Detailed Seismic Assessment

SpencerHolmes
engineers - surveyors - planners

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Scottish Harriers Clubhouse
Salisbury Terrace
Newtown
Wellington

TECHNICAL SUMMARY

The following table summarises the results of the Detailed Seismic Assessment, (DSA), completed using Part C of the Seismic Assessment of Existing Buildings document. The overall report provides a detailed assessment of the building's seismic performance, relative to the New Building Standard, *(%NBS)* and highlights the key seismic risks and presents recommendations for improvements to mitigate these risks. The table below presents a summary of the technical inputs to and findings of the assessment.

1. Building Information	
Building Name/ Description	Scottish Harriers Clubhouse
Street Address	Salisbury Terrace, Newtown, Wellington
Territorial Authority	Wellington City Council
No. of Storeys	2
Area of Typical Floor (approx.)	300m^2 Ground Floor, 330m2 First Floor
Year of Design (approx.)	Originally built in 1970 and extended in 1978
NZ Standards designed to	NZSS1900 (1965) and MOW code of Practice (1968) for the original building. NZS4203:1976 for the extension
Structural System including Foundations	Light weight timber roofing and walls at roof level with a combination of reinforced concrete and lightweight timber framed walls on shallow footings in both directions
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	From GWRC GIS Maps Combined Hazard – Low-Mod Ground Shaking – Low Liquefaction – outside zones of identified liquefaction potential Slope Failure – Low
Previous strengthening and/ or significant alteration	N/A
Heritage Issues/ Status	N/A
Other Relevant Information	
2. Assessment Information	
Consulting Practice	Spencer Holmes Limited
CPEng Responsible, including: • Name • CPEng number • A statement of suitable skills and experience in the seismic assessment of existing buildings	Thomas Smith 1016003 Thomas has a background of civil and structural design for domestic, industrial and commercial projects throughout New Zealand and has worked as an Engineer in New Zealand since 2012. Thomas was made an Associate of Spencer Holmes Limited in 2020.
Documentation reviewed, including: • date/ version of drawings/ calculations • previous seismic assessments	Original Structure - DWG 405/1 to 405/6 Extension Structure - DWG W1-W5
Geotechnical Report(s)	N/A
Date(s) Building Inspected and extent of inspection	2023
Description of any structural testing undertaken and results summary	N/A

Previous Assessment Reports	N/A
Other Relevant Information	N/A

3. Summary of Engineering Assessment Methodology and Key Parameters Used	
Occupancy Type(s) and Importance Level	IL2 – Commercial
Site Subsoil Class	B
<u>For a DSA:</u>	
Summary of how Part C was applied, including: • the analysis methodology(s) used from C2 • other sections of Part C applied	C2.3 – Elastic Force Based Assessment C5 – Concrete Buildings C9 – Timber Buildings
Other Relevant Information	

4. Assessment Outcomes	
Assessment Status (Draft or Final)	Final
Assessed *%NBS* Rating	34% (Seismic Grade C, Medium Risk, 5 – 10 times the risk of a new building on the site)
Seismic Grade and Relative Risk (from Table A3.1)	C
Comment on the nature of Secondary Structural and Non-structural elements/ parts identified and assessed	
Describe the Governing Critical Structural Weakness	

Structural Assessment Outcome 1 (Governs)	
Assessed Seismic Rating	34% (Seismic Grade C)
Element / Mode of Failure	Shear capacity timber framed walls
Options for Improvement	1. Reline walls and add steel frames as necessary
Structural Assessment Outcome 1 (Governs)	
Assessed Seismic Rating	43% (Seismic Grade C)
Element / Mode of Failure	Flexural capacity of the concrete walls out of plane
Options for Improvement	1. Add sprayed shear walls and larger foundations to existing walls

Brief

Spencer Holmes Limited has been commissioned by Wellington Scottish Athletics Club to undertake a Detailed Seismic Assessment, (DSA), of the existing Wellington Scottish Harrier Clubhouse located as part of the Prince of Wales Park complex, off Salisbury Terrace, Newtown, WELLINGTON.

Limitation of Report

This report has been prepared for the use of the Wellington Scottish Athletics Club, and any reliance on this report by third parties, without the written consent of Spencer Holmes Limited shall be at that parties own risk.

This assessment and report is limited to the Wellington Scottish Harrier Clubhouse located as part of the Prince of Wales Park complex, off Salisbury Terrace, Newtown, WELLINGTON

The structural assessment is based on the original construction drawings of the building that have been obtained from the Wellington City Council archives as well as a limited visual inspection. Where structure is not able to be sighted, the structure shown on the drawings has been assumed to be as-constructed.

No destructive tests or geotechnical investigations have been undertaken, nor are any considered appropriate.

The assessment is limited to section B1 Structure of the New Zealand Building Code (NZBC) for seismic loading, and no assessment of the compliance requirements of other sections of the NZBC has been undertaken.

Building Description

The site at Prince of Wales Park, Newtown is a two level structure with a light weight timber framed roof and a combination of timber framed and reinforced concrete walls. The building is 18m long and 13m wide and for the purpose of the assessment has been considered to be orientated longitudinally, east - west and transversely, north - south.

The building is a two-storey with the roof and floor are constructed with light weight timber framing. The building perimeter is predominantly reinforced concrete walls to the squash courts with lightweight timber framed walls to the ancillary portion of the building. The western end of the building are two squash courts with a reinforced concrete central wall. Jack frames sit on top of the concrete walls to support the roof framing. The squash courts have a suspended timber floor, whilst the rest of the building has a reinforced concrete slab on grade. The concrete walls are supported on shallow footings.

The perimeter reinforced concrete walls along each side and at the rear of the building retain some of the surrounding site.

Aerial Photo

Adjacent Structures

The building is well separated from neighbouring buildings, such that pounding is of no structural concern.

