Dunning Thornton consultants

Structural Engineers

Embassy Theatre (10 Kent Terrace) Detailed Seismic Assessment Report

Job number: 8651

Report prepared by: Rubie Inch

Report reviewed by: Ryan Clarke

Embassy Theatre (10 Kent Terrace) Detailed Seismic Assessment Report

Contents

Report Rev C 23/05/2025- Ryan Clarke

This report has been prepared for Wellington City Council for seismic assessment purposes to communicate the likely seismic capacity of the Embassy Theatre (10 Kent Terrace) structure. It shall not be used by others or for alternate purposes without the approval of Dunning Thornton Consultants Ltd.

1 Executive Summary

1.1 Background

This report has been carried out for Wellington City Council with the intention of establishing the likely seismic strength of the Embassy Theatre (10 Kent Terrace), using the MBIE publication *Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments,* July 2017 and expressed in terms of *%NBS* (New Building Standard).

The Embassy Theatre building is located at 10 Kent Terrace, Mount Victoria, Wellington on the corner of Majoribanks and Kent Terrace. DTC completed design and construction monitoring of earthquake strengthening on this building in two stages from 2001-2003 and 2009-2010. The 2001 redevelopment strengthening targeted 67%NBS of NZS4203:1992 and the 2009 strengthening targeted 80%NBS(IL3) of NZS1170.5 loadings. Both retrofits were done prior to the MBIE seismic assessment guidelines.

1.2 Building Description

The Embassy Theatre was originally constructed in 1924 as a reinforced concrete building with a riveted steel substructure on shallow concrete foundations. The original building consisted of a large theatre space behind a front three-storey concrete façade with concrete slabs at each level. The roof cladding is lightweight timber/corrugated iron supported by structural steel trusses that span to the encased steel columns in the walls. The main theatre has a large moulded plaster panel ceiling suspended from timber framing off the main steel roof trusses. The original structure has been modified over the years and underwent several strengthening/redevelopment stages between 2001 and 2010 (see Section 2 Building Description for more detail):

- The first stage (2001-2004) involved strengthening concrete shear walls, adding a Level 1 slab, and building two new basement cinemas.
- The second stage (2009-2010) included completing strengthening to the concrete shear walls, new steel roof bracing, and a lateral K-frame brace behind the stage.
- The third stage involved some minor fitout additions for the basement cinemas at the end of 2010. These alterations had no effect on the primary structure.

The building is currently configured as three cinemas with separate ground level retail, accessed from Kent Terrace and Majoribanks St. Its site coverage is approximately 1480 m^2.

The original 1924 structure provides the primary gravity system with the roof trusses and walls supported by the encased steel columns on shallow foundations. In both the longitudinal and transverse building directions, the original concrete perimeter walls with the encased steel frames provide the lateral-resisting system and are supplemented by the 2000s strengthening to these walls. The original building predates the 1931 Draft General Earthquake Building Bylaw and therefore is unlikely to have been specifically designed for seismic loadings. The building is heritage listed (list number 7500) as Historic Place Category 1 effective from 7th December 2001 (Heritage NZ). Any possible strengthening retrofits will need to consider the appearance and protection of heritage features.

There are neighbouring buildings on the southern and eastern boundaries of the Embassy. The building is separated from the neighbouring building by approximately 30 mm on the eastern side. The building on the southern boundary was demolished in early 2024.

The original structure is founded on shallow concrete pads and foundation beams below the original columns and walls. As part of the 2003 strengthening work, new screw piles were added to the strengthened shear wall foundations and the new basement was founded on screw piles and shallow foundation beams. The 2010 work involved adding Titan Ischebeck ground anchor micro piles and new foundation beams to the remaining shear walls.

A geotechnical desktop study has been completed by Tonkin and Taylor (T+T), see Appendix B for their full report. The subsoil classification is Class C and liquefaction is expected at this site. T+T have identified soil zones A, B, and C across the site. Zone A is rock in the south-east corner and the remaining soil (Zones B/C) are beach soils that are susceptible to liquefaction. The onset of liquefaction is expected to occur at 35 to 40% NBS, IL3 (Class C) ULS earthquake and is accompanied by cyclic displacement of the liquefied ground.

1.3 Assessed Seismic Rating

The results of the DSA indicate the building's earthquake rating to be 30*%NBS*(IL3) assessed in accordance with the guideline document The Seismic Assessment of Existing Buildings-Technical Guidelines for Engineering Assessments, dated July 2017. The earthquake rating assumes that Importance Level 3 (IL3), in accordance with the Joint Australian/ New Zealand Standard – Structural Design Actions Part 0, AS/NZS 1170.0:2002. IL3 was chosen as the building is designed to contain greater than 300 people in one area (crowds).

Therefore, this is a Grade D building following the NZSEE grading scheme. Grade D buildings represent a risk to occupants equivalent to 10-25 times that expected for a new building, indicating a high-risk exposure.

A building with an earthquake rating less than 34*%NBS* fulfils one of the requirements for the Territorial Authority to consider it to be an Earthquake-Prone Building (EPB) in terms of the Building Act 2004. A building rating less than 67*%NBS* is considered as an Earthquake Risk Building (ERB) by the New Zealand Society for Earthquake Engineering. Embassy Theatre (10 Kent Terrace) is therefore categorised as an Earthquake Risk Building and it also meets one of the criteria that could categorise it as an Earthquake Prone Building.

In accordance with the provisions of the Earthquake Prone Building requirements of the Building Act 2004, the determined earthquake rating requires the following actions:

- Notification of the findings of this report to the Territorial Authority as soon as practical.

The assessment identified the following structural weaknesses (SWs) in the building:

Table 1.1 - Structural Weaknesses

Description of Structural Weakness	Direction	Seismic Rating
ROOF		
Roof diaphragm – steel tension bracing	Longitudinal	60%NBS (IL3)
Roof diaphragm – steel compression members	Longitudinal	60%NBS (IL3)
Roof diaphragm – steel connections	Transverse	40%NBS (IL3)
Upper roof timber diaphragm and parapet (Grid 2 to 8)	Transverse	30%NBS (IL3)
WALLS/COLUMNS		
Concrete encased steel-frame walls along Grid A	Longitudinal	40%NBS (IL3)
Concrete encased steel-frame walls along Grid G	Longitudinal	67%NBS (IL3)
Concrete walls along Grid 2	Transverse	40%NBS (IL3)
Concrete walls along Grid 11	Transverse	40%NBS (IL3)

FOUNDATIONS		
Base shear takeout (non-liquified)	Transverse	40%NBS (IL3)
Base shear takeout (liquified)	Both	35%NBS (IL3)
Foundation bearing pressure (non-liquified)	Transverse	40%NBS (IL3)
Foundation bearing pressure (liquified)	Both	35%NBS (IL3)
FLOORS		
Western end L3 concrete diaphragm/wall drag tie to A/G shear walls	Longitudinal	30%NBS (IL3)
Western end L3 concrete diaphragm ties to Grid 2 wall	Transverse	50%NBS (IL3)
Western end L3 diaphragm in flexure	Longitudinal	45%NBS (IL3)
Western end L2 diaphragm in flexure	Longitudinal	75%NBS (IL3)
OTHER ELEMENTS		
K-frame on Grid 10	Transverse	80%NBS (IL3)
Façade parapets on Grid 1	Longitudinal	50%NBS (IL3)
Roof parapet on Grid 2	Longitudinal	45%NBS (IL3)
Main theatre plaster ceiling (hanging from roof trusses)	Longitudinal	30-40%NBS (IL3)

The Critical Structural Weaknesses (CSW) were found to be:

- Diaphragm connectivity to the main wall on at Grid A Level 3. This is limited to 30%NBS (IL3) by the lack of tie connection back to the concrete shear walls along Grid A. The main length of shear wall east of Grid 2 has limited capacity of the steel beam rivet connections and lack of bracing capacity of the intermediate wall due to large window openings.
- The combined capacity of the lightweight mono-pitched roof over the main theatre and the out-of-plane wall above truss level on Grid G at 30%NBS (IL3).
- The foundation capacity to resist horizontal base shear and vertical loads due to loss of support due to liquefaction of the subsoils at 35%NBS (IL3).
- The bracing capacity in the longitudinal direction of the plaster ceiling over the main theatre. The lateral load path is complex and lacking in substantial bracing capacity to resist the designated parts loading. If the bracing capacity is exceeded the ceiling may not automatically become unstable and present an automatic life safety hazard as it is hung from the roof trusses. For this reason, a range has been provided to indicate the likely risk and the uncertainty in this assessment at 30-40%NBS(IL3).

Pounding is not expected to be of structural significance between the Embassy and the eastern neighbouring building as this building is a two-storey timber structure which will not limit the displacements of the significantly heavier Embassy building.

The assessment identified no severe structural weaknesses (SSWs) in the building. A severe structural weakness is an element where its failure can potentially cause catastrophic collapse and for which the probable capacity may not be reliably assessed based on current knowledge.

1.4 Basis for the Assessment

This assessment has been based on the following information:

- Original blueprint drawings (dated 1923), Dunning Thornton “For Construction” structural drawings (dated 2003 and 2009)
- Original construction specification from 1923 and DTC specifications from 2003 and 2009 redevelopment/strengthening work
- DTC archived site reports from 2003
- Geotechnical analysis and report from T+T (August 2022), see Appendix B.
- Site investigations were conducted to understand the reinforcing in the existing structures walls and slabs. Concrete scanning of selected wall and slab zones was conducted by Concrete Structure Investigations Ltd and local breakouts were used to determine bars sizes. A summary report was provided on 29/02/2024 (Appendix C).
- Site inspections were conducted: general walkaround on 10/08/2022, reinforcing investigations on 29/04/2024 and main theatre roof/ceiling space on 20/05/2024.
- No previous ISA or DSA assessments were reviewed, however, DTC calculations for the 2001 to 2010 strengthening were available for reference.

1.5 Seismic Retrofit Options

We would be pleased to discuss with the client the likely scope of seismic improvement works once the client has had an opportunity to review this report and consider the level of seismic strength that is required. We note that further geotechnical site testing will likely be required to address geotechnical issues.

1.6 Disclaimer

This report has been prepared for the sole use of our client, Wellington City Council, for the particular brief and on the terms and conditions agreed with our client. It may not be used or relied on (in whole or in part) by anyone else, or for any other purpose or in any other context, without our prior written approval.

2 Building Description

2.1 General

The Embassy Theatre is located in Wellington on the corner of Kent Terrace and Majoribanks Street (see site plan, Figure 2.1). It was originally constructed in 1924 and underwent several stages of strengthening/redevelopment between 2001 and 2010. The following sections describe the site conditions and building structure.

Figure 2.1: Site location plan with Majoribanks St to the north and Kent Terrace to the West.

2.2 Site Conditions

Geotechnical information was provided by Tonkin and Taylor (T+T) (full report in Appendix B). T+T found the site subsoil had both Class B (rock) and Class C (shallow soil) with a moderate to high level of certainty. The exact location of the subsoil class boundary below the building is not known but it is likely that a larger area of the building is Subsoil Class C than Class B. T+T have provided a site plan with three soil zones (see Figure 2.2). Zone A is rock and not susceptible to liquefaction (purple, SE corner). However, Zone B (blue, NE corner) and Zone C (orange, western front half) are beach soils and both susceptible to liquefaction.

The soil layers contain beach and harbour deposits. The beach deposits are expected to experience liquefaction in an earthquake approximately 35% ULS IL3 subsoil Class C. In the harbour deposits, liquefaction is possible at approximately 40% ULS shaking (IL3, Class C). The expected liquefaction trigger level of 35 to 40% ULS shaking (IL3) limits the potential building %NBS as liquefication significantly reduces the soil's resistance to building base shear and foundation bearing capacity. The different soil classes and foundation types across the site may result in differential settlements and cyclic displacements.

The structural impacts of liquefaction and differential movement of liquefied/non-liquefied soil are considered in Section 5 - Results of the Detailed Seismic Assessment.

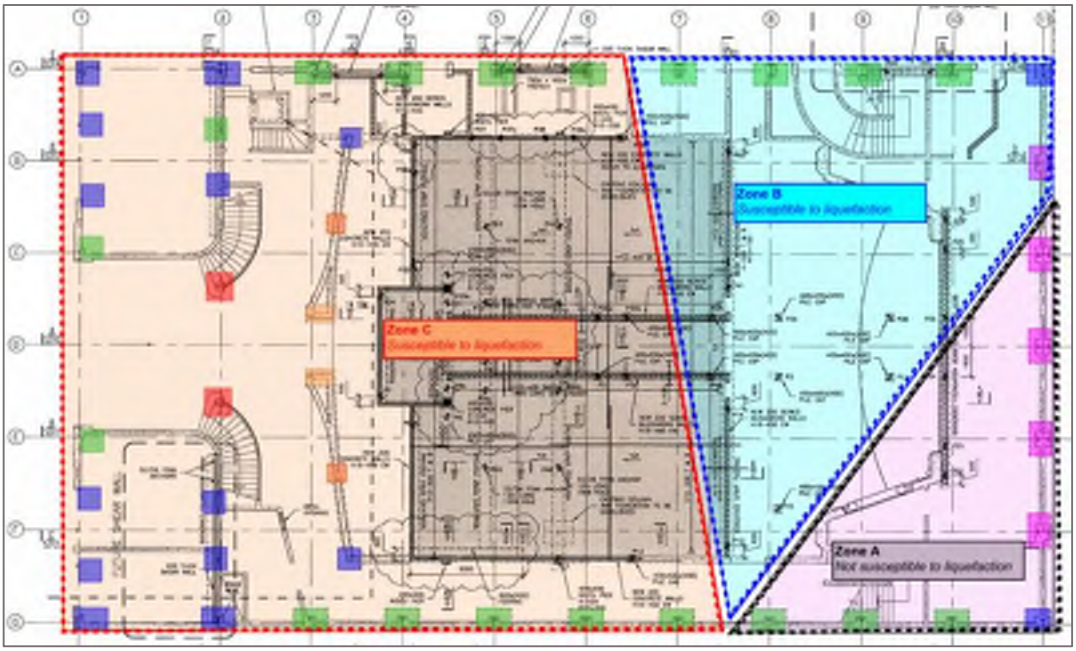


Figure 2.2: Soil zones from T+T with Zone A (purple, SE corner), Zone B (blue, NE corner) and Zone C (orange, front western half).

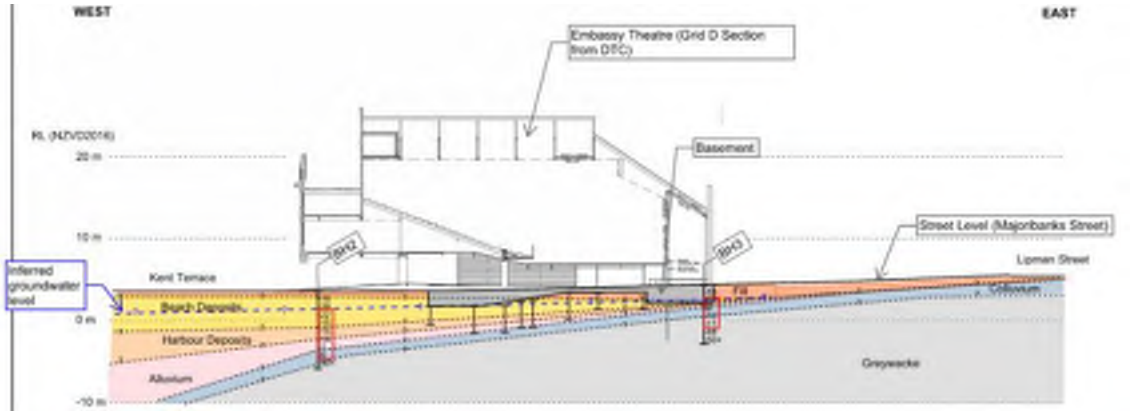


Figure 2.3: Soil profile through Grid D from T+T geotechnical report

2.3 Structural System

The original Embassy Theatre structure from 1924 is a combination of reinforced concrete slabs and walls supported by steel framing. The original walls are constructed from riveted steel beam/column frames encased by cast insitu reinforced concrete. The Grid A (north) wall has shopfront windows at street level between the encased steel columns, but the south elevation and eastern elevation walls are solid (Grids G and 11). The street front (western end) has a three-storey concrete façade with insitu concrete slabs at each level that are supported on concrete encased steel beams.

The roof structure is a semi-flexible diaphragm with a lightweight roof (timber purlins, sarking, and steel cladding) supported by jack studs onto the steel main roof trusses that are in turn retrofitted with cross-bracing to act as a roof diaphragm. The roof trusses span over the main theatre to the encased steel columns in the Grid A and G side walls. The original structure has

been modified over the years and underwent several strengthening stages between 2001 and 2010 (see below and Figure 2.4, Figure 2.5 and Figure 2.6 for reference):

- The first stage (2001-2004) involved strengthening concrete shear walls along Grid A and Grid 2, adding a Level 1 concrete slab, and building two new basement cinemas (block walls and concrete slabs on screw pile foundations). This first stage strengthened the building to 95%NBS of NZS4203:1992.
- The second stage (2009-2010) included completing the concrete shear wall strengthening, installing steel roof bracing to tie the building at the upper level, and bracing the eastern end with a K-frame and block wall. This stage was designed in accordance with NZS1170:2004 with the walls by equivalent static and the roof by parts. The strengthening was targeted to 80% NBS, IL3 seismic loads.
- The third stage involved some minor fitout additions for the basement cinemas at the end of 2010. This had no significant effect on the primary structure.

The original structure is founded on shallow concrete pads and foundation beams below the original columns and walls. As part of the 2003 strengthening work new screw piles were added to the strengthened shear wall foundations on Grid A. The new basement was also founded on screw piles and shallow foundation beams. The 2009 retrofit added Titan anchor micro piles and new foundation beams to the shear walls on Grid A and Grid 2.

Figure 2.4 shows the basic shape of the building and Figure 2.5 shows a section highlighting the different retrofit stages (blue = 2003, orange = 2009). Figure 2.6 shows the building grid references on the ground floor plan.

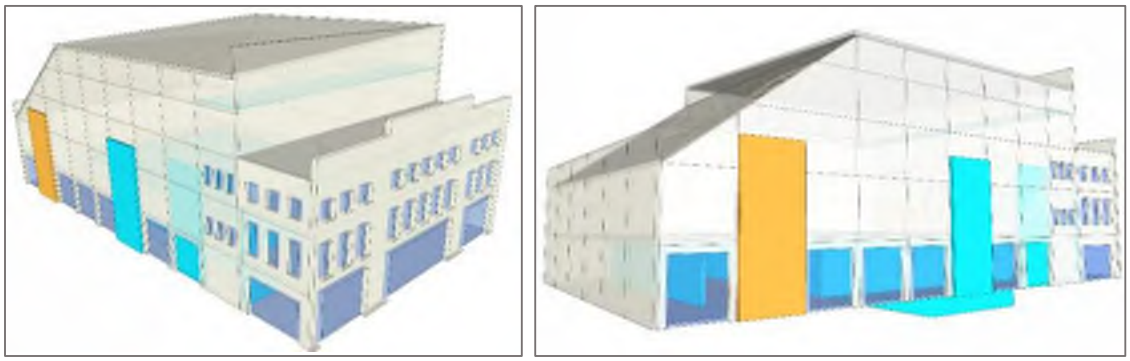


Figure 2.4: Embassy Theatre basic 3D shape from NW corner (right) and NE corner (left).

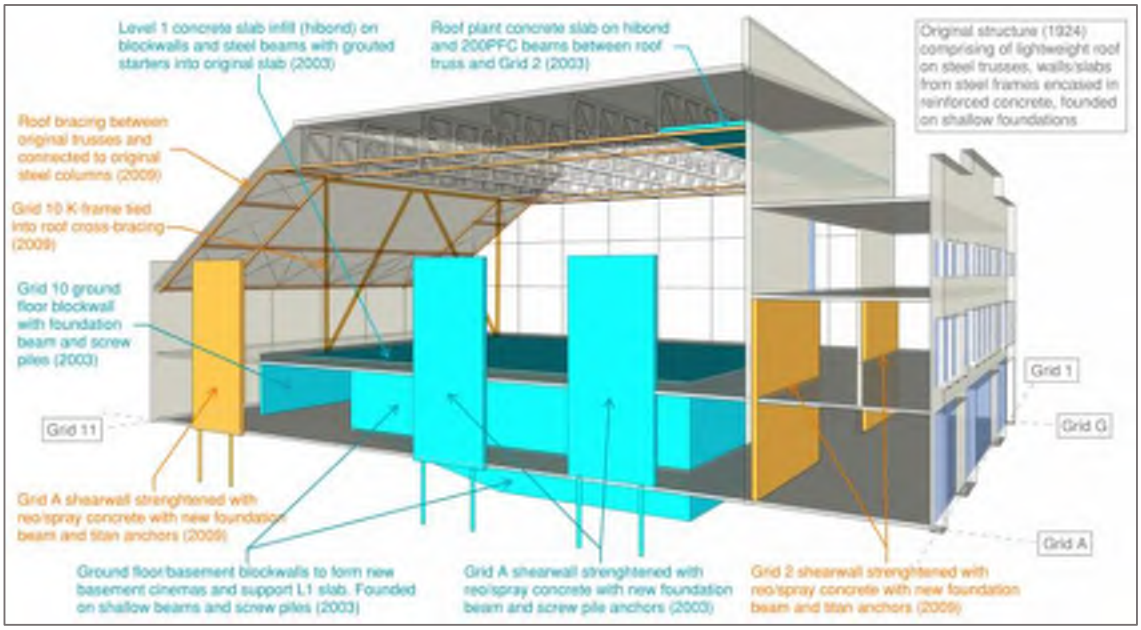


Figure 2.5: Section from North showing basic structure with different colours for different construction stages (grey = original 1924 building, blue = 2003 redevelopment, and orange = 2009 strengthening). The original Grid A (north) wall has been omitted for clarity in this view.

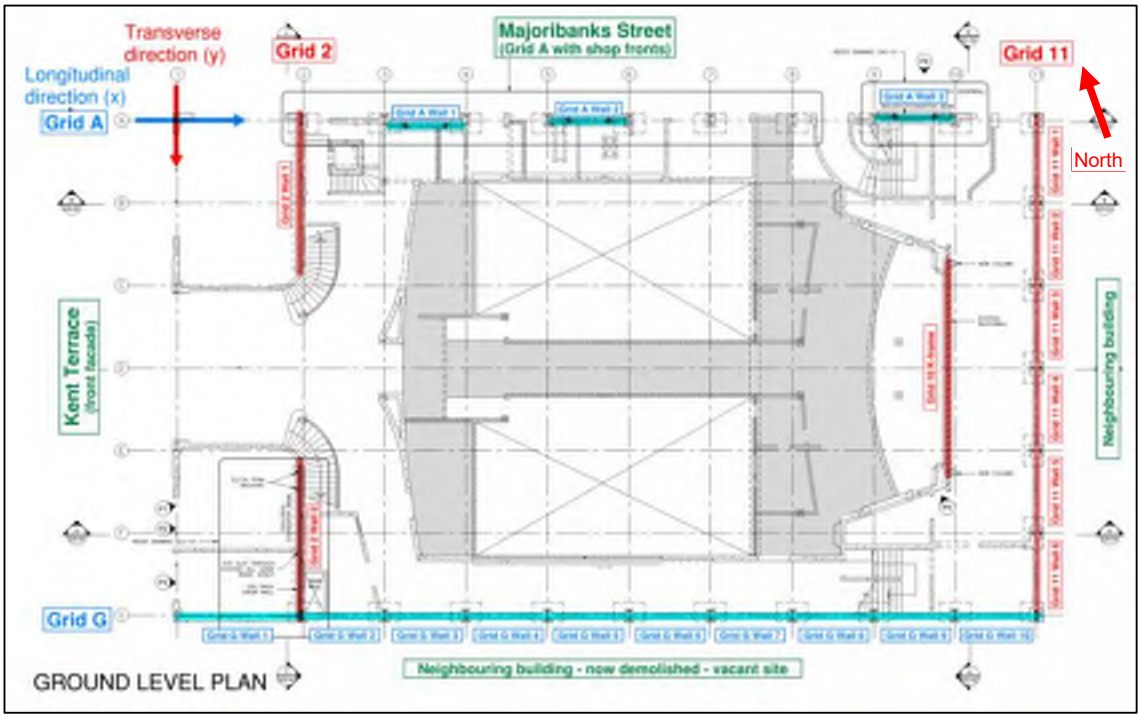


Figure 2.6: Ground level plan from 2009 structural drawings showing building grid numbering system and orientation with primary lateral resisting walls.

3 Detailed Seismic Assessment

3.1 Scope of Assessment

This report has been carried out for Wellington City Council with the intention of establishing the likely seismic strength of the Embassy Theatre (10 Kent Terrace), using the MBIE publication *Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments* (The Guidelines) and expressed in terms of *%NBS* (New Building Standard).

This assessment considers the seismic capacity of the primary structure (combination of original and strengthened elements) including walls, roof, floors, and foundations. The main theatre ceiling is the only secondary structural element considered in the assessment due to is size, location, and risk to occupants. No assessment is made to other secondary elements such as fitout or plant seismic restraints. This assessment was conducted using existing drawings, specifications, and site reports from WCC and DTC records, and local break out investigations. Site geotechnical analysis has been provided by Tonkin and Taylor Ltd (T+T).

3.2 Detailed Assessment Methodology

The assessment generally follows methods recommended in The Guidelines. The concrete elements have been assessed using the “Yellow Book,” Concrete Buildings Part C5 – Technical Proposal to Revise the Engineering Assessment Guidelines, November 2018. An assessment to the July 2017 “Red Book” guidelines only is not expected to change the outcome of the DSA as the structural steel components were the weaker elements.

This assessment uses the principles of the force-based equivalent static analysis because we do not consider there is significant ductility available in the original and newer structural components. The equivalent static loads have been applied to building components in the transverse and longitudinal directions to determine their %NBS and identify the critical structural weaknesses using the methodology in C2.3 of the Assessment Guidelines.

3.3 Assessment of Primary Lateral Load Resisting Elements

3.3.1 Seismic Weight, Shear Distribution and Torsion

The Embassy Theatre building was divided into areas (Areas 1 to 4) to reflect the contributing mass to the bracing systems. The areas and weights were adopted from the 2003 DTC calculations. Area 1 included the front (west end) with half the main roof, Area 2 included lower middle with the basement and L1 slab, Area 3 included the back walls with remaining half of the roof, and Area 4 included the back (east end) slabs and wall (see Figure 3.1 for reference to building Areas 1 to 4).

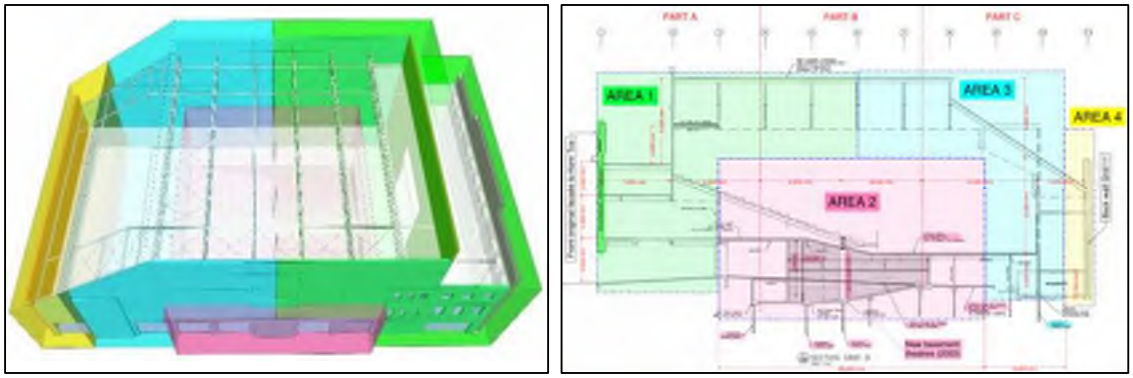


Figure 3.1: Areas 1 to 4 used to determine building weight and help distribution to different elements.

An equivalent static distribution has been assumed for both the longitudinal and transverse directions as per NZS1170.5. A ductility factor of μ = 1.25 has been used and a structural performance factor (Sp) of 1.0. Equivalent static analysis (ESA) was done on each area and these loads were used for the seismic demands on the different structural components.

Walls: The seismic loads from Areas 1+3+4 were distributed to the main structural walls on an assumed equal distribution in the transverse direction (50% each to Grid 2 and Grid 11 walls) and in the longitudinal direction (50% to Grid A and 50% to Grid G). Area 2 was excluded from the seismic loads on the perimeter walls. The load along each gridline was then distributed to the wall sections on that gridline. Given the roof is effectively a semi-flexible diaphragm accidental eccentricity was not considered in this analysis of the perimeter walls.

L1 Slab/Basement: Ground level and Level 1 have concrete slabs that provide rigid diaphragms to distribute lateral load. Area 2 contained the weight of the new L1 slab and basement which was considered as a separate structure inside the original main structure for the distribution of ESA loads. ESA seismic loads from Area 2 were assumed to be resisted by the basement walls and foundations. The L1 Hibond slab and basement walls also enable transfer of in-plane load from the side walls to the basement structure as a mechanism to take out base shear. The basement walls were checked for out-of-plane capacity as part of the passive base shear takeout mechanism and this limited the lateral displacement and takeout capacity of the basement structure.

3.3.2 Assessment of Primary Lateral Resisting Elements

The primary lateral resisting system is formed by the original concrete walls with encased steel frames on Grids A, G, 2 and 11. These are supplemented by the 2003/2009 reinforced sprayed concrete wall retrofits on Grids A and 2 (see Figure 3.2 on next page for overview of the structure). The different lateral system components were assessed:

- Original and retrofitted concrete walls were assessed using Concrete Buildings Part C5 – Technical Proposal to Revise the Engineering Assessment Guidelines, November 2018.
- Roof steel members and connections (both original and 2000s retrofits) were assessed using Part C6 of the guidelines. ESA derived roof loads were applied to a SpaceGass roof model to determine the demands on roof members and connections to the wall frames.
- Concrete encased steel members were also assessed using Part C6.5.4.

- Probable material properties were determined from Part C5.4 (concrete) and Part C6.4 (steel). Table 3.1: Probable materials properties from Part C guidelines. Table 3.1 shows the different properties used:

Table 3.1: Probable materials properties from Part C guidelines.

Concrete probable strengths:		**Steel probable strengths:**	
Original 1924 concrete	f'c = 30 MPa	Original 1924 reinforcing	fy = 284 MPa
2003/2009 concrete walls	f'c = 55 MPa	2003/2009 reinforcing	fy = 550 MPa
2003/2009 foundations	f'c = 40 MPa	1924 British steel members	fy = 231 MPa
Combined 1924/2000s	f'c = 43 MPa	2003/2009 steel members	fy = 350 MPa
		Rivets (pre-1960)	ft = 380 MPa

- According to the 1923 specification, original steel reinforcing is round bars with hooked ends at f_u = 28 tons/in^2 (f_u = 386 MPa). The assumed wall reinforcement of 3/8" rods at 12" each way (9.5mm dia at 305mm centres) was found in some wall sections during the concrete scanning, however, results were more varied. Investigations confirmed two-layer wall reinforcing with ½" (12.7mm bars) at centres varying between 230mm and 600mm in different wall sections. Connections between the concrete and steel frame were assumed to have no continuous reinforcing through the encased steel frame members. The lack of reinforcing continuity was confirmed by local breakout investigations on Grid A. The results of the reinforcing investigations have been used to revise some element capacities. Details for the original steelwork were more complete with member sizes and rivet connection details.

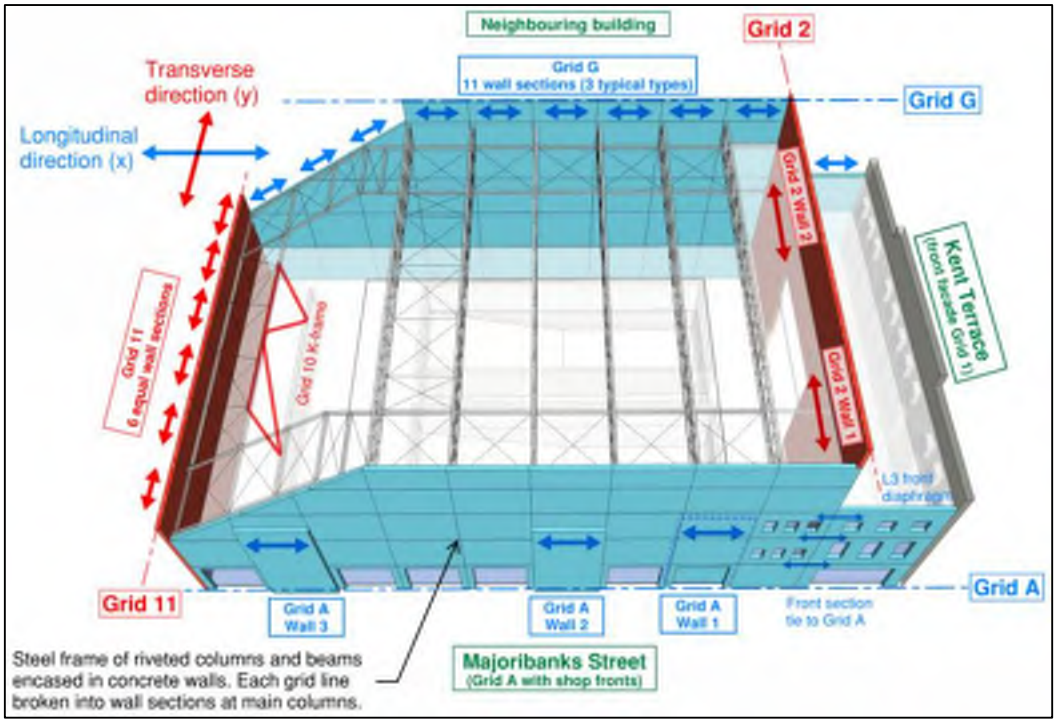


Figure 3.2: Concrete walls with steel frames provide the primary lateral resisting system in both the longitudinal and transverse directions.

To determine the seismic capacity of the combined wall/steel column mechanism on each grid line, the following component capacities were considered for each wall section. The capacity of the weakest element was reported as the %NBS of the grid line (also see Figure 3.3):

- Concrete wall capacities (flexure and shear) for typical wall sections on each gridline were assessed to Part C5 guidelines and compared to ESA demands.
- Flexural demands greater than the concrete wall section capacity are modelled to transfer load out of the wall into the steel columns at each end. This transfer is provided by the riveted connections between the beams and columns in the encased steel frame.
- The steel column axial capacity and splice connections capacities are compared to the load transferred into the column.
- At the base of the column, the baseplate hold-down bolts, and pad hold-down/bearing was compared to the column uplift and compression demands.
- The minimum capacity of these components is taken as the gridline %NBS.

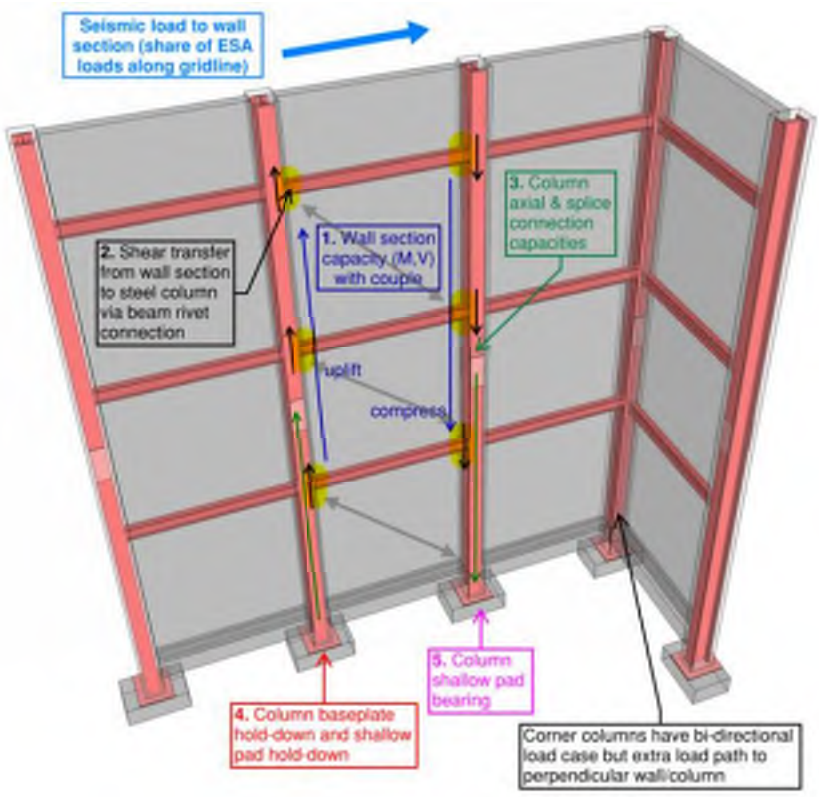


Figure 3.3: Combined concrete wall / steel column mechanism with load transfer to the foundations.

- For the Grid A wall/frame, the full wall elevation was modelled as a strut and tie mechanism rather than only typical wall sections for the gridline. Different foundation springs were also applied based on T+T's report. No struts were added at any window/shop openings. This model was to determine the load distribution effect of the encased steel frame and concrete wall sections in between the shear walls. The demands in the struts, ties, and spring reactions in the Grid A model were compared to the beam ties, columns, and foundations

capacities to determine the component and overall wall %NBS. The sensitivity of the model was then checked by removing ties with the largest loads.

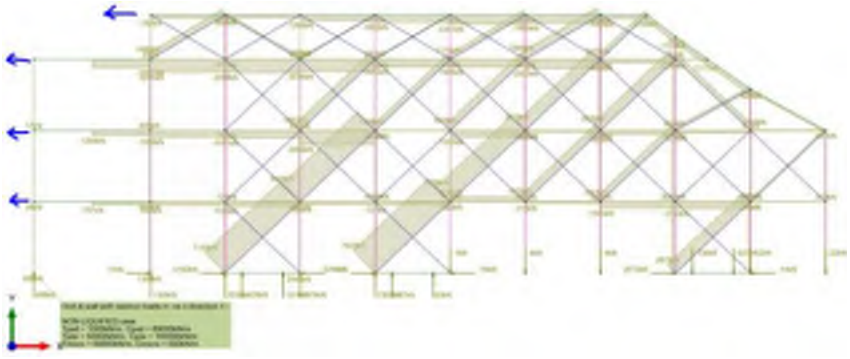


Figure 3.4: Strut and tie Grid A wall model under 50% longitudinal shear demands.

3.4 Assessment of Foundations

Tonkin+Taylor conducted a desktop geotechnical review of the site and building foundations. The full geotechnical report can be found in Appendix B.

- Capacities for shallow foundations, screw piles and Titan Ischebeck anchor micropiles capacities were assessed by T+T and used to determine the foundation %NBS. The onset of cyclic displacement which accompanies liquefaction as described in the T+T report has a marked influence on the performance of the screw piles and Titan micropile anchors.
- The base shear takeout was assessed using ESA derived loads and T+T foundation capacities for the non-liquefied and liquefied soil conditions.
- Non-liquefied bearing pressures beneath pad foundations were provided by T+T and compared to E+G+0.3Q loads for each pad. This included allowance for base shear takeout through inclined reactions with reduced vertical bearing capacity.
- Post-liquefaction performance, settlement, base shear takeout, and lateral spread have been considered qualitatively as a means to assess and rule out a geotechnical step change reduction in capacity.
    - For the theatre roof, a sensitivity check for vertical differential settlement and capacity design was made to verify the stability of the roof trusses under excessive settlement.
    - A qualitative assessment of the structures ability to deform and tolerate potential vertical splitting in the side walls due to the lateral spreading of the liquefiable soils and anchorage of the back corner to the rock. The geotechnical report highlighted the potential for cyclic displacements in Zone B and C in the order of 100mm to 200 mm (100mm probable) during shaking. This magnitude of lateral displacement is not believed to cause severe structural collapse and therefore has not had a geotechnical step-change factor of 2 applied.

3.5 Assessment of Out-of-Plane Loaded Parapets & Roof

- The parapet wall on Grid 1 acting out-of-plane has been assessed as part with ductility of 1.25.

- The parapet wall on Grid 2 spanning from the roof cross-bracing up to the top of the timber roof has been assessed as part with ductility of 2 with the encased steel columns and timber roof diaphragm connected behind.

- The lightweight timber-framed and sarked roof over the main theatre and the parapet wall on Grid G have been assessed as a combined capacity element for loading in the out-of-plane direction for the wall. The loading is based on parts with ductility of 2.

3.6 Assessment of Level 2 & 3 Concrete Diaphragms

The front section Level 2 and 3 floor diaphragms (west end between Grids 1 and 2) were assessed using pESA loads. The diaphragms consist of a 150mm thick insitu slab supported on encased steel beams spanning between Grid 1 and 2. Reinforcement scanning and local breakouts were undertaken to determine the reinforcement in the slab and the capacity compared to a section yield analysis of the diaphragm.

3.7 Assessment of the Main Theatre Ceiling

- The main theatre plaster ceiling bracing capacity was assessed using parts loading with a ductility of 2.

- The ceiling comprises a mix of flat plaster board elements supported on timber joists, intersected with a grid of set down, decorative, moulded plaster panels on light weight steel frame fixed back to timber joists. The joists span between trusses and are supported by timber hangers framed around the trusses. Framing is fixed with nails. The ceiling was assessed using Part C of the guidelines for diaphragm and timber/nail connection capacities. The figures below show a cross-section and photos of the ceiling/roof structure.

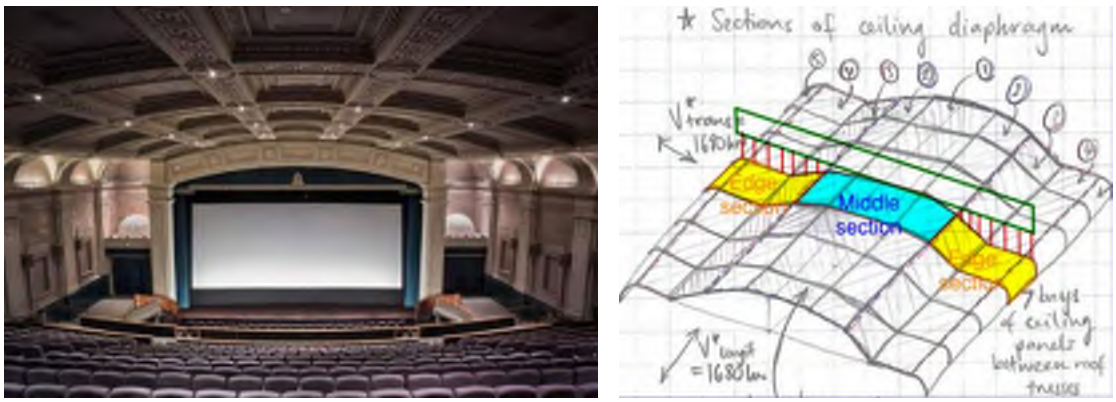


Figure 3.5: Photos of ceiling from inside main theatre with sketch of ceiling tributary to roof truss.

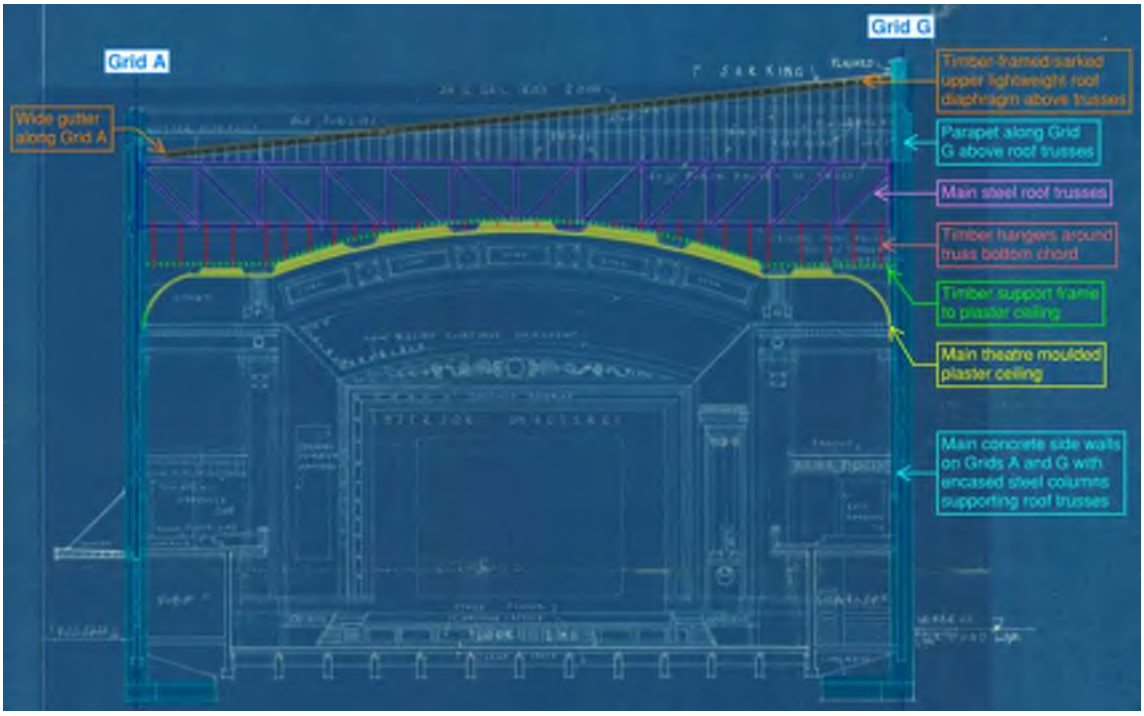


Figure 3.6: Cross-section from original drawings showing main theatre ceiling and lightweight timber roof supported on steel roof trusses, spanning to Grid A/G side walls.

Figure 3.7: Photos of ceiling framing and hangers to steel trusses from inside roof space.

3.8 Explanatory Statement

The following clarifications and limitations are noted:

- This report has been prepared for the sole use of Wellington City Council for the particular brief and on the terms and conditions stated in our engagement letter dated 22/03/2022 and agreed with our client. It may not be used or relied upon (in whole or in part) by anyone else, or for any other purpose or in any other

contexts, without our prior written agreement. This report may not be read or reproduced except in its entirety.

- The inspections of the building detailed in this report were undertaken to assist in the seismic assessment. This assessment does not specifically cover all building services, fire safety systems, building finishes, glazing or weather tightness.
- This assessment does not specifically include an assessment of the building's condition or any repairs and/or maintenance that may be required.
- Dunning Thornton Consultants Ltd. is not able to confirm that all possible damage, defects, conditions, or qualities have been identified. Therefore, the work done by Dunning Thornton Consultants Ltd. and our advice is on a reasonable endeavours basis.
- The assessment is based on the information available to us at the time of the assessment. Subsequent information may affect the results and conclusions of this assessment.
- We noted in our walkthrough inspection that seismic restraints have been provided to the roof level air handling plant. We have not reviewed the adequacy of these restraints.

4 Information Sources

4.1 Existing Documents

The following reference documents were used in undertaking this assessment:

Drawings:

- Original blueprint drawings dated 1923 from Wellington City Council records
- "For construction" structural drawings from 2003 (DTC job 3864)
- "For construction" structural drawings from 2009 (DTC job 3864-2009)

See Figure 4.1 for an example section from the original 1924 drawings, Figure 4.2 shows a 2003 section with new basement and Figure 4.3 shows the Grid A wall from the 2009 drawings.

Specifications:

- Original construction specification from 1923
- DTC specifications from 2003 and 2010 redevelopment/strengthening work

Other documents:

- DTC archived site reports from 2003
- Calculations from 2001 to 2010 redevelopment/strengthening work
- Geotechnical report from T+T (July 2022)
- Concrete scanning report from Concrete Structure Investigations (Feb 2024)

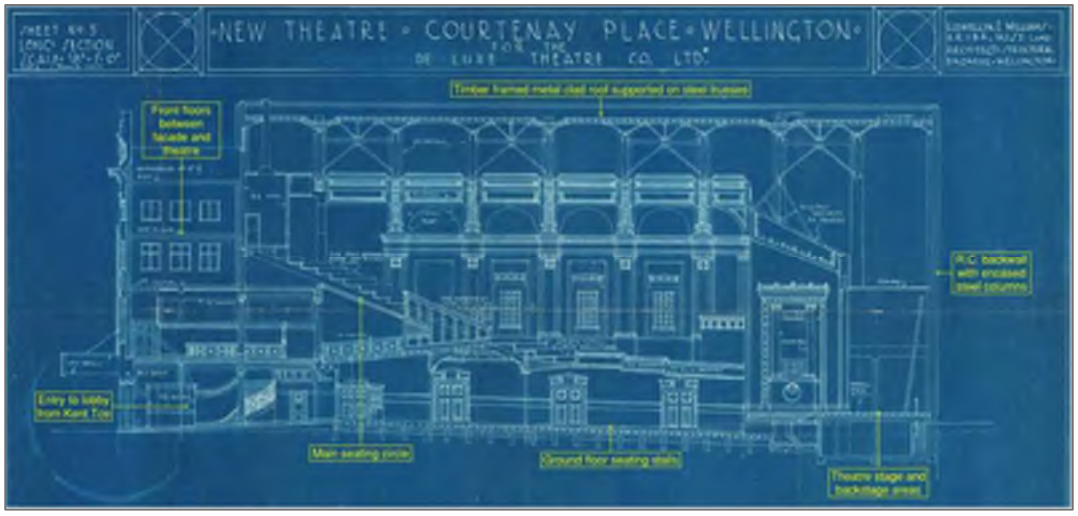


Figure 4.1: Longitudinal section from original blueprints (1923) showing building with single theatre with two seating tiers and front lobby with facade.

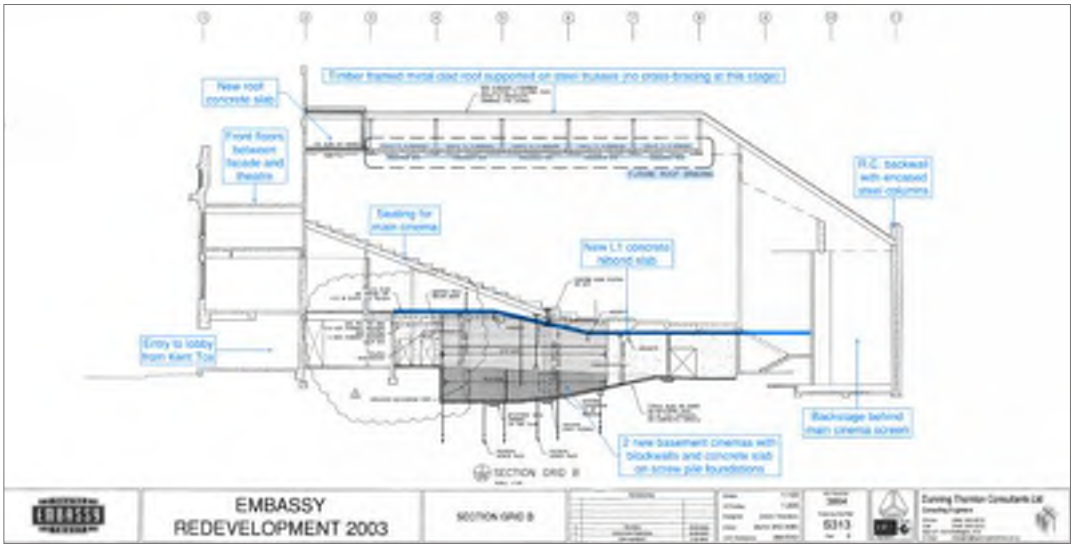


Figure 4.2: Longitudinal section from 2003 structural drawings showing conversion to three cinemas with new L1 slab to divide the original theatre into a main cinema with two basement cinemas below.

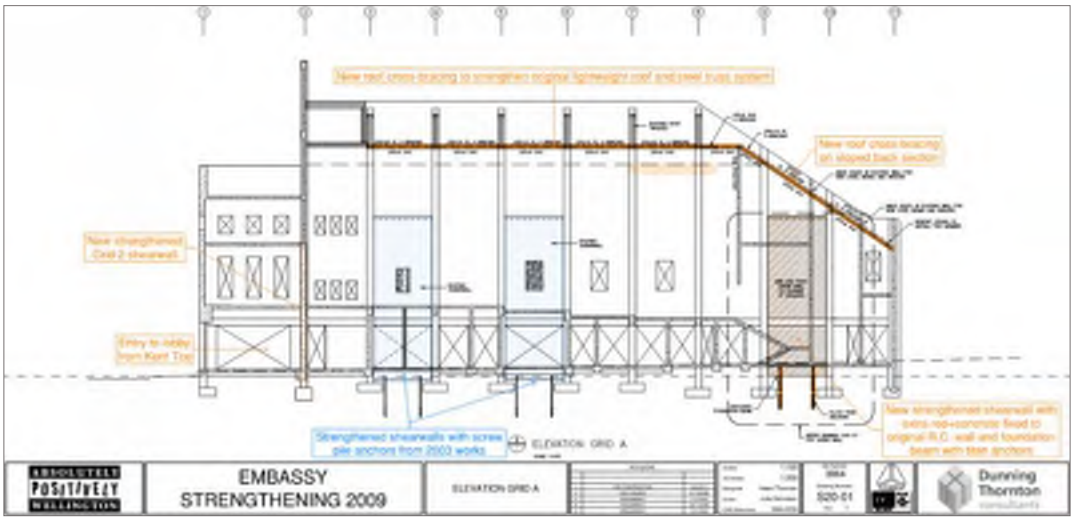


Figure 4.3: Longitudinal section from 2009 structural drawings showing strengthening to steel roof structure and adding to concrete shear walls on Grid A and Grid 2.

4.2 Building Investigation

This assessment was based on the existing documents described above. Limited on site investigations including reinforcement scanning and localised breakouts in locations advised by Dunning Thornton were also completed. Reinforcement scanning was undertaken by Concrete Structure Investigations Ltd. A summary reinforcement scan report was provided on 29/02/2024.

5 Results of the Detailed Seismic Assessment

5.1 Assessment of Primary Lateral Load Resisting Elements

The critical structural weaknesses (CSW) of the building are:

- The connection of the front Level 3 diaphragm between Grids 1 and 2 to the wall on Grid A. This is limited to **30%NBS(IL3)** primarily due to rivet shear failure at tie beam to column connection at Grid A2 within the Grid A wall. A value of 1 was used for the K_{dia} factor as per C5 of the guidelines as the connection failure lies within the immediate connecting wall element rather than the diaphragm itself. There are also multiple contributing load paths such as the wall section between Grid 1 and 2 that are capable of distributing some load between levels and there is some tie capacity from the stair landing and adjacent stairwell. The front section is tied into the longitudinal walls on Grid A and G at Level 3 but the weak riveted connection transfer continues along the Grid A wall. The same weakness is present at Grid G but the wall that immediately connects to the diaphragm is solid and without openings, so stability is not wholly reliant on these ties.

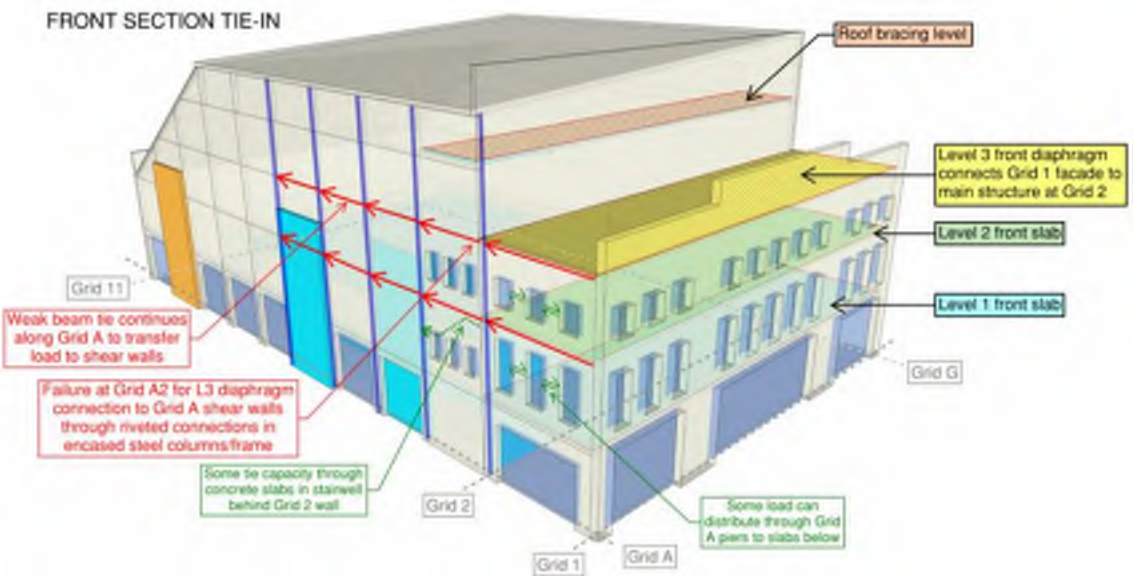


Figure 5.1: Diagram of the front section (slabs/facade) Level 3 tie-in to the Grid A shear wall.

- The upper roof structure above the main theatre trusses is constructed from purlins with sarking over spanning between unlined stud framed walls that are supported on the top chord of the roof trusses. The roof slopes up from an internal gutter along the inside edge of Grid A and attaches to the concrete wall on Grid G. The height of the Grid G wall is approximately 4.3m above the top of the trusses. Transversely, the sarked roof acting as a diaphragm and the Grid G wall (acting as a face-loaded cantilever) work together to resist the combined seismic loading. Loading has been assessed by parts with a ductility of 2. Both inwards and outwards directions have been considered for the transverse capacity as the wall has built in columns that form a 'T' section that has more capacity when loaded outwards from the building. The capacity of the sarked roof diaphragm is the same in either direction and has been taken from Table C9.3 of the guidelines. The inwards direction governs at 30%NBS(IL3) while the outwards direction is 40%NBS(IL3). We note that while the inwards direction has a lower NBS, the failure outcomes are likely different. Outwards loading will likely see the upper wall section losing stability and its attachment to the roof diaphragm, while inwards loading may result in the failed wall cantilever becoming buttressed by the roof diaphragm that may intern deflect the width of the gutter until the roof diaphragm abuts the Grid A wall. Post-failure stability is not guaranteed due to the excessive displacements required but it can be seen logically that a post failure redundant load path is possible.

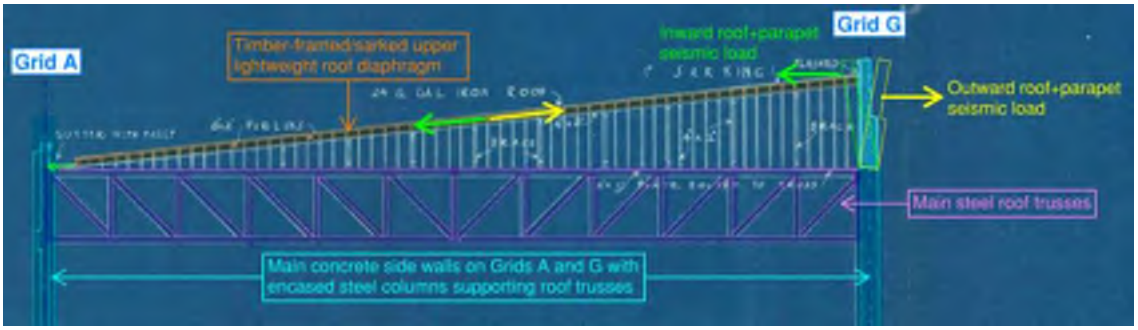


Figure 5.2: Upper roof diaphragm and parapet inwards and outwards loading cases.

- The ground conditions and loss of strength to the potentially liquefiable soil means the base shear resistance and bearing capacity of the structure is significantly reduced by liquefaction at 35% to 40%NBS(IL3) earthquake shaking. Large displacements are expected below the shallow pads once liquefaction is triggered. The worst case is expected at the Grid A pads on soil Zone B (lowest soil bearing capacities). Liquefaction results in a geotechnical step change in the capacity of the soils. This is not considered to lead to a structural step change in behaviour as explained in Section 5.5 so the structure will be limited by the liquefaction trigger value of **35%NBS(IL3)**. In the non-liquefied state, the lowest rated foundation component is the shallow foundation pads at corners G1/G2 in bearing and G11 for hold-down at 40%NBS(IL3). This result is due to seismic demands at the ends of shear walls.

- The plaster ceiling over the main theatre is in a largely original state and covers a large area from Grids A/2 to G/8. It is poorly braced, particularly in the longitudinal direction. Transversely, it is somewhat confined by the Grid A and G shear walls. The ceiling is made from plaster panels (flat panels and moulded fibre reinforced pieces) fixed and suspended from timber framing, spanning between roof trusses. The ceiling is hung from a multitude of rudimentary nailed timber frames slung over the bottom chord of the roof trusses with the occasional cross brace. See the photos of the ceiling framing. A capacity of **30-40%NBS(IL3)** was derived based on parts loads in the longitudinal direction at ductility of 2. The lateral load path is complex and lacking substantial capacity, it is ultimately limited by the capacity of the nail connections in the hanging frames. However, when the bracing capacity is exceeded, and because is it hung from multiple locations with moderately ductile nail connections, this does not mean it has automatically lost stability and become a life safety hazard. For this reason, we have given a range that indicates the likely relative risk but also the uncertainty in this assessment.

- The retrofit steel bracing roof diaphragm at the bottom of the main roof trusses transfers roof loads to the concrete shear walls. The capacities of the connections along Grid 8+ at the roof crank are exceeded by seismic demands in the transverse direction at 40%NBS(IL3). The steel connection to the original concrete wall is limited by concrete breakout. Additionally the SHS stub along Grid 11 between the original columns and retrofitted cross-bracing fails under moment from the roof seismic loads at 50%NBS. Connections at Grids A3 and G3 are exceeded in the roof longitudinal direction at 65%NBS(IL3). The tension and compression capacity of the roof bracing members can achieve 60%NBS(IL3).

5.2 Structural Weaknesses

Description of Structural Weakness	Direction	Seismic Rating	Comment
ROOF			
Roof diaphragm – steel tension bracing	Longitudinal	60%NBS (IL3)	Failure of tension cross-bracing under axial load
Roof diaphragm – steel compression members	Longitudinal	60%NBS (IL3)	Failure of SHS roof chords in compression
Roof diaphragm – steel connections	Transverse	40%NBS (IL3)	Failure of roof connections to original concrete wall at Grid 8+
Roof diaphragm – steel connections	Longitudinal	65%NBS (IL3)	Failure of roof bracing to column connections at A3/G3
Upper roof timber diaphragm and parapet (Grids A2 to G8)	Transverse	30%NBS (IL3)	Failure of diaphragm at Grid 2 / 8 and concrete wall along Grid G

WALLS/COLUMNS			
Concrete shear walls and steel-frame along Grid A	Longitudinal	40%NBS (IL3)	Encased beam tie between shear walls A2 and A5
Concrete shear walls and steel-frame along Grid G	Longitudinal	67%NBS (IL3)	Wall end/column uplift on Grid G10
Concrete walls/steel columns along Grid 2	Transverse	40%NBS (IL3)	End uplift on Grid 2 shear walls with eccentric anchors
Concrete walls/steel columns along Grid 11	Transverse	40%NBS (IL3)	Wall end/column uplift on Grid A11 and G11 (back wall corners)
FOUNDATIONS			
Base shear takeout (non-liquified)	Transverse	40%NBS (IL3)	Shear takeout from passive + sliding resistance of shallow pads and basement foundations
Base shear takeout (liquified)	Both	35%NBS (IL3)	Shear takeout from passive only resistance of pads and basement
Foundation bearing pressure (non-liquified)	Transverse	40%NBS (IL3)	Bearing on shallow pads at Grid A2/G2 shear wall end/corner
Foundation bearing pressure (liquified)	Both	35%NBS (IL3)	Bearing on shallow pads
FLOORS			
Western end L3 concrete diaphragm/wall drag tie to shear walls on Grids A and G	Longitudinal	30%NBS (IL3)	Front façade/slab loads failing the riveted connections at A2 and continuing along to A6
Western end L3 concrete diaphragm tie to Grid 2 shear wall	Transverse	50%NBS (IL3)	Front façade/slabs torsion causing tensile rivet failure at A2
Western end L3 floor diaphragm in flexure	Longitudinal	45%NBS (IL3)	Yielding reinforcing in the L3 concrete slab, spanning A to G
Western end L2 floor diaphragm in flexure	Longitudinal	75%NBS (IL3)	Yielding reinforcing in the L2 concrete slab, spanning A to G
SECONDARY ELEMENTS			
K-frame on Grid 10	Transverse	80%NBS (IL3)	Demands from roof level to steel K-frame members
Façade parapets on Grid 1	Longitudinal	50%NBS (IL3)	Concrete cantilever section and span between encased columns
Roof parapet on Grid 2	Longitudinal	45%NBS (IL3)	Concrete cantilever above roof bracing, spanning between encased columns on Grid 2
Main theatre plaster ceiling (suspended from roof trusses)	Longitudinal	30-40%NBS (IL3) range	Capacity of nail connections in timber hangers supporting ceiling panels up to roof trusses

Table 5.1 – Results of the Detailed Seismic Assessment.

5.3 Assessment Assumptions

The following assumptions have been used in deriving the seismic equivalent static load (ESA) demands for the building.

Parameter	Assumption	Comment
Date of Building Design	1923 / 2003 / 2009	As per original and retrofit drawings
Soil Class	C	From geotechnical report (Appendix B)
Building Importance Level	IL3	As per NZS1170.0:2004
Zone Factor, Z	0.40	Wellington
Longitudinal /Transverse Directions:		
Fundamental Period, T_1	0.4s	
Ductility of Structure, μ	1.25	Limited ductility of concrete structure
Structural Performance Factor, S_p	1.0	As per NZS1170.5:2004

Near Fault Factor, N(T, D)	1.0	As per NZS1170.5:2004
Seismic Coefficient, $C_d(T_1)$	1.07g	As per NZS1170.5:2004

Table 5.2 – Assessment parameters used to derive seismic loads.

5.4 Geotechnical Considerations

The geotechnical desktop study was conducted by Tonkin and Taylor (T+T). Liquefaction is expected at this site and T+T have identified three soil zones across the site. Zone A has rock (back SE corner), Zone B is softer soil that is susceptible to liquefaction (NE corner), and Zone C is soft and is also susceptible to liquefaction (front western half of building). The onset of liquefaction is expected to be a 35 to 40%NBS (IL3) ULS earthquake. In this case, all building base shear must be removed through passive soil pressure only as any sliding friction resistance has been lost. See T+T's geotechnical report for further information (Appendix B).

There is also potential for the building to be subject to significant differential lateral ground displacements along the interface between Zone A (rock) and Zone B (potential to liquify) as the front moves away from the rock-anchored SE corner by up to 200mm. T+T estimate that the lateral ground movement would induce 8 to 12 MN on the structure. This force would need to be resisted by the structural ties along the Zone A/Zone B interface, however, the structure is unlikely to have this capacity. The differential ground displacements predicted at 35%NBS(IL3) could cause significant ground damage to the Grid G and 11 walls, however, the potential splitting at ground level is not likely to lead to loss of gravity support to the roof trusses and to catastrophic collapse of the structure.

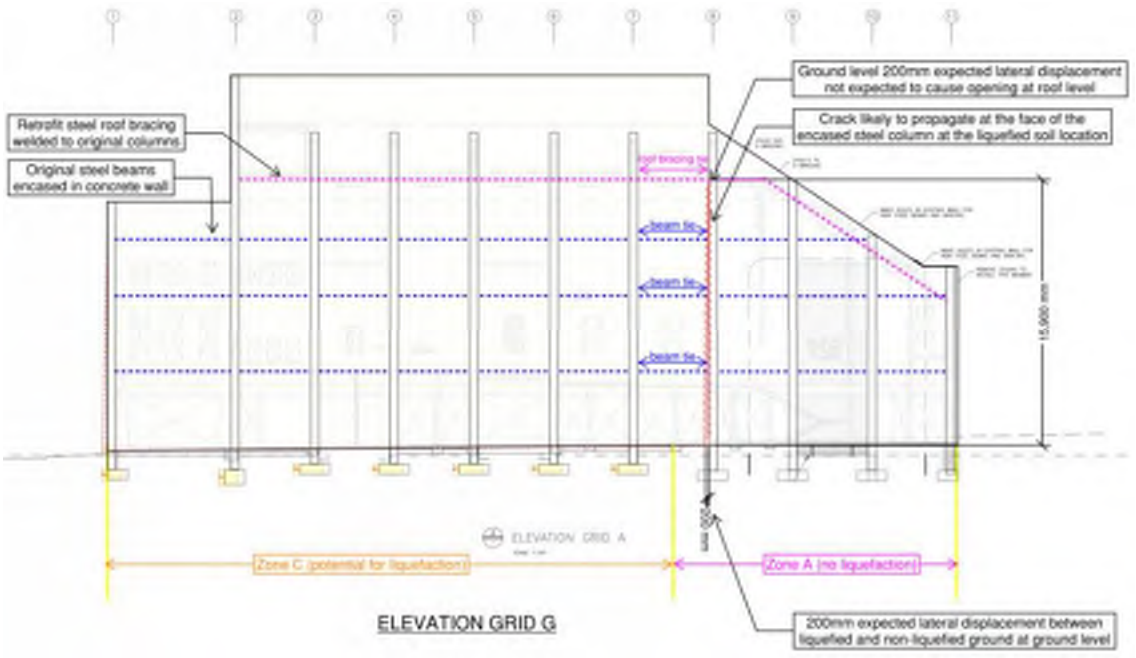


Figure 5.3: Grid G wall elevation with 200mm lateral displacement on Zone C foundation pads and no displacement in Zone A, leading to potential split along Grid 8G column.

5.5 Severe Structural Weaknesses

A severe structural weakness (SSW) is an element where its failure can potentially cause catastrophic collapse and for which the probable capacity may not be reliably assessed based on current knowledge. Structural systems likely to suffer a structural step change are required to have reduction factor 0.5 on the calculated NBS score.

There is significant likelihood that in a large seismic event, the building will move on its foundations due to lateral ground movement and liquefaction potential. However, the associated likelihood of this ground movement resulting in catastrophic collapse is considered unlikely. A structural step change in performance following the geotechnical step change in soil strength due to liquefaction has not been considered appropriate for the following reasons:

- The structural configuration and materials consist of a concrete encased steel frame and insitu concrete floors. These elements can have considerable inherent deformation capacity and robustness, particularly when compared with structures such as poorly tied unreinforced masonry structures, lightly reinforced concrete columns, or structures with Hollowcore floors.
- The long walls will also reduce differential settlement effects from localised pockets of liquefaction.
- The roof trusses over the main theatre have considerable capacity to ride out differential settlement of their foundations and the connections to the columns are capacity protected. More than 900mm differential settlement across the span is tolerable just to reach the yield capacity of the supporting column.
- Loss of base shear resistance is not considered to be catastrophic in itself, particularly as this leads to a loss of connection to the ground and therefore a potential reduction in shaking transferred to the structure.
- The structure is generally well-tied with Ground floor and Level 1 working together to resist differential lateral spreading.

The method for assessing the step change associated with geotechnical performance will be changing with the next revision of the assessment guidelines. We have been able to review these draft proposals. From our review, we do not expect the proposed changes when released to change our assessment of the step change performance.

5.6 Localised Construction Issues

Due to the main structure being built in 1924, there are limited original construction and connection details. This led to conservative assumptions about concrete work detailing and concrete connections to steelwork. The original structural steelwork drawings had more details to reference. Connections between the concrete and steel frame has been assumed to have no continuous reinforcing through the encased steel frame members. On site investigation has been conducted to understand the original reinforcement detailing. This included concrete scanning with some limited concrete removal in isolated areas to calibrate the scanning. This confirmed the lack of reinforcing continuity across the steel beams/columns. It also showed wall reinforcement as two-layers but with wider, more varied spacings. The overall capacities did not significantly change from the assumptions made in the original calculations.

6 Commentary of Seismic Risks

From our assessment, the Embassy Theatre (10 Kent Terrace) is likely to achieve **30*%NBS*(IL3).** Therefore, it is a **Grade D** (high risk) building. Following the grading scheme of the NZSEE, the relative risk to a New Building would be 10 to 25 times.

Building Grade	Percentage of New Building Strength (*%NBS*)	Approx. Risk Relative to a New Building	Life-safety Risk Description
A+	>100	<1	low risk
A	80 to 100	1 to 2 times	low risk
B	67 to 79	2 to 5 times	low or medium risk
C	34 to 66	5 to 10 times	medium risk
D	20 to 33	10 to 25 times	high risk
E	<20	more than 25 times	very high risk

Table 6.1 - NZSEE Grading Scheme

7 Seismic Retrofit Options

Improving the connections of the Level 2 and Level 3 diaphragms back to the main shear walls on Grids A is recommended. This may take the form of long tie beams fixed to the face of the shear walls and diaphragm edges.

The upper roof structure above the main theatre has a sarked timber diaphragm supported on stud frames that sit on top of the main steel roof trusses. The load path from the upper roof diaphragm to the steel trusses could be improved by bracing these stud frames in both orthogonal directions to the trusses below and by enhancing the connection of the wall to roof at Grid G.

Strengthening to the main theatre ceiling from inside the main roof space is likely to be possible. Adding bracing to the hangers and improving hanger connections is a priority. Further refixing of the plaster elements to the timber framing may also be required for enhanced diaphragm performance.

Whilst any improvements to the structure should target the highest NBS rating that is reasonably practicable, the building's improved NBS rating will ultimately be limited by the response of the ground. Improving the foundation performance is likely to be a major undertaking. This is likely to be very disruptive and costly depending on the target NBS of the upgrade. The works may involve installing new piles/micro pile foundations, significant foundations beams, new floor slabs, ground improvement, and will also likely involve complex temporary works for the structure above. The attached geotechnical report by Tonkin+Taylor has more detail on the potential ground improvements.

Appendix A Existing Drawings

The following existing drawings cover three sets, each for a different design/retrofit stage that has been completed on this building:

- Original blueprint drawings dated 1923 from Wellington City Council records
- “For construction” structural drawings from 2003 redevelopment (DTC 3864)
- “For construction” structural drawings from 2009 under DTC job 3864-2009

CONSENT NO. [A168] **SITE ADDRESS**

A 1611

EMBASSY THEATRE [00055:16:A1611]
[Part 1/2]

(DE LUXE)

KENT TCE.

De Luxe Theatre Co.
W. Williams – builder

ISSUE DATE

29.6.1924

Plans in
QA 89.

BUILDING APPLICATION FORM.

WELLINGTON,

Date, JUN 28 1923 19

To the City Engineer,
Wellington,

SIR,

I hereby apply for permission to erect a Picture Theatre in Courtenay Place Street, Section 24.25.26.27 part of Town Acre 308 for "De Luxe" Theatre Co of Wellington according to Plans and Specifications deposited herewith at the estimated cost of £ 41,000.—

Yours faithfully,

W Williamson

See L6 enclosed

Postal Address 175 Montreal St
Christchurch

W. A. T. LTD. C7004

C O N T R A C T

- For -

THE ERECTION and COMPLETION of a PICTURE THEATRE, COURTENAY PLACE, WELLINGTON, FOR THE DE LUXE THEATRE COMPANY LIMITED.

LLEWELLYN E. WILLIAMS.
A.R.I.B.A. M.I.S.E. LONDON
11 Grey Street,
W E L L I N G T O N.

March 1923.

THIS CONTRACT shall be carried out in accordance with the GENERAL CONDITIONS OF CONTRACT agreed to on the 17th day of February 1910 by the NEW ZEALAND INSTITUTE OF ARCHITECTS and the NEW ZEALAND FEDERATED BUILDERS ASSOCIATION.

MEMORANDA.

TENDERS CLOSE at Noon on MONDAY 19th March, 1923.
AMOUNT OF DEPOSIT: £250
DATE OF COMPLETION: State in Tender-this is important.
PENALTY FOR NON-COMPLETION - £5 a day
PERIOD OF MAINTENANCE: Ninety (90) days.

SPECIAL NOTE TO THE CONTRACTOR

to be attached to the Specification and considered part of Contract.

1. The Bills of Quantities shall be part of the Contract and shall be read in conjunction with the Plans and Specifications.

2. The p.c. amounts for the works in addition to the Contract shall include any profit the Contractor may think he is entitled to for the trouble he may be put to in attending on the sub-contractors as specified.

W. WILLIAMSON

BUILDER AND
CONTRACTOR

P.O. BOX 116

173 175 Montreal Street,
Christchurch, N. Z.

Telephones:
OFFICE AND FACTORY 3247
WORKS - - 2780
RESIDENCE - 1727

Telegrams: "Chips," Christchurch

28th June, 1923.

The Building Superintendant,
Wellington City Council,
WELLINGTON.

DE LUXE THEATRE.

Dear Sir,

In reply to your letter of the 27th inst. I enclose herewith building application form duly completed together with fee for £6 as required. I also enclose the sum of £33.15. 0 for hoarding deposit on 270 ft. of frontage at 2/6 per foot.

I have instructed my foreman, Mr. B. Keats, to call on you on Saturday morning when I trust you will be able to hand over to him these permits.

Yours faithfully,
For W. Williamson,
H S Cook

WELLINGTON CITY COUNCIL
CITY ENGINEER'S OFFICE
PERMIT MAY BE ISSUED
Drainage 33.15-0 by mtr
Water
Date 29.6.23
BUILDING SUPERINTENDENT

Stamps 1/- to cover exchange
H.S.C.

A 1611

SPECIFICATION of WORK required to be done and materials to be used in the erection and completion of a Theatre, Courtenay Place, Wellington, for The De Luxe Theatre Company, under the supervision and to the satisfaction of

LLEWELLYN M. WILLIAMS,
A.R.I.B.A., M.I.S.E.(Lond)
11 Grey Street,
WELLINGTON.

Phone 2016.
Box 1316.

THE SITE is at the corner of Courtenay Place and Majoribanks Street, and is approximately 99'0" by 170'0" and although the levels shewn are believed to be correct they are only given as approximate. The contractor shall carry down the piers and other supports to a solid base and to the satisfaction of the authorities and should the depths vary to those shewn on the drawings, then the contractor will be credited or debited as the case may be by an amount equal to the variation at the price as scheduled in the Bills of Quantities. Trial holes have been sunk and the Contractor must visit the site to view the nature of the ground.

The Contractor shall visit the site and make his own arrangements for the carrying out of the work. He shall give all notices, pay all fees, and carry out the work in strict accordance with the requirements of all By-laws governing the work. The Contractor shall be responsible for all damage to any person place or thing during the course of this contract, and must recompense, make good, or restore

as the case may demand to the full satisfaction of all parties concerned. The Contractor must particularly note that no extra or deduction will be recognised unless authorised in writing by the Architect and before any such variation is made a price must be fixed where possible or else reference made to the Bills of Quantity.

BRACING. The Contractor's attention is drawn to the fact that there are few cross walls, and he must provide temporary bracing to all walls until they are permanently supported or permanently set. This will be necessary owing to the possible effect of wind or earthquake.

OLD MATERIAL.

The old buildings now on the site are to be sold and the Contractor shall take over the site and remove any rubbish that may be left after the buildings have been cleared away. He shall grub up all old drains etc. he may come across in the course of the contract and prepare the site for the proper erection of the buildings as shewn on the drawings.

SETTING OUT.

The Contractor shall be responsible for the correct setting out of the work, and he shall rectify any errors he may make at his own expense.

THE BUILDINGS must be cleaned out from time to time during the progress of the work and thoroughly at completion when all floors shall be thoroughly scrubbed and all windows cleaned and all locks left in proper working order.

SEPARATE CONTRACTS.

The employers reserve to themselves the right to carry on during the period the works are in the Contractor's

hands any work appertaining to the business for which the buildings are being erected such as special fittings or electrical work or any extra work or sub-contractors work they may decide to do, and the Contractor shall give free access to the works of any persons necessary to the carrying out of this work. They shall also have the right to utilise any scaffolding etc. where by so doing they are not, in the opinion of the Architect, interfering with the progress of the works.

CLERK OF WORK'S OFFICE. The Contractor shall provide a suitable office for the Clerk of Works in a position to be determined upon by the Architect. The office shall consist of two rooms, the outer being 12'0" x 7'6" wide, the inner 4'6" x 7'6" wide. The outer office shall have a desk the full length of the office, and shall be lighted by a large window opening sufficiently for ventilation. The inner office shall have a smaller window, and shall be separated by a strong door with a yale lock. The outer door shall be strong and furnished with suitable lock and furniture. The walls shall be iron on 4" x 2" studding properly framed and roof shall be iron on rafters at 2'0" centres. The floor shall be 4" x 1" S. & G. flooring on 5" x 2" joists at 18" centres. The office is for the Clerk of Works exclusively, but when his duties cease the Contractor shall remove the building from the site. The roof may be a lean-to, but the ceiling height shall not be less than 7'0" at the lower side.

HOARDING.

The Employers reserve the right of advertising or letting for advertising the hoarding erected round the building, and the contractor shall not destroy or deface any advertisements

placed thereon. The reservation shall be for the term of the contract.

EXCAVATOR.

The Contractor shall excavate where required for all footings to all walls foundations and piers and wherever else required for the proper carrying out of the work. Where excavating near roadways and pining properties the Contractor shall perform all necessary work to prevent the earth from caving in, and he shall be held responsible for all damage that may occur, during the course of this work. Should the adjoining properties require underpinning the contractor must perform this work at his own expense building up in brick or concrete if necessary well wedged up with good hard bricks set in cement or finished hard up in good cement.

RAMMING.

The contractor shall consolidate the bottoms by ramming and shall fill in to foundations and remove all surplus spoil from the site.

PUMPING.

It is possible that the holes for piers etc. may have to be pumped during the excavating and filling. The contractor shall allow for all this work in his tender as it is absolutely necessary that the holes be kept free of water during the progress of his contract.

WATERPROOFING.

All concrete work round the entire building and including cross walls and piles shall be waterproofed with some concrete waterproofing compound in such proportions as will be effective. The portion above referred to

shall be that contained in a line drawn at a height of 18" above floor level and one drawn 18" below floor level. This clause is important. A guarantee that the work is watertight shall be obtained by the contractor from the supplier of the waterproofing compound and handed to the employers.

PILES. Where concrete piles are necessary the contractor shall drive same at a price to be stated by him at per foot when tendering. These shall be cylinder piles and the price given shall include all hoisting, tackle driving etc. and shall be for the pile complete and ready to receive the superstructure.

DRAINLAYER.

MATERIALS.

The Contractor shall supply and fix all materials necessary for the proper carrying out of the work according to the drawings and also to the complete satisfaction of the Council's Inspectors and any by-law that may govern the work. Only experienced and licensed tradesmen shall be employed. The Contractor shall excavate where required for drainers work to the depths and grades required and shall construct all manholes, inspection chambers etc. that may be required. Any concrete necessary for the work shall be as described under 'Concretor'.

THE PIPES shall be hard salt glazed socketted earthenware pipes set with cement compo and covered in concrete where required by the regulations. Each pipe shall be carefully wiped out and left free before another is laid.

ALL DRAINAGE required to be done in places where salt glazed

earthenware pipes are not practicable shall be through heavy cast-iron pipes with properly caulked joints and slung as directed. These pipes where shewing shall be neatly boxed in as described under "Carpenter" and shall conform in the boxing to adjacent work.

CONCRETOR.

The whole of the materials used shall be free from vegetable and loamy matter and shall be deposited on a clean surface in the proportions as hereafter specified. No salt water aggregate will be allowed and no stone larger than will pass through a 3/4" ring shall be used except for foundations.

PROPORTIONS.

The concrete shall be composed of one part of an approved brand of Portland cement to two parts of clean sharp sand to four parts of clean fresh water shingle all mixed in an approved batch mixer to the consistency of jelly.

CEMENT used throughout shall be of an approved brand and shall meet the requirements of the British Standard Specification for Portland Cement. No slag cement shall be used under any circumstances. The cement shall be kept dry and shall not be kept too long before being used. The cement used shall be fresh and samples shall be deposited with the Architect when requested.

AGGREGATE

The clause governing the size of the aggregate will be strictly enforced and any rejected material must be at once carted from the site. No shingle from the site shall be used.

WATER used shall be clean and shall be regularly gauged for each batch so that the consistency of the concrete shall be uniform.

DEPOSITING.

No concrete shall be thrown into a mould from a greater height than 12'0". Each beam, column and floor slab panel shall be filled in in one operation and where the work is left the joint must be over the beam or in the centre of the panel. The deposited concrete shall be kept free from vibration for at least eight hours.

GROUT WASH.

When concrete has set it shall be thoroughly cleaned off and a thin grout of neat cement washed over it before any additions are made to it.

BOXING and any false work shall be of such materials as will leave no stain. The boxing etc. must be held in place by stout wires drawn taut by any means, but bolts passing through the concrete will not be allowed. The wires shall be broken off below the surface of the concrete before any plastering is done. All boxing shall be perfectly rigid and carefully set and made so that the concrete when set will be of the form required. Columns shall be provided with means for cleaning out all rubbish before the pouring of the concrete is done. All forms shall be hosed down before pouring commences to clear out all rubbish and also to thoroughly wet the

forms. The forms shall not be open enough at the joints to allow the finer portions of the concrete to escape. Care shall be taken to see that the forms are properly cramped up. In all cases the concrete shall be thoroughly tamped in the forms so that all the steel is properly and efficiently covered and that as small a percentage of voids as possible is the result. Where the concrete is poured against adjoining property building paper shall be used to prevent the concrete adhering in any degree to the wall. This is an important point and the contractor's attention is drawn to it.

REMOVAL OF FORMS. No forms shall be removed until the work is strong enough to take 50% more than any load it has to sustain, no time being less than two days for walls, seven days for columns and three days for sides of beams, fourteen days for underside of beams and seven days for roof or floor slabs.

FOUNDATIONS shall be formed of concrete as before specified to the sizes shewn and reinforced as shewn on the drawings. In all cases the foundations shall rest on an absolutely solid bottom as the building is designed to sustain live loads and should any faulty ground be met with the attention of the Architect shall first be called to it and his instructions followed.

REINFORCEMENT.

THE STEEL used shall be as shewn on the drawings or such as the City Council Inspector may require under the by-laws. Any part not shewn reinforced shall be treated as the corresponding part shewn reinforced or as the Architect shall direct. The reinforcements shall consist of round steel bars free from welds, scaly rust etc. and shall be

manufactured by the 'Open Hearth' process, and shall be of the very best description. No re-rolled old materials will be allowed.

THE STEEL shall have an ultimate tensile strength of not less than 28-32 tons per square inch of section and an elongation of at least 15% on a test piece of 8" in length. All bending shall be done cold. Where the bars are very heavy the heating shall be done to a dull cherry red before bending. All oil or paint shall be thoroughly cleaned from the steel before it is embedded in the concrete. The steel when bent cold round its own diameter shall not bend or crack. In all joints in vertical reinforcement there shall be provided an over lap of at least twenty-four times the diameter of the upper bar. All hooks at ends of bars shall be of a ⊃ form. The inner diameter shall be equal to four times the diameter of the bar except where the hook fits over a main reinforcing bar. The length of the straight part beyond the curve shall be equal to four times the diameter of the bent bar. All tensile and shear reinforcement shall be hooked at each end.

WELDING. Any steel that takes tensile or shear stress shall not be welded and no part of the steelwork in this contract shall be welded unless permission has first been obtained in writing from the Architect.

PLACING. The steel shall be placed in the exact positions indicated on the drawings and once placed in position the steel shall not be disturbed.

IN PIERS AND COLUMNS the rods shall be put together as per drawings before being placed in the forms, the rods being kept apart by 1/2" bars placed diagonally with split ends placed about 5' centres.

ALL BEAM REINFORCEMENT shall be carefully put together, the rods

in the exact positions indicated on the drawings and undisturbed when the concrete is poured. The ends of all rods in all beams shall be bent as before mentioned.

RE-ADJUSTMENT OF METAL. It must be understood that steel reinforcement will be required in all concrete, but the Architect reserves the right to alter the shape or position of any reinforcement without additional cost to the employers provided that no more metal than is implied in the contract be used and that the altered sizes are procurable in the New Zealand market.

STAIRS. Entrance and exit shall be as shewn on the drawings formed in concrete as before specified, each stair reinforced with two 1/2" rods and carried to 6" concrete strings or spandrail walls all reinforced as for walls and poured at one operation.

CURTAIN WALLS, enclosure walls and wherever else indicated or directed shall be reinforced by 3/8" rods at 12" each way. Where openings occur in these walls a 1/2" rod shall run all round the opening overlapping 6" at each intersection.

CUTTING. No cutting for pipes etc. shall be done until the concrete is thoroughly set. If possible provision shall be made for pipes etc. before the concrete is poured.

BREEZE BRICKS shall be embedded in the concrete for fixing of joinery etc. wherever required for the purpose.

ALL CONCRETE FLOORS and stairs (entrance and exit) and the small area shall be finished smooth off the screed and of the thickness shewn with one of cement to one of sand while the concrete is being laid.

HOLES FOR PIPES ETC. Care should be taken to leave spaces where required for the introduction into the building for gas

or water pipes etc. electric conduits or other purposes. No punching of walls or floors will be allowed until the concrete is properly set.

PARAPETS shall be carried up as shewn with proper openings over rain water heads. The tops of all parapets shall be weathered to slope inwards.

VENTS. Leave holes where directed and fill in with expanded metal of small mesh.

WORKMANSHIP. All concreting shall be done as quickly and as efficiently as possible and to the entire satisfaction of the Architect. No concrete that has stood for more than half an hour shall be used. The face of all concrete shall be left sound and solid and free from excrescences.

TRENCHES and pier holes must be kept drained while the concrete is being deposited in them and afterwards for a period of ten hours.

ALL CONCRETE FLOORS, stairs, landings, areas, etc. shall be finished off the screed finished ready to take tiles,marble or whatever is specified herein, and must be left smooth and in a condition to take the material specified for it. In the case of the area,the lavatories and the exits,the top shall be finished smooth and hard with a full to the gulley or street as the case may be.

BOLTS. Provide and fix all bolts, straps and other fastenings necessary to take the circle waling pieces,struts,hangers etc. that may be necessary or shewn,of the sizes and shapes specified, shewn or as directed. The Contractor must allow for sufficient material.

CONCRETE PROJECTIONS. The Contractor shall carry out all work necessary to form the projections,cornices etc.that are shewn on the drawings and as will be detailed later.

HEATING ENGINEER.

Allow the sum of £ for a heating system and give every facility to the workmen engaged in installing same.

ELECTRICIAN.

The workmanship and materials throughout shall be of the best and all work shall be carried out in strict accordance with the rules and regulations of the City Council Inspectors and the Fire Underwriter's Inspectors in view of the new voltage. (Mr Robertson allow a lump sum until I can write this fully).

STRUCTURAL STEEL.

MATERIALS shall be the best of the kinds specified. All steel shall be British and 300 tons will be supplied to the Contractor on the site, and he shall allow the sum of £11:10/- per ton for the steel without any work whatever being allowed for above this amount. He shall erect all steelwork of the weights and shapes shewn, and he must add to the price above stated what sum he requires to finish the work complete. Any steel required over the amount stated shall be supplied by the Contractor and finished complete as before. The work shall be in accordance with British Standard Practice.

ALL STEEL shall be uniform in quality and weight and of British manufacture and shall be of standard sizes except where specially mentioned or shewn. Each member shall be straight, true to section and with clean smooth surface free from all imperfections.

GUSSETS ETC. All gussets, splicing pieces etc. shall be cut from plates no bar iron shall be allowed except for straps etc.

CLEANING DOWN.

The whole of the structural steel work shall be cleaned down with a wire brush before being embedded in the concrete or painted to remove all scale etc.

RIVETS shall be soft steel of approved manufacture.

DRILLING All bolt or rivet holes in any section shall be drilled. No pouching shall be done except in plates.

PITCH. All rivet holes shall be drilled to standard pitch accurately laid out by templates and spaced in a true line.

ASSEMBLING.

All work shall be carefully and accurately assembled.

DRIFTING. No drifting of holes will be allowed on any account. Any holes not being concentric shall be carefully reamed out to make them so before being rivetted or bolted.

RIVETTING. All rivets whenever practicable shall be machine driven and shall completely fill the holes.

CONNECTIONS.

Where different sections are to be rivetted together filling pieces shall be fitted to exactly pack the smaller member to the larger. Where rivetting is not possible the connections shall be made with tightly fitting turned bolts. Where ends of girders and joists are built into walls they shall be drilled and a 12" length of 1" rod shall pass through to act as a tie.

STEEL CONSTRUCTION.

All the steel beams and columns with the proper bases shall be fixed into position with all necessary fish plates, straps, bolts and connections wherever required. The various sizes and weights are shewn on the details and where shewn in one part of the work. This shall mean that corresponding parts shall be similarly treated.

COLLAPSIBLE GRILLES. Allow the sum of 8/- per square foot for collapsible grilles to Front Entrance, entrances to each shop, each exit on Majoribanks Street and to lavatory in Majoribanks Street. These grilles shall be 6'6" high with all necessary braces,guides,supports etc. complete.

PLASTERER.

All materials shall be of the best of the kinds specified and the work shall be carried out by qualified and experienced tradesmen.

CEMENT used shall be of an approved brand. 'Medusa' or 'Atlas' white cement shall be used on front elevation and on returns north and south side as far as main wall of Theatre.

~~SAND~~ Lime shall be the best and of an approved brand and run at least one month before using.

HAIR The hair shall be long curled cowhair free from grease and other impurities.

WALLS AND CEILINGS.

The walls and ceilings and beams throughout where not shewn to be tiled or covered with fibrous plaster or panelling shall be rendered,floated and set. One pound of hair shall be well beaten in and incorporated with every three cubic feet of coarse stuff.

ANGLES. All external angles inside the building except those on beams shall be finished in Keen's Cement on a Portland cement backing finished hard and true.

RENDERING shall be in the proportion of three parts sand to one part lime with the addition of hair as before mentioned. The walls and portions above dadoes shall be trowelled

hard and smooth with a steel trowel. The exterior walls shall be similarly treated but not to so pronounced a degree and in the proportion of two parts sand to one part cement.

THE WHOLE of the external wall surfaces including heads, sills, reveals, tops and backs of parapets and wherever else required except those portions already specified to be 'Medusa' or 'Atlas' shall be plastered in three coat work finishing at least 5/8" in thickness. Each coat shall be allowed to become hard before the next is applied. Sufficient toxement or other waterproofing compound must be incorporated to allow of the walls being left watertight. This part of the work is important especially on the more exposed north and south sides and the contractor must leave the building watertight and shall paint the walls outside should this be necessary to gain the desired effect.

THE SAND for the white cement used on front elevation and returns shall be silver said from Scorching Bay or as approved.

GENERAL. The plasterer shall make good after other trades and leave the work white and in thorough repair at completion.

TILING. The whole of the tiling on floors and walls shall be supplied by the employers, but the contractor shall take delivery of same and shall lay in the best manner possible with all materials and to the various designs and patterns as directed. The floor of entrance will not be of an intricate pattern but will be in mosaic. The walls will be to a pattern with skirting, checker bands and capping but not intricate. The wall tiling of entrance shall continue up the stairs at a height of 6'0" from the floor of vestibule and 6'0" from tread and shall finish where shewn on the longitudinal section sheet No.

MARBLE WORK.

The marble to entrance steps and stairs shall be best quality -white Sicilian marble 1½" thick with rounded nosings and returns finished with sand polished face for treads and polished face for risers. The risers shall be 3/4" thick. Each tread and each riser shall be in one piece. The treads and risers shall be properly bedded and shall be prepared to take the balustrading of stairs.

The newels at bottom and top and also on landings shall be to detail and executed in Calcola (Australian marble) properly jointed and secured.

The slabs at intersections of foyer rail with vamp rail shall be of Calcola marble properly supported on a strong iron support sufficiently strong to carry a large palm or figure.

TICKET BOX.

The marble on the face of the ticket box shall be Calcola and shall be cut and reversed and placed so as to form a chevron pattern as shown. The slabs shall be held in place by strong copper cramps fixed to the concrete sides of the ticket box. The surrounds shall be of brass as per detail properly mitred and brazed.

PROTECTION. All tile work and marble work must be protected until the building is completed. The Contractor must make good any work marked or disfigured as the result of the work of all the trades.

COLUMNS. The columns at entrance shall be of marble to detail and in one length for the shaft the echinus and abacus may be separate, the joint being made in the sinking under the annulets.

FIBROUS PLASTER WORK.

The contractor shall perform all decorative work as

shewn in fibrous plaster including that on sides and soffits of stage, circle balustrading and all enrichments on walls and ceilings of entrance, Foyer and auditorium and wherever else indicated. The whole of the work shall be clean, sharp, neatly mitred stopped when fitted into position and securely fixed with galvanized nails or screws. No wire nails shall be used on any account. All parts shall be to future detail. The underside of circle and bridge to circle splays and sides and sounding board to stage also auditorium ceiling shall be covered with fibrous sheets securely fixed with galvanized nails. All other surfaces shall be plastered as specified.

CARPENTER and JOINER.

All materials and workmanship shall be of the best of the kinds specified. The timber used throughout this contract shall be free from defects and shall be heart throughout. No sap or part of sap shall be used. All timber shall be dry and shall be stacked on the site within one month of signing the contract. Should there not be sufficient room on the site to accommodate the timber required for the work then the contractor must inform the Architect where the timber is stacked in order that the clause re stacking of timber may be observed. Should the timber not be stacked as required within one month of the signing of the contract then the contractor shall pay by way of liquidated damages the sum of £1 for every day or part thereof that the timber remains unstacked.

OUTSIDE DOORS and their frames shall be of heart of totara.

INSIDE DOORS. All joinery and partitions shall be heart of red pine selected for figure

EXPOSED FACES shall be wrought except where otherwise specified.

ALL JOINERY work over 9" wide shall be panelled and all joints and angles in joinery shall be glued and pinned. All joinery shall be roughly framed together and stacked for three months before glueing up.

FRAMES. All frames shall be properly built in and secured with 1½" x 3/16" wrought iron ties 3' apart screwed to frames one end and built 12" into concrete at other.

