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DETAILED SEISMIC ASSESSMENT REPORT

121396

RANGITIKEI DISTRICT COUNCIL

DETAILED SEISMIC ASSESSMENT

7 KING STREET, MARTON - DEPOT

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Document Control Record

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Signed		**Signed**	

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Contents

1 Executive Summary

1.1 Background

Resonant Consulting Ltd (Resonant) has been commission by Rangitikei District Council (RDC) to undertake a Detailed Seismic Assessment (DSA) of the building located at 7 King Street, Marton. The aim of the assessment is to determine the seismic rating of the building in relation to the New Building Standard (%NBS).

1.2 Building Description

The building at 7 King Street, Marton was designed circa 1982 by Lamong, Bycroft & Partners.

The building is currently used as depot for tools and offices.

1.3 Assessed Seismic Rating

The assessment has been completed in accordance with the New Zealand Society of Earthquake Engineering document – Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments, dated July 2017. The seismic rating assumes that Importance Level 2 (IL2), in accordance with the joint Australian/New Zealand Standard – Structural Design Actions Part 0, AS/NZS 1170.0:2002, is appropriate. Refer to Table 1 for a summary of the building's seismic rating.

Table 1:

Address - DSA		
Building	Seismic Rating (%NBS)	Seismic Grade
Depot – Longitudinal Direction	6% NBS	E
Depot – Transverse Direction	85% NBS	A
Office – Longitudinal Direction	100% NBS	A+
Office – Transverse Direction	80% NBS	A

The Seismic Grade has been determined in accordance with the NZSEE grading scheme. The overall building seismic rating for the Depot Building is governed by the roof bracing capacity. Refer to Section 8 for a summary of the %NBS scores, and commentary, for the various building structural components and to Appendix B for a Technical Summary Report.

1.4 Basis for the Assessment

The assessment has been based on the following information:

- Original Drawings by Lamont, Bycroft and partners - 1982
- Alterations Drawings by RDC - 1988
- On-site inspections:
    - By Gonzalo Sangra on the 17/11/2021.

1.5 Seismic Retrofit Options

A preliminary concept strengthening scheme, to achieve a capacity >67%NBS rating, has been enclosed in Section 10.

The following elements limit the capacity below 67%NBS:

- Truss top chord
- Truss bottom chord
- Roof bracing

2 Introduction

2.1 Overview

Rangitikei DC has engaged Resonant to assess the seismic capacity of the building located at 7 King Street. The intention of the assessment is to determine the building's ability to withstand earthquake loads in terms of the current New Zealand Building Standards and yield a score for the building expressed as "Percentage New Building Standard" (%NBS).

2.2 Scope of Work

As identified in our proposal dated 31/08/2021, the scope of works to be undertaken as part of the assessment:

- Detailed Seismic Assessment to determine the %NBS and identify any critical structural weaknesses.
- Provide an indicative remedial solution to strengthen the building to achieve a baseline %NBS rating.
- Provide a written report outlining the findings of the assessment.

2.3 Sources of Information

The assessment of 7 King Street is based on the following information:

- Original Drawings by Lamont, Bycroft and partners - 1982
- Alterations Drawings by RDC - 1988

All the documents have been obtained from Rangitikei District Council.

2.4 Site Investigation

A non-intrusive site investigation was carried out to confirm the information in the available documentation.

2.5 Exclusions

This report does not extend to an assessment of non-structural items such as cladding, ceilings, partitions, other fit-out related items, geotechnical ground conditions and latent defects.

It should be noted that for the purposes of this assessment the %NBS refers to the capacity and performance of the lateral load resisting system only. As Building Codes have evolved it is likely that an older building may not meet current Code requirements for aspects such as access and moisture detailing.

3 Background Regulations

3.1 Building Act 2004 and Earthquake Prone Buildings Amendment Act 2016

Before describing how the seismic analysis was completed, the regulatory requirements and definitions for earthquake prone buildings should be discussed.

The Building (Earthquake-prone Buildings) Amendment Act 2016 introduced major changes to the way earthquake-prone buildings are identified and managed under the Building Act.

Earthquake-prone Buildings

Under section 133AB of the Building Act (2004), the definition of earthquake-prone building is:

- A building or a part of a building is earthquake prone if, having regard to the condition of the building, or part, and to the ground on which the building is built, and because of the construction of the building or part
    - the building or part will have its ultimate capacity exceeded in a moderate earthquake, and
    - if the building or part were to collapse, the collapse would be likely to cause:
        - injury or death to persons in or near the building or on any other property, or
        - damage to any other property
- The above does not apply to a building that is used wholly or mainly for residential purposes unless the building:
    - comprises 2 or more storeys; and
    - contains 3 or more household units

A "moderate earthquake" is defined in Section 7 of the Building Regulations 2005"

> "…moderate earthquake means, in relation to a building, an earthquake that would generate shaking at the site of the building that is of the same duration as, but that is one-third as strong as the earthquake shaking (determined by normal measures of acceleration, velocity, and displacement) that would be used to design a new building at that site."

Whether a building, or part of a building, is earthquake-prone is determined by the territorial authority in whose district the building is situated.

For the purpose of the above subsection ultimate capacity and moderate earthquake have the meanings given to them by regulations. To assist with application, both ultimate capacity and moderate earthquake are terms defined in the Building (Specified Systems, Change the Use, and Earthquake-prone Buildings) Regulations 2005 (as amended).

These regulations define ultimate capacity as "The probable capacity to withstand earthquake actions and maintain gravity load support assessed by reference to the building and its individual elements or parts" and moderate earthquake as "In relation to a building, an earthquake that would generate shaking at the site of the building that is of the same duration as, but that is one-third as strong as, the earthquake shaking (determined by normal measures of acceleration, velocity, and displacement) that would be used to design a new building at that site if it were designed on 1 July 2017."

3.2 Ratings

The ratings provided within this report have been generated with respect to New Zealand Society for Earthquake Engineering (NZSEE) guidelines. They are often summarised as "%NBS rating" which reflects the design coefficient for a similar building designed today to current codes, referred to as the New Building Standard (NBS).

Per the NZSEE publication "The Seismic Assessment of Existing Buildings", Section A3.2.4 groups building ratings as follows:

3.2.1 Table NZSEE Grading Scheme

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

It should be noted that the demarcation between a C and D rating, 33% NBS, is aligned with the Building Act of 2004. Although these ratings are calculated in a linear manner, they are meant to represent an exponential scale of earthquake risk.

4 Building Description

4.1 General Building Description

The Building at 7 King Street is a single storey timber framed structure. The construction is predominantly structural timber with cantilevered posts and a pile foundation. The lightweight perimeter cladding is supported on shallow foundations and a slab on grade substructure. The development was designed and constructed circa 1982 with the site layout shown in Figure 4.1.1 below.

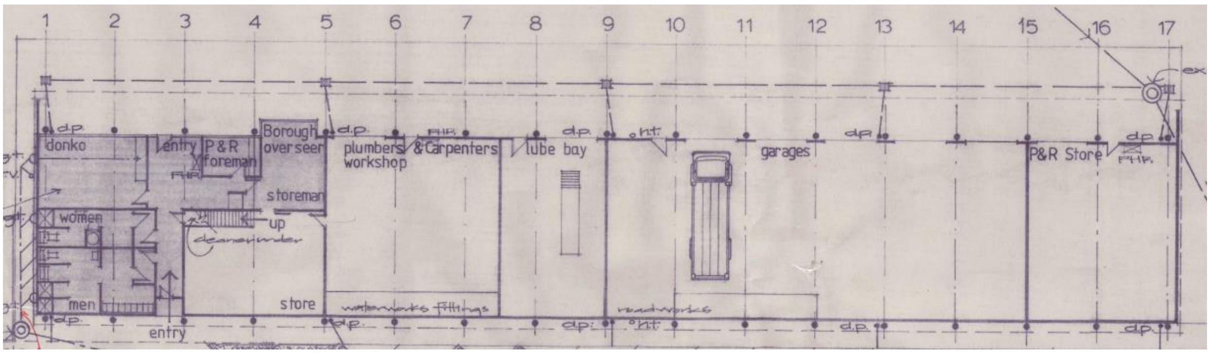
4.1.1 Overview of 7 King Street, Marton

The building has a total height of 7.0m at the apex of the timber truss, from finished floor level to top of the roof level. In plan the building has an approximately rectangular footprint measuring 57m x 13m. The façades consist of timber framed wall on a 1.2m high block wall on a shallow foundation.

4.2 Structural Description

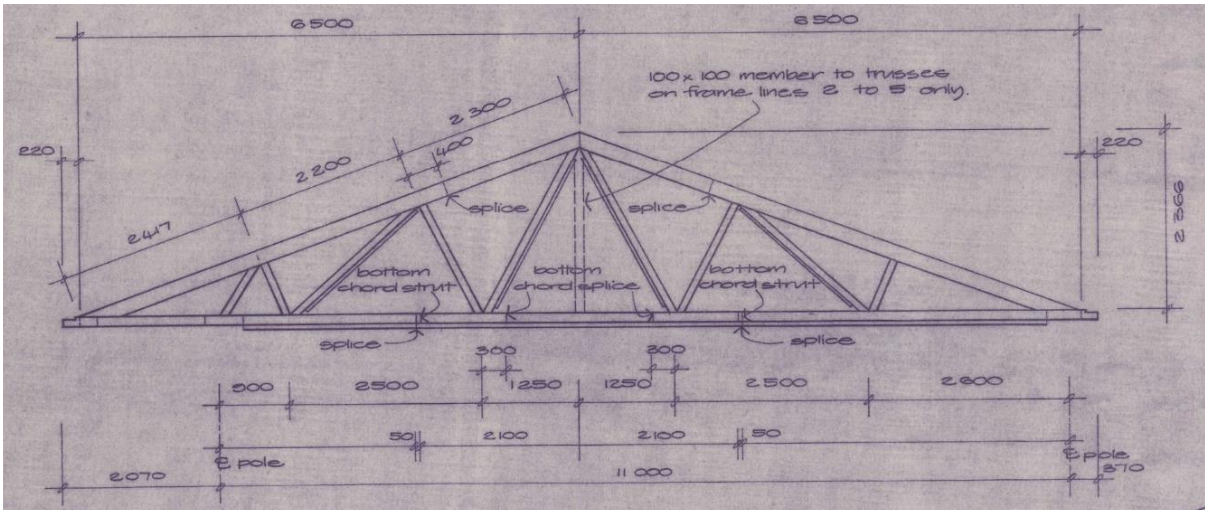
Superstructure

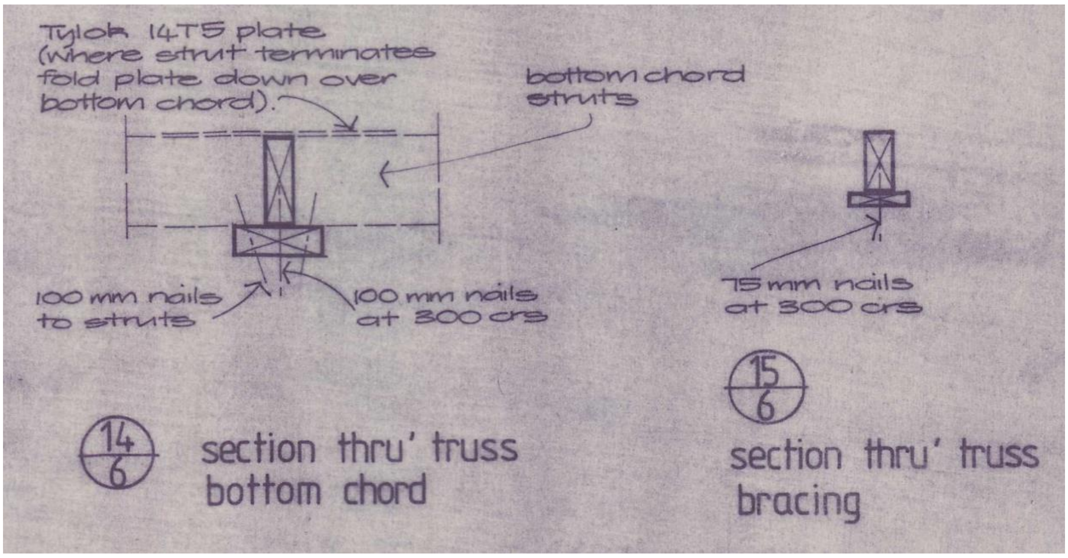
The building is divided into a varying grid system with timber portal (Grids 1-17) in the transverse direction (East - West) providing both gravity support for the light-weight roofing, supported on timber purlins, and lateral bracing support for the building.



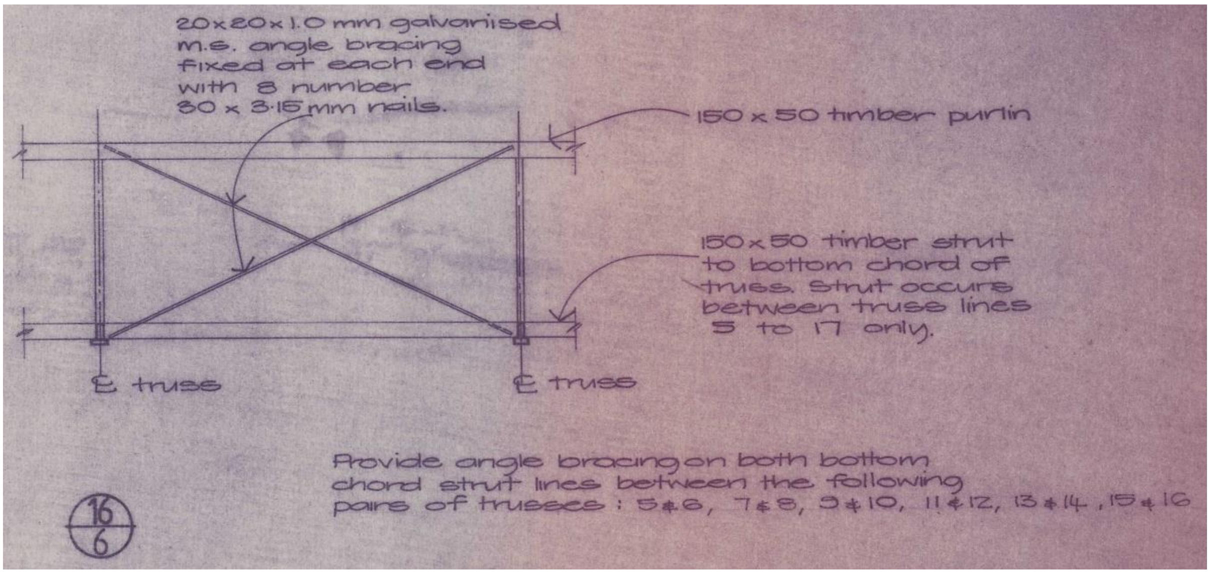
Timber trusses at 4m CRS (a total of 17):

Truss top chord 200 x 50.
Truss bottom chord 150x50 with 150x50 stiffener where shown.
Truss bracing 100x50 with 100x25 stiffener where shown.
Locate splices in chord members shown.
Splice in bottom chord stiffener to be located adjacent to chord strut.

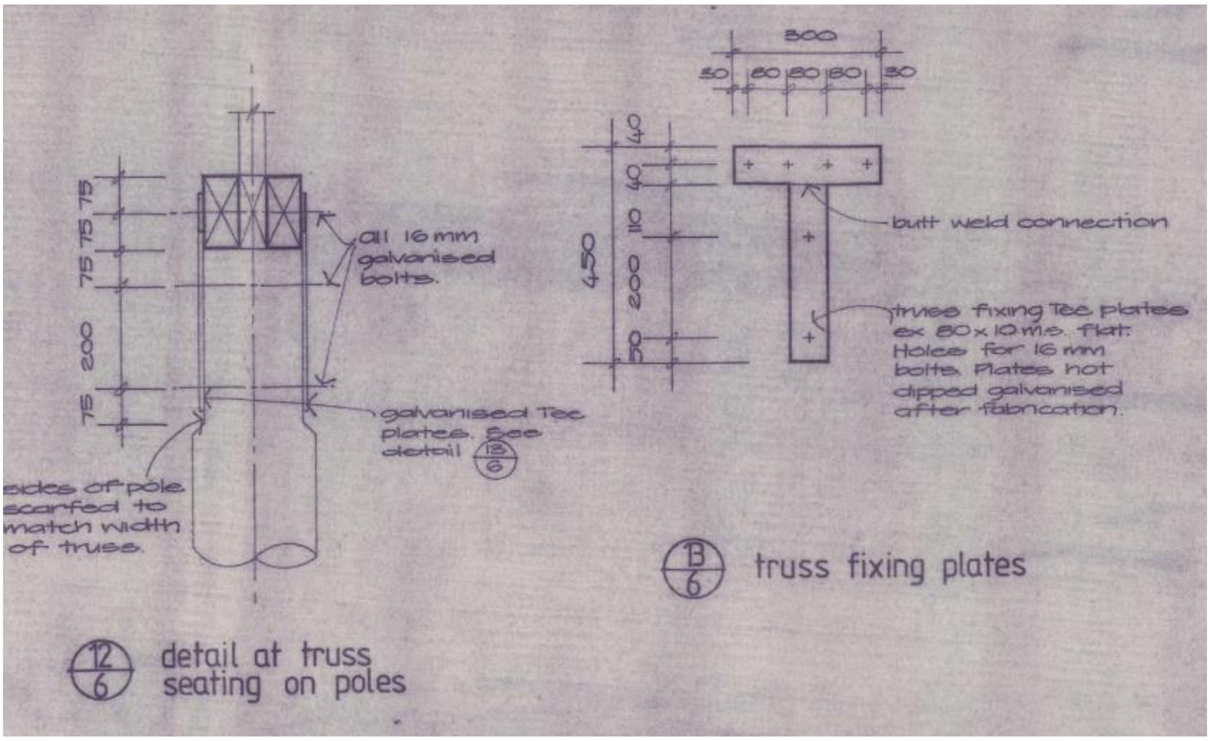




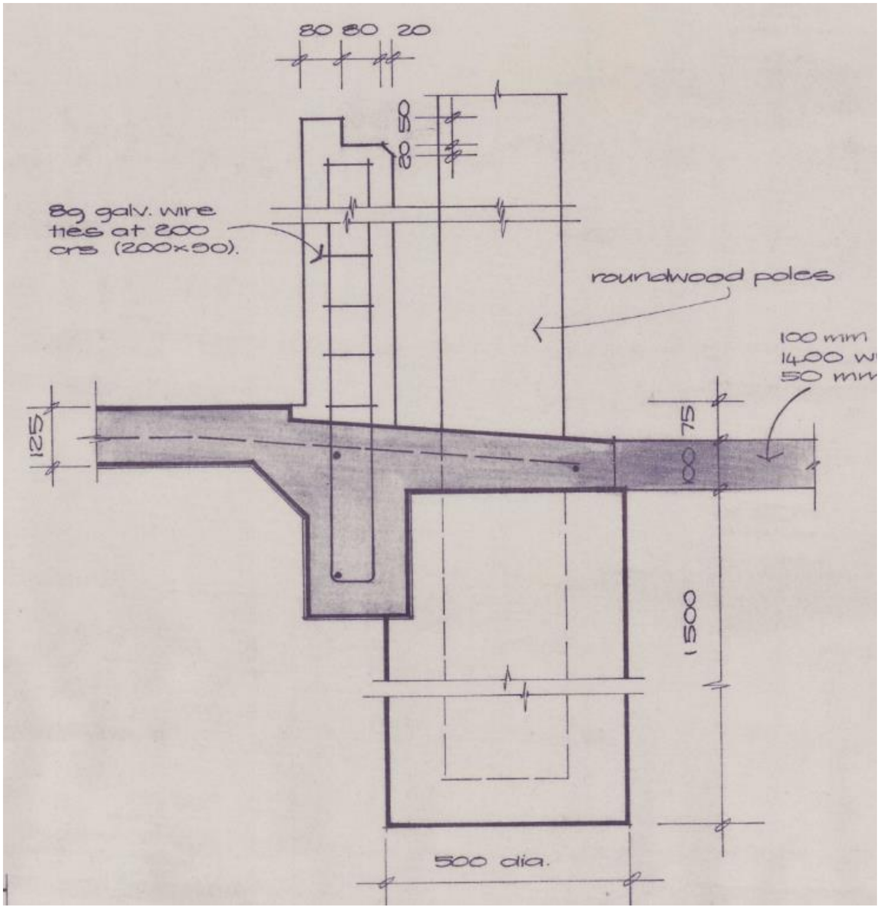
Timber struts to bottom chord with cross bracing to top chord (only to six paira of trusses).

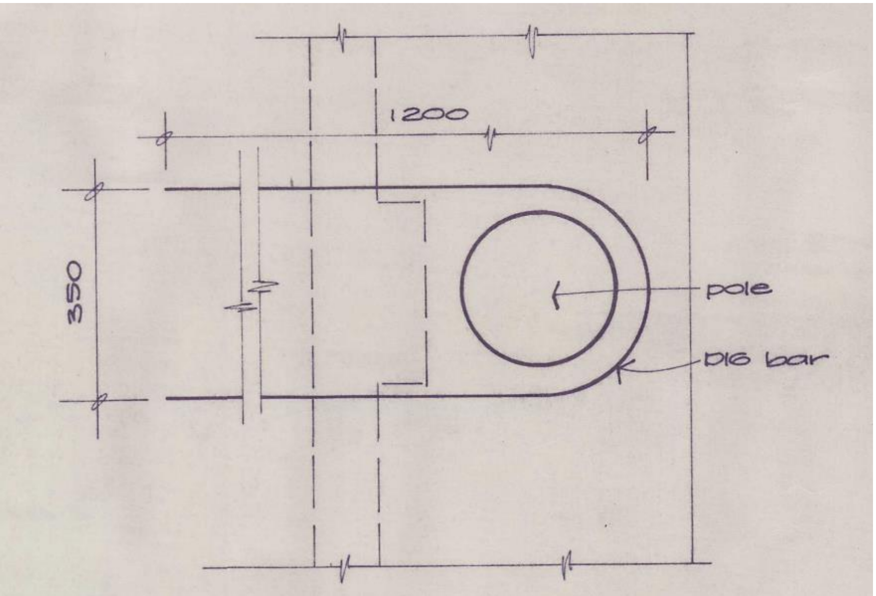


Timber trusses are supported on 250mm Dia cantilevered round posts with fixing plates both sides:



Post foundations are 500mm Dia x 1.5m deep piles with anchorages to the depot 125mm thick slab:





Seismic Rating Systems

In the longitudinal direction, the lateral earthquake loading is resisted by cantilevered piles (due to the height of walls and lack of it in a large part of the building, is assumed that walls are not part of the seismic resisting system, but they are part of the earthquake loads). In the transverse direction, the lateral earthquake loading is resisted by the same cantilevered posts that are part of the transverse frame with the trusses. This assessment covers seismic loading as the only lateral load and does not address wind loading on the structure.

Gravity system:

The roof sheeting is running over timber purlins (150x50mm at 900mm CRS.) that span between trusses (@4m CRS.) which are supported on timber poles founded with concrete piles.
Load path= Roof to purlins - Purlins to Trusses - trusses to posts – posts to piles.

Transverse Lateral load resisting system:

Lateral loads in the transverse direction are typically resisted by cantilevered poles. Roof cross bracing looks incomplete and for that reason, purlins transfer the EQ loads to the top chord and top cord transfer the loads to the pole.

