41 Weld Street, Hokitika

Detailed Seismic Assessment Report



Contact Details

Name: Emma Wardle

23 High Street
PO Box 365, Greymouth 7840
New Zealand

Telephone: +64 3 769 9330
Mobile: +64 27 609 0287

Document Details:

Date: 26 November 2019
Reference: 6-WMAK1.01 / 005GG
Status: DRAFT

Prepared By



Emma Wardle
Graduate Engineer Structures

Reviewed By



Julian Maranan
Senior Engineer Structural

Approved for Release By



Jamie Lester
Technical Principal – Building Structures

Contents

Executive Summary 5

1 General 1
- 1.1 Scope 1
- 1.2 Compliance 1
- 1.3 Means of Compliance 2
- 1.4 Limitation of the Report 3

2 Background Information 3
- 2.1 Source of Geotechnical Data 3
- 2.2 Source of Building Data 3

3 The Site 4
- 3.1 Topography 4
- 3.2 Regional Geology 4
- 3.3 Building Foundations 4
- 3.4 Seismicity 5
- 3.5 Ground Conditions 6

4 The Structure 11
- 4.1 Building Description 11
- 4.2 Primary Gravity Load Resisting System 11
- 4.3 Primary Lateral Load Resisting System 12
- 4.4 Site Inspection & Building Condition 12
- 4.5 Alterations 12
- 4.6 Assumptions 12

5 Detailed Seismic Assessment 13
- 5.1 Analysis Methodology 13
- 5.2 Gravity Loads 15
- 5.3 Earthquake Loads 15

6 Results of the Detailed Seismic Assessment 17
- 6.1 First Floor 17
- 6.2 Ground Floor 17
- 6.3 Summary of Assessed Seismic Capacity 19
- 6.4 Critical Structural Weakness 19

7 Conclusions and Recommendations 20

List of Figures

Figure 3-1 Geological map of the Hokitika area, Institute of Geological & Nuclear Sciences (IGNS, 2002). 4
Figure 3-2 Indicative Peak Ground Acceleration for the Westland District. 5
Figure 3-3 Indicative Mercalli intensities for the Westland District. 6
Figure 3-4 Map of available borehole information near the site on 41 Weld Street, Hokitika. Borehole 88151 is located 11m from the southwestern corner of the building. Cone Penetrometer Testing (CPT) data is also available from 127-137 Revell Street (CPT 109538), which is at 338m distance from the Site. 7
Figure 3-5 Ground Shaking Zones map for the Hokitika area 9
Figure 3-6 Scala penetrometer from BH_88151 location. The red line indicates ultimate bearing capacity of 300kPa (5 blows per 100mm). 10
Figure 4-1: D1, D2 and D3 11
Figure 5-1: D1 14
Figure 5-2: D2 15
Figure 5-3: Structural Acceleration Spectrum (5% damped) 16

List of Tables

Table 1-1 : Summary of assessed seismic capacity (%NBS) 5
Table 3-1 Borehole summary (BH_88151): 7
Table 4-1 : Material Properties 13
Table 5-1 : Material Densities 15
Table 5-2 : Seismic Design Coefficient Parameters 16
Table 5-3: Design Response Coefficient Parameters 16
Table 6-1: Assessed Seismic Capacity (%NBS) 19

Document History and Status

Revision	Date	Author	Reviewed by	Approved by	Status
01	26 Nov 2019	E. Wardle	J. Maranan	J. Lester	FINAL

Revision Details

Revision	Details
01	41 Weld Street, Hokitika Detailed Seismic Assessment

Executive Summary

Background

This report presents the Detailed Seismic Assessment (DSA) of the Mountain Jade Building located at 41 Weld Street, Hokitika. The building has been assessed as Importance Level 2 (IL2). The results of the assessment are presented and discussed to provide information to Kati Mahaaki Ki Makaawhio Ltd. as a part of their pre-purchase due diligence for the building.

Building Description

The property is a 2-storey complex constructed in 1993-94 (the south wing glass workshop was constructed at an earlier unknown date) and serves as a jade carving gallery, workshop, café, beauty spa, and backpackers accommodation. The building has an L-shaped plan and is located less than 200m north of the Hokitika River and approximately 300m south-west of the beach front.

Assessment Results

The seismic demands were calculated using an equivalent static analysis (ESA) procedure in accordance with the *New Zealand Structural Design Actions Standard, Part 5 – Earthquake Actions* (NZS1170.5).

The seismic capacities have been assessed in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) document *The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments.*

The assessed seismic capacities of the primary structural elements, based on the respective failure mode, are presented in the table below.

Table 1-1 : Summary of assessed seismic capacity (%NBS)

Level	Area	Element	%NBS		Failure Mode/Remarks
			X-dir	Y-dir	
First Floor	D1/D2	Steel Frame	15%	-	Combined Axial & Flexure
		Timber Walls	-	40%	Shear resistance of timber-framed walls
		PC Walls	67%	-	Out-of-plane bending, exterior wall along grid 2/Q-I
	D3	Steel Frame	40%	-	Combined Axial & Flexure
		Timber Walls	-	100%	Shear resistance of timber-framed walls
		Timber Floor Diaphragm	38%	95%	Shear resistance of MDF Panel, Connections not checked as there is no information
Ground Floor	D1	PC Walls	65%	40%	Combined axial & flexure
		PC Spandrels	60%	60%	Combined axial & flexure
		Diaphragm	50%	60%	Seating Length
		Foundation	100%	100%	Bearing
	D2	PC Walls	70%	70%	Shear, Combined Axial & Flexure
		PC Walls	67%	-	Out-of-plane bending, exterior wall along Grid 2/Q-I

			-	35%	Out-of-plane bending, exterior wall along 5/Q (PC31 & PC32)
			-	30%	Out-of-plane bending, exterior wall along 5/I (PC17 & PC16)
		PC Spandrels	80%	80%	Combined Axial & Flexure
		Diaphragm	25%	25%	Pull-out capacity M12 TCM inserts
		Foundation	100%	100%	Bearing
	D3	Steel Frame	40%	-	Combined Axial & Flexure
		Conc. Walls	-	38%	Shear resistance of original RC walls

The outcome of this assessment indicates that the existing Mountain Jade building has a seismic capacity of less than 34%NBS (IL2) in both directions. The assessed seismic capacity is governed by precast floor connection to the precast walls (25%NBS) and out-of-plane bending of precast wall along Grid 5/I (30%NBS).

The weakest lateral load resisting elements of this building are the steel portal frames. However, due to presence of an alternative load path (i.e. timber walls), these frames are unlikely to collapse under a significant earthquake and therefore not considered the limiting failure mechanism of this building.

The precast walls and precast floor are considered the critical structural weakness (CSW's) of this building with a limiting capacity of 25%NBS (IL2). Since the assessed capacity is less than 34%NBS it satisfies one of the requirements for an earthquake prone building (EPB).

In accordance with Table A3.1 of Assessment Guidelines, this building is considered very high risk with an Alpha Rating of E. The risk description can be considered to be relative life safety risks in the event of a large earthquake.

Recommendations

There is a legal requirement to strengthen the building to achieve a minimum 34%NBS although WSP recommends strengthening to at least 67%NBS.

The following is a conceptual strengthening solution to meet the minimum seismic capacity of 34%NBS:

First Floor

(1) Provide vertical braces (VB1) at locations marked on plan. Develop new roof bracing system on the roof level to create a clear load path for the inertial loads to transfer to the first floor via new vertical braces.
(2) Install fly braces to the existing roof beams at spacing required to develop the beams section capacity without flange buckling.

Ground Floor

(1) Provide out-of-plane lateral supports to the precast walls at grid lines 5/Q (PC31 and PC32) and 5/I (PC16 and PC17). This will include vertical and horizontal steel support to shorten the span PC walls in both directions.
(2) Install additional floor to wall anchors along grid line 1. Anchor bolts can be drilled and epoxied to the wall at 900mm spacing and place alternately with the existing M12 TCM inserts.

The scope of strengthening to 67%NSB includes the following:

(1) All items under 34%NBS strengthening

(2) Install additional vertical braces (VB2) at the first floor.
(3) Thicken the existing precast walls (pier and spandrels) along grid lines 10 and G.

The strengthening concepts are presented in Appendix C.

1 General

1.1 Scope

WSP has been commissioned by Kati Mahaaki Ki Makaawhio Ltd. (Makaawhio) to carry out a Detailed Seismic Assessment (DSA) of the existing Mountain Jade Building located at 41 Weld Street, Hokitika. The building is considered to have a Medium Consequence for loss of human life and therefor has been assessed as an Importance Level 2 (IL2) building in accordance with the New Zealand Building Code (NZBC).

Makaawhio has previously engaged WSP to complete an Initial Seismic Assessment (ISA) for the building in August 2019. This assessment and report provided a score of 40%NBS based on an Initial Evaluation Procedure (IEP). This assessment is above the threshold for Earthquake-Prone Buildings (EPBs) of 34%NBS, but below the threshold for Earthquake Risk Buildings of 67%NBS.

The purpose of the DSA is to determine the performance of the building and compare it with the requirements provided in the NZBC for a new building. The comparison is expressed as a percentage of the New Building Standard (%NBS).

The scope includes the following tasks:

- A desktop review of available building information;
- Walk through non-intrusive inspection of the building;
- Detailed structural assessment to determine the %NBS of the building

Assessment of the seismic capacity of the non-structural components of the building is not included in this DSA. Likewise, assessment of the geotechnical hazards such as liquefaction, lateral spreading, soil instability, and soil deformability are not part of the scope.

This report describes the methodology used in the assessment and presents the results of the DSA.

1.2 Compliance

This section contains a summary of the requirements of the various statutes and authorities that control activities in relation to Earthquake-Prone Buildings within the country at present.

1.2.1 Building Act 2004

This section incorporates the New Building (Earthquake-Prone Buildings) Amendment Act 2016 that came into effect from 1 July 2017. Several sections of the Building Act are relevant when considering structural requirements. The following provides an overview of sections typically relevant to the assessment of existing buildings.

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 the alteration. This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 133AB – Definition of Earthquake-Prone Buildings

This section defines a building or part of 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. The respective territorial authority determines whether a building or part of a building is earthquake-prone.

Section 133AE – Definition of Priority Building

©WSP | 26 November 2019

This section provides the definitions of priority buildings which generally includes buildings that are needed during emergency response; occupied for educational purposes as well as any Unreinforced Masonry buildings on busy thoroughfares with falling hazards. It also applies to buildings on strategic routes.

Section 133AF – Role of the Territorial Authority in identifying priority buildings

The section requires the Territorial Authority within the medium and high seismic risk zones to identify priority buildings in their districts.

Section 133AG – Territorial Authority must identify potentially Earthquake-Prone Buildings

This section requires the Territorial Authorities to identify buildings or parts of buildings that are potentially earthquake-prone within the set timeframe for each seismic risk area starting from 1 July 2017, which is the commencement date on which this section of the Act comes into force.

Section 133AH – Territorial Authority must request engineering assessment of potentially earthquake prone buildings

For buildings that are identified as potentially earthquake-prone, the Territorial Authority (TA) must ask the building owner to provide an engineering assessment.

Section 133AI – Obligations of Owners on receiving a request for engineering assessment

Upon notification by the Territorial Authority, the building owner must obtain an engineering assessment of the building and submit to the TA within 12 months.

Section 133AK – Territorial Authority must determine whether the building is earthquake-prone

Based on the engineering assessment, the TA must determine whether the building or a part of the building is earthquake-prone. If it is earthquake prone, the TA must issue an EPB notice and publish the building information on the EPB register.

Section 133AM – Deadline for completing seismic work

This section defines the timeframe that the owner of the building subject to EPB notice has to complete seismic strengthening work. The timeframe depends on the seismic risk zone and whether the building is considered a priority building or not and can range between 7.5 and 35 years.

1.2.2 The Seismic Assessment of Existing Buildings

The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments July 2017 (the Guidelines) provides a technical basis for engineers to carry out seismic assessments of existing buildings within New Zealand. The Guidelines support seismic assessments for a range of purposes. It forms local authority's decision-making on whether or not a building is earthquake-prone in terms of the Building Act 2004 and for property risk identification generally.

The principal outcome of a seismic assessment using the Guidelines is a rating expressed as a percentage of the new building standard that applies to an equivalent new building on the same site.

The Guidelines supersede the previous assessment guidance published in 2006 by NZSEE (the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes).

1.3 Means of Compliance

The detailed seismic assessment of the existing building has been undertaken in compliance with the following standards and guidelines:

- AS/NZS1170 Structural Design Actions, Parts 0, 1 and 5
- NZS3101:2006 Concrete Structures Standard
- NZS3404:1997 Steel Structures Standard
- The Seismic Assessment of Existing Buildings:
    - Part A – Assessment Objectives and Principles
    - Part C – Detailed Seismic Assessment:
        - *C1 – General Issues*
        - *C2 – Assessment Procedures and Analysis Techniques*
        - *C3 – Earthquake Demands*
        - *C4 – Geotechnical Considerations*
        - *C5 – Concrete Buildings*
        - *C6 – Structural Steel Buildings*
        - *C9 – Timber Buildings*
        - *C10 – Secondary Structural and Non-Structural Elements*

1.4 Limitation of the Report

The structural assessment is based on the limited information provided by the building owner/representative and on a limited visual inspection of the building. No destructive tests or specific geotechnical investigations have been undertaken at this stage.

The analysis and assessment are based on the building in its undamaged state. However, there may be deficiencies in the building that could cause the capacity of the building to be reduced. In this case, the current capacity of the building may be lower than that stated in this report.

Despite the use of best national and international practice, the analysis outcomes are affected by a certain level of uncertainty due to some assumptions and simplifications made during the assessment process. These include:

- Simplifications introduced in the analysis and numerical modelling of the existing structure, including boundary conditions (e.g. foundation fixity, soil stiffness, etc.);
- Assessment of material strength based on the indication provided in the Seismic Assessment of Existing Buildings Guidelines;
- Approximation in the assessment of the post-yield behaviour of structural elements;
- Assessment is based on the undamaged condition of the building.

The assessment is limited to Section B1 – Structure of the New Zealand Building Code, and no assessment of the compliance of other sections of the NZBC has been undertaken. Also, the assessment considers earthquake loads only; other loads such as wind and snow are not covered.

2 Background Information

2.1 Source of Geotechnical Data

The geotechnical information used as a basis in the assessment was obtained through a desktop study only. Data was adopted from published reports and nearby ground investigation where available.

2.2 Source of Building Data

The Detailed Seismic Assessment (DSA) has been carried out based on the information available in the record drawings and information gathered during the site inspection.

©WSP | 26 November 2019

2.2.1 Original Documentation

- Architectural and Structural 'For Tender' drawings, 1993/94, Lewis & Barrow Ltd.
- Initial Seismic Assessment, 22 August 2019, WSP Opus.

2.2.2 Site Inspection

A visual, non-intrusive site inspection was undertaken by WSP on the 12th September 2019. The inspection included a walk-through observation of the interior and exterior of the building.

The site inspection confirmed the general layout of the building as illustrated in the structural drawings. This included random check measurements and reinforcing scanning to compare the 'For Tender' drawings with the 'As-Built' structure. No discrepancies were noted.

3 The Site

3.1 Topography

The site on 41 Weld Street is generally flat. The ground level at the site is approximately at 2.7m RL with the terrain gently sloping down towards the Tasman Sea towards the west and the Hokitika River towards the south.

3.2 Regional Geology

Geological maps published by the Institute of Geological & Nuclear Sciences (IGNS, 2002) indicate the site geology comprises postglacial (Holocene) deposits, including *beach sand and gravel (Q1b)*, refer to Figure 3-1.



Figure 3-1 Geological map of the Hokitika area, Institute of Geological & Nuclear Sciences (IGNS, 2002).

3.3 Building Foundations

Foundation plans from the Westland District Council (WDC) property file indicates the existing building is founded on a mix of ground beams and shallow foundations, typically founded 2m below ground level. No photographic records of site excavations during construction or as-built information was available on the WDC property files. A copy of the Lewis and Bradford Drawings (1993) are included in Appendix B.

3.4 Seismicity

The site is located 24km west of the Alpine Fault and the following should be considered, when assessing the potential ground shaking intensities in a major earthquake scenario:

A Peak Ground Acceleration (PGA) model[1] from the Westland District (bordering indicates that an M8 earthquake (Alpine Fault rupture) would produce Peak Ground Accelerations (PGA's) of 0.5g for the Hokitika area, refer to Figure 3-2.

A map of Modified Mercalli intensities[1] from the Westland District indicate intensity MMXIII (*Severe - expected damage slight in specially designed structures; considerable damage in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned*), refer to Figure 3-3.



Figure 3-2 Indicative Peak Ground Acceleration for the Westland District.

[1] Westland District Council Lifelines Study. Alpine Fault Earthquake Scenario. WDC (2006)

©WSP | 26 November 2019

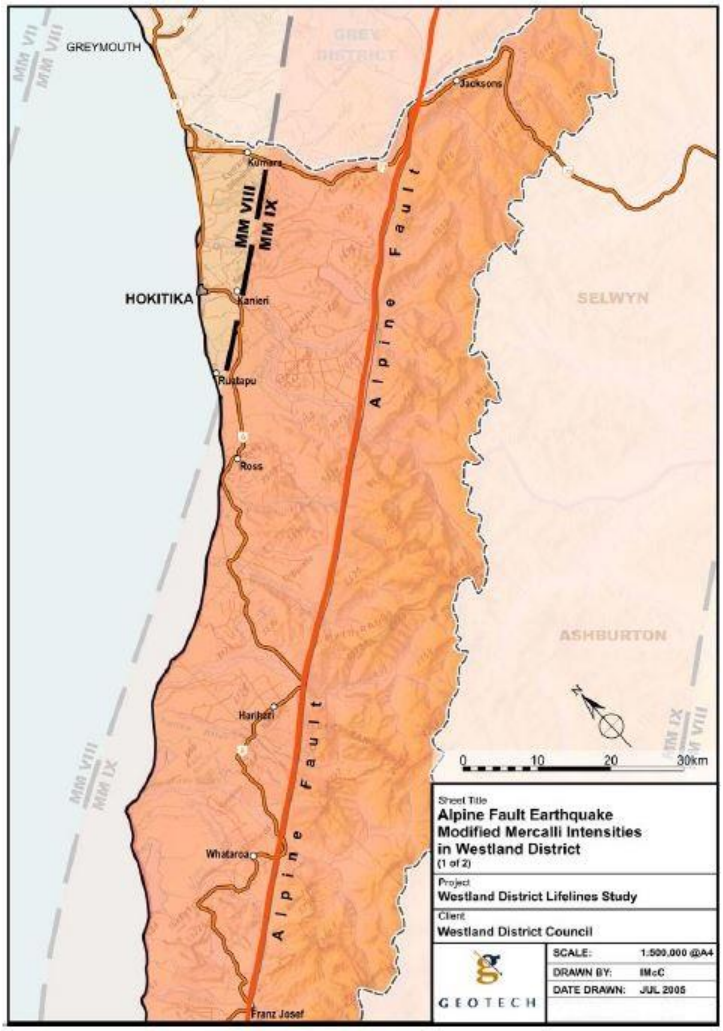


Figure 3-3 Indicative Mercalli intensities for the Westland District.

3.4.1 Importance Level

An Importance Level (IL) of 2 is considered appropriate for the liquefaction assessment of this site, in accordance with NZS 1170.5:2004.

3.4.2 Design Peak Ground Accelerations

According to NZS 1170.5:2004, the design PGA for Serviceability Limit State (SLS) and Ultimate Limit State (ULS) for a building of Importance level 2 should be 0.13g and 0.51g respectively.

3.5 Ground Conditions

3.5.1 Site soil stratigraphy

A review of New Zealand Geotechnical Database shows one borehole within 11m distance from the southwestern corner of the building (BH_88151). Cone Penetrometer Testing (CPT) data is also available from 127-137 Revell Street, which is at 338m distance from the Site (CPT_109538). For map of local borehole locations, refer to Figure 3-4.

©WSP | 26 November 2019



Figure 3-4 Map of available borehole information near the site on 41 Weld Street, Hokitika. Borehole 88151 is located 11m from the southwestern corner of the building. Cone Penetrometer Testing (CPT) data is also available from 127-137 Revell Street (CPT 109538), which is at 338m distance from the Site.

The machine auger borehole (BH_88151) was completed to a depth of 10.15m and included SPT testing at various intervals downhole. For a borehole summary, refer to Table 3-1.

Table 3-1 Borehole summary (BH_88151):

Soil type	From (m)	To (m)	Thickness (m)	Notes
Sandy **GRAVEL**	0.00	1.70	1.70	Imported material.
SILT with organic material	1.70	2.10	0.40	Ground water level at 2.00m. **Medium dense** (SPT at 1.70-2.05m).
Silty sandy **GRAVEL**	2.10	3.40	1.30	Saturated. **Medium dense** (SPT at 2.70-3.05m and 3.70-4.05m).
Sandy **SILT**	3.40	4.00	0.60	Saturated. Some stones.

©WSP | 26 November 2019

Coarse **SAND** with gravel and silt	4.00	6.70	2.70	Saturated. **Medium dense – Dense** (SPT at 4.70-5.05m; 5.70-6.05m and 6.70-7.05m).
Sandy **SILT**	6.70	7.10	0.40	Saturated. Some and stones.
Coarse **SAND** with silt	7.10	7.70	0.60	Saturated.
GRAVEL with sands and silt	7.70	10.15 (EOH)	2.45	**Very dense** (SPT at 7.70-8.05m; 8.70-9.05m and 9.70-10.05m).

The available CPT logs indicate soils dominated by sands, silty sands, sandy silt and gravelly sand. Estimated groundwater level (GVL) at 2.45m. Cone resistance values of >8MPa are generally found between 1.70-4.80m.

3.5.2 Sub soil class

On the basis of the available borehole information and regional geological maps Subsoil Class D has been selected as the most appropriate Subsoil class according to NZS1170.5:2004[2] .

3.5.3 Liquefiable soils

Liquefaction susceptible soils are typically saturated, recent (Holocene age) loose silt and sands. Liquefaction occurs when excess pore water pressures build up during cyclic earthquake shaking leading to loss of soil strength.

The Westland District Councils Lifelines Study[1] includes a map of liquefaction zones (corresponding to ground shaking zones) for the Hokitika area, refer to Figure 3-5. The Site is located in Zone 1a (estuarine deposits softer and finer than the rest of *Zone 1, which includes deep soils, more than 20m deep of moderate density).* Zone 1a is considered to have a higher liquefaction potential. It should be noted that the boundary to Zone 1a is approximate only and there is insufficient data and no in-situ testing to verify the liquefaction potential.