Wellington City Council Records

The following documentation associated with the buildings on the site being assessed is publicly available from Wellington City Council archives;

Consultant	Drawings
Graham Naish / Architect	Original Structure - DWG 405/1 to 405/6
	Extension Structure - DWG W1-W5

Original structural drawings of the building have been included and are contained in Appendix 1.

Geotechnical Desktop Investigation

The site address is Prince of Wales Park, Newtown, at the southern end of Salisbury Terrace on the City to Sea Walkway.

In accordance with the current design standard for earthquake actions, NZS 1170.5:2004;

- The (Seismic) Hazard Factor, (Z), for the building is 0.40, being located in Wellington.

The site has been located on the Wellington City Council District Plan map number 6 and this identifies that the site is located on the boundary of an identified Hazard (Ground Shaking or Fault Line) Area.

The Wellington Regional Council's Emergencies Hazards Maps Series (Earthquake) indicate;

- Fault Line Proximity – the site is located approximately 0.4km to the south east of the Wellington fault zone, which is a major fault requiring consideration of the "Near-fault factor" in accordance with NZS 1170.5: 2004 for buildings with periods of 1.5 seconds and above. This building's period is significantly below 1.5 seconds.

- Ground Shaking Hazard - the site is located in Zone 1 (low). Zone 1 being the least shaking, and Zone 5 has the greatest shaking of the five zone assessment.

- Liquefaction Potential - the site is located outside any liquefaction potential zone.

- Earthquake Induced Slope Failure – the site is located within Zone 1 (low) slope failure susceptibility zone, being the most favourable zone.

- Combined Hazard – the site is located within an interpolated "low-mod" hazard zone.

Structural System of Buildings

The timber framed roof and floor span transversely across the building and are supported by timber framing and are supported at mid span. The perimeter concrete and timber framed walls carry roof and floor loading, and are supported on shallow footings.

The foundations are shallow footings with a reinforced concrete slab on grade at the eastern end and the squash courts consist of timber flooring on timber piles.

The lateral load resisting system in both the transverse and longitudinal directions are the reinforced concrete shear walls, at both ground and first floor levels with roof loads transferred to in plane walls via timber framing.

Qualitative Structural Attributes

Positive attributes;

- Fully lined timber framed walls,
- Bracing at roof level, and
- Insitu reinforced concrete shear walls.

Negative attributes;

- Light weight timber framed roof and flooring,
- The irregular lateral load resisting systems in plan, and
- Inadequate reinforcement to the existing reinforced concrete walls.

Structural Assumptions for Detailed Seismic Assessment

The design has been assessed for compliance with AS/NZS 1170 "Structural Design Actions" as a means of compliance with the New Zealand Building Code section B1 Structure, and in particular NZS 1170.5: 2004 Part 5: "Earthquake Actions – New Zealand", the standard required by the Wellington City Council when assessing the strength of a building.

The building has been assessed using a comparison of our observations from site, and our review of the Wellington City Council archives documentation, to current design codes, as well as specific design assessment of key elements of the structure in the transverse and longitudinal directions.

The assessment has been undertaken in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidance documents "*The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*" dated July 2017.

The available drawings and information of the building provided specific details regarding the typical reinforcement details, however these were limited and it was assumed that the balance of the construction was built to a similar standard of detailing.

For this assessment the buildings material properties have been assumed using guidance from NZSEE "*Section C5 – Concrete Buildings*" and "*Section C9 – Timber Buildings*".

Seismic Loadings

We have undertaken a specific structural assessment of the building primary structure to the current seismic design loadings standard, NZS 1170.5:2004.

Item	**Parameter**	**Justification**
Importance Level	2	Normal commercial use
Annual Probability of Exceedance	1/500	Ultimate Limit State, Table 3.3
Soil Type	B	Rock
Hazard Factor, Z	0.40	Wellington
Period of Structure, T_1	<0.40 seconds	2-storey concrete wall structure

Structural Compliance

The structural compliance of the building was undertaken by comparison of the existing structure and the original design with current code requirements, with commentary provided on the form of construction that may contain significant flaws in the detailing.

Structural codes have changed and improved significantly, however the gravity and wind requirements, whilst becoming more specific, have not substantially changed in the intent since the earlier codes. Seismic requirements have become considerably more onerous as the engineering knowledge has improved, and this is described in more detail in the next sections.

The primary concern for this assessment of the buildings is seismic lateral loading given the age, weight and general construction of the building.

Earthquake Prone Building Legislation

The Building Act 2004 includes provisions whereby each Territorial Authorities are required to adopt a policy on dangerous, earthquake prone and insanitary buildings within its district within 18 months of the enactment of this section of the Building Act 2004, which occurred on 30 November 2004.

The Building (Earthquake-prone Buildings) Amendment Act 2016 came into effect on 1st July 2017. This Act contains significant changes to the previous system for identifying and remediating Earthquake-prone Buildings under the Building Act 2004. In accordance with this legislation an Earthquake-prone Building is defined as;

> *(1) A building or a part of a building is earthquake prone if, having regard to the condition of the building or part and to the ground on which the building is built, and because of the construction of the building or part,—*
>
> > *(a) the building or part will have its ultimate capacity exceeded in a moderate earthquake; and*
> >
> > *(b) if the building or part were to collapse, the collapse would be likely to cause—*
> >
> > > *(i) injury or death to persons in or near the building or on any other property; or*
> > > *(ii) damage to any other property.*
>
> *(2) Whether a building or a part of a building is earthquake prone is determined by the territorial authority in whose district the building is situated: see section 133AK.*
>
> *(3) For the purpose of subsection (1)(a), ultimate capacity and moderate earthquake have the meanings given to them by regulations*

The amended Building Regulations defines;

7 Earthquake-prone buildings: moderate earthquake defined

> *(1) For the purposes of section 133AB of the Act (meaning of earthquake-prone building), moderate earthquake means, in relation to a building, an earthquake that would generate shaking at the site of the building that is of the same duration as, but that is one-third as strong as, the earthquake shaking (determined by normal measures of acceleration, velocity, and displacement) that would be used to design a new building at that site if it were designed on the commencement date.*
>
> *(2) In this regulation, commencement date means the day on which section 133AB of the Act comes into force.*

For the purposes of illustrating how the new arrangements are intended to operate, a working technical interpretation of ultimate capacity (as at September 2016), and the proposed definition being consulted upon in the proposals for regulations, is:

> *Ultimate capacity means the building's probable capacity to withstand earthquake actions and maintain gravity load support calculated by reference to the building as a whole and its individual elements or parts*

The capacity of a building is able to be defined by analysis and using the compliance documents, however the collapse of a building is difficult to define and assess for any particular building.