STRINGERS and joists shall be of heart totara 7" x 2" and 6" x 2" respectively spaced as per plans. They shall be firmly wired to the piles by No. 8 galvanized wire sunk into the pile when green and stapled to the stringer .

CEILING JOISTS whenever shewn shall be 5" x 2" spaced at 18" centres and of heart of red pine.

SARKING shall be 8" x 1" firmly butted and cramped together with no opening anywhere as the roof has to be covered with Paroid or other approved material. The sarking shall be firmly nailed at each intersection.

GUTTERS shall be formed ready for the roofing contractor and shall be dressed smooth so that no ridges are left which may injure the roofing material. They shall be laid to the proper falls and the contractor shall consult with the roofing contractor regarding the proper falls required.

PARTITIONS shall be formed where shewn with 4" x 2" heart of red pine studs checked into 4" x 3" heads and sills and properly dwanged to receive lining or panelling as the case

may be. The studs shall be at 18" centres or to suit the setting out of the panelling and the dwanging shall be similarly accommodated. Frame for doors and other openings. Where the partitions are formed on the curve this part of the work shall be properly done.

LINING and PANELLING shall be securely nailed to the strapping or studding by double nailing. The strapping to concrete walls shall be done by securing to breeze blocks left in the concrete for the purpose of taking the strapping.

THE WALLS OF FOYER, the corridors from Foyer leading to circle, the walls of ladies and gentlemen's Lavatories,and wherever else shewn or indicated shall be square panelled to a height of 7'0" two panels in the height finished with a 7/8" plate rail supported on two small brackets to each muntin,all to detail. The panels or Foyer shall be 3 ply oak selected for figure, the skirting rails and muntins also plate rail and brackets shall be oak. The corridor leading to circle on each side shall be Queensland maple throughout and of similar design to panelling of Foyer. The panelling of cloakrooms shall be heart rimu but as specified above. All other panelling shall be heart rimu. Where walls are not plaster and are not specified as panelled or tiled they shall be lined with 4" x 5/8" T. & G. lining securely fixed by secret nailing to strapping to walls. All lining shall be vertical with no heading joints.

TEMPLATES. Provide all necessary furring, templates, boxing etc. wherever required for all trades throughout the building.

CLEAR OUT from time to time and care shall be taken that no debris of any sort is left under the floors at completion.

SPACING. Unless specified otherwise all spacing of joists rafters and studs shall be not more than 18" centres.

TRIMMERS Trimmers and trimming joists shall be 1" thicker than ordinary joists. Trim for all ceiling vents and manholes where shewn.

FLOOR JOISTS OF SHOPS shall be 5" x 2" heart totara well spiked to 6" x 3" stringers and wired and stapled on concrete piles as before specified for Theatre.

STAGE FLOOR. Joists shall be 5" x 2" laid on the concrete slab and wired thereto and firmly stapled.

DRESS CIRCLE JOISTS shall, unless shewn to be otherwise, be of oregon of the sizes shewn on the drawings and spaced as shewn. The oregon shall be of the clean and not merchantable class free from all defects. The waling pieces shall receive them on horizontal bearings well spiked to waling pieces and every third joist shall be bolted to straps let into the concrete. Provide and fix all necessary strutting for the support of the circle joists from cross girders.

SOFFIT OF DRESS CIRCLE. The undersides of all joisting of circle shall conform to the local bylaws and shall be covered with 26 gauge galvanised sheet iron with 1" laps and close tacked. Over this shall be placed the fibrous sheets securely fixed to the joisting of circle and specified under "Plasterer".

BIO BOX The walls floor and ceiling of bio box shall be covered on the lining and studding shewn by 26 gauge sheet iron with 1" lap close tacked and neatly taken round all openings. The door shall be iron clad to meet the requirements of the Fire Underwriters.

HERRING BONE STRUTTING. All joists shall be braced with rows of herringbone strutting of 2" x 2" Oregon securely nailed and fixed at 6'0" centres.

BLOCKING. The blockings forming the treads and risers of the circle flats shall be carefully cut and fitted and shall be formed of 4" x 2" oregon as shewn on detail.

FLOORING. The whole of the floors throughout the building except where specified or shewn to be otherwise shall be covered with 3" x 1" S. & G. and dressed selected all heart matai flooring, thoroughly seasoned cut in as long lengths as possible well cramped up and double nailed at each intersection. The flooring forming the treads in circle shall be laid in as long lengths as practicable and intersected and fitted at convenient points on the circle. The flooring shall be well cramped up and shall project beyond the seat risers 1/2" slightly rounded on the edge to form a nosing. This flooring shall be securely nailed with two nails at each intersection.

SEAT RISERS shall be of 6" x 5/8" Oregon T.G. and V.jointed well cramped up and nailed to blockings and fixed horizontally.

MARGINS 2" wide shall be securely fixed to all openings where necessary and shall be neatly mitred at angles.

AISLE STEPS. The aisle steps to circle shall be formed wherever shewn or necessary with 1½" treads and 3/4" risers and returns. The nosings of all seat risers shall be square with the nosings of the flats so that the carpets shall run properly down the aisles.

CUTTING INTO JOISTS. The Contractor shall not cut into any

joist or trimmer to accommodate any pipe or conduit to such an extent as will reduce the strength of the joist or trimmer.

SPLAYS TO STAGE. The splays to stage shall be formed of 5" x 2" heart rimu studs with all necessary top, bottom and intermediate plates, well braced and herring-boned at 6'0" centres all properly spiked and nailed and bolted to auditorium wall. The false ceiling and sounding board shall be similarly constructed and securely hung to roof principals as shewn on the section. The ceiling joists shall be 5" x 2" heart rimu.

WALLS OF BIO BOX. The walls of bio box shall be framed up with 4" x 2" heart rimu studs at 18" centres, well braced with 3" x 2" pieces at 2'0" centres and lined as before specified. Trim for door and opening 3'0" x 3'0" where directed.

GENERALLY. The whole of the work shall be left rigid true and straight with all necessary materials to enable fibrous etc. to be applied as the occasion demands.

VENTS IN CEILING. The Contractor shall form vents in the ceilings where shewn on the ceiling plan to the sizes indicated and shall fill in with lattice work as shewn. Each vent shall be taken up a distance of 4'0" with 3" x 2" studding and lined on the inside as for other lining specified. This is to mark the roof timbers from below.

LOUVRES The Contractor shall form the louvres as shewn on the drawings with proper sills heads and frames for the 22 gauge galvanised iron louvres all to detail. The outside of louvres shall be covered with galvanised iron on rough sarking well cramped up and nailed to studding.

ROOFING. The roofing shall be Paroid or other

approved material laid on the sarking in three layers with hot bitumen between each layer and with alternating joints well secured. The material shall have a good 4" lap and shall be unrolled and allowed to stretch in the sun before being laid. The contractor shall obtain a guarantee from the laying contractors that the roof is watertight and the laying contractor shall take whatever precautions he may think fit to give a watertight roof notwithstanding anything that may be specified to that effect. The material shall be well and carefully laid into the overflows over rain water heads and shall be generously coated with bitumen.

COVER the louvres and vents over exit doors with small mesh wire netting to keep out birds.

BALUSTRADES. The balustrades to stairs, Foyer and bridge to circle shall be as shewn securely fixed to floor and handrail as will be detailed. The balustrading to circle front shall be built up as detailed and finished with vertical lining one side and fibrous plaster on the remaining side. This will also be to future detail. The top rail of the circle balustrade shall be upholstered in Utrecht or other approved velvet of approved colour braided and close studded in a neat and workmanlike manner.

BARRIERS. The barrier round the orchestra well shall be framed up with 4" x 2" and covered with fibrous plaster to the detail. The fibrous plaster must be harder than ordinarily made to withstand the usage it will be subject to.

The barrier at entrance to stalls shall be as shewn framed up with 4" x 2" studding lined on

auditorium side and panelled on vestibule side as before specified. The barrier at back of middle entrance on circle shall be as shewn and shall match that shewn on the detail drawing.

FRAMES. All frames shall be securely fixed in the concrete and shall be solid rebated.

ALL JOINERS work shall be proceeded with as soon as possible after the signing of the tender to allow of the timber being well seasoned before being fixed. All timber in joinery that develops defects after being fixed shall be taken out and replaced.

DOORS generally shall have top rails single tenoned and shoulder tongued, middle and bottom rails double tenoned and shoulder tongued.

SIZES Where sizes are given or shewn the contractor shall check same with the actual work before proceeding with the execution of the detail.

PRIMING. All the outside work shall receive a coat of priming of best red lead and linseed oil before being fixed. All inside work shall receive one coat of raw linseed oil.

DOUBLE HUNG SASHES to lavatories and opening into area, in single ladies lavatory in Majoribanks Street and to men's lavatories in Majoribanks Street shall be installed with sashes $1\frac{3}{4}$" and all frames, sills, nosings, cords, weights, lifts and fasteners etc. complete in every detail.

ALL SASHES shewn except those above specified shall be steel of an approved section and manufacture and shall be fixed solidly in the concrete so that no water enters round them. They shall open as indicated on the drawings C indicating casement pivot hung to allow of easy cleaning of

the glass. F indicating fanlight hung to open out and fitted with approved opener. The casement shall be fitted with an approved fastener each casement. Provide and fix two steel sashes about 8" x 12" to apertures in bio box.

SHOP FRONTS shall be of Queensland maple and to detail the work being carefully done and of first-class workmanship. The squares above shop windows shall also be in maple, but the whole of the fronts of the shops in Majoribanks Street shall be in picked heart of red pine and to similar detail.

EXIT DOORS FROM STALLS shall be 7'0" x 6'0" x 2" in the clear and in two leaves as shewn on the drawing. A detail will be given of these doors which shall be hung on three 4" butts each leaf and shall open out only. They shall be each fitted with approved catches but not with locks or furniture except finger plates which shall be to approval. Similar doors to office entrance in Majoribanks Street.

DOORS TO STAGE shall be 2" to detail opening inwards and hung on three 4" butts and fitted with lock and furniture value £3 each door.

EXIT DOORS FROM CIRCLE leading to street on north side and court on south side shall be as for doors to stage as above.

LAVATORY DOORS shall be 6'8" x 2'8" x 1¾ four panelled, no mouldings, hung to 1¼" solid rebated with three 3" steel butts and fitted with approved lock and furniture value £2

each door. The door shall be of the material specified for the panelling immediately adjoining it.

BIO BOX DOOR shall meet the requirements of the Fire Underwriters as before specified.

DOOR TO MANAGER'S OFFICE and Cupboards shall be as specified for lavatory doors.

CABIN HOOKS shall be fixed to all doors one to each leaf - 4" for large doors and 3" for smaller doors with eye securely fixed to the door.

ARCHITRAVES shall be plain bevelled 5" by 1" and of the material of the door it frames shouldered at intersections and well nailed. Architraves shall be fitted to all door and window openings.

OVERDOORS shall be as shewn and to detail.

SKIRTINGS shall be 9" x 1½" plain bevelled of the material of the panelling it accompanies and where not matching any particular timber then it shall be selected red pine. The skirting shall be securely nailed to walls and scribed neatly to the floors.

MOSINGS shall be of 3" x 2" bevelled and fixed to all window openings. Where window boards are necessary they shall be fixed.

SHOP DOORS shall be as shewn to future detail hung to 6" x 3" solid rebated frames moulded slightly with transomes, hung on three 4" butts each leaf and two 3" butts each sash in transome. The Contractor shall allow the sum of £3 for each shop door for furniture, and he shall take delivery of and fix same.

EXIT DOORS shewing on Majoribanks Street shall be 7'0" x 6'0"x 2" heart totara in two leaves and as shewn. Each leaf hung on three 4" butts and each exit door fitted with approved panic bolt. The doors shall be hung on 5" x 3" solid rebated frames and shall open outwards.

BRIDGE TO CIRCLE shall be constructed as shewn on the detailed drawing.

SPANDRAILS shall be fitted in with turned balusters to detail.

CUPBOARDS. All cupboards shall be lined as previously specified for other rooms.

MUSIC ROOM and dressing rooms shall be as shewn, but the music room shall be fitted with shelving divided into pigeon-holes for music fixed securely to the walls and floor. There shall be twenty-six pigeon holes about 10" wide and 12" high formed with 1" shelving and 3/4" divisions. Provide and fix in addition 100' run of 1" shelving for music room to be fixed where directed on suitable supports. Door shall be as lavatory doors hung and with furniture as specified.

DRESSING ROOMS shall be provided with bench and basin as shewn the doors being as specified for lavatory doors.

IRON STAIRS TO MUSIC ROOM shall be spiral securely fixed and to approval.

EXIT STAIRS from circle shall be concrete as specified for entrance stairs and as shewn and in accordance with by-laws regulating exit stairs.

STAIRS TO STAGE shall be as shewn and formed with 12" x 2"

strings, 2" treads and 1½" risers all well housed, wedged, blocked and glued together. Provide and fix 4" x 4" square newels 4" x 3" handrail and 2" x 2" square balusters all to detail. The stairs shall be in selected heart red pine.

TICKET BOX shall be as shewn on the large detail herewith.

AWNING OVER FOOTPATH. The awning shall be constructed as detailed and shall be complete as regards purlins joists etc. The underside shall be lined with 4" x 1" T. & G. flooring and the joists shall be covered with 6" x 1" rough T & G boarding to take the covering all well cramped up and double nailed at each intersection. The fascia shall be divided into boxes which will each be fitted with a lighting point and glazed on the front face and provided with runners and slot with flap over to take the stencil letters indicating the programme. These boxes shall only occur in the sections marked. The remaining faces shall be stout sheet iron close tacked and as per detail. The canopy over entrance is detailed on Sheet No. and shall be executed with all necessary work and materials to complete it as per detail.

FOYER SEAT shall be built where shewn of oak to match the panelling. Legs 2" x 2" 6" x 1" bearers supporting seat 15" x 1⅛" made to slope back slightly. Back shall be formed with 5" x 1" top and bottom rail filled in with 3" x 1⅛" vertical rails spaced 1" apart. Only visible timbers shall be of oak.

PLUMBER.

All plumbing work shall be carried out with the best materials and only by competent and licensed tradesmen.

ALL JOINTS in lead shall be neatly wiped.

ALL WASTES shall be of lead and shall be trapped, and all traps shall be brasscapped.

FLASHINGS in any part of the work shall be done with 5 lbs lead tightly plugged with cast lead wedges pointed and left watertight.

R.W.P's shall be 6" diameter cast iron fixed with clips and shall have all necessary bends, shoes, heads etc. complete and all rainwater shall be conveyed as required by the Inspectors. The R.W. heads shall be of cast iron and shall be of sufficient size to cope with the water from the roofs and shall be of simple design. The pipes shall be in chases in the walls for a distance of 8'0" from path.

WATER SERVICE shall be laid from main in 1" galvanised iron pipes then through 3/4" pipes to each cistern in each W.C. each basin in lavatories and in dressing rooms and to sink in engine room.

LAVATORY BASINS shall be provided and fixed where shewn on approved cast iron supports and provided with wastes traps etc. all complete. Each basin and sink shall be fitted with nickel plated taps one marked 'hot' and the other marked 'cold'. Allow the sum of £3 for the purchase of each basin and sink without brackets taps or fittings.

W.Cs. Each W.C. shall be fitted up complete where shewn

with approved pedestal etc. with flap and cover value £2 each W.C. The cisterns, ball cocks etc. shall be of a pattern and design approved by the Inspectors. All shall be left clean and in proper working order.

ALL GUTTERS other than those over Theatre shall be of 5 lbs lead properly laid to falls with all necessary drips cesspools flashing and cover flashings etc:.

ROOF over shops portion shall be covered over the felt and sarking with 24 gauge galvanised corrugated iron screwed to timbers with galvanised screws and lead washers or 'Sun' brand galvanised leadheaded nails may be used. The laps shall be sufficient to ensure the roof being left watertight.

VENTS Provide and fix all necessary vents etc. in accordance with bylaws and leave all complete.

FIRE MAINS. Lay from street fire mains a 3" water main and fix one point in back of stalls near entrance doors and one point in circle near entrance door. Each point shall be fitted complete with a 2½" Hydrant value and hose union 40' of 2½" strong quality canvas hose to approval and brass nozzle. Each hose shall be accommodated in approved basket and the hydrant and basket shall be placed in recesses prepared for them.

LOUVRES shall be 22 gauge bent over top and bottom and fitted into frames provided by 'Carpenter'.

AWNING Carry out any necessary plumbing work in connection with the awning on Courtenay Place front and also on Majoribanks Street front.

TRAYS with proper outlets shall be provided and fixed under pedestals and under basin with flap over outlets.

WASTES The wastes from Men's cloak rooms will have to be taken to area on south side of building in which case the contractor shall consult the inspectors as to the best method of carrying out this work.

SEATINGS for all girders columns and joists shall be provided for the ends of such where resting on concrete.

STORAGE TANKS of the capacity and sizes necessary shall be provided and connected where directed by the Inspectors.

URINALS shall be installed where shewn to the value of £5 per stall and in accordance with regulations. The urinal in Majoribanks Street shall be fitted with automatic flushing cistern copper sponge pipes etc. to meet every requirement of the City Inspectors.

VENTILATING FIRE FLUE.

Carry the ventilating flue from bio box as shewn with proper cap,firmly stayed to roof and securely flashed. The flue shall be made so that a thickness of 1/2" asbestos packing insulates the surface of the inner ring from the outside outer.

FIREPROOF SHUTTER TO BIO BOX. The shutter shall be of asbestos sheet greatest thickness obtainable running in asbestos sheet grooves hung to pulleys on ceiling by some cord that will readily fuse and allow shutter to drop. The sizes of the holes cannot yet be given as this will depend on the position of the machines in the box.

PAINTER

MATERIALS and workmanship shall be the best of the kinds specified and all materials shall first be opened in the presence of the Architect if so desired.

WHITE LEAD shall be genuine and approved.

OIL shall be linseed and of approved brand.

STOPPING shall be of the best putty tinted for oiled work and shall be done on all work after the first coat is on.

ALL OUTSIDE AND INSIDE METAL WORK shall receive two good coats of best oil paint of approved colour. This shall include down pipes, vents, steel sashes etc. but not the roofing iron. It shall include, however, all structural steel work where visible and the paint in this case shall be 'Siderosthen' 'Graphulatum' or other steel paint applied as per the directions of the suppliers.

OUTSIDE WORK. All exposed woodwork including doors, windows, ventilators underside of awning and all soffits shall be primed and shall then receive two good coats of best oil paint of approved colour. The work must be solid and should it require an extra coat this must be done without expense to the employer.

ALL INSIDE WORK shall be similarly treated except that the panelling, foyer seats and doors in panelling shall be stained an approved tint.

FRENCH POLISHING The entrance swing doors shall be stopped and French polished to approval. Include also any woodwork to the ticket box except the shelves which shall be left bare.

WHITE WASHING. All walls and ceilings of engine room under stage, dressing rooms, all walls visible at back of stage shall be whitewashed with a fixed wash in as many coats as will give a good solid effect.

PAINT the walls of light area three good coats of white paint to give as much reflection as possible.

GLAZIER.

The whole of the glass shewn throughout the building including leadlights, glazing to canopy at entrance etc. shall be included in this contract and must be fixed in position, sprigged, back puttied and puttied and left whole and perfectly clean at completion. Windows on elevations and elsewhere except shop and shop doors shall be glazed with 21 oz. best British clear sheet glass free from serious blemish. The windows in Foyer shall be glazed with leadlights with reinforced cames to a design to detail and valued at 10/- per square foot. The Entrance doors and portions of ticket box where shewn shall be in Luxfer(Henry Brookes & Co.Ltd.Willis Street) to detail and value 20/- per foot. The glazing to canopy shall be in 1/4" plate acid hammered and rounded on edges, cut to shape and brilliant cut as per detail. The price for this work may be obtained from the above firm also. The glazing to shop fronts on both elevations shall be 1/4" British polished Plate glass well secured to the maple frames. The doors and transomes shall also be 1/4" British polished plate but bevelled with 1" bevel well secured to the squares shewn. This last shall apply to each elevation. The bowl over ticket box shall be

M. Feeney

RECEIVED
7 JUL 1964
HEALTH BRANCH

CITY ENGINEER'S DEPARTMENT

BUILDING BRANCH

CITY ENGINEER'S DEPARTMENT
BUILDING BRANCH
RECEIVED
6 JUL 1964
MERCER STREET, WELLINGTON

Date .....

To: HEALTH BRANCH:

For your requirements under the Health Act, 1956 -

No additional requirements. In order for approval

8/7/64

HEALTH

KERRIDGE ODEON CORPORATION LIMITED

THEATRE EMBASSY

TO WHOM IT MAY CONCERN

This is to authorise Francis Cowan to view the plans of the Embassy Theatre, Wellington.

1 October 1982

R.P. WILLIAMS
Manager

TO : The Building Division
Town Hall
WELLINGTON

COUNTER ENQUIRY | INTERNAL ENQUIRY

26.11.8C

BUILDING SUPERINTENDENT

Would you please obtain from the Building Permit records the following plans relating to the premises known as:-

Address: EMBASSY THEATRE BUILDING
9-11 KENT TCE
WELLINGTON

Building Permit No's A 1611

Delete each of the following that does not apply:

~~a) I am the owner of the property mentioned above~~

b) I am not the owner of the property mentioned above. The owner has authorised me to peruse the above-mentioned plans with the written permission attached ✓ (tick)

~~c) I am employed by the Wellington City Council Department, telephone extension~~

I understand that I will be charged $1.10 per file
$2.20 per roll plan
and I agree to pay $ 1-10 when the files are available.
(Not applicable to (c)).

Charge to Account
No. 061-900

Plans required by: P.G. ANDERSON
(Please print)

Signed... P G Anderson ...

COUNCIL PROCEEDURE FOR COPYING PLANS

Persons wanting copies can be taken to one of two companies which provide this service. Please indicate below which of the companies you wish to use by marking the box with a tick.

MULTIPRINT - WILLBANK HOUSE ☐ PLAN PRINTING CO - WAKEFIELD ST ☐

I am aware that Council has no control over the cost of the above service and that all charges are my responsibility.

Signed.....

RRIDGE ODEON CORPORATION LIMITED
& QUEEN STREET • P.O. BOX 2191 • AUCKLAND 1, N.Z.
HONE 796-780 • TELEGRAPHIC ADDRESS 'KERFILM' • TELEX NZ21400

DIVISION Buildings

311

ember 1987

ty Engineer
gton City Corporation
te Bag
INGTON

r Sir

: EMBASSY THEATRE - KENT TERRACE

I would be pleased if you would allow Mr Colin Orchiston of Orchiston & Associates, Registered Architects, access to any records you may be holding in respect of this building. Should Mr Orchiston require copies of selected documents, we would ask that these be provided on request.

Yours faithfully

N. F. Osmers

N F Osmers
BUILDINGS SUPERVISOR

NFO/LK

ORCHISTON and ASSOCIATES
ARCHITECTS
Hope Gibbons Building
58-70 Taranaki St
Wellington
P.O Box 9136 — Ph. 845-021

PLAN P.31
9-11 KENT TCE 12-10 MASONIC

A 1611
C 7650
C 32628
C 33252

PERMIT NO: A 1611 C 7650 C 30628 C 33252 Embassy Theatre ADDRESS : 9-11 KENT TCE

DATE 19-1-88

☐ a. I am the owner of the propetty mentioned above.

☑ b. I am not the owner of the property mentioned above. The owner has authorised me to peruse the above - mentioned plans with the written permission attached ✓

☐ c. I am employed by the Wellington City Council Department, telephone extension _____

* I agree to pay a Search/Print fee of $8.00. for the first plan file or microfiche print and $2.00. for each additional file/print and understand that this is non-refundable should the plans be unavailable. N.B. 24 hours notice is required for plan search but microfiche are available for viewing/printing on receipt of the fee.

Plans required by A.S.A.P

(PLease print)

Signed Colin R Duchars

COUNCIL PROCEDURE FOR COPYING PLANS

Persons requiring copies can be taken to one of two companies which provide this service. Please indicate below which of the companies you wish to use.

MULTIPRINT- WILLBANK HOUSE ☐ PLAN PRINTING CO - WAKEFIELD ST ☐

I am aware that Council has no control over the cost of this service and that all charges are my responsibility.

Signed Colin R Duchars

PLAN SEARCH AND PRINT REQUEST

PLEASE PRESENT RECEIPT
WHEN COLLECTING PLANS

ACCOUNT NUMBER 10485-400
DATE
AMOUNT DUE: $

GST NO.: 10-666-182

ADDRESS: 9 — 11 Kent Tce

CHARGES:

PERMIT UNDER 10 YEARS OLD (INCLUDES 1 PRINT FROM MICROFICHE)

1st Permit $12.00 (non-refundable if plans unavailable)

Each additional permit $3.00

Each microfiche print $3.00

WELLINGTON CITY COUNCIL BUILDING INSPECTION BRANCH RECEIVED 22 JUN 1992 WAKEFIELD STREET

PERMIT OVER 10 YEARS (ARCHIVE SEARCH)

1st Permit $25.00 (non refundable if plans unavailable)

Each additional permit $5.00

CARD OR LEDGER SEARCH ONLY $2.00 per property/enquiry

ARCHIVE PLANS SEARCH ACTIONED ON TUESDAY AND THURSDAY MORNINGS ONLY (but microfiche and cards are available to view on receipt of fee)

PLANS WILL BE HELD FOR 5 WORKING DAYS ONLY

I am/am not the owner of the property above.

I have the written permission of the owner ~~SCENE~~ SEE ATT (Attached)

I am employed by the Wellington City Council .....Dept Tel.....

PERMIT NUMBER/S..A16.11..... Plans required by:..Dunning Thornton.....

.....Missing..... Signed:.....

.....Embassy.....Theatre.... Address:..Box 27-153.....

COUNCIL PROCEDURE FOR COPYING PLANS:

Copies of plans can be obtained from:

MULTIPRINT - WILLBANK HOUSE REIGERS - BOND STREET

(Please nominate preference)

I am aware that Council has no control over the cost of this service and that all charges are my responsibility.

Signed:.....

COUNTER ENQUIRY

INTERNAL ENQUIRY

THE CHIEF BUILDING INSPECTOR:

Would you please obtain from the Building Permit records the following plans relating to the premises known as :- Embassy Theatre

Address: 9-11 Kent Terrace.

Building Permit No's. A 1611 / C 7630, C 30628 / / /

Delete each of the following that does not apply:

a) I am the owner of the property mentioned above.

b) I am not the owner of the property mentioned above. The owner has authorised me to peruse the abovementioned plans with the written permission attached. ✓ (tick)

c) I am employed by the Wellington City Council, _____ Department, telephone extension _____.

I understand that I will be charged $1.00 per file
$2.00 per roll plan
and I agree to pay $ 3 when the files are available.
(Not applicable to (c)).

See MR Minute Record.

Charge to Account
No.061-900.

Plans required by: Mr Francis J. Cowan
(Please print.)

Signed... F J Cowan (students)

45 Huntleigh Park Way
Ngaio Wgtn 4.

Embassy Theatre

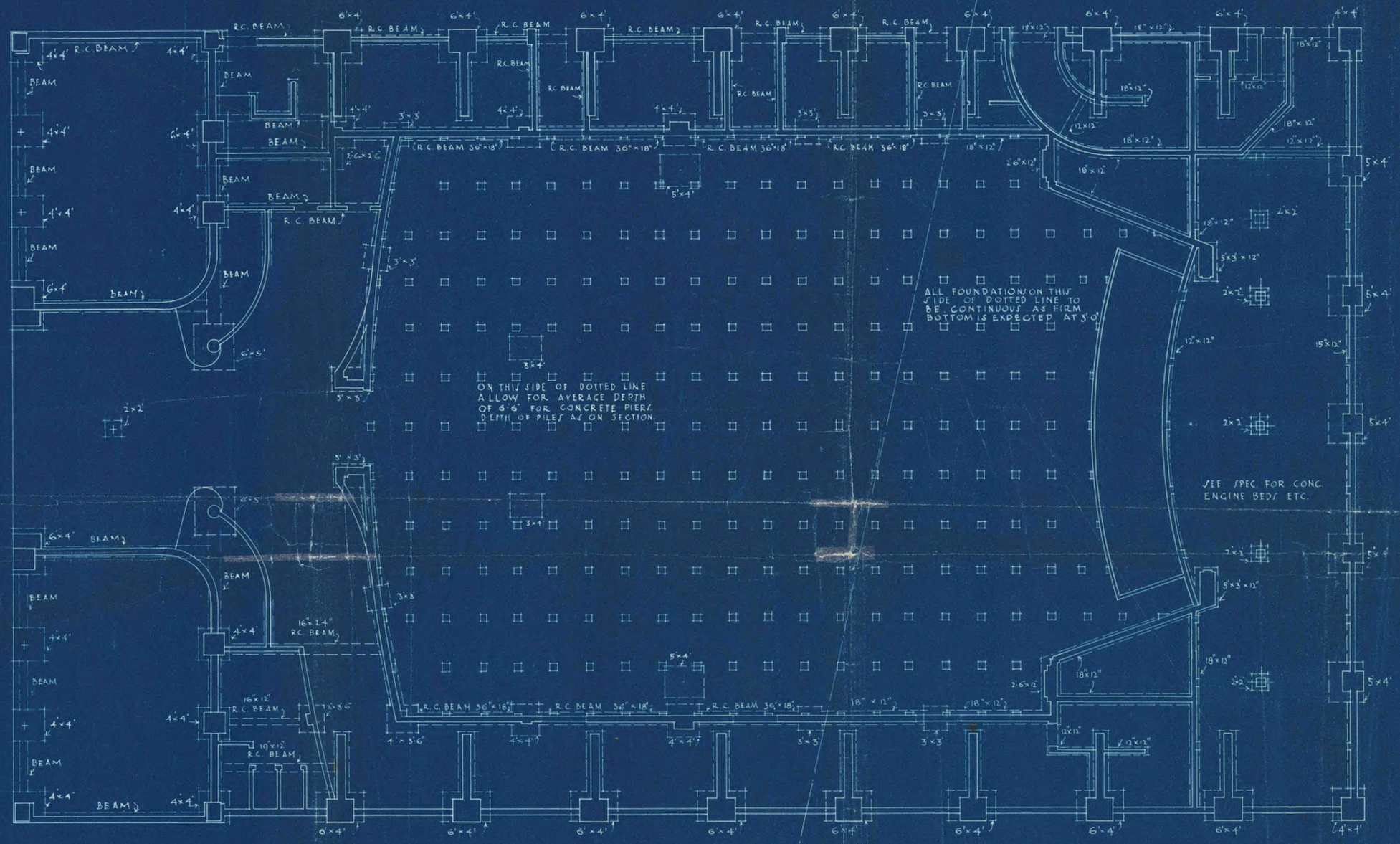
SHEET No 1
FOUNDATION PLAN
SCALE. 1/8" = 1'-0"
DATE. 1 March 1923

NEW THEATRE . COURTENAY PLACE . WELLINGTON.

FOR THE

DE LUXE THEATRE CO. LTD.

LLEWELLYN E. WILLIAMS.
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT. STRUCTURAL
ENGINEER. WELLINGTON.

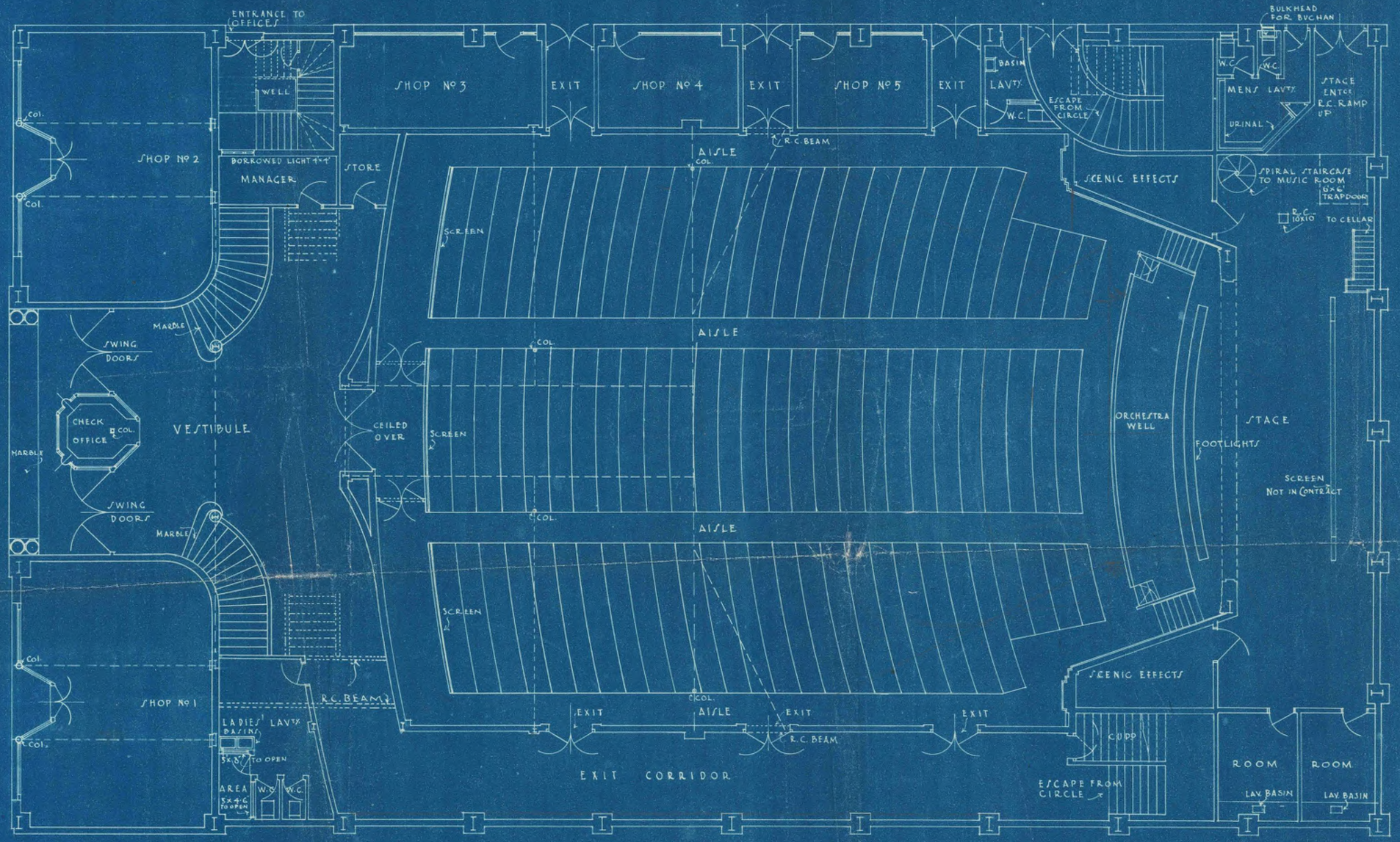


SHEET No 2
STALLS PLAN
SCALE • 1/8" = 1'-0"
DATE • 1 March 1923

• NEW THEATRE • COURTENAY PLACE • WELLINGTON •

FOR THE
DE LUXE THEATRE CO. LTD.

LLEWELLYN E. WILLIAMS •
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT • STRUCTURAL
ENGINEER • WELLINGTON •



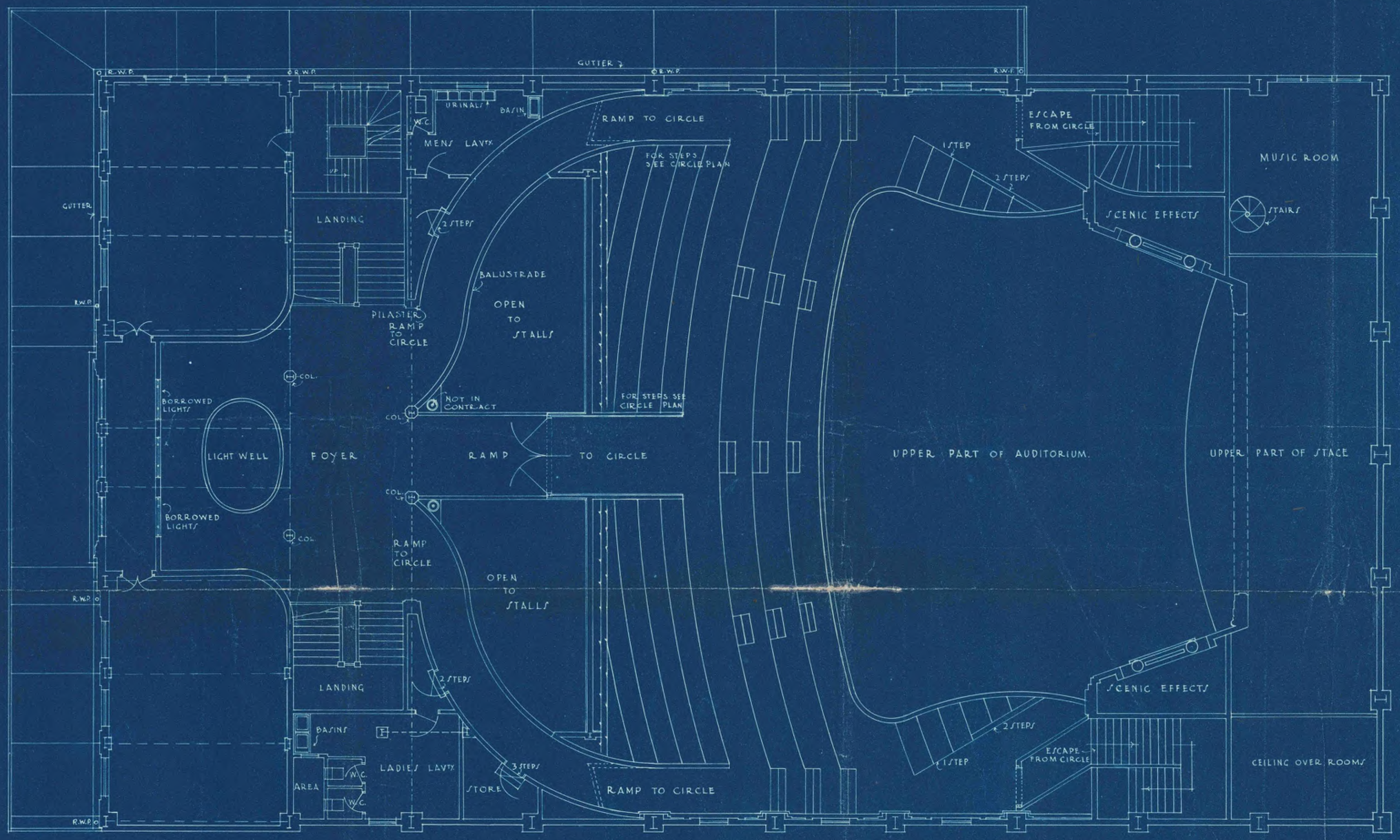
SHEET No 3
FOYER FLOOR PLAN
SCALE. 1/8" = 1'-0"
DATE. 1 march 1923

NEW THEATRE . COURTENAY PLACE . WELLINGTON .

FOR THE

DE LUXE THEATRE CO. LTD.

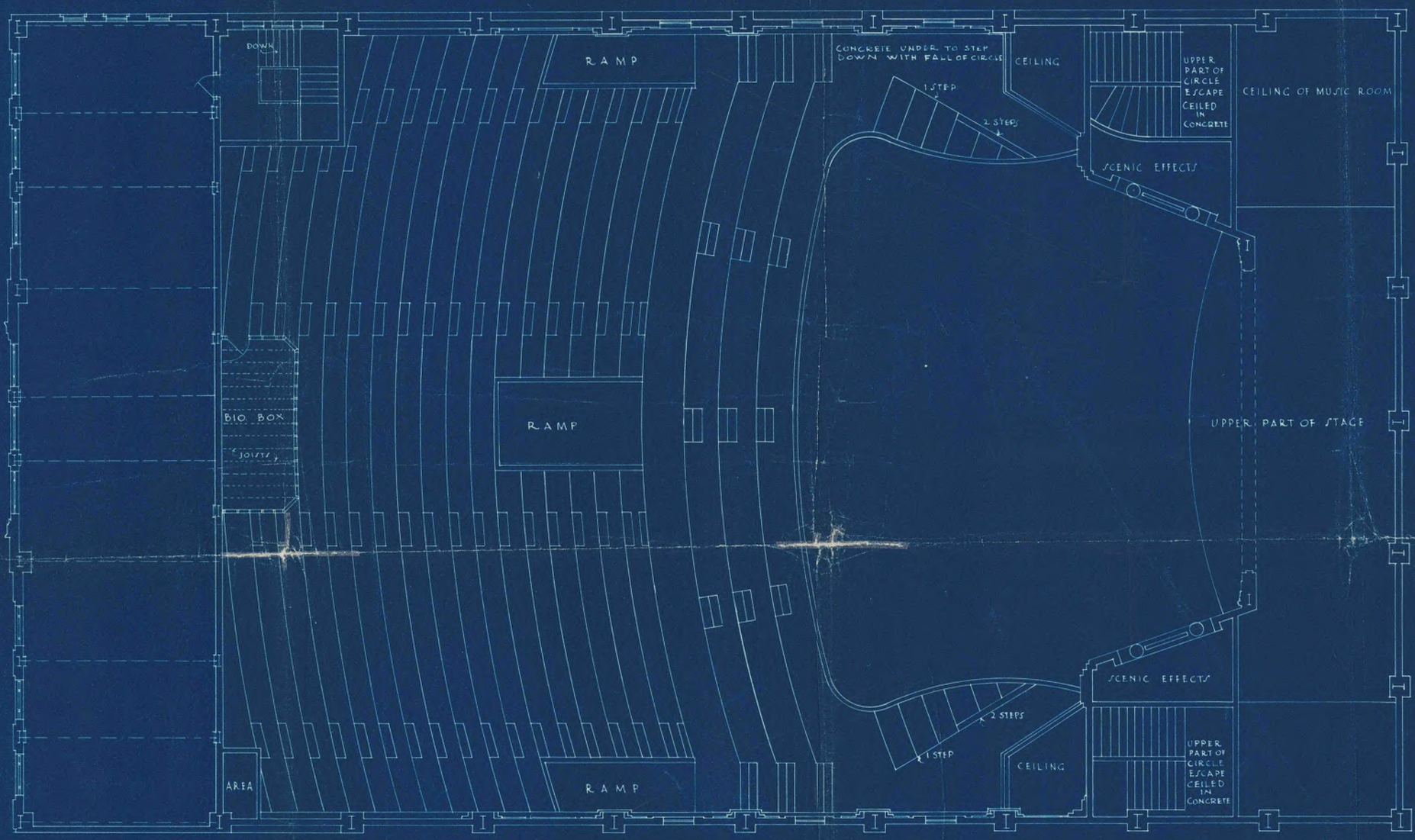
LLEWELLYN E. WILLIAMS .
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT . STRUCTURAL
ENGINEER . WELLINGTON .



SHEET No 4
CIRCLE PLAN
SCALE 1/8" = 1'-0"
DATE 1 March 1923

NEW THEATRE · COURTENAY PLACE · WELLINGTON ·
FOR THE
DE LUXE THEATRE CO. LTD.

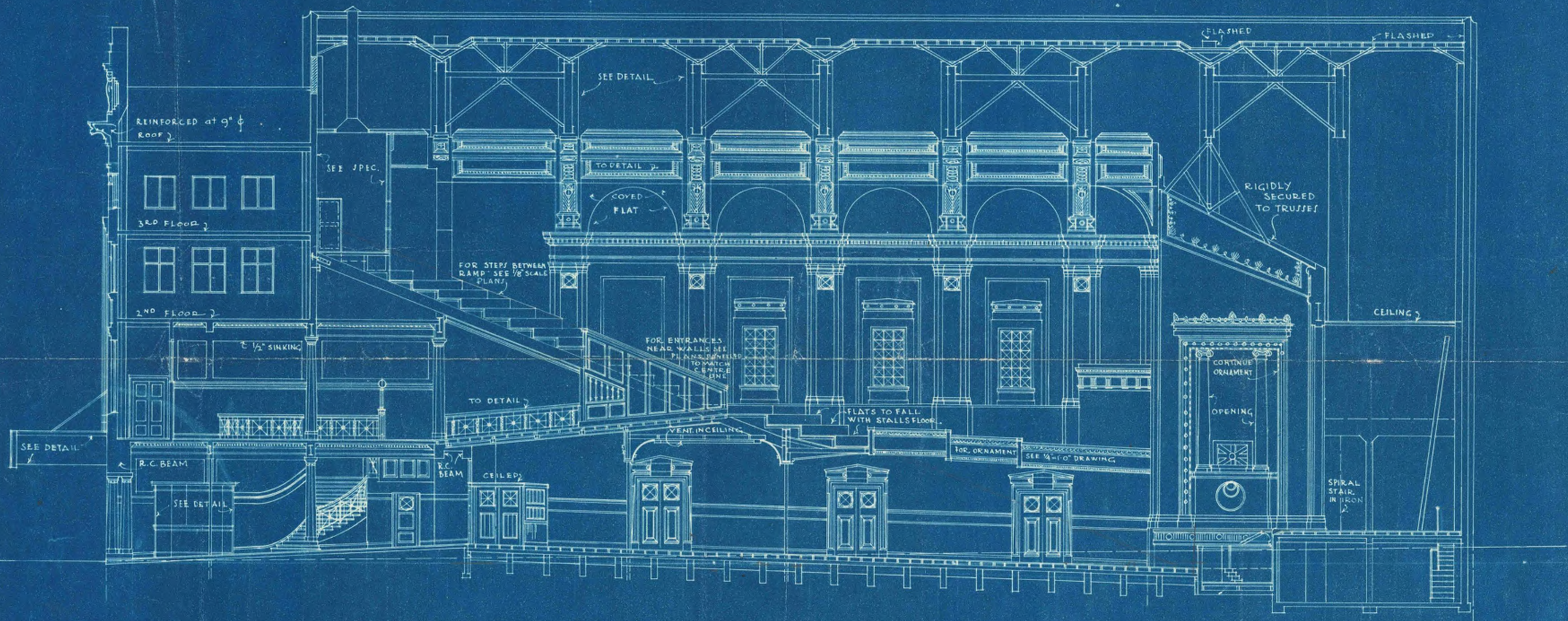
LLEWELLYN E. WILLIAMS ·
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT · STRUCTURAL
ENGINEER · WELLINGTON ·



SHEET No 5
LONG^L SECTION
SCALE. 1/8" = 1'-0"
DATE. 1. March 1923

NEW THEATRE . COURTENAY PLACE . WELLINGTON .
FOR THE
DE LUXE THEATRE CO. LTD.

LLEWELLYN E. WILLIAMS .
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT-STRUCTURAL
ENGINEER-WELLINGTON.



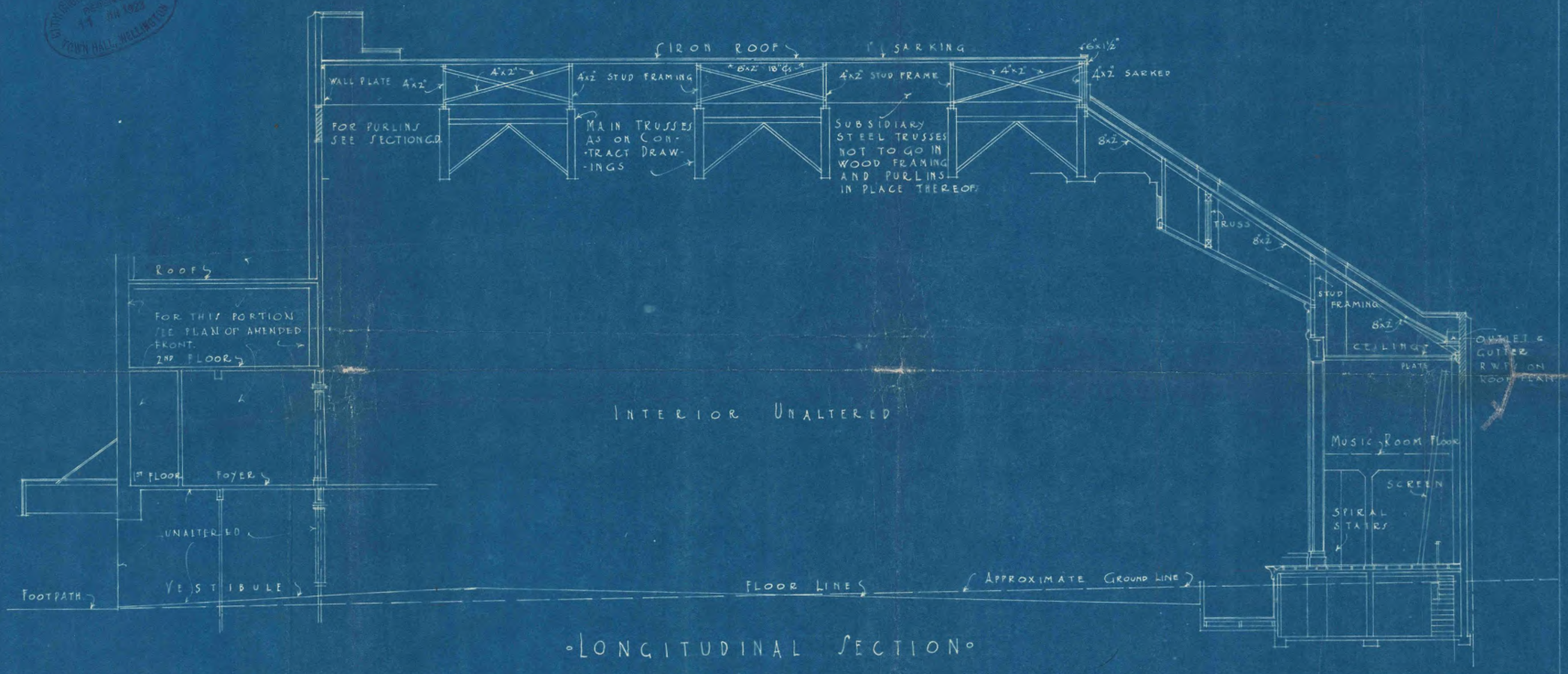
SHEET No. 5A
AMENDED PLAN
SCALE: 1/8" = 1'-0"
APRIL 1923

·NEW THEATRE · COURTENAY PLACE · WELLINGTON·

FOR THE

·DE LUXE THEATRE CO. LTD·

LLEWELLYN E. WILLIAMS
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT & STRUCT-URAL ENG^RS · W'GTON·



INTERIOR UNALTERED

·LONGITUDINAL SECTION·

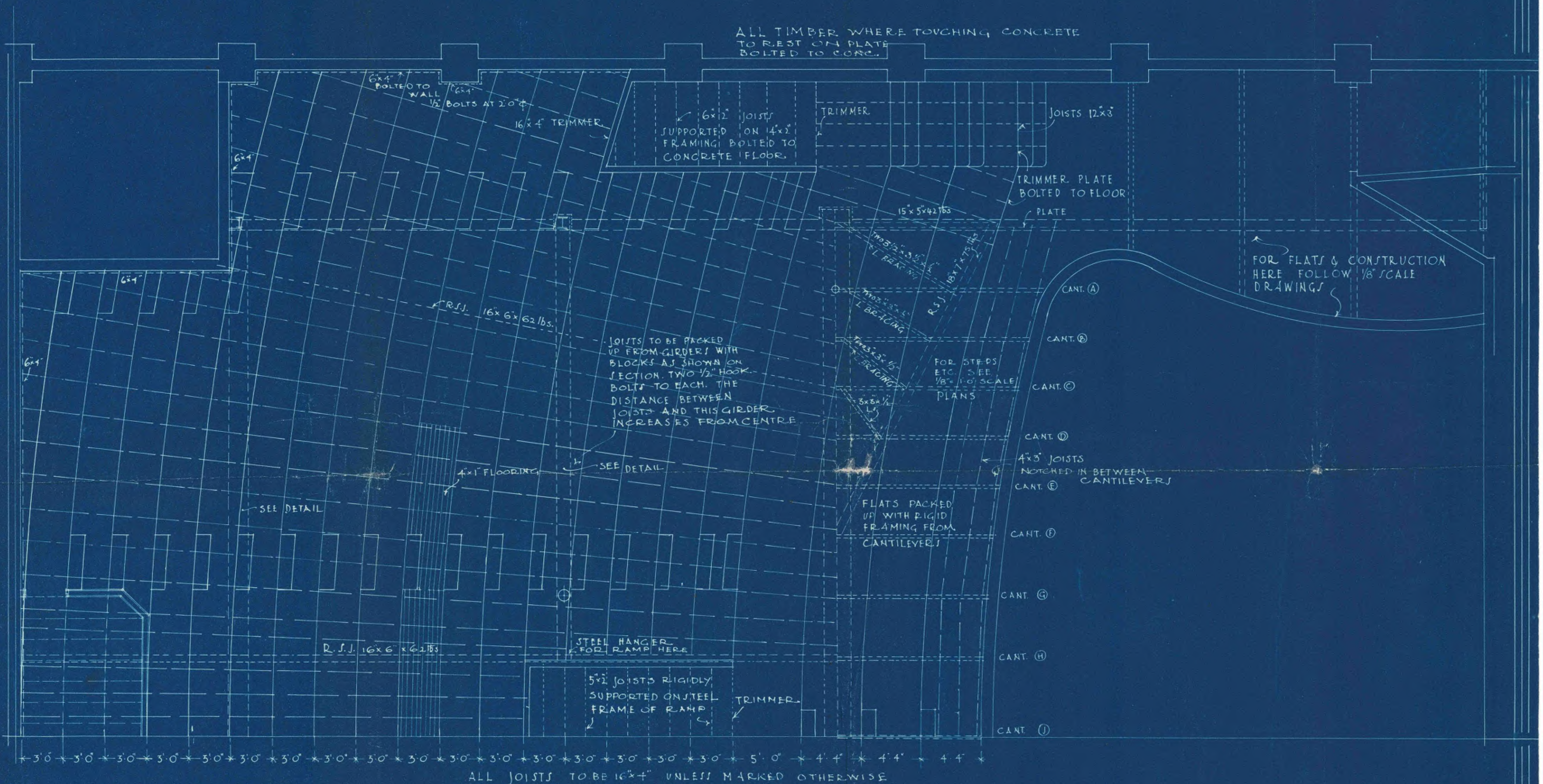
SHEET Nº
CIRCLE CONSn
SCALE • 1/4" = 1'-0"
DATE • 1 March 1923

•NEW THEATRE • COURTENAY PLACE • WELLINGTON•

FOR THE

DE LUXE THEATRE CO. LTD.

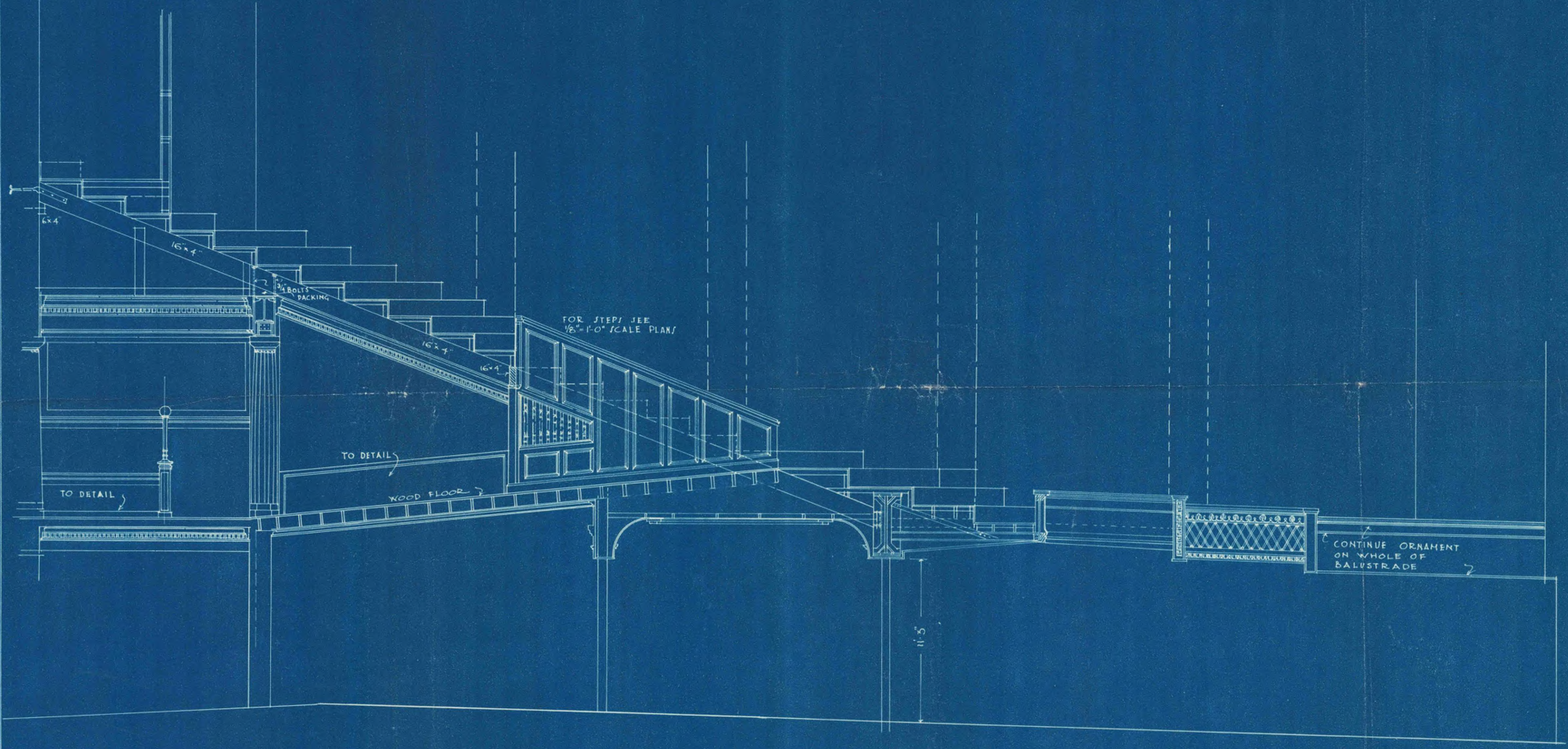
LLEWELLYN E. WILLIAMS •
A.R.I.B.A, M.I.S.E. LOND
ARCHITECT • STRUCTURAL
ENGINEER • WELLINGTON•



SHEET No 87
CIRCLE CONSN
SCALE 1/4 = 1'-0"
DATE. 1 MARCH 1923

NEW THEATRE · COURTENAY PLACE · WELLINGTON ·
FOR THE
DE LUXE THEATRE CO. LTD.

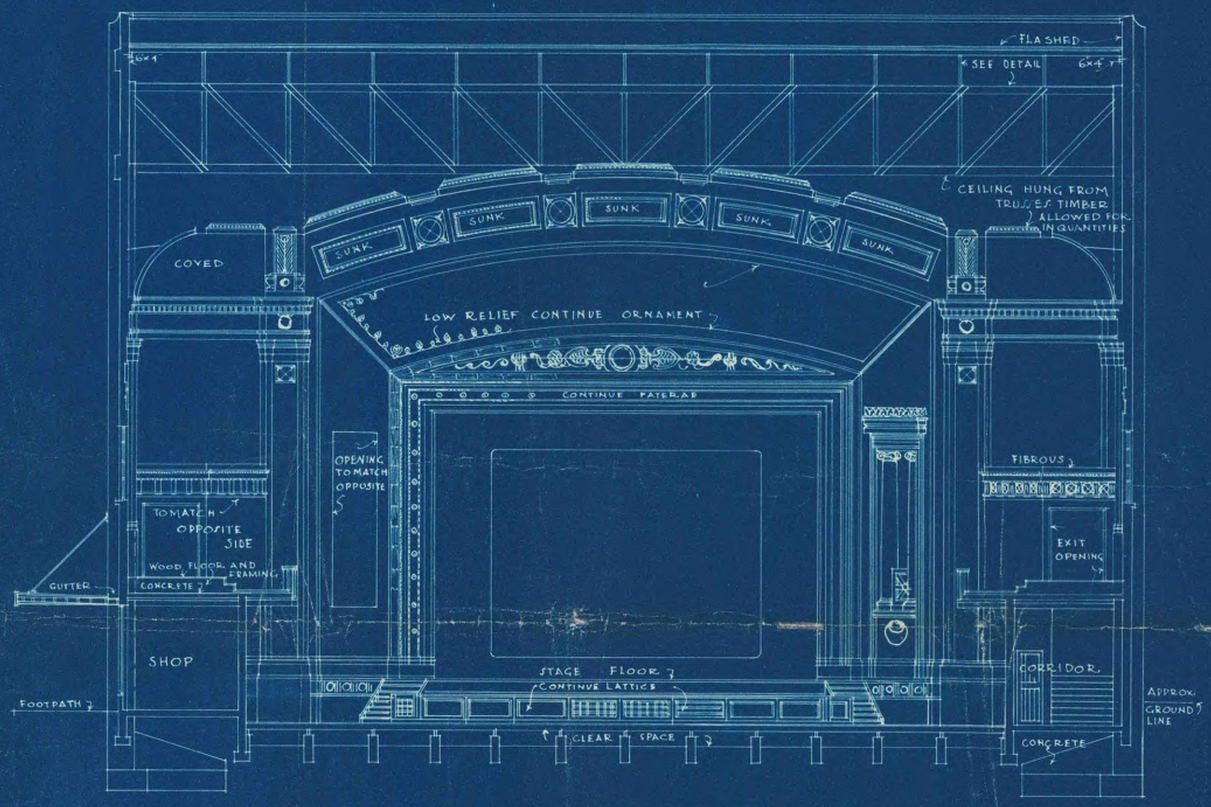
LLEWELLYN E. WILLIAMS ·
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT · STRUCTURAL
ENGINEER · WELLINGTON ·



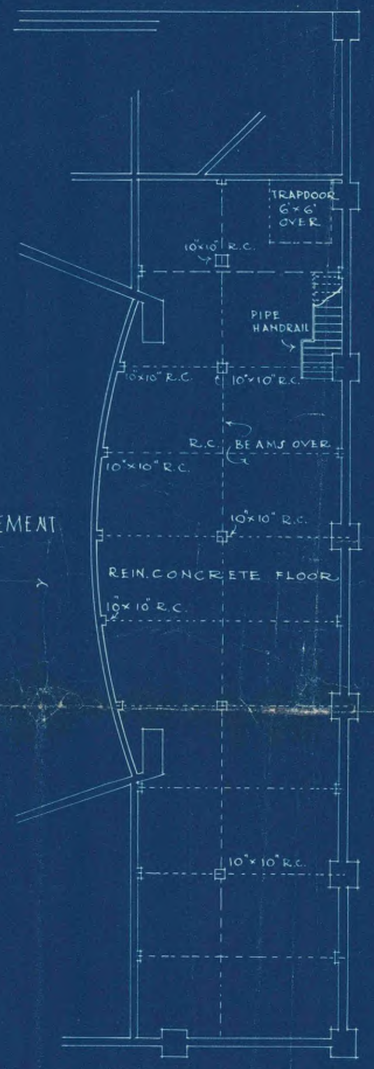
SHEET N° 8
CROSS SECTION
AND BASEMENT PLAN
SCALE ◦ 1/8" = 1'·0"
DATE ◦ 1 March 1923

◦NEW THEATRE ◦ COURTENAY PLACE ◦ WELLINGTON◦
FOR THE
DE LUXE THEATRE CO. LTD.

LLEWELLYN E. WILLIAMS ◦
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT ◦ STRVCTVRAL
ENGINEER ◦ WELLINGTON◦



SECTION ON C-D



PLAN OF BASEMENT UNDER STAGE

Shingles 7×2 Ht Ilans
Joists 6×2 Ht Ilans

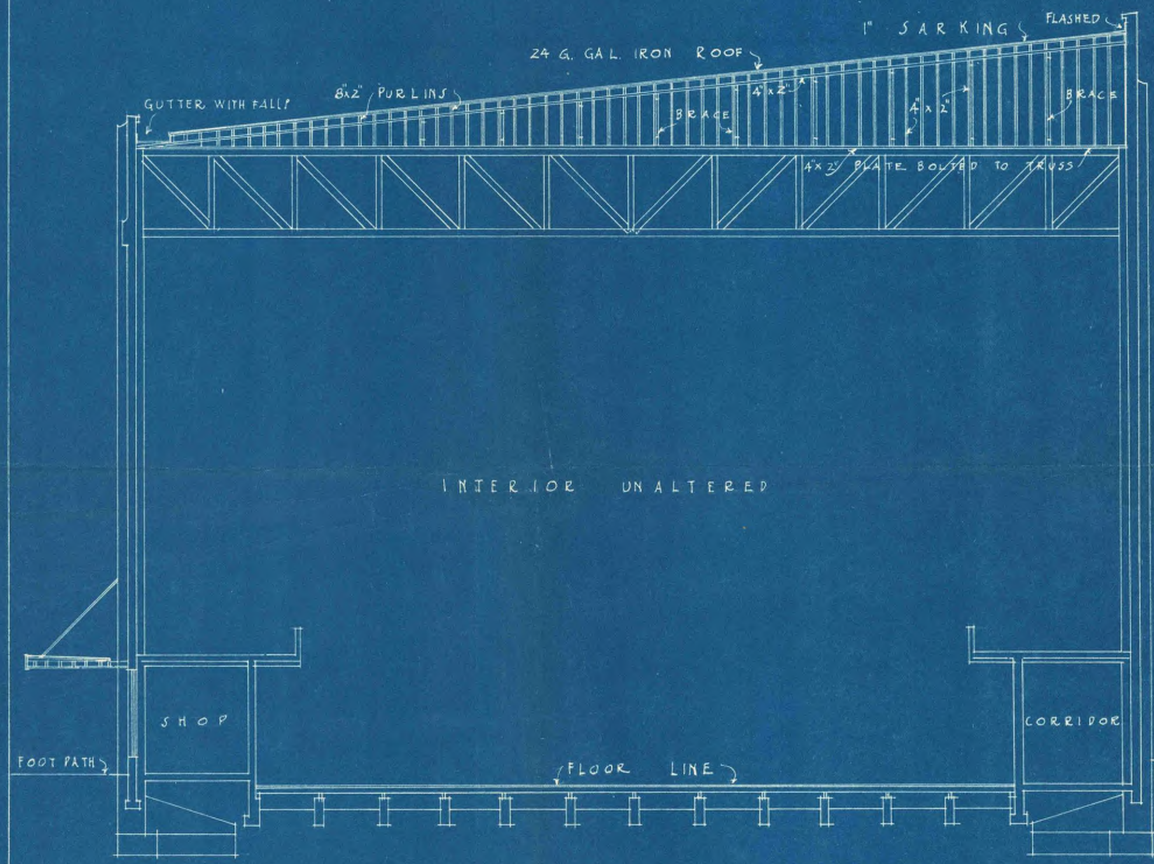
SHEET Nº 9A.
AMENDED PLAN
SCALE : 1/8" = 1'-0"
APRIL 1923

·NEW THEATRE · COURTENAY PLACE · WELLINGTON ·

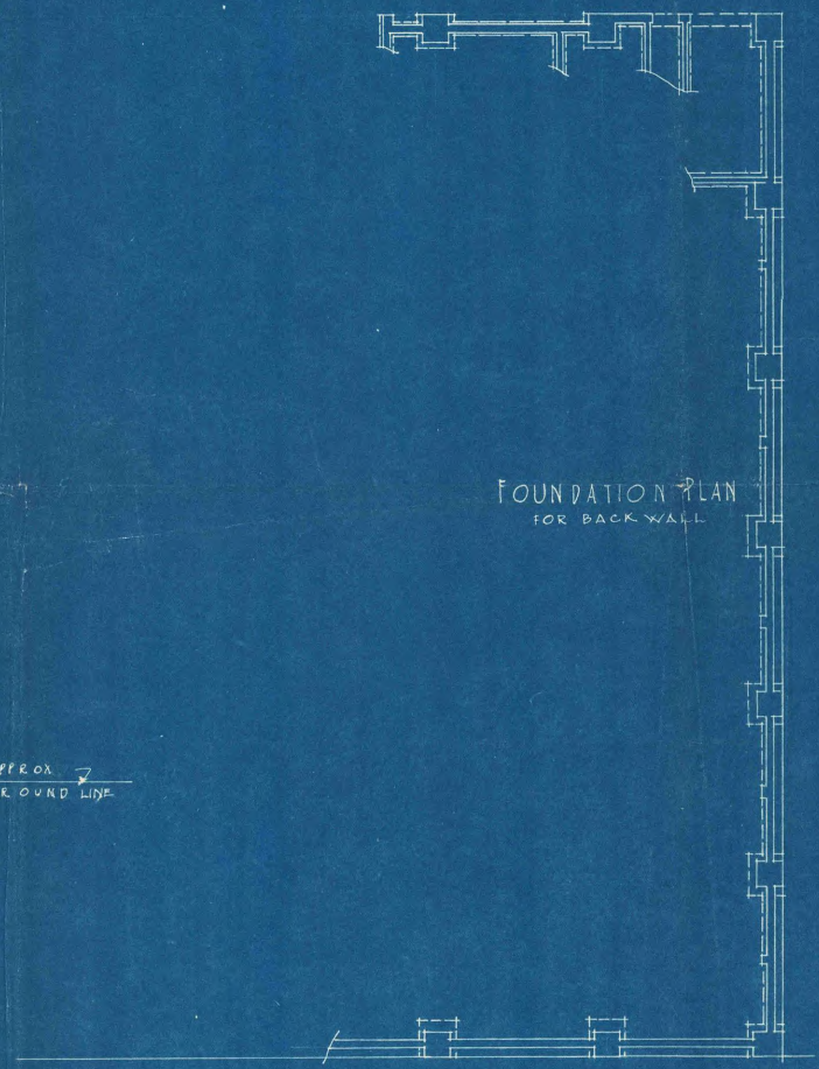
FOR THE

·DE LUXE THEATRE CO. LTD ·

LLEWELLYN E. WILLIAMS
A.R.I.B.A. M.I.S.E.
ARCHITECT & STRUCTURAL ENGR. W'GTON.



SECTION ON C·D



FOUNDATION PLAN
FOR BACK WALL

SHEET No
FRONT ELEVATION
SCALE. 1/8" = 1'-0"
DATE.

NEW THEATRE . COURTENAY PLACE . WELLINGTON.

FOR THE

DE LUXE THEATRE CO. LTD.

LLEWELLYN E. WILLIAMS.
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT. STRUCTURAL
ENGINEER. WELLINGTON.

Ms. Dolan

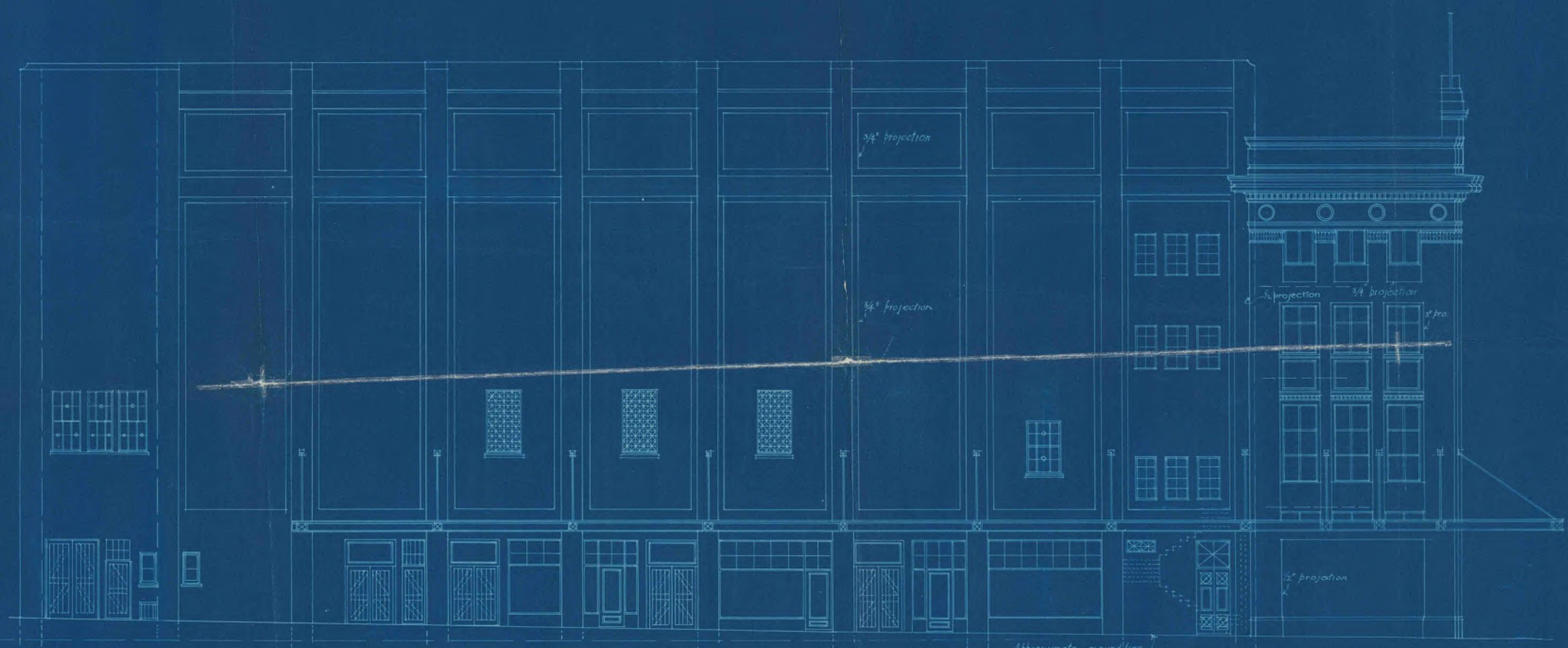
SHEET No 8
SIDE ELEVATION
SCALE · 1/8" = 1'-0"
DATE ·

· NEW THEATRE · COURTENAY PLACE · WELLINGTON ·

FOR THE

DE LUXE THEATRE CO. LTD.

LLEWELLYN E. WILLIAMS ·
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT · STRUCTURAL
ENGINEER · WELLINGTON ·



NOTE: for foundn see sheet No:

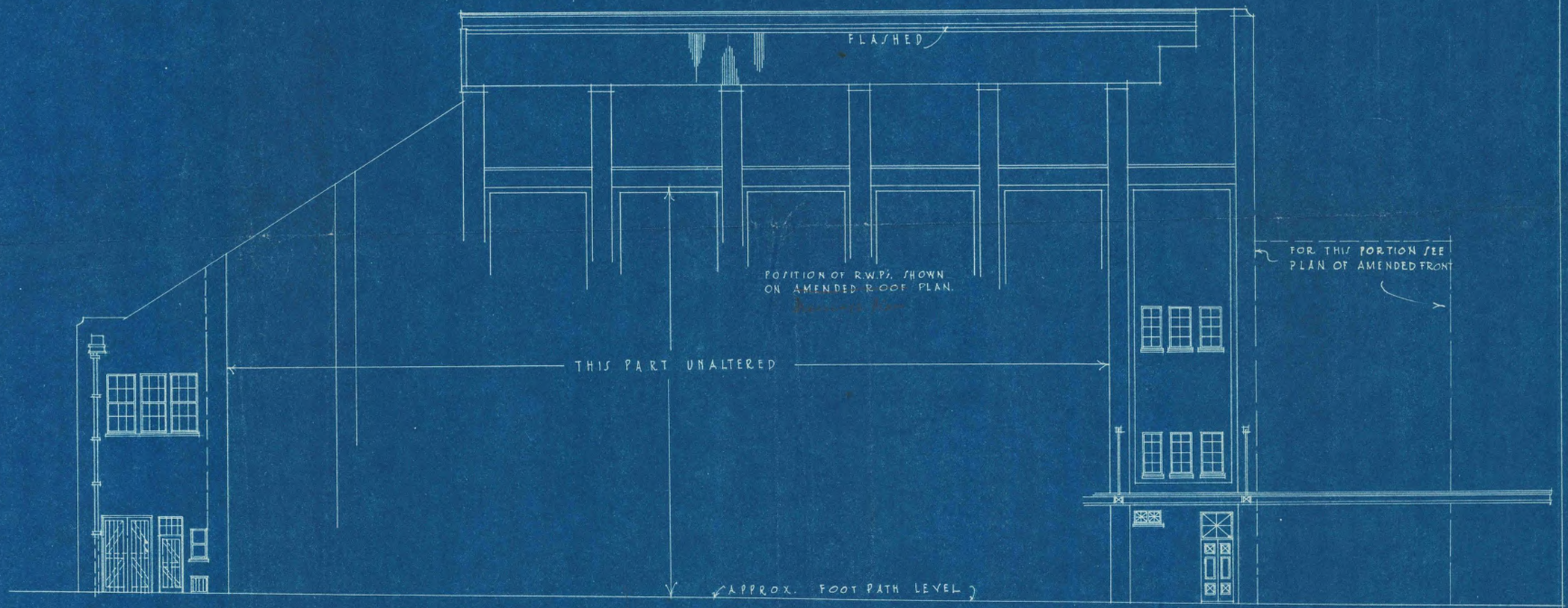
SHEET No 11A
AMENDED PLAN
SCALE: 1/8" = 1'-0"
APRIL 1923.

NEW THEATRE · COURTENAY PLACE · WELLINGTON ·

FOR THE

DE LUXE THEATRE CO. LTD.

LLEWELLYN E. WILLIAMS
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT & STRUCTURAL ENGR. W'GTON.



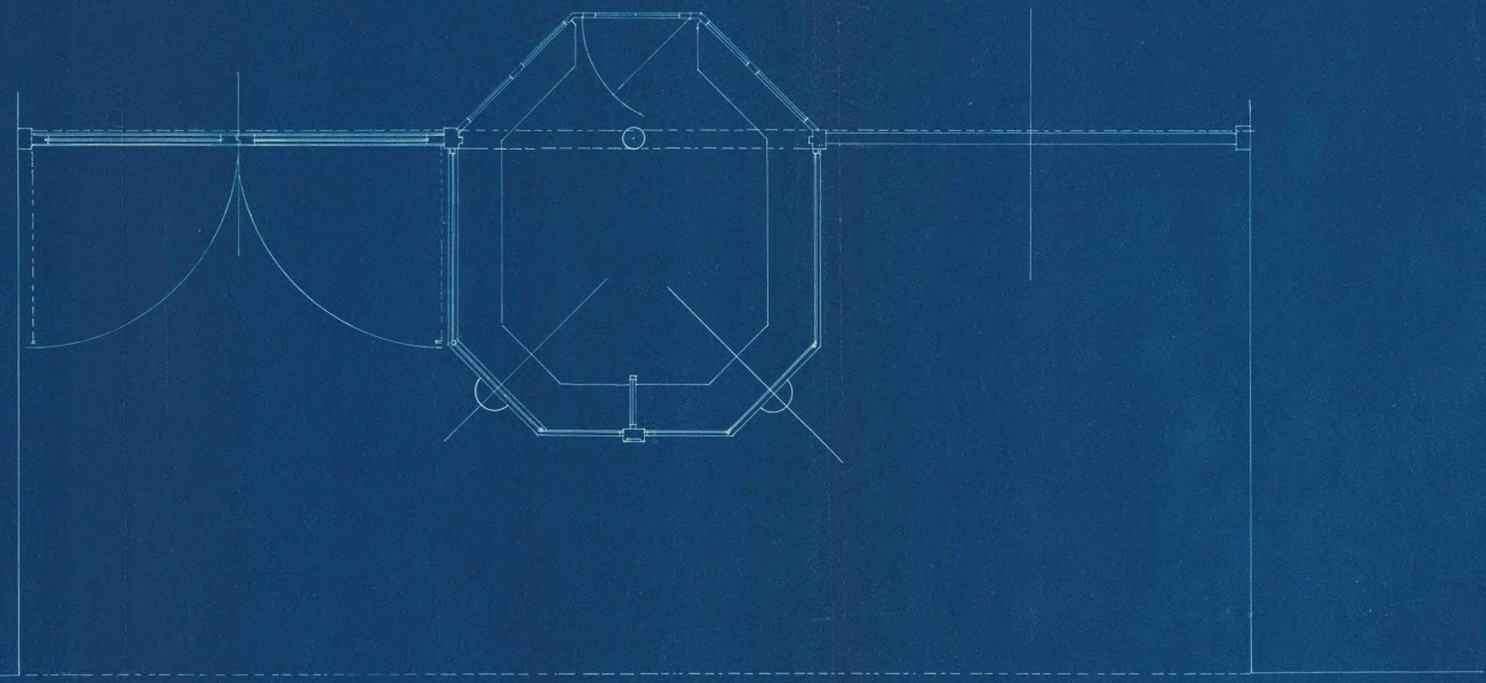
MAJORIBANKS STREET ELEVATION.

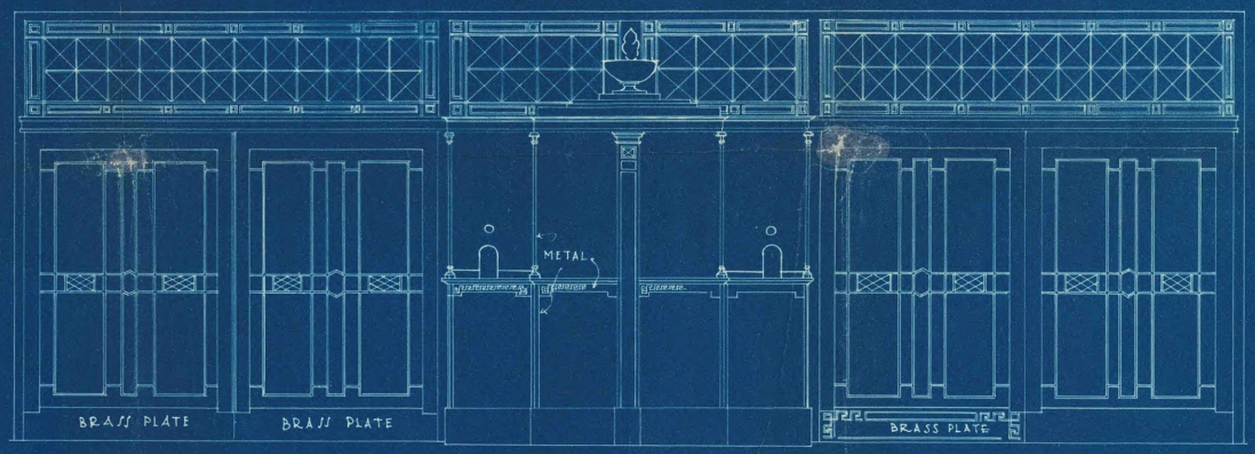
SHEET No
TICKET BOX & FRONT DOORS.
SCALE. 1/2" = 1'-0"
DATE. 1. March 1923

NEW THEATRE. COURTENAY PLACE. WELLINGTON.

FOR THE
DE LUXE THEATRE CO. LTD.

LLEWELLYN E. WILLIAMS.
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT. STRUCTURAL
ENGINEER. WELLINGTON.



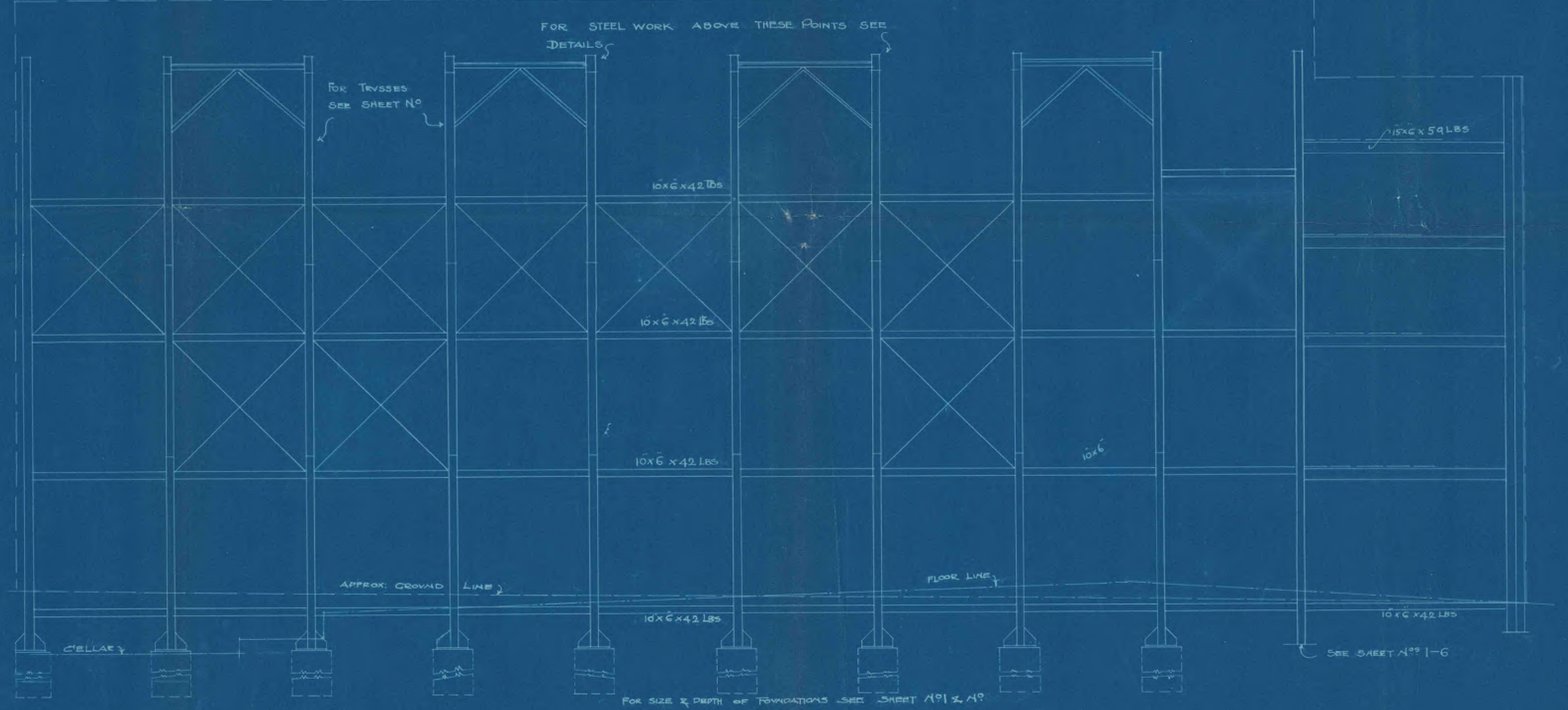


SHEET No 13
SIDE ELEVATION OF STEELWORK
SCALE. 1/8" = 1'-0

NEW THEATRE . COURTENAY PLACE . WELLINGTON.

FOR THE
DE LUXE THEATRE CO. LTD.

LLEWELLYN E. WILLIAMS.
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT . STRUCTURAL
ENGINEER . WELLINGTON.



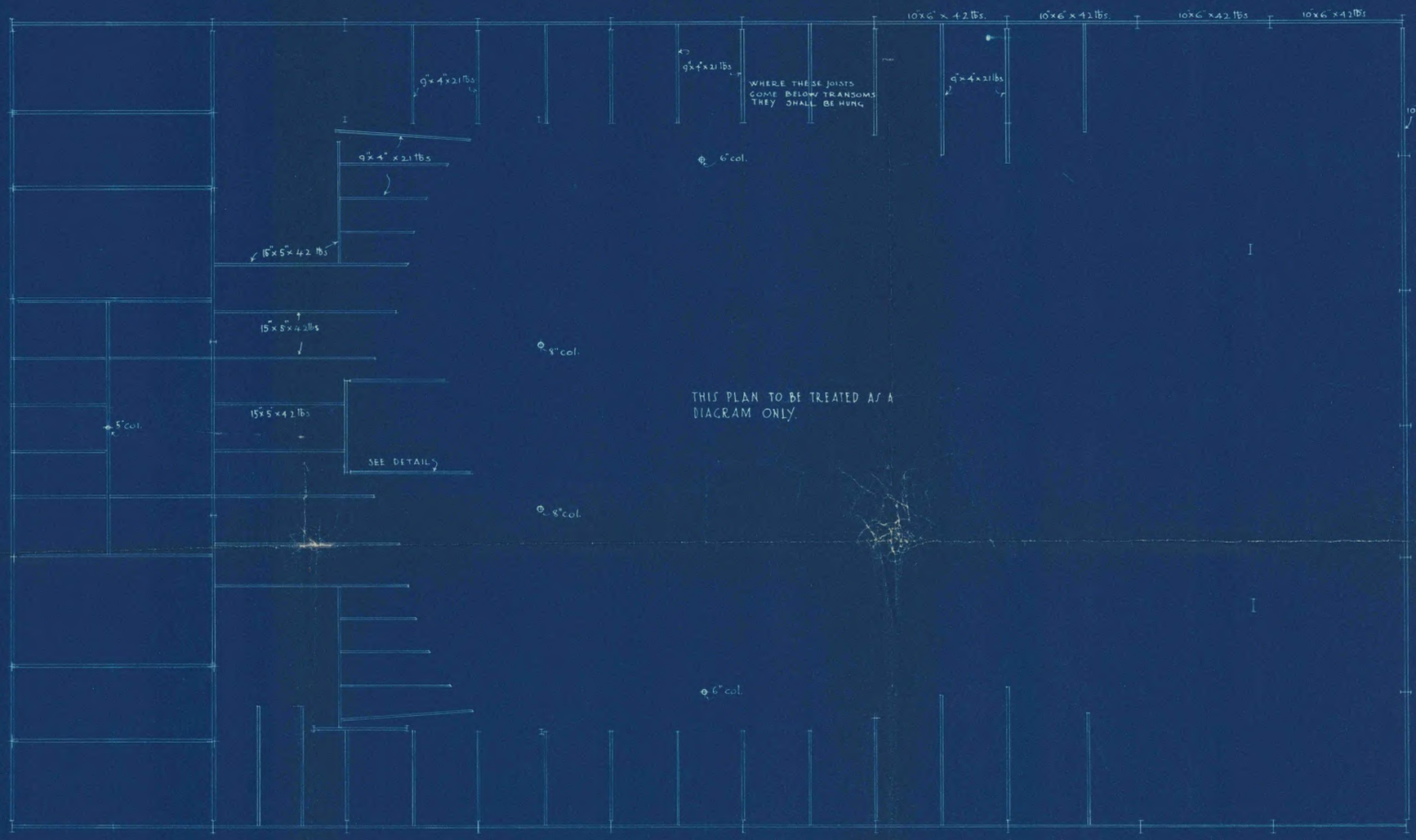
SHEET No 4
STEEL WORK PLAN
AT FOYER FLOOR LEVEL
SCALE. 1/8" = 1'-0"
DATE.

NEW THEATRE . COURTENAY PLACE . WELLINGTON.

FOR THE

DE LUXE THEATRE CO. LTD.

LLEWELLYN E. WILLIAMS.
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT. STRUCTURAL
ENGINEER. WELLINGTON.



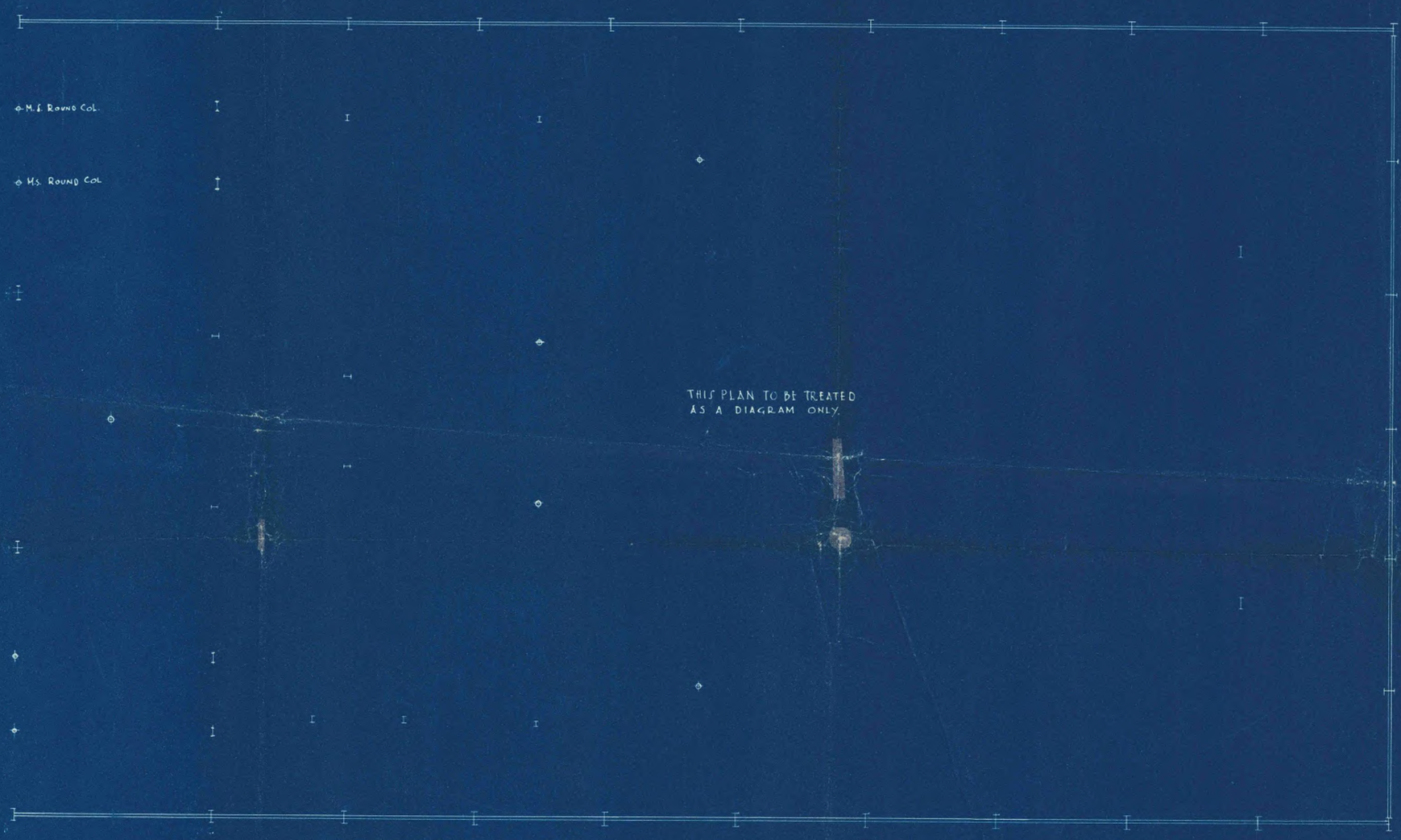
SHEET No 5
STEELWORK PLAN AT GROUND FLOOR LEVEL
SCALE. 1/8" = 1'-0"
DATE.

NEW THEATRE · COURTENAY PLACE · WELLINGTON

FOR THE

DE LUXE THEATRE CO. LTD.

LLEWELLYN E. WILLIAMS ·
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT · STRUCTURAL
ENGINEER · WELLINGTON ·

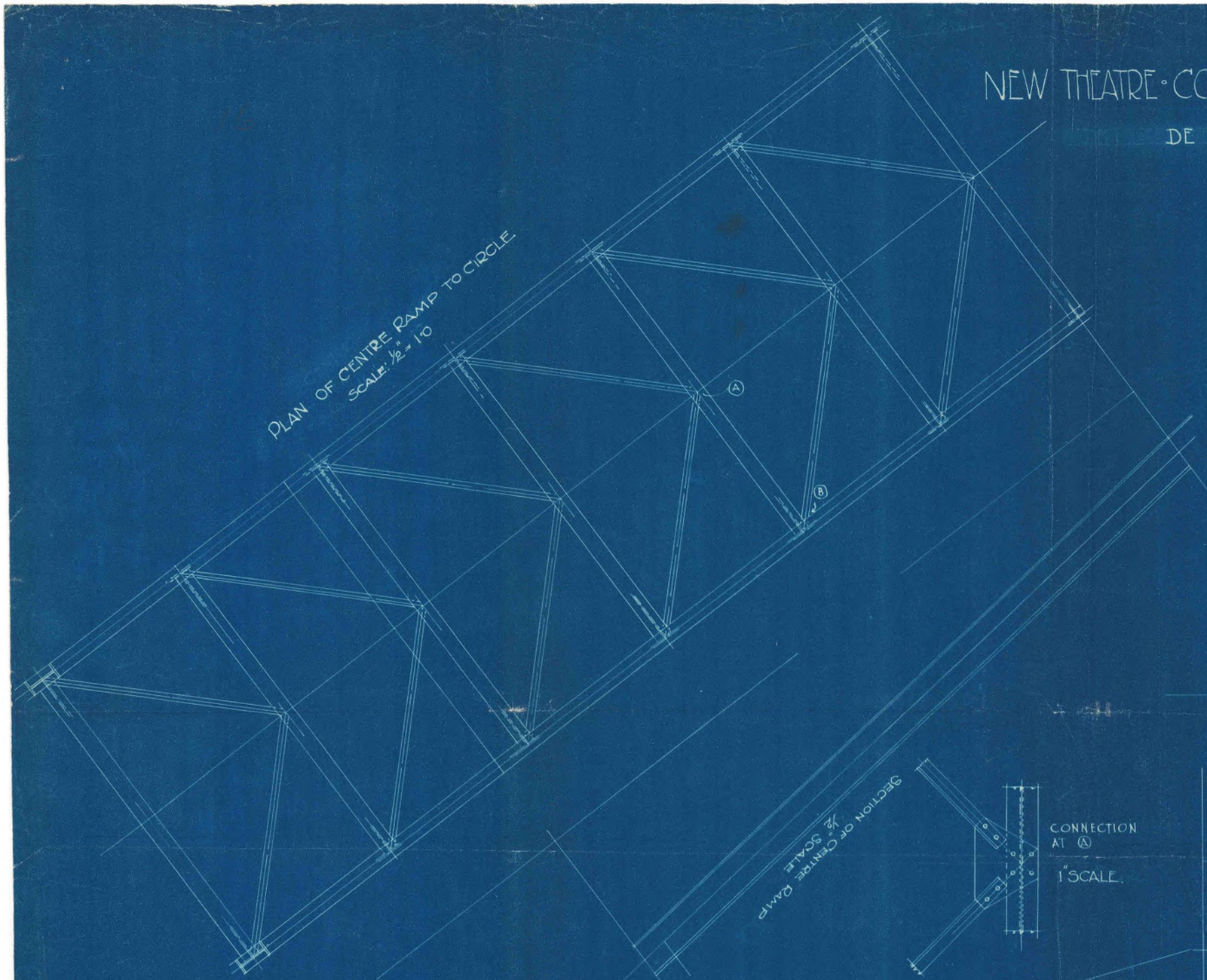


NEW THEATRE · COURTENAY PLACE · WELLINGTON ·

FOR THE

DE LUXE THEATRE CO. LTD.

