Further information to Process DSA Reports – WHA

Address	WCC Residence Team Comments	Robert Bird Response
16 Glenmore Street	1) The report notes the assessment uses the 2017 Red Book and 2018 Yellow Chapter. Please confirm the elements identified as <34%NBS are identified in accordance with the Red Book. 2) Please provide calculations that support the assessment. 3) Please outline the extent of inspections in the assessment summary report. 4) The assessment summary report notes the status as draft, please update. 5) Please confirm the %NBS of the building. There are contradictions between text, tables and summary report (though noting that all indicate <34%NBS).	1) The procedures in the 2017 version of the guide were used to confirm the status of all potentially-earthquake prone elements. Note that there is only a very small difference between the Red and Yellow book for the types of elements present in this building. 2) Complete calculations were included as an appendix in the report. 3) We have amended the report to note that we went twice before preparing the report, and describing the kind of observations made. 4) We have amended to show “Final” 5) We have corrected the %NBS to align.

WHAA Detailed Seismic Assessment
HUP2-T0-Seismic Assessments

16 Glenmore Street

Job Number: N0541
Revision: A01
Date of issue: 13 January 2025

Document Number:
N0541-RBG-WHA-XX-RP-ST-00001

Prepared for Wellington City Council

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Report Amendment Register

Issue Ref	Amended Section(s)	Issue/Amendment Details	Author(s)	Reviewer	Date
P01	N/A	Draft for Peer Review	[redacted]	[redacted]	24/04/24
P02	1.4, 5, 7	Response to Review Comments 1	[redacted]	[redacted]	3/5/24
P03	2	Statement of %NBS	[redacted]	[redacted]	6/5/24
A01	None	Issued as final	[redacted]	[redacted]	13/1/25

Executive Summary

Scope and Basis of Assumptions

Robert Bird Group NZ Limited (RBG) has been engaged by Wellington City Council (WCC) to complete a DSA of the residential building at 16 Glenmore Street, Thorndon, Wellington.

The buildings is known as the Whare Ahuru Apartments (WHAA). The original construction was consented as a two-storey building in 1950, but an additional storey was added in 1974/5. The additional storey is made from reinforced masonry on a composite concrete slab, which is supported by gravity columns through the interior and concrete walls on three external faces. The presence of concrete on three sides results in a highly torsional response in the gap between the two constructions. No strengthening appears to have been carried out as part of the additional storey. The connection between the original building and the additional storey was strengthened in 2013, but calculations have shown this is largely ineffective. There is a partial diaphragm at Ground Level, and complete concrete diaphragms at First Floor and Second Floor.

The building is founded on a gently sloping site, with approximately one storey of height change over the length of the building.

Reinforced concrete cantilever walls are the building's only structural system for resisting lateral loads, extending the entire height of the building. The exterior walls have numerous holes for windows and doors, with fewer holes for the interior walls.

Beca conducted a peer review of the DSA. They were specifically asked for their comment after completion of the Initial Assessment Form (refer Appendix Appendix B), and at completion of the detailed calculations, but have been in communication throughout the process. The peer review process generally followed the Engineering New Zealand Guidance titled *Practice Note 2: Peer Review*.

Results Summary

Refer to Table 1 below for a summary of the %NBS scores assigned to the critical elements of each structural component.

The structural form of the building varies around its perimeter and up its height, and numerous elements have undesirable failure modes or poor load paths. We have identified the most severe weakness as the connection between the 1950 and 1975 buildings, but seismic retrofit concepts will need to address potentially poorly-performing elements throughout the building. In particular, the Glenmore Street elevation has potentially severe failure modes in the ground storey piers between garage doors, in the spandrels below First Floor windows, in the piers between the ground storey windows, at the plywood wall between the 1950 and 1975 buildings, and out-of-plane response in the Second Floor concrete walls.

The seismic rating for this building is governed by the torsional response of the connection between the 1950 original building and the 1975 additional floor. **WHAA has an overall seismic score of 15%NBS.** In our seismic rating system this is designated as "Grade E", with a hazard to life more than 25 times that of a new building.

This DSA has been carried out in accordance with the November 2018 revision of section C5 for concrete buildings of the 2017 New Zealand Society for Earthquake Engineering (NZSEE) document The Seismic Assessment of Existing Buildings. Strictly speaking, since this building has been found to fall short of the performance level described for an Earthquake Prone Building (EPB), only the original concrete guidelines from 2017 should be used. However, guidance from Engineering New Zealand has noted that changes made in the November 2018 revision are likely most affect buildings with precast floors, concrete frame structures, and concrete buildings with a reasonable ductile response. WHAA falls outside of these characteristics.

Hence, we have considered our results gained from considering the 2018 revision of section C5 reasonable to report.

Element	%NBS (IL2)	Commentary
Plywood Infill Wall	15 %NBS	Governed by anchor tension breakout in the 75EA posts
Second Floor Block Walls	20 %NBS	Out of plane parts response as a cantilever
First Floor Concrete Walls	30 %NBS	Out of plane parts response as a cantilever on Grid F
First Floor External Piers	40 %NBS	Varied flexural and shear governed
First Floor External Spandrels	25-30 %NBS	Flexure governed
First Floor Diaphragm	70 %NBS	
Ground Floor Internal Concrete Walls	45-50 %NBS	Flexure governed
Ground Floor External Piers	65-70 %NBS	Varied flexural and shear governed
Ground Floor External Spandrels	40-45 %NBS	Flexure governed
Garage Level Internal Concrete Walls	40-45 %NBS	Flexure governed
Garage Level External Piers	35-40 %NBS	Varied flexure and shear governed
Foundations	70 %NBS	Local bearing

Recommendations

There are numerous discrepancies between the structural drawings across the history of the building, and numerous areas where the final design is not clear. Cracking has been seen on the exterior in places where the cracks are more likely to be the result of corrosion than of previous seismic damage. To retrofit the building a detailed site-measure and condition assessment will be needed, especially identifying the exact setting out of the reinforced concrete walls inside the different units, as some of these walls are indicated away from any ground-storey supporting structure.

The large-scale mapping of soil classes shows this site on the boundary between Soil Class B and C. An investigation should be carried out by a Geotechnical Engineer to provide certainty on this. If the site is on Soil Class B that would increase its level of seismic compliance, but not enough to remove the possibility it is an EBP. The unfavourable modes of failure found during the DSA mean we would still recommend wholescale retrofit if the nominal %NBS increases as a result of geotechnical investigation.

The parts and components loading from NZS1170.5 has been used throughout the HUP2 assessments to determine the performance of out-of-plane walls, and this methodology has often shown that the out-of-plane response is a critical performance measure for the 1974 extension. A study should be carried out generically comparing these out-of-plane loads to the proposed revision to the loading standard TS1170.5:2023. If this study confirms that the out-of-plane response does govern the performance of that element, seismic retrofit is recommended.

Seismic Retrofit Concepts

Enhancing the seismic performance of this building will require large-scale works, effectively replacing poorly performing elements with new systems.

To reach 34%NBS, the link between the 1950 and the 1975 buildings will need to be completed by replacing the plywood wall on the Glenmore elevation and by extending the interior walls from First Floor to connect to the Second Floor with in-situ concrete walls. The ceilings in the upper storey will need to be replaced with a structural plywood diaphragm to provide out-of-plane support.

To reach 67%NBS, in addition to the work for 34%NBS, new shear walls around the outside of the building will be needed, with new foundations. Preliminary assessment concentrates the new walls along the Glenmore street elevation. These works create the opportunity to increase the window sizes along the elevation.

Glossary

Detailed Seismic Assessment (DSA)	A quantitative seismic assessment carried out in accordance with Part A and Part C of the Engineering Assessment Guidelines.
Design Features Report (DFR)	A document that details the important decisions and outcomes regarding the design of a structure, including any proposed strengthening works.
Earthquake-prone Building (EPB)	As explained in Section A5.1.1 of the Engineering Assessment Guidelines; a building or part of a building that will have its ultimate capacity exceeded in a moderate earthquake. Additionally, if the building or part of a building were to collapse, the collapse would be likely to cause injury or death or damage to other properties. Whether a building or part of a building is considered earthquake prone is decided by the territorial authority that oversees the district where the building is.
Importance Level (IL)	Categorisation defined in the New Zealand Loadings Standard, AS/NZS 1170.0:2002 used to define the ULS shaking for a new building based on the consequences of failure. A critical aspect in determining new building standard.
Initial Seismic Assessment (ISA)	A seismic assessment carried out in accordance with Part A and Part B of the Engineering Assessment Guidelines.
Ultimate Limit State (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings.
New Building Standard (NBS)	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)%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.
(New Zealand) Building Code	Section B1 of the New Zealand Building Code (Schedule 1 to the Building Regulations 1992).
Non-structural element	An element within the building that is not considered to be part of either the primary or secondary structure.
Secondary structural element	A structural element that is not part of the primary structure.

Contents

Appendices

1. Introduction

The following report is a Detailed Seismic Assessment (DSA) and follows the technical document 'The Seismic Assessment of Existing Buildings' which has been developed for New Zealand.

The focus of the DSA is to achieve an understanding of the likely behaviour of the building in earthquakes by quantifying the strength and deformation capacities of the various structural elements, by checking the building's structural integrity against the loads/deformations (demands) that would be used for the design of a similar building on the same site.

As part of this process we have assessed the structural load paths of the building, the capacities of the structural elements, the likely inelastic mechanisms in the building, the global building response to earthquake shaking and then assigned an overall earthquake rating for the building.

1.1 Scope of Assessment

Robert Bird Group (New Zealand) Limited (RBG) has been engaged by Wellington City Council (WCC) to complete seismic assessments and provide concept strengthening designs – if needed – for specific buildings within its housing portfolio. The purpose of this work is to upgrade WCC's housing portfolio to meet the seismic strength standard detailed in the Deed of Grant (Minimal Housing Standard) Programme.

As part of this programme, RBG's work scope entails completing a DSA on the standalone residential building at 16 Glenmore Street, Thorndon, Wellington. For the purposes of the assessment, the building will be referred to as the 'WHAA' building, with layout and directions as shown in the site aerial plan in Figure 1:

Figure 1: WHAA building

The WHAA building is a four storey reinforced concrete structure which was originally constructed in 1950 and currently contains 14 residential flats. It is formed on a sloping site from the rear of the building, which is only three storeys, down to the street side of the building. The building had an additional storey added in 1974 and was strengthened in 2013 by adding some plywood infill walls beneath the 1974 extension. The building elevation from Glenmore street is shown below in Figure 2, which indicates the various construction stages.

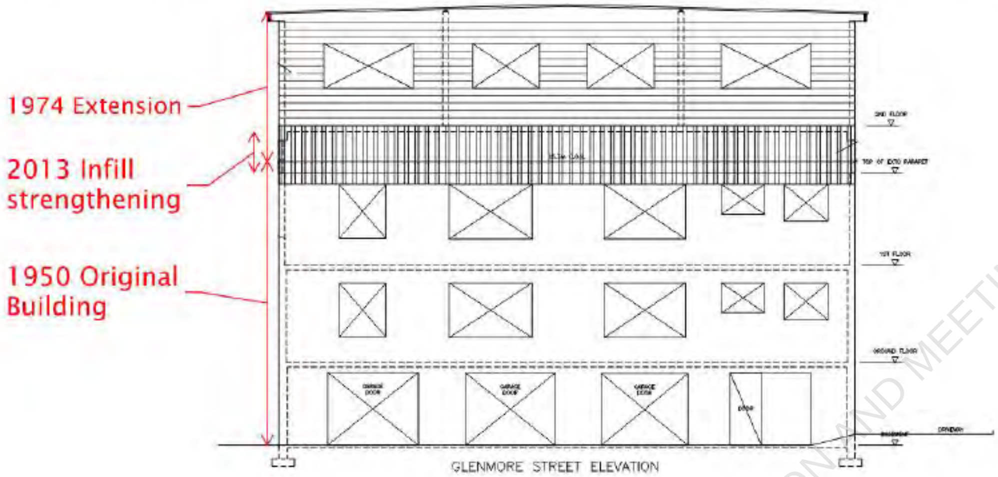


Figure 2: Front elevation of WHAA building

This DSA has been undertaken as part of Phase 2 of the Housing Upgrade Programme. The objective of this DSA is to establish the degree of seismic risk posed to WHAA. This assessment has been undertaken in accordance with the 'The Seismic Assessment of Existing Buildings', however it does not consider the November 2018 proposed revision to Section C5 because this report may be used to determine Earthquake-Prone Building status as per the Earthquake-Prone Building Amendment Act.

Two concept seismic retrofit schemes are included, showing viable options for improving the performance of this building two 34%NBS, and 67%NBS.

1.2 Regulatory Environment and Design Standards

EPBs are defined by the Building Amendment Act 2016 as buildings with ultimate capacities that are likely to be exceeded in a 'moderate earthquake,' hence posing a life safety risk to occupants. A 'moderate earthquake' is defined as one-third as strong, but of the same duration, as the shaking assumed when designing a new building. Thus, the lower threshold to designate a building as earthquake prone is referred to by the shorthand of "33%NBS". The New Zealand Society for Earthquake Engineering (NZSEE) recommends that buildings are strengthened to 67%NBS, and so this has become a widely adopted benchmark for performance.

The 2017 NZSEE Engineering Assessment Guidelines detail a method for assessing existing buildings against the contemporaneous building standards, especially NZS1170.5:2004. This benchmark of performance may not reflect changes in seismic design or assessment methodologies after 2017. This provides a way to rate existing buildings to understand the seismic risk posed to it relative to a new building in 2017. The primary focus of this procedure is life-safety risk. 'Probable' capacities and consideration of structural mechanisms that can form are allowed, provided these mechanisms do not constitute a significant life-safety hazard.

Territorial authorities (TAs) ultimately determine whether a building is earthquake prone. ISAs or DSAs prepared by engineers may be used by TAs to assist in this determination. TAs may request an engineering assessment from a building owner if the ISA process has flagged the building as potentially earthquake prone. In this case, the building owner will be given a timeframe to complete the assessment.

If a building has been identified by a TA as earthquake prone, that TA must issue an EPB notice that states the earthquake rating and deadline for completing seismic work on the building (amongst other items). For a 'normal' building in Wellington, this deadline typically entails 15 years. Buildings not identified as earthquake prone by a TA do not fall within the 2016 Building Amendment Act for EPBs. Hence, there is no legal obligation to strengthen such buildings.

Besides the 2017 NZSEE Engineering Assessment Guidelines, the following design standards were utilised in this DSA:

- NZS1170.0: 2002
- NZS1170.5: 2004
- NZS3101: 2006

1.3 Assessment Methodology

1.3.1 Procedure Overview

The DSA procedure adopted for this report is as follows:

1. Review existing information in the form of drawings, calculations, and specifications.
2. Complete an initial site visit to validate the current structure against the available documentation
3. Request site investigations where appropriate to confirm undocumented alterations if required
4. Establish the 100%NBS threshold by assessing the site seismic parameters and calculating the response spectra for the buildings.
5. Complete an initial simple lateral mechanism analysis (SLaMA) to understand the displacement and global ductility capacities of the buildings.
6. Calculate by spreadsheet the base shear demands and floor forces using the equivalent static analysis (ESA) procedure.
7. Model and analyse the buildings and individual components in either 2D or 3D using force-based procedures.
8. Complete structural calculations for key structural components.
9. Prepare a DSA report to summarise building component capacities, identify structural weaknesses, provide an overall %NBS score for the building.

The supporting calculations for this report have been peer reviewed by Beca

1.3.2 Sources of Information

RBG has been provided with limited architectural and structural specification and drawings of the 1950 building and 1974 extension, and structural drawings of the 2013 strengthening. These sources of information are more accurately described in Table 1 below:

Table 1: Sources of Information

Originator	Document	Date
Unknown	Original architectural and structural drawings, specification	1950
DeTerte & Kerr-Hislop	Structural extension drawings, calculations and specification	1974
Dunning Thornton Consultants	Seismic improvement drawings	2013
Concrete Structures Investigations	Independent Concrete Reinforcing Verification Report	2024

1.3.3 Loading Assumptions

Important permanent, superimposed dead loads and live loads used to calculate the seismic weight of WHAA are summarised in *Table 2* and *Table 3* below:

Table 2: Permanent loads for building assessment

Material	Permanent Load (G)
Lightweight Timber Roof	0.3 kPa
4No. 750L Roof Water Tanks	29 kN total
2" Composite Floor and Concrete Encased Steel Beams	4.9 kPa
5" Concrete Floor	3.2 kPa
4" – 5" Concrete Stair Flight and Landing	3.2 kPa
6" Concrete Walls and Lining	3.8 kPa
8" Concrete Walls and Lining	5.1 kPa
6" Partially Filled Concrete Block Walls and Lining	2.5 kPa
Timber Framed Wall and Lining	0.4 kPa

Table 3: Superimposed dead loads and live loads in accordance with NZS1170.1

Use	Level/Area	Superimposed Dead Load	Live Load (Q)
Residential Dwelling	1 to 3	0.4 kPa	1.5 kPa
Egress Corridors and Stairwells	1 to 3	0.1 kPa	4.0 kPa

The total seismic weight of WHAA was found to be approximately 9,500 kN. This weight was found considering a live load seismic combination factor of 0.3 and area reduction factor in accordance with NZS1170.0.

The seismic parameters used for calculating earthquake loads are outlined in *Table 4* below:

Table 4: Seismic parameters for building assessment

Parameter	Value	Notes
Design Working Life	50 years	Both the original building and the additional storey have exceeded their nominated design lives.
Importance Level	2	-
Site Subsoil Class	C Vs30 = 270 m/s	Based on WCC mapping Class IV in the Draft TS1170.5
Return Period Factor	1	-
Hazard Factor	0.40	Wellington
Near Fault Factor	1.0	A near-fault factor has not been considered due to the short period of the structure, but it is close to known faults and comment could be sought from a geotechnical engineer on potential local effects.
Period	0.4s in both directions	-
Structural Ductility and Performance Factor	μ 1.0, Sp 1.0	Diaphragms, foundation (ground beams, piles and pile caps) analysis. Shear walls with plain round bars.

1.3.4 Material Properties

Material properties used in assessment are based on the information in the architectural and structural construction drawings and specification, and in accordance with values outlined in Section C5 of the Engineering Assessment Guidelines. Refer to *Table 5* below for the adopted probable strengths used in the DSA calculations.

Table 5: Material probable strength for building assessment

Material	Probable Strength
Concrete	f'_c = 25 MPa
Reinforcing	f_y= 280 MPa f_u = 475 MPa

1.3.5 Modelling Philosophy

A 3D model was created on ETABS and subjected to lateral loads determined based on the seismic parameters outlined in *Table 4.*

The seismic load is calculated using ETABS automatically calculation function for ESA. A hand calculation was carried to validate the results from ETABS. The weight of the water tank is also included.

The stiffness modifiers for cracked section were assigned to all concrete members.

No gaps have been modelled between adjacent walls because the detailing shows well-detailed hook bars in all locations, providing anchorage and enforcing load sharing and displacement compatibility.

1.3.6 Structural Uncertainty

There are some ambiguities in the 1950 and 1975 drawings about the detailed setting out of the concrete walls inside the units. An overlay of the floor plans from the 1974 set shows unusual small offsets, and some walls with no apparent support. (The garage and first floor are each shown in blue below). For the purposes of this assessment all these walls are assumed to exist. Those walls provide lateral support, so if they are absent the loads will be higher in the remaining elements. A detailed site measure of the interior of the units will be needed to confirm the setting out of these walls.

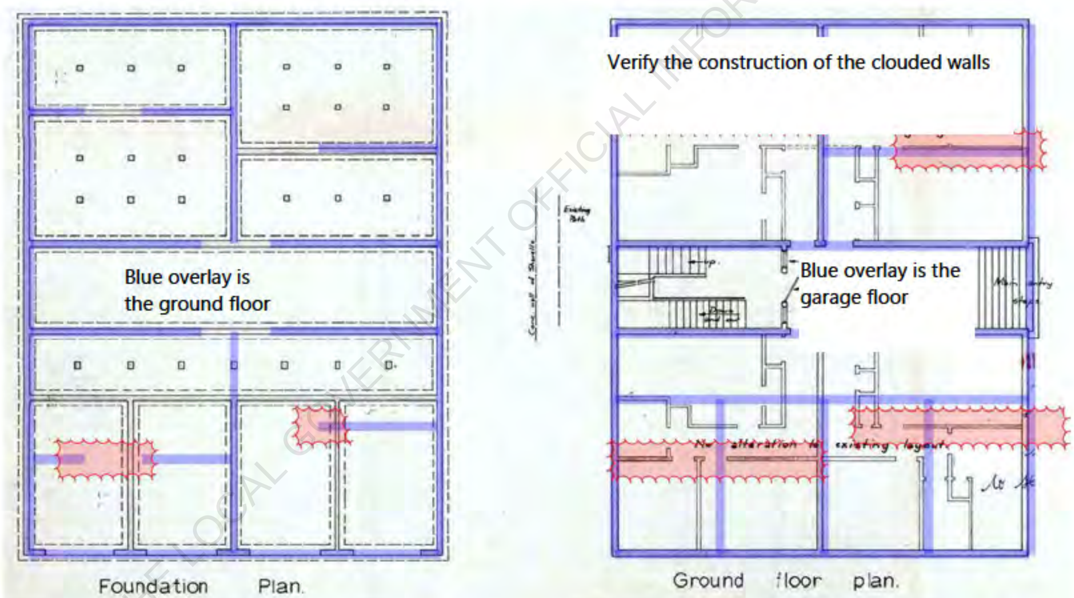


Figure 3: Extent of uncertainty

The construction of the "second floor support structure" is unclear, as conflicting information is shown in different areas of the 1975 drawings, and the 2013 strengthening plan does not show existing structure. The 1975 scheme showed a series of steel posts with masonry infill, but from the accessway into the area a different construction has been observed, as shown below. The detailed construction of the observed concrete elements needs to be confirmed with intrusive investigation. This is shown in more detail in the appendices.

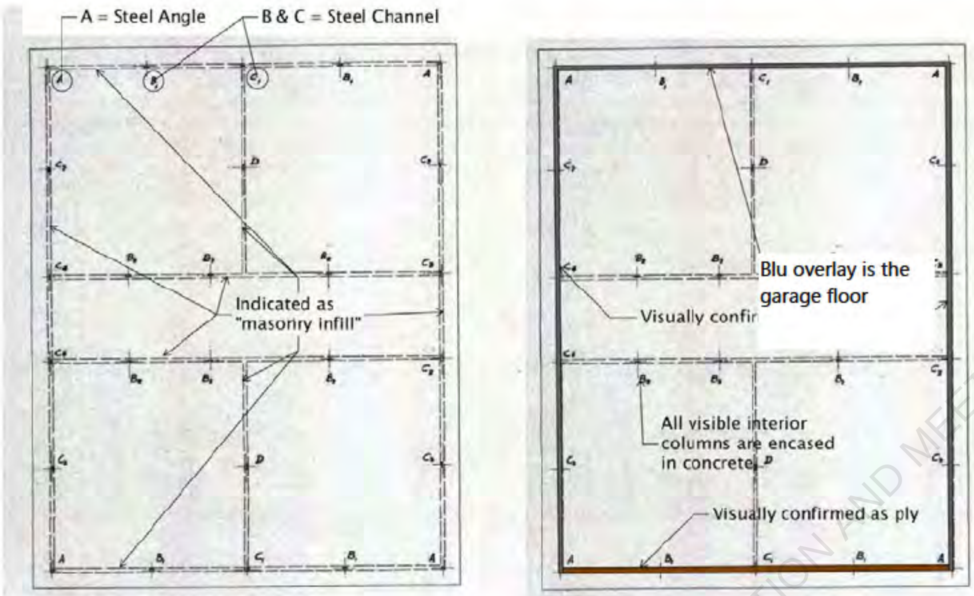
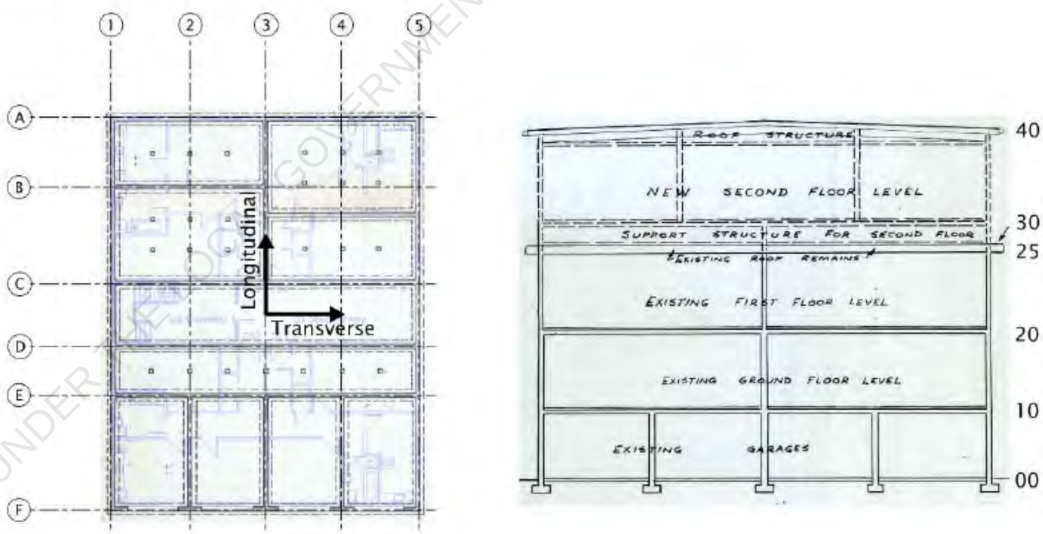


Figure 4: Extent of uncertainty

1.4 Building Description

The building is a three-storey concrete building. Concrete and reinforced blockwork walls form the primary vertical and lateral systems. The ground floor is partially timber and partially in-situ concrete, while the other two floors are concrete. The roof is lightweight timber.



KEY PLAN **KEY SECTION**

Figure 5: Naming Convention

The details of the primary structural system vary up the height of the building, as summarised in Figure 6 and elaborated in Figure 7. There are no interior walls that are continuous from the ground to the second floor roof, which potentially introduces transfers in the concrete diaphragms at every level. The presence and detailed setting out of the walls inside the units has not been confirmed, but the original drawings indicate wall locations at Ground and First that are offset from walls at Garage level.

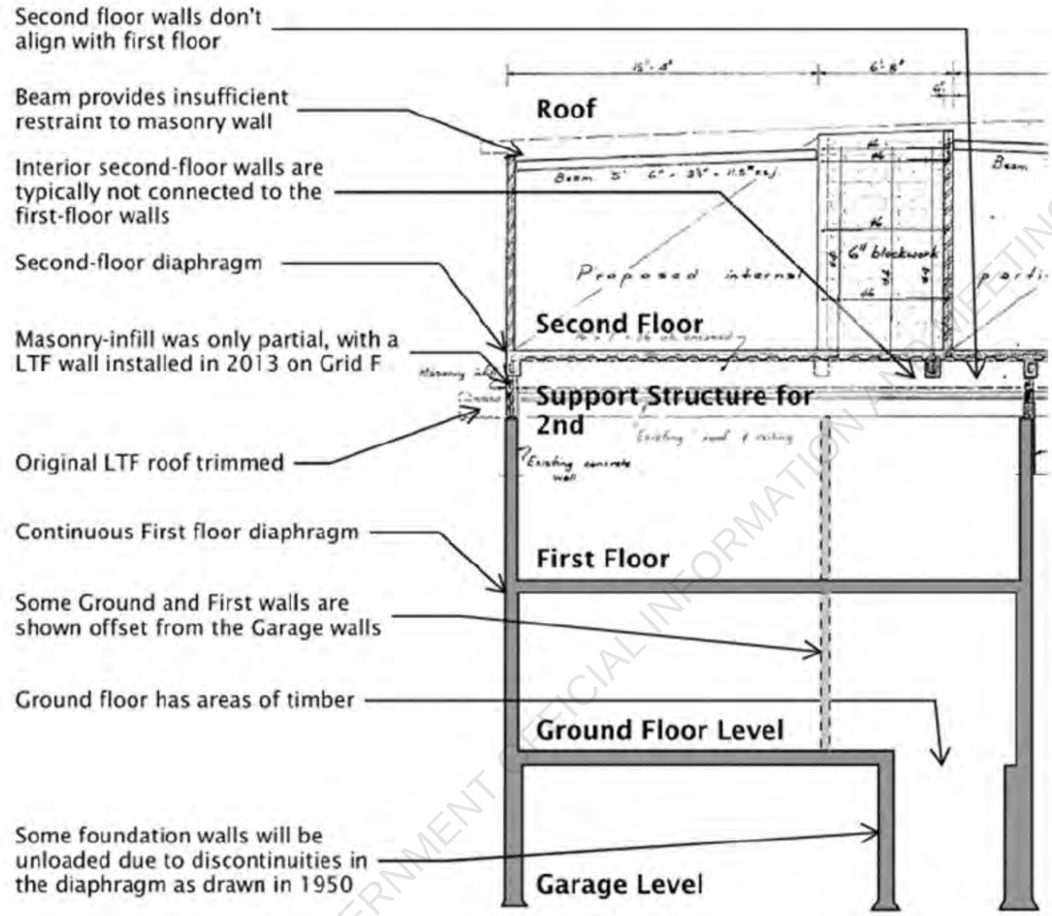


Figure 6: Structural system as it varies vertically

This means that the lateral load system is slightly different at each level and the wall thickness varies over the height, typically 8 in. at Garage and Ground and 6 in. at First Floor.

The general parameters for the building analysis are summarised in Table 6 below.

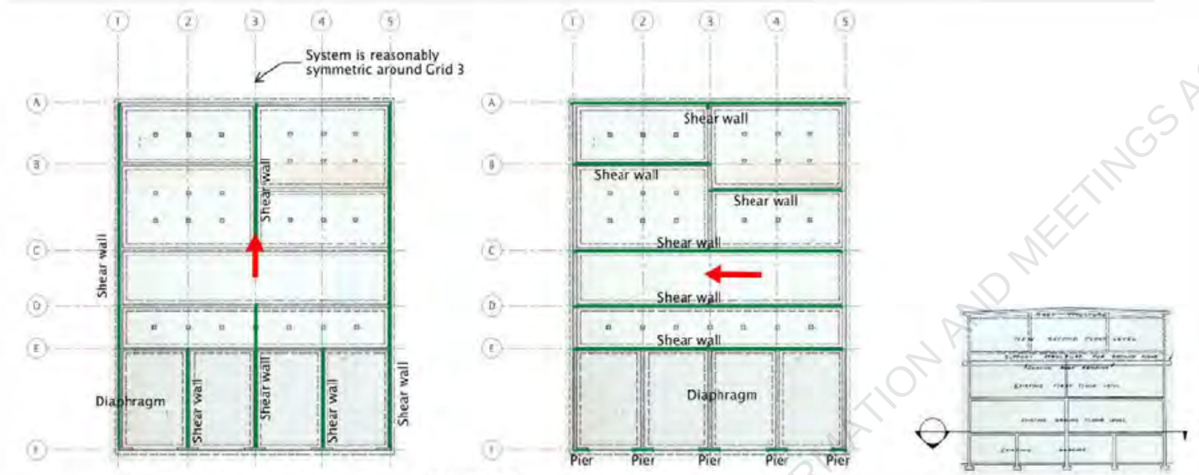
Table 6: Building Summary Information

Item	Details	Comment
Building name	WHAA	
Street Address	16 Glenmore Street	
Age	Original construction drawings dated 1950 Level 4 extension drawings dated 1974 Seismic Retrofit drawings dated 2013	
Description / Building Occupancy	Council residential flats	

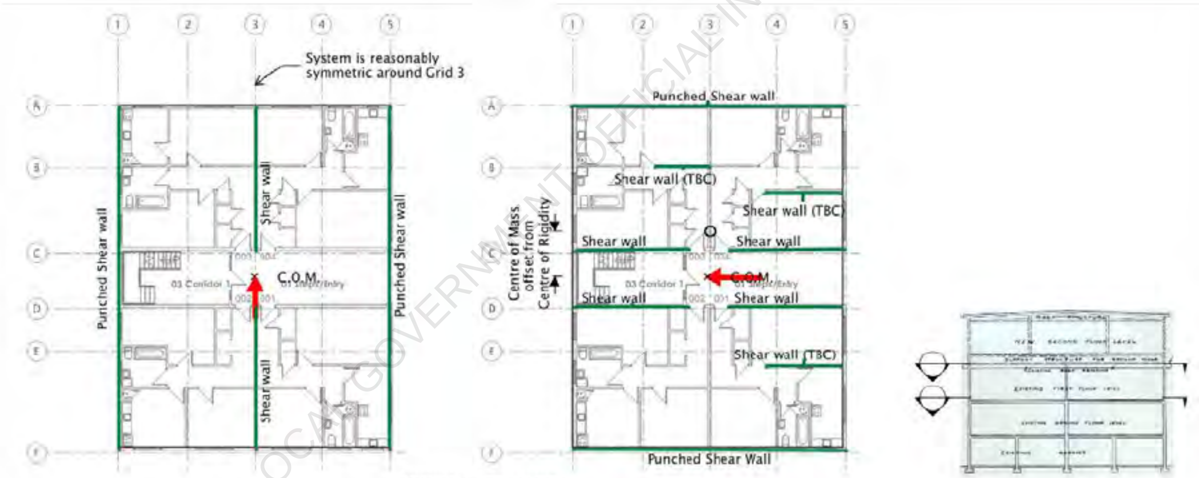
Importance Level	2	
Building Footprint / Floor Area	320	
No. of storeys / basements	4 level • Part-basement • Ground Floor • First Floor • Part-height storey / Second-storey extension • Second Floor	
Structural system	In-situ reinforced concrete shear walls up to the second storey, plywood-lined walls for the part-storey, reinforced masonry for the second storey.	The setting out of the concrete walls inside the units needs to be confirmed with a site measure. The construction details for the concrete support structure for the second floor need to be investigated.
Earthquake resisting system	In situ reinforced concrete shear walls in both directions. Note the extension level has reinforced masonry walls, and the extension diaphragm partially relies on plywood infill walls to transmit lateral load down to the original RC walls.	
Foundation system	Reinforced concrete shallow strip footings.	
Stair system	In-situ reinforced concrete	
Other notable features	Small water tank platform at roof level	
Past seismic strengthening	Plywood shear walls added in 2013 between the new Level 4 and original roof	
Construction information	Limited drawings and specifications for original 1950 construction and 1974 extension. Structural drawings for the 2013 strengthening.	
Likely Design Standards	Early NZSS 95 between 1935 and 1965 may have been followed. The 1974 extension likely followed NZS 1900 (1964), and the 2013 strengthening likely followed NZS1170.5 (2004).	
Heritage Status	None	
Seismic Risk Area		
Priority building status	None	
Other	None	

1.4.1 Structural Systems – Longitudinal and Transverse

The lateral system at each level is summarised below. Changes in wall locations and extents occur at the top of the garage and second floors. Access was not available to the interior of the units to confirm the presence or detailing of the interior walls, indicated below as "to be confirmed" (TBC).



GARAGE LEVEL



GROUND & FIRST FLOOR

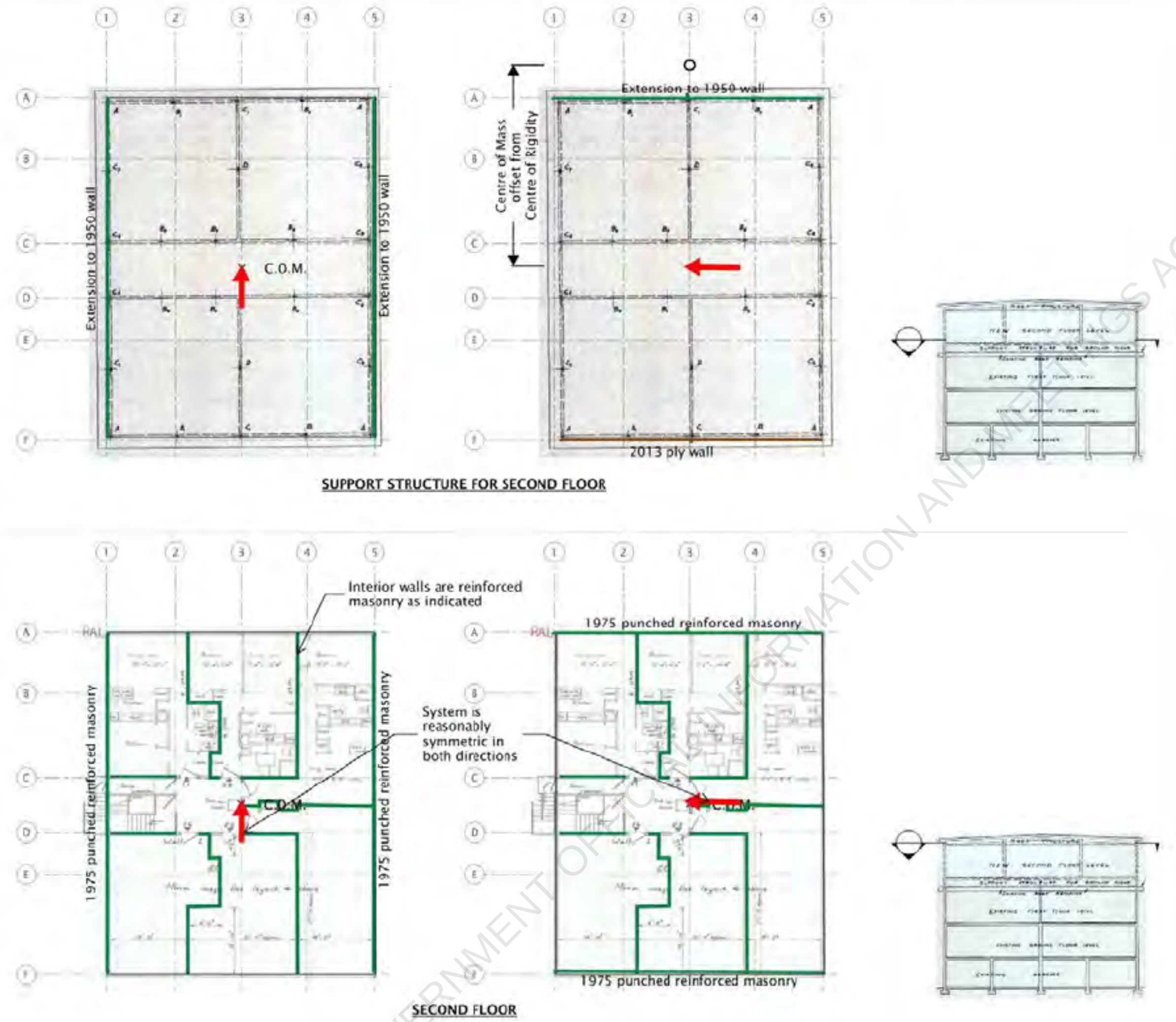


Figure 7: Structural system at each level

This variability in load paths requires computer modelling, since approximate hand methods would not accurately be able to evaluate the horizontal load transfers and relative stiffnesses correctly.