[2] NZS1170.5:2004. New Zealand Standard. Structural design actions. Part 5: Earthquake actions – New Zealand.

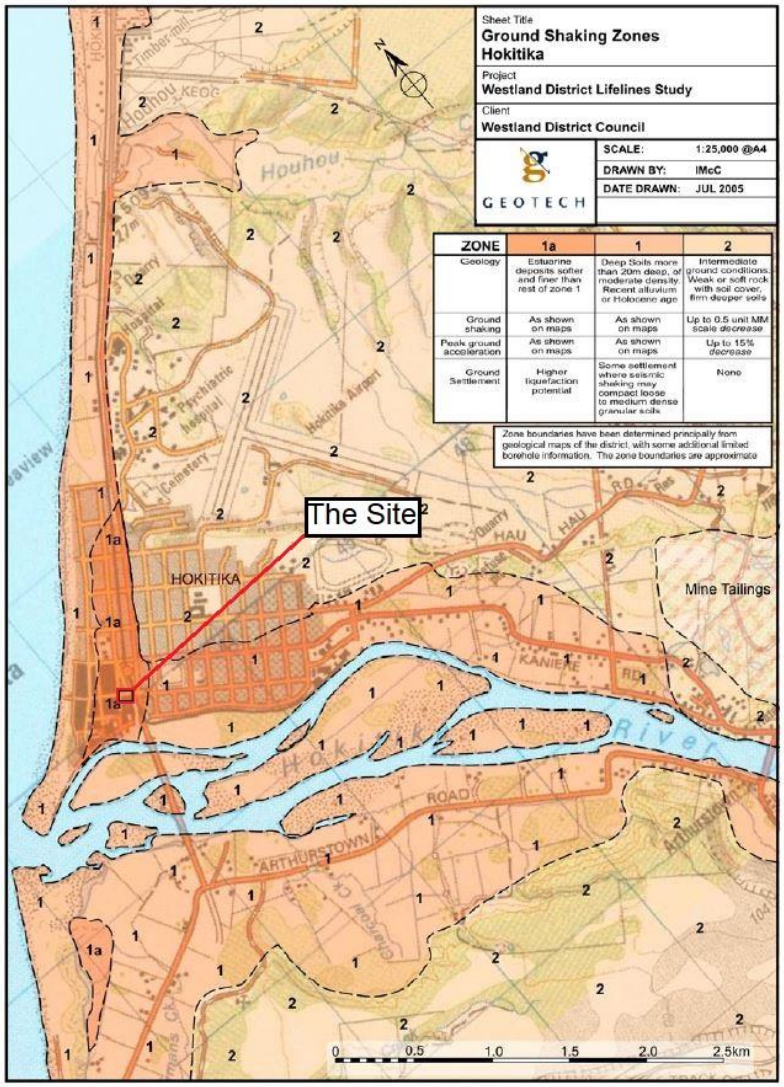


Figure 3-5 Ground Shaking Zones map for the Hokitika area.

Nearby borehole data, refer to Table 1, indicate that two relatively thin intervals of sandy SILT were intercepted in borehole BH_88151 at depths of 3.40-4.00 and 6.70-7.10m, which could be susceptible to liquefaction (saturated ground conditions, GVL at 2m). Density of soils derived from SPT testing indicate borehole soil densities generally ranging between Medium dense – Very dense, but no SPT testing was done in the two layers of sandy SILT.

CPT data (CPT_109538) indicate low cone resistance (2-4MPa), low sleeve friction (25-50kPa) combined with increased pore pressure (up to 800 kPa) between 7.25-7.75m. A brief liquefaction assessment (in **Cliq software**) indicates that liquefiable soils are present between 4.75 - 8.25m with potential for 60mm vertical settlement (volumetric settlement).

Considering the distance (338m) to the nearest CPT testing point these liquefaction predictions can only be inferred and ground conditions are likely to be variable.

3.5.4 Bearing capacity

The general foundation depth (2.00m) coincides approximately with the top of a layer of medium dense, silty sandy GRAVEL (2.10m). Scala penetrometer results from the nearby scala penetrometer testing at 24 Sewell Street borehole (BH_88151) indicate an ultimate bearing capacity of 300kPa (5 blows or more per 100mm) is present at depths below 1.60m, refer to Figure 3-6.

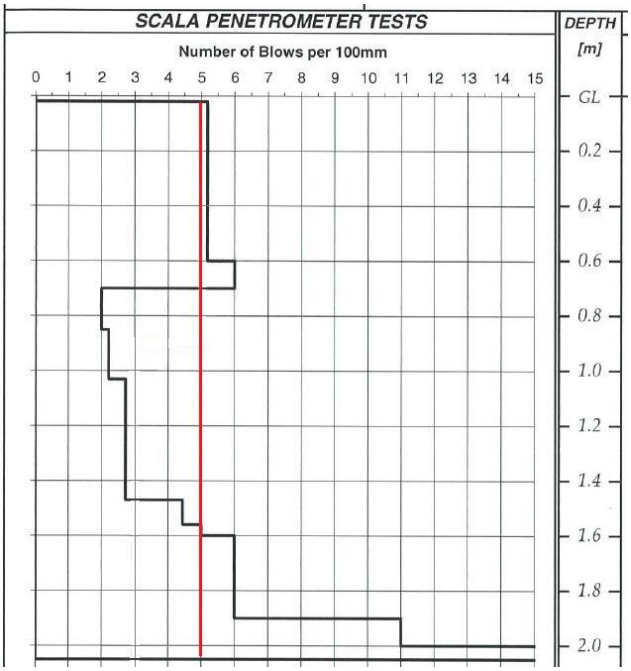


Figure 3-6 Scala penetrometer from BH_88151 location. The red line indicates ultimate bearing capacity of 300kPa (5 blows per 100mm).

It is noted that this is the static bearing capacity. Should liquefaction occur, this is likely to result in a reduction of available bearing capacity.

©WSP | 26 November 2019

4 The Structure

4.1 Building Description

The Mountain Jade building has a footprint area of approximately 1242m² with a floor area of approximately 2246m². The majority of the building was constructed in 1993/94, with the south-end glass workshop constructed at an earlier unknown date. The building is currently occupied by the Jade Factory and a café at ground floor; and offices, a beauty spa, and a backpacker's accommodation on the first floor. A large space at first floor level, marked 'gym' on the drawings, is currently unoccupied.

For the purposes of this assessment, the building has been divided into 3 distinct areas: D1, D2 and D3, as shown in Figure 4-1 below.

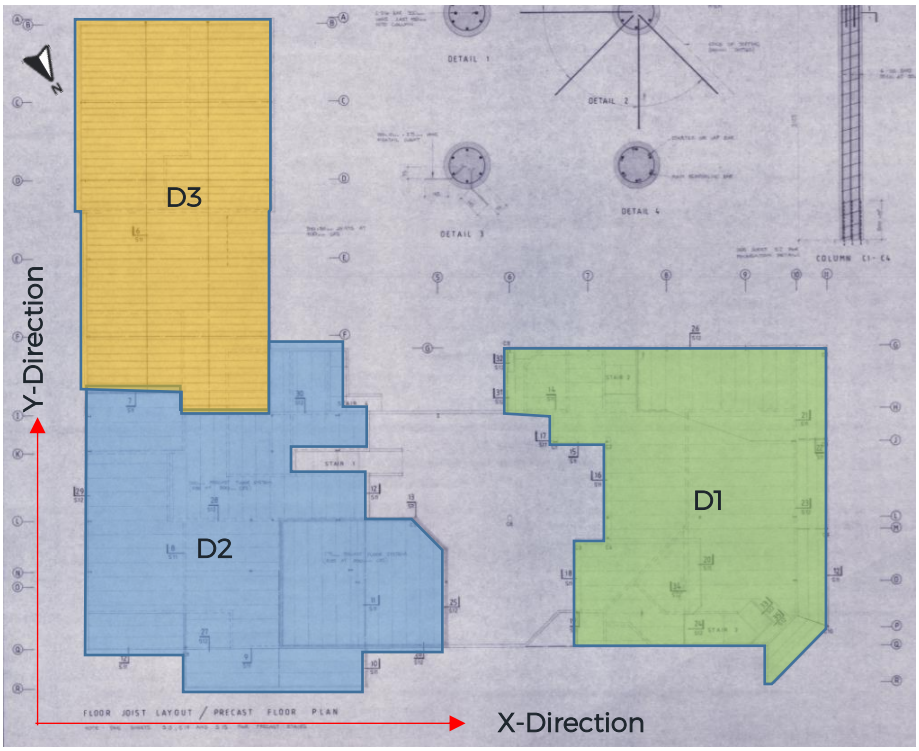


Figure 4-1: D1, D2 and D3

4.2 Primary Gravity Load Resisting System

4.2.1 D1 & D2

In the main retail part of the building, the lightweight roof (profiled metal) cladding is supported by steel portal frames spanning between walls and columns – internal columns are reinforced concrete (RC). The south-eastern external wall is RC tilt slab extending from floor to roof level. Other first floor walls are timber-framed, and ground floor walls are RC tilt slab.

The first floor is a rib-and-infill precast floor system with a large central void where the retail floor space extends up to roof level. This suspended floor slab system is supported on the precast concrete walls.

The ground floor is RC slab on grade. The structure's foundations are shallow RC barrette piles, typically 2.0m deep. The pile width ranges from 500mm to 900mm.

©WSP | 26 November 2019

4.2.2 D3

In the 'Existing Jade Workshop' original part of the building, the lightweight roof (profiled metal) cladding is supported by steel portal frames spanning the roof and extending down to ground floor level to bear on concrete foundations. Timber-framed walls line the upper floor, and the original concrete structure forms the ground floor walls.

The first floor is a timber particle board floor supported by timber joists spanning between the portal frames' steel beams. The ground floor is RC slab on grade. The original structure has been underpinned with shallow piles.

4.3 Primary Lateral Load Resisting System

4.3.1 D1 & D2

The lightweight roof with steel tension-only bracing in some locations, acts as a flexible diaphragm, transferring lateral loads to the steel portal frames in the x-direction, and to timber walls in the y-direction (refer Figure 4-1). A rib-and-infill precast concrete system acts as a rigid diaphragm at first floor level, transferring lateral loads to the concrete tilt panel walls at ground level. Along the eastern wall, tilt panels extend from ground floor to roof height. A void in the middle of the building presents some plan irregularity and out-of-plane flexure issues for the precast RC wall panels.

The precast wall panels are anchored to the concrete foundation via 450mm long fishtail steel plate cast 325mm into the foundation.

4.3.2 D3

In the 'Existing Jade Workshop' original part of the building, the lightweight roof acts as a flexible diaphragm, transferring lateral loads to the two-storey portal frames in the longitudinal direction. In the transverse direction, timber walls resist lateral forces at roof level and the original concrete structure resists lateral forces at ground level. The first-floor is timber, providing a flexible diaphragm.

4.4 Site Inspection & Building Condition

A site inspection was carried out by WSP Opus on 12 September 2019. The inspection involved a walkthrough visual inspection of the interior and exterior of the building and random measurements to confirm the as-built structure against the 'For Tender' drawings. No major discrepancies were found between the 'For Tender' drawings and the structure.

The building structure appears to be in good condition with no apparent structural damage, corrosion or deterioration.

4.5 Alterations

There have been two alterations to the building since the 1993/94 development, these are:

- In 2007, construction of a new timber-framed veranda roof over the escape deck at the rear of the building.
- In 2009, installation of an internal partition and roller door; and removal of a door and non-load bearing wall between the south-end glass workshop and the saw room. During these works, a concrete slab was constructed against the ground to form the garage floor.

4.6 Assumptions

The following assumptions were made during this assessment:

- It is assumed that the building has been constructed in accordance with the 'For Tender' drawings available at the building site. Some check measurements were obtained onsite, but this was not a full, detailed measure-up.
- The site soil conditions are adequate to support all the loads imposed by the building on its foundations. The Ultimate Bearing Capacity of the soil is assumed to be 300MPa. This assumption has taken into consideration the findings in the geotechnical desktop study that the layers prone to liquefaction at depth 3.4-6.7m, and water level 2.45m.
- Hidden damage/deterioration (if any) has no impact on the structural performance of the building.
- Material properties in Table 4-1 have been adopted for this assessment.

Table 4-1 : Material Properties

Material	Characteristic Strength (from Specification)	Probable Strength	Justification (from MBIE Guidelines)
Foundation Concrete	17.5MPa	26.3MPa	C5.4.2.2
Ground Floor Slab	25MPa	40MPa	Table C5.3
Concrete Columns	20MPa	30MPa	C5.4.2.2
Concrete Wall Panels	25MPA	40MPa	Table C5.3
Suspended Floor and Topping	25MPa	40MPa	Table C5.3
Original building Concrete	-	20MPa	Table C5.4.3.2
Grade 430 Reinforcement	Lower bound: 430MPa Upper bound: 500MPa	465MPa	Table C5C.4
Grade 300 Reinforcement	Lower bound: 300MPa Upper bound: 355MPa	328MPa	Table C5C.4
Welded Steel Wire Mesh	Lower bound: 575MPa Upper bound: 855MPa	715MPa	C5C.3
Structural Steel Grade 43	275MPa	316MPa	Table C6.2
Timber – Pinus radiata	No. 1 Framing Grade	Bending: 14.8MPa Comp. parallel: 12.7MPa Comp. parallel: 5.3MPa Shear: 2.4MPa	Table C9.1

5 Detailed Seismic Assessment

5.1 Analysis Methodology

The New Zealand standard methodology for assessing the earthquake performance of existing buildings is specified in the guidelines prepared by the Ministry of Business, Innovation and Employment (MBIE) and the New Zealand Society for Earthquake Engineering (NZSEE) jointly with Structural Engineering Society (SESOC) and New Zealand Geotechnical Society (NZGS). The force-based methodology described therein was used in this assessment.

The building has been assessed in three parts, marked D1, D2 and D3 in Figure 4-1. This simplification is acceptable because the three areas have independent floor diaphragms at first-floor level. The seismic resisting elements under each diaphragm resist seismic forces independently.

The first-floor structures in D1 and D2 were analysed as 'parts' because the mass and stiffness above the first floor are significantly less than that of the ground floor. The first floor of D3 was analysed as part of the main seismic resisting system since the steel frames extend from ground floor to the roof. The building was assessed using an equivalent static method for seismic analysis.

5.1.1 D1: Coffee Shop/Backpackers

The northernmost part of the building comprises two storeys: the ground floor houses the coffee shop, an office and part of the Jade Factory retail space, and the upper storey houses the backpacker's accommodation. Manual calculations were carried out to analyse the steel portal frames and timber walls at first floor level, treating the roof as a 'flexible diaphragm'.

The first-floor is constructed of a Rib and Infill system using precast ribs 125mm deep and 90mm in-situ concrete topping reinforced with 663 mesh. Ground-floor walls are 150mm precast concrete walls reinforced with D12 mild steel bars at 300mm centres each way. The first-floor slab is dowelled into the top of the precast walls at ground floor with D16 rebar. The structure was modelled and analysed in ETABS software, as shown in Figure 5-1 below. The precast walls were modelled as shell elements and the floor as a rigid diaphragm consisting of one-way spanning membranes. The walls above door and window openings were treated as spandrels and modelled as line elements.

The foundation is comprised of a system of slab on grade and shallow rectangular concrete piles (or barrette piles). To account for the foundation-structure interaction, the foundation was modelled as shell elements supported on compression-only soil springs.

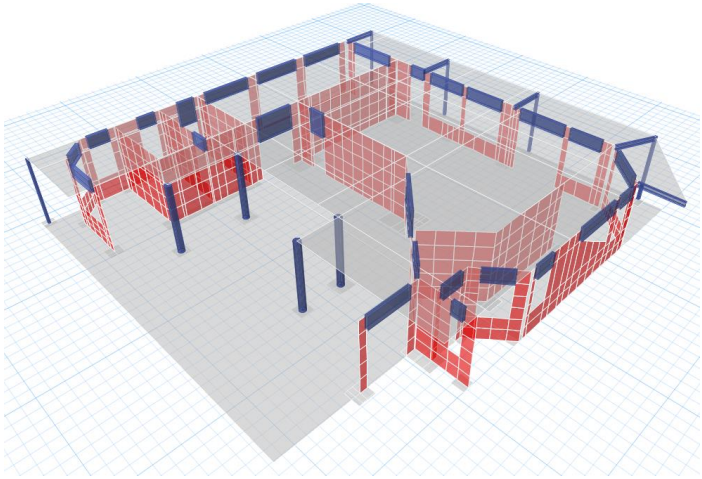


Figure 5-1: D1

5.1.2 D2: Retail Frontage

The easternmost part of the building comprises two storeys: the ground floor houses two retail shops, the jade carving workshops, toilets and storage, and the upper storey houses a beauty spa and offices. Manual calculations were carried out to analyse the steel portal frames and timber walls at first floor level, treating the roof as a 'flexible diaphragm'.

D2 comprises a similar seismic resisting structure to D1. The precast walls were modelled as shell elements and the floor as a rigid diaphragm consisting of one-way spanning

membranes. The walls above door and window openings were treated as spandrels and modelled as line elements.

The foundation is comprised of a system of slab on grade and shallow rectangular concrete piles (or barrette piles). To account for the foundation-structure interaction, the foundation was modelled as shell elements supported on compression-only soil springs.

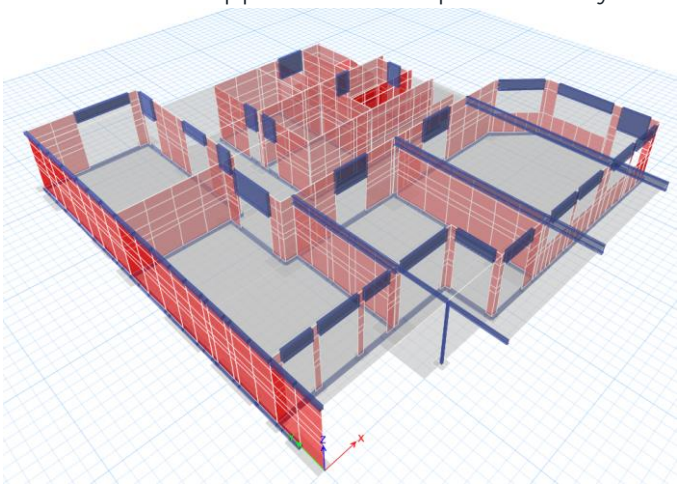


Figure 5-2: D2

5.1.3 D3: Original Workshop

The southernmost part of the building consists of an older reinforced concrete lower floor housing part of the backpacker's accommodation and some storage. The upper floor was built during the 1993 construction and is currently vacant.
Manual calculations were carried out to analyse the steel portal frames, timber walls, RC walls, and timber floor diaphragm.

5.2 Gravity Loads

The gravity loads include self-weight of the structure and internal fit-outs and linings, superimposed dead-loads, and occupancy live-loads. The self-weight of the structural elements is based on respective material unit weights which were taken as shown in Table 5-1 below.

Table 5-1 : Material Densities

Description	Density
Concrete	24kN/m^3
Steel	77kN/m^3
Timber	5.5kN/m^3

The current use of the building includes: a café, a hostel, offices, light industrial work rooms, retail space, and a beauty spa. Due to the wide range of uses of the building, an occupancy live-load of 3.0kPa was assumed based on AS1170.1 Table 3.1. If the building's use changes significantly in the future, this may impact the required occupancy live-load (e.g. if the building was used for book storage or heavy machinery).

5.3 Earthquake Loads

An Equivalent Static Analysis procedure has been adopted to determine the seismic demands on the building and the corresponding forces on the key structural elements. This procedure is appropriate for this building due to its short structural period and its modal response dominated by lower modes.

5.3.1 Specific Parameters for Seismic Analysis

Equivalent Static Method Seismic Design Coefficient

A structural acceleration spectrum based on NZS1170.5 was produced as shown below. From the acceleration response spectra, the unreduced peak acceleration is 1.093g for ULS (1/500-year event) with a structural period less than 0.50 seconds (refer Figure 5-3).

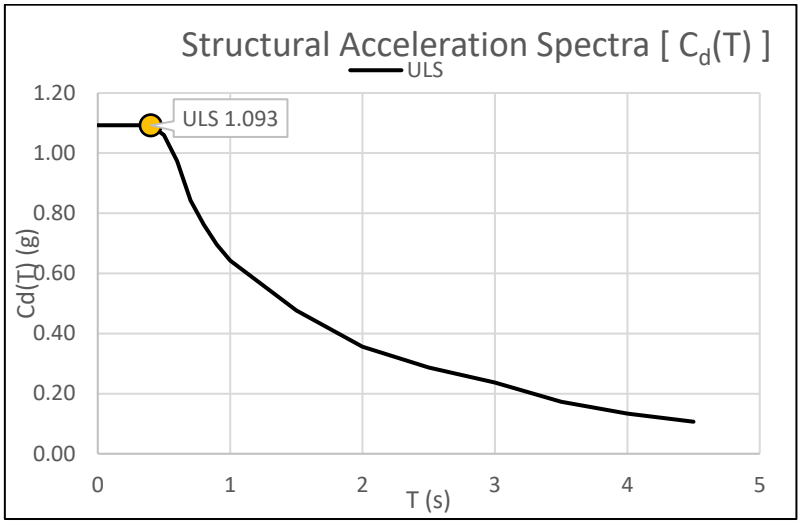


Figure 5-3: Structural Acceleration Spectrum (5% damped)

Site and structure parameters were used as shown in Table 5-2 below.

Table 5-2 : Seismic Design Coefficient Parameters

Description	ULS (1/500-year event)
Site Soil Classification	D
Hazard Factor, Z	0.45 (Hokitika)
Ductility, μ	1.25 (RC structure and fully welded portal frames) and 2.5 for timber walls
Structural performance factor, S_p	0.925
Factored structural ductility, k_μ	1.18
Horizontal design coefficient, $C_d(T)$	1.06
Importance Level	2

The ductility was chosen based on the structural elements' details on the available drawings. The precast walls that act as shear walls are typically singly-reinforced and the steel moment frames are made of non-compact sections.

Design Response Coefficient for Parts

A design response coefficient of 1.06 was calculated for ULS (1/500-year event). This was based on factors shown in Table 5-3 below, in addition to the factors above. This peak acceleration was used for D1 and D2.