PLAN OF CENTRE RAMP TO CIRCLE
SCALE: ½" = 1'0



SECTION OF CENTRE RAMP
½" SCALE

FOR HANGER SEE SHEET No

APPROX: GALLERY SLOPE

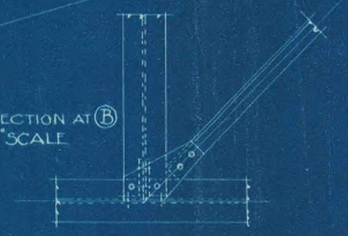
FOR FURTHER CONSTRUCTION HERE SEE ¼" PLAN OF CIRCLE

SEE DETAIL

MAIN GALLERY GIRDERS
¼" SCALE.

INTERMEDIATE GIRDER

CONNECTION AT Ⓐ
1" SCALE.



CONNECTION AT Ⓑ
1" SCALE

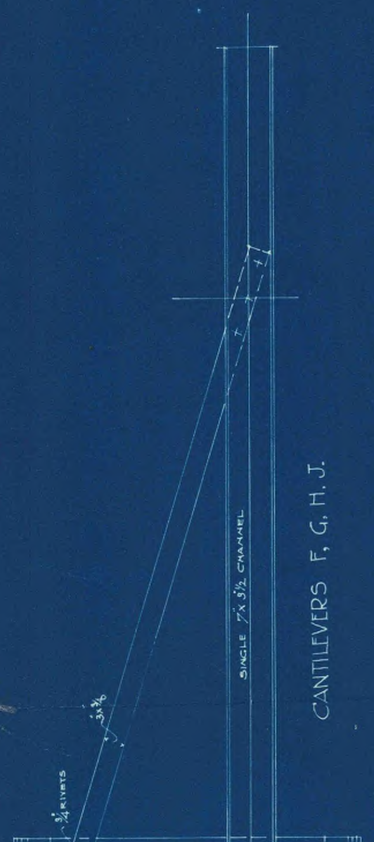
SECTION AT Ⓑ
1" SCALE.

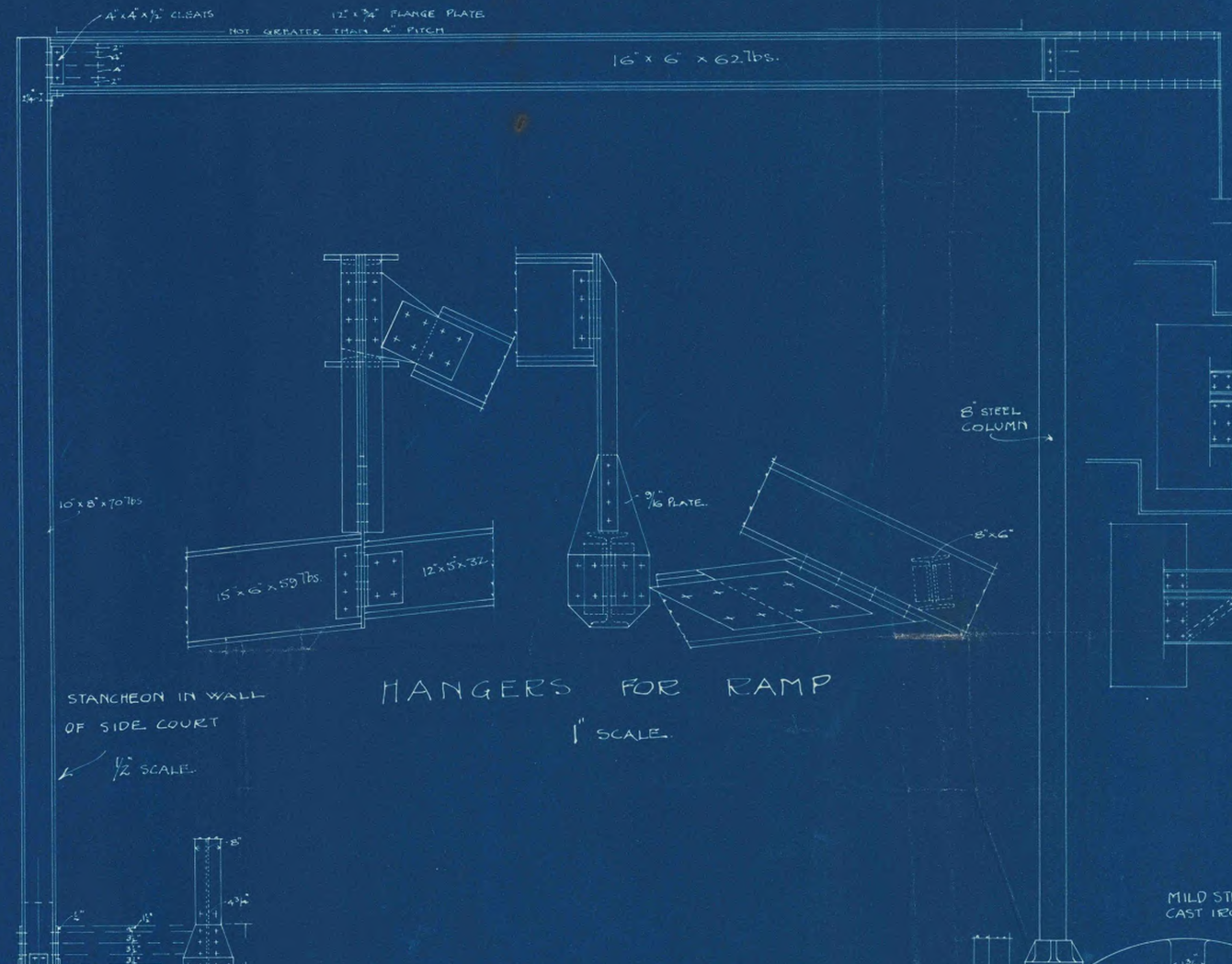
SHEET No
STEEL DETAILS
SCALE. AS NOTED

NEW THEATRE . COURTENAY PLACE . WELLINGTON.

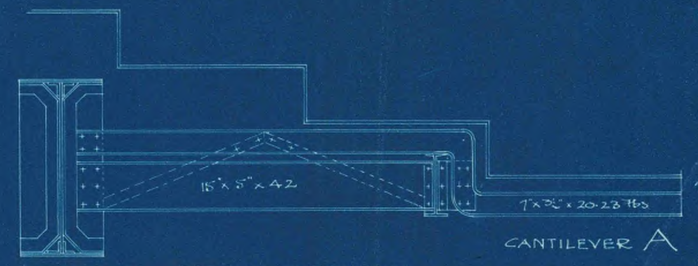
FOR THE
DE LUXE PICTURE THEATRE CO. LTD

LLEWELLYN E. WILLIAMS
A.R.I.B.A. M.I.S.E. Lond.
ARCHITECT . STRUCTURAL
ENGINEER . WELLINGTON.

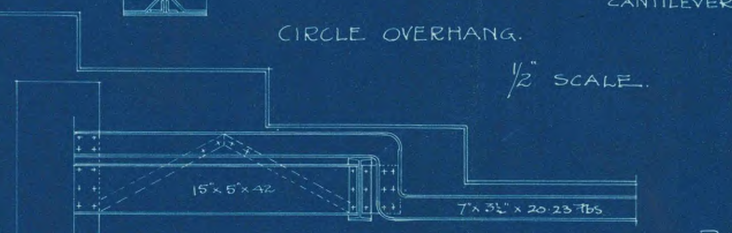




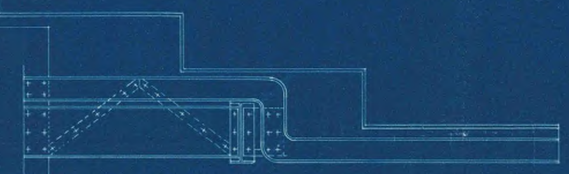
HANGERS FOR RAMP
1" SCALE.



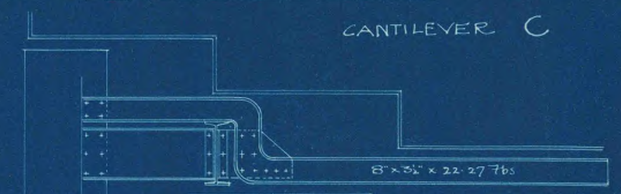
CANTILEVER A
CIRCLE OVERHANG.
½" SCALE.



CANTILEVER B



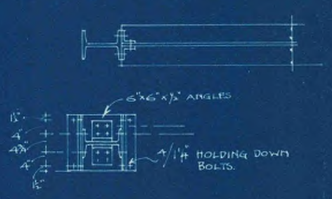
CANTILEVER C

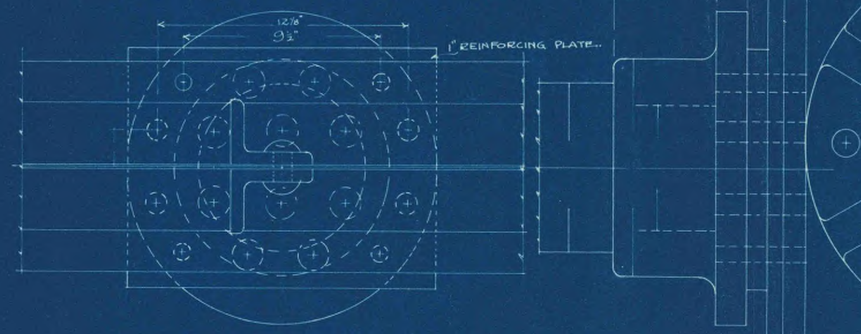


CANTILEVER D

CANTILEVER E

STANCHEON IN WALL
OF SIDE COURT
½" SCALE.





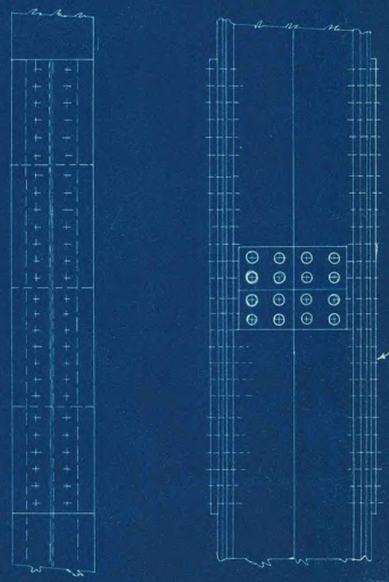
DETAILS OF CIRCLE STANCHEON 3" TO ONE FOOT.

SHEET No 16
STEELWORK DETAILS
SCALE · AS MARKED

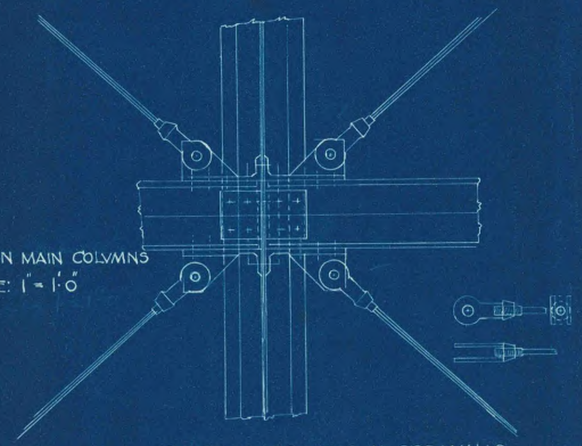
· NEW THEATRE · COURTENAY PLACE · WELLINGTON ·

FOR THE
DE LUXE THEATRE CO. LTD.

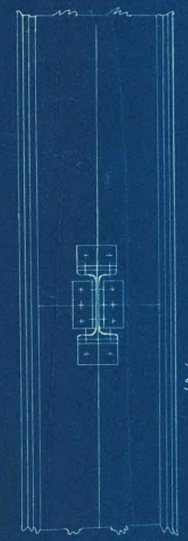
LLEWELLYN E. WILLIAMS
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT · STRUCTURAL
ENGINEER · WELLINGTON ·



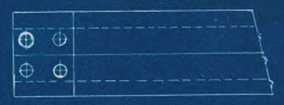
JOINTS IN MAIN COLVMNS
SCALE: 1" = 1·0"

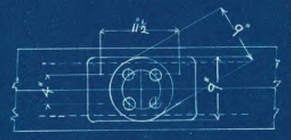


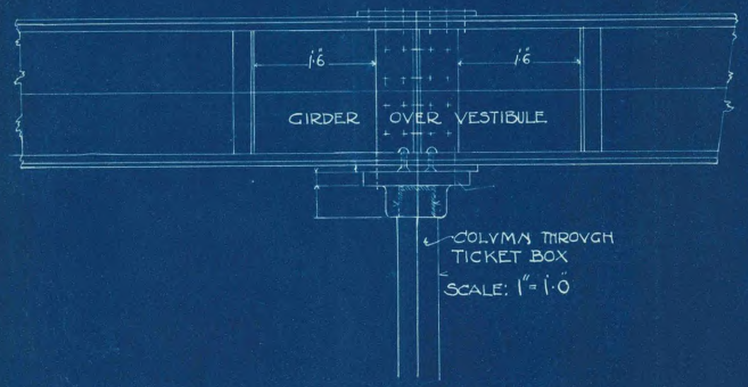
DIAGONAL BRACING IN SIDE WALLS
SCALE · 1" = 1·0"



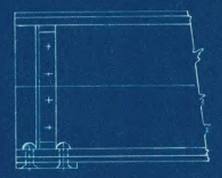
SECTION THROVGH
SIDE WALL TRANSOMES
SCALE · 1" = 1·0"







COLVMN THROVGH
TICKET BOX
SCALE: 1" = 1·0"

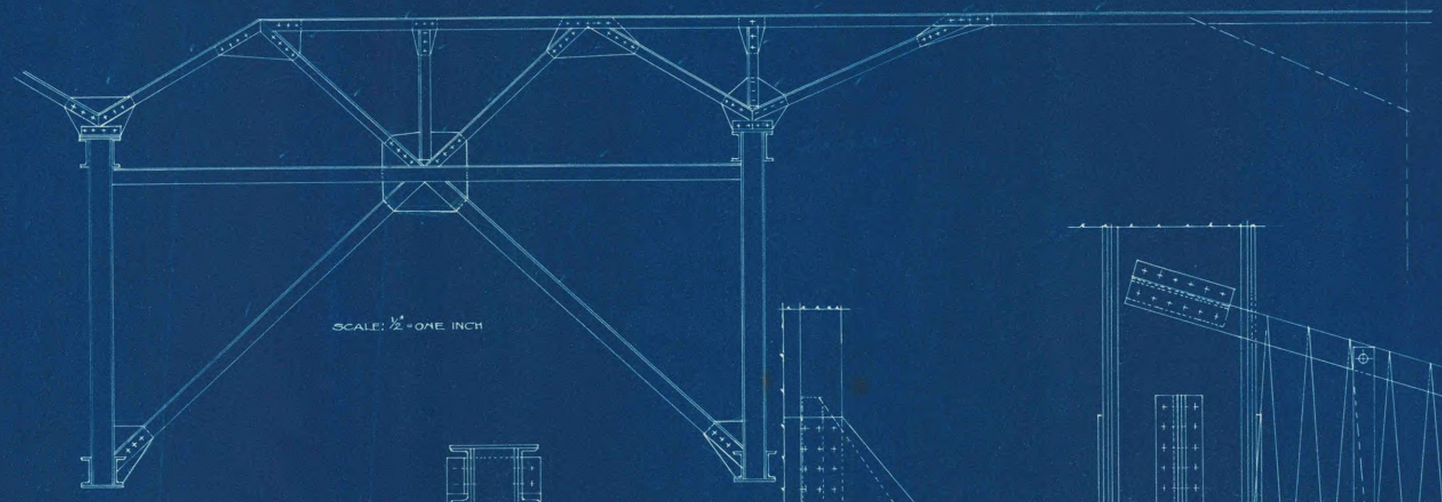


SHEET No 19
ROOF AND FOUNDATIONS TO MAIN STANCHIONS.
SCALE. AS MARKED.
DATE. 1 MARCH 1923

NEW THEATRE · COURTENAY PLACE · WELLINGTON.

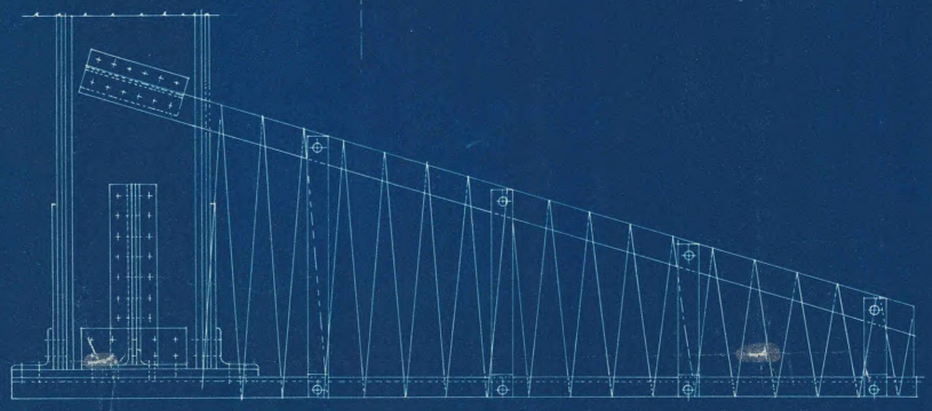
FOR THE DE LUXE THEATRE CO. LTD.

LLEWELLYN E. WILLIAMS.
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT. STRUCTURAL ENGINEER. WELLINGTON.



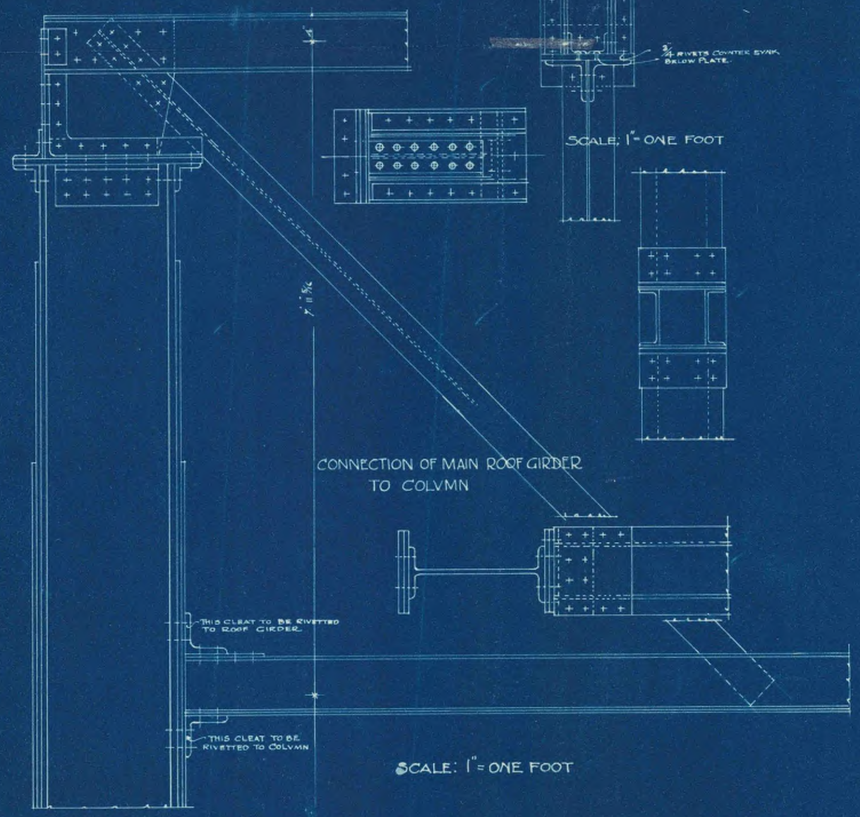
SCALE: ½" = ONE INCH

HALF END ELEVATION

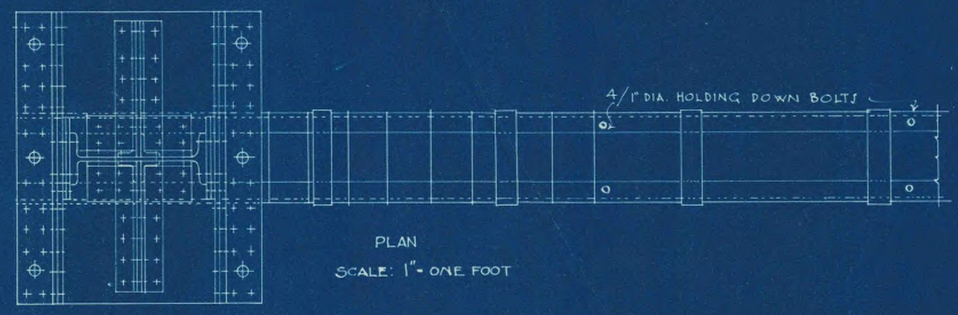


SIDE ELEVATION

SCALE: 1" = ONE FOOT



SCALE: 1" = ONE FOOT



PLAN

SCALE: 1" = ONE FOOT

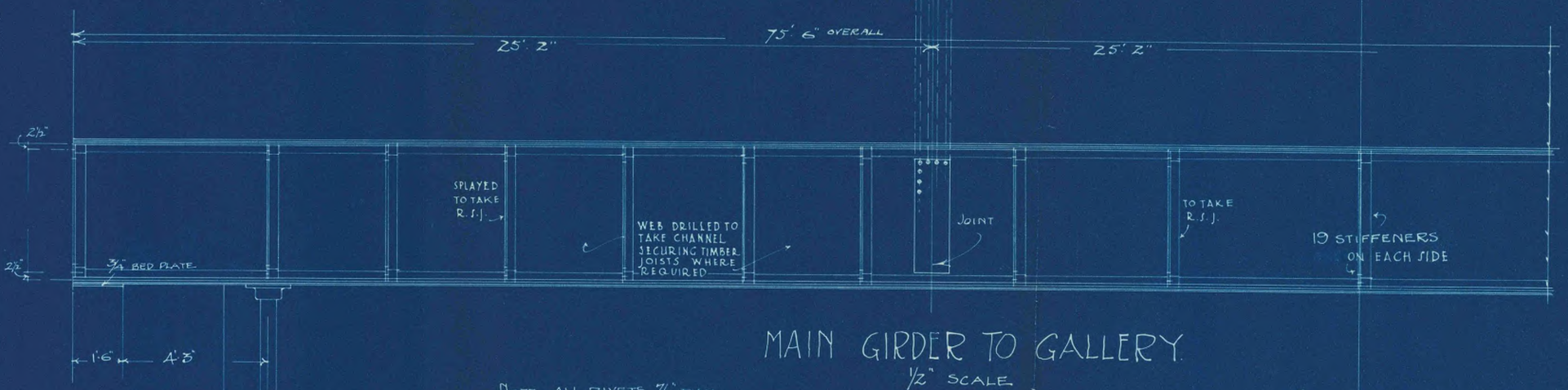
BASE AND CANTILEVER FOR MAIN COLVMNS

SHEET No 20
STEEL DETAILS
SCALES AS MARKED

◦NEW THEATRE◦ COURTENAY PLACE◦ WELLINGTON◦

FOR THE DE LUXE THEATRE CO LTD

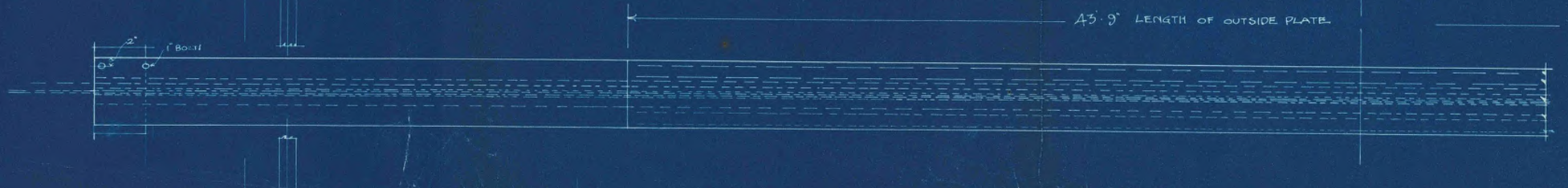
LLEWELLYN E. WILLIAMS
A.R.I.B.A. M.I.S.E. LOND
ARCHITECT◦STRUCTURAL
ENGINEER◦WELLINGTON◦

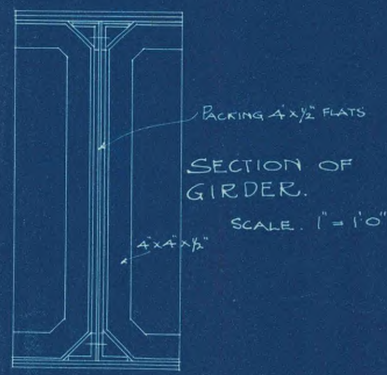


MAIN GIRDER TO GALLERY.

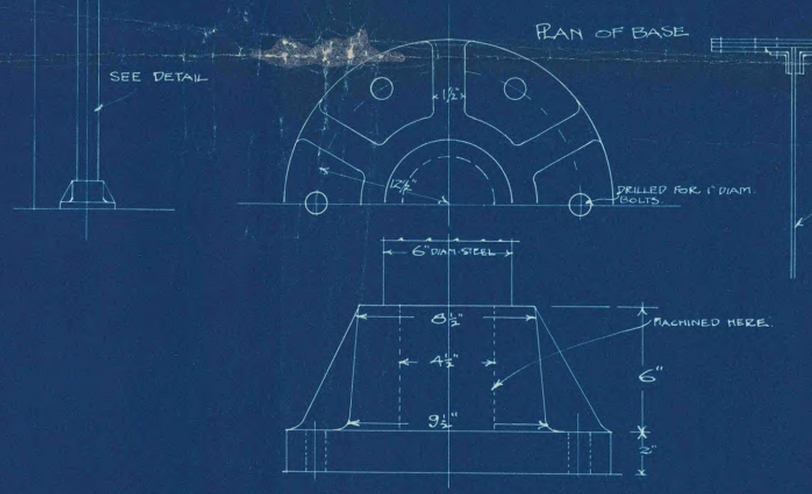
½" SCALE

NOTE. ALL RIVETS ⅞" DIAM.

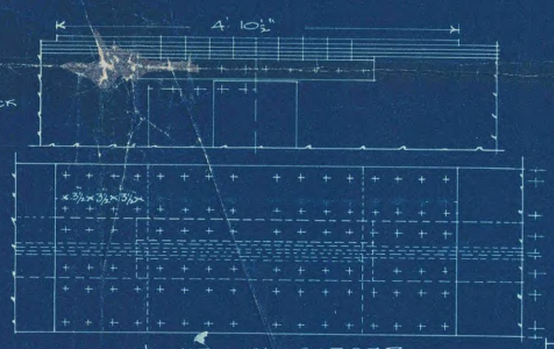




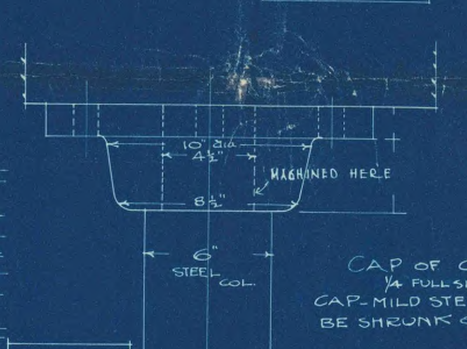
SECTION OF GIRDER.
SCALE. 1" = 1'0"



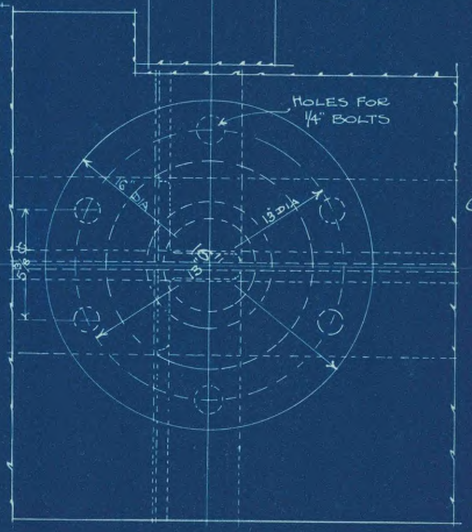
BASE OF CAP
¼ FULL SIZE



JOINT IN GIRDER



CAP OF COL.
¼ FULL SIZE.
CAP - MILD STEEL - TO BE SHRUNK ON.



PLAN OF CAP.
¼ FULL SIZE.

MAIN GALLERY GIRDERS & SUPPORTING COLUMNS.

NEW THEATRE COURTENAY PLACE WELLINGTON

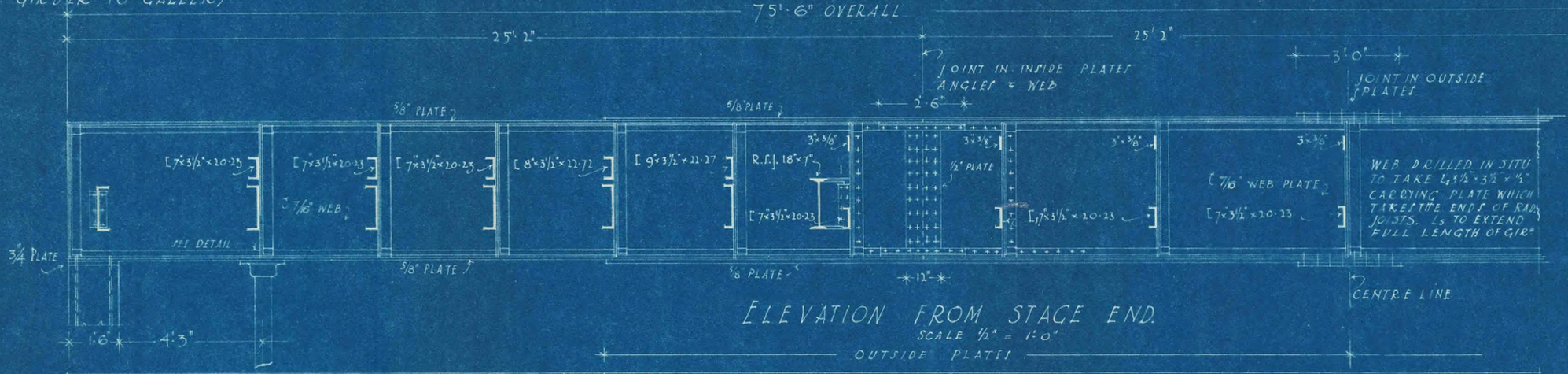
FOR

THE DE LUXE THEATRE CO. LTD.

SHEET Nº 204.

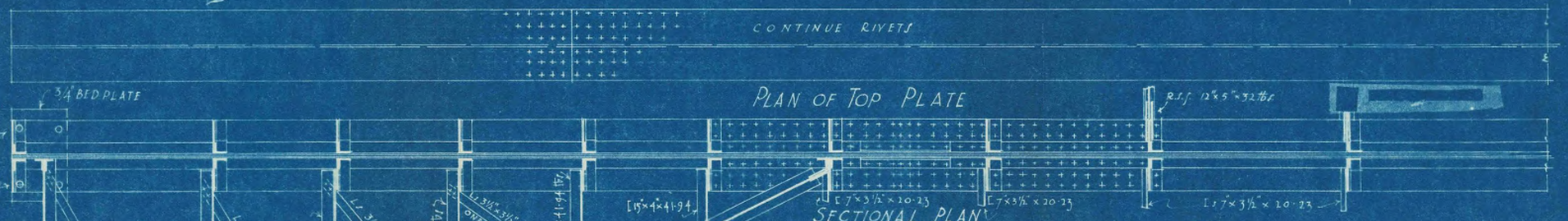
DETAILS OF MAIN GIRDER TO GALLERY.

LLEWELLYN E. WILLIAMS.
A.R.I.B.A. M.I.S.E. LOND.
ARCHITECT & STRUCTURAL ENGINEER · WELLINGTON.



ELEVATION FROM STAGE END.

SCALE ½" = 1'-0"



PLAN OF TOP PLATE

SECTIONAL PLAN

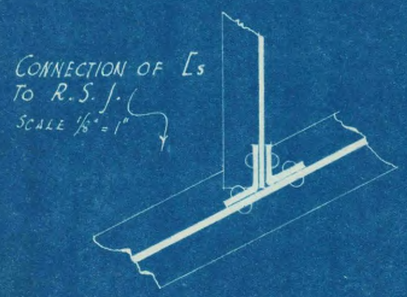
SCALE ½" - 1'-0"

ELEVATION & SECTION OF JOINT IN WEB, ANGLES & INSIDE PLATES.

SCALE: 1 INCH = 1'-0"

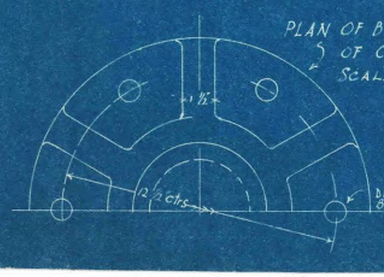
SECTIONAL PLAN OF CONNECTION BETWEEN SPLAYED GIRDER & MAIN GIRDER.

SCALE ⅛" = 1"



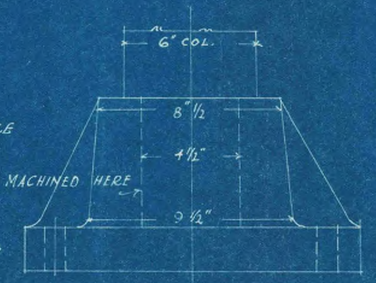
CONNECTION OF Ls TO R.S.J.

SCALE ⅛" = 1"



PLAN OF BASE OF COL.

SCALE ¼ FULL SIZE

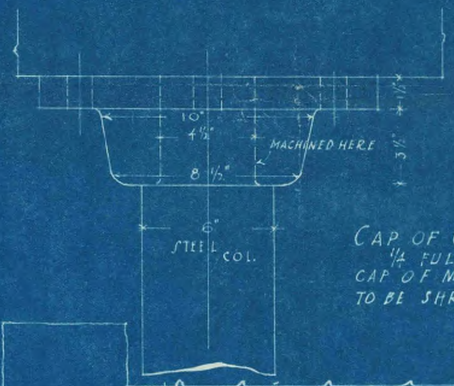


6" BASE OF COL. CAST IRON

SCALE ¼ FULL SIZE

SECTION THRO' MAIN GIRDER

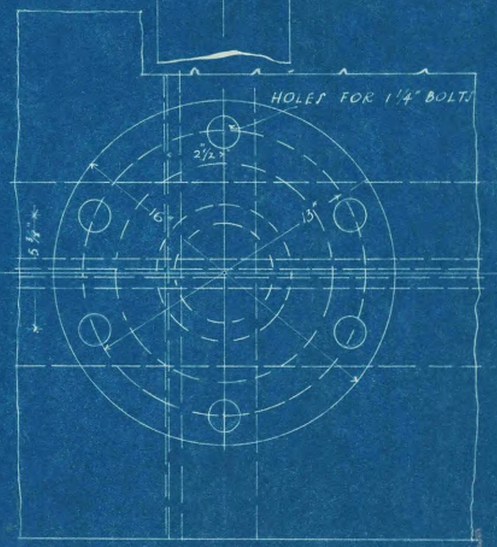
SCALE 1"=1'-0"



CAP OF COLUMN

¼ FULL SIZE

CAP OF MILD STEEL TO BE SHRUNK ON

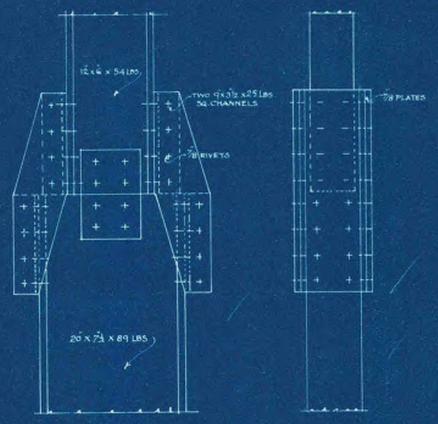


SHEET Nº
SHOP AND OFFICE
COLVMNS
SCALE: 1" = 1·0
DATE · 1 March 1923

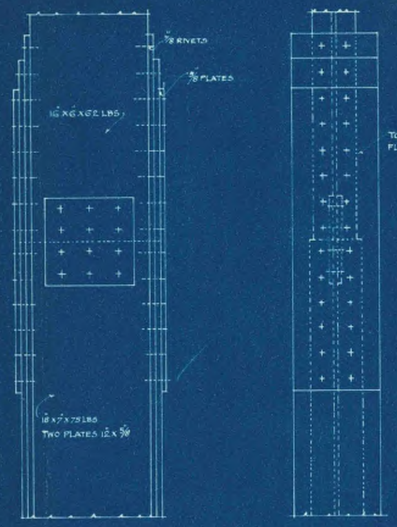
NEW THEATRE · COURTENAY PLACE · WELLINGTON

FOR THE
DE LUXE THEATRE CO. LTD.

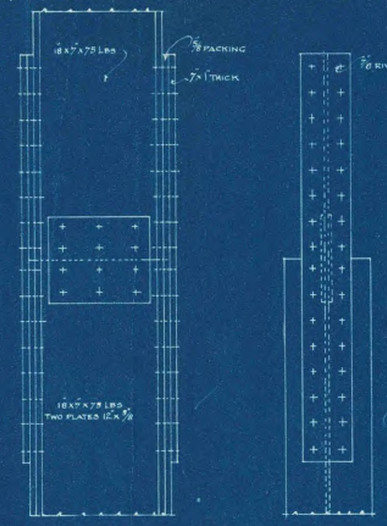
LLEWELLYN E. WILLIAMS
A.R.I.B.A. M.I.S.E. LOND.
ARCHITECT · STRUCTURAL
ENGINEER · WELLINGTON



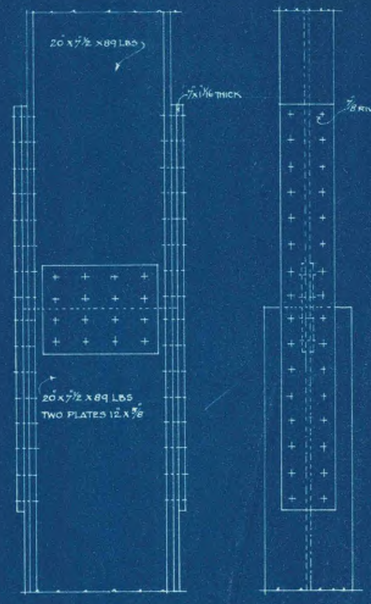
JVNCTION 2C AND 2D.



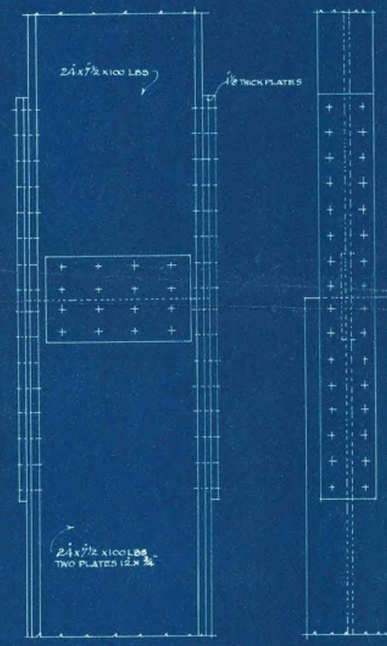
JVNCTION FOR 3C AND 3D TO 5C TO 5D



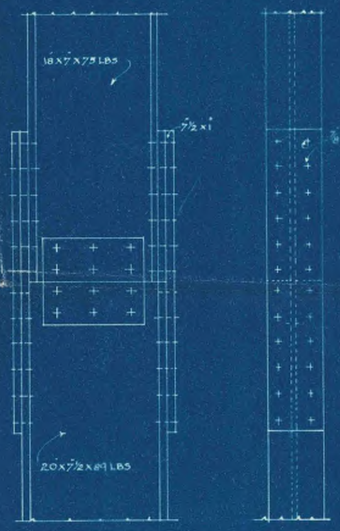
JVNCTION FOR 4C TO 4D, 6C TO 6D, 7C TO 7D



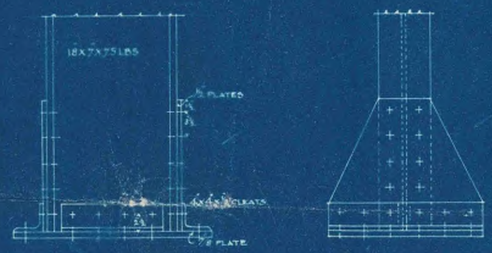
JVNCTION FOR 8C TO 8D



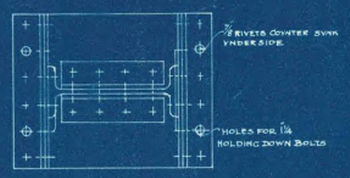
JVNCTION FOR 9C TO 9D

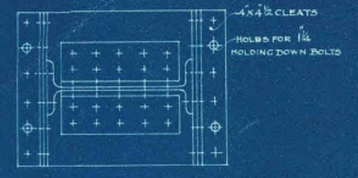


JVNCTION FOR 10D TO 10E

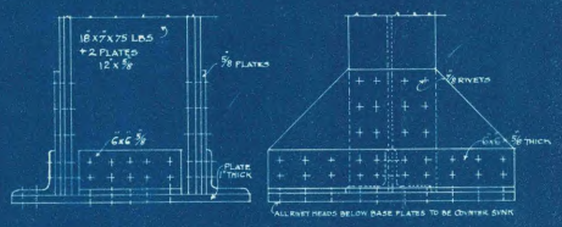


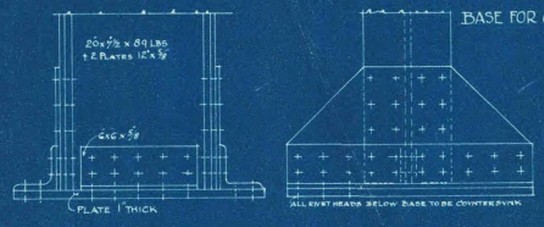
BASE FOR COLVMN 1ABCDE.

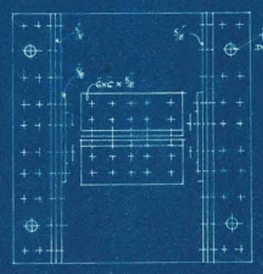




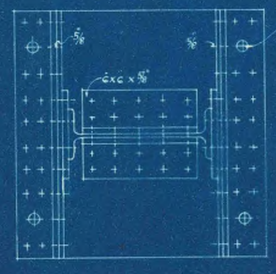
BASE FOR COLVMN 2ABCDE AND 10ABCDE







BASE FOR COLVMNS
3ABCDE
4ABCDE
5ABCDE
6ABCDE
7ABCDE



BASE FOR COLVMNS
8 ABCDE

IT IS PROPOSED TO OBTAIN COLVMNS FOR FIRST TWO TIERS IN SINGLE LENGTHS.
R.S. JOISTS CAN BE OBTAINED VP TO 40 FEET IN LENGTH.
SINGLE MEMBERS 1D 1E 2D 2E 3D 3E 4D 4E 5D 5E 6D 6E 7D 7E 8D 8E
9D 9E 10D 10E

SCALE: 1" = ONE FOOT

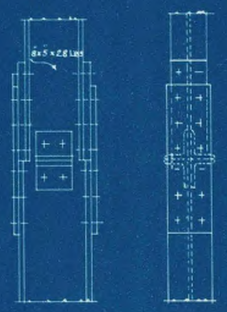
SHEET No 23
SHOP & OFFICE COLS.
SCALE - 1" = 1'-0"
DATE - 1 March 1923

NEW THEATRE · COURTENAY PLACE · WELLINGTON ·

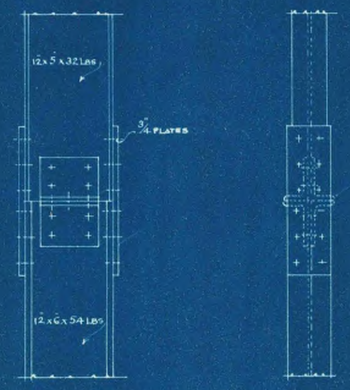
FOR THE

DE LUXE THEATRE CO. LTD.

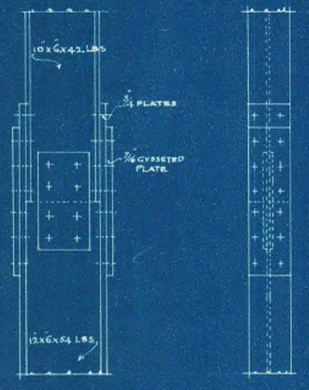
LLEWELLYN E. WILLIAMS ·
A.R.I.B.A., M.I.S.E. LOND.
ARCHITECT · STRUCTURAL
ENGINEER · WELLINGTON ·



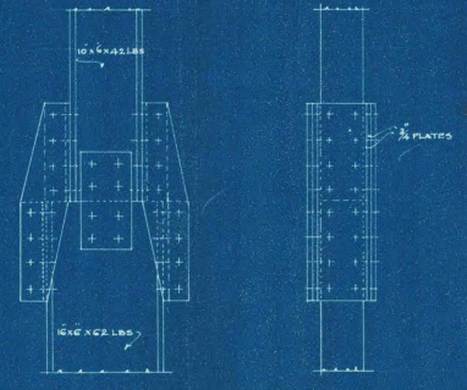
JVNCTIONS FOR 1A TO 1B, 2A TO 2B.
3A TO 3B, 4A TO 4B, 5A TO 5B, 6A TO 6B
7A TO 7B. 10A TO 10B.



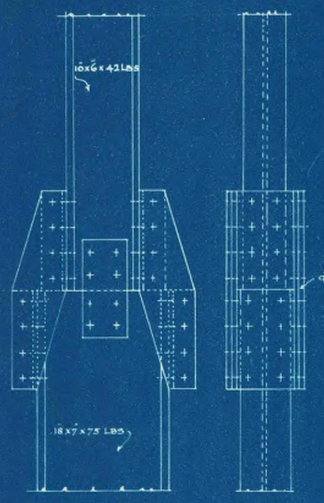
JVNCTIONS FOR 8A TO 8B AND 9A TO 9·6



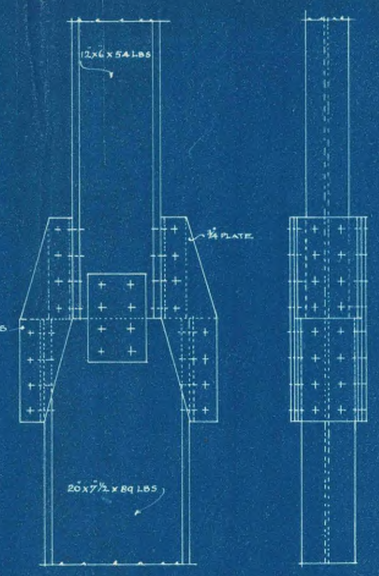
JVNCTIONS FOR 1B TO 1C, 2B TO 2C
10B TO 10C



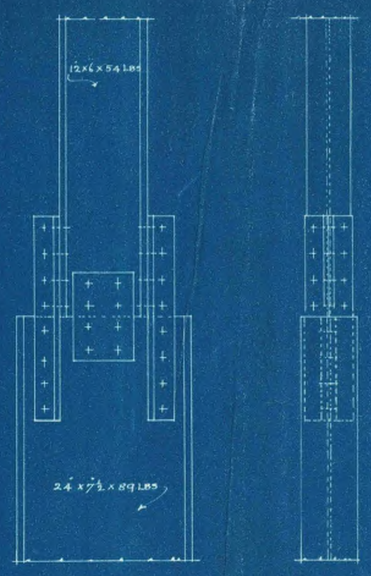
JVNCTIONS FOR 3B TO 3C AND 5B TO 5C.



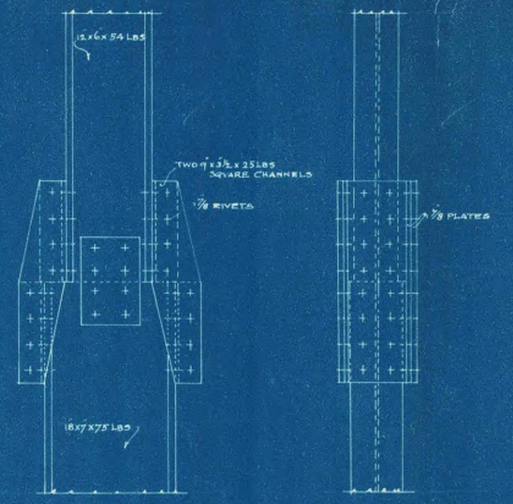
JVNCTIONS FOR 4B TO 4C, 6B TO 6C, 7B TO 7C



JVNCTIONS FOR 8B TO 8C.



JVNCTIONS FOR 9B TO 9C



JVNCTIONS FOR 1C TO 1D. AND 10C TO 10D

SCALE: 1" = ONE FOOT.

SHEET Nº 23.
DETAILS OF STEEL BETWEEN VESTIBULE & AUDITORIUM.
SCALE: 1" = 1'-0"

NEW THEATRE COURTENAY PLACE WELLINGTON FOR THE DE LUXE THEATRE CO LTD.

LLEWELLYN E. WILLIAMS
A.R.I.B.A. M.I.S.E.
ARCHITECT & STRUCTURAL ENGR. W'GTON.

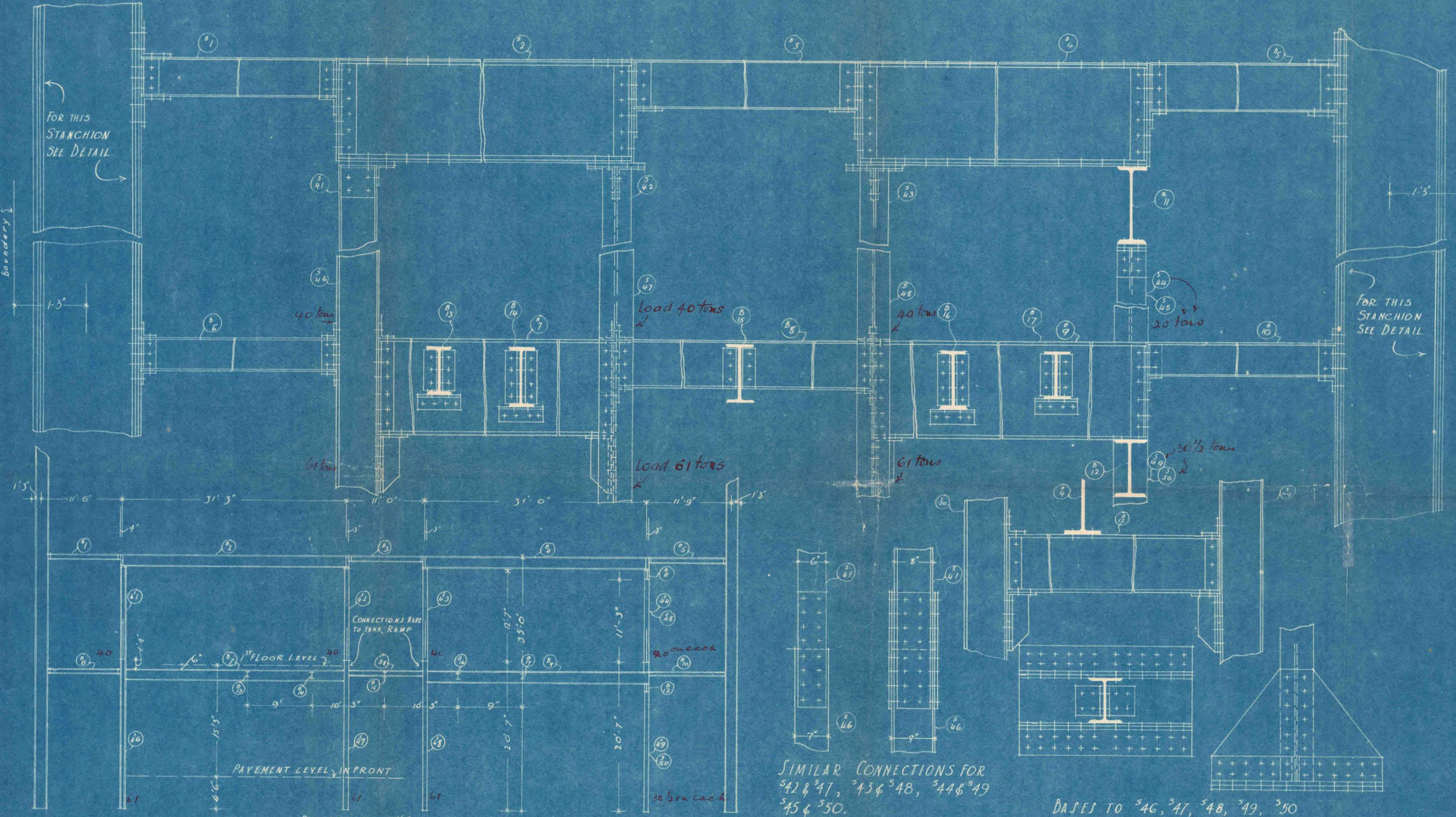


DIAGRAM TO 1/8" SCALE

STANCHIONS			
Nº	Length	Size	Wgt.
S41	12'-7"	8×6	35
S42	12'-7"	9×6	35
S43	12'-7"	8×6	35
S44	11'-3"	8×6	35
S45	11'-3"	8×6	35
S46	20-7"	9×7	58
S47	20'-7"	9×7	58
S48	20'-7"	9×7	58
S49	20'-7"	9×7	58
S50	20'-7"	9×7	58

BEAMS						
Nº	Length	Size	Wgt	Plates	Length	Rivets
B1	10'-2"	8×5	28			4 1/2" PITCH 7/8" DIA.
B2	31'-10"	20×7 1/2	89	2/12×5/8	31'-10"	
B3	10'-4 1/2	10×5	35			
B4	31'-7"	20×7 1/2	89	2/12×5/8	31'-7"	
B5	10'-5 1/4	10×5	35			
B6	10'-1 3/4	7×4	16			
B7	30'-9 3/4	20×7 1/2	89			
B8	10'-11"	10×5	30			
B9	31'-2" 1/2	20×7 1/2	89			
B10	10'-5 3/4	7×4	16			
B11	11'-6	16×6	62			
B12	10'-3 1/2	12×6	42			

CONNECTIONS AT STANCHION JOINTS										
	Nº	S41 S46	Nº	S42 S47	Nº	S43 S48	Nº	S44 S49	Nº	S45 S50
Plates	2	2'-0"×7"×5/8"	2	2'-0"×7"×5/8"	2	2'-0"×7"×5/8"	2	2'-0"×7"×5/8"	2	2'-0"×7"×5/8"
"	2	2'-0"×6"×1/2"	2	2'-0"×6"×1/2"	2	2'-0"×6"×1/2"	2	2'-0"×6"×1/2"	2	2'-0"×6"×1/2"
"	2	1'-0"×6"×1/2"	2	1'-0"×6"×1/2"	2	1'-0"×6"×1/2"	2	1'-0"×6"×1/2"	2	1'-0"×6"×1/2"
Angles			1	7"×3"×3"×1/2"	1	7"×3"×3"×1/2"				
"			2	1'-1"×3"×3"×1/2"	2	1'-1"×3"×3"×1/2"				
"			1	7"×3"×3"×1/2"	1	7"×3"×3"×1/2"				

BASES		
Plates	5	3'-0"×1'-10 1/2×7/8
Gussets	10	3'-0"×2'-0"×3/4"
Angles	10	3'-0"×6"×6"×3/4"
"	10	6"×6"×6"×3/4"

NOTE: IN ALL STEELWORK, BOLTED CONNECTIONS ARE NOT TO BE USED, EXCEPT WHERE APPROVED OF BY THE ARCHITECT.

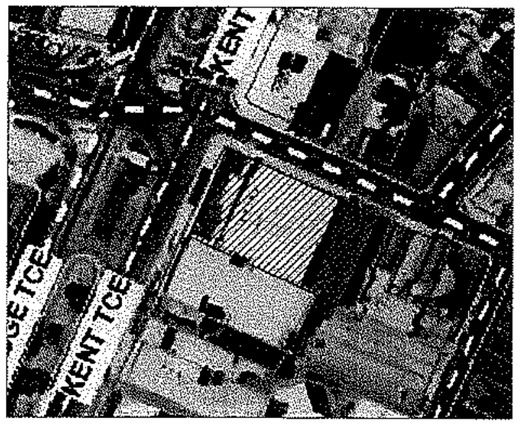
BEAM CONNECTIONS												
	Nº	B1	Nº	B2	Nº	B3	Nº	B4	Nº	B5	Nº	B11
Angles	4	7"×3"×3"×1/2"	4	1'-6"×3"×3"×1/2	4	9"×3×3"×1/2"	4	1'-6"×3"×3"×1/2"	4	9"×3"×3"×1/2"		
"	2	6"×6"×6"×1/2"			2	6"×6"×6"×5/8"			2	6"×6"×6"×5/8		
"	1	5"×3"×3"×3/8"							1	5"×3"×3"×3/8		
"												
Filler												
Tees												
Plate			2	1'-9 1/4×7"×1/2"			2	1'-9 1/4"×7"×1/2"				

	Nº	B6	Nº	B7	Nº	B8	Nº	B9	Nº	B10	Nº	B12
Angles	4	6"×3"×3"×1/2"	2	7"×3 1/2×3 1/2×1/2	2	6"×3"×3"×3/8	1	7"×3 1/2×3 1/2×1/2	1	7"×3"×3"×1/2"	2	7"×3 1/2×3 1/2×1/2
"	2	7"×3"×3"×1/2"	4	1'-6"×3"×3"×1/2"	4	9"×3"×3"×1/2"	4	1'-6"×3"×3"×1/2	4	6"×3"×3"×1/2	4	10"×3×3"×1/2"
"	2	6"×3×3×3/8"	2	7"×6"×6"×1/2"	2	7"×3"×3"×1/2"	1	7"×6"×6"×1/2"	1	6"×3"×3"×3/8	2	7"×6"×6"×1/2
"							1	7"×3"×3"×1/2"				
Filler			2	7"×6"×1/2"			1	7"×6"×1/2"			2	7"×6"×1/2"
Tees			2	11 1/2"×6"×4"×5/8			1	11 1/2"×6"×4"×5/8"			2	11 1/2"×6"×4"×5/8"
Plate							1	1'-8"×7"×1/2"				

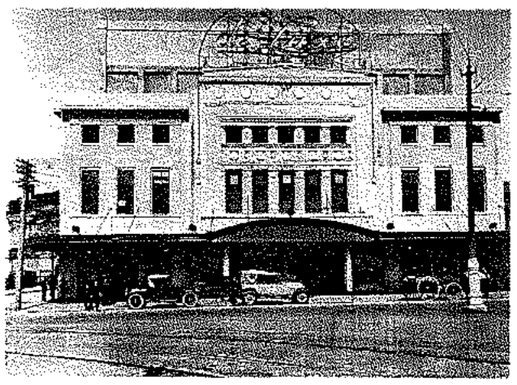
EMBASSY REDEVELOPMENT 2003

STRUCTURAL DRAWINGS

FOR CONSTRUCTION



SITE PLAN

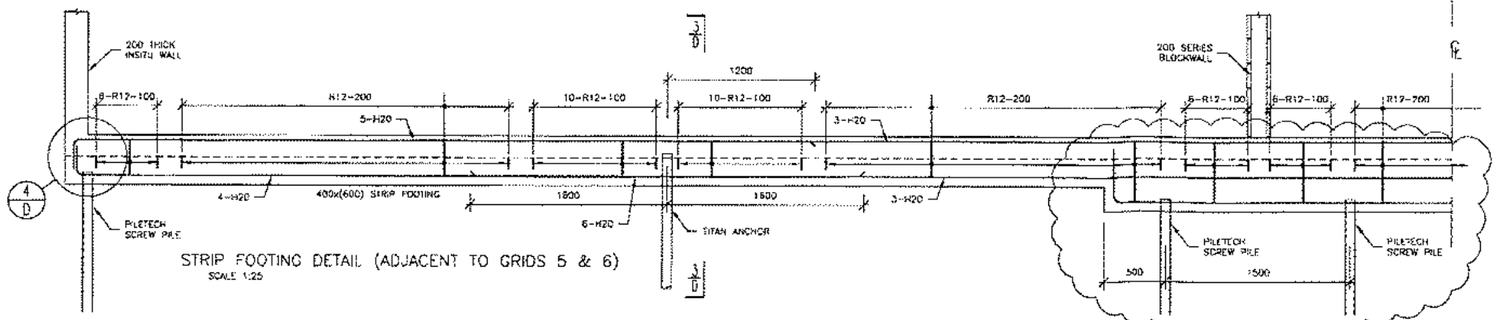


THEATRE EMBASSY TRUST

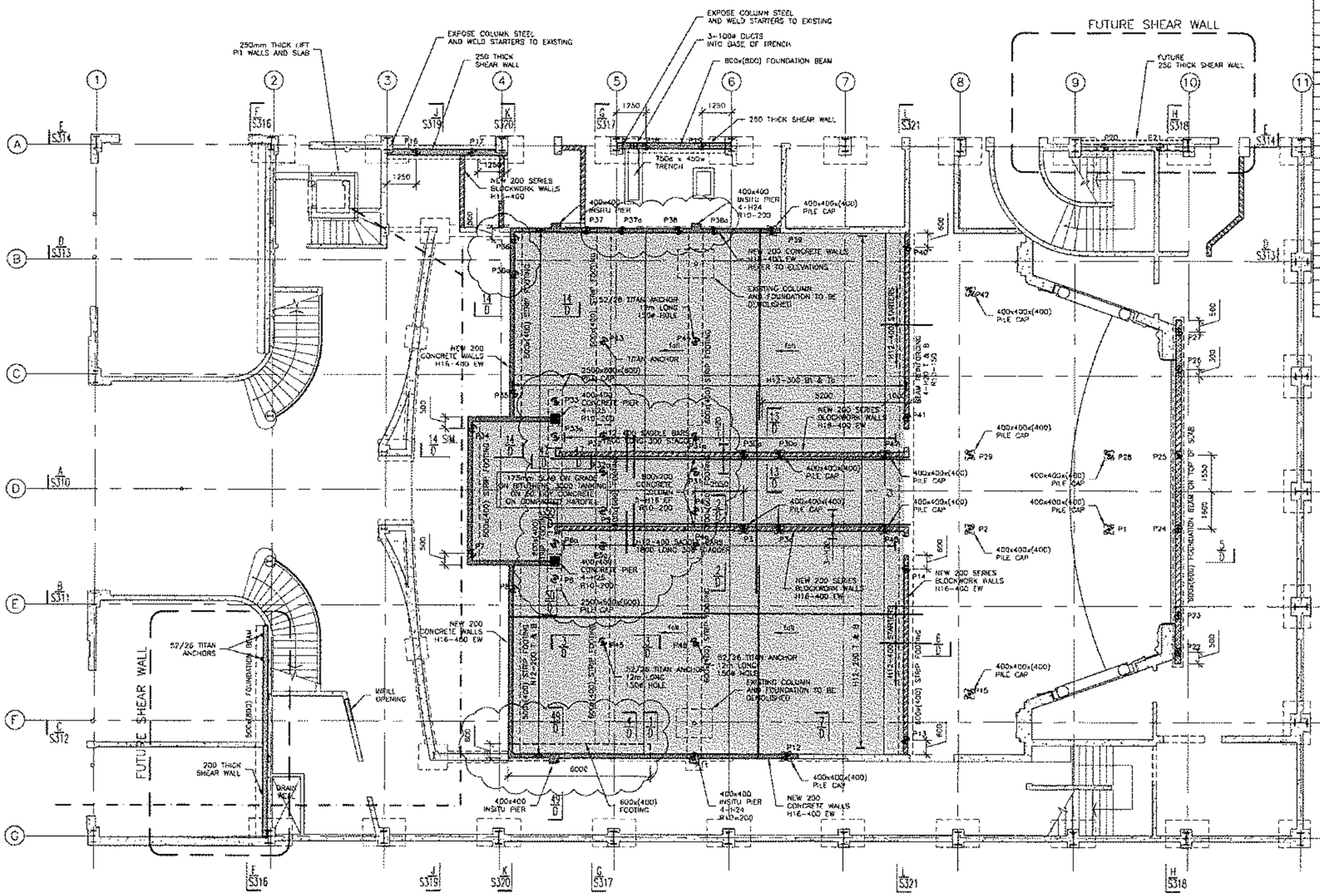
Dunning Thornton Consultants Ltd
Consulting Engineers

Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz
URL www.dunningthornton.co.nz

Job No. 3864



STRIP FOOTING DETAIL (ADJACENT TO GRIDS 5 & 6)
SCALE 1:25



PILE NO	G (kN)	Q (kN)	EQ (kN)	GWL (kN)	1.4G (kN)	1.2G + 1.6Q (kN)	G + 0.7Q (kN)	0.9G - 1.2GWL (kN)	G + 0.4Q - E (kN)	G + 0.4Q + E (kN)
1	103	86			144	261	163		137	137
2	156	58			218	344	225		195	195
3	262	157			367	566	372		325	325
4	340	264			476	830	525		446	446
5	251	124	230		351	500	338		71	531
6	129	324	125		181	513	286		94	344
7	78	25	125		109	134	96		-37	213
8	168	99	125		235	360	237		83	333
9	205	148	184		405	577	390		183	531
10	233	148	115		326	513	335		178	406
11	336	244			470	794	507		434	434
12	171	120	115		239	397	255		194	334
13	121	54	184		169	232	159		-41	327
14	143	64	184		200	274	168		-15	353
15	160	106			224	362	234		202	202
16									-350	750
17									-350	750
18									-350	750
19									-350	750
20									-350	750
21									-350	750
22	430		310						120	740
23	430		310						120	740
24	430		310						120	740
25	430		310						120	740
26	430		310						120	740
27	430		310						120	740
28	102	86			144	261	163		137	137
29	156	58			218	344	225		195	195
30	262	157			367	566	372		325	325
31	340	264			476	830	525		446	446
32	251	124	230		351	500	338		71	531
33	129	324	125		181	513	286		94	344
34	78	25	125		109	134	96		-37	213
35	168	99	125		235	360	237		83	333
36	205	148	184		405	577	390		183	531
37	233	148	115		326	513	335		178	406
38	336	244			470	794	507		434	434
39	171	120	115		239	397	255		194	334
40	121	54	184		169	232	159		-41	327
41	143	64	184		200	274	168		-15	353
42	160	106			224	362	234		202	202
43	96	91		-342	134	261	160	-324		
44	96	91		-342	134	261	160	-324		
45	96	91		-342	134	261	160	-324		
46	96	91		-342	134	261	160	-324		
47	175	105	184		245	378	249		33	401
48	175	105	184		245	378	249		35	401

THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

LEVEL 0 PILE LAYOUT & LOWER SLAB DETAILS

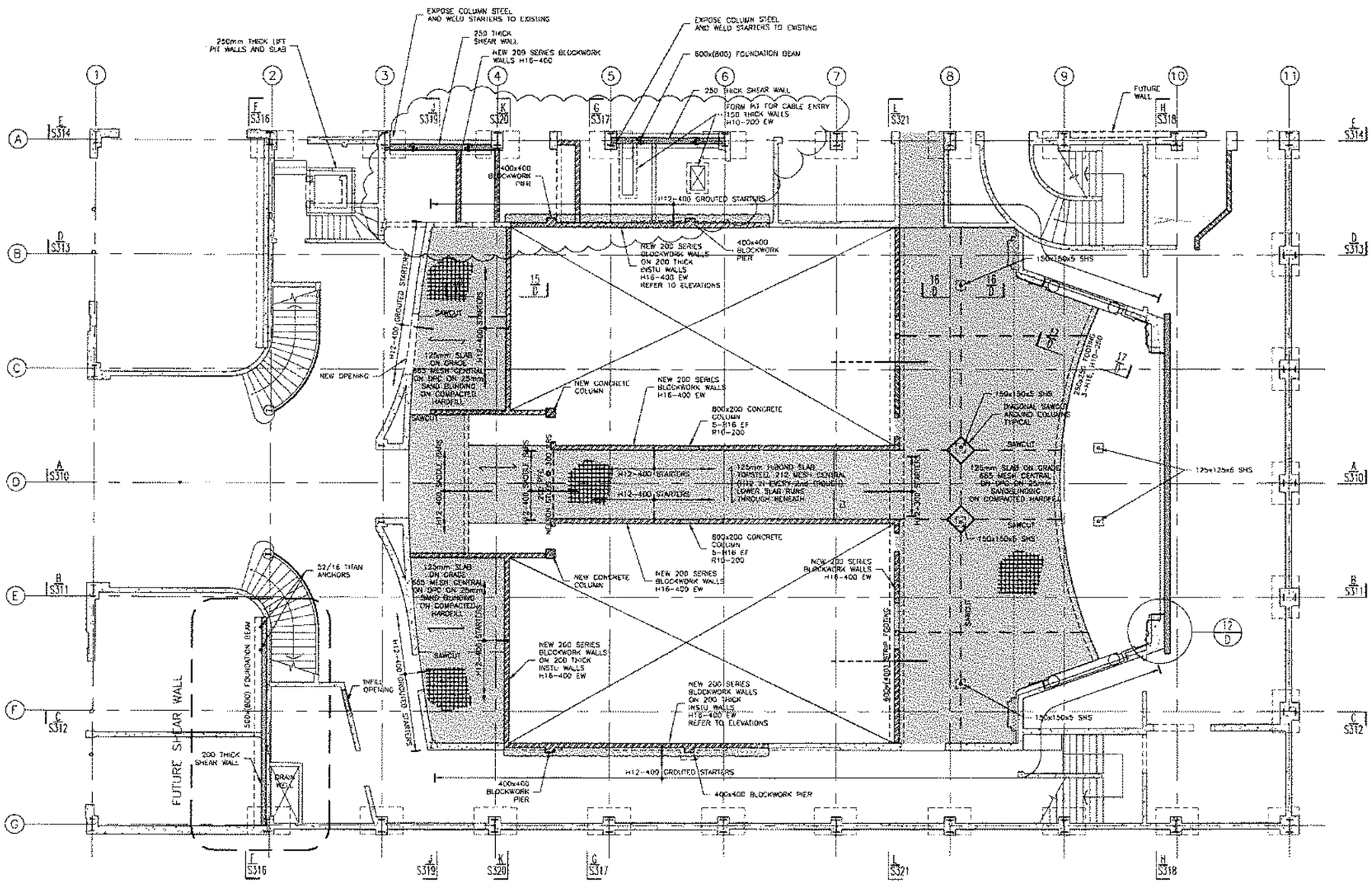
REVISIONS		
3	REVISED	22-07-2003
2	FOR CONSTRUCTION	20-06-2003
1	FOR CONSENT	3-06-2003
0	PILING TENDER	22-05-2003

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864
Drawing Number S300
Rev 3

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz



THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

LEVEL 0
SLAB PLAN

REVISIONS		
3	REVISED	22-07-2003
2	FOR CONSTRUCTION	20-06-2003
1	FOR CONSENT	3-06-2003
0	PILING TENDER	22-05-2003

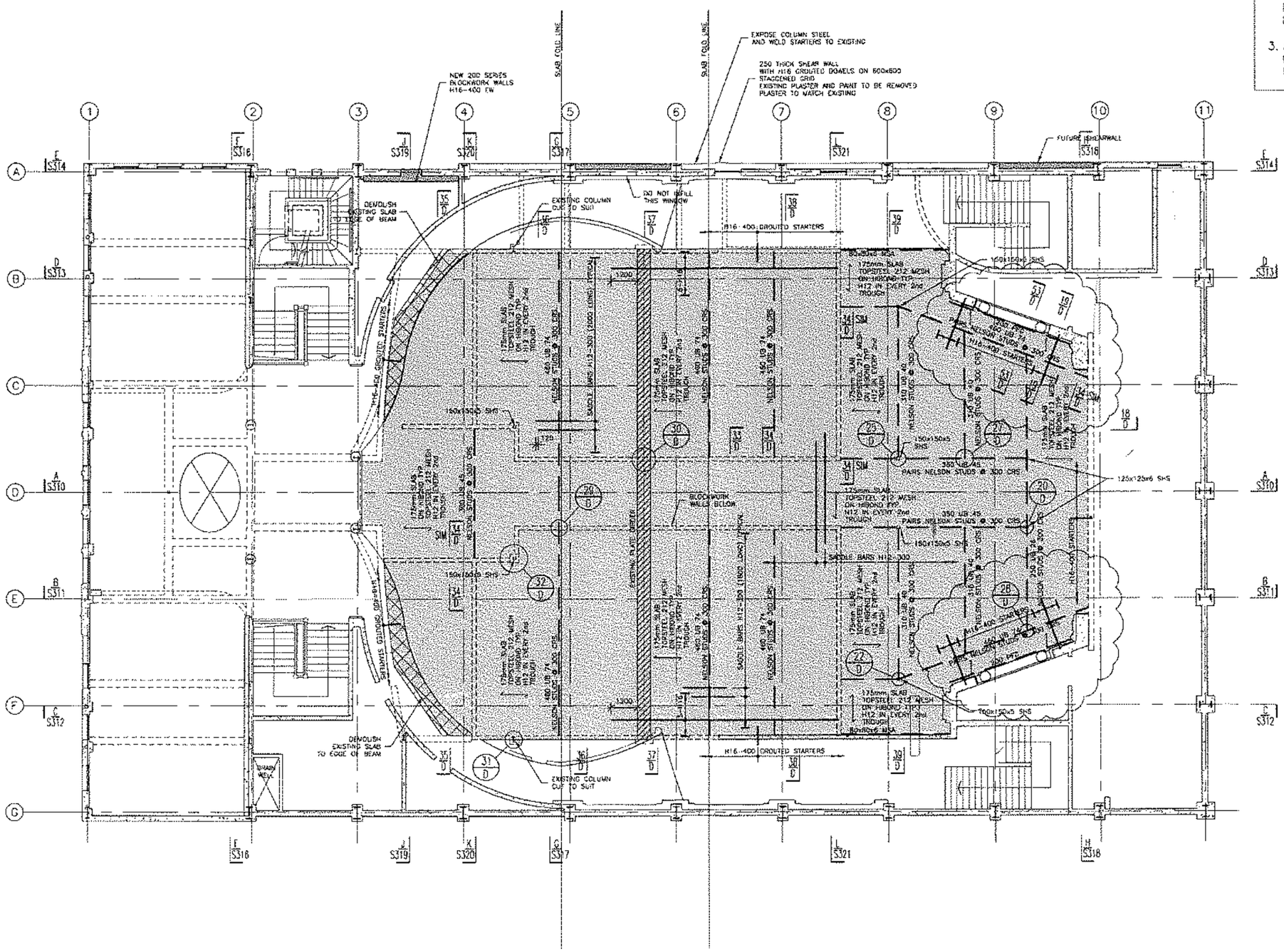
Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864
Drawing Number S301
Rev 3

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz

NOTE

1. HIBOND SLABS OVER 2.4m SPAN ARE TO BE PROPPED AT MIDSPAN DURING CONSTRUCTION
2. ALL BEAMS ARE TO BE PROPPED AT 1/3 POINTS DURING CONSTRUCTION
3. ALL STUDS ARE TO BE M20x95mm LONG NELSON STUDS



EMBASSY REDEVELOPMENT 2003	LEVEL 1	

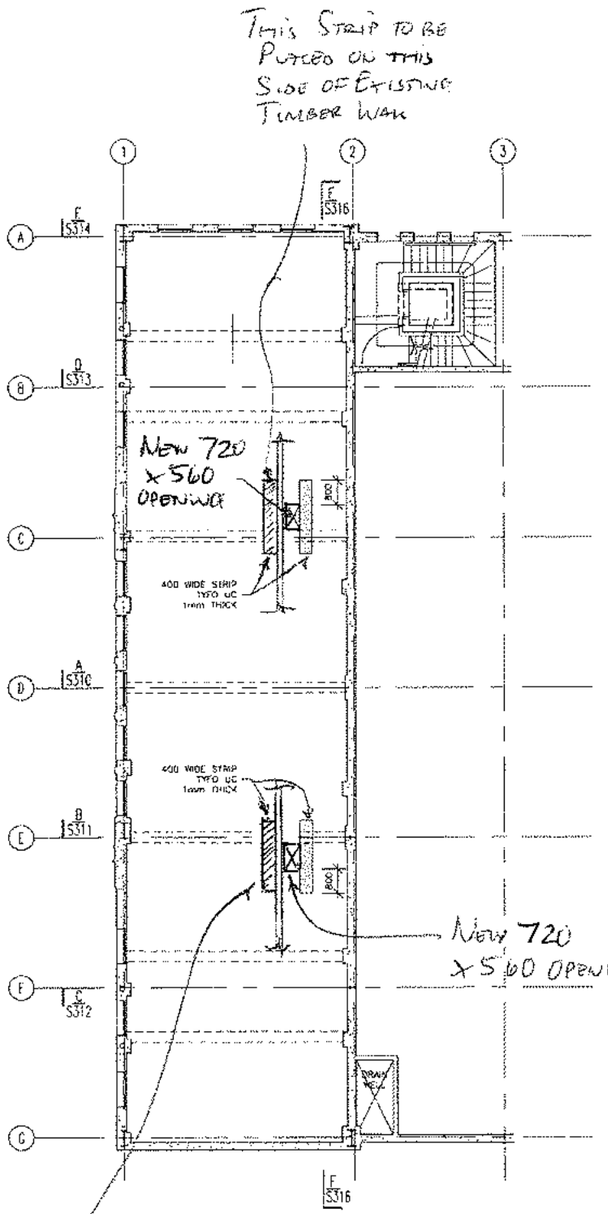
REVISIONS		
3	REVISED	22-07-2003
2	FOR CONSTRUCTION	20-06-2003
1	FOR CONSENT	2-06-2003
0	PILING TENDER	22-05-2003

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

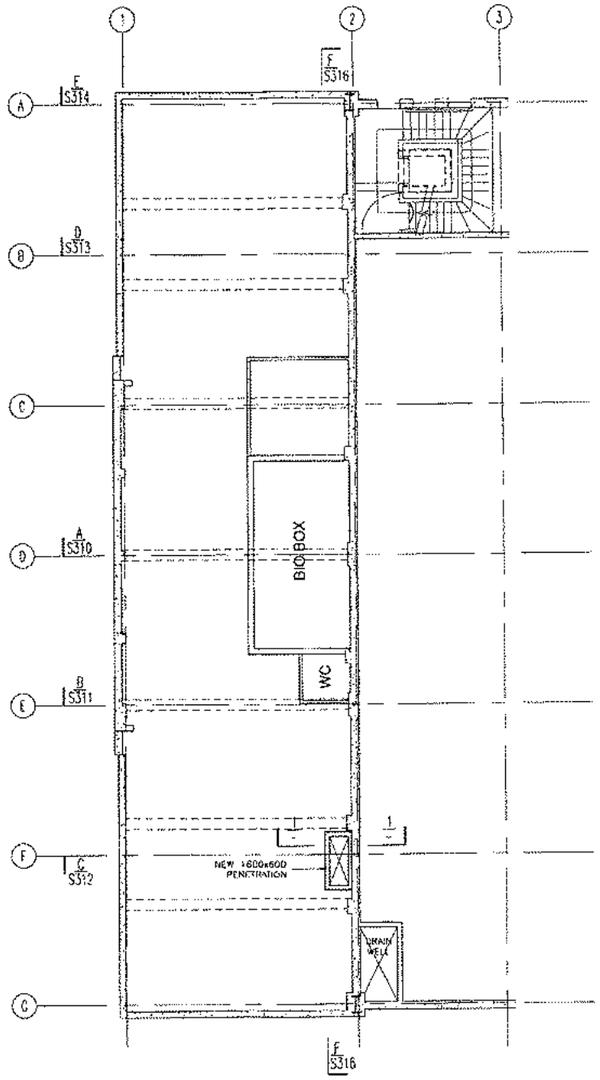
Job Number 3864

Drawing Number S302 Rev 3

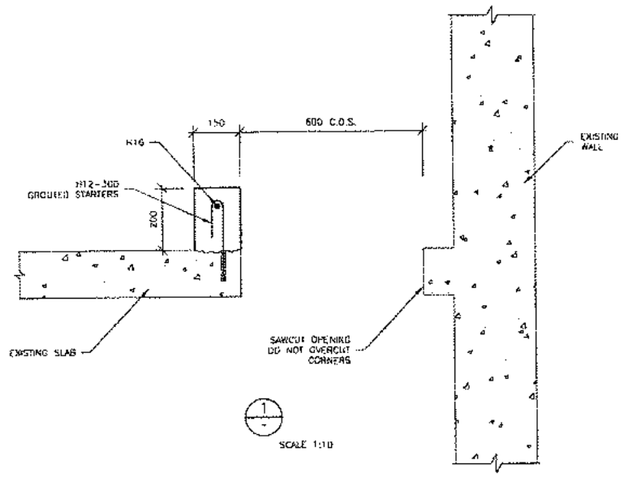
Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz



SECOND FLOOR SLAB PLAN
SCALE 1:100



THIRD FLOOR (ROOF) SLAB PLAN
SCALE 1:100



NOTE
CONFIRM DIMENSIONS AND POSITIONS OF PENETRATIONS ON SITE

THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

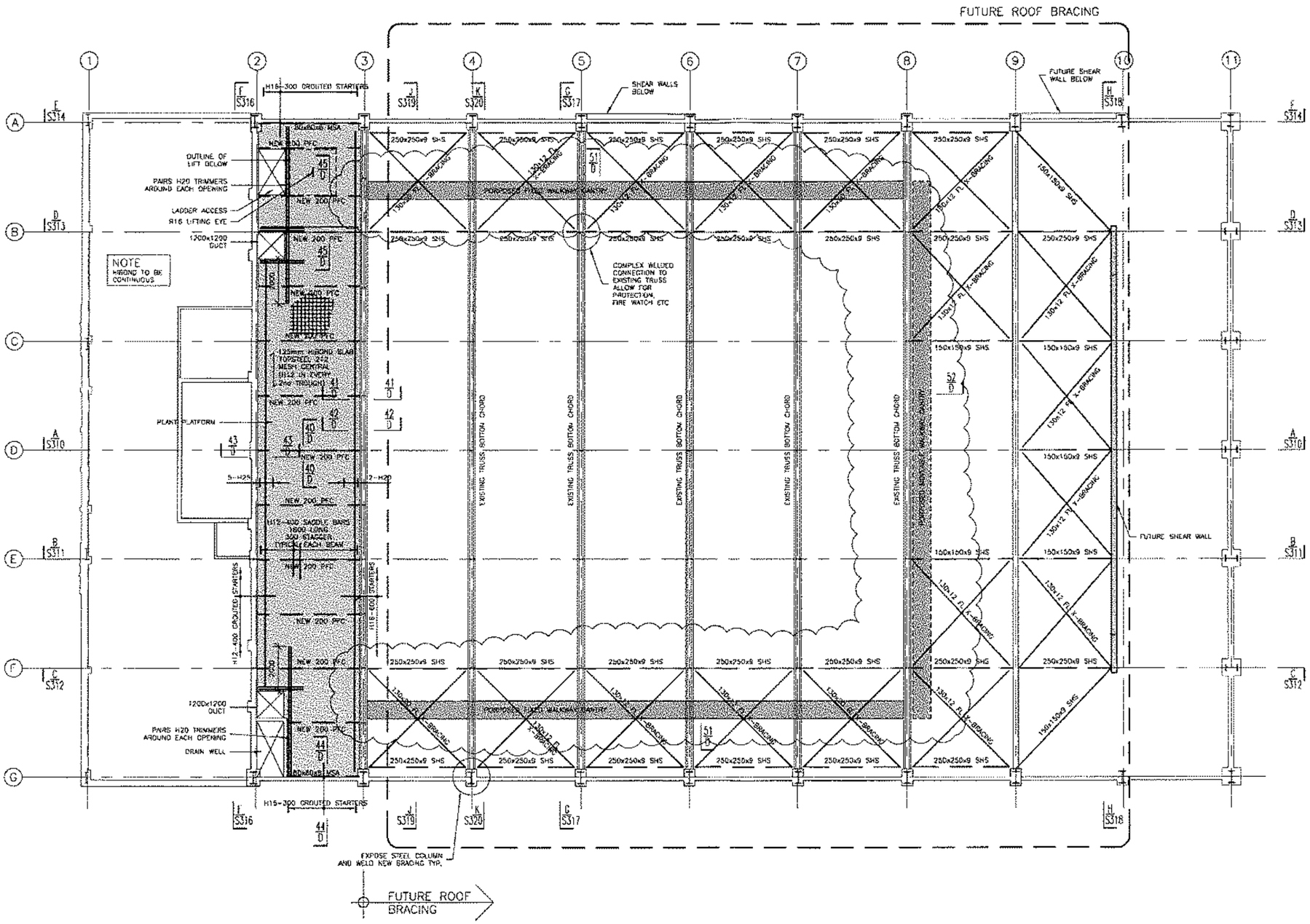
BOTTOM CHORD ROOF BRACING PLAN

REVISIONS		
1	FOR CONSTRUCTION	20-06-2003

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864
Drawing Number S303
Rev 2

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-183 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz



G:\3864\DtcDwgs\3864S300.dwg, 23-07-2003 1:33:59 p., HP LaserJet 4MV.pc3

THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

BOTTOM CHORD ROOF BRACING PLAN

REVISIONS		
3	REVISED	22-07-2003
2	FOR CONSTRUCTION	16-06-2003
1	FOR CONSENT	30-5-03

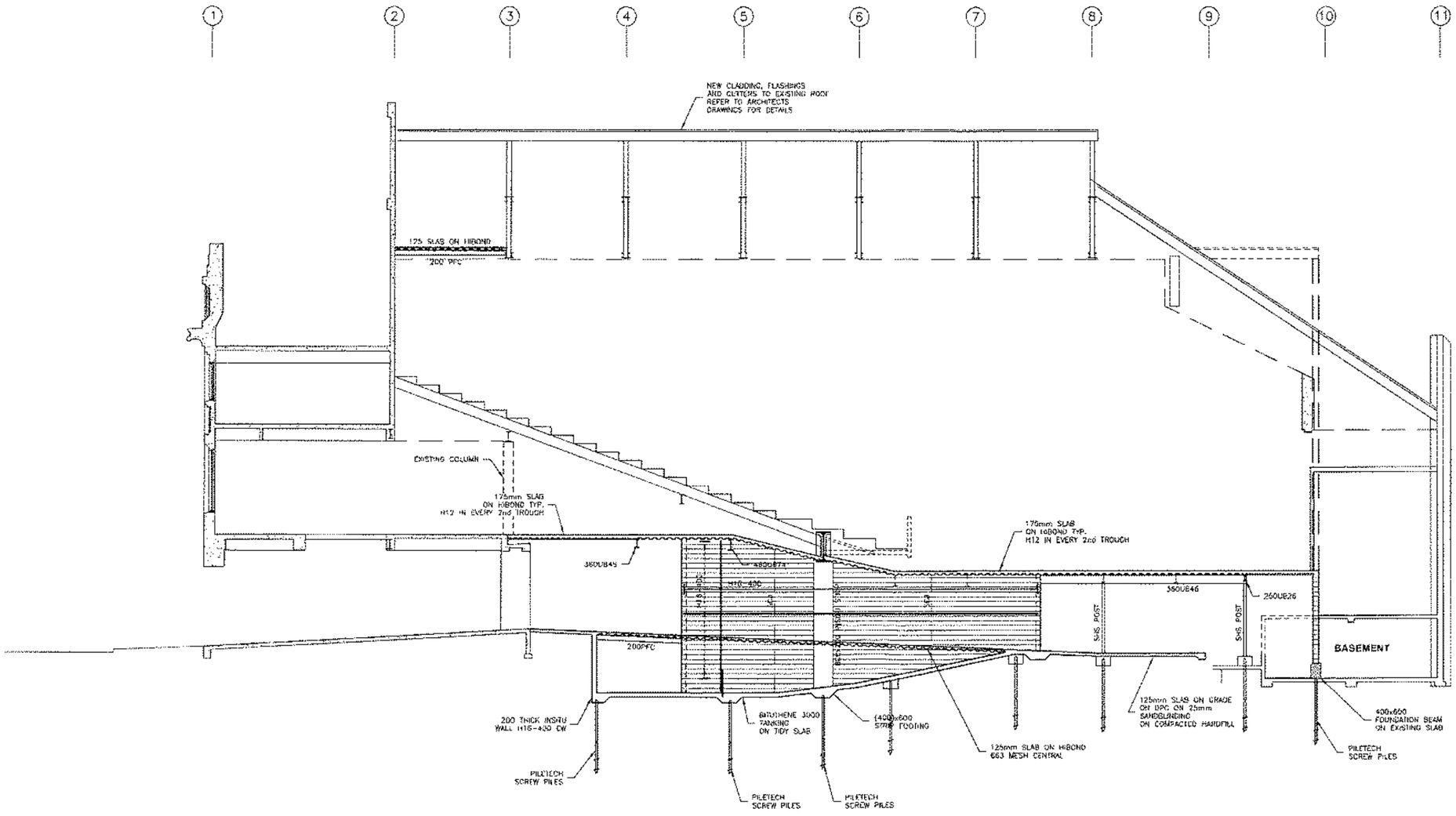
Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864

Drawing Number S305 Rev 3

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz



A S300 SECTION GRID D

SCALE 1:100

EMBASSY REDEVELOPMENT 2003

SECTION GRID D

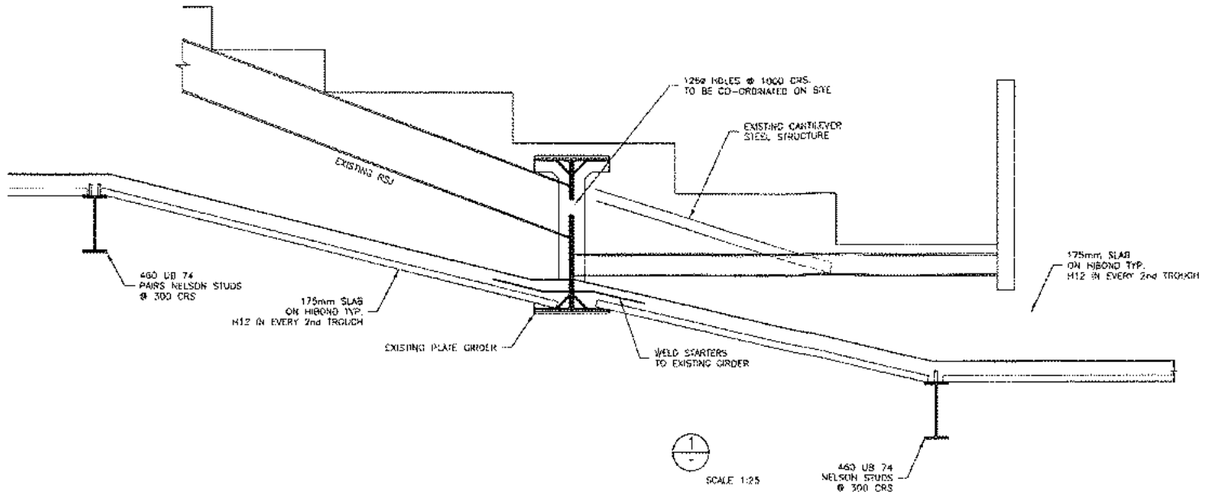
REVISIONS		
3	FOR CONSTRUCTION	20.06.2003
1	FOR CONSENT	3.06.2003
0	ISSUED FOR PRICING	29.05.2003

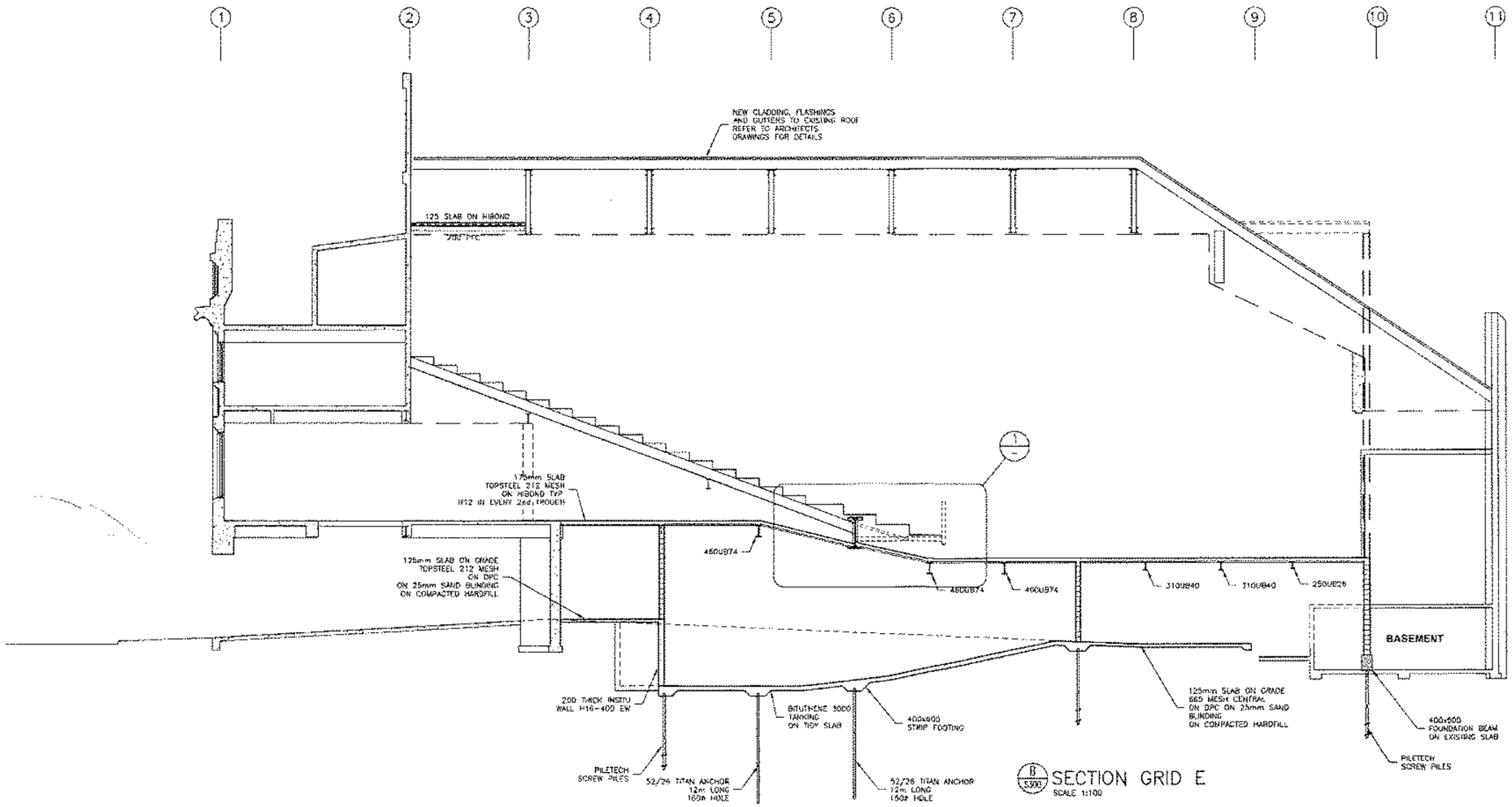
Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864

Drawing Number S310 Rev 2

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385 0312
Box 27-155 Wellington, N.Z.
E-mail stewgtns@dunningthornton.co.nz





EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

SECTION GRID E

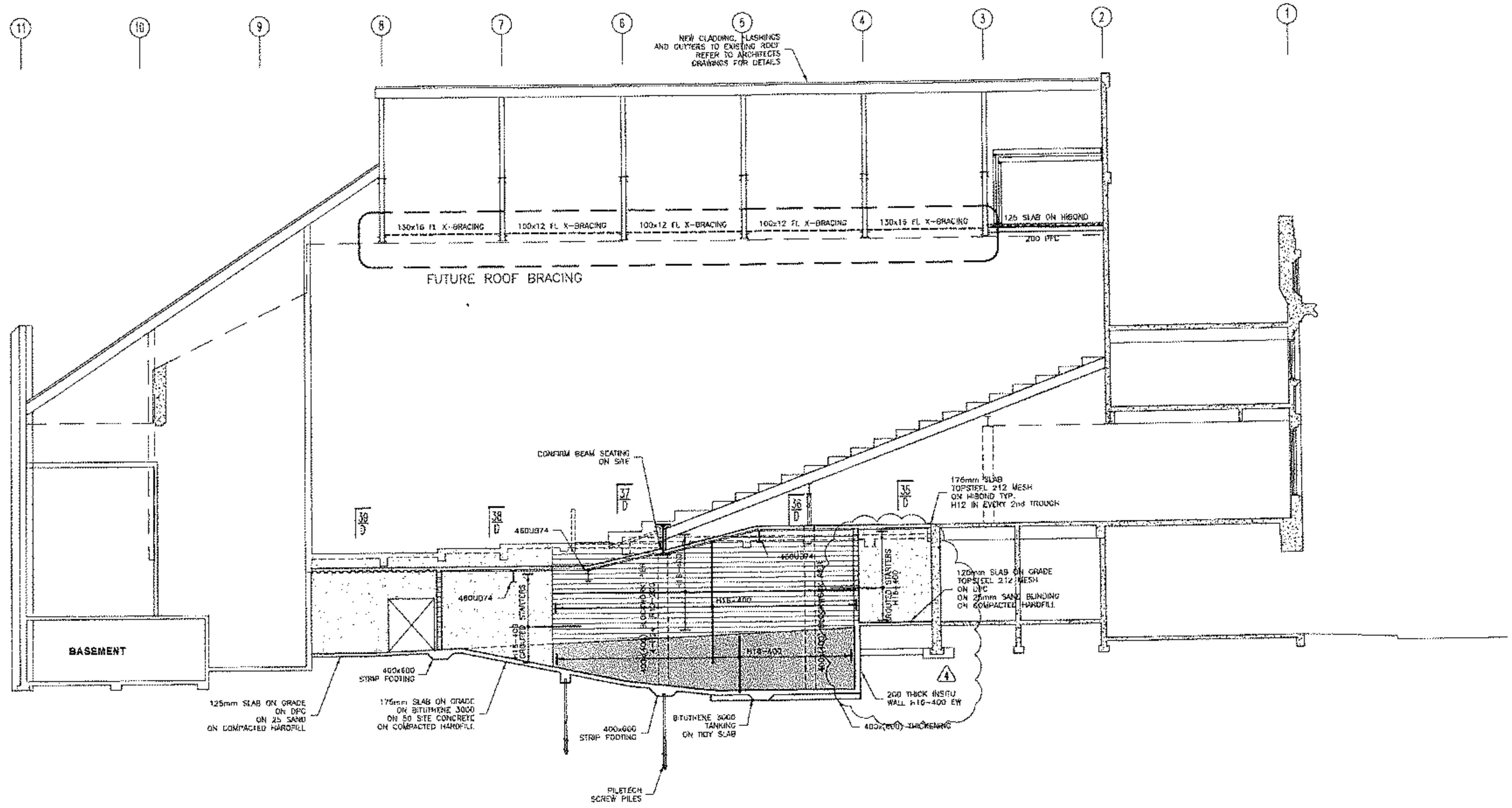
REVISIONS		
2	FOR CONSTRUCTION	29.09.2003
1	FOR CONSENT	3.09.2003
0	PILING TENDER	22.05.2003

