WCC Housing Detailed Seismic Assessments

Hanson Court Block A – Detailed Seismic Assessment

Wellington City Council

Reference: 523020

Revision: 3

2025-01-29

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Document control record

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Document control						aurecon
Report title		Hanson Court Block A – Detailed Seismic Assessment				
Document code		523020-001-REP-SS-001	**Project number**	523020		
File path		523020 - WCC Housing DSAs - Berkeley Dallard, Etona Apartments, Hanson Court - 5 Working Files\502 Engineering\02 Structures\Reports\HC - Block 1				
Client		Wellington City Council				
Client contact		Casey Zhang and Mario Venter	**Client reference**			
Rev	**Date**	**Revision details/status**	**Author**	**Reviewer**	**Verifier** (if required)	**Approver**
0	2023-03-09	Draft – For Client Review	[redacted]			
1	2023-12-19	Draft – Accounting for the peer review				
2	2024-09-03	Final				
3	2025-01-29	Final				
Current revision		**3**				

Approval			
Author signature	[redacted]	Approver signature	[redacted]
Name	[redacted]	Name	[redacted]
Title	Senior Structural Engineer	Title	Structures Practice Leader

Executive Summary

Scope and Basis of Assessment

Aurecon have been engaged by the Wellington City Council (WCC) to provide a Detailed Seismic Assessment (DSA) for the building located at the corner of Hanson and Hutchison Street in the Newtown, Wellington. The building is known as the **Hanson Court Block A Building.**

The DSA was generally completed in accordance *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017 (**Red Book**), including the *updated Section C5 – Concrete Buildings – Proposed Revision to the Engineering Assessment Guidelines*, dated November 2018 (the **Yellow Chapter**). These are collectively noted as the **Guidelines**.

The Building is considered to be an **Importance Level 2 (IL2)** structure, located on a **Site Subsoil Class B** site as defined by NZS 1170.5:2004.

Beca Ltd (Beca) was engaged by Wellington City Council to carry out an independent peer review of this Detailed Seismic Assessment. A copy of their Peer Review letter can be found in Appendix F.

Results Summary

The seismic rating of a building is generally limited by the lowest scoring element; therefore, the Building achieves an earthquake rating of **25%*NBS*(IL2)** in accordance with the **Guidelines**. This rating is based on the Critical Structural Weakness (**CSW**) of the reinforced concrete (RC) walls out-of-plane capacity at the roof level to resist seismic parts loading. Further investigations of the roof connections are required to confirm this rating. The Building also contains other distinct elements that are classified as structural weaknesses (**SW**).

A **SW** is an aspect of the building structure and/or the foundation soils that scores less than 100%*NBS* and a **CSW** is the lowest scoring structural weakness.

Although this building contains structural weaknesses, it is worth noting that this building is considered regular, has many wall elements, is structurally stiff and is well-tied together with a concrete in-situ diaphragm. Buildings that contain these characteristics typically perform "better" in large earthquake shaking when compared to other structures without these characteristics.

Beca conducted a peer review of the DSA following the issuance of the draft report. Based on their review, the %*NBS* score for RC Shear Walls in the transverse direction changed from 60% to 65%, while no other %*NBS* scores for the remaining structural elements were altered. The peer review did not affect the overall %*NBS* rating of the building.

The Table below presents a summary of the results based on the **Guidelines.**

Table: Summary of Elements - *%NBS* scores

Element:	%NBS(IL2):	Commentary:
RC Piers and Spandrel – Longitudinal Direction	50%	▪ The RC piers and spandrels have insufficient flexural capacity to resist 100% ULS loading. ▪ The plain round bars lap length limits the allowable steel stress. As a result, a single crack may form in the walls under a design level earthquake. This mechanism has an inferior energy dissipation capacity compared to modern walls.

RC Shear Walls – Transverse Direction	65%	▪ The RC shear walls have insufficient flexural capacity to resist 100% ULS loading. ▪ The plain round bars lap length limits the allowable steel stress. As a result, a single crack may form in the walls under a design level earthquake. This mechanism has an inferior energy dissipation capacity compared to modern walls. ▪ The RC foundations may also uplift and cause the building to rock.
Concrete Diaphragms:	100%	▪ The concrete diaphragm, reinforced with plain round bars, have sufficient capacity to transfer the diaphragm inertia and transfer loads to the RC lateral system.
RC Moment Frame with Block infill walls	30%	▪ The blockwalls within the RC frames, on Grids A and X, score 30%*NBS*(IL2) based on the walls out-of-plane capacity under seismic parts loading. The drawings indicate that the blockwall is unreinforced. To confirm if the blockwalls are unreinforced or reinforced we suggest further investigations is undertaken onsite. Presence of reinforcing steel in these walls may improve their %NBS score. ▪ The RC moment frames with block infill walls score 75%NBS based on the RC columns shear capacity for in-plane loading. The infill block wall causes flexural, and shear demands on the columns from the effective strut in the block walls. ▪ The remaining RC moment frames without blockwalls, have sufficient gravity carrying capacity under the expected ULS drifts.
Foundations:	100%	▪ The strip footing foundations can resist the soil bearing pressure demands and scores >100%*NBS*(IL2). ▪ The building is expected to slide at 40%ULS loading. However, the building sliding is not considered a life safety risk and therefore the score does not govern the building/foundation score.
Stairs	75%	▪ The stairs contain connections to the landings that are fixed with no allowance for sliding or seismic movement. As a result, the stairs may act as an unintentional strut in a design level earthquake. However, as the stairs are located next to a RC shear wall, the walls “protect” the stairs from attracting significant in-plane seismic loading and score 80%*NBS*(IL2) for in-plane loading. ▪ The stairs score 100% NBS for out-of-plane seismic parts loading.
Walls Out-of-Plane	25%	▪ The RC walls above Level 3 are cantilevering to support the roof system. This cantilever is as high as 4.7m in some locations. It is not expected that the roof system can provide restraint of the walls for out-of-plane loading. The walls score 25%*NBS*(IL2) for out-of-plane seismic parts loading.
Roof	70%	▪ The timber and aluminium roof is a flexible diaphragm and does not contain steel-cross braces or a plywood diaphragm. Therefore, the timber rafters must transfer seismic load from the roof to the RC walls by bending out-of-plane. ▪ The timber rafters score 70%*NBS*(IL2) for bending about the minor axis. ▪ The connections of the roof to the walls score at 100%*NBS*(IL2). However, information of the connections is incomplete and needs further investigation.
Canopies	100%	▪ The canopies have sufficient capacity to resist 100% ULS parts loading.

We note that the non-structural building elements (ceilings, lightweight partition walls, overhead services and plant and equipment etc) have not been explicitly considered in the seismic rating of the building. However, a desktop study of the available documentation did not identify any large plant, ceilings, and partitions that would raise concern.

Further Investigations

We recommend that further investigation be carried out to the following elements to provide a more accurate seismic score:

- Investigate the **connections of the timber roof elements to the RC shear walls**. The assessment to date has based the score on an assumed connection detail. Further clarity of the connection arrangement is recommended to provide a more accurate %*NBS* score.
- Investigate the **block walls to determine whether they contain reinforcement**. Presence of reinforcing steel in these walls may improve their *%NBS* score.

Recommendations

We recommend that the building is seismically strengthened considering a two-stage approach. Stage 1 would be to strengthen the building to a minimum seismic rating of greater than **34*%NBS*(IL2).** Based on our review, the seismic strengthening, to achieve greater than 34*%NBS*(IL2), would include, but not be limited to:

- Increase the **RC wall out-of-plane** capacity by installing a new roof diaphragm with new connections to the concrete walls. The roof diaphragm can be in the form of steel cross braces and steel beams.
- Increase the **blockwalls out-of-plane** capacity by installing steel strong-backs to the blockwalls and RC beams. Also introduce a seismic gap between the blockwalls and RC columns by saw cutting a gap.

Stage 2 would be to seismically strengthen the building to a minimum rating of 67*%NBS* (IL2). Based on our review, the seismic strengthening to achieve 67%*NBS*(IL2) would include, but not be limited to:

- Increase the **RC walls lateral capacity** by installing new RC overlay walls, reinforced and continuous doweled into the existing RC walls. New foundations will also be required.

We also recommend that part of any seismic upgrade or future fitout that the non-structural building elements (ceilings, internal walls, overhead services and plant and equipment etc) is seismically restrained to meet the current standards. It should be noted that no large plant was identified in the building that would need seismic support. No ceilings, partitions and façade were identified while studying the existing documentation that would raise concern.

We further recommend that in designing any seismic retrofit that the building owner should also consider the proposed increase in seismic hazard levels in Wellington. This would insulate the building against further future reductions in the seismic rating.

Contents

1 Introduction

1.1 Background

Aurecon have been engaged by the Wellington City Council (WCC) to provide a Detailed Seismic Assessment (DSA) for five apartment buildings in the Hanson Court complex on Hanson St. The buildings that have been assessed are buildings A, B, C, D & E. Refer to **Figure 1-1** for the site's location and layout.

This DSA report is for the **Hanson Court Block A Building**. **Figure 1-2** shows an elevation of the building.

The DSA focuses on life safety issues as the primary objective. This means that the earthquake scores or rating is based primarily on life safety considerations rather than damage to the building or its contents unless this might lead to damage to adjacent property. The earthquake rating assigned is, therefore, not reflective of serviceability performance.

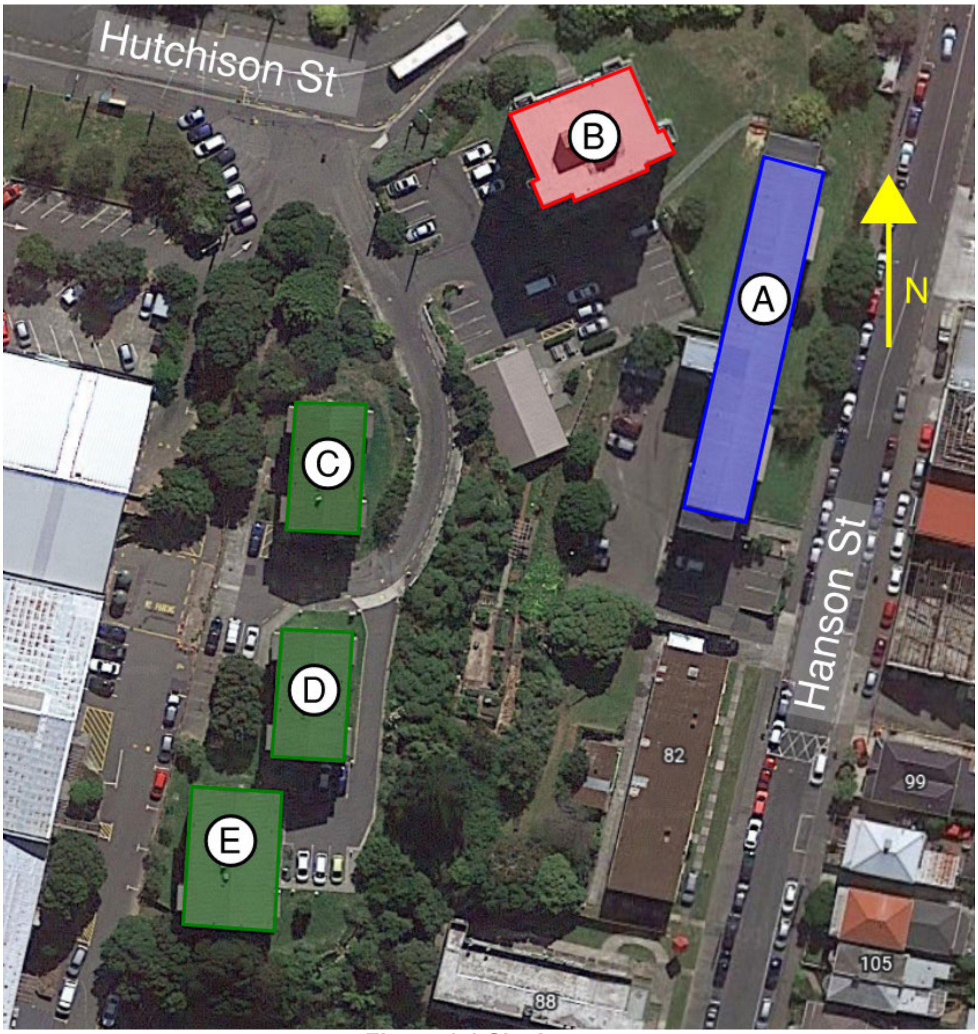


Figure 1-1 Site Layout

Figure 1-2 Building Site Photograph

1.2 Terminology and Key Definitions

See below for key terminology and key definitions as defined by the **Guidelines**. Refer to **Appendix A** for additional definitions.

- ***%NBS* (New Building Standard)*:*** The ratio of the ultimate capacity of a building as a whole or of an individual member/element and the ULS shaking demand for a similar new building on the same site, expressed as a percentage. Intended to reflect the expected seismic performance of a building relative to the minimum life safety standard required for a similar new building on the same site by Clause B1 of the New Zealand Building Code.
- **Design level/ULS earthquake:** Design level earthquake or loading is taken to be the seismic load level corresponding to the ULS seismic load for the building at the site as defined by NZS 1170.5:2004
- **Ductile/ductility**: Describes the ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake
- **Structural weakness (SW):** An aspect of the building structure and/or the foundation soils that scores less than 100*%NBS*.
- **Critical structural weakness (CSW):** The lowest scoring structural weakness determined from a DSA.

1.3 Building Description

Block A is an apartment building located at Hanson Court, located on the corner of Hutchison Road and Hanson Steet, Newtown, Wellington.

The building is a four-storey apartment block located towards the Eastern edge of the site. It was constructed in 1963 and is a reinforced concrete (RC) shear wall building. The building is rectangular in plan (60m x 8m) and 10.6m tall. Refer to Figure 1-3 for a typical building cross section and Figure 1-4 for a typical floor plan.

Each floor houses several individual apartments with an access gallery on the western side running north-south. An entrance lobby with stairs is located as an attached structure on the western elevation approximately 5.1m x 6.3m in plan. Stairs are also provided at the North and South corners on the building. The southern end on the building has a terraced lower ground floor housing two additional apartments.

The building has shear walls in the transverse direction (East-West walls) generally located between apartments, and on the external faces of the building in the longitudinal direction (North-South walls). All perimeter shear walls are 8” (200mm) thick 2 layers of reinforcement. The perimeter walls have numerous large openings for windows and doors. The openings have been trimmed with large diameter reinforcing bars. The internal walls, as well as the walls surrounding the stair cores are 6” (150mm) thick with a single layer of reinforcement.

The floors are 5” (125mm) thick reinforced concrete flat slabs spanning between the shear walls. The floors step down by 6” (150mm) where balconies and the access gallery are located.

The transverse shear walls extend up to meet the pitched roof which is formed with timber rafters spanning between the perimeter longitudinal walls and supporting lightweight aluminium roofing. Similarly, the Level 3 gib ceiling is supported by timber joists spanning between the Transverse walls. The roof joists are generally 4” x 2” (1270mm) at 2’ (610mm) spacing.

The balconies at level 3 are covered at roof height by steel canopies connected to the perimeter shear wall.

The structure is founded on a mixture of strip footings and pad foundations. The strip footings are generally 2’ x 4” (710mm) wide and 12” (305mm) thick. Pad foundations vary in size.

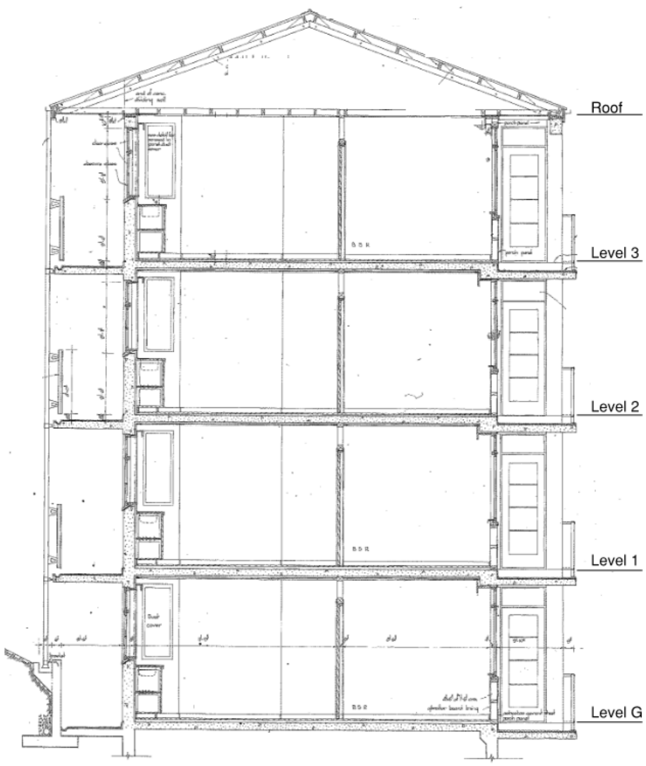


Figure 1-3 Building Cross Section

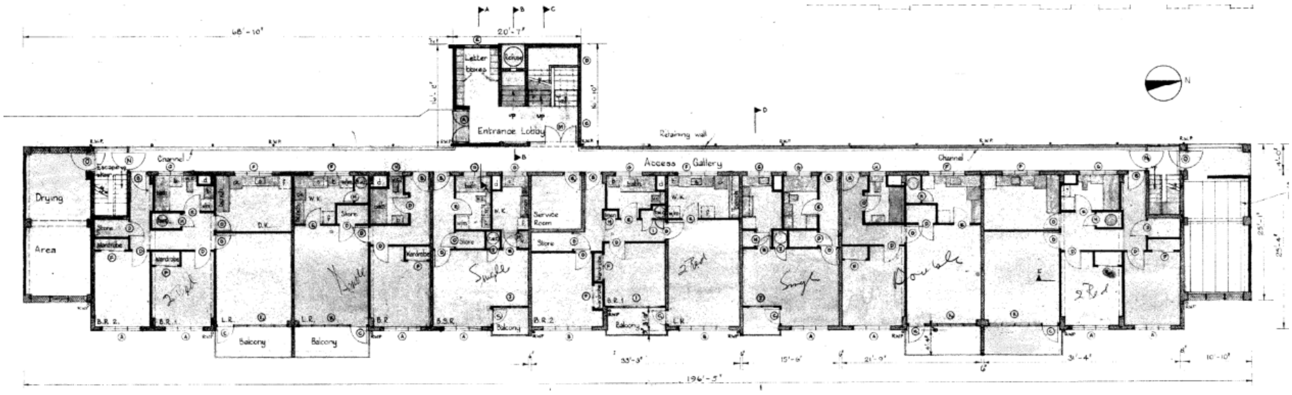


Figure 1-4 Typical Floor Plan

1.4 Previous Assessments

In 2009, Aurecon issued a report titled "*Hanson Court Podium and Tower Blocks Seismic Assessment Report*." The report indicated that the building achieved a seismic rating of **70%*NBS*(IL2)** in accordance the then current 2006 *NZSEE Assessment Guidelines.* The 70%*NBS* rating was the based on the capacity of the reinforced shear walls to resist seismic loading. All other elements scored 100%*NBS*(IL2).

Due to the date of the assessment, the assessment was not completed in accordance with *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017 (commonly known as the "**Red Book**").

Today the Red Book provides mandatory technical guidelines for engineers to use when carrying out seismic assessments of potential earthquake-prone buildings when required by the Territorial Authority. They should also be used by engineers for all seismic assessments.

In 2018, a proposed technical revision to *Section C5 of the Engineering Assessment Guidelines* (referred to as the "**Yellow Chapter**") was released by the engineering sector to provide the latest engineering knowledge on aspects involved in the assessment of concrete buildings, and to reflect what engineers learned from investigations following the Kaikōura earthquake.

1.5 Alterations and Maintenance

The building was subject to an upgrade in 2009 as part of the wider WCC Housing Upgrade project. Aurecon provided design input into the new entrance canopies, as documented by Architecture+, including Block A.

While no seismic strengthening was undertaken during the course of the alterations, substantial durability damage to the buildings was noted during the upgrade project. This damage related to corrosion of reinforcing and resulting loss of concrete cover. Works were undertaken to rectify these issues during the building upgrades.

1.6 Basis of Assessment

1.6.1 General

The DSA was generally completed in accordance *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017 (**Red Book**), including the *updated Section C5 – Concrete Buildings – Proposed Revision to the Engineering Assessment Guidelines*, dated November 2018 (the **Yellow Chapter**). These are collectively noted as the **Guidelines**.

1.6.2 Importance Level

The building has been assessed as an **Importance Level 2** (IL2) building and a design life of 50 years, in accordance with the New Zealand Building Code. A return period factor 'R' of 1.0 has therefore been used in accordance with NZS1170.5.

1.6.3 Site and subsoil class

Based on our review of the published geology and historic ground investigations, we are using the NZS 1170.5:2004 **site subsoil classification of B** for this site.

Geotechnical hazards such as liquefaction, landslide and lateral spreading are outside the scope of this assessment.

1.6.4 Hazard Zone Factor

The hazard zone factor Z determines the "seismic risk" area in accordance with NZS1170.5. There are different hazard zones factors depending on the buildings located in New Zealand. From NZS1170.5, we have used a hazard factor of **Z=0.40** for Wellington.

1.6.5 Scope

The assessment included undertaking the following:

- Retrieval and review of structural drawings, reports, calculations, and earlier models
- Conduct a walk through the building to establish that the building is generally in accordance with the plans (No intrusive investigations is allowed for)
- Create a detailed 3D ETABS model for the structure in accordance with the guidelines, based on the existing and strengthening structural drawings
- Non-Linear Analyses of the superstructure with consideration of site subsoil class and flexibility of shear walls and the foundations.
- Checking the walls, based on the analysis results and the detailing shown in the drawings.
- Assessment for the flat slab cast-in-situ diaphragms
- Assessment of the foundation including the strip footings and soil retaining structure in accordance with the updated geotechnical report
- Review of the secondary elements including stairs, and steel roof.
- Formal in-house verification by CPEng engineer
- Produce and issue a report
- Liaison and meetings as requested

Elements that are excluded from consideration and analysis in this DSA include, but are not limited to:

- Non-structural building elements (façade glass, ceilings, internal lightweight walls, overhead services and plant and equipment), although please note our observations with regards to these.

2 Assessed Seismic Risk

The results of the DSA assess the Building's earthquake rating to be **25%*NBS*(IL2)** in accordance with the **Guidelines**. This rating is based on the Critical Structural Weakness (**CSW**) of the reinforced concrete (RC) walls out-of-plane capacity at the roof level to resist seismic parts loading. The Building also contains other distinct elements that are classified as structural weaknesses (elements that score less than 100%*NBS*).

Therefore, this is a **Grade D** building following the NZSEE grading scheme. This may classify the building as earthquake prone to the New Zealand Building Act, subject to the Territorial Authority. A Grade D building imposes a risk more 10 to 25 times greater than a new building. Refer to **Table 2-1** that shows the relative seismic risk compared to a new building.

Details of the %NBS(IL2) scores are provided in **Table 6-1.**

Table 2-1 Relative seismic risk

Seismic Grade	*%NBS*(IL2)	Approx. risk relative to a similar new building	Relative life-safety risk description
A+	>100	<1	low risk
A	80 to 100	1 to 2 times	low risk
B	67 to 80	2 to 5 times	low to medium risk
C	33 to 67	5 to 10 times	medium risk
D	20 to 33	10 to 25 times	high risk
E	<20	more than 25 times	very high risk

A building with an earthquake rating less than 34%*NBS*, with the assessment undertaken utilising the Red Book, fulfils one of the requirements for the Territorial Authority to consider it to be an Earthquake-Prone Building (EPB) in terms of the Building Act 2004. A building rating less than 67%*NBS* is considered as an Earthquake Risk Building (ERB). The Building is therefore categorised as an Earthquake-Risk Building and meets one of the criteria that could categorise it as an Earthquake Prone Building by Wellington City Council as the Territorial Authority. We note that our assessment used the **Yellow Chapter**. An assessment using the Red Book would likely result in similar scores to the **Yellow Chapter**.

3 Structural System Description

3.1 Primary Lateral Load Resisting System

3.1.1 Vertical Lateral Resisting Elements

Longitudinal Direction

The lateral system in the longitudinal direction consists of in-situ 8" (200mm) thick RC piers and spandrels. These elements are reinforced with two layers of plain round 3/8" (9.5mm) diameter bars at 12" (305mm) spacing each way. The walls do not have end thickenings, but larger reinforcement trimmer bars are provided around wall openings. These are typically two 3/4" (20mm) diameter bars. Refer to

Figure 3-1 for a plan view showing the lateral load resisting elements in the longitudinal direction.

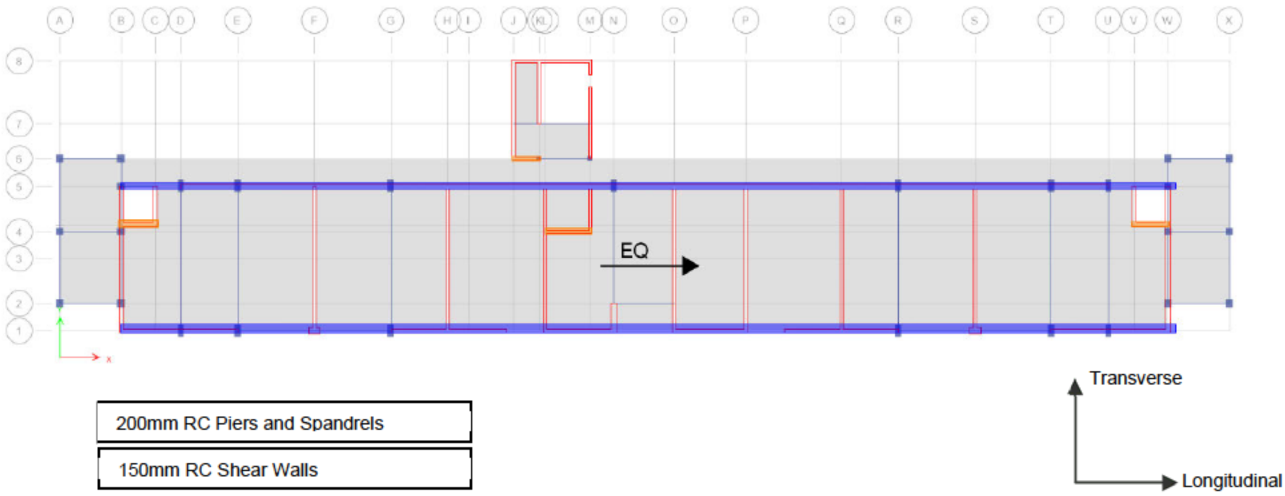


Figure 3-1 Lateral Load Resisting Elements in the Longitudinal Direction

Transverse Direction

The lateral system in the transverse direction consists of in-situ 8" (200mm) thick and 6" (150mm) RC shear walls. The walls are generally located between tenancies as well as around the stair core. The 200mm thick shear walls are located at the ends of the building and are reinforced with two layers of plain round 3/8" (9.5mm) diameter bars at 12" (305mm) spacing each way. The 150mm walls are located internally and are singly reinforced with 3/8" (9.5mm) diameter bars at 9" (230mm) spacing each way. The trimmer bars around wall openings are similar to the perimeter walls.

There are also RC moment frames in the transverse direction. Due to the flexible behaviour of the frames when compared to the RC shears, the frames due not resist significant lateral forces. We note that the frames along Grid A and X, contain block infill walls. These walls due act some lateral load.

The shear walls cantilever up from the 3rd floor to support the timber and aluminium roof structure. Refer to

Figure 3-2 for a plan view showing the lateral load resisting elements in the transverse direction.

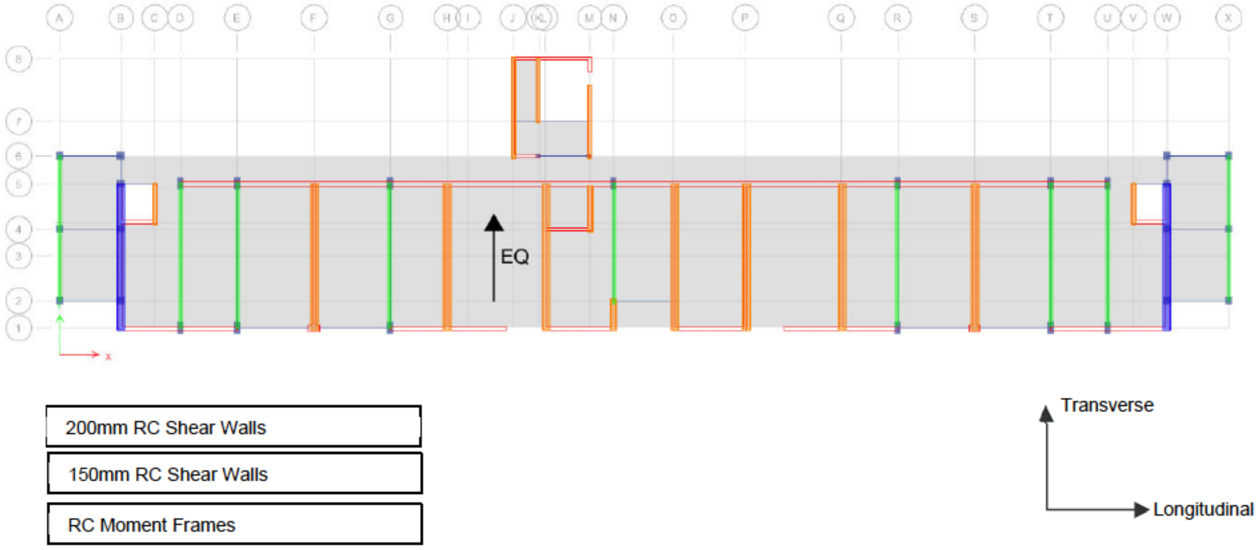


Figure 3-2 Lateral Load Resisting Elements in the Transverse Direction

3.1.2 Horizontal Lateral Resisting Elements

The horizontal lateral load resisting system consists of:

- The typical floor system of the building consists of a 150mm thick reinforced concrete flat slab spanning in both the longitudinal and transverse direction. The slabs are reinforced with plain round bars. The top reinforcement in the slabs is not continuous over the full span of the slab. Top bars are only located in hogging moment regions. Saddle bars and starter bars connect the floor diaphragm to the shear walls.
- The horizontal load is transferred from the floor slab, into the RC Shear Walls, by plain round starter reinforcement bars along the RC Shear Walls.

3.2 Gravity System

The typical floor system consists of a 125mm thick 2-way spanning RC flat slab. The slab is doubly reinforced at the walls and singly reinforced at all slab midspans. The slab is supported by the RC shear walls and RC moment frames. Gravity load is then transferred from the walls to the foundations. Refer to **Figure 3-3** for a section of typical wall to slab interface.

The timber joists support the timber and aluminium roof. The joists span to the RC shear walls.

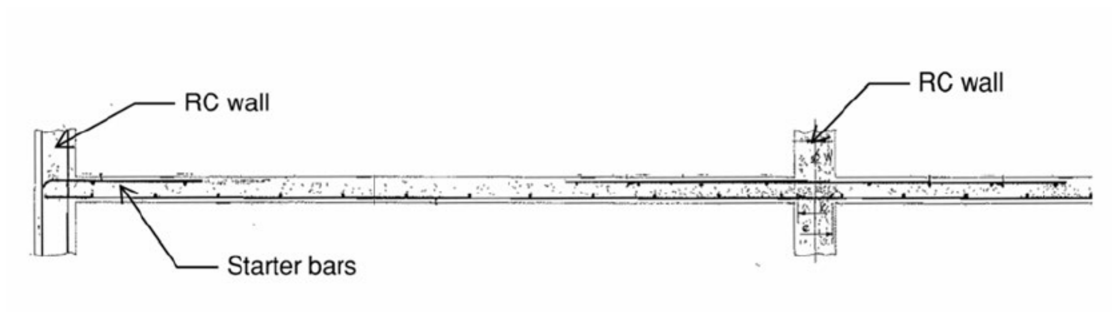


Figure 3-3 Section of typical wall to slab interface

3.3 Foundations

The building foundations consist of a combination of strip foundations and pad foundations. Concrete strip footings are located under all shear walls, these are typically 2'4" x 12" deep (710 x 300mm) strip footings on compacted hardfill. The foundations are reinforced with a single layer of bottom reinforcement, no top steel or steel stirrups have been placed in the strip footings.

Pad foundations are generally located under wall thickenings and columns. The pad foundations vary in size and are generally 18" deep (455mm). Similar to the strip footings, the pad foundations have a single layer of bottom reinforcement, and no stirrups or top reinforcement have been provided.

Refer to **Figure 3-4** for below the typical strip footing and pile layout and **Figure 3-5** for a typical strip footing section.

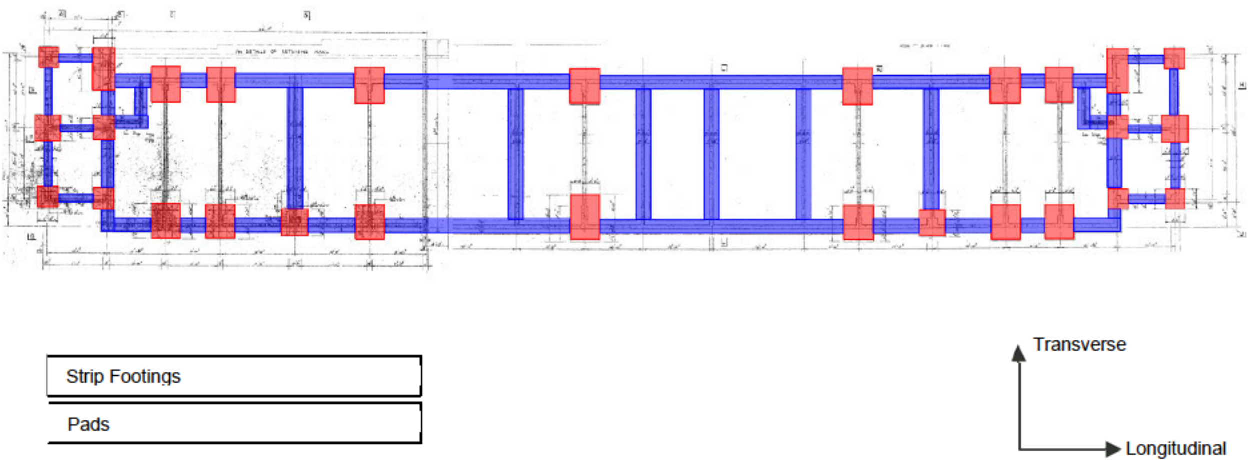


Figure 3-4 Plan view of foundations

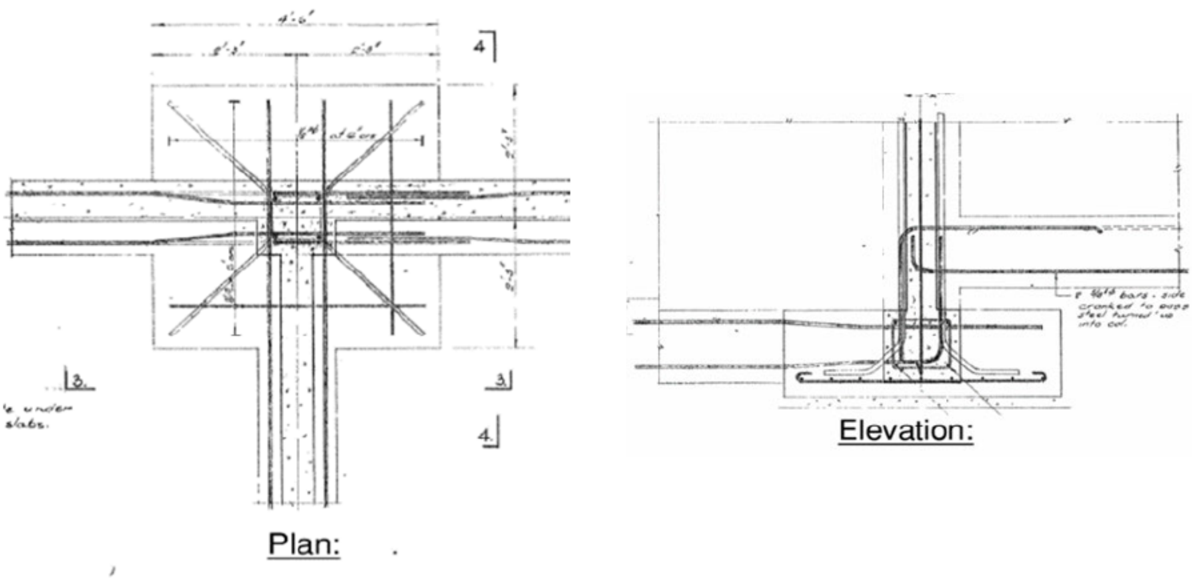


Figure 3-5 Pad Foundation Details

3.4 Subsoil

A geotechnical desktop study was performed as part of the assessment, refer to **Appendix G** for the report. The geology of the region is greywacke bedrock which underlies the site with a layer of colluvium and some fill material overlaying the greywacke. A number of active and inactive faults lie near the site, the most important of which is the active Wellington Fault, which lies approximately 2.7km northwest of the site. The site subsoil has been considered as **Subsoil Class B.**

The geotechnical investigation test pits suggest that the shallow foundations are likely to lie in moderately dense to dense gravels. The foundations are 0.95m to 1.45m below ground floor level.

