DETAILED ENGINEERING EVALUATION

WAIPA DISTRICT COUNCIL TE AWATMUTU OFFICE

(PROJECT)

WAIPA DISTRICT COUNCIL

(CLIENT)

Approved for Issue – Rev 5
February 2014

Detailed Engineering Evaluation

Project Name: **Waipa District Council Te Awamutu Office**
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1 Executive Summary

BCD Group Ltd has continued on from the Initial Evaluation Procedure (IEP) and undertaken a Detailed Engineering Evaluation (DEE) against NZS 1170.5:2004 for the Earthquake Strength of an existing building located at 101 Bank Street, Te Awamutu.

No non-destructive or destructive testing has been carried out on site as part of this Assessment; however original plans from 1973 and for the alterations carried out in 1997 have been used as a basis to determine a structural earthquake strength against New Building Standards (NBS).

Based on a Modal Analysis the Building has been assessed as 35% NBS for an Importance Level 4, Civil Defence facility, structure and 60% for an Importance Level 2, non-Civil Defence facility, structure.

2 Introduction

BCD Group Ltd has undertaken a Detailed Engineering Assessment of the building at the above address following a request by Waipa District Council. This assessment follows the Initial Assessment Procedure completed by BCD Group Ltd in September/October 2013.

Figure 2.1 Photo of the Southeast Elevation showing the original structure with the basement level in-filled.

The original building was designed during 1973 and consisted of a 3 storey building with attached council chambers. The current usage of the building is for Council Chambers and is the local Civil Defence headquarters, therefore making it an Importance Level 4 (IL4) structure.

2.1 Original – Main Building

The main building, originally designed and built circa 1973, is three levels high constructed using reinforced concrete. The basement level on grade was part basement and part open air parking that has since been enclosed to form office space. These in-fill walls, while made of block, have been installed in the 1990's. Any reinforcement within the walls is unknown and the capacity of the connections to the existing structure is unknown. Therefore, for the purposes of this DEE, we have only assumed minimal lateral restraint. The upper two levels are "open" plan with light weight internal partitions from floor to ceiling.

The main lateral structure is a two-directional shear core, centrally located and relatively symmetrical, with concrete floor plates which act as diaphragms attached on all sides. The gravity loads are resisted by a two way slab spanning between the central shear core and the beams and columns to the outside perimeter.

Detailing of the external concrete beam and columns mean that it will not provide any significant assistant to the lateral support.

The roof structure consists of tiles supported on timber purlins which span to steel RHS trusses connecting the top of the columns to the shear core.

It was noted on both the IEP inspection and subsequent visit, that the first floor has a significant sag in the southern most section. This has been estimated to be approximately 30mm using a string line.

2.2 Original – Council Chambers

The original Council Chambers consists of a two level building that was located outside of the main building but attached at first floor to first floor by a concrete walkway. This walkway has since been removed during the 1997 alterations.

The lateral structure was different to the main building as there is no shear core and uses frame action to restrain the roof structure. The first floor is restrained by shear walls that act as the external walls to the ground level. The first floor slab acts as a diaphragm and supports the gravity loads via two way action with a single internal column. Four external columns with beams provide the lateral restraint of the roof which is tiles on timber framing over a two way grillage of reinforced concrete beams.

2.3 Early 1990 Alterations

In-fill of the ground level with a new exterior wall formed of masonry blocks and glazing. The existing foundation was a basic turn down with two edged bars. Lateral resistance of new walls would be negligible on existing foundations. No record of works was encountered by BCD Group Ltd during this assessment.

2.4 1997 Alterations

Alterations to the existing buildings were undertaken to increase for the foot print area. The new area of building has introduced a new shear core adjacent the ex-Council Chambers. The Architecture was completed by Chow Hill Architects Limited and the Structural Engineer by Jones Gray Partnership. The new building is only 2 levels to match the ex-Council Chambers with masonry block shear walls to part of the ground level and column/beam lateral restraint to the roof level. The floor is of precast concrete flooring with an in-situ topping. The gravity loads are supported by concrete beams and masonry block columns/pilasters. During these alterations no major structural work was done to the main part of the original building. The new alterations were fixed to the original structure through the use of epoxied bars, bolts and a new access doorway was introduced in the 8” wall along the north-western face.

Figure 2.2 Northwest elevation showing original Council Chambers on the right and 1997 Alterations on the left.

3 NZSEE Initial Evaluation Procedure

The Initial Evaluation procedure calculations, for this building, were completed by BCD Group Ltd and are attached in Appendix B – Initial Evaluation Procedure

The original IEP conducted by BCD Group Ltd rated the building as 28% NBS in the longitudinal and 26% in the transverse orientation, resulting in Seismic Grade D.

This grade is based solely on a statistical analysis which critically takes into consideration structural age, construction materials, building geometries and geographical location.

4 Investigation

The main part of the investigation has been based on Historical records. These include the original 1973 plans (Structural and Architectural) and plans for the 1997 Alterations (Structural and Architectural). Part of the 1997 Specification has the Geotechnical report complete by Geocon Soil Testing Limited. The top 1m of soil profile is not shown and the underlying soil is shown to be cohesive soils.

A walkthrough was conducted, however for the most part little of the internal structure was able to be viewed due to non-structural linings. Of the structure that could be viewed there was little sign of poor quality and the roof structure appeared to be sound with no noticeable discolouration to the timber.

The floor in the southern part of the main structure has noticeable sagging at mid-span on both of the suspended slabs. An approximate measure carried out during the IEP stage assessed the mid-span to have sagged approximately 30mm. The main shear core to this part of the building appears unaltered.

The ex-Council Chambers have been altered into meeting rooms and the exterior walls have been significantly altered along two faces to accommodate the new layout. The new flooring has been connected to the existing structure using epoxied bars tied to the mesh.

5 Detailed Engineering Assessment

The Detailed Engineering Assessment calculations have been attached in Appendix D – Calculations.

5.1 Methodology

The building has been assessed as a whole 3D structure using SAP2000 using AS/NZS 1170 for the gravity and seismic loadings. A modal analysis using eigenvectors has been used.

As the plans for both major build phases were available, the dimensions used were based on the drawings rather than site measurements. As no destructive testing was undertaken, values for the material properties have been based on those specified in the Specification or assumed. Refer to Section 5.3 and Section 5.4 for further details.

5.2 Software

The following computer applications were used for the design:

Analysis Type	Software Used
General Design	BCD Group Ltd Design Spreadsheets
3D Model	SAP2000 Version 16

5.3 Known Material Properties

Based on the 1973 Original drawings

- Reinforcement HY60 specified for some of the bars. Based on a 1973 paper from University of Canterbury a yield strength of 58,000psi (≈400MPa) has been adopted

Based on the 1997 Alteration Specification

Reinforcement	HD & HR	430MPa
	D & R	300MPa
	Mesh	480MPa
Concrete		30MPa
Masonry Block		12MPa (17.5MPa for Grout)
Steel	Plate	300MPa
	Rolled	300MPa
	Hollow	250MPa

5.4 Design Assumptions

The visible structural concrete and steel appears to be in good condition. It has been assumed therefore that all structural elements have full sectional capacity and have not been affected by deterioration due to exposure to the elements.

1973 Original Building

Reinforcement	Mesh	55,000psi (≈380MPa)
	Bar	36,000psi (≈250MPa) for unspecified steel
Concrete	3000psi (≈20MPa) for all concrete	

5.5 Structural Form

The structural form of this building is a reinforced concrete building with a shear core and beam/column gravity frames to the perimeter. The 1997 Alterations introduced concrete block masonry for the shear walls in lieu of the in-situ concrete. The flooring was also changed from solid cast in-situ concrete to precast flat slabs with in-situ topping.

The roof is of concrete tiles over timber sarking for the original 1973 building and tin tiles over timber purlins for the 1997 Alterations. The beams supporting the roof over the new Council Chambers are steel rather than cast in-situ concrete beams.

5.6 Design Loads

Loads applied to the building have been determined using NZS 1170 parts 0: General, 1: Permanent, imposed and other actions and 5: Earthquake actions – New Zealand. The loadings due to parts 2: Wind actions and 3: Snow and ice actions have not been considered.

The building has been checked as an Importance Level 4 building, this means that it has been designated as a post-disaster emergency centre.

We have also reported on the NBS strength should you choose to remove the post disaster status and hence reduce the Importance level from 4 down to 2.

5.6.1 *Gravity loads*

Level/ Area	Use	Live Load	Superimposed Dead Loads
Floor Slab	General Office	3.0kPa	0.5kPa
Roof	Non-Access	0.25kPa	

Table 5.1 Imposed Gravity Loads

5.6.2 *Seismic Loads*

Seismic loads have been applied using SAP2000's in-built response spectrum and the following parameters

Soil D	Site Subsoil Class
Z = 0.17	Hazard Factor, Te Awamutu
R = 1.8	Return Period Factor; Importance Level 4 structure
N = 1	Near Fault Factor; no nearby faults
5%	Function Damping Ratio
μ = 1.25	Structural Ductility factor
s_p = 0.9	Structural Performance Factor

Should the Building be downgraded to an Importance Level 2, as it may have originally been designed for, then a Return Period Factor of 1.0 can be used.

6 Analysis Results

Figure 6.1 shows the 3D model created during the DEE. The following results are discussed using Gridline references; refer to Appendix A – Reference Floor Plans for locations of the gridlines used.

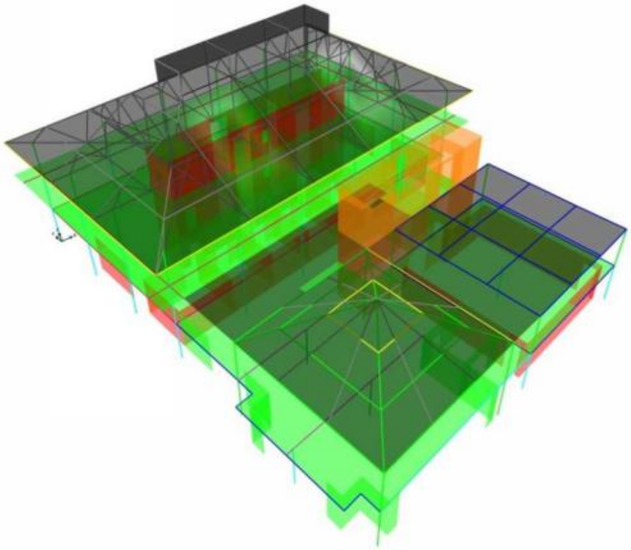


Figure 6.1 3D image of structural model from SAP2000

The floor levels refer to as follows; Level 1 – Basement, Level 2 – Ground Floor, Level 3 – 1st Floor.

6.1 Floor Slab

6.1.1 Gravity Loads

During the investigations it was noted that there was significant sagging of the part of the floor slab in one corner on both of the suspended floor levels. While this does not mean that the building is necessarily unsafe, the floor may have been loaded beyond the yield point of the steel and therefore may have sustained plastic deformation. Figure 6.2 and Figure 6.3 show the deflection contours for the short term loading of the suspended floors, level 2 and level 3 respectively. The maximum deflections are 4.5mm for both level 2 and 3 in the corner that has the sagging issue.

Figure 6.4, Figure 6.5, Figure 6.6 and Figure 6.7 show the short term Bending Demands for each direction and level as noted. The slab has sufficient capacity through the middle of the floor (approximately 72kN-m/m) and along the edges 37kN-m/m. While there are some concentration of demands about the corners of the shear core and the columns these do not exceed the capacity.

Figure 6.8, Figure 6.9, Figure 6.10 and Figure 6.11 show the ultimate Bending Demands for each direction and level as noted. The demand on the edges of the slab has increased to be more than the design capacity of the slab and therefore plastic deformations will have occurred locally. Given that it is unknown what the loading conditions have been, it is not possible to state when this has occurred. While the plastic deformation is not of concern for deflections, the fact that yielding may have occurred in the steel is. Issues of HY60 reinforcement not having ductile behaviour means that there may be failure of the steel reinforcement under earthquake loading due to shearing effects from the diaphragm forces. Therefore we have paid particular attention to the diaphragm stresses in this region.

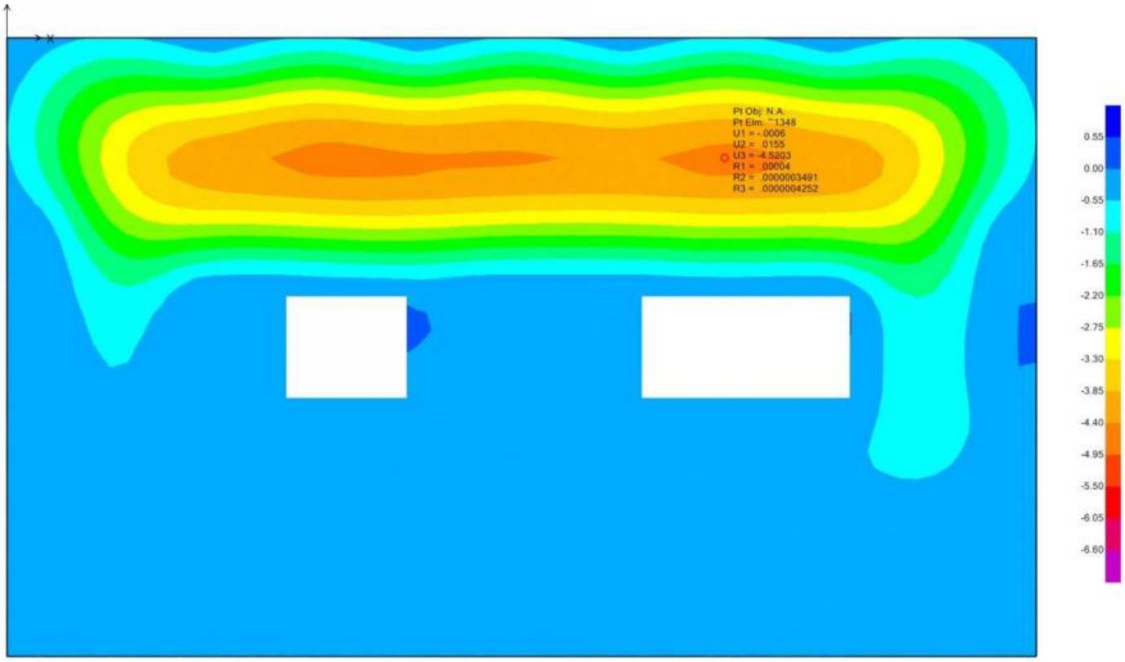


Figure 6.2 Plot of Deflections for Floor Slab at Level 2 (G + 0.7Q, UZ, units: N, mm)

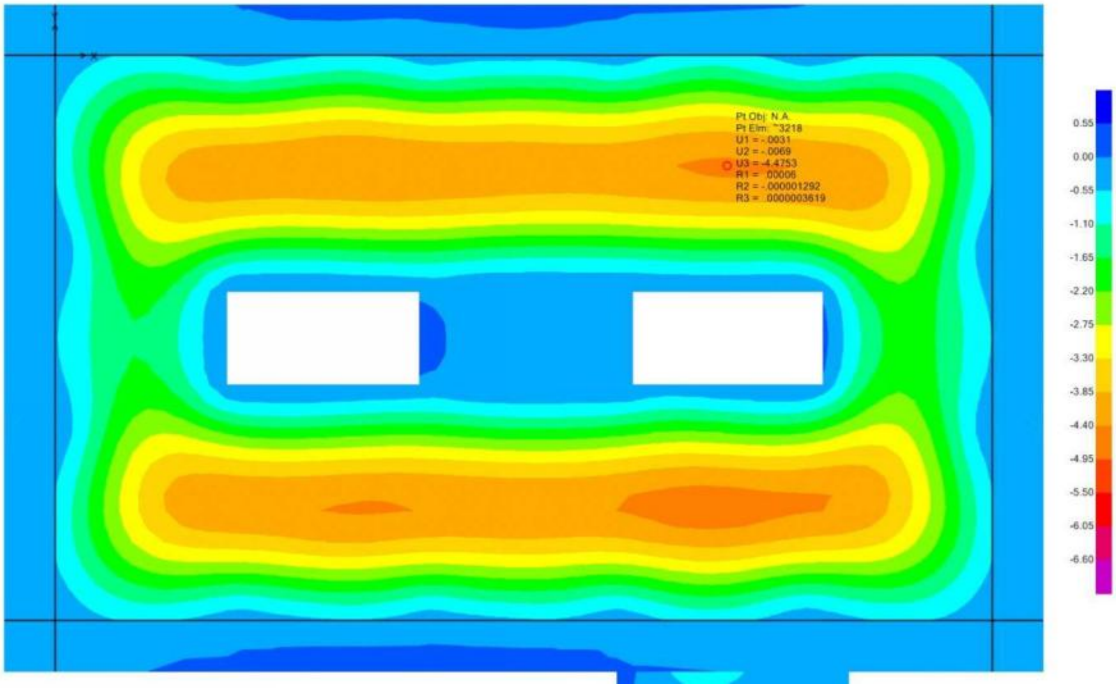


Figure 6.3 Plot of Deflections for Floor Slab at Level 3 (G + 0.7Q, UZ, units: N, mm)

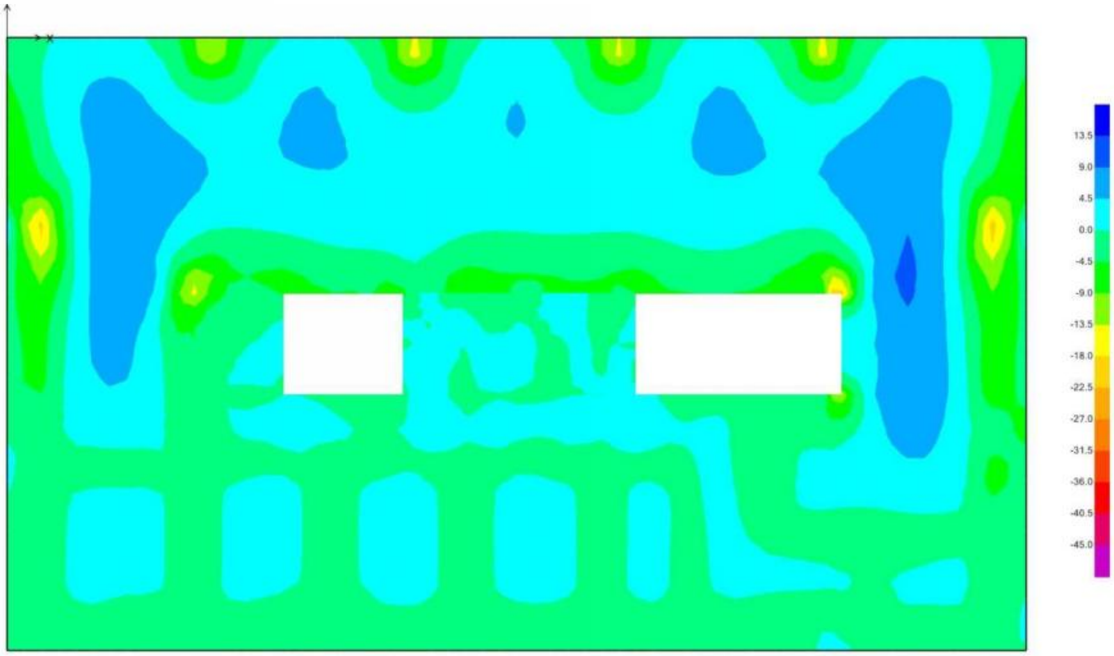


Figure 6.4 Plot of Bending Demand for Floor Slab at Level 2 (G + 0.7Q, M11, units: KN, m)

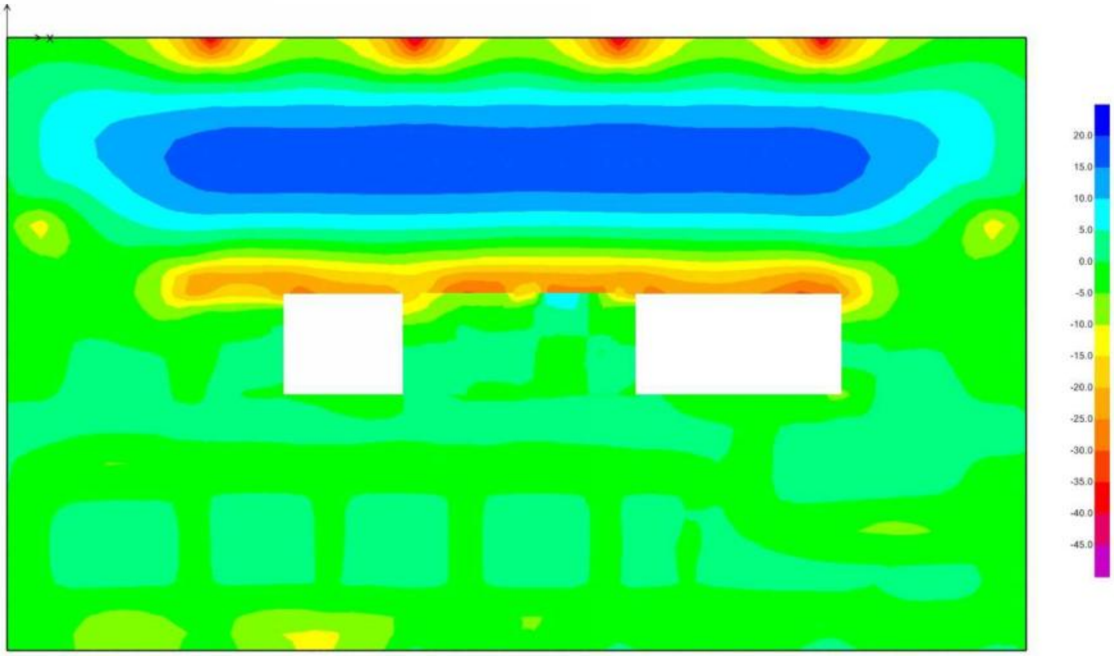


Figure 6.5 Plot of Bending Demand for Floor Slab at Level 2 (G + 0.7Q, M22, units: KN, m)

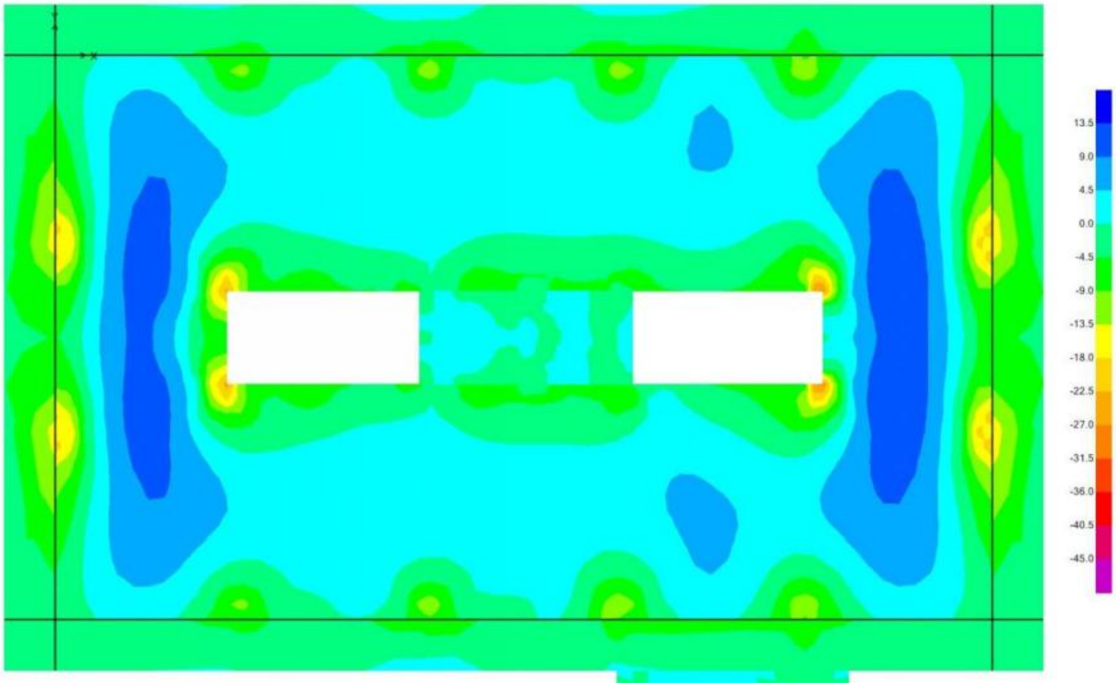


Figure 6.6 Plot of Bending Demand for Floor Slab at Level 3 (G + 0.7Q, M11, units: KN, m)

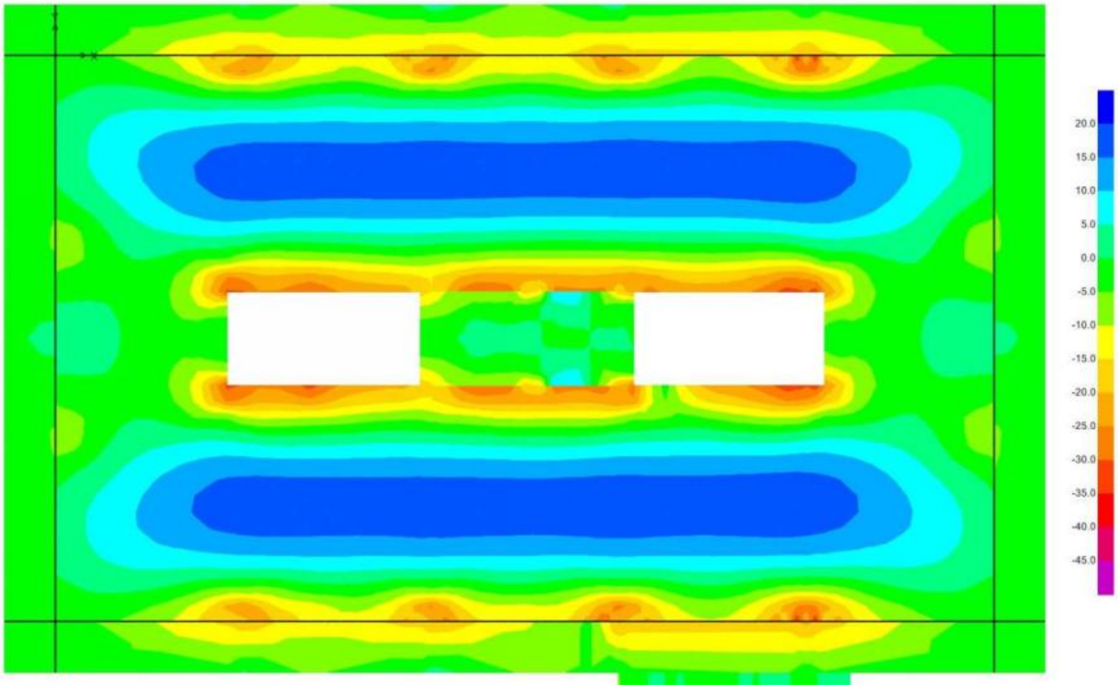


Figure 6.7 Plot of Bending Demand for Floor Slab at Level 3 (G + 0.7Q, M22, units: KN, m)

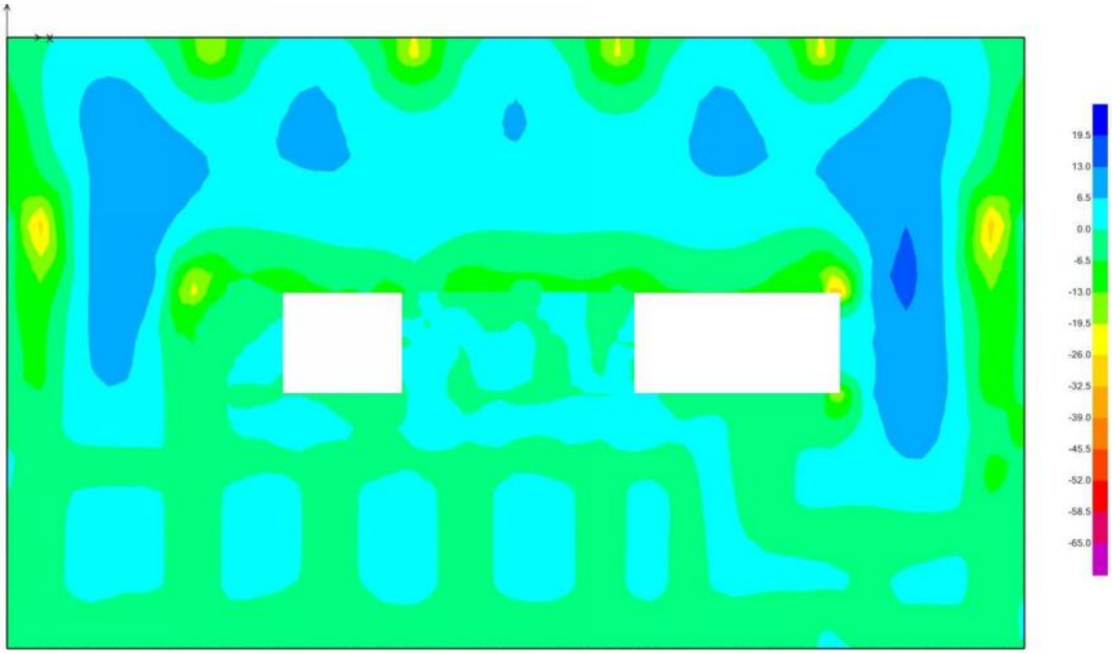


Figure 6.8 Plot of Bending Demand for Floor Slab at Level 2 (1.2G + 1.5Q, M11, units: KN, m)

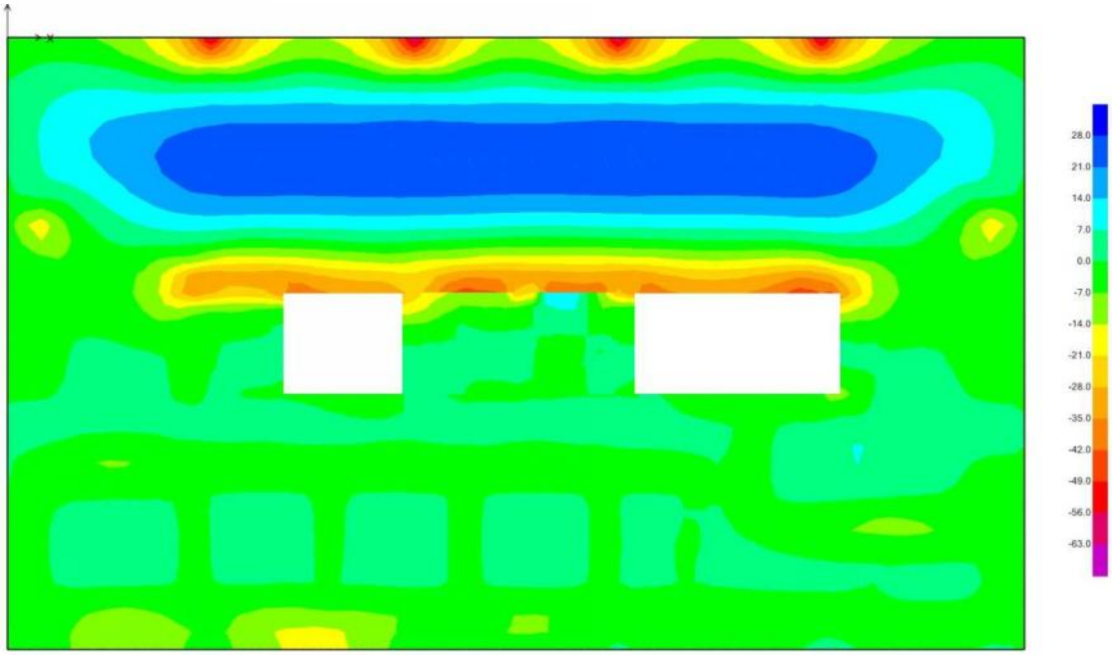


Figure 6.9 Plot of Bending Demand for Floor Slab at Level 2 (1.2G + 1.5Q, M22, units: KN, m)

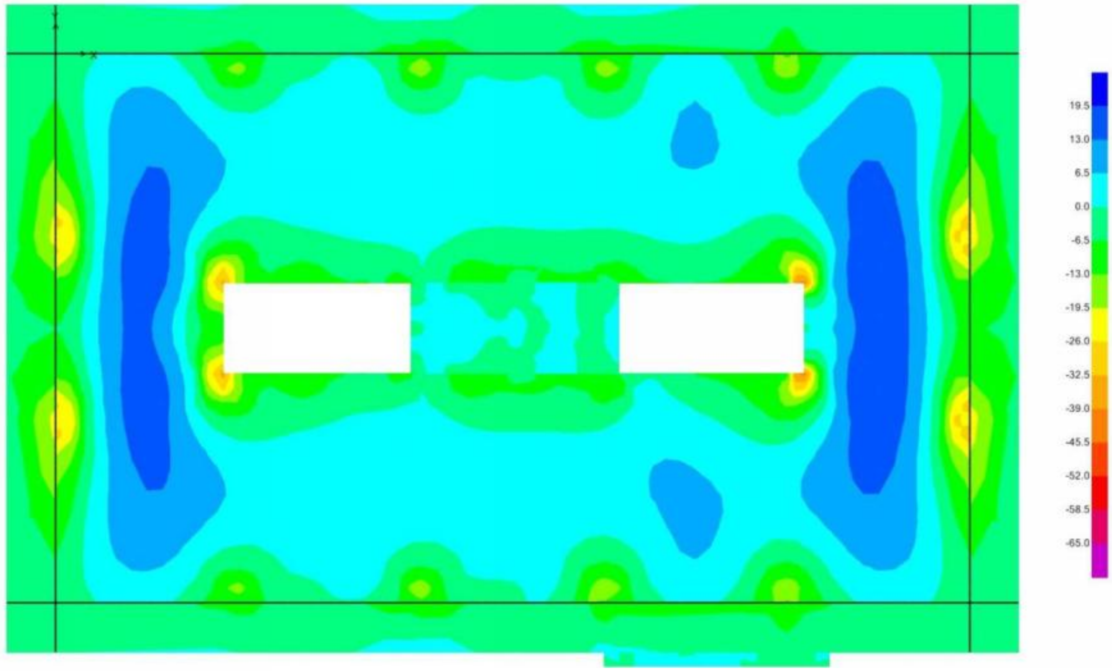


Figure 6.10 Plot of Bending Demand for Floor Slab at Level 3 (1.2G + 1.5Q, M11, units: KN, m)

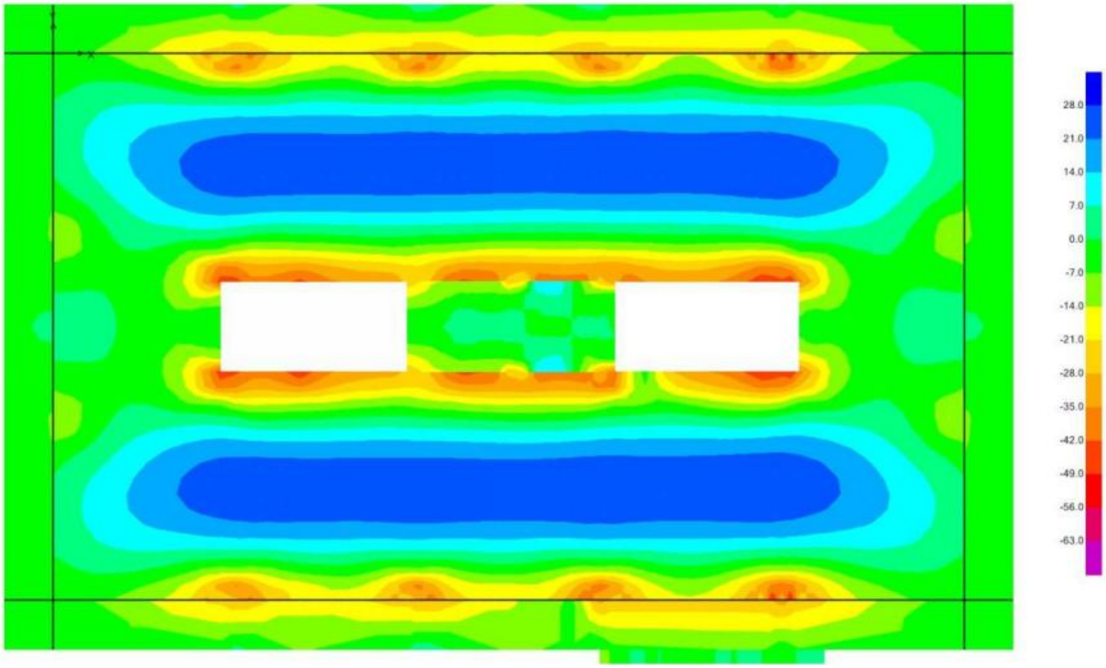


Figure 6.11 Plot of Bending Demand for Floor Slab at Level 3 (1.2G + 1.5Q, M22, units: KN, m)

6.1.2 Lateral Loads

The lateral forces from the seismic loads are transferred via the floor slabs to the shear walls. Due to the regular shape of the main building, the shear stresses through the diaphragm for level 3, Figure 6.12, is relatively symmetric with the maximum shear being approximately 0.5MPa at the Shear Core – diaphragm interface. The capacity of the interface is 0.56MPa from the steel alone. The capacity of the slab interface is also cast in-situ and further strength due to concrete shear may be considered.

The capacity of the 1997 flat slab for diaphragm actions is sufficient for elastic forces; however concern should be noted due to the lack of ductility within the diaphragm. The connection between the two stages is also of concern due to the nature of the connection, D12 bars have been epoxied into the existing suspended floors at 900mm centres. The expected shear capacity is 0.29MPa which is significantly less than the demand of >0.5MPa.