To be considered Earthquake Prone, the Building Act definition requires a building as a whole or part to have its Ultimate Capacity exceeded in a moderate earthquake, (currently for buildings of normal use this is 33% of the earthquake determined to have a 1/500 annual probability of exceedance).

Seismic Assessment Results

Longitudinal direction (East-West);

- Ceiling diaphragm was assessed to achieve approximately 100*%NBS*,
- Timber framed walls have been assessed to be approximately 60*%NBS*,
- Reinforced concrete walls have been assessed in plane to be approximately 67*%NBS*,
- Reinforced concrete walls have been assessed out of plane to be approximately 43*%NBS*,
- Floor diaphragms to be approximately 77*%NBS*,
- Extension was assessed to achieve approximately 87*%NBS*.

The seismic performance of the existing Wellington Scottish Harriers building in longitudinal direction has been assessed as;

60% New Building Standard

Transverse direction (North-South);

- Ceiling diaphragm was assessed to achieve approximately 78*%NBS*,
- Timber framed walls have been assessed to be approximately 34*%NBS*,
- Reinforced concrete walls have been assessed in plane to be approximately 80*%NBS*,
- Reinforced concrete walls have been assessed out of plane to be approximately 43*%NBS*,
- Floor diaphragms to be approximately 85*%NBS*,
- Extension was assessed to achieve approximately 100*%NBS*.

The seismic performance of the existing Wellington Scottish Harriers building in longitudinal direction has been assessed as

***34% New Building Standard*.**

Building Strength and Relative Risk

The table below taken from the NZSEE Guidelines provides the basis of a proposed grading system for existing buildings, as one way of interpreting the *%NBS* building score along with broad descriptions of the corresponding life-safety risk. It can be seen that occupants in *Earthquake Prone* buildings, (less than 34*%NBS*), are exposed to more than 10 times the risk that they would be in a similar new building. For buildings that are potentially *Earthquake Risk* (less than 67*%NBS*), but not *Earthquake Prone*, the risk is at least 5 times greater than that of an equivalent new building.

Building Grade	*%NBS*	Approx. Risk Relative to a New Building	Risk Description
A+	>100	Less than 1	Low
A	80 to 100	1 to 2 times	Low

B	67 to 79	2 to 5 times	Low or Medium
C	34 to 66	5 to 10 times	Medium
D	20 to 33	10 to 25 times	High
E	<20	More than 25 times	Very High

The New Zealand Society for Earthquake Engineering (which provides authoritative advice to the legislation makers, and should be considered to represent the consensus view of New Zealand structural engineers) classifies a building achieving greater than 67*%NBS* as "Low Risk", and having "Acceptable (improvement may be desirable)" building structural performance.

Based on the NZSEE grading system and the % New Building Standard achieved, the existing Wellington Scottish Harrier Clubhouse located as part of the Prince of Wales Park complex, off Salisbury Terrace, Newtown, WELLINGTON building is assessed to be;

34% New Building Standard - Seismic Risk Grade C

which is classified as a "Medium" risk building having 5 to 10 times the risk of a new building.

The assessment undertaken on the existing building is higher than the 33*%NBS* threshold for an earthquake prone building and lower than the 67*%NBS* threshold for an earthquake risk building, meaning that the building would be classified as being an earthquake risk.

With reference to the NZSEE building classification and based on this assessment, **there is no legal requirement to strengthen the building**.

Summary

We have completed a detailed seismic assessment on the existing Wellington Scottish Harrier Clubhouse located as part of the Prince of Wales Park complex, off Salisbury Terrace, Newtown, WELLINGTON.

The building has been assessed as 34*%NBS*, and is governed by the shear capacity of the first floor timber framed walls in the transverse direction. The assessment undertaken on the building is higher than the 33% threshold for an earthquake prone building and less than the 67*%NBS* threshold for an earthquake risk building.

The NZSEE has developed a grading system for the seismic performance of buildings, the building equates to a Seismic Risk Grade C, which is classified as a "Medium" risk building having a risk of 5 to 10 times higher than a new building.