1.4.3 Dilapidation Comment

Cracks have been observed in the exterior of the building. The location of the cracks does not correspond well with cracking that might arise from previous earthquakes, so we infer that these cracks are due to corrosion of the steel. The building has exceeded its notional design life, so it is possible for corrosion to have happened. This can only be determined by further investigation, including breaking out areas with large cracks to examine the steel directly. Refer to annotations in Inspection Photographs.

1.5 Site Geotechnical Conditions

No geotechnical investigation has been undertaken at the time of this assessment. However, based on council mapping information, Soil Class C has been adopted. These are classified as 'shallow soils' and can vary between clays, sands and gravels. The QuakeCore mapping indicates the site to have a time-averaged shear wave velocity to 30m, Vs,30 = 270 m/s. It can be observed from the maps that the site is bordering the delineation between Class B and Class C. However, Class B is unlikely because NZS1170.5 indicates a shear wave velocity of Vs,30 > 360 m/s for Class B.

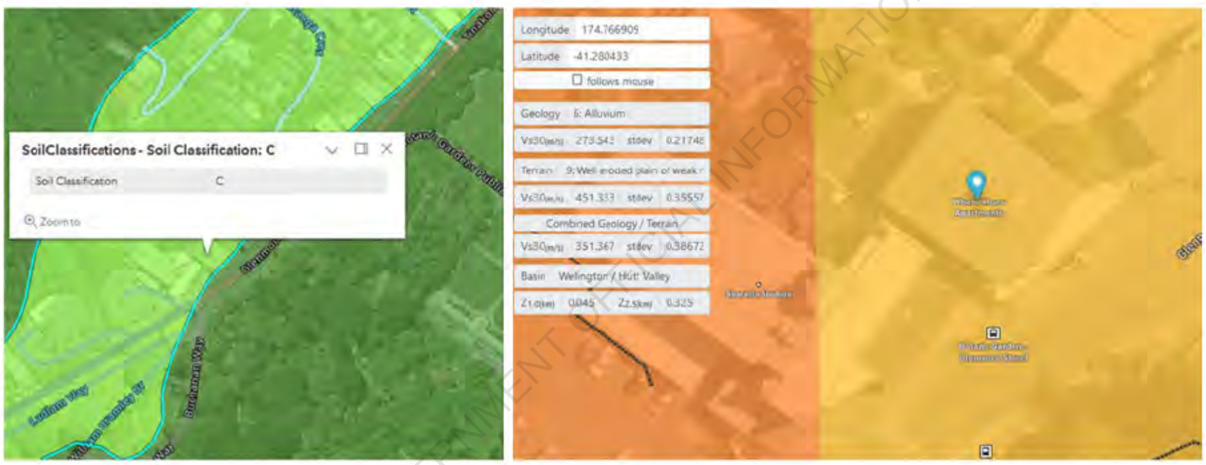


Figure 8: Geotechnical Context

1.6 Previous Assessments

There are no previous seismic assessments known to be undertaken at the time of writing this report. It is expected that some form of building assessment was undertaken to design the 2013 strengthening, but the only information made available from this was structural drawings alone.

2. Results of Seismic Assessment

The convoluted nature of the lateral load paths through this building is itself a major concern for its ability to withstand earthquake loading. Large-scale studies of building performance after earthquakes have generally shown that well-detailed regular structural arrangements perform better than irregular arrangements, even when irregular structures are designed for nominally greater loads. The ETABS modelling allows us to identify a probable hierarchy of element capacities for the seismic forces and displacements, and hence to estimate a level of shaking compared to the 2017 seismic design benchmark.

The analysis shows that the components of the Glenmore Street elevation limit the global capacity of the building. We have summarised the qualitative features of the model in Figure 9 below. The key features of this model are:

- The punched shear walls perform similar to a moment-resisting frame with rigid joints.
- In general the piers are stiffer than the spandrels, which will cause the rotations and damage to be concentrated in the spandrels.
- The shallow foundations don't provide restraint to rotation of the garage walls. They will pivot around the base. Typically free-standing walls cantilever from their base, but in effect these cantilever from the rigid structure above.
- The concrete floors will enforce deflection compatibility between adjacent piers at each level. This means the shorter (vertically) piers will experience relatively higher damage because any deflections will be a greater proportion of their length.

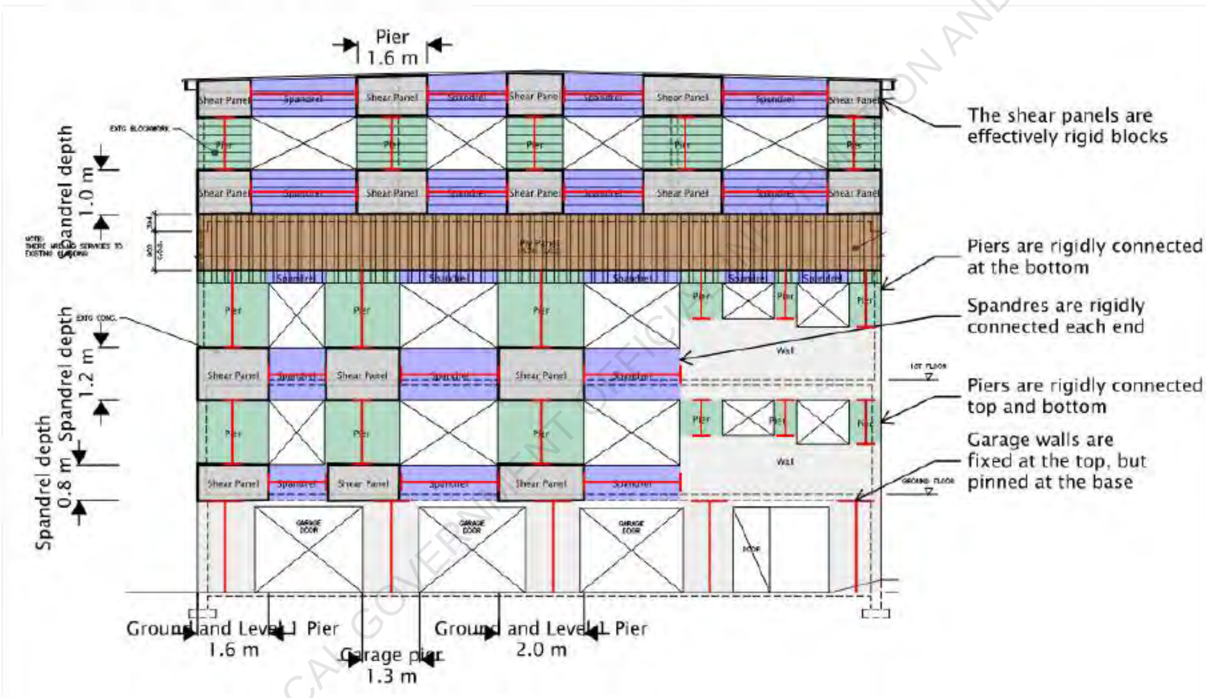


Figure 9: Glenmore Street Elevation Key Features

The ETABS modelling shows that the inherently torsional response at the second-floor support structure limits the performance of the system. The plywood allows relatively large displacements at the front of the building, in turn inducing second-order buckling in the wall stitching the 1950 building to the 1975 extension. A failure of this type in this location could be catastrophic, because the kinetic loads from the upper storey landing on the first floor will be very high.

The next-weakest elements in the system are the spandrel panels. These panels experience contraflexure as the structure tries to rotate. The spandrel panels try to restrain the rotation of the joints and the piers. Since there is no fixity in the walls at the bottom of the garage, once the spandrels fail there is nothing to restrain movement in the front wall. In a more typical building, where the base of the cantilever walls were fixed, the failure of a spandrel panel transfers load into those fixed bases, but here that is not possible.

Because they are not fixed against rotation at the base, and because they are shorter than the piers above, the garage walls are far more flexible than the structure above. They are supported against lateral movement by the concrete floor above the garages, but the garage concrete construction is also a torsional "C" shape. Once the garage walls begin to experience damage or significant rotation, they also begin to lose their ability to carry vertical loads, which would be potentially catastrophic..

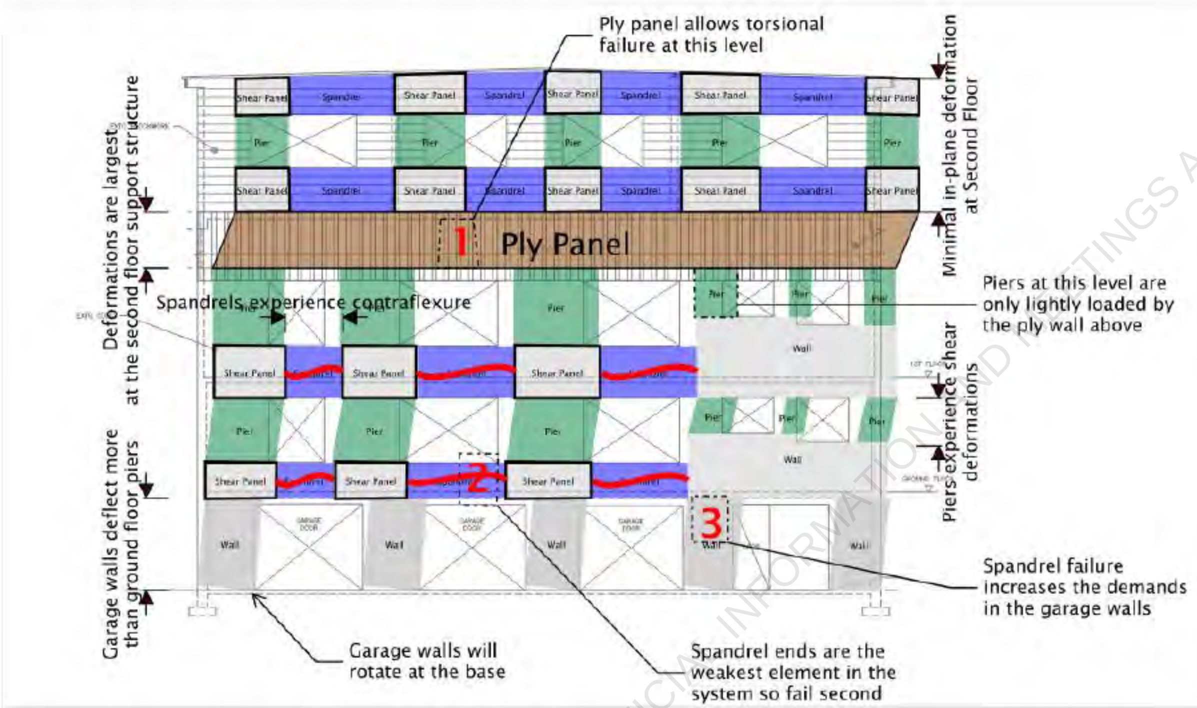


Figure 10: Qualitative results summary

The number and severity of these potential failure modes will require extensive retrofit.

This may be understood with reference to the different post-earthquake performance levels used by The Federal Emergency Management Agency (FEMA), as shown below. In Figure 11, the four performance states are explained. After low levels of shaking a building will ideally be operational or require only minor repairs. In NZBC the expectation is that at any level of shaking we achieve life safety for the occupants, even if a building is irreparably damaged. For a well-detailed modern building there is a substantial margin where increased shaking above the design level will not lead to collapse, but for WHAA that margin is small. Strengthening the building to 34%NBS may have only slight benefit if a larger earthquake happened. Therefore, while it may be possible to add strength to the system to achieve a good performance at 34%NBS or 67%NBS, it may be more effective to change the behaviour of the system by creating a more regular structural arrangement. This is discussed in more detail in Section 7 Concept Seismic Strengthening.

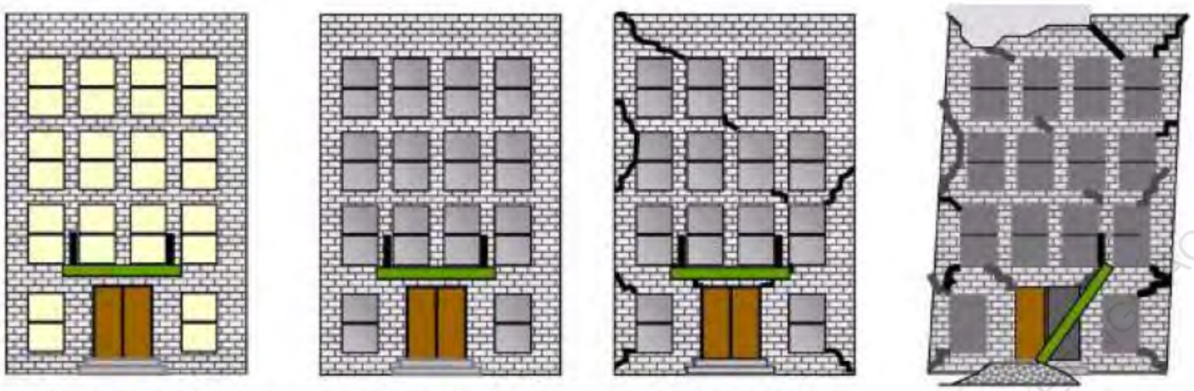


Figure 11: Extract from FEMA Earthquake Primer for Design Professionals

Table 7: Summary of Building Seismic Performance

Element	%NBS (IL2)	Commentary
Plywood Infill Wall	15 %NBS	Governed by anchor tension breakout in the 75EA posts
Second Floor Block Walls	20 %NBS	Out of plane parts response as a cantilever
First Floor Concrete Walls	30 %NBS	Out of plane parts response as a cantilever on Grid F
First Floor External Piers	40 %NBS	Varied flexural and shear governed
First Floor External Spandrels	25-30 %NBS	Flexure governed
First Floor Diaphragm	70 %NBS	
Ground Floor Internal Concrete Walls	45-50 %NBS	Flexure governed
Ground Floor External Piers	65-70 %NBS	Varied flexural and shear governed
Ground Floor External Spandrels	40-45 %NBS	Flexure governed
Garage Level Internal Concrete Walls	40-45 %NBS	Flexure governed
Garage Level External Piers	35-40 %NBS	Varied flexure and shear governed
Foundations	70 %NBS	Local bearing

The seismic rating for this building is governed by the torsional response of the connection between the 1950 original building and the 1975 additional floor. **WHAA has an overall seismic score of 15%NBS.** In our seismic rating system this is designated as "Grade E", with a hazard to life more than 25 times that of a new building.

3. Secondary Elements

The internal stairs from Ground to First are formed from in-situ concrete with no apparent seismic detailing for movement, but they also contain a landing at mid height that prevents an efficient strut forming between the storeys. The stairs are built into the walls on one side, and the connection is detailed showing a hooked bar which will allow them to transfer loads into the stairs. This is illustrated in the diagram below:

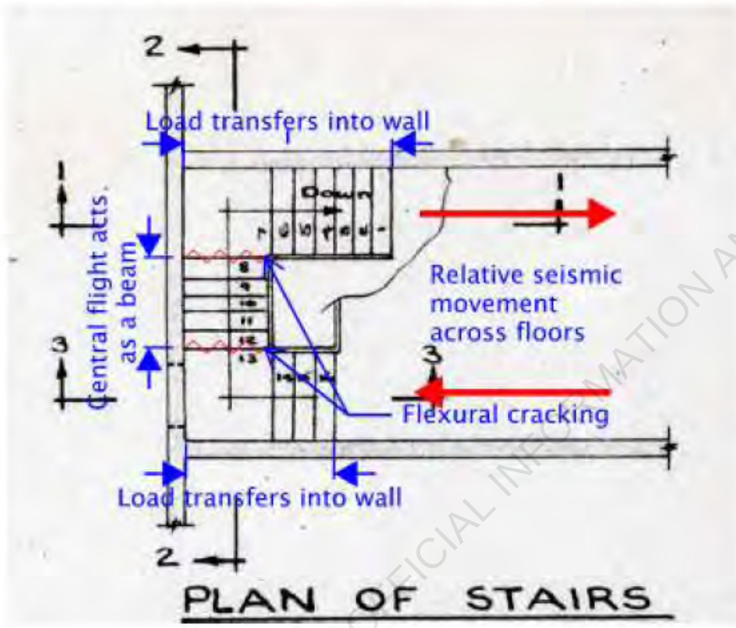


Figure 12: Stair load paths

This flexural cracking could make the stairs inoperable at higher levels of shaking, but is not likely to prevent use of the stairs at the levels of shaking which would cause primary structural damage. Due to the stiff connections to surrounding walls even high levels of shaking are unlikely to lead to the kind of buckling collapse that a direct stair flight could experience.

The concrete stairs represent a hazard to the occupants because once they are damaged it may make it difficult to evacuate the building. However, they are likely to perform adequately above levels of shaking that will cause one of the primary systems to fail, as described elsewhere in this report.

To enhance resilience at very high levels of shaking the concrete stairs with a flexible system able to accommodate the seismic movement as part of the retrofit works. This could be steel, or well-detailed timber.

4. Non-Structural Elements

There are water storage tanks above the roof. These have not been measured in detail, and no numerical work has been carried out on their performance. We were supplied photos of the general arrangement by WCC for this assessment.

There are four tanks of 750l each. In the 1974 drawings, they were intended to be supported on a substantial platform with an enclosure, but in reality they appear to be supported by two timber walls built above the 1974 block walls supporting a timber platform, refer to the photo below.

Figure 13: Roof tank support structure

From the photos provided, there is is a notional seismic restraint to the tanks on top of the platform, but this is likely to be inadequate for all but the lowest of seismic loads. The bolts connecting the platform to the support walls appear nominal, so the platform could become detached, and the joists for the platform appear to be unrestrained from rotation so can suffer rolling shear failures.

This platform will likely need to be redesigned as part of any seismic retrofit work for the building.

5. Risks from Adjacent Buildings

There are insignificant risks from adjacent buildings.

There is an existing retaining wall on the boundary of the site, which may support another building above. This wall is close to the end of its life, but is sufficiently far away from the building that it poses minimal risk.

6. Assessment of Seismic Risk

6.1 Seismic Risk and Performance Levels

This building has several elements with predicted poor performance that also have potentially catastrophic consequences. We have assessed the most severe of these weaknesses at 20%NBS (IL2), which corresponds to Grade D in the guidelines. Buildings with this level of seismic performance are potentially Earthquake Prone Buildings (EPB), though this determination must be made by the Territorial Authority.

Table 8: Relative Earthquake Risk

Building Grade	Percentage of New Building Strength (%*NBS*)	Approx. Risk Relative to a New Building	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

7. Concept Seismic Strengthening

Concept strengthening needs to address the weaknesses identified in the assessment calculations with two possible performance levels:

1. Ensure adequate performance for life-safety at 34%NBS as a minimum baseline to ensure this building is not potentially earthquake prone.
2. Ensure adequate performance for life-safety at 67%NBS as the client's preferred minimum level of performance.

An important aspect of the seismic assessing system which is not quantitively expressed is that the expectation is that buildings is unlikely to collapse when experiencing 100%NBS shaking.

All strengthening concepts here are caveated by the need to undertake more detailed investigations into the existing construction. The detailed information required is shown in the included investigation scoping sketches in this report, but in summary:

- The presence and detailed construction of concrete walls inside the units needs to be confirmed.
- The construction of the infill concrete walls, the Second Floor Support Structure, are completely unknown and must be thoroughly investigated.
- Numerous areas of cracking have been observed on the building exterior and interior, and intrusive investigation will be needed to confirm the cause and determine a remedial strategy in parallel with any strengthening.

7.1 Critical areas of seismic weakness in the primary system

The key parts of the building that require improvement are:

1. Plywood Wall Infill at Grid F
2. Out of Plane Restraint of Concrete and Masonry Walls
3. Perimeter RC Walls in-plane shear and flexure around openings
4. Improving the connections between the Second Floor and the First Floor
5. The concrete diaphragm at ground is incomplete

7.2 Concept 1 – Minimum additional structure

In this concept, additional structure is added to ensure that the critical elements identified in the DSA have enhanced performance above 34%NBS. The nature of some of these changes means that while it would be possible to fine-tune the scope of work to target exactly 34%NBS, there is minimal benefit compared to installing a more natural scope of work and achieving a higher %NBS.

1. Replace the plywood infill walls with reinforced concrete walls to remove the risk of gravity support loss from the plywood walls failing. This will also reduce the torsion effects on the mode shapes in the x-direction.
2. Connect the interior walls at First Floor to the underside of the existing Second Floor. This ensures as much load as possible is transmitted to the strongest elements in the building, reducing the load transfer into the front elevation from infilling the ply wall.

3. Add internal bracing and external strong-backs to the old roof level and new roof level to add a higher degree of out of plane restraint to the walls.

It would be possible to fine-tune the exact length of the additional connection to target 34%NBS, but completing the entire infill area will improve the performance at higher levels of shaking. It is likely that the enabling works will comprise a much larger proportion of the cost than the marginal difference of leaving out small areas of infill.

Figure 14: Effect of infill

The changes in pier shears are a function of two counter-balancing factors:

- Infilling the ply wall increases the load share into Grid F, because it is now more stiffly connected to the Second Floor
- Infilling the connections between First and Second floor and completing the diaphragm at Ground Level allows more load to be transferred into the stiff central walls.

This is evident from the increase in some First Floor piers, but a decrease below that, as load transfers away from Grid F to the interior grids at the diaphragm levels.

This scheme therefore significantly improves the performance below First Floor, but decreases the performance at First Floor, creating a requirement for localised strengthening to the central pier.

This scheme lifts the building performance to **50%NBS**.

Qualitatively, we would expect the building to experience irreparable damage, and there may be localised areas which could pose a risk to life safety.

7.3 Concept 2 – Demolish Second Floor

Seismic forces come from the inertia of building mass, so reducing the mass of a building also reduces its seismic loads. Demolishing the second floor entirely reduces the seismic mass considerably, but at the trade-off of a loss of amenity in the building.

This scheme lifts the building performance to **60%NBS**.

7.4 Concept 3 – Replace primary system on Grid F

The existing elevation on Grid F has a number of structural irregularities. Each irregularity in arrangement is a potential location for poor performance in higher levels of seismic load. In Concept 1, the main concept is regularising the overall building seismic system by transferring load to the central reinforced concrete walls. In this concept the irregularities in the front wall are addressed directly, so that regardless of the seismic load the system has a good seismic response profile.

1. Build new reinforced concrete walls attached the existing walls on Grid F and Grid 5 to effectively thicken the wall sections to a total 300mm thickness.
2. Remove or strengthen the spandrels that are failing in shear, allowing the front elevation to act as pure cantilever walls.
3. Install a concrete diaphragm between Grid D and Grid E at Ground Floor level.
4. Connect the interior walls at First Floor to the underside of the existing Second Floor. This ensures as much load as possible is transmitted to the strongest elements in the building, reducing the load transfer into the front elevation.

In this scheme it would be possible to achieve 100%NBS or better for Grid F, which would mean the performance of the building would become limited by other existing elements. Connecting the central walls to the Second Floor reduces loads in all the perimeter elements, improving the performance level of walls on the other critical face, Grid 5.

This scheme lifts the building performance to **75%NBS**.

8. Future Seismic Hazard

8.1.1 Revised National Seismic Hazard Model

In 2022, GNS Science released a revision of the National Seismic Hazard Model (NSHM), which is a set of updated guidelines for assessing the risk of earthquakes across the country. The model considers new scientific data and an improved understanding of seismic activity. It replaces the previous model developed in 2002.

The increase in seismic hazard anticipated with the revised NSHM in New Zealand varies depending on the location and type of earthquake. According to the Earthquake Commission and GNS Science, the expected increase in seismic hazard ranges from around 10% to 30% in some parts of the country, compared to the previous seismic hazard model. However, in other areas, such as the lower North Island, the increase in seismic hazard could be more significant, up to 50% or more.

The revised NSHM considers the likelihood of a major earthquake occurring in the Hikurangi subduction zone off the east coast of the North Island. This area is now considered to be at a higher risk of a large earthquake than previously thought, and the new NSHM reflects this increased risk.

Overall, the anticipated increase in seismic hazard with the new NSHM is significant and underscores the importance of ensuring buildings are earthquake-resistant and resilient.

MBIE is responsible for updating the Building Code in response to the NSHM. The Building Code sets minimum standards for building construction and design, and the updated code will reflect the latest seismic hazard information. The incorporation of the NSHM will require a determination from MBIE that will balance levels of risk and the cost/benefit of increasing seismic design loads.

As of February 2024, a draft Technical Specification TS 1170.5 has been released for feedback. TS 1170.5 is a result of Engineering New Zealand and MBIE collaborating to incorporate the 2022 revision of the NSHM into New Zealand's building regulations.

Engineering NZ has advised that the proposed Technical Specification will not affect %NBS scoring (and thus earthquake prone thresholds) as defined by EPB legislation effective from 1 July 2017, which relates NBS to the level of earthquake shaking. This does not necessarily reflect the future demands of building owners and tenants (or insurers) for a higher level of seismic strength/resilience, and this should be considered whenever reviewing seismic assessment information and/or strengthening advice.

8.1.2 Revised Site Spectra

Preliminary work using the draft Technical Standard shows that for this site the seismic loads will increase substantially, by around 45%, as shown in the figure below.

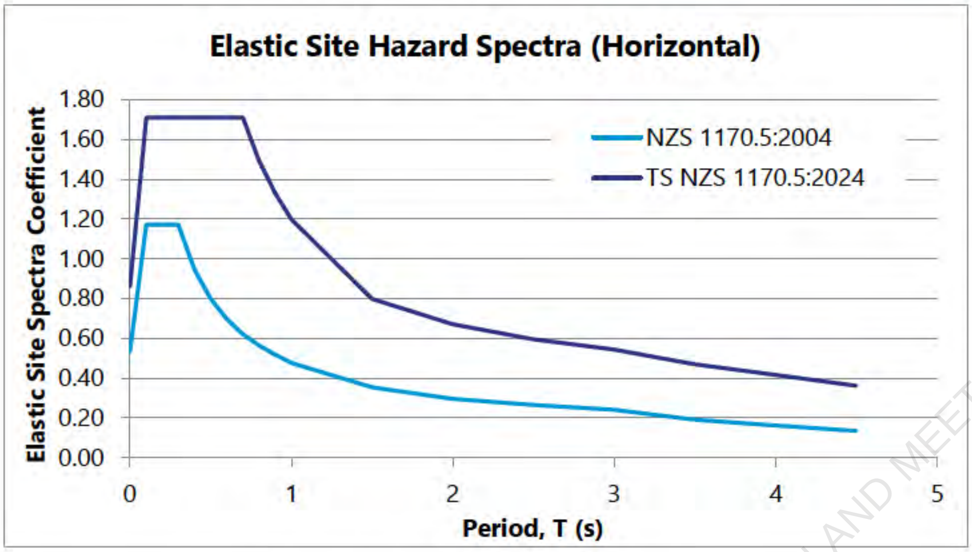


Figure 15 Revised Seismic Hazard Spectra

As discussed in Section 2, Results of Seismic Assessment, the seismic performance of buildings can be more influenced by structural arrangement than by detailed comparison of a force applied to the building versus capacities of individual elements. Strengthening WHAA to meet the notional target of 67%NBS (2017 benchmark) under current loading would result in a rating of 45% compared to the loads from the draft new standard (TS 1170.5:2023). For this reason, we recommend modes of seismic retrofit that will change the behaviour of the building rather than simply adding strength.

Appendix A
Sources of Information

HUP2-T0-Seismic Assessments

A-1 Property Documents

Refer to Table 1: Sources of Information.

A-2 Standards and Guidelines

Refer to 1.2, Regulatory Environment and Design Standards.

Appendix B
Initial Assessment Form

HUP2-T0-Seismic Assessments

AECOM Beca aurecon RobertBirdGroup

0 DOCUMENT CONTROL

N0541-RBG-WHAA-XX-DN-ST-00001

	Issue/Amendment	Author	Approver	Date
A	For Peer Review	s7(2)(a) [redacted] s7(2)(a) [redacted]	s7(2)(a) [redacted]	25.01.24
B	Included in DSA	s7(2)(a) [redacted]		24.04.24

1 SEISMIC ASSESSMENT – INITIAL REVIEW FORM

The purpose of this document is to provide a record of agreed initial parameters for a seismic assessment project.

Building Name:

WHA – 16 Glenmore Street

Structural Description: Describe the building	
Building Age/Year Constructed	Original construction drawings dated 1950 Level 4 extension drawings dated 1974 Seismic Retrofit drawings dated 2013  1950 Long-Section 1974 Cross-Section 2013 Strengthening
Previously strengthened? Y/N	Yes – ply shear walls added in 2013 between the new Second Floor and original roof
Location	Thorndon, Wellington
No. levels	4 level • Part-basement • Ground Floor • First Floor • Part-height storey / Second-storey extension • Second Floor
Plan Area (sq.m.)	320
Structural Form	Garage - In-situ reinforced concrete shear walls with partial diaphragm First – In-situ reinforced concrete shear walls with partial diaphragm Support structure for Second – In-situ reinforced concrete on three sides and reinforced concrete on the Glenmore elevation Second – Reinforced masonry with LTF roof
Roof Type	Light timber framing
Floor Type	In situ reinforced concrete
Foundation Type	Reinforced concrete shallow strip footings
Stair Type (Precast, Steel, etc)	Ground to First Floor - In-situ reinforced concrete First to second – LTF
Seismic Gaps (mm)/Pounding	N/A

Appendages/Parapets/Canopies	None
Precast Walls (reo type)	Critical ground floor walls are typically 225mm thick with 10mm bars at 300mm spacing each way, each face.
Veneers Present	No

Lateral Load-Resisting Mechanism (in each direction - confirm with drawings): Describe the lateral load resisting system in each direction	
Longitudinal:	In situ reinforced concrete shear walls
Transverse:	In situ reinforced concrete shear walls

Assessment Methodology
List components and proposed analysis method e.g. eqv static, pushover, modal analysis, rocking, force based, displacement based, part and portions, tributary area, flexible/rigid diaphragms

Type of analysis method:

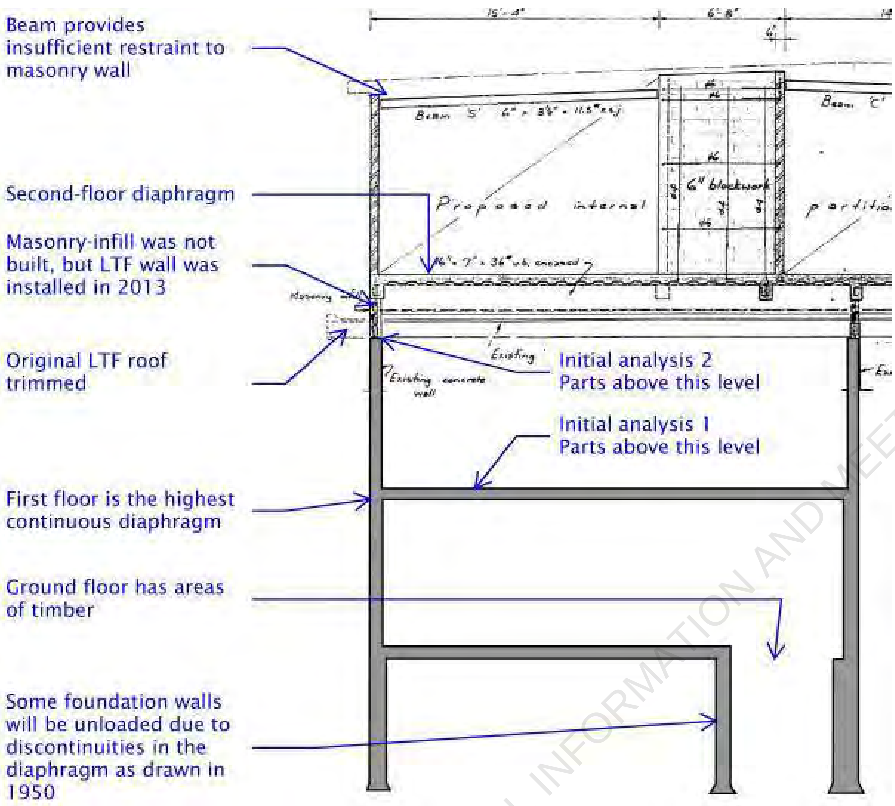
Initial investigations:

- Equivalent Static Analysis for the 2013 building
  - The presence of reinforced concrete connecting the 1950 and 1975 storeys means that the building has a severe vertical-system discontinuity, but does act as a single structure rather than one structure on top of another.
  - Initial check of force distribution through the walls using a rigid diaphragm assumption for each of the first and second floors.
- Parts and Portions for the Second Storey
  - Check masonry walls out-of-plane, assuming they are not effectively restrained by LTF roof

These analyses will provide minimum and maximum design shears for the critical structural elements:

- Masonry walls out-of-plane
- Strengthened part-storey, especially the components of the ply wall
- Primary walls at lower storey

Based on these analyses we can undertake shear and bending capacity checks for the primary elements, providing a realistic range of performance for the ETABS modelling.



Without prejudicing the results of this initial seismic investigation, the following results seem plausible:

- The performance of the building could be limited by the performance of the ply strengthening. This has three main potential weaknesses:
  - Brittle connections between the ply and concrete,
  - Inadequate stiffness, allowing high accelerations for the second floor,
  - Inadequate strength of wall components.

Detailed Analysis:

- 3D Equivalent Static Analysis
  - Models will be built for the cases as described above in the initial stage.
  - The lumped-mass models can be used to estimate floor acceleration spectra for comparison with parts loading in assessing the second floor.

Analysis method of diaphragms

In the initial assessments the diaphragm forces will not be considered directly. Assume rigid diaphragms for RC slabs and flexible diaphragm for both timber roof planes.

A first analysis using Pseudo-Equivalent Static Analysis will be carried out for the first floor. The second floor is formed with a steel grillage throughout and relatively short spans, and so by inspection does not govern the seismic performance of the system at that level.

The second-floor diaphragm is also likely to experience significant vertical design actions from wall overturning forces so a stick-frame model may be needed to assess the bending capacity of the cast-in steel grillage.

Initial Assessment of Ductility List the components of the structural system and the expected ductility to be achieved from them, eg plain round bar reinforced concrete moment frame ductility 1 – 1.25 or rocking	
Centrally reinforced concrete block walls	1.25 (deformed bars noted in the 1974 specification)
Centrally reinforced concrete walls	1.25 (plain bars assumed, but the detailing shows hooked laps throughout)
Double reinforced concrete walls	1.25
Steel connecting braces	1.25
Plywood shear walls	1.0 (Effectively elastic) The ply system in isolation could develop a relatively high ductility (μ>3), but the deflections associated with this level of ductility being activated would potentially create unacceptable accelerations from the storey above, and potentially second-order effects. Connections will be checked for overstrength forces from the ply; it expected these will limit the performance of the structure to an effectively elastic response.

Assessment Loadings: Loads to be used as part of assessment:	
Seismic Loadings	
Building Importance Level:	2
Site Subsoil Class:	C
Annual Probability of Exceedance:	1/500
Return Period Factor, Ru:	1.0
Near Fault Factor, N(T,D):	1.0
Hazard Factor, Z:	0.4
Code of the Day:	Early NZS between 1935 and 1965
S_p	1.0
Design Working Life (yrs):	50
Dead Loads/Superimposed Dead Loads	
Timber roof	0.2 kPa
6.5 inch RC composite floor slab	4.5 kPa DL, 0.6 kPa SDL
5 inch RC slab	3.2 kPa DL, 0.6 kPa SDL
Live Loads:	
Self contained dwellings	1.5 kPa
Balconies	2.0 kPa
Stairs & Corridors	4.0 kPa
Deflection Criteria	

ULS Deflection Limit (%)	TBC
Reason for Limit	

Material Properties:				
Material Rename material as appropriate		**Design Strength (MPa)**	**Strength Mod Factor**	**Assessment Strength (MPa)**
Reinforcement	Plain or Deformed bars?	Assume plain until confirmed on site		
	High Tensile (HT)			
	Medium Tensile (MT)	250	1.15	287
	Mild Steel (MS)			
Concrete	Foundations	15	1.5	22.5
	Slab on Grade	15	1.5	22.5
	Precast Panels or Shear Walls	15	1.5	22.5
	Columns			
	Beams			
Structural Steel	Beams	250	1.15	287
	Columns	250	1.15	287
	CHS			
	Plate	250	1.15	287
	Other members	250	1.15	287
Bolts				Assume 4.6/S
Weld Strength				Assume GP welds

Stiffness Reduction Factors in ETABS software:	
Columns Lower floors	TBC
Columns Upper floors	TBC
Beams	TBC
Walls	TBC
Diaphragms	TBC

Foundation Assessment Criteria:	
Geotechnical Report Available?	No
Foundation type:	Shallow concrete strip footings
Soil type:	Class C from WCC mapping; note – the site is on the border of B/C. From QuakeCore mapping Vs30 = 275 m/s
Geotechnical Investigation:	None
Ult. Bearing Pressure:	TBC
Sliding Resistance:	TBC

Pending Code/Guideline Changes to Take into Account :
Are there any upcoming code changes to take into account?

- TS1170.5 revises the soil class and parts loading.
    - Given the Vs30 value, it is likely that the site will not be worse than the assumption made for this analysis.
    - The parts loads are likely to be lower in TS1170.5, which would give a higher nominal %NBS for the assumed-critical ply load. However, the failure modes anticipated for the ply wall are not preferred (brittle connection failures), and so even if a higher number were reported, we would likely still recommend strengthening to improve the behaviour mode.

Kick-off Meeting:
Record minutes of the kick off meeting here, including key actions for people

Task / Note	Actioned By Who?

Additional Project-Specific Issues to take into account
E.g. Beam elongation, non-ductile mesh connection, minimal flexural steel, fracture issues, eccentric floor plate, bar anchoring, insufficient seating, unusual site characteristics, poor detailing

The 1974 specification calls for welding new galvanized bars and bolts to the 1950s reinforcing. The performance of welding between galvanised and mild steel members needs to be researched.

The 2013 strengthening shows relatively shallow post-fixed anchors into cover concrete; some research will be needed into the detailed performance of this fixing in cyclic loads. Since 2013 there has been a move to a more robust testing regime from the EU and it is possible that fixings which complied in 2013 will no longer comply.

Additional Project-Site Investigation Scope

The Basement level has had significant extensions not reflected in the 1950 or 1974 drawings. The new openings created to provide light into the extended spaces are shown on the 2013 strengthening scheme. There are also discrepancies between some details in the 1950s drawings for walls interior to the Ground Floor walls.

- Site measure of the as-build basement and potentially ground floor
- Drilling is needed to confirm the thickness of new basement walls
- Non-intrusive scanning is needed to confirm the reinforcing in any basement walls and confirm which interior walls on the ground floor are concrete inside each unit

A separate scoping sketch has been prepared showing this work.

The documentation provided via WCC archives for the 2013 strengthening is not as detailed as typical for the era. If more materials cannot be retrieved from the engineer then it may prompt further investigation scope.

Appendix C
Inspection Photographs

HUP2-T0-Seismic Assessments

Glenmore Street elevation

Rear elevation

Driveway elevation

Cracking at Glenmore/Driveway Corner (Grid F/5 intersection)

This may indicate advanced corrosion in the original reinforcing steel.

Cracking on Grid 1 pier.

This may indicate advanced corrosion in the original reinforcing steel.

Typical condition of visible interior concrete at garage level.

