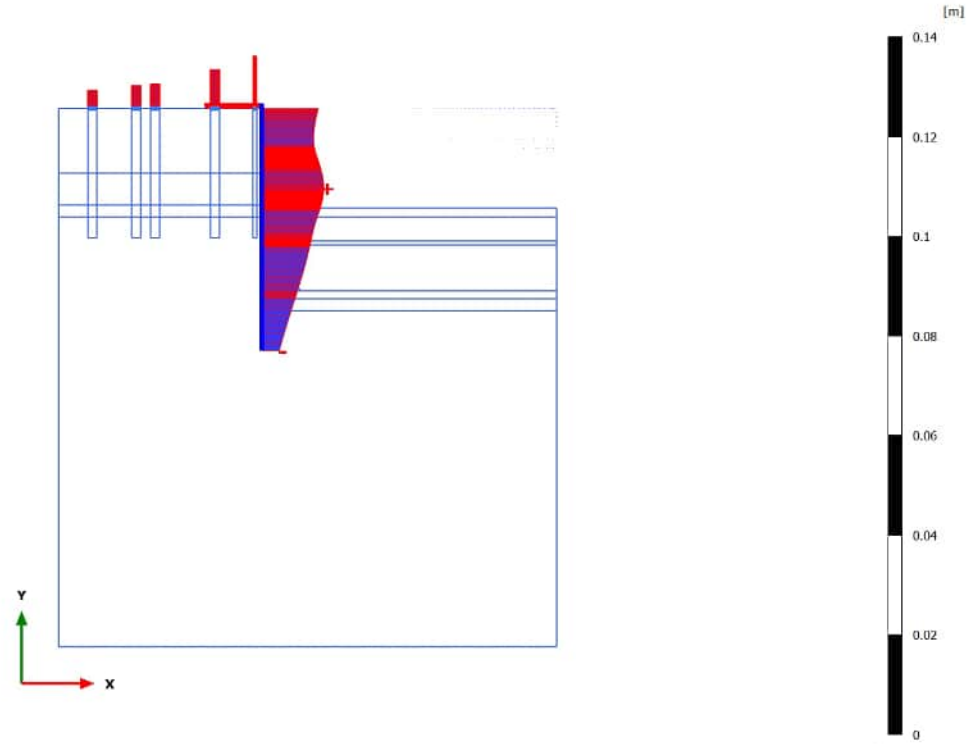


Total displacements u_x (scaled up 2.00*10³ times)

Maximum value = 0.3126*10^{-3} m (Element 8 at Node 79654)

Minimum value = -1.462*10^{-3} m (Element 27 at Node 25771)

3.1.1.1.2.19 Calculation results, Plate, Phase_7 [Phase_7] (7/339), Total displacements u_x

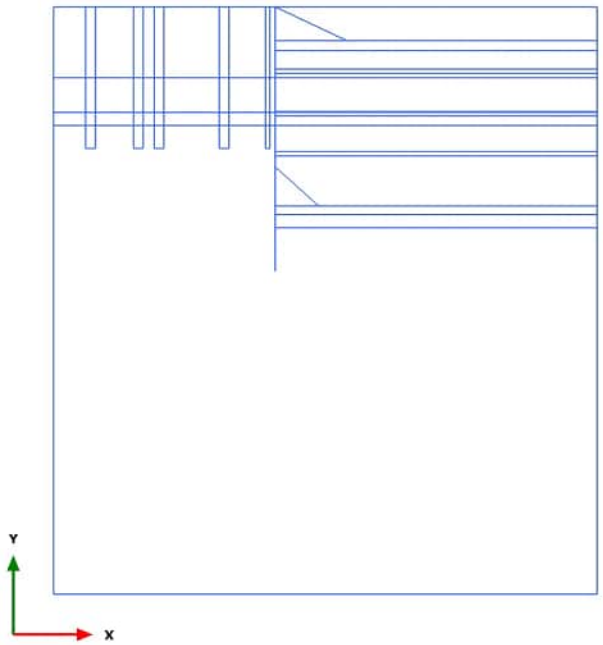


Total displacements u_x (scaled up 500 times)

Maximum value = 0.01246 m (Element 48 at Node 32363)

Minimum value = $3.512*10^{-3}$ m (Element 78 at Node 58394)

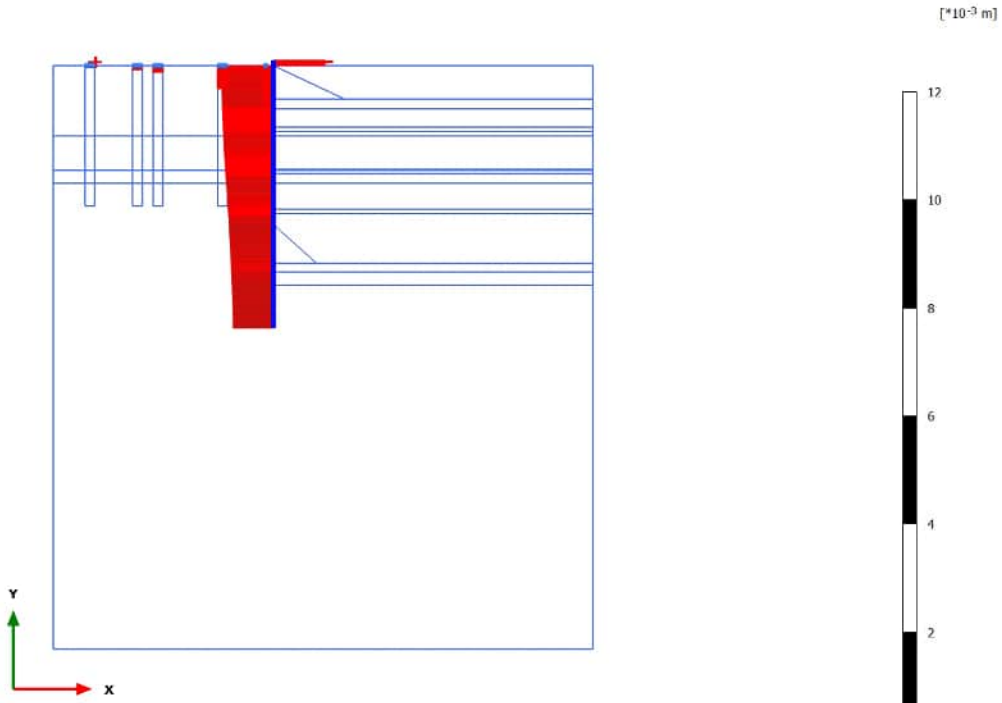
3.1.1.1.3.1 Calculation results, Plate, Initial phase [InitialPhase] (0/0), Total displacements u_y



Total displacements u_y (at true scale)

No results

3.1.1.1.3.2 Calculation results, Plate, Phase_2 [Phase_2] (2/3), Total displacements u_y

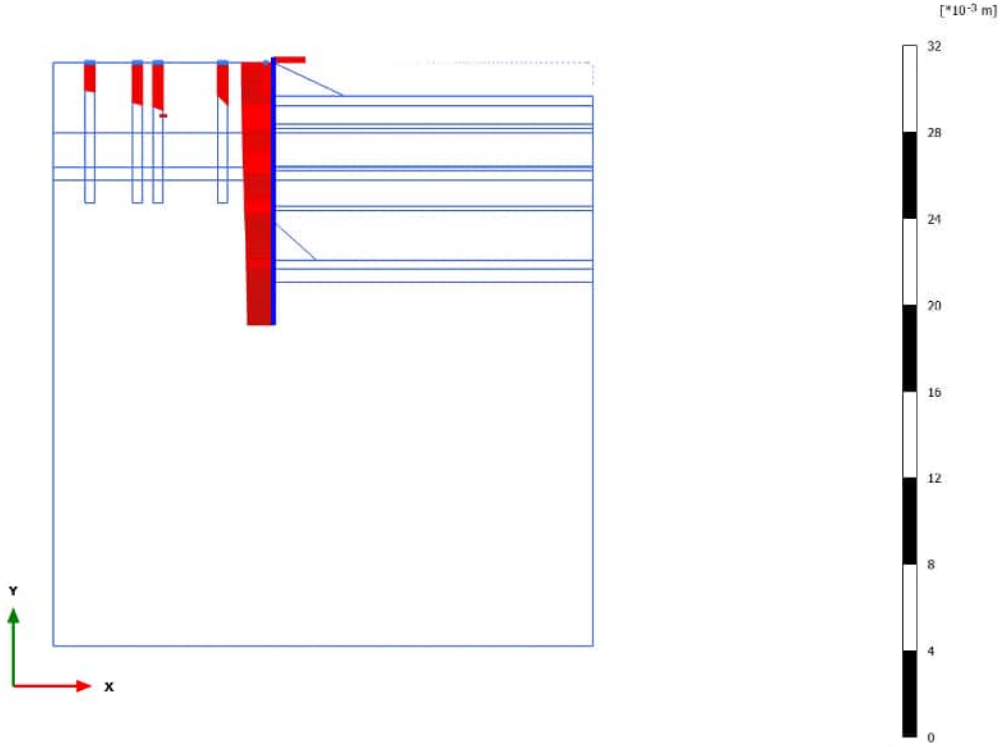


Total displacements u_y (scaled up $5.00*10^3$ times)

Maximum value = $5.199*10^{-6}$ m (Element 8 at Node 79710)

Minimum value = $-0.9596*10^{-3}$ m (Element 1 at Node 25713)

3.1.1.1.3.3 Calculation results, Plate, Phase_3 [Phase_3] (3/10), Total displacements u_y

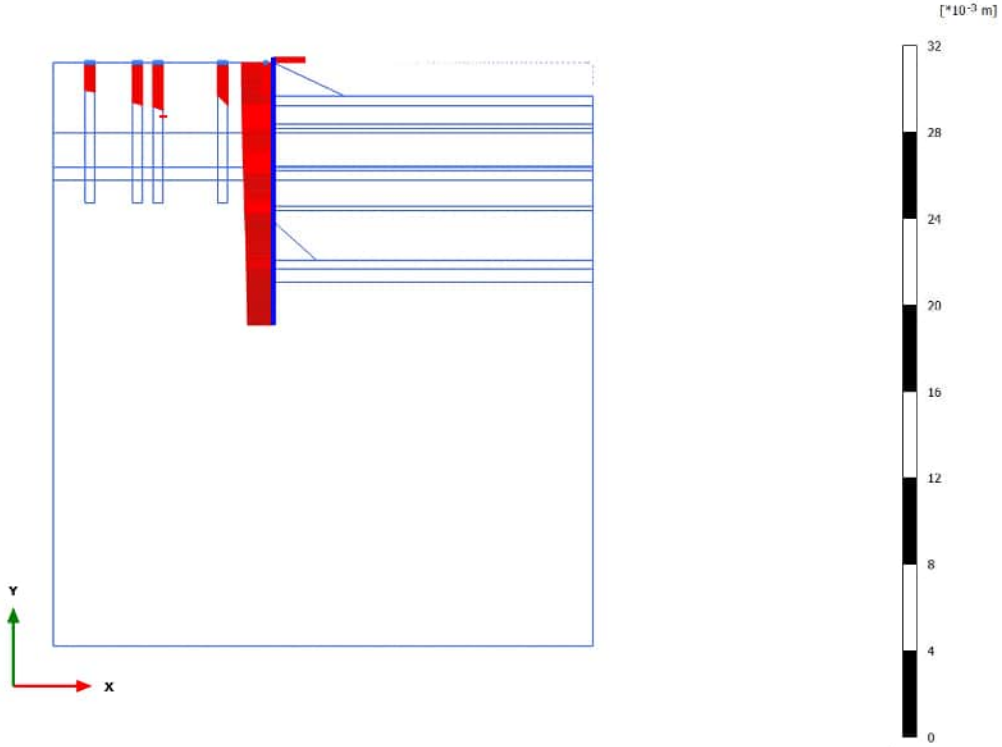


Total displacements u_y (scaled up 2.00*10³ times)

Maximum value = -1.132*10-3 m (Element 27 at Node 25771)

Minimum value = -2.223*10-3 m (Element 20 at Node 60207)

3.1.1.1.3.4 Calculation results, Plate, Phase_4 [Phase_4] (4/34), Total displacements u_y

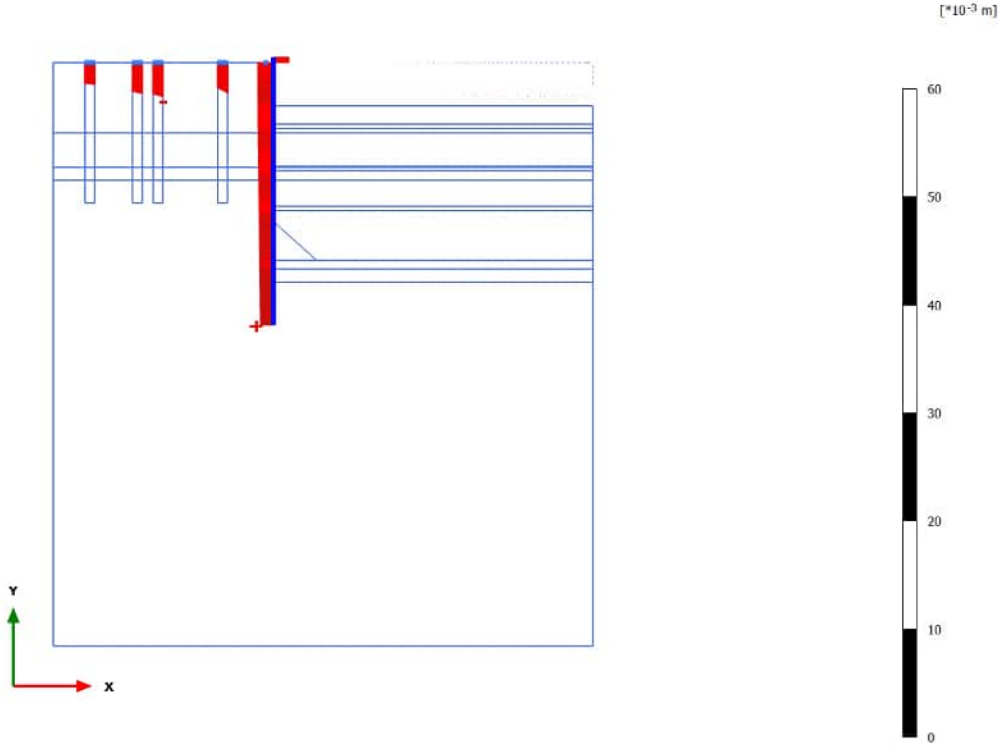


Total displacements u_y (scaled up 2.00*10³ times)

Maximum value = -1.134*10-3 m (Element 27 at Node 25771)

Minimum value = -2.223*10-3 m (Element 20 at Node 60207)

3.1.1.1.3.5 Calculation results, Plate, Phase_5 [Phase_5] (5/42), Total displacements u_y

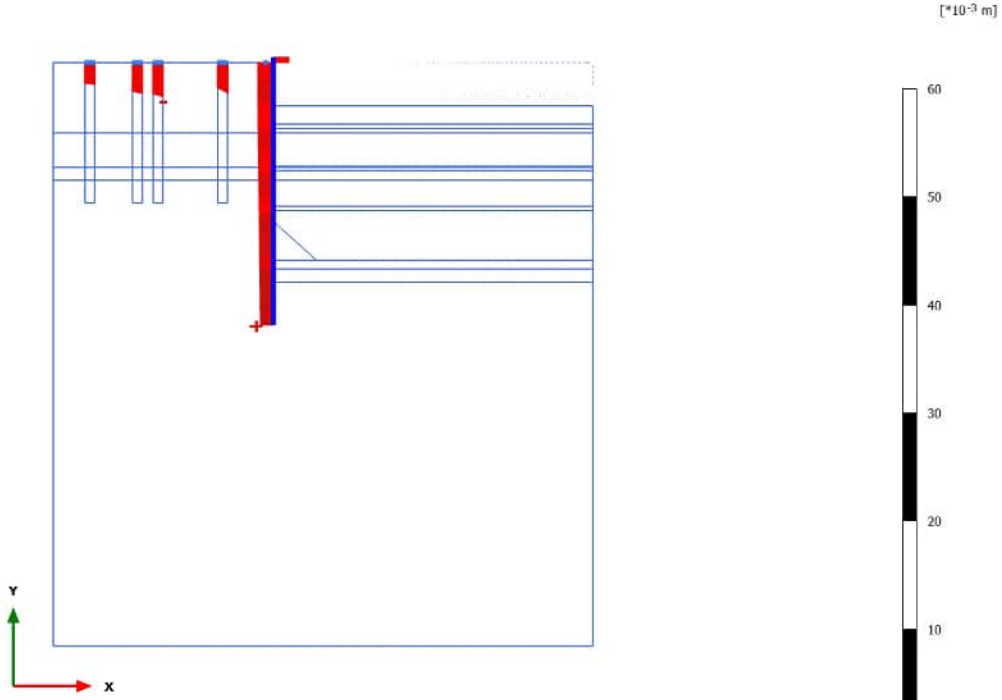


Total displacements u_y (scaled up 1.00*10³ times)

Maximum value = -1.222*10-3 m (Element 78 at Node 58394)

Minimum value = -3.161*10-3 m (Element 20 at Node 60207)

3.1.1.1.3.6 Calculation results, Plate, Phase_6 [Phase_6] (6/89), Total displacements u_y

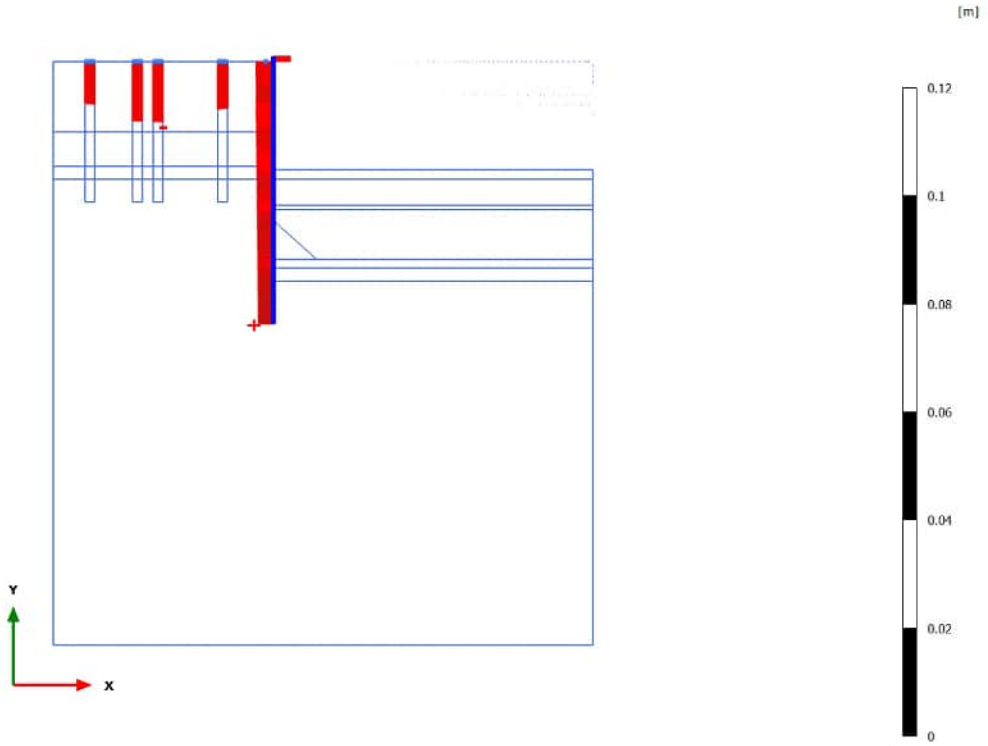


Total displacements u_y (scaled up 1.00*10³ times)

Maximum value = -1.222*10-3 m (Element 78 at Node 58394)

Minimum value = -3.161*10-3 m (Element 20 at Node 60207)

3.1.1.1.3.7 Calculation results, Plate, Phase_8 [Phase_8] (8/91), Total displacements u_y

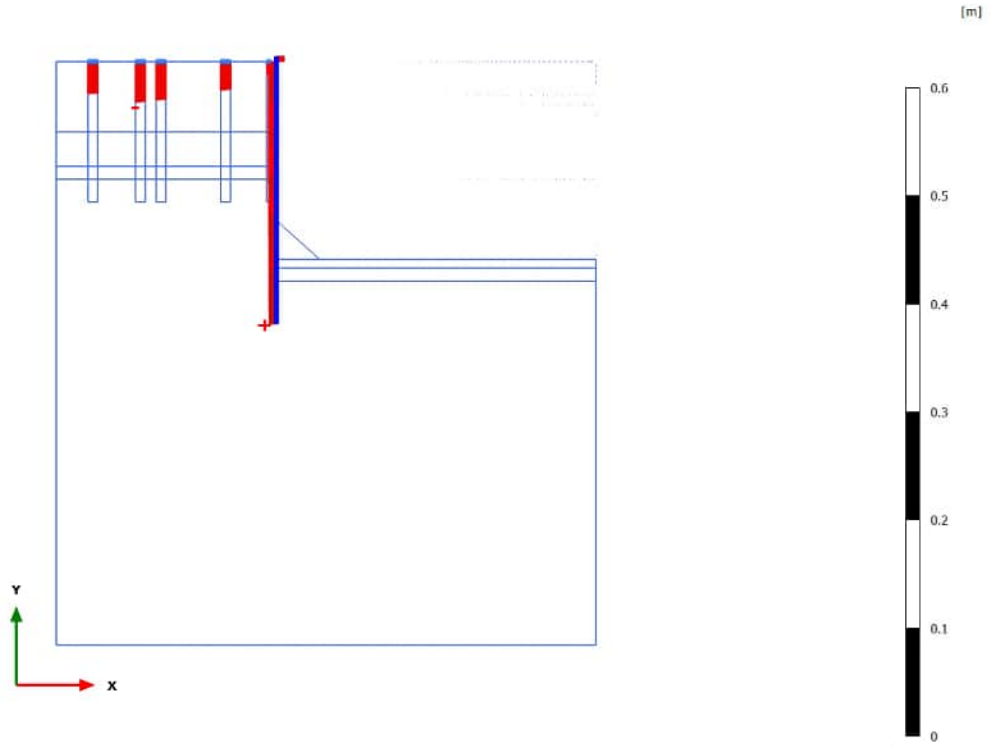


Total displacements u_y (scaled up 500 times)

Maximum value = $-2.816*10^{-3}$ m (Element 78 at Node 58394)

Minimum value = -0.01118 m (Element 20 at Node 60207)

3.1.1.1.3.8 Calculation results, Plate, Phase_9 [Phase_9] (9/166), Total displacements u_y

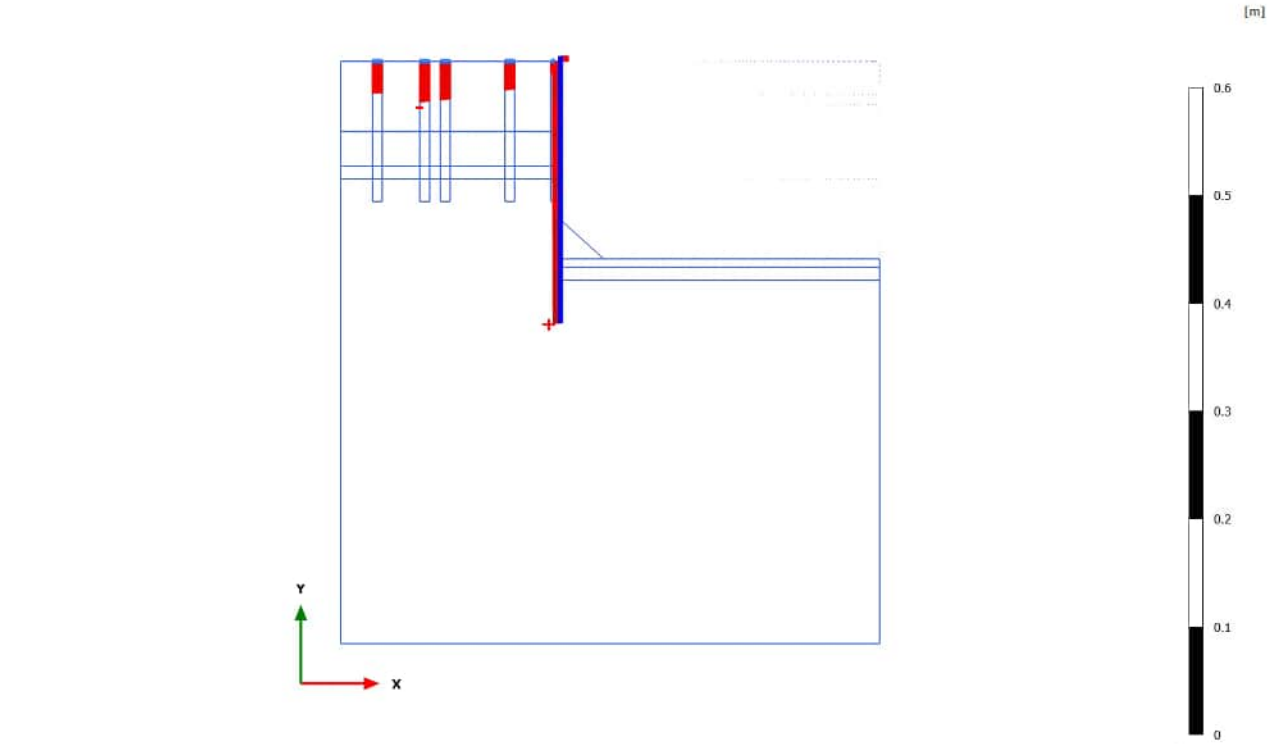


Total displacements u_y (scaled up 100 times)

Maximum value = $-6.993*10^{-3}$ m (Element 78 at Node 58394)

Minimum value = -0.03790 m (Element 9 at Node 73114)

3.1.1.1.3.9 Calculation results, Plate, Phase_10 [Phase_10] (10/168), Total displacements u_y

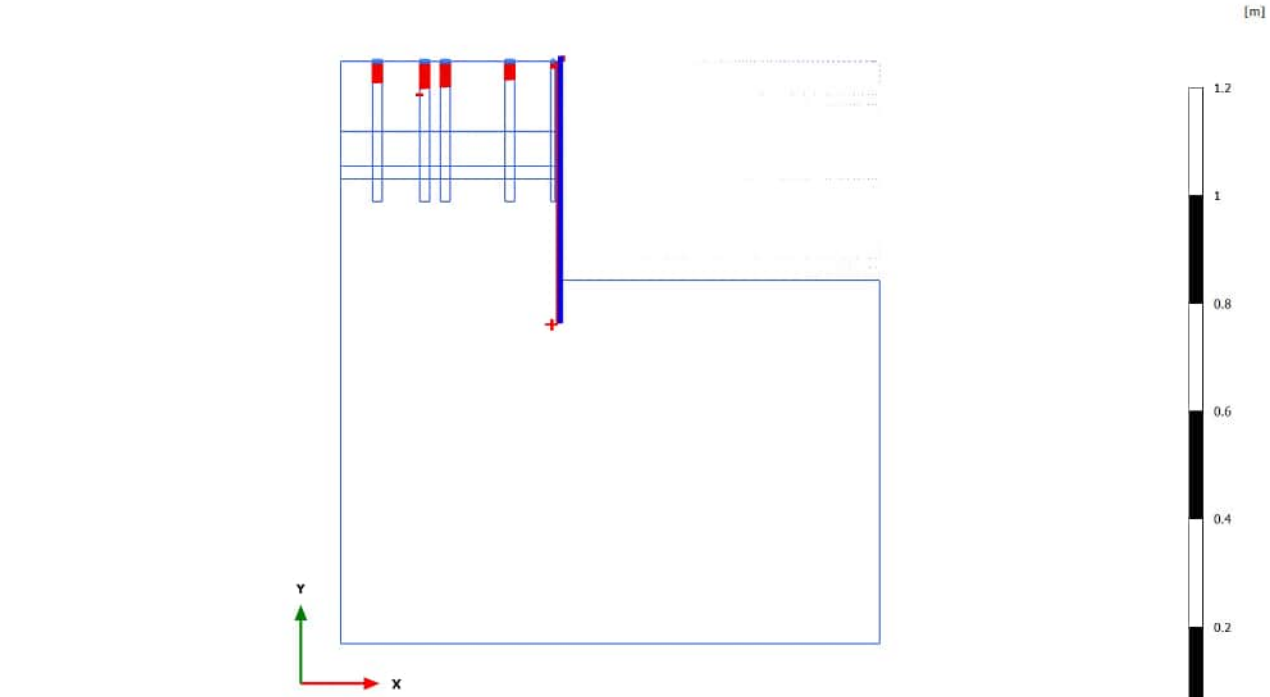


Total displacements u_y (scaled up 100 times)

Maximum value = $-6.992*10^{-3}$ m (Element 78 at Node 58394)

Minimum value = -0.03790 m (Element 9 at Node 73114)

3.1.1.1.3.10 Calculation results, Plate, Phase_11 [Phase_11] (11/218), Total displacements u_y

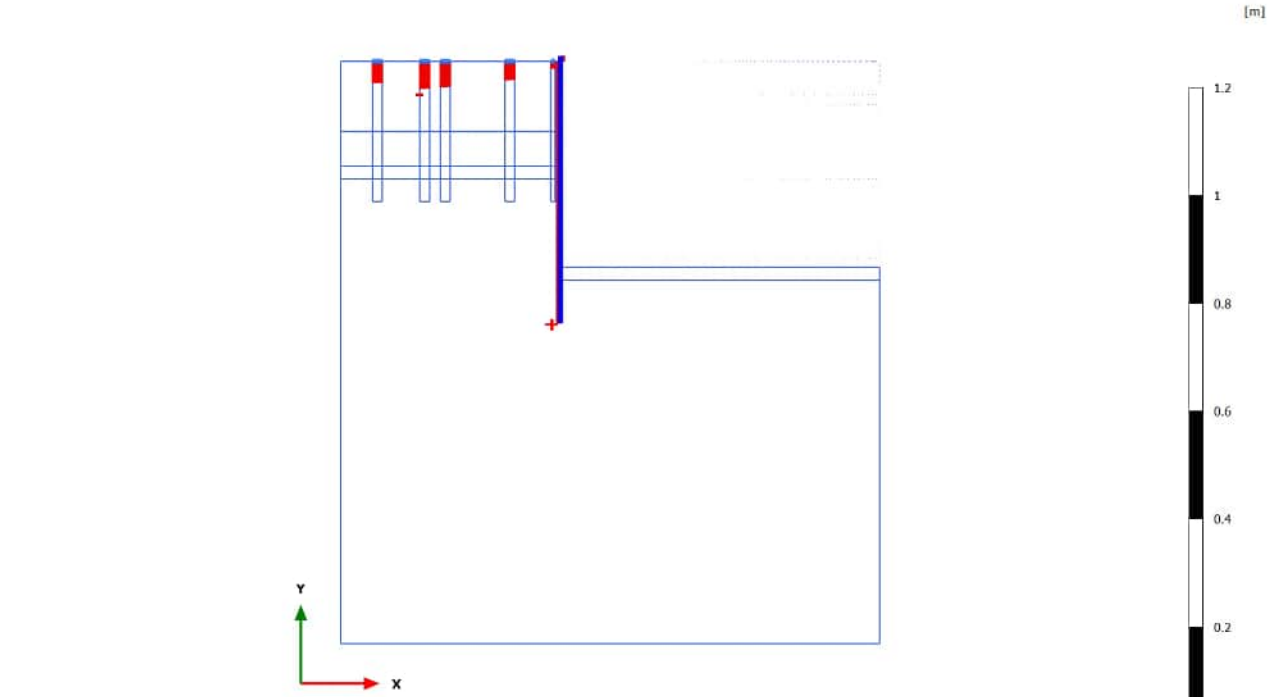


Total displacements u_y (scaled up 50.0 times)

Maximum value = $-7.856*10^{-3}$ m (Element 78 at Node 58394)

Minimum value = -0.05128 m (Element 9 at Node 73114)

3.1.1.1.3.11 Calculation results, Plate, Phase_12 [Phase_12] (12/221), Total displacements u_y

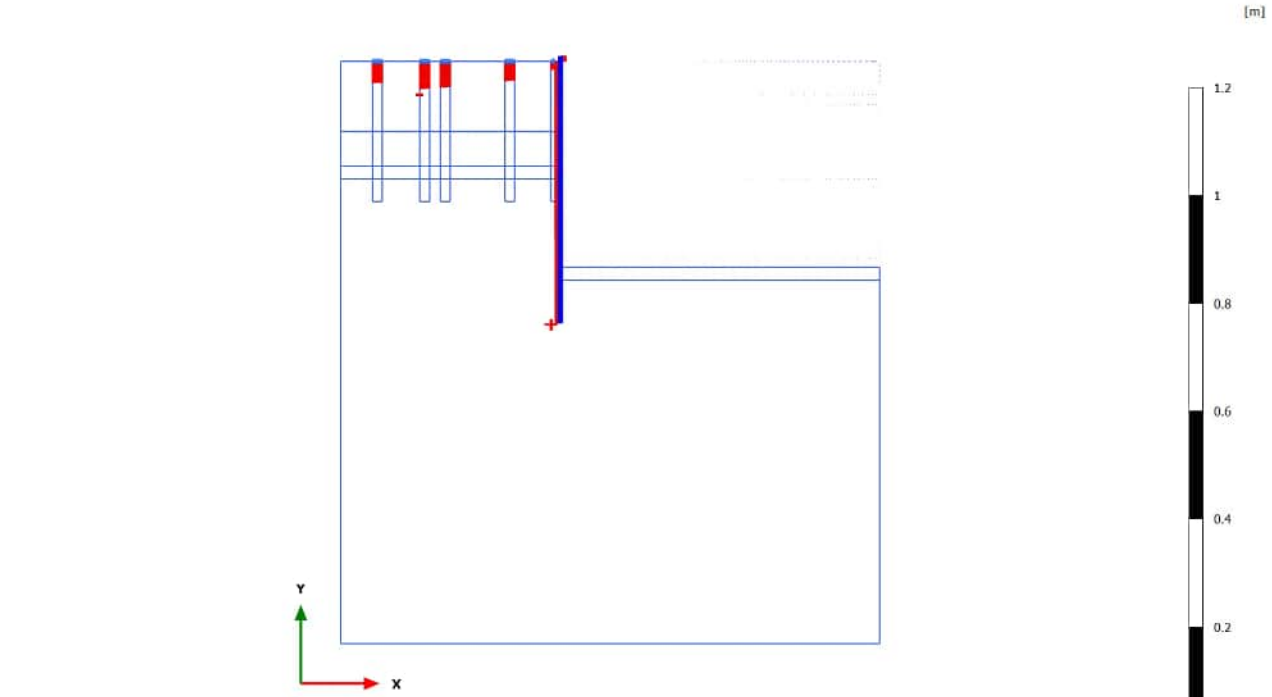


Total displacements u_y (scaled up 50.0 times)

Maximum value = $-7.854*10^{-3}$ m (Element 78 at Node 58394)

Minimum value = -0.05107 m (Element 9 at Node 73114)

3.1.1.1.3.12 Calculation results, Plate, Phase_13 [Phase_13] (13/226), Total displacements u_y

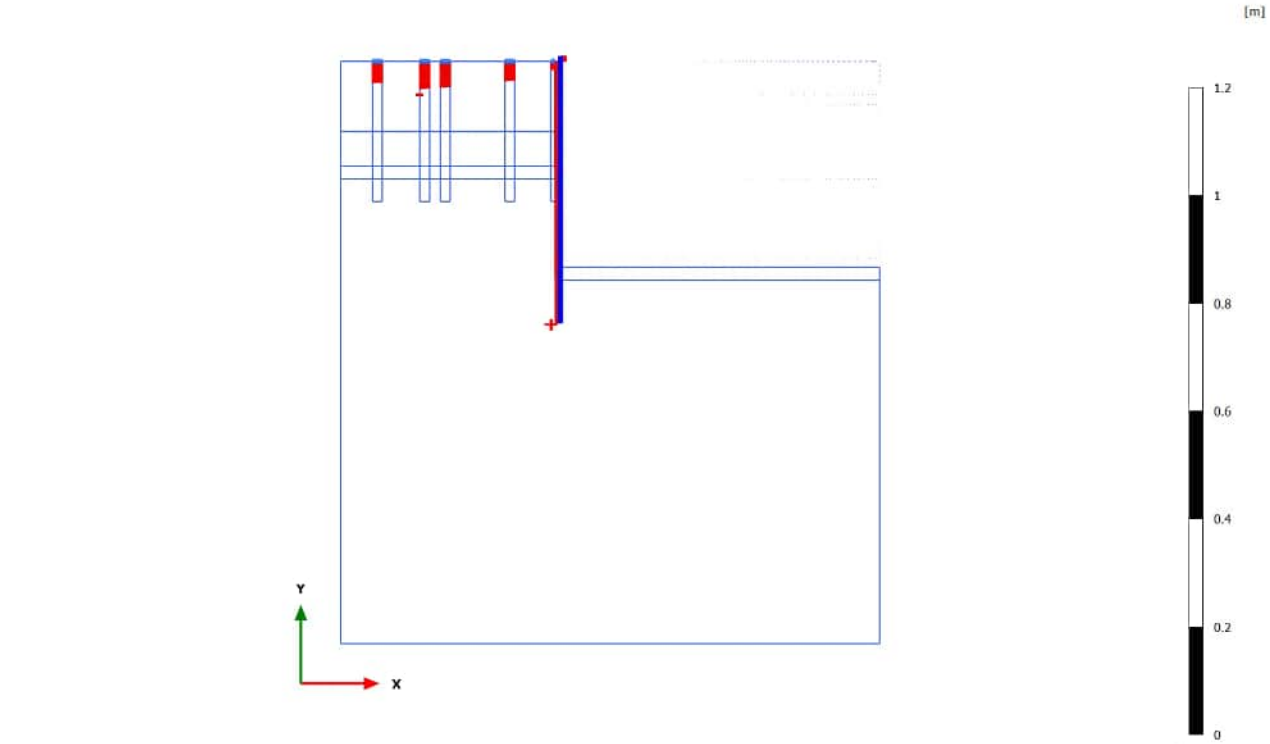


Total displacements u_y (scaled up 50.0 times)

Maximum value = -0.01022 m (Element 78 at Node 58394)

Minimum value = -0.05113 m (Element 9 at Node 73114)

3.1.1.1.3.13 Calculation results, Plate, Phase_14 [Phase_14] (14/229), Total displacements u_y

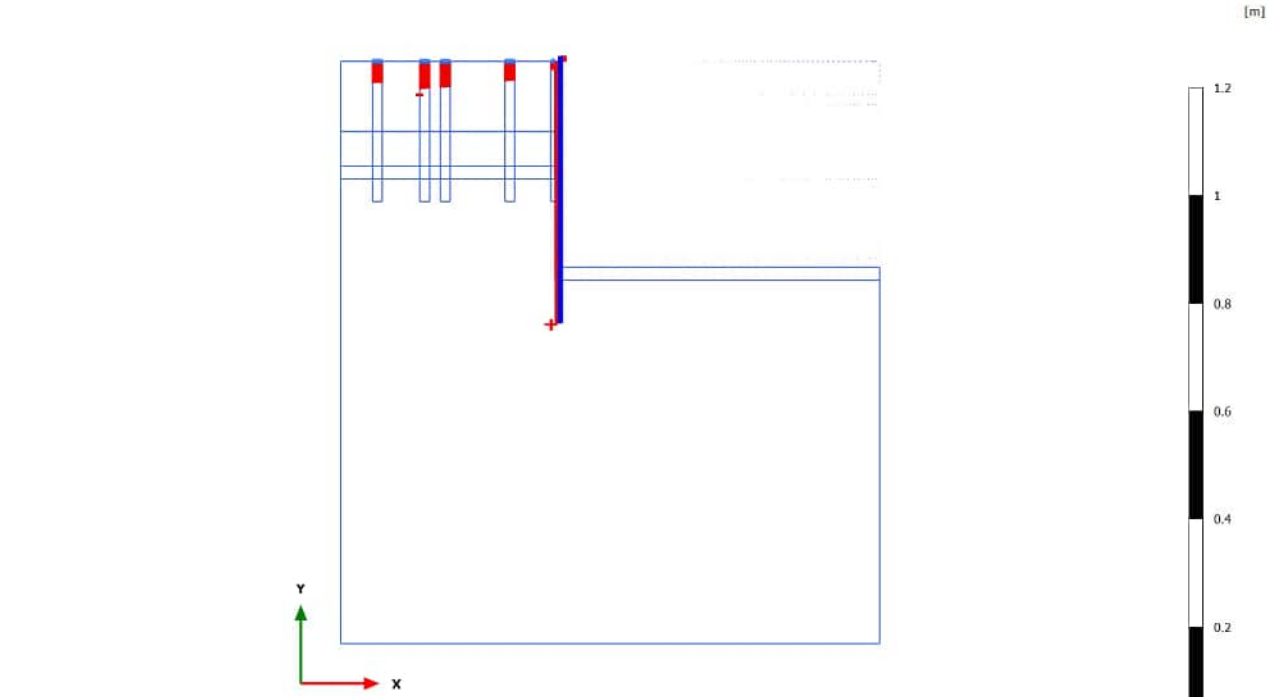


Total displacements u_y (scaled up 50.0 times)

Maximum value = -0.01037 m (Element 78 at Node 58394)

Minimum value = -0.05114 m (Element 9 at Node 73114)

3.1.1.1.3.14 Calculation results, Plate, Phase_15 [Phase_15] (15/245), Total displacements u_y

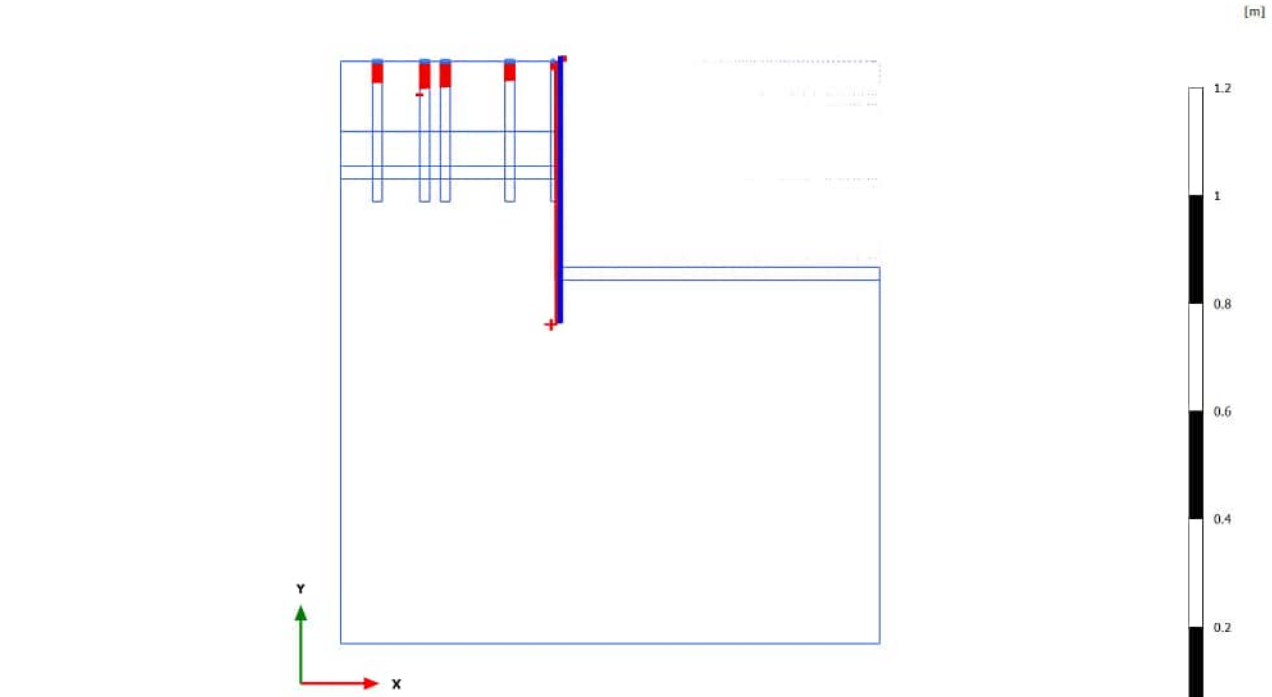


Total displacements u_y (scaled up 50.0 times)

Maximum value = -0.01038 m (Element 78 at Node 58394)

Minimum value = -0.05114 m (Element 9 at Node 73114)

3.1.1.1.3.15 Calculation results, Plate, Phase_16 [Phase_16] (16/248), Total displacements u_y

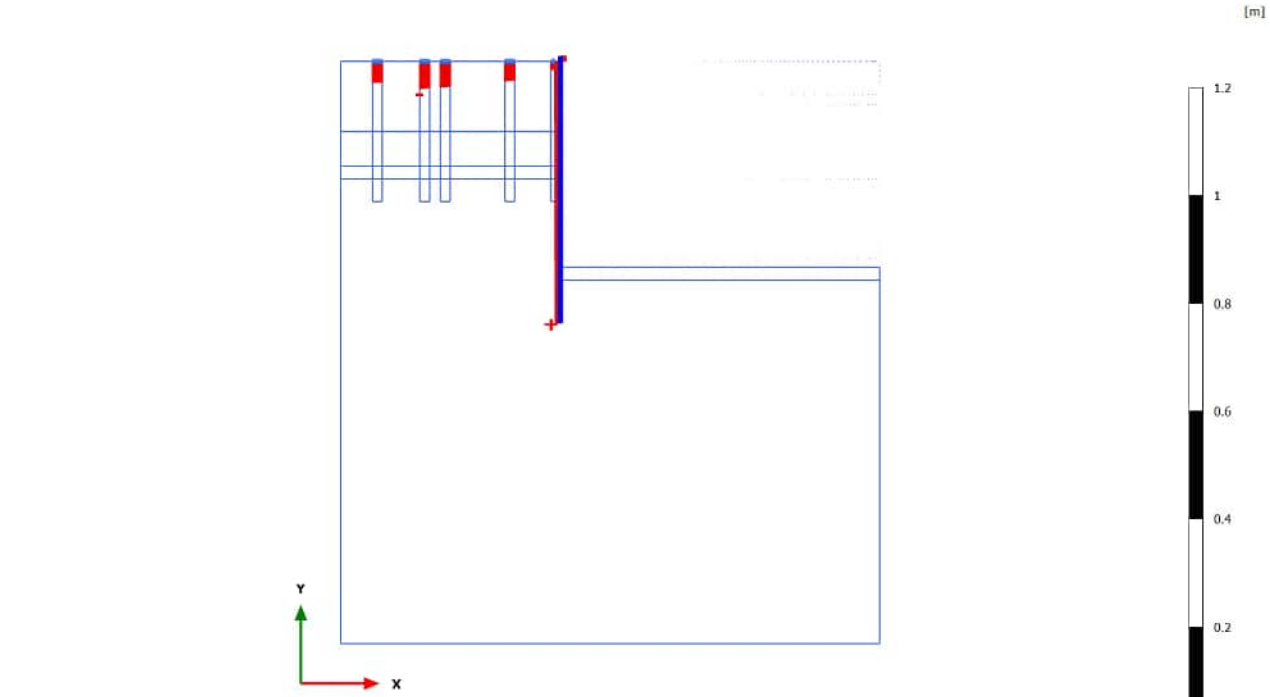


Total displacements u_y (scaled up 50.0 times)

Maximum value = -0.01038 m (Element 78 at Node 58394)

Minimum value = -0.05114 m (Element 9 at Node 73114)

3.1.1.1.3.16 Calculation results, Plate, Phase_17 [Phase_17] (17/253), Total displacements u_y

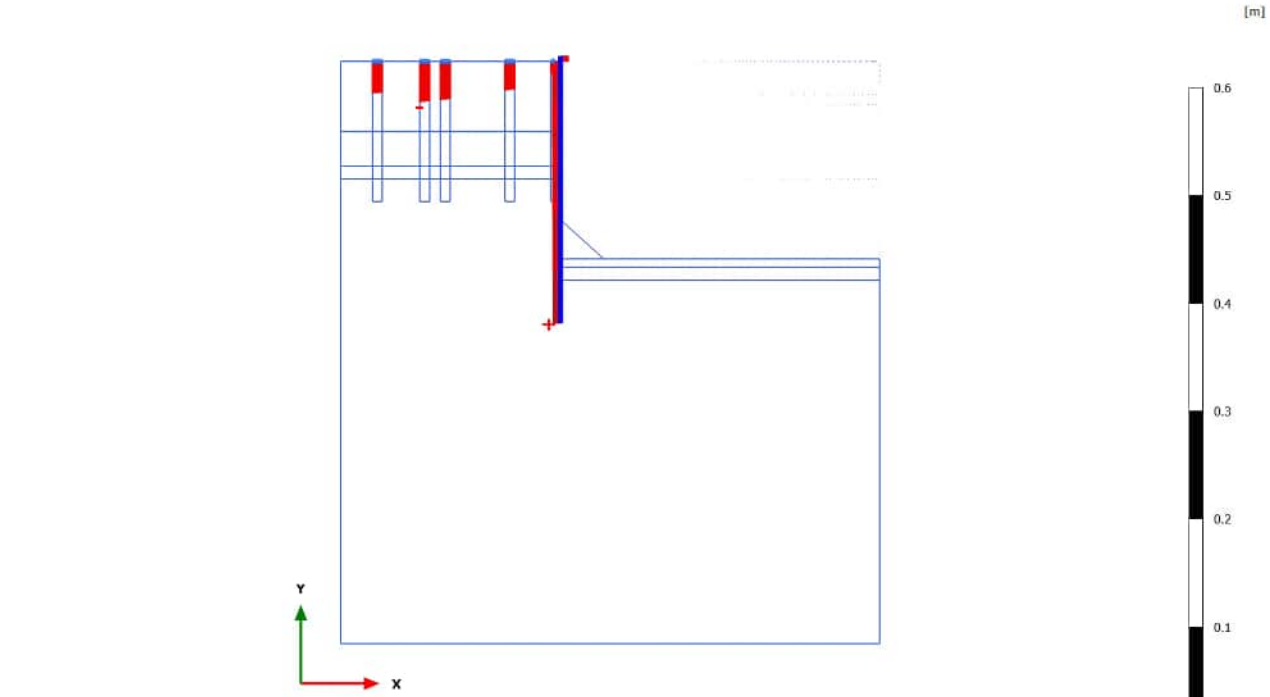


Total displacements u_y (scaled up 50.0 times)

Maximum value = -0.01030 m (Element 78 at Node 58394)

Minimum value = -0.05100 m (Element 9 at Node 73114)

3.1.1.1.3.17 Calculation results, Plate, Phase_18 [Phase_18] (18/258), Total displacements u_y

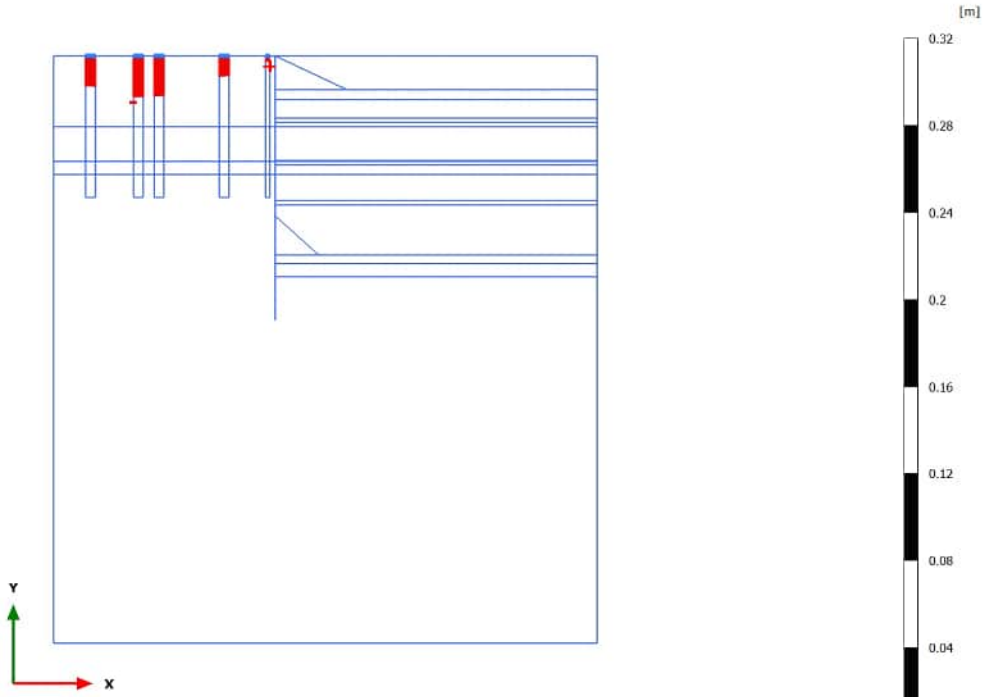


Total displacements u_y (scaled up 100 times)

Maximum value = $-6.922*10^{-3}$ m (Element 78 at Node 58394)

Minimum value = -0.03775 m (Element 9 at Node 73114)

3.1.1.1.3.18 Calculation results, Plate, Phase_1 [Phase_1] (1/310), Total displacements u_y

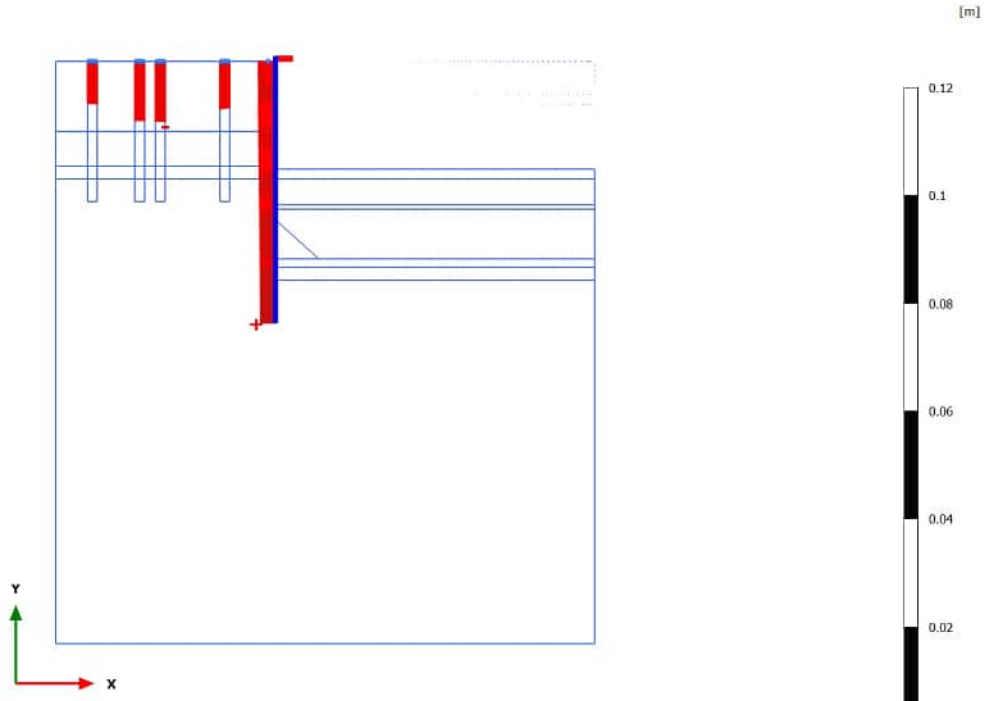


Total displacements u_y (scaled up 200 times)

Maximum value = $-2.446*10^{-3}$ m (Element 28 at Node 25748)

Minimum value = -0.01890 m (Element 9 at Node 73114)

3.1.1.1.3.19 Calculation results, Plate, Phase_7 [Phase_7] (7/339), Total displacements u_y

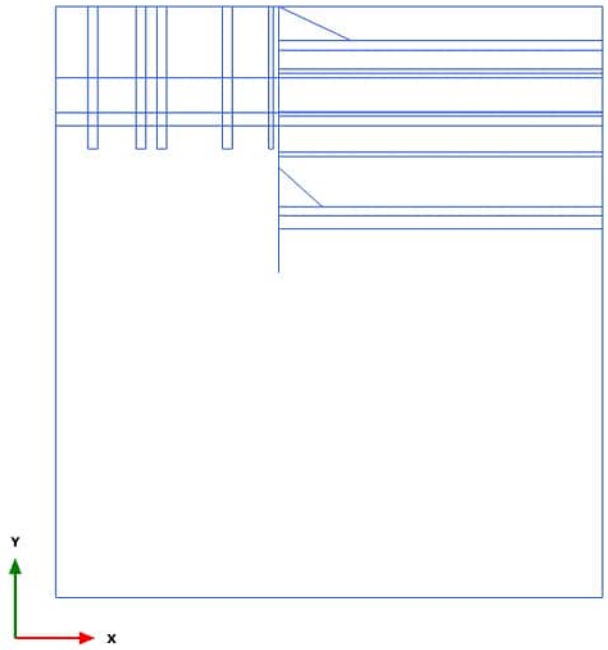


Total displacements u_y (scaled up 500 times)

Maximum value = $-2.817*10^{-3}$ m (Element 78 at Node 58394)

Minimum value = -0.01118 m (Element 20 at Node 60207)

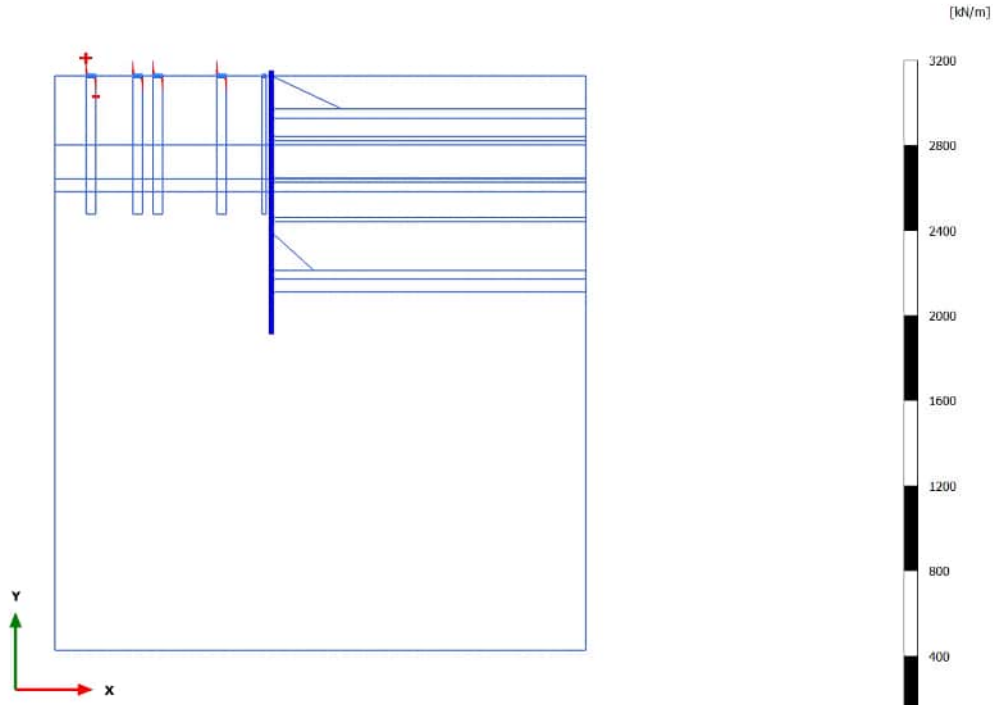
3.1.1.1.1 Calculation results, Plate, Initial phase [InitialPhase] (0/0), Shear forces Q



Shear forces Q (scaled up 1.00 times)

No results

3.1.1.1.2 Calculation results, Plate, Phase_2 [Phase_2] (2/3), Shear forces Q

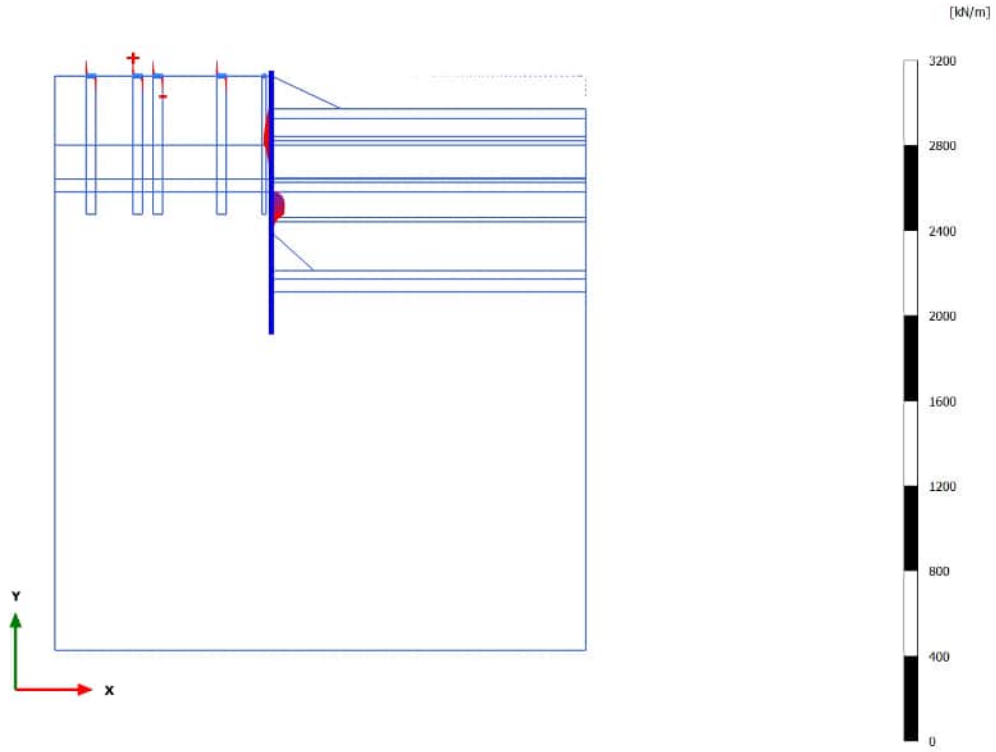


Shear forces Q (scaled up 0.0200 times)

Maximum value = 71.03 kN/m (Element 3 at Node 77761)

Minimum value = -71.14 kN/m (Element 8 at Node 79652)

3.1.1.1.3 Calculation results, Plate, Phase_3 [Phase_3] (3/10), Shear forces Q

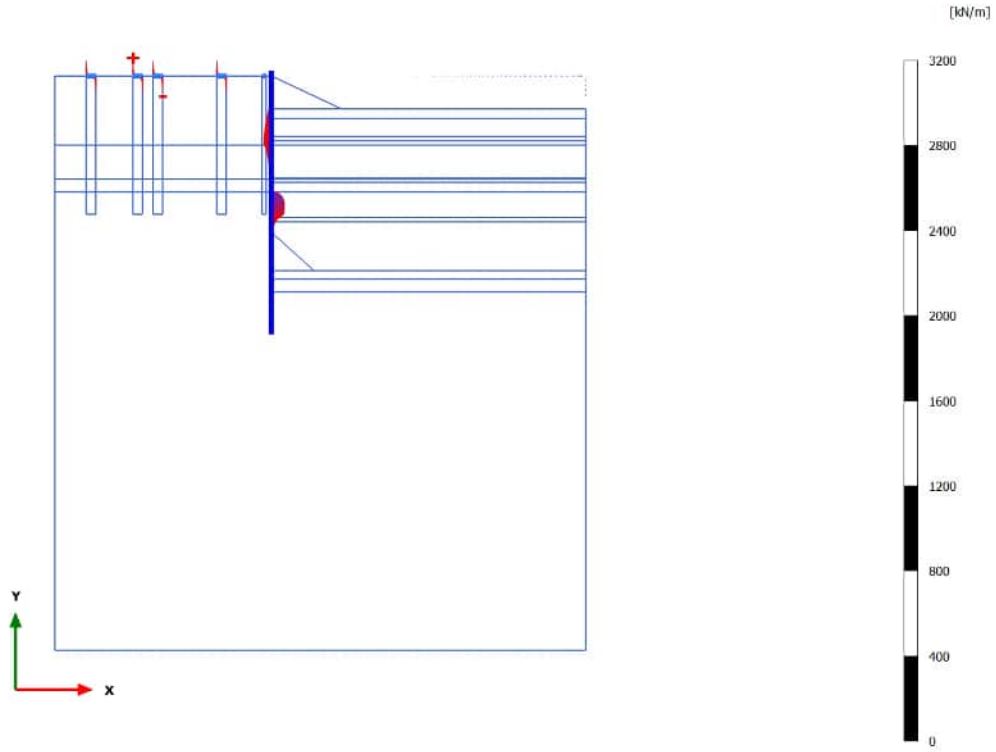


Shear forces Q (scaled up 0.0200 times)

Maximum value = 71.73 kN/m (Element 9 at Node 73088)

Minimum value = -71.77 kN/m (Element 20 at Node 60218)

3.1.1.1.4 Calculation results, Plate, Phase_4 [Phase_4] (4/34), Shear forces Q

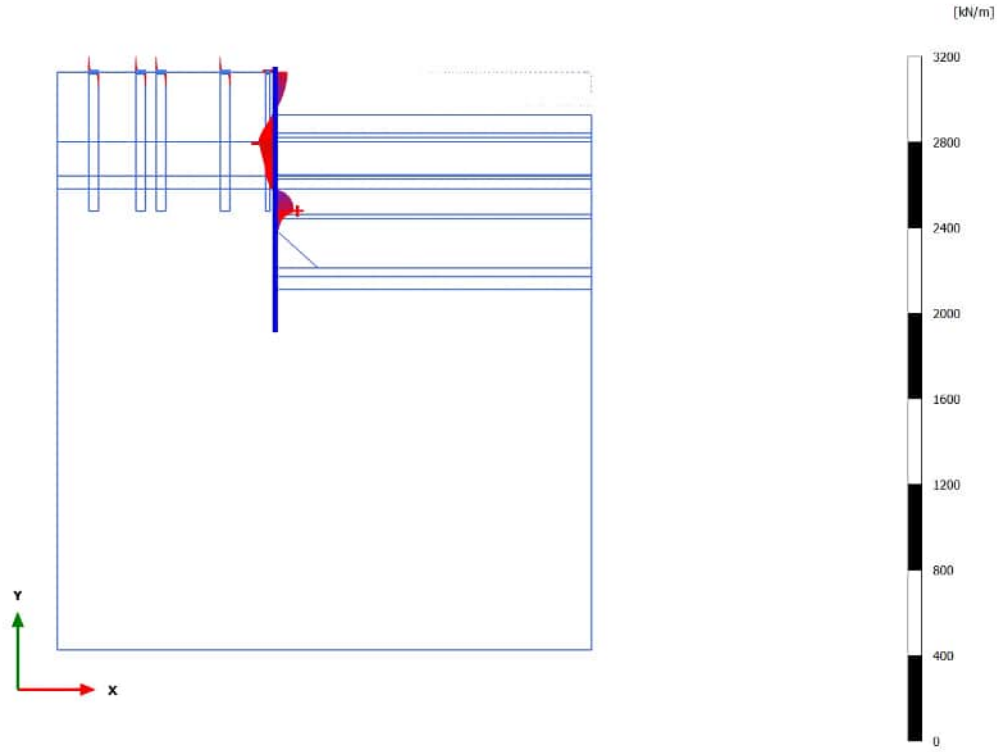


Shear forces Q (scaled up 0.0200 times)

Maximum value = 71.72 kN/m (Element 9 at Node 73088)

Minimum value = -71.76 kN/m (Element 20 at Node 60218)

3.1.1.1.5 Calculation results, Plate, Phase_5 [Phase_5] (5/42), Shear forces Q

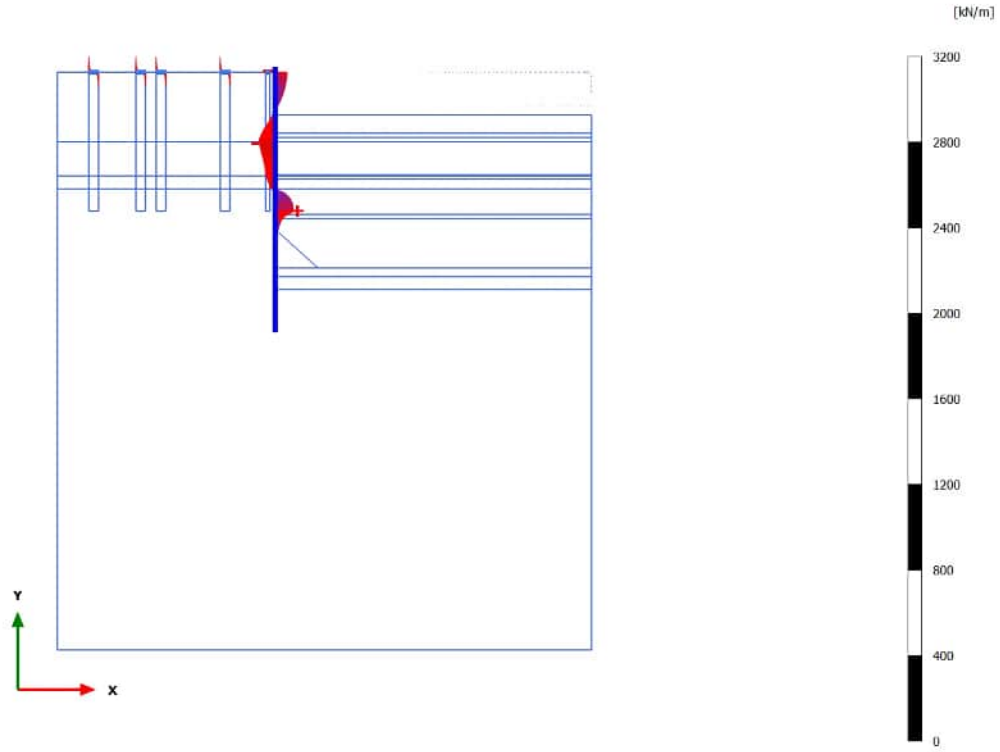


Shear forces Q (scaled up 0.0200 times)

Maximum value = 85.06 kN/m (Element 64 at Node 47087)

Minimum value = -78.01 kN/m (Element 45 at Node 31389)

3.1.1.1.6 Calculation results, Plate, Phase_6 [Phase_6] (6/89), Shear forces Q

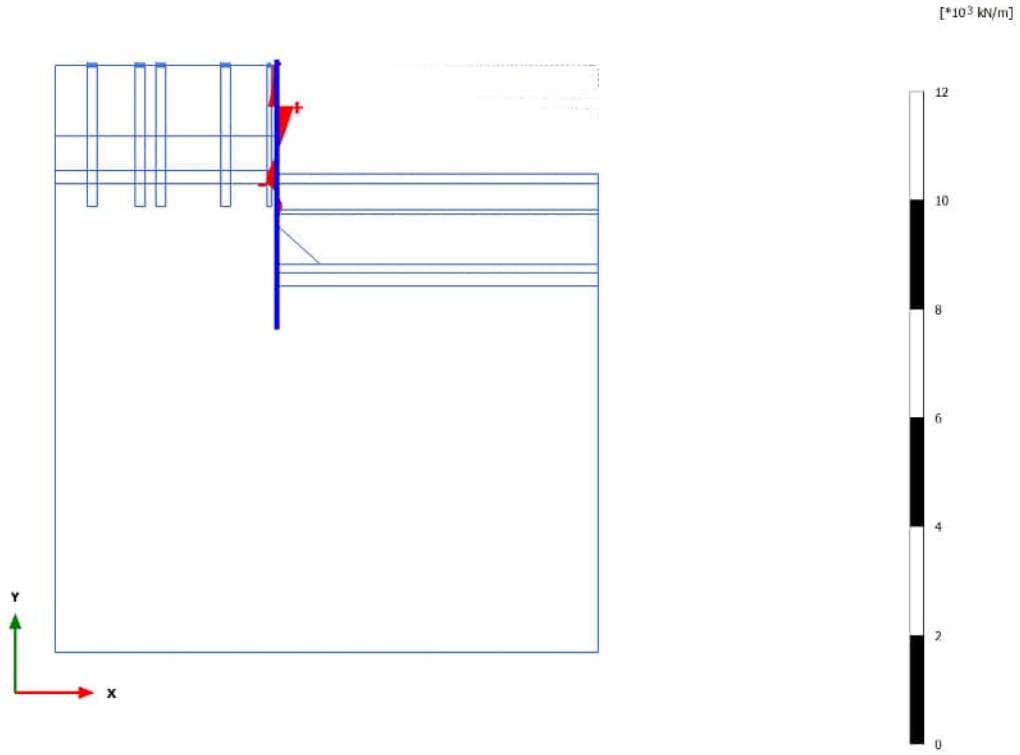


Shear forces Q (scaled up 0.0200 times)

Maximum value = 85.31 kN/m (Element 64 at Node 47087)

Minimum value = -77.53 kN/m (Element 45 at Node 31389)

3.1.1.1.7 Calculation results, Plate, Phase_8 [Phase_8] (8/91), Shear forces Q

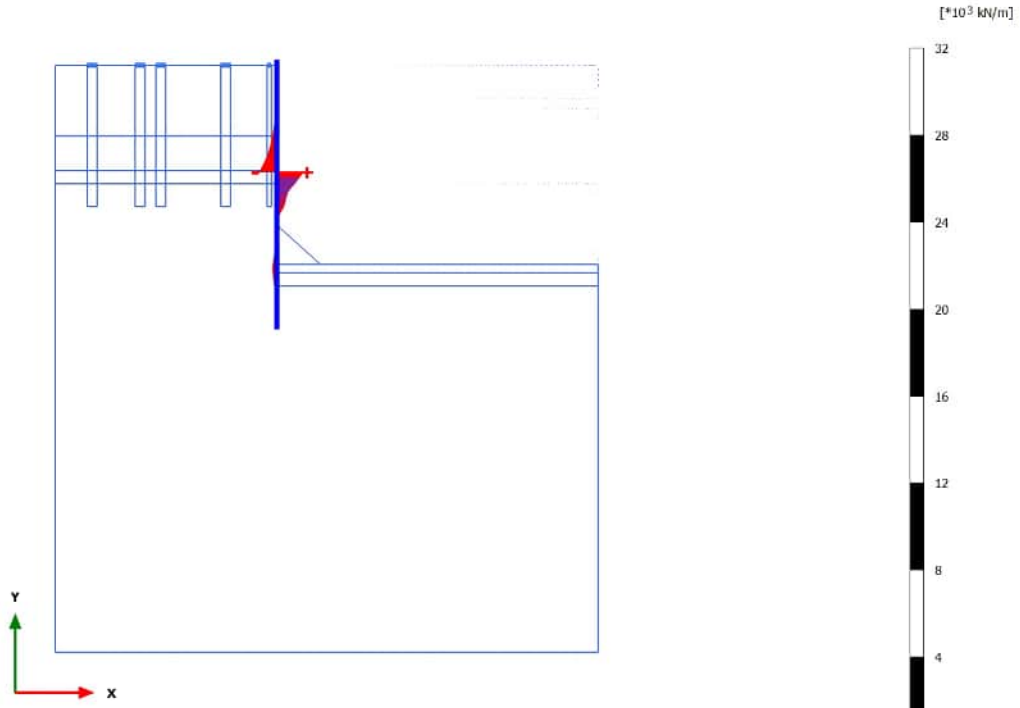


Shear forces Q (scaled up 5.00*10⁻³ times)