3.5 Stairs

There are three stair cores located within the building. The main entrance lobby is located centrally on the western face of the building. Two additional stairs are located at either the north or south end of the building. Refer to **Figure 3-6** that shows the locations of the stairs.

All the stairs are in-situ concrete stairs with a 5" thick throat. The connections of the stairs to the floors are fixed with no allowance for sliding or lateral movement of the building. **Figure 3-7** that shows a typical elevation of the stairs.

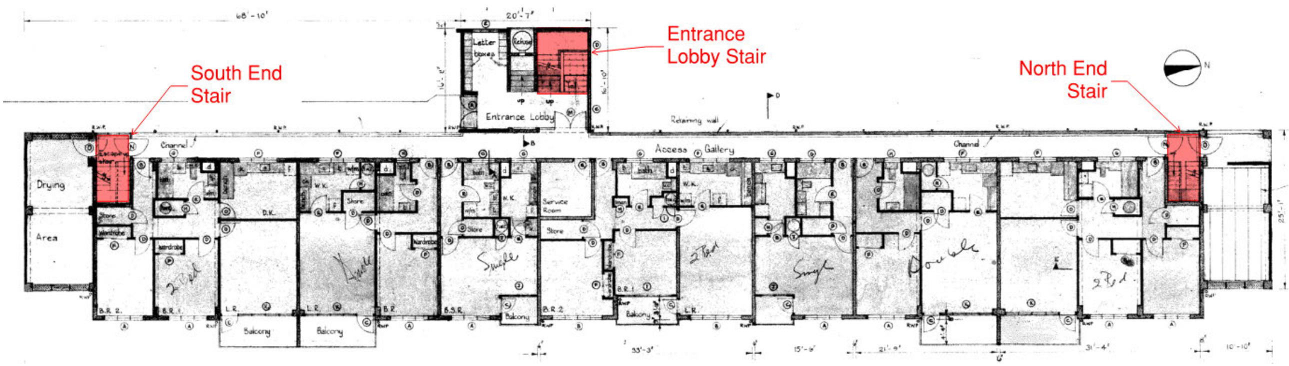


Figure 3-6 Stair Locations

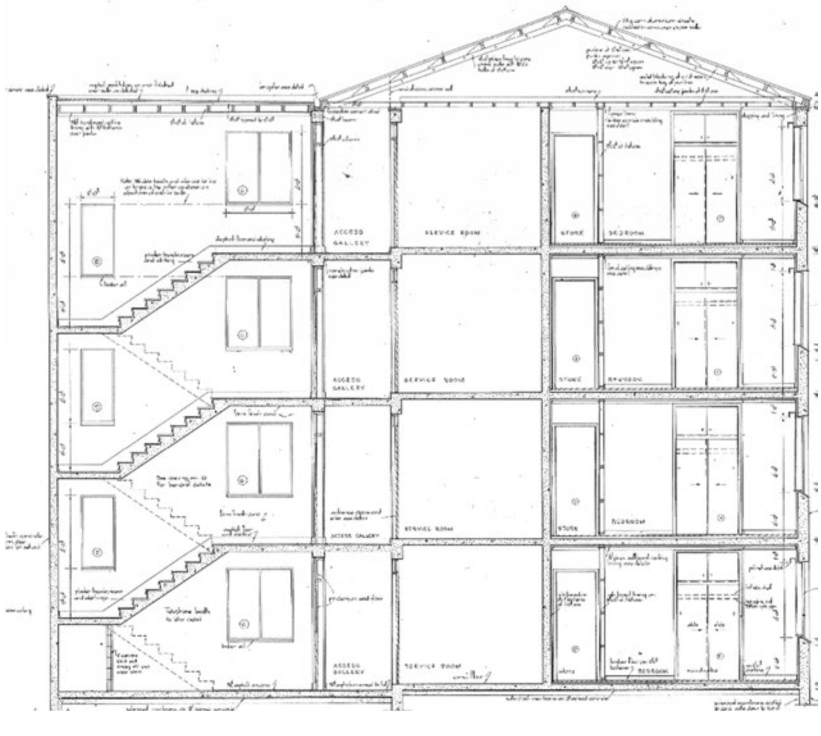


Figure 3-7 Entrance Stair Elevation

3.6 Roof

The building roof consists of timber joints spanning between concrete shear walls. The joists support timber purlins supporting aluminium roof sheeting. The joists are typically 4" x 2" timber beams spaced 2' (~610mm) apart. The joists are connected to the shear walls with ½" bolts at 2' (~610mm) spacing. Bolt embedment into the shear walls is not known.

The roof has no clearly defined diaphragm or bracing and therefore it has been assumed that the lateral loads distribute to the shear walls based on tributary area. Refer to **Figure 3-8** for a typical roof cross section.

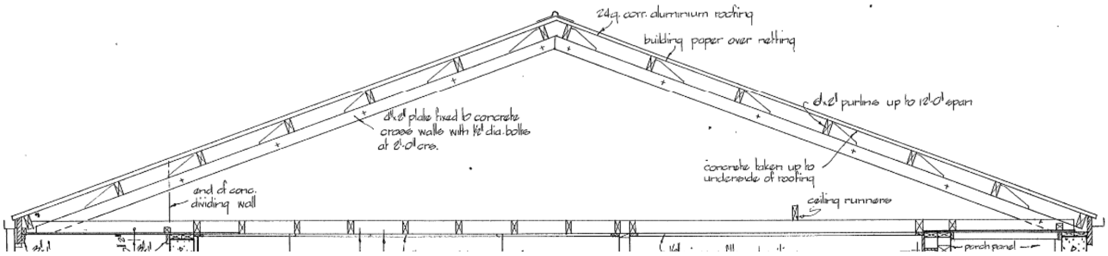


Figure 3-8 Roof Structure

3.7 Canopies

The building has steel canopies at roof level covering the balconies. The canopies are fixed to the building perimeter concrete wall with ½" bolts at 2' spacing. The bolts were cast into the concrete. Refer to Figure 3-9 for the balcony canopy plan.

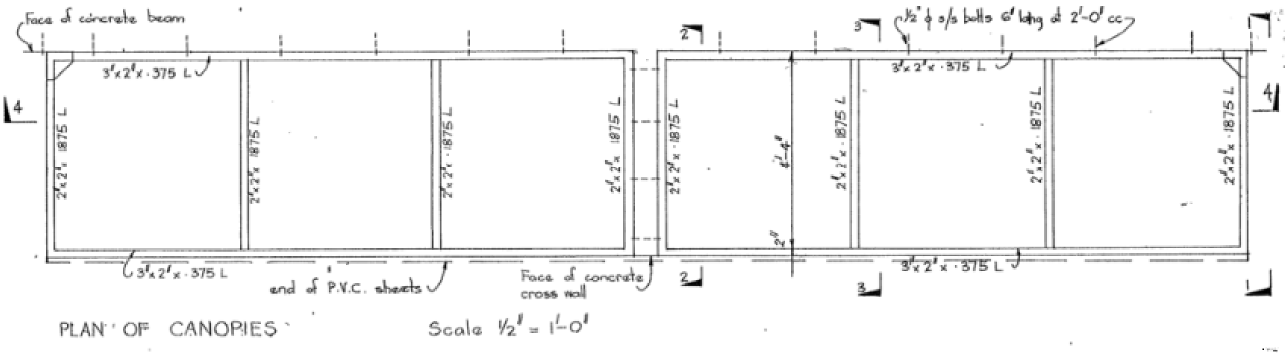


Figure 3-9 Balcony canopy plan

An additional canopy was added during 2009 alterations at the south-eastern entrance. This canopy is a timber roof structure with a steel portal frame providing gravity and lateral support. Refer to **Figure 3-10** for the South Entrance Canopy Plan.

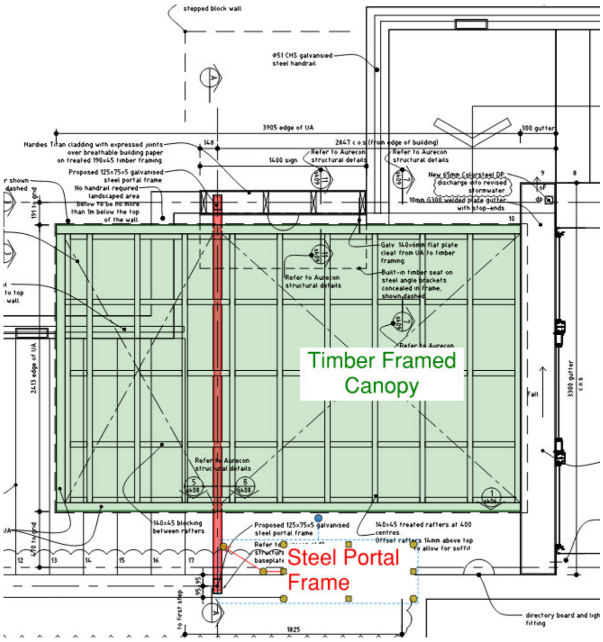


Figure 3-10 South Entrance Canopy Plan

3.8 Non-structural Building Elements

From our recent experience in evaluating similar buildings in Christchurch and Wellington, non-structural building elements (façade glass, ceilings, internal walls, overhead services etc.) constitute a significant portion of the repair/reinstatement cost following an earthquake. In a moderate seismic event, non-structural element damage may contribute heavily to downtime and repair costs and therefore the performance of these non-structural elements following a moderate seismic event could affect business continuity.

Assessment of these non-structural elements is not part of this DSA. However, a desktop study of the available documentation did not identify any large plant, ceilings, and partitions that would raise concern.

4 Assessment Methodology

4.1 Assessment Description

The DSA was completed in accordance with the **Guidelines.** The Guidelines provide solutions and methods for the assessment of existing buildings and give guidance for strengthening methodologies that are considered acceptable. Refer to **Appendix B** for the Assessment Inputs.

We have undertaken a stepped analysis approach to assess this building. We started with simpler elastic analysis methods and progressed with more complex analysis (non-linear and displacement-based analysis) to determine the seismic performance of the building.

4.2 Computer Modelling

4.2.1 Primary lateral resisting system

A computer model of the structure was developed using the ETABS computer program. Refer to **Figure 4-1** for the 3D View of the ETABS Model. The global structures behaviour was captured using non-linear equivalent static analysis. A Simple Lateral Mechanism Analysis (SLaMA) procedure was also undertaken to determine the global capacity of the structure.

The boundary supports were modelled with “compression-only” springs to capture the rocking behaviour of the building. The soil springs’ stiffnesses were modified by 50% and 200% of the recommended soil stiffness to get the lower and upper bounded dynamic properties of the building. The building was not sensitive to the different soil stiffnesses.

Finally, to assess the stair performance, the stairs were added to the 3D ETABS model to determine how much load they would attract given their proximity to shear walls.

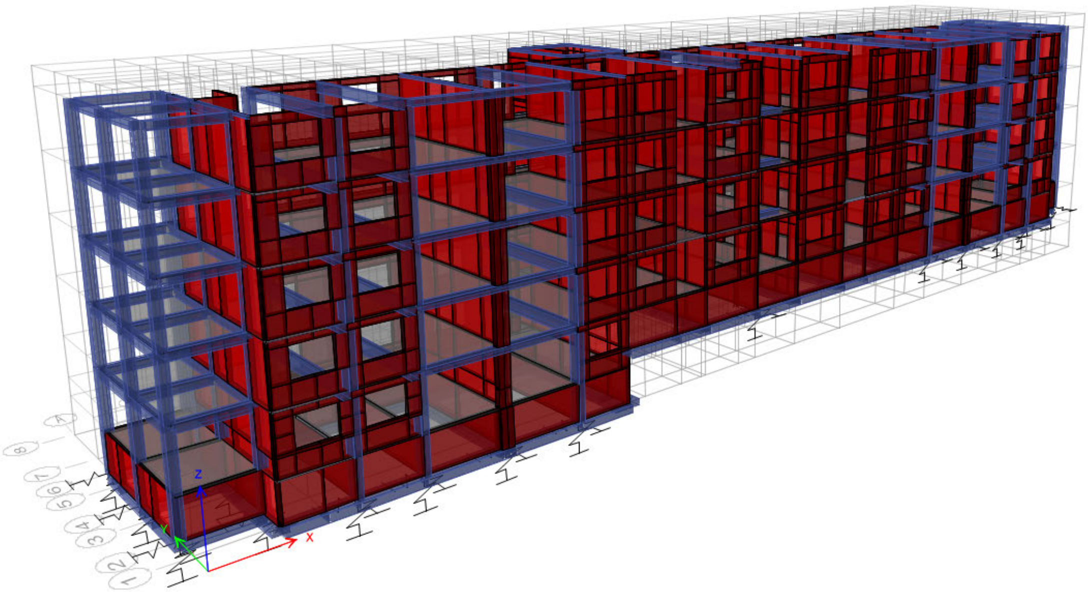


Figure 4-1 3D View of the Building ETABS Model

4.2.2 Diaphragms

The diaphragm acceleration demands were determined by the pESA method as recommended in NZS1170.5 C5.7.2.

These design accelerations/forces were then applied to the centre of mass of each diaphragm of the 3D ETABS model. For each diaphragm and for each direction of loading, the shear entering/exiting each vertical lateral resisting element (difference in shear above and below the level being considered) was extracted.

Due to the complexity of the diaphragms the diaphragm demands were assessed using the Grillage Method as recommended in the **Guidelines**. It is essentially an automated strut and tie analysis method to obtain demands. Capacities were determined using Appendix A of NZS 3101:2006. Refer to **Figure 4-2** for a plan view of a typical grillage model.

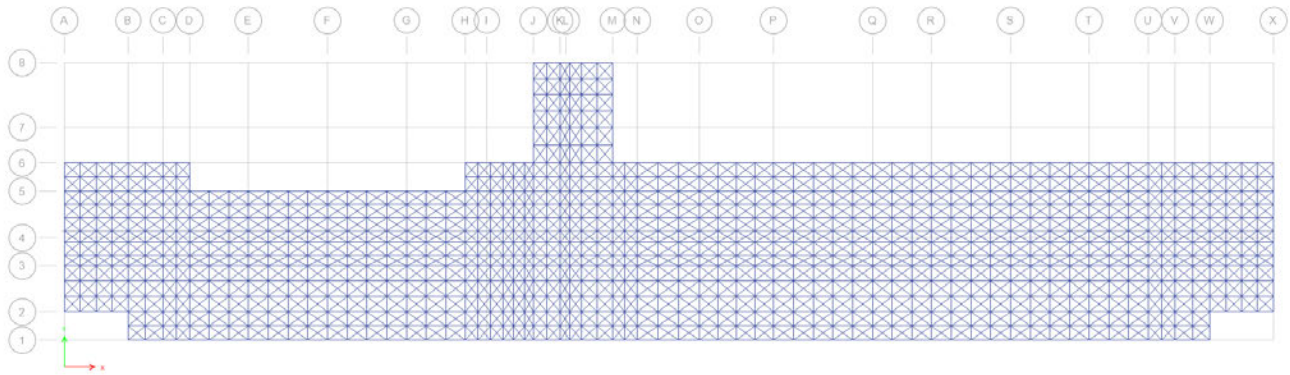


Figure 4-2 Grillage model

5 Peer Review

Following the issuance of the draft report, Beca undertook a peer review of the assessment. This process involved Beca reviewing the calculations prepared as part of the building assessment, providing comments and queries for Aurecon to address. These items were discussed with Beca at several meetings throughout the process.

Based on their review, the %*NBS* score for RC Shear Walls in the transverse direction changed from 60% to 65%, while no other %*NBS* scores for the remaining structural elements were altered. The peer review did not affect the overall %*NBS* rating of the building.

6 Assessment Results

6.1 Assessment Results Summary

The results of the DSA indicate that the Building's earthquake rating to be **25%*NBS*(IL2)** in accordance with the Guidelines. The earthquake rating is based on the lowest scoring element shown in **Table 6-1**.

Table 6-1 Summary of Elements - %NBS scores

Element:	%NBS(IL2):	Commentary:
RC Piers and Spandrel – Longitudinal Direction	50%	▪ The RC piers and spandrels have insufficient flexural capacity to resist 100% ULS loading. ▪ The plain round bars lap length limits the allowable steel stress. As a result, a single crack may form in the walls under a design level earthquake. This mechanism has an inferior energy dissipation capacity compared to modern walls.
RC Shear Walls – Transverse Direction	65%	▪ The RC shear walls have insufficient flexural capacity to resist 100% ULS loading. ▪ The plain round bars lap length limits the allowable steel stress. As a result, a single crack may form in the walls under a design level earthquake. This mechanism has an inferior energy dissipation capacity compared to modern walls. ▪ The RC foundations may also uplift and cause the building to rock.
Concrete Diaphragms:	100%	▪ The concrete diaphragm, reinforced with plain round bars, have sufficient capacity to transfer the diaphragm inertia and transfer loads to the RC lateral system.
RC Moment Frame Gravity Displacement with Block infill walls	30%	▪ The blockwalls within the RC frames, on Grids A and X, score 30%*NBS*(IL2) based on the walls out-of-plane capacity under seismic parts loading. The drawings indicate that the blockwall is unreinforced. ▪ The RC moment frames with block infill walls score 75%NBS based on the RC columns shear capacity for in-plane loading. The infill block wall causes flexural, and shear demands on the columns from the effective strut in the block walls. ▪ The remaining RC moment frames without blockwalls, have sufficient gravity carrying capacity under the expected ULS drifts.
Foundations:	100%	▪ The strip footing foundations can resist the soil bearing pressure demands and scores >100%*NBS*(IL2). ▪ The building is expected to slide at 40%ULS loading. However, the building sliding is not considered a life safety risk and therefore the score does not govern the building/foundation score.

Stairs	75%	▪ The stairs contain connections to the landings that are fixed with no allowance for sliding or seismic movement. As a result, the stairs may act as an unintentional strut in a design level earthquake. However, as the stairs are located next to a RC shear wall, the walls "protect" the stairs from attracting significant in-plane seismic loading and score 75%*NBS*(IL2) for in-plane loading. ▪ The stairs score 100% NBS for out-of-plane seismic parts loading.
Walls Out-of-Plane	25%	▪ The RC walls above Level 3 are cantilevering to support the roof system. This cantilever is as high as 4.7m in some locations. It is not expected that the roof system can provide restraint of the walls for out-of-plane loading. The walls score 25%*NBS*(IL2) for out-of-plane seismic parts loading.
Roof	70%	▪ The timber and aluminium roof is a flexible diaphragm and does not contain steel-cross braces or a plywood diaphragm. Therefore, the timber rafters must transfer seismic load from the roof to the RC walls by bending out-of-plane. ▪ The timber rafters score 70%*NBS*(IL2) for bending about the minor axis. ▪ The connections of the roof to the walls score at 100%*NBS*(IL2). However, information of the connections is incomplete and needs further investigation.
Canopies	100%	▪ The canopies have sufficient capacity to resist 100% ULS parts loading.

6.2 Structural Weaknesses

A structural weakness (**SW**) is an aspect of the building structure and/or the foundation that scores less than 100%*NBS*(IL2). The Critical Structural Weakness (**CSW**) is the lowest scoring structural weakness determined in the assessment. Based on the results of the DSA, the CSW for this building is:

- RC walls out-of-plane capacity at the roof level

See below for the other structural weaknesses for the elements considered in this DSA:

- Blockwalls out-of-plane capacity
- RC shear walls lateral capacity
- Stairs in-plane capacity
- RC Moment Frame Gravity Displacement with Block infill walls
- Roof system capacity

6.3 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.

There are no SSWs identified for this building.

6.4 Displacement and Inter-storey Drift

The building displacements up the height of the building obtained from our analyses for 100%ULS shaking are shown in **Figure 6-1** below.

Table 6-2 shows the structures time periods, global ductility demand at 100%ULS and the maximum inter-storey drift under 100%ULS shaking. The storey drift allows for the kdm modification factor and P-delta effects. In both directions, the drift is less than the design code limit of 2.5%.

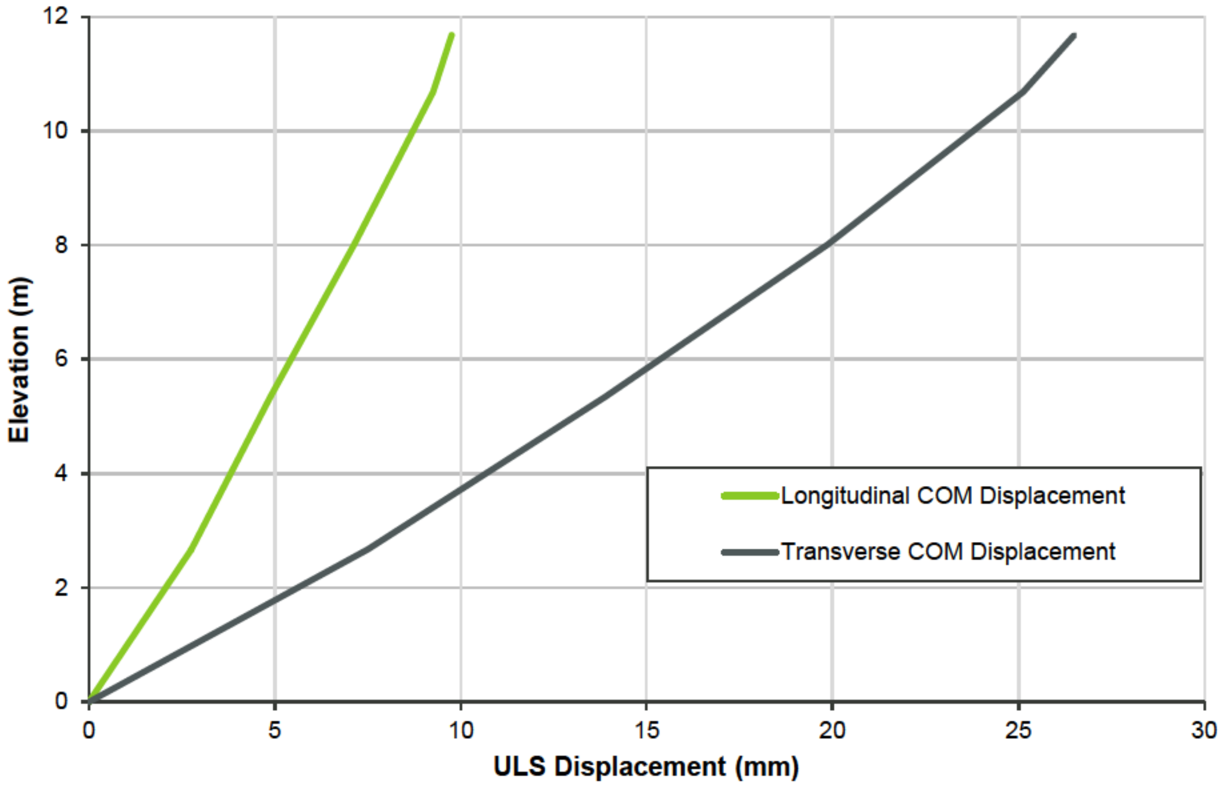


Figure 6-1 Estimated Building Displacements for 100% ULS shaking

Table 6-2 Estimated Time Periods, Global Ductility and Maximum Inter-Storey Drift for 100% ULS shaking

Direction	Fundamental Time Periods	Global Ductility	Maximum Inter-storey Drift
Longitudinal	<0.4s	~1.25	0.2%
Transverse	<0.4s	~1.25	0.5%

6.5 RC Shear Walls

Building Design

The building was constructed in the 1960s during a time where there were limited seismic requirements. The understanding of seismic engineering has vastly improved since the building was designed and the loading demand has increased significantly. Therefore, when a building of this age is assessed against the current code it starts at a significant disadvantage because it was designed to lesser loads.

The lateral system in this building contains doubly and singly reinforced walls with plain round bars. The plain reinforcing bars in these walls have insufficient lap lengths and lack proper end anchorages. In addition, the bars are spliced in potential plastic hinge regions (this causes bond degradation and potential bar slip) and lack lateral support to prevent reinforcement buckling.

However, it is worth noting that this building is considered regular, has many wall elements and is well-tied together with a concrete diaphragm that contain these characterises typically perform “better” in large earthquake shaking when compared to irregular structures.

Longitudinal and Transverse Direction

The RC piers, spandrels and shear walls flexural capacity at the ground level scores 50%*NBS* (IL2) in the longitudinal direction and 65%*NBS* (IL2) in the transverse direction.

At the ground level, a single crack is expected to form in the RC piers/walls under moderate earthquake shaking. Once a single crack forms, the plain round bars bond to the concrete deteriorates and slips. Once this occurs the piers and spandrels may exhibit a rocking response. Redistribution was considered to capture the elements post-yield rocking capacity.

Once a single crack forms, significant concrete cover spalling of the RC piers and spandrels and may increase the building displacements. Once the displacements increase, non-structural elements such as doors, windows and building services is expected to be significantly damaged. Once significant shear sliding occurs in the walls, the gravity carrying capacity of the walls may be lost. Refer to **Figure 6-2** and **Figure 6-3** for the building’s displacement shape under ULS shaking in each direction.

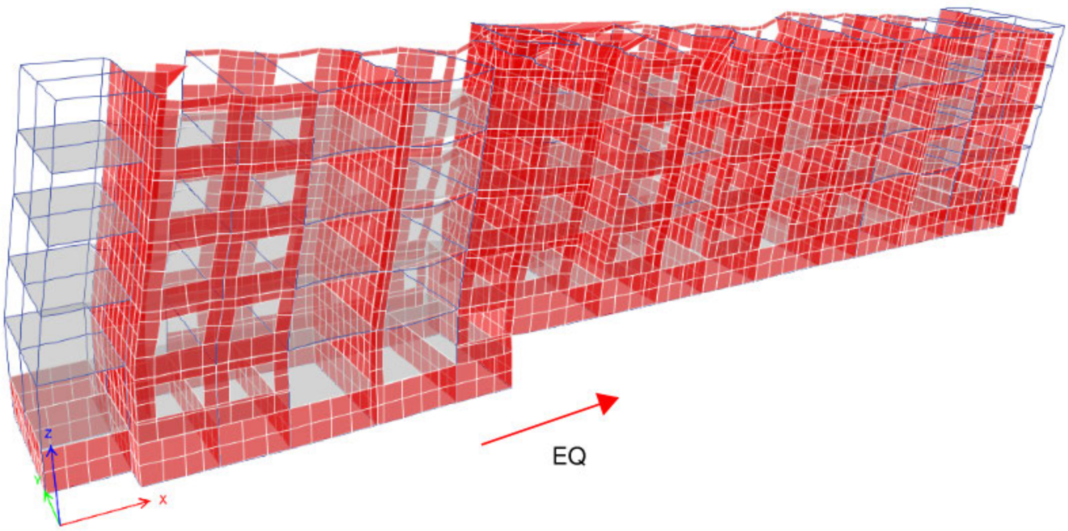


Figure 6-2 Buildings Displacement in the Longitudinal Direction at ULS demand

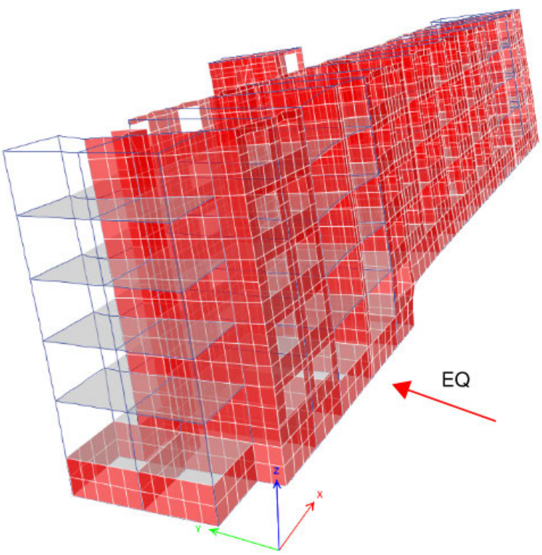


Figure 6-3 Buildings Displacement in the Transverse Direction at ULS demand

6.6 Concrete Diaphragms

The diaphragm tension capacity and the connection of the diaphragm to the main vertical lateral resisting elements scores **100%*NBS*(IL2)**.

The purpose of a diaphragm is to connect the discrete vertical elements of a structure together in the horizontal plane at regular intervals and be capable of transferring inertia, transfer and soil pressure forces to the lateral elements. The importance and behaviour of diaphragms was often overlooked until the Christchurch Earthquake in 2011, so it is common to find them deficient in older structures. In this building however, the diaphragm is cast in situ with ductile reinforcement and the concrete walls are regularly spaced, which reduces the forces that the diaphragm is required to transfer.

6.6.1 Typical Diaphragm

The diaphragms in both directions have sufficient capacity to reliably transfer 100% ULS inertia loads to the RC shear walls.

Diaphragm load must be transferred into the shear walls either at the ends of the wall (through compression bearing) or on the side walls (through shear-friction). Refer to **Figure 6-4** for the load transfer mechanism.

After considering redistribution, the plain round bars have sufficient capacity to transfer and collect the diaphragms inertia load to the RC walls and top of the floor plate. Refer to **Figure 6-5** that shows a grillage model of a typical floor plate in the transverse direction.

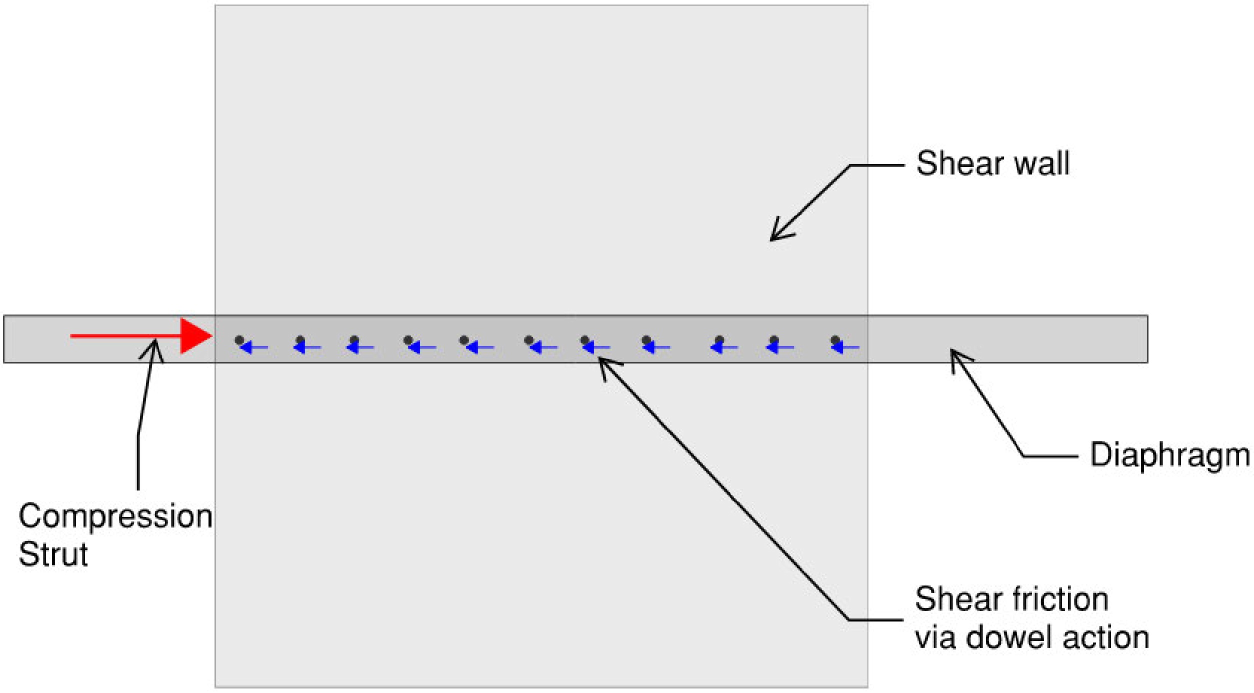


Figure 6-4 Shear wall elevation showing the load transfer mechanism

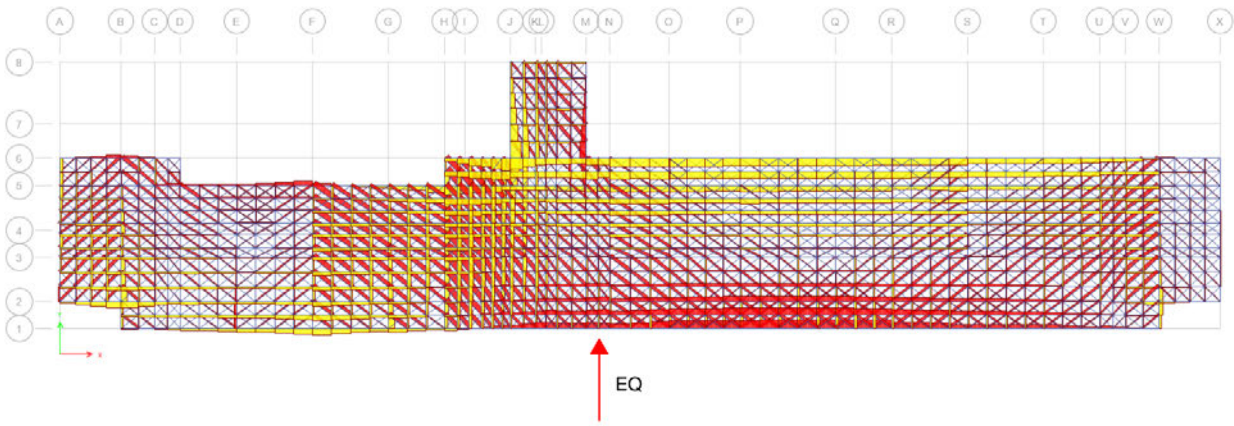


Figure 6-5 Grillage model of a typical floor plate in the transverse direction

6.7 Foundations

The building is supported on a combination of strip footings and pad foundations.

The building is supported by RC strip footings at each shear wall. The strip footings provide resistance to overturning of the building in the form of bearing pressure capacity. The footings were found to have sufficient capacity to resist the soil bearing demands. These footings score >100%*NBS*(IL2).

The strip footings only contain reinforced plain round bars at the bottom of the strip footing and no reinforcement at the top of the footing. The foundations were checked for bending and shear capacity to resist the bearing pressure as well as uplift demands. The foundation bending and shear capacity score >100%*NBS*(IL2).