Condition of the garage wall piers

Typical interior pile and general view of rear foundation wall

Typical interior concrete foundation wall

Interior of plywood wall on Glenmore Street elevation and cast-in steel beam

The tanks at roof level are supported on a timber platform with no apparent seismic system.

Appendix D
Assessment Summary

HUP2-T0-Seismic Assessments

A-3 Engineering Assessment Summary

The below summary tables are presented as per MBIE report guidelines:

1. Building Information	
Building Name/ Description	Whare Ahuru Apartments (WHAA)
Street Address	16 Glenmore Street, Thorndon
Territorial Authority	Wellington City Council
No. of Storeys	Four (4)
Area of Typical Floor (approx.)	310 m^2
Year of Design (approx.)	Original construction 1950 (Garages, Ground and First Floors) Third storey added 1974 (Second floor)
NZ Standards designed to	NZSS 95:1935
Structural System including Foundations	Shallow foundations Reinforced concrete shear walls Concrete diaphragms at First and Second Floors
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 ground is assumed to be historic colluvium, placed as part of the formation of the Tinakori hills. The rear of the site has a retaining wall with a steep slope above, so we assume that the rear of the building is placed on cut, while the front is approximately at street level which may be original ground. No significant settlement was apparent during our site walk-throughs, so we infer that the ground has adequate bearing capacity for in-service loads. The site is near the boundary between Soil Class B and C, so this should be investigated before any retrofit work is carried out.
Previous strengthening and/ or significant alteration	In 1975 an additional storey was added, comprising a composite-steel floor with steel gravity columns connected to the First Floor walls. A concrete wall was added to three elevations connecting the two. The Glenmore Street elevation was apparently left with purely gravity connections. In 2013 the LTF wall connecting the 1950 and 1975 buildings on the Glenmore Street elevation was strengthened. A plywood wall with new fixings was built along this frontage to provide torsional restraint to the front edge. This strengthening appears to be largely ineffective.
Heritage Issues/ Status	None.
Other Relevant Information	Cracking has been observed in several places around the façade. These cracks aren't in the likely areas for seismic cracking and so likely represent local corrosion. All these areas will need to be broken out and the condition investigated. If they are caused by corrosion then a more extensive condition

	report of the exterior walls should be carried out to determine the full extent of any problems.

2. Assessment Information	
Consulting Practice	Robert Bird Group
CPEng Responsible, including: • Name • CPEng number • A statement of suitable skills and experience in the seismic assessment of existing buildings[1]	[redacted] Alasdair has 23 years' experience in structural engineering. He has been engaged in seismic assessment and retrofit projects since 2006.
Documentation reviewed, including: • date/ version of drawings/ calculations[2] • previous seismic assessments	Refer to *Table 1: Sources of Information* Original drawings, 1950 Extension drawings, 1974 Strengthening drawings, 2013
Geotechnical Report(s)	None
Date(s) Building Inspected and extent of inspection	Site visits were carried out on 9/1/24 and 6/3/24.
Description of any structural testing undertaken and results summary	At the site visit of 6/3/24 concrete scanning was carried out to confirm the size and spacing of reinforcing in several locations in the ground storey. This determined that the drawings were reasonably accurate.
Previous Assessment Reports	No previous seismic assessments are available, nor any calculations or other supporting material for the 2013 strengthening.
Other Relevant Information	

3. Summary of Engineering Assessment Methodology and Key Parameters Used	
Occupancy Type(s) and Importance Level	Multi-unit residential Importance Level 2

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

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

Site Subsoil Class	C
For an ISA:	
Summary of how Part B was applied, including: • Key parameters such as μ, S_p and F factors • Any supplementary specific calculations	
For a DSA:	
Summary of how Part C was applied, including: • the analysis methodology(s) used from C2 • other sections of Part C applied	The load paths for this building are very complex, since there are numerous part-height shear walls, punched openings in shear walls, partial diaphragms, and a highly irregular connection between an original three-storey building and a fourth storey. Given the complexity, a complete building model was made using ETABS. MRSA and ESA were carried out for the elastic load distribution. All elements were checked against their elastic loads, and then post-elastic performance was evaluated to confirm whether elements were shear or moment governed, and whether any ductility could be developed. The out-of-plane responses of the walls were checked using the methods of 1170.5.
Other Relevant Information	None

4. Assessment Outcomes	
Assessment Status (Draft or Final)	Draft
Assessed *%NBS* Rating	20%NBS
Seismic Grade and Relative Risk (from Table A3.1)	Grade E
For an ISA:	
Describe the Potential Critical Structural Weaknesses	
Does the result reflect the building's expected behaviour, or is more information/ analysis required?	Yes – the ISA is sufficient Or No - a DSA is recommended[3]

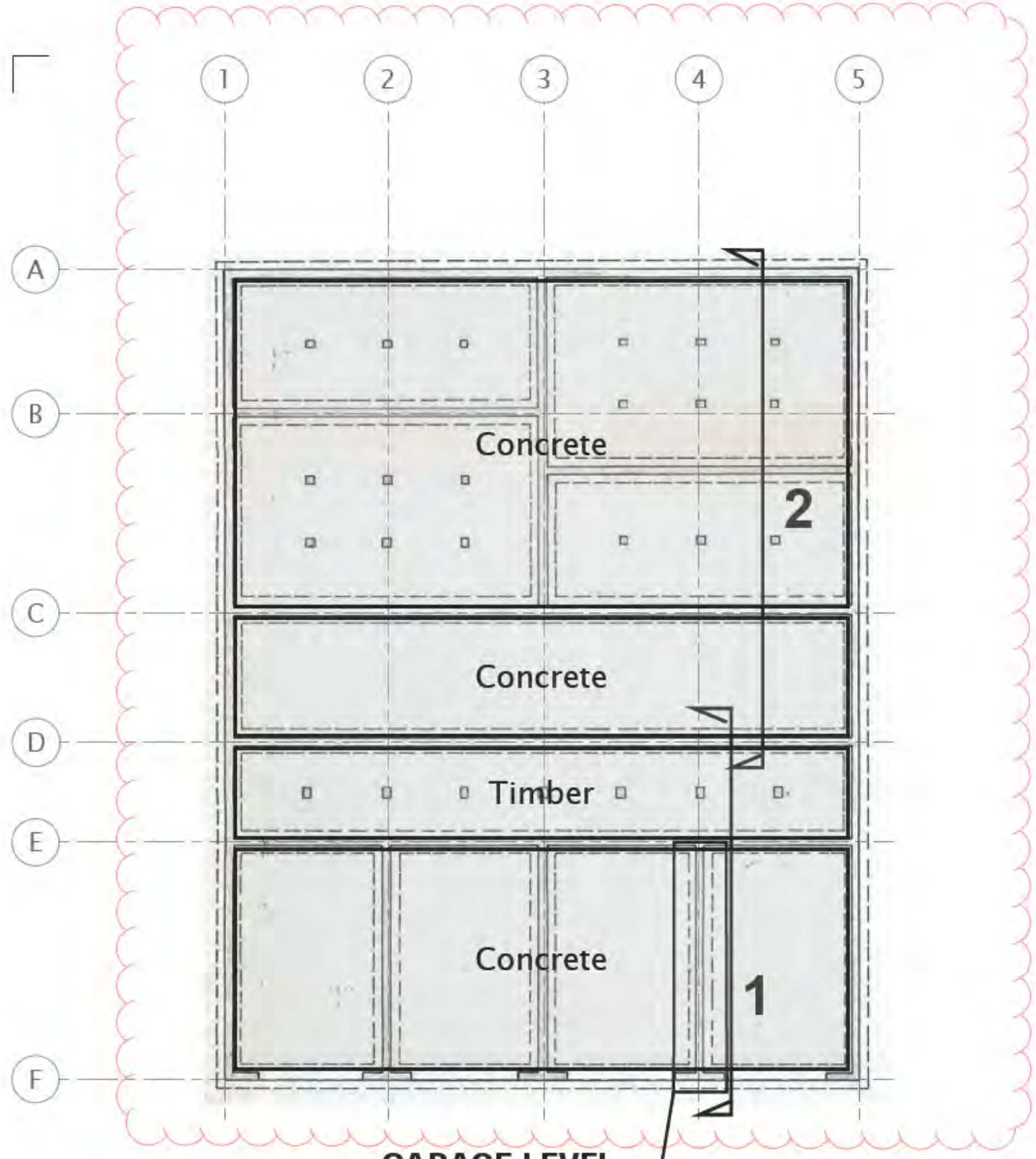
[3] Indicate what form should the DSA take/ what the specific areas to focus on are

If the results of this ISA are being used for earthquake prone decision purposes, <u>and</u> elements rating <34%*NBS* have been identified:	**Engineering Statement of Structural Weaknesses and Location**	***Mode of Failure and Physical Consequence* Statement(s)**
<u>For a DSA:</u>		
Comment on the nature of Secondary Structural and Non-structural elements/ parts identified and assessed	Water tanks have been installed at roof level. These are on a raised timber plinth which appears to have no lateral load system at all. The tanks could become unstable and fall onto the roof in a moderate earthquake.	
Describe the Governing Critical Structural Weakness	The governing weakness is the connection between the 1950 building and 1975 additional storey. This connection has several potentially brittle and low-strength aspects: • The connections between the singly-reinforced 1950s walls and the concrete infill are completely unknown. They may have inadequate embedment, are likely in confined concrete, and there may have been damage to the walls from installation of the connections. • The resulting shape is highly torsional, which concentrates out-of-plane displacements at the front of the walls on the Glenmore Street elevation.	
If the results of this DSA are being used for earthquake prone decision purposes, <u>and</u> elements rating <34%*NBS* have been identified (including Parts)[4]:	**Engineering Statement of Structural Weaknesses and Location** Connection between the 1950 and 1975 construction.	***Mode of Failure and Physical Consequence* Statement(s)** Loss of gravity support to the top storey, leading to general structural collapse.
Recommendations (*optional for EPB purposes*)		

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

Appendix E
Seismic Retrofit Concepts

HUP2-T0-Seismic Assessments



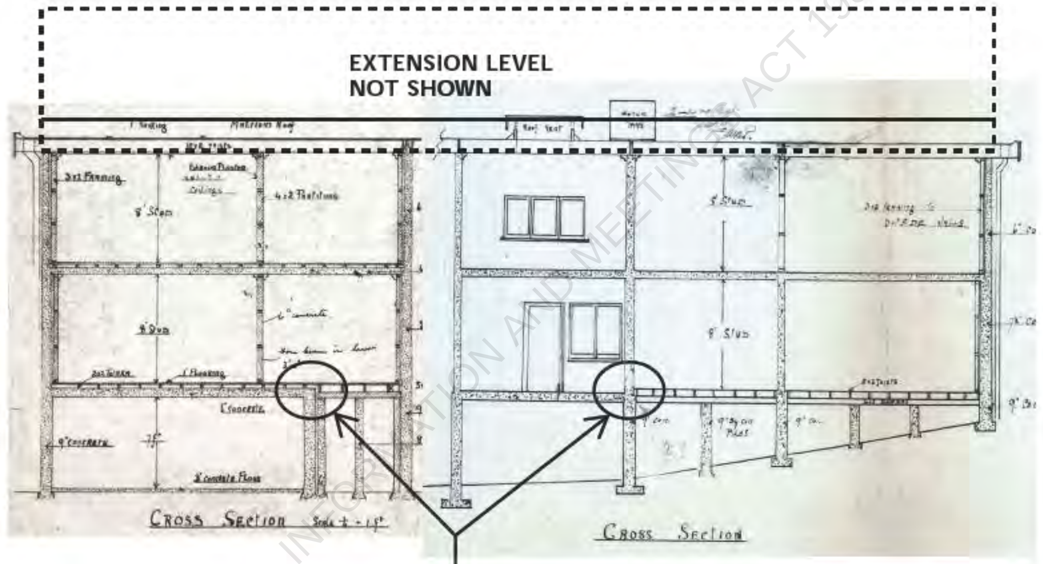
GARAGE LEVEL

BREAK OUT CONCRETE AT CRACKS TO
DETERMINE REINFORCING CONDITION

Notes:

Refer to Retrofit Sketches for updated wall layouts based on the site measure.

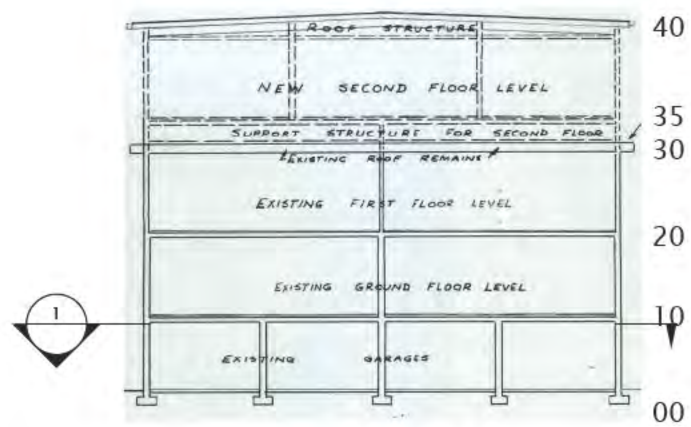
Scope of work indicated is what remains after CSI scanning and site measure of the public areas.



EXPOSE TOP AND BOTTOM OF
FLOOR PLATE TO DETERMINE
EXTENTS OF RC SLAB AT GF LEVEL.

SECTION 1

SECTION 2



KEY SECTION

PRINT DRAWINGS IN COLOUR

Rev	Revision Description	By	App	Date
A	DRAFT FOR REVIEW	s7(2)(a) [redacted]		24/1/24
B	ISSUE TO SUBCONTRACTOR			23/2/24
C	FOR INCLUSION IN DSA			31/5/24

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Wellington, 6011 New Zealand NZBN 9429 0421 10316

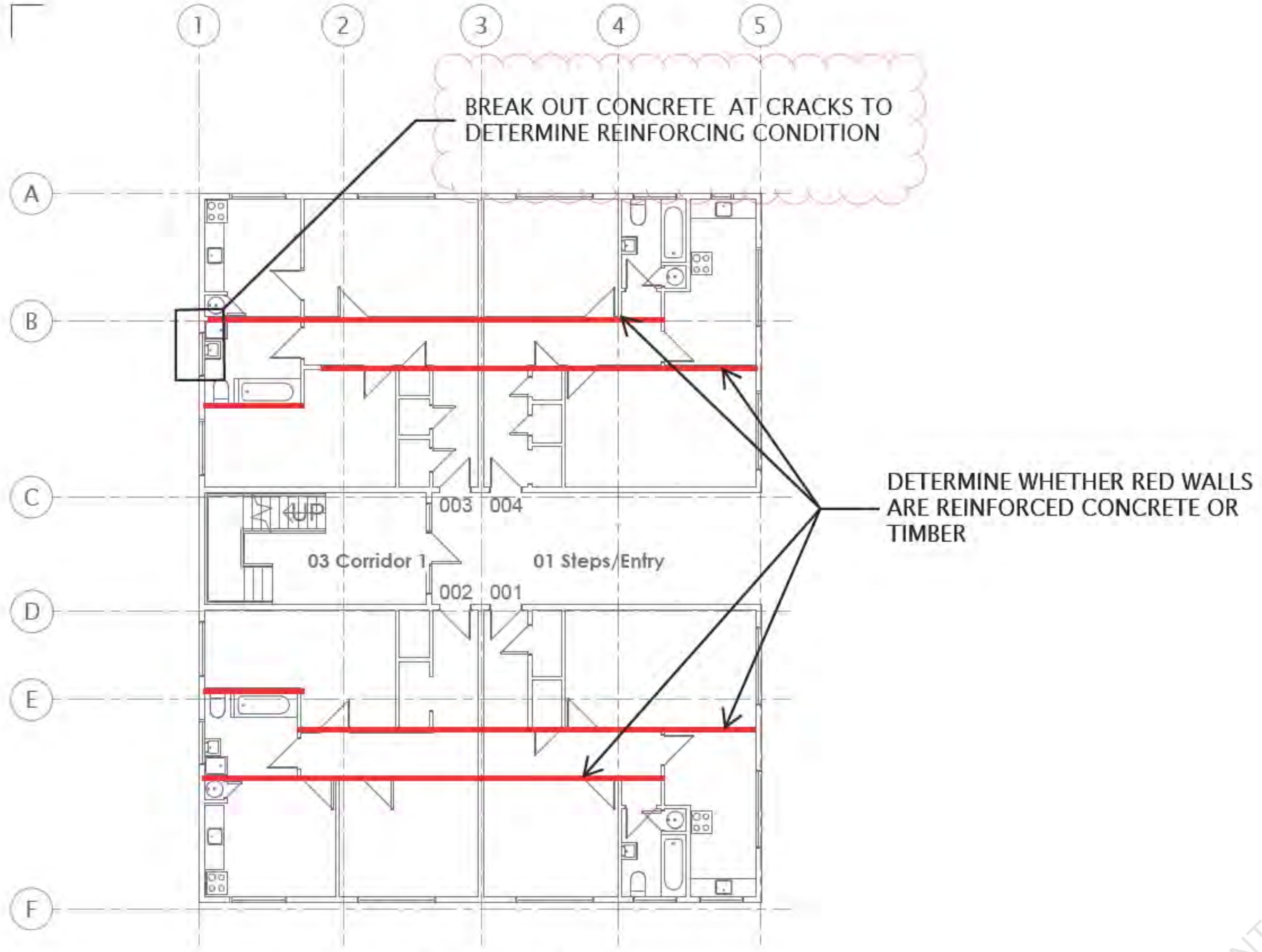
Client: WELLINGTON CITY COUNCIL
Project: N0541-WHA WHARE AHURU APARTMENTS
Title: INVESTIGATIONS - GARAGE FLOOR

Scale at A3: NTS
Drawn: s7(2)(a) [redacted]
Date: 23/2/2024
Designer: s7(2)(a) [redacted]
Drawing Number: N0541-RBG-WHAA-10-SK-ST-00001
Revision: C

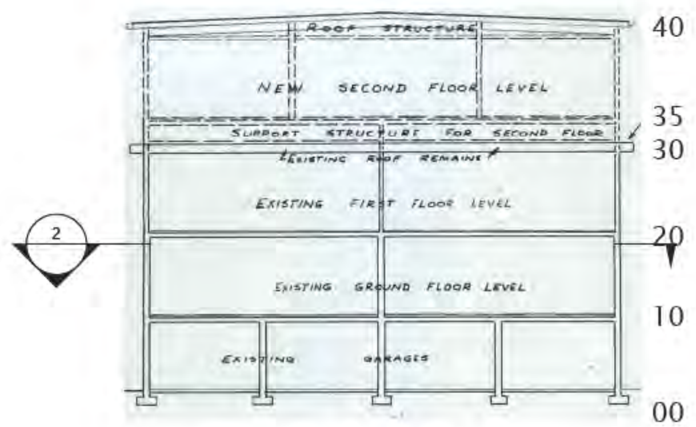
Notes:

Refer to Retrofit Sketches for updated wall layouts based on the site measure.

Scope of work indicated is what remains after CSI scanning and site measure of the public areas.



GROUND FLOOR



KEY SECTION

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Rev	Revision Description	By	App	Date
A	DRAFT FOR REVIEW	s7(2)(a) [redacted]		19/1/24
B	ISSUE TO SUBCONTRACTOR			23/2/24
C	FOR INCLUSION IN DSA			3/5/24

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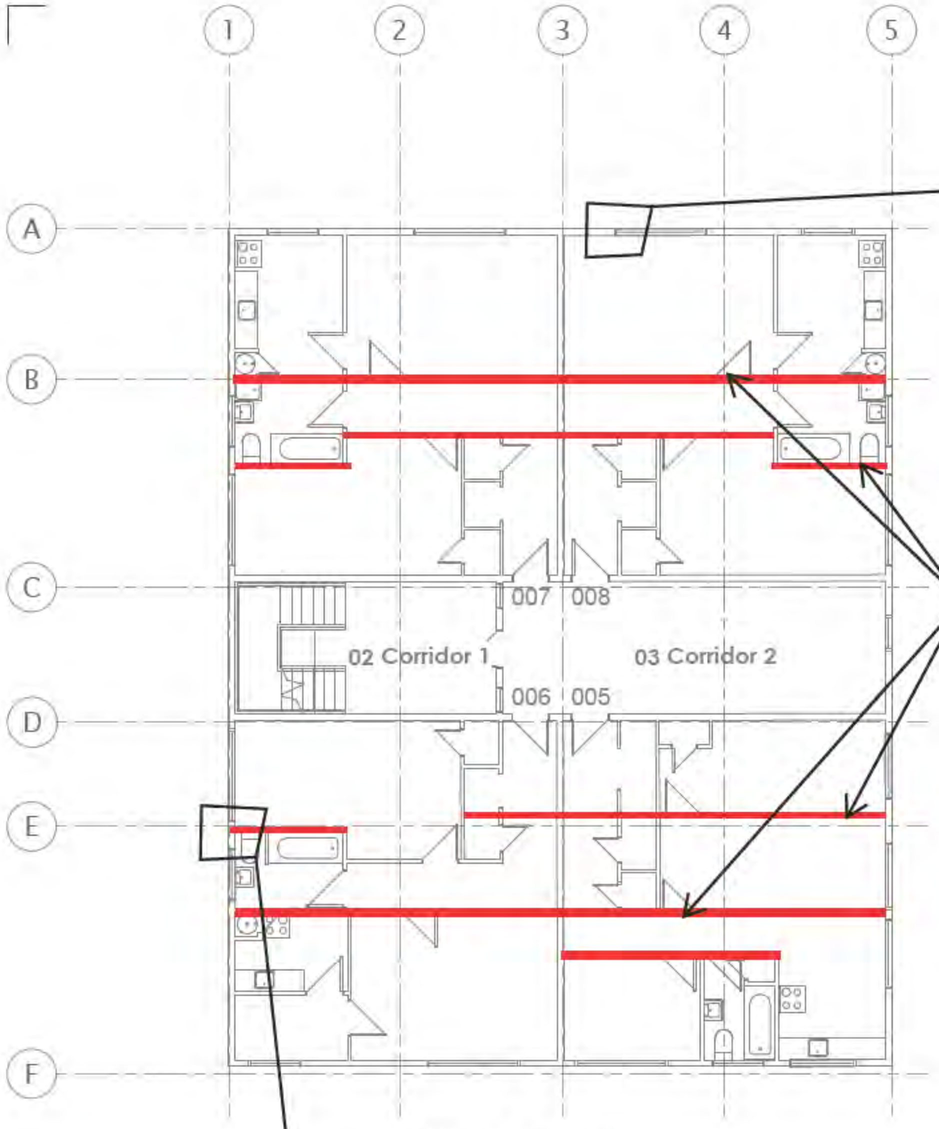
Project
N0541-WHA WHARE AHURU APARTMENTS

Title
INVESTIGATIONS - GROUND FLOOR

Scale at A3	Drawn
NTS	s7(2)(a) [redacted]
Date	**Designer**
23/2/2024	s7(2)(a) [redacted]

Drawing Number	Revision
N0541-RBG-WHAA-20-SK-ST-00002	C



BREAK OUT CONCRETE AT CRACKS TO DETERMINE REINFORCING CONDITION

DETERMINE WHETHER RED WALLS ARE REINFORCED CONCRETE OR TIMBER

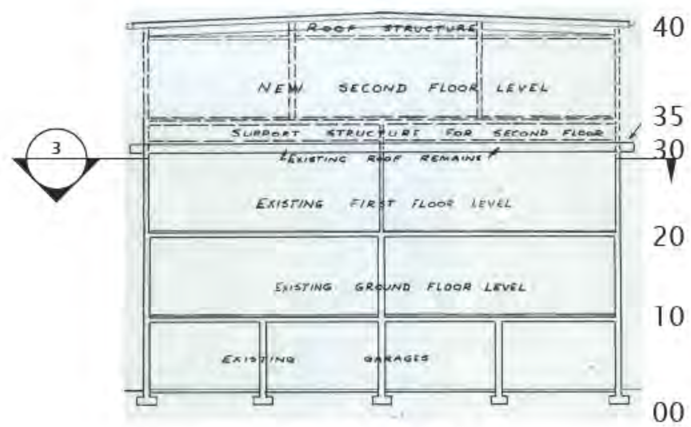
BREAK OUT CONCRETE AT CRACKS TO DETERMINE REINFORCING CONDITION

FIRST FLOOR

Notes:

Refer to Retrofit Sketches for updated wall layouts based on the site measure.

Scope of work indicated is what remains after CSI scanning and site measure of the public areas.



KEY SECTION

PRINT DRAWINGS IN COLOUR

Rev	Revision Description	By	App	Date
A	DRAFT FOR REVIEW	s7(2)(a) [redacted]		19/1/24
B	ISSUE TO SUBCONTRACTOR			23/2/24
C	FOR INCLUSION IN DSA			3/5/24

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Project: N0541-WHA WHARE AHURU APARTMENTS

Title: INVESTIGATIONS - FIRST FLOOR

Scale at A3: NTS

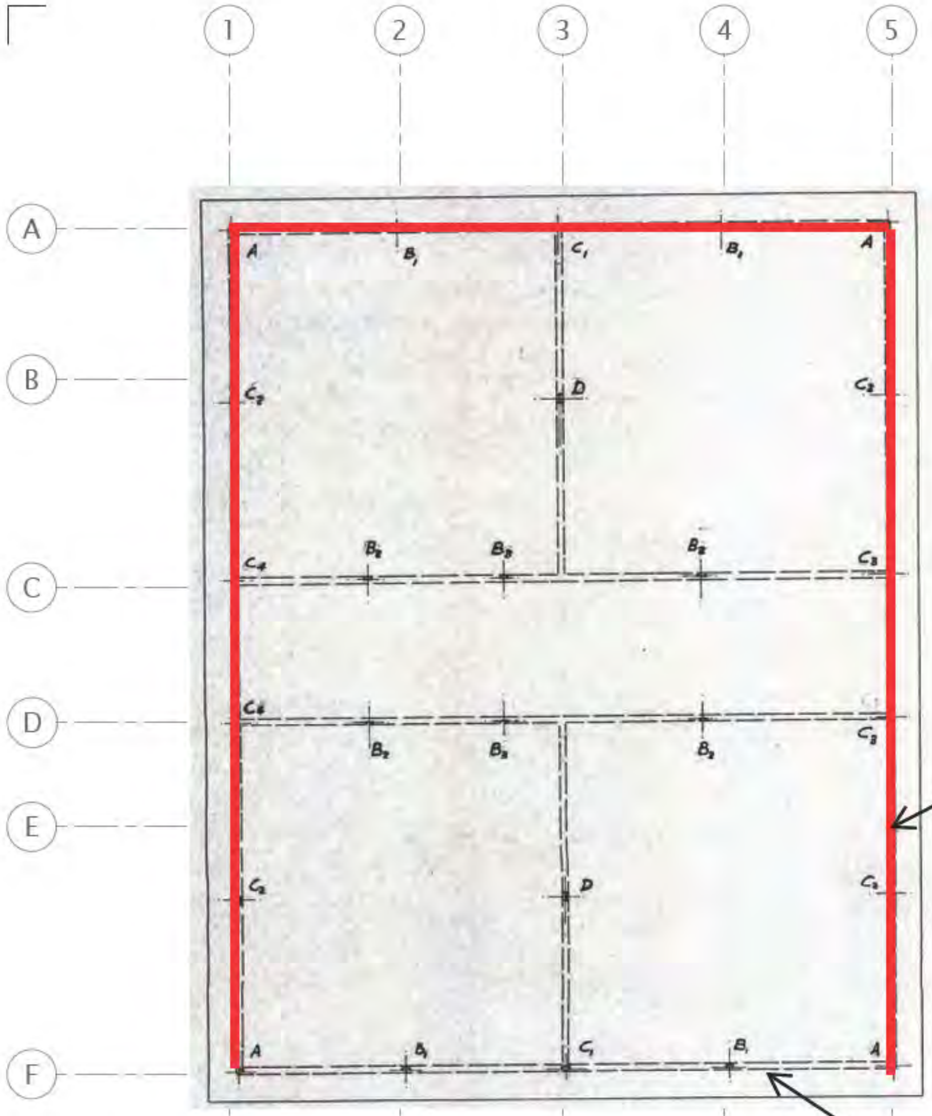
Drawn: s7(2)(a) [redacted]

Date: 23/2/2024

Designer: s7(2)(a) [redacted]

Drawing Number: N0541-RBG-WHAA-30-SK-ST-00003

Revision: C



SUPPORT STRUCTURE FOR
SECOND FLOOR

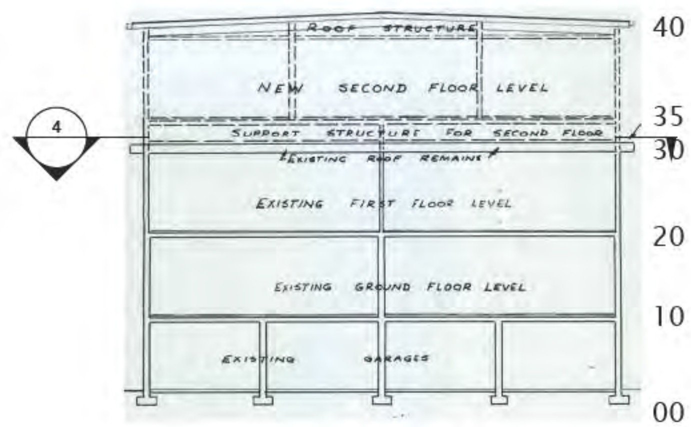
SCAN THE INFILL WALL TO
DETERMINE REINFORCING LAYOUT
BREAK OUT TWO AREAS AT THE
BASE AND TOP OF THE WALL TO
CONFIRM CONNECTION DETAILS

THE PLY INFILL IS BEING REPLACED
OR DEMOLISHED IN ALL SCHEMES

Notes:

Refer to Retrofit Sketches for updated wall layouts based on the site measure.

Scope of work indicated is what remains after CSI scanning and site measure of the public areas.



KEY SECTION

PRINT DRAWINGS IN COLOUR

Rev	Revision Description	By	App	Date
A	DRAFT FOR REVIEW	s7(2)(a) [redacted]		19/1/24
B	ISSUE TO SUBCONTRACTOR			23/2/24
C	FOR INCLUSION IN DSA			3/5/24

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Client
WELLINGTON CITY COUNCIL

Project
N0541-WHA WHARE AHURU APARTMENTS

Title
INVESTIGATIONS - SECOND FLOOR SUPPORT STRUCTURE

Scale at A3
NTS

Drawn
s7(2)(a) [redacted]

Date
23/2/2024

Designer
s7(2)(a) [redacted]

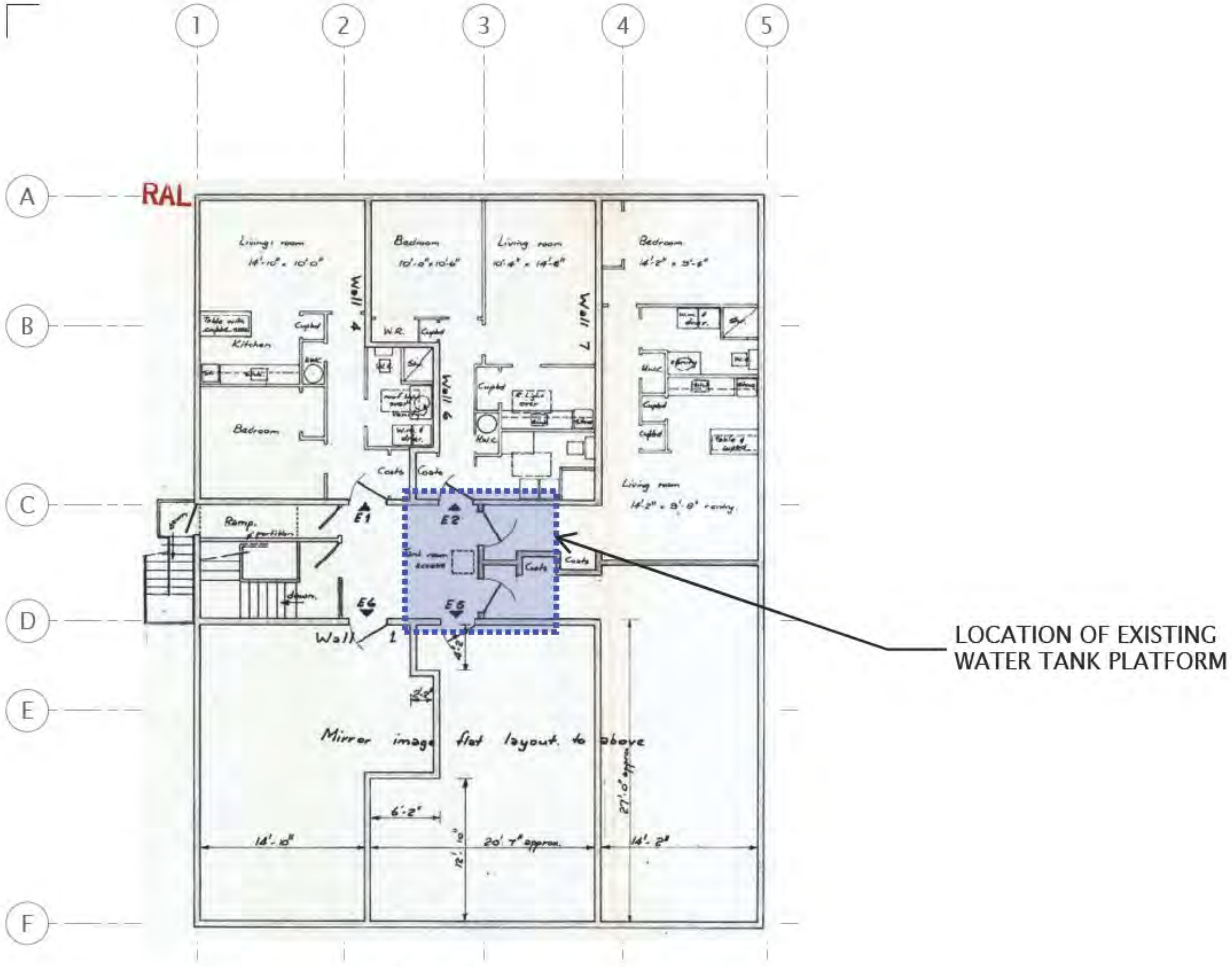
Drawing Number
N0541-RBG-WHAA-35-SK-ST-00004

Revision
C

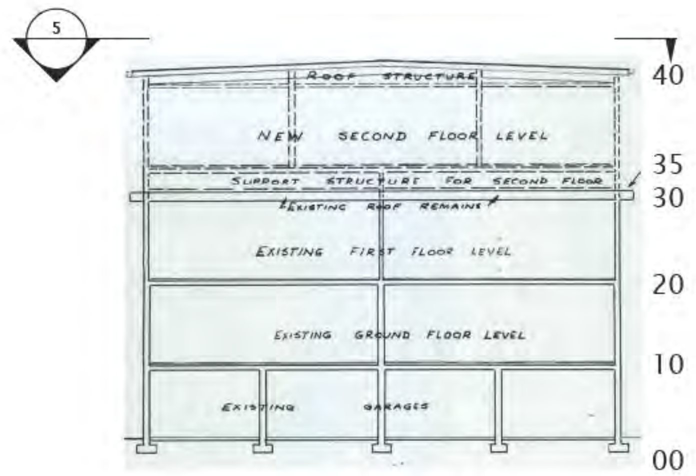
Notes:

Sufficient investigation has been conducted to confirm that the existing tanks are inadequately supported and restrained.

No further investigation is needed at this level.



ROOF



KEY SECTION

PRINT DRAWINGS IN COLOUR

Rev	Revision Description	By	App	Date
A	DRAFT FOR REVIEW	s7(2)(a) [redacted]		19/1/24
B	ISSUE TO SUBCONTRACTOR			23/2/24
C	FOR INCLUSION IN DSA			3/5/24

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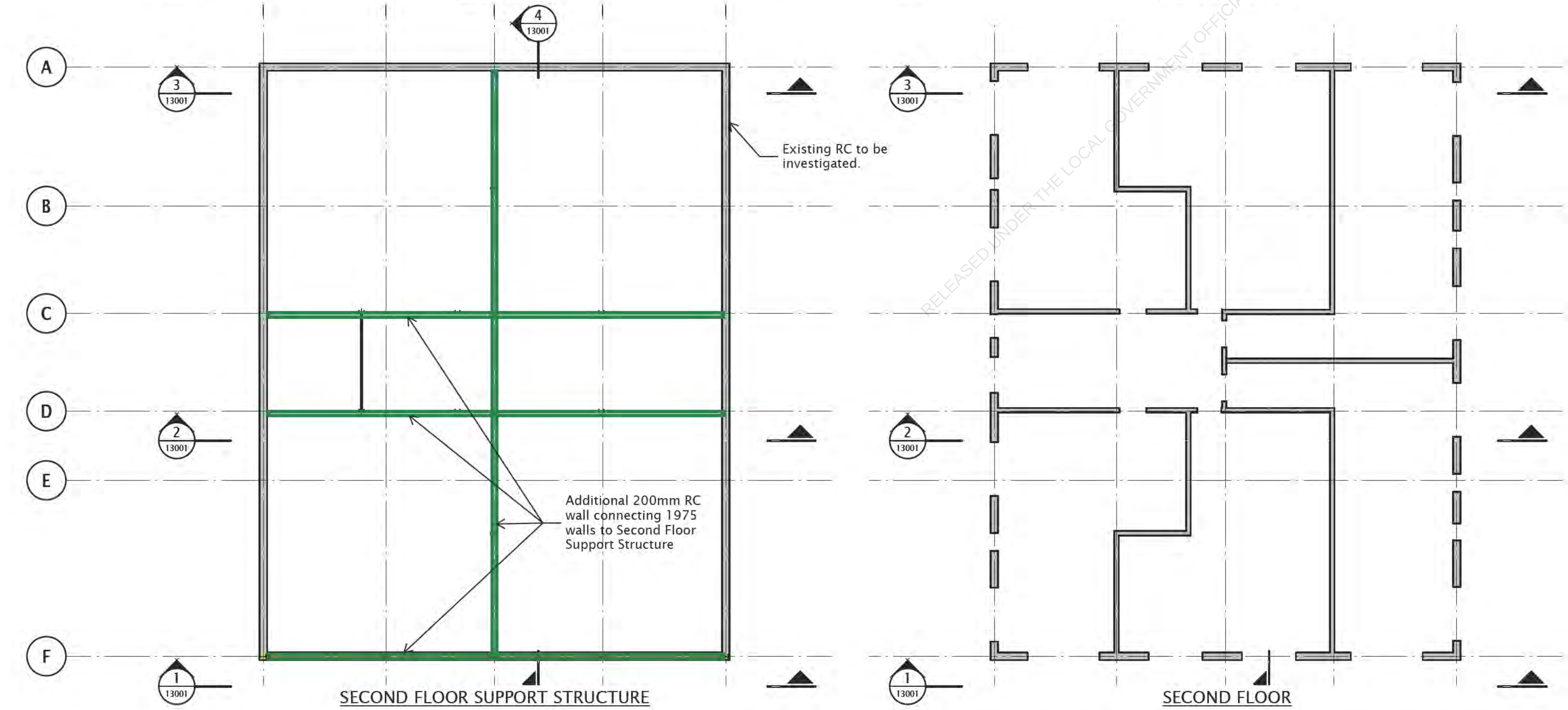
Client	WELLINGTON CITY COUNCIL
Project	N0541-WHA WHARE AHURU APARTMENTS
Title	INVESTIGATIONS - SECOND FLOOR
Scale at A3	NTS
Drawn	s7(2)(a) [redacted]
Date	23/2/2024
Designer	s7(2)(a) [redacted]
Drawing Number	N0541-RBG-WHAA-40-SK-ST-00005
Revision	C



Notes:

Seismic Strengthening to the level indicated in the drawing title.

