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Wigram Aerodrome - Harvard Lounge
BU 2556-002 EQ2
Detailed Engineering Evaluation
Quantitative Report
Version Final

8 Corsair Drive, Hornby

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Quantitative Report
Version Final

8 Corsair Drive, Hornby

Christchurch City Council

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Wigram Aerodrome – Harvard Lounge

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

8 Corsair Drive, Hornby

Background

The single storey building at 8 Corsair Drive, Hornby, Christchurch has been assessed for its safety during an earthquake. We have assessed the structure of the building to determine the current level of safety it affords during an earthquake, and have compared that level to the legal requirements.

This is a summary of the Quantitative report for the building structure, and is based in part on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 18th January 2012 and Qualitative report version draft issued on 27th February 2012.

Building Description

Wigram Aerodrome – Harvard Lounge is located at 8 Corsair Drive, Hornby, Christchurch. The single storey building is currently used as a function room which is available for hire. The original building was constructed in 1980 and 1981 based on the drawings provided by Christchurch City Council.

The building is approximately 30 m in length, 21 m wide and 6 m in height. Perimeter and interior walls consist of concrete filled masonry block wall. The roof is comprises of butynol fabric roofing on customwood supported by timber purlins on timber roof trusses. The timber roof trusses are connected to the steel rafter at lounge area located at the centre part of the building. The rest of the interior framing comprises of steel beams, timber beams and steel columns.

The floor is a reinforced concrete ground slab and the foundations consist of reinforced concrete strip footings.

Key Damage Observed

Key damage observed includes:

- Minor cracking between the external cladding and the top of the concrete masonry piers.

Building Capacity Assessment

GHD finds that the Wigram Aerodrome – Harvard Lounge achieves overall 42% New Building Standard (NBS) and is therefore considered “Earthquake Risk”.

Recommendations

It is recommended that:

- A strengthening scheme is developed to increase the seismic capacity of the building to at least 67% NBS.
- The current placard status of the building of green to remain as is.
- The building can still be used, as per CCC's policy to occupy "Earthquake Risk" buildings.
- Minor repairs are undertaken to fill the minor cracks identified at the top of the concrete masonry piers.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of Wigram Aerodrome – Harvard Lounge; a single storey function centre.

This is a Quantitative Assessment Report of the building structure; Quantitative Assessment involves full seismic review of the existing structure, which is discussed in this report. The structural investigation has been carried out in accordance with the requirements of the relevant New Zealand Standards and the New Zealand Society for Earthquake Engineering (NZSEE) Guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes.

2. Compliance

This section contains a brief summary of the requirements of the various statutes and authorities that control activities in relation to buildings in Christchurch at present.

2.1 Canterbury Earthquake Recovery Authority (CERA)

CERA was established on 28 March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18 April 2011. This act gives the Chief Executive Officer of CERA wide powers in relation to building safety, demolition and repair. Two relevant sections are:

Section 38 – Works

This section outlines a process in which the chief executive can give notice that a building is to be demolished and if the owner does not carry out the demolition, the chief executive can commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

This section enables the chief executive to require a building owner, insurer or mortgagee carry out a full structural survey before the building is re-occupied.

CERA now requires a detailed engineering evaluation to be carried out for all buildings (other than those exempt from the Earthquake Prone Building definition in the Building Act). The Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011 has been adopted by CERA for evaluations both qualitative and quantitative assessments.

The qualitative assessment is a desk-top and site inspection assessment. It is based on a thorough visual inspection of the building coupled with a review of available documentation such as drawings and specifications. The quantitative assessment involves analytical calculation of the buildings strength and may require non-destructive or destructive material testing, geotechnical testing and intrusive investigation.

Factors determining the extent of evaluation and strengthening level required include:

- The importance level and occupancy of the building
- The placard status and amount of damage
- The age and structural type of the building
- Consideration of any critical structural weaknesses
- The extent of any earthquake damage

2.2 Building Act

Several sections of the Building Act 2004 are relevant when considering structural requirements:

Section 112 – Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to any alteration. This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) be satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'. Regarding seismic capacity 'as near as reasonably practicable' has previously been interpreted by CCC as achieving a minimum of 67% NBS, however where practical achieving 100% NBS is desirable. The New Zealand Society for Earthquake Engineering (NZSEE) recommend a minimum of 67% NBS.

2.2.1 Section 121 – Dangerous Buildings

The definition of dangerous building in the Act was extended by the Canterbury Earthquake (Building Act) Order 2010, and it now defines a building as dangerous if:

- In the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
- In the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or
- There is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
- There is a risk that that other property could collapse or otherwise cause injury or death; or
- A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone if its ultimate capacity would be exceeded in a 'moderate earthquake' and it would be likely to collapse causing injury or death, or damage to other property. A moderate earthquake is defined by the building regulations as one that would generate ground shaking 33% of the shaking used to design an equivalent new building.

Section 124 – Powers of Territorial Authorities

This section gives the territorial authority the power to require strengthening work within specified timeframes or to close and prevent occupancy to any building defined as dangerous or earthquake prone.

Section 131 – Earthquake Prone Building Policy

This section requires the territorial authority to adopt a specific policy for earthquake prone, dangerous and insanitary buildings.

2.3 Christchurch City Council Policy

Christchurch City Council adopted their Earthquake Prone, Dangerous and Insanitary Building Policy in 2006. This policy was amended immediately following the Darfield Earthquake of the 4th September 2010.

The 2010 amendment includes the following:

- A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
- A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
- A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
- Repair works for buildings damaged by earthquakes will be required to comply with the above.

The council has stated their willingness to consider retrofit proposals on a case by case basis, considering the economic impact of such a retrofit.

We anticipate that any building with a capacity of less than 33% NBS (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% NBS of new building standard as recommended by the Policy.

If strengthening works are undertaken, a building consent will be required. A requirement of the consent will require upgrade of the building to comply 'as near as is reasonably practicable' with:

- The accessibility requirements of the Building Code.
- The fire requirements of the Building Code. This is likely to require a fire report to be submitted with the building consent application.

2.4 Building Code

The building code outlines performance standards for buildings and the Building Act requires that all new buildings comply with this code. Compliance Documents published by The Department of Building and Housing can be used to demonstrate compliance with the Building Code.

After the February Earthquake, on 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load)
- Serviceability Return Period Factor increased from 0.25 to 0.33 (80% increase in the serviceability design loads when combined with the Hazard Factor increase)

The increase in the above factors has resulted in a reduction in the level of compliance of an existing building relative to a new building despite the capacity of the existing building not changing.

3. Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The new building standard load requirements have been determined in accordance with the current earthquake loading standard (NZS 1170.5:2004 Structural design actions - Earthquake actions - New Zealand).

The likely capacity of this building has been derived in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidelines 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes' (AISPBE), 2006. These guidelines provide an Initial Evaluation Procedure that assesses a buildings capacity based on a comparison of loading codes from when the building was designed and currently. It is a quick high-level procedure that can be used when undertaking a Qualitative analysis of a building. The guidelines also provide guidance on calculating a modified Ultimate Limit State capacity of the building which is much more accurate and can be used when undertaking a Quantitative analysis.

The New Zealand Society for Earthquake Engineering has proposed a way for classifying earthquake risk for existing buildings in terms of %NBS and this is shown Figure 3-1 below.

Description	Grade	Risk	% NBS	Existing Building Structural Performance	Improvement of Structural Performance: Legal Requirement	Improvement of Structural Performance: NZSEE Recommendation
Low Risk Building	A or B	Low	Above 67	Acceptable (Improvement may be desirable)	The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34% NBS.	100% NBS desirable. Improvement should achieve at least 67% NBS
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally, Improvement recommended		Not recommended. Acceptable only in exceptional circumtances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement Required)	Unacceptable	Unacceptable

Figure 3-1 NZSEE Risk Classifications Extracted from Table 2.2 of the NZSEE 2006 AISPBE

Table 3-1 compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

Percentage of New Building Standard (%NBS)	Relative Risk (Approximate)
>100	<1 time
80-100	1-2 times
67-80	2-5 times
33-67	5-10 times
20-33	10-25 times
<20	>25 times

Table 3-1 %NBS compared to relative risk of failure

4. Building Description

4.1 General

Wigram Aerodrome – Harvard Lounge is located at 8 Corsair Drive, Hornby, Christchurch. The single storey building is currently used as a function room which is available for hire. The original building was constructed in 1980 and 1981 based on the drawings provided by Christchurch City Council.

Summary of Building key structural features:

- The building is approximately 30 m in length, 21 m wide and 6 m in height.
- Perimeter and interior walls consist of concrete filled masonry block wall.
- The roof is comprises of butynol fabric roofing on 20 mm customwood supported by timber purlins on roof trusses.
- The timber roof trusses are connected to the steel rafter at lounge area located at the centre part of the building.
- The rest of the interior framing comprises of steel beams, timber beams and steel columns.
- The floor is a reinforced concrete ground slab.
- The foundations consist of reinforced concrete strip footings.

Key structural details of the building are shown in Figure 2 below.

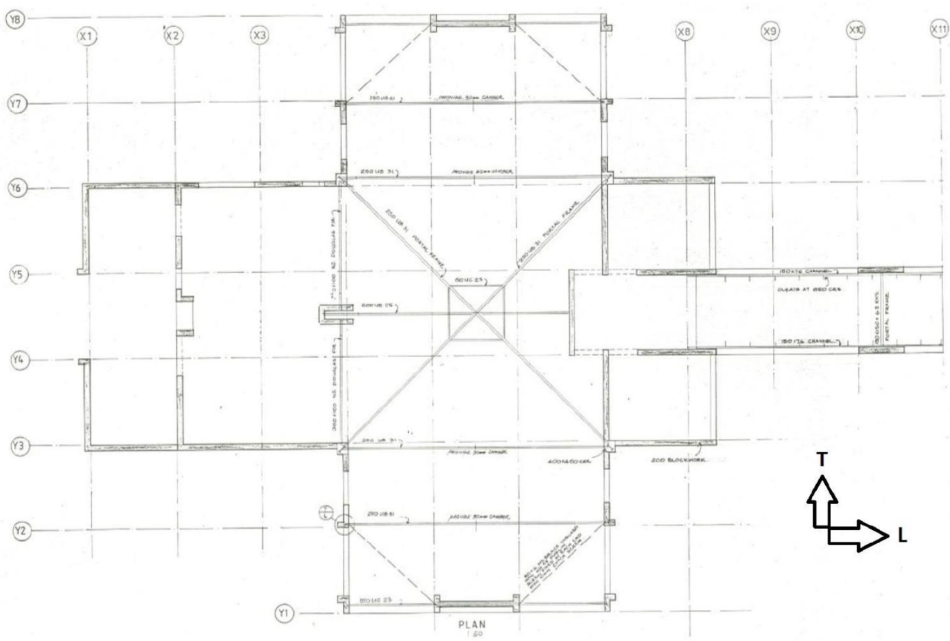


Figure 4-1Sketch Plan Showing Key Structural Elements

4.2 Gravity Load Resisting System

The gravity loads acting on the structure are resisted by concrete masonry block walls and a system of steel beams and concrete masonry piers.

Gravity loads from the roof are transferred through the timber purlins spanning between the timber rafters and steel beams. The loads are then transferred through the steel beams and timber framing and into the concrete masonry piers and walls and down into the foundations of the building.

The steel beams span over the function room area and are supported by concrete masonry piers at each end. The central steel beam sits on a large concrete masonry pier at one end and an internal concrete masonry wall at the other end. The roof structure in the central area of the function room consists of light steel cladding supported by timber purlins and timber rafters. The timber rafters span between the concrete masonry walls and a central steel RHS post as shown in Photograph 11. The steel post is supported by the central steel beam.

The gravity loads in the concrete masonry block wall areas to the north and south of the main function room area are resisted by a concrete masonry wall system. The roof structure consists of light steel cladding supported by timber framing spanning between concrete masonry walls.

The canopy over the entrance to the building is supported by RHS posts at the end closest to the road as shown in Photograph 3. The other end is supported by concrete masonry walls.

4.3 Lateral Load Resisting System

Lateral loads acting on the structure are resisted by concrete masonry walls in both the long and short directions of the building.

The building has a number of concrete masonry walls in the short and long directions which are laid out in a regular pattern throughout the building. During an earthquake the building is expected to behave in a relatively stiff manner due to the number of walls in both directions. Examples of the concrete masonry walls are shown in Photographs 8 and 15. Some diaphragm action from the roof spanning over the function room area is expected, transferring forces in the roof structure to the supporting concrete masonry piers and walls during an earthquake.

5. Assessment

5.1 Site Inspection

An inspection of the building was undertaken on the 18th of January 2012. Both the interior and exterior of the building were inspected. The building was observed to have a green placard in place. A large portion of the main structural components of the building were able to be viewed due to the exposed nature of the structure. The concrete masonry walls are unlined and the steel and timber framing is generally exposed. No inspection of the foundations of the structure was able to be undertaken.

The inspection consisted of observing the building to determine the structural systems and likely behaviour of the building during an earthquake. The site was assessed for damage, including observing the ground conditions, checking for damage in areas where damage would be expected for the structure type observed and noting general damage observed throughout the building in both structural and non-structural elements.

5.2 Investigation

5.2.1 Available Drawings

Table 5-1 outlines the construction drawings that were provided by CCC:

Item	Title	Sheet No.	Date
1	Plan and General Details	S1A	20/11/80
2	Foundation Plan and Details	S2A	20/11/80
3	Blockwork	S3A	20/11/80
4	Structural Steel	S4A	20/11/80
5	Revised Location Proposal	P1	11/81
6	Elevations Floor, Drainage, Electrical Plan	W1	11/81
7	Sections A-A, B-B, C-C Construction Details	W2	11/81
8	Sections D-D, E-E, F-F East Elev Kitchen Joinery	W3	11/81

Table 5-1 Construction drawings provided by CCC

The drawings have been used to confirm the structural systems, investigate potential critical structural weaknesses (CSW) and identify details which required particular attention.

No specification information and structural calculations for the building have been located.

Drawings are provided in Appendix C of this report.

5.3 Analysis and Modelling Methodology

The seismic assessment procedure determines the capacity of the structure to withstand seismic loading (as defined in the current New Zealand Standard 1170.5:2004) through structural analysis. The seismic capacity of the structure is measured as a proportion of New Building Standard (% NBS), the standard to which a new building must perform in terms of current design codes and standard. The weakest structural element of the structure is the element which governs the seismic capacity of the overall structure.

The methodology and approach adopted for the analysis and assessment is presented in the following sections.

5.3.1 Building Modelling

The three dimensional frame modelling of the Wigram Aerodrome – Harvard Lounge was undertaken to realistically simulate the effects of the applied loads on the structure under different loading conditions such as normal operation, earthquake and combinations thereof.

Each section, member and node of the model was defined using the physical dimensions, material properties and connection details from the available drawings described in Section 5.2.1. The structural software ETABS v.9.7.2 was used for the general modelling and analysis of the structure. The foundations were assumed to be pinned in the 3D model.

Figure 3 shows overall view of the model.

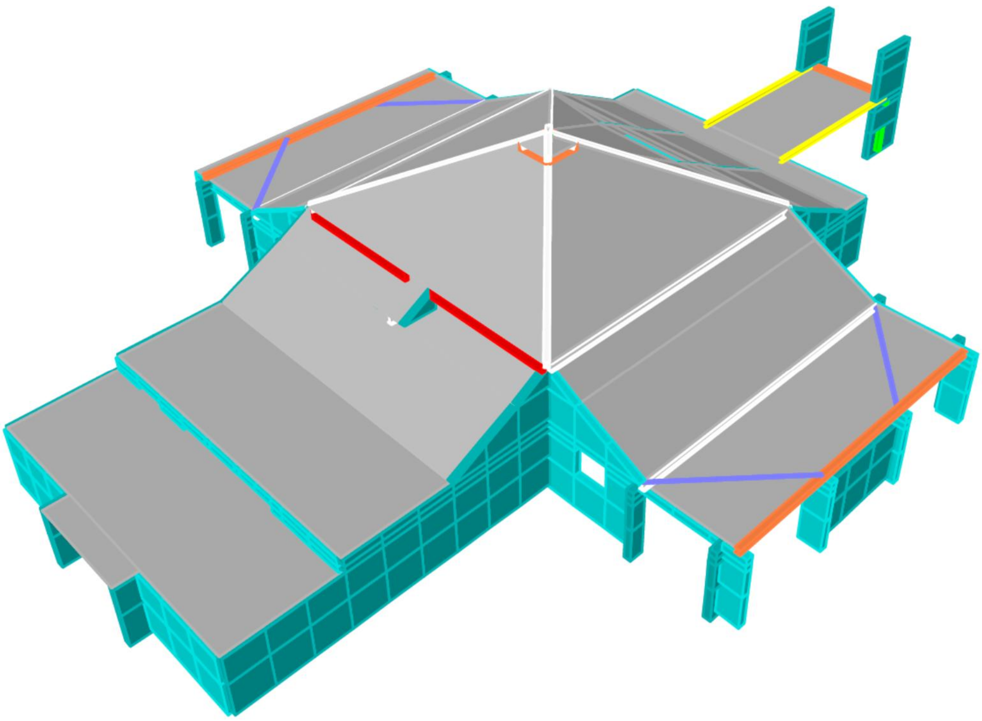


Figure 5-1: Model of building developed in Etabs

5.3.2 Loading Conditions

The loading conditions and load combinations used in the analysis of the structure was in accordance with AS/NZS 1170:2002.

5.3.3 Determination of % NBS

Upon determination of the critical loading conditions, each of the structural members that make up the Harvard Lounge was checked to determine % NBS of the members indicated as shown in the available drawings. Members demand and capacity ratio was computed and % NBS was calculated accordingly.

5.3.4 Seismic Design

The Wigram Aerodrome – Harvard Lounge structure was checked to the seismic design standards in accordance with the AS/NZ 1170.5:2004, NZBC Clause B1 Structure and New Zealand Society of Earthquake Engineering "Guidelines for Assessment and Improvement of the Structural Performance of Buildings in Earthquakes".

The seismic assessment was undertaken using the equivalent static method as described in Clause 6.2 of the NZS 1170.5 and the 3D models created in Etabs.

6. Damage Assessment

6.1 Surrounding Buildings

No damage to surrounding buildings was observed during our inspection of the site.

6.2 Residual Displacements and General Observations

No residual displacements of the structure were noticed during our inspection of the building.

Minor cracking was observed between the cladding and the top of the concrete masonry piers as can be seen in Photographs 5 and 9.

6.3 Ground Damage

There was no evidence of ground damage on the property.