Maximum value = 299.7 kN/m (Element 36 at Node 31403)

Minimum value = -198.6 kN/m (Element 60 at Node 41577)

3.1.1.1.8 Calculation results, Plate, Phase_9 [Phase_9] (9/166), Shear forces Q

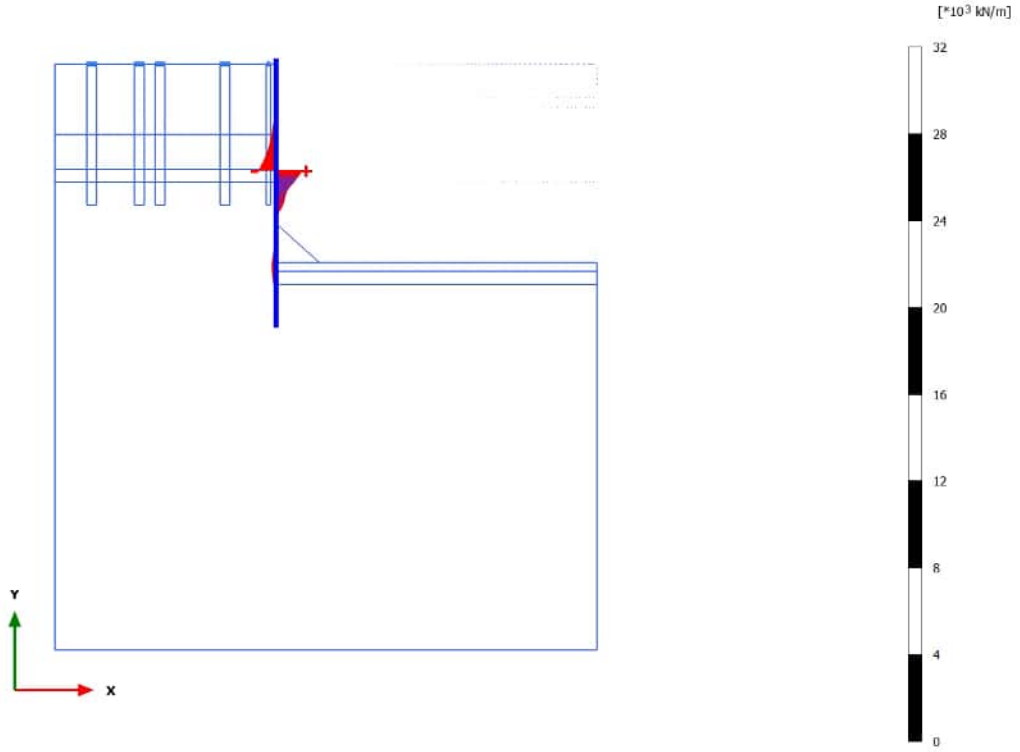


Shear forces Q (scaled up 2.00*10^{-3} times)

Maximum value = 1220 kN/m (Element 58 at Node 36587)

Minimum value = -813.4 kN/m (Element 57 at Node 36587)

3.1.1.1.9 Calculation results, Plate, Phase_10 [Phase_10] (10/168), Shear forces Q

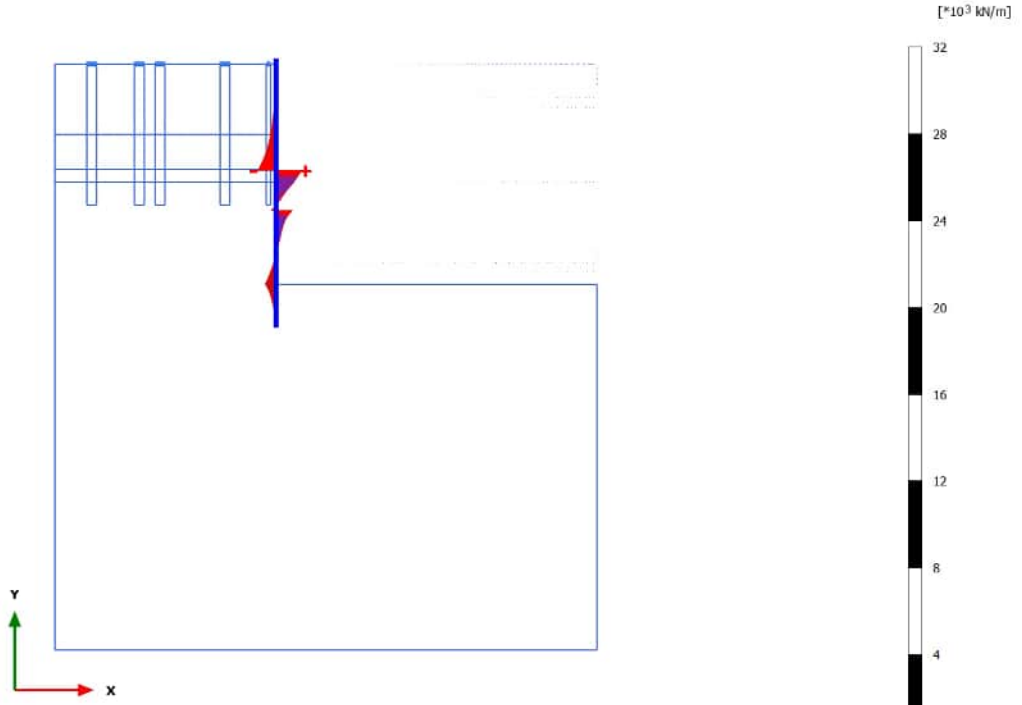


Shear forces Q (scaled up 2.00*10⁻³ times)

Maximum value = 1220 kN/m (Element 58 at Node 36587)

Minimum value = -813.4 kN/m (Element 57 at Node 36587)

3.1.1.1.10 Calculation results, Plate, Phase_11 [Phase_11] (11/218), Shear forces Q

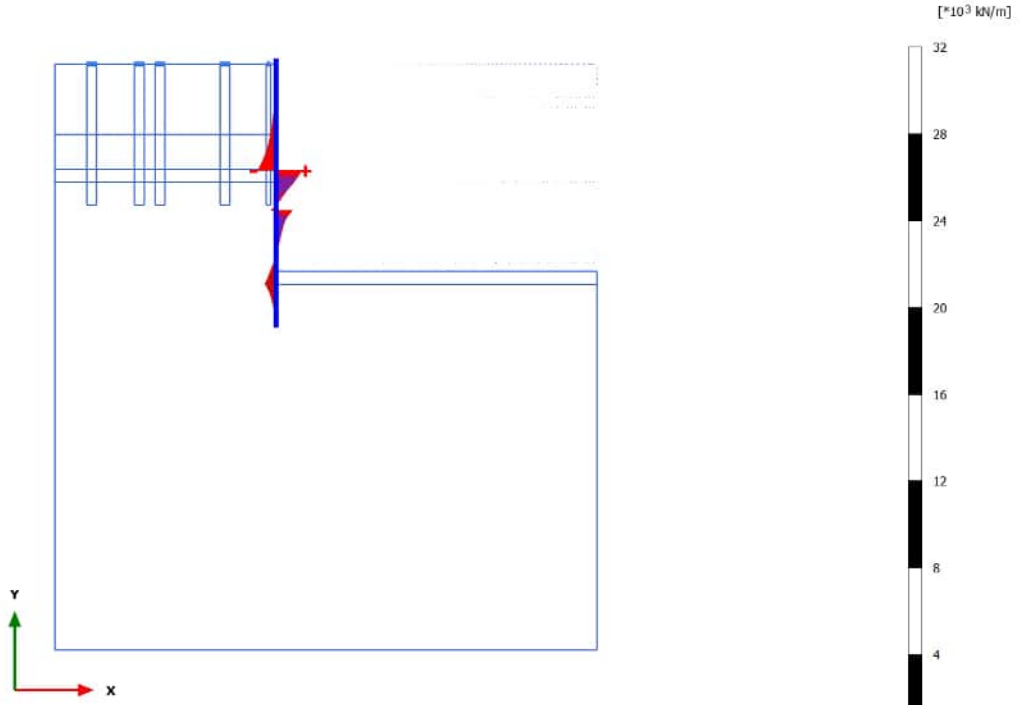


Shear forces Q (scaled up 2.00*10^{-3} times)

Maximum value = 1183 kN/m (Element 58 at Node 36587)

Minimum value = -844.2 kN/m (Element 57 at Node 36587)

3.1.1.1.11 Calculation results, Plate, Phase_12 [Phase_12] (12/221), Shear forces Q

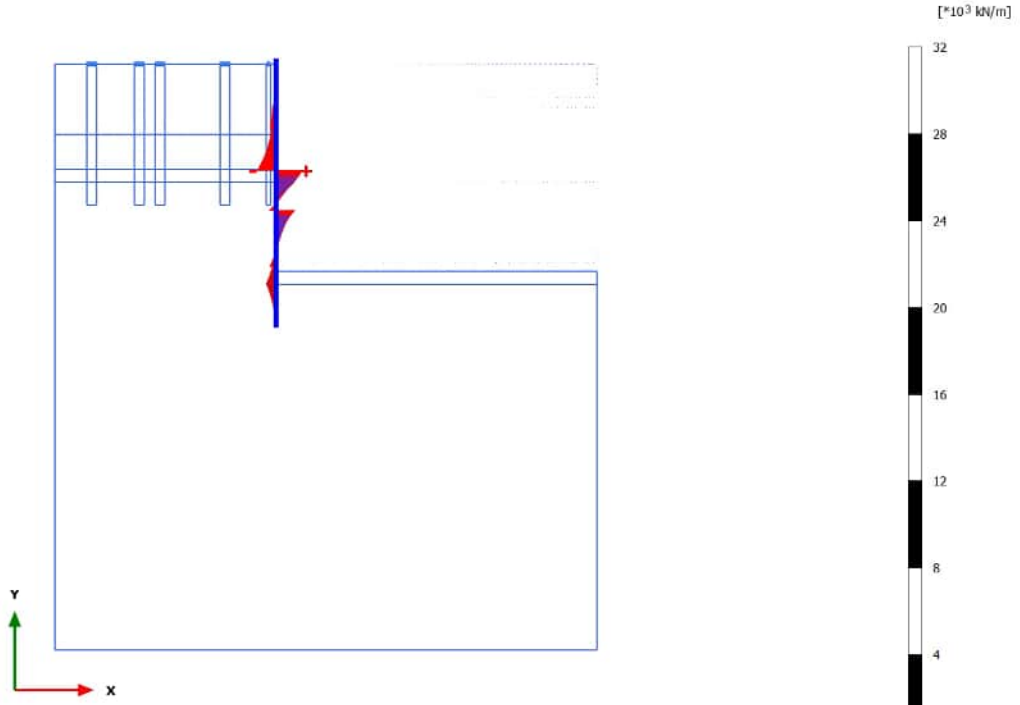


Shear forces Q (scaled up 2.00*10^{-3} times)

Maximum value = 1190 kN/m (Element 58 at Node 36587)

Minimum value = -848.5 kN/m (Element 57 at Node 36587)

3.1.1.1.12 Calculation results, Plate, Phase_13 [Phase_13] (13/226), Shear forces Q

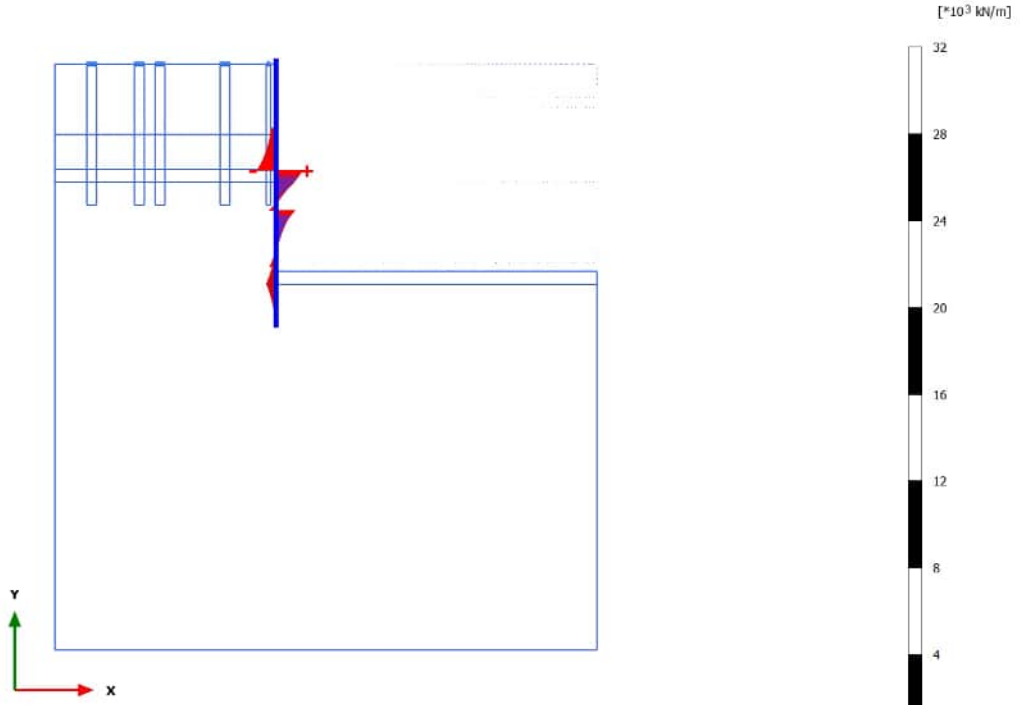


Shear forces Q (scaled up 2.00*10^{-3} times)

Maximum value = 1225 kN/m (Element 58 at Node 36587)

Minimum value = -869.1 kN/m (Element 57 at Node 36587)

3.1.1.1.13 Calculation results, Plate, Phase_14 [Phase_14] (14/229), Shear forces Q

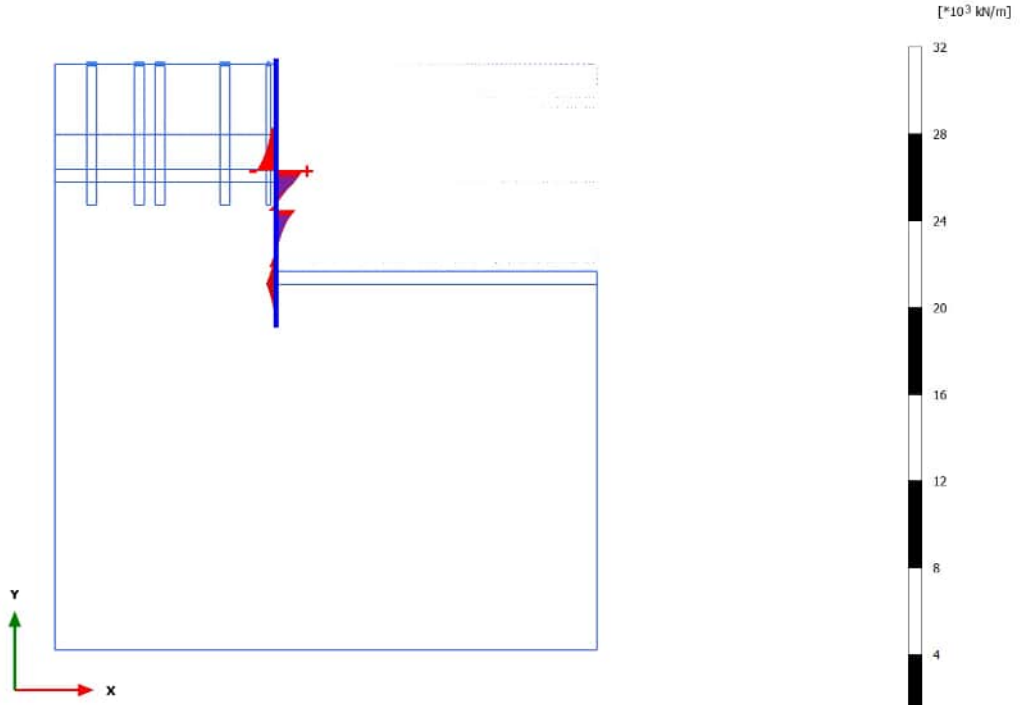


Shear forces Q (scaled up 2.00*10^{-3} times)

Maximum value = 1230 kN/m (Element 58 at Node 36587)

Minimum value = -878.6 kN/m (Element 57 at Node 36587)

3.1.1.1.14 Calculation results, Plate, Phase_15 [Phase_15] (15/245), Shear forces Q

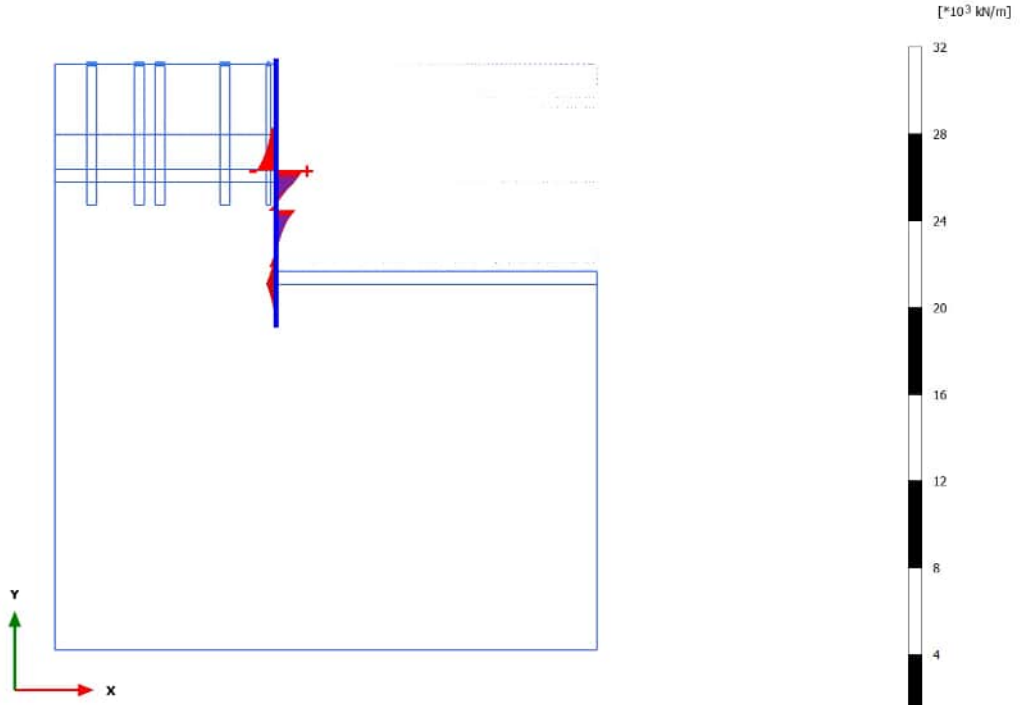


Shear forces Q (scaled up 2.00*10^{-3} times)

Maximum value = 1230 kN/m (Element 58 at Node 36587)

Minimum value = -878.5 kN/m (Element 57 at Node 36587)

3.1.1.1.15 Calculation results, Plate, Phase_16 [Phase_16] (16/248), Shear forces Q

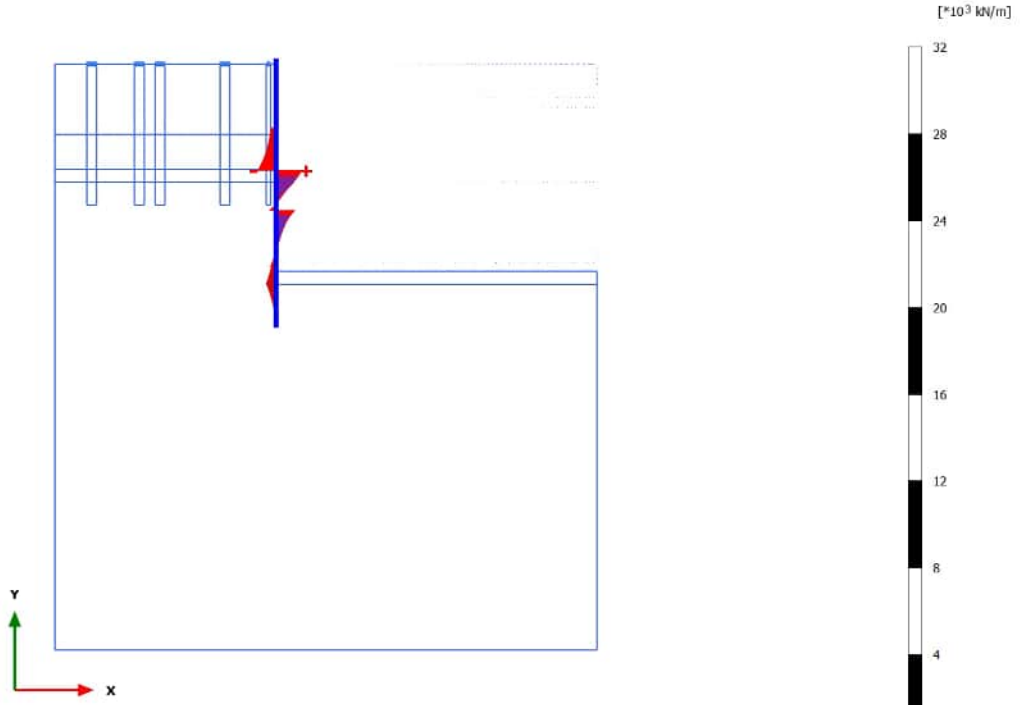


Shear forces Q (scaled up 2.00*10^{-3} times)

Maximum value = 1230 kN/m (Element 58 at Node 36587)

Minimum value = -878.5 kN/m (Element 57 at Node 36587)

3.1.1.1.16 Calculation results, Plate, Phase_17 [Phase_17] (17/253), Shear forces Q

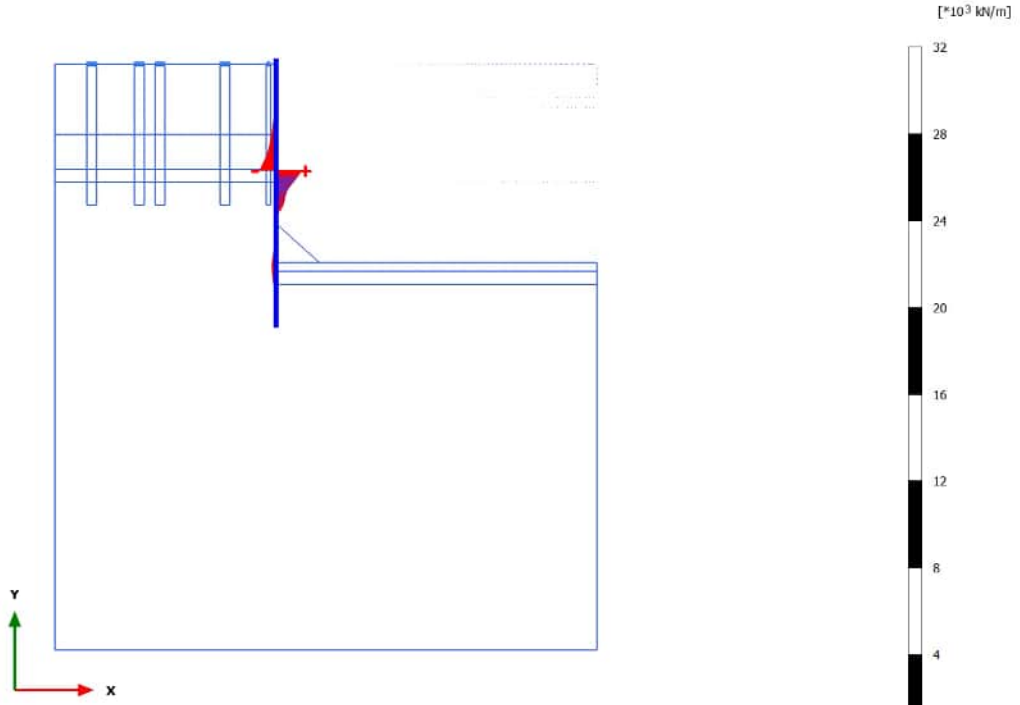


Shear forces Q (scaled up 2.00*10^{-3} times)

Maximum value = 1199 kN/m (Element 58 at Node 36587)

Minimum value = -847.5 kN/m (Element 57 at Node 36587)

3.1.1.1.17 Calculation results, Plate, Phase_18 [Phase_18] (18/258), Shear forces Q

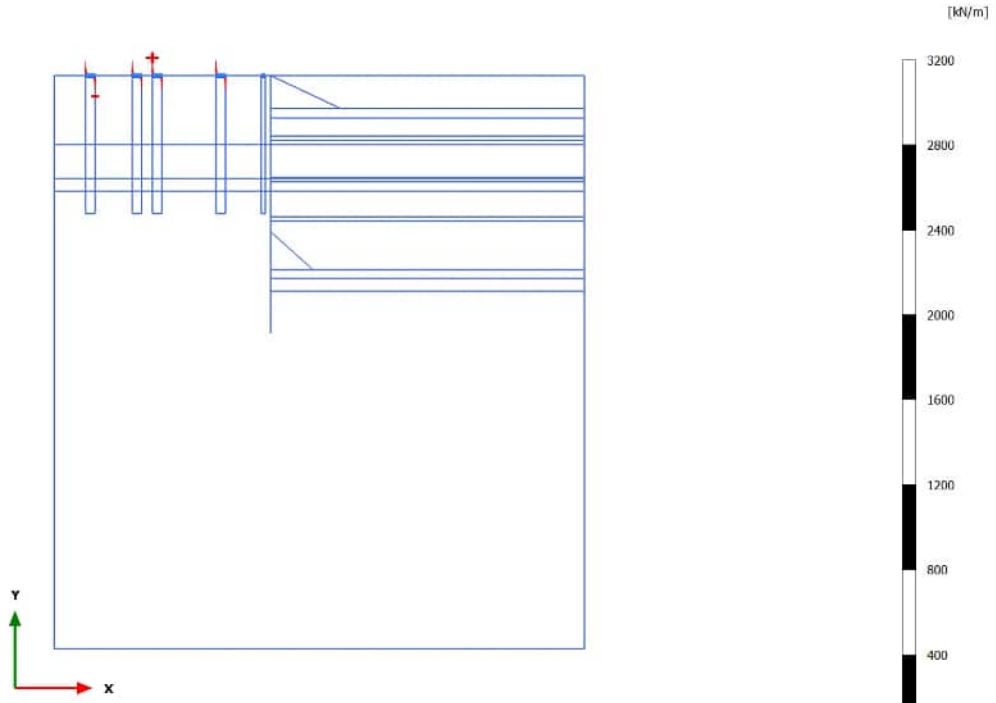


Shear forces Q (scaled up 2.00*10^{-3} times)

Maximum value = 1178 kN/m (Element 58 at Node 36587)

Minimum value = -765.8 kN/m (Element 57 at Node 36587)

3.1.1.1.18 Calculation results, Plate, Phase_1 [Phase_1] (1/310), Shear forces Q

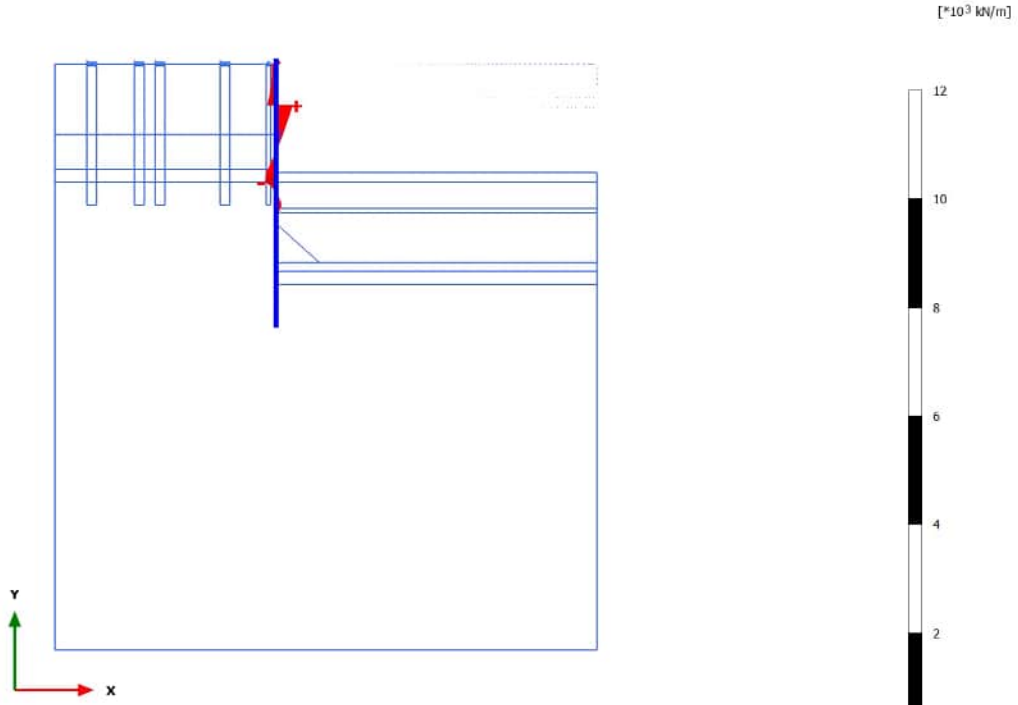


Shear forces Q (scaled up 0.0200 times)

Maximum value = 70.90 kN/m (Element 15 at Node 65583)

Minimum value = -70.92 kN/m (Element 8 at Node 79652)

3.1.1.1.19 Calculation results, Plate, Phase_7 [Phase_7] (7/339), Shear forces Q

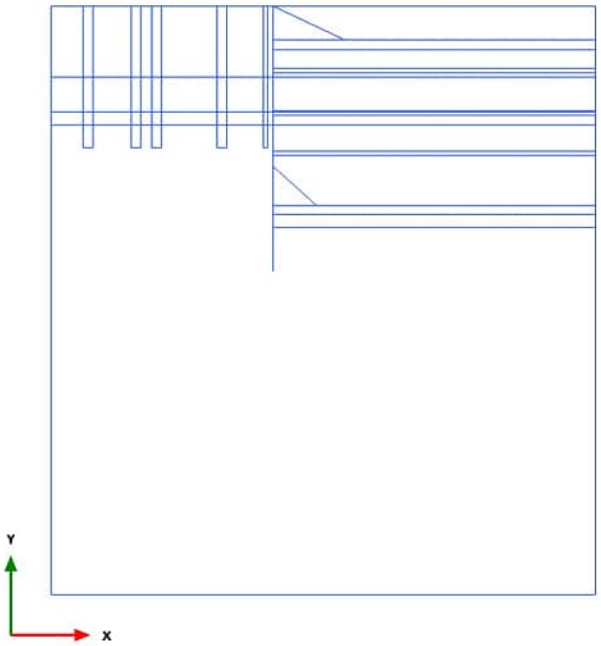


Shear forces Q (scaled up 5.00*10^-3 times)

Maximum value = 299.6 kN/m (Element 36 at Node 31403)

Minimum value = -198.4 kN/m (Element 60 at Node 41577)

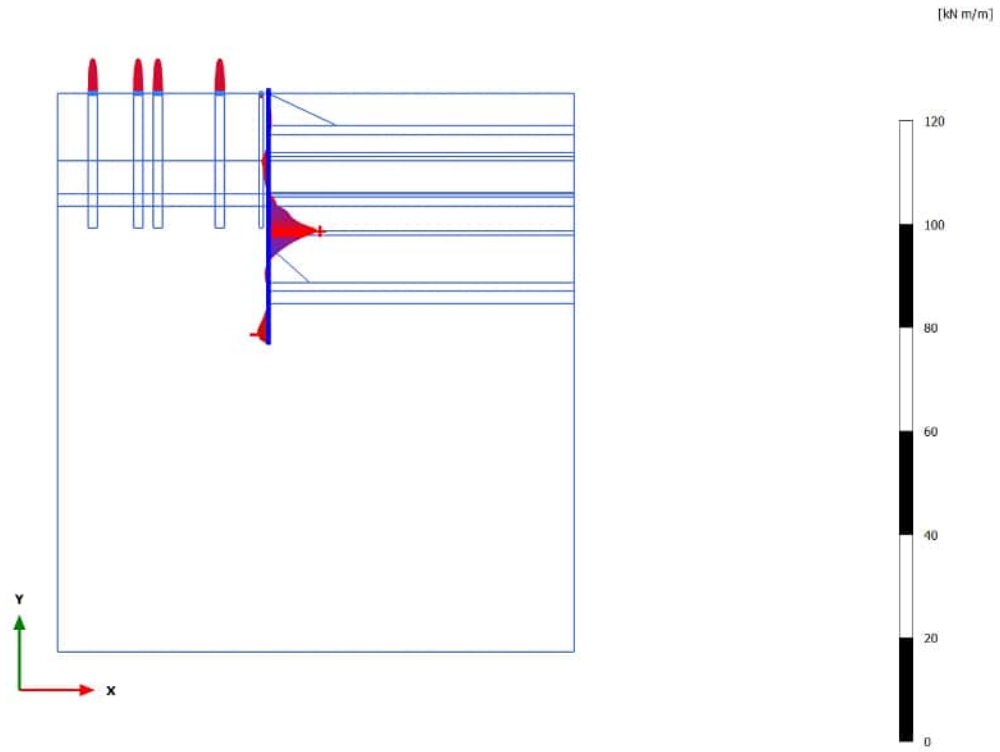
3.1.1.2.1 Calculation results, Plate, Initial phase [InitialPhase] (0/0), Bending moments M



Bending moments M (scaled up 1.00 times)

No results

3.1.1.2.2 Calculation results, Plate, Phase_2 [Phase_2] (2/3), Bending moments M

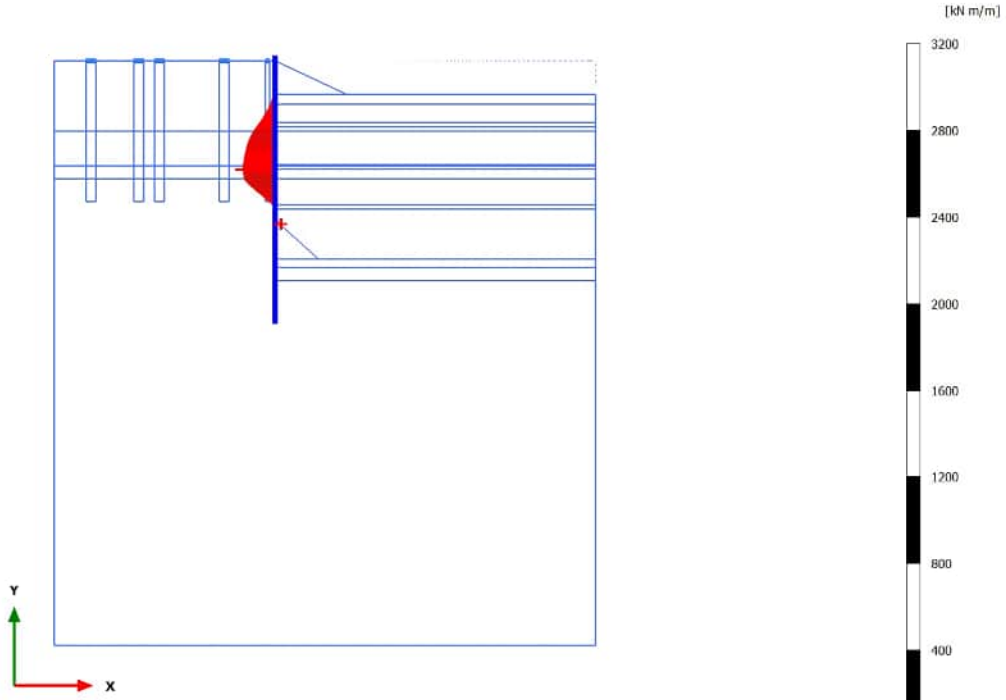


Bending moments M (scaled up 0.500 times)

Maximum value = 9.317 kN m/m (Element 65 at Node 47832)

Minimum value = -2.145 kN m/m (Element 78 at Node 58396)

3.1.1.2.3 Calculation results, Plate, Phase_3 [Phase_3] (3/10), Bending moments M

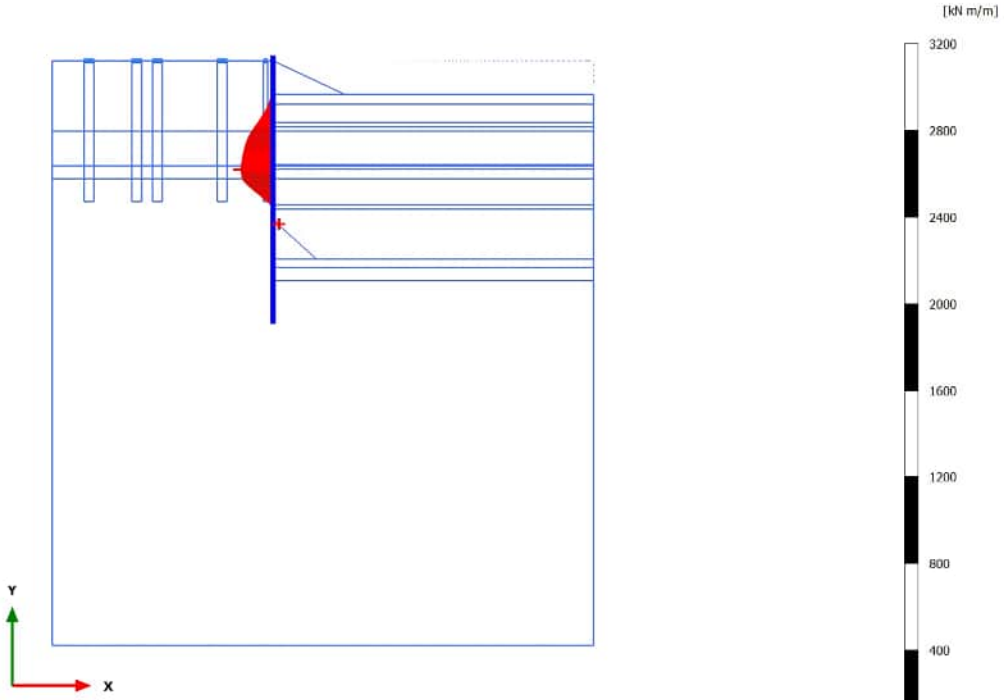


Bending moments M (scaled up 0.0200 times)

Maximum value = 6.928 kN m/m (Element 70 at Node 54038)

Minimum value = -148.0 kN m/m (Element 58 at Node 36589)

3.1.1.2.4 Calculation results, Plate, Phase_4 [Phase_4] (4/34), Bending moments M

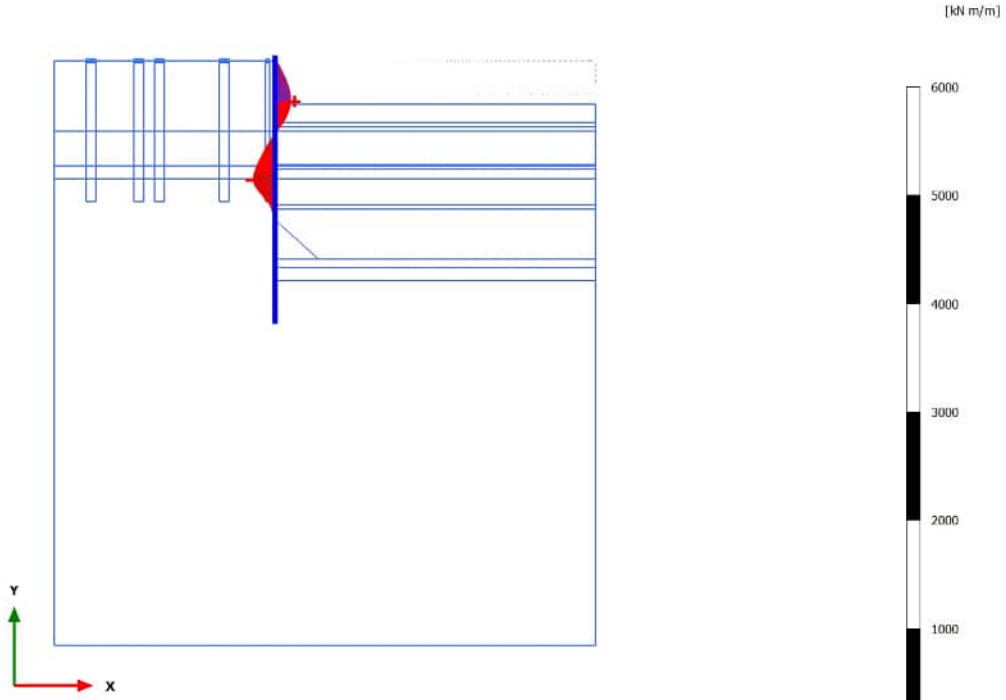


Bending moments M (scaled up 0.0200 times)

Maximum value = 6.873 kN m/m (Element 70 at Node 54038)

Minimum value = -148.0 kN m/m (Element 58 at Node 36589)

3.1.1.2.5 Calculation results, Plate, Phase_5 [Phase_5] (5/42), Bending moments M

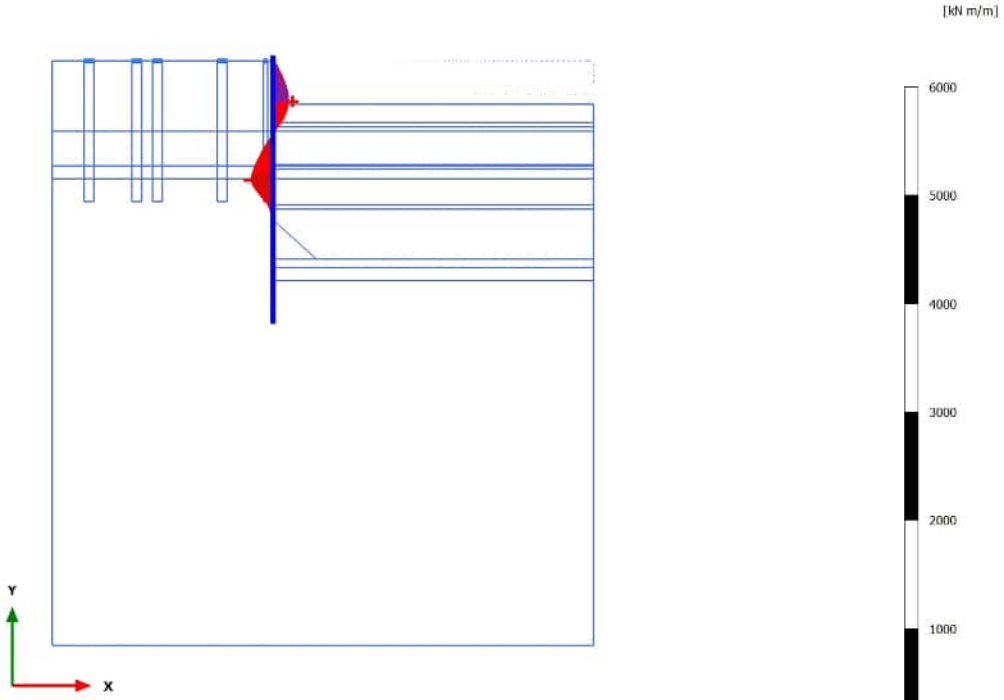


Bending moments M (scaled up 0.0100 times)

Maximum value = 144.2 kN m/m (Element 35 at Node 30678)

Minimum value = -207.0 kN m/m (Element 61 at Node 41577)

3.1.1.2.6 Calculation results, Plate, Phase_6 [Phase_6] (6/89), Bending moments M

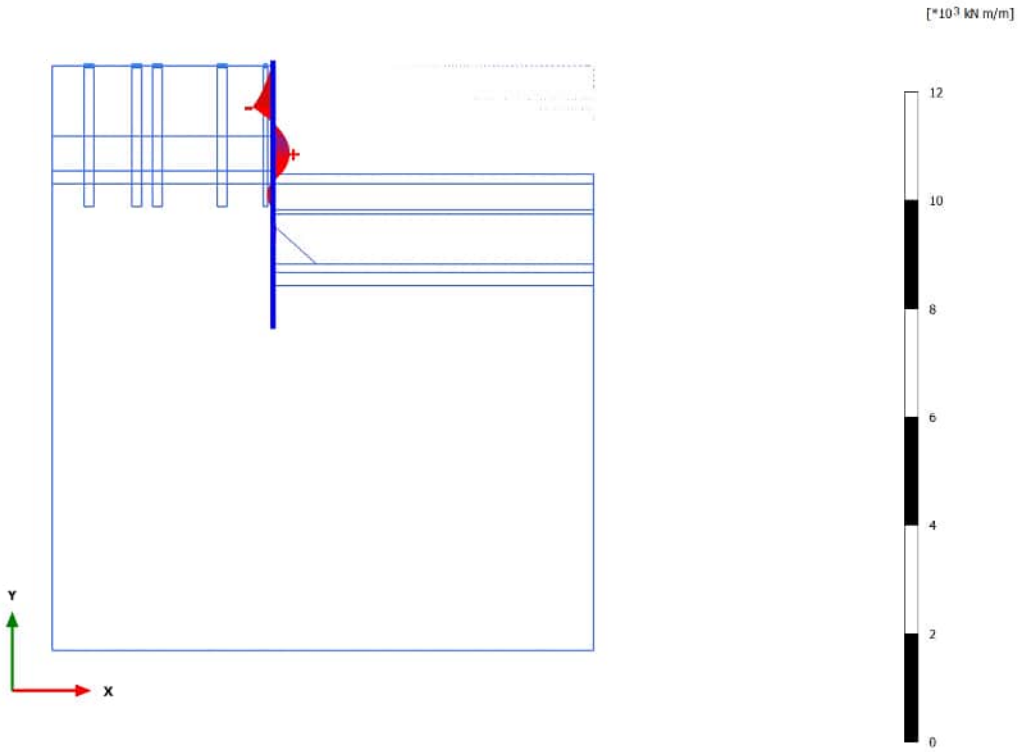


Bending moments M (scaled up 0.0100 times)

Maximum value = 143.4 kN m/m (Element 35 at Node 30678)

Minimum value = -207.5 kN m/m (Element 60 at Node 41577)

3.1.1.2.7 Calculation results, Plate, Phase_8 [Phase_8] (8/91), Bending moments M

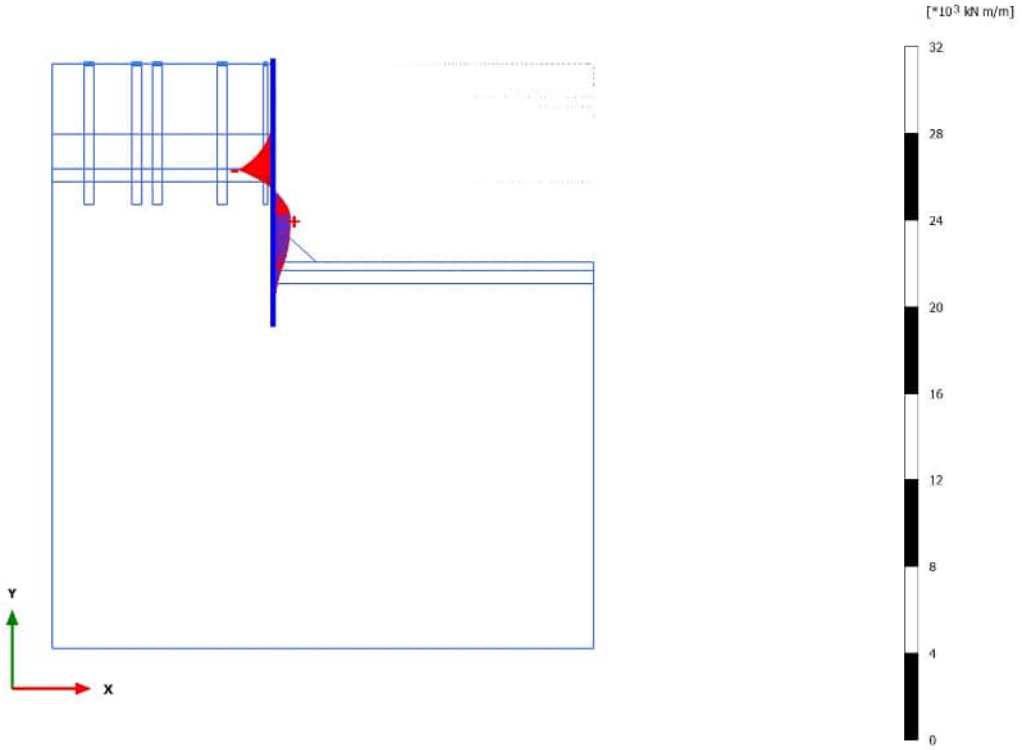


Bending moments M (scaled up 5.00*10^{-3} times)

Maximum value = 299.0 kN m/m (Element 49 at Node 32366)

Minimum value = -375.8 kN m/m (Element 35 at Node 31403)

3.1.1.2.8 Calculation results, Plate, Phase_9 [Phase_9] (9/166), Bending moments M

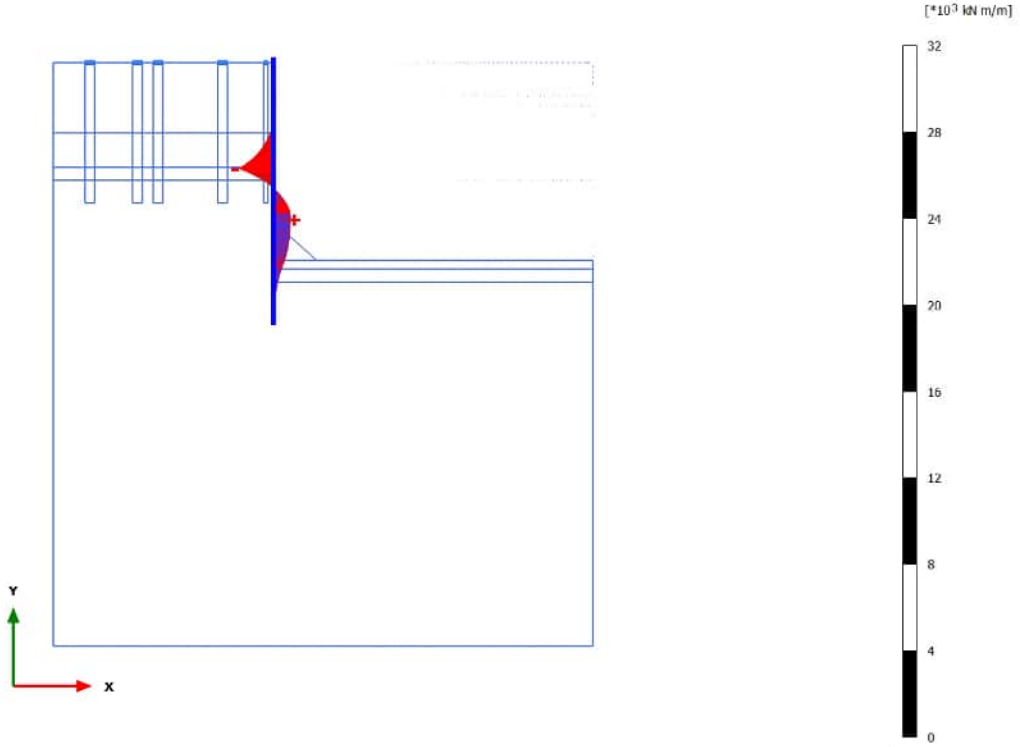


Bending moments M (scaled up 2.00*10^{-3} times)

Maximum value = 797.9 kN m/m (Element 69 at Node 53154)

Minimum value = -1584 kN m/m (Element 57 at Node 36587)

3.1.1.2.9 Calculation results, Plate, Phase_10 [Phase_10] (10/168), Bending moments M

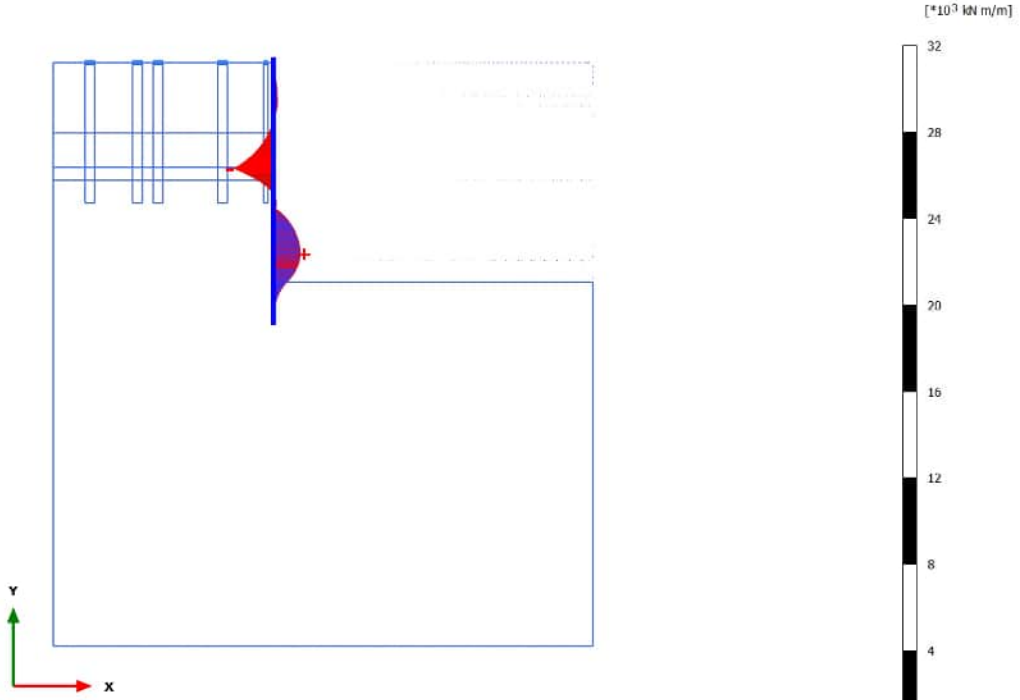


Bending moments M (scaled up 2.00*10^{-3} times)

Maximum value = 797.9 kN m/m (Element 69 at Node 53154)

Minimum value = -1584 kN m/m (Element 57 at Node 36587)

3.1.1.2.10 Calculation results, Plate, Phase_11 [Phase_11] (11/218), Bending moments M

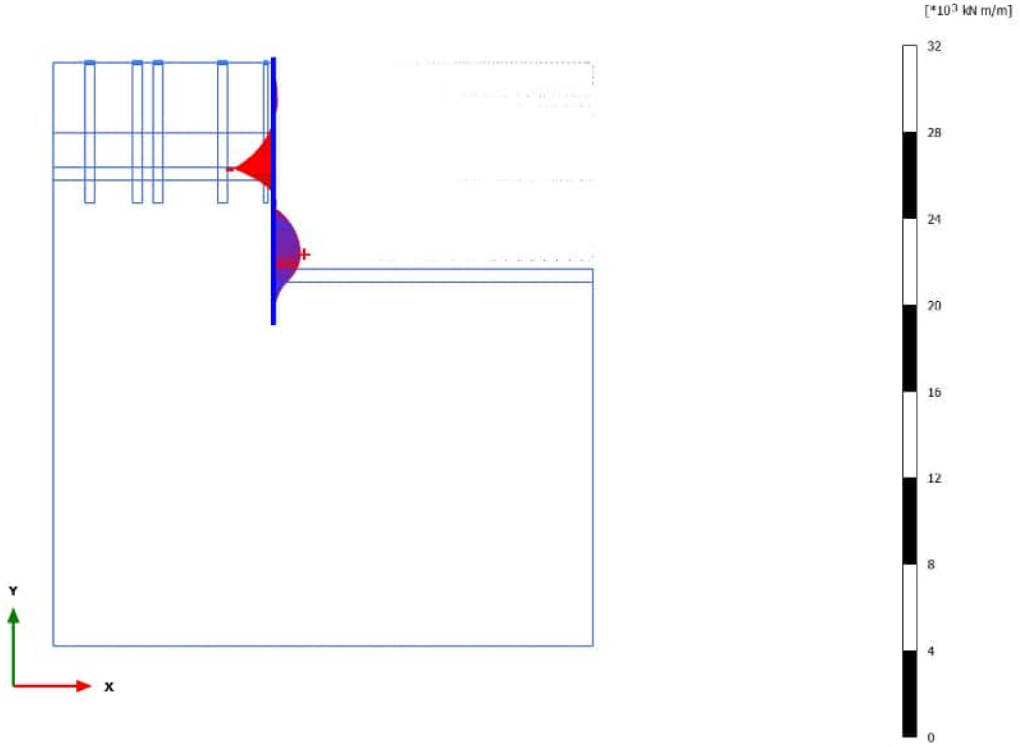


Bending moments M (scaled up 2.00*10^{-3} times)

Maximum value = 1243 kN m/m (Element 72 at Node 55745)

Minimum value = -1816 kN m/m (Element 58 at Node 36587)

3.1.1.2.11 Calculation results, Plate, Phase_12 [Phase_12] (12/221), Bending moments M

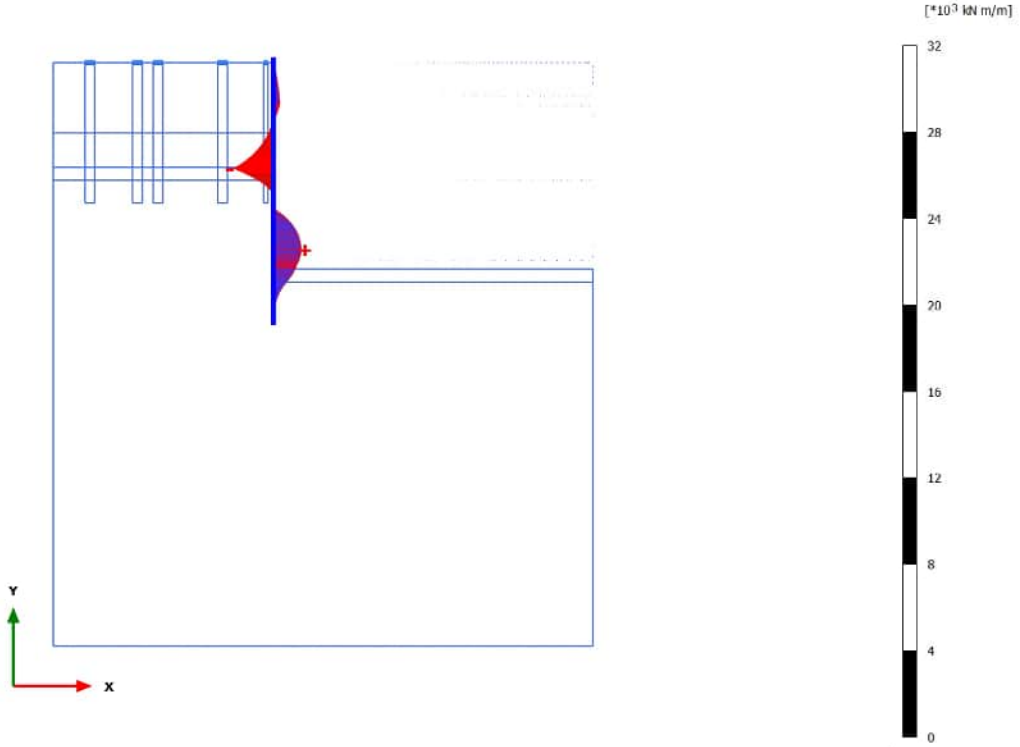


Bending moments M (scaled up 2.00*10^{-3} times)

Maximum value = 1252 kN m/m (Element 72 at Node 55745)

Minimum value = -1822 kN m/m (Element 57 at Node 36587)

3.1.1.2.12 Calculation results, Plate, Phase_13 [Phase_13] (13/226), Bending moments M

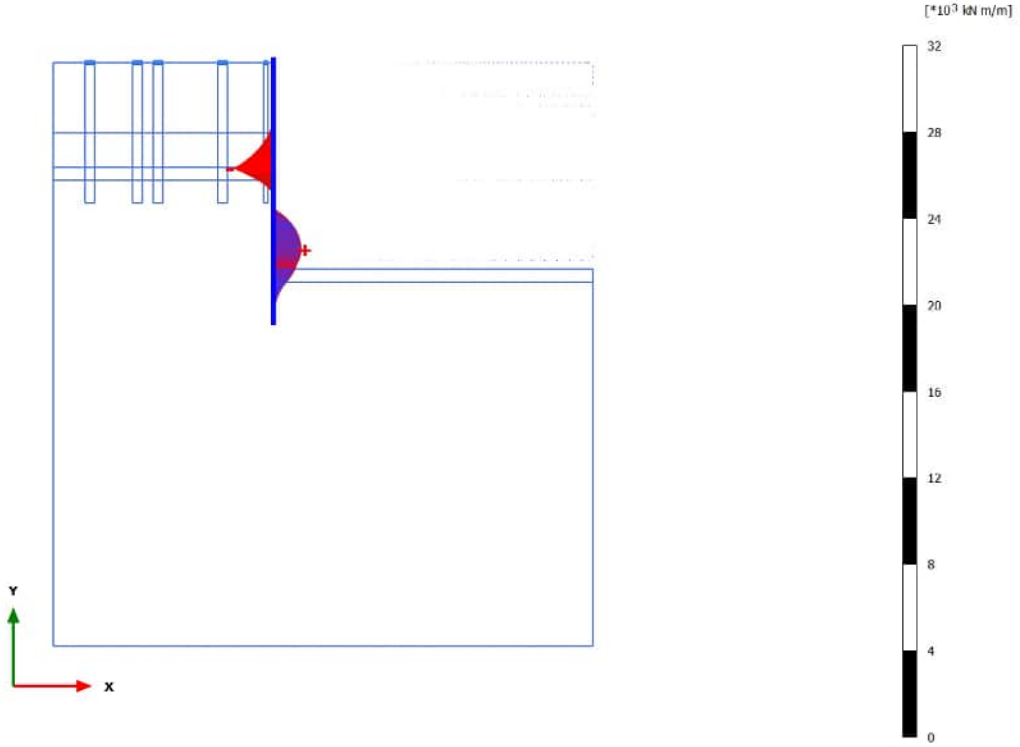


Bending moments M (scaled up 2.00*10^{-3} times)

Maximum value = 1279 kN m/m (Element 72 at Node 55525)

Minimum value = -1830 kN m/m (Element 57 at Node 36587)

3.1.1.2.13 Calculation results, Plate, Phase_14 [Phase_14] (14/229), Bending moments M

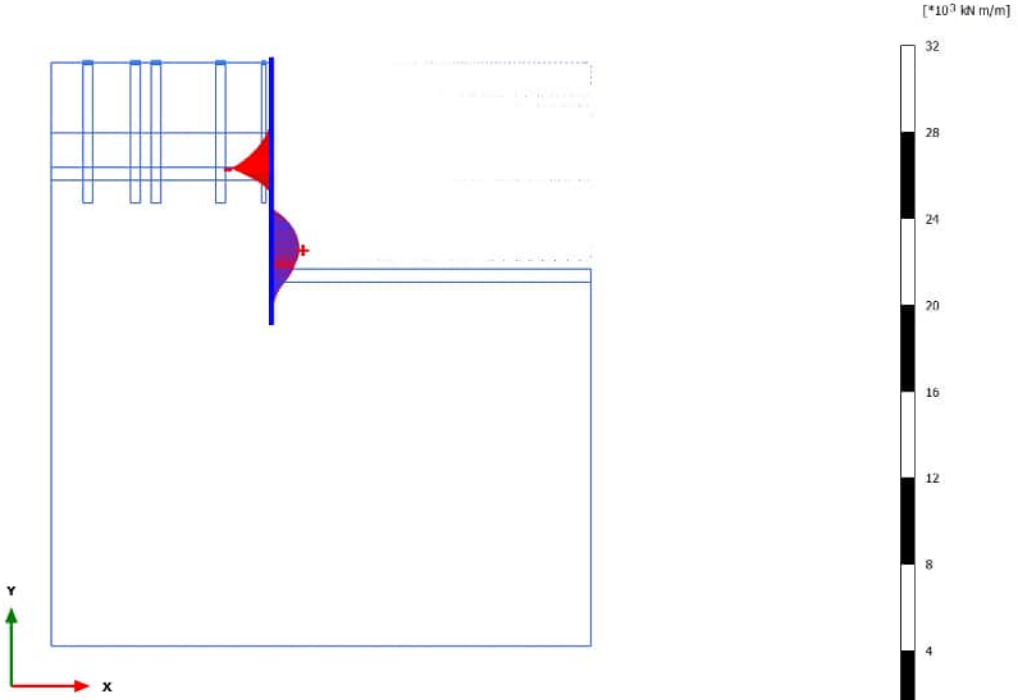


Bending moments M (scaled up 2.00*10^{-3} times)

Maximum value = 1279 kN m/m (Element 72 at Node 55525)

Minimum value = -1835 kN m/m (Element 58 at Node 36587)

3.1.1.2.14 Calculation results, Plate, Phase_15 [Phase_15] (15/245), Bending moments M

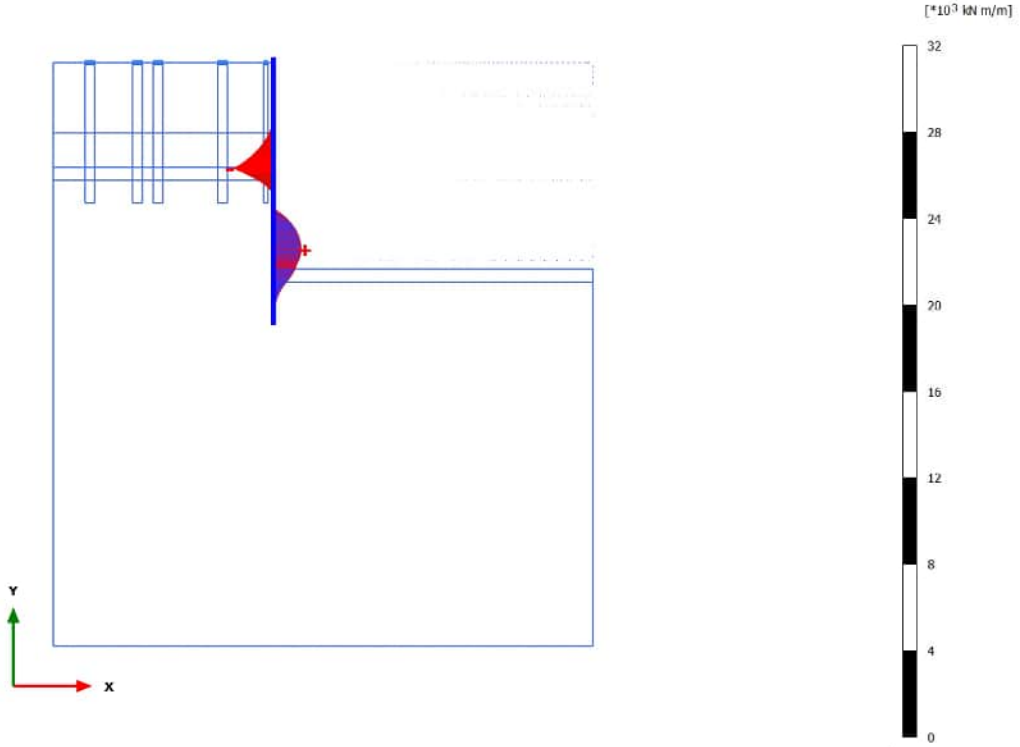


Bending moments M (scaled up 2.00*10^{-3} times)

Maximum value = 1279 kN m/m (Element 72 at Node 55525)

Minimum value = -1835 kN m/m (Element 58 at Node 36587)

3.1.1.2.15 Calculation results, Plate, Phase_16 [Phase_16] (16/248), Bending moments M

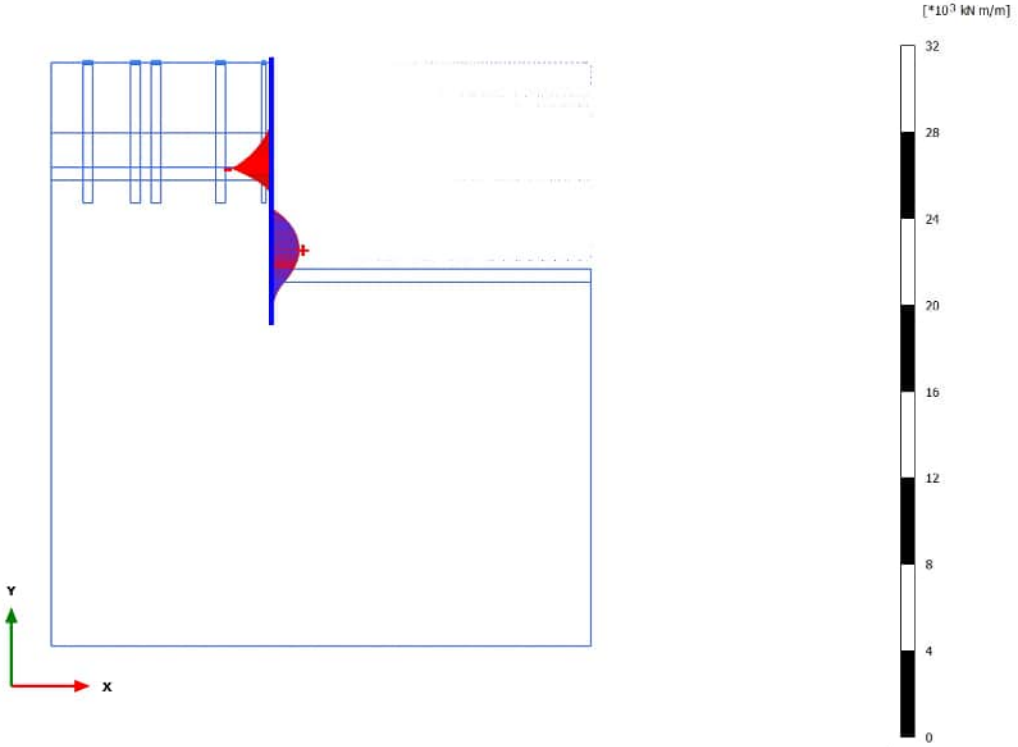


Bending moments M (scaled up 2.00*10^{-3} times)

Maximum value = 1279 kN m/m (Element 72 at Node 55525)

Minimum value = -1835 kN m/m (Element 58 at Node 36587)

3.1.1.2.16 Calculation results, Plate, Phase_17 [Phase_17] (17/253), Bending moments M

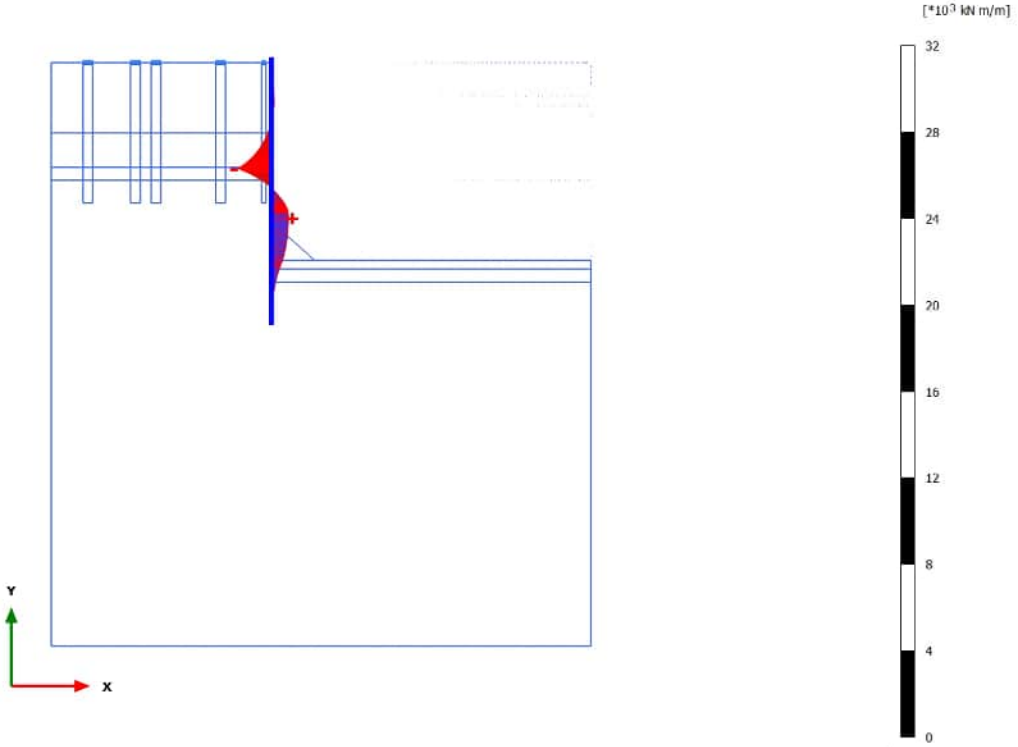


Bending moments M (scaled up 2.00*10^{-3} times)

Maximum value = 1289 kN m/m (Element 72 at Node 55525)

Minimum value = -1815 kN m/m (Element 58 at Node 36587)

3.1.1.2.17 Calculation results, Plate, Phase_18 [Phase_18] (18/258), Bending moments M

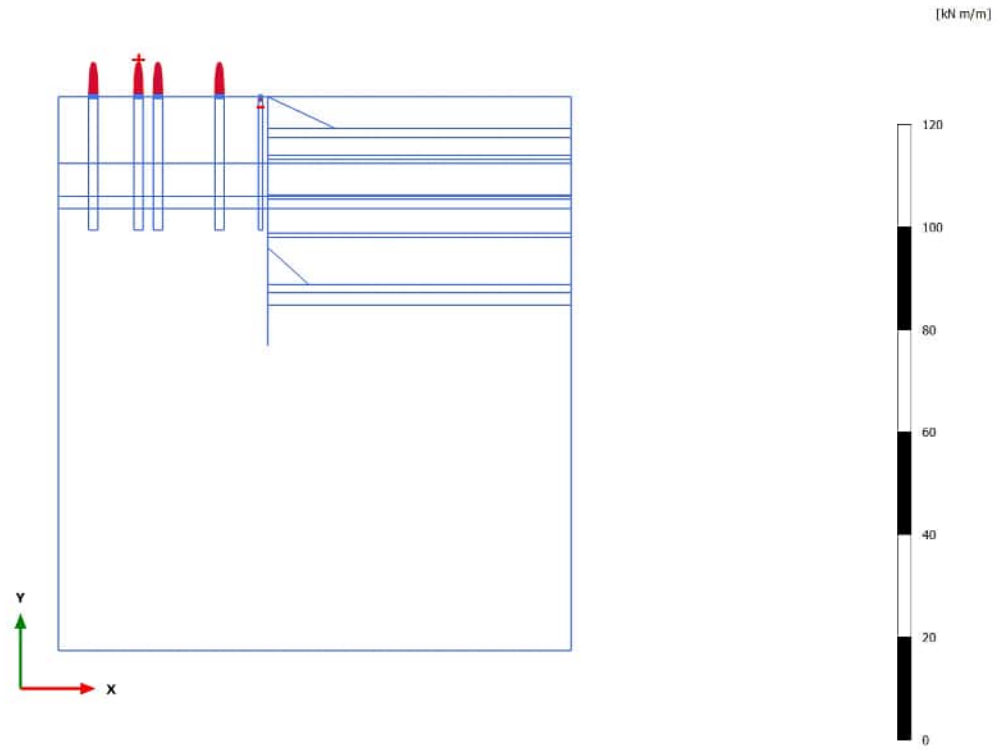


Bending moments M (scaled up 2.00*10^{-3} times)

Maximum value = 802.3 kN m/m (Element 69 at Node 53155)

Minimum value = -1544 kN m/m (Element 58 at Node 36587)

3.1.1.2.18 Calculation results, Plate, Phase_1 [Phase_1] (1/310), Bending moments M

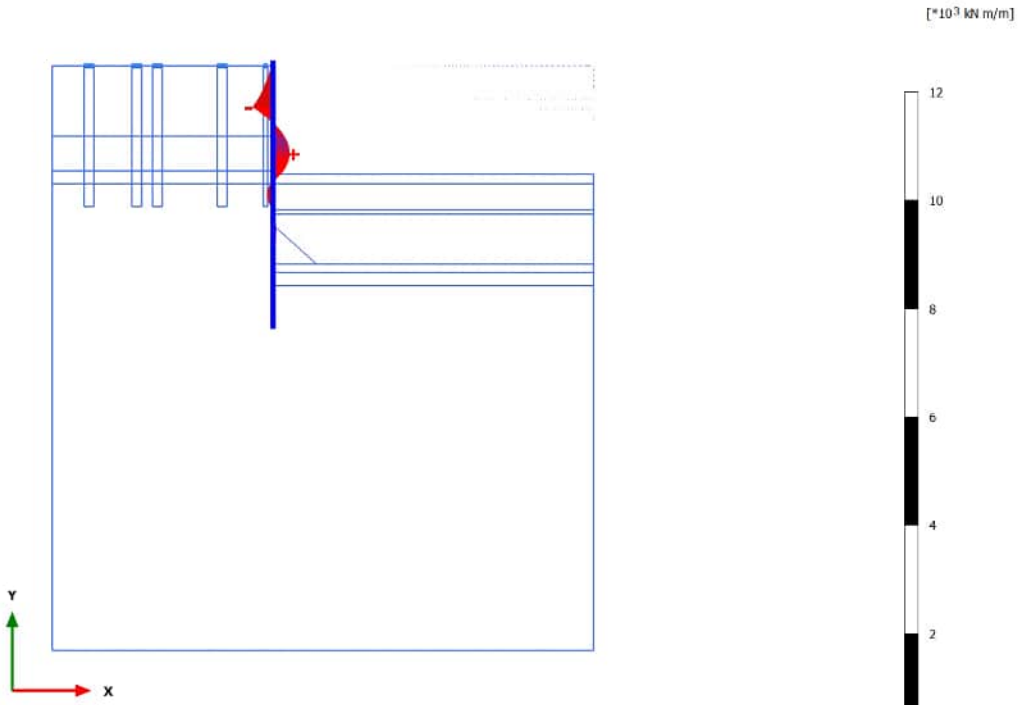


Bending moments M (scaled up 0.500 times)

Maximum value = 6.723 kN m/m (Element 11 at Node 72412)

Minimum value = -0.9821 kN m/m (Element 27 at Node 25761)

3.1.1.2.19 Calculation results, Plate, Phase_7 [Phase_7] (7/339), Bending moments M



Bending moments M (scaled up 5.00*10⁻³ times)

Maximum value = 298.9 kN m/m (Element 49 at Node 32366)

Minimum value = -375.7 kN m/m (Element 36 at Node 31403)

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APPENDIX B
Supplied Information

REPORT

Tonkin+Taylor

Downtown West - Downtown Car Park Redevelopment

Geotechnical and Groundwater Assessment Report

Prepared for
Precinct Properties Holdings Limited
Prepared by
Tonkin & Taylor Ltd
Date
December 2024
Job Number
1016043.1000 v6



Together we create and sustain a better world
www.tonkintaylor.co.nz

Document control

Title: Downtown West - Downtown Car Park Redevelopment					
Date	**Version**	**Description**	**Prepared by:**	**Reviewed by:**	**Authorised by:**
9/06/23	0	50% issue for Client review	RXSW	ABL	PJM
9/08/23	1	90% issue for Client review	RXSW	ABL	PJM
7/09/23	2	100% issue for Client review	RXSW	ABL	PJM
4/06/24	3	Updated 100% issue revised basement layout for Client review	RXSW	ABL	PJM
10/07/24	4	Updated 100% issue for Client review	RXSW	ABL	PJM
31/07/24	5	Final issue	RXSW	ABL	PJM
06/12/24	6	Updated final in response to s92 comments	RXSW	ABL	PJM

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Appendix G Basement Retention Analysis
Appendix H Combined Settlement Profiles

Executive summary

Precinct Properties Holding Limited (Precinct) has engaged Tonkin and Taylor Limited (T+T) to provide conceptual geotechnical, environmental engineering, civil and infrastructure consultancy services for the proposed Downtown West – Downtown Car Park Redevelopment. This report presents the concept geotechnical design and groundwater assessment for the development of the Downtown Car Park site as per our proposal dated 14 February 2023, with the key findings and conclusions summarised in the table below.