Strengthening scheme to be read in conjunction with investigation sketches.

Rev	Revision Description	By	App	Date
P01	for DSA report inclusion	s7(2)(a) [redacted]		24.4.24
P02	responding to peer review comments			3.5.24

Rev	Revision Description	By	App	Date

SCALE 1 2 3 4 5 6 7 8

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Client
WCC

Absolutely Positively Wellington City Council
Me Heke Ki Pōneke

Title
Concept 1 Plans
Minimal Additional Structure
50%NBS

Project
WHAA - 16 Glenmore Street

Scale at A1: NTS
Date: Refer revision block
Drawn: s7(2)(a) [redacted]
Designer: s7(2)(a) [redacted]
Design Checker: s7(2)(a) [redacted]
Approved: s7(2)(a) [redacted]

Concept Design

Drawing Number: N0541-WHAA-ZZ-SK-ST-00011
Revision: P02

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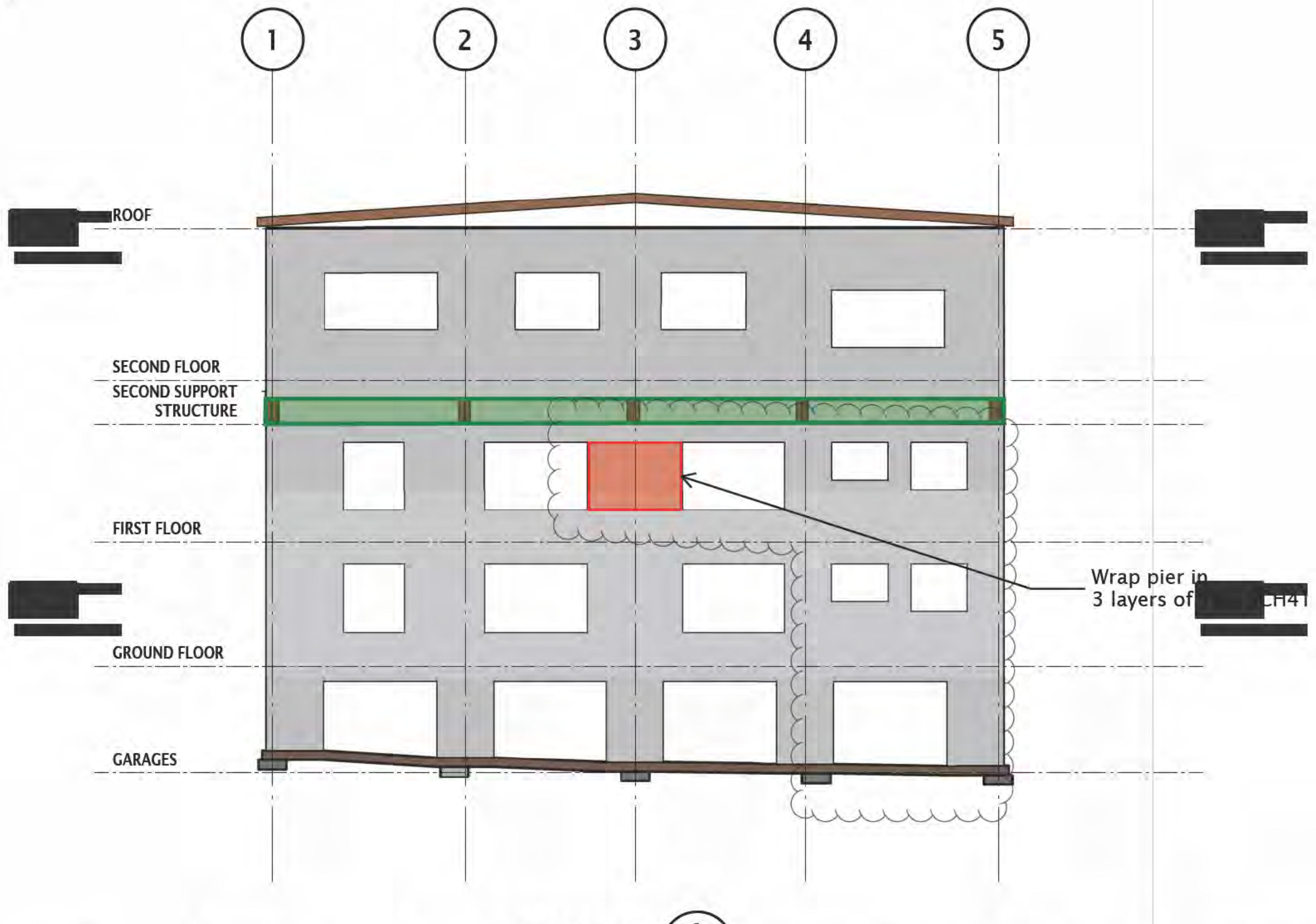
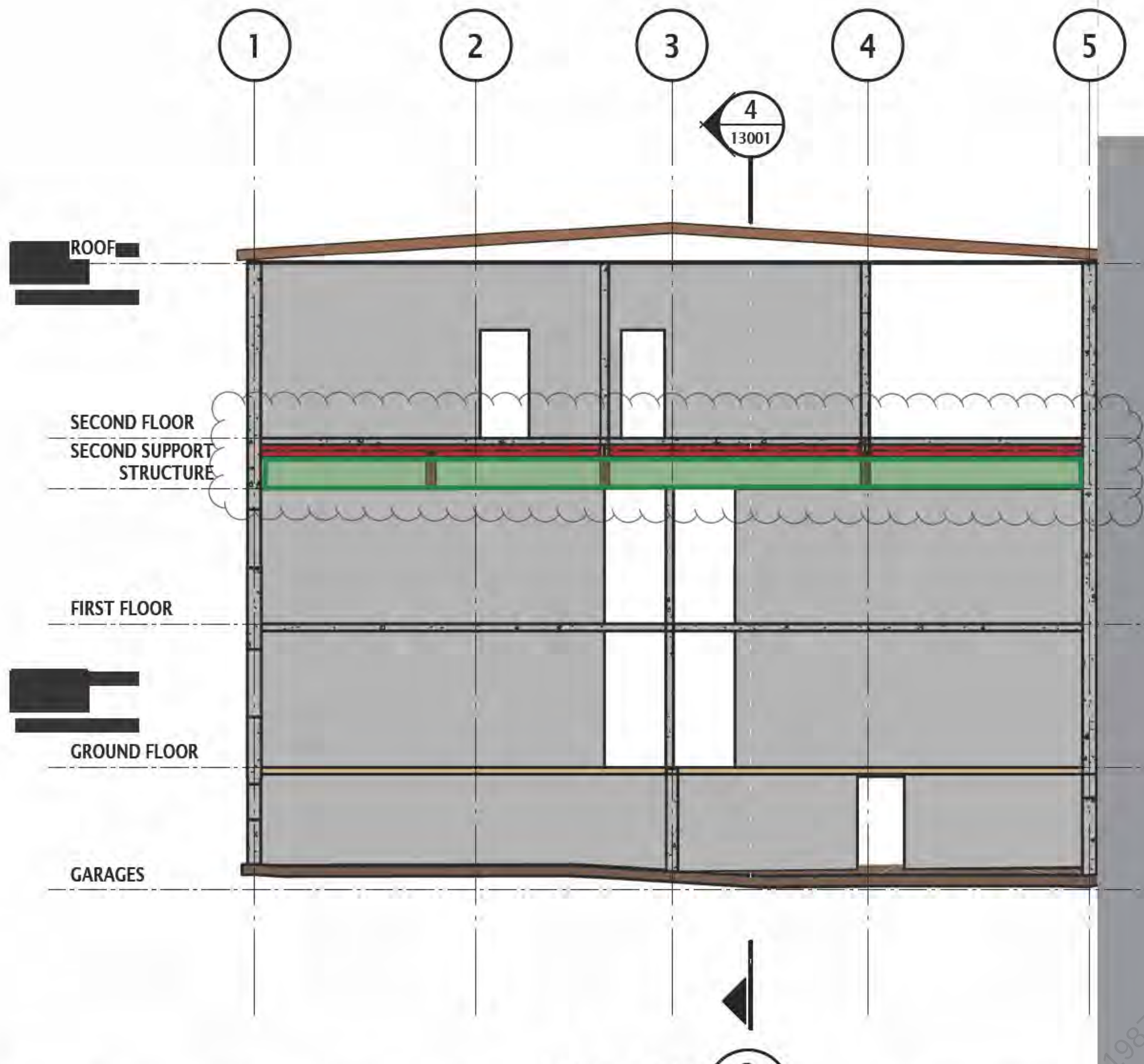
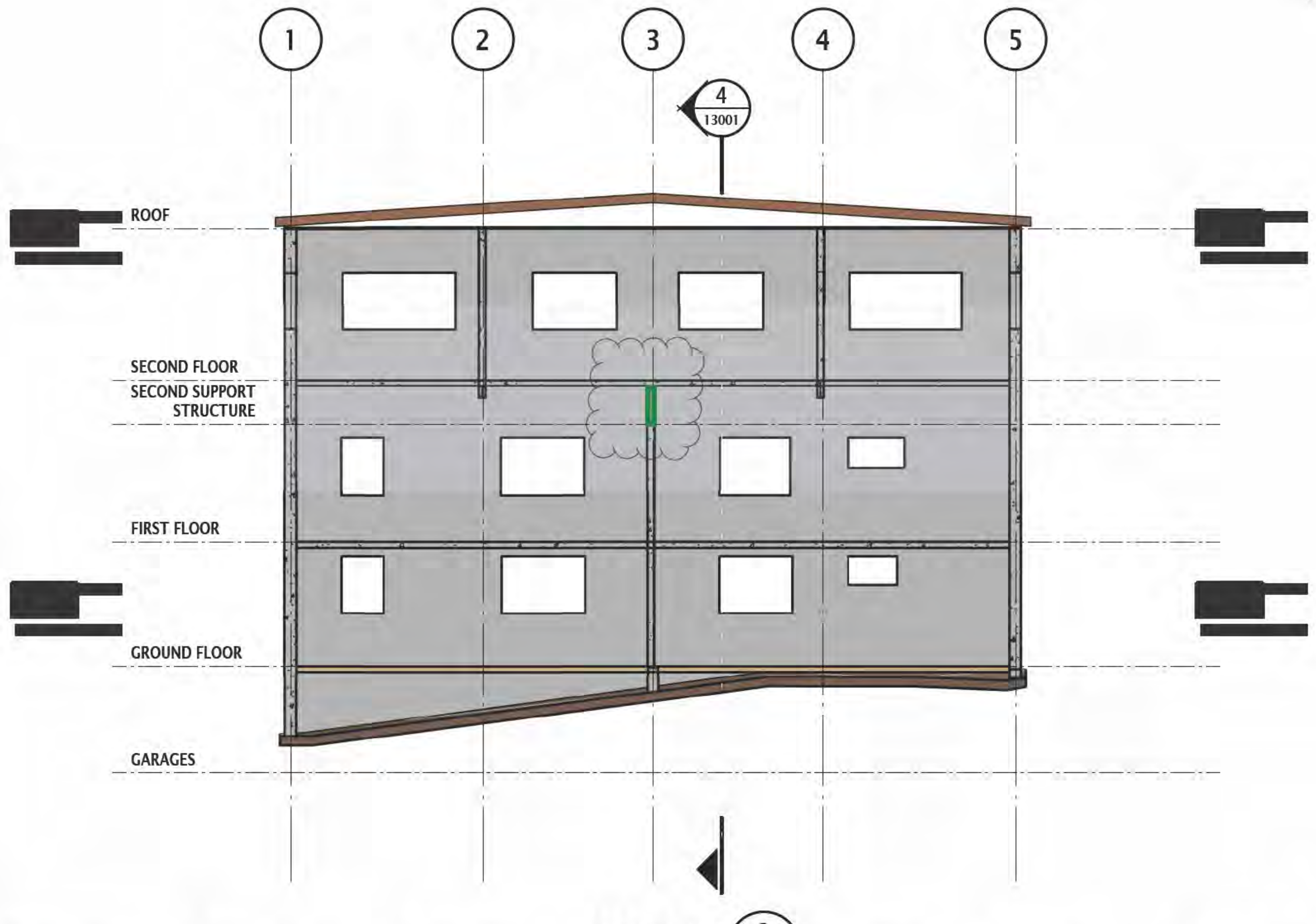


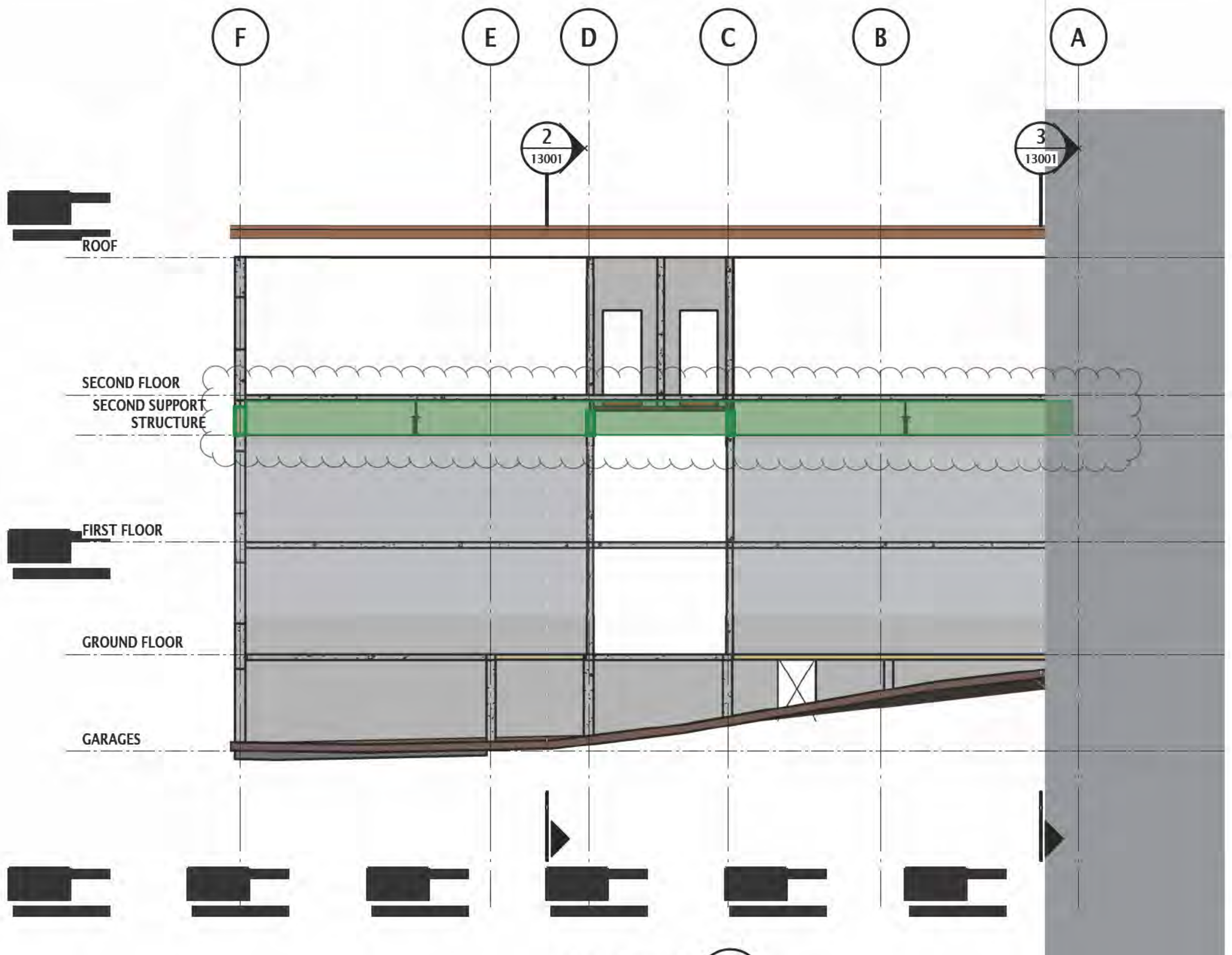
ELEVATION 1 (03001)
SCALE 1 : 100



ELEVATION 2 (03001)
SCALE 1 : 100



ELEVATION 3 (03001)
SCALE 1 : 100



ELEVATION 4 (03001)
SCALE 1 : 100

Notes:

Additional concrete wall

Rev	Revision Description	By	App	Date
[redacted]01	for DSA re[redacted]	s7(2)(a)		24.4.24
P02	respondin[redacted]iew comments	[redacted]		3.5.24

Rev	Revision Description	By	App	Date
	[redacted]	[redacted]		

SCALE 1 2 3 4 5 6 7 8

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Client
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Absolutely Positively Wellington City Council
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Title
Concept 1 [redacted]
Minimal Ad[redacted] Structure
50%NBS

Project
WHAA - 16 Glenmore Street

Scale at A1	Drawn	Design Checker
NTS	s7(2)(a)	s7(2)(a)
Date	[redacted]signer	[redacted]
[redacted]er revision bloc[redacted]	s7(2)(a)	s7(2)(a)

Concept Design

Drawing Number: N0541-WHAA-ZZ-SK-ST-00012
Revision: P02

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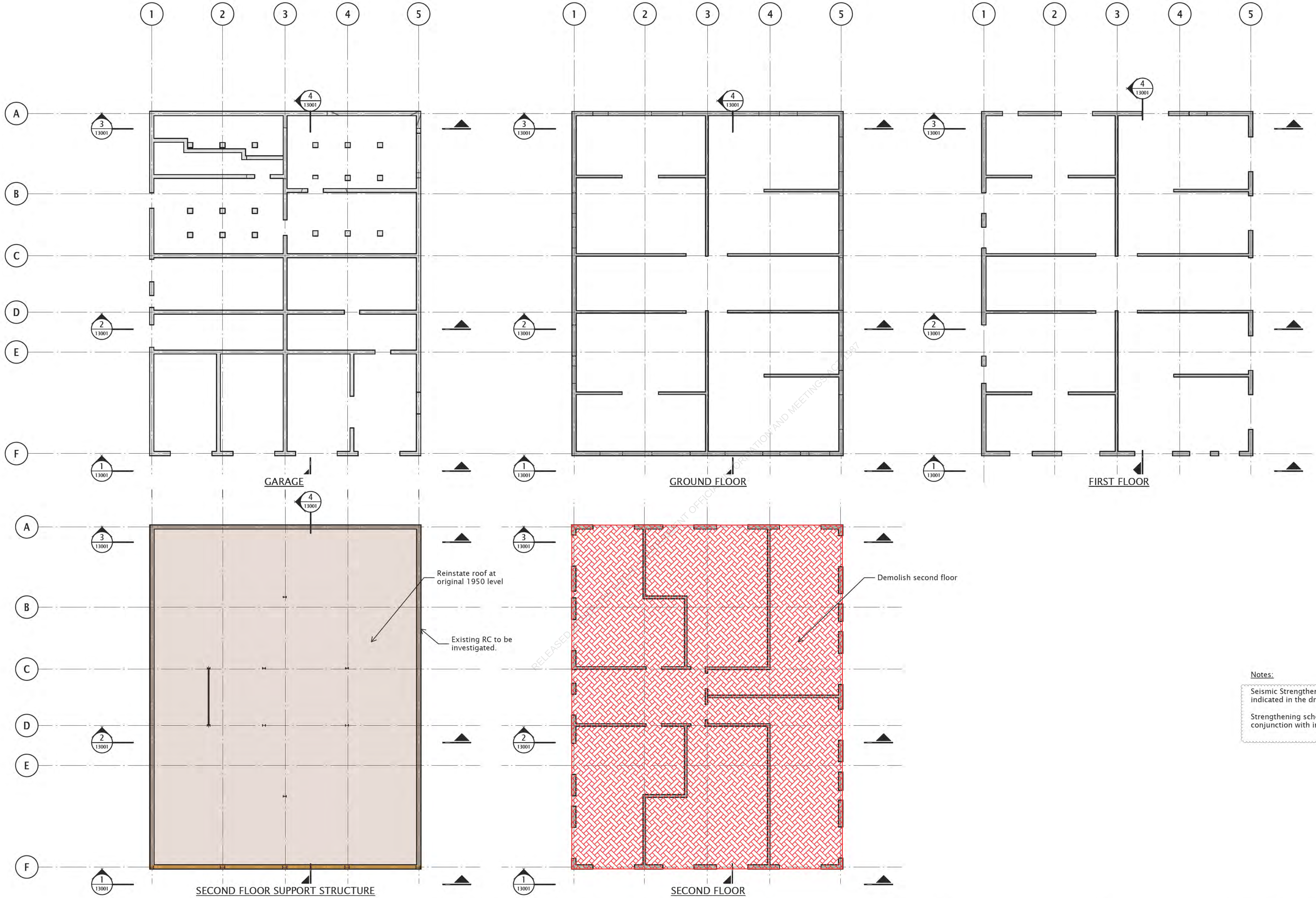

Notes:

Seismic Strengthening to the level indicated in the drawing title.

Strengthening scheme to be read in conjunction with investigation sketches.

Rev	Revision Description	By	App	Date
P01	for DSA report inclusion	s7(2)(a) [redacted]		24.4.24
P02	responding to peer review comments			3.5.24

Rev	Revision Description	By	App	Date

SCALE 1 2 3 4 5 6 7 8

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Client
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Title
Concept 2 Plans
Demolish Second Floor
60%NBS

Project
WHAA - 16 Glenmore Street

Scale at A1: NTS
Date: Refer revision block
Drawn: s7(2)(a) [redacted]
Designer: s7(2)(a) [redacted]
Design Checker: s7(2)(a) [redacted]
Approved: s7(2)(a) [redacted]

Concept Design

Drawing Number: N0541-WHAA-ZZ-SK-ST-00013
Revision: P02

PRINT DRAWINGS IN COLOUR

Notes:

Rev	Revision Description	By	App	Date
[redacted]01	for DSA re[redacted]	s7(2)(a)		24.4.24
P02	responding[redacted]iew comments	[redacted]		3.5.24

Rev	Revision Description	By	App	Date
[redacted]		[redacted]		

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Client
[redacted]WCC

Absolutely Positively Wellington City Council
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Title
[redacted]oncept 2 [redacted]
Demolish S[redacted]Floor
60%NBS

Project
WHAA - 16 Glenmore Street

Scale at A1: NTS
Date: [redacted]fer revision bloc[redacted]
Drawn: s7(2)(a)
[redacted]signer: s7(2)(a)
Design Checker: s7(2)(a)
[redacted]: s7(2)(a)

Concept Design

Drawing Number: N0541-WHAA-ZZ-SK-ST-00014
Revision: P02

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Notes:

Seismic Strengthening to the level indicated in the drawing title.

Strengthening scheme to be read in conjunction with investigation sketches.

Rev	Revision Description	By	App	Date
P01	for DSA report inclusion	[redacted]		24.4.24
P02	responding to peer review comments			3.5.24

Rev	Revision Description	By	App	Date

SCALE 1 2 3 4 5 6 7 8

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Client
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Title
Concept 3 Plans
Additional Walls
75%NBS

Project
WHAA - 16 Glenmore Street

Scale at A1	Drawn	Design Checker
NTS	[redacted]	[redacted]
Date	Designer	Approved
Refer revision block	[redacted]	[redacted]

Concept Design

Drawing Number	Revision
N0541-WHAA-ZZ-SK-ST-00015	P02

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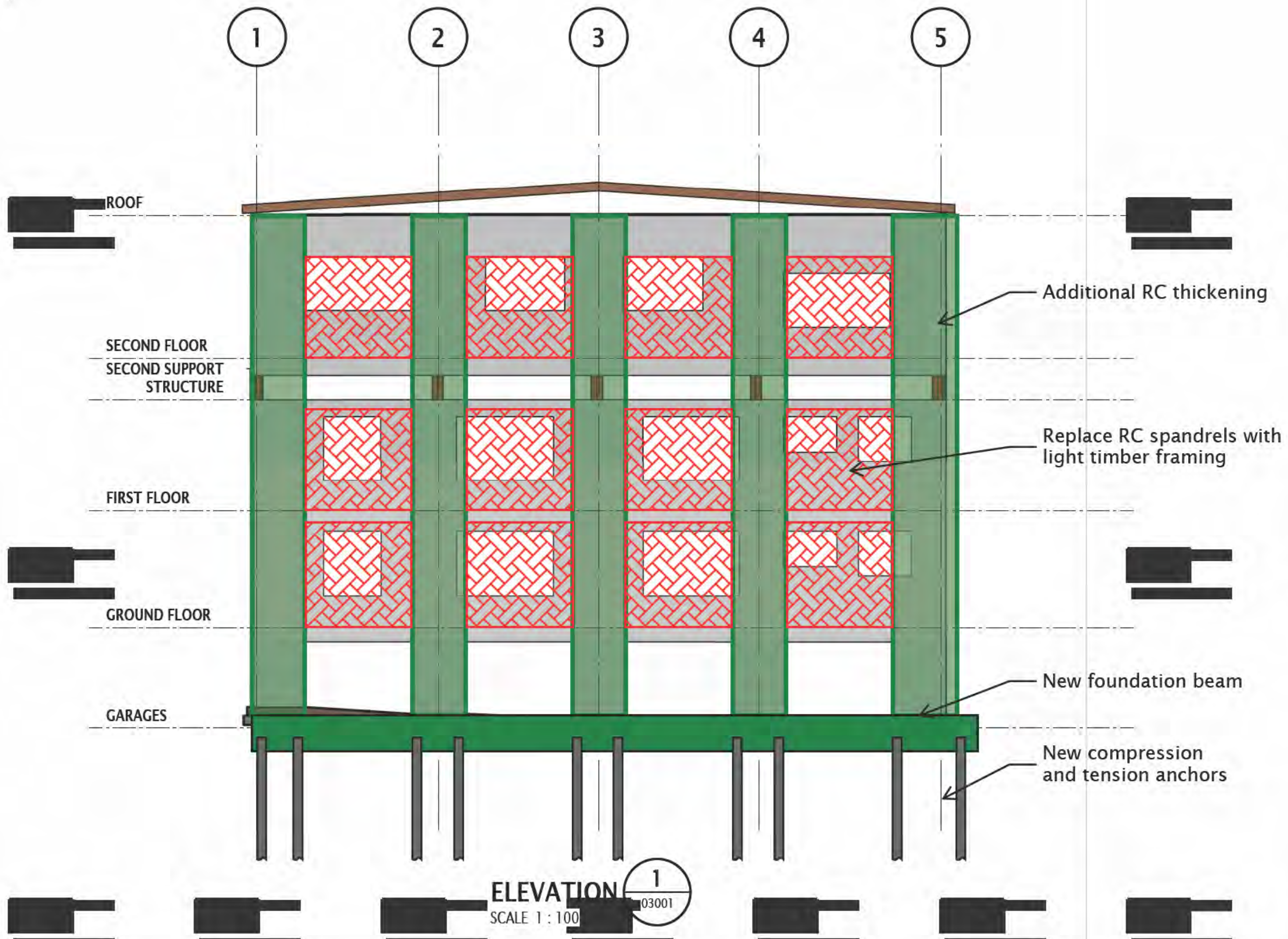

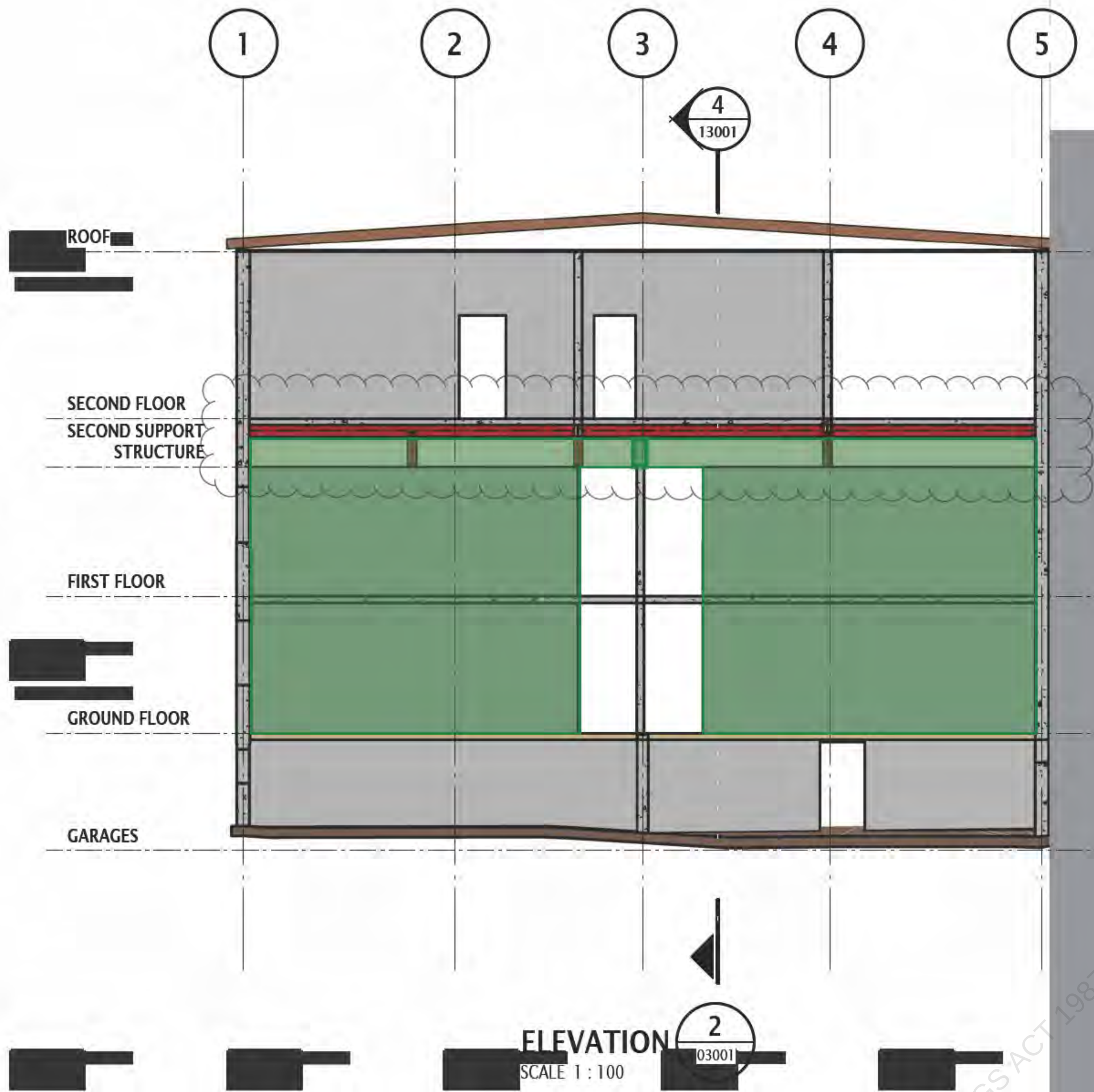

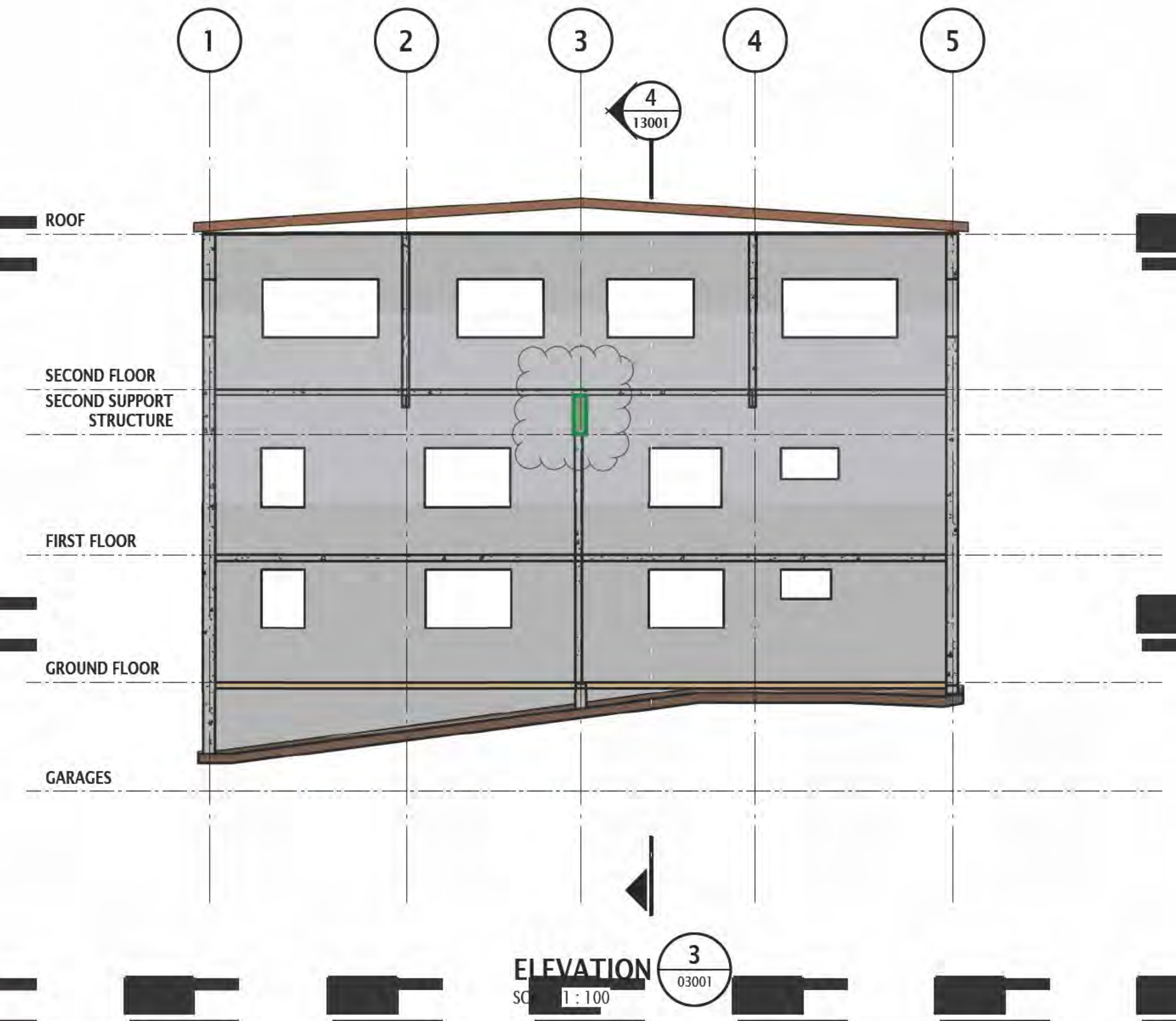

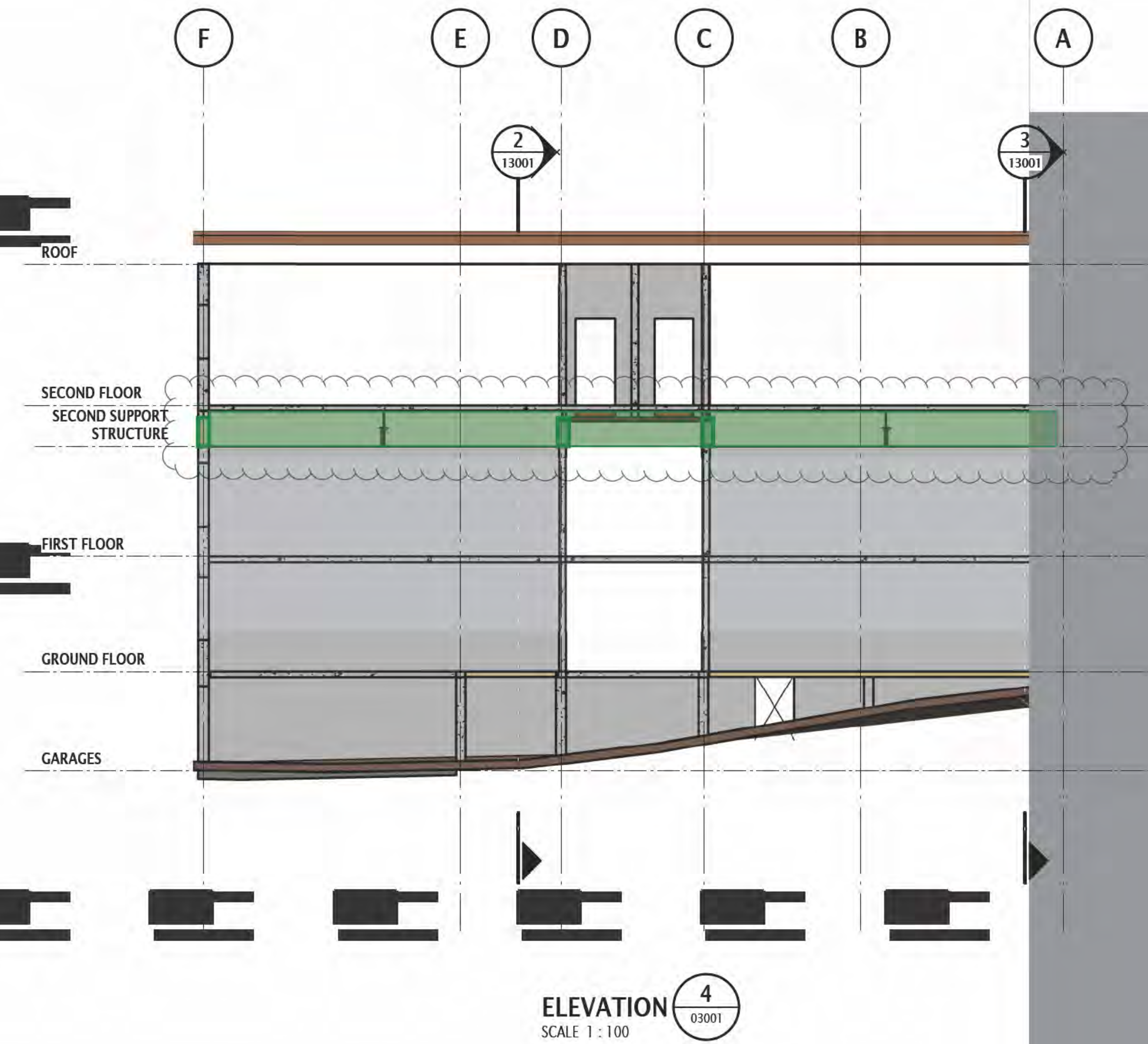

Notes:

Additional concrete wall

Rev	Revision Description	By	App	Date
[redacted]01	for DSA re[redacted]	s7(2)(a) [redacted]		24.4.24
P02	respondin[redacted]iew comments			3.5.24