7. Seismic Analysis

7.1 Seismic Parameters

Seismic loads were applied based on criteria specified by the New Zealand Code (NZS 1170.5:2004) and New Zealand Society of Earthquake Engineering (NZSEE).

The seismic assessment parameters are as tabulated below:

Site Classification	D
Importance Level	2
Hazard factor, (Z) (Table 3.3, NZS 1170.5:2004	0.30 (Christchurch)
and NZBC Clause B1 Structure)	
Annual Probability of Exceedance (Table 3.3, NZS 1170.0:2002)	1/500 (ULS)
Annual Probability of Exceedance (Table 3.3, NZS 1170.0:2002)	1/25 (SLS)
Return Period Factor (R_u), (Table 3.5, NZS 1170.5:2004)	1.0 (ULS)
Return Period Factor (R_s), (Table 3.5, NZS 1170.5:2004)	0.33 (SLS)
(NZBC B1 Clause 2.2.14c)	
Ductility Factor (μ) (Section 4.3.1.1, NZS 1170.5: 2004)	1.25
Performance Factor (S_p) (Section 4.4.2, NZS 1170.5:2004)	0.90
Liquefaction Potential	minor to moderate

8. Geotechnical Consideration

The terrain of the subject site is relatively flat at approximately 22 m above mean sea level. It is 1 km north of the Heathcote River, approximately 3 km southwest of the Avon River, and approximately 15 km west of the coast (Pegasus Bay) at New Brighton.

8.1 Published Information on Ground Conditions

8.1.1 Published Geology

The geological map of the area1 indicates that the site is underlain by Holocene alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation. The Springston Formation consists predominantly of dense alluvial gravel deposits.

Groundwater is indicated to be between 5 to 10 m bgl.

8.1.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that are no boreholes within 200m of the site. Borehole lithographic logs from beyond this distance show predominantly gravel and sandy gravel with varying quantities of clay beneath the ground surface.

8.1.3 EQC Geotechnical Investigations

The Earthquake Commission has not undertaken geotechnical testing in the area of the site.

8.1.4 Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has indicated the site is situated within the Green Zone, indicating that repair and rebuild may take place.

Land in the CERA green zone has been divided into three technical categories. These categories describe how the land in expected to perform in future earthquakes.

The site has been categorised as “N/A” – Urban Non-residential”. However, neighbouring residential properties have been categorised as TC1 (grey), indicating future land damage from liquefaction is unlikely.

8.1.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake (Figure 8-1) and all other earthquakes of the Canterbuty earthquake sequence shows no signs of liquefaction.

[1] Brown, L. J. and Weeber J.H. 1992: Geology of the Christchurch Urban Area. Institute of Geological and Nuclear Sciences 1:25,000 Geological Map 1. Lower Hutt. Institute of Geological and Nuclear Sciences Limited.

Figure 8-1 Post February 2011 Earthquake Aerial Photography [2]

8.1.6 Summary of Ground Conditions

From the information presented above, the ground conditions are indicated to comprise gravel and sandy gravel with varying quantities of clay beneath the ground surface.

8.2 Seismicity

8.2.1 Nearby Faults

There are many faults in the Christchurch region, however only those considered most likely to have an adverse effect on the site are detailed below.

Table 8-1 Summary of Known Active Faults[3,4]

Known Active Fault	Distance from Site (km)	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	125 km	NW	~8.3	~300 years

[2] Aerial Photography Supplied by Koordinates sourced from http://koordinates.com/layer/3185-christchurch-post-earthquake-aerial-photos-24-feb-2011/

[3] Stirling, M.W, McVerry, G.H, and Berryman K.R. (2002) A New Seismic Hazard Model for New Zealand, Bulletin of the Seismological Society of America, Vol. 92 No. 5, pp 1878-1903, June 2002.

[4] GNS Active Faults Database

Known Active Fault	Distance from Site (km)	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Greendale (2010) Fault	14 km	W	7.1	~15,000 years
Hope Fault	110 km	N	7.2~7.5	120~200 years
Kelly Fault	110 km	NW	7.2	150 years
Porters Pass Fault	70 km	NW	7.0	~1100 years
Port Hills Fault (2011)	8 km	SE	6.3	Not Estimated

Recent earthquakes since 22 February 2011 have identified the presence of a new active fault system / zone underneath Christchurch City and the Port Hills. Research and published information on this system is in development and not generally available and average recurrence intervals are yet to be established.

8.2.2 Ground Shaking Hazard

New Zealand Standard NZS 1170.5:2004 quantifies the Seismic Hazard factor for Christchurch as 0.30, being in a moderate to high earthquake zone. This value has been provisionally upgraded recently (from 0.22) to reflect the seismicity hazard observed in the earthquakes since 4 September 2010.

The recent seismic activity has produced earthquakes of Magnitude 6.3 with significant peak ground accelerations (PGA) across large parts of the city.

Conditional PGA's from the CGD5 indicate the PGA to be 0.28g during the 4 September 2010 earthquake, 0.28g on 22 February 2011, and 0.13g on 13 June 2011.

8.3 Slope Failure and/or Rockfall Potential

The site is located within Hornby, a flat suburb in western Christchurch. Global slope instability risk is considered negligible. However, any localised retaining structures and/or embankments should be further investigated to determine the site-specific slope instability potential.

8.4 Field Investigations

In order to further understand the ground conditions at the site, intrusive testing comprising one CPTU investigation was conducted at the site on 02 April 2012.

The location of the test is tabulated in Table 8-2.

[5] Canterbury Geotechnical Database (2012): "Conditional PGA for Liquefaction Assessment", Map Layer CGD5110 - 27 Sept 2012, retrieved 31/10/2012 from https://canterburygeotechnicaldatabase.projectorbit.com/

Table 8-2 Coordinates of Investigation Locations

Investigation	Depth (m bgl)	Easting (NZMG)	Northing (NZMG)
CPTU 001	1.0	2473140	5739968

The CPTU investigation was undertaken by McMillan Drilling Service on 02 April 2012, typically to a target depth of 20m below ground level. However, refusal was reached at depth of 1.0m due to the presence of dense gravels.

Interpretation of output graphs6 from the investigation showing Cone Tip Resistance (qc), Friction Ratio (Fr), Inferred Lithology and Inferred Liquefaction Potential are presented in Table 8-3.

8.5 Ground Conditions Encountered

8.5.1 Summary of CPT-Inferred Lithology

Table 8-3 Summary of CPT-Inferred Lithology

Depth (m)	Lithology [1]	Cone Tip Resistance q_c (MPa)	Friction Ratio Fr (%)
0 – 1.0	Surface soil	~15.0	1.0 – 2.0
>1.0	Gravel	> 20.0	~0.0

8.6 Liquefaction Assessment

8.6.1 Interpretation of Analysis

Overall, the site is considered unlikely to liquefy due to the following reasons:

- No observations of liquefaction post in earthquake aerial photography;
- Neighboring properties are classified as TC1 (grey); and,
- Anticipated presence of gravel and sandy gravel beneath the site.

8.7 Summary and Recommendations

The site appears to be situated on alluvial deposits, comprising gravel and sandy and silt. Associated with this the site is unlikely to liquefy.

Based on the information presented above, we recommend the following for the subject site:

- Standard foundations can be used for timber and concrete floors, in accordance with New Zealand Building regulations. For larger buildings, all foundations should be specifically-designed by a suitably qualified and experienced geotechnical engineer;
- Ground improvement works are not recommended.

[6] McMillans Drilling CPT data plots, Appendix X.

- A soil class of D (in accordance with NZS 1170.5:2004) should be adopted for the site.

9. Results

9.1 Summary of Results

The New Zealand Society for Earthquake Engineering (NZSEE) publication “Assessment & Improvement of Structural Performance of Buildings” (2006, Ref. b) and the relevant New Zealand material standards were used to provide a framework and method for the analysis.

Our analysis applied live loads, super imposed dead loads and seismic loads to the structure. The elements from Ground to the Roof level of the structure were then assessed against their respective load capacities.

The outcome of the three-dimensional model analysis and demand/capacity assessment is summarised in Table 5. Note that the values given represent the critical elements in the building, as these effectively define the building’s capacity. Other elements within the building will have significantly greater capacity when compared with the governing elements.

A diagrammatic plan is shown in Figure 5.

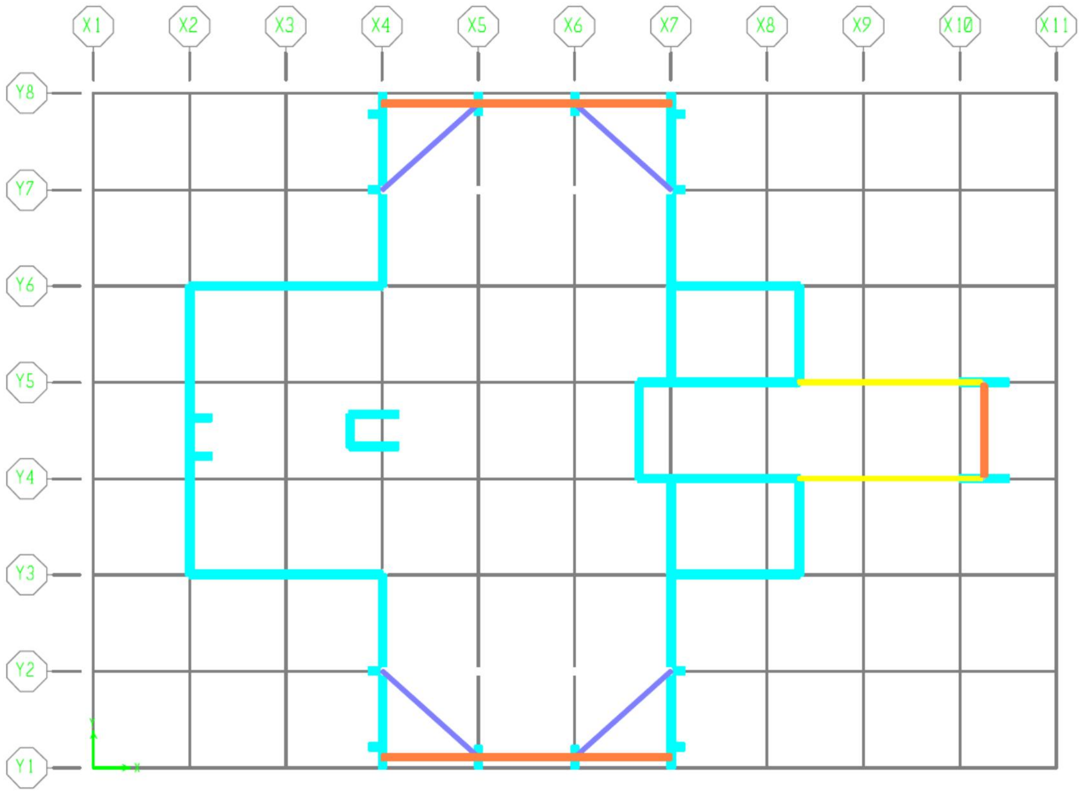


Figure 9-1: Plan Showing Gridlines

Table 9-1: Existing Building Element to % NBS

Level	Direction	Element	(% NBS)
Ground – Roof Level	Transverse	Masonry Walls	82%
		Timber Beams	-
		Steel Beams	>100%
		Steel Rafters	>100%
		Steel Columns	60%
	Longitudinal	Masonry Walls	51%
		Timber Beams	>100%
		Steel Beams	42%
		Steel Rafters	>100%
		Steel Columns	60%

9.1.1 Steel Rafters

All steel rafters are more than 100% NBS.

Key Drawings is located on Appendix D.

9.1.2 Steel Beams

Based on the analysis, the steel beams in the longitudinal direction were assessed to be having the lowest NBS score of 42% on Gridline Y2 and Gridline Y7. In the transverse direction, steel beams achieve ratings greater than 100% NBS. There are substantial numbers of steel beams that achieved NBS scores less than 67% NBS in the longitudinal direction. These fall within the “Earthquake Risk” category.

Key Drawings is located on Appendix D.

9.1.3 Timber Beams

All timber beams are more than 100% NBS.

Key Drawings is located on Appendix D.

9.1.4 Steel Columns

The steel columns in the longitudinal and transverse direction were assessed to have an NBS score of 60% on gridline Y7X4 and Y3X7.

Key Drawings is located on Appendix D.

9.1.5 Masonry Walls

Calculations showed that the masonry block walls in the longitudinal direction achieved a rating of 51% NBS on Gridline Y2 and Gridline Y7. There are substantial numbers of masonry walls that achieved NBS scores less than 67% NBS in the longitudinal direction. In the transverse direction, masonry walls achieve ratings greater than 67% NBS. These fall within the “Earthquake Risk” category.

Key Drawings is located on Appendix D.

9.1.6 Foundations

Due to the absence of ground damage and the non-susceptibility of the ground to liquefy, the foundations are not considered to be an “Earthquake Risk” for this building.

10. Conclusions

Our detailed seismic assessment shows that the overall building achieves 42% NBS. The building is therefore classified as an “Earthquake Risk”. A building with % NBS score in the range 34% to 67% NBS is between 5 to 10 times more likely than a similar building constructed to current loading standards to cause loss of life or serious injury during a seismic event.

Christchurch City Council should consider strengthening the building to 67% NBS, i.e. beyond Earthquake Risk.

11. Recommendations

Based from the results acquired in the quantitative analysis performed, the following recommendations are made:

- The current placard status of green remains.
- The building can still be used, as per CCC's policy to occupy "Earthquake Risk" buildings.
- A strengthening scheme is developed to increase the seismic capacity of the building to at least 67% NBS.
- Minor repairs are undertaken to fill the minor cracks identified at the top of the concrete masonry piers.

12. Limitations

12.1 General

This report has been prepared subject to the following limitations:

- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.
- This report is prepared for CCC to assist with assessing the remedial works required for council buildings and facilities. It is not intended for any other party or purpose.

12.2 Scope and Limitations of Geotechnical Investigation

The data and advice provided herein relate only to the project and structures described herein and must be reviewed by a competent geotechnical engineer before being used for any other purpose. GHD Limited (GHD) accepts no responsibility for other use of the data by third parties.

Where drill hole or test pit logs, cone tests, laboratory tests, geophysical tests and similar work have been performed and recorded by others under a separate commission, the data is included and used in the form provided by others. The responsibility for the accuracy of such data remains with the issuing authority, not with GHD.

The advice tendered in this report is based on information obtained from the desk study investigation location test points and sample points. It is not warranted in respect to the conditions that may be encountered across the site other than at these locations. It is emphasised that the actual characteristics of the subsurface materials may vary significantly between adjacent test points, sample intervals and at locations other than where observations, explorations and investigations have been made. Subsurface conditions, including groundwater levels and contaminant concentrations can change in a limited time. This should be borne in mind when assessing the data.

It should be noted that because of the inherent uncertainties in subsurface evaluations, changed or unanticipated subsurface conditions may occur that could affect total project cost and/or execution. GHD does not accept responsibility for the consequences of significant variances in the conditions and the requirements for execution of the work.

The subsurface and surface earthworks, excavations and foundations should be examined by a suitably qualified and experienced Engineer who shall judge whether the revealed conditions accord with both the assumptions in this report and/or the design of the works. If they do not accord, the Engineer shall modify advice in this report and/or design of the works to accord with the circumstances that are revealed.

An understanding of the geotechnical site conditions depends on the integration of many pieces of information, some regional, some site specific, some structure specific and some experienced based. Hence this report should not be altered, amended or abbreviated, issued in part and issued incomplete in any way without prior checking and approval by GHD. GHD accepts no responsibility for any circumstances which arise from the issue of the report which have been modified in any way as outlined above.

We trust the enclosed is acceptable. Please do not hesitate to contact the undersigned with any questions you may have.

13. References

- Drawings for Wigram Aerodrome – Harvard Lounge preferred by Dykes and Steven and Young Associates.
- Wigram Aerodrome – Harvard Lounge, BU 2556-002 EQ2, Detailed Engineering Evaluation, Qualitative Report, Version Draft; February 27, 2012, GHD Pty Ltd. - Christchurch

New Zealand Standard

- NZS 1170.0:2002 Structural Design Actions Part 0: General Principles
- NZS 1170.1:2002 Structural Design Actions Part 1: Permanent, Imposed and Other Actions
- NZS 1170.1:Supplement 1:2002 Structural Design Actions: Permanent, Imposed and Other Actions-Commentary
- NZS 1170.5:2004 Structural Design Actions Part 5: Earthquake Actions – New Zealand and NZBC Clause B1 Structure.
- NZS 3404: Part 1: 1997 Steel Structures Standard
- NZS 4230: 2004 Design of Reinforced Concrete Masonry Structures
- NZS 3603:1993 Timber Structures Standard
- New Zealand Society of Earthquake Engineering Guidelines for Assessment and Improvement of the Structural Performance of Buildings in Earthquake

Appendix A

Geotechnical Investigation Results and Analysis

CPT ANALYSIS NOTES

Soil Type

Interpretation using chart of Robertson & Campanella (1983). This is a simple but well proven interpretation using cone tip resistance (q_C) and friction ratio (f_R) only. No normalisation for overburden stress is applied. Cone tip resistance measured with the piezocone is corrected with measured pore pressure (u_C).

- sand (and gravel)
- silt-sand
- silt
- clay-silt
- clay
- peat

Liquefaction Screening

The purpose of the screening is to highlight susceptible soils, that is sand and silt-sand in a relatively loose condition. This is not a full liquefaction risk assessment which requires knowledge of the particular earthquake risk at a site and additional analysis. The screening is based on the chart of Shibata and Teparaksa (1988).

- high susceptibility
- medium susceptibility
- low susceptibility

High susceptibility is here defined as requiring a shear stress ratio of 0.2 to cause liquefaction with D_{50} for sands assumed to be 0.25 mm and for silty sands to be 0.05 mm.

Medium susceptibility is here defined as requiring a shear stress ratio of 0.4 to cause liquefaction with D_{50} for sands assumed to be 0.25 mm and for silty sands to be 0.05 mm.

Low susceptibility is all other cases.