Sections within the main body of the report contains further information and discussion, which puts into context the findings that are encapsulated within this summary. The findings and conclusions presented in this concept report will be updated during the next stage of design.

Key Consideration	Key findings and conclusions
Ground and groundwater model	The ground conditions beneath the Downtown Carpark site generally consists of Reclamation Fill, marine sediments and Tauranga Group sediments overlying East Coast Bays Formation (ECBF) rock. The elevation of the rock typically varies between RL -2 m on the south-east corner to RL -7 m over the north-west portion of the site. A 6-level basement is estimated to be at least 7 m below the deepest rock level. 5-levels of basement extend across the site footprint with the finished floor level for the 5th level at RL -14.3 m and the base of the excavation assumed to be at RL -16.3 m to accommodate the basement slab and over excavation for the foundation works. Basement level six accommodates the lift pits and water tanks, with the excavation level for these works extending up to RL -21 m. Groundwater levels for concept design are based on groundwater monitoring from the Quay Street Seawall strengthening project to the north, Commercial Bay and CRL tunnel projects to the east. The groundwater regime for the site is governed by the presence of the Waitemata Harbour 50 to 150 m north of the site, and in particular tidal recharge through service conduits. The site may also be impacted by the infilling of the graving dock and the conduit it may provide to the Waitemata Harbour. For concept design, a design static groundwater level has been adopted at **RL 1.4 m**. An elevated groundwater level has been adopted at **RL 2.5 m**. Further information on the ground and groundwater model is provided in **Section 3.** Both the ground and groundwater models will be confirmed in subsequent design phases through additional site-specific geotechnical investigations.
Retaining wall type for basement excavation	Sheet pile walls and diaphragm walls have been considered to retain the proposed basement excavation for concept design. Key requirements for the concept design include: • Top-down construction at the northern end of the site utilising permanent floor slab and temporary diagonal props. The permanent floor slab used to prop the northern side will extend part way along the north-eastern and north-western boundaries to transfer the loads acting on the floor slab from the northern wall. • Bottom-up construction for the remaining portion of the site providing access to the excavation from the south. • Anchored sheet piles along the southern and south-eastern sides of the site where rock is shallow (typically between RL -2 to -4 m). To achieve groundwater cut-off, the sheet piles are proposed to be embedded a minimum 1 m into the ECBF rock. The exposed rock face will be spot bolted and meshed as required. • 800 mm wide diaphragm wall that is either tied back using anchors or internally propped for the remaining excavation sides where rock is deeper. The

Key Consideration	Key findings and conclusions
	diaphragm wall is proposed to be full depth and taken a minimum 4 m below the base of the excavation (i.e. at least to RL -20.3 m). Further discussion and details for the proposed retention solution is provided in **Section 4.4**. Note there remains an option to use diaphragm wall instead of the proposed sheet piles or consider other retention options (such as secant pile walls), as the design for the overall development develops. The retaining wall options will be developed further in subsequent design phases.
Assessment of deformation and settlement effects	The potential for surface deformation of the surrounding ground due to the development has been assessed, with the key findings as follows. • The estimated ground settlement below neighbouring structures is estimated to be less than 12 mm, with differential gradients generally less than 1V:500H. Risk Category 1 in accordance with CIRIA PR30, with negligible damage to surrounding buildings expected due to the proposed development. • Total settlement less than 20 mm is estimated for surrounding pavement and underground services. The differential gradients, from settlements where services are oriented perpendicular to the excavation, are likely to be less than 1V:1,000H and is within the allowable tolerance for the various types of underground services surrounding the site.
Earthworks and construction considerations	The soil and rock materials are excavatable using conventional equipment and plant. Due to the complex historical land-use of the site, in-ground obstructions can be present in the form of cobbles, boulders and construction waste in the reclamation fill, remnants of the graving dock, old seawalls and associated structures, reinforced concrete piles for the existing Downtown Carpark building, and remnant foundations of historical commercial and workshop buildings that occupied the north-western and south-eastern portions of the site. Noise and vibration, due to driving of sheet pile walls, will also need to be considered as part of the construction noise and vibration monitoring plan.

1 Introduction

Precinct Properties Holding Limited (Precinct) has engaged Tonkin and Taylor Limited (T+T) to provide conceptual geotechnical, environmental engineering, civil and infrastructure consultancy services for the proposed Downtown West – Downtown Car Park Redevelopment (Downtown West).

This report presents the concept geotechnical design and groundwater assessment for the development of the Downtown Car Park site as per our proposal dated 14 February 2023. This report is intended to be used to support the Resource Consent application for Downtown West and should be read in conjunction with the civil infrastructure and flooding concept design report and the preliminary contaminated land site management report.

1.1 Scope of work

This concept geotechnical report is based on a desk study of existing geotechnical and groundwater information available at and near the site and our historical knowledge of the site and surrounding areas. Site-specific geotechnical investigations will be undertaken in subsequent design phases to confirm the ground and groundwater conditions assumed for the concept design.

The scope of work that has been undertaken for this concept design is as follows.

1 Development of preliminary ground and groundwater models using existing geotechnical information.
2 Development of preliminary soil and rock parameters for concept design.
3 Development of conceptual retaining wall types for the basement excavation, and a discussion of benefits and limitation of each retaining wall type for the site-specific constraints.
4 Development of conceptual foundation types and foundation capacities for preliminary geotechnical and structural design.
5 Preliminary assessment of effects due to the basement excavation on surrounding structures including impact of groundwater drawdown.
6 Discussion of construction methodologies and construction considerations.
7 Recommendations for geotechnical investigations during the preliminary design phase and further work required during subsequent design stages.

A draft groundwater and settlement monitoring and contingency plan has also been prepared for the site and is provided in Appendix B.

The current understanding of the proposed development is provided in Section 1.3 below. The geotechnical design for the redevelopment will be developed in parallel to the architectural, structural, civil and infrastructure design of the development.

1.2 Site description

The Downtown West site is located at 2 Lower Hobson Street, at the corner of Lower Hobson Street and Customs Street West within the Auckland CBD (legal description Lot 9 DP 60151. The proposed site to be developed is approximately 6,442 m^2 in area, with the site on land reclaimed in stages between 1850 and 1920 by the Auckland Harbour Board.

The site is relatively level, with the ground surrounding the site varying from RL 4 m to RL 5 m. A 9-storey (1 below ground level and 8 suspended floors) public carpark building known as the Downtown Carpark currently occupies the site. The carpark building was designed in 1968 and the concrete superstructure is supported on belled reinforced concrete piles with precast flooring. The building is approximately 70 m wide (parallel to Customs Street West) and 88 m long (parallel to Lower Hobson Street). The building is surrounded by MSocial Hotel immediately to the north, HSBC

and Aon Towers to the east, Custom Street West and Lower Hobson Street to the south and west respectively. Figure 1.1 shows the location of the site. The historical Tepid Baths are located across Lower Hobson Street to the west.

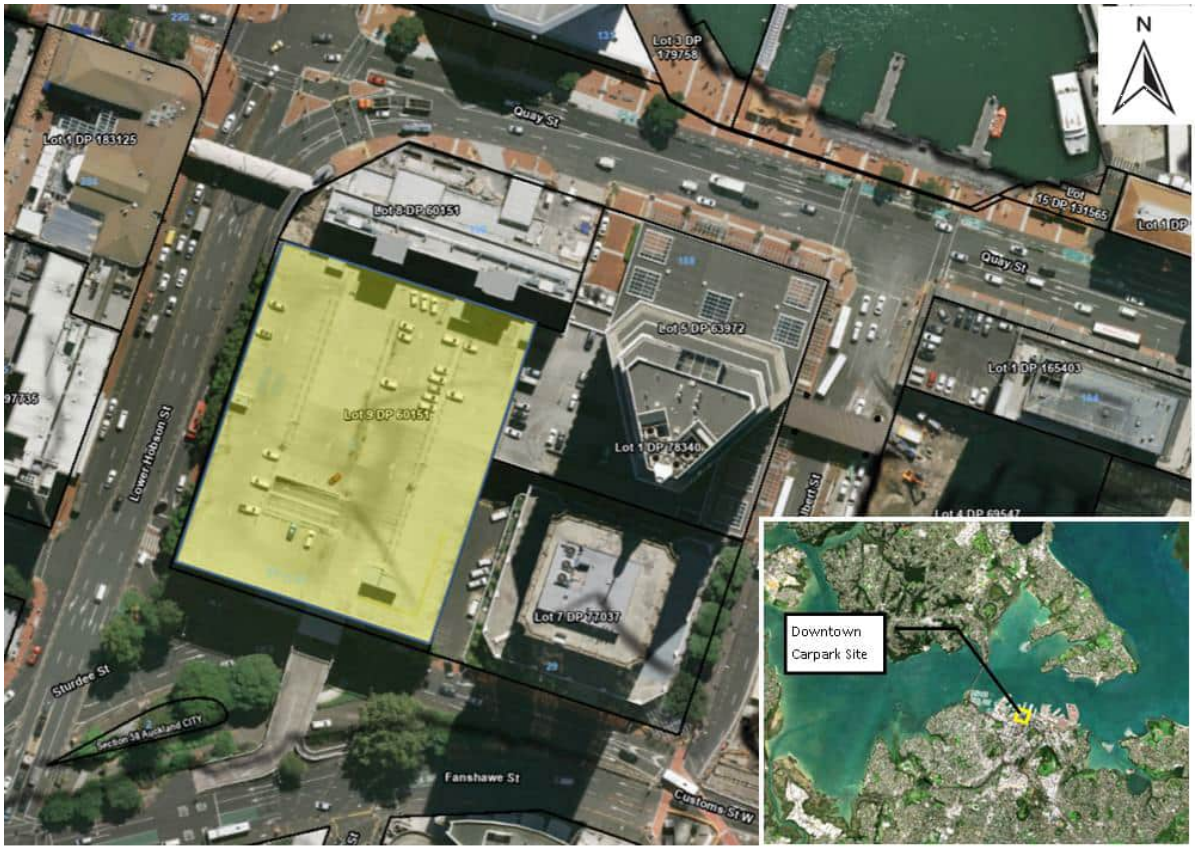


Figure 1.1: Downtown West site.

1.3 Proposed development

The proposed development includes the demolition of the existing downtown carpark building (together with the Lower Hobson Street pedestrian bridge and Customs Street West vehicle ramp located within part of the road reserve) and redevelopment of the site to provide for a mixed-use precinct providing for commercial, residential, retail, food and beverage and civic uses. The redevelopment involves three podium buildings, two towers and six levels of shared basement, including new public spaces and a new laneway network to provide connectivity within the city centre. In addition, the proposed development involves modifications to the podia of existing adjacent buildings (HSBC and AON) to facilitate the new laneway network.

The basement floor levels, and footprint are based on the Proposed Architectural Drawings for Resource Consent provided by the Architect, Warren and Mahoney and dated 9 December 2024. The following design floor levels have been adopted for this assessment:

- B5 Basement Level: FFL -14.3 m RL.
- B4 Basement Level: FFL -9.7 m RL.
- B3 Basement Level: FFL -5.1 m RL.
- B2 Basement Level: FFL -2.1 m RL.
- B1 Basement Level: FFL +0.90 m RL.
- GF Level: FFL +4.5 m RL.

An excavation level to -16.3 m RL has been adopted to allow for the basement slab and any over excavation.

An additional basement level B6 accommodates lift pits and water tanks, with the finished floor level for the lift pits and water tanks varying from FFL -17.3 m to FFL -19.8 m. The excavation for B6 level is located away from the perimeter of the site. It has been assumed that the excavation may need to extend up to -21.0 m RL to accommodate these works.

2 Site history

Historical information relating to the site has been collected from a variety of sources including the Auckland Council property file, site contamination enquiry, historical aerial photographs, archaeological assessments, and T+T project archives. This history focuses on on-site activities, except for the aerial photograph review where comments are also provided on readily observable surrounding land use. The information reviewed is summarised in the following sections.

2.1 Archaeological information

Reclamation to provide additional land, in addition to deeper harbours, was a central part of the early vision for Auckland. Figure 2.1 shows the original coastline relative to the subject site.

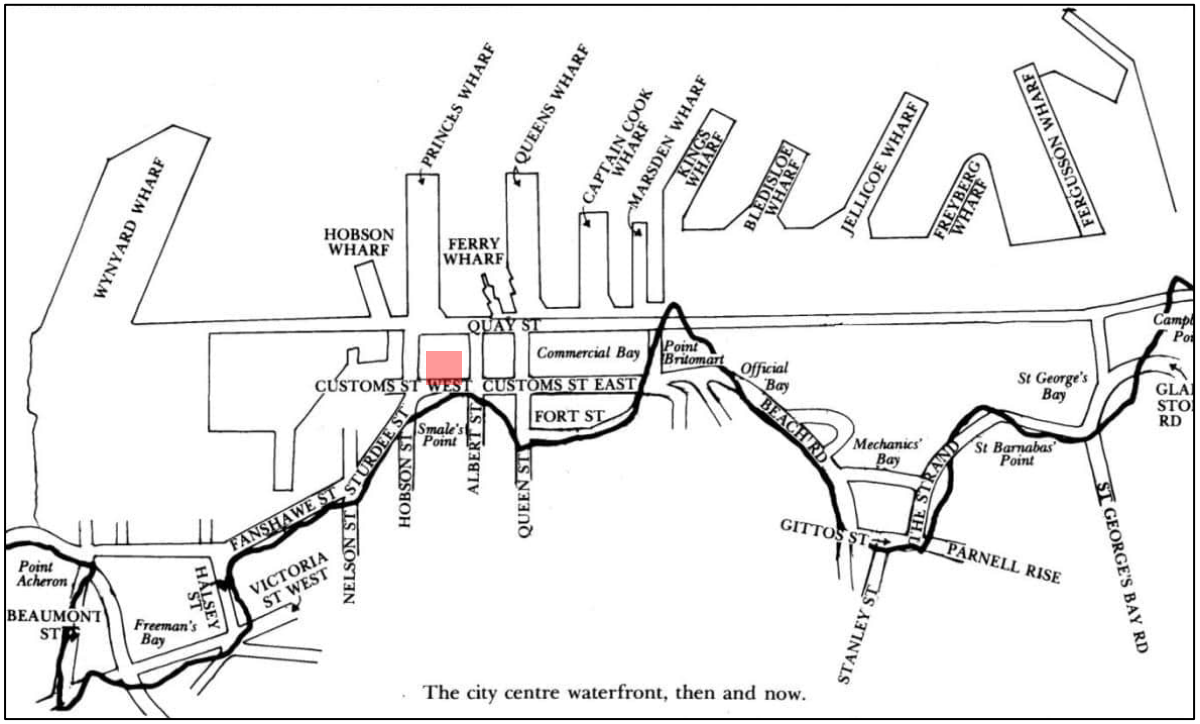


Figure 2.1: Reclamation and historical features in project area (from Barnett, 1981[1]).

Archaeological assessments completed by Clough & Associates[2,3,4] provide detailed accounts of the history of Auckland CBD, including the process of reclamation. The key findings are summarised below:

- Reclamation has been carried out in stages since the 1850s, with the first major reclamation to infill the area between Fort Street and Custom Street East starting in 1859. The process of construction involved creation of a seawall along what is now Customs Street East, with the use of material from Smales Point (the western headland) to provide fill.
- Throughout the 1870s, and early 1880s, a basalt sea wall along present-day Quay Street was constructed. Point Britomart was demolished, with manual labour (including horse drawn

[1] Barnett, S. 1981. A Picture Book of Old Auckland. Auckland: Benton Ross Ltd.
[2] Clough & Associates Ltd (2012), Key Historical Themes in the Development of the CBD Waterfront. Report prepared for Auckland Council.
[3] Clough & Associates Ltd (2014), Proposed Redevelopment, Downtown Shopping Centre, Queen Street, Auckland Central: Archaeological Assessment. Report prepared for RCP and Precinct Properties Ltd, dated October 2014.
[4] Clough & Associates Ltd (2018), Quay Street Seawall Upgrade Princes Wharf Section: Archaeological Assessment. Report prepared for Auckland Transport, dated April 2018.

carts) used to move the excess materials to points of ongoing reclamation. Dredging waste and construction debris were also noted to be used as infill.

- Throughout the 1870s-1900 existing wharves were removed, and drains constructed throughout the reclamations. Commercial/industrial buildings were constructed once the infill had stabilised, and the reclamation was complete. The site extends over an old graving dock which was backfilled, and the area above developed in the early 1900's (see Figure 2.2).

2.2 Historical aerial photographs

Historical aerial photographs from the T+T library and other sources (as shown in Table 2.1) have been reviewed. Relevant features of the site and surrounding land are summarised from each aerial photograph in Table 2.1.

Table 2.1: Summary of aerial photograph review

Date, run number and source	Key site features	Surrounding land features
1940 Auckland Council GeoMaps	Sturdee Street divides the site in a south-west to north-east alignment. The larger northern section of the site is vacant and appears to be either grass or gravel covered. Parts of it is being used for materials lay down / parking purposes. The smaller, triangular shaped southern section of the site is occupied by a single commercial building.	General port activities occupy the surrounds to the north. Commercial buildings occupy the immediate surrounds to the west, east and south.
1950 T+T Library (Run 1917 Photo 29)	A large commercial building has been established on the northern part of site.	No significant changes are evident.
1959 Auckland Council GeoMaps	No significant changes are evident.	No significant changes are evident.
1961 T+T Library (Run 3234 Photo 38)	No significant changes are evident.	No significant changes are evident.
1972 T+T Library (Run 4600 photo 11)	Sturdee Street no longer divides the site. The buildings that formerly occupied the northern and southern sections of the site have been removed and replaced by a single structure built over the entire site footprint including the former Sturdee Street road reserve.	A high rise building has been constructed immediately to the north between the site and Quay Street. And further development and construction of buildings has occurred in areas east of the site.
1980 T+T Library (Run SN5783/M14)	No significant changes are evident.	A large high-rise has been constructed immediately east of the site, replacing the smaller buildings that were established sometime between 1940 and 1950.
1987 T+T Library (Run SN8772/K4)	For the first time vehicles are visible parked on the roof of the building but no significant changes are evident.	Some redevelopment of buildings to the north and west, but no other significant changes are evident.

Date, run number and source	Key site features	Surrounding land features
1996 Auckland Council GeoMaps	No significant changes are evident.	Some redevelopment of buildings to the west, but no other significant changes are evident.
2001 Auckland Council GeoMaps	No significant changes are evident.	PWC tower, adjoining the sites north-eastern corner, is under construction.
2003 to 2016 Auckland Council GeoMaps	No significant changes are evident.	No significant changes are evident.
2017 Auckland Council GeoMaps	No significant changes are evident.	Former Downtown Shopping Centre has been removed and redevelopment of the site, located to the east, is underway.

2.3 Council property file review

The following information was identified during review of the property file:

- An undated plan hand annotated with dates from 1911 and 1912 show a “dock” covers most of the site. This is consistent with Ports of Auckland records and early photographs of Auckland City T+T has obtained for other projects which indicate that a boat graving dock (dry dock) was originally constructed at the site in the 1870s. The graving dock was subsequently infilled, prior to the reclamation of Quay Street (between Princess Wharf and the site) in 1923. Figure 2.2 below provides the timeline for the construction and reclamation of the graving dock.
- Plans dated from 1911 through 1947 show that a 3 level “warehousing” building occupied the south-eastern corner of the site, to the south of the graving dock and Sturdee Street. The Harbour Board workshops were also constructed in the north-western corner in the early 1940s.
- Plans dated 1968 show the proposed development of a “Car Parking Station” for Auckland City Council. The building includes 8 levels of parking, including the basement and roof levels, with a “service station” shown to occupy the south-western corner of the ground floor.
- Plans dated 1970 prepared for Shell Oil New Zealand Limited show the general layout of the “Downtown Service Station” within the “Downtown Parking Building”.
- Correspondence dated 1973 and 1975 refer to the transfer of the service station operations from “Dock Site Service Centre Ltd.” To “Paine Services Ltd.” With the addition of a rental vehicle operation, including “a limited amount of servicing of rental vehicles”.
- A dangerous goods license dated 1996 refers to the removal of 2 underground tanks, of 13,000 and 18,000 litre capacities, from a site occupied by “Downtown Auto Services”. The site is noted as being used as “Service station” with the future use proposed to be a “Garage”.
- Subsequent records relate to:
  - Alterations to the former service station/garage area for occupation by various restaurants and bars.
  - A barbers shop is noted as occupying a tenancy within the wider site.
  - Various alterations and improvements, including the addition of two floors, to the car parking facilities.

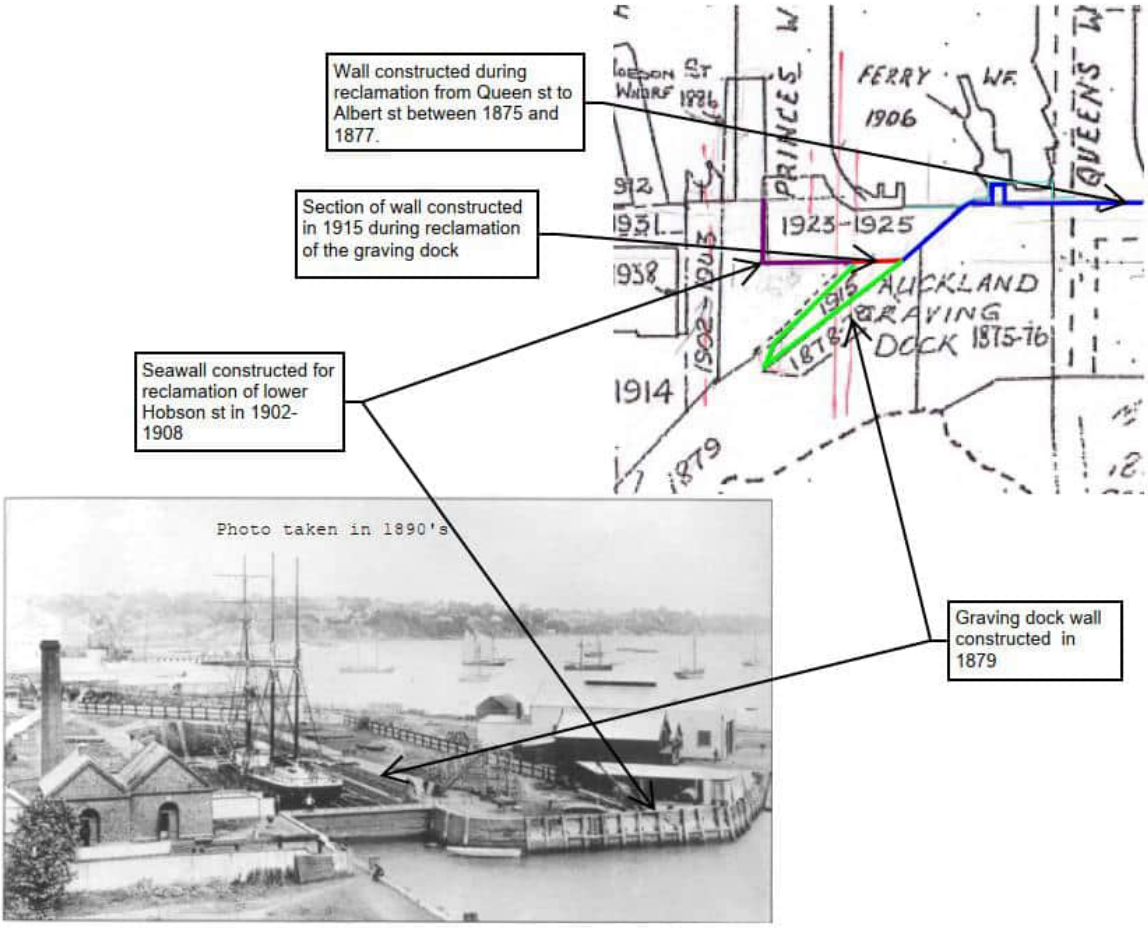


Figure 2.2: Construction and reclamation of the graving dock at the site.

2.4 Existing structure and remnant foundations

The existing carpark was designed in 1968 and comprises a concrete super structure with precast flooring and is founded on belled reinforced concrete piles. The ground floor of the structure is staggered where the south half is at a lower elevation than the north half. Building records indicate that the ground floor of the structure is a concrete slab on-grade, with some Fill likely beneath the floor slab.

The original 1968 design drawings show the length of piles are between 5.6 to 13.9 m, the shaft diameters are between 0.9 to 1.4 m and the bell diameters are between 1.5 to 2.3 m. These design drawings indicate that the pile lengths were to be adjusted during construction based on actual depth of rock. Based on geotechnical investigations in the area and the inferred elevation of the East Coast Bays Formation (ECBF) rock surface, and assuming a typical pile embedment of 3 m in the (ECBF) rock, the total pile lengths are estimated to vary between 6 m to 11 m. Pile lengths typically increase towards the north and west with the longest piles located in the northwest corner of the building where the depth to rock is deepest.

Design drawings for the existing Downtown Carpark structure showing the foundation layout are provided in Appendix C.

Prior to development of the Downtown Carpark structure, it is understood two multi-level structures occupied the site in south-east and north-west corners with the site split diagonally through the centre by Sturdee St. It is possible remnant foundations from these structures will still be present

below the site. Available records for the south-eastern structure (Appendix C) indicate the structure was built on shallow pad foundations. No records are available for the north-western structure, but it is likely to have been founded on driven timber pile foundations.

The majority of the piles will be exposed during the basement excavations but, where they conflict with the perimeter retaining walls or building piles installed from existing ground level, the piles may need to be removed to enable the new works.

3 Geotechnical model

3.1 Historical site investigations

Existing geotechnical and groundwater investigations at and near the Downtown West site have been used to develop a preliminary ground model for concept design, including:

- 8 No. machine drilled boreholes that were undertaken within the Downtown West footprint prior to construction of the existing carpark building in 1968.
- 2 No. machine drilled boreholes undertaken within the Downtown West footprint for the seismic assessment of the existing carpark building in 2020.
- Investigations at adjacent sites including investigations for the Millennium and Copthorne Hotels (now the MSocial Hotel) to the north, investigations for the Quay Street Seawall strengthening, and investigations for the development of the HSBC, AON, Commercial Bay developments, Watermark (85 Custom Street West) and Tepid Baths.

Existing groundwater monitoring information include groundwater levels measured using standpipe piezometers and vibrating wire piezometers over a period varying from 3 months to 3 years. The groundwater monitoring from the Quay Street seawall strengthening, Commercial Bay development and the CRL cut-and-cover tunnel excavation works has been used for this concept design.

A plan showing existing site investigations in the vicinity of the site is provided in Appendix A.

The ground and groundwater model for this concept design report was also supported by our historical knowledge of the area based on geotechnical and ground contamination assessment we have carried out in the vicinity between 1999 to 2022. An outline of these relevant works in the area are summarised in Table 3.1 below.

Table 3.1: Previous T+T investigations in the vicinity of the Downtown West.

Site	T+T Job Reference	Information sources
PwC Tower	16930	AMP Waterfront Tower Project – Geotechnical Report. Report prepared for AMP Asset Management Ltd, dated 1999.
Quay Street	28557.002	Quay Street Seawall Geotechnical and Ground Contamination Assessment. Report prepared for Auckland Transport, dated September 2012.
Copthorne Hotel	29493.1000vA	Ground Contamination Assessment, 196 Quay St, Auckland. Report prepared for Millennium & Copthorne Hotels NZ Ltd, dated November 2014. T+T reference no. 29493.1000; and Site Management Plan for Ground Contamination, 196 Quay St, Auckland. Report prepared for Millennium & Copthorne Hotels NZ Ltd, dated December 2014.
Tepid Baths	26753.001	Tepid Baths Redevelopment Geotechnical Report. Report prepared for Auckland City Council, dated September 2010.
West Plaza Centre, Albert Street	24700	3-15 Albert Street, Auckland Central, Geotechnical and Ground Contamination Desk Study Report. Report prepared for West Plaza Centre Limited, dated August 2007.
Precinct Downtown Shopping Centre	30108.3000v3	Downtown Development, Auckland, Preliminary Ground Contamination Assessment. Report prepared for Precinct Properties New Zealand Limited, dated January 2015.
Downtown Carpark	1012134	Downtown Carpark, Rock Level Investigation. Report prepared for Auckland Transport (AT), dated September 2020.

3.2 Geological setting

The ground conditions beneath the Downtown West generally consists of Reclamation Fill, marine sediments and Tauranga Group sediments overlying East Coast Bays Formation (ECBF) rock. The nature of these units is discussed in more detail in Section 3.3.

The geological boundary for the ECBF near the surface is located south of the site and is broadly consistent with the original "coastline" or cliff" that was shown in Figure 2.1. In addition, the geological map for Auckland shows Late Pliocene to mid-Pleistocene pumiceous river deposits belonging to the Tauranga Group immediately to the south of the site and Auckland Volcanic Field deposits consists of basalt and ash south east of the site.

Figure 3.1 presents an extract from the 1:250,000 geological map for the Auckland area is provided below showing the high-level geological setting in the vicinity of the site.

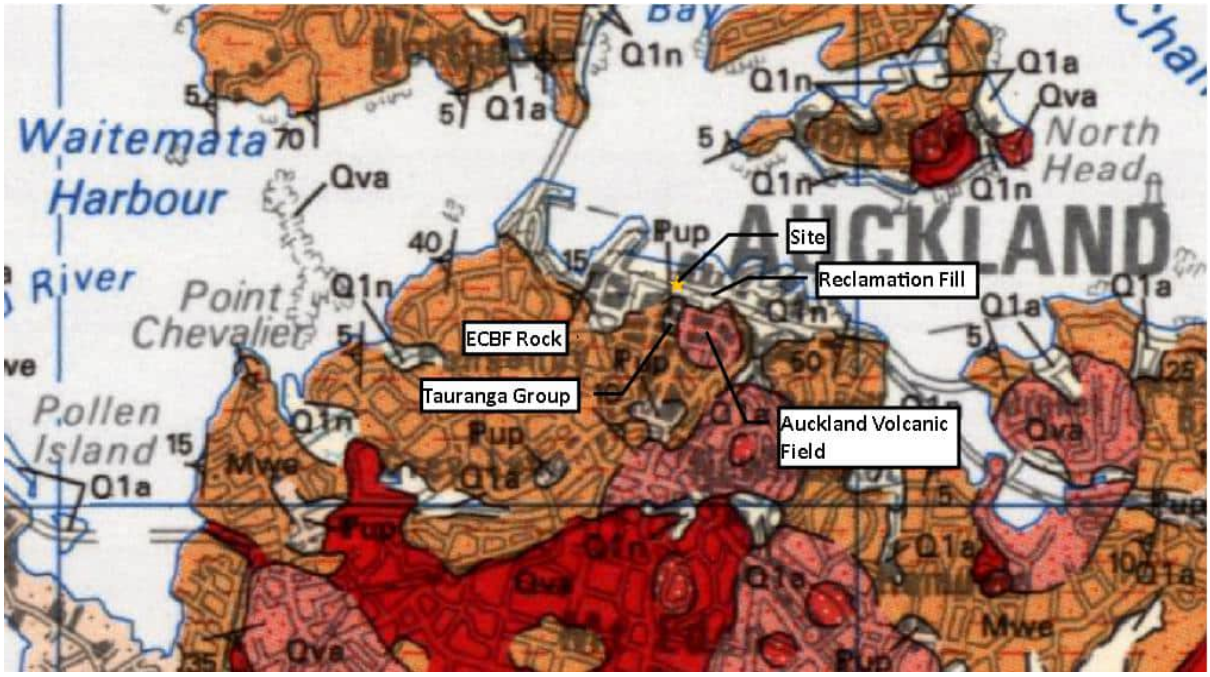


Figure 3.1: 1:250,000 Geological map of the Auckland area (Edbrooke, 2001).

3.3 Ground model

Based on historical investigations available at and near the site, the underlying ground conditions broadly consist of:

- Reclamation Fill; overlying.
- Recent marine sediments and Alluvium from the Tauranga Group; overlying.
- East Coast Bays Formation from the Waitemata group.

A discussion of each of the above units is provided below. Preliminary geotechnical cross-sections showing the variability in ground conditions is provided Appendix A. Note that the nature and continuity of the subsoils away from investigations locations is inferred, and it must be appreciated that actual conditions could vary from the assumed model. Further site investigations are proposed in the preliminary design stage to refine the geotechnical cross-sections presented here further.

Geotechnical parameters that have been adopted for the concept analysis and design are provided in Section 4.

3.3.1 Reclamation fill

Reclamation fill materials are present below ground level across the site. The thickness of the reclamation fill varies between 4 and 9 m and is generally comprised of both locally sourced and imported fill materials, dredged materials and Reclamation fill together with debris from earlier construction of seawalls and structures. The available records from boreholes within the site indicate that variable gravels, sands and soft to very stiff silts and clays will be encountered within the fill layers, with occasional basalt boulders, organics, timber, brick, porcelain, and other rubble.

As noted previously, the graving dock that was located across the site was infilled in 1923. The fill within the old graving dock was excavated from local sources of regional basement rock, typically comprising fine-grained cohesive soils (silts and clays). The old seawalls are reported to have been constructed using timber and masonry breast works constructed on rock rubble. Auckland Harbour Board drawings indicate that the graving dock walls may have later been replaced with mass concrete wall. It is unknown whether all or part of the walls were removed and to what extent any remnants of these walls were encountered during construction of the existing structure. As such, remnants of these walls may still be present at the site and encountered during the excavation of the basement for the Downtown West development, see also further discussion in Section 4.6.4.

3.3.2 Recent Marine Sediments and Alluvium (Takaanini Formation)

Tauranga Group sediments include recent marine “muds” typically comprising soft to stiff sandy silts and clays with significant organic content; and underlying Pleistocene-era alluvial sediments typically comprising soft to stiff pumiceous clays, silts and sands with some organic layers. Based on available geotechnical investigations this layer typically varies between 1 and 4 m in thickness.

3.3.3 East Coast Bays Formation rock

ECBF rock underlies the Auckland CBD, and typically comprises interbedded very weak to weak siltstone and sandstone. This unit often shows a well-developed weathering profile consisting of sands, silts and clays depending on the original parent lithology.

Historical records indicate the former sea cliffs prior to land reclamation were present to the south of the site (see Figure 2.1). The weathering profile in the top of the rock at the site has been affected by the historical coastal erosion processes in this area. The pre-European shoreline at the Downtown West site is more or less along the boundary with Customs Street, and as a result the rock in this area is highly variable, because of the presence of wave cut platforms, and possible caves and small cliffs or other steep rock interfaces. Natural Tauranga Group sediments and manmade fill were subsequently deposited over this rock surface.

3.4 Groundwater model

The groundwater model for the concept design is based on historical ground monitoring available from the Quay Street seawall, Commercial Bay development and CRL cut and cover tunnel works. The available monitoring data are discussed further below. Further site-specific ground water monitoring will be undertaken in the subsequent design phases to confirm the groundwater levels for design and construction.

Broadly the groundwater regime for the site is governed by the presence of the Waitemata Harbour 50 to 150 m north of the site, and in particular tidal recharge through service conduits. The site may also be impacted by the infilling of the graving dock and the conduit it may provide to the Waitemata Harbour. The site-specific groundwater levels are also considered to be impacted by the drained basements at the HSBC tower (formerly the PWC tower) to the east and West Plaza to the south-south east that extend below historical regional groundwater levels.

For concept design and resource consent application, groundwater levels have been adopted based on historical groundwater monitoring available in the vicinity of the site as presented in Appendix E. The monitoring results available from these positions, which include a number multi-level vibrating wire piezometer (VWP) installations, are plotted on Figure 3.2 below. The following conclusions can be drawn from the data:

- Static groundwater levels across the site up to RL 1.4 m have been observed. Groundwater pressures observed with depth across the multi-level VWP installations indicate full-hydrostatic pressures are not developed. Monitoring indicates the pressures are at approximately 85% of hydrostatic pressure.
- The groundwater levels are influenced by tides, however negligible fluctuations are observed (100 – 200 mm) within the VWP monitoring records available.
- Due to the close proximity to the harbour, it is considered that rainfall events have a negligible effect on the groundwater level but flows within fill material and stormwater pipes may locally affect groundwater levels. There is also potential for high connectivity with the harbour through open joints / faults which may be present within the ECBF rock.

For concept design, a design static groundwater level has been adopted at **RL 1.4 m**. An elevated groundwater level has been adopted at RL 2.5 m. These design cases should be confirmed by site-specific groundwater monitoring in subsequent design phase.

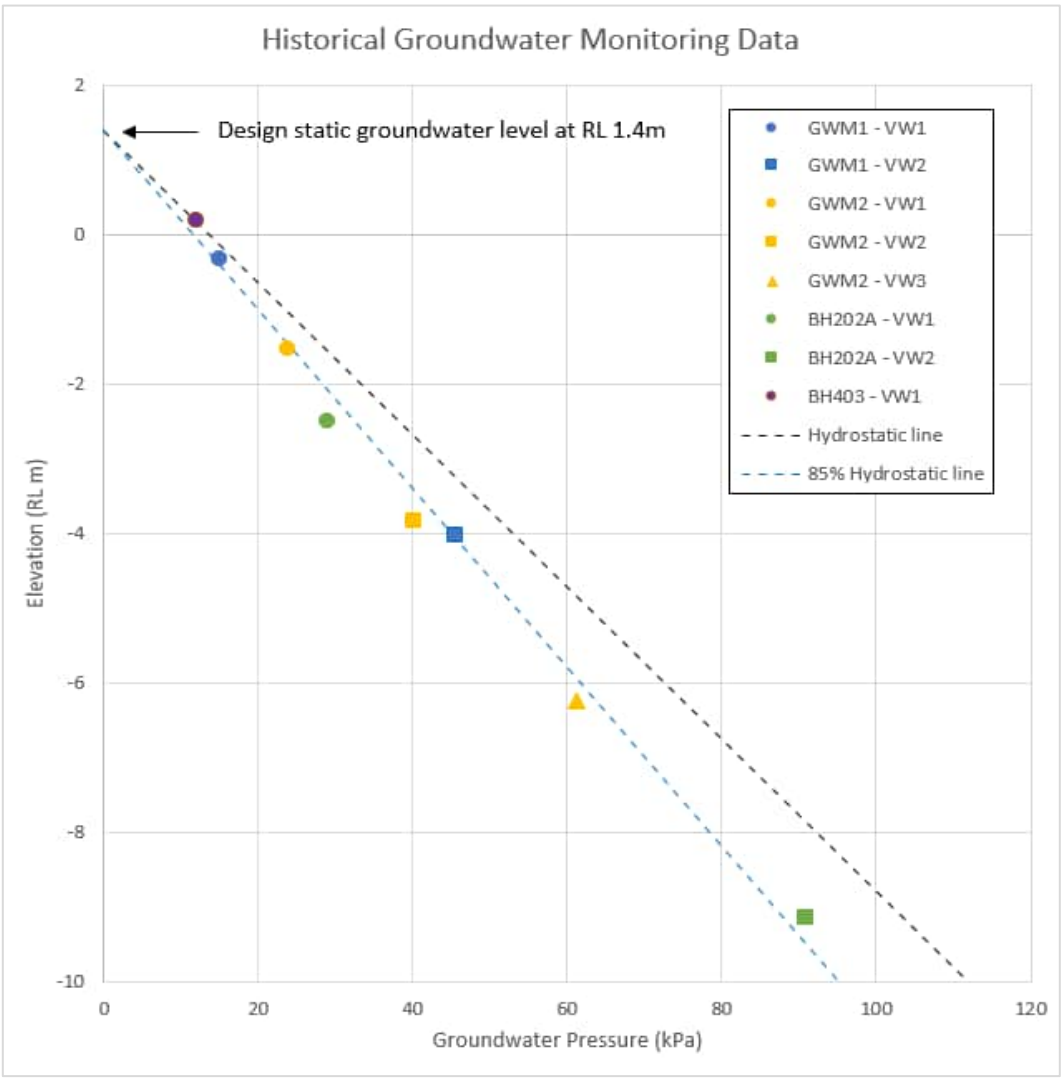


Figure 3.2: Historical groundwater monitoring records in the site vicinity.

4 Geotechnical implications for concept design

The assessment presented in this report is for the redevelopment of the site with a new structure consisting of up to six levels of basement, a podium structure over the site footprint and two towers.

For this assessment, the B5 basement, which extends over the site footprint, FFL has been assumed at RL -14.3 m. The bulk excavation level has been assumed at RL -16.3 m (allowing 1 m for basement slab and subbase, and 1 m over-excavation). The over excavation allows for site concrete, waterproofing, temporary sumps / underdrains and potential adjustments of excavation level as the design develops. This does not include local excavations for ground beams, water tanks and/or lift pits which are remote from the perimeter but may extend below this level. The likely localised excavation for water tanks and/ or lift pits has been accounted for in the groundwater drawdown assessment, with a localised excavation level of -21.0 m RL adopted. As the localised excavations are away from the perimeter walls, they are not expected to impact the retaining wall design.

For the assumptions noted above, a discussion of suitable foundation options, temporary and permanent groundwater considerations, retention options, seismic considerations and construction considerations are presented in subsequent sections.

4.1 Seismic considerations

4.1.1 Seismic shaking hazard

Seismic accelerations to be resisted by a structure are dependent upon the stiffness of the underlying soil/rock. Soft soils have the potential to amplify ground accelerations, requiring the structures built upon them to be designed to resist a higher seismic coefficient.

The ECBF rock level is expected to vary between approximate RL of -2 m to -7 m over the site footprint, and it is anticipated that the entire basement footprint would be founded upon ECBF rock (or at least <3 m weathered ECBF soil). Therefore, the site is considered to be classified as subsoil **Class B – Rock** in accordance with NZS 1170.5:2004 for the **design of building foundations**.

For the **design of retention structures** and to assess lateral actions on the basement walls, a subsoil **Class C – Shallow soil site** is considered appropriate to allow for the amplification of seismic accelerations within the retained Reclamation fill / marine sediments behind the wall.

The PGA for geotechnical design has been calculated in accordance with MBIE/ NZGS Module 1[5] and are summarised in Table 4.1 below. We have assumed a structure design life of 50 years and the new structure will be classified as Importance Level (IL) 3 to calculate the peak ground accelerations (PGA) for geotechnical design. The structure design life and IL should be confirmed with the structural engineer and architect as the design of the development progresses.

[5] MBIE/ NZGS Module 1. (2021). Overview of the guidelines.

Table 4.1: Design peak ground accelerations for geotechnical design

Design Element	Site Subsoil Class	Annual exceedance probability		PGA	Magnitude
Building Foundations	B – Rock	SLS	1 in 25	0.05	5.9
		ULS	1 in 1000	0.20 (0.19)	5.9 (6.5)
Retaining Walls	C – Shallow soil site	SLS	1 in 25	0.05	5.9
		ULS	1 in 1000	0.20 (0.19)	5.9 (6.5)

The lower bound PGA and Magnitude as per Module 1 (2021) is provided as bracketed values.

4.1.2 Liquefaction assessment

Liquefaction describes a process where significant excess pore water pressures are generated in typically loose, saturated, generally cohesionless soil during earthquake shaking, causing the soil to undergo a partial to near complete loss in strength and stiffness. The occurrence of liquefaction is dependent on several factors, including the intensity and duration of ground shaking, soil density, particle size distribution and the elevation of groundwater.

Based on our experience at the site and surrounding area, we expect thin layers within the Reclamation Fill and marine sediments may liquefy under ULS levels of earthquake shaking. Liquefaction is not expected to occur under SLS levels of earthquake shaking. Under ULS levels of earthquake shaking, typically only pockets of material are likely to liquefy and continuous liquefiable layers are unlikely to occur.

Liquefaction may occur within the soil retained by the perimeter retaining walls. Site specific investigations and more detailed analyses will be undertaken during detailed design to consider any increased lateral pressure loading on the basement walls if the surrounding material was to liquefy.

4.2 Foundation design considerations

Concept designs for the development indicate the superstructure will comprise two towers and three podium structures extending between approximately 30 m and 230 m above existing ground level, comprising up to 56 floor levels. Structures of this size will impose significant foundation loading, including large uplift loads from wind / seismic loading and groundwater uplift (where tanked basements are implemented).

The B5 and B6 basement levels are likely to be founded entirely below ECBF rock level across the site. This allows for consideration of either (or a combination of) the following foundation options:

- Shallow strip, pad or raft foundations bearing directly upon ECBF rock. Where required, ground anchors and/or tension piles may be required to resist high uplift loads.
- Piled foundations – comprising either driven steel piles or bored cast insitu concrete piles.

Further details on each option are provided in subsequent sections.

4.2.1 Shallow foundations

The subgrade across the assumed B5 and B6 basement level is anticipated to comprise of slightly to unweathered ECBF rock. Shallow foundations comprising of strip, pad or raft foundations bearing upon this stratum are considered suitable to support anticipated design loads.

A geotechnical ultimate bearing capacity of 4 MPa, with a strength reduction factor of ϕ_g = 0.5 may be used for preliminary design where the shallow foundations are directly bearing on ECBF rock. Foundations settlements are anticipated to be within typical building tolerances where the above values are adopted.

Where shallow foundations are considered, ground anchors or tension piles may be required to resist uplift loads. For the preliminary design of ground anchors that have a fixed length within ECBF rock, a geotechnical ultimate capacity of 1000 kPa, with a strength reduction factor of ϕ_g = 0.5 may be used. The minimum free length, maximum fixed length, borehole and strand diameters and the corrosion protection details for the ground anchors should be confirmed as part of detailed design.

Note the strength reduction factors noted above may be increased with site testing to prove ultimate geotechnical capacities.

4.2.2 Driven UC Piles

Driven steel UC piles embedded in ECBF rock are considered to be an appropriate founding method, although noise and vibration issues will need to be addressed as discussed further in Section 4.6.

Preliminary geotechnical design capacities for various UC pile sizes are presented in Table 4.2 below. The UCs should be driven to refusal with an appropriately sized hammer.

In accordance with AS2159[6], a strength reduction factor of ϕ_g = 0.5 should be applied to the geotechnical ultimate capacity, although the strength reduction factor could be increased to ϕ_g = 0.7 if 10% of the piles undergo Pile Dynamic Analysis (PDA) testing.

Table 4.2: Design parameters for Driven UC Piles into ECBF Rock

Pile Type	Geotechnical ultimate capacity (kN)	Maximum ULS load if PDA testing not undertaken (ϕ_g = 0.5) (kN)	Maximum ULS load if 10% of piles subjected to PDA testing (ϕ_g = 0.7) (kN)
310UC97	2,330	1,165	1,630
310UC137	3,390	1,645	2,370
310UC158	3,800	1,900	2,660
310UC198	4,750	2,375	3,325
305x305x240	5,760	2,880	4,030
356x406x287	6,890	3,445	4,820
356x406x393	9,430	4,715	6,600

Note: Larger steel sections may be used but are not available ex stock in NZ. Equipment is presently available to drive sections up to a maximum of 202 kg/m.

Installation of driven piles will generate significant noise and vibrations which will need to be considered. This is discussed further in Section 4.6.2.

4.2.3 Bored reinforced concrete pile foundations

Bored reinforced concrete piles embedded in the underlying ECBF rock are an alternative to shallow foundations. Table 4.3 presents geotechnical parameters for the design of bored pile foundations. These have been assessed based on published correlations with in-situ strength tests and our experience with similar geological units.

[6] AS2159 – 2009. *Australian Standard: Piling - Design and Installation*. SANZ

Table 4.3: Preliminary bored pile design parameters

Geological unit	Anticipated level of unit (RL m)	Geotechnical ultimate capacity (kPa)		Ultimate Limit State (ULS) capacity ϕg = 0.5 (kPa)	
		Skin friction[(2)]	End Bearing[(1,3)]	Skin friction[(2)]	End Bearing[(1,3)]
ECBF rock	-2 to -7	500 (smooth) 700 (grooved)	6,000	250 (smooth) 350 (grooved)	3,000

Notes:

1. Requires an embedment of at least 3 pile diameters into geological unit to mobilise end bearing capacity, otherwise reduce end bearing to ⅔ of recommended values

2. When assessing pile uplift capacity, a further reduction factor of 0.67 should be applied to the presented skin friction values.

3. Based on our experience with ECBF rock in Auckland CBD, including pile load testing undertaken as part of the Britomart station, a geotechnical ultimate bearing capacity of up to 8 MPa may be able to be achieved. Additional site-specific investigations and strength testing (i.e. UCS testing) will be necessary to confirm the competency of the ECBF rock at basement formation level.

Where piles are installed from current site levels, temporary support (i.e., casing / fluid support) will be required to support the Reclamation Fill, Marine sediments and Pleistocene Alluvium. Excavations with ECBF rock are expected to remain open during pile construction.

The basement retention presented in Section 4.4 has not been designed to support building loadings. Where this is proposed to be adopted also as a building foundation, the Structural engineer should review the vertical capacity and embedment accordingly.

Vertical spring stiffness values were estimated based on T+T records of testing a 600 mm diameter pile installed within very weak East Coast Bays Formation rock at a nearby site[7].

An appropriate vertical stiffness value (k_v) for grooved piles in Unweathered ECBF is 175 kPa/mm, up to a maximum displacement of 4 mm (i.e. 700 kPa/4 mm). To calculate a spring stiffness, this value should be multiplied by the corresponding pile diameter and spring spacing (nominated by the structural engineer).

Elastic shortening of the pile should be considered by the structural engineer. Based on preliminary load information, it is anticipated that elastic pile shortening will have a similar magnitude as the anticipated pile settlement/displacement.

4.3 Groundwater drawdown

4.3.1 General

The proposed six level basement excavation will extend below the groundwater and require temporary dewatering.

Dewatering of the site (and beyond) has the potential to cause settlement of the surrounding ground due to the additional pressure applied to the soil skeleton as groundwater is removed. The Reclamation fill and underlying marine deposits are considered highly compressible, and control of groundwater drawdown within these units is a key settlement risk. Groundwater drawdown within the ECBF bedrock underlying these deposits has been observed to result in negligible settlements in nearby projects such as Commercial Bay and City Rail Link (CRL).

A partially drained site is expected to be used for construction with an impermeable perimeter wall installed prior to earthworks to provide groundwater cut-off to the excavation. For concept design,

[7] Concrete Construction Group Pty (May 1998). Britomart Transport Centre Pile Load Test Report Rev 1.

the perimeter cut-off walls are to be founded within embedment into the ECBF rock to minimise potential drawdown of the overlying compressible soils. For the permanent structure, both a drained and tanked basement has been considered at this stage.

For conceptual design, a temporary excavation level has been adopted at **RL -16.3 m**, to allow for over dig, site concrete, waterproofing, temporary sumps / underdrains and potential adjustments of excavation level. Localised excavation to accommodate lift pits, water tanks etc. has also been considered for groundwater drawdown assessment (where appropriate, with a maximum excavation level of RL -21.0 m considered).

4.3.2 Methodology

Numerical analysis was undertaken using GeoStudio the finite element software package[8]. SEEP/w was used to assess the effects of the basement excavation on the surrounding groundwater regime and change in effective stress, which was then modelled in SIGMA/w to estimate ground settlement. A total of 4 No. analysis sections of groundwater drawdown and settlement have been undertaken as summarised below, and presented on Figure 4.1:

- Section 1a (West) – located at the south-west corner of the basement excavation. Modelled to assess the impact on underground services within Lower Hobson Street and Tepid Baths / Watermark Buildings. Diaphragm cut-off wall modelled.
- Section 1b (East) – located at the south-east corner of the basement excavation. Modelled to assess the impact on the neighbouring AON tower. Sheet pile cut-off wall modelled.
- Section 2a (West) – located at the north-west corner of the basement excavation. Modelled to assess the impact of excavation on underground services within Lower Hobson Street, 204 Quay Street and the MSocial Building to the north. Diaphragm cut-off wall modelled.
- Section 2b (East) – located in the north-east corner of the basement excavation. Modelled to assess the impact on the neighbouring HSBC and AON tower. Diaphragm cut-off wall modelled.

[8] GeoStudio 2021.3 SEEP/W & SIGMA/W module; version 11.2.2.23310.

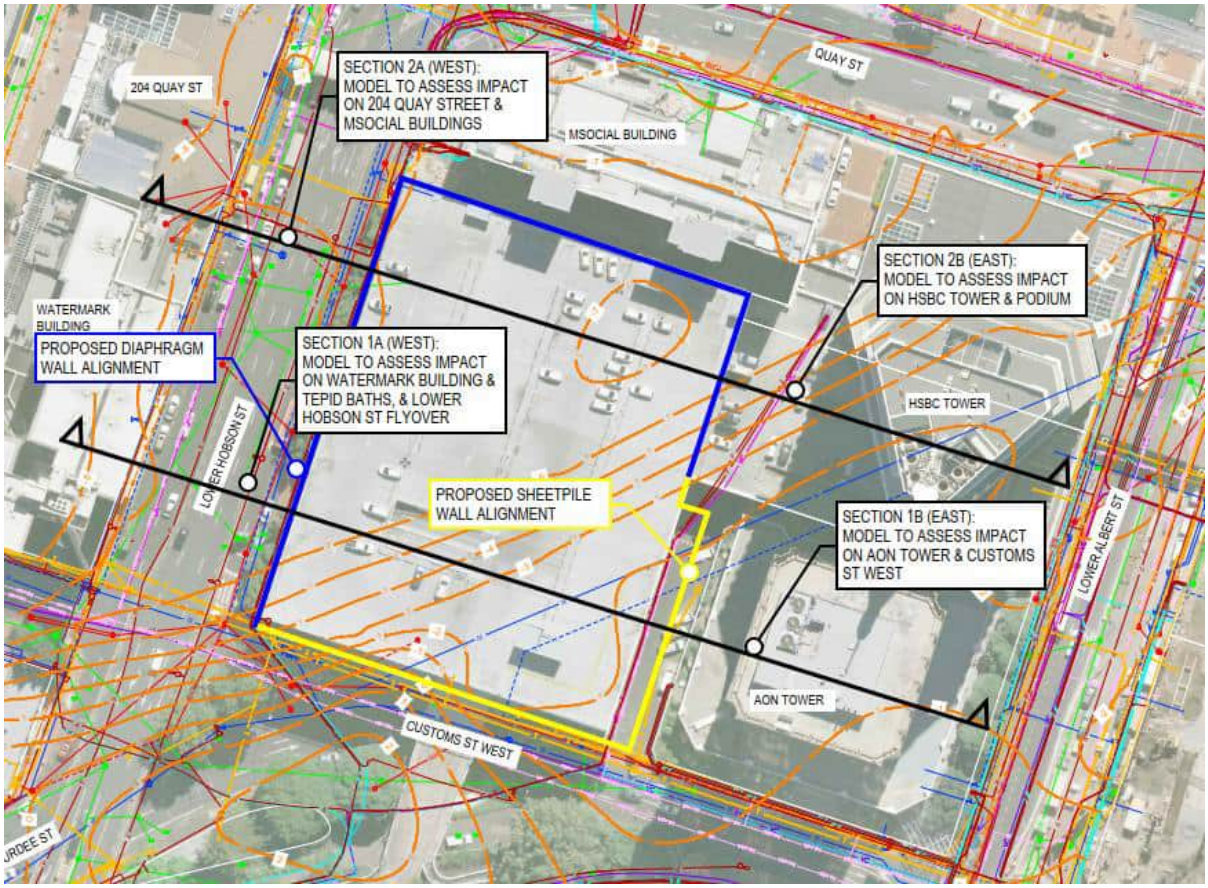


Figure 4.1: Indicative alignments of the groundwater drawdown analysis sections.

The following design cases were considered:

- Transient analysis for a 12-month drained construction period.
- Steady-state analysis for options of both a permanent drained and tanked basement.

4.3.3 Hydrogeological design parameters

Hydrogeological design parameters are presented in Table 4.4 below. These have been adopted based on the parameters derived and adopted from CRL and Commercial Bay developments in proximity to the site.

For concept design, settlements were assessed in SIGMA/W adopting 'isotropic elastic' soil models to model the stiffness of the soil / rock. Elastic modulus values were derived based on available in-situ testing from borehole and CPTs undertaken in proximity to the site, alongside our experience with similar geological units.

Table 4.4: Design parameters for groundwater and settlement assessment

Geological unit	Horizontal Permeability, k_h (m/s)	Vertical Permeability, k_v (m/s)	Equivalent Stiffness (MPa)
Reclamation Fill / Marine Sediments	3×10^{-5}	1×10^{-6}	6
Pleistocene Alluvium	2×10^{-7}	4×10^{-9}	20
Weathered ECBF	2×10^{-7}	4×10^{-9}	40
ECBF Rock	5×10^{-7}	5×10^{-8}	200

4.3.4 Initial groundwater conditions & historical drawdown

The initial groundwater conditions were defined in the SEEP/w model by applying the adopted groundwater pressures with depth at the model boundaries. The resulting modelled pre-construction groundwater levels have then been checked against available groundwater level monitoring records in the area.

When estimating potential settlements due to dewatering, it is important to allow for historical human and natural influences, otherwise settlements will likely be greatly overestimated. The historical changes in effective stresses mean that the soil has been effectively preconsolidated to some extent, and significant consolidation settlements due to groundwater drawdown are only expected to occur once groundwater decreases below previous low groundwater levels. Historical excavations at or near the site would have caused groundwater drawdown and settlement in the area of the site previously. Drawdown within this historical variation range is likely to result in negligible to minor settlements, as most consolidation settlements associated with this groundwater change have already occurred.

Given the high-density of development in the area surrounding the site, including the neighbouring 2-level basement at 188 Quay Street (HSBC Tower) and CRL tunnel excavation less than 100 m east of the site, it is anticipated significant historical drawdown / settlement has occurred.

4.3.5 Retaining wall permeability design parameters

Two wall types have been modelled for concept design:

Diaphragm Wall

An 800 mm diameter wide diaphragm wall has been modelled. The wall has been extended a minimum of 4 m below temporary excavation level to RL -20.3 m. **The design embedment of the wall will be confirmed during detailed design.**

The permeability of the walls has been modelled assuming some leakage during construction by assuming a permeability significantly greater than that of concrete. The following permeability values have been adopted:

- Construction phase: 1×10^{-10} m/s
- Permanent (i.e. where waterproofed above excavation level): 1×10^{-12} m/s (effectively impermeable)

Sheet pile walls

Sheet pile walls have been modelled in the south-east and south basement perimeter where ECBF rock is inferred to be encountered at shallower depths. The design has allowed for a minimum embedment into the ECBF rock of 1 m, which is anticipated to be achievable for a sheet pile for the

section size modelled (Arcelor AU25). A 1 m wide rock shelf and tie-back near the toe of the sheet pile has been assumed to protect the toe of the sheet pile.

For this assessment we have assumed a permeability of $k_s = 8 \times 10^{-9}$ m/s over a thickness of 0.25 m for the sheet piles. In practice, the permeability of the sheet piles will depend on the water tightness of the interlock between sheets. Leakage could occur at joints unless adequate sealants are applied to reduce flow.

4.3.6 Analysis results

Analysis results are summarised in Table 4.5 below.

Table 4.5: Summary of SEEP/W & SIGMA/W analyses

Design case		Maximum pressure head reduction (m)		Maximum Drawdown Settlement (mm)
		Surficial Deposits	ECBF Rock	
Section 1a (West)	Construction (12 months)	0.7	11.5	5
	Permanent Drained	0.7	11.4	
	Permanent Tanked	0.0	0.0	
Section 1b (East)	Construction (12 months)	0.4	18.2	2
	Permanent Drained	0.2	16.0	
	Permanent Tanked	0.0	0.0	
Section 2a (West)	Construction (12 months)	0.6	11.0	2
	Permanent Drained	0.6	11.0	
	Permanent Tanked	0.0	0.0	
Section 2b (East)	Construction (12 months)	0.3	10.7	2
	Permanent Drained	0.3	10.7	
	Permanent Tanked	0.0	0.0	

4.3.7 Hydrostatic uplift of basement slab

Where a tanked basement is adopted, the basement slab will also require assessment for hydrostatic uplift pressures should the basement be tanked. These uplifts pressures should consider potential long-term groundwater levels which may be expected over the life of the structure (i.e. sea level rise etc.).

These uplift pressures can be resisted using bored piles (refer to Table 4.3 for preliminary capacities) or through the use of tension anchors (refer Section 4.2.1).

4.3.8 Groundwater inflows

Inflow estimates are provided in Table 4.6 below.

Table 4.6: Construction stage and permanent groundwater inflows

Wall Type	Design inflow (assuming 90 m x 70 m floor area) (m^3/day)
Sheet Pile + ECBF open-cut	30 to 60
Diaphragm wall (full-length)	20 to 40

The upper range for construction allows for typically higher rates of inflow observed shortly after excavation has occurred while the lower rate is more typical summer conditions. Inflow rates exclude surface water and rainfall, which should be assessed separately. Groundwater stored within the soil material will also drain as the material is excavated and an allowance should be made to capture this as the excavation proceeds.

While significant inflows during construction through walls can be sealed (grouted) as the excavation progresses, groundwater inflows into the excavation could be significantly affected by the quality of the diaphragm and sheet pile wall construction. This risk can be mitigated by adopting good construction practices (such as sufficient overlap between the joint between the diaphragm and sheet pile wall) to ensure groundwater cut-off is achieved.

Significant inflows may occur due to presence of persistent faulting / open joints within the basement excavation which provide connectivity to the Waitemata Harbour, as was encountered during the Britomart Station excavation. If encountered, grouting of these features would be required to stop the flow, which may add significant cost / time delay to the project.

Where a fully drained basement is considered, an underdrainage system should be installed discharging via gravity or pumping to a reticulated stormwater system suitable to discharge groundwater flows estimated in Table 4.6 above. This drainage system should be designed by a suitably qualified engineer, and may typically comprise of:

- Minimum 200 mm thick permeable hardfill layer across the basement slab.
- 200 mm deep by 200 mm wide subsoil drains with perforated drain pipes (110 mm dia drainage coil) installed at approximately 6m centres north-south beneath the basement slab, interconnected for redundancy.
- Inspection points should be available at the ends of each subsoil pipe for clearing where required.

A subsoil collector drain should be installed inside the perimeter of the basement retaining wall to collect groundwater flow and re-direct to a discharge point.

4.3.9 Potential for groundwater mounding

Lateral groundwater flow may be impeded by the installation of diaphragm and sheet pile walls for the basement excavation. However, the potential for groundwater mounding is considered to be low given the reasonably high horizontal permeability of the reclamation fill and marine sediments, and as groundwater flow to the harbour are not impeded along Lower Hobson Street, along the eastern boundary of the site, and Lower Albert Street.

The potential for groundwater mounding will be considered further during detailed design, and if required mitigation measures such as installation of highly permeable trenches around the wall perimeter to redivert groundwater flows will be considered.

4.4 Retaining wall design considerations

4.4.1 General

To support the proposed basement excavation, the following retention options have been modelled:

- Diaphragm wall – to support the entire western and northern extents, and the northern section of the eastern perimeter wall.
- Steel sheet pile walls – to support the entire southern extent, and the southern section of the eastern perimeter wall.

Groundwater cut-off will be required to be maintained where the two wall types join. This may be done during construction by casting a steel sheet into the outside edge of the diaphragm wall. Should noise and vibration concerns not permit the use of the steel sheet piles (as discussed in Section 4.6), a diaphragm wall and/or secant pile walls may also be considered in these areas.

Geotechnical analyses of the retaining walls have been undertaken using pseudo non-linear finite element software WALLAP[9]. The design of the retaining walls has been undertaken to achieve an adequate factor of safety during construction and the long-term / permanent case, and to minimise wall deflections and potential ground settlement.

Retaining wall analyses for 3 No. design sections has been completed for the proposed basement excavation. The following summarises these design sections:

- Section 1 – north-west corner of the basement. This section represents the top-down construction methodology adopting an 800 mm thick Diaphragm wall and internal props/ permanent floor slabs across the northern section of the basement perimeter and the northeast and northwest extents as shown in Figure 4.2. The concept design allows for diagonal props from the pile capping beam to B01 basement level, however corner propping may be adopted as an alternative where possible.
- Section 2 – western basement perimeter. This section represents bottom-up construction methodology adopting an 800 mm thick Diaphragm wall and multi-level ground anchors to provide lateral restraint.
- Section 3 – eastern and southern basement perimeter. This section represents bottom-up construction methodology adopting a sheet pile wall and multi-level ground anchors to provide lateral restraint.

The location of the above sections is shown on Figure 4.2 below. The following sections provide details of the analysis undertaken.



Figure 4.2: Retaining wall modelling design section alignment.

[9] Geosolve (2013). *WALLAP. Anchored and Cantilevered Retaining Wall Analysis Program.* Version 6.05.

4.4.2 Geotechnical parameters

The parameters presented in Table 4.7 have been adopted for this assessment, which are based on the available geotechnical investigations in the site area and our experience with similar geological units.

Table 4.7: Retention geotechnical design parameters

Design parameter		Units	Reclamation Fill / Marine Sediments	Pleistocene Sediments	Weathered ECBF	ECBF rock
Unit weight	γ	kN/m^3	16.5	17.5	19	21
Effective friction angle	ϕ	deg	28	30	32	40
Effective cohesion	c'	kPa	3	5	7	100
Poisson's ratio	v'	-	0.3	0.3	0.3	0.25
Equivalent linear modulus of elasticity [(1)]	E'	MPa	6	20	40	400

To calculate the active and passive pressure coefficient, the following wall friction coefficients have been assumed for the wall types modelled:

- Concrete: assumed to be 2/3 of the soil friction angle on the active side, and ½ of the soil friction angle on the passive side of the wall.
- Steel: assumed to be ½ of the soil friction angle on the active side, and 1/3 of the soil friction angle on the passive side of the wall.

4.4.3 Wall structural parameters

Two wall types have been modelled for concept design:

An 800 mm diameter wide diaphragm wall has been modelled. The stiffness of the fully embedded retaining wall has been calculated adopting the following cracked section moduli for each stage in accordance with CIRIA guidance[10]:

- Construction stage – 75% of gross value.
- Long-term / elevated groundwater case – 50% of gross value.
- Seismic case – 25% of gross value.

Arcelor AU25 sheet piles have been modelled for preliminary design of the sheet pile wall.

The wall section properties adopted in the analysis are summarised in Table 4.8 below.

Table 4.8: Wall structural properties

Wall type	Modulus of Elasticity (GPa)	Second moment of inertia, I_{gross} (m^4 / m run)	Wall Flexural Stiffness (kNm^2 / m run)
800 mm thick D-wall	27.4	4.27×10^{-2}	1,170,000
Arcelor AU25 Sheet piles	210	5.62×10^{-4}	118,100

[10] CIRIA (2017). *CIRIA C760. Guidance on embedded retaining wall design.*

4.4.4 Temporary lateral support properties

Adopted temporary prop / anchor support parameters are given in Table 4.9 below.

Table 4.9: Temporary prop properties

Prop Type	Section area (m^2)	Elastic modulus, E (GPa)
B01 and B03 Partial floor support constructed top-down	0.4 x 1.0	27.4
310UC118 steel section	1.5×10^{-2}	210
450 x 9.53 CHS	1.34×10^{-2}	210
5 x 15.2 mm VSL strand anchors	7.17×10^{-4}	210
7 x 15.2 mm VSL strand anchors	1.003×10^{-3}	210
12 x 15.2 mm VSL strand anchors	1.720×10^{-3}	210

It is assumed that internal props or anchors installed at or near the ground level will be supported at the perimeter capping beam. Where lateral supports are installed at lower elevations, walers may be required to be incorporated as part of the design to distribute the loading between piles.

In the top-down case, partially constructed floors are modelled conservatively as continuous 0.4 m thick slabs of reinforced concrete. It is anticipated the floors may be construction in a 'U-shape' around the perimeter of the excavation to maintain and open pit near the centre of the excavation. However, to transfer shear load from the west and east walls, temporary propping may be required spanning across the southern extent of the floor slabs.

In WALLAP, the temporary floors are applied at the mid-slab elevation, modelled with a span equal to total basement width for each section.

4.4.5 Permanent lateral supports

The permanent floors are modelled as continuous slabs of reinforced concrete. The GF, B02, and B04 levels, which were not were not utilised as top-down supports, were modelled as 0.2 m thick slabs. The B05 floor has been modelled as a 0.8 m thick foundation. In WALLAP, the permanent floors are applied at the mid-slab elevation and the prop free length was adopted as a span equal to half of the basement width for each section (or maximum 30 m).

The following summarises the floor levels assumed for concept design:

- B05 Basement Level: FFL -14.3 m RL.
- B04 Basement Level: FFL -9.7 m RL.
- B03 Basement Level: FFL -5.1 m RL.
- B02 Basement Level: FFL -2.1 m RL.
- B01 Basement Level: FFL +0.9 m RL.
- GF Level: FFL +4.5 m RL.

4.4.6 Construction methodology

The following construction methodologies have been assumed in the Concept Design. The bulk excavation level provides an allowance for up to 1.0 m excavation below the B05 basement formation level for the base slab and sub-slab build up, drainage and contingency for disturbance / over-dig.

At the northern end of the site construction will be top-down with installation of the perimeter wall and internal plunge piles. Temporary diagonal props to the internal plunge piles will be installed prior to excavation to B01 level and construction of the floor slab. Similarly, temporary diagonal prop below the B04 basement level extending to the base of the excavation will be installed prior to construction of the basement floor slab and B04 basement floor. Corner props, as an alternative to diagonal props, may be utilised. Access for excavation will be from the south.

At the western end of the site, the basement excavation will be retained by a Diaphragm wall with four-rows of ground anchors founded within the ECBF rock and/or internal props.

The southern and south-eastern perimeter, a sheet pile wall is proposed with three-rows of ground anchors founded within the ECBF rock. Once excavation proceeds to be ECBF rock, a 1 m wide shelf would be formed below the wall and an open-cut excavation would be undertaken vertically within the ECBF. Temporary support with rock bolts with mesh facing and/or shotcrete may be required to stabilise the rock cut as excavation proceeds. Horizontal drains may also be required in the rock cut to temporarily relieve groundwater pressures near the cut face. Alternatively, a diaphragm wall with three-rows of ground anchors and/or internal props may be considered.