Report prepared by:
Spencer Holmes Limited

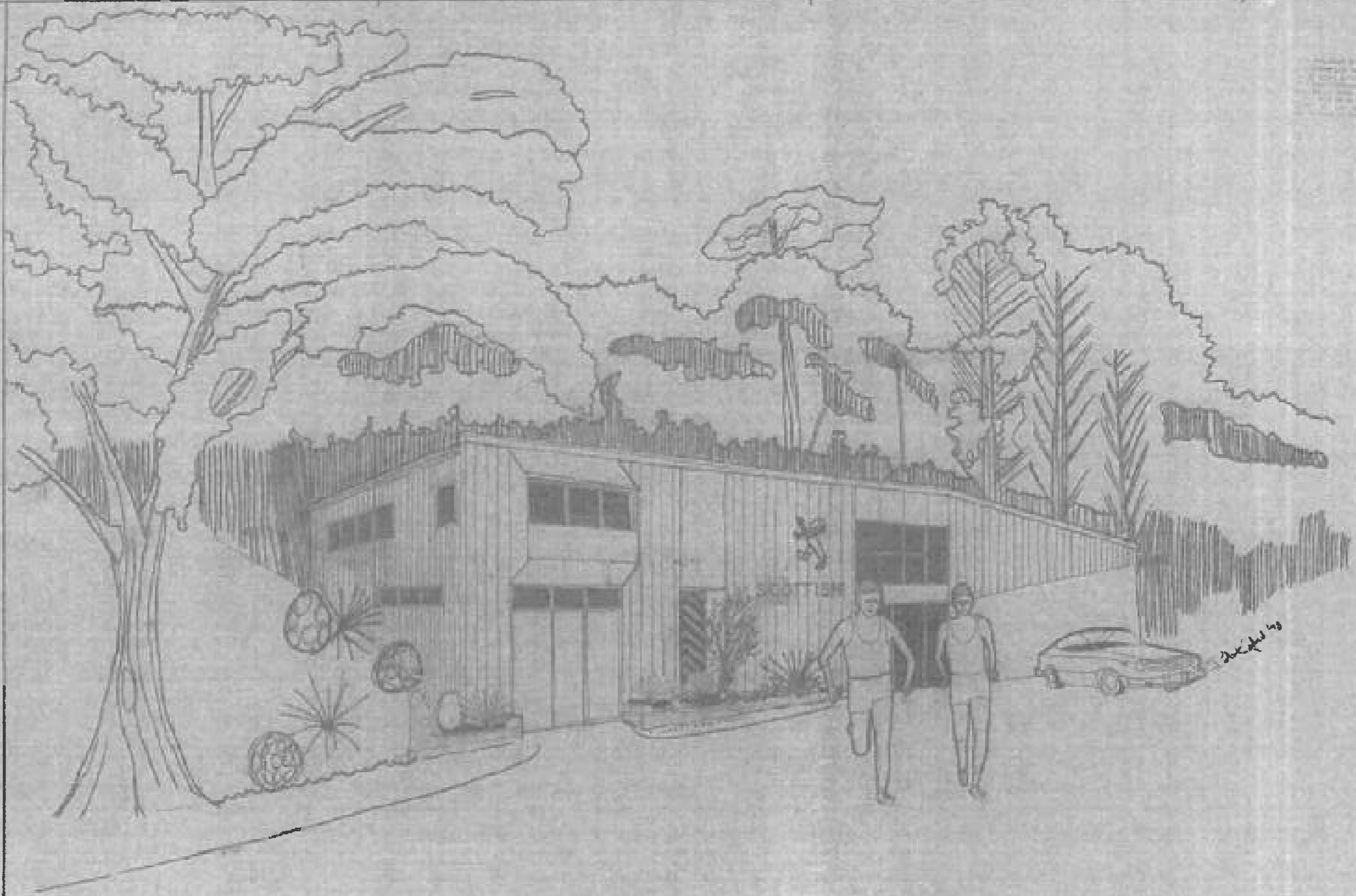
Thomas Smith
Associate
BE, CPEng, CMEngNZ, IntPE(NZ)

Report reviewed by:

Philip McConchie
Director

APPENDIX 1

Original Structural Drawings



SCOTTISH HARRIERS CLUBHOUSE

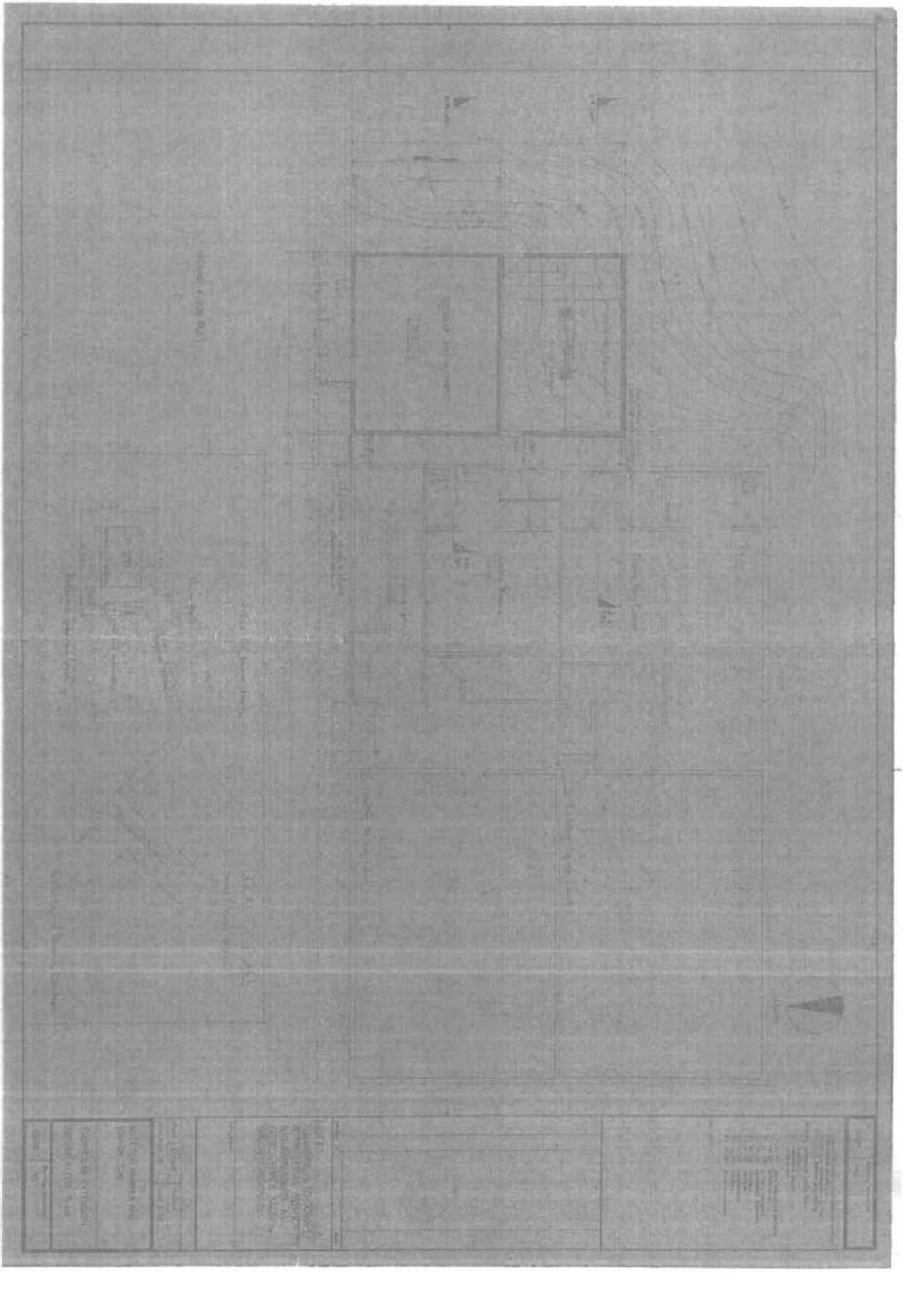
figured dimensions shall take precedence over scaled dimensions
report any discrepancy to the architect before proceeding