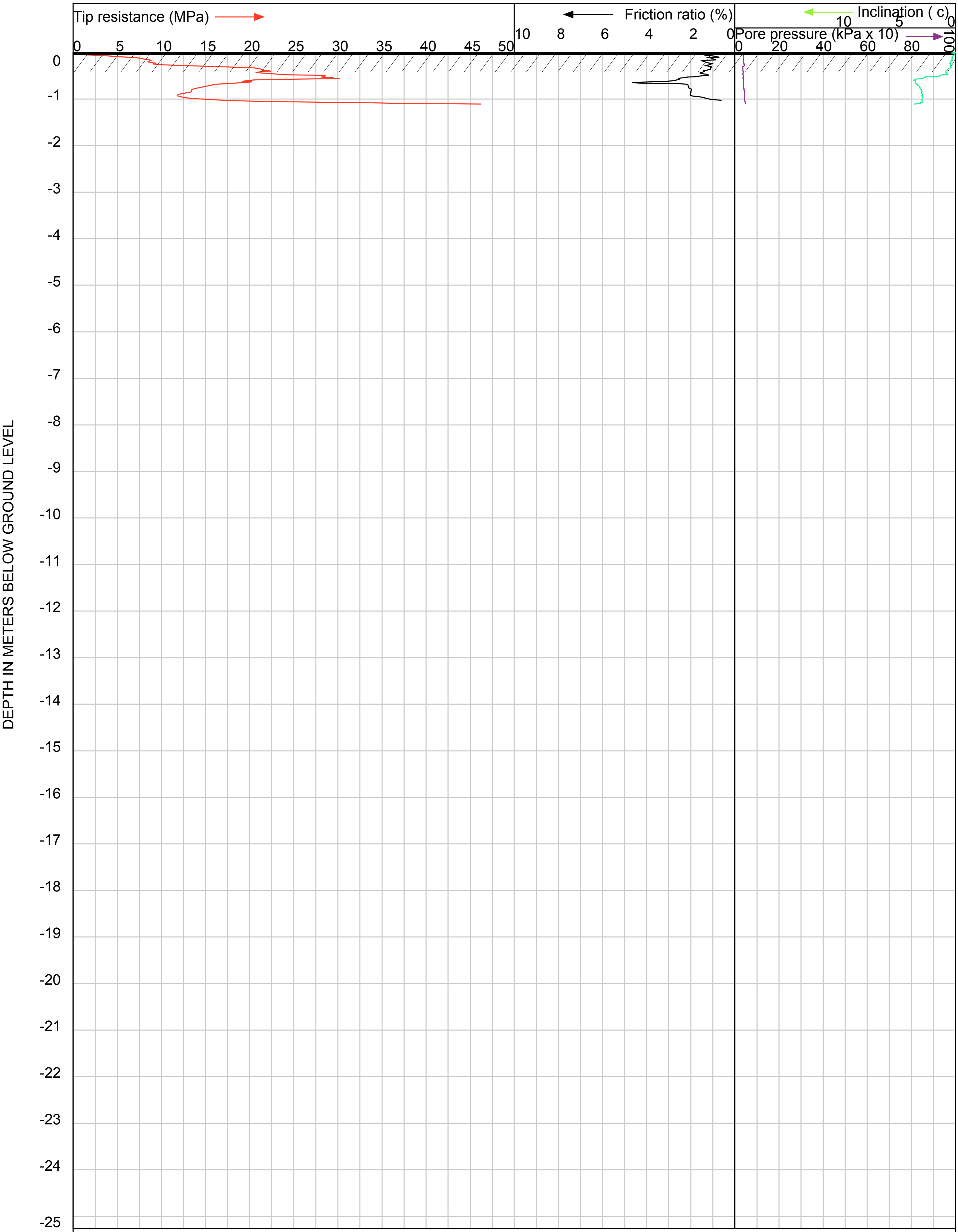
Relative Density (D_R)

Based on the method of Baldi et. al. (1986) from data on normally consolidated sand.

Undrained Shear Strength (S_U)

Derived from the bearing capacity equation using $S_U = (q_C - \sigma_{VO})/15$.

McMILLAN
DRILLING SERVICES

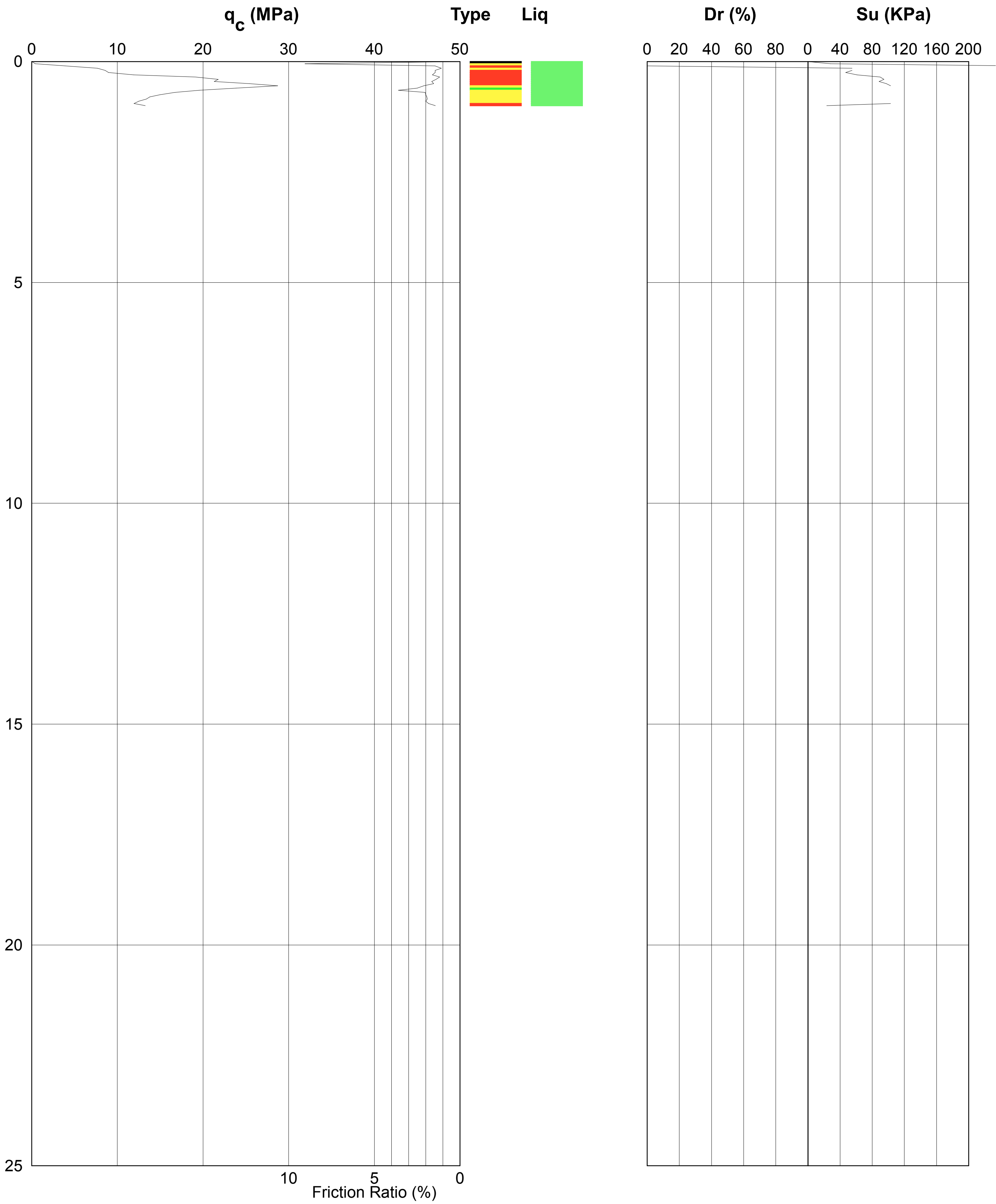


CLIENT : GHD
LOCATION : Christchurch Various (CCC Properties)
DATE : 2-4-2012
OPERATOR : H. Pardoe
REMARK 1 : CPTu24
REMARK 2 : Effective Refusal

JOB # : 10386
TEST # : 24

McMILLAN DRILLING SERVICES
120 High St Southbridge CANTERBURY NZ
Ph +64 3 324 2571 Fax +64 3 324 2431
www.drilling.co.nz

PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No:	10386	Date:	2-4-2012
CPT No:	CPTu24	Operator:	H. Pardoe
Project:	GHD	Remark:	Effective Refusal
Location:	Christchurch Various (CCC Properties)		

McMILLAN DRILLING SERVICES

Appendix B
Photographs

Photograph 1 View of entrance canopy.

Photograph 2 View of the entrance from Corsair Drive.

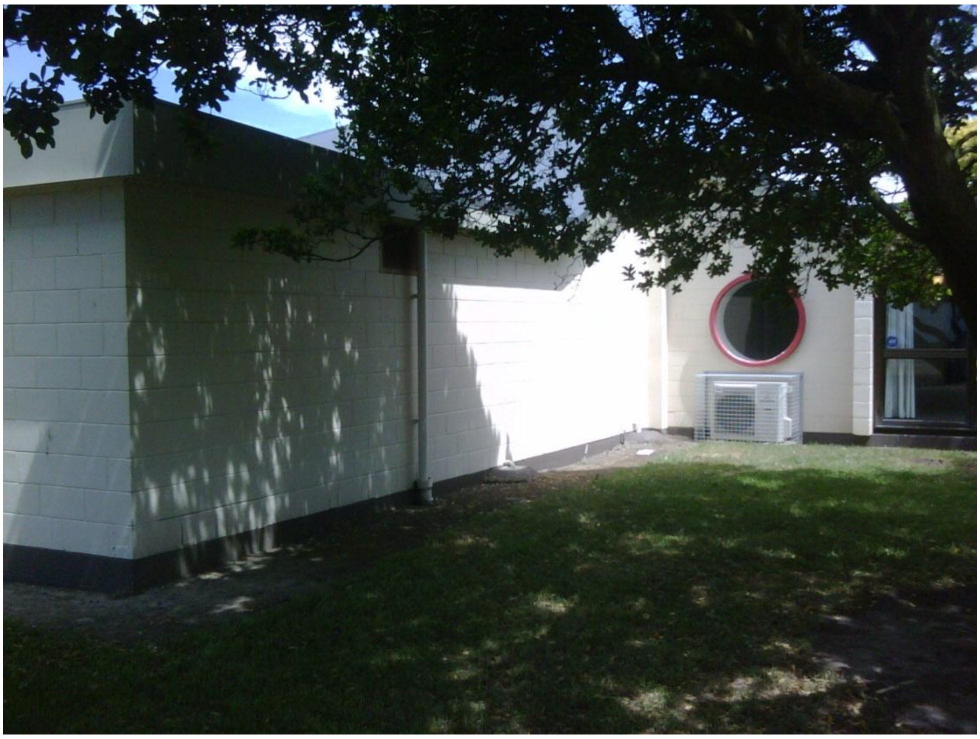
Photograph 3 **Connection at base of RHS posts supporting the entrance canopy.**

Photograph 4 **View looking east.**

Photograph 5 **Cracking between the external cladding and the top the concrete masonry piers.**

Photograph 6 **View from the south.**

Photograph 7 External concrete masonry piers.



Photograph 8 Concrete masonry walls on the north-west face of the building.

Photograph 9 **Further cracking between the external cladding and the top the concrete masonry piers.**

Photograph 10 **Internal view of lounge area and concrete masonry piers.**

Photograph 11 Steel RHS post supporting the steel rafters.

Photograph 12 View from the north

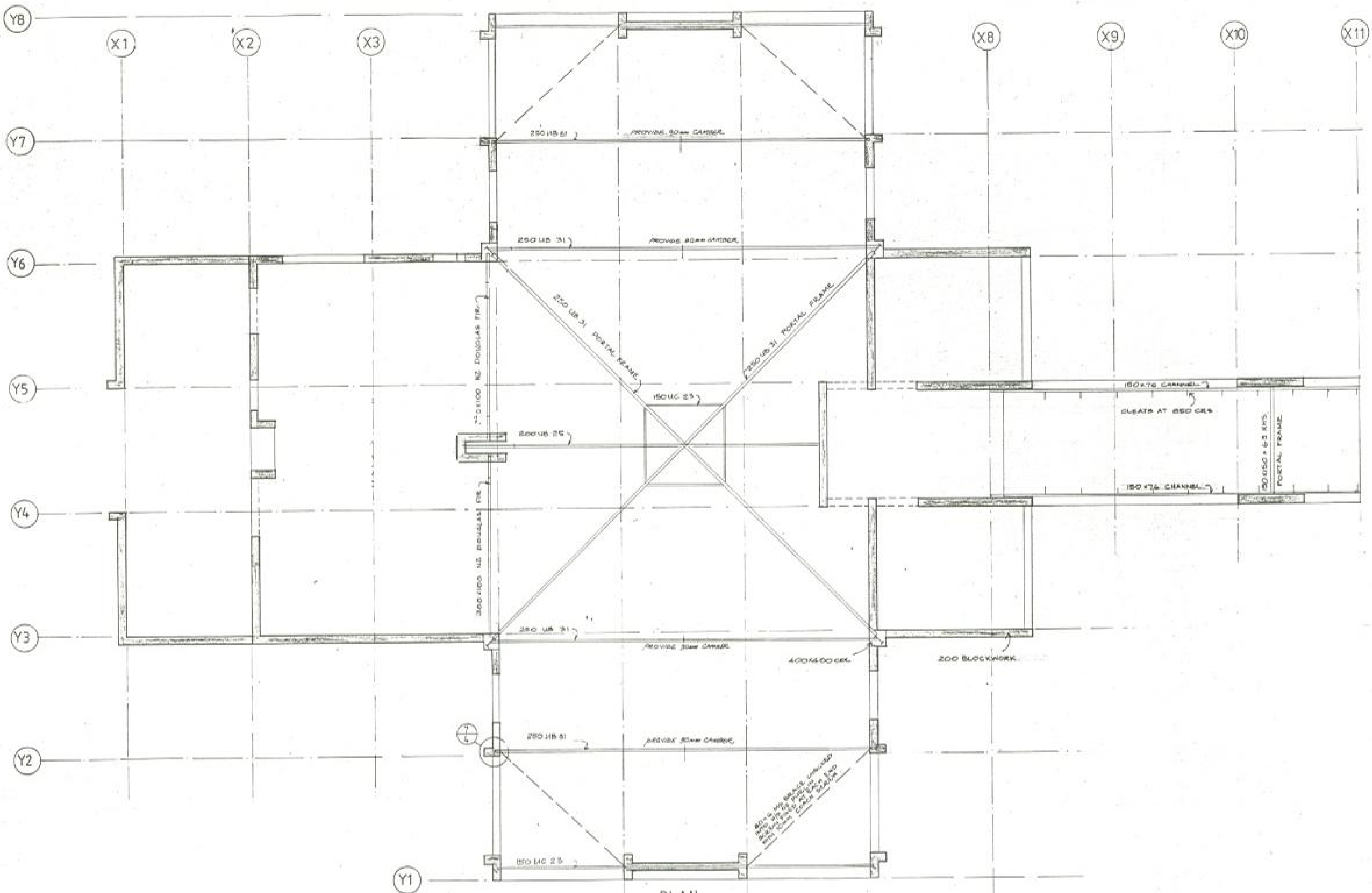
Photograph 13

Photograph 14 View at entrance canopy

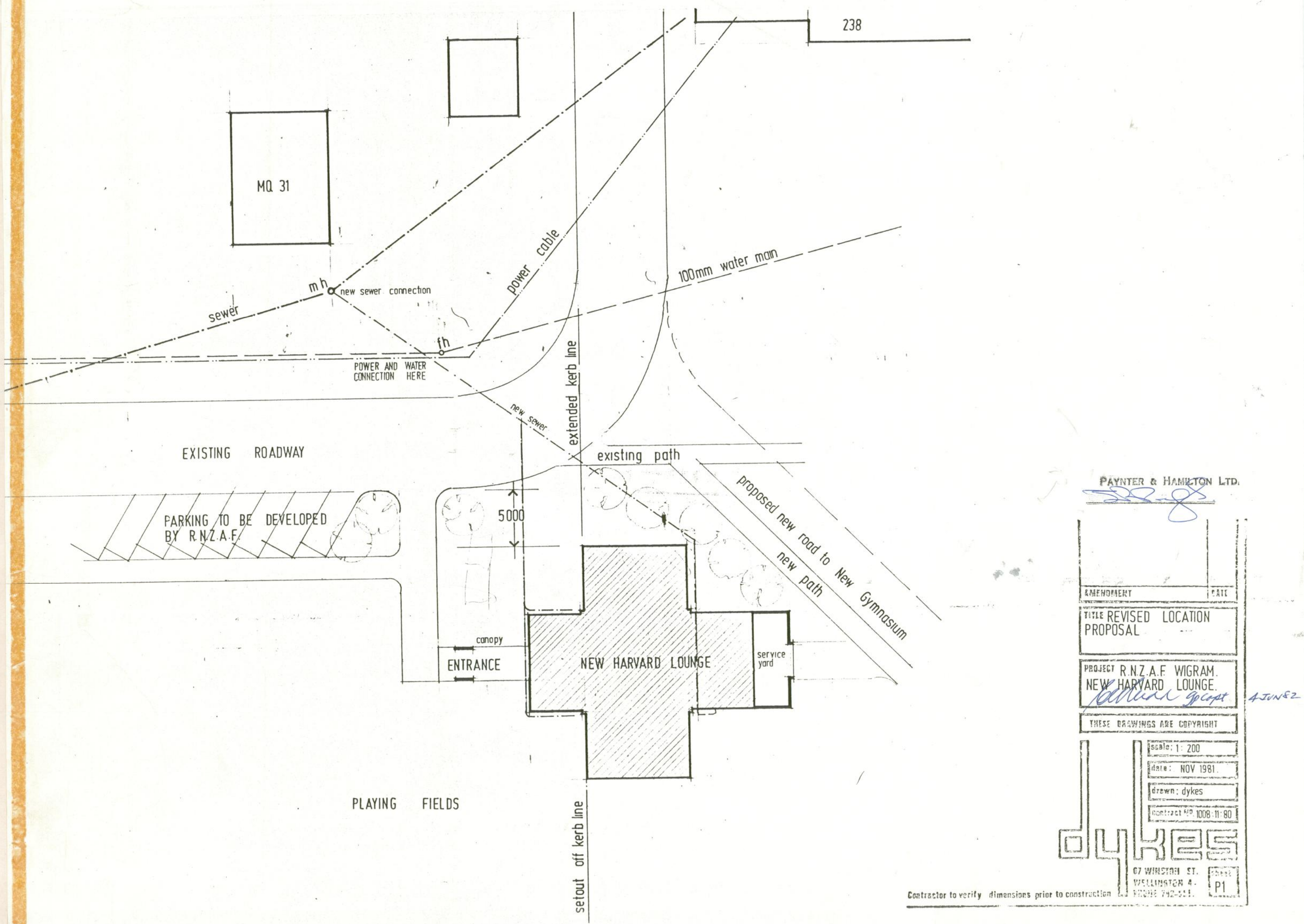
Photograph 15 External concrete masonry piers