The level 2 diaphragm (ground floor), Figure 6.13, has a better distribution of forces between the existing and new suspended slabs. The bars have ductility and so, some yielding can occur allowing for the forces to spread evenly. The diaphragm stresses along the joints tend to be fairly uniform with some stress concentrations due to the floor slab connection with the walls below.

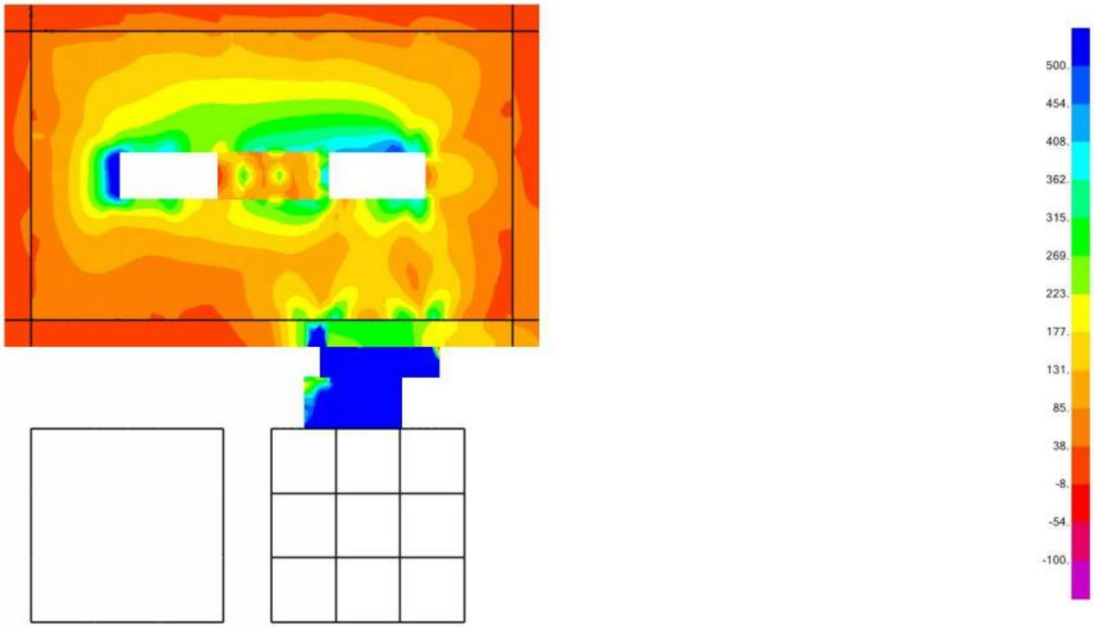


Figure 6.12 Plot of Shear stresses (kPa) in the Level 3 slab due to seismic excitation

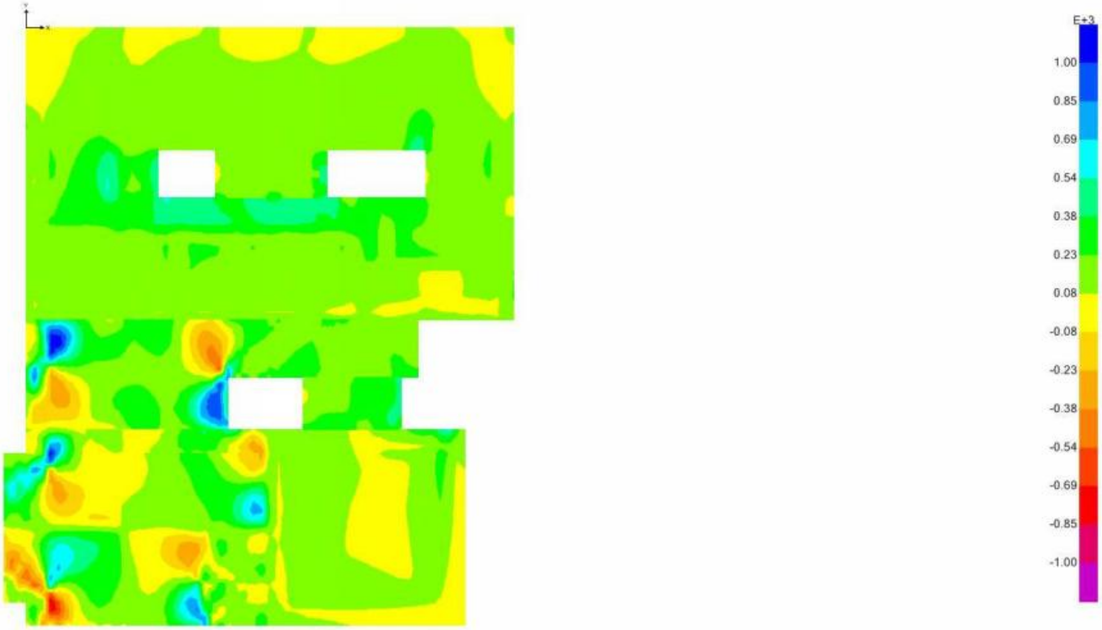
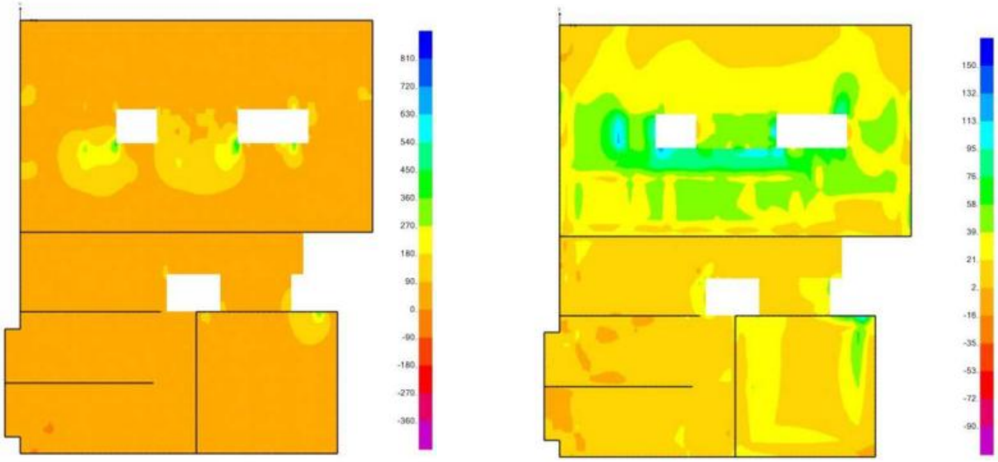


Figure 6.13 Plot of shear stresses (kPa) in the Level 2 slab due to seismic excitation



Y direction resultant (F22)

Shear resultant (F12)

Figure 6.14 Plot of force resultants (kN) in the Level 2 slab due to seismic excitation

The force is relatively constant throughout the slab. Based on using epoxied bars (using Ramset™ Chemset™ Injection 101 Plus) the capacity/demand is 17% due to pullout failure of the dowels. The shear capacity/demand of the connection is 45% at the maximum. It should be noted that the tie bars are grade 300 and therefore have ductility and are able to spread the load and basement shear walls on either side of the joint will continue to provide lateral support should the diaphragms start to separate.

6.2 Shear Core – Original 1973 Building

The shear core of the original structure has not been changed or altered since their construction.

The shear core for the model X direction are relatively long compared to those of the model Y direction, the walls are also thicker with 8” thick compared to 6” thick. Figure 6.15 and Figure 6.16 shows the shear forces through the building. The walls have a capacity of 90% NBS for Shear Wall line A and 100% NBS for Shearwall Line C, the maximum stress concentration in the lintel is due to the model formation rather than the building.

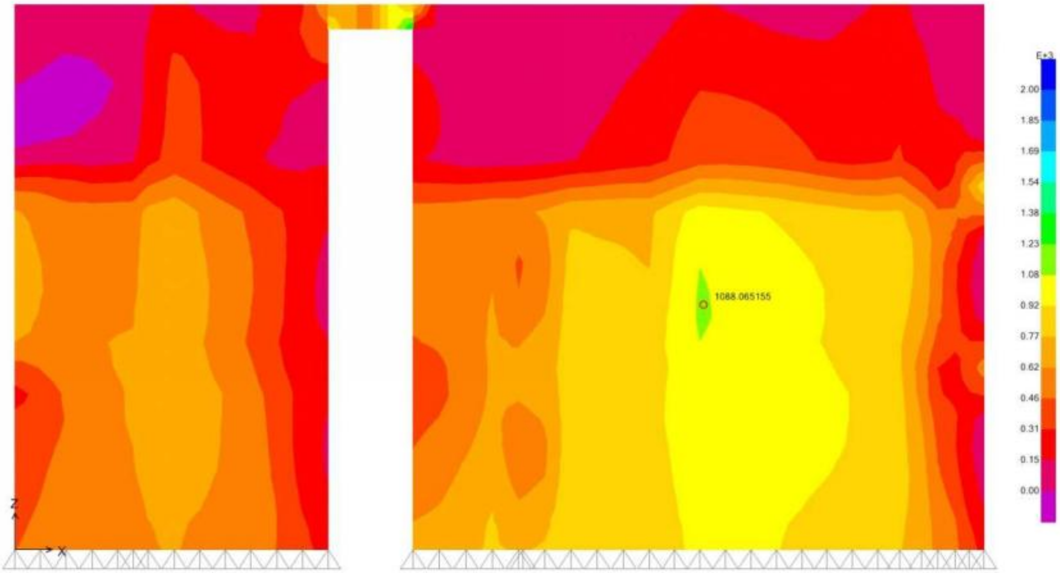


Figure 6.15 Plot of shear stresses (kPa) Shear Core 1 GL A

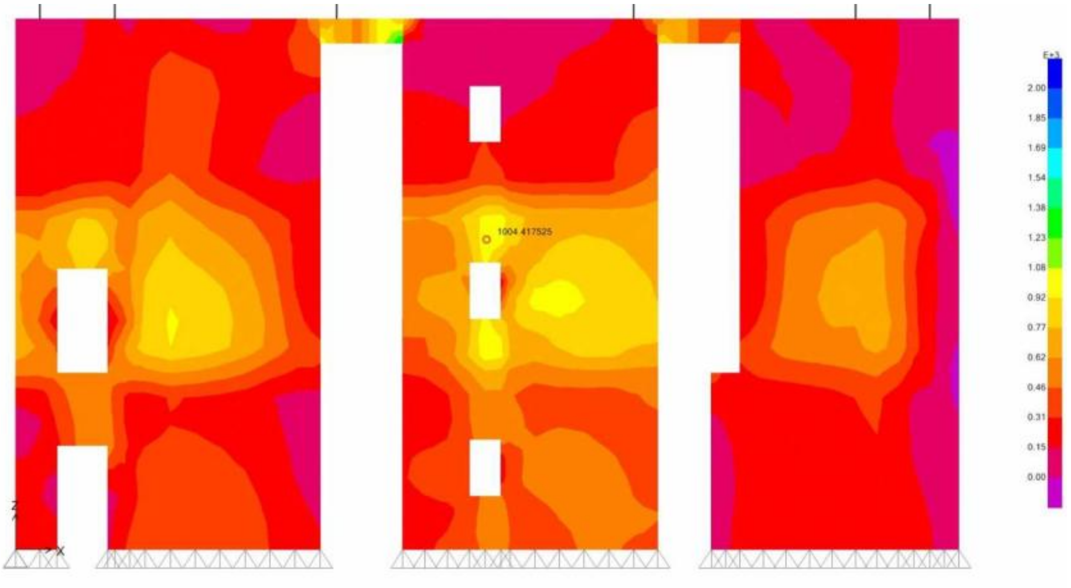
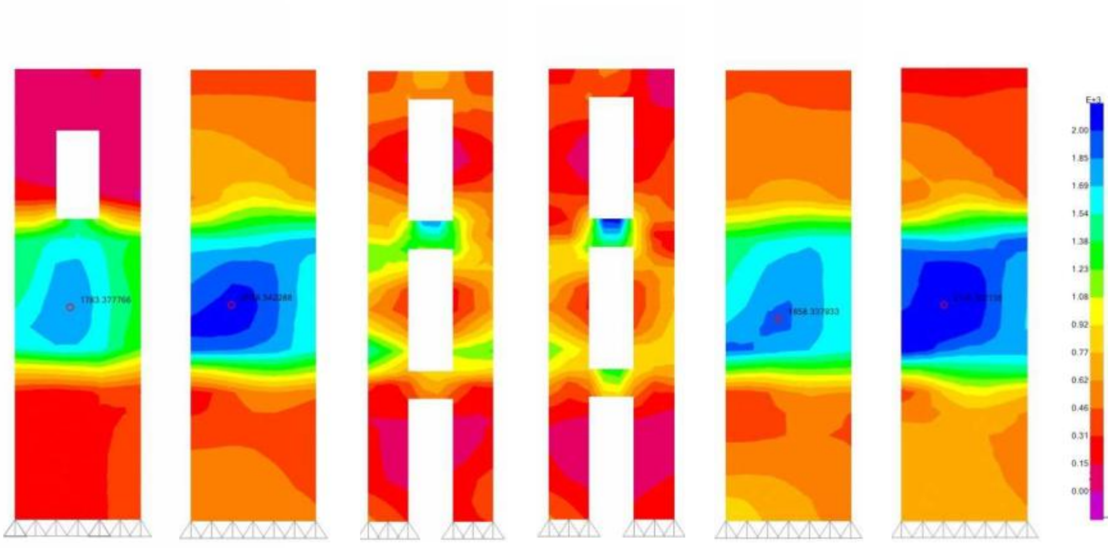


Figure 6.16 Plot of shear stresses (kPa) Shear Core 1 GL C

The shear walls for the model Y-direction being of thinner thickness and only a single layer of reinforcement means that a capacity of 45% NBS is achieved. This is due to high stresses between levels 2 and 3. The ground floor has numerous walls orientated in the Y-direction that are tied into the floor slab that reduces the demand on the lower walls in the shear core. Figure 6.17 shows stresses for the walls on the shear lines as noted; due to the penetrations in walls along Gridline 4 and 4a, large concentrations of stress in the walls take place compared to the long and relatively solid walls in the X direction.



GL 1 *GL 2* *GL 4* *GL 4a* *GL 5* *GL 6*

Figure 6.17 Plot of shear stresses (kPa) Shear Core 1

6.3 Frames – Original 1973 Building

The frames were originally designed for gravity loads only. The detailing of the splice locations and connections is well documented in the plans. The assessment shows that the beams are dominated by gravity and that the demand from seismic action is low.

The columns between level 3 and the roof structure undergone the most seismic response. This is due the roof being relatively flexible compared to the concrete suspended floors below. The columns resist the lateral excitation of the roof that connects directly to each frame line. For the frames along Gridlines 1 and 6 the mid-level columns have high moments due to seismic excitation. The ratio of capacity/demand is approximately 70%.

6.4 Shear Core – 1997 Alterations

A new shear core was introduced in the 1997 Alterations to house the new lift and a third set of stairs. The materials used for the new core were block masonry rather than in-situ concrete, and use grade 430MPa reinforcing bars. The stairs have been cast hard against the block walls, but given the stiffness of the walls this has not been considered an issue.

The longer shear walls in the model X direction have large penetrations between levels 2 and 3, which lead to a concentration of stresses around the openings. However, the walls provide a capacity of greater than 100% NBS.

The shorter shear walls in the model Y direction display an issue due to connections to the floor diaphragms. Subsequently, we have calculated large stress concentrations at the floor level. This is evident in Figure 6.20 GL 3 where the shear stress exceeds 1.2MPa.

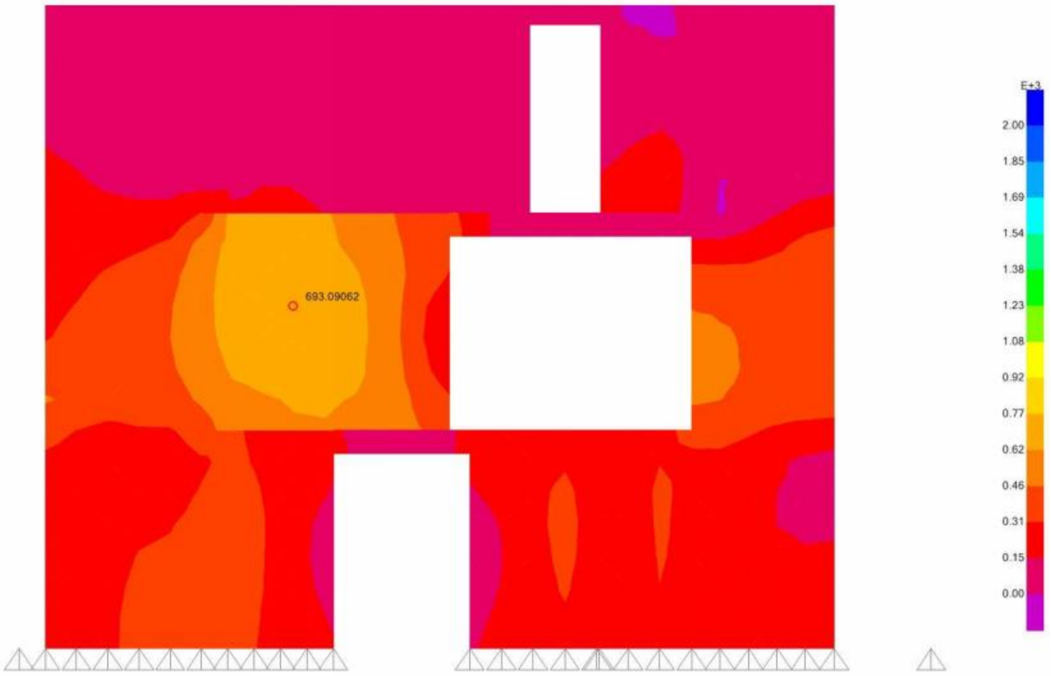


Figure 6.18 Plot of shear stresses (kPa) Shear Core 2 GL A

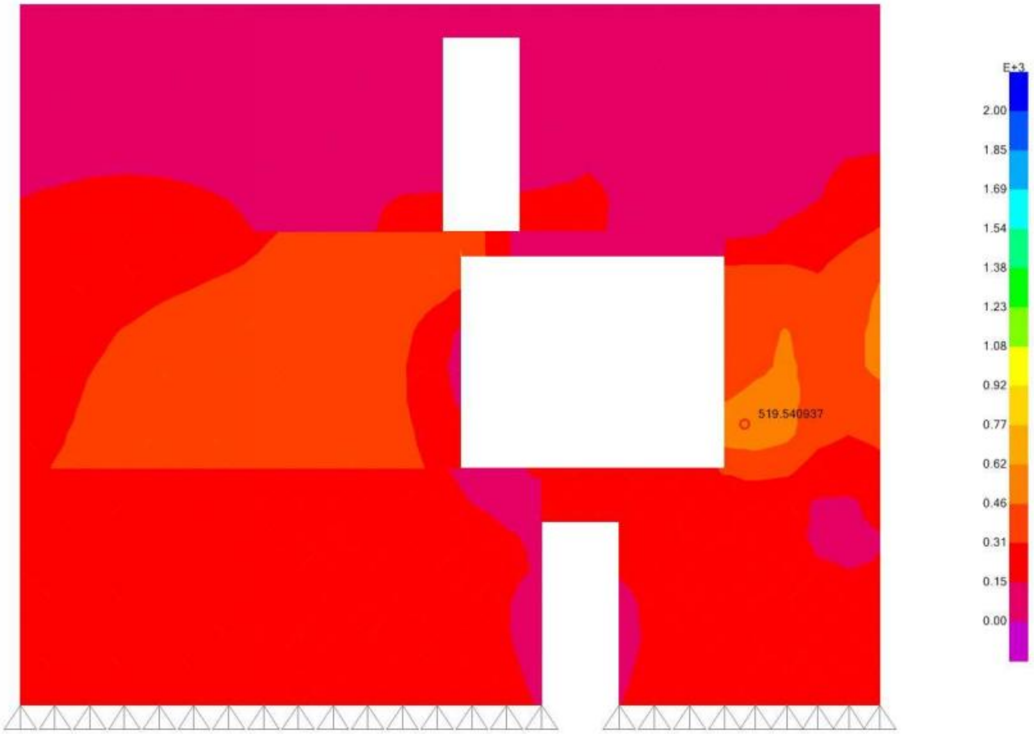
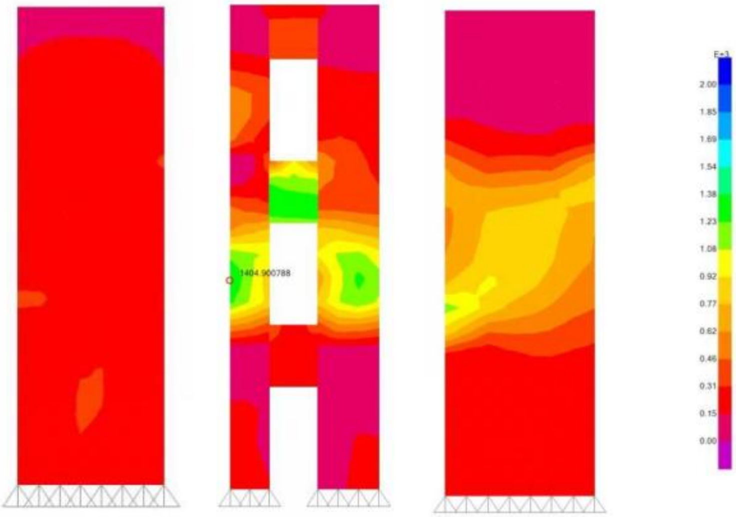


Figure 6.19 Plot of shear stresses (kPa) Shear Core 2 GL B



GL 1 GL 2 GL 3

Figure 6.20 Plot of shear stresses (kPa) Shear Core 2

The shear wall along Gridline 2, has large stresses due the openings for the lift doors.

Shear Core 2 has 85% NBS risk due to the stresses in the wall along GL 2.

6.5 Ground Floor Walls – 1997 Alterations

Figure 6.21 shows the shear stress of all the block masonry walls is low, less than 0.6MPa, there the walls are rated to greater than 100% NBS.

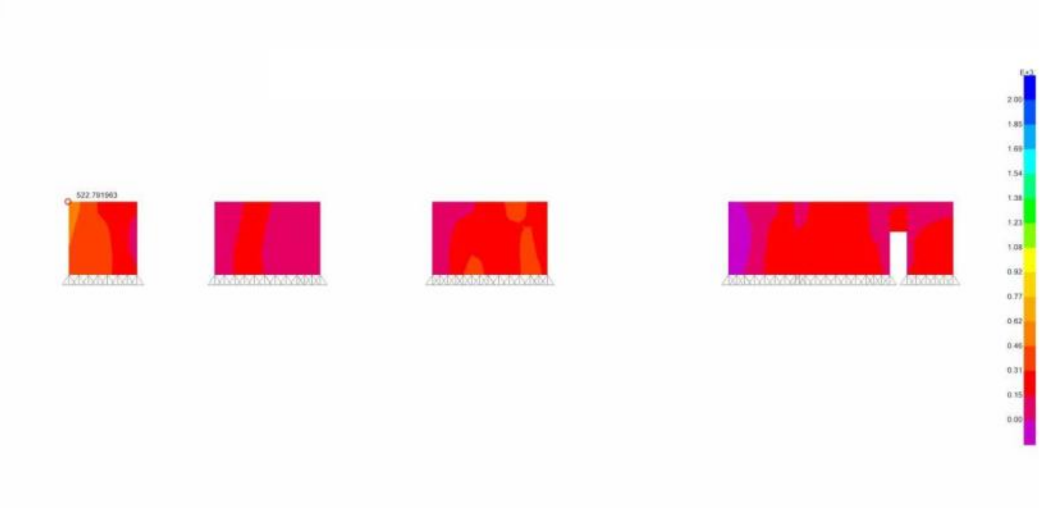


Figure 6.21 Plot of shear stresses (kPa) 1997 Ground Walls

6.6 Frames – 1997 Alterations

The 1997 alterations frames were designed to restrain the lower level roof during a seismic event. While the analysis shows that the columns have sufficient capacity, the detailing of the columns are for non-ductile performance and do not fully restrain the longitudinal bars. We estimate a rating of >100% NBS based on elastic design.

6.7 Uplift Forces

The building capacity is approximately 40% NBS.

7 Critical Structural Weaknesses

The following are structural weaknesses noted during the Assessment. These do not necessarily mean that this will cause failure of the building but may lead to decommissioning of the building post-earthquake.

7.1 Columns to ex-Council Chambers and new Council Chambers

Columns have been designed to transfer seismic loads to the first floor (level 2) slab via bending. The columns each have 8 bars; however drawings illustrate the newer columns only have 2 legs of stirrup at 200mm centres. The longitudinal bars do not have sufficient support based on current design standards. The ductile capacity of these columns needs to be considered as 1, or elastic.

7.2 Suspended Slab Deformation

As stated in Section 6.1.1 the steel in the floor slab may have yielded due to the large deformations of the slab to occur. While the floor still has some capacity, repeated high levels loading and unloading will cause further deformation and may even cause to failure of the slab. We recommend that this section have floor loads restricted to 3kPa or less, typical office space.

7.3 Suspended Slab to Shear Core Connections – 1973 Original Building

As stated in Section 6.1.1 the steel in the floor slab may have yielded due to the large deformations of the slab to occur. There have also been concerns raised over the ductile performance of the HY60 bars in cyclic loadings. This means that there is high degree of uncertainty to the level of repeat performance that can be expected of the connection under large seismic events.

Therefore we recommend that a detailed review of the diaphragm be completed by a Chartered Professional Engineer after any significant seismic event (greater than magnitude 5).

7.4 Use of Non-Ductile Mesh to Suspended Slab Diaphragm

The 1997 Alterations used 665 mesh as the steel reinforcement to the topping. This mesh is non-ductile and if yielding occurs along the wall faces then failure is likely to occur and the floor slab will separate from the walls.

Therefore we recommend that a detailed review of the diaphragm be completed by a Chartered Professional Engineer after any significant seismic event (greater than magnitude 5).

7.5 Connection 1997 Suspended Slab to Original Suspended Slab

During the 1997 Alterations additional suspended concrete slab was introduced to the building. This was connected to the existing via epoxied bars and slab edge has been scrabbled back where the new slab was poured up against. The depth of penetration into the existing slab is 100mm and during seismic excitation tensile forces may exceed the tensile capacity of the joint, however shear through the bars will still occur. We believe that the ex-Council Chambers will provide lateral support to the 1997 Alterations but the Main Building can no longer provide support to the 1997 Alterations in the models Y direction. The tension transfer required between the existing Main Building suspended floor and the new flat slab is insufficient for the building to work as a whole and therefore the building is considered to achieve a lower NBS risk rating. Please note that a seating angle under the 1997 Alteration floor slab that will support the joint for gravity loads even if complete failure of the tensile capacity occurs.

8 Conclusions

The Modal analysis has produced the following results

- The floor slab joint between the two building phases has not been designed for diaphragm over strength forces and may start to split during a serviceability level event, however due to the construction of the joint with an angle supporting the precast units, sudden collapse is unlikely to occur. We recommend remedial to this joint in order to increase the building strength.
- Should the floor joint start to fail as per the previous point then critical wall in shear core of the 1973 building has been assessed as 35% NBS at Importance Level 4.
  Should the building be downgraded to an Importance level 2, non-civil defence rating, structure then the assessment increases 60% NBS.
- If the joint is strengthened to provide sufficient capacity along its length and the building has been considered as a whole structure and no separation of the building phases occur then the 1973 shear core has been assessed as 45% NBS for an Importance Level 4 structure. The shear core has been reassessed as 80% NBS for an Importance Level 2 structure.
- The columns to the original building have been assessed as 70% NBS risk between levels 2 and 3 for an Importance Level 4 structure.

- The 1997 shear core (block masonry) has been assessed as 85% NBS risk between levels 2 and 3 for an Importance Level 4 structure.
- The central shear core does not have over strength reactions transferred in to the foundations. Currently this has been approximated as 40% NBS risk for an Importance Level 4 structure.
- Higher levels of shear wall ductility have not been considered as part of this analysis. This will provide an increase to the % NBS for the walls, however the diaphragms and foundations need to be still considered with a ductility of 1. Should the floor joint be strengthened and the foundations improved then the shear walls will provide above 70% NBS utilising a higher ductility level.

Critical Weakness Element	Importance Level 4	Importance Level 2
Wall (Original) (with floor split)	35%	60%
Wall (Original) (without floor split)	45%	80%
Frames (Original)	70%	>100%
Wall (1997 Alterations)	85%	>100%
Foundations	40%	70%

Table 8.1 Table of Approximate % NBS for Critical Weaknesses of varying Importance Levels

9 Recommendations

Importance Level 4 Structure – Civil Defence facility

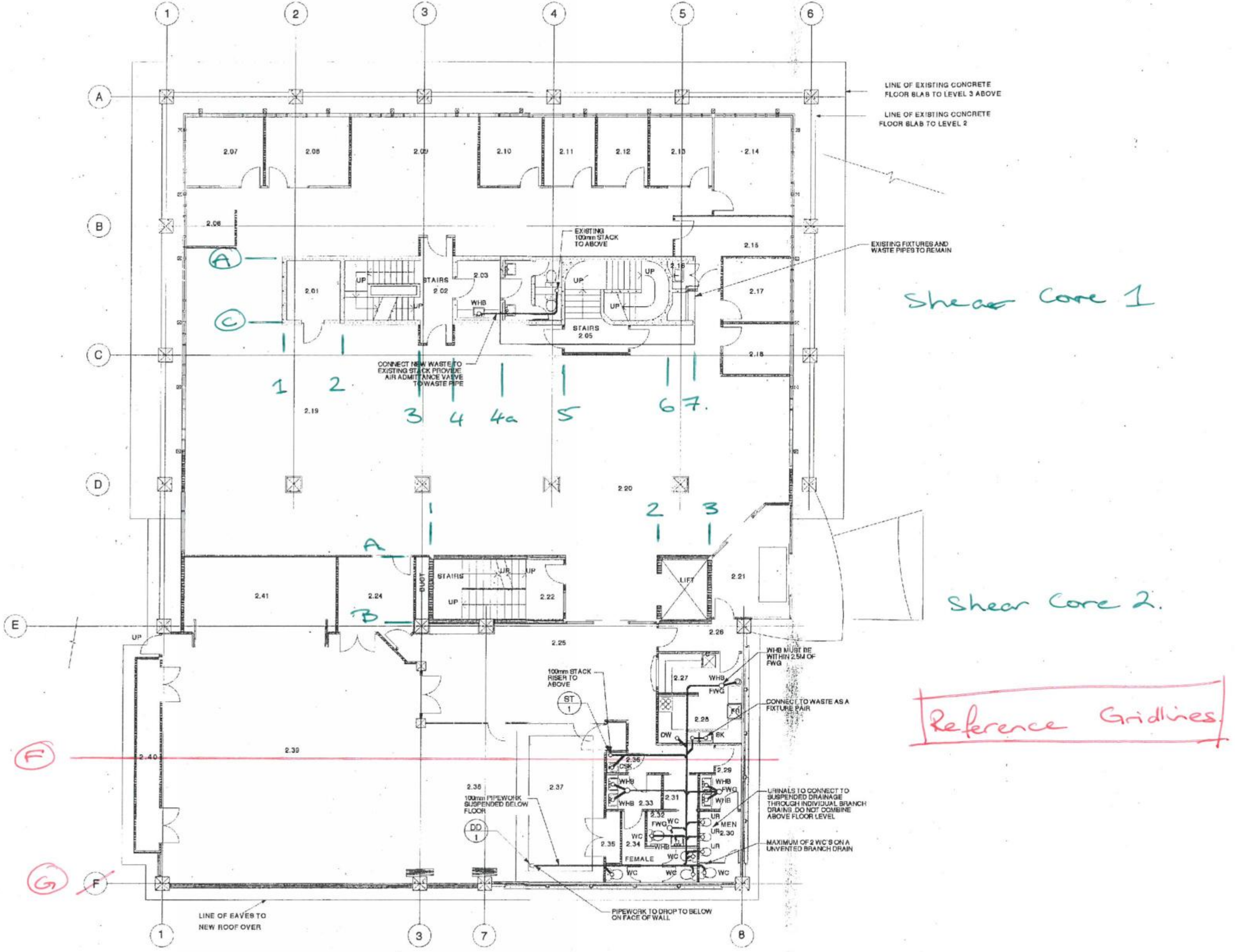
- Strengthening of the Floor Joint between the 1973 building and the 1997 alterations. Options for strengthening this joint include:
  - Mechanically fix to the underside using, for example, plates and bolts
  - Using a Fibre Reinforced Polymer over the joint by drilling and epoxying
- Strengthening of Foundations for overturning

  A geotechnical investigation would need to be undertaken to determine the strength and liquefaction potential of the soil underlying the structure. Options for strengthening include:
  - New piles, possible screw piles that are installed in short lengths due to head height restrictions
  - New shallow foundation beams to provide a rafting effect for the shear core
- Complete a detailed geotechnical investigation that will confirm/alter the design assumption on ground conditions and seismic soil type category.

Importance Level 2 Structure – non-Civil Defence facility

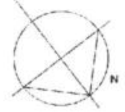
- Strengthening of the Floor Joint between the 1973 building and the 1997 alterations. Options for strengthening this joint are similar as for the Importance Level 4 structure.
- Complete a detailed geotechnical investigation that will confirm/alter the design assumption on ground conditions and seismic soil type category.

Appendix A – Reference Floor Plans



ROOM SCHEDULE - NEW

2.01	STORE
2.02	STAIRS
2.03	WC LOBBY
2.04	FEMALE WC
2.05	STAIRS
2.06	COPY, PRINTER
2.07	OFFICE, ENVIRON SAFETY MANAGER
2.08	HEALTH OFFICERS
2.09	PLANNERS
2.10	OFFICE, PLANNER
2.11	OFFICE, BUILDING
2.12	OFFICE, BUILDING
2.13	OFFICE, BUILDING
2.14	PUBLIC SAFETY MANAGER
2.15	MICROFICH, PRINTER
2.16	CLEANERS CUPBOARD
2.17	INTERVIEW 1
2.18	INTERVIEW 2
2.19	GENERAL OFFICE AREA
2.20	RECEPTION
2.21	WIND LOBBY
2.22	STAIRS
2.23	PATCH PANEL CUPBOARD
2.24	STORE/ANTE
2.25	LOBBY/WAITING
2.26	CORRIDOR
2.27	SERVERY/BAR
2.28	KITCHEN
2.29	WC LOBBY
2.30	MALE WC
2.31	ACCESSIBLE WC LOBBY
2.32	ACCESSIBLE WC
2.33	WC LOBBY
2.34	FEMALE WC
2.35	CIVIL DEFENCE STORE
2.36	CLEANERS CUPBOARD
2.37	COMMITTEE ROOM 1
2.38	COMMITTEE ROOM 2
2.39	COUNCIL CHAMBERS
2.40	STORE
2.41	STORE



FITTINGS LEGEND

- WC PAN
- WALL HUNG URINAL
- EXISTING STAINLESS STEEL URINAL
- WASH HAND BASINS
- SHOWER TRAY
- SINK
- FRIDGE
- CHILLER

WALL ELEMENTS KEY

- EXISTING 75X50 TIMBER FRAMED INTERNAL PARTITIONS
- NEW TIMBER FRAMED PARTITIONS
- EXISTING CONCRETE WALLS
- NEW CONCRETE WALLS
- 2.30 NEW ROOM NUMBERS

REFER TO DIAGRAMMATIC DRAWING H140 FOR PIPE SIZES

Scale 1: 100 Original A1
0m 1.0 2.0 3.0 4.0 5.0

Issue	Details	Date
1	General Revisions	04/08/97
Orig	Preliminary Issue	16/07/97

Symbols

- RV 5 :Relief Vent & No.
- ST 5 :Stack & No.
- DP 5 :Downpipe & No.
- GR 5 :Grease Vent & No.
- DR 5 :Drain Riser & No.
- DD 5 :Drain Dropper & No.
- :Pipe Riser/Fixture
- :Pipe Dropper
- :Sanitary Pipe, C Water pipe or Drain
- :Vent Pipe or Hot Water pipe
- FU's 206 :Fixture Units

B	:Bath
BI ORG	:Back Inlet ORG
BO	:Balcony Outlet
COS	:Clear Out to Surface
CSK	:Cleaners Sink
CD	:Condensate
CTS	:Connected to sewer
CU	:Copper
CWD	:Cold Water Dropper
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GIT	:Grease Interceptor Trap
GVB	:Group Vented Branch
GV	:Group Vent
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Project

WAIPA DISTRICT
COUNCIL NEW SERVICE
CENTRE & LIBRARY

Drawing Title

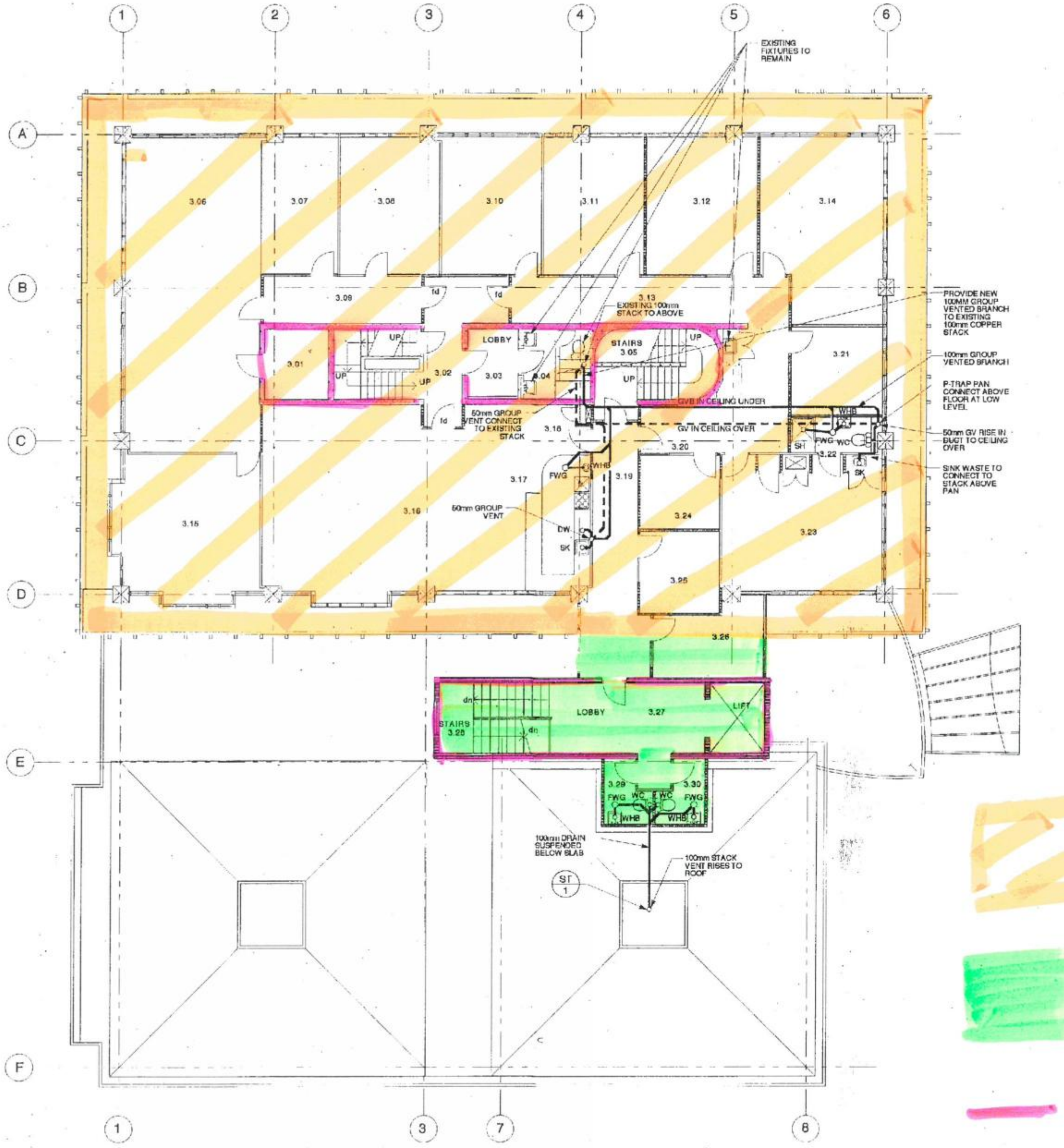
HYDRAULIC SERVICES:
LEVEL 2 SANITARY
DRAINAGE LAYOUT

Job No.	Drawn	Designed	Date	Scale
7891	SMO	Peter Downey	16/07/97	1:100 @ A1

THE CONTRACTOR: must verify all dimensions on the site prior to commencement of work.