Load path= Roof to purlins - Purlins to Trusses - Trusses to posts – Posts to piles.

Longitudinal Lateral load resisting system:

Lateral loads in the longitudinal direction are resisted by the cantilevered poles as well. Trusses bottom chords and top chords are connected with cross bracing and struts only to 5 bays and some struts are missing, this means that the roof longitudinal EQ forces will be transferred to the post by the flexural capacity of the top chords and the bottom chord out of plane.

Load path= Roof to purlins - Purlins to top chord – top chord to posts to one side and top chord to bottom chord on the other side – Posts to piles.

5 Geotechnical Conditions

A geotechnical report was not supplied.

Soil assumption to check the pile = 28º of angle of shear resistance or 50kPa of undrained shear strength, the worst scenario.

6 Seismic Analysis

6.1 Seismic Parameters

Building Ductility

Ductility is a measure of the ability of a building to resist the earthquake forces/energy by inelastic deformation. Under current design standards the level of ductility is generally determined by:

- Identifying an appropriate mechanism that can sustain inelastic deformations without leading to collapse of a building.
- The ability to achieve an appropriate level of structural detailing to ensure that the chosen ductile mechanism is achievable.
- Code limitations on the inter-storey deflections for the structure.

The choice of ductility factor affects the load level selected for the design and the complexity of detailing required. Generally, the higher the ductility demand, the lower the loading, but the more stringent the detailing requirements. Ductility demands typically vary between $\mu = 1.0$ for elastic, $\mu = 1.25$ for nominally ductile, $\mu = 3.0$ for limited ductile and $\mu = 6.0$ for fully ductile. A sufficient quantity and placement of reinforcing steel or well-designed bolted or welded steel beam-column connections could imply that a minimal level of ductility could be achieved without creating brittle failure mechanisms that might compromise life safety for any occupants.

The current guidelines "The Seismic Assessment of Existing Buildings" require the assessor to determine the ductility demand and ductility capacity of the structure rather than assume a ductility factor. This is generally done by undertaking the Simple Lateral Mechanism Analysis (SLaMA). The SLaMA is a simple nonlinear analysis technique that provides an estimate of the global probable capacity of the primary lateral structure of the building.

The building assessed is typically of timber frame, built in 1982. The building was assesed for nominal ductility $\mu = 1.25$.

Typically, instead of assuming an appropriate ductility factor, the required ductility factor is determined by following the Force-Based SLaMA Procedure described in Section C2.3 of the NZSEE Guidelines.

Site Geology

The site geology can have significant impact on the level of loading imparted on a building during an earthquake. Deep, soft soil conditions tend to amplify the ground motions, increasing the forces on a building structure. The interpreted subsoil Class is D classification in accordance with the available geotechnical report was used to determine the elastic site hazard spectrum for the horizontal loading 'C(T)' (section 3 NZ S1170.5:2004).

Importance Level

The Importance Level of a building is a classification from NZS 1170.0. Increasing importance levels trigger higher factors of safety in design or analysis. The building is designated Importance Level 2 (IL2). The building is a depot building with offices, however as the total expected occupancy is less than 5000 people it is not classified as IL3.

TABLE 3.2

IMPORTANCE LEVELS FOR BUILDING TYPES—NEW ZEALAND STRUCTURES

Importance level	Comment	Examples
1	Structures presenting a low degree of hazard to life and other property	Structures with a total floor area of <30 m^2 Farm buildings, isolated structures, towers in rural situations Fences, masts, walls, in-ground swimming pools
2	Normal structures and structures not in other importance levels	Buildings not included in Importance Levels 1, 3 or 4 Single family dwellings Car parking buildings
3	Structures that as a whole may contain people in crowds or contents of high value to the community or pose risks to people in crowds	Buildings and facilities as follows: (a) Where more than 300 people can congregate in one area (b) Day care facilities with a capacity greater than 150 (c) Primary school or secondary school facilities with a capacity greater than 250 (d) Colleges or adult education facilities with a capacity greater than 500 (e) Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities (f) Airport terminals, principal railway stations with a capacity greater than 250 (g) Correctional institutions (h) Multi-occupancy residential, commercial (including shops), industrial, office and retailing buildings designed to accommodate more than 5000 people and with a gross area greater than 10 000 m^2 (i) Public assembly buildings, theatres and cinemas of greater than 1000 m^2 Emergency medical and other emergency facilities not designated as post-disaster Power-generating facilities, water treatment and waste water treatment facilities and other public utilities not designated as post-disaster Buildings and facilities not designated as post-disaster containing hazardous materials capable of causing hazardous conditions that do not extend beyond the property boundaries

The design working life of the structure is 50 years. Combined with the IL2 classification, a Return Period Factor “R” of 1.0 has been used for the analysis.

Site Spectra

The site spectra (m = 1.25 for depot and 3.5 for office) is given by:

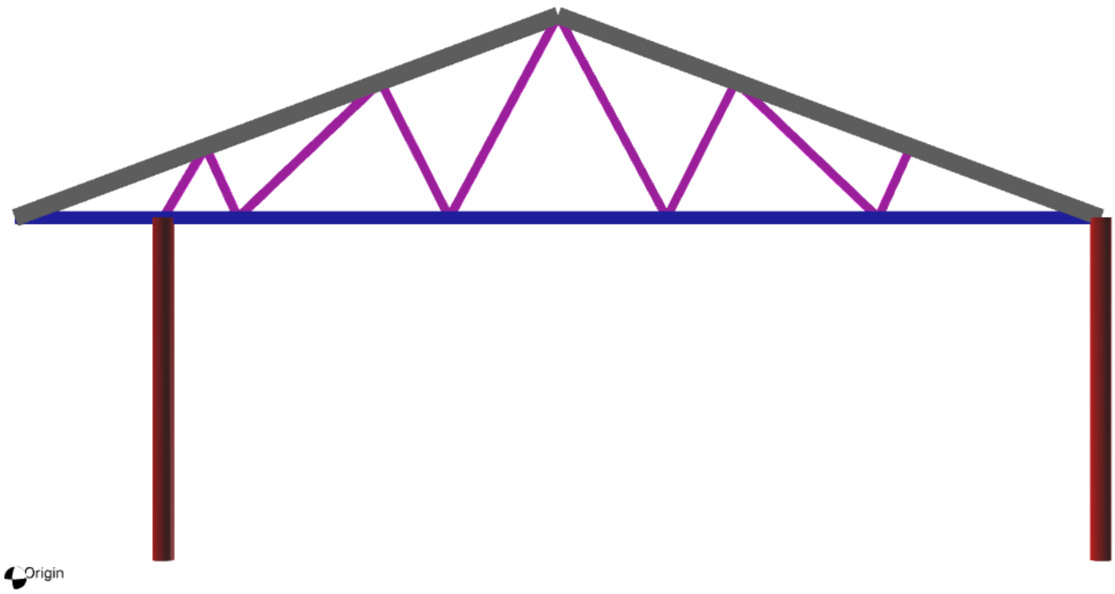
C(T) = Ch(T)*Z*R*N(T,D)

Building						
Structural System	**T_s**	**C_h(T)**	**Z**	**R**	**N(T,D)**	**C(T)**
DEPOT	0.4	3.00	0.30	1.0	1	**0.73**
OFFICE	0.4	3.00	0.30	1.0	1	**0.26**

6.2 Building Analysis Method

The lateral load resisting systems for the building consists of cantilevered posts connected by trusses. Linear methods are generally appropriate for systems with a nominal ductility of 1.25. Because of the overall low ductility demand on the building, an Equivalent Static Analysis was adopted as recommended by "The Seismic Assessment of Existing Buildings – Assessment Procedures and Analysis Techniques" guidelines Part C2 Section 2.6.2 Table C2.1. The assessment was conducted in accordance with Part C6 of guidelines "The Seismic Assessment of Existing Building – Structural Steel Buildings" and Part C5 of guidelines "The Seismic Assessment of Existing Building – Reinforced Concrete Buildings"

Representative 2D frames in the building were modelled.



6.3 Timber Framed Office

There is a timber framed office inside the building. Walls demand and capacity has been assessed using ductility 3.5.

6.4 Analysis Assumptions

General Assumptions

- In calculating the self-weight of the structure 24kN/m3 was used for all reinforced concrete elements. Timberweights were calculated from the member sizes. Lightweight roof elements have been assumed to be 0.25kPa. Mezzanine floor self-weight is assumed to be 0.6kPa.
- The following Live Loads & SDLs have been allowed for mezzanine floor:
    - Mezzanine = 2.0kPa (mezzanine space do not seems able to hold big loads due to the lack of space, difficulty of moving loads and some sheets were missing – during the site visit no loads were added on the mezzanine)
    - Roof LL = 0.25kPa
- Load combinations used in the analysis are as required by NZ S1170.0.
- The building has been designated as an Importance Level 2 (IL2). The design working life of 50 years has been used, giving a return period factor of 1.0.
- The Hazard factor, Z for Marton is 0.30.
- The subsoil class for the site assumed is D – Deep Soils.
- The member capacities have been assessed using the New Zealand Concrete Standard NZ S3101:2006, New Zealand Steel Structures Standard NZS 3404 Parts 1 and 2:1997 and the guidelines "The Seismic Assessment of Existing Buildings".
- All building materials have been assumed to be in acceptable condition. Allowances for corrosion, spalling or any other latent structural defects has not been considered as part of this assessment.
- Member capacities were calculated per the sizes and dimensions given on the structural drawings, and have not been verified by field observation or measurement.
- The building has not been checked for wind loads.

Material Properties

Material properties have accounted for the probable strengths. Factors for various materials have been obtained from guidelines "The Seismic Assessment of Existing Buildings". For concrete a probable strength factor of 1.5 has been used while for reinforcing steel a factor 1.3 has been used. For structural steel, a factor of 1.15 was used. Refer as follows for probable strengths used for the assessment.

- Reinforced Concrete Elements

Probable Compressive Strength	f'_c	=	25MPa - insitu
	f'_c	=	25MPa - slab
Probable Yield Strength of Reinforcement	$f_{y,p}$	=	494MPa (HD and HR Steel)
	$f_{y,p}$	=	358MPa (D and R Steel)
	$f_{y,p}$	=	300MPa (Mesh Steel)

- Timber Elements – No1 Framing

Species	Grade	Bending	Compression parallel	Tension parallel	Shear in beams	Compression perpendicular	Modulus of elasticity (GPa)
1. Moisture condition – Dry (m/c = 16% or less)							
Radiata pine	No. 1 framing	17.7	20.9	10.6⁺	3.8	8.9	8.0

- Poles – Normal Density

f_b = 38 MPa

- Office Walls capacity

Plasterboard to one side = 50 BU/m

Plasterboard to both side = 60 BU/m

7 Seismic Assessment Approach

A discussion on the seismic assessment approach is presented in the sections below, followed by a summary of the building's overall capacity in the Section 8.

For the assessment of buildings with timber frames as the primary lateral load resisting systems, the structures have been assessed in accordance with Part C9 – "Timber Buildings" in the new seismic assessment guidelines "The Seismic Assessment of Existing Buildings – Technical Guidelines". The member capacities for determining the %NBS of various structural elements have been assessed as follows.

The probable material strengths of beams, columns and braces are defined in accordance with Section C9.5 - Material Properties. The beam and column components are assessed using ductility principles by using a nominal ductility factor of 1.25. The connections are assessed elastically by using a ductility factor of 1.0 to ensure the correct hierarchy is formed, to suppress brittle failure mechanisms.

7.1 Foundations

The foundations for the posts consist of a deep pile restrained at top by the concrete slab and for the walls on strip footings. Since no soil test was found, the foundations have been assessed assuming soft soils.

7.2 Drifts

Building frame in-plane drift have been calculated from an Equivalent Static Method assessed in SPACE Gass model. The drifts have been determined in accordance with NZS 1170.5 Section 7.

8 Seismic Assessment Results

The seismic %NBS scores for the lateral structure, gravity structure and secondary structural elements for both directions of loading are summarized in the tables as follows, along with commentary on the results and potential options for strengthening to a higher % NBS (refer to structural calcs on Appendix B):

8.1 Building Capacity

Structural Component	Structural Weakness or Deficiencies	Assessed %NBS Score	Comments about mode of failure, physical consequences and potential options for strengthening to higher %NBS
Transversal-Direction			
Purlins	Bending Capacity	100%	
Truss top chord	Axial Capacity	100%	
Truss bottom chord	Axial Capacity	100%	
Truss diagonals	Axial Capacity	100%	
Timber Pole	Bending Capacity	85%	
Timber Pole fixing	Bolt and plate capacity	100%	
Foundation	Soil Horizontal Capacity	100%	
Drifts	ULS deflection SLS deflection	75% 70%	
Overall %NBS for Transversal Direction Loading		**70% (IL2)**	

Structural Component	Structural Weakness or Deficiencies	Assessed %NBS Score	Comments about mode of failure, physical consequences and potential options for strengthening to higher %NBS
Longitudinal-Direction			
Purlins	Bending Capacity	100%	
Truss Top Chord	Bending Capacity out of plane	6%	The building is not properly braced in the longitudinal direction. EQ loads need to be transferred from the roof to the poles and only the bending capacity out of plane of the top chord can do that
Truss Bottom Chord	Bending Capacity out of plane	12%	On the East side of the building, there is an overhang eaves and the bottom chord transfer the load from the top chord to the cantilevered pole
Foundation	Soil Horizontal Capacity	100%	
Drifts	ULS deflection SLS deflection	82% 74%	
Overall %NBS for Longitudinal Direction Loading		**6% (IL2)**	**Governed by the lack of bending capacity out of plane of truss top chord.**

9 Severe Structural Weaknesses

The general process of the DSA is determining the probable seismic capacity of the structure and relating this to the ULS loading demands. The intention is also to ensure with reasonable satisfaction that the building can withstand higher levels of shaking. This is referred to as the structural resilience and is a necessary aspect of the buildings behaviour if it is to deliver the overall expected seismic performance.

There are potentially some aspects of a buildings behaviour which may not be adequately captured within these general assessment procedures but are likely to have a step change response resulting in sudden (brittle) and / or progressive, but complete collapse of the buildings gravity load support system in shaking greater than that represented by %ULS shaking. These building aspects are referred to as Severe Structural Weaknesses (SSWs). Potential severe structural weaknesses are described in C1 of “The Seismic Assessment of Existing Buildings”.

The building has been reviewed for the SSW’s described above and it has been found that the building does not contain the above Severe Structural Weaknesses.

10 Concept Strengthening & Investigation

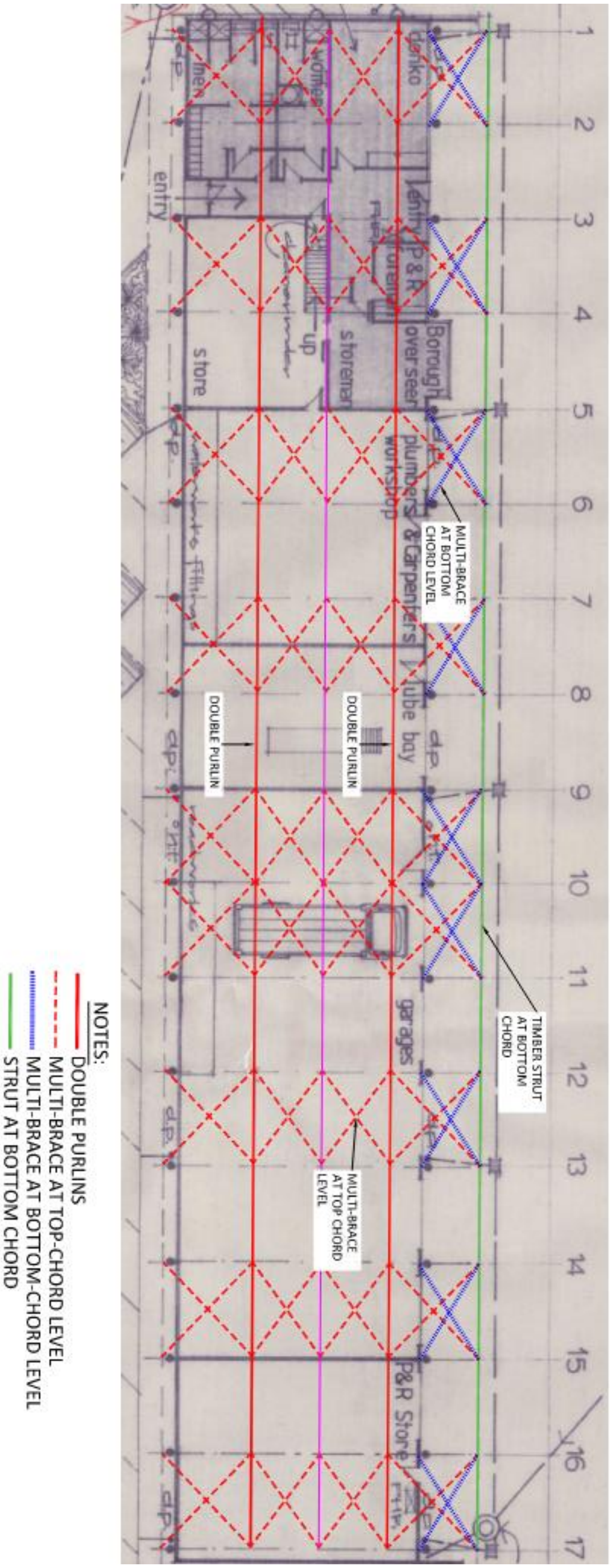
The detailed seismic assessment of the building at 7 King Street, Marton has found that several components of the building have a seismic score of less than 100%NBS. The following section summarises the deficiencies in the building and provides concept strengthening to achieve a higher 67% NBS score for the structural components.

The detailed seismic assessment identified the following as having a seismic score less than 67% NBS:

- Longitudinal bracing
- Top Chord bending capacity out of plane
- Bottom Chord bending capacity out of plane

Conceptual Preliminary Strengthening Scheme (refer to SK at following page):

- Multi Brace at roof level
- Double Purlins to work as multi-brace struts
- Multi-brace eaves at bottom chord level
- Strut to bottom chord at eaves
- Due to the poor conditions of the poles and to prevent further deterioration, we recommend to brace posts with four equally spaced SS multi braces "belts" with tensioner and to paint the poles with a protective coat to extend the working life of the poles.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women
men
entry
store
storeman
Borough overseer
plumbers & carpenters workshop
lube bay
garages
P&R Store
MULTI-BRACE AT BOTTOM CHORD LEVEL
DOUBLE PURLIN
DOUBLE PURLIN
TIMBER STRUT AT BOTTOM CHORD
MULTI-BRACE AT TOP CHORD LEVEL
NOTES:
DOUBLE PURLINS
MULTI-BRACE AT TOP-CHORD LEVEL
MULTI-BRACE AT BOTTOM-CHORD LEVEL
STRUT AT BOTTOM CHORD

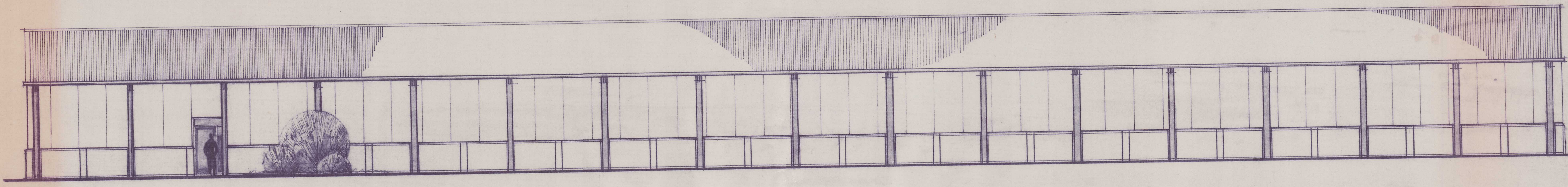
11 Explanatory Notes

- This assessment contains the professional opinion of Resonant as to the matters set out herein, in the light of the information available to it during preparation, using its professional judgment and acting in accordance with the standard of care and skill normally exercised by professional engineers providing similar services in similar circumstances. No other express or implied warranty is made as to the professional advice contained in this report.
- The assessment is also based on information that has been provided to Resonant from other sources or by other parties. The assessment has been prepared strictly on the basis that the information that has been provided is accurate, complete and adequate. To the extent that any information is inaccurate, incomplete or inadequate, Resonant takes no responsibility and disclaims all liability whatsoever for any loss or damage that results from any conclusions based on information that has been provided to Resonant.
- We have prepared this report in accordance with the brief as provided and our terms of engagement. The information contained in this report has been prepared by Resonant at the request of its client, Rangitikei District Council and is exclusively for its use and reliance. It is not possible to make a proper assessment of this assessment without a clear understanding of the terms of engagement under which it has been prepared, including the scope of the instructions and directions given to and the assumptions made by Resonant. The assessment will not address issues which would need to be considered for another party if that party's particular circumstances, requirements and experience were known and, further, may make assumptions about matters of which a third party is not aware. No responsibility or liability to any third party is accepted for any loss or damage whatsoever arising out of the use of, or reliance on this assessment by any third party.