The concept design basement retention layout is presented in Appendix G, and design sketches for the respective WALLAP sections are also provided.

The following summarises the construction methodology assumed for the WALLAP analysis:

<u>Section 1 – Top-down Diaphragm wall</u>

1 Install 800 mm thick Diaphragm wall with a toe level at RL -20.3 m.
2 Excavate no deeper than RL 0.0 m and form 1(v):2(h) temporary berm to B01 FFL (RL +0.90 m).
3 Install 310UC118 temporary prop founding upon internal plunge piles at 5 m c/c spacing (or alternatively corner propping).
4 Excavate berm and continue excavation no deeper than RL 0.0 m and install 400 mm thick B01 level floor slab.
5 Excavate no deeper than RL -6.0 m and install 400 mm thick B03 level floor slab.
6 Excavate to no deeper than RL -10.7 m and form 1(v):1(h) temporary berm to basement excavation level (RL -16.3 m).
7 Install 450x9.53 temporary prop founding on the basement excavation floor (or leave berm in place until B04 slab construction).
8 Excavate berm and continue excavation to no deeper than RL -16.3 m and install B05 level slab.
9 Install B04 and B02 level slab.
10 Remove the lower and upper-level temporary prop and install GF slab.
11 Apply elevated groundwater design scenario.
12 Apply seismic design scenario.

<u>Section 2 – Anchored Diaphragm wall</u>

1 Install 800 mm thick Diaphragm wall with a toe level at RL -20.3 m.
2 Excavate no deeper than RL 2.0 m and install 7 x 15.2 mm VSL strand ground anchors at 2.5 m c/c spacing, at RL 2.5 m. Pre-stress anchor to 150 kN.
3 Continue excavation to no deeper than RL -1.8 m and install 7 x 15.2 mm VSL strand ground anchors at 2.5 m c/c spacing, at RL -1.3 m. Pre-stress anchor to 200 kN.

4 Excavate no deeper than RL -6.5 m and install 12 x 15.2 mm VSL strand ground anchors at 2.0 m c/c spacing, at RL -6.0 m. Pre-stress anchor to 400 kN.

5 Excavate no deeper than RL -11.0 m and install 7 x 15.2 mm VSL strand ground anchors at 2.5 m c/c spacing, at RL -10.5 m. Pre-stress anchor to 200 kN.

6 Excavate to no deeper than RL -16.3 m and install B05 slab.

7 Install B4 and B3 slabs, and remove low-level ground anchors at RL -10.5 m and RL -6.0 m.

8 Install B2 slab and remove ground anchor at RL -1.3 m.

9 Install B1 and GF slab and remove ground anchor at RL 2.5 m.

10 Apply elevated groundwater design scenario.

11 Apply seismic design scenario.

Section 3 – Anchored Sheet pile wall

1 Install Arcelor AU25 sheet piles with a minimum toe embedment of 1 m into ECBF rock.

2 Excavate no deeper than RL 2.0 m and install 5 x 15.2 mm VSL strand ground anchor at RL 2.5 m. Pre-stress anchor to 150 kN.

3 Continue excavation to no deeper than RL -1.0 m and 5 x 15.2 mm VSL strand ground anchor at RL -0.5 m. Pre-stress anchor to 200 kN.

4 Continue excavation to no deeper than RL -4.5 m and 5 x 15.2 mm VSL strand ground anchor at RL -4.0 m. These are to act as passive anchors.

5 Install 1 m wide level bench at RL -4.5 m and excavate vertical rock-cut (with temporary support with rock bolts and mesh / shotcrete as required) to no deeper than RL -16.3 m.

6 Install permanent basement wall and internal floor slabs. Backfill between sheet piles and permanent wall.

The section analysed above represents the 'critical' section adopting the highest depth to rock based on the current ground profile. The elevation of the rock shelf and anchor supports will be adjusted as the depth to rock and ground level behind the excavation changes (generally rising to the south / east). This will be considered during detailed design.

4.4.7 Surcharge loading

The analyses have been undertaken adopting a design construction traffic surcharge of 12 kPa. The neighbouring structures in close proximity to the development are all anticipated to be on piled foundations, and therefore will impose negligible surcharges at the ground surface.

4.4.8 Seismic loading

For the design of piled retaining walls permanently propped by the internal basement structure recommendations for 'stiff' walls (i.e. deflection<0.4%H) have been adopted as recommended by NZGS Module 6: Earthquake Resistant Retaining Wall Design. This seismic load component is calculated by the following formula and applied midway up the retained height (i.e. modelled as uniform distribution):

$\Delta P_E = 0.6.K_h.\gamma.H^2$

Where: K_h is the earthquake acceleration design coefficient determined from MBIE module 6 eq.5-1
γ = bulk unit weight of retained soil (adopt typical 17.5 kN/m^3)
H = Permanent retained height

4.4.9 Analysis Results

Analysis results are summarised in Table 4.10 below.

Table 4.10: WALLAP Analysis Results

	Section			**Design Section 1**	**Design Section 2**	**Design Section 3**
	Location			Northern section	Western perimeter	South and south-east perimeter
Wall	Retained ground level		mRL	4.0	4.0	4.0
	Design pile toe		mRL	-20.3	-20.3	-5.5
	Wall Type			800 mm D-Wall	800 mm D-Wall	AU25 Sheet pile
Description of props	Prop / anchor 1	Prop Type	-	310UC118	7 x 15.2 mm VSL ground anchor	5 x 15.2 mm VSL ground anchor
		Level	mRL	4.3	2.5	2.5
		Spacing	m	5.0	2.5	3.0
		Inclination	deg.	27	30	45
		Pre-stress	kN	-	150	200
	Prop / anchor 2	Prop Type	-	0.4 m thick floor slab (B01)	7 x 15.2 mm VSL ground anchor	5 x 15.2 mm VSL ground anchor
		Level	mRL	0.9	-1.8	-0.5
		Spacing	m	-	2.0	3.0
		Inclination	deg.	0	30	35
		Pre-stress	kN	-	200	200
	Prop / anchor 3	Prop Type	-	0.4 m thick floor slab (B03)	12 x 15.2 mm VSL ground anchor	5 x 15.2 mm VSL ground anchor
		Level	mRL	-5.1	-6.0	-4.0
		Spacing	m	-	2.5	3.0
		Inclination	deg.	0	20	20
		Pre-stress	kN	-	400	0
	Prop / anchor 4	Prop Type	-	450 x 9.53	7 x 15.2 mm VSL ground anchor	-
		Level	mRL	-10.3	-10.5	
		Spacing	m	5.0	2.5	
		Inclination	deg.	45	20	
		Pre-stress	kN	-	200	
Design actions – temporary	Deflection (ground level)		mm	8	20	17
	Deflection (maximum)		mm	22	21	17
	Max. Deflection Level		mRL	-10.9	-0.35	+4.0
	Bending moment (max)		kNm/m	690	525	140
	Shear force (maximum)		kN/m	690	515	155
	Max. force at support **(horizontal loads[(1)])**	Prop / anchor 1	kN/m	260	270	210
		Prop / anchor 2	kN/m	1090	180	250
		Prop / anchor 3	kN/m	450	1790	325

	Section			**Design Section 1**	**Design Section 2**	**Design Section 3**
	Location			Northern section	Western perimeter	South and south-east perimeter
		Prop / anchor 4	kN/m	1770	1070	-
Actions – **static/seismic**	Bending moment (max)		kNm/m	630	320	
	Shear force (maximum)		kN/m	680	530	-
	Static force applied to prop	GF	kN/m	45	60	-
		B01	kN/m	285	170	-
		B02	kN/m	65	200	-
		B03	kN/m	1075	685	-
		B04	kN/m	675	725	-
		B05	kN/m	915	930	-

(1) Prop / anchor forces are presented as horizontal forces at the wall face only. Where supports are installed at an inclination, the actual force within the support will need to be calculated.

4.4.10 Further Retaining Wall design considerations

The retaining wall design presented meets the minimum requirements to achieve acceptable deflections for both groundwater and mechanical settlements, for the purposes of a Resource Consent application.

During preliminary design, detailed analysis using finite element software will be undertaken to consider the following:

1 The wall design currently allows for a minimum 4 m toe embedment for the diaphragm wall sections, and minimum 1 m toe embedment for the sheet pile wall sections. Detailed design will test sensitivity for a fault zone / joint at the base of the excavation. This may require a contingency for an extension of the pile embedment, or installation of low-level anchors / props to stabilise the base of the wall. Further analysis should be undertaken to confirm suitable design options as part of detailed design.

2 There is potential for water pressures within the ECBF rock to cause the diaphragm wall panels to 'lift' off the rock face (i.e. wall deflection in excess of relaxation in the rock). This could result in a gap to form between the wall and the rock face affecting shear transfer and affecting leakage flows. This may require consideration of pre-stressing of the lower floor levels (B04 / B05) and/or installation of low-level ground anchors and will be considered as part of detailed design.

3 The south-east portion of the site is located at the former land cliff-edge prior to the reclamation of the Downtown area, and at the top of a rock-cut platform. It is anticipated the rock surface will be highly variable, with local erosional features at the top of the rock forming an undulating rock surface and localised 'hard' areas of higher strength rock that were more resistant to erosion. This may result in an increase in retained height or a reduced pile embedment into ECBF rock, respectively, and may require contingency to install additional low-level anchors to control deflections and/or prevent failure at the toe of the piles.

4.5 Assessment of deformations and settlement effects

4.5.1 Mechanisms of settlement effects

The potential for surface deformation of the surrounding ground due to the development has been assessed as detailed in the following sections. The main contributing factors potentially causing ground settlements near the site that could impact on the immediately adjacent buildings, services or infrastructure include:

1 Excavation will extend below groundwater levels and so will result in local groundwater drawdown. Consolidation of the ground due to groundwater drawdown may occur due to the reduction in porewater pressures and increase in effective stress in the soil as groundwater seeps into the excavation and will be dependent on time.
2 As the excavation proceeds and the perimeter walls take load from the retained soil, lateral deflections will result. Mechanical settlement of the ground is associated with the deformation of the retaining walls. The associated ground settlements will occur relatively quickly and are expected to rapidly diminish with distance from the excavation.

4.5.2 Deformation criteria for buildings

Ground deformation can affect adjacent buildings and infrastructure (buried services and road pavements) through changes in grade and elongation/horizontal strain. A risk level of Aesthetic Damage of 'negligible' to 'slight damage' as defined in CIRIA PR30[11] is considered appropriate for neighbouring buildings. The limiting criteria of less than 1(V):500(H) differential settlement and less than 10 mm total settlement (i.e., negligible effects) have been adopted for this preliminary assessment.

Table 4.11: Typical values of maximum settlements for building damage risk assessment

Risk cat.	Maximum Differential Settlement	Maximum Settlement of Building (mm)	Description of Risk
1	Less than 1 in 500	Less than 10	Negligible: superficial damage unlikely
2	1 in 500 to 1 in 200	10 to 50	Slight: possible superficial damage which is unlikely to have structural significance
3	1 in 200 to 1 in 50	50 to 75	Moderate: expected superficial damage and possible structural damage to building, possible damage to relatively rigid pipelines.
4	Greater than 1 in 50	Greater than 75	High: expected structural damage to buildings and rigid pipelines or possible damage to other pipelines.

Where 'slight damage' risk (category 2) may occur, further assessment, as per Burland (2012)[12] will be required to considered to assess likely effects on potentially affected structures.

[11] CIRIA (1996). *Project Report 30: Prediction and effects of ground movements caused by tunnelling in soft ground beneath urban areas*. March 1996.
[12] Burland, J.B. (2012). *Chapter 26 Building Response to Ground Movements* in ICE Manual of Geotechnical Engineering. Institution of Civil Engineers.

4.5.3 Deformation criteria for underground services

While many types of utilities can accommodate high levels of differential settlement, certain types can be susceptible to damage. In general, a utility's tolerance to settlement depends upon the construction type/material, existing condition, and whether the utility runs parallel or perpendicular to the excavation. Utilities running perpendicular to the excavation works are considered to be at the highest risk of damage. Utilities which run parallel and are near the excavation works may experience horizontal displacement associated with ground loss at the excavation face; however, they will experience a much gentler differential settlement.

The methodology to assess the effects on utilities is based on the method on O'Rourke and Trautman (1982)[13] – which provides guidance on allowable differential settlement for various utility construction types.

Table 4.12: Allowable differential settlement based on utility type

Utility type	Maximum allowable differential settlement (V:H)
Brick unlined	1:245
Welded steel pipe	1:122
Cast in-situ concrete	1:173
PVC & HDPE	1:67
Reinforced concrete pipe	1:229
Ductile iron pipe	1:229
Vitrified clay pipe	1:299
Cast iron pipe	1:150 – 1:500 (varies based on diameter)

Estimations of potential damage to a utility have been based off the calculated ground surface settlement profiles perpendicular to the excavation. This is likely to over-estimate the actual differential settlement experienced by the utility as:

- Where a utility crosses oblique to the alignment, the estimated differential settlement is expected to be lower than if the utility was crossing perpendicular to the alignment.
- It has been assumed that the differential settlement affecting each recorded utility is equal to the differential settlement at the ground surface. In reality the differential settlements at depth are likely to be less than at the ground surface, and consequently the settlement estimates are conservative.

4.5.4 Cumulative settlement and effects on neighbouring buildings

Table 4.13 below presents a summary of the estimated total and differential settlements resulting from the combined effects of wall deformations and groundwater induced settlements on the neighbouring structures surrounding the development. Settlement profiles with distance perpendicular to the excavation are presented in Appendix H.

This assessment indicates the potential effects due to the proposed basement excavation are generally within Risk Category 1 – Negligible in accordance with CIRIA PR30, and negligible damage is expected to occur as a result.

[13] O'Rourke, T D, and C H Trautmann. 1982. Buried pipeline response to tunnel ground movements. In Europipe 82 Conf., Basel, Switzerland, paper 1.

Table 4.13: Estimated settlements below neighbouring structures

Building	Basement + Foundation Types	Date of Construction	Estimated ground settlement below structure (mm)		Assessed Risk of Building Damage due to Ground Settlement[1]
			Total	Differential	
MSocial Building	Main Tower – piled foundations to rock. Front of house – shallow foundations.	1970s (upgraded in 2017, including external re-cladding).	<12	<1(v):1,000(h)	Very low – modern building on piled foundations socketed into rock. Front of house building is over 15 m from the excavation and groundwater is controlled/ recharged by the harbour.
HSBC Tower	Main Tower – 2-Level basement + piled foundations to rock. Podium – piled foundations.	2002	<12 <8	<1(v):1,000(h) <1(v):3,000(h)	Very low – modern building on piled foundations socketed into rock. Groundwater is controlled/ recharged by the harbour.
AON Tower	Double level basement + pad foundations bearing on rock.	1970s	<10	<1(v):600(h)	Very low – modern building with two level basement, bearing on rock. Basement floor slab higher tolerance to deformations. Groundwater is controlled/ recharged by the harbour.
204 Quay Street	Unknown.	1940s	<5	<1(v):5,000(h)	Very low – 30 m+ distant from excavation and very low total and differential ground settlement assessed. Groundwater is controlled/ recharged by the harbour.

Building	Basement + Foundation Types	Date of Construction	Estimated ground settlement below structure (mm)		Assessed Risk of Building Damage due to Ground Settlement[1]
			Total	Differential	
Tepid Baths	Composite shallow + driven piled foundations.	1914 (upgraded in 2010, including upgrades to piled foundations).	<7	<1(v):4,000(h)	Very low – 40 m+ distant from excavation and Low total and differential ground settlement assessed. Main structure is supported on piled foundations socketed into rock. Limited existing historic fabric generally supported by the new structure.
Watermark Building (85 Customs St West)	Partial level basement + piled foundations.	2000	<7	<1(v):4,000(h)	Very low – modern building, 30 m+ distant, piled foundations socketed into rock. Low total and differential ground settlement assessed. ground water control/ recharge by harbour.
Hobson St Flyover	Piled foundations.	Late 1980s / early 1990s.	<12	<1(v):1,000(h)	Very low – piled foundations and low differential movements assessed.

Note:

1. onsidering maximum total and different ground settlement, foundation type and age of structure. Assessed risk of building damage is a joint assessment by Geotechnical and Structural Engineer.

4.5.5 Settlement effects on underground services and pavements

Table 4.14 below presents a summary of the estimated total and differential settlements beneath services surround the development. Settlement profiles with distance perpendicular to the excavation are presented in Appendix H.

Table 4.14: Estimated settlement below services within pavement

Street	Services within pavement	Service orientation to excavation	Estimated ground settlement below structure (mm)	
			Total	Differential*
Customs St West	Wastewater – 150VC / CONC & 225AC	Parallel	9	-
	Stormwater – 225 Ceramic	Parallel	4	-
	Water – 250/600 CLS & 175CI	Parallel	1	-
	Telecommunications – Chorus + Vodafone	Parallel + Perpendicular	8 – 10	-
	Vector – Gas	Parallel	9	
Lower Hobson St	Wastewater – 225VC / AC	Parallel + Perpendicular	13 – 17	<1(v):1,000(h)
	Stormwater – 300 to 1050 CONC	Parallel + Perpendicular	13 – 17	<1(v):1,000(h)
	Water – 200CLS	Parallel	13 – 17	-
	Power & Telecommunications (Chorus + Vodafone)	Parallel + Perpendicular	13 – 17	-
Private Accessway between HSBC Tower / AON Tower	Power & Telecommunications (Vector + Vodafone)	Parallel	10 – 12	-

* Differential settlements estimated for services which are orientated perpendicular to the excavation only. Where the services run parallel to the excavation face, negligible differential settlements are expected to occur.

4.6 Other construction considerations

4.6.1 Earthworks and excavating through rock

The underlying ground conditions at the site are likely to be excavatable using conventional equipment and plant, such as a 20-tonne digger with a toothed bucket. The ECBF rock generally has an unconfined compressive strength of 1 to 5 MPa, with possible seams of cemented rock of up to 20 MPa strength.

Sections of the basement excavation are proposed to be supported by sheet piles as shown in Figure 4.2 above. An open excavation through rock is proposed in these areas. The use of vertical open cuts with spot bolting as required has been adopted for a number of basement projects in the Auckland CBD. Allowances should be made for mapping of the vertical rock face, installation of rock bolts and grouting as the excavation proceeds, to support the rock and reduce groundwater seepage respectively.

4.6.2 Earthworks volumes

Approximate volumes for the bulk excavation are provided below for the purpose of construction planning. These approximate volumes are based on the current understanding of basement footprint (Resource consent draft drawing dated 07-06-2024), current basement level and existing geotechnical investigations at the site. We recommend an appropriate contingency is applied to these volumes including bulking factors for storage of material or transport away from the site.

- Total estimated bulk excavation – 130,000 m^3.
- Estimated volume for plunge columns and DWalls - 4500 m^3.

Split based on geological unit:

- Fill/ Reclamation Fill/ Marine Sediments – 54,000 m^3.
- East Coast Bays Formation soil – 3,500 m^3.
- East Coast Bays Formation rock – 72,500 m^3.

4.6.3 Noise & vibrations

Anchored sheet piles have been considered as a potential retention solution along the southern and south-eastern perimeter of the site. The sheet piles are proposed to be driven through the reclamation fill and marine sediments with an embedment of at least 1 m into the ECBF rock. The noise and vibration that will be generated during the driving of the sheet piles, particularly into the ECBF rock, will need to be considered as part of the construction noise and vibration monitoring plan.

Pertinent structures that are likely to be impacted by noise and vibration due to sheet piling include:

- The AON building which is located within 10 m of the proposed sheet piles and is supported on pile foundations that are embedded into ECBF rock.
- The HSBC building, Tepid Baths and the Watermark buildings (85 Custom Street West), which are located between 30 and 50 m from the proposed sheet piles. All of the buildings are expected to be supported on pile foundation embedded into ECBF rock.

M-Social and other neighbouring structures are located greater than 50 m from the proposed sheet piling and the impact on these structures are expected to be minimal.

4.6.4 In-ground obstructions

Due to the extensive historical land-use across the site, consideration of in-ground obstructions will be required for design and construction, including; foundation layout, basement excavation, and pile / perimeter wall construction. The following historical land-uses should be considered:

Reclamation Fill & Graving Dock

The Reclamation Fill is likely to be highly variable and is inferred to consist of fine-grained soils through to cobbles, boulders and construction waste material. The graving dock which extended across majority of the site was infilled in 1923. The material used to infill this is not known and remnants of the graving dock and associated structure may still be present at the site. Therefore, potential construction issues may arise due to obstruction from large boulders within the Reclamation Fill, old graving dock structure, relict wharfs and old seawalls. In particular this may impact the driveability of sheet piles and/or excavation of bored piles. Further investigations and assessment are proposed in subsequent design phases to assess this.

Existing Downtown Car Park foundations

The existing belled reinforced concrete pile foundations will also need to be removed as the excavation proceeds. The existing piles may need to be cut down where they conflict with construction of the foundations for the proposed development and to reduce the potential for hard points beneath the B5 floor level. Appendix C provides the historical pile layout for the existing carpark building.

The presence of these existing foundations should be considered as part of the foundation / superstructure design and set-out to avoid pile clashes where possible, particularly if piled foundations are adopted for the development with installation undertaken at existing ground levels.

Historical (pre-carpark) foundations

Remnants of the old seawalls and graving dock walls may still be present at the site and can be encountered during the excavation of the basement for the Downtown West development.

In addition, prior to construction of the existing carpark building, commercial/ workshop buildings were noted to occupy the northwestern and southeastern portions of the site. It is not known if the foundations of these structures were fully removed during the construction of the existing Downtown Carpark structure and may still be present and encountered during the excavation of the basement for the Downtown West development.

4.6.5 Potential contamination

There may be contaminants within the basement excavation spoil that limits disposal to a clean landfill site. A preliminary site investigation (PSI) study and a preliminary contamination site management plan (CSMP) have been undertaken for the site and are provided separately.

5 Auckland Unitary Plan – Operative in Part

We have reviewed the Auckland Unitary Plan (AUP) rules regarding the take, use, damming and diversion of groundwater that are relevant to the proposed basement excavation. The AUP provides for the take, using damming and diversion of groundwater and drilling in association with excavation. The AUP includes several requirements that must be met for developments to be considered as a permitted activities with regard to groundwater, which are listed in Activity Table E7.4.1 and Permitted Activity Standards E7.6.1.10 and E7.6.1.6.

The assessment of compliance with these rules is summarised below and identifies a number of non-compliances. On the basis of our assessment the project is likely to be considered as a restricted discretionary activity in accordance with AUP, Table E7.4.1 (A20) and (A28).

Table 5.1: AUP E7 engineering assessment

E7 Taking, using, damming and diversion of water and drilling - E7.6.1 PERMITTED ACTIVITY STANDARDS		
E7.6.1.6 Dewatering or groundwater level control associated with a groundwater diversion permitted under Standard E7.6.1.10, all of the following must be met.		
(A27) Permitted Activity Standards: Exemptions	**Geotechnical Interpretation of Compliance**	**Permitted activity compliance**
(1) The water take must not be geothermal water;	Geothermal water not expected at the site.	Yes
(2) The water take must not be for a period of more than 10 days where it occurs in peat soils, or 30 days in other types of soil or rock; and	Groundwater diversion/ take will be for a period greater than 30 days.	No
(3) The water take must only occur during construction.	Groundwater take may extend beyond construction if drained basement is adopted	No
E7 Taking, using, damming and diversion of water and drilling - E7.6.1 PERMITTED ACTIVITY STANDARDS		
E7.6.1.10. Diversion of Groundwater Caused by any Excavation (including trench), or tunnel		
(A27) Permitted Activity Standards: Exemptions	**Geotechnical Interpretation of Compliance**	**Permitted activity compliance**
(1) All of the following activities are exempt from the Standards E7.6.1.10(2) – (6):		
(a) pipes cables or tunnels including associated structures which are drilled or thrust and are less than 1.2 m in external diameter;	There are not expected to be any pipes cables or tunnels ≥1.2 m.	Yes
(b) pipes including associated structures up to 1.5m in external diameter where a closed face or earth pressure balanced machine is used;	N/A due to compliance with 1(a) above.	Yes
(c) piles up to 1.5 m in external diameter are exempt from these standards;	All piles are expected to be <1.5 m diameter.	Yes
(d) diversions for no longer than 10 days; or	Groundwater diversion due to construction of the basement will be longer than 10 days.	No
(e) diversions for network utilities and road network linear trenching activities that are progressively opened, closed and stabilised	The groundwater diversion is for purposes other than network utilities and road network linear trenching activities.	No

E7 Taking, using, damming and diversion of water and drilling - E7.6.1 PERMITTED ACTIVITY STANDARDS		
E7.6.1.6 Dewatering or groundwater level control associated with a groundwater diversion permitted under Standard E7.6.1.10, all of the following must be met.		
(A27) Permitted Activity Standards: Exemptions	**Geotechnical Interpretation of Compliance**	**Permitted activity compliance**
where the part of the trench that is open at any given time is no longer than 10 days.		
(2) Any excavation that extends below natural groundwater level, must not exceed:		
(a) 1ha in total area: and	(a) The development area does not exceed 1ha in total area	Yes
(b) 6m depth below the natural ground level	(b) The maximum excavation depths will be greater than 6 m below existing ground levels	No
(3) The natural groundwater level must not be reduced by more than 2m on the boundary of any adjoining site.	(3) The natural groundwater level in the surficial soils is not expected to be reduced by more than 2 m at the site boundary.	Yes
(4) Any structure, excluding sheet piling that remains in place for no more than 30 days, that physically impedes the flow of groundwater through the site must not:		
(a) impede the flow of groundwater over a length of more than 20 m; and	(a) The development will impede the flow of groundwater over a length of more than 20 m (site measures approximately 88 m by 70 m)	No
(b) extend more than 2 m below the natural groundwater level.	(b) The structure will extend more than 2 m below natural groundwater levels	No
(5) The distance to any existing building or structure (excluding timber fences and small structures on the boundary) on an adjoining site from the edge of any:		
(a) trench or open excavation that extends below natural groundwater level must be at least equal to the depth of the excavation;	(a) Several neighbouring structures will be inside the specified envelope.	No
(b) tunnel or pipe with an external diameter of 0.2 - 1.5 m that extends below natural groundwater level must be offset 2 m or greater; or	(b) Any site connections are likely to be greater than 2 m from neighbouring structures.	Yes
(c) a tunnel or pipe with an external diameter of up to 0.2 m that extends below natural groundwater level has no separation requirement.	(c) No comment required	Yes
(6) The distance from the edge of any excavation that extends below natural groundwater level, must not be less than:		
(a) 50 m from the Wetland Management Areas Overlay;	The proposed excavation that extends below natural groundwater level is located more than 50m from Wetland Management Areas Overlay	Yes
(b) 10 m from a scheduled Historic Heritage Overlay; or	The proposed excavation that extends below natural groundwater level is	Yes

E7 Taking, using, damming and diversion of water and drilling - E7.6.1 PERMITTED ACTIVITY STANDARDS		
E7.6.1.6 Dewatering or groundwater level control associated with a groundwater diversion permitted under Standard E7.6.1.10, all of the following must be met.		
(A27) Permitted Activity Standards: Exemptions	**Geotechnical Interpretation of Compliance**	**Permitted activity compliance**
	approximately 30m from the closest scheduled Historic Heritage overlay	
(c) 10 m from a lawful groundwater take	(c) A specific search of groundwater takes adjacent to the site has not been undertaken. However, we expect a groundwater take consent may have been required for the current HSBC building present immediately to the north-west (188 Quay Street) and West Plaza development to the south-west (3-15 Albert Street). Groundwater take consent also existing for the Commerical Bay precinct development further to the east.	No

6 Recommendations for site investigations and monitoring work

6.1 Recommendations for site investigations

Further geotechnical site investigations are proposed during the preliminary design stage. The site investigations will be used to confirm the geotechnical and groundwater model that have been adopted for this concept design. Additional site investigations to inform detailed design will consist of:

- Four machine drilled boreholes to a depth of 20 m below ground level. Unconfined compressive strength (UCS) tests and other suitable laboratory tests will be undertaken to confirm design parameters.

  The location of the boreholes will be finalised based on access to machinery and following further detailed review of underground services. However, generally the boreholes will be located as follows:

  - Two boreholes near the north-east corner and perimeter of the site. There is currently limited information in this area and rock levels may be impacted by the graving dock. These boreholes will also be used to assess impact on the groundwater regime due to the backfilling of the graving dock.
  - One borehole near the south-west corner to confirm depth to rock and for groundwater monitoring at the southern perimeter of the proposed excavation.
  - If feasible, one borehole within the building footprint and in the south-west portion of the building. This will be used to refine the depth to rock in an area where there is currently limited information.
- The four machine boreholes will be completed with a nested standpipe piezometer to monitor groundwater levels in the upper reclamation fills and marine sediments, and groundwater levels in the ECBF rock. These groundwater monitoring wells will be part of the long-term groundwater monitoring system during construction.
- Up to four cone penetration tests (CPTs) are proposed to refusal (typically 5 m to 11 m). The CPTs will be used mainly to confirm the design parameters and liquefaction assessment presented in this concept design report.
- These investigations should be coordinated with contamination sampling and testing.

6.2 Construction survey and monitoring

A draft GSMCP is provided in Appendix B. The GSMCP provides draft proposed monitoring of the basement excavation works and surrounding areas to assess if the ground and groundwater conditions are consistent with the design analyses and the response of structures are within design tolerances.

7 Risk register

Presents identified geotechnical risks and opportunities for the proposed development of the Downtown West. The presented risks are those identified at this time and may not be a complete list. Risks (including Safety in Design considerations) should be regularly reviewed by the project team as the design and construction progresses.

Table 7.1: Geotechnical risk register

ID	Risk	Further assessment proposed	Residual risk	Likelihood of residual risk
1	The depth to unweathered ECBF rock varies from that assumed for basement retention design and pile embedment	Further site-specific investigations proposed in preliminary design. Detailed design will consider contingencies to extend diaphragm wall, sheet pile and foundation piles should rock be deeper than expected. Consider proof drilling at pile locations during construction, particularly where sheet pile retention is proposed (i.e. south and south-east perimeter). Sheet pile retention design to consider contingency options where sheet piles extend deeper than ground model allows. This may include additional low-level ground anchor supports.	Depth to rock varies over short distances (i.e. erosional channels in rock)	Low to medium
2	Obstructions during excavation works from the existing carpark pile foundations, historical foundations (pre-carpark), remnants of old seawalls and boulders and waste material that may be present in the reclamation fill	Detailed design of the proposed foundations for the development to consider current carpark foundation layout, particularly for any piles that are to be formed from ground surface. Existing reinforced concrete piles to be cut down below the B4 floor levels Design to consider contingency if obstructions impact the driveability of sheet piles or interference during diaphragm wall / secant pile installation.	Unknown location, magnitude and size of obstructions	Medium to high
3	Collapse of pile shaft and anchor bores within the reclamation fill and marine sediments	Temporary support of pile shafts required through reclamation fill and marine sediments. Options include using temporary casing or drilling with a slurry (bentonite or polymer)	Collapsing of pile or anchor holes	Medium
4	Storage of excavated soil on site, such as stockpiling of soil above the proposed retaining walls may cause excessing wall deformation	Maintain a designated area behind the back of the retaining walls where storage of soil or plant and machinery is not permitted	Due to limited space available at the site, designated areas to stockpile fill may be difficult	Low
5	Clash between proposed temporary	Pile layouts for neighbouring buildings are known. Design to consider anchor	Unexpected differences	Low

ID	Risk	Further assessment proposed	Residual risk	Likelihood of residual risk
	ground anchors and the pile foundation of neighbouring building	inclination and length so that they generally do not extend into the neighbouring building footprint. If this is not feasible design shall consider the layout of ground anchors to avoid clashes with the existing pile foundations beneath neighbouring structures	between actual foundation locations and available as-built plans	
6	High groundwater inflows during construction due to open joints / fault zone through ECBF rock providing hydraulic connection to harbour	Frequent geotechnical observations during basement excavation to identify risk areas. Contingency option to grout joint / fault zones to provide seal and reduce groundwater inflows.	Construction difficulties installing grout seal. Grout volumes required to provide seal.	Low
7	Strong Calcite cemented bed making drilling, excavation and driven pile installation difficult	Further geotechnical investigations proposed to assess competency of ECBF rock within basement excavation. Detailed design of sheet pile retention to consider contingency options should minimum 1 m embedment into ECBF rock not be achieved. This may include additional low-level anchors and/or pre-augering of the sheet-piles.	Isolated beds of strong cemented material not identified during investigation	Low
8	Water pressures in ECBF rock causing the diaphragm wall to 'lift' off the rock face. Impacts on shear transfer between wall and rock interface and affecting leakage flows	Detailed design to consider impacts and contingency options, including pre-stressing of lower floor levels and/or installation of low-level ground anchors.	Gaps between the ECBF rock and diaphragm results in increased likelihood of leakage.	Low

8 Applicability

This report has been prepared for the exclusive use of our client Precinct Properties Holdings Limited, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

We understand and agree that our client will submit this report as part of an application for resource consent and that Auckland Council as the consenting authority will use this report for the purpose of assessing that application.

Recommendations and opinions in this report are based on data from discrete investigation locations. The nature and continuity of subsoil away from these locations are inferred but it must be appreciated that actual conditions could vary from the assumed model.

Tonkin & Taylor Ltd
Environmental and Engineering Consultants

Report prepared by:



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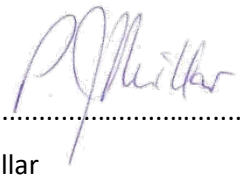
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Project Director

6-Dec-24
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Appendix A Site Plan and Geological Cross-sections

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LEGEND

- PROPERTY BOUNDARY
- ECBF CONTOUR (1m INTERVAL)
- GEOMAPS WATER LINE
- GEOMAPS ABANDONED WATER LINE
- WATER HYDRANT
- WATER VALVE
- GEOMAPS STORMWATER LINE
- GEOMAPS ABANDONED STORMWATER LINE
- STORMWATER CATCHPIT
- STORMWATER MANHOLE
- GEOMAPS WASTEWATER LINE
- GEOMAPS ABANDONED WASTEWATER LINE
- WASTEWATER MANHOLE
- CRL UNDERGROUND ROUTE
- CHORUS
- VECTOR COMMUNICATION - UNDERGROUND ROUTE TRENCH
- VECTOR COMMUNICATION - UNDERGROUND INSTALLED IN PLACE
- GAS
- GAS VALVE
- VODAFONE
- FIBRE OPTIC
- POWER
- LAMP
- EXISTING BOREHOLE LOCATIONS
- EXISTING CONE PENETRATION TEST LOCATIONS

204 QUAY ST
QUAY ST
MSOCIAL
DOWNTOWN CARPARK SITE
WATERMARK BUILDINGS
LOWER HOBSON ST
HSBC TOWER
TEPID BATHS
AON TOWER
LOWER ALBERT ST
STURDEE ST
FANSHAWE ST

BH_64279 BH_64270 CPT_TT2811 CPT_TT2835 CPT_TT48734 CPT_TT48735 CPT_TT2810 CPT_TT2809 801_BH11 BH_64269 801_BH7 CPT_TT48731 CPT_TT2808 CPT_TT2807 BH_64268 CPT_TT2806 BH_64278 BH_64274 CPT_TT48732 16930_BH6 15000_166 CPT_TT48730 15000_165 CPT_TT2832 CPT_TT48733 CPT_TT2824 801_BH10 15000_171 16930_BH5 8676_BH3 15000_167 CPT_TT48729 15000_164 BH_64277 801_BH12 15000_168 15000_BH26 16930_BH4 CPT_TT2830 CPT_TT2828 15000_169 CPT_TT2823 BH_64266 CPT_TT2812 2020_BH02 16930_BH1 CPT_TT2822 801_BH5 15000_170 BH_64276 801_BH14 BH_64267 CPT_TT2813 CPT_TT2821 CPT_TT2829 BH_66381 16930_BH2 CPT_TT2814 CPT_TT2820 801_BH13 CPT_TT2815 BH_127924 801_BH6 26753_BH1 2020_BH01 CPT_TT2819 801_BH15 CPT_TT2816 BH_127794 CPT_TT2818 801_BH18 26753_BH2 CPT_TT2817 BH_66380 15000_105 BH_66390

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DRAFTING CHECK		
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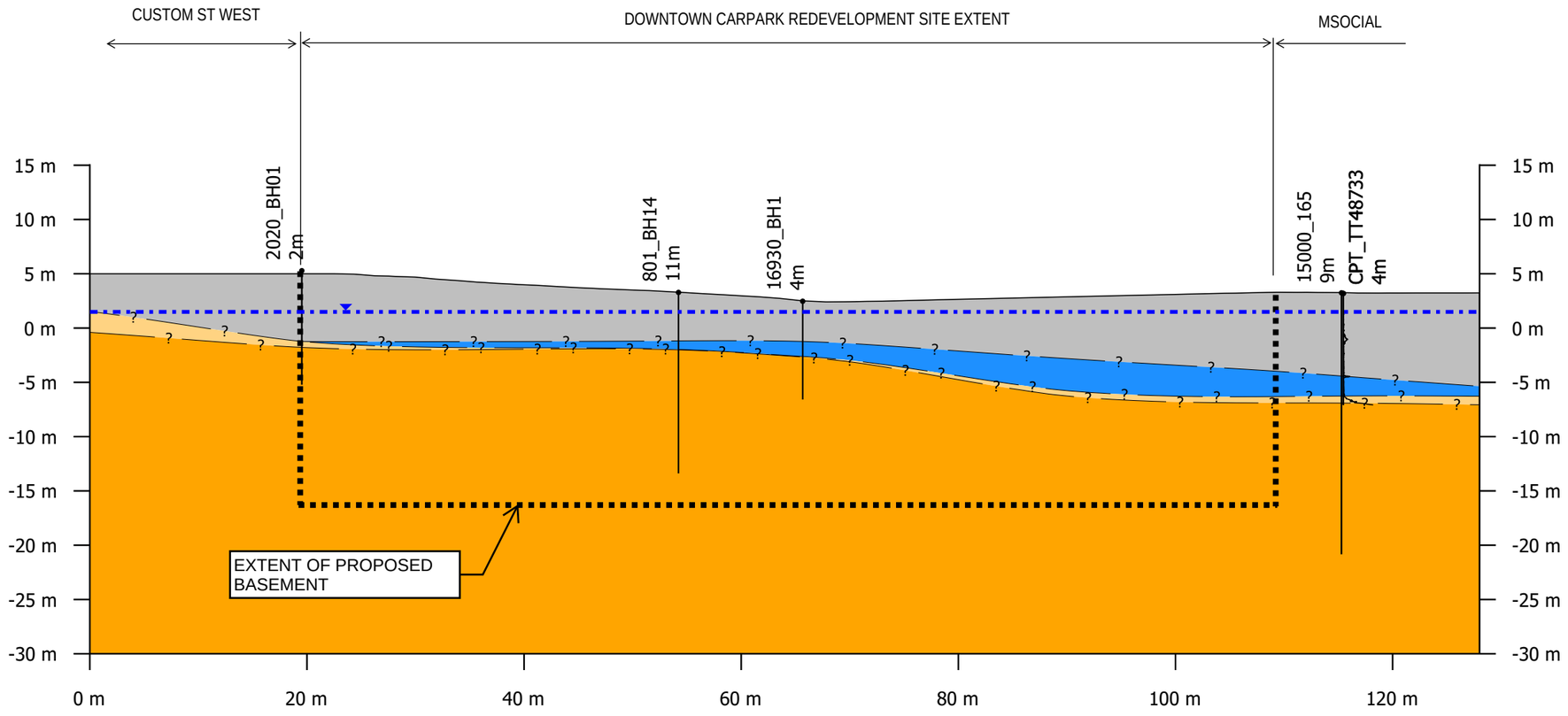
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PROJECT No.	1016043.1000	
DESIGNED	KASC	May.23
DRAWN	JC	May.23
CHECKED		
APPROVED		DATE

CLIENT PRECINCT PROPERTIES HOLDINGS LIMITED
PROJECT DOWNTOWN CARPARK REDEVELOPMENT
TITLE GEOTECHNICAL LAYOUT PLAN
UNDER REVISION 1
SCALE (A3) 1:750 FIG No. FIGURE 1 REV 1

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Section 5

Legend

- Fill
- Marine Sediments
- Tauranga Group
- Weathered ECBF
- ECBF N>50

Scale: 1:500

Vertical exaggeration: 1x

0m 40m

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REV	DESCRIPTION	CAD	CHK	DATE
	NOTES			

DESIGNED	JOWI	8.6.23	DRAWING STATUS	FOR INFORMATION
DRAWN	JOWI	8.6.23		
DESIGN CHECKED	ABL	Xxx. XX	PROJECT PHASE	CONCEPT
DRAWING CHECKED	ABL	Xxx. XX		

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APPROVED DATE

CLIENT **PRECINCT PROPERTIES HOLDINGS LIMITED**

PROJECT **DOWNTOWN CARPARK REDEVELOPMENT**

TITLE GEOLOGICAL SECTION
SECTION 5

SCALE (A3) 1:500 REV A

Appendix B Draft groundwater and settlement monitoring and contingency plan

REPORT

Tonkin+Taylor

Downtown West - Downtown Car Park Redevelopment

Groundwater and Settlement Monitoring and Contingency Plan



Prepared for
Precinct Properties Holding Limited
Prepared by
Tonkin & Taylor Ltd
Date
December 2024
Job Number
1016043.1000 v4

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Document control

Title: Downtown West - Downtown Car Park Redevelopment					
Date	**Version**	**Description**	**Prepared by:**	**Reviewed by:**	**Authorised by:**
June 2023	0	50% Draft for Client Review	ABL	PJM	PJM
September 2023	1	100% Draft for Client Review	ABL	PJM	PJM
June 2024	2	Updated 100% Draft for Client Review	ABL	PJM	PJM
July 2024	3	Final issue	ABL	PJM	PJM
December 2024	4	Updated final issue following s92 comments	ABL	PJM	PJM

Distribution:

Precinct Properties Holding Limited	PDF copy
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Appendix A Construction Monitoring and Instrumentation plan

Appendix B Summary of monitoring requirements

Appendix C **Piezometer construction records**

Appendix D **Groundwater monitoring records**

1 Introduction

Precinct Properties Holding Limited (Precinct) has engaged Tonkin and Taylor Limited (T+T) to prepare a Groundwater and Settlement Monitoring Contingency Plan (GSMCP) for the proposed basement construction at the Downtown West - Downtown Car Park Redevelopment site (Downtown West).

This draft GSMCP is applicable for up to six levels of basement construction and assumes a basement excavation generally to RL -16.3 m, though with localised excavation of up to RL -21 m to accommodate water tanks and lift pits. This draft GSMCP has been prepared to supplement the resource consent application for the Downtown West and will be updated as the design for the project develops. This draft GSMCP should be read in conjunction with the concept geotechnical and groundwater assessment report for the Downtown West development.

1.1 Project description

The Downtown West site is located at 2 Lower Hobson Street, at the corner of Lower Hobson Street and Customs Street West within the Auckland CBD (legal description Lot 9 DP 60151). The proposed site to be developed is approximately 6,442 m^2 in area, with the site on land reclaimed in stages between 1850 and 1920 by the Auckland Harbour Board.

The site is relatively level, with the ground surrounding the site varying from RL 4 m to RL 5 m. A 9-storey (1 below ground level and 8 suspended floors) public carpark building known as the Downtown Carpark currently occupies the site. The building is surrounded by MSocial Hotel immediately to the north, HSBC and Aon Towers to the east, Custom Street West and Lower Hobson Street to the south and west respectively. Figure 1.1 shows the location of the site. The historical Tepid Baths are located across Lower Hobson Street to the west.

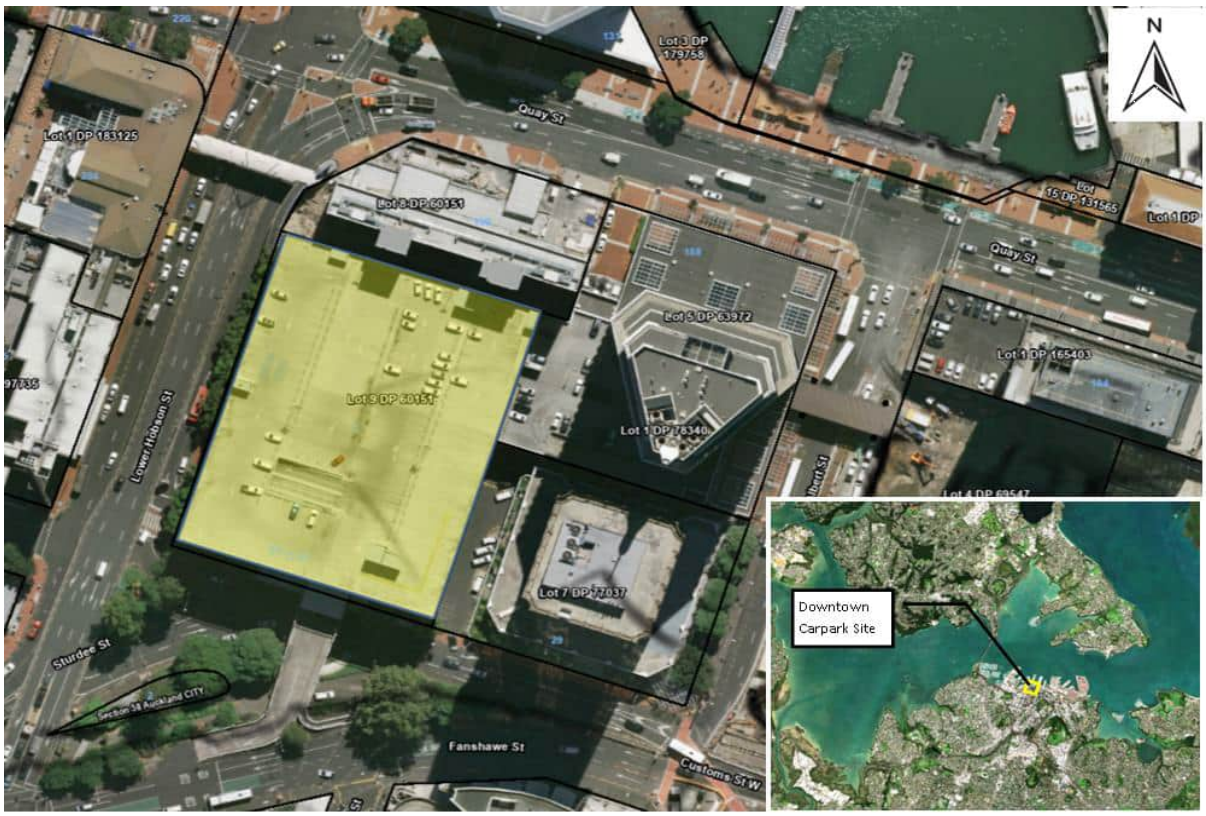


Figure 1.1: Downtown West site.

The proposed development for the site consists of two multi-storey towers, three podium structures and up to six level basement that extends across the full footprint of the site (with the exception of the sixth level of basement which is a localised level to accommodate water tanks and lift pits).

1.2 Definitions

AC	The Auckland Council.
Commencement of dewatering	Excavation below the groundwater table and/or commencing taking any groundwater from the basement excavation excluding the installation of the pile walls.
Completion of excavation	When all bulk excavation below groundwater level has been completed. The permanent retaining walls have been propped with the building floors and all foundation/footing excavations within 10 m of the retaining wall have been completed.
Completion of temporary dewatering	When all the external base slab and basement walls are essentially completed, with permanent drainage systems in place, and the structures internal support mechanisms, including basement floors have been completed.
Completion of construction	When the Certificate of Completion is issued by AC.
Significant damage	When damage is considered to affect serviceability or structural integrity.
Serviceability damage	The degree of damage is minor i.e. when doors and windows start sticking.
Damage	Includes aesthetic, serviceability and significant damage.
Alert Level	Monitoring reaches a level close to, or equal to the design value, which is above the level where potential damage could occur, and requires a review to assess the future trend.
Alarm Level	Monitoring reaches the design value, and/or level close to which damage could occur, and requires immediate action including the cessation of ground dewatering and other construction activities that may have an effect on ground deformation.
The Council	Auckland Council (Team Leader, Water Allocation, NRSI) or nominated AC staff acting on The Team Leader's behalf.
NRSI	Natural Resources and Specialist Input, Auckland Council.
RL	Reduced Level.

Services	Includes for example fibre optic cables, sanitary drainage, gas and water mains, power and telephone, road infrastructure assets such as footpaths, kerbs, catch-pits, pavements and street furniture.

2 Monitoring

2.1 Summary

Monitoring of the excavation works and surroundings shall be undertaken to check that the ground deformations and groundwater conditions are consistent with the design analyses and that the response of neighbouring structures are within adopted design tolerances.

The monitoring requirements include location of monitoring points, frequency of monitoring, action trigger levels, response procedures and reporting requirements are detailed in the following sections. Survey monitoring locations are shown in the Construction Monitoring Instrumentation Plan Figure 1, Appendix A.

The proposed construction phase monitoring to assess the effects on ground surrounding the site and of the services and structures identified above shall include:

- Pre and post-basement construction condition surveys of the potentially affected building and services.
- Regular visual external survey of the surrounding ground, pavement, associated street infrastructure and structures during construction to identify any deterioration of pre-construction baseline conditions.
- Precise levelling survey of markers/pins on the buildings and maintained existing retention structures surrounding the excavation. The marks shall be set as low as practicable on structures on elements in direct connection with the foundation level (i.e. columns or perimeter footings).
- Precise levelling survey of pavement line levels surrounding the site at ground deformation marks.
- Survey of the retaining wall capping beam at approximately 15 m spacing. The number and spacing of locations may be altered depending on the layout including lateral support spacing and basement access points.
- Retaining wall deformation profile measurement via inclinometers installed within the walls at critical locations, allowing for site access constraints that limit where measurements may be safely undertaken.
- Groundwater monitoring of piezometer installations to assess any relationship between groundwater drawdown and ground deformation.

Monitoring results will be collated by the project Geotechnical Engineer and compared with the specified trigger levels. These records will be held by the project Geotechnical Engineer and will be available for inspection as required.

2.2 Reporting of monitoring records

2.2.1 General

Survey points will be monitored by the Contractor and results will be provided to a qualified Geotechnical Engineer for their review at the frequency set out in the Resource Consent conditions and the following sections. Survey results should be submitted to the project Geotechnical Engineer within three working days of taking the readings.

Monitoring results will be collated by the project Geotechnical Engineer and compared with the specified trigger levels. These records will be held by the project Geotechnical Engineer and will be available for inspection as required. If results exceed the trigger levels mitigation measures will be implemented and results will be reported to the Consents Manager as described in Section 7 to 8.

2.2.2 Pre-dewatering baseline readings

Building dilapidation and pre-condition surveys will be required prior to commencement of construction.

Baseline readings for ground and building deformation and retaining wall deformation shall be established prior to commencement of excavation.

Baseline readings for groundwater shall be established with weekly readings for at least one month prior to the commencement of dewatering. These baseline readings shall be submitted to the Council for approval prior to the commencement of excavation.

2.2.3 Reporting intervals and requirements

During excavation, all monitoring records as detailed in this report shall be compiled and submitted to the Council at **three monthly intervals** from the commencement of excavation until **six months** after the completion of basement construction, when the basement is sealed (or at such time following the completion of basement construction that stable measurements are demonstrated and written approval is granted from the Council).

Each report shall include the following:

1 Monitoring records presented in a tabulated format as well as on a timeline plot.
2 Comparison of monitoring data with trigger levels and the assumed design models.
3 Previous results set out with an explanation of any trends.
4 A construction progress summary.
5 Any other information relevant to the reporting period (i.e. exceedance of trigger levels and contingency measures being undertaken).

3 Groundwater monitoring

3.1 Monitoring piezometer details and proposed drawdown trigger levels

Groundwater levels shall be monitored at four planned locations, as set out in Table 3.1, subject to accessibility to the locations and more detailed underground service location check. Measurements shall be accurate to ±100mm for all readings.

Table 3.1: Groundwater drawdown planned locations and drawdown trigger levels

ID	Location	Screen Depth (mRL)[2]	Trigger limits below lowest baseline level[1] (m)	
			Alert Trigger Level 1	Alert Trigger Level 2
MW01	Northeast corner of the site targeting infilled graving dock	-4 to 3	0.7	1.0
MW02	Southeast corner of the site	-1 to 3	0.7	1.0
MW03	Southwest corner of the site	-4 to 3	0.7	1.0
MW04	Northwestern corner – to be confirmed based on access	-4 to 3	0.7	1.0

Note:

1 Baseline readings shall be agreed upon in writing with the Council prior to dewatering.

2 The screen for the monitoring wells are proposed within the reclamation fill and marine sediments. The screen depths will be be confirmed following drilling of the boreholes.

See Figure 1 in Appendix A for the monitoring well planned locations.

Should any of the monitoring bores be damaged and become inoperable during the basement construction monitoring period, then the Council is to be informed and a new monitoring bore is to be drilled at a nearby location in consultation with the Team Leader.

3.2 Groundwater monitoring intervals

The groundwater level in the piezometers shall be measured at the following intervals:

Prior to dewatering	Weekly in the month prior to commencement of dewatering.
During temporary dewatering	Weekly, until the completion of temporary dewatering, or at such time following the completion of excavation that stable measurements are demonstrated and written approval is granted from The Team Leader (AC) to reduce monitoring frequency.
After completion of dewatering for construction	Monthly for minimum six (6) months or until a consistent pattern of groundwater records are obtained in which no evidence of adverse effects is apparent and groundwater levels are above alert trigger levels.

The monitoring frequency may be changed if approved by the Team Leader Central Monitoring. Any change shall be specified in the GSMCP. In addition, the monitoring period post Completion of Temporary Dewatering may be extended, by the Team Leader, Central Monitoring, Resource Consenting and Compliance, if measured groundwater levels are not consistent with inferred seasonal trends or predicted groundwater movement.

The project Geotechnical Engineer will review the results of this monitoring and compare with the trigger levels detailed in the Resource Consent Conditions. If the alert or alarm levels in Table 4.1 are reached, the actions outlined in Section 7 shall be carried out.

4 Retaining wall deflection monitoring

4.1 General

The retention system shall be monitored via survey monitoring points installed along the perimeter capping beam at approximately 15 m spacing, and deformation down the retaining wall will be measured via inclinometers installed within the walls at critical locations. The number and spacing of locations may be altered depending on the layout including lateral support spacing and basement access points.

Proposed locations of the survey monitoring points are shown in Figure 1 in Appendix A.

Survey measurements shall be accurate to ±2 mm. A record of the survey results should be submitted to the Geotechnical Engineer within one working day.

4.2 Retaining wall monitoring intervals

Surveys shall be carried out at the following intervals:

Prior to dewatering	At least two baseline surveys to establish baseline readings
During dewatering, until one month after completion of excavation	Weekly (minimum) and at each 2 metres depth of excavation or when changes to the propping system are being carried out
From one month after completion of excavation, until completion of temporary dewatering	Fortnightly until completion of temporary dewatering.

Monitoring frequency may be reduced if stable measurements are demonstrated and written approval is granted from the Council.

4.3 Retaining wall deflection trigger levels

The results of this monitoring are to be compared with the design assumptions and baseline readings. Trigger levels at which actions are required to be undertaken are summarised in Table 4.1. If the Trigger Levels are reached, the actions outlined in Section 7 shall be carried out.

Table 4.1: Trigger levels for retaining wall deflections

Monitoring ID	Location	Alert trigger level	Alarm trigger level
Survey peg on retaining wall	Top of wall along northern boundary	15 mm	20 mm
	Top of wall along western and eastern boundary	20 mm	25 mm
	Top of wall along southern boundary	15 mm	20 mm
Inclinometers (01 to 03)	Inclinometer in the retaining walls	20 mm	25 mm

Note: Inclinometers are proposed to be installed within the Diaphragm wall. Inclinometers are currently not allowed for along the southern and southeastern perimeter where anchored sheet piles are proposed. However, inclinometer over this section will be considered as required.

5 Ground and building deformation monitoring

5.1 General

Ground and building deformation monitoring marks shall be established on the pavements and buildings surrounding the excavation area at the locations shown on Figure 1 in Appendix A, subject to permission from neighbouring properties.

Survey measurements shall be accurate to ±2 mm. A record of the survey results should be maintained on site and forwarded to the project Geotechnical Engineer within one working day.

5.2 Settlement monitoring intervals

Surveys shall be carried out at the following intervals:

Prior to dewatering	At least two baseline surveys to establish baseline readings.
During temporary dewatering	Weekly.
Post temporary dewatering	Monthly for six months or until such a time following the completion of temporary dewatering that stable measurements are demonstrated and written approval is granted from the Council.

Monitoring frequency may be reduced following the completion of excavation if stable measurements are demonstrated and written approval is granted from the Council.

5.3 Settlement trigger levels

Monitoring data are to be compared with the design assumptions and baseline readings. Alert and Alarm Levels at which actions are required to be undertaken are summarised in Table 5.1. If the alert or alarm levels in Table 4.2 are reached, the actions outlined in Section 7 shall be carried out.

Table 5.1: Ground and building deformation mark trigger levels

Settlement survey mark ID	Location	Total settlement (mm)		Differential settlement	
		Alert level	Alarm level	Alert level	Alarm level
Ground survey pins	Custom Street West	14	20	1V:500H[1]	1V:250H[1]
	Lower Hobson Street	17	25		
	Private accessway between site and AON/ HSBC tower	14	20		
Building survey pins	MSocial	8	12	1:1000 between any two Building deformation marks	1:500 between any two Building deformation marks
	HSBC Tower	8	12		
	AON Tower	8	12		
	Hobson St Flyover	8	12		
	204 Quay Street	7	10		
	Tepid Baths	7	10		
	Watermark Building (85 Custom St West)	7	10		

6 Condition surveys

6.1 Underground service condition surveys

6.1.1 General

The condition surveys should comprise pressure testing or leak detection (acoustic) surveys carried out on the water main and pressure testing or CCTV inspection of the stormwater and wastewater pipes within 20 m of the site.

6.1.2 Condition survey intervals

A condition survey of the public services immediately adjacent to the site shall be carried out before the commencement of dewatering. The survey shall be repeated no earlier than six months after the completion of basement construction.

Following the post-construction survey, a report shall be submitted to the Council within 15 working days demonstrating that the services surveyed have not been damaged.

If the survey indicates damage to the services since the pre-construction survey, a determination of the cause of damage identified shall be reported by a Chartered Professional Engineer, together with a methodology for repair of any damage caused wholly or in part by the construction works.

6.2 Building condition surveys

6.2.1 Accessibility

If a structural condition survey requires access to a building or property, then in the event that access is declined by the owner or subject to unreasonable terms, the Council shall be notified as soon as is practicable. A report from a Chartered Professional Engineer shall be provided identifying whether any alternate risk monitoring options are available that do not involve access to the third party property. The report shall clearly state whether the identified alternative monitoring options will be sufficient to monitor any settlement risk to the property/s to which the monitoring relates and whether they will provide sufficient early detection warning to enable contingency measures to be implemented. Written approval from the Council must be obtained before any alternative monitoring option is implemented.

6.2.2 Recommended building condition survey locations

A visual inspection of the surrounding ground and buildings within 20 m of the project boundary. Survey shall be carried out for the structures in the immediate vicinity of the development, as set out in the following table. For the internal building survey, the survey shall cover the basement levels and the ground floor.

Table 6.1: List of buildings requiring condition surveys

Address	Legal Title	External survey	Internal Survey	Settlement Pins (Yes/No)
MSocial (196 Quay St)	Lot 8 DP 60151	✓	✓	Yes
HSBC Tower (188 Quay St)	Lot 5 DP 63972 and Lot 1 DP 7834	✓	✓	Yes
AON Tower (21/29 Custom St West)	Lot 7 DP 77037	✓	✓	Yes
Tepid Baths (100 Custom St West)	Lot 2 DP 184176	✓	✓	Yes
Watermark (85 Custom St West/ 1 Lower Hobson St)	Lot 2 DP 197735	✓	×	Yes
204 Quay Street	Lot 1 DP 183125	✓	×	No
Lower Hobson Street flyover	N/A	✓	×	Yes

6.2.3 Pre and post-construction condition survey

A pre-construction condition survey of the specified buildings shall be carried out prior to the commencement of dewatering.

The survey report shall confirm the existing condition of the building and enable the sensitivity of the buildings to potential damage caused by groundwater and ground settlement changes to be accurately determined. Major features of the building and site developments shall be recorded including location, type of construction, age and present conditions including defects.

The survey shall include, but not be limited to, the following:

1 A description of the type of foundations.

2 A description of existing levels of damage considered to be of an aesthetic or superficial nature.

3 A description of existing levels of damage considered affecting the serviceability of the building where visually apparent without recourse to intrusive or destructive investigation.

4 An assessment as to whether existing damage may or may not be associated with actual structural damage and an assessment of the susceptibility of the buildings/structures to further movement and damage, to the extent predicted.

5 Photographic evidence of existing observable damage.

6 Confirmation of the installation of deformation pins as required by this specification.

7 A review of proposed Alarm and Alert Levels to confirm they are appropriately set and confirmation that any ground settlement less than the Alarm Level will not cause damage.

8 An assessment of whether the monitoring frequency is appropriate.

9 An assessment of whether the location and density of existing deformation pins is adequate and appropriate for the effective detection of change to building and structure condition.

Within six months of completion of construction, a post-construction survey covering the matters detailed above shall be completed the building. The survey report shall include a determination of the cause of damage identified (if any) since the pre-construction or previous survey and steps to repair it.

A copy of the pre and post-construction reports shall be forwarded to the Council within 15 working days of completing the reports along with a certificate from the Chartered Surveyor or Chartered Professional Engineer who has certified that the survey has been completed in a professional manner and is an accurate assessment of the condition of the building concerned.

6.2.4 Construction phase inspections

A visual inspection undertaken by the Principal Contractor at regular intervals shall monitor any deterioration or further cracking of any pre-existing cracks on the following:

- The ground within 10 m of the excavation.
- External facades of the buildings identified for structural condition surveys.

A photographic record of the surveys is to be maintained including the time, date and any observations for each inspection. This record is to be maintained on site and forwarded to T+T typically monthly but within one day if any deterioration or cracking is observed.

6.2.5 Condition survey intervals

A visual inspection of the surrounding ground and specified buildings shall be carried out at the following intervals:

Prior to dewatering	Pre-construction survey as detailed in Section 6.2.3.
From Commencement of Dewatering to Completion of Dewatering	Weekly visual inspections as detailed in Section 6.2.4.
After completion of basement construction	Post-construction survey within six months of completion of construction as detailed in Section 6.2.3.

6.3 Underground service condition surveys

6.3.1 General

The condition surveys should comprise pressure testing or leak detection (acoustic) surveys carried out on the water main and pressure testing or CCTV inspection of the stormwater and wastewater pipes within 20 m of the site.

6.3.2 Condition survey intervals

A condition survey of the public services immediately adjacent to the site shall be carried out before the commencement of dewatering. The survey shall be repeated no earlier than six months after the completion of basement construction.

Following the post-construction survey, a report shall be submitted to the Council within 15 working days demonstrating that the services surveyed have not been damaged.

If the survey indicates damage to the services since the pre-construction survey, a determination of the cause of damage identified shall be reported by a Chartered Professional Engineer, together with a methodology for repair of any damage caused wholly or in part by the construction works.

7 Alert and alarm trigger level response procedures

7.1 Response procedure if the alert trigger levels are exceeded

If any of the monitoring alert settlement trigger levels are reached then one or more of the contingency options described in Section 8 of this plan should be carried out together with:

1 Notify the Project Manager.

2 Notify the Council, in writing within one working day of the trigger level being exceeded, with details of any actions being undertaken.

3 Survey all monitoring marks within a 50 m radius of the monitoring point, and compare them to settlement trigger levels.

4 Review the monitoring data, as-built details and geology and compare against the assumptions made in the design analyses.

5 Submit a report by a Chartered Professional Engineer to the Council for approval, within one week of trigger level exceedance, which provides analyses of all monitoring data, including wall deflection monitoring, relating to the exceedance of any of the trigger levels and any recommendations for remedial actions and time frames for implementing these actions. If no remedial actions are considered necessary, then justification for this viewpoint is required.

6 All monitoring pins and groundwater levels within 50 m shall be surveyed at two day intervals until such time the written report in (5) has been approved by the Council.

If considered necessary:

1 Increase monitoring frequency.

2 Develop a detailed contingency plan and submit to the Council.

7.2 Response procedure if the alarm trigger levels are exceeded

If any of the monitoring alarm settlement trigger levels are reached then the following shall be carried out in addition to the actions outlined in the previous section:

1 Cease further lowering of the water table or any other activity which has the potential to cause further deformations.

2 Submit a written report by a Chartered Professional Engineer to the Council for approval, within one week of alarm trigger Level exceedance, which provides analyses of all monitoring data and any recommendations for remedial action.

3 Once approved by the Council the recommendations shall be implemented.

Dewatering and/or construction may be resumed once the Council provides written notice to the Consent Holder that the Council is satisfied that damage to buildings, structures and services is unlikely with or without any approved additional mitigation measures to be undertaken or that the Council is satisfied that owners of potentially affected buildings, structures and services have given written approval for dewatering and/or construction to continue.

8 Contingency Options

8.1 General

If any of the monitoring trigger levels are exceeded the general response will be as detailed in Section 7. Specific actions will be selected depending on the exact nature of the problem. Possible contingency actions are detailed in the following sections.

8.2 Retaining wall deflection contingency measures

In the event of retaining wall deflections exceeding the monitoring trigger levels, a review of the retaining design model shall be carried out to assess the potential increased load in the wall, anchors and existing props, and potential increase in mechanical ground deformations.

If required, the following actions may be taken:

1 Reduce load on the wall by allowing water to drain from behind the wall. This would be temporary while one of the other mitigation measures is undertaken.

2 Place a temporary berm of soil in front of the wall.

3 Install additional props and/ or anchors.

8.3 Groundwater drawdown contingency measures

8.3.1 Groundwater recharge during construction

One option to lessen the effect of groundwater dewatering during the excavation in a local area is to inject further water into the ground through shallow injection wells or a trench.

The shallow well injection system will likely consist of the following:

1 2 m to 6 m deep, 150 mm diameter wells spaced at approximately 10 m centres (depending on degree of drawdown).

2 A water supply manifold connected to a header tank with float valve and flow meter.

The location of the wells will be dependent on where the groundwater has been drawn down. Following the implementation of the injection wells the groundwater levels will be monitored closely.

8.3.2 Options to reduce temporary and permanent drawdown

In the event of groundwater drawdown exceeding the monitoring trigger levels, one or more of the following actions may be taken:

1 Grout anchor points, wall connections or other areas of excessive seepage.

2 Shallow reinjection wells to inject potable water into the ground to mitigate the change in groundwater level (groundwater recharge).

3 Construct a grout curtain to reduce inflow.

4 Install an impermeable wall inside the diaphragm/ sheet pile wall, designed to resist hydraulic pressure.

5 Locally tank floor to extend the drainage path.

6 Convert to a tanked basement.

8.4 Ground and building deformation contingency measures

In the event of ground or building deformation exceeding the monitoring trigger levels, one or more of the following actions shall be taken:

1 Check public safety is maintained.

2 Review the potential causes for ground of building deformations.

3 Discuss the situation with the property/service owner that may be affected.

4 Monitor the rate of settlement (assuming that other steps have been undertaken to address the cause).

5 Undertake remedial works including foundation strengthening.

6 Accept and reinstate resultant damage.

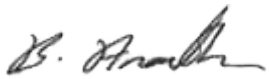
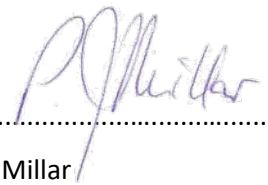
9 Applicability

This report has been prepared for the exclusive use of our client Precinct Properties Holding Limited, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

We understand and agree that our client will submit this report as part of an application for resource consent and that Auckland Council as the consenting authority will use this report for the purpose of assessing that application.

Recommendations and opinions in this report are based on data from discrete investigation locations. The nature and continuity of subsoil away from these locations are inferred but it must be appreciated that actual conditions could vary from the assumed model.

Tonkin & Taylor Ltd
Environmental and Engineering Consultants

Report prepared by:	Authorised for Tonkin & Taylor Ltd by:
	
.....	.....
Ananth Balachandra	Peter Millar
Geotechnical Engineer	Project Director

6-Dec-24
p:\1016043\1016043.1000\2 geotechnical\issueddocuments\gsmcp-dtc-december 2024-rev 4_final.docx

Appendix A Construction Monitoring and Instrumentation plan

COPYRIGHT ON THIS FIGURE IS RESERVED DO NOT SCALE FROM THIS FIGURE - IF IN DOUBT, ASK.

C:\12dSynergy\data\ALBTCAD\DCR Concept and Resource Consent Stage_1352\CAD\FIG\1016043.1000-GEO-F1.dwg 2023-May-26 9:36:57 AM Plotted By: JONALD CASTRO

LEGEND

- PROPERTY BOUNDARY
- ECBF CONTOUR (1m INTERVAL)
- GEOMAPS WATER LINE
- GEOMAPS ABANDONED WATER LINE
- WATER HYDRANT
- WATER VALVE
- GEOMAPS STORMWATER LINE
- GEOMAPS ABANDONED STORMWATER LINE
- STORMWATER CATCHPIT
- STORMWATER MANHOLE
- GEOMAPS WASTEWATER LINE
- GEOMAPS ABANDONED WASTEWATER LINE
- WASTEWATER MANHOLE
- CRL UNDERGROUND ROUTE
- CHORUS
- VECTOR COMMUNICATION - UNDERGROUND ROUTE TRENCH
- VECTOR COMMUNICATION - UNDERGROUND INSTALLED IN PLACE
- GAS
- GAS VALVE
- VODAFONE
- FIBRE OPTIC
- POWER
- LAMP

LEGEND FOR MONITORING

- PROPOSED GROUNDWATER MONITORING WELL
- PROPOSED RETAINING WALL INCLINOMETER
- PROPOSED GROUND SURVEY PEG (NUMBERING TBC)
- PROPOSED BUILDING SURVEY PIN (NUMBERING TBC)
- PROPOSED SURVEY PEG ON RETAINING WALL (NUMBERING TBC)

QUAY ST
MSocial
MW04
IN01
MW01
IN03
IN02
HSBC Tower
Watermark Building
LOWER HOBSON ST
MW03
LOWER ALBERT ST
AON Tower
MW02
Tepid Baths
STURDEE ST
FANSHAWE ST

A3 SCALE 1:750
0 5 10 15 20 25 30 35 40 (m)

ORIGINAL IN COLOUR

DRAFT

CHECK PRINT	INITIAL	DATE
BACKDRAFTING REQUIRED		
READY TO ISSUE		
CAD SELF CHECK		
DESIGN CHECK		
DRAFTING CHECK		
BACKDRAFTER - EDITS MADE		
BACKCHECKER - EDITS CONFIRMED		

Tonkin & Taylor
Exceptional thinking together www.tonkintaylor.co.nz

NOTES:
1. AERIAL PHOTO, STORMWATER LINE, WASTEWATER LINE AND WATER LINE SOURCED FROM AUCKLAND COUNCIL GEOMAPS, LICENSED FOR RE-USE UNDER THE CREATIVE COMMONS ATTRIBUTION 4.0 NEW ZEALAND LICENCE (CC BY 4.0). CAPTURE DATE 24/05/2023.
2. VODAFONE AND CRL INFORMATION SUPPLIED BY BEFORE U DIG. REF 11197173 - Vodafone Plan.pdf AND "11197172 - CityLink Plan (002).pdf".
3. CHORUS INFORMATION SUPPLIED BY CHORUS, REF "CHORUS.pdf".
4. VECTOR POWER, COMMUNICATION AND GAS SUPPLIED BY VECTOR, REF "VECTOR ELECTRICITY.pdf" , "VECTOR COMMUNICATION.pdf" AND "VECTOR GAS.pdf".

