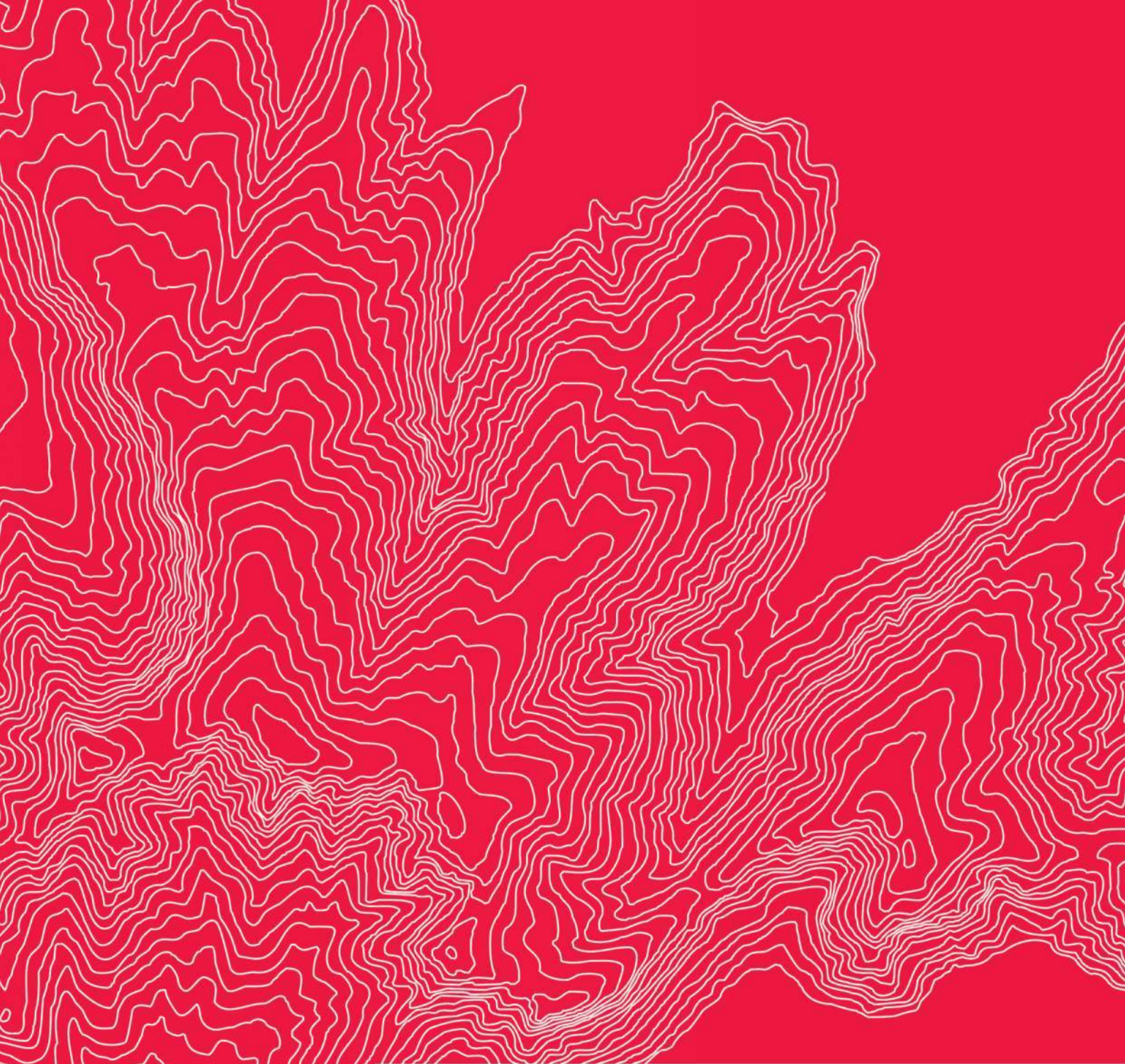




Detailed Seismic Assessment

Version - A

**eliot
sinclair**

36 Weld Street, Hokitika

Prepared for Westland District Council

503048

Detailed Seismic Assessment

36 Weld Street, Hokitika
Prepared for Westland District Council
503048

Quality Control Certificate

Eliot Sinclair & Partners Limited
eliotsinclair.co.nz

Action	Name	Signature	Date
Prepared by:	Sanjeev Raturi Structural Engineer BEngTech, PGCertCivilEng (EQ), CMEngNZ, CPEng sanjeev.raturi@eliotsinclair.co.nz		25 September 2026
Reviewed by:	Travers Armstrong Structural Engineer Director BE(Hons) Civil CMEngNZ CPEng travers.armstrong@eliotsinclair.co.nz		25 September 2026
Directed and approved for release by:	Travers Armstrong Structural Engineer Director BE(Hons) Civil CMEngNZ CPEng travers.armstrong@eliotsinclair.co.nz		25 September 2026
Status:	Version - A		
Release date:	25 September 2026		
Distributed to:	Westland District Council		

Version History

Status	Description	Author	Release Date
A	DSA Report Issue	SR	25 September 2026

Contents

Executive Summary	1
1. Introduction	3
2. Property Description	4
2.1. Site Description	4
2.2. Building Description	4
2.3. Gravity Structure	5
2.4. Seismic Lateral Structure	6
3. Building Investigations	6
3.1. Document Review	6
3.2. Site Inspections	6
3.3. Observations	8
4. Geotechnical considerations	9
4.1. Subsoil description	9
4.2. Foundations	9
5. Seismic Assessments	9
5.1. Quantitative Assessment Methodology	9
5.2. Assessment Parameters (DSA)	10
6. Seismic Grades & Relative Risk	14
7. Conclusions	14
8. Concept strengthening – Primary Structure	15
8.1. Purpose	15
8.2. Scope	15
8.3. Strengthening Works required to achieve 34% NBS(IL2)	16
8.4. Further investigations and information required.	17
Disclaimer	18
Appendix A. DSA Summary Sheets	
Appendix B. Existing Building Plans	
Appendix C. Geotechnical Report	
Appendix D. 34 %NBS Concept Upgrade Sketches	

Executive Summary

Background

Eliot Sinclair & Partners Limited (Eliot Sinclair) has been engaged by Westland District Council to undertake a Detailed Seismic Assessment (DSA) of the Council building at 36 Weld Street, Hokitika. This report has been prepared to summarise the seismic assessment and the associated results.

Building Description

The HQ Building was originally designed by the Ministry of Works and constructed circa 1948 as the Hokitika Post Office. The original reinforced concrete building had an approximate footprint of 610 m² and a total floor area of 1,220 m² over two levels. Its structure comprised reinforced concrete walls, floors, gravity elements and a concrete roof.

During the early 1990s, the building was altered and extended to accommodate the Hokitika Council offices. The works included additions of approximately 170 m² to the southern side and 63 m² to the northern side at ground-floor level. A further floor of approximately 575 m² was constructed over the existing roof deck, creating a three-level building.



Figure 1. View of building (Southern elevation)– from weld street.

The primary gravity structure comprises a steel-framed roof supported on cold-formed steel purlins and transverse portal trusses, with timber ceiling joists below. The suspended floors are reinforced concrete two-way slabs supported by perimeter walls and internal beams, columns and walls. The exterior and internal walls are reinforced concrete and support portions of the floor structure. The ground floor comprises a 127 mm thick reinforced concrete slab, although it is unclear whether this acts as a slab-on-ground or a suspended slab due to potential settlement of the underlying fill.

The foundations comprise reinforced concrete ground beams spanning between piles supported on foundation pads approximately 3.6 m below ground-floor level.

Seismic loads are transferred through roof bracing, steel portal frames, timber-framed walls and diaphragm action in the roof and floor systems. Reinforced concrete floor diaphragms transfer seismic forces to internal and external reinforced concrete shear walls, which then transfer the loads to the foundations through flexure and shear.

Assessed Seismic Rating

Based on the results of the seismic assessment, the building is categorized as having a seismic rating of 25 %NBS which categorizes it as a Grade D building following the NZSEE grading scheme. Grade D buildings represent a risk to building occupants equivalent to 10-25 times that expected for a new building, indicating a high earthquake risk exposure.

A building with a seismic rating less than 34%NBS is considered to be an Earthquake-Prone Building in terms of the Earthquake-Prone Buildings Amendment Act 2016 and a building rating less than 67%NBS as an Earthquake Risk Building by the New Zealand Society of Earthquake Engineering. The building at 36 Weld Street, Hokitika is therefore categorized as an Earthquake-Prone Building.

Conclusions & Recommendations

- a) Based on the seismic assessment outlined in this report, the building has an assessed seismic rating of 25% NBS, as limited by:
 - i) Flexural/shear capacity of concrete column piles in the transverse direction
 - ii) In plane capacity of portal frame connections at third storey in the transverse direction
 - iii) In place capacity of plasterboard and steel flat bracing at the third storey in the longitudinal direction
- b) Based on the above seismic rating, the building is classified as a being in the Grade D category under the NZSEE grading scheme (aka Earthquake Prone). More information on this is provided in Sections 5 & 6 of this report.
- c) Eliot Sinclair & Partners supports the recommendation of the NZSEE that it is desirable to seismically strengthen earthquake risk & Earthquake prone buildings to as near as reasonably practical to that of a new building; but, as a minimum, seismic improvements should achieve at least 67%NBS.
- d) As requested, a conceptual seismic strengthening scheme to achieve a target level of 34% NBS has been provided in Section 8 of this report.
- e) This executive summary is a limited précis of our observations and conclusions. We recommend that this report is read in full. Where any question arises as to the scope or interpretation of the seismic assessment for this building Eliot Sinclair & Partners Ltd should be consulted.

1. Introduction

Eliot Sinclair & Partners Limited (Eliot Sinclair) has been engaged by Westland District Council to undertake a Detailed Seismic Assessment (DSA) of the Council building at 36 Weld Steet, Hokitika.

This DSA summarised in this report, has been undertaken generally in accordance with the EQ-Assess guidelines "The Seismic Assessment of Existing Buildings" issued July 2017.

The purpose of undertaking the DSA is to quantitatively establish the approximate ultimate seismic structural capacity of the existing building with a focus on life safety rather than damage avoidance.

1.1. Scope of Assessment

The scope of work undertaken by Eliot Sinclair includes:

- a) Review of relevant information on the building which has been provided to Eliot Sinclair, including:
 - i) Extracts from the original structural drawings dated circa 1947. Only selected sheets of the original construction drawings were available in the property files. Accordingly, it has been assumed that the construction methodology in other areas of the building, for which no plans were available, was similar to that shown in the available drawings
 - ii) Major alterations and additions were undertaken circa 1990. Apart from a floor plan, no further information regarding these works was available. Accordingly, much of the analysis is based on visual inspections of the members and connections undertaken by Eliot Sinclair in September 2021 and April 2024.
 - iii) The building is operated as an office building with an occupancy of 300 people or less and is therefore classified as importance level 2 in accordance with AS/NZS 1170.0:2002.
- b) Undertake site inspections of the property for the purpose of identifying:
 - i) The nature and general condition of primary structure of the building.
 - ii) Other conditions that could impact on the seismic performance of the building.
- c) Analyse the primary building structural systems based on the information gained from the review of the drawings and knowledge of the detailing used for structures of this era.
- d) Quantitative evaluation of the capacity of the critical structural elements of the building and the seismic demands (internal forces and ductility) on these elements, as derived from the analytical models.
- e) Produce a report summarising the findings of the DSA.

1.2. Limitations

- a) This report has been prepared by Eliot Sinclair & Partners at the request of our client and is exclusively for our client's use for the purpose for which it is intended in accordance with the agreed scope of work. Eliot Sinclair & Partners accepts no responsibility or liability to any third party for any loss or damage whatsoever arising out of the use of or reliance on this report by that party or any party other than our client.
- b) The inspections of the building discussed in this report have been restricted to those required to assist in the structural assessment of the building structure for seismic loads only. This assessment does not consider gravity or wind loading or cover building services or fire safety systems, or the building finishes, glazing system or the weather tightness envelope.
- c) Eliot Sinclair & Partners have not undertaken an assessment of the ducting, services and plant. We have also not checked whether tall or heavy furniture has been seismically restrained or not. These issues are outside the scope of this assessment but could be the subject of further investigation.

- d) Unless otherwise noted within this report, no geotechnical, subsurface or slope stability assessments have been undertaken.
- e) Eliot Sinclair & Partners is not able to give any warranty or guarantee that all possible damage, defects, conditions or qualities have been identified. The work done by Eliot Sinclair & Partners and the advice given is therefore on a reasonable endeavours basis.
- f) The assessment is based on various assumptions as outlined in Section 5 of this report.
- g) Eliot Sinclair & Partners has not considered any environmental matters and accepts no liability, whether in contract, tort, or otherwise for any environmental issues.
- h) The basis of Eliot Sinclair & Partners advice and our responsibility to our client is set out above and in the terms of engagement with our client.

2. Property Description

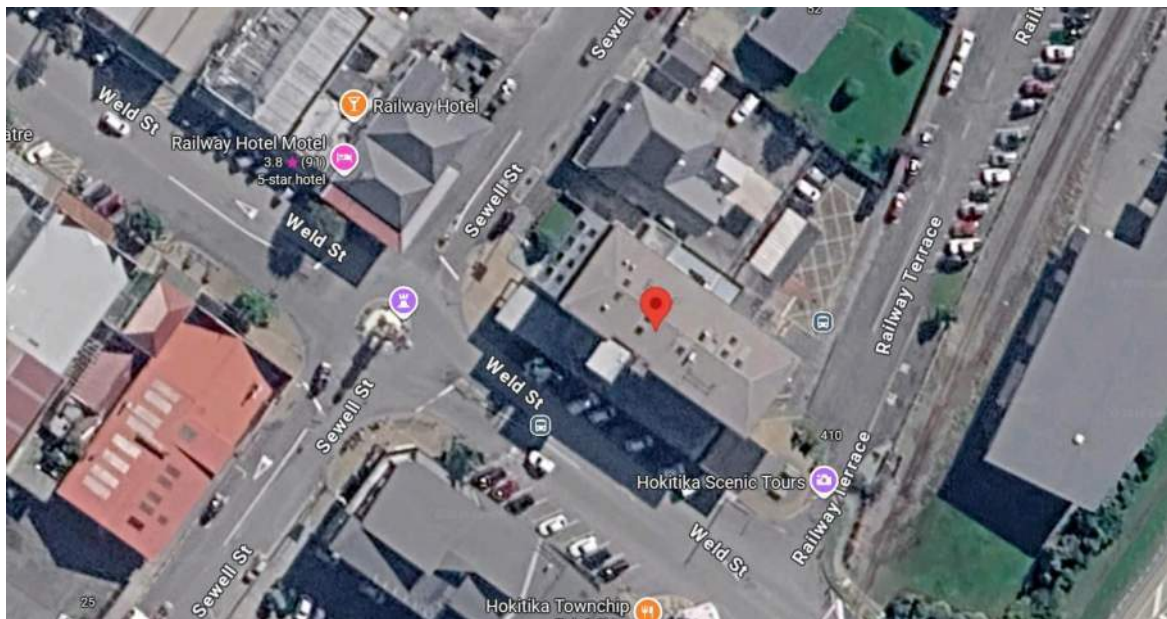


Figure 2. Site plan of the building at 36 Weld street. (Source Google maps)

2.1. Site Description

2.2. Building Description

The original HQ Building was designed by the Ministry of works and was constructed circa 1948 as the Hokitika Post Office. With a footprint of approximately 610m² and a total floor level of 1220m² over two levels, the building was a reinforced concrete structure comprising a mixture of punctured shear walls, concrete floor and gravity structure and a concrete roof.

The building was altered and extended in the early 1990s to accommodate the conversion to the Hokitika Council Building (HQ Building). This conversion added approx. 170m² to the southern side at ground floor, 63m² to the northern side at ground level and a new flat structural steel and timber framed butynol lined roof was constructed over the entire existing roof deck to create a new ~575m²

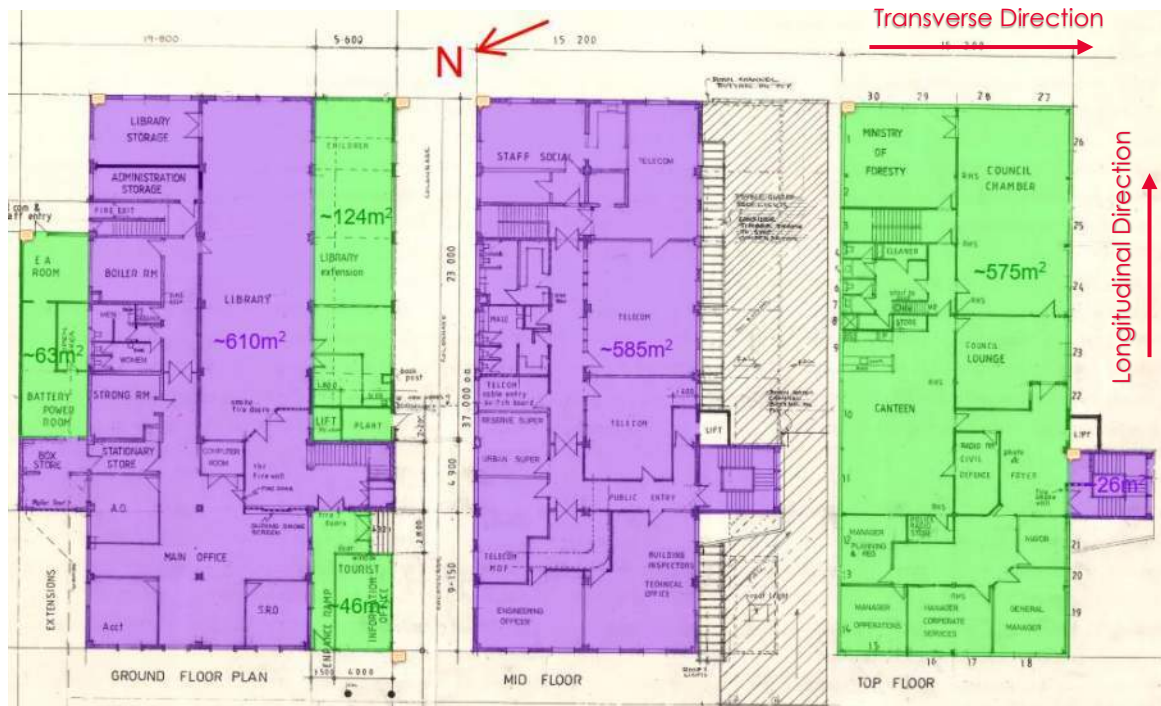


Figure 3. Floor plan of the buildings after alterations and additions in circa 1990. Additions (green) to the original building (purple)

third floor. At some later stage (exact age unknown), the flat butynol roof was over roofed with a steel farmed metal clad pitched roof.

2.3. Gravity Structure

Based on our review of the available documentation and a site visit to inspect the visible structure, we understand that the primary gravity load-resisting systems for this building comprise:

- **Roof:** Profiled metal roofing supported on cold-formed steel purlins, which span longitudinally between the top chords of transverse steel portal trusses located on grids 1-9. The ceiling structure consists of timber joists spanning longitudinally between the bottom chords of the transverse portal trusses. These timber joists support plasterboard ceiling linings on the underside and a previous ply-lined, butynol-clad roofing above. The portal columns rest on the concrete floor directly over the main reinforced gravity columns below. A central SHS column also props the portal truss at midspan.
- **Suspended floors:** The suspended floors comprise a 152mm thick slab on the first floor and a 127mm thick slab on the second floor. The second-floor slab originally formed the roof of the building before the addition of the top floor in the 1990s. These concrete floors are two-way slabs supported by perimeter concrete walls and internal reinforced concrete beams, columns, or walls.
- **Walls:** The exterior of the building consists of 203mm thick reinforced in situ concrete walls, heavily punctured to accommodate windows and doors. Additionally, there are 6, 8, and 10-inch reinforced in situ concrete walls separating stairwells, strong rooms, and general circulation and amenity areas. These walls support a tributary area of the reinforced concrete floor slab system.
- **Ground floor:** The ground floor is a 127mm thick reinforced in situ concrete slab. It is unclear whether this slab is supported on grade or intended to span as a suspended slab following the settlement of the fill layer below the building. Refer to Section 5 for further discussion.
- **Foundations:** The foundations comprise a grid of 914mm deep by 305mm wide reinforced in situ concrete ground beams, primarily located on grid lines, which span between 406mm reinforced

in situ concrete piles. These piles are supported on 660mm thick foundation pads founded approximately 3.6 meters below the ground floor slab level.

2.4. Seismic Lateral Structure

Based on our review of the available documentation and a site visit to inspect the visible structure, we understand that the primary lateral load resisting system for this building comprises:

Second Floor

- **Longitudinal:** At the upper pitched-roof level, longitudinal seismic actions from the roof are collected by the tension-only roof bracing and transferred to the plywood- and plasterboard-lined ceiling diaphragm. The ceiling diaphragm then distributes these actions to the supporting steel braced frames and plasterboard-lined timber-framed walls.

At the lower butynol flat-roof level, the seismic inertial forces generated by the seismic weight of the ceiling system and the lightweight steel- and timber-framed walls are transferred through the plywood- and plasterboard-lined ceiling diaphragm to the supporting plasterboard-lined timber-framed walls and steel braced frames.

- **Transverse:** Roof bracing and purlins transfers the seismic loads from the roof to the in-plane portal frames. Seismic loads from the ceiling system and plasterboard lined timber framed walls are transferred via diaphragm action to the in plane loaded portal frames.

First Floor & Ground Floor

- **Longitudinal & Transverse:** Seismic loads associated with these floors and the second floor above are transferred through diaphragm action in the reinforced in-situ concrete floors to the in-plane loaded reinforced in situ concrete shear walls, located both internally and externally to the floor plate. These shear walls then transfer the load to the foundations via flexure and shear action.

3. Building Investigations

3.1. Document Review

The following documents were reviewed in addition to undertaking site inspections to gain an understanding of the building design and construction to assist with locating areas of potential weaknesses:

- i) Only selected sheets were available from the original building drawings circa 1947
- ii) Single sheet showing plan of the additions and alterations carried out in the 1990s.
- iii) DSA by Simco Consulting limited dated November 2020
- iv) Drawings of the as-built structure by Simco Consulting Limited.

3.2. Site Inspections

The building was inspected by Eliot Sinclair & Partners in September 2021 and April 2024. The following summary defines the scope of the observations undertaken:

- a) Visual observations of the building exterior wall elevations undertaken from ground level.
- b) Visual observation of interior walls, floors and ceilings throughout the building. Wall, floor and ceiling linings were not specifically removed expect for: the area around the portal frame legs at the top storey where a localised area of the wall lining was removed to observe portal base connections to the concrete.
- c) Visual observation within the roof space of third storey.

- d) Localised removal of wall linings in the vicinity of the steel rafter connections to the 1990s South Library Extension at ground-floor level.
- e) Visual observations of the paving and ground around the buildings for indication of ground movement, lateral spread, and liquefaction.
- f) A site-specific geotechnical investigation was carried out by Eliot Sinclair and report Issued dated 26 June 2024 (job reference #503048).
- g) The foundations were viewed from the exterior only. No direct access or specific detailed or intrusive investigation was carried out.
- h) Determine basic internal geometric measurements such as building heights and lengths at key locations as deemed necessary at the time of the site visit.
- i) Other than as noted above, no intrusive site investigations were undertaken.

3.3. Observations

Table 1. Photos which record examples of typical condition observed to the building.



Photo 1 – Exterior view to southern elevation of building.



Photo 2 – Internal view to library extension to south end circa 1990



Photo 3 – Internal view (southern staircase)



Photo 4 – internal view to portal knee connection at second floor ceiling level



Photo 5 – Internal view of main portal rafter at roof level.



Photo 6 – Internal view of roof valley beam connections

4. Geotechnical considerations

A geotechnical investigation of the site was undertaken on 22 and 23 April 2024 which included shallow hand augers and dynamic cone penetrometer (DCP) tests, and deep CPT testing. All onsite investigations were undertaken by Canterbury Geotest Ltd under the supervision of Eliot Sinclair. The geotechnical report was issued dated 26 June 2024.

4.1. Subsoil description

The site subsoil category is Class D – Deep or soft soil sites.

The soil profile at the site consists of silts overlaying sands and then gravels. The pad footings are assumed to be founded on the sand layer at approximately 2.8m bgl. The thickness of the sand layer beneath the pad footings increases from approximately 0.6m at the western end, to 2.6m at the eastern end. Liquefaction is not expected to occur during a design SLS seismic event, with liquefaction of some lenses within the silt and sand layers occurring during a design ULS event. As a result of greater thickness of sand below the eastern end of the building, there is the potential for differential settlement to occur in a post-earthquake liquefied state.

4.2. Foundations

The foundations comprise a 127mm reinforced-concrete slab with 914mm x 305mm deep reinforced-concrete beams arranged in a rectangular grid. The beams are connected to reinforced-concrete column piles, which extend approximately 3.0 m into square bearing pads measuring 2.08 m by 2.08 m and 0.66 m deep.

5. Seismic Assessments

The methodology adopted for the detailed seismic assessment of this building are generally as outlined in the EQ-Assess Guidelines.

5.1. Quantitative Assessment Methodology

Our methodology is briefly summarised below:

- a) Review of documents available to us, as outlined in Section 3.1 of this report, to identify the main structural elements and any apparent “structural weaknesses” that may significantly reduce the seismic performance of the building.
- b) Visual inspection of key elements of the building.
- c) Analysis methodology - An ETABS model was developed with the column piers extended to the foundation pads below ground level. Lateral restraint was represented using soil springs assigned at 0.5 m depth increments, with stiffness parameters adopted from the geotechnical report. A sensitivity analysis was undertaken by varying the soil spring stiffness values between 50% and 200% of the adopted values. The most adverse response was used for the design assessment.

The global structure was analysed using the equivalent static seismic analysis procedure in accordance with NZS 1170.5. Seismic weights were calculated at each level based on the applicable dead and live loads in accordance with NZS 1170.1.

The structure comprises the primary building levels and a lightweight structure at the uppermost level, Level 3. The roof structure was modelled as part of the overall building for gravity loading, seismic mass, diaphragm action and load-path purposes. The seismic mass of the roof was retained at its actual elevation.

The additional top force normally generated automatically by ETABS was not applied at the roof level. This reflects the adopted design basis that the additional 8% top force is not required where

the uppermost level comprises a lightweight structure. The roof was instead assessed independently using the parts loading procedure in accordance with NZS 1170.5.

For the global structure analysis, Level 2 was treated as the uppermost storey for the purposes of determining seismic shear. Accordingly, the additional 8% storey shear component was applied at Level 2, in addition to the seismic shear generated by the seismic mass at that level under the equivalent static analysis procedure.

- d) Hand analysis of selected critical elements of the building to determine the likely failure mechanisms of these subassemblies, and the whole building.
- e) Determination of the likely seismic capacity of the building compared with an equivalent new building at the site based on our inspections, any structural weaknesses identified, our calculations, and our engineering judgment.

5.2. Assessment Parameters (DSA)

5.2.1. Material Design Standards

Various aspects of the following New Zealand Building Code compliance documents have been used in conjunction the EQ-Assess Guidelines to assist with the assessment of the seismic capacity of the building:

- New Zealand Loadings Standards - NZS1170(set).
- New Zealand Timber Structures Standard – NZS3603:1993.
- New Zealand Timber Framed Buildings – NZS3604:2011.
- New Zealand Steel Structures Standard NZS 3404:1997.
- New Zealand Concrete Structures standard NZS 3101:2006.
- EQ-Assess guidance modules.

5.2.2. Assessment Load Parameters

General

For the purposes of consideration of loading, this structure is Importance Level 2 in accordance with AS/NZS 1170.0:2002.

Permanent Loads

Building self-weight = calculated for each element

Imposed Loads

Roof = 0.25kPa, $\psi_e = 0.0$ – roof

Floors = 3.0kPa, $\psi_e = 0.3$ – Floors

Seismic Loads: Ultimate limit State

Site subsoil category = D – In accordance with Geotechnical report.

Hazard Factor 'Z' = 0.45, (Location: Hokitika)

Return Period Factor = 1.0 (1/500year earthquake)

Near fault factor = 1.0

Assumed structural ductility =

- $\mu = 3.0$ for timber framed buildings as per EQ-Assess guidance module C9 (paragraph C9.4.2).
- $\mu = 1.25$ for steel structures.
- $\mu = 1.0-2.0$ for concrete frames and shear walls

Exclusions

Other loadings, including wind, snow and serviceability limit state earthquake loads have not been considered as part of this seismic assessment.

Probable Material Strengths

In accordance with the EQ-Assess Guidelines, the seismic capacity of the existing building elements have been assessed using probable material strengths and reduced strength reduction factors. These are as follows:

- Timber – Materials as per SESOC EQ assessment guidance tables C9.2 and C9.3.
- Steel – $F_{y\text{Prob}} = 270\text{MPa}$ as per SESOC EQ assessment guidance tables C6B.2.
- Concrete - $f_c = 30\text{MPa}$ as per SESOC EQ assessment guidance tables C5.3.

General Assumptions

The results of the Detailed Seismic Assessment are reported as a %NBS. The %NBS value contains uncertainty due to the assumptions and simplifications which are made during the assessment. The primary assumptions include, but are not limited to:

- i) The existing construction information supplied is an accurate record of the building. The information used to undertake the seismic assessment is listed in Section 3.
- ii) Simplifications made in the analysis, including boundary conditions such as foundation fixity.
- iii) Assessments of material strengths based on limited drawings, specifications and site inspections.
- iv) The normal variation in material properties which change from batch to batch.
- v) Approximations made in the assessment of the capacity of each element, especially when considering the post-yield behaviour.
- vi) Only a sample of the drawings was available from the council records, and detailed information for all building elements was not available. For the purposes of this assessment, it has been assumed that the remaining parts of the building for which information was unavailable were detailed similarly to those for which information was available.
- vii) The capacity of the portal frame leg hold downs at the second floor was estimated based on an equivalent cast in situ anchor. The anchor type and installation method have not been confirmed, making it difficult to reliably estimate the connection capacity. More detailed investigation would be required to confirm this.
- viii) The library extension to the south had new rafters installed and connected to the existing reinforced concrete columns. The connection could not be observed and would need further investigation to confirm the capacity of this connection.
- ix) The second-floor wall steel cross bracing element. It has been assumed these elements are 75 x 10mm steel plate elements, Grade 250 MPa.
- x) No information was available on the lift shaft construction. For seismic weights, it has been assumed as concrete however more details would be required on the lift shaft construction.

- xi) We did not have any available documentation specifying the in-plane timber framed bracing elements for the third storey. As such we have assumed a bracing capacity of 2.1 kN/m for plasterboard fixed to both sides of the wall in accordance with part C9 of the EQ-Assess Guideline documentation which, in this case, is equivalent to a GIB GS2-nom bracing element

5.2.3. Structural Weaknesses

A Structural Weakness is an aspect of the building structure and/or the foundation soils that score less than 100% New Building Standard (%NBS). The Detailed Seismic Assessment identified structural weaknesses in most areas of the building.

5.2.4. Critical Structural Weakness

A critical structural weakness is the lowest scoring structural weakness determined from a DSA. The flexural capacity of the concrete foundation piles in the transverse direction along the East and West elevations were determined as the Critical Structural Weakness (CSW).

5.2.5. Severe Structural Weaknesses

A Severe Structural Weakness (SSW) is a defined structural weakness that is potentially associated with catastrophic collapse and for which the capacity may not be reliably assessed based on current knowledge. Aspects that must be assessed as SSWs in a DSA have been predetermined and are listed in Section C1.5.3 of the EQ Assess guidelines. No SSWs have been identified for this building.

5.2.6. Seismic Assessment Results

The results of the Detailed Seismic Assessment are summarised in Table 2 below. Note that the values given represent the worst performing elements in the building, as these effectively define the building's capacity. Other elements within the building may have significantly greater capacity when compared with the governing elements.

Table 2. Seismic Capacity of Primary Structural Elements (%NBS)

Structural Element	Load Direction	%NBS	Limiting component
Second Floor Portal Frame In-plane Grids 1-9	Across	25%	Portal Leg hold down connections
Second floor Plasterboard Bracing In- plane	Along	25%	Bracing capacity of plasterboard walls & Steel flat bracing
First Floor West Shear wall Grid 1	Across	60%	Flexural Capacity of Shear wall
First floor Middle Shear Wall along the Corridor Grid B	Along	40%	Flexural Capacity of Shear wall
Ground Floor Boiler Room Shear Wall Grid 7	Across	30%	Flexural Capacity of Shear wall
Ground Floor South Shear Wall Grid C	Along	55%	Flexural Capacity of Shear wall
Foundations Concrete Column Piles East & West End Column at intersection of Grids 9&B	Across	25%	Flexural capacity of concrete column
Foundations Concrete Column Piles North End Column at intersection of Grids 8&C	Along	35%	Flexural capacity of concrete column

Table 2 indicates that the overall seismic rating for the building is 25%NBS for an importance level 2 building as defined by the New Zealand Standard – Structural Design Actions AS/NZS1170.0:2002.

The seismic rating of the structure at 36 Weld Street, Hokitika is governed by the Flexural capacity of the columns piles foundation in the transverse direction.

We note that we did not have any available documentation specifying the in-plane timber framed bracing elements for the third storey. As such we have assumed a bracing capacity of 2.0 kN/m for plasterboard fixed to both sides of the wall in accordance with part C9 of the EQ-Assess Guideline documentation which, in this case, is approximately equivalent to a GIB GS1-nom bracing element.

6. Seismic Grades & Relative Risk

For this assessment, the building's earthquake resistance is expressed as a "Percentage of New Building Standard" (%NBS). The %NBS seismic rating is intended to provide a measure of the ultimate seismic structural capacity of a building relative to the minimum that would meet the current New Zealand Building Code requirements for a new building constructed on the same site

The following table by NZSEE provides the basis of a proposed grading system for existing buildings, as one way of interpreting the %NBS seismic rating. Table 3 taken from the EQ-Assess Guidelines provides the basis of a generally accepted grading system for existing buildings, as one way of interpreting the %NBS seismic rating.

Table 3. Relative Earthquake Risk

Percentage of New Building Standard (%NBS)	Alpha rating	Approximate risk relative to a new building	Life-safety Risk Description
>100	A+	Less than or comparable to	Low risk
80-100	A	1-2 times greater	Low risk
67-79	B	2-5 times greater	Low or Medium risk
34-66	C	5-10 times greater	Medium risk
20-33	D	10-25 times greater	High risk
<20	E	25 times greater	Very High risk

Table 3 shows that occupants of an Earthquake Prone building (%NBS less than 33%, Grade D and E) are exposed to up to more than 25 times the risk during an earthquake than that of occupants of a similar new building. For buildings that are potentially Medium Risk (67%>%NBS>34%), the risk is 5 to 10 times greater than that of an equivalent new building. Broad descriptions of the life-safety risk can be assigned to these building grades accordingly.

Based on the results of the seismic assessment, the building is categorized as a Grade D building following the NZSEE grading scheme. Grade E buildings represent a high risk to building occupants equivalent to 10-25 times that expected for a new building, indicating a high earthquake risk exposure.

A building with a seismic rating less than 34%NBS is considered to be Earthquake-Prone in terms of the Earthquake-Prone Buildings Amendment Act 2016. The building at 36 Weld Street, Hokitika, Christchurch is therefore categorized as Earthquake-Prone.

7. Conclusions

- Based on the seismic assessment, the building has a seismic rating of 25%NBS for an importance level 2 building as defined by NZS1170.0:2002.
- Based on this seismic rating, the building is therefore categorized as a Grade D building following the NZSEE grading scheme. The building is considered to be Earthquake Prone.
- Grade D buildings represent a high risk to building occupants equivalent to 10-25 times that expected for a new building, indicating a very high earthquake risk exposure.
- The seismic rating for the building is governed by different elements in each area. Refer to Table 2 for further detail.

- e) The decision for continued occupancy of the building remains with the owner and/or tenant of the building.
- f) Eliot Sinclair & Partners supports the recommendation of the NZSEE that it is desirable to seismically strengthen earthquake risk buildings to as near as reasonably practical to that of a new building but, as a minimum, seismic improvements should achieve at least 67%NBS.
- g) Site specific geotechnical investigation has been undertaken as part of this DSA as we consider the performance of the foundation to be one of the critical aspect governing the seismic capacity of the building. However, if seismic strengthening is to be undertaken, then further geotechnical testing should be carried out to confirm the quality of the existing fill material under the building ground floor slab.

8. Concept strengthening – Primary Structure

Following the above seismic assessment, we have prepared a seismic strengthening concept scheme to bring the building up to a target level of 34% NBS (IL 2). This section is to be read in conjunction with the mark-up plans in appendix C.

8.1. Purpose

The purpose of this concept strengthening design is to summarise the current structural concept design as developed to date for the seismic strengthening of the building at 36 Weld Street. This design can be used for continuing coordination purposes and for re-validating the project budget. However, noting the conceptual nature of the information provided, appropriate design and measurement contingencies need to be allowed for in any cost estimates. Any cost estimate should also make an allowance to complete further investigations as outlined in Section 8.4.

This design also records key assumptions and identifies structural related issues that are yet to be resolved.

This report should be read in conjunction with the site-specific Geotechnical Report, and Structural Drawings produced for this concept design.

8.2. Scope

The primary object of the proposed concept design is to improve the seismic strength of the building to a target level of 34%NBS based on an IL2 building in accordance with NZS1170.0:2002. The scope of the proposed improvement work is limited to structurally designed elements associated with the seismic strengthening of the building. The seismic assessment and strengthening does not consider wind or snow loading or cover building services or fire safety systems, or the building finishes, glazing systems or the weather tightness envelope.

The primary objective of the seismic strengthening is to reduce the life safety risk to the building occupants during an ultimate limit state earthquake to that associated with a seismic rating of 34%NBS. Table 3 indicates that a building which has been seismically strengthened to 34%NBS is a Grade C building following the NZSEE grading scheme. Grade C buildings represent a life safety risk to occupants of 5 to 10 times that expected for a new building, indicating a medium risk.

The proposed seismic strengthening work is not specifically intended to reduce the potential for damage to occur to the building during an earthquake. As such, damage is still expected to occur to the building during a significant earthquake.

The seismic strengthening work outlined in this report is proposed to be undertaken in accordance with Section 112 of the New Zealand Building Act 2004. That is, once the structural building works are

completed, the building structure will continue to comply with the requirements of Clause B1 of the New Zealand Building Code to at least the same extent it did prior to the work being undertaken.

To enhance the overall seismic capacity of the primary lateral structure, the following strengthening works are proposed:

8.3. Strengthening Works required to achieve 34% NBS(IL2)

8.3.1. Level 3 Roof Structure

The lateral strength of the existing roof structure relies on the strength and stiffness offered by the existing ply and plasterboard clad ceiling diaphragm to distribute loads to the following bracing elements:

- Strengthening of the existing portal column hold down fixings to the floor on grids A&C/1-9.
- Strengthening of the steel braced frame on grid B/6-8 and installation of new SHS braced frames to the grid A&C exterior walls.
- Strengthening of the existing rafter connection to the grid 3 & 4 concrete shear walls near grid C.

8.3.2. First Floor

- In selected locations along grids 3 & 4, install structural steel ties to strengthen the lateral connection between the existing shear walls and the floor diaphragms.
- Demolish and replace existing brick walls with timber framed walls.

8.3.3. Ground Floor

- Construct new reinforced concrete facing walls on the interior face of the existing Grid 7. This new reinforced wall will act compositely with the existing shear walls and could be constructed as boxed and poured in situ concrete walls or sprayed concrete walls.
- New SED ply Shear wall at the south end of the building to provide bracing to the library extension in the E-W direction.
- Investigate/strengthen rafter to RC column connections at grids C1 to C9 for the south library extension.
- In selected locations along grids 3 ,4 & 7, install structural steel ties to strengthen the lateral connection between the existing shear walls and the floor diaphragms.

8.3.4. Foundations

The building superstructure is supported on a series of reinforced concrete piles which project from the underside of the building down to pad foundations founded at approx. 3.6m below the ground floor. These piles exhibit relatively low transverse reinforcing content and are vulnerable to a brittle shear failure mechanism under significant seismic lateral loading. The brittle failure of these piles could compromise the gravity load support of the building.

To mitigate the risk to the support of the building, two strengthening options have been considered as described below. The suitability of each option is dependent on the quality of the ~3.0m thickness of existing fill material located below the building:

Option 1 – Foundation Pads: This option is deemed appropriate if the existing fill consists of relatively compact gravelly material. Should the existing piles experience lateral failure, the gravity load from the building's superstructure will be transferred from the piles to newly constructed reinforced concrete foundation pads, founded approximately 1.1 meters below the ground floor level. These new

foundations will bear directly on the existing fill layer. Although some vertical settlement is expected after the load is redistributed, the building's gravity load support will be maintained.

Option 2 – Foundation Pads & Piles: This option is deemed appropriate if the existing fill consists of relatively soft and uncompacted silt. In the event of lateral failure of the existing piles, the gravity load from the building's superstructure will be transferred from the existing piles to newly constructed reinforced concrete foundation pads, which will be supported by newly constructed reinforced concrete piles. These new piles will extend approximately 2.0 meters and bear directly on top of the original concrete foundation pads, ensuring the building's gravity load support is maintained.

8.4. Further investigations and information required.

The following section summarises the outstanding issues identified to date that need to be resolved as the design progresses through the developed and detailed design phases:

Confirmation of the connections between structural elements which includes the following:

- The anchor type and installation method for the portal frame hold downs to the concrete. More detailed investigation would be required to confirm this.
- Connection of the rafters to the reinforced concrete columns for the library extension to the south.
- Connection of the rafters to the reinforced concrete columns for the EA and Battery Power room extension to the North.
- Confirmation of the steel flat bracing connections at the second floor and post hold downs at the location of this bracing.
- Confirmation of the lift shaft construction material and its connection to the building.
- Investigation on the support structure and connections for canopy to the south end for lateral capacity.
- Undertake a regime of reinforcing scanning by a specialist contractor to verify the typical reinforcing layout of key structural shear walls throughout the building.

Confirmation of the proposed foundation strengthening concept requires the following additional geotechnical investigations:

- Investigate the existing fill material located below the ground floor slab to confirm its composition and compaction. This will necessitate cutting localized areas of the slab to enable hand auger and scala penetrometer testing of the fill in selected locations.
- Confirm the potential presence of a void space between the underside of the ground floor slab and the top of the fill during the fill investigations. If a void is confirmed, it will need to be locally grout-filled before foundation strengthening begins.
- Once the properties of the fill material are known, additional geotechnical analysis of the lateral & vertical capacity of the foundations will be required to validate the concept strengthening design.
- Coordination of the proposed strengthening work with the architect and services engineers is necessary to confirm the strengthening concept. For example, reliance on the existing interior timber-framed walls and ceiling/roof diaphragm at level 3 to brace the roof of the building has been noted. If alterations to the existing structure are required to facilitate any refit of these spaces, additional roof bracing may be needed.
- Early contractor involvement is recommended to critique the buildability of the proposed strengthening work.

Disclaimer

This report has been prepared by Eliot Sinclair & Partners Limited ("Eliot Sinclair") only for the intended purpose as a Detailed Seismic Assessment.

The report is based on:

- Desktop review of available building information and reports.
- On – site investigation of as-built structure.

Where data supplied by other external sources, including previous site investigation reports, have been relied upon, it has been assumed that the information is correct unless otherwise stated. No responsibility is accepted by Eliot Sinclair for incomplete or inaccurate data supplied by other parties.

Whilst every care has been taken during our investigation and interpretation of describe conditions e.g., subsurface conditions available data to ensure that the conclusions drawn, and the opinions and recommendations expressed are correct at the time of reporting, Eliot Sinclair has not performed an assessment of all possible conditions or circumstances that may exist at the site.

Variations in conditions may occur between investigatory locations and there may be conditions such as include examples e.g. subsoil strata and features that were not detected by the scope of the investigation that was carried out or have been covered over or obscured over time. Eliot Sinclair does not provide any warranty, either express or implied, that all conditions will conform exactly to the assessments contained in this report.

The exposure of conditions or materials that vary from those described in this report, or occurrence of additional strong seismicity, or any update to the Building Act, NZBC or MBIE's Guidance may require a review of our recommendations. Eliot Sinclair should be contacted to confirm the validity of this report should any of these occur.

This report has been prepared for the benefit of Westland District Council and the regulatory authority for the purposes as stated above. No liability is accepted by Eliot Sinclair or any of their employees with respect to the use of this report, in whole or in part, for any other purpose or by any other party.

Appendix A. DSA Summary Sheets

1. Building Information	
Building Name/ Description	WDC HQ Building The original HQ Building was designed by the Ministry of works and was constructed circa 1948 as the Hokitika Post Office. With a footprint of approximately 610m² and a total floor level of 1220m² over two levels, the building was a reinforced concrete structure comprising a mixture of punctured shear walls, concrete floor and gravity structure and a concrete roof. The building was altered and extended in the early 1990s to accommodate the conversion to the Hokitika Council Building (HQ Building). This conversion added approx. 170m² to the southern side at ground floor, 63m² to the northern side at ground level and an additional floor was constructed over the entire existing roof deck to create a new ~575m² third floor.
Street Address	36 Weld Street, Hokitika
Territorial Authority	Westland District Council
No. of Storeys	3
Area of Typical Floor (approx.)	Ground Floor: 843m² First Floor: 610m² Second Floor: 575m²
Year of Design (approx.)	Circa 1948 with additions/alterations in circa 1990.
NZ Standards designed to	Unknown.
Structural System including Foundations	Roof: Profiled metal roofing supported on cold-formed steel purlins, which span longitudinally between the top chords of transverse steel portal trusses located on grids 1-9. The ceiling structure consists of timber joists spanning longitudinally between the bottom chords of the transverse portal trusses. These timber joists support plasterboard ceiling linings on the underside and a previous ply-lined, butynol-clad roofing above. The portal columns rest on the concrete floor directly over the main reinforced gravity columns below. A central SHS column also props the portal truss at midspan. Suspended floors: The suspended floors comprise a 152mm thick slab on the first floor and a 127mm thick slab on the second floor. The second-floor slab originally formed the roof of the building before the addition of the top floor in the 1990s. These concrete floors are two-way slabs supported by perimeter concrete walls and internal reinforced concrete beams, columns, or walls. Walls: The exterior of the building consists of 203mm thick reinforced in situ concrete walls, heavily punctured to accommodate windows and doors. Additionally, there are 6, 8, and 10-inch reinforced in situ concrete walls separating stairwells, strong rooms, and general circulation and amenity areas. These walls support a tributary area of the reinforced concrete floor slab system. Ground floor: The ground floor is a 127mm thick reinforced in situ concrete slab. It is unclear whether this slab is supported on grade or intended to span as a suspended slab following the settlement of the fill layer below the building. Refer to Section 5 for further discussion. Foundations: The foundations comprise a grid of 914mm deep by 305mm wide reinforced in situ concrete ground beams, primarily located on grid lines, which span between 406mm reinforced in situ concrete piles. These piles are supported on 660mm thick foundation pads founded approximately 3.6 meters below the ground floor slab level.

Does the building comprise a shared structural form or shares structural elements with any other adjacent titles?	No.
Key features of ground profile and identified geohazards	Ground expected to be similar to TC2 category for liquefaction
Previous strengthening and/ or significant alteration	Additions and Alterations in circa 1990. Addition of a third storey office. South library extension and extension to the north end.
Heritage Issues/ Status	None Known.
Other Relevant Information	-
2. Assessment Information	
Consulting Practice	Eliot Sinclair & Partners
CPEng Responsible, including: - Name - CPEng number - A statement of suitable skills and experience in the seismic assessment of existing buildings¹	Sanjeev Raturi PGCertCivilEng(EQ) CPENG (1027800) Practice Field is Design, Assessment and Construction monitoring of low-rise structures. with 9 plus years' experience in seismic assessment & strengthening.
Documentation reviewed, including: - date/ version of drawings/ calculations² - previous seismic assessments	- Only selected sheets were available from the original building drawings circa 1947 - Single sheet showing plan of the additions and alterations carried out in the 1990s. - DSA by Simco Consulting limited dated November 2020 - Drawings of the as-built structure by Simco Consulting Limited.
Geotechnical Report(s)	Geotechnical report by Eliot Sinclair dated 26 June 2024 job ref # 503048.
Date(s) Building Inspected and extent of inspection	Visual inspection only. September 2021 and April 2024.
Description of any structural testing undertaken and results summary	None taken.
Previous Assessment Reports	DSA by Simco Consulting Limited dated 10 November 2020 Job Ref#20005 Assessed as being less than 34% NBS.

¹ This should include reference to the engineer's Practice Field being in Structural Engineering, and commentary on experience in seismic assessment and recent relevant training