Rev	Revision Description	By	App	Date
[redacted]		[redacted]		

SCALE 1 2 3 4 5 6 7 8

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Client
[redacted]WCC [redacted]

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Title
[redacted]oncept 3 [redacted]
Additional [redacted]
[redacted]%NBS

Project
WHAA - 16 Glenmore Street

Scale at A1: [redacted]NTS
Date: [redacted]er revision bloc[redacted]
Drawn: s7(2)(a) [redacted]
[redacted]signer: s7(2)(a) [redacted]
Design Checker: s7(2)(a) [redacted]
[redacted] s7(2)(a) [redacted]

Concept Design

Drawing Number: N0541-WHAA-ZZ-SK-ST-00016
Revision: P02

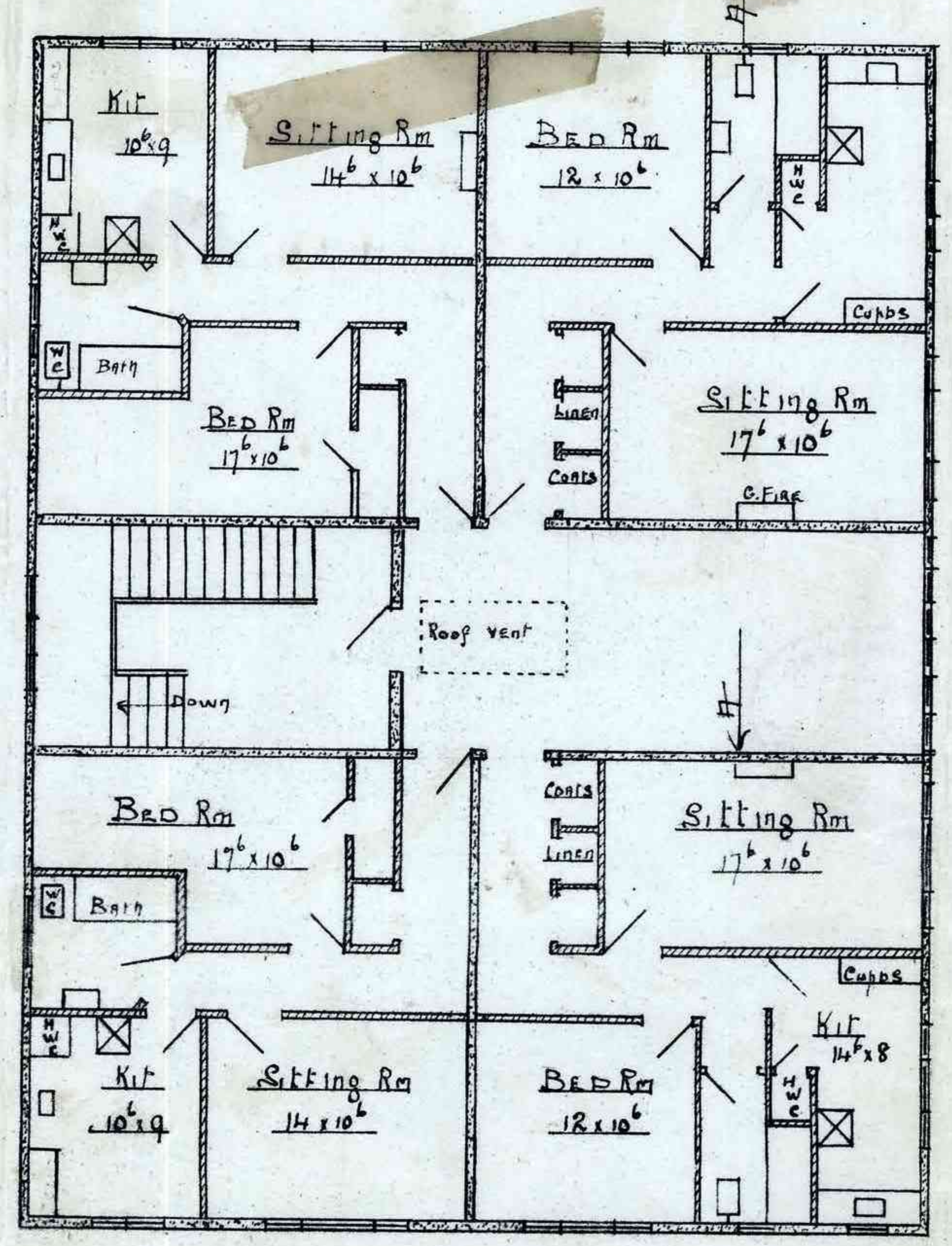
PRINT DRAWINGS IN COLOUR

Appendix F

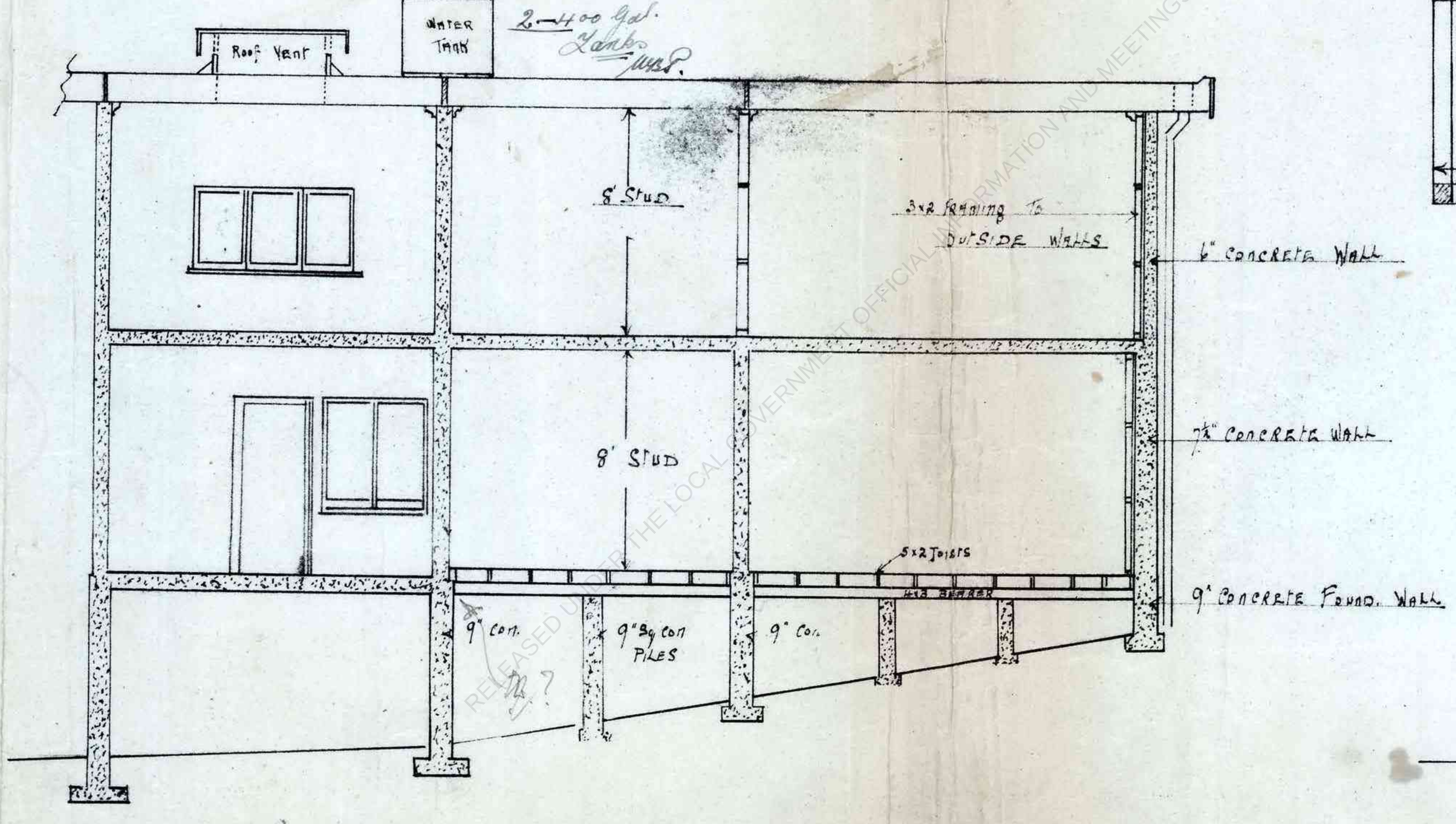
Original Structural Drawings

HUP2-T0-Seismic Assessments

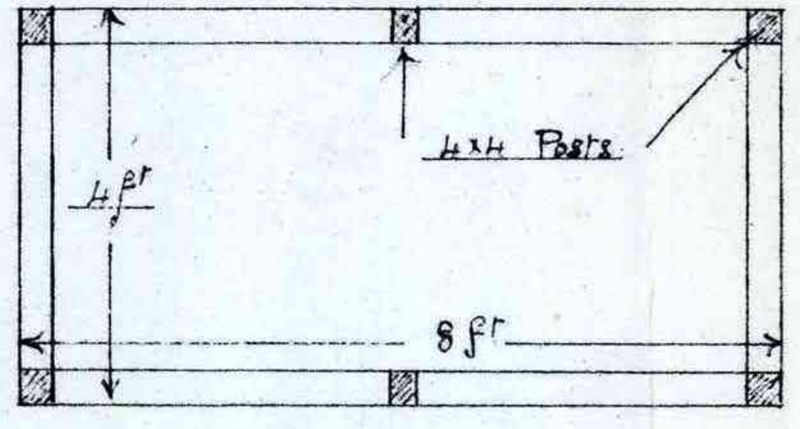
Proposed Flats for W. B. Potter Glenmore St Wellington



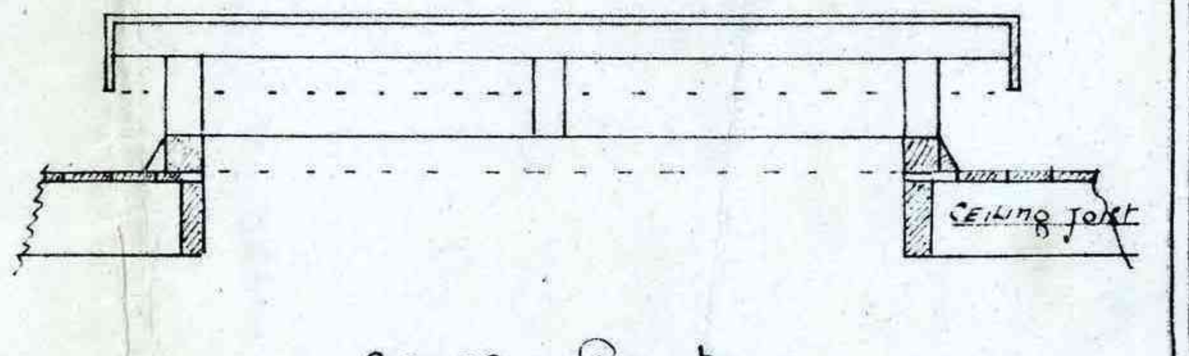
First Floor Plan



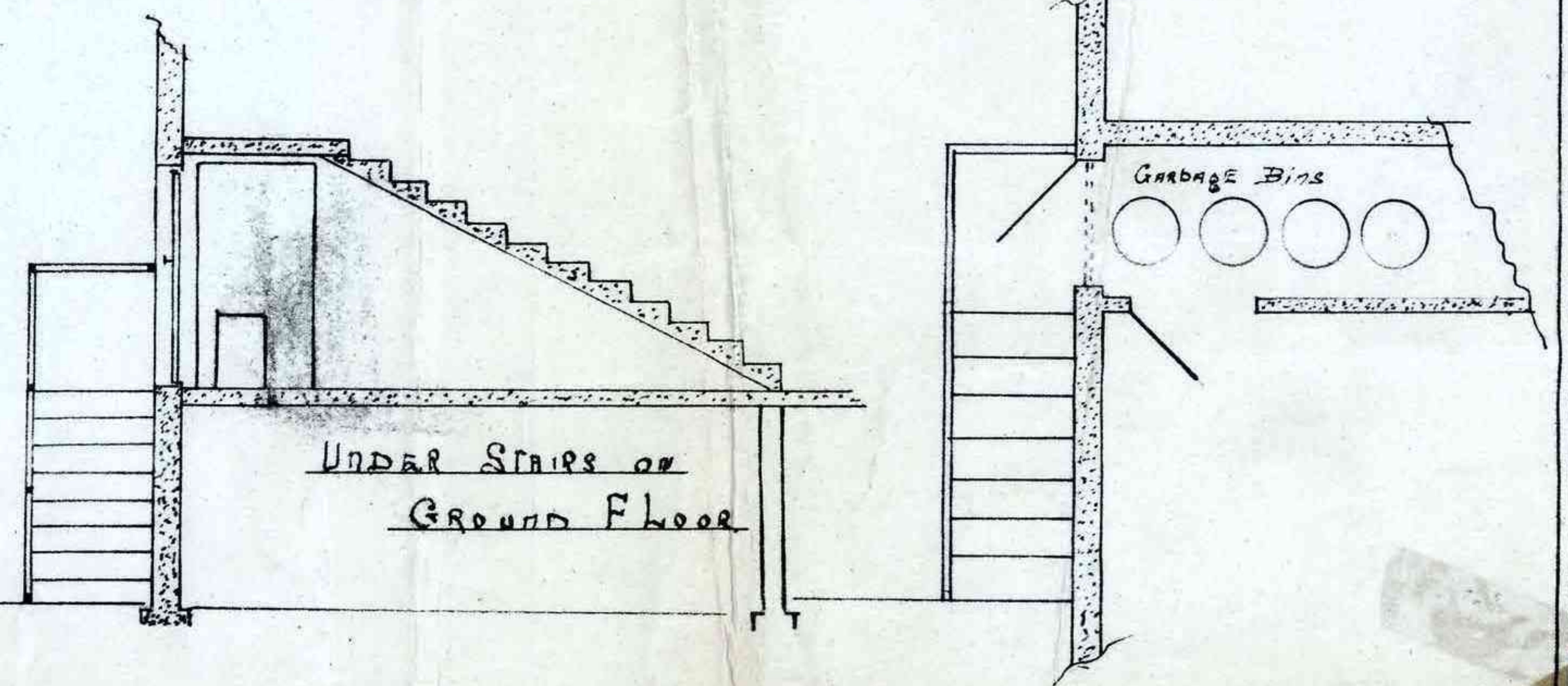
Cross Section



Roof Vent

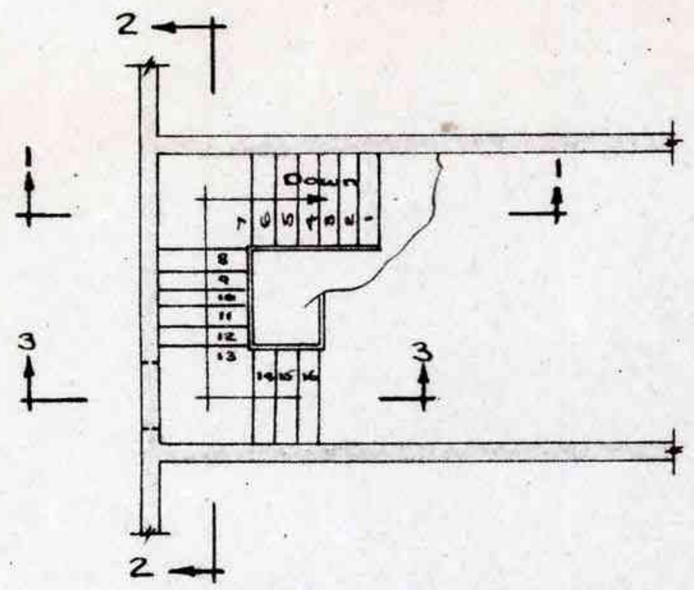


Cross Section

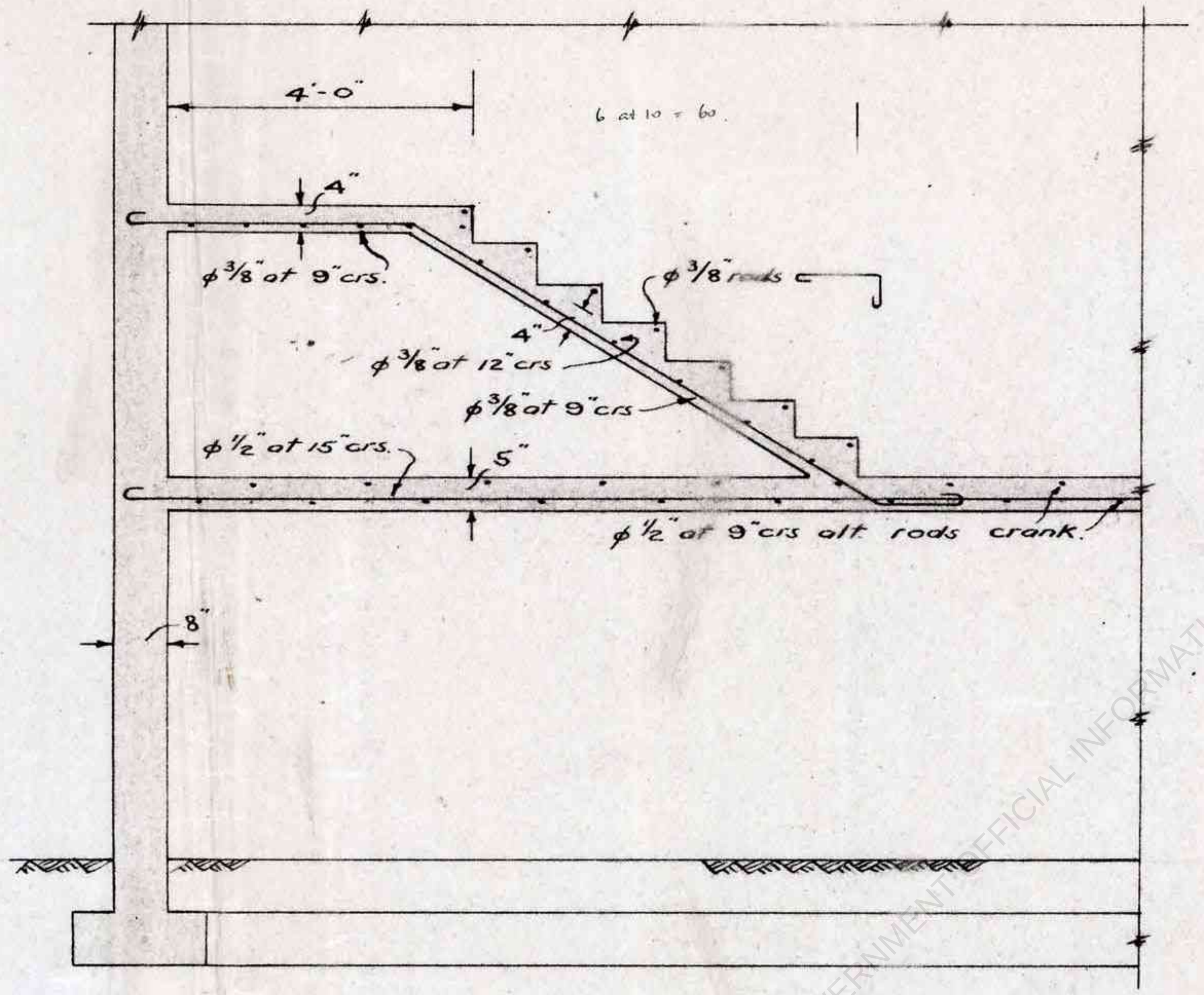


Details of Refuse Disposal

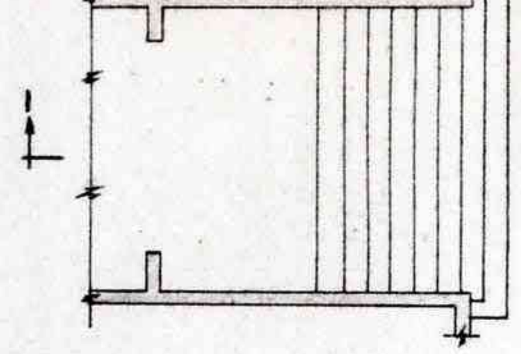

PROPOSED BLOCK of 8 FLATS
for
W. B. POTTER
on
SECTIONS 1 & 2 GLENMORE S^t
WELLINGTON.
4 GARAGES 18^6 x 11^6
CONCRETE FLOORS
11^6 WIDE
GARAGES
WASH H 7^6 x 9^4
WASH H 7^6 x 9^4
COR ALUMINIUM ROOF
4x2 FRAMING
ASBESTOS WALLS
2" CONCRETE FLOOR
6" CONCRETE
WASH HOUSES
CITY ENGINEER'S DEPARTMENT
RECEIVED
21 JUL 1950
MERCER ST. WELLINGTON.
KIT 9 x 9^6
SITTING RM 14^6 x 10^6
FLAT 3 & 7
BED RM 12 x 10^6
FLAT 4 & 8
KIT 14^6 x 8
Conc. Wall 6"
BED RM 17 x 10^6
SITTING RM 17 x 10^6
Roof Vent
Down
Concrete Steps
BED RM 17 x 10^6
FLAT 1 & 5
SITTING RM 17 x 10^6
FLAT 2 & 6
6" Conc. Wall
KIT 14^6 x 8
KIT 9 x 9^6
SITTING RM 14^6 x 10^6
BED RM 12 x 10^6
ENTRANCE to 4 GARAGES
GROUND AND FIRST FLOOR PLAN Scale 1/8 - 1 ft
NOTE: 5/8 Verticals at Windows as shown on window at left hand end.
SIDE ELEVATION
APP. GROUND LEVEL
APPROVED
SUBJECT TO CONDITIONS AS SET OUT IN CONSENT No. 15421
DISTRICT BUILDING CONTROLLER.
Date 31/7/50
FRONT ELEVATION
1" SARKING
MALTHOID ROOF
10x2 JOISTS
3x2 FRAMING
FIBROUS PLASTER WALLS & CEILINGS
4x2 PARTITIONS
8' STUD
6" CONCRETE
6" CONCRETE
8' STUD
8" CONCRETE
2x2 TOPARM
1" FLOORING
5x2 FLOOR JOISTS
6" CONCRETE
9" CONCRETE
9" CONCRETE
7 ft
8x8 CONCRETE PILES
3" CONCRETE FLOOR
CROSS SECTION Scale 1/4 - 1 ft



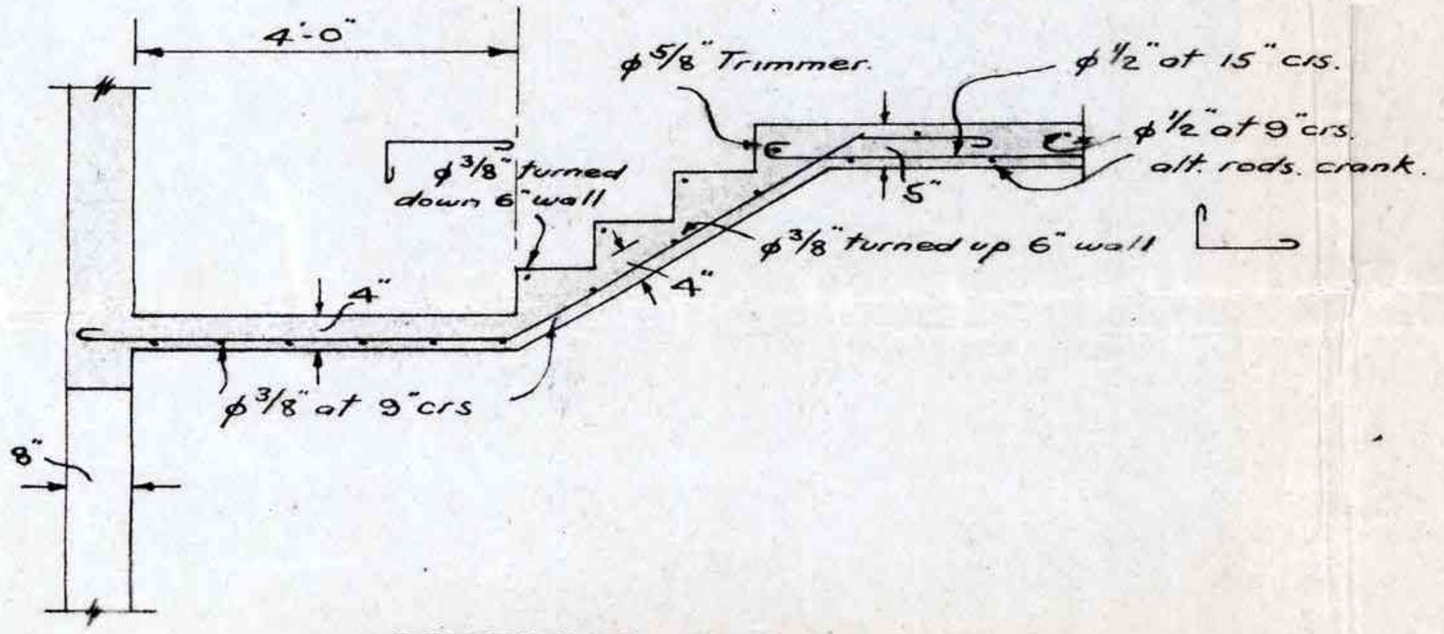
PLAN OF STAIRS
SCALE 1/8" = 1'-0"



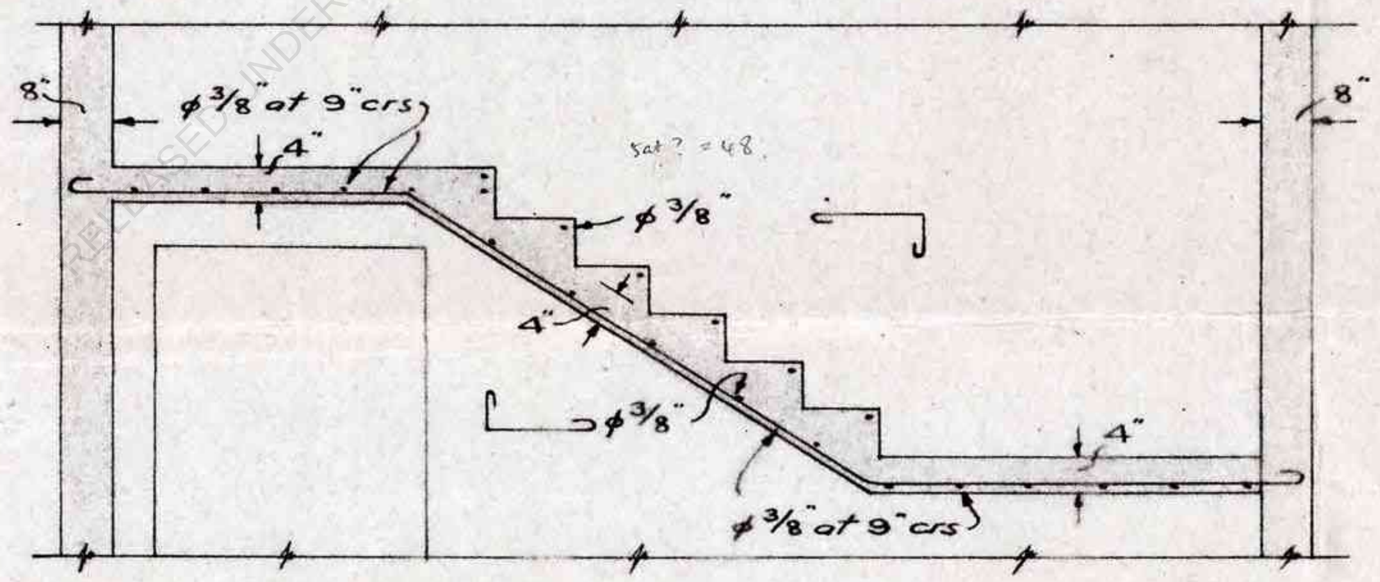
SECTION 1-1



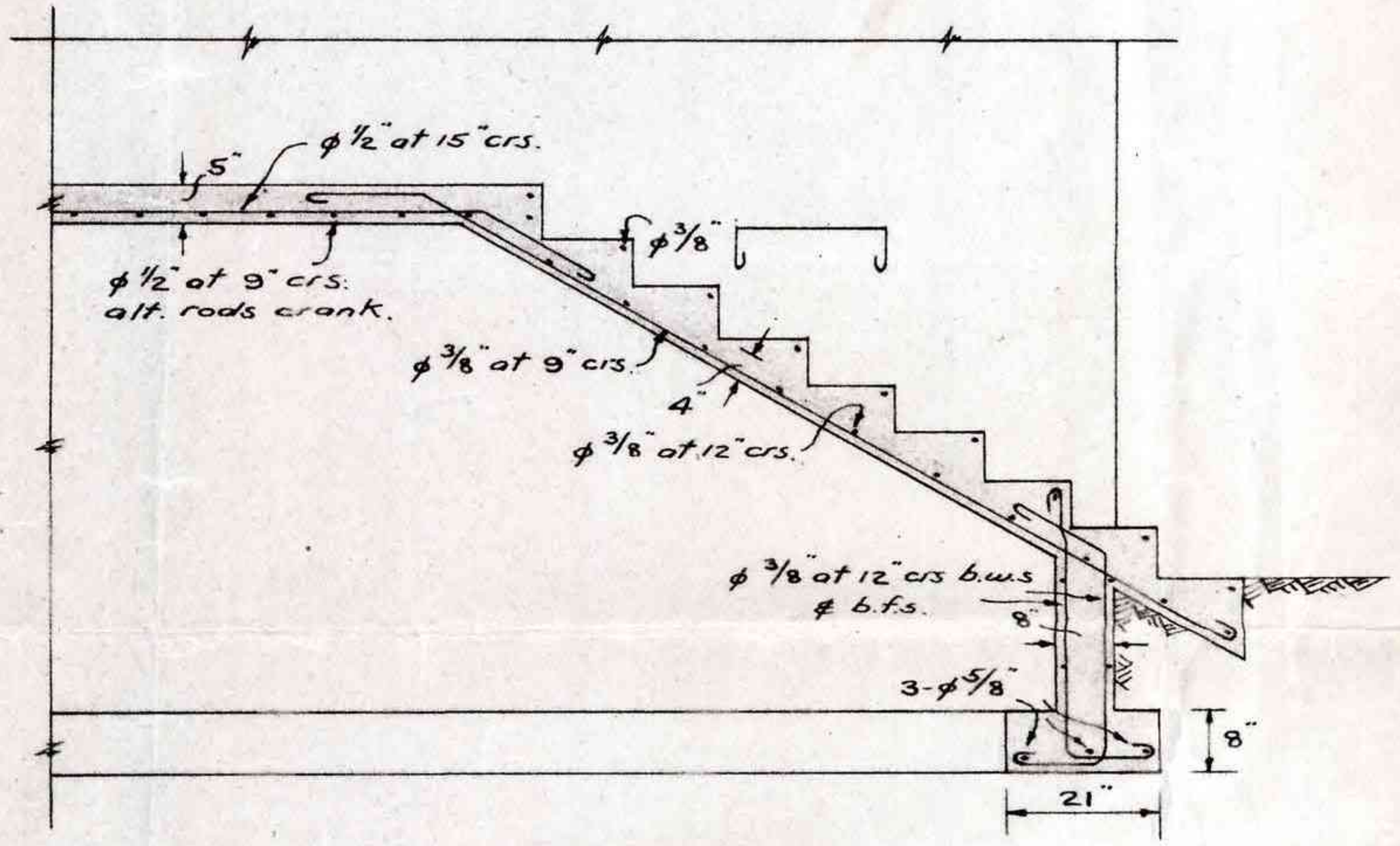
PLAN OF STEPS
AT MAIN ENTRANCE
SCALE 1/8" = 1'-0"



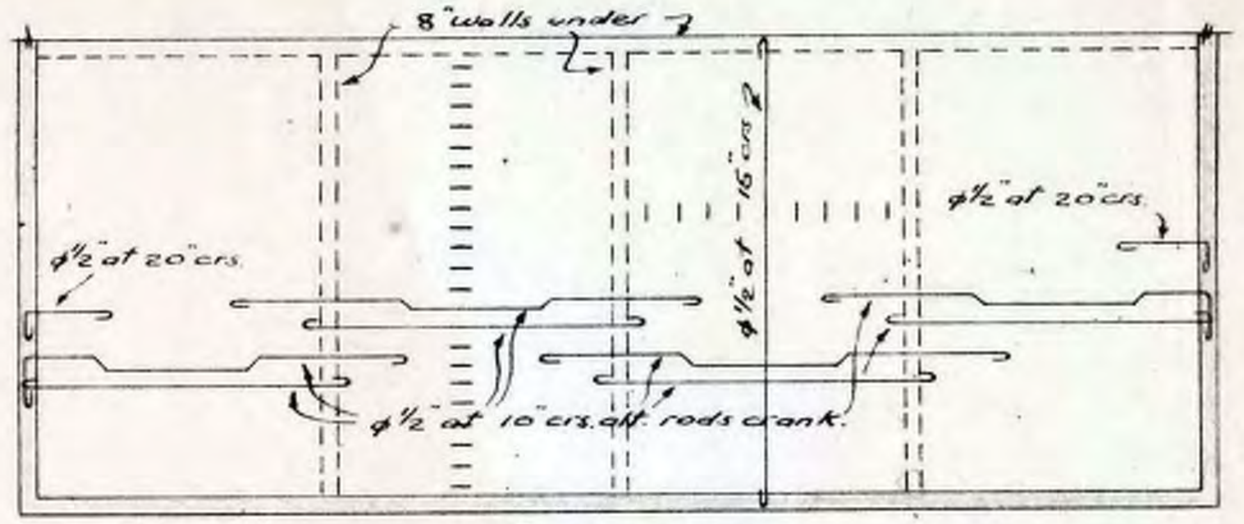
SECTION 3-3



SECTION 2-2



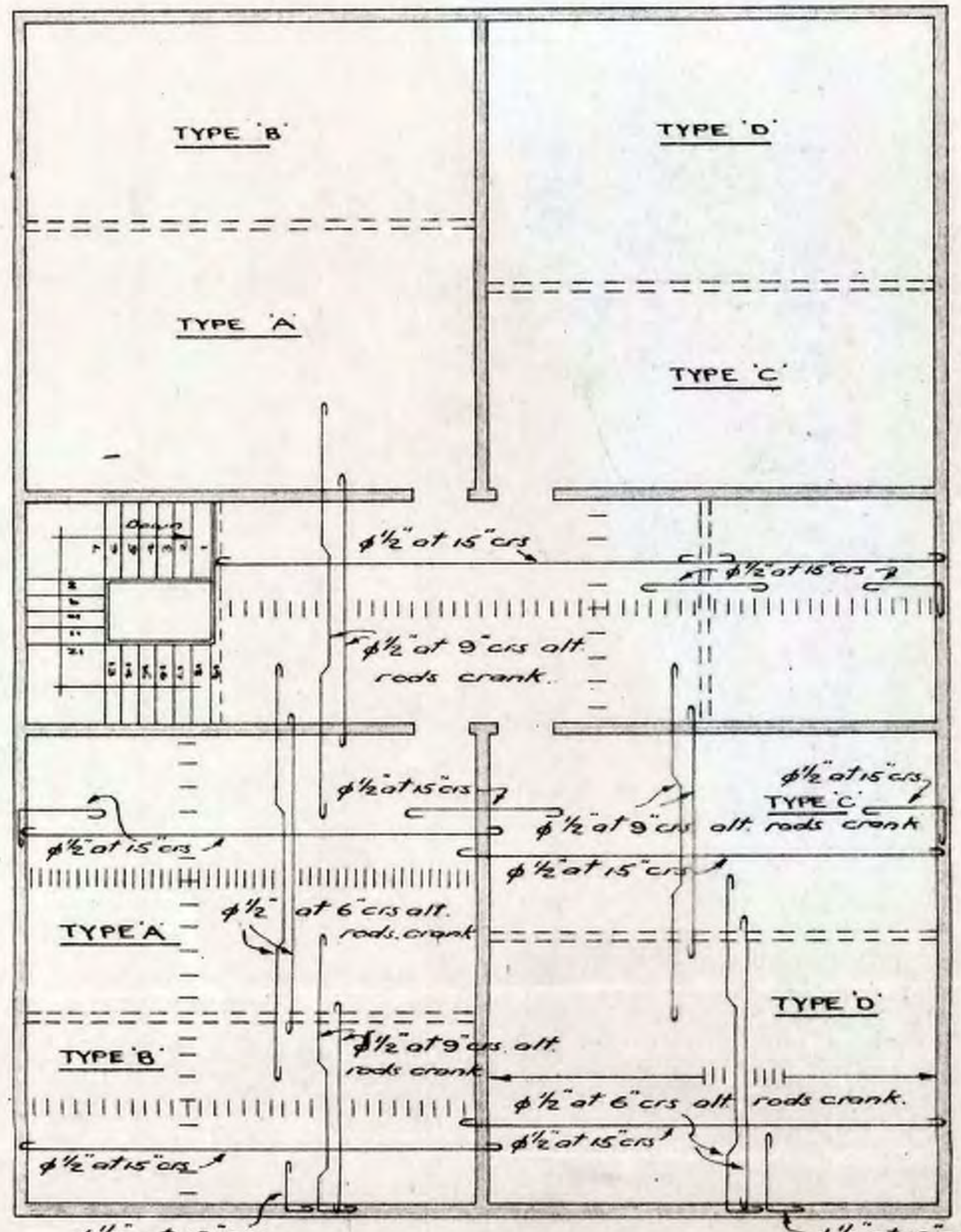
SECTION 1-1



5" THICK SLAB

DETAIL OF SLAB OVER GARAGES

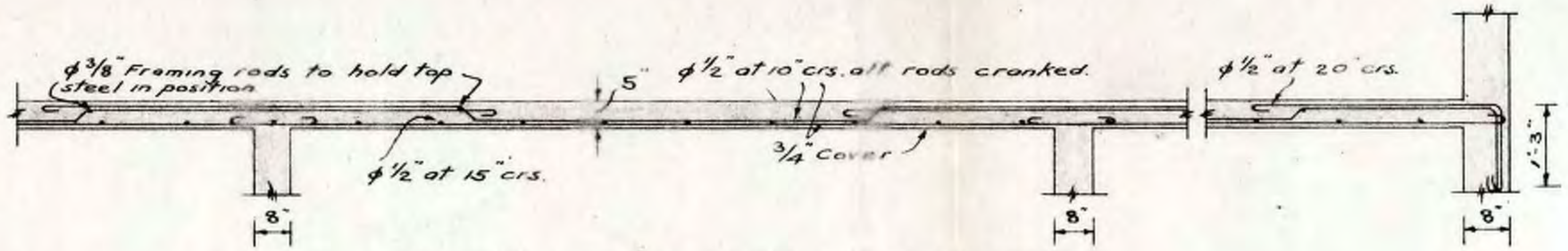
SCALE 1/8" = 1'-0"



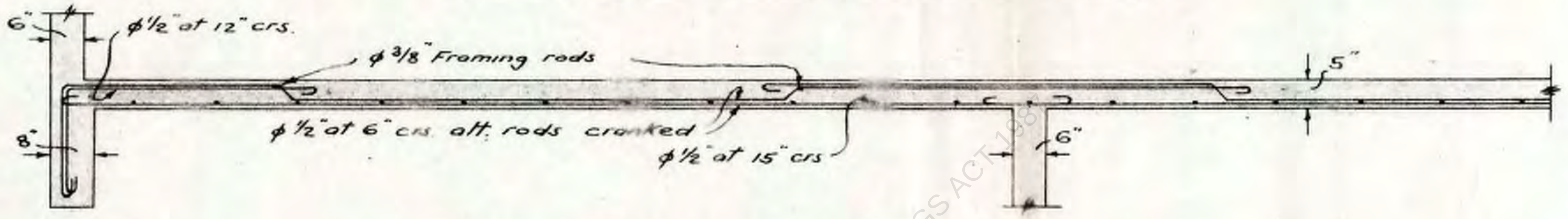
5" THICK SLAB.