Table 5-3: Design Response Coefficient Parameters

Description	ULS (1/500-year event)
Part Risk Factor	0.9
Part Ductility, μ_p	1.00
Height of uppermost seismic mass	3.425
Height of attachment of part	3.425m (first floor level where steel frames are attached)

©WSP | 26 November 2019

5.3.2 Seismic Weight

The seismic weight comprises the permanent weight of the structure and a percentage of the occupancy live load. A live-load of 0.25kPa was considered for the roof loads with a combination factor of 0, i.e. no contribution to seismic weight. A live-load of 3kPa was considered for the floors with a combination factor of 0.3.

5.3.3 Seismic Base Shear

The seismic base shear is the product of the seismic weight, the acceleration corresponding to the structural period of the building, and the seismic parameters inherent to the building ductility.

6 Results of the Detailed Seismic Assessment

6.1 First Floor

6.1.1 D1/D2

Steel portal frames over the first floor of D1 and D2 are made of 200UB25 sections (note this is a non-compact section). These frames provide the main lateral load resisting elements in the x-direction. The critical seismic demands on these frames were analysed using Microstran and manual calculations. The result of the assessment indicated that the portal frames are 15%NBS governed by the flexural capacity of the beams.

Timber-framed Gib-lined walls provide the main lateral load resisting elements in the y-direction. The seismic demands on these walls were analysed using manual calculations. The result of the assessment indicated that the timber-framed walls are 40%NBS.

Precast tilt slab walls at first floor provide lateral load resistance along the south-eastern wall in the y-direction. The seismic demands on these walls were analysed using manual calculations. The result of the assessment indicated that the precast slab walls are 67%NBS.

6.1.2 D3

Steel portal frames of D3 are made of UB sections. These frames provide the main lateral load resisting elements in the x-direction at the first floor. The critical seismic demands on these frames were analysed using SAP2000 and manual calculations. The result of the assessment indicated that the portal frames are 40%NBS governed by the flexural capacity of the columns.

Timber-framed Gib-lined walls provide the main lateral load resisting elements in the y-direction. The seismic demands on these walls were analysed using manual calculations. The result of the assessment indicated that the timber-framed walls are 25%NBS. Although the capacity is low, this is not considered critical because of the presence of an alternative load path which is the cantilever action of the steel column via weak axis bending.

The timber floor diaphragm transfers shear to the portal frames through shear resistance of the MDF panel flooring. This was assessed using manual calculations. The result of the assessment indicated that the timber floor diaphragm has 38%NBS and 95%NBS seismic capacity in the x and y directions respectively.

6.2 Ground Floor

6.2.1 D1

Precast walls were modelled in ETABS. In the x-direction, precast walls achieved a result of 65%NBS for shear and combined axial and flexural loads and gave a result of 40%NBS in the y-direction.

Precast spandrels were modelled in ETABS giving a result of 60%NBS in the x-direction and 60%NBS in the y-direction.

The rib-and-infill diaphragm was modelled in ETABS giving a result of 50%NBS in the x-direction and 60%NBS in the y-direction.

Foundations were modelled in ETABS giving a required bearing pressure of 200kPa in the x-direction and y-direction. This is equivalent to 100%NBS for a soil with an ultimate bearing capacity of 300kPa.

6.2.2 D2

Precast walls were modelled in ETABS. In the x-direction, precast walls achieved a result of 70%NBS for shear and combined axial and flexural loads, and gave a result of 67%NBS for out-of-plane bending in the exterior wall along Grid 2/Q-I. In the y-direction, precast walls achieved a result of 35%NBS for out-of-plane bending in the exterior wall along Grid 5/Q, and 30%NBS for out-of-plane bending in the exterior wall along Grid 5/I.

Precast spandrels were modelled in ETABS giving a result of 80%NBS in the x-direction and 80%NBS in the y-direction.

The rib-and-infill diaphragm was modelled in ETABS giving a result of 25%NBS in the x-direction and 25%NBS in the y-direction. This is governed by the pull-out capacity of the slab anchors (M12 TCM inserts) to the precast walls due to the out-of-plane forced induced by the pre-cast walls.

Foundations were modelled in ETABS giving a result of 67%NBS in the x-direction and 67%NBS in the y-direction.

6.2.3 D3

The steel portal frame along grid line D is the most critical frame in the x-direction with an assessed seismic capacity of 40%NBS.

The original RC walls provide the main lateral load resisting elements in the y-direction. The critical seismic demands on these frames were analysed using manual calculations. The result of the assessment indicated 38%NBS.

6.2.4 Concrete Stairs

The stairs are typically constructed of reinforced concrete. Based on the available drawings, there is no rigid connection between the stair landing and the stair but there is also no clear indication if it can slide. There is no detail provided for the stair to floor connection. Due to incomplete information, the concrete stairs were not assessed.

©WSP | 26 November 2019

6.3 Summary of Assessed Seismic Capacity

Table 6-1: Assessed Seismic Capacity (%NBS)

Level	Area	Element	%NBS		Failure Mode/Remarks
			X-dir	Y-dir	
First Floor	D1/D2	Steel Frame	15%	-	Combined Axial & Flexure
		Timber Walls	-	40%	Shear resistance of timber-framed walls
		PC Walls	67%	-	Out-of-plane bending, exterior wall along grid 2/Q-I
	D3	Steel Frame	40%	-	Combined Axial & Flexure
		Timber Walls	-	100%	Shear resistance of timber-framed walls
		Timber Floor Diaphragm	38%	95%	Shear resistance of MDF Panel, Connections not checked as there is no information
Ground Floor	D1	PC Walls	65%	40%	Combined axial & flexure
		PC Spandrels	60%	60%	Combined axial & flexure
		Diaphragm	50%	60%	Seating Length
		Foundation	100%	100%	Bearing
	D2	PC Walls	70%	70%	Shear, Combined Axial & Flexure
		PC Walls	67%	-	Out-of-plane bending, exterior wall along Grid 2/Q-I
			-	35%	Out-of-plane bending, exterior wall along 5/Q (PC31 & PC32)
			-	30%	Out-of-plane bending, exterior wall along 5/I (PC17 & PC16)
		PC Spandrels	80%	80%	Combined Axial & Flexure
		Diaphragm	25%	25%	Pull out capacity of M12 TCM inserts.
		Foundation	100%	100%	Bearing
	D3	Steel Frame	40%	-	Combined Axial & Flexure
		Conc. Walls	-	38%	Shear resistance of original RC walls

See Figure 1 for the reference x and y directions.

6.4 Critical Structural Weakness

In accordance with the MBIE Guidelines, the lowest-scoring structural weakness determined from a DSA is considered to be a Critical Structural Weakness (CSW). The following elements have been assessed as having a %NBS less than 34%, and are therefore CSWs:

- D1: steel portal frames
- D2: steel portal frames, out-of-plane bending along exterior walls along 5/I, and the diaphragm for pull-out capacity of the M12 TCM inserts
- D3: steel portal frames

Failure of the portal frames is unlikely to lead to progressive collapse and a threat to life-safety to the building occupants as there is an alternative load-path through weak axis bending of the portal frames.

©WSP | 26 November 2019

Failure of the M12 TCM inserts does present a life-safety hazard.

7 Conclusions and Recommendations

The seismic capacities have been assessed in accordance with the MBIE Guidelines "The Seismic Assessment of Existing Buildings – Technical Design Guidelines for Engineering Assessments", July 2017. The earthquake rating is based on Importance Level 2 (IL2).

The assessed capacity is based on the undamaged state of the building and the roof and wall-diaphragm connections were assumed to be adequate to develop the member capacity.

The outcome of this assessment indicates that the existing Mountain Jade building has a seismic capacity of less than 34%NBS (IL2) in both directions. The assessed seismic capacity is governed by the precast floor connections to the precast walls (25%NBS) and out-of-plane bending of the precast wall along Grid 5/I (30%NBS).

The weakest lateral load resisting elements of this building are the steel portal frames. However, due to presence of an alternative load path (i.e. timber walls), these frames are unlikely to collapse under a significant earthquake and therefore not considered the limiting failure mechanism of this building.

The precast walls and precast floor are considered the critical structural weakness (CSW's) of this building with a limiting capacity of 25%NBS (IL2). Since the assessed capacity is less than 34%NBS it satisfies one of the requirements for an earthquake prone building (EPB).

In accordance with Table A3.1 of Assessment Guidelines, this building is considered a very high risk building with an Alpha Rating of E. The risk description can be considered to be relative life safety risks if a large earthquake occurs.

There is a legal requirement to strengthen the building with seismic capacity of less than 34%NBS although we recommend strengthening to at least 67%NBS.

The following are the conceptual strengthening solution to meet the minimum seismic capacity of 34%NBS:

First Floor

(1) Provide vertical braces at wall locations marked on plan. Develop new roof bracing system on the roof level to create a clear load path for the inertial loads to transfer to the first floor via new vertical braces.

(2) Install fly braces to the existing roof beams at spacing required to develop the beams section capacity without flange buckling.

Ground Floor

(3) Provide out-of-plane lateral supports to the precast walls at grid lines 5/Q (PC31 and PC32) and 5/I (PC16 and PC17). This will include vertical and horizontal steel support to shorten the span PC walls in both directions.

(4) Install additional floor to wall anchors along grid line 1. Anchor bolts can be drilled and epoxied to the wall at 900mm spacing and place alternately with the existing M12 TCM inserts.

The scope of strengthening to 67%NSB includes the following:

(4) All items under 34%NBS strengthening

(5) Install additional vertical braces (VB2) at the first floor

©WSP | 26 November 2019

(6) Thicken the existing precast walls (pier and spandrels) along grid lines 10 and G

A conceptual strengthening sketch of the proposed strengthening has been prepared with preliminary sizing and scoping for costing purposes. This is presented in Appendix C. The sizes, quantities and arrangements shall be finalised during the detailed design.

Prior to detailed design, we recommend that an invasive site investigation is conducted to further check the following:

- Roof connections
- Wall-diaphragm connections
- Stair to floor connections
- Extent of corrosion to steel (if any)
- Site specific geotechnical investigation and report.

©WSP | 26 November 2019

Appendix A

Assessment Summary Report Template

Appendix A – Assessment Summary Report Template

1. Building Information	
Building Name/ Description	Mountain Jade Building
Street Address	41 Weld Street, Hokitika
Territorial Authority	Westland District Council
No. of Storeys	2
Area of Typical Floor (approx.)	Ground Floor Area: 1,242m² First-Floor Area: 1,004m²
Year of Design (approx.)	1993
NZ Standards designed to	NZS 4203:1986
Structural System including Foundations	The property is a 2-storey complex constructed in 1993-94 (the south-end glass workshop was constructed at an earlier unknown date) and serves as a jade carving gallery/workshop and backpackers accommodation. Lightweight roof (profiled metal) cladding is supported by steel rafters spanning between walls and RC columns. The eastern and southern external walls are reinforced concrete tilt slab extending from floor to roof level. The western and northern facade walls are lightweight timber on the upper floor, and reinforced concrete on the lower floor. Upper floor internal walls are lightweight timber and the lower floor internal walls are reinforced concrete. Walls rest on concrete ground beams and ground floor loads are taken by a concrete slab. The suspended floor is rib and infill construction, providing a good diaphragm, but the void in the middle of the building does present some plan irregularity. This suspended floor slab system is supported on the precast concrete walls. Longitudinal and transverse seismic loads are taken by the reinforced concrete walls acting as shear walls. The building was found to be in good condition. The building foundation is comprised of rectangular concrete footing with a typical depth of 2m from the ground floor level.
Does the building comprise a shared structural form or shares structural elements with any other adjacent titles?	N/A
Key features of ground profile and identified geohazards	Situated on a flat site; located less than 200m north of Hokitika River and approx. 300m south-west of the beach front. Liquefaction risk unknown but from knowledge of the surrounding area the building is likely underlain by gravels.
Previous strengthening and/ or significant alteration	N/A
Heritage Issues/ Status	None

©WSP | 26 November 2019

Other Relevant Information	None
2. Assessment Information	
Consulting Practice	WSP
CPEng Responsible, including: • Name • CPEng number • A statement of suitable skills and experience in the seismic assessment of existing buildings[3]	Jamie Lester CPEng 260347 Current practice field is Structural Engineering design and analysis including seismic assessments and strengthening design of building structures.
Documentation reviewed, including: • date/ version of drawings/ calculations[4] • previous seismic assessments	July 1993 For Tender Drawings
Geotechnical Report(s)	None – geotechnical desktop study as outlined above.
Date(s) Building Inspected and extent of inspection	12 September 2019 – Detailed Seismic Assessment. Non-intrusive investigation and walkthrough. Check measurements taken.
Description of any structural testing undertaken and results summary	None.
Previous Assessment Reports	Initial Seismic Assessment (WSP Opus 22 August 2019)
Other Relevant Information	None

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

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

3. Summary of Engineering Assessment Methodology and Key Parameters Used	
Occupancy Type(s) and Importance Level	IL2 assumed
Site Subsoil Class	Class D (Deep Soil)
For an ISA:	
Summary of how Part B was applied, including: • Key parameters such as μ, S_p and F factors • Any supplementary specific calculations	N/A
For a DSA:	
Summary of how Part C was applied, including: • the analysis methodology(s) used from C2 • other sections of Part C applied	Equivalent Static Analysis Sections C1, C2, C3, C5, C6 and C9 were used
Other Relevant Information	N/A

©WSP | 26 November 2019

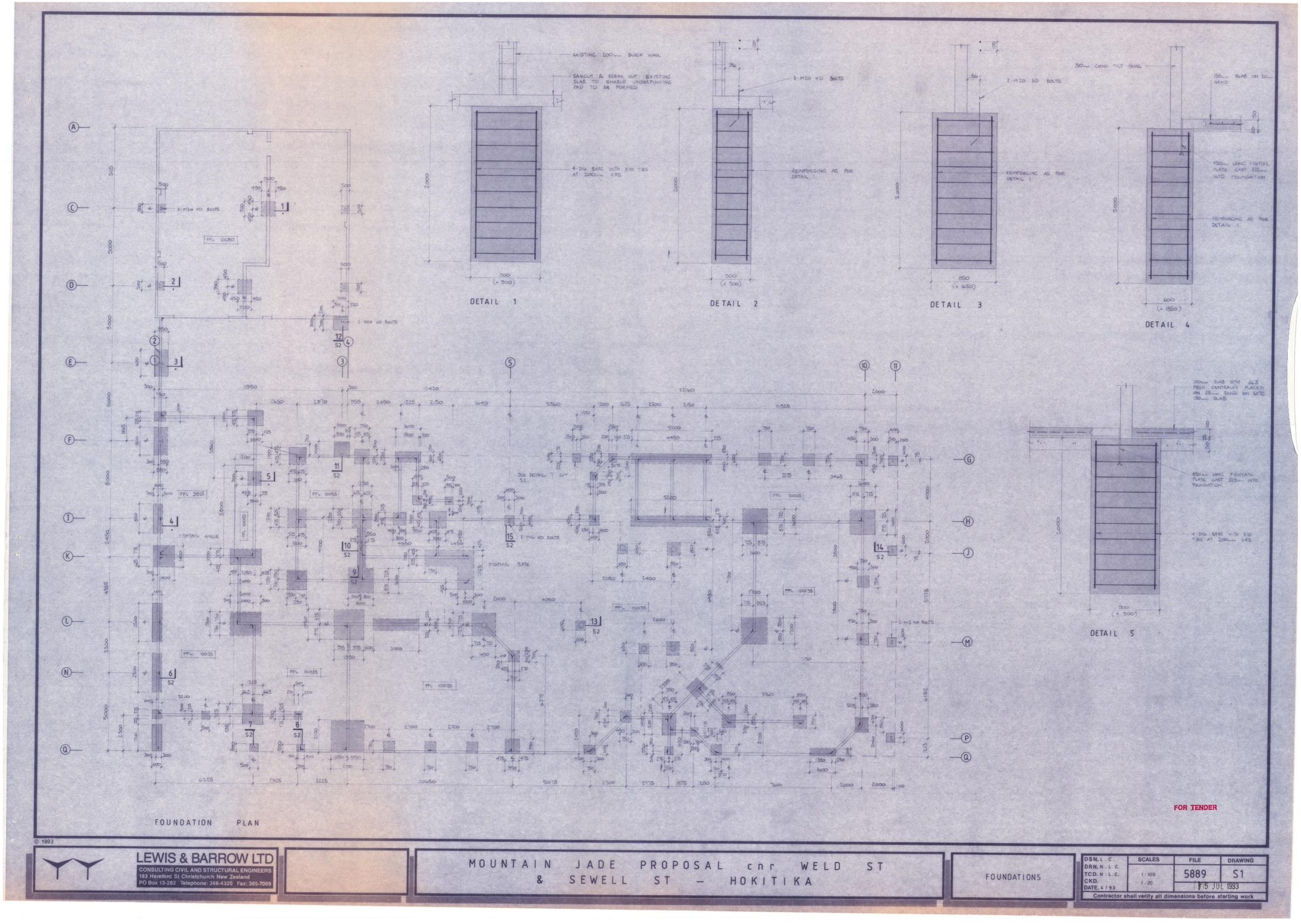
4. Assessment Outcomes		
Assessment Status (Draft or Final)	Final	
Assessed *%NBS* Rating	<34%NBS	
Seismic Grade and Relative Risk (from Table A3.1)	Grade E: Very High Risk	
For an ISA:		
Describe the Potential Critical Structural Weaknesses	N/A	
Does the result reflect the building's expected behaviour, or is more information/ analysis required?	N/A	
If the results of this ISA are being used for earthquake prone decision purposes, and elements rating <34*%NBS* have been identified:	**Engineering Statement of Structural Weaknesses and Location** N/A	***Mode of Failure and Physical Consequence* Statement(s)** N/A
For a DSA:		
Comment on the nature of Secondary Structural and Non-structural elements/ parts identified and assessed	None.	
Describe the Governing Critical Structural Weakness	Axial & Flexural Failure of the steel portal frames Shear resistance of the timber-framed walls	
If the results of this DSA are being used for earthquake prone decision purposes, and elements rating <34*%NBS* have been identified (including Parts)[5]:	**Engineering Statement of Structural Weaknesses and Location**	***Mode of Failure and Physical Consequence* Statement(s)**
Recommendations (*optional for EPB purposes*)	Strengthening recommendation to 67%NBS is attached in Appendix C.	