6.8 Stairs

The Department of Building and Housing (now MBIE) issued their Practice Advisory 13 in response to concerns about stair collapse and damage observed in the Christchurch earthquake. The primary concern of this Practice Advisory is stairs with sliding support details in mid to high-rise buildings. For these types of stairs, the recommendation is that the stair flights be detailed so that the stairs are free to slide but with sufficient sliding ledge support width available.

The stairs are concrete cast in-situ. The connections of the stairs to the landings are fixed with no allowance for sliding or lateral movement of the building. The flights are cast into the face of the landings with plain round bars.

The stairs were added to the 3D ETABS model to determine how much load they would attract given their proximity to shear walls. The analysis results revealed that the entrance stairs attract some moment and axial demands, even though the stairs are surrounded by RC shearwalls. Therefore, the entrance stair does act as an unintentional strut in a design level earthquake. The entrance stair scores 75%*NBS* based on the stairs tension and moment capacity at the stairs knee joint. Refer to **Figure 6-6** that shows the stair's behaviour during ULS earthquake shaking.

The south and north end of the stairs scores 100% *NBS* (IL2). These stairs are only 600mm wide and therefore does attract significant seismic load.

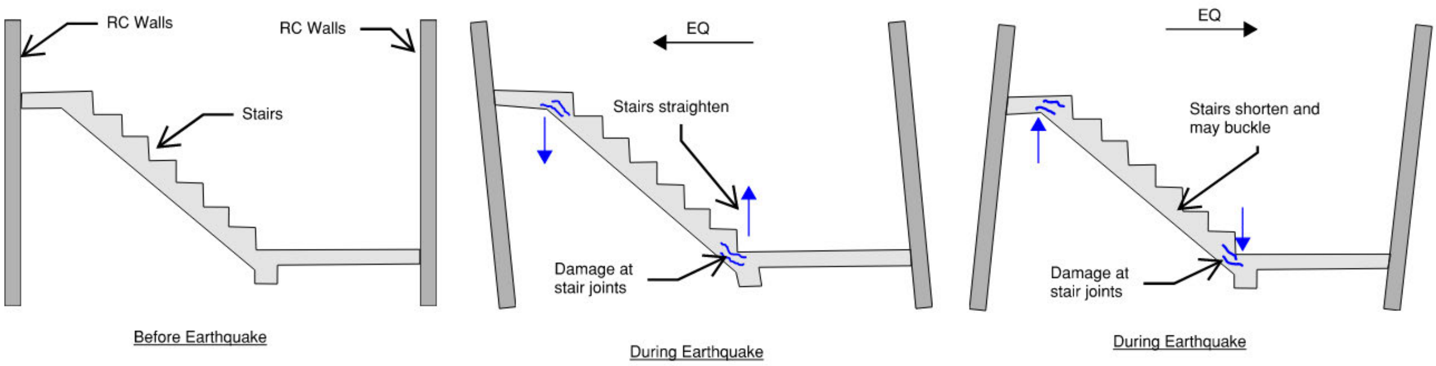


Figure 6-6 Stair's behaviour during ULS earthquake shaking

6.9 Concrete Walls Out-of-plane

The building's concrete walls cantilever up from Level 3 to roof level providing support the timber roof rafters and ceiling. The walls are approximately 2.6m high along its eastern and western edges and reaches up to 4.7m high along the roof apex.

The concrete walls about level 3 are considered cantilevers as the walls have been assessed based on the roof structure not effectively tying the walls together at high level. The roof structure would need to form a reliable diaphragm to restrain the walls out-of-plane. The roof structure as discussed in the section below has timber joists with bolted connections to the concrete walls.

The walls score 25%*NBS*(IL2) out-of-plane seismic parts loading. We note that if these walls were restrained at roof level, they would score greater than 67%NBS (IL2).

Walls located below Level 3 score 100%*NBS*. Refer to Figure 6-7Figure that shows the RC shear wall stress distribution based on out-of-plane seismic parts loading.

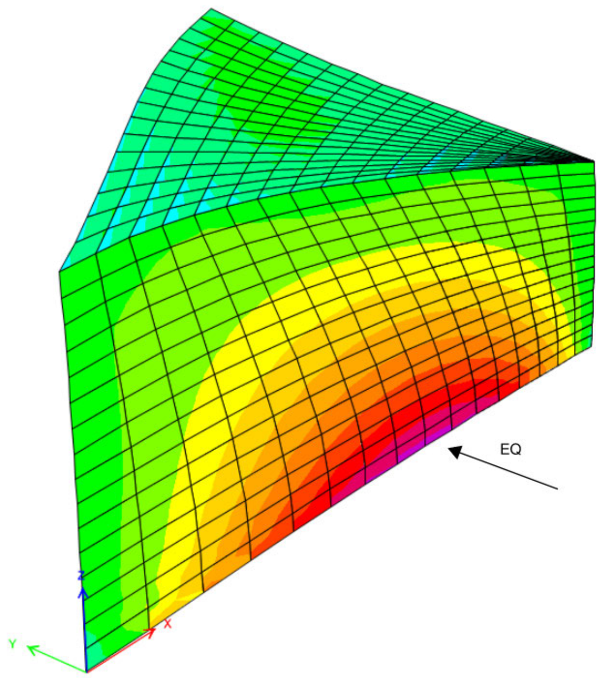


Figure 6-7 RC shear wall stress distribution based on out-of-plane seismic parts loading

6.10 Roof

The building's roof comprises of timber joists spanning in the building's transverse direction between concrete walls. The aluminium sheeting and timber purlins are not considered to form an effective diaphragm to transfer the lateral loads into the shear walls. The roof joists have been assessed based on tributary area, and therefore are required to bend out of plane to resist lateral loads in the longitudinal direction of the building.

The 4"x2" joists span up approximately 6.0m in some locations and these score 70%*NBS*(IL2) governed by combined in-plane and out-of-plane bending.

The joists connect to a timber end plate running along the concrete shear walls. The timber plate is bolted to the shear walls by ½" bolts. The existing documentation does not indicate the connection of the joists to the timber plate. Additionally, the bolt embedment is not known, and therefore the score of the connections cannot accurately be made. Further site investigation can be undertaken to survey these connections, the possible failure of these connections is closely related to the score of the concrete walls out-of-plane.

6.11 RC Moment Frame Gravity Displacement with Block infill walls

The blockwalls within the RC frames, on Grids A and X, score 30%*NBS*(IL2) based on the walls out-of-plane capacity under seismic parts loading. The drawings indicate that the blockwall is unreinforced. The out-of-plane resistance of block infill is based upon an arching model of the infill in the bounding frame. Once the capacity of the block infill is exceeded, the block may detach and fall away from the building. Refer to **Figure 6-8** that shows the blockwall displacement shape under ULS earthquake shaking.

To confirm if the blockwalls are unreinforced or reinforced we suggest further investigations is undertaken onsite. Presence of reinforcing steel in these walls may improve their %NBS score. We also suggest confirming the extend of the blockwalls.

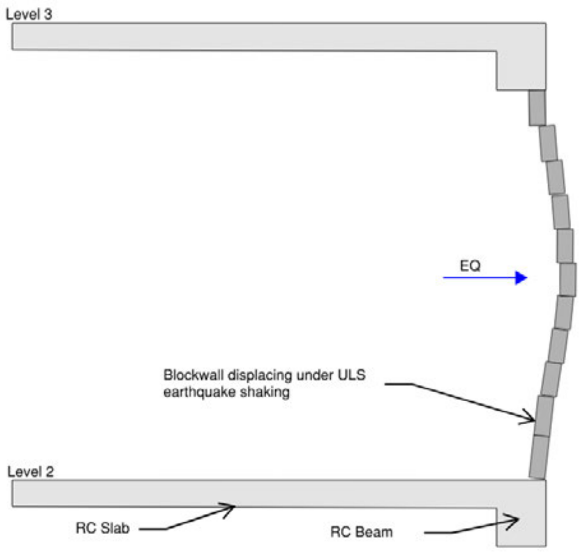


Figure 6-8 Blockwall out-of-plane

The RC moment frames with block infill walls score 75%NBS based on the RC columns shear capacity for in-plane loading. The infill block wall causes flexural, and shear demands on the columns from the effective strut in the block walls.

The remaining RC moment frames without blockwalls, have sufficient gravity carrying capacity under the expected ULS drifts.

7 Strengthening

We recommend that the building is seismically strengthened considering a two-stage approach. Stage 1 would be to strengthen the building to a minimum seismic rating of greater than **34*%NBS*(IL2).** Based on our review, the seismic strengthening, to achieve greater than 34*%NBS*(IL2), would include, but not be limited to:

- Increase the **RC wall out-of-plane** capacity by installing a new roof diaphragm with new connections to the concrete walls. The roof diaphragm can be in the form of steel cross braces and steel beams.
- Increase the **blockwalls out-of-plane** capacity by installing steel strong-backs to the blockwalls and RC beams. Also introduce a seismic gap between the blockwalls and RC columns by saw cutting a gap.

Stage 2 would be to seismically strengthen the building to a minimum rating of 67*%NBS* (IL2). Based on our review, the seismic strengthening to achieve 67%*NBS*(IL2) would include, but not be limited to:

- Increase the **RC walls lateral capacity** by installing new RC overlay walls, reinforced and continuous doweled into the existing RC walls. New foundations will also be required.

The strengthening options recommended are only of a schematic level detail and a detailed design will be required for Building Consent and construction documents. It is noted that the schematic design presented is one structural solution and there may be other solutions for the building. We envisage that the strengthening work would be completed in stages (i.e., floor by floor or groups of floors) to minimise occupant disruption. We note that the noise due to drilling and other construction activities will have impact on the building occupants.

We also recommend that part of any seismic upgrade or future fitout that the non-structural building elements (façade glass, ceilings, internal walls, overhead services and plant and equipment etc) is seismically restrained to meet the current standards.

We further recommend that in designing any seismic retrofit that the building owner should also consider the proposed increase in seismic hazard levels in Wellington. This would insulate the building against further future reductions in the seismic rating.

8 Future Code Changes

8.1 Hazard Zone Factor

The results of the updated National Seismic Hazard Model (NSHM) were released in October 2022. The previous update to the NSHM was in 2010. Since then, the science behind estimating earthquake rates and understanding and complexity of ground motion modelling have significantly advanced.

The NZSM provides the basis for setting the seismic demands in the design code NZS1170.5. Although the results are not a design standard or design loadings standard, they provide an indication of how the code may reflect the updated seismic hazard in future revisions. A possible outcome of this review will be an increase in the hazard zone factor, Z, for the Wellington region. This factor is used to determine the seismic risk for the area and hence the design standard for new buildings.

A future increase in the Hazard Factor will lead to an increase in the design level for new buildings in Wellington and potentially increase the standard required for existing buildings to achieve 100%NBS when assessed against that new standard.

8.2 Basin Edge Effects

The 2016 Kaikōura earthquake exposed the concept of the "basin edge effects." The basin edge efforts cause amplification of ground shaking due to the presence of soft soils in the sedimentary basin and cause larger peak ground accelerations than expected. The edge efforts are currently not incorporated in the Earthquake actions design code NZS 1170.5.

The basin edge effects have the potential to significantly increase the design standard for new buildings in particular locations in Wellington and potentially may increase the standard required for existing buildings to achieve 100%NBS (IL2) when assessed against that new standard. The "basin edge effects" is currently being discussed and reviewed by industry experts with no fixed timeframe when it will be introduced into the design standards. We note that the Hanson St housing complex location is less likely to be impacted by basin edge effects than other sites in the Wellington.

8.3 Seismic Guidelines

The **Yellow Chapter**, dated November 2018, provides the latest engineering knowledge on aspects involved in the assessment of concrete buildings, and to reflect what engineers learned from the Kaikōura earthquake.

However, its impact to the industry to still being assessed before it can be incorporated into regulation. Therefore, some aspects of the Guidelines may potentially change and hence affect the standard required for existing buildings to achieve 100%*NBS*(IL2).

9 Conclusions and Recommendations

9.1 Conclusion

The results of the DSA indicate the Building's earthquake rating to be ***25% NBS*(IL2)** in accordance with **The Guidelines**. This rating is based on the Critical Structural Weakness (CSW) of RC walls out-of-plane capacity at the roof level to resist seismic parts loading. The Building also contains other distinct elements that are classified as structural weaknesses.

9.2 Recommendations

To achieve a minimum rating of **67%*NBS*(IL2),** we consider the Building structure must be seismically strengthened. The seismic retrofit would include strengthening elements as described in **Section 7.**

We further recommend that in designing any seismic retrofit that the building owner should also consider the proposed increase in seismic hazard levels in Wellington. This would insulate the building against further future reductions in the seismic rating.

10 Explanatory Notes

- The information contained in this report has been prepared by Aurecon at the request of Wellington City Council and is exclusively for Wellington City Council's use and reliance. It is not possible to make a proper assessment of this review without a clear understanding of the terms of engagement under which it has been prepared, including the scope of the instructions and directions given to and the assumptions made by Aurecon. The report will not address issues which would need to be considered for another party if that party's particular circumstances, requirements and experience were known and, further, may make assumptions about matters of which a third party is not aware. Aurecon 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.
- This report contains the professional opinion of Aurecon as to the matters set out herein, in the light of the information available to it during preparation, using its professional judgment and acting in accordance with the standard of care and skill usually exercised by professional engineers providing similar services in similar circumstances. Aurecon is not able to give any warranty or guarantee that all possible damage, defects, conditions or qualities have been identified.
- The report is based on information that has been provided to Aurecon from other sources or by other parties. The report has been prepared strictly on the basis that the information that has been provided is accurate, complete and adequate, except where otherwise identified during site investigation inspections. To the extent that any information is inaccurate, incomplete or inadequate, Aurecon takes no responsibility and disclaims all liability whatsoever for any loss or damage that results from any conclusions based on information that has been provided to Aurecon.
- The inspections of the building discussed in this report have been undertaken to inspect the structure and confirm the adequacy of the existing drawings. This report does not address building defects. Where site inspections were undertaken, they were restricted to visual inspections with intent to determine existing building main structural elements only.
- We have not undertaken a review of secondary elements such as ceilings, building services, plant and partitions.

A

Appendix A - Definitions and Acronyms

Definitions and Acronyms

ADRS	Acceleration-displacement response spectrum
Brittle	A brittle material or structure is one that fractures or breaks suddenly once its probable yield capacity is exceeded. A brittle structure has little tendency to deform before it fractures.
Critical Structural Weakness (CSW)	The lowest scoring structural weakness determined from a DSA. For an ISA all structural weaknesses are considered to be potential CSWs.
Damping	The value of equivalent viscous damping corresponding to the energy dissipated by the structure, or its systems and elements, during the earthquake. It is generally used in nonlinear assessment procedures. For elastic procedures, a constant 5% damping as per NZS 1170.5:2004 is used.
Design Level or ULS earthquake	Design level earthquake or loading is taken to be the seismic load level corresponding to the ULS seismic load for the building at the site as defined by NZS 1170.5:2004 (refer to Section C3)
Detailed Seismic Assessment (DSA)	A seismic assessment carried out in accordance with Part C of these guidelines
Diaphragm	A horizontal structural element (usually a suspended floor or ceiling or a braced roof structure) that is strongly connected to the vertical elements around it and that distributes earthquake lateral forces to vertical elements, such as walls, of the primary lateral system. Diaphragms can be classified as flexible or rigid.
Ductile or Ductility	Describes the ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake
Elastic Analysis	Structural analysis technique that relies on linear-elastic assumptions and maintains the use of linear stress-strain and force-displacement relationships. Implicit material nonlinearity (e.g. cracked section) and geometric nonlinearity may be included. Includes equivalent static analysis and modal response spectrum dynamic analysis.
Flexible diaphragm	A diaphragm which for practical purposes is considered so flexible that it is unable to transfer the earthquake loads to shear walls even if the floors/roof are well connected to the walls. Floors and roofs constructed of timber, and/or steel bracing in a URM building, or precast concrete without reinforced concrete topping fall in this category. A diaphragm with a maximum horizontal deformation along its length that is greater than or equal to twice the average inter-storey drift. In a URM building a diaphragm constructed of timber and/or steel bracing.
Initial Seismic Assessment (ISA)	A seismic assessment carried out in accordance with Part B of these guidelines. An ISA is a recommended first qualitative step in the overall assessment process.

Nonlinear analysis	Structural analysis technique that incorporates the material nonlinearity (strength, stiffness and hysteretic behaviour) as part of the analysis. Includes nonlinear static (pushover) analysis and nonlinear time history dynamic analysis.
Non-structural item	An item within the building that is not considered to be part of either the primary or secondary structure. Non-structural items such as individual window glazing, ceilings, general building services and building contents are not typically included in the assessment of the building's earthquake rating.
OTM	Overturning moment.
Primary gravity structure	Portion of the main building structural system identified as carrying the gravity loads through to the ground. Also required to carry vertical earthquake induced accelerations through to the ground. May also incorporate the primary lateral structure.
Primary lateral structure	Portion of the main building structural system identified as carrying the lateral seismic loads through to the ground. May also be the primary gravity structure.
Probable capacity	The expected or estimated mean capacity (strength and deformation) of a member, an element, a structure as a whole, or foundation soils. For structural aspects this is determined using probable material strengths. For geotechnical issues the probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
Rigid diaphragm	A diaphragm that is not a flexible diaphragm
Secondary structure	Portion of the structure that is not part of either the primary lateral or primary gravity structure but, nevertheless, is required to transfer inertial and vertical loads for which assessment/design by a structural engineer would be expected. Includes precast panels, curtain wall framing systems, stairs and supports to significant building services items
Serviceability limit state (SLS)	Limit state as defined in AS/NZS 1170.0:2002 (or NZS 4203:1992) being the point at which the structure can no longer be used as originally intended without repair
Severe structural weakness (SSW)	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
Simple Lateral Mechanism Analysis (SlaMA)	An analysis involving the combination of simple strength to deformation representations of identified mechanisms to determine the strength to deformation (pushover) relationship for the building as a whole
Single-degree-of- freedom (SDOF)	A simple inverted pendulum system with a single mass
Structural element	Combinations of structural members that can be considered to work together; e.g. the piers and spandrels in a penetrated wall, or beams and columns in a moment resisting frame

Structural member	Individual items of a building structure, e.g. beams, columns, beam-column joints, walls, spandrels, piers
Structural sub-system	Combination of structural elements that form a recognisable means of lateral or gravity load support for a portion of the building: e.g. moment resisting frame, frame/wall. The combination of all of the sub-systems creates the structural system.
Structural system	Combinations of structural elements that form a recognisable means of lateral or gravity load support; e.g. moment resisting frame, frame/wall. Also used to describe the way in which support/restraint is provided by the foundation soils.
Structural weakness (SW)	An aspect of the building structure and/or the foundation soils that scores less than 100%NBS. Note that an aspect of the building structure scoring less than 100%NBS but greater than or equal to 67%NBS is still considered to be a SW even though it is considered to represent an acceptable risk.
Ultimate Limit State (seismic)	A term defined in regulations that describes the limiting capacity of a building for it to be determined to be an earthquake-prone building. This is typically taken as the probable capacity but with the additional requirement that exceeding the probable capacity must be associated with the loss of gravity support (i.e. creates a significant life safety hazard).
Ultimate limit state (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings.
XXX%NBS	The ratio of the ultimate capacity of a building as a whole or of an individual member/element and the ULS shaking demand for a similar new building on the same site, expressed as a percentage. Intended to reflect the expected seismic performance of a building relative to the minimum life safety standard required for a similar new building on the same site by Clause B1 of the New Zealand Building Code.
XXX%ULS shaking (demand)	Percentage of the ULS shaking demand (loading or displacement) defined for the ULS design of a new building and/or its members/elements for the same site. For general assessments 100%ULS shaking demand for the structure is defined in the version of NZS 1170.5 (version current at the time of the assessment) and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016. For engineering assessments undertaken in accordance with the EPB methodology, 100%ULS shaking demand for the structure is defined in NZS 1170.5:2004 and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016 (with appropriate adjustments to reflect the required use of NZS 1170.5:2004). Refer also to Section C3.

B

Appendix B – Assessment Inputs

Assessment Inputs

Structural Layout

The building layout, member sizes, detailing and material grades have been taken from available design drawings and calculations. A site inspection of the interior and exterior was carried out to confirm that the drawings and documentation was generally in accordance with the as-built configuration. The following drawing documentation was available at the time of the assessment:

- Existing Structural drawings titled "*Hanson Street Flats Development Block 1*" dated 1965

Dead, Superimposed Dead Loads and Live Loads.

See Table below for the Dead, Superimposed dead loads and Live Loads used in the assessment. The self-weight of the walls, frame members and slabs are calculated by the structural analysis program based on the input section size and unit weight. The design live loads were adopted as indicated as per structural drawings and in accordance with NZS1170.1 loading.

Table: Dead, Superimposed dead loads and Live Loads used in the assessment

Load Type	Load
Dead Load	Calculated by the structural analysis program based on the input section size and unit weight
Super Imposed Dead Load	0.5 kPa
Live Load	0.25kPa for inaccessible roof 5kPa for plantroom 1.5kPa for apartment levels 4.0kPa for stairwells

Seismic Weight

The seismic mass was calculated based on the NZS 1170.5:2004 loading combination W = G + ΨEQu, where ΨE = 0.0 for roof. Where applicable, an area reduction factor was also applied to the live load in accordance with clause 3.4.2 of AS/NZS 1170.1:2002.

Wind Loads

Consideration of wind loads is outside the scope of this assessment.

Seismic loading

The seismic loads were determined in accordance with NZS1170.5 with the following parameters.

Table: Seismic parameters for building assessments

Parameter	Value
Design Working Life	50
Importance level	2
Site Subsoil Classification	B
Hazard Factor (Z)	0.4

Material Properties

The following material properties and corresponding characteristic and probable strengths were used as per the Assessment Guideline Tables C5.3, C5.4 and Section C6. No material specification regarding the concrete and steel used at the time was found in the structural drawings. No physical materials testing has been undertaken to validate the assumed material properties.

Table: Material properties

Item	Characteristic Design Strength (MPa)	Assessment (Probable) Strength (MPa)
Reinforcing Steel – Beams	275 MPa	324 MPa
Concrete	20 MPa	30 MPa
Structural Steel	300 MPa	345 MPa

Geotechnical Parameters

The following parameters, taken from the *Geotechnical Parameters for Hanson Court - Detailed Seismic Assessment (DSA),* by Aurecon, dated 03/02/23, was used to assess the strip footings and base-shear takeout.

Table 3.1: Geotechnical parameters and capacities for building assessments

Parameters	Values
Soil Bearing Capacity	600kPa
Subgrade modulus	5MPa to 20MPa
Friction coefficient	0.35. The friction capacity is considered to develop within 15mm to 20mm displacement.
Soil Density	20.5kN/m^3

C

Appendix C – Importance Level Description

Importance Level Description

Importance Levels for Building Types – New Zealand Structures

Importance Level:	Comment:	Example:
1	Structures presenting a low degree of hazard to life and other property	Structures with a total floor area of <30 m2 Farm buildings, isolated structures, towers in rural situations Fences, masts, walls, in-ground swimming pools
2	Normal structures and structures not in other importance levels	Buildings not included in Importance Levels 1, 3 or 4 Single family dwellings and Car parking buildings
3	Structures that as a whole may contain people in crowds or contents of high value to the community or pose risks to people in crowds	Buildings and facilities as follows: a) Where more than 300 people can congregate in one area b) Day care facilities with a capacity greater than150 c) Primary school or secondary school facilities with a capacity greater than 250 d) Colleges or adult education facilities with a capacity greater than 500 e) Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities f) Airport terminals, principal railway stations with a capacity greater than 250 g) Correctional institutions h) Multi-occupancy residential, commercial (including shops), industrial office and retailing buildings designed to accommodate more than 5000 people and with a gross area greater than 10 000m2 i) Public assembly buildings, theatres and cinemas of greater than 1000m2 Emergency medical and other emergency facilities not designated as post-disaster Power-generating facilities, water treatment and wastewater treatment facilities and other public utilities not designated as post-disaster Buildings and facilities not designated as post-disaster containing hazardous materials capable of causing hazardous conditions that do not extend beyond the property boundaries

4	Structures with special post-disaster functions	Buildings and facilities designated as essential facilities Buildings and facilities with special post-disaster function Medical emergency or surgical facilities Emergency service facilities such as fire, police stations and emergency vehicle garages Utilities or emergency supplies or installations required as backup for buildings and facilities of Importance Level 4 Designated emergency shelters, designated emergency centres and ancillary facilities Buildings and facilities containing hazardous materials capable of causing hazardous conditions that extend beyond the property boundaries
5	Special structures (outside the scope of this Standard-acceptable probability of failure to be determined by special study)	Structures that have special functions or whose failure poses catastrophic risk to a large area (e.g. 100 km2) or a large number of people (e.g., 100 000) Major dams, extreme hazard facilities

Annual Probability of Exceedance

Design Working Life:	Importance Level:	Annual probability of exceedance for ultimate limit states			Annual probability of exceedance for serviceability limit states	
		Wind	Snow	Earthquake	SLS1	SLS2 Importance level 4 only
Construction equipment	2	1/100	1/50	1/100	1/25	-
Less than 6 months	1 2 3 4	1/25 1/100 1/250 1/1000	1/25 1/50 1/100 1/250	1/25 1/100 1/250 1/1000	- 1/25 1/25 1/25	-
5 years	1 2 3 4	1/25 1/250 1/500 1/1000	1/25 1/50 1/100 1/250	1/25 1/250 1/500 1/1000	- 1/25 1/25 1/25	-
25 years	1 2 3 4	1/50 1/250 1/500 1/1000	1/25 1/50 1/100 1/250	1/50 1/250 1/500 1/1000	- 1/25 1/25 1/25	- - - 1/250
50 years	1 2 3 4	1/100 1/500 1/1000 1/2500	1/50 1/150 1/250 1/500	1/100 1/500 1/1000 1/2500	- 1/25 1/25 1/25	- - - 1/500
100 years or more	1 2 3 4	1/250 1/1000 1/2500 *	1/150 1/250 1/500 *	1/250 1/1000 1/2500 *	- 1/25 1/25 1/25	- - - *

D

Appendix D – Assessment Summary

Assessment Summary

1. Building Information	
Building Name/ Description:	Hanson Court Block A
Street Address	Hanson Court complex on Hanson St
Territorial Authority	Wellington City Council
No. of Storeys	4 Storeys
Area of Typical Floor (approx.)	Approx. 480m^2 per floor
Year of Design (approx.)	1963
NZ Standards designed to	N/A
Structural System including Foundations	Lateral system consists of RC shear walls, spandrels, and piers. Foundation system is RC strip footings
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	The site subsoil classification, in terms of NZS1170.5:2004 Clause 3.1.3, is Class B.
Previous strengthening and/ or significant alteration	N/A
Heritage Issues/ Status	N/A
Other Relevant Information	N/A

2. Assessment Information	
Consulting Practice	Aurecon NZ Ltd
CPEng Responsible, including: • Name • CPEng number • A statement of suitable skills and experience in the seismic assessment of existing buildings	▪ [redacted] ▪ 21 years' experience as a structural engineer with significant experience in the seismic assessment of existing buildings
Documentation reviewed, including: • date/ version of drawings/ calculations • previous seismic assessments	▪ Existing Structural drawings titled "*Hanson Street Flats Development Block 1*" dated 1965
Geotechnical Report(s)	Geotechnical Parameters for Hanson Court - Detailed Seismic Assessment (DSA), by Aurecon, dated 03/02/23. Geotechnical desktop study Appendix G
Date(s) Building Inspected and extent of inspection	12/2022 Visual external, no material test or intrusive investigation has been carried out.
Description of any structural testing undertaken and results summary	N/A
Previous Assessment Reports	2009 Aurecon DSA report.
Other Relevant Information	N/A

3. Summary of Engineering Assessment Methodology and Key Parameters Used	
Occupancy Type(s) and Importance Level	2
Site Subsoil Class	B

For a DSA: Summary of how Part C was applied, including: • the analysis methodology(s) used from C2 • other sections of Part C applied	Equivalent Static Analysis and Slama The DSA was completed in accordance with the Guidelines. The Guidelines provide solutions and methods for the assessment of existing buildings and give guidance for strengthening methodologies that are considered acceptable. We have undertaken a stepped analysis approach to assess this building. We started with simpler elastic analysis methods and progressed with more complex analysis (non-linear and displacement-based analysis) to determine the seismic performance of the building.
Other Relevant Information	N/A

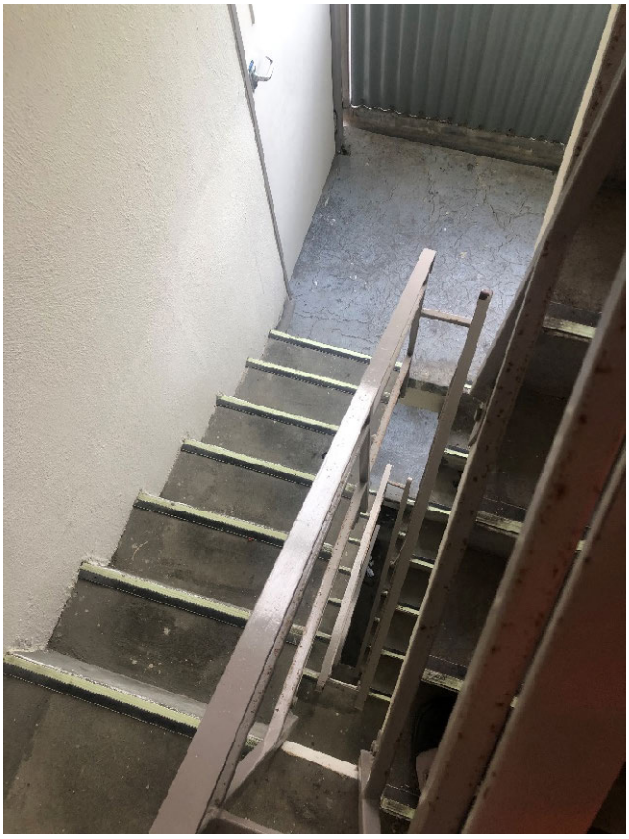
4. Assessment Outcomes		
Assessment Status	Final	
Assessed %NBS Rating	25%	
For a DSA:		
Comment on the nature of Secondary Structural and Non-structural elements/ parts identified and assessed	Non-structural elements have not been assessed at this stage.	
Describe the Governing Critical Structural Weakness	RC Shear Wall Lateral Capacity	
If the results of this DSA are being used for earthquake prone decision purposes, and elements rating <34%NBS have been identified (including Parts):	Engineering Statement of Structural Weaknesses and Location: ▪ RC out-of-plane capacity	Mode of Failure and Physical Consequence Statement(s): ▪ The RC walls above level 3 are cantilevering to support the roof system, and it is not expected that the roof system can provide restraint of the walls for out-of-plane loading. The failure of RC wall out-of-plane may impose potential life-safety hazard to building users.
Recommendations (Optional for EPB purposes)	Strengthening should be undertaken to increase the structure's rating to a minimum of 67%NBS(IL2) if feasible.	

E

Appendix E – Building Photographs


Photograph 1: Western Elevation


Photograph 2: Blockwork infill wall



Photograph 3: Typical concrete stairs

Photograph 4: Accessway on western elevation

Photograph 5: Stair Core Southern Elevation

F

Appendix F – Peer Review Letter

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12 December 2023

Peter Mora, Casey Zhang, Mario Venter
Wellington City Council
PO Box 2199
Wellington 6140

Dear Peter, Casey, Mario

Peer Review of DSA Blocks A, B, C, D and E, Hanson Court Apartments, Newtown, Wellington

Beca Ltd (Beca) has been engaged by Wellington City Council to carry out an independent peer review of Aurecon's Detailed Seismic Assessment (DSA) for the Hanson Court buildings located at the corner of Hanson and Hutchison Street, Newtown, Wellington. It consists of the following buildings: Block A(1), B(Tower), C(4), D(2) and E(3).

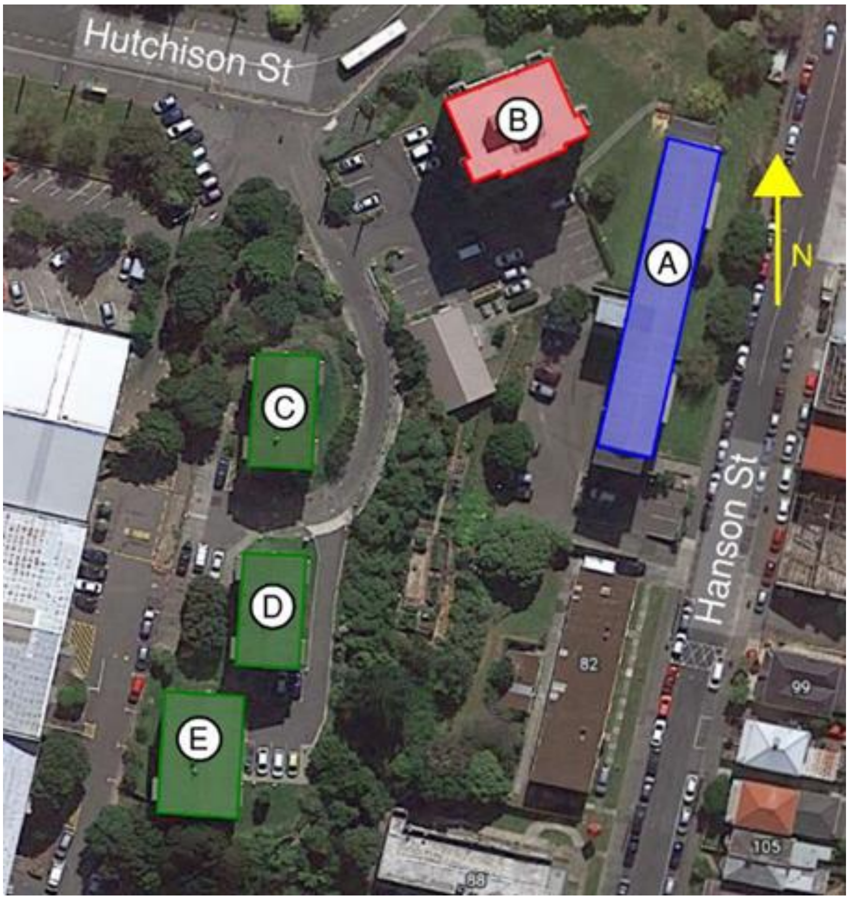


Fig. 1: Plan showing layout of Block A, B, C, D and E

make everyday better.

1.1 Information Received

Building		Document Code	Date	Revision
Drawings	Binder-All Blocks			
Block A (1)	Detailed Seismic Assessment Report -Draft (Final)	523020-001-REP-SS-001	09/03/2023	0
	Hanson Block 1 DSA Peer Review Calculations	523020-0000-STR	04/2023	1
Block B (Tower)	Block B DSA Report-Draft	523020-001-REP-SS-006	04/2023	1
	Hanson Tower DSA Peer Review Calculations	523020-0000-STR	04/2023	1
	Foundations	-	18/07/2023	
	RC Walls w Sp=0.9	-	18/07/2023	-
	Tower Diaphragm Laps	-	18/07/2023	-
	Block B Foundations	-	07/08/2023	2
Block C (4) and D (2)	Block C and D DSA Report	523020-001-REP-SS-002&4	05/05/2023	1
	Hanson Block 2 and 4 DSA Peer Review Calculations	523020-0000-STR	04/2023	1
	Block C and D RC Walls and ADRS	-	18/07/2023	-
	Block E DSA Report	523020-001-REP-SS-003	05/05/2023	1
Block C (4)	Hanson Block 3 DSA Peer Review Calculations	523020-0000-STR	04/2023	1
Geotechnical Report by Aurecon	Geotechnical parameters for Hanson Court-Detailed Seismic Assessment (DSA)	P523020	03/02/2023	A

1.2 Scope of Beca's Review

Beca was asked to undertake a peer review of the DSAs Block A, B, C, D and E with focus on identifying what items are above and below 34%*NBS*.(IL2)

1.3 Buildings Description

Hanson Court Apartments comprises of 5 buildings-Block A, B, C, D and E.