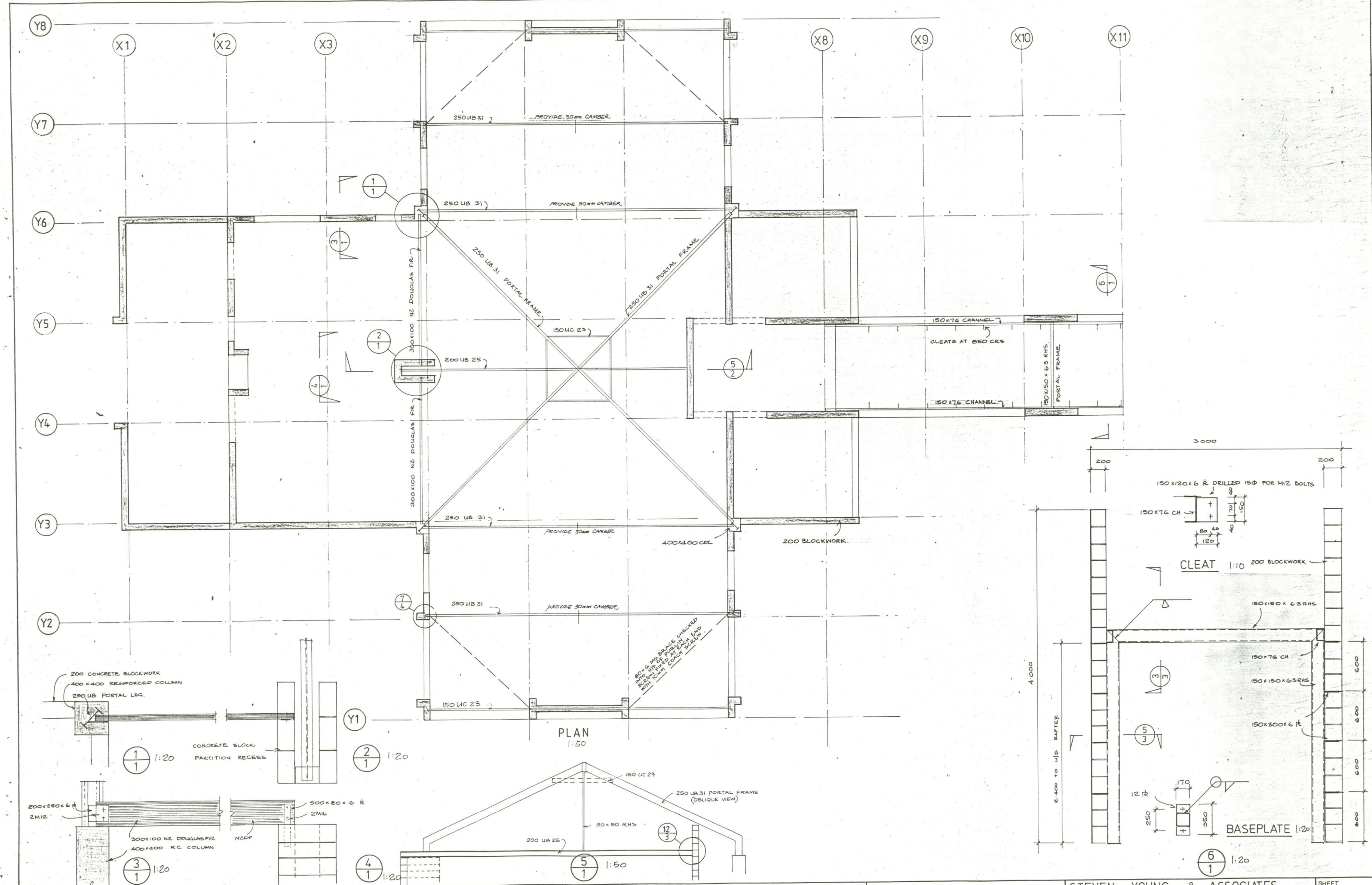
Appendix C

Original Drawings


Y8
Y7
Y6
Y5
Y4
Y3
Y2
Y1
X1
X2
X3
X8
X9
X10
X11
250 UB 31
PROVIDE 30mm CAMBER
250 UB 31 PORTAL FRAME
150 UC 23
200 UB 25
NZ DOUGLAS FIR
300x100 NZ DOUGLAS FIR
150x76 CHANNEL
CLEATS AT 850 CRS
150x150 + 63 RHS PORTAL FRAME
400x400 CRS
200 BLOCKWORK
150 UC 23

PLAN
1:50


238
MQ 31
sewer
m h
new sewer connection
power cable
100mm water main
fh
POWER AND WATER
CONNECTION HERE
new sewer
extended kerb line
EXISTING ROADWAY
existing path
proposed new road to New Gymnasium
new path
PARKING TO BE DEVELOPED
BY R.N.Z.A.F.
5000
canopy
ENTRANCE
NEW HARVARD LOUNGE
service
yard
PLAYING FIELDS
setout off kerb line
PAYNTER & HAMILTON LTD.
AMENDMENT
DATE
TITLE REVISED LOCATION
PROPOSAL
PROJECT R.N.Z.A.F. WIGRAM.
NEW HARVARD LOUNGE.
Gpcapt
4 JUN 82
THESE DRAWINGS ARE COPYRIGHT
scale: 1: 200
date: NOV 1981.
drawn: dykes
contract No. 1008:11:80
dykes
67 WINSTON ST.
WELLINGTON 4.
PHONE 742-551.
sheet
P1
Contractor to verify dimensions prior to construction



REVISION A

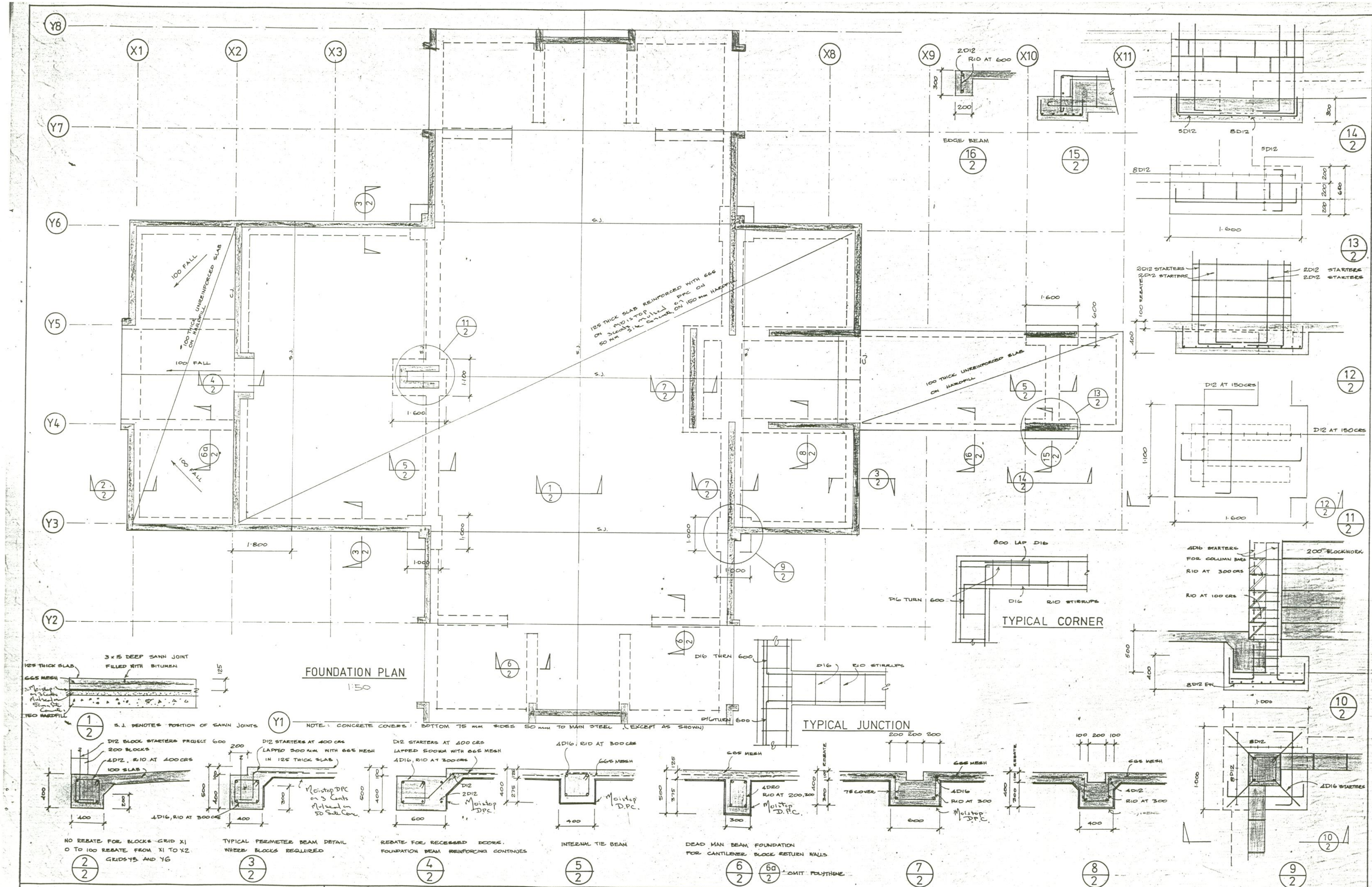
STEVEN YOUNG & ASSOCIATES
CONSULTING ENGINEERS
THIRD FLOOR - MUTUAL BLDG
27 DIXON ST WELLINGTON
P O BOX 6168
TEL 850 190

SHEET S1A

PLAN & GENERAL DETAILS

HARVARD LOUNGE RNZAF BASE WIGRAM

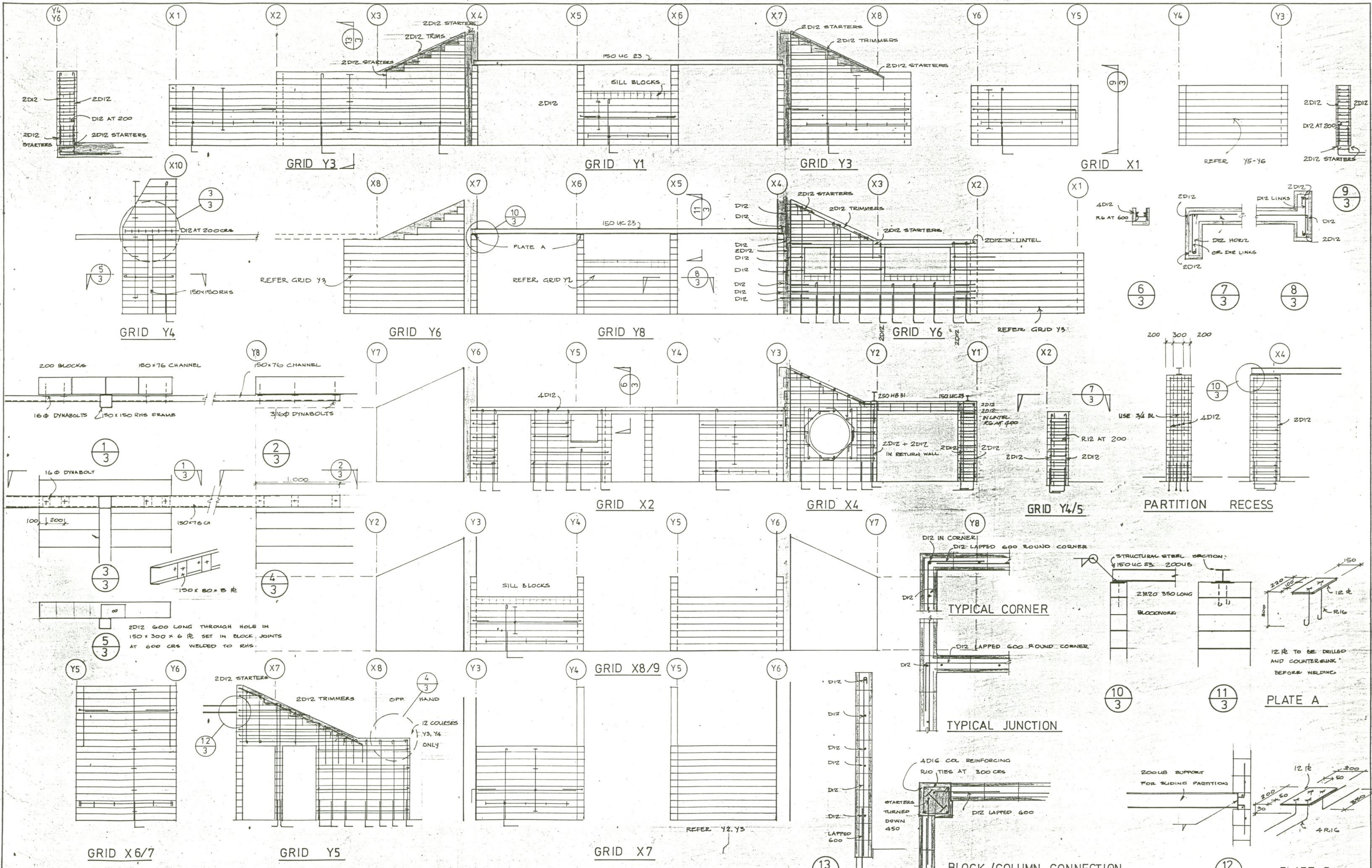
DRAWN S.Y.	CHECKED	SCALES	SERIES OF
TRACED	DATE 20:11:80	1: 20, 50, 10	REF



FOUNDATION PLAN & DETAILS	HARVARD LOUNGE RNZAF BASE WIGRAM	REVISION A	SHEET S2A

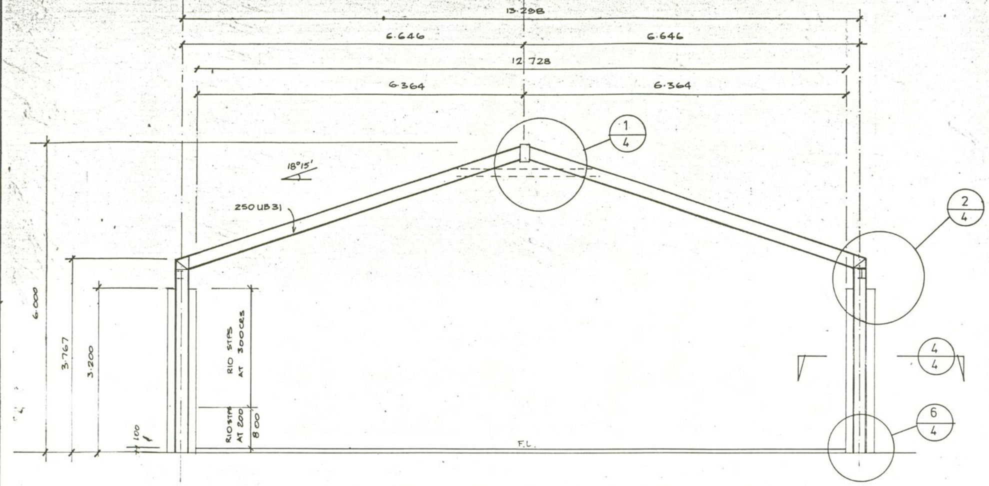
STEVEN YOUNG & ASSOCIATES
CONSULTING ENGINEERS
THIRD FLOOR - MUTUAL BLDG
27 DIXON ST WELLINGTON
P O BOX 6168
TEL 850 190

DRAWN	CHECKED	SCALES	SERIES OF
SY	DATE 20 11 80	1: 20,50	
TRACED			REF

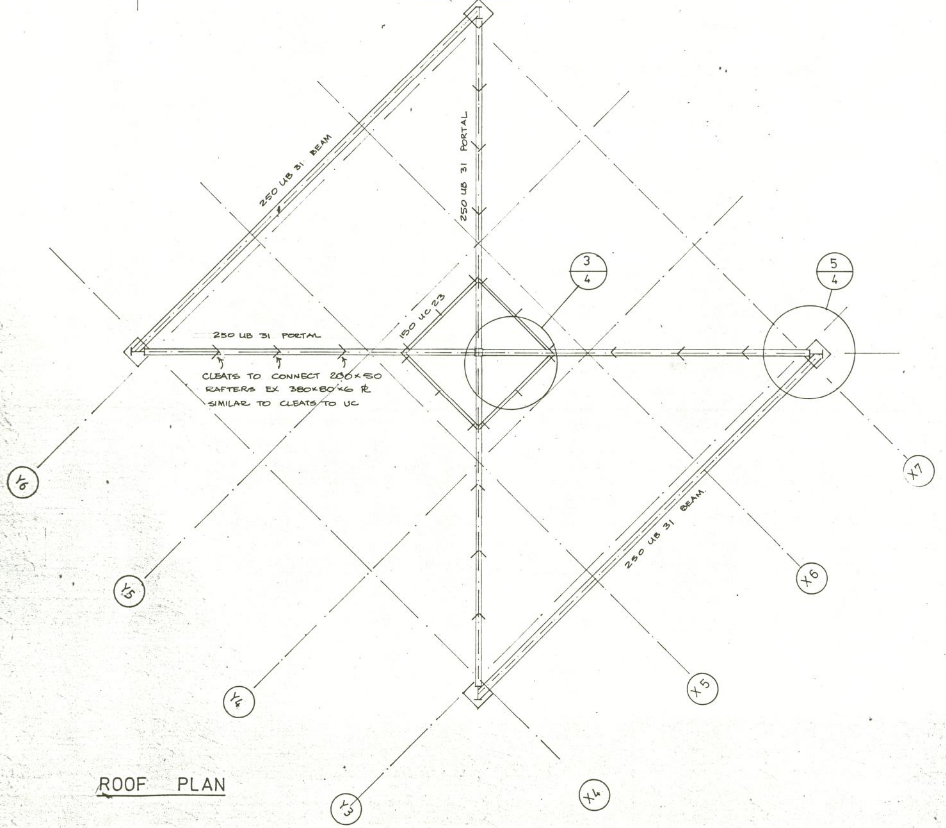


NOTE: ALL REINFORCING D12 BARS UNLESS SHOWN OTHERWISE - GENERALLY AT 600CRS EACH WAY, ALSO 400 & 200 CRS WHERE SHOWN
BLOCKWORK: ALL CELLS FILLED WITH GROUT.