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

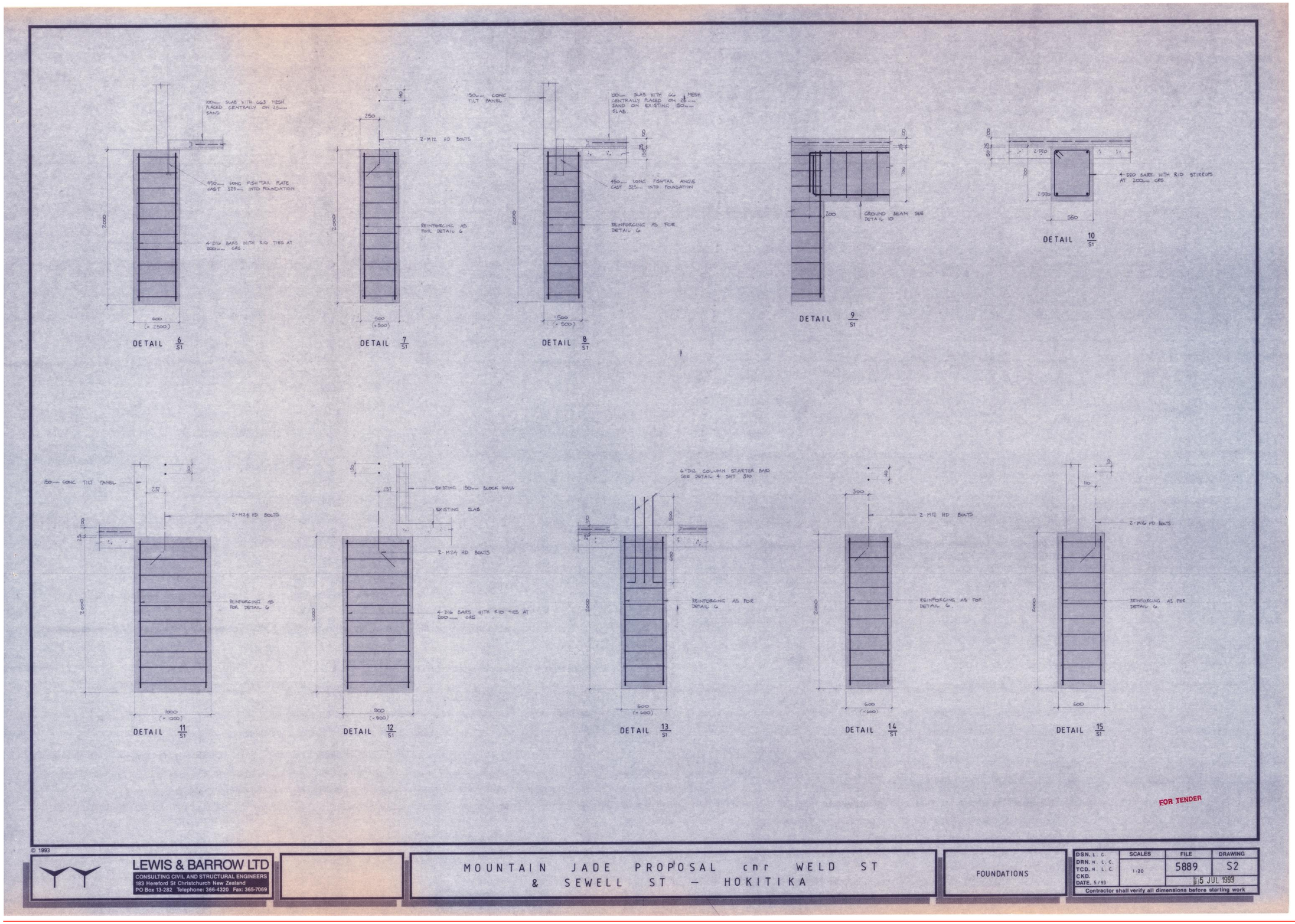
Appendix B

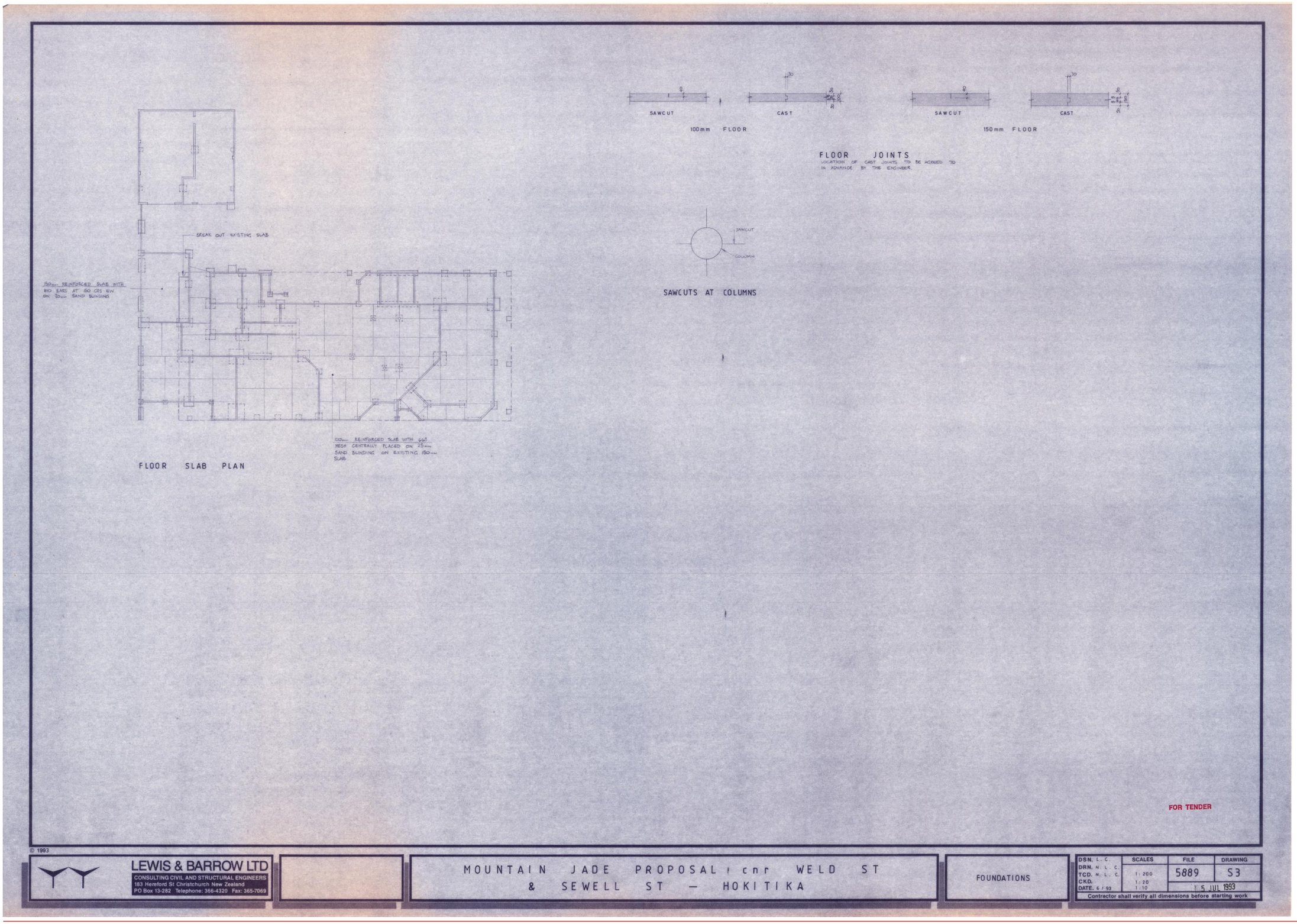
Record Drawings

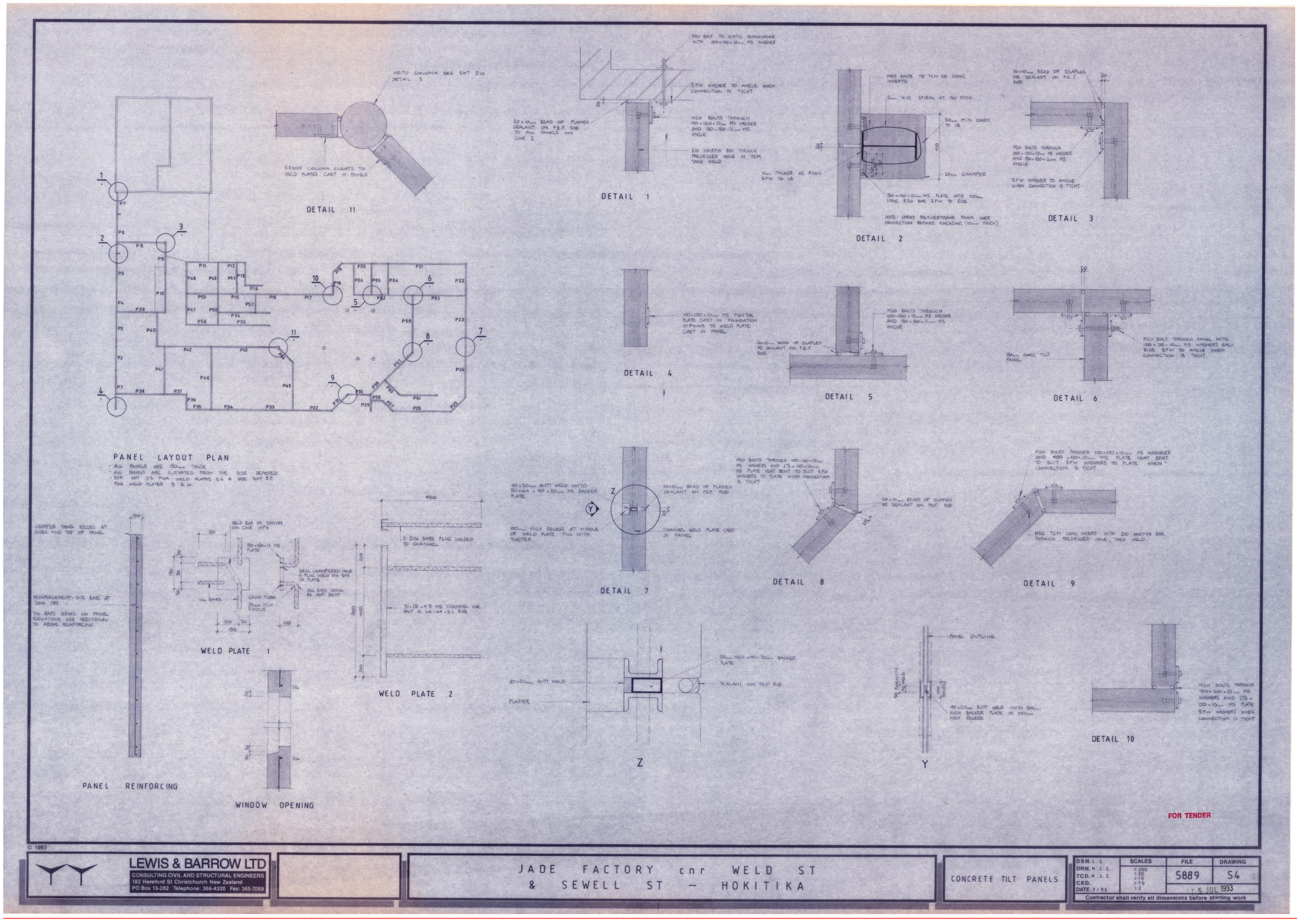
©WSP | 26 No



©WSP | 26 November 2019


100mm SLAB WITH 665 MESH PLACED CENTRALLY ON 25mm SAND
450mm LONG FISHTAIL PLATE CAST 325mm INTO FOUNDATION
4-D16 BARS WITH R10 TIES AT 200mm CRS.
2000
600 (x 2500)
DETAIL 6/S1
250
2-M12 HD BOLTS
REINFORCING AS FOR DETAIL 6
500 (x 500)
DETAIL 7/S1
150mm CONC TILT PANEL
100mm SLAB WITH 665 MESH CENTRALLY PLACED ON 25mm SAND ON EXISTING 150mm SLAB
450mm LONG FISHTAIL ANGLE CAST 325mm INTO FOUNDATION
REINFORCING AS FOR DETAIL 6
500 (x 500)
DETAIL 8/S1
200
GROUND BEAM SEE DETAIL 10
DETAIL 9/S1
2-D20
4-D20 BARS WITH R10 STIRRUPS AT 200mm CRS
560
DETAIL 10/S1
150mm CONC TILT PANEL
257
2-M24 HD BOLTS
REINFORCING AS FOR DETAIL 6
1000 (x 1000)
DETAIL 11/S1
EXISTING 150mm BLOCK WALL
EXISTING SLAB
2-M24 HD BOLTS
4-D16 BARS WITH R10 TIES AT 200mm CRS
900 (x 900)
DETAIL 12/S1
6-D12 COLUMN STARTER BARS SEE DETAIL 4 SHT S10
REINFORCING AS FOR DETAIL 6
600 (x 600)
DETAIL 13/S1
300
2-M12 HD BOLTS
REINFORCING AS FOR DETAIL 6.
600 (x 600)
DETAIL 14/S1
2-M16 HD BOLTS
REINFORCING AS FOR DETAIL 6.
600
DETAIL 15/S1
FOR TENDER
© 1993
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069
MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST – HOKITIKA
FOUNDATIONS
DSN. L. C.
DRN. N. L. C.
TCD. N. L. C.
CKD.
DATE. 5/93
SCALES 1:20
FILE 5889
DRAWING S2
15 JUL 1993
Contractor shall verify all dimensions before starting work


BREAK OUT EXISTING SLAB
150mm REINFORCED SLAB WITH H10 BARS AT 600 CRS EW ON 50mm SAND BLINDING
100mm REINFORCED SLAB WITH 665 MESH CENTRALLY PLACED ON 25mm SAND BLINDING ON EXISTING 150mm SLAB
FLOOR SLAB PLAN
SAWCUT
CAST
100mm FLOOR
SAWCUT
CAST
150mm FLOOR
FLOOR JOINTS
LOCATION OF CAST JOINTS TO BE AGREED TO IN ADVANCE BY THE ENGINEER.
SAWCUT
COLUMN
SAWCUTS AT COLUMNS
FOR TENDER
© 1993
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069
MOUNTAIN JADE PROPOSAL : cnr WELD ST & SEWELL ST — HOKITIKA
FOUNDATIONS
DSN. L. C.
DRN. N. L. C.
TCD. N. L. C.
CKD.
DATE. 6 / 93
SCALES
1 : 200
1 : 20
1 : 10
FILE
5889
DRAWING
S3
15 JUL 1993
Contractor shall verify all dimensions before starting work


INSITU COLUMN SEE SHT S10 DETAIL 3
5FWAR COLUMN CLEATS TO WELD PLATES CAST IN PANELS
DETAIL 11
M10 BOLT TO EXTG BLOCKWORK WITH 100x100x3mm MS WASHER
5FW WASHER TO ANGLE WHEN CONNECTION IS TIGHT
20x10mm BEAD OF FLAMEX SEALANT ON P.E.F. ROD TO ALL LINE 2 PANELS ON
M10 BOLTS THROUGH 100x100x10mm MS WASHER AND 150x150x12mm MS ANGLE
D10 HAIRPIN BAR THROUGH PREDRILLED HOLE IN TCM TACK WELD
DETAIL 1
M20 BOLTS TO TCM ED CONC INSERTS
5mm WIRE SPIRAL AT 150 PITCH
50mm MIN COVER TO UB
20mm CHAMFER
10mm PACKER AS REQD 5FW TO UB
150x100x10mm MS PLATE WITH 100mm LONG R20 BAR 5FW TO SIDE
NOTE: SPRAY POLYURETHANE FOAM OVER CONNECTION BEFORE ENCASING (10mm THICK)
DETAIL 2
20x10mm BEAD OF SILAFLEX MS SEALANT ON P.E.F. ROD
M20 BOLTS THROUGH 100x100x10mm MS WASHER AND 150x150x12mm MS ANGLE
5FW WASHER TO ANGLE WHEN CONNECTION IS TIGHT
DETAIL 3
100x100x10mm MS TISHTAIL PLATE CAST IN FOUNDATION
12 FWAR TO WELD PLATE CAST IN PANEL
DETAIL 4
M20 BOLTS THROUGH 150x100x10mm MS WASHER AND 150x150x12mm MS ANGLE
20x10mm BEAD OF SILAFLEX MS SEALANT ON P.E.F. ROD
DETAIL 5
M20 BOLT THROUGH PANEL WITH 100x100x10mm MS WASHERS EACH SIDE 5FW TO ANGLE WHEN CONNECTION IS TIGHT.
150mm CONC TILT PANEL
DETAIL 6
PANEL LAYOUT PLAN
- ALL PANELS ARE 150mm THICK
- ALL PANELS ARE ELEVATED FROM THE SIDE DENOTED
- SEE SHT S5 FOR WELD PLATES 1 & 4 SEE SHT S9 FOR WELD PLATES 5 & 6.
CHAMFER PANEL EDGES AT SIDES AND TOP OF PANEL
REINFORCEMENT - D12 BARS AT 300 CRS
D12 BARS SHOWN ON PANEL ELEVATIONS ARE ADDITIONAL TO ABOVE REINFORCING
PANEL REINFORCING
WELD BAR AS SHOWN ON CNR WPS
150x150x12 MS PLATE
D16 BARS
DRILL CHAMFERED HOLE & PLUG WELD D16 BAR TO PLATE
ALL BARS SHALL BE HEAT BENT
GRIND FLUSH
35mm MIN RADIUS
WELD PLATE 1
450
2-D16 BARS PLUG WELDED TO CHANNEL
51x25x4.5 MS CHANNEL OR HALF A 64x64x5.2 RHS
WELD PLATE 2
WINDOW OPENING
40x20mm BUTT WELD ONTO 50mm HIGH x 40x20mm MS BACKER PLATE
100mm HIGH RECESS AT MIDDLE OF WELD PLATE. FILL WITH PLASTER
20x10mm BEAD OF FLAMEX SEALANT ON P.E.F. ROD
CHANNEL WELD PLATE CAST IN PANEL
DETAIL 7
M20 BOLTS THROUGH 100x100x10mm MS WASHERS AND 275x100x10mm MS PLATE HEAT BENT TO SUIT 5FW WASHERS TO PLATE WHICH CONNECTION IS TIGHT
20x10mm BEADS OF SILAFLEX MS SEALANT ON P.E.F. ROD
DETAIL 8
M20 BOLTS THROUGH 100x100x10mm MS WASHERS AND 400x100x10mm MS PLATE HEAT BENT TO SUIT 5FW WASHERS TO PLATE WHICH CONNECTION IS TIGHT
M20 TCM CONC INSERT WITH D10 HAIRPIN BAR THROUGH PREDRILLED HOLE, TACK WELD
DETAIL 9
50mm HIGH x 40x20mm BACKER PLATE
40x20mm BUTT WELD
PLASTER
SEALANT ON P.E.F. ROD
Z
PANEL OUTLINE
50 BACKING ROD
20 SEALANT
40x20mm BUTT WELD ONTO 80mm HIGH BACKER PLATE IN 100mm HIGH RECESS
Y
M20 BOLTS THROUGH 100x100x10mm MS WASHERS AND 275x100x10mm MS PLATE 5FW WASHERS WHEN CONNECTION IS TIGHT
DETAIL 10
FOR TENDER
© 1993
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069
JADE FACTORY cnr WELD ST & SEWELL ST – HOKITIKA
CONCRETE TILT PANELS
DSN. L. C.
DRN. N. L. C.
TCD. N. L. C.
CKD.
DATE. 9/93
SCALES 1:200 1:20 1:10 1:25 1:2
FILE 5889
DRAWING S4
15 JUL 1993
Contractor shall verify all dimensions before starting work



©WSP | 26 November 2019

P 20 5·8T

P 21 5·2T

P 22 2·2T

P 23 5·6T

P 24 3·2T

P 25 1·5T

P 26 7·7T

P 27 1·8T

P 28 1·0T

P 29 2·6T

P 30 1·4T

P 31 2·0T

P 32 3·9T

P 33 4·8T

P 34 4·8T

P 35 2·0T

P 36 1·8T

DETAIL X

NOTE : 'X' OPPOSITE HANDED ABOVE DETAILS OF P20, P24, P25, P26, P27, P28, P30, P31 AND P32.

FOR TENDER

© 1993

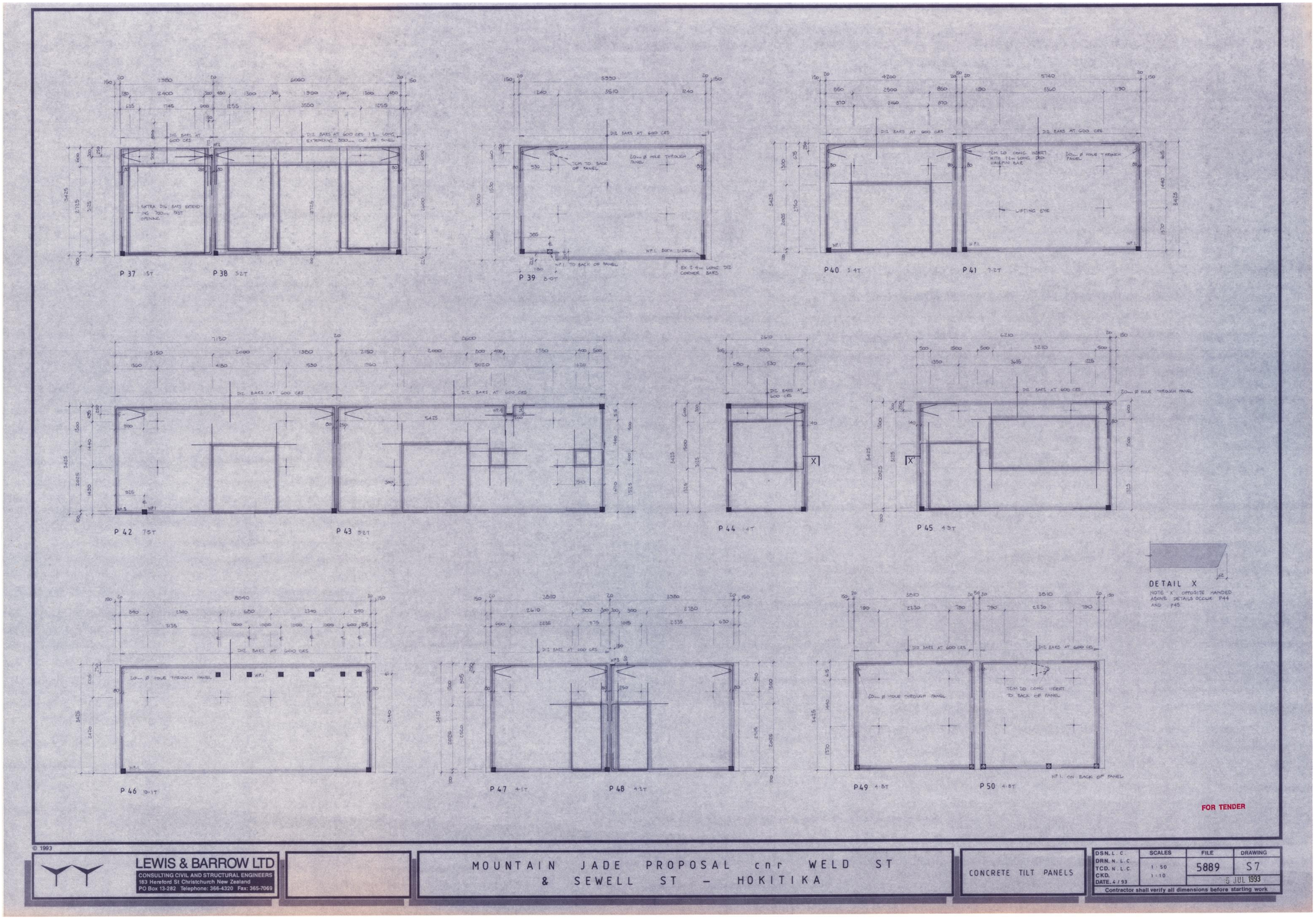
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069

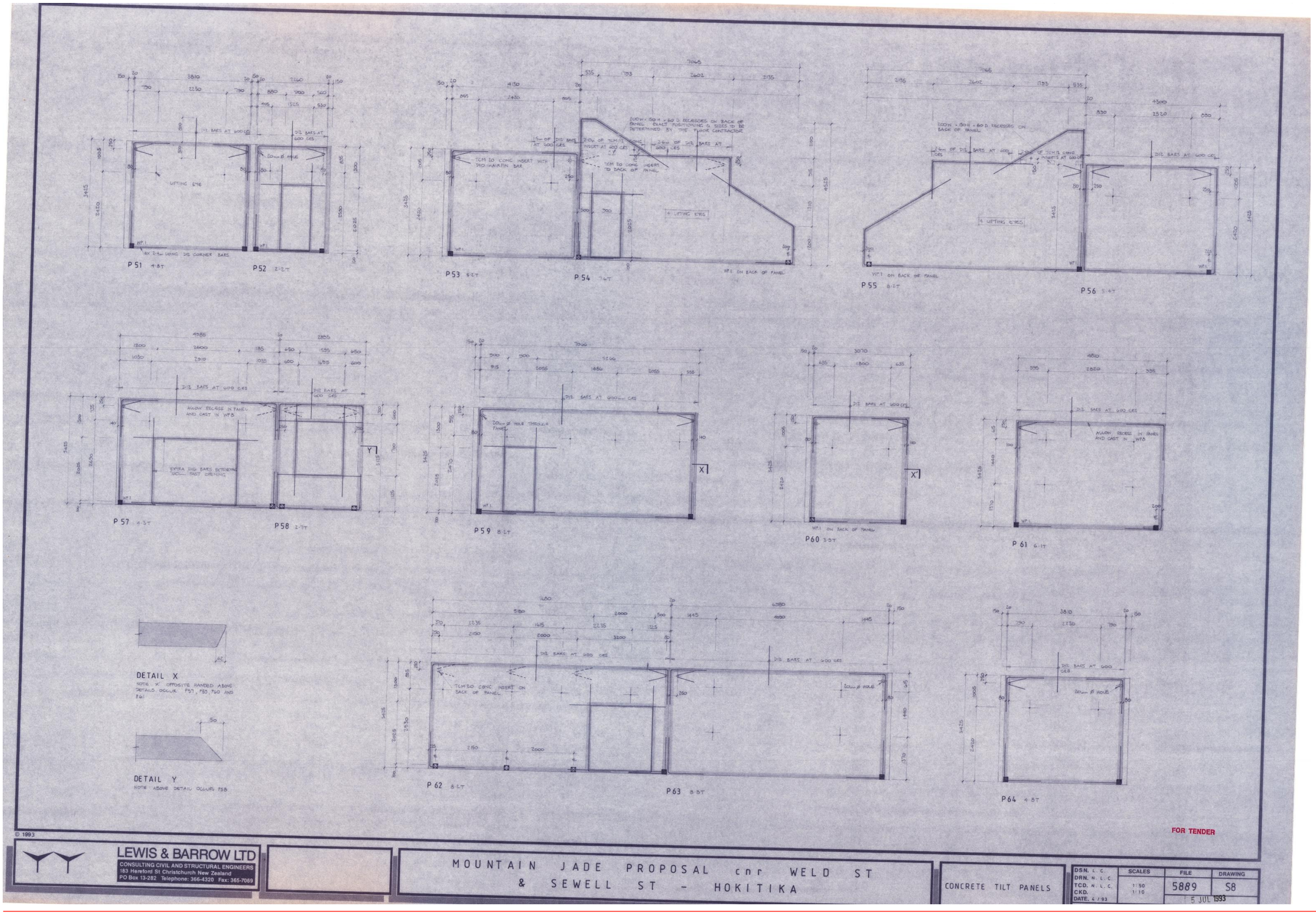
MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST – HOKITIKA