FIRST FLOOR SLAB

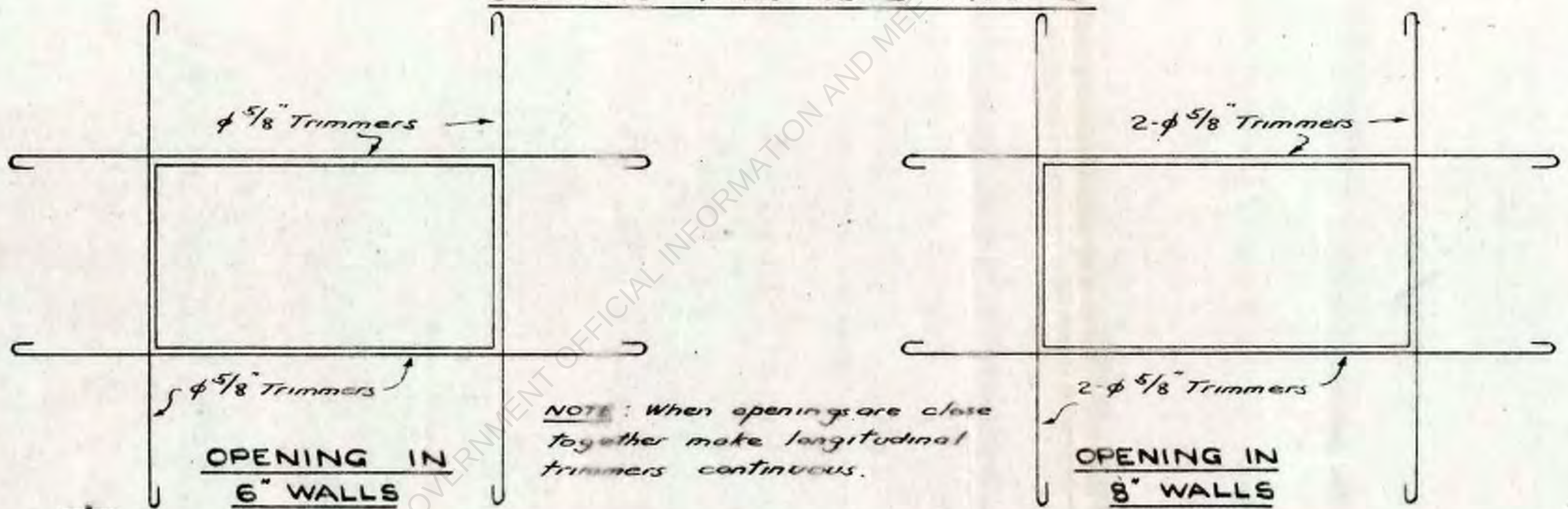
SCALE 1/8" = 1'-0"



SECTION THRU SLAB OVER GARAGES



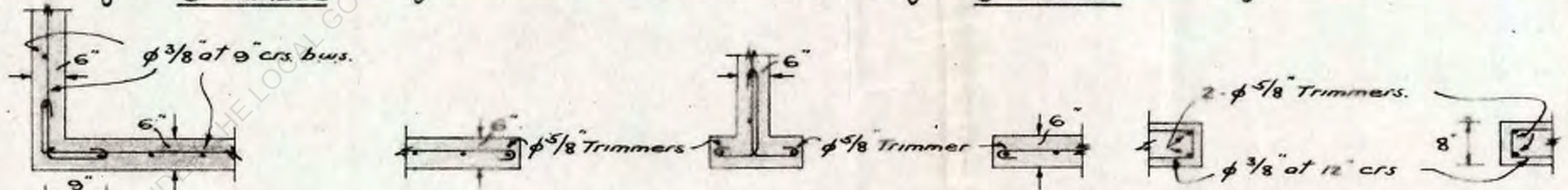
SECTION THRU SLAB TYPE "D"



NOTE: When openings are close together make longitudinal trimmers continuous.

OPENING IN 6" WALLS

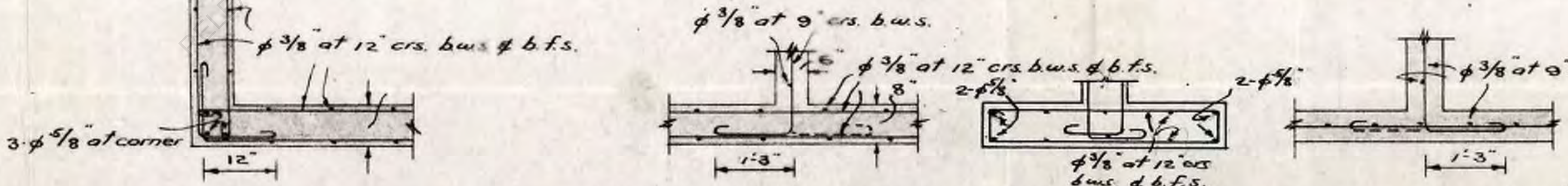
OPENING IN 8" WALLS



6" WALL

DETAIL OF 6" WALL AT ENTRANCE TO FLATS

DETAIL AT OPENINGS IN EXTERIOR WALL



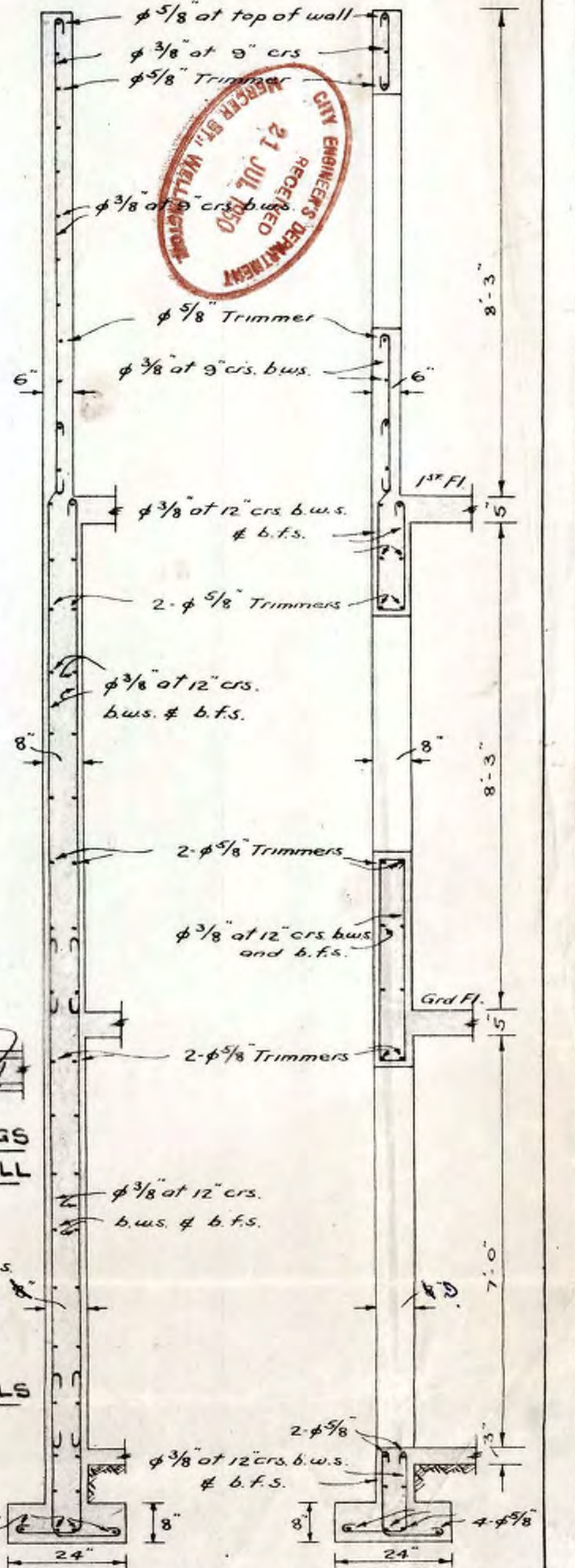
8" WALL DETAIL AT CORNER

JUNCTION 6" WALL & 8" WALL

9" WALL BETWEEN GARAGE DOORS.

JUNCTION OF 6" WALLS

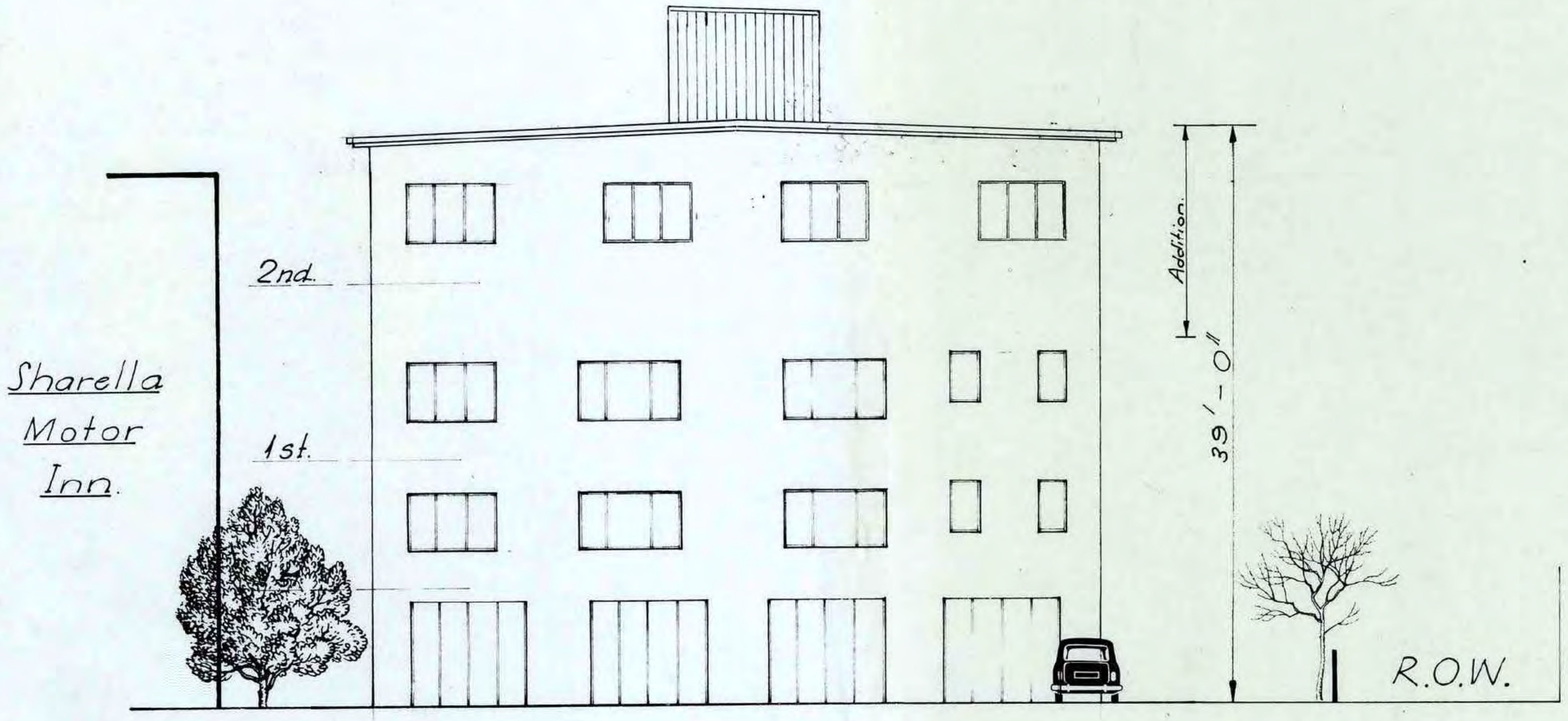
SCALE ½" = 1'-0"



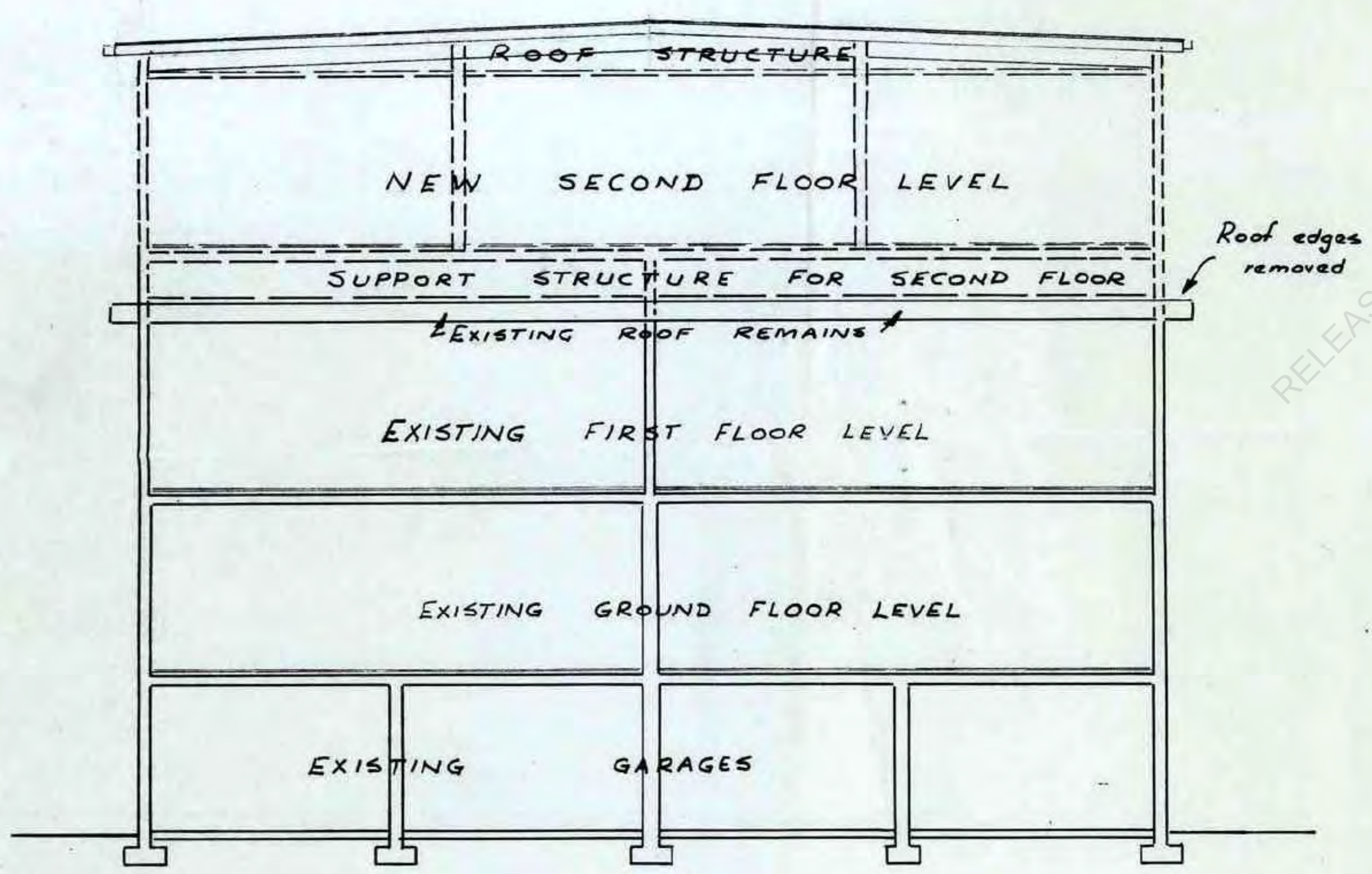
TYPICAL DETAIL - EXTERIOR WALLS BETWEEN OPENINGS AT OPENINGS

NOTE: *Footings*:- All footings to be taken down to solid and to be of the following sizes. All garage walls to have 24" x 8" reinforced with 4-φ5/8" rods. All other footings to be 21" x 8" reinforced with 3-φ5/8" rods.
Walls:- Exterior walls to be of sizes shown in typical cross section. All interior walls to be 8" thick below ground floor level, otherwise 6" for full height.

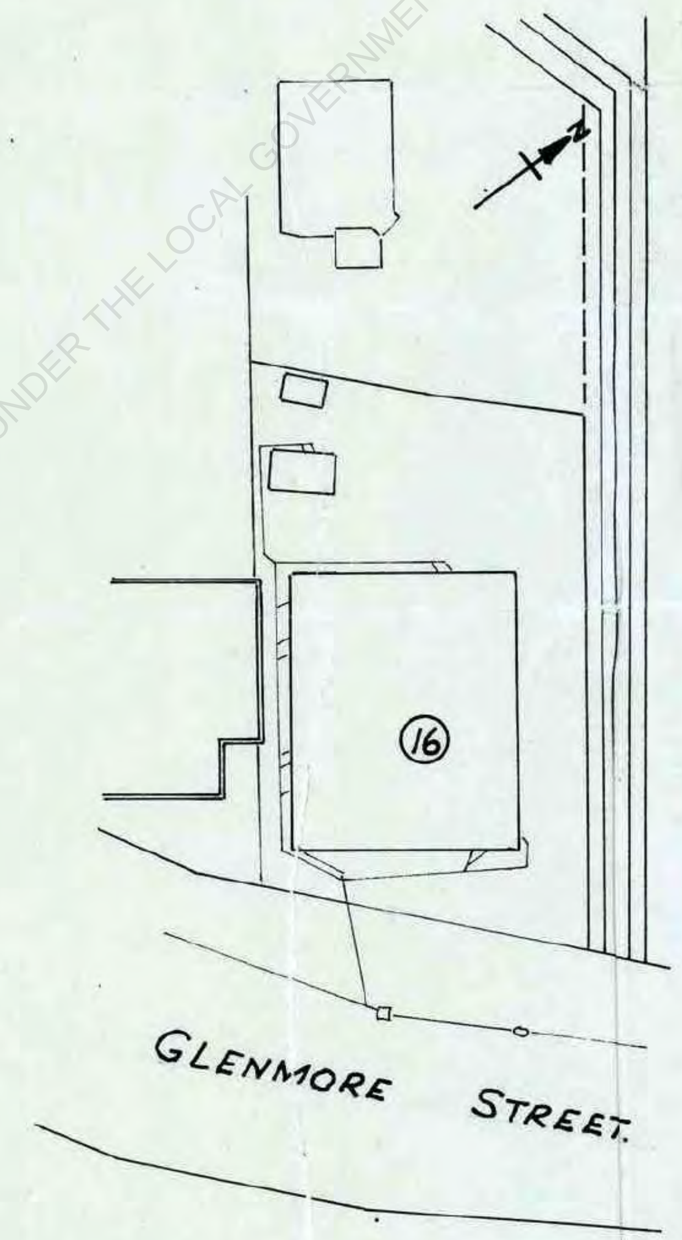
CITY ENGINEER'S DEPARTMENT RECEIVED 21 JUL 1950 MERCER ST. WELLINGTON



ELEVATION TO
GLENMORE ST.

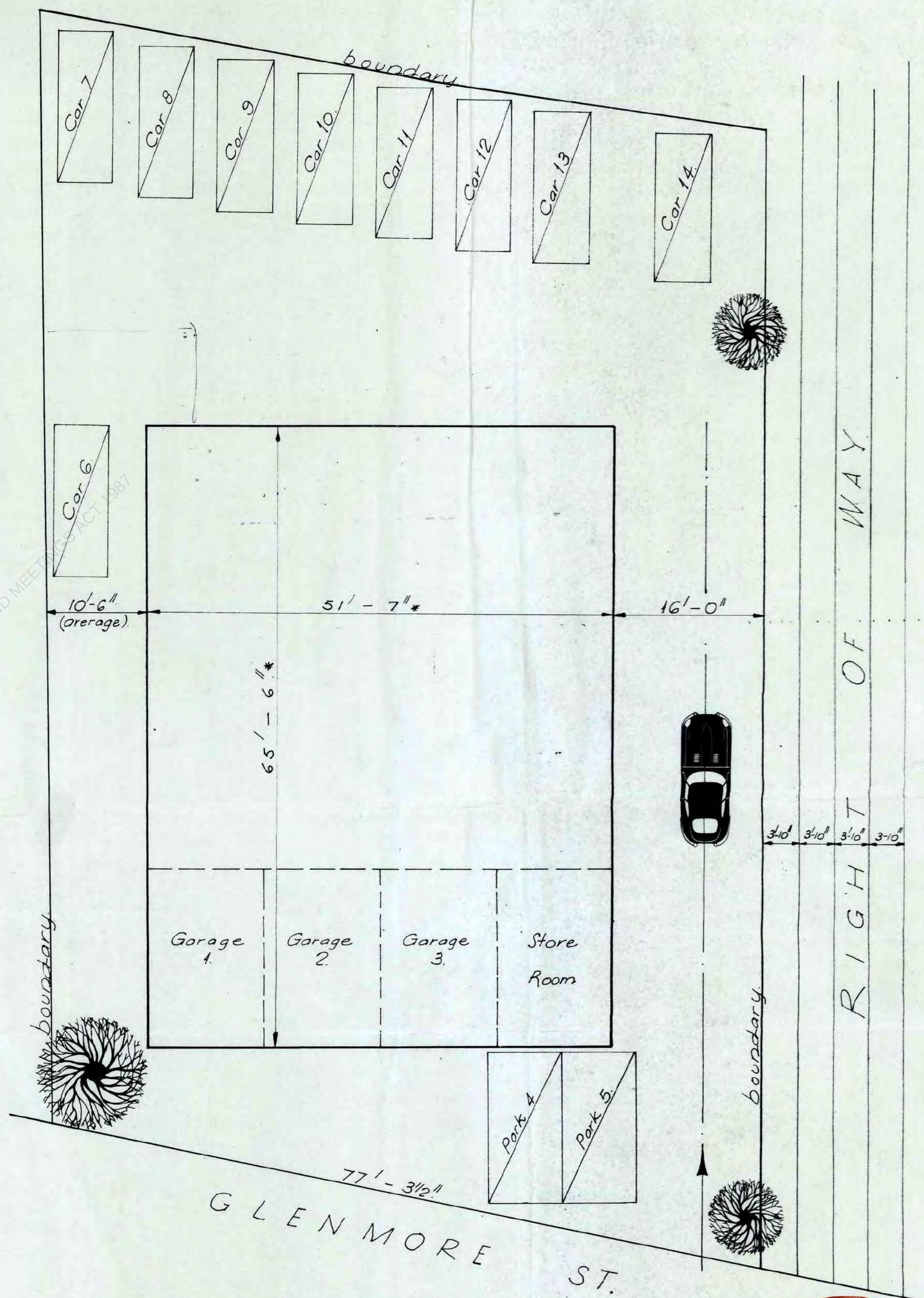


TYPICAL CROSS-SECTION.
(WITHOUT PENTHOUSE).



V.C.C. T25.

LOCALITY PLAN



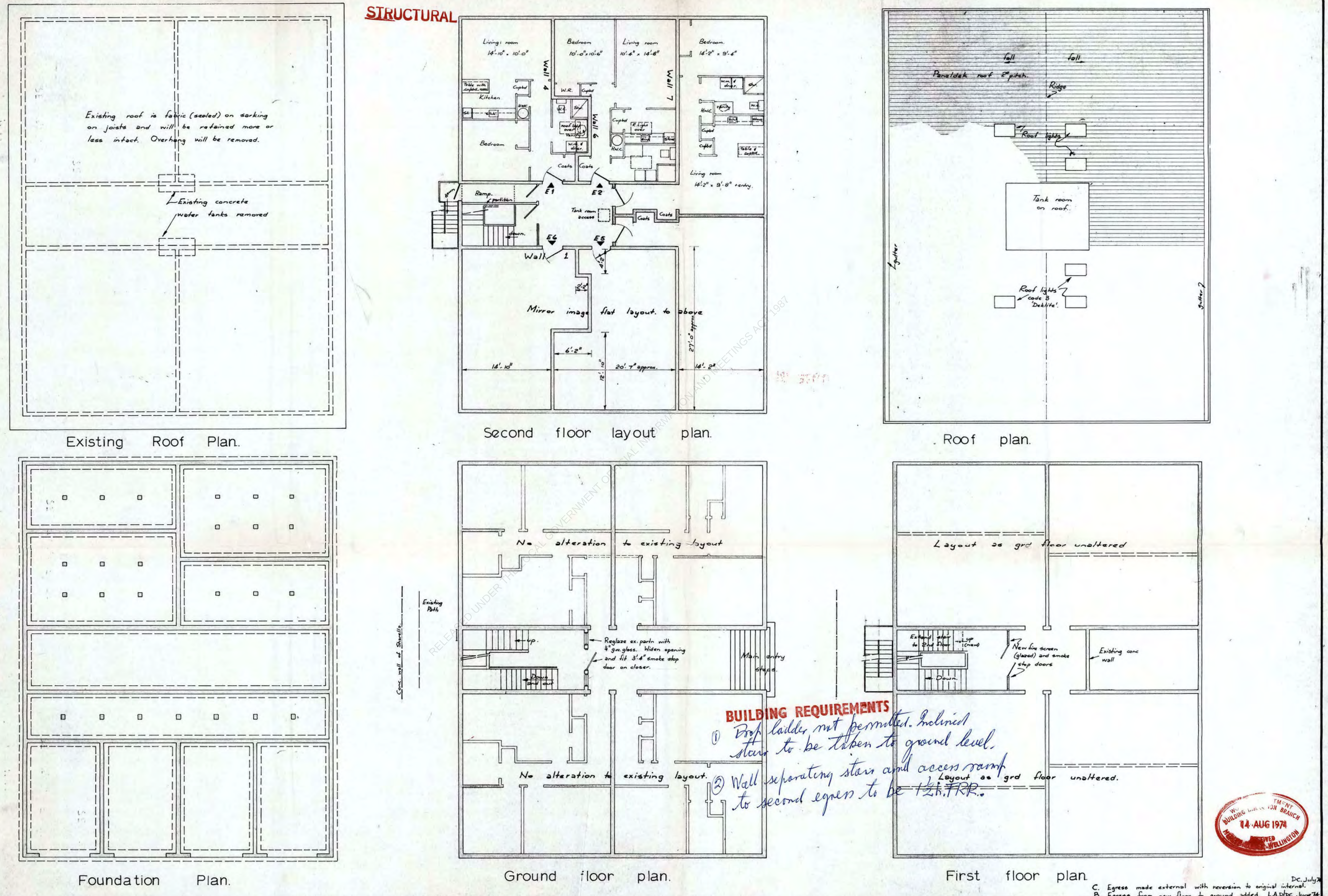
SITE PLAN.

* All dimensions are to be checked.

WORKS DEPARTMENT
BUILDING INSPECTION BRANCH
22 MAR 1974
RECEIVED
MERCER STREET, WELLINGTON

Amended Feb 74. Cutler.

PROPOSED ADDITION TO FAYMAR FLATS at 16 GLENMORE STREET, NORTHLAND for Mr. F. HAVILL	SITE PLAN, ELEVATION, and CROSS-SECTION.	DeTERTE & KERR-HISLOP CONSULTING CIVIL AND STRUCTURAL ENGINEERS. 2nd FLOOR "WOODCRAFT HOUSE" 75 WILLIS STREET, WELLINGTON, N.Z. PHONE 40-835	SCALE 1/8" = 1'0"
			PASSED / Dwn. D.Scull.
			Tcd.
			DATE Oct '73 / Chkd.
			D.K. 1163/1A



AMENDED PLAN
APPROVED BY [signature] DATE 23/8/74

BUILDING REQUIREMENTS
1. Drop ladder not permitted. Inclined stairs to be taken to ground level.
2. Wall separating stairs and access ramp to second egress to be 1½ h. FRR.

WELLINGTON BUILDING INSPECTION DEPARTMENT — RECEIVED 14 AUG 1974

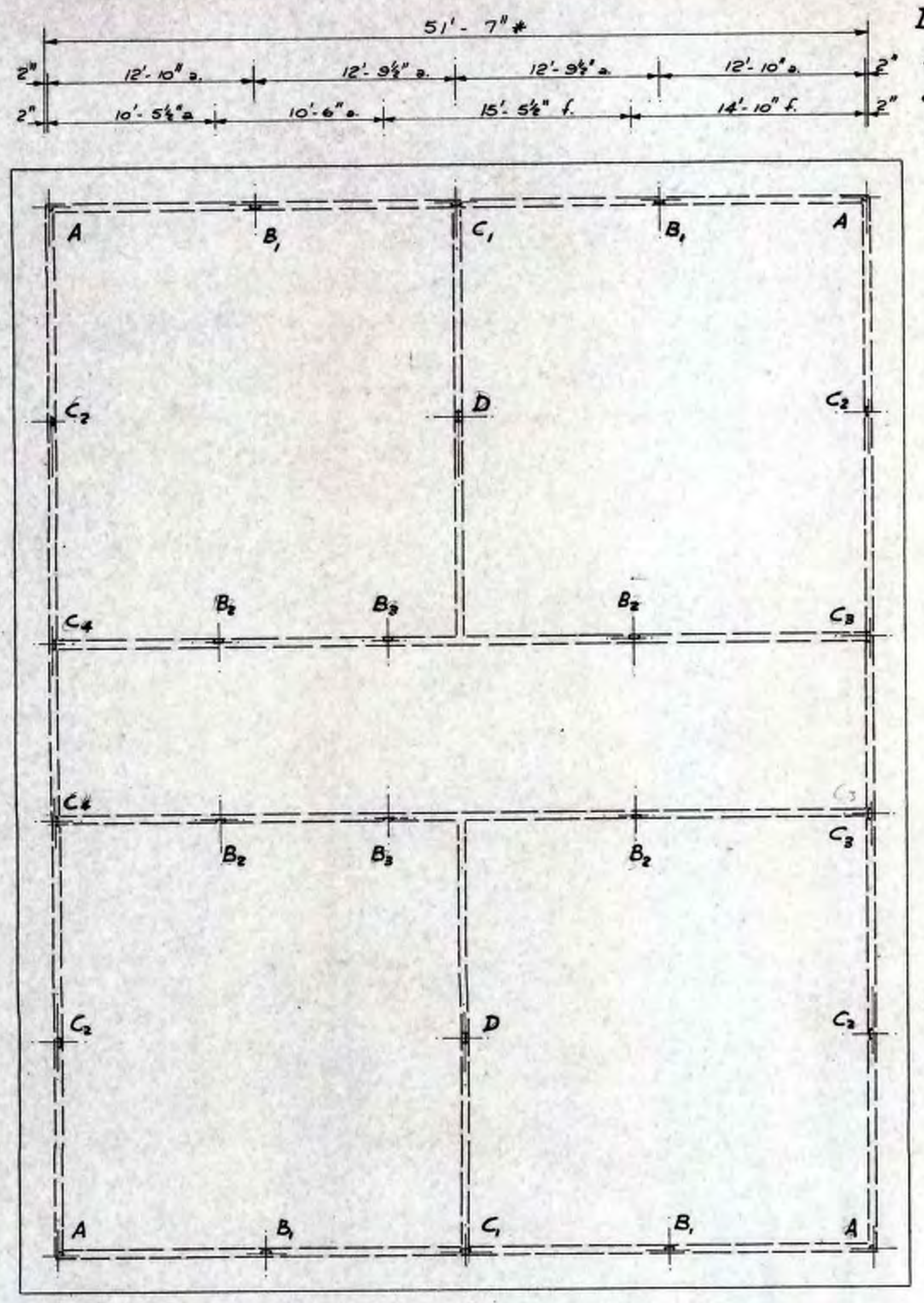
PROPOSED ADDITION TO FAYMAR FLATS
at 16 GLENMORE STREET, NORTHLAND
for Mr F. HAVILL.

LAYOUT PLANS,
EXISTING and PROPOSED.

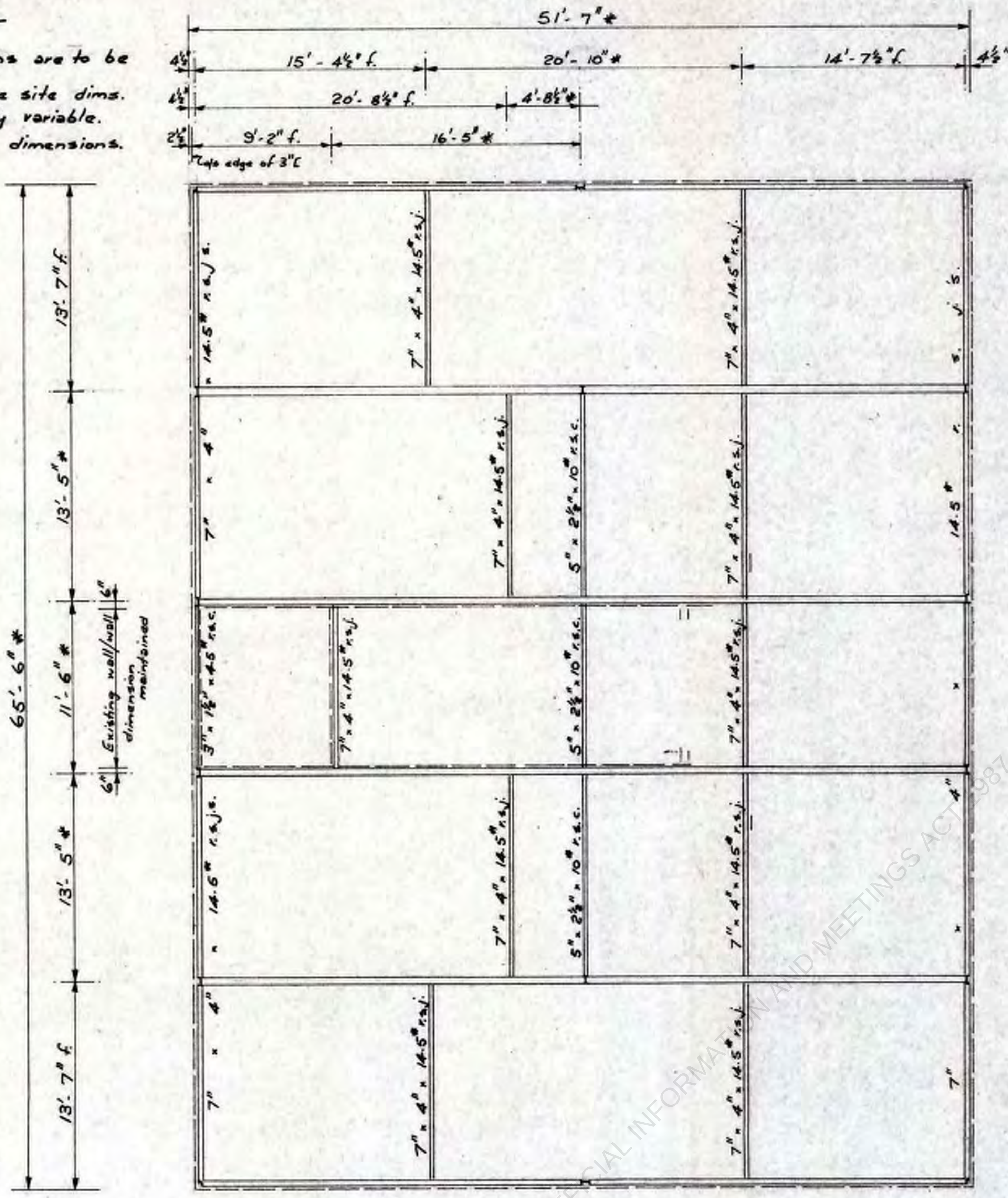
DeTERTE & KERR-HISLOP
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
2nd FLOOR "WOODCRAFT HOUSE"
75 WILLIS STREET,
WELLINGTON, N.Z.
PHONE 40-835

SCALE ⅛" to 1'-0"			
PASSED	Dwn.	Cutler	D.K. 1163/2C
	Tcd.		
DATE Mar 74	Chkd.		

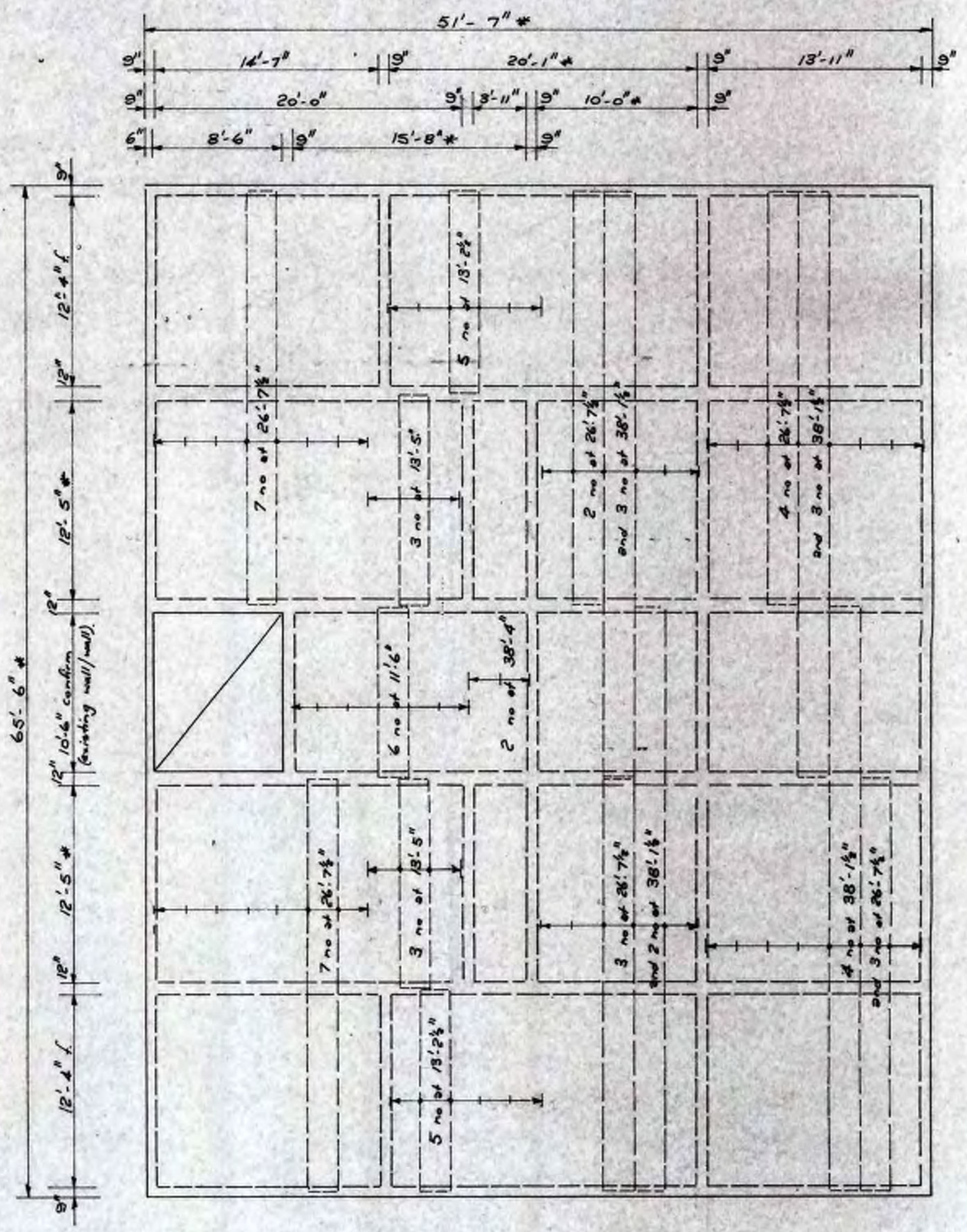
Dimensions:-
All dimensions are to be checked. * are site dims.
'a' are slightly variable.
'f' are fixed dimensions.