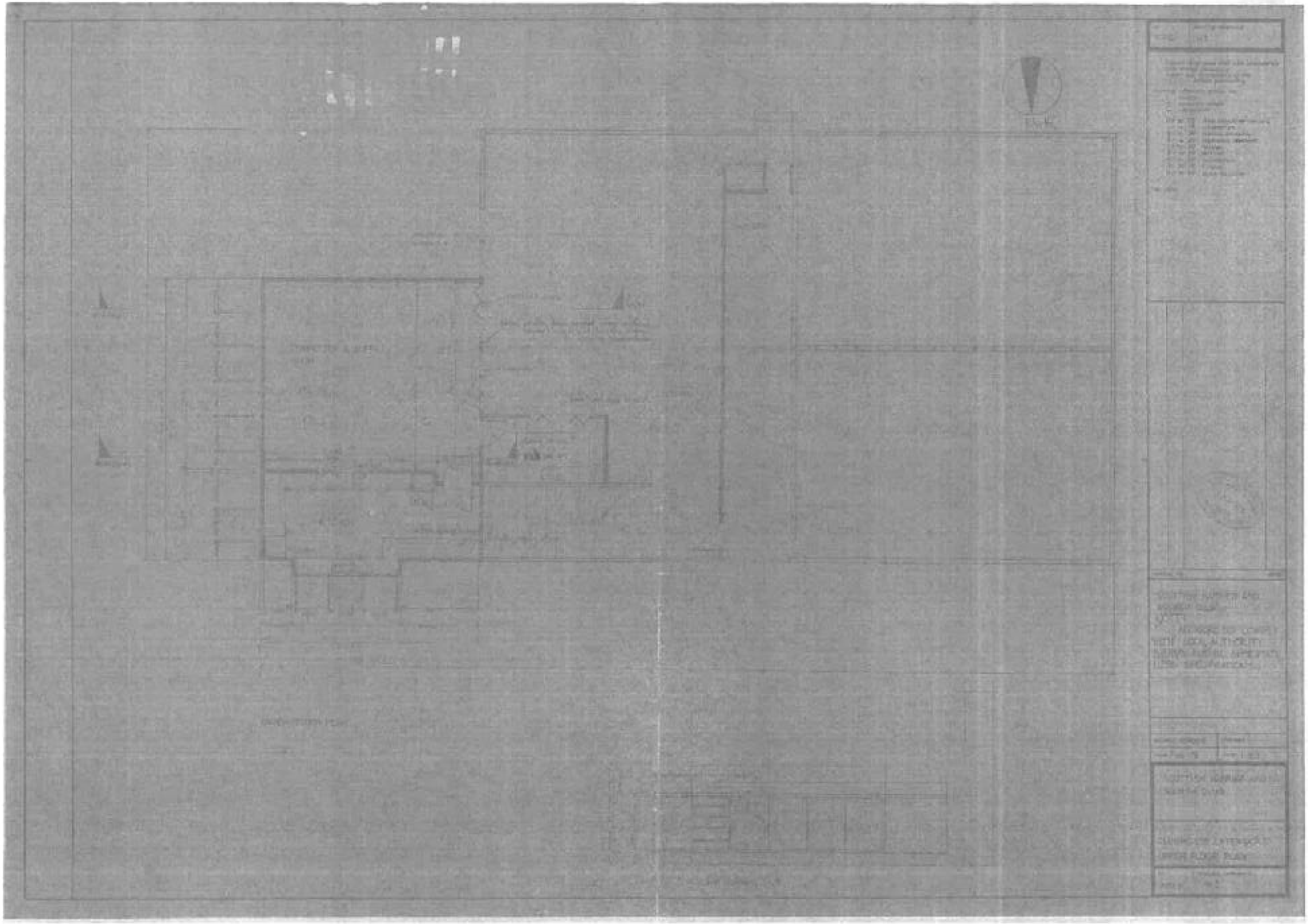
drawing reference prefix key
- (0) general
- (1) substructure
- (2) primary elements
- (21) external walls
- (22) internal walls
- (23) floors
- (24) stairs
- (27) roofs
- (28) frames
- (3) secondary elements
- (4) finishes
- (5) piped services
- (6) wired services
- (7) fittings
- (9) siteworks