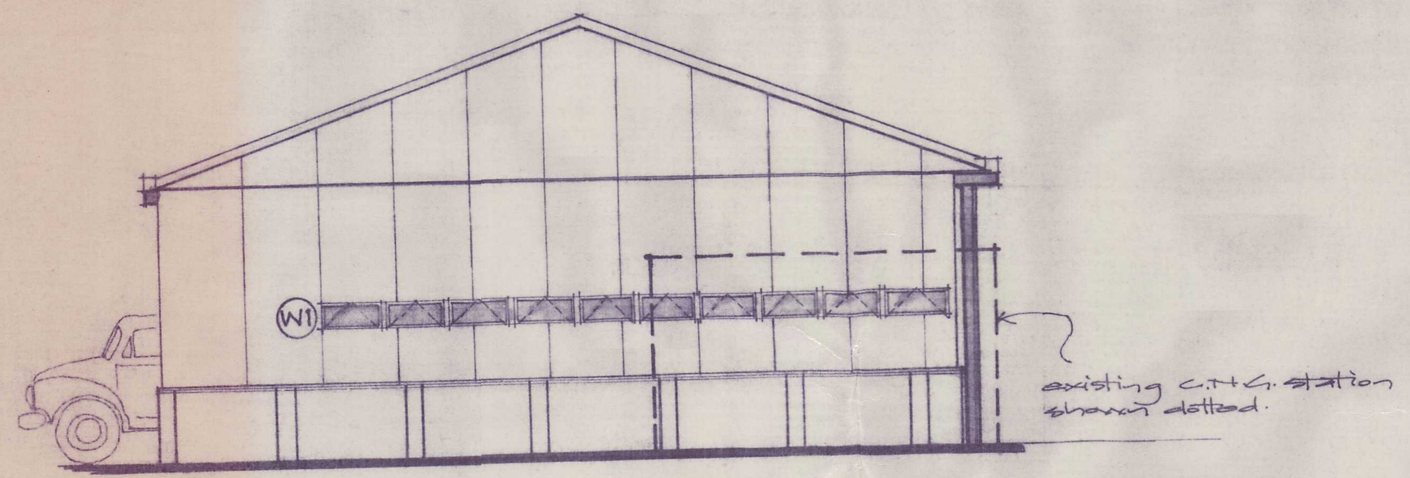
APPENDIX A

EXISTING DRAWINGS

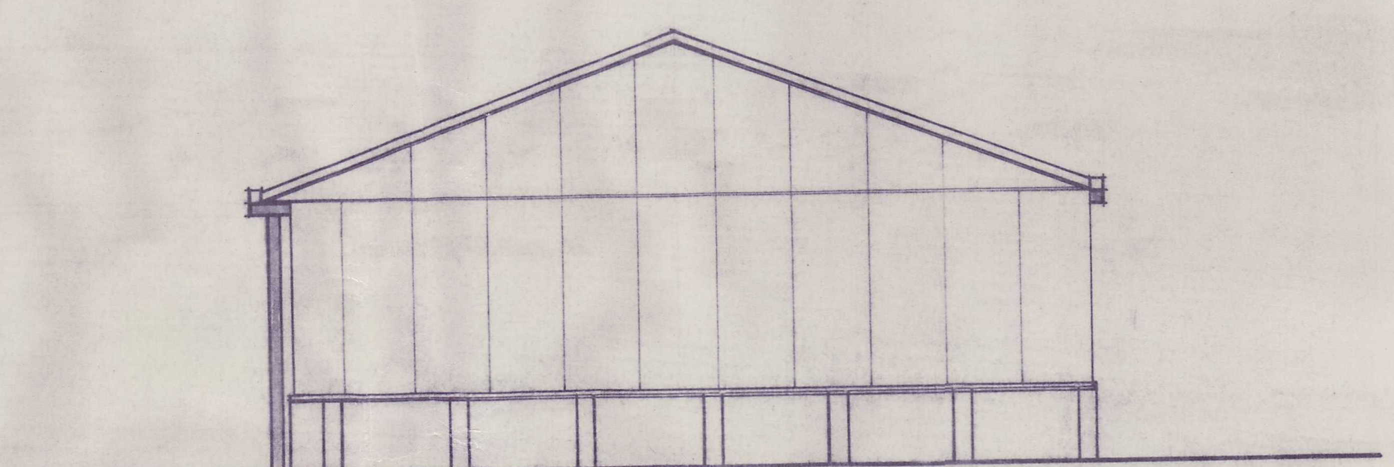

river run
base course
sand
pea metal
crushed dust
plant mix
existing 300 mm ø 1:100
NB :- Siteworks not in contract (see spec).
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17
existing manhole
d.p.
donko
Entry
P&R foreman
Borough overseer
plumbers workshop
& Carpenters
lube bay
garages
P&R Store
storeman
up
women
men
store
waterworks fittings
roadworks
entry
pipe storage
mezzanine over shaded area
g.t.
t.v.
h.t.
EXISTING BUILDING
Rangitikei County Council c.n.g. station
new manhole
existing 300 mm ø
Baden Powell Street
King Street
invert 158.297
100 mm ø watermain
THIS SEWER IS NOW INSIDE BUILDING :
940
GT
MH
legend
water supply
sewer pipe
stormwater pipe
d.p. downpipe
g.t. gulley trap
manhole
h.t. hosetap
t.v. terminal vent
stopcock tobybox
'HUMES' catchpit
fire hose reel
all stormwater and sewer drains 100 mm at 1 in 60 fall.
north
amended 15 June 1982
amended 19 July 1982
amended Nov' 1982
MARTON BOROUGH COUNCIL
New Depot
Site & Drainage Plan
LAMONT, BYCROFT & PARTNERS
ARCHITECTS ENGINEERS VALUERS
TOWN PLANNERS LANDSCAPE ARCHITECTS
162 Wicksteed Street, Wanganui Phone 53-959
Scale 1:200
Designed BGB
Drawn JGK
Date 30.4.82
Job number 704
Sheet number WD 1



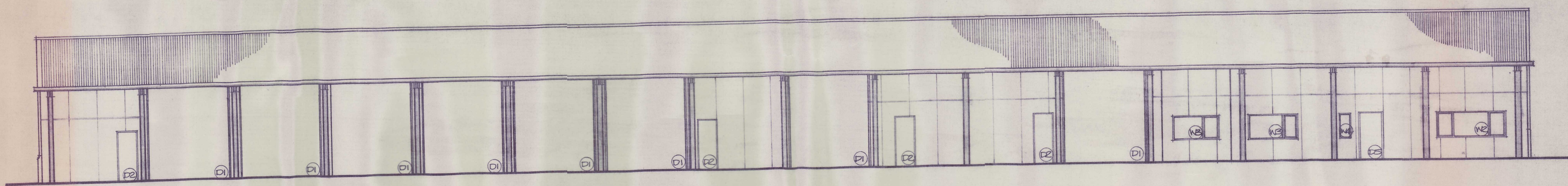
south west scale 1:100



north west scale 1:100

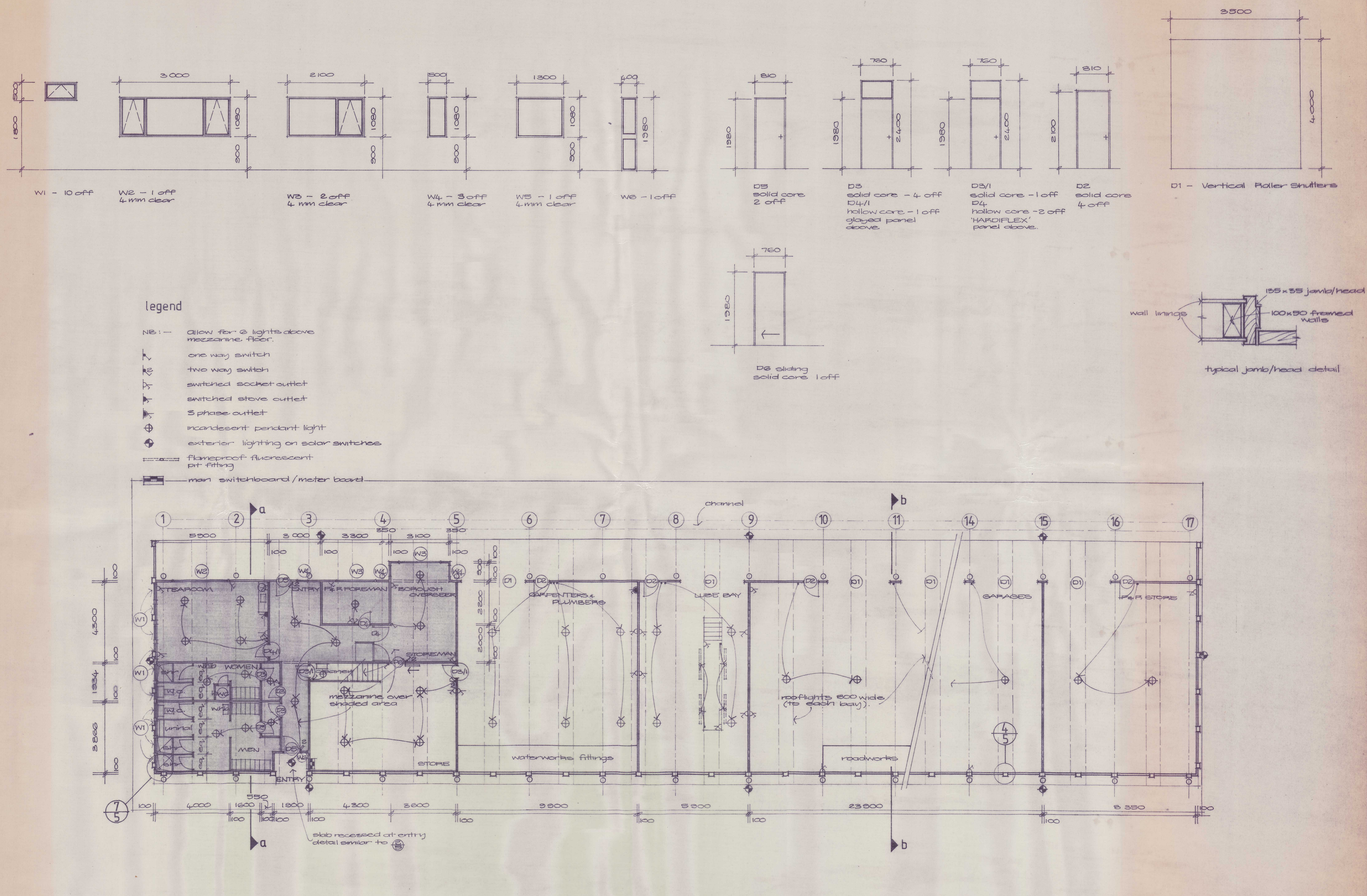


south east

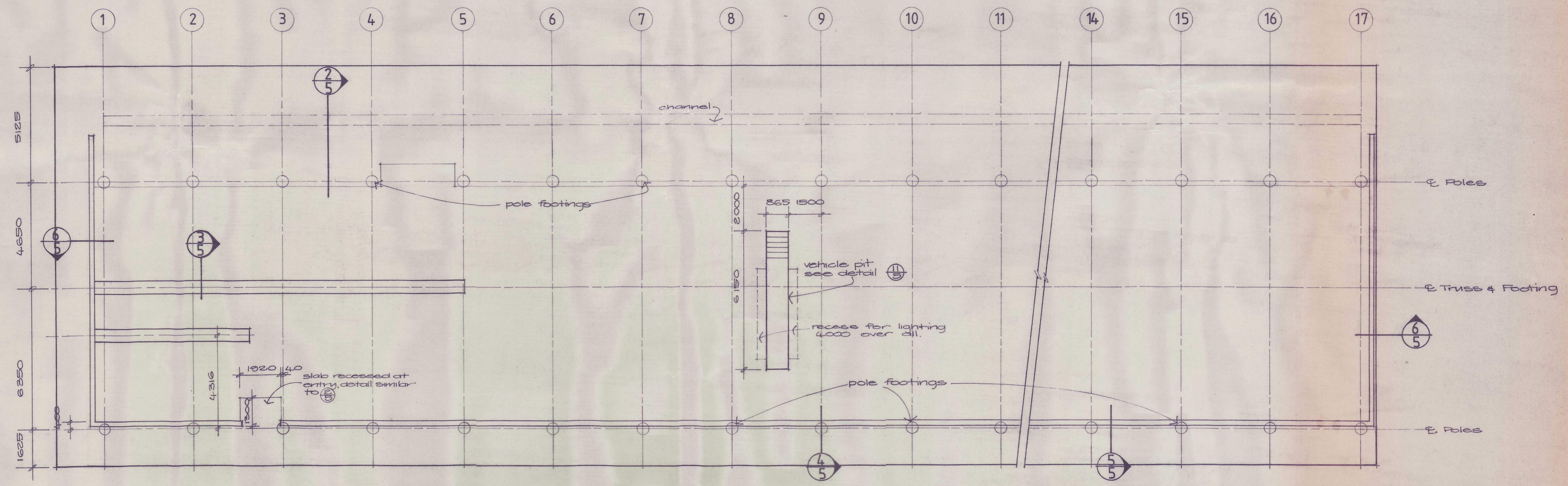


north east

MARTON BOROUGH COUNCIL New Depot	Job number 704
Elevations	Sheet number WD 2
LAMONT, BYCROFT & PARTNERS ARCHITECTS ENGINEERS VALUERS TOWN PLANNERS LANDSCAPE ARCHITECTS 162 Wicksteed Street, Wanganui Phone 53-959	Scale: 1:100 Designed: BCB Drawn: KK Date: 30 4 82

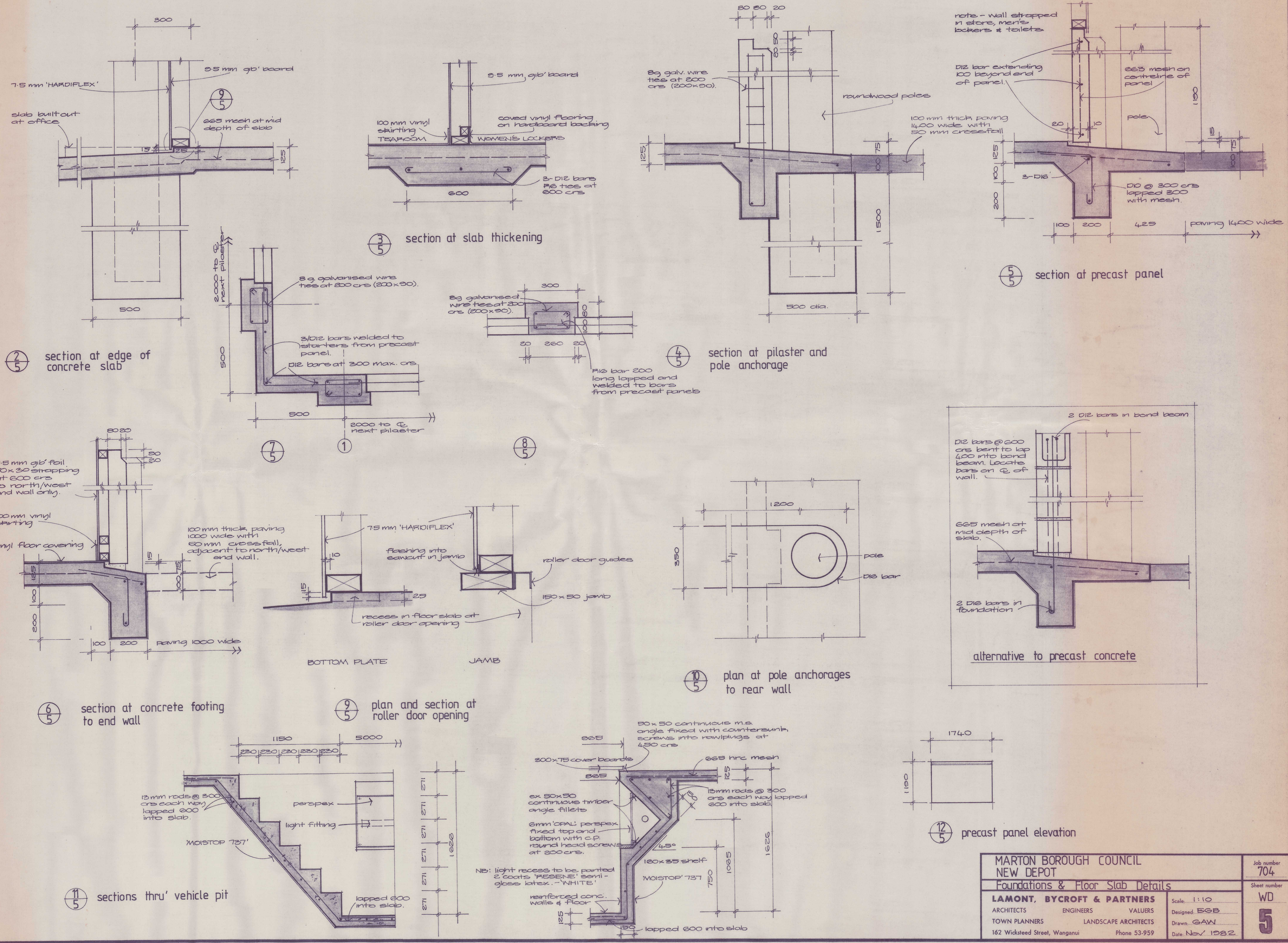


MARTON BOROUGH COUNCIL NEW DEPOT	Job number 704
Floor Plan Window / Door Sched. Electrical	Sheet number WD
LAMONT, BYCROFT & PARTNERS ARCHITECTS ENGINEERS VALUERS TOWN PLANNERS LANDSCAPE ARCHITECTS 162 Wicksteed Street, Wanganui Phone 53-959 Scale: 1:100 1:50 1:5 Designed: EGB Drawn: GAW Date: Nov' 1982	3



1/4 floor slab / foundation plan

MARTON BOROUGH COUNCIL NEW DEPOT		Job number 704
Floor Slab / Foundation Plan		Sheet number WD
LAMONT, BYCROFT & PARTNERS ARCHITECTS ENGINEERS VALUERS TOWN PLANNERS LANDSCAPE ARCHITECTS 162 Wicksteed Street, Wanganui Phone 53-959	Scale: 1:100 Designed: EBB. Drawn: GAW. Date: Nov '1982	4



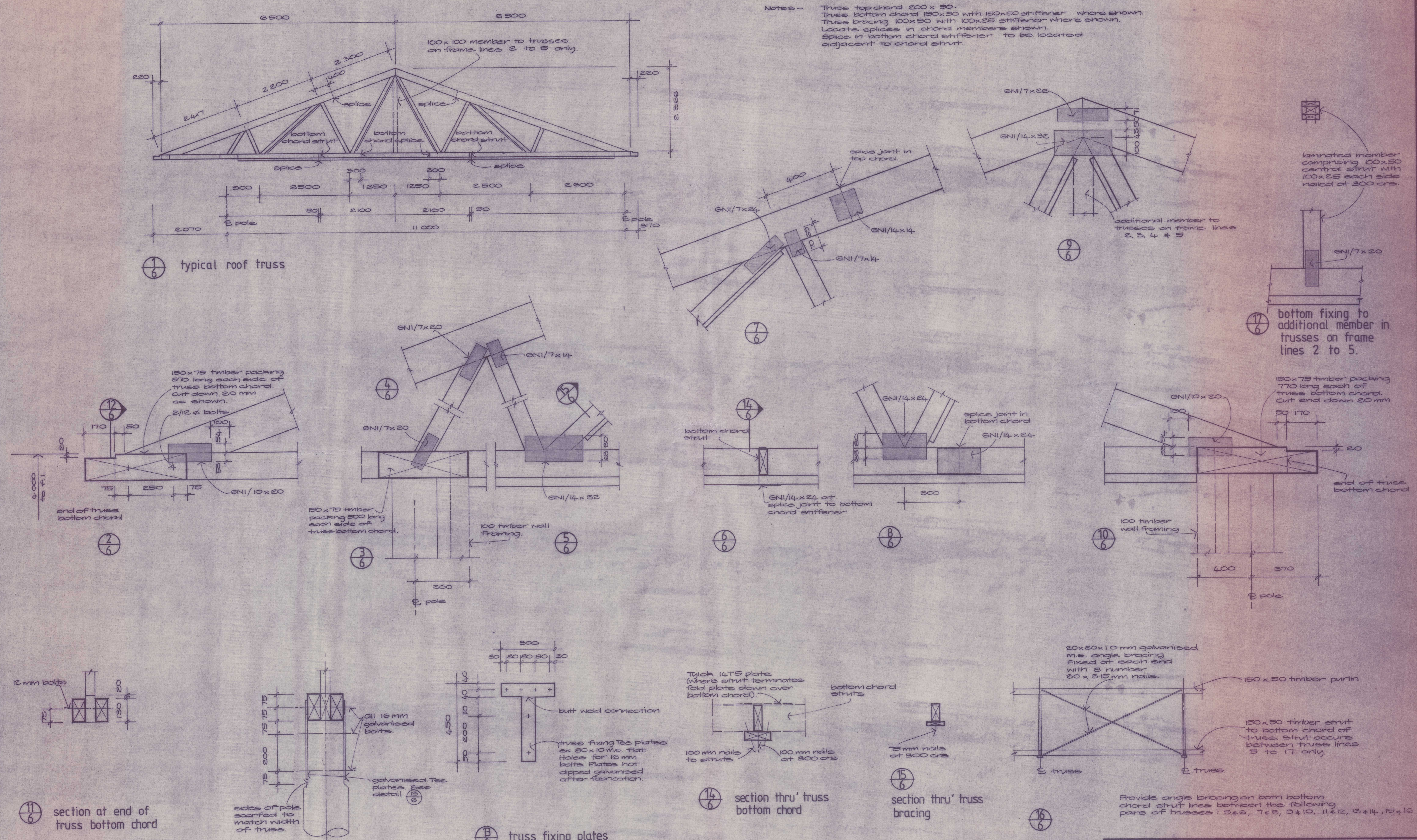
MARTON BOROUGH COUNCIL
NEW DEPOT
Foundations & Floor Slab Details

Job number 704

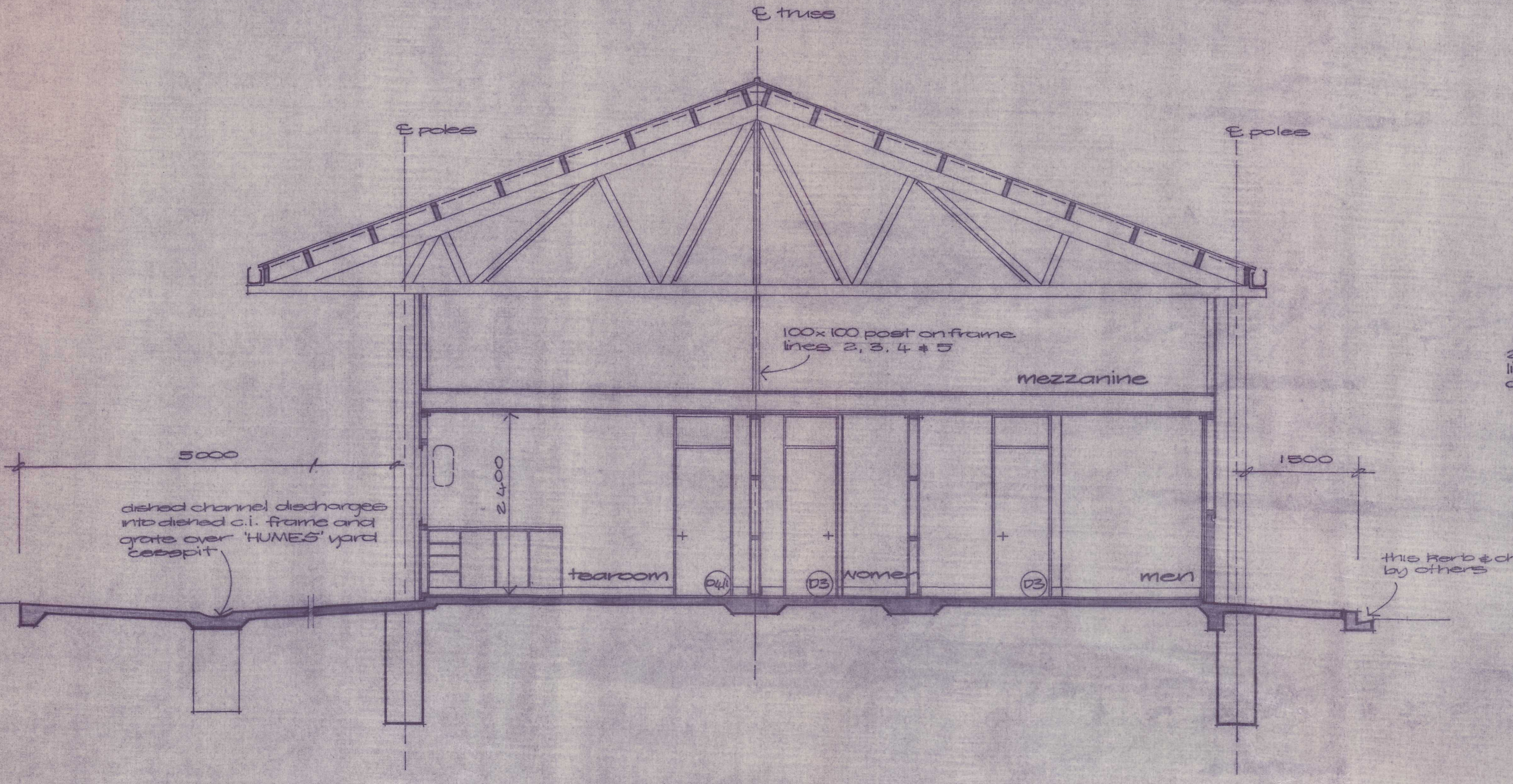
Sheet number WD 5

LAMONT, BYCROFT & PARTNERS
ARCHITECTS ENGINEERS VALUERS
TOWN PLANNERS LANDSCAPE ARCHITECTS
162 Wicksteed Street, Wanganui Phone 53-959