² Or justification of assumptions if no drawings were able to be obtained

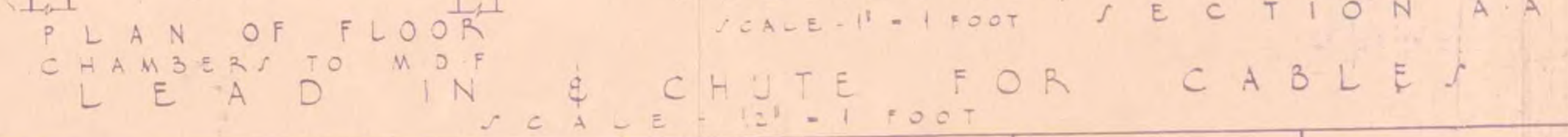
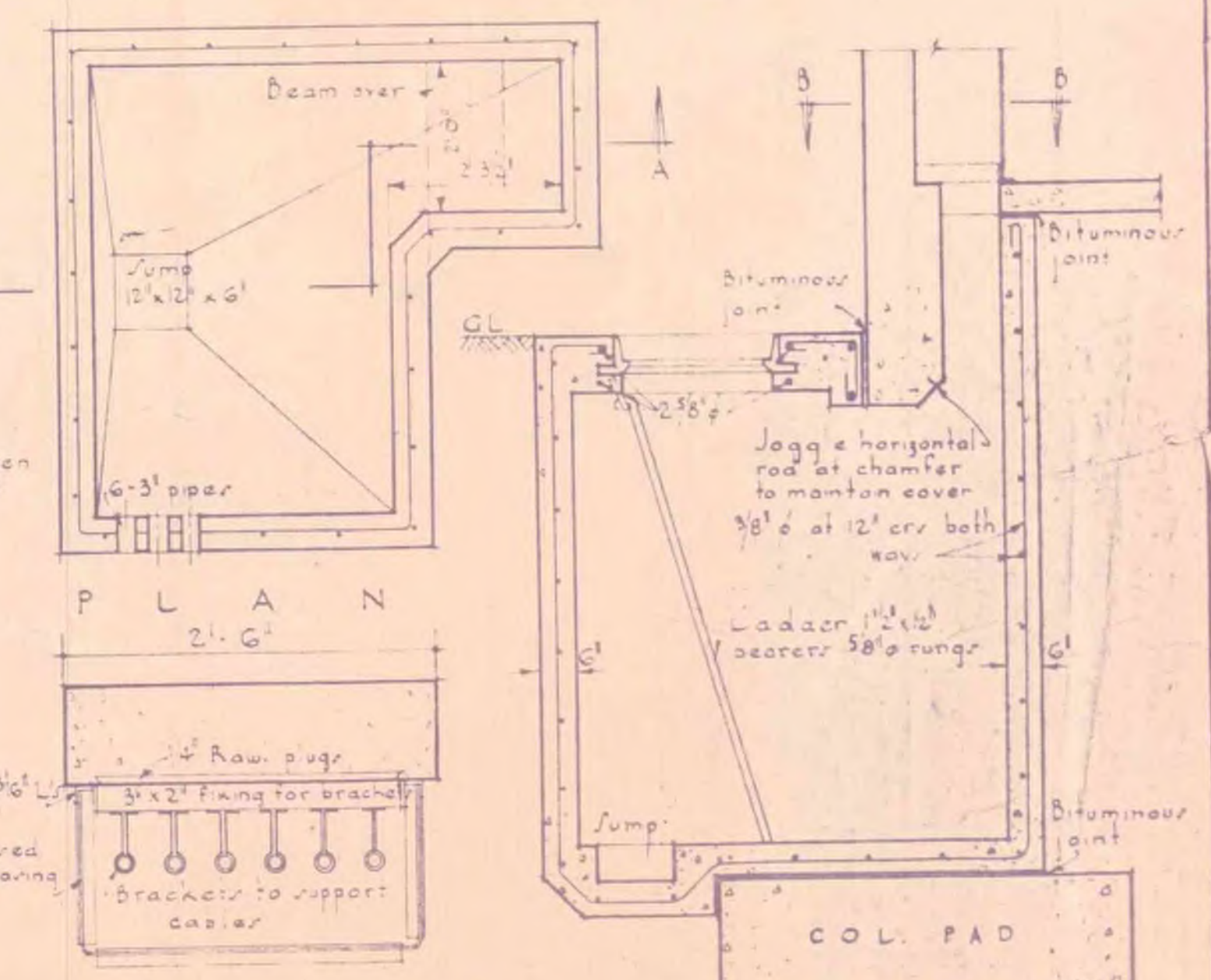
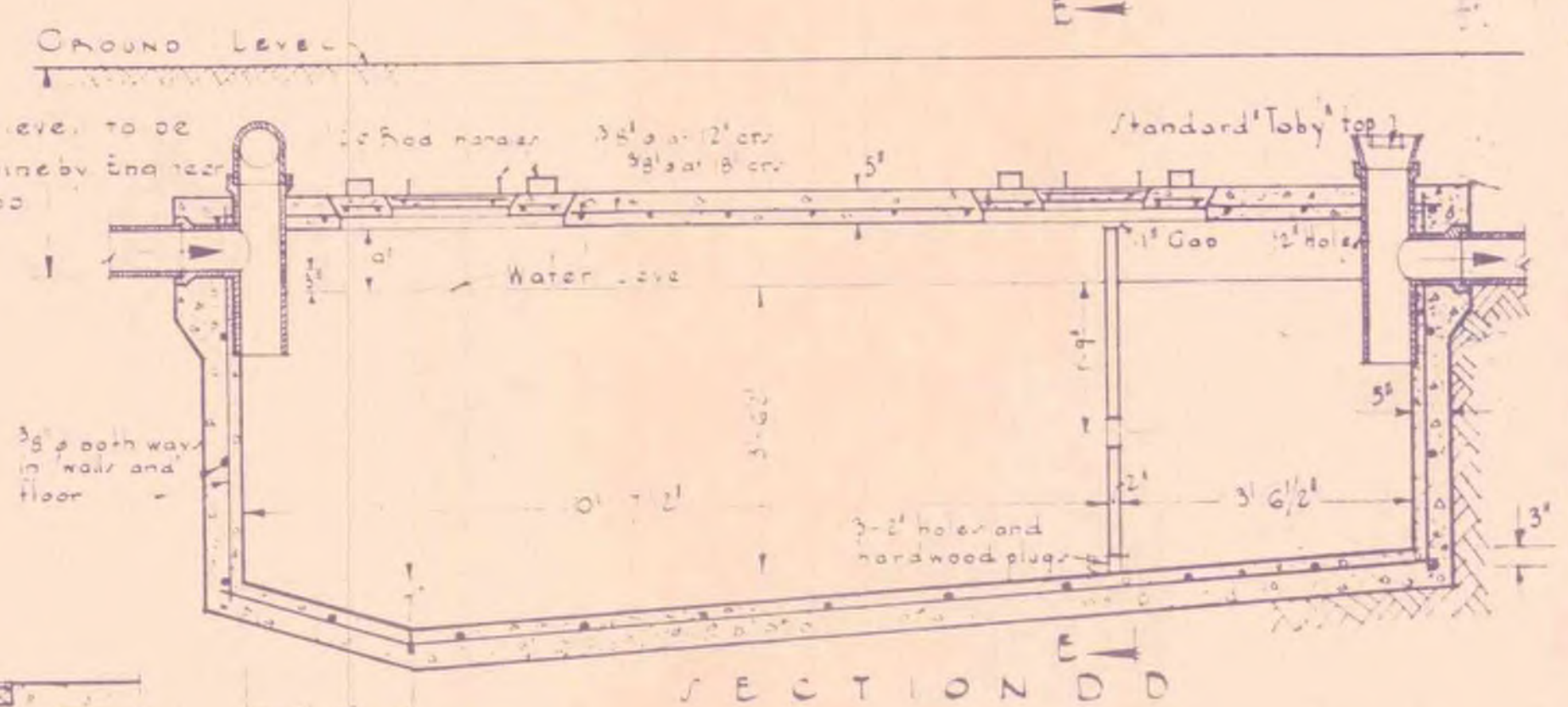
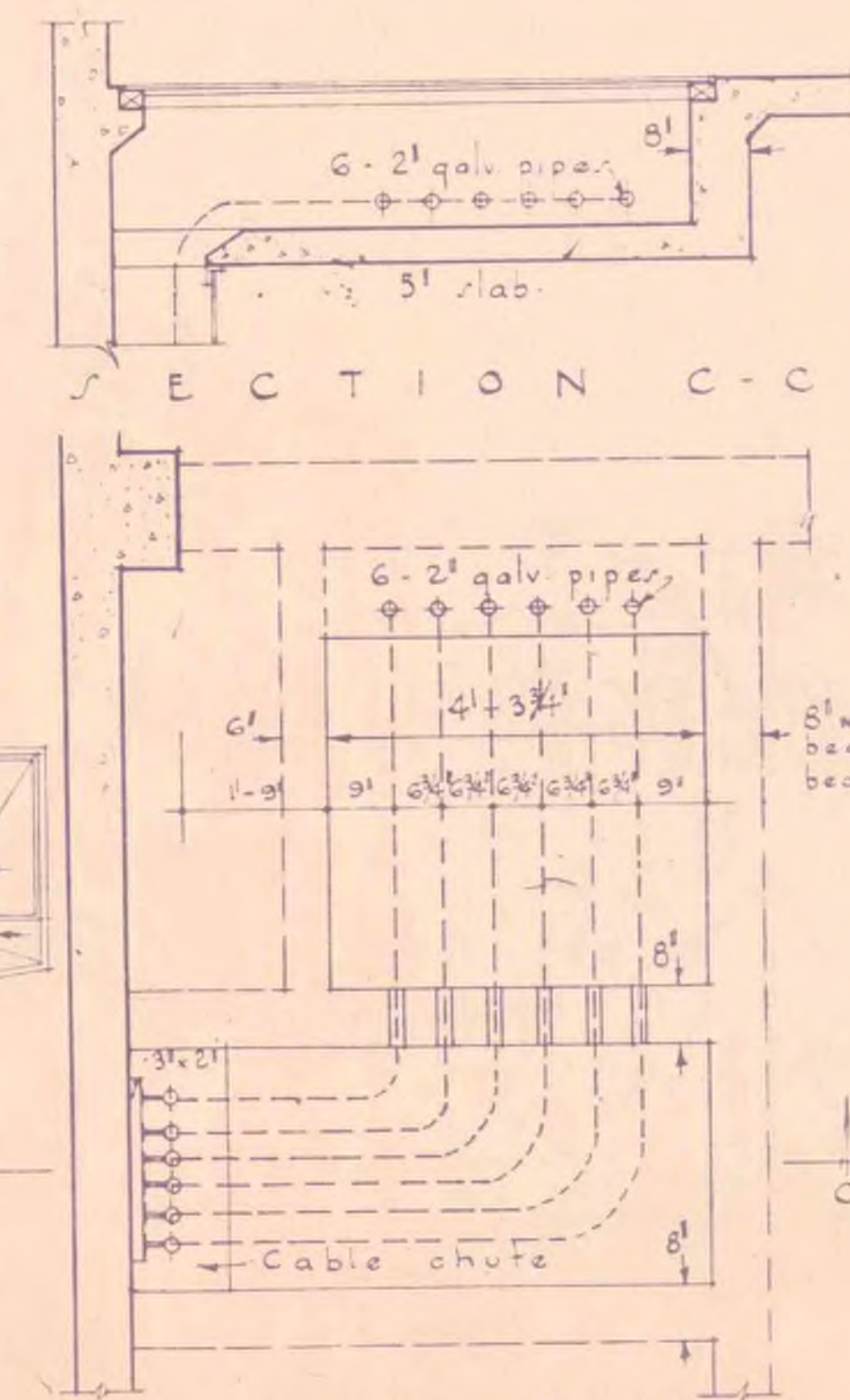
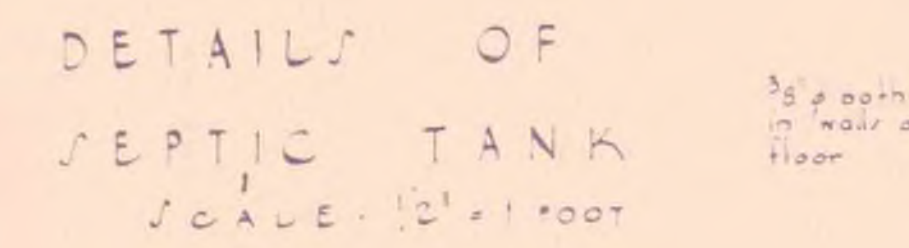
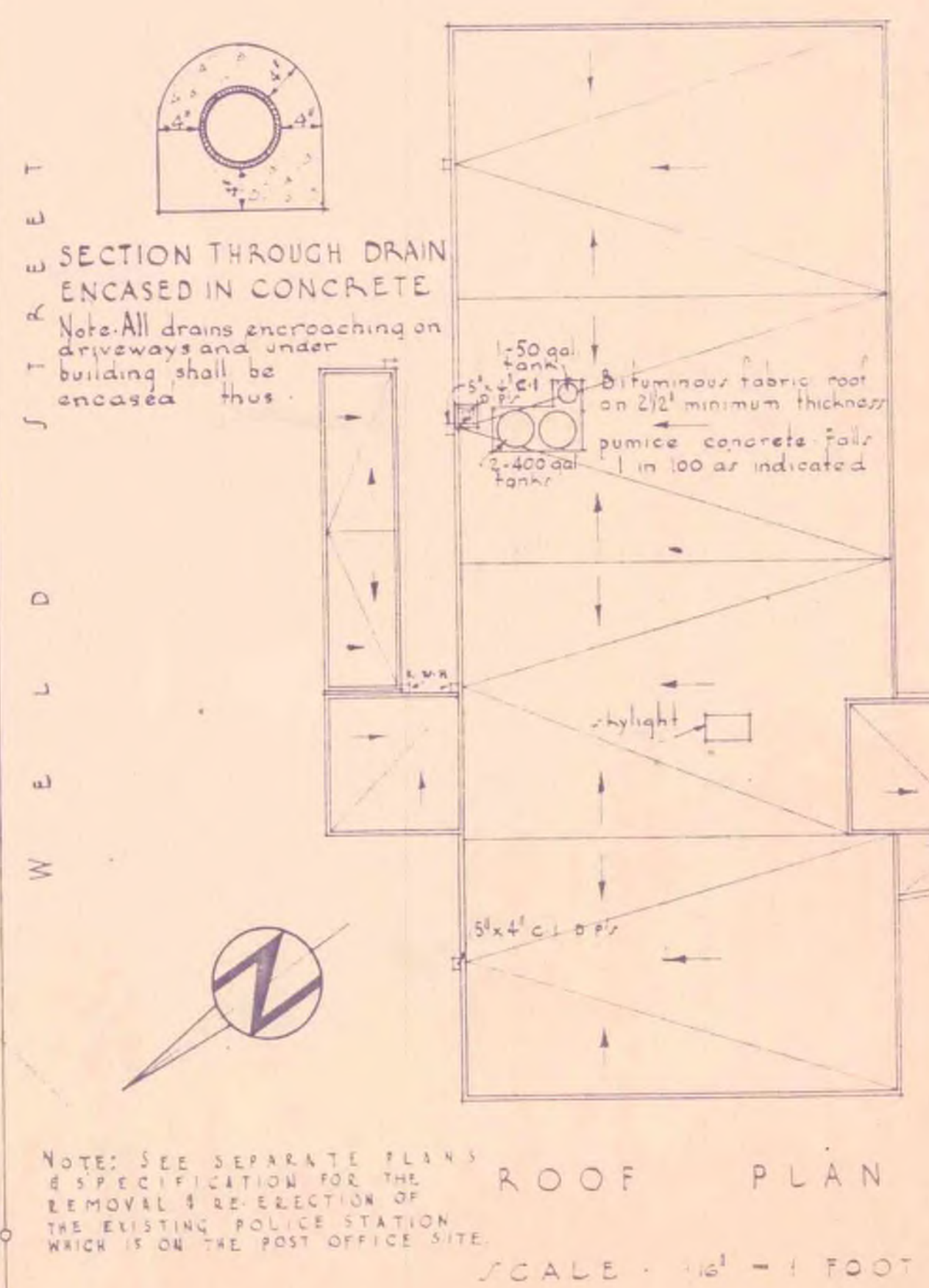
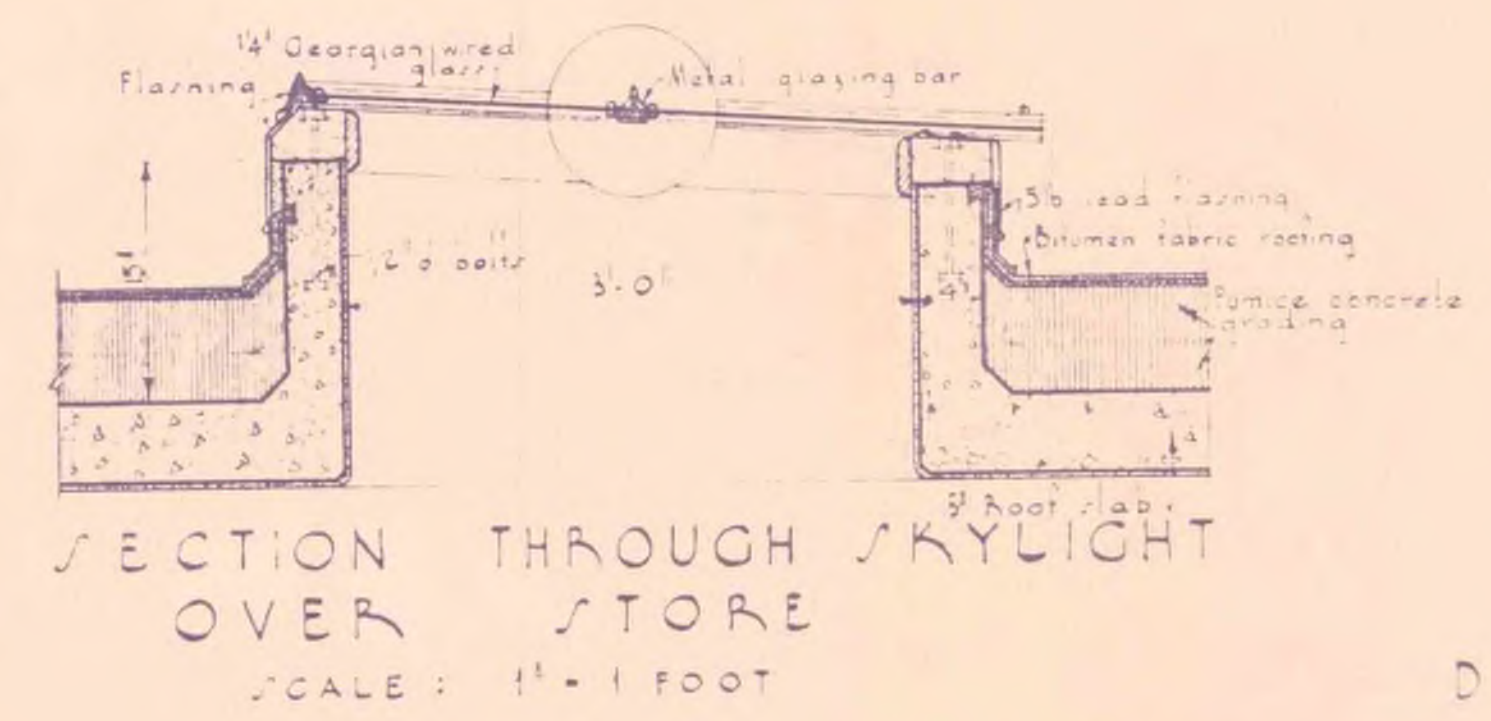
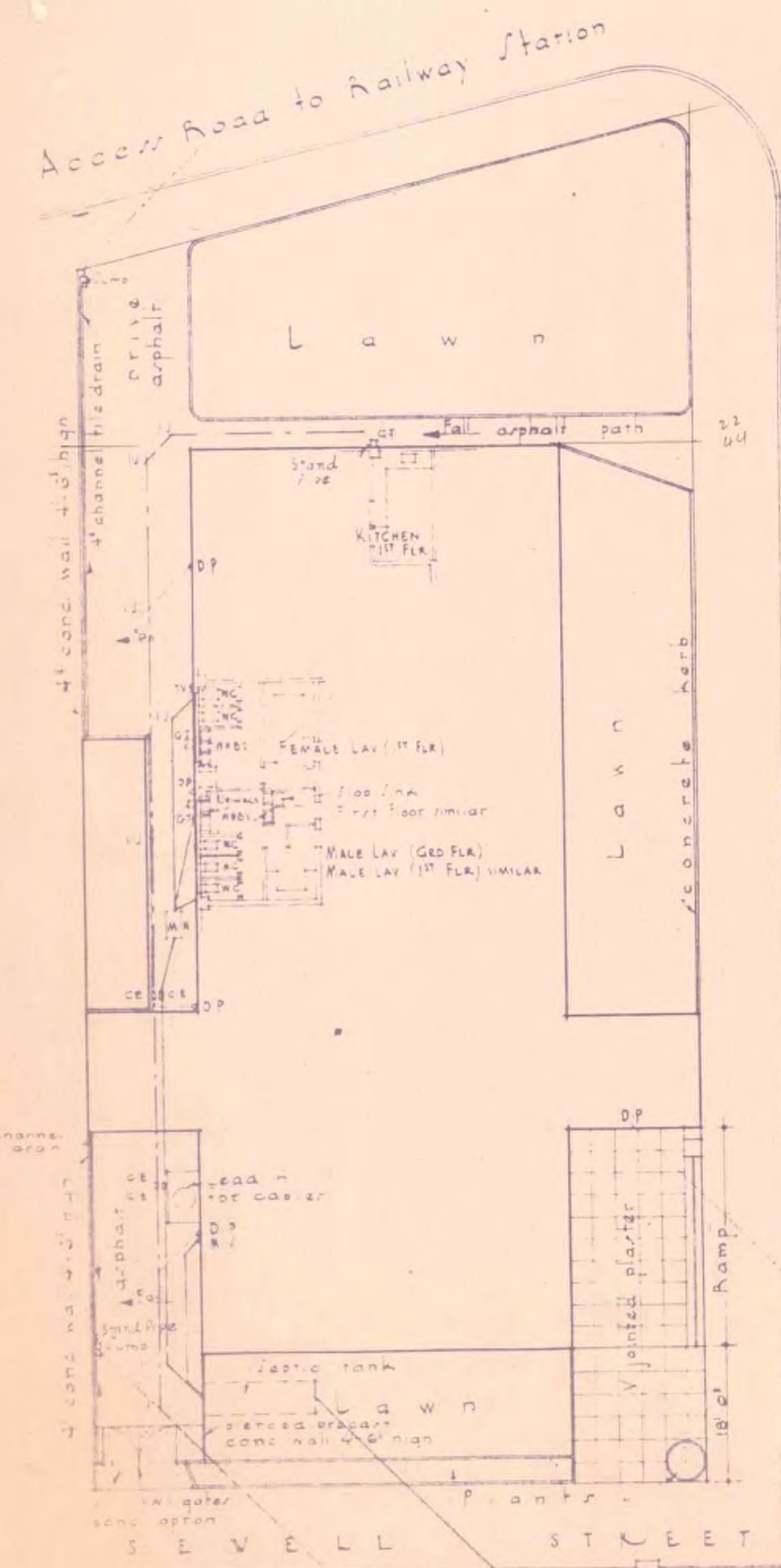
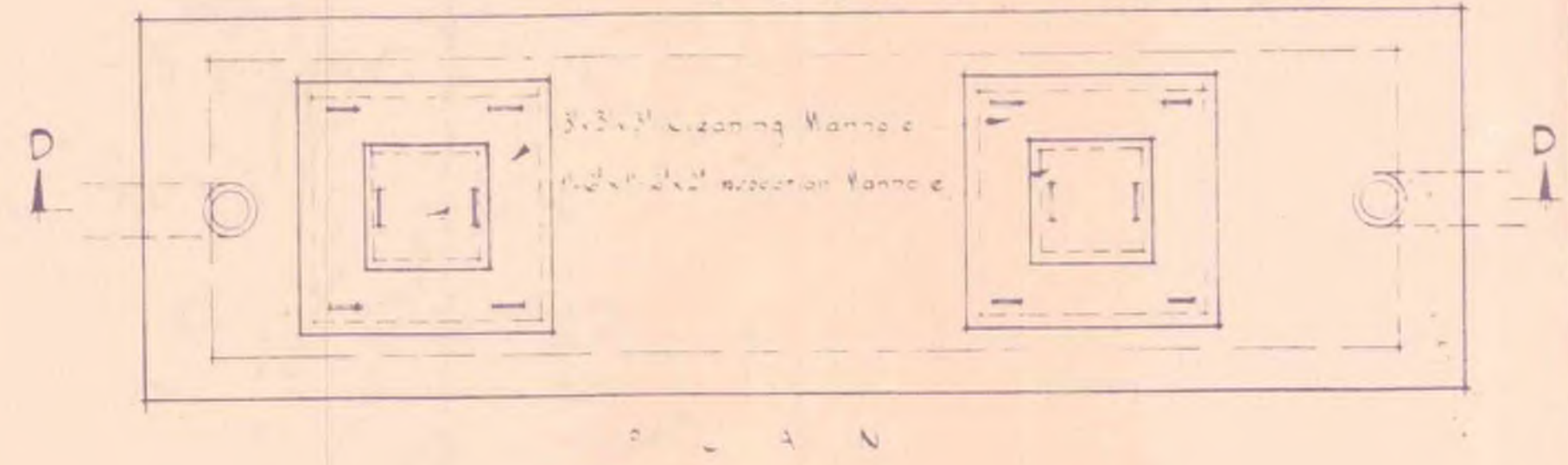
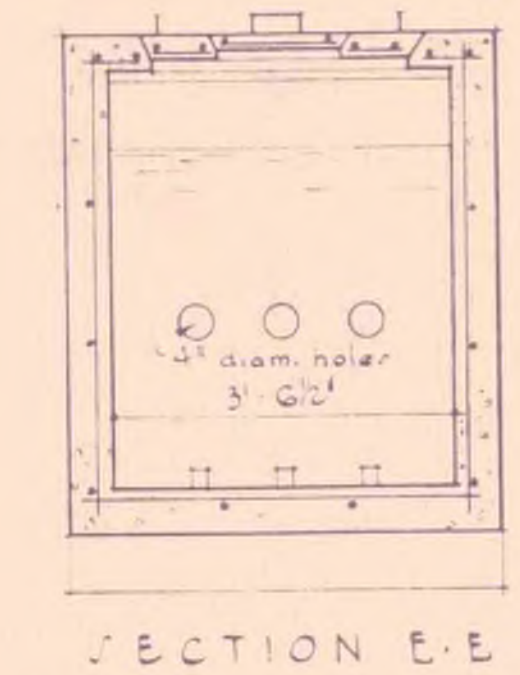
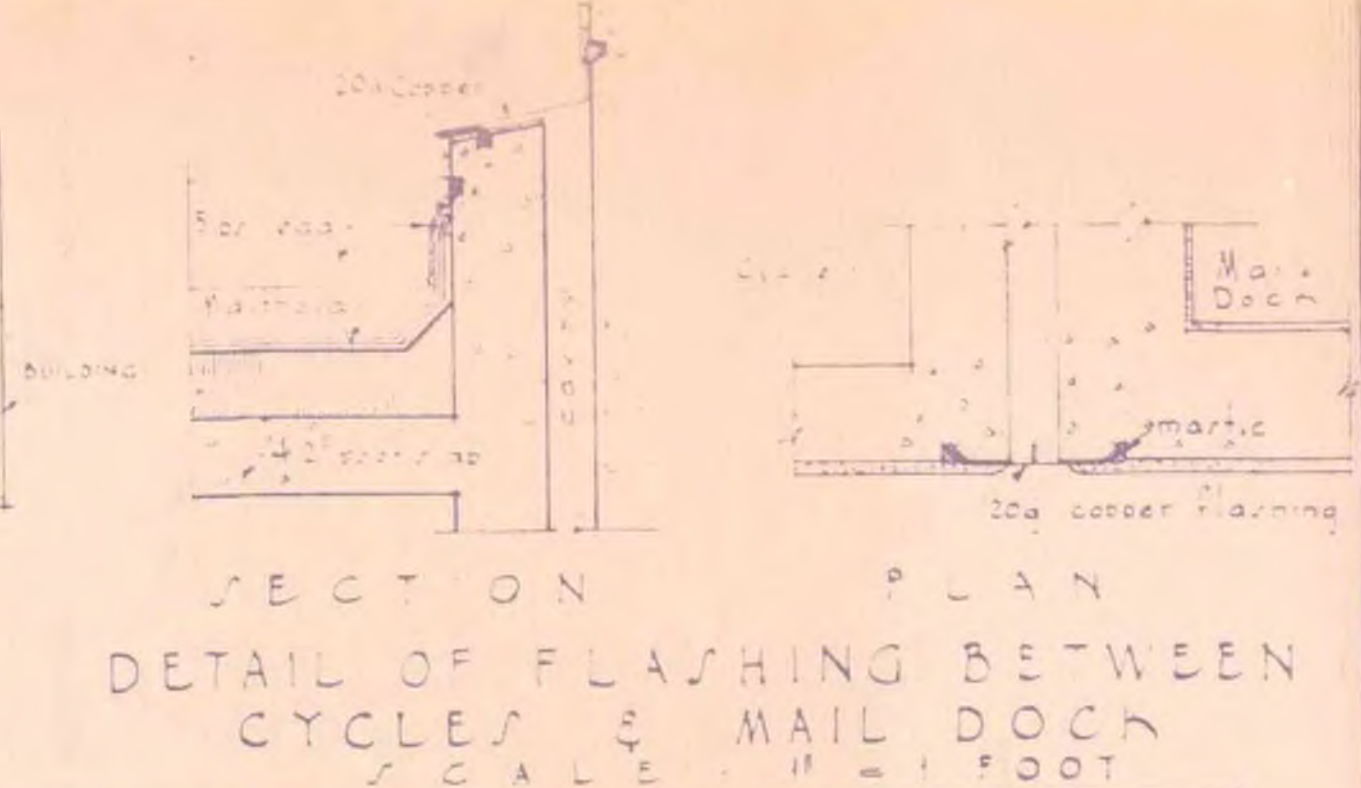
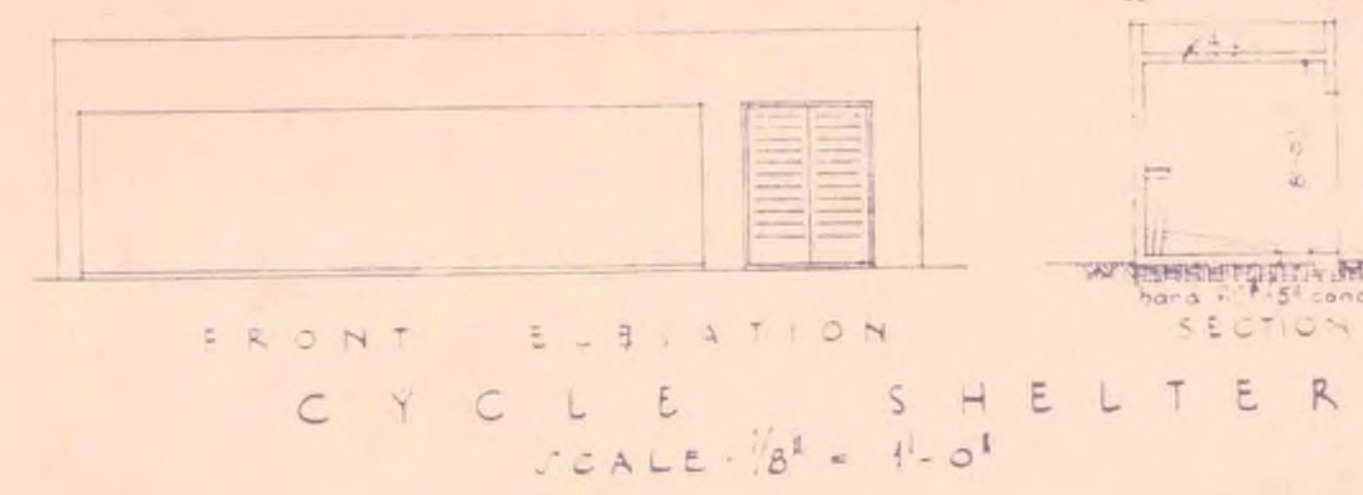
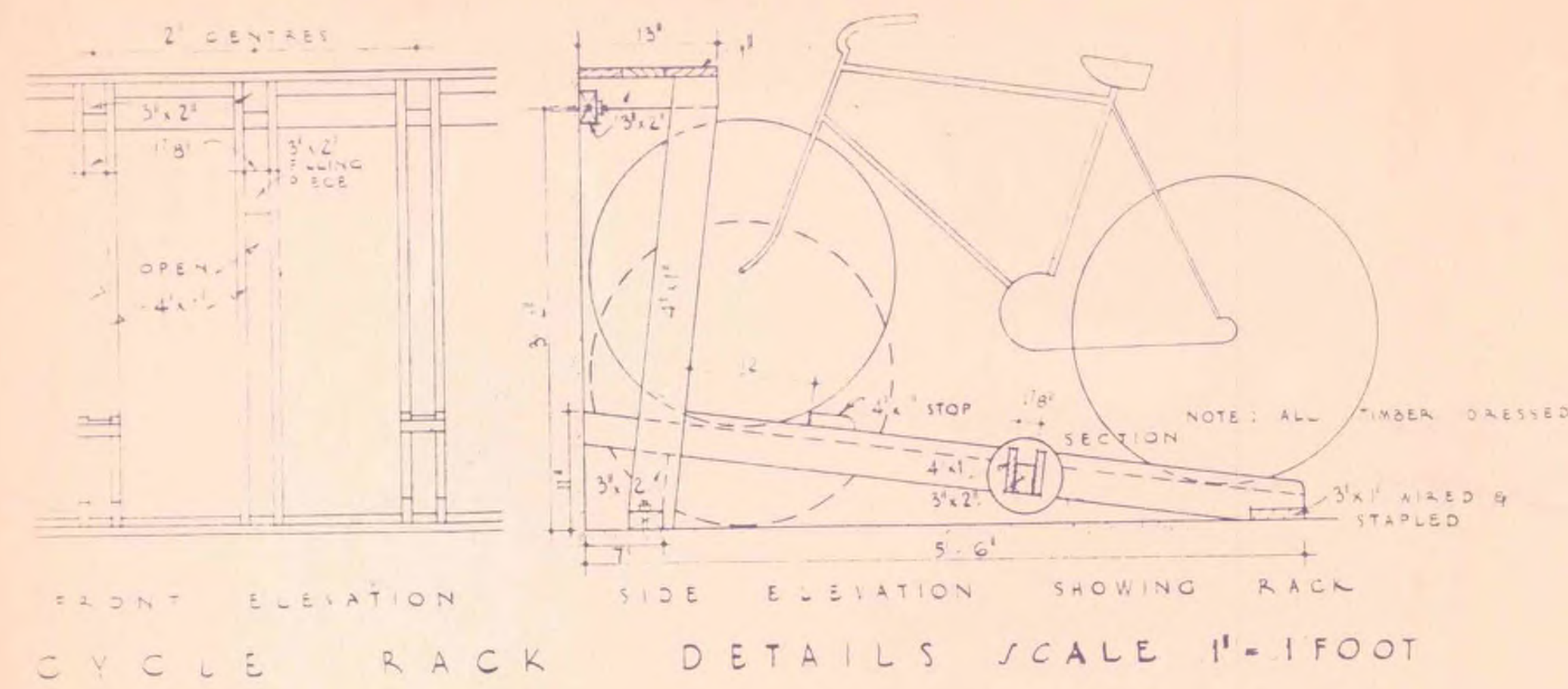
Other Information	Relevant -
----------------------	---------------

3. Summary of Engineering Assessment Methodology and Key Parameters Used	
Occupancy Type(s) and Importance Level	Office Building < 300 people – Importance level 2
Site Subsoil Class	Site subsoil class D as per Geotechnical report.
For an ISA:	
Summary of how Part B was applied, including: - Key parameters such as μ, S_p and F factors - Any supplementary specific calculations	N/A
For a DSA:	
Summary of how Part C was applied, including: - the analysis methodology(s) used from C2 - other sections of Part C applied	Elastic, force-based procedure. Modules C1, C2, C3, C4, C5, C6 & C9 An ETABS model was developed with the column piers extended to the foundation pads below ground level. Lateral restraint was represented using soil springs assigned at 0.5 m depth increments, with stiffness parameters adopted from the geotechnical report. A sensitivity analysis was undertaken by varying the soil spring stiffness values between 50% and 200% of the adopted values. The most adverse response was used for the design assessment. The global structure was analysed using the equivalent static seismic analysis procedure in accordance with NZS 1170.5. Seismic weights were calculated at each level based on the applicable dead and live loads in accordance with NZS 1170.1. The structure comprises the primary building levels and a lightweight structure at the uppermost level, Level 3. The roof structure was modelled as part of the overall building for gravity loading, seismic mass, diaphragm action and load-path purposes. The seismic mass of the roof was retained at its actual elevation. The additional top force normally generated automatically by ETABS was not applied at the roof level. This reflects the adopted design basis that the additional 8% top force is not required where the uppermost level comprises a lightweight structure. The roof will instead be assessed independently using the parts loading procedure in accordance with NZS 1170.5. For the global structure analysis, Level 2 was treated as the uppermost storey for the purposes of determining seismic shear. Accordingly, the additional 8% storey shear component was applied at Level 2, in addition to the seismic shear generated by the seismic mass at that level under the equivalent static analysis procedure.
Other Relevant Information	-

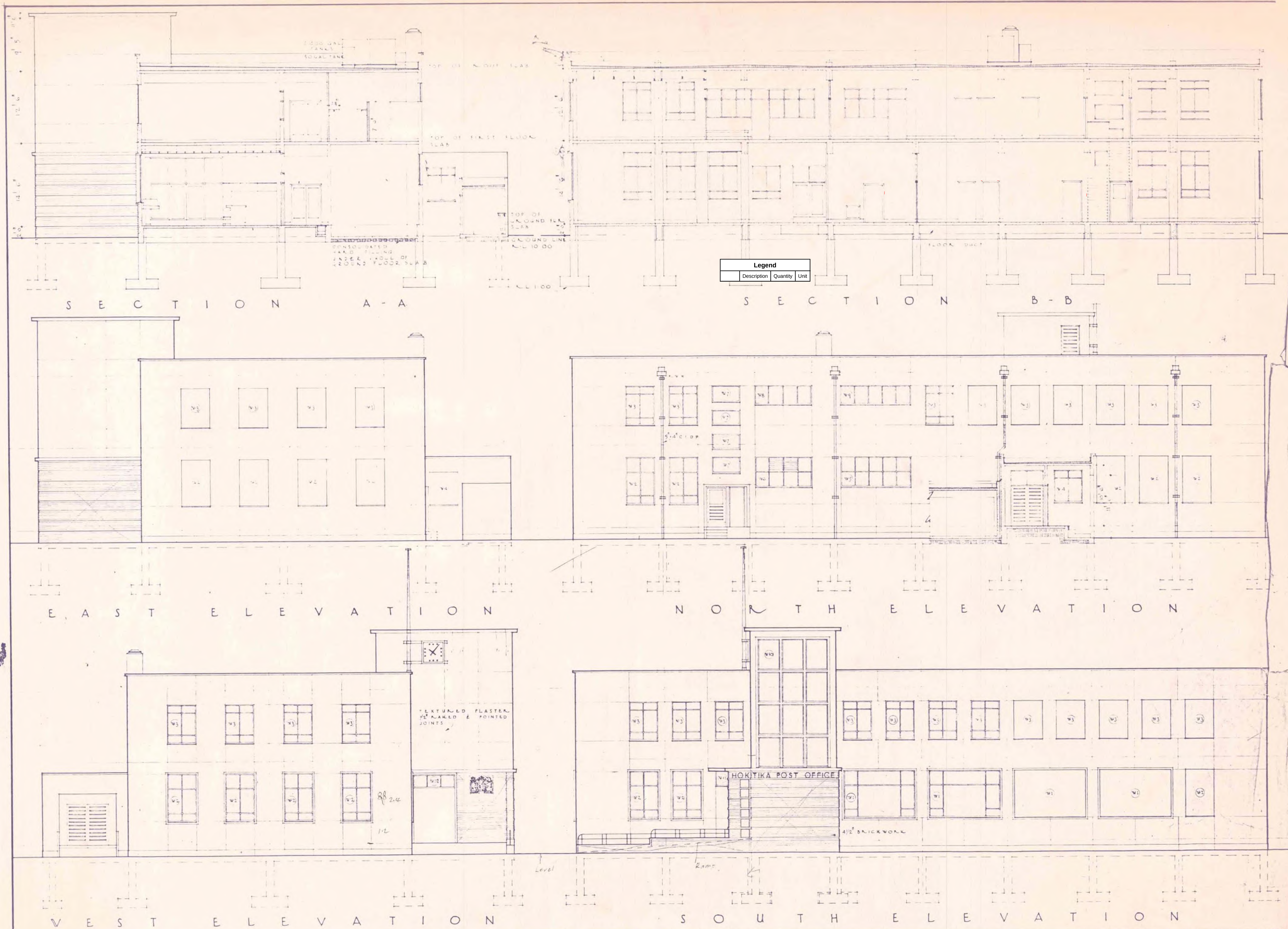
4. Assessment Outcomes		
Assessment Status (Draft or Final)	Final	
Assessed %NBS Rating	25 %NBS	
Seismic Grade and Relative Risk (from Table A3.1)	Grade D –High risk – 10-25 times greater than that of a new building.	
For a DSA:		
Comment on the nature of Secondary Structural and Non-structural elements/ parts identified and assessed	N/A –Analysis of relevant primary structure only.	
Describe the Governing Critical Structural Weakness	See below.	
If the results of this DSA are being used for earthquake prone decision purposes, <u>and</u> elements rating <34%NBS have been identified (including Parts)³:	Engineering Statement of Structural Weaknesses and Location	Mode of Failure and Physical Consequence Statement(s)
	Foundation Column piles flexural capacity (Transverse direction)	Columns are likely to exceed their flexural and shear capacity, and the secondary load path of foundation beam bearing would provide gravity support to the building.
	Third storey portal frames hold downs (Transverse direction)	The portal frame hold downs are expected to exceed their capacity. The Portal frames are likely to displace excessively. Partial collapse is likely in open areas without any internal timber framed walls.
	Third storey in plane plasterboard bracing and steel flat bracing. (Longitudinal direction)	The plasterboard walls and steel flat bracing are expected to displace excessively. Partial collapse is likely in open areas without any internal timber framed walls.
Recommendations (optional for EPB purposes)	We support the recommendation of the EQ-Assess guidance that it is desirable to seismically strengthen earthquake risk buildings to as near as reasonably practical to that of a new building; but, as a minimum, seismic improvements should achieve at least 67%NBS.	

³ If a building comprises a shared structural form or shares structural elements with other adjacent titles, information about the extent to which the low scoring elements affect, or do not affect the structure.

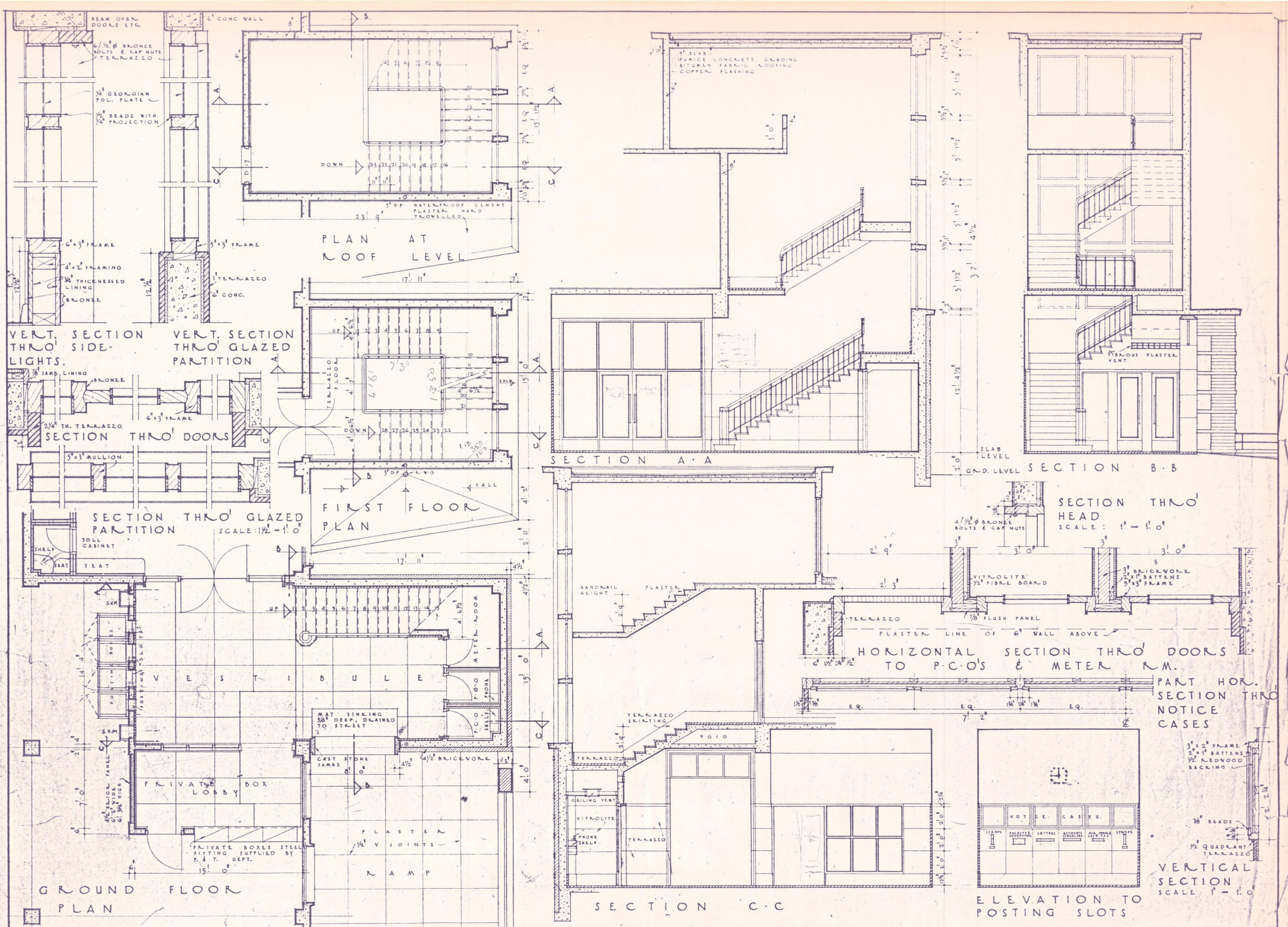
Appendix B. Existing Building Plans



4643	SCALES	NEW POST OFFICE BUILDING HOKITIKA	P.V.D
1	1/16" = 1' 0"	LAYOUT DRAINAGE & ROOF PLANS	126351
2	1/8" = 1' 0"	DETAILS OF SEPTIC TANK CYCLES ETC.	
3	1/2" = 1' 0"		
4	1" = 1' 0"		



46/43 NNC NOV/47 NNC 1/47 1/10/47	SCALES $\frac{1}{8}'' = 1' 0''$	NEW POST OFFICE BUILDING HOKITIKA ELEVATIONS & SECTIONS	P.W.D. 126351
--	------------------------------------	---	------------------



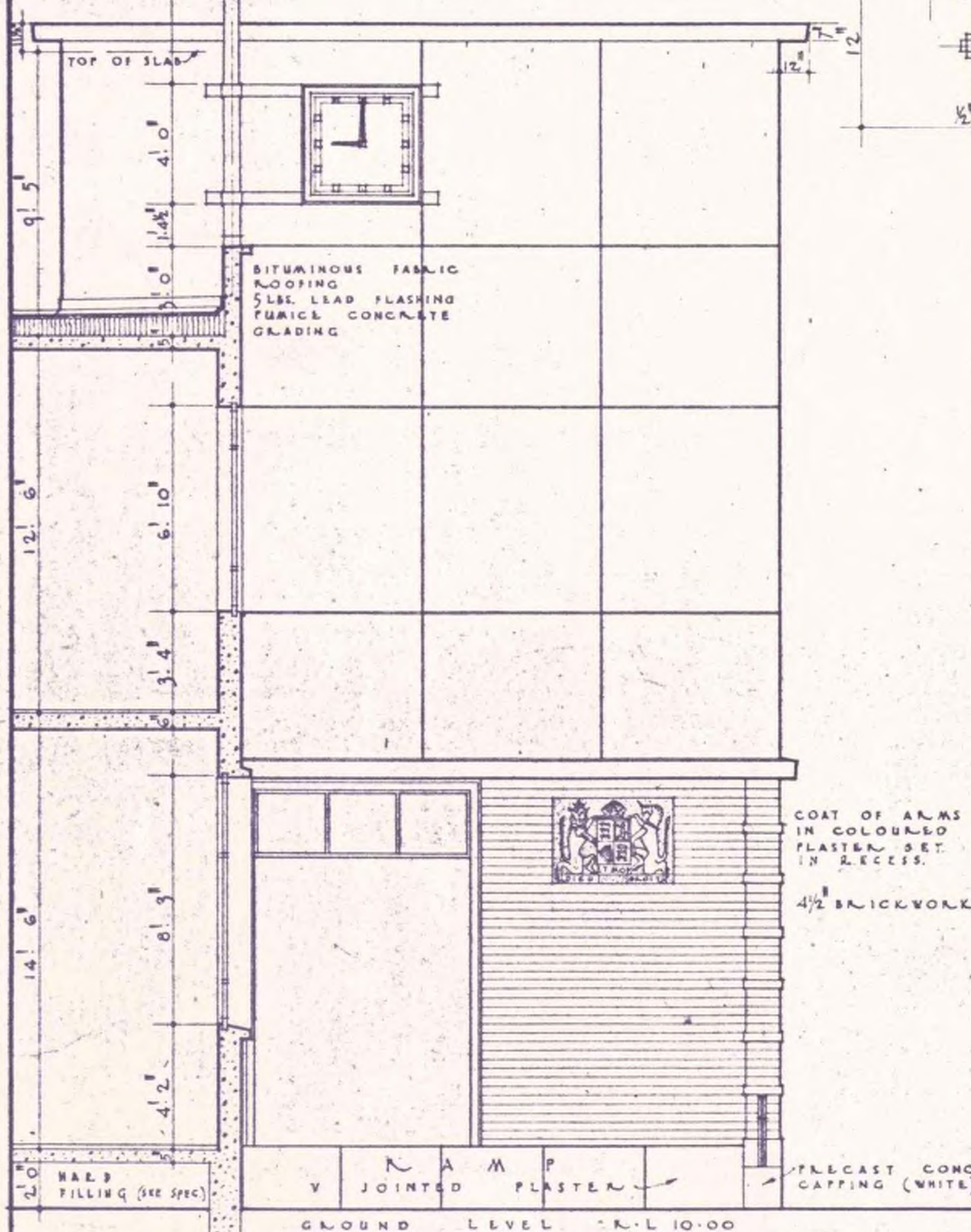
DETAILS OF
GROUND & FIRST
FLOOR WINDOWS
SCALE 3" 1:0"

ELEVATION

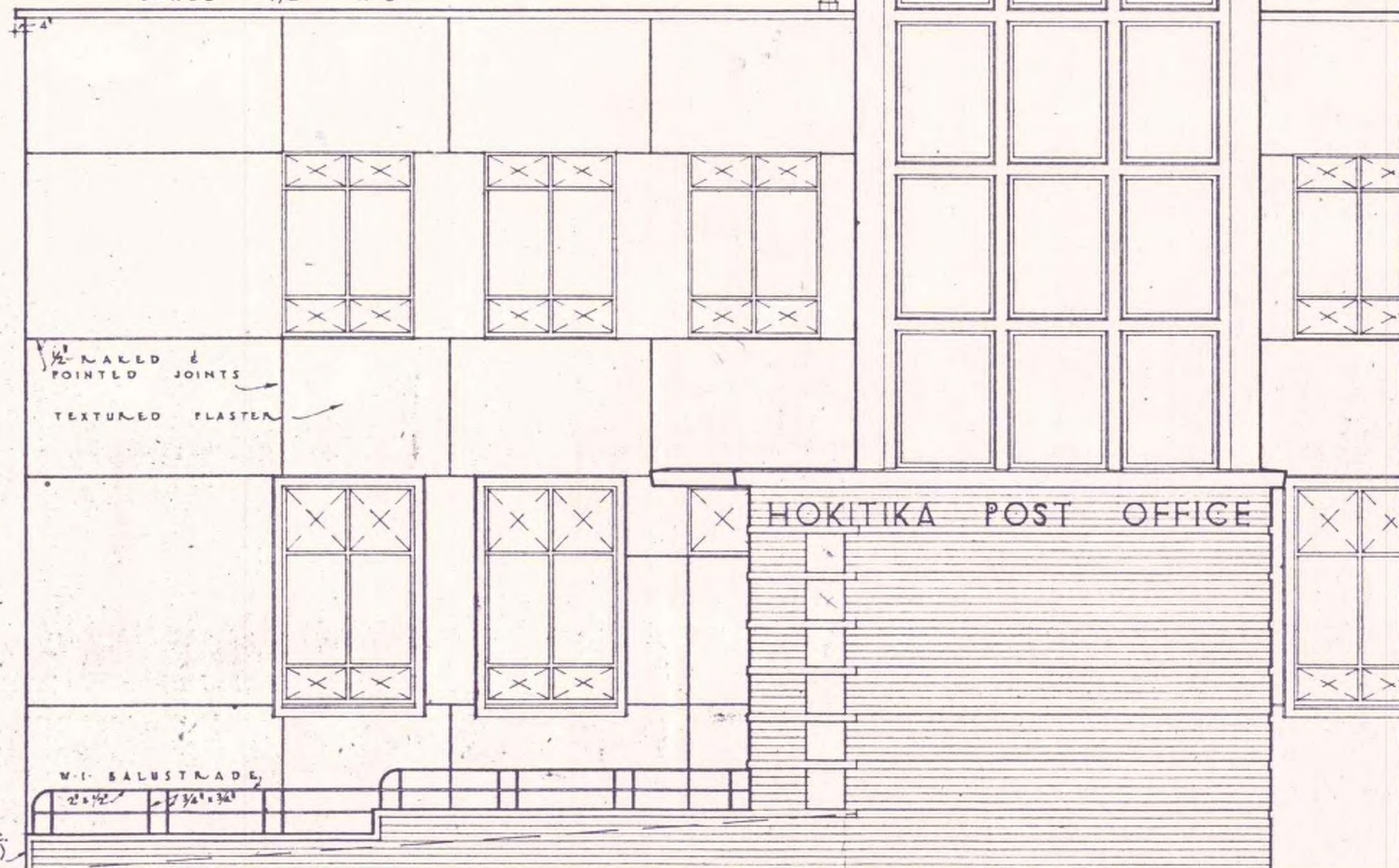
PART HORIZONTAL
SECTION THRO' CLOCK
& FLAGPOLE BRACKET
SCALE 1 1/2" 1' 0"

SECTION

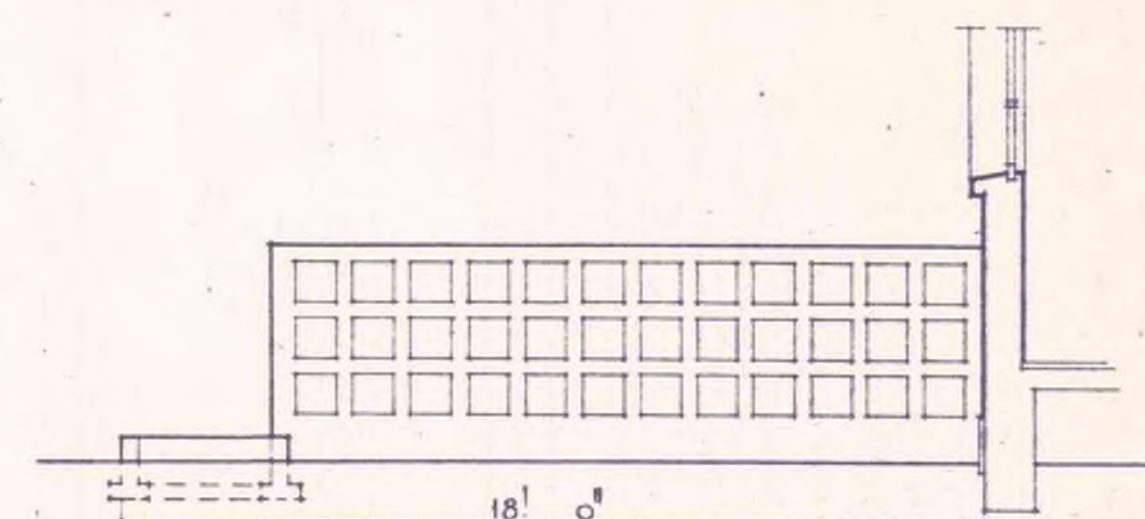
DETAILS OF BRONZE
LETTERING ETC.
SCALE 3" 1.0"



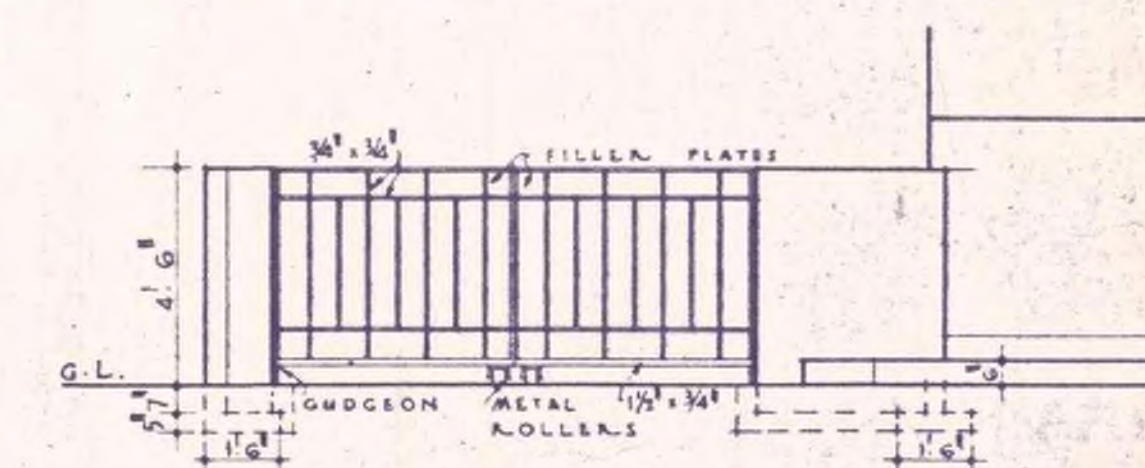
TYPICAL
WALL
SECTION



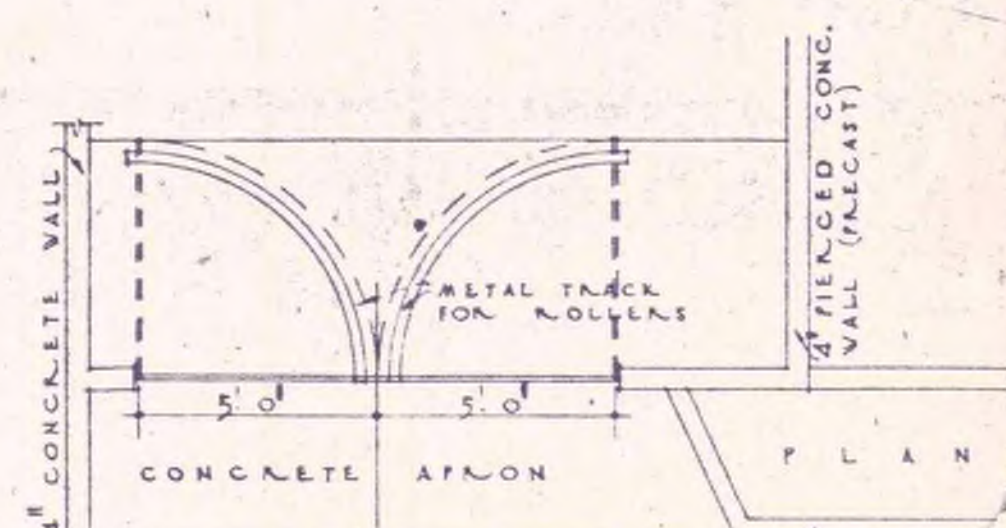
PART SOUTH ELEVATION



ELEVATION OF
PIERCED WALL



ELEVATION OF GATES

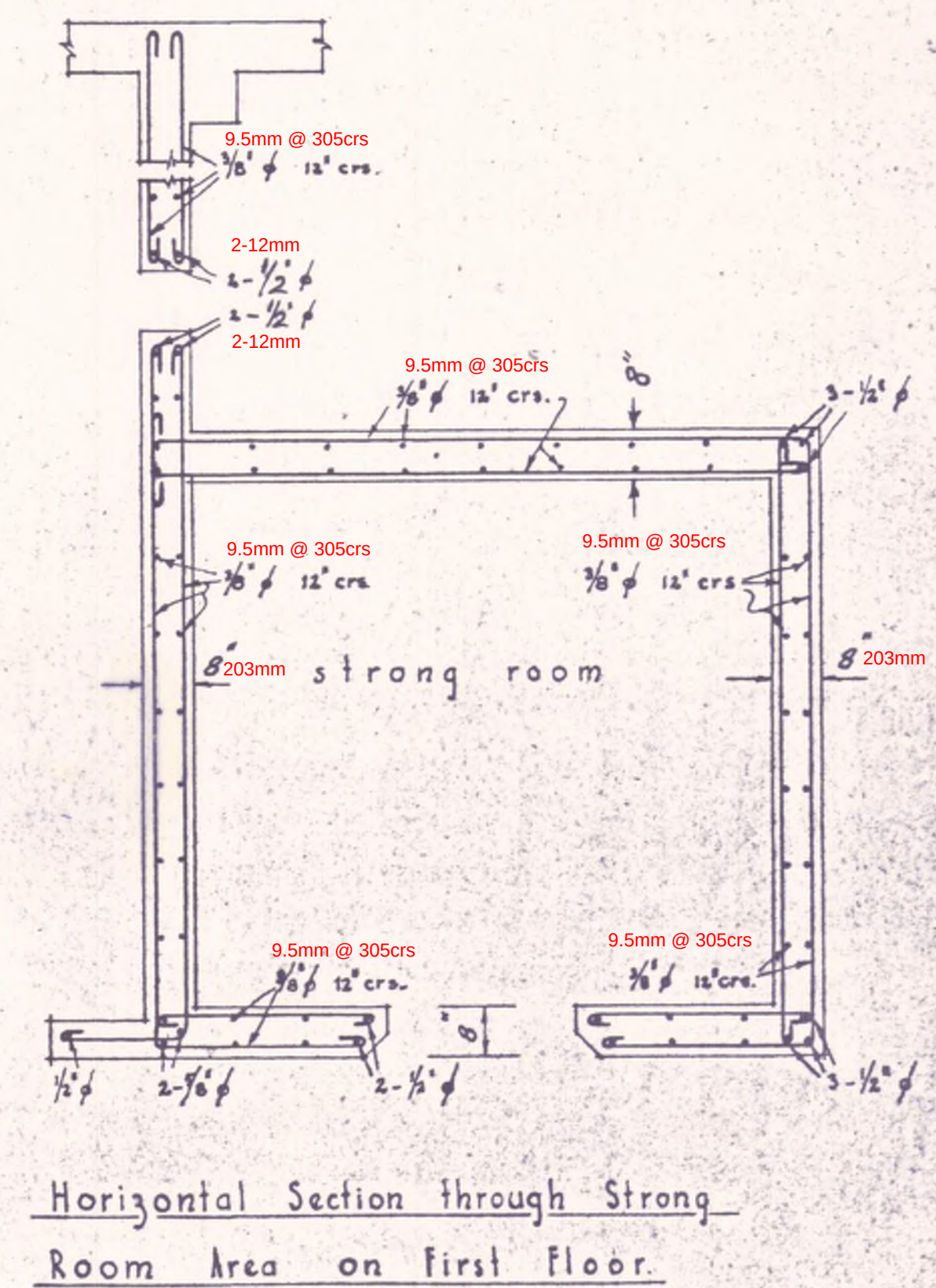
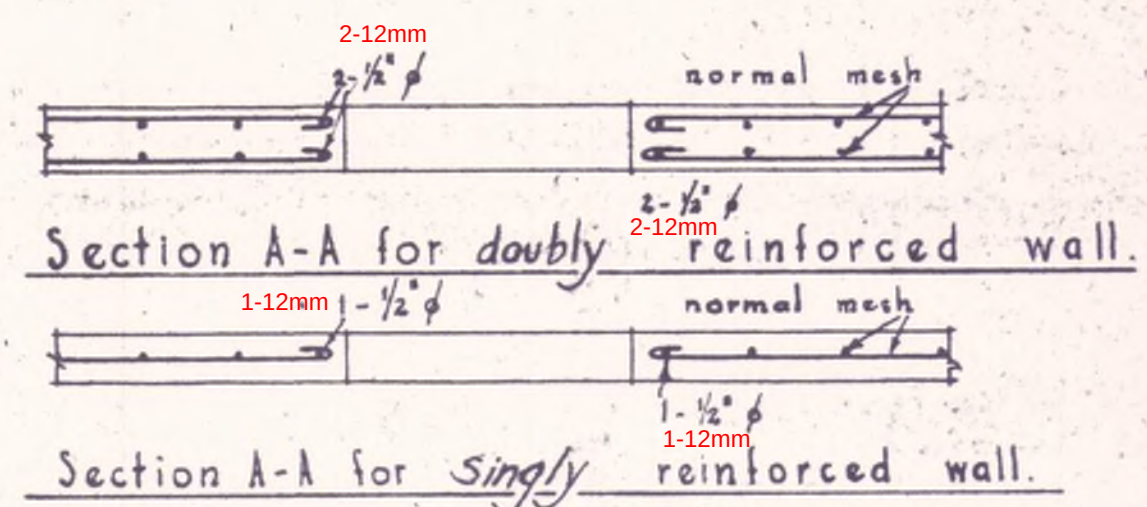
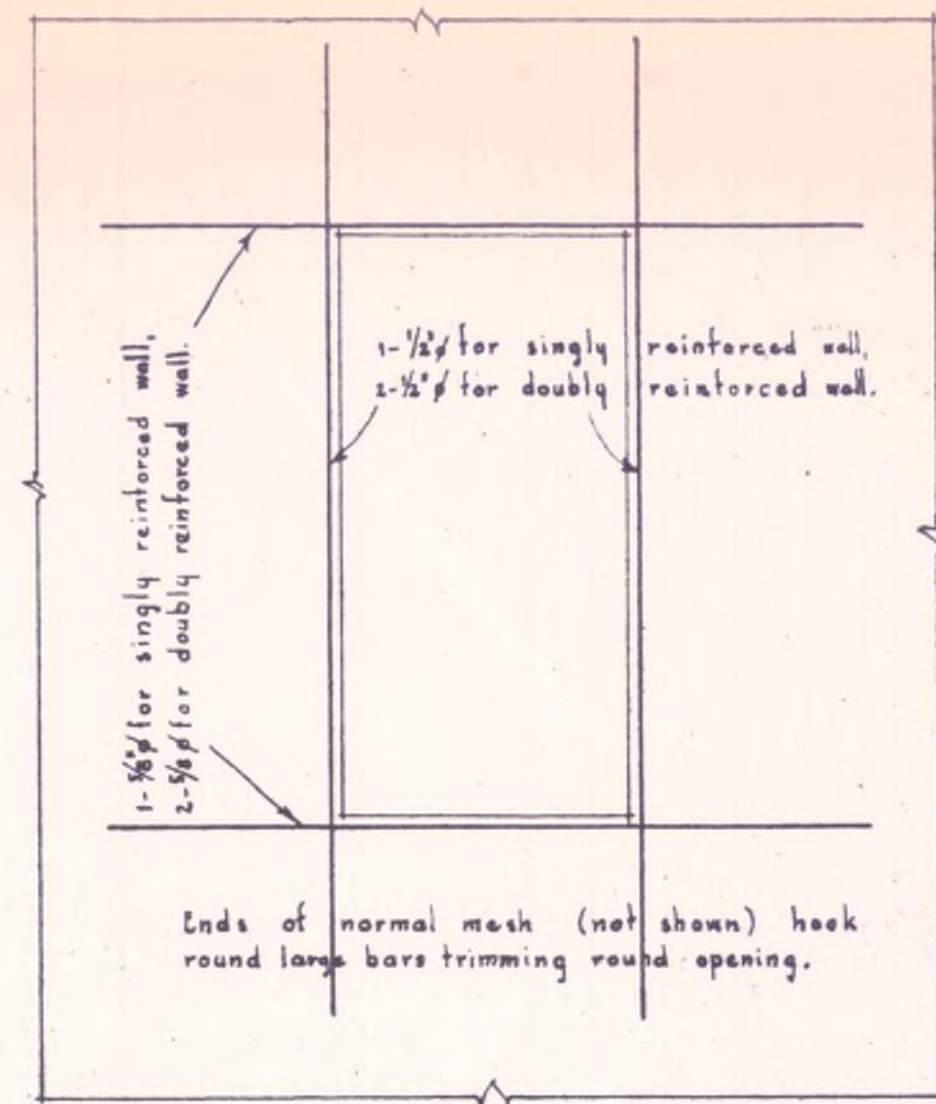
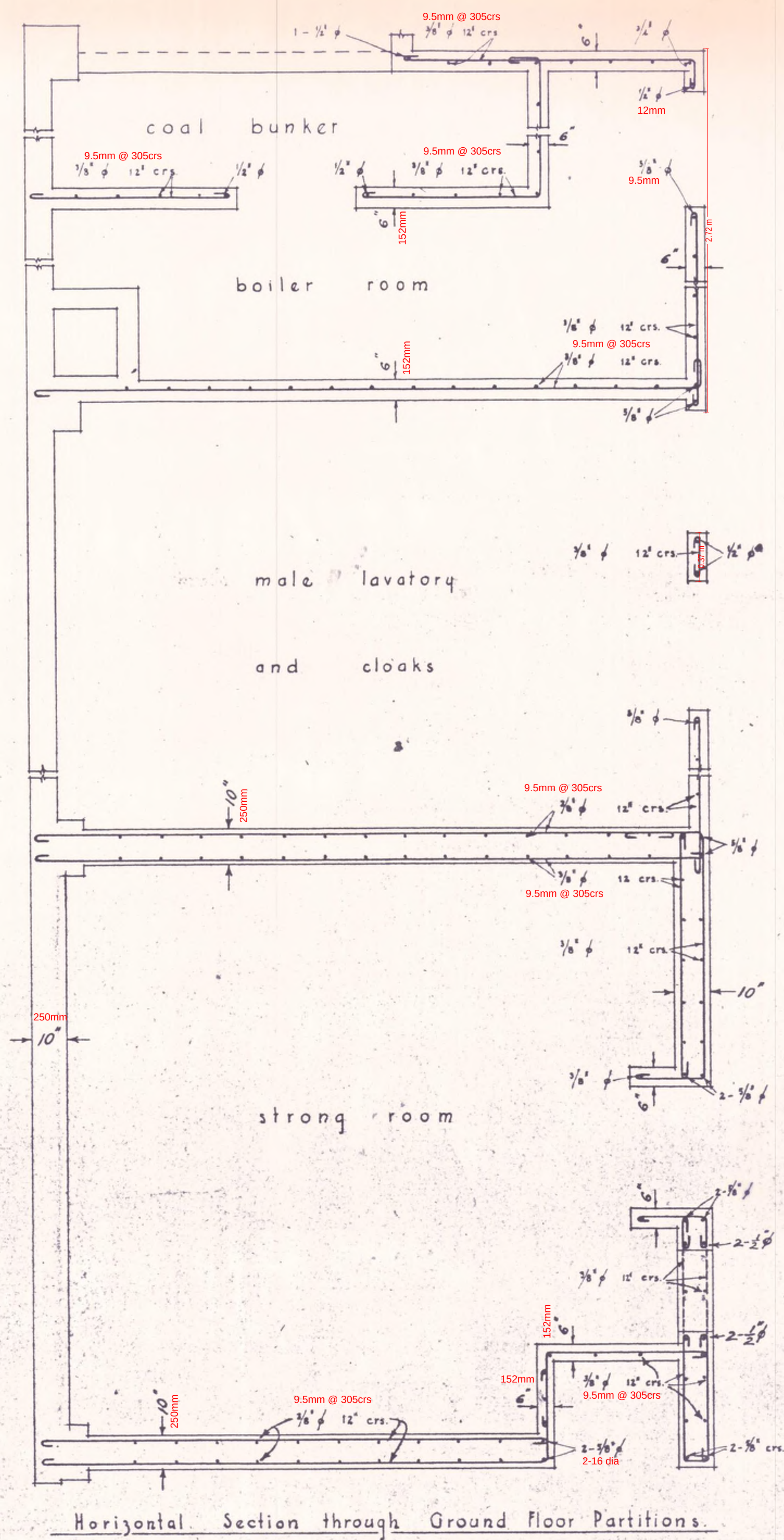
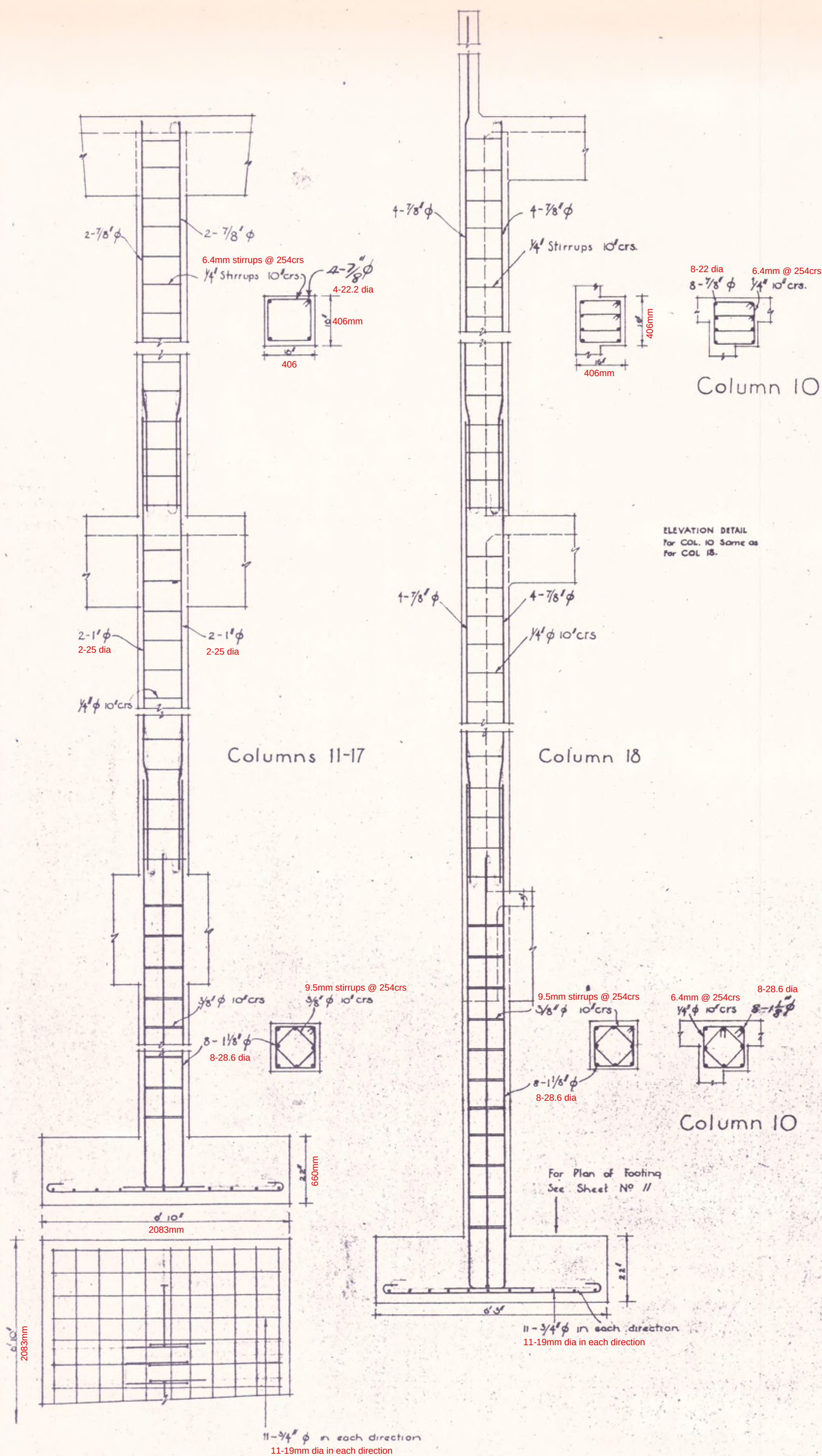