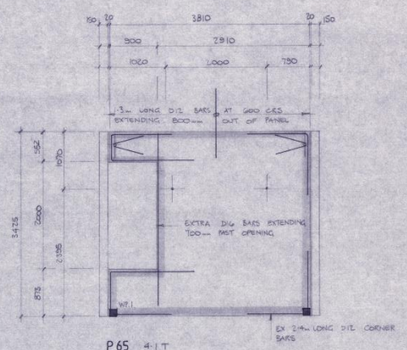
CONCRETE TILT PANELS

	SCALES	FILE	DRAWING
DSN. L. C. DRN. N. L. C. TCD. N. L. C. CKD. DATE. 4 / 93	1:50 1:10	5889	S6
		5 JUL 1993	

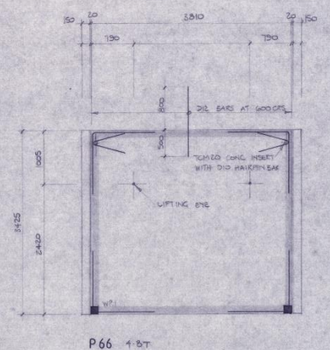
Contractor shall verify all dimensions before starting work


P 37 1ST
P 38 3.2T
P 39 8.0T
P 40 3.4T
P 41 7.2T
P 42 7.5T
P 43 9.2T
P 44 4T
P 45 4.3T
P 46 10.1T
P 47 4.1T
P 48 4.3T
P 49 4.8T
P 50 4.8T
DETAIL X
NOTE 'X' OPPOSITE HANDED ABOVE DETAILS OCCUR P44 AND P45
FOR TENDER
© 1993
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069
MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST – HOKITIKA
CONCRETE TILT PANELS
DSN. L. C.
DRN. N. L. C.
TCD. N. L. C.
CKD.
DATE. 4 / 93
SCALES 1 : 50 1 : 10
FILE 5889
DRAWING S7
6 JUL 1993
Contractor shall verify all dimensions before starting work

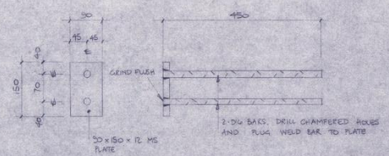

P51 4·8T
P52 2·2T
P53 5·2T
P54 7·6T
P55 8·2T
P56 5·4T
P57 4·3T
P58 2·7T
P59 8·2T
P60 3·3T
P61 6·1T
P62 8·2T
P63 8·8T
P64 4·8T
DETAIL X
DETAIL Y
FOR TENDER
© 1993
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069
MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST - HOKITIKA
CONCRETE TILT PANELS
DSN. L.C.
DRN. N.L.C.
TCD. N.L.C.
CKD.
DATE. 4/93
SCALES 1:50 1:10
FILE 5889
DRAWING S8
5 JUL 1993



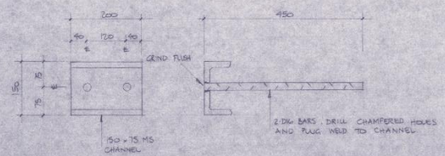
P65 4·1T



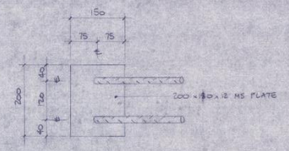
P66 4·8T

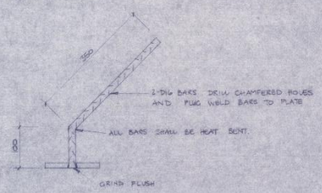


WELD PLATE 5



WELD PLATE 6
WELD PLATE 8 SIMILAR BUT 300mm LONG





WELD PLATE 7
WELD PLATE 9 SIMILAR BUT 300mm LONG

FOR TENDER

© 1993

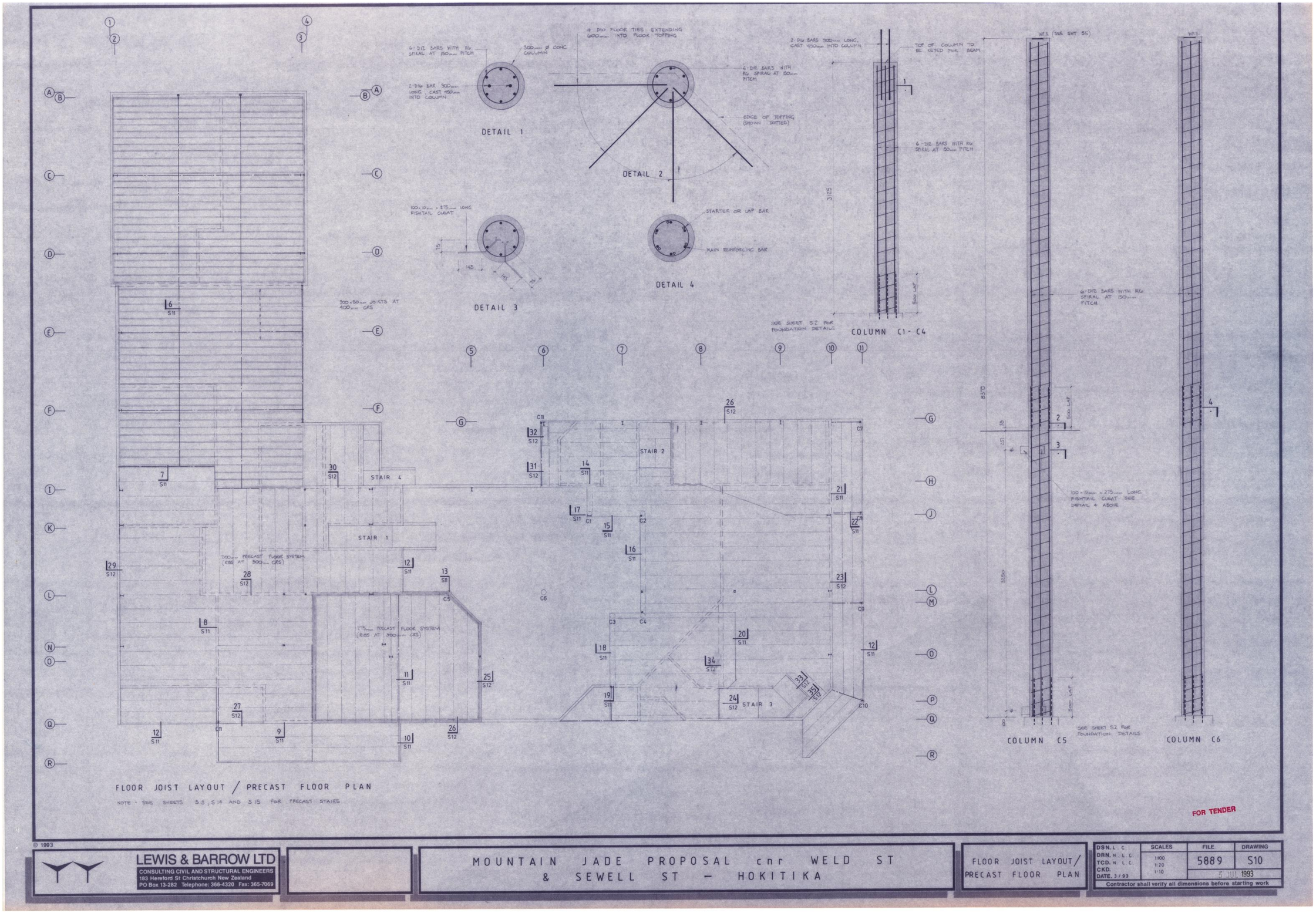
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069

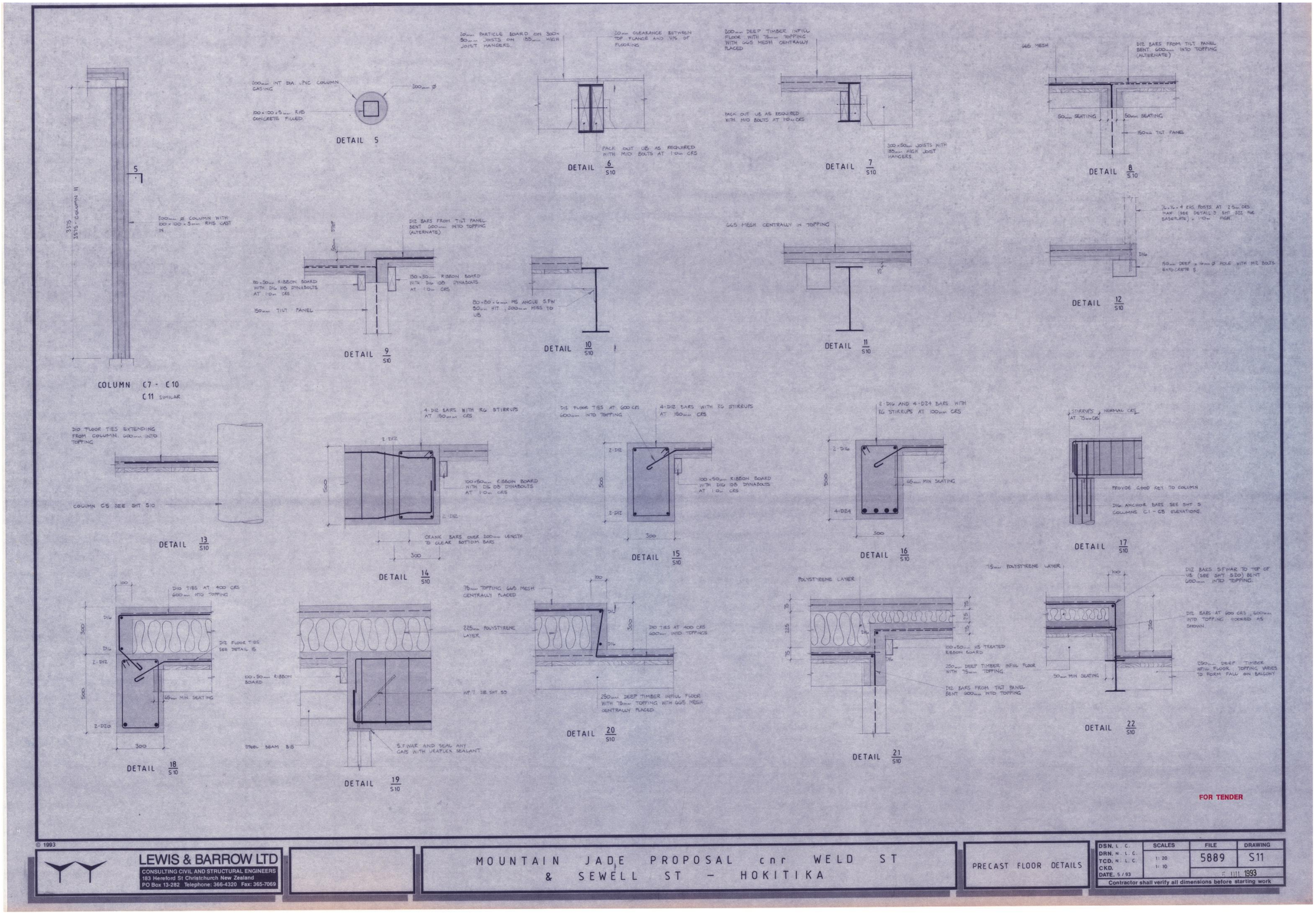
MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST – HOKITIKA

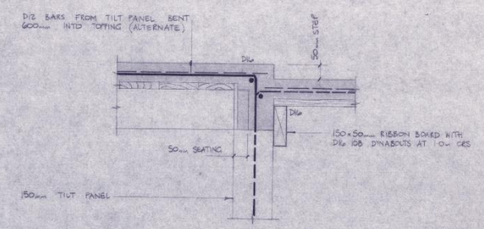
CONCRETE TILT PANELS

DSN. L. C.	SCALES	FILE	DRAWING
DRN. N. L. C.		5889	S9
TCD. N. L. C.	1:50 1:7·5		
CKD.		5 JUL 1993	
DATE. 4/93			

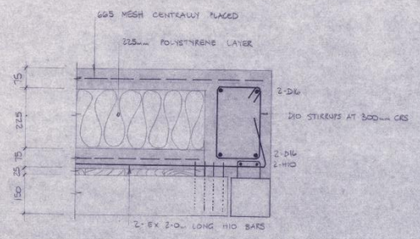
Contractor shall verify all dimensions before starting work


6 DIA BARS WITH RG SPIRAL AT 150mm PITCH
300mm Ø CONC COLUMN
2 DIA BAR 300mm LONG, CAST 450mm INTO COLUMN
DETAIL 1
4 DIA FLOOR TIES EXTENDING 600mm INTO FLOOR TOPPING
6 DIA BARS WITH RG SPIRAL AT 50mm PITCH
EDGE OF TOPPING (SHOWN DOTTED)
DETAIL 2
100 x 10mm x 275mm LONG FISHTAIL CLEAT
DETAIL 3
STARTER OR LAP BAR
MAIN REINFORCING BAR
DETAIL 4
2 DIA BARS 300mm LONG, CAST 450mm INTO COLUMN
TOP OF COLUMN TO BE KEYED INTO BEAM
6 DIA BARS WITH RG SPIRAL AT 50mm PITCH
3125
500 LAP
SEE SHEET S12 FOR FOUNDATION DETAILS
COLUMN C1 - C4
W.P.S. (SEE SHT S5)
6 DIA BARS WITH RG SPIRAL AT 150mm PITCH
100 x 10mm x 275mm LONG FISHTAIL CLEAT SEE DETAIL 4 ABOVE
8570
3250
SEE SHEET S12 FOR FOUNDATION DETAILS
COLUMN C5
COLUMN C6
300 x 50mm JOISTS AT 400mm CRS
200mm PRECAST FLOOR SYSTEM (RIBS AT 500mm CRS)
175mm PRECAST FLOOR SYSTEM (RIBS AT 300mm CRS)
STAIR 1
STAIR 2
STAIR 3
STAIR 4
FLOOR JOIST LAYOUT / PRECAST FLOOR PLAN
NOTE - SEE SHEETS S13, S14 AND S15 FOR PRECAST STAIRS
FOR TENDER
© 1993
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069
MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST - HOKITIKA
FLOOR JOIST LAYOUT/ PRECAST FLOOR PLAN
DSN. L. C.
DRN. N. L. C.
TCD. N. L. C.
CKD.
DATE. 3/93
SCALES 1:100 1:20 1:10
FILE 5889
DRAWING S10
5 JUL 1993
Contractor shall verify all dimensions before starting work

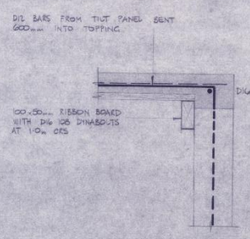

COLUMN C7 - C10
C11 SIMILAR
3375
3515 - COLUMN 11
5
800mm Ø COLUMN WITH 100 x 100 x 5mm RHS CAST IN.
DETAIL 5
200mm INT DIA. uPVC COLUMN CASING
200mm Ø
100x100x5mm RHS CONCRETE FILLED
DETAIL 6 / S10
20mm PARTICLE BOARD ON 300x50mm JOISTS ON 180mm HIGH JOIST HANGERS.
20mm CLEARANCE BETWEEN TOP FLANGE AND U/S OF FLOORING
PACK OUT UB AS REQUIRED WITH M10 BOLTS AT 1.0m CRS
DETAIL 7 / S10
200mm DEEP TIMBER INFILL FLOOR WITH 75mm TOPPING WITH GD5 MESH CENTRALLY PLACED
PACK OUT UB AS REQUIRED WITH M10 BOLTS AT 1.0m CRS
300x50mm JOISTS WITH 180mm HIGH JOIST HANGERS
DETAIL 8 / S10
GD5 MESH
D12 BARS FROM TILT PANEL BENT 600mm INTO TOPPING (ALTERNATE)
50mm SEATING
50mm SEATING
150mm TILT PANEL
DETAIL 9 / S10
STRIP
D12 BARS FROM TILT PANEL BENT 600mm INTO TOPPING (ALTERNATE)
100x50mm RIBBON BOARD WITH D16 105 DYNABOLTS AT 1.0m CRS
150mm TILT PANEL
150x50mm RIBBON BOARD WITH D16 105 DYNABOLTS AT 1.0m CRS
DETAIL 10 / S10
80x80x6mm MS ANGLE S.F.W 50mm FIT ; 200mm MISS TO UB
GD5 MESH CENTRALLY IN TOPPING
DETAIL 11 / S10
2x75x4 EAS POSTS AT 2.5m CRS MAX (SEE DETAIL 3 SHT S22 FOR BASEPLATE) x 1.1m HIGH
D16
150mm DEEP x 14mm Ø HOLE WITH M12 BOLTS EXPO CRETE S.
DETAIL 12 / S10
D10 FLOOR TIES EXTENDING FROM COLUMN. 600mm INTO TOPPING
COLUMN C5 SEE SHT S10
DETAIL 13 / S10
4-D12 BARS WITH R6 STIRRUPS AT 150mm CRS
2-D12
500
100x50mm RIBBON BOARD WITH D16 105 DYNABOLTS AT 1.0m CRS
2-D12
CRANK BARS OVER 200mm LENGTH TO CLEAR BOTTOM BARS
300
DETAIL 14 / S10
D12 FLOOR TIES AT 600 CRS 600mm INTO TOPPING
4-D12 BARS WITH R6 STIRRUPS AT 150mm CRS
2-D12
500
100x50mm RIBBON BOARD WITH D16 105 DYNABOLTS AT 1.0m CRS
2-D12
300
DETAIL 15 / S10
2-D16 AND 4-D24 BARS WITH R6 STIRRUPS AT 100mm CRS
2-D16
500
65mm MIN SEATING
4-D24
300
DETAIL 16 / S10
STIRRUPS AT NORMAL CRS AT 75mm CRS
PROVIDE GOOD KEY TO COLUMN
D16 ANCHOR BARS SEE SHT 5 COLUMNS C1 - C5 ELEVATIONS
DETAIL 17 / S10
D10 TIES AT 400 CRS 600mm INTO TOPPING
D16
D12 FLOOR TIES SEE DETAIL 15
2-D12
500
100x50mm RIBBON BOARD
65mm MIN SEATING
2-D20
300
DETAIL 18 / S10
100
75mm TOPPING, GD5 MESH CENTRALLY PLACED
225mm POLYSTIRENE LAYER
W.T. SEE SHT S5
Steel BEAM B/B
S.F.W AR AND SEAL ANY GAPS WITH WEATHER SEALANT
DETAIL 19 / S10
D10 TIES AT 400 CRS 600mm INTO TOPPING
D16
300
250mm DEEP TIMBER INFILL FLOOR WITH 75mm TOPPING WITH GD5 MESH CENTRALLY PLACED
DETAIL 20 / S10
POLYSTIRENE LAYER
15
225
15
100x50mm H5 TREATED RIBBON BOARD
250mm DEEP TIMBER INFILL FLOOR WITH 75mm TOPPING
D12 BARS FROM TILT PANEL BENT 600mm INTO TOPPING
DETAIL 21 / S10
75mm POLYSTIRENE LAYER
100
D12 BARS S.F.W AR TO TOP OF U/S (SEE SHT S.20) BENT 600mm INTO TOPPING.
D12 BARS AT 400 CRS 600mm INTO TOPPING HOOKED AS SHOWN
300
50mm MIN SEATING
250mm DEEP TIMBER INFILL FLOOR TOPPING VARIES TO FORM FALL ON BALCONY
DETAIL 22 / S10
FOR TENDER
© 1993
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069
MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST - HOKITIKA
PRECAST FLOOR DETAILS
DSN. L. C.
DRN. N. L. G.
TCD. N. L. C.
CKD.
DATE. 5 / 93
SCALES 1:20 1:10
FILE 5889
DRAWING S11
2 1111 1993
Contractor shall verify all dimensions before starting work