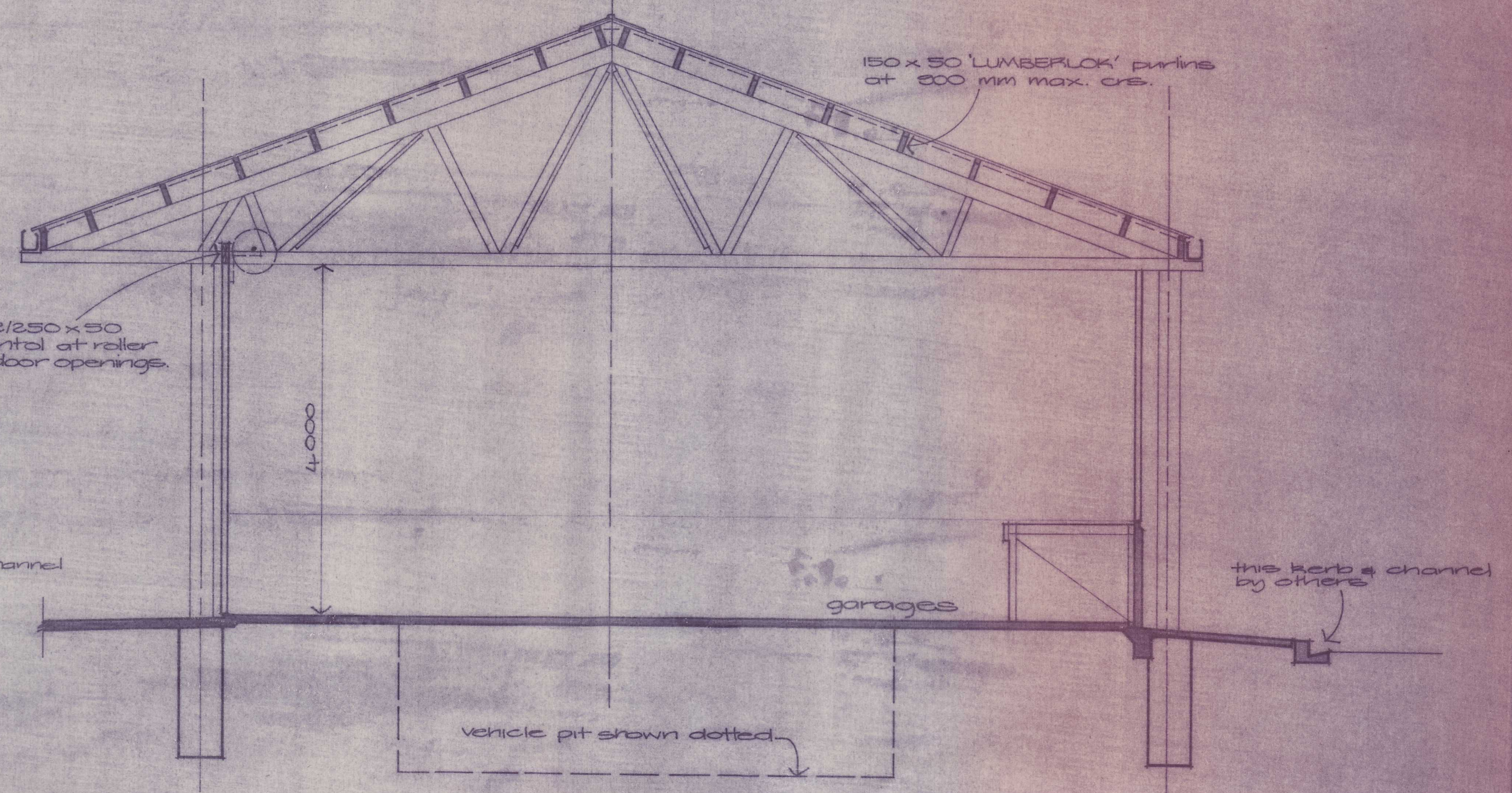
Scale 1:10
Designed EGB
Drawn GAW
Date Nov' 1982



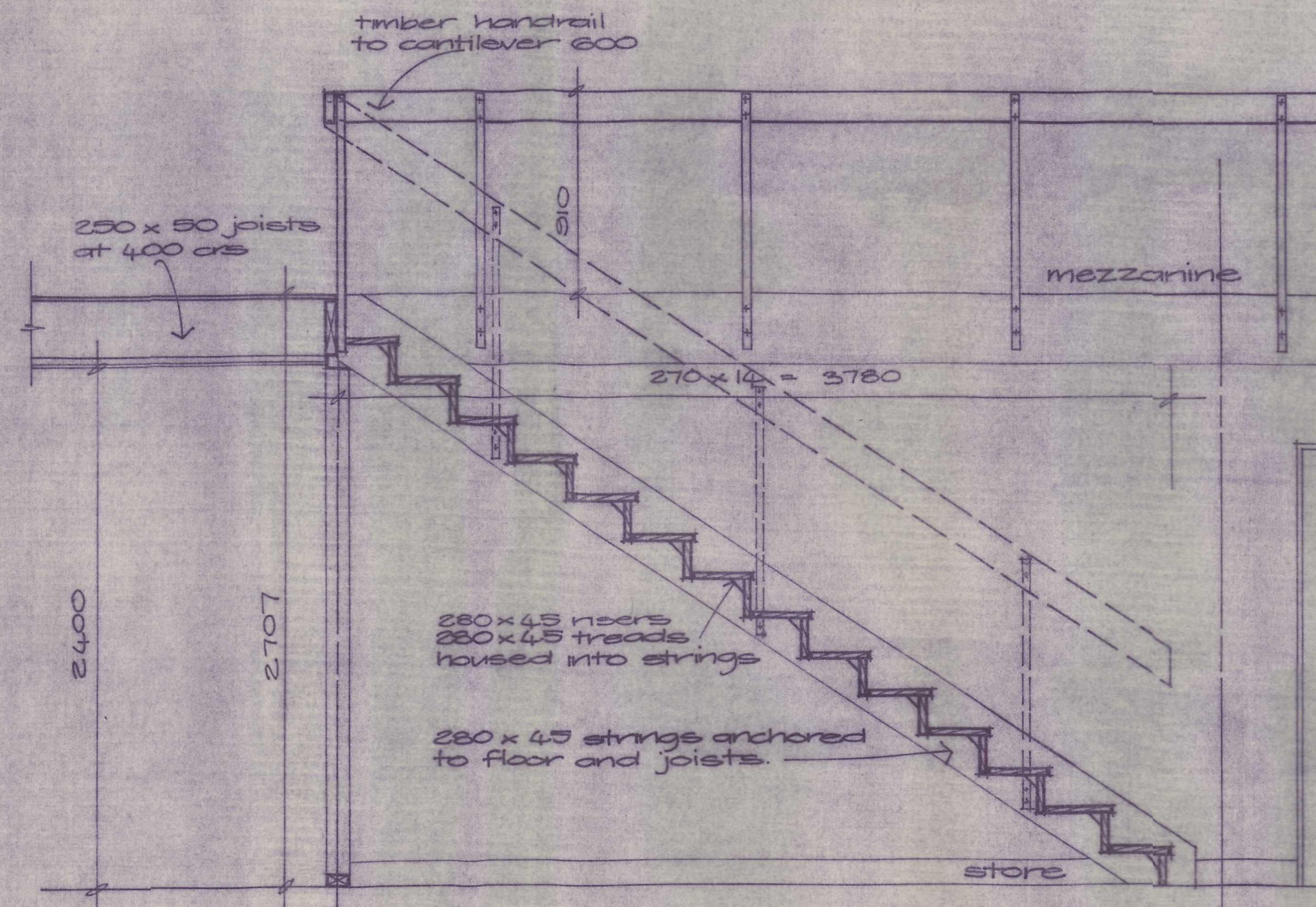
MARTON BOROUGH COUNCIL NEW DEPOT		Job number 704
Roof Framing and Details		Sheet number WD 6
LAMONT, BYCROFT & PARTNERS ARCHITECTS ENGINEERS VALUERS TOWN PLANNERS LANDSCAPE ARCHITECTS 162 Wicksteed Street, Wanganui Phone 53-959	Scale: 1:10 1:50 Designed: EGB Drawn: GAW Date: Oct' 1982	

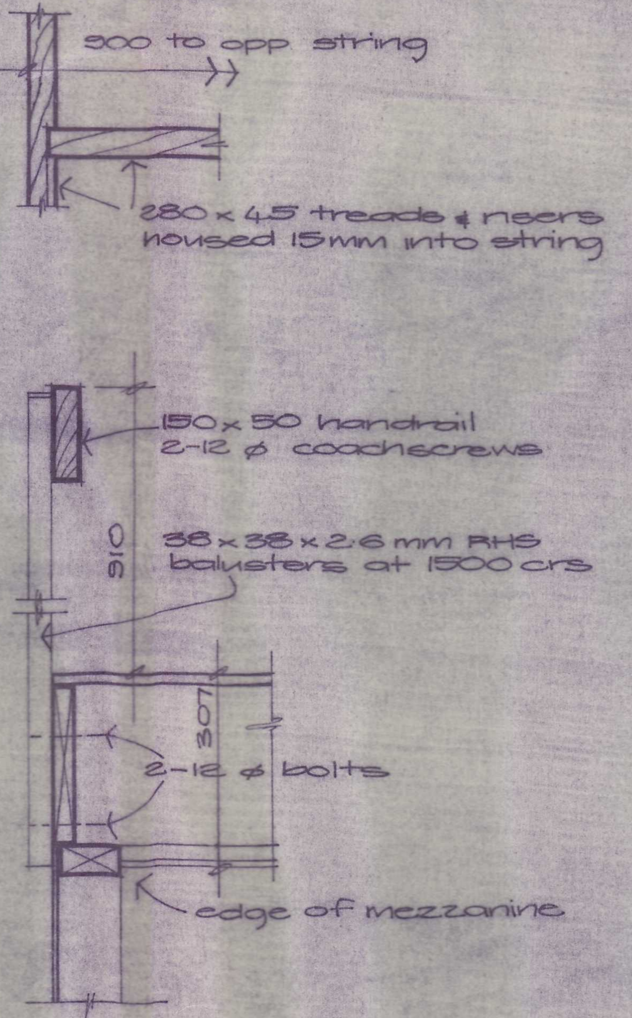


section a-a 1:50

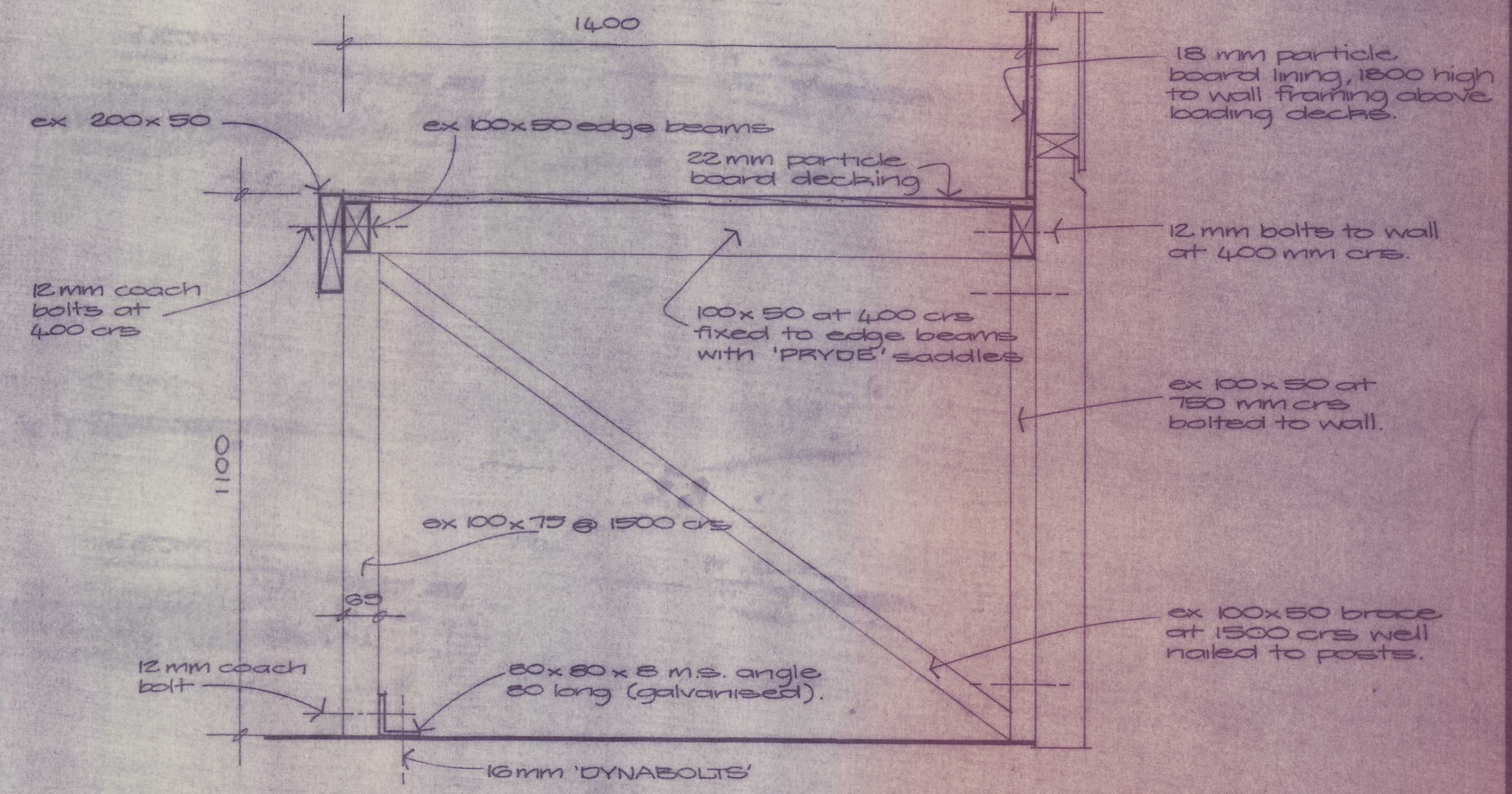


section b-b 1:50



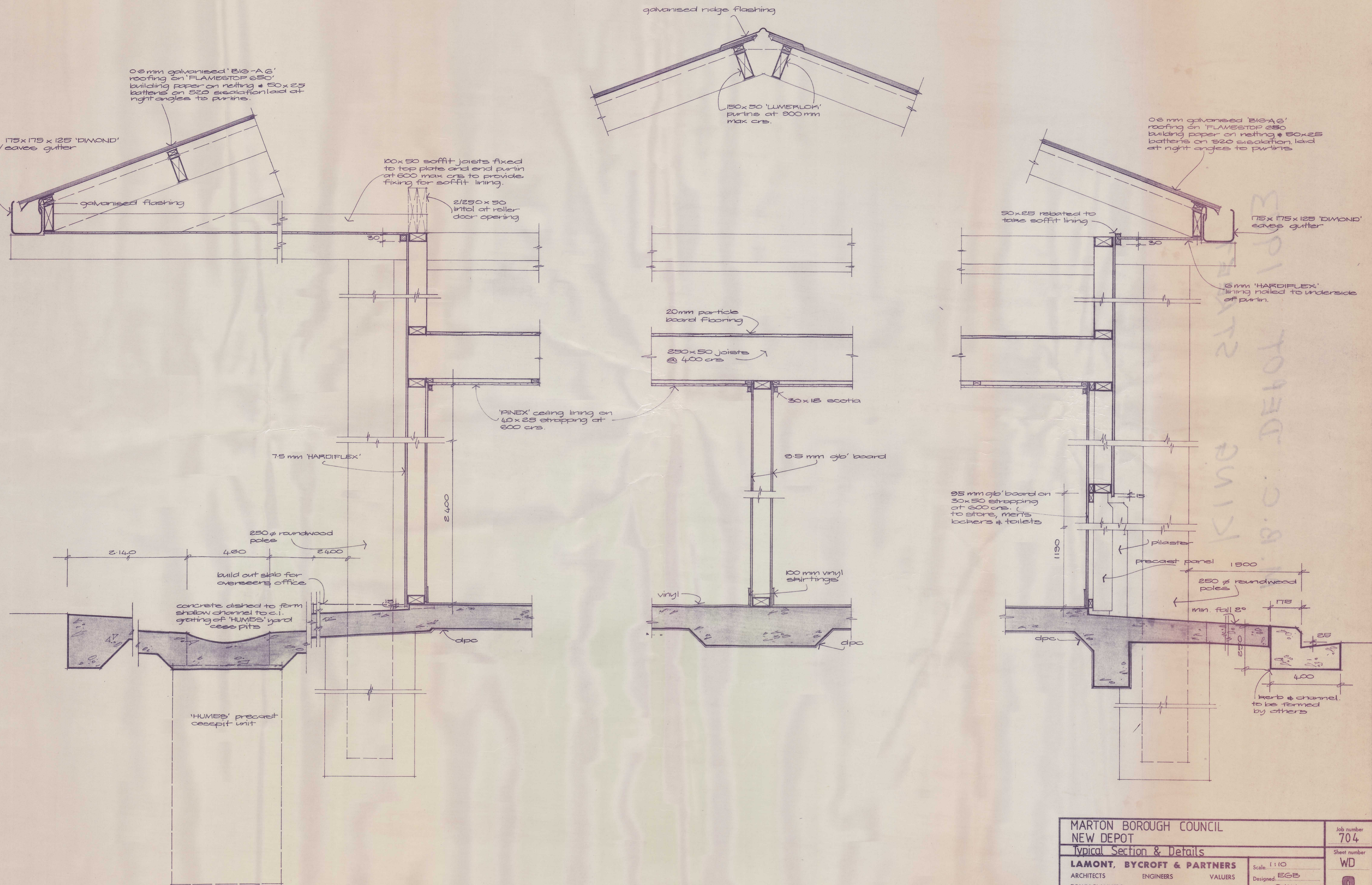


section at stair and mezzanine 1:25



platform to garages and plumbers workshop 1:10

MARTON BOROUGH COUNCIL NEW DEPOT Cross Sections		Job number 704
LAMONT, BYCROFT & PARTNERS ARCHITECTS ENGINEERS VALUERS TOWN PLANNERS LANDSCAPE ARCHITECTS 162 Wicksteed Street, Wanganui Phone 53-959	Scale: as shown Designed: BBB Drawn: GAW Date: Nov' 1982	Sheet number WD 7



MARTON BOROUGH COUNCIL
NEW DEPOT
Typical Section & Details

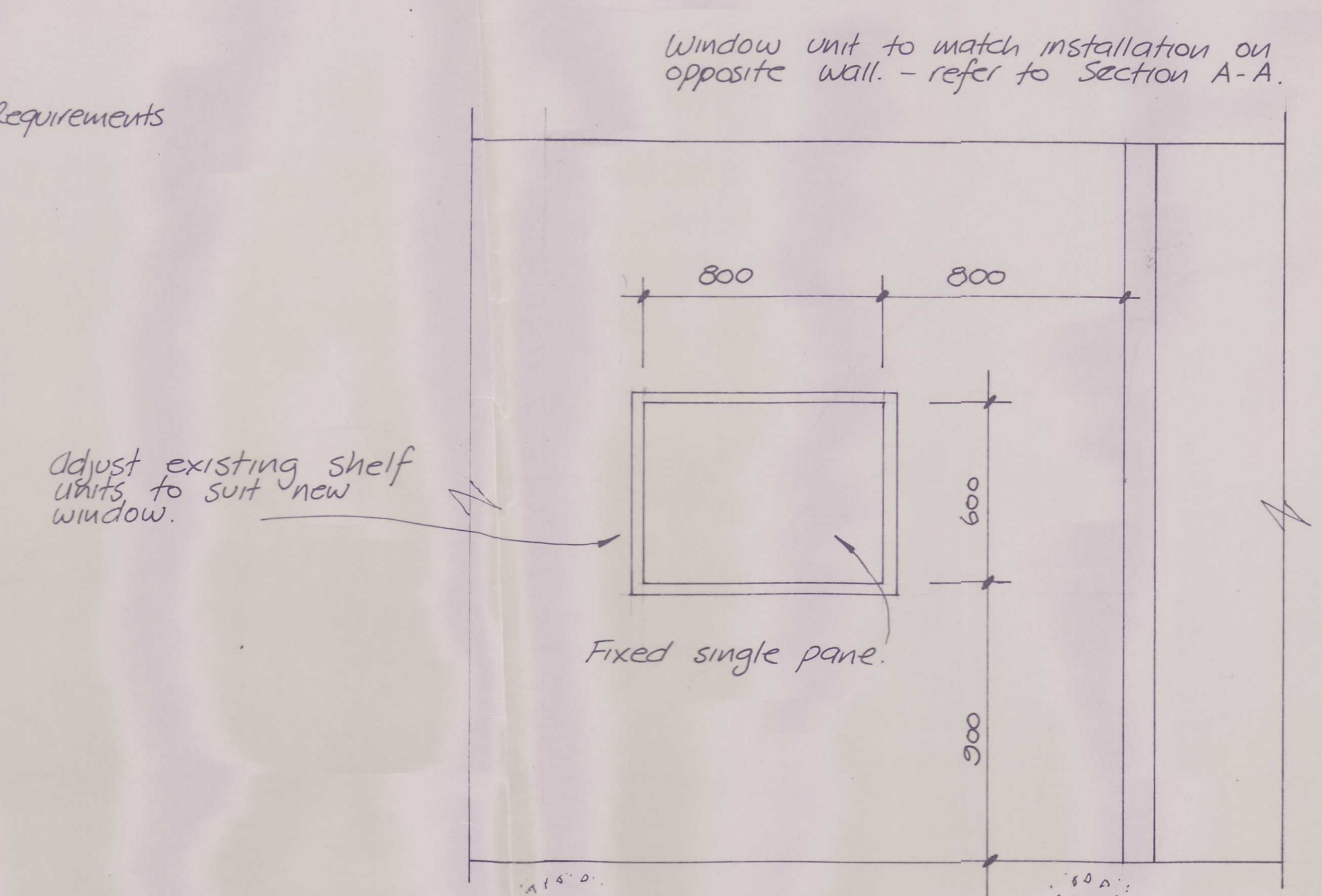
Job number	704
Sheet number	WD 8
Scale	1:10
Designed	EGB
Drawn	GAW
Date	Nov' 1982

LAMONT, BYCROFT & PARTNERS
ARCHITECTS ENGINEERS VALUERS
TOWN PLANNERS LANDSCAPE ARCHITECTS
162 Wicksteed Street, Wanganui Phone 53-959