- The Block A is a rectangular in plane 60m x 8m, four-storey apartment block located towards the Eastern edge of the site. It was constructed circa 1963. The main lateral resisting system in both longitudinal and transverse directions consist of 200mm thick perimeter and 150mm thick internal reinforced concrete (RC) shear walls. The floor structure is 125mm thick reinforced concrete (RC) flat slabs. The structure is founded on a mixture of strip footings and pad foundations, vary in size, and a slab on grade. The roof is formed of the timber structure.

- The Tower Building-Block B is a rectangular in plane 21m x 15m, nine-storey apartment block located towards Hutchison Street at the Northern edge of the site. It was constructed circa 1967. The main lateral resisting system in both longitudinal and transverse directions consist of 200mm thick perimeter and 150mm thick internal reinforced concrete (RC) shear walls. The floor structure is 125mm thick reinforced concrete (RC) flat slabs. The structure is founded on a mixture of strip footings and pad foundations, vary in size. The roof is formed of the timber structure.
- Block C (4) and D (2). Each block is rectangular in plane 17.5mx9.3m, four-storey apartment block located to western edge of the site. They were constructed circa 1964. The main lateral resisting system in both longitudinal and transverse directions consist of 200mm thick perimeter and 150mm thick internal reinforced concrete (RC) shear walls above 1st floor. The floor structure is 150mm thick reinforced concrete (RC) flat slabs. The roof is formed of the timber structure. The structure of Block C is founded on strip footings and slab on grade. The structure of Block D is founded on strip footings, slab on grade and reinforced concrete pile foundations joined by ground beams to the centre and eastern sides of the building.
- Block E (3) building is rectangular in plane 19mx12m, four-storey apartment block located to western edge of the site. They were constructed circa 1964. The main lateral resisting system in both longitudinal and transverse directions consist of 200mm thick perimeter and 150mm thick internal reinforced concrete (RC) shear walls above 1st floor. The floor structure is 150mm thick reinforced concrete (RC) flat slabs. The structure is founded on strip footings, slab on grade and reinforced concrete pile foundations joined by ground beams to the southern side of the building. The roof is formed of the timber structure.

1.4 Aurecon's Seismic Assessments Results

Aurecon has determined that the buildings achieved the following earthquake score less than 34%*NBS*.

Building	Revision 0 before peer review July 2023	Revision 1 after peer review Dec 2023
Block A (1)	30%*NBS* RC Moment Frame with Block infill walls on Grids A and X	Aurecon suggested that further investigations would be undertaken on site to confirm the extent and present of the reinforcement in the block walls
	25%*NBS* Out-of- Plane capacity of RC walls located above level 3	Minimum score of 25%NBS (IL2) for Out-of- Plane capacity of RC walls remain until site investigations carried out to confirm the structure.
Block B (Tower)	Min score 45%*NBS* RC Shear Walls have insufficient flexural and ductility capacity in Longitudinal Direction.	Min score 45%*NBS*
Block C (4) and D (2)	30%*NBS* Stairs. Out-of-plane flexural capacity of RHS stringers	100 %*NBS* Reviewed dimensions of stringer and updated score
	25%*NBS* Out-of- Plane capacity of RC walls located above level 3	Min score of out of plane (OOP) capacity of RC wall located above Level 3, is

		25%NBS (IL2). Everything below level 3 already scores ≥34%NBS.
Block E (3)	30%*NBS* Stairs. Out-of-plane flexural capacity of RHS stringers	100*%NBS* Reviewed dimensions of stringer and updated score
	25%*NBS* Out-of- Plane capacity of RC walls located above level 3	Final conclusion: Min score of out of plane (OOP) capacity of RC wall located above Level 3, based on Aurecon report, is 25*%NBS* (IL2). Everything below level 3 already scores ≥34%NBS

These buildings were assessed in accordance with the guideline document '*The Seismic Assessment of Existing Buildings-Technical Guidelines for Engineering Assessments'*, dated July 2017, updated Section C5-*Concrete Buildings-Proposed Revision to the Engineering Assessment Guidelines* dated 2018.

All buildings were an Importance level 2 (IL2) structure, located on a Site Subsoil Class B site for Blocks A and B and a Site Subsoil Class C site for Blocks C, D and E in accordance with Aurecon's geotechnical report dated 03/02/2023.

1.5 Peer Review Summary

Based on our review of the available information provided to us and our discussions with Aurecon, we have provided the review comments as listed in the peer review register for each block separately. The peer review of each block was completed, and we comment as followings:

<u>Block A (1)</u>

- **RC Moment Frame with Block infill walls located on Grids A and X.**

Aurecon suggested that further investigations would be undertaken on site to confirm the extent and present of the reinforcement in the block walls. %NBS score of these items should be reviewed based on the results of the investigation.

- **Out-of- Plane capacity of RC walls located above level 3.**

There was no sufficient information provided. The further investigations on site should be carried out to confirm the extent of the reinforcement in the walls and %NBS score of these items should be reviewed based on the results of the investigation.

These items were closed out.

Conclusion: Minimum score of 25%*NBS* (IL2) for Out-of- Plane capacity of RC walls remains until the investigations carried out to confirm the structure.

<u>Block B -Tower</u>

Min score, based on Aurecon report, is **45%NBS** (IL2) for shear walls in Longitudinal direction. They have insufficient flexural and ductility capacity.

<u>Block C (4) and D (2)</u>

- **Stairs. Out-of-plane (OOP) flexural capacity of RHS stringers.**

We initially raised some questions around whether the right thickness of RHS stringer's sections was used for the assessments and were RHS stringers considered as a part of the system not as single element. Aurecon reviewed their assessment and calculations and achieved a score of 100%*NBS*. The comments were closed out.

- **Out-of- Plane capacity of RC walls located above level 3.**

The further investigations on site should be carried out to confirm the connection details of the timber roof structure to the walls and %NBS score of these items should be reviewed based on the results of the investigation.

Conclusion: Only the walls at the top floor would be required minor strengthening in order to achieve 34%NBS(IL2), unless Aurecon's on-site investigation confirms that there is good roof diaphragm connection then the score for the OOP may better. Min score of out of plane (OOP) capacity of RC wall located above Level 3, based on Aurecon report, is **25%NBS** (IL2). Everything below level 3 already scores ≥34%NBS.

Block E (3)

- **Stairs. Out-of-plane (OOP) flexural capacity of RHS stringers.**

We initially raised some questions around whether the right thickness of RHS stringer's sections was used for the assessments and were RHS stringers considered as a part of the system not as single element. Aurecon reviewed their assessment and calculations and achieved a score of 100%*NBS*. The comments were closed out.

- **Out-of- Plane capacity of RC walls located above level 3.**

The further investigations on site should be carried out to confirm the connection details of the timber roof structure to the walls and %NBS score of these items should be reviewed based on the results of the investigation.

Conclusion: Only the walls at the top floor would be required minor strengthening in order to achieve 34%NBS(IL2), unless Aurecon's on-site investigation confirms that there is good roof diaphragm connection then the score for the OOP may better. Min score of out of plane (OOP) capacity of RC wall located above Level 3, based on Aurecon report, is **25%NBS** (IL2). Everything below level 3 already scores ≥34%NBS. .

The updated Reports for Block A, B, C D and E based on the results of the peer review recorded in the registers and our discussions were not provided to us.

1.6 Conclusion

After completion of the peer review, we comment as followings:

- Block A, C, D and E are all rated less 34 *%NBS* (IL2).
- Block B is rated greater 34 *%NBS* (IL2).

We have prepared a peer review register for each block attached and all items are now closed out. We have no further comments.

Attached is our PS2 – Design Review, indicating that we believe on reasonable grounds that the design of the structural framing is generally in compliance with the Building Code Part B1 – Structure.

Specific exclusions to our checks and scope are as follows:

Geotechnical review. No review of the geotechnical engineering and overall ground conditions and results has been undertaken.

Plant and equipment. This exclusion extends to seismic restraint of the equipment and serviceability criteria.

Serviceability criteria and analysis for plant, equipment and operation of the plant has been excluded.

Secondary and tertiary structure and non-structural elements.

Any other structural elements that have not been assessed by Aurecon.

Durability.

The following documents are attached to this letter:

Peer Review Registers for Block A, B,C, D and E, dated December 2023.

Please contact the undersigned should you wish to discuss any aspect of the peer review.

Yours sincerely

[redacted]

[redacted]

Technical Director - Structural Engineer

on behalf of

Beca Limited

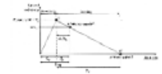
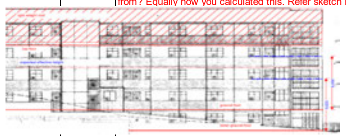
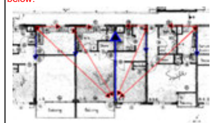
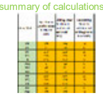
[redacted]

Sensitivity: General

Review Register

JOB NAME	Hanson Court Blocks A
JOB NUMBER	5275360
ELEMENTS	
DATE	12/12/2023
Reviewers	[redacted] s(7)(2)(a)

ISSUE DESCRIPTION

No.	ITEM / ELEMENT	Reference	Beca's Comments		Designer Respond		Closeout Comments		Designer Respond	STATUS
			Date	Comment	Date	Comment	Date	Comment	Comment	
1	General Comment.									
1	Building Analysis	Calculations	13/07/2023	The calculations indicate that the 3/8" round bar have adequate lap lengths. RC wall summary of the wall performance notes that a single crack will form at the base of the walls and resulting in slippage of the bars therefore limiting their capacity. I understand that the single crack, due to minimal vertical reinforcing, will result in localised bars strains that limit the rotation capacity but if the bars have more than enough anchorage length I wouldn't have expected this to limit the wall capacity. Please confirm the wall rotational capacities are an that this was used in the analyis model.	27.09.23	For the rotational capacity of the walls, in accordance with the guidelines, the smaller value among the rocking plastic capacity, deformed bars' plastic capacity, and the out-of-plane stability plastic rotation is used to determine the plastic rotation capacity of the wall. For instance, if a plain round bar wall has a rocking plastic capacity of 5.0% (using C5.40) and a deformed plastic capacity of 1.0% (C5.41), then the plastic rotation capacity of the wall is considered as 1.0%. Regarding plain round bars with sufficient development length, once the tension capacity in the bar is reached, the bond between the concrete and the bar is lost, and the wall starts to rock. Please see below for an example of the force vs. displacement plot of a wall with plain round bars, as given in the C5 guidelines seminar by Concrete NZ. The calculated wall rotation capacities are found on pages 54 and 61 of the calculations. 	6/10/2023	Just to clarify: The bar anchor length is sufficient to allow the bars to yield. If the bars yield (not slip, as it is noted the bars have adequate anchorage length) a single crack will occur limiting the wall rotational capacity. However, the walls are likely to rock at foundation level due to insufficient restoring weight. Therefore, the wall plastic rotation capacity limited to it's ability to rock. Question: what damping can you get from a rocking system? Is the soil likley to deform plastically therefore is ratchetting a posibility at high deformations? Or have you limited the rocking capacity to account for this? The ADRS should have allowed for the benefit of rocking. 1/11/23: Note, medium damping of 7-10% used. Bearing considered & bearing capacity not expected to be exceeded at ULS loads thereofre ratcheting unlikely. NFC.	We have considered Median damping in accordance with Table C2D.1. Based on this, we obtain a damping value between 7-10%. The majority of the walls are governed by out-of-plane lateral stability, with some rocking walls. The calculated wall rotation capacities can be found on pages 54 and 61 of the calculations. Regarding soil plasticity, based on our calculations, we do not exceed the bearing capacity at ULS displacements; therefore, soil plasticity and ratcheting are not expected. ADRS takes into account the benefit of rocking.	Closed
2	Seismic Demands		17/07/2023	The transverse wall assessment notes the wall capacity can support a ductility of 1.5 but propsoses using a Sp =1.0 (instead of the standard 0.9). The justification being the walls are not expected fair better than a mu=1.5. However, this could be said of all checks made using the guidelines but the guidelines do not appear to recommend using the higher Sp=1.0 for limited ductile elements. For example, the guidelines recommends calculating the diaphragm demands based on a mu=1.25 & Sp=0.9. Please review.	27.09.23	Our calculations for the transverse walls have a ductility capacity of 1.25, not 1.5. Please refer to page 53, which shows this. We agree that Sp = 0.9 can be used. If we consider an Sp = 0.9, then the %NBS in the transverse direction is 58% / 0.9 = 64% NBS. We note that we are at the top of the spectrum, and the percentage of NBS is 58% when using Sp = 1.0. We will update the %NBS for the walls from 60% to 65%.	6/10/2023	Noted, ductility limits and Sp values reviewed. NFC		Closed
3	Global Capacity curves		17/07/2023	The Combined Wall (1 to 3) capacities shows the combined capacity reduces once the walls exceed their capacity. Once the wall capacity is exceeded does this mean the wall doesn't contribute to the global stability under subsequent cyclic loads? Has the global wall check been carried out for the initial case that the first wall exceeds it capacity (small displacment) or the where the final case where the one wall resists all the load (larger displacment)?	27.09.23	Once the wall capacity (considering plain round bar steel) is exceeded, the wall will rock. Therefore, the wall will contribute to the global stability under subsequent cyclic loads based on the wall's rocking capacity. The global wall analysis was undertaken using the SLAMA method. We have examined the global capacity under two conditions: 1) The displacement capacity at the beginning of the degrading portion of the plot. 2) The maximum displacement when all the steel in the walls has slipped, and all the walls are rocking. Both cases yield similar % NBS.	6/10/2023	Noted, both cases considered and yielded similar results. NFC		Closed
4	Wall lateral load distribution in transverse direction		17/07/2023	The building varies in height from one end to the other therefore the wall stiffness will vary along the building. How has this been accounted for the the push over in the transverse direction? How have the ADRS curves been generated given the change in building height?	27.09.23			Please confirm what the first mode, period and the effective heights are for the transverse direction. Is the ADRS curves sensitive to the effective assumed? 14/11/23: Give you note that the effective height is critical, can you clarify where your effective height of 6.6m is taken from? Equally how you calculated this. Refer sketch below..  6/12/23: Noted response considered pushover curves for both heightsand these resulted in similar results. NFC.	Please refer to page 53. The time period is less than 0.4s, and the effective height is 6.6m. Yes, the ADRS is sensitive to the assumed effective height, like all ADRS curves. 5/12/2023 The effective height is measured from the ground level and calculated using equation C2.8 in the guidelines. Additionally, we have analysed the Single Degree of Freedom (SDOF) structure, considering the lower ground floor as the reference level. This results in a different effective weight (12,200kN) and effective height (9.5m). Plotting this on the Acceleration-Displacement Response Spectrum (ADRS) curve, we obtain a similar %NBS when compared to the ADRS using the ground level as the reference level, i.e., 55%-60%NBS (IL2)	Closed
5	Foundation Sliding Capacity		17/07/2023	The base shear capacity is based on the combined passive pressure and base friction. However, the building is not uniform in profile (one end is 4 storeys & the other 5 storeys). How has this been accounted for as the shorter, stiffer end will attract more lateral load whereas the tall end contributes more to the weight, that is, friction? Do the retaining wall seismic loads contribute to the baseshear demands?	27.09.23	We acknowledge that the building is not uniform in profile, with one end having 4 storeys and the other 5 storeys. To account for this, we have incorporated this variation into our ETABS model. This enables us to represent the fact that the shorter and stiffer end will experience a greater elastic lateral load. We acknowledge that we have conducted a global sliding check rather than assessing the individual weight on each pad foundation and its resulting shear friction capacity. However, it's important to note that all the pads are interconnected with ground beams, which means that the foundation is likely to move as a single unit. In our opinion, the sliding of the structure can be beneficial as it increases the building's damping, increase the effective period of the structure and hence reduces the buildings accelerations. The sliding of the structure is not considered a life safety hazard. Regarding the retaining wall, seismic loads contribute to the base shear demands, and the presence of retaining walls may indeed increase these demands. However, may lead to the building sliding earlier in a design-level earthquake. Again, sliding will increase the building's damping, increase the effective period of the structure and hence reduces the buildings accelerations. This is considered advantageous in a design level earthqauke.	6/10/2023	Please confirm the assessment of the transverse walls allows for the potential for sliding. That is, if the central wall slides before it rocks then won't this change the current ADRS curve? 14/11/23: You mention that the foundation is suitably interlocked therefore the building will slide as a whole. Are you saying the floor slab and ground beams act as a diaphragm and have the capacity to do so? A quick look suggest the ground beam tie capacity may be critical. Refer below.  6/12/23: Noted, local sliding resistance is such that only minimal transfer is required. NFC.	The assessment of the transverse walls has taken into account the potential for sliding. It is important to note that the foundations are interconnected; thus, one wall cannot slide without dragging the other walls along with it. However, we have not regarded sliding as a limiting factor to control the shear demand on the wall. Limiting shear on the walls is deemed an unreliable and uncertain mechanism, as determining the appropriate overstrength factor to consider for the walls' sliding capacity poses challenges in assessing the hierarchy of strength. i.e. what overstrength factor would Beca consider on the walls sliding capacity to check the hierachary of strength? 5/12/2023 We have assessed each wall's shear capacity (based on the wall's flexural capacity) in the transverse direction against the sliding resistance (0.35 x Wall weight). Our calculations indicate that the walls possess a greater sliding resistance than their shear capacity. In some cases, an additional 20kN is required to prevent the wall from sliding. Upon inspection, it appears that the existing diaphragm can bear this additional weight. Therefore, we are not relying on the ground-level diaphragm for sliding. Please see the summary of calculations below 	Closed

6	Diaphragm Assessment		17/07/2023	In the transverse direction, wall H resists signiciantly more load than the other walls. I assume this is because it is at the step in the building height, however, it is still the full (taller) building height. Has the support (restraint) for this located at basement and ground level (then again, the ground level ground beams migh be providing the restraint at ground level)? Also, wall O appears to transfer its load at 2 floor level. Why? Is the diaphragm modelled as rigid or semi ridgid?	27.09.23	We acknowledge that, based on our calculations, Wall H carries a greater load than the other walls in the transverse direction. This is because Wall H is situated at the step location, and consequently, it functions as a vertically propped cantilever (by the upper and lower diaphragm), making it elastically stiffer than the other walls. In reality, we anticipate that the load will be evenly distributed among the walls in an ULS earthqauke. Wall O is not as wide and, therefore, not as stiff as the other walls. As a result, it accumulates load on level 3 and then transfers the load to level 2, where the other walls are significantly stiffer. The diaphragm is modeled as semi-rigid. We acknowledge that, in our opinion, during a design-level earthquake, the diaphragm will soften and behave as a flexible diaphragm, spanning between the numerous RC walls and limiting the transfer of forces.	6/10/2023	Is the building response sensitive to a whether or not the floor is rigid or semi-rigid (note, before it softens off it's initally rigid)? Wall H - refer to comments on sliding. Wall O response is interesting but likely to change once wal H rocks &/or slides. 14/11/23 You note that you modelled the diaphragm as semi-rigid. Did you do a quick displacement check to confirm that this was in fact the case? 6/12/23: noted, check carried out to confirm that the diaphragm is semi-rigid. NFC.	We have modeled the diaphragm as semi-rigid; therefore, this is the most 'realistic' diaphragm assumption. Please refer to the above in regard to sliding. Noted regarding Wall O. 5/12/2023. Yes, we conducted a quick displacement check to confirm that the diaphragm should be modeled as semi-rigid, i.e., the semi-rigid diaphragm displacements are more than double those of the rigid diaphragm.	Closed
7	Diaphragm Capacity		17/07/2023	The capacity of the connection of the slab to the walls is based on dowel action of the tie bars as it is assumed no scabbling undertaken. However, is it possible the slabs were cast across the top of the walls (quite common for insitu slabs), therefore is shear friction is possible?	27.09.23	We agree that it's possible that the slabs were cast across the top of the walls, and shear friction might be in play. However, this would simply contribute to the shear interface capacity. Since the diaphragms already achieve a 100% NBS score, this would only enhance the %NBS	6/10/2023	Noted, wall/floor connection scores greater than 100% based on concervative assumptions. NFC		Closed
8	Slab gravity monets		17/07/2023	It appears the slab FE's axis are not in alignment. Did you consider aligning them to make reading the results a little easier? In addition, a number of the panel elements do not node out with the neighbouring panel. Are the panels discontinuous across grid lines as the analysis implies'	27.09.23	We agree that some of the plate elements do not align with certain wall lines. However, the edge constraints in ETABs have been turned on, allowing the area objects to provide continuity as if the nodes were aligned. Therefore, the moment demand will be "correct."	6/10/2023	Just to clarfiy, have you checked the displacements along the panel edges to ensure they edge is continuous as believed? 14/11/23: NFC	Yes, we have checked the displacements along the panel edges to ensure that each edge is continuous as believed.	Closed
9	Slab reinforcing		17/07/2023	The slab has round bars. Have the longer lap lengths of slab reinforcing been accounted for assessing the slab tie capacity (that is, is continuity of the tie reinforcing provided)'	27.09.23	Yes, we have considered the longer lap lengths of the slab reinforcing when assessing the slab tie capacity. Please refer to pages 107-120 for details	6/10/2023	Noted, lap length for round bars considered. NFC		Closed
10	End wall analysis		17/07/2023	The elevation of the end walls indicate that the walls include beams and columns (has the presence of these been observed on site). Have these been included in the FEA of the end walls?	27.09.23	Yes, the beams and columns have been modelled in the end walls. The presence of these have not been observed on site	6/10/2023	Noted, beams and columns modeled as per drawings.		Closed
11	Block Out-of-plane score		17/07/2023	The out-of-plane capacity of the blockwork appears to be based on the walls being unreinforced but the wall elevation provided seems to indicate that it is reinforced (starter bars seem to be provided). Please confirm if it is or is not reinforced.	27.09.23	The calculations have assumed no reinforcement in the middle portions of the block walls. Consequently, we have considered arching action in accordance with the guidelines. The drawings indicate that there are two starter bars over a length of 3.6m and no reinforcement in the middle portions of the block walls. Our DSA suggests conducting site investigations to confirm the extent of the block walls	6/10/2023	Noted, DSA recommends site investigation work should be carried out to confirm extent of wall reinforcing. NFC.		Closed
12	Timber roof framing		17/07/2023	It is noted that there is no information on the timber roof connections therefore it has not been assessed. To complete the assessment of the roof structure you should consider the connections. Is a site investigation being proposed to confirm the timber connections?	27.09.23	As mentioned in our DSA report, further onsite investigations are required to confirm the timber connections. If this connections is found to have sufficient capacity to act as a tie for the walls OOP, it would increase the OOP %NBS of the wall	6/10/2023	Noted, DSA recommends site investigation work should be carried out to confirm extent of wall reinforcing. NFC.		Closed
										Open
										Open
1										Open
2										Open
3										Open
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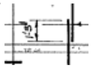
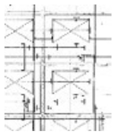
7										Open

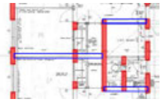
Sensitivity: General

Peer Review Register

JOB NAME: **Hanson Court Block B**
JOB NUMBER: **5275360**
ELEMENTS
DATE: **12/12/2023**
Reviewers

ISSUE DESCRIPTION

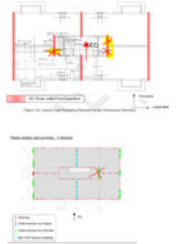
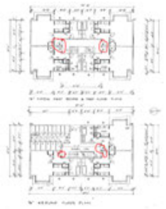
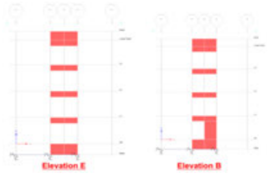
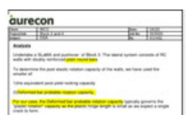
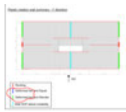
No.	ITEM / ELEMENT	Reference	Beca's Comments		Designer Respond		Closeout Comments		Designer Response	STATUS
			Date	Comment	Date	Comment	Date	Comment	Comment	
1	Reinforcing type	DSA calculation report	14/06/2023	The DSA report notes that the building has plain round bars. Given the year the building was constructed there is a chance it has deformed bars. Has an intrussive investigation been undertaken to confirm that the bars a round as noted in report?	18/07/2023	An intrusive investigation has been undertaken to confirm that the bars are round. Please refer to the picture below, which shows an example of the plain round bars. It is worth noting that since the majority of the main walls are governed by out-of-plane instability, whether the bars are deformed or plain becomes a moot point. 	26/07/2023	Noted, reinforcing type confirmed on site. No further comment (NFC)		Closed
2	Reinforcing lap length		14/06/2023	The calculations indicate a lap length of 450mm for the 9.5mm vert bars but the drawings show 1'-3" (380mm). Which is correct? 	18/07/2023	We agree that the 9.5mm vertical bars have a development length of 380mm. Based on this development length, the bars can achieve an allowable stress of 312MPa instead of the assumed 324MPa as stated in the Yellow Chapter. Since the difference in bar stress is within 5% of each other, there is no change in %NBS wall scores.	26/07/2023	Note 5% difference but this may be critical for scores close to 3. Please review & ensure change in allowable bar stress won't affect element scores close to these limits. 11/8/23 Noted, scores are not	We have once again reviewed the 5% difference in stress steel and have concluded that there is no change in the %NBS. The walls in the long direction score 45%, while the walls in the opposite direction score 60%. Therefore, we are not approaching the 34% and 67% limits. All other elements score 100%NBS.	Closed
3	Building Periods		14/06/2023	The first mode has a period of 1.8s but only 21% mass participation. This seems unusual. What is the deformed shape for this period?	18/07/2023	This is a torsional mode.	26/07/2023	Noted. NFC		Closed
4	Spandrels		14/06/2023	The calcs state that all spandrels have been cracked so they don't take any load. What type of cracking is being refered to as simple concrete cracking doesn't mean they can't take load. Further more, they have a similar detailing to the piers (and in many places more depth) therefore may have more capacity than the piers (mainly around the exterior). Confirm that the spandrels have been included in the analysis (they apper to on the images but their stiffness may have been set as zero) and over all wall capacity.  In addition, the spandrels are effectively deep beams therefore they may need to be assessed using strut and	18/07/2023	The spandrels were initially considered in terms of stiffness and strength. However, during our iterative process, it was discovered that the majority of the spandrels are shear-governed and therefore do not contr bute to the seismic resistance of the building. According to our calculations, the deep spandrels are expected to yield at less than 0.1% and reach their ultimate rotation at 0.4%. Consequently, the spandrels are not expected to contribute to the lateral resistance of the building during a design-level earthquake. It should be noted, however, that the spandrels' gravity carrying capacity is expected to be maintained. We agree that in some locations, the spandrels have more depth than the piers and hence a potential for a greater flexural capacity than the piers. However, for the lower level piers they have large compression loads on them which drives up their capacity and ensures the spandrels yield before the piers. At the higher levels, the piers may yield before the spandrels. However, we cannot form a column-sway mechanism because of the internal walls. Regarding the assessment of the spandrels using strut and tie, it would be inappropriate as the spandrels have plain round bars. Strut and tie analysis requires plasticity in the beams, which is not present in this case.	26/07/2023	Noted, spandrels yeild early and have low rotational limits. A strut & tie assessment of the wall is possible up to first yield but, given you are allowing some ductility in your push-over can appreciate this may not be suiatble. NFC		Closed
5	Seismic coefficient		14/06/2023	The seismic coefficient is based on a mu=1.5 & Sp=1.0. Why 1.5? And given you have used 1.5, why Sp=1.0 (though a rocking mechanism is indicated, the ADRS curves appear to be for a limited flexural response and you're treated as a medium energy dissipation (not medium to high) therefore an Sp=0.9 seems reasonable)?	18/07/2023	An Sp=1.0 was chosen since the majority of the walls are governed by out-of-plane (OOP) lateral instability, which is considered a brittle failure mode. Consequently, there is limited redundancy in the system once the walls reach their OOP lateral instability rotation. The level of redundancy in the system is an important factor to consider when deciding on the appropriate value of Sp. However, we have no objections to changing the value of Sp to 0.9. We have updated the wall calculations considering an Sp=0.9. This has resulted in no change in %NBS.	26/07/2023	Please clarfiy how a reduction in demands by 10% results in zero change to %NBS score. 11/8/23 Noted 10% change doesn't signifcantly affect ADRS curve's. NFC	The capacity curves intersect with the demand curves on the degrading portion of the plot; therefore, a 10% change does not significantly impact the %NBS due to the curve's non-linear nature. For example, in the Y-direction, utilizing Sp=1.0 results in a %NBS of 57%, which rounds to 60%NBS. Similarly, with Sp=0.9, the %NBS equals 62%, which also rounds to 60%NBS.	Closed

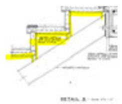
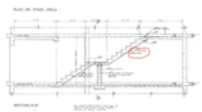
6	Wall flexural capacity		14/06/2023	When assessing the flexural capacity of the walls have the return flanges been taken into account? 	18/07/2023	The return flanges were initially considered in terms of stiffness and strength. However, during our iterative process, it was discovered that the majority of the wall-to-return-flange interfaces did not have sufficient shear friction capacity to allow the walls to act compositely. The horizontal reinforcement is insufficent to effective mobilisation of the flanges. We anticipate the formation of cracks at the wall-to-flange return interface, causing the walls to behave as individual rectangular sections during a design-level earthquake. It should be noted that in the ETABS model, gaps were introduced between the wall and return flanges to ensure they do not function as a single element. Therefore, the building's stiffness is based on rectangular walls rather than walls with return flanges.	26/07/2023	Noted, insufficient shear to allow composite action between perpendicular walls. NFC		Closed
7	Wall rotation limit		21/06/2023	The wall plastic rotation limits appear to be for simple cantilevers (typicall wall elevation shown with small coupling beams with minimal impact on wall performance) but the perimeter walls have more substancial coupling beams that will affect the wall response, plus the central longitudinal walls are not simple retangles How have these been assessed?	18/07/2023	The walls have been assessed as simple retangular cantilevers for the following reasons: 1) The majority of the spandrels are shear governed, meaning they do not contribute to the seismic resistance of the building. Additionally, it should be noted that once a spandrel beam cracks, there is no restoring component that forces this crack to close. 2)We anticipate the formation of cracks at the wall-to-return-wall interface, causing the walls to behave as individual rectangular sections. The interfaces between the walls and return flanges did not possess sufficient shear friction capacity to enable composite action. Furthermore, the horizontal reinforcement is insufficient to	26/07/2023	Did you consider shear hinging of the beams as per ASCE-SE1-41 (table 10-13)?k 11/8/23 Noted shear hinging considered but drift limit 0.3% therefore small. NFC	Yes, we have considered shear hinging of the beams following ASCE-SE1-41 (table 10-13). However, the table indicates a plastic rotation of only 0.3%. Consequently, this results in a probable rotation capacity for the typical spandrels of less than 0.5%. Anticipating a building drift of 1.1% in the transverse direction and 2.2% in the longitudinal direction, we expect the spandrels to experience a loss of lateral capacity well before the building achieves its ultimate limit state drifts.	Closed
8	Seismic Drifts		21/06/2023	1% drfits for a shear walled building at ground floor was high and I assume is due to foundation rotations. Has a sensitivity check been carried out for upper and lower values for the soil stiffness?	18/07/2023	We have conducted a sensitivity analysis by modifying the spring stiffness to 50% and 200% of the original spring stiffness. However, the dynamic properties of the building did not show significant changes under these modifications.	26/07/2023	Noted, sensity check for varying foundation stiffness carried out. NFC		Closed
9	Foundations		21/06/2023	The foundation bearing pressures are quite high in places. Have the foundations been checked to see if they can cope with these? Have +/- directions been considered? Have 45deg actions been consider (eg 100% / 30% case)?	18/07/2023	Please see attached calculations showing the +/- directions in both the x and y directions. Based on our calculations, the foundations still scores 100%NBS. In regards to the 100%/30% case, as we have a ducitlity greater than 1.25 then in accordacne with NZS1170.5 this load case does not need to be considered. We are satisifed that the +/- directions in both the x and y directions captures the behaviour of the foundations.	26/07/2023	Not quite. As noted in the commentary of NZS1150.5, the biaxial response is considered as part of the capacity design approach. That is, either design for the combined overstrength reactions on the foundations (allows for a earthquake not perpendicular to the building axis) or 100% 30% non-ductile load cases. Please review. 11/8/23 Updated foundation response for combined	Please refer to the attached document for the updated calculations regarding the foundations. These calculations consider 100%/30% load cases with mu=1.25 loading and Sp=0.9, utilizing an equivalent static force vector. Based on the revised calculations, the foundations still achieve a score of 100%NBS.	Closed
10	Diaphragm		21/06/2023	The FEA of the gravity demands on the floor plates have elements that don't node out along some wall lines This affects the plate continuity (moment demands) across the walls in these locations. Please review.	18/07/2023	We agree that some of the plate elements do not align with certain wall lines. However, the edge constraints in ETABs have been turned on, allowing the area objects to provide continuity as if the nodes were aligned. Therefore, the moment demand will be "correct.'	26/07/2023	Noted, floor gravity moments considered correct. NFC.		Closed
11	Floor Grillage Model		21/06/2023	Has a +/- review been carried in the grillage model for each direction? How was the load redistribution carried out (there appears to be a large jump in compression load between grids 3-1 & 16-18)? There still appears to be large tension demands between 1-2 & 17-18. How are these resisted? are the bars adequately anchored to resist these loads?	18/07/2023	We have not undertaken a +/- review in the grillage model for each direction. We are satified that the building is sufficently symmetrical that a +/- review will result in same %NBS (i.e a 100%NBS). Redistr bution was carried out by applying tension limits to the grillage tie elements. The large tension demands between 1-2 & 17-18 are resisted by the reinforcement in the slab. Please see attatched calculations that shows that the bars are adequately anchored to resist these loads	26/07/2023	Does the tension redistribution (due to bar yielding) account for the reinforcing strain limits? 11/8/23 Noted, bar strains reviewed and within acceptable limits. NFC	The tension redistr bution does not account for strain limits. However, in the Y-direction, we only require 15% redistribution and in the X-directions, we need 25% redistr bution. These values are below the acceptable force-based redistr bution limit. Furthermore, we have observed that the pESA methods utilized time periods of 0.8s in both directions, while the actual period of the buildings is approximately 1.5s. Therefore, our diaphragm analysis is considered conservative, and the diaphragm still achieves a score of 100%NBS. If we were to use the larger time periods, it is likely that no redistribution	Closed
12	Wall OOP capacity		21/06/2023	The wall parts loading seems high at 18,9kPa (parts coefficient = 2.0) for a 200mm thick wall. Could you confirm how this was calculated?	18/07/2023	We agree the 18.9kPa for a 200mm thick wall is wrong. The parts loading should be 0.2m x 25kN/m3 x 2.0g = 10kPa. As the wall OOP scores 100%NBS using 18.9kPa, then the wall OOP still scores 100%NBS using 10kPa	26/07/2023	Noted, Loads reviewed and updated. NFC		Closed
13	Masonry walls		21/06/2023	The URM walls are assessed as vertical spanning. Is there any benefit in considering them both veritical and horizontal spanning?	18/07/2023	The URM walls are expected to crack and collapse at loads below 34%NBS. However, considering the location of these walls, they are not considered a life safety concern.	26/07/2023	Walls not considered a life safety risk		Closed

Peer Review

JOB NAME	Hanson Court Blocks C and D.
JOB NUMBER	5275360
ELEMENTS	
DATE	12/12/2023
Reviewers	