PROJECT No.	1016043.1000	
DESIGNED	ABL	May.23
DRAWN	JC	May.23
CHECKED		
APPROVED		DATE

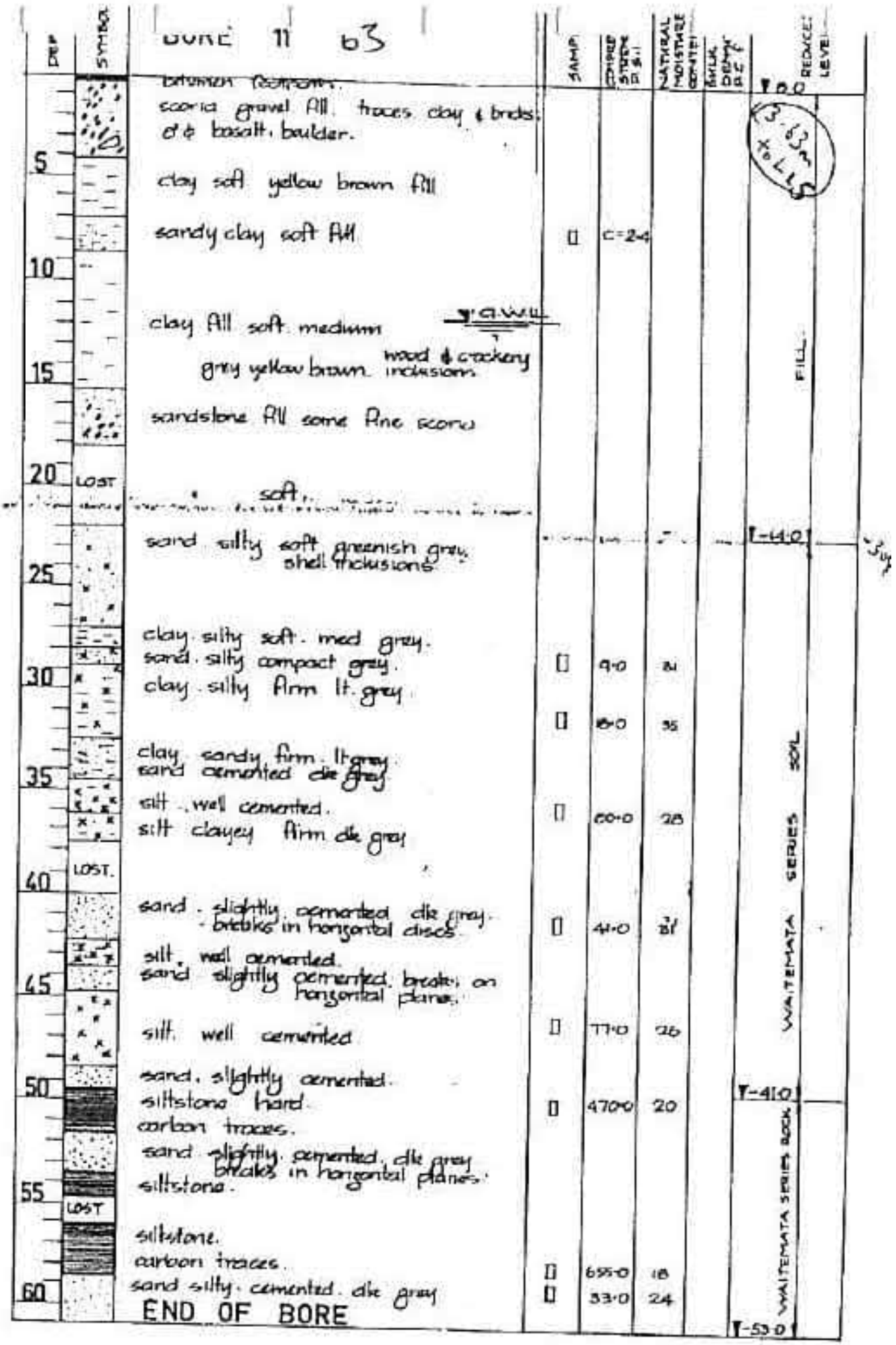
CLIENT **PRECINCT PROPERTIES HOLDINGS LIMITED**
PROJECT **DOWNTOWN CARPARK REDEVELOPMENT**
TITLE CONSTRUCTION MONITORING AND INSTRUMENTATION PLAN
SCALE (A3) 1:750 FIG No. FIGURE 1 REV 1

Appendix D Previous ground investigation results

801_BH11

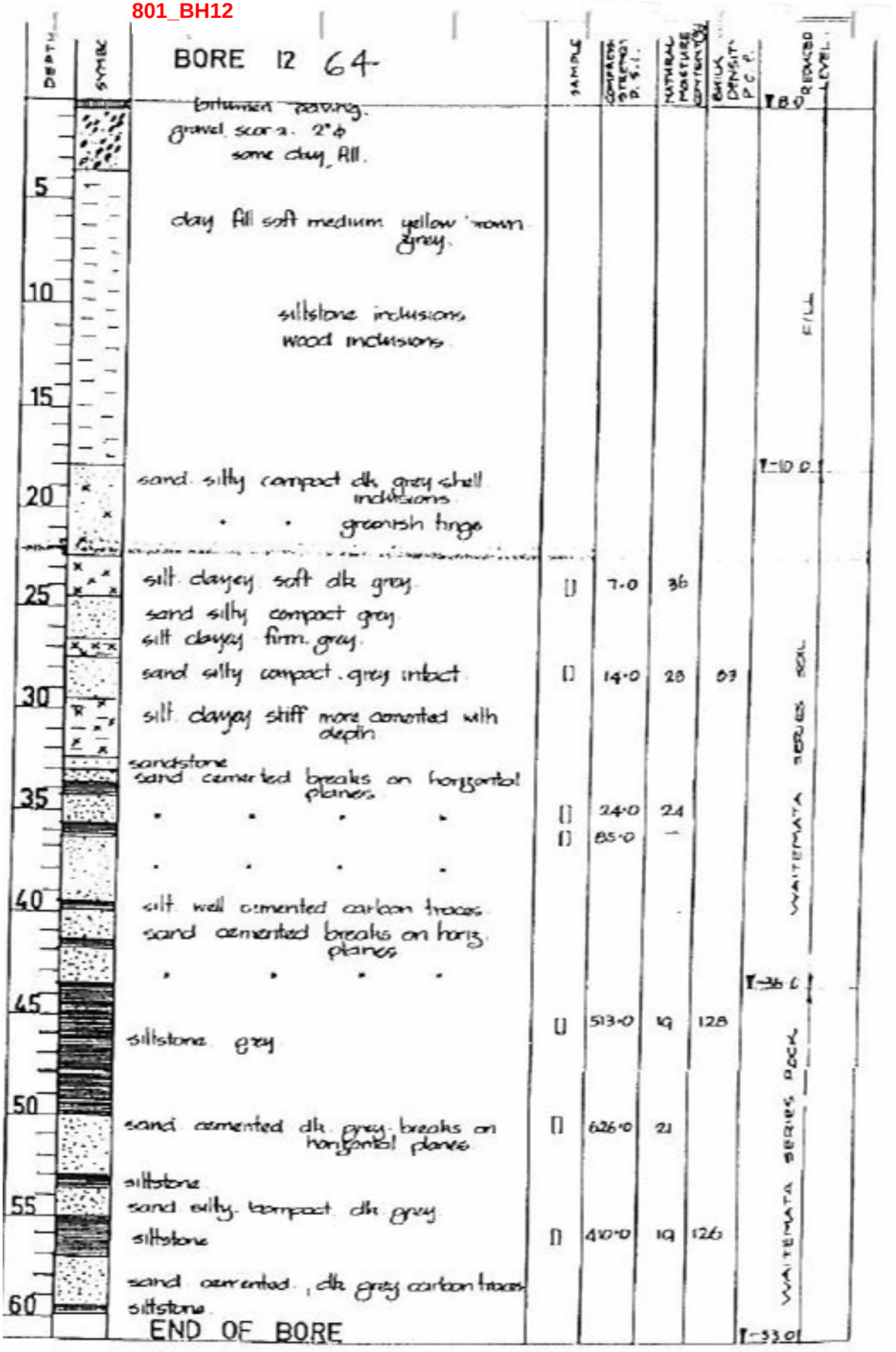
BH_TT66664

BH_64334



BH_TT66665

801_BH12



BH_64335

BH_TT66657

TONKIN & TAYLOR LTD.

D/Base N° 88

BOREHOLE LOG

BOREHOLE NO 3

SHEET 1 OF 2

PROJECT: Lower Hobson St. Bridge LOCATION: Lower Hobson St. JOB NO: 8676

CO-ORDINATES: See Dwg 8676-1

RL: 3.90m

DATUM: L & S

DRILL TYPE Mayhew 250

DRILL METHOD: Open-barrel HMLC

DRILL FLUID: Water

HOLE STARTED: 16/7/88

HOLE FINISHED: 16/7/88

DRILLED BY: Drillwell Ltd

LOGGED BY MH CHECKED BY: CJF

DRILLING AND TESTS						ENGINEERING DESCRIPTION						GEOLOGICAL	
FLUID LOSS	WATER	CORE RECOVERY	METHOD/CASING	SAMPLES, TESTS	RL (m) / DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	SOIL NAME, PLASTICITY OR PARTICLE SIZE CHARACTERISTICS, COLOUR, SECONDARY AND MINOR COMPONENTS	MOISTURE CONDITION	SHEAR STRENGTH OR RELATIVE DENSITY	ESTIMATED SHEAR STRENGTH, kPa	ORIGIN TYPE, MINERAL COMPOSITION, DEFECTS, STRUCTURE	UNIT
		0	WASH		1.0			NO CORE : Cuttings are GRAVELS (Basecourse?) 1.0m ? ?					
				• 96kPa *LV	2.0			CLAY, silty, yellow-brown and grey mottles, some pockets of SILT - some pieces of mudstone & sandstone h.w. > 50mm		St - Vst		FILL fine texture	FILL
		100		• 121kPa	3.0			Matrix of ROCK (80%) and SOIL (20%) Rock is sandstone and mudstone, HW, weakly cemented. Soil is SILT and CLAY yellow-brown mottles		Extremely Low (Rock)	N/A	FILL Rock and Silt	
		100		Too • Hard	4.0								
		100		Too • Hard									
		100		Too • Hard									
		100		Too • Hard	5.0								
		0	OPEN BARREL	N=5				SAND (fine), dark grey, some sea shells				Marine Sediments	
		100			6.0					VL-L	N/A		KAIHU GROUP
					7.0			- grades silty, some bands of SAND, clayey, silty					
					8.0			- grades clayey					
		100		N=5									
					9.0			CLAY, plastic, lt grey-brown, some dk brown mottles minor silt		S - F?			
	30				10.0			Rock and soil (PTO)					

BH_64327

BH_TT66657

D/Base N° 88

TOMKIN & TAYLOR LTD.

BOREHOLE LOG

BOREHOLE NO 3

SHEET 2 OF 2

PROJECT: Lower Hobson St Bridge LOCATION: Lower Hobson St JOB NO: 8676

CO-ORDINATES: See Dwg 8676-1 DRILL TYPE Mayhew 250

DRILL METHOD: Open-barrel HMLC

RL: 3.90m

DATUM: L & S

DRILL FLUID: Water

HOLE STARTED: 16/7/88

HOLE FINISHED: 16/7/88

DRILLED BY: Drillwell Ltd

LOGGED BY MH CHECKED BY: CJF

DRILLING AND TESTS						ENGINEERING DESCRIPTION					GEOLOGICAL	
FLUID LOSS	WATER	CORE RECOVERY	METHOD/CASING	SAMPLES, TESTS	RL (m) DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	SOIL NAME, PLASTICITY OR PARTICLE SIZE CHARACTERISTICS, COLOUR, SECONDARY AND MINOR COMPONENTS	MOISTURE CONDITION	SHEAR STRENGTH OR RELATIVE DENSITY	ESTIMATED SHEAR STRENGTH, kPa	ORIGIN TYPE, MINERAL COMPOSITION, DEFECTS, STRUCTURE	UNIT
		100		N=70+				NO CORE : SOIL?					
		66		UC	11.0			ROCK and SOIL : Rock is interbedded SAND-STONE and MUDSTONE, SW., weakly to moderately cemented, dk grey Soil is SAND (fine) dk grey, silty.		Ext. Low - Very Low Strength (ROCK)	N/A	ROCK and SOIL	Waitemata Group Formation
		0		N=100+	12.0								
		100			13.0								
		100		N=100+	14.0								
		100			15.0								
		100		N=100+	16.0			End of Borehole @ 16.1m					

BH_64327

BH_66381

ARTP76

NEW ZEALAND GEOLOGICAL SURVEY

LOG OF DRILL HOLE

HOLE No. ART 43

PROJECT Auckland Rapid Transit FEATURE tunnel alignment LOCATION Customs St West

GRID REF. M.O.W. CO-ORD 703810.6N 299896.4E DATUM L & S RL GROUND 3.73m

ANGLE FROM HORIZONTAL 90° DIRECTION PHOTO FILE No. COLLAR

DESCRIPTION OF CORE	STRUCTURES
FORMATION NAME / ROCK TYPE / DESCRIPTION OF CORE	JOINTS; VEINS; SEAMS; SHATTER; SHEAR & CRUSH ZONES; FOLIATION; SCHISTOSITY; LINEATIONS.
FILL	concrete pavement
	fill, stiff clays and weathered Waitemata Gp sands
	basalt sand layer
	Fe stained sands and silts, occ corestones of sandst and siltst
	well cem sst frags
	Fe stained
MARINE SANDS	grey/green marine sands with numerous shell frag
	sands loose
	stiff grey clays
	interb uncem sands
	f sands grad to med sst poorly cemented
	interb w siltst
WAITEMATA GROUP	f sst gr to med sst rel uncemented
	siltst with concr layer
	rel well cem f sst interb siltst
	f sst gr to med sst with interb siltst
	med and f sst well cem siltst
	f sst grad to med sst
	siltst with thin layers of f sst
	f sst with siltst lenses
	siltstone
	fine sandstone grading to coarse sst mod cem
	siltst with interb cem f sst
	med sst gr to c sst, mod cem
	siltstone in thick layer
	fine sandstone
	grad to c sandstone

tube sample

soft material washed by drilling

fractures at 70° in clay

2 jts partly cem dip 60° and 70°

bedding horizontal

fracture in sandstone dips 70° v rough surface

bedding dips 20°

bedding dips 30°

R.Q.D.
79
91
87
54
70
82
100
77
27

WEATHERING: UW Unweathered; SW Slightly Weathered; MW Moderately Weathered; HW Highly Weathered; CW Completely Weathered

HARDNESS: VH Very Hard; H Hard; MH Moderately Hard; M Medium; S Soft; VS Very Soft

1 10 20 10^2 10^3 _ nat fractures/m of core

COMMENTS: grinding of core

ENGINEERING GEOLOGY SECTION

LOGGED I.R.Brown DRILLED G.Hadfield

DATE 28/5/74 DATE 16-17/5/7

TRACED B.R. DRILL Acker

CHECKED CORE BOXES

LENGTH 22.86m

SOURCE: NZGD

2020_BH01

BOREHOLE LOG

BOREHOLE No.: BH01

SHEET: 1 OF 3

PROJECT: Downtown Car Park
JOB No.: 1012134.1000
LOCATION: 31 Customs Street West, Auckland

CO-ORDINATES: 5920984.16 mN, 1757327.82 mE (NZTM2000)
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND:
R.L. COLLAR:
DATUM:
SURVEY: GIS\Web map viewer

DRILLED BY: Lei
LOGGED BY: JELE
CHECKED: CRB
START DATE: 05/09/2020
FINISH DATE: 05/09/2020
CONTRACTOR: McMillan Drilling

GEOLOGICAL UNIT	DESCRIPTION OF CORE (SOIL: Classification, colour, consistency / density, moisture, plasticity; ROCK: Weathering, colour, fabric, name, strength, cementation)
Fill	CONCRETE: 0.00 - 0.15m
	HYDRO-EXCAVATION (NO RECOVERY): 0.15 - 2.00 m Sandy GRAVEL [INFERRED].
	2.00 - 2.10 m: Clayey SILT with minor gravel; brown. Firm, moist, low plasticity. Gravel: angular, medium.
	2.20 - 3.30 m: Silty CLAY with minor gravel; brown. Firm, moist, medium to high plasticity. Gravel: fine to medium.
	3.30 - 3.60 m: Silty fine SAND; grey. Moist, well graded.
	3.60 - 4.00 m: Silty CLAY; brownish grey. Soft to firm, moist, medium to high plasticity.
	4.00 - 4.50 m: Core loss.

Rock Weathering: UW, EW, HW, MW, SW, CW
Rock Strength: VS, S, MS, M, W, EW
Sampling Method: HVAC (0.15 - 2.00 m); HQTT (2.00 - 3.30 m); HQTT (3.60 - 4.00 m)
Core Recovery (%): 0; 100; 66
Testing
RL (m) / Depth (m): 5 / 4 / 3 / 2 / 1; 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5
Graphic Log
ROCK DEFECTS: Defect Log; Fracture Spacing (mm): 2000, 600, 200, 60, 20; RQD (%); Description & Additional Observations
Fluid Loss (%): 25, 50, 75
Water Level: 05/09/2020
Casing
Installation
Core Box No

COMMENTS: Water level inferred at 2.65m b.g.l. by driller.

Hole Depth 10.5m

Scale 1:25

Rev 0.7

General Log - 4/11/2020 9:21:00 AM - Produced with Core-GS by GeRoc

BH_TT166760

BH_150878

SOURCE: NZGD

2020_BH01

BOREHOLE LOG

BOREHOLE No.: BH01

SHEET: 2 OF 3

Tonkin+Taylor

PROJECT: Downtown Car Park

JOB No.: 1012134.1000

LOCATION: 31 Customs Street West, Auckland

CO-ORDINATES: 5920984.16 mN, 1757327.82 mE (NZTM2000)

DIRECTION:

ANGLE FROM HORIZ.: -90°

R.L. GROUND:

R.L. COLLAR:

DATUM:

SURVEY: GIS\Web map viewer

DRILLED BY: Lei

LOGGED BY: JELE

CHECKED: CRB

START DATE: 05/09/2020

FINISH DATE: 05/09/2020

CONTRACTOR: McMillan Drilling

GEOLOGICAL UNIT	DESCRIPTION OF CORE (SOIL: Classification, colour, consistency / density, moisture, plasticity; ROCK: Weathering, colour, fabric, name, strength, cementation)	Rock Weathering	Rock Strength	Sampling Method	Core Recovery (%)	Testing	RL (m)	Depth (m)	Graphic Log	ROCK DEFECTS: Defect Log	Fracture Spacing (mm)	RQD (%)	Description & Additional Observations	Fluid Loss (%)	Water Level	Casing	Installation	Core Box No
Fill	5.20 - 5.75 m: Silty CLAY with minor sand; grey. Soft to firm, moist, medium to high plasticity. Sand: fine.			HQTT	86		0	5.5										
Fill	5.75 - 6.00 m: Silty CLAY, light grey. Stiff, moist, medium plasticity. *6.00m:* Grades to soft to firm.			HQTT	86		-1	6.0										Box 1, 0.0-6.1m
Fill	6.20 - 6.50 m: Silty CLAY with minor sand; brown. Firm, moist, low to medium plasticity. Sand: fine. *6.50m:* Trace of decomposed wood.			HQTT	86		-1	6.5										Box 1, 0.0-6.1m
East Coast Bays Formation	6.50 - 6.90 m: Core loss.			HQTT	73		-2	7.0										Box 1, 0.0-6.1m
East Coast Bays Formation	6.90 - 7.10 m: Clayey SILT; grey. Very stiff, moist, low plasticity. (Moderately to highly weathered SILTSTONE)			HQTT	73		-2	7.0										Box 1, 0.0-6.1m
East Coast Bays Formation	7.10 - 7.50 m: Slightly to moderately weathered, grey, interbedded SILTSTONE and SANDSTONE. Extremely weak to very weak, weakly cemented, sub-horizontal.			HQTT	73		-2	7.5										Box 1, 0.0-6.1m
East Coast Bays Formation	7.50 - 8.30 m: Slightly to moderately weathered, grey, SANDSTONE. Very weak, weakly cemented, sub-horizontal.			HQTT	73		-3	8.0										Box 1, 0.0-6.1m
East Coast Bays Formation	8.30 - 9.00 m: Unweathered to slightly weathered, grey, SILTSTONE. Very weak, weakly cemented, sub-horizontal.			HQTT	100		-3	8.5										Box 2, 6.1-8.8m
East Coast Bays Formation	9.00 - 10.50 m: Unweathered to slightly weathered, grey, interbedded SILTSTONE and SANDSTONE. Extremely weak to very weak, weakly cemented, sub-horizontal.			HQTT	100		-4	9.0										Box 2, 6.1-8.8m
East Coast Bays Formation				HQTT	100		-4	9.5										Box 2, 6.1-8.8m

COMMENTS: Water level inferred at 2.65m b.g.l. by driller.

Hole Depth 10.5m

Scale 1:25

Rev 0

General Log - 4/11/2020 9:21:01 AM - Produced with Core-GS by GeRoc

BH_TT166760

BH_150878

SOURCE: NZGD

2020_BH01

BOREHOLE LOG

BOREHOLE No.: BH01

SHEET: 3 OF 3

PROJECT: Downtown Car Park

JOB No.: 1012134.1000

LOCATION: 31 Customs Street West, Auckland

CO-ORDINATES: 5920984.16 mN, 1757327.82 mE (NZTM2000)

DIRECTION:

ANGLE FROM HORIZ.: -90°

R.L. GROUND:

R.L. COLLAR:

DATUM:

SURVEY: GIS\Web map viewer

DRILLED BY: Lei

LOGGED BY: JELE

CHECKED: CRB

START DATE: 05/09/2020

FINISH DATE: 05/09/2020

CONTRACTOR: McMillan Drilling

GEOLOGICAL UNIT	DESCRIPTION OF CORE SOIL: Classification, colour, consistency / density, moisture, plasticity ROCK: Weathering, colour, fabric, name, strength, cementation	Rock Weathering (UW, EW, HW, MW, SW)	Rock Strength (VS, S, MS, M, W, VW, EW)	Sampling Method	Core Recovery (%)	Testing	RL (m)	Depth (m)	Graphic Log	ROCK DEFECTS: Defect Log	Fracture Spacing (mm) (2000, 600, 200, 60, 20)	RQD (%)	Description & Additional Observations	Fluid Loss (%) (25, 50, 75)	Water Level	Casing	Installation	Core Box No
East Coast Bays Formation	9.00 - 10.50 m: Unweathered to slightly weathered, grey, interbedded SILTSTONE and SANDSTONE. Extremely weak to very weak, weakly cemented, sub-horizontal.			HQTT	100		-5	10.5										Box 3, 8.8-10.5m
	10.5m: END OF BOREHOLE					12/38 for 75mm **N>=50** Solid	-6	11.0										
								11.5										
							-7	12.0										
								12.5										
							-8	13.0										
								13.5										
							-9	14.0										
								14.5										

COMMENTS: Water level inferred at 2.65m b.g.l. by driller.

Hole Depth 10.5m

Scale 1:25

Rev 0

General Log - 4/11/2020 9:21:01 AM - Produced with Core-GS by GeRoc

BH_TT166760

BH_150878

SOURCE: NZGD

2020_BH01

Tonkin+Taylor

CORE PHOTOS

BOREHOLE No.: **BH01**

PROJECT: Downtown Car Park	LOCATION: 31 Customs Street West, Auckland	JOB No.: 1012134.1000
CO-ORDINATES: 5920984.16 mN (NZTM2000) 1757327.82 mE R.L.: DATUM:	DRILL TYPE: DRILL METHOD: RC	HOLE STARTED: 05/09/2020 HOLE FINISHED: 05/09/2020 DRILLED BY: McMillan Drilling LOGGED BY: JELE CHECKED: CRB



0.00-6.10m



6.10-8.80m

General Log - 4/11/2020 9:21:01 AM - Produced with Core-GS by GeRoc

BH_TT166760

BH_150878

SOURCE: NZGD

2020_BH01

Tonkin+Taylor

CORE PHOTOS

BOREHOLE No.: **BH01**

SHEET: 2 OF 2

PROJECT: Downtown Car Park	LOCATION: 31 Customs Street West, Auckland	JOB No.: 1012134.1000
CO-ORDINATES: 5920984.16 mN (NZTM2000) 1757327.82 mE R.L.: DATUM:	DRILL TYPE: DRILL METHOD: RC	HOLE STARTED: 05/09/2020 HOLE FINISHED: 05/09/2020 DRILLED BY: McMillan Drilling LOGGED BY: JELE CHECKED: CRB



8.80-10.50m

General Log - 4/11/2020 9:21:01 AM - Produced with Core-GS by GeRoc

SOURCE: NZGD

2020_BH02

BOREHOLE LOG

BOREHOLE No.: BH02

SHEET: 1 OF 3

Tonkin+Taylor

PROJECT: Downtown Car Park
JOB No.: 1012134.1000
LOCATION: 31 Customs Street West, Auckland

CO-ORDINATES: 5921037.30 mN, 1757320.80 mE (NZTM2000)
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND:
R.L. COLLAR:
DATUM:
SURVEY: GIS\Web map viewer

DRILLED BY: Andrew. K
LOGGED BY: VELA
CHECKED: CRB
START DATE: 28/08/2020
FINISH DATE: 31/08/2020
CONTRACTOR: McMillan Drilling

GEOLOGICAL UNIT	DESCRIPTION OF CORE SOIL: Classification, colour, consistency / density, moisture, plasticity ROCK: Weathering, colour, fabric, name, strength, cementation	Rock Weathering (UW SW MW HW CW)	Rock Strength (XS VS MS W EW)	Sampling Method	Core Recovery (%)	Testing	RL (m)	Depth (m)	Graphic Log	ROCK DEFECTS: Defect Log	Fracture Spacing (mm) (2000 600 200 60 20)	RQD (%)	Description & Additional Observations	Fluid Loss (%) (25 50 75)	Water Level	Casing	Installation	Core Box No
Fill	CONCRETE: GL - 0.15m																	
	HYDRO-EXCAVATION (NO RECOVERY): 0.15 - 0.75m 0.15 - 0.22m: Dark brownish-black, fine to coarse, asphalt and greywacke GRAVEL with minor greywacke cobbles. 0.20 - 0.40m: Brown SAND with some silt. Moist. Sand is medium. 0.40 - 0.50m: Light brownish-grey CLAY 0.50 - 0.75m: Brown, fine to medium, SAND with sandstone fragments and basalt cobbles (sand is locally weakly cemented)			HVAC	0			0.5										
	SAND, some gravel; brown. Moist, poorly graded; sand, fine to medium; gravel, coarse, subangular, Brick, ceramic and concrete.			HA	100		4	1.0										
	Silty CLAY, minor gravel; brown. Soft to firm, moist, high plasticity; gravel, fine, subangular.							1.5										
	GRAVEL, some silt, minor clay and sand; brown. Moist, well graded; gravel, fine to coarse, subangular to angular, Brick, concrete and scoria; sand, fine.			HQTT	17		3	2.0										
	CORE LOSS: 1.71 - 2.70m (Brick fragment caught in core barrel)							2.5										
	GRAVEL; grey and reddish brown. Poorly graded; gravel, medium to coarse, subangular to angular, Concrete, basalt and scoria.			HQTT	50	● 32/13 kPa	2	3.0								120mm PQ		
	Gravelly CLAY, some silt; dark grey. Soft, moist, high plasticity; gravel, fine to medium, subangular to angular, Brick, basalt and mudstone; Slight organic odour.																	
	CORE LOSS: 3.20 - 3.70m (Concrete caught in core barrel driven into soft clay below)							3.5										
	GRAVEL & COBBLES; dark greyish-black. Poorly graded; gravel, coarse, subangular to angular, Basalt, cobbles, Basalt.			HQTT	30		1	4.0										
	CORE LOSS: 3.85 - 4.20m (Basalt cobble caught in core barrel)																	
	GRAVEL & COBBLES, minor silt; dark greyish black. Poorly graded; gravel, coarse, subangular to angular, Basalt, cobbles, Basalt.			HQTT	54								*4.20 - 4.70m:* Slow drilling progress due to basalt					
	CORE LOSS: 4.47 - 4.70m							4.5										
Marine Sediments	SILT, minor sand and clay; light greenish-grey. Very soft to soft, moist, low plasticity; sand, fine; Inclusions of shell fragments.						0											

COMMENTS: Hydro-excavation undertaken to 0.75m depth. Crew switched to hand auger to between 0.75m and 1.5m depth.Fill encountered between 8.5 - 9.05 m may be buttress fill placed at the time of the old seawall construction.

Hole Depth 11.75m

Scale 1:25

Rev 0

General Log - 4/11/2020 9:21:23 AM - Produced with Core-GS by GeRoc

BH_TT166761

BH_150879

SOURCE: NZGD

2020_BH02

BOREHOLE LOG

BOREHOLE No.: BH02

SHEET: 2 OF 3

Tonkin+Taylor

PROJECT: Downtown Car Park
JOB No.: 1012134.1000
LOCATION: 31 Customs Street West, Auckland

CO-ORDINATES: 5921037.30 mN, 1757320.80 mE (NZTM2000)
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND:
R.L. COLLAR:
DATUM:
SURVEY: GIS\Web map viewer

DRILLED BY: Andrew. K
LOGGED BY: VELA
CHECKED: CRB
START DATE: 28/08/2020
FINISH DATE: 31/08/2020
CONTRACTOR: McMillan Drilling

GEOLOGICAL UNIT	DESCRIPTION OF CORE SOIL: Classification, colour, consistency / density, moisture, plasticity ROCK: Weathering, colour, fabric, name, strength, cementation	Rock Weathering	Rock Strength	Sampling Method	Core Recovery (%)	Testing	RL (m)	Depth (m)	Graphic Log	ROCK DEFECTS: Defect Log	Fracture Spacing (mm)	RQD (%)	Description & Additional Observations	Fluid Loss (%)	Water Level	Casing	Installation	Core Box No
Marine Sediments	Sandy SILT, trace clay; grey. Very soft to soft, moist, dilatant; sand, fine; Abundant inclusions of shell fragments.			HQTT	100	8/2 kPa	-1	5.5										
	Clayey SILT; grey. Soft, moist, medium plasticity.																	
	SILT, minor sand, trace clay; grey. Very soft to soft, moist, dilatant; sand, fine; Inclusions of shell fragments.																	
	CORE LOSS: 5.79 - 6.70m			HQTT	9			6.0										
	Clayey SILT; greyish-brown. Soft, moist, low plasticity.						-2	6.5										
	SILT, minor sand, trace clay; grey. Very soft to soft, wet, dilatant; sand, fine; Inclusions of shell fragments.																	
	CORE LOSS: 7.10 - 7.70m			HQTT	40	0/0 kPa Sample turning in core barrel		7.0										Box 1, 0.0-7.7m
	SILT, some clay, minor sand; grey. Very soft to soft, wet; sand, fine; Inclusions of shell fragments.							7.5										
	Silty SAND, trace clay; grey. Moist, dilatant. sand, fine; Inclusions of shell fragments.			HQTT	80		-3	8.0										
	Clayey SILT; grey. Soft, moist, low plasticity.																	
	CORE LOSS: 8.35 - 8.53m																	
Fill	Gravelly COBBLES; dark grey and red. Loose, moist, poorly graded; cobbles, highly weathered to moderately weathered, basalt; gravel, medium to coarse, subangular, highly weathered to moderately weathered, basalt.			HQTT	58		-4	8.5										
	CORE LOSS: 8.65 - 8.75																	
Marine Sediments	Gravelly COBBLES (as above)							9.0										
	CORE LOSS: 9.05 - 9.50																	
	Clayey SILT, minor sand; grey. Very soft, wet, non-plastic; sand, fine to coarse, shell and volcanic.			HQTT	35		-5	9.5										
	CORE LOSS: 9.70 - 9.85																	
	Clayey SILT, minor sand (as above)																	

COMMENTS: Hydro-excavation undertaken to 0.75m depth. Crew switched to hand auger to between 0.75m and 1.5m depth.Fill encountered between 8.5 - 9.05 m may be buttress fill placed at the time of the old seawall construction.

Hole Depth 11.75m

Scale (A4) 1:25

File (A4)

General Log - 4/11/2020 9:21:24 AM - Produced with Core-GS by GeRoc

BH_TT166761

BH_150879

SOURCE: NZGD

2020_BH02

BOREHOLE LOG

BOREHOLE No.: BH02

SHEET: 3 OF 3

Tonkin+Taylor

PROJECT: Downtown Car Park
JOB No.: 1012134.1000
LOCATION: 31 Customs Street West, Auckland

CO-ORDINATES: 5921037.30 mN, 1757320.80 mE (NZTM2000)
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND:
R.L. COLLAR:
DATUM:
SURVEY: GIS\Web map viewer

DRILLED BY: Andrew. K
LOGGED BY: VELA
CHECKED: CRB
START DATE: 28/08/2020
FINISH DATE: 31/08/2020
CONTRACTOR: McMillan Drilling

GEOLOGICAL UNIT	DESCRIPTION OF CORE (SOIL: Classification, colour, consistency / density, moisture, plasticity; ROCK: Weathering, colour, fabric, name, strength, cementation)	Rock Weathering	Rock Strength	Sampling Method	Core Recovery (%)	Testing	RL (m)	Depth (m)	Graphic Log	ROCK DEFECTS: Defect Log	Fracture Spacing (mm)	RQD (%)	Description & Additional Observations	Fluid Loss (%)	Water Level	Casing	Installation	Core Box No
East Coast Bays Formation	Highly weathered, grey, SILTSTONE. Extremely Weak, Recovered as (SILT with minor clay and trace sand, grey. Stiff to very stiff, wet to moist, low plasticity. Sand; fine.).			HQTT	31		-6	10.5										
	CORE LOSS: 10.15 - 10.70 (Due to basalt cobble being caught in barrel.																	
	Moderately weathered, grey, SILTSTONE. Extremely Weak, Recovered as (SILT with some sand and minor clay, grey. Very stiff, moist, non plastic. Sand; fine.).			HQTT	100							0						
	Slightly weathered, grey speckled white, SANDSTONE. Extremely Weak, sub-horizontal, moderately cemented, Recovered as (SAND, with minor silt and clay, grey speckled white. Dense, moist, poorly graded.).			HQTT	100			11.0				95	11.00m: DD 11.12m: DD 11.16m: DD 11.21m: J, 15° dip, PL, SM, VN 11.25m: BF, 0° dip, PL, SM 11.37m: DD 11.43m: J, 25° dip, PL, R, VN, VE(Silt) 11.62m: DD					Box 2, 7.7-11.8m
	Unweathered, grey, SANDSTONE. Very Weak, sub-horizontal. - INTERBEDDED WITH - Unweathered, grey, SILTSTONE. Very Weak, sub-horizontal, moderately thick.						-7	11.5										
	11.75m: END OF BOREHOLE						-8	12.0 12.5 13.0										
							-9	13.5 14.0										
							-10	14.5										

COMMENTS: Hydro-excavation undertaken to 0.75m depth. Crew switched to hand auger to between 0.75m and 1.5m depth.Fill encountered between 8.5 - 9.05 m may be buttress fill placed at the time of the old seawall construction.

Hole Depth 11.75m

Scale 1:25

Rev 0.7

General Log - 4/11/2020 9:21:24 AM - Produced with Core-GS by GeRoc

BH_TT166761

BH_150879

SOURCE: NZGD

2020_BH02

Tonkin+Taylor

CORE PHOTOS

BOREHOLE No.: **BH02**

SHEET: 1 OF 1

PROJECT: Downtown Car Park	LOCATION: 31 Customs Street West, Auckland	JOB No.: 1012134.1000
CO-ORDINATES: 5921037.30 mN (NZTM2000) 1757320.80 mE R.L.: DATUM:	DRILL TYPE: Portable DRILL METHOD: RC	HOLE STARTED: 28/08/2020 HOLE FINISHED: 31/08/2020 DRILLED BY: McMillan Drilling LOGGED BY: VELA CHECKED: CRB



0.00-7.70m



7.70-11.75m

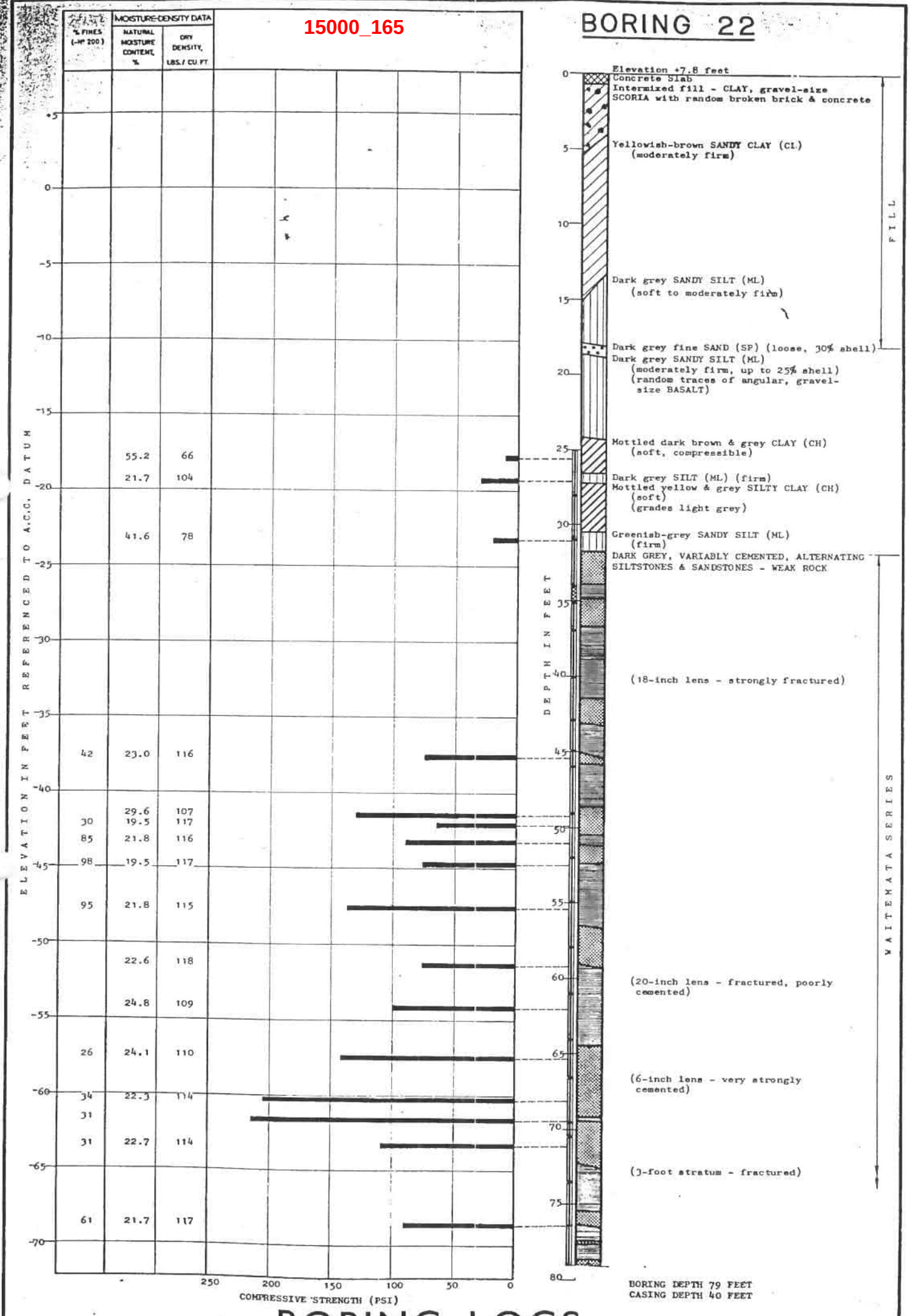
General Log - 4/11/2020 9:21:24 AM - Produced with Core-GS by GeRoc

BH_TT166761

BH_150879

15000_165

BORING 22



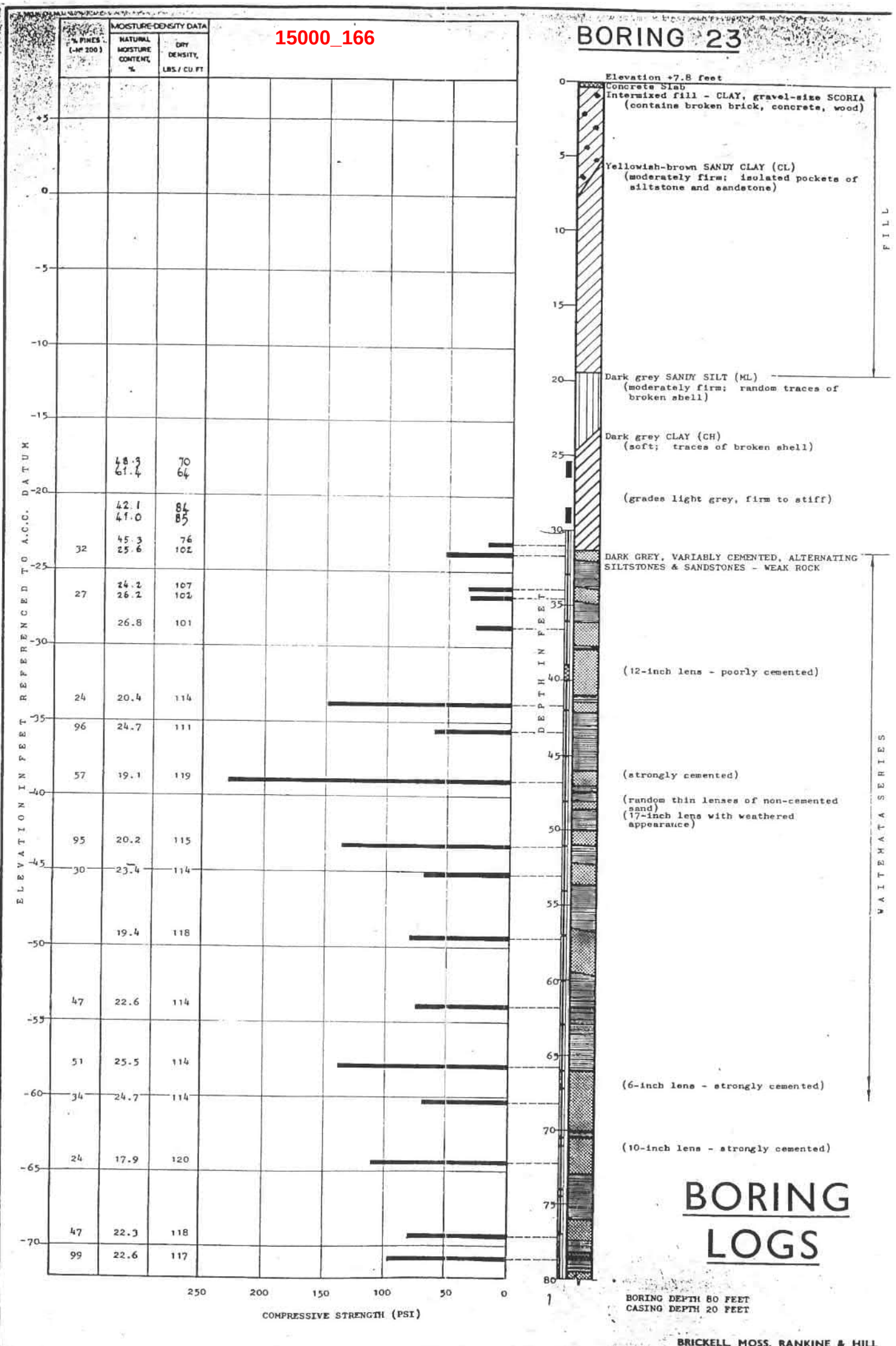
BORING DEPTH 79 FEET
CASING DEPTH 40 FEET

BORING LOGS

BRICKELL, MOSS, RANKINE & HILL

15000_166

BORING 23



BORING LOGS

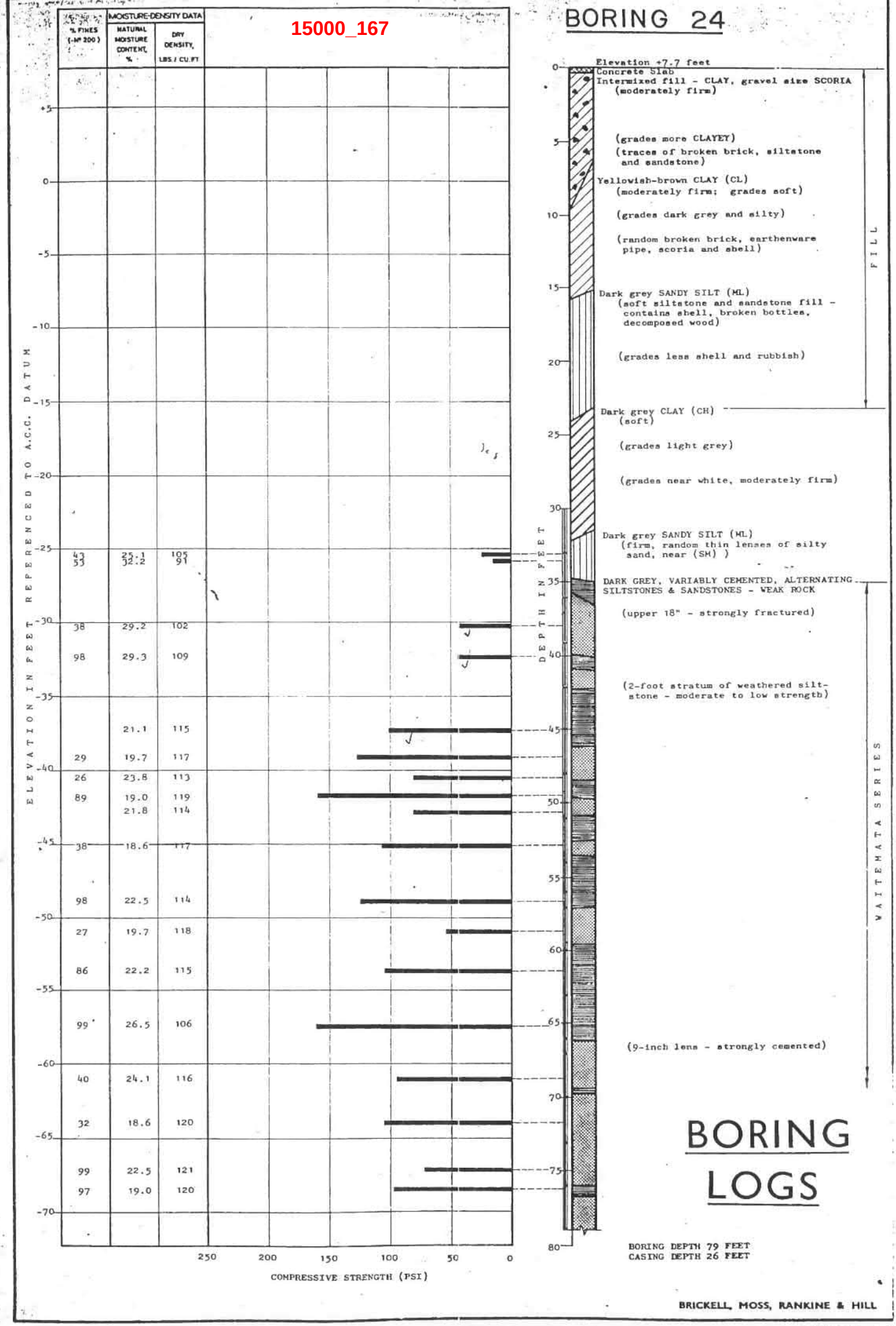
BORING DEPTH 80 FEET
CASING DEPTH 20 FEET

BRICKELL, MOSS, RANKINE & HILL

PLATE B-21

15000_167

BORING 24



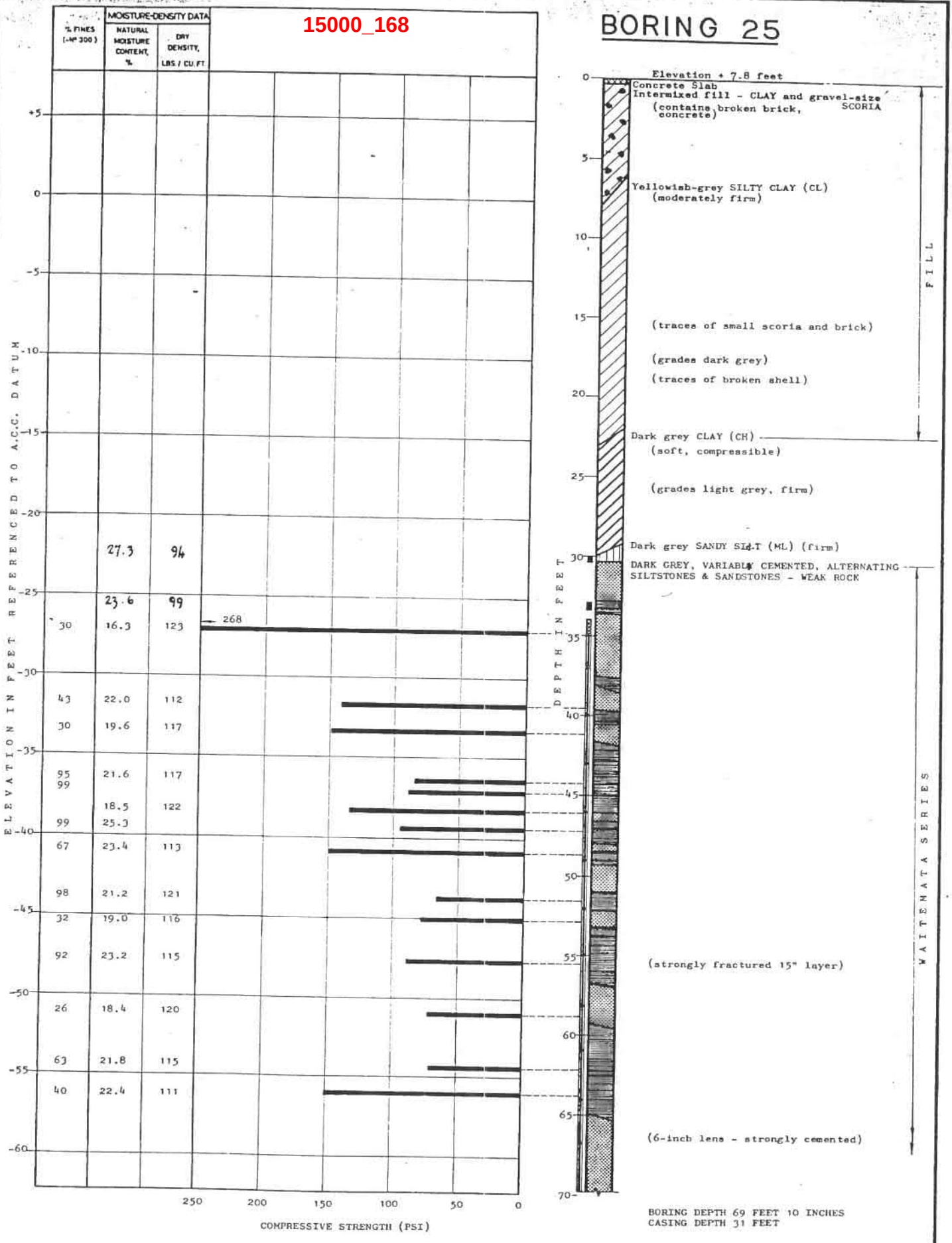
BORING LOGS

BORING DEPTH 79 FEET
CASING DEPTH 26 FEET

BRICKELL, MOSS, RANKINE & HILL

15000_168

BORING 25



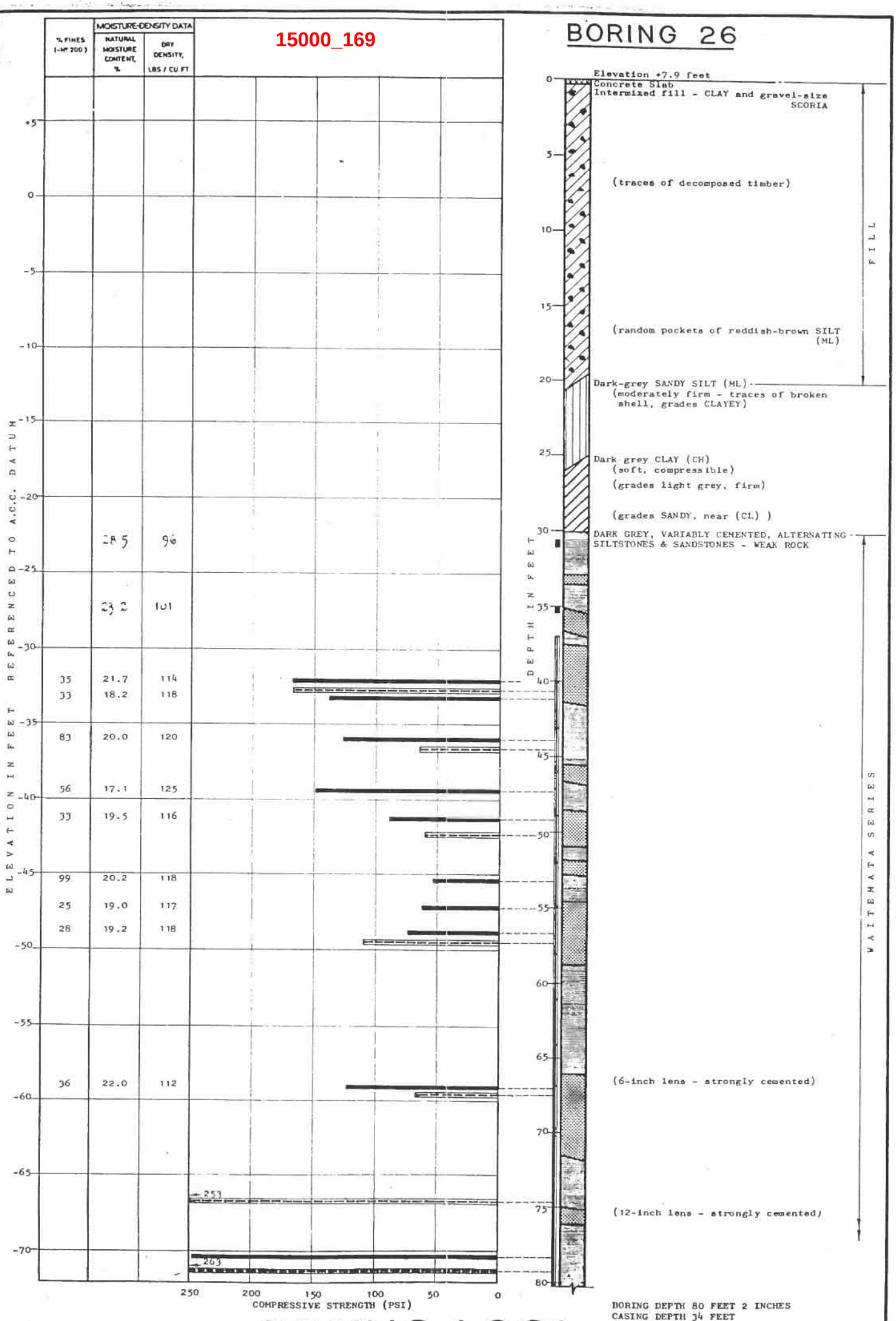
BORING DEPTH 69 FEET 10 INCHES
CASING DEPTH 31 FEET

BORING LOGS

BRICKELL, MOSS, RANKINE & HILL

15000_169

BORING 26



BORING DEPTH 80 FEET 2 INCHES
CASING DEPTH 34 FEET

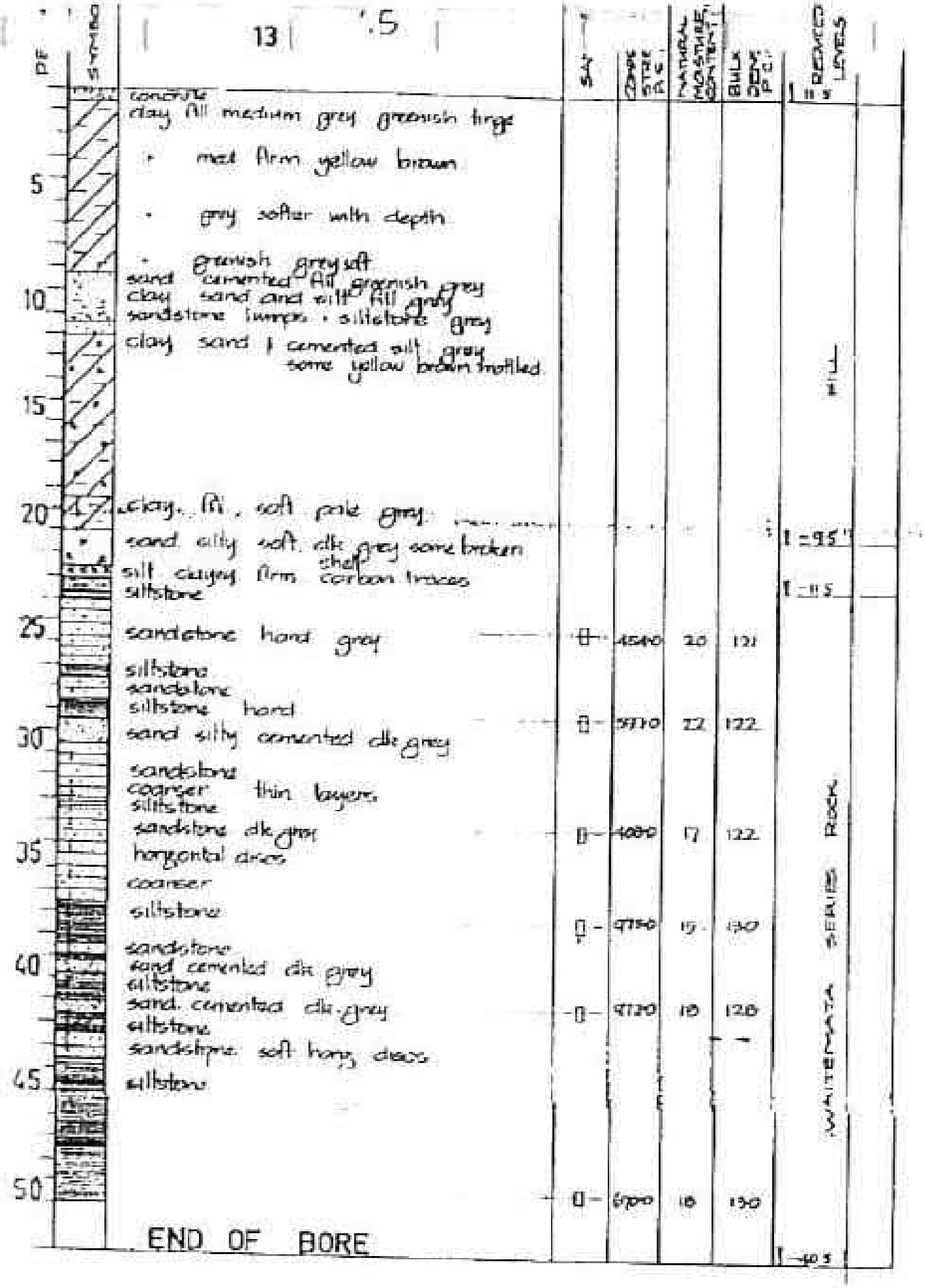
BORING LOGS

BRICKELL, MOSS, RANKINE & HILL

801_BH13

BH_TT66666

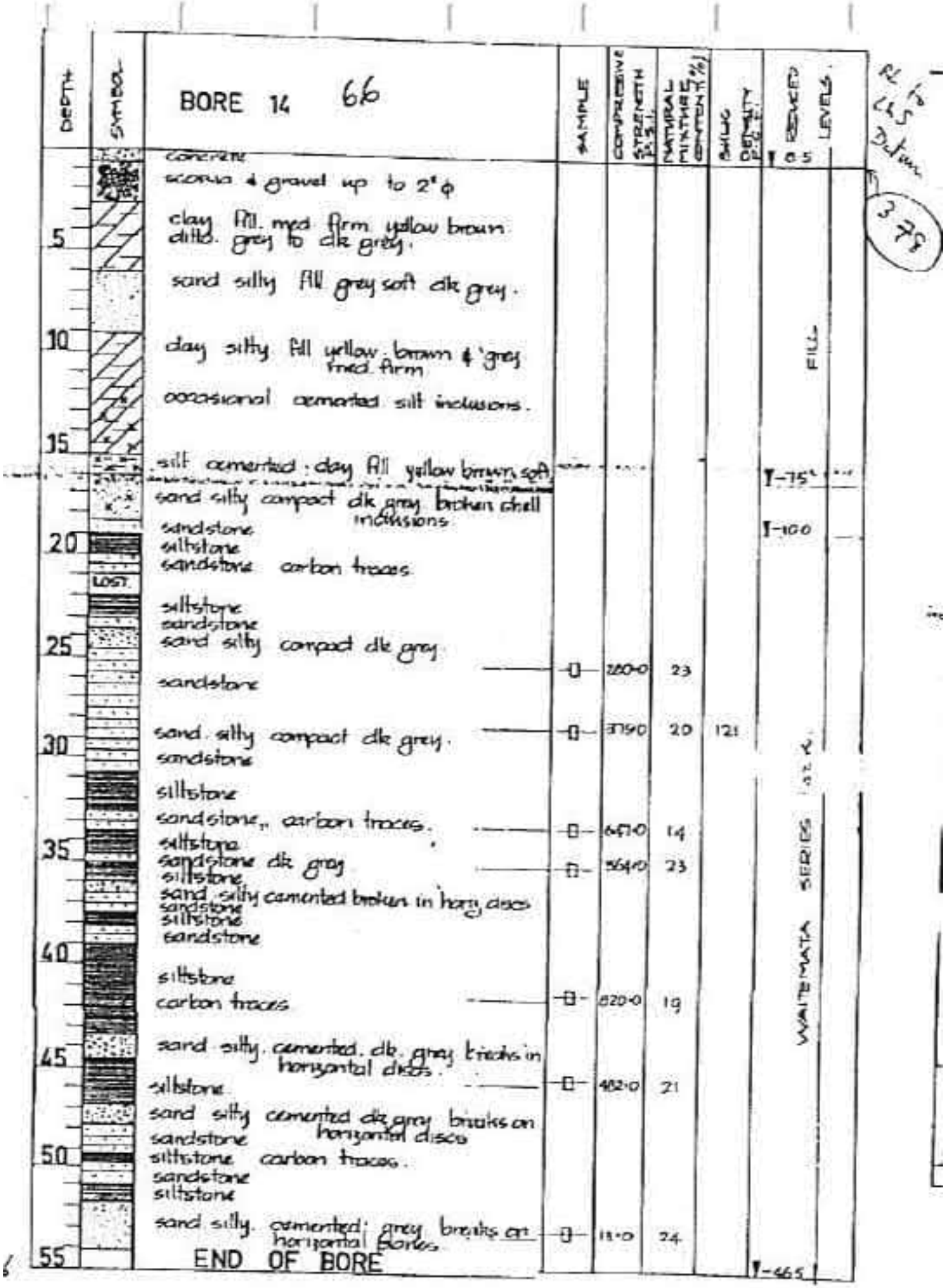
BH_64336



801_BH14

BH_TT66667

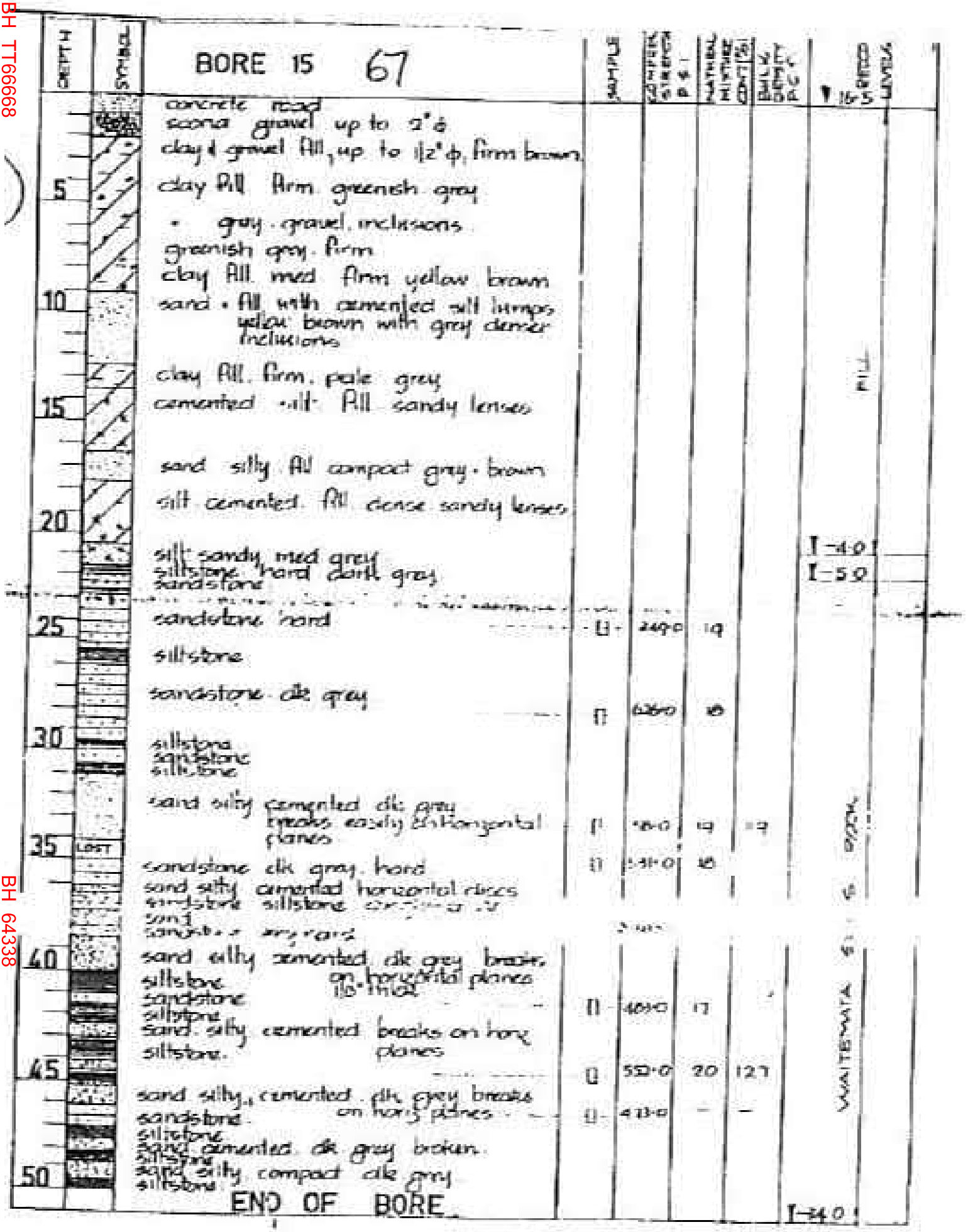
BH_64337



801_BH15

BH_TT66668

BH_64338



BH_64274

BH_TT66604

RECORD OF BOREHOLE 211

SHEET 1 OF 3 | BH211

JOB NAME : Quay Street West Underpass

CLIENT : Auckland City Council

JOB NO. : 2409275

LOCATION : Lower Hobson Street

COORDS : 6482788.40N, 2667722.90E

ELEV.: 3.59 DATUM: LANDS & SURVEY

DEPTH (m)	LGD	SYM	STRATA DESCRIPTION	ELEVATN (metres)	SAMPLE DEPTH (m)	SAMPLE TYPE	FIELD TESTS SPT	FIELD TESTS VANE	FIELD TESTS OTHER
			0.00 Asphaltic Concrete	3.59	0.00				DB
			0.15 Medium dense grey GRAVEL,moderately weathered,moderately strong,subangular Basalt [FILL]	3.44					
			0.80 Loose reddish brown sandy fine to medium GRAVEL,moderately weathered,moderately strong vesicular Basalt [FILL]	2.79					
1			1.50 — becoming brown	2.09					
2			2.50 Firm orange brown mottled grey/black/white SILT-CLAY,some fine sand,trace shells,moist,highly plastic, intermixed with fine to medium gravel sized silt clasts and grey SAND,some silt,wet,non plastic [FILL]	1.09	2.50	‡			
3			3.00 — becoming grey	0.59	3.00	●	1 6 4 10	C = 44 CR= 10	
					3.50	‡			
4			4.00 — with clasts of coarse gravel sized SILTSTONE and SANDSTONE	-0.41	4.00	‡			
					4.50	●	1 1 1 2	C = 30 CR= 8	

OBSERVATIONS:

SAMPLES
- ● Small disturbed sample
- ‡ Large disturbed sample
- ▮ Undisturbed tube sample
- ▯ Undisturbed core sample
- ↓ SPT Split spoon sample

FIELD TESTS
- SPT = Standard Penetration Test (blows/150mm, N=blows/300mm)
- C = Cohesion as measured direct with shear vane (kPa)
- CR = Remoulded C
- CC = Corrected Reading (kPa)

PILCON VANE DIAL NO.: DR3969

DATE STARTED : 5-Apr-98 CORE DIA.: 90 mm RIG : Trailer

DATE FINISHED: 5-Apr-98 LOGGED BY: M.Watson CONTRACTOR: Prodrill

BECA CARTER HOLLINGS AND FERNER LTD. Ph: (09) 3773 410

BH_64274

BH_64274

BH_TT66604

BH_64274

RECORD OF BOREHOLE 211

SHEET 2 OF 3 | BH211

JOB NAME : Quay Street West Underpass

CLIENT : Auckland City Council

JOB NO. : 2409275

LOCATION : Lower Hobson Street

COORDS : 6482788.40N, 2667722.90E

ELEV.: 3.59 DATUM: LANDS & SURVEY

DEPTH(m)	LGD	SYM	STRATA DESCRIPTION	ELEVATN (meters)	SAMPLE DEPTH(m)	SAMPLE TYPE	FIELD TESTS SPT	FIELD TESTS VANE	FIELD TESTS OTHER
						~			
					5.50	‡		C = 40 CR= 18	
			5.60 — with trace wood	-2.01					
			5.80 — Very loose dark grey SAND,minor fine gravel sized shell fragments;saturated,non plastic, intermixed with orange brown mottled white SILT-CLAY,trace fine sand,moist,highly	-2.21					
6			6.00 — plastic [TAURANGA GROUP SEDIMENTS(Upper)]	-2.41	6.00		1	C = 16	OB
			No recovery				1		
							1		
							2		
			6.50 — No recovery, washings indicate grey silty SAND	-2.91					
7									
			7.50 — Soft grey SILT-CLAY,some fine sand,moist,highly plastic [TAURANGA GROUP SEDIMENTS(Upper)]	-3.91	7.50	●	0		OB
							0		
							0		
							0		
8			9.00 — Loose grey greenish brown SAND,trace fine gravel sized shells;moist,non plastic [TAURANGA GROUP SEDIMENTS(Upper)]	-4.41	8.00	‡			
			8.50 — Firm light grey mottled green and dark grey speckled white SILT,minor fine sand,minor clay,trace rootlets;moist, moderately plastic [TAURANGA GROUP SEDIMENTS(Lower)]	-4.91	8.50	‡		C = 26	
			8.80 — Dense light grey mottled orange and speckled white faintly bedded silty SAND,minor clay;moist,non plastic [WAITEMATA GROUP]	-5.21					
9			9.00 — very stiff	-5.41	9.00	●	1	C = 130	+
							3		
							4		
							7		
			9.60 — becoming dark grey	-6.01					
					9.80	‡			
			9.95 — bedding disturbed,grey layer of 10mm moderately weathered,extremely weak SILTSTONE	-6.36					

OBSERVATIONS:

SAMPLES

- ● Small disturbed sample
- ‡ Large disturbed sample
- ▮ Undisturbed tube sample
- ▯ Undisturbed core sample
- ↓ SPT Split spoon sample

FIELD TESTS

- SPT = Standard Penetration Test (blows/150mm,N=blows/300mm)
- C = Cohesion as measured direct with shear vane (kPa)
- CR = Remoulded C
- CC = Corrected Reading (kPa)

PILCON VANE DIAL NO.: DR3969

DATE STARTED : 5-Apr-98 CORE DIA.: 90 mm RIG : Trailer

DATE FINISHED: 5-Apr-98 LOGGED BY: M.Watson CONTRACTOR: Prodrill

BECA CARTER HOLLINGS AND FERNER LTD. Ph: (09) 3773 410

BH_64274

BH_TT66604

BH_64274

RECORD OF BOREHOLE 211

SHEET 3 OF 3 | BH211

JOB NAME : Quay Street West Underpass

CLIENT : Auckland City Council

JOB NO. : 2409275

LOCATION : Lower Hobson Street

COORDS : 6482788.40N, 2667722.90E

ELEV.: 3.59 DATUM: LANDS & SURVEY

DEPTH (m)	LGD	SYM	STRATA DESCRIPTION	ELEVATN (metres)	SAMPLE DEPTH (m)	SAMPLE TYPE	FIELD TESTS SPT	FIELD TESTS VANE	FIELD TESTS OTHER	
			10.00 wavy orange SILT-CLAY seam 3mm, soft sediment deformation?	-6.41						
			10.20 with 10mm seam, subhorizontal light grey mottled orange SILT-CLAY; moist, highly plastic	-6.61	10.30		25		TT	SPT=25/80mm
			10.50 Grey moderately weathered, extremely weak SANDSTONE [WAITEMATA GROUP]	-6.91			26			
							25			
							51+			
11			11.00 Grey moderately weathered, extremely weak SILTSTONE [WAITEMATA GROUP]	-7.41	11.00					
			11.40 Grey moderately weathered, extremely weak SANDSTONE [WAITEMATA GROUP]	-7.81						
			11.60 becoming slightly weathered, very weak	-8.01	11.50					
			11.90 with subhorizontal laminated layers of brown carbonaceous material	-8.31						
12					12.00		50			SPT=50/150
							50+			
					12.50					
			12.85 Grey slightly weathered, very weak SANDSTONE [WAITEMATA GROUP]	-9.26						
13			12.95	-9.36						
			13.00	-9.41	13.00					
			13.10 Grey slightly weathered, very weak SILTSTONE [WAITEMATA GROUP]	-9.51						
			13.20 becoming extremely weak	-9.61						
			13.40 Grey slightly weathered, very weak SANDSTONE	-9.81						
			becoming weak		13.50		50			SPT=50/90mm
			Grey slightly weathered, very weak SILTSTONE [WAITEMATA GROUP]				50+			
			END OF BOREHOLE 211 . 13.59m BELOW GROUND SURFACE							
14										

OBSERVATIONS:

SAMPLES
- Small disturbed sample
- Large disturbed sample
- Undisturbed tube sample
- Undisturbed core sample
- SPT Split spoon sample

FIELD TESTS
- SPT = Standard Penetration Test (blows/150mm, N=blows/300mm)
- C = Cohesion as measured direct with shear vane (kPa)
- CR = Remoulded C
- CC = Corrected Reading (kPa)

PILCON VANE DIAL NO.: DR3969

DATE STARTED : 5-Apr-98 CORE DIA.: 90 mm RIG : Trailer

DATE FINISHED: 5-Apr-98 LOGGED BY: M.Watson CONTRACTOR: Prodrill

BECA CARTER HOLLINGS AND FERNER LTD. Ph: (09) 3773 410

15000_170

BH_64276

BH_TT66606

BH_64276

RECORD OF BOREHOLE 213

SHEET 1 OF 3 | BH213

JOB NAME : Quay Street West Underpass

CLIENT : Auckland City Council

JOB NO. : 2409275

LOCATION : Lower Hobson St

COORDS : 6482721.30N, 2667672.10E

ELEV.: 3.54 DATUM: LANDS & SURVEY

DEPTH (m)	LGD	SYM	STRATA DESCRIPTION	ELEVATN (metres)	SAMPLE DEPTH(m)	SAMPLE TYPE	FIELD TESTS SPT	FIELD TESTS VANE	FIELD TESTS OTHER
			0.00 Asphaltic Concrete	3.54					
			0.10 Brown GRAVEL,some sand,minor silt,trace clay,moist,non plastic. Gravel,brown grey moderately weathered,moderately strong subangular Basalt	3.44					
			0.80 with red brown scoriaceous Basalt	2.74					
1			0.95 with some reddish brown silty SAND	2.59					
			1.00	2.54	1.00	↕			GB
			1.10 Loose brown silty sandy GRAVEL,minor clay,moist,moderately plastic	2.44					
			Firm orange brown SAND,some silt,minor clay,minor fine to medium gravel,moist,non plastic Gravel,grey,moderately weathered,very weak to extremely weak SILTSTONE						
			1.35	2.19					
			1.45	2.09					
			Soft orange mottled white,red/orange and grey SILT-CLAY,minor fine sand,moist,highly plastic		1.50	▮		C = 36 CR= 24	
			with some orange brown fine sand,wet [FILL]						
2					2.00	↓	0		
							1		
			2.35 with a moderately thick layer of reddish brown highly weathered, extremely weak SANDSTONE	1.19			1/2		
			2.55 with some coarse GRAVEL,grey mottled orange and orange brown moderately to highly weathered,very weak to extremely weak SILTSTONE, with carbonaceous flecks [FILL]	0.99	2.50	↕			
			2.85	0.69					
3			Loose bluish grey mottled black and greenish brown/grey silty fine SAND,minor clay,moist,moderately plastic [FILL]		3.00	▮		C = 70 CR= 14	
			3.45 bottom of tube contains grey fine SAND,moist,non plastic,	0.09					
			3.50 intermixed with reddish brown highly weathered extremely weak SANDSTONE and stiff greenish brownish grey SILT-CLAY,moist,highly plastic.	0.04	3.50	↓	3		
							3		
			Firm greenish brownish grey highly weathered,very weak SANDSTONE and reddish brown highly weathered,extremely weak SANDSTONE [FILL]				4/7		
4			4.00	-0.46	4.00	↕			
			with a thin layer of soft grey mottled white and orange brown SILT-CLAY,some gravel,moist,highly plastic Gravel,coarse,grey,moderately weathered,very weak SILTSTONE						
			4.40	-0.86					
			Firm grey mottled reddish orange and brown moderately weathered extremely weak SANDSTONE [FILL]		4.50	↓	1		THP
							2		
							2/4		