PLAN

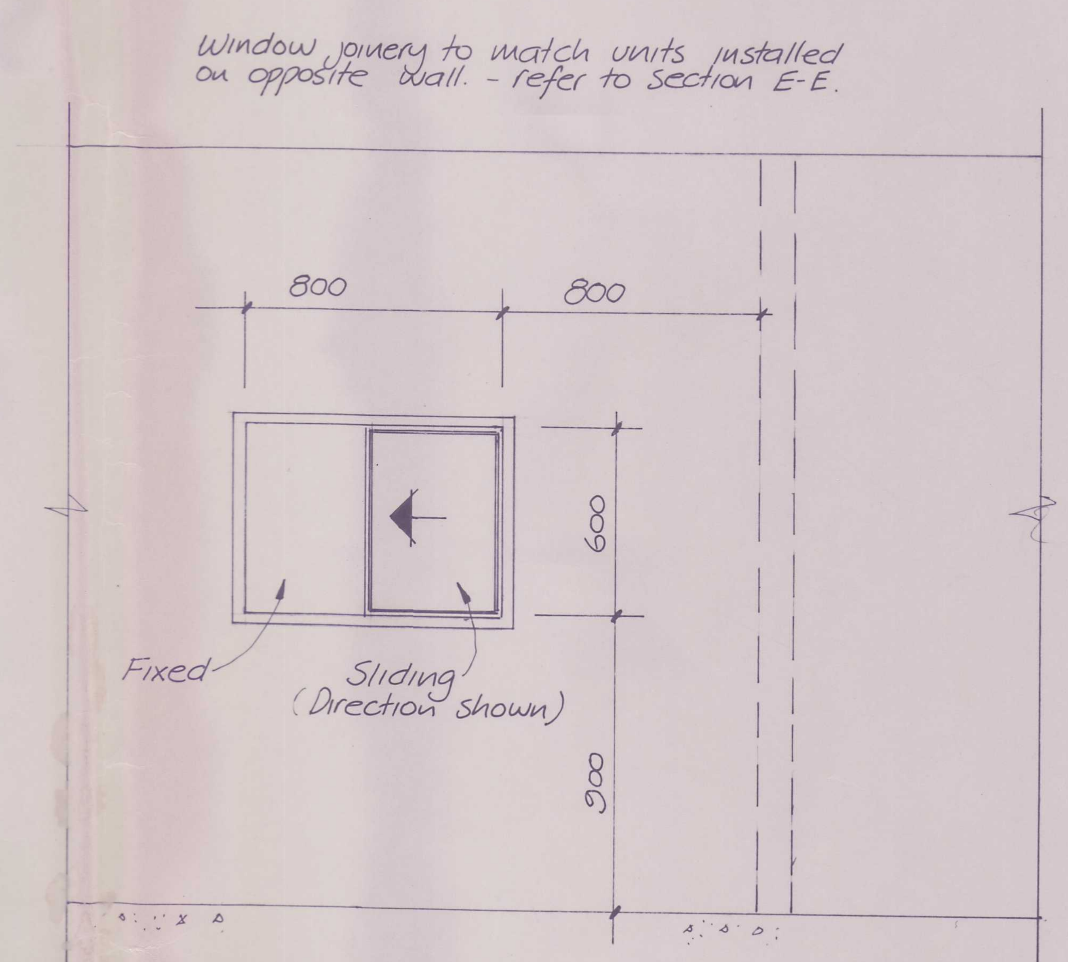
Scale 1:50

External/Internal Partition Wall Relocation Requirements



Section B-B

Scale 1:20



Section A-A

Scale 1:20

NAME	FLD BK	DATE
SURVEY		
DESIGN		
DRAWN B Knox		Oct 89
CHECKED		
AMENDED		

APPROVED [signature]

COUNTY ENGINEER

Date 10/89

RANGITIKEI COUNTY COUNCIL

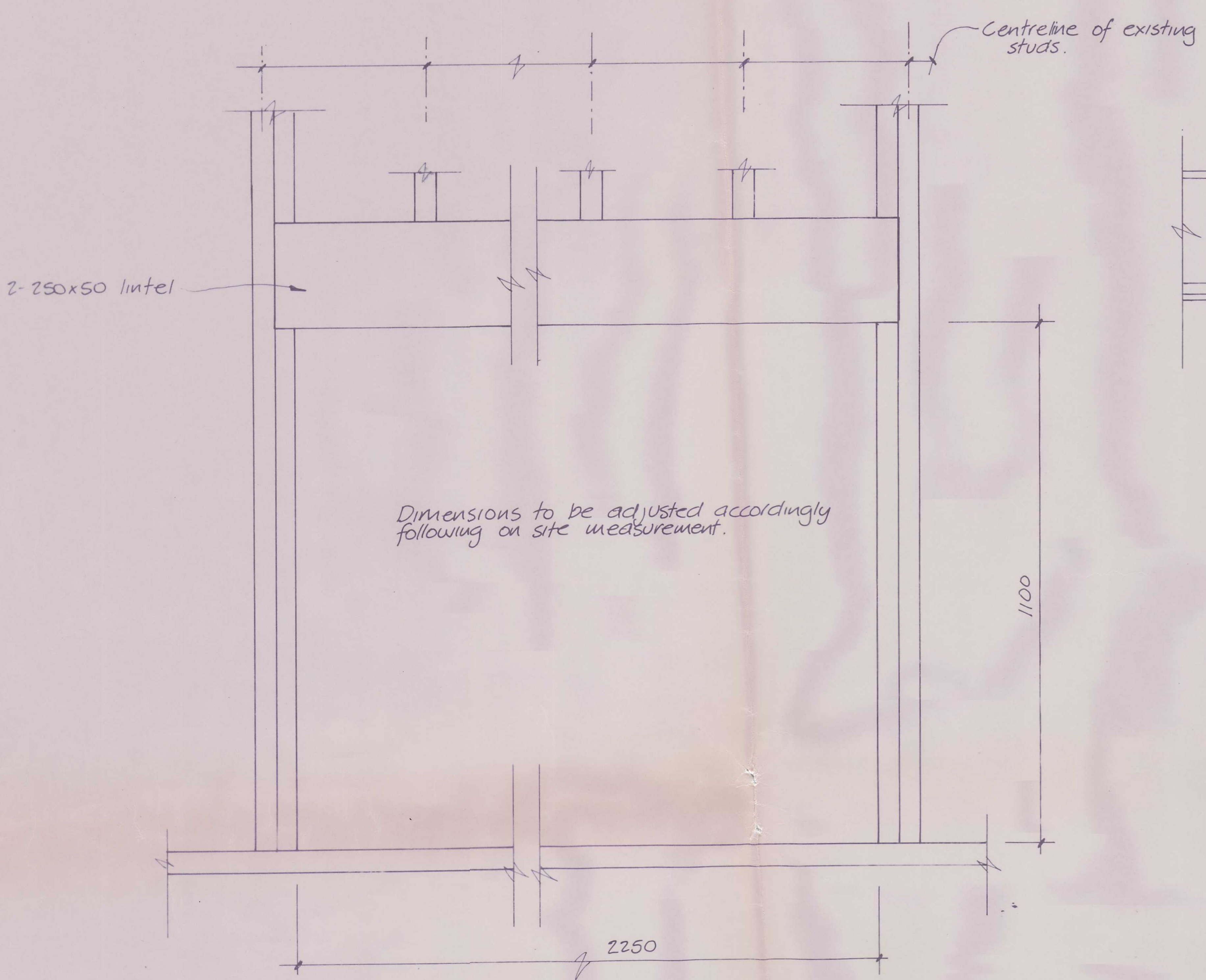
MARTON BOROUGH JUNCTION DEPOT ALTERATIONS

SCALE As Shown.

CON. No.

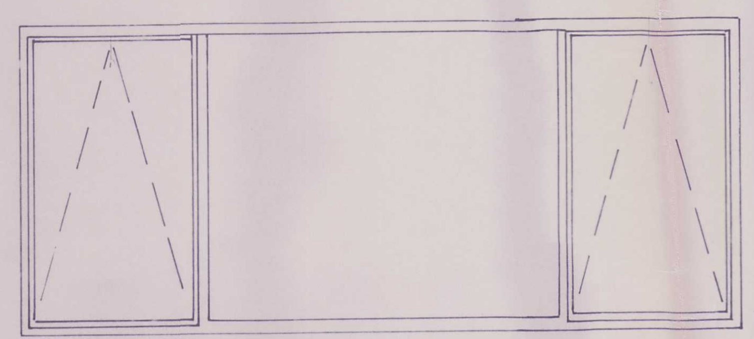
PLAN 1 OF 4

212 a



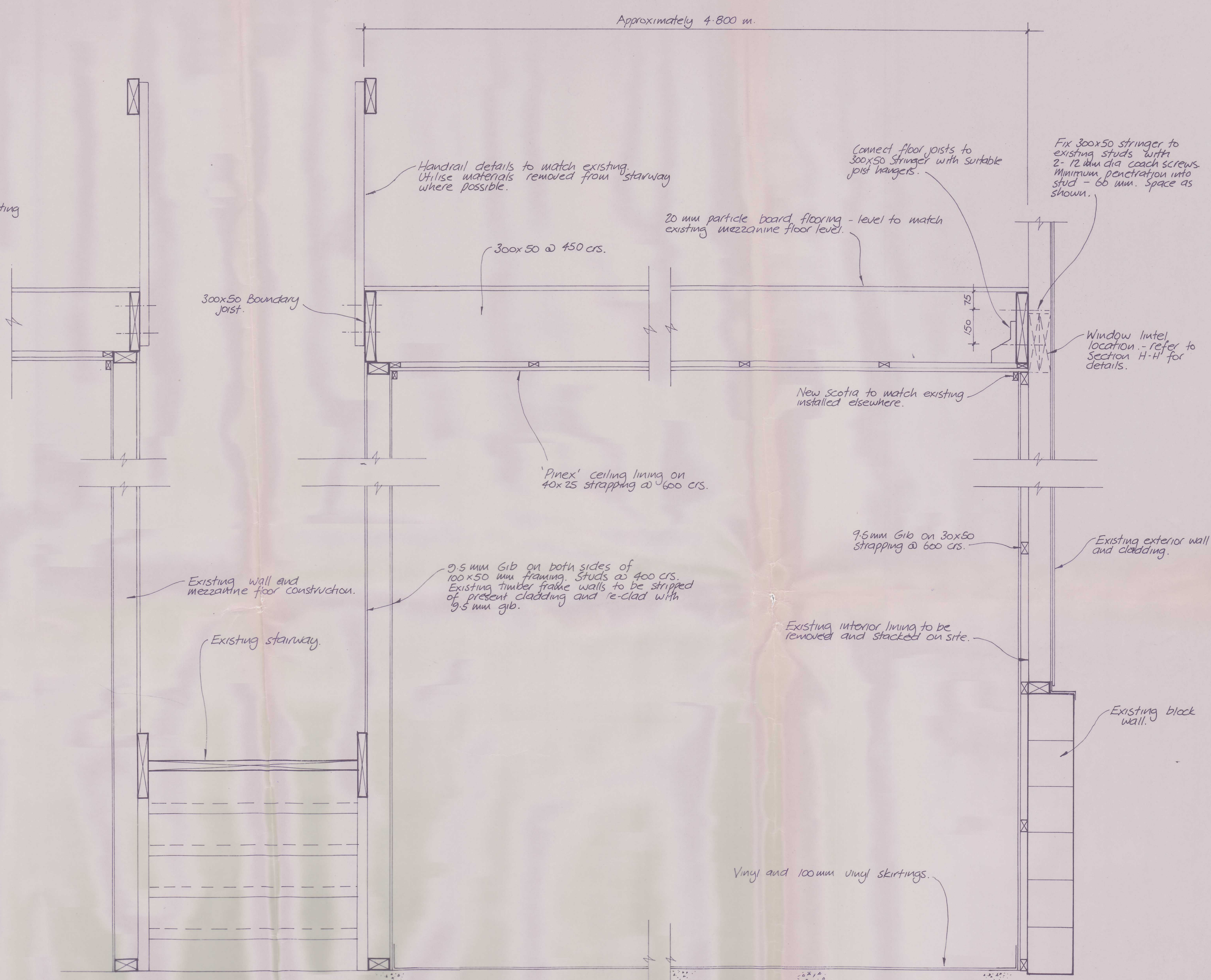
Scale 1:10

Section H-H (1 of 2)



n.t.s.

Section H-H (2 of 2)



Section C-C

Scale 1:10

NAME	FLD BK	DATE
SURVEY		
DESIGN		
DRAWN B.Knox	~	Oct 80
CHECKED		
AMENDED		

APPROVED

COUNTY ENGINEER

Date 10/89

RANGITIKEI COUNTY COUNCIL

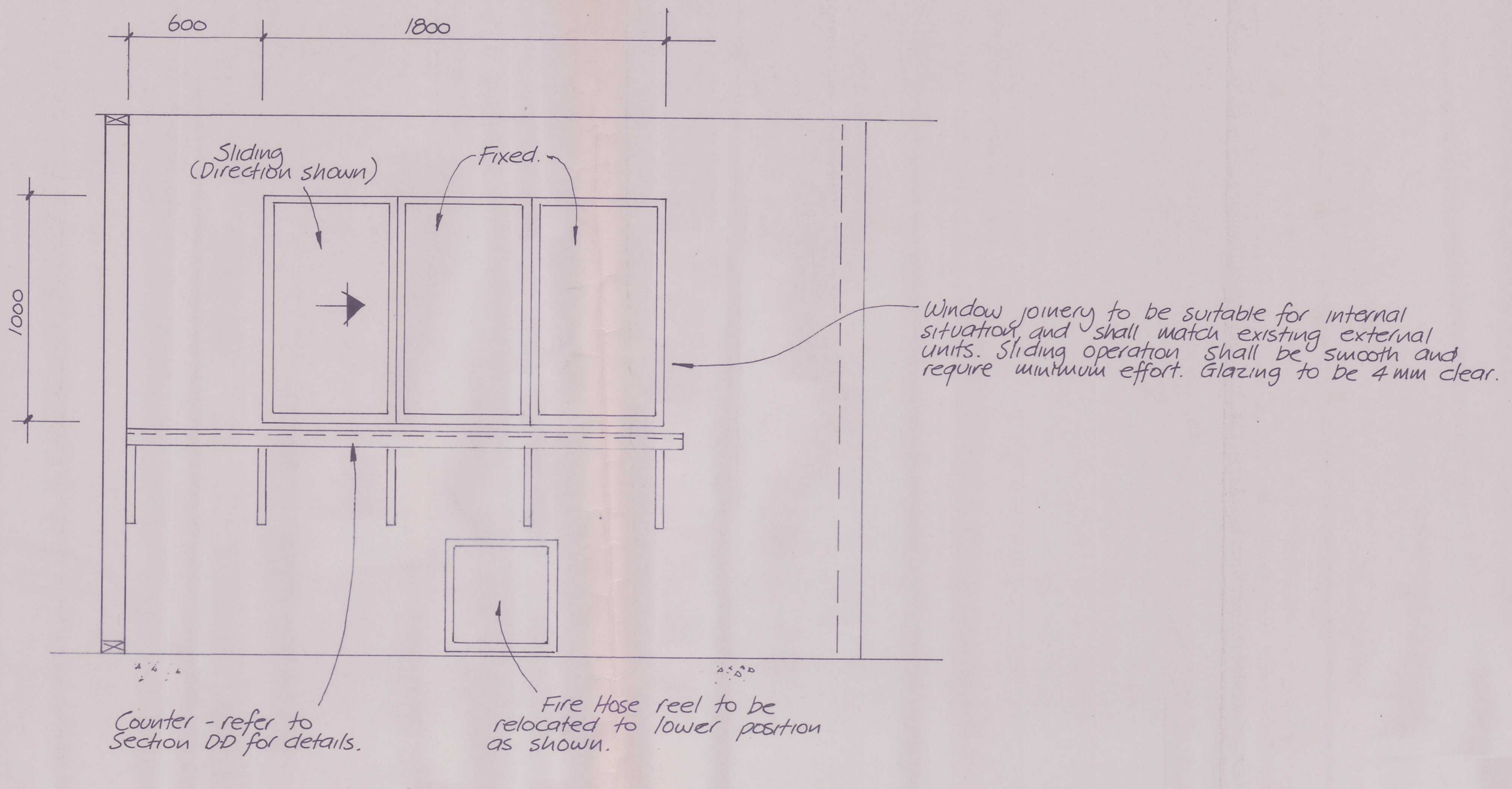
MARTON BOROUGH JUNCTION DEPOT ALTERATIONS.

SCALE As Shown.

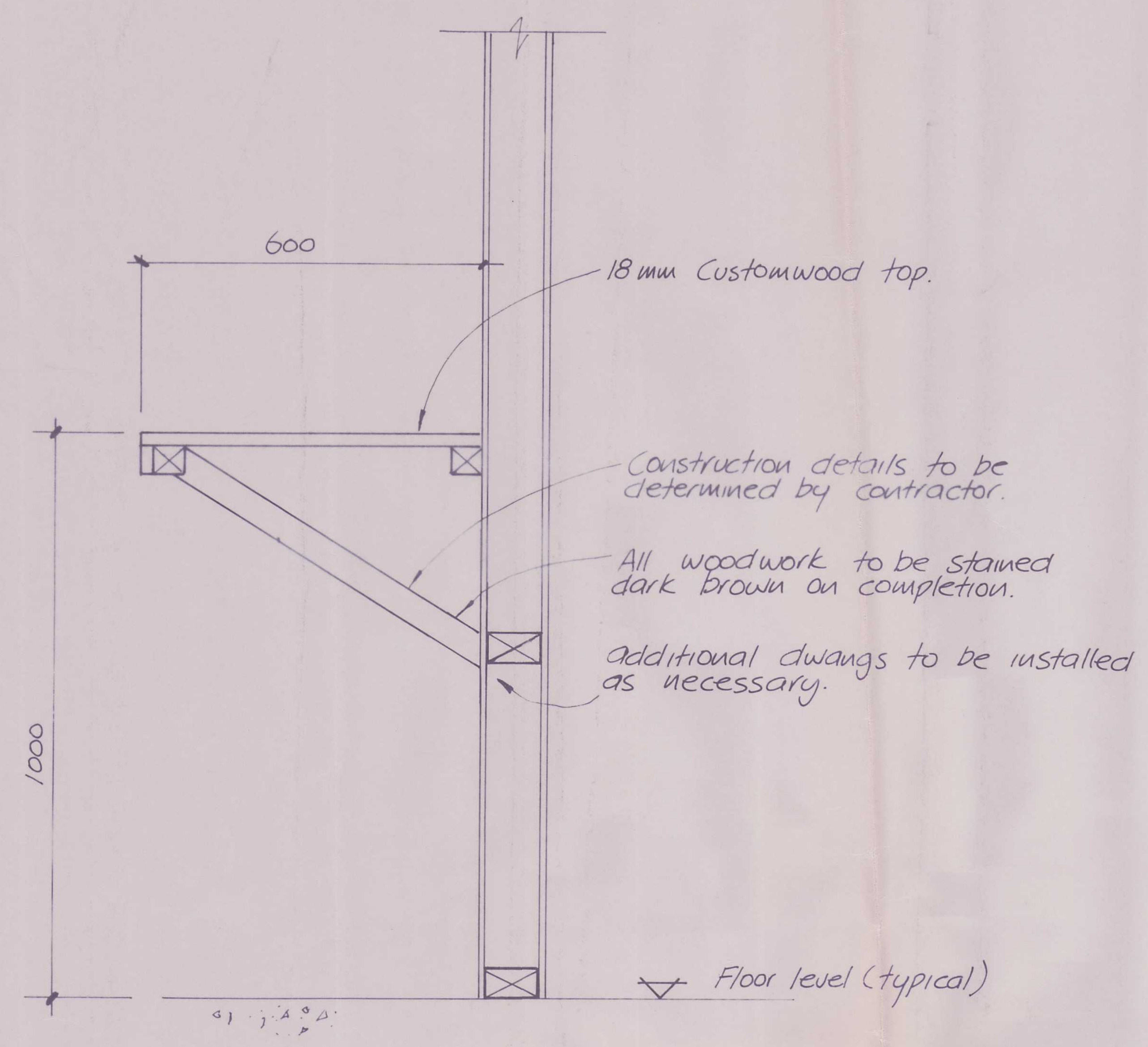
CON. No.