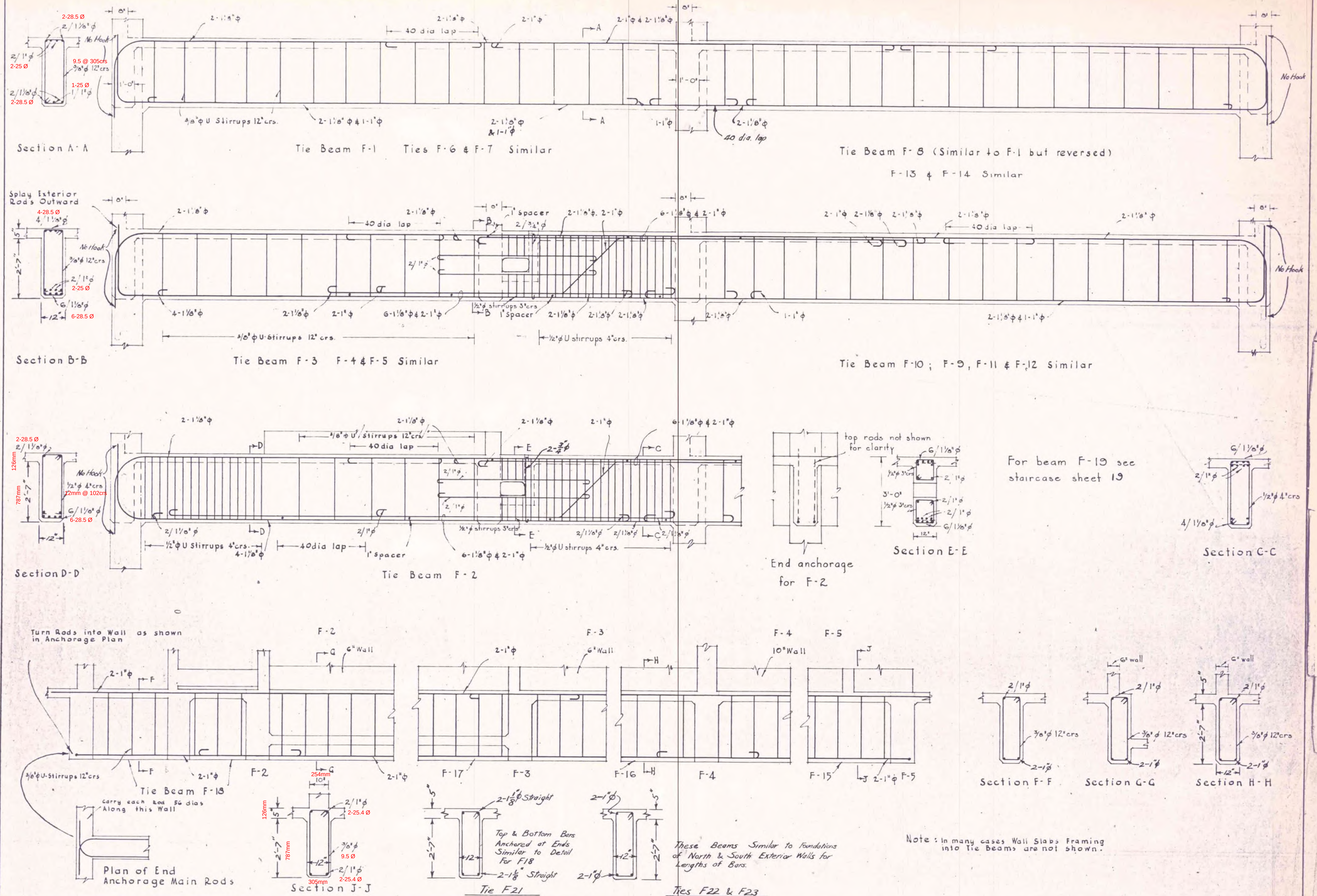
PLAN
DETAILS OF
TO MAIL DO

6 46/43
N.N.C NOV/
N.N.C 47
APRIL 1947

S	C	A	L	E	S
1/4"	—	1	0"		
1/2"	—	1	0"		
3"	—	1	0"		

NEW POST OFFICE BUILDING HOKITIKA
EXTERIOR DETAILS





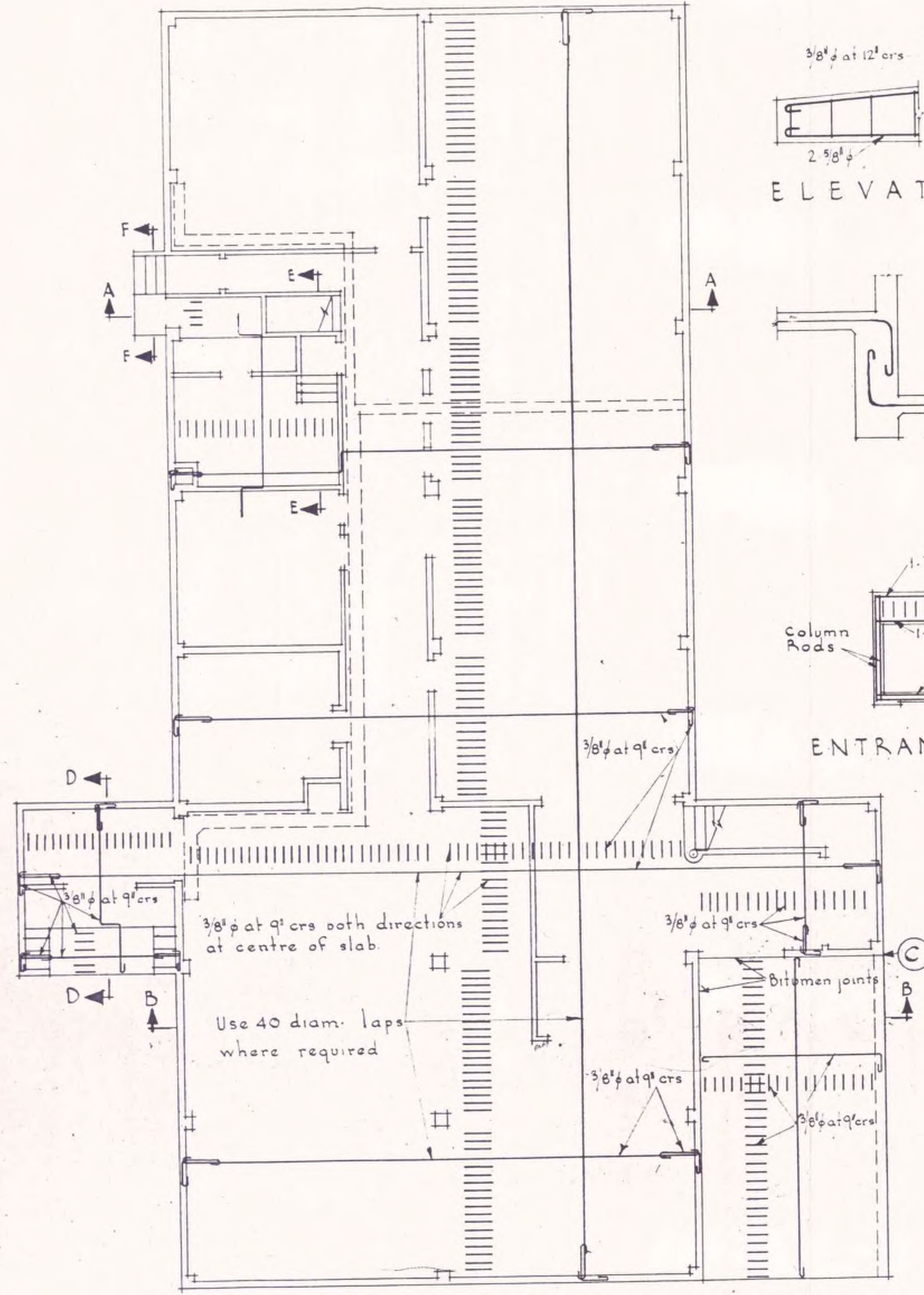
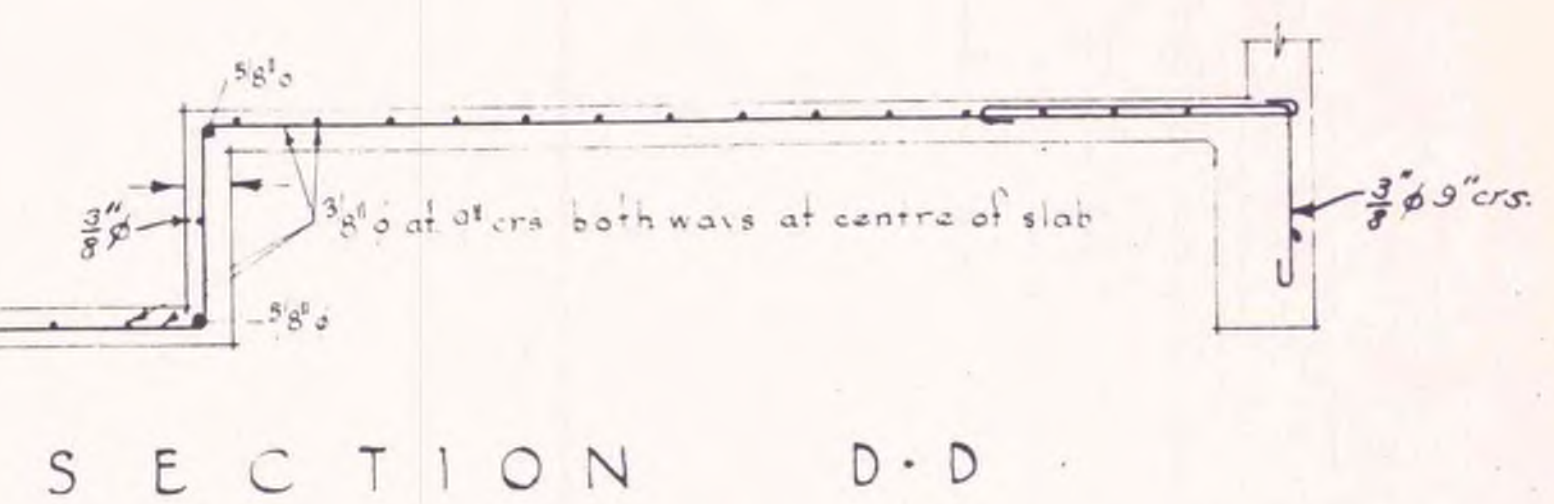
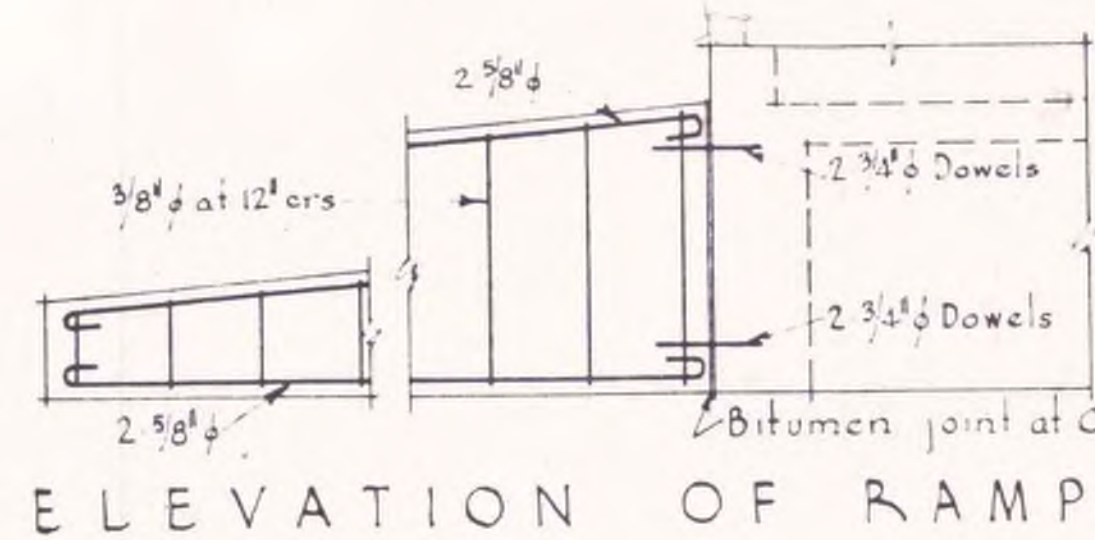
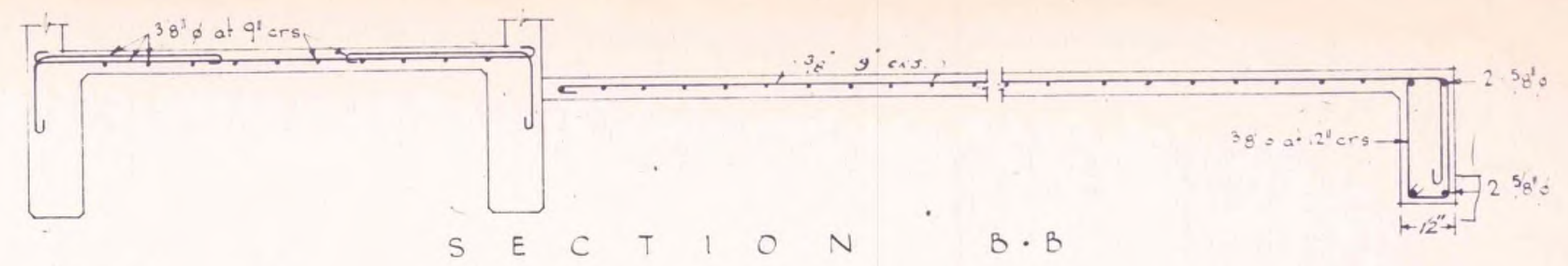
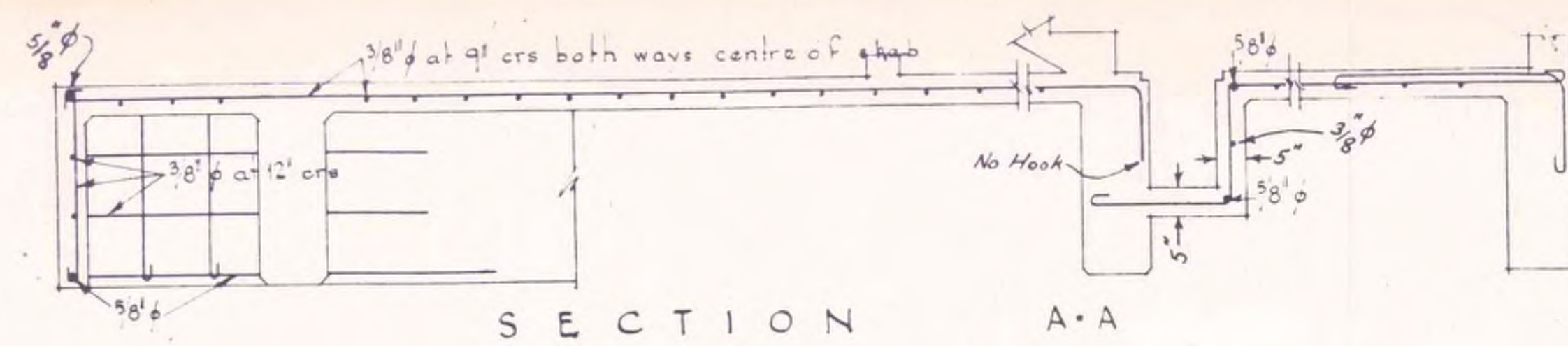
JOB No. 46/43
SHEET 13 (of 14)
DRAWN BY: M.M.K. P.E.B.
TRACED BY: J.A.R. 47.
CHECKED BY: J.A.R. 47.
APR 47.

SCALES
1/2" = 1 FT.

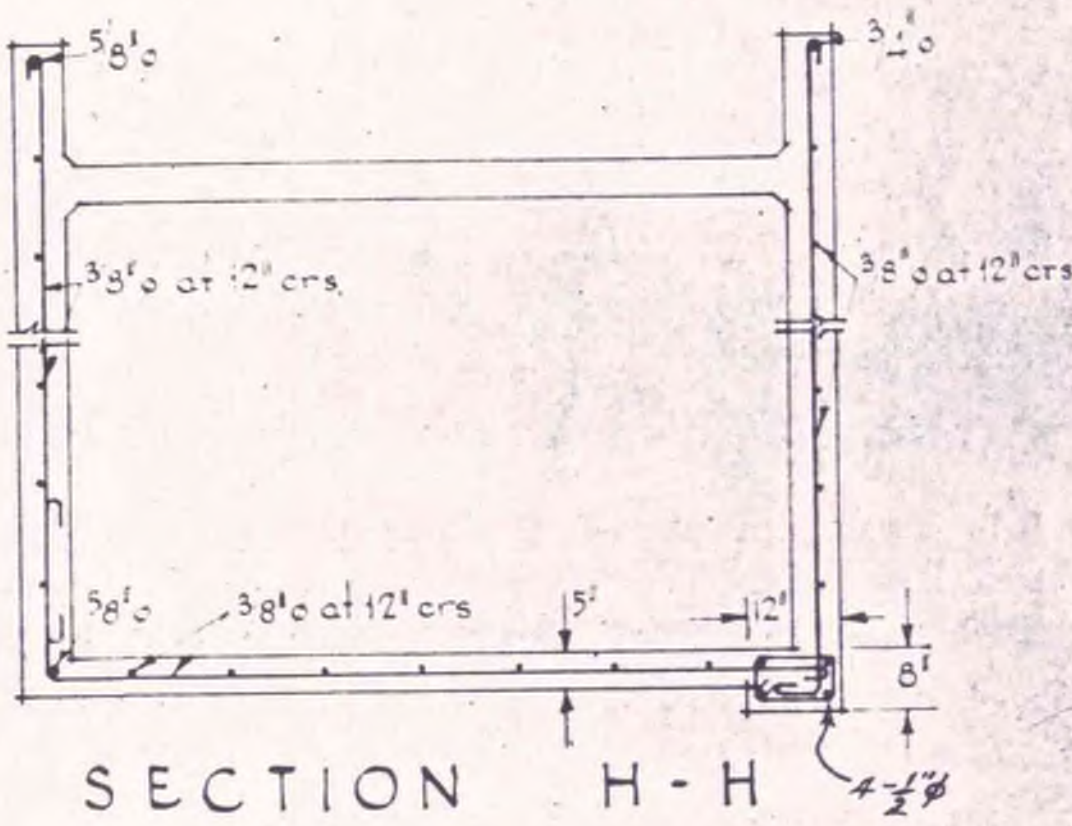
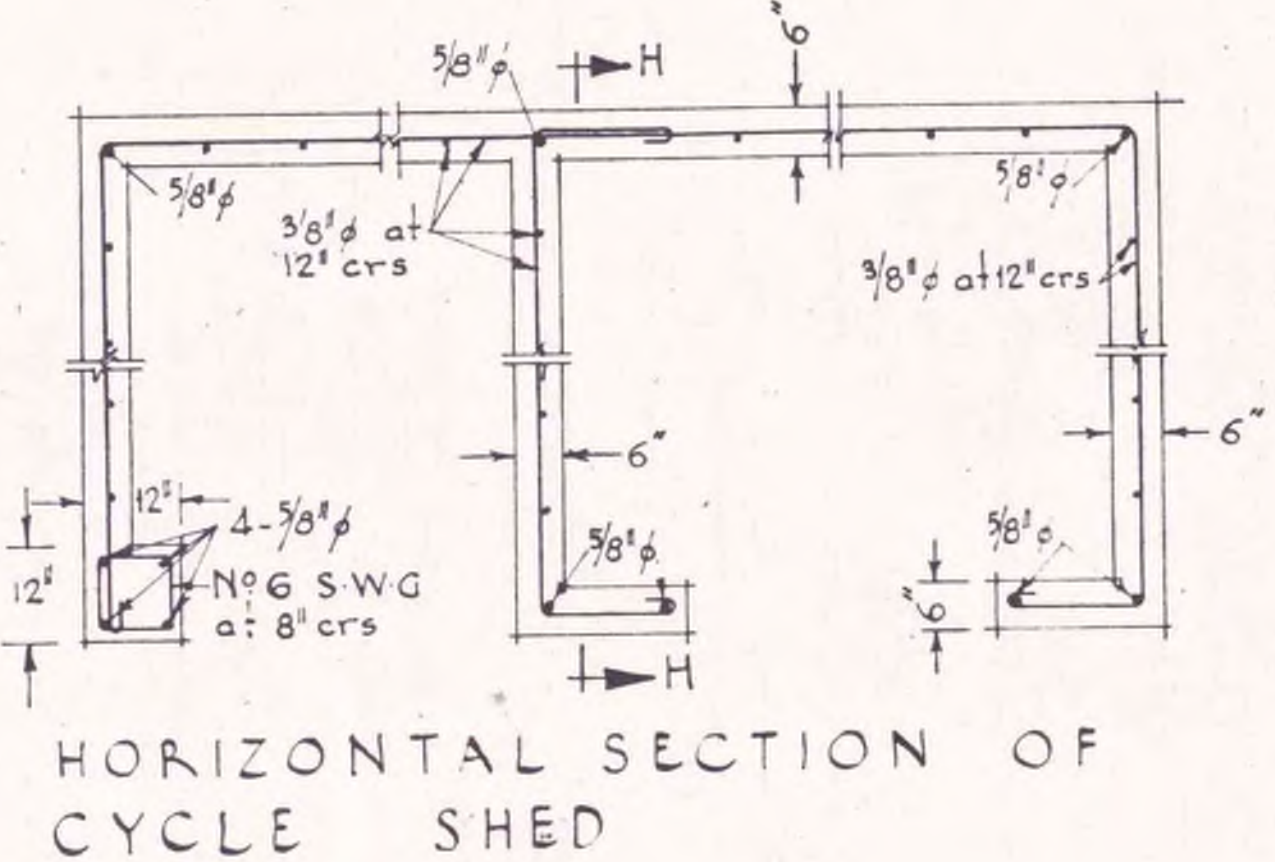
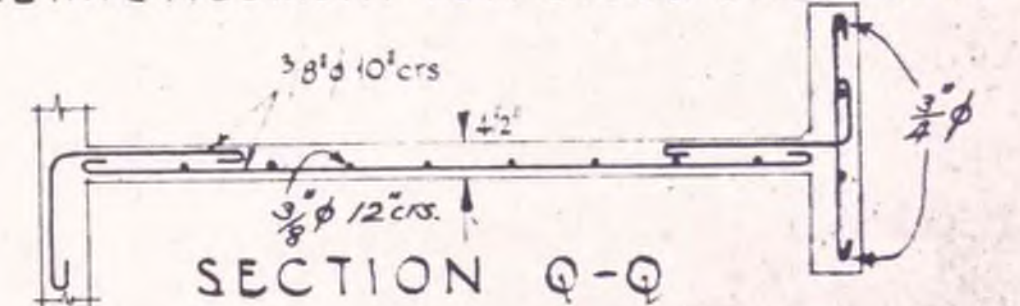
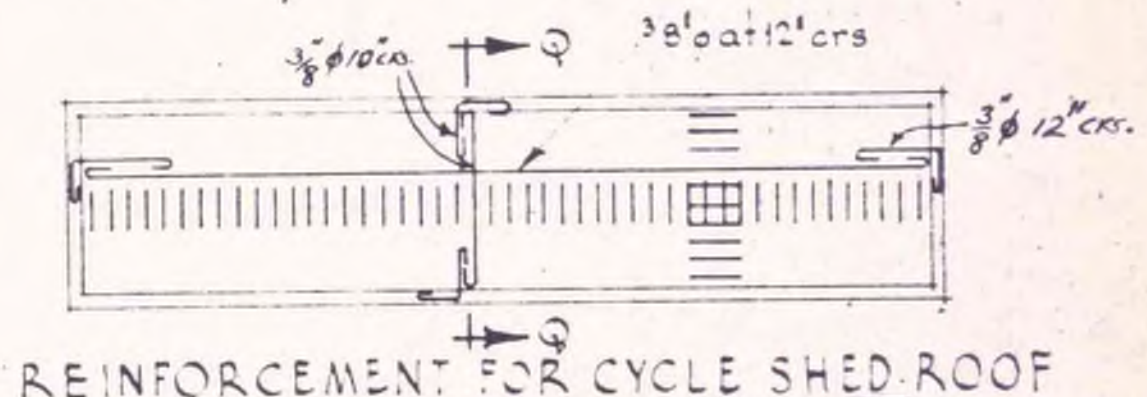
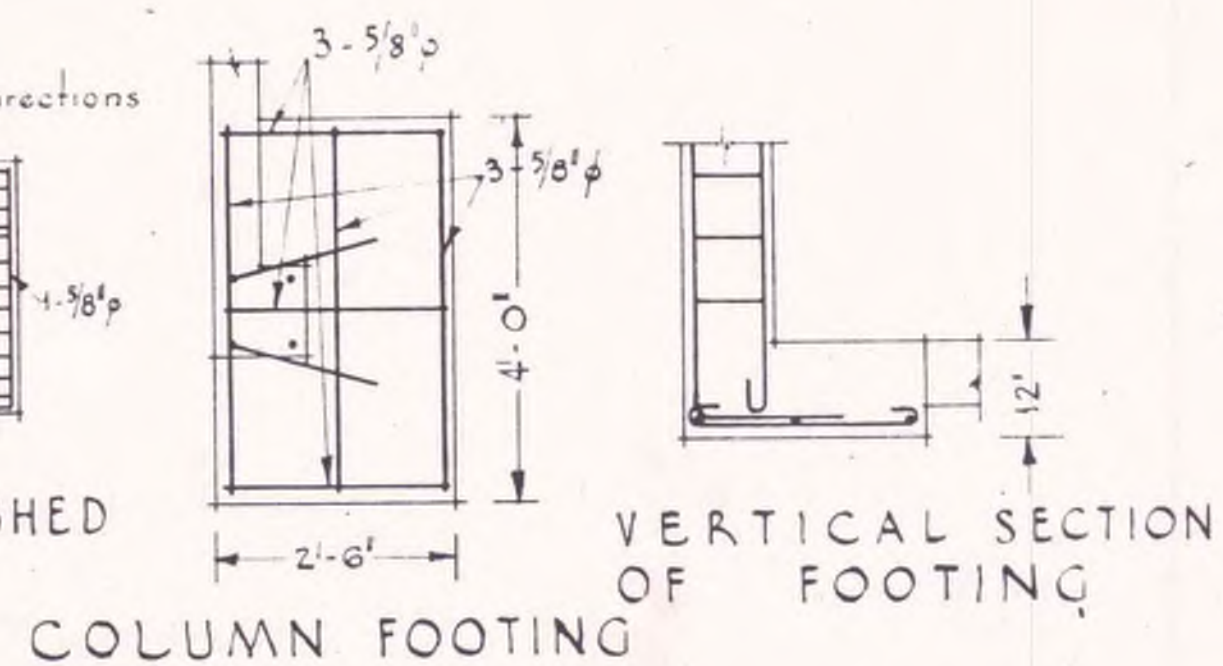
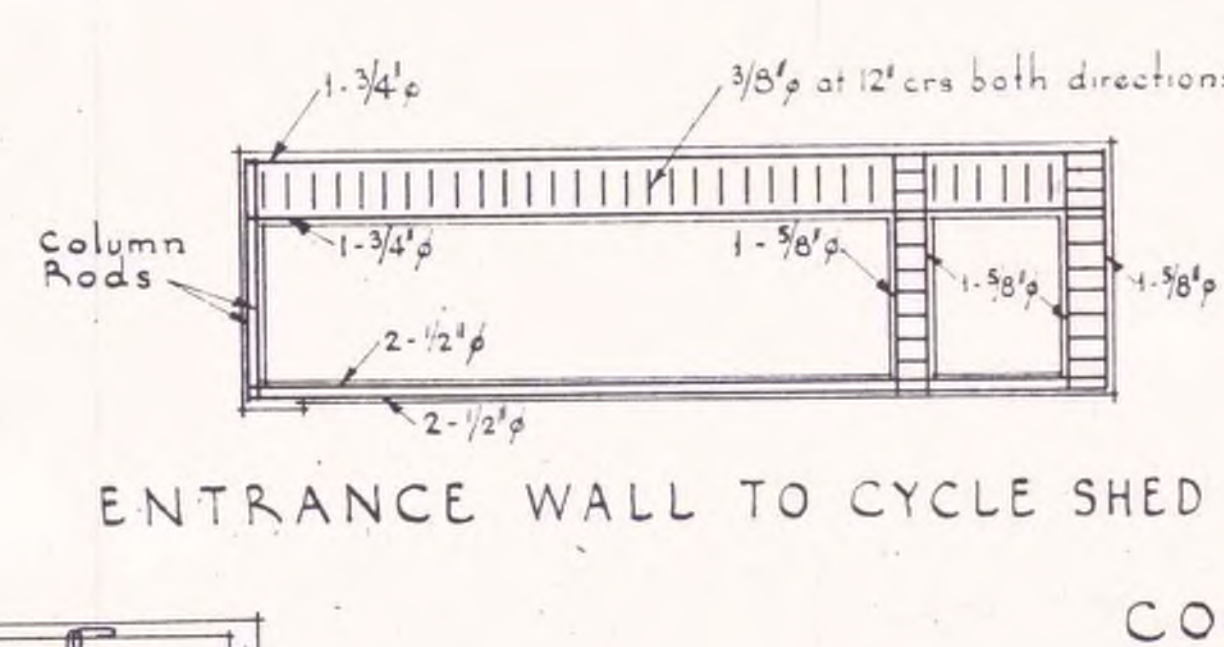
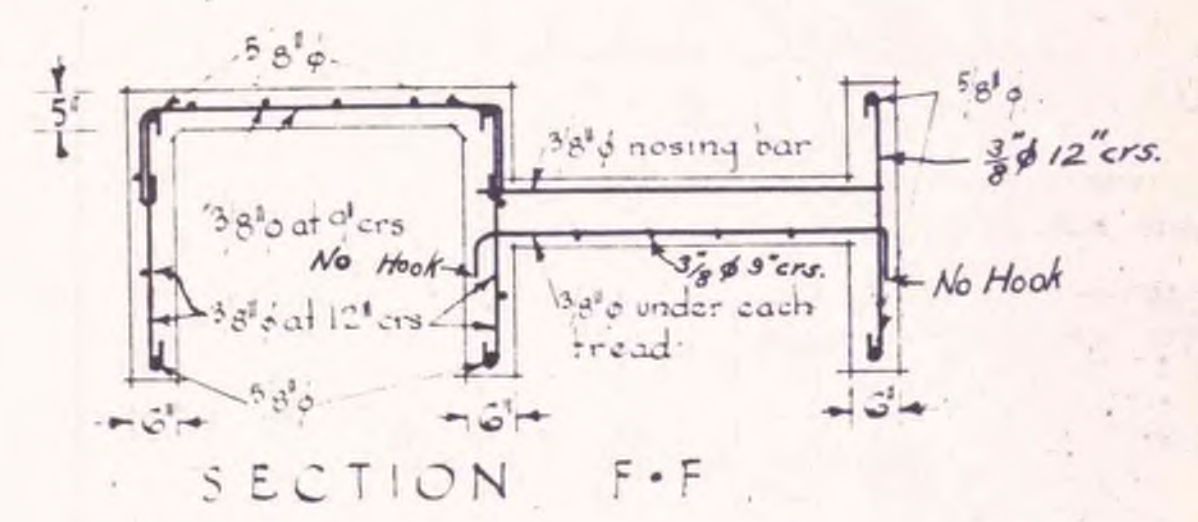
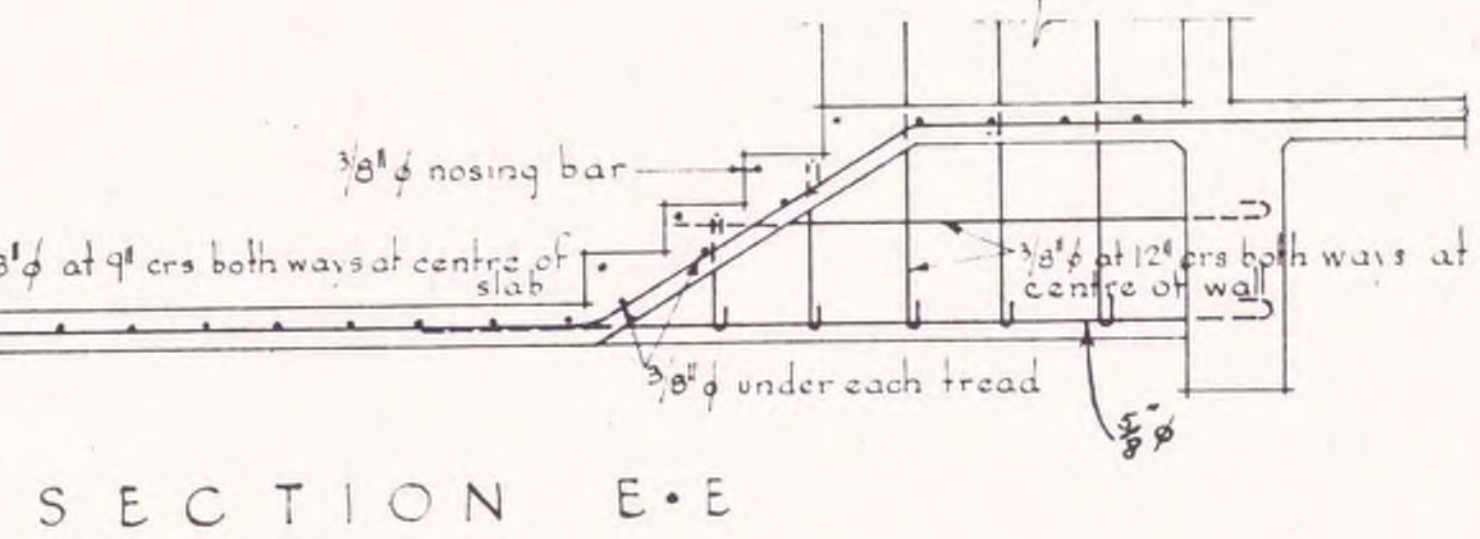
NEW POST OFFICE BUILDING HOKITIKA FOUNDATION TIE BEAMS

Sheet No. 13
in Set of
P.W.D. 12635/





GROUND FLOOR REINFORCING PLAN [5" SLAB]
FOUNDATION TIE BEAMS NOT SHOWN FOR CLARITY.



46/43

C.M.C.
K.E.B. Jan 48
G.M.A. Jan 48
S.B. April 48

SCALES

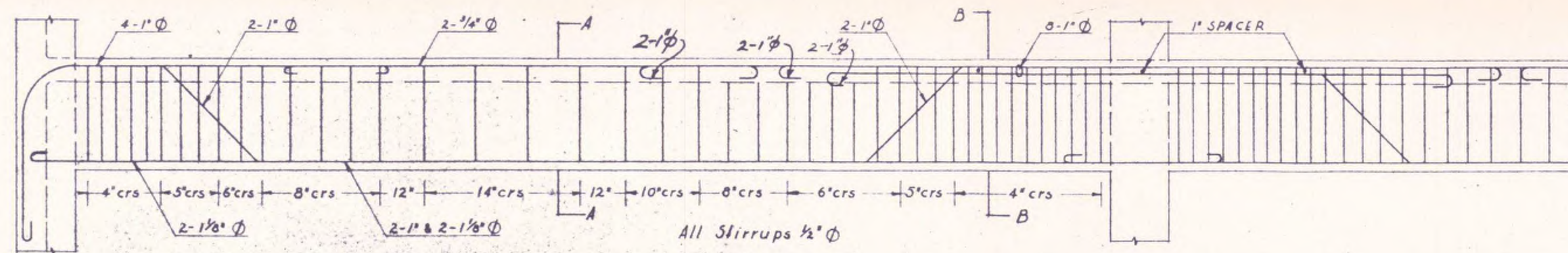
1/8" = 1 FOOT
1/2" = 1 FOOT

NEW POST OFFICE BUILDING HOKITIKA

GROUND FLOOR SLAB & MISCELLANEOUS DETAILS

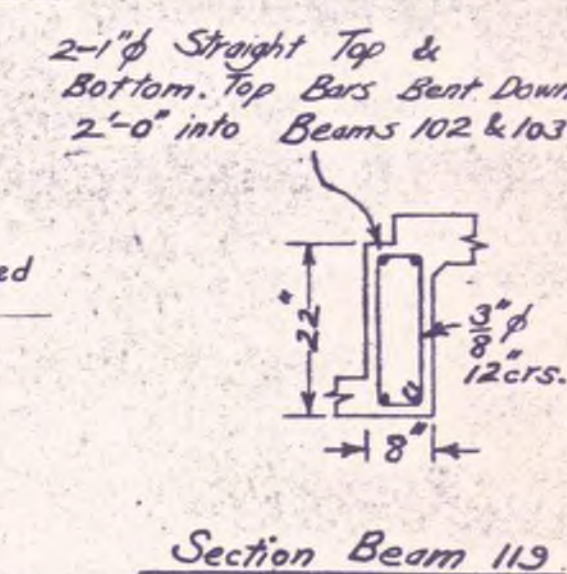
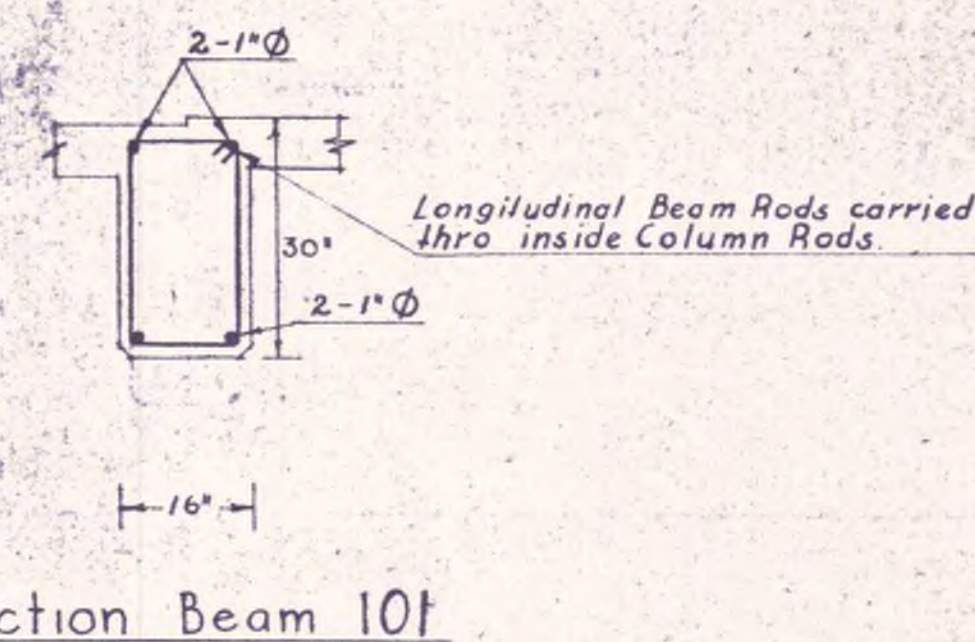
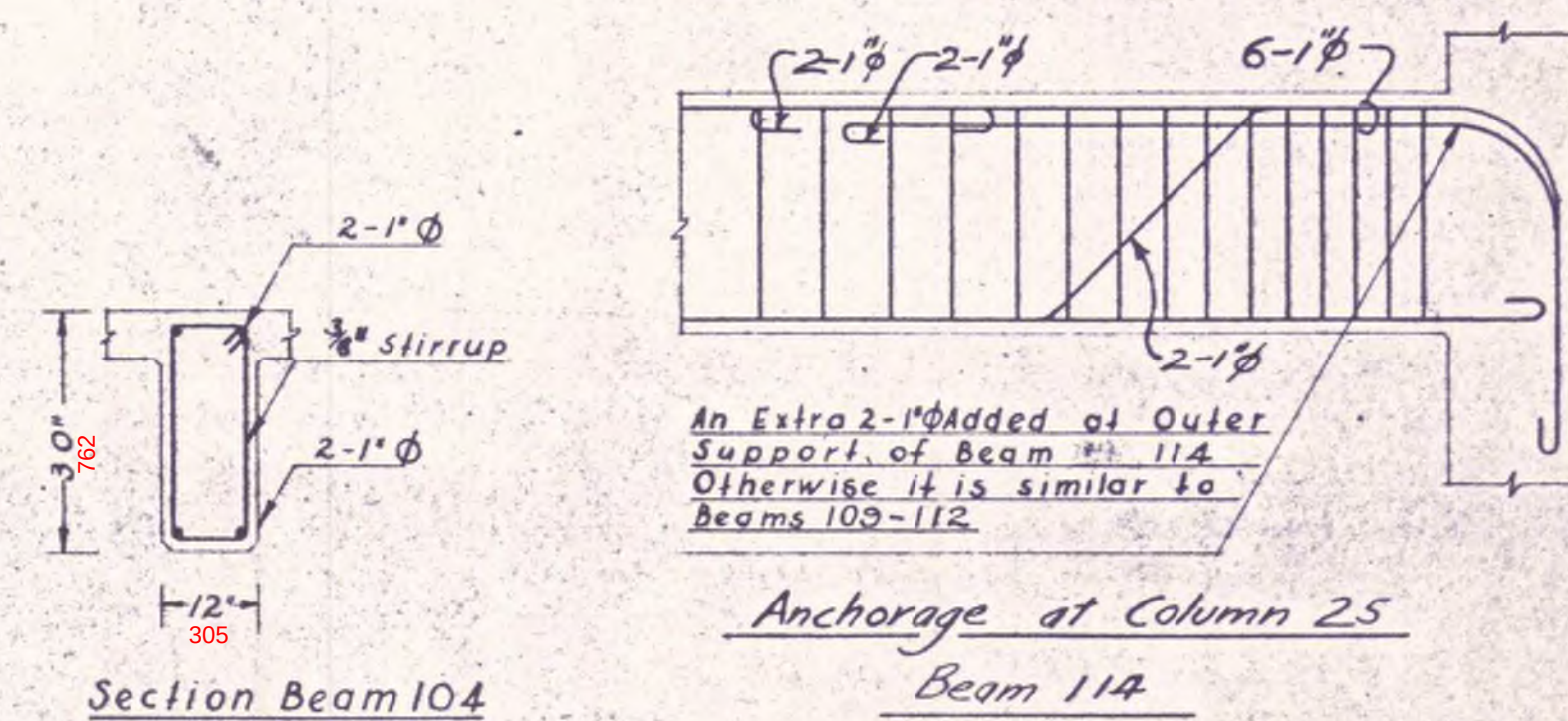
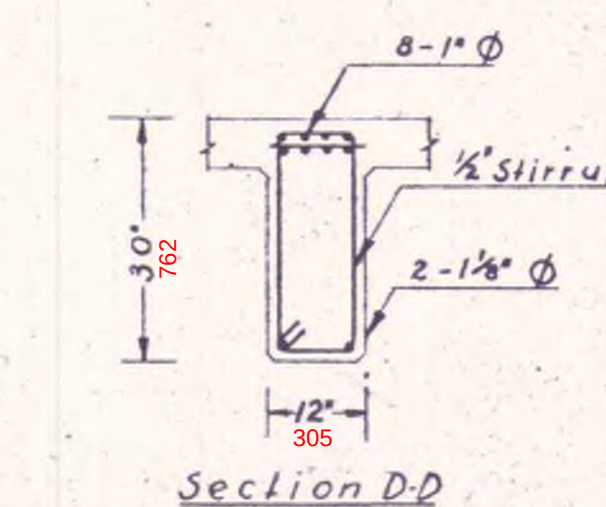
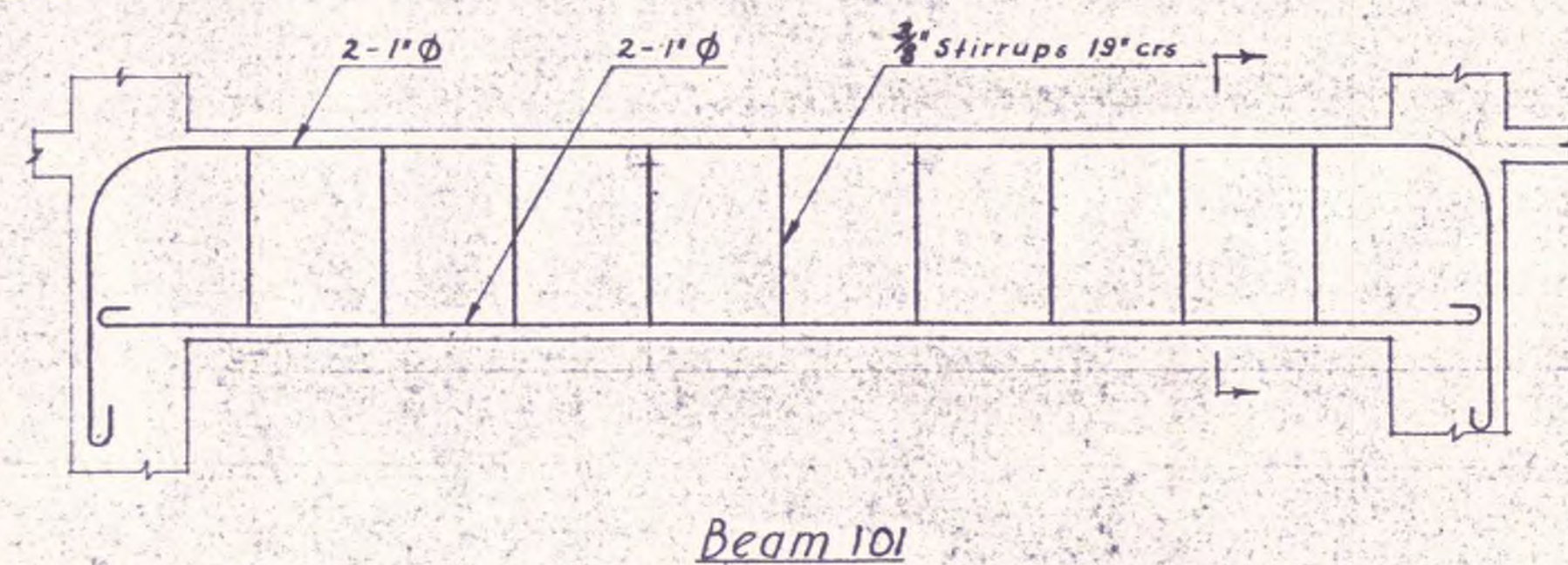
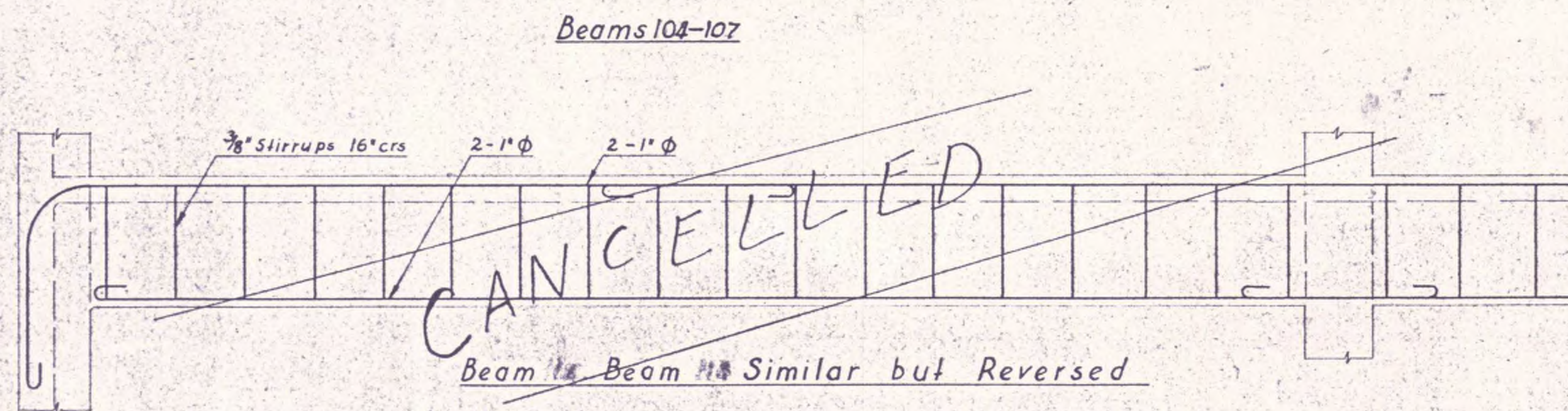
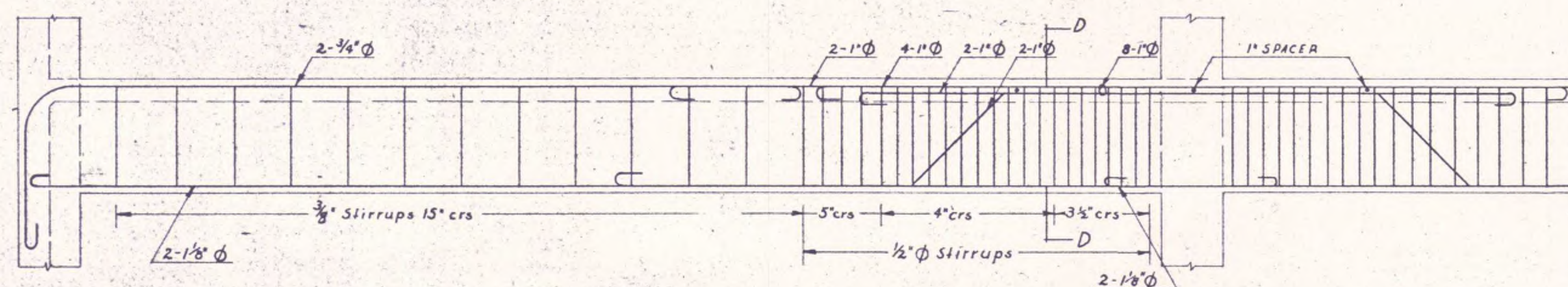
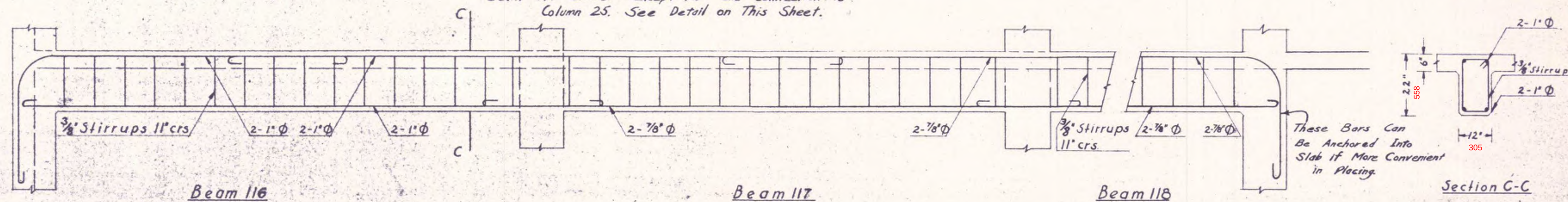
SHEET NO 14
IN SET OF 25

P.W.D 126351



First Floor Beams 102, 103, 108. Beams 109-113, 115 Similar but Reversed

Beam 114 Similar except for end connection to Column 25. See Detail on This Sheet.



An Extra 2-1" Added at Outer Support of Beam 114
Otherwise it is similar to Beams 109-112

Anchorage at Column 25
Beam 114

2-1" ϕ Straight Top &
Bottom. Top Bars Bent Down
2'-0" into Beams 102 & 103

Longitudinal Beam Rods carried thro inside Column Rods.

46/43

15	MASS. MAE
G. McK	Sept. 46
R.F.	Sept. 46
G. McK. Rm. 28	
J.B.	April 1948

SCALES

 $\frac{1}{2}'' = 1 \text{ FT.}$

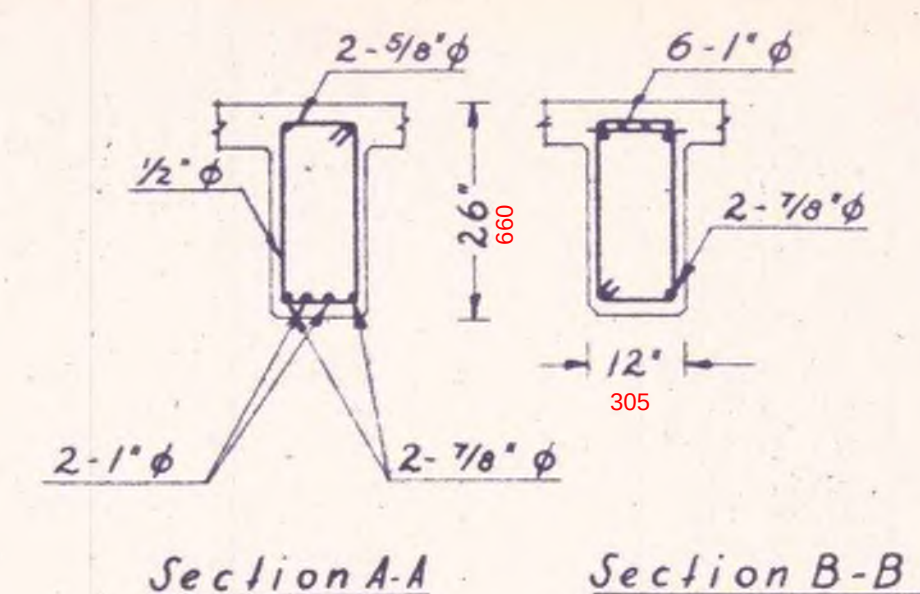
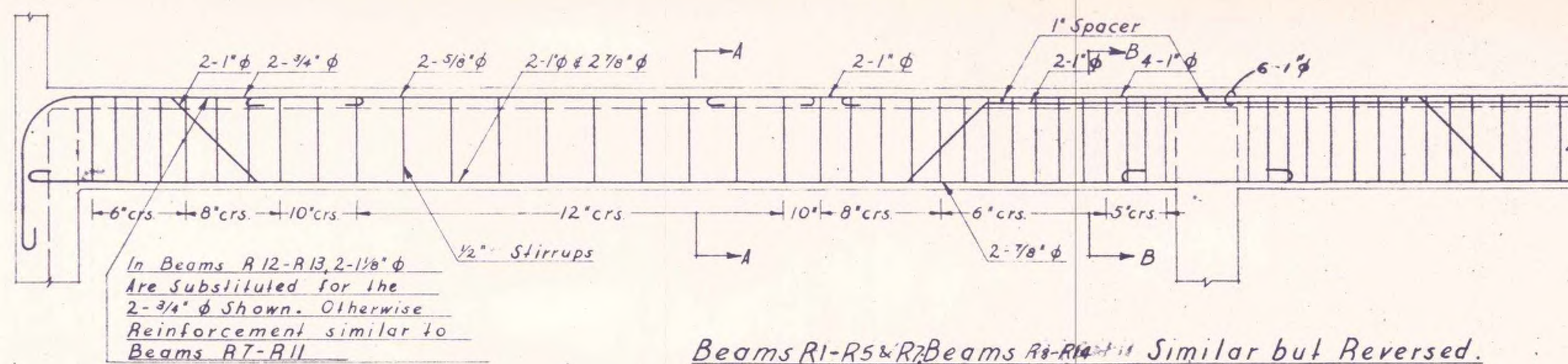
NEW POST OFFICE BUILDING HOKITIKA
1st FLOOR BEAMS

1st FLOOR BEAMS

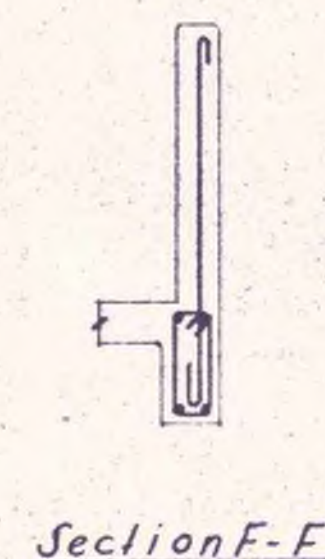
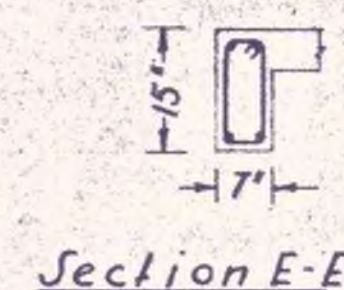
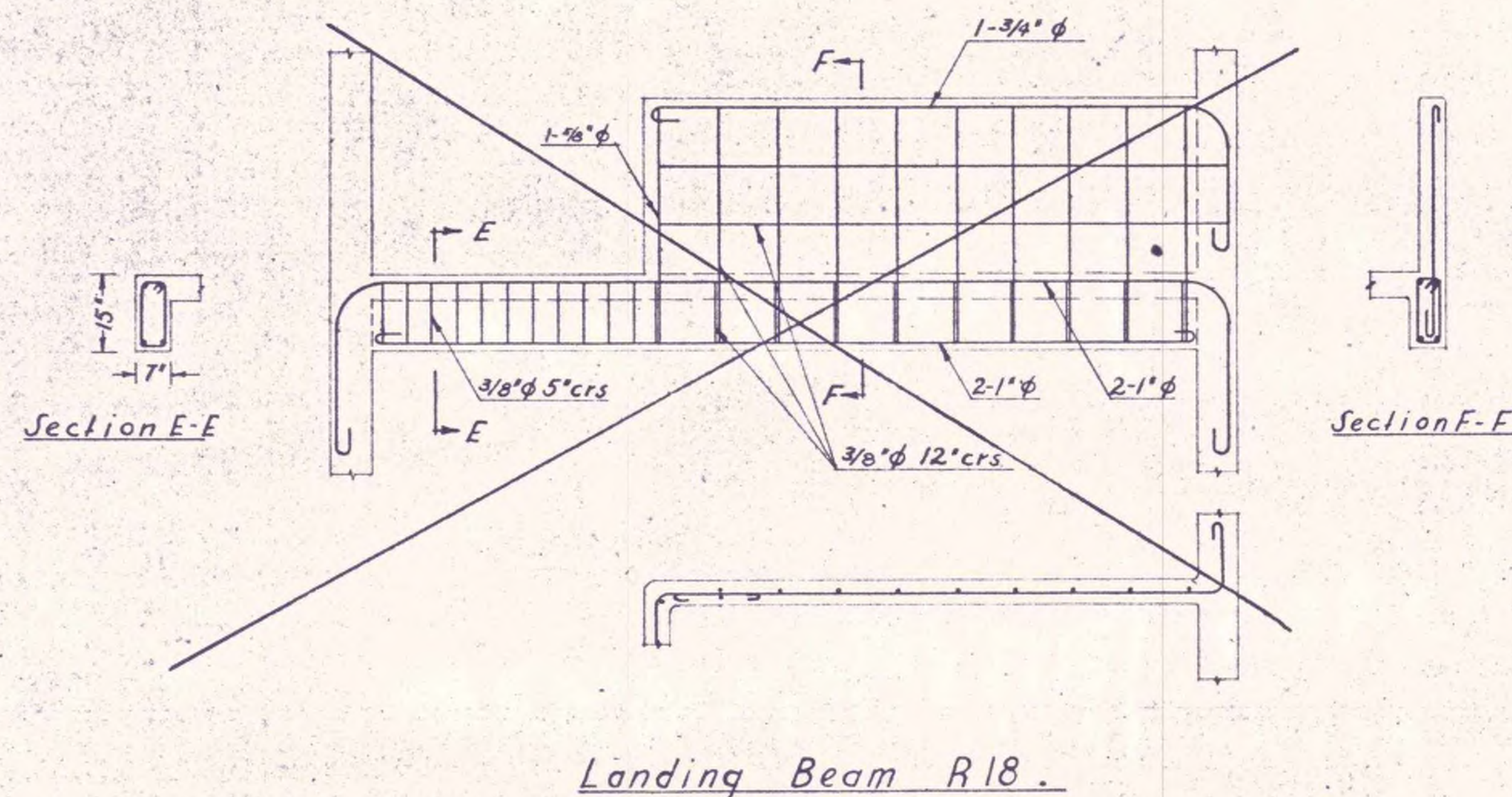
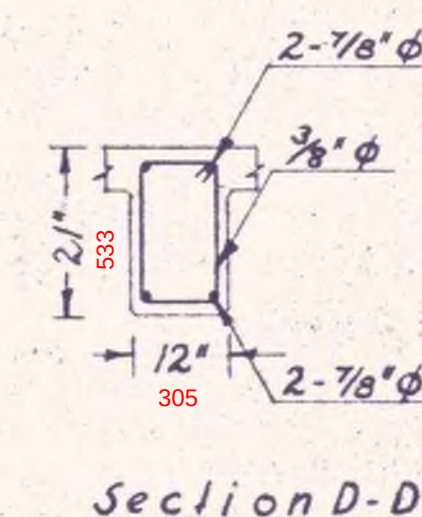
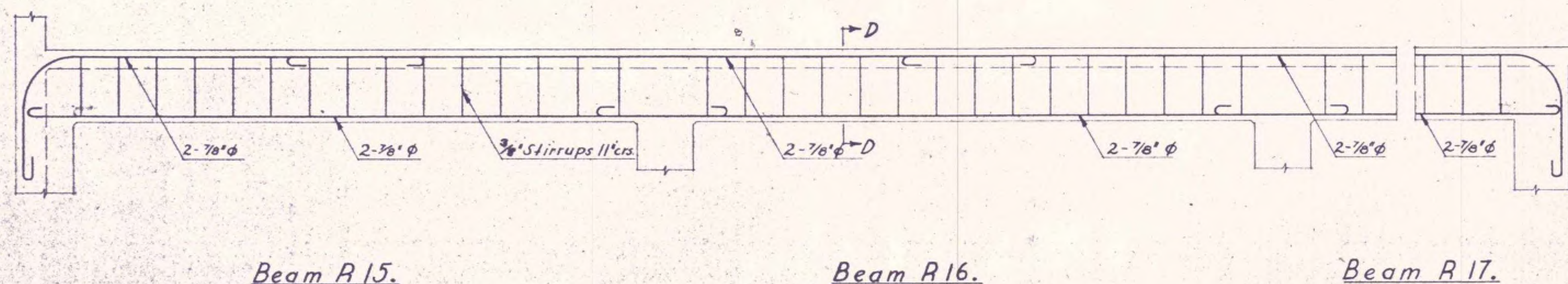
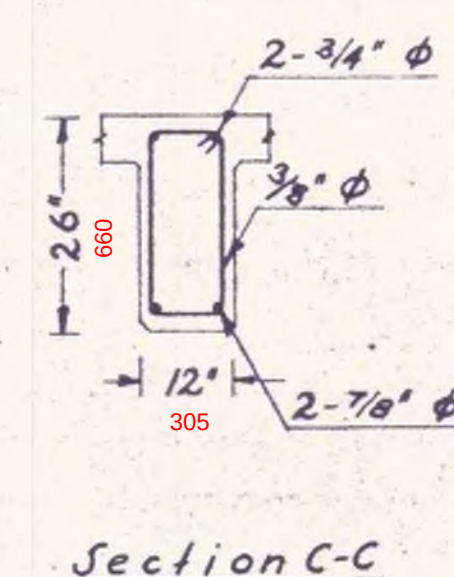
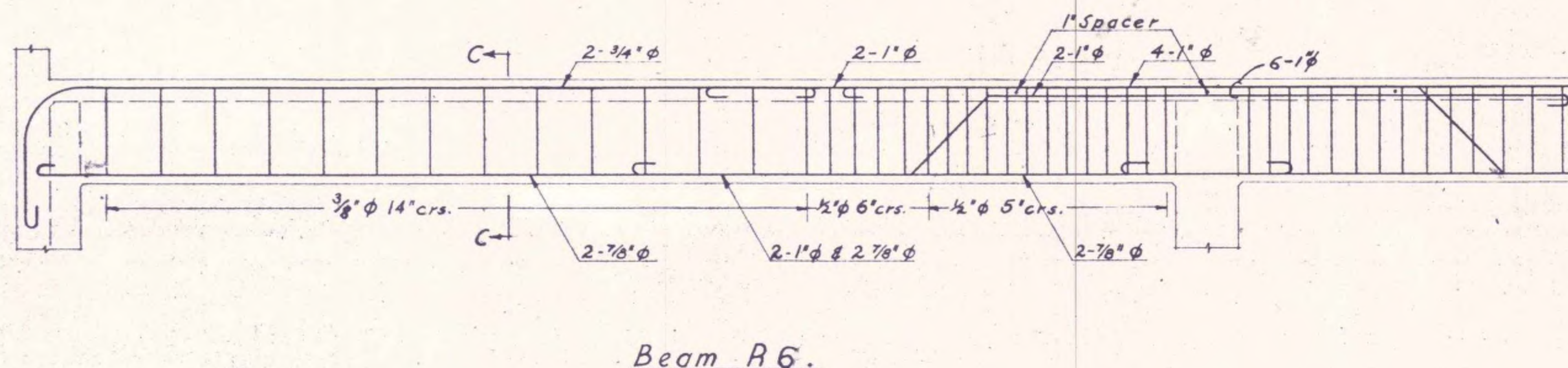
Sheet No. 15
in Set of 25

P.W.D. 126351





Beams R1-R5 & R7, Beams R8-R11 Similar but Reversed.