OBSERVATIONS:
THP=Too Hard to Penetrate with shear vane

SAMPLES
- ● Small disturbed sample
- ↕ Large disturbed sample
- ▮ Undisturbed tube sample
- ▯ Undisturbed core sample
- ↓ SPT Split spoon sample

FIELD TESTS
- SPT = Standard Penetration Test (blows/150mm,N=blows/300mm)
- C = Cohesion as measured direct with shear vane (kPa)
- CR = Remoulded C
- CC = Corrected Reading (kPa)

PILCON VANE DIAL NO.: DR2122

DATE STARTED : 21-Jan-98 CORE DIA.: 90 mm RIG : Trailer Mounted

DATE FINISHED: 21-Jan-98 LOGGED BY: M.Watson CONTRACTOR: Prodrill

BECA CARTER HOLLINGS AND FERNER LTD. Ph: (09) 3773 410

BH_64276

BH_TT66606

BH_64276

RECORD OF BOREHOLE 213

SHEET 2 OF 3 | BH213

JOB NAME : Quay Street West Underpass

CLIENT : Auckland City Council

JOB NO. : 2409275

LOCATION : Lower Hobson St

COORDS : 6482721.30N, 2667672.10E

ELEV.: 3.54 DATUM: LANDS & SURVEY

DEPTH (m)	LGD	SYM	STRATA DESCRIPTION	ELEVATN (metres)	SAMPLE DEPTH (m)	SAMPLE TYPE	FIELD TESTS SPT	FIELD TESTS VANE	FIELD TESTS OTHER
					5.00	‡			
			5.45 Grey coarse GRAVEL,moderately weathered weak SILTSTONE	-1.91					
			5.50 Loose grey speckled white fine SAND,some silt,minor clay,minor fine gravel,moist,non plastic. Gravel,shells. [TAURANGA GROUP SEDIMENTS(Upper)]	-1.96	5.50	‡			
6					6.00	▮		C = 38 CR= 14	
			6.50 bottom of tube contains stiff grey SILT-CLAY,moist,highly plastic	-2.96	6.50	↓	3		
			6.65 Medium dense grey fine SAND,wet,non plastic,intermixed with grey SILT-CLAY,wet,highly plastic [TAURANGA GROUP SEDIMENTS(Upper)]	-3.11			4		
							6		
							10		
7			7.00 Medium dense grey fine to medium SAND,wet,non plastic. [TAURANGA GROUP SEDIMENTS(Upper)]	-3.46	7.00	‡			
			7.35 Hard grey SILT-CLAY,wet,highly plastic [WAITEMATA GROUP]	-3.81					
			7.50 Medium dense grey silty fine SAND,minor clay,wet,moderately plastic, intermixed with grey SILT-CLAY,wet,highly plastic [WAITEMATA GROUP]	-3.96	7.50	↓	6		THP
							8		
							12		
							20		
			7.95 with some fine gravel sized wood fragments	-4.41					
8			8.00 Dense grey fine SAND,some silt,trace clay,wet,non plastic [WAITEMATA GROUP]	-4.46	8.00	‡			
					8.50	‡			
9			9.00 Intermixed dense grey fine SAND,some silt,trace clay,wet,non plastic with very stiff grey SILT-CLAY,wet,highly plastic [WAITEMATA GROUP]	-5.46	9.00	‡	5		
							15		
							40		
							55		
					9.50				TT

OBSERVATIONS:
THP=Too Hard to Penetrate with shear vane

SAMPLES
- ● Small disturbed sample
- ‡ Large disturbed sample
- ▮ Undisturbed tube sample
- ▯ Undisturbed core sample
- ↓ SPT Split spoon sample

FIELD TESTS
- SPT = Standard Penetration Test (blows/150mm,N=blows/300mm)
- C = Cohesion as measured direct with shear vane (kPa)
- CR = Remoulded C
- CC = Corrected Reading (kPa)

PILCON VANE DIAL NO.: DR2122

DATE STARTED : 21-Jan-98 CORE DIA.: 90 mm RIG : Trailer Mounted

DATE FINISHED: 21-Jan-98 LOGGED BY: M.Watson CONTRACTOR: Prodrill

BECA CARTER HOLLINGS AND FERNER LTD. Ph: (09) 3773 410

BH_64276

BH_TT66606

BH_64276

RECORD OF BOREHOLE 213

SHEET 3 OF 3 | BH213

JOB NAME : Quay Street West Underpass

CLIENT : Auckland City Council

JOB NO. : 2409275

LOCATION : Lower Hobson St

COORDS : 6482721.30N, 2667672.10E

ELEV.: 3.54 DATUM: LANDS & SURVEY

DEPTH (m)	LGD	SYM	STRATA DESCRIPTION	ELEVATN (metres)	SAMPLE DEPTH(m)	SAMPLE TYPE	FIELD TESTS SPT	FIELD TESTS VANE	FIELD TESTS OTHER
			10.20 Moderately thickly interbedded with grey slightly weathered very weak SILTSTONE and grey moderately weathered, extremely weak SANDSTONE [WAITEMATA GROUP]	-6.46	10.00				
					10.50		28		SPT=22 95mm
							22 50+		
11					11.00				
					11.50				
			11.70 — laminated with carbonaceous material	-8.16					
12					12.00		50 50+		SPT=50/125mm
			END OF BOREHOLE 213 , 12.13m BELOW GROUND SURFACE						
13									
14									

OBSERVATIONS:
THP=Too Hard to Penetrate with shear vane

SAMPLES
- Small disturbed sample
- Large disturbed sample
- Undisturbed tube sample
- Undisturbed core sample
- SPT Split spoon sample

FIELD TESTS
- SPT = Standard Penetration Test (blows/150mm, N=blows/300mm)
- C = Cohesion as measured direct with shear vane (kPa)
- CR = Remoulded C
- CC = Corrected Reading (kPa)

PILCON VANE DIAL NO.: DR2122

DATE STARTED : 21-Jan-98 CORE DIA.: 90 mm RIG : Trailer Mounted

DATE FINISHED: 21-Jan-98 LOGGED BY: M.Watson CONTRACTOR: Prodrill

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RECORD OF BOREHOLE 214

SHEET 1 OF 3 | BH214

JOB NAME : Quay Street West Underpass

CLIENT : Auckland City Council

JOB NO. : 2409275

LOCATION : Lower Hobson St

COORDS : 6482760.10N, 2667686.50E

ELEV.: 3.59 DATUM: LANDS & SURVEY

DEPTH (m)	LGD	SYM	STRATA DESCRIPTION	ELEVATN (meters)	SAMPLE DEPTH (m)	SAMPLE TYPE	FIELD TESTS SPT	FIELD TESTS VANE	FIELD TESTS OTHER
			0.00 Asphaltic Concrete	3.59					
			0.15 Grey fine to coarse GRAVEL;moderately weathered,moderately strong,sub angular Basalt	3.44					
					0.50	↕			
			0.70	2.89	0.70				OB
			0.80 Dark grey gravelly SILT-CLAY,minor sand,wet,highly plastic,organic odour present. Gravel,grey,moderately weathered,moderately strong,sub angular Basalt.	2.79	0.80			C = 70 CR= 40	
1			1.00 Orange brown sandy GRAVEL,some silt, minor clay;wet,non plastic. Gravel,reddish brown highly weathered weak basalt. [FILL]	2.59	1.00	↕			
			Soft to very stiff orange brown mottled grey and orange fine sandy SILT-CLAY intermixed with grey moderately weathered extremely weak SILTSTONE and orange grey highly weathered extremely weak SANDSTONE. [FILL]						
			1.50 with mottles of white	2.09	1.50	↓	0	C = 100	+
							0		
							1		
2							1		
			2.40 Soft to firm orange brown mottled white,grey and brown SILT-CLAY with clasts of fine gravel sized moderately weathered,very weak SILTSTONE. [FILL]	1.19	2.40	↕			
					2.50			C = 40 CR= 8	
			2.85 Very stiff to hard grey mottled brown grey and orange highly weathered,extremely weak SILTSTONE [FILL]	0.74					
3			3.00 with a thin layer of brown sandy fine GRAVEL,wet,non plastic	0.59 0.49	3.00	↓	1		THP
			3.10 with a thin layer of black silty SAND,minor clay,wet,moderately plastic, with a thin layer of medium gravel sized red brick				1		
							1		
							2		
			Soft grey mottled orange brown SILT-CLAY,some fine sand,some fine to medium gravel,wet,highly plastic Gravel,orange brownish grey,moderately weathered,very weak SANDSTONE and grey moderately weathered weak SILTSTONE.						
4			4.00 with a thin layer of soft dark grey silty SAND,minor clay;wet,moderately plastic, and with a lense of stiff brown SILT-CLAY,trace fibrous material,moist,highly plastic	-0.41	4.00	↕			
			4.15	-0.56					
			Stiff grey mottled orange brownish grey intermixed grey moderately weathered very weak SILTSTONE and brownish grey mottled orange highly weathered,very weak SANDSTONE [FILL]		4.50	↓	3		THP
							4		
							5		
							9		

OBSERVATIONS:
THP=Too Hard to Penetrate with shear vane

SAMPLES
- ● Small disturbed sample
- ↕ Large disturbed sample
- ▮ Undisturbed tube sample
- ▯ Undisturbed core sample
- ↓ SPT Split spoon sample

FIELD TESTS
- SPT = Standard Penetration Test (blows/150mm,N=blows/300mm)
- C = Cohesion as measured direct with shear vane (kPa)
- CR = Remoulded C
- CC = Corrected Reading (kPa)

PILCON VANE DIAL NO.: DR2122

DATE STARTED : 20-Jan-98 CORE DIA.: 90 mm RIG : Trailer Mounted

DATE FINISHED: 20-Jan-98 LOGGED BY: M.Watson CONTRACTOR: Prodrill

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BH_64277

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BH_64277

RECORD OF BOREHOLE 214

SHEET 2 OF 3 | BH214

JOB NAME : Quay Street West Underpass

CLIENT : Auckland City Council

JOB NO. : 2409275

LOCATION : Lower Hobson St

COORDS : 6482760.10N, 2667686.50E

ELEV.: 3.59 DATUM: LANDS & SURVEY

DEPTH(m)	LGD	SYM	STRATA DESCRIPTION	ELEVATN (meters)	SAMPLE DEPTH(m)	SAMPLE TYPE	FIELD TESTS SPT	FIELD TESTS VANE	FIELD TESTS OTHER
					-				
			5.50 No recovery	-1.91	5.50	↕			
			5.65 Hard grey moderately weathered, very weak SANDSTONE with a thin layer of brownish orange highly weathered extremely weak SANDSTONE. [FILL]	-2.06					
			5.75	-2.16					
			5.85	-2.21					
			5.90 with a thin layer of brownish orange SANDSTONE	-2.31					
6			6.00 Hard grey moderately weathered very weak SILTSTONE	-2.41	6.00	↓	0		
			Medium dense dark grey SAND, some silt, minor clay, minor fine gravel; wet, non plastic. Gravel; shell fragments. [FILL]				1		
							1/2		
			6.50 Loose grey SAND, minor fine to medium gravel, wet, non plastic intermixed with soft grey SILT-CLAY, trace fine sand, wet, highly plastic. Gravel; shell fragments and wood [FILL]	-2.91	6.50	▮			
			No recovery in tube						
7			7.00 Loose grey SAND, minor fine to medium gravel, moist, non plastic, intermixed with soft grey SILT-CLAY, trace fine sand, moist, highly plastic. Gravel, shell fragments. [TAURANGA GROUP SEDIMENTS (Upper)]	-3.41	7.00	↕		C = 12 CR= 6	
			7.15 with a thin layer of very stiff orange brown SILT-CLAY, trace fine sand, moist, highly plastic	-3.76					
			7.40	-3.81					
			7.55 with a thin layer of firm bluish grey SILT-CLAY, minor fine sand, wet, highly plastic, speckled dark brown with organics	-3.96	7.50	↓	2	C = 66 CR= 10	
			Loose grey silty fine SAND, trace clay, wet, non plastic. [WAITEMATA GROUP]				3		
			7.90 becoming medium dense	-4.31			4/7		
8					8.00	↕			
			8.15 becoming dense	-4.56					
					8.50	↕			
9					9.00	↓	4		
							6		
							9/15		
			9.55 intermixed with grey slightly weathered, very weak SILTSTONE and slightly weathered very weak SANDSTONE	-5.96					
					9.90		40		SPT=50/152mm

OBSERVATIONS:
THP=Too Hard to Penetrate with shear vane

SAMPLES
- ● Small disturbed sample
- ↕ Large disturbed sample
- ▮ Undisturbed tube sample
- ▯ Undisturbed core sample
- ↓ SPT Split spoon sample

10/50+ FIELD TESTS
- SPT = Standard Penetration Test (blows/150mm, N=blows/300mm)
- C = Cohesion as measured direct with shear vane (kPa)
- CR = Remoulded C
- CC = Corrected Reading (kPa)

PILCON VANE DIAL NO.: DR2122

DATE STARTED : 20-Jan-98 CORE DIA.: 90 mm RIG : Trailer Mounted

DATE FINISHED: 20-Jan-98 LOGGED BY: M.Watson CONTRACTOR: Prodrill

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BH_64277

BH_TT66607

BH_64277

RECORD OF BOREHOLE 214

SHEET 3 OF 3 | BH214

JOB NAME : Quay Street West Underpass

CLIENT : Auckland City Council

JOB NO. : 2409275

LOCATION : Lower Hobson St

COORDS : 6482760.10N, 2667686.50E

ELEV.: 3.59 DATUM: LANDS & SURVEY

DEPTH(m)	STRATA LGD	STRATA SYM	STRATA DESCRIPTION	ELEVATN (meters)	SAMPLE DEPTH(m)	SAMPLE TYPE	FIELD TESTS SPT	FIELD TESTS VANE	FIELD TESTS OTHER	
			10.00 Moderately thickly interbedded with grey slightly weathered very weak SANDSTONE and grey slightly weathered, very weak SILTSTONE	-6.41	10.00				TT	
					10.40					
11					11.00					
					11.50					
			11.65 — laminated with carbonaceous material	-8.06						
12					12.00		28			SPT=22/25mm
							22/50+			
					12.40					
13					13.00		50/50+			SPT=50/95mm
			END OF BOREHOLE 214 . 13.10m BELOW GROUND SURFACE							
14										

OBSERVATIONS:
THP=Too Hard to Penetrate with shear vane

SAMPLES
- Small disturbed sample
- Large disturbed sample
- Undisturbed tube sample
- Undisturbed core sample
- SPT Split spoon sample

FIELD TESTS
- SPT = Standard Penetration Test (blows/150mm,N=blows/100mm)
- C = Cohesion as measured direct with shear vane (kPa)
- CR = Remoulded C
- CC = Corrected Reading (kPa)

PILCON VANE DIAL NO.: DR2122

DATE STARTED : 20-Jan-98 CORE DIA.: 90 mm RIG : Trailer Mounted

DATE FINISHED: 20-Jan-98 LOGGED BY: M.Watson CONTRACTOR: Prodrill

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RECORD OF BOREHOLE 215

SHEET 1 OF 3 | BH215

JOB NAME : Quay Street West Underpass

CLIENT : Auckland City Council

JOB NO. : 2409275

LOCATION : Lower Hobson St

COORDS : 6482794.50N, 2667699.80E

ELEV.: 3.69 DATUM: LANDS & SURVEY

DEPTH(m)	LGD	SYM	STRATA DESCRIPTION	ELEVATN (metres)	SAMPLE DEPTH(m)	SAMPLE TYPE	FIELD TESTS SPT	FIELD TESTS VANE	FIELD TESTS OTHER
			0.00 Asphaltic Concrete	3.69					
	ooo		0.15 Dense brownish grey GRAVEL,fine to coarse,moderately weathered,moderately strong,vesicular,subangular Basalt	3.54					
			0.70 Loose orange mottled light brown silty clayey SAND,wet,moderately plastic [FILL]	2.99	0.70	↕			DB
			0.90 mottled with black SILT-CLAY,wet,highly plastic	2.79					
			0.95	2.74					
1			1.00 becoming hard greenish brown SILT-CLAY	2.69	1.00	↕			
	×××		Very stiff brown mottled reddish orange and grey SILT-CLAY,some sand,trace gravel,moist,highly plastic. Gravel,reddish orange,highly weathered,weak Basalt.						
	×××		1.50 Soft, brownish grey and orange extremely weathered,extremely weak SILTSTONE [FILL]	2.19	1.50	↓	1 1 1 2		THP
2	×××								
	×××		2.50 Brownish greenish grey highly weathered,extremely weak SANDSTONE,intermixed with soft grey SILT-CLAY,moist,highly plastic and loose brownish grey SAND,some silt,minor clay,moist,non plastic [FILL]	1.19	2.50	↕			
3	×××				3.00	↓	1 1 2 3		THP
	×××								
4	×××		4.00 Soft to very stiff brownish grey mottled orange and grey SILT-CLAY,trace fine sand,moist,highly plastic with pockets of grey,moderately weathered very weak SILTSTONE [FILL]	-0.31	4.00	↕			
	×××		4.15 with a moderately thin layer of grey,moderately weathered to highly weathered,very weak SANDSTONE,stained orange along defects	-0.46					
	×××		4.50	-0.81	4.50	↓	1 4 4 8		THP
	×××		4.60 Stiff grey brown SILT-CLAY,trace fine sand,moist,highly plastic. Grey,moderately weathered,extremely weak SILTSTONE	-0.91					

OBSERVATIONS:
THP=Too Hard to Penetrate with shear vane

SAMPLES
- Small disturbed sample
- Large disturbed sample
- Undisturbed tube sample
- Undisturbed core sample
- SPT Split spoon sample

FIELD TESTS
- SPT = Standard Penetration Test (blows/150mm,N=blows.300mm)
- C = Cohesion as measured direct with shear vane (kPa)
- CR = Remoulded C
- CC = Corrected Reading (kPa)

PILCON VANE DIAL NO.: DR2122

DATE STARTED : 19-Jan-98 CORE DIA.: 90 mm RIG : Trailer Mounted

DATE FINISHED: 20-Jan-98 LOGGED BY: M.Watson CONTRACTOR: Prodrill

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BH_64278

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BH_64278

RECORD OF BOREHOLE 215

SHEET 2 OF 3 | BH215

JOB NAME : Quay Street West Underpass

CLIENT : Auckland City Council

JOB NO. : 2409275

LOCATION : Lower Hobson St

COORDS : 6482794.50N, 2667699.80E

ELEV.: 3.69 DATUM: LANDS & SURVEY

DEPTH (m)	STRATA LGD	STRATA SYM	STRATA DESCRIPTION	ELEVATN (metres)	SAMPLE DEPTH (m)	SAMPLE TYPE	FIELD TESTS SPT	FIELD TESTS VANE	FIELD TESTS OTHER
					-				
			5.50 Medium dense grey mottled orange SAND,wet,non plastic,intermixed with very stiff to hard grey SILT-CLAY,trace fine sand,wet,highly plastic [FILL]	-1.81	5.50	‡			
6					6.00	↓	0		THP
							0		
							1		
							1		
					6.50	▮			
7			7.00 Loose grey SAND,minor silt,minor fine gravel sized shells,trace clay;wet,non plastic. [TAURANGA GROUP SEDIMENTS(Upper)]	-3.31	7.00	‡			
					7.50	▮		C = 30 CR= 10	
8			8.00 Soft grey sandy SILT-CLAY,trace fine gravel sized shells,wet,highly plastic [TAURANGA GROUP SEDIMENTS(Upper)]	-4.31	8.00	↓	1		
							2		
							1		
							1		
			8.50 Stiff light grey mottled brownish orange speckled white SILT-CLAY,some fine sand,moist,highly plastic [TAURANGA GROUP SEDIMENTS(Lower)]	-4.81	8.50	‡			
9					9.00	↓	5	C = 150+	
							7		
							10		
							17		
					9.50	‡			
			9.79 — with trace fine sand	-6.21					

OBSERVATIONS:
THP=Too Hard to Penetrate with shear vane

SAMPLES
- ● Small disturbed sample
- ‡ Large disturbed sample
- ▮ Undisturbed tube sample
- ▯ Undisturbed core sample
- ↓ SPT Split spoon sample

FIELD TESTS
- SPT = Standard Penetration Test (blows/150mm,N=blows/300mm)
- C = Cohesion as measured direct with shear vane (kPa)
- CR = Remoulded C
- CC = Corrected Reading (kPa)

PILCON VANE DIAL NO.: DR2122

DATE STARTED : 19-Jan-98 CORE DIA.: 90 mm RIG : Trailer Mounted

DATE FINISHED: 20-Jan-98 LOGGED BY: M.Watson CONTRACTOR: Prodrill

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RECORD OF BOREHOLE 215

SHEET 3 OF 3 | BH215

JOB NAME : Quay Street West Underpass

CLIENT : Auckland City Council

JOB NO. : 2409275

LOCATION : Lower Hobson St

COORDS : 6482794.50N, 2667699.80E

ELEV.: 3.69 DATUM: LANDS & SURVEY

DEPTH (m)	LGD	SYM	STRATA DESCRIPTION	ELEVATN (meters)	SAMPLE DEPTH(m)	SAMPLE TYPE	FIELD TESTS SPT	FIELD TESTS VANE	FIELD TESTS OTHER
			10.00 Brownish pinkish grey mottled orange SILT-CLAY,minor fine sand,trace gravel sized wood,moist,highly plastic	-6.31	10.00	‡		C = 56 CR= 28	
			10.20 Medium dense grey silty clayey SAND;wet,moderately plastic	-6.51					
			10.30	-6.61					
			10.35 Very stiff grey SILT-CLAY,trace fine sand,moist,highly plastic.	-6.66					
			Dense grey SAND;wet,non plastic (WAITEMATA GROUP)		10.50	↓	31		THP
							21/52		
11					11.00				TT
			11.50 Moderately thickly interbedded with grey slightly weathered,very weak SILTSTONE and SANDST	-7.81					
12					12.00		50/50+		SPT=50/115mm
			12.55 thinly laminated with carbonaceous material	-8.86	12.50	▯			
13					13.00	▯			
					13.50		32		SPT=20/35mm
			END OF BOREHOLE 215 , 13.70m BELOW GROUND SURFACE				20/52+		
14									

OBSERVATIONS:
THP=Too Hard to Penetrate with shear vane

SAMPLES
- ● Small disturbed sample
- ‡ Large disturbed sample
- ▮ Undisturbed tube sample
- ▯ Undisturbed core sample
- ↓ SPT Split spoon sample

FIELD TESTS
- SPT = Standard Penetration Test (blows/150mm,N=blows/300mm)
- C = Cohesion as measured direct with shear vane (kPa)
- CR = Remoulded C
- CC = Corrected Reading (kPa)

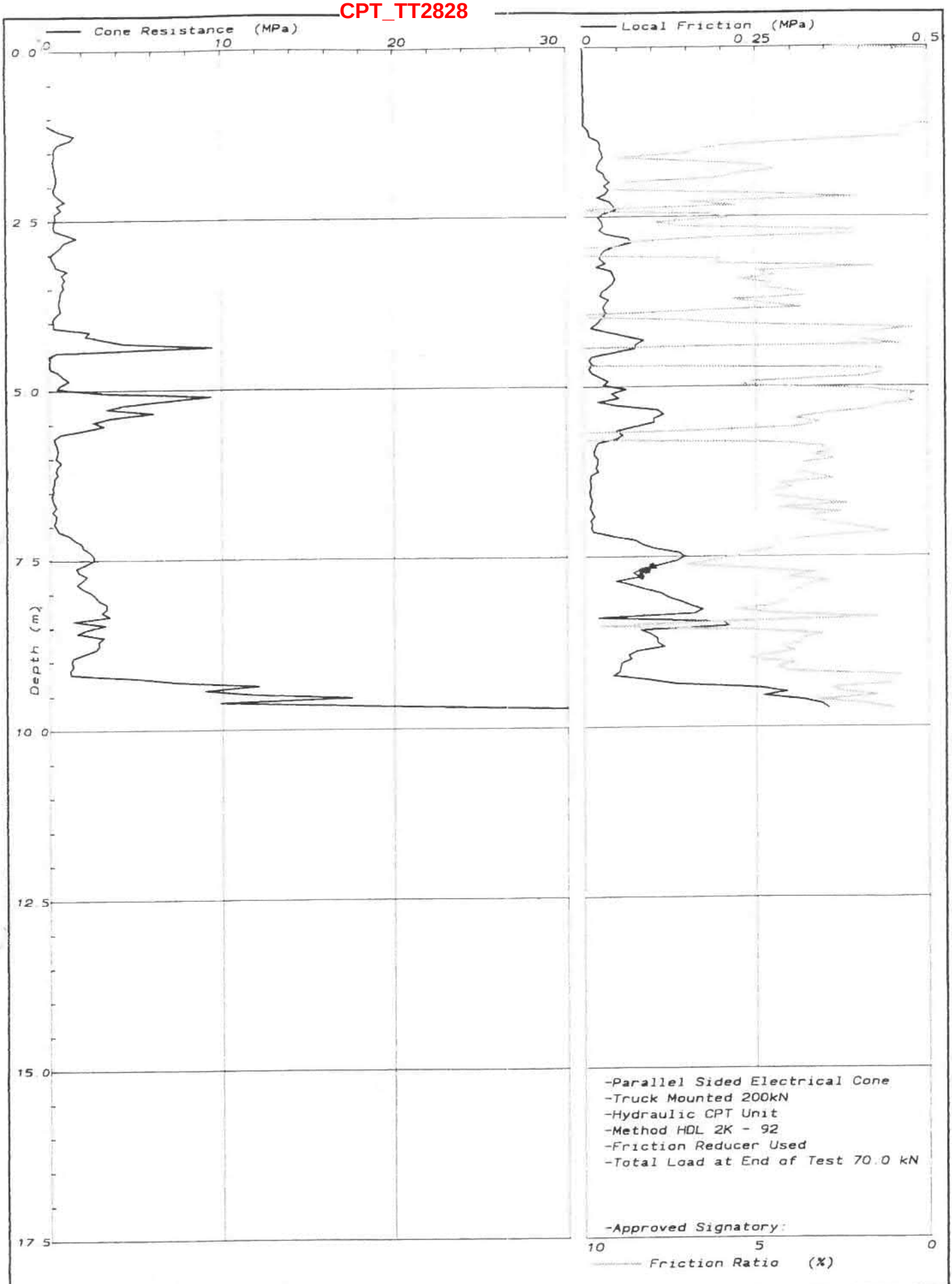
PILCON VANE DIAL NO.: DR2122

DATE STARTED : 19-Jan-98 CORE DIA.: 90 mm RIG : Trailer Mounted

DATE FINISHED: 20-Jan-98 LOGGED BY: M.Watson CONTRACTOR: Prodrill

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		Sheet of	CPT 229

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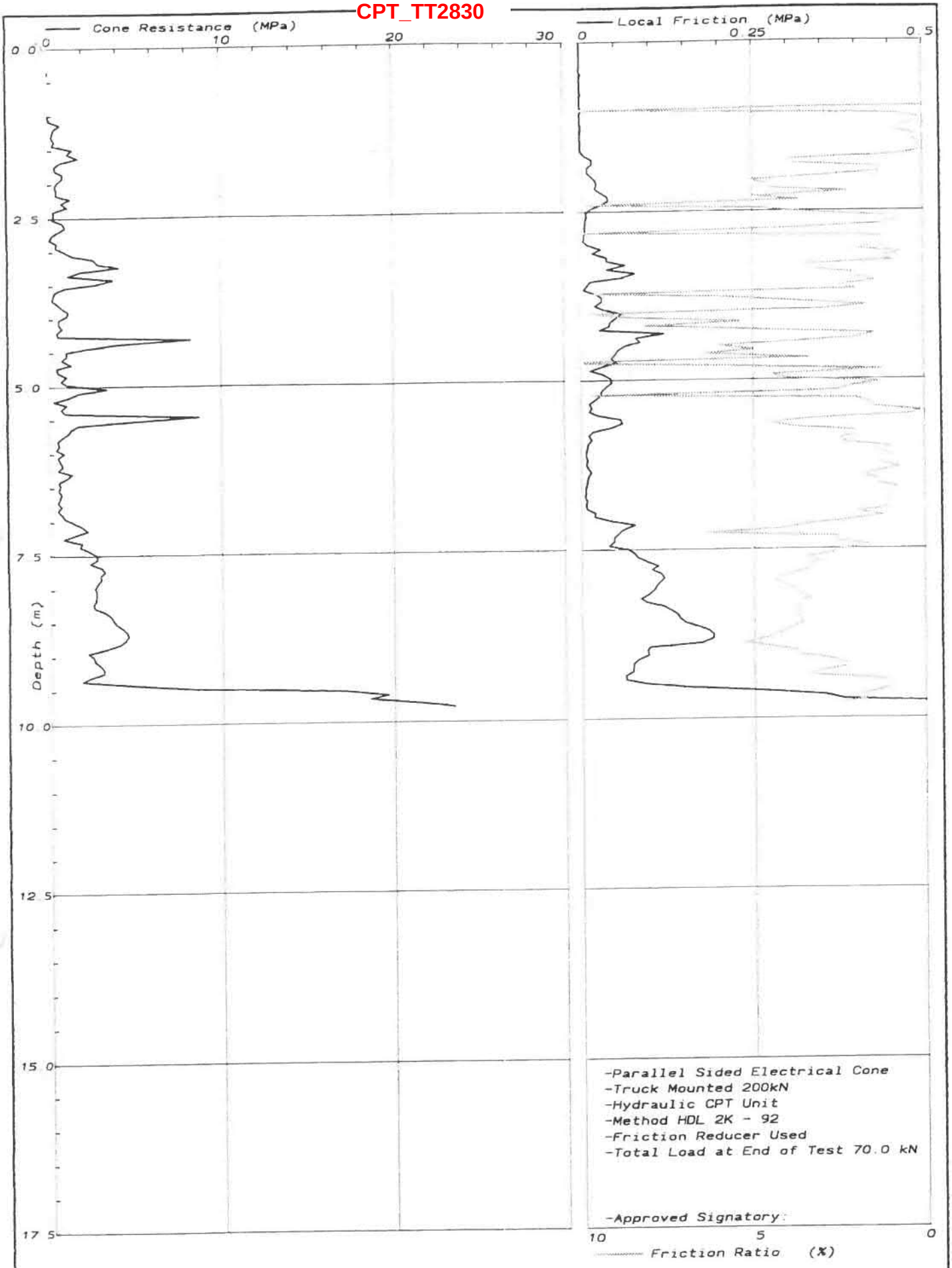
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		6/4/1998	G E BAIRD
		Sheet of	CPT 231

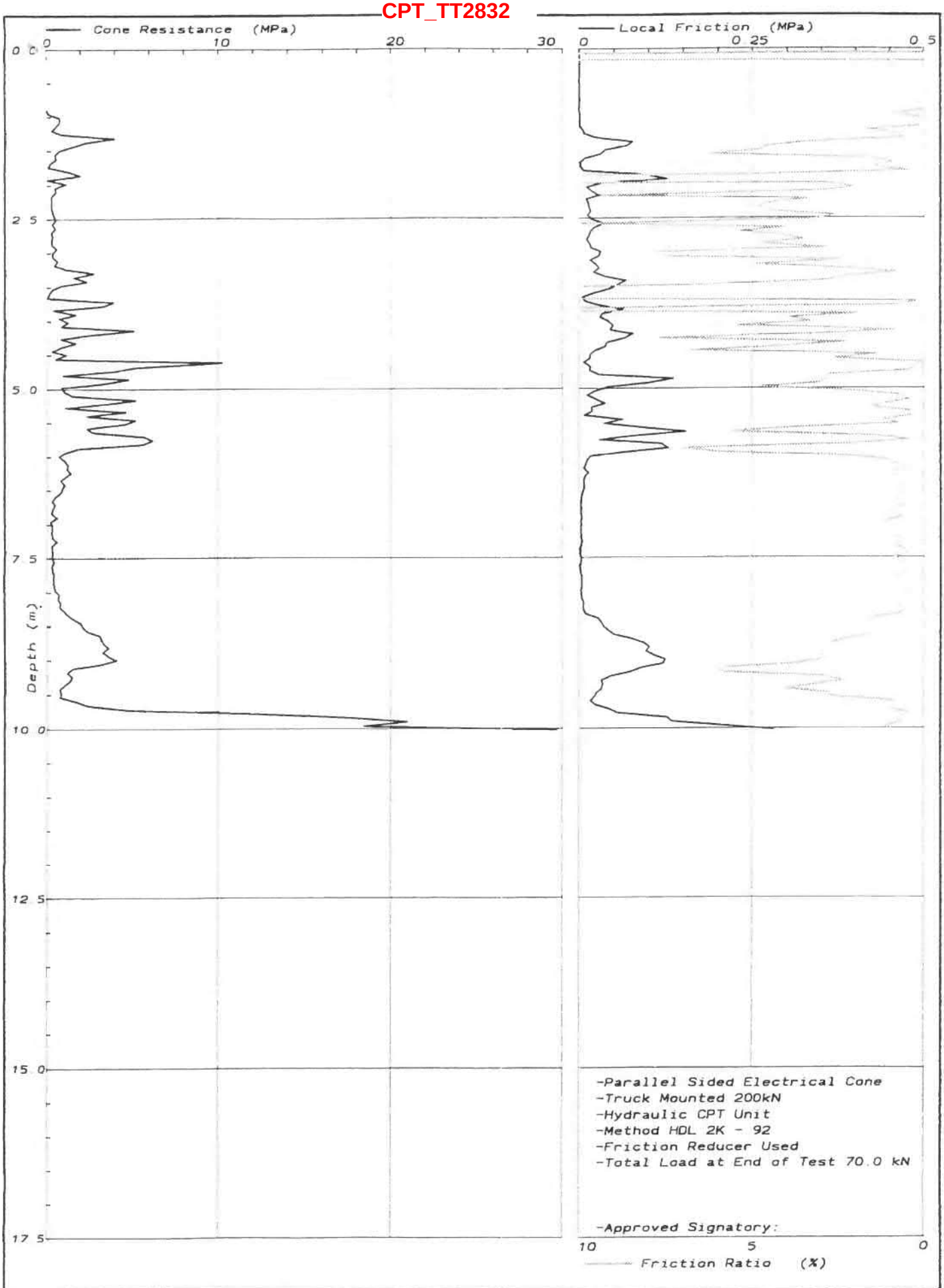
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CPT_TT2832



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QUAY STREET WEST	6/4/1998	G E BAIRD
GEOTECH INVESTIGATION	Sheet of	CPT 232

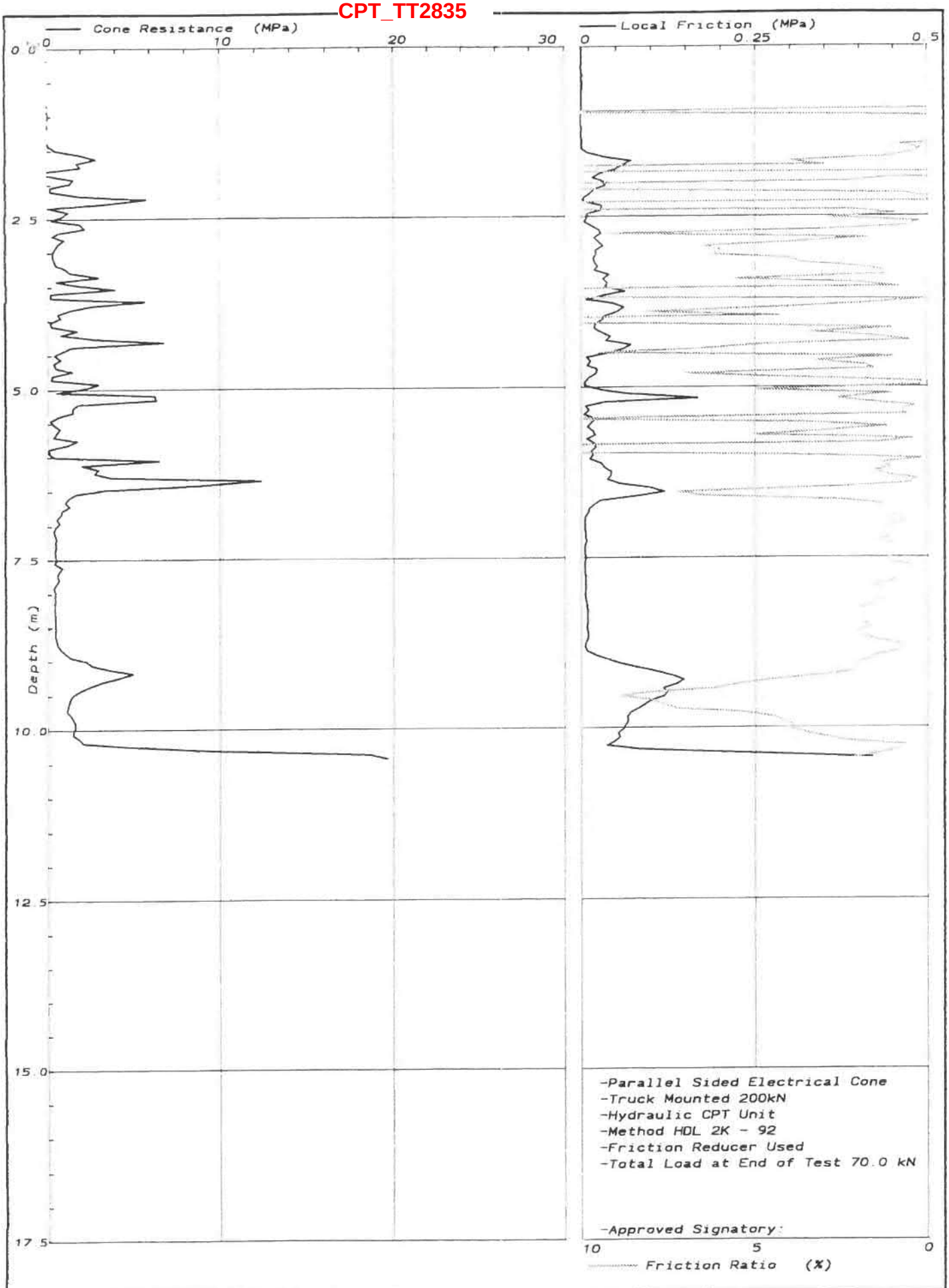
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CPT_TT2835



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		6/4/1998	G E BAIRD
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Appendix E Historical Groundwater Monitoring

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Source: Tonkin + Taylor Map Viewer (NZ) Date: 19/04/2023 5:33 PM

LEGEND

New Zealand Hillshade

VW @-0.33mRL
Ave (winter) 1.2mRL
Min (winter) 1mRL
Max (winter) 1.5mRL

VW@ -4.03mRL
Ave (winter) 0.6mRL
Min (winter) 0.3RL
Max (winter) 0.9mRL

GMW1

GMW2

VW @-1.53mRL
Ave (winter) 0.9mRL
Min (winter) 0.5mRL
Max (winter) 1.3mRL

VW @-3.83mRL
Ave (winter) 0.25mRL
Min (winter) 0RL
Max (winter) 0.6mRL

VW @-6.23mRL
Ave (winter) 0mRL
Min (winter) -0.2RL
Max (winter) 0.4mRL

VW @-1.23mRL (Fill)
Ave (winter) 0.45mRL
Min (winter) 0.2mRL
Max (winter) 0.7mRL

VW in rock showed WL to vary between 0 and 0.5mRL

BH401

VW @-2.49RL (ECBF Rock)
Ave 0.46mRL
Min 0.15mRL
Max 0.82mRL
VW @-9.14RL (ECBF Rock)
Ave 0.12mRL
Min -0.16mRL
Max 0.47mRL

BH202A

VW @-0.11RL (Fill)
Ave 1.57mRL
Min -0.14mRL
Max 2.51mRL

VW @0.33RL (ECBF rock)
Ave 2.86mRL
Min 2.28mRL
Max 3.38mRL

VW @-7.67 (ECBF rock)
Ave 0.32mRL
Min 0mRL
Max 0.97mRL

BH202B

BH251

BH402

VW @0.01mRL (Fill)
Ave (winter) 1.1mRL
Min (winter) 0.8mRL
Max (winter) 1.5mRL

VW @-4m RL, drawdown from 1mRL to -1 mRL over a three month period (ECBF rock)

VW @-14m, potentially faulty?

A3 SCALE 1:1,132
0 10 20 30 40 50 m

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NOTES:

CRS: NZGD 2000 New Zealand Transverse Mercator Credits: Eagle Technology, Land Information New Zealand, Earthstar Geographics, Eagle Technology, Land Information New Zealand, GEBCO, Community maps contributors

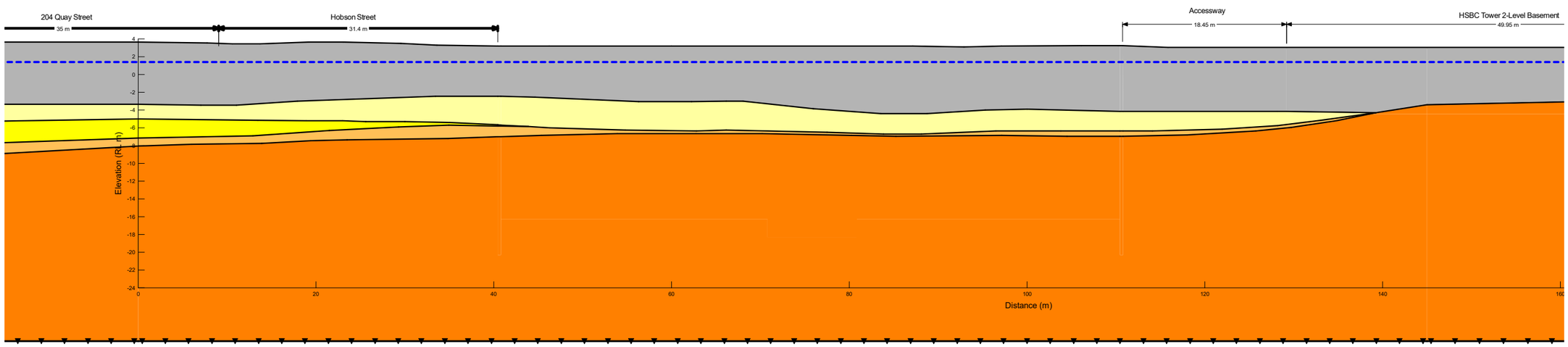
LOCATION PLAN

PROJECT No.	1016043.1000		
DESIGNED	ABL	APR.23	
DRAWN	-WEB-	APR.23	
CHECKED			
APPROVED		DATE	

CLIENT **PRECINCT**
PROJECT **DOWNTOWN CARPARK DEVELOPMENT**
TITLE GROUNDWATER MONITORING INFO
SCALE (A3) 1:1,132 FIG No. FIGURE 1. REV 0

Appendix F Groundwater Seepage Analysis

Color	Name	Vol. WC. Function	Ky'/Kx' Ratio
	ECBF rock	ECBF rock	0.1
	Hydraulic Fill	Hydraulic Fill	0.0333
	Marine Sediments	Marine Sediments	0.0333
	Pleistocene Alluvium	Pleistocene Alluvium	0.02
	Weathered ECBF	Weathered ECBF	0.02



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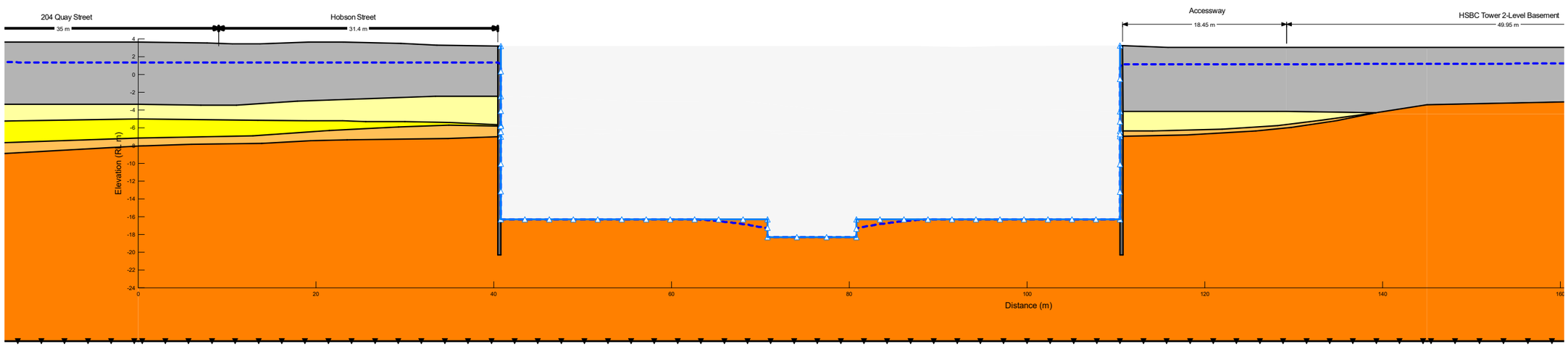
Project: 2. Section 2 - North (West to East).gsz	
Project Number: 1016043.1000	
A3 Scale: 1:500	Vertical Exaggeration: 1
Analysed by: Ric Wilkinson	Checked By: Ananth Balachandra

Analysis Details:
1. Analysis Scenario: 1a. Existing Condition
2. Method: Steady-State
3. Elapsed Time: 0 d

Analysis Description:

Directory: C:\Users\rxsw\OneDrive - Tonkin + Taylor Group Ltd\Desktop\Downtown Carpark - SEEPW\Rev4 - S92\

Color	Name	Sat Kx (m/sec)	Vol. WC. Function	Ky'/Kx' Ratio
	ECBF rock		ECBF rock	0.1
	Hydraulic Fill		Hydraulic Fill	0.0333
	Marine Sediments		Marine Sediments	0.0333
	Pleistocene Alluvium		Pleistocene Alluvium	0.02
	Secant (construction)	1e-10		1
	Weathered ECBF		Weathered ECBF	0.02



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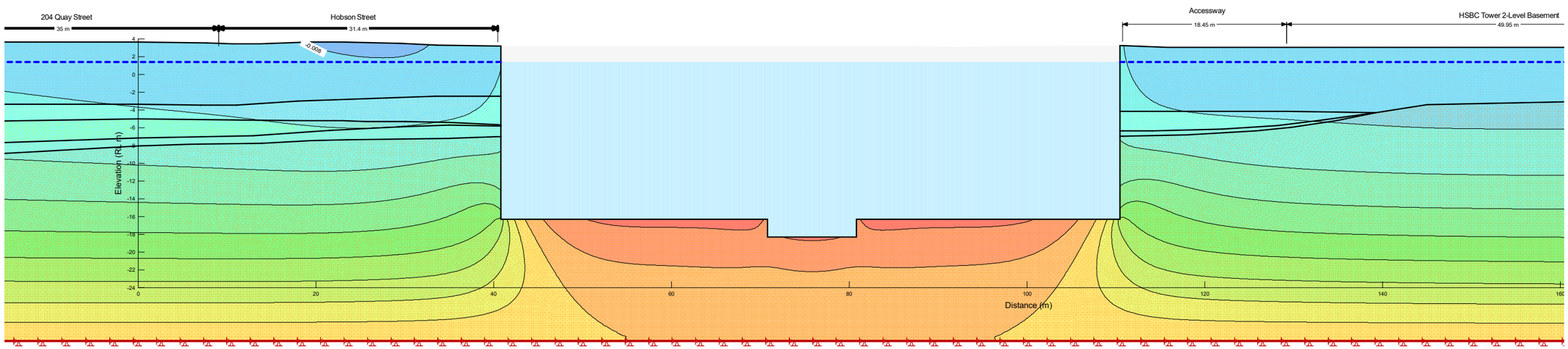
Project: 2. Section 2 - North (West to East).gsz	
Project Number: 1016043.1000	
A3 Scale: 1:500	Vertical Exaggeration: 1
Analysed by: Ric Wilkinson	Checked By: Ananth Balachandra

Analysis Details:
1. Analysis Scenario: 1b. Temporary Excavation
2. Method: Transient
3. Elapsed Time: 365 d

Analysis Description:

Directory: C:\Users\rxsw\OneDrive - Tonkin + Taylor Group Ltd\Desktop\Downtown Carpark - SEEPW\Rev4 - S92\

Color	Name	Stress Material Model	Unit Weight (kN/m³)	Effective Elastic Modulus (kPa)	Effective Poisson's Ratio	K0	Vol. WC. Function	Ky'/Kx' Ratio
	ECBF rock	Isotropic Elastic	21	200,000	0.3	1	ECBF rock	0.1
	Hydraulic Fill	Isotropic Elastic	16	6,000	0.35	1	Hydraulic Fill	0.0333
	Marine Sediments	Isotropic Elastic	16	6,000	0.35	1	Marine Sediments	0.0333
	Pleistocene Alluvium	Isotropic Elastic	17.5	20,000	0.35	1	Pleistocene Alluvium	0.02
	Weathered ECBF	Isotropic Elastic	19	40,000	0.3	1	Weathered ECBF	0.02



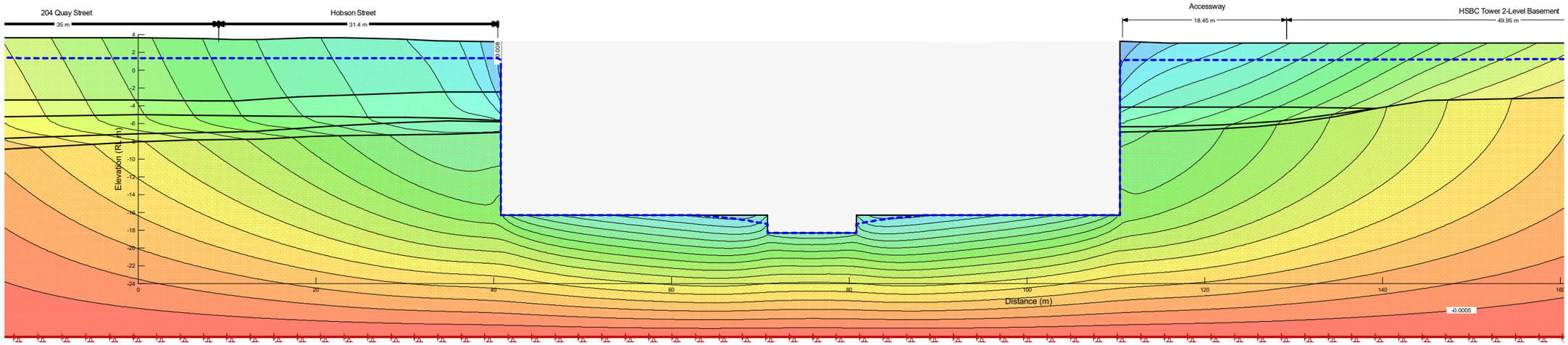
Tonkin+Taylor

Project: 2. Section 2 - North (West to East).gsz	
Project Number: 1016043.1000	
A3 Scale: 1:500	Vertical Exaggeration: 1
Analysed by: Ric Wilkinson	Checked By: Ananth Balachandra

Analysis Description:

Directory: C:\Users\rxsw\OneDrive - Tonkin + Taylor Group Ltd\Desktop\Downtown Carpark - SEEPW\Rev4 - S92\

Color	Name	Stress Material Model	Unit Weight (kN/m³)	Effective Elastic Modulus (kPa)	Effective Poisson's Ratio	K0	Vol. WC. Function	Ky'/Kx' Ratio
	ECBF rock	Isotropic Elastic	21	200,000	0.3	1	ECBF rock	0.1
	Hydraulic Fill	Isotropic Elastic	16	6,000	0.35	1	Hydraulic Fill	0.0333
	Marine Sediments	Isotropic Elastic	16	6,000	0.35	1	Marine Sediments	0.0333
	Pleistocene Alluvium	Isotropic Elastic	17.5	20,000	0.35	1	Pleistocene Alluvium	0.02
	Weathered ECBF	Isotropic Elastic	19	40,000	0.3	1	Weathered ECBF	0.02



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Project: 2. Section 2 - North (West to East).gsz	
Project Number: 1016043.1000	
A3 Scale: 1:500	Vertical Exaggeration: 1
Analysed by: Ric Wilkinson	Checked By: Ananth Balachandra

Analysis Details:
1. Analysis Scenario: Load/Deformation
2. Method: Load/Deformation
3. Elapsed Time: 1 d
4. Initial Stress and PWP Conditions from Parent Analysis: In Situ [(last)]
5. Final PWP Conditions from Other GeoStudio Analysis: 1b. Temporary Excavation [(last)]

Analysis Description:

Directory: C:\Users\rxsw\OneDrive - Tonkin + Taylor Group Ltd\Desktop\Downtown Carpark - SEEPW\Rev4 - S92\

Appendix G Basement Retention Analysis

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C:\12dSynergy\data\ALBTCAD\DCR Concept and Resource Consent Stage_1352\CAD\FIG\1016043.1000-GEO-F1.dwg 2023-May-26 9:36:57 AM Plotted By: JONALD CASTRO

LEGEND

- PROPERTY BOUNDARY
- ECBF CONTOUR (1m INTERVAL)
- GEOMAPS WATER LINE
- GEOMAPS ABANDONED WATER LINE
- WATER HYDRANT
- WATER VALVE
- GEOMAPS STORMWATER LINE
- GEOMAPS ABANDONED STORMWATER LINE
- STORMWATER CATCHPIT
- STORMWATER MANHOLE
- GEOMAPS WASTEWATER LINE
- GEOMAPS ABANDONED WASTEWATER LINE
- WASTEWATER MANHOLE
- CRL UNDERGROUND ROUTE
- CHORUS
- VECTOR COMMUNICATION - UNDERGROUND ROUTE TRENCH
- VECTOR COMMUNICATION - UNDERGROUND INSTALLED IN PLACE
- GAS
- GAS VALVE
- VODAFONE
- FIBRE OPTIC
- POWER
- LAMP

204 QUAY ST

QUAY ST

MSOCIAL BUILDING

DESIGN SECTION 1
800mm THICK D-WALL WITH TOP-DOWN CONSTRUCTION OF B1 LEVELS AND TEMPORARY HIGH-LEVEL AND LOW-LEVEL PROP SUPPORTS

DESIGN SECTION 2
800mm THICK D-WALL WITH FOUR ROWS OF GROUND ANCHORS EMBEDDED INTO ECBF ROCK

WATERMARK BUILDING

GROUND ANCHORS MAY BE RESTRICTED DUE TO NEIGHBOURING BASEMENT + DEPTH TO ECBF ROCK

HSBC TOWER

HOBSON ST

4m

TEMPORARY PROP MAY BE REQUIRED FOR LOAD TRANSFER IN THE EAST-WEST DIRECTION.

DESIGN SECTION 3

OPTION 1: AU25 SHEETPILE WALL EMBEDDED 1M INTO ECBF ROCK WITH THREE ROWS OF GROUND ANCHORS INTO ECBF ROCK. FORM 1m WIDE BENCH AND OPEN-CUT WITHIN ROCK TO FORMATION LEVEL.

OPTION 2: FULLY EMEDDED 800mm THICK D-WALL WITH FOUR ROWS OF GROUND ANCHORS EMBEDDED INTO ECBF ROCK

TEPID BATHS

AON TOWER

CUSTOMS ST WEST

STURDEE ST

FANSHAWE ST

A3 SCALE 1:750

0 5 10 15 20 25 30 35 40 (m)

ORIGINAL IN COLOUR

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☐ BACKDRAFTING REQUIRED	CHECK PRINT	
☐ READY TO ISSUE	INITIAL	DATE
CAD SELF CHECK		
DESIGN CHECK		
DRAFTING CHECK		
BACKDRAFTER - EDITS MADE		
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NOTES:
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2. VODAFONE AND CRL INFORMATION SUPPLIED BY BEFORE U DIG. REF 11197173 - Vodafone Plan.pdf AND "11197172 - CityLink Plan (002).pdf".
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PROJECT No.	1016043.1000		
DESIGNED	KASC	May.23	
DRAWN	JC	May.23	
CHECKED			
		APPROVED	DATE

CLIENT PRECINCT PROPERTIES HOLDINGS LIMITED

PROJECT DOWNTOWN CARPARK REDEVELOPMENT

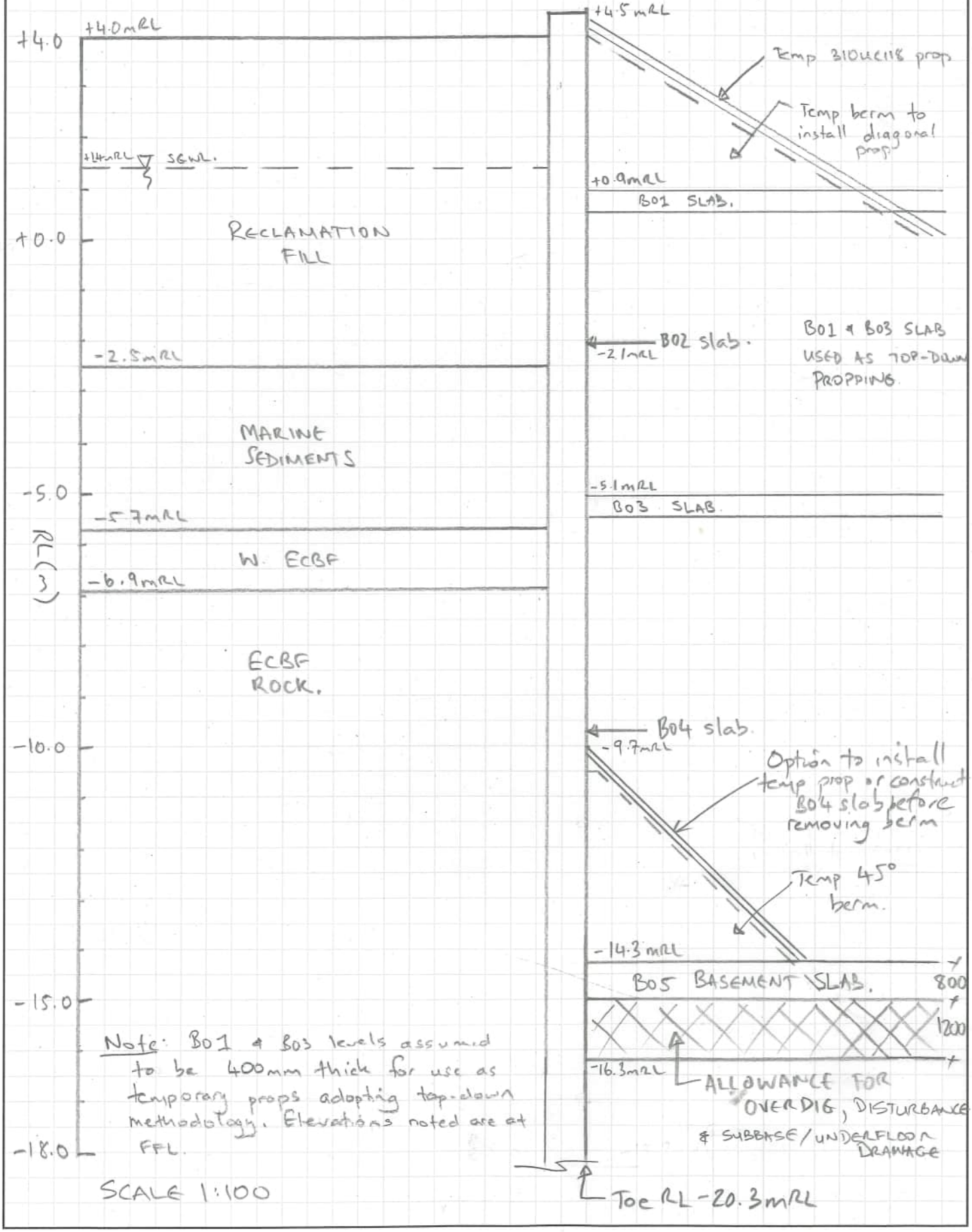
TITLE GEOTECHNICAL LAYOUT PLAN

UNDER REVISION 1

SCALE (A3) 1:750 FIG No. FIGURE 1 REV 1

Tonkin+Taylor	Project: Downtown Carpark	Computed: RXSW 23/5/2024	Office: AKL
Description:		Checked: 20	Job No: 1016043.1
		Revised: 20	File:
		Checked: 20	Sheet No.

SECTION 1 - TOP DOWN CONSTRUCTION



Note: B01 & B03 levels assumed to be 400mm thick for use as temporary props adopting top-down methodology. Elevations noted are at FFL.

SCALE 1:100

Units: kN,m

INPUT DATA

SOIL PROFILE

Stratum no.	Elevation of top of stratum	Soil types Left side	Right side
1	4.00	1 Fill	1 Fill
2	-2.50	2 Marine Sediment	2 Marine Sediment
3	-5.70	5 Weathered ECBF	3 Pleistocene Alluvium
4	-6.90	4 ECBF	4 ECBF

SOIL PROPERTIES

-- Soil type -- No.	Description (Datum elev.)	Bulk density kN/m3	Young's Modulus Eh,kN/m2 (dEh/dy)	At rest coeff. Ko (dKo/dy)	Consol state. NC/OC (Nu)	Active limit Ka (Kac)	Passive limit Kp (Kpc)	Cohesion kN/m2 (dc/dy)
1	Fill	16.50	6000	0.530	OC (0.300)	0.309 (1.300)	3.868 (5.395)	3.000d
2	Marine Sediment	16.50	6000	0.530	OC (0.300)	0.309 (1.300)	4.369 (5.836)	3.000d
3	Pleistocene Alluvium	17.50	20000	0.500	OC (0.300)	0.283 (1.241)	4.369 (5.836)	5.000d
4	ECBF	21.00	400000	0.357	OC (0.250)	0.180 (0.978)	8.892 (9.405)	100.0d
5	Weathered ECBF	19.00	40000	0.470	OC (0.300)	0.259 (1.185)	4.964 (6.343)	7.000d
6	Engineered Fill	19.00	40000	0.384	OC (0.300)	0.197 (1.027)	7.588 (8.432)	0.0d

Additional soil parameters associated with Ka and Kp

------- Soil type ------- No.	Description	--- parameters for Ka --- Soil friction angle	Wall adhesion coeff.	Back-fill angle	--- parameters for Kp --- Soil friction angle	Wall adhesion coeff.	Back-fill angle
1	Fill	28.00	0.670	0.00	28.00	0.500	0.00
2	Marine Sediment	28.00	0.670	0.00	30.00	0.500	0.00
3	Pleistocene Alluvium	30.00	0.670	0.00	30.00	0.500	0.00
4	ECBF	40.00	0.670	0.00	40.00	0.500	0.00
5	Weathered ECBF	32.00	0.670	0.00	32.00	0.500	0.00
6	Engineered Fill	38.00	0.670	0.00	38.00	0.500	0.00

GROUND WATER CONDITIONS

Density of water = 10.00 kN/m3

	Left side	Right side
Initial water table elevation	1.40	1.40

Automatic water pressure balancing at toe of wall : No

Water press. profile no.	Left side Point no.	Elev. m	Piezo elev. m	Water press. kN/m2	Right side Point no.	Elev. m	Piezo elev. m	Water press. kN/m2
1	1	1.40	1.40	0.0	1	0.00	0.00	0.0
2	1	1.40	1.40	0.0	1	-6.00	-6.00	0.0
3	1	1.40	1.40	0.0	1	-16.30	-16.30	0.0
	2	-5.90	1.40	73.0				
	3	-20.30	-11.30	90.0				
4	1	2.50	2.50	0.0	1	-15.10	-15.10	0.0
5	1	1.40	1.40	0.0	1	-15.10	-15.10	0.0

WALL PROPERTIES

Type of structure = Fully Embedded Wall
Elevation of toe of wall = -20.30
Maximum finite element length = 1.20 m
Youngs modulus of wall E = 2.7400E+07 kN/m2
Moment of inertia of wall I = 0.042700 m4/m run
E.I = 1.1700E+06 kN.m2/m run
Yield Moment of wall = Not defined

STRUTS and ANCHORS

Prop no.	Elev.	Prop spacing m	Cross-section area sq.m	Youngs modulus kN/m2	Free length m	Inclin-ation (degs)	Pre-stress /prop kN	Strut or Anchor	Allow tension ?	L/R
1	0.70	1.00	0.400000	2.740E+07	30.00	0.00	0	Strut	No	R
2	-5.30	1.00	0.400000	2.740E+07	30.00	0.00	0	Strut	No	R
3	4.30	5.00	0.015000	2.100E+08	9.80	27.00	0	Strut	No	R
4	-14.70	1.00	0.800000	2.740E+07	30.00	0.00	0	Strut	No	R
5	-9.80	1.00	0.200000	2.740E+07	30.00	0.00	0	Strut	No	R
6	-2.20	1.00	0.200000	2.740E+07	30.00	0.00	0	Strut	No	R
7	4.40	1.00	0.200000	2.740E+07	30.00	0.00	0	Strut	No	R
8	-10.30	5.00	0.013400	2.000E+08	6.10	45.00	100.0	Strut	No	R

HORIZONTAL and MOMENT LOADS/RESTRAINTS

Load no.	Elevation	Horizontal load kN/m run	Moment load kN.m/m run	Moment restraint kN.m/m/rad	Partial factor (Category)
1	-13.20	61.50	0	0	N/A
2	-10.90	61.50	0	0	N/A
3	-8.60	61.50	0	0	N/A
4	-6.30	61.50	0	0	N/A
5	-4.00	61.50	0	0	N/A
6	-1.70	61.50	0	0	N/A
7	0.60	61.50	0	0	N/A
8	2.90	61.50	0	0	N/A

SURCHARGE LOADS

Surch -arge no.	Elev.	Distance from wall	Length parallel to wall	Width perpend. to wall	Surcharge ----- kN/m2 ----- Near edge	Far edge	Equiv. soil type	Partial factor/ Category
1	4.00	2.00(L)	100.00	20.00	12.00	=	N/A	N/A

Note: L = Left side, R = Right side

CONSTRUCTION STAGES

Construction stage no.	Stage description
1	Change EI of wall to 877500 kN.m2/m run Reset wall displacements to zero at this stage
2	Apply surcharge no.1 at elevation 4.00
3	Apply water pressure profile no.1 No analysis at this stage
4	Excavate to elevation 4.00 on RIGHT side Toe of berm at elevation 0.00 Width of top of berm = 0.01 Width of toe of berm = 8.00
5	Install strut or anchor no.3 at elevation 4.30
6	Excavate to elevation 0.00 on RIGHT side
7	Install strut or anchor no.1 at elevation 0.70
8	Apply water pressure profile no.2 No analysis at this stage
9	Excavate to elevation -6.00 on RIGHT side
10	Install strut or anchor no.2 at elevation -5.30
11	Apply water pressure profile no.3 No analysis at this stage
12	Excavate to elevation -10.70 on RIGHT side Toe of berm at elevation -16.30 Width of top of berm = 0.30 Width of toe of berm = 5.90
13	Install strut or anchor no.8 at elevation -10.30
14	Excavate to elevation -16.30 on RIGHT side
15	Fill to elevation -15.10 on RIGHT side with soil type 6
16	Install strut or anchor no.4 at elevation -14.70
17	Install strut or anchor no.5 at elevation -9.80
18	Remove strut or anchor no.8 at elevation -10.30
19	Install strut or anchor no.6 at elevation -2.20
20	Remove strut or anchor no.3 at elevation 4.30
21	Install strut or anchor no.7 at elevation 4.40
22	Change EI of wall to 585000 kN.m2/m run Allow wall to relax with new modulus value
23	Apply water pressure profile no.4
24	Apply water pressure profile no.5
25	Change EI of wall to 292500 kN.m2/m run Allow wall to relax with new modulus value
26	Apply load no.1 at elevation -13.20
27	Apply load no.2 at elevation -10.90
28	Apply load no.3 at elevation -8.60
29	Apply load no.4 at elevation -6.30
30	Apply load no.5 at elevation -4.00
31	Apply load no.6 at elevation -1.70
32	Apply load no.7 at elevation 0.60
33	Apply load no.8 at elevation 2.90

FACTORS OF SAFETY and ANALYSIS OPTIONS

Stability analysis:
 Method of analysis - Strength Factor method
 Factor on soil strength for calculating wall depth = 1.00

Parameters for undrained strata:
 Minimum equivalent fluid density = 5.00 kN/m3
 Maximum depth of water filled tension crack = 0.00 m

Bending moment and displacement calculation:
 Method - 2-D finite element model
 Open Tension Crack analysis? - No
 Active limit arching modelled? - Yes
 Non-linear Modulus Parameter (L) = 24.80 m

Boundary conditions:
 Length of wall (normal to plane of analysis) = 1000.00 m

 Width of excavation on Left side of wall = 60.00 m
 Width of excavation on Right side of wall = 60.00 m

 Distance to rigid boundary on Left side = 60.00 m
 Distance to rigid boundary on Right side = 60.00 m

Elevation of rigid lower boundary = -47.75

Lower rigid boundary at elevation -47.75 - Rough
Rigid boundary on Left side - Rough
Rigid boundary on Right side - Rough
Wall / soil interface - Smooth

OUTPUT OPTIONS

Stage no.	Stage description	Output options: Displacement Bending mom. Shear force	Active, Passive pressures	Graph. output
1	Change EI of wall to 877500kN.m2/m run	No	No	No
2	Apply surcharge no.1 at elev. 4.00	No	No	No
3	Apply water pressure profile no.1	No	No	No
4	Excav. to elev. 4.00 on RIGHT side	Yes	No	No
5	Install prop no.3 at elev. 4.30	No	No	No
6	Excav. to elev. 0.00 on RIGHT side	Yes	Yes	Yes
7	Install prop no.1 at elev. 0.70	No	No	No
8	Apply water pressure profile no.2	No	No	No
9	Excav. to elev. -6.00 on RIGHT side	Yes	Yes	Yes
10	Install prop no.2 at elev. -5.30	No	No	No
11	Apply water pressure profile no.3	No	No	No
12	Excav. to elev. -10.70 on RIGHT side	No	No	No
13	Install prop no.8 at elev. -10.30	No	No	No
14	Excav. to elev. -16.30 on RIGHT side	Yes	Yes	Yes
15	Fill to elev. -15.10 on RIGHT side	No	No	No
16	Install prop no.4 at elev. -14.70	No	No	No
17	Install prop no.5 at elev. -9.80	No	No	No
18	Remove prop no.8 at elev. -10.30	No	No	No
19	Install prop no.6 at elev. -2.20	No	No	No
20	Remove prop no.3 at elev. 4.30	No	No	No
21	Install prop no.7 at elev. 4.40	No	No	No
22	Change EI of wall to 585000kN.m2/m run	No	No	No
23	Apply water pressure profile no.4	No	No	No
24	Apply water pressure profile no.5	No	No	No
25	Change EI of wall to 292500kN.m2/m run	No	No	No
26	Apply load no.1 at elev. -13.20	No	No	No
27	Apply load no.2 at elev. -10.90	No	No	No
28	Apply load no.3 at elev. -8.60	No	No	No
29	Apply load no.4 at elev. -6.30	No	No	No
30	Apply load no.5 at elev. -4.00	No	No	No
31	Apply load no.6 at elev. -1.70	No	No	No
32	Apply load no.7 at elev. 0.60	No	No	No
33	Apply load no.8 at elev. 2.90	Yes	No	No
*	Summary output	Yes	-	Yes

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Units: kN,m

Stage No. 4 Excavate to elevation 4.00 on RIGHT side
Toe of berm at elevation 0.00
Width of top of berm = 0.01
Width of toe of berm = 8.00

STABILITY ANALYSIS of Fully Embedded Wall according to Strength Factor method
Factor of safety on soil strength

Stage No.	Ground level Act.	Ground level Pass.	Prop Elev.	FoS for toe elev. = -20.30: Factor of Safety	Moment equilib. at elev.	Toe elev. for FoS = 1.000	Wall Penetration	Direction of failure
4	4.00	4.00	Cant.	Conditions not suitable for FoS calc.				