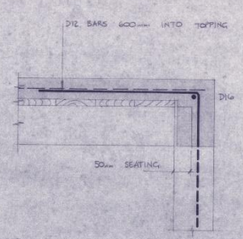
DETAIL 23 S10



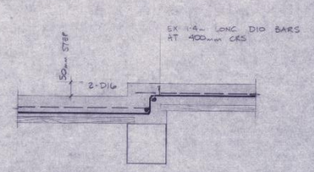
DETAIL 24 S10



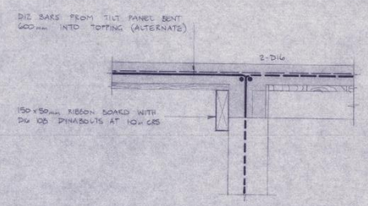
DETAIL 25 S10



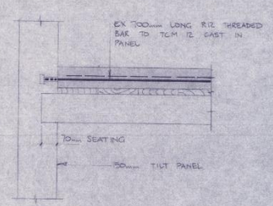
DETAIL 26 S10



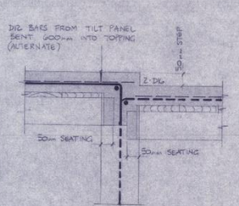
DETAIL 27 S10



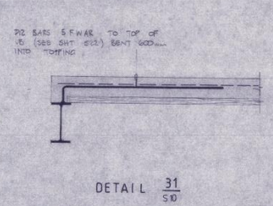
DETAIL 28 S10



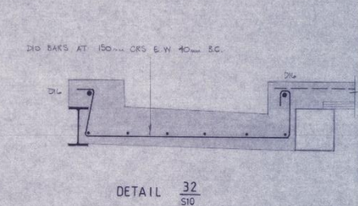
DETAIL 29 S10



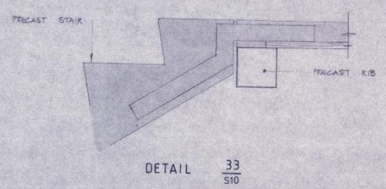
DETAIL 30 S10



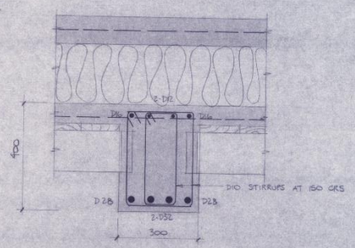
DETAIL 31 S10



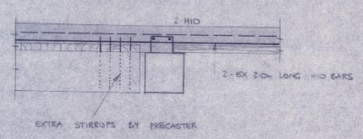
DETAIL 32 S10



DETAIL 33 S10



DETAIL 34 S10



DETAIL 35 S10

FOR TENDER

© 1993

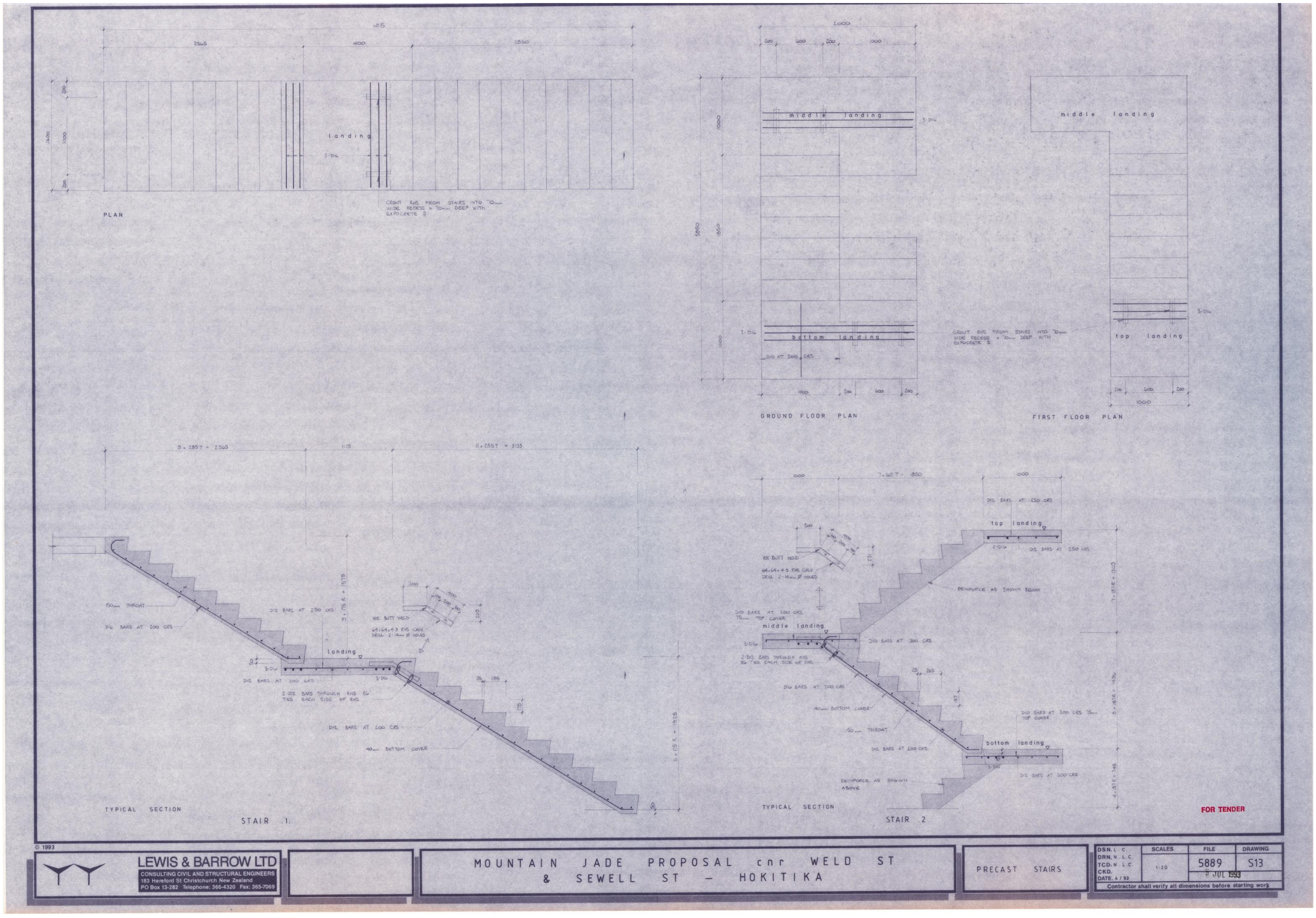
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069

MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST – HOKITIKA

PRECAST FLOOR DETAILS

DSN. L.C.	SCALES	FILE	DRAWING
DRN. N.L.C.	1:10	5889	S12
TCD. N.L.C.			
CKD.		- JUL 1993	
DATE. 6/93			

Contractor shall verify all dimensions before starting work



© 1993

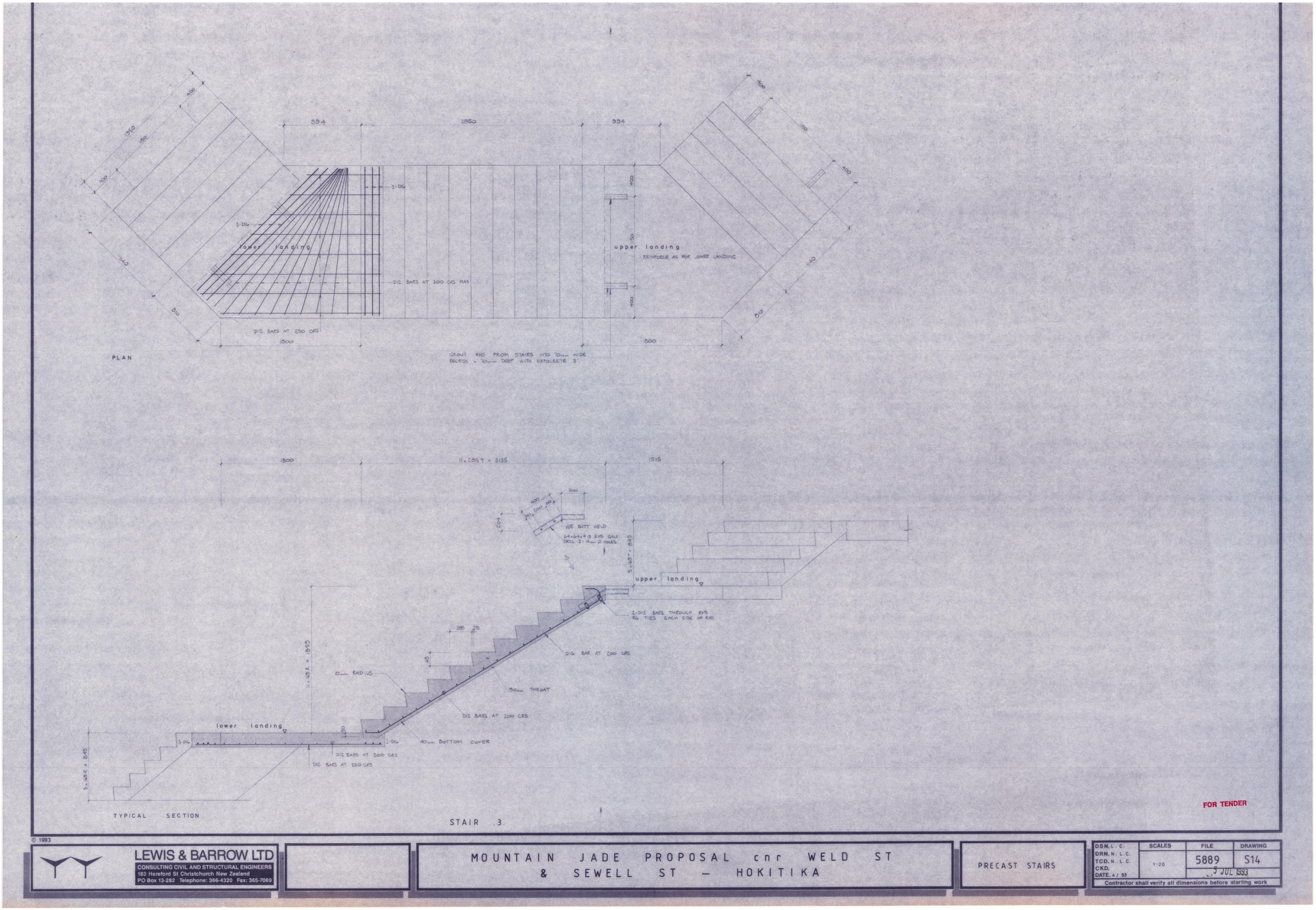
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069

MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST – HOKITIKA

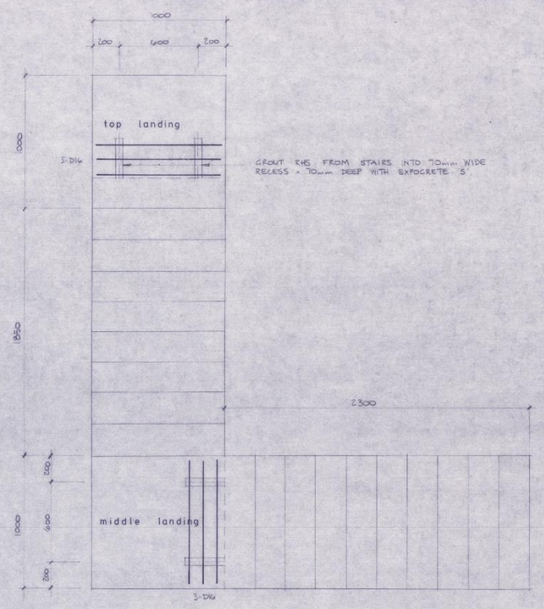
PRECAST STAIRS

DSN. L. C.	SCALES	FILE	DRAWING
DRN. N. L. C.	1:20	5889	S13
TCD. N. L. C.		8 JUL 1993	
CKD.			
DATE. 4 / 93			

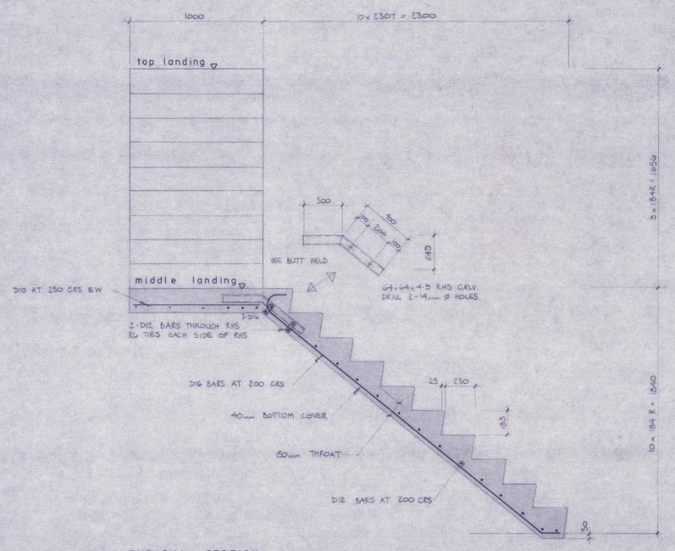
Contractor shall verify all dimensions before starting work



©WSP | 26 November 2019



PLAN



TYPICAL SECTION

STAIR .4.

FOR TENDER

© 1993

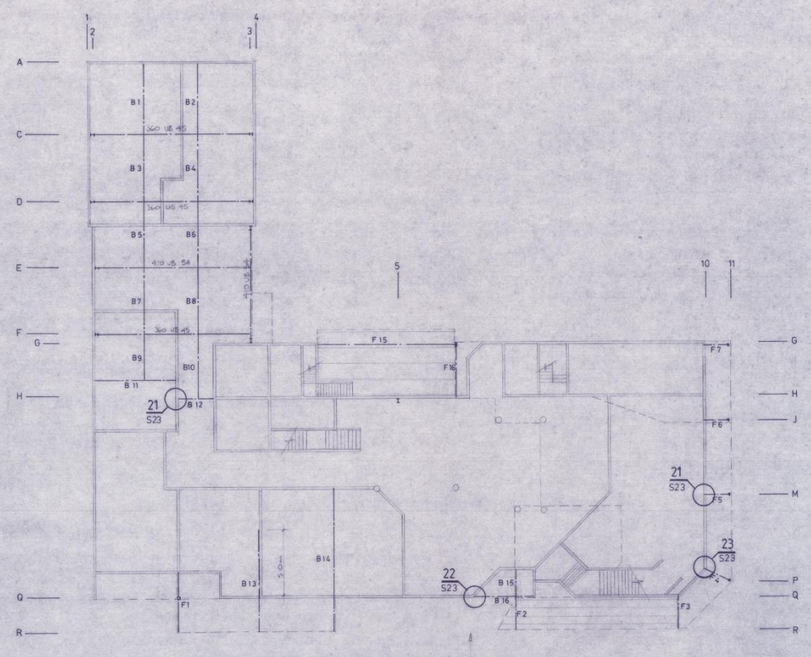
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069

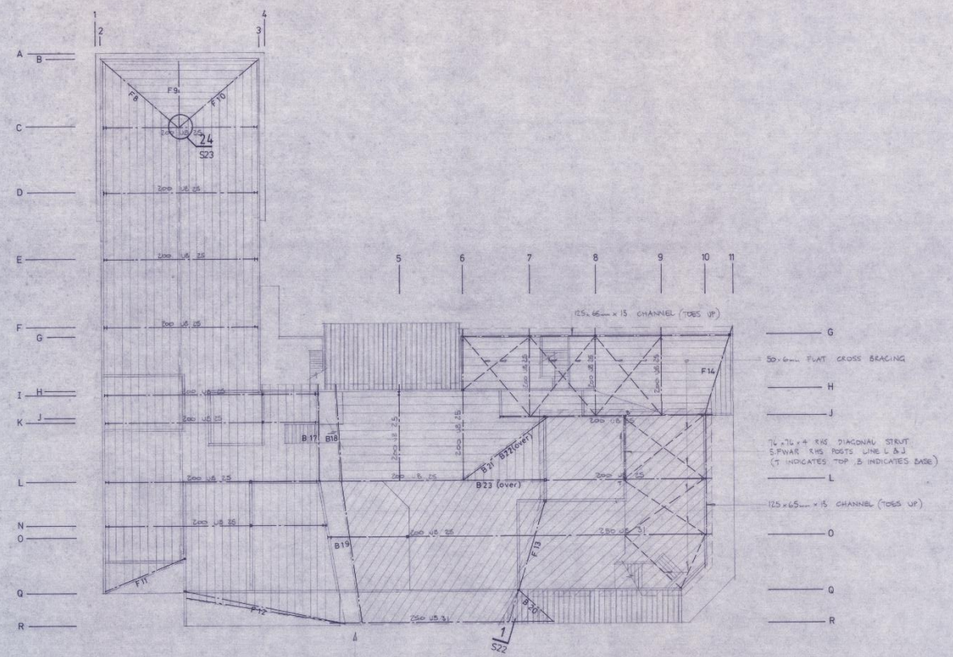
MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST - HOKITIKA

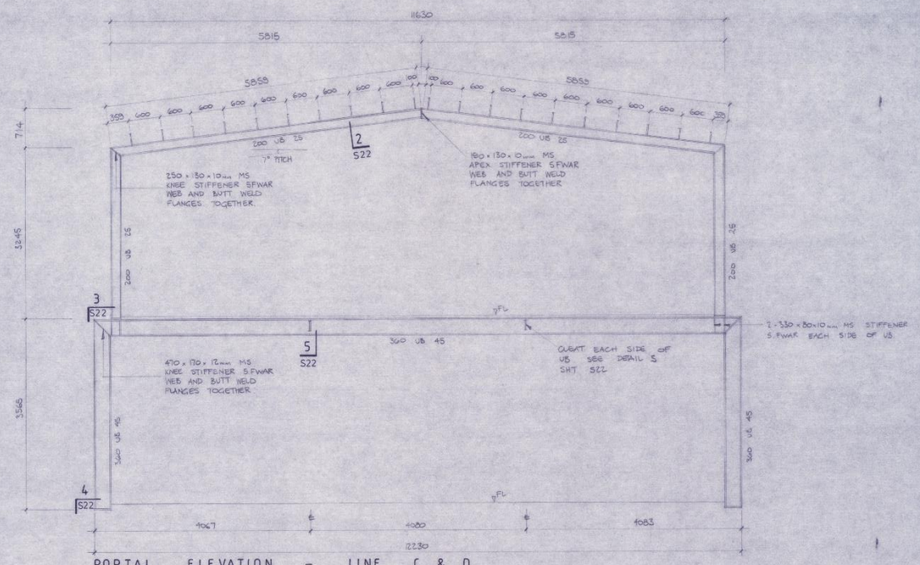
PRECAST STAIRS

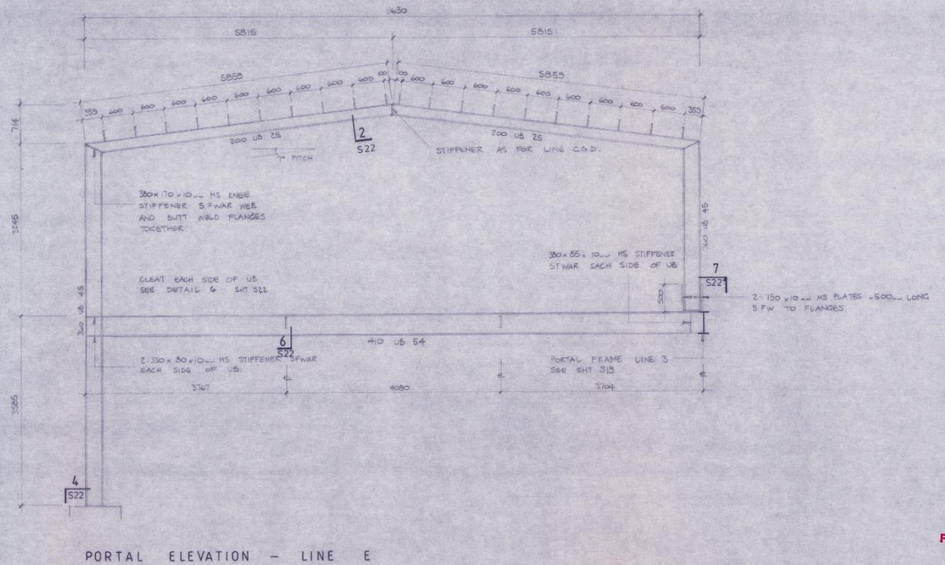
DSN. L.C.	SCALES	FILE	DRAWING
DRN. N.L.C.	1:20	5889	S15
TCD. N.L.C.			
CKD.		5 JUL 1993	
DATE. 6/93			

Contractor shall verify all dimensions before starting work









FOR TENDER

© 1993

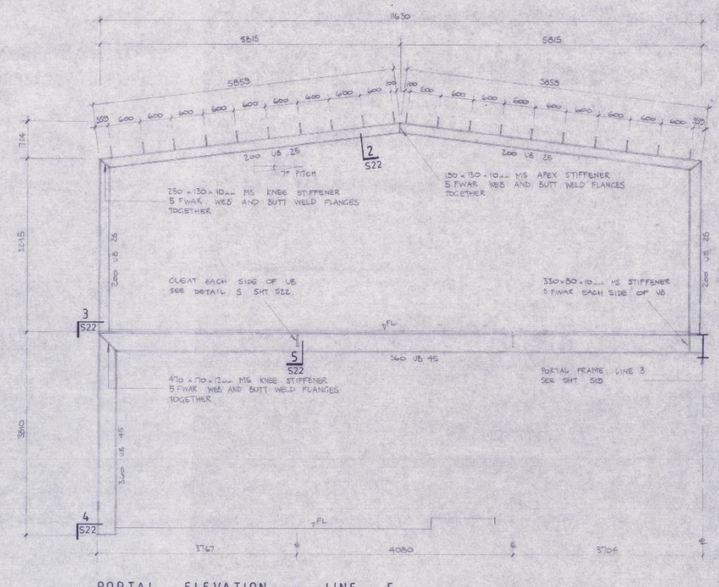
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069

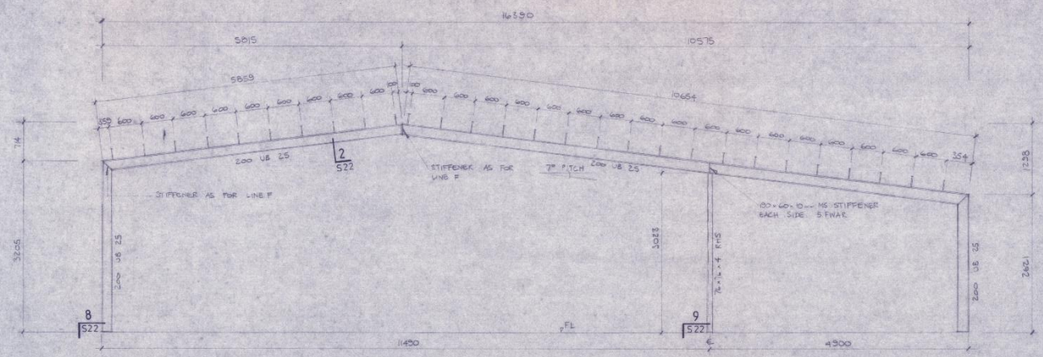
MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST – HOKITIKA