Amendment
1

Drawing No.
H 120



🟧 1973 Original building

🟩 1997 Alterations

▬ walls designated as shear walls.

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3.03	WC LOBBY
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3.05	STAIRS
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3.07	PHOTOCOPY
3.08	EXECUTIVE SECRETARYS
3.09	CORRIDOR
3.10	OFFICE, ESSENTIAL SERVICES MANAGER
3.11	OFFICE, ENVIRONMENTAL SERVICES MANAGER
3.12	OFFICE, CORPORATE SERVICES MANAGER
3.13	CORRIDOR
3.14	OFFICE, C.E.O
3.15	OFFICE, MANAGEMENT ACCOUNTING
3.16	CAFETERIA
3.17	KITCHEN
3.18	PATCH PANEL CUPBOARD
3.19	CORRIDOR
3.20	CORRIDOR
3.21	EXECUTIVE SECRETARYS
3.22	WC
3.23	MAYORS OFFICE
3.24	OFFICE INTERVIEW
3.25	COMMUNICATIONS OFFICE
3.26	SECRETARY
3.27	LOBBY
3.28	STAIRS
3.29	WC
3.30	WC

FITTINGS LEGEND

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- NEW CONCRETE WALLS
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REFER TO DIAGRAMMATIC DRAWING H140 FOR PIPE SIZES

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Project

WAIPA DISTRICT
COUNCIL NEW SERVICE
CENTRE & LIBRARY

Drawing Title

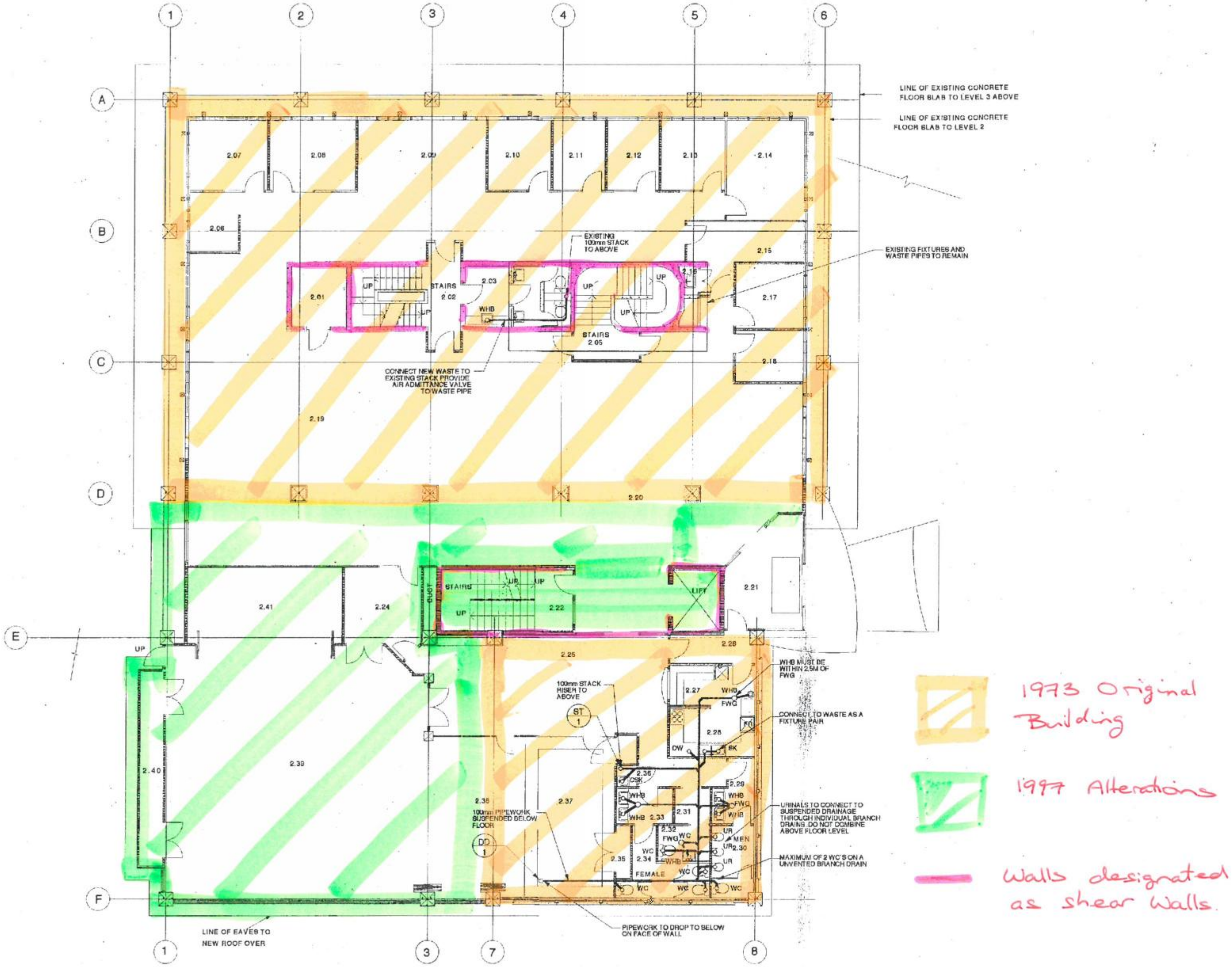
HYDRAULIC SERVICES:
LEVEL 3 SANITARY
DRAINAGE LAYOUT

THE CONTRACTOR: must verify all dimensions on the site prior to commencement of work.

Amendment 1

Job No.	Drawn	Designed	Date	Scale
7891	SMO	Peter Downey	16/07/97	1:100 @ A1

Drawing No.
H 130



🟧 1973 Original Building

🟩 1997 Alterations

▬ Walls designated as shear walls.

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2.09	PLANNERS
2.10	OFFICE, PLANNER
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2.12	OFFICE, BUILDING
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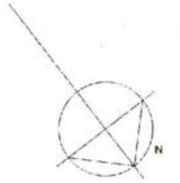

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REFER TO DIAGRAMMATIC DRAWING H140 FOR PIPE SIZES

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0m 1.0 2.0 3.0 4.0 5.0

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Orig	Preliminary Issue	16/07/97

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Project

WAIPA DISTRICT
COUNCIL NEW SERVICE
CENTRE & LIBRARY

Drawing Title

HYDRAULIC SERVICES:
LEVEL 2 SANITARY
DRAINAGE LAYOUT

Job No.	Drawn	Designed	Date	Scale
7891	SMO	Peter Downey	16/07/97	1:100 @ A1

THE CONTRACTOR: must verify all dimensions on the site prior to commencement of work.

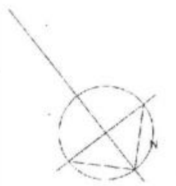
Amendment
1

Drawing No.
H 120

🟪 walls, concrete or block

ROOM SCHEDULE - NEW

1.01	STORE
1.02	STAIRS
1.03	WC LOBBY
1.04	WC
1.05	STORE
1.06	OFFICE / ASSET SERVICES
1.07	OFFICE / ASSET SERVICES MAN.
1.08	OFFICE / I.M.S.
1.09	CORRIDOR
1.10	CORRIDOR
1.11	CORRIDOR
1.12	OFFICE
1.13	OFFICE / ROADING
1.14	STORE / HOLD
1.15	SICK BAY
1.16	PHOTOCOPY ROOM
1.17	STORE
1.18	STORE
1.19	STORE
1.20	CORRIDOR
1.21	CORRIDOR
1.22	CORRIDOR
1.23	BOILER ROOM
1.24	I.S STORE TELECOM TERM. PT.
1.25	STORE
1.26	CORRIDOR
1.27	WC LOBBY
1.28	FEMALE WC
1.29	STORE
1.30	LIFT PLANT ROOM
1.31	LOBBY/ STAIRS
1.32	STORE
1.33	LOBBY / CORRIDOR
1.34	PATCH PANEL CUPBOARD
1.35	SERVER
1.36	PRINTER
1.37	INFORMATION SERVICES
1.38	RUBBISH HOLD
1.39	CORRIDOR
1.40	CLEANERS CUPBOARD
1.41	OFFICE, ASSET MANAGER PARKS
1.42	OFFICE, DOG CONTROL
1.43	OFFICE, ADMIN SERV. MAN.
1.44	RECORDS/PAYROLL/TELEPHONISTS
1.45	LUNDIA SHELVING



FITTINGS LEGEND

- WC PAN
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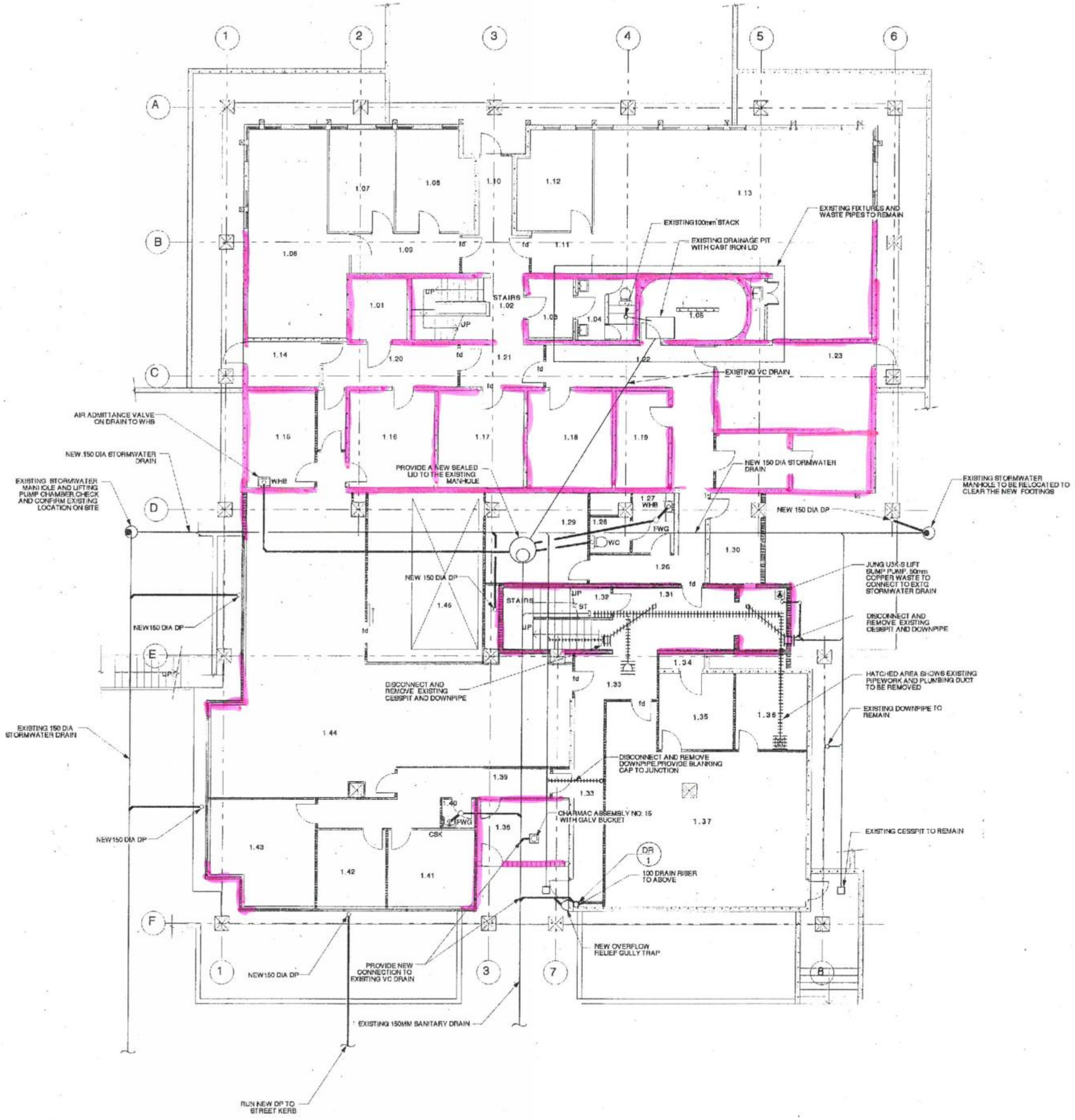
- EXISTING STAINLESS STEEL URINAL
- WASH HAND BASINS
- SHOWER TRAY
- SINK
- FRIDGE
- DRINKS CHILLER



SCHEDULE OF HYDRAULIC SERVICES DRAWINGS

100 SERIES DRAWINGS ~ SANITARY SYSTEMS DESCRIPTION

H 110	LEVEL 1 SANITARY DRAINAGE LAYOUT
H 120	LEVEL 2 SANITARY DRAINAGE LAYOUT.
H 130	LEVEL 3 SANITARY DRAINAGE LAYOUT
H 140	SANITARY DIAGRAMMATIC LAYOUT

REFER TO DIAGRAMMATIC DRAWING H140 FOR PIPE SIZES

NOTES

1. THIS SERVICES DESIGN HAS BEEN PREPARED IN ACCORDANCE WITH THE REQUIREMENTS OF AUSTRALIAN STANDARD 3500 2. AND THE NZBC HANDBOOK. FOLLOW THOSE DOCUMENTS TO ACHIEVE COMPLIANCE WITH THE NZBC.

2. ALL DRAINAGE, SOIL, WASTE AND VENT PIPE AND FITTINGS SHALL BE JAMES HARDIE PIPE LINES "SUPER TUFF" SEWER PIPE IN ACCORDANCE WITH AS/NZS 1260 AND SHALL BE INSTALLED WITH SOLVENT WELDED JOINTS, [EXPANSION JOINTS EXCEPTED].

3. ALL PLUMBING, DRAINAGE AND MATERIALS SHALL BE INSTALLED IN ACCORDANCE WITH THE RELEVANT STANDARDS, CODES, BYLAWS. CONTRACTOR TO INSPECT SITE TO SATISFY HIMSELF OF PROPOSED WORKING CONDITIONS.

4. THESE DRAWINGS ARE A GUIDE FOR DRAIN POSITIONS ONLY. REFER TO THE ARCHITECTURAL AND EQUIPMENT LAYOUT DRAWINGS FOR EXACT POSITIONS OF FIXTURES AND EQUIPMENT. CONTRACTOR IS RESPONSIBLE FOR COORDINATION OF HIS SERVICES WITH WORK OF OTHER TRADES AND DISCIPLINES. NOTIFY ARCHITECT IF DOUBT EXISTS AS TO ANY ASPECT OF DESIGN FEASIBILITY.

5. CONTRACTOR TO VERIFY THE POSITIONS AND DEPTHS OF ALL EXISTING UTILITY SERVICE PIPES WITHIN AND ADJACENT TO THE SITE THAT AFFECT THE WORKS AND COORDINATE THE WORKS WITH THESE SERVICES **PRIOR TO COMMENCING ANY WORKS.**

6. GIVE 48 HOURS NOTICE AND RECEIVE APPROVAL, PRIOR TO THE CLOSING DOWN OF ANY SERVICES.

7. ALL WORK TO BE CARRIED OUT BY REGISTERED TRADESMEN STRICTLY IN ACCORDANCE WITH THE RELEVANT ACT/S.

8. NOTICE AT START OF WORK TO BE GIVEN AND NO WORK TO BE CONCEALED UNTIL EXAMINED BE TA'S INSPECTOR.

9. BASINS TO HAVE 40 DIA **WASTE OUTLETS**, TRAPS AND WASTE PIPES. SINKS TO HAVE 40 DIA **WASTE OUTLETS**, TRAPS AND WASTE PIPES, CLEANERS SINKS, SHOWERS AND BAR SINKS TO HAVE 40 DIA WASTE OUTLETS, TRAPS AND WASTE PIPES

10. FWG'S MUST BE IN THE SAME ROOM AS THE FIXTURE THEY SERVE.

11. MAX DISTANCE, FIXTURE TO FWG UNTRAPPED = 1.2 METRES
MAX DISTANCE FIXTURE TO FWG TRAPPED = 2.5 METRES [ALL BASINS MUST BE TRAPPED]

12. MIN FALL 100 DIA SANITARY PIPES = 1.60 [1:80 WITH APPROVAL FROM ARCHITECT]
MIN FALL 100 DIA STORMWATER DRAINS = 1:80
MIN FALL 65, 50, 40 DIA PIPES = 1:40.

13. ALL PIPES UNDER THE SLAB-ON-GROUND ARE DRAINS AND SHALL BE LAID IN A CORRECTLY EXCAVATED TRENCH WITH CORRECT FALL IN APPROVED BEDDING MATERIAL. BACKFILL AND COMPACTION SHALL BE TO THE APPROVAL OF THE STRUCTURAL ENGINEER AND THE TA. A MINIMUM OF 25 MM CLEARANCE SHALL BE ACHIEVED/MAINTAINED BETWEEN THE SOFFIT OF ANY PIPE AND THE UNDERSIDE OF ANY CONCRETE SLAB.

Scale 1: 100 Original A1
0m 1.0 2.0 3.0 4.0 5.0

Issue	Details	Date
2	General Revisions	04/09/97
1	Consent Issue	12/08/97
Orig	Preliminary Issue	14/07/97

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SH	:Shower
SK	:Sink
SNDU	:Sanitary Napkin Disposal Unit
SP	:Sanitary Plumbing
SW	:Stormwater
TA	:To Above
TB	:To Below
TG	:Test Gate
TD	:Tundish
UR	:Urinal
VB	:Vanity Basin
VC	:Vent Cowl
VPR	:Vent Pipe Riser
VCP	:Vitrified Clay Pipe
WM	:Washing Machine
W&T	:Waste & Trap
WC	:Water Closet
WHB	:Wash Hand Basin

Architect

Chow:Hill
ARCHITECTS LTD
DEVELOPMENT CONSULTANTS
119 Collingwood Street, P.O. Box 19208, Hamilton.
Telephone 07-834 0343. Fax 07-834 2156.

Hydraulic Consultant

HYDRAULIC
SERVICES
CONSULTANTS
4th Floor Westpac Building
135-151 Broadway, Newmarket
Fax 64-9-520 7739
Ph 64-9-520 7738

Project

WAIPA DISTRICT COUNCIL
HEAD OFFICE ALTERATION &
REFURBISHMENT

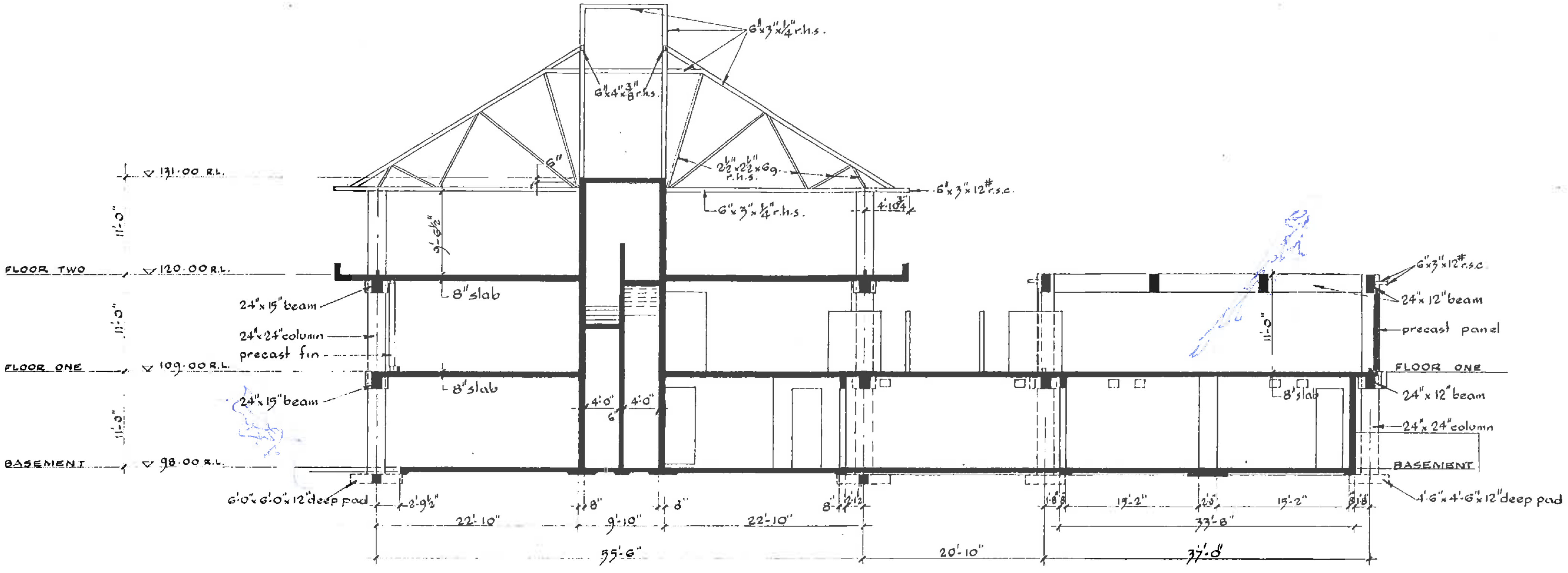
Drawing Title

HYDRAULIC SERVICES:
LEVEL 1 SANITARY
DRAINAGE LAYOUT

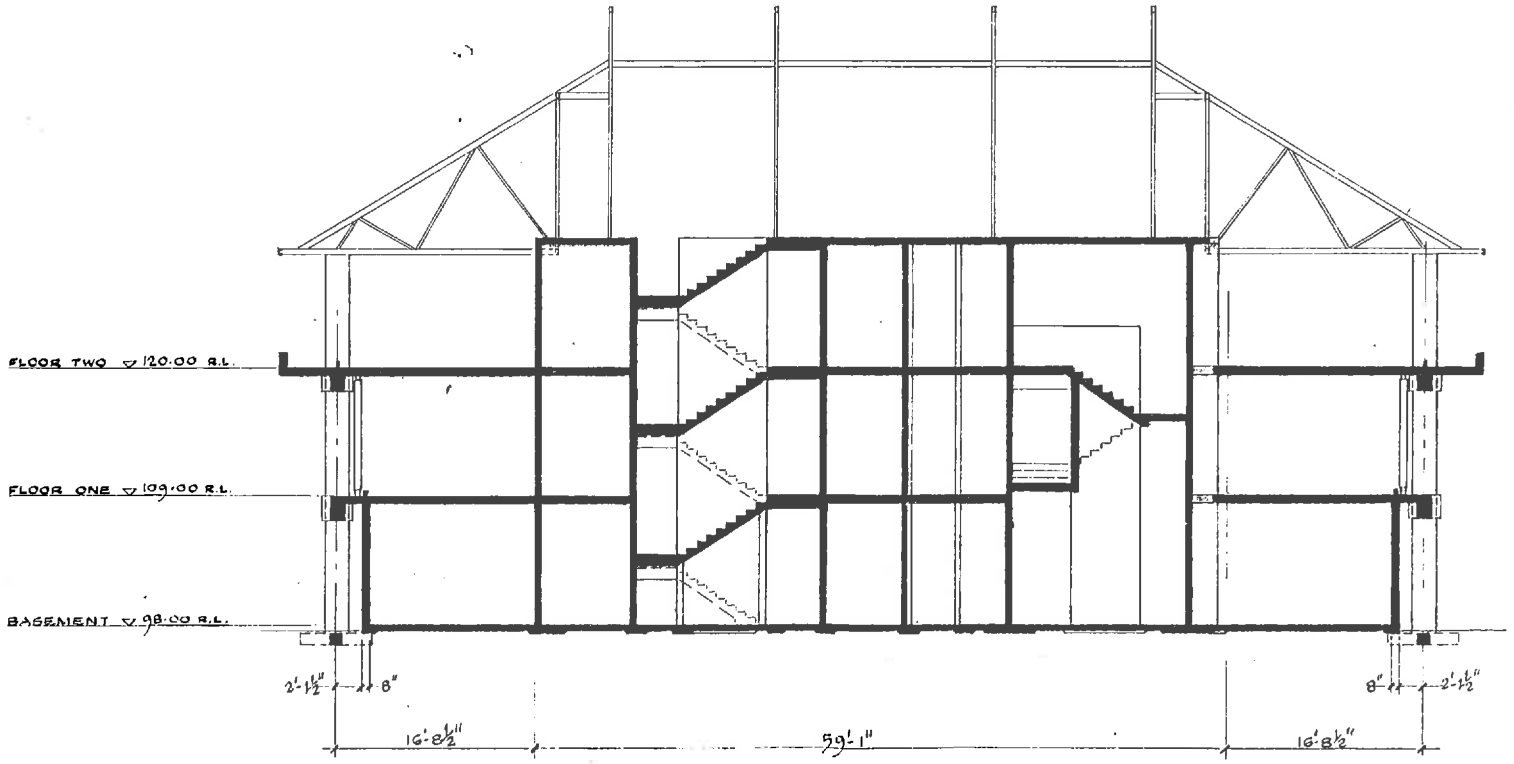
THE CONTRACTOR: must verify all dimensions on the site prior to commencement of work.

Amendment
2

Job No.	Drawn	Designed	Date	Scale	Drawing No.
7891	SMO	Peter Downey	15/07/97	1:100 @ A1	H 110



CROSS-SECTION ⅛"=1ft.



LONGITUDINAL SECTION ⅛"=1ft.

DATE	REVISION

DRAWN J. Burton
CHECKED
APPROVED
DATE May '73
SCALE ⅛"=1ft.
JOB No 736

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THE CONTRACTOR MUST CHECK ALL DIMENSIONS ON SITE

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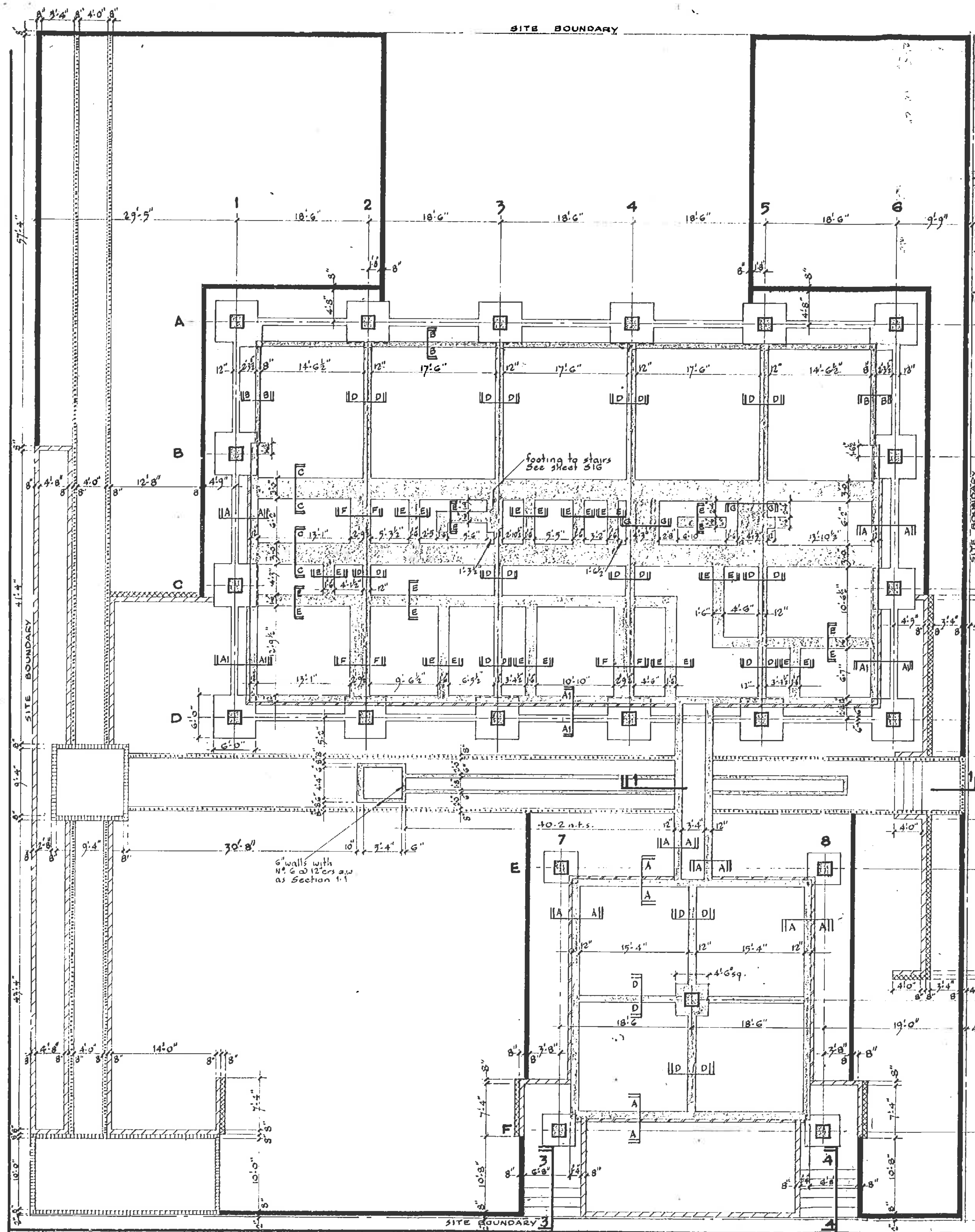
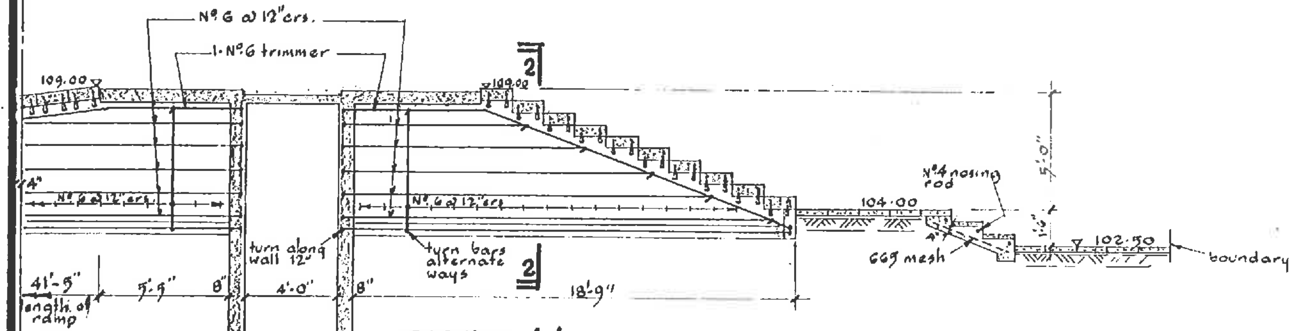
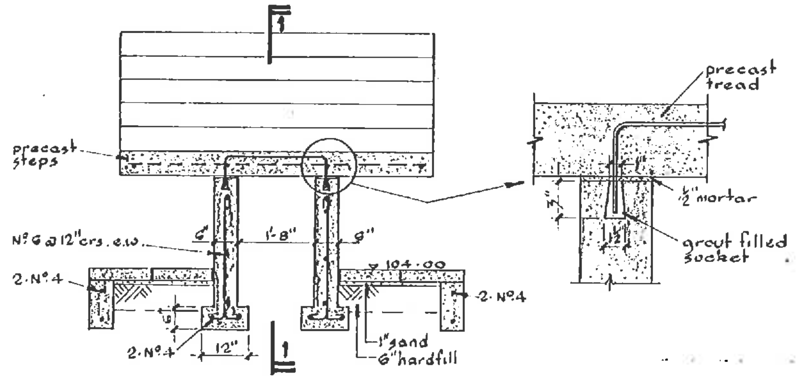
WAIPA COUNTY COUNCIL OFFICE
STRUCTURAL
CROSS-SECTIONS

DRG. No
450/S1

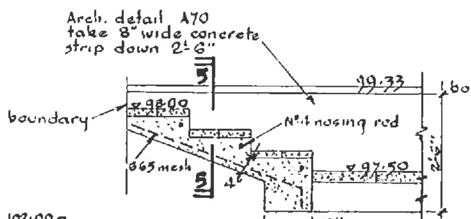




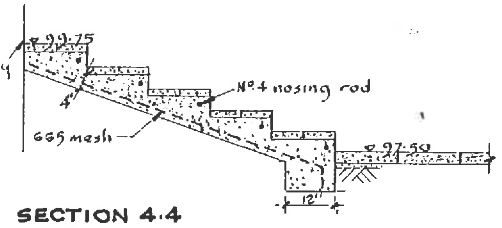
SECTION 1·1
¼" = 1ft.



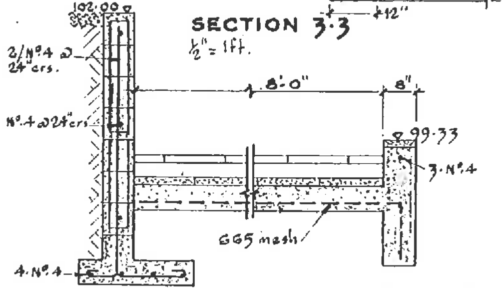
SECTION 2·2
½" = 1ft.



SECTION 3·3
½" = 1ft.

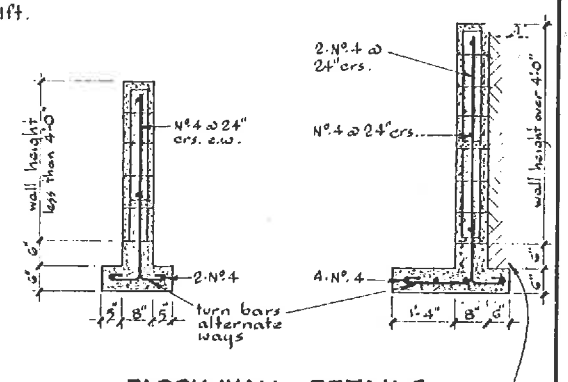


SECTION 4·4
½" = 1ft.



SECTION 5·5 ½" = 1ft.

For levels see Architects drg. 1/1
For details A·A to G·G see sheet 53



BLOCK·WALL DETAILS
½" = 1ft.

	ARCHITECTS DETAIL	
▤	A64	2-No 4 6" wide x 12" deep concrete strip
▥	A66	2-No 4 6" wide x 12" deep concrete strip
▨	A67	See blockwork detail
▩	A68	1-No 4 8" wide x 6" deep mowing strip - joints @ 25'-0" crs
■	A70	2-No 4 8" wide x 12" deep concrete strip
▨	A78	1-No 4 4" wide x 3" deep concrete strip
	A61	2-No 4 8" wide x 4" deep concrete strip @ high level on Line 'D'

FOUNDATION PLAN
⅛" = 1ft.