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall
Analysis options
Length of wall perpendicular to section = 1000.00m
2-D finite element model. Active limit arching modelled.
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Left side 60.00 from wall — Rough boundary
Right side 60.00 from wall — Rough boundary
Lower rigid boundary at elevation -47.75 — Rough boundary

*** Wall displacements reset to zero at stage 1

Node no.	Y coord	Nett pressure kN/m2	Wall disp. m	Wall rotation rad.	Shear force kN/m	Bending moment kN.m/m	Prop forces kN/m	EI of wall kN.m2/m
1	4.40	0.00	0.006	3.30E-04	0.0	-0.0		877500
2	4.30	0.00	0.006	3.30E-04	0.0	-0.0		877500
3	4.00	0.00	0.006	3.30E-04	0.0	0.0		877500
		-2.52	0.006	3.30E-04	0.0	0.0		
4	2.90	-4.62	0.005	3.31E-04	-3.9	-1.5		877500
5	2.50	-4.14	0.005	3.32E-04	-5.7	-3.4		877500
6	1.40	-3.08	0.005	3.42E-04	-9.7	-12.0		877500
7	0.70	1.55	0.004	3.54E-04	-10.2	-19.1		877500
8	0.60	2.30	0.004	3.57E-04	-10.0	-20.1		877500
9	0.00	6.19	0.004	3.72E-04	-7.4	-25.5		877500
10	-0.85	6.79	0.004	3.99E-04	-1.9	-29.5		877500
11	-1.70	7.33	0.003	4.27E-04	4.1	-28.7		877500
12	-2.20	7.67	0.003	4.42E-04	7.8	-25.7		877500
13	-2.50	7.97	0.003	4.51E-04	10.2	-23.0		877500
14	-3.25	8.39	0.003	4.66E-04	16.3	-13.2		877500
15	-4.00	8.82	0.002	4.71E-04	22.8	1.4		877500
16	-4.65	9.18	0.002	4.64E-04	28.6	18.1		877500
17	-5.30	9.38	0.002	4.43E-04	34.6	38.6		877500
18	-5.70	8.25	0.002	4.22E-04	38.2	53.3		877500
		-8.74	0.002	4.22E-04	38.2	53.3		
19	-6.00	-4.25	0.002	4.02E-04	36.2	64.3		877500
20	-6.30	-0.85	0.001	3.78E-04	35.5	75.0		877500
21	-6.90	0.92	0.001	3.20E-04	35.5	96.1		877500
		-86.41	0.001	3.20E-04	35.5	96.1		
22	-7.75	-33.61	0.001	2.27E-04	-15.5	95.1		877500
23	-8.60	1.71	0.001	1.47E-04	-29.1	69.7		877500
24	-9.80	9.08	0.001	7.54E-05	-22.6	36.0		877500
25	-10.30	9.66	0.001	5.77E-05	-17.9	25.9		877500

(continued)

Stage No.4 Excavate to elevation 4.00 on RIGHT side
Toe of berm at elevation 0.00
Width of top of berm = 0.01
Width of toe of berm = 8.00

Node no.	Y coord	Nett pressure	Wall disp.	Wall rotation	Shear force	Bending moment	Prop forces	EI of wall
		kN/m2	m	rad.	kN/m	kN.m/m	kN/m	kN.m2/m
26	-10.70	8.80	0.001	4.74E-05	-14.2	19.5		877500
27	-10.90	7.00	0.001	4.33E-05	-12.7	16.8		877500
28	-12.05	4.50	0.001	2.78E-05	-6.0	6.9		877500
29	-13.20	2.19	0.001	2.14E-05	-2.2	2.9		877500
30	-13.95	0.98	0.001	1.93E-05	-1.0	1.9		877500
31	-14.70	0.56	0.001	1.80E-05	-0.4	1.4		877500
32	-15.10	0.17	0.001	1.74E-05	-0.3	1.2		877500
33	-16.30	0.07	0.001	1.58E-05	-0.2	1.0		877500
34	-17.15	-0.07	0.000	1.49E-05	-0.2	0.9		877500
35	-18.00	-0.11	0.000	1.41E-05	-0.2	0.7		877500
36	-19.15	-0.04	0.000	1.34E-05	-0.3	0.4		877500
37	-20.30	0.46	0.000	1.32E-05	-0.1	0.0		0
38	-20.60	0.06	0.000	0	0.0	0.0		0
39	-24.70	0.00	0.000	0	0.1	0.0		0
40	-28.80	0.01	0.000	0	0.2	0.0		0
41	-33.60	-0.00	0.000	0	0.2	0.0		0
42	-38.40	-0.00	0.000	0	0.1	0.0		0
43	-43.08	-0.00	0.000	0	0.1	0.0		0
44	-47.75	-0.05	0.000	0	-0.0	0.0		---

LEFT side

Node no.	Y coord	Water press.	Effective stresses: Vertic -al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
1	4.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	4.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
3	4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
		0.00	0.00	0.00	16.18	0.00	0.00a	5999
4	2.90	0.00	18.76	1.90	88.77	7.44	7.44	5999
5	2.50	0.00	26.00	4.14	116.75	11.31	11.31	5999
6	1.40	0.00	46.19	10.38	194.86	21.89	21.89	5999
7	0.70	7.00	51.88	12.13	216.89	24.87	31.87	5999
8	0.60	8.00	52.68	12.38	219.97	25.33	33.33	5999
9	0.00	14.00	57.37	13.83	238.12	27.80	41.80	5999
10	-0.85	22.50	63.79	15.81	262.95	31.22	53.72	5999
11	-1.70	31.00	70.00	17.73	286.97	34.56	65.56	5999
12	-2.20	36.00	73.58	18.84	300.80	36.52	72.52	5999
13	-2.50	39.00	75.70	19.49	309.01	37.74	76.74	5999
		39.00	75.70	19.49	348.27	37.74	76.74	5999
14	-3.25	46.50	80.94	21.11	371.18	40.60	87.10	5999
15	-4.00	54.00	86.11	22.71	393.76	43.46	97.46	5999
16	-4.65	60.50	90.54	24.08	413.12	45.91	106.41	5999
17	-5.30	67.00	94.93	25.44	432.31	48.28	115.28	5999
18	-5.70	71.00	97.62	26.27	444.04	48.97	119.97	5999
		71.00	97.62	17.04	528.95	32.87	103.87	39991
19	-6.00	74.00	100.38	17.75	542.63	37.47	111.47	39991
20	-6.30	77.00	103.13	18.47	556.28	41.06	118.06	39991
21	-6.90	83.00	108.61	19.89	583.51	44.99	127.99	39991
		83.00	108.61	0.00	1906.20	0.00	83.00a	399914
22	-7.75	91.50	118.05	0.00	1990.12	19.47	110.97	399914
23	-8.60	100.00	127.46	0.00	2073.75	40.01	140.01	399914
24	-9.80	112.00	140.69	0.00	2191.39	48.26	160.26	399914
25	-10.30	117.00	146.19	0.00	2240.29	50.45	167.45	399914

Run ID. 1_North_800DWall_topdown+propped_gw2 | Sheet No.
Downtown Carpark Redevelopment | Date:30-05-2024
800mm D-Wall - Top Down + Propped | Checked :

(continued)

Stage No.4 Excavate to elevation 4.00 on RIGHT side
Toe of berm at elevation 0.00
Width of top of berm = 0.01
Width of toe of berm = 8.00

LEFT side

Node no.	Y coord	Water press.	Effective stresses: Vertic -al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
26	-10.70	121.00	150.58	0.00	2279.35	51.57	172.57	399914
27	-10.90	123.00	152.77	0.00	2298.87	51.43	174.43	399914
28	-12.05	134.50	165.38	0.00	2410.95	54.67	189.17	399914
29	-13.20	146.00	177.96	0.00	2522.78	57.99	203.99	399914
30	-13.95	153.50	186.15	0.00	2595.60	60.31	213.81	399914
31	-14.70	161.00	194.33	0.00	2668.35	63.04	224.04	399914
32	-15.10	165.00	198.69	0.00	2707.12	64.40	229.40	399914
33	-16.30	177.00	211.76	0.00	2823.35	69.03	246.03	399914
34	-17.15	185.50	221.01	0.00	2905.60	72.29	257.79	399914
35	-18.00	194.00	230.26	0.00	2987.80	75.59	269.59	399914
36	-19.15	205.50	242.76	0.00	3098.95	80.11	285.61	399914
37	-20.30	217.00	255.25	0.00	3210.04	84.85	301.85	399914
38	-20.60	220.00	258.51	0.00	3239.02	85.82	305.82	399914
39	-24.70	261.00	303.03	0.00	3634.84	101.79	362.79	399914
40	-28.80	302.00	347.54	0.00	4030.63	117.80	419.80	399914
41	-33.60	350.00	399.69	0.00	4494.29	136.53	486.53	399914
42	-38.40	398.00	451.89	0.00	4958.42	155.28	553.28	399914
43	-43.08	444.75	502.78	0.00	5410.96	173.55	618.30	399914
44	-47.75	491.50	553.73	1.64	5863.98	191.81	683.31	399914

RIGHT side

Node no.	Y coord	Water press.	Effective stresses: Vertic -al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
1	4.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	4.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
3	4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
		0.00	0.00	0.00	4.97b	2.52	2.52	5999
4	2.90	0.00	18.15	1.71	26.16b	12.06	12.06	5999
5	2.50	0.00	24.75	3.75	32.14b	15.45	15.45	5999
6	1.40	0.00	42.90	9.36	51.81b	24.98	24.98	5999
7	0.70	0.00	54.45	12.93	63.96b	30.32	30.32	5999
8	0.60	0.00	56.10	13.44	65.55b	31.03	31.03	5999
9	0.00	0.00	66.00	16.50	74.37b	35.61	35.61	5999
10	-0.85	8.50	71.53	18.20	86.17b	38.42	46.92	5999
11	-1.70	17.00	77.05	19.91	104.74b	41.23	58.23	5999
12	-2.20	22.00	80.30	20.92	118.74b	42.85	64.85	5999
13	-2.50	25.00	82.25	21.52	125.58b	43.77	68.77	5999
		25.00	82.25	21.52	177.32b	43.77	68.77	5999
14	-3.25	32.50	87.13	23.02	178.18b	46.22	78.72	5999
15	-4.00	40.00	92.00	24.53	183.27b	48.64	88.64	5999
16	-4.65	46.50	96.23	25.84	202.56b	50.74	97.24	5999
17	-5.30	53.00	100.45	27.14	219.22b	52.90	105.90	5999
18	-5.70	57.00	103.05	27.95	229.59b	54.72	111.72	5999
		57.00	103.05	22.99	303.90b	55.61	112.61	19996
19	-6.00	60.00	105.30	23.63	285.52b	55.72	115.72	19996
20	-6.30	63.00	107.55	24.27	270.64b	55.91	118.91	19996
21	-6.90	69.00	112.05	25.54	283.64b	58.06	127.06	19996
		69.00	112.05	0.00	1936.77	100.41	169.41	399914
22	-7.75	77.50	121.40	0.00	2019.90	67.08	144.58	399914

(continued)

Stage No.4 Excavate to elevation 4.00 on RIGHT side
Toe of berm at elevation 0.00
Width of top of berm = 0.01
Width of toe of berm = 8.00

RIGHT side

Node no.	Y coord	Water press.	Effective stresses: Vertic-al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
23	-8.60	86.00	130.75	0.00	1916.67	52.30	138.30	399914
24	-9.80	98.00	143.95	0.00	1843.64b	53.18	151.18	399914
25	-10.30	103.00	149.45	0.00	1788.75b	54.78	157.78	399914
26	-10.70	107.00	153.85	0.00	1826.90b	56.77	163.77	399914
27	-10.90	109.00	156.05	0.00	1857.85b	58.43	167.43	399914
28	-12.05	120.50	168.70	0.00	1960.74b	64.16	184.66	399914
29	-13.20	132.00	181.35	0.00	2069.62b	69.81	201.81	399914
30	-13.95	139.50	189.60	0.00	2146.73b	73.34	212.84	399914
31	-14.70	147.00	197.85	0.00	2217.08b	76.47	223.47	399914
32	-15.10	151.00	202.25	0.00	2265.99b	78.23	229.23	399914
33	-16.30	163.00	215.45	0.00	2373.64b	82.97	245.97	399914
34	-17.15	171.50	224.80	0.00	2459.71b	86.35	257.85	399914
35	-18.00	180.00	234.15	0.00	2545.14b	89.69	269.69	399914
36	-19.15	191.50	246.80	0.00	2655.27b	94.15	285.65	399914
37	-20.30	203.00	259.45	0.00	2761.65b	98.39	301.39	399914
38	-20.60	206.00	262.75	0.00	2822.86b	99.76	305.76	399914
39	-24.70	247.00	307.85	0.00	3196.82b	115.79	362.79	399914
40	-28.80	288.00	352.95	0.00	3601.91b	131.79	419.79	399914
41	-33.60	336.00	405.75	0.00	4067.45b	150.53	486.53	399914
42	-38.40	384.00	458.55	0.00	4536.59b	169.28	553.28	399914
43	-43.08	430.75	509.98	0.00	4994.61b	187.55	618.30	399914
44	-47.75	477.50	561.40	3.02	5433.47b	205.86	683.36	399914

Note: 83.00 a Soil pressure at active limit
123.45 p Soil pressure at passive limit
5433.47 b Passive limit reduced because of berm

Units: kN,m

Stage No. 6 Excavate to elevation 0.00 on RIGHT side

STABILITY ANALYSIS of Fully Embedded Wall according to Strength Factor method
Factor of safety on soil strength

Stage No.	Ground level Act.	Ground level Pass.	Prop Elev.	FoS for toe elev. = -20.30: Factor of Safety	FoS for toe elev. = -20.30: Moment equilib. at elev.	Toe elev. for FoS = 1.000	Wall Penetr-ation	Direction of failure
6	4.00	0.00	4.30	12.455	n/a	-1.91	1.91	L to R

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall
Analysis options
Length of wall perpendicular to section = 1000.00m
2-D finite element model. Active limit arching modelled.
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Left side 60.00 from wall — Rough boundary
Right side 60.00 from wall — Rough boundary
Lower rigid boundary at elevation -47.75 — Rough boundary

*** Wall displacements reset to zero at stage 1

Node no.	Y coord	Nett pressure	Wall disp.	Wall rotation	Shear force	Bending moment	Prop forces	EI of wall
		kN/m2	m	rad.	kN/m	kN.m/m	kN/m	kN.m2/m
1	4.40	0.00	0.007	-8.80E-04	0.0	0.0		877500
2	4.30	0.00	0.007	-8.80E-04	0.0	-0.0	-89.2	877500
		0.00	0.007	-8.80E-04	-89.2	0.0		
3	4.00	0.00	0.008	-8.75E-04	-89.2	-26.8		877500
		3.33	0.008	-8.75E-04	-89.2	-26.8		
4	2.90	8.82	0.009	-7.83E-04	-82.5	-120.9		877500
5	2.50	10.72	0.009	-7.21E-04	-78.6	-153.1		877500
6	1.40	20.04	0.010	-4.80E-04	-61.7	-230.9		877500
7	0.70	29.14	0.010	-2.81E-04	-44.5	-268.4		877500
8	0.60	30.12	0.010	-2.50E-04	-41.5	-272.7		877500
9	0.00	38.19	0.010	-5.74E-05	-21.0	-291.7		877500
		29.93	0.010	-5.74E-05	-21.0	-291.7		
10	-0.85	28.22	0.010	2.28E-04	3.7	-298.7		877500
11	-1.70	27.67	0.010	5.11E-04	27.5	-285.3		877500
12	-2.20	27.69	0.009	6.68E-04	41.3	-268.1		877500
13	-2.50	28.01	0.009	7.58E-04	49.7	-254.4		877500
14	-3.25	28.61	0.008	9.56E-04	70.9	-209.3		877500
15	-4.00	29.44	0.008	1.10E-03	92.7	-148.1		877500
16	-4.65	30.19	0.007	1.19E-03	112.0	-81.6		877500
17	-5.30	30.52	0.006	1.22E-03	131.8	-2.4		877500
18	-5.70	28.53	0.006	1.21E-03	143.6	52.7		877500
		-21.25	0.006	1.21E-03	143.6	52.7		
19	-6.00	-8.93	0.005	1.18E-03	139.0	94.8		877500
20	-6.30	-0.06	0.005	1.14E-03	137.7	136.2		877500
21	-6.90	6.83	0.004	1.02E-03	139.7	218.8		877500
		-218.13	0.004	1.02E-03	139.7	218.8		
22	-7.75	-107.34	0.003	7.96E-04	1.4	258.8		877500
23	-8.60	-28.41	0.003	5.64E-04	-56.3	221.2		877500
24	-9.80	11.80	0.002	3.21E-04	-66.3	133.2		877500
25	-10.30	23.12	0.002	2.55E-04	-57.5	101.5		877500
26	-10.70	23.38	0.002	2.13E-04	-48.2	80.3		877500

(continued)

Stage No.6 Excavate to elevation 0.00 on RIGHT side

Node no.	Y coord	Nett pressure	Wall disp.	Wall rotation	Shear force	Bending moment	Prop forces	EI of wall
		kN/m2	m	rad.	kN/m	kN.m/m	kN/m	kN.m2/m
27	-10.90	20.55	0.002	1.96E-04	-43.8	71.2		877500
28	-12.05	14.71	0.002	1.27E-04	-23.6	34.3		877500
29	-13.20	8.19	0.002	9.35E-05	-10.4	16.9		877500
30	-13.95	4.24	0.002	8.14E-05	-5.7	11.4		877500
31	-14.70	2.72	0.002	7.30E-05	-3.1	8.3		877500
32	-15.10	1.16	0.002	6.94E-05	-2.3	7.3		877500
33	-16.30	0.61	0.002	6.08E-05	-1.3	5.3		877500
34	-17.15	-0.06	0.001	5.61E-05	-1.0	4.4		877500
35	-18.00	-0.35	0.001	5.23E-05	-1.2	3.5		877500
36	-19.15	-0.14	0.001	4.88E-05	-1.5	1.8		877500
37	-20.30	2.19	0.001	4.75E-05	-0.3	0.0		0
38	-20.60	0.12	0.001	0	0.0	0.0		0
39	-24.70	0.01	0.001	0	0.3	0.0		0
40	-28.80	0.04	0.001	0	0.4	0.0		0
41	-33.60	0.01	0.001	0	0.5	0.0		0
42	-38.40	-0.00	0.001	0	0.5	0.0		0
43	-43.08	0.00	0.000	0	0.5	0.0		0
44	-47.75	-0.22	0.000	0	-0.0	0.0		---

At elev. 4.30 Prop force = 89.2 kN/m run (horiz.)
= 100.1 kN/m run (inclined)

LEFT side

Node no.	Y coord	Water press.	Effective stresses: Vertic-al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
1	4.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	4.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
3	4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
		0.00	0.00	0.00	16.18	3.33	3.33	5999
4	2.90	0.00	18.76	1.90	88.77	8.82	8.82	5999
5	2.50	0.00	26.00	4.14	116.75	10.72	10.72	5999
6	1.40	0.00	46.19	10.38	194.86	20.04	20.04	5999
7	0.70	7.00	51.88	12.13	216.89	22.14	29.14	5999
8	0.60	8.00	52.68	12.38	219.97	22.12	30.12	5999
9	0.00	14.00	57.37	13.83	238.12	24.19	38.19	5999
10	-0.85	22.50	63.79	15.81	262.95	27.20	49.70	5999
11	-1.70	31.00	70.00	17.73	286.97	30.33	61.33	5999
12	-2.20	36.00	73.58	18.84	300.80	32.25	68.25	5999
13	-2.50	39.00	75.70	19.49	309.01	33.53	72.53	5999
		39.00	75.70	19.49	348.27	33.53	72.53	5999
14	-3.25	46.50	80.94	21.11	371.18	36.51	83.01	5999
15	-4.00	54.00	86.11	22.71	393.76	39.56	93.56	5999
16	-4.65	60.50	90.54	24.08	413.12	42.17	102.67	5999
17	-5.30	67.00	94.93	25.44	432.31	44.52	111.52	5999
18	-5.70	71.00	97.62	26.27	444.04	44.73	115.73	5999
		71.00	97.62	17.04	528.95	4.61	75.61A	39991
19	-6.00	74.00	100.38	17.75	542.63	15.00	89.00A	39991
20	-6.30	77.00	103.13	18.47	556.28	22.45	99.45	39991
21	-6.90	83.00	108.61	19.89	583.51	30.31	113.31	39991
		83.00	108.61	0.00	1906.20	0.00	83.00a	399914
22	-7.75	91.50	118.05	0.00	1990.12	0.00	91.50a	399914
23	-8.60	100.00	127.46	0.00	2073.75	14.81	114.81	399914
24	-9.80	112.00	140.69	0.00	2191.39	39.17	151.17	399914
25	-10.30	117.00	146.19	0.00	2240.29	46.58	163.58	399914
26	-10.70	121.00	150.58	0.00	2279.35	48.22	169.22	399914
27	-10.90	123.00	152.77	0.00	2298.87	47.51	170.51	399914

Run ID. 1_North_800DWall_topdown+propped_gw2 | Sheet No.
Downtown Carpark Redevelopment | Date:30-05-2024
800mm D-Wall - Top Down + Propped | Checked :

(continued)

Stage No.6 Excavate to elevation 0.00 on RIGHT side

LEFT side

Node no.	Y coord	Water press.	Effective stresses: Vertic-al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
28	-12.05	134.50	165.38	0.00	2410.95	49.02	183.52	399914
29	-13.20	146.00	177.96	0.00	2522.78	50.21	196.21	399914
30	-13.95	153.50	186.15	0.00	2595.60	51.14	204.64	399914
31	-14.70	161.00	194.33	0.00	2668.35	53.30	214.30	399914
32	-15.10	165.00	198.69	0.00	2707.12	54.06	219.06	399914
33	-16.30	177.00	211.76	0.00	2823.35	58.48	235.48	399914
34	-17.15	185.50	221.01	0.00	2905.60	61.46	246.96	399914
35	-18.00	194.00	230.26	0.00	2987.80	64.64	258.64	399914
36	-19.15	205.50	242.76	0.00	3098.95	69.25	274.75	399914
37	-20.30	217.00	255.25	0.00	3210.04	74.92	291.92	399914
38	-20.60	220.00	258.51	0.00	3239.02	75.05	295.05	399914
39	-24.70	261.00	303.03	0.00	3634.84	91.09	352.09	399914
40	-28.80	302.00	347.54	0.00	4030.63	107.22	409.22	399914
41	-33.60	350.00	399.69	0.00	4494.29	126.13	476.13	399914
42	-38.40	398.00	451.89	0.00	4958.42	145.10	543.10	399914
43	-43.08	444.75	502.78	0.00	5410.96	163.61	608.36	399914
44	-47.75	491.50	553.73	1.64	5863.98	182.03	673.53	399914

RIGHT side

Node no.	Y coord	Water press.	Effective stresses: Vertic-al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
1	4.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	4.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
3	4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
4	2.90	0.00	0.00	0.00	0.00	0.00	0.00	0.0
5	2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.0
6	1.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
7	0.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
8	0.60	0.00	0.00	0.00	0.00	0.00	0.00	0.0
9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
		0.00	0.00	0.00	16.18	8.26	8.26	5999
10	-0.85	8.50	5.53	0.00	37.56	12.98	21.48	5999
11	-1.70	17.00	11.05	0.00	58.93	16.66	33.66	5999
12	-2.20	22.00	14.30	0.52	71.51	18.55	40.55	5999
13	-2.50	25.00	16.25	1.12	79.05	19.52	44.52	5999
		25.00	16.25	1.12	88.52	19.52	44.52	5999
14	-3.25	32.50	21.13	2.63	109.83	21.90	54.40	5999
15	-4.00	40.00	26.01	4.14	131.15	24.12	64.12	5999
16	-4.65	46.50	30.24	5.45	149.63	25.98	72.48	5999
17	-5.30	53.00	34.47	6.75	168.12	28.00	81.00	5999
18	-5.70	57.00	37.07	7.56	179.50	30.20	87.20	5999
		57.00	37.07	4.30	191.17	39.86	96.86	19996
19	-6.00	60.00	39.33	4.94	201.02	37.93	97.93	19996
20	-6.30	63.00	41.58	5.58	210.87	36.51	99.51	19996
21	-6.90	69.00	46.09	6.85	230.57	37.48	106.48	19996
		69.00	46.09	0.00	1350.30	232.13	301.13	399925
22	-7.75	77.50	55.46	0.00	1433.59	121.34	198.84	399925
23	-8.60	86.00	64.83	0.00	1516.92	57.23	143.23	399925
24	-9.80	98.00	78.07	0.00	1634.62	41.37	139.37	399925
25	-10.30	103.00	83.59	0.00	1683.69	37.46	140.46	399925
26	-10.70	107.00	88.00	0.00	1722.95	38.83	145.83	399925
27	-10.90	109.00	90.21	0.00	1742.59	40.95	149.95	399925

(continued)

Stage No.6 Excavate to elevation 0.00 on RIGHT side

RIGHT side

Node no.	Y coord	Water press.	Effective stresses: Vertic-al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
28	-12.05	120.50	102.92	0.00	1855.56	48.31	168.81	399925
29	-13.20	132.00	115.63	0.00	1968.62	56.02	188.02	399925
30	-13.95	139.50	123.93	0.00	2042.40	60.89	200.39	399925
31	-14.70	147.00	132.23	0.00	2116.24	64.58	211.58	399925
32	-15.10	151.00	136.66	0.00	2155.63	66.91	217.91	399925
33	-16.30	163.00	149.97	0.00	2273.90	71.87	234.87	399925
34	-17.15	171.50	159.40	0.00	2357.74	75.53	247.03	399925
35	-18.00	180.00	168.83	0.00	2441.64	78.99	258.99	399925
36	-19.15	191.50	181.61	0.00	2555.27	83.39	274.89	399925
37	-20.30	203.00	194.40	0.00	2669.00	86.73	289.73	399925
38	-20.60	206.00	197.74	0.00	2698.69	88.93	294.93	399925
39	-24.70	247.00	243.47	0.00	3105.27	105.08	352.08	399925
40	-28.80	288.00	289.36	0.00	3513.34	121.19	409.19	399925
41	-33.60	336.00	343.29	0.00	3992.82	140.13	476.13	399925
42	-38.40	384.00	397.39	0.00	4473.91	159.10	543.10	399925
43	-43.08	430.75	450.23	0.00	4943.71	177.60	608.35	399925
44	-47.75	477.50	503.17	0.00	5414.43	196.25	673.75	399925

Note: 91.50 a Soil pressure at active limit
123.45 p Soil pressure at passive limit
89.00A Arching - soil pressure below active limit

Units: kN,m

Stage No. 9 Excavate to elevation -6.00 on RIGHT side

STABILITY ANALYSIS of Fully Embedded Wall according to Strength Factor method
Factor of safety on soil strength

Stage No.	Ground level Act.	Ground level Pass.	Prop Elev.	FoS for toe elev. = -20.30: Factor of Safety	Moment equilib. at elev.	Toe elev. for FoS = 1.000: Toe elev.	Wall Penetr-ation	Direction of failure
9	4.00	-6.00		More than one prop. No FoS calc.				

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall
Analysis options
Length of wall perpendicular to section = 1000.00m
2-D finite element model. Active limit arching modelled.
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Left side 60.00 from wall — Rough boundary
Right side 60.00 from wall — Rough boundary
Lower rigid boundary at elevation -47.75 — Rough boundary

*** Wall displacements reset to zero at stage 1

Node no.	Y coord	Nett pressure	Wall disp.	Wall rotation	Shear force	Bending moment	Prop forces	EI of wall
		kN/m2	m	rad.	kN/m	kN.m/m	kN/m	kN.m2/m
1	4.40	0.00	0.006	-1.28E-03	0.0	0.0		877500
2	4.30	0.00	0.006	-1.28E-03	0.0	-0.0	-17.0	877500
		0.00	0.006	-1.28E-03	-17.0	0.0		
3	4.00	0.00	0.006	-1.28E-03	-17.0	-5.1		877500
		6.21	0.006	-1.28E-03	-17.0	-5.1		
4	2.90	11.99	0.008	-1.27E-03	-7.0	-18.0		877500
5	2.50	13.92	0.008	-1.26E-03	-1.8	-19.7		877500
6	1.40	23.05	0.010	-1.24E-03	18.6	-11.2		877500
7	0.70	31.85	0.011	-1.24E-03	37.8	8.3	-259.7	877500
		31.85	0.011	-1.24E-03	-222.0	8.3		
8	0.60	32.36	0.011	-1.24E-03	-218.8	-13.7		877500
9	0.00	39.71	0.011	-1.19E-03	-197.1	-138.7		877500
10	-0.85	50.03	0.012	-9.83E-04	-159.0	-290.3		877500
11	-1.70	60.53	0.013	-6.46E-04	-112.0	-405.8		877500
12	-2.20	66.65	0.013	-4.01E-04	-80.2	-454.0		877500
13	-2.50	70.27	0.013	-2.42E-04	-59.7	-475.0		877500
14	-3.25	80.14	0.013	1.73E-04	-3.3	-499.0		877500
15	-4.00	90.13	0.013	5.91E-04	60.6	-478.0		877500
16	-4.65	98.91	0.013	9.23E-04	122.0	-419.0		877500
17	-5.30	107.54	0.012	1.19E-03	189.1	-318.2		877500
18	-5.70	113.25	0.011	1.32E-03	233.3	-233.8		877500
		71.00	0.011	1.32E-03	233.3	-233.8		
19	-6.00	74.00	0.011	1.38E-03	255.0	-160.6		877500
		46.91	0.011	1.38E-03	255.0	-160.6		
20	-6.30	48.25	0.011	1.43E-03	269.3	-81.9		877500
21	-6.90	59.39	0.010	1.42E-03	301.6	88.3		877500
		-304.58	0.010	1.42E-03	301.6	88.3		
22	-7.75	-170.66	0.009	1.27E-03	99.6	234.7		877500
23	-8.60	-66.33	0.008	1.03E-03	-1.1	257.7		877500
24	-9.80	-22.08	0.007	7.14E-04	-54.1	208.6		877500
25	-10.30	-1.58	0.006	6.04E-04	-60.1	178.8		877500

(continued)

Stage No.9 Excavate to elevation -6.00 on RIGHT side

Node no.	Y coord	Nett pressure	Wall disp.	Wall rotation	Shear force	Bending moment	Prop forces	EI of wall
		kN/m2	m	rad.	kN/m	kN.m/m	kN/m	kN.m2/m
26	-10.70	5.50	0.006	5.28E-04	-59.3	154.6		877500
27	-10.90	13.53	0.006	4.94E-04	-57.4	142.9		877500
28	-12.05	15.96	0.005	3.44E-04	-40.4	85.8		877500
29	-13.20	12.82	0.005	2.55E-04	-23.9	49.9		877500
30	-13.95	8.57	0.005	2.19E-04	-15.9	35.6		877500
31	-14.70	6.36	0.005	1.92E-04	-10.3	26.1		877500
32	-15.10	3.59	0.005	1.81E-04	-8.3	22.5		877500
33	-16.30	2.22	0.004	1.56E-04	-4.8	15.1		877500
34	-17.15	0.50	0.004	1.43E-04	-3.6	11.9		877500
35	-18.00	-0.40	0.004	1.32E-04	-3.6	9.0		877500
36	-19.15	-0.11	0.004	1.24E-04	-3.9	4.6		877500
37	-20.30	5.43	0.004	1.21E-04	-0.8	0.0		0
38	-20.60	0.16	0.004	0	0.0	0.0		0
39	-24.70	0.02	0.003	0	0.4	0.0		0
40	-28.80	0.07	0.003	0	0.6	0.0		0
41	-33.60	0.03	0.003	0	0.8	0.0		0
42	-38.40	0.01	0.002	0	0.9	0.0		0
43	-43.08	0.02	0.001	0	1.0	0.0		0
44	-47.75	-0.44	0.000	0	0.0	0.0		---

At elev. 4.30 Prop force = 17.0 kN/m run (horiz.)
= 19.0 kN/m run (inclined)

At elev. 0.70 Prop force = 259.7 kN/m run

		LEFT side						
			Effective stresses				Total	Adjusted
Node no.	Y coord	Water press.	Vertic -al	Active limit	Passive limit	Earth pressure	earth pressure	soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
1	4.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	4.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
3	4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
		0.00	0.00	0.00	16.18	6.21	6.21	5999
4	2.90	0.00	18.76	1.90	88.77	11.99	11.99	5999
5	2.50	0.00	26.00	4.14	116.75	13.92	13.92	5999
6	1.40	0.00	46.19	10.38	194.86	23.05	23.05	5999
7	0.70	7.00	51.88	12.13	216.89	24.85	31.85	5999
8	0.60	8.00	52.68	12.38	219.97	24.36	32.36	5999
9	0.00	14.00	57.37	13.83	238.12	25.71	39.71	5999
10	-0.85	22.50	63.79	15.81	262.95	27.53	50.03	5999
11	-1.70	31.00	70.00	17.73	286.97	29.53	60.53	5999
12	-2.20	36.00	73.58	18.84	300.80	30.65	66.65	5999
13	-2.50	39.00	75.70	19.49	309.01	31.27	70.27	5999
		39.00	75.70	19.49	348.27	31.27	70.27	5999
14	-3.25	46.50	80.94	21.11	371.18	33.64	80.14	5999
15	-4.00	54.00	86.11	22.71	393.76	36.13	90.13	5999
16	-4.65	60.50	90.54	24.08	413.12	38.41	98.91	5999
17	-5.30	67.00	94.93	25.44	432.31	40.54	107.54	5999
18	-5.70	71.00	97.62	26.27	444.04	42.25	113.25	5999
		71.00	97.62	17.04	528.95	0.00	71.00A	39991
19	-6.00	74.00	100.38	17.75	542.63	0.00	74.00A	39991
20	-6.30	77.00	103.13	18.47	556.28	2.74	79.74A	39991
21	-6.90	83.00	108.61	19.89	583.51	13.98	96.98A	39991
		83.00	108.61	0.00	1906.20	0.00	83.00a	399914
22	-7.75	91.50	118.05	0.00	1990.12	0.00	91.50a	399914
23	-8.60	100.00	127.46	0.00	2073.75	0.00	100.00a	399914
24	-9.80	112.00	140.69	0.00	2191.39	0.00	112.00a	399914
25	-10.30	117.00	146.19	0.00	2240.29	0.00	117.00a	399914

Run ID. 1_North_800DWall_topdown+propped_gw2 | Sheet No.
Downtown Carpark Redevelopment | Date:30-05-2024
800mm D-Wall - Top Down + Propped | Checked :

(continued)

Stage No.9 Excavate to elevation -6.00 on RIGHT side

		LEFT side						
			Effective stresses				Total	Adjusted
Node no.	Y coord	Water press.	Vertic -al	Active limit	Passive limit	Earth pressure	earth pressure	soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
26	-10.70	121.00	150.58	0.00	2279.35	2.79	123.79	399914
27	-10.90	123.00	152.77	0.00	2298.87	7.52	130.52	399914
28	-12.05	134.50	165.38	0.00	2410.95	13.15	147.65	399914
29	-13.20	146.00	177.96	0.00	2522.78	16.02	162.02	399914
30	-13.95	153.50	186.15	0.00	2595.60	16.79	170.29	399914
31	-14.70	161.00	194.33	0.00	2668.35	18.61	179.61	399914
32	-15.10	165.00	198.69	0.00	2707.12	18.77	183.77	399914
33	-16.30	177.00	211.76	0.00	2823.35	22.77	199.77	399914
34	-17.15	185.50	221.01	0.00	2905.60	25.23	210.73	399914
35	-18.00	194.00	230.26	0.00	2987.80	28.10	222.10	399914
36	-19.15	205.50	242.76	0.00	3098.95	32.75	238.25	399914
37	-20.30	217.00	255.25	0.00	3210.04	40.03	257.03	399914
38	-20.60	220.00	258.51	0.00	3239.02	38.56	258.56	399914
39	-24.70	261.00	303.03	0.00	3634.84	54.63	315.63	399914
40	-28.80	302.00	347.54	0.00	4030.63	70.86	372.86	399914
41	-33.60	350.00	399.69	0.00	4494.29	89.92	439.92	399914
42	-38.40	398.00	451.89	0.00	4958.42	109.10	507.10	399914
43	-43.08	444.75	502.78	0.00	5410.96	127.88	572.63	399914
44	-47.75	491.50	553.73	1.64	5863.98	146.51	638.01	399914

		RIGHT side						
			Effective stresses				Total	Adjusted
Node no.	Y coord	Water press.	Vertic -al	Active limit	Passive limit	Earth pressure	earth pressure	soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
1	4.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	4.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
3	4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
4	2.90	0.00	0.00	0.00	0.00	0.00	0.00	0.0
5	2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.0
6	1.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
7	0.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
8	0.60	0.00	0.00	0.00	0.00	0.00	0.00	0.0
9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
10	-0.85	0.00	0.00	0.00	0.00	0.00	0.00	0.0
11	-1.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
12	-2.20	0.00	0.00	0.00	0.00	0.00	0.00	0.0
13	-2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.0
14	-3.25	0.00	0.00	0.00	0.00	0.00	0.00	0.0
15	-4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
16	-4.65	0.00	0.00	0.00	0.00	0.00	0.00	0.0
17	-5.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
18	-5.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
19	-6.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
		0.00	0.00	0.00	29.18	27.09	27.09	19997
20	-6.30	3.00	2.25	0.00	39.01	28.48	31.48	19997
21	-6.90	9.00	6.75	0.00	58.67	28.60	37.60	19997
		9.00	6.75	0.00	1000.49	378.58	387.58	399934
22	-7.75	17.50	16.10	0.00	1083.64	244.66	262.16	399934
23	-8.60	26.00	25.46	0.00	1166.81	140.33	166.33	399934
24	-9.80	38.00	38.67	0.00	1284.29	96.08	134.08	399934
25	-10.30	43.00	44.18	0.00	1333.26	75.58	118.58	399934
26	-10.70	47.00	48.58	0.00	1372.45	71.29	118.29	399934
27	-10.90	49.00	50.79	0.00	1392.06	67.99	116.99	399934

(continued)

Stage No.9 Excavate to elevation -6.00 on RIGHT side

RIGHT side

Node no.	Y coord	Water press.	Effective stresses: Vertic-al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
28	-12.05	60.50	63.47	0.00	1504.83	71.20	131.70	399934
29	-13.20	72.00	76.17	0.00	1617.73	77.20	149.20	399934
30	-13.95	79.50	84.46	0.00	1691.45	82.23	161.73	399934
31	-14.70	87.00	92.76	0.00	1765.24	86.24	173.24	399934
32	-15.10	91.00	97.19	0.00	1804.62	89.18	180.18	399934
33	-16.30	103.00	110.49	0.00	1922.92	94.55	197.55	399934
34	-17.15	111.50	119.93	0.00	2006.85	98.72	210.22	399934
35	-18.00	120.00	129.39	0.00	2090.91	102.50	222.50	399934
36	-19.15	131.50	142.20	0.00	2204.83	106.85	238.35	399934
37	-20.30	143.00	155.04	0.00	2319.00	108.59	251.59	399934
38	-20.60	146.00	158.39	0.00	2348.83	112.40	258.40	399934
39	-24.70	187.00	204.45	0.00	2758.34	128.60	315.60	399934
40	-28.80	228.00	250.92	0.00	3171.55	144.79	372.79	399934
41	-33.60	276.00	305.87	0.00	3660.09	163.90	439.90	399934
42	-38.40	324.00	361.36	0.00	4153.49	183.09	507.09	399934
43	-43.08	370.75	415.86	0.00	4638.12	201.87	572.62	399934
44	-47.75	417.50	470.74	0.00	5126.04	220.94	638.44	399934

Note: 117.00 a Soil pressure at active limit
123.45 p Soil pressure at passive limit
96.98A Arching - soil pressure below active limit

Units: kN,m

Stage No. 12 Excavate to elevation -10.70 on RIGHT side
Toe of berm at elevation -16.30
Width of top of berm = 0.30
Width of toe of berm = 5.90

STABILITY ANALYSIS of Fully Embedded Wall according to Strength Factor method
Factor of safety on soil strength

Stage No.	Ground level Act.	Ground level Pass.	Prop Elev.	FoS for toe elev. = -20.30: Factor of Safety	Moment equilib. at elev.	Toe elev. for FoS = 1.000	Wall Penetration	Direction of failure
12	4.00	-10.70		More than one prop. No FoS calc.				

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall
Analysis options
Length of wall perpendicular to section = 1000.00m
2-D finite element model. Active limit arching modelled.
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Left side 60.00 from wall Rough boundary
Right side 60.00 from wall Rough boundary
Lower rigid boundary at elevation -47.75 Rough boundary

*** Wall displacements reset to zero at stage 1

Node no.	Y coord	Nett pressure	Wall disp.	Wall rotation	Shear force	Bending moment	Prop forces	EI of wall
		kN/m2	m	rad.	kN/m	kN.m/m	kN/m	kN.m2/m
1	4.40	0.00	0.006	-1.19E-03	0.0	0.0		877500
2	4.30	0.00	0.006	-1.19E-03	0.0	-0.0	-25.3	877500
		0.00	0.006	-1.19E-03	-25.3	0.0		
3	4.00	0.00	0.006	-1.19E-03	-25.3	-7.6		877500
		6.68	0.006	-1.19E-03	-25.3	-7.6		
4	2.90	12.73	0.008	-1.17E-03	-14.6	-29.3		877500
5	2.50	15.02	0.008	-1.15E-03	-9.0	-34.1		877500
6	1.40	24.45	0.010	-1.11E-03	12.7	-32.8		877500
7	0.70	33.53	0.010	-1.09E-03	33.0	-17.1	-176.0	877500
		33.53	0.010	-1.09E-03	-143.0	-17.1		
8	0.60	34.31	0.010	-1.09E-03	-139.6	-31.3		877500
9	0.00	42.03	0.011	-1.04E-03	-116.7	-108.4		877500
10	-0.85	52.95	0.012	-9.02E-04	-76.3	-190.7		877500
11	-1.70	64.03	0.013	-6.96E-04	-26.6	-235.0		877500
12	-2.20	70.58	0.013	-5.61E-04	7.0	-240.0		877500
13	-2.50	74.57	0.013	-4.79E-04	28.8	-234.7		877500
14	-3.25	84.88	0.013	-2.97E-04	88.6	-191.1		877500
15	-4.00	95.37	0.013	-1.73E-04	156.2	-99.9		877500
16	-4.65	104.59	0.014	-1.44E-04	221.2	22.4		877500
17	-5.30	113.74	0.014	-2.23E-04	292.1	188.8	-650.2	877500
		113.74	0.014	-2.23E-04	-358.1	188.8		
18	-5.70	121.25	0.014	-2.78E-04	-311.1	54.9		877500
		124.34	0.014	-2.78E-04	-311.1	54.9		
19	-6.00	112.87	0.014	-2.82E-04	-275.5	-32.8		877500
20	-6.30	105.55	0.014	-2.57E-04	-242.7	-110.4		877500
21	-6.90	108.90	0.014	-1.39E-04	-178.4	-236.8		877500
		246.70	0.014	-1.39E-04	-178.4	-236.8		
22	-7.75	75.18	0.014	1.28E-04	-41.6	-315.0		877500

(continued)

Stage No.12 Excavate to elevation -10.70 on RIGHT side
Toe of berm at elevation -16.30
Width of top of berm = 0.30
Width of toe of berm = 5.90

Node no.	Y coord	Nett pressure	Wall disp.	Wall rotation	Shear force	Bending moment	Prop forces	EI of wall
		kN/m2	m	rad.	kN/m	kN.m/m	kN/m	kN.m2/m
23	-8.60	76.19	0.014	4.43E-04	22.8	-335.3		877500
24	-9.80	77.60	0.013	8.51E-04	115.0	-262.6		877500
25	-10.30	78.19	0.013	9.82E-04	154.0	-196.1		877500
26	-10.70	78.67	0.012	1.05E-03	185.4	-128.4		877500
		-162.33	0.012	1.05E-03	185.4	-128.4		
27	-10.90	-51.22	0.012	1.08E-03	164.0	-94.5		877500
28	-12.05	-48.95	0.011	1.10E-03	106.4	60.4		877500
29	-13.20	-68.24	0.010	9.66E-04	39.0	150.6		877500
30	-13.95	-51.85	0.009	8.33E-04	-6.0	160.7		877500
31	-14.70	-19.94	0.008	7.03E-04	-32.9	141.7		877500
32	-15.10	0.03	0.008	6.42E-04	-36.9	126.9		877500
33	-16.30	9.31	0.007	4.99E-04	-31.3	82.8		877500
34	-17.15	7.83	0.007	4.30E-04	-24.0	59.0		877500
35	-18.00	3.64	0.007	3.82E-04	-19.2	40.7		877500
36	-19.15	1.15	0.006	3.42E-04	-16.4	19.6		877500
37	-20.30	21.72	0.006	3.29E-04	-3.3	0.0		0
38	-20.60	0.15	0.006	0	0.0	0.0		0
39	-24.70	0.03	0.005	0	0.4	0.0		0
40	-28.80	0.09	0.004	0	0.6	0.0		0
41	-33.60	0.05	0.003	0	1.0	0.0		0
42	-38.40	0.03	0.003	0	1.1	0.0		0
43	-43.08	0.04	0.002	0	1.3	0.0		0
44	-47.75	-0.59	0.000	0	0.0	0.0		---

At elev. 4.30 Prop force = 25.3 kN/m run (horiz.)
= 28.3 kN/m run (inclined)
At elev. 0.70 Prop force = 176.0 kN/m run
At elev. -5.30 Prop force = 650.2 kN/m run

LEFT side

Node no.	Y coord	Effective stresses: Water press.	Vertic -al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
1	4.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	4.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
3	4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
		0.00	0.00	0.00	16.18	6.68	6.68	5999
4	2.90	0.00	18.76	1.90	88.77	12.73	12.73	5999
5	2.50	0.00	26.00	4.14	116.75	15.02	15.02	5999
6	1.40	0.00	46.19	10.38	194.86	24.45	24.45	5999
7	0.70	7.00	51.88	12.13	216.89	26.53	33.53	5999
8	0.60	8.00	52.68	12.38	219.97	26.31	34.31	5999
9	0.00	14.00	57.37	13.83	238.12	28.03	42.03	5999
10	-0.85	22.50	63.79	15.81	262.95	30.45	52.95	5999
11	-1.70	31.00	70.00	17.73	286.97	33.03	64.03	5999
12	-2.20	36.00	73.58	18.84	300.80	34.58	70.58	5999
13	-2.50	39.00	75.70	19.49	309.01	35.57	74.57	5999
		39.00	75.70	19.49	348.27	35.57	74.57	5999
14	-3.25	46.50	80.94	21.11	371.18	38.38	84.88	5999
15	-4.00	54.00	86.11	22.71	393.76	41.37	95.37	5999
16	-4.65	60.50	90.54	24.08	413.12	44.09	104.59	5999
17	-5.30	67.00	94.93	25.44	432.31	46.74	113.74	5999
18	-5.70	71.00	97.62	26.27	444.04	50.25	121.25	5999
		71.00	97.62	17.04	528.95	53.34	124.34	39991

Run ID. 1_North_800DWall_topdown+propped_gw2 | Sheet No.
Downtown Carpark Redevelopment | Date:30-05-2024
800mm D-Wall - Top Down + Propped | Checked :

(continued)

Stage No.12 Excavate to elevation -10.70 on RIGHT side
Toe of berm at elevation -16.30
Width of top of berm = 0.30
Width of toe of berm = 5.90

LEFT side

Node no.	Y coord	Effective stresses: Water press.	Vertic -al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
19	-6.00	73.12	101.26	17.98	547.01	39.75	112.87	39991
20	-6.30	73.47	106.65	19.38	573.79	32.08	105.55	39991
21	-6.90	74.18	117.43	22.18	627.29	34.72	108.90	39991
		74.18	117.43	0.00	1984.62	172.52	246.70	399914
22	-7.75	75.18	134.37	0.00	2135.20	0.00	75.18a	399914
23	-8.60	76.19	151.27	0.00	2285.48	0.00	76.19a	399914
24	-9.80	77.60	175.08	0.00	2497.22	0.00	77.60a	399914
25	-10.30	78.19	184.99	0.00	2585.33	0.00	78.19a	399914
26	-10.70	78.67	192.91	0.00	2655.76	0.00	78.67a	399914
27	-10.90	78.90	196.87	0.00	2690.97	0.00	78.90a	399914
28	-12.05	80.26	219.62	0.00	2893.22	0.00	80.26a	399914
29	-13.20	81.62	242.34	0.00	3095.23	0.00	81.62a	399914
30	-13.95	82.50	257.14	0.00	3226.87	3.24	85.75	399914
31	-14.70	83.39	271.94	0.00	3358.43	22.61	106.00	399914
32	-15.10	83.86	279.83	0.00	3428.57	34.06	117.92	399914
33	-16.30	85.28	303.48	0.00	3638.90	45.03	130.31	399914
34	-17.15	86.28	320.23	0.00	3787.80	51.44	137.72	399914
35	-18.00	87.28	336.97	0.00	3936.66	56.70	143.98	399914
36	-19.15	88.64	359.61	0.00	4137.99	65.77	154.42	399914
37	-20.30	90.00	382.25	0.00	4339.27	86.63	176.63	399914
38	-20.60	93.00	385.51	0.00	4368.24	76.28	169.28	399914
39	-24.70	134.00	430.03	0.00	4764.06	91.21	225.21	399914
40	-28.80	175.00	474.54	0.00	5159.86	106.49	281.49	399914
41	-33.60	223.00	526.69	0.00	5623.52	125.08	348.08	399914
42	-38.40	271.00	578.89	6.16	6087.64	144.12	415.12	399914
43	-43.08	317.75	629.78	15.30	6540.18	163.01	480.76	399914
44	-47.75	364.50	680.73	24.45	6993.21	181.81	546.31	399914

RIGHT side

Node no.	Y coord	Effective stresses: Water press.	Vertic -al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
1	4.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	4.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
3	4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
4	2.90	0.00	0.00	0.00	0.00	0.00	0.00	0.0
5	2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.0
6	1.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
7	0.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
8	0.60	0.00	0.00	0.00	0.00	0.00	0.00	0.0
9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
10	-0.85	0.00	0.00	0.00	0.00	0.00	0.00	0.0
11	-1.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
12	-2.20	0.00	0.00	0.00	0.00	0.00	0.00	0.0
13	-2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.0
14	-3.25	0.00	0.00	0.00	0.00	0.00	0.00	0.0
15	-4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
16	-4.65	0.00	0.00	0.00	0.00	0.00	0.00	0.0
17	-5.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
18	-5.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0

(continued)

Stage No.12 Excavate to elevation -10.70 on RIGHT side
Toe of berm at elevation -16.30
Width of top of berm = 0.30
Width of toe of berm = 5.90

RIGHT side

Node no.	Y coord	Water press.	Effective stresses: Vertic-al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
19	-6.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
20	-6.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
21	-6.90	0.00	0.00	0.00	0.00	0.00	0.00	0.0
22	-7.75	0.00	0.00	0.00	0.00	0.00	0.00	0.0
23	-8.60	0.00	0.00	0.00	0.00	0.00	0.00	0.0
24	-9.80	0.00	0.00	0.00	0.00	0.00	0.00	0.0
25	-10.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
26	-10.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
		0.00	0.00	0.00	252.04b	241.00	241.00	399946
27	-10.90	0.00	4.20	0.00	130.12b	130.12	130.12p	399946
28	-12.05	0.00	28.35	0.00	129.21b	129.21	129.21p	399946
29	-13.20	0.00	52.51	0.00	149.86b	149.86	149.86p	399946
30	-13.95	0.00	68.27	0.00	162.59b	137.60	137.60	399946
31	-14.70	0.00	84.03	0.00	176.12b	125.94	125.94	399946
32	-15.10	0.00	92.44	0.00	182.16b	117.89	117.89	399946
33	-16.30	0.00	117.69	0.00	550.53b	121.00	121.00	399946
34	-17.15	8.50	127.09	0.00	1101.60b	121.39	129.89	399946
35	-18.00	17.00	136.50	0.00	1191.78b	123.34	140.34	399946
36	-19.15	28.50	149.25	0.00	1298.49b	124.77	153.27	399946
37	-20.30	40.00	162.04	0.00	1395.23b	114.91	154.91	399946
38	-20.60	43.00	165.38	0.00	1512.55b	126.13	169.13	399946
39	-24.70	84.00	211.32	0.00	1846.22b	141.18	225.18	399946
40	-28.80	125.00	257.84	0.00	2270.97b	156.40	281.40	399946
41	-33.60	173.00	313.12	0.00	2752.38b	175.03	348.03	399946
42	-38.40	221.00	369.33	0.00	3251.43b	194.08	415.08	399946
43	-43.08	267.75	424.90	0.00	3747.59b	212.97	480.72	399946
44	-47.75	314.50	481.18	0.00	4201.61b	232.40	546.90	399946

Note: 81.62 a Soil pressure at active limit
149.86 p Soil pressure at passive limit
4201.61 b Passive limit reduced because of berm

Units: kN,m

Stage No. 14 Excavate to elevation -16.30 on RIGHT side

STABILITY ANALYSIS of Fully Embedded Wall according to Strength Factor method
Factor of safety on soil strength

Stage No.	Ground level Act.	Ground level Pass.	Prop Elev.	FoS for toe elev. = -20.30: Factor of Safety	Moment equilib. at elev.	Toe elev. for FoS = 1.000: Toe elev.	Wall Penetr-ation	Direction of failure
14	4.00	-16.30		More than one prop. No FoS calc.				

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall
Analysis options
Length of wall perpendicular to section = 1000.00m
2-D finite element model. Active limit arching modelled.
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Left side 60.00 from wall — Rough boundary
Right side 60.00 from wall — Rough boundary
Lower rigid boundary at elevation -47.75 — Rough boundary

*** Wall displacements reset to zero at stage 1

Node no.	Y coord	Nett pressure	Wall disp.	Wall rotation	Shear force	Bending moment	Prop forces	EI of wall
		kN/m2	m	rad.	kN/m	kN.m/m	kN/m	kN.m2/m
1	4.40	0.00	0.006	-1.11E-03	0.0	0.0		877500
2	4.30	0.00	0.006	-1.11E-03	0.0	-0.0	-35.8	877500
		0.00	0.006	-1.11E-03	-35.8	0.0		
3	4.00	0.00	0.007	-1.10E-03	-35.8	-10.8		877500
		7.11	0.007	-1.10E-03	-35.8	-10.8		
4	2.90	13.49	0.008	-1.07E-03	-24.5	-43.9		877500
5	2.50	16.19	0.008	-1.05E-03	-18.6	-52.5		877500
6	1.40	25.96	0.009	-9.82E-04	4.6	-61.1		877500
7	0.70	35.33	0.010	-9.37E-04	26.1	-50.7	-104.1	877500
		35.33	0.010	-9.37E-04	-78.1	-50.7		
8	0.60	36.40	0.010	-9.31E-04	-74.5	-58.3		877500
9	0.00	44.48	0.011	-8.78E-04	-50.2	-95.9		877500
10	-0.85	55.99	0.011	-7.73E-04	-7.5	-120.9		877500
11	-1.70	67.66	0.012	-6.64E-04	45.1	-105.5		877500
12	-2.20	74.65	0.012	-6.12E-04	80.6	-74.2		877500
13	-2.50	79.02	0.013	-5.92E-04	103.7	-46.6		877500
14	-3.25	89.79	0.013	-5.95E-04	167.0	54.3		877500
15	-4.00	100.80	0.013	-7.06E-04	238.5	205.7		877500
16	-4.65	110.50	0.014	-9.24E-04	307.1	382.6		877500
17	-5.30	120.10	0.015	-1.29E-03	382.1	606.2	-1012.7	877500
		120.10	0.015	-1.29E-03	-630.7	606.2		
18	-5.70	127.71	0.015	-1.51E-03	-581.1	363.7		877500
		167.44	0.015	-1.51E-03	-581.1	363.7		
19	-6.00	152.82	0.016	-1.60E-03	-533.1	196.9		877500
20	-6.30	140.80	0.016	-1.64E-03	-489.0	43.9		877500
21	-6.90	135.79	0.017	-1.58E-03	-406.0	-223.9		877500
		515.58	0.017	-1.58E-03	-406.0	-223.9		
22	-7.75	218.66	0.018	-1.28E-03	-94.0	-398.5		877500
23	-8.60	86.43	0.019	-8.93E-04	35.7	-411.6		877500
24	-9.80	77.60	0.020	-3.95E-04	134.1	-315.9		877500
25	-10.30	78.19	0.020	-2.37E-04	173.1	-240.0	-352.8	877500
		78.19	0.020	-2.37E-04	-179.8	-240.0		

(continued)

Stage No.14 Excavate to elevation -16.30 on RIGHT side

Node no.	Y coord	Nett pressure	Wall disp.	Wall rotation	Shear force	Bending moment	Prop forces	EI of wall
		kN/m2	m	rad.	kN/m	kN.m/m	kN/m	kN.m2/m
26	-10.70	78.67	0.020	-1.13E-04	-148.4	-305.7		877500
27	-10.90	78.90	0.020	-4.01E-05	-132.7	-334.5		877500
28	-12.05	80.26	0.020	4.63E-04	-41.1	-434.8		877500
29	-13.20	81.62	0.019	1.02E-03	51.9	-424.1		877500
30	-13.95	82.50	0.018	1.36E-03	113.5	-363.2		877500
31	-14.70	83.39	0.017	1.62E-03	175.7	-255.9		877500
32	-15.10	83.86	0.017	1.72E-03	209.2	-179.2		877500
33	-16.30	85.28	0.014	1.75E-03	310.6	133.2		877500
		-336.22	0.014	1.75E-03	310.6	133.2		
34	-17.15	-214.78	0.013	1.56E-03	76.5	275.1		877500
35	-18.00	-91.47	0.012	1.30E-03	-53.7	261.9		877500
36	-19.15	-10.93	0.010	1.03E-03	-112.6	138.2		877500
37	-20.30	163.95	0.009	9.47E-04	-24.6	0.0		0
38	-20.60	0.10	0.009	0	0.0	0.0		0
39	-24.70	0.03	0.007	0	0.3	0.0		0
40	-28.80	0.09	0.006	0	0.5	0.0		0
41	-33.60	0.07	0.005	0	0.9	0.0		0
42	-38.40	0.06	0.004	0	1.2	0.0		0
43	-43.08	0.06	0.003	0	1.5	0.0		0
44	-47.75	-0.70	0.000	0	0.0	0.0		---

At elev. 4.30 Prop force = 35.8 kN/m run (horiz.)
= 40.2 kN/m run (inclined)
At elev. 0.70 Prop force = 104.1 kN/m run
At elev. -5.30 Prop force = 1012.7 kN/m run
At elev. -10.30 Prop force = 352.8 kN/m run (horiz.)
= 499.0 kN/m run (inclined)

LEFT side

Node no.	Y coord	Water press.	Effective stresses: Vertic -al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
1	4.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	4.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
3	4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
		0.00	0.00	0.00	16.18	7.11	7.11	5999
4	2.90	0.00	18.76	1.90	88.77	13.49	13.49	5999
5	2.50	0.00	26.00	4.14	116.75	16.19	16.19	5999
6	1.40	0.00	46.19	10.38	194.86	25.96	25.96	5999
7	0.70	7.00	51.88	12.13	216.89	28.33	35.33	5999
8	0.60	8.00	52.68	12.38	219.97	28.40	36.40	5999
9	0.00	14.00	57.37	13.83	238.12	30.48	44.48	5999
10	-0.85	22.50	63.79	15.81	262.95	33.49	55.99	5999
11	-1.70	31.00	70.00	17.73	286.97	36.66	67.66	5999
12	-2.20	36.00	73.58	18.84	300.80	38.65	74.65	5999
13	-2.50	39.00	75.70	19.49	309.01	40.02	79.02	5999
		39.00	75.70	19.49	348.27	40.02	79.02	5999
14	-3.25	46.50	80.94	21.11	371.18	43.29	89.79	5999
15	-4.00	54.00	86.11	22.71	393.76	46.80	100.80	5999
16	-4.65	60.50	90.54	24.08	413.12	50.00	110.50	5999
17	-5.30	67.00	94.93	25.44	432.31	53.10	120.10	5999
18	-5.70	71.00	97.62	26.27	444.04	56.71	127.71	5999
		71.00	97.62	17.04	528.95	96.44	167.44	39991
19	-6.00	73.12	101.26	17.98	547.01	79.70	152.82	39991
20	-6.30	73.47	106.65	19.38	573.79	67.32	140.80	39991
21	-6.90	74.18	117.43	22.18	627.29	61.61	135.79	39991
		74.18	117.43	0.00	1984.62	441.39	515.58	399914

Run ID. 1_North_800DWall_topdown+propped_gw2 | Sheet No.
Downtown Carpark Redevelopment | Date:30-05-2024
800mm D-Wall - Top Down + Propped | Checked :

(continued)

Stage No.14 Excavate to elevation -16.30 on RIGHT side

LEFT side

Node no.	Y coord	Water press.	Effective stresses: Vertic -al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
22	-7.75	75.18	134.37	0.00	2135.20	143.47	218.66	399914
23	-8.60	76.19	151.27	0.00	2285.48	10.25	86.43	399914
24	-9.80	77.60	175.08	0.00	2497.22	0.00	77.60a	399914
25	-10.30	78.19	184.99	0.00	2585.33	0.00	78.19a	399914
26	-10.70	78.67	192.91	0.00	2655.76	0.00	78.67a	399914
27	-10.90	78.90	196.87	0.00	2690.97	0.00	78.90a	399914
28	-12.05	80.26	219.62	0.00	2893.22	0.00	80.26a	399914
29	-13.20	81.62	242.34	0.00	3095.23	0.00	81.62a	399914
30	-13.95	82.50	257.14	0.00	3226.87	0.00	82.50a	399914
31	-14.70	83.39	271.94	0.00	3358.43	0.00	83.39a	399914
32	-15.10	83.86	279.83	0.00	3428.57	0.00	83.86a	399914
33	-16.30	85.28	303.48	0.00	3638.90	0.00	85.28a	399914
34	-17.15	86.28	320.23	0.00	3787.80	0.00	86.28a	399914
35	-18.00	87.28	336.97	0.00	3936.66	35.65	122.94	399914
36	-19.15	88.64	359.61	0.00	4137.99	73.13	161.77	399914
37	-20.30	90.00	382.25	0.00	4339.27	163.79	253.79	399914
38	-20.60	93.00	385.51	0.00	4368.24	75.76	168.76	399914
39	-24.70	134.00	430.03	0.00	4764.06	82.97	216.97	399914
40	-28.80	175.00	474.54	0.00	5159.86	92.67	267.67	399914
41	-33.60	223.00	526.69	0.00	5623.52	108.57	331.57	399914
42	-38.40	271.00	578.89	6.16	6087.64	126.27	397.27	399914
43	-43.08	317.75	629.78	15.30	6540.18	144.70	462.45	399914
44	-47.75	364.50	680.73	24.45	6993.21	163.50	528.00	399914

RIGHT side

Node no.	Y coord	Water press.	Effective stresses: Vertic -al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
1	4.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	4.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
3	4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
4	2.90	0.00	0.00	0.00	0.00	0.00	0.00	0.0
5	2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.0
6	1.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
7	0.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
8	0.60	0.00	0.00	0.00	0.00	0.00	0.00	0.0
9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
10	-0.85	0.00	0.00	0.00	0.00	0.00	0.00	0.0
11	-1.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
12	-2.20	0.00	0.00	0.00	0.00	0.00	0.00	0.0
13	-2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.0
14	-3.25	0.00	0.00	0.00	0.00	0.00	0.00	0.0
15	-4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
16	-4.65	0.00	0.00	0.00	0.00	0.00	0.00	0.0
17	-5.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
18	-5.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
19	-6.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
20	-6.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
21	-6.90	0.00	0.00	0.00	0.00	0.00	0.00	0.0
22	-7.75	0.00	0.00	0.00	0.00	0.00	0.00	0.0
23	-8.60	0.00	0.00	0.00	0.00	0.00	0.00	0.0
24	-9.80	0.00	0.00	0.00	0.00	0.00	0.00	0.0
25	-10.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0

(continued)

Stage No.14 Excavate to elevation -16.30 on RIGHT side

RIGHT side

Node no.	Y coord	Water press.	Effective stresses: Vertic-al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
26	-10.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
27	-10.90	0.00	0.00	0.00	0.00	0.00	0.00	0.0
28	-12.05	0.00	0.00	0.00	0.00	0.00	0.00	0.0
29	-13.20	0.00	0.00	0.00	0.00	0.00	0.00	0.0
30	-13.95	0.00	0.00	0.00	0.00	0.00	0.00	0.0
31	-14.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
32	-15.10	0.00	0.00	0.00	0.00	0.00	0.00	0.0
33	-16.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
		0.00	0.00	0.00	940.47	421.50	421.50	399965
34	-17.15	8.50	9.35	0.00	1023.61	292.56	301.06	399965
35	-18.00	17.00	18.70	0.00	1106.78	197.41	214.41	399965
36	-19.15	28.50	31.37	0.00	1219.38	144.21	172.71	399965
37	-20.30	40.00	44.05	0.00	1332.12	49.85	89.85	399965
38	-20.60	43.00	47.36	0.00	1361.57	125.66	168.66	399965
39	-24.70	84.00	92.83	0.00	1765.88	132.93	216.93	399965
40	-28.80	125.00	138.88	0.00	2175.33	142.59	267.59	399965
41	-33.60	173.00	193.80	0.00	2663.63	158.50	331.50	399965
42	-38.40	221.00	249.99	0.00	3163.23	176.21	397.21	399965
43	-43.08	267.75	305.99	0.00	3661.19	194.63	462.38	399965
44	-47.75	314.50	363.20	0.00	4169.88	214.20	528.70	399965

Note: 86.28 a Soil pressure at active limit
123.45 p Soil pressure at passive limit

Units: kN,m

Stage No. 15 Fill to elevation -15.10 on RIGHT side with soil type 6

STABILITY ANALYSIS of Fully Embedded Wall according to Strength Factor method
Factor of safety on soil strength

Stage No.	Ground level Act.	Ground level Pass.	Prop Elev.	FoS for toe elev. = -20.30: Factor of Safety	Moment equilib. at elev.	Toe elev. for FoS = 1.000: Toe elev.	Wall Penetr-ation	Direction of failure
15	4.00	-15.10		More than one prop. No FoS calc.				

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall
Analysis options
Length of wall perpendicular to section = 1000.00m
2-D finite element model. Active limit arching modelled.
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Left side 60.00 from wall — Rough boundary
Right side 60.00 from wall — Rough boundary
Lower rigid boundary at elevation -47.75 — Rough boundary

*** Wall displacements reset to zero at stage 1

Node no.	Y coord	Nett pressure	Wall disp.	Wall rotation	Shear force	Bending moment	Prop forces	EI of wall
		kN/m2	m	rad.	kN/m	kN.m/m	kN/m	kN.m2/m
1	4.40	0.00	0.006	-1.11E-03	0.0	0.0		877500
2	4.30	0.00	0.006	-1.11E-03	0.0	-0.0	-35.8	877500
		0.00	0.006	-1.11E-03	-35.8	0.0		
3	4.00	0.00	0.007	-1.11E-03	-35.8	-10.7		877500
		7.11	0.007	-1.11E-03	-35.8	-10.7		
4	2.90	13.48	0.008	-1.07E-03	-24.4	-43.8		877500
5	2.50	16.18	0.008	-1.05E-03	-18.5	-52.4		877500
6	1.40	25.94	0.009	-9.83E-04	4.7	-60.9		877500
7	0.70	35.31	0.010	-9.38E-04	26.1	-50.4	-104.6	877500
		35.31	0.010	-9.38E-04	-78.5	-50.4		
8	0.60	36.38	0.010	-9.32E-04	-75.0	-58.1		877500
9	0.00	44.46	0.011	-8.79E-04	-50.7	-96.0		877500
10	-0.85	55.96	0.011	-7.74E-04	-8.0	-121.4		877500
11	-1.70	67.63	0.012	-6.64E-04	44.5	-106.5		877500
12	-2.20	74.60	0.012	-6.12E-04	80.1	-75.5		877500
13	-2.50	78.97	0.013	-5.91E-04	103.1	-48.1		877500
14	-3.25	89.74	0.013	-5.92E-04	166.4	52.4		877500
15	-4.00	100.75	0.013	-7.02E-04	237.8	203.3		877500
16	-4.65	110.45	0.014	-9.18E-04	306.4	379.7		877500
17	-5.30	120.04	0.015	-1.28E-03	381.3	602.9	-1009.7	877500
		120.04	0.015	-1.28E-03	-628.3	602.9		
18	-5.70	127.67	0.015	-1.50E-03	-578.8	361.3		877500
		167.15	0.015	-1.50E-03	-578.8	361.3		
19	-6.00	152.54	0.016	-1.59E-03	-530.8	195.2		877500
20	-6.30	140.53	0.016	-1.63E-03	-486.9	42.9		877500
21	-6.90	135.58	0.017	-1.57E-03	-404.0	-223.8		877500
		513.45	0.017	-1.57E-03	-404.0	-223.8		
22	-7.75	216.85	0.018	-1.27E-03	-93.7	-397.4		877500
23	-8.60	84.76	0.019	-8.83E-04	34.5	-410.8		877500
24	-9.80	77.60	0.020	-3.85E-04	131.9	-317.8		877500
25	-10.30	78.19	0.020	-2.25E-04	170.9	-242.9	-350.2	877500
		78.19	0.020	-2.25E-04	-179.3	-242.9		

(continued)

Stage No.15 Fill to elevation -15.10 on RIGHT side with soil type 6

Node no.	Y coord	Nett pressure	Wall disp.	Wall rotation	Shear force	Bending moment	Prop forces	EI of wall
		kN/m2	m	rad.	kN/m	kN.m/m	kN/m	kN.m2/m
26	-10.70	78.67	0.020	-9.98E-05	-147.9	-308.5		877500
27	-10.90	78.90	0.020	-2.62E-05	-132.2	-337.2		877500
28	-12.05	80.26	0.020	4.80E-04	-40.6	-436.9		877500
29	-13.20	81.78	0.019	1.04E-03	52.5	-425.6		877500
30	-13.95	83.62	0.018	1.38E-03	114.6	-364.3		877500
31	-14.70	85.32	0.017	1.64E-03	177.9	-255.9		877500
32	-15.10	86.92	0.016	1.74E-03	212.4	-178.1		877500
33	-16.30	79.55	0.014	1.77E-03	312.2	138.0		877500
		-340.53	0.014	1.77E-03	312.2	138.0		
34	-17.15	-216.23	0.013	1.57E-03	75.6	279.7		877500
35	-18.00	-91.65	0.012	1.30E-03	-55.2	265.3		877500
36	-19.15	-10.62	0.010	1.04E-03	-114.0	139.7		877500
37	-20.30	165.72	0.009	9.51E-04	-24.9	0.0		0
38	-20.60	0.11	0.009	0	0.0	0.0		0
39	-24.70	0.03	0.007	0	0.3	0.0		0
40	-28.80	0.09	0.006	0	0.6	0.0		0
41	-33.60	0.06	0.005	0	0.9	0.0		0
42	-38.40	0.05	0.004	0	1.2	0.0		0
43	-43.08	0.06	0.002	0	1.5	0.0		0
44	-47.75	-0.68	0.000	0	0.0	0.0		---

At elev. 4.30 Prop force = 35.8 kN/m run (horiz.)
= 40.1 kN/m run (inclined)
At elev. 0.70 Prop force = 104.6 kN/m run
At elev. -5.30 Prop force = 1009.7 kN/m run
At elev. -10.30 Prop force = 350.2 kN/m run (horiz.)
= 495.2 kN/m run (inclined)

LEFT side

Node no.	Y coord	Effective stresses: Water press.	Vertic -al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
1	4.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	4.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
3	4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
		0.00	0.00	0.00	16.18	7.11	7.11	5999
4	2.90	0.00	18.76	1.90	88.77	13.48	13.48	5999
5	2.50	0.00	26.00	4.14	116.75	16.18	16.18	5999
6	1.40	0.00	46.19	10.38	194.86	25.94	25.94	5999
7	0.70	7.00	51.88	12.13	216.89	28.31	35.31	5999
8	0.60	8.00	52.68	12.38	219.97	28.38	36.38	5999
9	0.00	14.00	57.37	13.83	238.12	30.46	44.46	5999
10	-0.85	22.50	63.79	15.81	262.95	33.46	55.96	5999
11	-1.70	31.00	70.00	17.73	286.97	36.63	67.63	5999
12	-2.20	36.00	73.58	18.84	300.80	38.60	74.60	5999
13	-2.50	39.00	75.70	19.49	309.01	39.97	78.97	5999
		39.00	75.70	19.49	348.27	39.97	78.97	5999
14	-3.25	46.50	80.94	21.11	371.18	43.24	89.74	5999
15	-4.00	54.00	86.11	22.71	393.76	46.75	100.75	5999
16	-4.65	60.50	90.54	24.08	413.12	49.95	110.45	5999
17	-5.30	67.00	94.93	25.44	432.31	53.04	120.04	5999
18	-5.70	71.00	97.62	26.27	444.04	56.67	127.67	5999
		71.00	97.62	17.04	528.95	96.15	167.15	39991
19	-6.00	73.12	101.26	17.98	547.01	79.42	152.54	39991
20	-6.30	73.47	106.65	19.38	573.79	67.06	140.53	39991
21	-6.90	74.18	117.43	22.18	627.29	61.40	135.58	39991
		74.18	117.43	0.00	1984.62	439.27	513.45	399914

Run ID. 1_North_800DWall_topdown+propped_gw2 | Sheet No.
Downtown Carpark Redevelopment | Date:30-05-2024
800mm D-Wall - Top Down + Propped | Checked :

(continued)

Stage No.15 Fill to elevation -15.10 on RIGHT side with soil type 6

LEFT side

Node no.	Y coord	Effective stresses: Water press.	Vertic -al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
22	-7.75	75.18	134.37	0.00	2135.20	141.67	216.85	399914
23	-8.60	76.19	151.27	0.00	2285.48	8.57	84.76	399914
24	-9.80	77.60	175.08	0.00	2497.22	0.00	77.60a	399914
25	-10.30	78.19	184.99	0.00	2585.33	0.00	78.19a	399914
26	-10.70	78.67	192.91	0.00	2655.76	0.00	78.67a	399914
27	-10.90	78.90	196.87	0.00	2690.97	0.00	78.90a	399914
28	-12.05	80.26	219.62	0.00	2893.22	0.00	80.26a	399914
29	-13.20	81.62	242.34	0.00	3095.23	0.16	81.78	399914
30	-13.95	82.50	257.14	0.00	3226.87	1.12	83.62	399914
31	-14.70	83.39	271.94	0.00	3358.43	1.93	85.32	399914
32	-15.10	83.86	279.83	0.00	3428.57	3.05	86.92	399914
33	-16.30	85.28	303.48	0.00	3638.90	4.13	89.40	399914
34	-17.15	86.28	320.23	0.00	3787.80	4.82	91.10	399914
35	-18.00	87.28	336.97	0.00	3936.66	40.57	127.85	399914
36	-19.15	88.64	359.61	0.00	4137.99	77.96	166.60	399914
37	-20.30	90.00	382.25	0.00	4339.27	169.17	259.17	399914
38	-20.60	93.00	385.51	0.00	4368.24	80.04	173.04	399914
39	-24.70	134.00	430.03	0.00	4764.06	87.05	221.05	399914
40	-28.80	175.00	474.54	0.00	5159.86	96.63	271.63	399914
41	-33.60	223.00	526.69	0.00	5623.52	112.49	335.49	399914
42	-38.40	271.00	578.89	6.16	6087.64	130.18	401.18	399914
43	-43.08	317.75	629.78	15.30	6540.18	148.63	466.38	399914
44	-47.75	364.50	680.73	24.45	6993.21	167.44	531.94	399914

RIGHT side

Node no.	Y coord	Effective stresses: Water press.	Vertic -al	Active limit	Passive limit	Earth pressure	Total earth pressure	Adjusted soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
1	4.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	4.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
3	4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
4	2.90	0.00	0.00	0.00	0.00	0.00	0.00	0.0
5	2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.0
6	1.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
7	0.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
8	0.60	0.00	0.00	0.00	0.00	0.00	0.00	0.0
9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
10	-0.85	0.00	0.00	0.00	0.00	0.00	0.00	0.0
11	-1.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
12	-2.20	0.00	0.00	0.00	0.00	0.00	0.00	0.0
13	-2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.0
14	-3.25	0.00	0.00	0.00	0.00	0.00	0.00	0.0
15	-4.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
16	-4.65	0.00	0.00	0.00	0.00	0.00	0.00	0.0
17	-5.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
18	-5.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
19	-6.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
20	-6.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0
21	-6.90	0.00	0.00	0.00	0.00	0.00	0.00	0.0
22	-7.75	0.00	0.00	0.00	0.00	0.00	0.00	0.0
23	-8.60	0.00	0.00	0.00	0.00	0.00	0.00	0.0
24	-9.80	0.00	0.00	0.00	0.00	0.00	0.00	0.0
25	-10.30	0.00	0.00	0.00	0.00	0.00	0.00	0.0

(continued)

Stage No.15 Fill to elevation -15.10 on RIGHT side with soil type 6

		RIGHT side						
			Effective stresses				Total	Adjusted
Node no.	Y coord	Water press.	Vertic -al	Active limit	Passive limit	Earth pressure	earth pressure	soil modulus
		kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2	kN/m2
26	-10.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
27	-10.90	0.00	0.00	0.00	0.00	0.00	0.00	0.0
28	-12.05	0.00	0.00	0.00	0.00	0.00	0.00	0.0
29	-13.20	0.00	0.00	0.00	0.00	0.00	0.00	0.0
30	-13.95	0.00	0.00	0.00	0.00	0.00	0.00	0.0
31	-14.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
32	-15.10	0.00	0.00	0.00	0.00	0.00	0.00	0.0
		0.00	0.00	0.00	0.00	0.00	0.00	39996
33	-16.30	0.00	22.80	4.50	173.01	9.86	9.86	39996
		0.00	22.80	0.00	1143.21	429.93	429.93	399961
34	-17.15	8.50	32.16	0.00	1226.39	298.83	307.33	399961
35	-18.00	17.00	41.52	0.00	1309.62	202.50	219.50	399961
36	-19.15	28.50	54.20	0.00	1422.36	148.72	177.22	399961
37	-20.30	40.00	66.90	0.00	1535.29	53.45	93.45	399961
38	-20.60	43.00	70.21	0.00	1564.79	129.93	172.93	399961
39	-24.70	84.00	115.80	0.00	1970.07	137.02	221.02	399961
40	-28.80	125.00	161.98	0.00	2380.71	146.54	271.54	399961
41	-33.60	173.00	217.04	0.00	2870.29	162.42	335.42	399961
42	-38.40	221.00	273.31	0.00	3370.64	180.13	401.13	399961
43	-43.08	267.75	329.31	0.00	3868.54	198.57	466.32	399961
44	-47.75	314.50	386.41	0.00	4376.21	218.13	532.63	399961