Scales	1:100, 1:25
A3 Scales	1:200, 1:50
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864
Drawing Number S311
Rev 2

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0319
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail drewgtn@dunningthornton.co.nz



SECTION GRID F
SCALE 1:100

EMBASSY THEATRE TRUST

EMBASSY REDEVELOPMENT 2003

SECTION GRID 10

REVISIONS		
4	REVISED	29-08-2003
3	REVISED	22-07-2003
2	FOR CONSTRUCTION	20-06-2003
1	FOR CONSENT	3-06-2003
0	PILING TENDER	22-05-2003

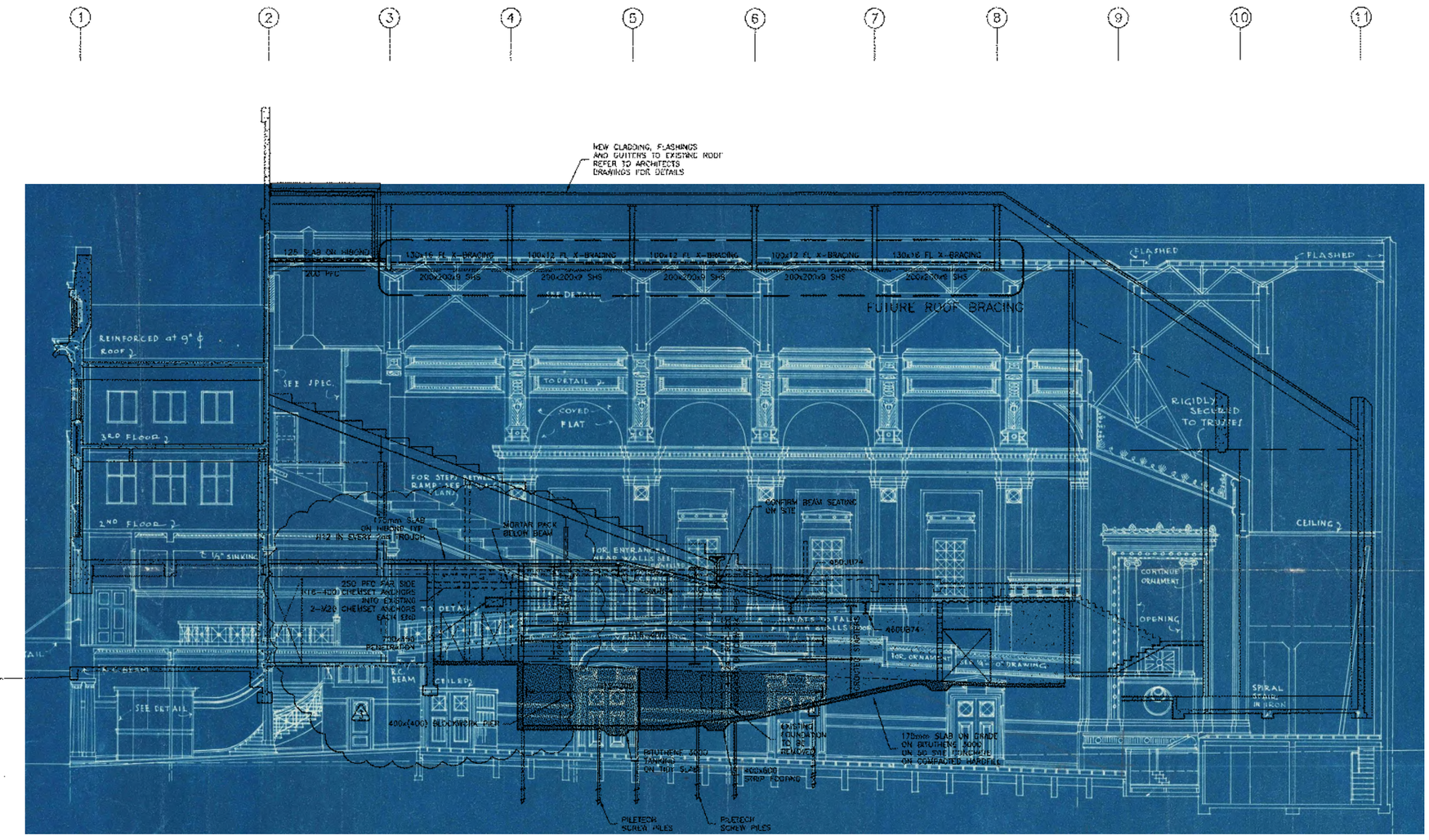
Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864
Drawing Number S312
Rev 4

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtnwgtn@dunningthornton.co.nz

G:\3864\Dtc\Dwgs\3864S300.dwg, 02/09/2003 09:56:43



D S300 SECTION GRID B

SCALE 1:100

G:\3864\Dwgs\3864S300.dwg, 02/09/2003 09:56:22

EMBASSY THEATRE

EMBASSY REDEVELOPMENT 2003

SECTION GRID B

REVISIONS		
3	REVISED	29-09-2003
2	FOR CONSTRUCTION	26-08-2003
1	FOR CONSENT	3-06-2003

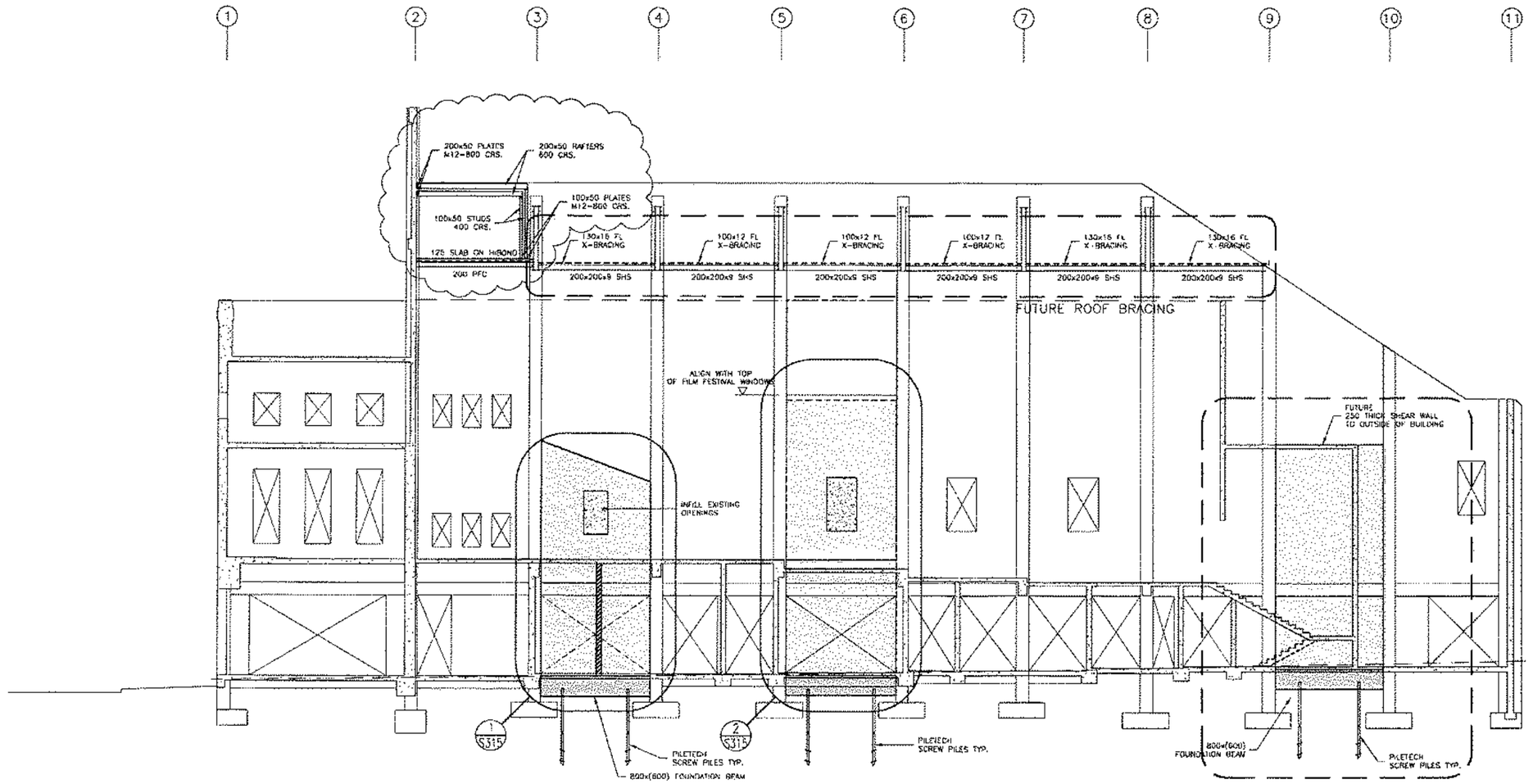
Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864

Drawing Number S313 Rev 3

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-6019
Fax (644) 385 6312
Box 27-155 Wellington, N Z
E-mail dtewgtn@dunningthornton.co.nz



E S300 ELEVATION GRID A

SCALE 1:100

THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

ELEVATION GRID A

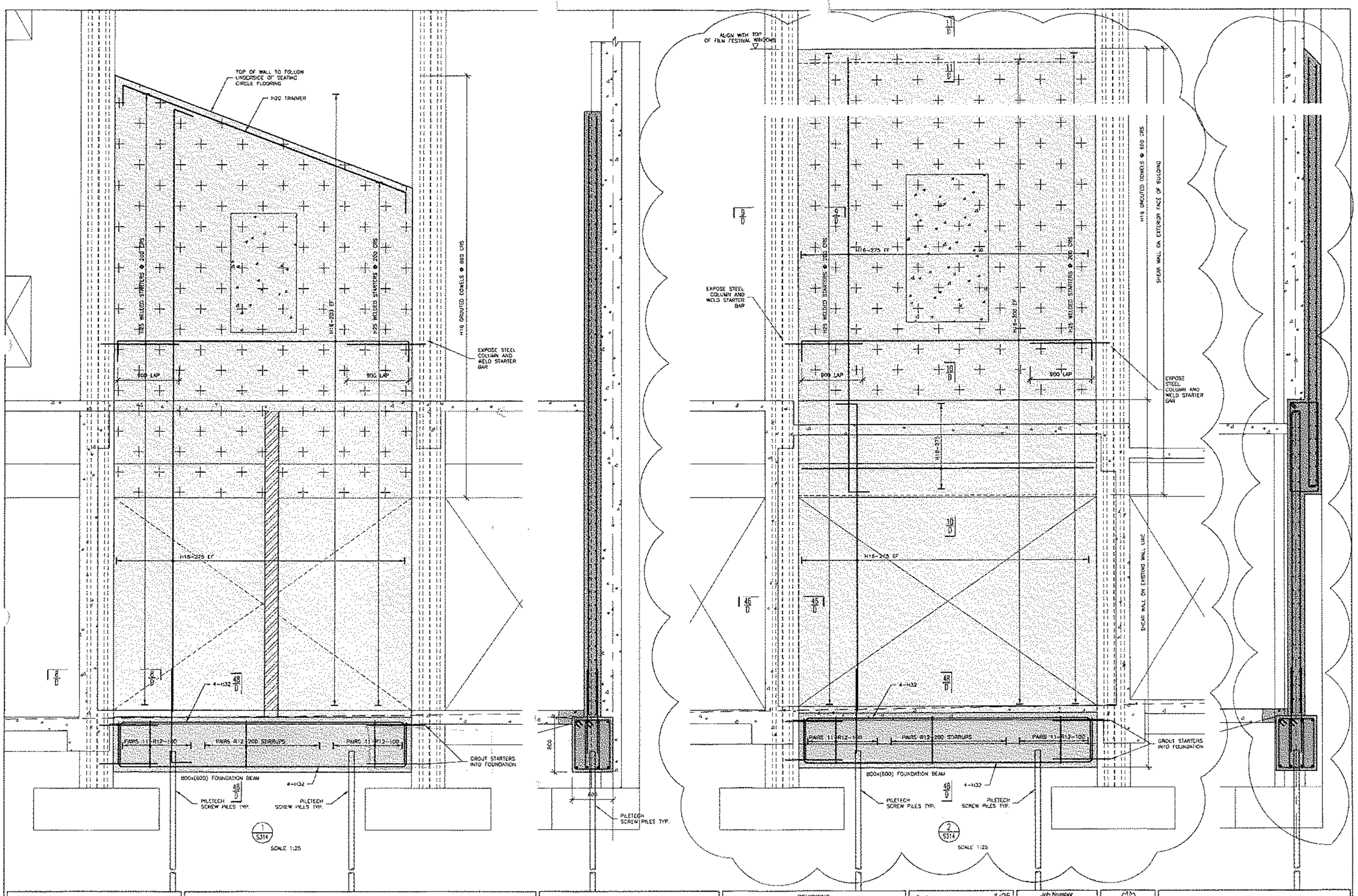
REVISIONS		
3	REVISED	22-07-2003
2	FOR CONSTRUCTION	20-05-2003
1	FOR CONSENT	2-06-2003

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864

Drawing Number S314 Rev 3

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27 153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz



THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

GRID A
SHEARWALLS 1 & 2

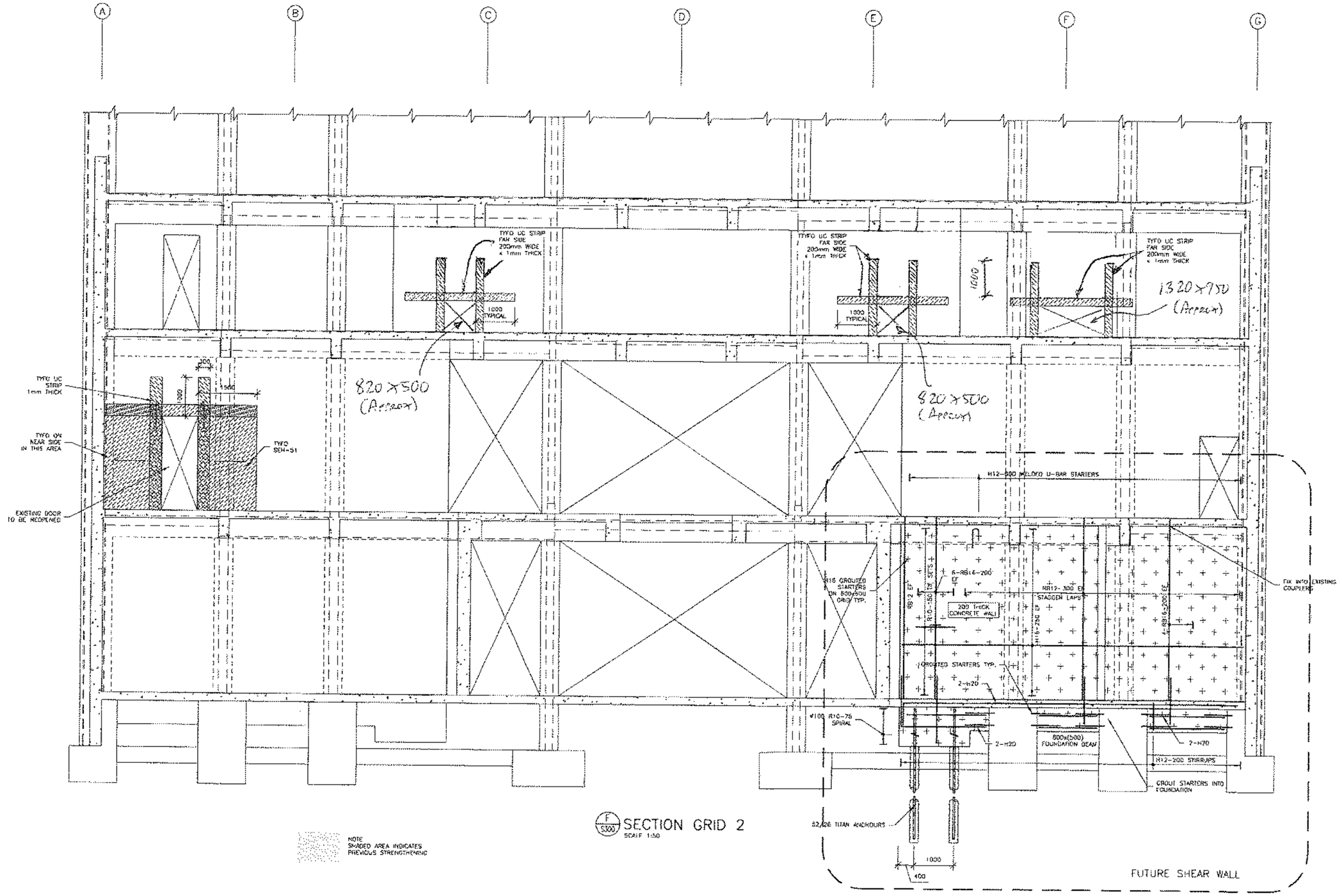
REVISIONS		
3	REVISED	22-07-2003
2	FOR CONSTRUCTION	20-06-2003
1	FOR CONSENT	3-06-2003

Scales	1:25
As Scales	1:50
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864
Drawing Number S315
Rev 3

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz



F S300 SECTION GRID 2
SCALE 1:50

NOTE
SHADED AREA INDICATES PREVIOUS STRENGTHENING

THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

SECTION GRID 2

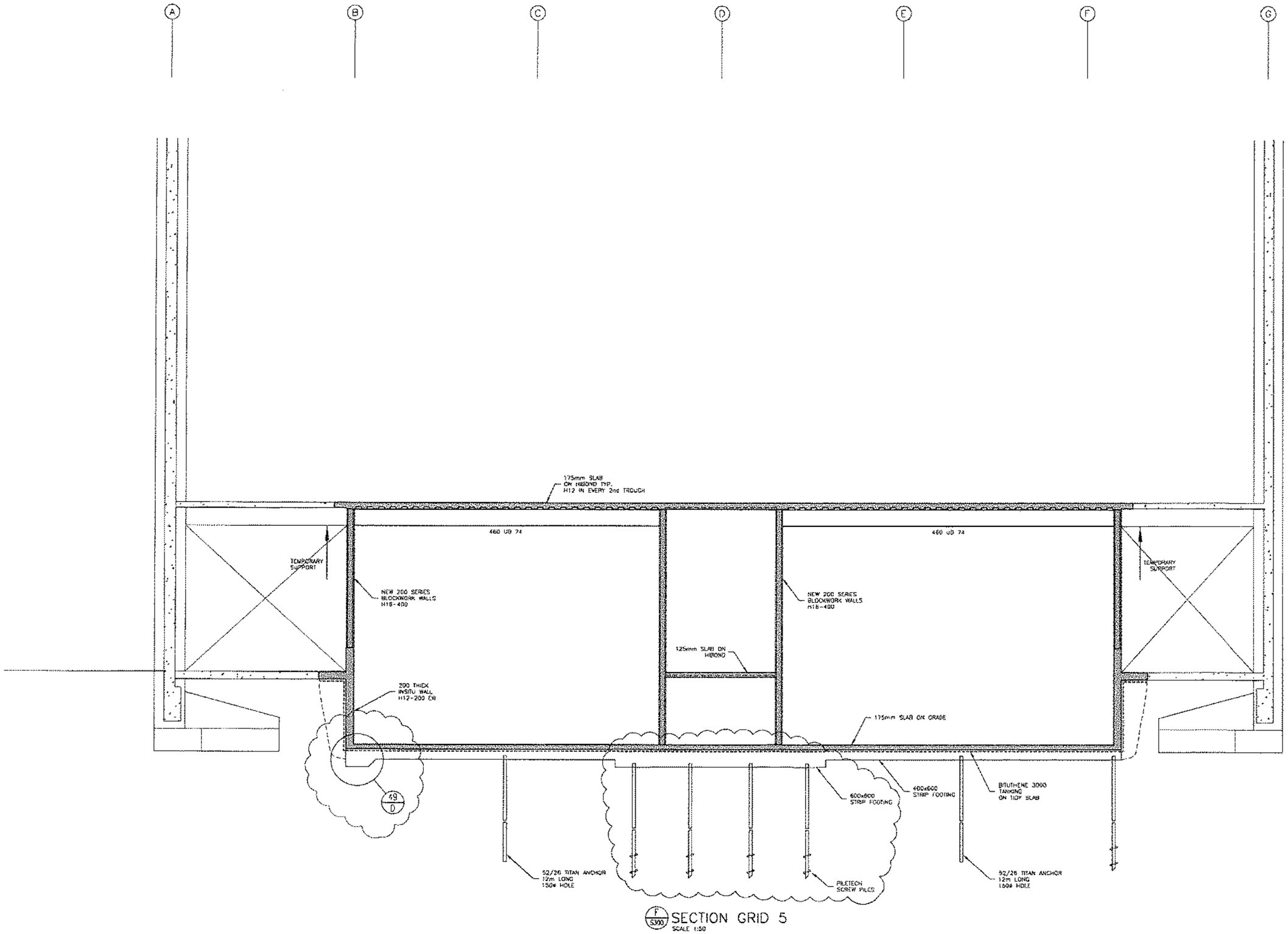
REVISIONS		
2	FOR CONSTRUCTION	20-06-2003
1	FOR CONSENT	3-06-2003

Scales	1:50
A3 Scales	1:100
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864
Drawing Number S316
Rev 2 3

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385 0312
Box 27-159 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz



F S300 SECTION GRID 5
SCALE 1:50

THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

SECTION GRID 5

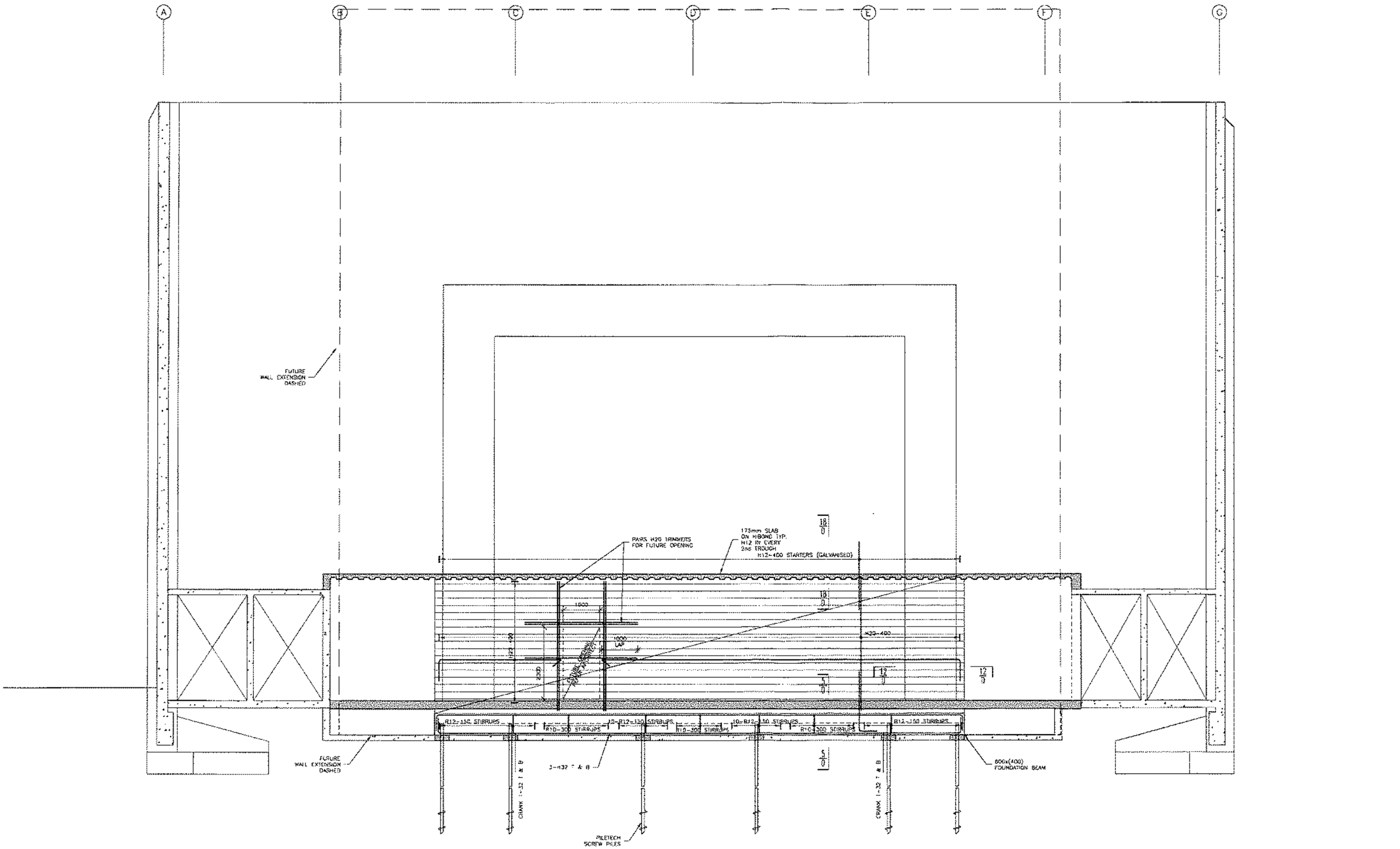
REVISIONS		
X	REVISED	22-07-2003
2	FOR CONSTRUCTION	20-06-2003
1	FOR CONSENT	3-06-2003

Scales	1:50
A3 Scales	1:100
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864
Drawing Number S317
Rev 3

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz



THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

SECTION GRID 10

REVISIONS		
2	FOR CONSTRUCTION	20-08-2003
1	FOR CONSENT	3-06-2003

Scales	1:50
A3 Scales	1:100
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

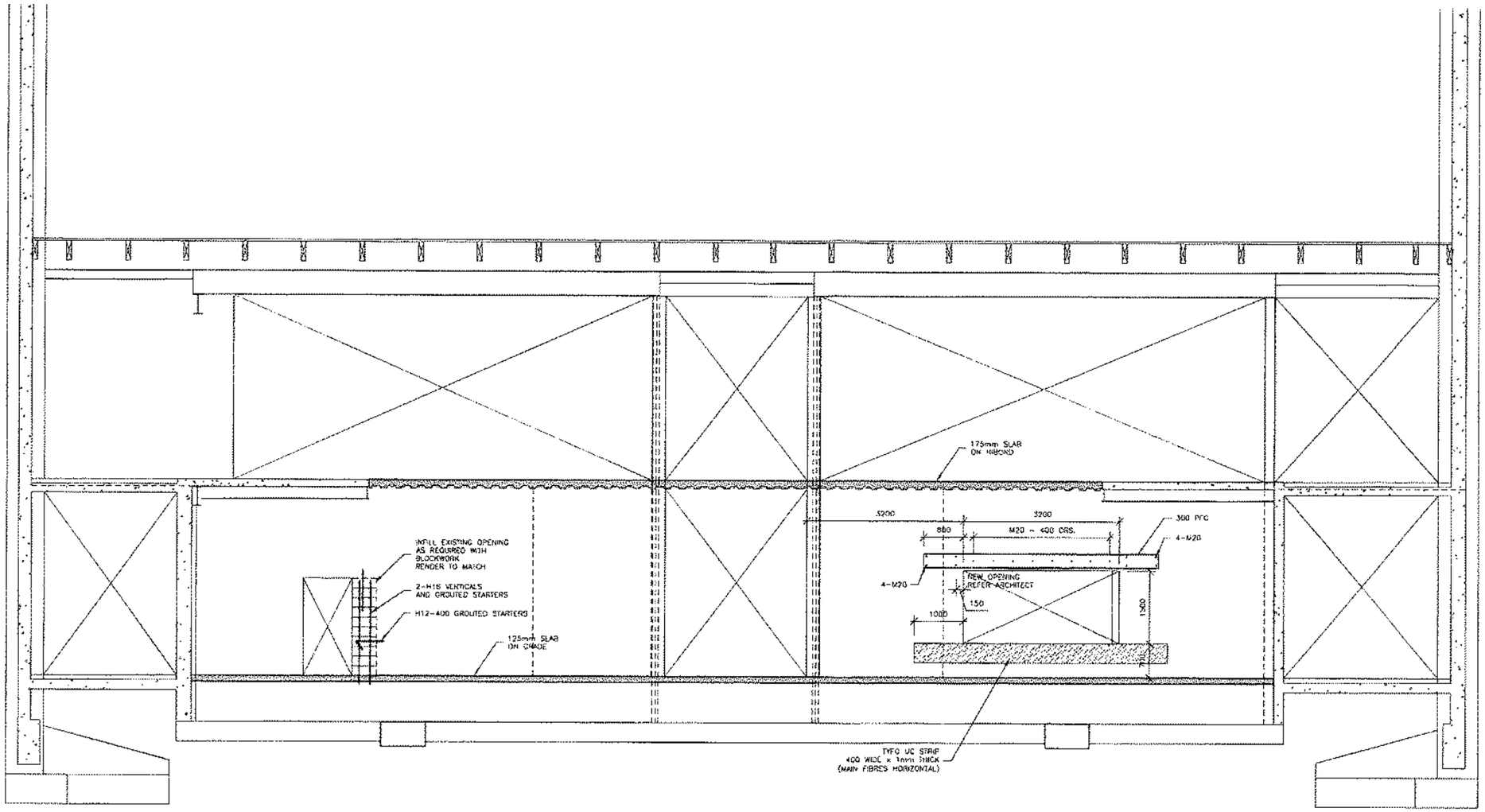
Job Number 3864

Drawing Number S318 Rev 2

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-2212
Box 27-163 Wellington, N.Z.
E-mail otcwgtn@dunningthornton.co.nz

G F E D C B A



J/S300 SECTION GRID 3+
SCALE 1:50

THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

SECTION GRID 3

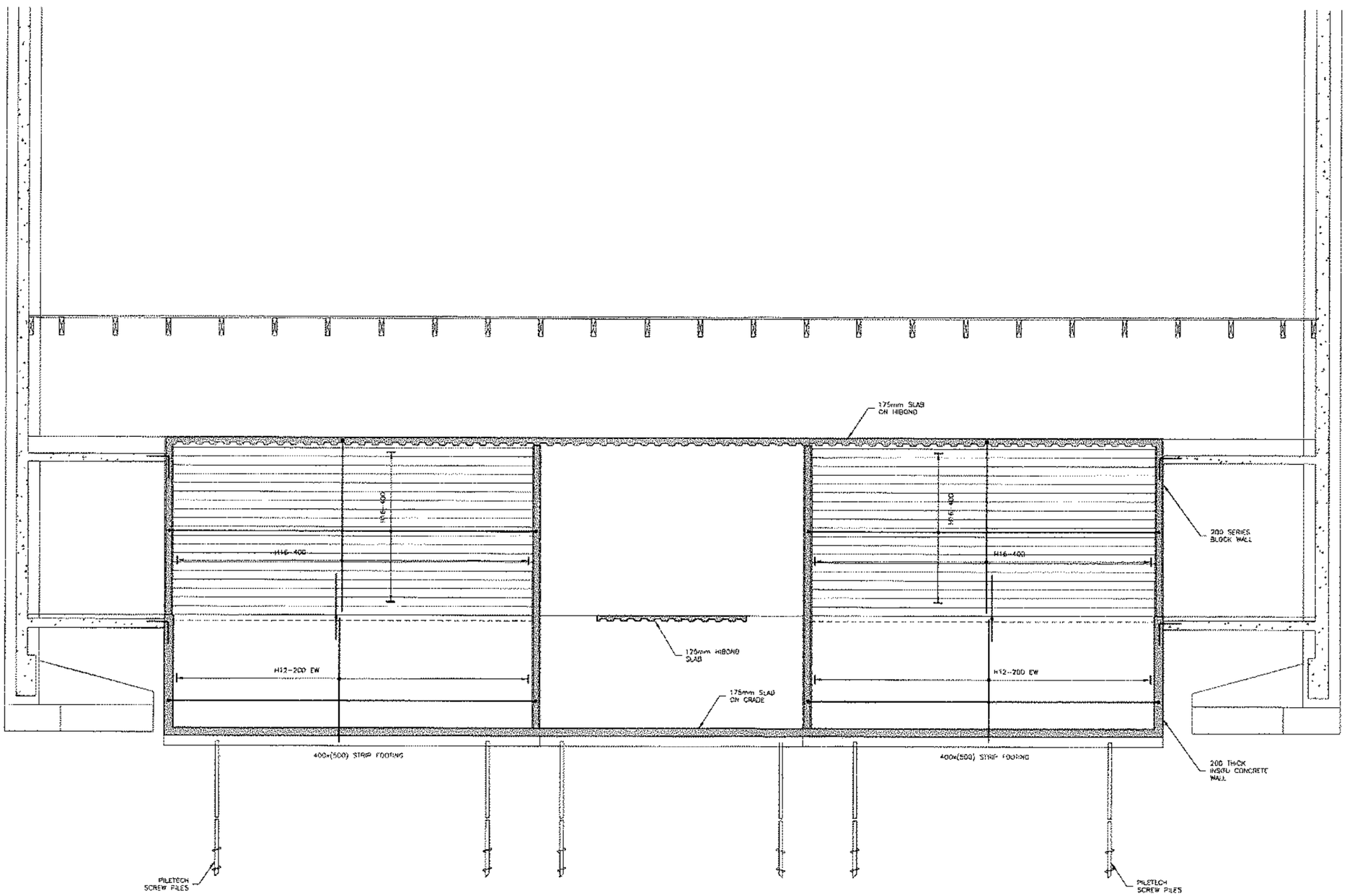
REVISIONS		
1	FOR CONSTRUCTION	20-08-2003

Scales	1:50
A3 Scales	1:100
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864
Drawing Number S319
Rev 1

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz



K S300 SECTION GRID 4
SCALE 1:50

THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

SECTION GRID 4

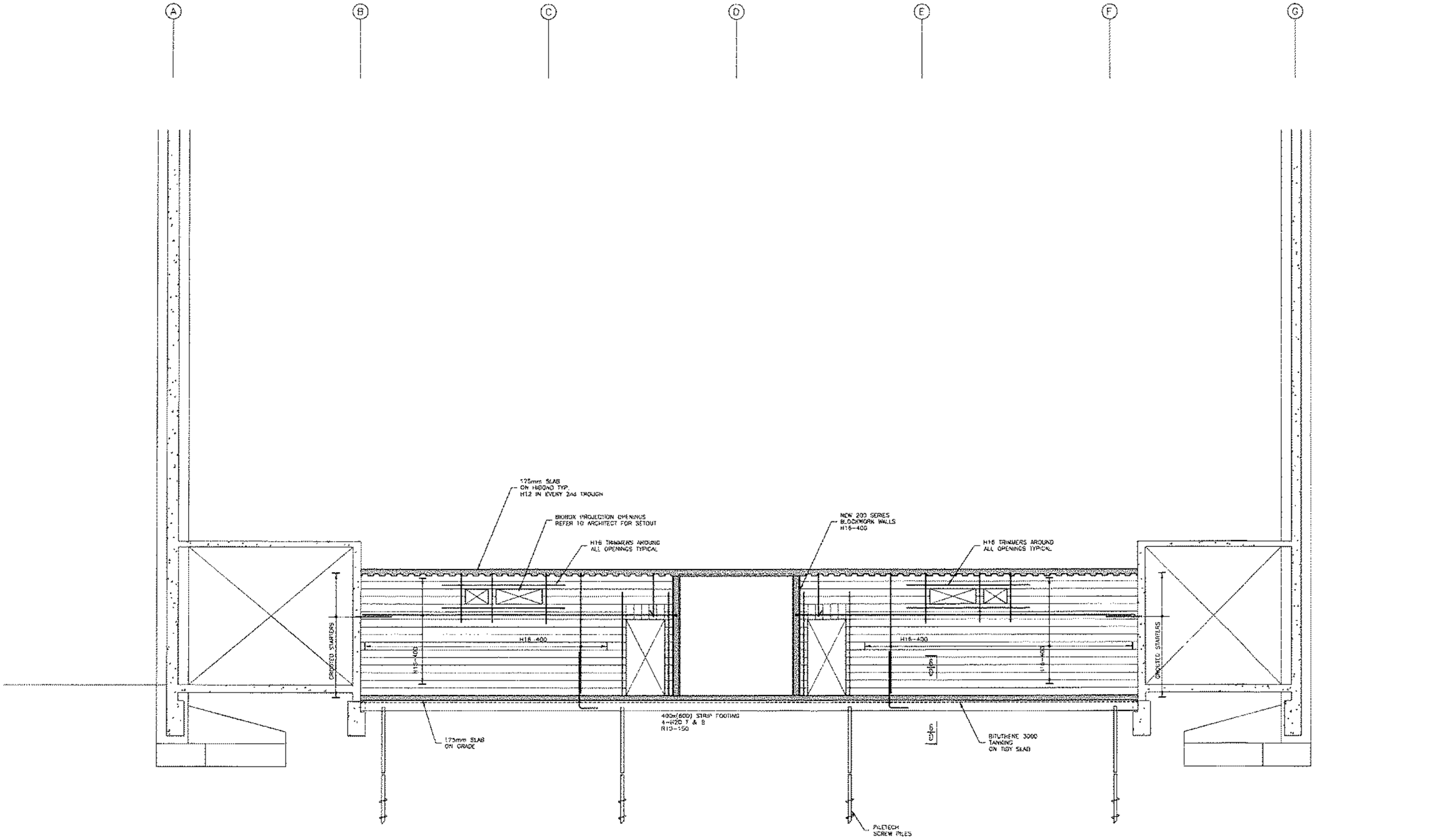
REVISIONS		
1	FOR CONSTRUCTION	20-06-2003

Scales	1:50
A3 Scales	1:100
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864
Drawing Number S320
Rev 1

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz



1/S300 SECTION GRID 7+
SCALE 1:50

THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

SECTION GRID 7+

REVISIONS		
1	FOR CONSTRUCTION	20-06-2003

Scales	1:50
A3 Scales	1:100
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

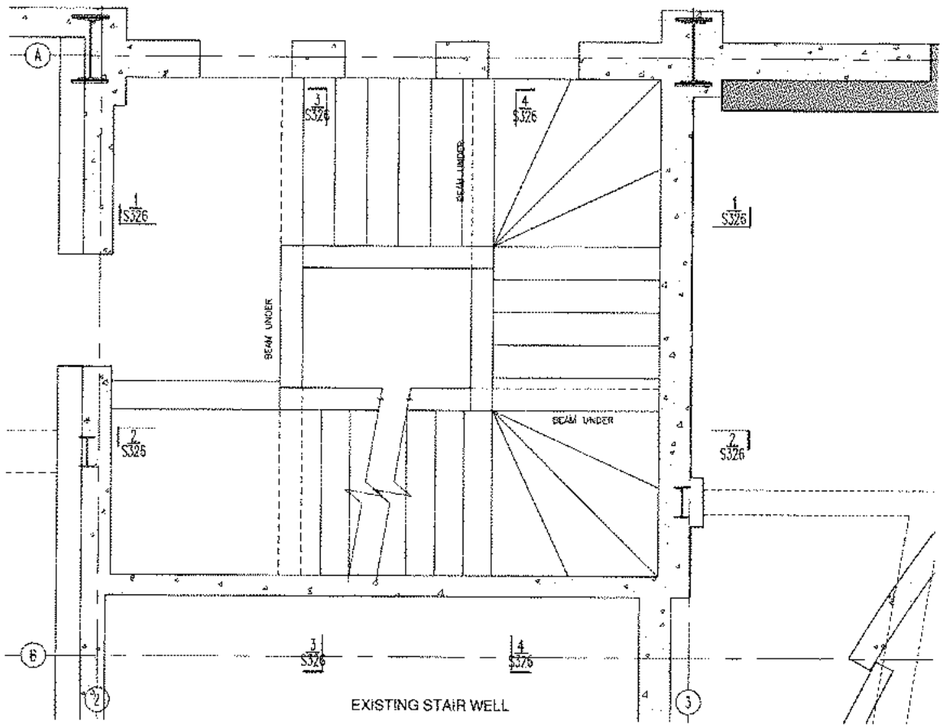
Job Number 3864

Drawing Number S321

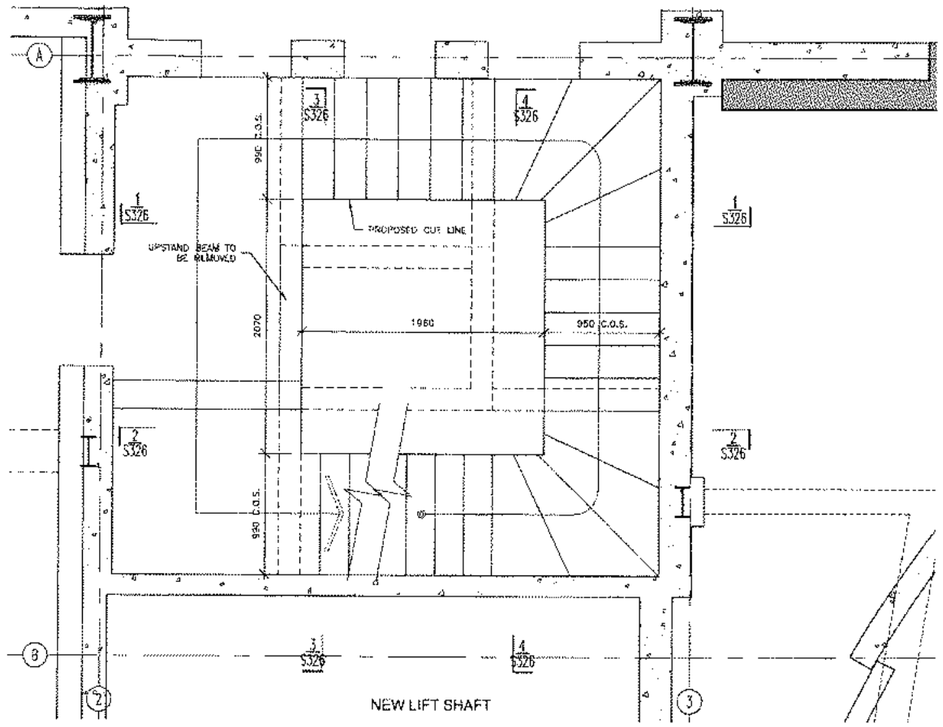
Rev 1

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-9019
Fax (644) 385-9312
Box 27-153 Wellington, N.Z.
E-mail dtewgtn@dunningthornton.co.nz



EXISTING STAIR WELL



NEW LIFT SHAFT

EMBASSY REDEVELOPMENT 2003

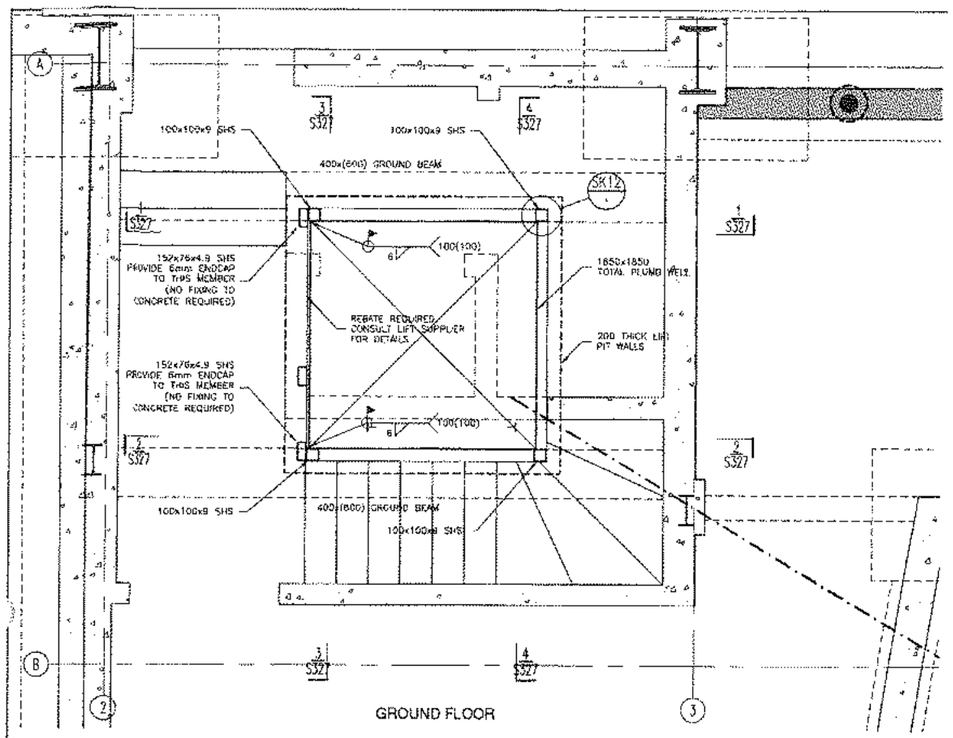
STAIRWELL SETOUT

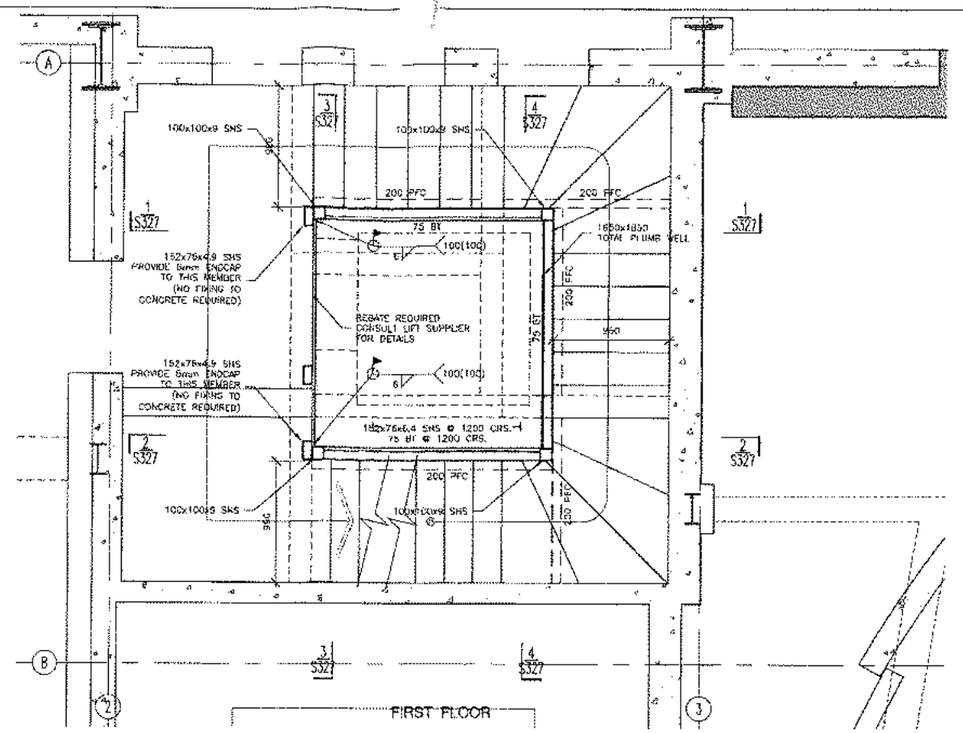
REVISIONS		
B	FOR CONSTRUCTION	29-08-2003
A	PRELIMINARY	4-07-2003

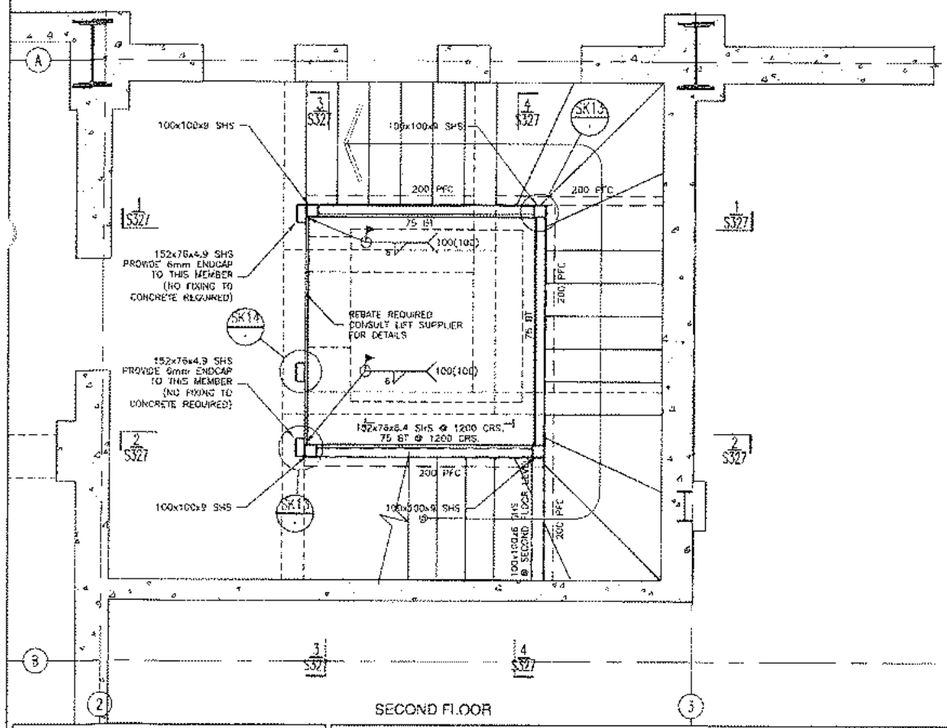
Scales	1:25
A3 Scales	1:50
Designed	Scott MARSHALL
Drawn	Martin WILLIAMS
CAD Reference	3864S325

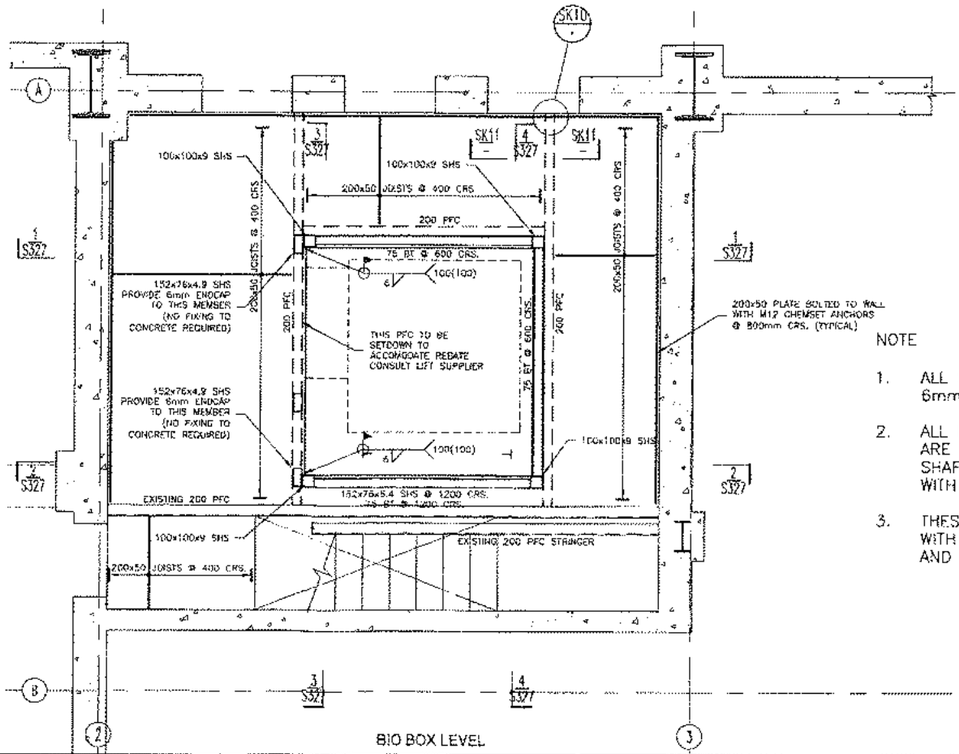
Job Number 3864
Drawing Number S325
Rev 0

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz









NOTE

1. ALL STEEL / STEEL CONNECTIONS TO BE 6mm FILLET WELDS ALL ROUND
2. ALL PFC STAIR SUPPORT MEMBERS ARE TO BE FULLY WELDED TO LIFT SHAFT CORNER MEMBERS WITH 6mm FILLET WELD ALL ROUND
3. THESE DRAWINGS ARE TO BE READ IN CONJUNCTION WITH THE LIFT SUPPLIERS DRAWINGS AND ARCHITECTS DETAILS

EMBASSY REDEVELOPMENT 2003

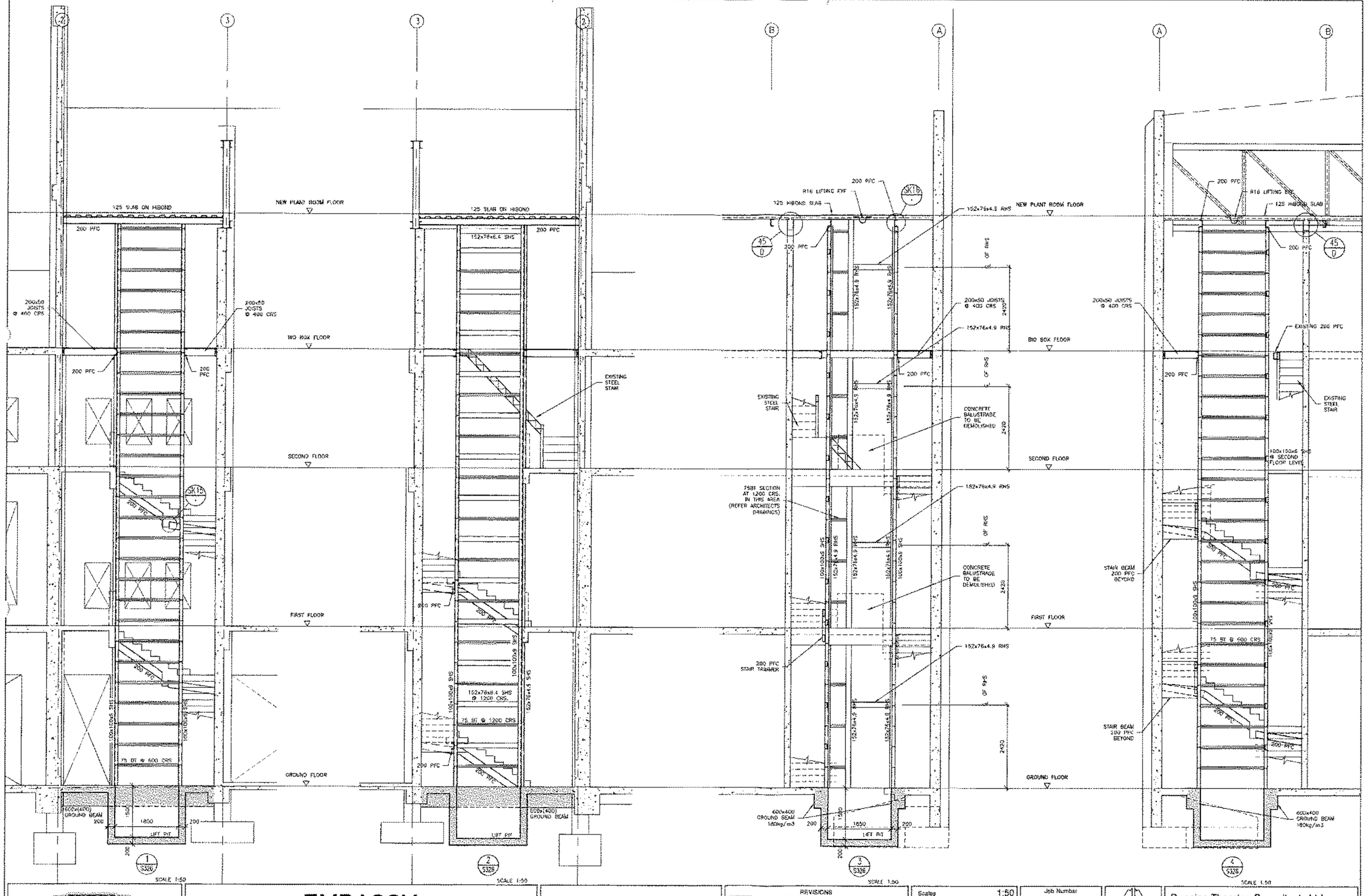
LIFT SHAFT FRAME PLANS

REVISIONS		
3	FOR CONSTRUCTION	20-08-2003
A	PRELIMINARY	4-07-2003

Scales	1:25
A3 Scales	1:50
Designed	Scott MARSHALL
Drawn	Martin WILLIAMS
CAD Reference	3864S325

Job Number 3864
Drawing Number S326
Rev 0

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385 0019
Fax (644) 385-0312
Box 27-153 Wellington, NZ
E-mail elewgen@dunningthornton.co.nz



EMBASSY REDEVELOPMENT 2003

LIFT SHAFT ELEVATIONS

REVISIONS		
B	FOR CONSTRUCTION	20-06-2003
A	PRELIMINARY	4-07-2003

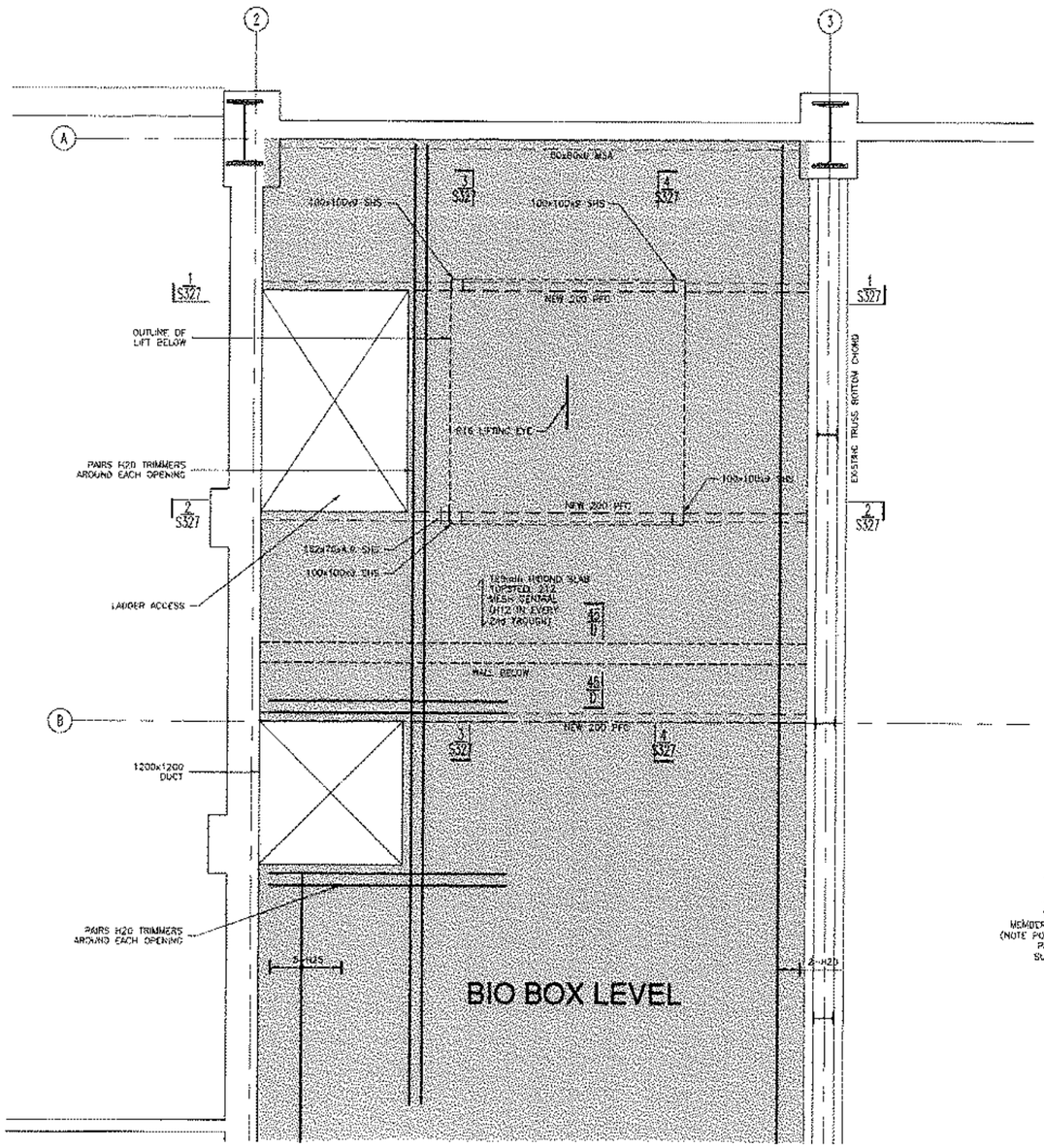
Scales	1:50
A3 Scales	1:100
Designed	Scott MARSHALL
Drawn	Martin WILLIAMS
CAD Reference	3864S325

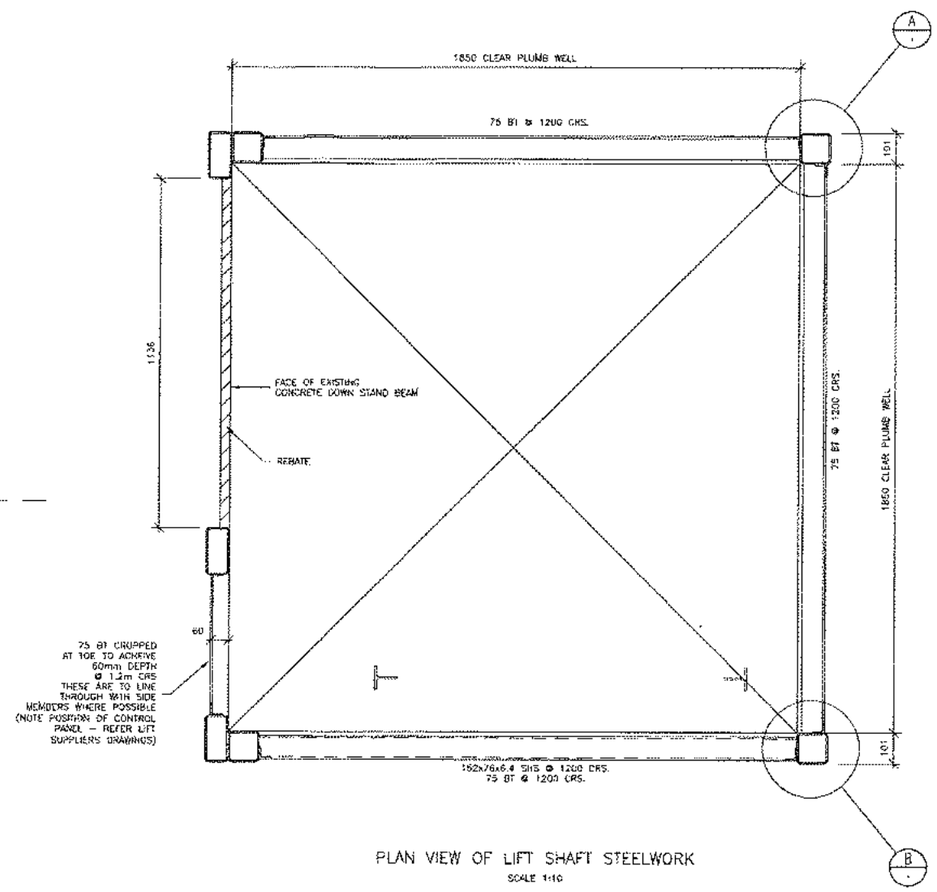
Job Number 3864

Drawing Number S327

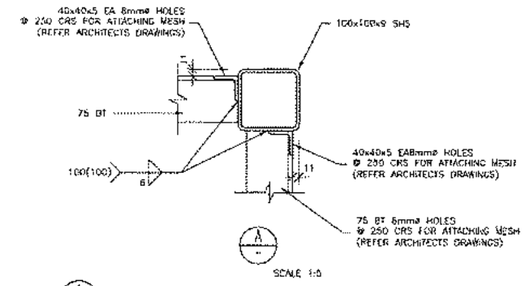
Rev 0

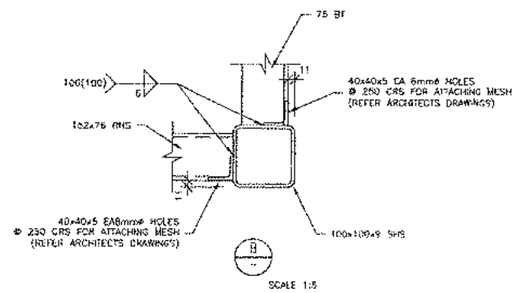
Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-152 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz





PLAN VIEW OF LIFT SHAFT STEELWORK
SCALE 1:10





EMBASSY REDEVELOPMENT 2003

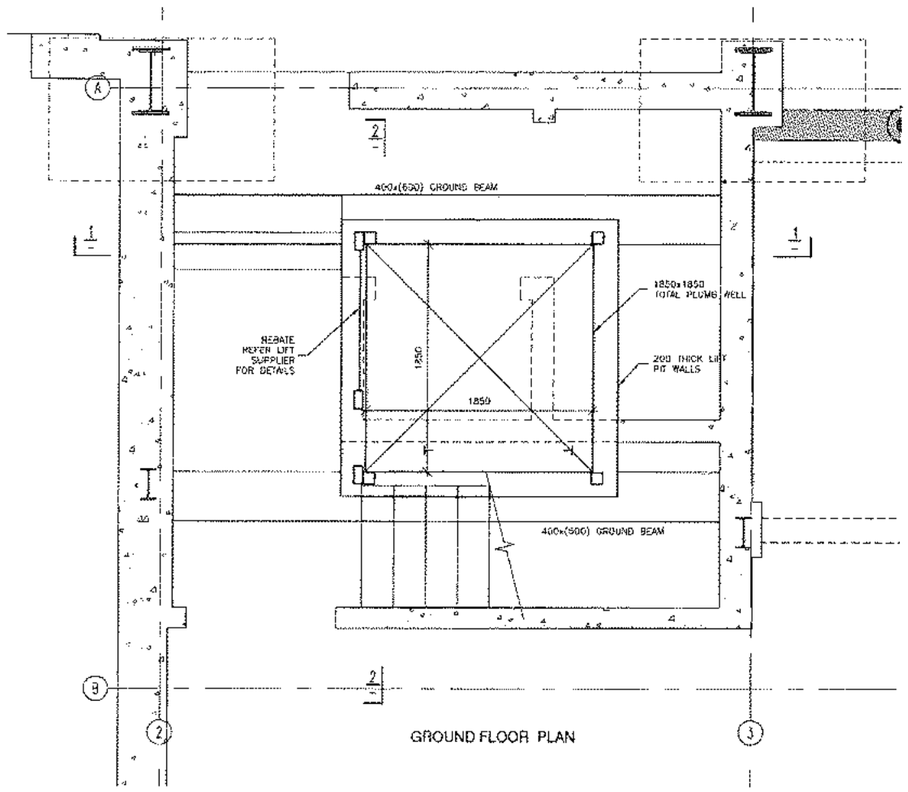
LIFT SHAFT
PLANTROOM SLAB

REVISIONS		
0	FOR CONSTRUCTION	20-08-2003
A	PRELIMINARY	4-07-2003

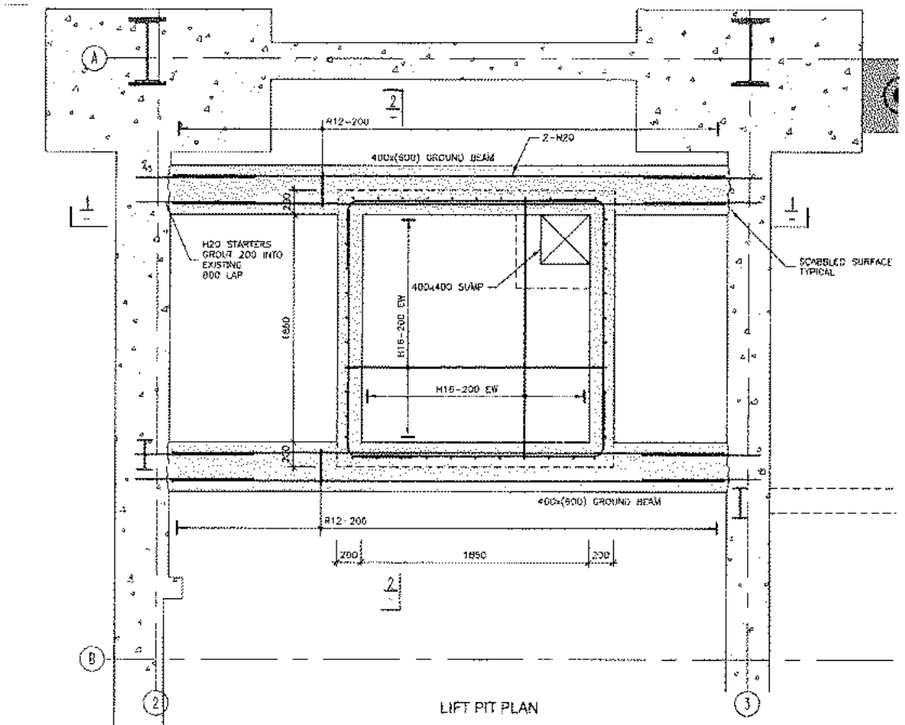
Scales	1:5 1:10 1:25
A3 Scales	1:10 1:20 1:50
Designed	Scott MARSHALL
Drawn	Martin WILLIAMS
CAD Reference	3864S325

Job Number 3864
Drawing Number S328
Rev 0

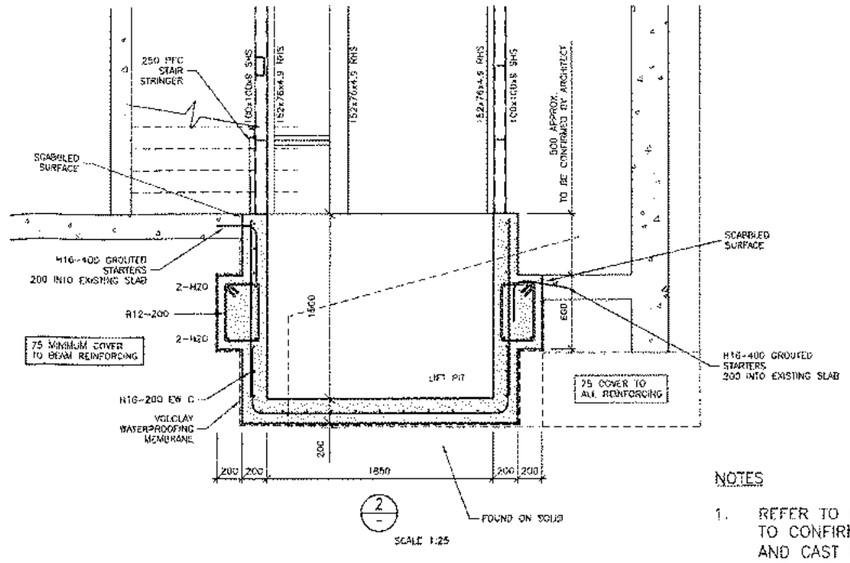
Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385 0019
Fax (644) 385 0312
Box 27-158 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz



GROUND FLOOR PLAN



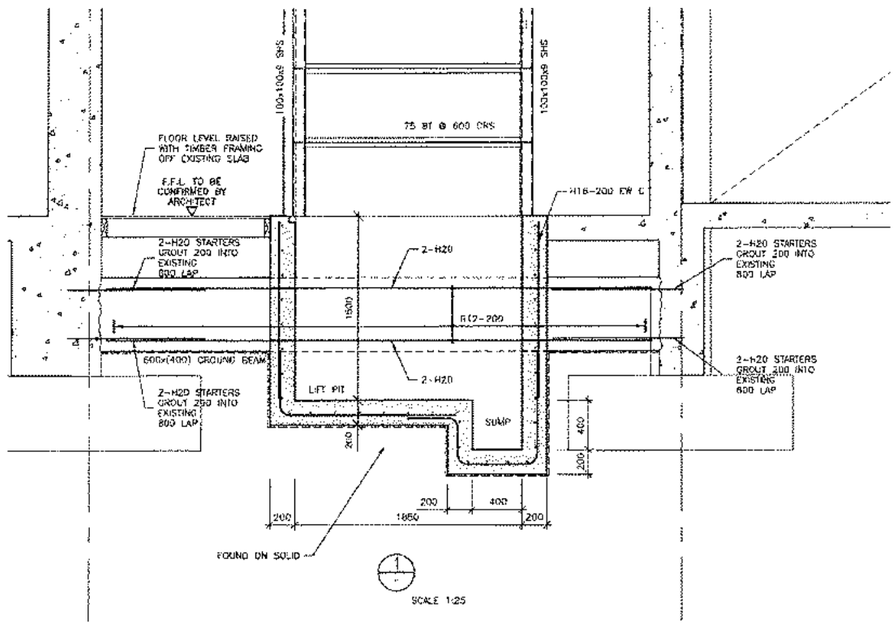
LIFT PIT PLAN



SCALE 1:25

NOTES

1. REFER TO LIFT SUPPLIERS DRAWINGS TO CONFIRM ALL DIMENSIONS AND CAST IN'S
2. LOCATION OF EXISTING FOUNDATIONS TO BE CONFIRMED ON SITE



SCALE 1:25

G:\3864\DtcDwgs\3864S330.dwg, 27/08/2003 16:09:35

EMBASSY THEATRE

EMBASSY REDEVELOPMENT 2003

LIFT PIT

REVISIONS		
0	FOR CONSTRUCTION	26-08-2003

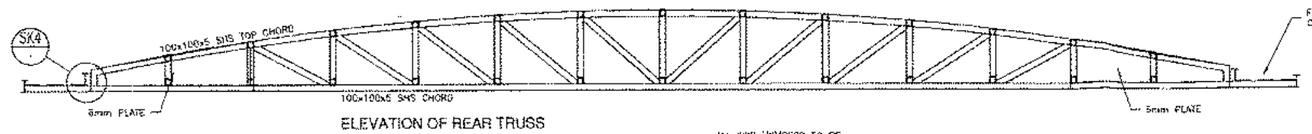
Scales	1:25
A3 Scales	1:50
Designed	Scott Marshall
Drawn	Martin Williams
CAD Reference	3864S330

Job Number 3864

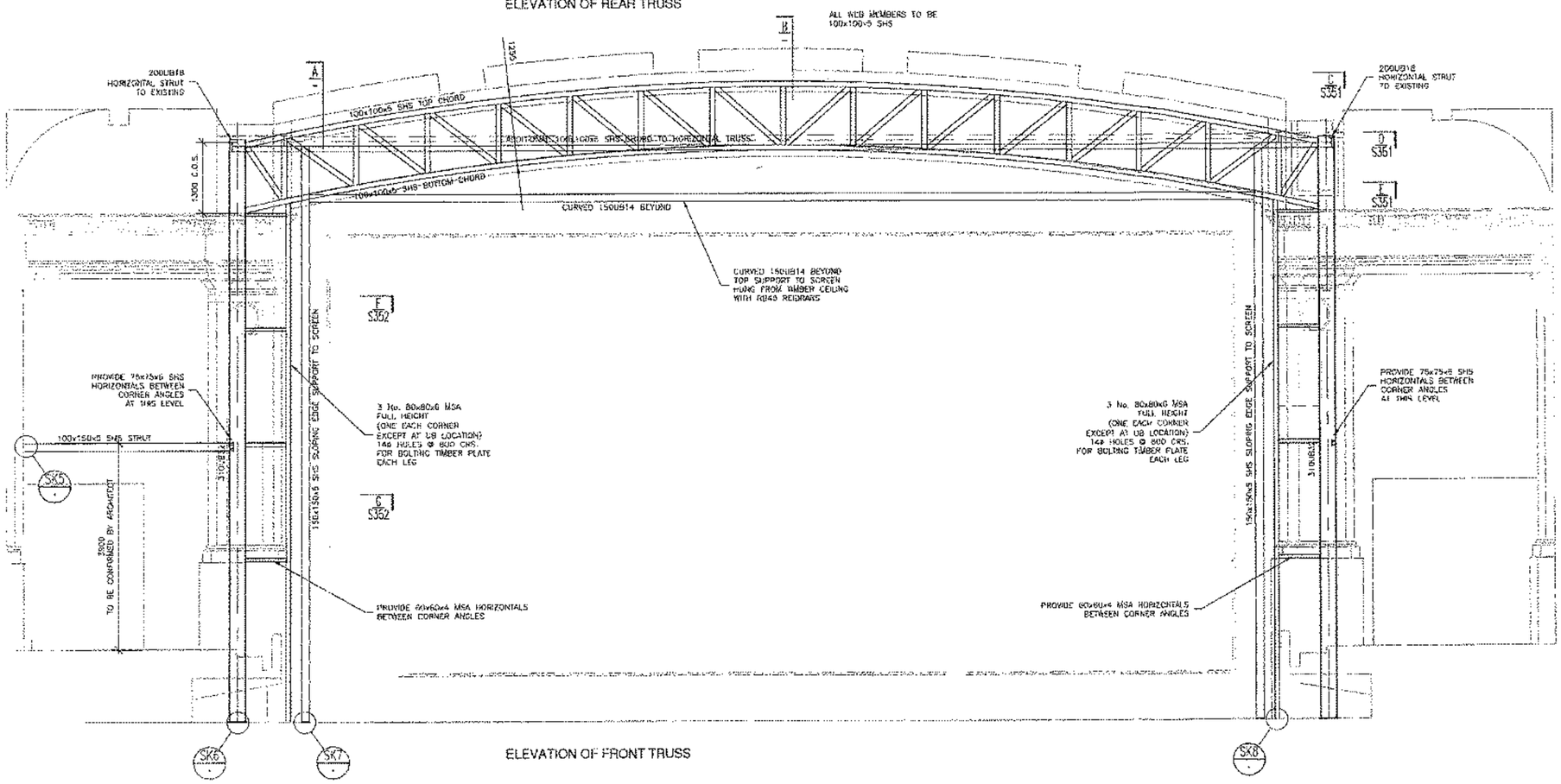
Drawing Number S330 Rev 0

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-6019
Fax (644) 385-6312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz

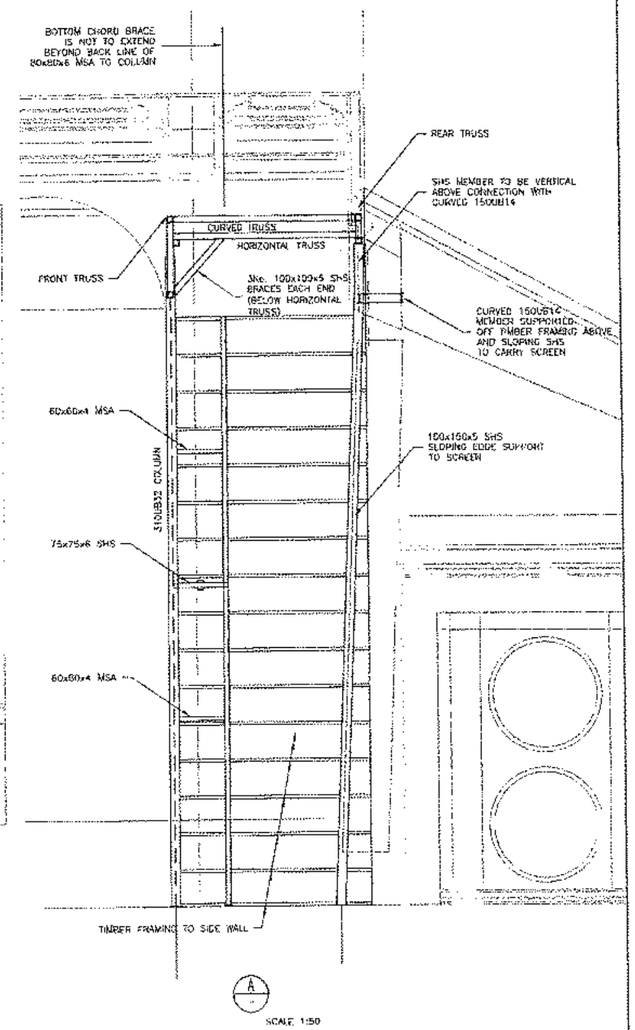
ALL STEEL/STEEL CONNECTIONS ARE TO BE 6mm F.W.A.R. UNLESS NOTED OTHERWISE

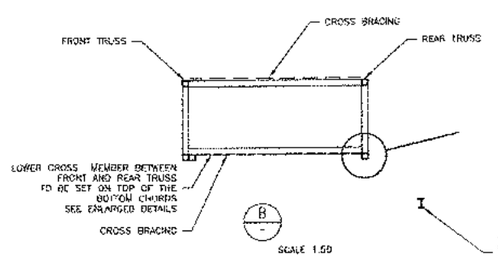


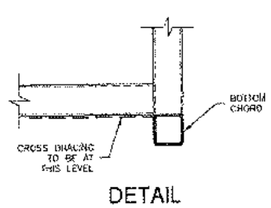
ELEVATION OF REAR TRUSS



ELEVATION OF FRONT TRUSS







DETAIL

CURVED 150UB14 MEMBER SUPPORTED OFF TIMBER FRAMING ABOVE TO CARRY SCREEN

EMBASSY REDEVELOPMENT 2003

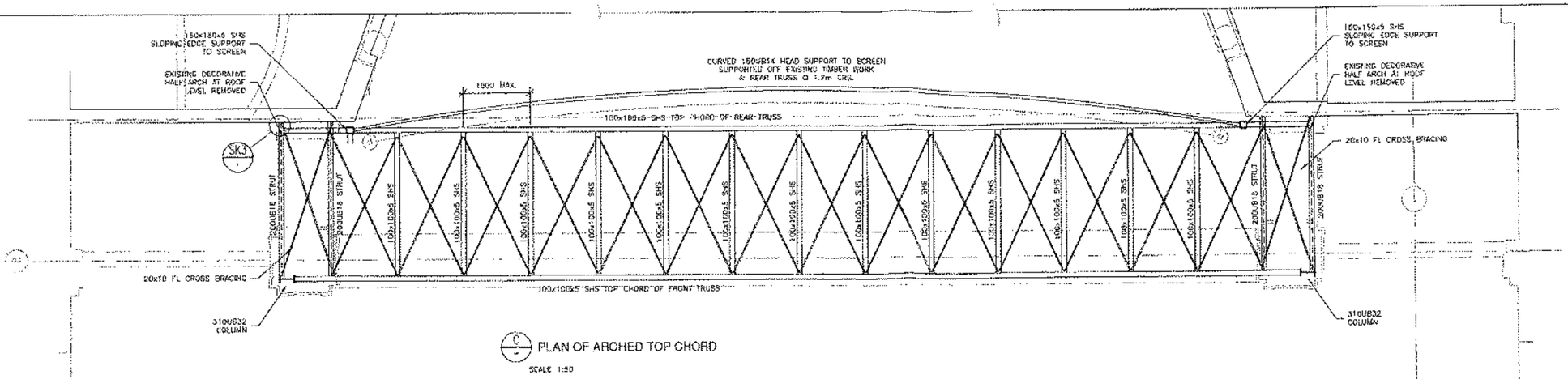
PROCENIUM ARCH ELEVATION

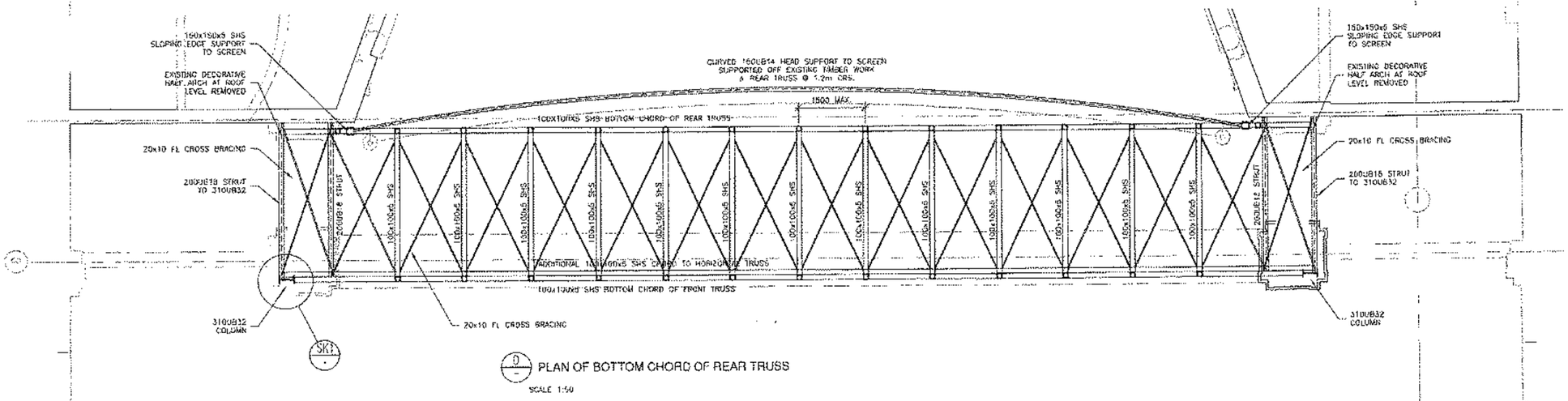
REVISIONS		
0	FOR CONSTRUCTION	18-08-2003
B	REVISED	14-08-2003
A	FOR PRICING	6-08-2003

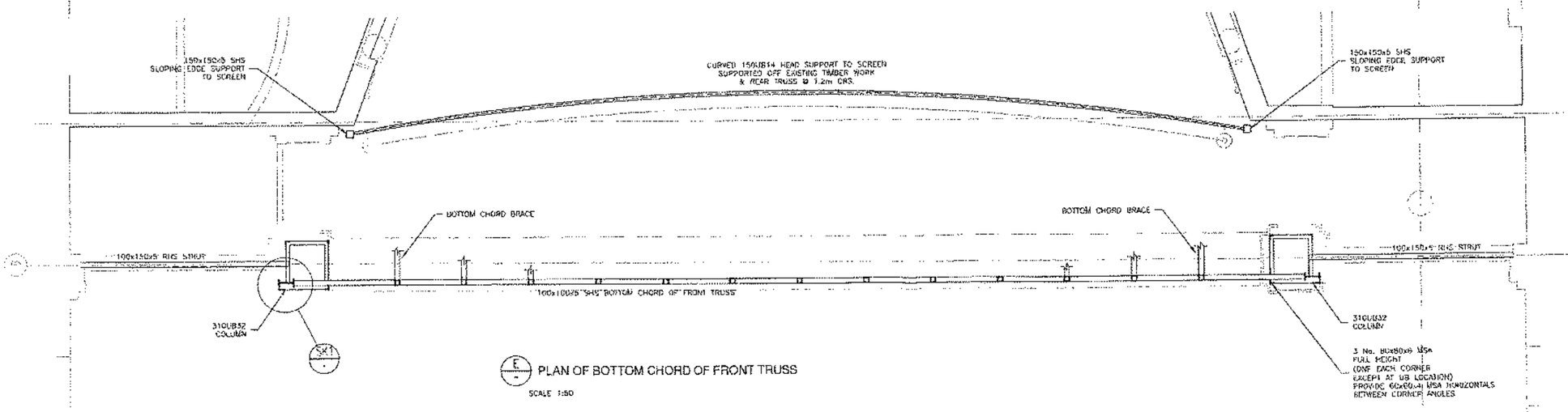
Scales	1:50
A3 Scales	1:100
Designed	Scott Marshall
Drawn	Martin Williams
CAD Reference	3864S350

Job Number 3864
Drawing Number S350
Rev 0

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z
E-mail str.wgtn@dunningthornton.co.nz







EMBASSY REDEVELOPMENT 2003

PROCENIUM ARCH TRUSS PLANS

REVISIONS		
B	FOR CONSTRUCTION	19-06-2003
B	REVISED	14-06-2003
A	FOR PRICING	6-06-2003

Scales	1:50
A3 Scales	1:100
Designed	Scott Marshall
Drawn	Martin Williams
CAD Reference	3864S350

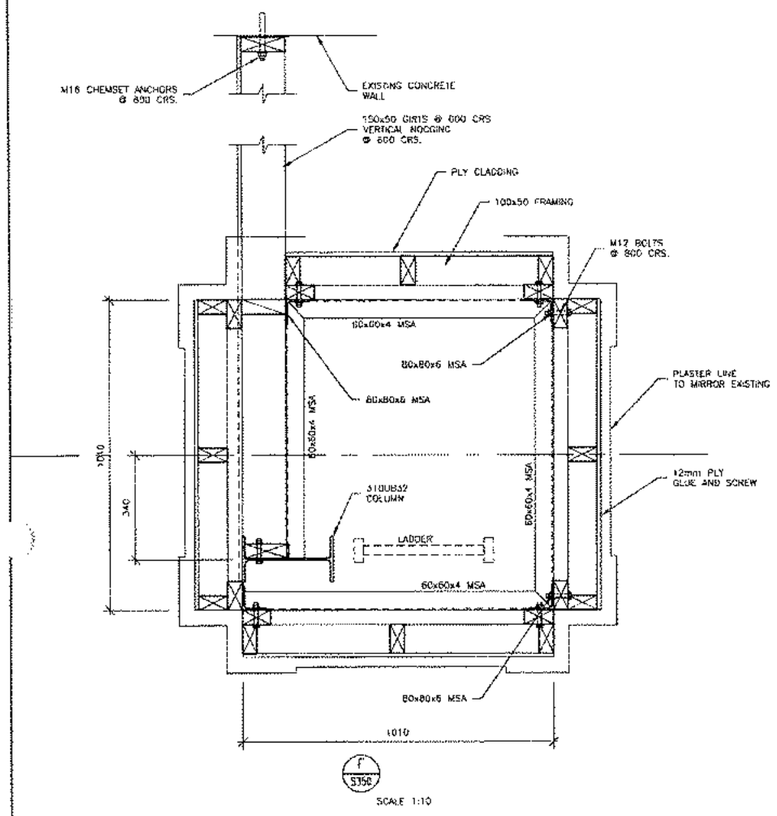
Job Number 3864

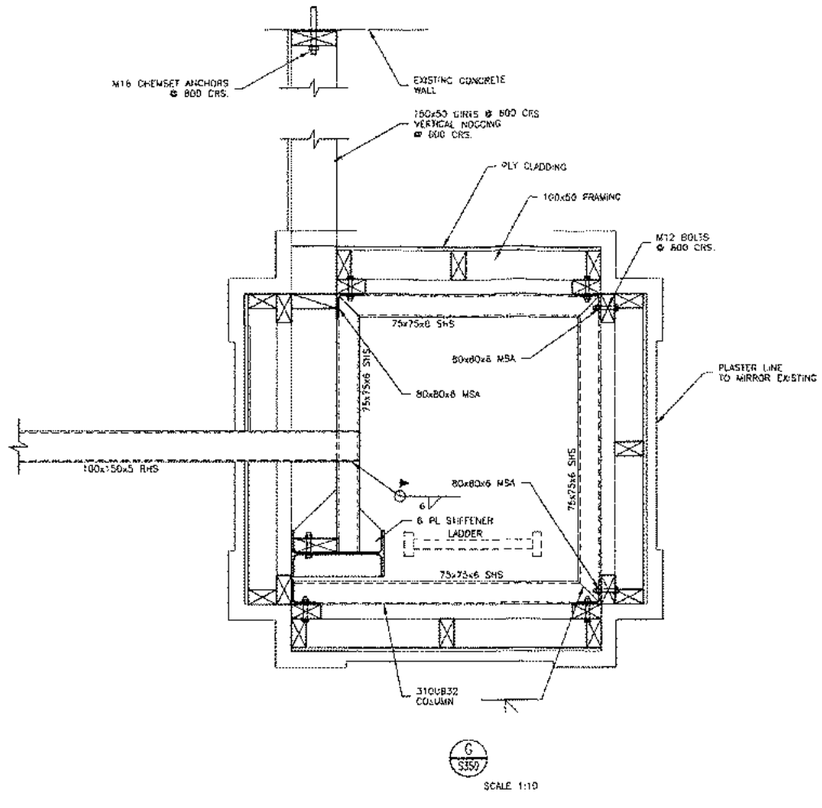
Drawing Number S351

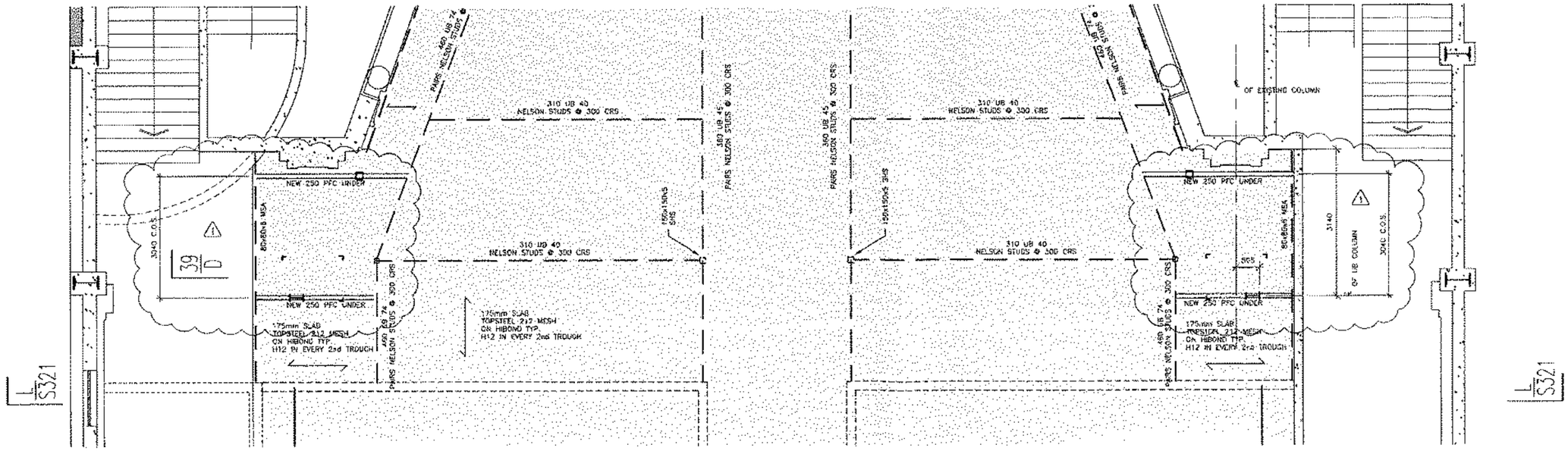
Rev 0

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-159 Wellington, N.Z.
E-mail etowgtn@dunningthornton.co.nz







ADDITIONAL SLAB STEELWORK

EMBASSY THEATRE	EMBASSY REDEVELOPMENT 2003	PROCENIUM ARCH SLAB PLAN AND DETAILS

REVISIONS		
1	REVISED	4-09-2003
0	FOR CONSTRUCTION	15-08-2003
B	REVISED	14-08-2003
A	FOR PRICING	6-08-2003

Scales	1:10 1:50
A3 Scales	1:20 1:100
Designed	Scott Marshall
Drawn	Martin Williams
CAD Reference	3864S350

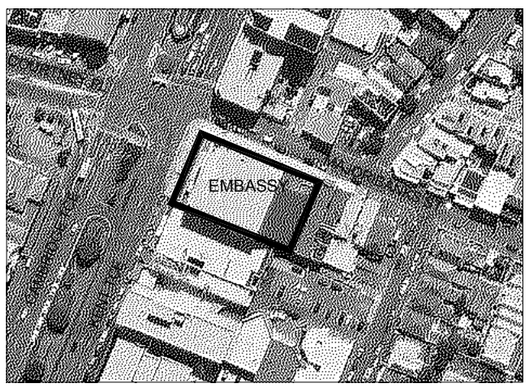
Job Number 3864
Drawing Number S352
Rev 1

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtowgtn@dunlingthornton.co.nz