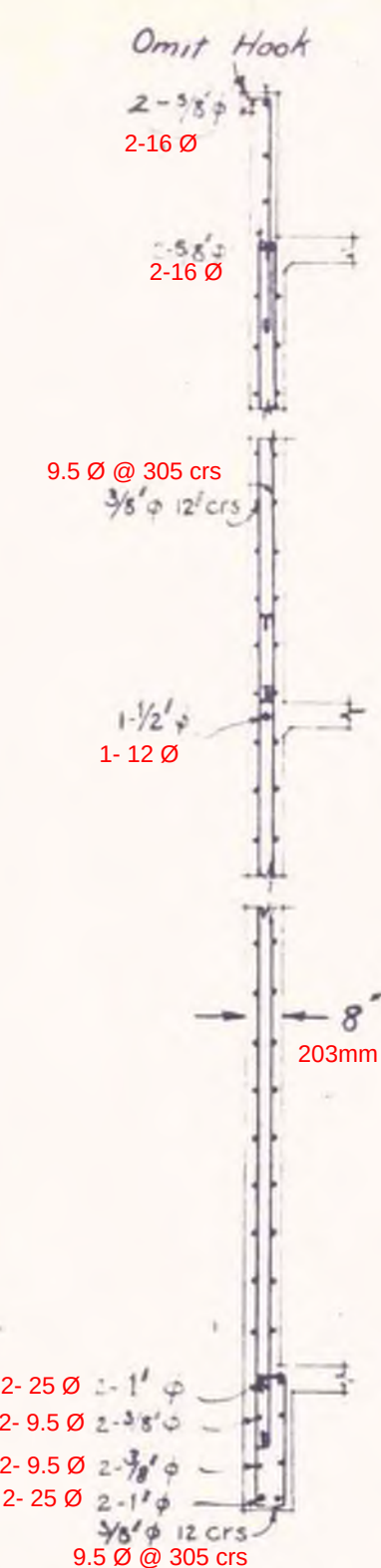
JOB No. 46/43
SHEET 16
DRAWN BY G.M.R. Sep 46
TRACED R.F. Oct 46
CHECKED J.P. April 48

SCALES
1/2" = 1 FT.

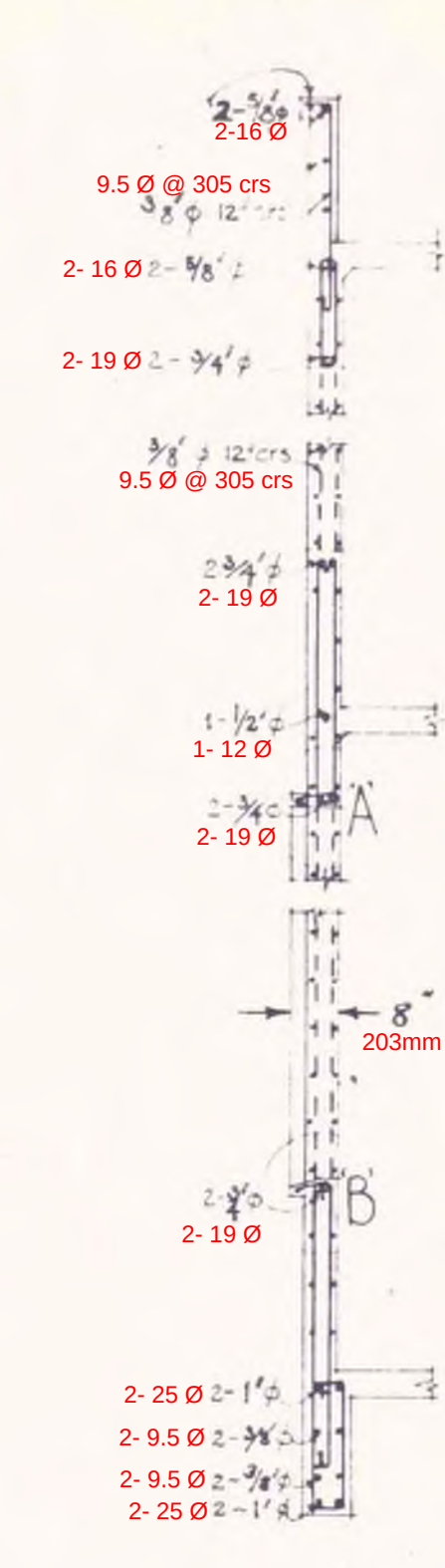
NEW POST OFFICE BUILDING HOKITIKA
ROOF BEAMS

Sheet No. 16
in Set of 25
P.W.D. 126351

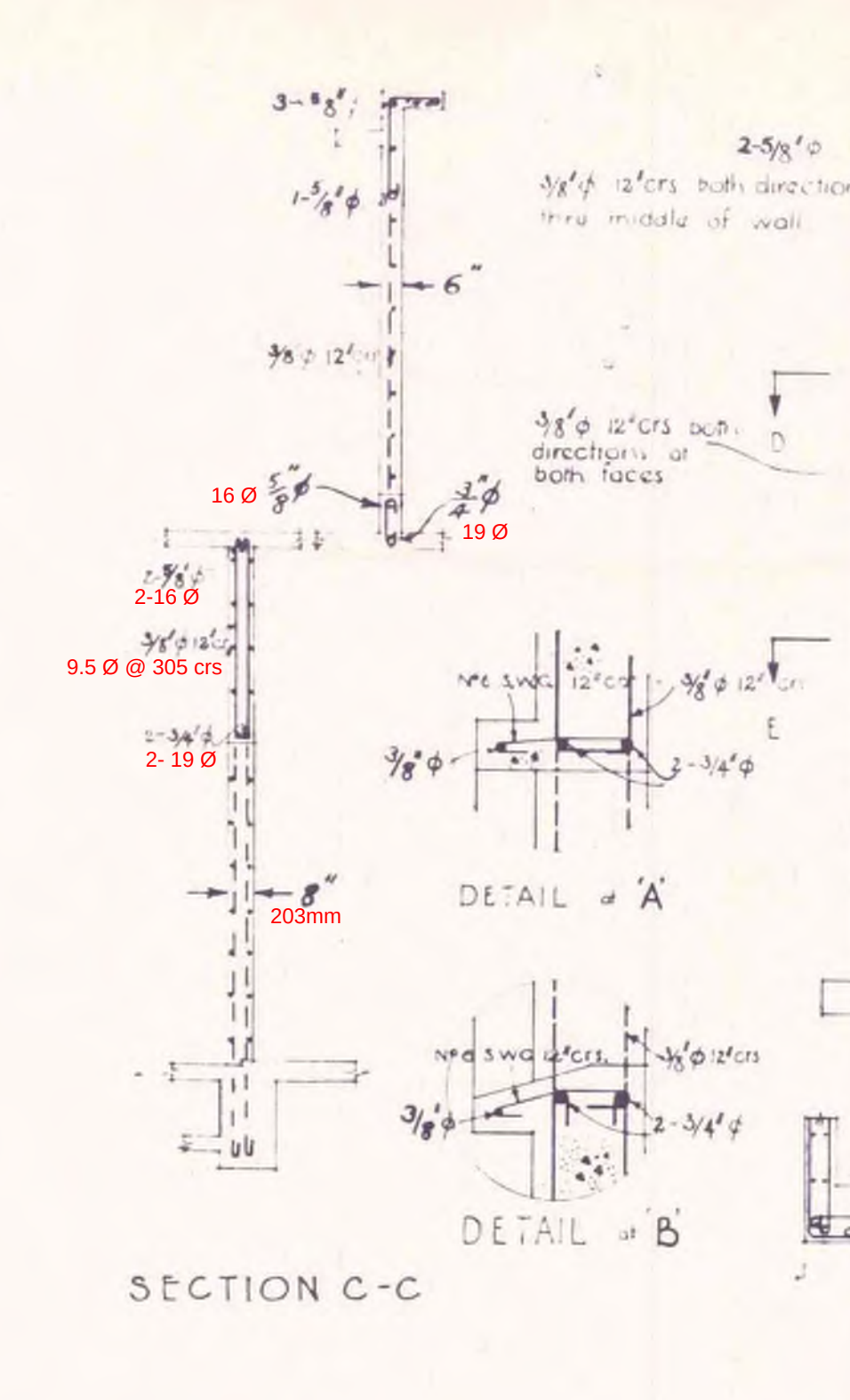




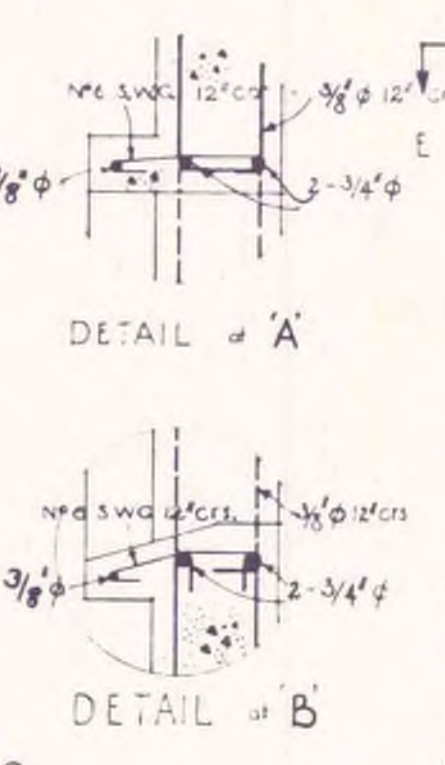
SECTION A-A



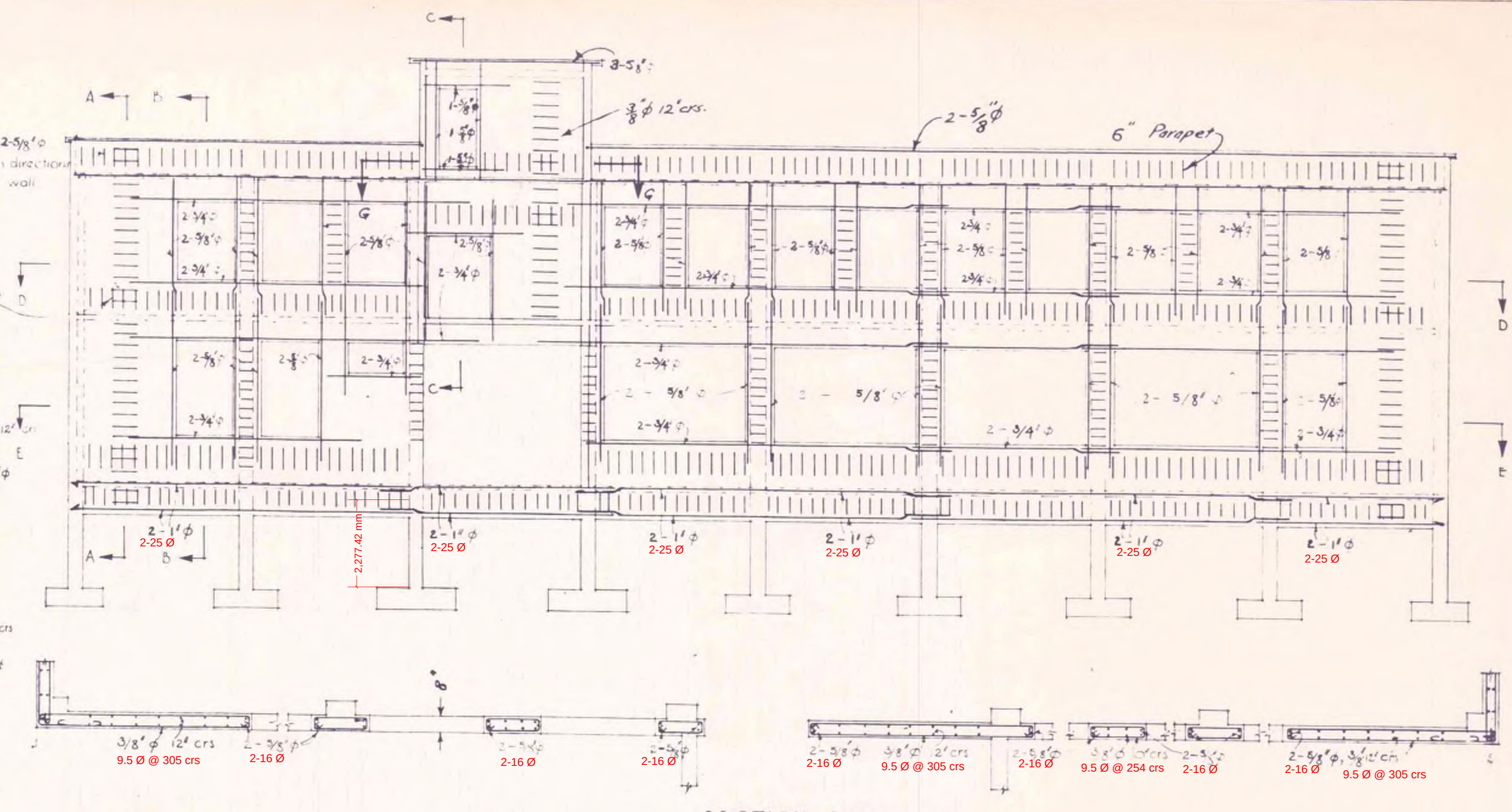
SECTION B-B



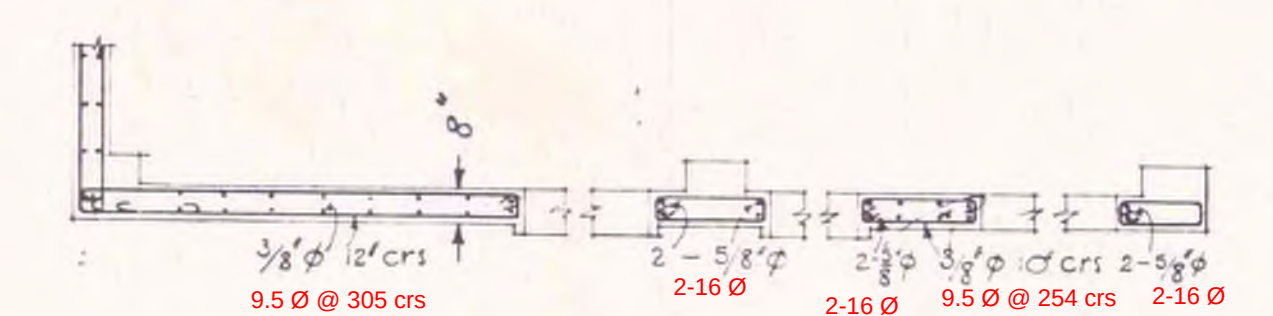
SECTION C-C



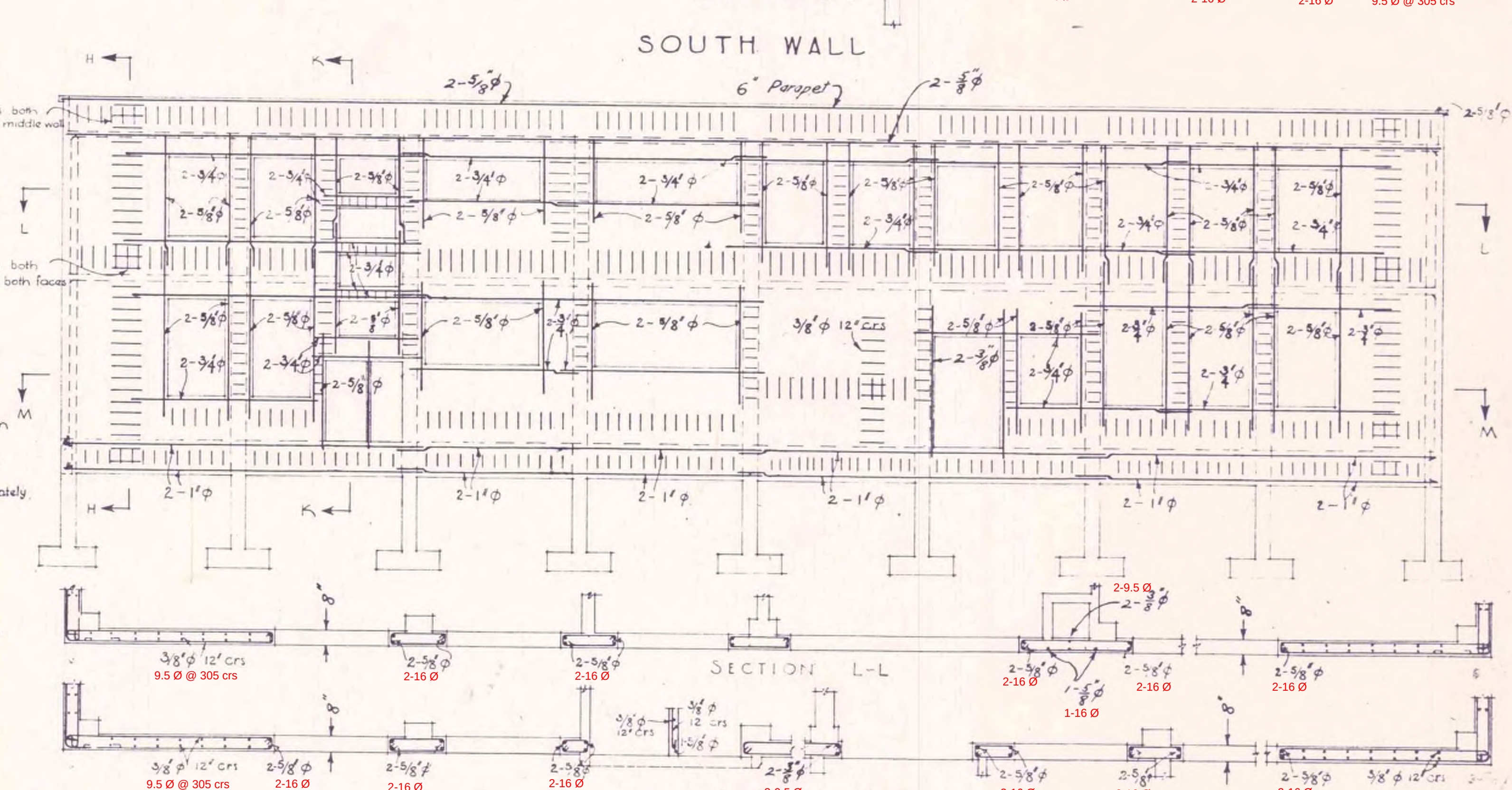
DETAIL A



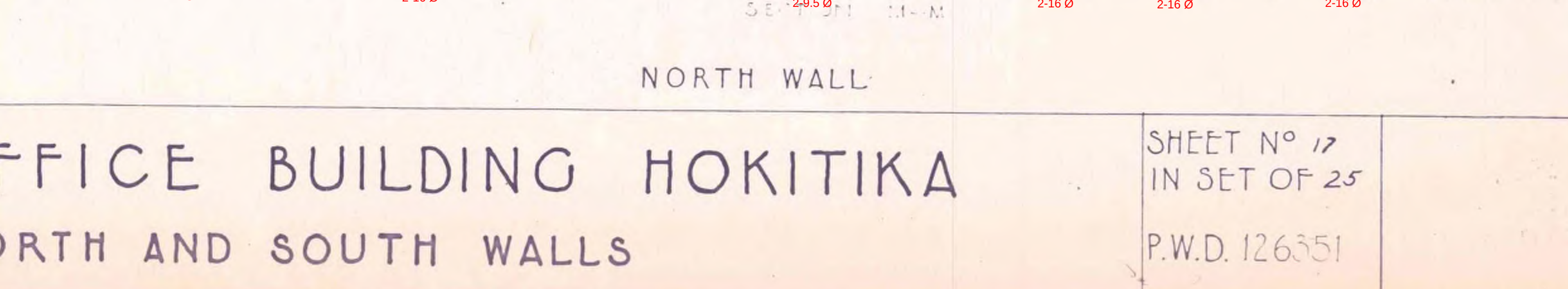
SECTION D-D



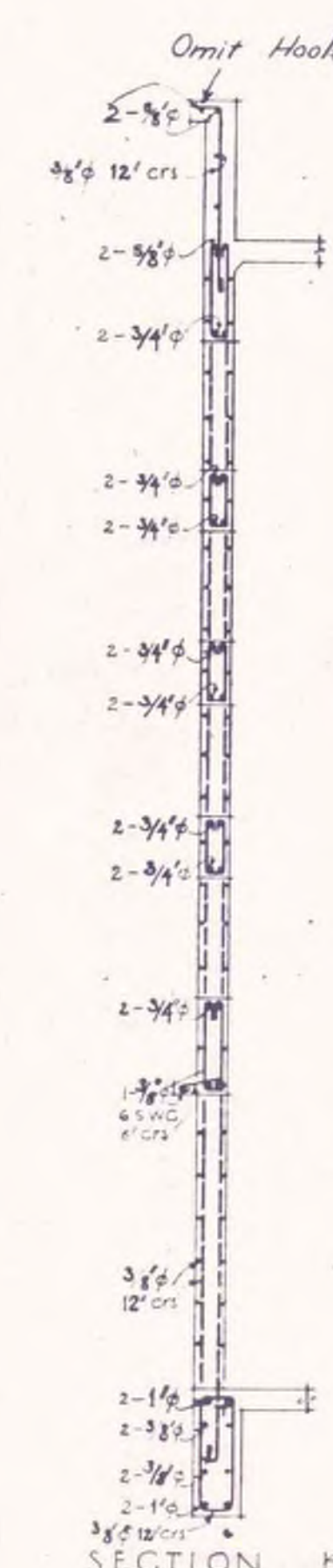
SECTION E-E



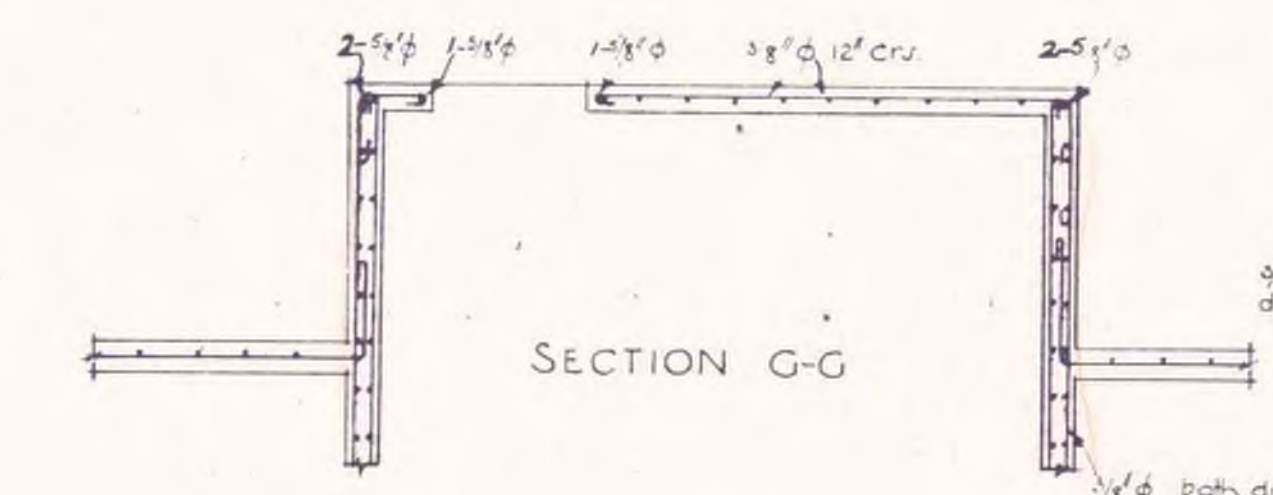
SOUTH WALL



NORTH WALL



SECTION K-K



SECTION G-G

SECTION H-H Similar to SECTION A-A

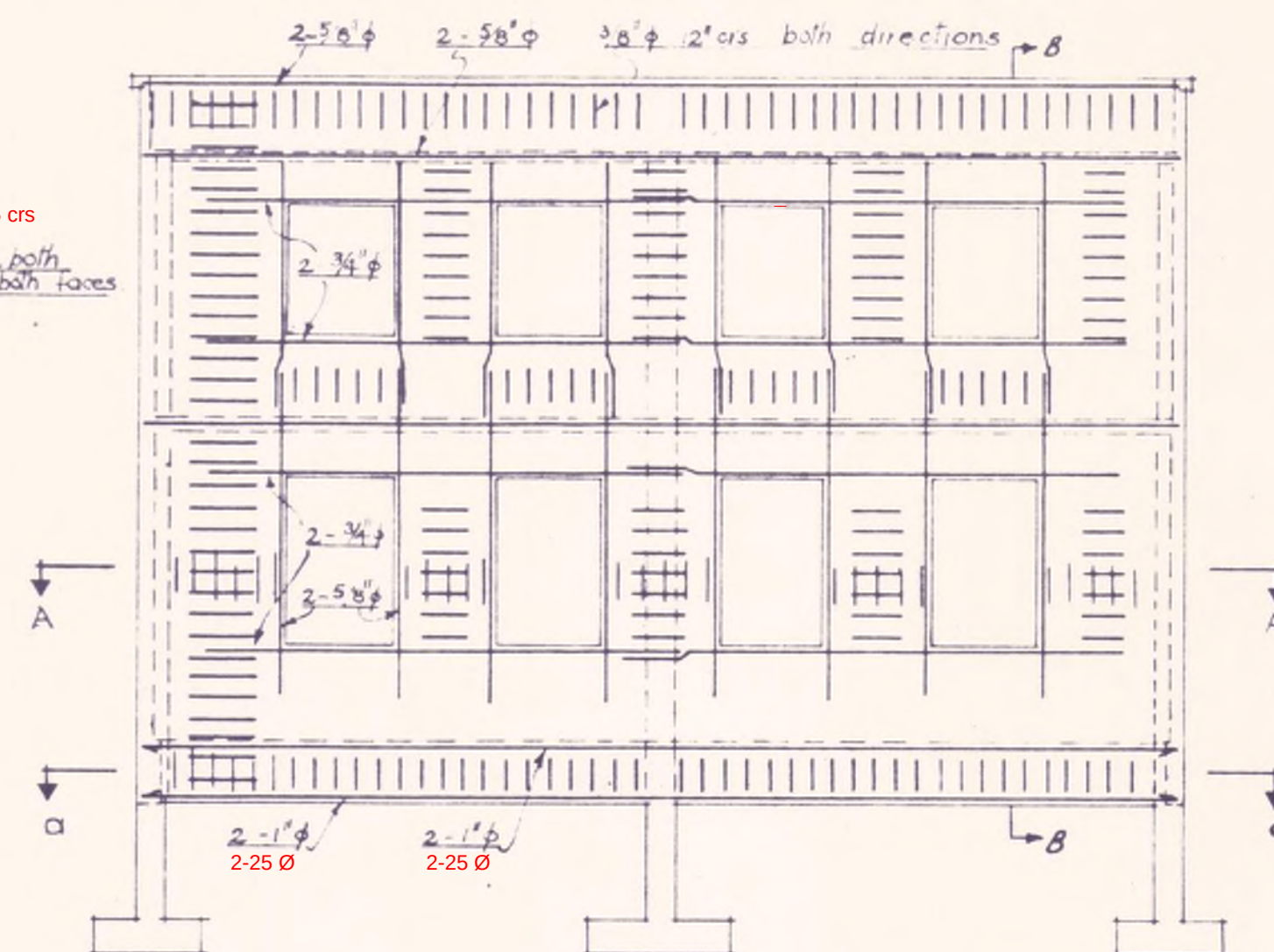
- 1 LAPS AND ANCHORAGE OF WALL REINFORCEMENT.**
- a All bars except heavy bars round wall openings, shall be hooked with standard hooks.
 - b For bars with hooks laps shall be 40 diameters in length.
 - c For bars without hooks laps shall be 54 diameters in length.
 - d Heavy horizontal bars at openings shall be in lengths as long as possible. Where laps in these bars are necessary they shall occur in piers between openings and shall be formed by joggling one bar above the other.
 - e Laps in 3/8" diameter horizontal wall reinforcement shall be staggered along the length of the wall.
 - f Laps in 3/8" diameter vertical wall reinforcement may be made if required immediately above ground floor slab level.
 - f Horizontal bars at openings shall be anchored 54 diam. Vertical bars at openings shall be anchored 4C diameters.
- 2 COVER.**
- a Cover for 3/8" wall reinforcement shall 1/4" min & 1 1/2" max. for exterior face of wall and 3/4" min and 1" max for interior face of wall. These covers shall be increased below ground level by placing horizontal steel on outside above ground level and on inside below ground level.
 - b Main reinforcement of columns, beams and around openings shall have 1 1/2" cover.
 - c Steel in column footings shall have 3" cover. Steel in wall footings shall have 2" cover.
 - d Normal roof slab, floor slab, column and footing reinforcing omitted for clarity, on elevation details.
 - e All Slab steel to have a minimum of 3/4" cover at top and bottom.

46/43
 17
 O.M.K.
 H.E.G. DEC 97
 T.H.S. JAN 98
 J.B. APR 98

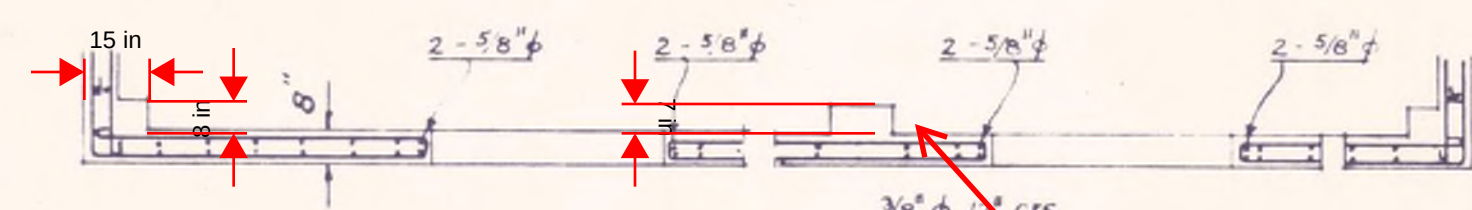
NEW POST OFFICE BUILDING HOKITIKA NORTH AND SOUTH WALLS

SHEET N° 17
 IN SET OF 25
 P.W.D. 126351

9.5mm @ 305 crs
3/8" φ 12" crs both directions both faces

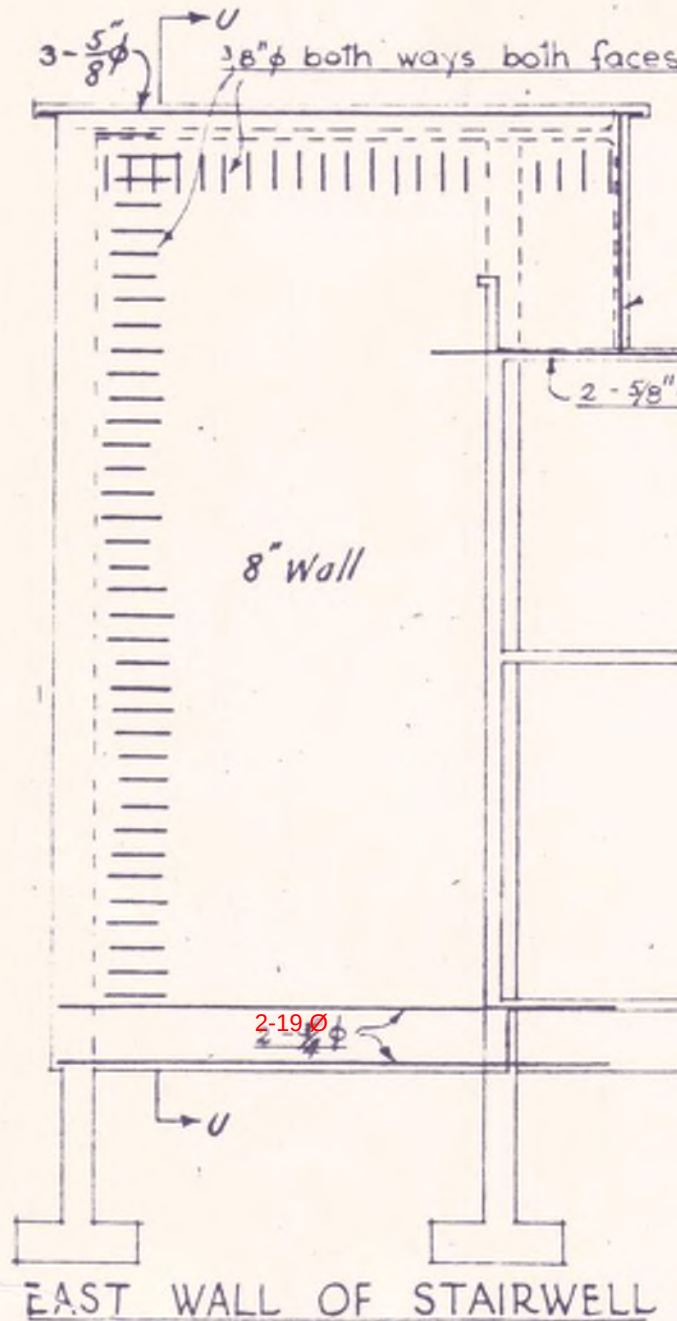


EAST AND WEST WALLS

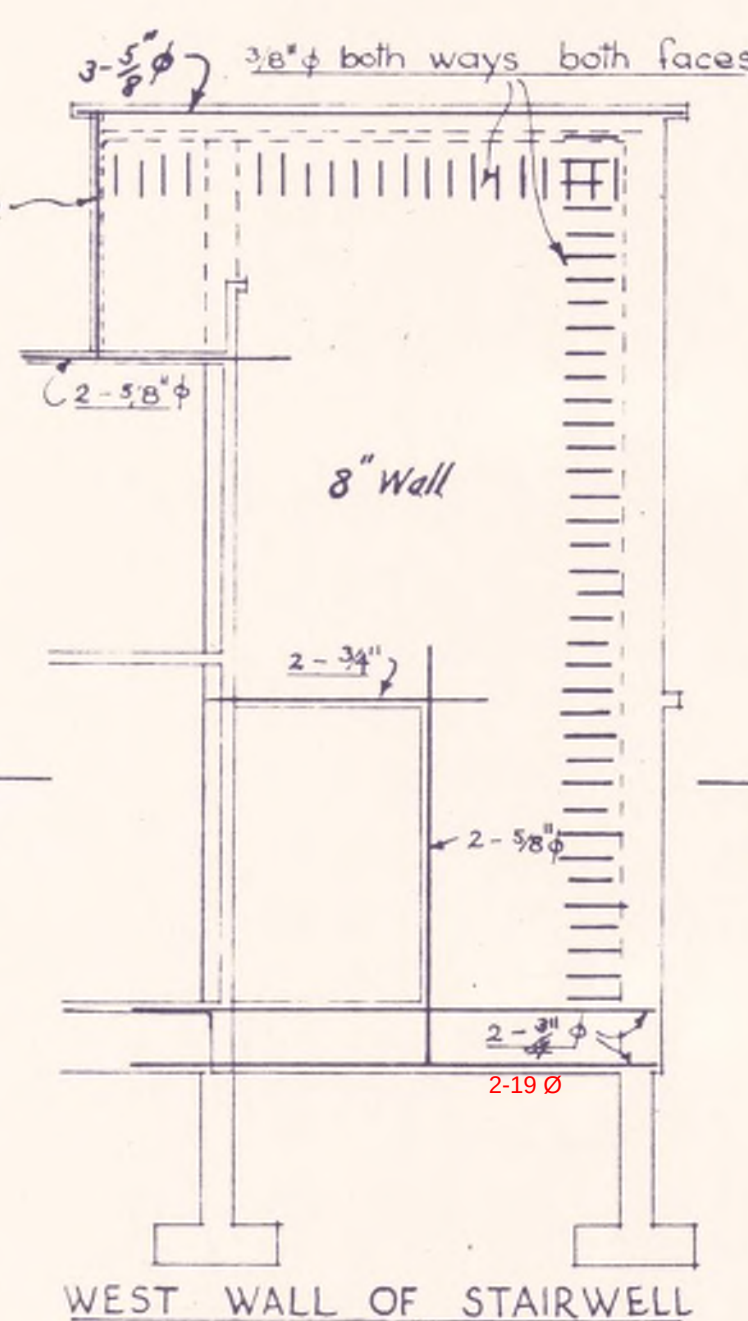


SECTION A-A

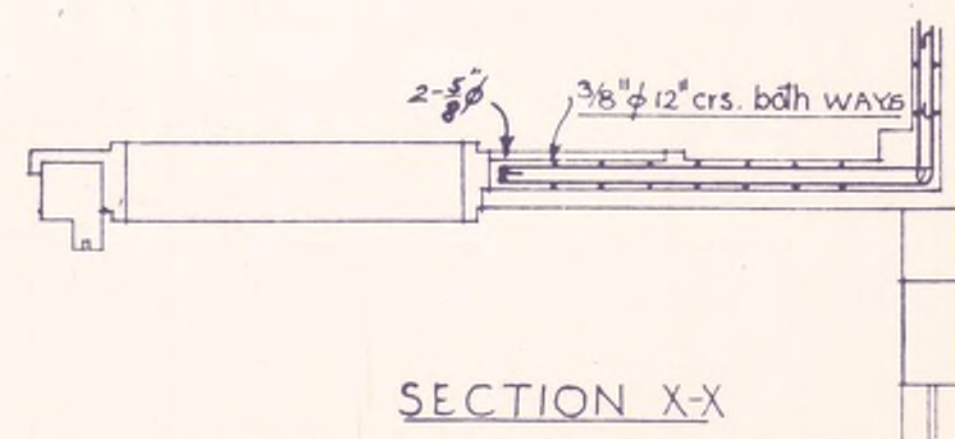
Add 23.7 kN to wall
Wt for columns
8.5m/2 * 15in*8in*3
*24kN/m3 = 23.7 kN



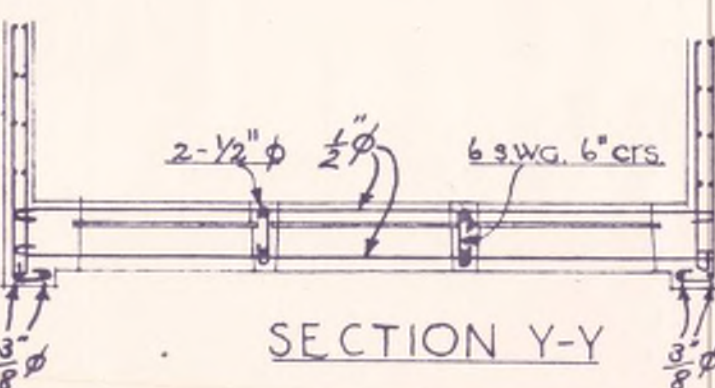
EAST WALL OF STAIRWELL



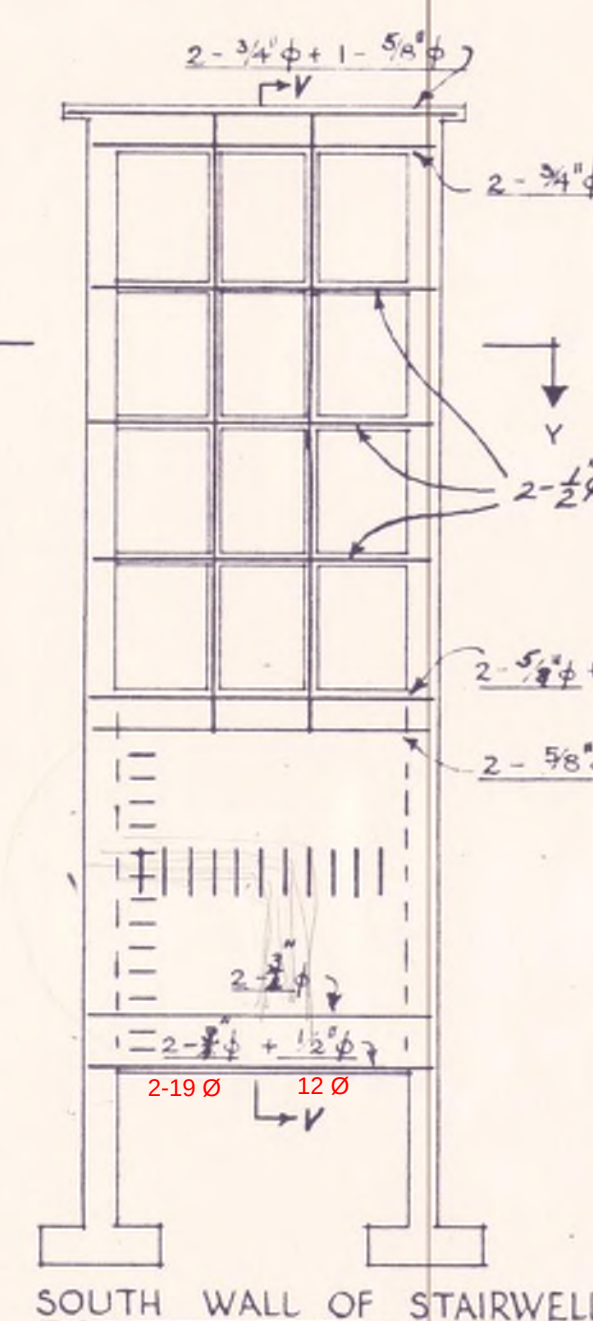
WEST WALL OF STAIRWELL



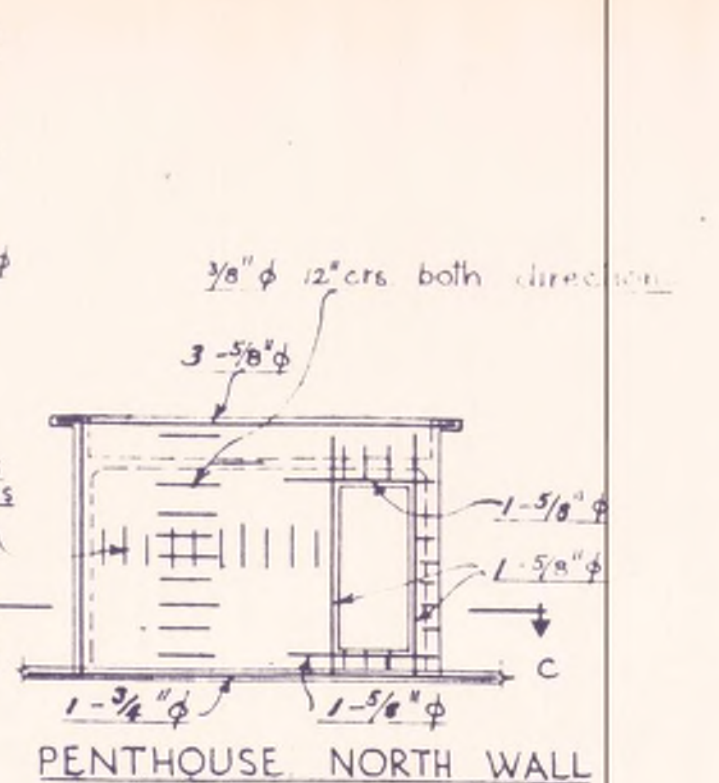
SECTION X-X



SECTION Y-Y



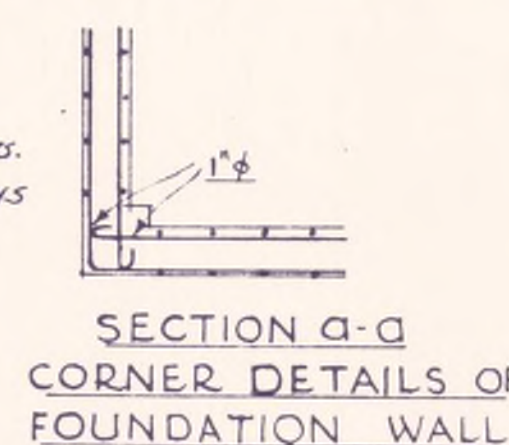
SOUTH WALL OF STAIRWELL



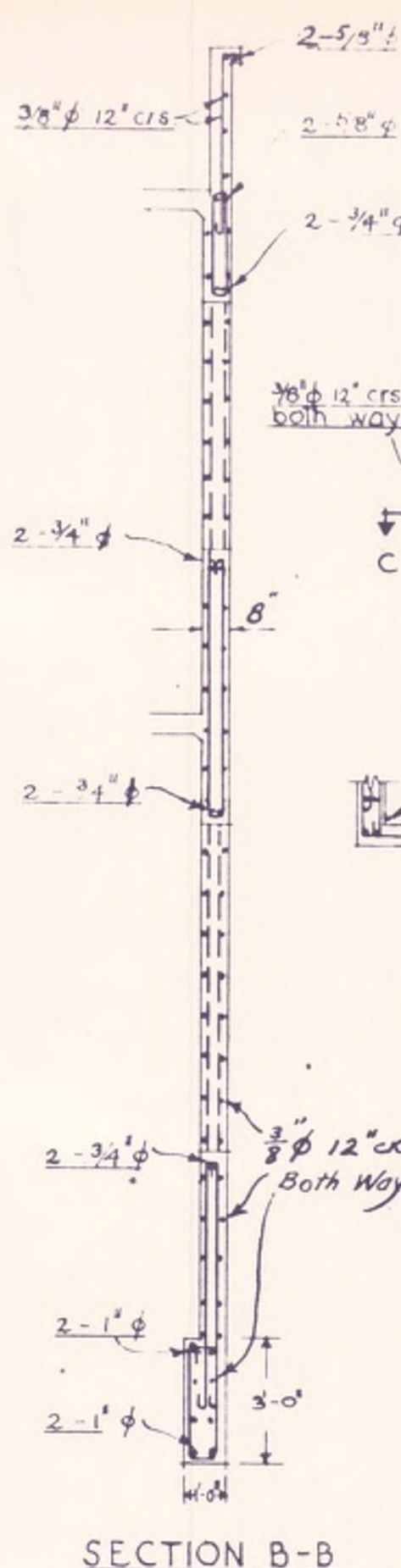
PENTHOUSE NORTH WALL



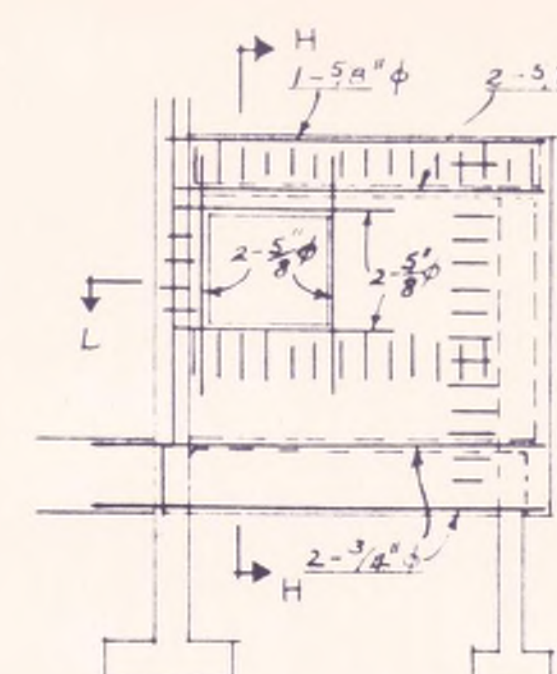
SECTION C-C
See Also Sheet 16



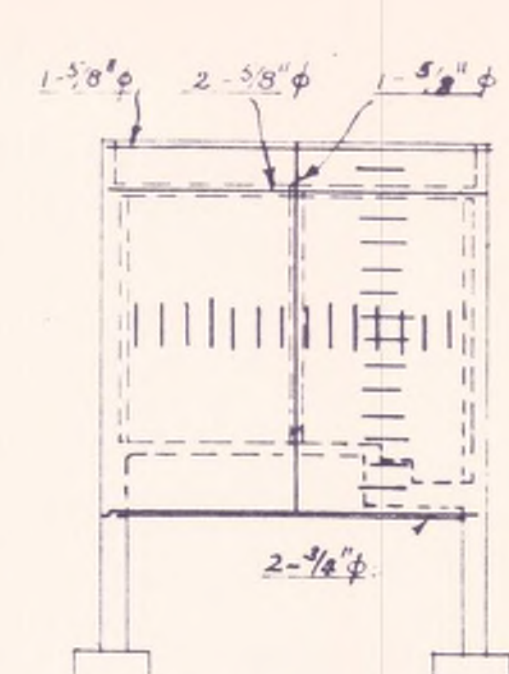
SECTION A-A
CORNER DETAILS OF
FOUNDATION WALL