DATE	REVISION

DRAWN	J. Burton	DATE	June 1 '73
CHECKED		SCALE	As shown
APPROVED		JOB No	736

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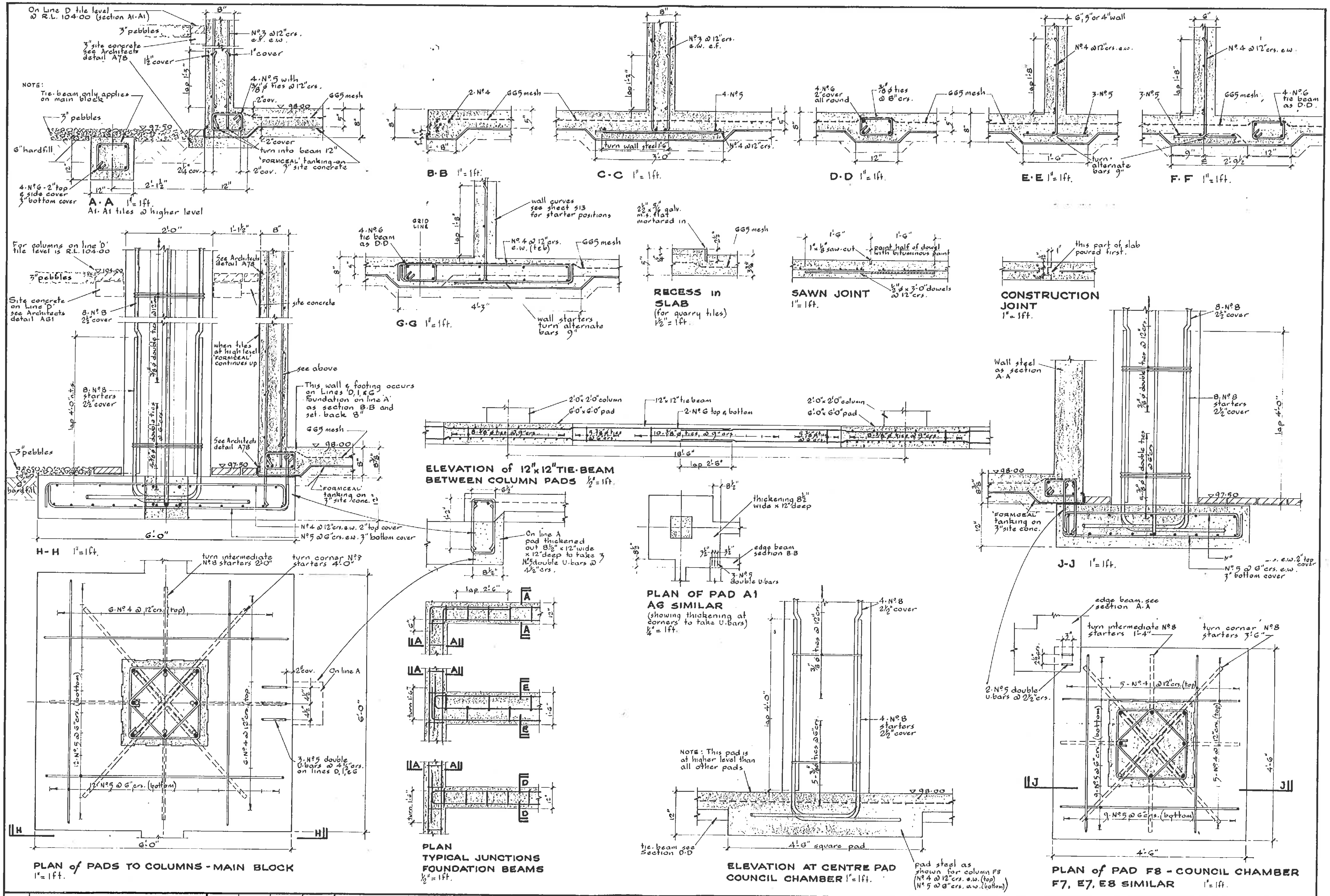
WAIPA COUNTY COUNCIL OFFICE
FOUNDATION PLAN & ENTRANCE DETAILS

DRG. No
450/S2



DATE	REVISION	DRAWN	J. Burton	DATE	June '73
		CHECKED		SCALE	As shown
		APPROVED		JOB No	736

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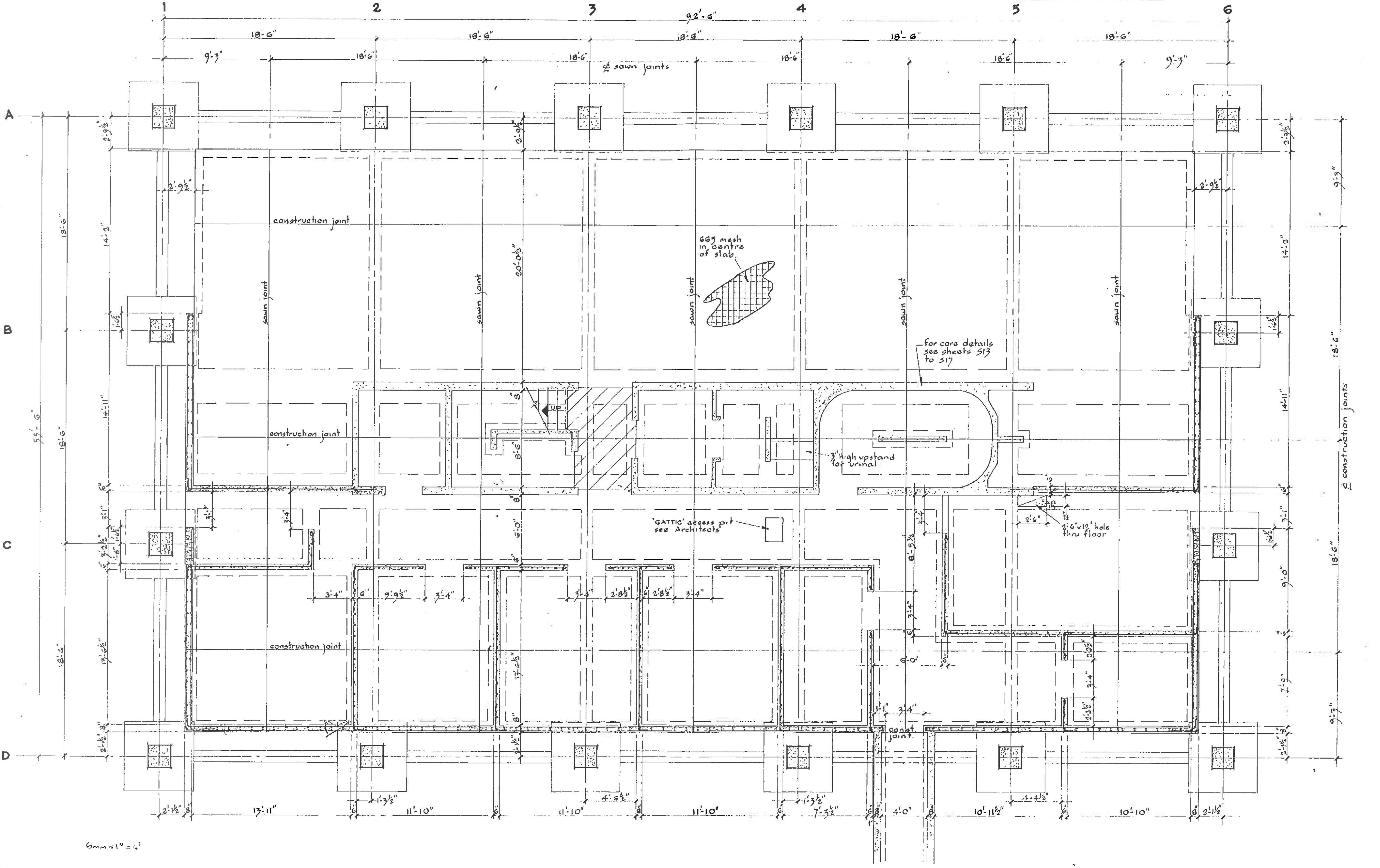
WAIPA COUNTY COUNCIL OFFICE
FOUNDATION DETAILS

DRG. No
450/S3



BASEMENT PLAN

¼" = 1ft.

▨ shaded area depressed 1¼"

DATE	REVISION

DRAWN J.Burton DATE May '73
CHECKED SCALE ¼" = 1ft.
APPROVED JOB No 736

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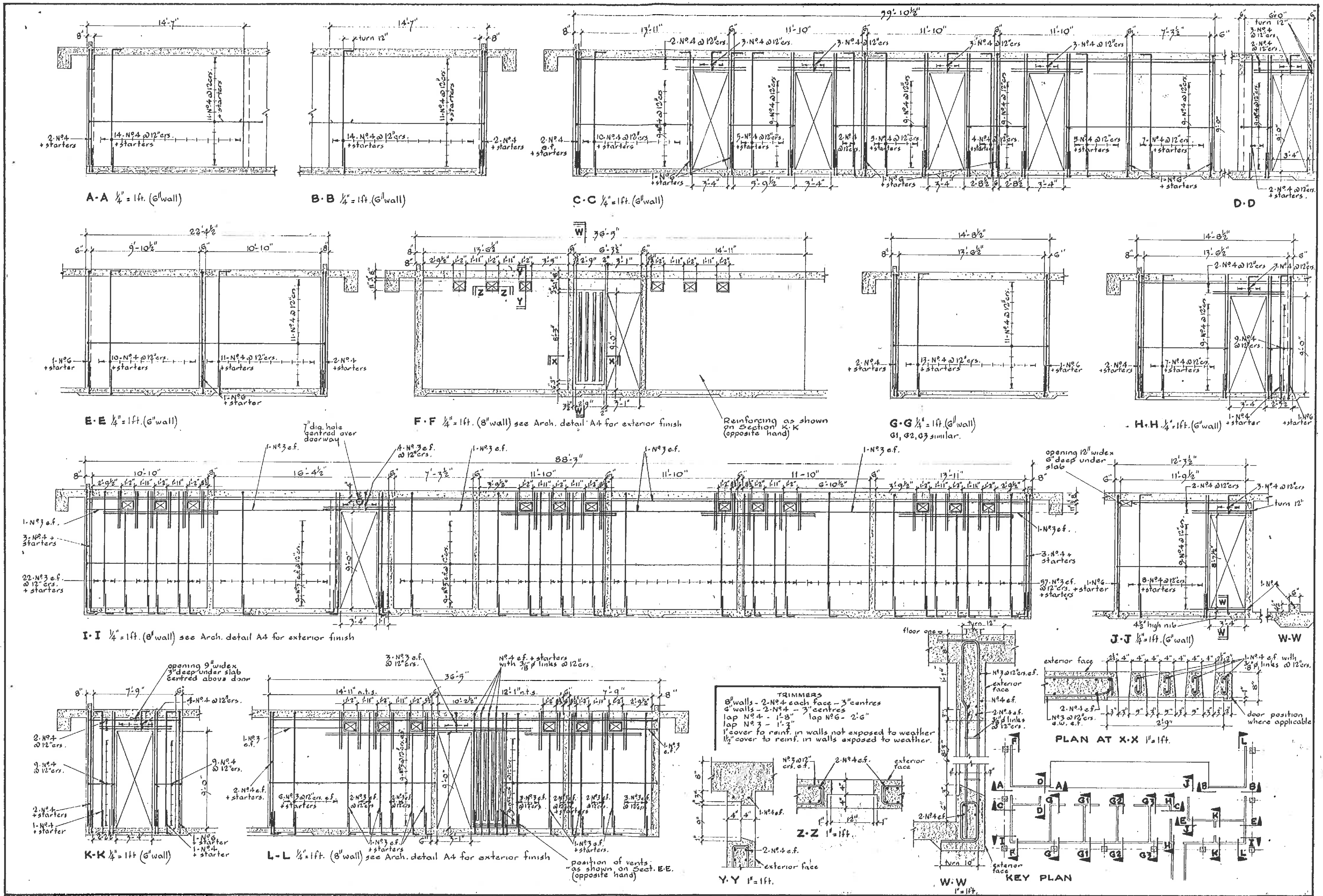
WAIPA COUNTY COUNCIL OFFICE
MAIN BLOCK
BASEMENT FLOOR PLAN

DRG. No
450/S4



TRIMMERS
8" walls - 2-N° 4 each face - 3" centres
6" walls - 2-N° 4 - 3" centres
lap N° 4 - 1'-8" lap N° 6 - 2'-6"
lap N° 3 - 1'-3"
1" cover to reinf. in walls not exposed to weather
1½" cover to reinf. in walls exposed to weather.

DATE	REVISION

DRAWN J. Burton
CHECKED
APPROVED
DATE May 24 '73
SCALE 1/4" = 1ft.
JOB N° 736

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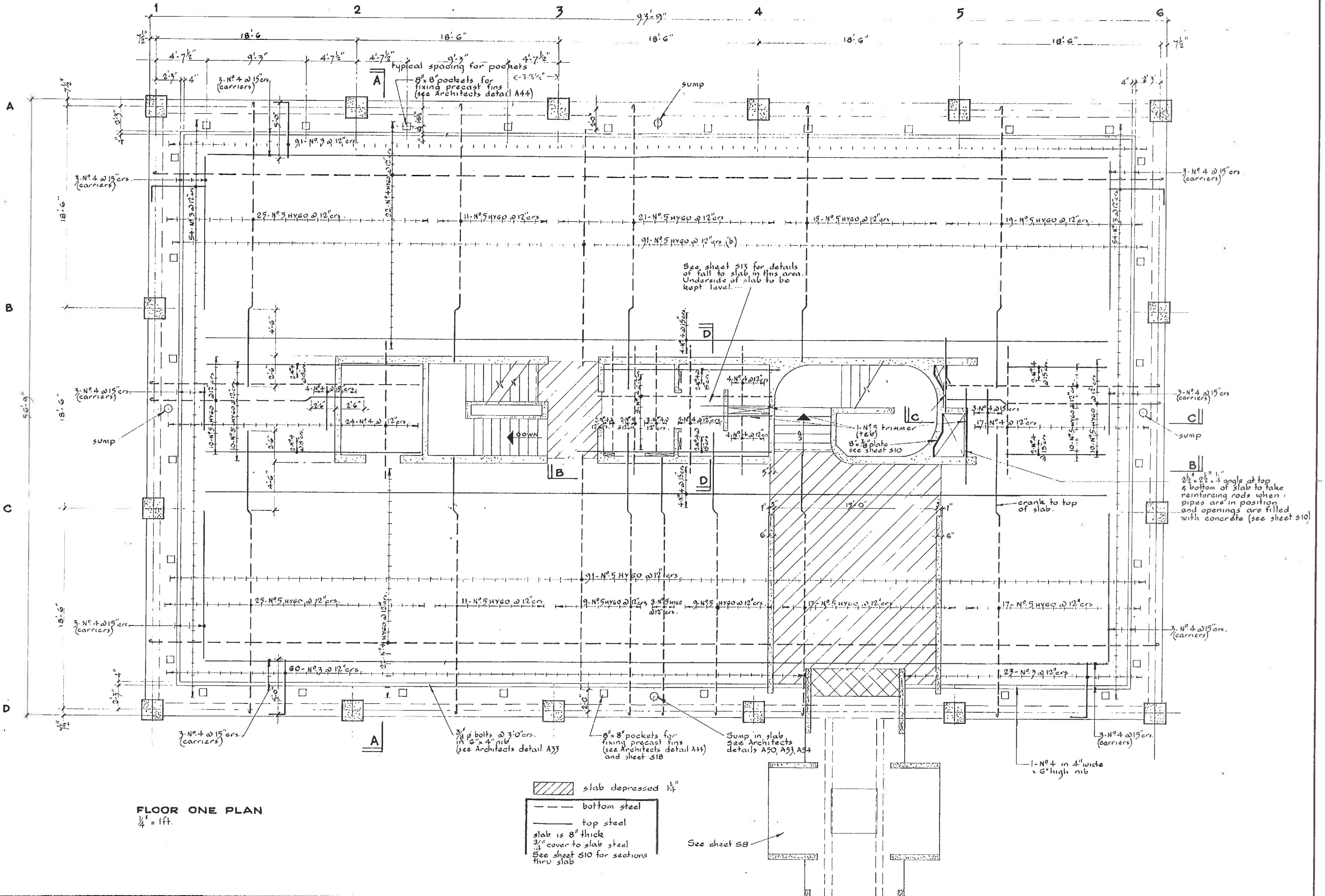
WAIPA COUNTY COUNCIL OFFICE
MAIN BLOCK
BASEMENT WALLS

DRG. N°
450/56



FLOOR ONE PLAN

¼" = 1ft.

DATE	REVISION

DRAWN	J.Burton	DATE	June '73
CHECKED		SCALE	¼" = 1ft.
APPROVED		JOB Nº	736

THE CONTRACTOR MUST CHECK ALL DIMENSIONS ON SITE

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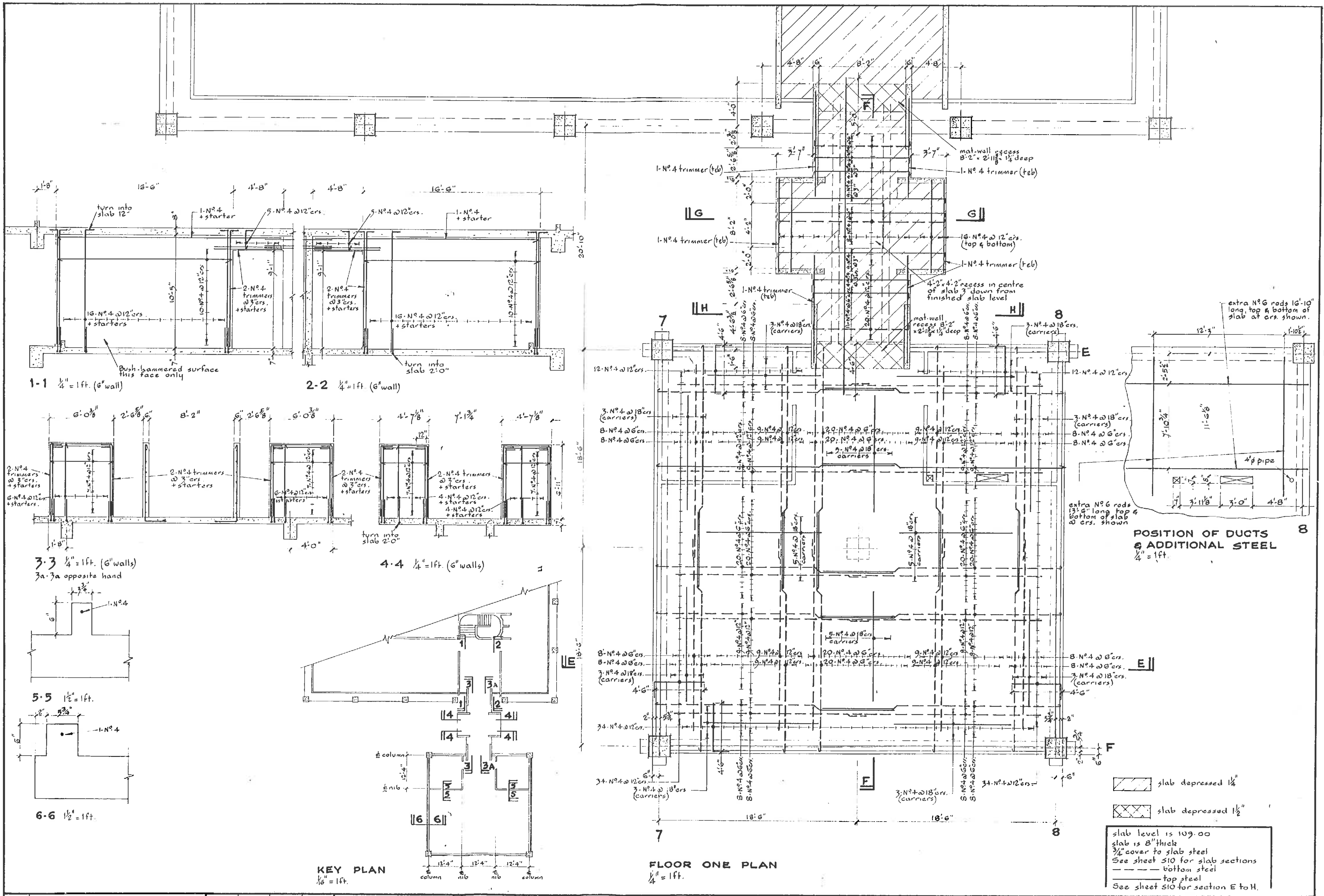
WAIPA COUNTY COUNCIL OFFICE
MAIN BLOCK
FLOOR ONE PLAN

DRG. Nº
450/S7



1-1 ¼"=1ft. (6"wall)

2-2 ¼"=1ft. (6"wall)

3-3 ¼"=1ft. (6"walls)
3a-3a opposite hand

4-4 ¼"=1ft. (6"walls)

5-5 1½"=1ft.

6-6 1½"=1ft.

KEY PLAN
1/16"=1ft.

FLOOR ONE PLAN
¼"=1ft.

POSITION OF DUCTS & ADDITIONAL STEEL
¼"=1ft.

▨ slab depressed 1¼"

☒ slab depressed 1½"

slab level is 109.00
slab is 8" thick
¾" cover to slab steel
See sheet 510 for slab sections
– – – – bottom steel
———— top steel
See sheet 510 for section E to H.

DATE	REVISION

DRAWN J.Burton DATE June 5 '73
CHECKED SCALE ¼"=1ft.
APPROVED JOB No 736

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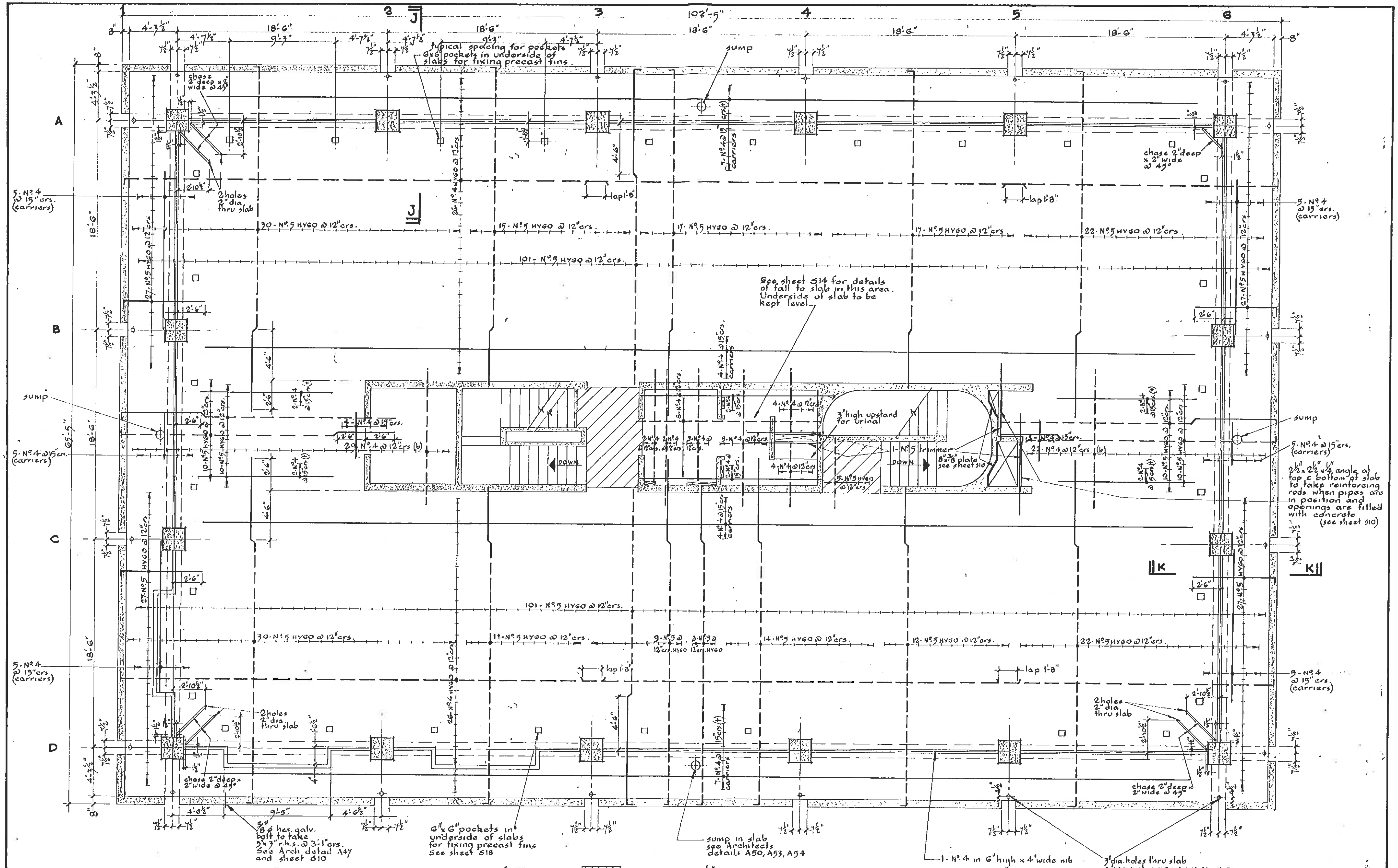
WAIPA COUNTY COUNCIL OFFICE
COUNCIL CHAMBER
FLOOR ONE PLAN

DRG. No
450/58



FLOOR TWO PLAN

¼" = 1ft.

▨ slab depressed ¼"

— — — bottom steel
——— top steel
slab is 8" thick
¾" cover to slab steel

Lap Nº4 - 1'-8"
Lap Nº5 - 2'-1"

DATE	REVISION

DRAWN	J. Burton	DATE	May '73
CHECKED		SCALE	¼" = 1ft.
APPROVED		JOB Nº	736

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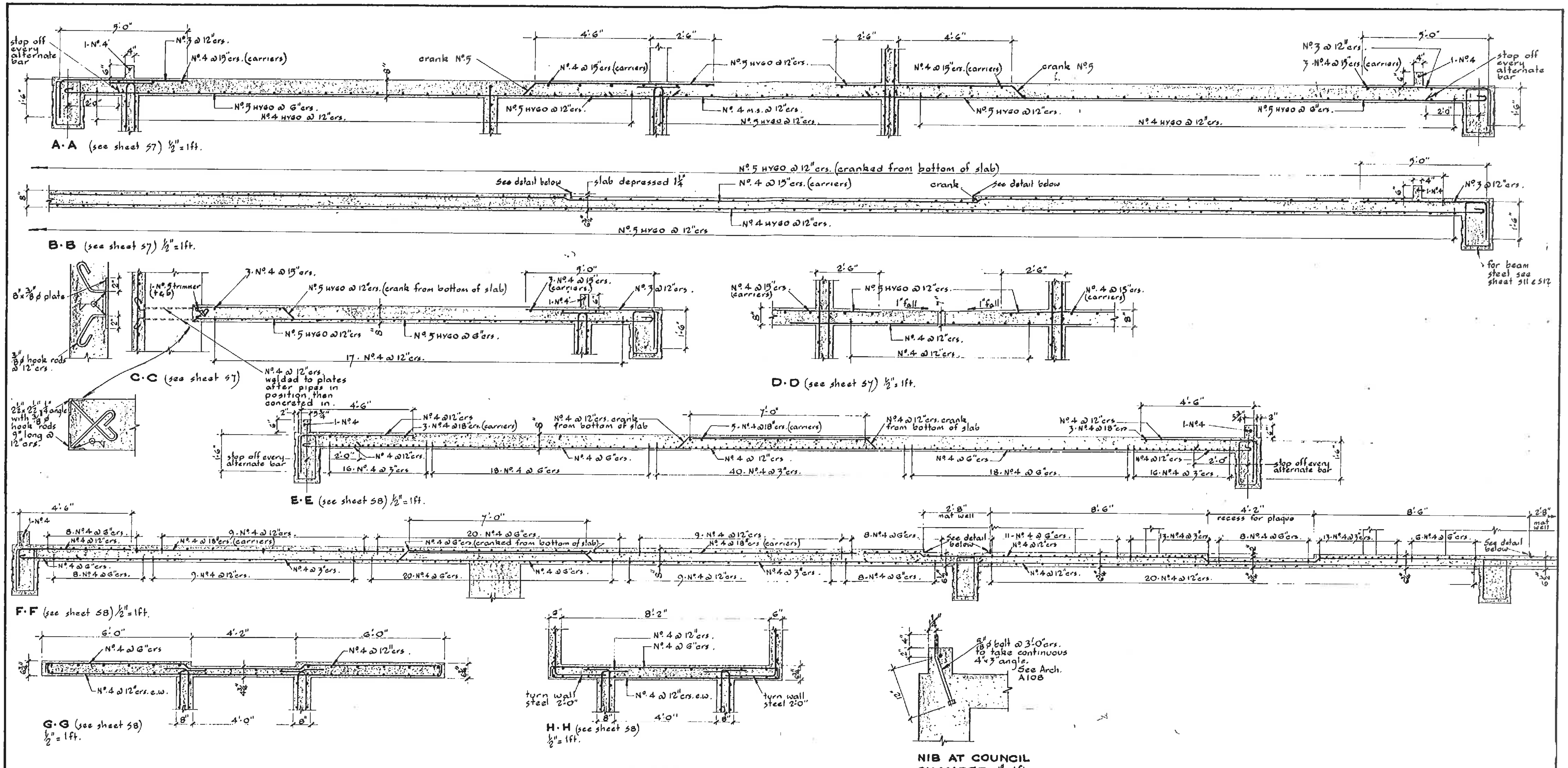
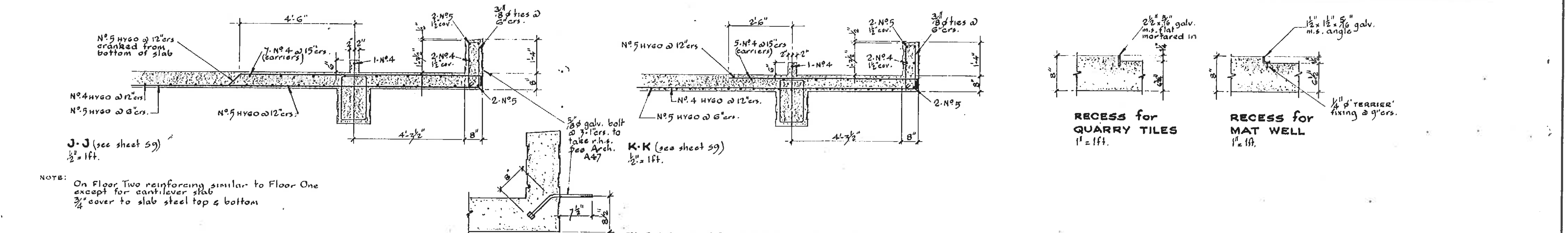
WAIPA COUNTY COUNCIL OFFICE
MAIN BLOCK
FLOOR TWO PLAN

DRG. Nº
450/S9



FLOOR ONE SECTIONS



NOTE: On Floor Two reinforcing similar to Floor One except for cantilever slab
¾" cover to slab steel top & bottom

FLOOR TWO SECTIONS

DATE	REVISION

DRAWN J. Burton DATE July '73
CHECKED SCALE As shown
APPROVED JOB No 736

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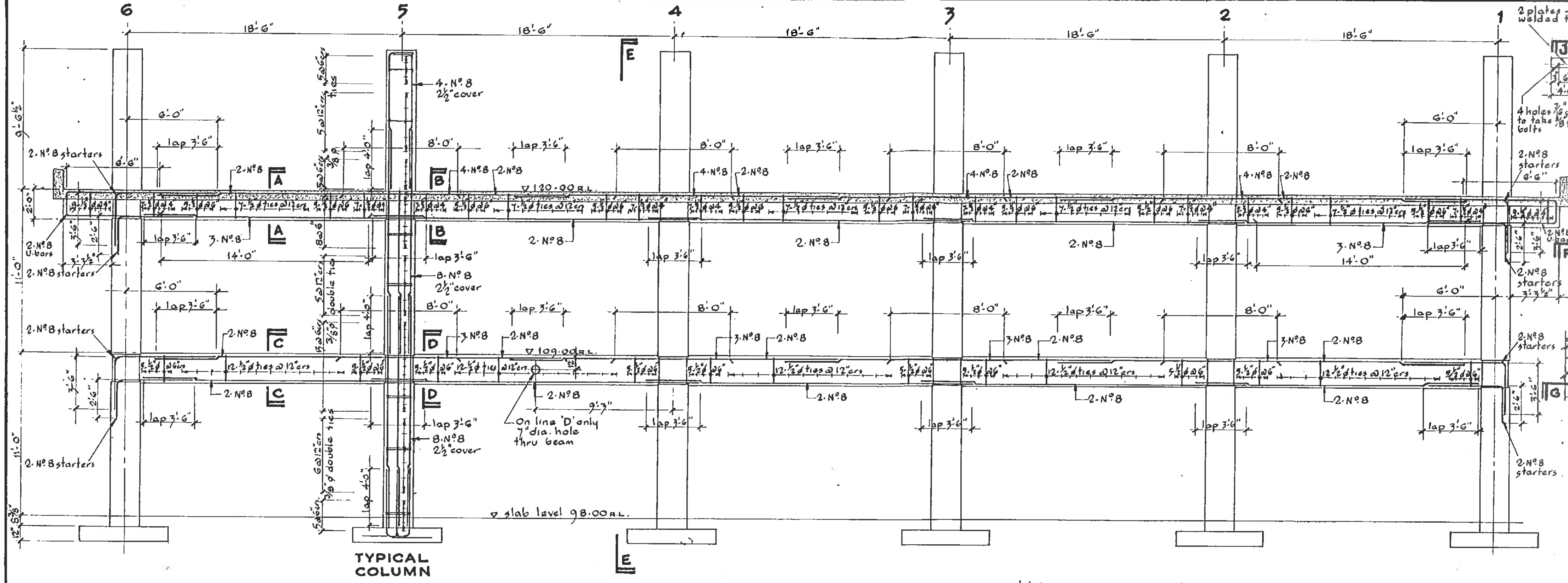
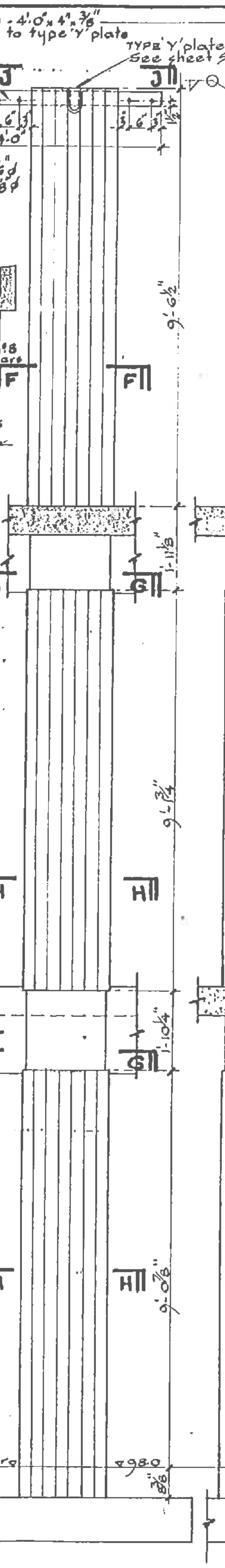
WAIPA COUNTY COUNCIL OFFICE
FLOORS ONE & TWO
SLAB SECTIONS

DRG. No 450/S10

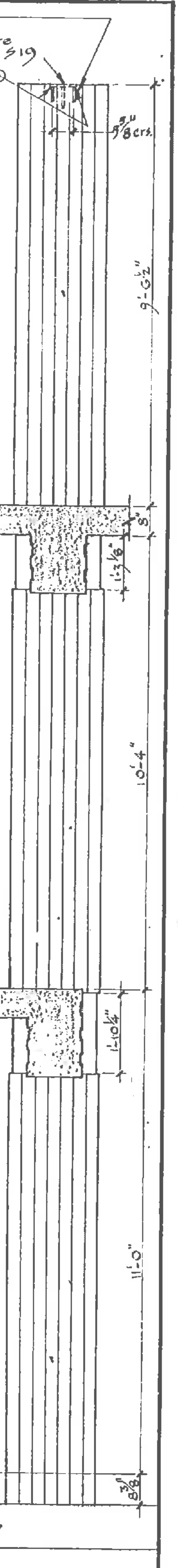


TYPICAL COLUMN

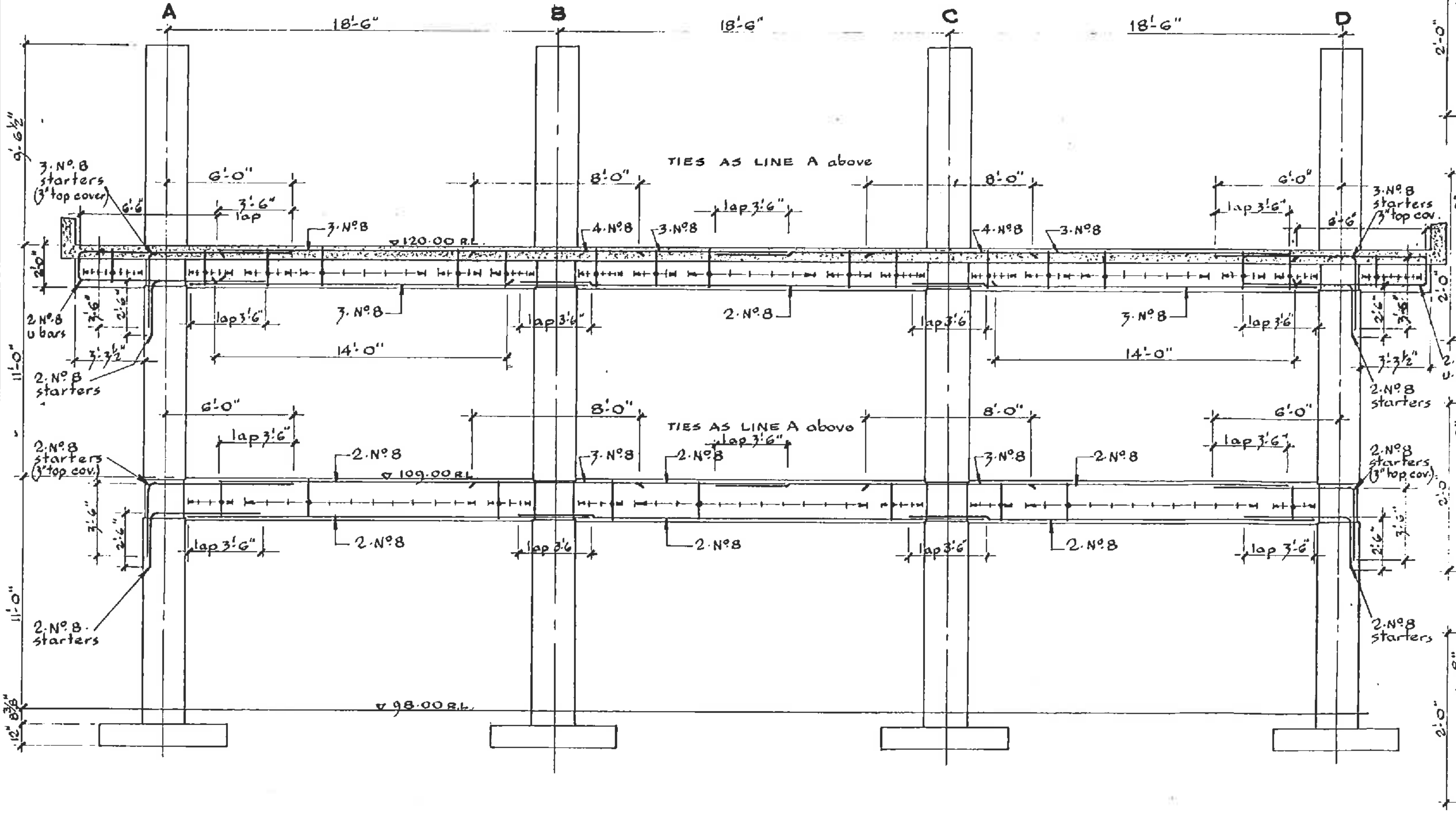
FRAME ON LINE 'A' ¼" = 1ft. (viewed from outside) LINE 'D' SIMILAR



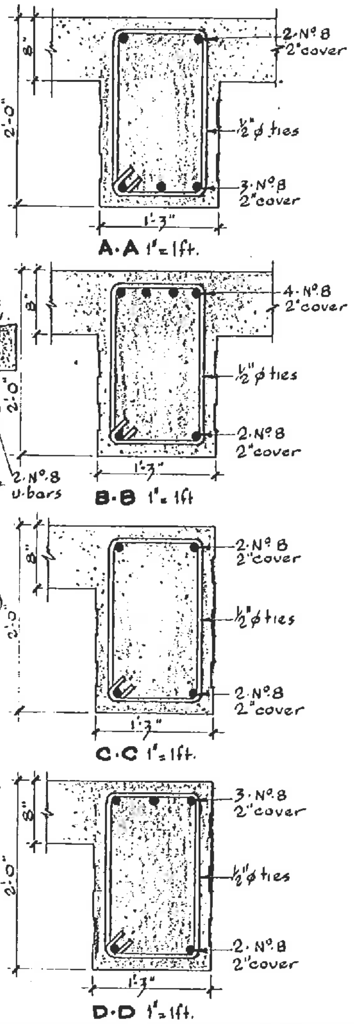
ELEVATION of COLUMN ½" = 1ft.