ISSUE DESCRIPTION

No.	ITEM / ELEMENT	Reference	Beca's Comments		Designer Respond		Closeout Comments		STATUS
			Date	Comment	Date	Comment	Date	Comment	
1	General Comment.								
1		Calcs page 27 and 28	23.06.2023	There is no 150mm RC concrete shear walls in transverse direction located each side of stairs above 1st floor level. There is only short length of 200mm RC shear wall at Ground floor level. Refer to architectural and structural drawing. Please review the assessment.  	13.07.23	The image displayed corresponds to the ground floor. We acknowledge that there are no shear walls on Grid E, and only a short shear wall on Grid B. These factors have been considered in our assessment. Please refer to the snippets from our ETABS model for further reference. No assessment review required as this has been taken into consideration. 	31.07.2023	Noted	Closed
2		Calcs page 42 and 44	23.06.2023 31.07.2023	It's stated that lateral system consists of RC walls with 2 layers of plain round bars. However, deformed bars in regards of rotation capacity are mentioned on the same page. Please clarify this matter and confirm which bars, plain or deformed were used for assessment. Please review calculations as required   Noted that "the walls do contain plain round bars" Please amend "deformrd bar" to "plain bar" on the ADRS summary page for both directions X &Y 	13.07.23 27.09.23	The walls do contain plain round bars. However, in accordance with the Guidelines, to determine the plastic rotation capacity to smaller of the following needs to be considered: 1) Rocking Plastic Rotation, θr 2) Deformed Bars Plastic Rotation, θp 3) The onset of OOP wall lateral instability, θp No calculation review required. The walls do contain plain round bars; however, in accordance with the guidelines, the smaller value between the rocking plastic capacity, deformed bars plastic capacity and the Out-of-Plane stability plastic rotation is used to determine the plastic rotation capacity of the wall. For instance, if a plain round bar wall has a rocking plastic capacity of 5.0% (using C5.40) and a deformed plastic capacity of 1.0% (C5.41), then the plastic rotation capacity of the wall is considered as 1.0%.	12.10.23	Noted	Closed
3	RC shear walls	Calcs page 16 and 38	23.06.2023	The lap length of existing plain bars is Ld prov=425mm and demand- Ld=1013mm or fy, splice=227MPa as it's shown on page 16. fy=324MPa was used for the assessment. On page 38 was mentioned that assessment and %NBS is based on development length of plain bars. Please clarify how %NBS was determined	13.07.23	The %NBS of the lateral system was determined using the ADRS method. For walls that did not have sufficient development lengths, their steel stress was reduced to match the allowable steel stress specified in the Guidelines. For walls that had sufficient development lengths, their steel stress = 324MPa.	31.07.2023	Noted	Closed
4			23.06.2023	Please clarify the reason of using plain bars for assessment? Bars are not clearly denoted on drawings as plain or deformed and also no specification was provided to us for confirmation. Plain and deformed bars could be used for design in mid 60 in accordance with ClC5B.1 and Table C5.B1 of the Guidelines C5 "Yellow". Given the year the building was constructed there is a chance it has deformed bars. Has an intrusive investigation been undertaken to confirm that the bars a round as noted in report? Please clarify this matter.	13.07.23	An intrusive investigation has been undertaken to confirm that the bars are round. Please refer to the picture below, which shows an example of the plain round bars. It is worth noting that since the majority of the walls are governed by out-of-plane instability, whether the bars are deformed or plain becomes a moot point. 		Noted	Closed

5	RC floor diaphragm(s)		23.06.2023	Please confirm ductility used for assessment of capacity of connection details of floor diaphragm to shear wall.	13.07.23	The diaphragm demands were calculated using the pESA method, considering an Equivalent Static Analysis (ESA) vector with ductility factor mu = 1.25. Consequently, the connections are assessed considred a ductility factor of 1.25		Noted	Closed
6	General Comment.		23.06.2023	We suggest to clarify in the report and calculation set that Block 2 is indicated as "Block D" and Block 4 -as "Block C". Currently, it's not very clear.	13.07.23	Noted.		Noted	Closed
7	Connection detail shear wall to foundation		23.06.2023	Was shear friction capacity of connection detail shear wall to foundation assessed to be able to transfer the loads? What is %NBS?	13.07.23	Yes, the shear friction capacity of the connection detail between the shear wall and foundation was assessed. A friction coefficient of 0.6 was applied in calculating the shear friction capacity for the shear wall-to-foundation connection. The overall shear capacity of the walls was determined by taking the minimum value between the shear capacity specified in the Yellow Book and the shear friction capacity. However, based on our calculations, the flexural capacity of the walls was found to govern over the walls shear capacity and shear friction capacity.		Noted	Closed
8	Stairs		23.06.2023 31.07.2023	Steel stringers are 5"x2.5" RHS 11.79lbs. We comment as followings 1- this is equivalent to 127x64 RHS 2 - weight of the section is indicated as 11.79lbs. This is 11.79lbs per foot and equivalent to 17.6kg/m 3 - in accordance with the data presented in the Table (AISC) it'll be 127x64x6.3mm not 2mm as used for assessment (pp.119-123) 4 - Please review the assessment of stringer capacity and %NBS  1 - Please confirm on site thickness of stair stringer and amend calculation accordingly 2 - Please review calculations of stairs using "horizontal truss" method as discussed. 3 - Please update %*NBS* score of the structure accordingly	13.07.23 27.09.23	As discussed in our DSA report, there is uncertainty in the thickness of the stair stringer and onsite investigations is required to confirm this thickness. We agree that the stairs %NBS would increase if the stairs steel thickness was 6mm. We agree that the stairs thickness is likely to be 6.0mm. Based on 6.0mm thickness, the stairs scores 100%NBS. We will update the %NBS score in the DSA report to 100%NBS subject to onsite investigations.	12.10.23	Noted. Stair score %NBS has to be updated in the report	Closed
9	Stairs		23.06.2023 31.07.2023	RHS stringer was assessed as a single element (beam). However, there are vertical and horizontally located steel plates approx. 9 mm thickness (3/8"x2'1/2" wide) welded to each RHS stringer to supports concrete steps. There are also 2RHS at mid-landing level. 2-RHS stringers and steel plates are acting as a horizontal truss under lateral earthquake loads. Please review the assessment and %NBS of stair structure  Please refer to comments Item 8, dated on 02.08.2023	13.07.23 27.09.23	As discussed in our DSA report, there is uncertainty in the thickness of the stair stringer and onsite investigations is required to confirm this thickness. We agree that the stairs %NBS would increase if the stairs thickness was 6mm. Refer to comment 8.	27.09.23	Noted. Stair score %NBS has to be updated in the report	Closed
10	Stairs Ground Level/1st Floor Level	dwg S139/11 calcs page 117	23.06.2023	We note there is no top reinforcing at the mid-landing. Has this been considered in the assessment of the stair given negative moments could develop here. Is stair's structure able to accommodate the displacement of the main structure?  	13.07.23	Based on our observations of the existing drawings, there is top reinforcement at the mid-landing. This reinforcement has sufficient capacity to resist the stairs negative moment. Yes, the stairs can accommodate the displacement of the main structure. This is at the ground level where the buildings drift is smallest under a design level earthquake.		Noted	Closed

11	Stairs	calcs p.120 and the assessment inputs Appendix B	23.06.2023	Please clarify how fy=264 MPa for assessing steel stringer was determined? Probable strength fy=345 Mpa is indicated for structural steel. Please review the calculations and update %NBS Table: Material properties Item / Characteristic Design Strength (MPa) / Assessment (Probable) Strength (MPa) Reinforcing Steel – Beams / 275 MPa / 324 MPa Concrete / 20 MPa / 30 MPa Structural Steel / 300 MPa / 345 MPa	13.07.23	The hollow section was assumed to have a fy= 250MPa and therefore the probable strength fy,p = 250 x 1.1 = 270MPa (this is within 5% of 264MPa). We will update our DSA report showing 250MPa and 270MPa. No %NBS update is required until onsite investigations is undertake to confirm the stairs steel thickness.		Noted	Closed
12	Non-structural		23.06.2023 31.07.2023	The image capture is from Google Map image dated June 2019. Please confirm if life safety issue might be caused.  This is from Google Maps. Please review and clarify this matter. Is there any life safety risk?	13.07.23 27.09.23	From the existing drawings and our site investigations no chimney was observed. Can Beca please clarify where they obtained this photo from? Our understanding is that this is not a chimney but instead is a light-weight roof vent. As it is light-weight this is not considered a life-safety hazard. We will add to our DSA report that further investigations is required to confirm the roof vent material.	27.09.23	Noted. DSA report to be updated and note added that further investigations is required to confirm the roof vent material.	Closed
13	Secondary and Non-structural		23.06.2023	Are any services located in the roof space should be assessed and restrained?	13.07.23	From our onsite investigations, we could not get access to the roof space and therefore could not determine if there is any services to be restrained in the roof.	31.07.2023	Closed with action subject to this matter highlighted in the DSA report and noted that additional investigation will be required to confirm the existance, condition and bracing of the existing services.	Closed
14	Shear walls	page 37 calcs	23.06.2023 31.07.2023	Pleas clarify the followings: 1 - the choice of 11% damping in the ADRS curve? Specifically, considering the presence of round bars in the walls and the limited impact of ductility? 2 - what modal participation factor and the modal mass coefficient are utilized in the ADRS Curve? Ductility mu=1.25 was used for the assessment. The hysteretic damping =3 for Concrete wall structural system (Medium). Assuming the inherent damping=5, 5+3=8 not 11. Please clarify this matter. Refer Table C2D.1 Guidelines	13.07.23 27.09.23	The damping values for the ADRS can be found on page 51 for the Y-direction and page 57 for the X-direction. The hysteretic damping is taken from Table C2D.1 in the guidelines. Median damping is considered to account for the expected plain round hysteretic shape, resulting in a total damping range between 5% and 10%. The modal participation factor for each primary mode exceeds 60%, and the modal mass coefficient is 0.83, as stated in the ADRS calculations. We are confused. Our calculates on page 51, shows the damping to equal 8% not 11%.		Noted	Closed

15	Shear walls		23.06.2023	What failure mechanism of RC shear walls is- flexure or shear? Please clarify and provide reference to calculation pages to confirm shear capacity of RC walls	13.07.23	The walls are flexurally governed. Please see attached for the capacity calculations.		Noted	Closed
16	North and South Shear walls.		23.06.2023	Capacity of wall out of plane. 200mm thickness wall with 2 layers of REO in both directions is supported (restrained) at 3 sides -by external wall and internal RC walls and RC floor. Was it taken into consideration? Please confirm. Please confirm coeff. phi used for the assessment of flexural and shear capacity of the wall	13.07.23	The 200mm thick wall, reinforced with 2 layers of REO in both directions, and supported on three sides (external wall, internal RC walls, and RC floor), has been taken into consideration for our out-of-plane (OOP) parts assessments. Shear strength is considered with a phi value of 0.85, while flexural strength is considered with a phi value of 1.0. These values align with the Guidelines.		Noted. Strength reduction factor phi=1 should be used for flexure or shear. Refer to C5.5.1.4 Guidelines	Closed
17	Internal Shear wall in Longitudinal direction.		23.06.2023 12.10.2023	Wall REO is 10mm DIA @230mm crs both ways. Was it taken into account that the wall is restrained at RC floo at 3d floor level and by external RC wall (2 way supported). Was it also considered that wall is partially supported by ceiling structure and by timber purlins @ approx. 900mm crs at the top level? A proportion of the lateral load imposed by the roof structure will be transmitted to the RC external perimeter wall, which in turn redistributes the force back to the internal wall at the timber ceiling level. Please confirm Coeff. phi used for the assessment of flexural and shear capacity of the wall. Please clarify the model used to assess the wall capacity-was it supported on 1 side only? Please clarify this matters, review calculations and update %NBS We reviewed the OOP of the longitudinal wall currently scoring 25% and discussed this internally and wonder if a few more investigations could confirm the life safety score for this item. Could you consider the following: 1.Investigate whether there is a lap length at the floor level. If there is no lap in the plastic hinge, could potentially consider ductility mu > 1 (e.g. mu = 2). and/or 2.Reviewing the score regarding its life safety risk by confirming the connection between diaphragm and wall. If a good connection is confirmed between diaphragm and wall and then review whether the life-safety risk is present.Wall should be checked as supported at floor level and restrained by external concrete wall on one side only 3. Undertake on-site investigation to assess the capacity of the roof and ceiling structure and their connection details to RC internal and external walls structure.	13.07.23 Copy of respond from Aurecon- refer Email from Aurecon received on 25/10/23	Our assessment considered that the wall is restrained at the RC floor at the 3rd-floor level and supported by an external RC wall (two-way support). We also considered that the wall is partially supported by the ceiling structure and timber purlins, spaced at approximately 900mm intervals at the top level. However, the connection between the RC wall and timber purlins is unknown. Therefore, the ceiling structure was not relied upon in assessing the wall's out-of-plane (OOP) behaviour. As mentioned in our DSA report, further onsite investigations are required to confirm the wall-to-ceiling connection. If this connection is found to have sufficient capacity to act as a tie, it would increase the OOP %NBS of the wall. Shear strength is considered with a phi value of 0.85, while flexural strength is considered with a phi value of 1.0. These values align with the Guidelines. Based on the above, there will be no change in the %NBS until further onsite investigation are undertaken. 1 - Based on the above, there will be no change in the %NBS until further onsite investigations are undertaken. We have re-examined the structural drawings, and they indicate a lap joint (see below) where we anticipate the maximum moment in the wall due to out-of-plane loading. Considering the lap's location, achieving a ductility greater than 1.25 seems unlikely. Additionally, it's worth noting, as outlined in the guidelines, that experimental testing has demonstrated that straight plain bar laps are prone to failure before the bar yields, even when the lap length theoretically provides enough support to develop the bar's probable yield strength. This failure occurs due to the loss of chemical bond caused by the plain bar contracting due to the Poisson effect. Consequently, even if the lap meets the necessary length, the wall won't retain its moment capacity; instead, the mc capacity will degrade once the capacity is exceeded.	03.11.2023	Noted	Closed
18	Foundations		23.06.2023	Please clarify Sp factor used to determine loads acting on foundations	13.07.23	Sp =0.9 was used for the foundations.		Noted. Sp=1 should be used for design, however Sp=0.9 is accepted for assessment in this particular case due to Foundations been assesed to achieve score >100%NBS	Closed
17 Cont.	Internal Shear wall in Longitudinal direction.			Queries dated 12.10.2023 -See above	Copy of respond from Aurecon- refer Email from Aurecon received on 25/10/23	2 - We believe that the walls pose a life safety hazard even if there is a "good" connection between the diaphragm and wall. We highlight, that the 150mm thick walls effectively cantilevers 4.7m with some restraint from the side walls. If the walls' capacity is exceeded due to out-of-plane loading, and the earthquake changes direction, requiring the walls to resist in-plane loading, there is no lateral stiffness or strength left to counter the in-plane forces. This lack of resistance causes the roof to become unstable, leading to excessive displacements. These displacements can result in the roof losing support, creating a life safety hazard. Additionally, if the walls yield out-of-plane, it compromises the roof's torsional resistance, potentially making the roof unstable. We've also re-examined the walls supported at the floor level, restrained by an external concrete wall on one side only. Based on our calculations, the walls score less than 34%NBS. Furthermore, using yield line theory, our non-conservative evaluation also yielded a score less than 34%NBS. 3 - We agree that onsite investigations are necessary. This recommendation was included in our DSA report, and we have emphasized it consistently throughout the peer review process. We have been in discussions with the client, and we are currently confirming the presence of asbestos in the ceiling before proceeding with the onsite investigations. Thes investigations will establish the connection between the wall and ceiling. Additionally, w	03.11.2023	Noted	Closed

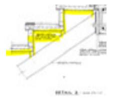
Beca

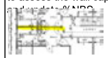
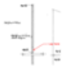
Peer Review DSA

JOB NAME: **Hanson Court Block E.**
JOB NUMBER: **5275360**
ELEMENTS
DATE: **12/12/2023**
Reviewers

ISSUE DESCRIPTION

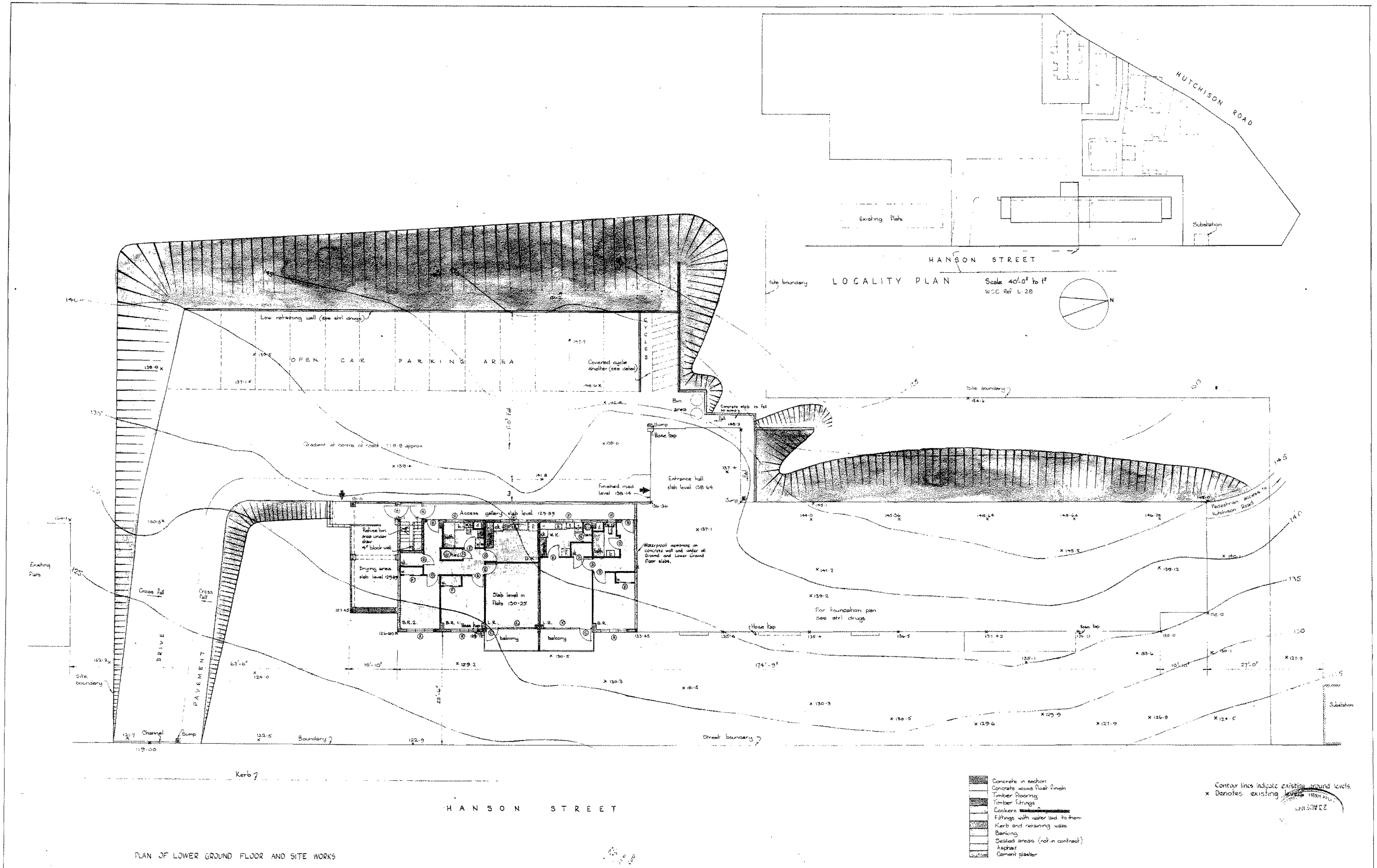
No.	ITEM / ELEMENT	Reference	Beca's Comments Date	Beca's Comments Comment	Designer Respond Date	Designer Respond Comment	Closeout Comments Date	Closeout Comments Comment	STATUS
1	General Comment.								
1		Calcs page 45	23.06.2023	There is no 150mm RC concrete shear walls in transverse direction located on left (south) side of stairs above 1st floor level. Refer to architectural and structural drawing. Please review the assessment as required. 		We acknowledge that there are no 150mm RC concrete shear walls in transverse direction located on left (south) side of stairs above 1st floor level. These factors have been considered in our assessment. Please refer to the snippets from our ETABS model for further reference. No assessment review required as this has been taken into consideration. 	02.08.2023	Noted. It was stated on page "RC Walls Summary" of the updated calculations received on 18.07.2023 that "shear capacity at the splice locations is expected to be exceeded at 40%ULS loading"	Closed
2		Calcs page 44 and 48	23.06.2023 31.07.2023	It's stated that lateral system consists of RC walls with 2 layers of plain round bars. However, deformed bars in regards of rotation capacity are mentioned on the same page. Please clarify this matter and confirm which bars, plain or deformed were used for assessment. Please review calculations as required Noted that "the walls do contain plain round bars" Please amend "deformrd bar" to "plain bar" on the ADRS summary page for both directions X &YHowever, 	27.09.23	The walls do contain plain round bars. However, in accordance with the Guidelines, to determine the plastic rotation capacity to smaller of the following needs to be considered: 1) Rocking Plastic Rotation, θr 2) Deformed Bars Plastic Rotation, θp 3) The onset of OOP wall lateral instability, θp No calculation review required. The walls do contain plain round bars; however, in accordance with the guidelines, the smaller value between the rocking plastic capacity, deformed bars plastic capacity and the Out-of-Plane stability plastic rotation is used to determine the plastic rotation capacity of the wall. For instance, if a plain round bar wall has a rocking plastic capacity of 5.0% (using C5.40) and a deformed plastic capacity of 1.0%	12.10.2023	Noted	Closed
3			23.06.2023	Please clarify the reason of using plain bars for assessment? Bars are not clearly denoted on drawings as plain or deformed and also no specification was provided to us for confirmation. Plain and deformed bars could be used for design in mid 60 in accordance with ClC5B.1 and Table C5.B1 ofthe Guidelines C5 "Yellow". Please clarify this matter.		An intrusive investigation has been undertaken to confirm that the bars are round. Please refer to the picture below, which shows an example of the plain round bars. It is worth noting that since the majority of the walls are governed by out-of-plane instability, whether the bars are deformed or plain becomes a moot point. 	02.08.2023	Noted	Closed
4	RC floor diaphragm(s)		23.06.2023	Please confirm ductility used for assessment of capacity of connection details of floor diaphragm to shear wall.		The diaphragm demands were calculated using the pESA method, considering an Equivalent Static Analysis (ESA) vector with ductility factor mu = 1.25. Consequently, the connections are assessed considred a ductility factor of 1.25.	02.08.2023	Noted	Closed
5	Connection detail shear wall to foundation		23.06.2023	Was shear friction capacity of connection detail shear wall to foundation assessed to be able to transfer the loads? What is %NBS?		Yes, the shear friction capacity of the connection detail between the shear wall and foundation was assessed. A friction coefficient of 0.6 was applied in calculating the shear friction capacity for the shear wall-to-foundation connection. The overall shear capacity of the walls was determined by taking the minimum value between the shear capacity specified in the Yellow Book and the shear friction capacity. However, based on our calculations, the flexural capacity of the walls was found to govern over the walls shear capacity and shear friction	02.08.2023	Noted	Closed
6	Stairs		23.06.2023 28.07.2023	Steel stringers are 5"x2.5" RHS 11.79 bs. We comment as followings 1- this is equivalent to 127x64 RHS 2 - weight of the section is indicated as 11.79lbs. This is 11.79lbs per foot and equivalent to 17.6kg/m 3 - in accordance with the data presented in the Table (AISC) it'll be 127x64x6.3mm not 2mm as used for assessment (pp.119-123) 4 - Please review the assessment of stringer capacity and %NBS  1 - Please confirm on site thickness of stair stringer and amend calculation accordingly 2 - Please review calculations of stairs using "horizontal truss" method as discussed.	27.09.23	As discussed in our DSA report, there is uncertainty in the thickness of the stair stringer and onsite investigations is required to confirm this thickness. We agree that the stairs %NBS would increase if the stairs steel thickness was 6mm. We agree that the stairs thickness is likely to be 6.0mm. Based on 6.0mm thickness, the stairs scores 100%NBS. We will update the %NBS score in the DSA report to 100%NBS subject to onsite investigations.	12.10.2023	Noted. Stair score %NBS has to be updated in the report	Closed

7	Stairs	calcs page 115-120	23.06.2023 31.07.2023	RHS stringer was assessed as a single element (beam). However, there are vertical and horizontaly local steel plates approx 9 mm thickness (3/8"x2'1/2" wide) welded to each RHS stringer to supports concrete steps. There are also 2RHS at mid-landing level. 2-RHS stringers and steel plates are acting as a horizontal truss under lateral earthquake loads. Please review the assessment and %NBS of stair structure  Please refer to comments Item 6, dated on 02.08.2023	27.09.23	As discussed in our DSA report, there is uncertainty in the thickness of the stair stringer and onsite investigations is required to confirm this thickness. We agree that the stairs %NBS would increase if the stairs steel thickness was 6mm. Refer to comment 6.	12.10.2023	Noted. Stair score %NBS has to be updated i the report	Closed
8	Stairs Ground Level/1st Floor Level	dwg S139/11 calcs page 115-120	23.06.2023	We note there is no top reinforcing at the mid-landing. Has this been considered in the assessment of the stair given negative moments could develop here. Is stair's structure able to accomodate the displaceme t of the main structure?  		Based on our observations of the existing drawings, there is top reinforcement at the mid-landing. This reinforcement has sufficient capacity to resist the stairs negative moment. Yes, the stairs can accommodate the displacement of the main structure. This is at the ground level where the buildings drift is smallest under a design level earthquake.	02.08.2023	Noted	Closed
9	Stairs	calcs p.calcs page 118	23.06.2023 31.07.2023	Please clarify how fy=264 MPa for assessing steel stringer was determined? Probable strength fy=345 Mpa is indicated for structural steel. Please review thecalculations and update %NBS  Noted. OK, Please update %NBSowever %NBS score after on site investion. Feref also to comments Item 6 and 7	27.09.23	The hollow section was assumed to have a fy= 250MPa and therefore the probable strength fy,p = 250 x 1.1 = 270MPa (this is within 5% of 264MPa). We will update our DSA report showing 250MPa and 270MPa. No %NBS update is required until onsite investigations is undertake to confirm the stairs steel thickness. Refer to comment 6.	12.10.2023	Noted. Stair score %NBS has to be updated i the report	Closed
10	Non-structural		23.06.2023 31.07.2023	There is a structure located above the top of the roof of Block E (3) and it looks like a chimney. Was the assessment of this structure carried out? Is it brick or masonry? Please clarify the structure and provide %NBS Photo of Block 4 - similar to Block   The image capture is from Google Map image dated June 2019. Please confirm if life safety issue might be caused	27.09.23	From the existing drawings and our site investigations no chimney was observed. Can Beca please clarify where they obtained this photo from? Our understanding is that this is not a chimney but instead is a light-weight roof vent. As it is light-weight this is not considered a life-safety hazard. We will add to our DSA report that further investigations is required to confirm the roof vent material.	12.10.2023	Noted. DSA report to be updated and note added that further investigations is required to confirm the roof vent material.	Closed

11	Non-structural		23.06.2023	Are any services located in the roof space should be assessed and restrained?		From our onsite investigations, we could not get access to the roof space and therefore could not determine if there is any services to be restrained in the roof.	02.08.2023	Closed with action subject to this matter highlighted in the DSA report and noted that additional investigation will be required to confirm the existance, condition and bracing of the existing services	Closed
12	Shear walls	page 59 calcs	23.06.2023	Pleas clarify the followings: 1 - the choice of 7% damping in the ADRS curve? Specifically, considering the presence of round bars in the walls and the limited impact of ductility? 2 - what modal participation factor and the modal mass coefficient are utilized in the ADRS Curve?		The hysteretic damping is taken from Table C2D.1 in the guidelines. Median damping is considered to account for the expected plain round hysteretic shape, resulting in a total damping range between 5% and 10%. The modal participation factor for each primary mode exceeds 60%, and the modal mass coefficient is 0.83, as stated in the ADRS calculations	02.08.2023	Noted	Closed
13	Shear walls		23.06.2023	What failure mechanism of RC shear walls is- flexure or shear? Please clarify and provide reference to calculation pages to confirm shear capacity of RC wal		The walls are flexurally governed. Please see attached for the capacity calculations.	02.08.2023	Noted	Closed
14	North and South Shear walls.		23.06.2023	Capacity of wall out of plane. 200mm thickness wall with 2 layers of REO in both directions is supported (restrained) at 3 sides -by external wall and internal RC walls and RC floor. Was it taken into consideratio Please confirm. Please confirm coeff. phi used for the assessment of flexural and shear capacity of the wall.		The 200mm thick wall, reinforced with 2 layers of REO in both directions, and supported on three sides (external wall, internal RC walls and RC floor), has been taken into consideration for our out-of-plane (OOP) parts assessments. Shear strength is considered with a phi value of 0.85, while flexural strength is considered with a phi value of 1.0. These values align with the Guidelines	02.08.2023	Noted. Strength reduction factor phi=1 should be used for flexure or shear. Refer to C5.5.1.4 Guidelines	Closed
15	Southern Internal Shear wall in Longitudinal direction.		23.06.2023	Wall REO is 10mm DIA @230mm crs both ways. Was it taken into account that the wall is restrained at RC floo at 3d floor level and by external RC wall and by (2 way supported). Was it also considered that wall is partially supported by ceiling structure and by timber purlins @ aprox 900mm crs at at the top level? A proportion of the lateral load imposed by the roof structure will be transmitted to the RC external perimeter wall, which in turn redistr butes the force back to the internal wall at the timber ceiling level. Please confirm coeff. phi used for the assessment of flexural and shear capacity of the wall. Please clarify the model used to assess the wall capacity-was it supported on 1 side only? Please clarify this matters, review calculations and update %NBS 		Our assessment considered that the wall is restrained at the RC floor at the 3rd-floor level and supported by an external RC wall (two-way support). We also considered that the wall is partially supported by the ceiling structure and timber purlins, spaced at approximately 900mm inter at the top level. However, the connection between the RC wall and timber purlins is unknown. Therefore, the ceiling structure was not relied upon in assessing the wall's out-of-plane (OOP) behaviour. As mentioned in our DSA report, further onsite investigations are required to confirm the wall-to-ceiling connection. If this connection is found to have sufficient capacity to act as a tie, it would increase the OOP %NBS of the wall. Shear strength is considered with a phi value of 0.85, while flexural strength is considered with a phi value of 1.0. These values align with the Guidelines.		Noted	Closed
			12.10.2023	We reviewed the OOP of the longitudinal wall currently scoring 25% and discussed this internally and wonder if a few more investigations could confirm the life safety score for this item. Could you consider the following: 1.- Investigate whether there is a lap length at the floor level. If there is no lap in the plastic hinge, could potentially consider ductility mu > 1 (e.g. mu = 2). and/or 2.- Reviewing the score regarding its life safety risk by confirming the connection between diaphragm and wall. If a good connection is confirmed between diaphragm and wall and then review whether the life safety risk is present.Wall should be checked as supported at floor level and restrained by external concrete wall on one side only 3. - Undertake on-site investigation to assess the capacity of the roof and ceiling structure and their connection details to RC internal and external walls structure.	Copy of respond from Aurecon- refer Email from Aurecon received on 25/10/23	1 - Based on the above, there will be no change in the %NBS until further onsite investigations are undertaken. We have re-examined the structural drawings, and they indicate a lap joint (see below) where we anticipate the maximum moment in the wall due to out-of-plane loading. Considering the lap's location, achieving a ductility greater than 1.25 seems unl kely. Additionally, it's worth noting, as outlined in the guidelines, that experimental testing has demonstrated that straight plain bar laps are pr to failure before the bar yields, even when the lap length theoretically provides enough support to develop the bar's probable yield strength. This failure occurs due to the loss of chemical bond caused by the plain bar contracting due to the Poisson effect. Consequently, even if the lap meets the necessary length, the wall won't retain its moment capacity; instead, the moment capacity will degrade once the capacity is exceeded. 	03.11.2023		
16		page 37 calcs	23.06.2023	Please clarify the reason of using Sp=1 for mu=1.25 for the assessment of Block 3 and Sp=0.9, mu=1.25 Block 2 and 4? Please review and update calculations and the %NBD accordinc	-	Both Block 3 and Block 2 and 4 used a Sp=0.9. Please refer to the ADRS calculations showing Sp=0.9. No change in %NBS.	02.08.2023	Noted	Closed
17	Foundations		23.06.2023	Please clarify Sp factor used to determine loads acting on foundations		Sp =0.9 was used for the foundations.	02.08.2023	Noted. Sp=1 should be used for design, however Sp=0.9 is accepted for assessment in this particular case due to Foundations been assesed to achieve score >100%NBS	Closed
15 Contin.	Southern Internal Shear wall in Longitudinal direction.			Queries dated 12.10.2023 -See above	Copy of respond from Aurecon- refer Email from Aurecon received on 25/10/23	2 - We believe that the walls pose a life safety hazard even if there is a "good" connection between the diaphragm and wall. We highlight, that the 150mm thick walls effectively cantilevers 4.7m with some restraint from the side walls. If the walls' capacity is exceeded due to out-of-plane loading, and the earthquake changes direction, requiring the walls to resist in-plane loading, there is no lateral stiffness or strength left to counter the in-plane forces. This lack of resistance causes the roof to become unstable, leading to excessive displacements. These displacements can result in the roof losing support, creating a life safety hazard. Additionally, if the walls yield out-of-plane, it compromises the roof's torsional resistance, potentially making the roof unstable. We've also re-examined the walls supported at the floor level, restrained by an external concrete wall on one side only. Based on our calculations, the walls score less than 34%NBS. Furthermore, using yield line theory, our non-conservative evaluation also yielded a score less than 34%NBS. 3 - We agree that onsite investigations are necessary. This recommendation was included in our DSA report, and we have emphasized it consistently throughout the peer review process. We have been in discussions with the client, and we are currently confirming the presence of asbestos in the ceiling before proceeding with the onsite investigations. These investigations will establish the connection	03.11.2023	Noted	Closed