EMBASSY
STRENGTHENING 2009

STRUCTURAL DRAWINGS

FOR CONSTRUCTION



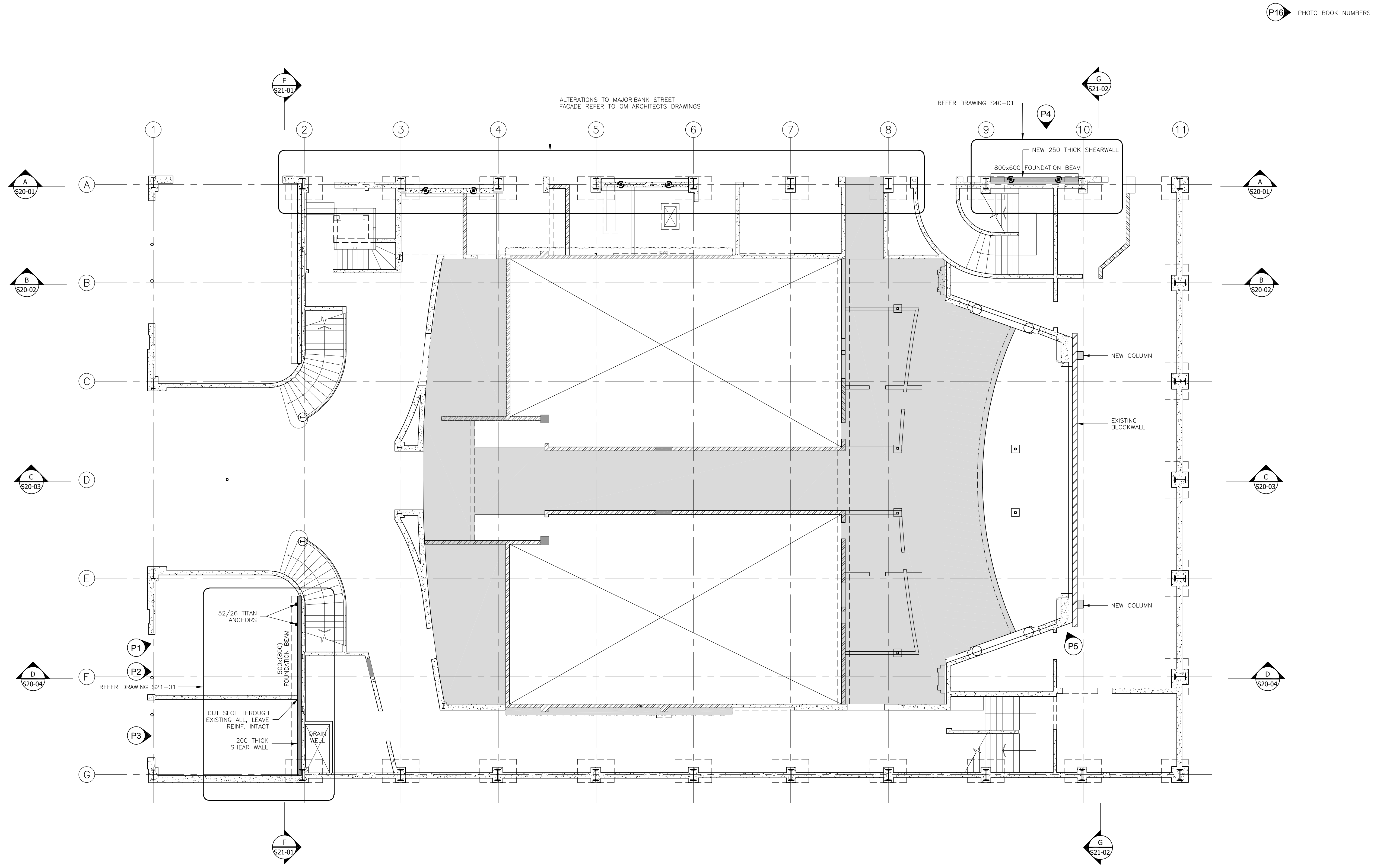
SITE PLAN

Dunning Thornton consultants

Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz
URL www.dunningthornton.co.nz

Job No. 3864

Original Size A1



Original Scale

Original Size A1

ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

LEVEL 0
SLAB PLAN

	REVISIONS	
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Julie nicholson
CAD Reference	3864-2009

Job Number 3864
Drawing Number S2-01
Rev 1

Dunning Thornton consultants

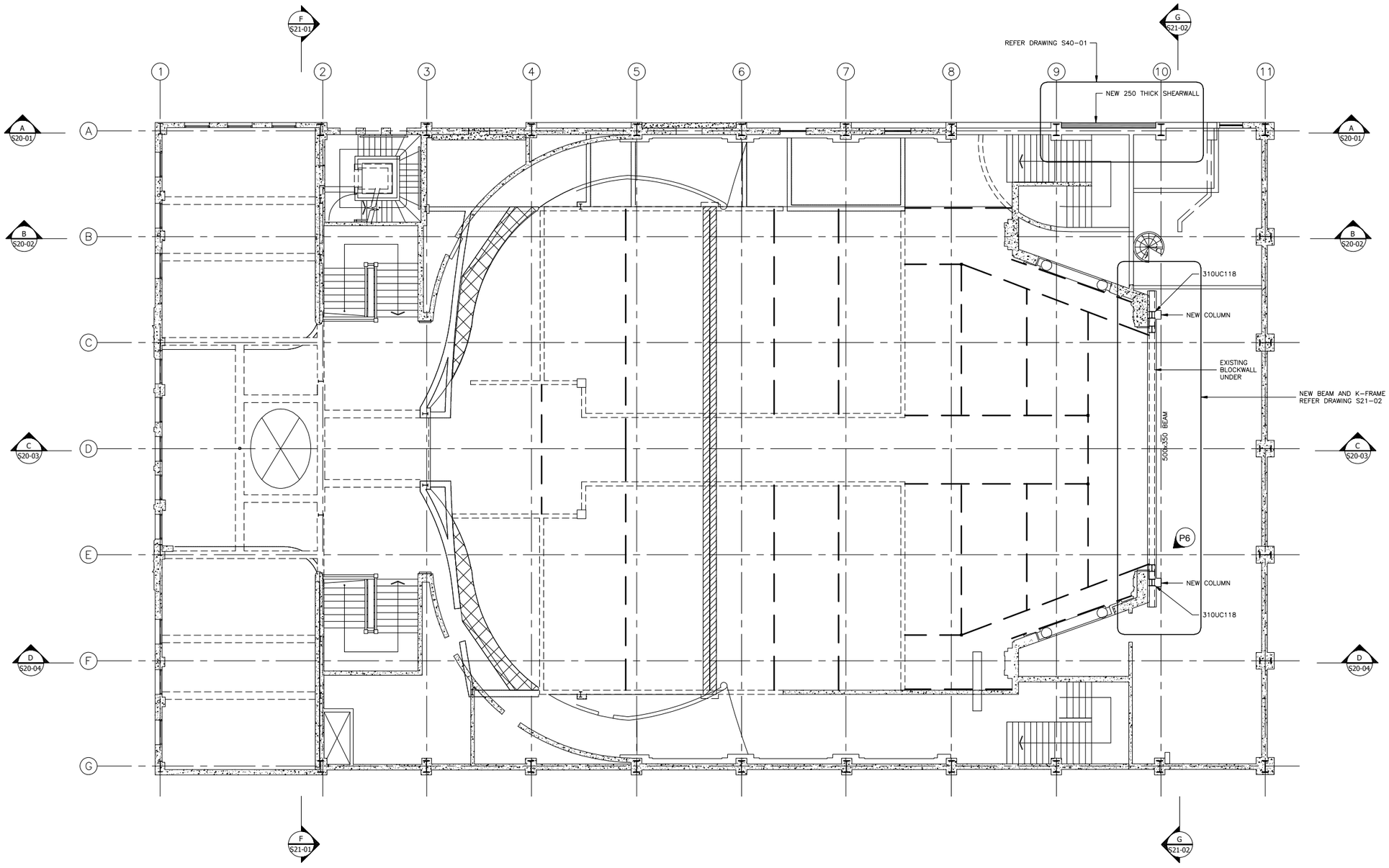
p: 385 0019 f: 385 0312 e: dtcwgtn@dunningthornton.co.nz P.O. Box 27-153 Wellington

IF IN DOUBT ASK

DO NOT SCALE

Original Size A1

P16 PHOTO BOOK NUMBERS



Original Size A1 Original Scale 100 50 0 10

ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

LEVEL 1
SLAB PLAN

REVISIONS		
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Julie nicholson
CAD Reference	3864-2009

Job Number 3864
Drawing Number S3-01
Rev 1

ACENZ IANZ JAS-ANZ ISO 9001

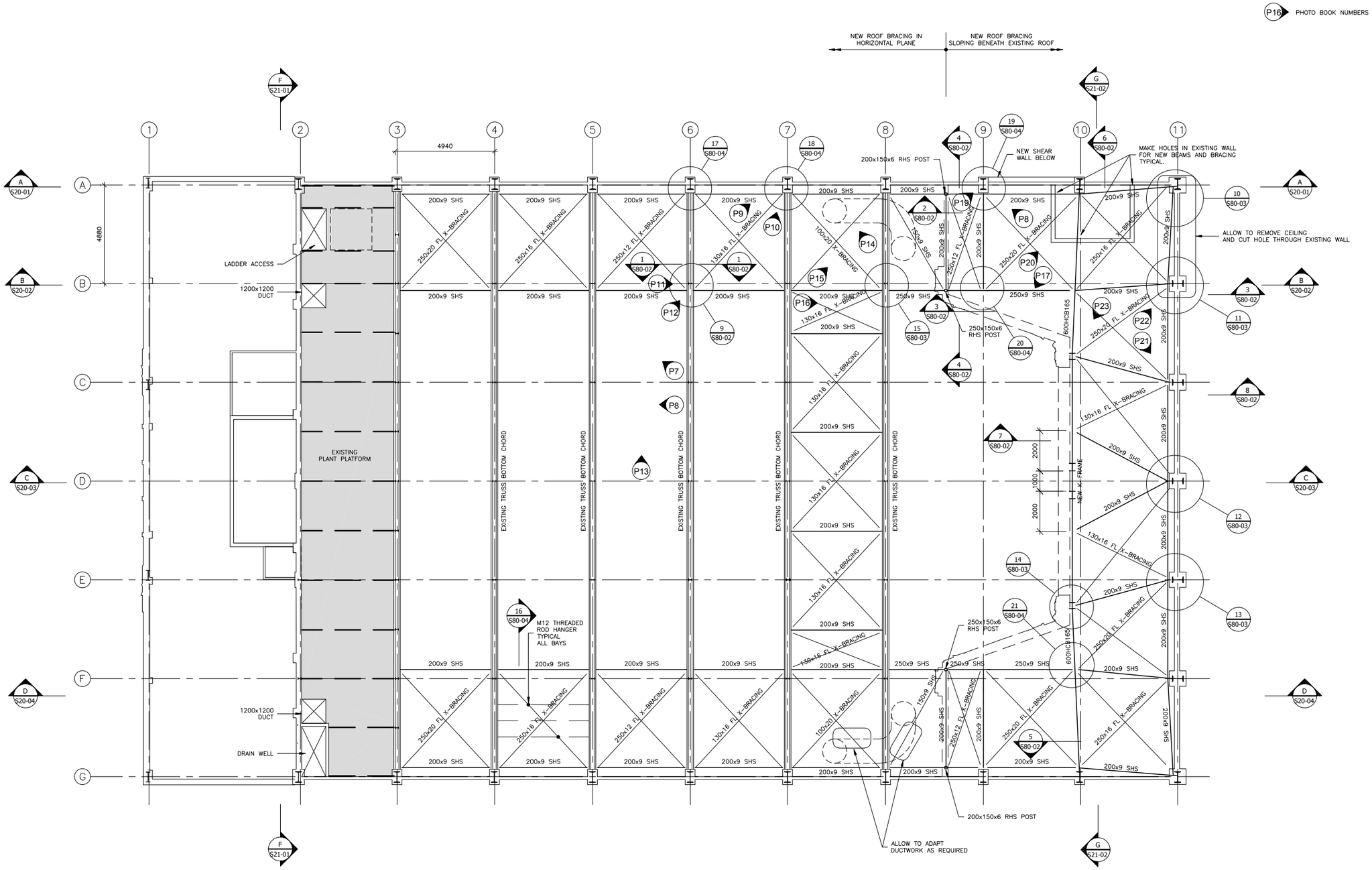
Dunning Thornton consultants

IF IN DOUBT ASK

DO NOT SCALE

p: 385 0019 f: 385 0312 e: dtlowgin@dunningthornton.co.nz P.O. Box 27-153 Wellington

Original Size A1



NOTE:
> 120mm FLAT TO BE GRADE 350

ABSOLUTELY POSITIVELY WELLINGTON	
EMBASSY STRENGTHENING 2009	
BOTTOM CHORD ROOF BRACING PLAN	

REVISIONS		
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Julie NICHOLSON
CAD Reference	3864S300

Job Number 3864
Drawing Number S4-01
Rev 1

ISO 9001

Dunning Thornton consultants

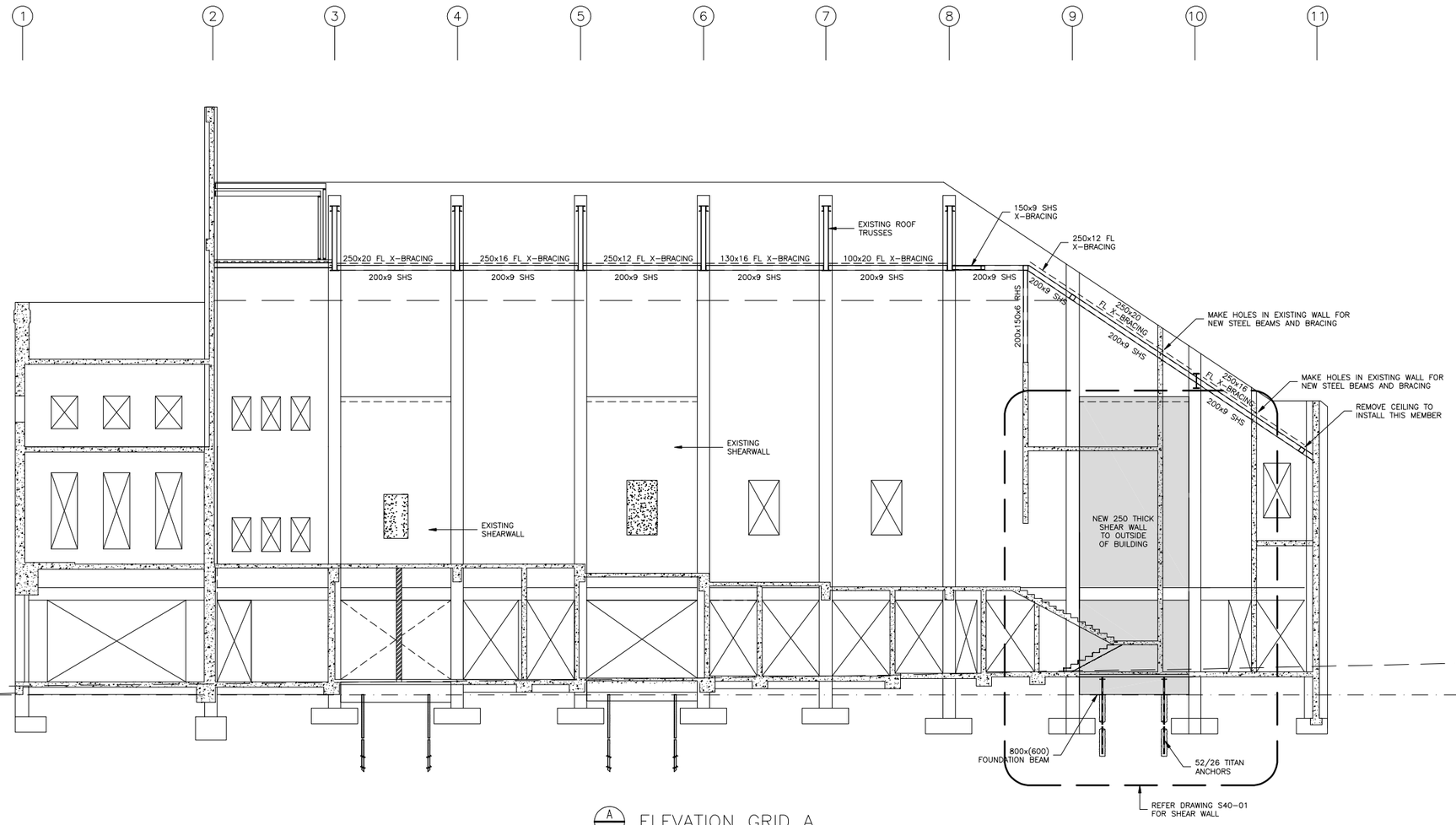
IF IN DOUBT ASK

DO NOT SCALE

p: 385 0019 f: 385 0312 e: dtcwgtn@dunningthornton.co.nz P.O. Box 27-153 Wellington

Original Size A1 Original Scale 100 50 0 10

Original Size A1



A ELEVATION GRID A

SCALE 1:100

Original Size A1 Original Scale

100 50 0 10

G:\3864_Drawings\Drawings_2009\dwg, 12/04/2010 8:40:36 a.m., 1:1

ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

ELEVATION GRID A

REVISIONS		
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Julie Nicholson
CAD Reference	3864-2009

Job Number 3864

Drawing Number S20-01 Rev 1

ISO 9001

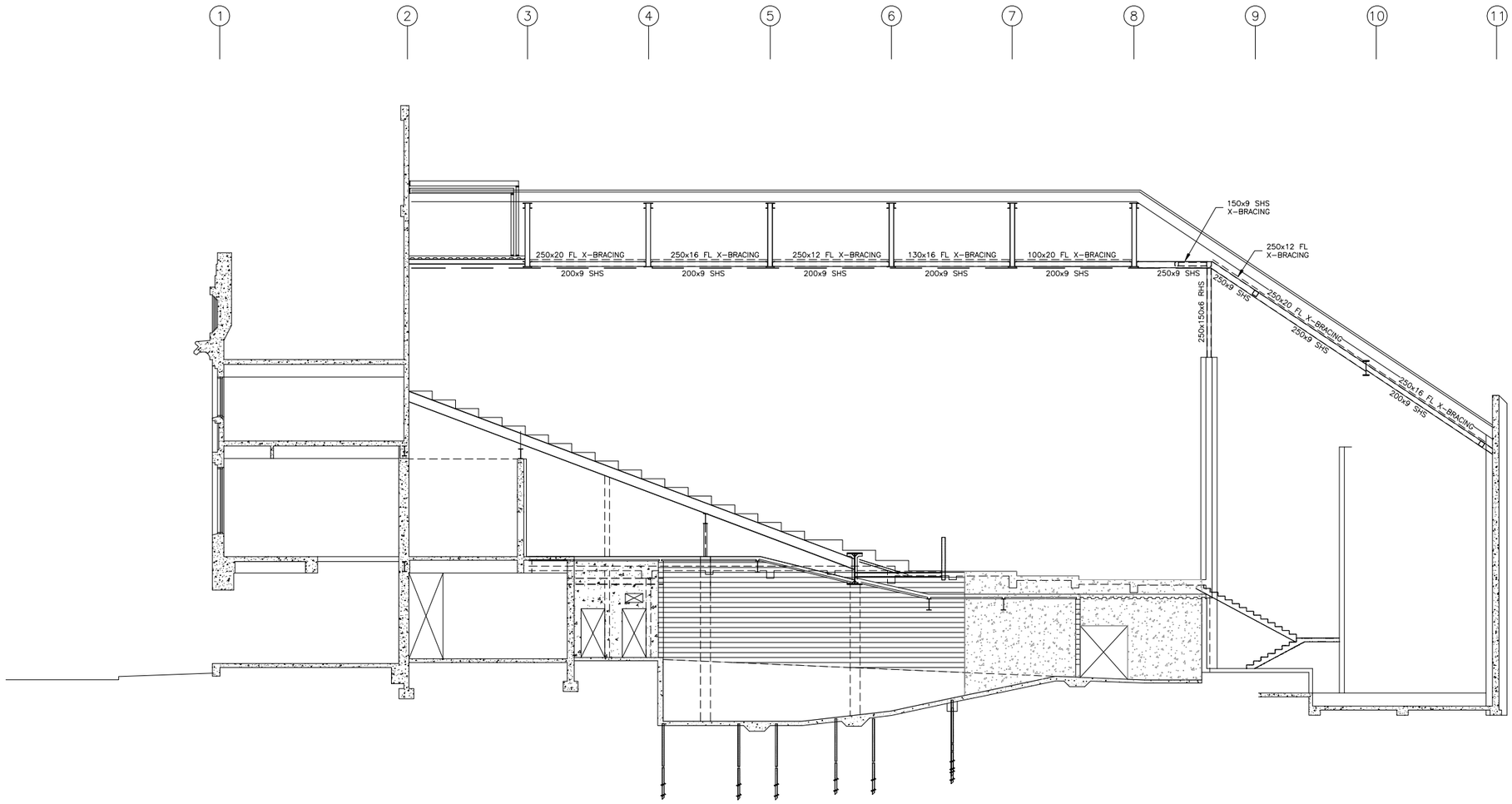
Dunning Thornton consultants

p: 385 0019 f: 385 0312 e: dtcwgtn@dunningthornton.co.nz P.O. Box 27-153 Wellington

IF IN DOUBT ASK

DO NOT SCALE

Original Size A1



B SECTION GRID B

SCALE 1:100

Original Size A1 Original Scale 100 50 0 10

G:\3864_Drawings\dwgs_2009\dwg_S20-02.dwg, 12/24/2010 6:46:39 p.m., 1:1

ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

SECTION GRID B

REVISIONS		
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Julie NICHOLSON
CAD Reference	3864-2009

Job Number 3864

Drawing Number S20-02

Rev 1

ISO 9001

Dunning Thornton consultants

p: 385 0019 f: 385 0312 e: dtowgtn@dunningthornton.co.nz P.O. Box 27-153 Wellington

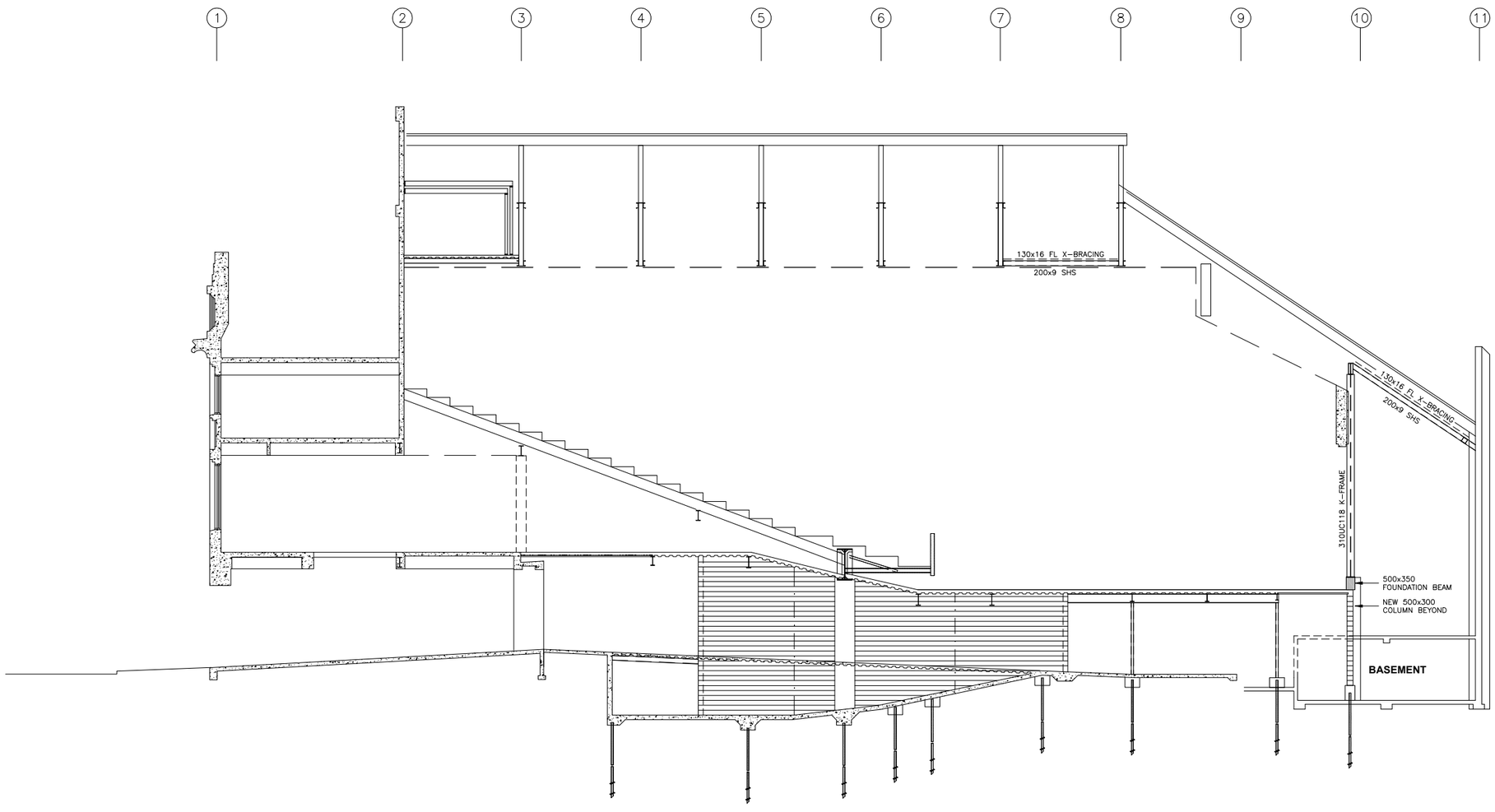
IF IN DOUBT ASK

DO NOT SCALE

Original Size A1

100 50 0 10 Original Scale

Original Size A1



C SECTION GRID D
SCALE 1:100

ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

SECTION GRID D

REVISIONS		
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Julie NICHOLSON
CAD Reference	3864-2009

Job Number 3864

Drawing Number S20-03 Rev 1

ISO 9001

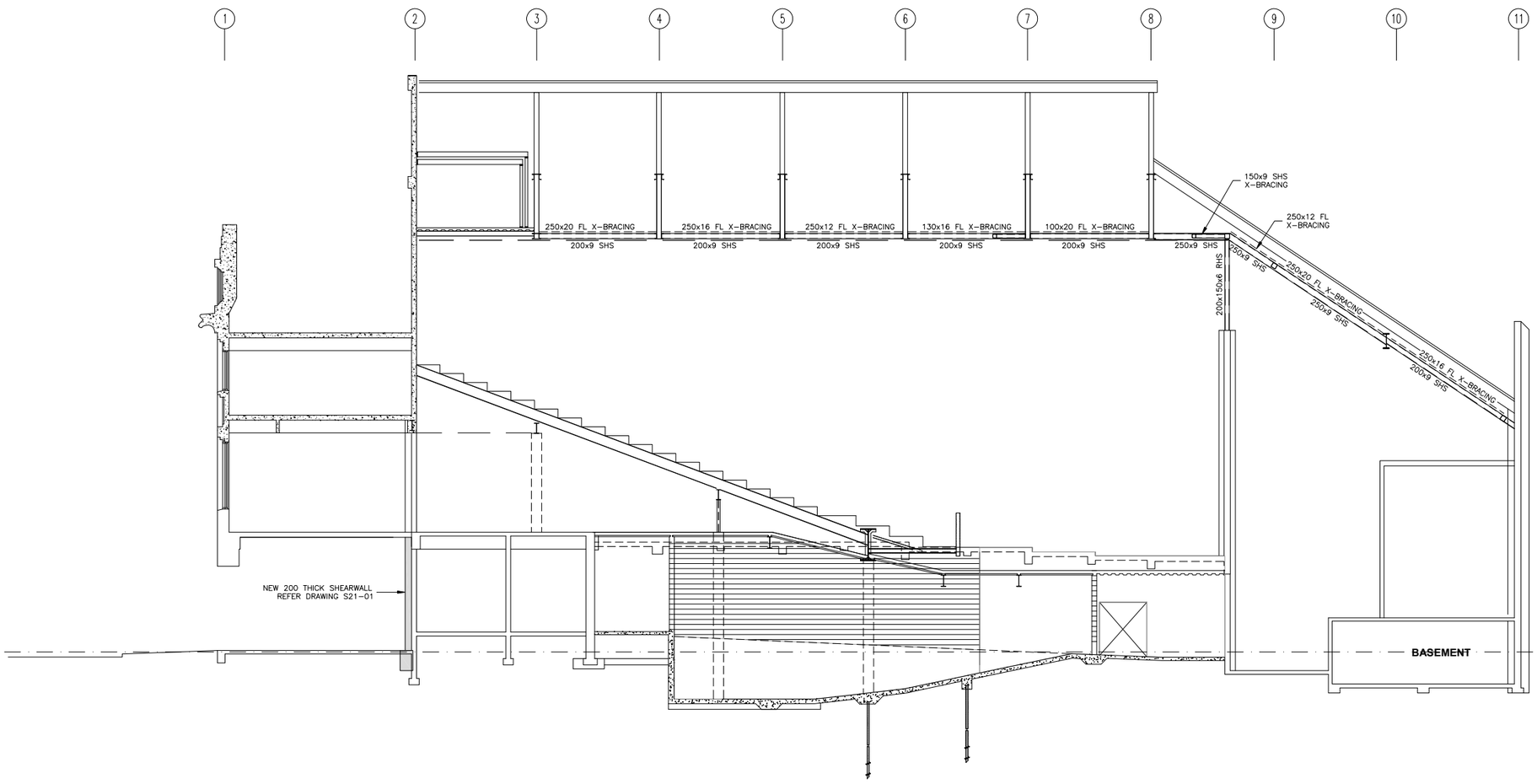
Dunning Thornton consultants

IF IN DOUBT ASK

DO NOT SCALE

p: 385 0019 f: 385 0312 e: dtcwgtn@dunningthornton.co.nz P.O. Box 27-153 Wellington

Original Size A1



D SECTION GRID F
SCALE 1:100

100 50 0 10

Original Scale

Original Size A1

G:\3864_Drawings\drawings_2009.dwg, 12/04/2010 8:40:41 a.m., 1:1

ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

SECTION GRID F

REVISIONS		
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Julie NICHOLSON
CAD Reference	3864-2009

Job Number 3864

Drawing Number S20-04

Rev 1

ISO 9001

Dunning Thornton consultants

p: 385 0019 f: 385 0312 e: dtcwgtn@dunningthornton.co.nz P.O. Box 27-153 Wellington

IF IN DOUBT ASK

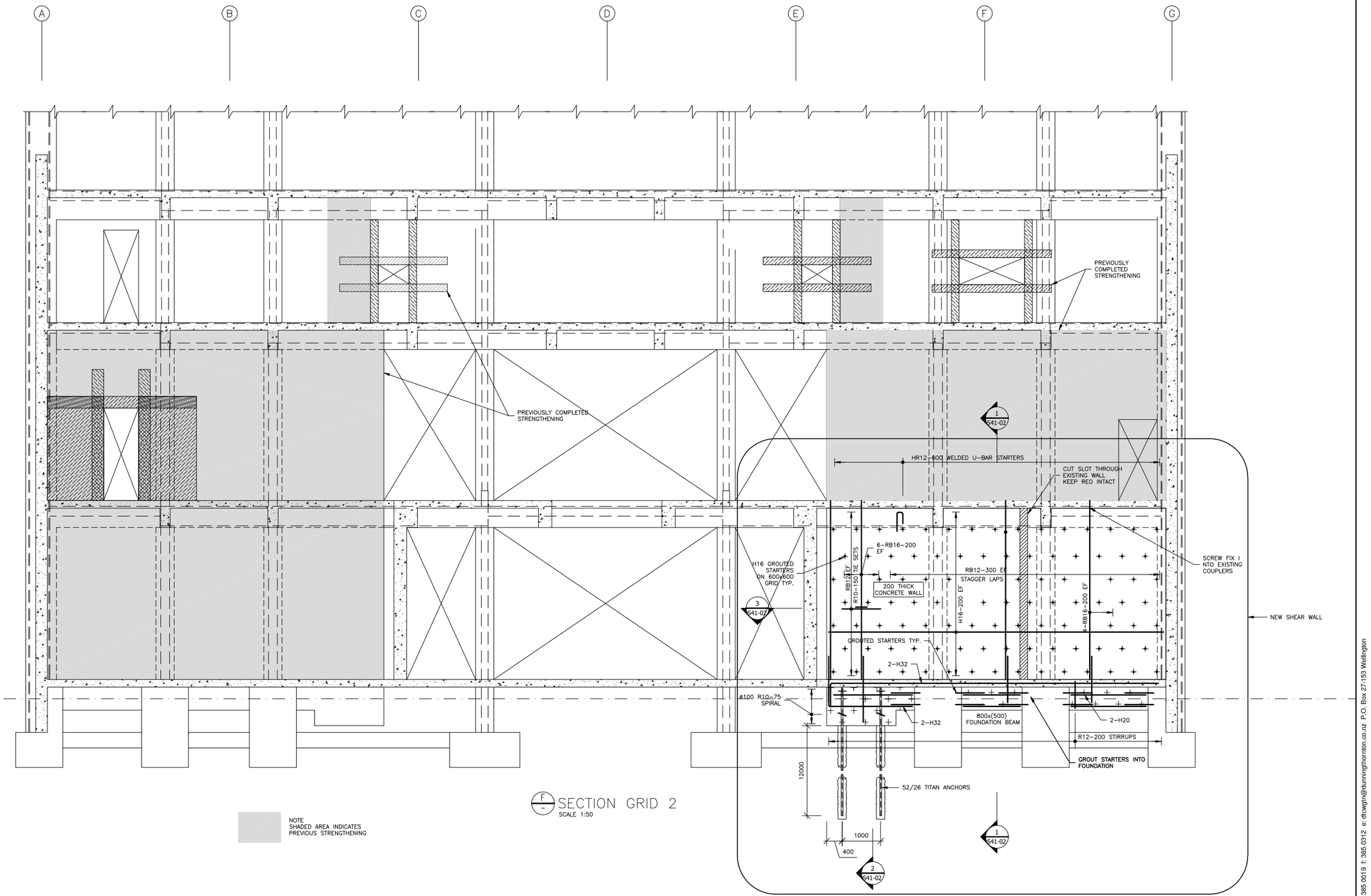
DO NOT SCALE

Original Size A1

100 50 0 10

Original Scale

Original Size A1



F SECTION GRID 2
SCALE 1:50

NOTE
SHADED AREA INDICATES PREVIOUS STRENGTHENING

ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

SECTION GRID 2

REVISIONS		
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Julie NICHOLSON
CAD Reference	3864-2009

Job Number 3864

Drawing Number S21-01 Rev 1

ISO 9001

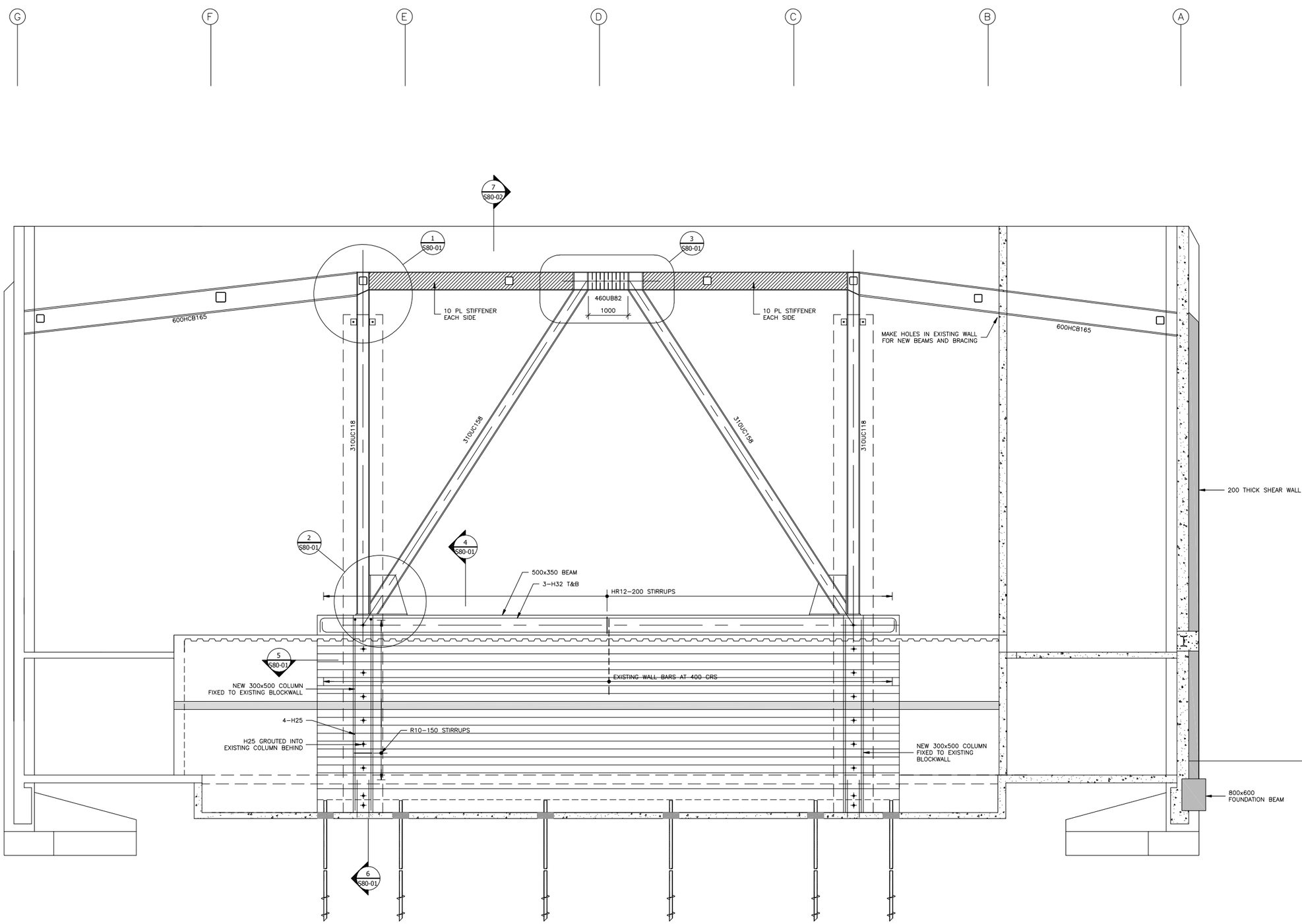
Dunning Thornton consultants

p: 385 0019 f: 385 0312 e: dtowgtn@dunningthornton.co.nz P.O. Box 27-153 Wellington

IF IN DOUBT ASK

DO NOT SCALE

Original Size A1



100 50 0 10

Original Scale

Original Size A1

G:\3864_Drawings\dcs\engs_2009.dwg, 10/24/2010 8:46:49 a.m., 1:1

ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

SECTION GRID H

REVISIONS		
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Julie NICHOLSON
CAD Reference	3864-2009

Job Number 3864

Drawing Number S21-02

Rev 1

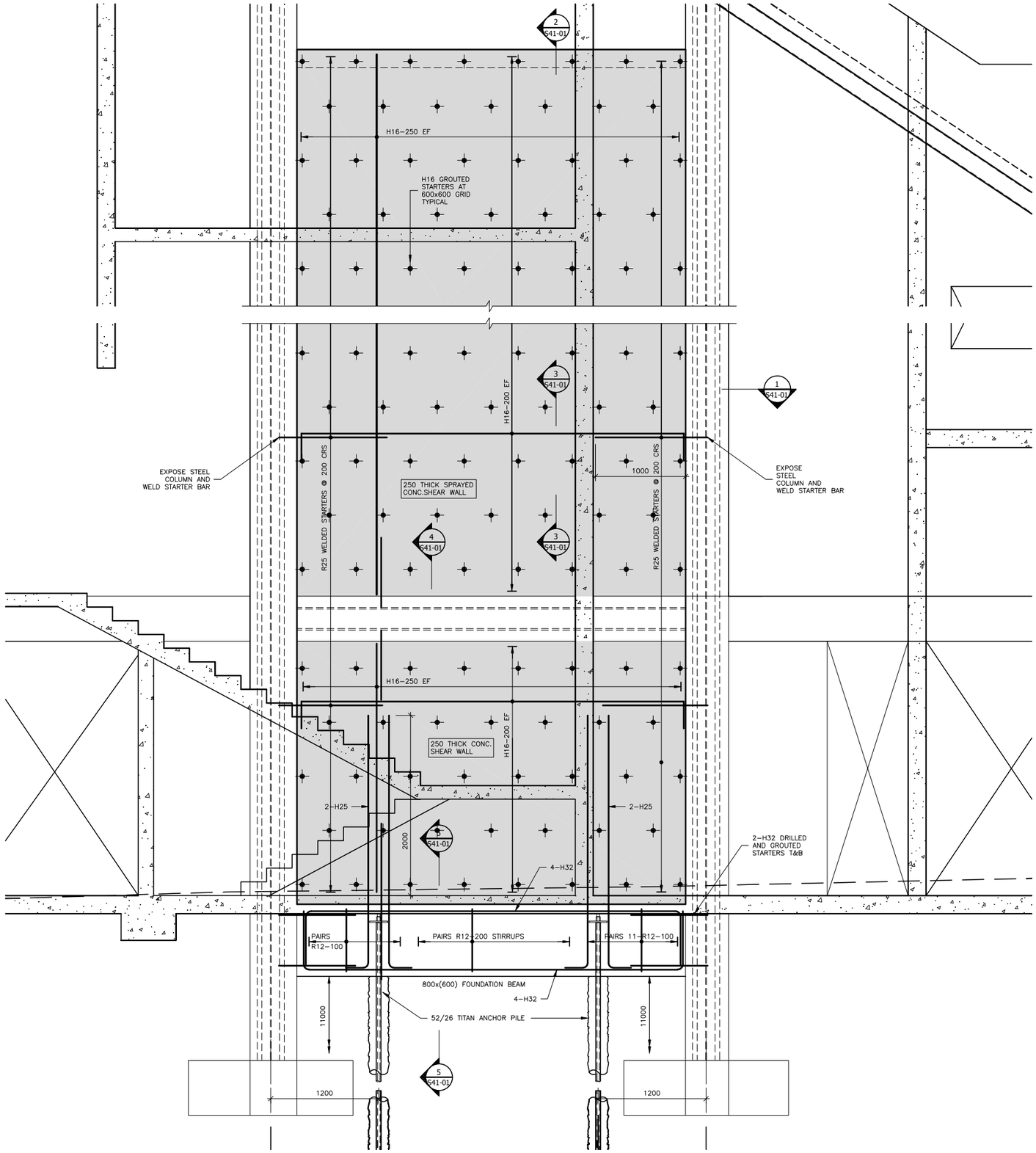
ISO 9001

Dunning Thornton consultants

p: 385 0019 f: 385 0312 e: dtcwgtn@dunningthornton.co.nz P.O. Box 27-153 Wellington

IF IN DOUBT ASK

DO NOT SCALE



ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

GRID A
SHEARWALL

REVISIONS		
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:25
A3 Scales	1:50
Designed	Adam Thornton
Drawn	Julie NICHOLSON
CAD Reference	3864-2009

Job Number 3864
Drawing Number S40-01
Rev 1

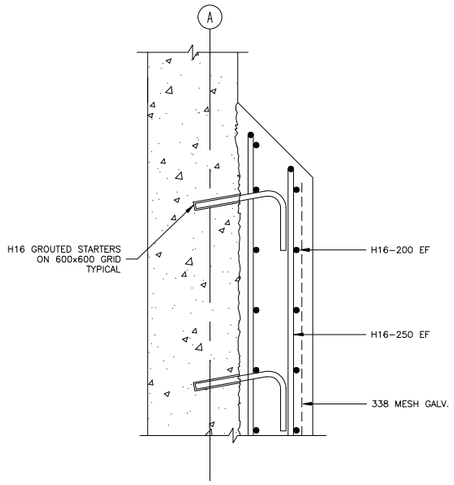
ISO 9001

Dunning Thornto consultan

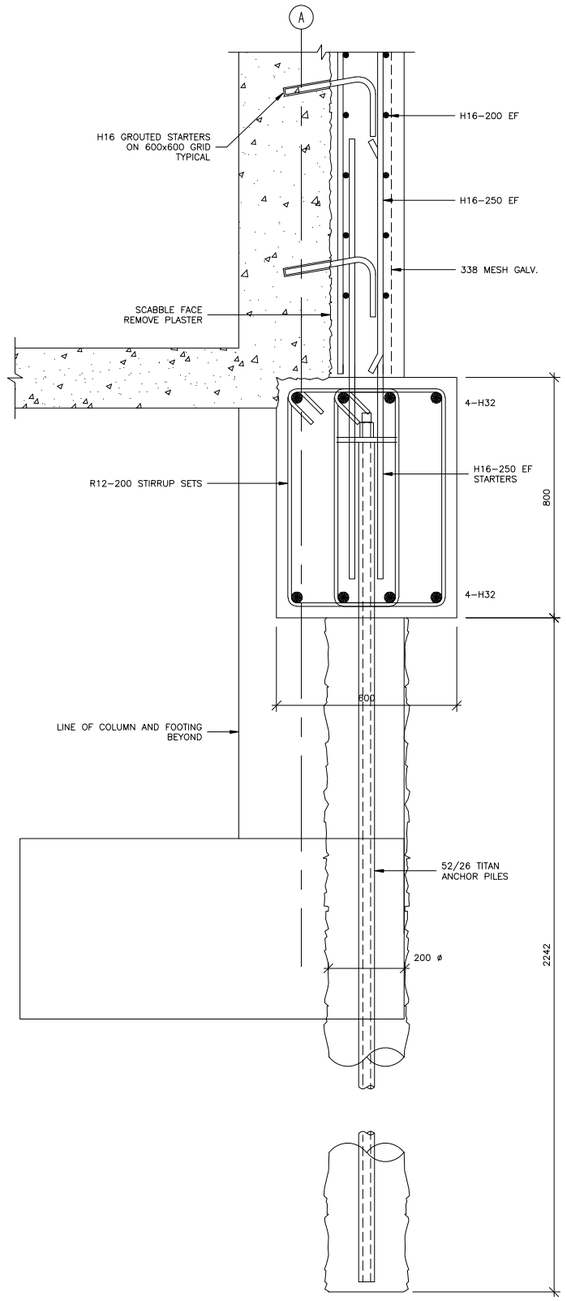
IF IN DOUBT ASK

DO NOT SCALE

Original Size A1

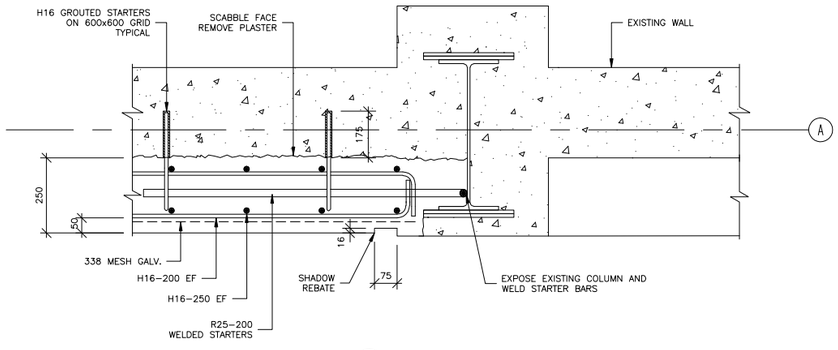


2 S40-01 SCALE 1:10

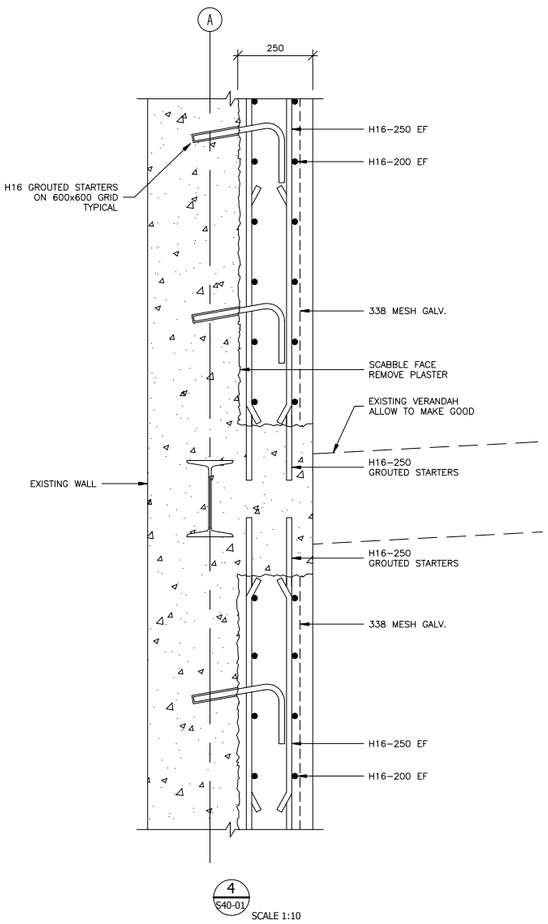


5 S40-01 SCALE 1:10

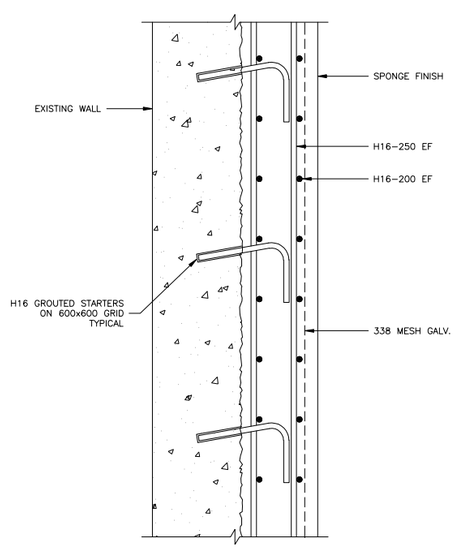
INSIDE



1 S40-01 SCALE 1:10



4 S40-01 SCALE 1:10



3 S40-01 TYPICAL SCALE 1:10

Original Scale

100 50 0 10

Original Size A1

C:\3864_Drawings\drawings_EP09.dwg, 12/04/2010 8:46:48 a.m., 1:1

ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

SHEARWALL DETAILS SHEET 1

REVISIONS		
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:10
A3 Scales	1:20
Designed	Adam Thornton
Drawn	Julie NICHOLSON
CAD Reference	3864-2009

Job Number 3864

Drawing Number S41-01 Rev 1

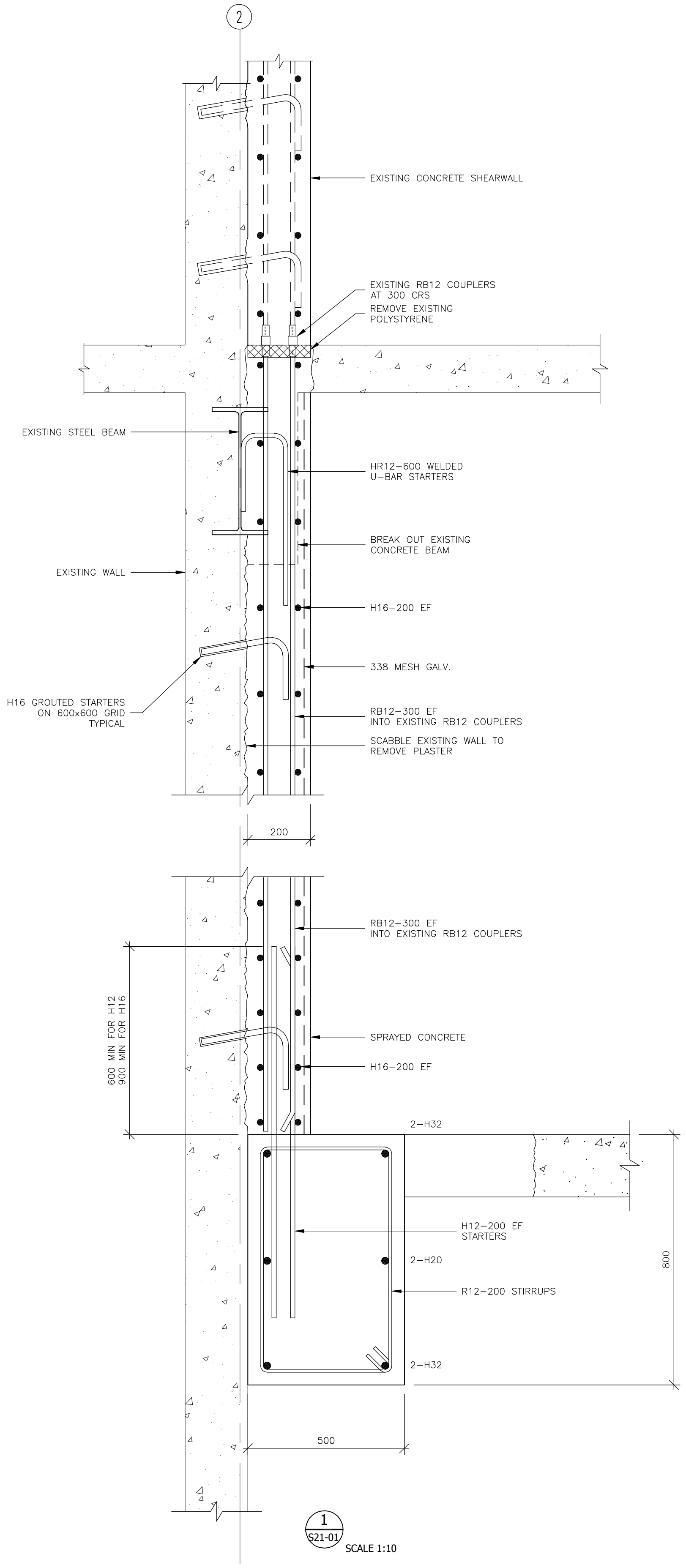
ISO 9001

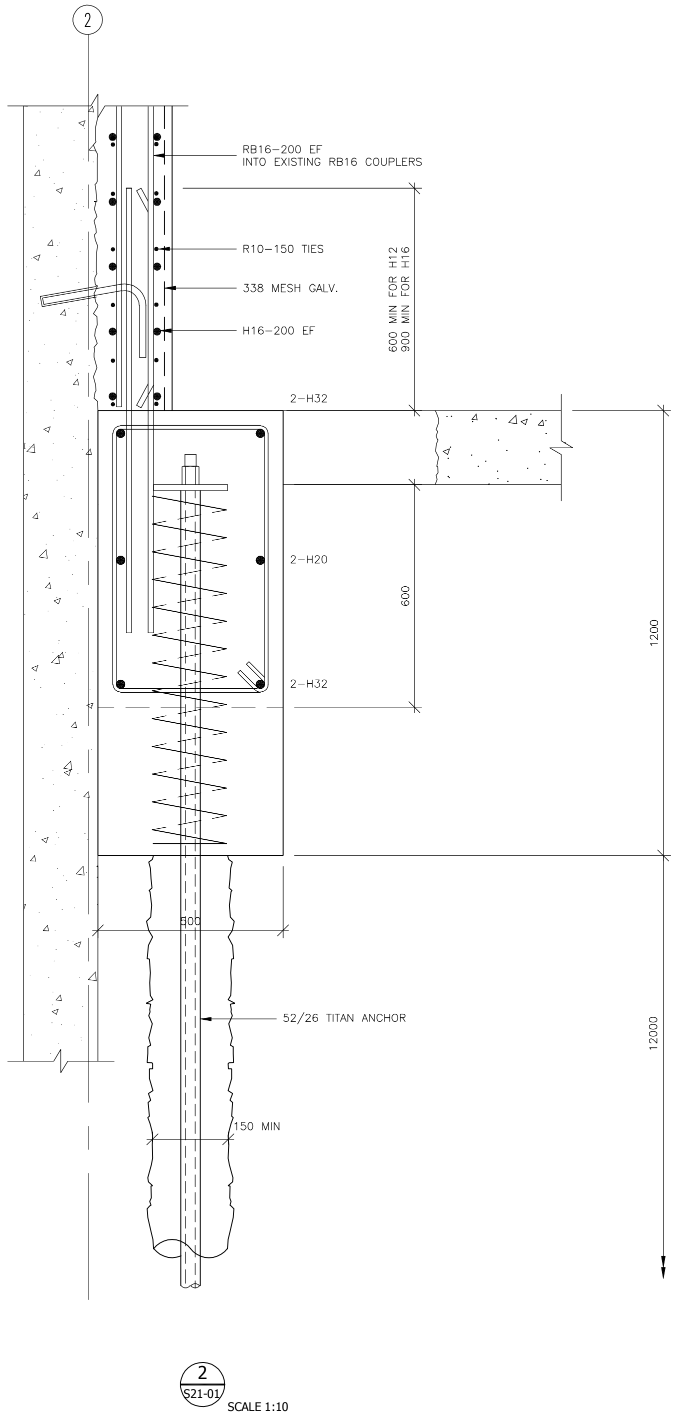
Dunning Thornton consultants

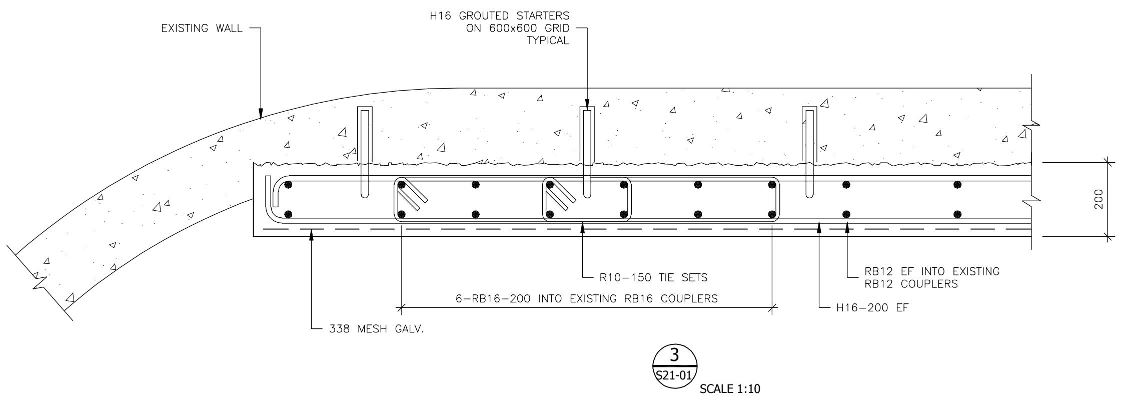
IF IN DOUBT ASK

DO NOT SCALE

p: 385 0019 f: 385 0312 e: dtcwgtn@dunningthornton.co.nz P.O. Box 27-153 Wellington







Original Size A1

Original Scale

100 50 0 10

Original Size A1

ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

SHEARWALL DETAILS
SHEET 2

REVISIONS		
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:10
A3 Scales	1:20
Designed	Adam Thornton
Drawn	Julie NICHOLSON
CAD Reference	3864-2009

Job Number 3864
Drawing Number S41-02
Rev 1

ACENZ
ISO 9001
JAS-ANZ

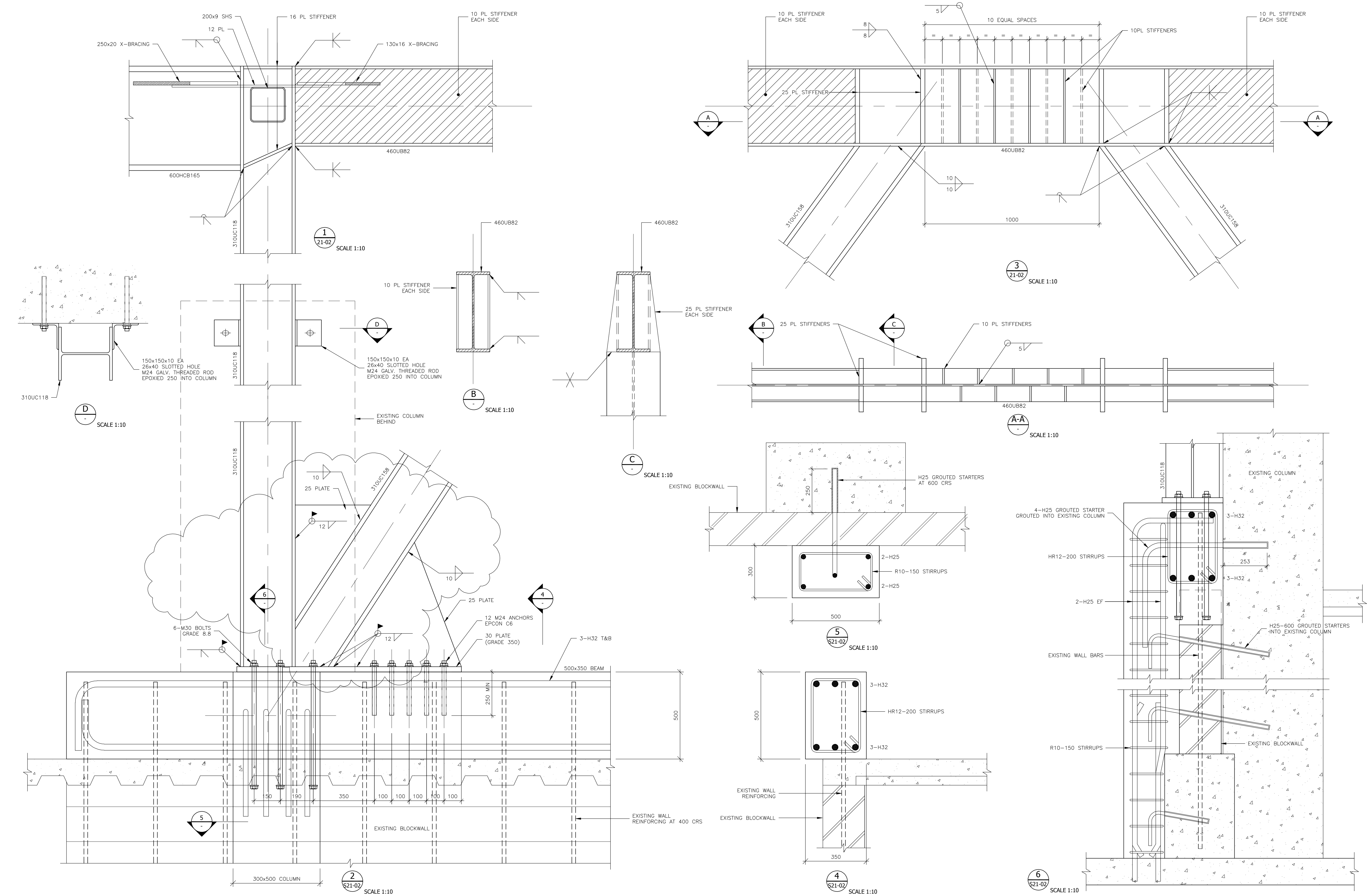
Dunning Thornton consultants

p: 385 0019 f: 385 0312 e: dtwgtn@dunningthornton.co.nz P.O. Box 27-153 Wellington

G:\3864_Drawings\drawings_2009.dwg, 12/04/2010 8:46:50 a.m., 1:1

IF IN DOUBT ASK

DO NOT SCALE



ABSOLUTELY POSITIVELY WELLINGTON	EMBASSY STRENGTHENING 2009	K-FRAME DETAILS

REVISIONS

1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:100
A3 Scales	1:20
Designed	Adam Thornton
Drawn	Julie NICHOLSON
CAD Reference	3864-2009

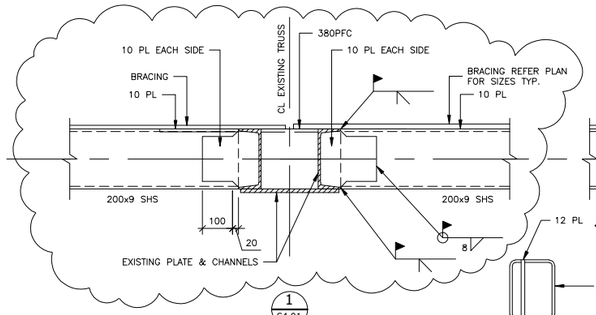
Job Number 3864
Drawing Number S80-01
Rev 1

ACENZ ISO 9001

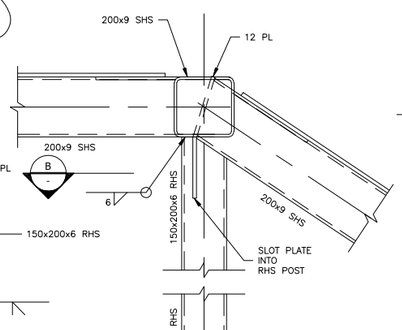
Dunning Thornton consultants
p: 385 0019 f: 385 0312 e: dtcwgtn@dunningthornton.co.nz P.O. Box 27-153 Wellington

IF IN DOUBT ASK

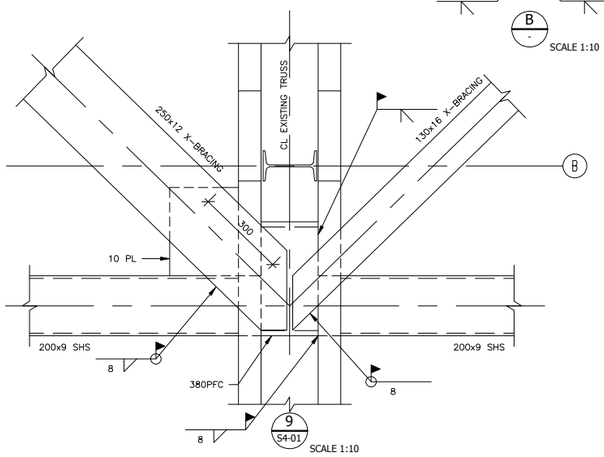
DO NOT SCALE



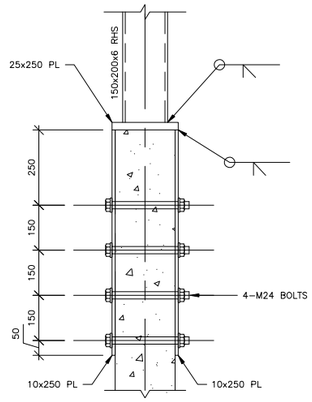
1 S4-01 SCALE 1:10



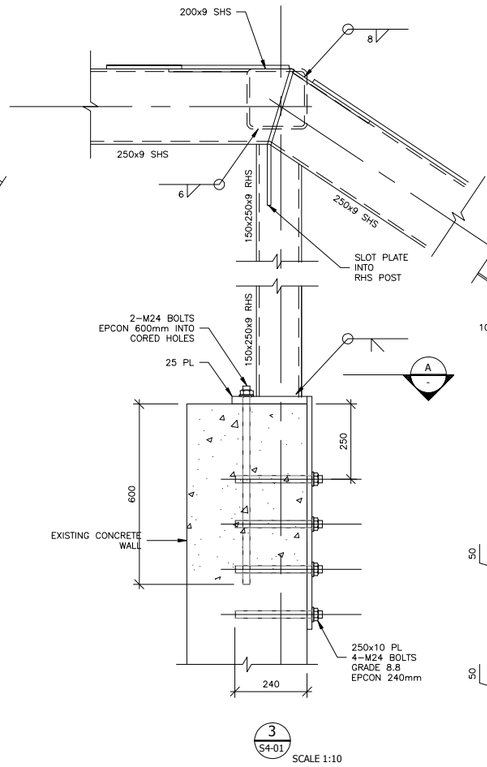
B - SCALE 1:10



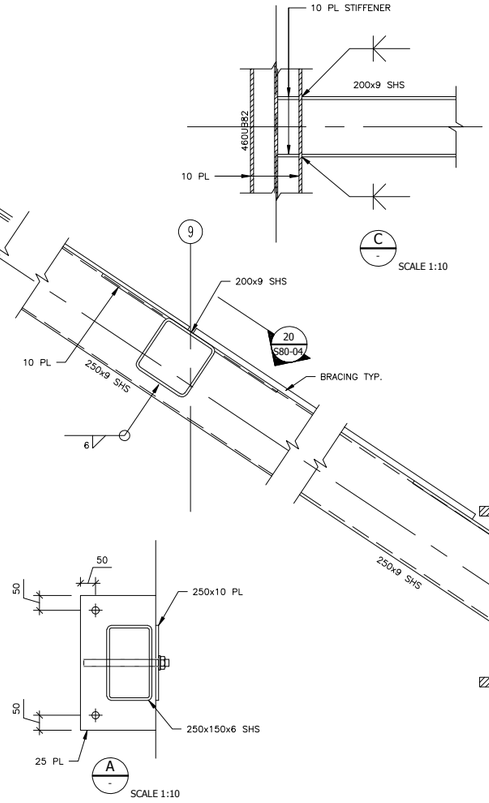
9 S4-01 SCALE 1:10



2 S4-01 SCALE 1:10

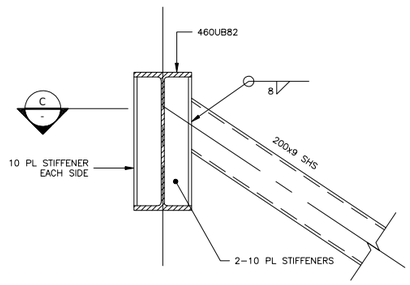


3 S4-01 SCALE 1:10

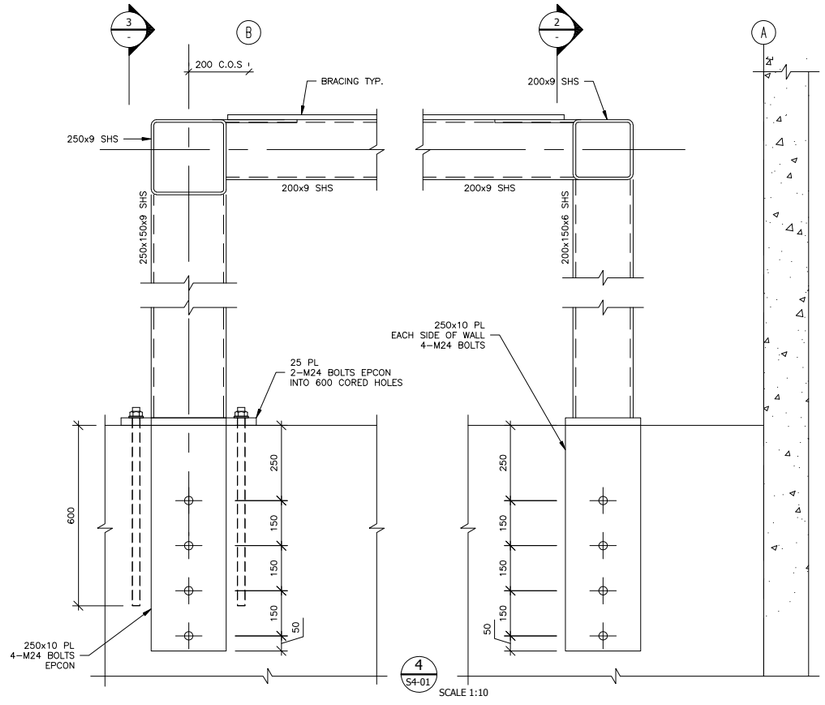


A - SCALE 1:10

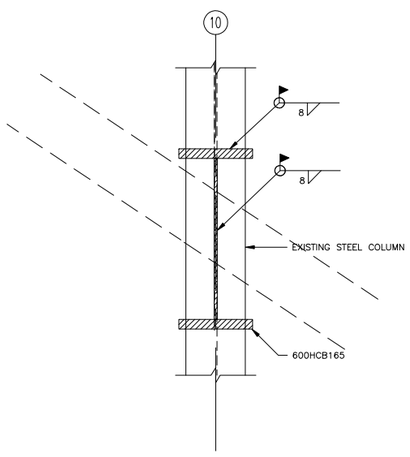
C - SCALE 1:10



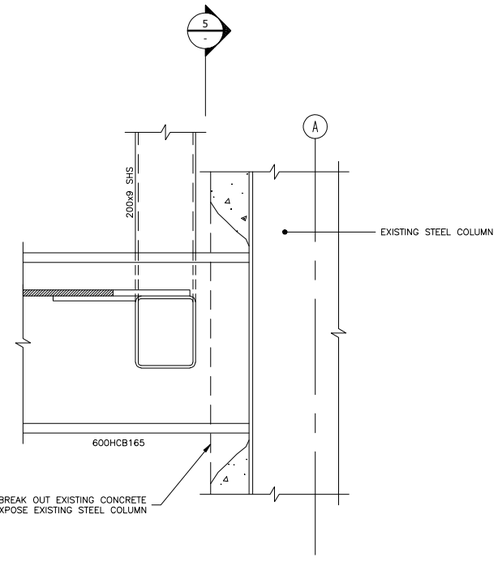
7 S4-01 SCALE 1:10



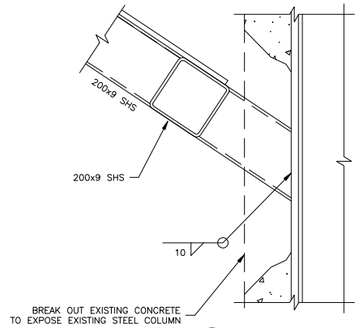
4 S4-01 SCALE 1:10



5 S4-01 SCALE 1:10



6 S4-01 SCALE 1:10



8 S4-01 SCALE 1:10

ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

STEEL DETAILS

	REVISIONS	
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:100
A3 Scales	1:20
Designed	Adam Thornton
Drawn	Julie NICHOLSON
CAD Reference	3864-2009

Job Number 3864
Drawing Number S80-02
Rev 1

ISO 9001

Dunning Thornton consultants

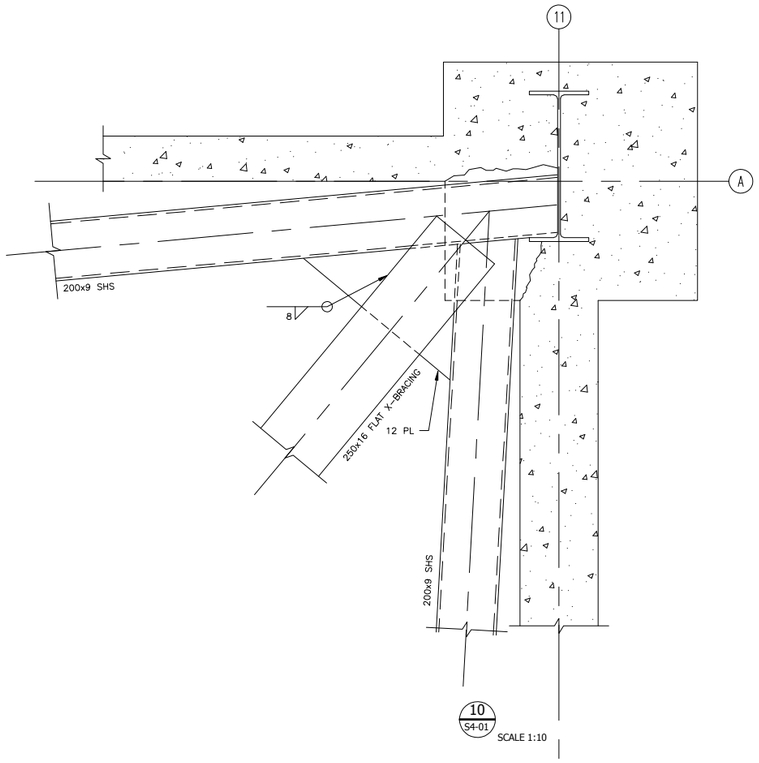
IF IN DOUBT ASK

DO NOT SCALE

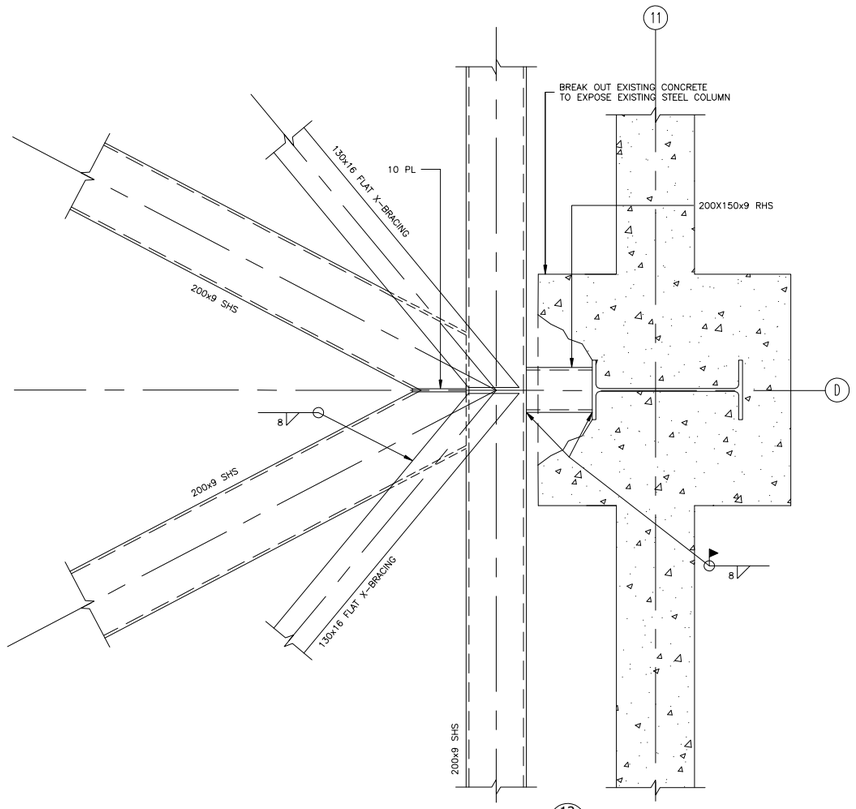
p: 385 0019 f: 385 0312 e: dtowgtn@dunningthornton.co.nz P.O. Box 27-153 Wellington

Original Size A1 Original Scale 100 50 0 10

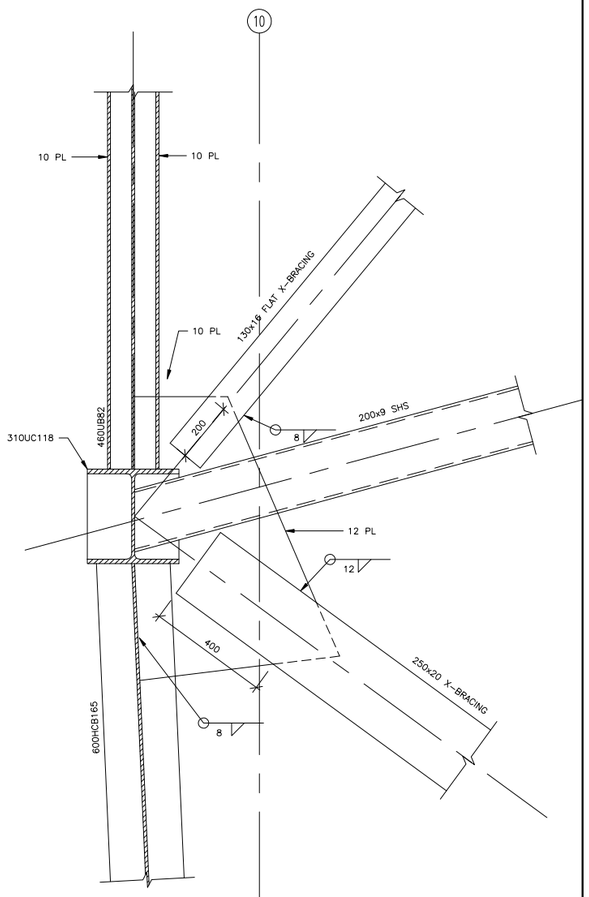
G:\3864_Drawings\drawings_2009.dwg, 12/04/2010 8:46:53 a.m., 1:1

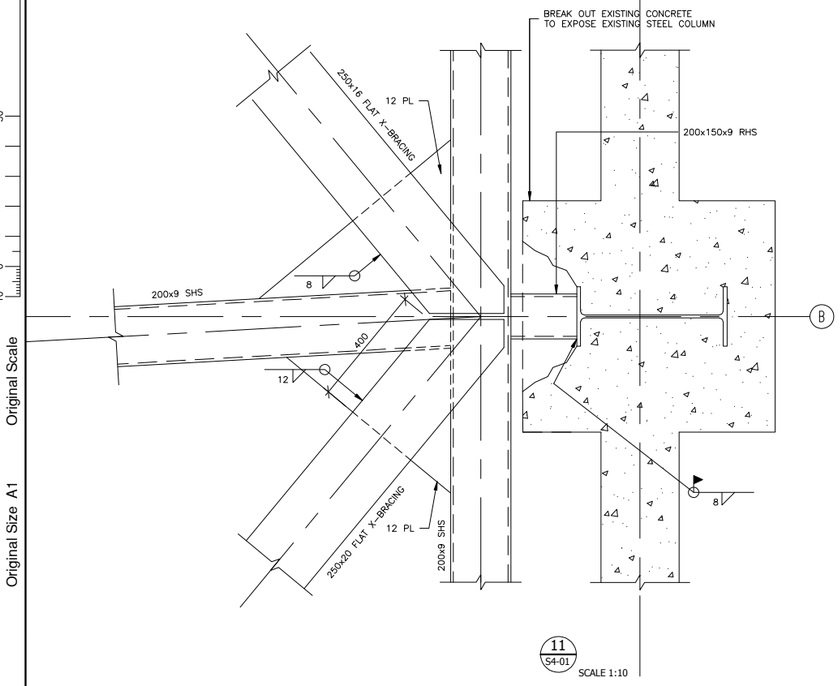


10 S4-01 SCALE 1:10

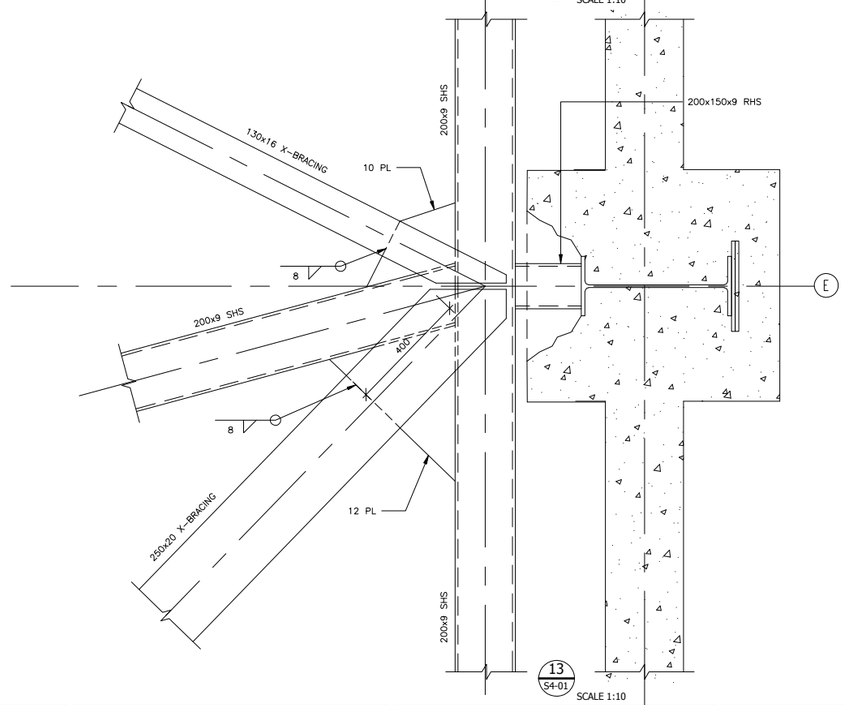


12 S4-01 SCALE 1:10



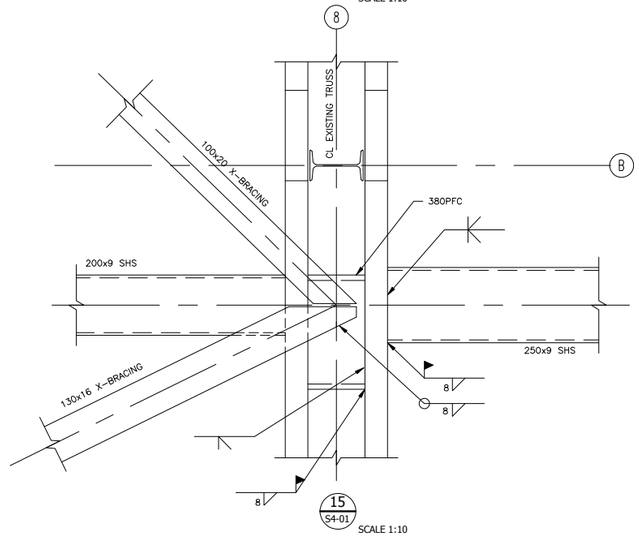


11 S4-01 SCALE 1:10



13 S4-01 SCALE 1:10

14 S4-01 SCALE 1:10



15 S4-01 SCALE 1:10

ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

STEEL DETAILS

REVISIONS		
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
A	PRELIMINARY	7/12/2009

Scales	1:100
A3 Scales	1:20
Designed	Adam Thornton
Drawn	Julie NICHOLSON
CAD Reference	3864-2009

Job Number 3864
Drawing Number S80-03
Rev 1

ISO 9001

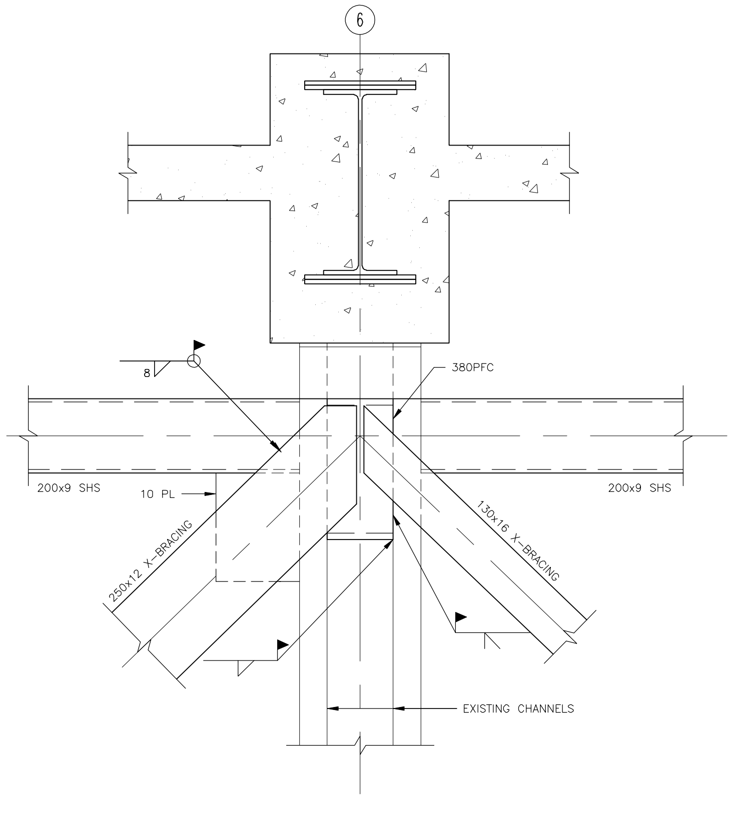
Dunning Thornton consultants

p: 385 0019 f: 385 0312 e: dtcwgtn@dunningthornton.co.nz P.O. Box 27-153 Wellington

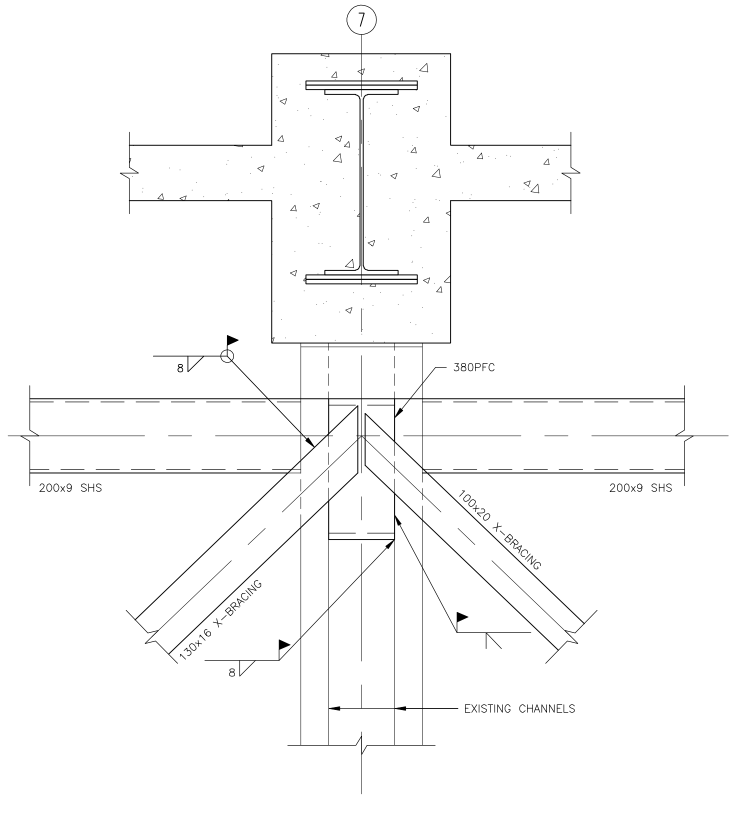
IF IN DOUBT ASK

DO NOT SCALE

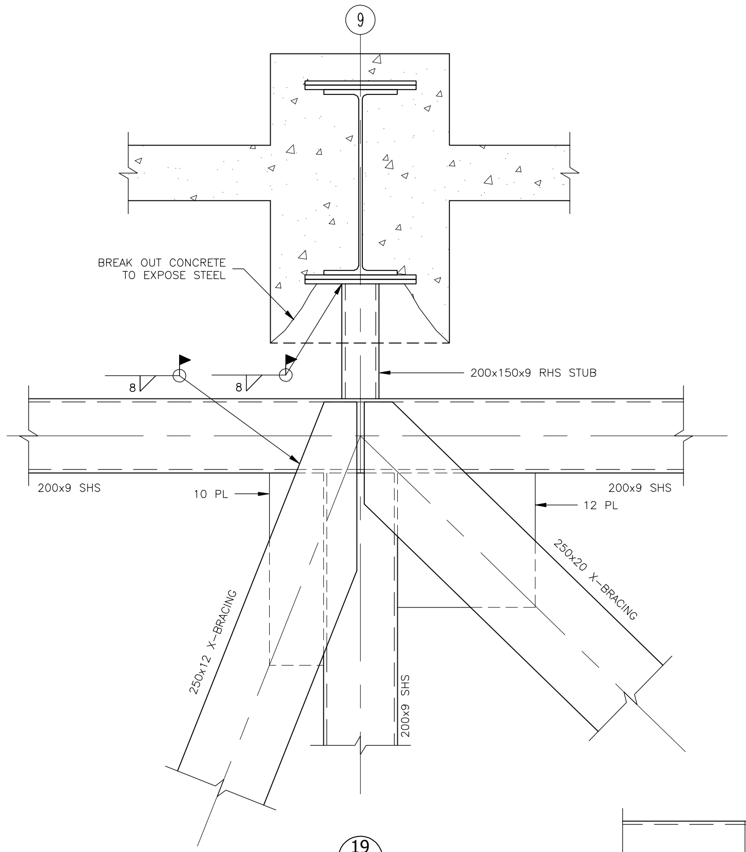
Original Size A1



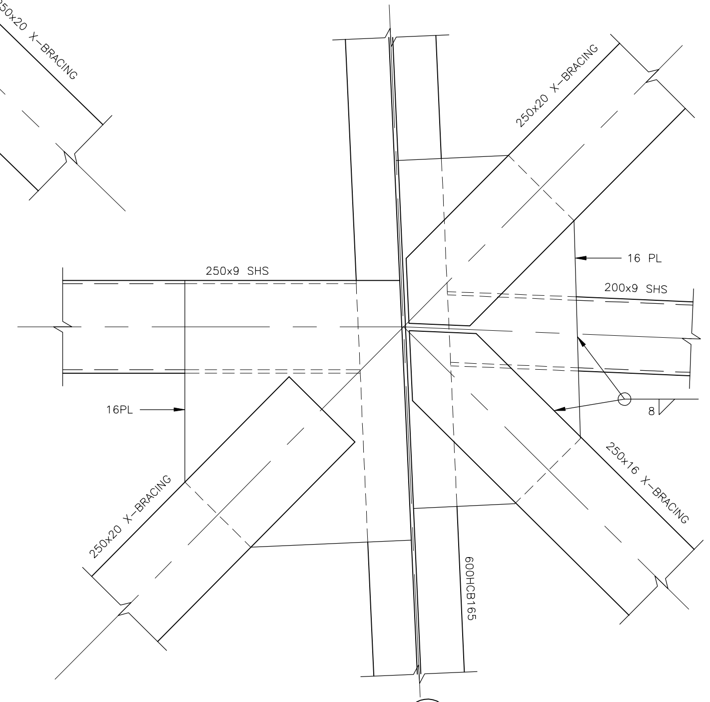
17
S4-01
SCALE 1:10



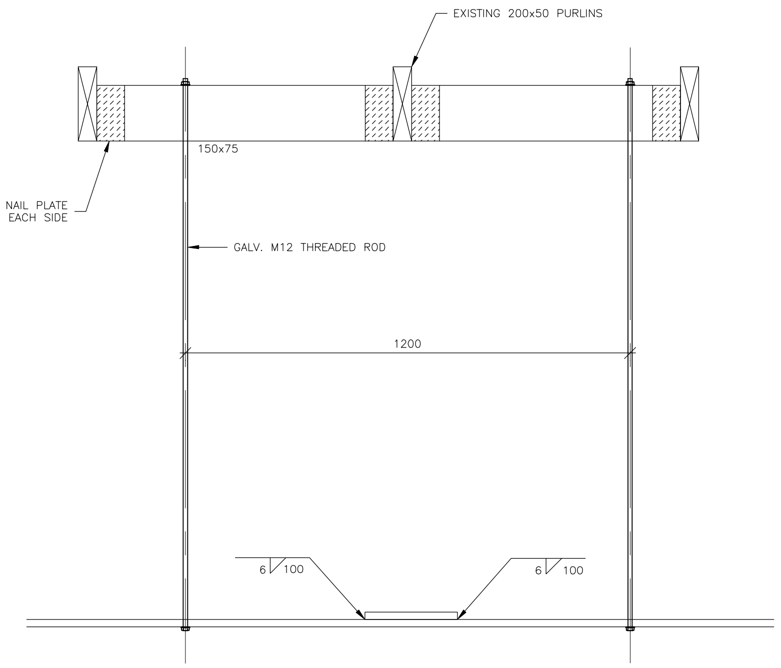
18
S4-01
SCALE 1:10



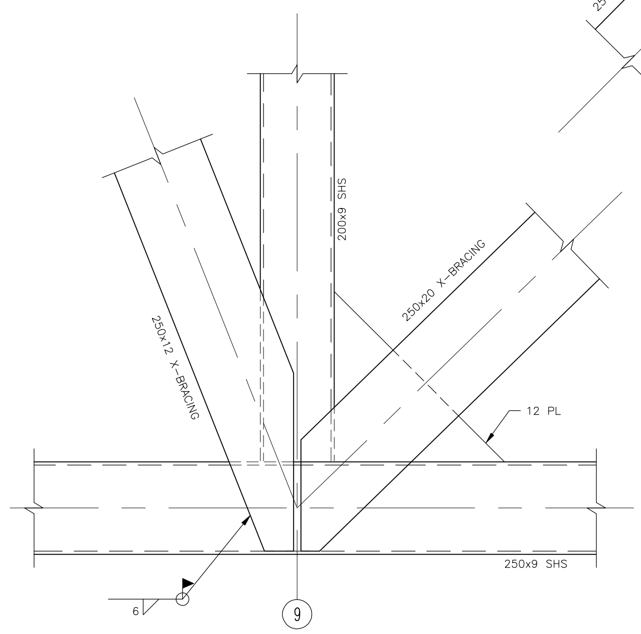
19
S4-01
SCALE 1:10



21
S4-01
SCALE 1:10



16
S4-01
SCALE 1:10



20
S4-01
SCALE 1:10

Original Scale

Original Size A1

ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

STEEL DETAILS

REVISIONS		
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
A	PRELIMINARY	7/12/2009

Scales	1:100
A3 Scales	1:20
Designed	Adam Thornton
Drawn	Julie NICHOLSON
CAD Reference	3864-2009

Job Number 3864
Drawing Number S80-04
Rev 1

ACENZ
JAS-ANZ
ISO 9001

Dunning Thornton consultants

p: 385 0019 f: 385 0312 e: dtcwgtn@dunningthornton.co.nz P.O. Box 27-153 Wellington

IF IN DOUBT ASK

DO NOT SCALE

Appendix B Geotechnical Report

5 October 2022
Job No: 1020459.0000

Wellington City Council
PO BOX 2199
WELLINGTON 6140

Attention: Tane Dunne

Dear Tane

Embassy Theatre Wellington, Geotechnical Seismic Desktop Assessment

1 Introduction

This report presents a desktop geotechnical seismic assessment for the Embassy Theatre, Wellington. The study was undertaken by Tonkin & Taylor Ltd (T+T) at the request of Wellington City Council in accordance with our letter of engagement dated 10 May 2022. The project structural engineer is Dunning Thornton Consultants Ltd (DTC).

This report summarises the agreed geotechnical parameters required by DTC to carry out their Detailed Seismic Assessment (DSA).

This report presents:

- A summary of findings of a desktop assessment.
- Factual information.
- Potential for liquefaction at the site and associated geotechnical consequences.
- A summary of geotechnical issues associated with the site.
- Agreed geotechnical parameters to support DTC's DSA.

2 Desktop assessment

2.1 Site description

Conclusion	Information reviewed
• The site is located at 10 Kent Terrace, Mt Victoria, Wellington. • The site is bordered by Majoribanks Street to the north, and Kent Terrace to the west. There are existing buildings to the east and south. • Site legal description: Lots 24 to 27 DP 240. • Property parcel areas (total): approximately 1,500 m^2. • The site footprint is wholly occupied by a multi-storey building.	• Aerial photo sourced from Terralink International (Copyright 2002-2005).

Tonkin & Taylor Ltd | Harbour Tower, Level 4, 2 Hunter Street, Wellington 6011, New Zealand
PO Box 2083, Wellington 6140 P +64-4-381 8560 F +64-9-307 0265 E wlg@tonkintaylor.co.nz

- The site is flat, with the surrounding land gently sloping in the north-west direction (<10°).
- The site is approximately 160 m south (landward) of the pre-1900 shoreline.
- The site is approximately 400 m south of the current shoreline.

2.2 Ground and groundwater conditions

Conclusion	Information reviewed
• Available borehole data within the site comprises three boreholes up to 9.5 m depth drilled in 1992. Refer to Figure A1, Appendix A for investigation locations. • The inferred soil profile is presented in Table 2.1 and Figures A2 to A4, Appendix A. • Groundwater measurements taken during drilling indicate a water level of approximately 1.8 m below ground level. Groundwater level of 1.8m below ground level has been assumed in this geotechnical assessment. • The boreholes have been inferred to terminate at the top of the rock.	• 1:50,000 geological map 22 (Begg, J.G.; Mazengarb, C., 1996). • "An engineering geological investigation of the seismic subsoil classes in the central Wellington commercial area". (Semmens.S.,2011) • T+T field borehole logs (BH01 to BH03) dated 15 to 18 November 1992 (Refer Appendix B) and notes.