E-E ½" = 1ft.



FRAME ON LINE '1' ¼" = 1ft. (viewed from outside) LINE 6 SIMILAR

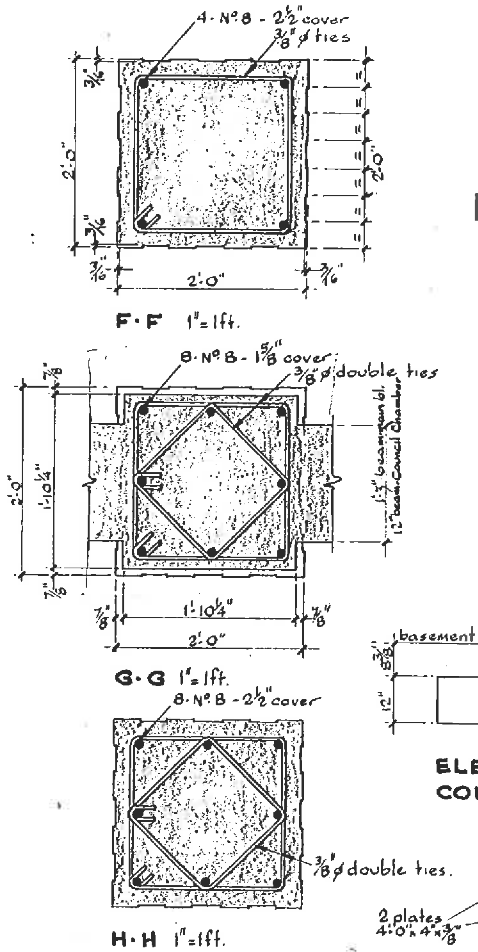


A-A 1" = 1ft.

B-B 1" = 1ft.

C-C 1" = 1ft.

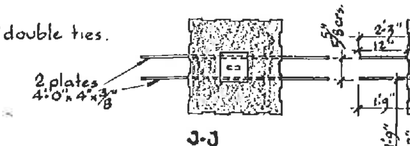
D-D 1" = 1ft.



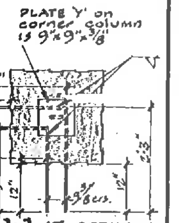
F-F 1" = 1ft.

G-G 1" = 1ft.

H-H 1" = 1ft.



J-J



J-J AT CORNERS

DATE	REVISION

DRAWN J. Burton DATE June 1'73
CHECKED SCALE As Shown
APPROVED JOB No 736

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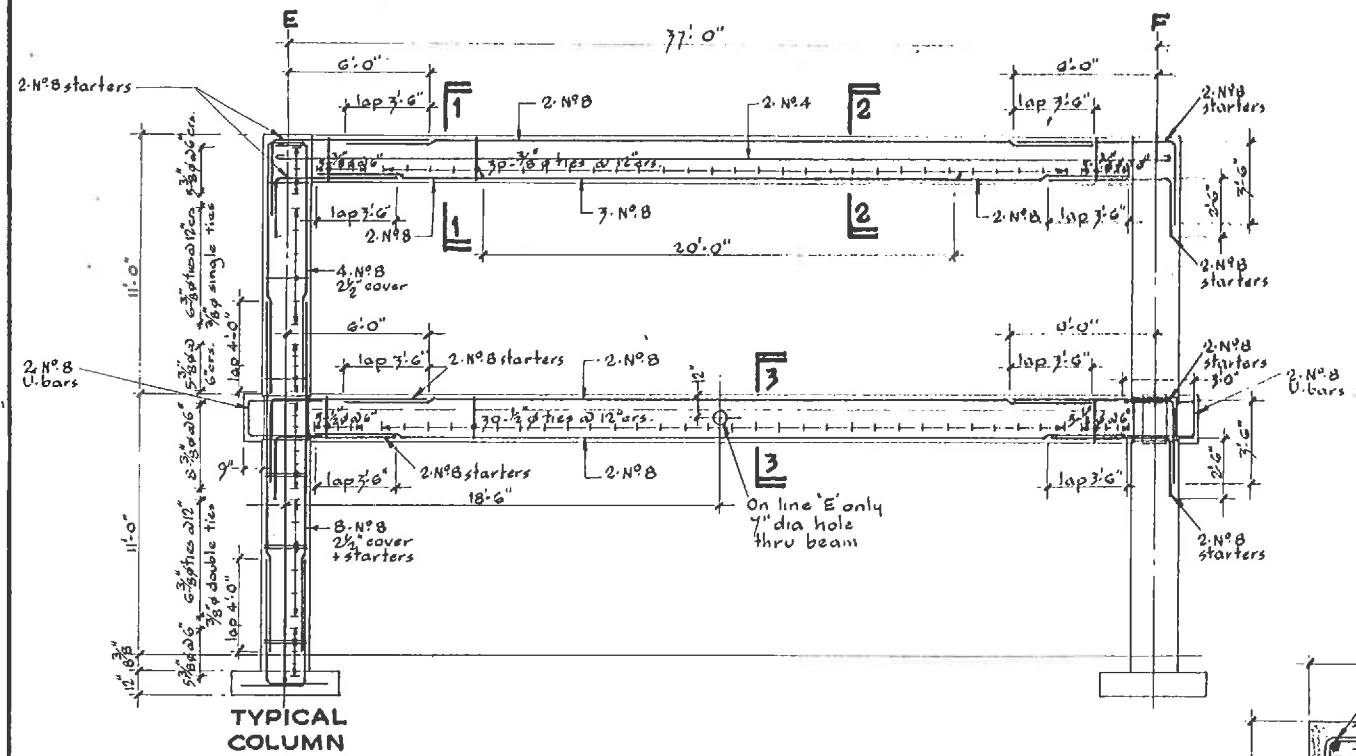
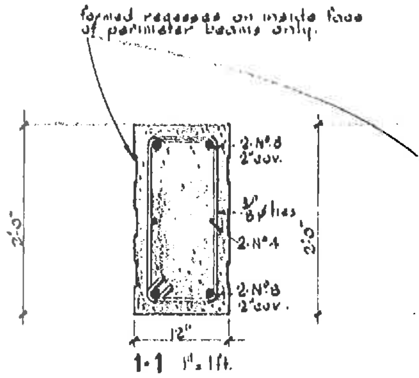
WAIPA COUNTY COUNCIL OFFICE
MAIN BLOCK
FRAME ELEVATIONS & COLUMNS

DRG. No
450/S11

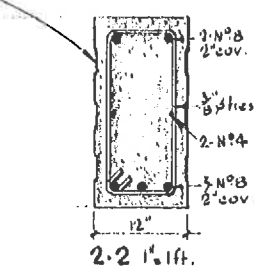


FRAME ON LINE 7

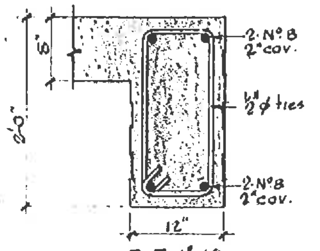
LINES B, E, & F SIMILAR



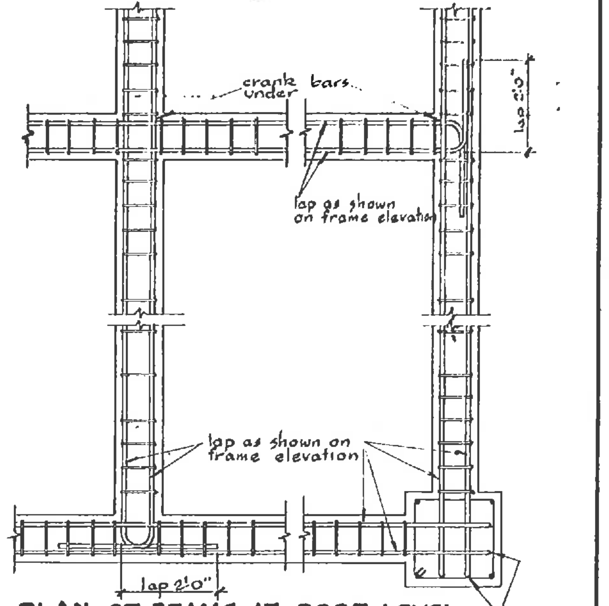
1-1 1"=1ft.



2-2 1"=1ft.

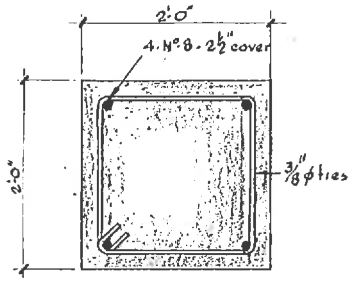


3-3 1"=1ft.

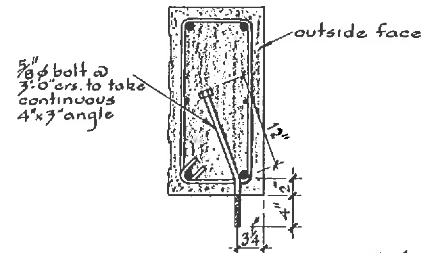


PLAN OF BEAMS AT ROOF LEVEL

½"=1ft.

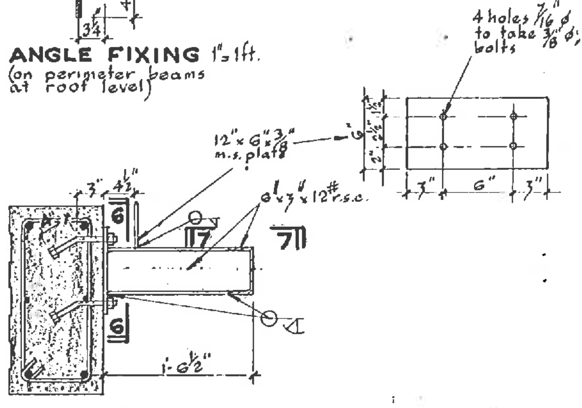


4-4 1"=1ft.

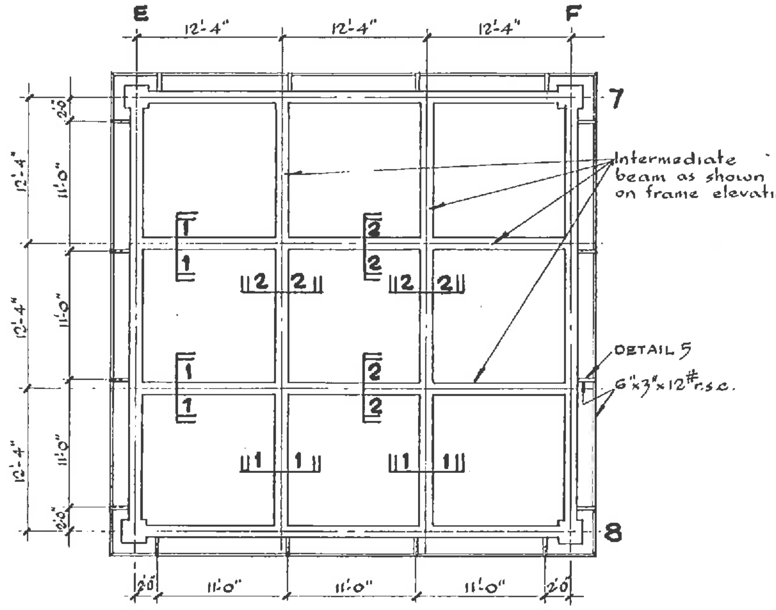


ANGLE FIXING 1"=1ft.

(on perimeter beams at roof level)

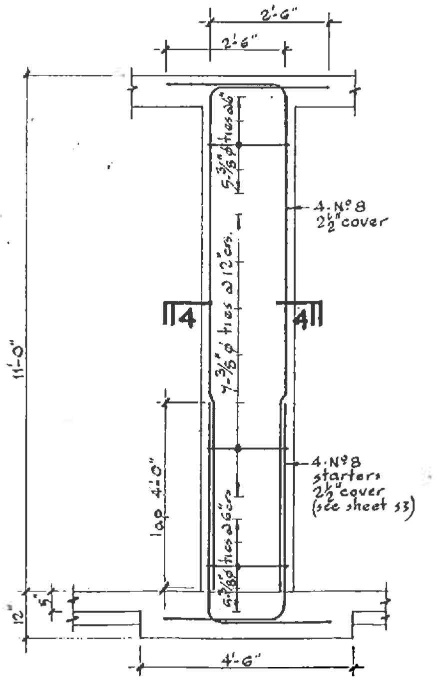


DETAIL 5 1"=1ft.

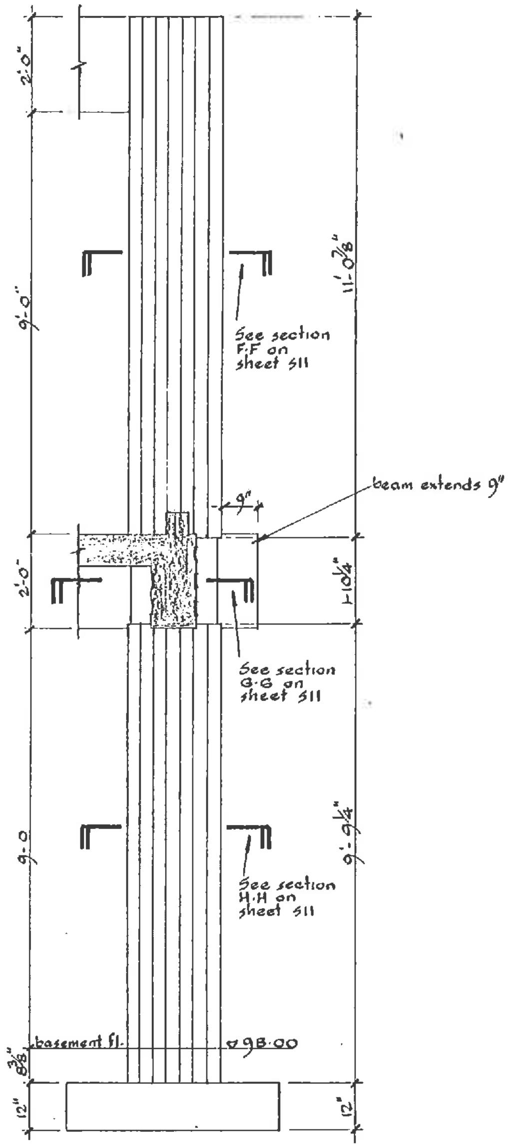


PLAN of BEAMS AT ROOF LEVEL

⅛" = 1ft. (All beams 24" deep x 12" wide)

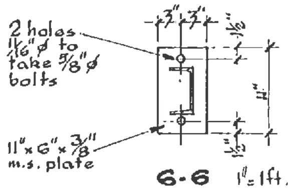


CENTRE COLUMN ½"=1ft.

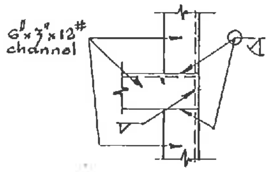


ELEVATION of COLUMN

½"=1ft.



6-6 1"=1ft.



7-7 1"=1ft.

DATE	REVISION

DRAWN	J. Burton	DATE	July '73
CHECKED		SCALE	As shown
APPROVED		JOB Nº	736

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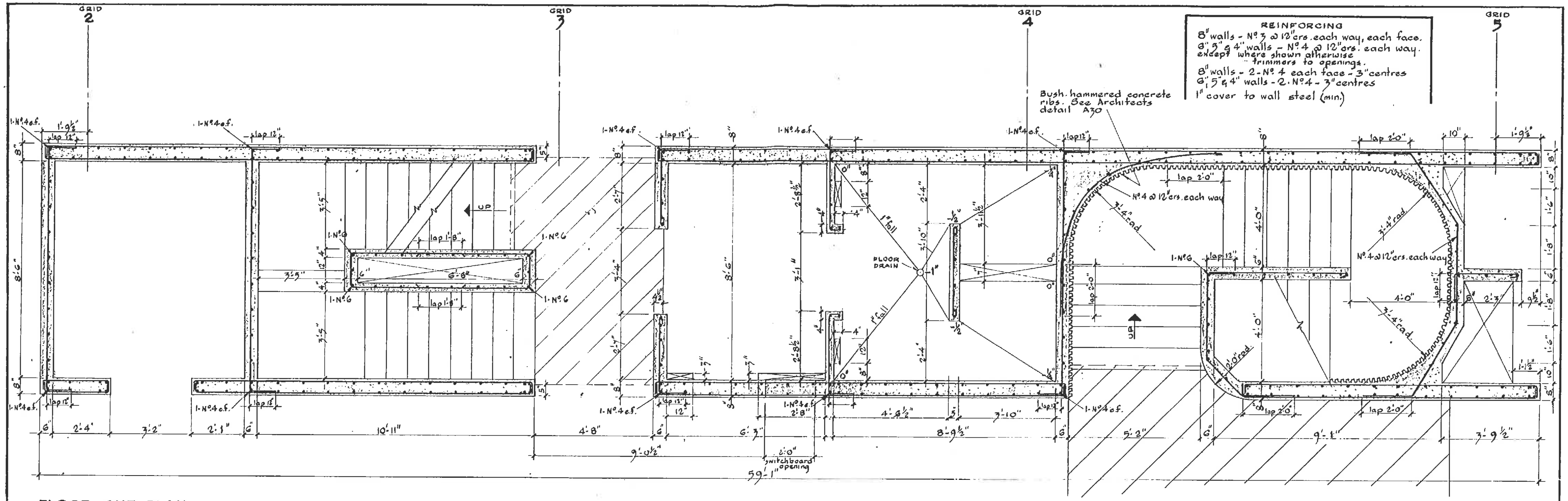
P.O. BOX 30444 LOWER HUTT TEL. 61-483 LOWER HUTT

WAIPA COUNTY COUNCIL OFFICE
COUNCIL CHAMBER
FRAME ELEVATIONS & COLUMNS

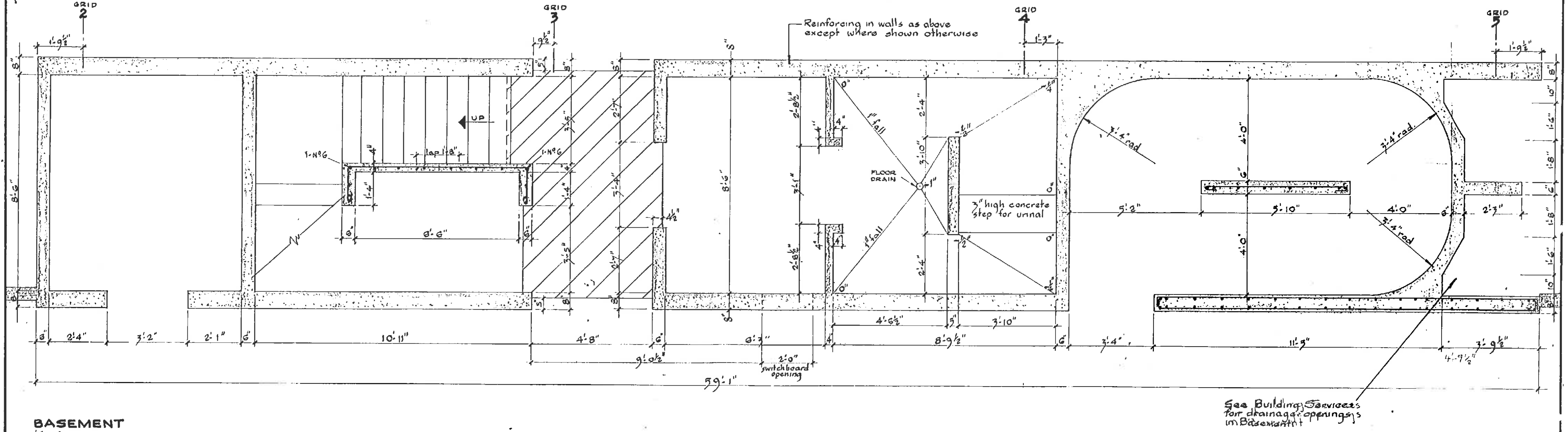
DRG. Nº
450/S12

REINFORCING

8" walls - Nº 3 @ 12"crs. each way, each face.
6", 5" & 4" walls - Nº 4 @ 12"crs. each way.
except where shown otherwise
trimmers to openings.
8" walls - 2 - Nº 4 each face - 3" centres
6", 5" & 4" walls - 2 · Nº 4 - 3" centres
1" cover to wall steel (min.)



FLOOR ONE PLAN
½" = 1ft.



BASEMENT
½" = 1ft.

▨ shaded area depressed 1¼"

DATE	REVISION

DRAWN	J.Burton	DATE	May 23'73
CHECKED		SCALE	½" = 1ft.
APPROVED		JOB Nº	736.

THE CONTRACTOR MUST CHECK ALL DIMENSIONS ON SITE

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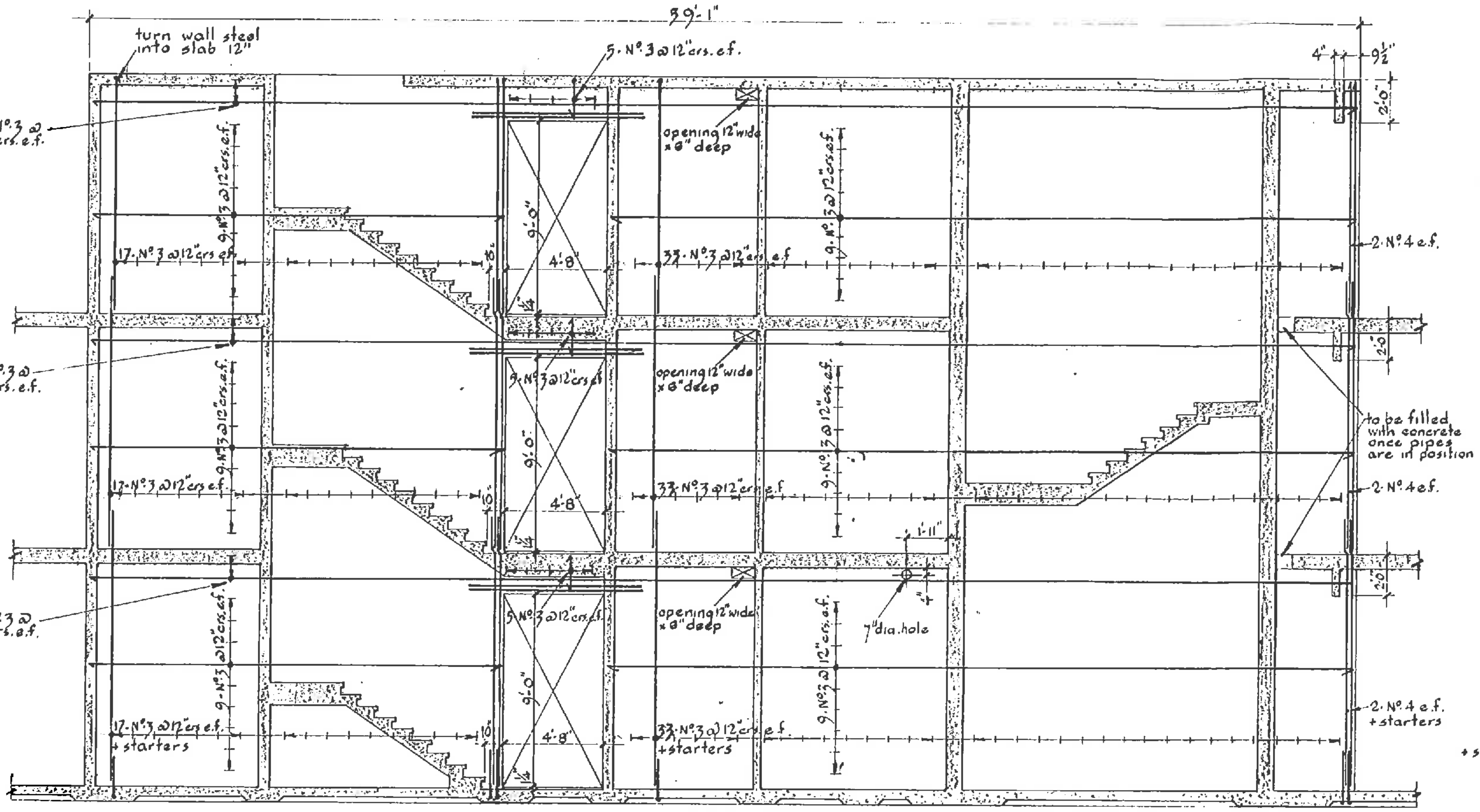
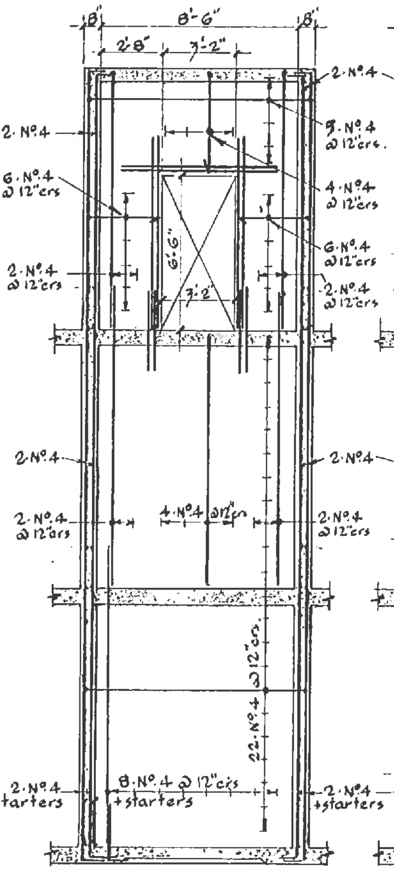
WAIPA COUNTY COUNCIL OFFICE
MAIN BLOCK
CORE PLANS · BASEMENT & FLOOR ONE

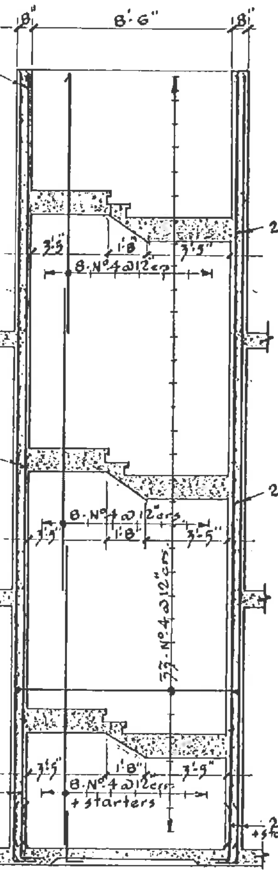
DRG. Nº.
450/S13



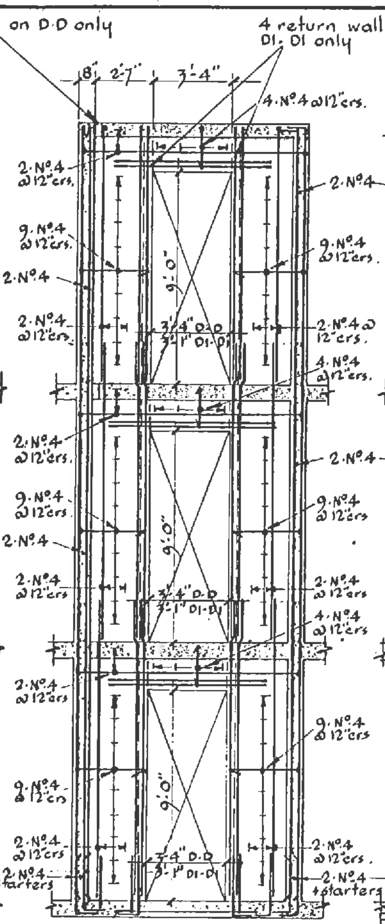
A-A 8" wall
¼" = 1ft.



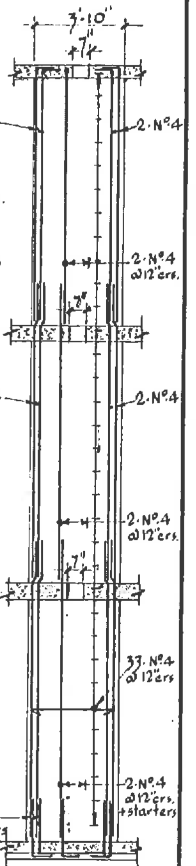
B-B 6" wall
¼" = 1ft.



C-C 6" wall
¼" = 1ft.



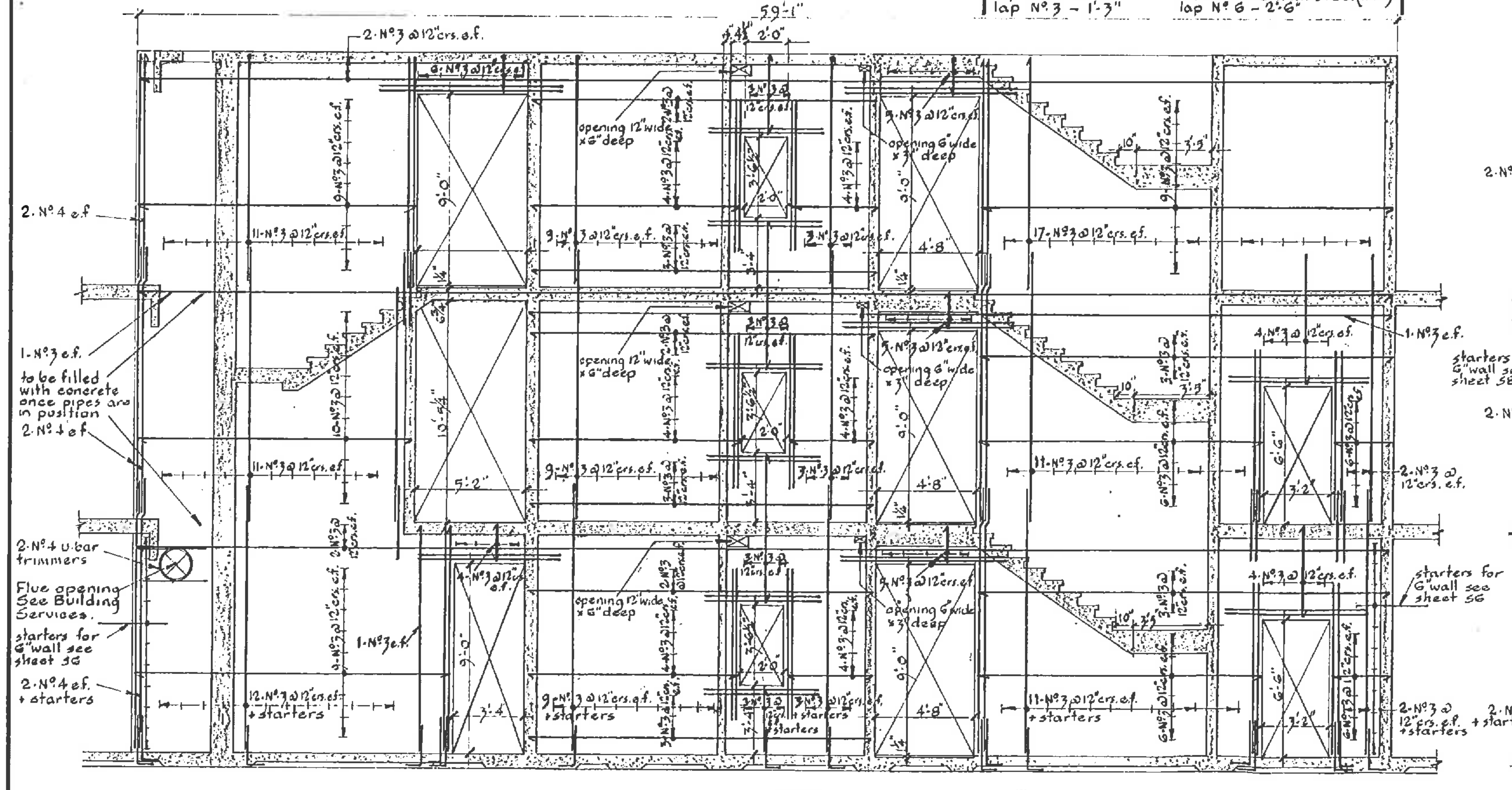
D·D 6" wall
D1·D1 4" wall with return
¼" = 1ft.



E·E 5" wall
¼" = 1ft.

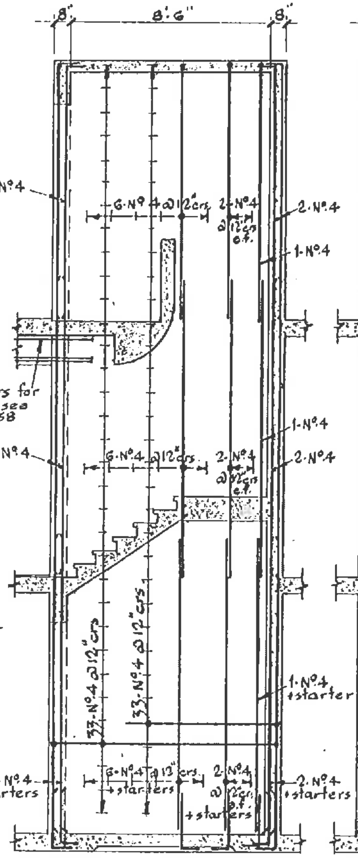
NOTE:
FOR POSITION OF PLATES TO TAKE ROOF TRUSSES SEE SHEET 519

TRIMMERS
8" walls – 2·N°4 each face – 3" centres
6", 5", 4" walls – 2·N°4 – 3" centres
lap N°4 – 1'-8"
lap N°3 – 1'-3"
1" cover to wall steel (min)
lap N°6 – 2'-6"

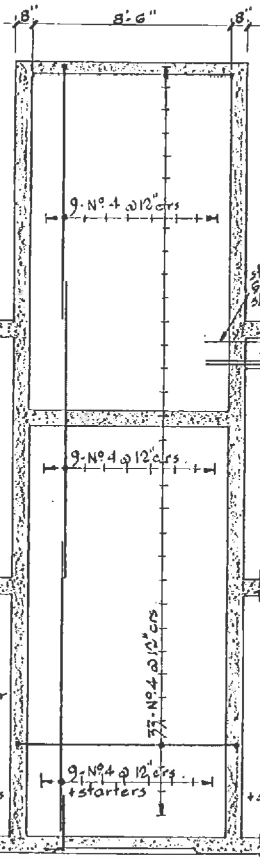


NOTE:
WALL STEEL RANDOM LENGTH LAP – MIN. 15'·0"

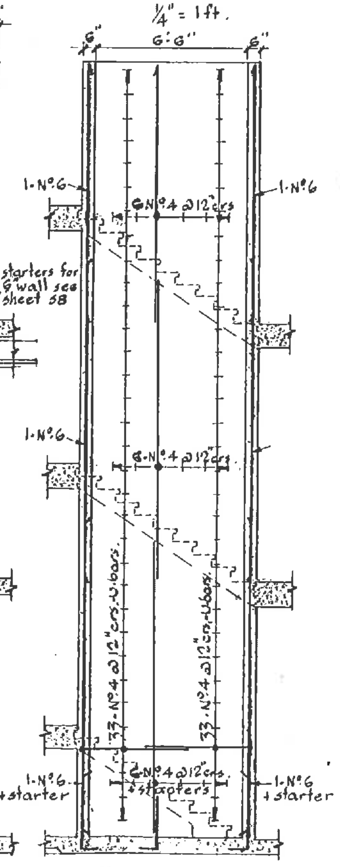
F·F 8" wall
¼" = 1ft.