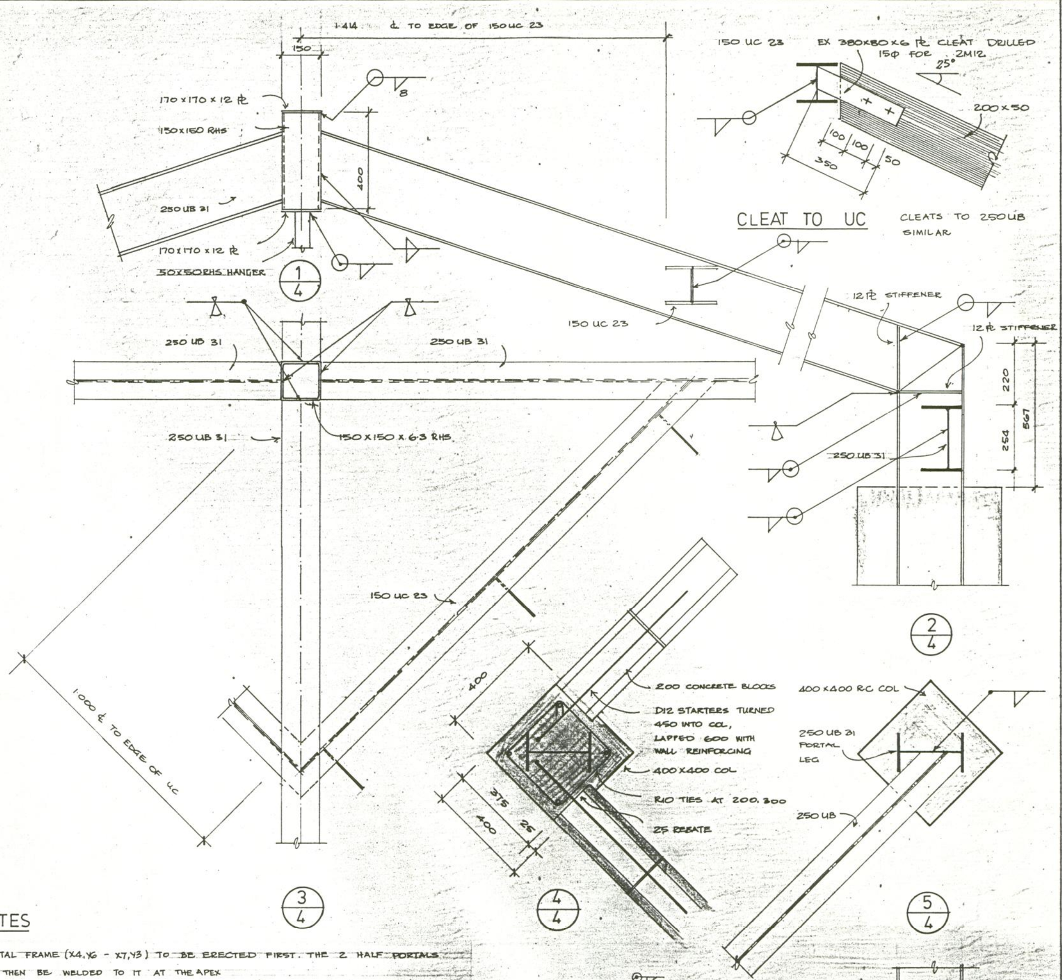
BLOCKWORK	HARVARD LOUNGE RNZAF BASE WIGRAM	REVISION A	STEVEN YOUNG & ASSOCIATES CONSULTING ENGINEERS THIRD FLOOR - MUTUAL BLDG 27 DIXON ST. WELLINGTON P O BOX 6168 TEL 850 190	SHEET S3A
			DRAWN SY / TRACED / CHECKED / DATE 20 11 80 / SCALES 1: 20, 50	SERIES OF / REF



FRAME (X4,Y6 - X7,Y3) FRAME (X4,Y3 - X7,Y6) SIMILAR

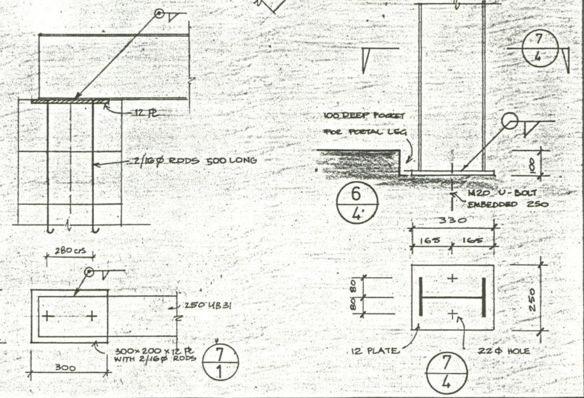


ROOF PLAN

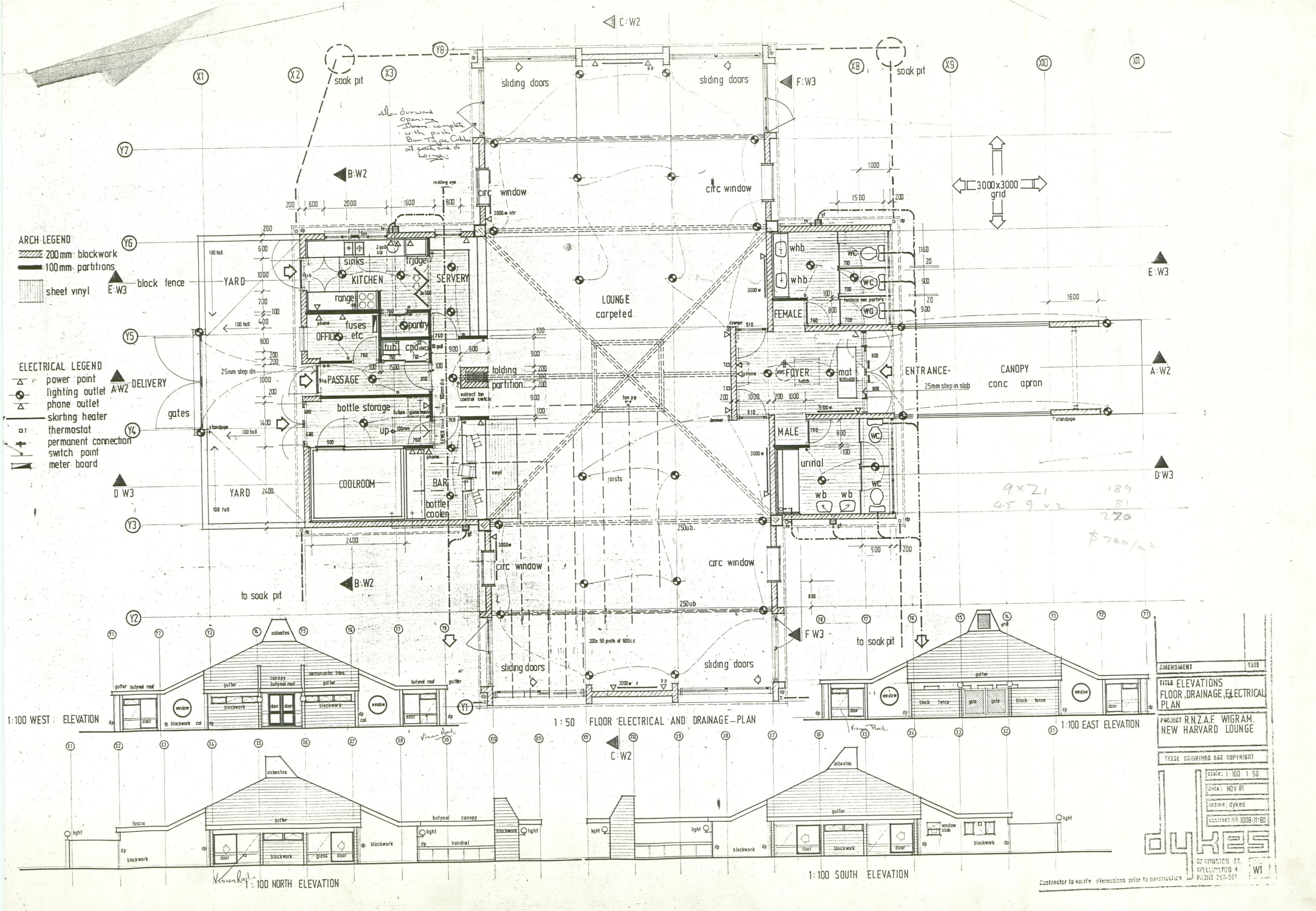


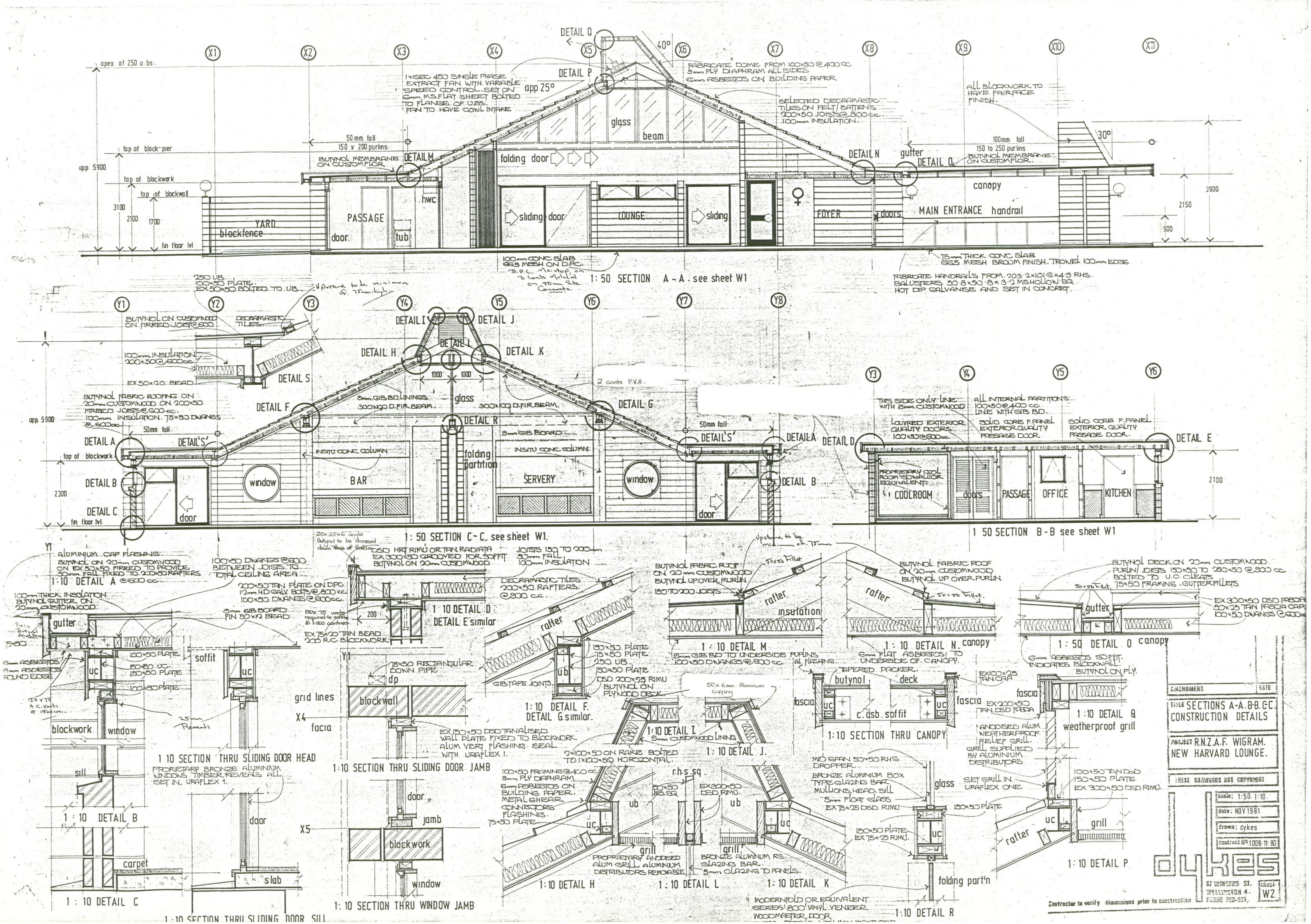
NOTES

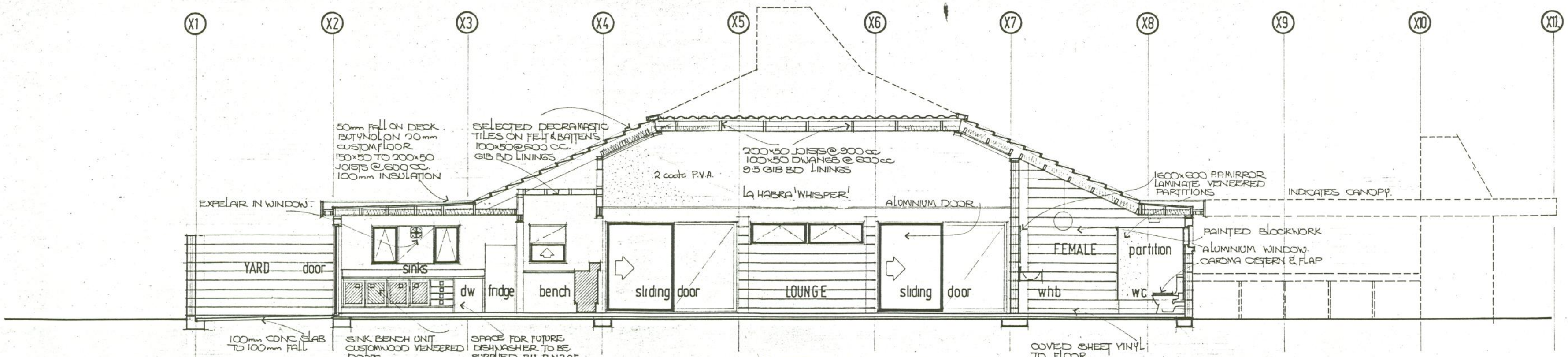
1. PORTAL FRAME (X4,Y6 - X7,Y3) TO BE ERECTED FIRST. THE 2 HALF PORTALS MAY THEN BE WELDED TO IT AT THE APEX
2. THE HORIZONTAL STIFFENERS AT THE KNEE JOINT TO BE SITE WELDED AFTER 250 UB 31 BEAMS ERECTED. THIS WILL FACILITATE MAKING THE 250 UB TO PORTAL LEG CONNECTION AFTER LOCATING AT REQUIRED LEVEL.
3. CLEATS TO CONNECT 200 x 50 RAFTERS EX 380 x 80 x 6 PL SIMILAR TO "CLEAT TO UC". CLEATS AT 45° TO AXIS OF RAFTER IN PLAN, 25° TO HORIZONTAL (PARALLEL TO 200 x 50 RAFTERS)
4. SECTION OF PORTAL LEG TO BE ENCASED IN CONCRETE NOT TO BE PAINTED



STRUCTURAL STEELWORK	HARVARD LOUNGE RNZAF BASE WIGRAM	REVISION A	STEVEN YOUNG & ASSOCIATES CONSULTING ENGINEERS THIRD FLOOR - MUTUAL BLDG 27 DIXON ST WELLINGTON P O BOX 6168 TEL 850 190	SHEET S4A
			DRAWN SY / TRACED / CHECKED / DATE 20 11 80 / SCALES 1:10, 50	SERIES OF / REF


ARCH LEGEND
200mm blockwork
100mm partitions
sheet vinyl
ELECTRICAL LEGEND
power point
lighting outlet
phone outlet
skirting heater
thermostat
permanent connection
switch point
meter board
soak pit
sliding doors
circ window
LOUNGE carpeted
KITCHEN
SERVERY
sinks
fridge
range
pantry
OFFICE
fuses etc
PASSAGE
bottle storage
COOLROOM
BAR
bottle cooler
folding partition
YARD
block fence
DELIVERY
gates
FEMALE
MALE
whb
wc
urinal
wb
FOYER
mat
ENTRANCE
CANOPY
conc apron
25mm step in slab
3000x3000 grid
to soak pit
B:W2
C:W2
F:W3
E:W3
A:W2
D:W3
1:100 WEST ELEVATION
1:50 FLOOR ELECTRICAL AND DRAINAGE-PLAN
1:100 EAST ELEVATION
1:100 NORTH ELEVATION
1:100 SOUTH ELEVATION
Vision Rail
TITLE ELEVATIONS FLOOR, DRAINAGE, ELECTRICAL PLAN
PROJECT R.N.Z.A.F. WIGRAM. NEW HARVARD LOUNGE
THESE DRAWINGS ARE COPYRIGHT
scale: 1:100 1:50
date: NOV 81
drawn: dykes
contract No 1008:11:80
dykes
W1
Contractor to verify dimensions prior to construction
9 x 21
4.5 9 x 2
189
81
270
$ 740/m²