G

Appendix G – Existing Drawings



WELLINGTON CITY CORPORATION
CITY ENGINEER'S DEPARTMENT
ARCHITECTURAL BRANCH

HANSON STREET FLATS DEVELOPMENT - BLOCK 1

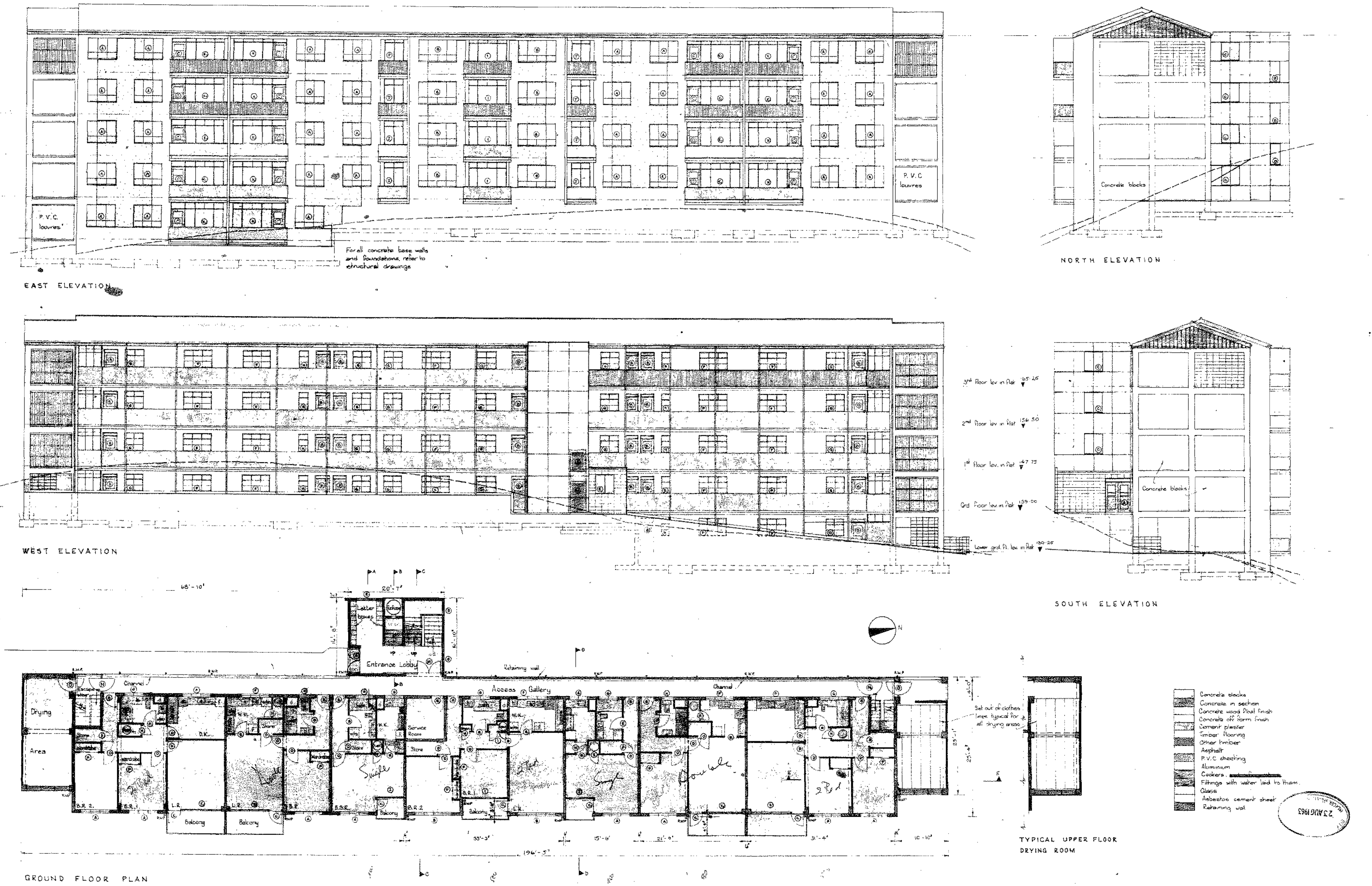
SITE PLAN 1/8" to 1'-0"

CONTRACT No 2145

TRACING No AL/33/1

Designed	M.C.
Drawn	C.J.F.
Traced	C.J.F.
Checked	
Approved	

G. I. B. Thomas M.N.Z.I.E.
City Engineer Wellington N.Z.



WELLINGTON CITY CORPORATION
CITY ENGINEER'S DEPARTMENT
ARCHITECTURAL BRANCH

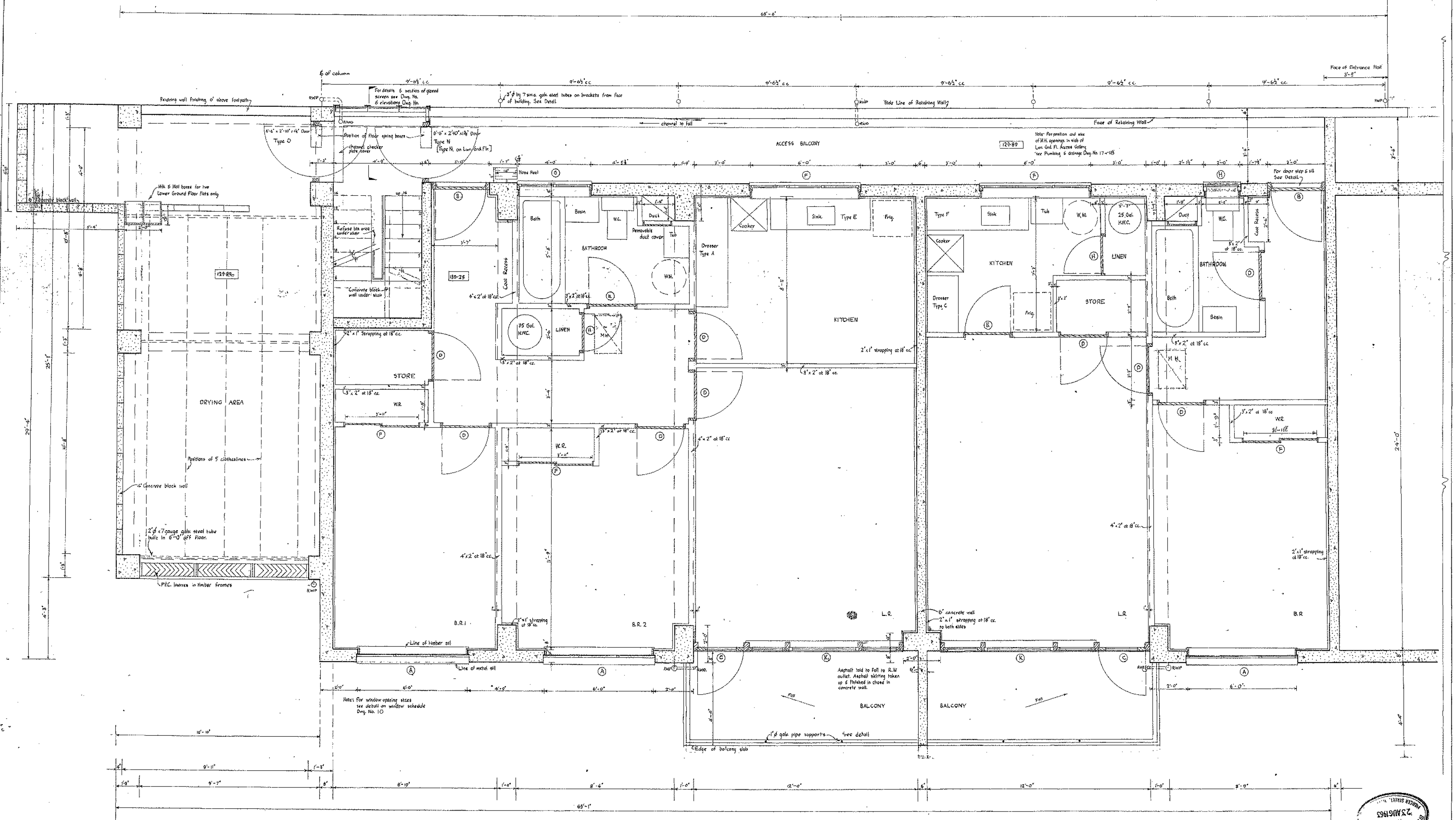
HANSON STREET FLATS DEVELOPMENT - BLOCK I

1/8" Scale PLANS AND ELEVATIONS

CONTRACT No 2145

TRACING No AL/33/2		
Designed	M.L.	
Drawn	C.J.F.	April 1963
Traced	C.J.F.	
Checked	[illegible]	
Approved		

G. I. B. Thomas MNZIE.
City Engineer Wellington NZ



LOWER GROUND FLOOR PLAN

Scale: 1/4" = 1'-0"

WELLINGTON CITY CORPORATION
CITY ENGINEER'S DEPARTMENT
ARCHITECTURAL BRANCH

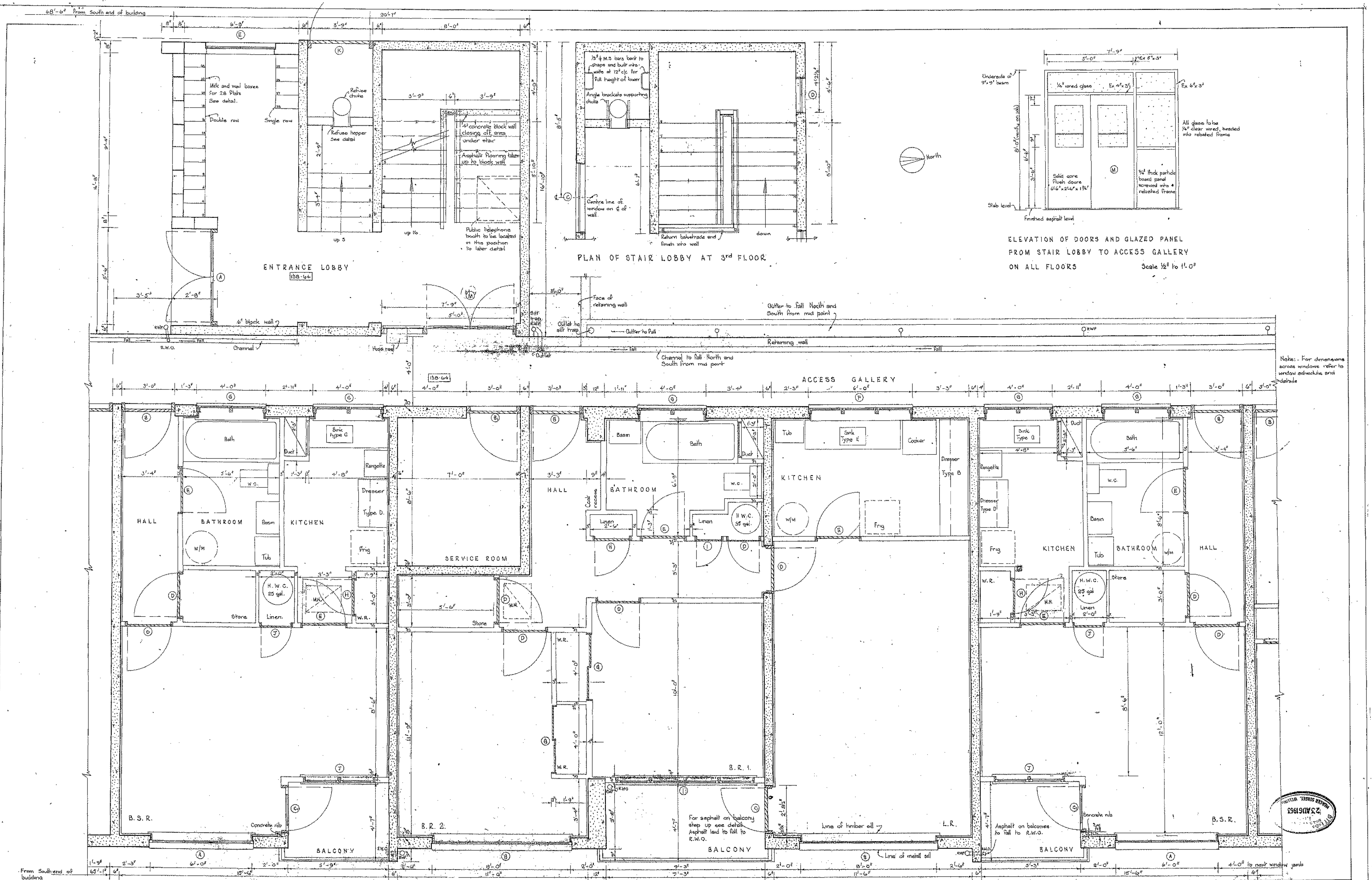
HANSON STREET FLATS DEVELOPMENT - BLOCK 1

1/4" Scale Plan South End of Building - North End of Building Similar.

CONTRACT No 2145

TRACING	No AL33/3
Designed	M.L.
Drawn	I.E.W. 10 April 1963
Traced	M.S.
Checked	
Approved	

F.B.C. Jeffreys M. Inst.C.
City Engineer Wellington



WELLINGTON CITY CORPORATION
CITY ENGINEER'S DEPARTMENT
ARCHITECTURAL BRANCH

HANSON STREET FLATS DEVELOPMENT - BLOCK 1

1/2" SCALE PLAN OF CENTRE PART OF BUILDING

CONTRACT No 2145

TRACING No AL/33/4

Designed	M.L.	
Drawn	M.W.	April 1965
Traced	M.S.	
Checked		
Approved		

City Engineer Wellington N.Z.

WELLINGTON CITY CORPORATION
CITY ENGINEER'S DEPARTMENT
ARCHITECTURAL BRANCH

HANSON STREET FLATS DEVELOPMENT - BLOCK I

1/2 SCALE ELEVATIONS

CONTRACT No	TRACING No AL/33/5
2145	Designed M.L.
	Drawn I.E.W. April 1965
	Traced M.S.
	Checked
	Approved
	G. L. B. Thomas M.N.Z.I.E. City Engineers Wellington N.Z.

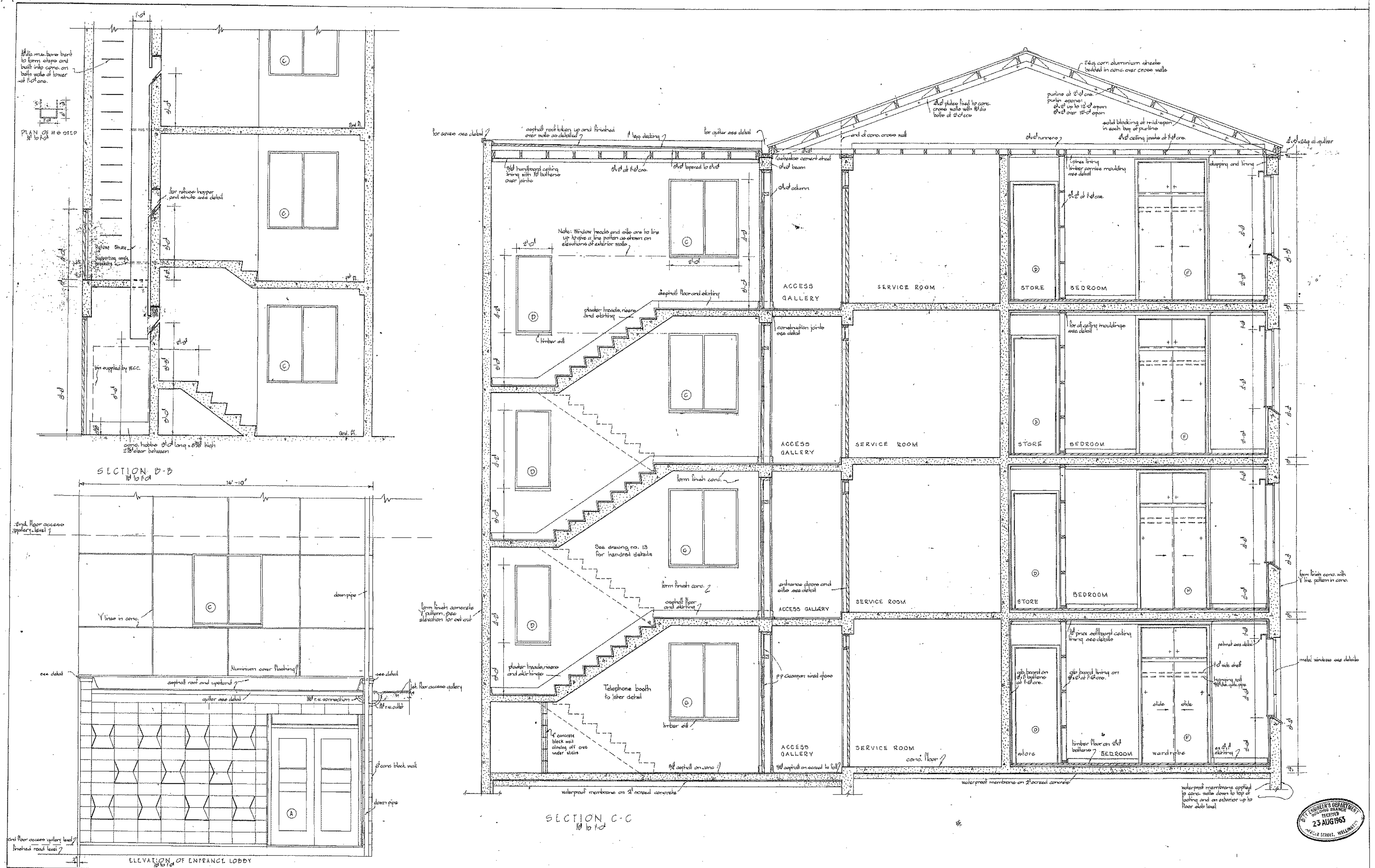
Scale: ½" = 1'-0"

WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT ARCHITECTURAL BRANCH	HANSON STREET FLATS DEVELOPMENT - BLOCK 1 1/2" SCALE ELEVATIONS	CONTRACT No 2145

TRACING No AL 33/6

Designed	M.L.	
Drawn	I.E.W.	April 1965
Traced	M.S.	
Checked		
Approved		

G. L. B. Thomas M.N.Z.I.E.
City Engineer Wellington N.Z.

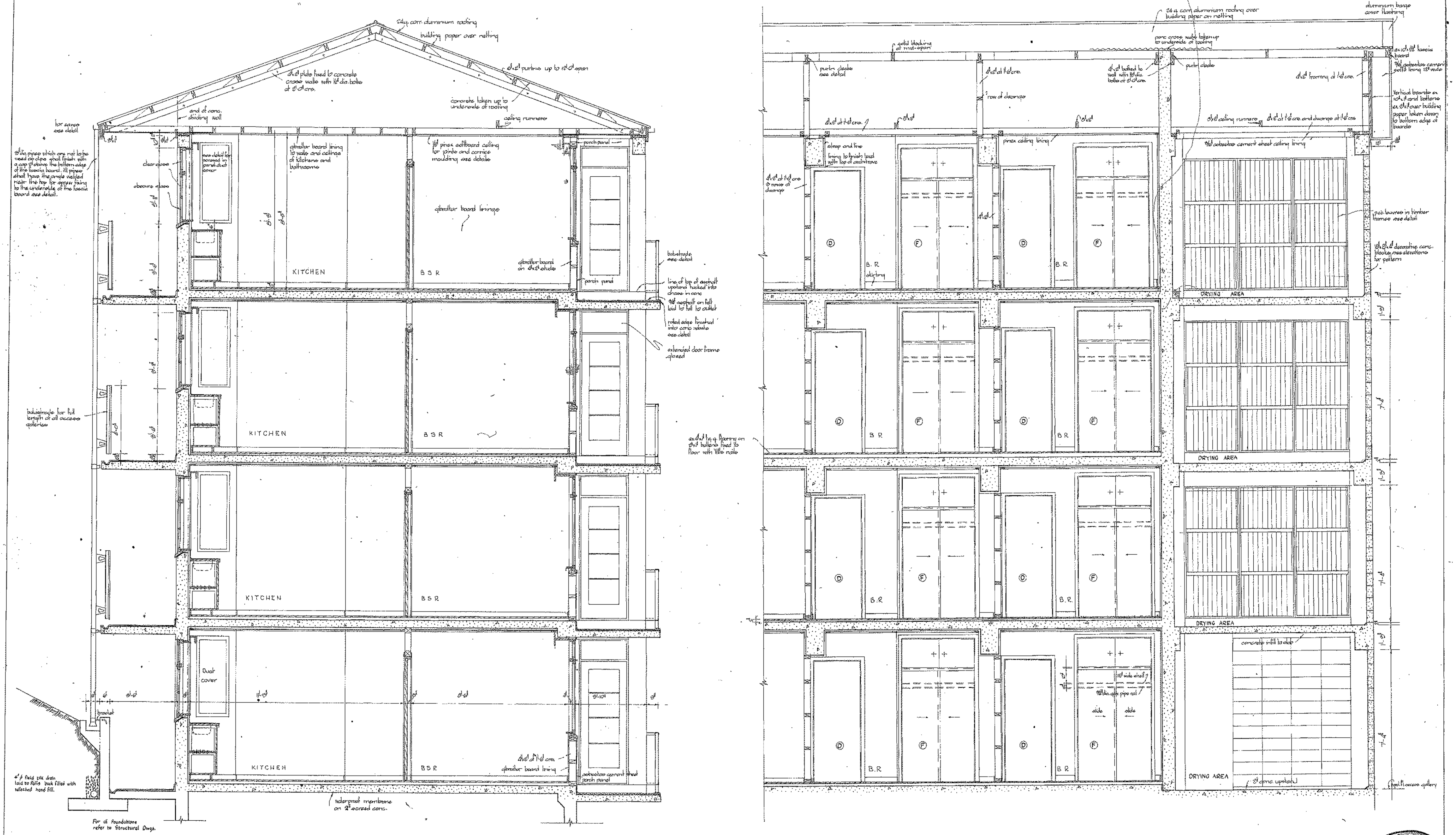


WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT ARCHITECTURAL BRANCH	HANSON STREET FLATS DEVELOPMENT - BLOCK I 1/2" SCALE CROSS SECTIONS

CONTRACT No 2145

TRACING No AL 33/7		
Designed	M.L.	
Drawn	I.E.W.	April 1963
Traced	M.S.	
Checked		
Approved		

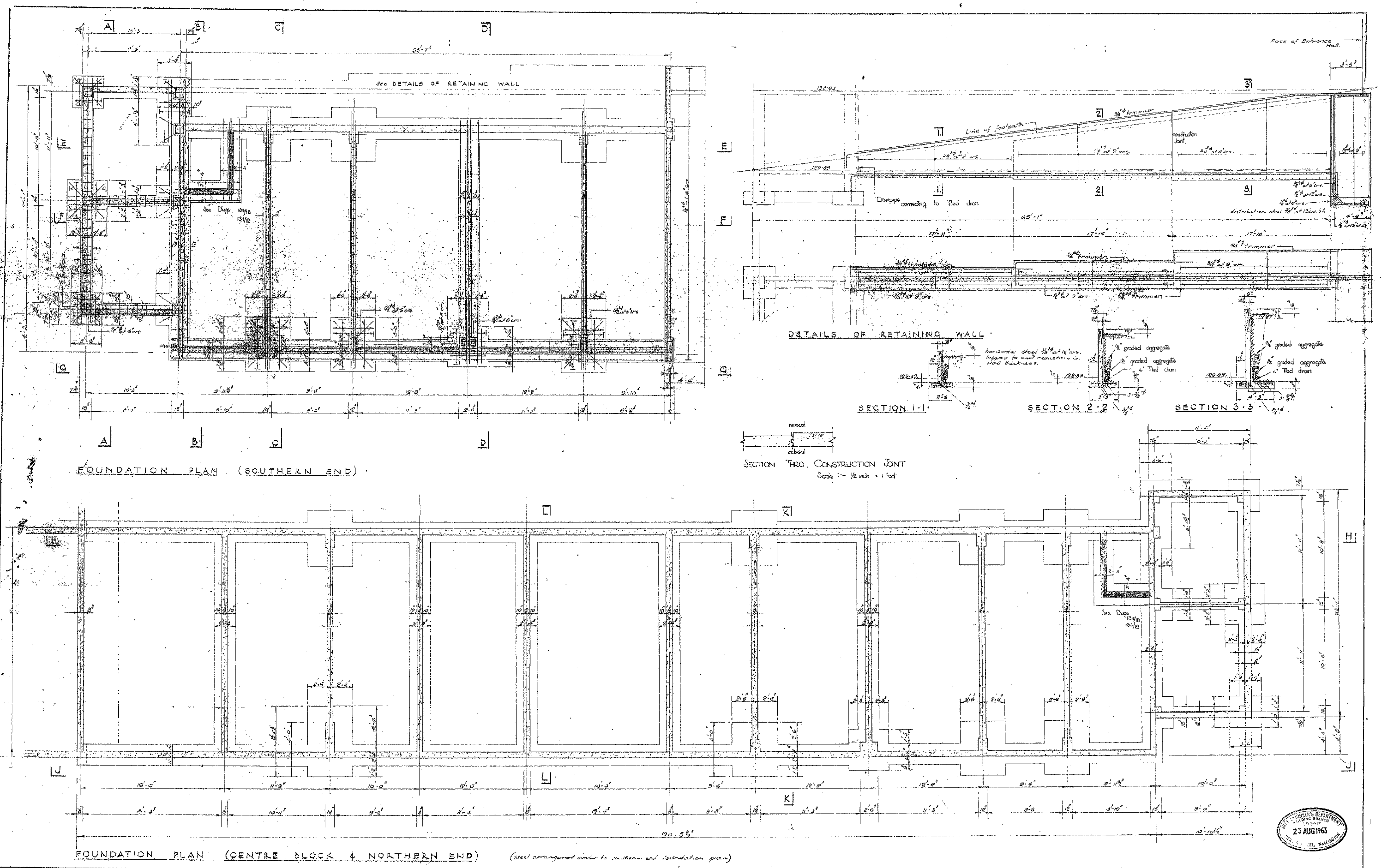
City Engineer Wellington N.Z.



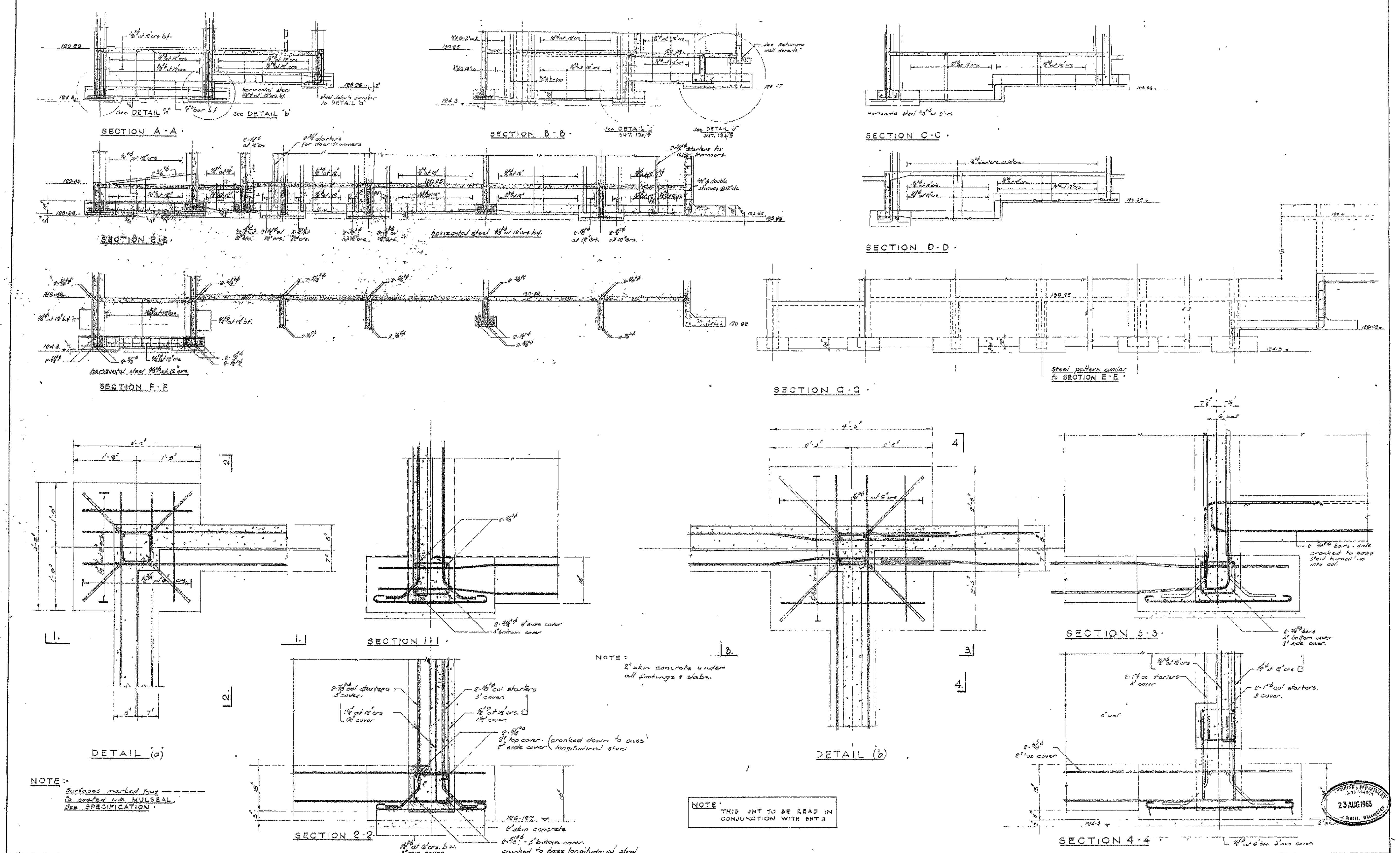
SECTION D-D
1/2" to 1'-0"

SECTION E-E
1/2" to 1'-0"

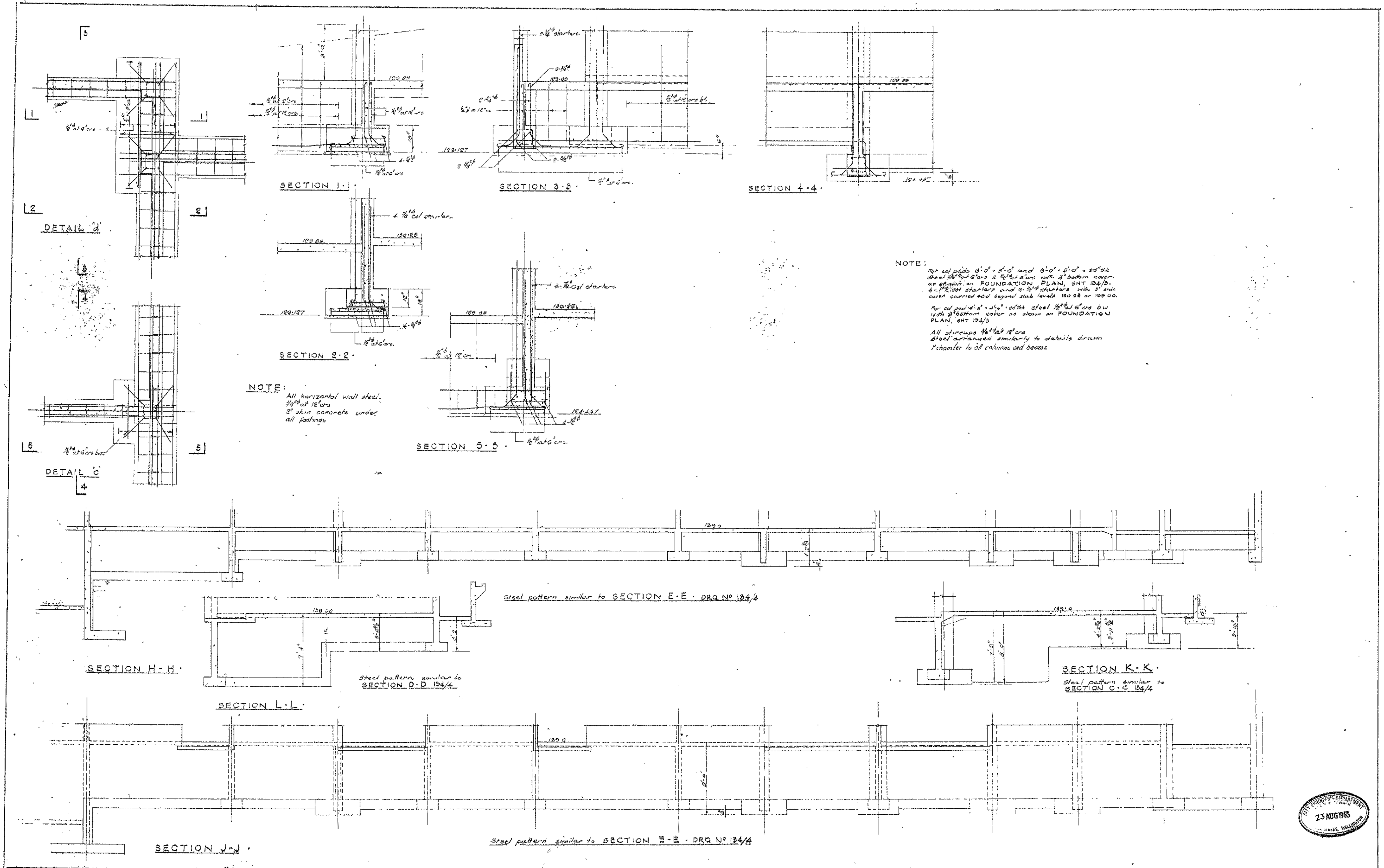
WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT ARCHITECTURAL BRANCH	HANSON STREET FLATS DEVELOPMENT - BLOCK I 1/2" SCALE CROSS SECTIONS	CONTRACT No 2145	TRACING No AL/33/8 Designed M.L. Drawn I.E.W. April 1963 Traced M.M.S. Checked Approved G.F.L.B. Thomas M.N.Z.I.E. City Engineer Wellington N.Z.



WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT STRUCTURAL BRANCH	HANSON STREET FLATS DEVELOPMENT — BLOCK 1 FOUNDATION PLAN & RETAINING WALL	CONTRACT No. 2145	TRACING No. STR.134/3 DRAWN W.H.P. APRIL '63 TRACED CHECKED APPROVED 18-4-63 C.I.B. THOMAS CITY ENGINEER. WELLINGTON N.Z.



WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT STRUCTURAL BRANCH	HANSON STREET FLATS DEVELOPMENT - BLOCK 1 FOUNDATIONS SECTIONS AND DETAILS	CONTRACT No. 2145	TRACING No. STR. 134/4 DRAWN W.H.F. APRIL '63 TRACED CHECKED APPROVED G.L.S. THOMAS CITY ENGINEER - WELLINGTON N.Z.