PLAN 2 OF 4

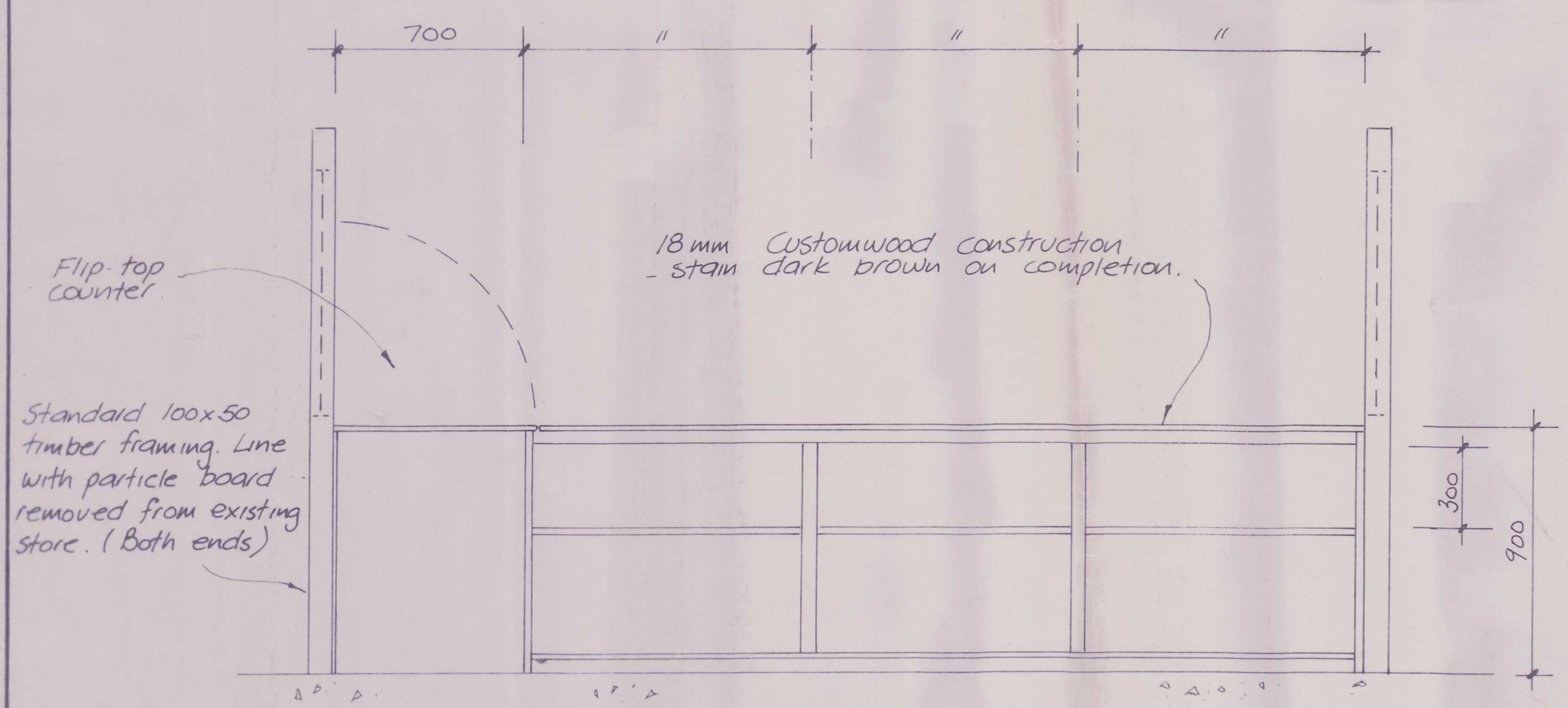
212a



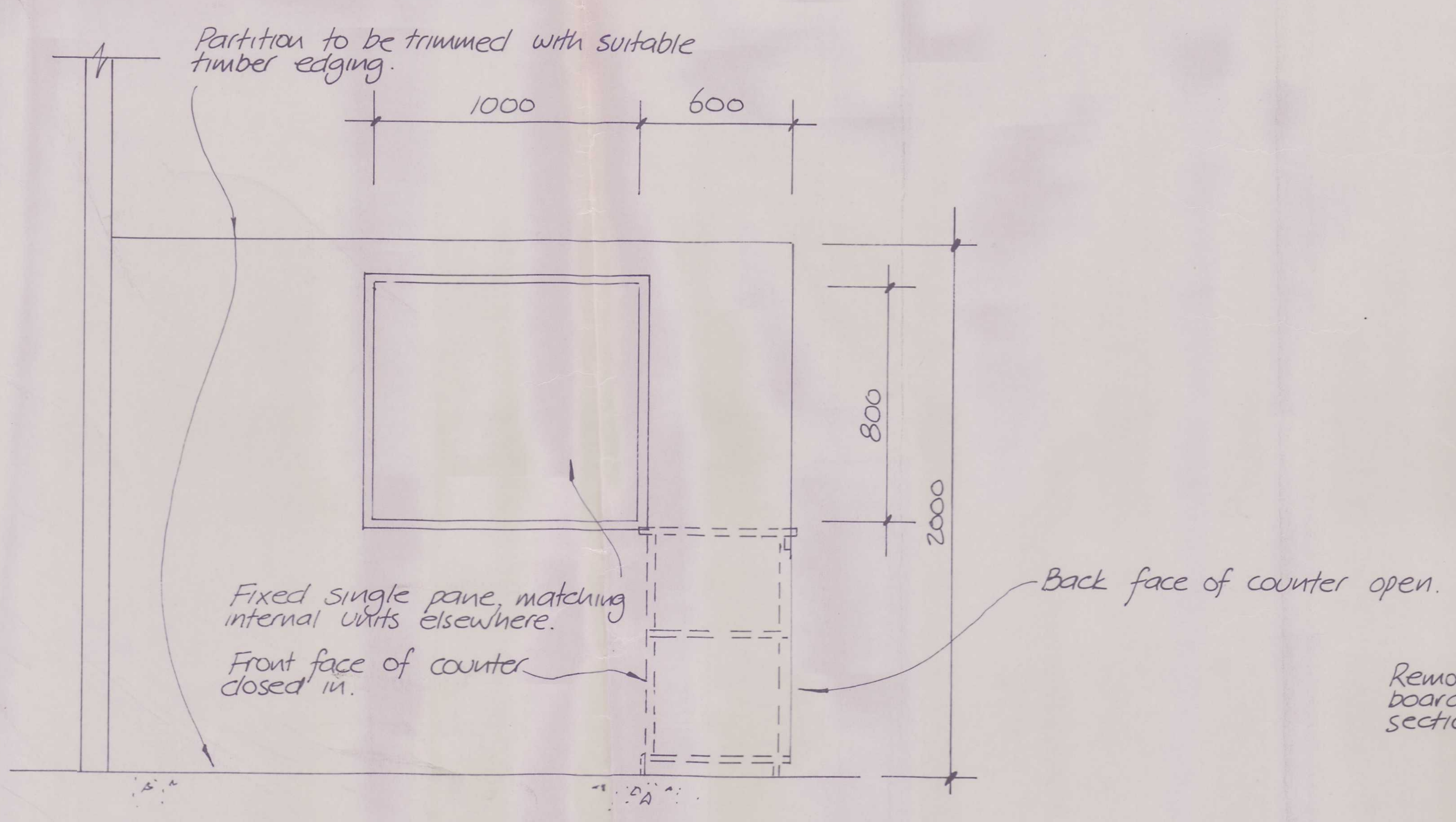
Section E-E
Scale 1:20



Section D-D
Scale 1:10

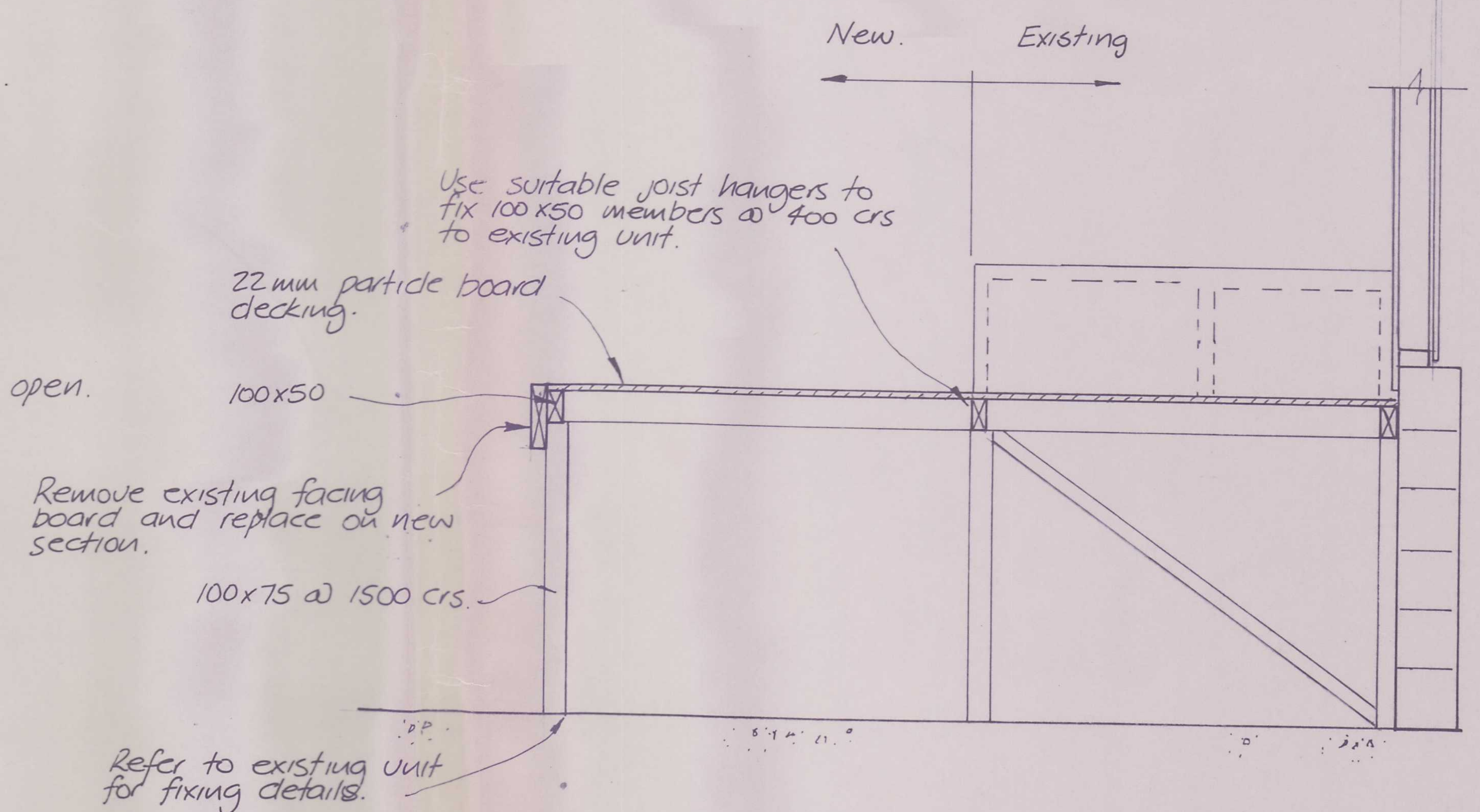


Section F-F
Scale 1:20



Section G-G
Scale 1:20

Note: Details to be repeated at opposite end of counter.



Section I-I
Scale 1:20

	NAME	FLD BK	DATE
SURVEY			
DESIGN			
DRAWN	B Knox		Oct '89
CHECKED			
AMENDED			

APPROVED
COUNTY ENGINEER
Date 10/89

RANGITIKEI COUNTY COUNCIL

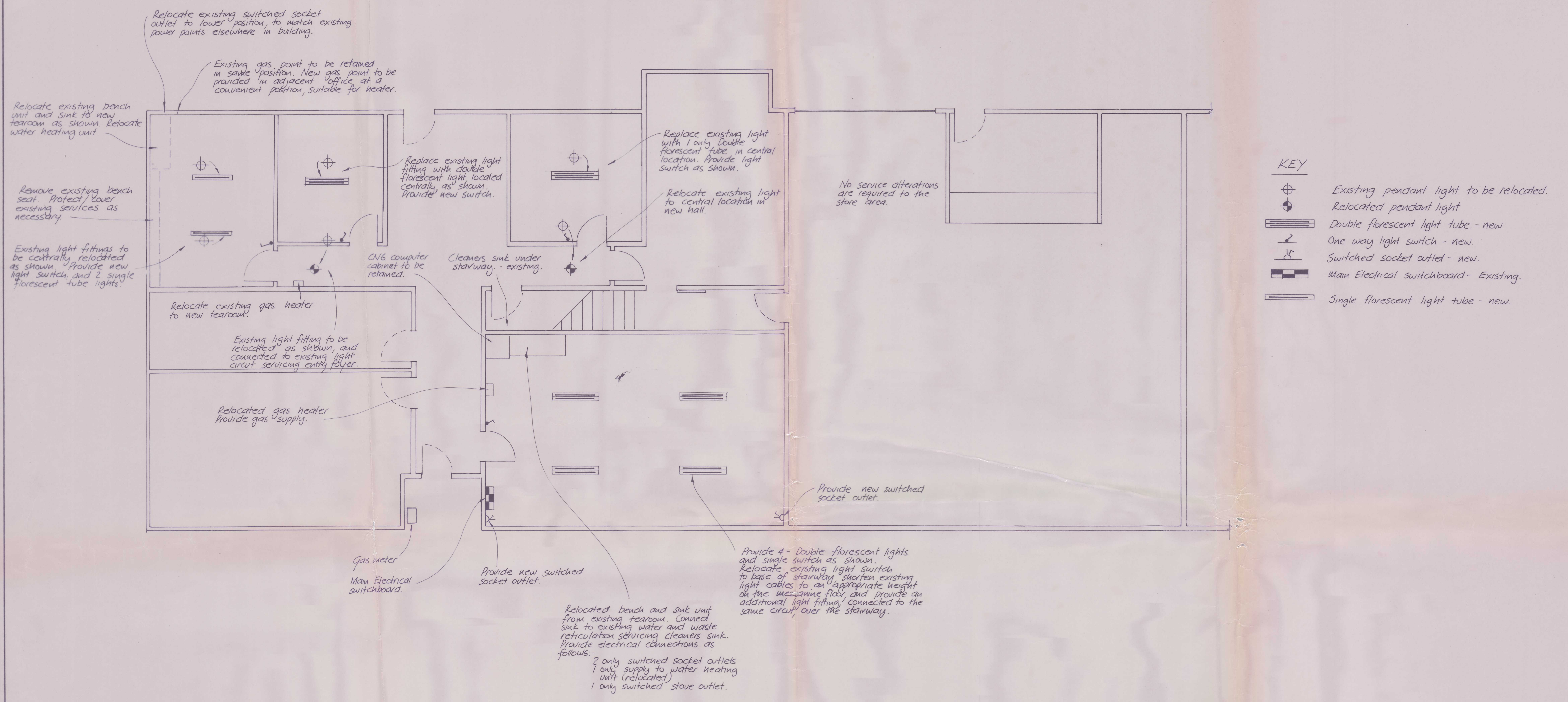
MARTON BOROUGH JUNCTION DEPOT ALTERATIONS

SCALE
As Shown.

CON. No.

PLAN 3 OF 4

212 a



PLAN

Service Relocation / Installation Details.

NOTE: Layout detailed based on new floorplan and excludes partition walls removed.

	NAME	FLD BK	DATE
SURVEY			
DESIGN			
DRAWN	B Knox		[illegible]
CHECKED			
AMENDED			

APPROVED [signature]
COUNTY ENGINEER
Date 10/89

RANGITIKEI COUNTY COUNCIL

MARTON BOROUGH JUNCTION DEPOT ALTERATIONS

SCALE 1:50

CON. No. ~

PLAN 4 OF 4

212

APPENDIX B

ASSESSMENT REPORT

1 - SCOPE

Estimation of seismic rating (%NBS) of Depot located at 7 King Street, Marton

2 - SUPPORT DOCS:

- LAMONT, BYCROFT & PARTNERS - Original Construction plans - Date: 1982
- NO-NAME - Depot Alteration Construction plans - Date: 1989
- NO NAME - Current internal layout - Date: 2020

3 - BUILDING INFORMATION

Gravity Loading:

- Timber framing Walls - OFFICE	0.35	kPa
- Timber framing Walls - DEPOT	0.3	kPa
- Depot Roof (no ceiling)	0.25	kPa
- Mezzanine	0.6	kPa

Live load:

- Roof LL	0.25	kPa
- Floor LL	1.5	kPa
- Stairs LL	2	kPa

Geometry:

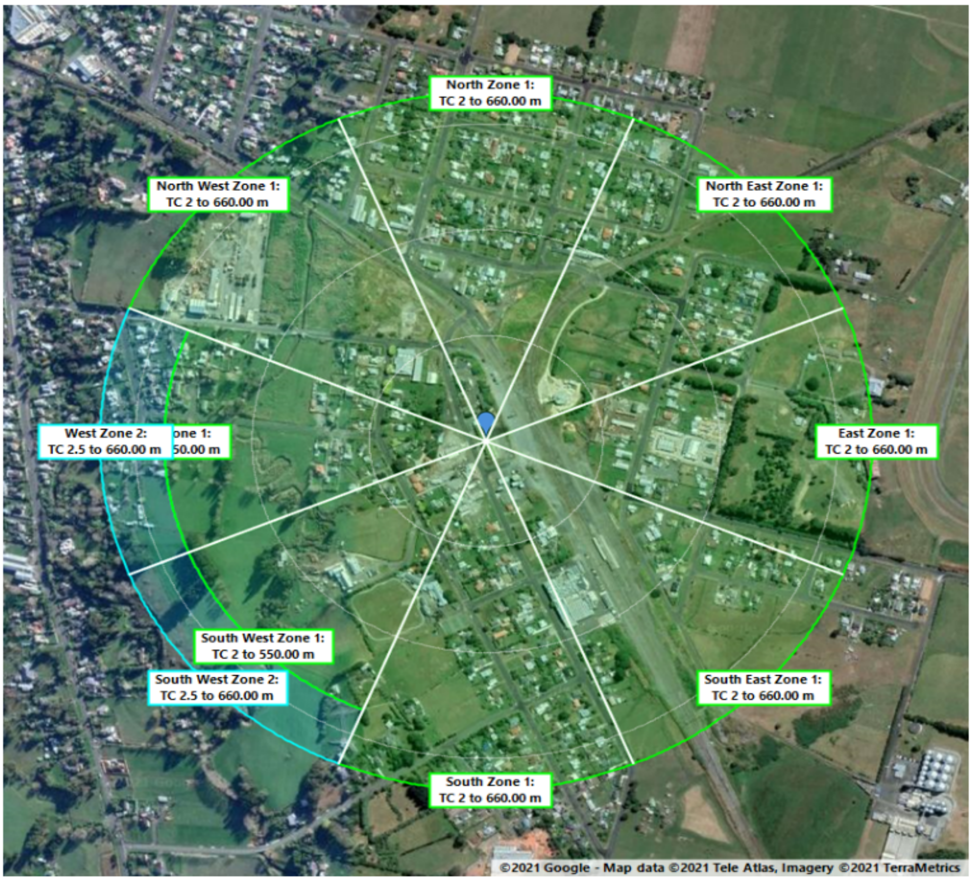
- Internal walls stud height	2.4	m
- External walls stud height	4	m
- Roof pitch	20	°

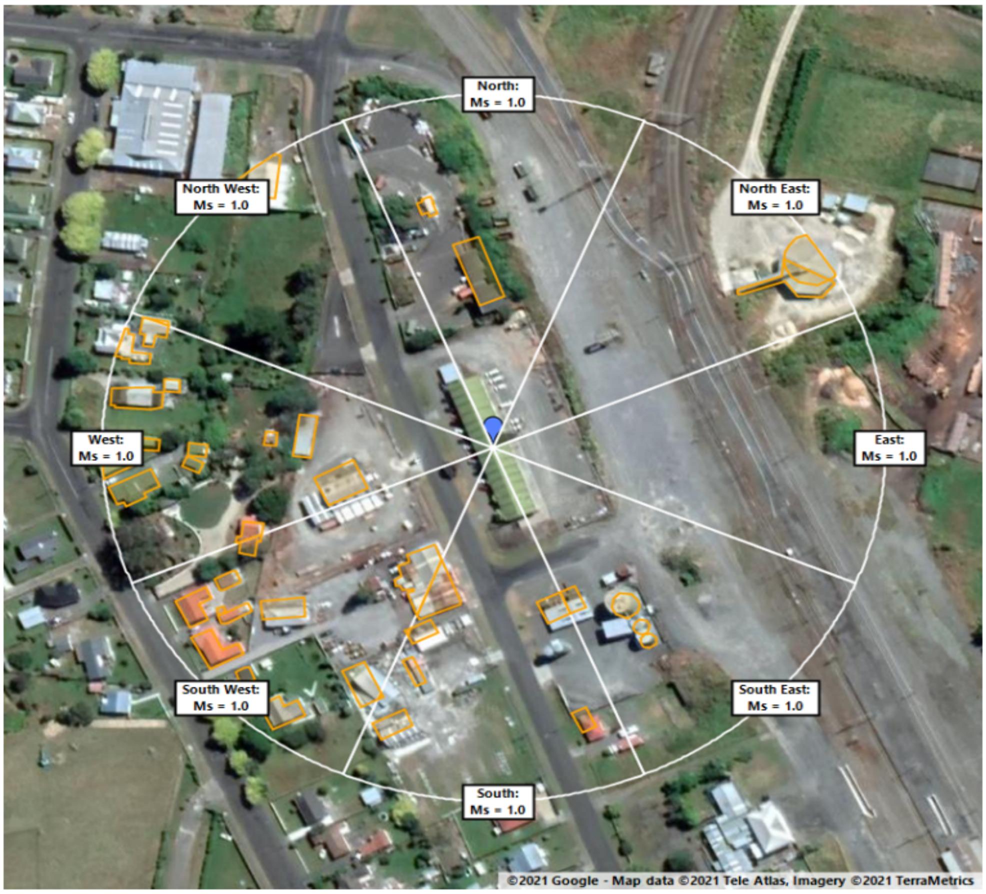
Soil Parameters (assumtions):

- Safe bearing pressure	$\phi_{sls}q_u$ =	70	kPa
- Undrained sheat Strength	S_u =	50	kPa

Wind Loads:

```
STRUCTURE:      OTHER            LATITUDE:       -40.085771      CRITICAL DIRECTION:   North West
HEIGHT (h):     8.00 m           LONGITUDE:      175.389221      Md:                   1.00
                                 ELEVATION:      140.00 m        Mc:                   1.00
                                                                 TC:                   2.00
                                 WIND REGION:    NZ2             Mz,cat:               0.9640
                                 ULTIMATE ARI:   500 YEARS       Ms:                   1.0
                                 ULTIMATE VR:    45 m/s          Mh:                   1.0
                                                                 Mlee:                 1.0
                                                                 Mel:                  1.0
                                                                 Mt:                   1.0
                                                                 Vdes,θ:               43.38 m/s
                                                                 qdes,θ:               1.1291 kPa
```


North Zone 1:
TC 2 to 660.00 m
North West Zone 1:
TC 2 to 660.00 m
North East Zone 1:
TC 2 to 660.00 m
West Zone 2:
TC 2.5 to 660.00 m
East Zone 1:
TC 2 to 660.00 m
South West Zone 1:
TC 2 to 550.00 m
South West Zone 2:
TC 2.5 to 660.00 m
South East Zone 1:
TC 2 to 660.00 m
South Zone 1:
TC 2 to 660.00 m
©2021 Google - Map data ©2021 Tele Atlas, Imagery ©2021 TerraMetrics


North:
Ms = 1.0
North West:
Ms = 1.0
North East:
Ms = 1.0
West:
Ms = 1.0
East:
Ms = 1.0
South West:
Ms = 1.0
South East:
Ms = 1.0
South:
Ms = 1.0
©2021 Google - Map data ©2021 Tele Atlas, Imagery ©2021 TerraMetrics

EQ Loads:

ULS - DUCT 1.25

T	0.40	*Time peroid [seconds]*
Soil Class	D	*Soil Class*
$C_h(T)$	3.00	*Accerleration spectra*
Z	0.30	*Hazard Factor*
R	1.00	*Return peroid factor*
N(T,D)	1.00	*Near fault factor*
μ	1.25	*Ductility*
S_p	0.93	*Structural performance*
k_μ	1.14	*Manual input for class E soils required*
$C_d(T_1)$	0.73	*Horizontal design co-efficient*

ULS - DUCT 3.5

T	0.40
Soil Class	D
$C_h(T)$	3.00
Z	0.30
R	1.00
N(T,D)	1.00
μ	3.50
S_p	0.70
k_μ	2.43
$C_d(T_1)$	0.26

ULS - DUCT 1.0

T	0.40	*Time peroid [seconds]*
Soil Class	D	*Soil Class*
$C_h(T)$	3.00	*Accerleration spectra*
Z	0.30	*Hazard Factor*
R	1.00	*Return peroid factor*
N(T,D)	1.00	*Near fault factor*
μ	1.00	*Ductility*
S_p	1.00	*Structural performance*
k_μ	1.00	*Manual input for class E soils required*
$C_d(T_1)$	0.90	*Horizontal design co-efficient*

SLS - DUCT 1

T	0.40	*Time peroid [seconds]*
Soil Class	D	*Soil Class*
$C_h(T)$	3.00	*Accerleration spectra*
Z	0.30	*Hazard Factor*
R	0.25	*Return peroid factor*
N(T,D)	1.00	*Near fault factor*
μ	1.00	*Ductility*
S_p	0.70	*Structural performance*
k_μ	1.00	*Manual input for class E soils required*
$C_d(T_1)$	0.16	*Horizontal design co-efficient*

4 - DEPOT STRUCTURE SEISMIC ASSESMENT

4.1 - Structural elements capacity

As per table C9.1 Material Strengths from DEE guidelines Section C9 - radiata pine No1 Framing timber assumed

Species	Grade	Bending	Compression parallel	Tension parallel	Shear in beams	Compression perpendicular	Modulus of elasticity (GPa)
1. Moisture condition – Dry (m/c = 16% or less)							
Radiata pine	No. 1 framing	17.7	20.9	10.6+	3.8	8.9	8.0