1: 50 SECTION A - A . see sheet W1
1: 50 SECTION C - C. see sheet W1.
1 50 SECTION B - B see sheet W1
DETAIL Q
DETAIL P
DETAIL M
DETAIL N
DETAIL O
apex of 250 u.bs.
app 25°
40°
30°
1×SEC 450 SINGLE PHASE EXTRACT FAN WITH VARIABLE SPEED CONTROL. SET ON 6mm M.S.FLAT SHEET BOLTED TO FLANGE OF U.B.S. FAN TO HAVE COWL INTAKE
FABRICATE DOME FROM 100×50 @400 cc 9mm PLY DIAPHRAM ALL SIDES 6mm ASBESTOS ON BUILDING PAPER
SELECTED DECRAMASTIC TILES ON FELT/BATTENS 200×50 JOISTS @ 300 cc. 100mm INSULATION.
ALL BLOCKWORK TO HAVE FAIRFACE FINISH.
50mm fall
150 x 200 purlins
100mm fall
150 to 250 purlins
BUTYNOL MEMBRANE ON CUSTOMFLOR.
top of block pier
top of blockwork
top of blockwall
fin floor lvl
YARD
blockfence
PASSAGE
door
hwc
tub
folding door
glass
beam
sliding door
LOUNGE
sliding
FOYER
doors
MAIN ENTRANCE handrail
canopy
gutter
100mm CONC SLAB 665 MESH ON D.P.C.
75mm THICK CONC SLAB 665 MESH BROOM FINISH. TROWEL 100mm EDGE
FABRICATE HANDRAILS FROM 203.2×101.6×4.9 RHS. BALUSTERS 50.8×50.8×3.2 MS HOLLOW SQ. HOT DIP GALVANISE AND SET IN CONCRETE.
250 UB 100×50 PLATE EX 50×50 BOLTED TO UB.
DETAIL I
DETAIL J
DETAIL L
DETAIL H
DETAIL K
DETAIL S
DETAIL F
DETAIL G
DETAIL R
DETAIL A
DETAIL'S'
DETAIL B
DETAIL C
DETAIL D
DETAIL E
BUTYNOL ON CUSTOMWOOD ON FIRRED JOISTS @ 600
DECRAMASTIC TILES
100mm INSULATION 200×50 @ 600 cc.
EX 50×20 BEAD
BUTYNOL FABRIC ROOFING ON 20mm CUSTOMWOOD ON 200×50 FIRRED JOISTS @ 600 cc. 100mm INSULATION. 75×50 DWANGS @ 600 cc.
9mm GIB BD LININGS 300×100 D.FIR BEAM.
300×100 D.FIR BEAM.
2 coats P.V.A.
9mm GIB BOARD
INSITU CONC COLUMN
folding partition
window
BAR
SERVERY
door
THIS SIDE ONLY LINE WITH 8mm CUSTOMWOOD
ALL INTERNAL PARTITIONS 100×50 @ 400 cc LINE WITH GIB BD.
LOUVRED EXTERIOR QUALITY DOORS 100×50 @ 600 cc.
SOLID CORE F.PANEL EXTERIOR QUALITY PASSAGE DOOR
SOLID CORE F.PANEL EXTERIOR QUALITY PASSAGE DOOR.
PROPRIETARY COOL ROOM SDWALLOR EQUIVALENT.
COOLROOM
PASSAGE
OFFICE
KITCHEN
ALUMINIUM CAP FLASHING. BUTYNOL ON 20mm CUSTOMWOOD ON EX 50×50 FIRRED TO PROVIDE 50mm FALL FIXED TO 200×50 RAFTERS
1:10 DETAIL A @ 600 cc.
100×50 DWANGS @ 600 BETWEEN JOISTS TO TOTAL CEILING AREA
100mm THICK INSULATION BUTYNOL GUTTER ON 20mm CUSTOMWOOD
gutter
9mm GIB BOARD FIN 50×12 BEAD
200×50 TAN PLATE ON DPC 12mm HD GALV BOLTS @ 800 cc 100×50 DWANGS @ 600 cc
6mm ASBESTOS 9mm ASBESTOS ROUND EDGE
150×50 PLATE
50×50 cc. 150×50 PLATE
100×50 PLATE
uc
soffit
blockwork
window
sill
1: 10 DETAIL B
carpet
1 : 10 DETAIL C
1 10 SECTION THRU SLIDING DOOR HEAD
PROPRIETARY BRONZE ALUMINIUM WINDOWS TIMBER REVEALS ALL SET IN URAFLEX 1.
slab
1:10 SECTION THRU SLIDING DOOR SILL
25×25×6 angle Butynol to be dressed down face of fascia
DSD HRT RIMU OR TAN RADIATA EX 300×50 GROOVED FOR SOFFIT BUTYNOL ON 20mm CUSTOMWOOD
JOISTS 150 TO 200mm 50mm FALL 100mm INSULATION
1 10 DETAIL D
DETAIL E similar
EX 75×20 TAN BEAD 200 R.C. BLOCKWORK.
grid lines
X4
blockwall
facia
75×50 RECTANGULAR DOWN PIPE
dp
EX 150×50 DSD TANALISED WALL PLATE FIXED TO BLOCKWORK ALUM VERT FLASHING. SEAL WITH URAFLEX 1.
1:10 SECTION THRU SLIDING DOOR JAMB
door
jamb
X5
blockwork
window
1: 10 SECTION THRU WINDOW JAMB
DECRAMASTIC TILES 200×50 RAFTERS @ 300 c.c.
rafter
150×50 PLATE 75×50 PLATE 250 UB 150×50 PLATE DSD 200×25 RIMU BUTYNOL ON PLYWOOD DECK.
GIBTAPE JOINTS
ub
1:10 DETAIL F. DETAIL G similar.
50×6mm Aluminium capping
1:10 DETAIL I 9mm CUSTOMWOOD LINING
1: 10 DETAIL J.
2×100×50 ON RAKE BOLTED TO 1×100×50 HORIZONTAL.
100×50 FRAMING @ 400 cc 9mm PLY DIAPHRAM 6mm ASBESTOS ON BUILDING PAPER. METAL SHEAR CONNECTORS FLASHING. 75×50 PLATE
r.h.s sq.
50×50 RHS SQ
EX 200×50 DSD RIMU.
ub
uc
grill
PROPRIETARY ANODISED ALUM GRILL ALUMINIUM DISTRIBUTORS REMOVABLE
BRONZE ALUMINIUM RS. GLAZING BAR. 5mm GLAZING TO PANELS.
1: 10 DETAIL H
1: 10 DETAIL L
1: 10 DETAIL K
BUTYNOL FABRIC ROOF ON 20mm CUSTOMWOOD BUTYNOL UP OVER PURLIN
150 TO 200 JOISTS
50×50 Fillet
rafter
insulation
1: 10 DETAIL M
9mm GIB BD TO UNDERSIDE PURLINS 100×50 DWANGS @ 600 cc.
BUTYNOL FABRIC ROOF ON 20mm CUSTOMWOOD BUTYNOL UP OVER PURLIN
1: 10 DETAIL N. canopy
6mm FLAT ASBESTOS TO UNDERSIDE OF CANOPY.
AL FLASHING
TAPERED PACKER.
butynol
deck
fascia
c.dsb.soffit
1:10 SECTION THRU CANOPY
EX 100×25 TAN CAP
EX 200×50 TAN DSD FASCIA
BUTYNOL DECK ON 20mm CUSTOMWOOD PURLIN/JOISTS 150×50 TO 250×50 @ 300 cc 75×50 FRAMING . GUTTER FILLETS
50×50 Fillet
EX 300×50 DSD FASCIA 50×25 TAN FASCIA CAP 100×50 DWANGS @ 600 cc
1: 50 DETAIL O canopy
6mm ASBESTOS SOFFIT. INDICATES BLOCKWALL. BUTYNOL ON PLY.
MID SPAN 50×50 RHS DROPPER.
BRONZE ALUMINIUM BOX TYPE GLAZING BAR MULLIONS, HEAD, SILL 5mm FLOAT GLASS EX 75×25 DSD RIMU
glass
150×50 PLATE
150×50 PLATE. EX 75×25 RIMU.
folding part'n
KODERNFOLD OR EQUIVALENT SERIES 800 VINYL VENEER WOODMASTER DOOR
1:10 DETAIL R
ANODISED ALUM WEATHERPROOF RELIEF GRILL. GRILL SUPPLIED BY ALUMINIUM DISTRIBUTORS
SET GRILL IN URAFLEX ONE
100×50 TAN DSD 150×50 PLATE EX 300×50 DSD RIMU.
1 : 10 DETAIL Q weatherproof grill
1: 10 DETAIL P
AMENDMENT
DATE
TITLE SECTIONS A-A.B-B.C-C. CONSTRUCTION DETAILS
PROJECT R.N.Z.A.F. WIGRAM. NEW HARVARD LOUNGE.
THESE DRAWINGS ARE COPYRIGHT
scale: 1:50. 1:10.
date: NOV 1981
drawn: dykes
contract NO. 1008.11.80
dykes
57 WINSTON ST. WELLINGTON 4. PHONE 792-551.
sheet W2
Contractor to verify dimensions prior to construction



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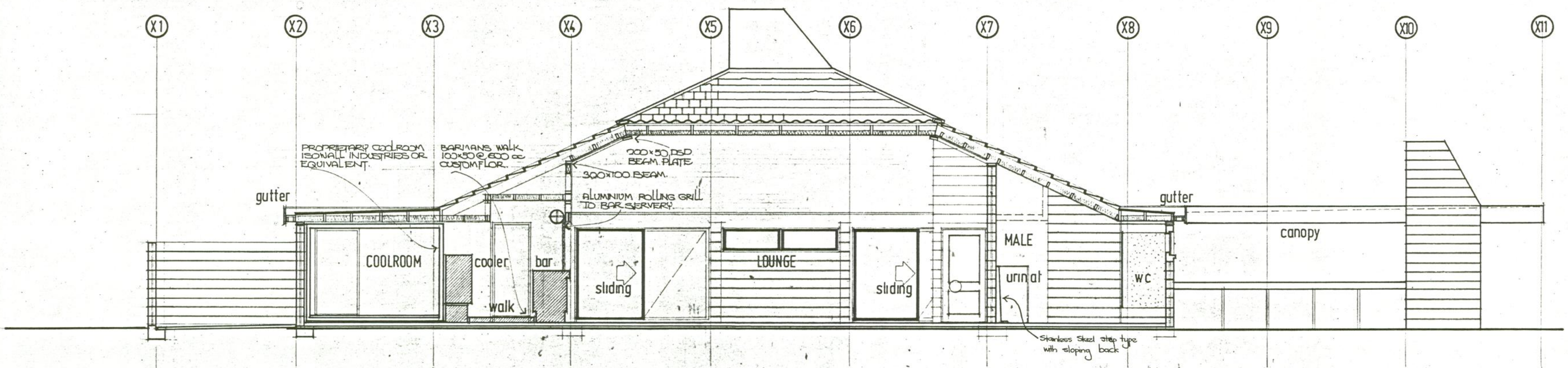

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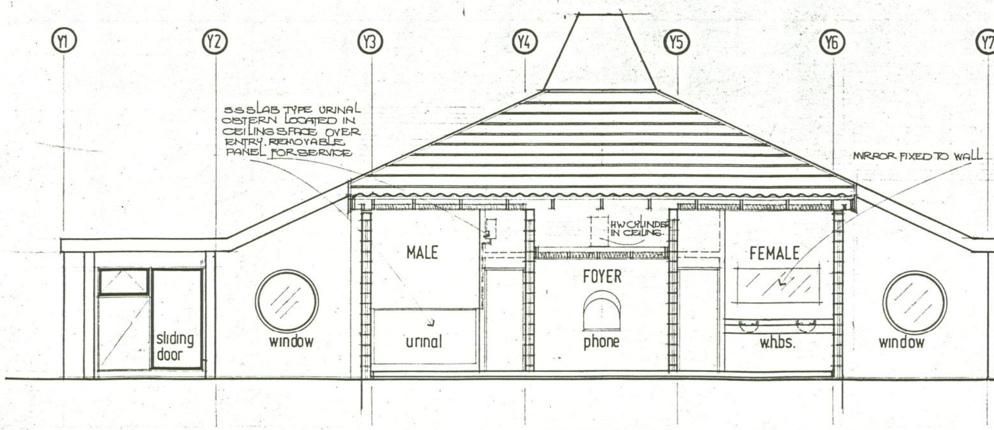

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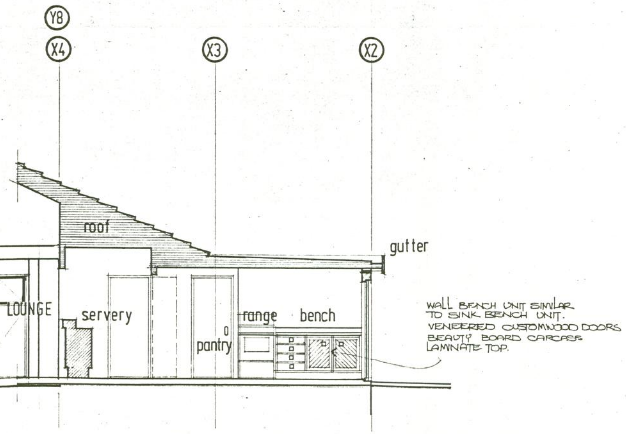


1 : 50 EAST ELEVATION KITCHEN

AMENDMENT	DATE

TITLE SECTIONS D-D. E-E. F-F. EAST ELEV KITCHEN JOINERY

PROJECT R.N.Z.A.F. WIGRAM NEW HARVARD LOUNGE .

scale: 1 : 50
date: NOV 81
drawn: dykes
contract No. 1008.11.80

dykes

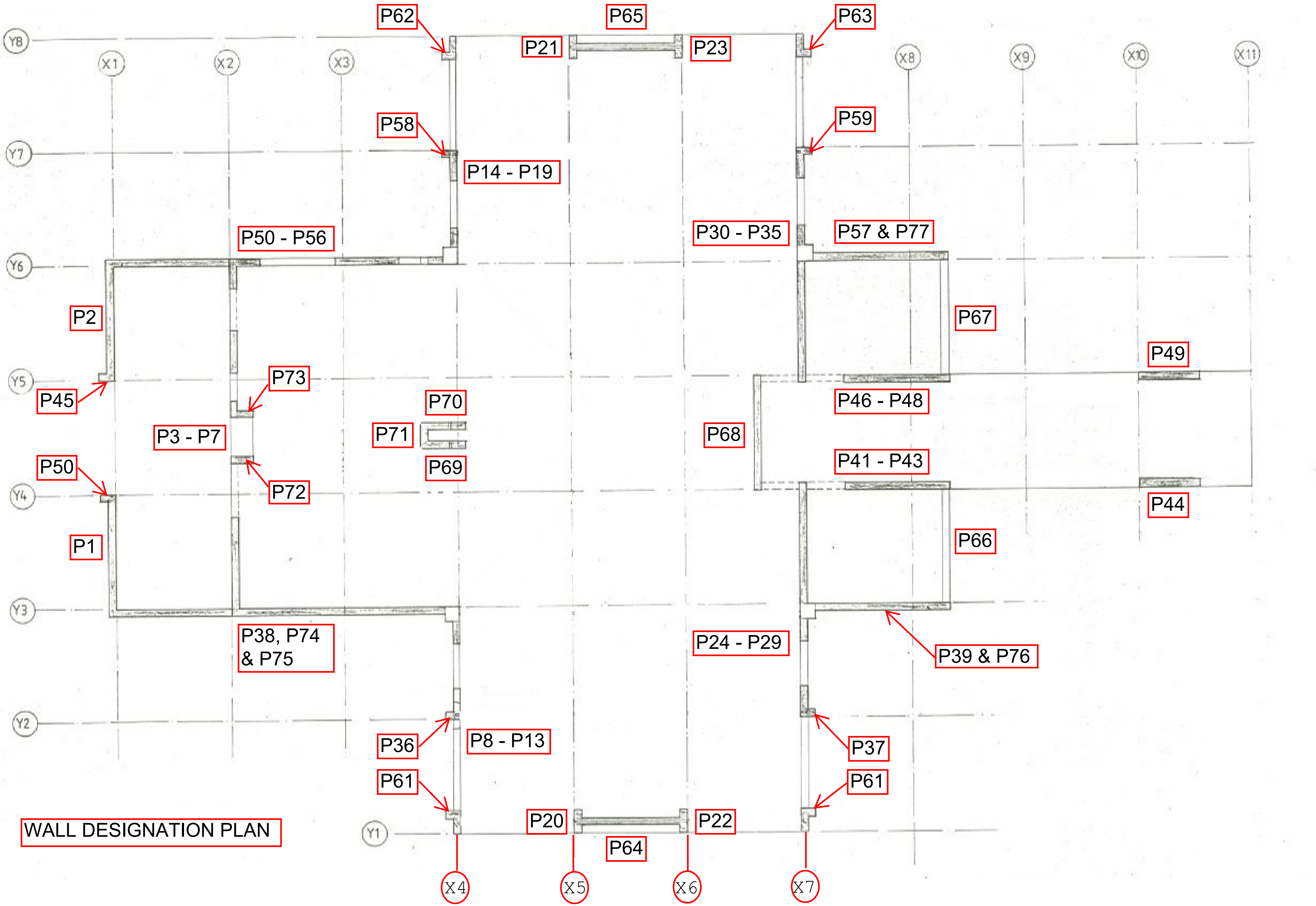
67 WINSTON ST. WELLINGTON 4. PHONE 757-551.