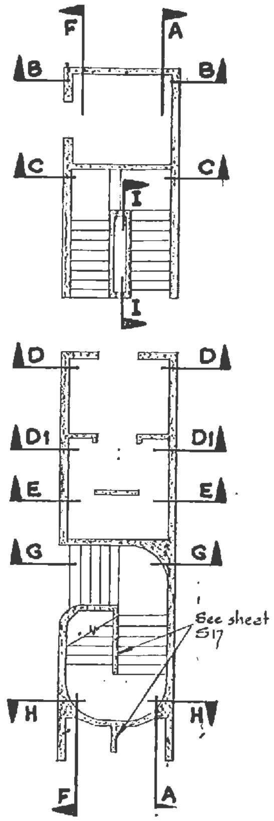
G·G 6" wall
¼" = 1ft.



H·H 6" wall
¼" = 1ft.



I·I 4" wall
¼" = 1ft.



KEY PLAN

DATE	REVISION

DRAWN	J. Burton	DATE	May 24 '73
CHECKED	CMJ	SCALE	¼" = 1ft.
APPROVED	CMJ	JOB N°	736

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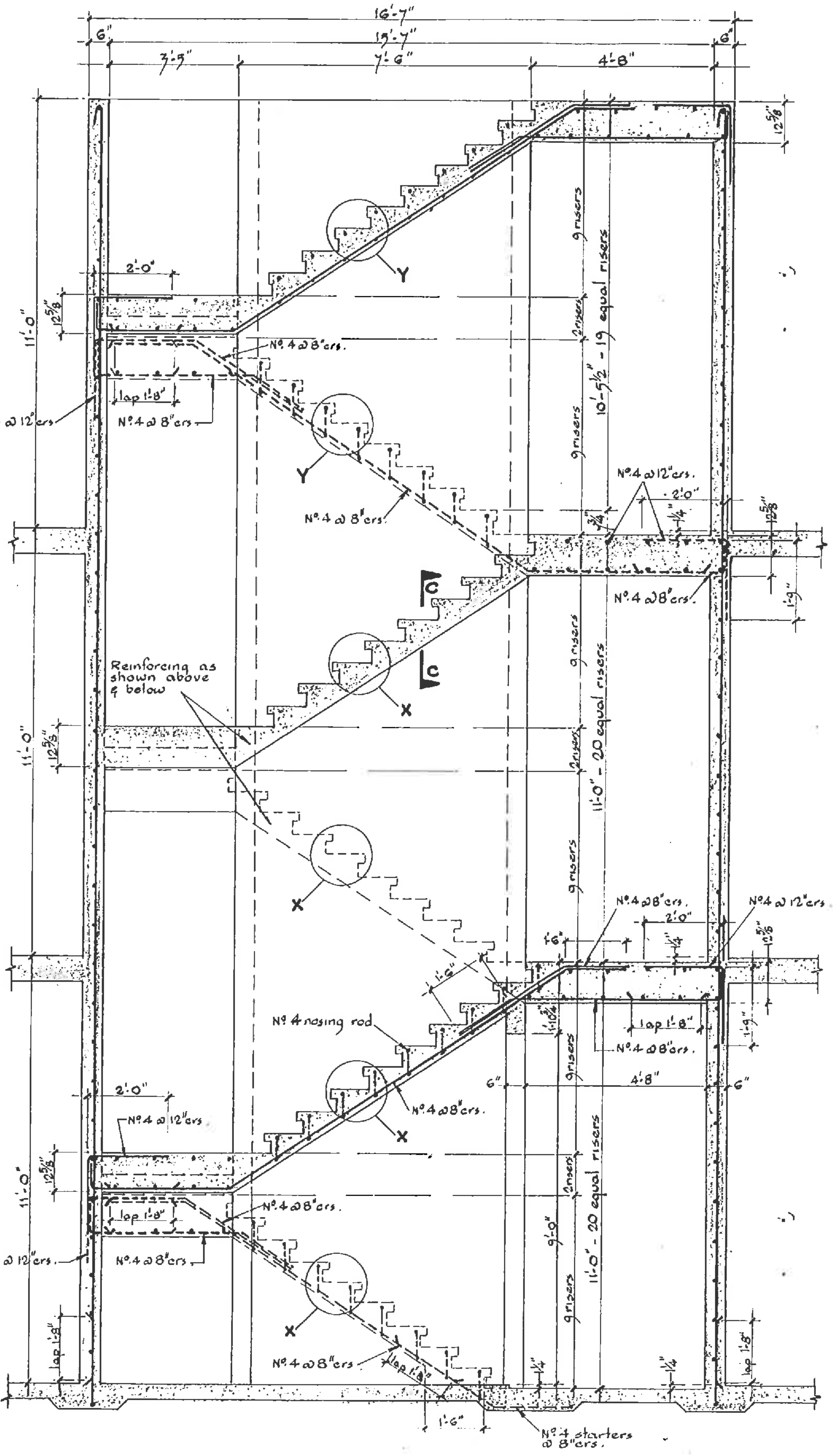
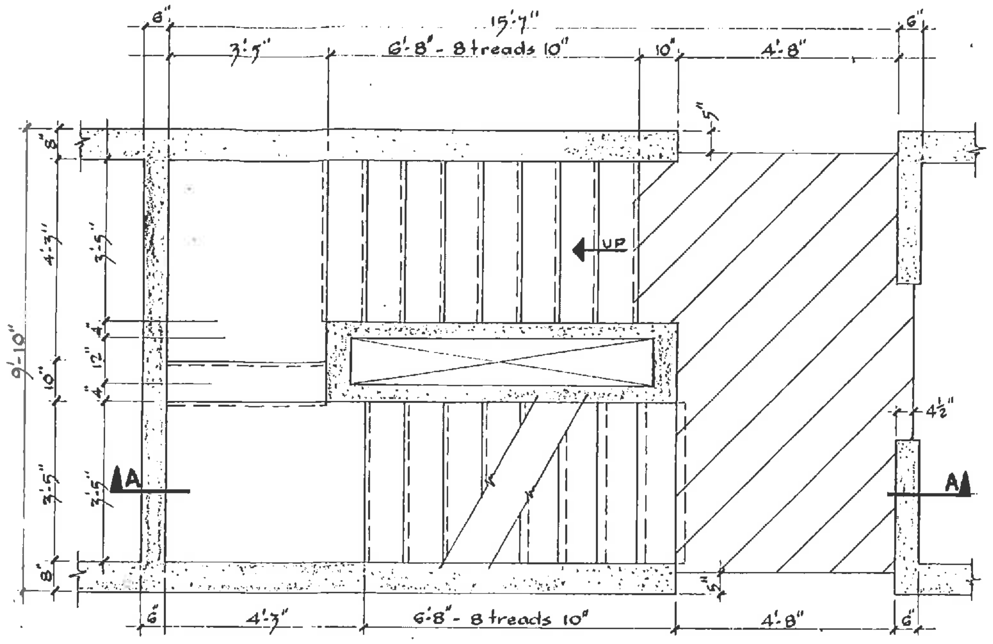
WAIPA COUNTY COUNCIL OFFICE
MAIN · BLOCK
CORE WALL ELEVATIONS

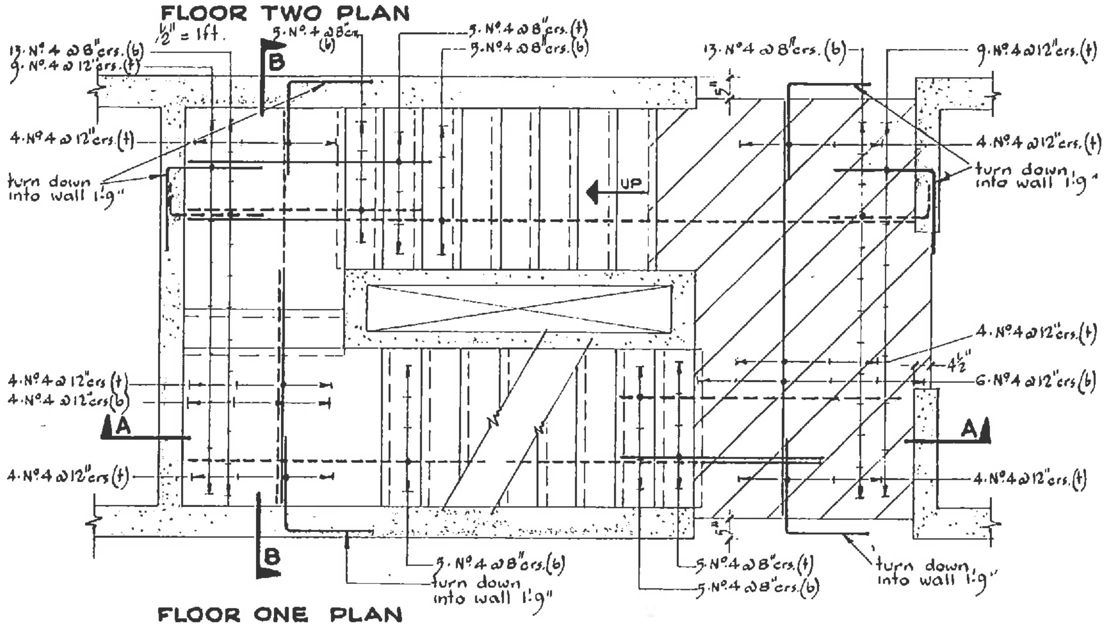
DRG. N°
450/515



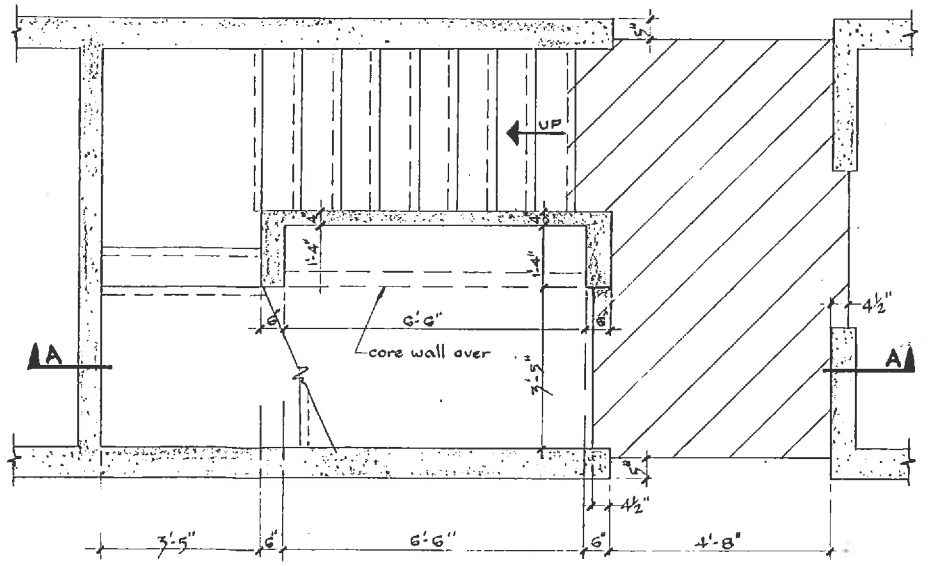
SECTION A·A
½" = 1ft.



FLOOR TWO PLAN
½" = 1ft.

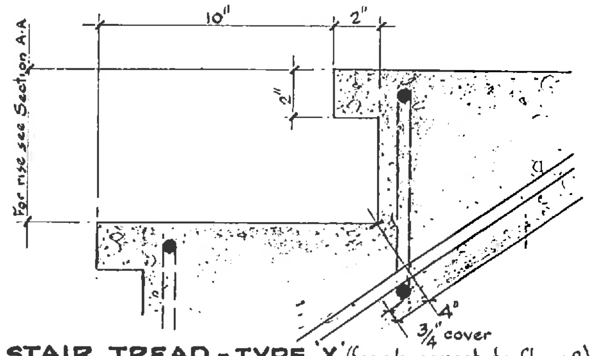


FLOOR ONE PLAN
½" = 1ft.

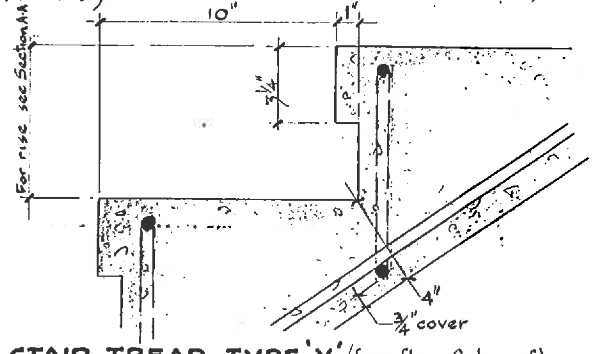


BASEMENT PLAN
½" = 1ft.

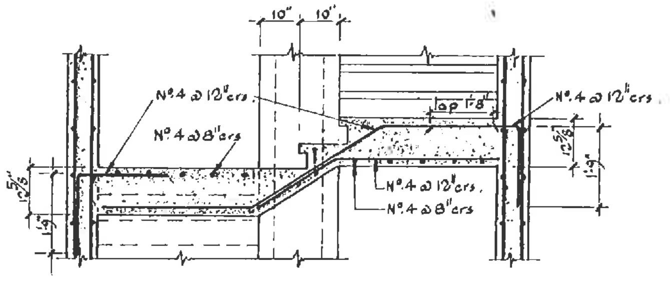
▨ shaded area depressed ¼"



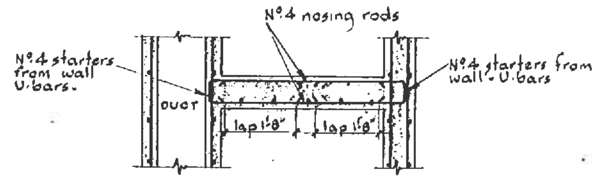
STAIR TREAD - TYPE 'X' (from basement to floor 2)
¼ full size



STAIR TREAD - TYPE 'Y' (from floor 2 to roof)
¼ full size



SECTION B·B (basement & floor two similar)
½" = 1ft.



SECTION C·C
½" = 1ft.

¾" cover to landing & stair reinforcing
----- denotes bottom steel (in plan)

DATE	REVISION

DRAWN J. Burton DATE May 21 '73
CHECKED CMS SCALE As shown
APPROVED CMS JOB Nº 736

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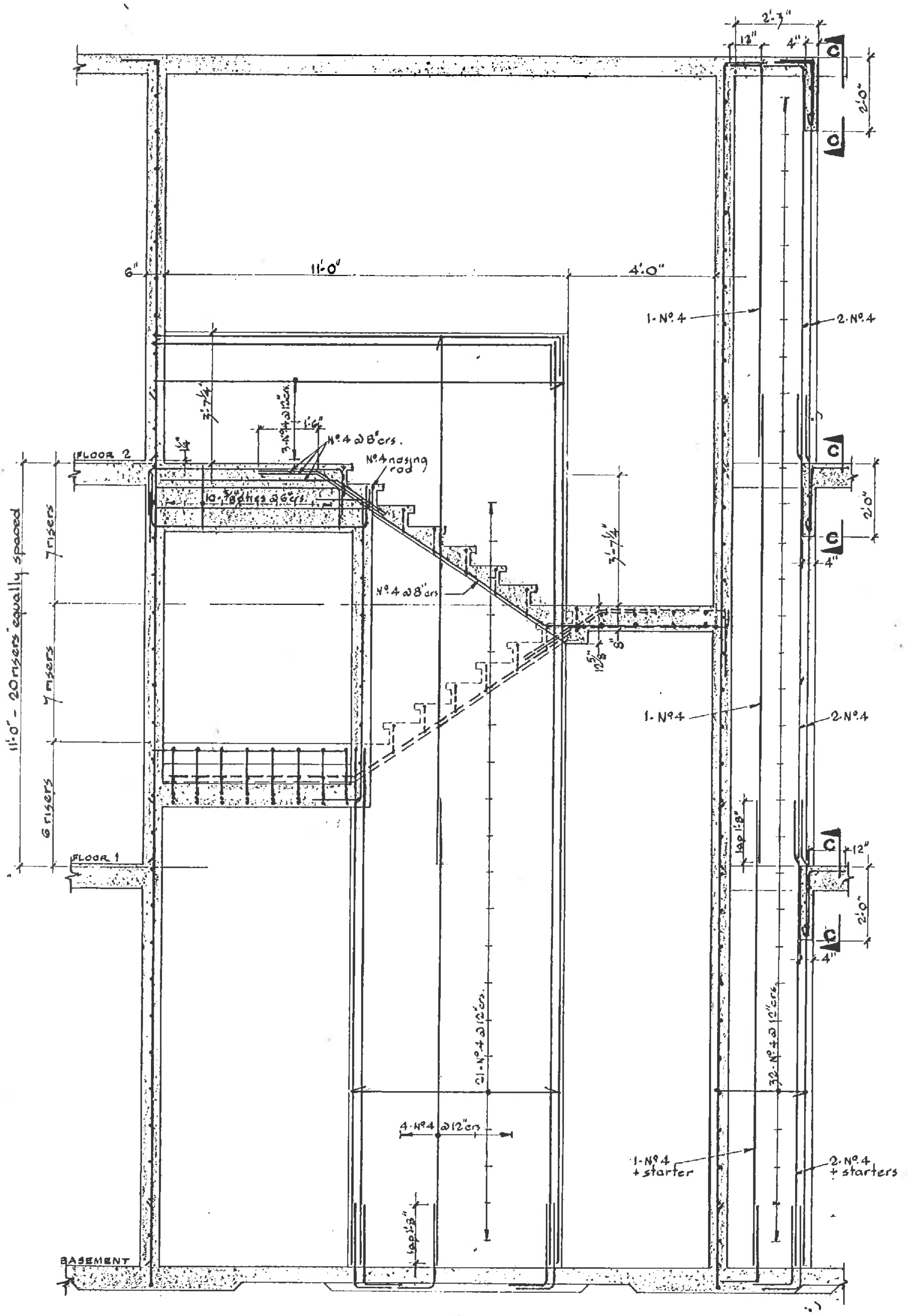
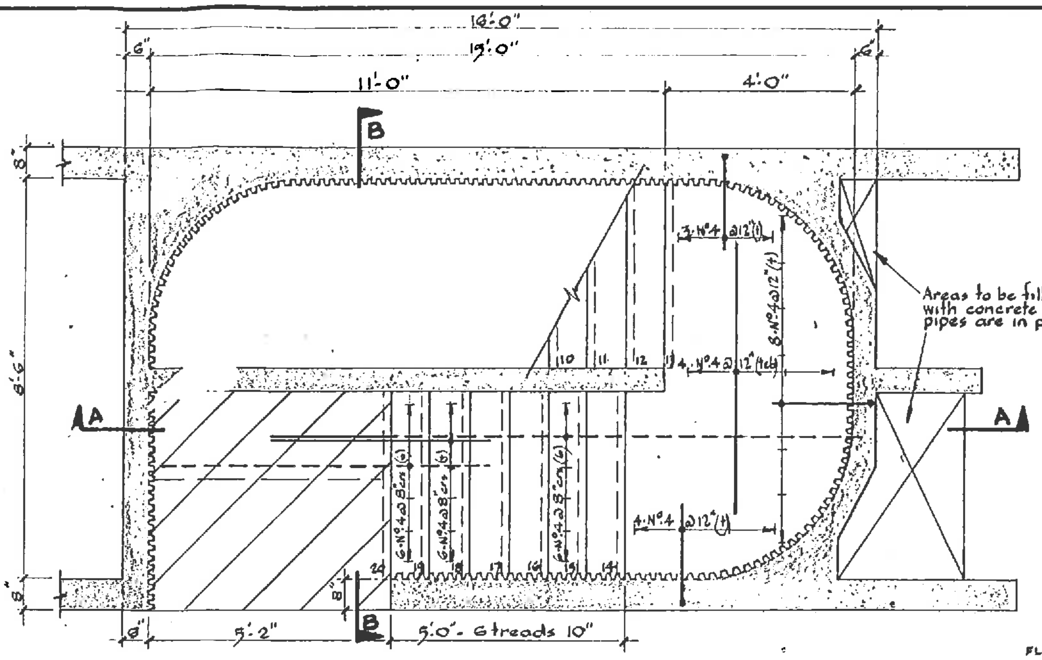
WAIPA COUNTY COUNCIL OFFICE
MAIN BLOCK
STAIR TWO

DRG. Nº 450/S16



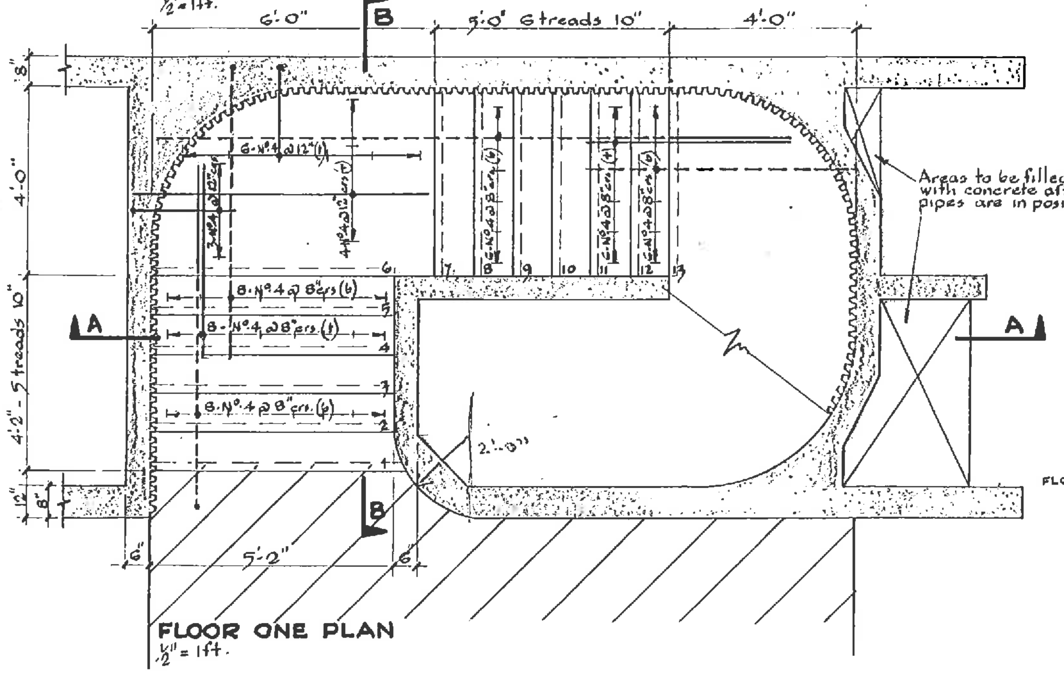
SECTION A·A

½" = 1ft.



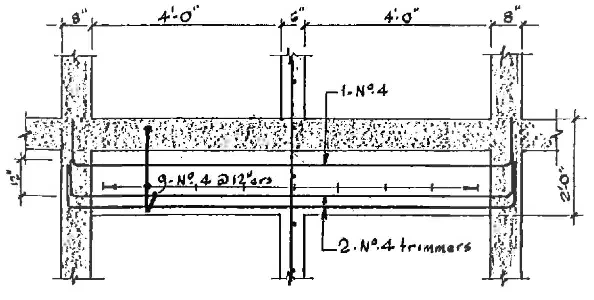
FLOOR TWO PLAN

½" = 1ft.



FLOOR ONE PLAN

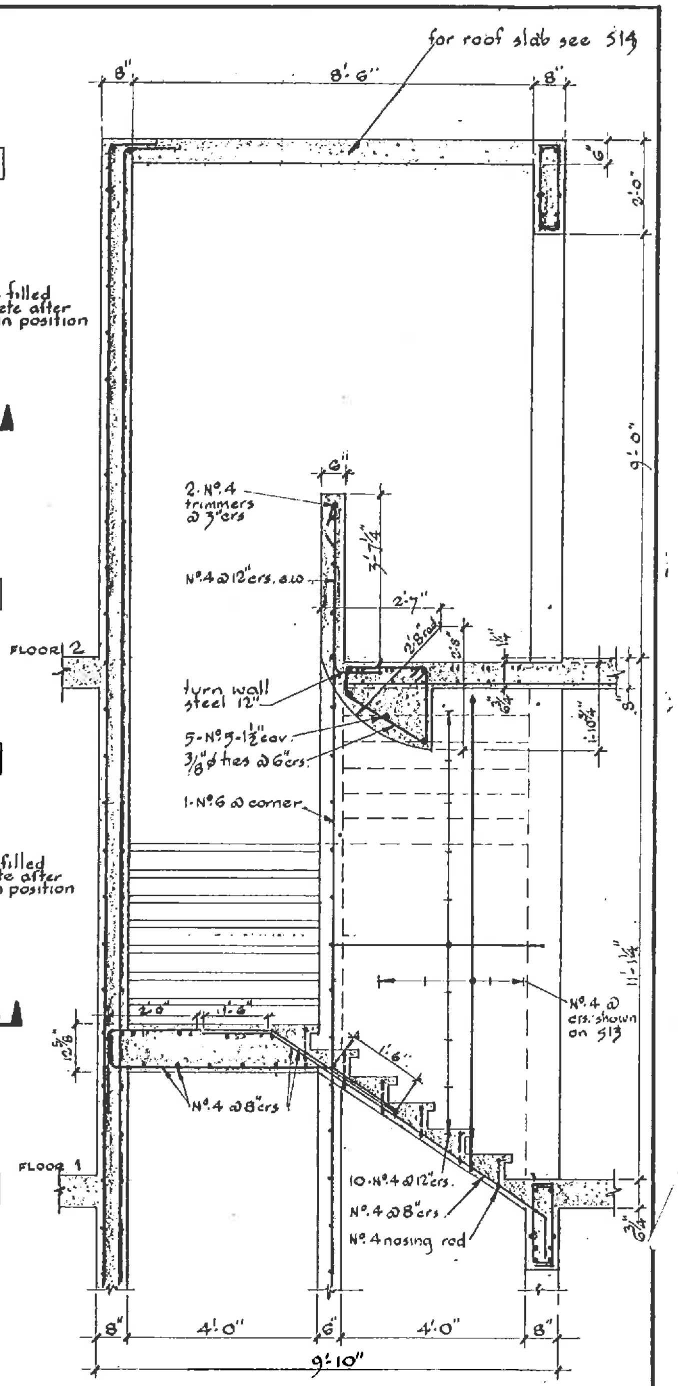
½" = 1ft.



SECTION C·C

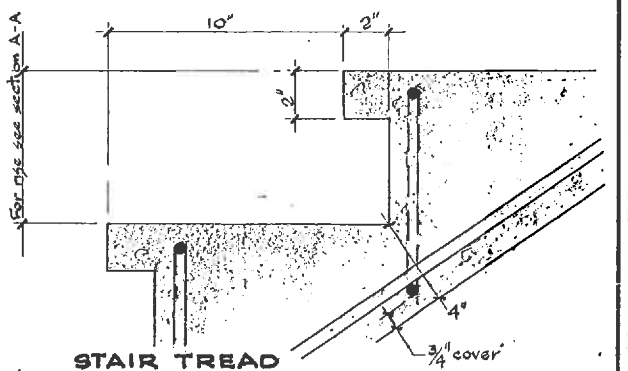
½" = 1ft.

¾" cover to landing & stair reinforcement
– – – – denotes bottom steel in plan.



SECTION B·B

½" = 1ft.



STAIR TREAD

¼ full size

▨ shaded area depressed 1¼"

DATE	REVISION	DRAWN	J. Burton	DATE	May 23 '73	THIS DRAWING IS COPYRIGHT AND MAY NOT BE REPRODUCED WITHOUT PERMISSION	
		CHECKED		SCALE	As shown		
		APPROVED		JOB Nº	736	THE CONTRACTOR MUST CHECK ALL DIMENSIONS ON SITE	

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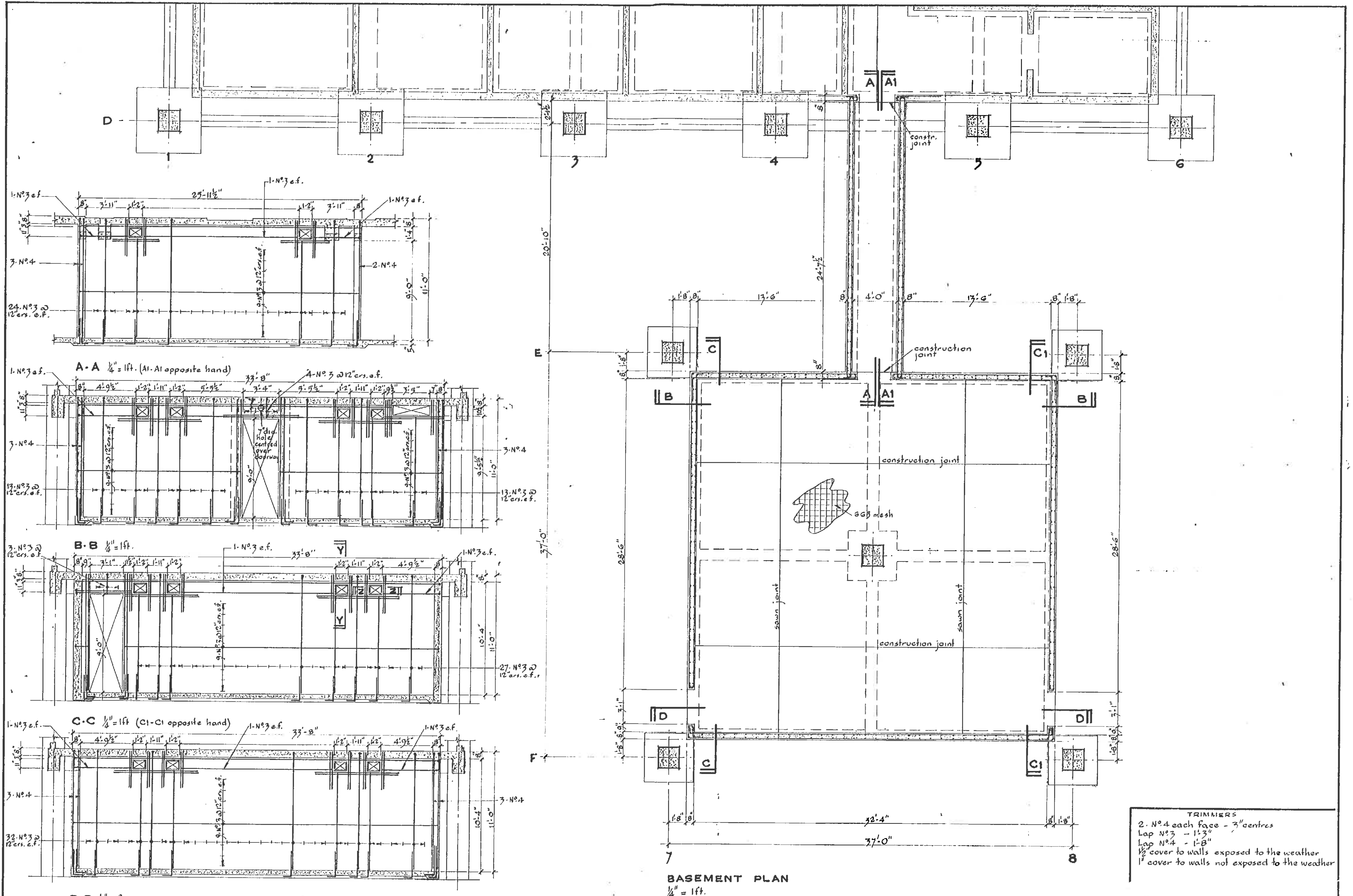
WAIPA COUNTY COUNCIL OFFICE
MAIN BLOCK
STAIR ONE

DRG. Nº
450/S17



A·A ¼" = 1ft. (A1-A1 opposite hand)

B·B ¼" = 1ft.

C·C ¼" = 1ft (C1-C1 opposite hand)

D·D ¼" = 1ft.

BASEMENT PLAN
¼" = 1ft.

TRIMMERS
2. N°4 each face - 3" centres
Lap N°3 - 1'-3"
Lap N°4 - 1'-8"
1½" cover to walls exposed to the weather
1" cover to walls not exposed to the weather

DATE	REVISION

DRAWN J. Burton DATE May '73
CHECKED SCALE ¼" = 1ft.
APPROVED JOB No 736

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WAIPA COUNTY COUNCIL OFFICE
COUNCIL CHAMBER
BASEMENT PLAN & WALL ELEVATIONS

DRG. No
450 / S5

Appendix B – Initial Evaluation Procedure

EARTHQUAKE PRONE ASSESSMENT

PROJECT NAME:	Earthquake Prone Assessment for Waipa District Council 101 Bank Street Te Awamutu
PREPARED FOR:	Waipa District Council Attn: Leonie Spalding

Executive Summary

BCD Group has undertaken a structural assessment in respect to the earthquake proneness of an existing building located at 101 Bank Street, Te Awamutu.

The result of the structural assessment categorises the building as Potentially Earthquake Prone with a Seismic Grade D to 26% of New Building Standard (NBS) and therefore under the NZ Building Act further action is required.

Figure 1 View of building from Roache Street

Introduction

BCD Group has undertaken a structural assessment of an existing Waipa District Council building located at 101 Bank Street, Te Awamutu. This assessment is in accordance with the NZSEE Study Group Recommendations and follows the initial assessment procedures outlined in the Study Group Draft, October 2005. The recommendations contained within the NZSEE Draft have been adopted nationwide as best practice for completing this task.

Investigation

We visited site on the 6th of September 2013 to visually inspect the layout and structure of the existing building. No opening up work was undertaken as part of the visual inspection.

It was verbally noted by our client that the building was initially constructed in 1973 and drawings supplied by the client were dated 1973 and therefore back up this assumption. Major alteration and additions have taken place to the building since its completion.

In the early 1990s a section of the basement which was an open space for car parking was filled in and turned into additional office space. The altered area is noticeable as the infill section is constructed out of block masonry and this is the only building elements which used masonry block.

In the late 1990’s the chambers were extended over a basement and ground floor section. This addition appears to have included additional stair wells and lifts which service all 3 levels. No seismic joints were apparent in the additions and therefore it is assumed the structure is to now work as one element.

Calculations prepared by Jones Grey Partnership Consulting Engineer (JGPCE) have been viewed. Although the extended chamber section has been designed against the revised building code, the design engineer does not complete a design review of the whole building against the revised building code (NSZ 4203:1992). In the body of the calculations we noted the following extract which explained the lateral resisting assumptions when taking into account the new structure.

Roof level over new Chamber comments from 1997 Calculations

Extract from 1997 Calculations: “Lateral forces at roof level will be distributed to the 1st floor by 4No. concrete columns in bending. This is appropriate for both directions and design will take account of this”

The additional chamber roof level has been completely connected to the existing building, and JGPCE have introduced additional bracing elements to transfer this section of roof down to the 1st floor. Therefore the existing 1st floor has not been reviewed at the time of this extension.

1st floor supporting new Chamber floor from 1997 Calculations

Extract from 1997 Calculations: “Lateral forces at 1st floor level will be distributed to foundation level by existing concrete walls adjacent to the new building and the new block walls in the service location. The flat slabs will act as a rigid diaphragm to distribute these forces to those locations. Consider these forces more closely.

Firstly consider forces at the 1st floor level in the NW-SE direction. Lateral forces will be taken by 200 thick concrete walls on lines G, E and D. By inspection, this would appear to be sufficient.

Likewise in the SW-NE direction loads taken by 200 thick concrete wall on lines 1, 3 and 7 and new walls in the main block in rooms 1.15, 1.16, 1.17, 1.18 and 1.19”

As a result of the above comments along with our review of the calculations prepared by JGPCE we believe that the existing structure was not re-checked against NZS4203:1992 while designing the chamber section and therefore until a full structural assessment of the entire structure is completed we must review the IEP for the entire building using the original design age of 1973.

Building Form

The main structure that forms the overall building development consists of a suspended concrete ground and first floor and a structural steel truss roof structure.

Lateral stability is provided by a centrally located concrete shear wall core. The Shear core consists of a number of internal singular reinforced 6” and 8” thick reinforced walls. The section from Basement to Ground has a large number of shear walls, however once above ground the structural system relies on the main shear core and external concrete frames.

Drawings viewed suggest that the beam column joints have not been detailed to absorb induced ductile forces, however we expect that they would assist to some level during a seismic event.