key plan

consultants

drawn	checked
date	scale





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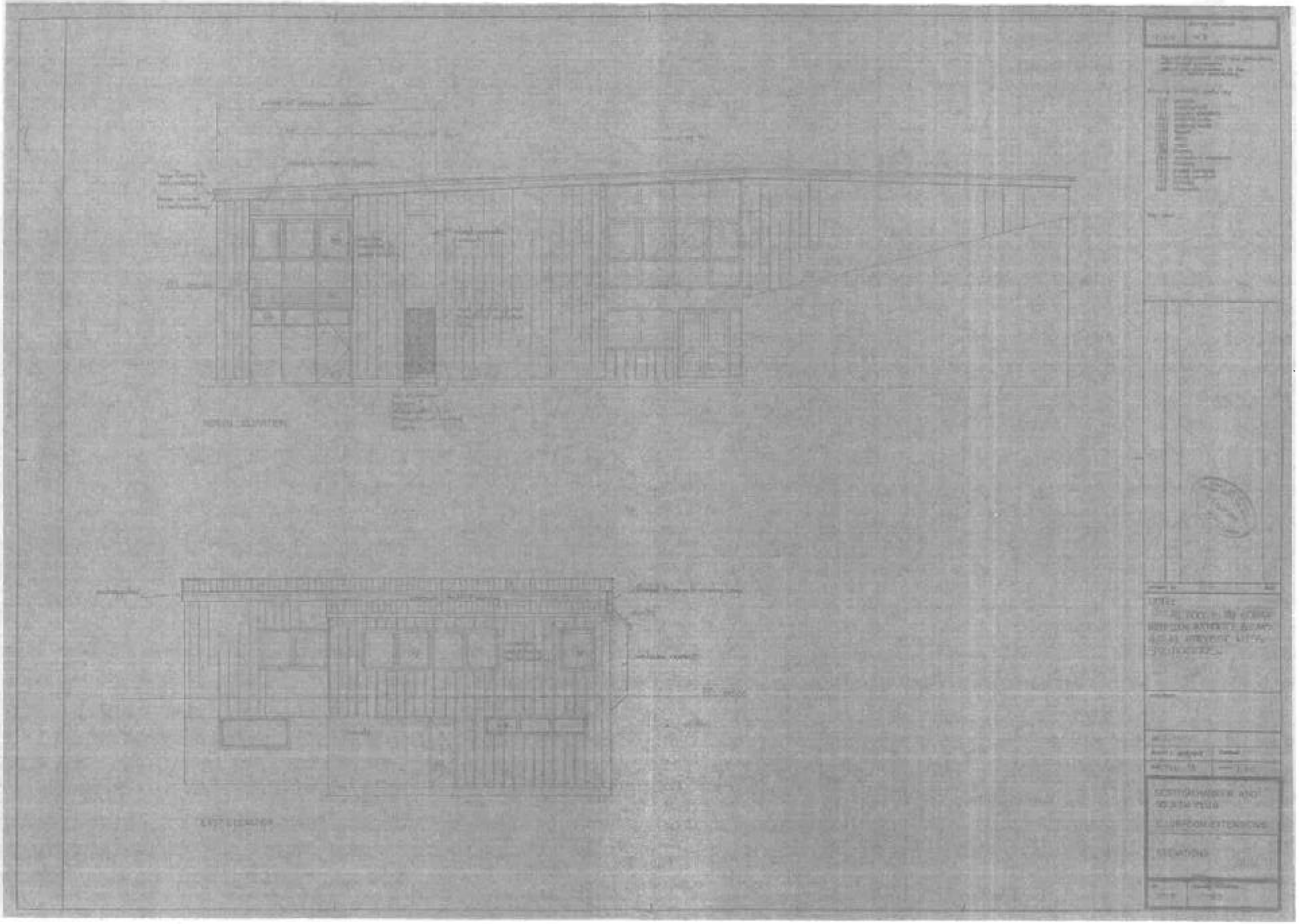
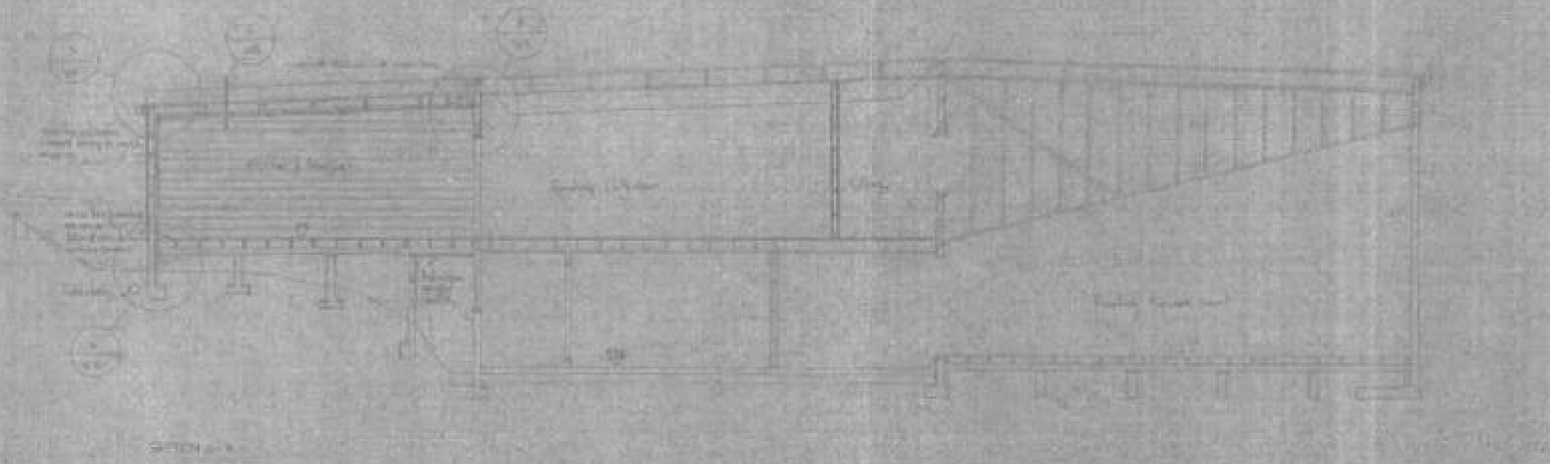
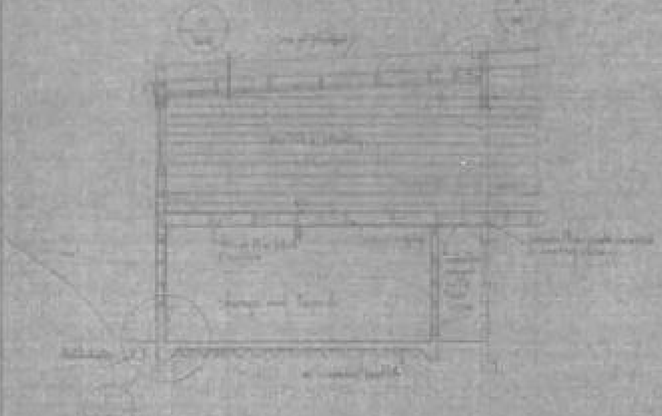