sheet W3

Contractor to verify dimensions prior to construction

Appendix D

Key Drawings



WALL DESIGNATION PLAN

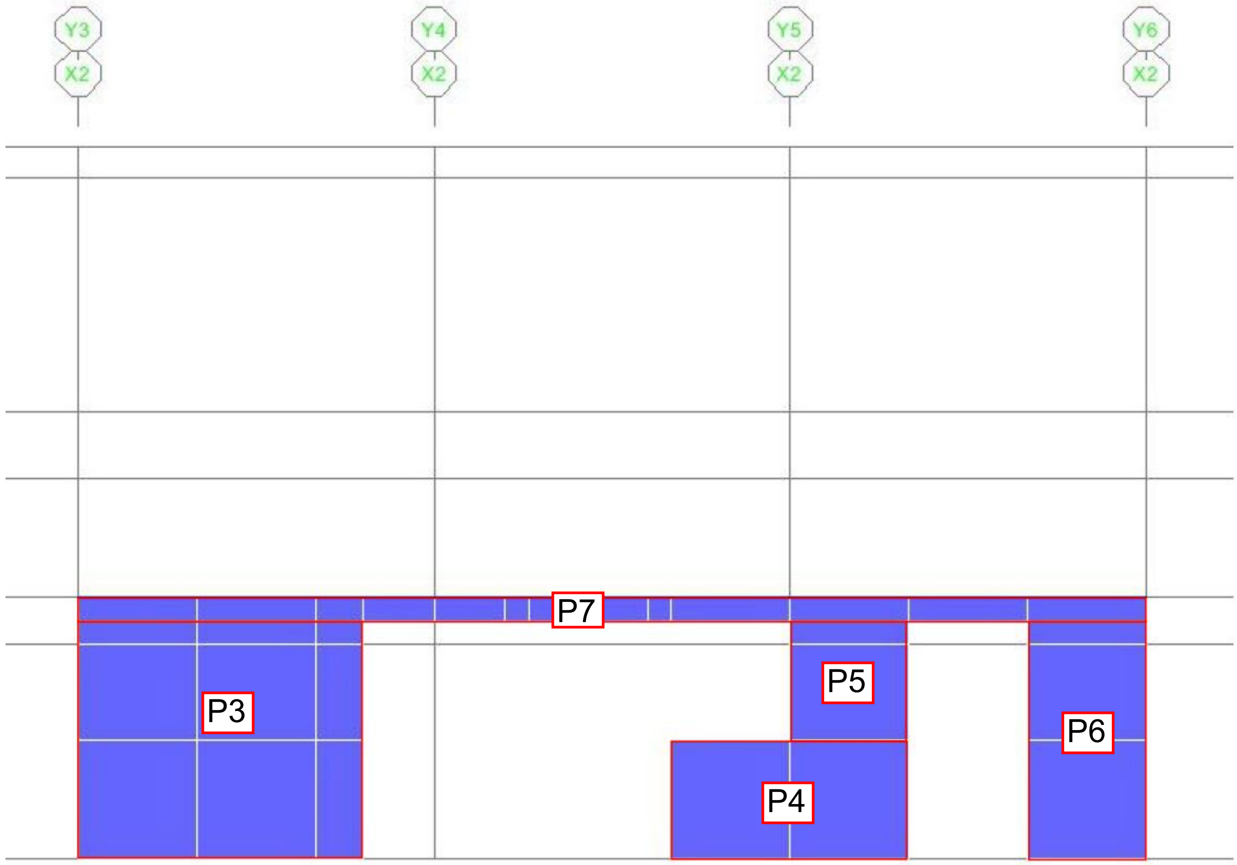


Figure 1: WALL DESIGNATION AT GRIDLINE X2

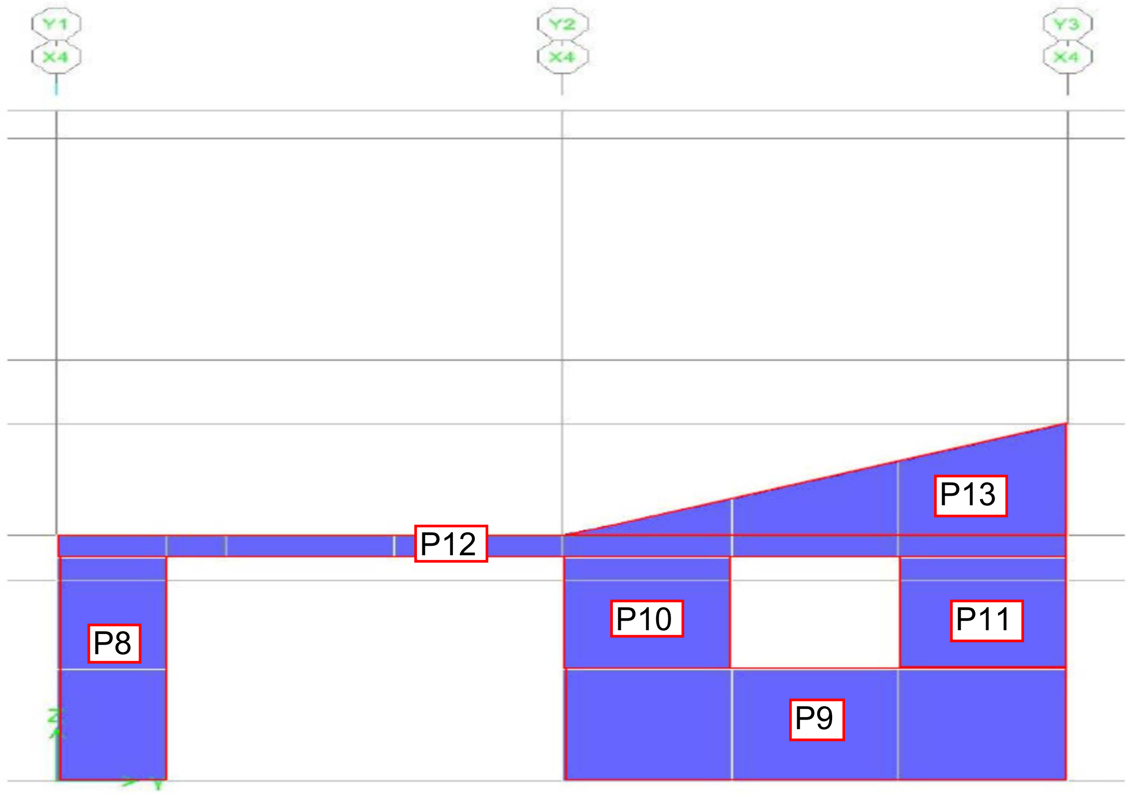


Figure 2: WALL DESIGNATION AT GRIDLINE X4 - FROM Y1 TO Y3

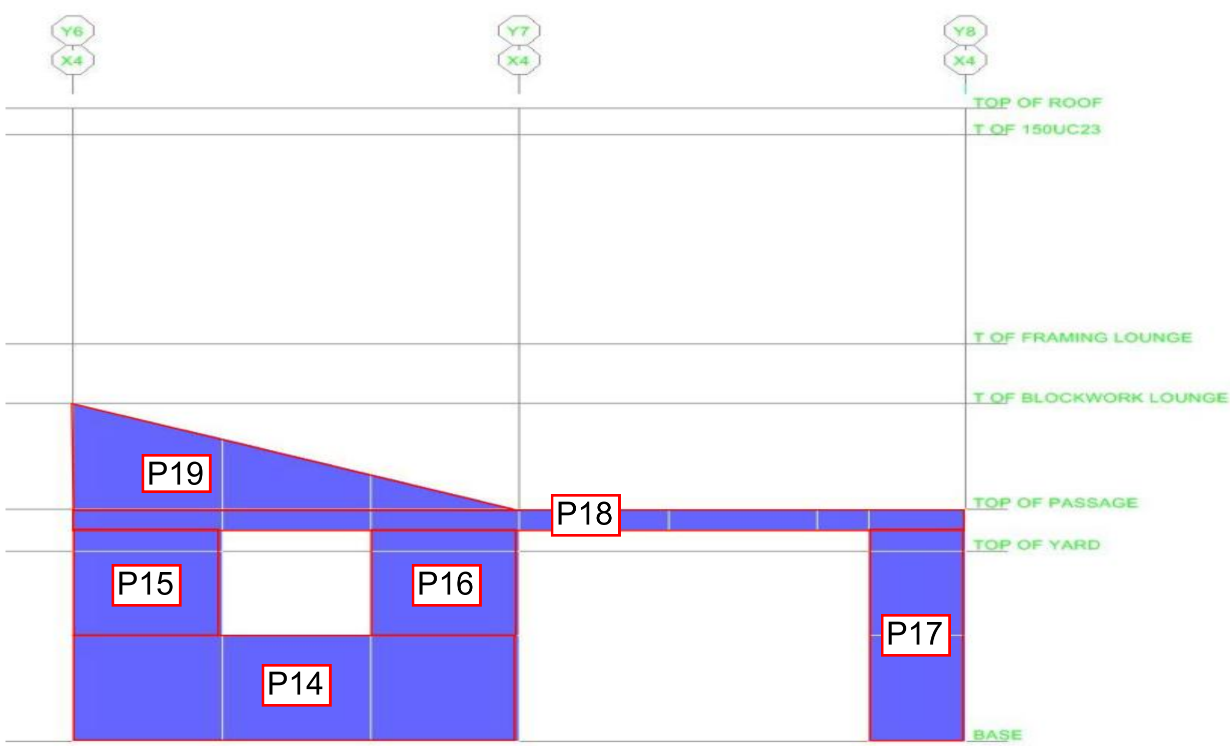


Figure 3: WALL DESIGNATION AT GRIDLINE X4 - FROM GRIDLINE Y6 TO Y8

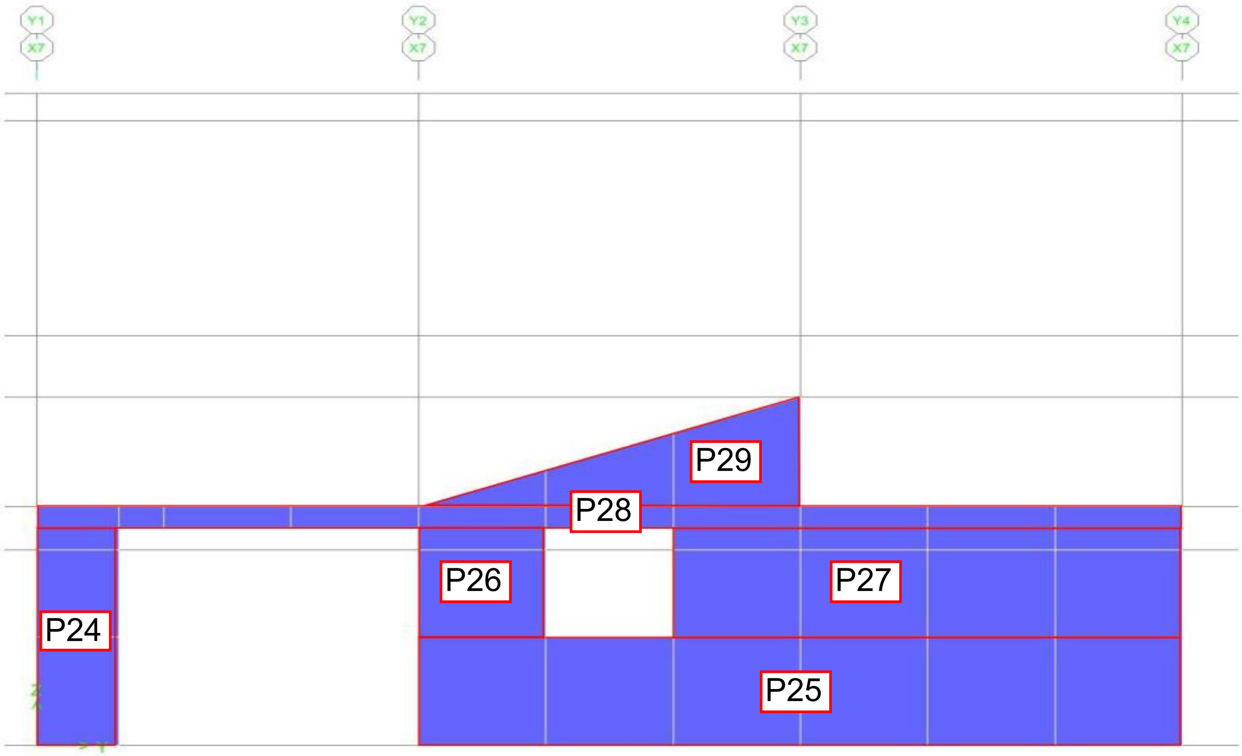


Figure 4: WALL DESIGNATION AT GRIDLINE X7 - FROM GRIDLINE Y1 TO Y4

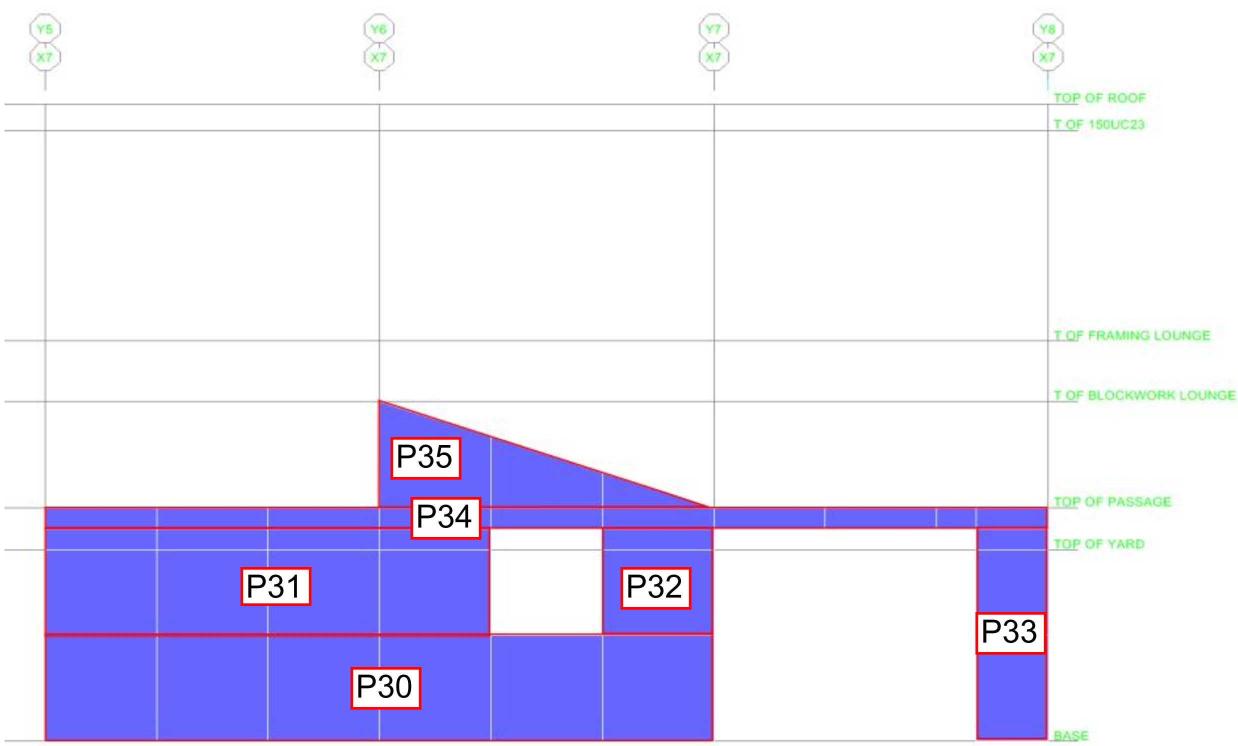


Figure 5: WALL DESIGNATION AT GRIDLINE X7 - FROM GRIDLINE Y5 TO Y8

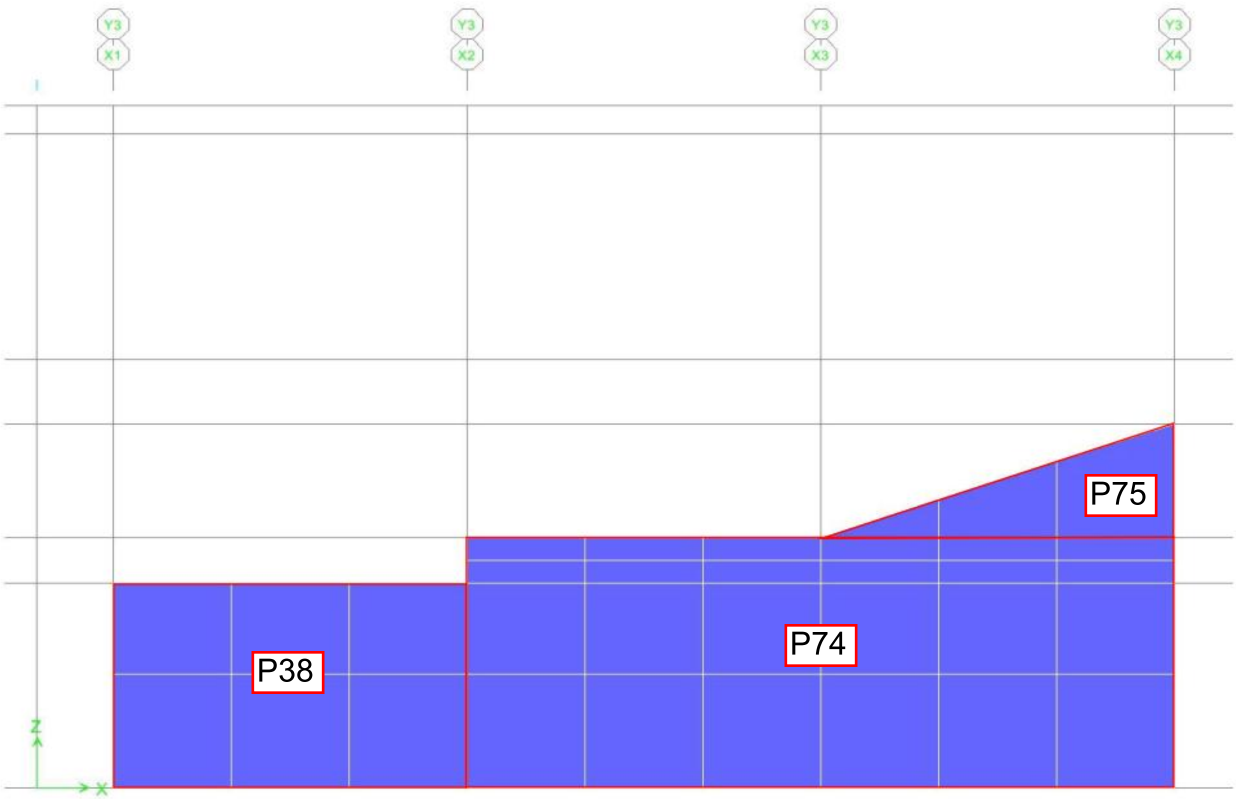


Figure 6: WALL DESIGNATION AT GRIDLINE Y3 - FROM GRIDLINE X1 TO X4

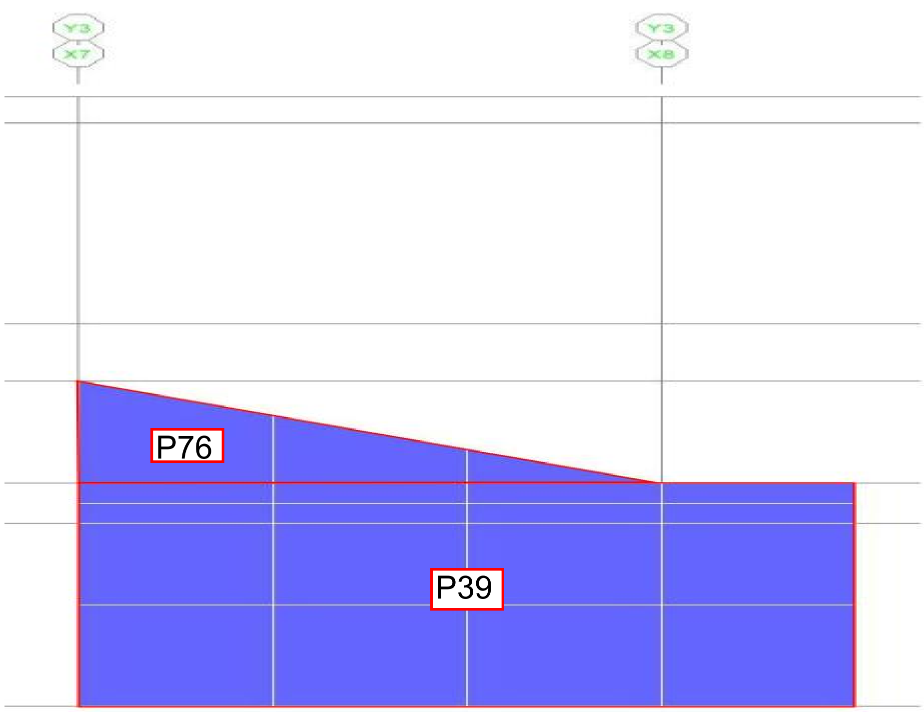


Figure 7: WALL DESIGNATION AT GRIDLINE Y3 - FROM GRIDLINE X7 TO X8