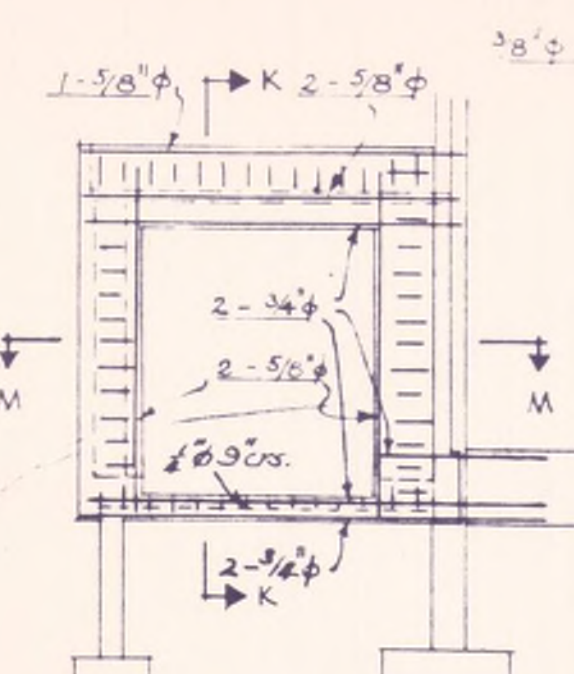
SECTION B-B



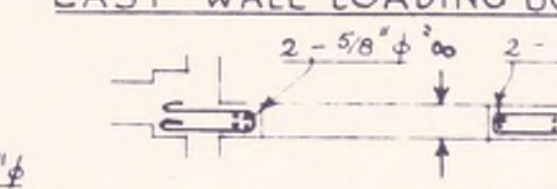
EAST WALL LOADING DOCK



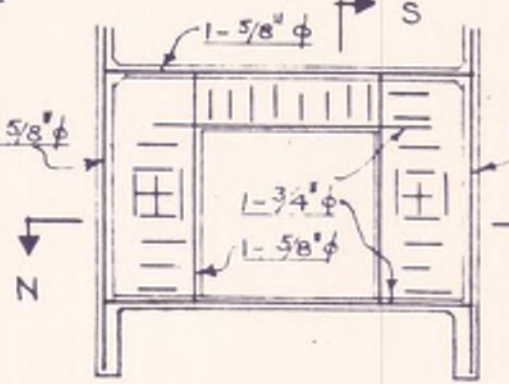
NORTH WALL LOADING DOCK



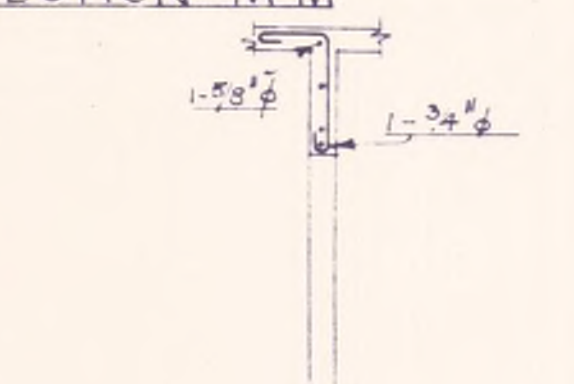
WEST WALL LOADING DOCK



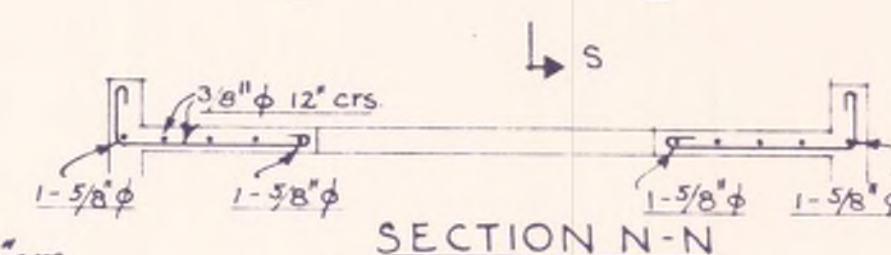
SECTION L-L



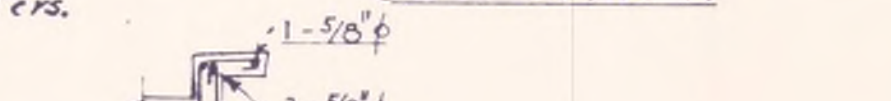
LOADING DOCK
PARTITION WALL



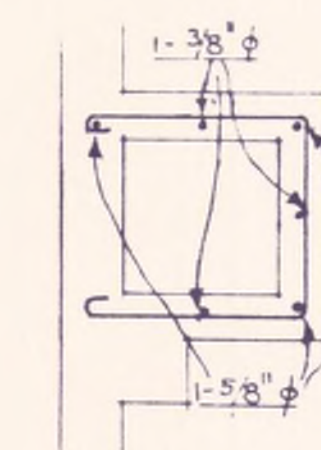
SECTION M-M



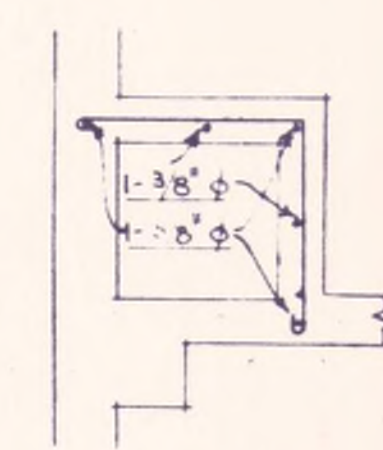
SECTION N-N



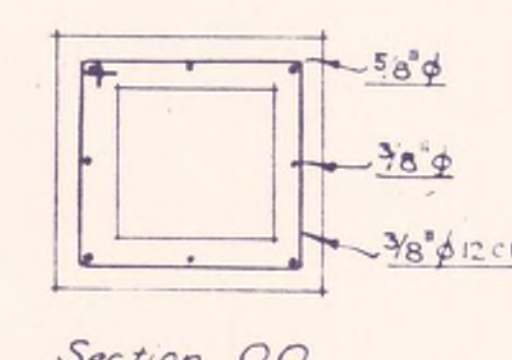
SECTION S-S



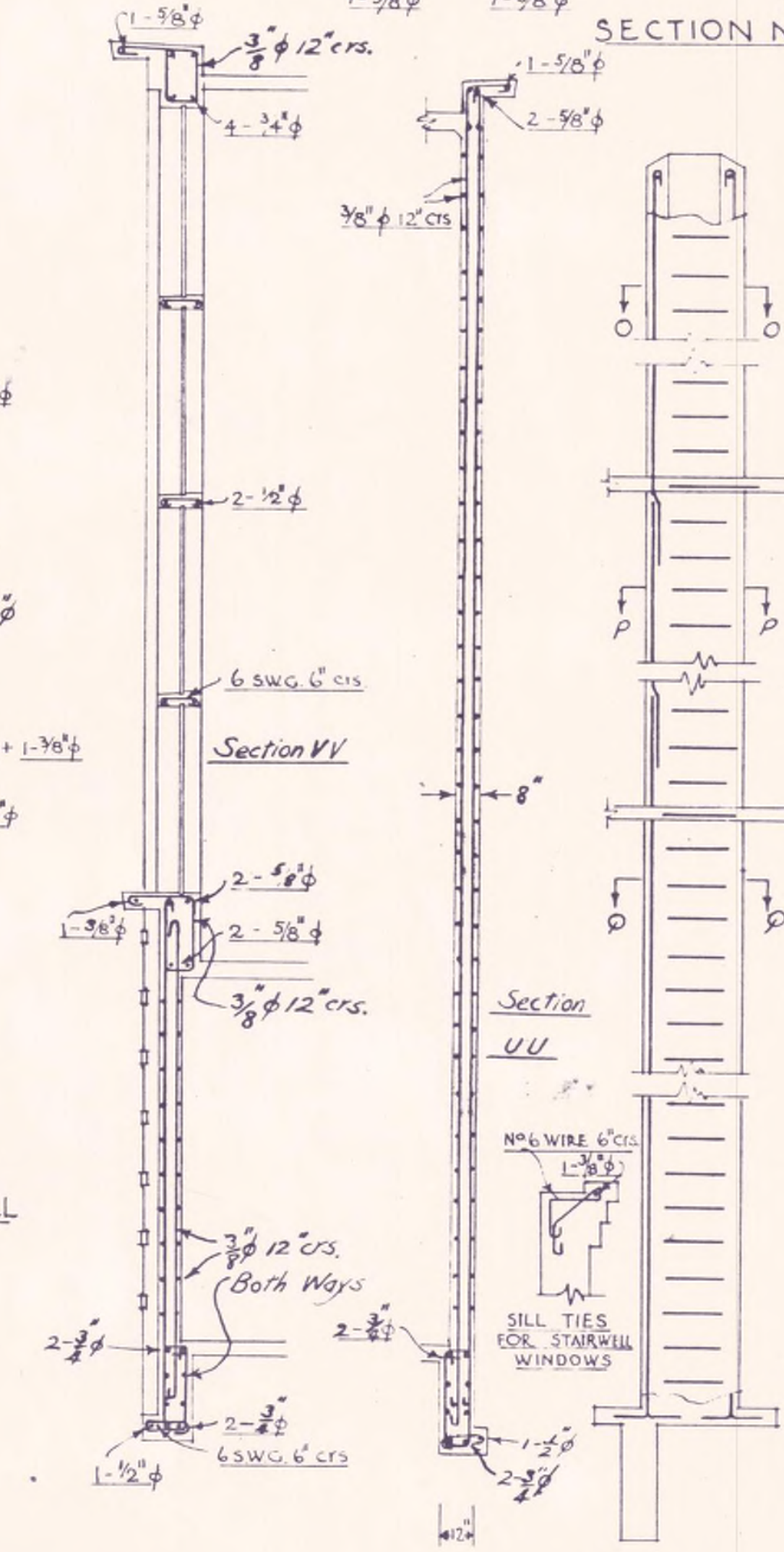
SECTION PP



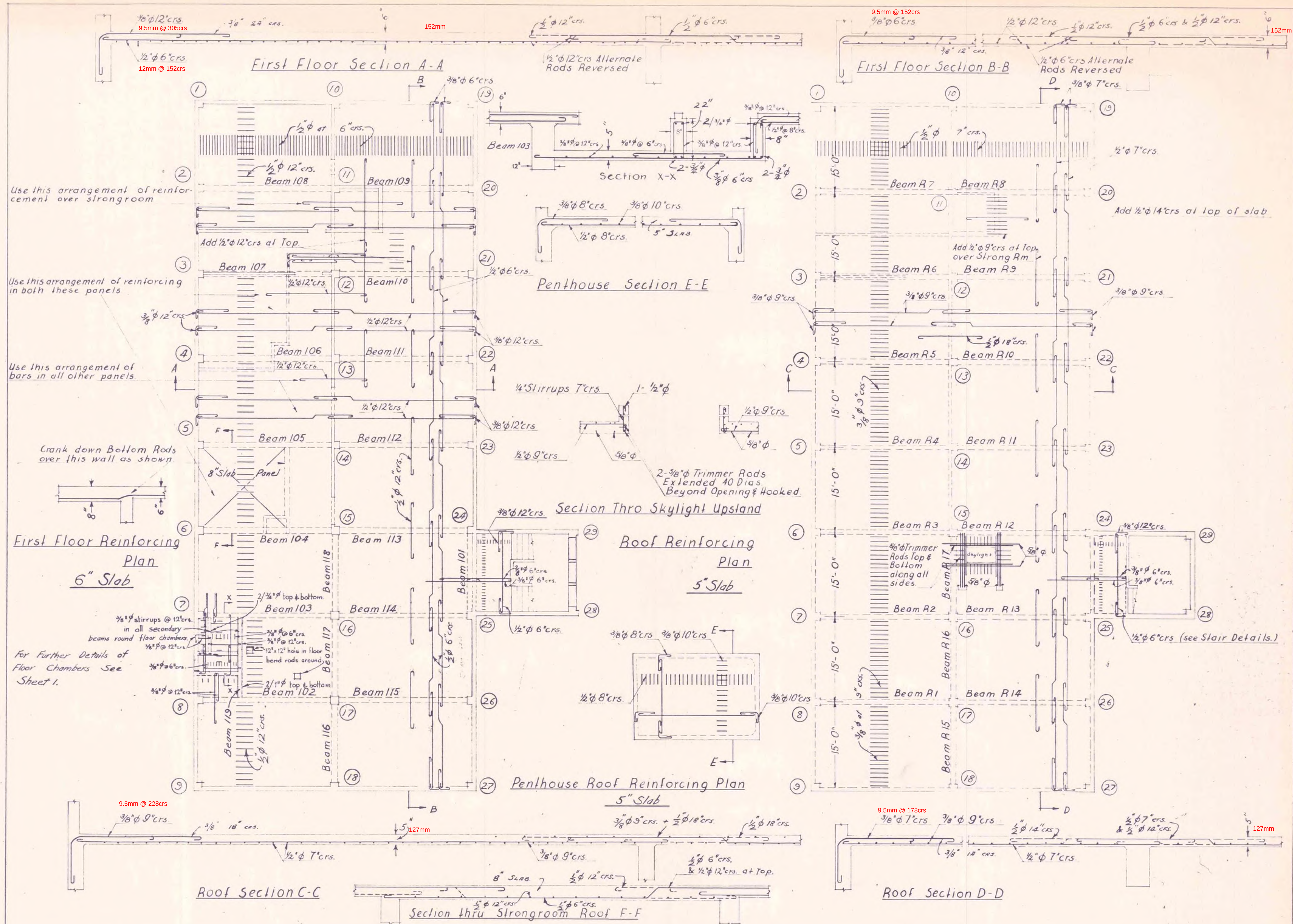
SECTION QQ



SECTION OO



Elevation of
Chimney

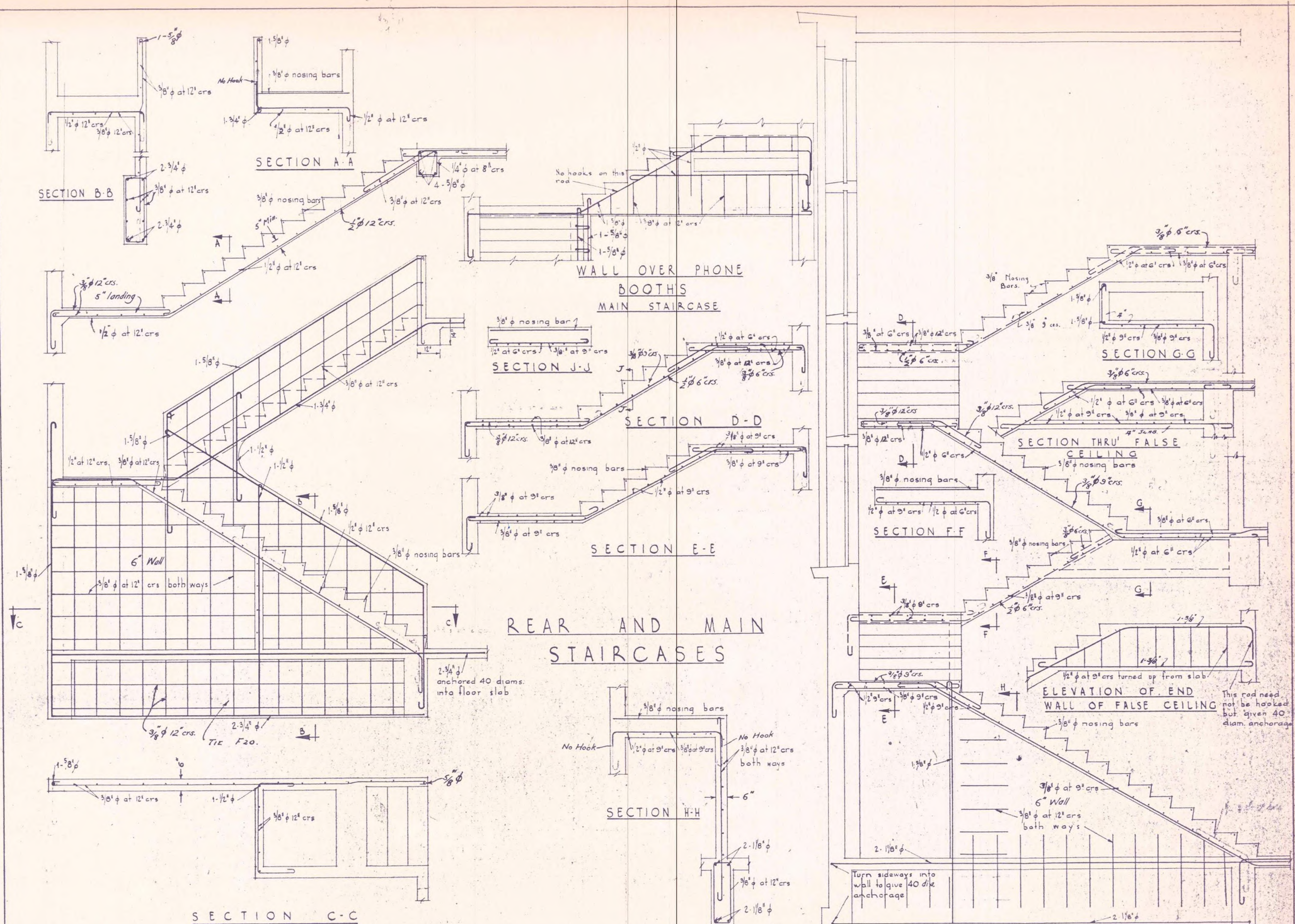


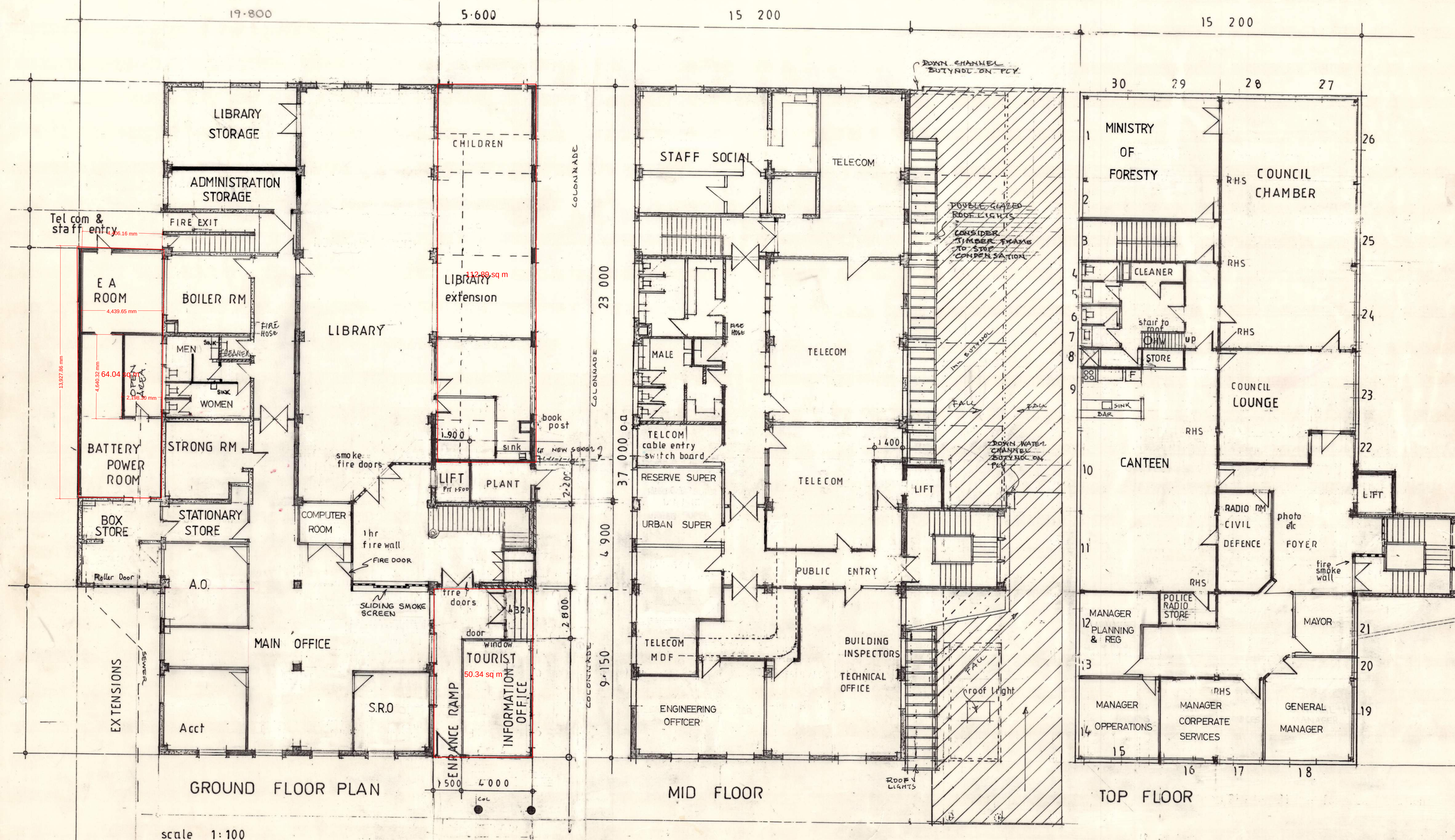
46/43
 G.M.H. Oct 40
 R.F. Oct 40
 T.M.A.
 2/3 April 78

SCALES
 1/8" = 1/2" = 1 FT.

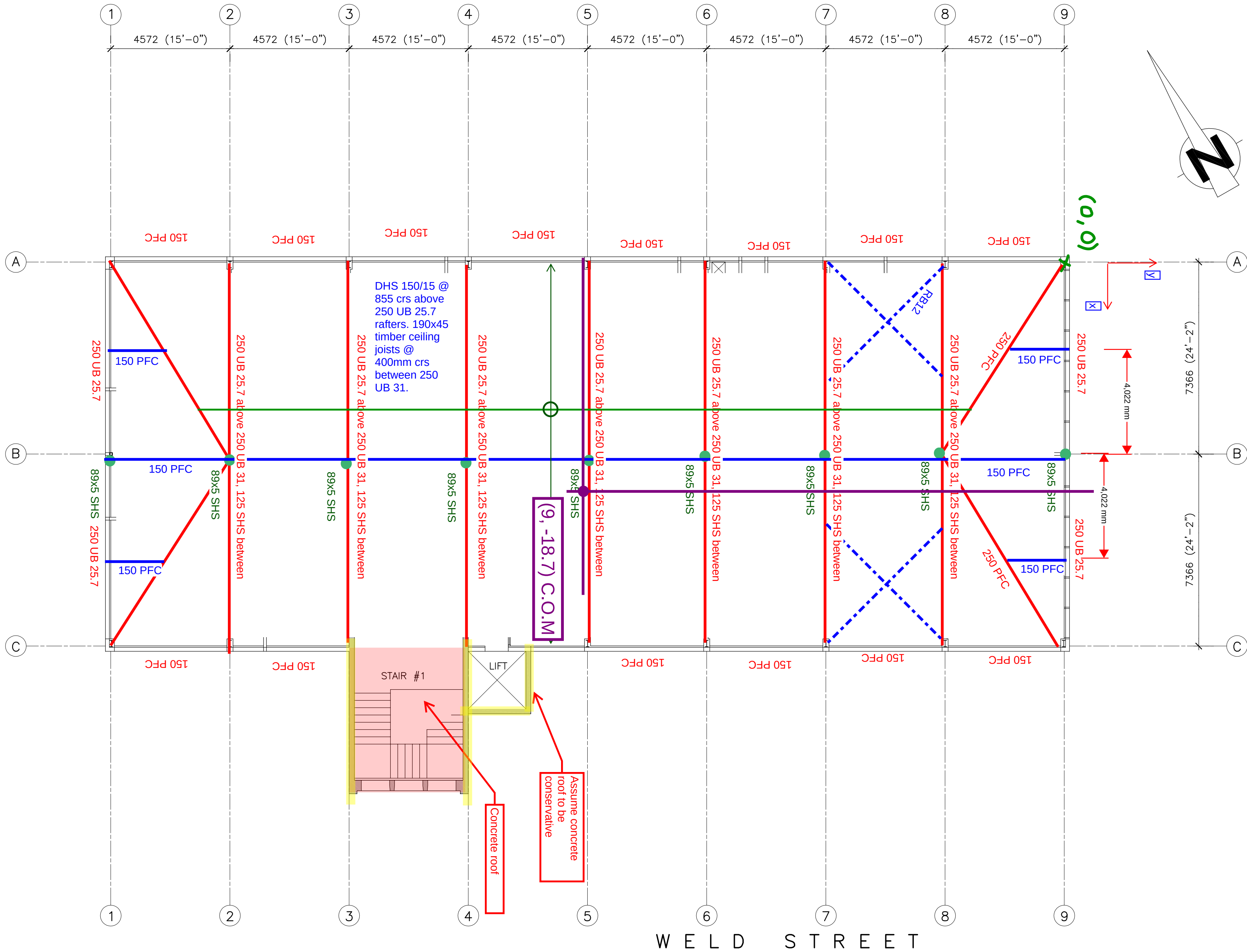
NEW POST OFFICE BUILDING HOKITIKA 1st. FLOOR, ROOF AND PENTHOUSE ROOF DETAILS

Sheet No. 19
 in Set of 25
 P.W.D. 126351



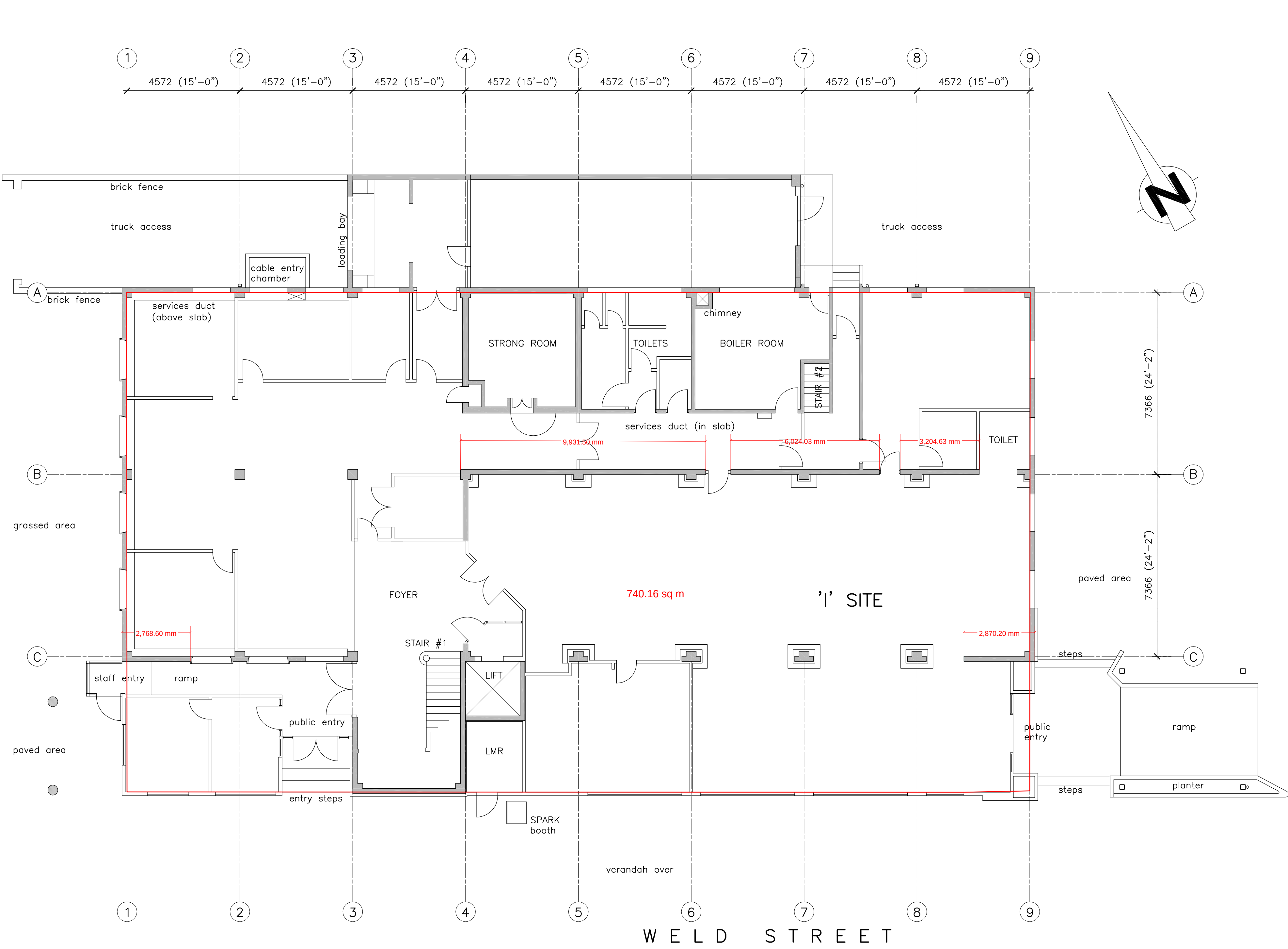


issue 22 10 91



Roof framing

ISS.	123456	qwertyuiop
DATE	REASON FOR ISSUE	
SIMCO Consulting Limited		
JOB NAME		
Westland District Council		
Building Strengthening		
Weld Street - Hokitika		
DRAWING NAME		
Existing Second Floor Plan		
Layout and Demolition		
CAD FILE	SCALE (at A2 paper size)	
AutoCAD	1 : 100	
Job No.	DRAWING	ISSUE
20050	S1.3	1



1	123456	qwertuio
ISS.	DATE	REASON FOR ISSUE
SIMCO Consulting Limited		
JOB NAME Westland District Council Building Strengthening Weld Street - Hokitika		
DRAWING NAME Existing Ground Floor Plan Layout and Demolition		
CAD FILE AutoCAD	SCALE (at A2 paper size) 1 : 100	
JOB No. 20050	DRAWING S1.1	ISSUE 1



1	123456	qwertyuiop
ISS.	DATE	REASON FOR ISSUE

SIMCO

Consulting Limited

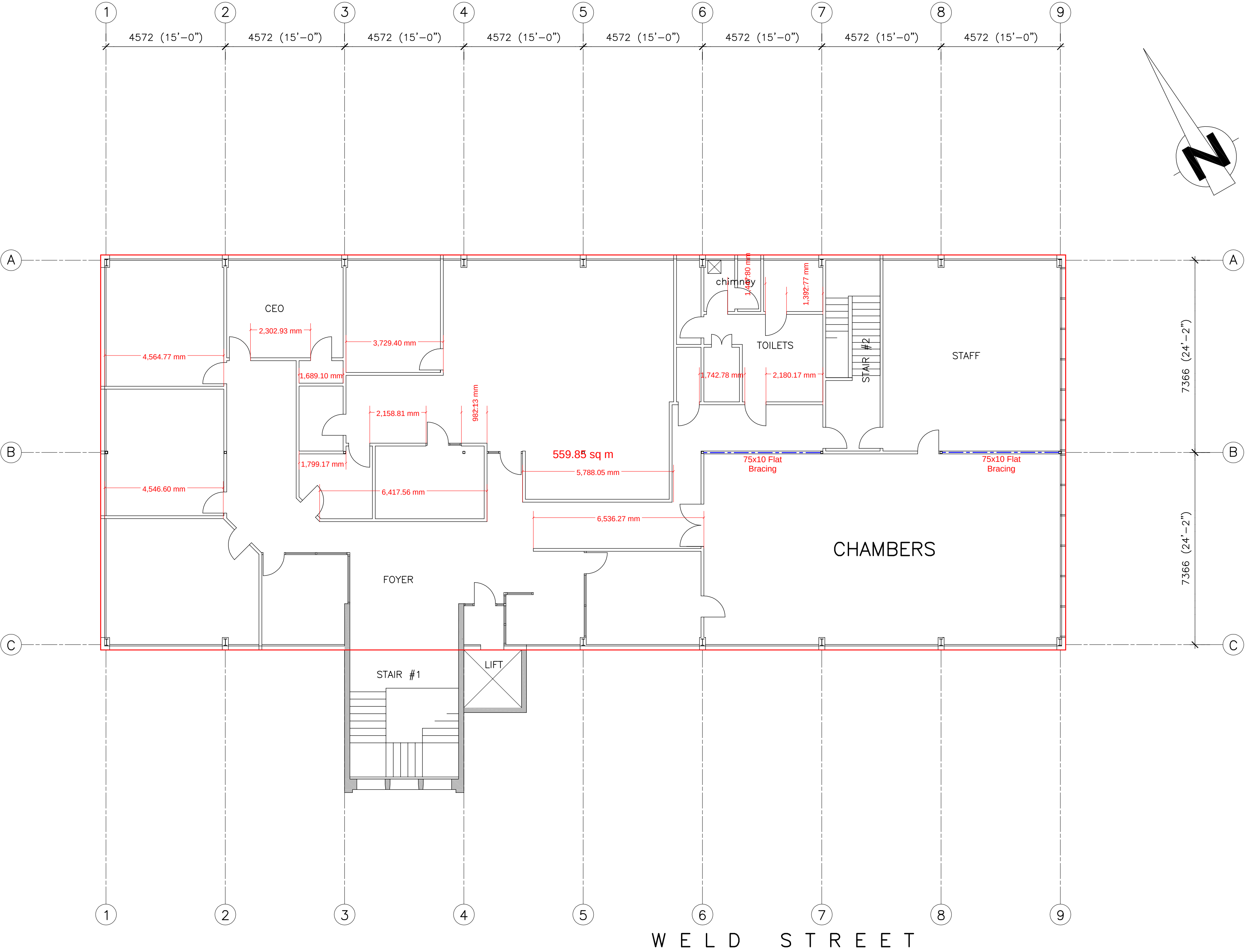
JOB NAME

Westland District Council
Building Strengthening
Weld Street - Hokitika

DRAWING NAME

Existing First Floor Plan
Layout and Demolition

CAD FILE	SCALE (at A2 paper size)	
AutoCAD	1 : 100	
JOB No.	DRAWING	ISSUE
20050	S1.2	1



1	123456	qwertyuiop
ISS.	DATE	REASON FOR ISSUE

SIMCO

Consulting Limited

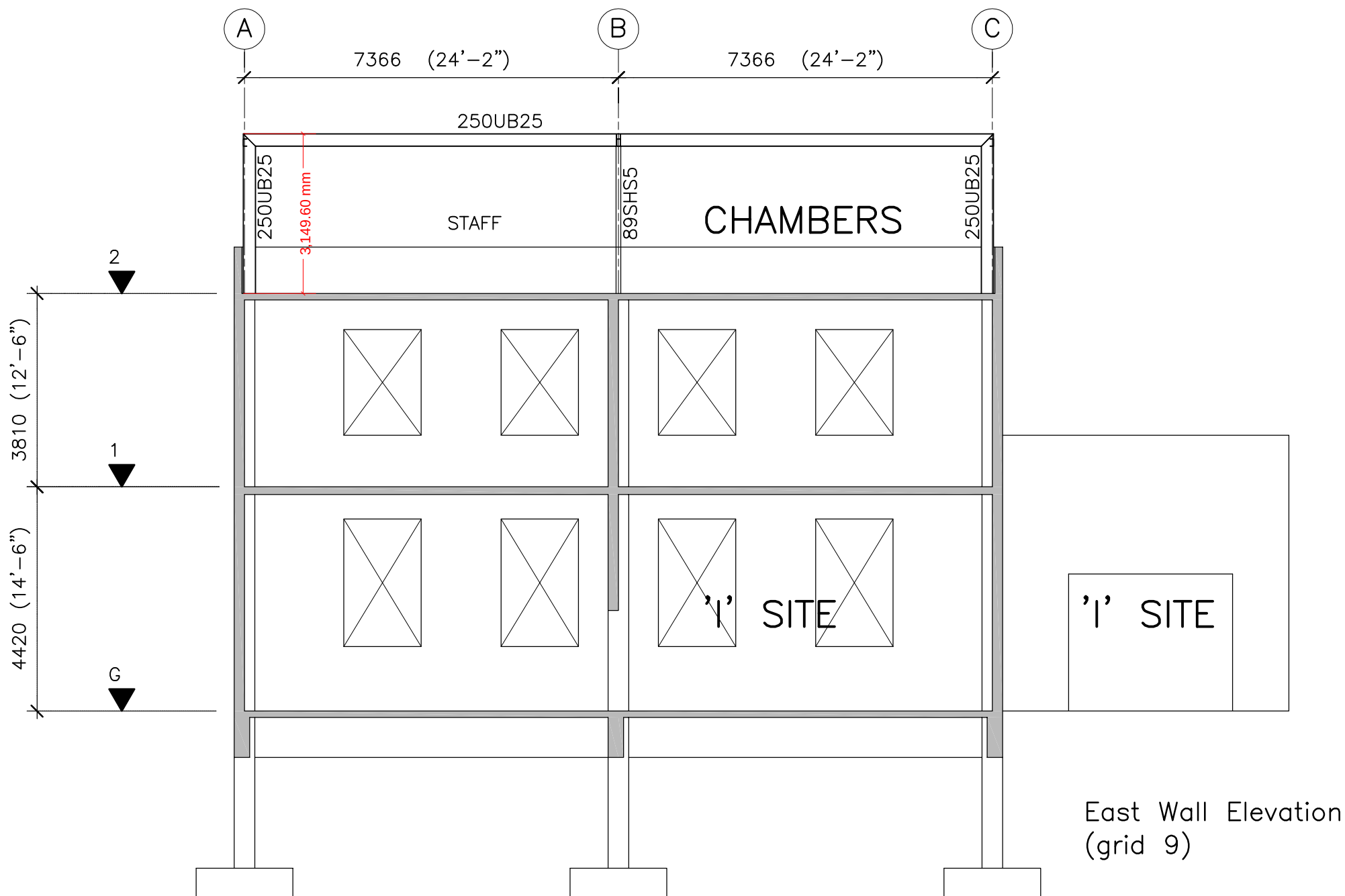
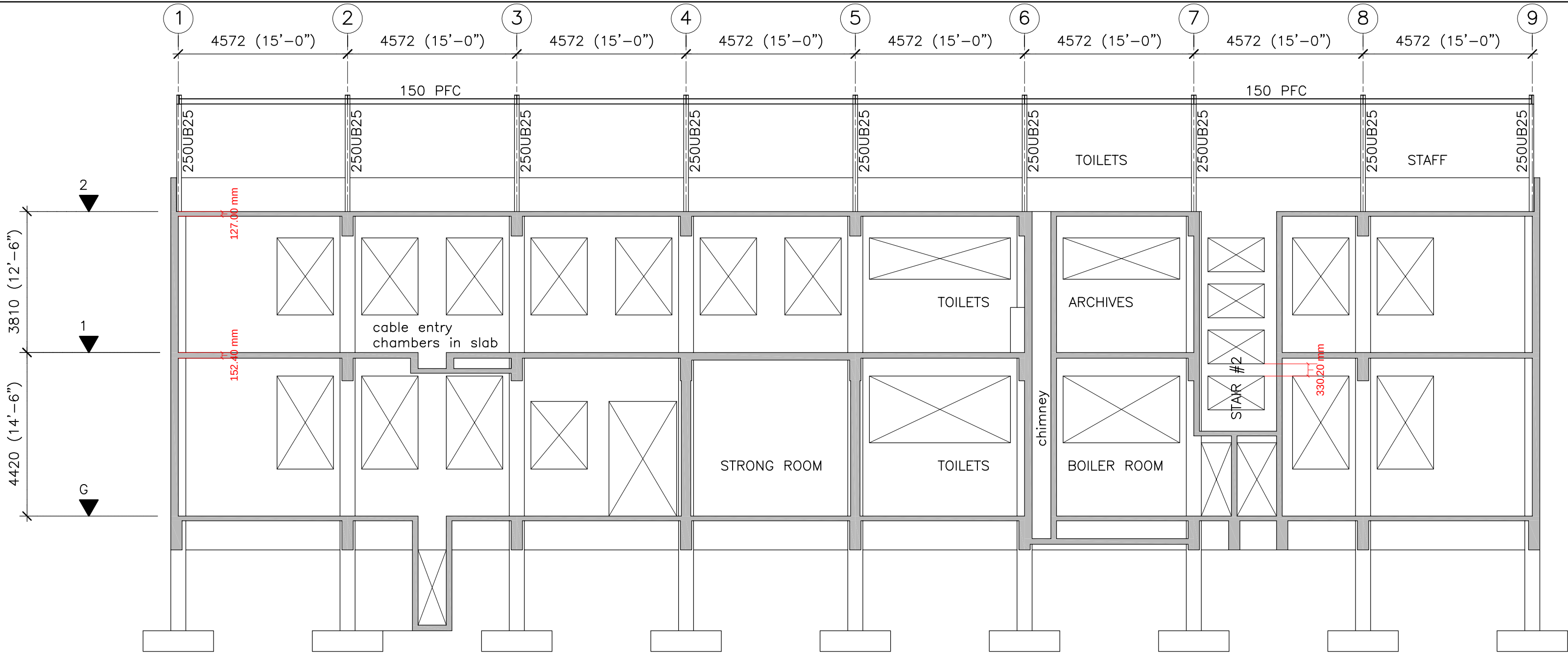
JOB NAME

Westland District Council
Building Strengthening
Weld Street - Hokitika

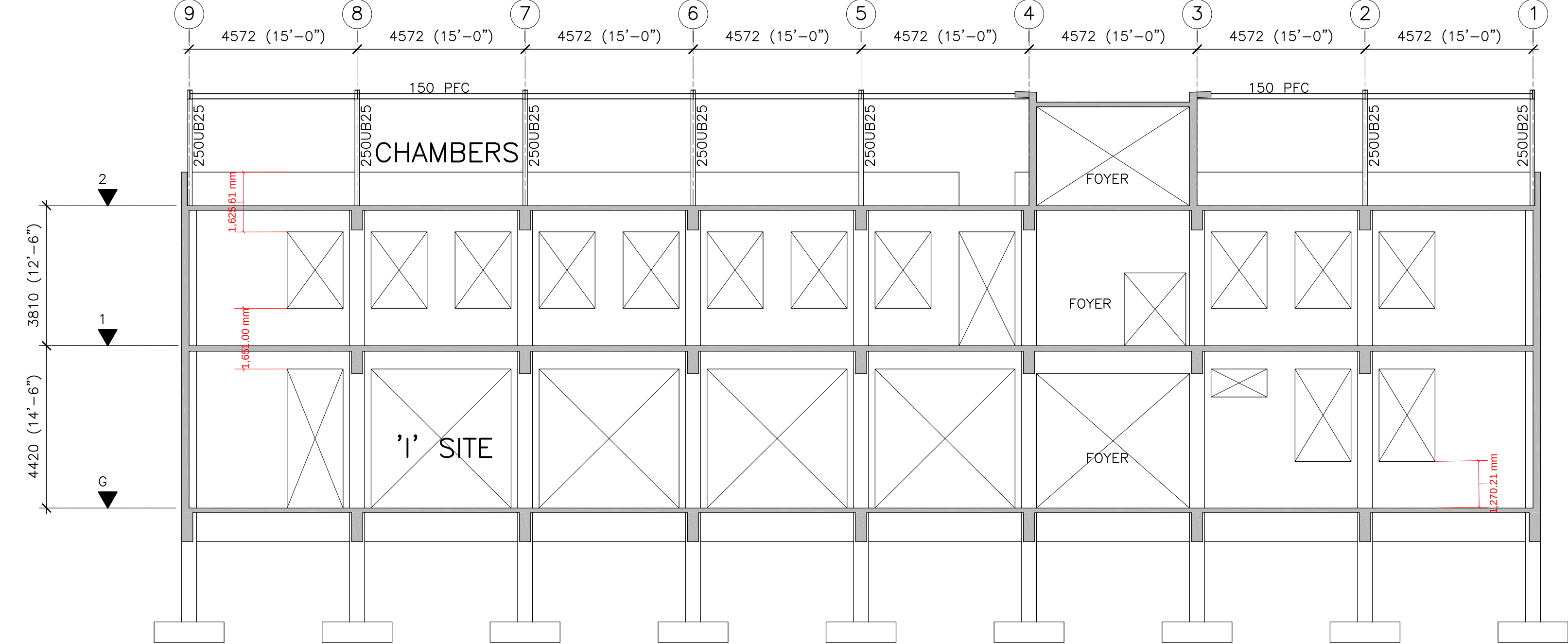
DRAWING NAME

Existing Second Floor Plan
Layout and Demolition

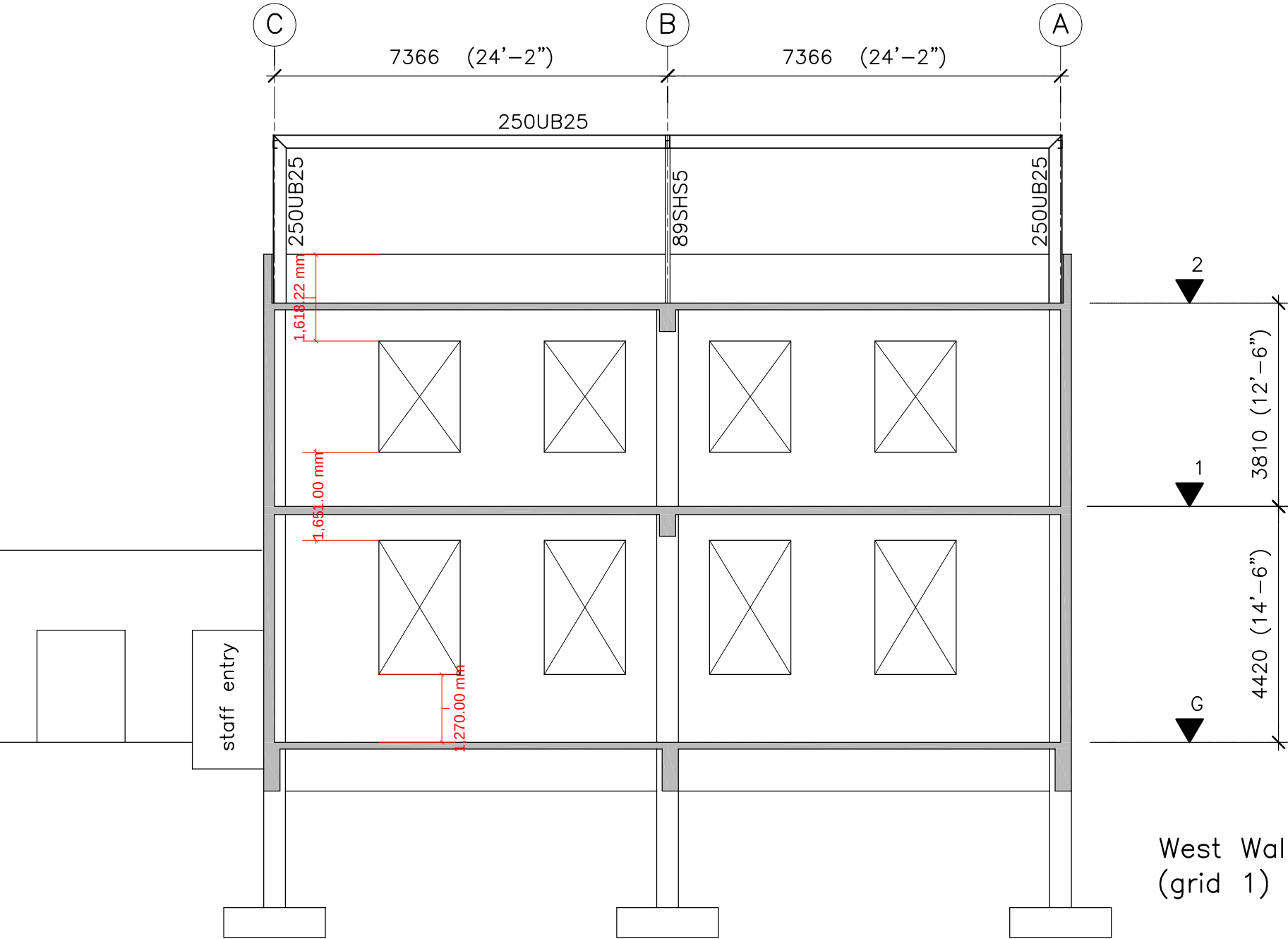
CAD FILE	SCALE (at A2 paper size)	
AutoCAD	1 : 100	
JOB No.	DRAWING	ISSUE
20050	S1.3	1



1	123456	qwertyuiop
ISS.	DATE	REASON FOR ISSUE
SIMCO		
Consulting Limited		
JOB NAME		
Westland District Council		
Building Strengthening		
Weld Street - Hokitika		
DRAWING NAME		
Existing Wall Elevations		
North, East (grids A, 9)		
CAD FILE	SCALE (at A2 paper size)	
AutoCAD	1 : 100	
JOB No.	DRAWING	ISSUE
20050	S1.4	1

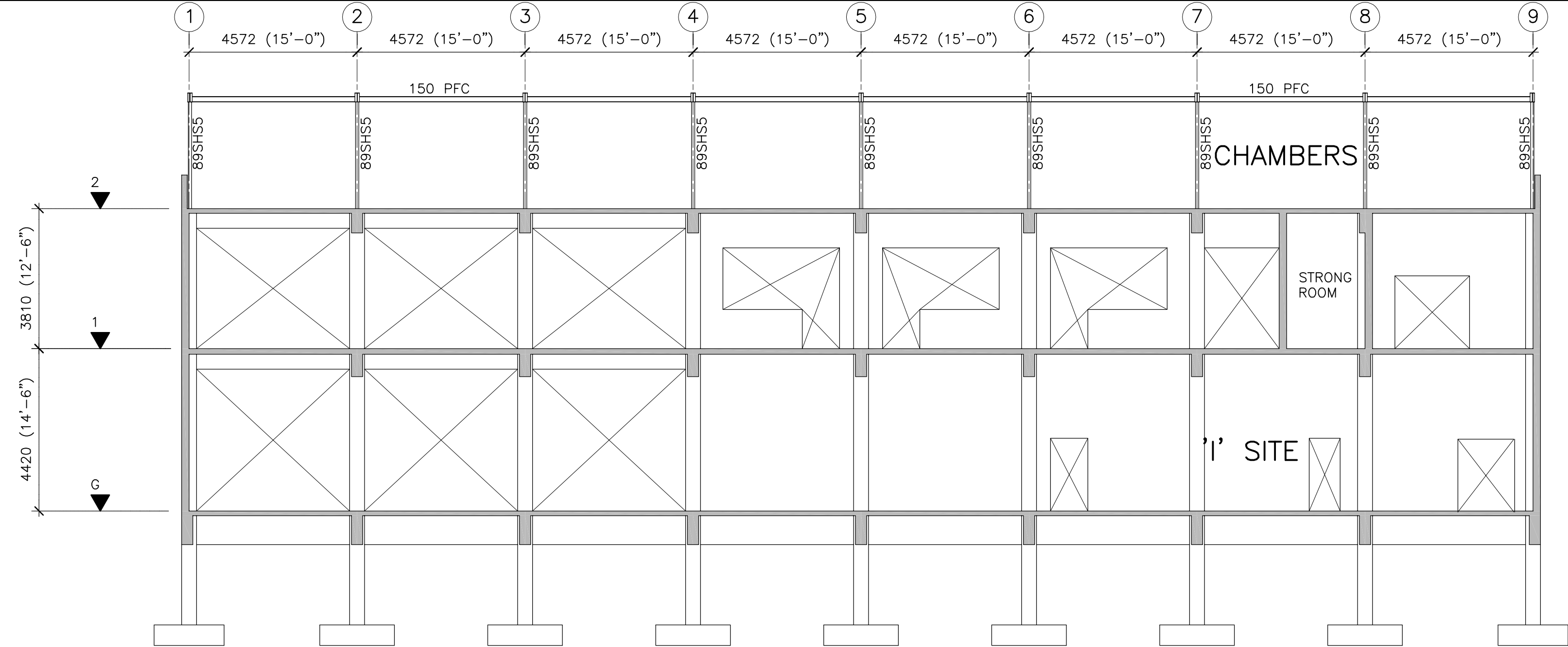


South Wall Elevation
(grid C)

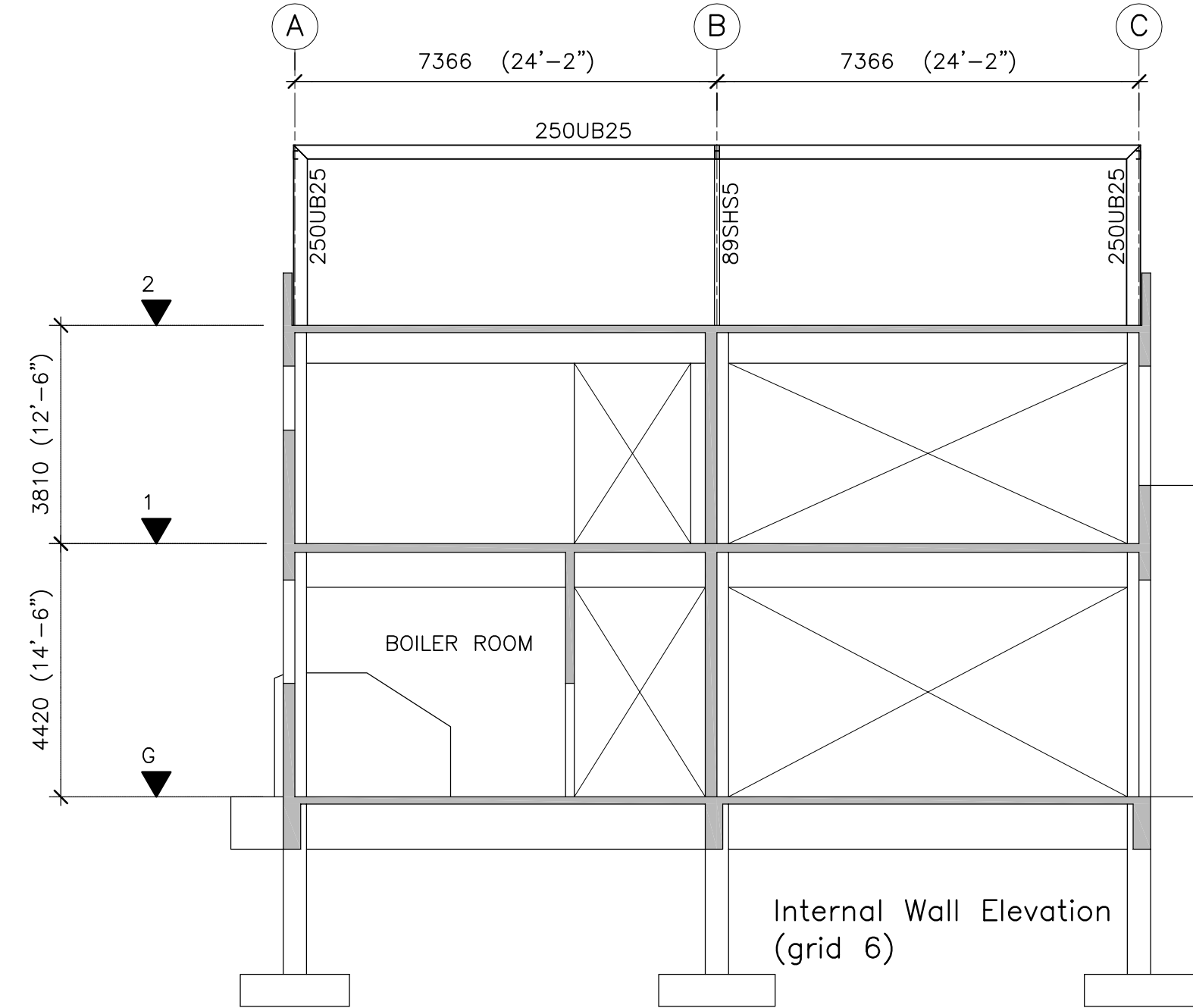


West Wall Elevation
(grid 1)

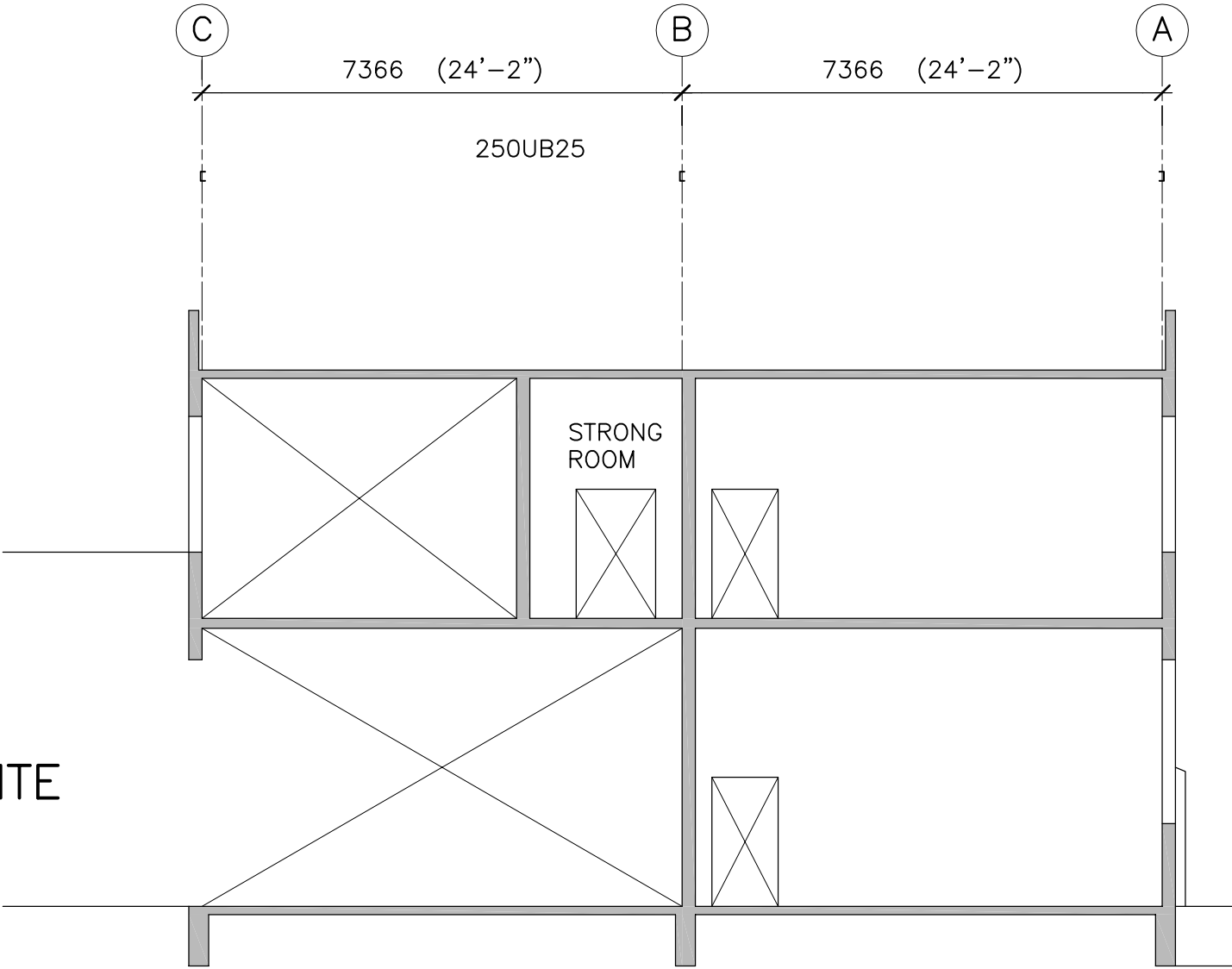
1	123456	qwertyuiop
ISS.	DATE	REASON FOR ISSUE
SIMCO Consulting Limited		
JOB NAME Westland District Council Building Strengthening Weld Street - Hokitika		
DRAWING NAME Existing Wall Elevations South, West (grids C, 1)		
CAD FILE AutoCAD		SCALE (at A2 paper size) 1 : 100
JOB No. 20050	DRAWING S1.5	ISSUE 1



Internal Wall Elevation
(grid B)



Internal Wall Elevation
(grid 6)



Internal Wall Elevation
(grid 7/8)

1	123456	qwertyuiop
ISS.	DATE	REASON FOR ISSUE
SIMCO Consulting Limited		
JOB NAME		
Westland District Council		
Building Strengthening		
Weld Street - Hokitika		
DRAWING NAME		
Existing Wall Elevations		
grids B, 6, 7/8		
CAD FILE	SCALE (at A2 paper size)	
AutoCAD	1 : 100	
JOB No.	DRAWING	ISSUE
20050	S1.6	1

Appendix C. Geotechnical Report

The background of the cover is a topographic map with white contour lines on a red field. The lines are dense and irregular, representing a hilly or mountainous terrain. The map covers the entire upper and left portions of the cover, with a white rectangular area on the bottom right containing the title and other text.

36 Weld Street Geotechnical Report

Version A

36 Weld Street, Hokitika

Prepared for Westland District Council

503048

**eliot
sinclair**

36 Weld Street Geotechnical Report

36 Weld Street, Hokitika
Prepared for Westland District Council
503048

Quality Control Certificate
Eliot Sinclair & Partners Limited
eliotsinclair.co.nz

Action	Name	Signature	Date
Prepared by:	David Hatton Geotechnical Engineer BE (Hons) Civil		26 June 2024
Reviewed by:	Andrei Cotiga Geotechnical Engineer BE (Hons) Civil ME CPEng		26 June 2024
Directed and approved for release by:	Travers Armstrong Structural Engineer Director BE(Hons) Civil CMEngNZ CPEng		26 June 2024
Status:	Version A		
Release date:	26 June 2024		
Distributed to:	Westland District Council		

Version History

Status	Description	Author	Release Date
A	First issue of document	David Hatton	26 June 2024

Contents

1. General	4
1.1. Introduction	4
1.2. Scope of Work	4
1.3. Site description	4
2. Existing geotechnical Information	5
2.1. Site Geology and Topography	5
2.2. Faults	5
2.3. Soil Subsoil Class	5
2.4. Nearby borehole records	6
2.5. Eliot Sinclair Nearby Deep Investigation Data	6
3. Site investigation	6
3.1. General	6
3.2. CPT	7
3.3. Hand Augers	7
3.4. Dynamic Cone Penetrometer Testing	7
3.5. Groundwater	8
4. Ground Model	8
5. Liquefaction analysis	1
5.1. Assessment Method	1
5.2. Liquefaction susceptibility	1
5.3. Vertical Settlement due to Liquefaction (index value)	2
5.4. Liquefaction Severity Number (LSN)	2
5.5. Lateral Displacement	2
5.6. Assessed Technical Category	3
6. Foundation Discussions	3
6.1. Static Case	3
6.2. Earthquake Case	5
6.3. Post-earthquake (liquefaction) Case	6
7. Conclusions	7
8. Disclaimer	7

Appendix A. Site Plan

Appendix B. ES Site investigation logs

Appendix C. Liquefaction analysis

1. General

1.1. Introduction

Eliot Sinclair have been engaged by the Westland District Council (WDC) to undertake a seismic assessment and provide seismic strengthening design for the existing building at 36 Weld Street in Hokitika. This report describes the ground conditions encountered at the site and provides geotechnical guidance to inform the seismic assessment and strengthening of the existing building.

This report is valid for two years from the date of issue.

1.2. Scope of Work

Eliot Sinclair were engaged to provide the following scope of geotechnical engineering services:

- a) Undertake a review of available data from the NZGD, WCRC hazard maps, and the Institute of Geological and Nuclear Sciences (GNS) Active Faults database.
- b) Arrange for a third-party CPT Contractor to undertake two Cone Penetration Tests/DPSH (if required) to 10-15m below ground level or to practical refusal. Also arrange for a concrete cutting contractor to cut through hard-stand areas for the testing.
- c) Undertake four Dynamic Cone Penetration tests (DCP's) to 2m depth (or practical refusal), to investigate the bearing capacity of the shallow soils.
- d) Undertake two shallow hand-auger test holes to 3m depth (or practical refusal), to investigate the nature of the shallow soils.
- e) Calculate the risk of liquefaction and liquefaction-induced settlement using the site-specific CPT results.
- f) Prepare a Geotechnical Report that summarises the results of the investigation, the risk of liquefaction, and provides geotechnical parameters that can be used to evaluate the performance of the existing foundations along with foundation upgrades that may be required.

1.3. Site description

The site is located within the centre of the Hokitika township on the north side of Weld Street. The building at the site is primarily a 3-level reinforced in-situ concrete building first constructed in 1948, that has had a number of significant alterations since it was constructed. The building is currently used as offices for the WDC and as a visitor information centre.



Figure 1. Overview showing site location (Eliot Sinclair, 2024).

2. Existing geotechnical Information

2.1. Site Geology and Topography

The Geological Map¹ of the area notes the site is underlain with Holocene shoreline deposits (Q1) consisting of Beach sand and gravel underlying present day coastal plain.

The site is relatively flat with a very gentle slope down towards the northwest. The existing building is surrounded by lawns and paved footpaths or parking areas.

2.2. Faults

The nearest active fault is the Alpine Fault, recorded on the GNS Active Faults Database², which lies approximately 23km south-east of the site. Based on available data, the site is located outside the minimum 20m fault avoidance zone recommended by the Ministry for the Environment³.

2.3. Soil Subsoil Class

Based on our geological assessment and in accordance with NZS1170.5, Section 3.1.3, the site subsoil classification, we consider a conservative site subsoil category "Class D - Deep or soft soil sites" is appropriate for the site.

¹ <https://data.gns.cri.nz/mapservice/apps/geology/>

² [Data.gns.cri.nz/af/](https://data.gns.cri.nz/af/)

³ Planning for Development of Land on or Close to Active Faults: A Guideline to Assist Resource Management Planners in New Zealand (Published July 2003).

2.4. Nearby borehole records

We have searched the NZGD website⁴ for borehole records near the site. The following boreholes are located within 200m of the site.

Borehole ID	Distance	Soil profile
BH_88151	90m SW	0 – 1.7m Gravel FILL 1.7 – 2.1m Brown SILT with organics 2.3 – 6.8m Medium dense to dense Sandy GRAVEL/Gravelly SAND 6.8 – 7.7m Gravelly sandy SILT/silty SAND 7.7 – 10m Very Dense Sandy GRAVEL
BH_193234	140m SW	0 – 1.4m Gravel FILL 1.4 – 2.3m Very Soft SILT with some fibrous organics 2.3 – 9.5m Medium dense to dense Sandy GRAVEL
BH_193233	165m SW	0 – 0.6m Gravel FILL 0.6 – 2.3m Very Soft SILT with wood fragments 2.3 – 4.2m Loose Silty SAND with wood fragments 4.2 – 6.5m Medium dense SAND 6.5 – 11m Dense to very dense sandy GRAVEL 11 – 15.5m Stiff to very stiff SILT

2.5. Eliot Sinclair Nearby Deep Investigation Data

In December 2020 Eliot Sinclair carried out two Cone Penetration tests (CPTs) at 53 Weld Street, Hokitika, which is located at around 75m south of the site.

The nearby CPT testing was carried out to practical refusal before then proceeding with Dynamic Probe Super Heavy (DPSH) testing. DPSH tests generally indicate the inferred gravels extend to at least 15m bgl where testing terminated at the target depth.

3. Site investigation

3.1. General

A geotechnical investigation of the site was undertaken on 22 and 23 April 2024 which included shallow hand augers and dynamic cone penetrometer (DCP) tests, and deep CPT testing. The investigation was undertaken as part of a wider investigation that also included 41 Weld Street where the existing building is also undergoing a seismic strengthening assessment. A total of eight locations were investigated, four around each building.

A CPT and DCP were undertaken at each end of the building at 36 Weld Street (Locations 1 and Location 2). A hand auger and DCP were undertaken at Location 5 and Location 6 to the north-east and south-west of the building. Where investigation locations were on pavement, the hardstanding was broken out using an excavator to provide access to the underlying soils.

All onsite investigations were undertaken by Canterbury Geotest Ltd under the supervision of Eliot Sinclair.

⁴ <https://www.nzgd.org.nz/>



Figure 2. Geotechnical investigation locations for 36 and 41 Weld Street.

3.2. CPT

Four CPTs at locations 1 to 4 were advanced until refusal in dense material was encountered. CPT 1 at the west end of 36 Weld Street refused at 3.4m bgl, and CPT 2 at the east end refused at 5.5m bgl.

Based on our local knowledge of the subsurface conditions, and the nearby borehole information it is more likely than not that the termination depth of the CPT's coincides with an underlying gravel deposit that is prevalent across Hokitika and the wider area.

The CPT data can be found in Appendix B.

3.3. Hand Augers

Hand augers were undertaken at locations 5 to 8 where they were progressed to refusal. Location 5 was to the north-east side of 36 Weld Street which encountered clayey and sandy silt to a depth of 1.5m bgl where refusal in gravel was encountered. Location 6 was to the south-west side of 36 Weld Street and encountered similar clayey and sandy silt but to a greater depth of 2.9m before refusal in gravel was encountered.

3.4. Dynamic Cone Penetrometer Testing

A DCP test was undertaken adjacent to each CPT and hand auger. DCPs at locations 1,2,5,6 was undertaken around the building at 36 Weld Street. Below any surficial topsoil or fill layers the DCP recorded 1 blow per 100mm to a depth of 1m bgl. From 1m bgl this increased to 2 to 3 blows per 100mm until 1.4m bgl where it then increased to greater than 3 blows per 100mm until refusal was encountered. DCP1 and DCP2 refused at 3.0m bgl and 2.9m bgl, respectively, with DCP 5 refusing at 2.3m bgl and DCP6 at 3.1m bgl.

These results generally indicate the upper soils have a relatively low but consistent geotechnical ultimate bearing capacity across the footprint of the building, in the region of 100kPa. The bearing capacity increases with depth and 300kPa is generally available from 1.5m bgl.

Full DCP profiles are provided in Appendix B.

3.5. Groundwater

Groundwater was not encountered in our shallow testing at up to 2.9m depth. Ground water has been recorded at 2.8m bgl on nearby ES sites in the past.

4. Ground Model

The CPT data was used within CPeT-IT v2.3 software to infer the ground model based on soil behaviour type (SBT) ratios. We used this along with the hand auger and DCP logs to infer a generalised soil profile for the site.

The interpretation of the soil conditions indicates three broad layers are present. A soft clayey silt, sandy silt, and silty sand to 2.8 to 2.9m bgl, then sand and silty sand to between 3.6m at the western end of the building and 5.5m at the eastern end of the building where gravel was encountered.

The software used to represent the subsurface conditions infers the soil type from the CPT data. As no soil samples are taken during this test method, care should be taken in this regard.

For the purposes of this report, we have adopted a GWT of 2.8m bgl.

Refer to Appendix B for the CPT data.