Table 2.1: Summary of inferred subsurface ground conditions

Layer No.	Geological Unit	Description	Depth to top of layer (m)	Thickness (m)	SPT N
1	Reclamation Fill [1]	Sandy GRAVEL. Loose to medium dense	0	0.5 to 2.0	13 (one test only) [1]
2	Beach Deposits	SAND, some gravel. Loose to medium dense.	0.5 to 2.0	0.0 to 3.5	6 to 29
3	Harbour Deposits	SILT, trace gravel. Firm to stiff, non-plastic to low plasticity.	2.0 to 4.5	2.0 to 4.5	6 to 15
4	Alluvial Deposits	SILT, some sand. Stiff.	3.0 to 6.0	1.0 to 2.0	11 (one test only)
5	Colluvium / CW Rock	GRAVEL. Dense to very dense.	0.8 to 9.0	0.5 to 1.0	48 to 50+
6	Rock [2]	Sandstone, highly weathered or better. Weak to moderately strong.	2.0 to 9.5	-	-

Notes:

1. SPT N = 13 proven for the fill; descriptions indicate that the density varies between loose and medium dense (inferred N = 5 to 15).
2. Rock was not recovered and logged in the BH logs within the site, description of rock inferred from nearby investigations at adjacent sites.

2.3 Active faults

Conclusion	Information reviewed
• The Wellington Fault lies approximately 2.2 km northwest from the site. • The Wellington Fault is included in Table 3.6 of NZS 1170.5:2004 as a major fault requiring near fault factors when assessing structural design actions. • There are numerous other active and inactive faults mapped nearby in Wellington city. Bathymetric survey of the Wellington Harbour identified the active Aotea Fault. The Aotea Fault is inferred to project onshore and extend southward beneath Te Aro. Although the precise onshore location is currently inferred, the alignment of the fault is located approximately 40 m west of the site. The inferred location is poorly constrained, and for this reason GNS has only published the offshore fault alignment. • The Aotea Fault is not considered a major fault according to NZS 1170.5:2004.	• GNS Online database of active faults • NZS1170.5: 2004 Section 3.1.3 and Table 3.6 • "The structure and seismic potential of the Aotea and Evans Bay Faults, Wellington, New Zealand" (Barnes, P.M. et al, 2018) • Kaiser, A.E., et. al., 2019. Updated 3D Basin model and NZS 1170.5 subsoil class and site period maps for the Wellington CBD: Project 2017-GNS-03-NHRP. GNS Science consultancy report 2019/01.

2.4 Previous earthquakes

Conclusion	Information reviewed
• Kaikōura Earthquake (14 November 2016 at 12:02am) Location: 15 km northeast of Culverdon Magnitude: M_L 7.8 Focal depth: 15 km Intensity felt at site: PGA 0.18g recorded at Wellington Te Papa Museum (TEPS), approximately 450 m northwest of the site. • Cook Strait Earthquake (21 July 2013 at 5.09pm) Location: Cook Strait, distance 51km Magnitude: M_L 6.5 Focal depth: 13km Intensity felt at site: PGA 0.12g recorded at Wellington Te Papa Museum (TEPS), approximately 450 m northwest of the site. • Lake Grassmere Earthquake (16 August 2013 at 2.31pm) Location: Lake Grassmere, distance 72km Magnitude: M_L 6.6 Focal depth: 8km Intensity felt at site: PGA 0.1g recorded at Wellington Te Papa Museum (TEPS), approximately 450 m northwest of the site. • No known evidence of ground damage at the site as a consequences of these earthquakes.	• Earthquake magnitude source of data: http://geonet.org.nz/

2.5 Existing building foundations

Conclusion	Information reviewed
Available information provided by the structural engineer indicate the following stages of foundation construction: • The original building (1923) was founded on shallow pad foundations (various sizes). • A 2003 retrofit of the building included the removal of existing shallow foundations located towards the centre of the building, installation of a basement, and the installation of screw pile and rock anchor foundations. • A retrofit in 2009 included the installation of micro-pile foundations. The following foundation types are considered in the assessment: • Square shallow pad foundations ranging in size from 0.9m to 1.2m to an approximate depth of 0.5m to 1.5m below ground level. Depths based on that shown on drawings; no construction records found. • Rectangular shallow pad foundations ranging in size from 1.5m by 0.9m to 1.8m by 1.5m, to an approximate depth of 0.5m to 1.5m below ground level. Depths based on that shown on drawings; no construction records found. • Screw pile foundations (37 No.) with 219mm shaft outer diameter and 600mm helix diameter founded 5m to 10.5m below base of pile cap/ground beam around the perimeter of the building, and 1.4m to 8m below the underside of the basement. • Screw pile foundations (17 No.) in conjunction with a 200mm diameter grouted rock anchor extending below the base of the screw pile. • Rock anchor foundations (6 No.) with 200mm diameter founded 4.3m to 5m below the underside of the basement. • Micropile foundations (4 No.) measuring 200mm in diameter and 12m long founded in rock (length based on that shown on drawings; no construction records found).	• No as-built drawing available for the foundations • "Embassy Redevelopment 2003" by DTC dated August 2003 (Refer Appendix C). • "Embassy strengthening 2009" by DTC, dated April 2010 (Refer Appendix C). • "New Theatre, Courtenay Place, Wellington" by Llewellyn E Williams Architect Structural Engineer dated March 1923 (Refer Appendix C).

3 Geotechnical engineering considerations

3.1 Seismic site subsoil class

Conclusion	Information reviewed
• Site subsoil class is assessed to vary across the site between Class B – Rock, and Class C – Shallow soil site. • Level of certainty in the above assessment is moderate to high. • The level of certainty in the location of the subsoil class boundary is low. It is likely that a larger area of the building footprint is Subsoil Class C than Class B. • If required, further investigations could provide an opportunity to inform the demarcation of the Class B and Class C zones.	• Refer Section 2.2. • NZS1170.5:2004 • Kaiser, A.E., et. al., 2019. *Updated 3D Basin model and NZS 1170.5 subsoil class and site period maps for the Wellington CBD*.

3.2 Ground shaking hazard

The seismic hazard in terms of peak ground acceleration (PGA) and magnitude (M) for the site has been assessed based on NZGS/MBIE Module 1 (2016). Table 3.1 presents the return periods for earthquakes with various 'unweighted' peak ground accelerations (PGA) with a corresponding earthquake magnitude.

Table 3.1: Ground seismic hazard

NZS 1170.5 Limit State	PGA (g)	Effective magnitude M_{eff}	Return period (years)
Subsoil Class C – Shallow Soil Site			
Ultimate limit state (ULS)	0.59	7.1	1000
Serviceability limit state (SLS)	0.11	6.2	25
Subsoil Class B – Rock Site			
Ultimate limit state (ULS)	0.45	7.1	1000
Serviceability limit state (SLS)	0.09	6.2	25

Note:

PGA and effective magnitude has been assessed based on MBIE/NZGS Module 1 (2016), for the following:

Building design life	50 years – advised by DTC
Building importance level	IL3 (NZS 1170.0:2004, Table 3.2) – advised by DTC
Return period factor, Ru	1.3 for 1000yr; 1.0 for 500yr; 0.5 for 100yr; and 0.25 for 25yr return period (NZS 1170.5:2004, Table 3.5)
Subsoil class	B (rock) and C (shallow soil) – refer Section 4.4.1
Return period PGA coefficient, $C_{0,1000}$	0.44 (Bridge Manual Table 6A.1)
Site subsoil class factor, f	1.0 for Subsoil Class B, 1.33 for Subsoil Class C (Bridge Manual Section 6.2)
PGA	$C_{0,1000}$ x Ru/1.3 x f x g (Bridge Manual Section 6.2)
Effective Magnitude, Meff	7.1 for 1000yr and 500yr and 6.2 for 100yr and 25yr return period (Bridge Manual Table 6A.1)

A revision to Module 1 (2016) was issued in November 2021. The geotechnical seismic assessment presented in this report has been undertaken using the parameters presented in Table 3.1 from Module 1 (2016) as per the guidance notes on the MBIE website for the seismic assessment of existing buildings.

4 Liquefaction assessment

4.1 Liquefaction potential

The triggering of liquefaction, for each soil layer identified as being susceptible to liquefaction, has been assessed in accordance with the procedure of Idriss and Boulanger (2014). The method is based on empirical relationships with the SPT 'N'/CPT 'qc' and fines content. SPT data from three boreholes have been used in the assessment. The conclusions are summarised in Table 4.1.

Table 4.1: Liquefaction potential

Layer No.	Description	Conclusion
1	Reclamation Fill	The reclamation fill is above groundwater level and is not expected to liquefy.

Layer No.	Description	Conclusion
2	Beach Deposits	Widespread liquefaction of loose to medium dense sand possible in an earthquake shaking with PGA > 0.2g, M7.1 (approx. 35% ULS IL3 Subsoil Class C).
3	Harbour Deposits	Widespread liquefaction of non-plastic and low plasticity silt possible in an earthquake shaking with PGA > 0.23g, M7.1 (approx. 40% ULS IL3 Subsoil Class C).
4	Alluvial Deposits	Localised pockets of liquefaction within layers/lenses of low plasticity silt and medium dense sand possible in an earthquake shaking with PGA > 0.25g, M7.1 (45% ULS IL3 Subsoil Class C).
5	Colluvium / CW Rock	This layer is dense and is not susceptible to liquefaction.
6	Rock	This layer is not susceptible to liquefaction.

There is a high level of uncertainty in the liquefaction assessment because:

- There is no CPT or laboratory test data available.
- Recorded groundwater levels at the site were taken during drilling only.

Specific investigations would be required to confirm/revise the conclusions of the liquefaction assessment outlined above.

4.2 Liquefaction consequences

Considering the potential for liquefaction described in Section 4.1, consequences of liquefaction at have been identified as listed in Table 4.2.

Table 4.2: Liquefaction consequences

ID	Issue	Comments
1	Cyclic displacement (Ground lurch)	• Cyclic displacement is not expected at the south-eastern corner of the building where rock is shallow (Zone A of Figure A5, Appendix A). For the rest of the building (Zone B and C of Figure A5, Appendix A) cyclic displacement of the order of 100mm to 200 mm is possible (100mm probable) at the site during shaking.
2	Lateral spread	• Not expected as the site is relatively flat and no nearby free face present.
3	Reduced support to shallow foundations	• Liquefaction beneath a shallow foundation will result in a reduction of support to that foundation and associated settlement of that foundation. • See Section 5.
4	Reduced support to piles	• Reduced shaft resistance of pile foundations penetrating liquefied beach and harbour deposits. • Reduced end bearing for piles founded in or near liquefied soils.
5	Negative skin friction (NSF)	• Post liquefaction settlements as a result of reconsolidation of liquefied soils could impose down drag on deep foundations.
6	Free field settlement	• Severe to moderate earthquake shaking (> 35% ULS IL3 Subsoil Class C) could cause surface settlement of up to 120 mm. This settlement would be nil in Zone A and increasing to the north and west.

ID	Issue	Comments
7	Sand boils	• Sand boils at the site around the perimeter of the building are possible as crust thickness is less than 2m.
8	Uplift pressure (liquefaction induced)	• Liquefaction induced uplift pressure on the underside of the basement is possible as a result of liquefaction of the beach and harbour deposits immediately underlying the basement slab. The uplift pressure on the 2.65m deep (west) basement would be in the order of 50 kPa.
9	Reduced passive capacity of basement walls	• As a result of liquefaction of the beach and harbour deposits the passive resistance of the basement walls (north, west and southern walls only) are likely to be reduced. Refer Section 6.2.1.

5 Geotechnical issues identified

Several geotechnical issues that have potential to affect the buildings seismic performance are presented in Table 5.1.

Table 5.1: Geotechnical Issues

ID	Issue	Comments
1	Reduced support to shallow foundations.	• Refer Section 6.1.1.
2	Limited vertical capacity of existing piles (screw piles and micropiles).	• Refer Section 6.1.2.
3	Limited lateral capacity of piles (screw piles and micropiles).	• Refer Section 6.2.2.
4	Differential settlement	Differential movement of the building may result from settlement attributed to the following three mechanisms: • Post-liquefaction free field settlement as reported in Table 4.2, ID 6. • Ground settlement as a result of sand boil(s). This is in addition to free field settlement described above. • Deviatoric stresses induced by foundation loads. Settlement can be derived using the vertical spring stiffnesses provided in Table 6.2 and Table 6.3.
5	Different foundation types	• The building is founded on both deep and shallow foundations with different behaviours under vertical loading (stiffness difference). This is further exacerbated by liquefaction. • Structural analysis to consider these differences in stiffnesses, refer Section 6.1.
6	Cyclic Displacement	• Cyclic displacement of the ground will impose a lateral load on the piles in Zone B and C. • A lateral load is induced in the structure due to the foundations and basement in the south-eastern corner of the building (Zone A of Figure A5, Appendix A) being “anchored” in rock, and the remainder of the foundations and underground structures (Zone B and C of Figure A5, Appendix A) are subjected to lateral kinematic loads. The anchorage force in Zone A is assessed to be in the order of 8 to 12MN.

ID	Issue	Comments
		• Should the structure not be able to tolerate this force, the lateral movement of the ground and building between Zones A and B could be up to the magnitude of the cyclic displacement (refer ID 1, Table 4.2).
7	Earthquake ground rupture	• The location of the Aotea Fault is poorly constrained and there is a risk that this fault may be at or very close to the site.

6 Assessment of existing foundations

In line with Section C4 of the Assessment Guidelines, the capacities presented in this section need not be reduced by a strength reduction factor.

If the parameters presented in this section prove to be critical to the assessment, the structural engineer is to discuss this with the geotechnical engineer, to allow review.

Structural parameters used in the assessment were provided by DTC. The geotechnical assessment is based on founding conditions presented in Sections 2.2 and 4.1.

6.1 Vertical Assessment

6.1.1 Vertical Capacity of Shallow Foundations

The agreed geotechnical parameters with DTC for the shallow foundations are provided in Appendix D for their structural assessment. These include the assessment of the limiting (probable) bearing capacities and range of compressive spring stiffness for each existing footing type, presented in Table 6.1 and Table 6.2 respectively.

The calculated bearing capacity assumes that the foundations are not subjected to overturning moments i.e. foundations are not eccentrically loaded and the full footing area is assumed to be providing uniform support. A range of bearing capacities where the footing is subjected to horizontal loading (e.g. used to take out base shear) is provided, where H is the imposed horizontal load on the footing and V is the vertical load.

Table 6.1: Probable Bearing Capacity of Shallow Pad Foundations

H/V	Net Ultimate (Limiting) Bearing Capacity (kPa)				
	Zone A	Zone B		Zone C	
	No liquefaction	No liquefaction	Liquefaction	No liquefaction	Liquefaction
0	1200	700	50 to 180	1030	50 to 180
0.1	860	500	-	750	-
0.2	620	350	-	530	-
0.3	430	250	-	370	-
0.4	300	170	-	250	-

Note:
Refer Figure A5 in Appendix A for Zone classification.

Table 6.2: Vertical spring stiffnesses of Shallow Pad Foundations

Zone	Compressive Spring Stiffness (kPa/mm)	
	No liquefaction	Liquefaction
A	50 to 250	-
B	10 to 40	1 to 8
C	10 to 40	1 to 8

Note:
Refer Figure A5 in Appendix A for Zone classification.

The structural analysis should consider the range of stiffnesses, including performing sensitivities (e.g. one pad being 'soft' and adjacent pads 'h favourable result.

Very low. Too low to be of use.
Is there a reasonable settlement we can work to in the foundations to check if a step change occurs in the structure?

Foundations in compression may be modelled bearing capacity.

6.1.2 Pile foundations

The agreed geotechnical parameters with DTC for the deep foundations are provided in Appendix F for their structural assessment.

Table 6.3Error! Reference source not found. summarises the estimates of the probable geotechnical capacities and spring stiffnesses of the existing pile foundations. These capacities were assessed using the available drawings included in Appendix C. The capacity and spring stiffness values provided are applicable to the no liquefaction case only (up to 35% ULS IL3 for Subsoil Class C).

Table 6.3: Vertical capacity and spring stiffnesses of deep foundations (No liquefaction case)

Pile Location	Pile Type	Assumed Pile Length (m)	Probable Vertical Capacity (kN)		Spring Stiffnesses (kN/mm)	
			Compression	Tension	Compression	Tension
Perimeter (Grid A)	Micropile founded in rock	12	-[2]	1500	-[2]	15 to 60
Perimeter (Grid A)	Screw Pile	9.5	1000	500	20 to 100	10 to 50
Perimeter (Grid A)	Screw Pile with rock anchor	Screw Pile: 7.3 Rock Anchor: 1.3	1000	500	20 to 100	10 to 50
Basement (P37 – P39)	Screw Pile	7.5	1000	-[3]	20 to 100	-[3]
Basement (P37 – P39)	Screw Pile with rock anchor	Screw Pile: 7 Rock Anchor: 2	1000	-[3]	20 to 250	-[3]
Basement (remaining piles)	Screw Pile	3.5	1000	-[3]	20 to 100	-[3]
Basement (remaining piles)	Screw Pile with rock anchor	Screw Pile: 3.5 Rock Anchor: 3	1000	-[3]	20 to 250	-[3]
Shallow basement (Approx. Grid 8 to 10)	Screw Pile	2	1000	-[3]	20 to 100	-[3]
Grid 10	Rock Anchor	4.5	1200	1200	50 to 250	15 to 75

Note:

1. Spring stiffness provided are applicable up to the probable (limiting) compression and tension reported.
2. DTC advised that compression load will not be transferred to the micropiles.
3. The tension capacity and spring stiffness of this pile type has not been assessed. DTC advised that there is no tension loading on this pile type.

The structural analysis should consider the range of stiffnesses, including performing sensitivities (e.g. one pile being 'soft' and adjacent piles hard and vice versa etc) and allow for the least favourable result. Foundations may be modelled as elastic plastic i.e. plastic beyond the ultimate geotechnical capacity.

6.2 Lateral Assessment

6.2.1 Shallow foundations and basement

The lateral resistance for base shear take-out can be derived from the passive resistance behind the basement walls and pad foundations and sliding resistance beneath the shallow pad foundations.

6.2.1.1 Passive resistance

this is written like a new design. There are no choices for us here.

The load-displacement plots for passive resistance are included in Appendix D for the pad foundations and Appendix E for the basement walls.

6.2.1.2 Sliding

It is preferable to resist base shear through passive resistance as inducing a lateral load on shallow foundations reduces the bearing capacity significantly. If sliding resistance is required to take out base shear, the sliding coefficient beneath the existing footings may be taken as 0.65 (where sliding resistance = 0.65 x the vertical load on a footing not supported by piles). The sliding coefficient assumes that the concrete is cast directly onto ground without DPM or other types of membrane beneath that can affect the sliding resistance. The sliding coefficient presented is for the no liquefaction case only.

Where sliding capacity is considered, the foundations bearing capacity shall be reduced based on the ratio of horizontal and vertical load (H/V) being applied to the foundation, refer Table 6.1.

6.2.2 Pile foundations

Table 6.4 presents the lateral pile behaviour and assessment under the four stages of an earthquake.

Table 6.4: Pile lateral behaviour during an earthquake

Scenario No.	Description	Comments
1	Start of earthquake (no liquefaction) Up to 35% ULS(IL3) Subsoil Class C ground shaking.	• 100% of base shear resisted by passive resistance of buried elements (refer Section 6.2.1). The pile foundations will be subject to the displacement required to mobilise the passive resistance. • Lateral pile (LPile) analyses of the screw pile and micropile foundations were undertaken by varying the displacement at the pile head and the results are presented in Appendix F.
2	Liquefaction triggered but no lateral ground movement	• Loss of lateral support to piles.

Scenario No.	Description	Comments
		• Pile behaviour for this scenario has not been assessed per discussion with DTC.
3	Ground lurch (cyclic displacement of liquefied ground) occurs. During shaking.	• If the building moves less than the cyclic displacement (ID 1, Table 4.2), the soil movement acts as a kinematic load on the pile (in addition to base shear). • Pile behaviour for this scenario has not been assessed per discussion with DTC.
4	Lateral spreading occurs. Towards end of shaking, possibly post shaking.	• Lateral spreading is not expected at the site and pile behaviour for this scenario has not been assessed.

7 Further assessment

If strengthening of the building is required, further specific geotechnical engineering input will be required as outlined below.

- Site-specific geotechnical investigations.
- Foundation optioneering and concept design.
- Preliminary design.
- Developed design.
- Detailed design.
- Construction monitoring.

Information presented in this report is not to be applied to any aspect of the strengthening design without review.

8 Applicability

This report has been prepared for the exclusive use of our client Wellington City Council, 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.

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

Report prepared by:	Authorised for Tonkin & Taylor Ltd by:
	
.....	.....
Emily Peebles Geotechnical Engineer	Dr EngLiang Chin Project Director

Technical review by Bhavesh Rama (Senior Geotechnical Engineer).

5-Oct-22
p:\1020459\workingmaterial\3. report\1020459-rpt-gt-001-v1_embassy theatre.docx

Appendix A Figures

- Figure A1: Site Plan
- Figures A2-A4: Cross Sections
- Figure A5: Zone Classification

LEGEND

- Contour
- Property boundary
- Site bounday
- Cross section
- Borehole (1992)

A3 SCALE: 1:500

0 5 10 15 20 25 (m)

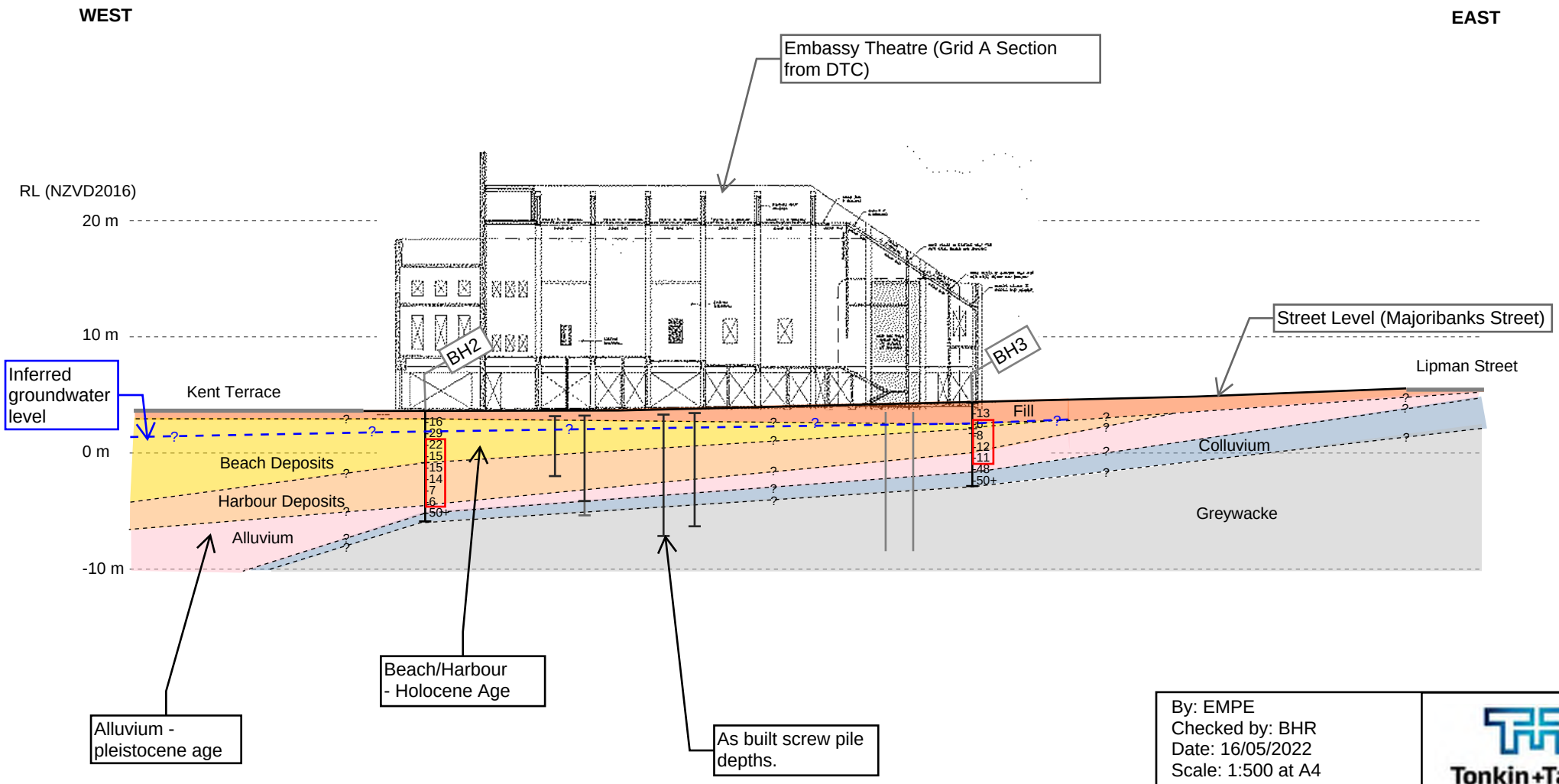
1. Property Boundary,Street Name,Street Number sourced from the LINZ Data Service and licensed for re-use under the Creative Commons Attribution 3.0 New Zealand licence.
2. World Imagery Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

Created On:	13/05/2022
Created By:	EPeebles
Approved By:	
TT Proj Ref:	1020459
TT Map Ref:	TTMAPREF1433463001.902

Tonkin+Taylor
105 Carlton Gore Road, Newmarket, Auckland
www.tonkintaylor.co.nz

WELLINGTON CITY COUNCIL
EMBASSY THEATRE
Site Plan

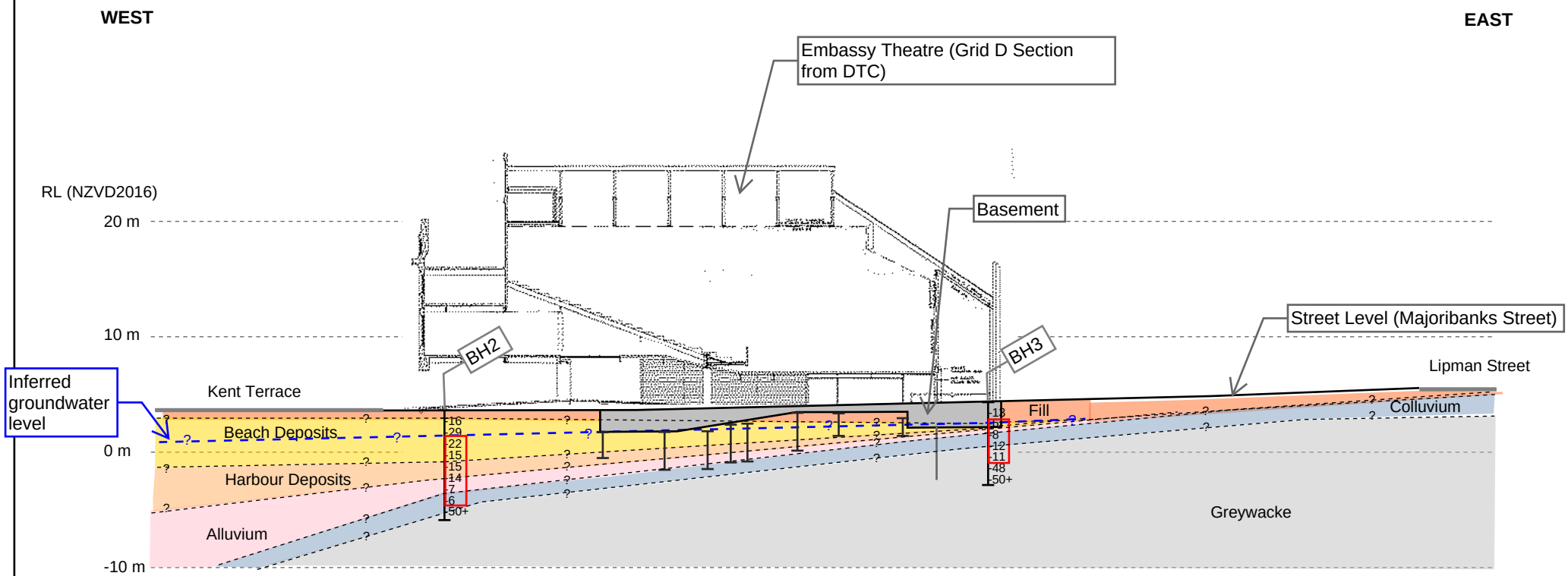
FIGURE No. **A1**



By: EMPE
Checked by: BHR
Date: 16/05/2022
Scale: 1:500 at A4
Rev: 0

Tonkin+Taylor

WELLINGTON CITY COUNCIL
EMBASSY THEATRE
FIGURE A2: CROSS-SECTION (WEST - EAST)
Grid A



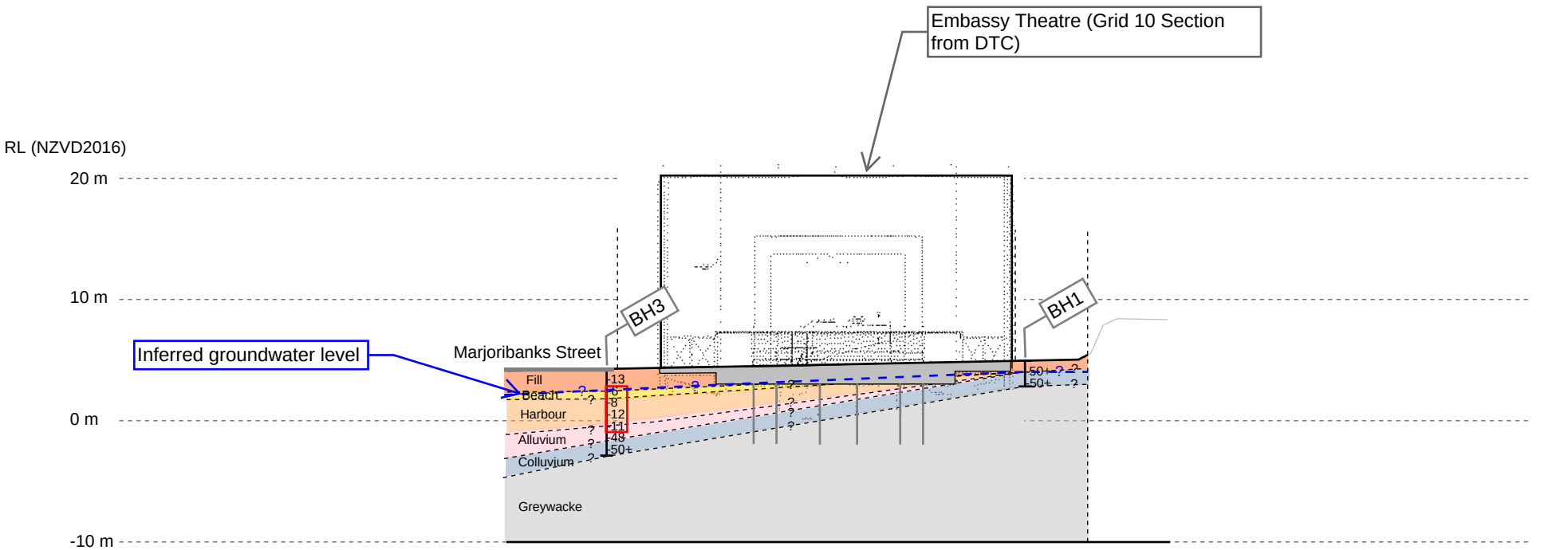
By: EMPE
Checked by: BHR
Date: 16/05/2022
Scale: 1:500 at A4
Rev: 0

Tonkin+Taylor

WELLINGTON CITY COUNCIL
EMBASSY THEATRE
FIGURE A3: CROSS-SECTION (WEST - EAST)
Grid D

NORTH

SOUTH



5m

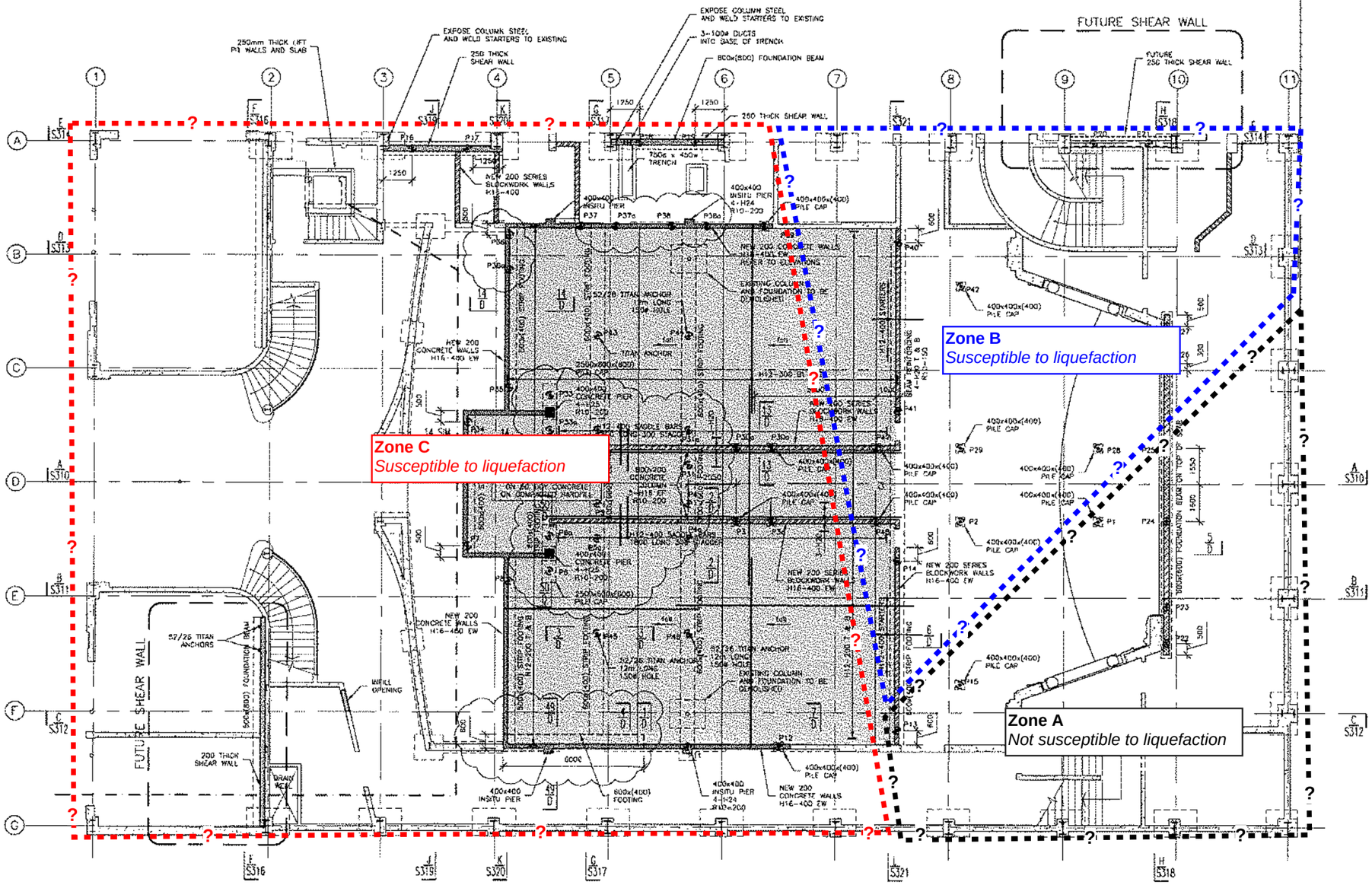
By: EMPE
Checked by: BHR
Date: 16/05/2022
Scale: 1:500 at A4
Rev: 0

Tonkin+Taylor

WELLINGTON CITY COUNCIL
EMBASSY THEATRE
FIGURE A4: CROSS-SECTION (North - South)

Embassy Theatre DSA, Wellington
T+T Ref: 1020459

Figure A5: Zone Classification
Rev: 1
Date: 2/06/2022
Drawn: EMPE
Checked: BHR / ELC



THEATRE EMBASSY TRUST	EMBASSY REDEVELOPMENT 2003	LEVEL 0 PILE LAYOUT & LOWER SLAB DETAILS

REVISIONS		
3	REVISED	22-07-2003
2	FOR CONSTRUCTION	20-06-2003
1	FOR CONSENT	3-06-2003
0	PILING TENDER	22-05-2003

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864
Drawing Number S300
Rev 3

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz

G:\3864\Dtc\Dwgs\3864S300.dwg, 23-07-2003 1:34:32 p., HP LaserJet 4MV.pc3

Appendix B Investigation Records

TONKIN & TAYLOR LTD
CONSULTING ENGINEERS

SITE DETAILS

FIELD LOG

BOREHOLE No: 1
SHEET 1 OF 1

PROJECT: THEATRE ROYAL LOCATION: MAJORIBANK ST., WELLINGTON JOB No: 80132

CO-ORDINATES
RL.
DATUM

DRILL TYPE: TRUCK MOUNTED RIG
DRILL METHOD: WASH DRILLING
DRILL FLUID:

HOLE STARTED: 15/11/92
HOLE FINISHED: 15/11/92
DRILLED BY: PENDLETON CORP.
LOGGED BY: GRF CHECKED:

GEOLOGICAL	DRILLING AND TESTS								ENGINEERING DESCRIPTION						
GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY	METHOD	CASING	TESTS.	SAMPLES	RL (m) / DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/ DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa) / COMPRESSIVE STRENGTH (MPa)	DEFECT SPACING (mm)	SOIL DESCRIPTION: Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION: Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.
BASECOURSE								0.0				D			CONCRETE
RECLAMATION FILL															GRAVEL, (Fine/Large), greywacke, yellow brown, dense.
			100		5FS	40 blows for 100mm		1.0				VD			GRAVEL (Fine/med), greywacke, brown, very dense.
COLLUVIUM			100		5FS	40 blows for 90mm		2.0				VD			BOREHOLE TERMINATED AT 2.1m
								3.0							

TONKIN & TAYLOR LTD
CONSULTING ENGINEERS

SITE DETAILS

FIELD LOG

BOREHOLE No: 2
SHEET 1 OF 1

PROJECT: THEATRE ROYAL — LOCATION: MAJORIBANK ST., WELLINGTON — JOB No: 80132

CO-ORDINATES — DRILL TYPE: TRUCK MOUNTED RIG — HOLE STARTED: 16/11/92

HOLE FINISHED: 17/11/92

RL. — DRILL METHOD: WASH DRILLING — DRILLED BY: PENDLETON CORP.

DATUM — DRILL FLUID: — LOGGED BY: GRF CHECKED:

GEOLOGICAL	DRILLING AND TESTS								ENGINEERING DESCRIPTION						
GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY	METHOD	CASING	TESTS.	SAMPLES	RL (m) / DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/ DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa) / COMPRESSIVE STRENGTH (MPa)	DEFECT SPACING (mm)	SOIL DESCRIPTION: Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION: Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.
BASECOURSE								0.0							ASPHALTIC CONCRETE BASECOURSE
RECLAMATION FILL															GRAVEL (fine/med.), sandy, loose, brown-grey
OLD BEACH DEPOSITS			80	SPT		N=16	2 5 8	1.0				MD			SAND (coarse), bands of gravels (fine/med.), grey-black, med. dense
			100	SPT		N=29	5 10 19	2.0			M	MD			
							7	3.0			M	MD			SAND (coarse), bands of gravels (fine/med.), grey-black, dense
			20	SPT		N=22	10 12								
			100	SPT		N=15	6 10 5	4.0			M	MD			SILT, grey, sl. plastic, very stiff
ALLUVIUM/ HARBOUR SEDIMENT			60	SPT		•WC=17.5% N=15	6 9 6	5.0			M	MD			SILT, grey, sl. plastic, occ. gravel (fine to medium),
			80	SPT		N=14	5 9 5	6.0			M	MD			
			100	SPT		N=7	5 4 3	7.0			M	L			SILT, grey, sl. plastic, light grey, occ. organics
			80	SPT		•WC=21.8% N=6	2 4 2	8.0			M	L			
COLLUVIUM			100	SPT		50 blows for 150mm 40 blows for 130mm	50/150 50/130	9.0			D	VD			GRAVEL (Fine/med.) dense, brown
								10.0							BOREHOLE TERMINATED AT 9.5m

TONKIN & TAYLOR LTD
CONSULTING ENGINEERS

SITE DETAILS

FIELD LOG

BOREHOLE No: 3
SHEET 1 OF 1

PROJECT: THEATRE ROYAL LOCATION: MAJORIBANK ST., WELLINGTON JOB No: 80132

CO-ORDINATES
RL.
DATUM

DRILL TYPE: TRUCK MOUNTED RIG
DRILL METHOD: WASH DRILLING
DRILL FLUID:

HOLE STARTED: 17/11/92
HOLE FINISHED: 18/11/92
DRILLED BY: PENDLETON CORP.
LOGGED BY: GRF CHECKED:

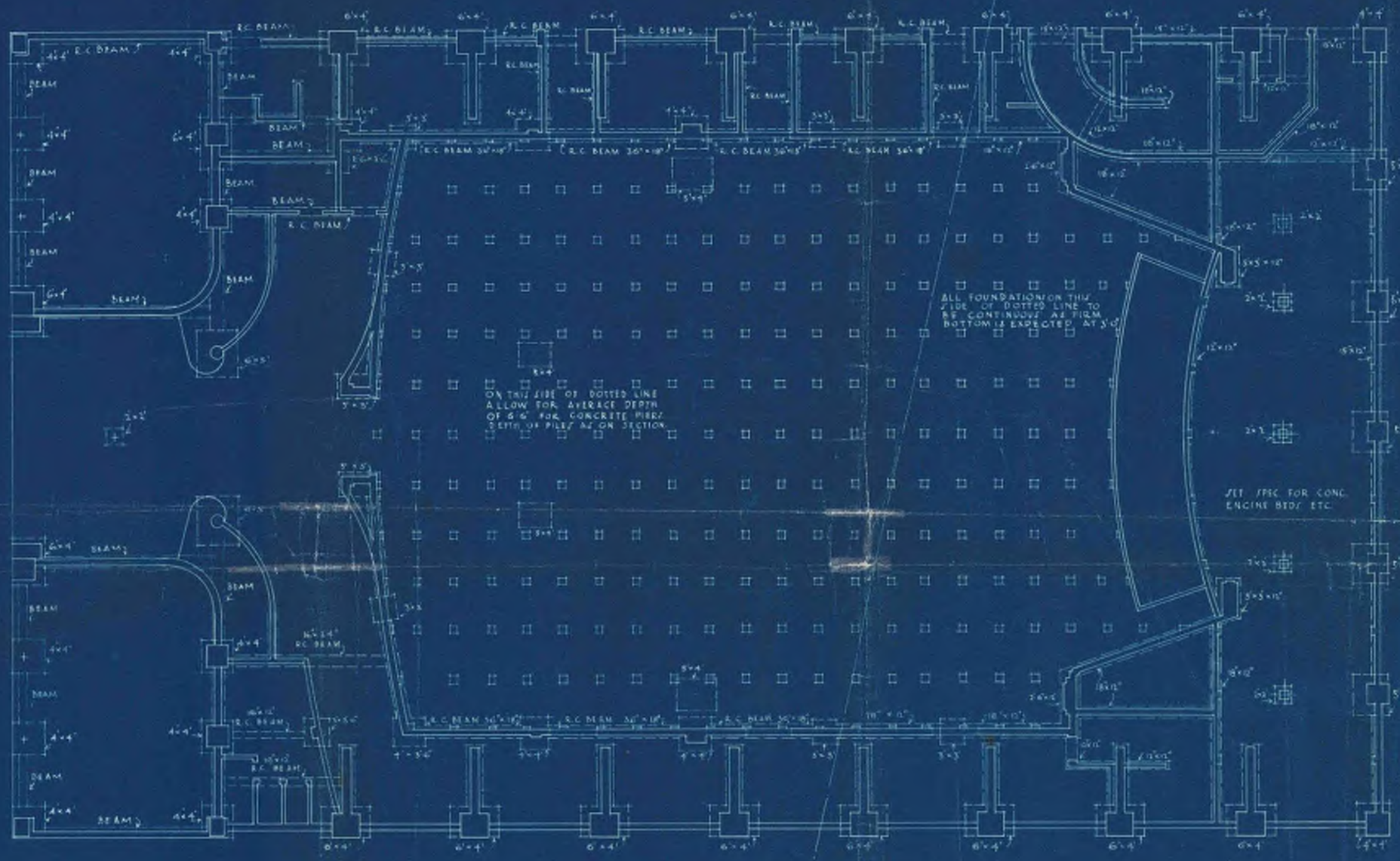
GEOLOGICAL	DRILLING AND TESTS								ENGINEERING DESCRIPTION						
GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY	METHOD	CASING	TESTS.	SAMPLES	RL (m) / DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/ DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa) / COMPRESSIVE STRENGTH (MPa)	DEFECT SPACING (mm)	SOIL DESCRIPTION: Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION: Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.
BASECOURSE															100mm ASPHALTIC CONCRETE
RECLAMATION FILL			0			1/6 N=13		1.0							GRAVEL (Fine/heavy), sandy, in greywacke matrix, medium-dense — rounded stones
OLD BEACH DEPOSITS		▽	100	SPT		3/4/2 N=6		2.0							SAND, fine, gravelly, loose
SILT			100	SPT		WC=20% 2/2/6 N=8		3.0							SILT, grey, loose, with occ. bands off fine to med. gravels.
			90	SPT		4/4/8 N=12		4.0							
			80	SPT		WC=17.9% 4/5/6 N=11		5.0							SILT, brown, medium-dense, brown to yellow-brown
COLLUVIUM			50	SPT		10/14/34 N=48		6.0							GRAVEL (Fine/medium), brown, dense.
			100	SPT		42 blows For 170 mm		7.0							BOREHOLE TERMINATED AT 7.2m

Appendix C Drawings

SHEET No 1
FOUNDATION PLAN
SCALE: 1/8" = 1'-0"
DATE - 1 March 1923

NEW THEATRE · COURTENAY PLACE · WELLINGTON ·
FOR THE
DE LUXE THEATRE CO. LTD.

LLEWELLYN E. WILLIAMS ·
A.R.I.B.A., M.I.S.E. Lond.
ARCHITECT · STRUCTURAL
ENGINEER · WELLINGTON ·



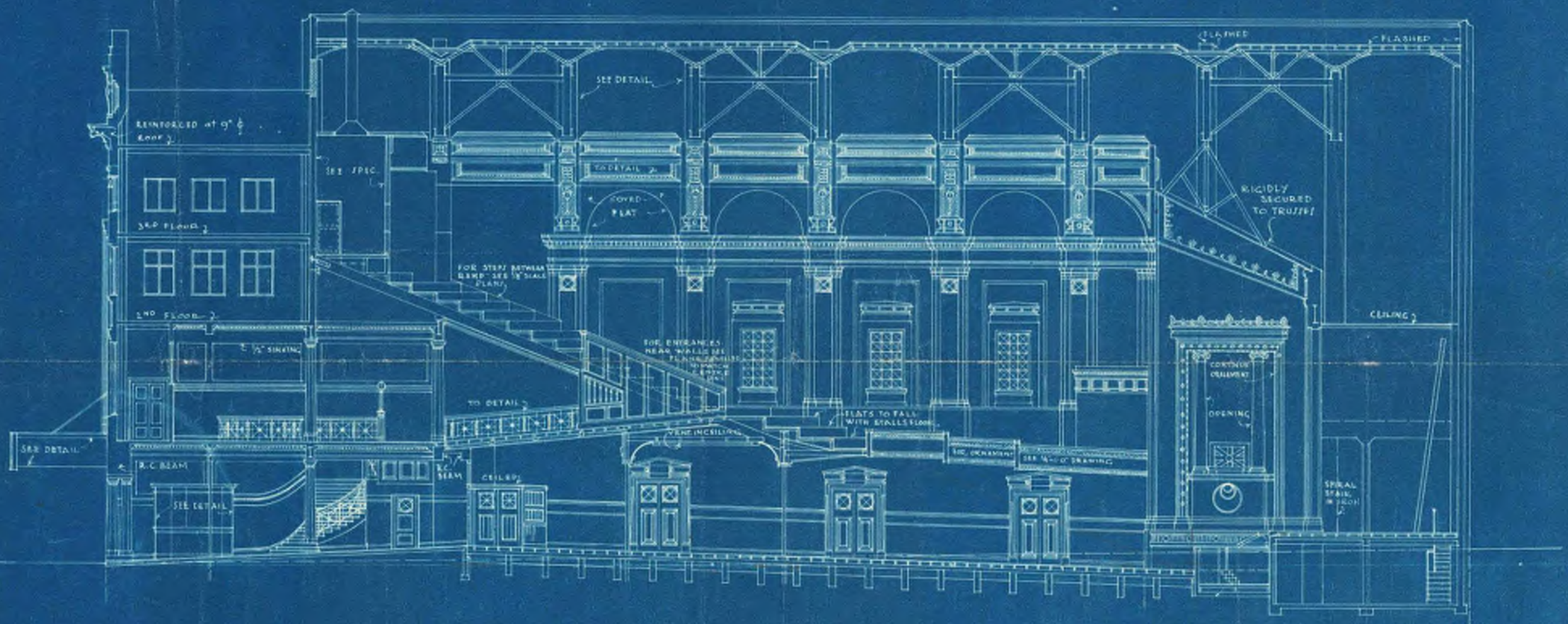
SHEET No 5
LONG^L SECTION
SCALE. 1/8" = 1'-0"
DATE - 1 March 1913

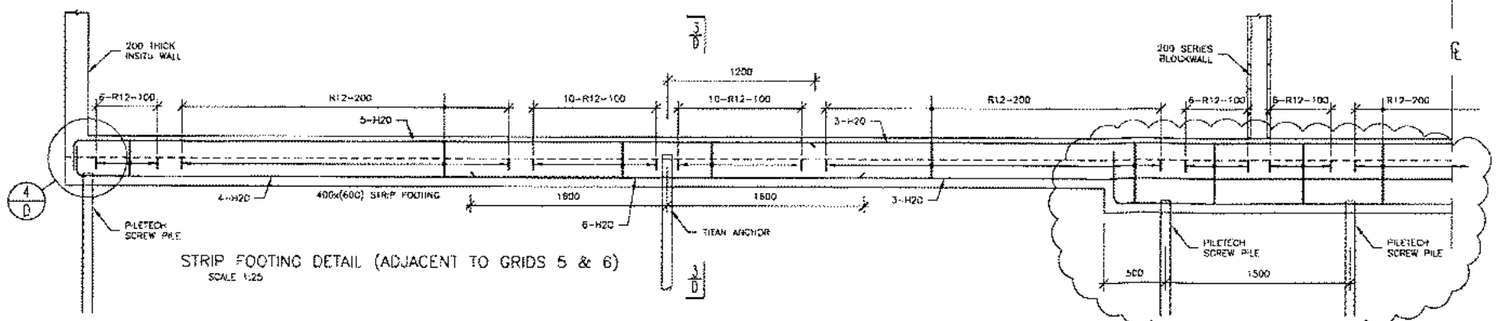
NEW THEATRE . COURTENAY PLACE . WELLINGTON .

FOR THE

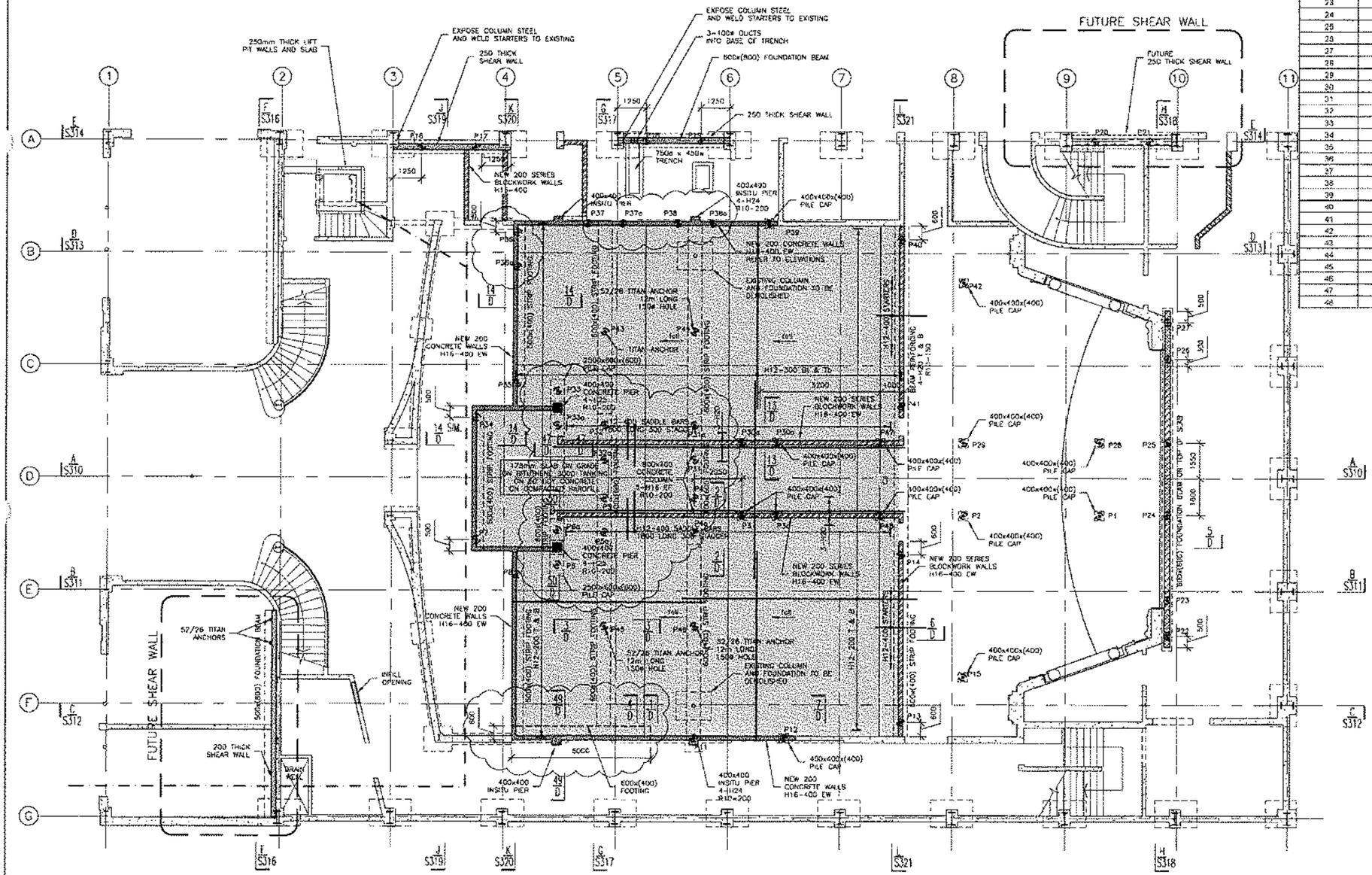
DE LUXE THEATRE CO. LTD.

LLEWELLYN E WILLIAMS .
A.R.I.B.A., M.I.S.E. Lond.
ARCHITECT-STRUCTURAL
ENGINEER-WELLINGTON.





STRIP FOOTING DETAIL (ADJACENT TO GRIDS 5 & 6)
SCALE 1:25



PILE NO	G (kN)	Q (kN)	EQ (kN)	CWL (kN)	1.4G (kN)	1.2G + 1.6Q (kN)	G + 0.7Q (kN)	0.9G - 1.2GWL (kN)	G + 0.4Q - E (kN)	G + 0.4Q + E (kN)
1	103	86			144	261	163		137	137
2	156	96			218	344	225		195	195
3	262	157			367	566	372		325	325
4	340	264			476	830	525		446	446
5	251	124	230		351	500	338		71	531
6	129	224	125		181	513	286		64	344
7	78	25	125		109	134	96		-37	213
8	168	99	125		235	360	237		63	333
9	289	144	184		405	577	390		183	531
10	233	146	115		326	613	335		178	406
11	336	244			470	794	507		434	434
12	171	120	115		239	397	255		194	334
13	121	54	184		169	232	159		-41	327
14	143	84	184		200	274	188		-15	363
15	160	105			224	362	234		202	202
16									-350	750
17									-350	750
18									-350	750
19									-350	750
20									-350	750
21									-350	750
22	430		310						120	740
23	430		310						120	740
24	430		310						120	740
25	430		310						120	740
26	430		310						120	740
27	430		310						120	740
28	103	86			144	261	163		137	137
29	156	98			218	344	225		195	195
30	262	157			367	566	372		325	325
31	340	264			476	830	525		446	446
32	251	124	230		351	500	338		71	531
33	129	224	125		181	513	286		64	344
34	78	25	125		109	134	96		-37	213
35	168	99	125		235	360	237		63	333
36	289	144	184		405	577	390		183	531
37	233	146	115		326	613	335		178	406
38	336	244			470	794	507		434	434
39	171	120	115		239	397	255		194	334
40	121	54	184		169	232	159		-41	327
41	143	84	184		200	274	188		-15	363
42	160	105			224	362	234		202	202
43	96	91		-342	134	261	160	-324		
44	96	91		-342	134	261	160	-324		
45	96	91		-342	134	261	160	-324		
46	96	91		-342	134	261	160	-324		
47	175	105	184		245	378	249		33	401
48	175	105	184		245	378	249		33	401

PILE LOADS

THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

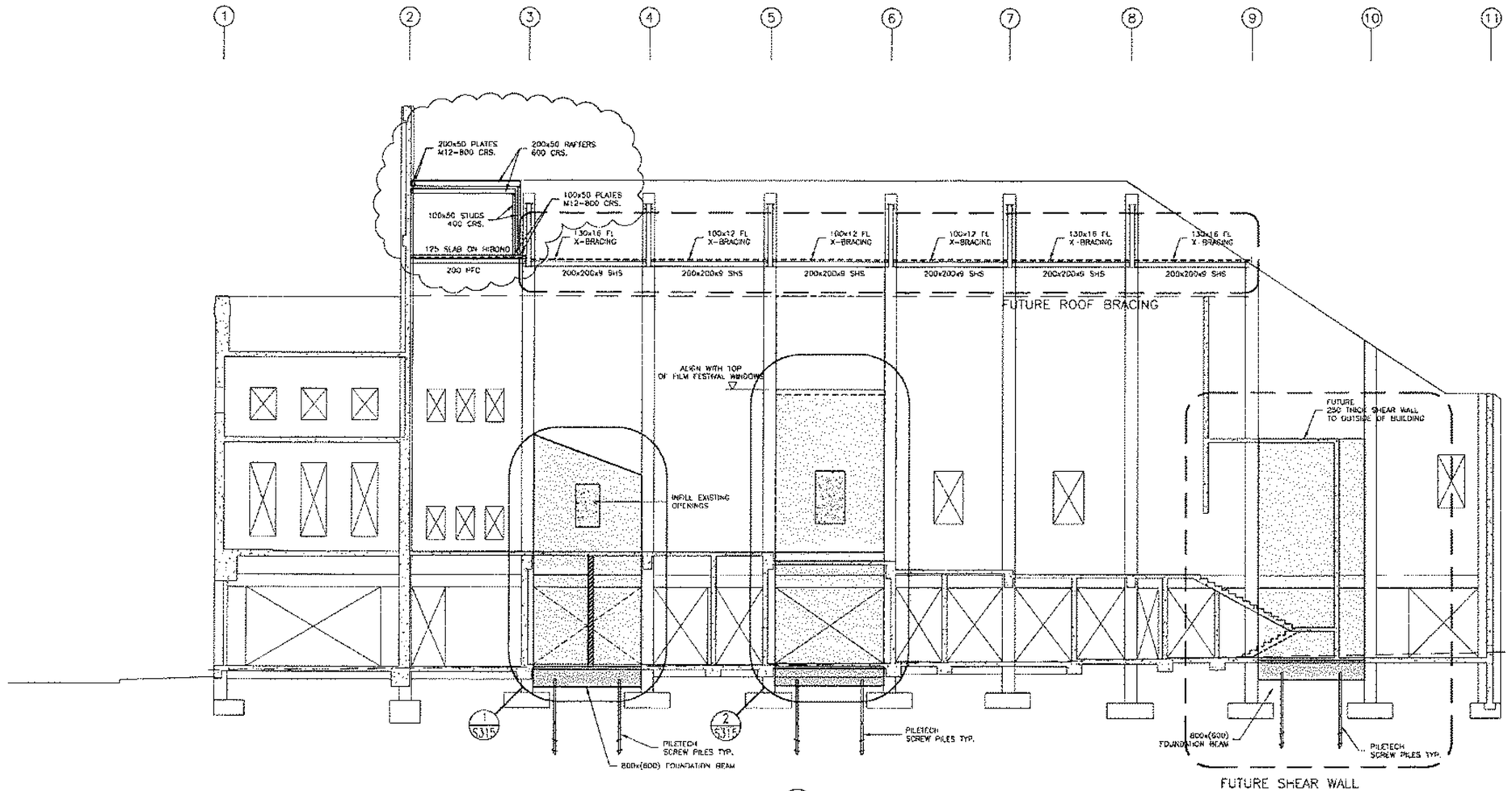
LEVEL 0 PILE LAYOUT & LOWER SLAB DETAILS

REVISIONS		
3	REVISED	22-07-2003
2	FOR CONSTRUCTION	22-05-2003
1	FOR CONSENT	3-05-2003
0	PILING TENDER	22-05-2003

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864
Drawing Number S300
Rev 3

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz



E S300 ELEVATION GRID A

SCALE 1:100

THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

ELEVATION GRID A

REVISIONS		
3	REVISED	22-07-2003
2	FOR CONSTRUCTION	20-06-2003
1	FOR CONSENT	3-06-2003

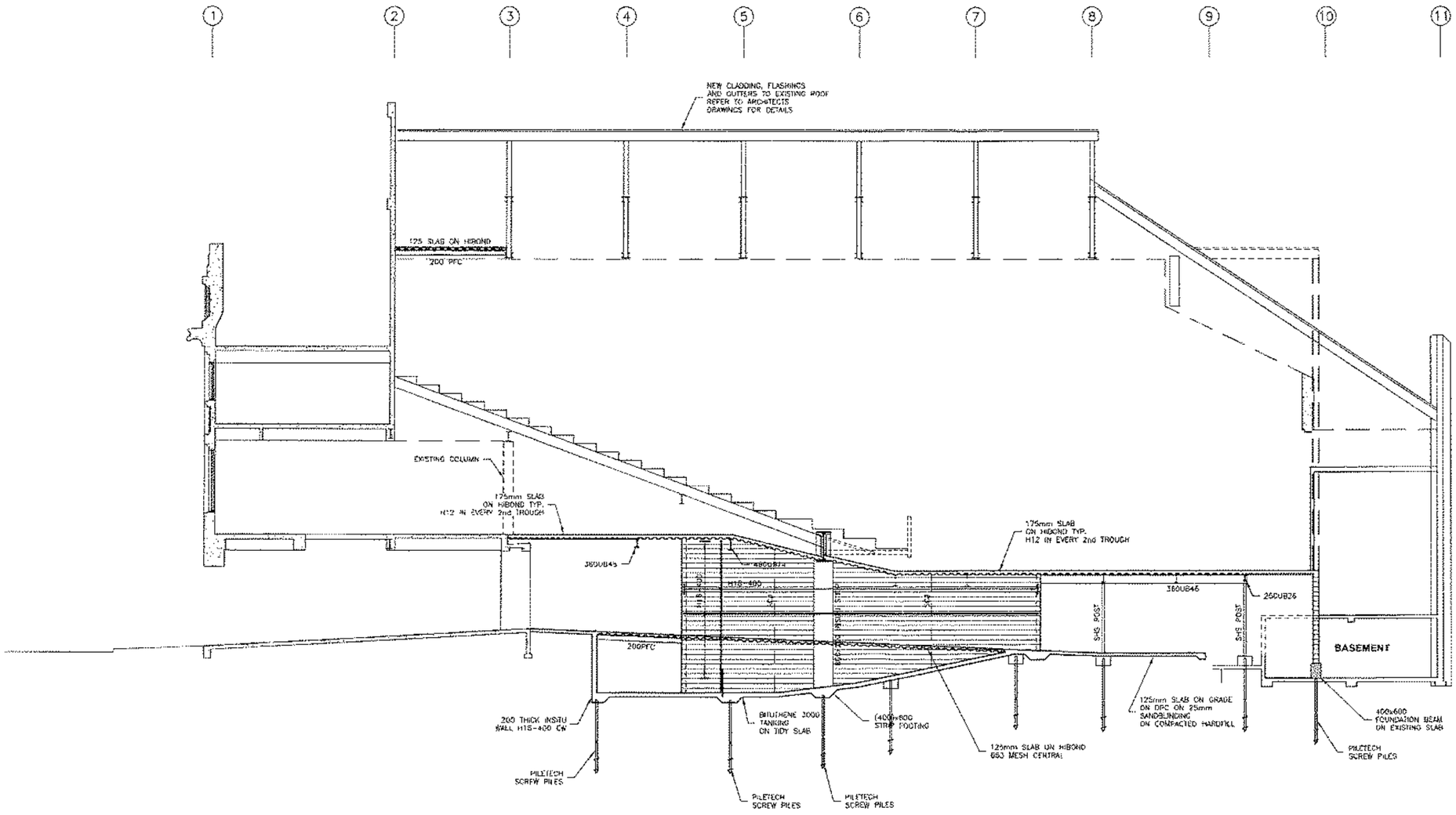
Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864

Drawing Number S314 Rev 3

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0015
Fax (644) 385-0312
Box 27 153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz



A S300 SECTION GRID D
SCALE 1:100

EMBASSY REDEVELOPMENT 2003

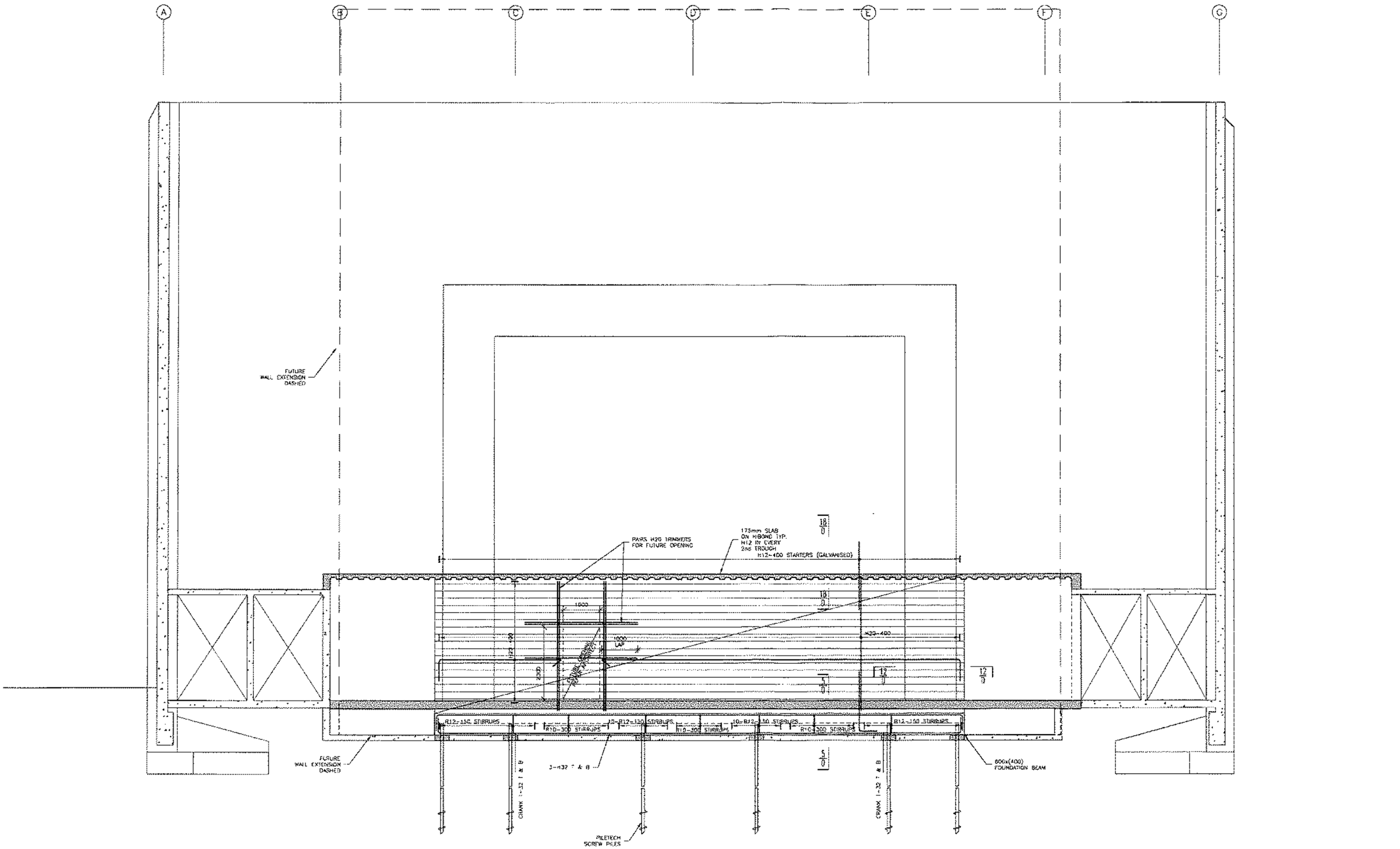
SECTION GRID D

REVISIONS		
2	FOR CONSTRUCTION	20.06.2003
1	FOR CONSENT	3.06.2003
0	PRIME TRANSFER	27-05-2003

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864
Drawing Number S310
Rev 2

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-163 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz



THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

SECTION GRID 10

REVISIONS		
2	FOR CONSTRUCTION	20-08-2003
1	FOR CONSENT	3-06-2003

Scales	1:50
A3 Scales	1:100
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

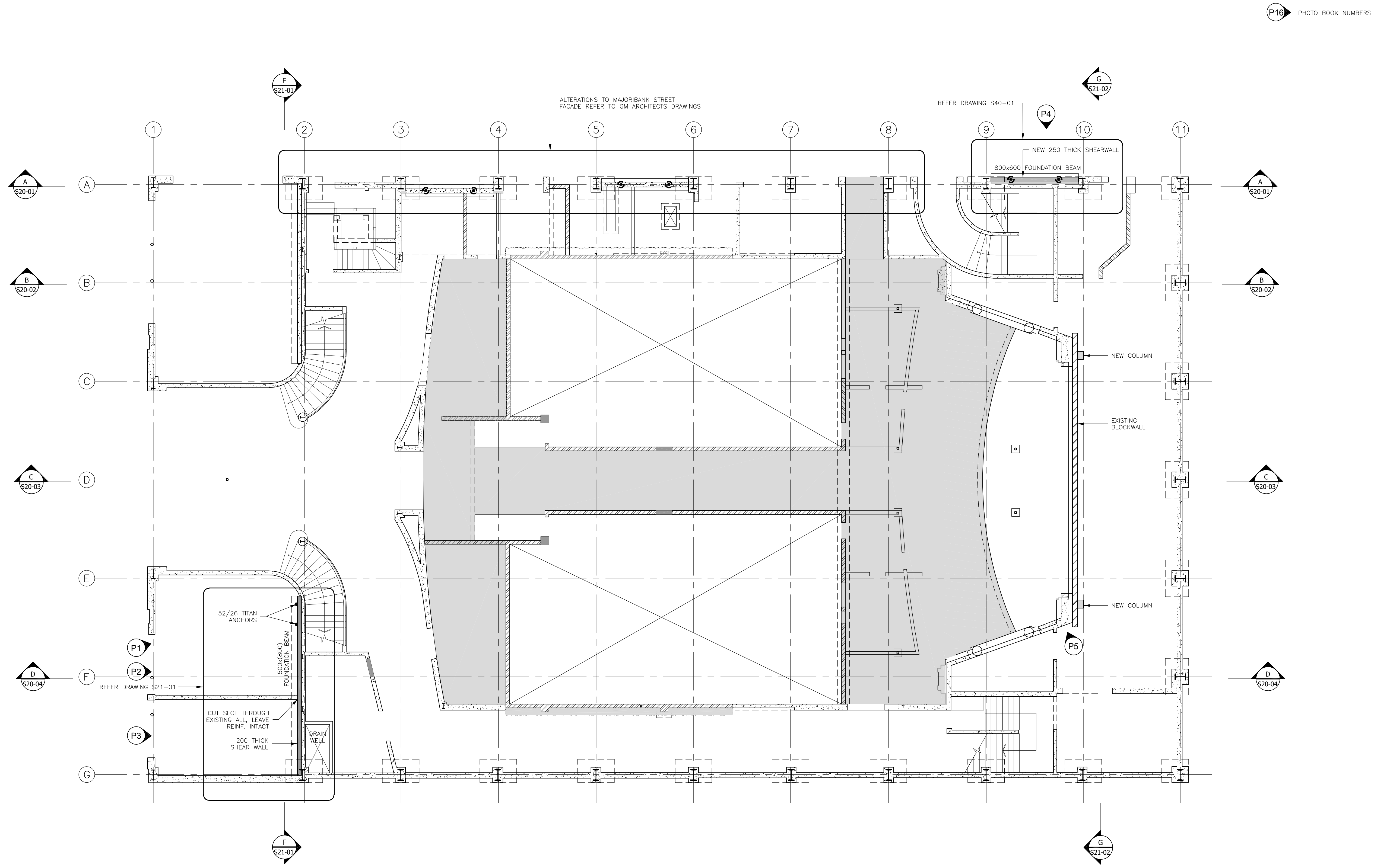
Job Number 3864

Drawing Number S318 Rev 2

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-2212
Box 27-163 Wellington, N.Z.
E-mail otcwgtn@dunningthornton.co.nz

Original Size A1



Original Scale

Original Size A1

ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

LEVEL 0
SLAB PLAN

REVISIONS		
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Julie nicholson
CAD Reference	3864-2009

Job Number 3864
Drawing Number S2-01
Rev 1

ACENZ
ISO 9001

Dunning Thornton consultants

p: 385 0019 f: 385 0312 e: dtcwgtn@dunningthornton.co.nz P.O. Box 27-153 Wellington

IF IN DOUBT ASK

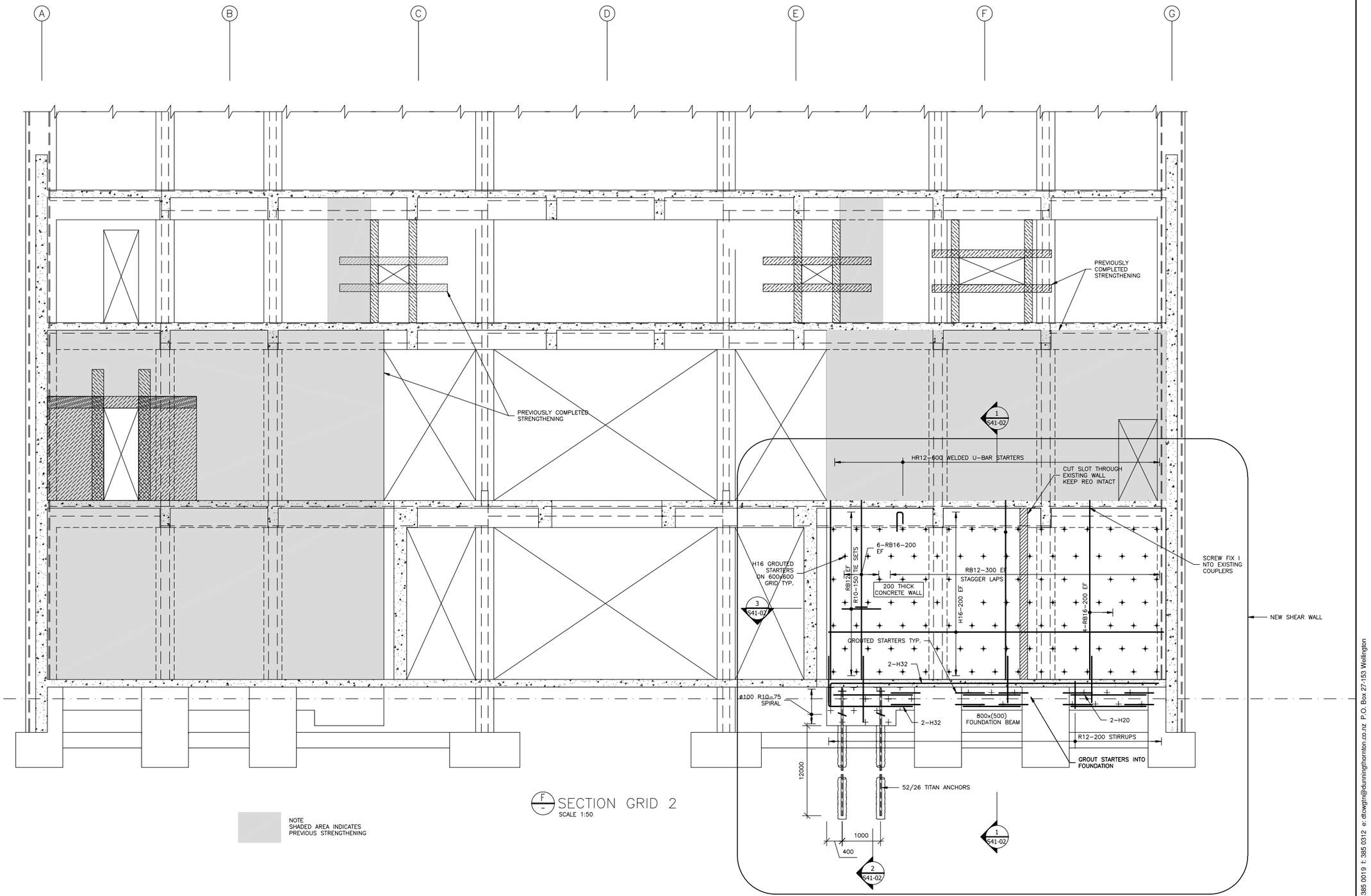
DO NOT SCALE

Original Size A1

100 50 0 10

Original Scale

Original Size A1



F SECTION GRID 2
SCALE 1:50

NOTE
SHADED AREA INDICATES PREVIOUS STRENGTHENING

ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

SECTION GRID 2

REVISIONS		
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Julie NICHOLSON
CAD Reference	3864-2009

Job Number 3864

Drawing Number S21-01 Rev 1

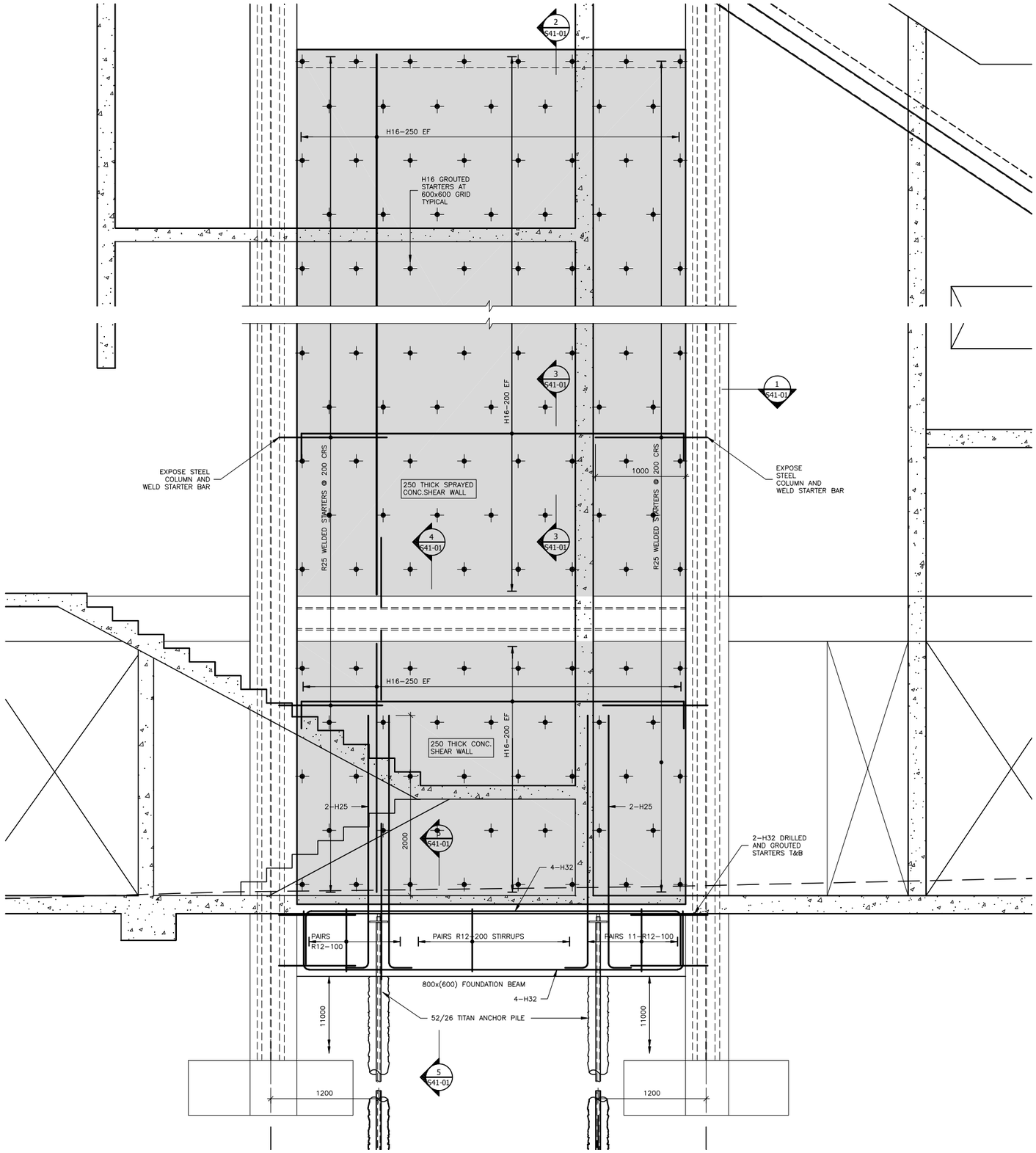
ISO 9001

Dunning Thornton consultants

p: 385 0019 f: 385 0312 e: dtcwgtn@dunningthornton.co.nz P.O. Box 27-153 Wellington

IF IN DOUBT ASK

DO NOT SCALE



ABSOLUTELY POSITIVELY WELLINGTON

EMBASSY STRENGTHENING 2009

GRID A
SHEARWALL

REVISIONS		
1	FOR CONSTRUCTION	9/04/2010
0	FOR CONSENT	21/12/2009
C	PRELIMINARY	7/12/2009
B	PRELIMINARY	20/11/2009
A	PRELIMINARY	9/11/2009

Scales	1:25
A3 Scales	1:50
Designed	Adam Thornton
Drawn	Julie NICHOLSON
CAD Reference	3864-2009

Job Number 3864
Drawing Number S40-01
Rev 1

ISO 9001

Dunning Thornto consultan

IF IN DOUBT ASK

DO NOT SCALE

Appendix D Shallow Foundation Geotechnical Parameters

- Summary figure of agreed geotechnical parameters.
- Figure D1: Passive load-displacement curve for 1.5m deep pad foundation.

Zone C
Susceptible to liquefaction.

kv = 10 to 40 kPa/mm
kv (liquefied conditions) = 1 to 8 kPa/mm

Refer to the orange line on Figure 1 for Passive capacity load-displacement plot for pad foundations.

Note that the bearing capacity in liquefied conditions reduces significantly if horizontal load is experienced at the base of the footing.

No Liquefaction

H/V	Net Ultimate (Limiting) B.C (kPa)
0	1030
0.1	750
0.2	530
0.3	370
0.4	250

Liquefaction

H (kN)	Net Ultimate B.C (liquefied conditions) (kPa)
0	50 to 180

Embassy Theatre DSA, Wellington
T+T Ref: 1020459

Shallow Foundation Bearing Capacity and Springs
Rev: 0
Date: 1/06/2022
Drawn: EMPE
Checked: BHR / ELC
Page 1/2

Shallow Foundation Plan

- 1.22 x 1.22 m (4' x 4')
- 1.82 x 1.22 m (6' x 4')
- 1.82 x 1.52m (6' x 5')
- 1.22 x 1.52m (4' x 5')
- 0.91 x 0.91 m (3' x 3')
- 0.91 x 1.52 m (3' x 5')

All pad foundations assumed to be founded 1.5m bgl. Foundations below original 1923 basement assumed to be founded 0.5m below basement level.

Zone B
Susceptible to liquefaction.

No Liquefaction

H/V	Net Ultimate (Limiting) B.C (kPa)
0	700
0.1	500
0.2	350
0.3	250
0.4	170

Liquefaction

H (kN)	Net Ultimate B.C (liquefied conditions) (kPa)
0	50 to 180

kv = 10 to 40 kPa/mm
kv (liquefied conditions) = 1 to 8 kPa/mm

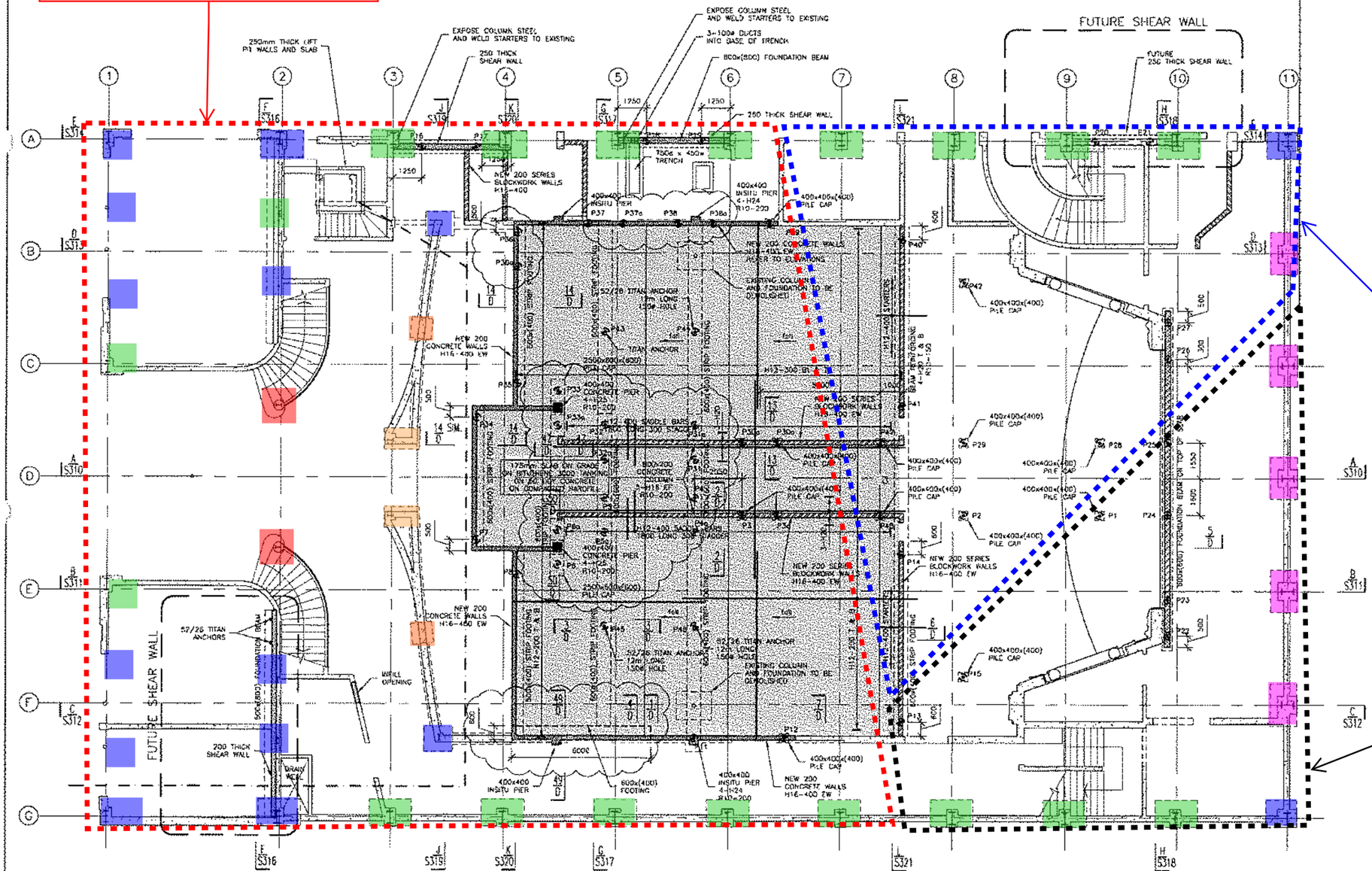
Refer to the orange line on Figure D1 for Passive capacity load-displacement plot for pad foundations.

Note that the bearing capacity in liquefied conditions reduces significantly if horizontal load is experienced at the base of the footing.

Zone A
Not susceptible to liquefaction.

H/V	Net Ultimate (Limiting) B.C (kPa)
0	1200
0.1	860
0.2	620
0.3	430
0.4	300

kv = 50 to 250 kPa/mm



THEATRE EMBASSY TRUST	EMBASSY REDEVELOPMENT 2003	LEVEL 0 PILE LAYOUT & LOWER SLAB DETAILS	

REVISIONS		
3	REVISED	22-07-2003
2	FOR CONSTRUCTION	20-06-2003
1	FOR CONSENT	3-06-2003
0	PILING TENDER	22-05-2003

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300
Job Number	3864
Drawing Number	S300
Rev	3

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz

G:\3864\Dtc\Dwgs\3864S300.dwg, 23-07-2003 1:34:32 p., HP LaserJet 4MV.pc3

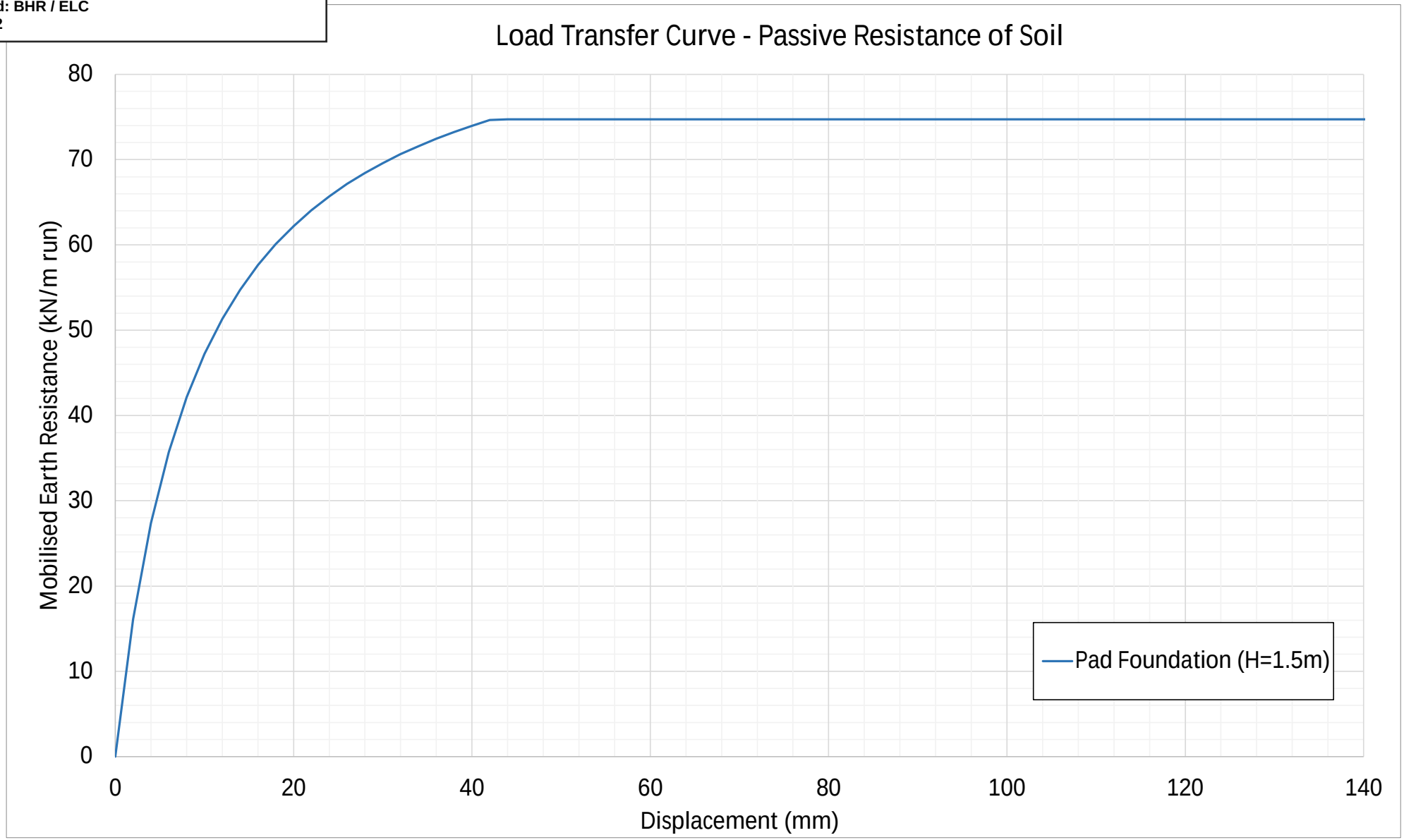


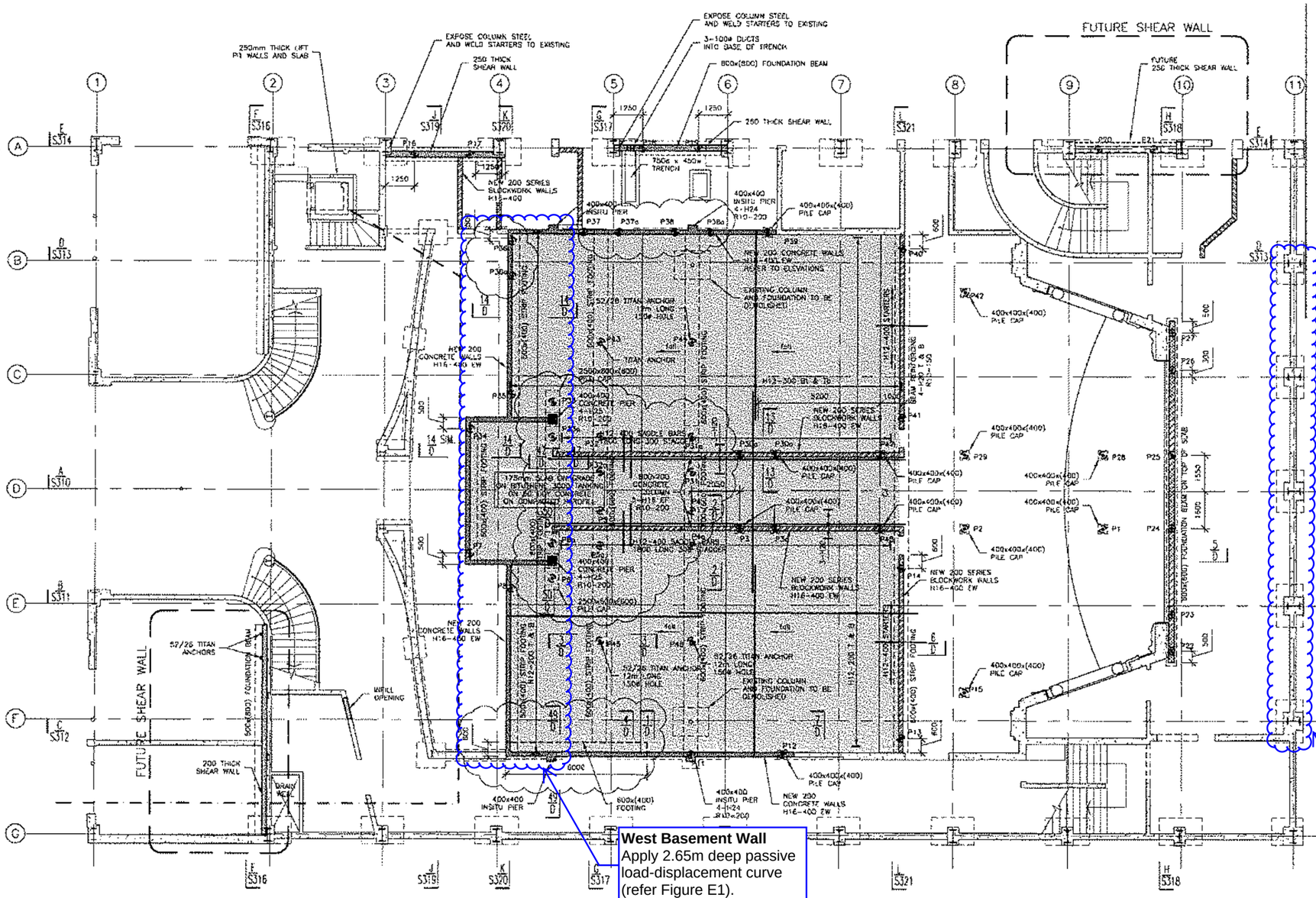
Figure D1. Passive load displacement curve for 1.5m deep pad foundation.

Appendix E Basement wall load-displacement curve

- Summary figure outlining basement walls.
- Figure E1: Passive load-displacement curve for 2.65m deep west basement wall.
- Figure E2: Passive load-displacement curve for 2.65m deep east basement wall.

Embassy Theatre DSA, Wellington
T+T Ref: 1020459

Basement Passive Load-Displacement Curves
Rev: 0
Date: 15/07/2022
Drawn: EMPE
Checked: BHR
Page 1/3



West Basement Wall
Apply 2.65m deep passive load-displacement curve (refer Figure E1).

East Basement Wall
Apply 1.8m deep passive load-displacement curve (refer Figure E2).

THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

LEVEL 0 PILE LAYOUT & LOWER SLAB DETAILS

REVISIONS		
3	REVISED	22-07-2003
2	FOR CONSTRUCTION	20-06-2003
1	FOR CONSENT	3-06-2003
0	PILING TENDER	22-05-2003

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864
Drawing Number S300
Rev 3

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz

G:\3864\Dtc\Dwgs\3864S300.dwg, 23-07-2003 1:34:32 p., HP LaserJet 4MV.pc3

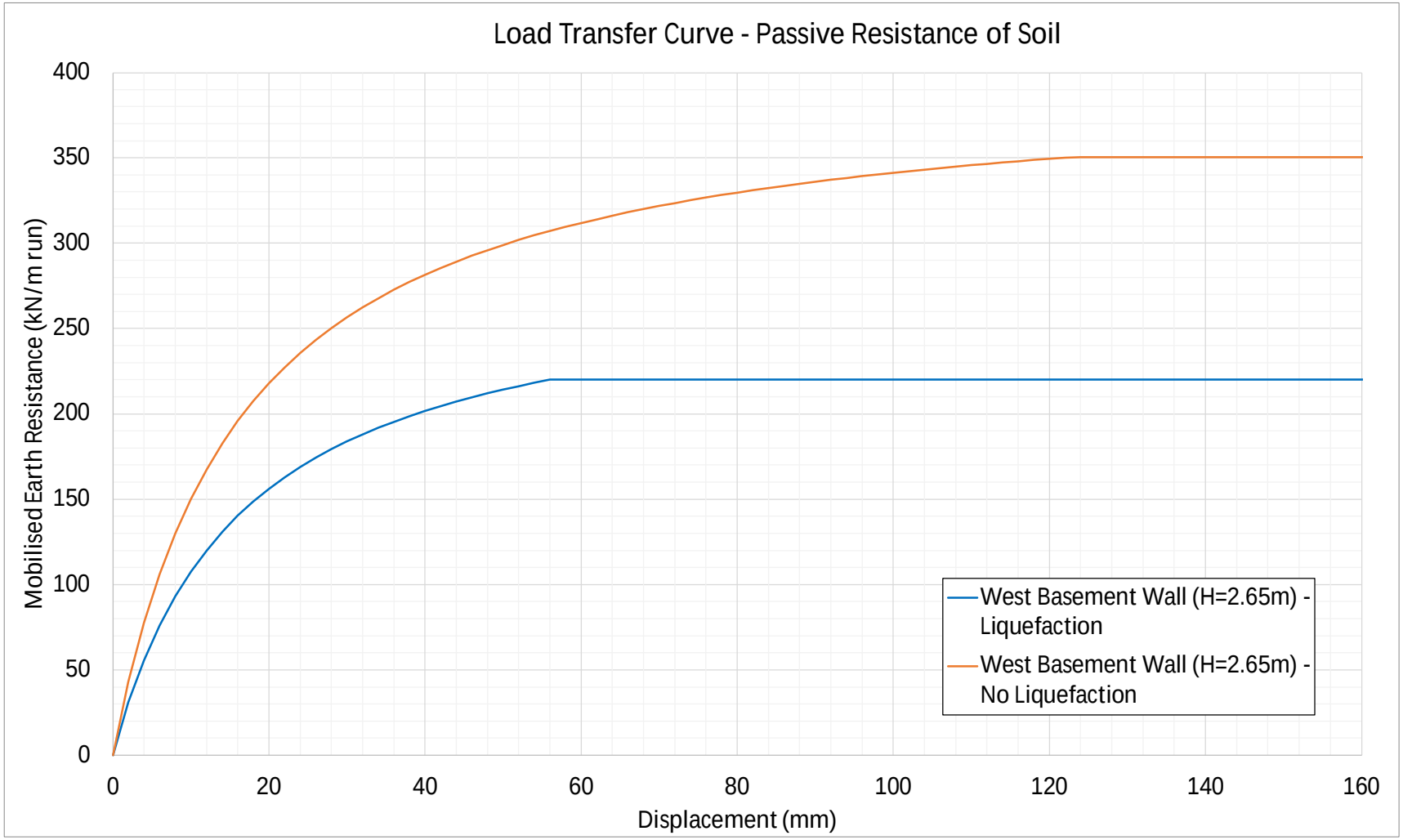


Figure E1. Passive load-displacement curve for 2.65m deep west basement wall.

Note that the mobilised earth resistance (kN/m) is per metre of basement wall perpendicular to direction of movement.

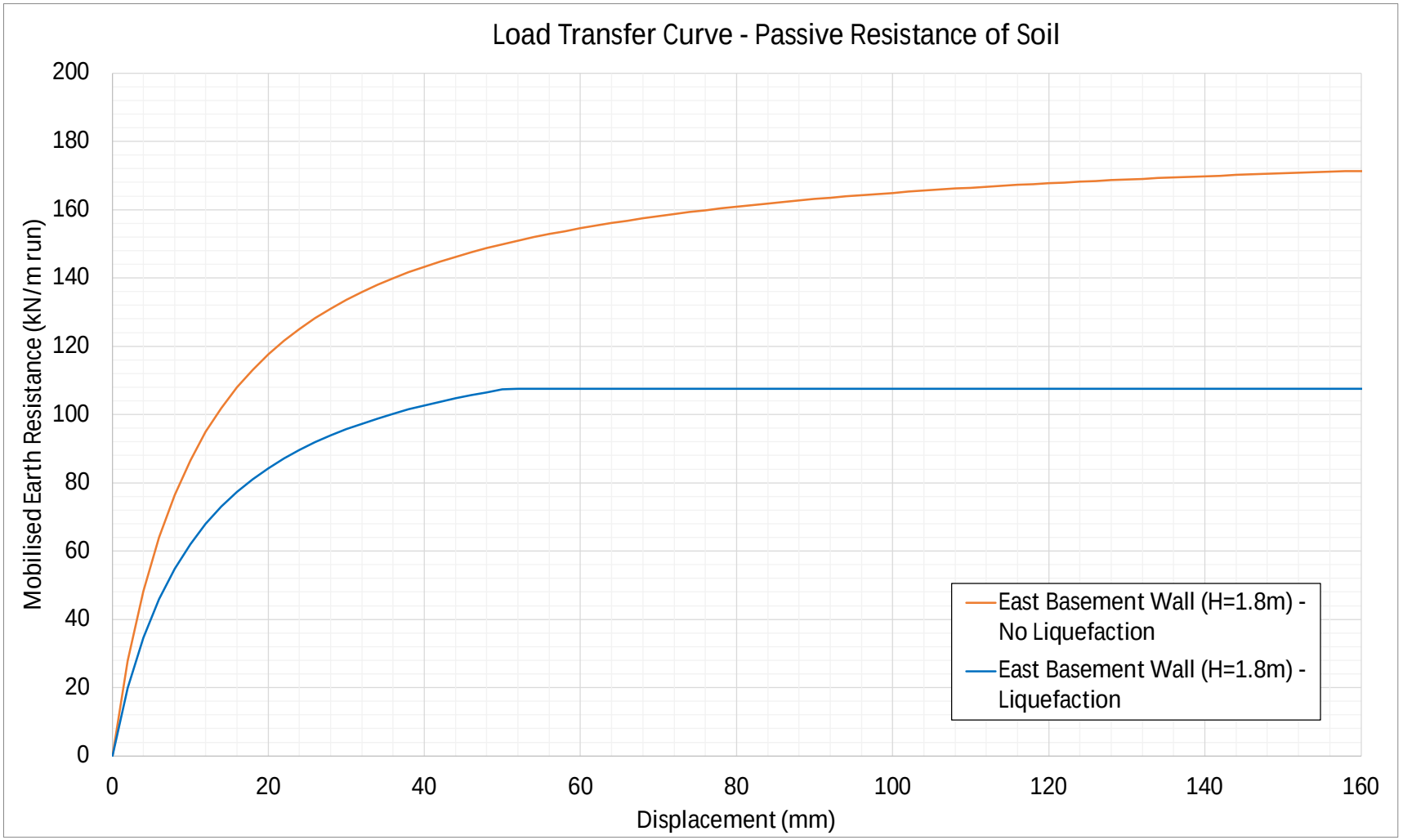


Figure E2. Passive load-displacement curve for 1.8m deep east basement wall.

Note that the mobilised earth resistance (kN/m) is per metre of basement wall perpendicular to direction of movement.

Embassy Theatre DSA, Wellington
T+T Ref: 1020459

Basement Passive Load-Displacement Curves
Rev: 0
Date: 15/07/2022
Drawn: EMPE
Checked: BHR
Page 3/3

Appendix F Deep Foundation Geotechnical Parameters

- Summary figure of agreed geotechnical parameters for screw pile and rock anchor foundations.
- LPile Analyses for screw pile and rock anchor foundations.
- Summary figure of agreed geotechnical parameters for micropile foundations.
- LPile Analyses for micropile foundations.

Embassy Theatre DSA, Wellington
T+T Ref: 1020459

Pile Foundation Bearing Capacity and Springs (No Liquefaction Case)
Rev: 0
Date: 25/07/2022
Drawn: EMPE
Checked:
Page:

Perimeter Piles
Pile type: Screw pile

Probable compression capacity = 1000 kN
Probable tension capacity = 500 kN

Compression springs = 20 to 100 kN/mm
Tension springs = 10 to 50 kN/mm

Refer LPile Analysis 1 and 2.

Basement Piles
Pile type: Screw pile and Screw pile/Rock anchor combo

Probable compression capacity = 1000 kN
Compression springs screw pile only = 20 to 100 kN/mm
Compression springs screw pile / rock anchor combo = 20 to 250 kN/mm

Refer LPile Analysis 3 for P37 to P39.
Analysis 4 for remaining screw piles.
Analysis 5 for Screw pile / rock anchor combo

Foundation Plan (2003 Strengthening)

SP - Screw Pile Length
RA - Rock Anchor Length

- Screw Pile Only (2003) 600 diameter helix
- Screw Pile and Rock Anchor Combination (2003) 200 diameter rock anchors
- Micropile (2009)
- Shallow foundation (1924)

Shallow Basement Piles
Pile type: Screw pile

Probable compression capacity = 1000 kN

Compression springs = 20 to 100 kN/mm

Refer LPile Analysis 6

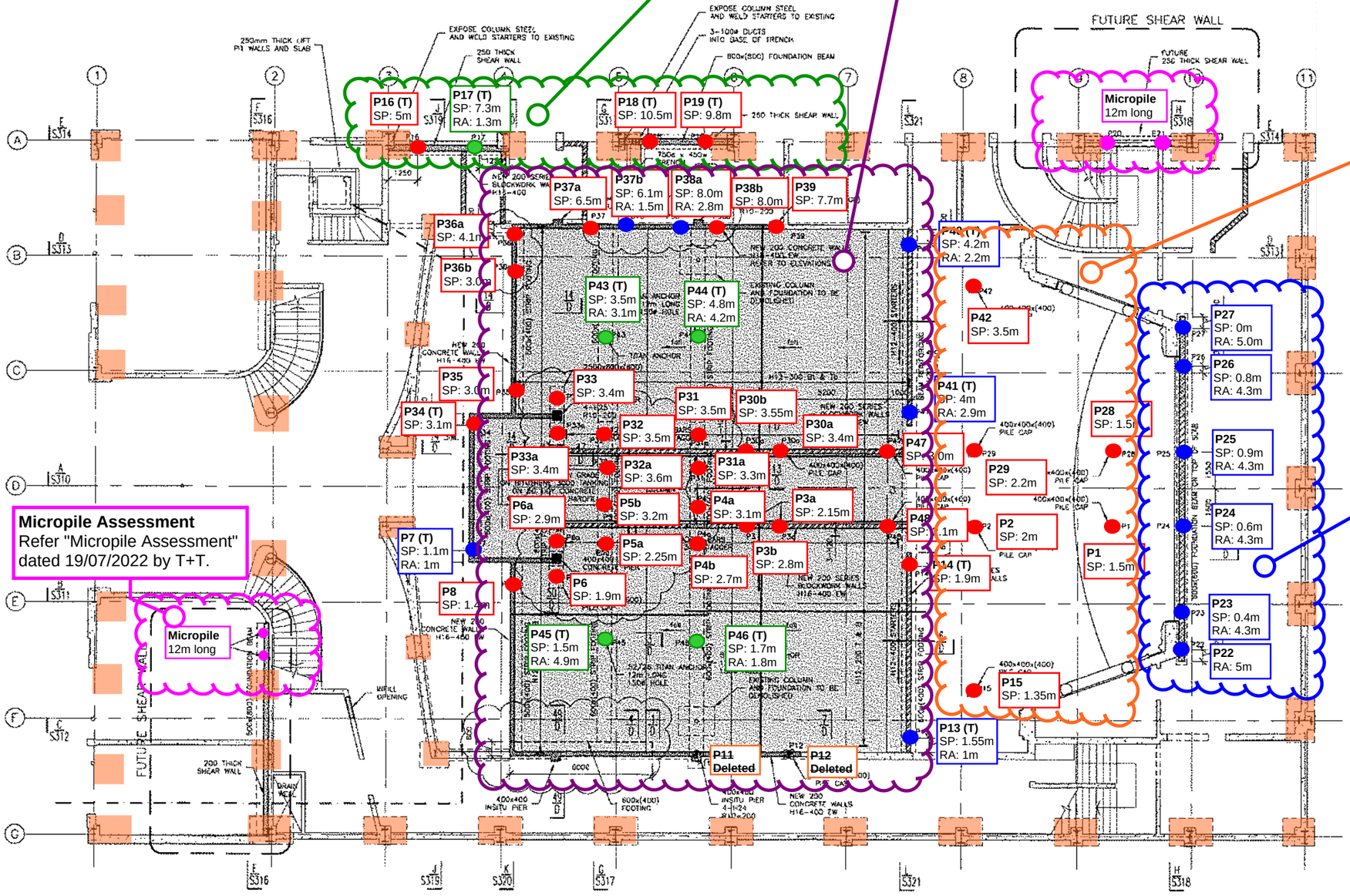
Grid 10 Rock Anchors
Pile type: Rock Anchor

Probable compression capacity = 1200 kN
Probable tension capacity = 1200 kN

Compression springs = 50 to 250 kN/mm
Tension springs = 15 to 75 kN/mm

Refer LPile Analysis 7 and 8.

Micropile Assessment
Refer "Micropile Assessment" dated 19/07/2022 by T+T.



THEATRE EMBASSY TRUST	EMBASSY REDEVELOPMENT 2003	LEVEL 0 PILE LAYOUT & LOWER SLAB DETAILS

REVISIONS		
3	REVISED	22-07-2003
2	FOR CONSTRUCTION	20-06-2003
1	FOR CONSENT	3-06-2003
0	PILING TENDER	22-05-2003

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864
Drawing Number S300
Rev 3

ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-0019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz

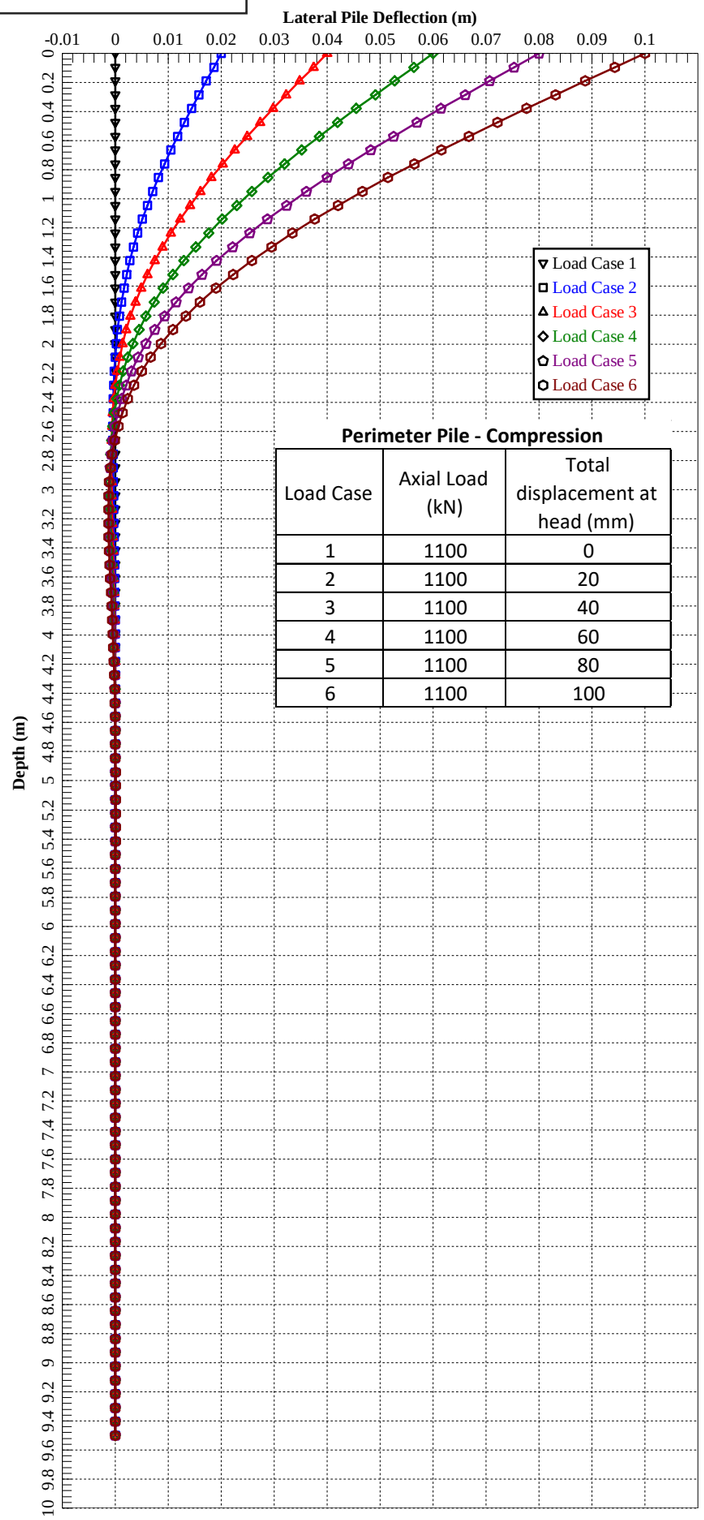
G:\3864\DtcDwgs\3864S300.dwg, 23-07-2003 1:34:32 p., HP LaserJet 4MV.pc3

Embassy Theatre DSA, Wellington
T+T Ref: 1020459
Pile Assessment
Rev: 0
Date: 25/07/2022

Pile Properties
D = 0.219m
EI = 6,734 kNm^2
EA = 1,890,660 kN

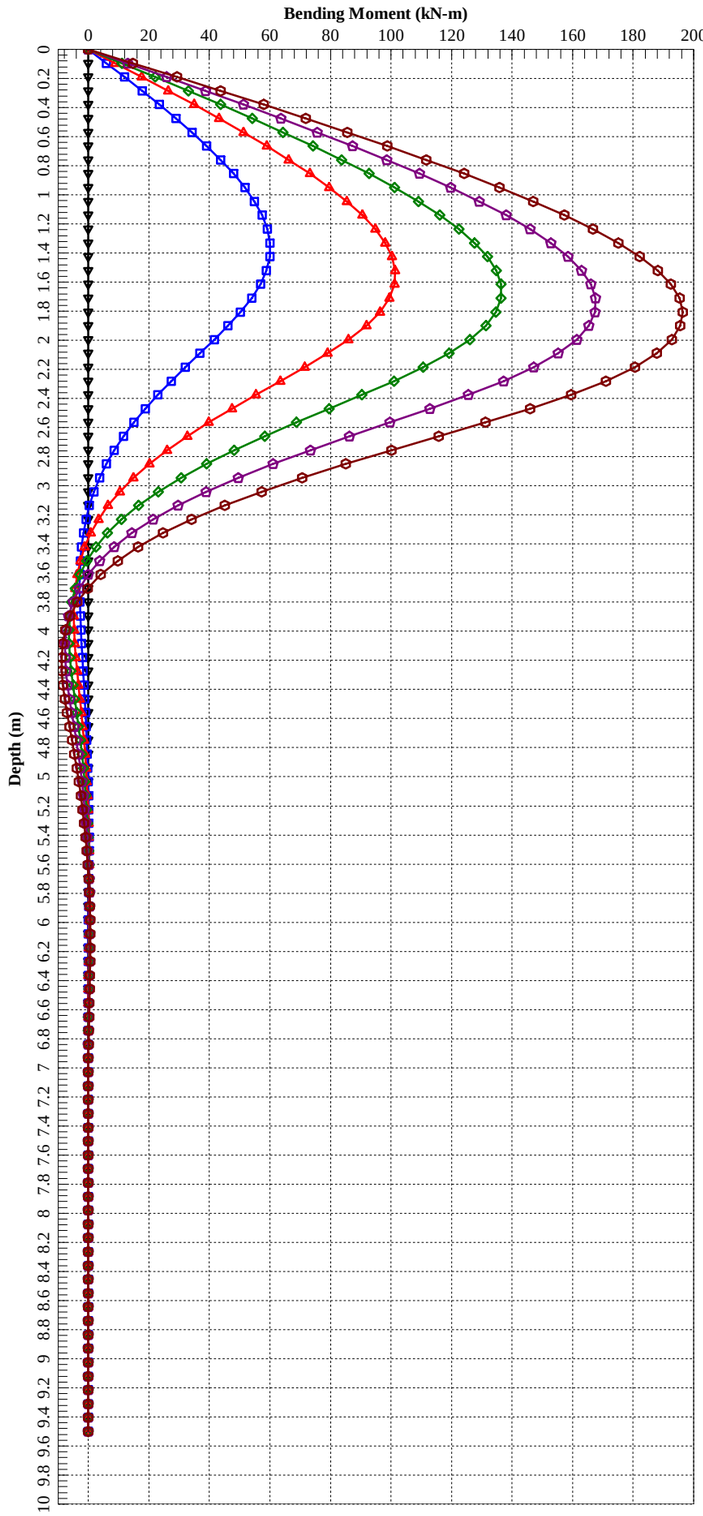
Perimeter Pile - No Liquefaction - Compression

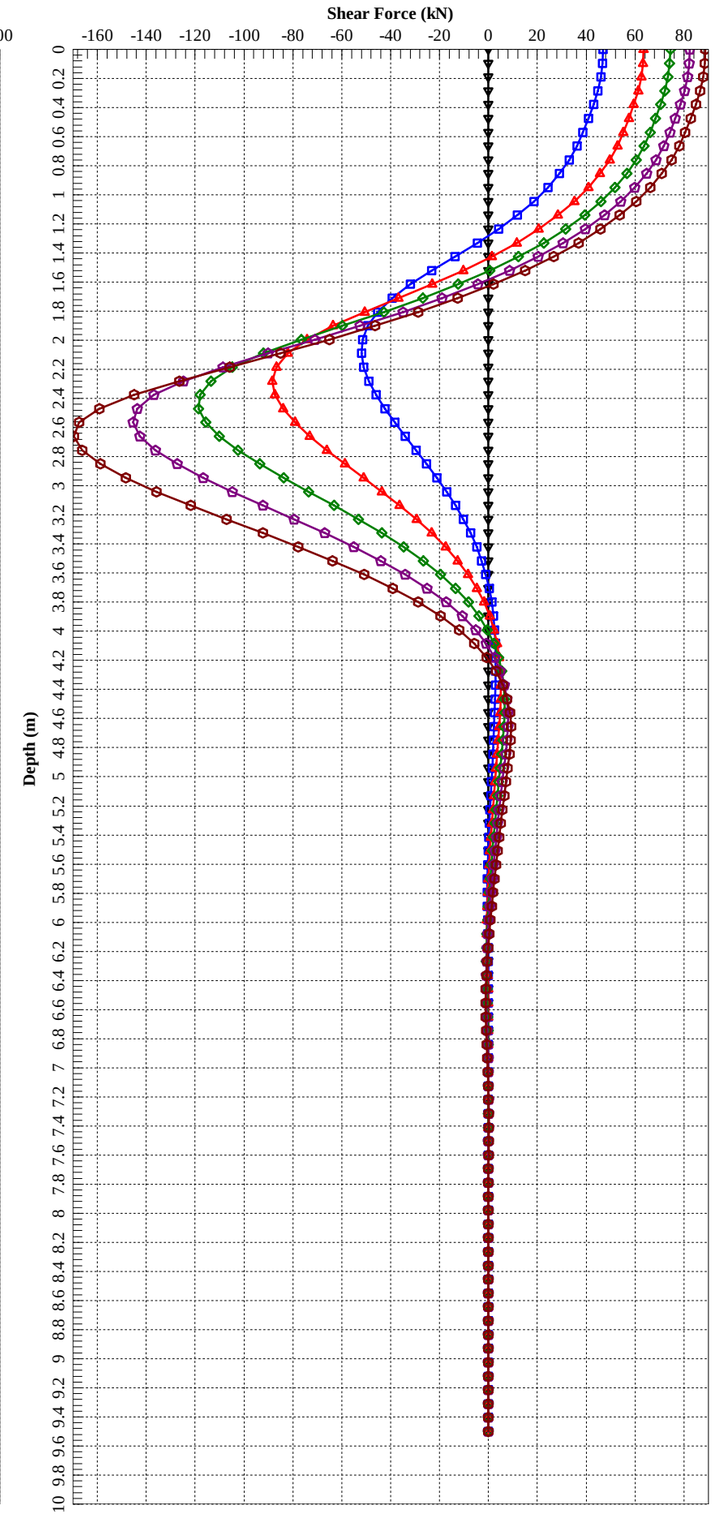
Analysis 1

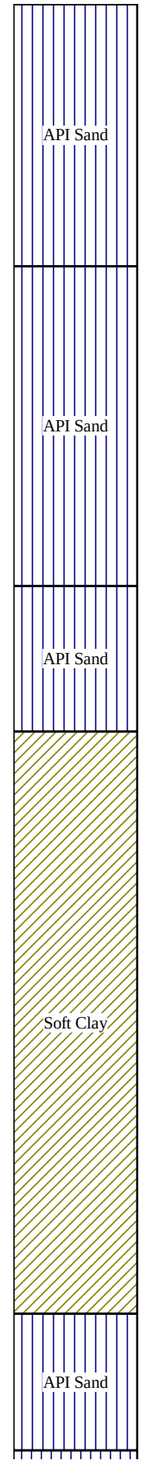


Perimeter Pile - Compression

Load Case	Axial Load (kN)	Total displacement at head (mm)
1	1100	0
2	1100	20
3	1100	40
4	1100	60
5	1100	80
6	1100	100





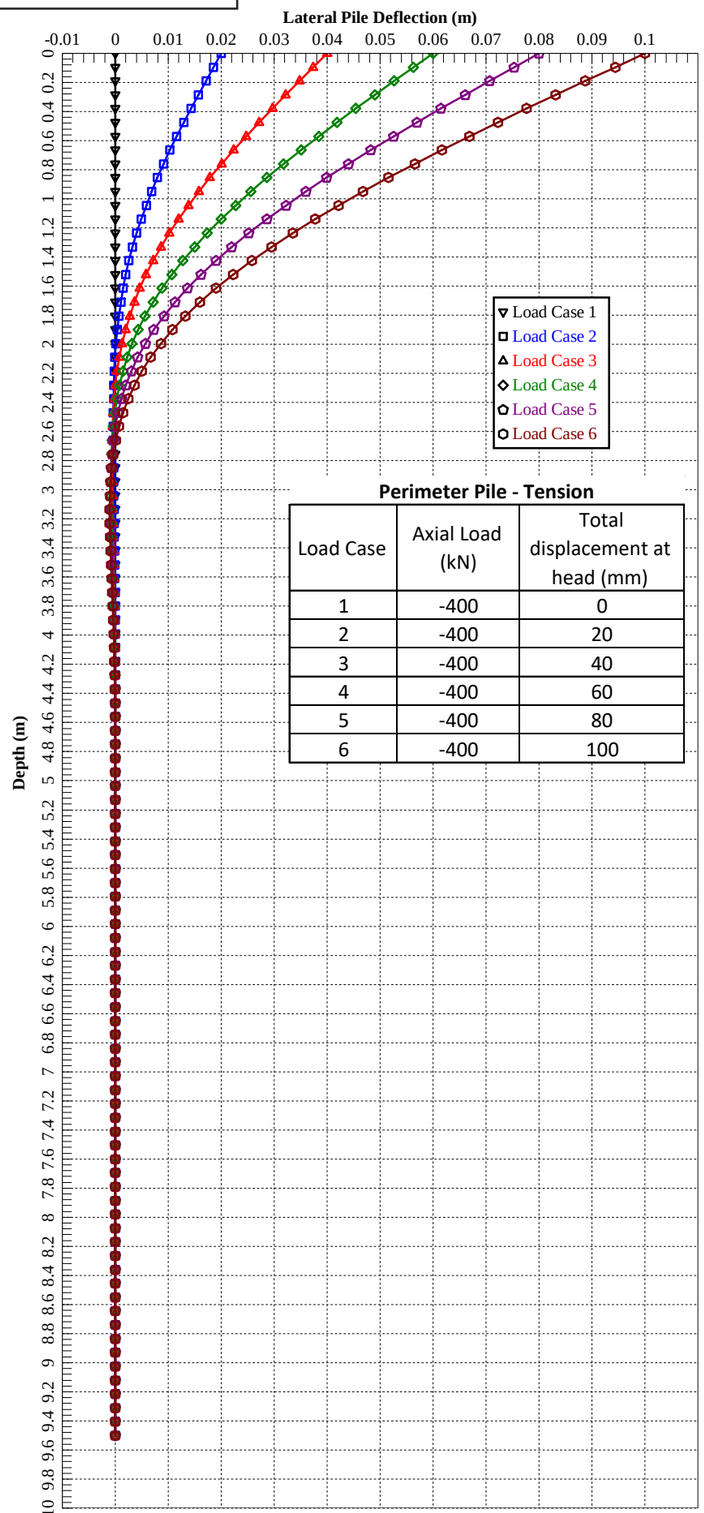


Embassy Theatre DSA, Wellington
T+T Ref: 1020459
Pile Assessment
Rev: 0
Date: 25/07/2022

Pile Properties
D = 0.219m
EI = 4955 kNm²
EA = 876200 kN

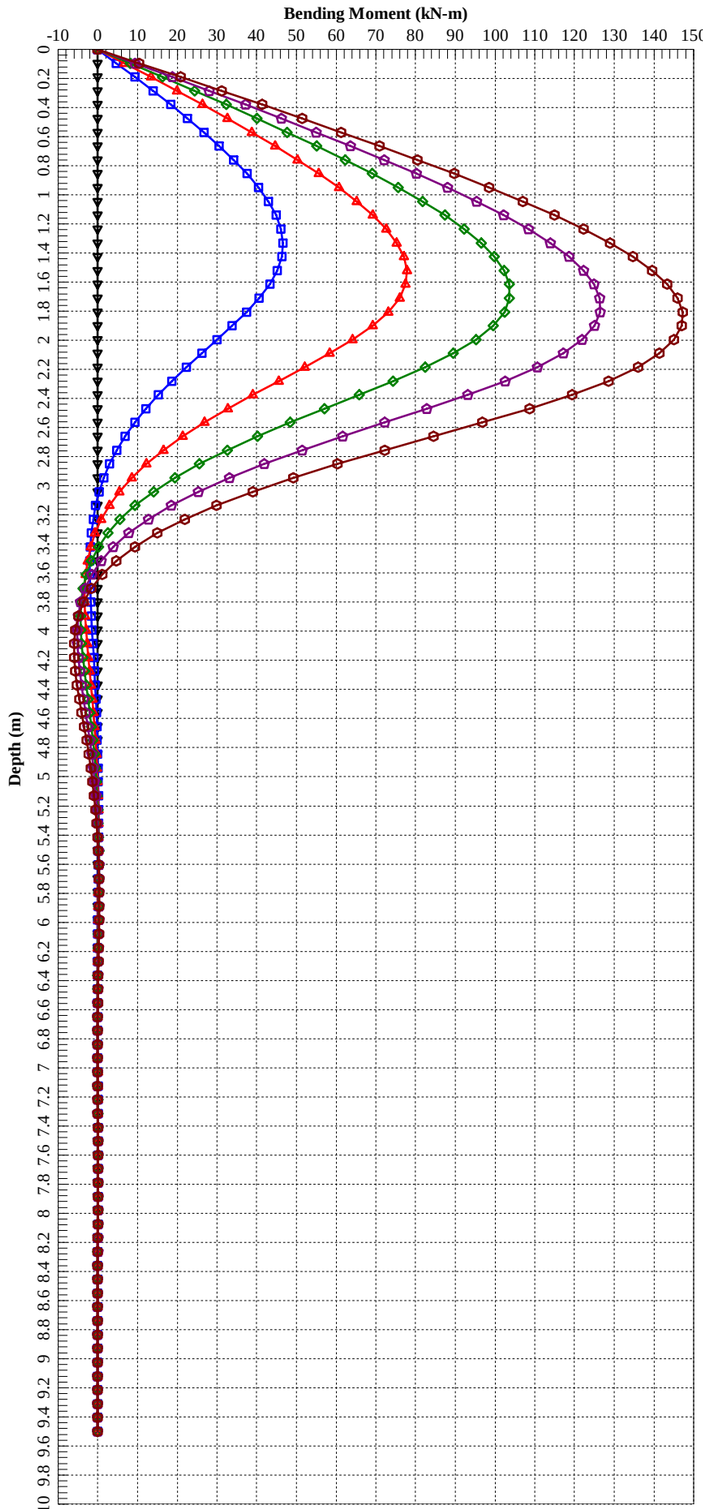
Perimeter Pile - No Liquefaction - Tension

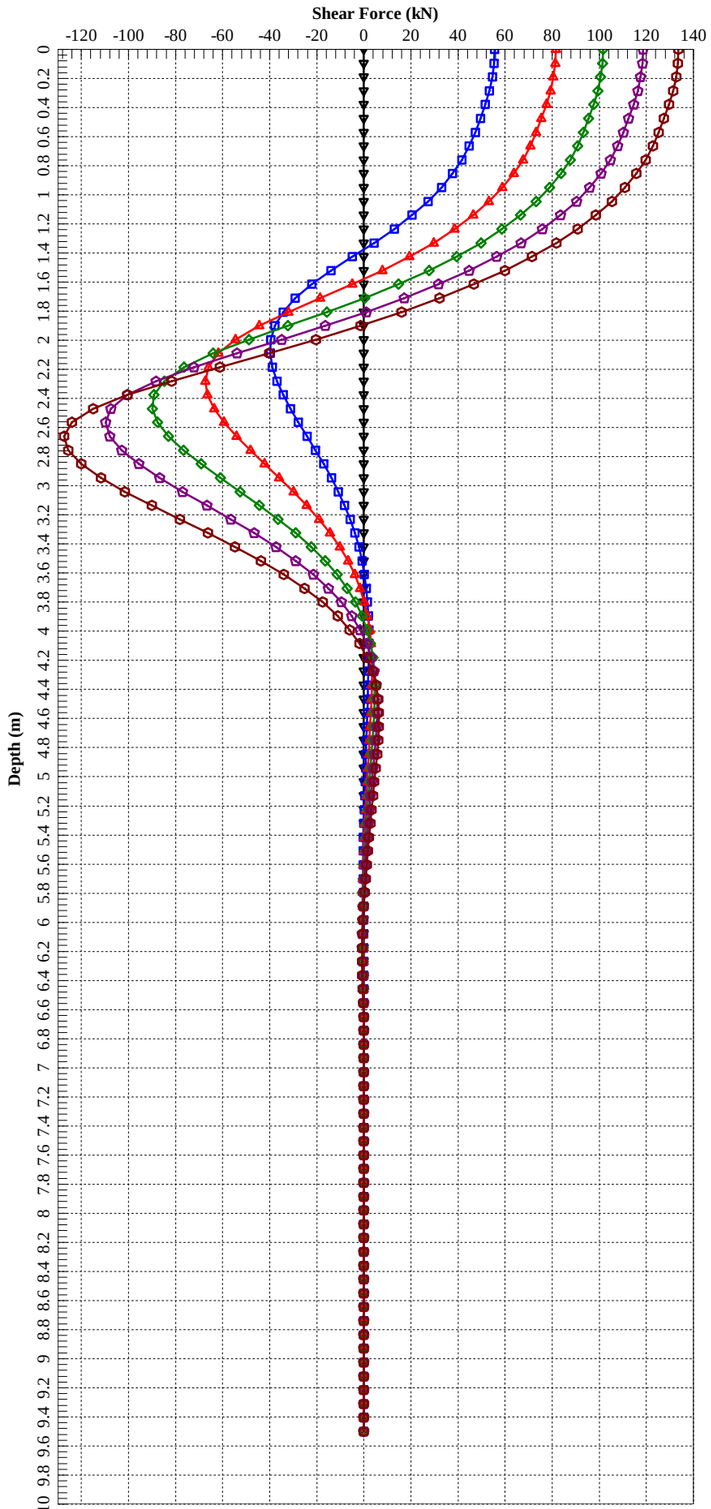
Analysis 2



Perimeter Pile - Tension

Load Case	Axial Load (kN)	Total displacement at head (mm)
1	-400	0
2	-400	20
3	-400	40
4	-400	60
5	-400	80
6	-400	100



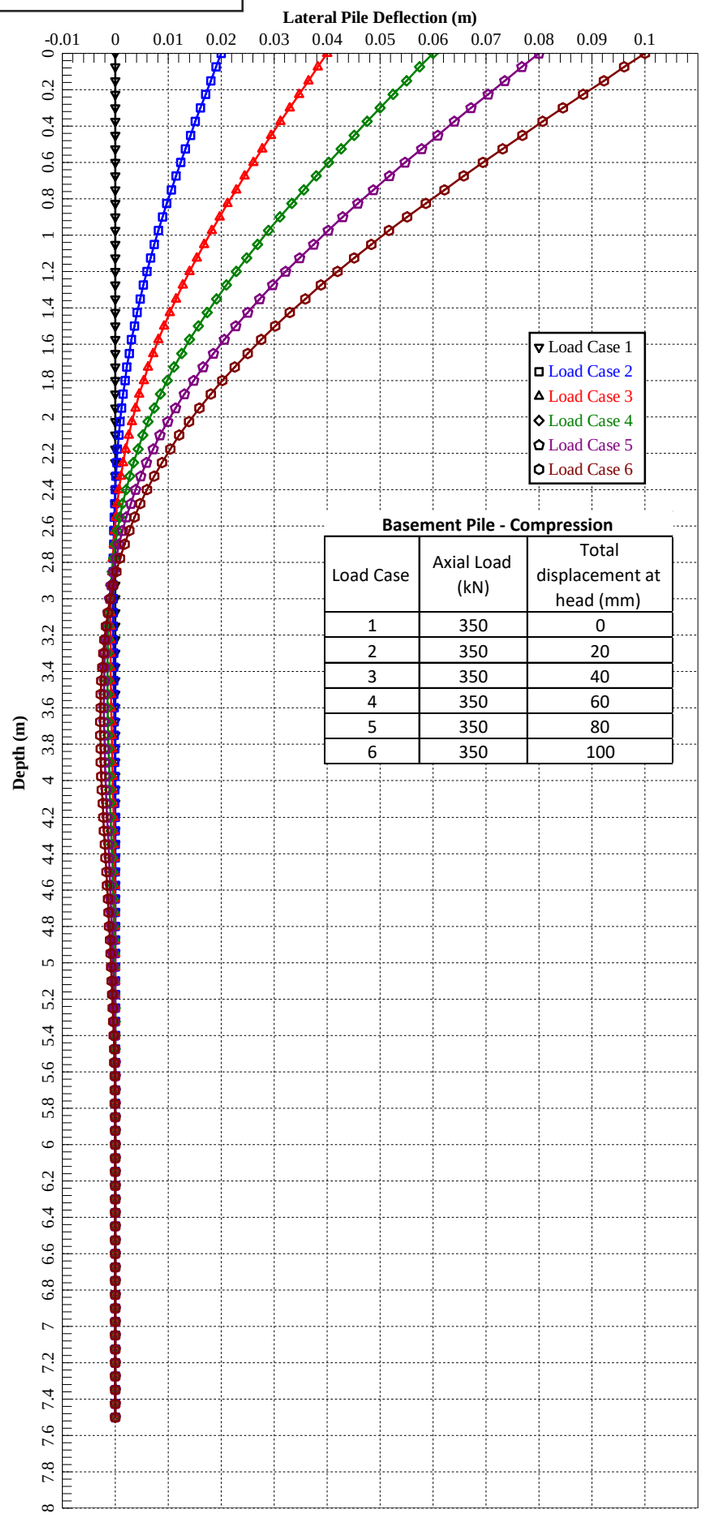


Embassy Theatre DSA, Wellington
T+T Ref: 1020459
Pile Assessment
Rev: 0
Date: 25/07/2022

Pile Properties
D = 0.219m
EI = 6,734 kNm^2
EA = 1,890,660 kN

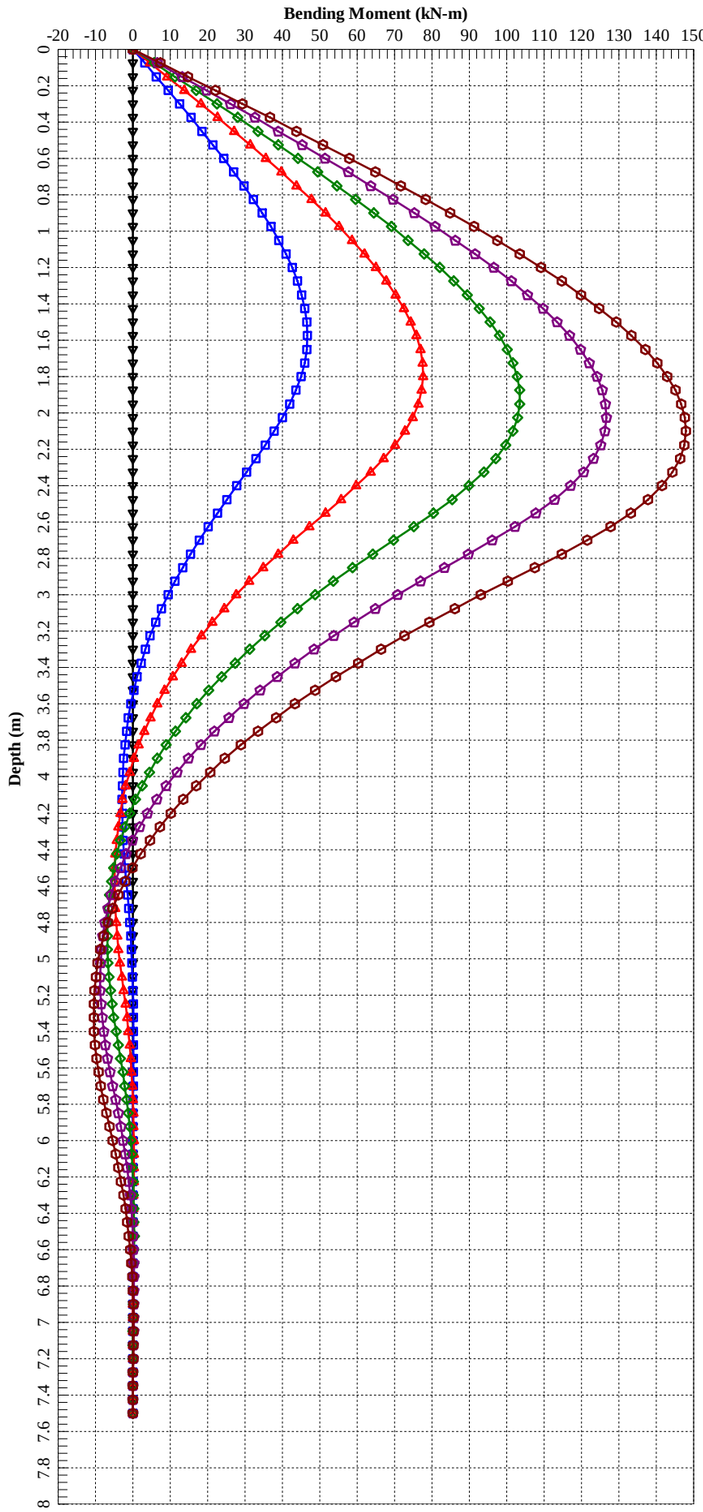
Basement Pile - No Liquefaction - Compression (P37 to P39 only)

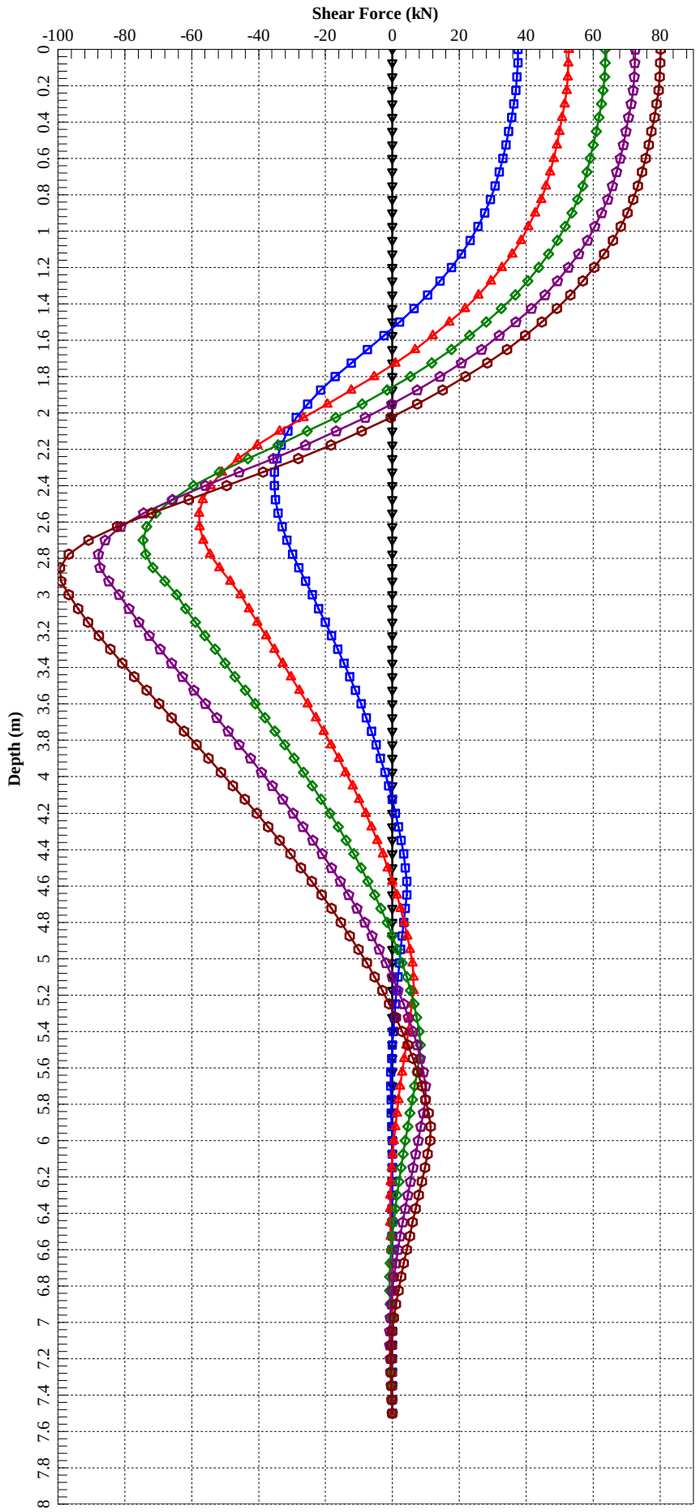
Analysis 3



Basement Pile - Compression

Load Case	Axial Load (kN)	Total displacement at head (mm)
1	350	0
2	350	20
3	350	40
4	350	60
5	350	80
6	350	100



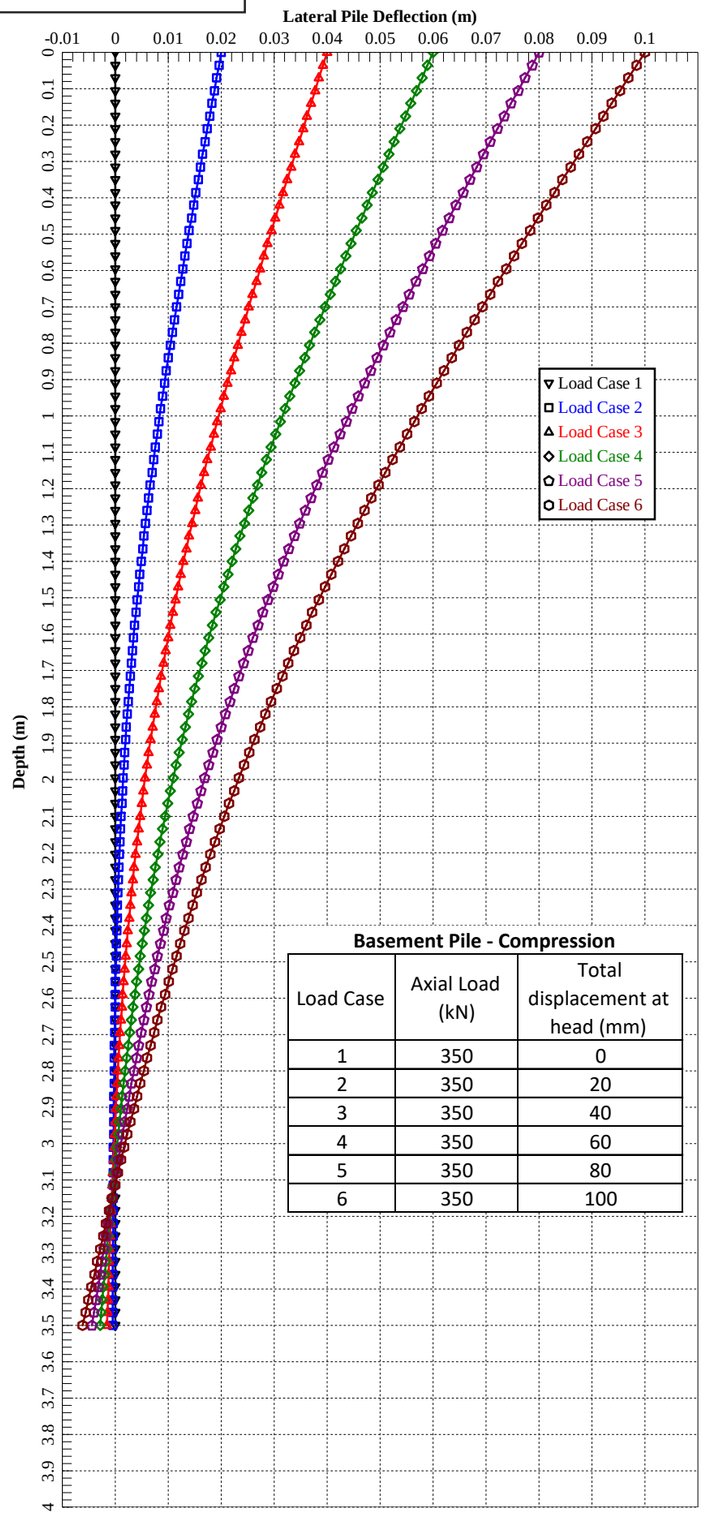


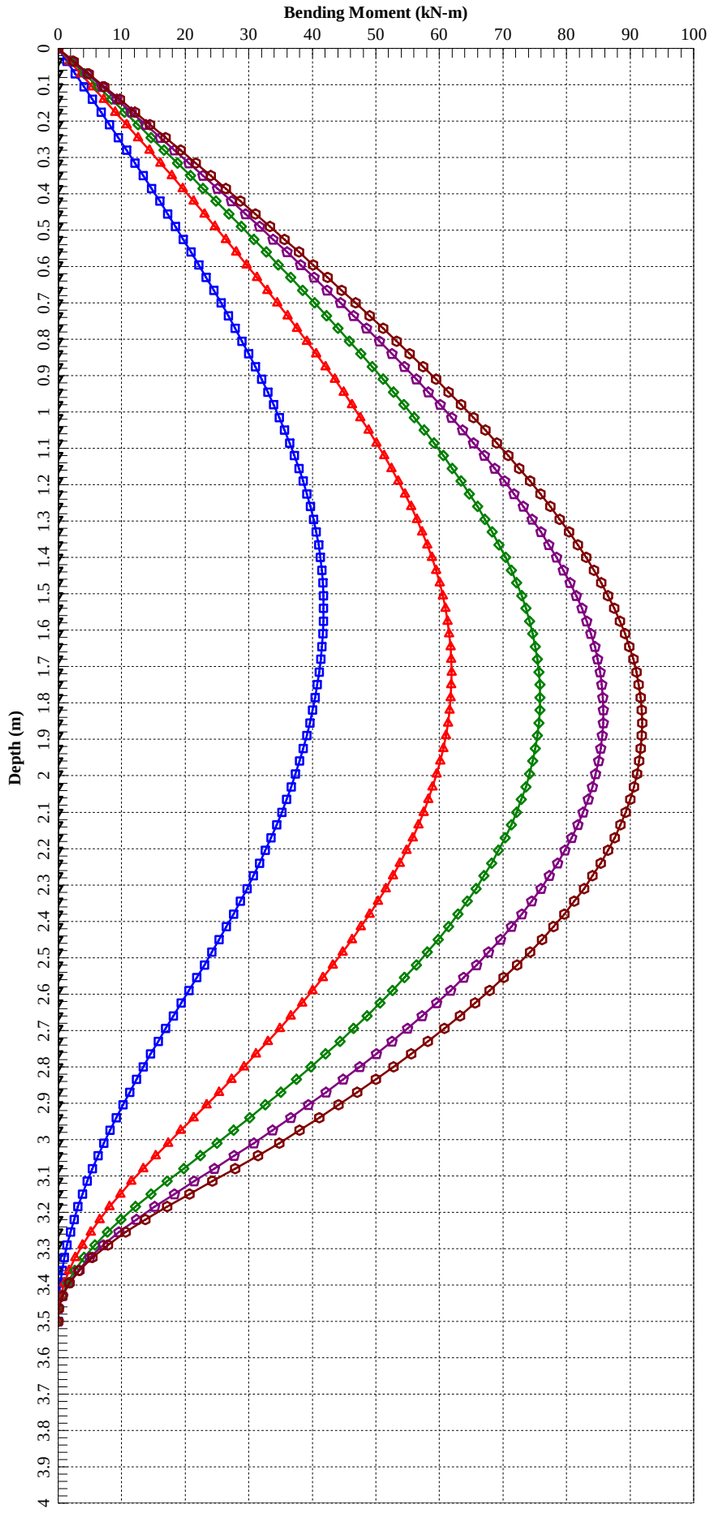
Embassy Theatre DSA, Wellington
T+T Ref: 1020459
Pile Assessment
Rev: 0
Date: 25/07/2022
Page 5/9

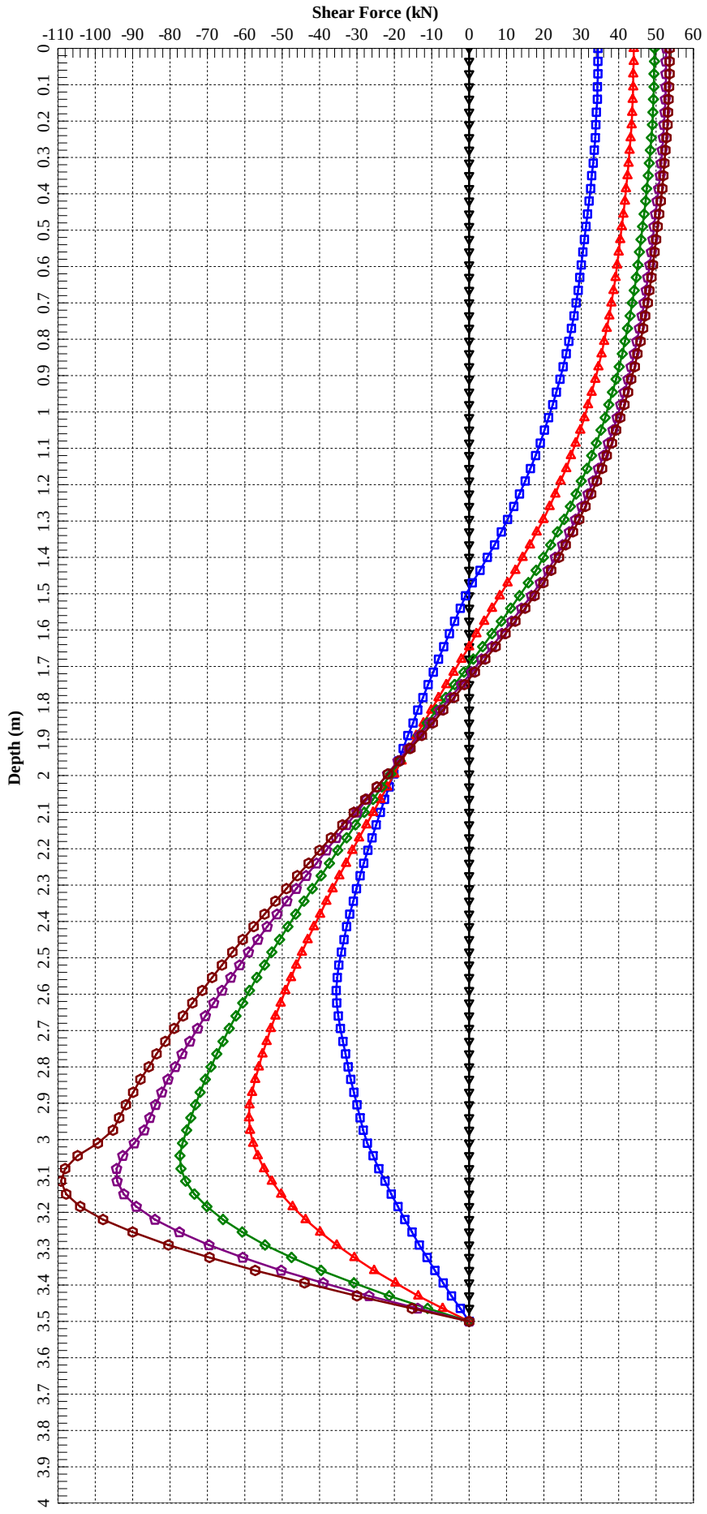
Pile Properties
D = 0.219m
EI = 6,734 kNm^2
EA = 1,890,660 kN

Basement Pile - No Liquefaction - Compression (Remaining screw piles)

Analysis 4







Basement Pile - Compression

Load Case	Axial Load (kN)	Total displacement at head (mm)
1	350	0
2	350	20
3	350	40
4	350	60
5	350	80
6	350	100

Embassy Theatre DSA, Wellington
T+T Ref: 1020459
Pile Assessment
Rev: 0
Date: 25/07/2022

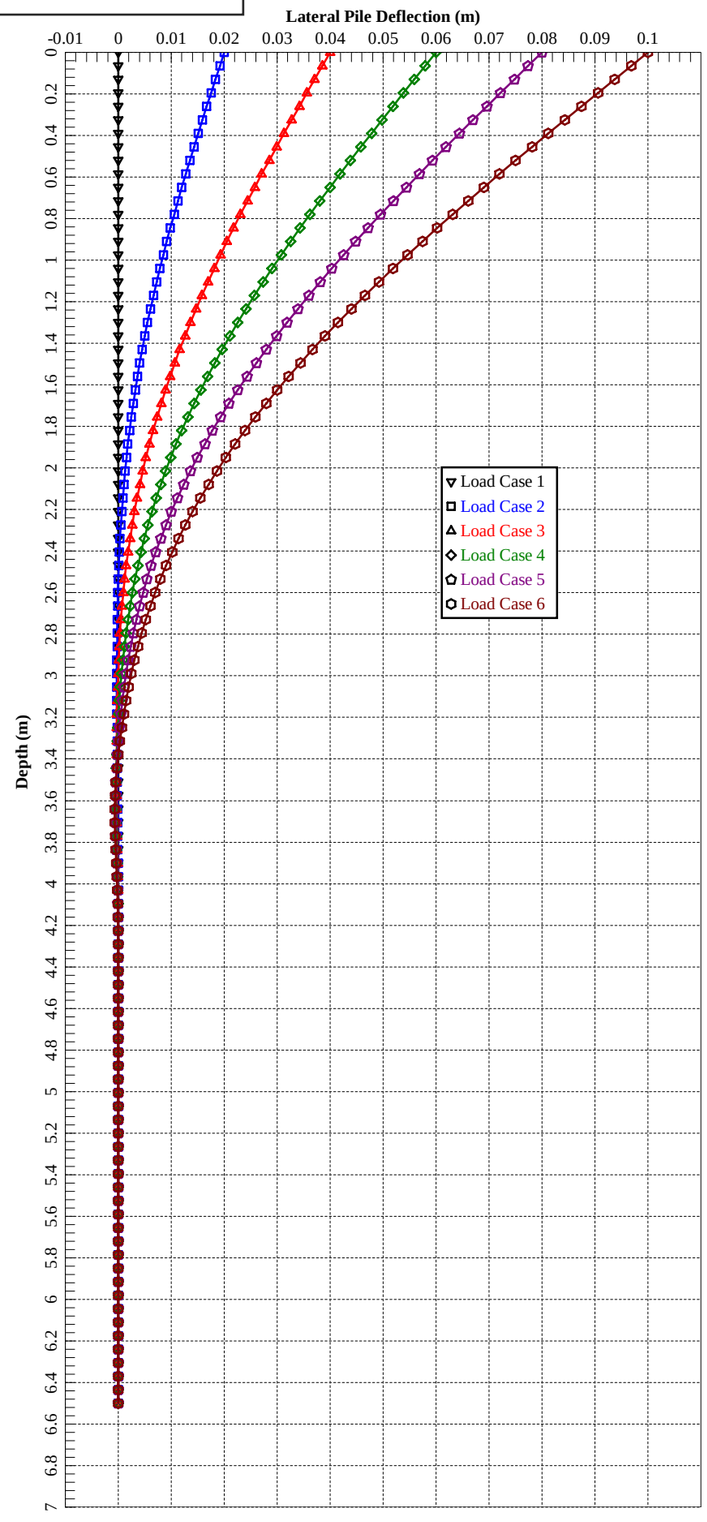
Pile Properties 0 to 3.5m
D = 0.219m
EI = 6,734 kNm²
EA = 1,890,660 kN
Pile Properties >3.5m
D = 0.200m
EI = 1,783 kNm²
EA = 1,107065 kN

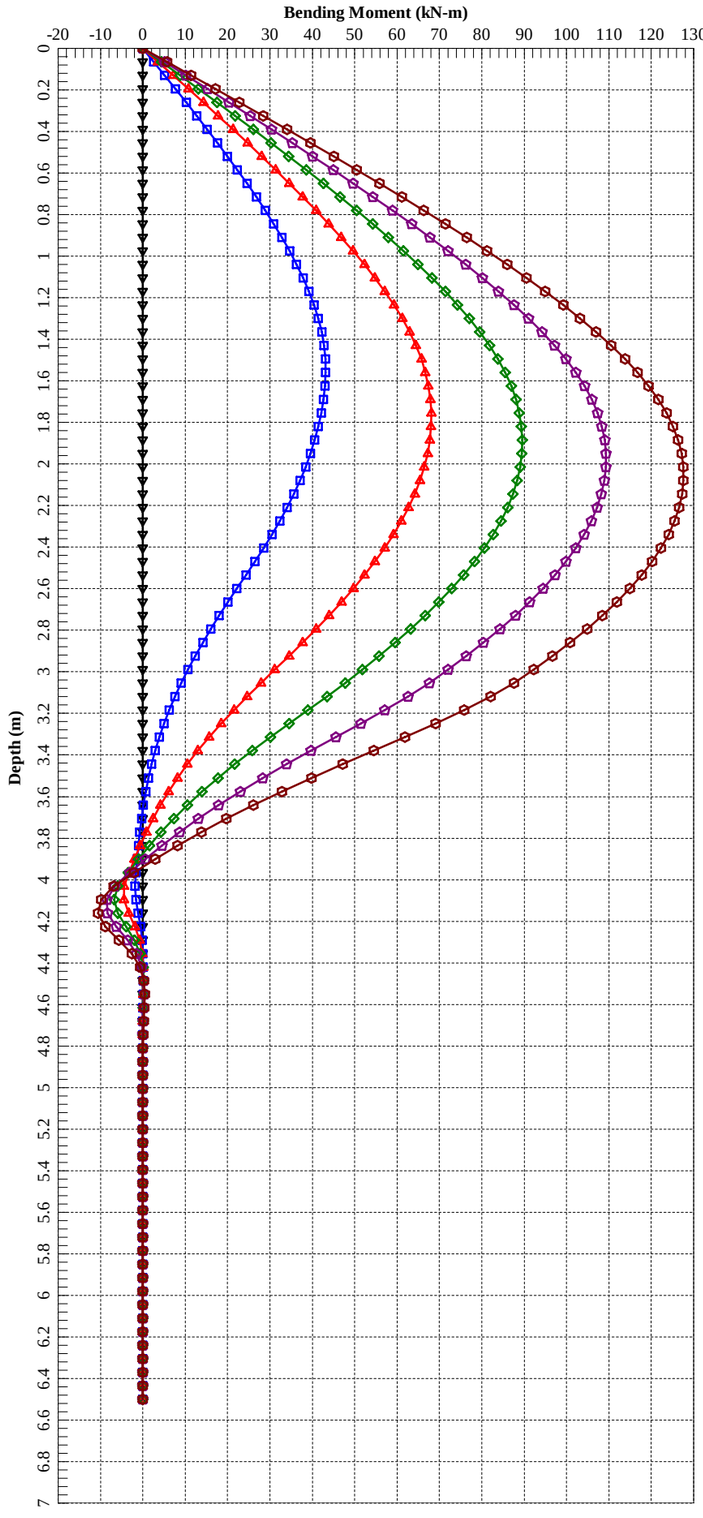
Basement Pile - No Liquefaction - Compression (Remaining screw pile and rock anchor combination)

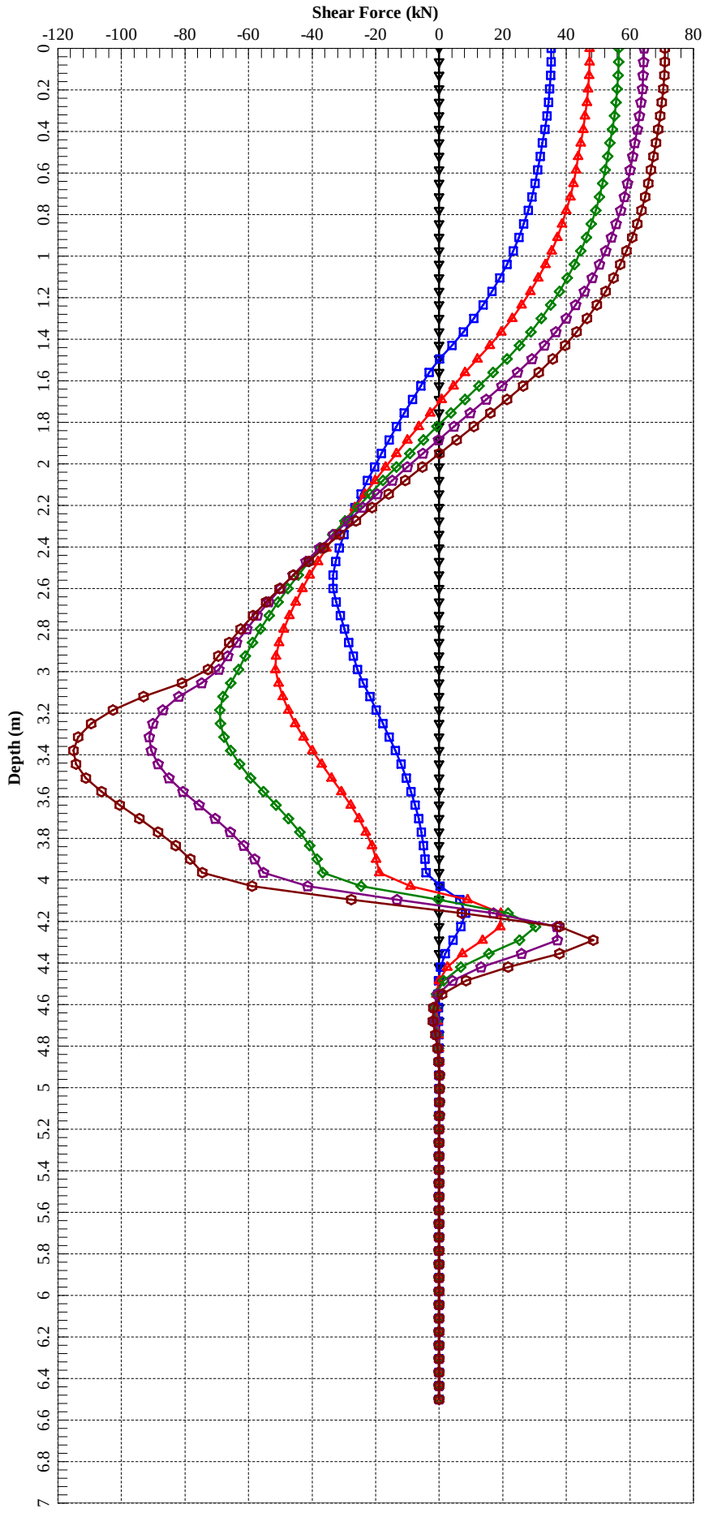
Analysis 5

Screw pile

Rock anchor





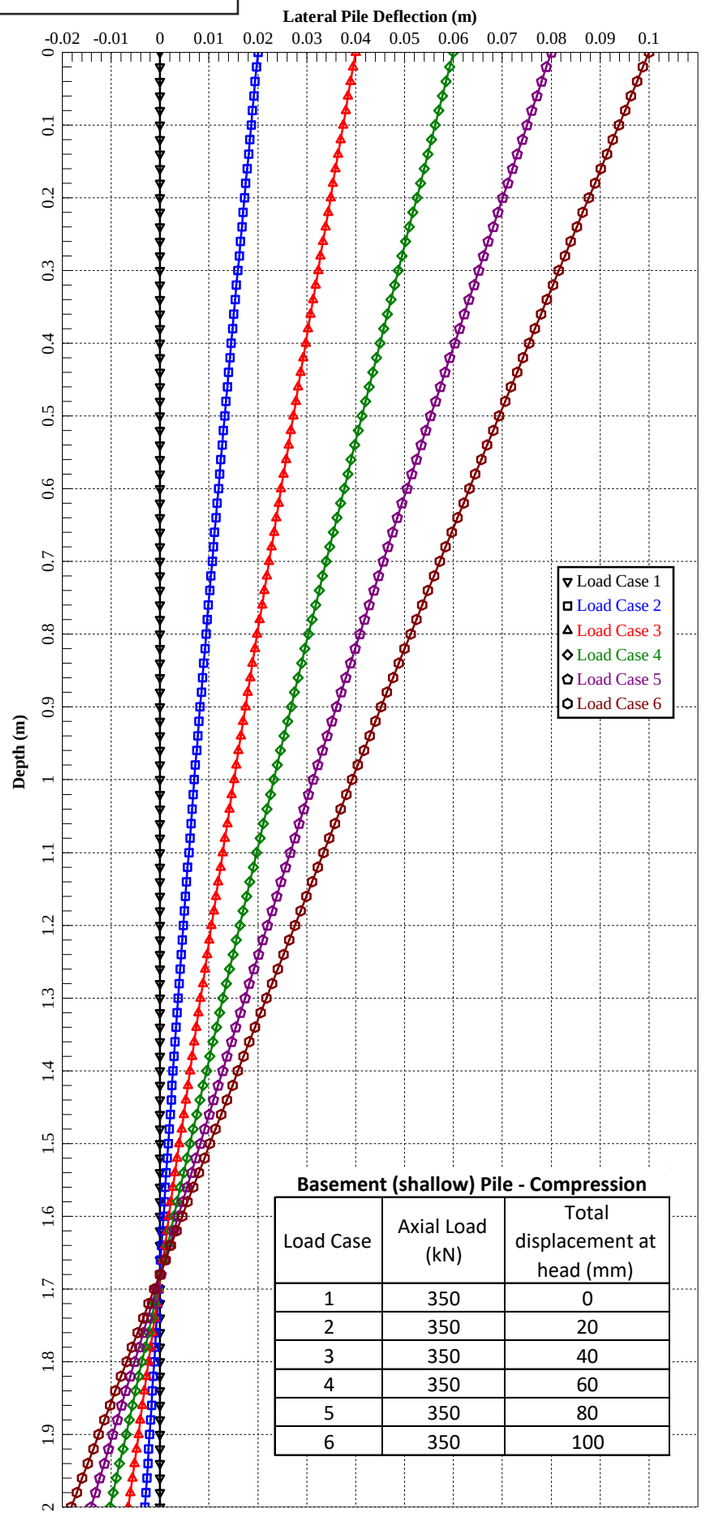


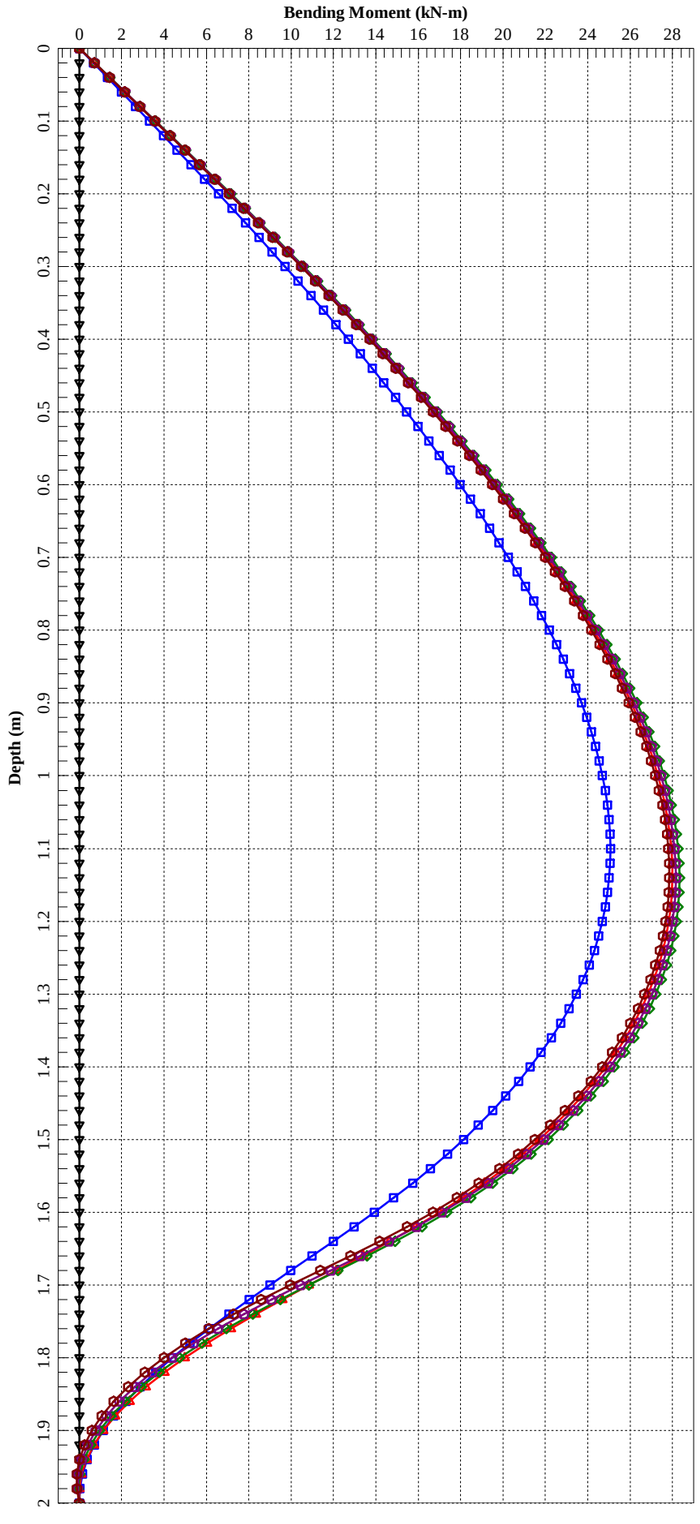
Embassy Theatre DSA, Wellington
T+T Ref: 1020459
Pile Assessment
Rev: 0
Date: 25/07/2022

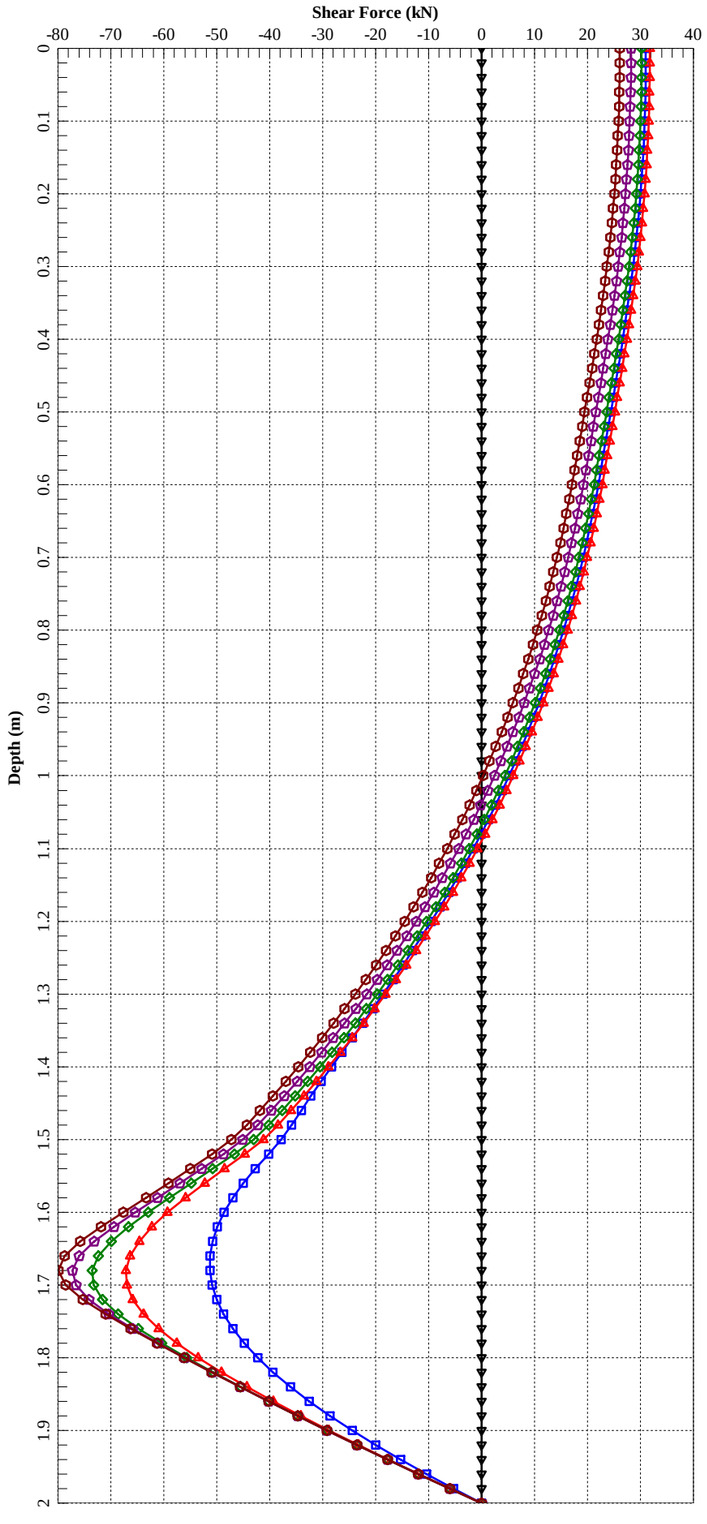
Pile Properties
D = 0.219m
EI = 6,734 kNm^2
EA = 1,890,660 kN

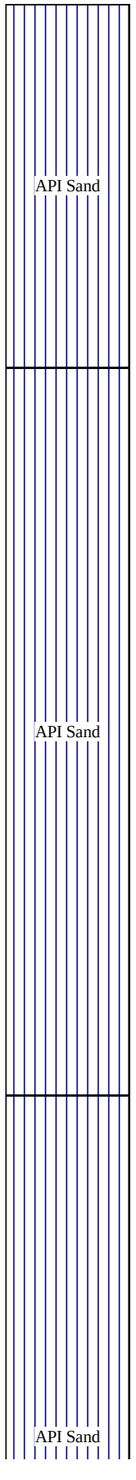
Shallow Basement Pile - No Liquefaction - Compression

Analysis 6









Basement (shallow) Pile - Compression

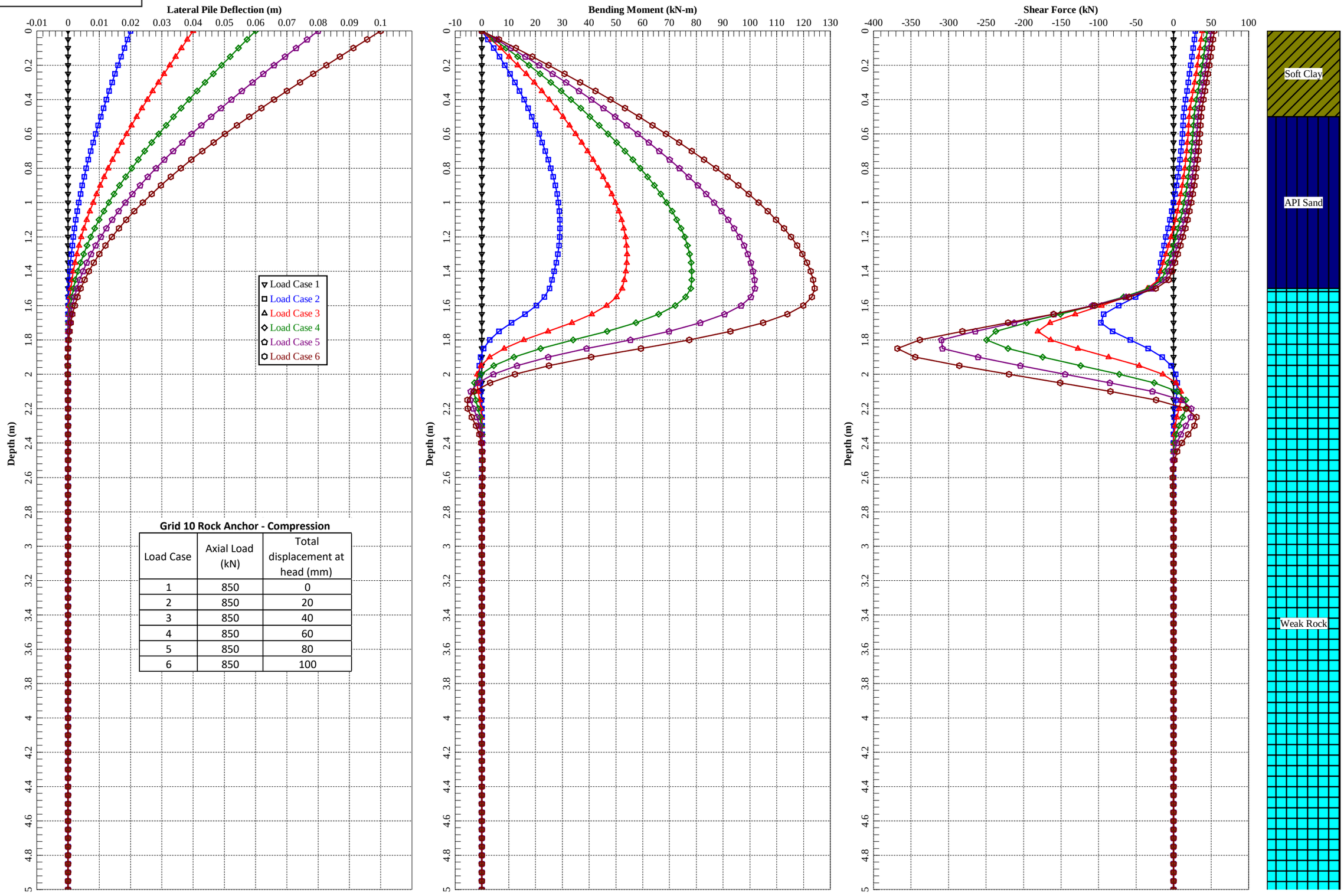
Load Case	Axial Load (kN)	Total displacement at head (mm)
1	350	0
2	350	20
3	350	40
4	350	60
5	350	80
6	350	100

Embassy Theatre DSA, Wellington
T+T Ref: 1020459
Pile Assessment
Rev: 0
Date: 25/07/2022
Page 8/9

Pile Properties
D = 0.2m
EI = 1,783 kNm^2
EA = 1,107,065 kN

Rock Anchors (Grid 10) - No Liquefaction - Compression

Analysis 7



Grid 10 Rock Anchor - Compression

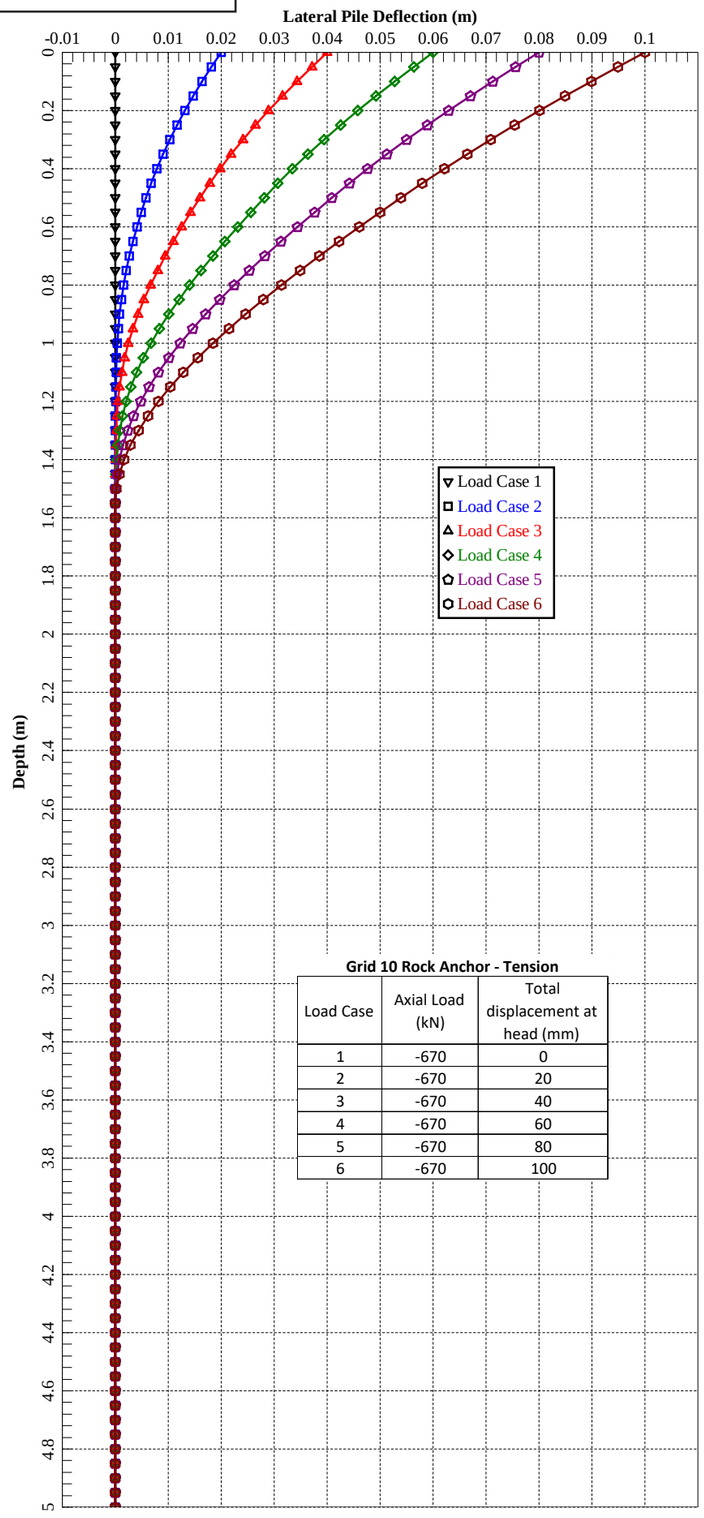
Load Case	Axial Load (kN)	Total displacement at head (mm)
1	850	0
2	850	20
3	850	40
4	850	60
5	850	80
6	850	100

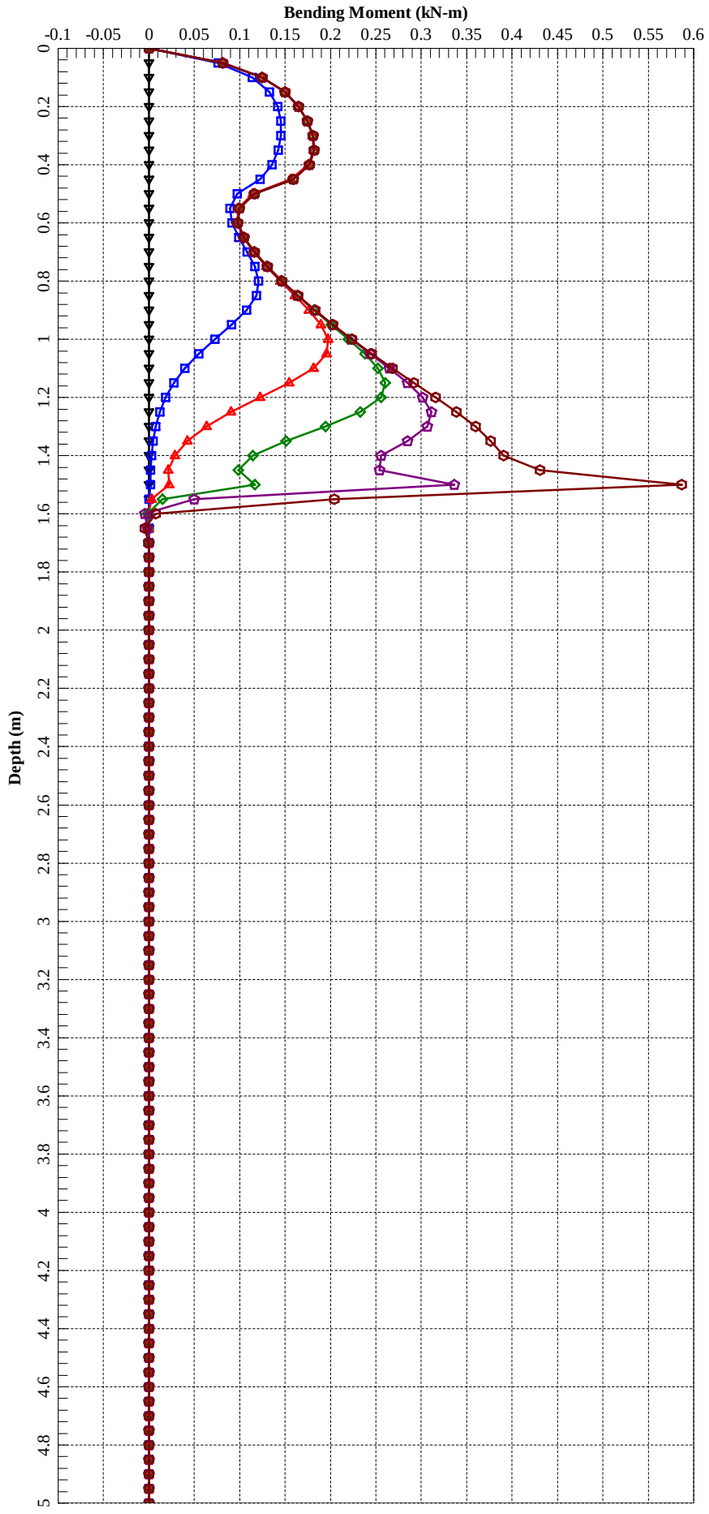
Embassy Theatre DSA, Wellington
T+T Ref: 1020459
Pile Assessment
Rev: 0
Date: 25/07/2022
Page 9/9

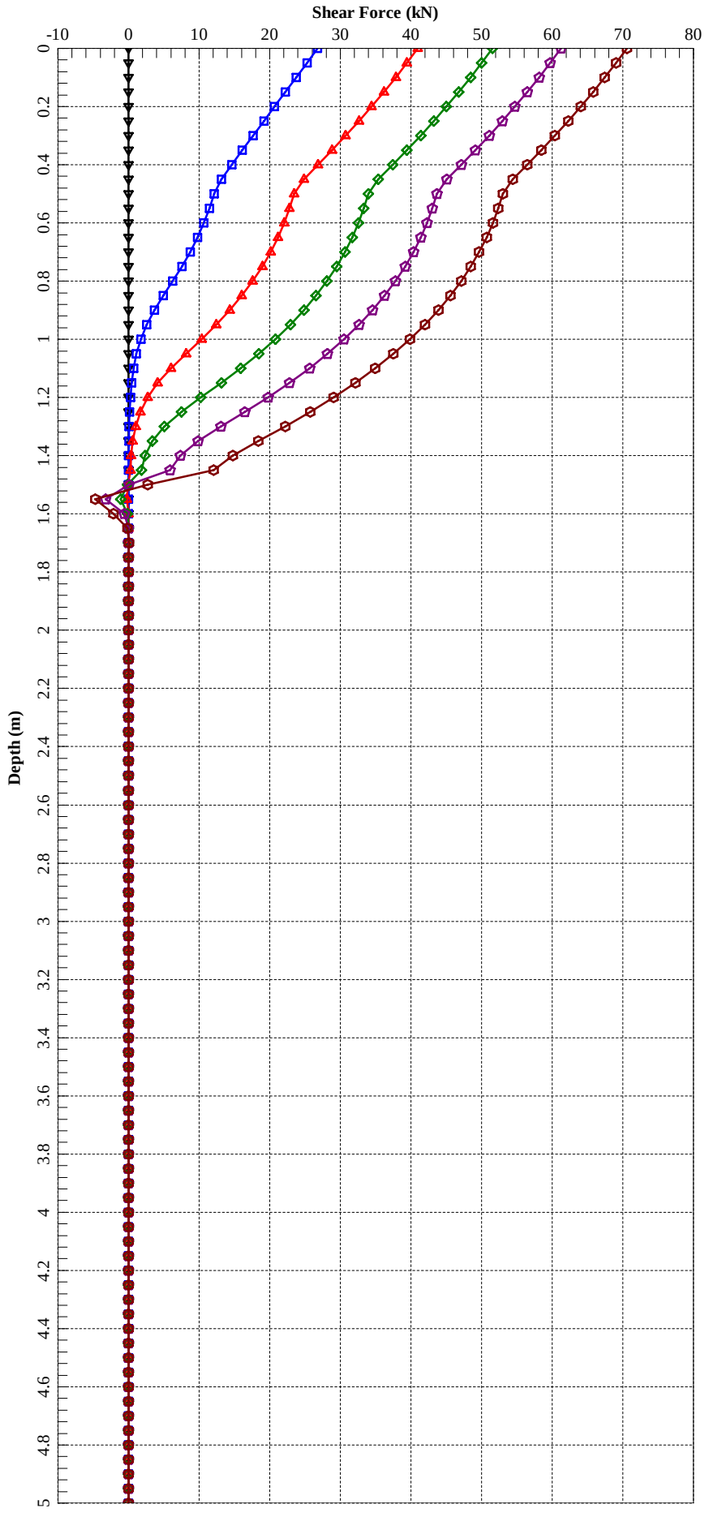
Pile Properties
D = 0.2m
EI = 3 kNm^2
EA = 92,737 kN

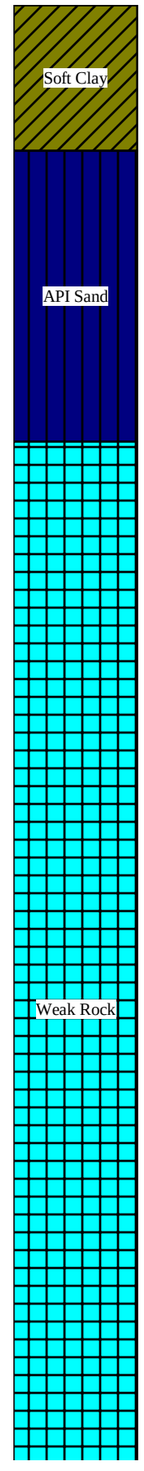
Rock Anchors (Grid 10) - No Liquefaction - Tension

Analysis 8









Grid 10 Rock Anchor - Tension

Load Case	Axial Load (kN)	Total displacement at head (mm)
1	-670	0
2	-670	20
3	-670	40
4	-670	60
5	-670	80
6	-670	100

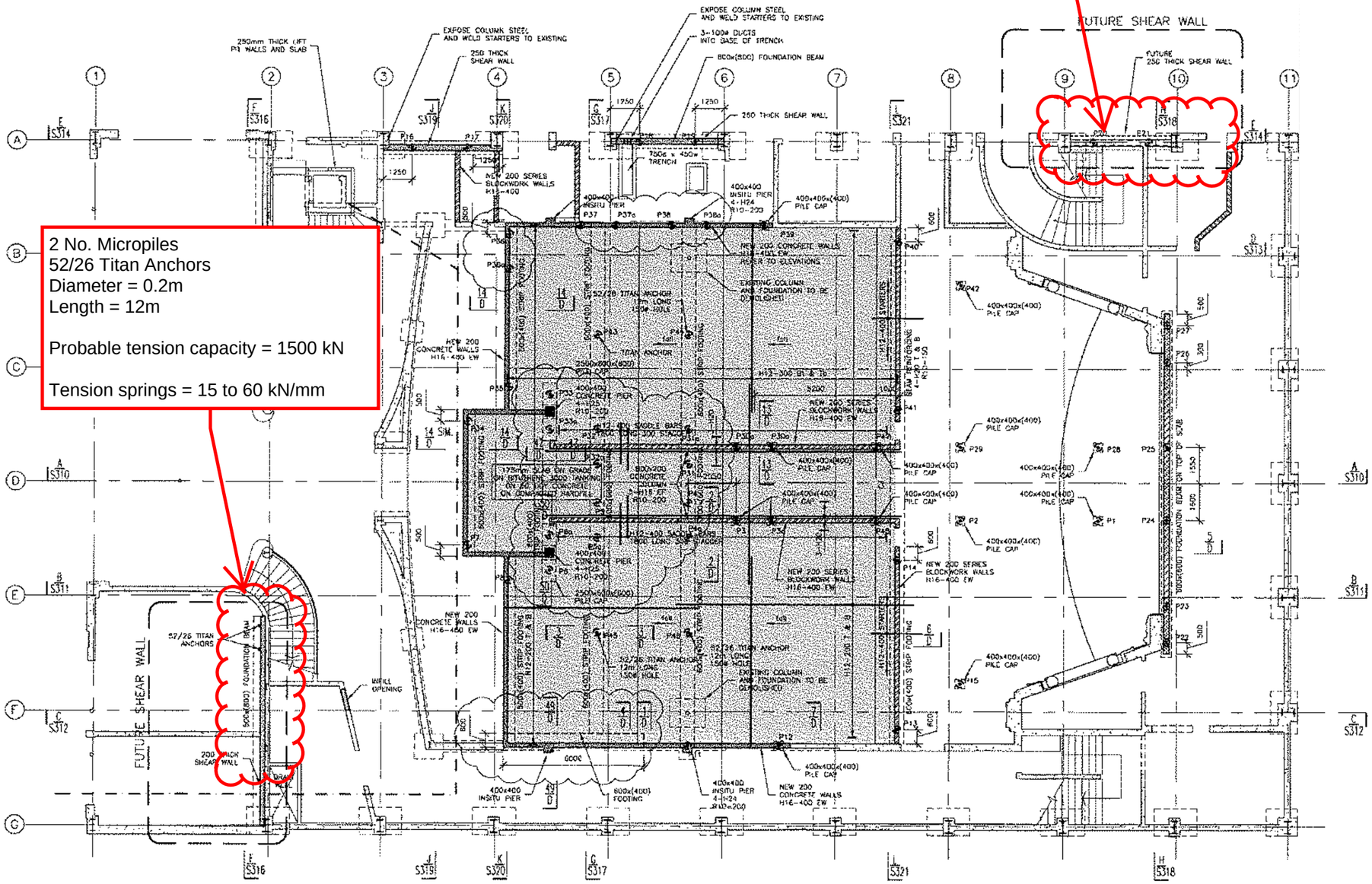
Embassy Theatre DSA, Wellington
T+T Ref: 1020459

Micropile Assessment
Rev: 0
Date: 19/07/2022
Drawn: EMPE
Checked: BHR
Page 1/3

2 No. Micropiles
52/26 Titan Anchors
Diameter = 0.2m
Length = 12m

Probable tension capacity = 1500 kN

Tension springs = 15 to 60 kN/mm



2 No. Micropiles
52/26 Titan Anchors
Diameter = 0.2m
Length = 12m

Probable tension capacity = 1500 kN

Tension springs = 15 to 60 kN/mm

G:\3864\Dtc\Dwgs\3864S300.dwg, 23-07-2003 1:34:32 p., HP LaserJet 4MV.pc3

THEATRE EMBASSY TRUST

EMBASSY REDEVELOPMENT 2003

LEVEL 0 PILE LAYOUT & LOWER SLAB DETAILS

REVISIONS		
3	REVISED	22-07-2003
2	FOR CONSTRUCTION	20-06-2003
1	FOR CONSENT	3-06-2003
0	PILING TENDER	22-05-2003

Scales	1:100
A3 Scales	1:200
Designed	Adam Thornton
Drawn	Martin WILLIAMS
CAD Reference	3864S300

Job Number 3864
Drawing Number S300
Rev 3

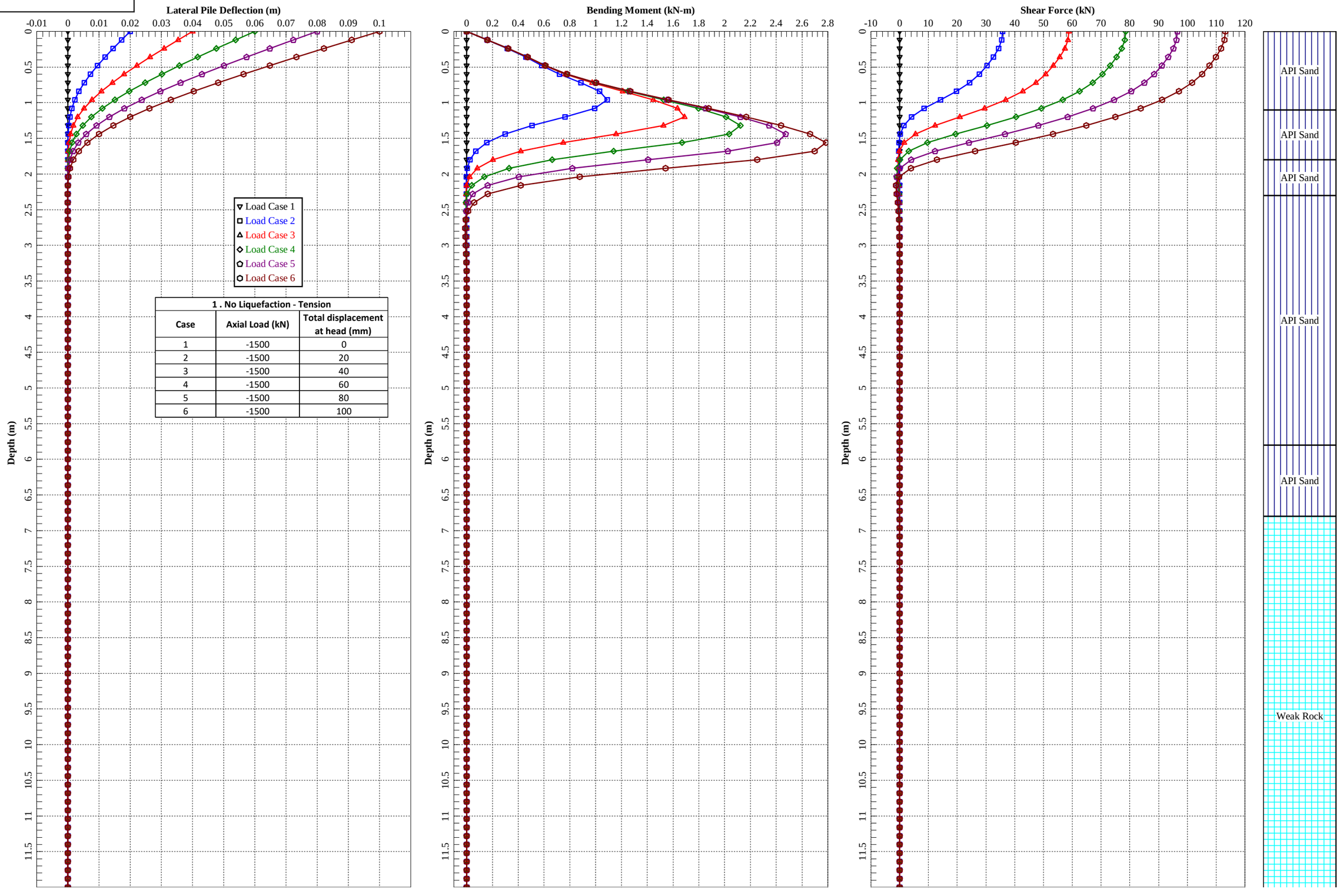
ISO 9001

Dunning Thornton Consultants Ltd
Consulting Engineers
Phone (644) 385-9019
Fax (644) 385-0312
Box 27-153 Wellington, N.Z.
E-mail dtcwgtn@dunningthornton.co.nz

Embassy Theatre DSA, Wellington
T+T Ref: 1020459
Micropile Assessment
Rev: 0
Date: 19/07/2022
Page 2/3

Micropile Properties
D = 0.2m
E = 7350 MPa
A = 0.0314 m^2
I = 5.71E-06 m^4

Micropile - No liquefaction - Tension Load.