Plan of existing roof showing new frame support positions.



Steelwork framing plan for new second floor.

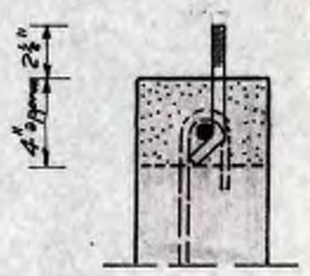


Floor slab and unit layout plan for new second floor.

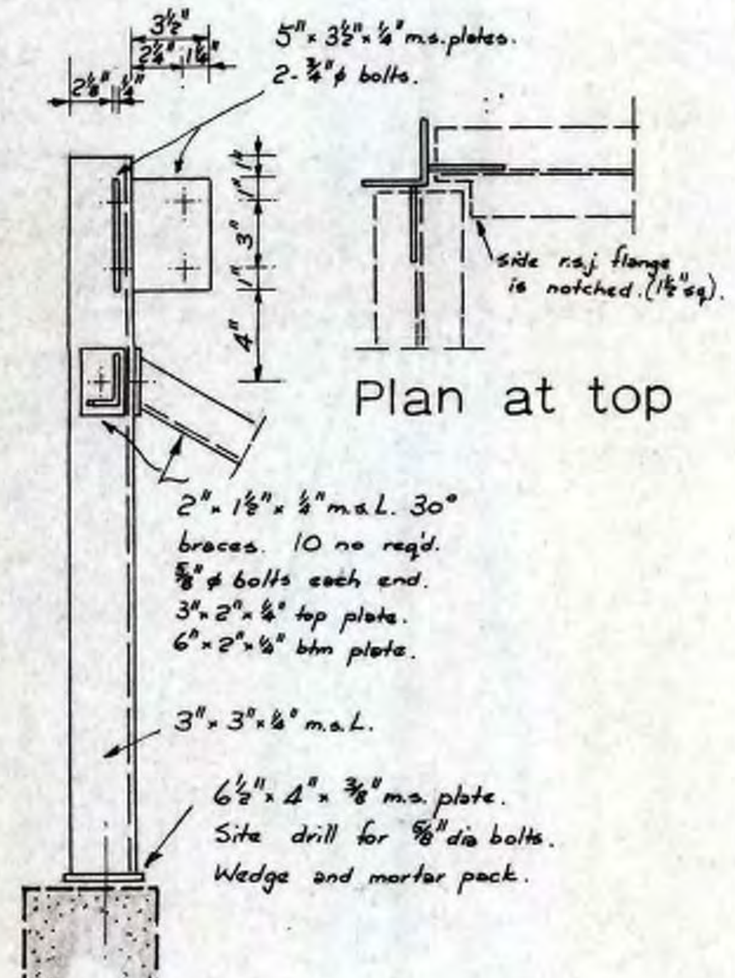
Schedule of Dimond Hi-Bond 2" V 22g floor units.			
Length	Count	Total	
38'-4"	2	12	
38'-1½"	5+7	2	} Laid in staggered pattern.
26'-7½"	7+7+5+7	26	
13'-5"	3+3	6	
13'-2½"	5+5	10	
11'-6"	6	6	One to be cut down.

Existing top

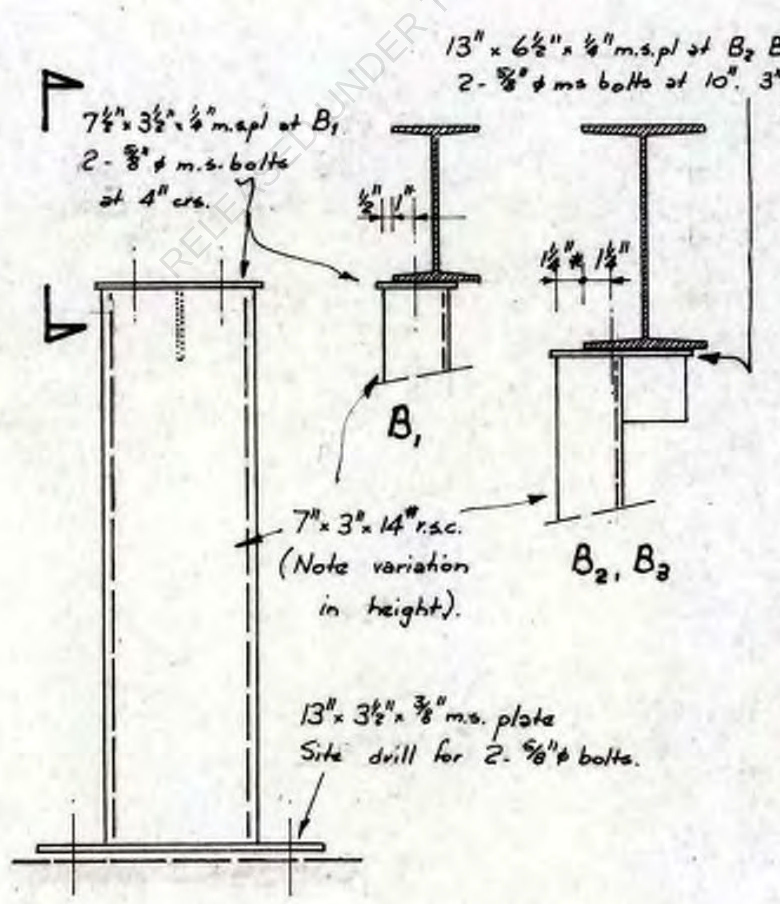
Break away concrete to fully expose top ⅝" dia trimmer for 12". Weld on ⅝" dia galv rag bolts to project 2½" and make good concrete with suitable mortar. 2 bolts per fixing.



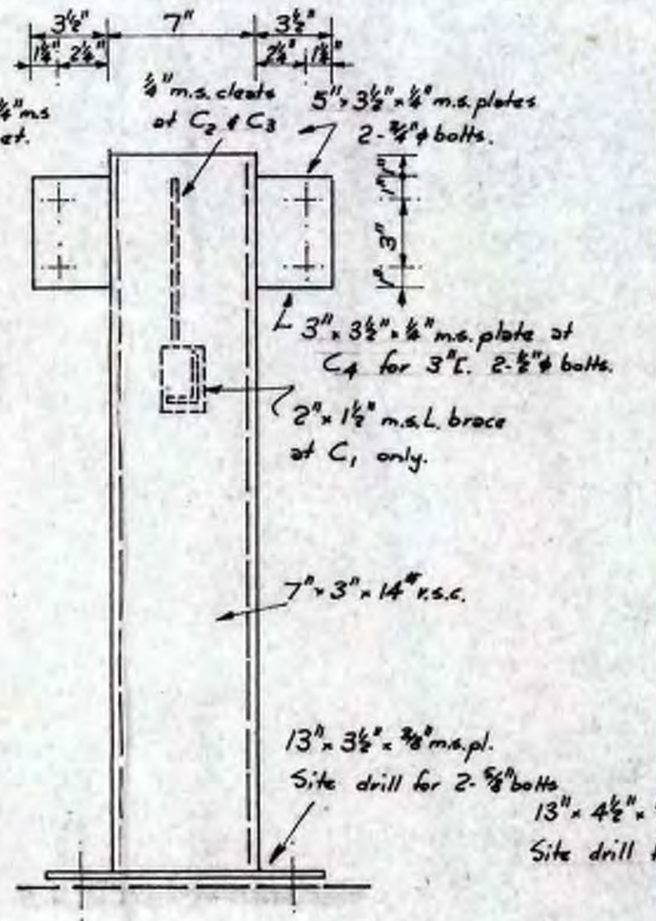
Proposed fixing.



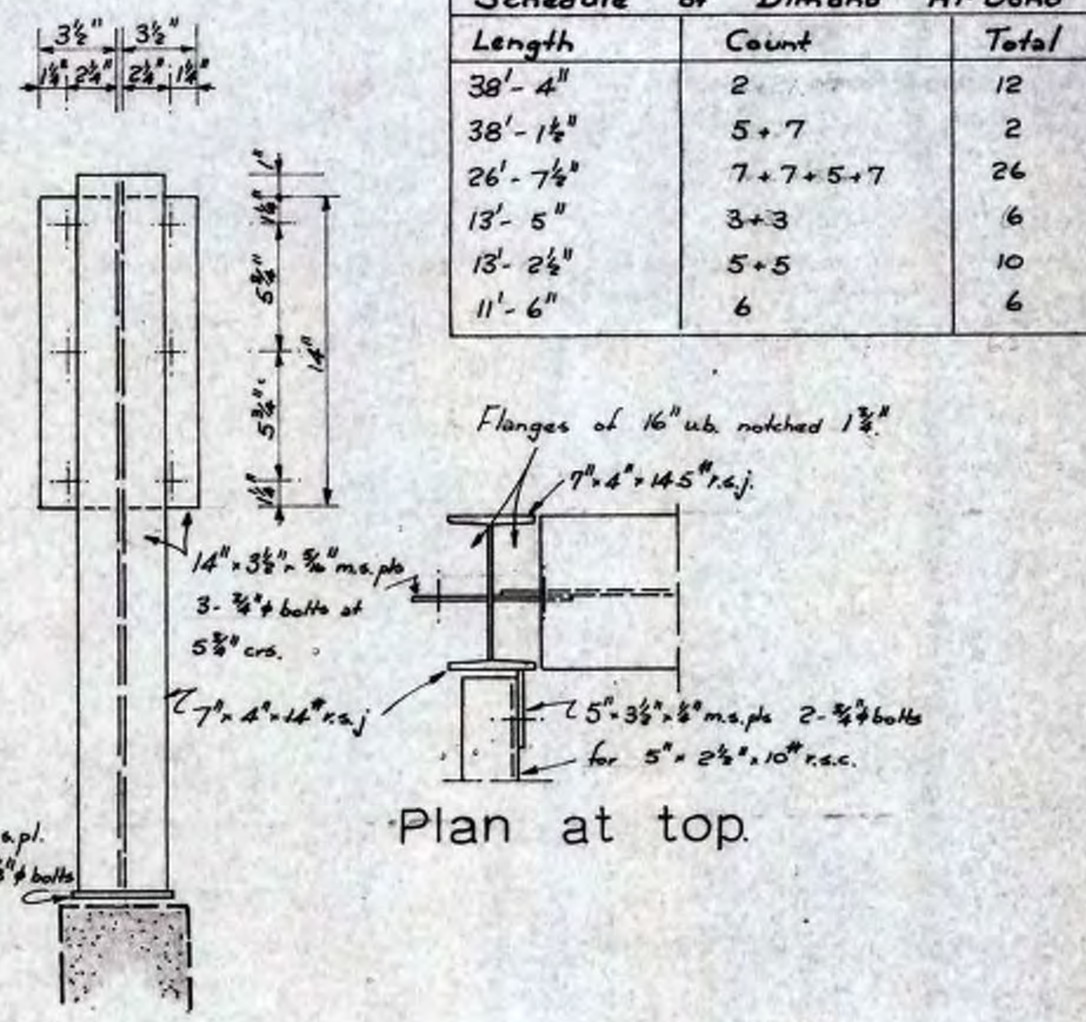
Elevation Type 'A' support.



Elevation Type 'B' support.



Elevation Type 'C' support



Elevation Type 'D' support.

PROPOSED ADDITION TO FAYMAR FLATS at 16 GLENMORE STREET, NORTHLAND for Mr. F. HAVILL.

SECOND FLOOR FRAMING PLANS and SUPPORT POST DETAILS.

DeTERTE & KERR-HISLOP
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
2nd FLOOR "WOODCRAFT HOUSE" 75 WILLIS STREET, WELLINGTON, N.Z.
PHONE 40-835

SCALE ⅛" & 1½" to 1'-0"			
PASSED	Dwn.	Cutler	D.K. 1163/3
- -	Tcd.		
DATE Mar '74	Chkd.		

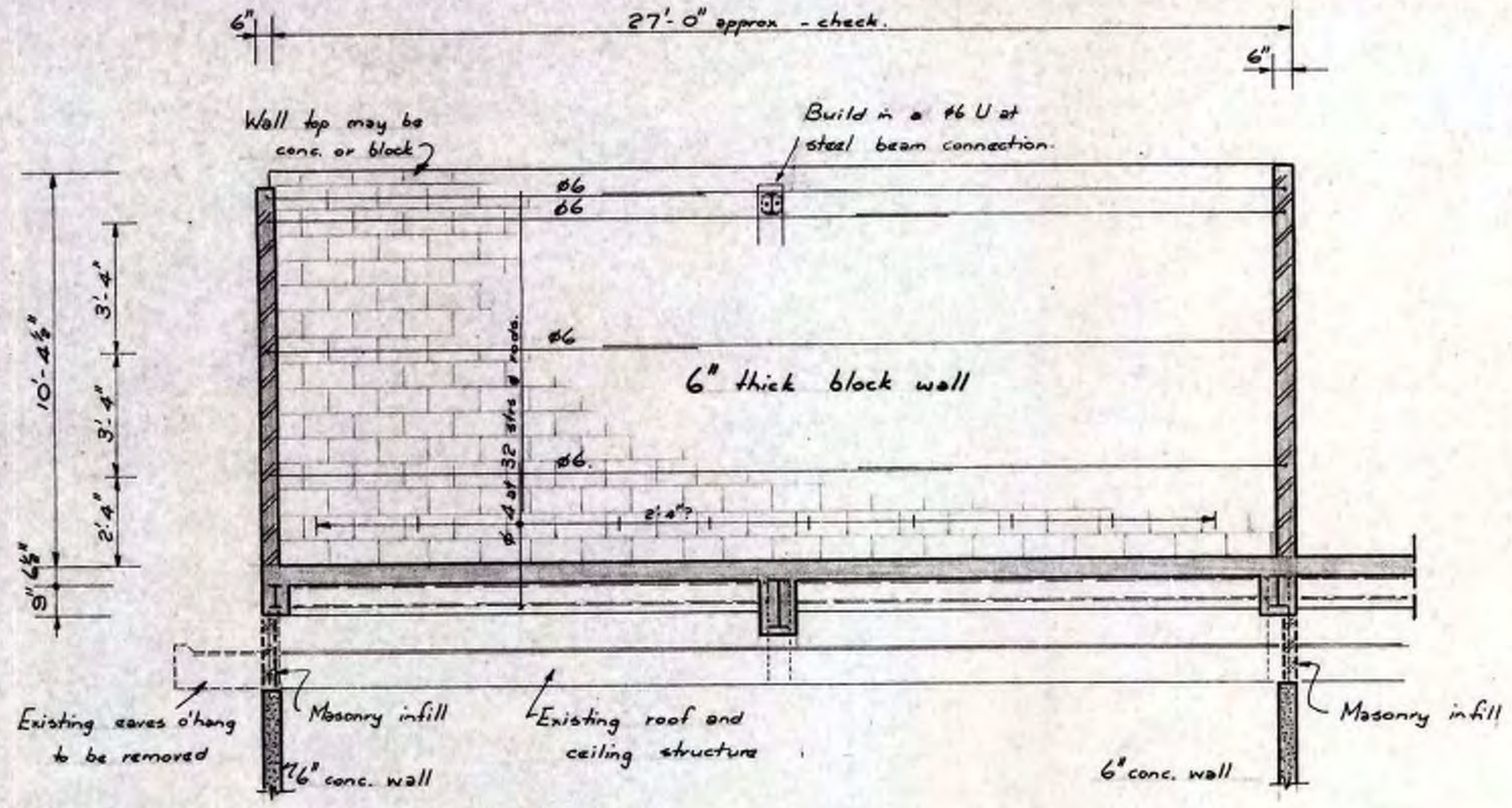
PROPOSED ADDITION TO FAYMAR - FLATS
at 16 GLENMORE STREET, NORTHLAND
for Mr. F. HAVILL.

STEEL FRAMING - ELEVATIONS and DETAILS.

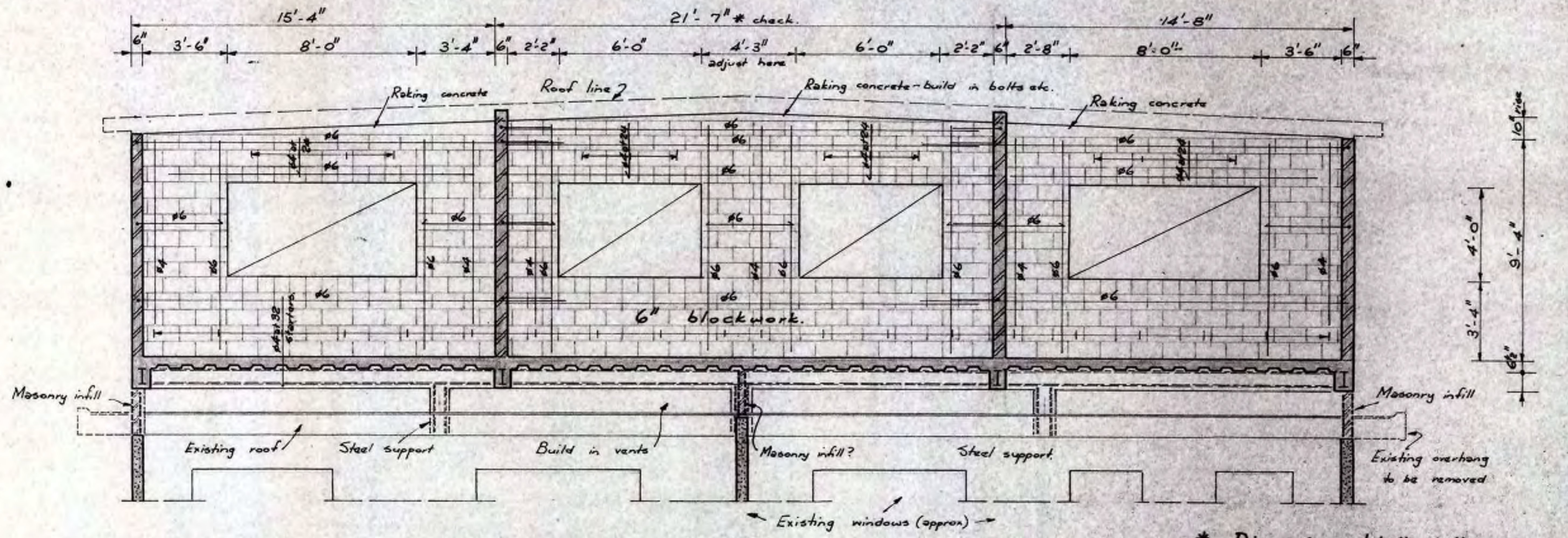
DeTERTE & KERR-HISLOP
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
2nd FLOOR "WOODCRAFT HOUSE" 75 WILLIS STREET, WELLINGTON, N.Z.
PHONE 40-835

SCALE ⅜"; 1½" to 1'0"
PASSED
Dwn. Cutter
Tcd.
Chkd.
DATE Mar '74.

D.K. 1163/4

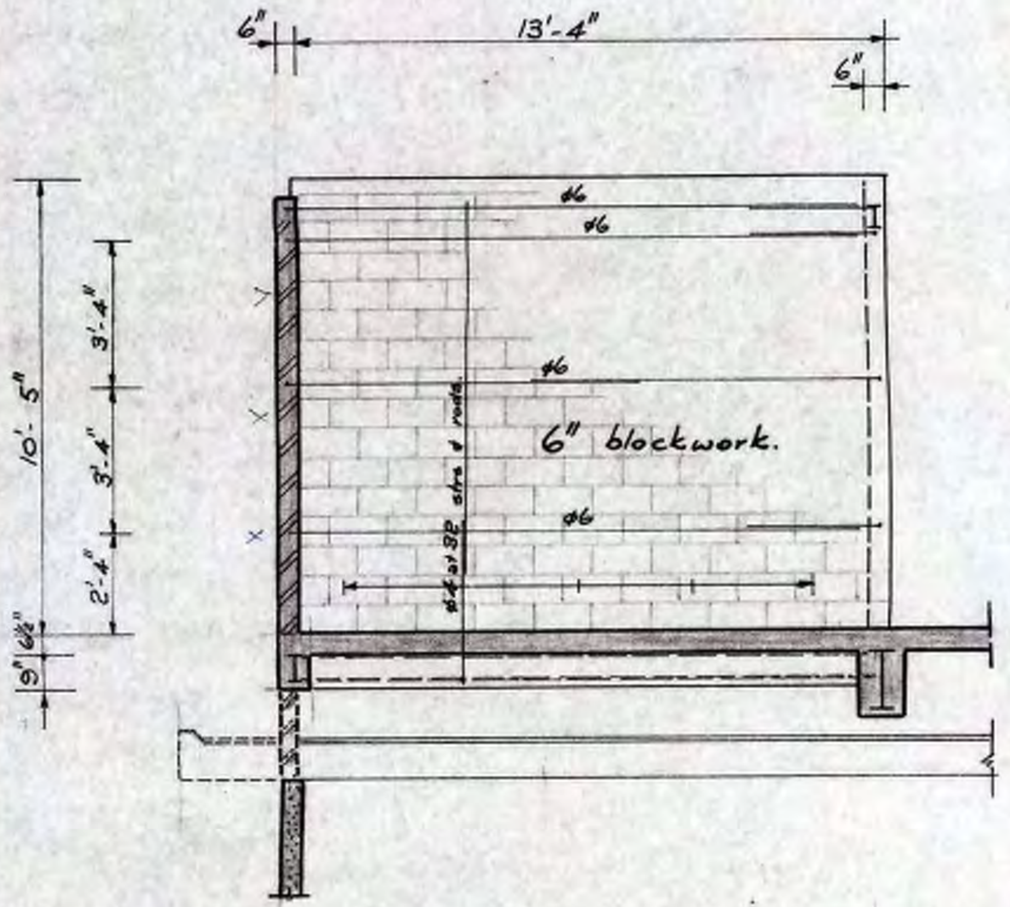


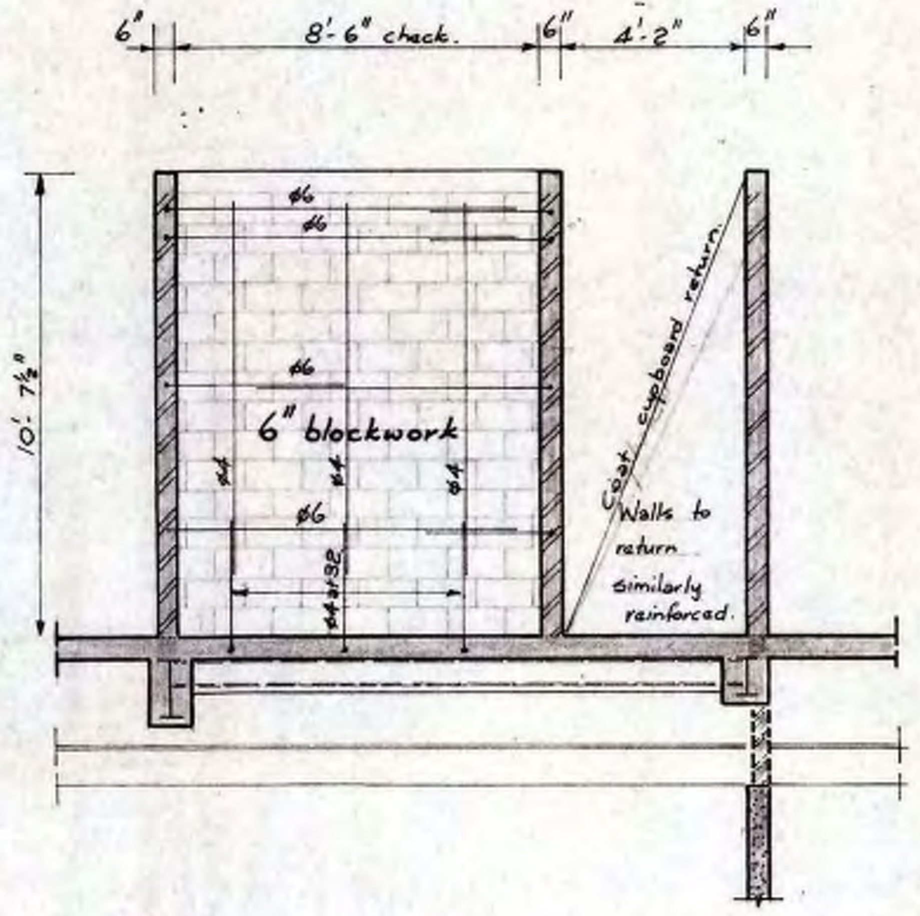
Elevation of wall 7. (2 of).



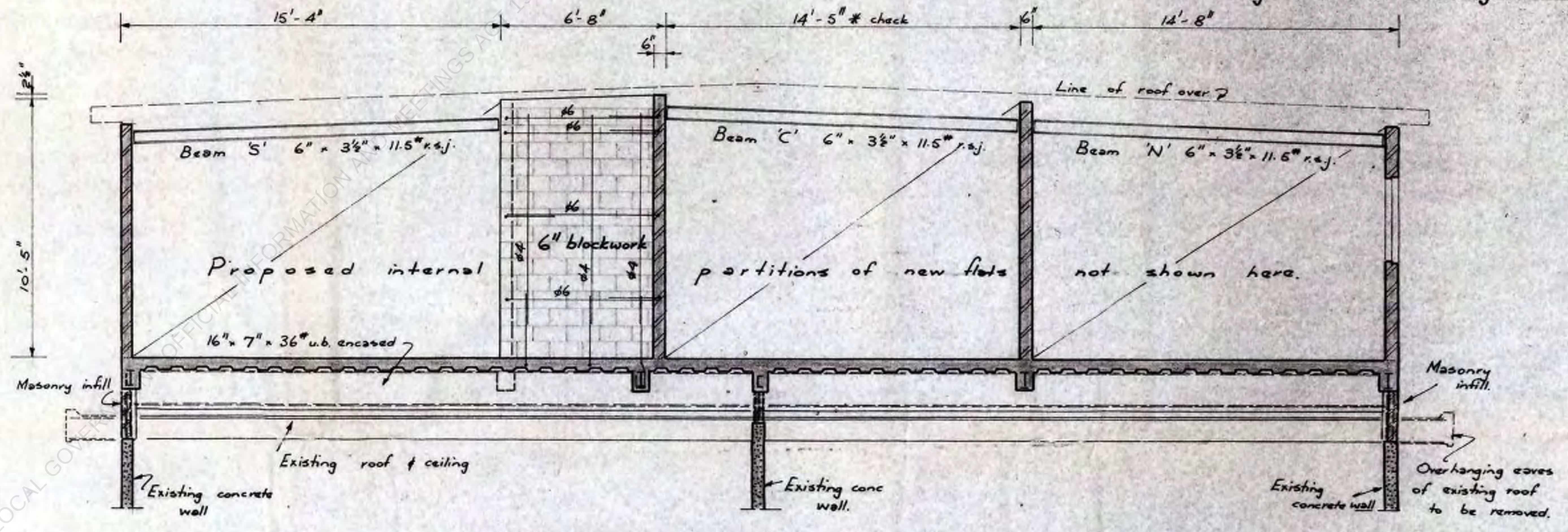
Elevation of rear wall (Front similar)

* Dimensions labelled thus are subject to site measurement and adjustment if necessary.

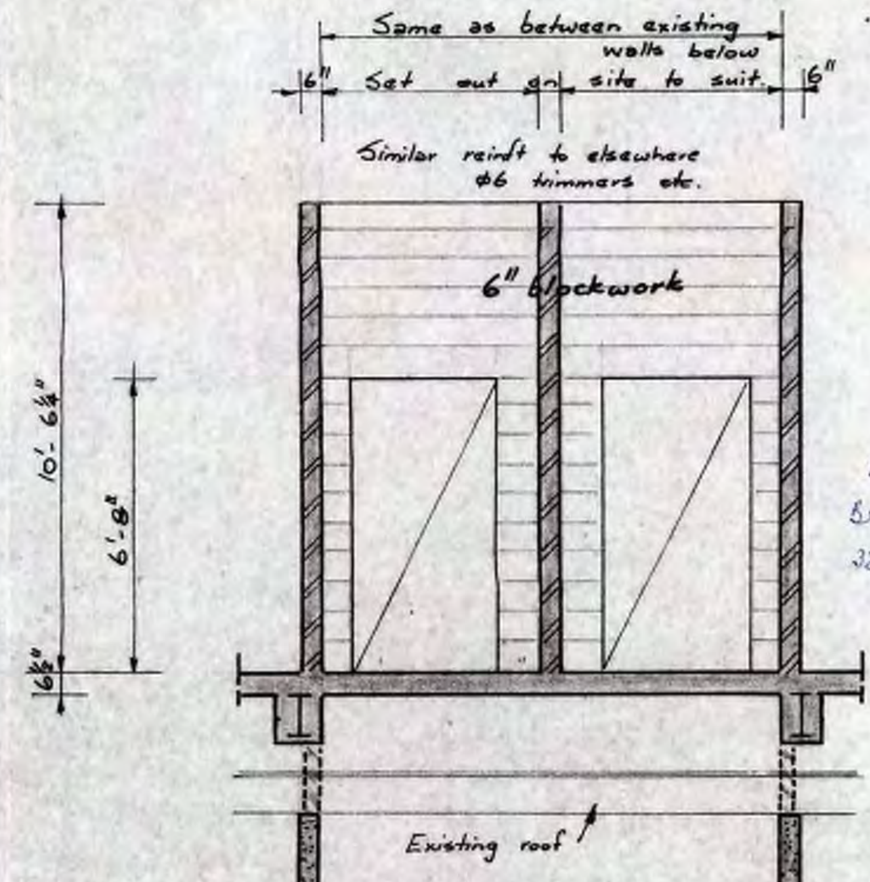


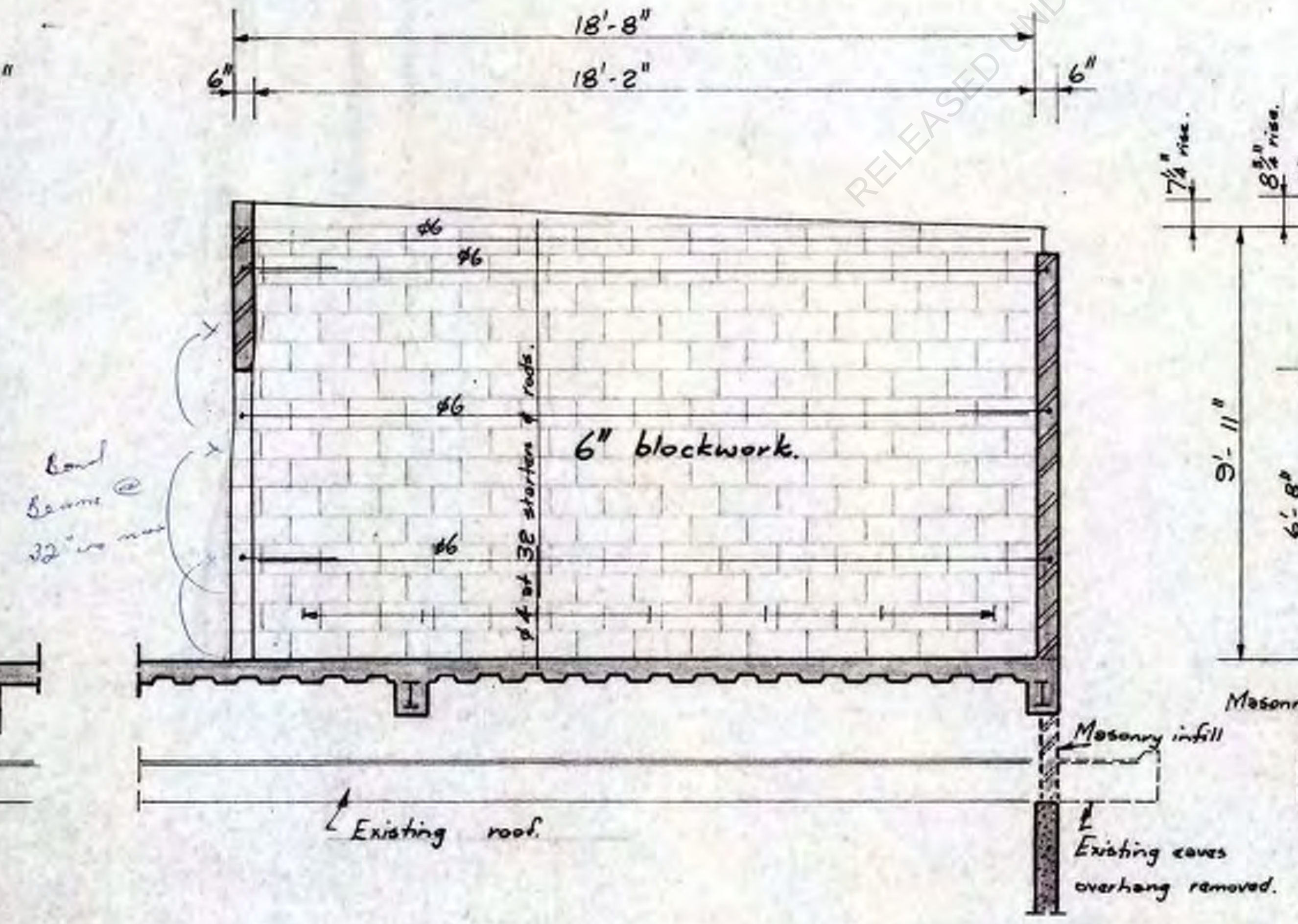


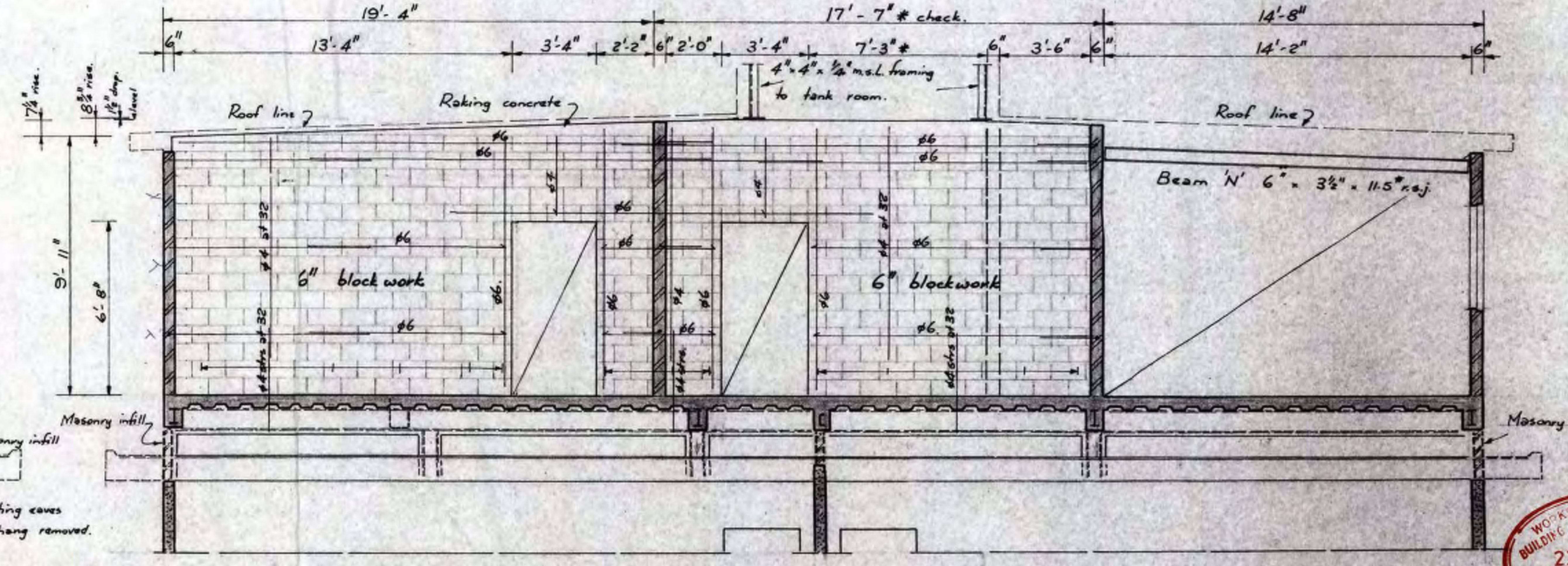
Elevation - wall 4 (2 of). Elevation - wall 6 (2 of).



Elevation of main framing over flats and wall 5 (Rear shown, front similar).







Elevation of wall 3 Elevation of wall 2. Elevation of wall 1 (Front shown, rear similar).