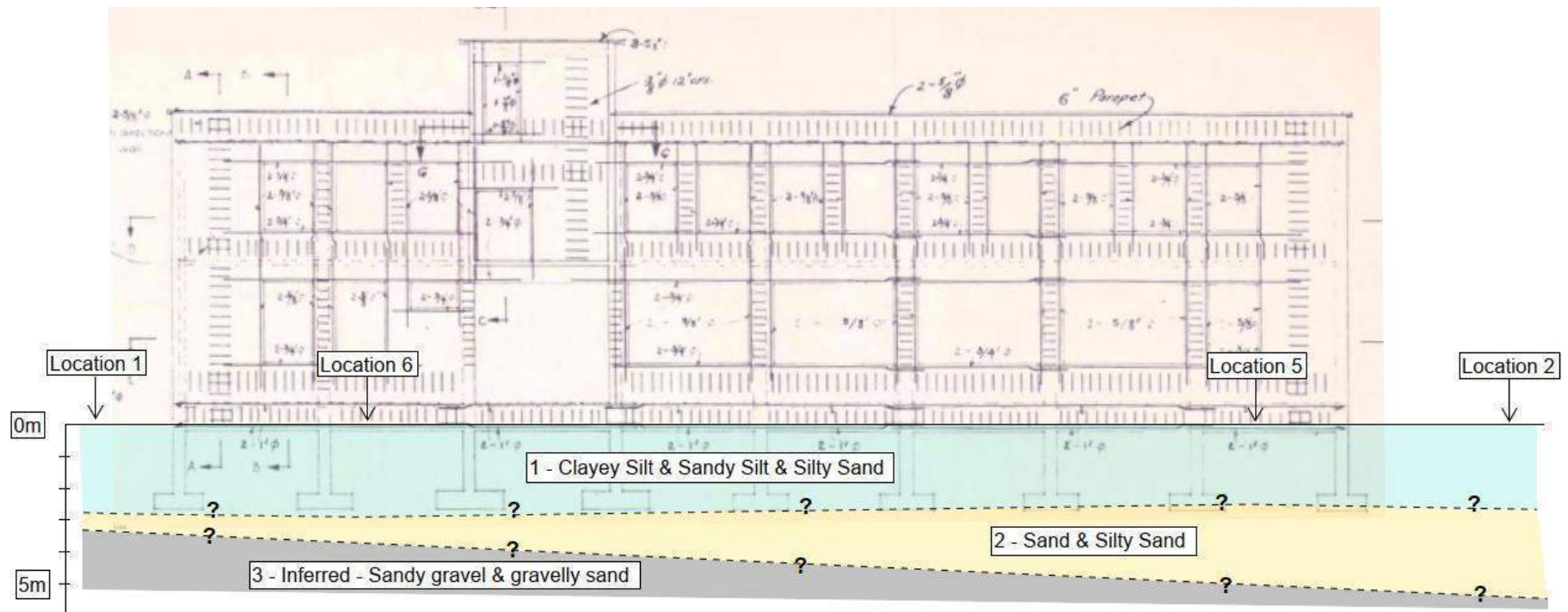


Figure 3. Inferred soil profile beneath 36 Weld Street in the East-west orientation.

Table 1. Ground model soil properties

Layer	Depth to top (m bgl)	Thickness (m)	Typical q_c (MPa)	Unit weight (kN/m ³)	Phi	Cohesion
1 – Clayey Silt, Sandy Silt, Silty Sand	0	2.8 – 2.9	0.5 – 2.0	17	26°	3
2 – Sand and Silty Sand	2.8 – 2.9	0.6 – 2.6	10 – 20	18	29°	0
3 – Inferred sandy gravel and gravelly sand	3.4 – 5.5	-	>35	20	32°	0

5. Liquefaction analysis

The BECA 2021 West Coast Regional Liquefaction Assessment Report⁵ identifies the site as belonging to “Liquefaction Damage is Possible”. Refer to Figure 4.



Figure 4. Map of west coast region liquefaction assessment (Beca, Map I7, 2023).

5.1. Assessment Method

The calculation of liquefaction triggering was undertaken using the method outlined in Boulanger & Idriss (2014), and the estimation of post-liquefaction induced settlements using the method outlined by Zhang et al (2002). The liquefaction analysis was calculated using CLiq software.

The site specific CPT data was analysed for both the Serviceability Limit State (SLS) and the Ultimate Limit State (ULS) levels of earthquake shaking in Hokitika region as per NZGS Module 1 showing below:

- SLS (25-year return period) Case 1: M6.5, PGA 0.13g.
- ILS (100-year return period) Case 2: M6.5, PGA 0.27g.
- ULS (500-year return period) Case 3: M6.7, PGA 0.53g.

Based on our analysis of the CPT data we have assumed the design groundwater depth to be 2.8m bgl during the static condition, and 2.0m bgl during the seismic loading.

Please refer to Appendix C for the Liquefaction Analysis Report.

5.2. Liquefaction susceptibility

Analysis of the CPT1 and CPT 2 data indicates that isolated bands within layers 1 and 2 of the soil profile are susceptible to liquefaction. Liquefaction is not triggered during a design SLS level event, with some

⁵ <https://www.wcrc.govt.nz/publications/natural-hazard-reports>

liquefaction occurring during an ILS level event, and a greater level occurring during a design ULS event.

With the ground water set to 1.0m bgl to assess the near surface soils susceptibility to liquefaction, a design ULS level event was found to trigger liquefaction between approximately 1.5m bgl and 3.0m bgl, and again from 4.5 m bgl to 5.5m bgl.

The results of CPT 3 and CPT4 which were located to the south near 41 Weld Street are not discussed here but show a general agreement with the results from CPT1 and CPT2.

5.3. Vertical Settlement due to Liquefaction (index value)

The liquefaction-induced 'index' settlement values were calculated using the method by Zhang et al (2002)¹² for a range of parameters that are estimated from the four basic CPT parameters (depth, cone tip resistance, skin friction and pore water pressure) and represent 'free-field' settlements. Therefore, the settlements shown in Table 1 are not an exact vertical movement, but only index values for interpretation of relative susceptibility to the damaging effect of liquefaction.

Table 2. Liquefaction-induced 'index' settlement values

Test No.	Depth of CPT test (m bgl)	Liquefaction-induced 'index' settlements (mm)			MBIE Equivalent land classification at test location
		SLS1 (M6.5, 0.13g)	ILS (M6.5, 0.27g)	ULS (M6.7, 0.53g)	
CPT1	11.04	0	9	12	TC2
CPT2	6.91	0	4	26	TC2

5.4. Liquefaction Severity Number (LSN)

The liquefaction severity number (LSN) is a parameter developed to reflect the more damaging effects of shallow liquefaction on residential land and shallow foundations. The estimated LSN values for CPT 1 and CPT 2 are summarised in Table 3.

Table 3. Maximum LSN

Event	Maximum LSN Range	Predominant Performance
SLS/ILS	0 - 2	Little to no expression of liquefaction
ULS	4 - 8	Little to no expression of liquefaction

5.5. Lateral Displacement

The site is near-level, and the nearest watercourse (i.e. Hokitika River) is located around 240m south of the site. Assuming a free bank face of 4.8m (necessary for software conditions) the calculated amount of global lateral displacement in an ULS event is within the range of MBIE 'Minor to Moderate' extent.

Lateral displacement and stretching is not considered to be an issue for the site.

5.6. Assessed Technical Category

The TC land classification system is primarily intended for application to residential land. However, it also gives a useful indication of the relative vulnerability to liquefaction and earthquake-induced land deformation for non-residential land, as the site. Based on the liquefaction hazard discussed above, we have assessed the predicted earthquake-induced land deformation around the site to be **equivalent to residential Technical Category 2 (TC2)**.

6. Foundation Discussions

6.1. Static Case

In static case the foundations are expected to bear exclusively onto the 2m x 2m x 0.56m pads located at a depth of 2.8m bgl. The pads bear onto a variable thickness of sand and silty sand layer as depicted in Figure 3. The ultimate bearing capacity of the pads at that depth is expected to be approximately 730kPa when calculated using a simplified B1/VM4 method. Using a strength reduction factor of 0.5, the design bearing strength (as defined by B1/VM4) is estimated at 365kPa.

The general geometry is shown in Figure 5a, together with the assumed soil properties.

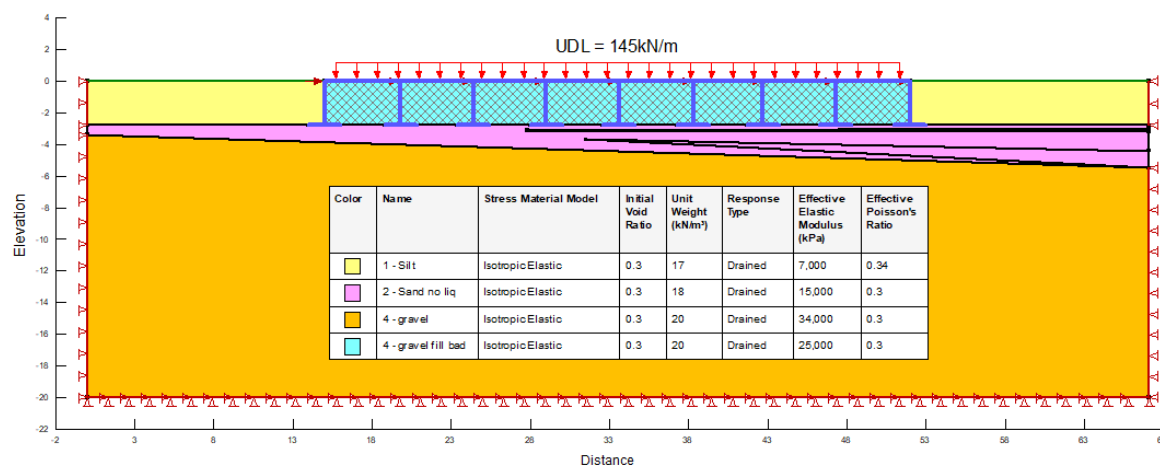


Figure 5a. FEA Model geometry

We further estimated the total settlements using the entire cross-section frame model as per Figure 5b.

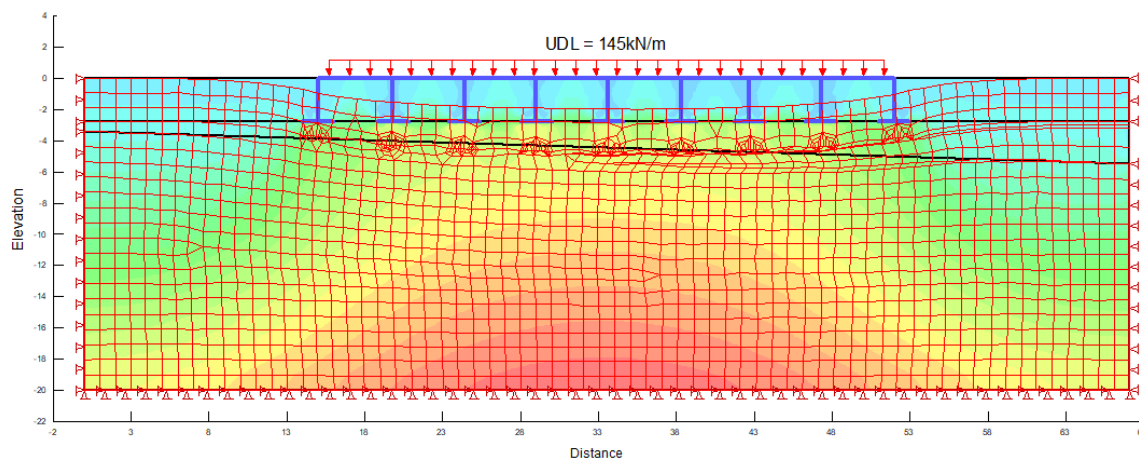


Figure 5b. Middle frame – static case settlement

The differential elastic settlement between the ends and middle of the beam was estimated at 47mm as per Figure 6 and Figure 7.

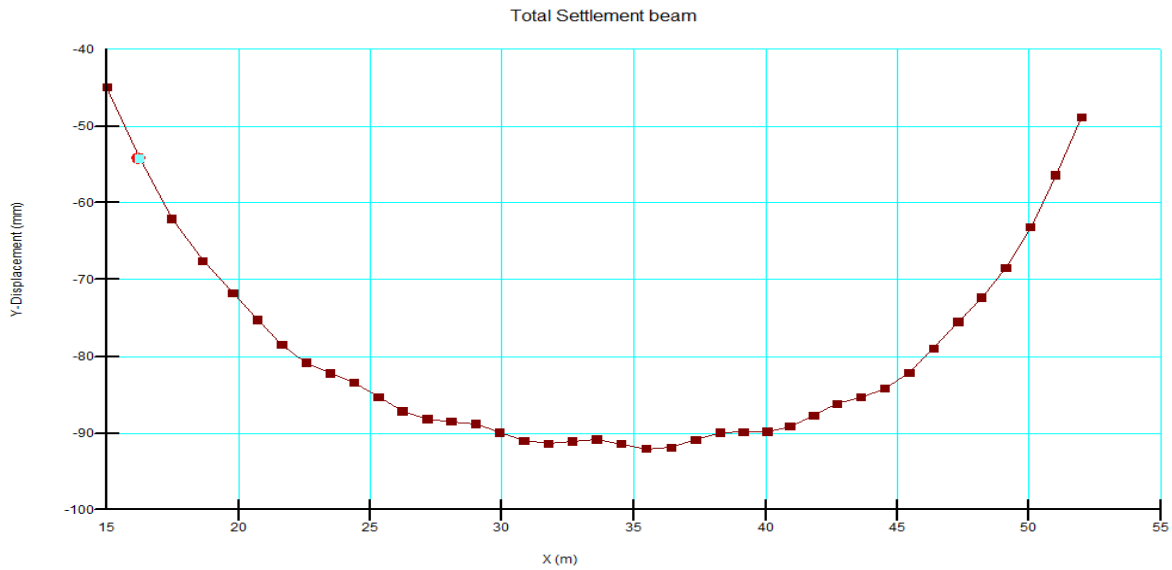


Figure 6. Beam settlement

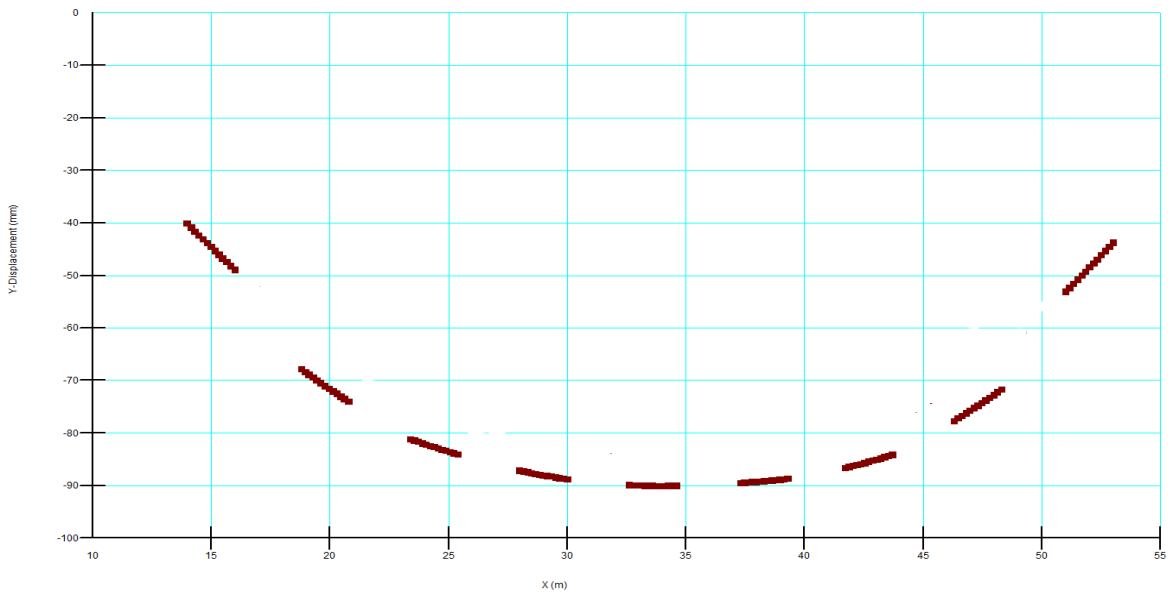


Figure 7. Individual pad settlement

6.2. Earthquake Case

During a ULS earthquake event we estimate alongside the vertical UDL, a point load of 480kN has been applied to each pile. The lateral deformation of the piles is represented in Figure 8.

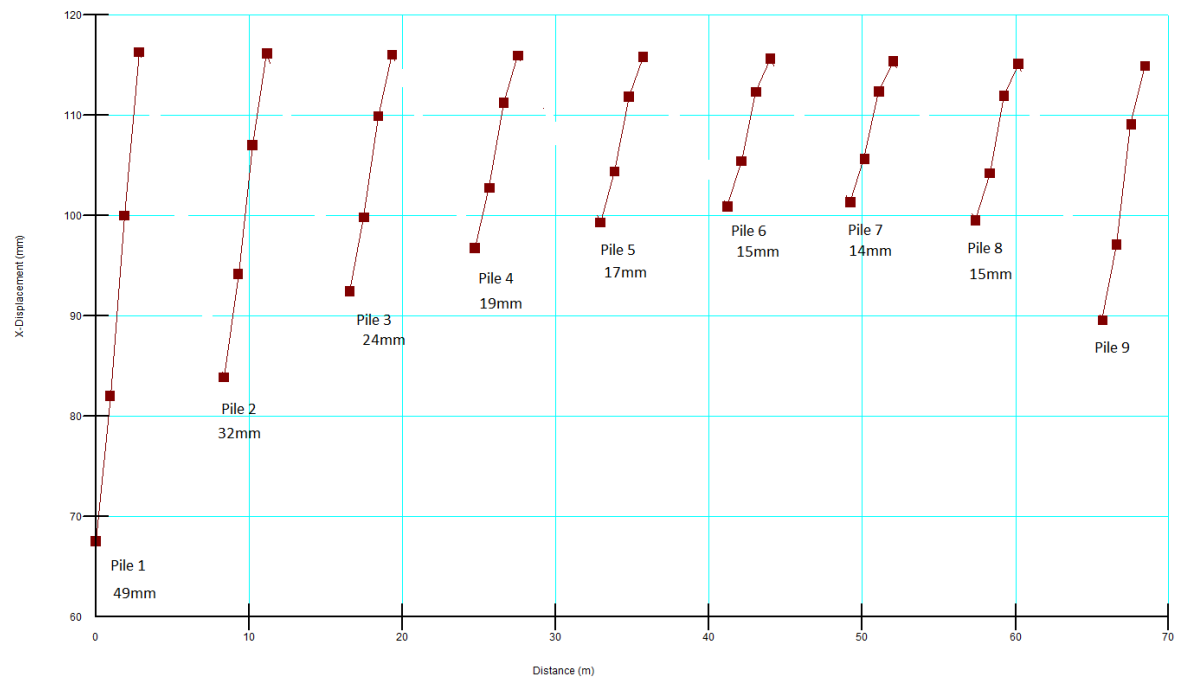


Figure 8. Pile lateral deformation.

Pile 1 is estimated to experience the largest lateral deformation between the top connection into the beam and the bottom connection into the 2m x 2m pad. The deformation is estimated at 49mm.

Because of this deformation, the maximum Bending Moment is estimated to occur at this position, and we represent the graph in Figure 9.

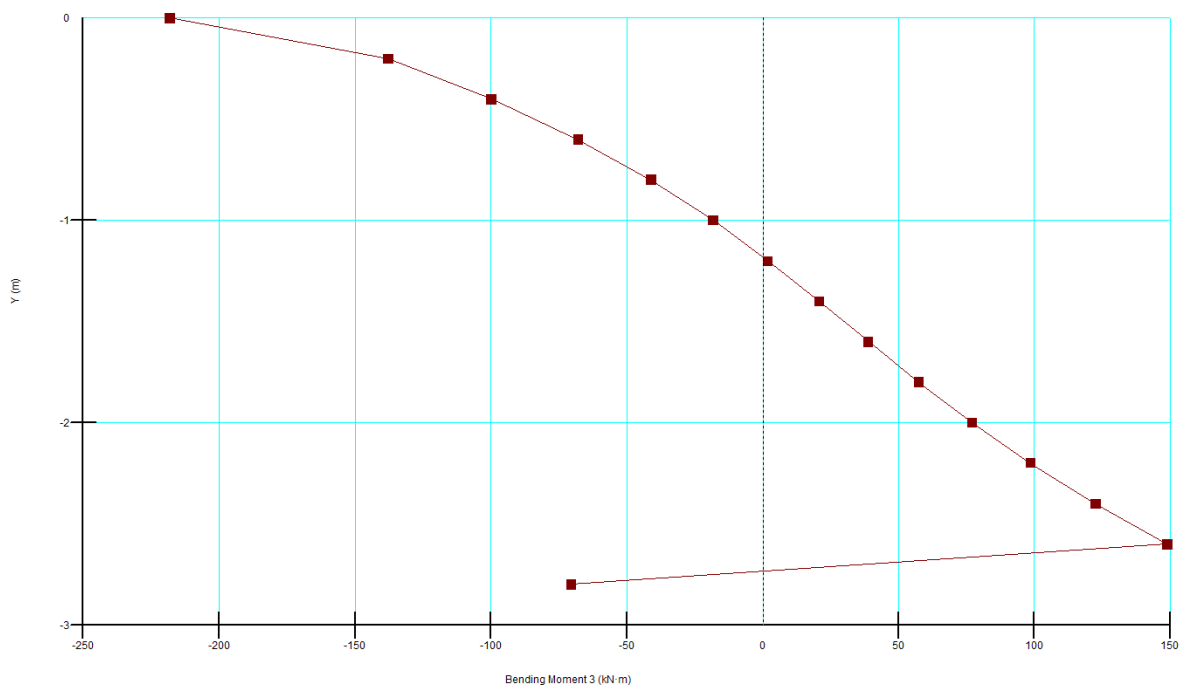


Figure 9. BM of Pile 1 – Mmax=218kNm at beam connection level

Furthermore, we calculated a series of lateral spring stiffnesses, varying with depth, to be used by structural engineers.

Lateral pile spring values have been calculated assuming a 250mm square pile extending from the ground beams down to the foundation pad. The values provided in Table 5 can be used to assess the lateral deflections during a seismic event. It is recommended that a sensitivity analysis using 50% and 200% spring stiffness is carried out.

Table 4. Soil springs for a 250mm square pile - 36 Weld Street

Depth (m bgl)	k_s (kN/m)	50% of k_s (kN/m)	200% of k_s (kN/m)
0.0	100	50	200
0.5	4700	2350	9400
1.0	8000	4000	16000
1.5	10500	5250	21000
2.0	12000	6000	24000
2.5	13400	6700	26800
2.8	14600	7300	29200

6.3. Post-earthquake (liquefaction) Case

Continuing the analysis, we assumed the lateral shear force developed during earthquake has led to the failure of all of the piles. Hence, the building's vertical loads are fully supported by the existing ground beam sitting onto the gravel fill.

In this case, liquefaction of discrete layers of sand and silty sand have liquefied and the strength parameters of those layers have been greatly degraded.

We are unsure of the quality of gravel fill between the pile pads and the beam's underside and have thus allowed for relatively poor performance.

As most of the liquefaction is predicted to occur towards the eastern side of the building (corresponding to CPT 2), it is unsurprising to see that differential settlement tends to accentuate towards Pile 7 & Pile 8. The estimated settlement at the beam level is presented in Figure 10.

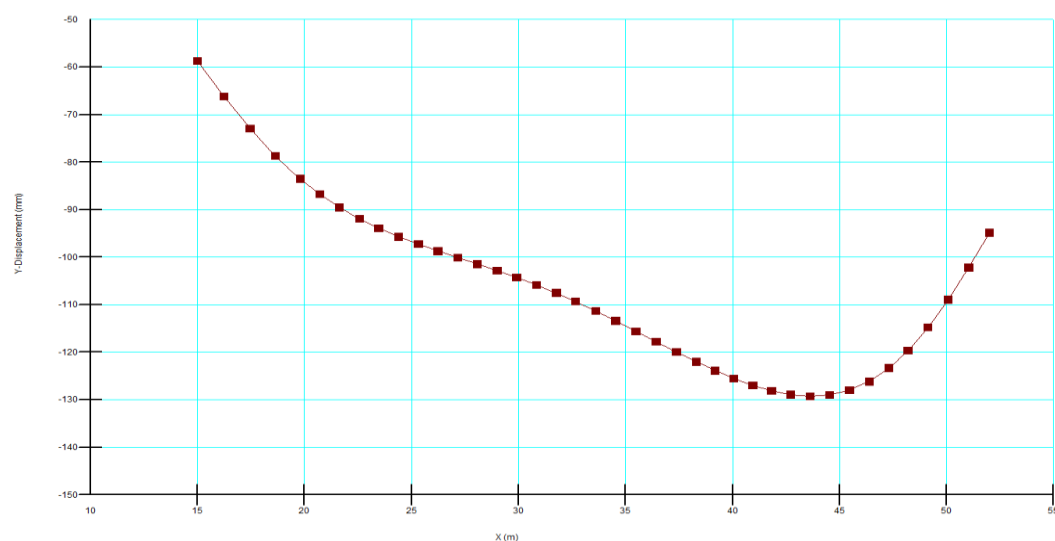


Figure 10. Beam settlement in post-liquefaction case, following complete pile failure.

7. Conclusions

The soil profile at the site consists of silts overlaying sands and then gravels. The pad footings are assumed to be founded on the sand layer at approximately 2.8m bgl. The thickness of the sand layer beneath the pad footings increases from approximately 0.6m at the western end, to 2.6m at the eastern end.

Liquefaction is not expected to occur during a design SLS seismic event, with liquefaction of some lenses within the silt and sand layers occurring during a design ULS event. As a result of greater thickness of sand below the eastern end of the building, there is the potential for differential settlement to occur in a post-earthquake liquefied state.

In static case, the pads are estimated to have an Ultimate Bearing Capacity of 730kPa and the differential settlements across the building length are estimated at approximately 50mm.

In earthquake case, it is expected that the lateral shear force from the building will lead to the loss of some of the piles (possibly all). Following the earthquake action, the ground will likely liquefy with a greater amount being predicted towards the eastern end (CPT2).

Without the piles, the vertical forces of the building will be distributed to the beam which will now bear on an uncertain quality of gravel fill. The differential settlement in post-earthquake (liquefaction) case is predicted to be in the region of 50mm over a 10m length.

These results are based on preliminary structural estimated loads as well as soil strength parameters derived from CPT tests. They should be regarded as "best fit" approximations and sensitivity analysis of $\pm 50\%$ should be allowed for. If the sensitivity analysis shows that the structure may reach critical failure, additional refinement of either the loads or the soil's assumed strength parameters should be sought as part of additional investigations and reporting.

8. Disclaimer

This report has been prepared by Eliot Sinclair & Partners Limited ("Eliot Sinclair") only for the intended purpose as described in Section 1 of this report. Our analysis is based on a visual inspection, shallow and deep soil investigations of the site on 26 April 2024 comprising shallow Dynamic Cone Penetration (DCP) testing, shallow hand auger testing and CPT around the existing building.

The report is based on the most recent version of the Ministry of Business, Innovation and Employment Guidelines.

Where data supplied by Westland District Council or other external sources, including previous site investigation reports, have been relied upon, it has been assumed that the information is correct unless otherwise stated. No responsibility is accepted by Eliot Sinclair for incomplete or inaccurate data supplied by other parties.

Whilst every care has been taken during our investigation and interpretation of subsurface conditions to ensure that the conclusions drawn, and the opinions and recommendations expressed are correct at the time of reporting, Eliot Sinclair has not performed an assessment of all possible conditions or circumstances that may exist at the site. Variations in conditions may occur between investigatory locations and there may be conditions such as subsoil strata or features at depth that were not detected by the scope of the investigation that was carried out or have been covered over or obscured over time. Additionally, on-going seismicity in the general area may lead to deterioration or additional ground settlement that could not have been anticipated at the time of writing this report.

Eliot Sinclair does not provide any warranty, either express or implied, that all conditions will conform exactly to the assessments contained in this report.

Should the exposure of soil conditions that vary from those described in this report, or the requirements of MBIE's guidelines, NZ Standards or the NZBC that relate to foundations and floors be updated, a review of our recommendations may be required. Eliot Sinclair should be contacted to confirm the validity of this report should any of these occur.

This report has been prepared for the benefit of Westland District Council for the purposes as stated above. No liability is accepted by Eliot Sinclair or any of their employees with respect to the use of this report, in whole or in part, for any other purpose or by any other party.

Appendix A. Site Plan



DISCLAIMER
© Eliot Sinclair and Partners Ltd. This drawing and all its information is only to be used for its intended purpose. All rights reserved.

- NOTES
- Contractors to verify all dimensions and the location of all underground services on site prior to commencing work.
 - Unless noted otherwise, all work shall be undertaken in accordance with the NZBC and any relevant Territorial Authority Engineering Standards and Specifications as a minimum standard.

LEGEND

CPT + Dynamic Cone
Penetrometer (DCP)

HA + DCP

LINZ NZ Primary Parcels

A	GIS	29.05.24	Preliminary
REV.	DRAWN	DATE	NOTE

CLIENT

WESTLAND DISTRICT COUNCIL

DESIGNED	GIS
DRAWN	GIS
CHECKED	DSH
APPROVED	29.05.24 DSH

STATUS	PRELIMINARY
SCALE	1:700 [A3]

TESTING LOCATIONS PLAN
FROM CORE-GS
36 Weld Street
Hokitika

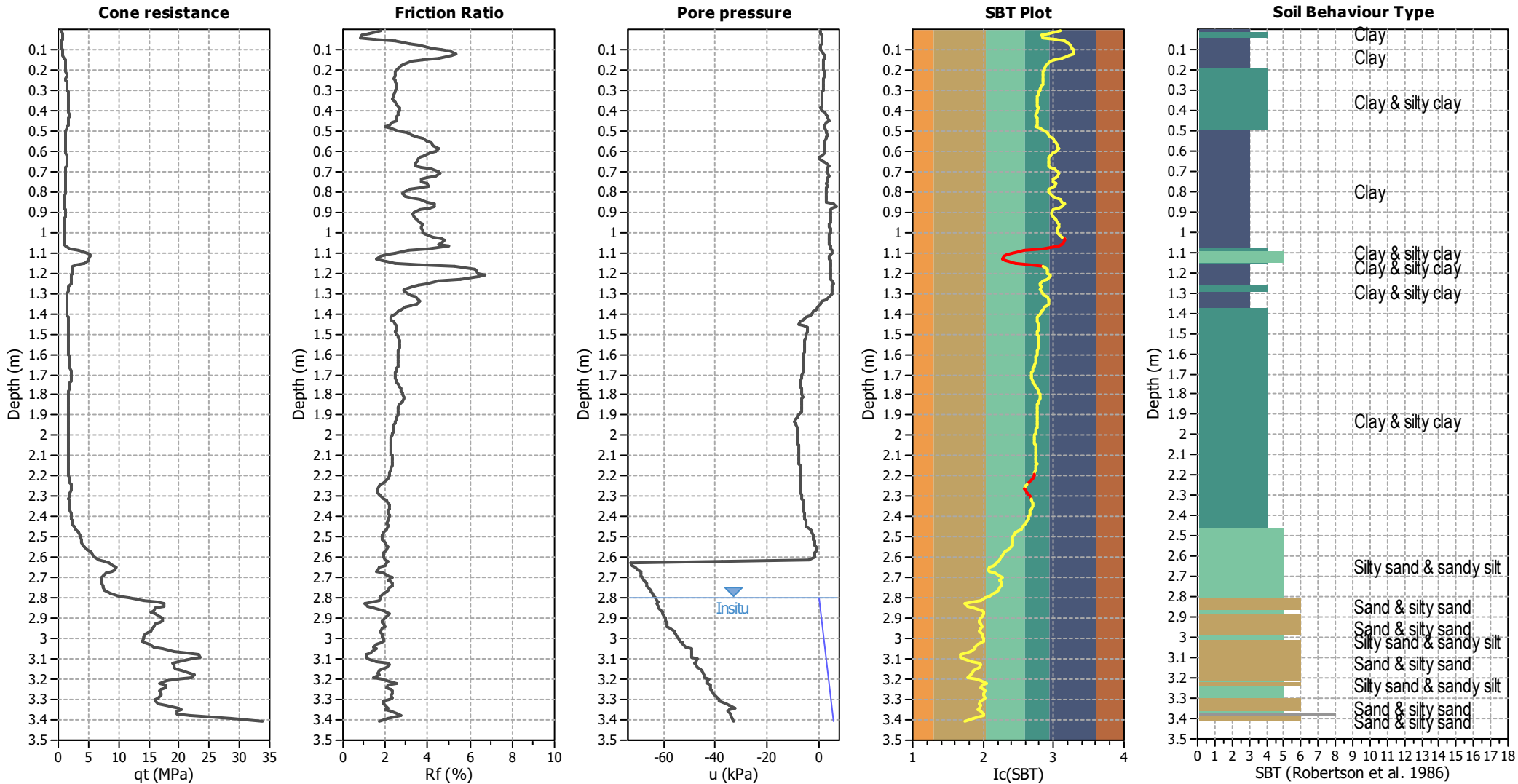
TEST LOCATION PLAN
XX

PROJECT	SET	SHEET	REV.
503048	01	XXXX	A



Appendix B. ES Site investigation logs

CPT basic interpretation plots



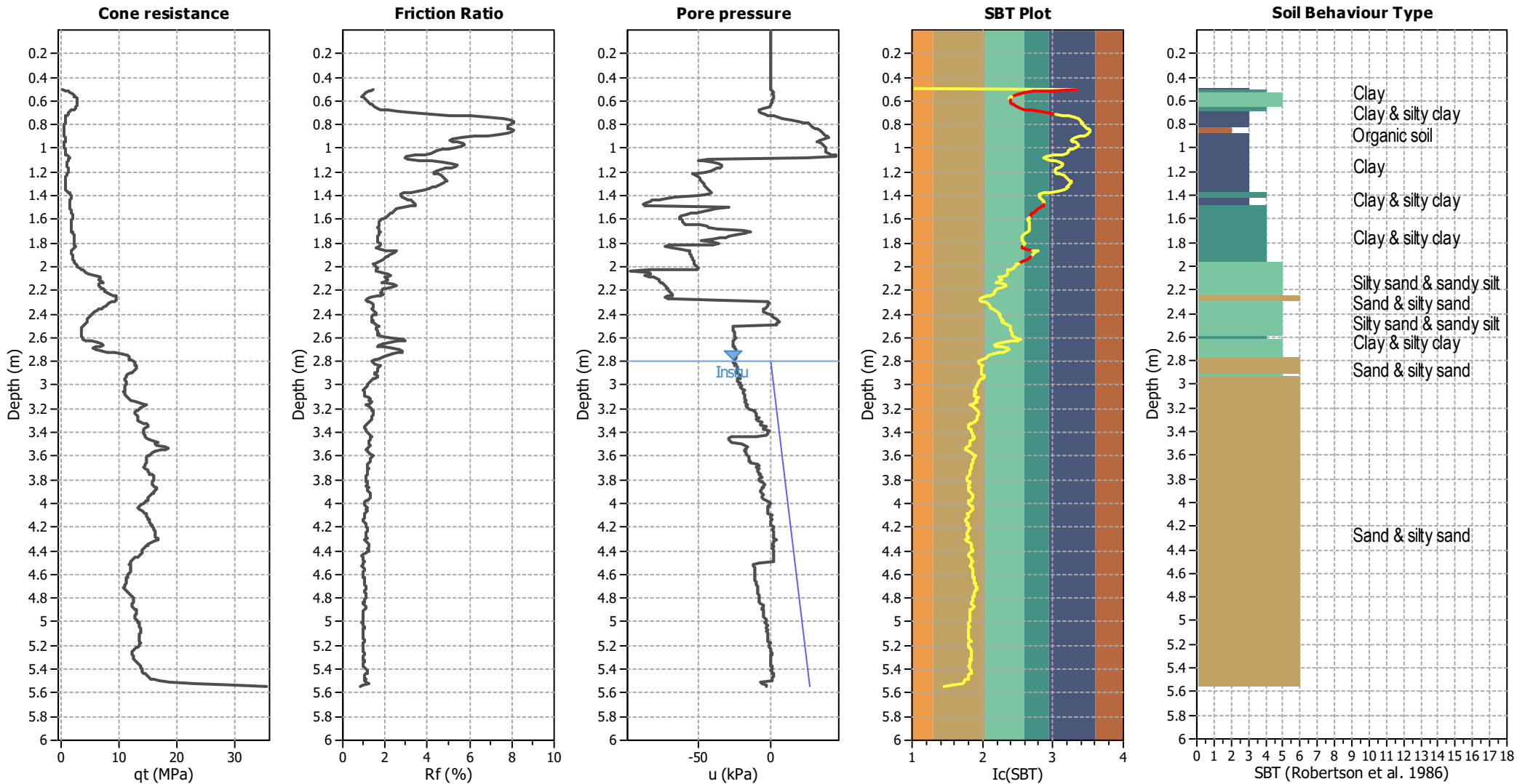
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _g applied:	Yes
Earthquake magnitude M _w :	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.80 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT basic interpretation plots



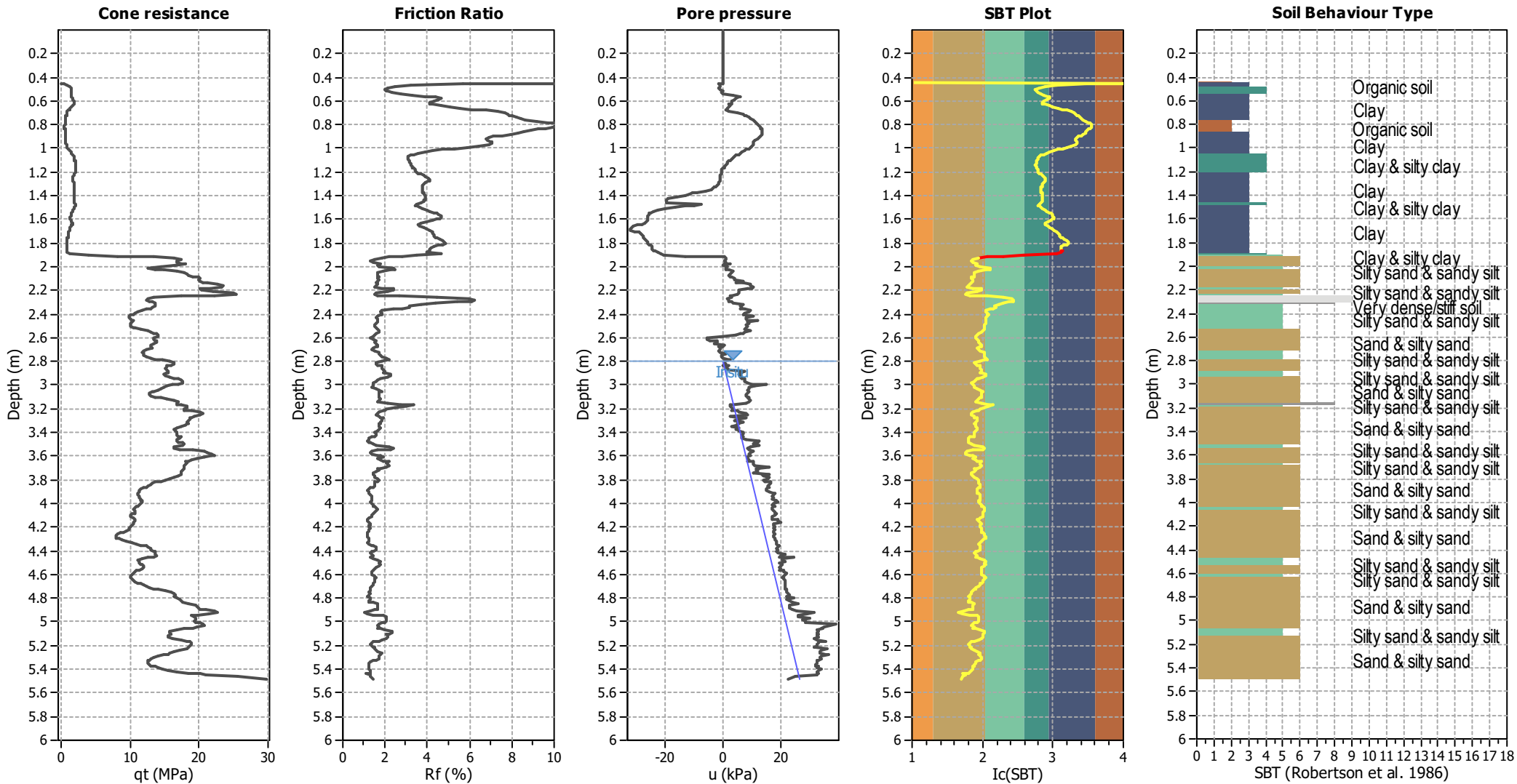
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _g applied:	Yes
Earthquake magnitude M _w :	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.80 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT basic interpretation plots



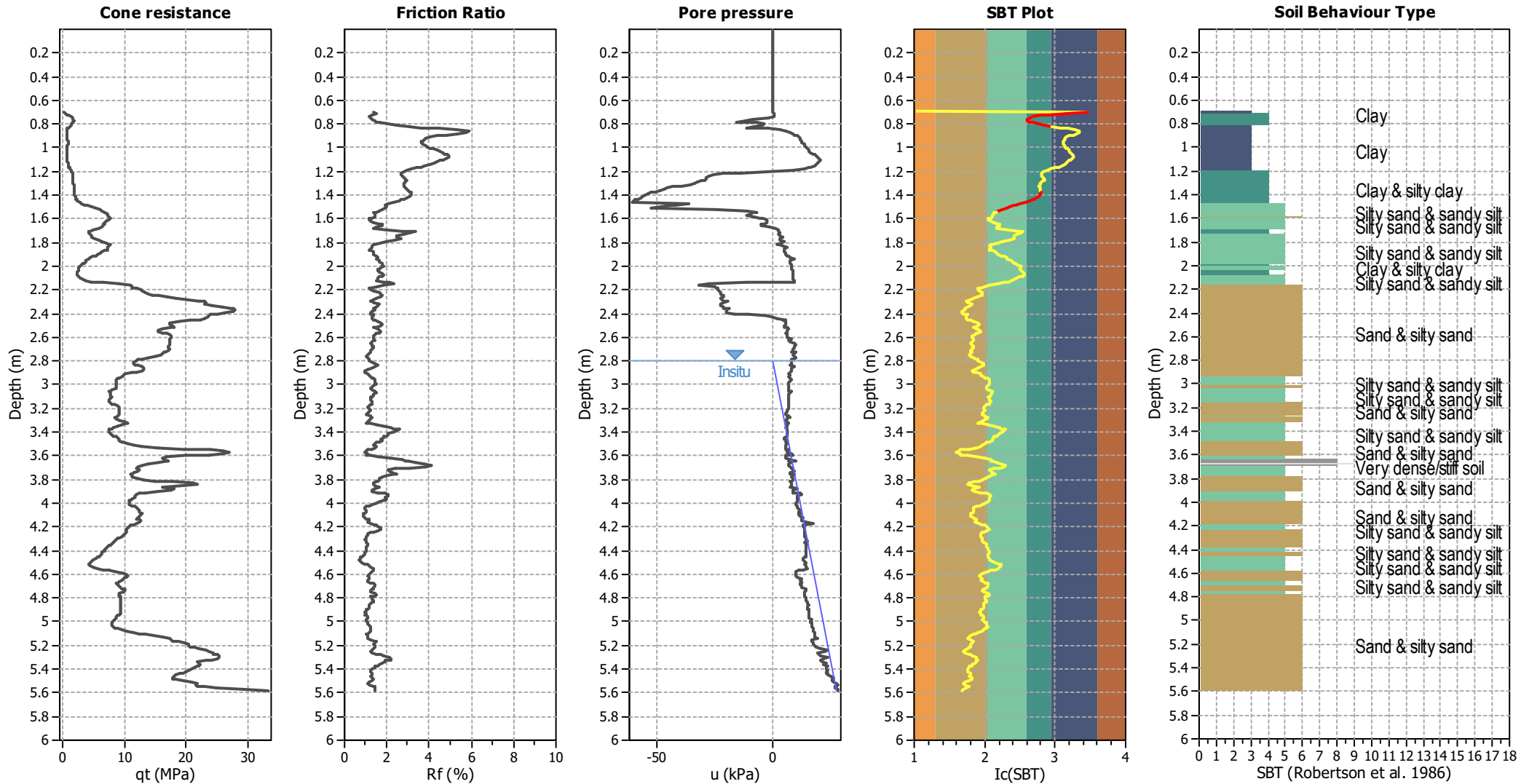
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.80 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT basic interpretation plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _g applied:	Yes
Earthquake magnitude M _w :	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.80 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Site Investigation Record

Client: Westland District Council c/- RDB Project Management

Technical Category:

Date Tested: 22-Apr-2024

Site: 36 & 41 Weld Street, Hokitika

Lot:

Log Sheet No.: 1 of 1

D.P.:

Project No.: 503048

Dynamic Cone Penetrometer (DCP) Test Results															Depth (m)	Soil Profile	
Number of Blows per 100mm																Test Location 01	Water
1	2	3	4	5	6	7	8	9	10	11	12	13	14				
															0.2	EOH: 0m - refer to CPT01 and CPT02 for soil profiles	Groundwater Not Encountered
															0.4		
															0.6		
															0.8		
															1.0		
															1.2		
															1.4		
															1.6		
															1.8		
															2.0		
															2.2		
															2.4		
															2.6		
															2.8		
>>20															3.0		
>>20															3.2		
— 01 - - - 02															Site Plan: (Not to Scale)		
Comments: Dynamic Cone Penetrometers: Undertaken adjacent to CPT01 and CPT02.																	
Field Staff: Geotest		Prepared By: DSH		Soil Profile From:													
Job Manager: TIA		Approved By: AC		☐ Hand Auger ☐ Spade Hole ☐ Test Pit													

Note: This record identifies the geotechnical conditions encountered at the noted test location(s) only. It is possible that ground conditions could be different away from the point(s) of testing.

Site Investigation Record

Client: Westland District Council c/- RDB Project Management

Site: 36 & 41 Weld Street, Hokitika

Technical Category:

Lot:

D.P.:

Date Tested: 22-Apr-2024

Log Sheet No.: 1 of 1

Project No.: 503048

Dynamic Cone Penetrometer (DCP) Test Results															Depth (m)	Soil Profile		
Number of Blows per 100mm																Test Location 03	Water	
1	2	3	4	5	6	7	8	9	10	11	12	13	14					
																EOH: 0m - refer to CPT03 and CPT04 for soil profiles	Groundwater Not Encountered	
— 03 - - - 04															Site Plan: (Not to Scale)			
Comments: Dynamic Cone Penetrometers: Undertaken adjacent to CPT03 and CPT04.																		
Field Staff: Geotest		Prepared By: DSH		Soil Profile From:														
Job Manager: TIA		Approved By: AC		☐ Hand Auger ☐ Spade Hole ☐ Test Pit														

Note: This record identifies the geotechnical conditions encountered at the noted test location(s) only. It is possible that ground conditions could be different away from the point(s) of testing.

Site Investigation Record

Client: Westland District Council c/- RDB Project Management

Site: 36 & 41 Weld Street, Hokitika

Technical Category:

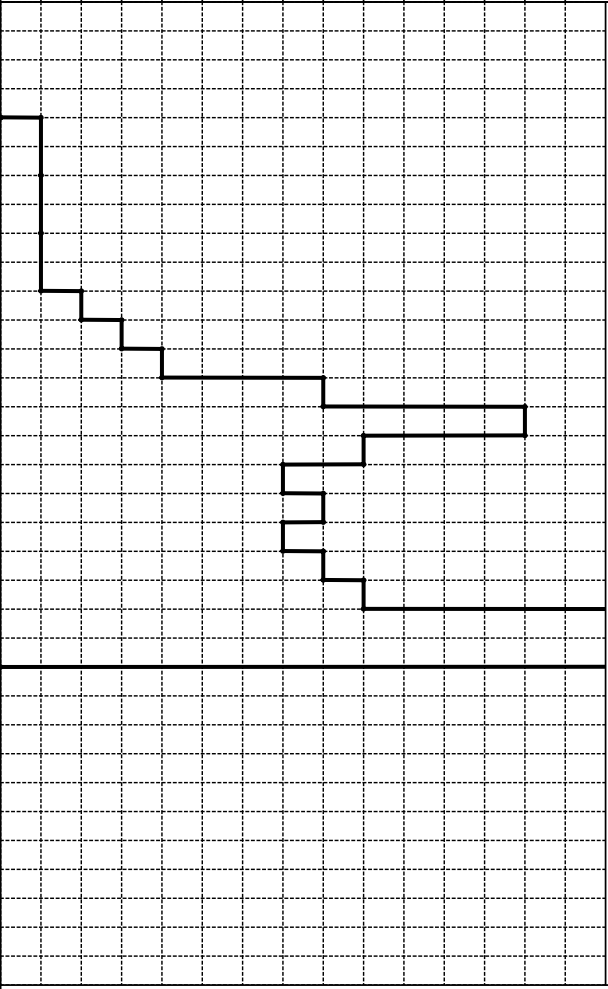
Lot:

D.P.:

Date Tested: 22-Apr-2024

Log Sheet No.: 1 of 1

Project No.: 503048

Dynamic Cone Penetrometer (DCP) Test Results															Depth (m)	Soil Profile	
Number of Blows per 100mm																Test Location 05	Water
1	2	3	4	5	6	7	8	9	10	11	12	13	14				
															0.2	FILL: fine to coarse GRAVEL, with some cobbles. Moist; Chipseal pavement and basecourse.	Groundwater Not Encountered
															0.4	Clayey SILT; grey with some orange mottling. Low plasticity; moist.	
															0.6		
															0.8		
															1.0		
															1.2	Sandy SILT; grey. Low plasticity; moist.	
															1.4		
															1.6	EOH: 1.5m - Target Depth: Refusal in gravel.	
															1.8		
															2.0		
															2.2		
															2.4		
															2.6		
															2.8		
															3.0		
															3.2		

05

Comments:

Field Staff:	Prepared By:	Soil Profile From:
Geotest	DSH	☒ Hand Auger
Job Manager:	Approved By:	☐ Spade Hole
TIA	AC	☐ Test Pit

Site Plan: (Not to Scale)



Note: This record identifies the geotechnical conditions encountered at the noted test location(s) only. It is possible that ground conditions could be different away from the point(s) of testing.

Site Investigation Record

Client: Westland District Council c/- RDB Project Management

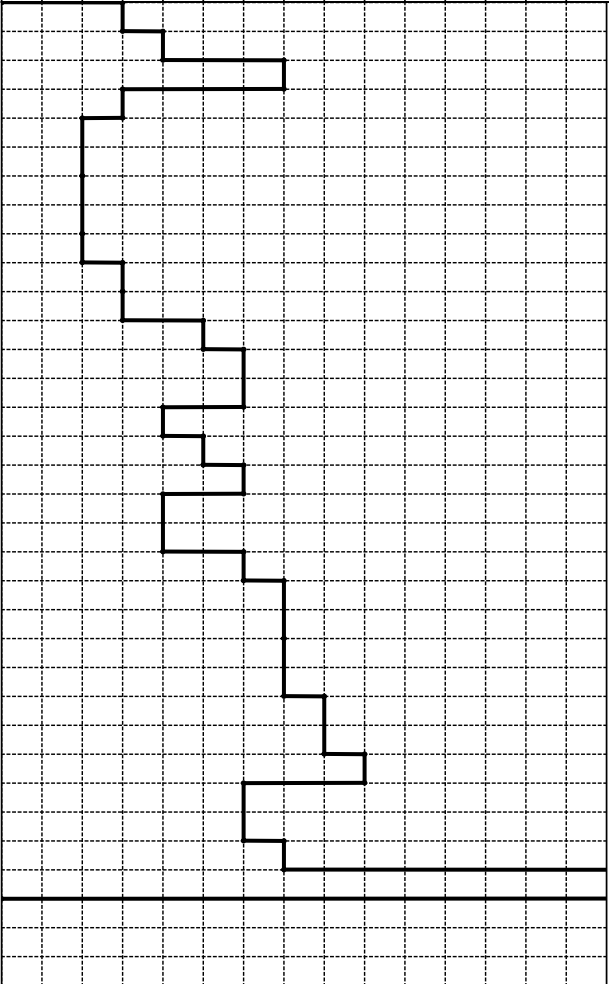
Technical Category:

Date Tested: 22-Apr-2024

Site: 36 & 41 Weld Street, Hokitika

Lot: D.P.:

Log Sheet No.: 1 of 1 Project No.: 503048

Dynamic Cone Penetrometer (DCP) Test Results															Soil Profile		
Number of Blows per 100mm															Depth (m)	Test Location 06	Water
1	2	3	4	5	6	7	8	9	10	11	12	13	14				
																FILL: sandy TOPSOIL; brown. Moist. FILL: sandy fine to coarse GRAVEL; grey. Moist. Clayey silty fine to coarse SAND, with trace gravel; light brown. Gravel, fine to medium. Clayey SILT; grey. Low plasticity; moist. Clayey silty fine SAND; grey. Low plasticity; wet to saturated. EOH: 2.9m - Target Depth: Refusal in gravel.	Groundwater Not Encountered
06																	
Comments:																	
Field Staff:		Prepared By:		Soil Profile From:													
Geotest		DSH		☒ Hand Auger													
Job Manager:		Approved By:		☐ Spade Hole													
TIA		AC		☐ Test Pit													

Site Plan: (Not to Scale)



Note: This record identifies the geotechnical conditions encountered at the noted test location(s) only. It is possible that ground conditions could be different away from the point(s) of testing.

Site Investigation Record

Client: Westland District Council c/- RDB Project Management

Site: 36 & 41 Weld Street, Hokitika

Technical Category:

Lot:

D.P.:

Date Tested: 22-Apr-2024

Log Sheet No.: 1 of 1

Project No.: 503048

Dynamic Cone Penetrometer (DCP) Test Results															Depth (m)	Soil Profile	
Number of Blows per 100mm																Test Location 07	Water
1	2	3	4	5	6	7	8	9	10	11	12	13	14				
>>17																EOH: 0m - Target Depth: Unable to penetrate hard ground with hand auger.	
07 Comments: Dynamic Cone Penetrometer: Unable to penetrate surface layer at this location.																	
Field Staff: Geotest		Prepared By: DSH		Soil Profile From:													
Job Manager: TIA		Approved By: AC		☒ Hand Auger ☐ Spade Hole ☐ Test Pit													

Site Plan: (Not to Scale)

Note: This record identifies the geotechnical conditions encountered at the noted test location(s) only. It is possible that ground conditions could be different away from the point(s) of testing.

Site Investigation Record

Client: Westland District Council c/- RDB Project Management

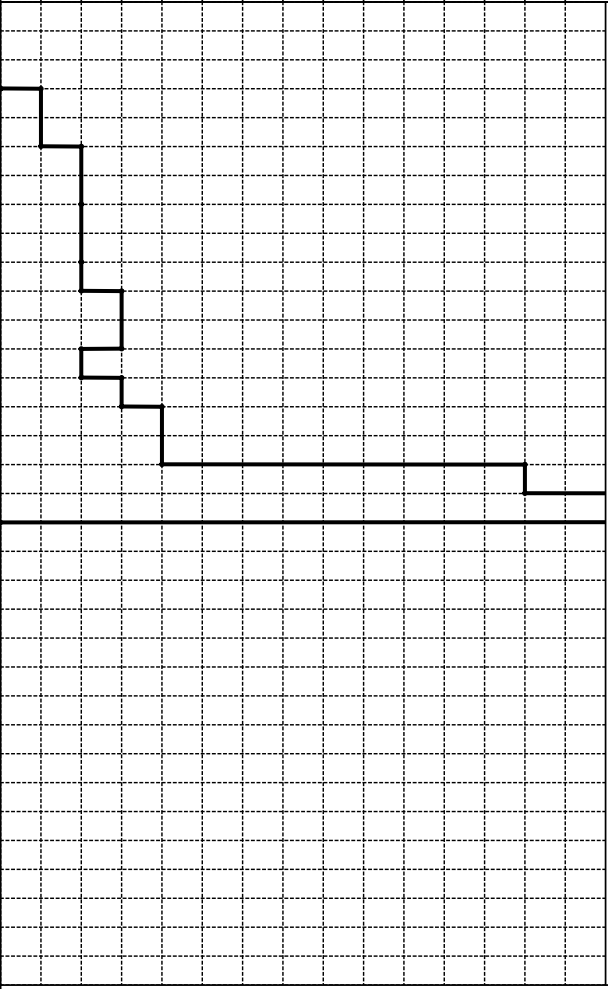
Technical Category:

Date Tested: 22-Apr-2024

Site: 36 & 41 Weld Street, Hokitika

Lot: D.P.:

Log Sheet No.: 1 of 1 Project No.: 503048

Dynamic Cone Penetrometer (DCP) Test Results															Depth (m)	Soil Profile	
Number of Blows per 100mm																Test Location 08	Water
1	2	3	4	5	6	7	8	9	10	11	12	13	14				
															0.2	FILL: GRAVEL, with some cobbles. Moist; Chipseal pavement and basecourse.	Groundwater Not Encountered
															0.4	Sandy SILT, with trace gravel; brown. Moist; gravel, fine.	
															0.6	Gravelly SAND; brown. Moist to wet; gravel, fine.	
															0.8	Clayey SILT; grey. High plasticity; moist.	
															1.0		
															1.2		
															1.4		
															1.6		
															1.8	EOH: 1.7m - Target Depth: Refusal in gravel.	
															2.0		
															2.2		
															2.4		
															2.6		
															2.8		
															3.0		
															3.2		

08

Comments:

Field Staff:	Prepared By:	Soil Profile From:
Geotest	DSH	☒ Hand Auger
Job Manager:	Approved By:	☐ Spade Hole
TIA	AC	☐ Test Pit

Site Plan: (Not to Scale)



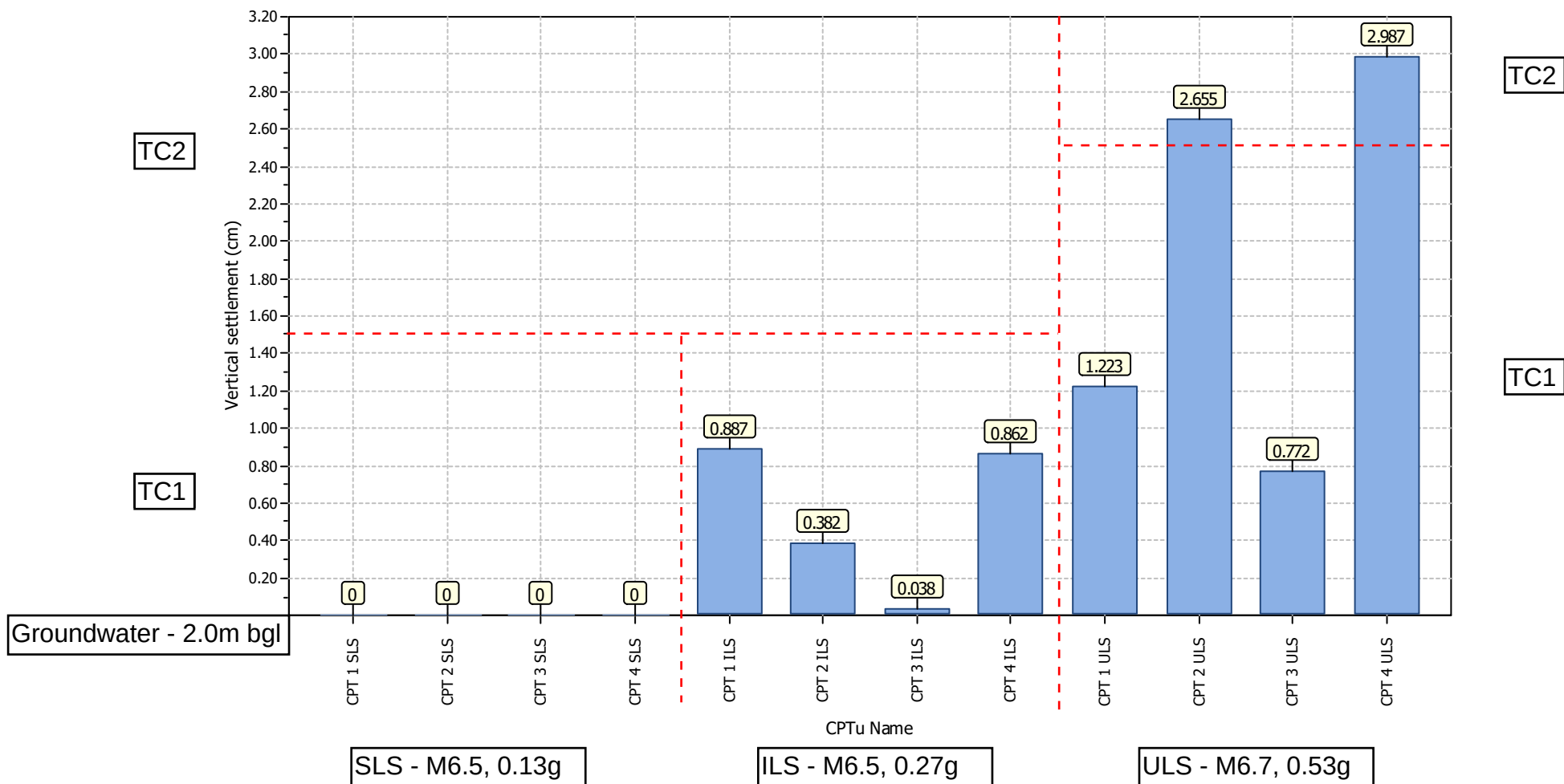
Note: This record identifies the geotechnical conditions encountered at the noted test location(s) only. It is possible that ground conditions could be different away from the point(s) of testing.