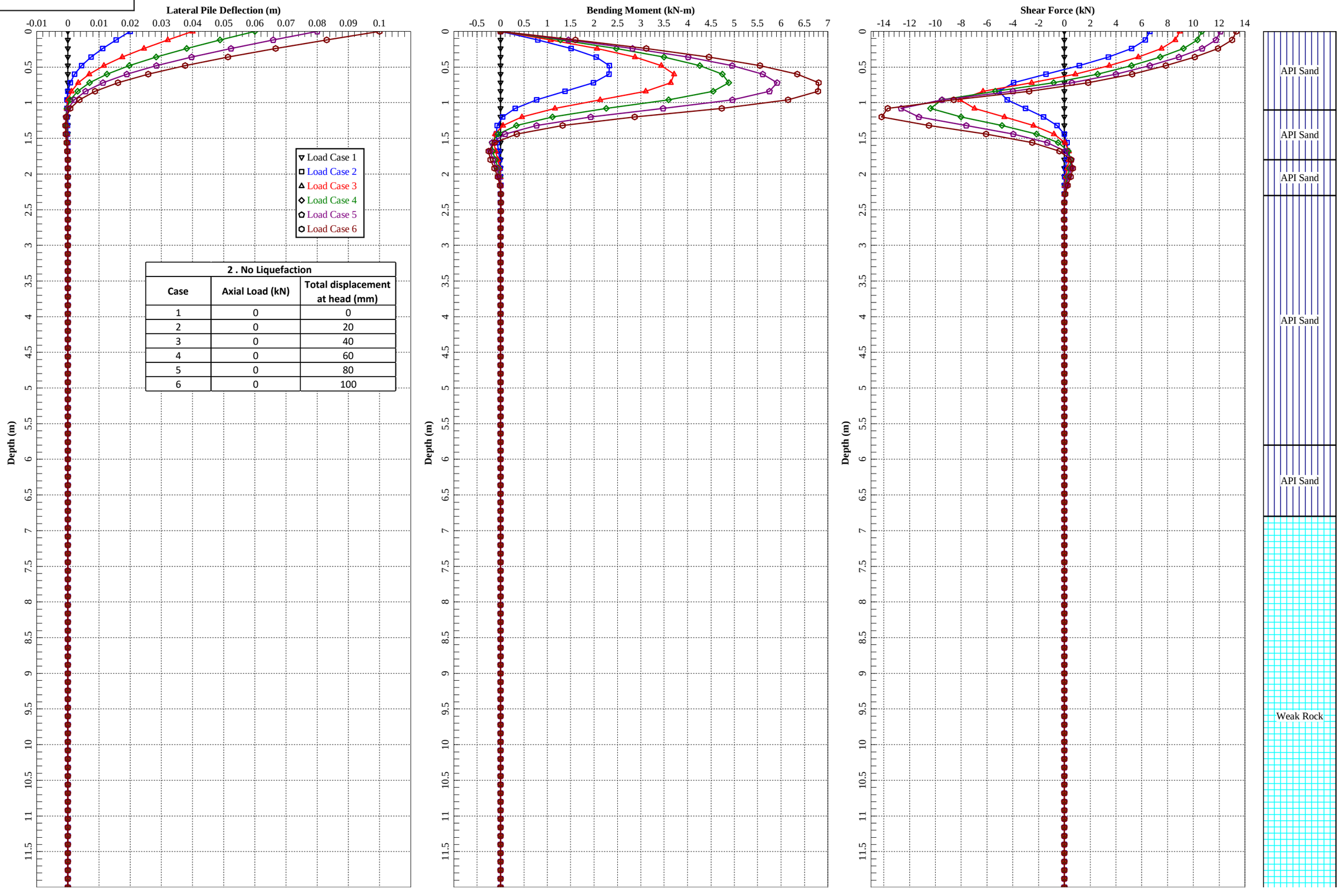
1 . No Liquefaction - Tension

Case	Axial Load (kN)	Total displacement at head (mm)
1	-1500	0
2	-1500	20
3	-1500	40
4	-1500	60
5	-1500	80
6	-1500	100

Embassy Theatre DSA, Wellington
T+T Ref: 1020459
Micropile Assessment
Rev: 0
Date: 19/07/2022
Page 3/3

Micropile Properties
D = 0.2m
E = 7350 MPa
A = 0.0314 m^2
I = 5.71E-06 m^4

Micropile - No liquefaction - No applied load.



2 . No Liquefaction

Case	Axial Load (kN)	Total displacement at head (mm)
1	0	0
2	0	20
3	0	40
4	0	60
5	0	80
6	0	100

Appendix C Concrete Scan Report

Independent Concrete Reinforcing Verification Report

Project: Embassy Theatre Strengthening

Engineer	***Dunning Thornton Consultants Ltd***
	Attn: Rubie Inch
Client:	***Wellington City Council***
	Attn: Karla Rivers
Date:	***29/02/2024***

Job Number: 234158

Professional Procedure

This report was written by:

Pablo Mitchell

Email: pablo@csiscan.nz
Phone: +64 (0) 27 443 3558
Public Key SHA1 Digest:96 80 33 FD 30 9E CA 46 85 C6 BF 7C 86 01 FB FD 2F CD 73 AD

This report was reviewed by:

Daniel Traeger

Email: daniel@csiscan.nz
Phone: +64 (0) 27 2305843
Public Key SHA1 Digest: 99 01 03 72 8D 10 16 39 C0 7B 81 CC 46 E2 7B 5E 0B DA 11 E3

Table of Contents

Table of Figures

1 Project Scope

Concrete Structure Investigations Limited (CSI) were engaged by Wellington City Council to undertake scanning services at the Embassy Theatre, Wellington.

Our brief was supplied by Rubie Inch of Dunning Thornton for scanning various structural elements to determine the reinforcement layout. The locations investigated were provided by Rubie in a marked-up plan. All structural elements supplied in brief were able to be scanned, alternative scan locations were chosen by CSI where required due to the site conditions. The information/data taken on site has been analysed, peer-reviewed and provided in the below report.

2 Building Plans and Scanning Locations

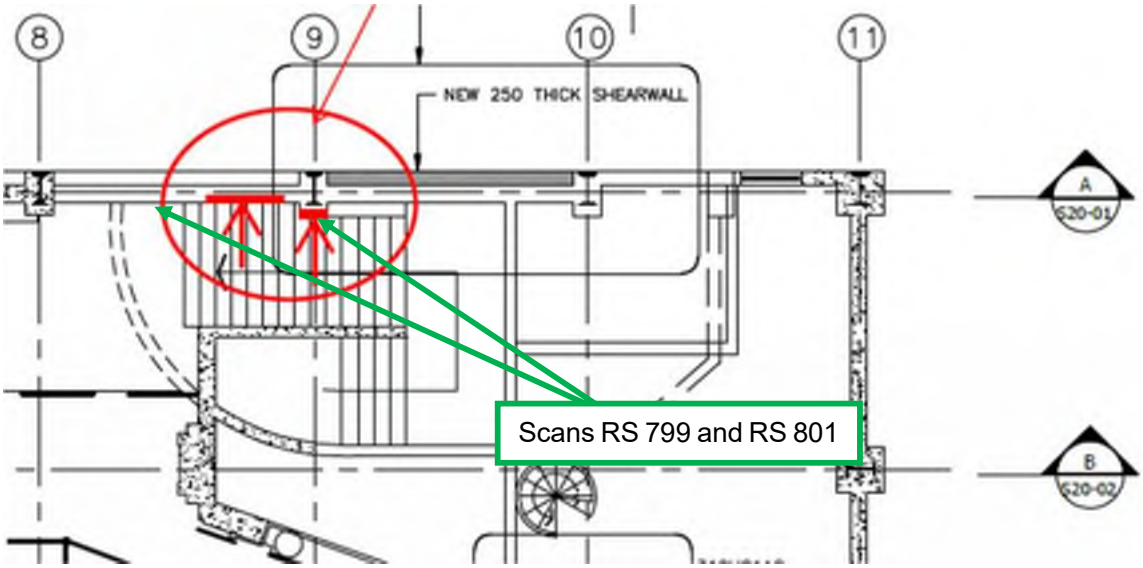


Figure 1: Scans Grid A 8/9 Level 1

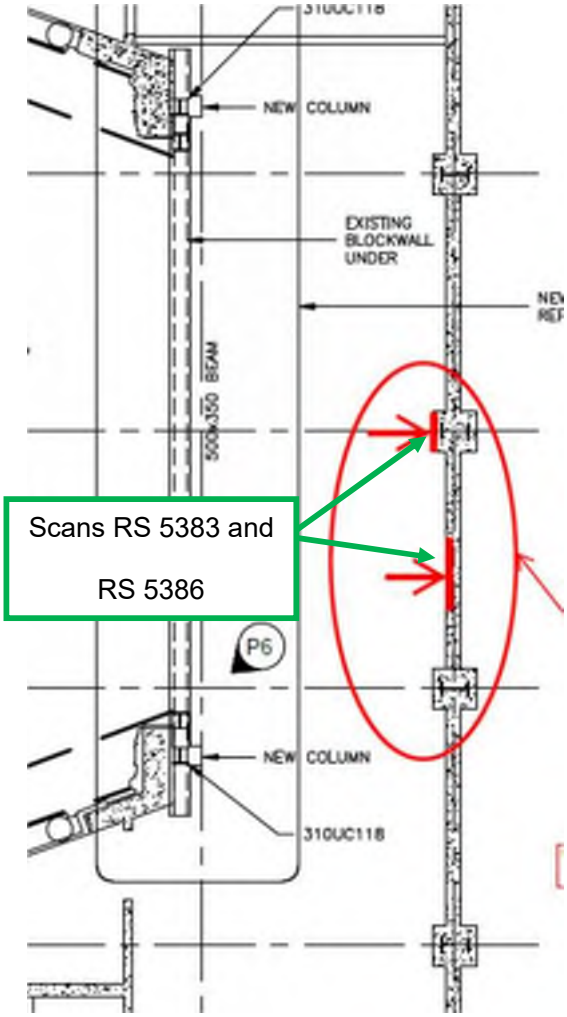


Figure 2: Scans Grid 11E/F Level 1

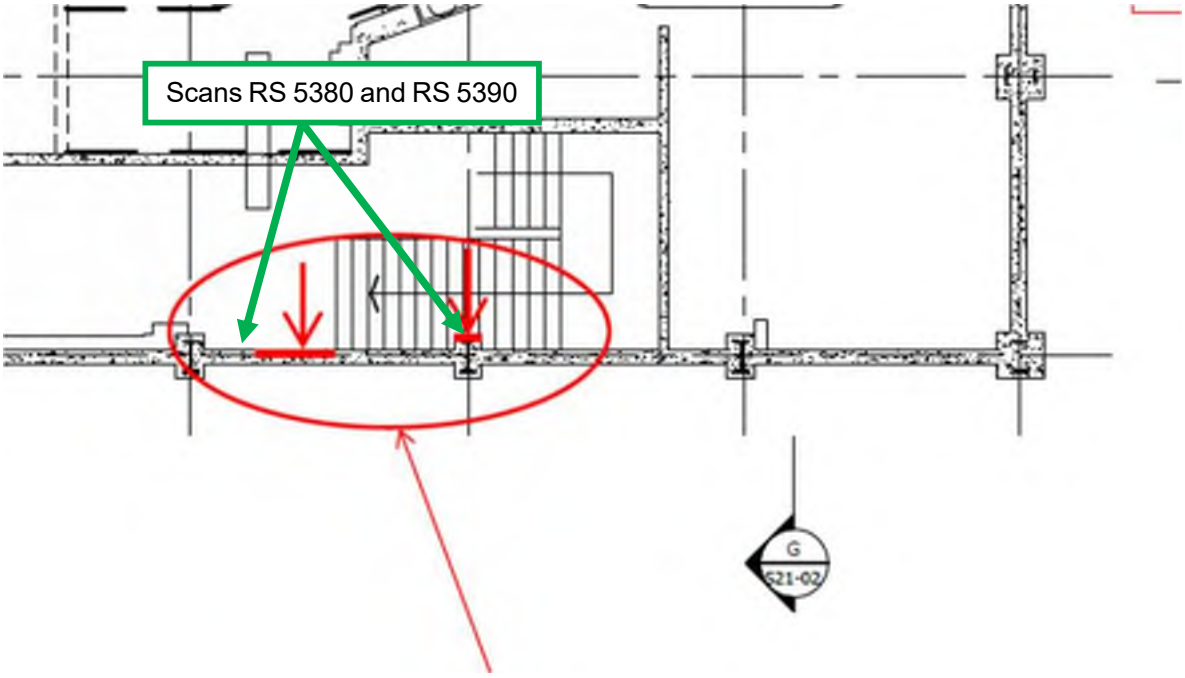


Figure 3: Scans Grid G 8/9 Level 1

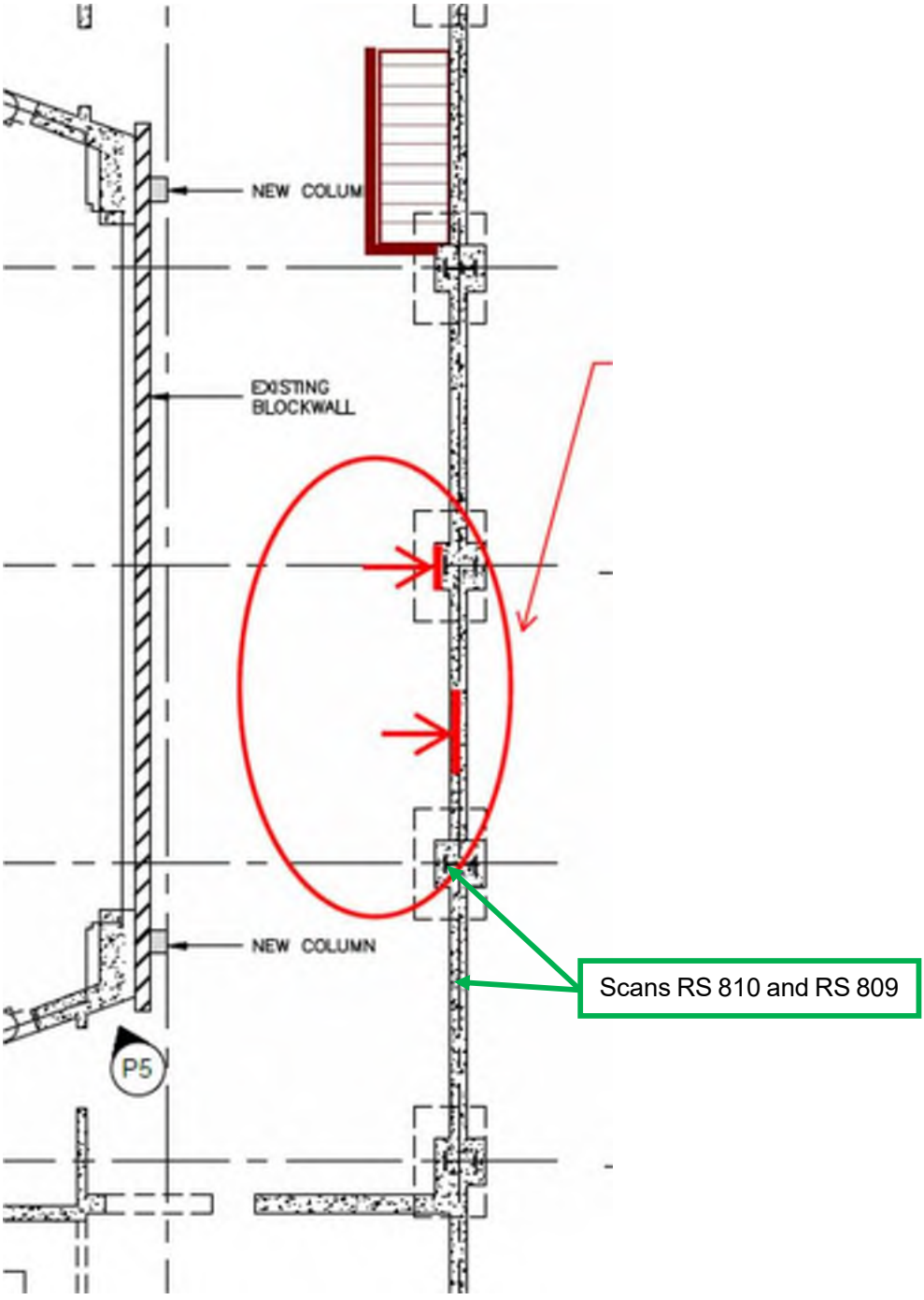


Figure 4: Scans Grid 11 E/F Basement Level 0

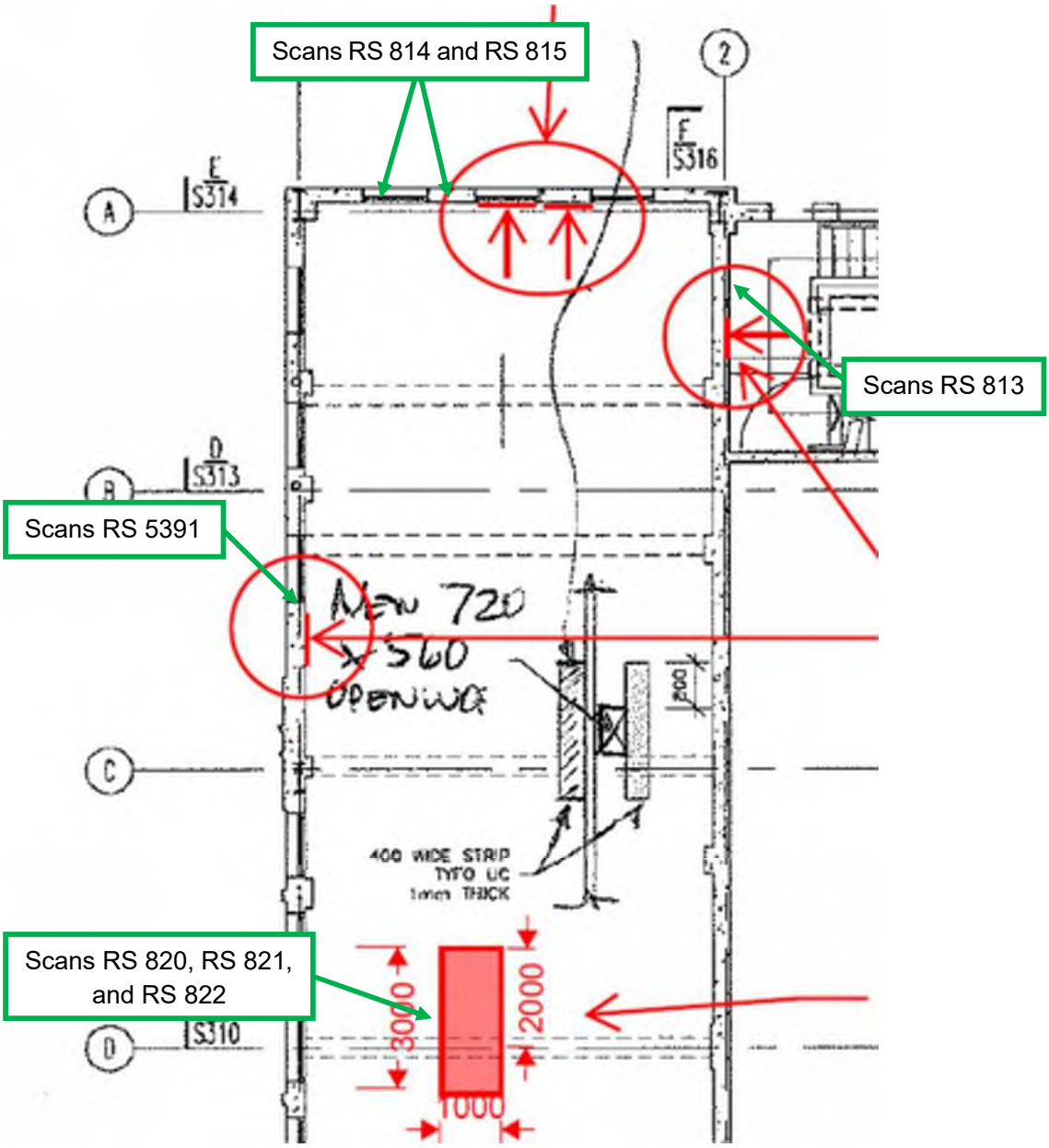


Figure 5: Scans Locations Level 2

3 Scans

3.1 Level 0

3.1.1 Scans grid 11 E/F

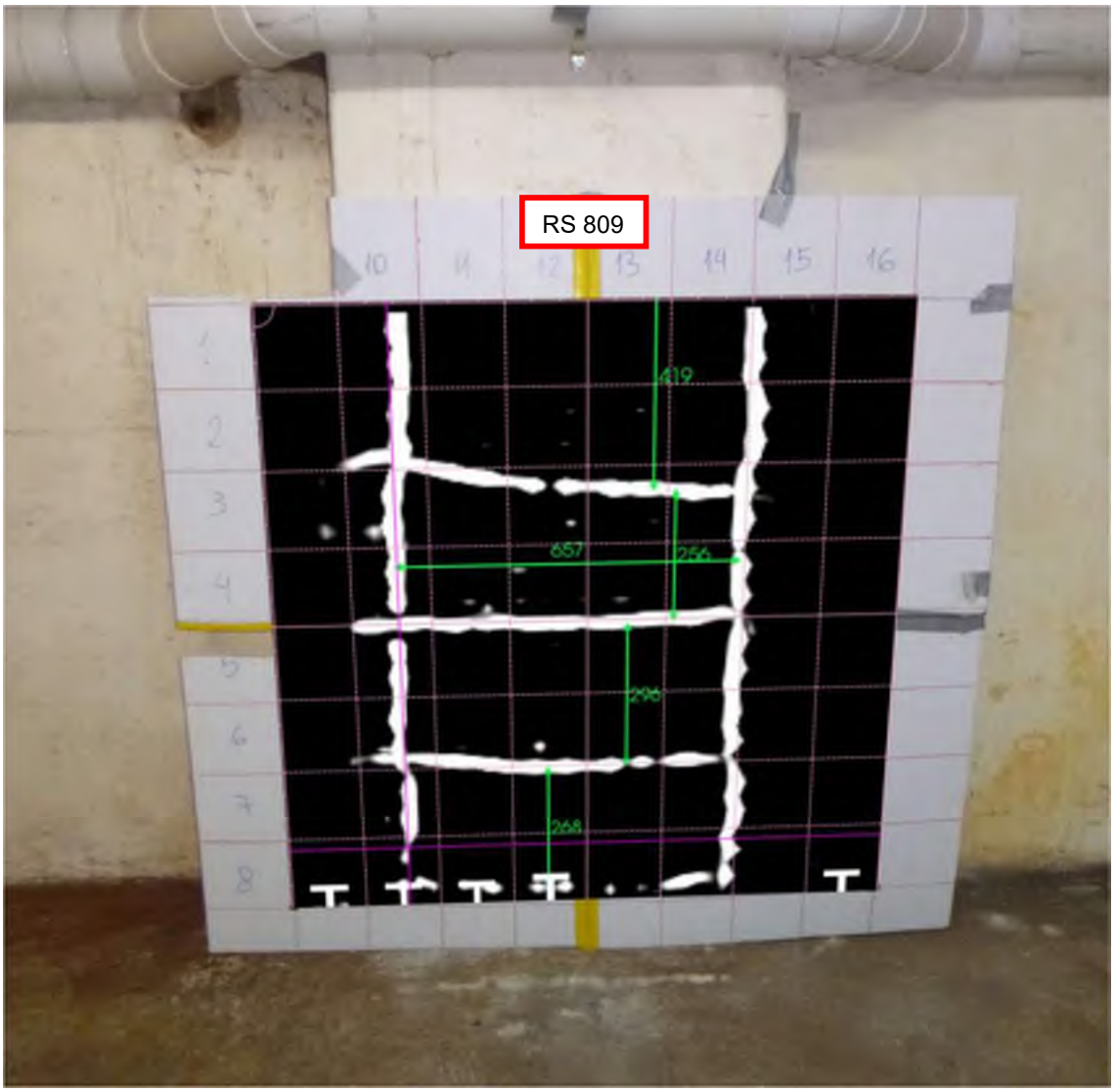


Figure 6: Column Scan Grid 11-E

Figure 7: Wall Scan Grid 11- E/F

3.1.1.1 RS809

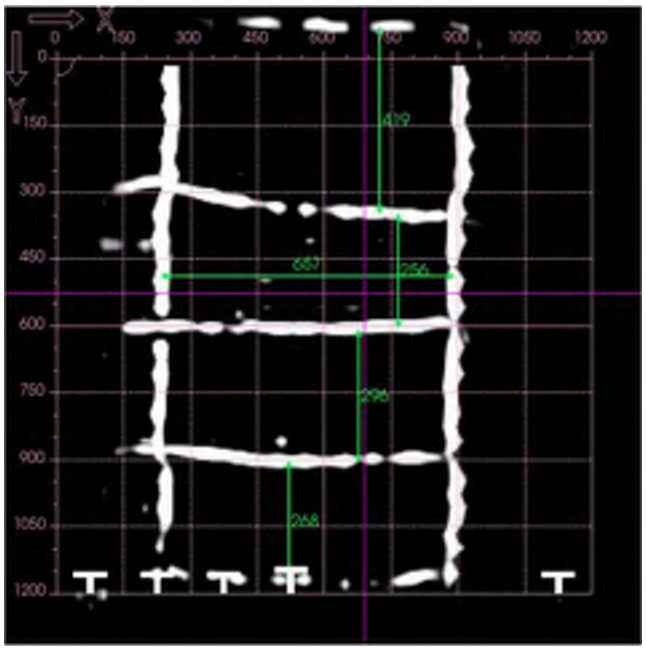


Figure 8: Scan RS809

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	7.0
Filename	RS_054210001_000809.hscan	Timestamp	-
Comment:	Column Scan taken on Grid 11/E. The top scan margin is approx. 1300mm above the floor, and x=150 aligns with the left edge of the column Reinforcing bars: • min. concrete cover approx. 64mm • vertical bars c/c approx. 650mm • stirrups c/c approx. from 260mm to 420mm Additional reinforcing: • horizontal bars are visible at a cover of approx. 350-400mm • c/c is approx. 630mm • possibility of wall reinforcing developing through the column o further investigations are required for confirmation Steel column (as per drawings): • cannot be confirmed from the available data o further investigations are required to confirm the presence / absence		

3.1.1.2 Adjacent column Grid 11/F

Vertical Reinforcement approx. 25mm	
Horizontal (Stirrups) Approx. 12mm	

Figure 9: Column Grid 11/F

3.1.1.3 RS810

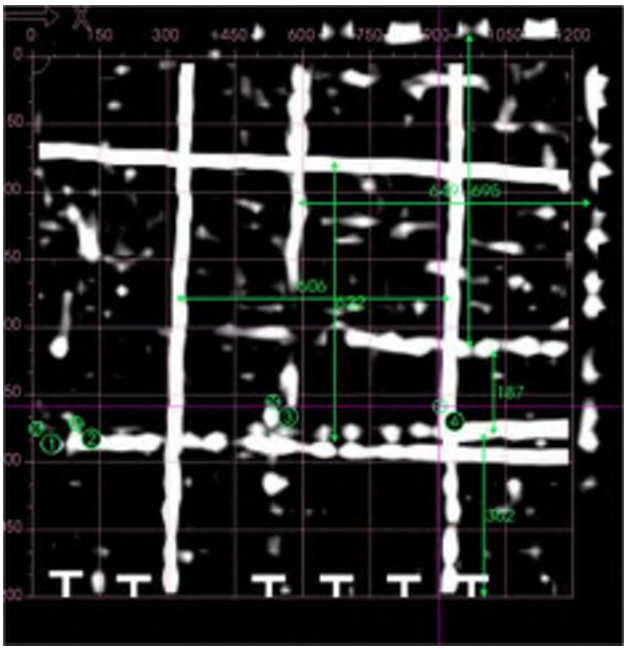


Figure 10: Scan RS810

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	5.6
Filename	RS_054210001_000810.hscan	Timestamp	-
	Wall Scan taken on Grid 11/E-F, as depicted in Figure 7. The Left scan margin is approx. 720mm from the column 11/E and the top scan margin is 1300mm above the floor. Increased moisture content is present in the bottom of the scan, within the concrete Reinforcing bars: • two layers of reinforcement bars • vertical and horizontal bars connect ○ First layer: ▪ min. concrete cover approx. 48mm ▪ horizontal bars, c/c approx. 620mm ▪ vertical bars, c/c approx. 600mm ○ Second layer: ▪ horizontal bars, c/c from approx. 570mm and 645mm, respectively ▪ vertical bars, c/c approx. 540mm and 640mm, respectively Starter bars: • suspected starter bars at annotations 1, 2, 3 and 4 • bars terminate approx. 550mm above the slab • for c/c refer to RQ811		

3.1.1.4 RQ811 and RQ812

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	5.6
Filename	RQ 811 and RQ 812	Timestamp	-
Comment:	Wall Scans not displayed RQ811: Horizontal scan taken approx. 250mm above the floor on grid 11 between columns E to F Vertical bars: • two layers of reinforcement bars o first layer: ▪ min. concrete cover approx. 55mm ▪ vertical bars, c/c approx. 600mm Starter bars: • bars appear to be arranged mostly in pairs with a close c/c o increased c/c between the pairs • average c/c is approx. 319mm RQ812: Horizontal scan taken approx. 1250mm above the floor on grid 11 between column E to F Vertical bars: • two layers of reinforcement bars o first layer: ▪ min. concrete cover approx. 49mm ▪ average c/c is approx. 603mm o second layer: ▪ bars, average c/c is approx. 608mm		

3.2 Level 1

3.2.1 Scans Grid A 8/9

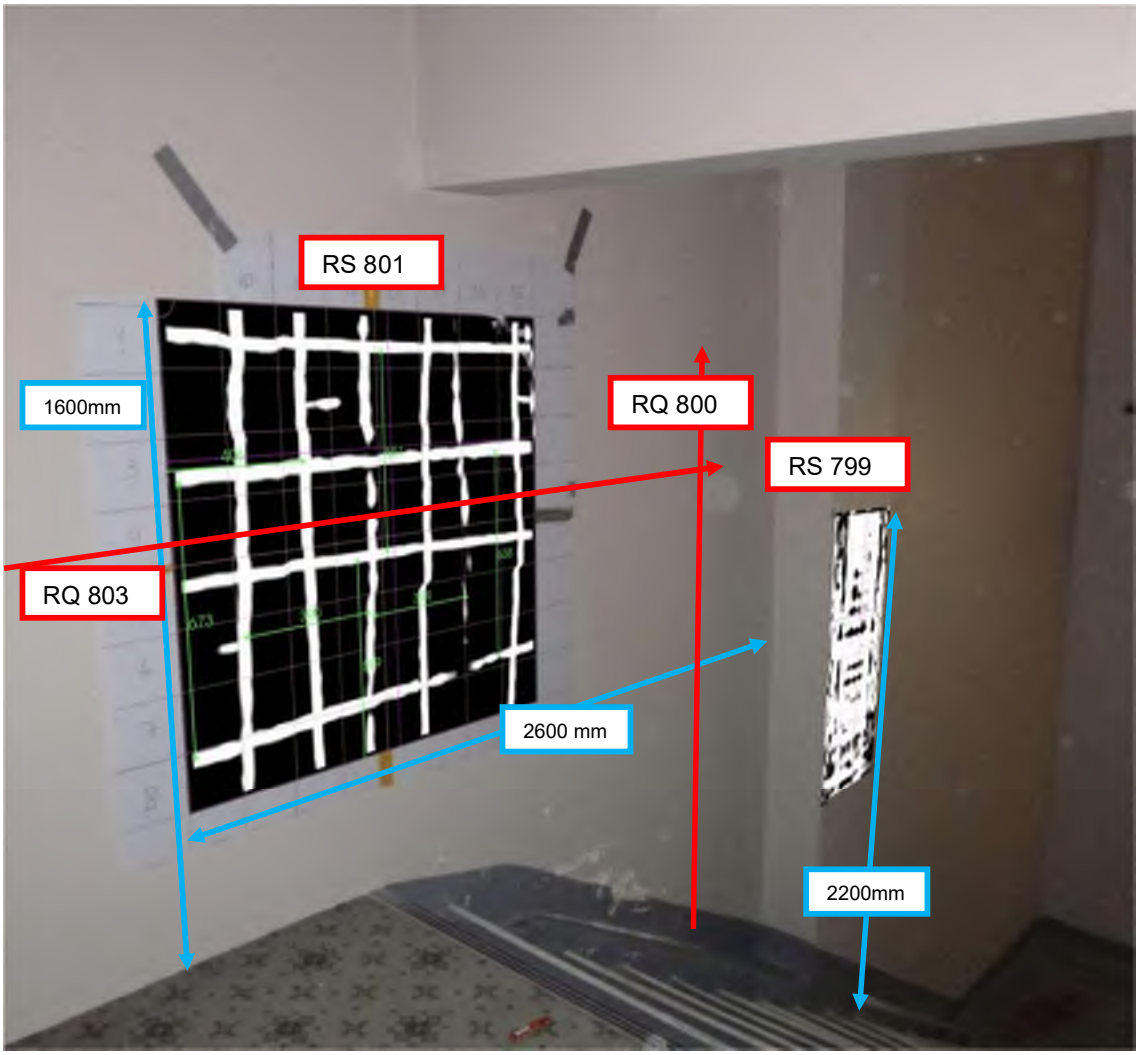


Figure 11: Wall, Column Scan Grid A- 8/9

3.2.1.1 RS801

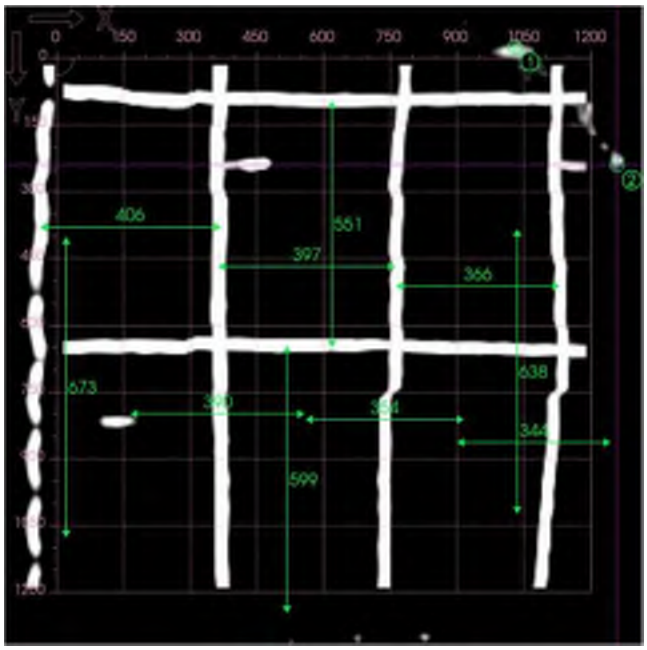


Figure 12: Scan RS801

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	7.0
Filename	RS_054210001_000801.hscan	Timestamp	-
Comment:	Wall Scan taken on Grid A/8-9, as depicted in Figure 11. The right scan margin is approx. 1400mm from the column and 1600mm above the floor. Scan data indicate the possibility for increased moisture content within the concrete Reinforcing bars: • two layers of reinforcement bars • vertical and horizontal bars connect ○ first layer: ▪ min. concrete cover approx. 53mm ▪ horizontal bars, c/c approx. 530mm and 600mm ▪ vertical bars, c/c between approx. 370mm to 400mm ○ second layer ▪ horizontal bars, c/c approx. is approx. 640mm and 560mm ▪ vertical bars, c/c between is approx. 340mm to 390mm ▪ annotations 1 and 2; a portion of a diagonal bar is visible		

3.2.1.2 RS799

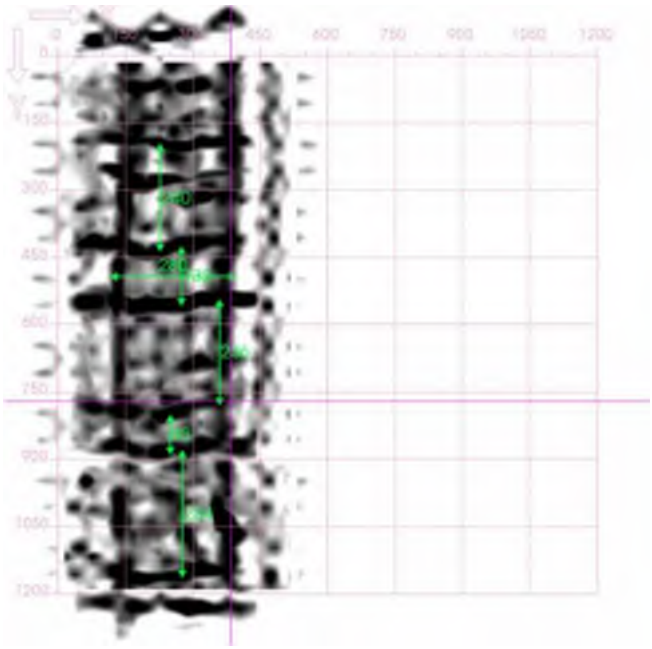


Figure 13: Scan RS799

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	7.0
Filename	RS_054210001_000799.hscan	Timestamp	-
Comment:	Column Scan taken on column A/9, as shown in Figure 11. The top scan margin is approximately 2200mm above the stairs. Structural steel column: • flange width is approx. 320mm • wire-stirrups ○ c/c varies from approx. 90mm to 290mm		

3.2.1.3 RQ800 and RQ803

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	7.0
Filename	RQ800 and RQ803	Timestamp	-
Comment:	Wall Scans not displayed RQ800: Vertical scan taken left of the column A/9 Scan distance 2400mm Horizontal steel beam: • approx. x=800 • concrete cover approx. 85mm • flange width is approx. 180mm Horizontal bars: • two layers of reinforcement bars ○ first layer: ▪ min. concrete cover approx. 23mm ▪ bars, c/c approx. 620mm to 770mm ○ second layer: ▪ bars, c/c approx. 600mm RQ803: Horizontal scan taken on the fourth pass of scan RS 801 Scan distance 3500mm Vertical bars: • two layers of reinforcement bars ○ first layer: ▪ min. concrete cover approx. 31mm ▪ bars, c/c approx. 400mm ○ second layer ▪ bars, c/c approx. 400mm		

3.2.2 Scans Grid G 8/9

Figure 14: Wall Scan Grid G- 8/9

3.2.2.1 RS5380

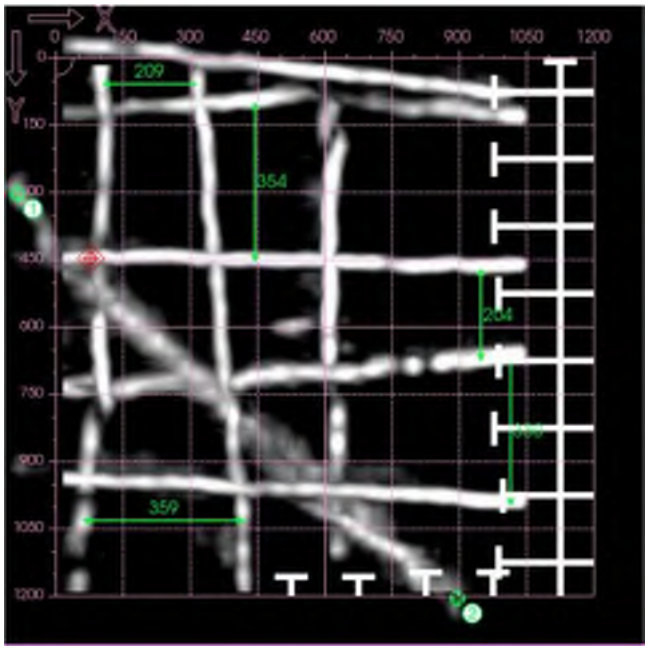


Figure 15: Scan RS5380

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	6.0
Filename	RS_070140002_005380.hscan	Timestamp	-

Comment:	Wall
	
	Scan taken on Grid G/8-9, as depicted in Figure 14. The right scan margin is 75mm from the column and the top scan margin is 1500mm above the floor.
	
	Scan data indicate the possibility for increased moisture content within the concrete
	
	Reinforcing bars:
	• reinforcement is irregularly placed
	• annotations 1 and 2; diagonal bar
	• vertical and horizontal bars connect
	  ○ horizontal bars, c/c from approx. 200mm to 360mm
	  ○ vertical bars, c/c from approx. 210mm to 360mm
	• min. concrete cover approx. 31mm		

3.2.3 Scan Grid G9

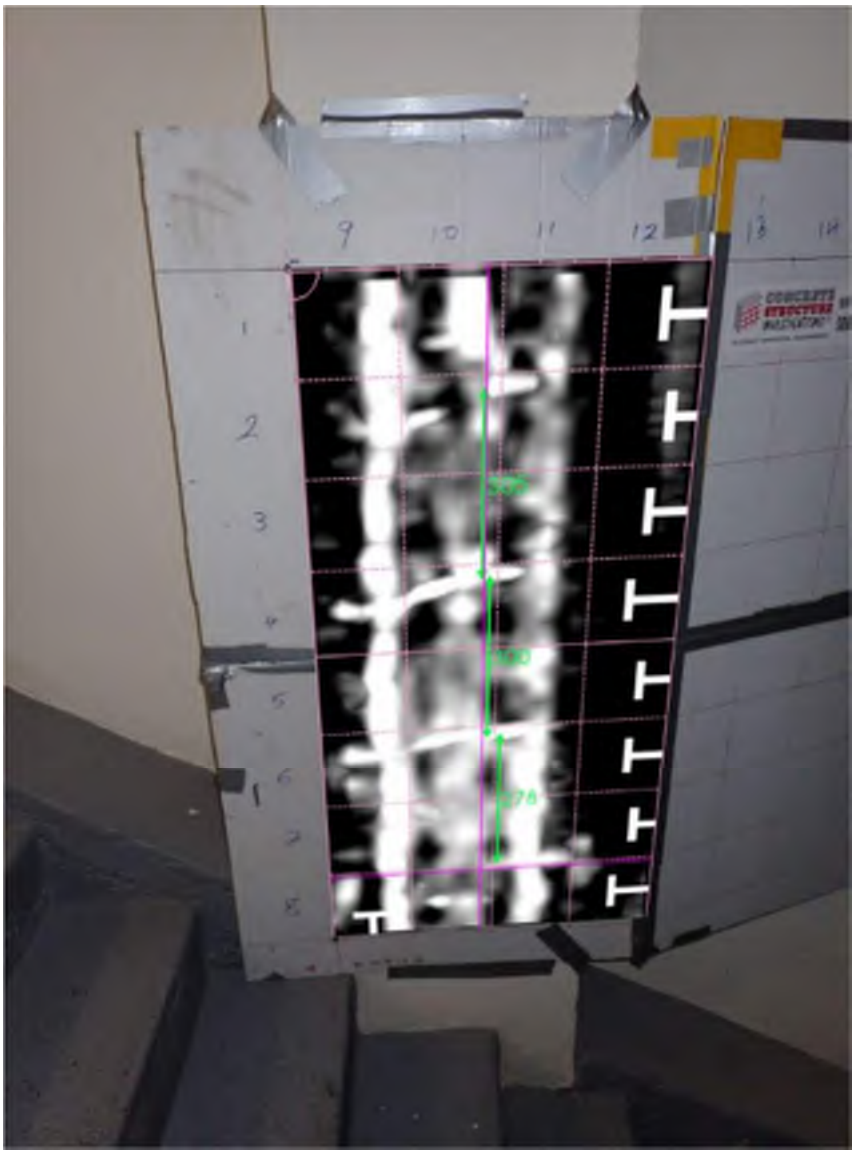


Figure 16: Column Scan Grid G9

3.2.3.1 RS5390

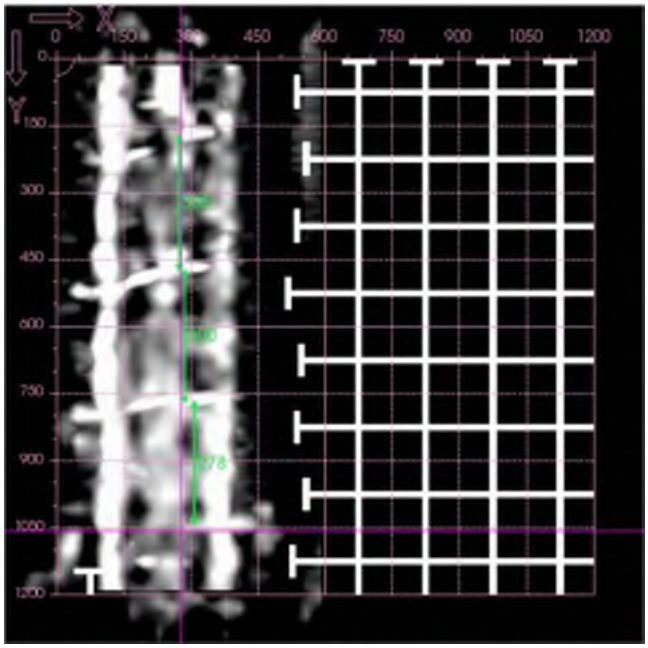


Figure 17: Scan RS5390

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	5.0
Filename	RS_070140002_005390.hscan	Timestamp	-
Comment:	Column Scan taken on Grid G/9, as shown in Figure 16. The top scan margin is approximately 1350mm from the stairs. Structural steel column: • flange width is approx. 320mm • wire-stirrups ○ c/c varies from approx. 278mm to 300mm		

3.2.4 Scans Grid 11 D/E

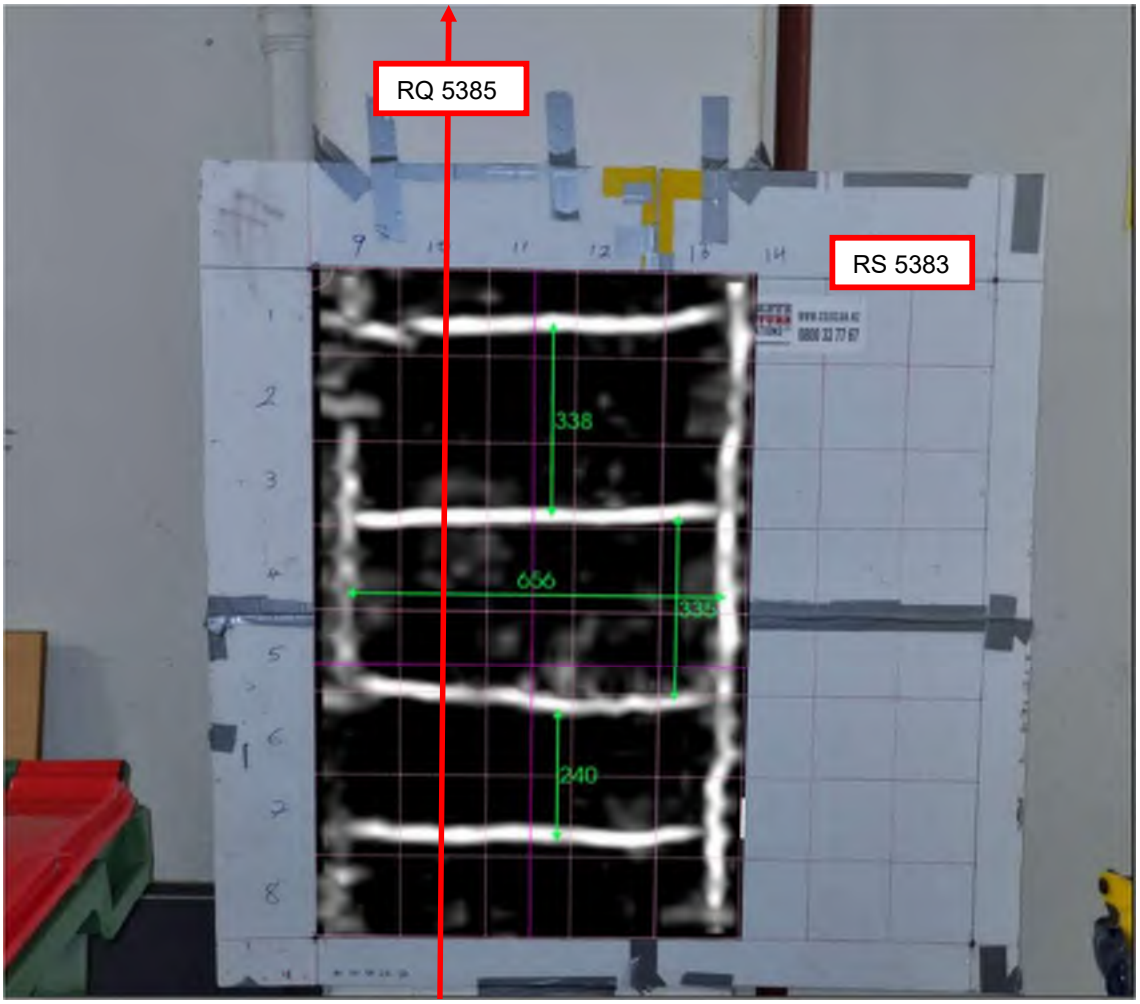


Figure 18: Column Scan Grid 11-D

3.2.4.1 RS5383

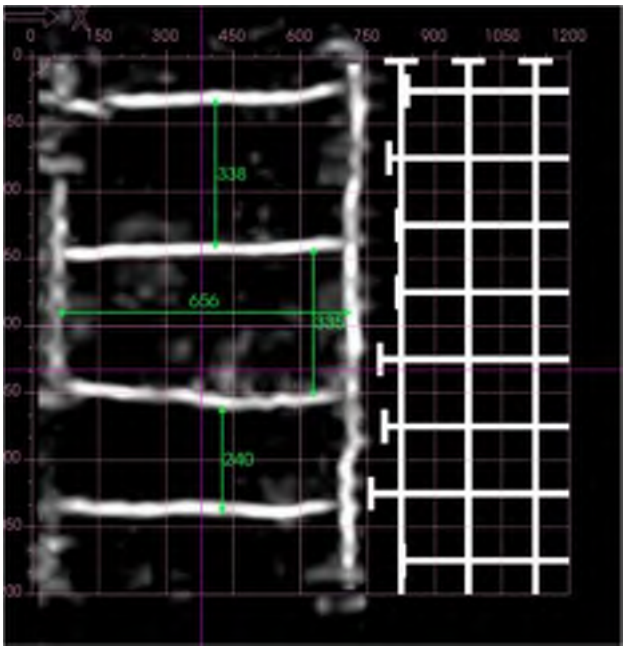


Figure 19: Scan RS5383

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	6.0
Filename	RS_070140002_005383.hscan	Timestamp	-
Comment:	Column Scan taken on Grid 11/D, as shown in Figure 18 Reinforcing bars: • min. concrete cover approx. 62mm • vertical bars c/c approx. 650mm • stirrups c/c approx. from 240mm to 340mm Starter bars: • indications for starter bars are given, although these cannot be confirmed with certainty ○ suspected termination at y=290 ○ further investigations required to confirm the starter bars with certainty Additional reinforcing: • horizontal bars are visible at a cover of approx. at 300-400mm • c/c is approx. 450 and 1070mm, respectively • possibility of wall reinforcing developing through the column ○ further investigations are required for confirmation Steel column (as per drawings): • cannot be confirmed from the available data • further investigations are required to confirm the presence / absence		

3.2.4.2 RQ5385

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	6.0
Filename	RQ_070140002_005385.hscan	Timestamp	-
Comment:	Column Scans not displayed RQ5385: Vertical scan taken on column Grid 11/D Reinforcing bars: • min. concrete cover approx. 37mm • stirrups average c/c approx. 290mm, max 370mm, min. 160mm		

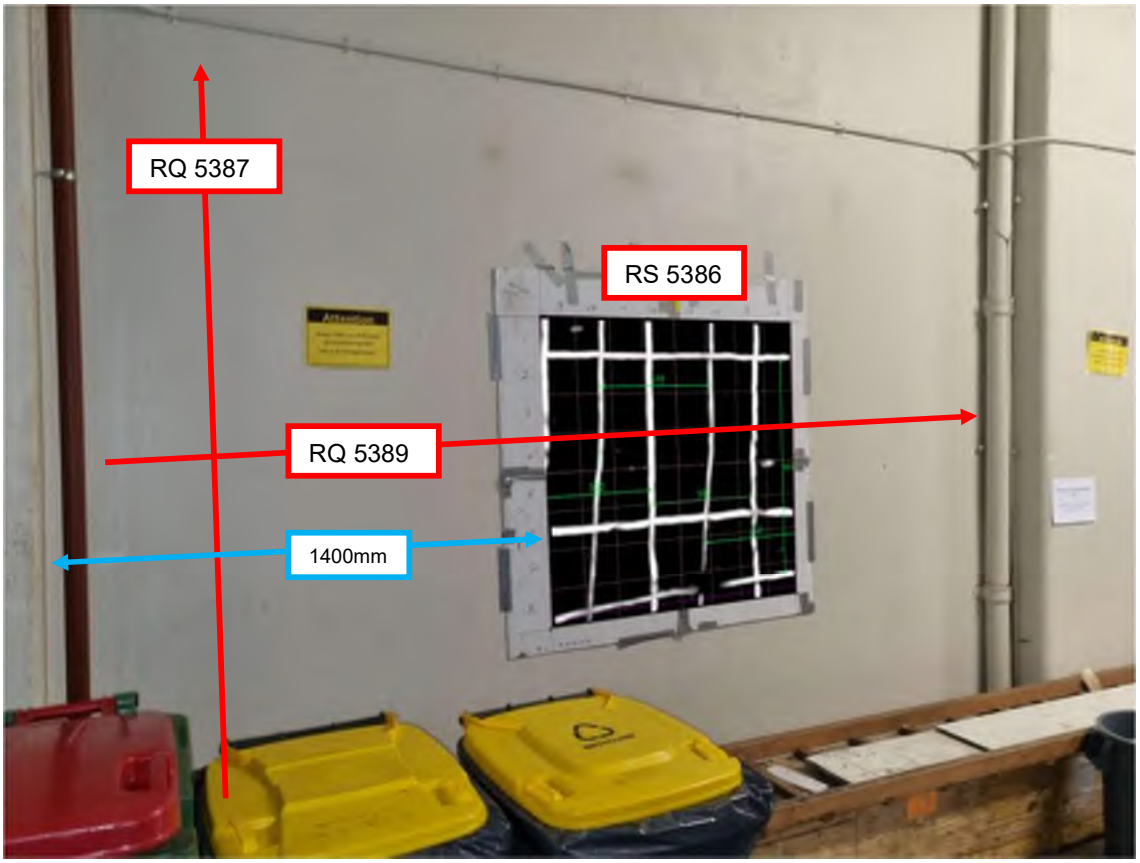


Figure 20: Wall Scan Grid 11 D/E

3.2.4.3 RS5386

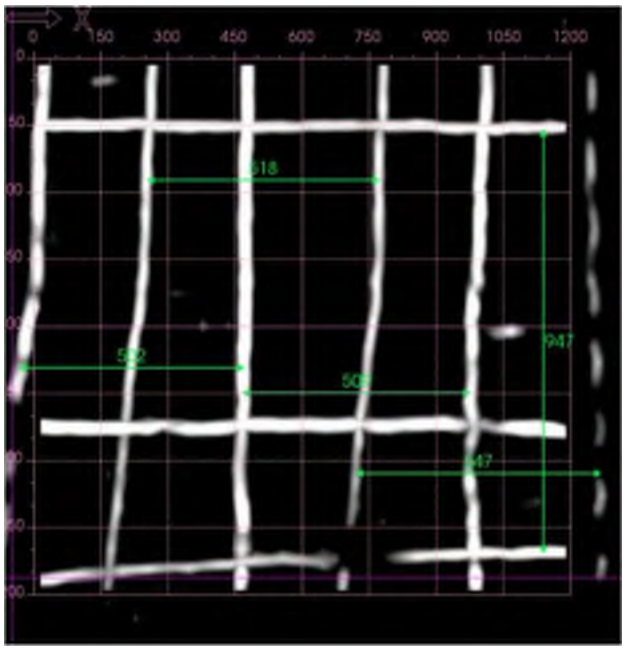


Figure 21: Scan RS5386

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	6.0
Filename	RS_070140002_005386.hscan	Timestamp	-
Comment:	Wall Scan taken on Grid 11 between columns D and E, as depicted in Figure 20. The left scan margin is approx. 1400mm from the column grid 11/D Reinforcing bars: • two layers of reinforcement bars • vertical and horizontal bars connect ○ first layer: ▪ min. concrete cover approx. 23mm ▪ horizontal bars, c/c approx. 940mm ▪ vertical bars, c/c approx. 515mm and 540mm ○ second layer: ▪ only one horizontal bar is visible within the scan at y=825 ▪ vertical bars, c/c approx. 500mm and 510mm, respectively		

3.2.4.4 RQ5387 and RQ5389

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	6.0
Filename	RQ5387 and RQ5389	Timestamp	-
Comment:	Wall Scan not displayed Scans taken on Grid line 11 between columns D and E, as depicted in Figure 20. RQ5387: Vertical scan taken right from column grid 11/D Horizontal bars: • min. concrete cover approx. 64mm • bars, average c/c approx. 440mm, min. 160mm - max. 690mm RQ5389: Horizontal scan taken on grid 11 between columns D and E, on pass 3/4 from Scan RS5386 Vertical bars: • two layers of reinforcement bars o first layer: ▪ min. concrete cover approx. 21mm ▪ bars, average c/c approx. 500mm, max. 790mm – min. 250mm o second layer: ▪ bars, average c/c approx. 550mm, max. 700mm – min. 470mm		

3.3 Level 2

3.3.1 Scan Grid 2- A/B

Figure 22: Wall Scan Grid 2- A/B

3.3.1.1 RS813

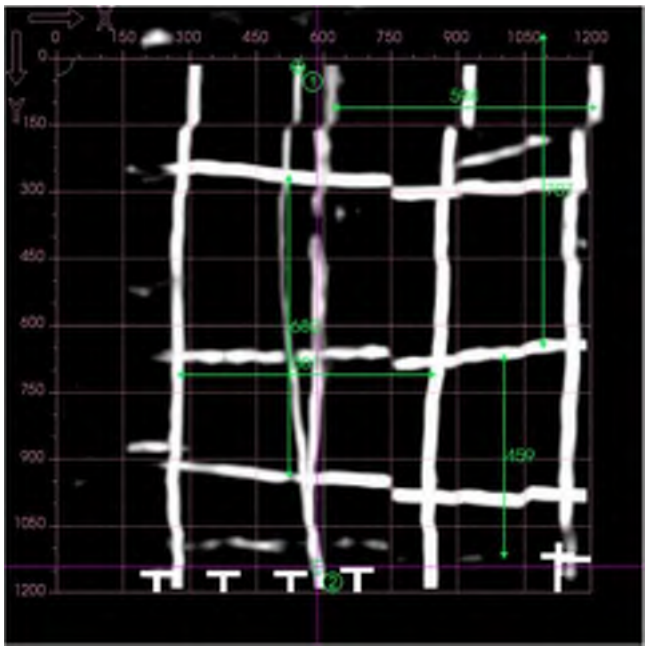


Figure 23: Scan RS813

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	6.0
Filename	RS_054210001_000813.hscan	Timestamp	-

Comment:	Wall
	
	Scan taken on Grid 2/A-B, as depicted in Figure 22. Left edge aligns with y=150, top scan margin is 1300mm above the floor.
	
	Reinforcing bars:
	• two layers of reinforcement bars
	• vertical and horizontal bars connect
	  ○ first layer:
	  ▪ min. concrete cover approx. 65mm
	  ▪ horizontal bars, c/c approx. 680mm
	  ▪ vertical bars, c/c approx. 580mm
	  ○ second layer:
	  ▪ horizontal bars, c/c approx. 460mm and 710mm
	  ▪ vertical bars, c/c approx. 590mm
	
	Low cover detection between annotations 1 & 2
	• min. cover of 9mm
	• unclear whether it is a service or bar
	• does not connect to any reinforcement		

3.3.2 Scan Grid A-1 /2

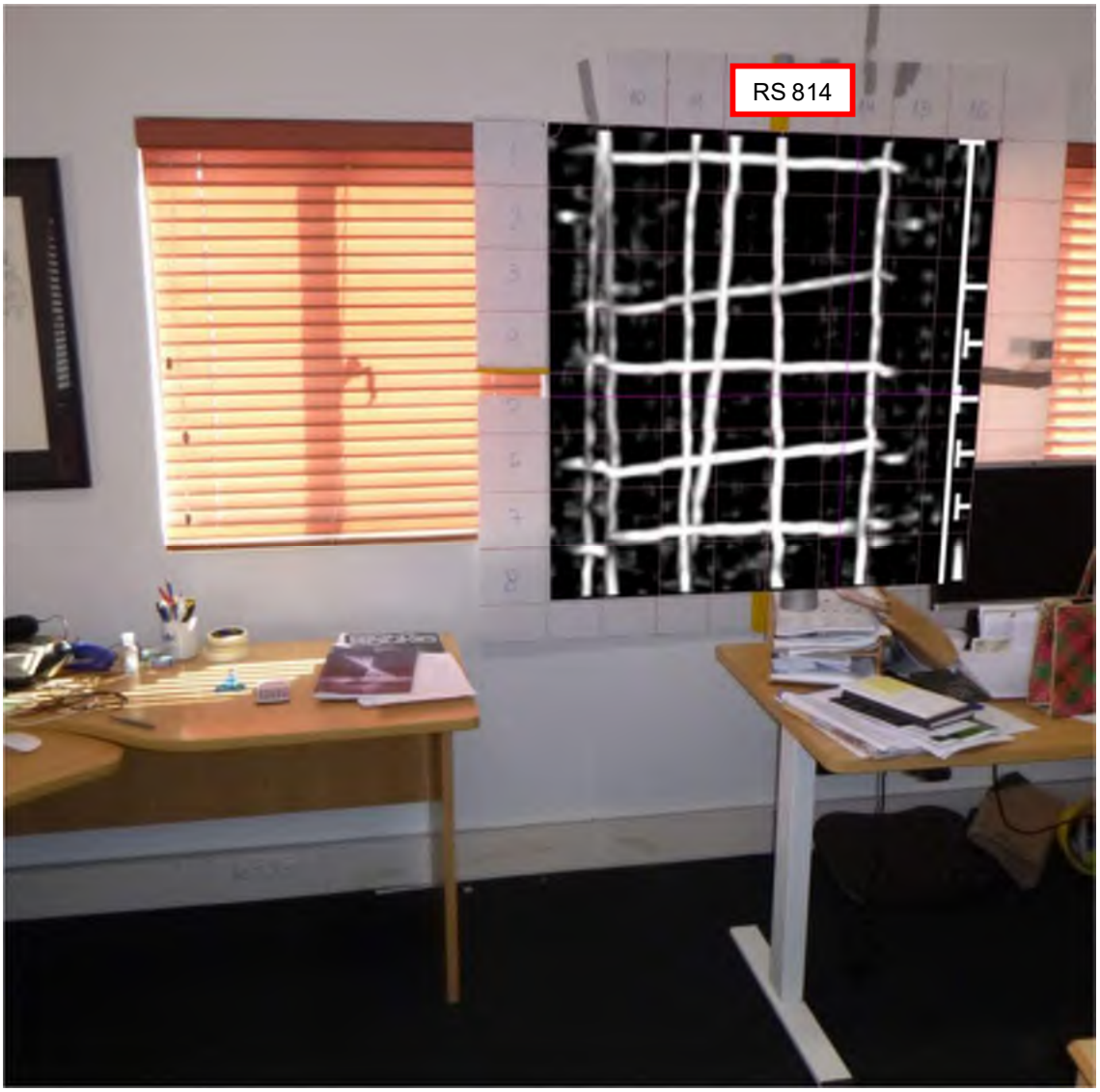


Figure 24: Wall Scan Grid A- 1/2 Between Windows

3.3.2.1 RS814

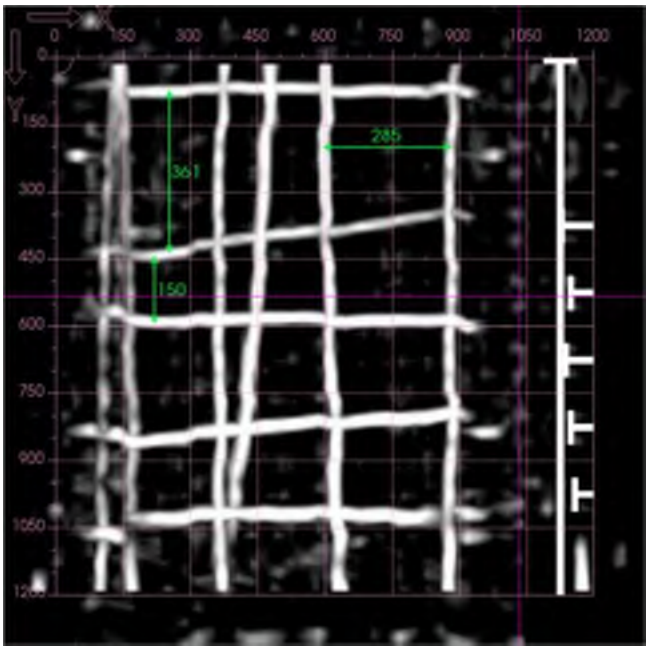


Figure 25: Scan RS814

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	5.6
Filename	RS_054210001_000814.hscan	Timestamp	-
Comment:	Wall Scan taken on Grid A/1-2, as depicted in Figure 24. The top scan margin is approx. 1970mm above the floor. Scan data indicate the possibility for increased moisture content within the concrete Reinforcing bars: • two layers of reinforcement bars • vertical and horizontal bars connect ○ first layer: ▪ min. concrete cover approx. 18mm ▪ horizontal bars, average c/c approx. 240mm ▪ vertical bars, average c/c approx. 240mm ○ second layer ▪ horizontal bars, average c/c approx. 240mm ▪ vertical bars, average c/c approx. 280mm		

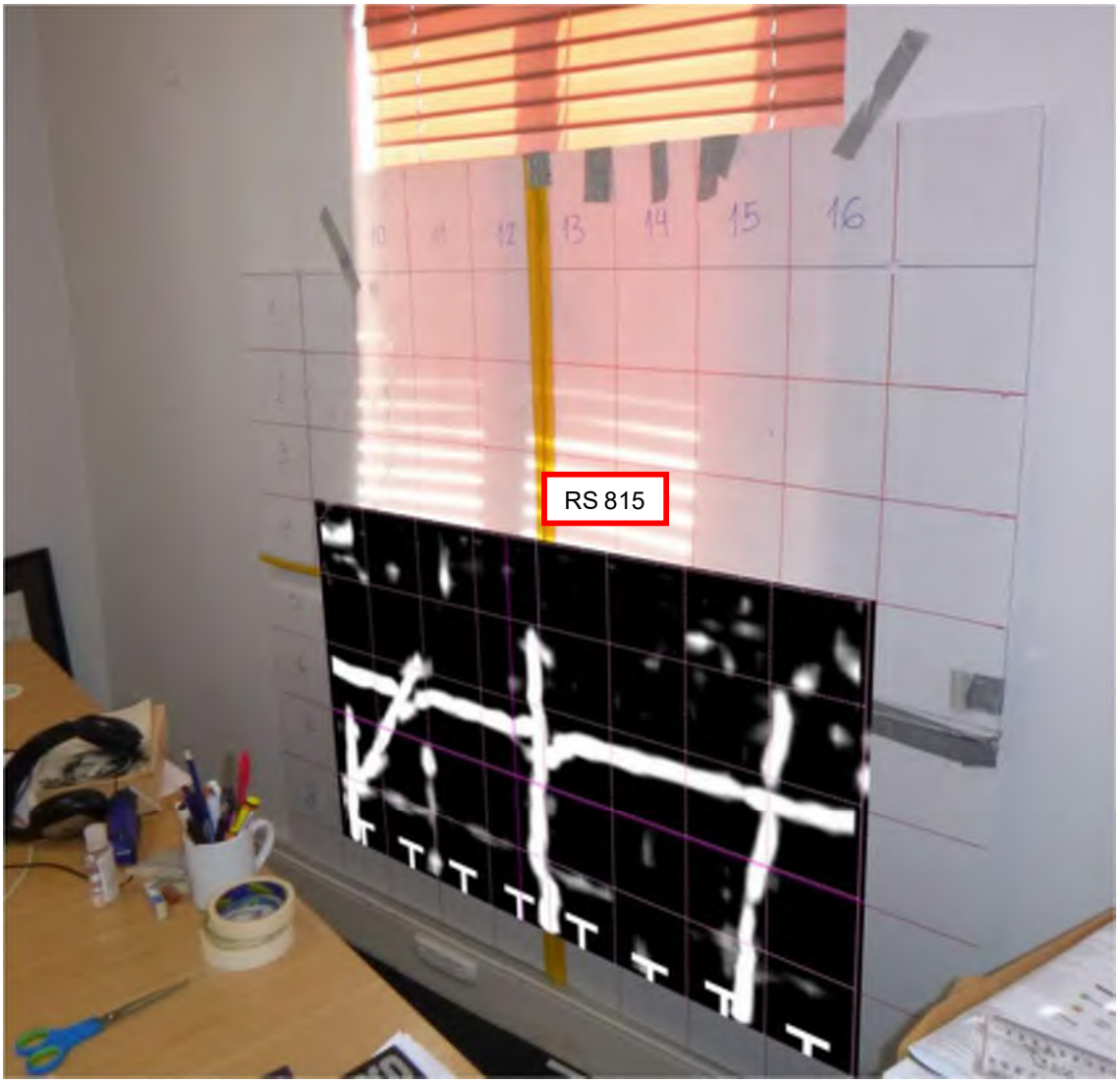


Figure 26: Wall Scan Grid A - 1/2

3.3.2.2 RS815

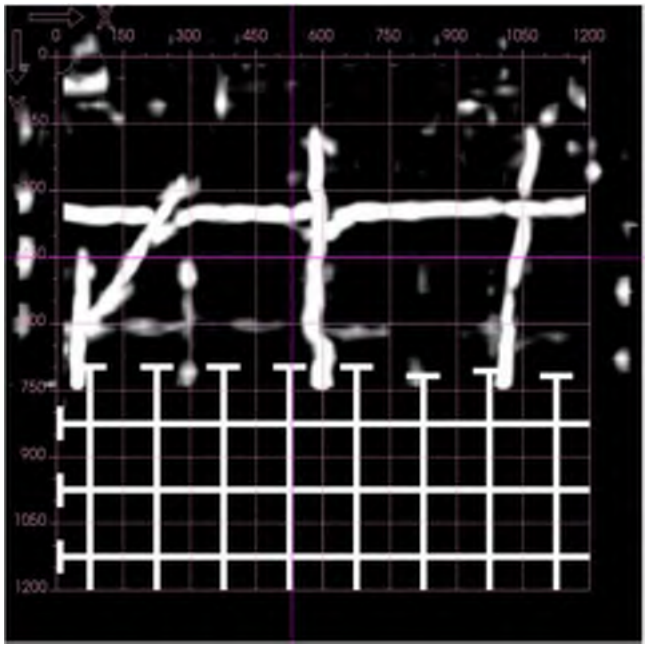


Figure 27: Scan RS815

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	5.6
Filename	RS_054210001_000815.hscan	Timestamp	-
Comment:	Wall Scan taken on Grid A/1-2, as depicted in Figure 26. The left scan margin is approx. 940mm from the column grid A/1. Reinforcing bars: • two layers of reinforcement bars • vertical and horizontal bars connect ○ first layer: ▪ min. concrete cover approx. 45mm ▪ horizontal bars, c/c approx. 440mm ▪ vertical bars, c/c approx. 650mm and 480mm, respectively ○ second layer ▪ horizontal bars, average c/c approx. 430mm ▪ vertical bars, c/c approx. 530mm and 410mm, respectively		

3.3.3 Scan Grid 1- C/B

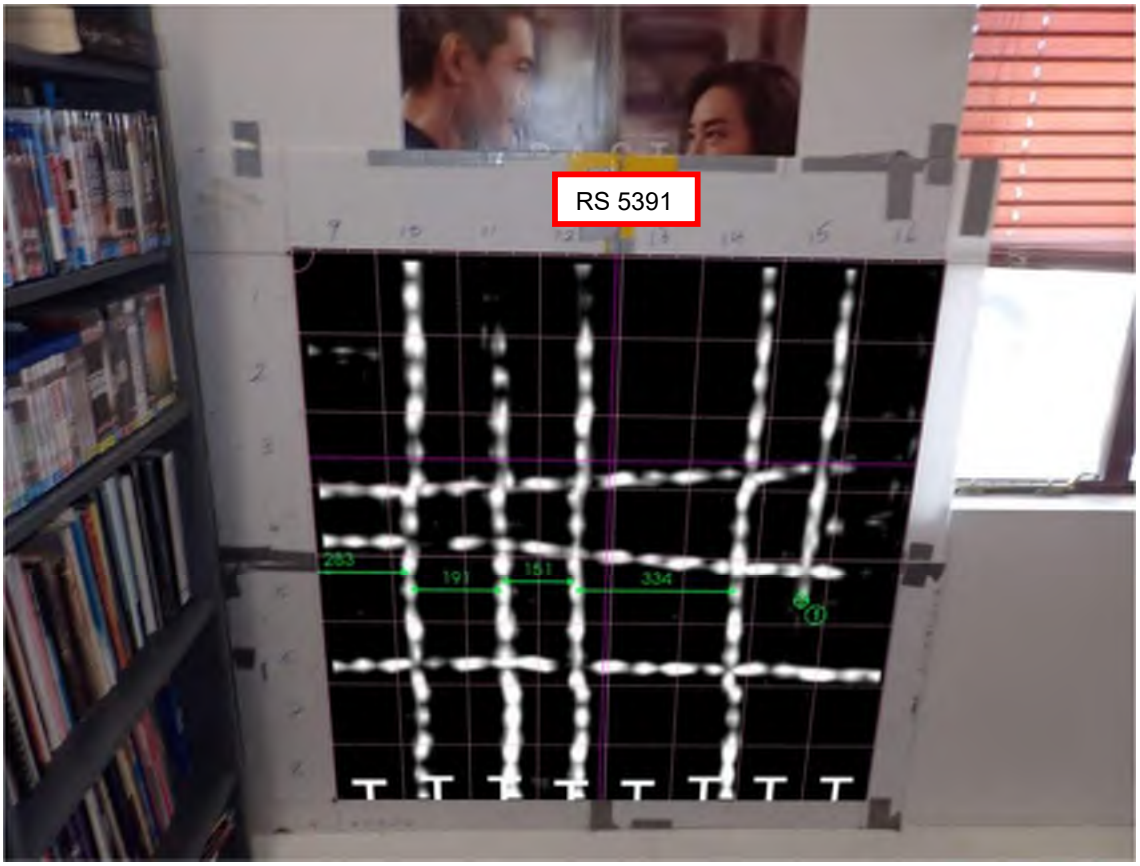


Figure 28: Wall Scan Grid 1- C/B

3.3.3.1 RS5391

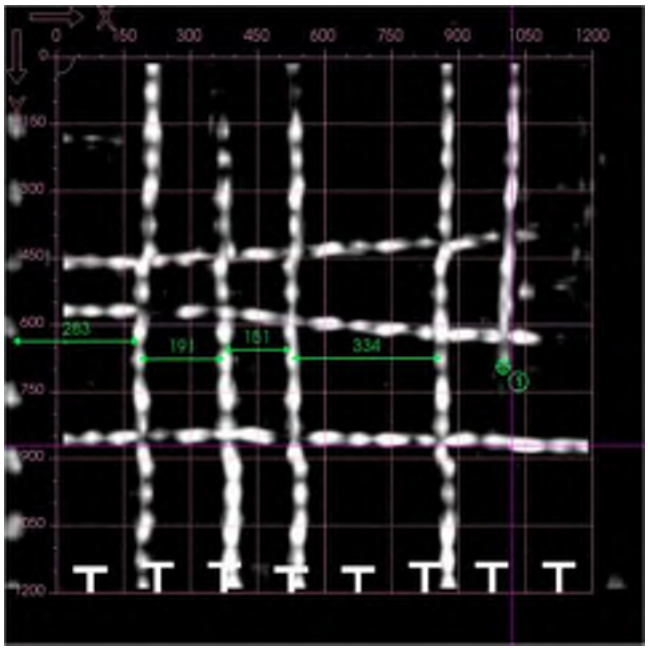


Figure 29: Scan RS5391

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	5.0
Filename	RS_070140002_005391.hscan	Timestamp	-
Comment:	Wall Scan taken on Grid 1/C-B, as depicted in Figure 28. Reinforcing bars: • two layers of reinforcement bars • vertical and horizontal bars connect ○ first layer: ▪ min. concrete cover approx. 45mm ▪ horizontal bars, c/c approx. min. 100mm - max. 280mm ▪ vertical bars, c/c varies from approx. 150mm to 330mm • annotations 1; bar emerges from the top and terminates at annotation ○ second layer: ▪ horizontal bars, c/c approx. 430mm and 550mm, respectively ▪ vertical bars, average c/c approx. 280mm		

3.3.4 Scan Grid 1/2 – D/C

Figure 30: Slab Scan Grid 1/2 - D/C

3.3.4.1 RS820

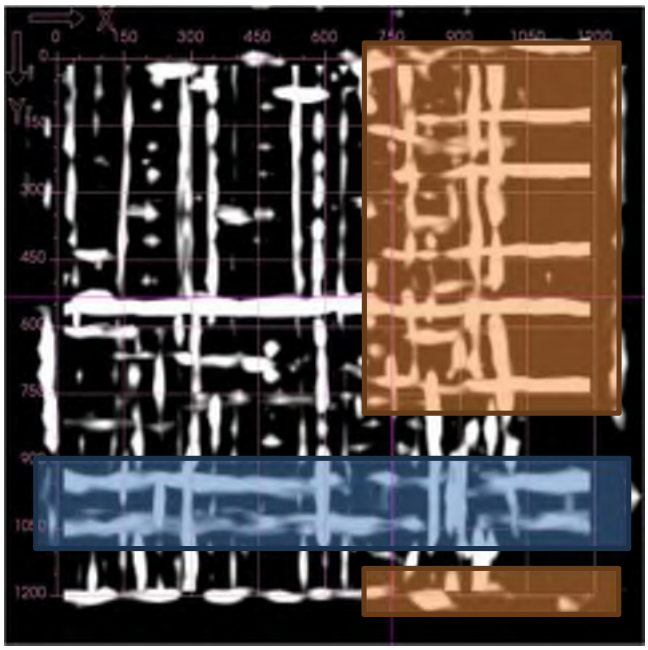


Figure 31: Scan RS820

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	7.0
Filename	RS_054210001_000820.hscan	Timestamp	-
Comment:	Slab Scan taken on Grid D/1-2, as depicted in Figure 30. The top scan margin is approx. 8000mm from the wall at grid G, the right scan margin is approx. 1780mm from the wall at grid 1 Steel beam: • highlighted in blue • flange width is approx. 150mm Slab reinforcement: • one layer: ○ c/c average of 600mm, parallel to numbered grid lines (x-direction) ○ c/c average of 118mm, parallel to lettered grid lines (y-direction) • additional bars: ○ bars in x-direction ▪ orange highlighted areas ▪ bars emerge from the right and terminate at around x=750 ▪ average c/c approx. 159mm ▪ the bars sit on top of the above-mentioned layer ▪ the bars connect to the above-mentioned layer ▪ possible structural change at the right scan margin, visual inspection from below is recommended ○ bars in y-direction ▪ bars emerge from the bottom of the scan ▪ where possible the termination is determined to be between y=510-665		

	▪ refer to RS821 for second termination point ▪ the number of bars / c/c is indeterminable

3.3.4.2 RS821

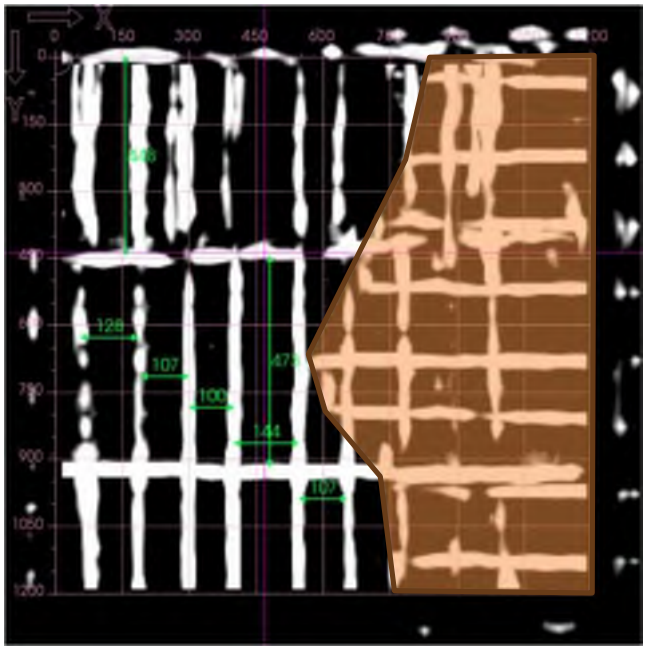


Figure 32: Scan RS821

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	7.0
Filename	RS_054210001_000821.hscan	Timestamp	-
Comment:	Slab Scan taken on Grid D/1-2, as depicted in Figure 30. The top scan margin is approx. 9200mm from the wall grid G, right scan margin is approx. 1780mm from the wall grid 1. Slab reinforcement: • one layer: o c/c average is 450mm, parallel to numbered grid lines (x-direction) o c/c average is 116mm, parallel to lettered grid lines (y-direction) • additional bars: o bars in x-direction ▪ orange highlighted area ▪ bars emerge from the right and terminate between around x=600-750 ▪ c/c average is 153mm ▪ the bars sit on top of the above-mentioned layer ▪ the bars connect to the above-mentioned layer ▪ possible structural change at the right scan margin, visual inspection from below is recommended o bars in y-direction ▪ bars emerge from top of scan and terminate between y=310-455 ▪ c/c average is 145mm		

	▪ refer to RS820 for second termination point ▪ the number of bars / c/c is indeterminable

3.3.4.3 RS822

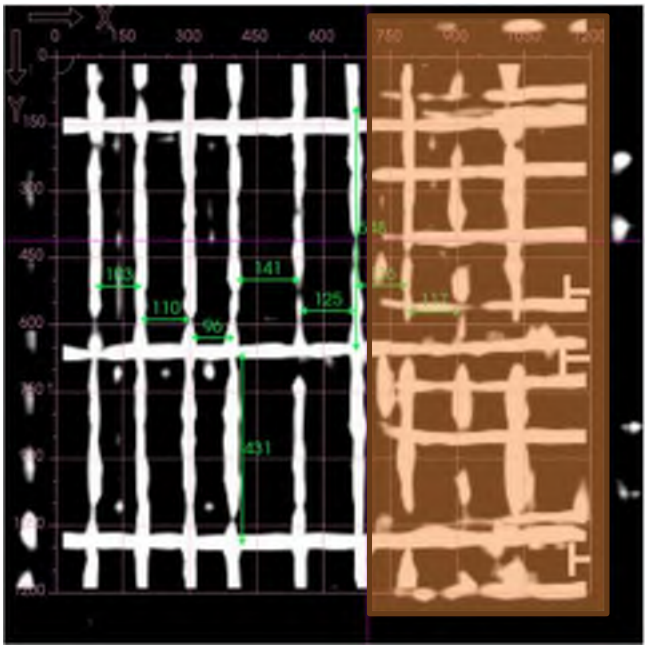


Figure 33: Scan RS822

Project name:	234158	Customer:	-
Location:	-	Object:	-
User:	-	Permittivity:	7.0
Filename	RS_054210001_000822.hscan	Timestamp	-
Comment:	Slab Scan taken on Grid line D-C/1-2, as depicted in Figure 30. The top scan margin is approx. 10400mm from the wall at grid G, the right scan margin is approx. 1780mm from the wall grid 1. Slab reinforcement: • one layer: o c/c average of 450mm, parallel to numbered grid lines (x-direction) o c/c average of 159mm, parallel to lettered grid lines (y-direction) • additional bars: o bars in x-direction ▪ in orange highlighted areas ▪ bars emerge from the right and terminate at around x=705 ▪ average c/c approx. 159mm ▪ the bars sit on top of the above-mentioned layer ▪ the bars connect to the above-mentioned layer ▪ possible structural change at the right scan margin, visual inspection from below is recommended		

4 Conclusion of Results

Note: No bar diameter were able to be determined by scanning due to technical limitations. Where required, bar diameter must be established through physical investigation.

4.1 Level 0 and Level 1 – Grid 11

4.1.1 Columns

The columns on level 0 and level 1 consist of longitudinal bars in the corners, enclosed by stirrups.

Some indications for possible starter bars can be seen in the column scan on level 1, although these cannot be confirmed with certainty without additional investigations.

Horizontal bars at greater depth are visible. These bars are potentially part of the wall reinforcement propagating through the column. Additional investigations are required to confirm this.

A steel beam as shown on the provided drawings cannot be confirmed which may be due to limitations of the used device. Additional scanning with a different antenna is required to either, confirm the existence or absence of a steel beam.

Exposed reinforcement on column 11/F revealed round, plain bars. The longitudinal bars and stirrups have diameters of 25mm and 12mm, respectively.

4.1.2 Walls

The walls on level 0 and level 1 contain two layers of reinforcement with large and varying centre spacings for both, horizontal and vertical bars.

For details refer to the scan section.

4.2 Level 1 – Grid A and G

4.2.1 Columns

The columns on grid A and G consist of a structural steel column with stirrups. The flange width is estimated to be 320mm. The signal characteristics of the stirrups indicate a very small diameter.

4.2.2 Walls

The walls contain two layers of reinforcement with large and varying centre spacing for both, horizontal and vertical bars.

For details refer to the scan section.

4.3 Level 2

4.3.1 Grid 2

4.3.1.1 Internal Wall

The scan taken on an internal wall on level 2 shows two layers of reinforcement with varying centre spacings.

4.3.2 Grid A

4.3.2.1 Wall

The wall contains two layers of reinforcement with average centre spacings between 240-280mm.

4.3.2.2 Spandrel

The spandrel consists of two layers of reinforcement with varying centre spacings.

4.3.3 Grid 1

4.3.3.1 Wall

The wall consists of two layers of reinforcement with varying centre spacings.

4.3.4 Grid D/1-2

4.3.4.1 Slab

An area covering 1200x3600mm was scanned on grid D. The slab contains one layer of reinforcement with a wide centre spacing for bars in one direction and close centre spacing in the other direction.

A structural steel beam is apparent on grid D, the flange width is estimated to be 150mm. Additional bars, either lapping bars or saddle bars are sitting on top of the slab reinforcing and extend past either side of the beam.

A possible change of structure is noticeable on the right scan margins, parallel to grid 1. In addition, bars emerge from the same direction and are located over the possible structural change. These bars have a similar centre spacing and cover as the bars over the steel beam. Further investigations are required to establish the development of the bars and the possible structural change.

For details refer to the scan section.

Note: All scan results and interpretation are a professional estimate based on the limitations of the hardware, software and environment. If the location, size and depth of the reinforcing steel bars must be determined to an absolute certainty, then a physical examination of those bars is required.

Our investigation and report are limited to those areas specifically identified within this report, for the sole purpose of the scope identified. Unless stated otherwise, we have not inspected

framing or any other parts of the structure which are covered, concealed or inaccessible and there is the possibility that different conditions exist elsewhere within the subject structure.

5 GPR Limitations

GPR Limitations within concrete structures:

Note: The below limitations do not cover all possible limitations but only the most common.

Data Collection and Interpretation – The technician providing the GPR results is potentially the biggest limiting factor involved with this science. Technicians must not only be trained in operating the technology, they also must have a sound understanding of the material/structure and application in each specific situation; however, teaching an individual how to interpret the data they receive with the equipment can take long periods of time and ongoing training. The highest quality equipment operated by an inexperienced technician will offer little information to the customer as the ability to interpret the data is essential. In short, the technology/science of ground penetrating radar is only as good as the operator's expertise and education in data collection and interpretation.

Moisture – Moist or 'green' concrete can be problematic for GPR as the presence of moisture will reflect/inhibit the passage of the radar pulse and thereby limit penetration and data quality.

Depth Penetration – The depth range of GPR is limited by the electrical conductivity of the medium, the transmitted centre frequency and the radiated power. As conductivity increases, the penetration depth decreases. Higher frequencies do not penetrate as far as lower frequencies but give better resolution. The best penetration is achieved in dry materials such as granite, limestone, and dry concrete.

Size of Target – There are two main ways in which GPR is limited when discussing the size of a target. GPR technology is unable to determine the diameter of the target being located. Dimensions of objects can in certain circumstances be given within tolerances which are specific to the site conditions and scanner used. As a rule of thumb, objects smaller than half the size of the wavelength cannot be detected. Larger objects may also not be detected, depending on the size and orientation. The wavelength is correlated with the centre frequency of the antenna used.

Obstructions – Targets may be obstructed by objects positioned in front of them, prohibiting the wave propagation to the target. Closely spaced neighbouring objects may also prohibit the detection of targets beneath.

6 PS 200 Limitations

	Reliable Measurements
Base Material	▪ Reinforced concrete. ▪ Reinforced CMU. ▪ (Brick)

Detected Objects	▪ Rebars which comply with one of the following standards:
	  ▪ DIN 488
	  ▪ ASTM A615/A 615/M-01b
	  ▪ CAN/CSA-G30. 18-M92
	  ▪ JIS G 3112
	  ▪ GB 50012-2002 (China)
	▪ Metal conduit if sufficient spacing from rebars.

Environment / Working conditions	▪ Smooth and flat surfaces.
Rebar layout **Direction**	▪ Rebars lying orthogonal or within ±5° of a right angle to the scanning direction. ▪ Reinforcement running parallel to the surface.
Rebar layout **Diameter**	▪ Neighbouring bars with a similar diameter. ▪ Standard diameters from 6-36mm, up to a depth of 60mm, if s:c ≥ 2:1. ▪ Calibrated bar sizes: {6,8,10,12,14,16,20,25,28,30,36}mm ▪ Device is calibrated to be accurate to within ± 1 of the above intervals; however, this is strongly affected by environmental conditions.

Rebar layout **Spacing**	▪ Minimum bar spacing either 36mm (1.4 in)
	OR
	The minimum ratio of spacing to coverage (s:c), whichever value is greater.
	▪ Minimum ratio of spacing to coverage
	  ▪ Depth calculation / bar diameter estimation s:c = min 2:1
	  ▪ Bar location s:c = min 1.5:1

Rebar layout **Depth calculation**	▪ Neighbouring bars at similar depth/size.
	▪ For accurate depths readings minimum depth of 20mm (0.8 in). For readings at all min. 10mm.
	▪ Maximum depth determination differs with rebar diameter.
	▪ Typically if diameter is known (DIN) Image Scan / Block Scan:
	  ▪ ≤ 10mm: up to 100mm
	  ▪ ≤ 14mm: up to 120mm
	  ▪ ≤30mm: up to 140mm
	  ▪ 36mm: up to 160mm
	Linear scan: up to 100mm
	▪ Accuracy for unknown bar diameter: ± 10% for depths greater than 20mm.