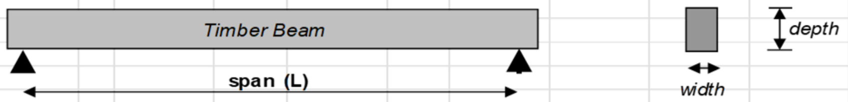
ϕ = 1

Purlins	ϕMn =	2.17 kNm	Strong axis	
	ϕMn =	1.11 kNm	Weak axis	
	ϕNnc =	7.6 kN	Compression	
Top Chord	ϕMn =	1.11 kNm	Weak axis	
	ϕNnc =	120 kN	Compression	
Bottom Chord	ϕMn =	1.11 kNm	Out of plane	
	ϕNnc =	9 kN	Compression	
	ϕNnt =	44 kN	Tension	k1 x k4 x ft x A x 1
Bottom Chord stiffener	ϕMn =	3.96 kNm	Out of plane	
	ϕNnc =	18.9 kN	Compression	
Diagonals	ϕNnc =	10.9 kN		
Timber Pole	ϕMn =	42.1 kNm	New pole	
	ϕMn =	25.3 kNm	Existing pole - poor conditions assume 60%	
	ϕNnc =	928 kN	New pole	
	ϕNnc =	556.8 kN	Existing pole - poor conditions assume 60%	
Foundation	ϕM =	30.0 kNm		

Purlins:

TIMBER BEAM DESIGN

NZS 3603:1993



Beam Size =	50 (width) x 150 (depth) mm		Z =	187500 mm^3
Span of Beam (L) =	4000 mm			
Effective Span (L_{ay}) =	4000 mm	Loaded Form	UDL	
Bending Moment (M*) =	kNm *(ULS)*	0 kNm *(SLS)*		
Shear Force (V*) =	kN *(ULS)*	0 kN *(SLS)*		

Load Duration	Brief (wind and Earthquake Loads)	k_1 =	1	*Table 2.4*
Timber Grade	No1 Framing	Bending strength f_b =	17.7 MPa	
		Compressive Strength f_c =	20.9 MPa	
	dry	Tension Strength f_t =	8.8 MPa	
		Stress in Shear f_s =	3.8 MPa	
		Modulus of Elasticity E =	8000 MPa	
		Lower Bound Mod of E E_{lb} =	6000 MPa	
		Φ =	1	
Number of Parallel Supports	1	k_4 =	1	*Table 2.7*
Assume no Grid system		k_5 =	1	*Cl 2.9.2*
max of	$S = 1.35 \times \frac{(L_{ay} \times}{b}((d/b)^2 - 1)^{0.5})^{0.5}$ = 20.3	k_8 =	0.65	*Cl 3.2.5.2* *Table 2.8*
	or $3 \times d/b$ = 9.00			

BENDING CAPACITY

$M_n = k_1 k_4 k_5 k_8 f_b Z$ = 2.17 kNm *Cl 3.2.4*

ΦM_n = 2.17 kNm **O.K.**

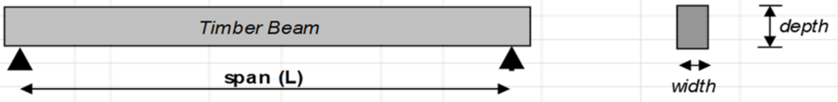
SHEAR CAPACITY

$V_n = k_1 k_4 k_5 f_s A_s$ = 19.00 kN A_s = (2bd)/3 = 5000 mm^2 *Cl 3.2.3.1*

ΦV_n = 19.00 kN **O.K.**

TIMBER BEAM DESIGN

NZS 3603:1993



Beam Size =	150	x	50	mm	**Z =** 62500	mm^3
	(width)		*(depth)*			
Span of Beam (L) =	4000	mm				
Effective Span (L_{ay}) =	4000	mm	**Loaded Form**	UDL		
Bending Moment (M*) =		kNm *(ULS)*	0	kNm *(SLS)*		
Shear Force (V*) =		kN *(ULS)*	0	kN *(SLS)*		

Load Duration — Brief (wind and Earthquake Loads) — k_1 = 1 — *Table 2.4*

Timber Grade — No1 Framing — dry

Bending strength f_b =	17.7	MPa
Compressive Strength f_c =	20.9	MPa
Tension Strength f_t =	8.8	MPa
Stress in Shear f_s =	3.8	MPa
Modulus of Elasticity E =	8000	MPa
Lower Bound Mod of E E_{lb} =	6000	MPa
Φ =	1	

Number of Parallel Supports — 1 — k_4 = 1 — *Table 2.7*

Assume no Grid system — k_5 = 1 — *Cl 2.9.2*

max of $S = 1.35 \times \frac{(L_{ay} \times}{b} ((d/b)^2 - 1)^{0.5})^{0.5}$ = 0.0 — k_8 = 1.00 — *Cl 3.2.5.2*, *Table 2.8*

or $3 \times d/b$ = 1.00

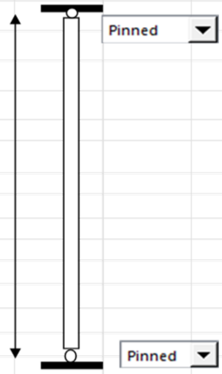
BENDING CAPACITY

$M_n = k_1 k_4 k_5 k_8 f_b Z$ = 1.11 kNm — *Cl 3.2.4*

ΦM_n = 1.11 kNm — **O.K.**

NZS 3603:1993

COMPRESSIVE CAPACITY (Buckling about XX axis)



Axial Load (N^*_c) = [ ] kN *(ULS)*

Bending Moment (M^*) = 0 kNm *(ULS)*

$N^*_c < \Phi N_{ncx}$ — *L_{ax} = 4000* — *Fig 3.5*

$N_{ncx} = k_1 k_8 f_c A$ — *Cl 3.3.4* — k_{10} = 1

$S_2 = k_{10}L/d$ = 80.0 *or smaller of* L_{ax}/d = 80.00 — *Cl 3.3.3.2*

S_2 = **80.0**

k_8 = 0.05 — *Based upon S_2*

k_1 = 1

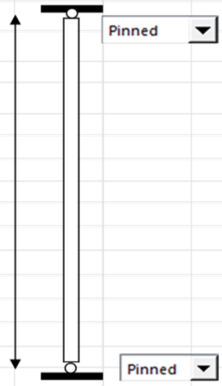
f_c = 20.9 MPa

A = 7500 mm^2

Beam Size = 150 *(width)* x 50 *(depth)* mm

ΦN_{ncx} = 7.6 kN

COMPRESSIVE CAPACITY (Buckling about YY axis)



Axial Load (N^*_c) = 0 kN *(ULS)*

Clear span (L_{ay}) = 4000 mm *(if = 0 then fully restained)*

$N^*_c < N_{ncy}$ — *Fig 3.5*

$N_{ncy} = k_1 k_8 f_c A$ — *Cl 3.3.4*

L_{ay} = 4000

$S_3 = k_{10}L_y/b$ = **26.7** *or smaller of* L_{ay}/b = 26.7 — k_{10} = 1 — *Cl 3.3.3.2*

S_3 = **26.7**

k_8 = 0.41 — *Based upon S_3*

k_1 = 1

f_c = 20.9 MPa

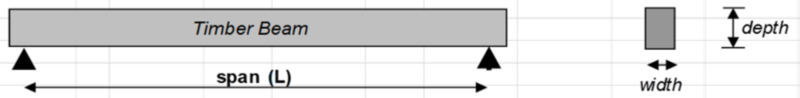
A = 7500 mm^2

ΦN_{ncy} = 63.8 kN

Top Chord:

TIMBER BEAM DESIGN

NZS 3603:1993



Beam Size =	150 (width)	x 50 (depth)	mm	**Z =** 62500 mm^3
Span of Beam (L) =	7000	mm		
Effective Span (L_{ay}) =	900	mm	**Loaded Form**	UDL
Bending Moment (M*) =		kNm *(ULS)*	0	kNm *(SLS)*
Shear Force (V*) =		kN *(ULS)*	0	kN *(SLS)*

Load Duration	Brief (wind and Earthquake Loads)		k_1 = 1	*Table 2.4*
Timber Grade	No1 Framing	Bending strength f_b =	17.7 MPa	
		Compressive Strength f_c =	20.9 MPa	
	dry	Tension Strength f_t =	8.8 MPa	
		Stress in Shear f_s =	3.8 MPa	
		Modulus of Elasticity E =	8000 MPa	
		Lower Bound Mod of E E_{lb} =	6000 MPa	
		Φ =	1	
Number of Parallel Supports	1		k_4 = 1	*Table 2.7*
Assume no Grid system			k_5 = 1	*Cl 2.9.2*

max of $S = 1.35 \times \frac{(L_{ay} \times}{b} ((d/b)^2 - 1)^{0.5})^{0.5} = 0.0$ k_8 = 1.00   *Cl 3.2.5.2*

or $3 \times d/b$ = 1.00   *Table 2.8*

BENDING CAPACITY

$M_n = k_1\, k_4\, k_5\, k_8\, f_b\, Z$ = 1.11 kNm   *Cl 3.2.4*

ΦM_n = 1.11 kNm   **O.K.**

SHEAR CAPACITY

$V_n = k_1\, k_4\, k_5\, f_s\, A_s$ = 19.00 kN $A_s = (2bd)/3$ = 5000 mm^2 *Cl 3.2.3.1*

ΦV_n = 19.00 kN   **O.K.**

NZS 3603:1993

COMPRESSIVE CAPACITY (Buckling about XX axis)

Axial Load (N^*_c) = kN *(ULS)*

Bending Moment (M^*) = 0 kNm *(ULS)*

$N^*_c < \Phi N_{ncx}$

L_{ax} = 2500

Fig 3.5

$N_{ncx} = k_1 k_8 f_c A$ *Cl 3.3.4*

k_{10} = 1

$S_2 = k_{10}L/d$ = 16.7 *or smaller of* L_{ax}/d = 16.67 *Cl 3.3.3.2*

S_2 = 16.7

k_8 = 0.83 *Based upon S_2*

k_1 = 1

f_c = 20.9 MPa

A = 7500 mm^2

Pinned

Pinned

Beam Size = 50 *(width)* x 150 *(depth)* mm

ΦN_{ncx} = 130 kN

COMPRESSIVE CAPACITY (Buckling about YY axis)

Axial Load (N^*_c) = 0 kN *(ULS)*

Clear span (L_{ay}) = 900 mm *(if = 0 then fully restained)*

$N^*_c < N_{ncy}$

Fig 3.5

$N_{ncy} = k_1 k_8 f_c A$ *Cl 3.3.4*

L_{ay} = 900

$S_3 = k_{10}L_y/b$ = 18.0 *or smaller of* L_{ay}/b = 18.0

k_{10} = 1 *Cl 3.3.3.2*

S_3 = 18.0

k_8 = 0.77 *Based upon S_3*

k_1 = 1

f_c = 20.9 MPa

A = 7500 mm^2

Pinned

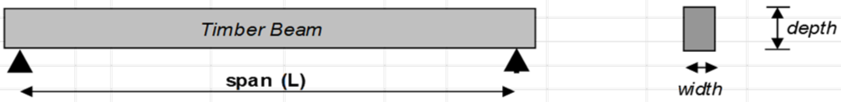
Pinned

ΦN_{ncy} = 120 kN

Bottom Chord:

TIMBER BEAM DESIGN

NZS 3603:1993



Beam Size =	50 (width)	x	150 (depth)	mm	$Z =$ 187500	mm^3
Span of Beam (L) =	11000	mm				
Effective Span (L_{ay}) =	2500	mm	Loaded Form	UDL		
Bending Moment (M*) =		kNm *(ULS)*	0	kNm *(SLS)*		
Shear Force (V*) =		kN *(ULS)*	0	kN *(SLS)*		

Load Duration	Brief (wind and Earthquake Loads)	$k_1 = 1$		*Table 2.4*
Timber Grade	No1 Framing	Bending strength f_b =	17.7 MPa	
		Compressive Strength f_c =	20.9 MPa	
	dry	Tension Strength f_t =	8.8 MPa	
		Stress in Shear f_s =	3.8 MPa	
		Modulus of Elasticity E =	8000 MPa	
		Lower Bound Mod of E E_{lb} =	6000 MPa	
		Φ =	1	
Number of Parallel Supports	1	$k_4 = 1$		*Table 2.7*
Assume no Grid system		$k_5 = 1$		*Cl 2.9.2*

max of $S = 1.35 \times \frac{(L_{ay} \times ((d/b)^2 - 1)^{0.5})^{0.5}}{b}$ = 16.1 $k_8 = 0.86$ *Cl 3.2.5.2* *Table 2.8*

or $3 \times d/b$ = 9.00

BENDING CAPACITY

$M_n = k_1 k_4 k_5 k_8 f_b Z =$ 2.85 kNm *Cl 3.2.4*

ΦM_n = 2.85 kNm **O.K.**

SHEAR CAPACITY

$V_n = k_1 k_4 k_5 f_s A_s =$ 19.00 kN $A_s = (2bd)/3 =$ 5000 mm^2 *Cl 3.2.3.1*

ΦV_n = 19.00 kN **O.K.**

NZS 3603:1993

COMPRESSIVE CAPACITY (Buckling about XX axis)

Pinned

Axial Load (N^*_c) = kN *(ULS)*

Bending Moment (M^*) = 0 kNm *(ULS)*

$N^*_c < \Phi N_{ncx}$ L_{ax} = *11000* *Fig 3.5*

$N_{ncx} = k_1k_8f_cA$ *Cl 3.3.4* k_{10} = 1

$S_2 = k_{10}L/d$ = 73.3 *or smaller of* L_{ax}/d = 73.33 *Cl 3.3.3.2*

S_2 = 73.3

k_8 = 0.06 *Based upon S_2*

k_1 = 1 Pinned

f_c = 20.9 MPa

A = 7500 mm^2

Beam Size = 50 *(width)* x 150 *(depth)* mm

ΦN_{ncx} = **9 kN**

COMPRESSIVE CAPACITY (Buckling about YY axis)

Axial Load (N^*_c) = 0 kN *(ULS)* Pinned

Clear span (L_{ay}) = 900 mm *(if = 0 then fully restained)*

$N^*_c < N_{ncy}$ *Fig 3.5*

$N_{ncy} = k_1k_8f_cA$ *Cl 3.3.4*

L_{ay} = *900*

$S_3 = k_{10}L_y/b$ = 18.0 *or smaller of* L_{ay}/b = 18.0 k_{10} = 1 *Cl 3.3.3.2*

S_3 = 18.0

k_8 = 0.77 *Based upon S_3*

k_1 = 1

f_c = 20.9 MPa

A = 7500 mm^2

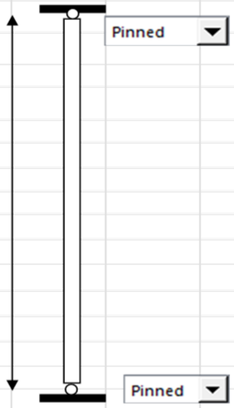
Pinned

ΦN_{ncy} = **120 kN**

TRUSS DIAGONALS:

COMPRESSIVE CAPACITY (Buckling about YY axis)

Axial Load (N^*_c) =	0	**kN** *(ULS)*		
Clear span (L_{ay}) =	2700	**mm**	*(if = 0 then fully restained)*	
$N^*_c < N_{ncy}$				*Fig 3.5*
$N_{ncy} = k_1 k_8 f_c A$			*Cl 3.3.4*	
			L_{ay} = 2700	
$S_3 = k_{10}L_y/b$ =	**54.0**	*or smaller of*	**L_{ay}/b = 54.0**	**k_{10} =** 1 *Cl 3.3.3.2*
S_3 =	**54.0**			
k_8 =	0.10	*Based upon S_3*		
k_1 =	1			
f_c =	20.9	**MPa**		
A =	5000	**mm^2**		



ΦN_{ncy} = 10.9 kN

NORMAL DENSITY POLE:

Bending capacity =

Diam=	**250**	mm
Φ	**1.00**	
k1	**1.00**	
k4	**1.00**	
k20	**0.85**	
k21	**0.85**	
k22	**1.00**	
fb	**38.00**	MPa
Z	1533979	mm3
ΦMb	42.1	kNm

For EQ case

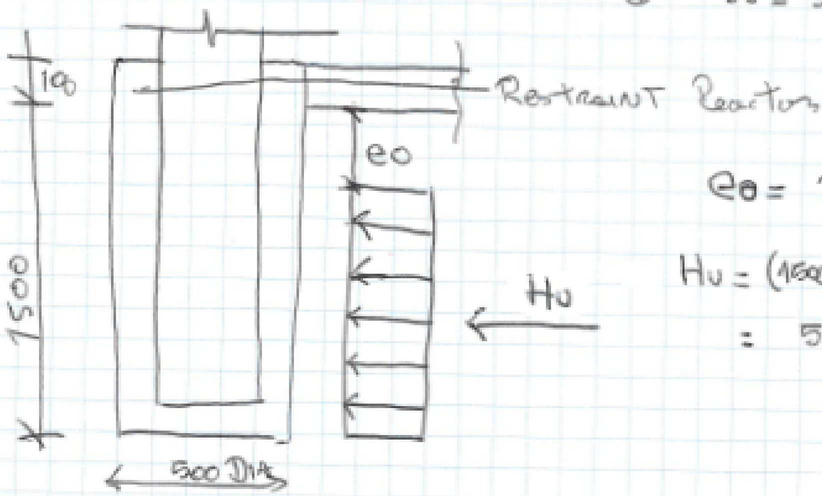
Compression capacity =

Diam=	**250**	mm
Φ	**1.00**	
k1	**1.00**	
k4	**1.00**	
k20	**1.00**	
k21	**0.90**	
k22	**1.00**	
fc	**21.00**	MPa
A	49087	mm2
ΦNc	927.8	kN

For EQ case

Check foundation capacity

① $S_u = 50\ kPa$

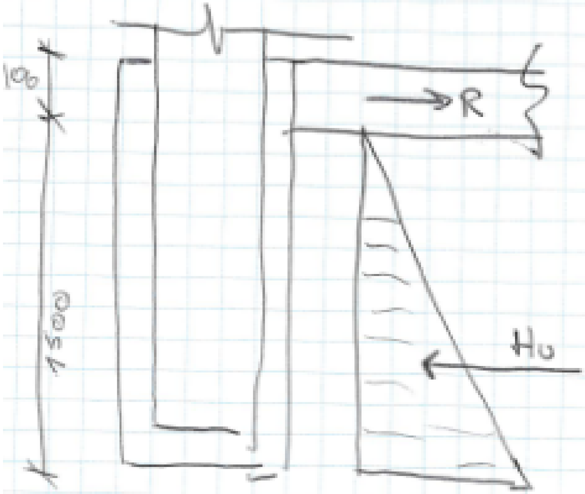


$e_o = 375\ mm$

$H_u = (1500 - 375) \times 2 \times 50 \times 500mm$

$= 56\ kN$

$\phi M = 56kN \times 0.99m = 55\ kNm$



$\phi = 28°$ $\gamma = 18\ kN/m^3$

$K_p = 2.8$

$H_u = 2.8 \times 18kN/m^3 \times 0.5m \times \frac{1.5m^2}{2}$

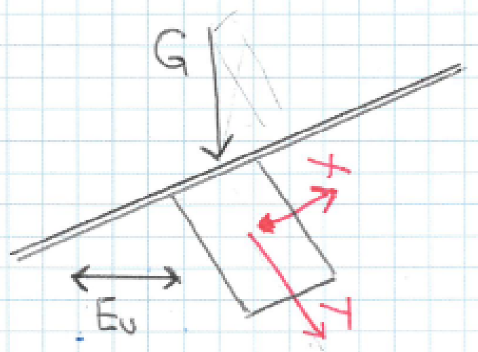
$= 28\ kN$

$\phi M = 28\ kN \times 1.05 = 30\ kNm$

4.2 - Transversal capacity

Eq from purlins to top chord

PURLINS



$G = 0.25\ kPa \times 0.9m = 0.225\ kN/m$

$E_U = 0.225 \times 0.73 = 0.16\ kN/m$

L / C => ① G + Eu → X = 0.225 x Sen(20) + 0.16 x cos(20) = 0.07
Y = 0.225 x Cos(20) + 0.16 x Sen(20) = 0.27

② G − Eu → X = 0.225 x Sen(20) + 0.16 x cos 20 = 0.23
Y = 0.225 x Cos(20) − 0.16 . SIN(20) = 0.16

① → $M^*_x = 0.27\ kN/m \times 4^2_m/8 = 0.54\ kN\,m$

$M^*_y = 0.07 \times 4^2/8 = 0.14\ kN\,m$

$$\frac{0.54}{1.23} + \frac{0.14}{0.63} = 0.66 < 1$$

② → $M^*_x = 0.23 \times 4^2/8 = 0.46\ kN/m$

$M^*_y = 0.16 \times 4^2/8 = 0.32\ kN/m$

$$\frac{0.46}{1.23} + \frac{0.32}{0.63} = 0.88 < 1$$

$\boxed{100\%\ NBS}$

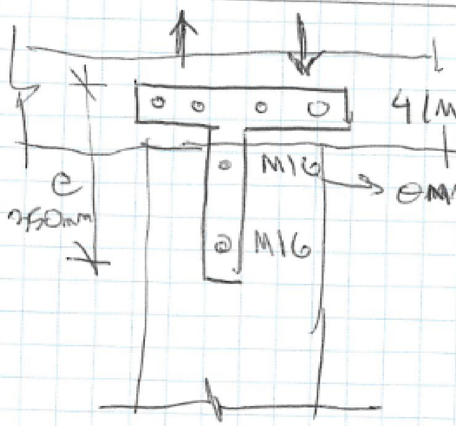
$V^*_x = 0.23 \times \frac{4}{2} = 0.46\ kN$ → 2 / WIRE DOGS FIXING

$\boxed{OK}$

FIXING PLATE:

27

Check Fixing Plate



4/M6 → By inspection 100% NBS

→ OMITTED – NO EDGE DISTANCE

$V^* = 6.9$ kN (MODEL)

to duct 1

BOLT CAPACITY = 13 kN

OK – 100% NBS

Check Bending capacity of plate

M^* = 8.5kN x 0.35m /2 (two plates) = 1.5 kN

Plate capacity = $\frac{80mm^2 \times 10}{6} \times 250$ MPa = 2.7 kNm

OK – 100% NBS

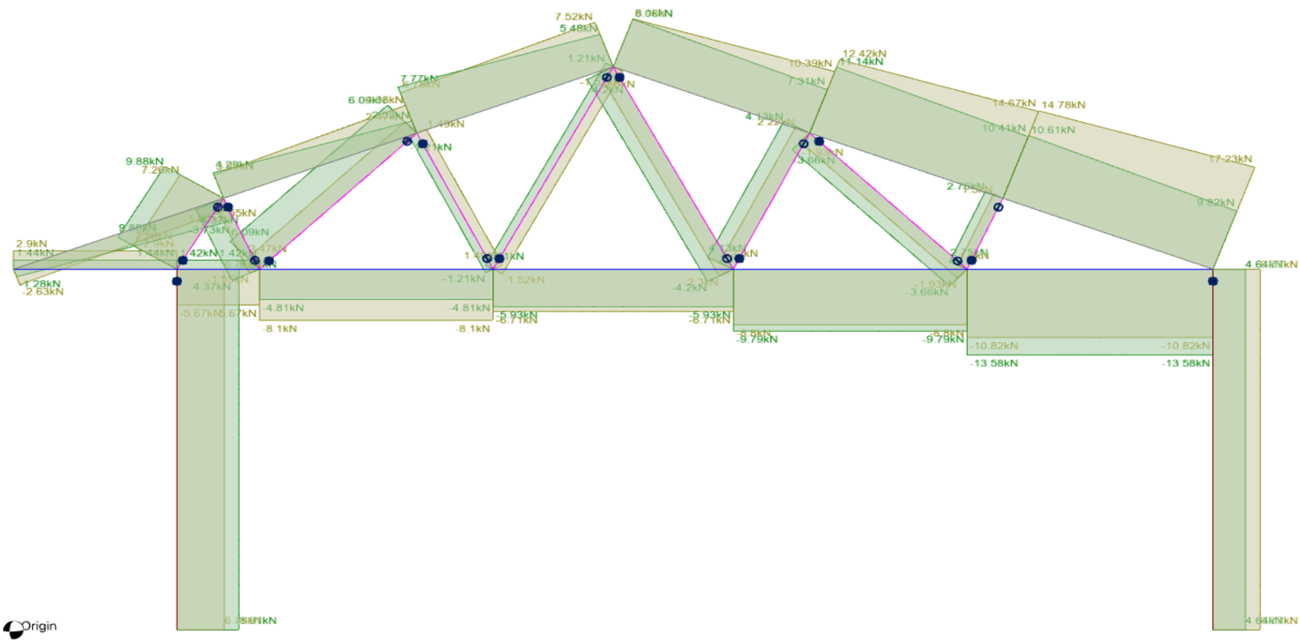
<u>SPACE GASS MODEL FOR LOAD CASES G+Eu AND G-Eu:</u>