Appendix C. Liquefaction analysis

Project title : 36 Weld Street

Location : 36 Weld Street, Hokitika

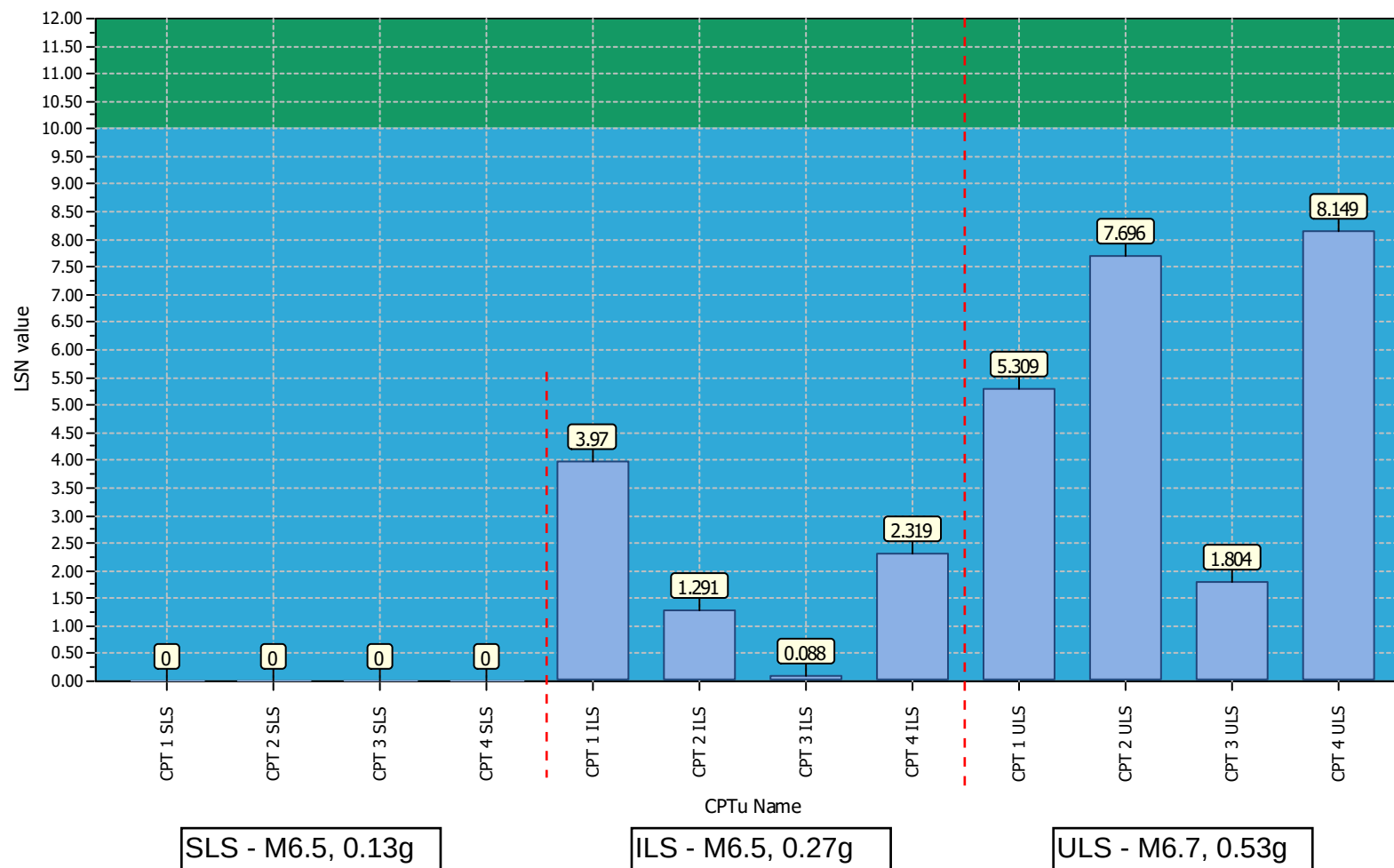
Overall vertical settlements report

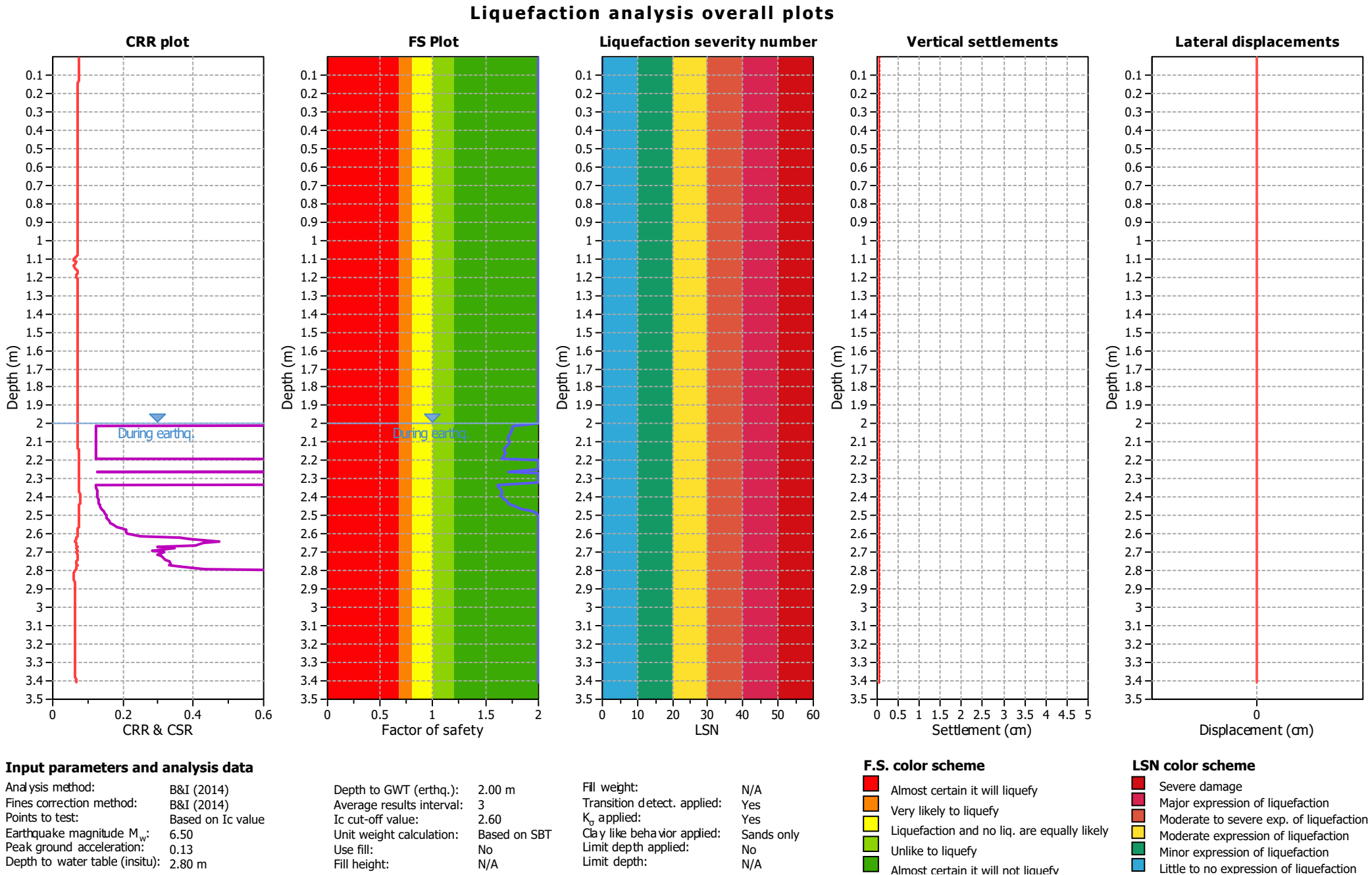


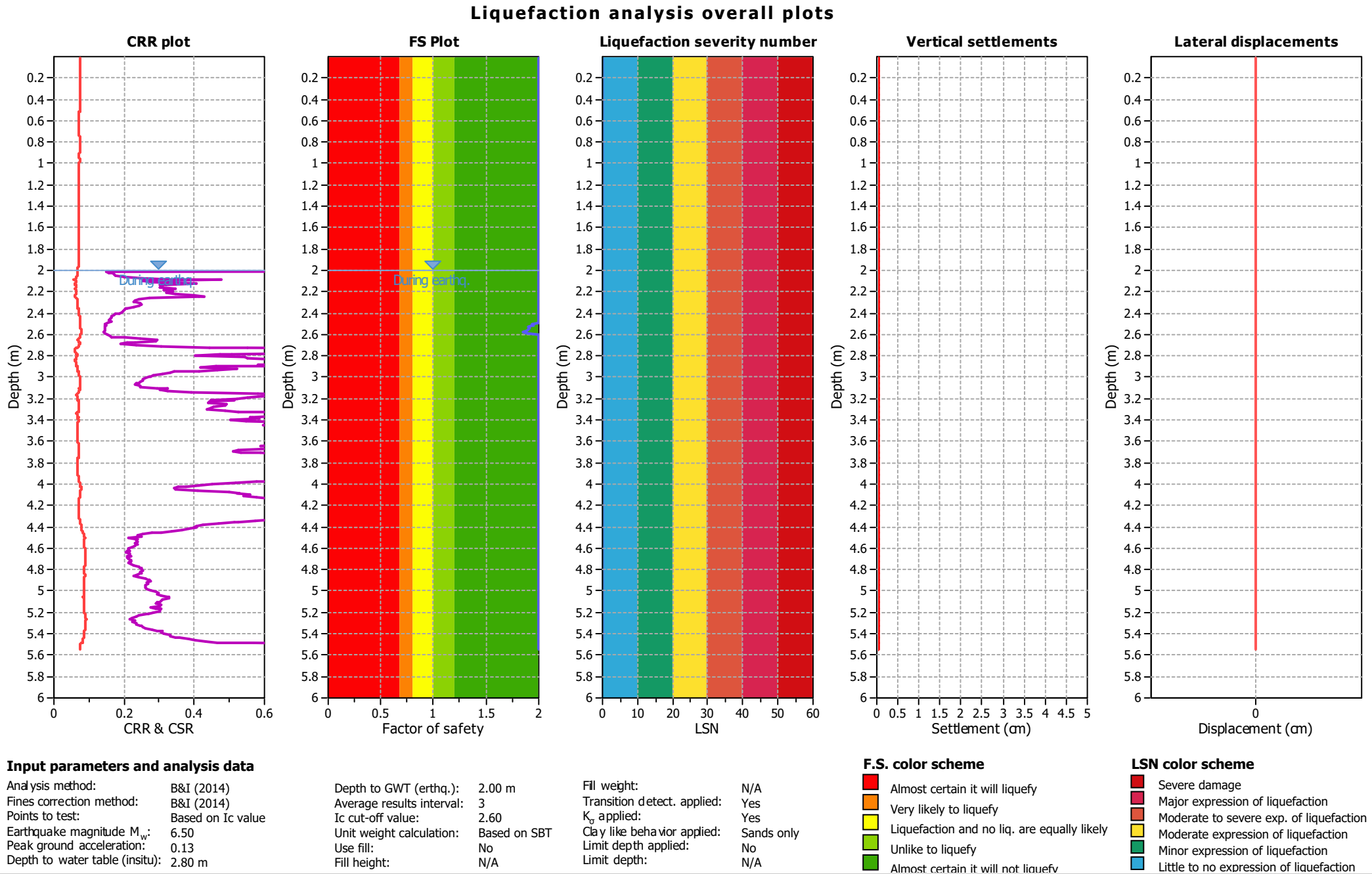
Project title : 36 Weld Street

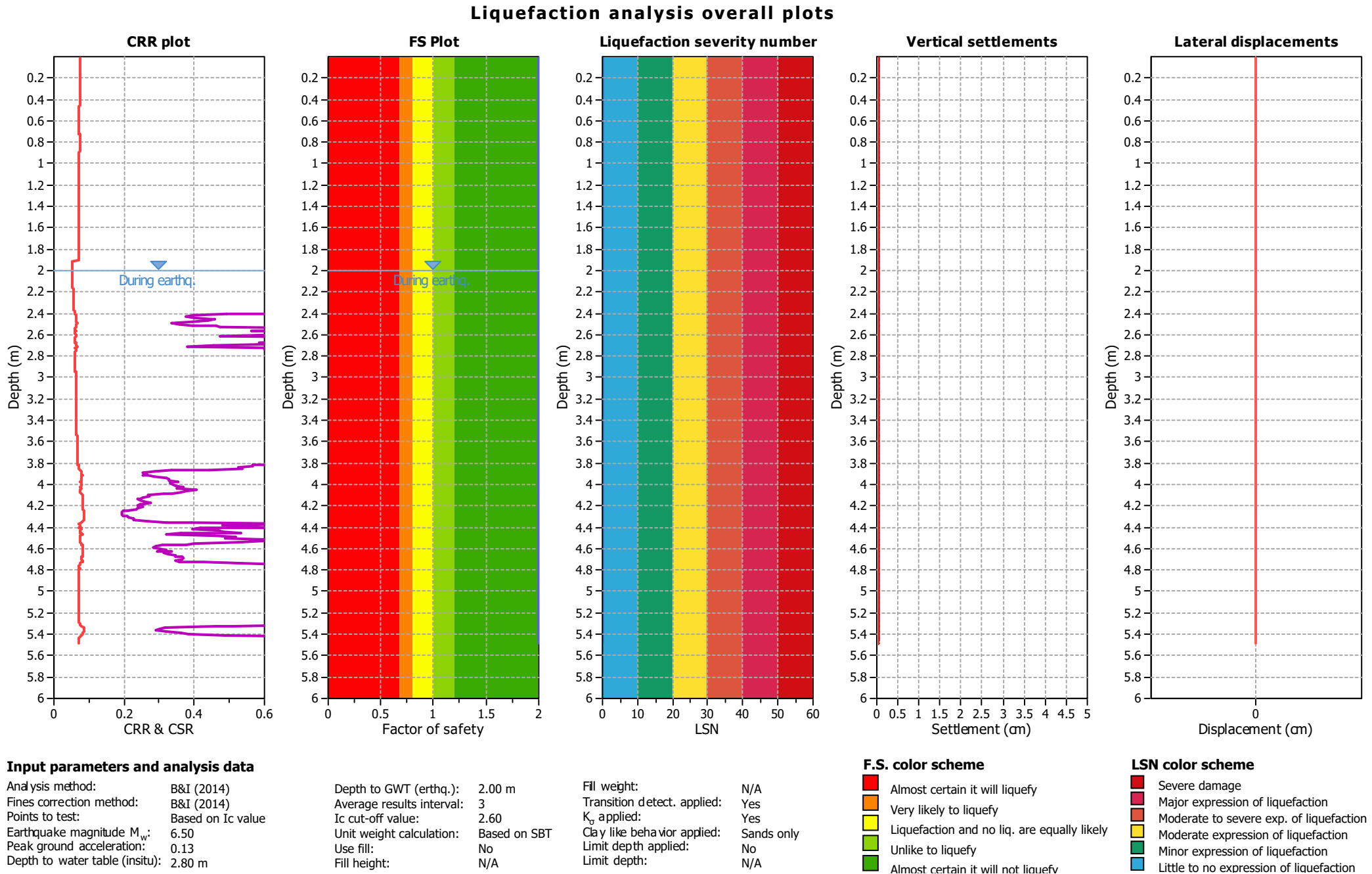
Location : 36 Weld Street, Hokitika

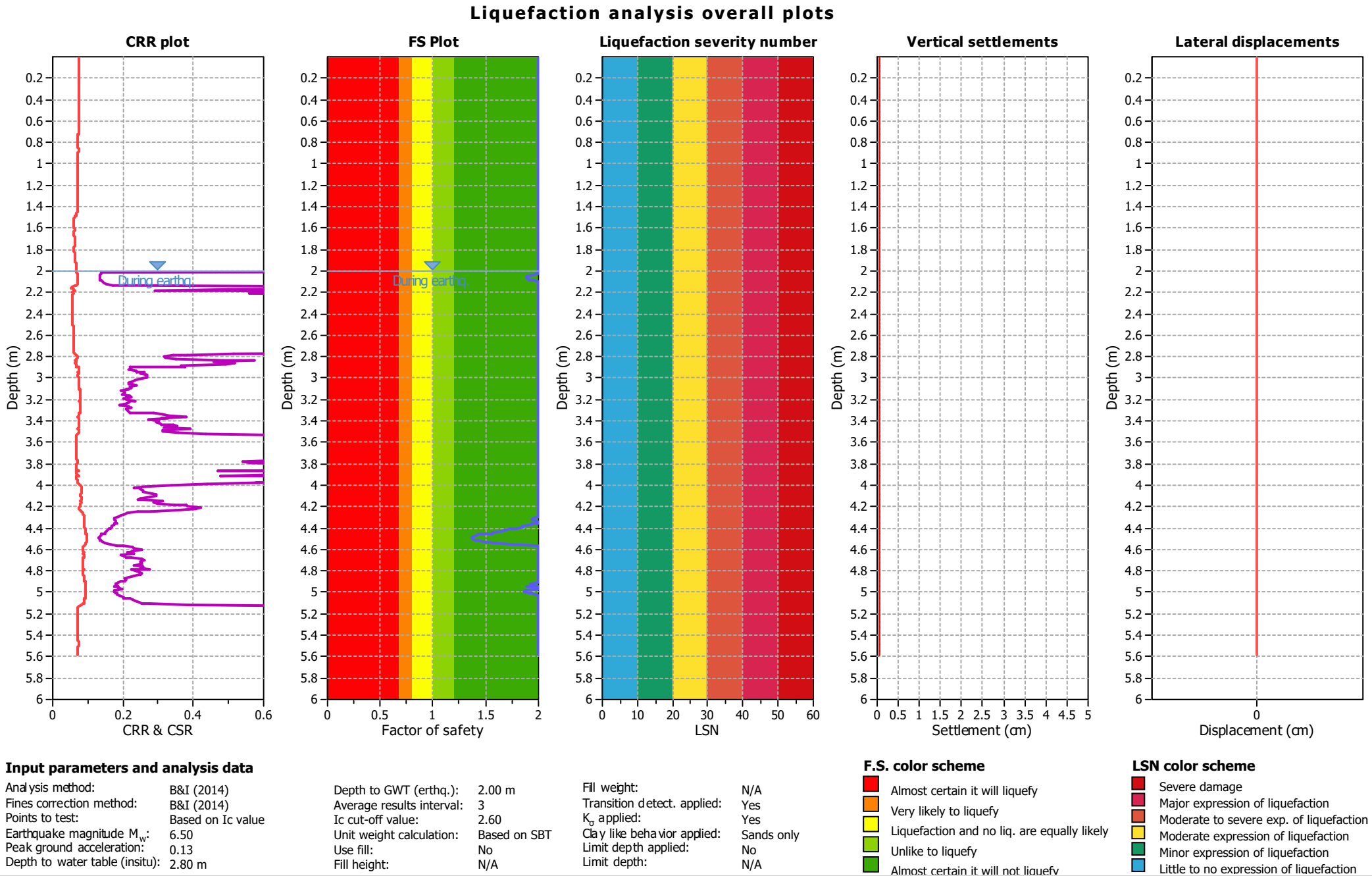
Overall Liquefaction Severity Number report

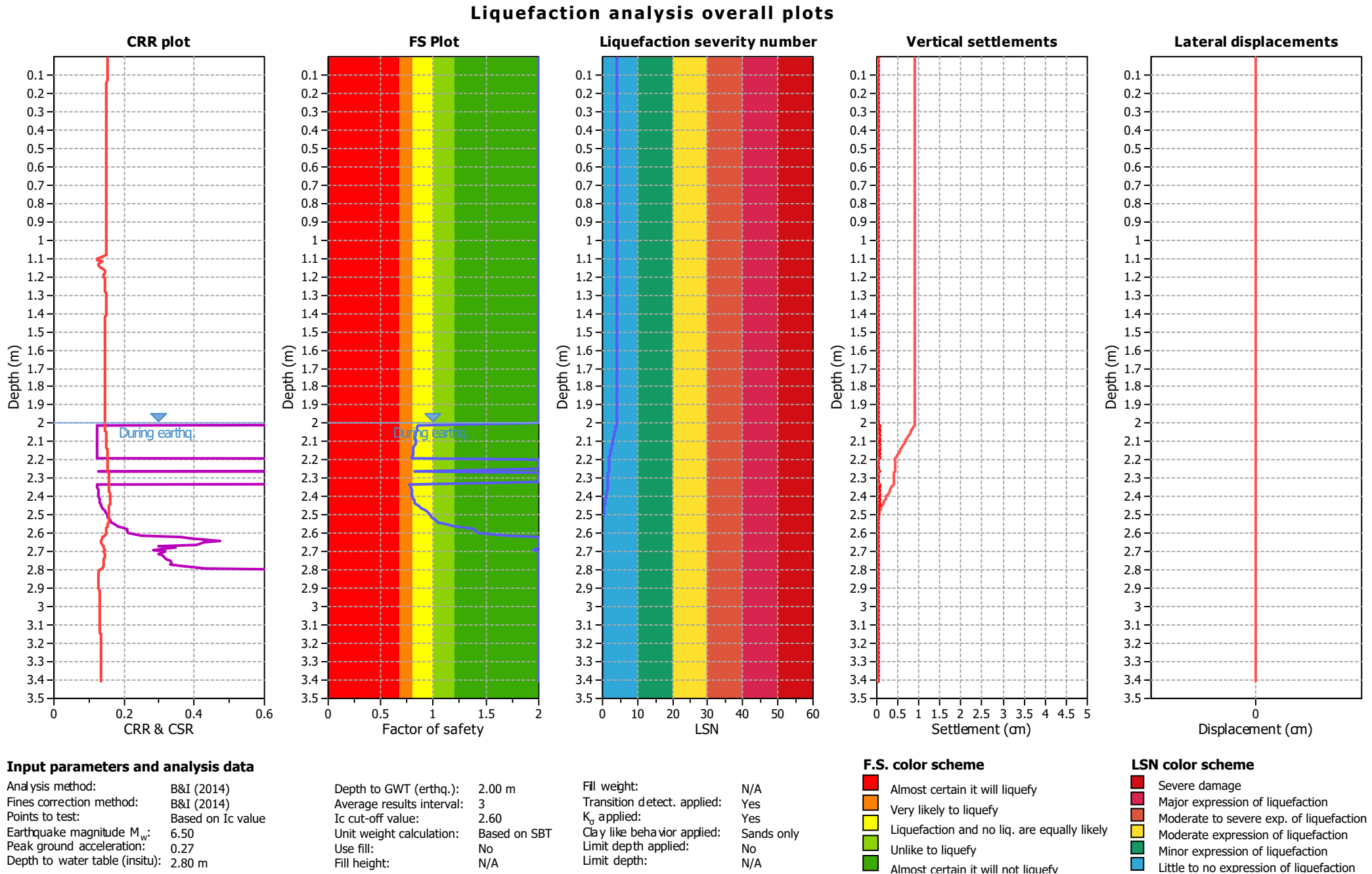


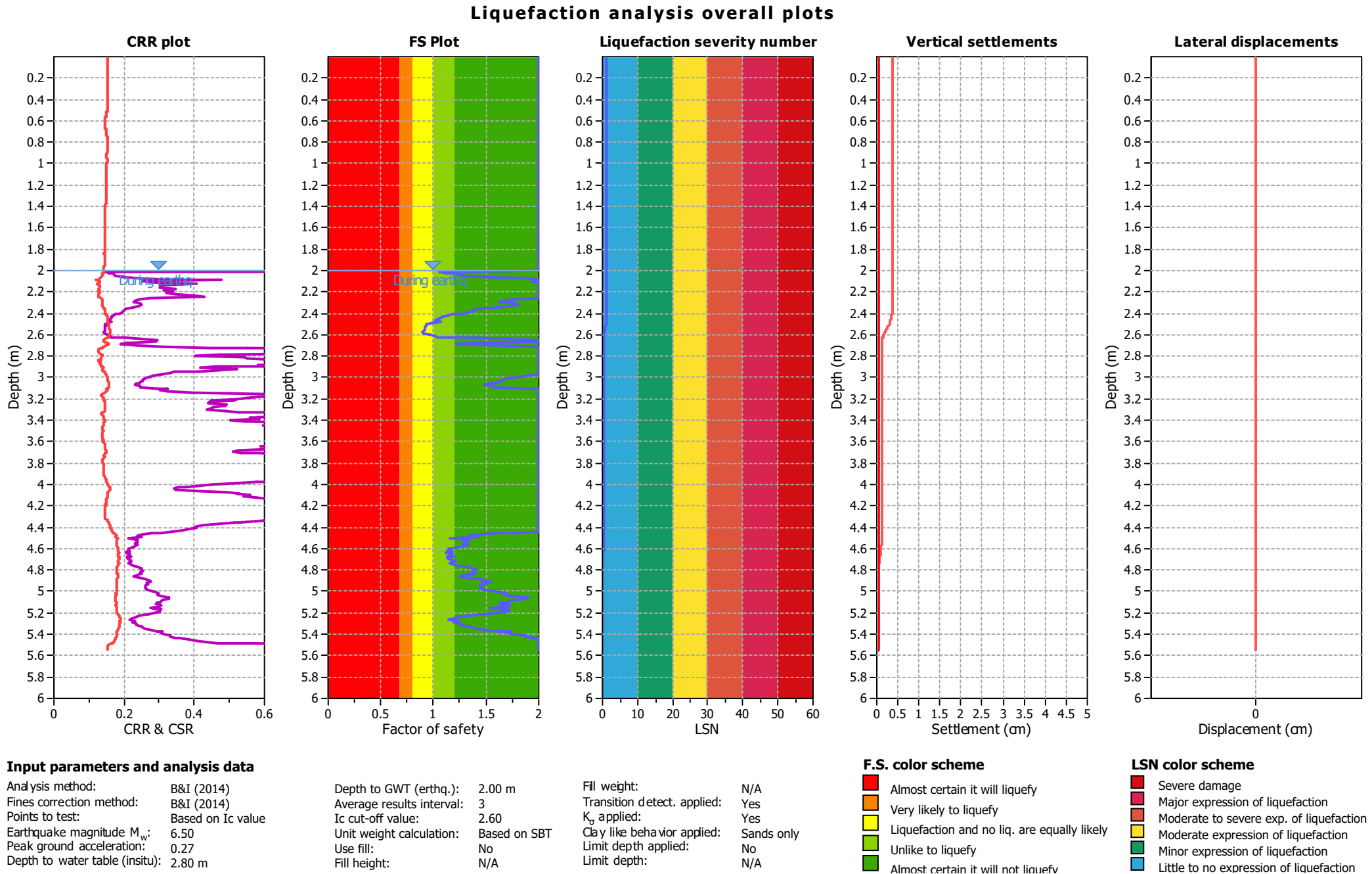


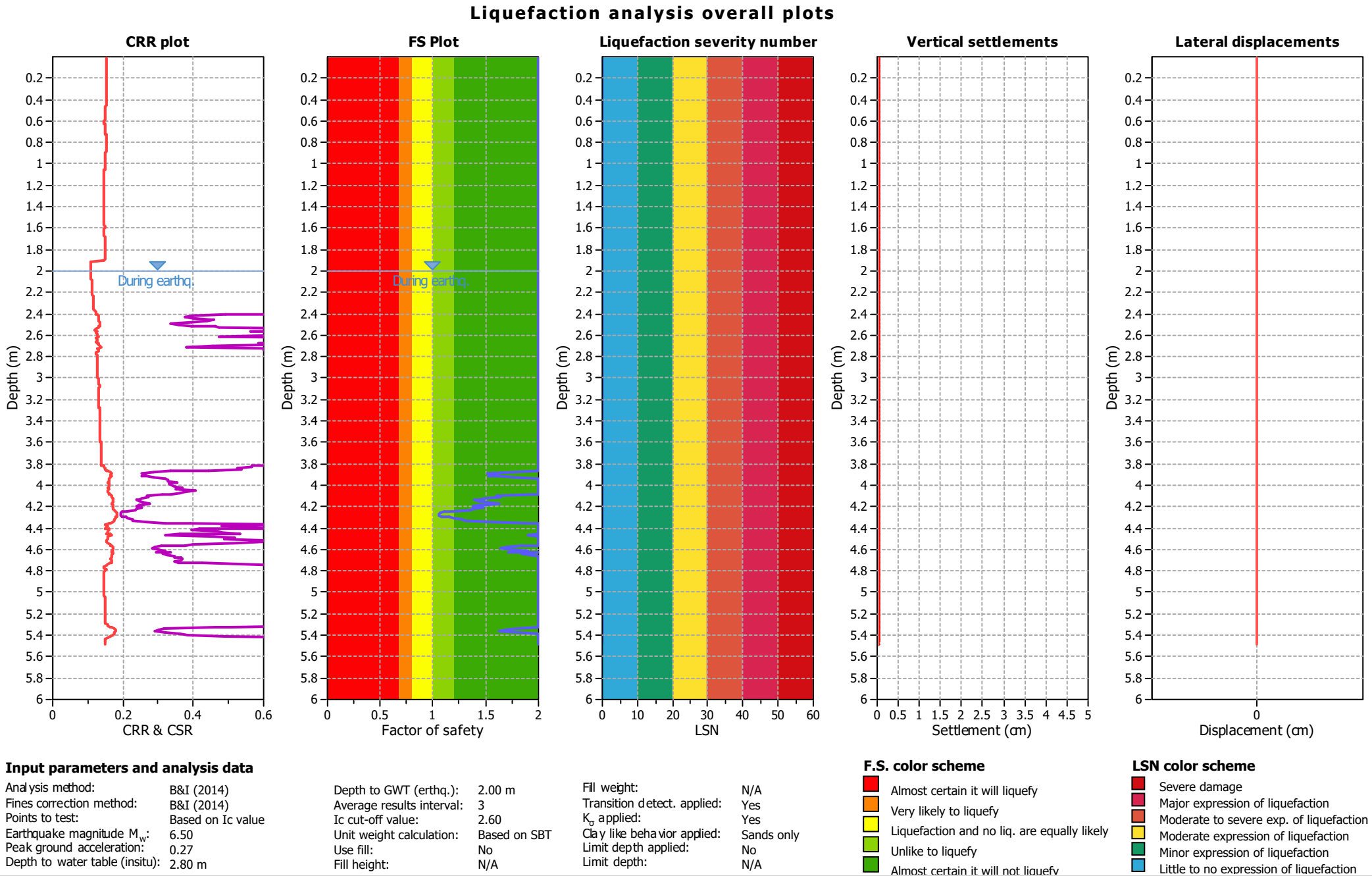




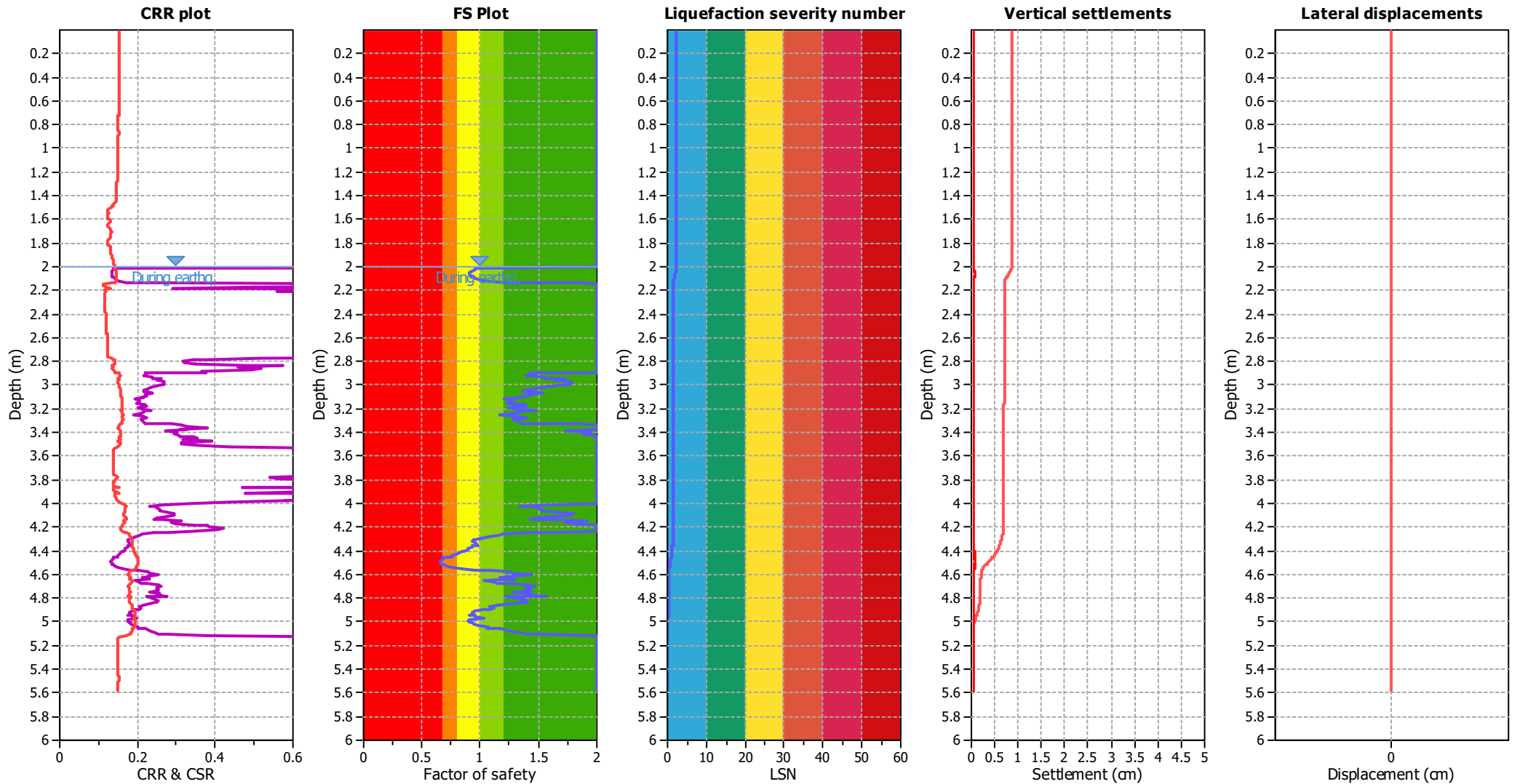








Liquefaction analysis overall plots



Input parameters and analysis data

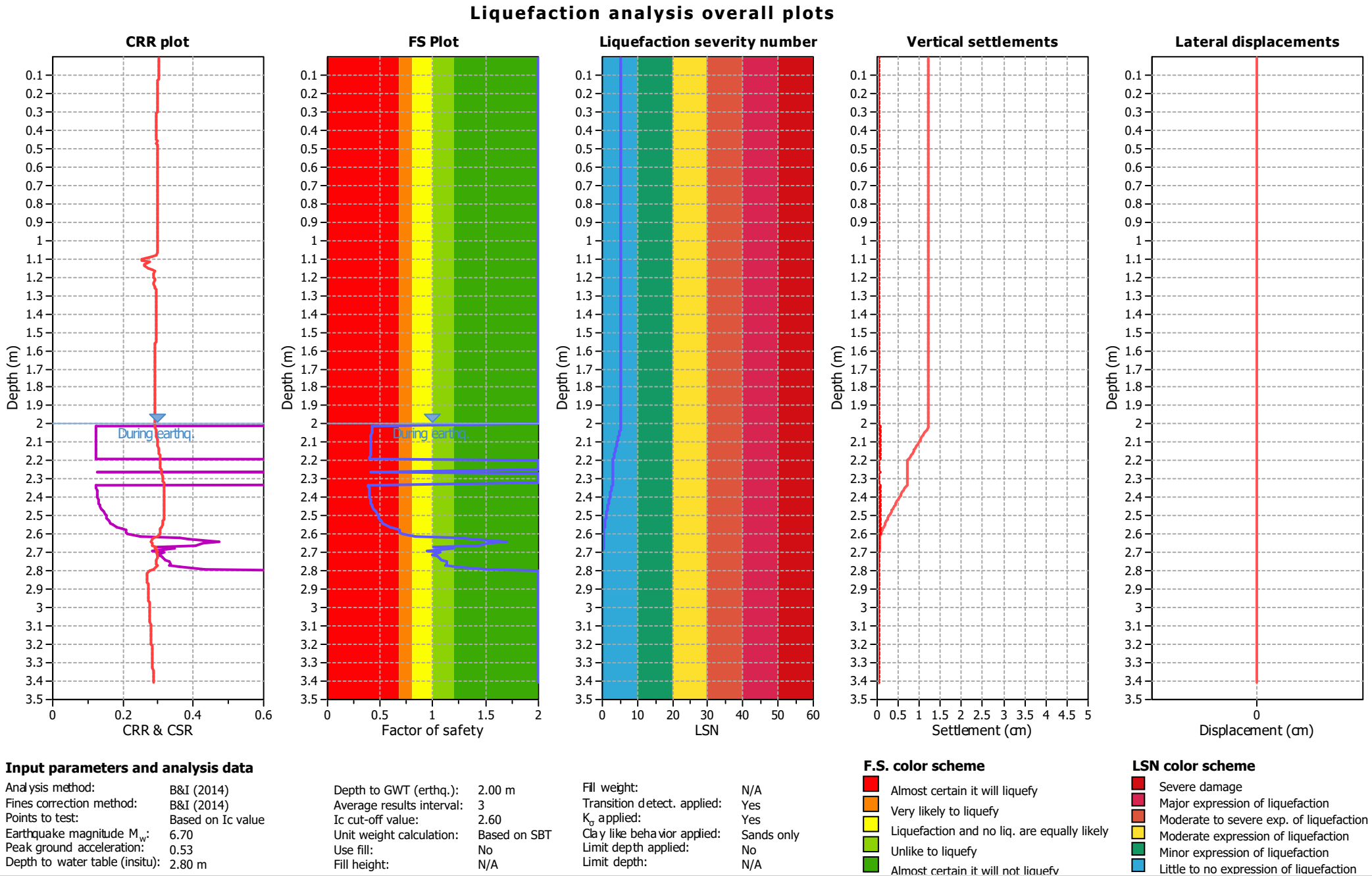
Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.27	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.80 m	Fill height:	N/A	Limit depth:	N/A

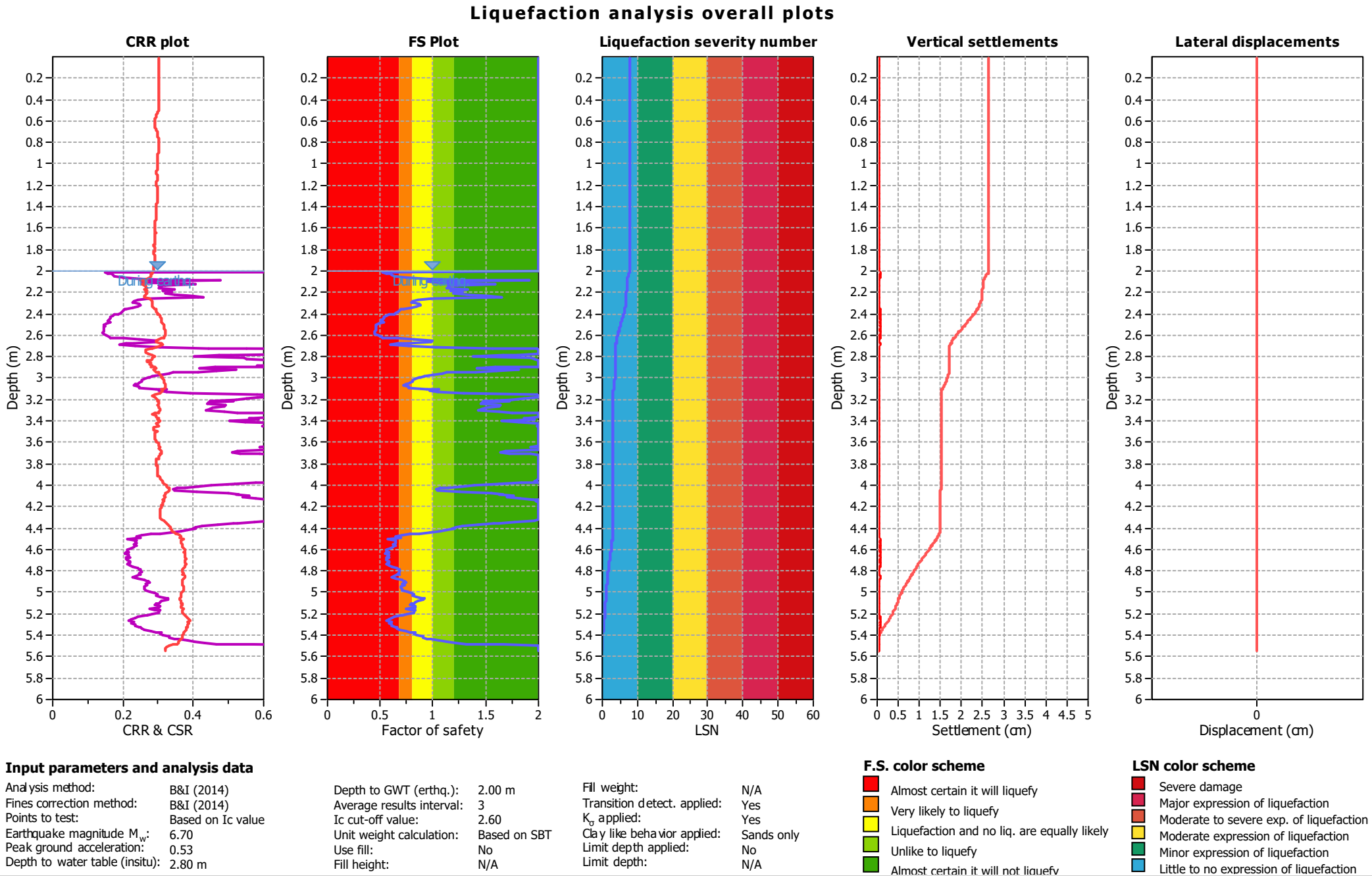
F.S. color scheme

Red	Almost certain it will liquefy
Orange	Very likely to liquefy
Yellow	Liquefaction and no liq. are equally likely
Green	Unlike to liquefy
Dark Green	Almost certain it will not liquefy

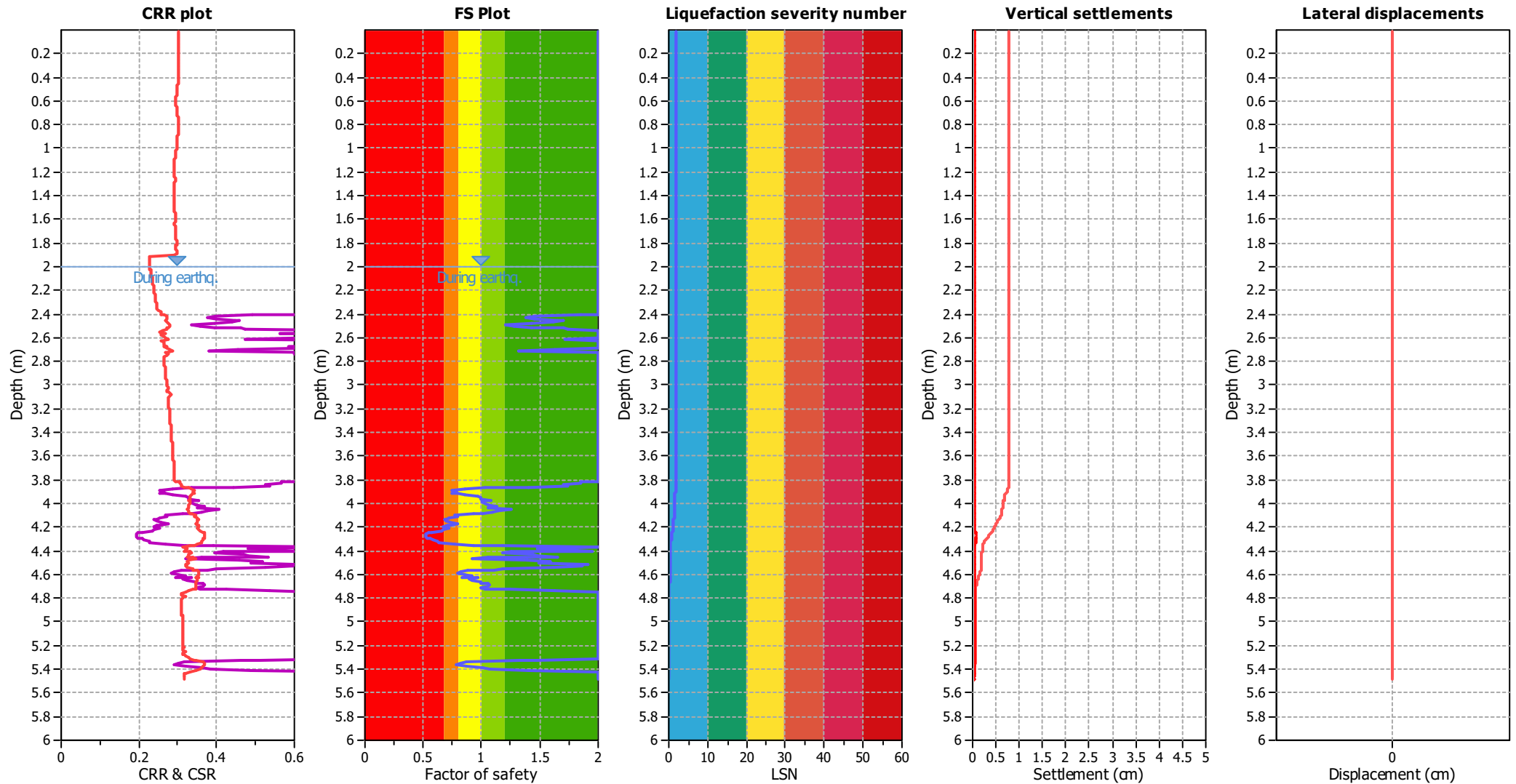
LSN color scheme

Red	Severe damage
Orange	Major expression of liquefaction
Yellow	Moderate to severe exp. of liquefaction
Green	Moderate expression of liquefaction
Dark Green	Minor expression of liquefaction
Blue	Little to no expression of liquefaction





Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method: B&I (2014)
 Fines correction method: B&I (2014)
 Points to test: Based on Ic value
 Earthquake magnitude M_w : 6.70
 Peak ground acceleration: 0.53
 Depth to water table (insitu): 2.80 m

Depth to GWT (earthq.): 2.00 m
 Average results interval: 3
 Ic cut-off value: 2.60
 Unit weight calculation: Based on SBT
 Use fill: No
 Fill height: N/A

Fill weight: N/A
 Transition detect. applied: Yes
 K_0 applied: Yes
 Clay like behavior applied: Sands only
 Limit depth applied: No
 Limit depth: N/A

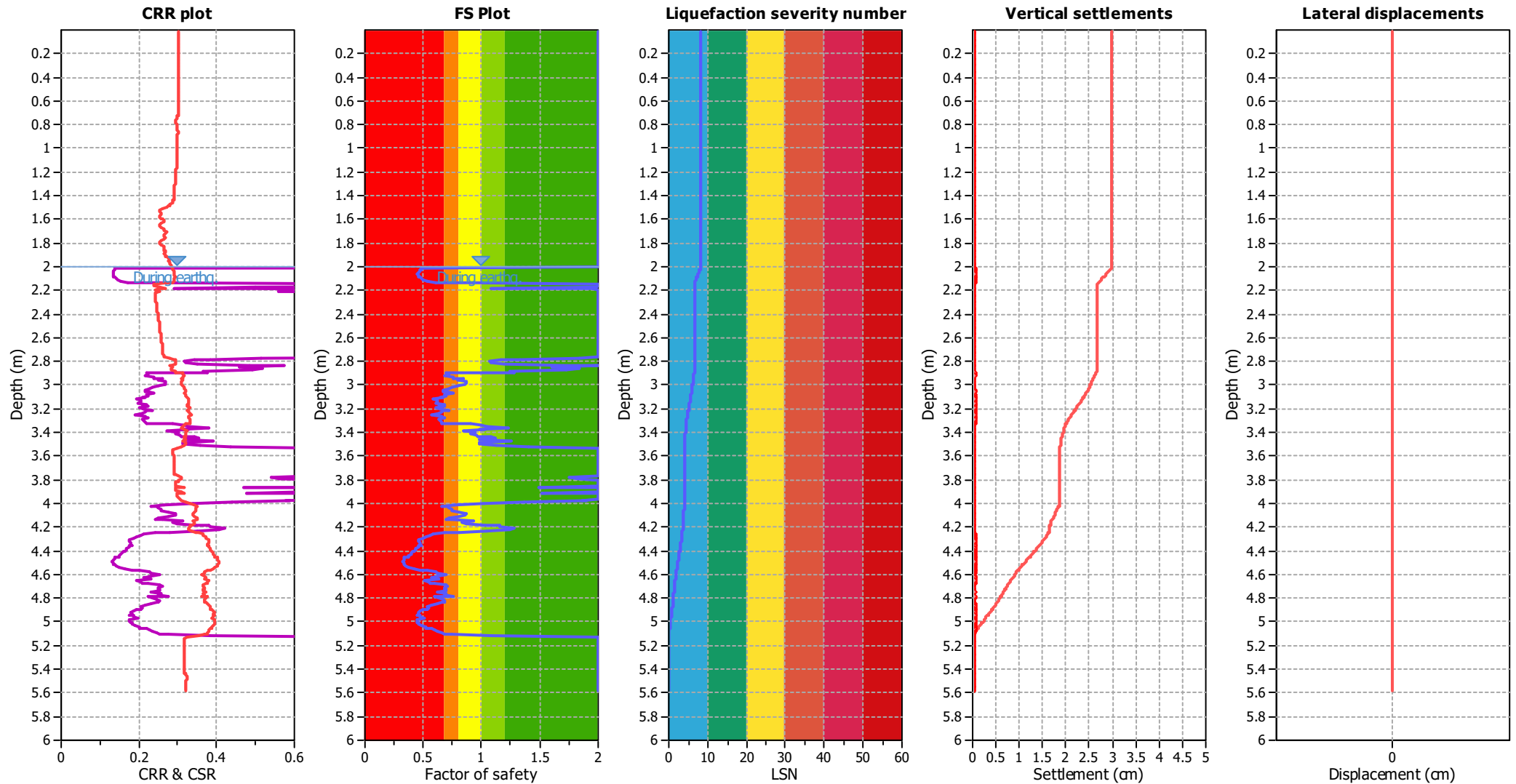
F.S. color scheme

Almost certain it will liquefy
 Very likely to liquefy
 Liquefaction and no liq. are equally likely
 Unlike to liquefy
 Almost certain it will not liquefy

LSN color scheme

Severe damage
 Major expression of liquefaction
 Moderate to severe exp. of liquefaction
 Moderate expression of liquefaction
 Minor expression of liquefaction
 Little to no expression of liquefaction

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_0 applied:	Yes
Earthquake magnitude M_w :	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.53	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.80 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

Red	Almost certain it will liquefy
Orange	Very likely to liquefy
Yellow	Liquefaction and no liq. are equally likely
Light Green	Unlike to liquefy
Dark Green	Almost certain it will not liquefy

LSN color scheme

Red	Severe damage
Dark Red	Major expression of liquefaction
Orange	Moderate to severe exp. of liquefaction
Yellow	Moderate expression of liquefaction
Light Green	Minor expression of liquefaction
Dark Green	Little to no expression of liquefaction

Appendix D. 34 %NBS Concept Upgrade Sketches

NOTES

Clarifications
The primary objective of the seismic strengthening is to reduce the life safety risk to the building occupants during an ultimate limit state earthquake to that associated with a seismic rating of 34%NBS based on an IL2 building in accordance with NZS1170.0:2002.

These concept drawings should be read in conjunction with the associated Detailed Seismic Assessment Report by Eliot Sinclair.

The proposed seismic strengthening work is not specifically intended to reduce the potential for damage to occur to the building during an earthquake. As such, damage is still expected to occur to the building during a significant earthquake.

The scope of the proposed improvement work is limited to structurally designed elements associated with the seismic strengthening of the building. The seismic assessment and strengthening does not consider wind or snow loading or cover building services or fire safety systems, or the building finishes, glazing systems or the weather tightness envelope.

The seismic assessment and strengthening of the building have been undertaken in general accordance with the "Seismic Assessment of Existing Buildings - Technical Guidelines for Engineering Assessments" and the following New Zealand Building Code compliance documents:

- New Zealand Loadings Standards - NZS1170(set)
- New Zealand Concrete Structures Standard - NZS3101:2006
- New Zealand Steel Structures Standard - NZS3404:1997
- New Zealand Timber Structures Standard - NZS3603:1993

The seismic strengthening work is proposed to be undertaken in accordance with Section 112 of the New Zealand Building Act 2004. That is, once the structural building works are completed, the building structure will continue to comply with the requirements of Clause B1 of the New Zealand Building Code to at least the same extent it did prior to the work being undertaken.

Assumptions
The proposed seismic strengthening work detailed herein have been developed to concept level only for the purpose of enabling a contractor to establish a preliminary cost estimate for the work. The design is subject to confirmation of the following:

- Detailed geotechnical investigation and report to confirm the bearing capacity and suitability of the existing site to support the proposed foundation loads.
- Feedback on buildability and construction methodologies from Contractor.
- Coordination with any proposed renovation/refit to the interior fitout.
- Completion of developed and detailed structural design and documentation.
- Building consent from the Westland District Council who may require upgrades of fire safety systems and accessible features.

Construction Methodology/Sequence
The following sets out the assumed construction methodology & sequence assumed as part of the concept design:

1. Strip out the existing internal fitout to the extent required to accommodate the proposed new fitout works.
2. Cut out sections of floor slab and excavate through existing subbase fill material as required to accommodate the proposed new foundation beams & pads. Notify the Geotech Engineer to review the founding subgrade.
3. Roughen the faces existing foundation beams where they join to the proposed new foundations. Roughening to NZS3109 Type B.
4. Drill and clean all holes required to the exposed faces of the existing foundations ready to receive new starter bars.
5. Mobilise a small excavator with a 500dia posthole auger attachment capable of augering to a depth of ~2.0m below the base of the excavated foundations. Refer to drawings for locations. Notify the Geotech Engineer to review the auger holes.
6. Place posthole reinforcing cages. Notify the Structural Engineer to review reinforcing prior to pouring concrete up to the underside of the proposed foundations. Roughen top of postholes to NZS3109 Type B.
7. Place foundation reinforcing cages and epoxy in starter bars. Notify the Structural Engineer to review foundation reinforcing pour to poring foundation concrete up to finished slab level.
8. Remove existing paint and roughen the faces of all existing concrete walls which are to receive new concrete strengthening walls. Roughening to NZS3109 Type B.
9. Place all reinforcing cages complete with all necessary drill and epoxy starter bars. Notify the Structural Engineer to review reinforcing.
10. New concrete strengthening walls can be boxed and poured insitu concrete or constructed with sprayed concrete e.g. <http://www.southislandshotcrete.co.nz/>
11. Install all structural steel & timber strengthening work as detailed in the structural drawings. Notify the Structural Engineer to review prior to closing in.

General

1. All work shall comply with the New Zealand Building Code.
2. Do not scale. Refer any discrepancies to the Architect/Engineer.
3. The Contractor shall check all dimensions onsite prior to commencing work.
4. The Contractor shall provide Producer Statements for the following work trades:
 - Main contractor – PS3
 - Site reinforced and poured/sprayed concrete – PS3
 - Structural steelwork fabrication & erection – PS3
5. The form of the producer statements shall be equivalent to the Christchurch City Council standard form B-085.

Sediment Control Management Plan

1. The Contractor/Site Manager is responsible for providing effective erosion protection and sediment control during the entire construction period. Refer to the Westcoast Regional Council for guidance.
2. Sediment control measures shall be taken where appropriate to remove coarse silt and debris from stormwater runoff leaving the site, either overland, via a piped stormwater system.
3. The effectiveness of the measures is to be reviewed immediately after rain or at least weekly by the Contractor and, if necessary, further controls put in place to prevent excess sediment or debris from entering the Westland District Council stormwater system and waterways.
4. The Contractor shall undertake any other practical measure at their cost to comply with good erosion and sediment control practice.

Excavations & Hardfill

1. Excavations for the foundations and ground slab are to be inspected by the Geotechnical Engineer to confirm an ultimate bearing capacity of 380kPa. The final depth of excavation shall be determined by the Geotechnical Engineer.
2. Prior to pouring concrete, the foundation excavations shall be thoroughly cleaned of all water and loose materials.
3. Provide a minimum 150mm thick layer of AP40 hardfill below all ground slabs. Hardfill shall be compacted to a minimum dry density of 2150kg/m3.

Concrete

1. All concrete work and associated reinforcing shall comply with the requirements of NZS3109:1997 'Concrete Construction'.
2. Concrete mixes shall comply with NZS3109 & NZS3104 be as follows:
 - Foundation concrete: 25MPa compressive strength, 19mm aggregate, normal grade.
 - Sprayed/poured concrete walls: 40MPa compressive strength, 13mm aggregate, special grade.
 - Other concrete: 30MPa compressive strength, 19mm aggregate, normal grade.
 - Refer to the Architect for any special finishing requirements for exterior paths, patios and the driveway.
3. All concrete to be well consolidated by a mechanical vibrator and carefully worked around reinforcement and into corners of the formwork.
4. Epoxy resin for installation of reinforcing starters bars and steel studs/anhors shall be Hilti HIT-RE 500V4.
5. The interior concrete surface finish shall comply with 'U3' in accordance with NZS 3114.

Reinforcing

1. Reinforcement steel must comply with the requirements of AS/NZS 4671:2001: Bar designations shown on drawings are to be interpreted as follows -
 - Bars prefixed H – Grade 500E MA, deformed
 - Bars prefixed D – Grade 300E, deformed
 - Bars prefixed HR – Grade 500E MA, plain
 - Bars prefixed R – Grade 300E, plain
 - Mesh prefix SE – Grade 500E MA
2. Minimum lap length for D bars to be 40 x bar diameter; for H bars to be 60 x bar diameter.
3. All bars not lapped are to terminate with a 90° bends unless noted otherwise.
4. Minimum concrete covers (unless stated otherwise):
 - 75mm side and bottom cover against ground;
 - 50mm top, bottom and side cover against boxing, DPM and exposed to exterior environment;
 - 30mm if protected from weather (i.e. internal);
 - All other situations to be as per NZS 3101:2006 unless shown otherwise on the drawings.
5. All mesh to be Grade 500E Ductility Class E welded wire mesh, with 225mm min lap or to Manufacturer's specification, whichever is greater.

Steelwork

1. All steelwork, fabrication, welding and erection shall comply with NZS3404:1997.
2. All Cold formed steel hollow sections (CHS, SHS & RHS) shall comply with AS/NZS 1163:2016 grade C350L0.
3. All hot-rolled bars and sections (UB, UC, PFC, EA, UA & bars, etc) shall comply with AS/NZS3679:2016 grade 300.
4. All steel plate shall comply with AS/NZS 3678:2016 grade 350.
5. Welding electrodes shall be selected for the grade of steel being welded and in accordance with AS/NZS 1554. The nominal tensile strength of the weld material shall not be less than 480MPa.
6. Unless noted otherwise in the structural drawings, all lines of contact shall be welded using 6mm structural purpose fillet weld all round unless noted otherwise.
7. Welding inspection and quality control shall comply with NZS 3404, AS/NZS 5131 and AS/NZS 1554 as appropriate for the welding being undertaken. The extent of non-destructive examination shall be as set out below:
 - 100% of all SP & GP welds shall be visually scanned.
 - 100% of all full penetration butt welds shall be Visually examined (VT).
 - 100% of full penetration butt welds to the portal frame knee joint stiffeners shall be ultrasonically tested (UT).
8. The various methods of Non-Destructive Examination shall be in accordance with Section 6 of AS/NZS 1554.1 or AS/NZS 1554.5 as appropriate. Imperfection levels shall not exceed the maximum permissible levels given in Section 6 of AS/NZS 1554.1. It is the Contractor's responsibility to clearly demonstrate that all testing requirements of this specification have been met.
9. Unless noted otherwise in the structural drawings, all bolts shall be M20 8.8/S hot dip galvanised.
10. Holes for bolts to be 2mm larger diameter than the bolt diameter, unless noted otherwise.
11. All interior steelwork shall be prepared and prime painted with Dulux Zincanode 402 in accordance with the Dulux specification [DuSpec NZSD1053](#).
12. All exterior structural steel shall be hot dip galvanised to AS/NZS2312: thermal Contractor's responsibility to clearly demonstrate that all testing requirements of this specification have been met.
13. Unless noted otherwise in the structural drawings, all bolts shall be M20 8.8/S hot dip galvanised.
14. Holes for bolts to be 2mm larger diameter than the bolt diameter, unless noted otherwise.
15. All interior steelwork shall be prepared and prime painted with Dulux Zincanode 402 in accordance with the Dulux specification DuSpec NZSD1053.
16. All exterior structural steel shall be hot dip galvanised to AS/NZS2312: HDG600 or thermal zinc sprayed to TSZ300S. Refer to the Architect for specification of topcoats and colours.



NOTES
1. Contractors to verify all dimensions and the location of all underground services on site prior to commencing work.
2. Unless noted otherwise, all work shall be undertaken in accordance with the NZBC and any relevant Territorial Authority Engineering Standards and Specifications as a minimum standard.
DISCLAIMER
© Eliot Sinclair and Partners Ltd. This drawing and all its information is only to be used for its intended purpose. All rights reserved.

REV.	DRAWN	DATE	NOTE
A	SR	25.09.26	Preliminary Concept

CLIENT



DESIGNED	XX
DRAWN	XX
REVIEWED	XX
APPROVED	XX
STATUS	PRELIMINARY
SCALE	NTS

34% NBS SEISMIC STRENGTHENING CONCEPT

36 Weld Street, Hokitika

GENERAL NOTES & SITE PLAN

PROJECT	REV.
503048	A
SET	SHEET
SC	S01





NOTES

1. Contractors to verify all dimensions and the location of all underground services on site prior to commencing work.

2. Unless noted otherwise, all work shall be undertaken in accordance with the NZBC and any relevant Territorial Authority Engineering Standards and Specifications as a minimum standard.

DISCLAIMER

© Eliot Sinclair and Partners Ltd. This drawing and all its information is only to be used for its intended purpose. All rights reserved.