Reinforcing used in the construction appears to have been HY60 grade bars. We note that historically this reinforcing does not perform well under strain elongation and therefore it is assumed this building needs to remain nominally elastic (μ=1.25) in order to avoid diaphragm and beams ductile yielding failure.

The current building is designated as a civil defence facility. Therefore the earthquake importance level is increased to level 4. As result the outcome strength of your IEP was reduced by a 0.6 multiplier when compared with “typical” 2-3 level building in the town centre.

Should this building no longer be required as a civil defence facility then the base strength result would be increased by a 1.67 multiplier.

We were unable to view the calculations or drawings for the chamber extension; there we cannot confirm the earthquake level of the entire building was reviewed for at such time. We noted no seismic joints in the Chamber extensions and therefore the entire building should have been reassessed against the New Zealand loading code NZS4203:1992. Should the building have been fully rechecked in the late 1990’s then this could give grounds to alter this report. However as this information was not available we have adopted an F factor equal to 1.0 for the purpose of this investigation.

NZSEE Initial Evaluation

The Initial Evaluation Procedure (IEP) calculations for this building are attached in Appendix A

The recommendations contained within the NZSEE Draft have been adopted by local councils for assessing building stock for earthquake strength levels. This assessment procedure grades the building according to several criteria and compares the result against the NBS.

The calculation has revealed this building as Potentially Earthquake Prone with a Seismic Grade D to 26% of New Building Standard (NBS).

Should the building classification no longer be required as a civil defence facility then it is rated as Seismic Grade C to 43%NBS.

Key Definitions

The definition of an earthquake prone building is set out in section 122 of the Building Act 2004, and in the related Building Regulation SR2005/32 that defines a “moderate earthquake”.

A moderate earthquake, in relation to section 122 of the Building Act can be defined as:

"an earthquake that would generate shaking at the site of a building that is of the same duration as, but that is one third as strong as, the earthquake shaking that would be used to design a new building at that site"

A building is earthquake prone if, having regard to its condition and to the ground on which it is built, and because of its construction, the building:

- Will have its ultimate capacity exceeded in a moderate earthquake
- Would be likely to collapse causing:
    - Injury or death to persons in the building or to persons on any other property; or
    - Damage to any other property

In general terms a building risk classification against NBS can be summarized as follows:

Table 1: Building Rick Classification

Description	Classification	Risk	% NBS	Building Act 2004
Low Risk Building	-	Low	>67%	Acceptable
Moderate Risk Building	Earthquake Risk Building	Moderate	34% - 66%	Legally Acceptable, improvements recommended
High Risk Building	Earthquake Prone Building	High	<33%	Unacceptable, improvements required under the Building Act

Detailed Engineering Evaluation

Given the form of this building, we believe that specific calculations and modeling of the existing structure against the most recent design standards (AS/NZS 1170) will likely illustrate improved new building earthquake strength, however to undertake this we would look to undertake a full detailed engineering evaluation. This would include the confirmation of concrete reinforcing content, concrete strength and other miscellaneous items. This data would then be compared with the existing documentation to allow a full desk top analysis to take place.

Although the legal minimum strength is 34% NBS we would recommend you consider extending the investigation so you can obtain the minimum level of 67% NBS.

Using analysis software SAP2000, we would create a 3D model your building, inputting site established information and look to identify specific areas that might fail. Therefore we strengthen only those areas identified. In some cases it has also illustrated an improved rating which leads to no further work being required.

The identification of weak elements susceptible to damage ensures that any strengthening measures are targeted to provide the maximum benefits, while significantly reducing the construction cost. This will prove to be the most cost effective way of re-strengthening your building.

Fee to undertake this process is likely to be around $30,000 + GST plus disbursements. However in our experience traditional strengthening without the use of a non-linear model greatly increases the construction cost. Therefore our fee pays for itself as the reduced construction costs will far exceed the cost of the analysis.

Conclusions

This building is classified as earthquake prone, and under the Building Act further action is required.

We recommend 3D non-linear modelling and assessment of your building using structural modelling software such as SAP2000 or ETABS.

Findings presented as a part of this project are for the sole use of Waipa District Council. The findings are not intended for use by other parties.

Regards,

Blair Currie
DIRECTOR

Appendix A – IEP

Table IEP-1: Initial Evaluation Procedure – Step 1

Table IEP-1 **Initial Evaluation Procedure Step 1** **Page 1....**

(Refer Table IEP - 2 for Step 2; Table IEP - 3 for Step 3, Table IEP - 4 for Steps 4, 5 and 6)

Building Name Waipa DC.
Location 101 Bank Street
Te Awamutu.

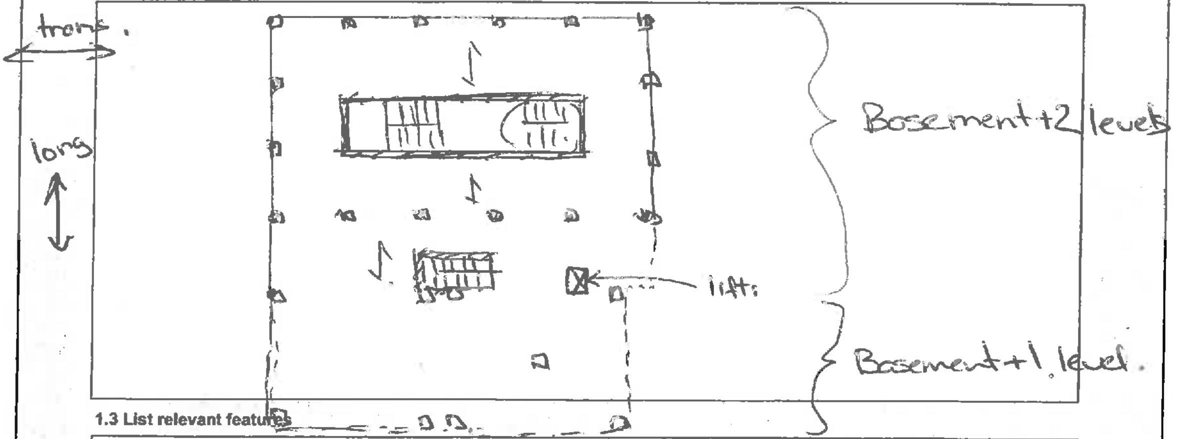
Ref. 13-257
By BSC
Date 6/9/13.

Step 1 - General Information

1.1 Photos (attach sufficient to describe building)

See attached Report.

1.2 Sketch of building plan



1.3 List relevant features

- Concrete Frames. + Shear wall cores.
- Insitu floor slabs (heavy).
- Steel Truss roof
- Civil defence centre.

1.4 Note information sources

tick as appropriate

Visual Inspection of Exterior ☑
Visual Inspection of Interior ☑
Drawings (note type) ☑
Specifications ☐
Geotechical Reports ☐
Other (list) ☐

Limited Structural drawing of 1973 none of the alterations were wiewed.

- Built over multiple Stages. Basement extended early 1990's. which was on infill of space previously built over.
- Chambers were extend late 1990's, no obvious signs of seismic joints ∴ engineers assumed to have accounted for addition impacts on existing building with extension.

Table IEP-2: Initial Evaluation Procedure – Step 2

Table IEP-2 **Initial Evaluation Procedure Step 2** **Page 2....**

(Refer Table IEP - 1 for Step 1; Table IEP - 3 for Step 3; Table IEP - 4 for Steps 4, 5 and 6)

Building Name Waipa DC **Ref.** 13-257
Location 101 Bank Street **By** BJC
Direction Considered: a) Longitudinal [✓ b) Transverse]
(Choose worse case if clear at start. Complete IEP-2 and IEP-3 for each if in doubt) **Date** 6/9/13

Step 2 - Determination of $(\%NBS)_b$

2.1 Determine nominal $(\%NBS) = (\%NBS)_{nom}$

a) Date of Design and Seismic Zone

tick as appropriate

- **Pre 1935** ☐ See also notes 1, 3
- **1935-1965** ☐
- **1965-1976** **Seismic Zone; A** ☐
  - **B** ☑
  - **C** ☐
- **1976-1992** **Seismic Zone; A** ☐ See also note 2
  - **B** ☐
  - **C** ☐
- **1992-2004** ☐

b) Soil Type

From NZS1170.5:2004, Cl 3.1.3

- **A or B Rock** ☐
- **C Shallow Soil** ☐
- **D Soft Soil** ☑
- **E Very Soft Soil** ☐

From NZS4203:1992, Cl 4.6.2.2
(for 1992 to 2004 only and only if known)

- **a) Rigid** ☐
- **b) Intermediate** ☐

c) Estimate Period, *T* 0.4 Seconds

Can use following:

$T = 0.09h_n^{0.75}$ for moment-resisting concrete frames
$T = 0.14h_n^{0.75}$ for moment-resisting steel frames
$T = 0.08h_n^{0.75}$ for eccentrically braced steel frames
$T = 0.06h_n^{0.75}$ for all other frame structures
$T = 0.09h_n^{0.75}/A_c^{0.5}$ for concrete shear walls
$T \leq 0.4$sec for masonry shear walls

Where h_n = height in m from the base of the structure to the uppermost seismic weight or mass.
$A_c = \Sigma A_i(0.2 + L_{wi}/h_n)^2$
A_i = cross-sectional shear area of shear wall i in the first storey of the building, in m^2
l_{wi} = length of shear wall i in the first storey in the direction parallel to the applied forces, in m
with the restriction that l_{wi}/h_n shall not exceed 0.9

d) $(\%NBS)_{nom}$ determined from Figure 3.3 5 $(\%NBS)_{nom}$

Note 1: For buildings designed prior to 1965 and known to be designed as public buildings in accordance with the code of the time, multipy $(\%NBS)_{nom}$ by 1.25.
For buildings designed 1965 - 1976 and known to be designed as public buildings in accordance with the code of the time, multiply $(\%NBS)_{nom}$ by 1.33 - Zone A
1.2 - Zone B 1.2

1

Note 2: For reinforced concrete buildings designed between 1976-84 multiply $(\%NBS)_{nom}$ by 1.2 1

Note 3: For buildings designed prior to 1935 multiply $(\%NBS)_{nom}$ by 0.8 except for Wellington where the factor may be taken as 1. 1 6 $(\%NBS)_{nom}$

Continued over page

Table IEP-2:Initial Evaluation Procedure – Step 2

Table IEP-2 **Initial Evaluation Procedure Step 2** **Page 2....**

(Refer Table IEP - 1 for Step 1; Table IEP - 3 for Step 3; Table IEP - 4 for Steps 4, 5 and 6)

Building Name Waipa DC.
Location 101 Bank St, Te Awamutu.
Direction Considered: [✓ **a) Longitudinal**] **b) Transverse**
(Choose worse case if clear at start. Complete IEP-2 and IEP-3 for each if in doubt)
Ref. 13-257
By BJC
Date 6/9/13

Step 2 - Determination of $(\%NBS)_b$

2.1 Determine nominal $(\%NBS) = (\%NBS)_{nom}$

a) Date of Design and Seismic Zone

tick as appropriate

Pre 1935		☐	See also notes 1, 3
1935-1965		☐	
1965-1976	**Seismic Zone; A**	☐	
	B	☑	
	C	☐	
1976-1992	**Seismic Zone; A**	☐	See also note 2
	B	☐	
	C	☐	
1992-2004		☐	

b) Soil Type

From NZS1170.5:2004, Cl 3.1.3	**A or B Rock**	☐
	C Shallow Soil	☐
	D Soft Soil	☑
	E Very Soft Soil	☐
From NZS4203:1992, Cl 4.6.2.2	**a) Rigid**	☐
(for 1992 to 2004 only and only if known)	**b) Intermediate**	☐

c) Estimate Period, *T* 0.6 Seconds

Can use following:

$T = 0.09h_n^{0.75}$ for moment-resisting concrete frames
$T = 0.14h_n^{0.75}$ for moment-resisting steel frames
$T = 0.08h_n^{0.75}$ for eccentrically braced steel frames
$T = 0.06h_n^{0.75}$ for all other frame structures
$T = 0.09h_n^{0.75}/A_c^{0.5}$ for concrete shear walls
$T \leq 0.4\text{sec}$ for masonry shear walls

Where h_n = height in m from the base of the structure to the uppermost seismic weight or mass.
$A_c = \Sigma A_i(0.2 + L_{wi}/h_n)^2$
A_i = cross-sectional shear area of shear wall i in the first storey of the building, in m^2
l_{wi} = length of shear wall i in the first storey in the direction parallel to the applied forces, in m
with the restriction that l_{wi}/h_n shall not exceed 0.9

d) $(\%NBS)_{nom}$ determined from Figure 3.3 5 $(\%NBS)_{nom}$

Note 1: For buildings designed prior to 1965 and known to be designed as public buildings in accordance with the code of the time, multipy $(\%NBS)_{nom}$ by 1.25.
For buildings designed 1965 - 1976 and known to be designed as public buildings in accordance with the code of the time, multiply $(\%NBS)_{nom}$ by 1.33 - Zone A
1.2 - Zone B

1.2

Note 2: For reinforced concrete buildings designed between 1976-84 multiply $(\%NBS)_{nom}$ by 1.2

1

Note 3: For buildings designed prior to 1935 multiply $(\%NBS)_{nom}$ by 0.8 except for Wellington where the factor may be taken as 1.

1

6 $(\%NBS)_{nom}$

Continued over page

1

Table IEP-2: Initial Evaluation Procedure – Step 2 continued

Table IEP-2 Initial Evaluation Procedure Step 2 continued **Page 3....**

2.2 Near Fault Scaling Factor, Factor A

If $T \leq 1.5$sec, Factor A = 1

a) Near Fault Factor, *N(T,D)* 1
(from NZS1170.5:2004, Cl 3.1.6)

b) Near Fault Scaling Factor = 1/N(T,D) **Factor A** 1

2.3 Hazard Scaling Factor, Factor B

a) Hazard Factor, *Z, for site* 0.17
(from NZS1170.5:2004, Table 3.3)

b) Hazard Scaling Factor

For pre 1992 = $1/Z$
For 1992 onwards = Z_{1992}/Z

(Where Z_{1992} is the NZS4203:1992 Zone Factor from accompanying Figure 3.5(b)) **Factor B** 5.88

2.4 Return Period Scaling Factor, Factor C

a) Building Importance Level 4
(from NZS1170.0:2004, Table 3.1 and 3.2)

b) Return Period Scaling Factor from accompanying Table 3.1 **Factor C** 0.6

2.5 Ductility Scaling Factor, D

a) Assessed Ductility of Existing Structure, μ 1.25
(shall be less than maximum given in accompanying Table 3.2)

b) Ductility Scaling Factor

For pre 1976 = k_μ
For 1976 onwards = 1

(where k_μ is NZS1170.5:2004 Ductility Factor, from accompanying Table 3.3) **Factor D** 1.14 – T = 0.4 sec. – Trans.

1.21 – T = 0.6 – Long.

2.6 Structural Performance Scaling Factor, Factor E

a) Structural Performance Factor, S_p from accompanying Figure 3.4 0.925

b) Structural Performance Scaling Factor = $1/S_p$ **Factor E** 1.08

2.7 Baseline %NBS for Building, $(\%NBS)_b$ (equals $(\%NSB)_{nom}$ x A x B x C x D x E) 26 Trans 28 Long.

Table IEP-3: Initial evaluation procedure – Step 3

Table IEP-3 **Initial Evaluation Procedure Step 3** **Page ......**

(Refer Table IEP - 1 for Step 1; Table IEP - 2 for Step 2; Table IEP - 4 for Steps 4, 5 and 6)

Building Name Waipa DC. **Ref.** 13-257
Location 101 Bank St. **By** BJC
Direction Considered: **a) Longitudinal** Similar **b) Transverse**
(Choose worse case if clear at start. Complete IEP-2 and IEP-3 for each if in doubt) **Date** 6/9/13

Step 3 - Assessment of Performance Achievement Ratio (PAR)
(Refer Appendix B - Section B3.2)

Critical Structural Weakness		**Building Score**	**Effect on Structural Performance** *(Choose a value - Do not interpolate)*		
3.1 Plan Irregularity *Effect on Structural Performance*			Severe	Significant	Insignificant
	Factor A	1	0.4 max	0.7	1
Comment					
3.2 Vertical Irregularity *Effect on Structural Performance*			Severe	Significant	Insignificant
	Factor B	1	0.4 max	0.7	1
Comment					
3.3 Short Columns *Effect on Structural Performance*			Severe	Significant	Insignificant
	Factor C	1	0.4 max	0.7	1
Comment					

3.4 Pounding Potential
(Estimate D1 and D2 and set D = the lower of the two, or =1.0 if no potential for pounding)

a) Factor D1: - Pounding Effect
Select appropriate value from Table

Note:
Values given assume the building has a frame structure. For stiff buildings (eg with shear walls), the effect of pounding may be reduced by taking the co-efficient to the right of the value applicable to frame buildings.

Factor D1 1

Table for Selection of Factor D1	Severe	Significant	Insignificant
Separation	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Alignment of Floors within 20% of Storey Height	0.7	0.8	1
Alignment of Floors not within 20% of Storey Height	0.4	0.7	0.8

b) Factor D2: - Height Difference Effect
Select appropriate value from Table

Factor D2 1

Table for Selection of Factor D2	Severe	Significant	Insignificant
	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Height Difference > 4 Storeys	0.4	0.7	1
Height Difference 2 to 4 Storeys	0.7	0.9	1
Height Difference < 2 Storeys	1	1	1

Factor D 1 ***(Set D = lesser of D1 and D2 or.. set D = 1.0 if no prospect of pounding)***

3.5 Site Characteristics - *(Stability, landslide threat, liquefaction etc)* *Effect on Structural Performance*			Severe	Significant	Insignificant
	Factor E	1	0.5 max	0.7	1

3.6 Other Factors

Factor F 1 **For ≤ 3 storeys - Maximum value 2.5, otherwise - Maximum value 1.5. No minimum.**

Record rationale for choice of Factor F:

3.7 Performance Achievement Ratio (PAR)
(equals A x B x C x D x E x F) 1

Table IEP-4: Initial evaluation procedure – Steps 4, 5 and 6

Table IEP- 4 **Initial Evaluation Procedure Steps 4, 5 and 6** **Page ...**

(Refer Table IEP - 1 for Step 1; Table IEP - 2 for Step 2; Table IEP - 3 for Step 3)

Building Name Waipa DC Te Awamutu
Location 101 Bank St.
Ref. 13-257
By BSC
Date 6/9/13

Step 4 - Percentage of New Building Standard (*%NBS*)

	Longitudinal	Transverse
4.1 Assessed Baseline (*%NBS*)$_b$ (from Table IEP - 1)	28	26
4.2 Performance Achievement Ratio (PAR) (from Table IEP - 2)	1	1
4.3 PAR x Baseline (*%NBS*)b	28	26
4.4 Percentage New Building Standard (*%NBS*) (Use lower of two values from Step 3.3)		26

Step 5 - Potentially Earthquake Prone?
(Mark as appropriate)

%NBS > 33 NO
%NBS ≤ 33 [✓ YES]

Step 6 - Potentially Earthquake Risk?
(Mark as appropriate)

%NBS ≥ 67 NO
%NBS < 67 [✓ YES]

Step 7 - Provisional Grading for Seismic Risk based on IEP

Seismic Grade D

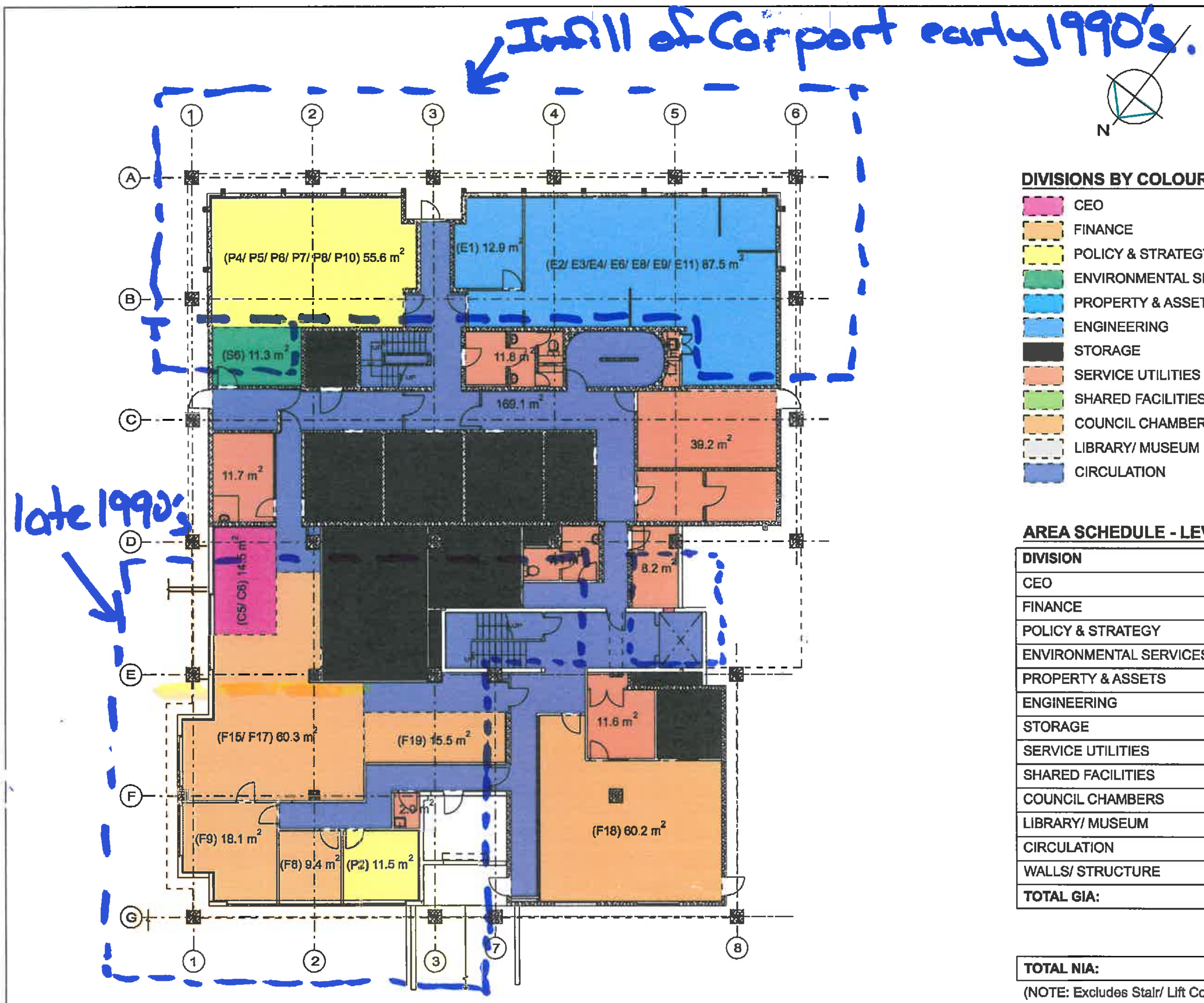
Evaluation Confirmed by... .... **Signature**

Blair Currie.................................... **Name**

230516.................................... **CPEng. No**

Relationship between Seismic Grade and *%NBS*:

Grade:	A+	A	B	C	D	E
%NBS:	> 100	100 to 80	80 to 67	67 to 33	33 to 20	< 20



DIVISIONS BY COLOUR KEY:

- 🟪 CEO
- 🟧 FINANCE
- 🟨 POLICY & STRATEGY
- 🟩 ENVIRONMENTAL SERVICES
- 🟦 PROPERTY & ASSETS
- 🟦 ENGINEERING
- ⬛ STORAGE
- 🟧 SERVICE UTILITIES
- 🟩 SHARED FACILITIES
- 🟧 COUNCIL CHAMBERS
- ⬜ LIBRARY/ MUSEUM
- 🟦 CIRCULATION

AREA SCHEDULE - LEVEL 1 (L1):

DIVISION	AREA
CEO	14.5 m²
FINANCE	163.5 m²
POLICY & STRATEGY	67.1 m²
ENVIRONMENTAL SERVICES	11.3 m²
PROPERTY & ASSETS	0.0 m²
ENGINEERING	100.4 m²
STORAGE	120.0 m²
SERVICE UTILITIES	98.1 m²
SHARED FACILITIES	0.0 m²
COUNCIL CHAMBERS	0.0 m²
LIBRARY/ MUSEUM	0.0 m²
CIRCULATION	169.1 m²
WALLS/ STRUCTURE	45.2 m²
TOTAL GIA:	**789.2 m²**

TOTAL NIA:	**653.8 m²**

(NOTE: Excludes Stair/ Lift Cores & Service Utility Areas)

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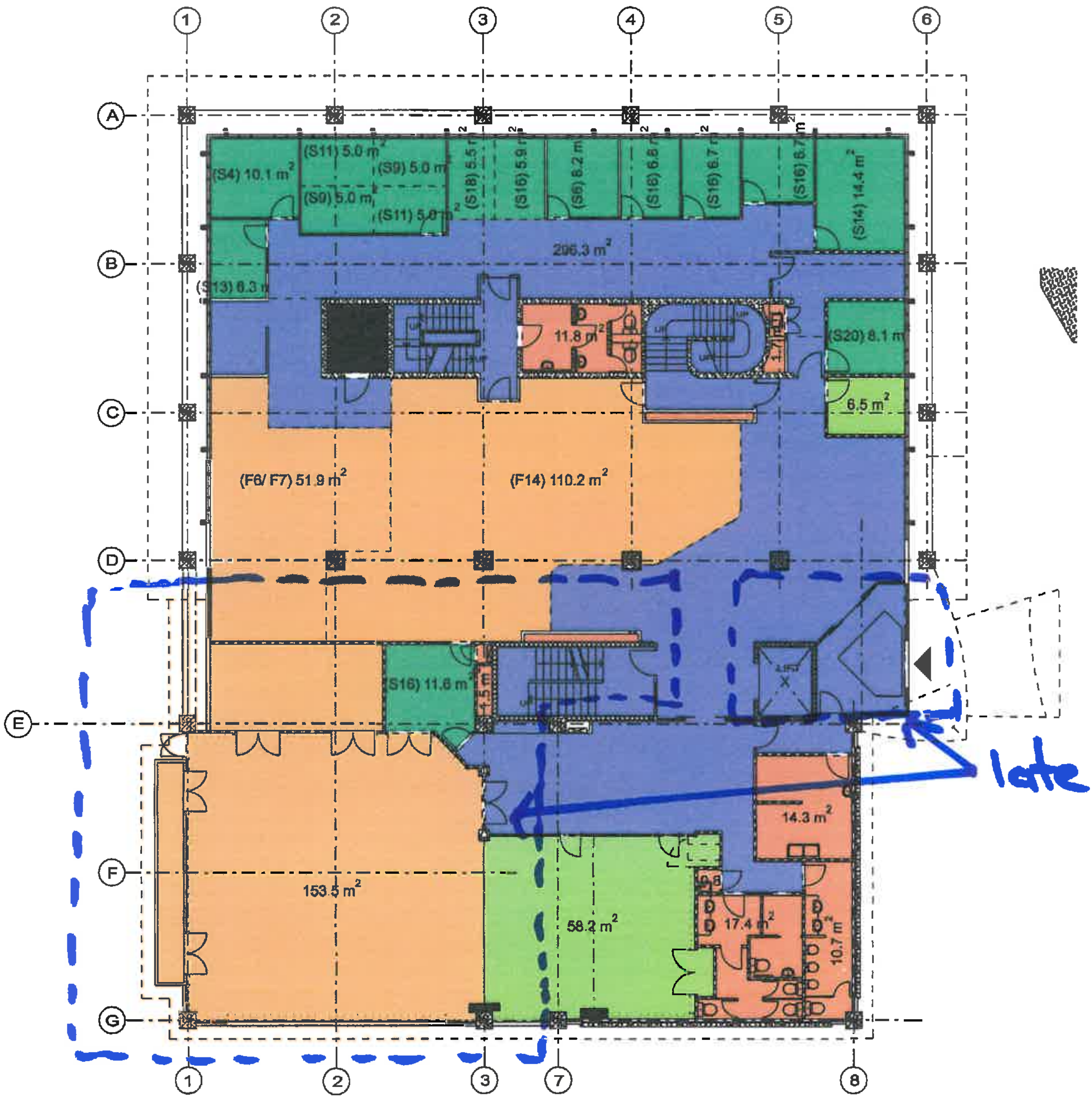

late 1990's

DIVISIONS BY COLOUR KEY:

- 🟪 CEO
- 🟧 FINANCE
- 🟨 POLICY & STRATEGY
- 🟩 ENVIRONMENTAL SERVICES
- 🟦 PROPERTY & ASSETS
- 🟦 ENGINEERING
- ⬛ STORAGE
- 🟧 SERVICE UTILITIES
- 🟩 SHARED FACILITIES
- 🟧 COUNCIL CHAMBERS
- ⬜ LIBRARY/ MUSEUM
- 🟦 CIRCULATION

AREA SCHEDULE - LEVEL 2 (L2):

DIVISION	AREA
CEO	0.0 m²
FINANCE	162.1 m²
POLICY & STRATEGY	0.0m²
ENVIRONMENTAL SERVICES	107.3 m²
PROPERTY & ASSETS	0.0 m²
ENGINEERING	0.0 m²
STORAGE	6.1 m²
SERVICE UTILITIES	60.4 m²
SHARED FACILITIES	64.7 m²
COUNCIL CHAMBERS	153.5 m²
LIBRARY/ MUSEUM	0.0 m²
CIRCULATION	296.4 m²
WALLS/ STRUCTURE	31.3 m²
TOTAL GIA:	**881.8 m²**

TOTAL NIA:	769.4 m²

(NOTE: Excludes Stair/ Lift Cores & Service Utility Areas)

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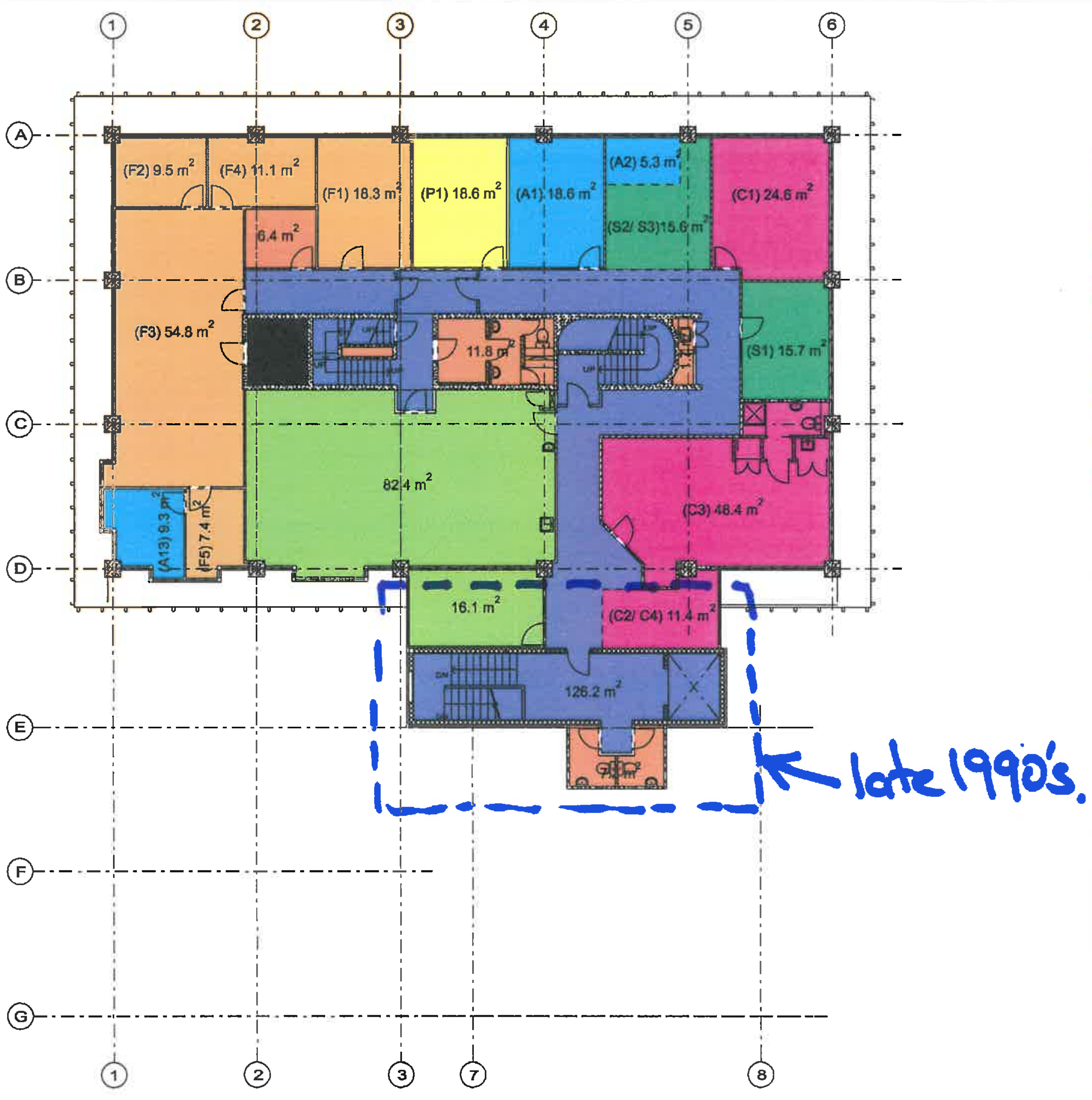

late 1990's.