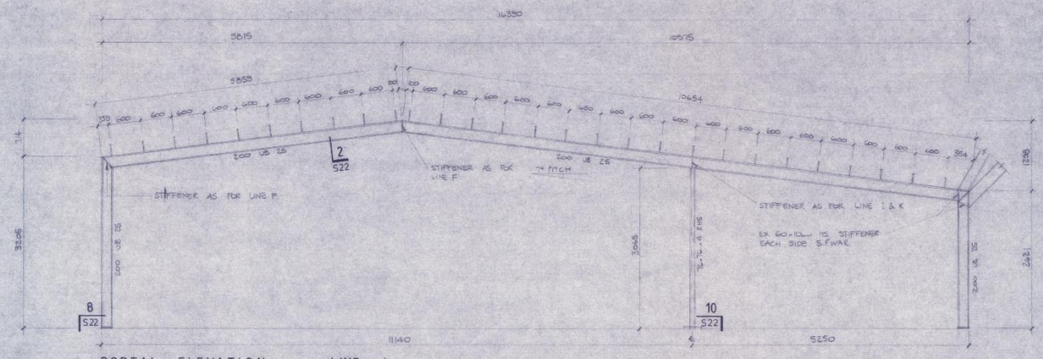
STEELWORK

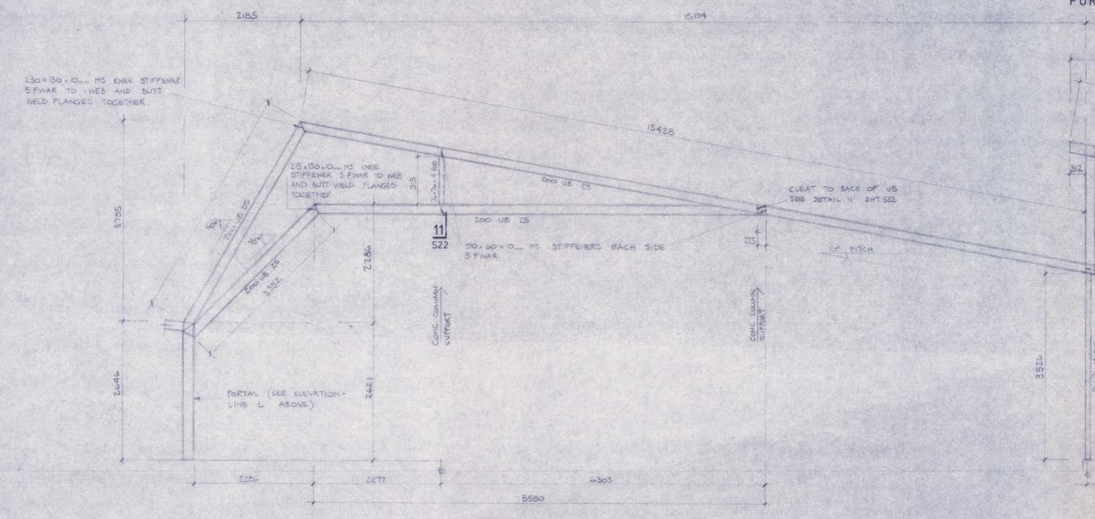
DSN. L. C.	SCALES	FILE	DRAWING
DRN. N. L. C.	1:200	5889	S16
TCD. N. L. C.	1:50		
CKD.		5 JUL 1993	
DATE. 3 / 93			

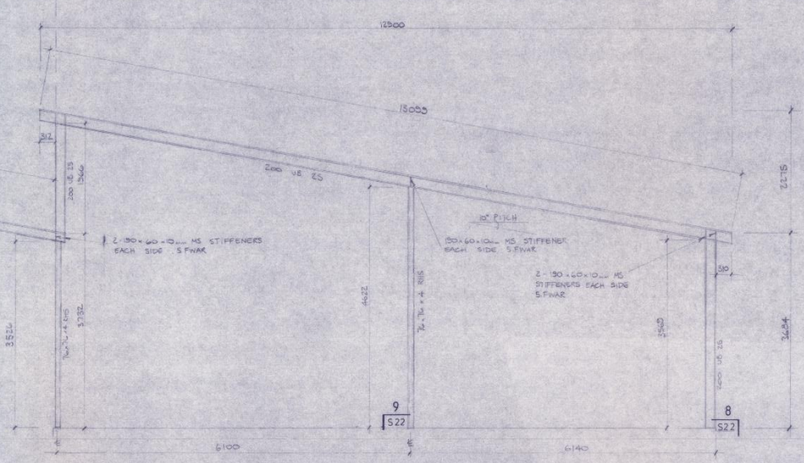
Contractor shall verify all dimensions before starting work











FOR TENDER

© 1993

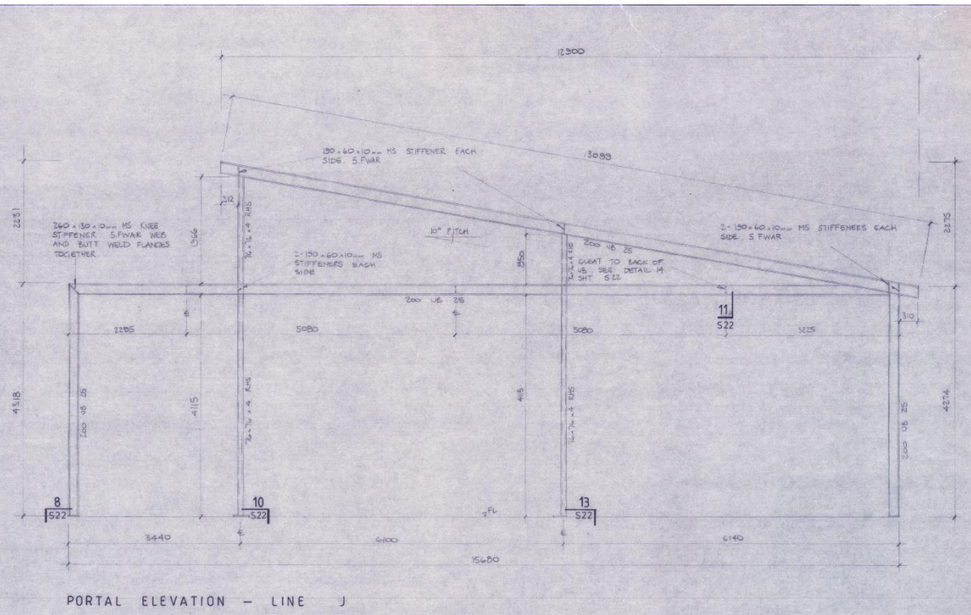
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069

MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST – HOKITIKA

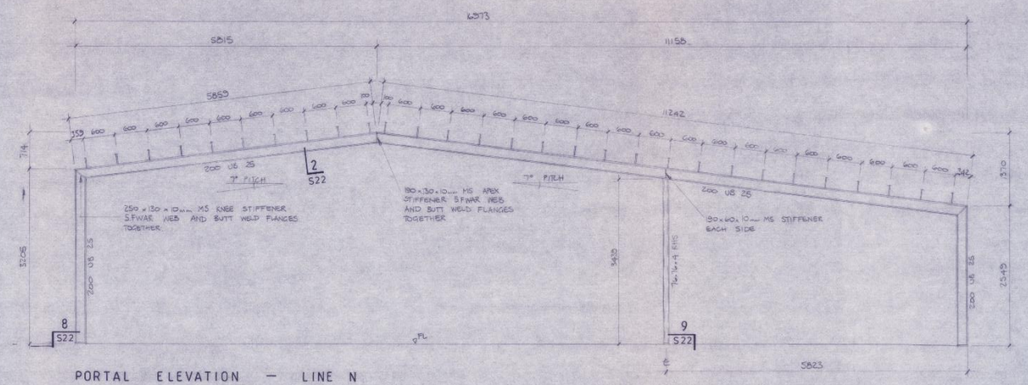
STEELWORK

DSN. L. C	SCALES	FILE	DRAWING
DRN. N. L. C.	1:50	5889	S17
TCD. N. L. C.			
CKD.		5 JUL 1993	
DATE. 3 / 93			

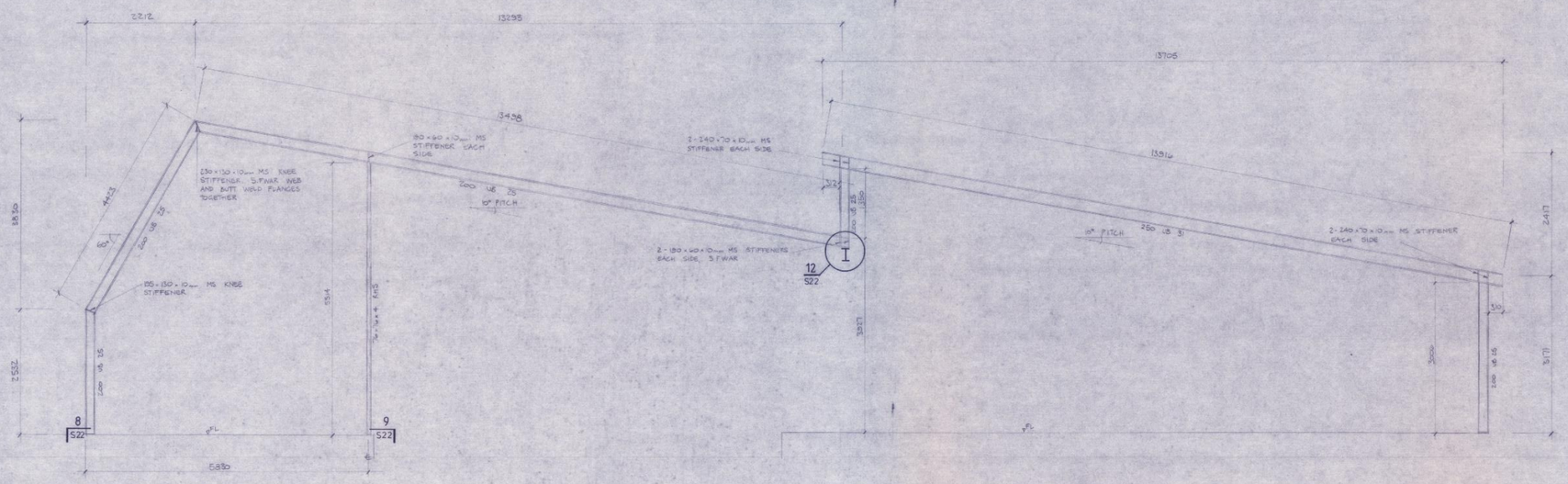
Contractor shall verify all dimensions before starting work



PORTAL ELEVATION — LINE J



PORTAL ELEVATION — LINE N



PORTAL ELEVATION — LINE O

FOR TENDER

© 1993

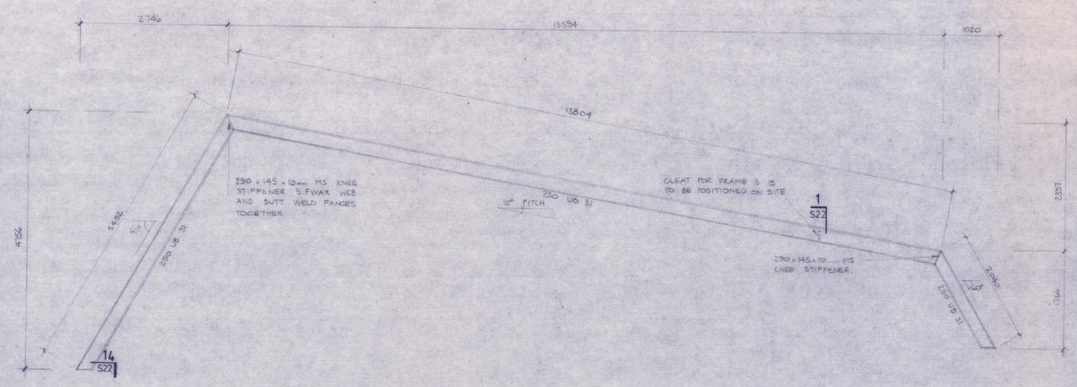
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069

MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST HOKITIKA

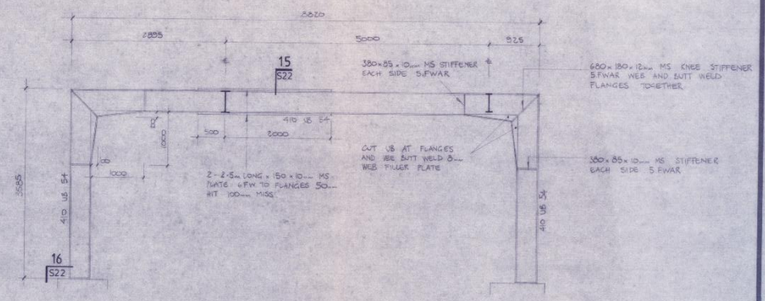
STEELWORK

DSN. L. C.	SCALES	FILE	DRAWING
DRN. N. L. C.	1:50	5889	S18
TCD. N. L. C.			
CKD.		5 JUL 1993	
DATE. 3 / 93			

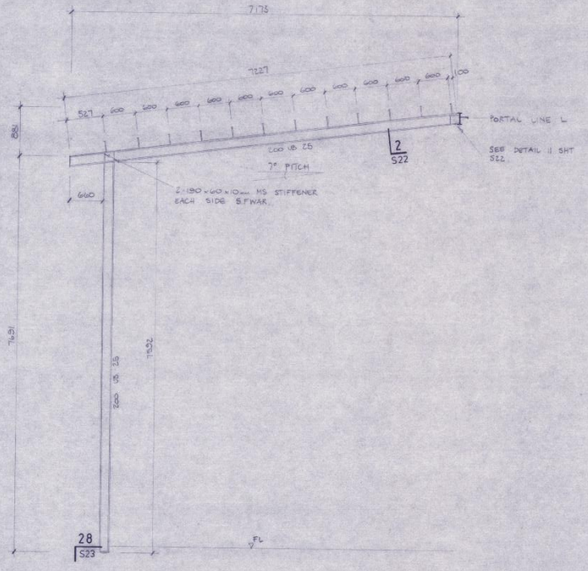
Contractor shall verify all dimensions before starting work.



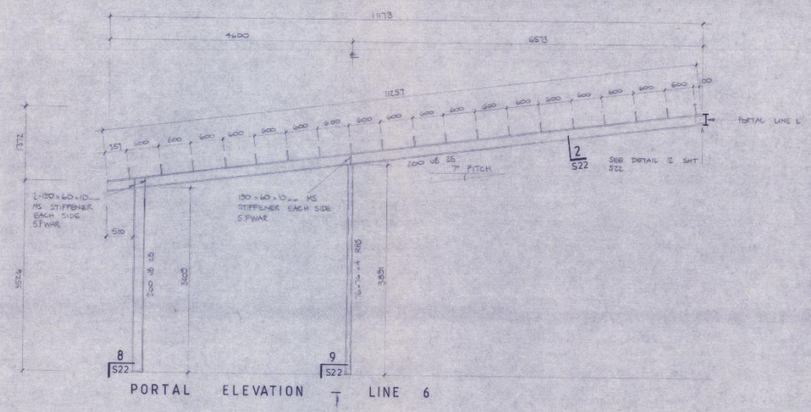
PORTAL ELEVATION — LINE R



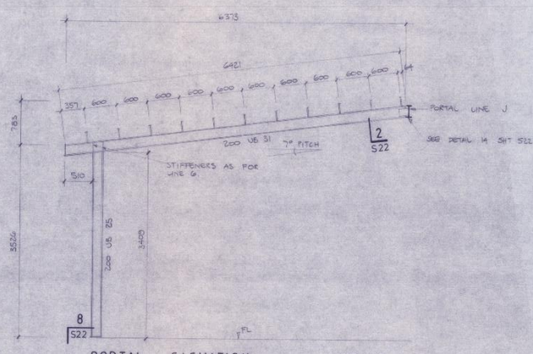
PORTAL ELEVATION — LINE 3



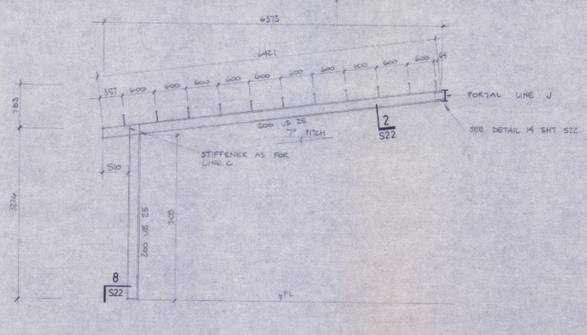
PORTAL ELEVATION — LINE 5



PORTAL ELEVATION — LINE 6



PORTAL ELEVATION — LINE 7



PORTAL ELEVATION — LINE 8 & 9

FOR TENDER

© 1993

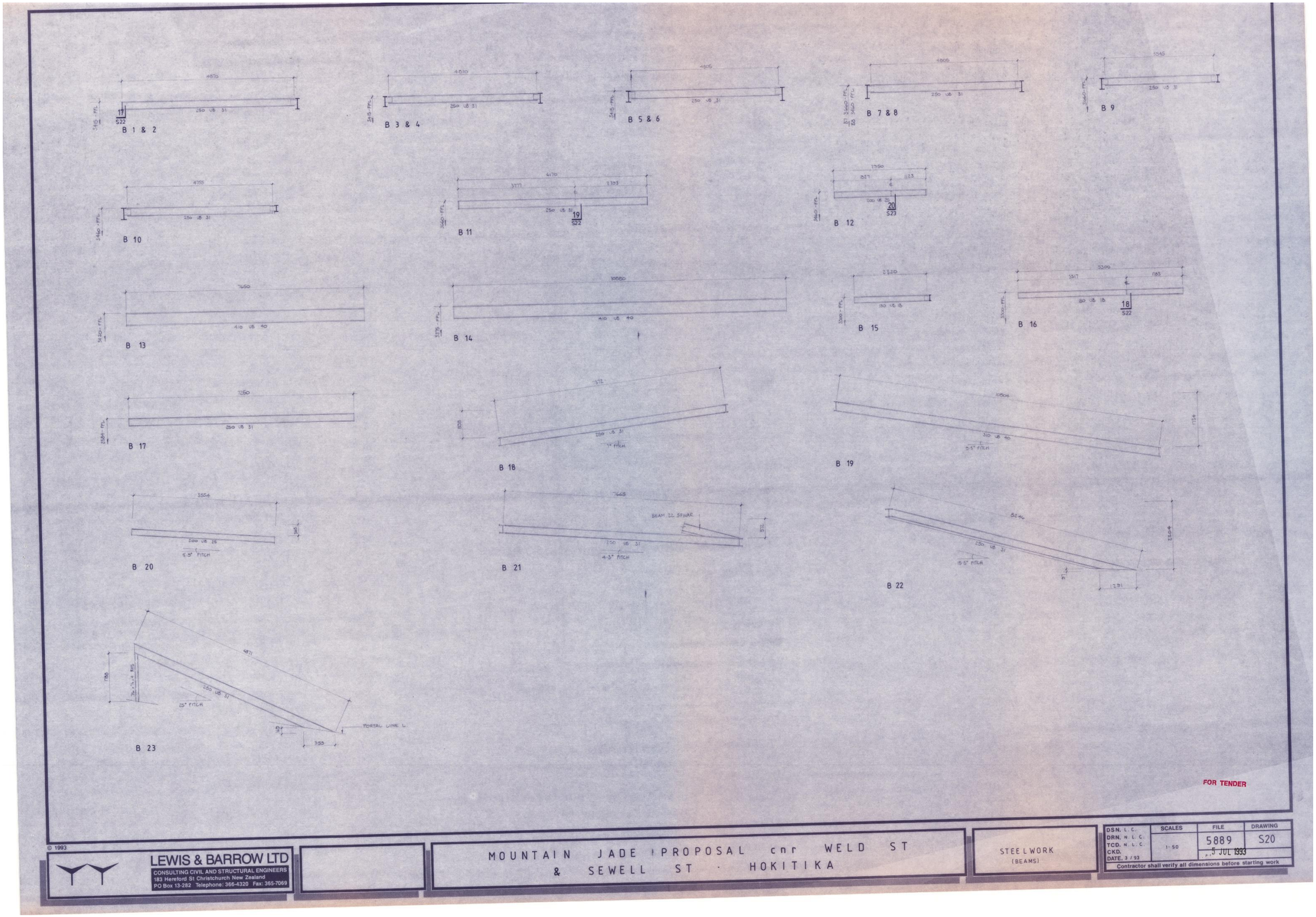
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069

MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST — HOKITIKA

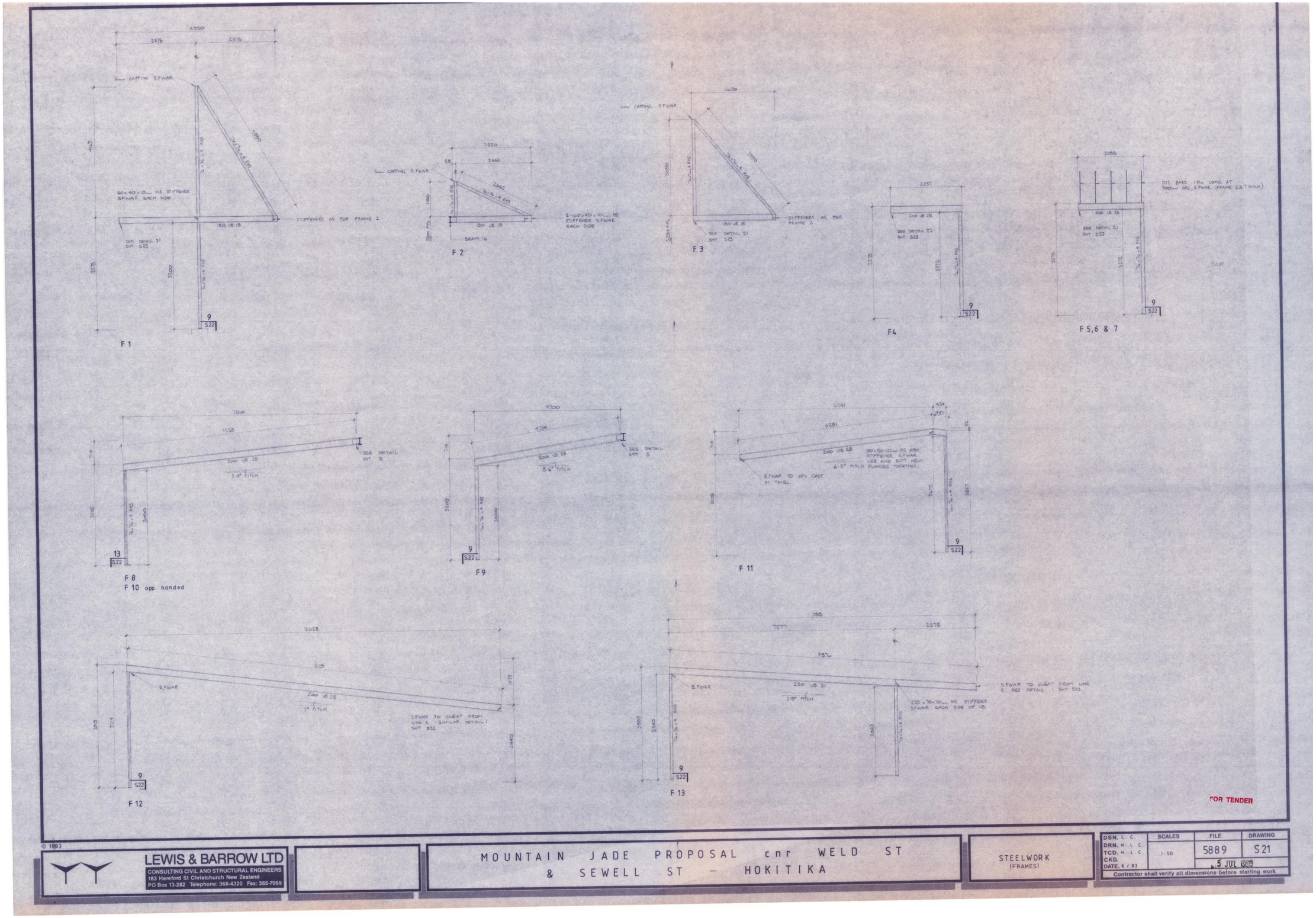
STEELWORK

DSN. L.C.	SCALES	FILE	DRAWING
DRN. N.L.C.	1:50	5889	S19
TCD. N.L.C.			
CKD.			
DATE. 3/93		5 JUL 1993	

Contractor shall verify all dimensions before starting work



©WSP | 26 November 2019



©WSP | 26 November 2019

F 14

F 15

F 16

DETAIL 1

DETAIL 2

DETAIL 3

DETAIL 4

DETAIL 5

DETAIL 6

DETAIL 7

DETAIL 8

DETAIL 9

DETAIL 10

DETAIL 11

DETAIL 12

DETAIL 13

DETAIL 14

DETAIL 15

DETAIL 16

DETAIL 17

DETAIL 18

DETAIL 19

FOR TENDER

© 1993

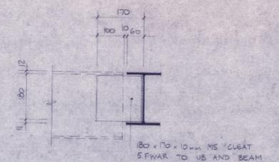
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069

MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST – HOKITIKA

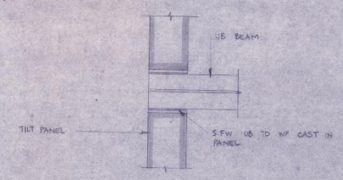
STEELWORK (FRAMES)

DSN. L. C.	SCALES	FILE	DRAWING
DRN. N. L. C.	1:50	5889	S22
TCD. N. L. C.	1:10		
CKD.		5 JUL 1993	
DATE. 6 / 93	Contractor shall verify all dimensions before starting work		

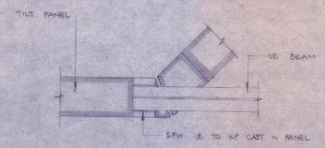
©WSP | 26 November 2019



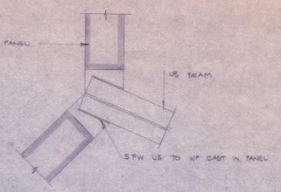
DETAIL 20 / S20



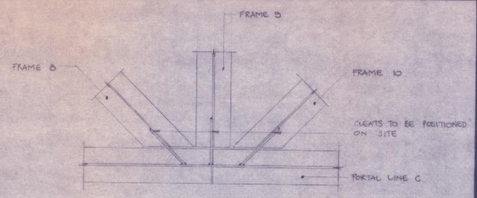
DETAIL 21 / S16



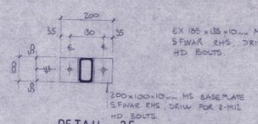
DETAIL 22 / S16



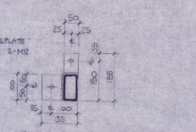
DETAIL 23 / S16



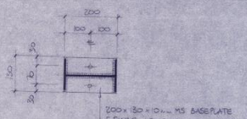
DETAIL 24 / S16



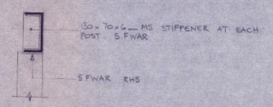
DETAIL 25



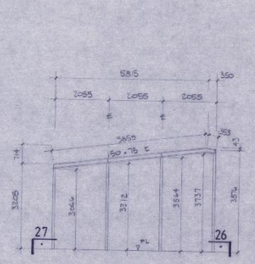
DETAIL 26
27 - OPPOSITE HANDED

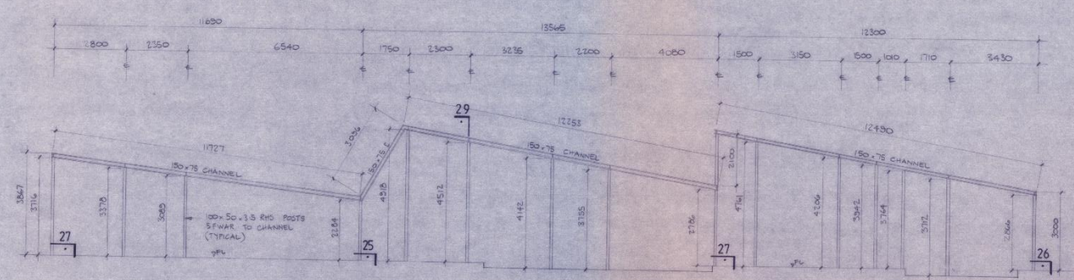


DETAIL 28 / S19



DETAIL 29





STRUCTURAL WINDOW FRAMES – NORTH ELEVATION

FOR TENDER

© 1993

LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069

MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST – HOKITIKA

STEELWORK

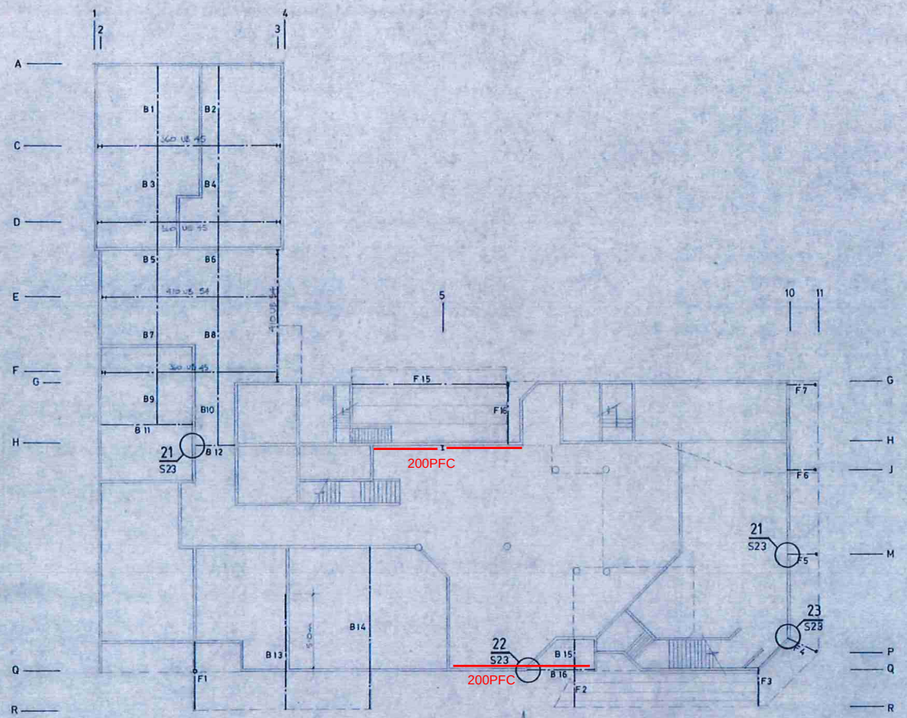
DSN. L. C.	SCALES	FILE	DRAWING
DRN. N. L. C.	1:100 1:10	5889	S23
TCD. N. L. C.			
CKD.			
DATE. 6/93		5 JUL 1993	

Contractor shall verify all dimensions before starting work

Appendix C

34%-67%NBS Strengthening Concept

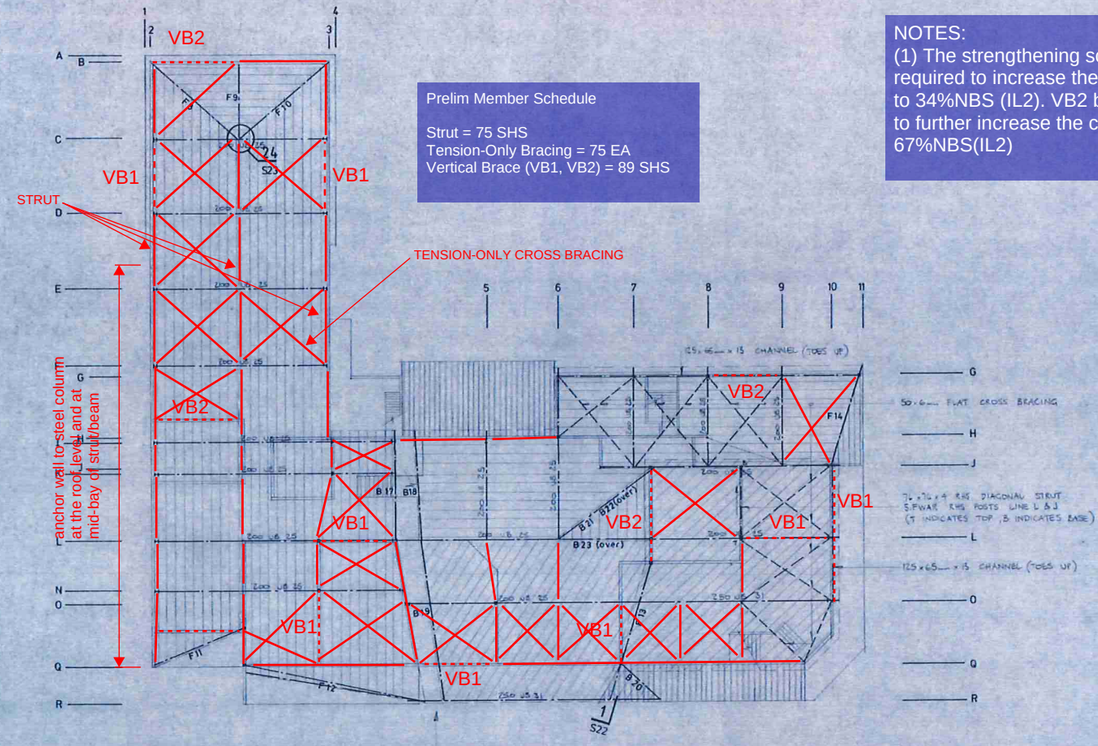
SK-1



FIRST FLOOR STEELWORK PLAN

BEAMS AND FRAMES ARE ELEVATED FROM SIDE DENOTED
BEAMS ELEVATED ON SHT S20
FRAMES ELEVATED ON SHT S21 AND S22

DIRECTION OF STEELWORK ELEVATIONS



Prelim Member Schedule

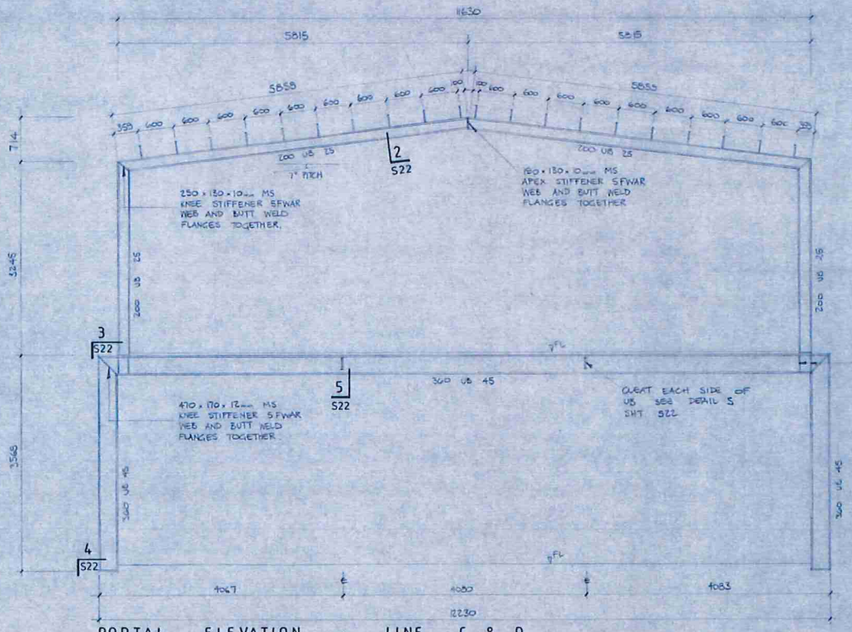
Strut = 75 SHS
Tension-Only Bracing = 75 EA
Vertical Brace (VB1, VB2) = 89 SHS

NOTES:
(1) The strengthening scope shown is required to increase the seismic capacity to 34%NBS (IL2). VB2 braces are required to further increase the capacity to 67%NBS(IL2)

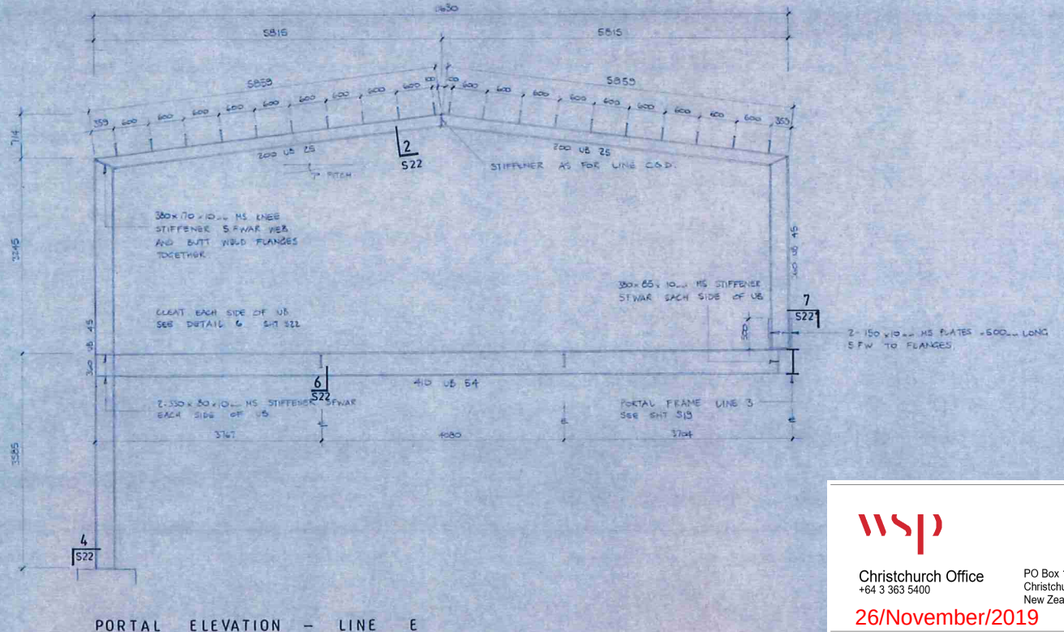
ROOF STEELWORK PLAN

BEAMS AND FRAMES ARE ELEVATED FROM SIDE DENOTED
BEAMS ELEVATED ON SHT S20
FRAMES ELEVATED ON SHT S21 AND S22

DIRECTION OF STEELWORK ELEVATIONS



PORTAL ELEVATION – LINE C & D



PORTAL ELEVATION – LINE E

wsp
Christchurch Office
+64 3 363 5400
PO Box 1482
Christchurch 8140
New Zealand

26/November/2019

© 1993

LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069

MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST – HOKITIKA

STEELWORK

DSN. I. C.	SCALES	FILE	DRAWING
DRN. N. L. C.	1:200	5889	S16
TCD. N. L. C.	1:50		
CKD.		5 JUL 1993	
DATE. 3 / 93			

Contractor shall verify all dimensions before starting work

SK-02

200 mm RC wall thickening

75 EA Tension Bracing

M16 ANCHORS AT 800mm AT ATTACHED TO ALTERNATE JOIST

M16 ANCHORS AT 900mm AP WITH EXISTING INSERTS

Note:
(1) 200mm RC wall thickening and 75EA plan bracing are required for the 64%NBS strengthening

200PFC

200PFC

DETAIL 1

DETAIL 2

DETAIL 3

DETAIL 4

COLUMN C1 - C4

COLUMN C5

COLUMN C6

STAIR 1

STAIR 2

STAIR 3

STAIR 4

FLOOR JOIST LAYOUT / PRECAST FLOOR PLAN

NOTE - SEE SHEETS S13, S14 AND S15 FOR PRECAST STAIRS

wsp
Christchurch Office
+64 3 363 5400
PO Box 1482
Christchurch 8140
New Zealand
26/November/2019

© 1993

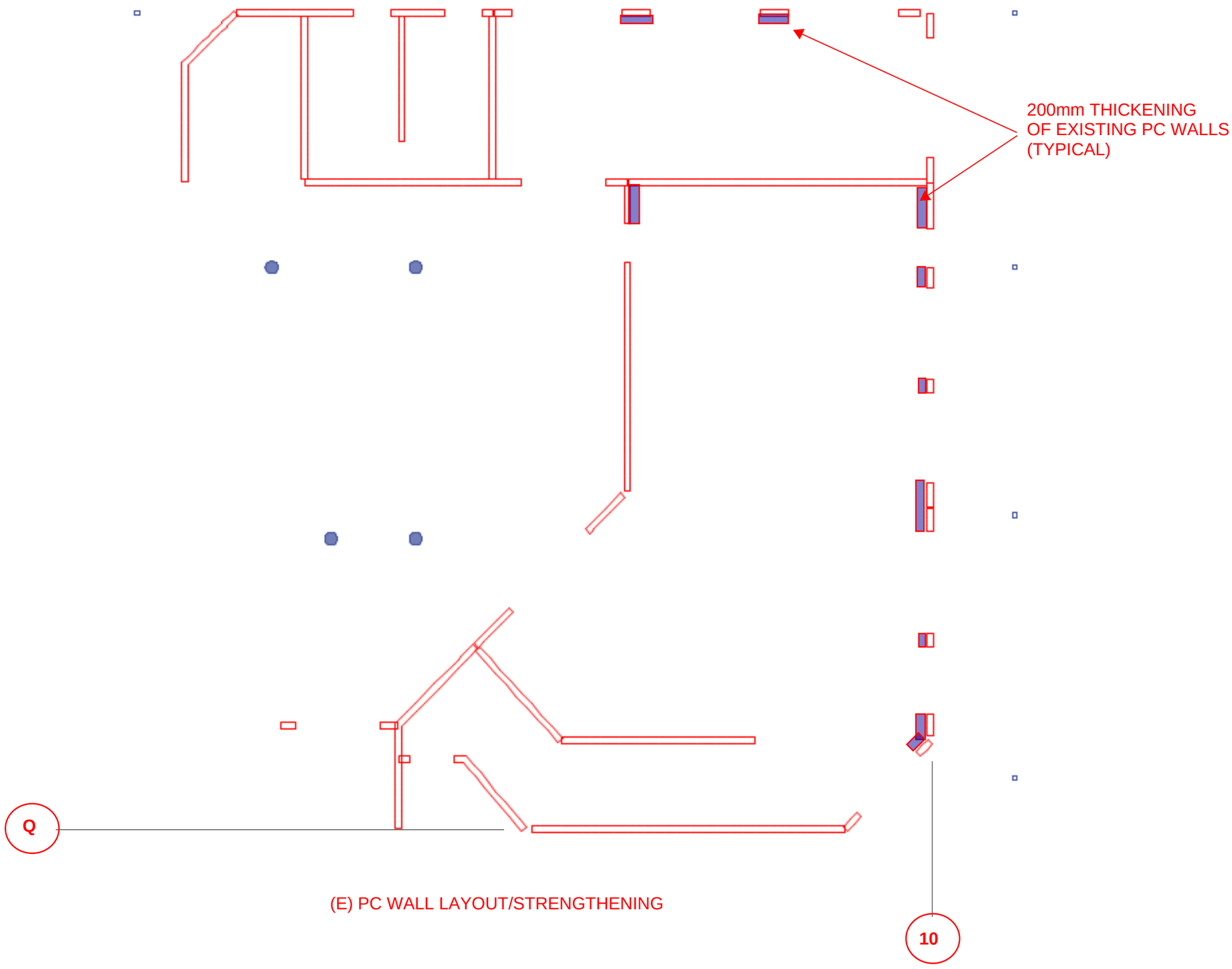
LEWIS & BARROW LTD
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
PO Box 13-282 Telephone: 366-4320 Fax: 365-7069

MOUNTAIN JADE PROPOSAL cnr WELD ST & SEWELL ST – HOKITIKA

FLOOR JOIST LAYOUT / PRECAST FLOOR PLAN

DSN. L. C.	SCALES	FILE	DRAWING
DRN. N. L. C.	1:100	5889	S10
TCD. N. L. C.	1:20		
CKD.	1:10	5 JUL 1993	
DATE. 3/93			

Contractor shall verify all dimensions before starting work



(E) PC WALL LAYOUT/STRENGTHENING

NOTES:

(1) This strengthening scope is required to increase the seismic capacity to 67%NBS.

(2) New Reinforcement to be adequately anchored to the foundation.

(3) Thickening to be on the interior face of the existing PC walls, includes pier and spandrels.

(4) Additional concrete thickness to be 200mm, UNO.

REVISION	AMENDMENT	APP	DATE

wsp
Christchurch Office
+64 3 363 5400
PO Box 1482
Christchurch 8140
New Zealand

26/November/2019

SCALE @ A1

ORIGINAL SIZE A1

DRAWN | DESIGNED | APPROVED

DRAWING VERIFIED | DESIGN VERIFIED | APPROVED DATE

PROJECT

TITLE

OPUS PROJECT NO. | SUITABILITY

PROJ-ORIG-VOL-LVL-TYPE | SHEET NO. SK-03 | REVISION

www.wsp.com/en-NZ