REV.	DRAWN	DATE	NOTE
A	SR	25.09.26	Preliminary

CLIENT



DESIGNED
DRAWN
REVIEWED
APPROVED

xx
xx
xx
xx

STATUS
SCALE

PRELIMINARY
1:100 [A2]

34% NBS SEISMIC
STRENGTHENING CONCEPT

36 Weld Street, Hokitika

FOUNDATION PLAN

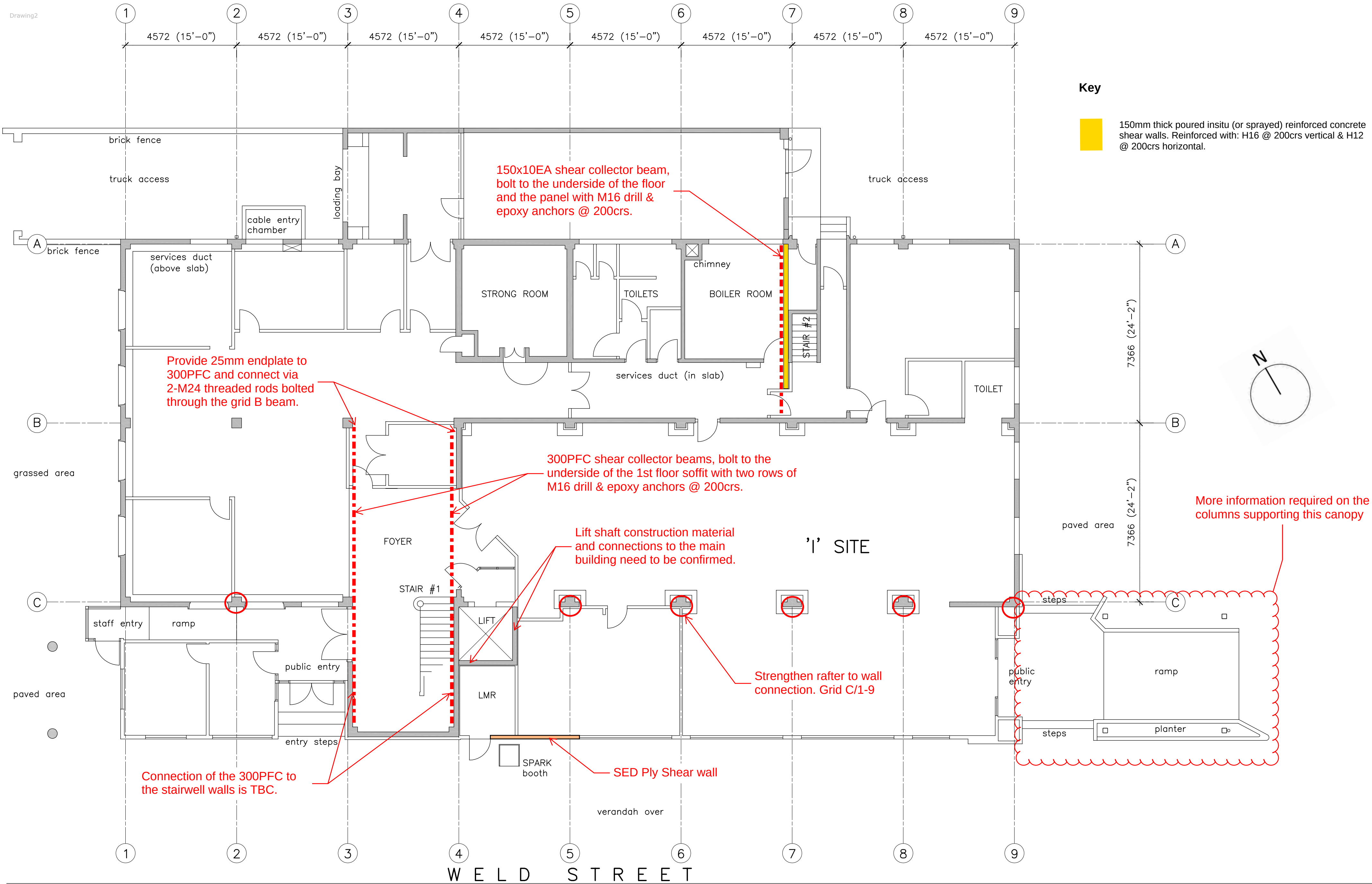
PROJECT
503048

REV.
A

SET
SC

SHEET
S02





NOTES

1. Contractors to verify all dimensions and the location of all underground services on site prior to commencing work.

2. Unless noted otherwise, all work shall be undertaken in accordance with the NZBC and any relevant Territorial Authority Engineering Standards and Specifications as a minimum standard.

DISCLAIMER

© Eliot Sinclair and Partners Ltd. This drawing and all its information is only to be used for its intended purpose. All rights reserved.

REV.	DRAWN	DATE	NOTE
A	SR	25.09.26	Preliminary Concept

CLIENT



DESIGNED
DRAWN
REVIEWED
APPROVED

xx
xx
xx
xx

STATUS
SCALE

PRELIMINARY
1:XXX [A1]

34% NBS SEISMIC
STRENGTHENING CONCEPT

36 Weld Street, Hokitika

GROUND FLOOR PLAN

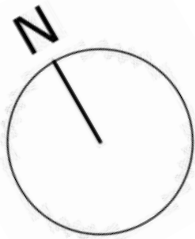
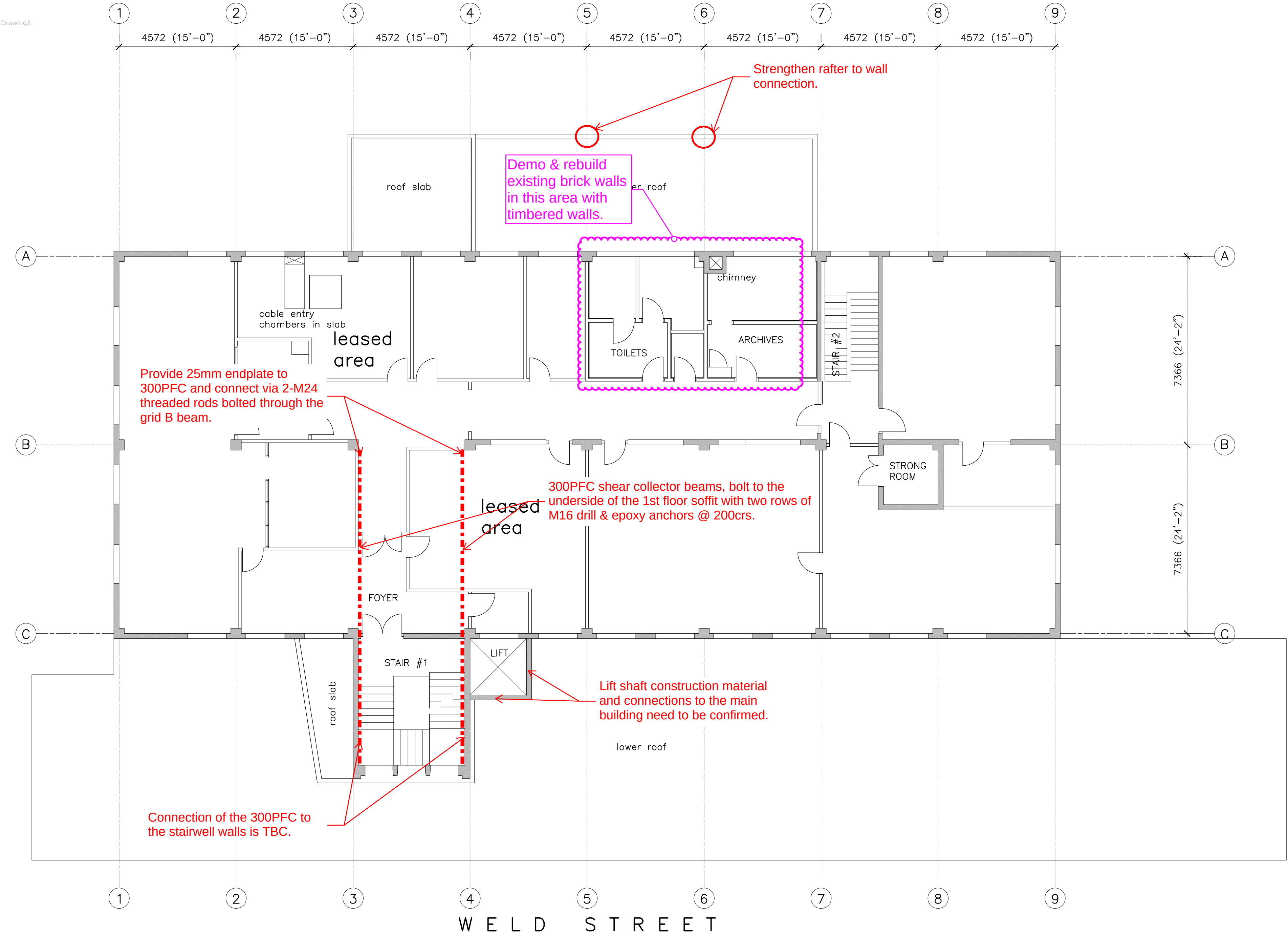
PROJECT
503048

REV.
A

SET
SC

SHEET
S03





NOTES

1. Contractors to verify all dimensions and the location of all underground services on site prior to commencing work.

2. Unless noted otherwise, all work shall be undertaken in accordance with the NZBC and any relevant Territorial Authority Engineering Standards and Specifications as a minimum standard.

DISCLAIMER

© Eliot Sinclair and Partners Ltd. This drawing and all its information is only to be used for its intended purpose. All rights reserved.

REV.	DRAWN	DATE	NOTE
A	SR	25.09.26	Preliminary Concept

CLIENT



DESIGNED	xx
DRAWN	xx
REVIEWED	xx
APPROVED	xx
STATUS	PRELIMINARY
SCALE	1:100 [A2]

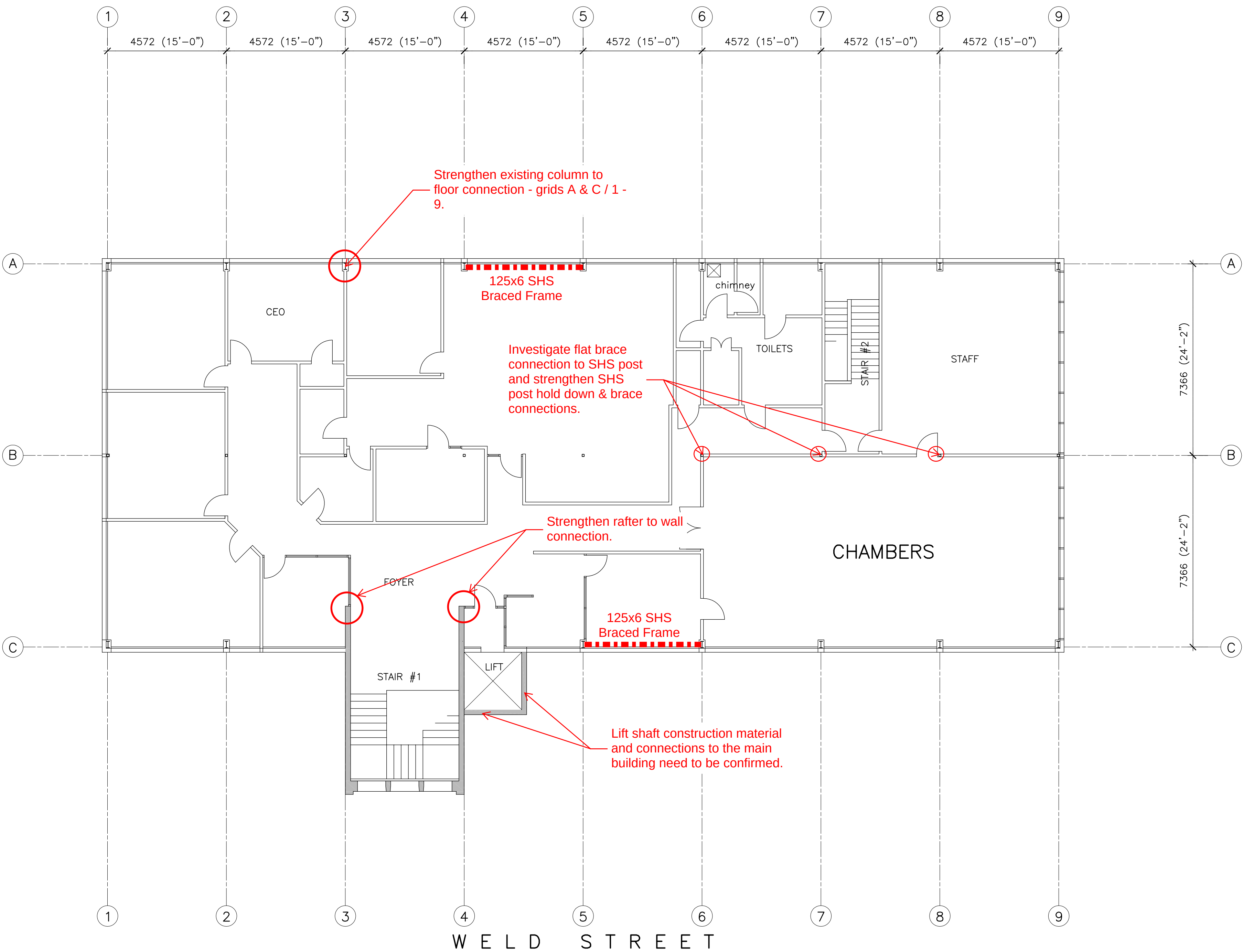
34% NBS SEISMIC STRENGTHENING CONCEPT

36 Weld Street, Hokitika

FIRST FLOOR PLAN

PROJECT	REV.
503048	A
SET	SHEET
SC	S04





NOTES

1. Contractors to verify all dimensions and the location of all underground services on site prior to commencing work.

2. Unless noted otherwise, all work shall be undertaken in accordance with the NZBC and any relevant Territorial Authority Engineering Standards and Specifications as a minimum standard.

DISCLAIMER

© Eliot Sinclair and Partners Ltd. This drawing and all its information is only to be used for its intended purpose. All rights reserved.

REV.	DRAWN	DATE	NOTE
A	SR	25.09.26	Preliminary Concept

CLIENT



DESIGNED
DRAWN
REVIEWED
APPROVED

xx
xx
xx
xx

STATUS
SCALE

PRELIMINARY
1:100 [A2]

34% NBS SEISMIC
STRENGTHENING CONCEPT

36 Weld Street, Hokitika

SECOND FLOOR PLAN

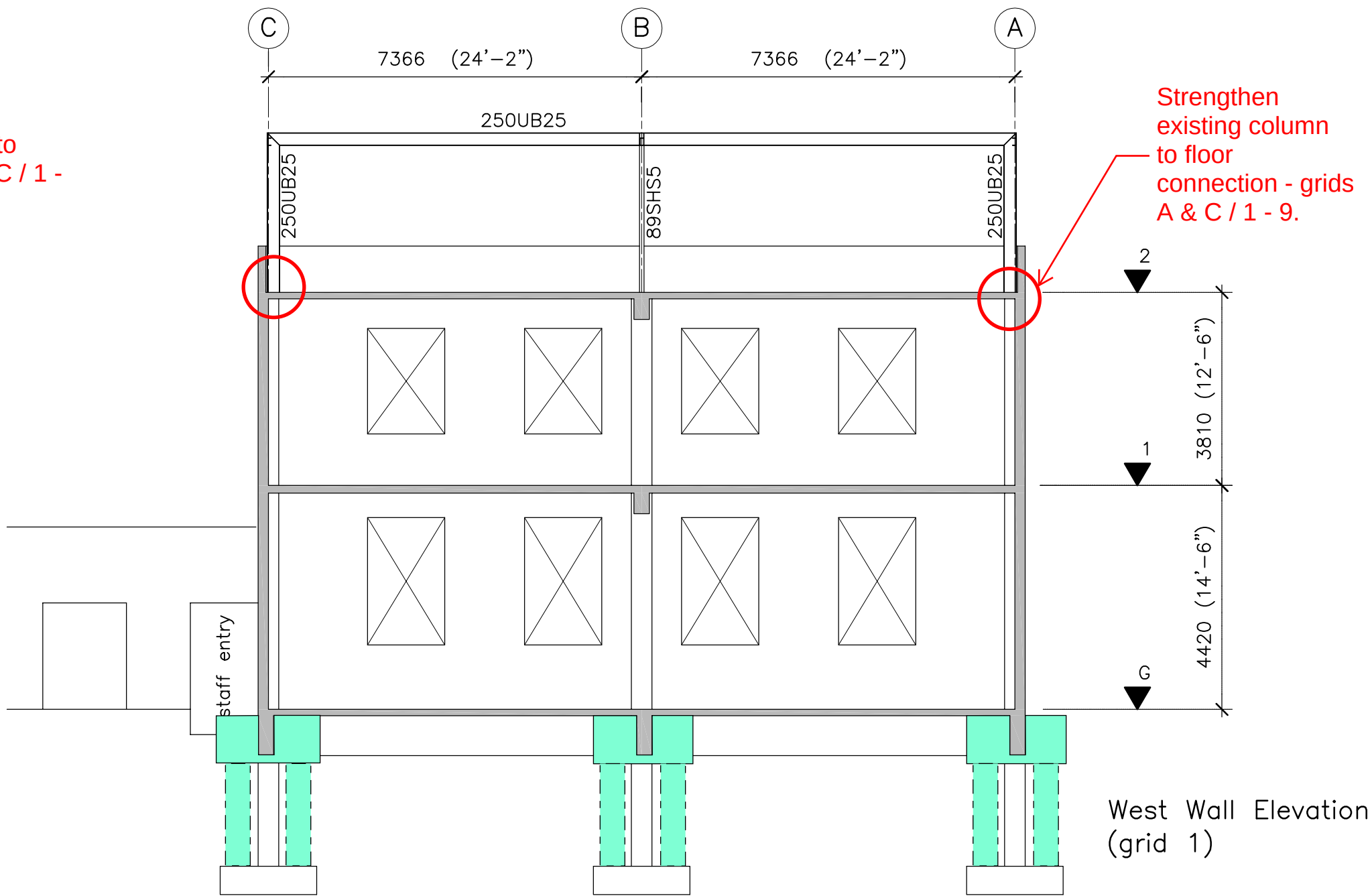
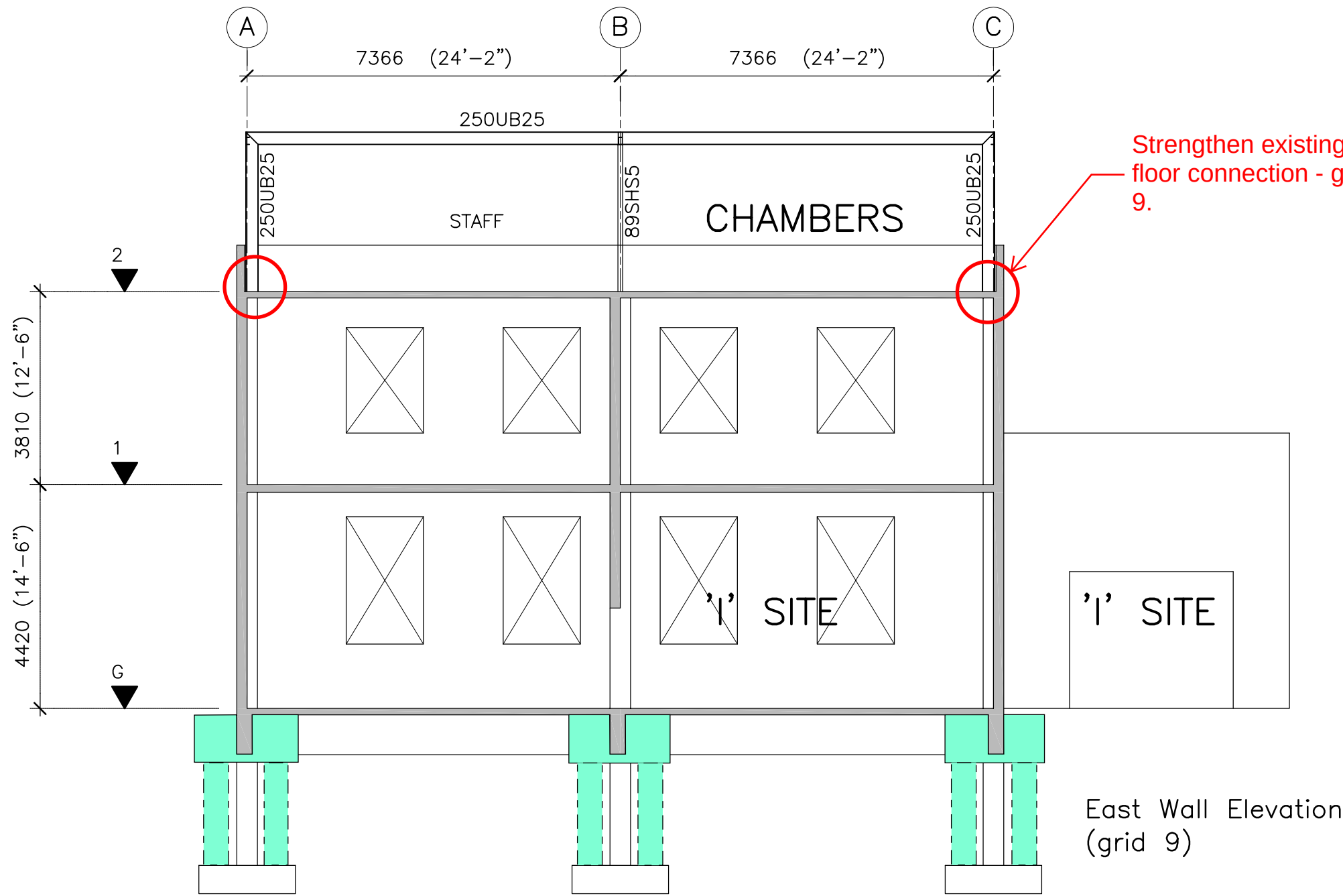
PROJECT
503048

REV.
A

SET
SC

SHEET
S05





KEY

 = Poured insitu reinforced concrete foundations. Refer to sheet S02 & S10-S11 for extent & indicative details.

NOTES

- Contractors to verify all dimensions and the location of all underground services on site prior to commencing work.
- Unless noted otherwise, all work shall be undertaken in accordance with the NZBC and any relevant Territorial Authority Engineering Standards and Specifications as a minimum standard.

DISCLAIMER

© Eliot Sinclair and Partners Ltd. This drawing and all its information is only to be used for its intended purpose. All rights reserved.

REV.	DRAWN	DATE	NOTE
A	SR	25.09.26	Preliminary Concept

CLIENT



DESIGNED
DRAWN
REVIEWED
APPROVED

xx
xx
xx
xx

STATUS
SCALE

PRELIMINARY
1:100 [A2]

34% NBS SEISMIC
STRENGTHENING CONCEPT

36 Weld Street, Hokitika

GRID 1 & 9 ELEVATIONS

PROJECT
503048

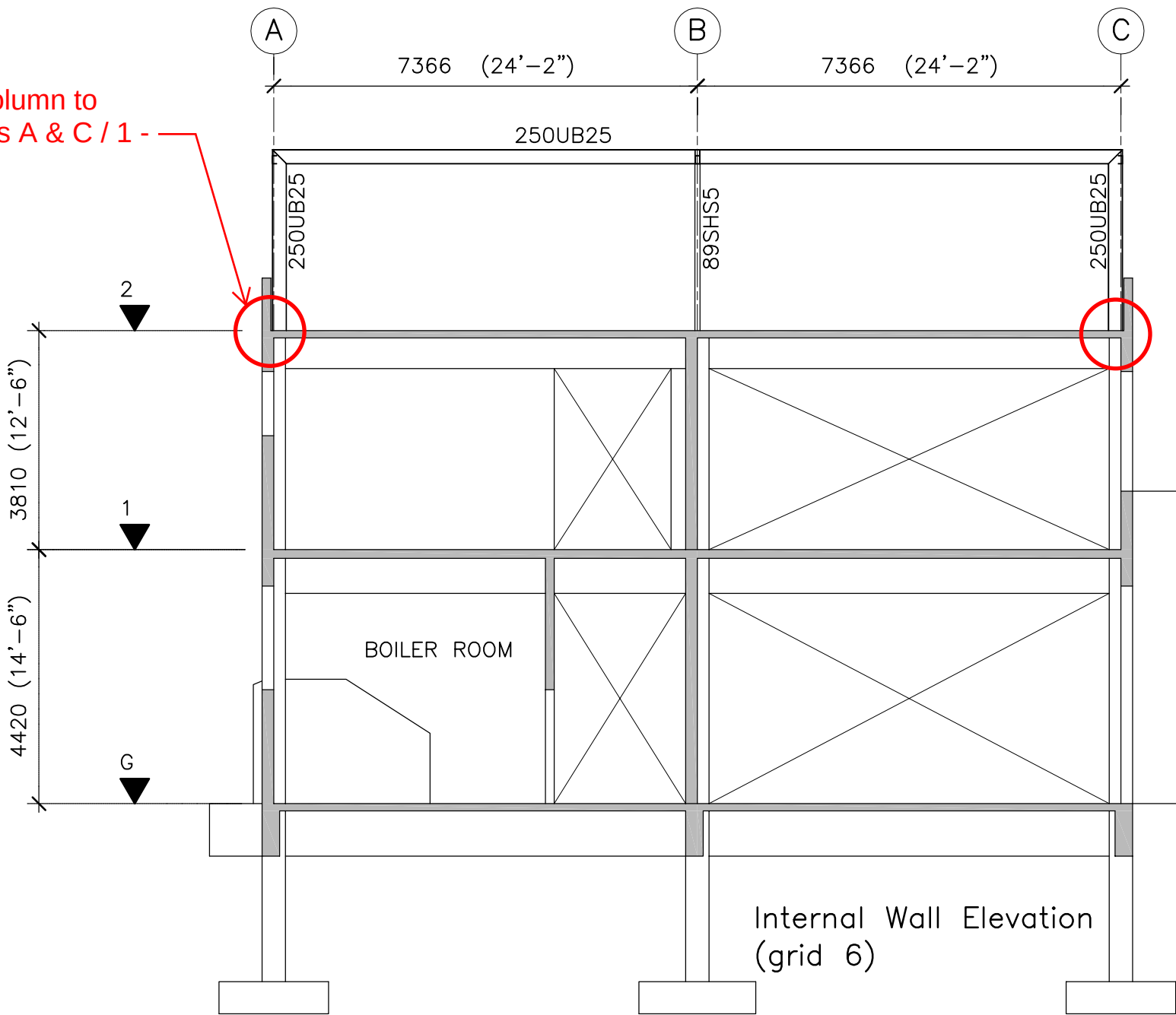
REV.
A

SET
SC

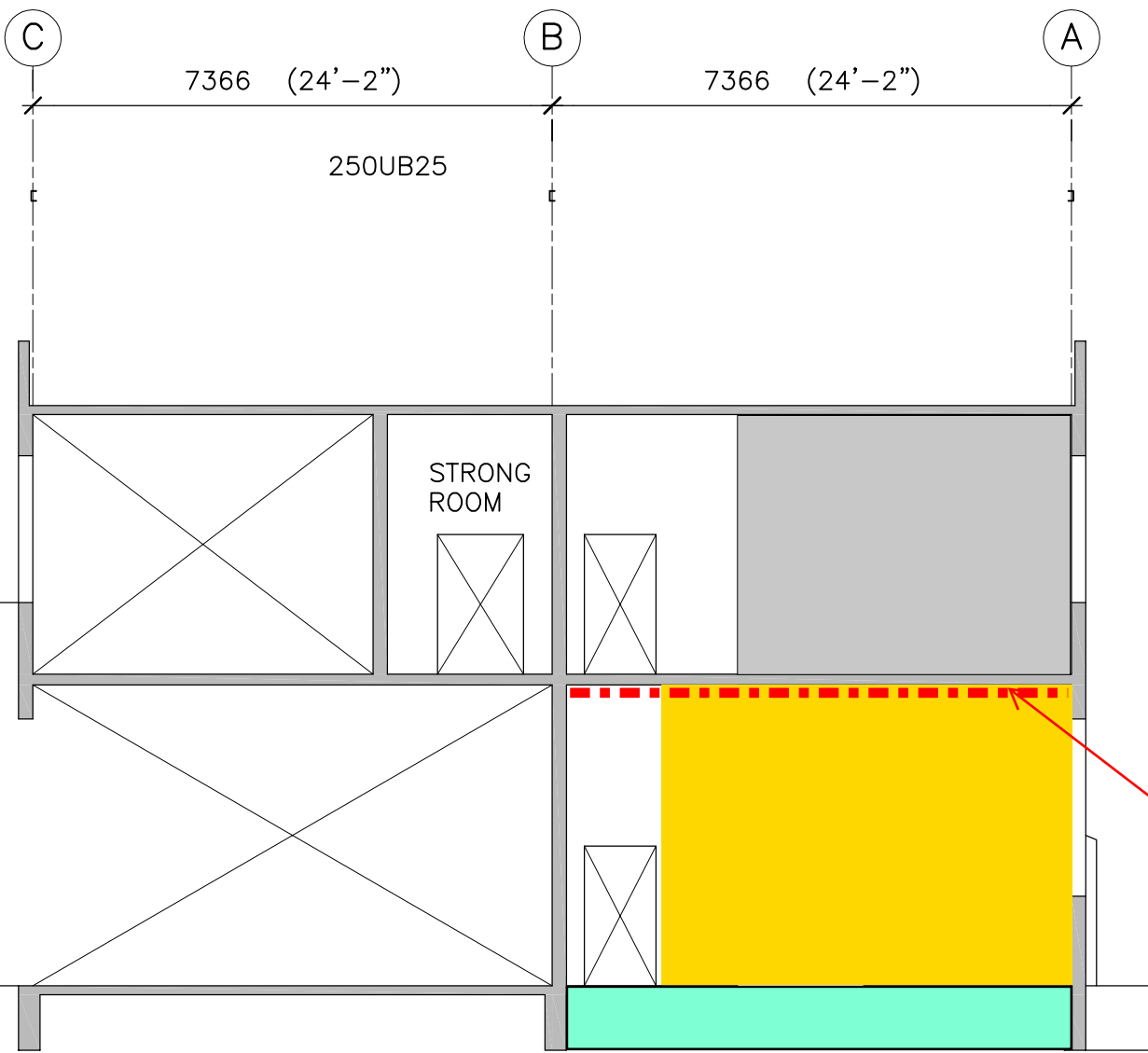
SHEET
S06



Strengthen existing column to floor connection - grids A & C / 1 - 9.



'I' SITE



150x10EA shear collector beam

Key

-  150mm thick poured insitu (or sprayed) reinforced concrete shear walls. Reinforced with: H16 @ 200crs vertical & H12 @ 200crs horizontal.
-  Poured insitu reinforced concrete foundations. Refer to sheet S10-S11 for indicative details.

NOTES

- Contractors to verify all dimensions and the location of all underground services on site prior to commencing work.
- Unless noted otherwise, all work shall be undertaken in accordance with the NZBC and any relevant Territorial Authority Engineering Standards and Specifications as a minimum standard.

DISCLAIMER

© Eliot Sinclair and Partners Ltd. This drawing and all its information is only to be used for its intended purpose. All rights reserved.

REV.	DRAWN	DATE	NOTE
A	SR	25.09.26	Preliminary Concept

CLIENT



DESIGNED
DRAWN
REVIEWED
APPROVED

xx
xx
xx
xx

STATUS
SCALE

PRELIMINARY
1:100 [A2]

34% NBS SEISMIC STRENGTHENING CONCEPT

36 Weld Street, Hokitika

GRID 6 & 7.5 ELEVATIONS

PROJECT
503048

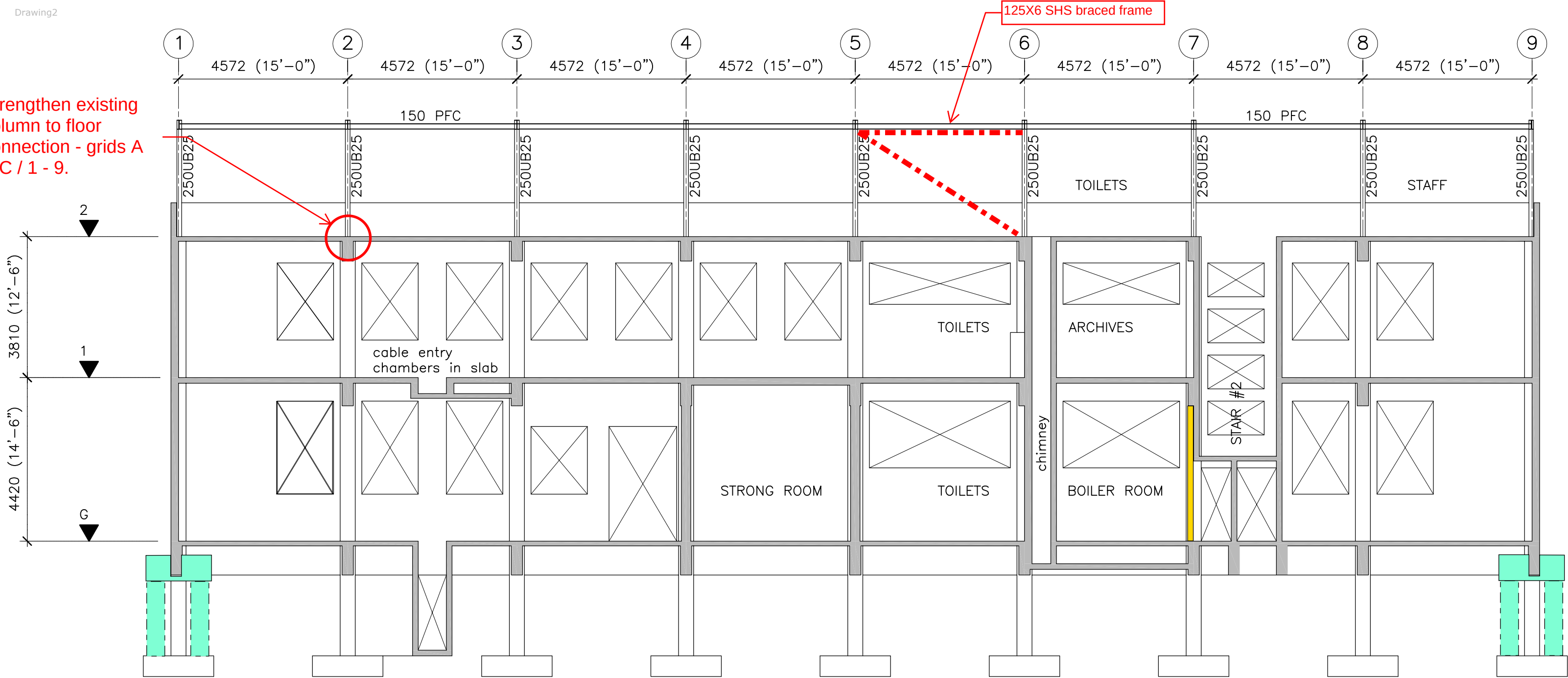
REV.
A

SET
SC

SHEET
S07



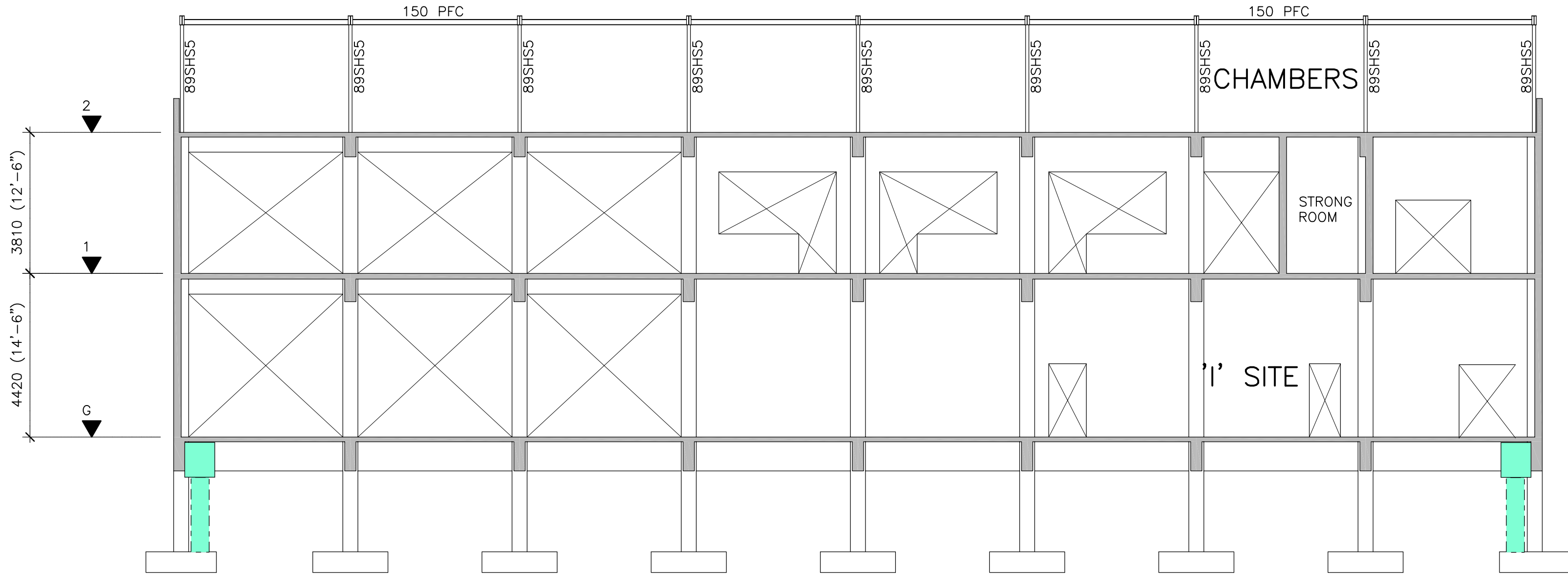
Strengthen existing column to floor connection - grids A & C / 1 - 9.



Key

- 150mm thick poured insitu (or sprayed) reinforced concrete shear walls. Reinforced with: H16 @ 200crs vertical & H12 @ 200crs horizontal.
- Poured insitu reinforced concrete foundations. Refer to sheet S02 & S10-S11 for extent & indicative details

North Wall Elevation (grid A)



Internal Wall Elevation (grid B)

NOTES

- Contractors to verify all dimensions and the location of all underground services on site prior to commencing work.
- Unless noted otherwise, all work shall be undertaken in accordance with the NZBC and any relevant Territorial Authority Engineering Standards and Specifications as a minimum standard.

DISCLAIMER

© Eliot Sinclair and Partners Ltd. This drawing and all its information is only to be used for its intended purpose. All rights reserved.

REV.	DRAWN	DATE	NOTE
A	SR	25.09.26	Preliminary Concept

CLIENT

DESIGNED	xx
DRAWN	xx
REVIEWED	xx
APPROVED	xx
STATUS	PRELIMINARY
SCALE	1:100 [A2]

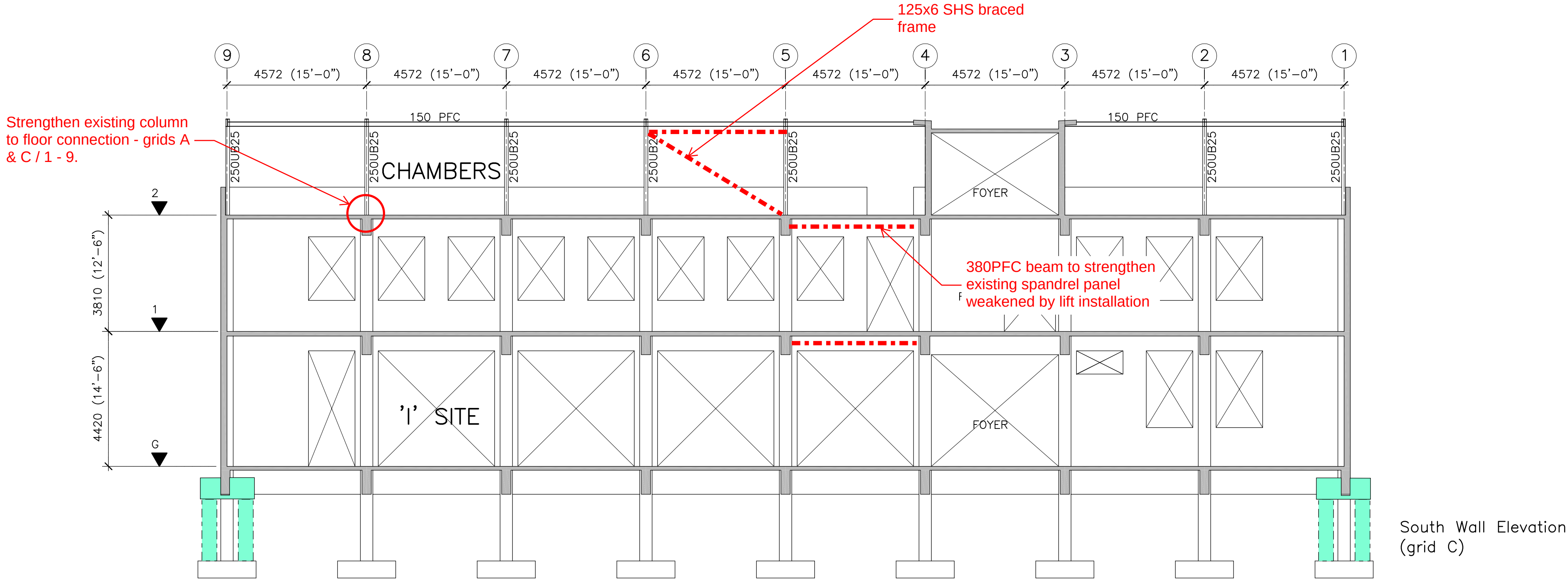
34% NBS SEISMIC STRENGTHENING CONCEPT

36 Weld Street, Hokitika

GRID A & B ELEVATIONS

PROJECT	REV.
503048	A
SET	SHEET
SC	S08





Key



Poured insitu reinforced concrete foundations. Refer to sheet S02 & S10-S11 for extent & indicative details

NOTES
1. Contractors to verify all dimensions and the location of all underground services on site prior to commencing work.
2. Unless noted otherwise, all work shall be undertaken in accordance with the NZBC and any relevant Territorial Authority Engineering Standards and Specifications as a minimum standard.
DISCLAIMER
© Eliot Sinclair and Partners Ltd. This drawing and all its information is only to be used for its intended purpose. All rights reserved.

REV.	DRAWN	DATE	NOTE
A	SR	25.09.26	Preliminary Concept

CLIENT

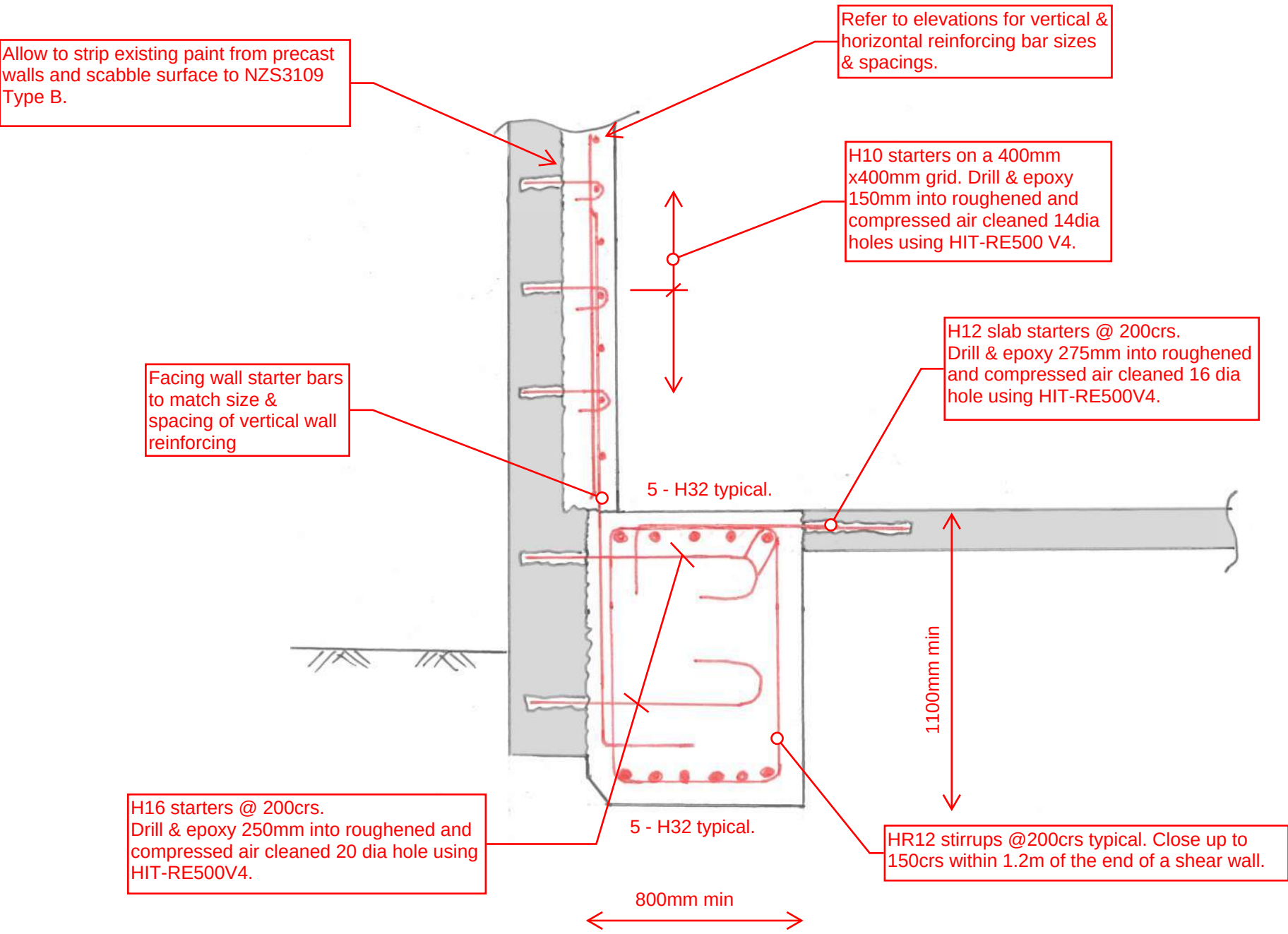


DESIGNED	XX
DRAWN	XX
REVIEWED	XX
APPROVED	XX
STATUS	PRELIMINARY
SCALE	1:100 [A2]

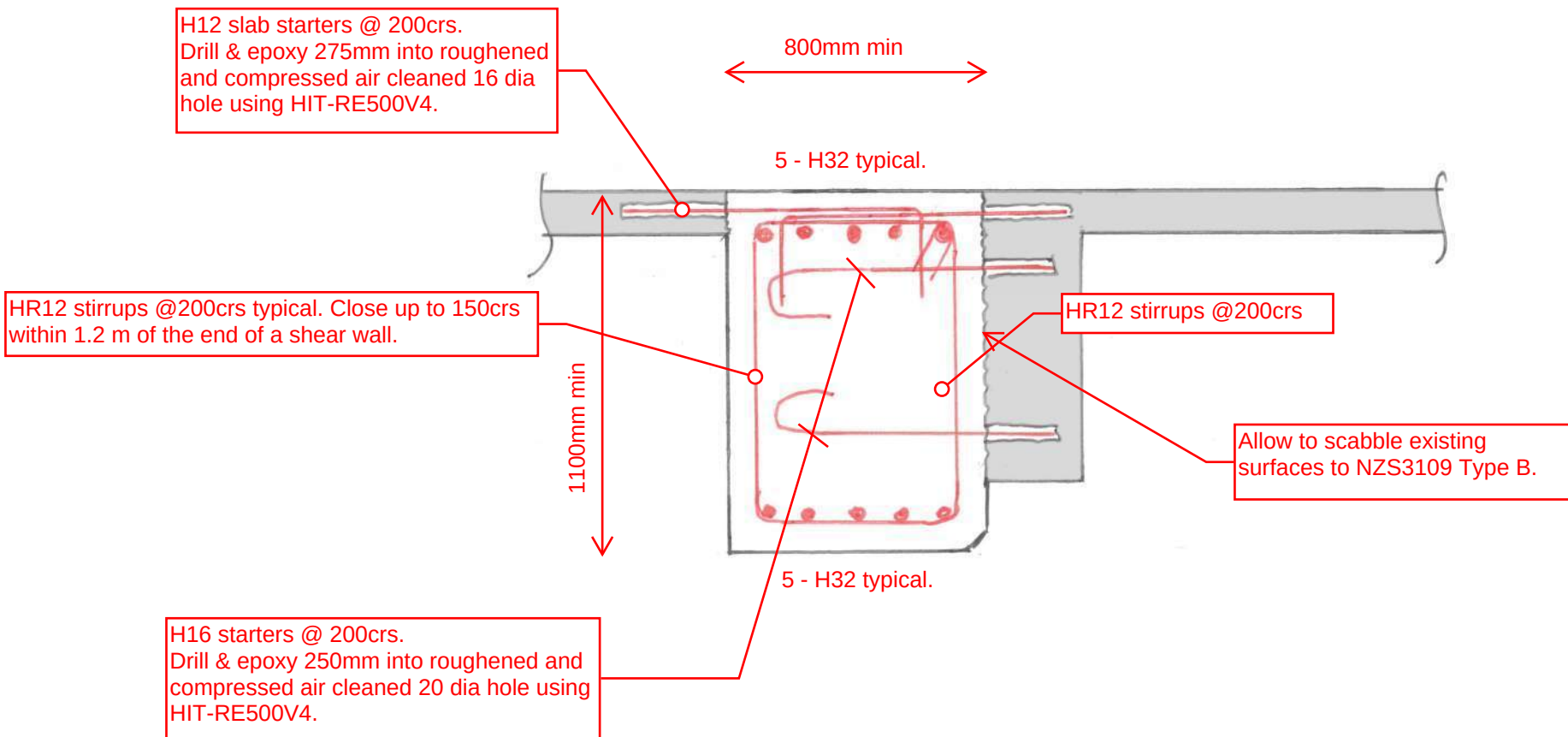
34% NBS SEISMIC STRENGTHENING CONCEPT
36 Weld Street, Hokitika
GRID C ELEVATION

PROJECT	REV.
503048	A
SET	SHEET
SC	S09

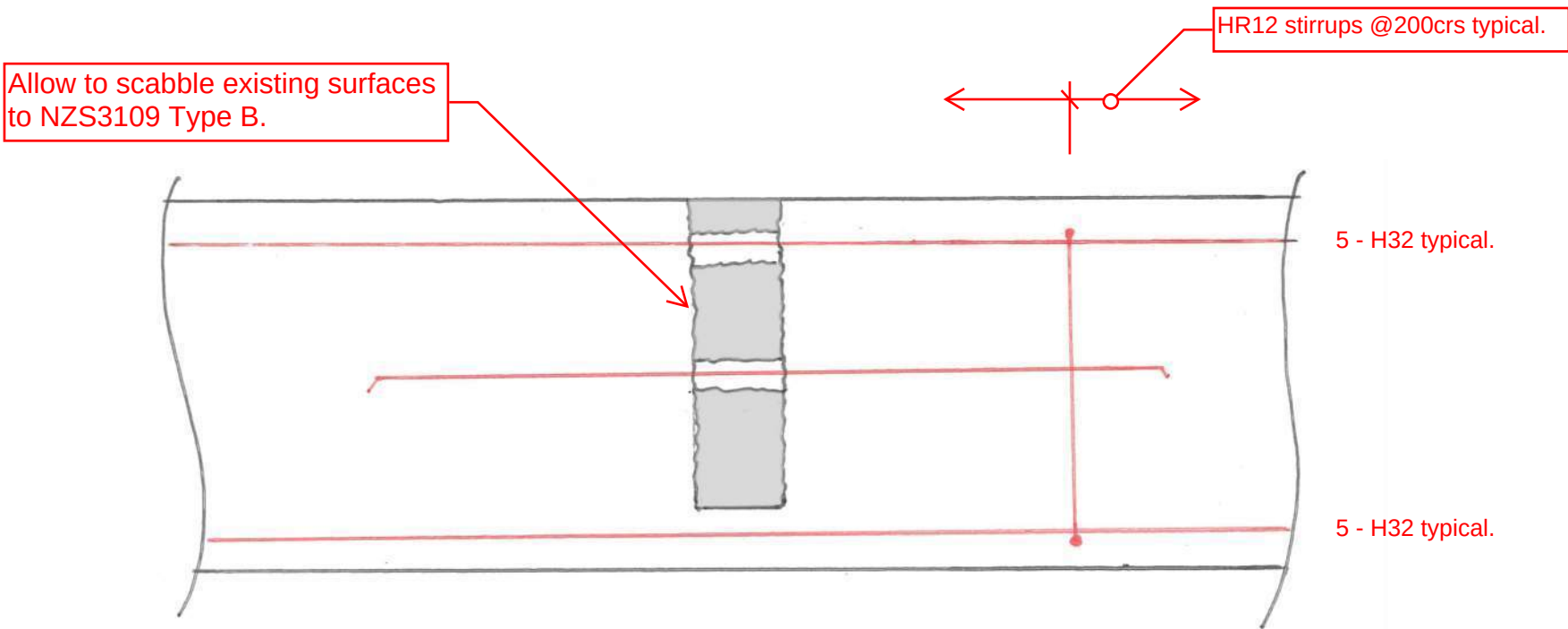




Typical New Concrete Facing Wall to New Foundation Connection



Typical New Foundation to Existing Foundation Connection - Parallel



Typical New Foundation to Existing Foundation Connection - Perpindicular

NOTES
1. Contractors to verify all dimensions and the location of all underground services on site prior to commencing work.
2. Unless noted otherwise, all work shall be undertaken in accordance with the NZBC and any relevant Territorial Authority Engineering Standards and Specifications as a minimum standard.
DISCLAIMER
© Eliot Sinclair and Partners Ltd. This drawing and all its information is only to be used for its intended purpose. All rights reserved.

REV.	DRAWN	DATE	NOTE
A	SR	25.09.26	Preliminary Concept

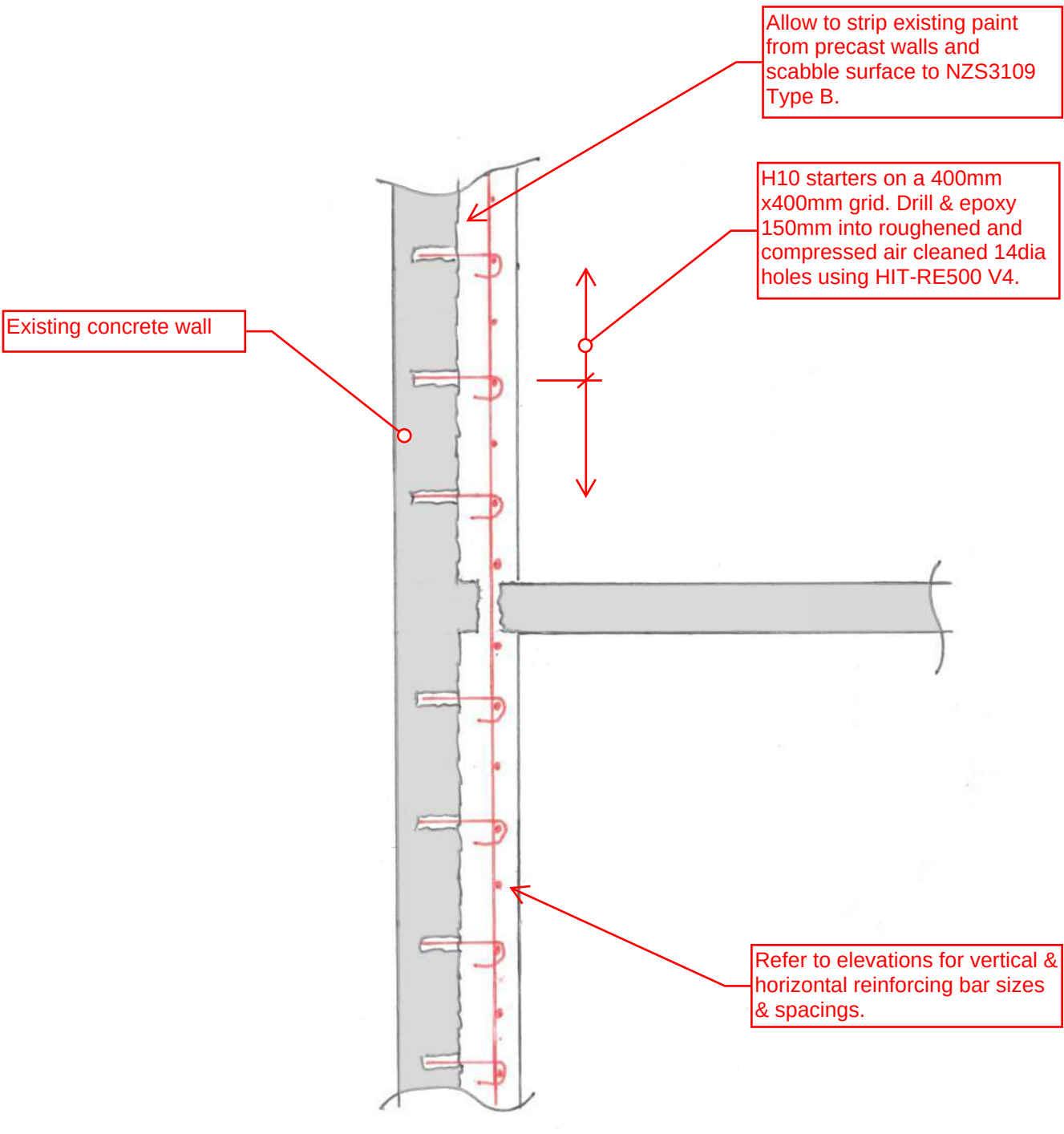


DESIGNED	XX
DRAWN	XX
REVIEWED	XX
APPROVED	XX
STATUS	PRELIMINARY
SCALE	NTS

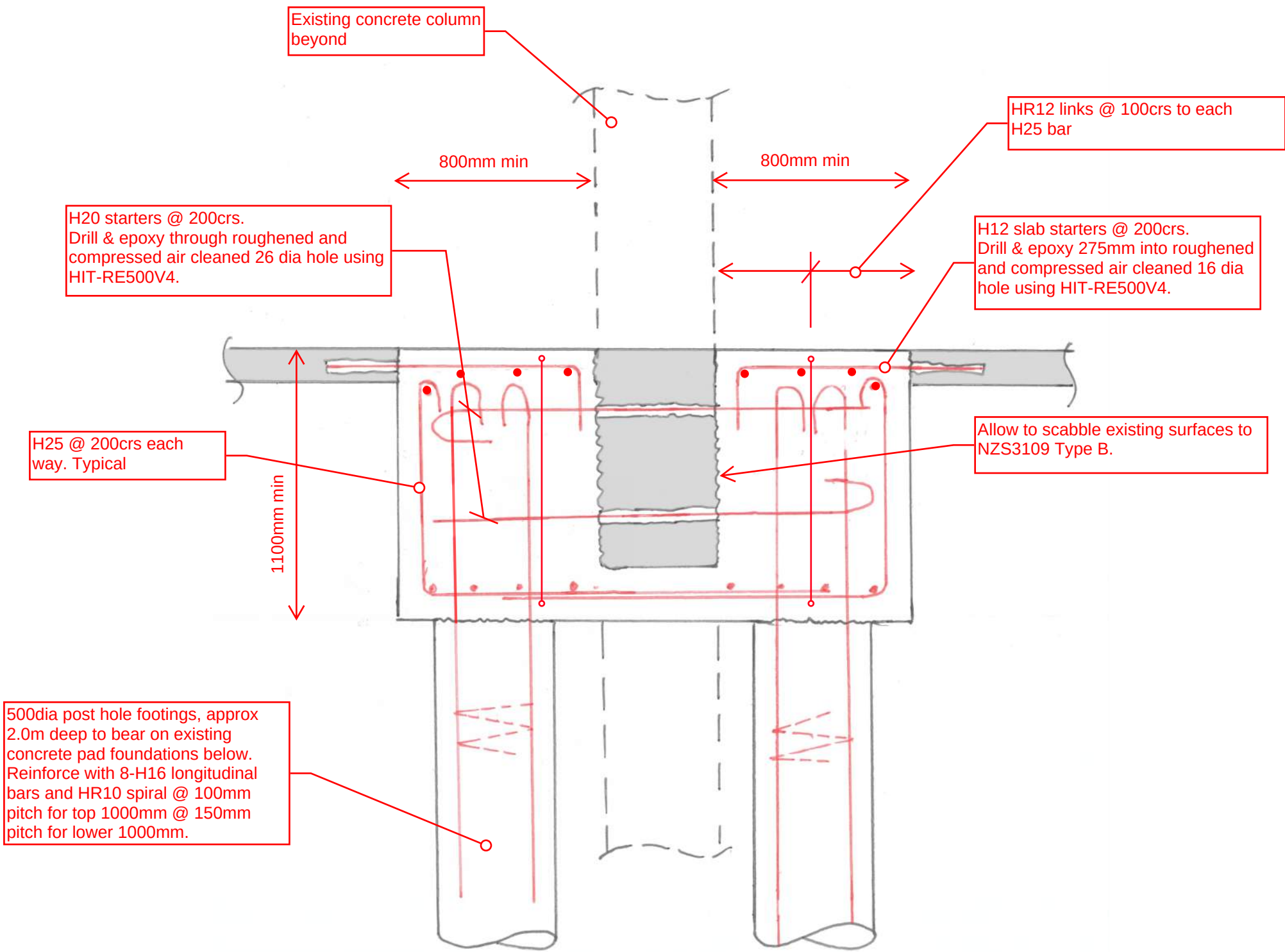
34% NBS SEISMIC STRENGTHENING CONCEPT
36 Weld Street, Hokitika
TYPICAL DETAILS

PROJECT	REV.
503048	A
SET	SHEET
SC	S10





Typical New Concrete Facing Wall to Floor Junction



Typical New Foundation Connection @ Existing Grid Intersections

NOTES
1. Contractors to verify all dimensions and the location of all underground services on site prior to commencing work.
2. Unless noted otherwise, all work shall be undertaken in accordance with the NZBC and any relevant Territorial Authority Engineering Standards and Specifications as a minimum standard.

DISCLAIMER
© Eliot Sinclair and Partners Ltd. This drawing and all its information is only to be used for its intended purpose. All rights reserved.

REV.	DRAWN	DATE	NOTE
A	SR	25.09.26	Preliminary Concept

CLIENT



DESIGNED	XX
DRAWN	XX
REVIEWED	XX
APPROVED	XX
STATUS	PRELIMINARY
SCALE	NTS

34% NBS SEISMIC STRENGTHENING CONCEPT
36 Weld Street, Hokitika
TYPICAL DETAILS

PROJECT	REV.
503048	A
SET	SHEET
SC	S11