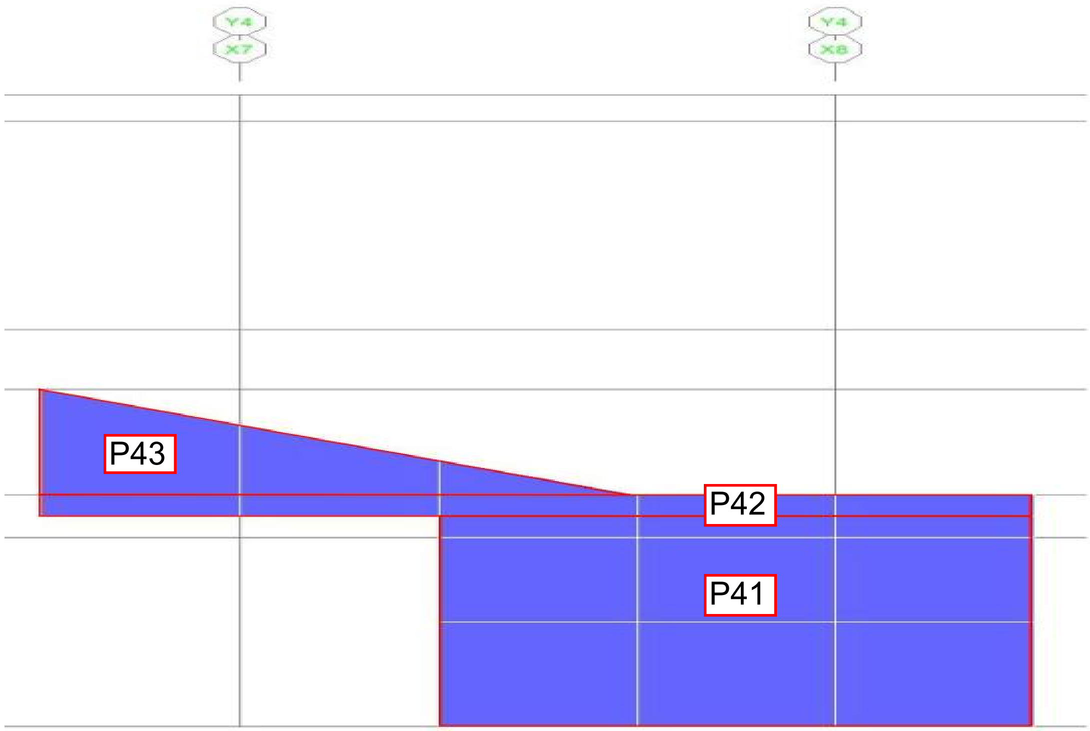


Figure 8: WALL DESIGNATION AT GRIDLINE Y4 - FROM X7 TO X8

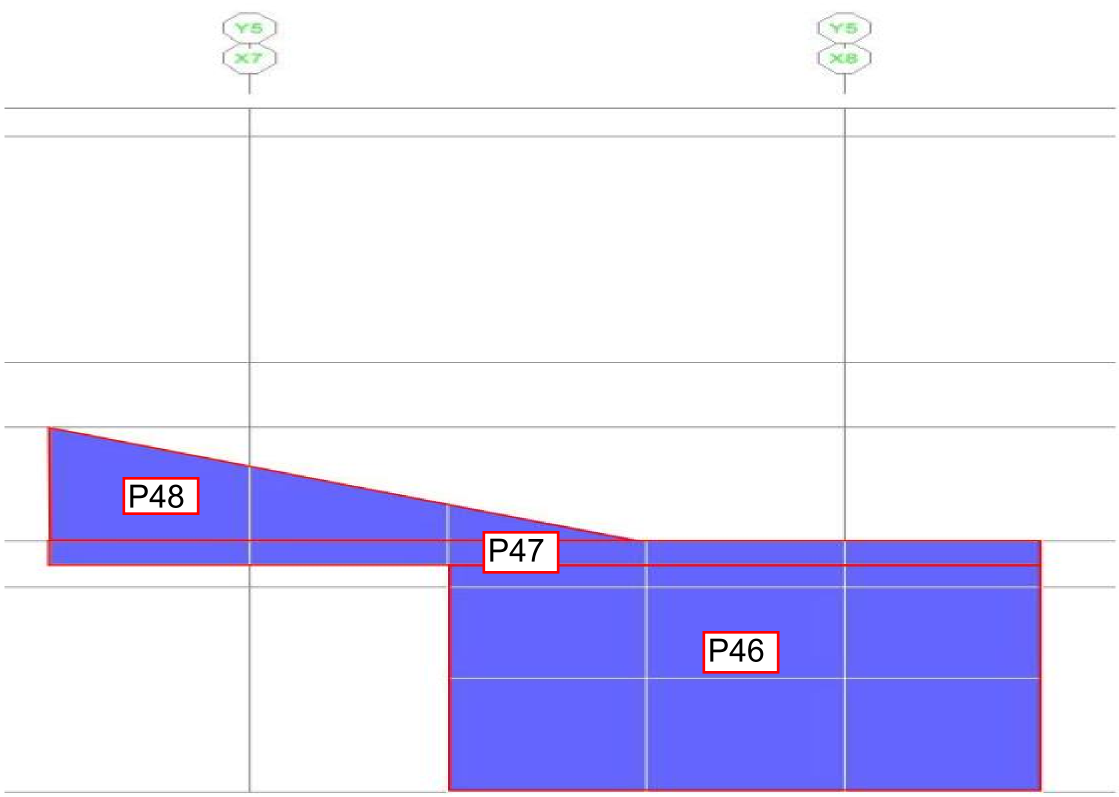


Figure 9: WALL DESIGNATION AT GRIDLINE Y5 - FROM GRIDLINE X7 TO X8

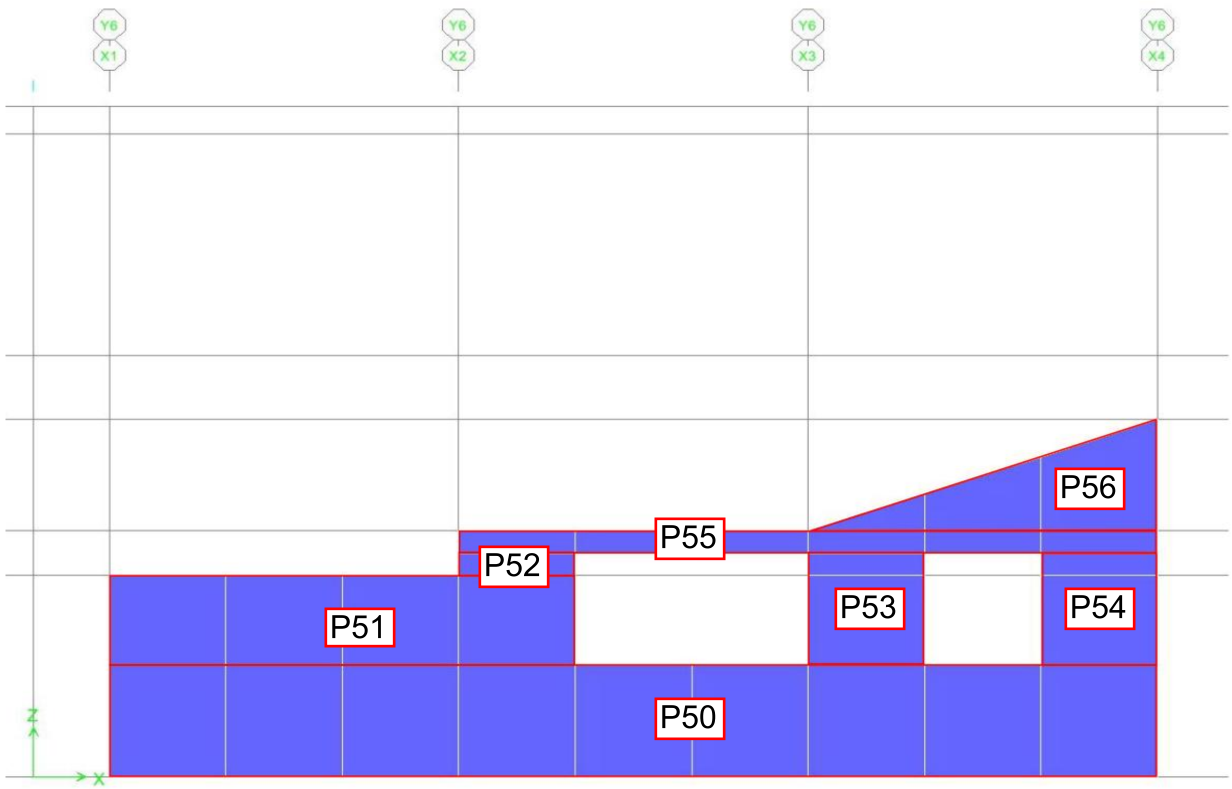


Figure 10: WALL DESIGNATION AT GRIDLINE Y6 - FROM X1 TO X4

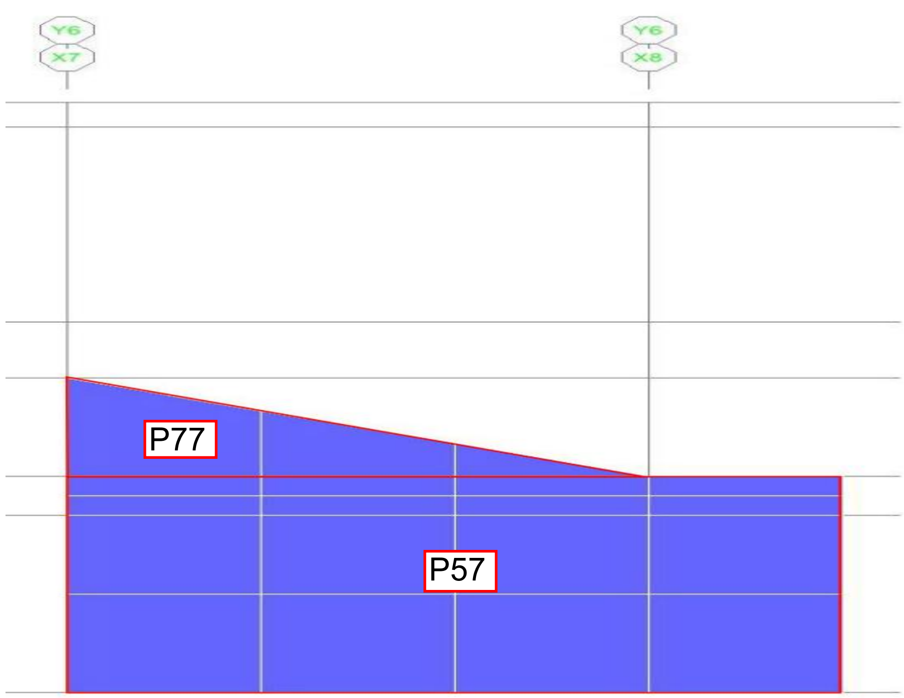
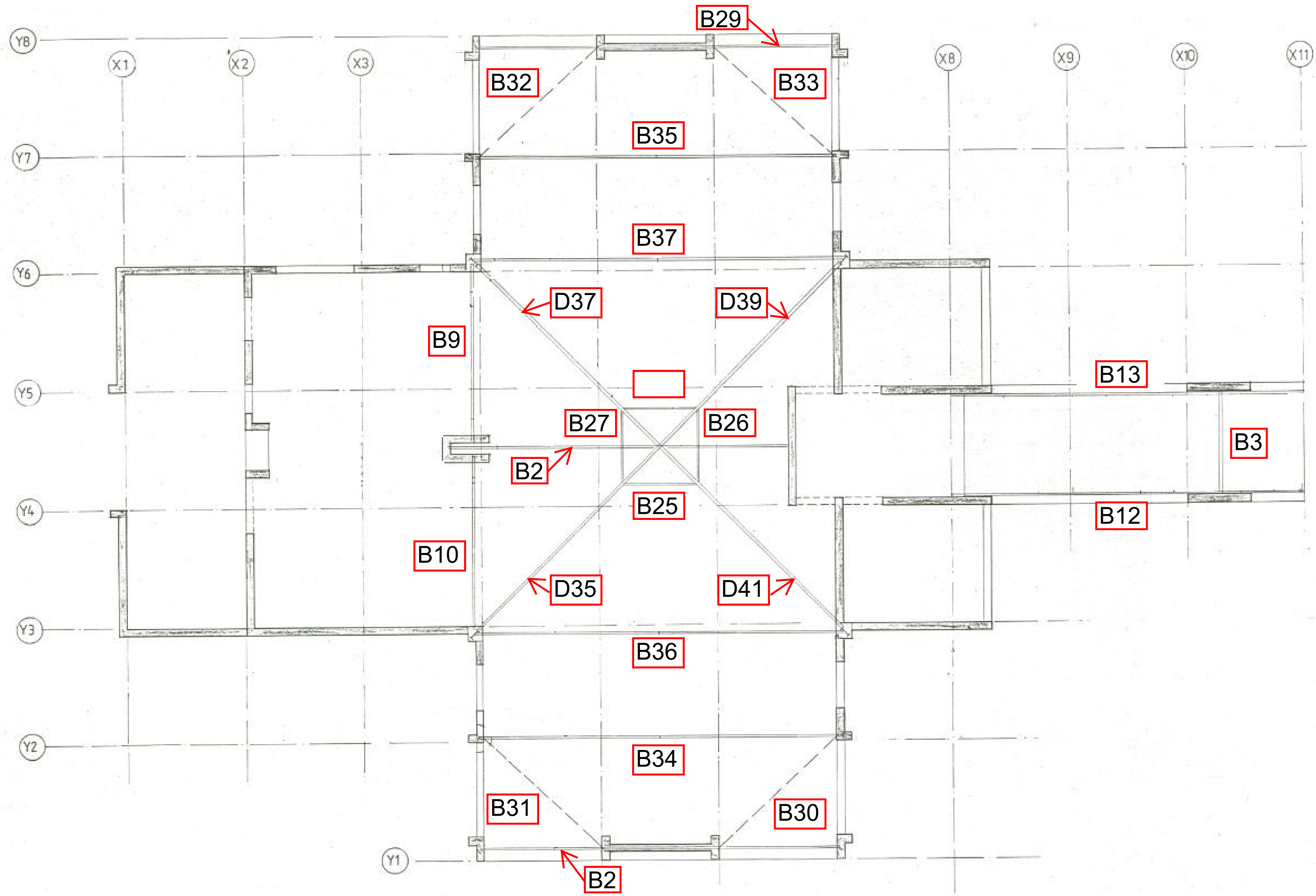
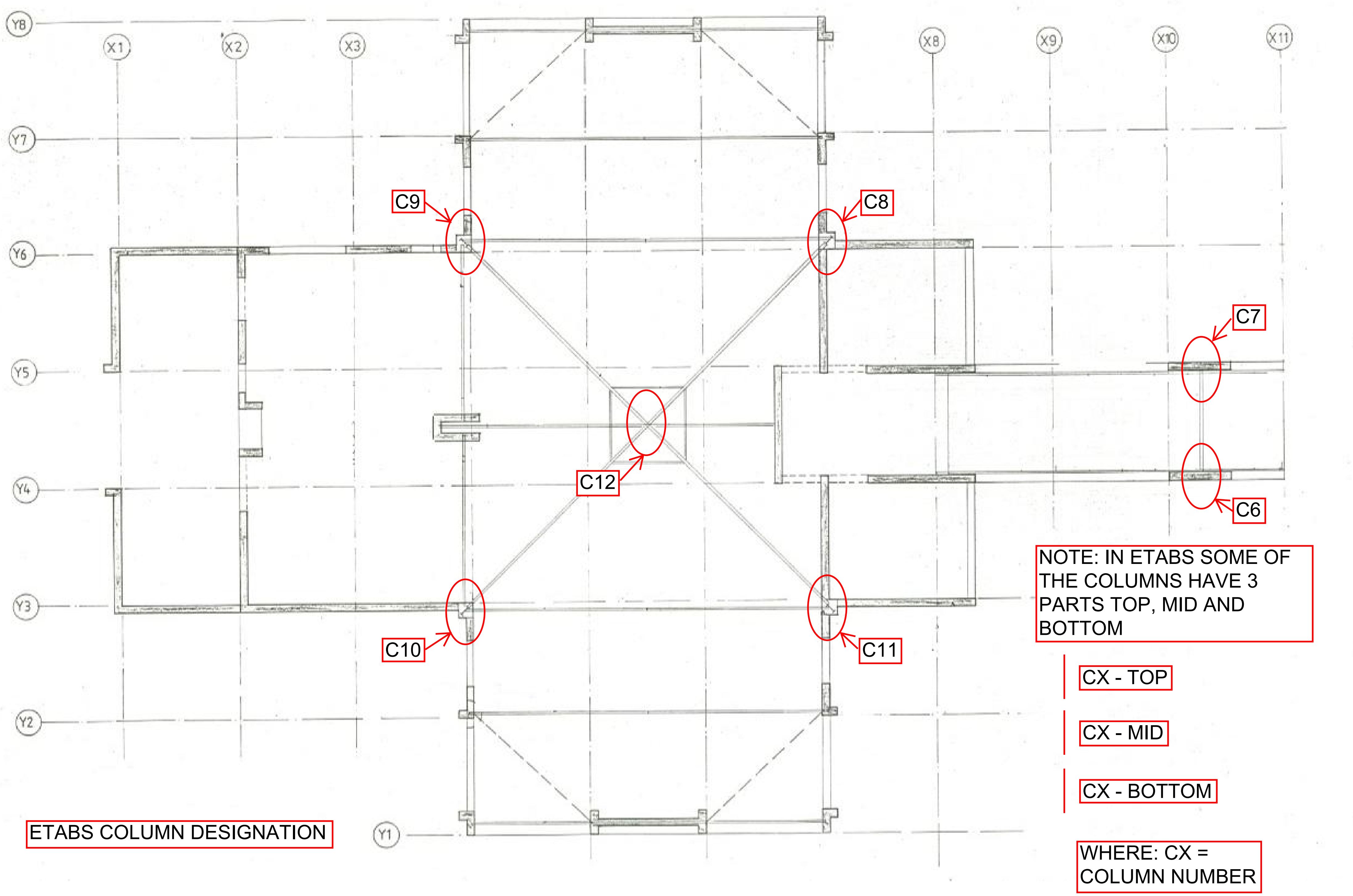


Figure 11: WALL DESIGNATION AT GRIDLINE Y6 - FROM X7 TO X8



ETABS FRAMING DESIGNATION



Y8
X1
X2
X3
X8
X9
X10
X11
Y7
Y6
Y5
Y4
Y3
Y2
Y1
C9
C8
C7
C12
C6
C10
C11
NOTE: IN ETABS SOME OF THE COLUMNS HAVE 3 PARTS TOP, MID AND BOTTOM
CX - TOP
CX - MID
CX - BOTTOM
WHERE: CX = COLUMN NUMBER
ETABS COLUMN DESIGNATION

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