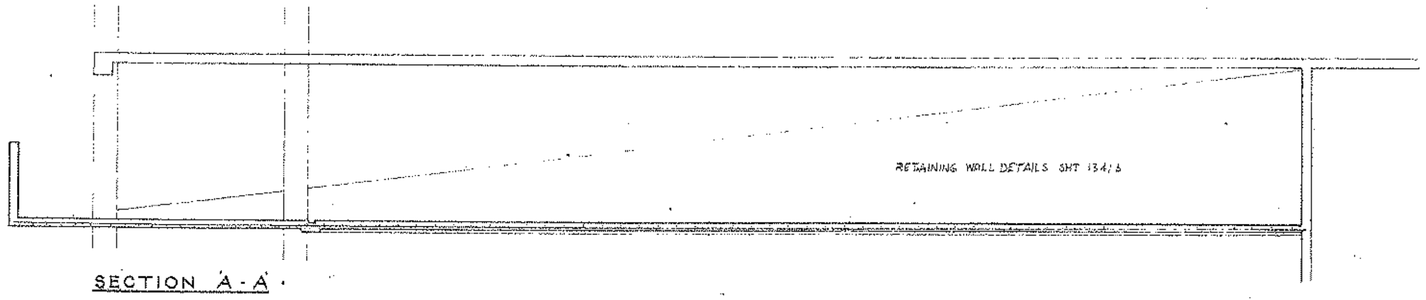


WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT STRUCTURAL BRANCH	HANSON· STREET FLATS DEVELOPMENT – BLOCK I· FOUNDATIONS· SECTIONS AND DETAILS·	CONTRACT No. 2145·

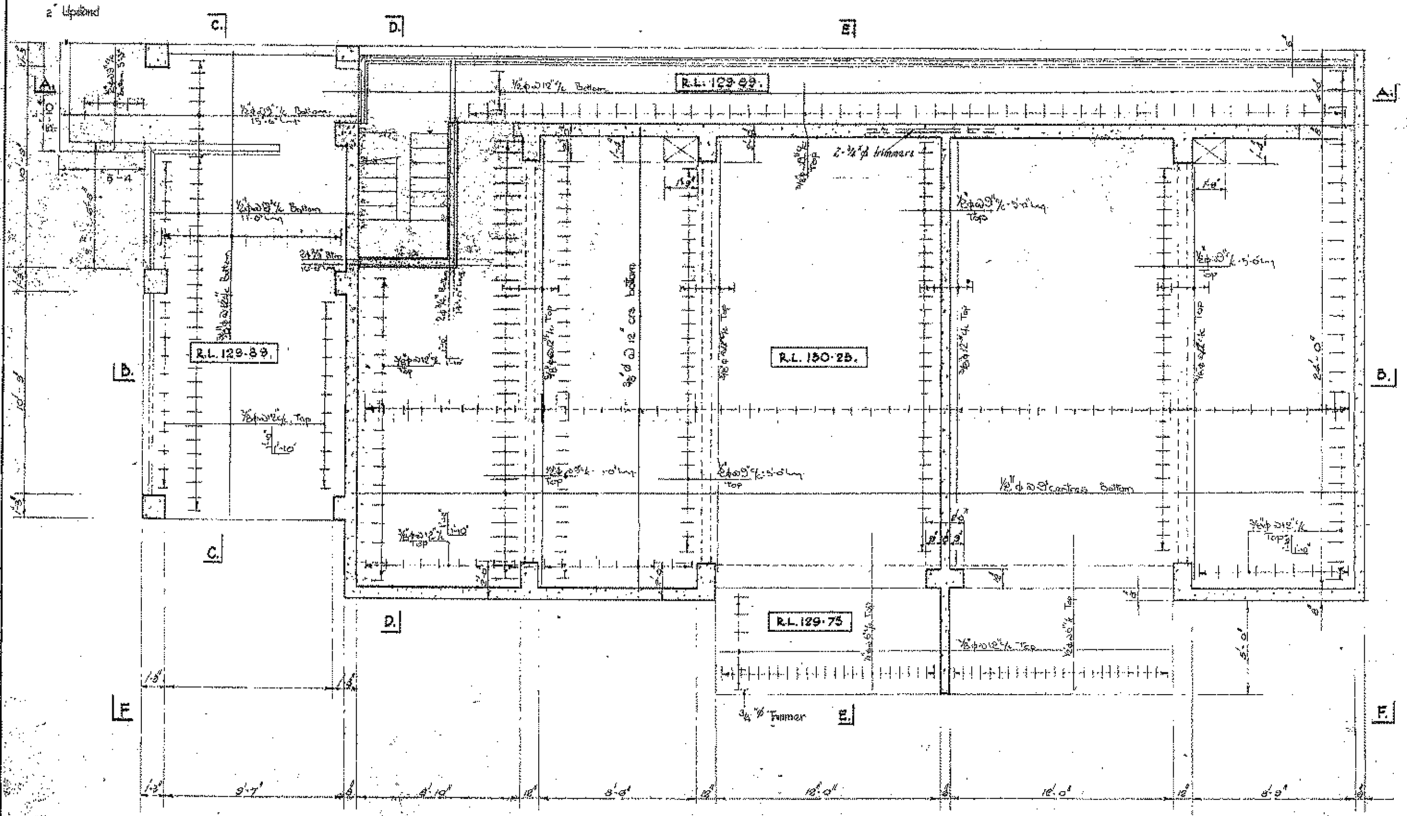
TRACING No. STR 134/8

DRAWN	W.H.P	APRIL '63
TRACED		
CHECKED		
APPROVED		

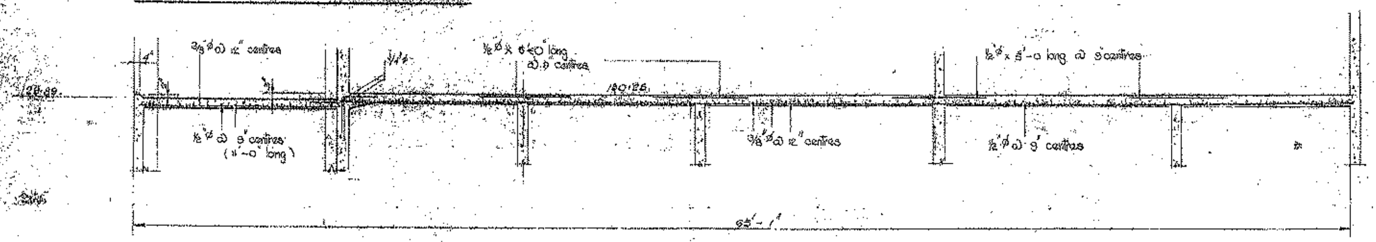
G.L. THOMAS
CITY ENGINEER, WELLINGTON N.Z.



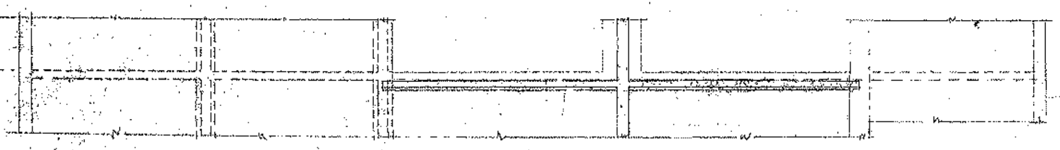
SECTION A-A

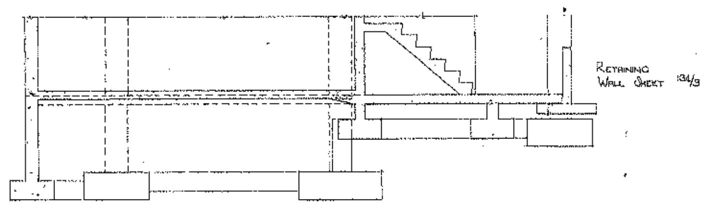


LOWER GROUND FLOOR PLAN

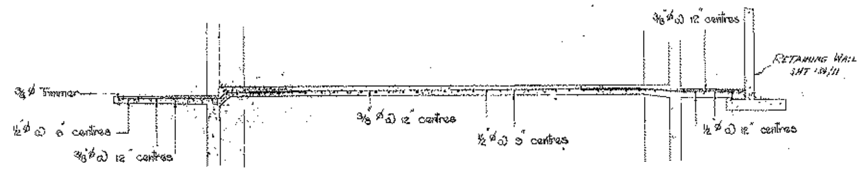


SECTION B-B
130·25





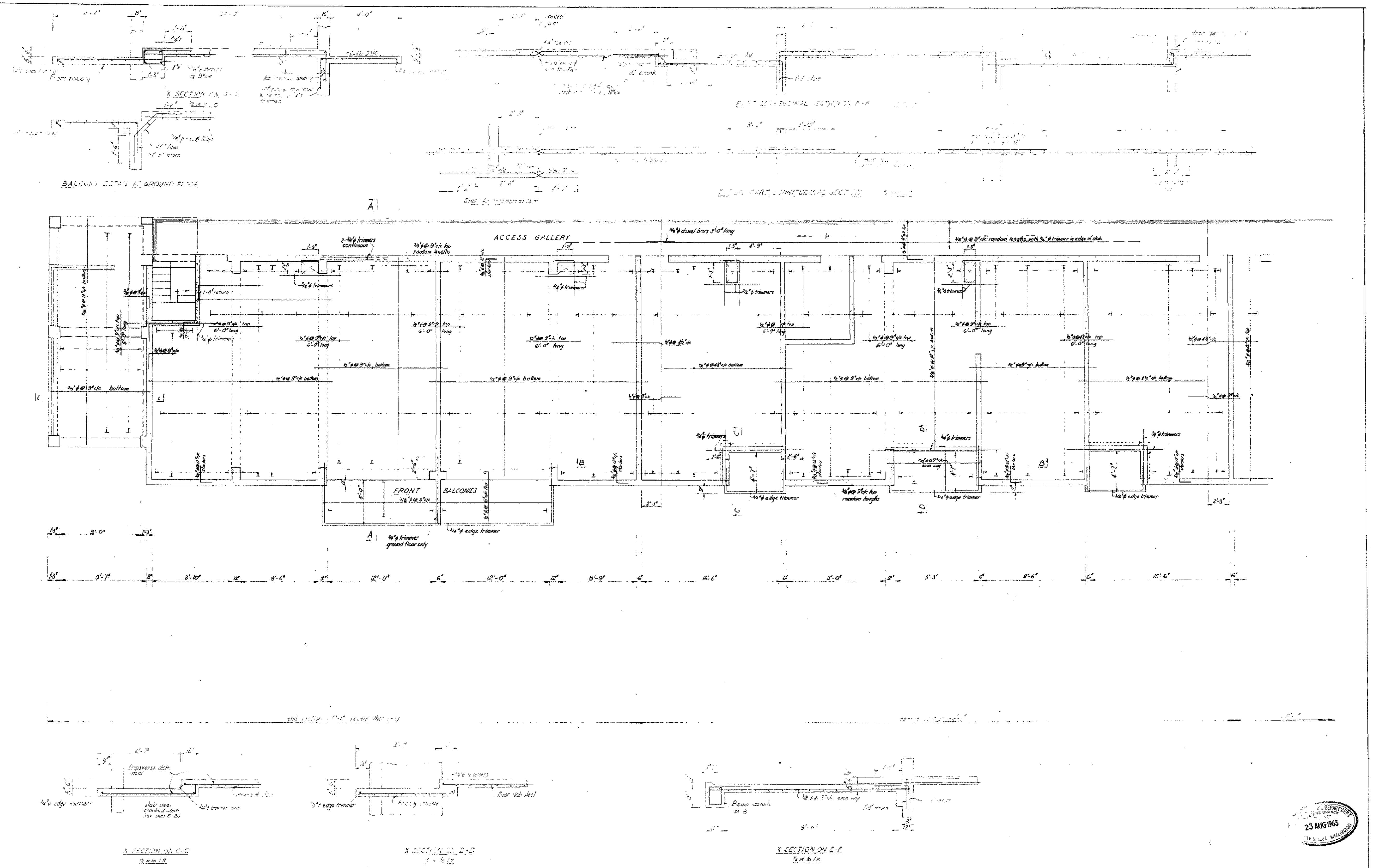
SECTION D-D



SECTION E-E

WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT STRUCTURAL BRANCH.	HANSON STREET FLATS DEVELOPMENT — BLOCK I. LOWER GROUND FLOOR PLAN.	CONTRACT No.	TRACING No. STR 134/8

DRAWN.	W.H.P. & D.S.	APRIL '63.
TRACED.		
CHECKED.		
APPROVED.		
G. I. B. THOMAS CITY ENGINEER - WELLINGTON, N.Z.		



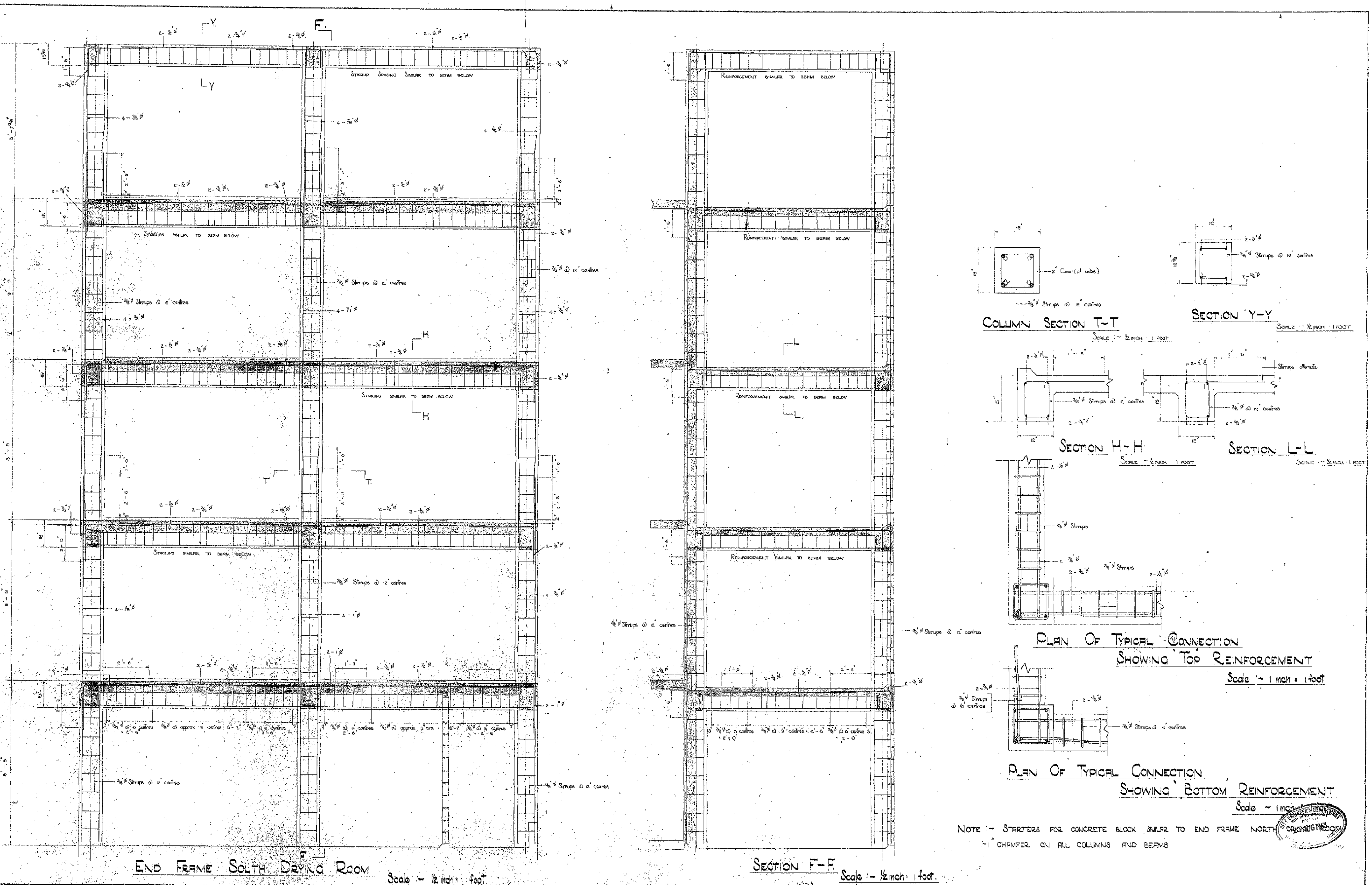
WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT STRUCTURAL BRANCH	HANSON STREET FLATS DEVELOPMENT — BLOCK 1. TYPICAL FLOOR PLAN.	CONTRACT No.

TRACING No. STR 134/7

DRAWN. April '65
TRACED.
CHECKED.
APPROVED.

C. I. B. THOMAS.
CITY ENGINEER · WELLINGTON N.Z.

23 AUG 1965

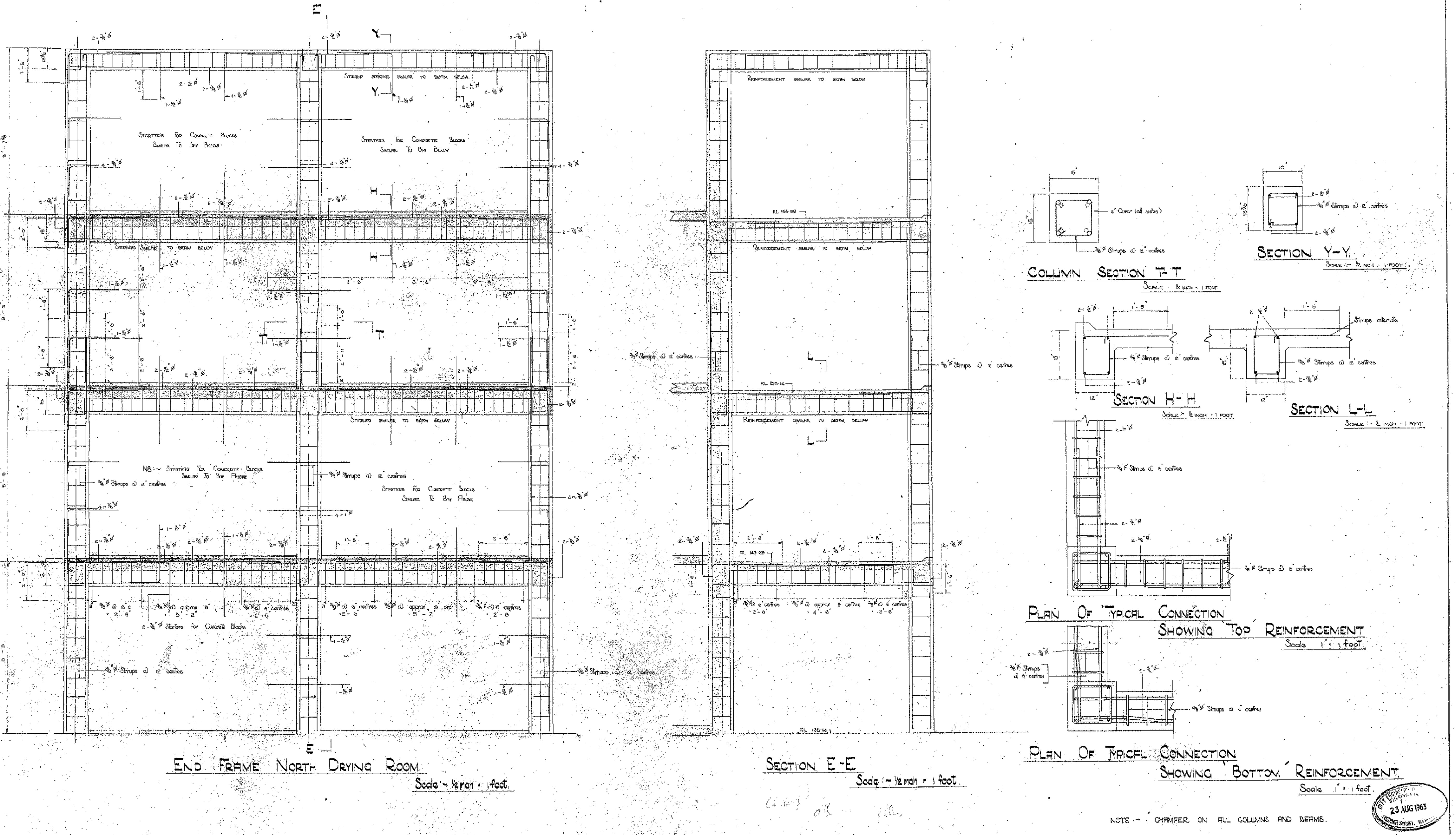


WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT STRUCTURAL BRANCH	HANSON STREET FLATS DEVELOPMENT — BLOCK 1. COLS. & BEAMS - DRYING BAY SOUTHERN END.	CONTRACT No. 2145.

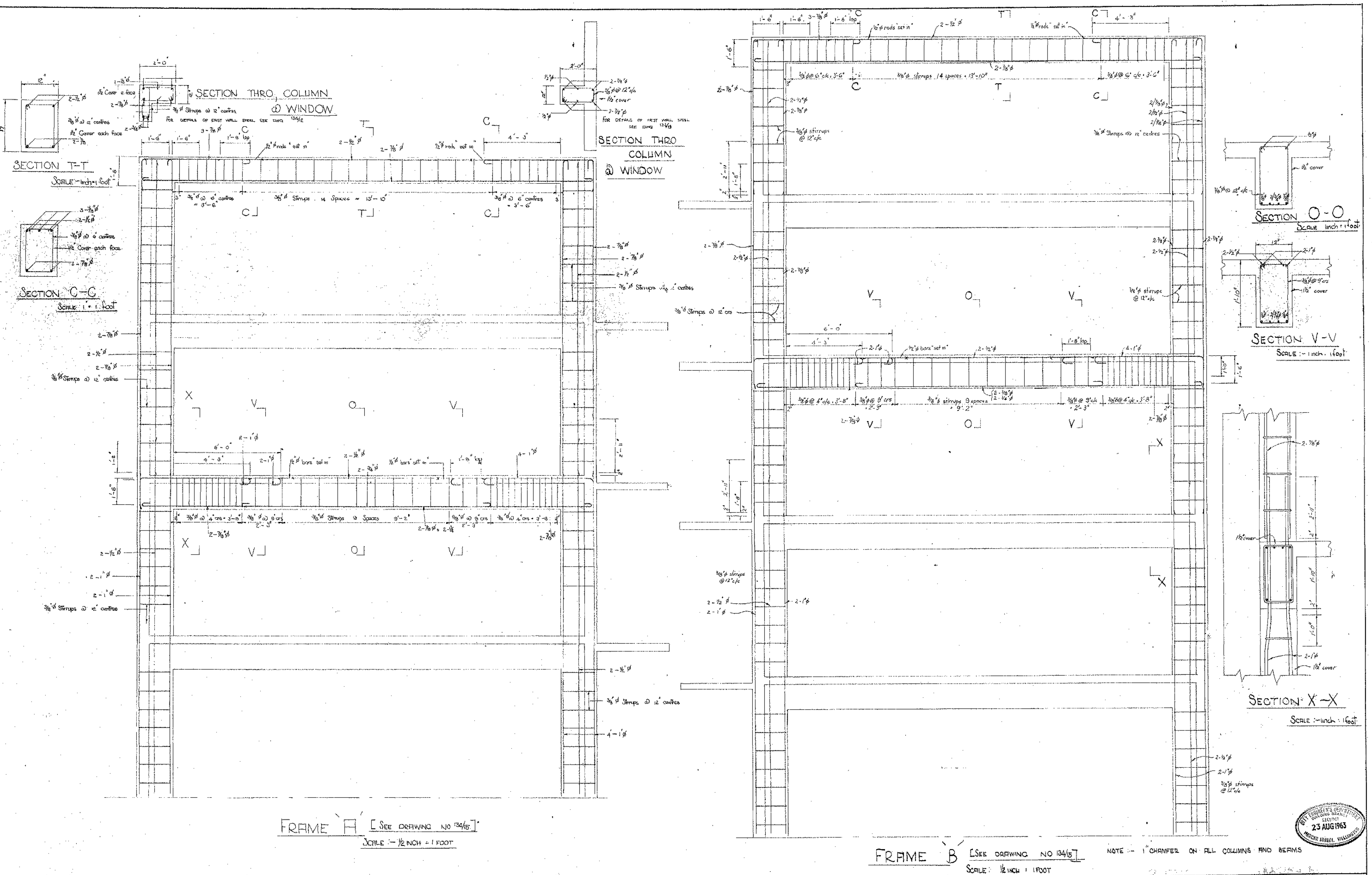
TRACING No. STR.134/8

CALCULATIONS	J. G. FROHN	MARCH 1963
DRAWN	C C & W.H.F	" "
TRACED	G. CURRIE	
CHECKED		
APPROVED		

F B C JEFFREYS M.I.C.E.
CITY ENGINEER WELLINGTON N.Z.



WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT STRUCTURAL BRANCH	HANSON STREET FLATS DEVELOPMENT — BLOCK I. COLS. & BEAMS - DRYING BAY NORTH END.	CONTRACT No. 2145.	TRACING No. STR. 134/9 CALCULATIONS J.G. FROST MARCH 1962 DRAWN C. COPPE TRACED CHECKED APPROVED 18.6.62 F.B.C. JEFFERIS M.I.C.E. CITY ENGINEER WELLINGTON N.Z.



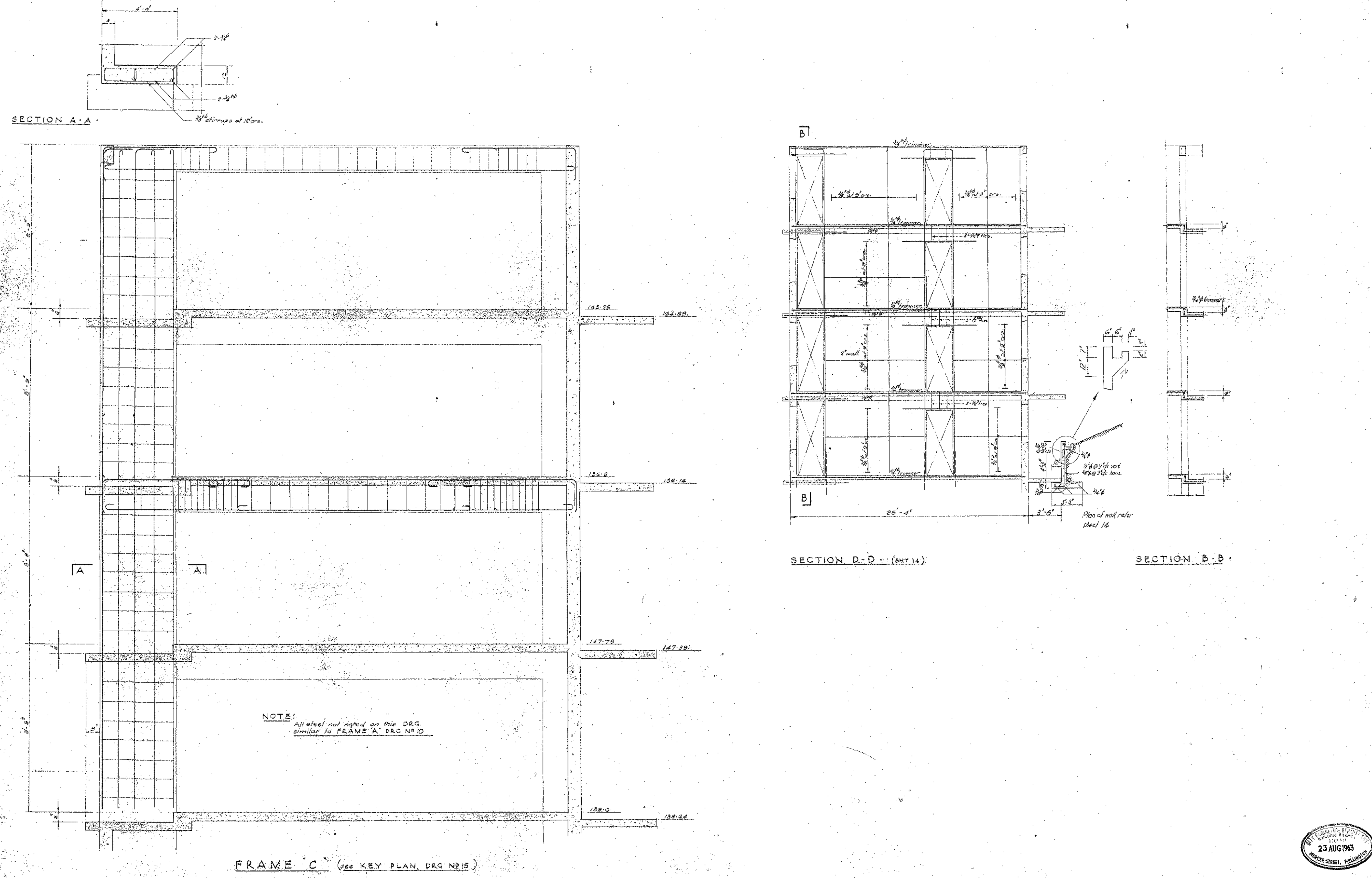
WELLINGTON CITY CORPORATION
CITY ENGINEER'S DEPARTMENT
STRUCTURAL BRANCH

HANSON STREET FLATS DEVELOPMENT – BLOCK 1

MAIN FRAMES.

CONTRACT No.	TRACING No. STR 134/10	
2145	COMPUTATIONS	J C F
	DRAWN	C.C. Apr. 63
	TRACED	C.C. Aug. 63
	CHECKED	
	APPROVED	

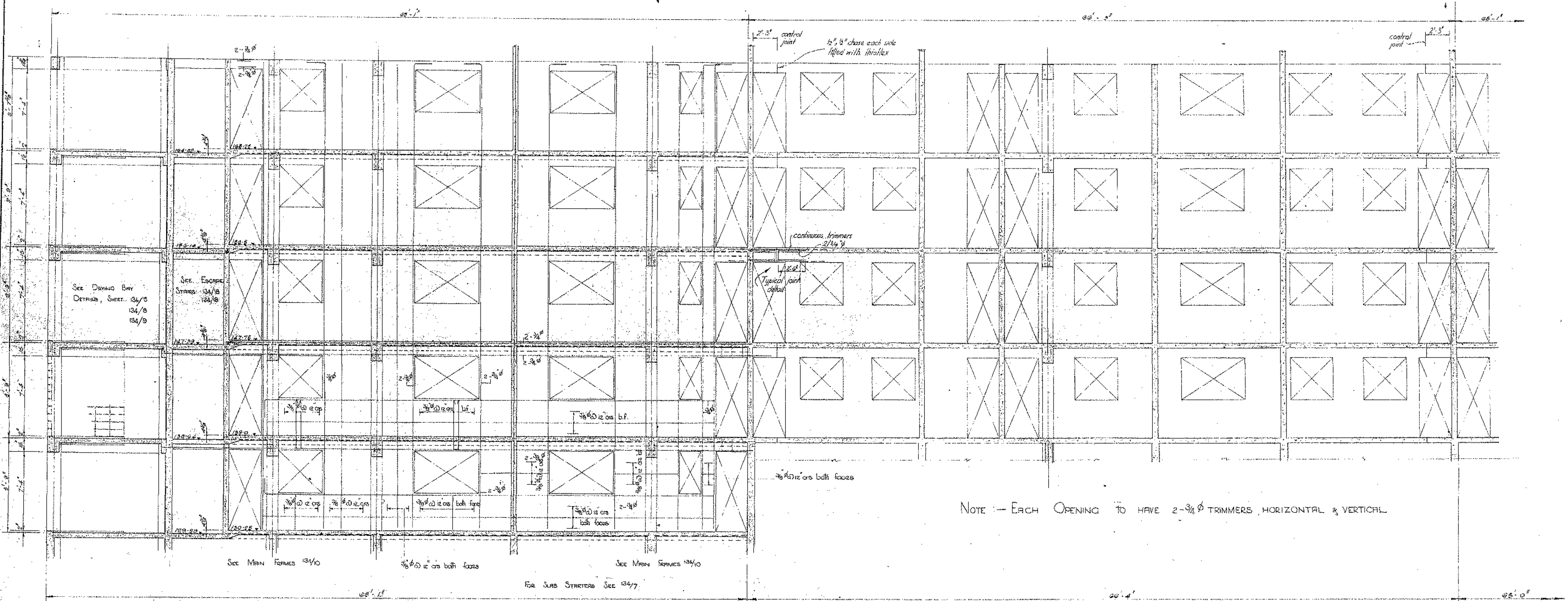
G. T. B. THOMAS M.N.Z.I.E.
CITY ENGINEER WELLINGTON N.Z.



WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT BRANCH	HANSON STREET FLATS DEVELOPMENT — BLOCK I · FRAME 'C' & CROSS WALL DETAILS ·	CONTRACT No. 2145 ·	TRACING No. STR 134/11

DRAWN	W. H. F.	Apr. '63
TRACED		
CHECKED		
APPROVED		

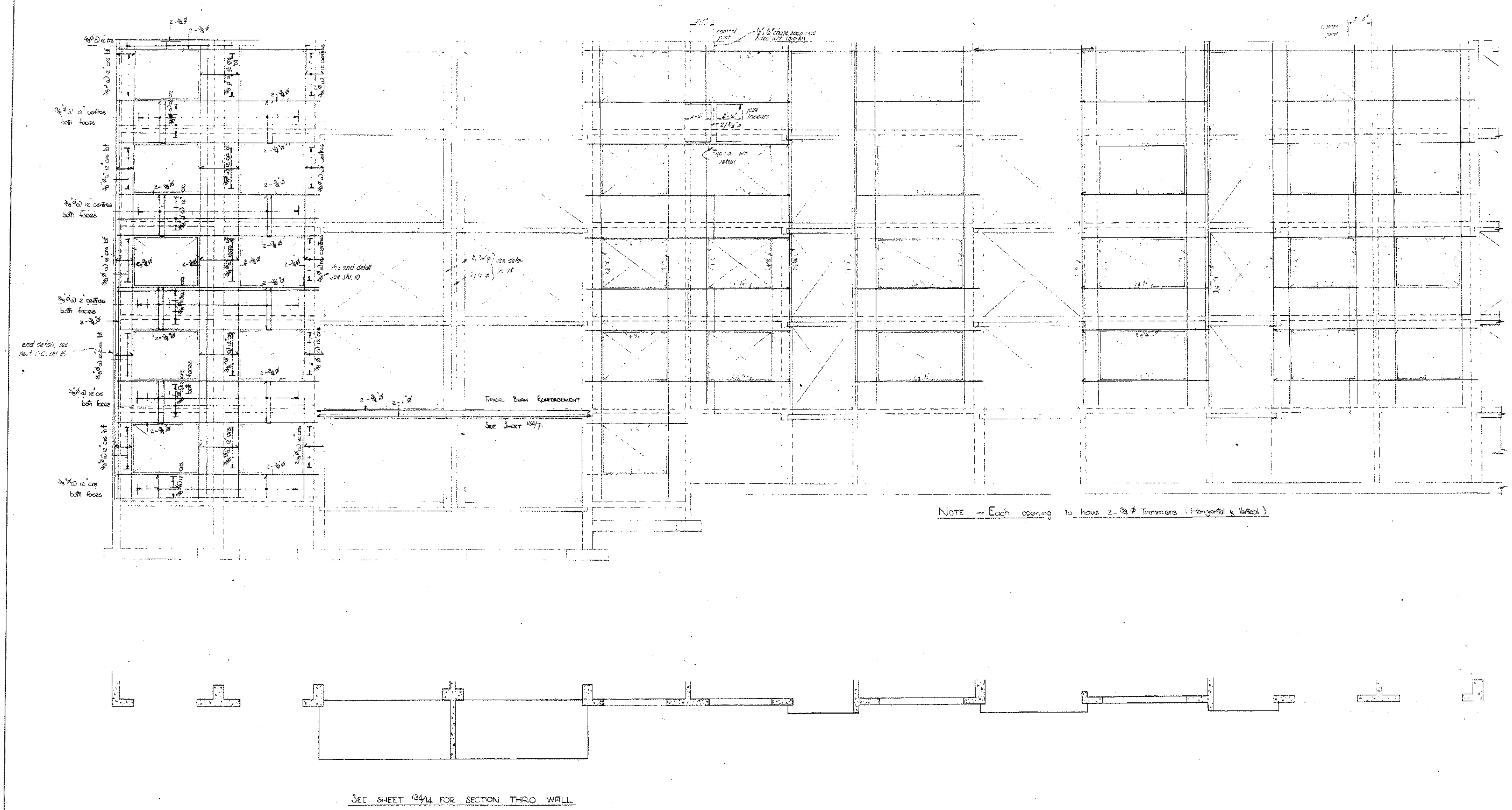
G. I. B. THOMAS
CITY ENGINEER - WELLINGTON. N.Z.



Pattern of steel shown repeats in all bays and all floors

See see sheet 134/14 for section thro wall

WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT STRUCTURAL BRANCH	HANSON STREET FLATS DEVELOPMENT — BLOCK 1. ELEVATION - WEST WALL.	CONTRACT No. 2145.	TRACING No. STR 134/12		
			DRAWN	W.H.F.	April '63
			TRACED		
			CHECKED		
			APPROVED		12.8.63
			G. L. B. THOMAS CITY ENGINEER - WELLINGTON. N.Z.		