DIVISIONS BY COLOUR KEY:

- ▨ CEO
- ▨ FINANCE
- ▨ POLICY & STRATEGY
- ▨ ENVIRONMENTAL SERVICES
- ▨ PROPERTY & ASSETS
- ▨ ENGINEERING
- ■ STORAGE
- ▨ SERVICE UTILITIES
- ▨ SHARED FACILITIES
- ▨ COUNCIL CHAMBERS
- ▨ LIBRARY/ MUSEUM
- ▨ CIRCULATION

AREA SCHEDULE - LEVEL 3 (L3):

DIVISION	AREA
CEO	74.4 m²
FINANCE	101.1 m²
POLICY & STRATEGY	18.6 m²
ENVIRONMENTAL SERVICES	31.3 m²
PROPERTY & ASSETS	33.2 m²
ENGINEERING	0.0 m²
STORAGE	6.1 m²
SERVICE UTILITIES	28.2 m²
SHARED FACILITIES	98.5 m²
COUNCIL CHAMBERS	0.0 m²
LIBRARY/ MUSEUM	0.0 m²
CIRCULATION	126.2 m²
WALLS/ STRUCTURE	31.2 m²
TOTAL GIA:	**548.8 m²**

TOTAL NIA:	**463.3 m²**

(NOTE: Excludes Stair/ Lift Cores & Service Utility Areas)

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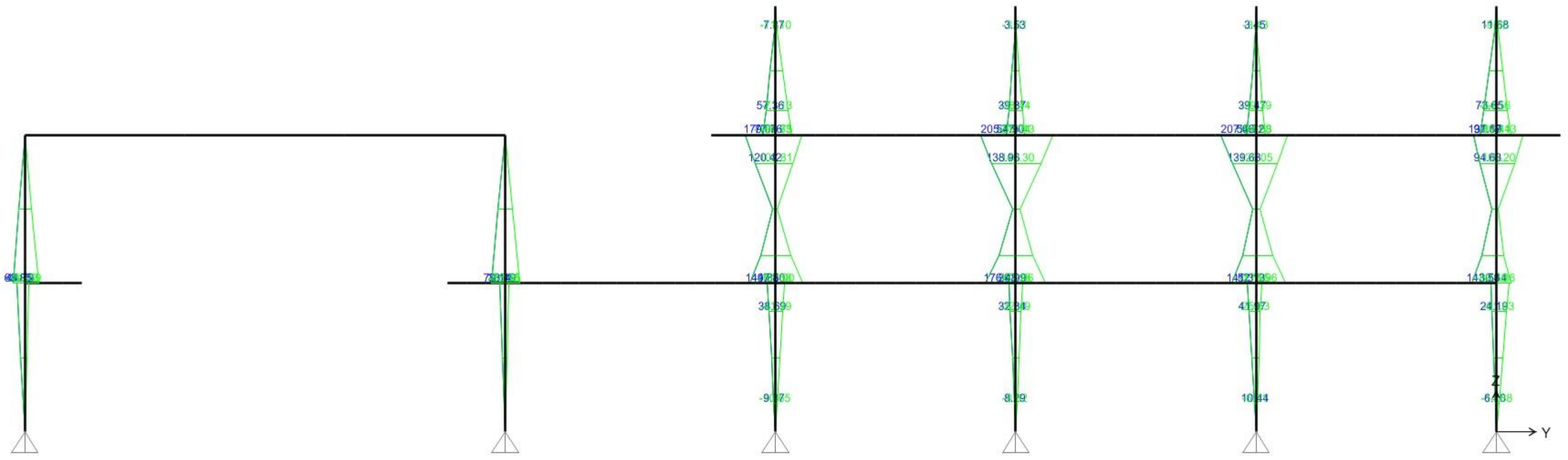
Appendix C – Additional Output



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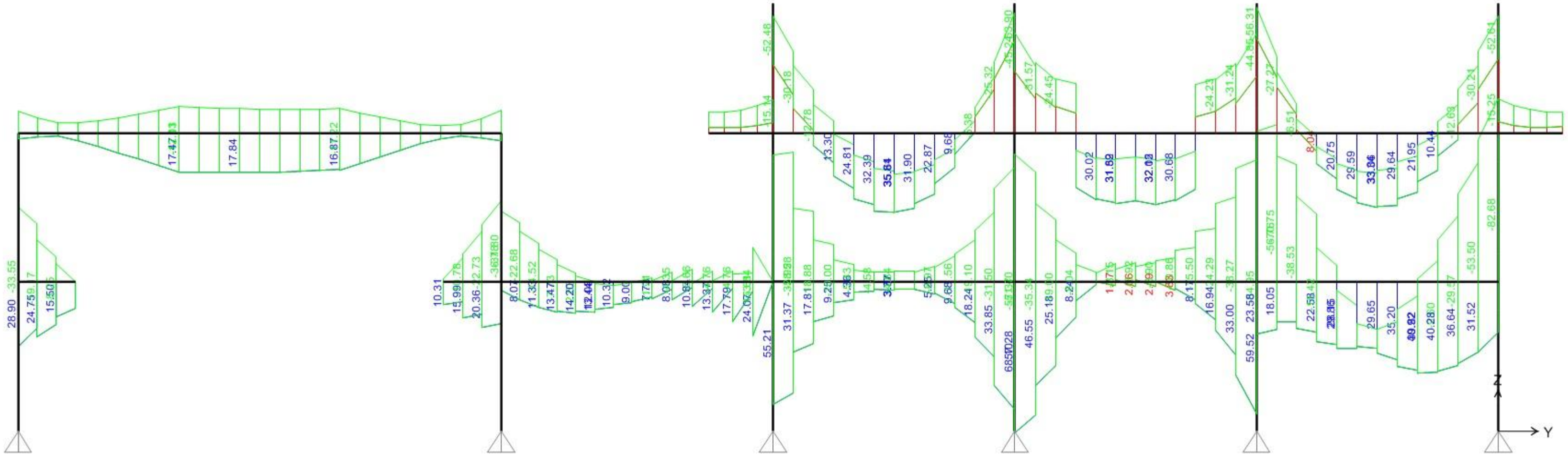

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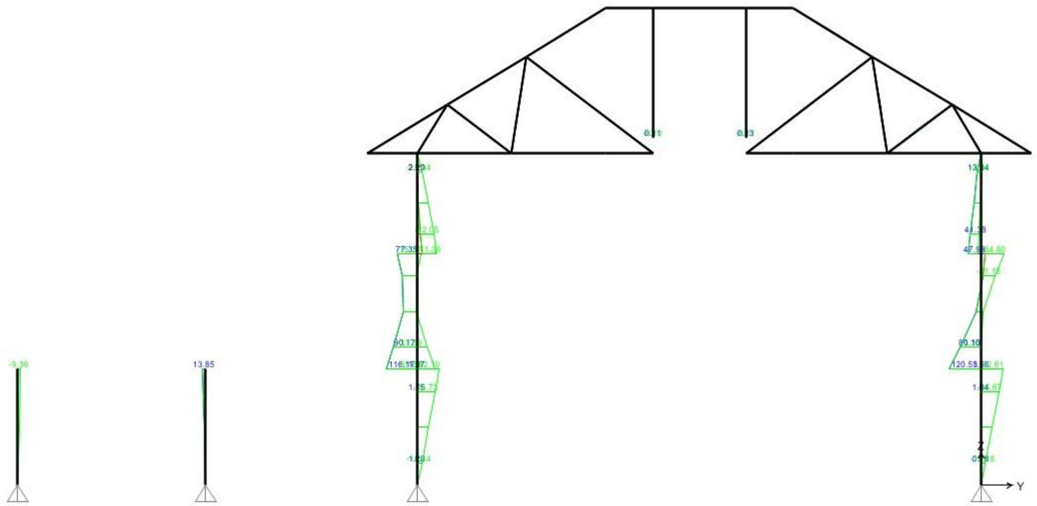

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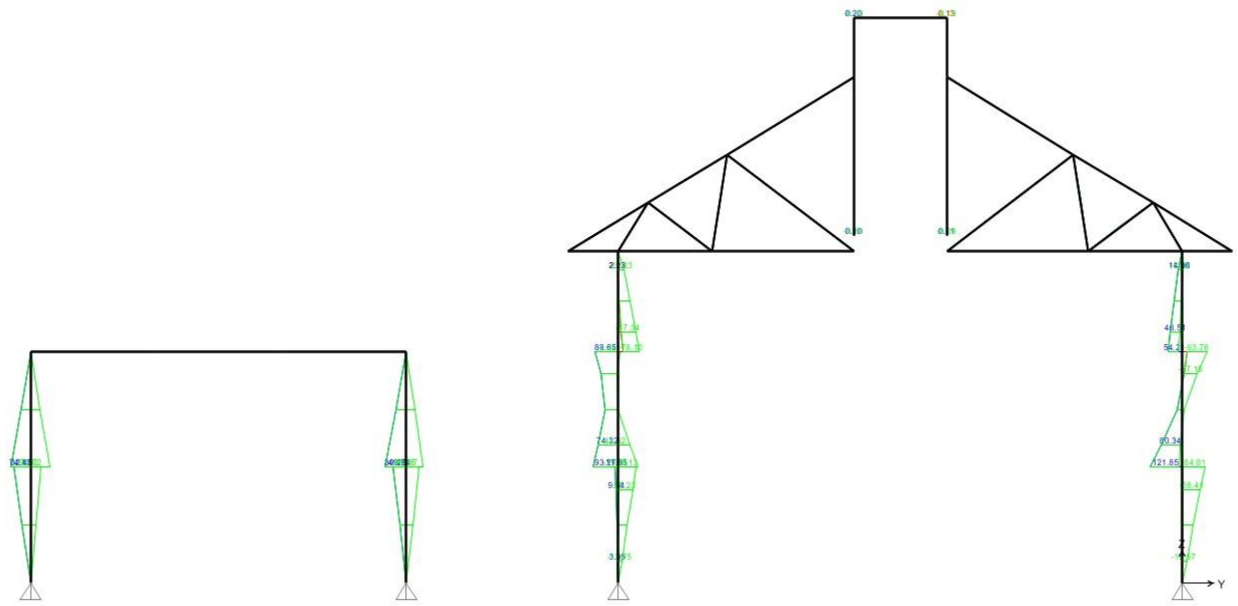

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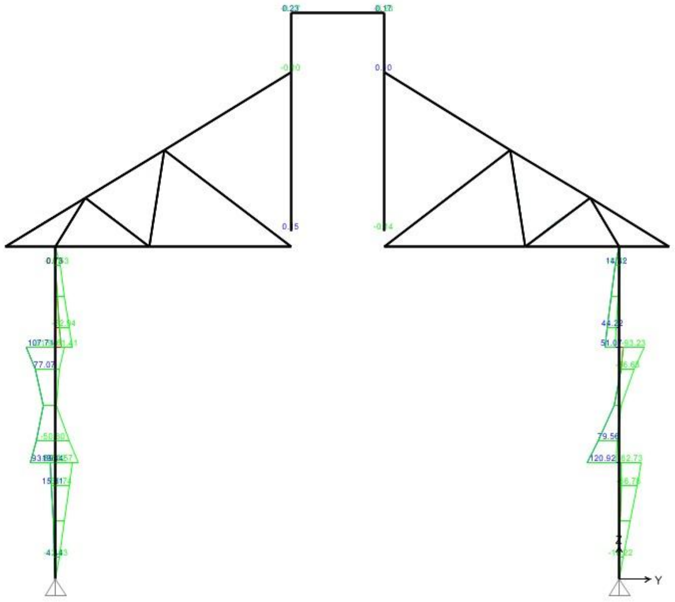
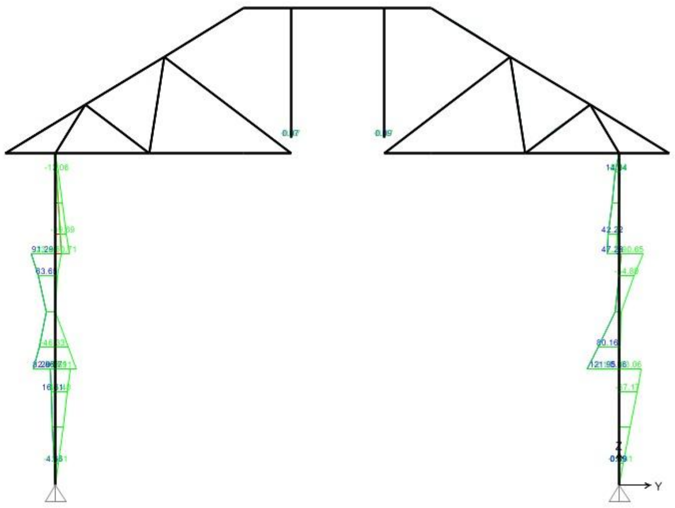


Frame - Grid Line 4 M22 Seismic



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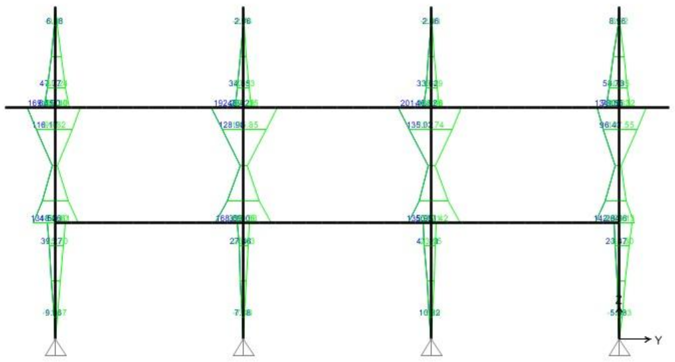
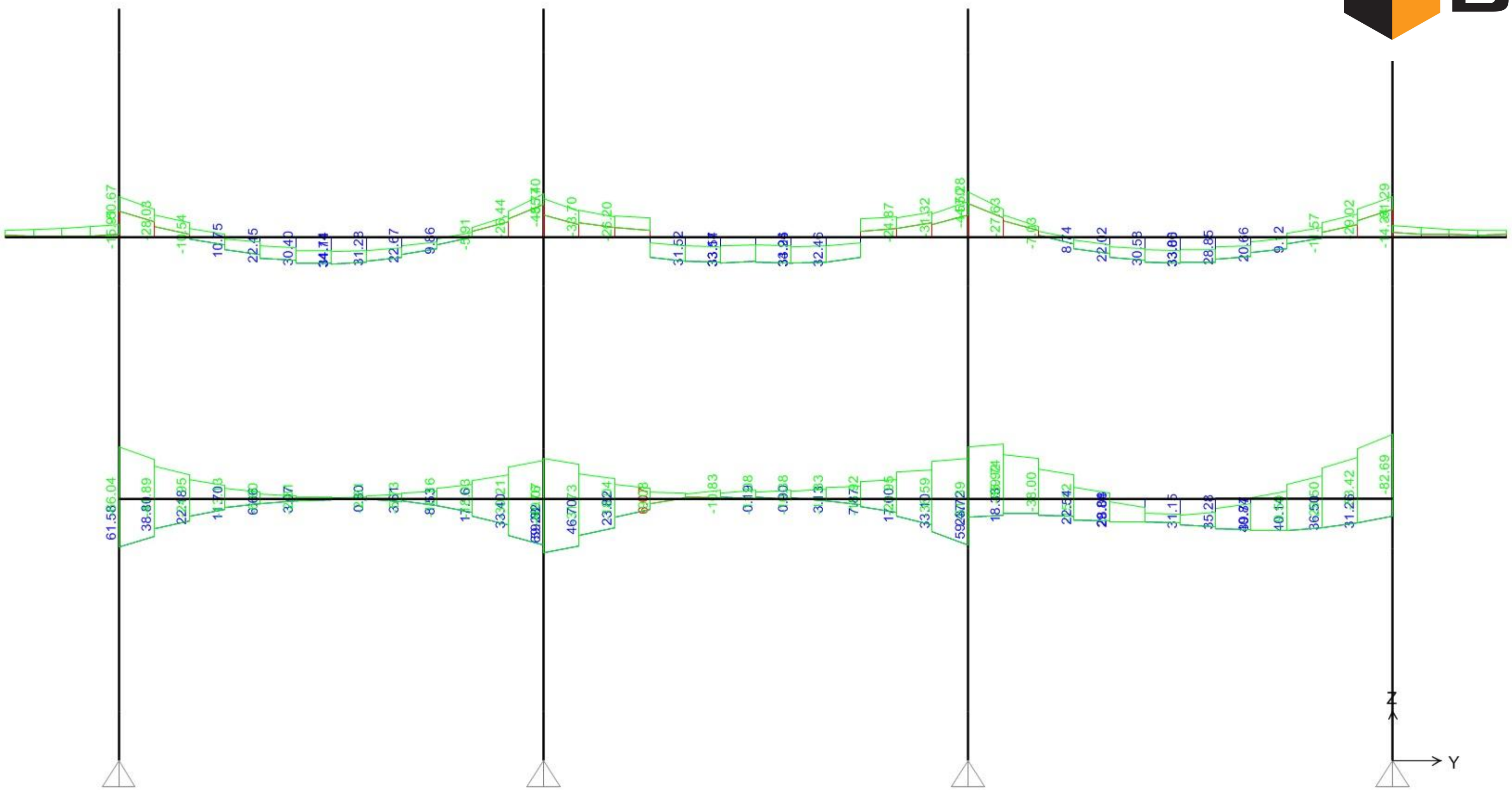


Frame - Grid Line 6 M22 Seismic



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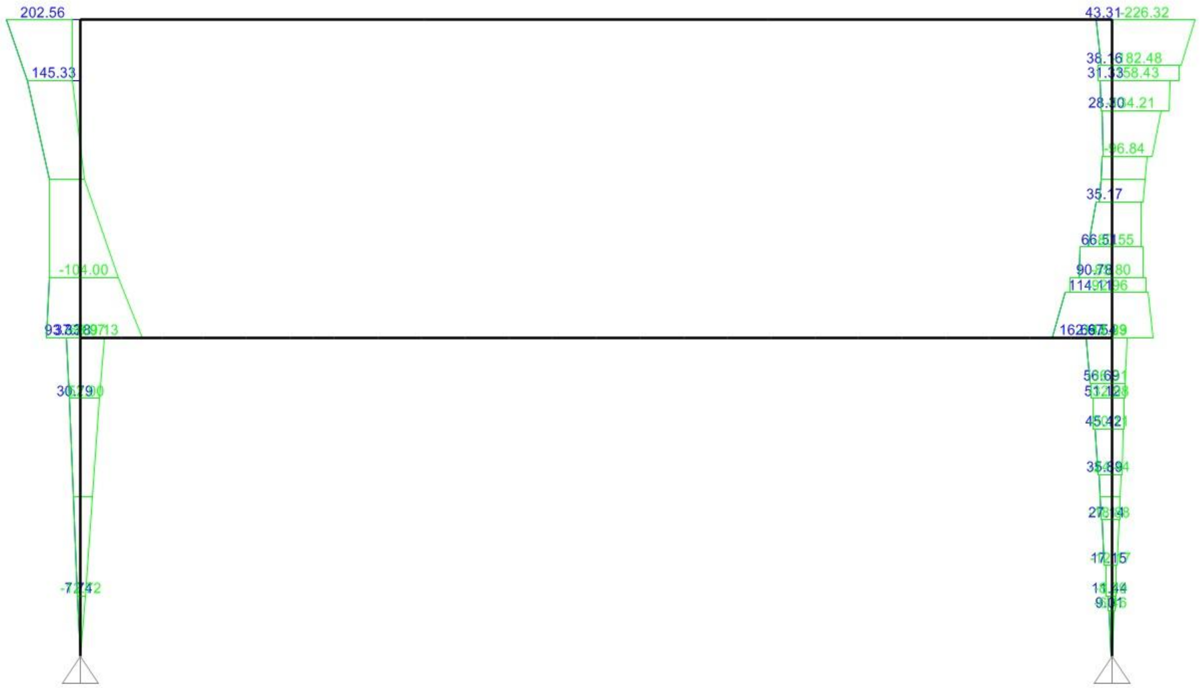
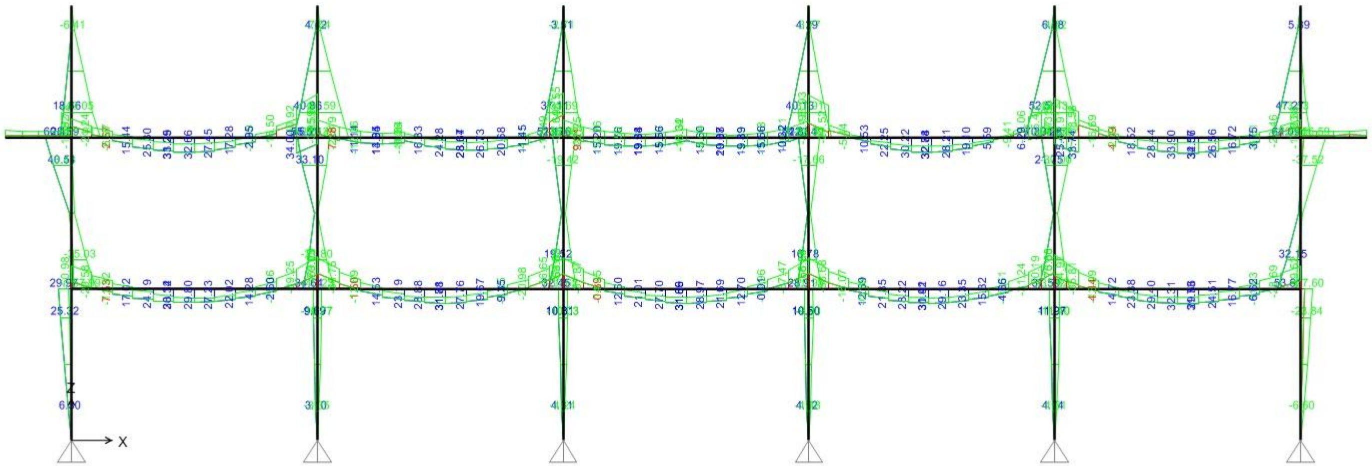
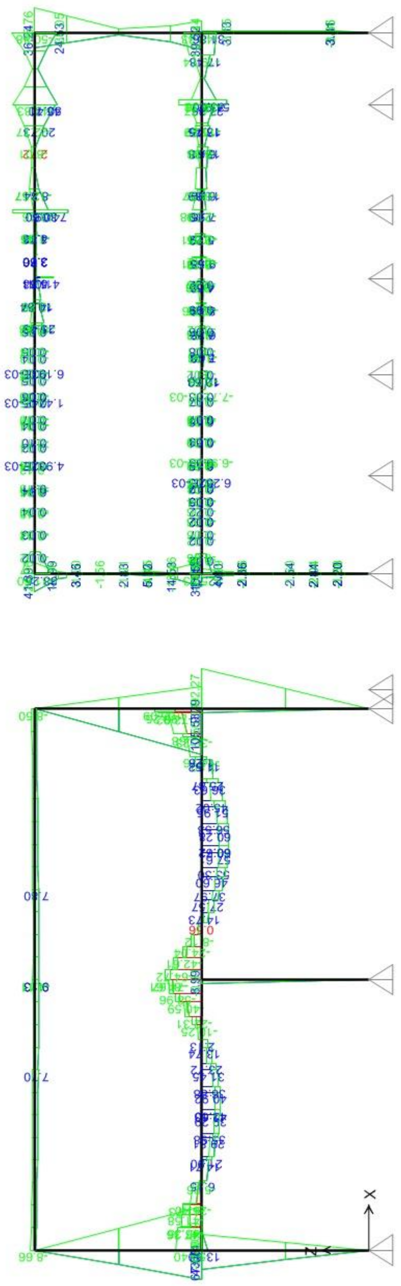


Frame - Grid Line 7 M22 Seismic

Frame - Grid Line 8 M22 Seismic



Frame - Grid Line A M33 Seismic



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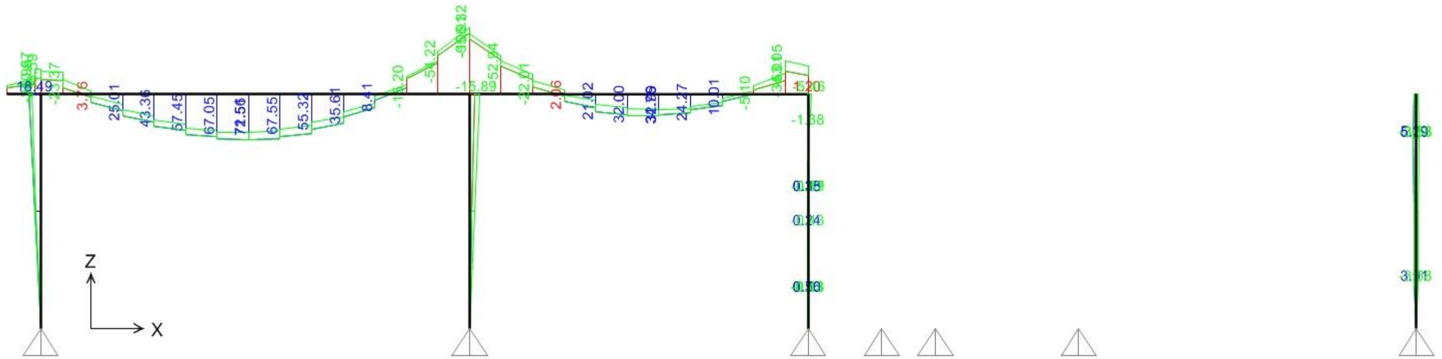

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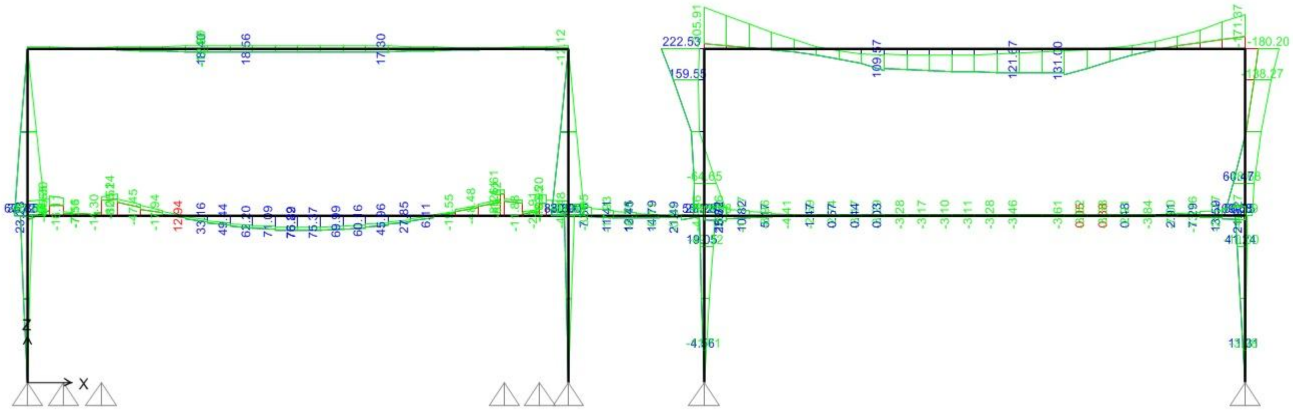

Frame - Grid Line G M33 Seismic

Appendix D – Calculations

DESIGN CALCULATIONS FOR DIAPHRAGM CAPACITY

8" Floor Diaphragm

Internal Capacity

Mesh	No 5 HY60 @ 12"		
d_b	15.875 mm	f'_c	20 MPa
s	304.8 mm	t	203.2 mm
f_y	400 mm	v_c	0.76 MPa
A_s	649.4 mm2/m		
q_s	259.8 N/mm	q_c	154.5 N/mm
ϕ	0.75		
ϕq	310.7 N/mm	*or kN/m*	
ϕv	1.53 MPa		

Shear Transfer at Edge

d_b	15.875 mm	ϕ	0.75	
f_y	400 MPa			
		ϕq	$=\phi*0.62*A_s*f_y$	*Note:* $\tau \cong 0.62 f_y$
s	304.8 mm			
A_s	649.4 mm^2/m	ϕq	120.8 N/mm	*or kN/m*
		ϕv	0.59 MPa	

Therefore use:

Topping	No 5 HY60 @ 12" mesh in 203.2mm concrete topping with a compressive strength of 20MPa
Starters	RB15.875s @ 304.8mm centres

DESIGN CALCULATIONS FOR DIAPHRAGM CAPACITY
75 + 75 Flat Slab Diaphragm

Internal Capacity

Mesh	665		
d_b	5.3 mm	f'_c	30 MPa
s	150 mm	t	75 mm
f_y	500 mm	v_c	0.93 MPa
A_s	147.1 mm2/m		
q_s	73.5 N/mm	q_c	69.8 N/mm
ϕ	0.75		
ϕq	107.5 N/mm	*or kN/m*	
ϕv	1.43 MPa		

Shear Transfer at Edge

d_b	5.3 mm	ϕ	0.75	
f_y	500 MPa			
		ϕq	$=\phi*0.62*A_s*f_y$	*Note:* $\tau \cong 0.62f_y$
s	150 mm			
A_s	147.1 mm^2/m	ϕq	34.2 N/mm	*or kN/m*
		ϕv	0.46 MPa	

Therefore use:

Topping	665 mesh in 75mm concrete topping with a compressive strength of 30MPa
Starters	RB5.3s @ 150mm centres

DESIGN CALCULATIONS FOR SHEAR WALL CAPACITY
Shear Core 2 - 20 Series Walls

Internal Capacity

Mesh

d_b	12 mm	f'_c	17.5 MPa
#			
s	900 mm	t	75 mm
f_y	300 mm	v_c	0.71 MPa
A_s	0.0 mm2/m		
q_s	0.0 N/mm	q_c	53.3 N/mm
ϕ	0.75		
ϕq	40.0 N/mm	*or kN/m*	
ϕv	0.53 MPa		

Shear Transfer at Edge

d_b	12 mm	ϕ	0.75	
f_y	300 MPa			
		ϕq	$=\phi*0.62*A_s*f_y$	*Note:* $\tau \cong 0.62 f_y$
s	900 mm			
A_s	125.7 mm^2/m	ϕq	17.5 N/mm	*or kN/m*
		ϕv	0.23 MPa	

Pull out (Based on RamsetTM ChemsetTM Injection 101 Plus)

ϕN	14.1 kN	*100mm embedment*
X_{nc}	0.87	*20MPa*
X_{ne}	1	*50mm edge distance*
X_{nae}	1	*Anchor spacing, edge*
X_{nai}	1	*Anchor spacing, internal*
X_{nt}	1	*Service Temperature, <35*
X_{nw}	1	*Water in hole, no*
X_{nk}	1	*Tension zone, No*
s	900 mm	
ϕN	13.63 kN/m	

Therefore use:

Topping	mesh in 75mm concrete topping with a compressive strength of 17.5MPa
Starters	D12s @ 900mm centres

DESIGN CALCULATIONS FOR SHEAR WALL CAPACITY
Shear Core 1 - 8" Walls

Internal Capacity

Mesh	No 3 @ 12"			
d_b	9.525 mm	f'_c	20 MPa	
#	2			
s	304.8 mm	t	203.2 mm	
f_y	250 mm	v_c	0.76 MPa	
A_s	467.6 mm2/m			
q_s	116.9 N/mm	q_c	154.5 N/mm	
ϕ	0.75			
ϕq	203.5 N/mm	*or kN/m*		
ϕv	1.00 MPa			

Shear Transfer at Edge

d_b	9.525 mm	ϕ	0.75	
f_y	250 MPa			
		ϕq	$=\phi*0.62*A_s*f_y$	*Note:* $\tau \cong 0.62f_y$
s	304.8 mm			
A_s	233.8 mm^2/m	ϕq	27.2 N/mm	*or kN/m*
		ϕv	0.13 MPa	

Therefore use:

Topping	No 3 @ 12" mesh in 203.2mm concrete topping with a compressive strength of 20MPa
Starters	RB9.525s @ 304.8mm centres

DESIGN CALCULATIONS FOR SHEAR WALL CAPACITY
Shear Core 1 - 6" Walls

Internal Capacity

Mesh	No 4 @ 12"			
d_b	12.7 mm	f'_c	20 MPa	
#	1			
s	304.8 mm	t	152.4 mm	
f_y	250 mm	v_c	0.76 MPa	
A_s	415.6 mm2/m			
q_s	103.9 N/mm	q_c	115.9 N/mm	
ϕ	0.75			
ϕq	164.8 N/mm	*or kN/m*		
ϕv	1.08 MPa			

Shear Transfer at Edge

d_b	12.7 mm	ϕ	0.75	
f_y	250 MPa			
		ϕq	$=\phi*0.62*A_s*f_y$	*Note:* $\tau \cong 0.62f_y$
s	304.8 mm			
A_s	415.6 mm^2/m	ϕq	48.3 N/mm	*or kN/m*
		ϕv	0.32 MPa	

Therefore use:

Topping	No 4 @ 12" mesh in 152.4mm concrete topping with a compressive strength of 20MPa
Starters	RB12.7s @ 304.8mm centres

PROJECT: **Waipa DC TA Office DEE**

JOB No. **13-257** DATE : **23-12-13**

BY: **AJW** PAGE: 6

DESIGN CALCULATIONS FOR SHEAR WALL CAPACITY

Shear Core 2 - 20 Series Walls

Internal Capacity

Mesh	YD12@600		
d_b	12 mm	f'_c	17.5 MPa
#	1		
s	600 mm	t	190 mm
f_y	430 mm	v_c	0.71 MPa
A_s	188.5 mm2/m		
q_s	81.1 N/mm	q_c	135.1 N/mm
ϕ	0.75		
ϕq	162.1 N/mm	*or kN/m*	
ϕv	0.85 MPa		

Shear Transfer at Edge

d_b	12 mm	ϕ	0.75	
f_y	430 MPa			
		ϕq	$=\phi*0.62*A_s*f_y$	*Note:* $\tau \cong 0.62 f_y$
s	600 mm			
A_s	188.5 mm^2/m	ϕq	37.7 N/mm	*or kN/m*
		ϕv	0.20 MPa	

Therefore use:

Topping	YD12@600 mesh in 190mm concrete topping with a compressive strength of 17.5MPa
Starters	RB12s @ 600mm centres

DESIGN CALCULATIONS FOR SHEAR WALL CAPACITY

Shear Core 2 - 20 Series Walls

Internal Capacity

Mesh	YD12@600		
d_b	12 mm	f'_c	17.5 MPa
#	1		
s	800 mm	t	190 mm
f_y	430 mm	v_c	0.71 MPa
A_s	141.4 mm2/m		
q_s	60.8 N/mm	q_c	135.1 N/mm
ϕ	0.75		
ϕq	146.9 N/mm	*or kN/m*	
ϕv	0.77 MPa		

Shear Transfer at Edge

d_b	12 mm	ϕ	0.75	
f_y	430 MPa			
		ϕq	$=\phi*0.62*A_s*f_y$	*Note:* $\tau \cong 0.62 f_y$
s	800 mm			
A_s	141.4 mm^2/m	ϕq	28.3 N/mm	*or kN/m*
		ϕv	0.15 MPa	

Therefore use:

Topping	YD12@600 mesh in 190mm concrete topping with a compressive strength of 17.5MPa
Starters	RB12s @ 800mm centres

DESIGN SPREADSHEET
Main Floor Slab

f'_c	3000	psi	f'_c	3000	psi
t	8	in	t	8	in
No	5		No	5	
d_b	0.625	in	d_b	0.625	in
f_y	58000	psi	f_y	58000	psi
s	6	in	s	12	in
d'	1	in	d'	1	in
d	7	in	d	7	in
A_s	0.614	in^2/ft	A_s	0.307	in^2/ft
ϕ	0.85		ϕ	0.85	
α	0.85		α	0.85	
ϕM_n	194160	lb-in/ft	ϕM_n	101477.68	lb-in/ft
ϕM_n	72.0	kN-m/m	ϕM_n	37.6	kN-m/m

DESIGN SPREADSHEET

Ex Coucnil Chambers Floor Slab

f'_c	3000	psi	f'_c	3000	psi
t	8	in	t	8	in
No	4		No	4	
d_b	0.5	in	d_b	0.5	in
f_y	58000	psi	f_y	58000	psi
s	3	in	s	6	in
d'	1	in	d'	1	in
d	7	in	d	7	in
A_s	0.785	in^2/ft	A_s	0.393	in^2/ft
ϕ	0.85		ϕ	0.85	
α	0.85		α	0.85	
ϕM_n	242220.29	lb-in/ft	ϕM_n	128315.3	lb-in/ft
ϕM_n	89.8	kN-m/m	ϕM_n	47.6	kN-m/m

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DESIGN SPREADSHEET
Beam Capacities - 1973 Original Building

Section A -ve, B +ve, C, D -ve			Section A +ve, D -ve			Section B -ve,		
B	15	in	B	15	in	B	15	in
D	24	in	D	24	in	D	24	in
f'_c	3000	psi	f'_c	3000	psi	f'_c	3000	psi
d'	2	in	d'	2	in	d'	2	in
#	2		#	3		#	4	
d_b	1	in	d_b	1	in	d_b	1	in
f_y	36000	psi	f_y	36000	psi	f_y	36000	psi
d	22	in	d	22	in	d	22	in
A_s	1.571	in^2	A_s	2.356	in^2	A_s	3.142	in^2
ϕ	0.85		ϕ	0.85		ϕ	0.85	
α	0.85		α	0.85		α	0.85	
ϕM_n	1021929.5	lb-in	ϕM_n	1506246.3	lb-in	ϕM_n	1972797.9	lb-in
ϕM_n	115.5	kN-m	ϕM_n	170.2	kN-m	ϕM_n	222.9	kN-m

DESIGN SPREADSHEET

Column Capacities - 1973 Original Building

	Section F			**Section G**			**Section H**	
B	24	in	B	22.25	in	B	24	in
D	24	in	D	22.25	in	D	24	in
f'_c	3000	psi	f'_c	3000	psi	f'_c	3000	psi
d'	2.5	in	d'	1.625	in	d'	2.5	in
#	2		#	3		#	3	
d_b	1	in	d_b	1	in	d_b	1	in
f_y	36000	psi	f_y	36000	psi	f_y	36000	psi
d	21.5	in	d	20.625	in	d	21.5	in
A_s	1.57	in^2	A_s	2.36	in^2	A_s	2.36	in^2
ϕ	0.85		ϕ	0.85		ϕ	0.85	
α	0.85		α	0.85		α	0.85	
ϕM_n	1011220.3	lb-in	ϕM_n	1433158.6	lb-in	ϕM_n	1500175.5	lb-in
ϕM_n	114.3	kN-m	ϕM_n	161.9	kN-m	ϕM_n	169.5	kN-m

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DESIGN SPREADSHEET

Beam Capacities - 1997 Alterations

600x300 Concrete			600x400 A Concrete +ve			600x400 A Concrete -ve		
B	300	mm	B	400	mm	B	400	mm
D	600	mm	D	600	mm	D	600	mm
f'_c	30	MPa	f'_c	30	MPa	f'_c	30	MPa
d'	50	mm	d'	50	mm	d'	50	mm
#	2		#	4		#	2	
d_b	16	mm	d_b	24	mm	d_b	28	mm
f_y	430	MPa	f_y	430	MPa	f_y	430	MPa
d	550	mm	d	550	mm	d	550	mm
A_s	402.1	mm^2	A_s	1809.6	mm^2	A_s	1231.5	mm^2
ϕ	0.85		ϕ	0.85		ϕ	0.85	
α	0.85		α	0.85		α	0.85	
ϕM_n	79.2	kN-m	ϕM_n	338.5	kN-m	ϕM_n	235.9	kN-m

DESIGN SPREADSHEET

Beam Capacities - 1997 Alterations

600x400 B Concrete +ve			600x400 B Concrete -ve		
B	400	mm	B	400	mm
D	600	mm	D	600	mm
f'_c	30	MPa	f'_c	30	MPa
d'	50	mm	d'	100	mm
#	3		#	4	
d_b	28	mm	d_b	28	mm
f_y	430	MPa	f_y	430	MPa
d	550	mm	d	500	mm
A_s	1847.3	mm^2	A_s	2463.0	mm^2
ϕ	0.85		ϕ	0.85	
α	0.85		α	0.85	
ϕM_n	345.1	kN-m	ϕM_n	403.4	kN-m

DESIGN SPREADSHEET
Beam Capacities - 1997 Alterations

500x500 Concrete +ve			500x500 Concrete -ve		
B	500	mm	B	500	mm
D	500	mm	D	500	mm
f'_c	30	MPa	f'_c	30	MPa
d'	50	mm	d'	50	mm
#	4		#	4	
d_b	24	mm	d_b	28	mm
f_y	430	MPa	f_y	430	MPa
d	450	mm	d	450	mm
A_s	1809.6	mm^2	A_s	2463.0	mm^2
ϕ	0.85		ϕ	0.85	
α	0.85		α	0.85	
ϕM_n	277.4	kN-m	ϕM_n	367.7	kN-m

DESIGN SPREADSHEET
Column Capacities - 1997 Alterations

	600x600 Concrete		**400x400 Concrete**		**390x390 Masonry**
B	600 mm	B	400 mm	B	390 mm
D	600 mm	D	400 mm	D	390 mm
f'_c	30 MPa	f'_c	30 MPa	f'_c	12 MPa
d'	50 mm	d'	50 mm	d'	55 mm
#	3	#	2	#	2
d_b	24 mm	d_b	24 mm	d_b	16 mm
f_y	430 MPa	f_y	430 MPa	f_y	430 MPa
d	550 mm	d	350 mm	d	335 mm
A_s	1357.2 mm^2	A_s	904.8 mm^2	A_s	402.1 mm^2
ϕ	0.85	ϕ	0.85	ϕ	0.85
α	0.85	α	0.85	α	0.85
ϕM_n	263.4 kN-m	ϕM_n	109.4 kN-m	ϕM_n	46.0 kN-m