Note: 80.26 a Soil pressure at active limit
123.45 p Soil pressure at passive limit

Units: kN,m

Summary of results

STABILITY ANALYSIS of Fully Embedded Wall according to Strength Factor method
Factor of safety on soil strength

				FoS for toe elev. = -20.30		Toe elev. for FoS = 1.000		
Stage No.	Ground level Act.	Pass.	Prop Elev.	Factor of Safety	Moment equilib. at elev.	Toe elev.	Wall Penetr -ation	Direction of failure
1	4.00	4.00	Cant.	Conditions not suitable for FoS calc.				
2	4.00	4.00	Cant.	Conditions not suitable for FoS calc.				
3	4.00	4.00		No analysis at this stage				
4	4.00	4.00	Cant.	Conditions not suitable for FoS calc.				
5	4.00	4.00		No analysis at this stage				
6	4.00	0.00	4.30	12.455	n/a	-1.91	1.91	L to R
7	4.00	0.00		No analysis at this stage				

All remaining stages have more than one prop - FoS calculation n/a

Units: kN,m

Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 1000.00m
2-D finite element model. Active limit arching modelled.
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Left side 60.00 from wall Rough boundary
Right side 60.00 from wall Rough boundary
Lower rigid boundary at elevation -47.75 Rough boundary

Bending moment, shear force and displacement envelopes

Node	Y	Displacement		Bending moment		Shear force	
no.	coord	maximum	minimum	maximum	minimum	maximum	minimum
		m	m	kN.m/m	kN.m/m	kN/m	kN/m
1	4.40	0.008	-0.000	0.0	-0.0	0.0	-45.0
2	4.30	0.009	-0.000	0.3	-3.1	0.0	-89.2
3	4.00	0.009	-0.000	1.1	-26.8	0.0	-89.2
4	2.90	0.009	-0.000	9.3	-120.9	26.2	-82.5
5	2.50	0.010	-0.000	15.5	-153.1	31.9	-78.6
6	1.40	0.010	-0.000	47.7	-230.9	55.4	-61.7
7	0.70	0.011	-0.000	94.9	-268.4	77.6	-222.0
8	0.60	0.011	-0.000	76.7	-272.7	0.1	-218.8
9	0.00	0.011	-0.000	17.2	-291.7	0.1	-197.1
10	-0.85	0.012	-0.000	0.2	-298.7	5.6	-159.0
11	-1.70	0.013	-0.000	0.3	-405.8	58.6	-112.0
12	-2.20	0.013	-0.000	12.5	-454.0	94.5	-80.2
13	-2.50	0.013	-0.000	36.9	-475.0	117.7	-59.7
14	-3.25	0.013	-0.000	130.3	-499.0	181.6	-3.3
15	-4.00	0.013	-0.000	274.8	-478.0	253.6	0.0
16	-4.65	0.014	-0.000	450.5	-419.0	322.9	0.0
17	-5.30	0.015	-0.000	684.5	-318.2	398.5	-691.5
18	-5.70	0.016	-0.000	420.0	-233.8	233.3	-638.0
19	-6.00	0.017	-0.000	237.1	-160.6	255.0	-585.6
20	-6.30	0.018	-0.000	136.2	-111.4	269.3	-539.6
21	-6.90	0.019	0.000	218.8	-263.5	301.6	-453.6
22	-7.75	0.021	0.000	258.8	-439.1	99.6	-122.2
23	-8.60	0.023	0.000	257.7	-471.0	194.3	-56.3
24	-9.80	0.024	0.000	208.6	-414.7	328.4	-362.1
25	-10.30	0.024	0.000	178.8	-473.1	173.1	-301.1
26	-10.70	0.024	0.000	154.6	-505.2	185.4	-252.4
27	-10.90	0.024	0.000	142.9	-517.2	164.0	-227.4
28	-12.05	0.024	0.000	85.8	-520.7	106.4	-41.1
29	-13.20	0.022	0.000	150.6	-425.6	226.9	-23.9
30	-13.95	0.020	0.000	160.7	-364.3	359.8	-15.9
31	-14.70	0.018	0.000	254.3	-255.9	547.5	-379.4
32	-15.10	0.017	0.000	148.6	-179.2	212.4	-256.8
33	-16.30	0.015	0.000	181.7	-0.0	312.2	-31.3
34	-17.15	0.014	0.000	297.0	-0.0	76.5	-24.0
35	-18.00	0.013	0.000	269.4	-0.0	0.0	-66.0
36	-19.15	0.012	0.000	139.7	-0.0	0.0	-115.8
37	-20.30	0.011	0.000	0.0	0.0	0.0	-24.9
38	-20.60	0.011	0.000	0.0	0.0	0.0	-14.7
39	-24.70	0.009	0.000	0.0	0.0	0.4	-14.4
40	-28.80	0.008	0.000	0.0	0.0	0.6	-14.2
41	-33.60	0.007	0.000	0.0	0.0	1.0	-13.8
42	-38.40	0.005	0.000	0.0	0.0	1.2	-13.5
43	-43.08	0.004	0.000	0.0	0.0	1.5	-13.3

Run ID. 1_North_800DWall_topdown+propped_gw2 | Sheet No.
Downtown Carpark Redevelopment | Date:30-05-2024
800mm D-Wall - Top Down + Propped | Checked :

Bending moment, shear force and displacement envelopes

Node	Y	Displacement		Bending moment		Shear force	
no.	coord	maximum	minimum	maximum	minimum	maximum	minimum
		m	m	kN.m/m	kN.m/m	kN/m	kN/m
44	-47.75	0.000	0.000	0.0	0.0	0.0	-14.7

Maximum and minimum bending moment and shear force at each stage

Stage	Bending moment				Shear force			
no.	maximum	elev.	minimum	elev.	maximum	elev.	minimum	elev.
	kN.m/m		kN.m/m		kN/m		kN/m	
1	1.1	-5.70	-1.3	-8.60	0.4	-9.80	-2.3	-6.90
2	30.4	-6.90	-6.5	-0.85	11.6	-5.70	-9.2	-8.60
3	No calculation at this stage							
4	96.1	-6.90	-29.5	-0.85	38.2	-5.70	-29.1	-8.60
5	No calculation at this stage							
6	258.8	-7.75	-298.7	-0.85	143.6	-5.70	-89.2	4.30
7	No calculation at this stage							
8	No calculation at this stage							
9	257.7	-8.60	-499.0	-3.25	301.6	-6.90	-222.0	0.70
10	No calculation at this stage							
11	No calculation at this stage							
12	188.8	-5.30	-335.3	-8.60	292.1	-5.30	-358.1	-5.30
13	185.5	-5.30	-332.7	-8.60	291.4	-5.30	-355.6	-5.30
14	606.2	-5.30	-434.8	-12.05	382.1	-5.30	-630.7	-5.30
15	602.9	-5.30	-436.9	-12.05	381.3	-5.30	-628.3	-5.30
16	No calculation at this stage							
17	No calculation at this stage							
18	684.5	-5.30	-520.3	-12.05	398.5	-5.30	-686.0	-5.30
19	No calculation at this stage							
20	664.8	-5.30	-520.7	-12.05	374.9	-5.30	-683.3	-5.30
21	No calculation at this stage							
22	615.6	-5.30	-424.9	-12.05	366.4	-5.30	-636.8	-5.30
23	675.5	-5.30	-506.8	-12.05	533.8	-14.70	-691.5	-5.30
24	657.4	-5.30	-500.7	-12.05	508.1	-14.70	-674.5	-5.30
25	573.4	-5.30	-391.2	-12.05	474.4	-14.70	-597.7	-5.30
26	577.0	-5.30	-411.0	-12.05	517.7	-14.70	-600.7	-5.30
27	583.0	-5.30	-430.5	-12.05	540.0	-14.70	-605.7	-5.30
28	594.5	-5.30	-428.3	-12.05	545.2	-14.70	-619.1	-5.30
29	603.5	-5.30	-427.6	-12.05	547.5	-14.70	-659.2	-5.30
30	617.4	-5.30	-427.8	-12.05	532.7	-14.70	-676.5	-5.30
31	626.2	-5.30	-427.8	-12.05	532.5	-14.70	-679.4	-5.30
32	627.2	-5.30	-427.9	-12.05	532.5	-14.70	-679.9	-5.30
33	626.1	-5.30	-427.8	-12.05	532.5	-14.70	-679.6	-5.30

Summary of results (continued)

Maximum and minimum displacement at each stage

Stage no.	Displacement maximum	elev.	minimum	elev.	Stage description
	m		m		
1	0.000	-20.30	-0.000	4.40	Change EI of wall to 877500kN.m2/m run
2	0.002	4.40	0.000	4.40	Apply surcharge no.1 at elev. 4.00
3	No calculation at this stage				Apply water pressure profile no.1
4	0.006	4.40	0.000	4.40	Excav. to elev. 4.00 on RIGHT side
5	No calculation at this stage				Install prop no.3 at elev. 4.30
6	0.010	0.00	0.000	4.40	Excav. to elev. 0.00 on RIGHT side
7	No calculation at this stage				Install prop no.1 at elev. 0.70
8	No calculation at this stage				Apply water pressure profile no.2
9	0.013	-3.25	0.000	4.40	Excav. to elev. -6.00 on RIGHT side
10	No calculation at this stage				Install prop no.2 at elev. -5.30
11	No calculation at this stage				Apply water pressure profile no.3
12	0.014	-7.75	0.000	4.40	Excav. to elev. -10.70 on RIGHT side
13	0.014	-7.75	0.000	4.40	Install prop no.8 at elev. -10.30
14	0.020	-10.90	0.000	4.40	Excav. to elev. -16.30 on RIGHT side
15	0.020	-10.90	0.000	4.40	Fill to elev. -15.10 on RIGHT side
16	No calculation at this stage				Install prop no.4 at elev. -14.70
17	No calculation at this stage				Install prop no.5 at elev. -9.80
18	0.022	-10.70	0.000	4.40	Remove prop no.8 at elev. -10.30
19	No calculation at this stage				Install prop no.6 at elev. -2.20
20	0.022	-10.70	0.000	4.40	Remove prop no.3 at elev. 4.30
21	No calculation at this stage				Install prop no.7 at elev. 4.40
22	0.022	-10.70	0.000	4.40	Change EI of wall to 585000kN.m2/m run
23	0.023	-10.90	0.000	4.40	Apply water pressure profile no.4
24	0.023	-10.90	0.000	4.40	Apply water pressure profile no.5
25	0.024	-10.90	0.000	4.40	Change EI of wall to 292500kN.m2/m run
26	0.024	-10.90	0.000	4.40	Apply load no.1 at elev. -13.20
27	0.024	-10.90	0.000	4.40	Apply load no.2 at elev. -10.90
28	0.024	-10.90	0.000	4.40	Apply load no.3 at elev. -8.60
29	0.024	-10.90	0.000	4.40	Apply load no.4 at elev. -6.30
30	0.024	-10.90	0.000	4.40	Apply load no.5 at elev. -4.00
31	0.024	-10.90	0.000	4.40	Apply load no.6 at elev. -1.70
32	0.024	-10.90	0.000	4.40	Apply load no.7 at elev. 0.60
33	0.024	-10.90	0.000	4.40	Apply load no.8 at elev. 2.90

Run ID. 1_North_800DWall_topdown+propped_gw2 | Sheet No.
Downtown Carpark Redevelopment | Date:30-05-2024
800mm D-Wall - Top Down + Propped | Checked :

Summary of results (continued)

Prop forces at each stage (horizontal components)

Stage no.	Strut no. 1 at elev. 0.70		Strut no. 2 at elev.-5.30		Strut no. 3 at elev. 4.30	
	kN/m run	kN/prop	kN/m run	kN/prop	kN/m run	kN/prop
6	---	---	---	---	89.18	445.89
9	259.75	259.75	---	---	16.96	84.80
12	175.97	175.97	650.20	650.20	25.26	126.29
13	176.56	176.56	647.03	647.03	25.18	125.89
14	104.11	104.11	1012.73	1012.73	35.84	179.20
15	104.64	104.64	1009.68	1009.68	35.76	178.80
18	90.11	90.11	1084.53	1084.53	37.73	188.65
20	151.35	151.35	1058.20	1058.20	---	---
22	160.11	160.11	1003.19	1003.19	---	---
23	178.83	178.83	1087.78	1087.78	---	---
24	152.40	152.40	1050.67	1050.67	---	---
25	175.84	175.84	954.81	954.81	---	---
26	175.72	175.72	959.08	959.08	---	---
27	174.63	174.63	965.76	965.76	---	---
28	172.21	172.21	982.04	982.04	---	---
29	170.74	170.74	1024.10	1024.10	---	---
30	176.23	176.23	1066.10	1066.10	---	---
31	197.76	197.76	1074.86	1074.86	---	---
32	249.88	249.88	1073.87	1073.87	---	---
33	284.83	284.83	1072.41	1072.41	---	---

Stage no.	Strut no. 4 at elev.-14.70		Strut no. 5 at elev.-9.80		Strut no. 6 at elev.-2.20	
	kN/m run	kN/prop	kN/m run	kN/prop	kN/m run	kN/prop
18	190.97	190.97	232.42	232.42	---	---
20	193.12	193.12	235.94	235.94	slack	slack
22	243.68	243.68	307.93	307.93	slack	slack
23	779.51	779.51	477.48	477.48	slack	slack
24	728.08	728.08	456.99	456.99	slack	slack
25	785.85	785.85	559.84	559.84	0.68	0.68
26	852.51	852.51	584.87	584.87	slack	slack
27	896.67	896.67	631.11	631.11	slack	slack
28	907.75	907.75	663.33	663.33	slack	slack
29	912.29	912.29	675.43	675.43	slack	slack
30	912.02	912.02	674.32	674.32	17.96	17.96
31	911.65	911.65	673.16	673.16	54.52	54.52
32	911.63	911.63	673.04	673.04	62.87	62.87
33	911.72	911.72	673.24	673.24	59.03	59.03

Run ID. 1_North_800DWall_topdown+propped_gw2 | Sheet No.
Downtown Carpark Redevelopment | Date:30-05-2024
800mm D-Wall - Top Down + Propped | Checked :

Stage no.	Strut no. 7 at elev. 4.40		Strut no. 8 at elev.-10.30	
	kN/m run	kN/prop	kN/m run	kN/prop
13	---	---	14.14	70.71
14	---	---	352.83	1764.16
15	---	---	350.17	1750.87
22	1.67	1.67	---	---
23	2.17	2.17	---	---
24	3.27	3.27	---	---
25	1.21	1.21	---	---
26	1.30	1.30	---	---
27	1.62	1.62	---	---
28	2.21	2.21	---	---
29	2.41	2.41	---	---
30	14.72	14.72	---	---
31	11.49	11.49	---	---
32	13.66	13.66	---	---
33	44.98	44.98	---	---

*** Convergence errors have occurred in at least one Construction Stage.
The errors are cumulative, and the results of all stages must be inspected for significant out of balance moment or shear at the toe of the wall.

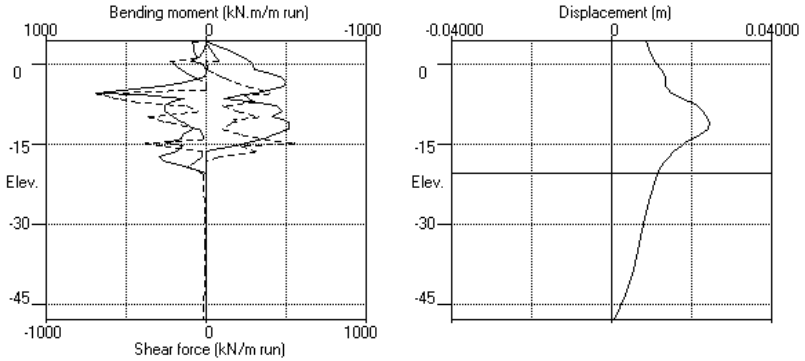
Failure of the iterative procedure to converge to an equilibrium solution may be due to a very high ratio of soil stiffness to wall stiffness. The data should be reviewed to see if realistic values have been specified

Out of balance shear forces.

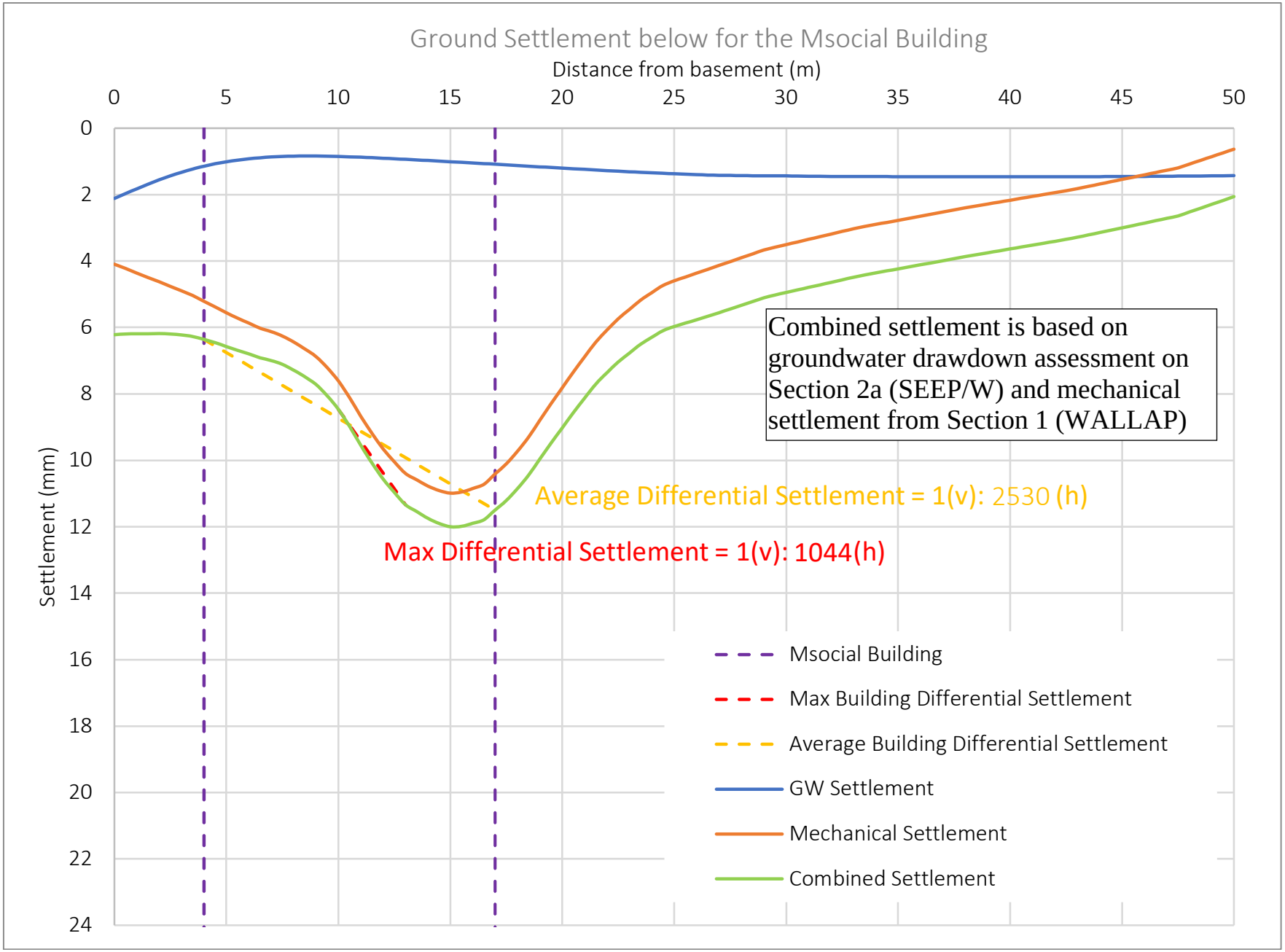
Percentage Error	Interpretation
< 2%	Generally acceptable
2% to 4%	Use with caution
> 4%	Should not be used

Units: kN,m

Bending moment, shear force, displacement envelopes



Appendix H Combined Settlement Profiles


Ground Settlement below for the Msocial Building
Distance from basement (m)
0
5
10
15
20
25
30
35
40
45
50
Settlement (mm)
0
2
4
6
8
10
12
14
16
18
20
22
24
Combined settlement is based on groundwater drawdown assessment on Section 2a (SEEP/W) and mechanical settlement from Section 1 (WALLAP)
Average Differential Settlement = 1(v): 2530 (h)
Max Differential Settlement = 1(v): 1044(h)
Msocial Building
Max Building Differential Settlement
Average Building Differential Settlement
GW Settlement
Mechanical Settlement
Combined Settlement

REPORT

Millenium & Copthorne Hotels NZ Ltd

Copthorne Harbour City - 196 Quay Street
Geotechnical Investigation

Report prepared for:

MILLENIUM & COPTHORNE HOTELS NZ LTD

Report prepared by:

Tonkin & Taylor Ltd

Distribution:

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Tonkin & Taylor Ltd (FILE)	1 copy

December 2014

T&T Ref: 29493.1



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Appendix C:	**Cone penetration tests**
Appendix D:	**Laboratory test results**
Appendix E:	**CPT liquefaction plots**
Appendix F:	**Lateral pile analyses**

1 Introduction

This report presents the geotechnical investigations and assessment for the Copthorne Hotel Development at 196 Quay Street, Auckland CBD. Written authorisation to proceed with this work was received on 9 October 2014 (ref 7) following our offer of services of 6 September 2013 (ref 18). The scope and objectives of the assessment were to undertake:

- Seven cone penetration tests to identify the depth to underlying rock and enable an assessment of the liquefaction potential;
- Two machine drilled boreholes, one adjacent to a CPT and the other angled to investigate the existing foundation;
- Laboratory test to assess the strength of the underlying rock mass and classification tests for liquefaction potential;
- A geotechnical report including:
    - An assessment of the liquefaction potential of the underlying geological units
    - Lateral pile analyses on four piles
    - Assessment of the vertical capacity of 4 piles.

Based upon the available geotechnical information and analyses undertaken, the following comments are made with regard to the geotechnical issues associated with Copthorne Hotel:

- The site is underlain by weak and potentially compressible fill and marine sediments. Very weak rock is encountered at a depth of approximately 10 metres;
- In terms of seismic risk, the site is classified as a Class C (shallow soil) site and the risk of liquefaction is considered low;
- Vertical and lateral capacity of a number of the existing piles have been presented.
- Lightly loaded shallow foundations may be used to support non-settlement sensitive structures, subject to size and load limitations.

2 Site Description

2.1 Project Description

The Copthorne Hotel is located on the south eastern corners of Quay Street and Lower Hobson Street, Auckland CBD. The site (Lot 8 DP 60151) has an area of 2408m^2 and is approximately 71m (west-east) by 35m (north-south). Commercial buildings are located beyond all property boundaries, with the Auckland Harbour located approximately 50 metres to the north.

The existing building has a footprint of approximately 70 metres by 10 metres. It consists of a thirteen storey tower structure above ground level, with the potential development involving an additional two light weight floors. The existing tower is founded on reinforced concrete piles (some belled) embedded in the underlying ECBF rock.

The following team are involved in the redevelopment of the structure:

- Client: Millenium & Copthorne Hotel NZ Ltd.
- Architect: Susan Lee Architects Ltd
- Structural engineer : Structure Design

The proposed development involves redevelopment of the existing ground floor level with new concrete slabs and an outdoor extension to the north (ref 13). The existing single level structures are founded on shallow foundations. Ground floor slabs are suspended, which will have avoided

additional fill being placed which may have caused ground settlement. The following footing pressures for the existing foundations have been calculated by Structure Design (ref 14):

- 100 kPa (working) on a 300mm wide footing
- 60 kPa (working) on a 600 mm wide footing.

It is understood the proposed ground floor slab re-development is to consist of a concrete slab poured on polystyrene (to avoid suspending the slab) and if possible using shallow foundations.

2.2 Geological information

Published geological information (ref 6) indicates the site is underlain by reclamation fill overlying weathered soil of the East Coast Bays Formation. Ports of Auckland records (ref 12) and early photographs of Auckland City provide an indication of early site use and dates of reclamation. The boundaries of the various reclamations are difficult to accurately map, with approximate dates presented on Figure 1 in Appendix A. The southern part of the site was likely to have been reclaimed in about 1886 as part of the Hobson St Wharf, with the northern part of the site reclaimed between 1923 and 1925 as part of the Quay Street reclamation. Immediately to the eastern side of the site, graving docks were located. The graving dock, orientated in a north-east to south west direction was initially constructed in 1875 with subsequent extensions towards the north-west in 1878 and 1915. These graving docks were subsequently infilled, prior to the reclamation of Quay Street (between Princess Wharf and the site) in 1923.

2.3 Subsurface investigation

The subsurface investigations consisted of 2 machine drilled boreholes and 7 cone penetration tests. The investigation was undertaken between 29 October and 17 November 2014, with the duration of the investigation lengthened due to both access difficulties and machinery issues. Locations of the investigation holes are included on Figure 1 within Appendix A.

The machine boreholes were completed by DCN Drilling under the supervision of a senior engineering technician from T&T. Borehole logs and photographs of recovered core are included in Appendix B. Drilling was undertaken using open barrel techniques with hand vane shear strength measurements undertaken in the ends of the barrel. Standard penetration tests, using a trip wire release, were undertaken within the underlying rockmass.

Borehole 1 was drilled at an inclination of 6° off the vertical adjacent to Pile 46. The purpose of the hole to drill through the bell of Pile 46 to confirm the presence of the bell, the integrity of the concrete and the founding level of the pile. A summary of the pile investigation is presented in Table 2.1. Pile 46 is a 1.22 metre diameter pile with a 2.13 metre diameter bell founding at RL -11.3m (L&S Datum) - (ref 11).

Table 2.1: Pile 46 details

		Design drawings (ref 11)	From BH1
Pile diameter	m	1.22 (4')	-
Bell diameter	m	2.13 (7')	-
Rock level (L&S Datum)	m	-6.0	-6.5
Founding level (ACC Datum)	m	-12.5(-41')	-
Founding level (L&S Datum)	m	-11.3	-8.9

The cone penetration tests were undertaken by DCN Drilling and Ground Investigations Ltd. The tests were undertaken using a 20 tonne cone, measuring cone tip resistance, sleeve friction and dynamic pore water pressure. CPT profiles are included in Appendix C. All CPTs were predrilled through the surficial hardfill / concrete, with CPTs 1 and 7 hitting obstructions and the tests being repeated.

2.4 Laboratory tests

Samples from the two machine boreholes were returned to Geotechnics laboratory for classification and strength testing. A summary of the results are included in Tables 2.2 and 2.3, with laboratory results included in Appendix D.

Table 2.2: Atterberg Limit tests – BH2

Depth (metres)	Soil description	Liquid Limit	Plastic Limit	Plasticity Index
2.4	Uncontrolled fill – clayey silt	70	32	38
5.2	Hydraulic fill – sl. clayey silt	67	30	37
8.4	Marine Sediments- sl. clayey silt	60	27	33

Table 2.3: Rock unconfined compressive strength tests

Borehole	Depth (metres)	Bulk density (t/m^3)	Unconfined Compressive Strength (MPa)	Modulus of Elasticity (MPa)
1	10.5	2.10	1.5	253
1	12.5	2.08	0.56	29
2	14.5	2.08	3.3	779

2.5 Subsurface conditions

Based upon the geotechnical investigations undertaken, five geological units have been identified at the site: Uncontrolled fill, hydraulic fill, marine sediments, alluvium and East Coast Bays Formation.

Uncontrolled fill. A veneer of hardfill and concrete covers the site. Beneath this a layer of firm to stiff clayey silts occur. This fill includes lenses of basaltic gravels and isolated rare organics. Measured strengths of the uncontrolled fill varied between 38 kPa and 66 kPa, with cone tip resistances of between 0.4 MPa and 1 MPa.

Hydraulic Fill. Reclamation fill was historically placed by either dredging or pumping marine sediments from the sea bed behind a scoria or basalt bund. The fill generally described as a slightly clayey to slightly sandy silt with beds of shells. This hydraulic fill is often difficult to distinguish from the underlying marine sediments onto which it was placed. The strength of the hydraulic fill ranges between 18 kPa and 30 kPa (undrained shear strength) and CPT cone tip resistances of 0.2 MPa to 0.5 MPa. The cone penetration tests at the eastern end of the site (CPT 6 and 7) identified a thin dense gravel layer, which may be the edge of a reclamation bund. The hydraulic fill is approximately 4 metres thick.

Marine sediments. The original sea bed consisted of slightly clayey silt. The measured undrained shear strength of the sediment varies between 15 kPa and 26 kPa, with cone tip values measured between 0.3MPa and 0.6 MPa. The marine sediments are approximately 3 metres thick.

Alluvium / Weathered ECBF. A stiff layer of clayey to silty sand was encountered above the ECBF rock. The boreholes logged it as alluvium and residually weathered ECBF, with inferences from the CPTs unable to differentiate between the two units of similar strength. The alluvium is a Pleistocene aged (<1 million years old) alluvial sediments of the Tauranga Group. The weathered rockmass is formed from the insitu residual weathering. Cone tip resistances of 2 MPa to 10 MPa were measured, with a thickness of up to 2.5 metres.

ECBF rock. Very weak interbedded sandstone and siltstone of the East Coast Bays Formation exists beneath the site. This geological unit underlies the majority of Auckland forming the sea cliffs around foreshore of northern and eastern beaches. The strength of the rock varies between 0.5 MPa and 3.3 MPa (unconfined compressive strength). The surface level of the rockmass varies between RL-6.5m to RL -8.5m in the north western corner of the site.

Table 2.4: Summarised geological profile

Top of geological unit[1] (m)	Geological unit	Description	Typical undrained strength - Su (kPa)
3.5	Hardfill	Concrete and coarse to fine gravels	N/A
3.0 to 3.1	Uncontrolled fill	Firm to stiff clayey silts	30 to 50
0.5 to -1	Hydraulic fill	Soft to firm clayey and sandy silt	18 to 30
-3 to -3.5	Marine Sediments	Soft clayey and sandy silt	15 to 26
-6 to -6.5	Alluvium	Very stiff clayey silt	100
-6.5 to -8.5	ECBF rock	Very weak siltstone	>500 kPa
Note (1) Top of geological is approximate only, with measurements made in terms of the reduced level in terms of &S 1946 i.e. MSL			

The groundwater level is likely to be partially affected by tidal fluctuations, especially due to the likely presence of leaking stormwater pipes discharging beneath Quay Street. The following levels provide guidance on likely groundwater levels:

- Mean Sea Level RL +0.01
- Mean High Water Springs RL + 1.40
- Highest recorded tide (1936) RL +2.25m

3 Geotechnical Considerations

3.1 General

The following geotechnical issues have been addressed within this desktop report:

- Seismic hazard
- Pile capacity analyses (vertical and lateral)
- Design parameters for shallow foundations

Recommendations and opinions in this report are based on data from geotechnical investigations undertaken on this and adjacent sites using information from our geotechnical database. The nature and continuity of subsoil conditions away from these historic investigations is inferred but it must be appreciated that actual conditions could vary from the assumed model.

3.2 Seismic Issues

3.2.1 Site Subsoil class

Seismic accelerations to be resisted by a structure are dependent upon the stiffness of the underlying soil/rock. Soft soils have the potential to amplify ground accelerations, requiring structures built upon them to be designed to resist a higher seismic coefficient.

The NZGS guidelines (ref 5) recommend the use of the NZS1170 (ref 7) seismic loadings for liquefaction analysis in the absence of site specific seismic modelling. In accordance with the New Zealand Standard and on the basis of the readily available geotechnical data, the site is classified as a Class C (shallow soil) site.

We assess the proposed structure to have an importance level of [2] as defined in NZS 1170 (ref 6). For the proposed structure, the design earthquakes for serviceability and ultimate limit states are therefore 1 in 25 year and 1 in 500 year events respectively. The design peak ground accelerations using this importance level and site seismic classification are summarised in Table 3.1 below.

Table 3.1 - Design peak ground accelerations

Design life (years)[note (1)]	Serviceability limit state (SLS)		Ultimate limit state (ULS)	
	Annual exceedance probability	Peak ground acceleration	Annual exceedance probability	Peak ground acceleration
50	25	0.043	500	0.17
Note (1) Design life and importance level to be verified by the structural engineer/architect/owner, as appropriate. If different from than that assumed, or if this changes during the project life then these values and the opinions in this report will require review.				

3.2.2 Susceptibility to Liquefaction

Seismic liquefaction occurs when excess pore pressures are generated in loose, saturated, generally cohesionless soil during earthquake shaking, causing the soil to undergo a partial to complete loss of shear strength. Such a loss of shear strength can result in settlement, bearing capacity failure and/or horizontal movement of the soil mass. The occurrence of liquefaction is

dependent on several factors, including the intensity and duration of ground shaking, soil density, particle size distribution, and elevation of the groundwater table.

The liquefaction susceptibility of material at the site has been assessed using cone penetration tests and the laboratory plasticity tests. The CPT analyses indicate an extremely low risk of liquefaction in the SLS event within thin lenses potentially liquefiable in the ULS event. Liquefaction may be induced in return periods of approximately 1 in 150 years (ref 10 – fig C3.3). The CPT liquefaction analyses profiles for the ULS event are included in Appendix E.

3.2.3 Consequences of liquefaction - general

Liquefaction related structural damage can generally be characterised by one of, or a combination of, the following ground responses:

(1) Ground settlement
(2) Bearing capacity failure
(3) Lateral spreading

In addition, liquefaction related land damage can generally be characterised by one of, or a combination of, the following features that may also affect ground bearings slabs, pavements and services:

(a) Ground cracking (lateral spreading and/or land oscillation)
(b) Undulating ground surfaces (resulting from differential movement, which may result from lateral spreading, soil densification and/or ejection of liquefiable soils)
(c) Changes to gravitational drainage patterns (both reticulated and surface water systems)
(d) Inundation of the ground surface (or sub-surface structures) with sand and silt

These effects can also lead to increased or decreased vulnerability to liquefaction from subsequent earthquake events and/or an increase/decrease of vulnerability to flooding (resulting from ground level changes).

3.2.4 Potential consequences of liquefaction at this site

Based upon available information and analyses undertaken, the following are potential consequences of liquefaction at this site:

- Liquefaction and densification of the underlying marine sediments, with subsequent surface settlement: liquefaction induced settlements from the ULS seismic event have been calculated at up to 20 to 30mm.
- Lateral spreading towards the Quay Street sea wall to the north: For lateral spreading to occur, the potentially liquefiable layers need to be continuous. Based upon the CPTs undertaken, continuous liquefiable layers are unlikely to be present and therefore the potential risk for lateral spreading is considered minor.
- Lateral loads (dynamic, kinematic and from lateral spreading) being imposed on the existing piles: The magnitude of the kinematic soil loading on a pile from the liquefied soil is proportional to the magnitude of soil settlement. As calculated settlements are relatively small (<30mm), kinematic soil displacements and the potential lateral loading on piles are anticipated to be negligible.

3.3 Foundation Assessment

3.3.1 Existing Piles

The machine borehole at Pile 46 (refer Section 2.3) indicated the rock level was at a similar level to that inferred from the 1968 geotechnical investigations (±0.5m). But the pile founding level was considerably higher (2.4m higher) than originally inferred. The founding level for the Pile 46 was measured at 2.4 metres below rock level. It is considered likely that the belled pile was taken to sufficient depth into the rock to develop the pile bell (≈2m) and still have a temporary steel casing above – slightly socketed into rock. Therefore pile embedment lengths are likely to be less than originally designed, with a minimum embedment into rock of 2.5 metres.

3.3.2 Vertical pile capacity

The existing piles consist of bored reinforced concrete piles founding in the underlying ECBF rock (ref 3). Several of the piles were constructed with enlarged bases (bells). The capacity of a range of the existing piles has been undertaken using the assessed design parameters presented in Table 3.2 and published design methodologies (ref 15 – pp313).

Table 3.2: Pile design parameters

Design parameter			Units	Magnitude
Geotechnical ultimate end bearing of pile founded at least 3 pile diameters into ECBF rock (linearly reduced to ⅓ of this value for a pile with no embedment within ECBF rock)		f_{ub}	kPa	6,000
Geotechnical ultimate skin friction within ECBF rock (for belled piles, the lower 2 metres of skin friction is ignored)		f_{su}	kPa	500
Design parameters for calculating uplift capacity of "cone" pullout Note (1)	Effective friction angle of ECBF rock	ϕ	deg	40
	Effective cohesion of ECBF rock	c'	kPa	50
	Submerged unit weight of overlying soil	γ'	kN/m^3	7
Note (1) Alternatively adopting a cylindrical cone failure through the rockmass of the diameter of the bell				

The calculated pile capacities, based upon interpolated ECBF rock levels are presented in Table 3.3 below. Given the uncertainty of the construction and the potential variability of the surface of the underlying ECBF rock (ref 1), a strength reduction factor (ϕ_g) of 0.5 is considered appropriate when comparing with Ultimate Limit State loads. Pile numbers refer to the original construction drawings (ref 2).

The settlement of the piles may be estimated using a vertical spring applied at the base of the pile. Based upon a rock stiffness (E') of 300 MPa, the following vertical spring stiffness (k_v) can be used:

$$k_v = \frac{400}{B} (\text{kPa/mm})$$

where B = diameter of pile (metres)

Table 3.3: Pile design parameters

Pile No			1	31	38	44
Original pile design loads (ref 4) note (1)	Compression	kips	1,280	450	225	1,280
	Tension	kips	1,280	55	-	1,280
	Compression – ULS	kN	5,696	3,004	1,502	5,696
	Tension - ULS	kN	5,696	245	-	5,696
Pile shaft diameter		m	1.22	0.91	0.91	1.22
Pile bell diameter		m	2.29	1.22	0.91	2.29
Design pile founding RL (ACC datum)		m	-14.0	-11.6	-12.5	-11.6
Design pile founding RL (L&S datum)		m	-12.8	-10.4	-11.3	-10.4
Estimated RL of ECBF rock (L&S datum)		m	-7	-6.5	-7	-6.5
Pile founding level (ref Sec 3.3.1) – L&S		m	-9.5	-9	-9.5	-9
Geotechnical ultimate capacities(3)	Shaft capacity – compression note (2)	kN	958	715	3,574	958
	End bearing	kPa	4,728	5,366	5,832	4,728
	End Bearing	kN	19,472	6,273	3,793	19,472
	Total compression	kN	20,431	6,988	7,366	20,431
	Uplift capacity	kN	9,000	4,790	3,574	9,000

Note (1) Design loads are the maximum of unfactored working load or unfactored seismic load.
1 kip = 1,000lb = 4.45kN
To obtained ULS loads, gravity loads are factored by 1.5 and seismic loads factored by 1.0.

(2) Based upon skin friction within ECBF rock only

(3) Geotechnical ultimate capacities need to be factored by a strength reduction factor to calculate the maximum ultimate limit state load

3.3.3 Lateral pile capacity

An assessment of the lateral deformation of four piles has been undertaken in accordance with our proposal (ref 18). The piles were modelled with a range of potential seismic lateral loads to enable the projects structural engineer, Structure Design, to interpolate the lateral stiffness of the foundation piles. The analysis of the lateral springs has been undertaken using proprietary software (ref 3) which calculates non-linear p-y curves for the geological model developed. Four representative piles to be analysed were selected by Structure Design. A summary of the geotechnical model developed for the four piles is presented in Table 3.4.

Structural details of the existing piles have been assessed from historical records (ref 3&11). Pile concrete has been modelled as linear elastic, with the second moment of area calculated from the sectional properties. A concrete stiffness of 26 GPa has been adopted. The pile vertical loads have been calculated by Structure Design (ref 14). A range of lateral loads have been analysed, as the actual pile load will depend upon the stiffness of each pile. The range of lateral loads varies up to 800 kN. The top of the pile has been modelled as fixed; i.e. full moment fixity into the pile cap. A summary of the pile model is presented in Table 3.5.

Table 3.4: Lateral pile geological model

Geological unit	Unit weight – γ (kN/m^3) [note (1)]	Undrained shear strength – Su (kPa)	Effective friction angle - ϕ (deg)	Non-linear spring model
Gravel Fill	18	-	35	Sand – "Reese"
Uncontrolled Fill	17	40	-	
Hydraulic Fill / Marine sediments	17	20	-	Soft clay – "Matlock"
Alluvium / RW ECBF	18	100	-	Stiff clay without free water
ECBF Rock	20	380	-	Stiff clay without free water
Note (1) Effective (or buoyant) unit weight used below RL 1.1m (2.4metres depth)				

Table 3.5: Lateral Pile Model

Pile No		1	22	31	44
Pile Diameter (m)		1.22	0.91	0.91	1.22
Pile Bell diameter (m)		2.34	1.22	1.22	2.29
Pile vertical load (kN)		8,600	1,400	1,400	8,600
Pile detail / top of geological unit (in terms of L&S Datum)	Gravel fill	+3.5	+3.5	+3.5	+3.5
	Uncontrolled Fill	+3.0	+3.0	+3.0	+3.0
	Hydraulic Fill / MS	0.0	0.0	0.0	0.0
	Alluvium / RW ECBF	-6.5	-6.5	-6	-6
	ECBF Rock	-7	-7	-6.5	-6.5
	Inferred top of bell	-7.5	-7.5	-7.0	-7.0
	Pile founding level	-9.5	-9.5	-9	-9
Pile detail / top of geological unit (below ground level)	Gravel fill	0	0	0	0
	Uncontrolled Fill	0.5	0.5	0.5	0.5
	Hydraulic Fill / MS	3.5	3.5	3.5	3.5
	Alluvium / RW ECBF	10	10	9.5	9.5
	ECBF Rock	10.5	10.5	10.0	10.0
	Inferred top of bell	11	11	10.5	10.5
	Pile founding level	13	13	12.5	12.5

Lateral pile analyses for the four piles analysed are included in Appendix F. For each pile analysed the following is presented:

- A summary geological section
- A numerical presentation of input and output data. Twelve seismic load cases were analysed with various magnitudes of lateral loads
- A graph of the lateral deformation of the pile from the 12 lateral loads analysed
- A graph of the bending moments within the pile for the 12 lateral loads analysed
- A graph of the shear force within the pile for the 12 lateral loads analysed

- A graph of the pile head deflection for the 12 lateral loads analysed
- A graph of the maximum bending moment within the pile for the 12 lateral loads analysed

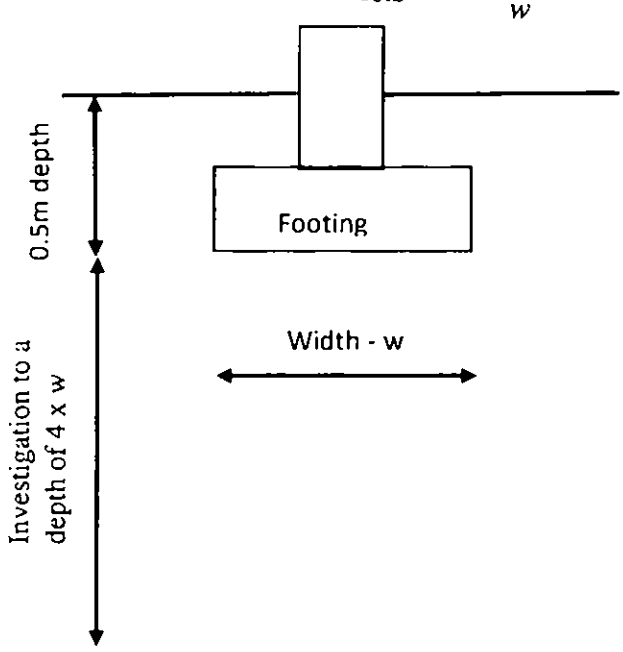
3.3.4 Foundations for single level extension

The redevelopment of the Copthorne Hotel will involve new single level structures on the northern side of the existing structure. The geotechnical investigation has identified the surficial soils consist of a thin "crust" of hardfill and stiff reclamation fill, becoming softer and more compressible with depth. With "bedrock" at approximately RL -6.5m (10 metres depth).

The following foundation options may be considered to support this structure:

- Piles extended to the underlying rock. Detailed design and design parameters would need to be assessed, but the following pile options may be considered:
  - Driven timber poles extended to the underlying rock at a depth of approximately 10 metres. The piles would need to be pre-drilled through the surficial hardfill, with consideration given to the likely minor vibrations generated during installation
  - Driven universal columns extended into the underlying rockmass with an approximate length of 12 metres. Vibrations generated by driving are likely to be slightly greater than that for timber poles.
  - Screw piles extended to the top of the rock mass. Minimal vibrations are generated by pile installation.
- Shallow foundation of limited size and load – to avoid overstressing the underlying soft and potentially compressible reclamation fill and marine sediments (as shown below). There remains the risk of settlement of these foundations that may result in minor cracking. To minimise this risk the foundations would need to be designed in accordance with the following:
  - Foundations 0.5m below the surface (to minimise potential effects of the seasonal shrink swell of soil and to minimise the risk of overstressing underlying soils).
  - Maximum applied ultimate limit state stress (q_{ULS}) over a foundation width (w) calculated as follows:

$$q_{ULS} = 25 + \frac{25}{w} \leq 100 \quad \text{(kPa)}$$



  - Structural detailing to accommodate potential differential settlements
  - Following excavation of shallow foundations, it is recommended that hand augured boreholes (with insitu strength testing) be undertaken to confirm the above design parameters are applicable. The boreholes should extend to a depth of at least "4 × foundation width" and be placed at a spacing of approximately 5 to 10 metres along the foundations. In the event that the boreholes indicate the ground is weaker than anticipated, settlement reducing piles or undercut and replacement with ground structural polystyrene may be required.

4 Applicability

This report has been prepared for the benefit of Millenium & Copthorne Hotels NZ Ltd with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

During excavation and construction, the site should be examined by an engineer competent to judge whether the exposed subsoils are compatible with the inferred conditions on which the report has been based. We would be pleased to provide this service to you and believe your project would benefit from the continuity. However, it is important that we be contacted if there is any variation in subsoil conditions from those described in the report.

Tonkin & Taylor& LTD

Environmental and Engineering Consultants

Report prepared by:	Authorised for Tonkin & Taylor Ltd by:
.....	.....
Andrew Langbein	C J Freer
Senior Geotechnical Engineer	Project Director

acl

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5 References

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   b. (May 1968) Bored Piles Layout & Part Schedule. Dwg No : S.2B.H.1
   c. (May 1968) Bored Piles Part Schedule. Dwg No : S.3B.H.2

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13 Structure Design Ltd (13 October 2014 @ 7:50pm). Email from Ashley Smith to Andrew Langbein. *RE: Soil disturbance & Geotech testing; Copthorne Harbour City 196-200 Quay Street.*

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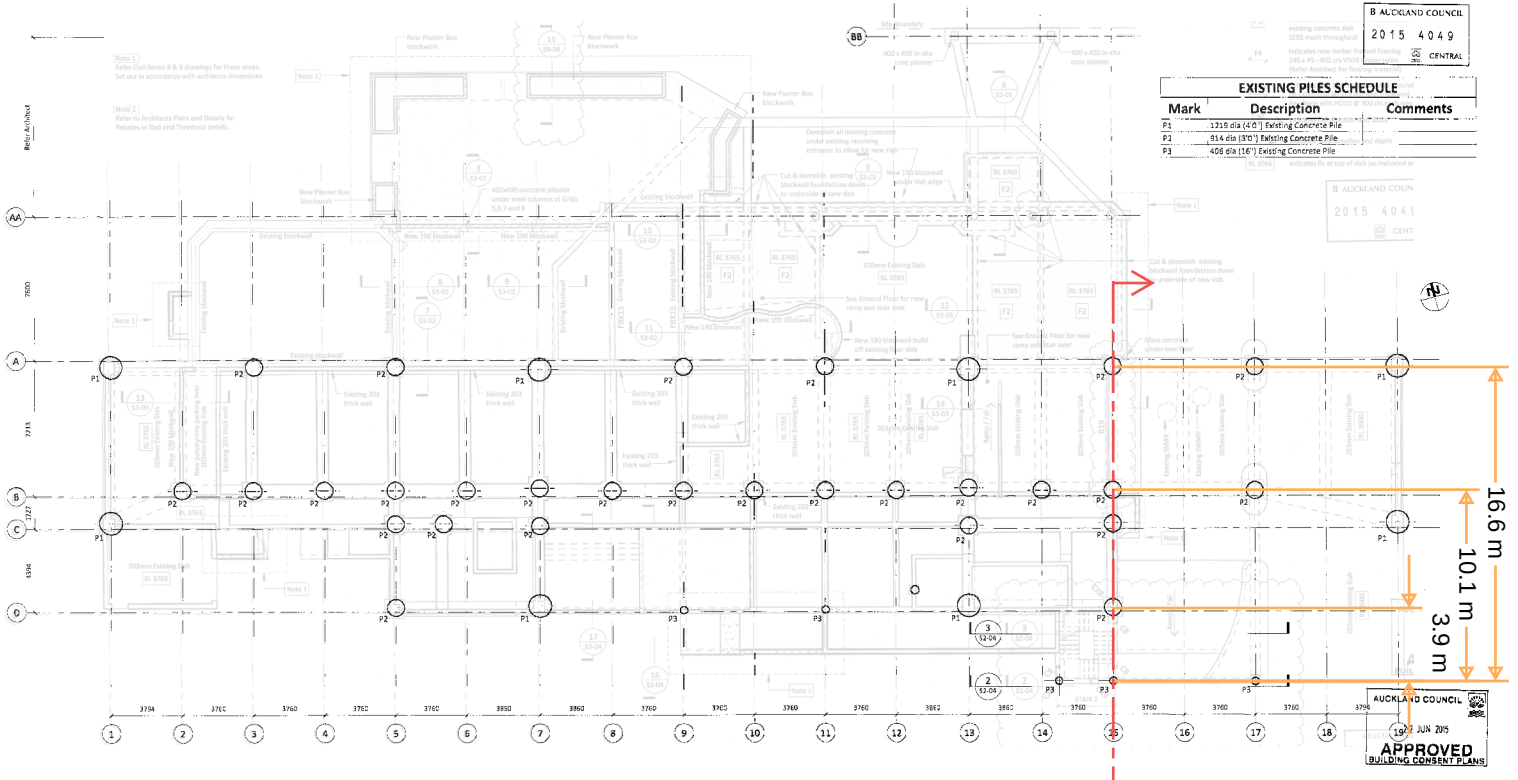
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A1



NOTE
Locations of new penetrations for services through existing concrete are to be confirmed on site and approved by the Structural Engineer prior to cutting.
In all cases penetrations are not to cut through existing reinforcing, the Contractor is to scan for reinforcing to confirm locations.
No penetrations are permitted through the shear wall coupling beams on grids 7 and 13.

A	24/04/15	ISSUE FOR CONSENT
B	12/05/15	Structural Design Update
C	29/05/15	Structural Design Update
REV	DATE	BY

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1:100 Scale 0 1000 2000 3000 4000 5000 6000
1:1 Scale 0 20 30 40 50 60

This SCALE BAR is provided to ensure the sheet is printed to its proper size. If this is not the case it is not to be used for measuring dimensions.

NOTES:
1. These drawings are to be read in conjunction with all relevant Architects and Engineers drawings and Specifications.
2. The structural slabs shall be treated as general arrangements, refer to Architects drawings for levels and setout.
3. Refer to engineer for any dimensions not given or obtainable from the structural details. Do not scale from drawings.

ARCHITECT:
SUSAN LEE ARCHITECTS LTD
Tse Architects

ENGINEER:
Structure Design

PROJECT TITLE:
COPTHORNE HARBOURCITY REFURBISHMENT
196 - 200 Quay Street, Auckland City

SHEET TITLE:
PILE LAYOUT PLAN

DRAWN BY:	G.R	SCALE:	1:100
APPROVED	AS	FILES NAME	
JOB No:	1551	SHEET No:	S1-00
REV	4		

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APPENDIX B
CMW Gap Analysis Letter

19 February 2026 Document Ref: AKL2025-0064AE | Rev 1

Millenium & Copthorne Hotels New Zealand Limited
Cc Forme Planning Limited

Attention: Louise Borton and Kay Panther Knight

Dear Louise and Kay,

RE: Downtown Carpark Developments – T&T report update
2 LOWER HOBSON STREET, AUCKLAND CBD

1.0 INTRODUCTION

Modifications from the previous letter Rev0 are indicated in blue.

1.1 Scope

CMW Geosciences (CMW) was engaged by Millenium & Copthorne Hotels New Zealand Limited to carry out a geotechnical assessment of the potential impacts to the adjacent M Social Hotel of the proposed Downtown Carpark development along the shared boundary located at 2 Lower Hobson Street, Auckland CBD.

The scope of work and associated terms and conditions of our engagement were detailed in our services proposal letter referenced AKL2025-0064AA, Rev. 0 dated 14 April 2025 and Variation Order (email) dated 21 November 2025.

This letter is to inform Millenium & Copthorne Hotels New Zealand Limited of potential geotechnical hazards, present gap analysis based on our assessment and to present our recommendations for geotechnical investigations and geotechnical works for the proposed development. This letter may be used in discussions with Precinct Properties Holdings Limited to define preliminary requirements and mitigation strategies.

1.2 Reviewed documents

The following documents have been reviewed during the assessment and preparation of this report. These are attached in Appendix B.

1. Downtown West – Downtown Car Park Redevelopment Geotechnical and Groundwater Assessment Report (Ref. 1016043.2000 v3 dated October 2025) by Tonkin & Taylor Ltd.
2. Travelodge extensions by Peddle, Thorp & Horman, dated December 1978: Existing ground floor plan (job 779167, drawings D1), relocation of existing underground fuel oil tank (job 048035, drawings M1) and oil tank enclosure (drawing 76145:S1)

3. Extension to Downtown Auckland Travelodge by Brickell, Moss, Rankine & Hill, dated 1980: Details new lift to eastern block (Drawings 76318-S7A and S8).
4. Travelodge Downtown – Alterations to staff facilities by Wass Buller & Associates Ltd dated May 1986: Layout, Site floor Plan, Floor Plan, Elevations Cross sections (Project 8527, Drawings SD 191, (00)1,(0)1, (0)2) and Foundation Plan (Job R/4971 Drawing 80F9 sheet 1 of 1).
5. Geotechnical Preliminary Assessment Report by CMW Geosciences (ref. AKL2025-0064AB rev.1 dated 13 June 2025)
6. Assessment of effects from Downtown West Development Structural Memorandum, Structus, dated 11 June 2025.
7. Statement of Evidence of Darren Mitchel before Commissioners appointed by Auckland Council, dated 17 June 2025.

1.3 Initial assessment

A 2D Plaxis finite element analysis was undertaken as part of CMW Preliminary Assessment Report (ref. AKL2025-0064AB rev1 dated 13 June 2025) to assess the effects of the proposed 20 m deep basement excavation on the adjacent M Social Hotel. The modelling, incorporating HS Small soil parameters, high-tide groundwater conditions, reduced long-term concrete stiffness, hotel pile loading, staged construction, and a 25-year return period earthquake, indicates a maximum lateral displacement of approximately 55 mm, occurring at the 914 mm diameter hotel piles near Grids B-15 and C-15, with movements concentrated toward the lower half of the diaphragm wall.

Separate assessments by T+T indicate combined settlement of the hotel of approximately 12 mm, with limited differential settlement.

These findings highlighted the need for detailed design to explicitly address tidal groundwater effects, potential lateral movement of existing piles, slab and wall stiffness sensitivity, and rock mass behaviour, together with stringent construction tolerances, enhanced real-time monitoring, and early-warning systems to manage excavation-induced risks.

2.0 REVIEW AND ANALYSIS

It is noted that the geotechnical studies provided were prepared to support the Resource Consent application. Consequently, several geohazards are intended to be revisited at the detailed design stage. However, based on our professional expertise, these matters warrant consideration at the current stage, as they have the potential to affect the overall feasibility of the project and, if not adequately addressed, could result in irreversible damage to the neighbouring property.

Accordingly, critical conditions to construction should be identified and addressed at this stage rather than deferred as described in the following table.

Item	Geohazard	Assessment outcome (T&T report)	Assessment outcome CMW on updated report	Additional works/ recommendations	Comments
1	Lateral variation of geotechnical layers	Reclamation fill thickness & nature variation (section 3.4.1 p.19)	Potential variability of reclamation fill (nature, permeability), roof of the weathered and unweathered ECBF will have consequences on diaphragm wall design and construction	Monitoring of diaphragm wall construction with deepening with lateral variation? Deepening would rquire modification of the reinforcemntn and increase of movements	Construction challenges in reclamation fill noted by T&T in Seismic strengthening Auckland's waterfront (2021)
2	Groundmodel parameters	Permeability parameters based on near-by Commercial bay projects (section 4.4.3 p.27) Equivalent stiffness defined in table 4.4 section 4.4.3 p.27 Geotechnical parametrer defined for retaining structure in section 4.5.2 p.33	Equivalent stiffness of 20MPa in the Pleistocene alluvium is very optimistic Ultimate end bearing @10MPa in ECBF - can it be justified based on dilatometer testing (not pushed to pressure limit)?	Site specific testing of permeability coefficients / stiffness / bearing capacity Sensitivity testing on parameters in the meantime	AS2159 – 2009. Australian Standard: Piling - Design and Installation. SANZ skin friction
3	Seismicity	Class B-rock for foundations Class C - shallow soil site for the diaphragm wall IL3 M=5.9 ULS=0.20 SLS=0.05	Soil classes are coherent with known geotechnical conditions, foundation and retaining structure geometry. Peak ground acceleration and magnitude are consistent with Module 1 for IL3 structure with a 50-year design life		
4	Liquefaction / Cyclic softening	Discrete bands within the marine sediments and reclamation fill (discontinuous) Site specific investigations to be conducted at detailed design (section 4.1.2 p.17)	Lateral spread noted by T&T in Seismic strengthening Auckland's waterfront (2021)	Inconsistency between the 2 projects to be assessed. Exising CPT allow for this assessment to be conducted at this stage	
5	Global stability / Lateral stretch		Lateral spread noted by T&T in Seismic strengthening Auckland's waterfront (2021)	See above	
6	Stability/Diaphragm wall	Max deflection = 24mm (section 4.5.10 p37) Potential for panel "lift off" the face of the rock in case of water pressures in the ECBF to be remediated by ground anchors/pre-stressing lower levels- to be considered at detailed design (section 4.5.11 p.38)	Max deflection 55mm based on Plaxis 2D model with loss of lateral confinement around piles	Assuming M Social hotel is on piled foundation as it is not. Taken only 12kPa surcharge Noted that displacement were at the surface => therefore OK but not	
7	Groundwater impact (mounding/draining)	Groundwater monitoring (3 months/3 years) Groundwater mounding risk is assessed as low due to high permeability within the reclamation fill (section 4.4.9 p.31) to be reassessed at detailed design with possible remediation by highly permeable trenches Groundwater & settlement monitoring - Appendix B	The shallow GWT influenced by the tidal recharge comes up to 2m bgl at BH01 location (M Social side? There is 2 BH01-named GEO and CT?) is likely to rise with the presence of the diaphragm wall Long-term changes to groundwater flow paths caused by the diaphragm wall acting as a cut-off Consequences : impact on slabs, underground services, modifications in effective stress (limited impact on skin friction?), corrosion on structure not designed for?	Hydrological modelling of temporary works and permanent solution to estimate impact and to be provided with full-data set of groundwater monitoring covering tidal recharge over a full-year	Highly permeable trenches down to effective depths (at least 2m bgl) will be difficult to build considering the position of the diaphragm wall in respect to the property boundary

8	Saline intrusion	Managed through cutoff walls on the northern/eastern/western sides, possible imperfections in the wall, construction joints, or natural discontinuities within the rock mass could provide pathways for seawater ingress [which] could result in elevated groundwater inflows during construction or long-term operation [and] require localised grouting to seal joints (section 4.7.2 p.44)	Ineffectiveness of groundwater cutoff wall could result in fine contents migration and loss of support of the M Social building Feasibility of seal joints if discontinuities in ECBF when discovered at earthworks stage are linked to artesian GWT		
9	Dewatering	Analysis conducted for 2 cases (transient/temporarary works and steady-state) with SEEP/W and SIGMA/W show induced settlements of less than 5mm Dewatering induced settlements for M Social building are ,10mm (section 4.6.4 p 41) Groundwater inflows are estimated at 20-40 m3/day at diaphragm wall (section 4.4.8 p.24)		Tidal influence to be taken into account as per comments in preliminary	
10	Aggressive environment		see comment on saline intrusion Depending on the environment aggressivity and importance level of the retaining structures, appropriate remediations will be required (for example lining for the sheetpile wall).		
11	Construction methodology (impact on M Social lot)		Diaphragm wall construction: - Horizontal ground movements induced by trench excavation and panel installation. - Vertical settlement or heave due to stress relief, slurry pressure imbalance, or concrete placement. - Loss of bentonite/polymer slurry into permeable strata or defects (potential ground loss). - Surface settlement of services caused by slurry infiltration. - Trench collapse at shallow depth affecting M Social lot/ Erosion beneath guide walls, resulting in loss of near-surface support. -Vibrations from excavation equipment, reinforcement cage handling, or concrete placement and resonance effects on M Social buildings. Excavation of the ECBF will require chiseling (drop of a chisel) which may cause noise and vibration or use of a rotary tool. - Guide Wall encroachment onto M Social property boundaries. - Load transfer from guide walls or construction plant onto adjacent ground or structures. - stability of guide wall (usually 0.8 to 1.5m deep/ bgl) Inappropriate panel sequencing leading to cumulative ground movements. - Delays between excavation and concreting, increasing instability risk. - Emergency response limitations in constrained urban	Guide wall / stability of trenches (temporary works design => has to be part of a Hold point for review) Surcharge of 12 kPa taken into account (just for traffic?) seems insufficient for the part of M Social building on shallow foundations Clarification of the diaphragm wall construction methodology at an early stage is essential to assess risk to M Social property and also as an input for the noise management.	Likely to impact Noise/Vibration

			environments/Delayed response to exceedance of movements threshold.		
		Temporary ground anchors borehole collapse to be supported by casing or slurry (section 7 table 7.2 risk 3)	Use of grouted temporary anchors will prevent their removal. Permanent anchors will limit development of M Social lot.	Self drilling anchors are an option Does the client allow for drilling under the property? Penalty for not remonving anchors?	Likely to impact Noise/Vibration
		Clash between proposed temporary ground anchors and the pile foundation of neighbouring building (section 7 table 7.2 risk 5) - anchors layout to avoid clashes with existing based on as-built	Interaction between ground anchors and existing piles reducing skin friction Possible inconsistency between as-built and built piles Depending on length, with a few degrees deviation horizontally/vertically designed ground anchors could still interact with existing piles	What is T&T proposed methodology to ensure ground anchors are not interfering with M Social foundations?	Likely to impact Noise/Vibration
		"where piles conflict with perimeter RW[…][they] may need to be removed" (section 2.4 p13) Inground obstructions (section 4.7.6 p.39)	Existing/remnant foundations of parking (piles) vs diafragm wall deviation/construction. Piles (up to 11m) inferred to be longer on the northern site on the site (section 2.4 p.13/Appendix C), ie M Social side	How do you remove the piles along the boundary? Possible to encroach onto M Social property or create cavity. Backfilling with uncompacted material to allow for the diaphrag wall to be constructed may cause over-excavation, cavities, impact on M Social foundations. Clarification of the pile removal methodology at an early stage is essential to assess risk to M Social property and also as an input for the noise management.	
		Driven UC piles noise and vibration (section 4.7 p.)+ sheetpiles noise and vibration (section)	Driven UC piles and sheetpile installation might require hammering to penetrate the ECBF and gain sufficient embedment. Refusal of the sheetpiles on the ECBF required hammering at a nearby project (Commercial Bay) which generated significant noise and vibration.	Clarification of the piles and sheetpiles construction methodology and refusal management on ECBF at an early stage is essential to assess risk to M Social property and also as an input for the noise management. Alternatively soldier piles and shotcrete could be considered as a retaining solution with less noise and vibration disruption	Likely to impact Noise/Vibration
		Proposed excavations and retaining structures may cause settlements and displacements on the M Social structures. Risk is elevated due to spatial variability and uncertainty associated with reclaimed fill.		Instrumentation is recommended to establish baseline structural and ground behaviour, detect early ground movements during excavation and retaining works, provide trigger-based control for construction activities and protect M Social structures. We recommend the following instrumentation to be included in the consent conditions: • baseline monitoring prior to works and increase monitoring frequency during critical excavation stages. • survey settlement markers between the excavation and the building to monitor vertical ground movements. • building settlement points and prisms on the structure to detect total and differential settlement. • Inclinometers between the excavation and the structure to monitor lateral ground deformation. • Monitor retaining wall movements using survey prisms and, where appropriate, in-place inclinometers. • Groundwater fluctuation monitoring along the northern boundary • Consider deep settlement gauges or extensometers to differentiate settlement within fill and underlying soils. • Adopt trigger levels to enable early intervention if movements exceed predicted performance.	

12	Works impacting on future development of M Social lot	Noise and vibration: piles would limit the noise and vibration	Vibration would transmit through ECBF to M Social building	Remediation?	
			Diaphragm wall acting as a groundwaterflow barrier/cutoff		
			Has the diaphragm wall long term deflection and/or induced settlement been considered?		
			Sensitivity of Diaphragm wall to future works: what would be the future restrictions to constrcution? If the diaphragm wall is sensitive to excavation works, should it be displacements further within Downtown carpark lots to allow full development of the M Social site?		
			Construction of a diaphragm wall as a permanent retaining structure will impact posibility for M Social to develop a basement and make basement contruction more onerous as (for example) no ground anchors could be used.		
			Temporary anchors removal/uncertainty: Temporary ground anchors which could not be removed after construction will obstruct and raise issues for construction of basement levels		Likely to impact Noise/Vibration
			Unsupported lateral loads from diaphragm wall, if excavation is required at the M Social location for the development of underground levels		

3.0 CLOSURE

Additional important information regarding the use of your CMW report is provided in the '*Using your CMW Report*' document attached to this report.

This report has been prepared for use by inform Millenium & Copthorne Hotels New Zealand Limited in relation to the Downtown Carpark development along the shared boundary located at 2 Lower Hobson Street, Auckland CBD. in accordance with the scope, proposed uses and limitations described in the report. Should you have further questions relating to the use of your report please do not hesitate to contact us.

Where a party other than inform Millenium & Copthorne Hotels New Zealand Limited seeks to rely upon or otherwise use this report, the consent of CMW should be sought prior to any such use. CMW can then advise whether the report and its contents are suitable for the intended use by the other party.

For and on behalf of CMW Geosciences

Prepared by:



Hai An Nguyen-Bui
Associate Geotechnical Engineer

Reviewed and authorised by:



Martin Larisch
Principal Geotechnical Engineer

Distribution: 1 electronic copy to Millenium & Copthorne Hotels New Zealand Limited via email
Original held at CMW Geosciences

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APPENDIX C

Building conditions modifications and recommendations

Comments for Kay	Consent condition	Page	Geohazard/risk	Relating to item no. of CMW Gap Analysis Letter AKL2025-0064AE Rev.1	Initial wording	Recommended change	Formal consent condition (proposed wording)	Justification
Important	38–41, 164–165 (Monitoring frequency)	14–15, 50–51	Insufficient monitoring frequency and continuity, Undetected diaphragm wall and building movement during critical stages	2, 6, 7, 12	Monitoring frequency defined in GSMCP; reactive increases after triggers	Require continuous monitoring during high-risk stages with controlled relaxation	Monitoring shall include continuous automated groundwater monitoring and stage-dependent monitoring frequencies during diaphragm wall construction, excavation and dewatering. Continuous automated monitoring of diaphragm wall performance and adjacent structures (including M Social) shall be undertaken during diaphragm wall construction, excavation and dewatering. Monitoring frequency may only be reduced following demonstration that movements are stable and within acceptable limits, to the satisfaction of a SQEP and the Council Monitoring frequency shall be proactively adjusted based on risk and not solely in response to trigger exceedances	CMW notes tidal effects and parameter uncertainty require continuous and proactive monitoring Also addresses highest risk phase and ensures early detection of unexpected behaviour
Important	41 (Pre-construction surveys)	15	Baseline surveys not linked to risk	8, 12	Surveys required before works	Require building sensitivity assessment	Pre-construction condition surveys shall include assessment of building sensitivity, foundation type, and development of property-specific movement thresholds for each affected structure	Need to relate predicted movements to actual building vulnerability
Important	90–91 (Construction methodology)	27	Excavation and support methodology may not control deformation	5, 6, 9, 11	Methodology required based on T+T report	Require independent design verification and deformation control	A detailed draft construction methodology must be based on detailed design and shall be submitted at the earthworks/building (?) consent application, including excavation sequence, temporary works, dewatering strategy, predicted deformations, and mitigation measures to control movement	CMW notes uncertainty in modelling and lack of demonstrated methodology to control impacts
Important	106 (Access limitations)	30	Reduced monitoring coverage at adjacent sites	12	Alternative monitoring allowed if access unavailable	Require conservative approach where access denied	Where access to adjacent properties is not available, conservative movement thresholds shall be adopted and supplementary monitoring implemented to ensure equivalent protection	Ensures neighbouring buildings remain protected even with limited access
May be relaxed	157 (Dewatering notification)	47	Dewatering may commence without adequate review	7, 9	10-day notice required	Require updated modelling and monitoring submission	The 10-day notification period requirement is acceptable where all relevant information, including groundwater monitoring results and updated modelling, has been provided to the Council in advance and is considered sufficient to support commencement of dewatering.	Addresses uncertainty in groundwater behaviour
Very Important	158 (Retaining wall design)	47	Underestimation of wall deflection and ground movement	5, 6, 8, 11, 12	Design per T+T report	Require independent FE analysis and SSI assessment	**Must be carried out with Detailed Design carried out based on Detailed Design report.** Retaining wall and excavation design shall be verified by independent finite element analysis, including soil–structure interaction and parameter variability, to demonstrate predicted movements are acceptable	CMW identified discrepancies in modelling and insufficient treatment of SSI
Important	162–163 (Alert and Alarm Levels)	49–50	Delayed detection and response to excessive deformation, risking damage to adjacent structures	2, 6, 7, 12	Monitoring increased only after exceedance; notification within 24 hours; periodic measurements	Modify to require continuous monitoring and immediate notification at Alarm Level	Monitoring of all relevant parameters (including wall deflection, ground movement, and building deformation) shall be undertaken on a continuous basis using automated systems (real-time data). In the event of any Alarm Level exceedance, the consent holder shall immediately notify the Council and the SQEP upon detection, and implement contingency measures. Alert Level exceedances shall trigge immediate review of construction activities and mitigation measures	Current conditions rely on periodic/reactive monitoring and allow delays (e.g. 24 hours) in notification. Given CMW concerns about uncertainty and potential rapid changes during excavation and dewatering, continuous monitoring and immediate notification are required to prevent damage
May be relaxed	164–165 (Detailed GSMCP)	50–51	Monitoring plan based on incomplete data	1, 2, 7, 8, 9	GSMCP required before dewatering	Require additional investigation and parameter validation	The GSMCP shall include additional investigation where existing data is insufficient, and full justification of stiffness, permeability and groundwater parameters used in design	CMW notes boreholes and testing are limited and statistical representativeness has not been demonstrated
May be relaxed Ask structural engineer if 6 is enough or where?	166 + Schedule B (M Social + buildings)	51–53	Risk of damage to adjacent structures	6, 8, 11, 12	Detailed surveys and monitoring required	Require building-specific risk assessment and limits	Building-specific assessments shall define acceptable movement limits, monitoring requirements, and mitigation measures for each structure, including M Social Hotel	Directly addresses key stakeholder risk
Very Important	171(Dewatering)-173 (Diaphragm wall monitoring)	57	Undetected wall movement leading to damage to adjacent structures	5, 6, 8, 11, 12	Monitoring required (not clearly continuous or instrumented)	Require mandatory continuous monitoring using data loggers and inclinometers	Monitoring of groundwater, vertical movement of the buiding and diaphragm wall performance shall be undertaken using automated (real-time) data loggers to enable continuous measurement. A minimum of three (3) inclinometers shall be installed along the length of the diaphragm wall, with monitoring undertaken continuously throughout diaphragm wall construction, excavation, and dewatering, and maintained until movements have stabilised to the satisfaction of a SQEP and the Council	CMW identified uncertainty in predicted wall deflections and risk to adjacent buildings. Continuous monitoring with sufficient instrumentation is required to detect deviations early and manage risk proactively
Important	175 (Notification of exceedance / incident)	58	Delay in notifying exceedances or incidents, leading to unmanaged risk to adjacent structures and stakeholders	6, 7, 11, 12	Notification to Council required (within a specified timeframe, e.g. 24 hours)	Require immediate notification to Council and affected asset owners	In the event of any exceedance of Alert or Alarm Levels, or any incident with potential to affect adjacent structures or services, the consent holder shall immediately notify the Council and all directly affected asset owners (including M Social Hotel) upon detection, without delay.	Immediate notification is critical to ensure timely response, particularly over weekends or public holidays. This is especially important for sensitive stakeholders such as M Social Hotel, where delays could increase risk to occupants and operations
Important	176 (Monitoring reporting / communication)	58	Lack of timely communication of monitoring results to affected stakeholders, leading to unmanaged risk	6, 7, 11, 12	Monitoring results reported to Council	Require monitoring data to be communicated to affected stakeholders	All monitoring data, including groundwater levels, wall movements, and building deformation, shall be made available to the Council and directly affected stakeholders (including M Social Hotel) in a timely manner. Where automated monitoring is implemented, access to near real-time data shall be provided where practicable. Regular summary reports shall also be provided at a frequency agreed with the Council and stakeholders.	Given the sensitivity of adjacent structures and the operational nature of M Social Hotel, timely access to monitoring data is essential to allow stakeholders to understand risks, respond appropriately, and maintain safe operation. Continuous monitoring is only effective if information is shared promptly
Very Important	Advice notes 15–17	62	Non-enforceable requirements; ambiguity in design/monitoring expectations	2, 5, 11	Advisory wording using “should”; unclear reference to drawings	Convert to enforceable condition and clarify scope	All requirements currently identified as Advice Notes 15–17 shall be incorporated into enforceable consent conditions. All instances of “should” shall be replaced with “shall”. Any reference to drawings or documents shall be updated to reflect only those that have been prepared and submitted, or otherwise require submission for Building Consent and revision of conditions	



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