23 AUG 1963

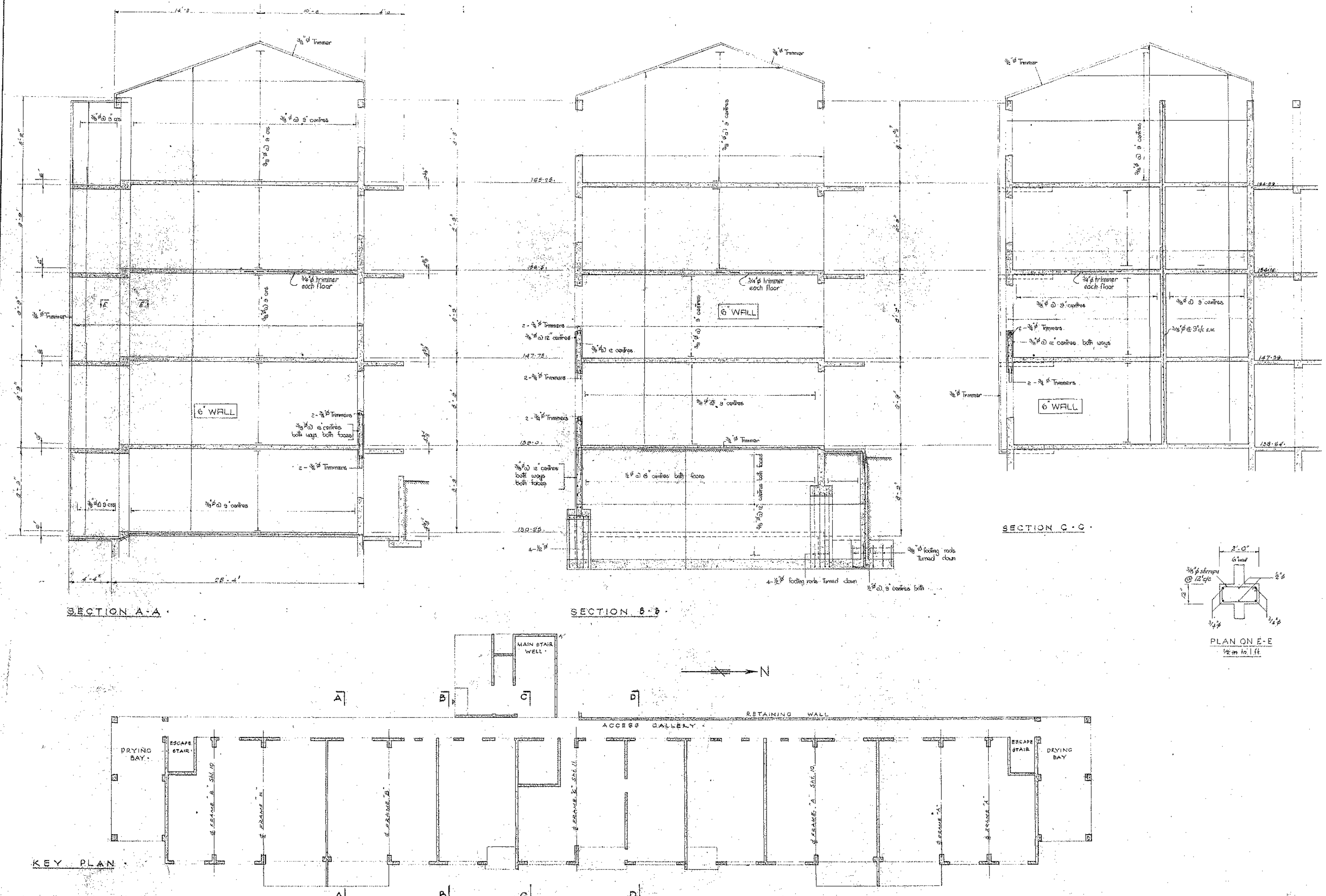
WELLINGTON CITY CORPORATION CITY ENGINEERS DEPARTMENT. STRUCTURAL BRANCH.	HANSON STREET FLATS DEVELOPMENT — BLOCK 1 · ELEVATION - EAST WALL ·

CONTRACT No. 2145

TRACING No. STR 134/13

CALCULATION		
DRAWN	E.J. Allomee D.S.	April '63
TRACED		
CHECKED		
APPROVED.		

P.B.C. Jefferys M.I.C.E.
City Engineer, Wellington, New Zealand.

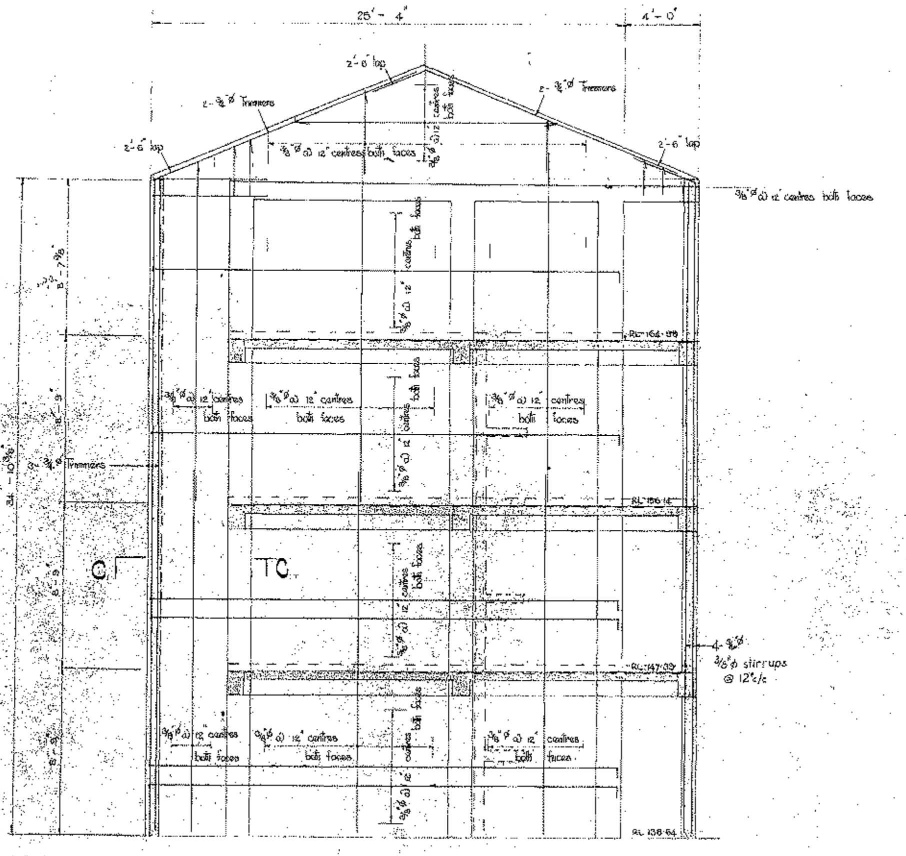


23 AUG 1963

WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT STRUCTURAL BRANCH	HANSON STREET FLATS DEVELOPMENT — BLOCK I · KEY PLAN & CROSS WALL DETAILS ·	CONTRACT No. 2145 ·	TRACING No. STR 134/14

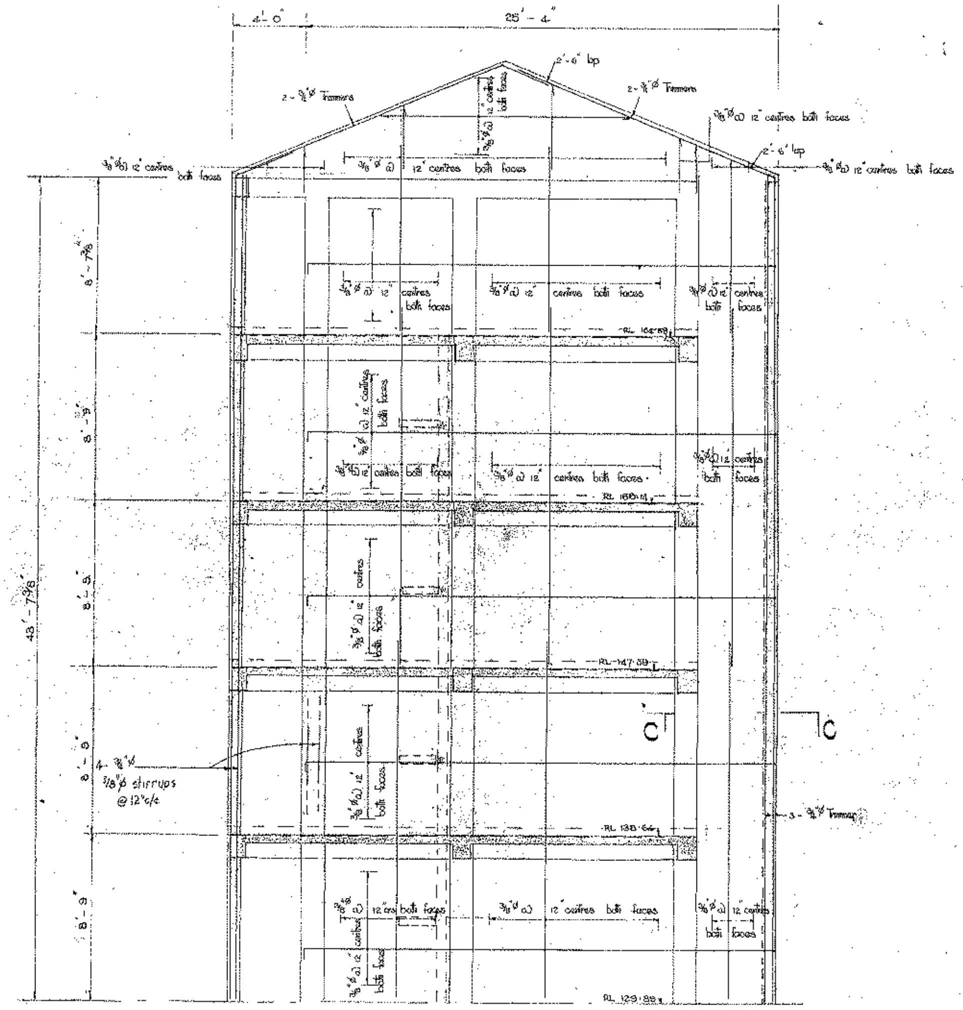
CALCULATIONS	J. G. FROMM	APRIL 1963
DRAWN	W.H.P.F.	APRIL 1963
TRACED		
CHECKED		
APPROVED		

G. I. B. THOMAS M.NZIE
CITY ENGINEER, WELLINGTON N.Z.



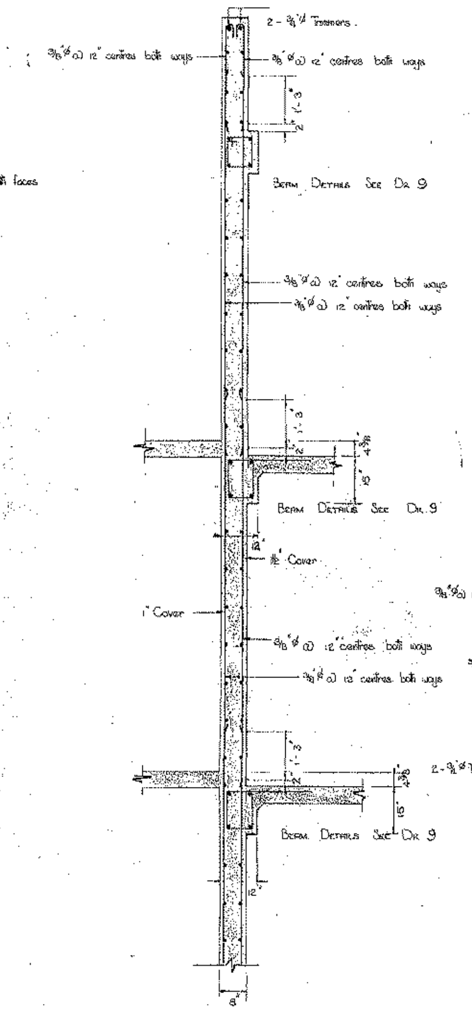
DIVIDING WALL NORTH DRYING ROOM.
SECTION H-H   Scale :- 1/4 inch = 1 foot

NOTE :- COLUMN AND BEAM STEEL SIMILAR TO SHEET 134/9

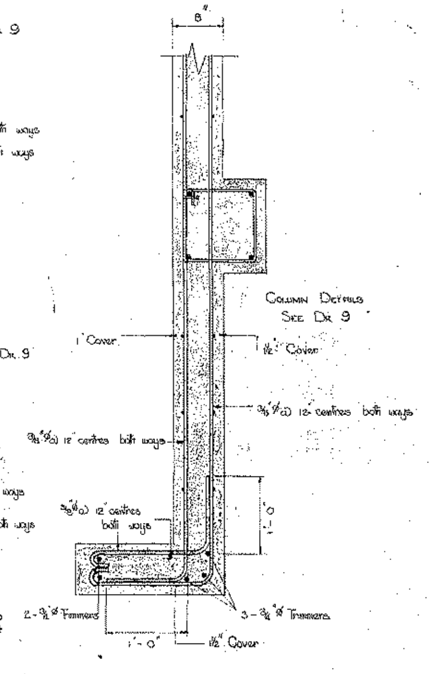


DIVIDING WALL SOUTH DRYING ROOM
SECTION G-G.   Scale :- 1/4 inch = 1 foot.

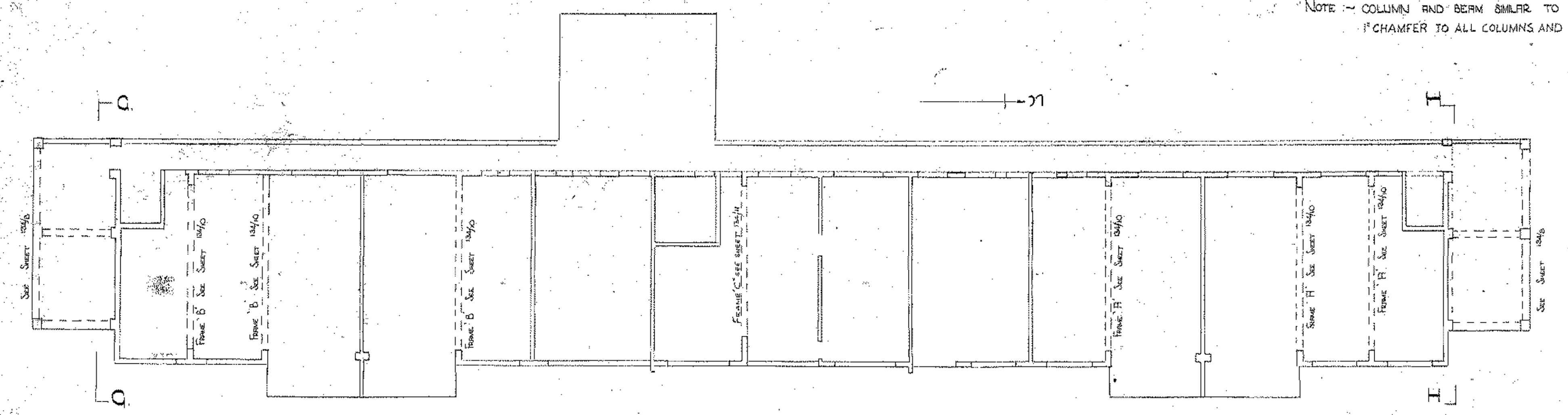
NOTE :- COLUMN AND BEAM SIMILAR TO SHEET 134/8
1" CHAMFER TO ALL COLUMNS AND BEAMS.



TYPICAL WALL SECTION
Scale : 1/2 inch = 1 foot.



SECTION C-C
Scale : 1 inch = 1 foot.



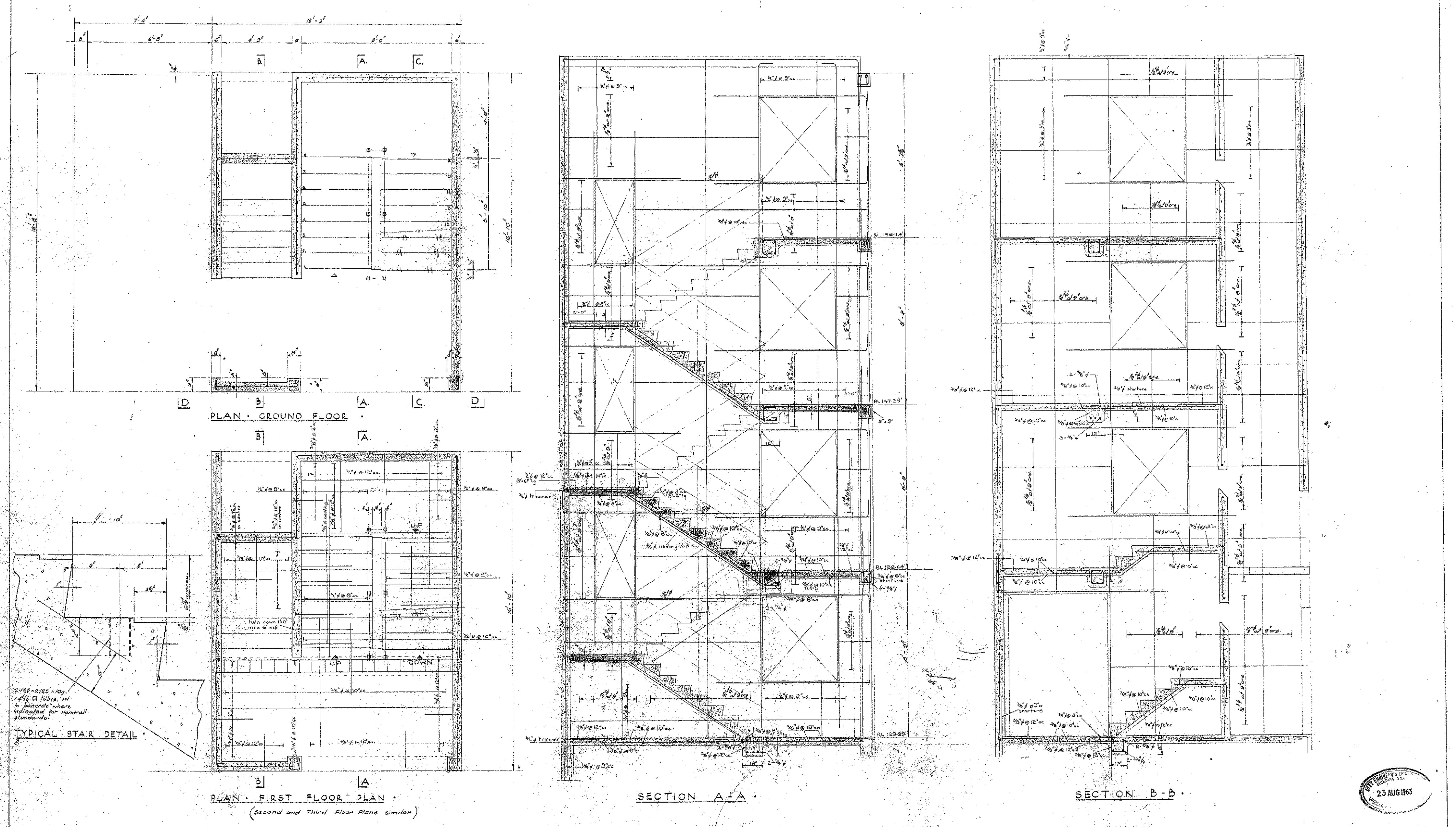
KEY PLAN   Scale :- 1/8 inch = 1 foot

WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT STRUCTURAL BRANCH	HANSON STREET FLATS DEVELOPMENT - BLOCK 1. DRYING BAYS - CROSS WALLS.	CONTRACT No. 2145.

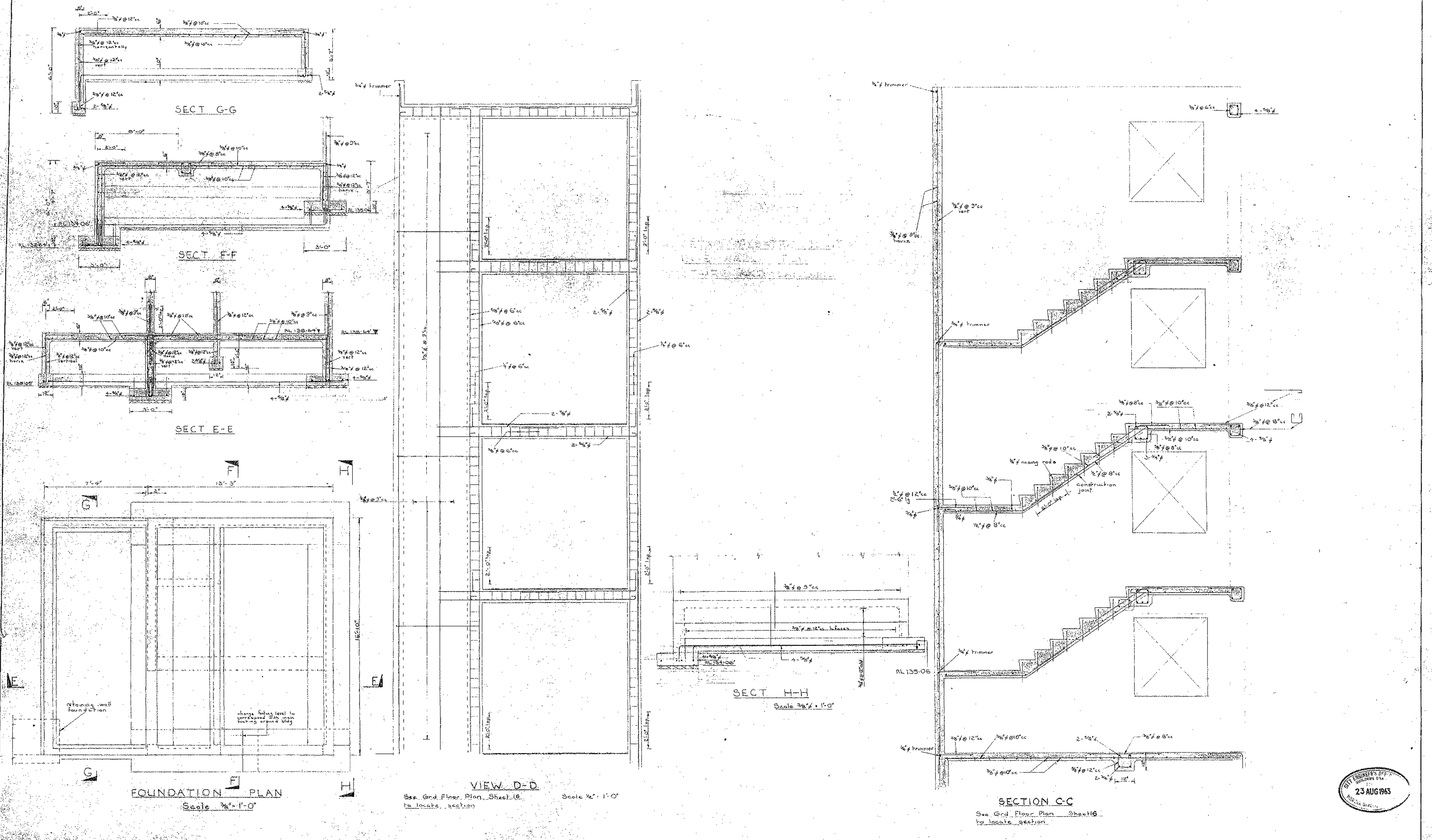
TRACING No. STR 134/15.

CALCULATIONS	J. G. Frohn	March 1963.
DRAWN	C. Carrie	April 1963
TRACED	"	"
CHECKED		
APPROVED		8.4.63

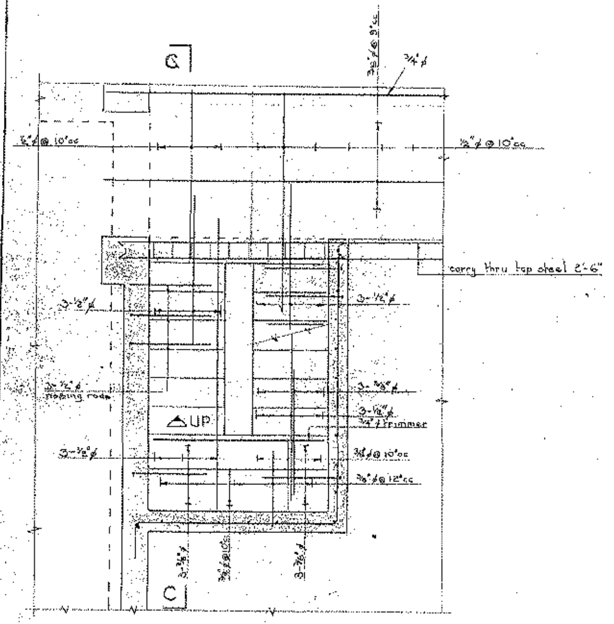
G. I. B. Thomas, M.N.Z.I.E.
City Engineer, Wellington, N.Z.



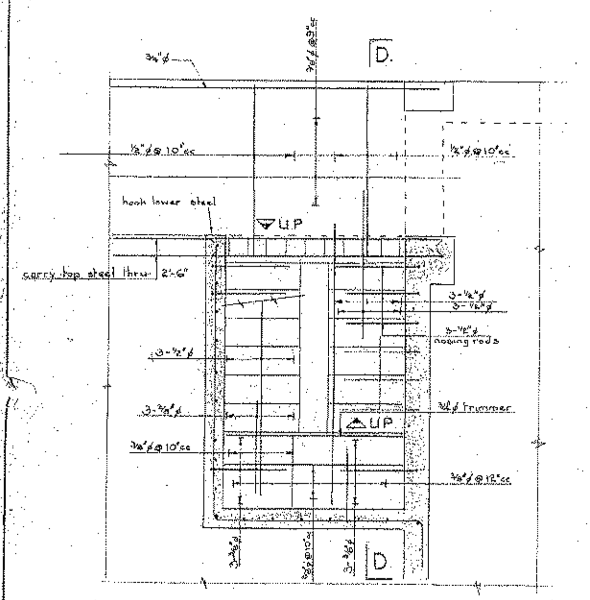
WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT STRUCTURAL BRANCH	HANSON STREET FLATS DEVELOPMENT — BLOCK I · MAIN STAIR DETAILS ·	CONTRACT Nº	TRACING Nº STR 124/16 DRAWN. J.B. W.H.F. APRIL '63 TRACED. CHECKED APPROVED G. I. S. THOMAS CITY ENGINEER · WELLINGTON N.Z.



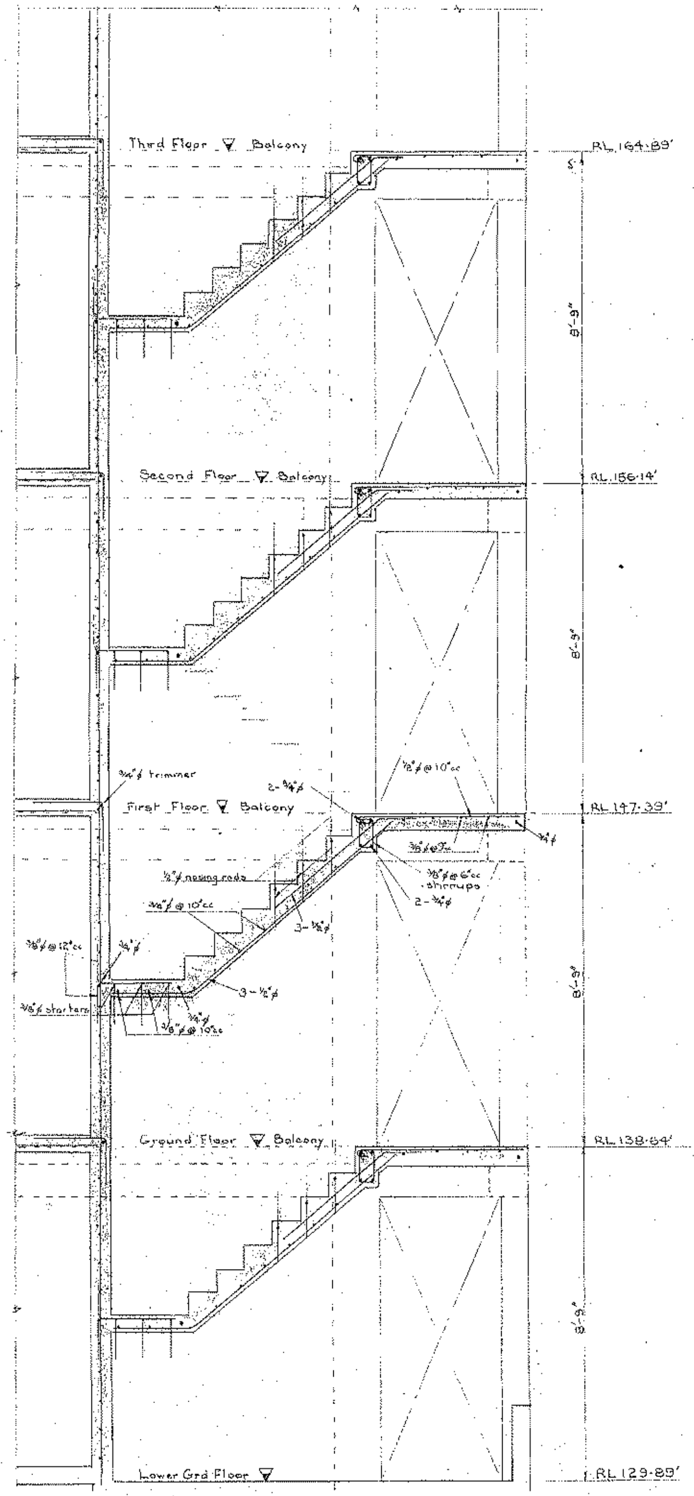
WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT STRUCTURAL BRANCH	HANSON STREET FLATS DEVELOPMENT — BLOCK 1. MAIN STAIR DETAILS.	CONTRACT No. 2145.	TRACING No. STR 134/17 DRAWN J.S. April '63 TRACED CHECKED APPROVED



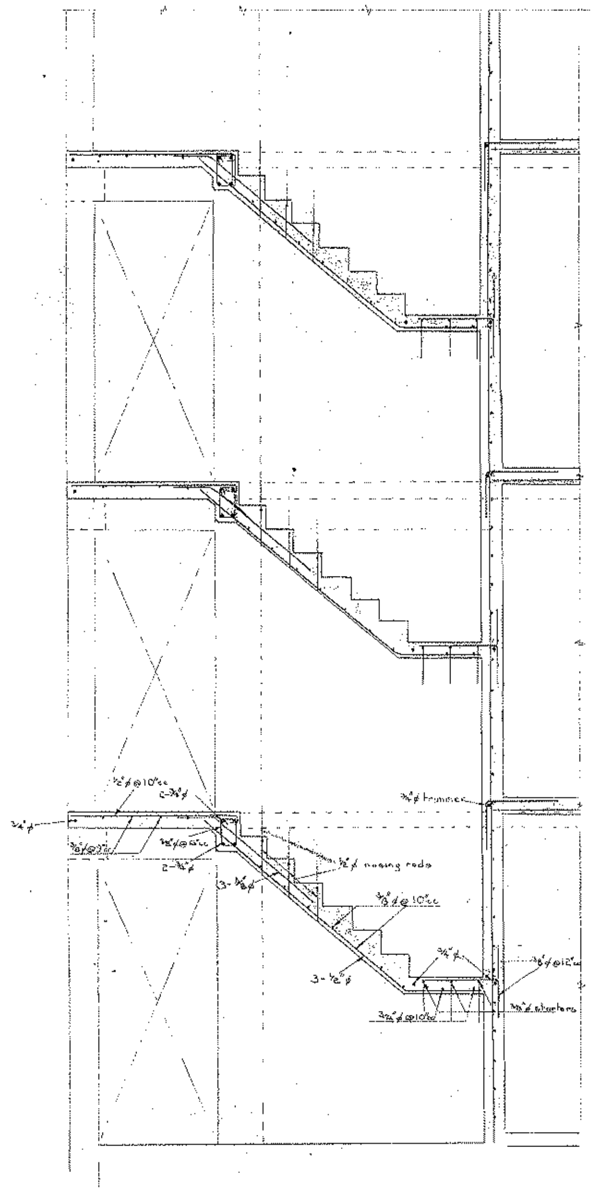
GROUND FLOOR PLAN
SOUTH END - ESCAPE STAIR WELL



FIRST FLOOR PLAN
NORTH END ESCAPE STAIR WELL



SECTION C-C



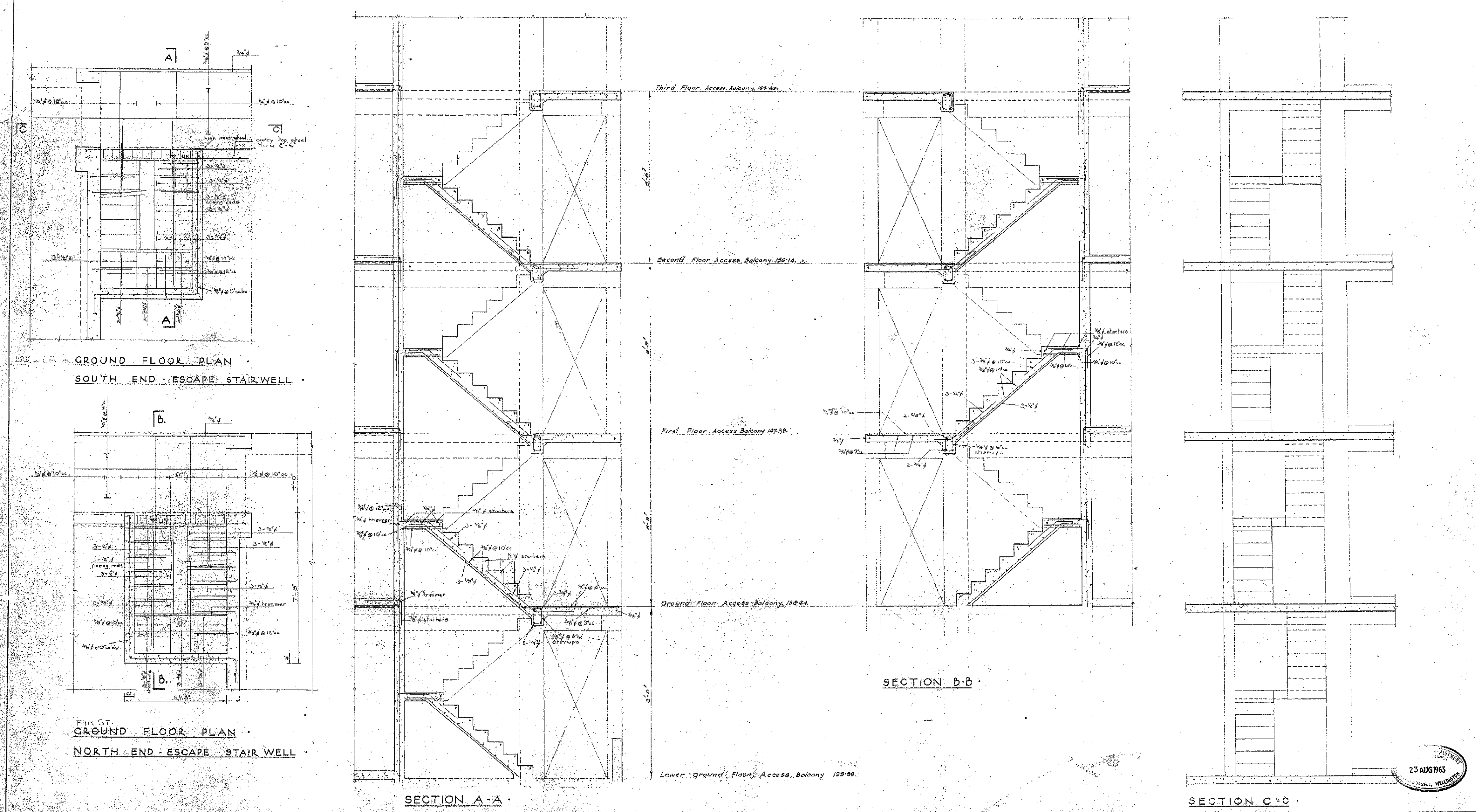
SECTION D-D

WELLINGTON CITY CORPORATION CITY ENGINEER'S DEPARTMENT STRUCTURAL BRANCH	HANSON STREET FLATS DEVELOPMENT —— BLOCK I DETAILS OF ESCAPE STAIRWAYS	CONTRACT No. 2145

TRACING No. STR 134/8

DRAWN	J. B.	April '63
TRACED		
CHECKED		
APPROVED		

G. I. S. THOMAS.
CITY ENGINEER, WELLINGTON N.Z.



WELLINGTON CITY CORPORATION	HANSON STREET FLATS DEVELOPMENT — BLOCK 1 ·
CITY ENGINEER'S DEPARTMENT	DETAILS OF ESCAPE STAIRWAYS ·
STRUCTURAL BRANCH	

CONTRACT No. 2145

TRACING No. STL 134/9

DRAWN	J.B. / W.H.F. APR.
TRACED	
CHECKED	
APPROVED	

G. I. D. THOMAS
CITY ENGINEER · WELL

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