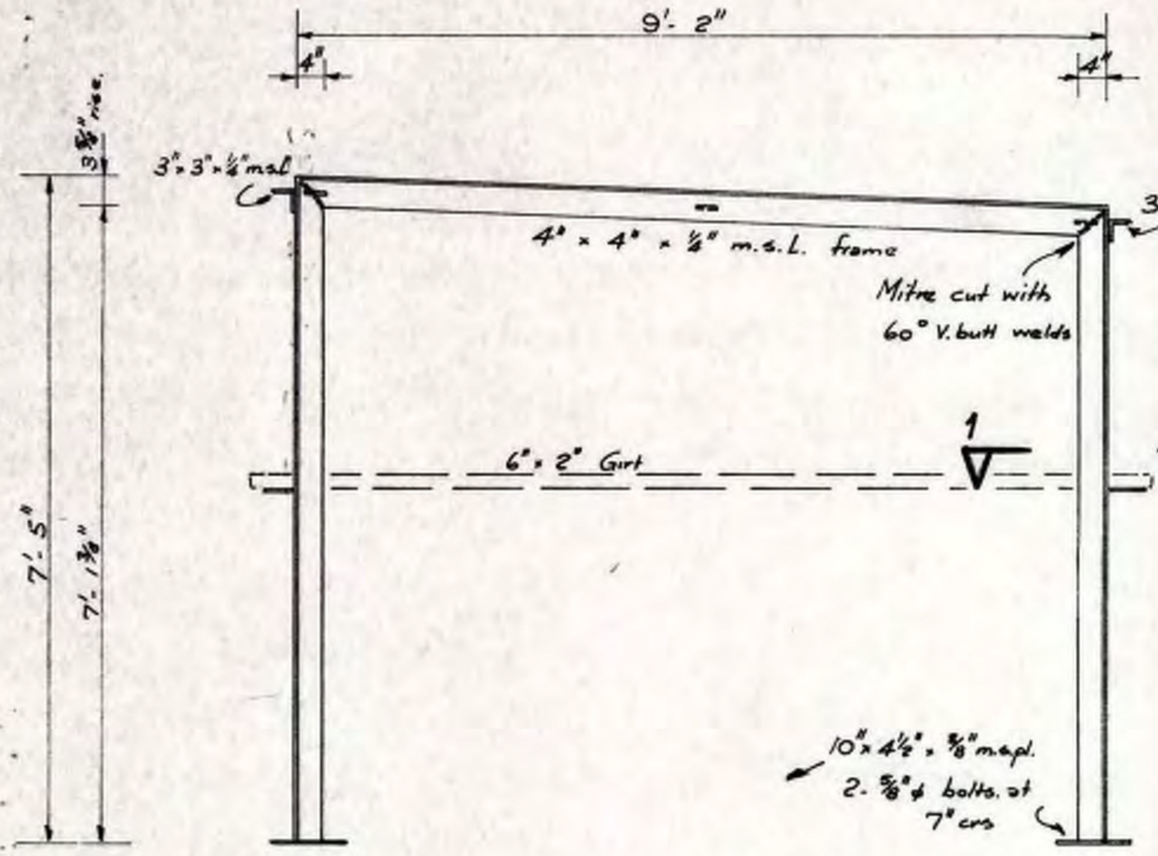
PROPOSED ADDITION TO FAYMAR FLATS
at 16 GLENMORE STREET, NORTHLAND
for Mr. F. HAVILL.

ELEVATIONS OF BLOCK WALLS.

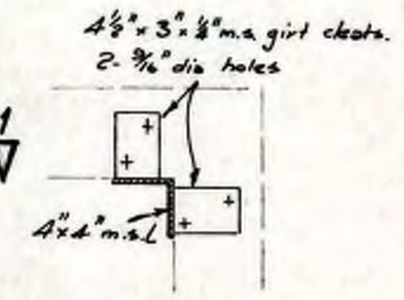
DeTERTE & KERR-HISLOP
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
2nd FLOOR "WOODCRAFT HOUSE"
75 WILLIS STREET,
WELLINGTON, N.Z.
PHONE 40-835

SCALE ¼"		
PASSED	Dwn.	Cutler
	Tcd.	
DATE Mar 74	Chkd.	

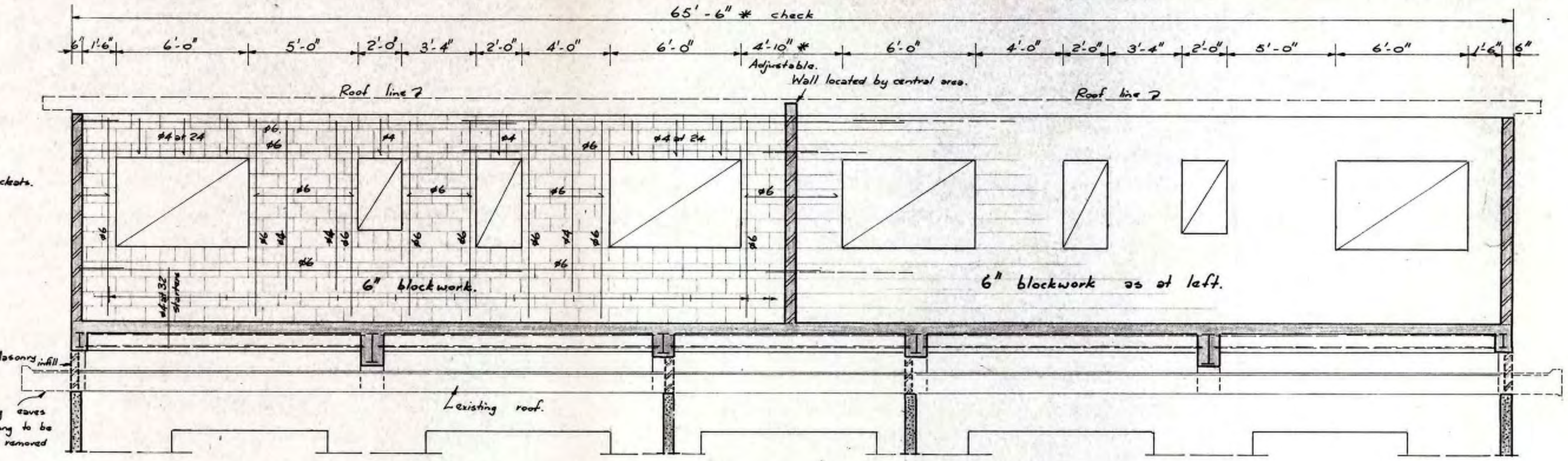
D.K. 1163/5



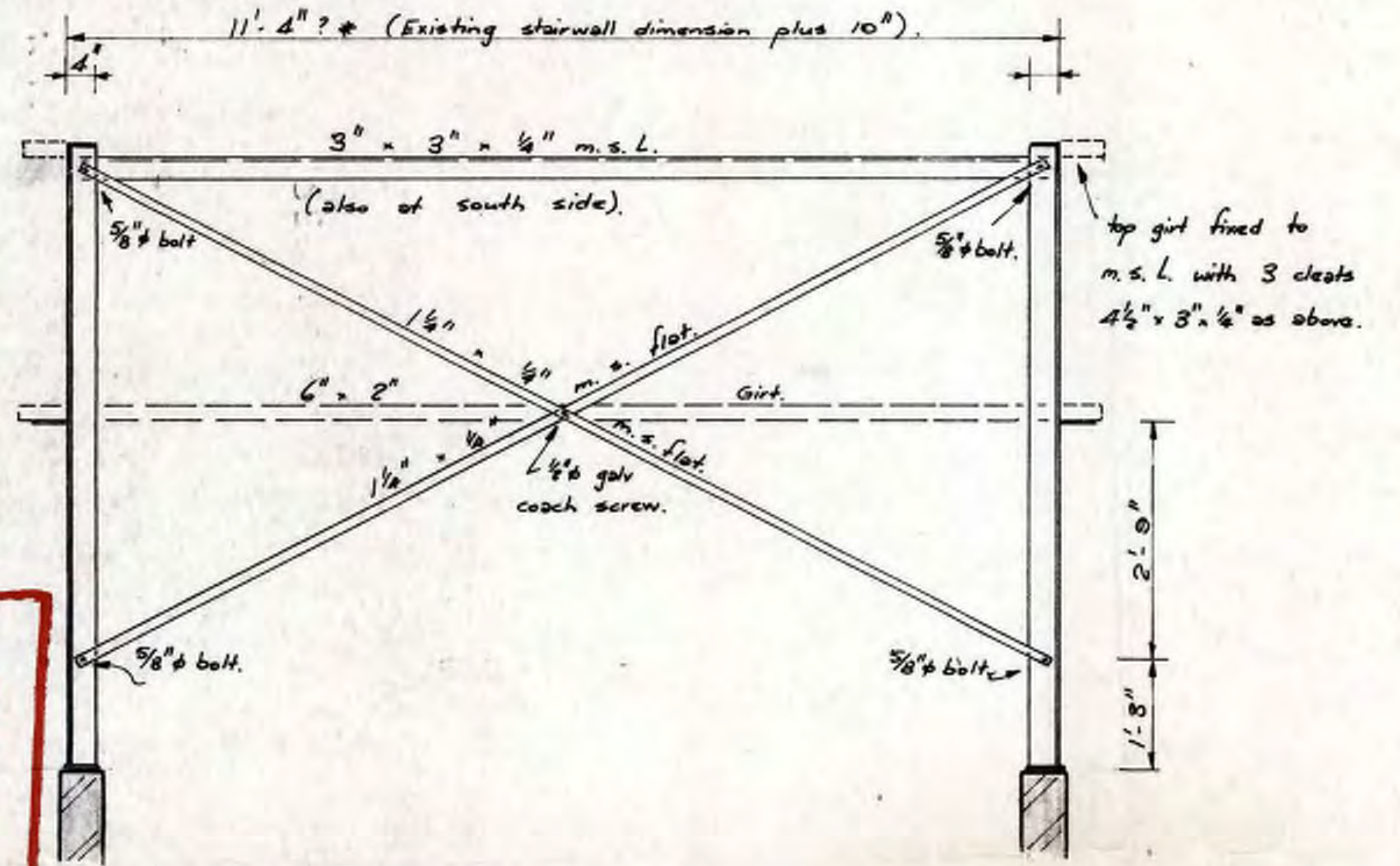
Side wall frame
(Two required at front and rear sides).



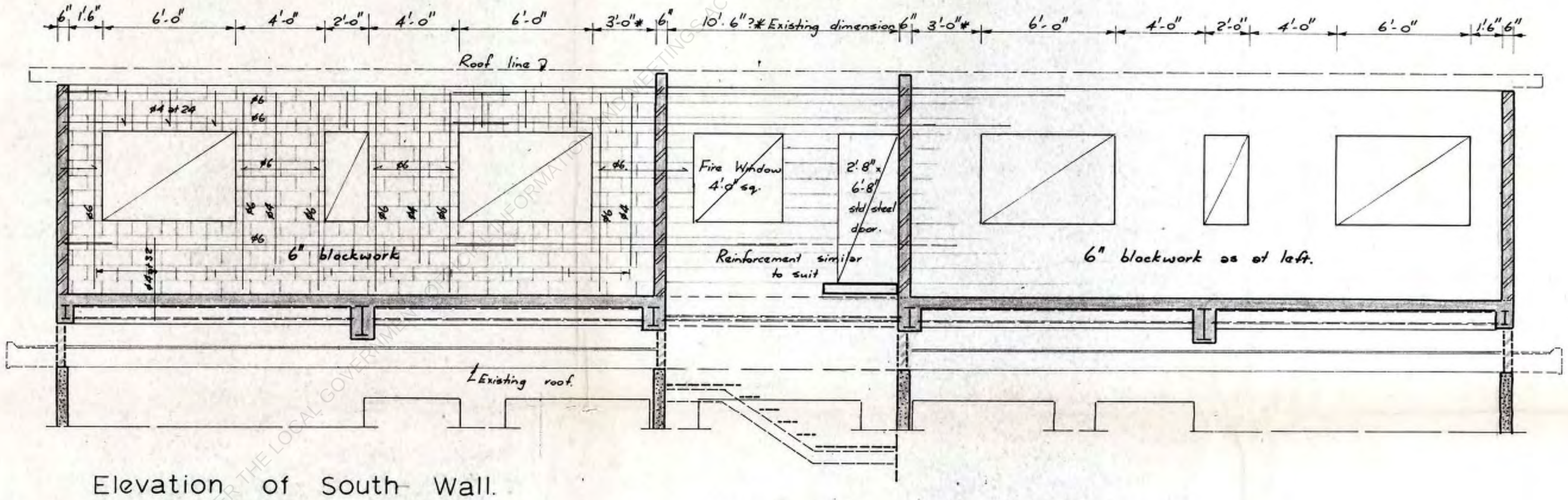
Sect 1-1.



Elevation of North Wall.



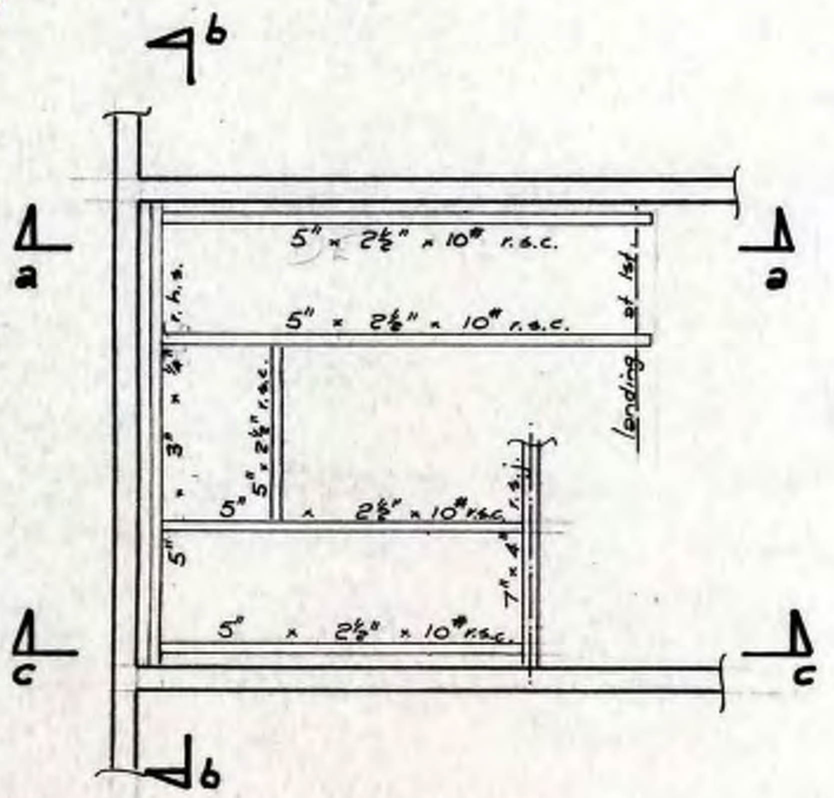
End wall frame
(North side shown, South side similar)



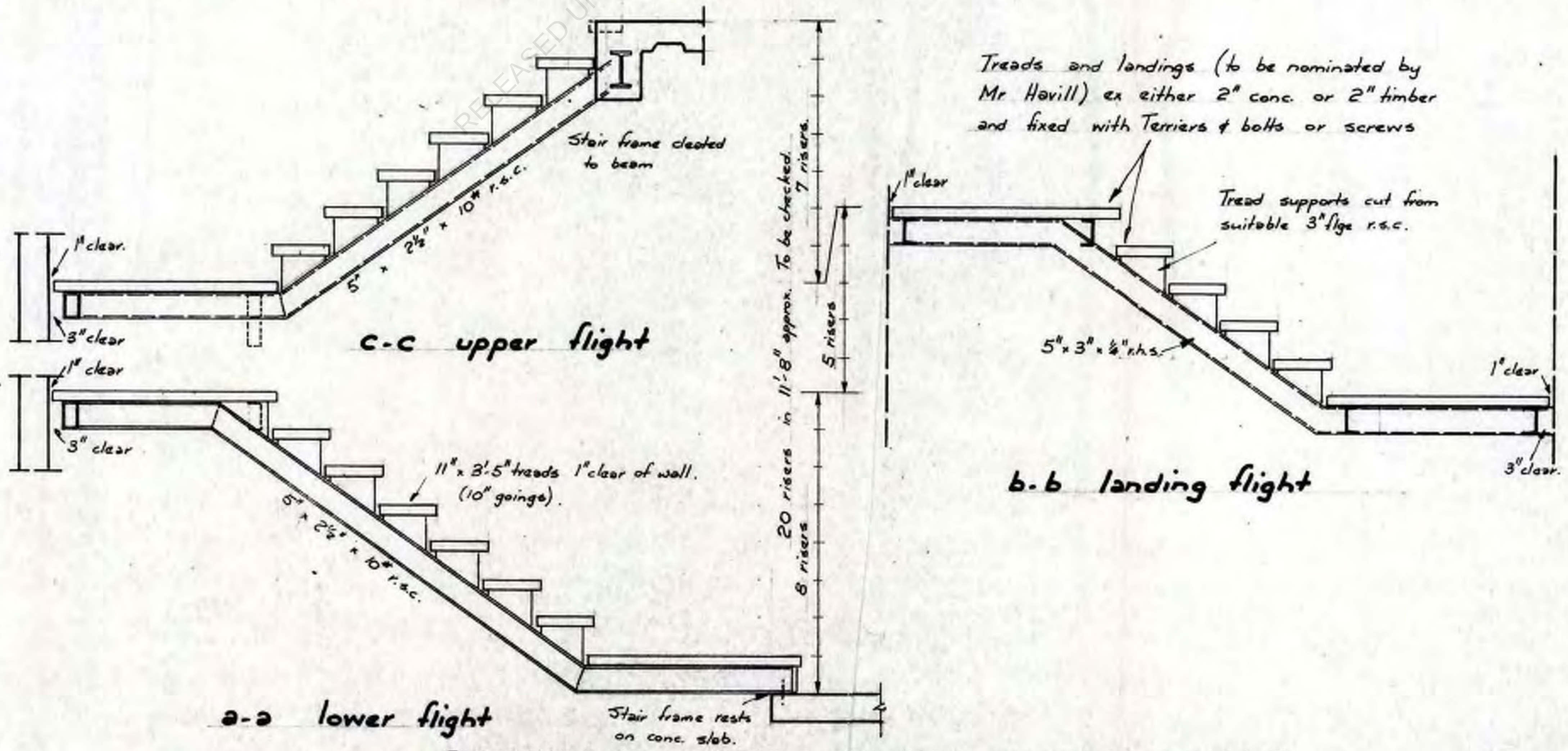
Elevation of South Wall.

Window schedule (nominal sizes.).

8'-0" x 4'-0"	2 x 2	= 4 off
6'-0" x 4'-0"	2 x 2 + 4 x 2	= 12 off
2'-0" x 4'-0"	2 x 2	= 4 off
2'-0" x 3'-4"		2 off.
Measure (8'-0" shown) x 4'-0" stair		1 off



Plan of main stair framing.



PROPOSED MAIN STAIR CONSTRUCTION.

A. Corrected for ext. egress. Stair details added. Cutler July 74.

PROPOSED ADDITION TO FAYMAR FLATS at 16 GLENMORE STREET, NORTHLAND for Mr. F. HAVILL.	ELEVATIONS OF SIDE BLOCK WALLS, TANKROOM AND STAIRCASE	DeTERTE & KERR-HISLOP CONSULTING CIVIL AND STRUCTURAL ENGINEERS 2nd FLOOR "WOODCRAFT HOUSE" 75 WILLIS STREET, WELLINGTON, N.Z. PHONE 40-835

SCALE ⅛"; ¼"; 1" to 1'-0"		
PASSED	Dwn.	Cutler
	Tcd.	
DATE Mar 74	Chkd.	

D.K. 1163/6A

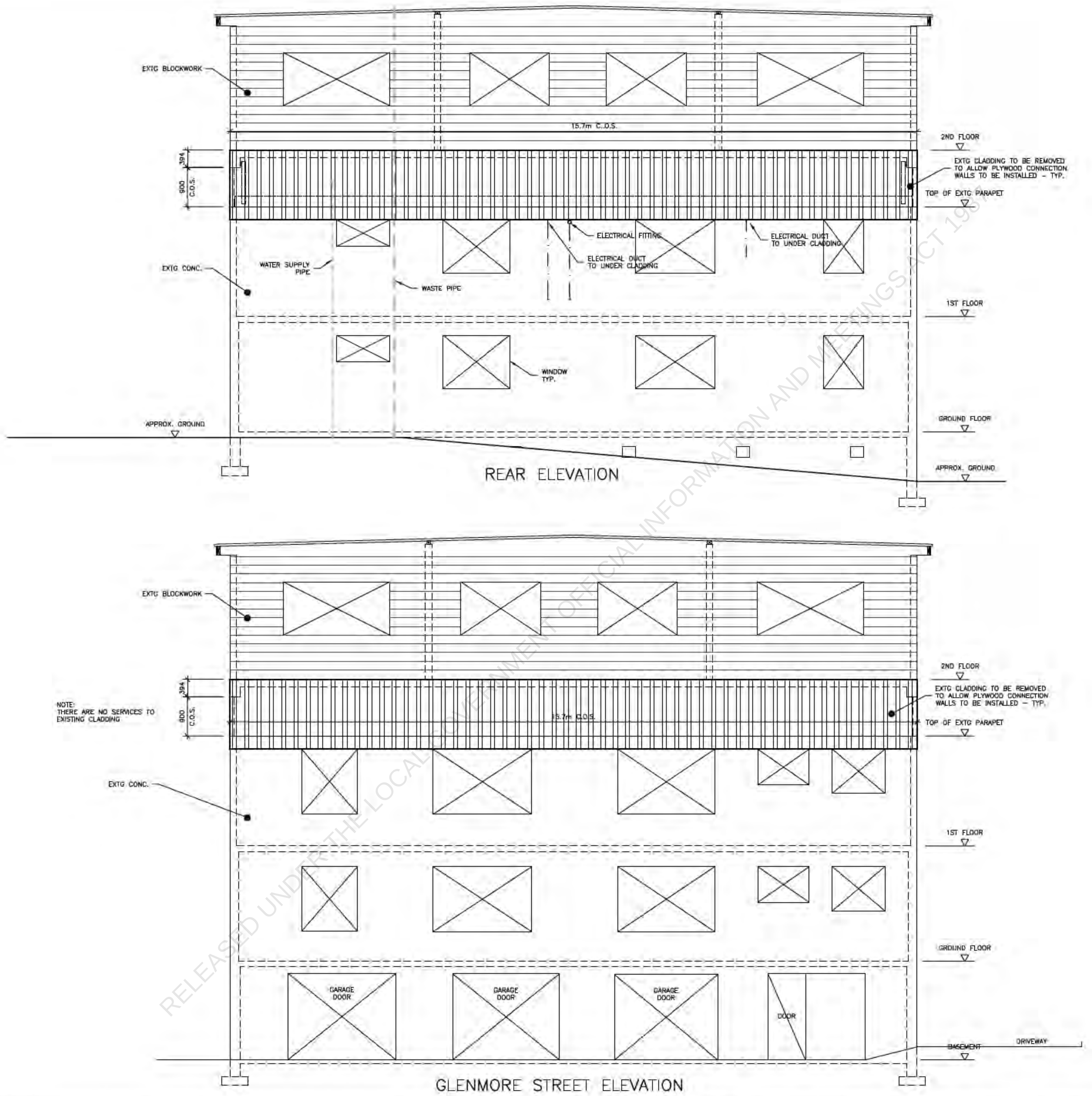
PROPOSED ADDITION TO FAYMAR FLATS
at 16 GLENMORE STREET, NORTHLAND
for Mr. F. HAVILL.

FLOOR SETTING UP and
ROOF AND GENERAL
DETAILS.

DeTERTE & KERR-HISLOP
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
2nd FLOOR "WOODCRAFT HOUSE"
75 WILLIS STREET,
WELLINGTON, N.Z.
PHONE 40-835

SCALE ⅛", ½" to 1'·0"			
PASSED		Dwn.	Cutler
		Tcd.	
DATE Mar '74		Chkd.	

D.K. 1163/7



Original Size A1

Original Scale

NOTES:

1. REFER TO ARCHITECT FOR DETAILS ON MAINTAINING EXISTING SERVICES THROUGH PERIMETER WALLS.
2. REFER ARCHITECTS DRAWINGS FOR ALL WORKS TO REINSTATE EXTERNAL METAL CLADDING

Absolutely Positively Wellington
Me Heke Ki Pōneke
Wellington City Council

16 GLENMORE STREET SEISMIC IMPROVEMENT

ELEVATIONS 1

REVISIONS		
0	FOR CONSENT	5/08/2013

Scales	1:50
A3 Scales	1:100
Designed	s7(2)(a)
Drawn	s7(2)(a)
CAD Reference	6042s01

Job Number 6042
Drawing Number S02
Rev 0

ISO 9001

Dunning Thornton consultants

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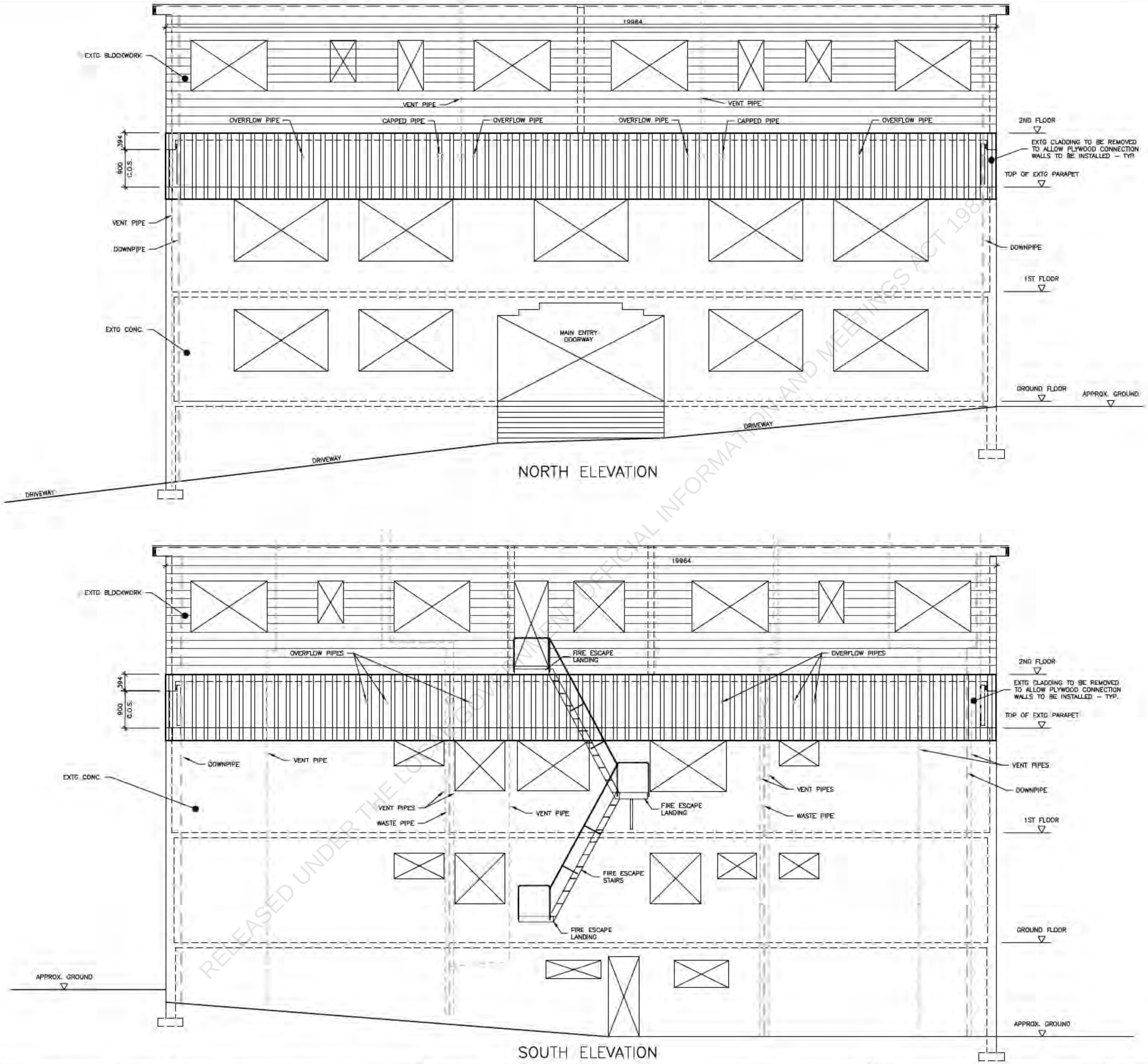
IF IN DOUBT ASK

DO NOT SCALE

Original Size A1



Original Scale
100
50
10 0

Original Size A1

NOTES:

1. REFER TO ARCHITECT FOR DETAILS ON MAINTAINING EXISTING SERVICES THROUGH PERIMETER WALLS.
2. REFER ARCHITECTS DRAWINGS FOR ALL WORKS TO REINSTATE EXTERNAL METAL CLADDING

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Wellington City Council

16 GLENMORE STREET SEISMIC IMPROVEMENT

ELEVATIONS 2

REVISIONS		
0	FOR CONSENT	5/06/2013

Scales	1:50
A3 Scales	1:100
Designed	s7(2)(a)
Drawn	s7(2)(a)
CAD Reference	6042s01

Job Number
6042
Drawing Number
S03
Rev 0

ISO 9001

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IF IN DOUBT ASK

DO NOT SCALE

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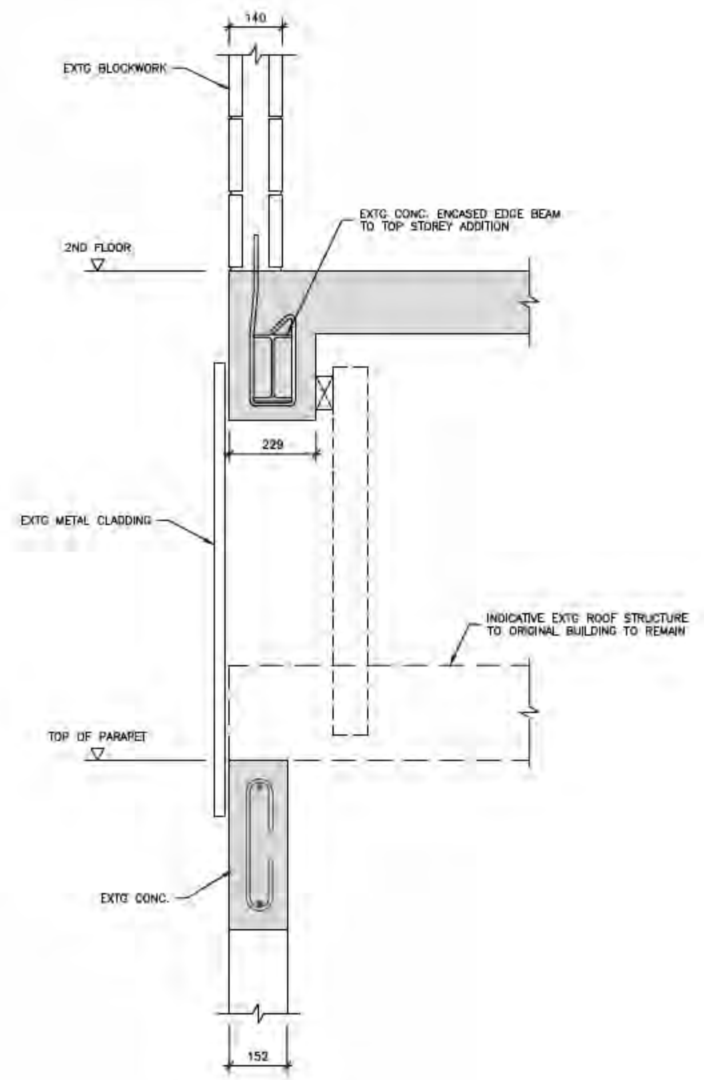
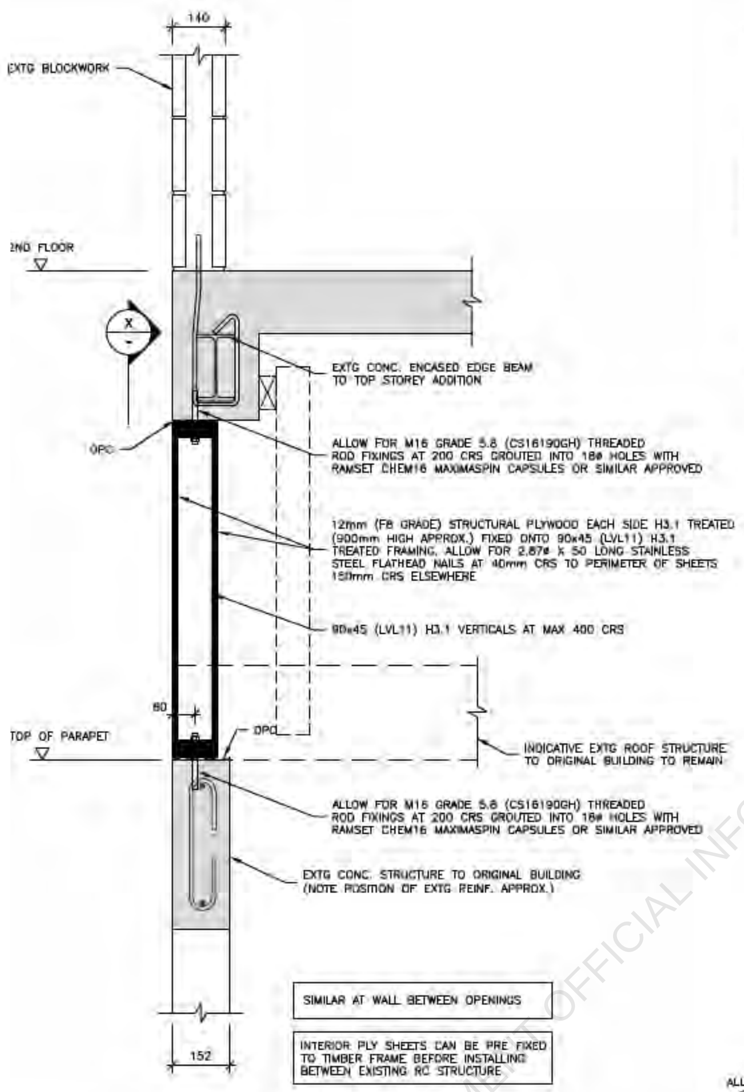
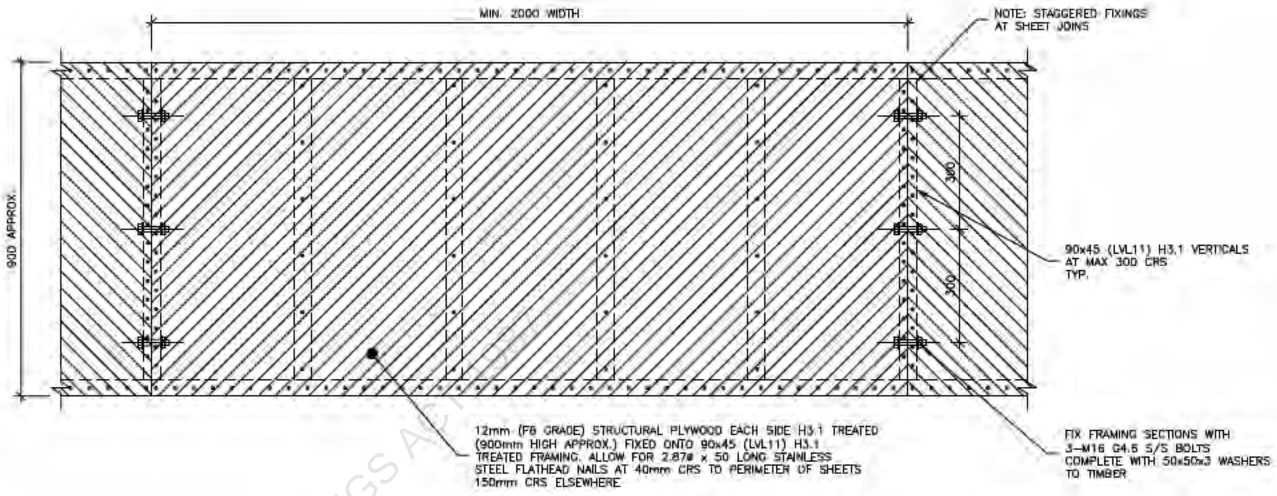


EXISTING CONDITION TO BUILDING PERIMETER
SCALE 1:10

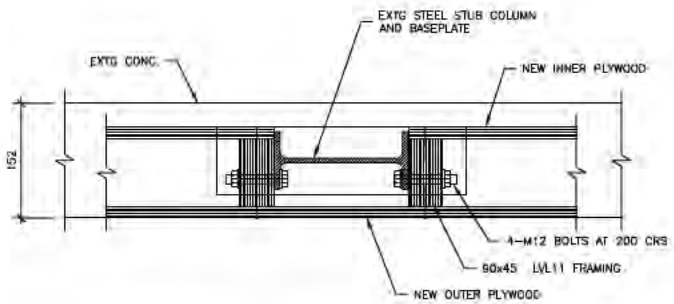


PROPOSED DETAIL TO BUILDING PERIMETER
SCALE 1:10

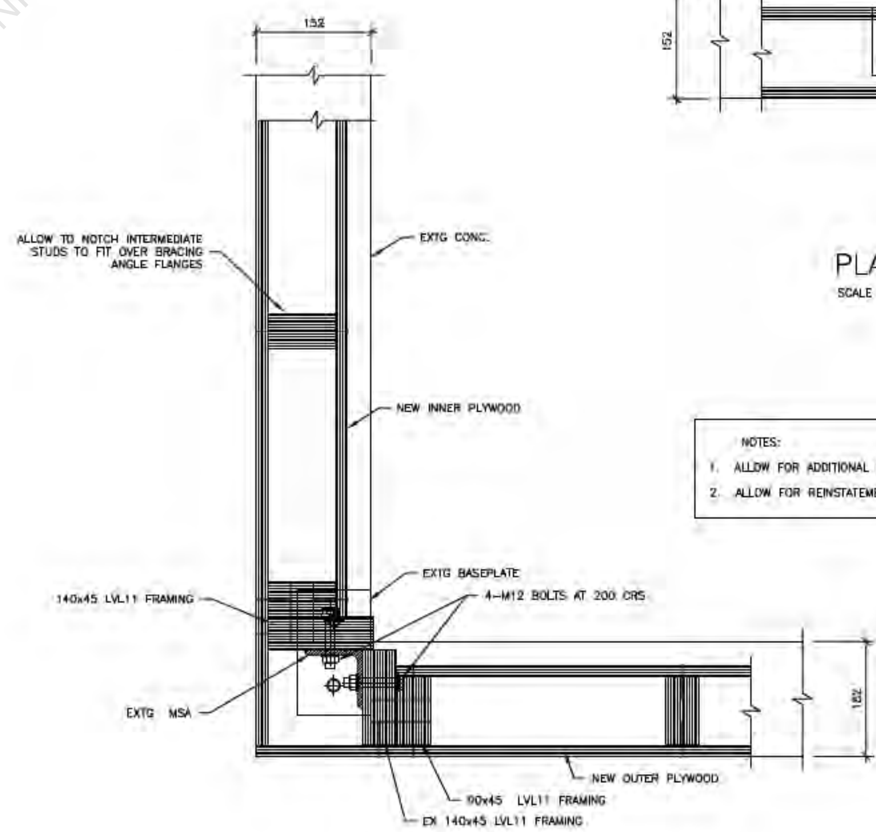


X ELEVATION OF NEW PLYWOOD LINING
SCALE 1:10

NOTE:
TYPICAL ALLOW TWO PREFABRICATED FRAMING SECTIONS APPROX. 2m LONG BETWEEN EXISTING COLUMNS



PLAN DETAIL – MID COLUMNS
SCALE 1:5



PLAN DETAIL – CORNER COLUMNS
SCALE 1:5

NOTES:
1. ALLOW FOR ADDITIONAL FRAMING, FIXINGS, SEALS ETC TO ARCHITECTURAL DETAILS
2. ALLOW FOR REINSTATEMENT OF METAL CLADDING OR SIMILAR TO ARCHITECTS DETAILS

Original Size A1
Original Scale
Original Size A1

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16 GLENMORE STREET SEISMIC IMPROVEMENT

DETAILS

REVISIONS		
0	FOR CONSENT	5/06/2013

Scales	1:5, 1:10
A3 Scales	1:10, 1:20
Designed	s7(2)(a)
Drawn	s7(2)(a)
CAD Reference	6042s01

Job Number 6042
Drawing Number S04
Rev 0

Dunning Thornton consultants

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