Groof = 0.25kPa x 4m = 1 kPa

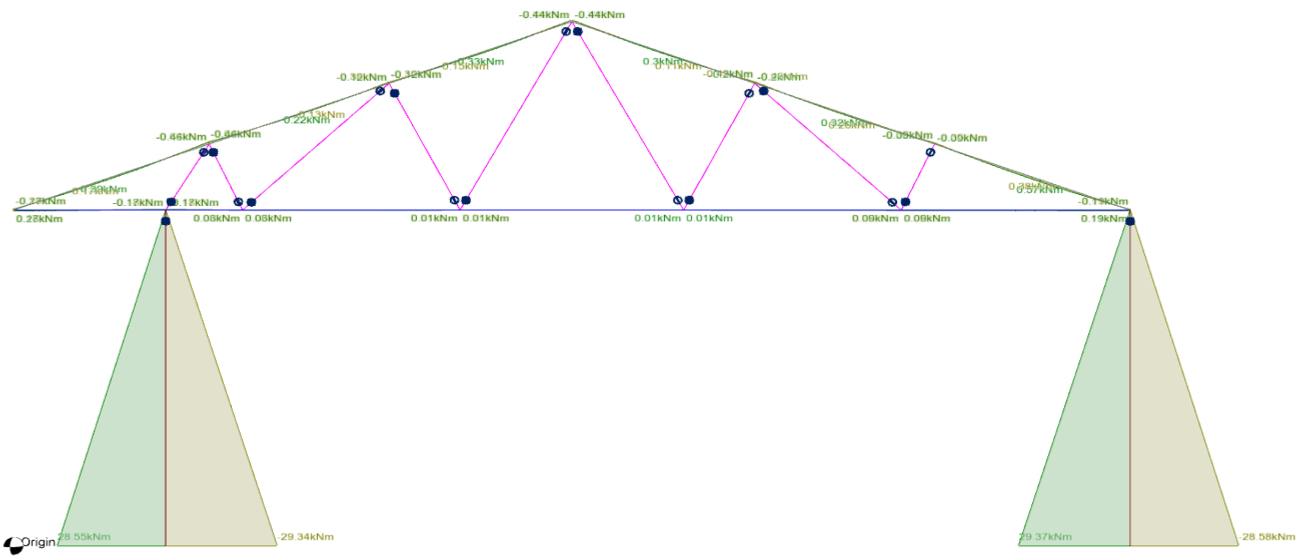
Eu = 1 kPa x 0.73 = 0.73 kPa UDL at roof

Eu = 0.3 kPa x 0.73 x 4m x 2.2m = 1.92 kN Wall OOP point loads (both sides)

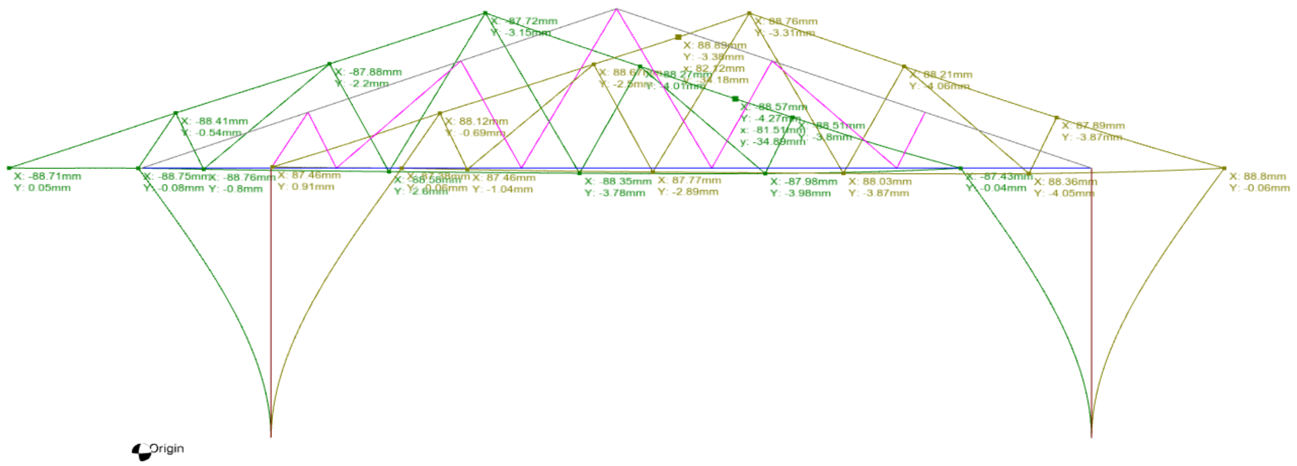
Axial forces



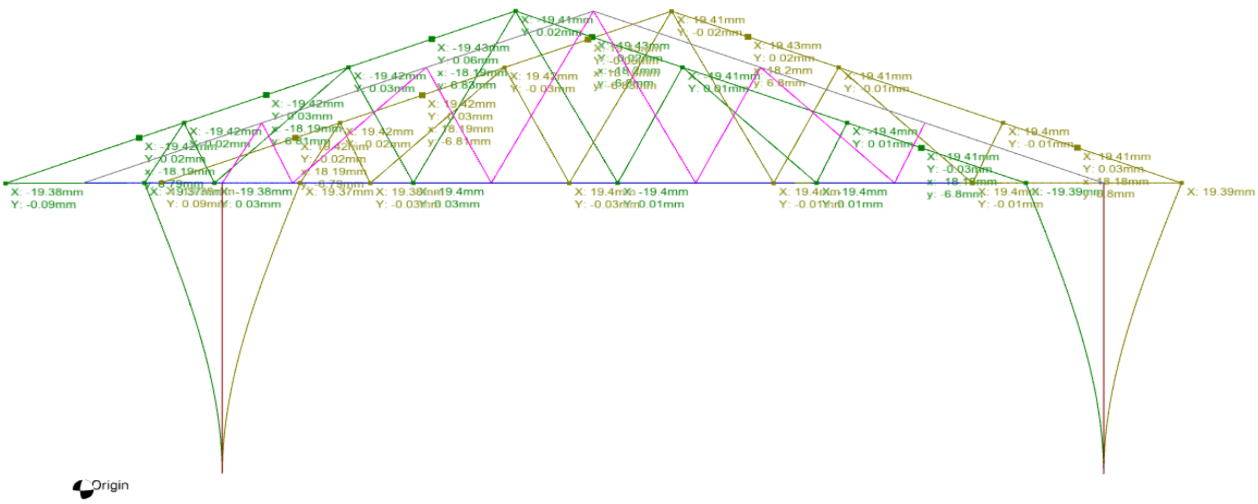
Bending moment



ULS Deflection



SLS Deflection



CHECK TRANSVERSAL SEISMIC %NBS:

Purlins = 100% NBS

Top Chord = 17.2kN / 21.6 kN = 0.8 => 100% NBS

Bottom Chord = 13.6kN / 44 kN = 0.32 => 100% NBS

Diagonal = 9.9kN / 10.9 kN = 0.91 => 100% NBS

Post = 29.4kNm / 25.3 kNm + 6.7 kN / 557 kN = 1.17 => 85% NBS

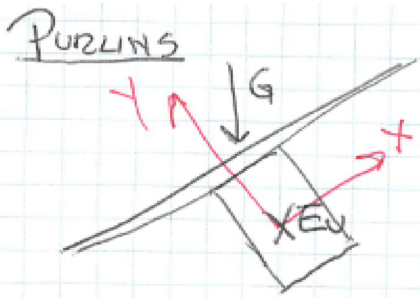
Foundation = 29.4kNm / 30 kNm + 6.7 kN / 801 kN = 0.98 => 100% NBS

ULS DEFLECTION = 1.25 (m) x 1.2 (kdm) x 88.8 mm = 133mm
2.5% H = 100mm => 75% NBS

SLS DEFLECTION = 4000mm / 19mm = L/210 < L/300 ==> 70% NBS

4.3 - Longitudinal capacity

PURLINS



$G = 0.225\ kN/m$

$E_u = 0.225 \times 0.73 \times 4m = 0.7\ kN$

$M_y^* = 0.225 \times \sin(20) \times \frac{4^2}{8} = 0.15\ kNm$

$M_x^* = 0.225 \times \cos(20) \times \frac{4^2}{8} = 0.42\ kNm$

$\frac{0.42}{2.17} + \frac{0.15}{1.11} + \frac{0.7}{7.6} = 0.42 \rightarrow$ 100% NBS

TOP CHORD

THERE IS SOME CROSS - BRACING AT ROOF LEVEL BUT IT IS INCOMPLETE - LONG ASSUMED 14m

$M^*_{OOP} = 4m \times 0.225\ kN/m \times 0.73 \times \frac{14^2}{8} = 17.9\ kNm$

N^* (from MODEL) for G = 13.5 kN

$\frac{17.9}{1.1} + \frac{13.5}{21.6} = 16.9 \Rightarrow$ $\boxed{6\%\ NBS}$

IF ASSUME 7m LONG $\rightarrow M^* = 4.5\ kNm$

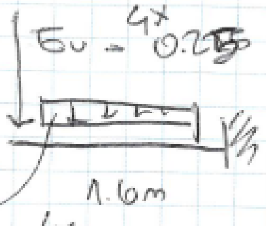
$\Rightarrow$ $\boxed{21\%\ NBS}$

Bottom chord

Longitudinal cross-bracing (beam) only to a couple of trusses → Bottom chord will transfer loads as cantilever OOP

$E_u = 4 \times 0.25 \times 0.73 \times \frac{14}{2} = 5.1\ kN$

$V^* = 6.3\ kN$



1.6m

→ $M^* = 9.1\ kNm$ OOP

UDL = $4 \times 0.25 \times 0.73 = 0.72\ kN/m$
ceiling

N^* (Model) = 2.2 kN

$\frac{9.1\ kNm}{1.11\ kNm} + \frac{2.2}{189} = 8.3$ → $\boxed{12\%\ NBS}$

Cantilever post & foundation

$M^* = (6.3\ kN) \times 4.2\ m = 27\ kNm$

Pole → 100 % NBS

Foundation → 100 % NBS

DEFLECTION (CANTILEVER PL)

Cantilever Length = 4 m
PL = 6.3 KN ULS
E = 8700 MPa I = 1.9E+08 mm4
deflection = 80.6 mm

DEFLECTION (CANTILEVER PL)

Cantilever Length = 4 m
PL = 1.386 KN SLS
E = 8700 MPa I = 1.9E+08 mm4
deflection = 17.7 mm

CHECK LONGIT SEISMIC %NBS:

Purlins = 100% NBS

Top Chord = 6% NBS

Bottom Chord = 12% NBS

Diagonal = 9.9kN / 10.9 kN = 0.91 => 100% NBS

Post = 73% NBS

Foundation = 87% NBS

ULS DEFLECTION = 1.25 (m) x 1.2 (kdm) x 81 mm = 122mm
2.5% H = 100mm => 82% NBS

SLS DEFLECTION = 4000mm / 18mm = L/222 < L/300 ==> 74% NBS

5 - TIMBER FRAMED OFFICE SEISMIC ASSESMENT
5.1 - BRACE DEMAND

SEISMIC WEIGHT - MEZZANINE LEVEL:

G =

MEZZANINE	265m² x 0.6Kpa =	159.0 KN
WALLS	1.2m x (160m x 0.4Kpa) =	76.8 KN
	TOTAL =	235.8 KN

Q =

MEZZANINE	260m² x 1.5Kpa + 5 x 2kPa =	400.0 KN

ψE = 0.30

$W_i = G + \psi Q$ = 355.8 KN

Cd(T) ULS = 0.26

ULS → V = Wt x Cd(T) = 92.5 KN

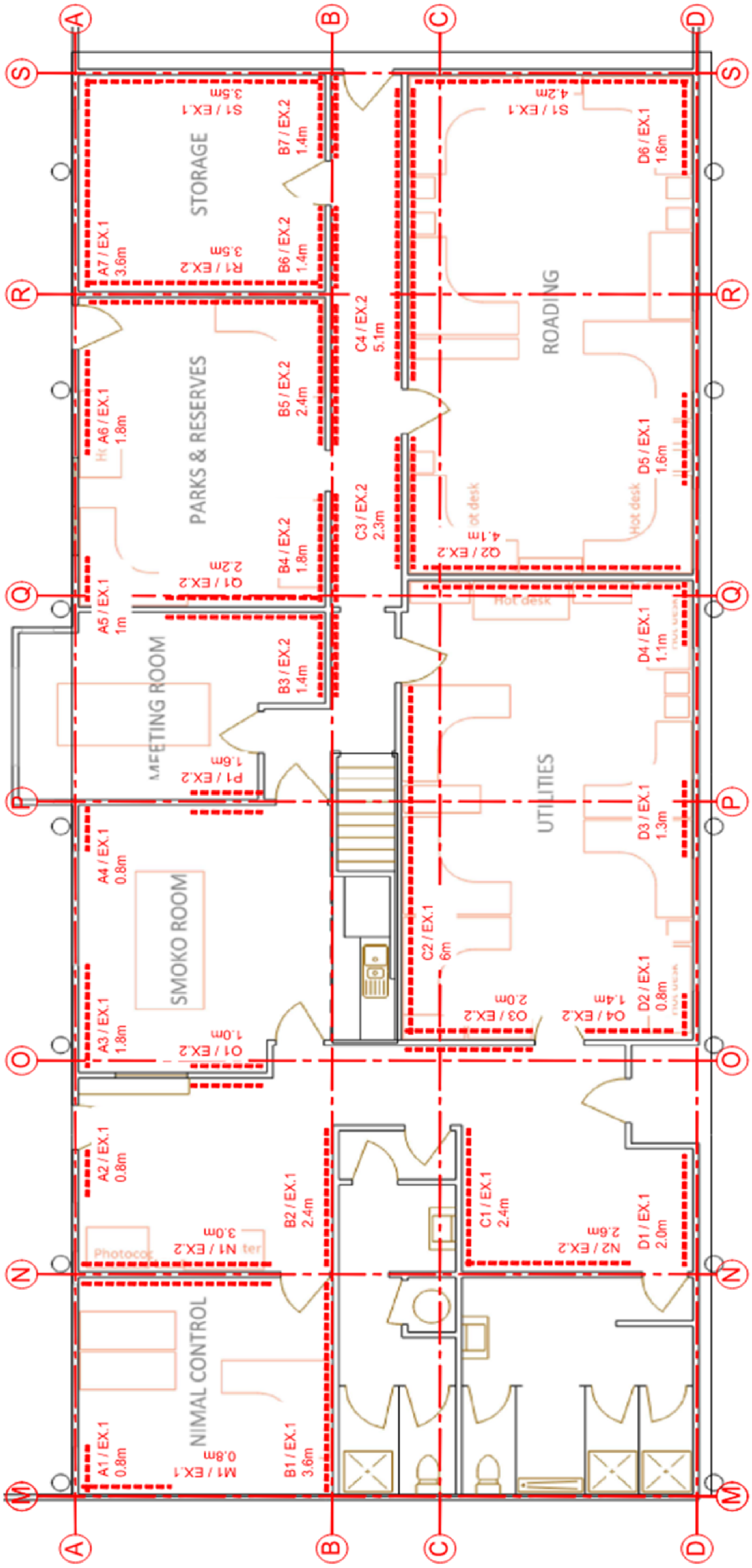
EQ demand = 92.5KN x 20BUS/KN = **1850 BUS**

5.2 - BRACE CAPACITY

Min Brace capacity = 80% NBS

Wall Ex.1 = exiting wall - plasterboard 1 side

Wall Ex.2 = extisting wall - plasterboard both side



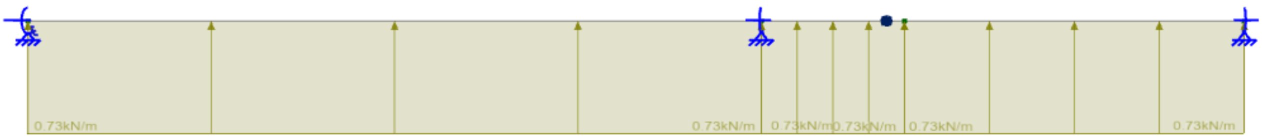
Typical element height	2.4										
Override data			Element		Bracing element data				EQ bracing capacity (BU)		
Height	Wind capacity	EQ capacity	ID		Element type	Element length (m)	Angle to braceline (°)	Element height (m)	Capacity per metre	Capacity along	Capacity across
			A	1	Ex.1	0.80		2.4	0	-	-
			A	2	Ex.1	0.80		2.4	0	-	-
			A	3	Ex.1	1.80		2.4	50	90	-
			A	4	Ex.1	0.80		2.4	0	-	-
			A	5	Ex.1	1.00		2.4	50	50	-
			A	6	Ex.1	1.80		2.4	50	90	-
			A	7	Ex.1	3.60		2.4	50	180	-
			B	1	Ex.1	3.60		2.4	50	180	-
			B	2	Ex.1	2.40		2.4	50	120	-
			B	3	Ex.2	1.40		2.4	60	84	-
			B	4	Ex.2	1.80		2.4	60	108	-
			B	5	Ex.2	2.40		2.4	60	144	-
			B	6	Ex.2	1.40		2.4	60	84	-
			B	7	Ex.2	1.40		2.4	60	84	-
			C	1	Ex.1	2.40		2.4	50	120	-
			C	2	Ex.1	6.00		2.4	50	300	-
			C	3	Ex.2	2.30		2.4	60	138	-
			C	4	Ex.2	5.10		2.4	60	306	-
			D	1	Ex.1	2.00		2.4	50	100	-
			D	2	Ex.1	0.80		2.4	0	-	-
			D	3	Ex.1	1.30		2.4	50	65	-
			D	4	Ex.1	1.10		2.4	50	55	-
			D	5	Ex.1	1.60		2.4	50	80	-
			D	6	Ex.1	1.60		2.4	50	80	-
			M	1	Ex.1	0.80		2.4	0	-	-
			N	1	Ex.2	3.00		2.4	60	-	180
			N	2	Ex.2	2.60		2.4	60	-	156
			O	1	Ex.2	1.00		2.4	60	-	60
			O	2	Ex.2	2.00		2.4	60	-	120
			O	3	Ex.1	1.40		2.4	50	-	70
			P	1	Ex.2	1.60		2.4	60	-	96
			Q	1	Ex.2	2.20		2.4	60	-	132
			Q	2	Ex.2	2.20		4.1	60	-	77
			R	1	Ex.2	3.50		2.4	60	-	210
			S	1	Ex.1	3.50		2.4	50	-	175
			S	2	Ex.1	4.10		2.4	50	-	205

[*] = Bracing element has greater than 120BU/m.
Verify hold down connection if timber subfloor

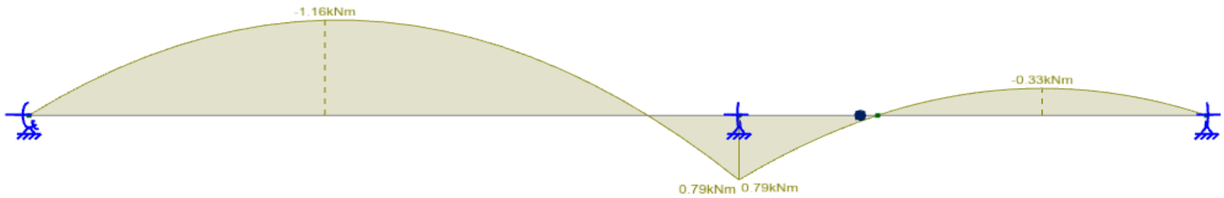
Earthquake	**along**	**across**
Achieved	**2458**	**1481**
Demand	**1850**	**1850**
	OK 133%	NG 80%

6 - CONCEPT STRENGTHENING TO 67% NBS

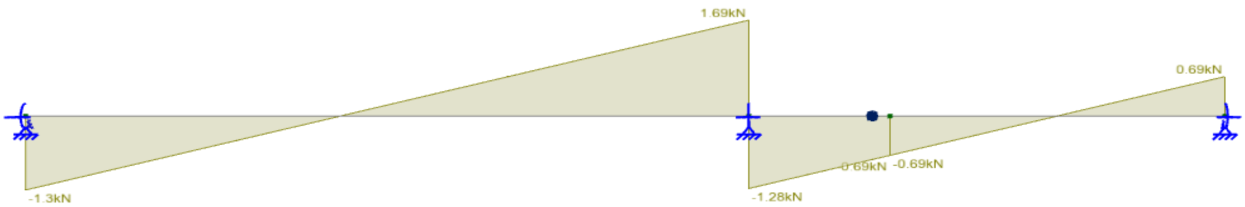
* CROSS BRACE THE ROOF AT TOP CHORD LEVEL TO REDUCE TOP CHORD SPAN OOP



[M]



[V]



M* = 1.16 kNm (out of plane)
N* = 12.7 kN (from transversal frame model)

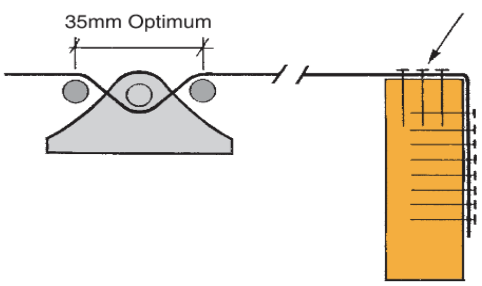
1.16 kNm / 1.1kNm + 12.7kN / 120 kN (compression capacity for 4.5m long) => 86% NBS

R* = 1.69 + 1.28 = 3 kN (REACTION)

USE MULTIBRACE TO CROSS BRACE THE TOP CHORD AND THE BOTTOM CHORD ON THE EAVES

Loadings

3 nails top edge, 8 nails vertical face (not in same line)



0.91mm x 53mm G300 Z275 GALVANISED STEEL
0.9mm x 53mm STAINLESS STEEL 304-2B

Tension	Multi-Brace Only	Multi-Brace With Tensioner*
Characteristic Load	14.8kN	14.8kN
Elongation 0.2mm/m/kN including nail slip		
End nail fixing - 11 x LUMBERLOK Product Nails 30mm x 3.15 dia. if Multi-Brace is folded over timber face. Otherwise use 15 Product Nails.		