ELEVATIONS



SECTION A-A

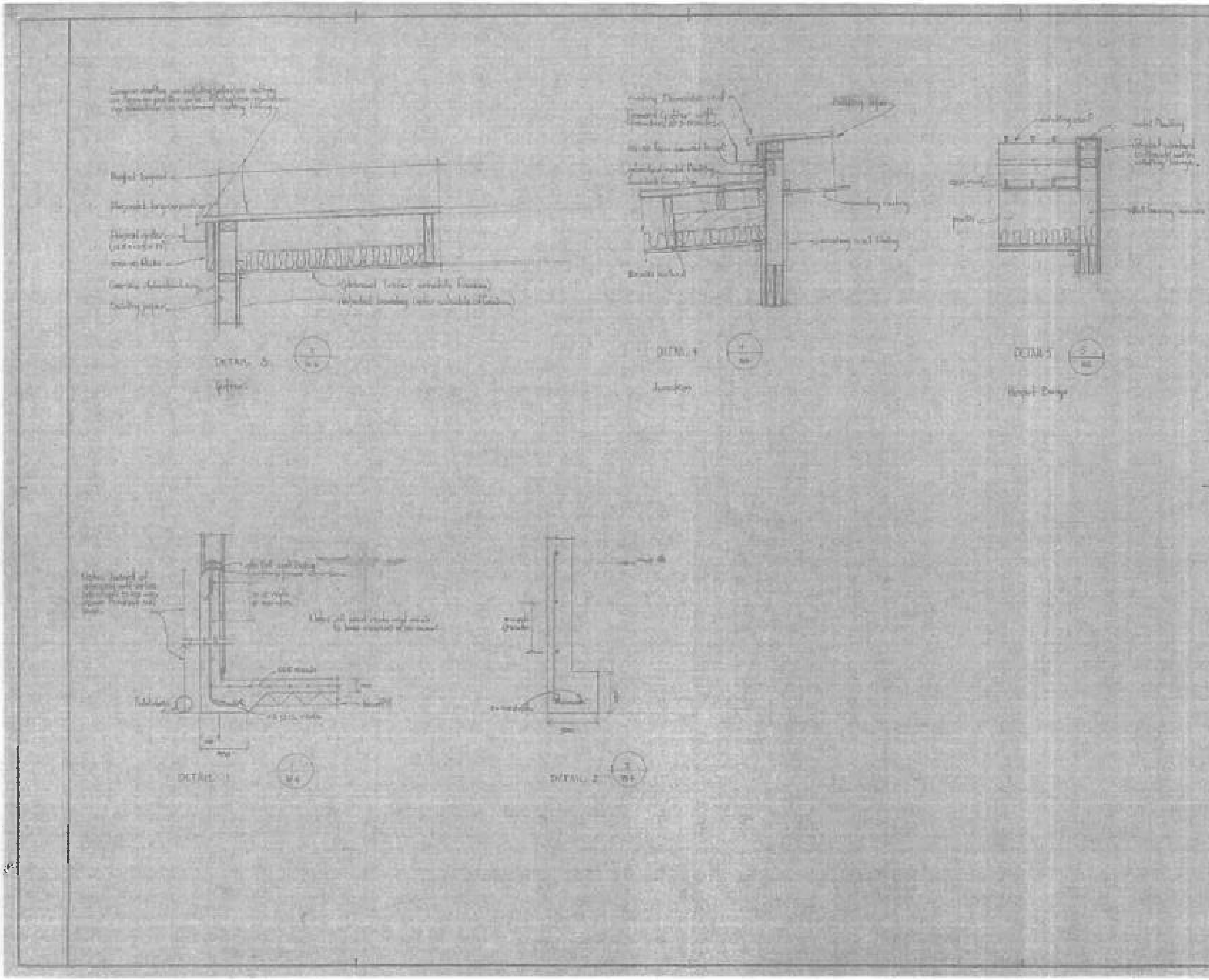


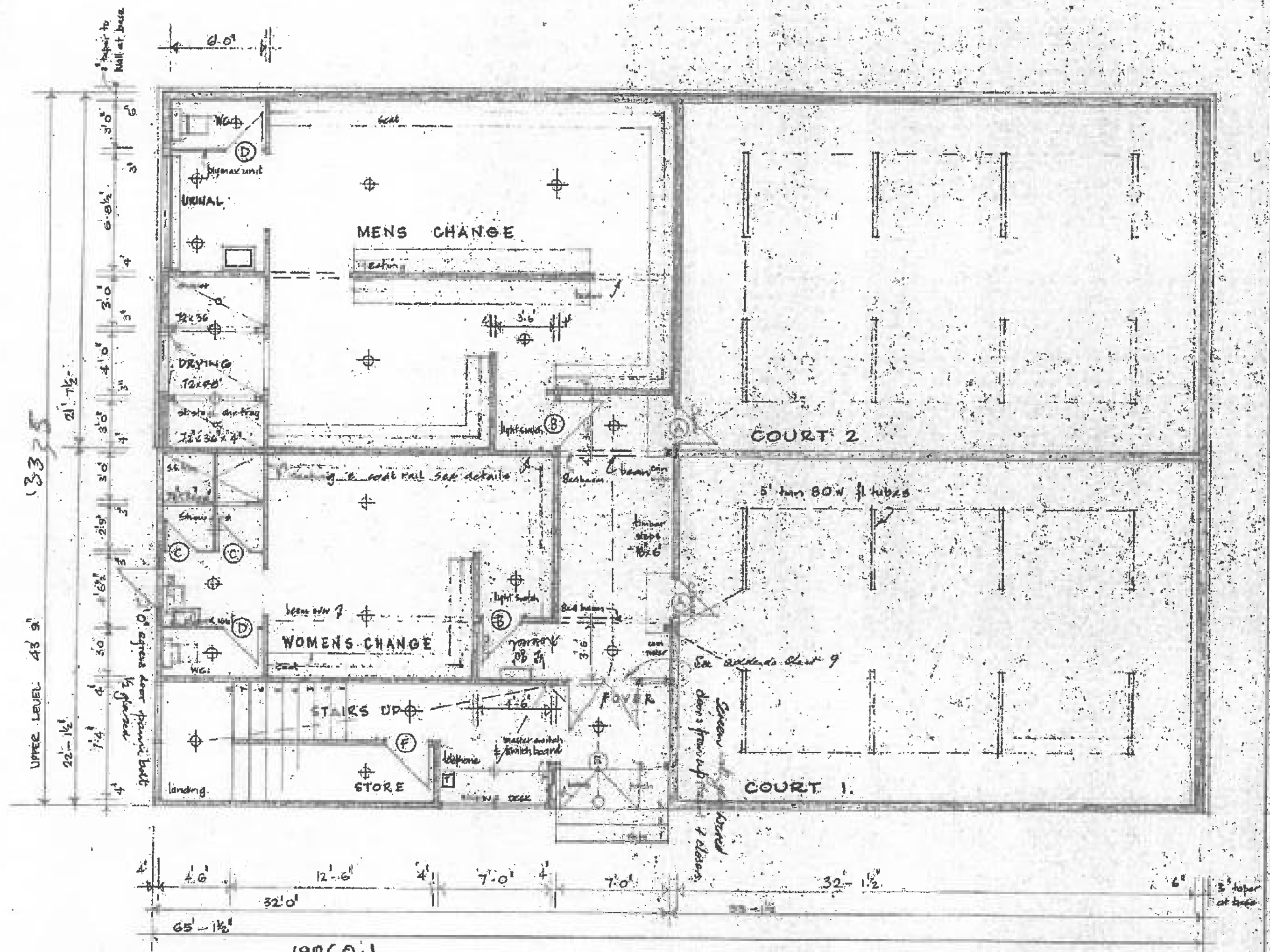
SECTION B-B

SCHEDULE OF FINISHES

ROOM	WALLS N	E	S	W	Ceiling	Floor	Skirting	Windows
FIRST FLOOR								
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		
GROUND FLOOR								
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		

(CLUBHOUSE EXTENSION)

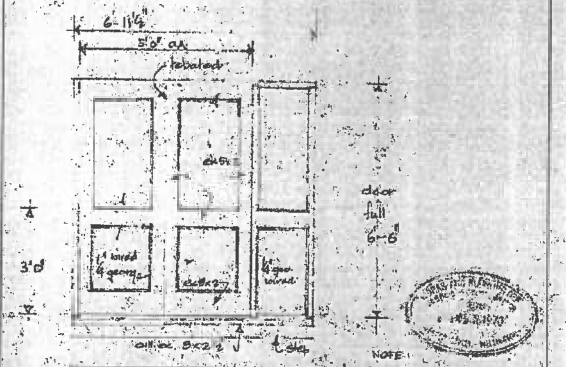

DETAIL 4
DETAIL 5
DETAIL 1
DETAIL 2



GROUND FLOOR PLAN

scale ¼" = 1' 0".

ELECTRICAL LAYOUT:
- ceiling outlets bayonet type.
- exterior bulkhead light.
- switch positions. Coin operated meters to courts
- 5' twin 80 watt fluor tubes to both courts.
- Main switchboard - reverse meter to outside.

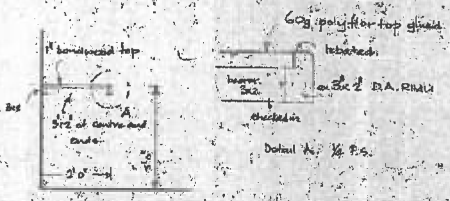


DOORS TYPE E 1 - OFF

scale ½" = 1' 0"

DOOR - SCHEDULE

TYPE	SIZE	DETAILS	C' STRIPS
A	6'6" x 2'4"	Solid core to dwg. paint finish	double
B	6'6" x 2'8"	Solid core. (vent to bot.) varnish	double
C	6'0" x 2'0"	Plymax embossed alum.	—
D	6'0" x 2'6"		—
E	6'6" x 2'6"	Glazed ¼" wired mid rail paint	— see above
F	6'6" x 2'8"	Solidcore ply - varnish	single
G	6'6" x 2'6"	H.C.F. Ply veneer on HB. varnish	single
H	6'6" x	pair glazed ¼" wired midrail - varnish	
J	6'6" x 3'4"	Best Quality solid paint finish	double.



BOOKING DESK

scale ½" = 1' 0"

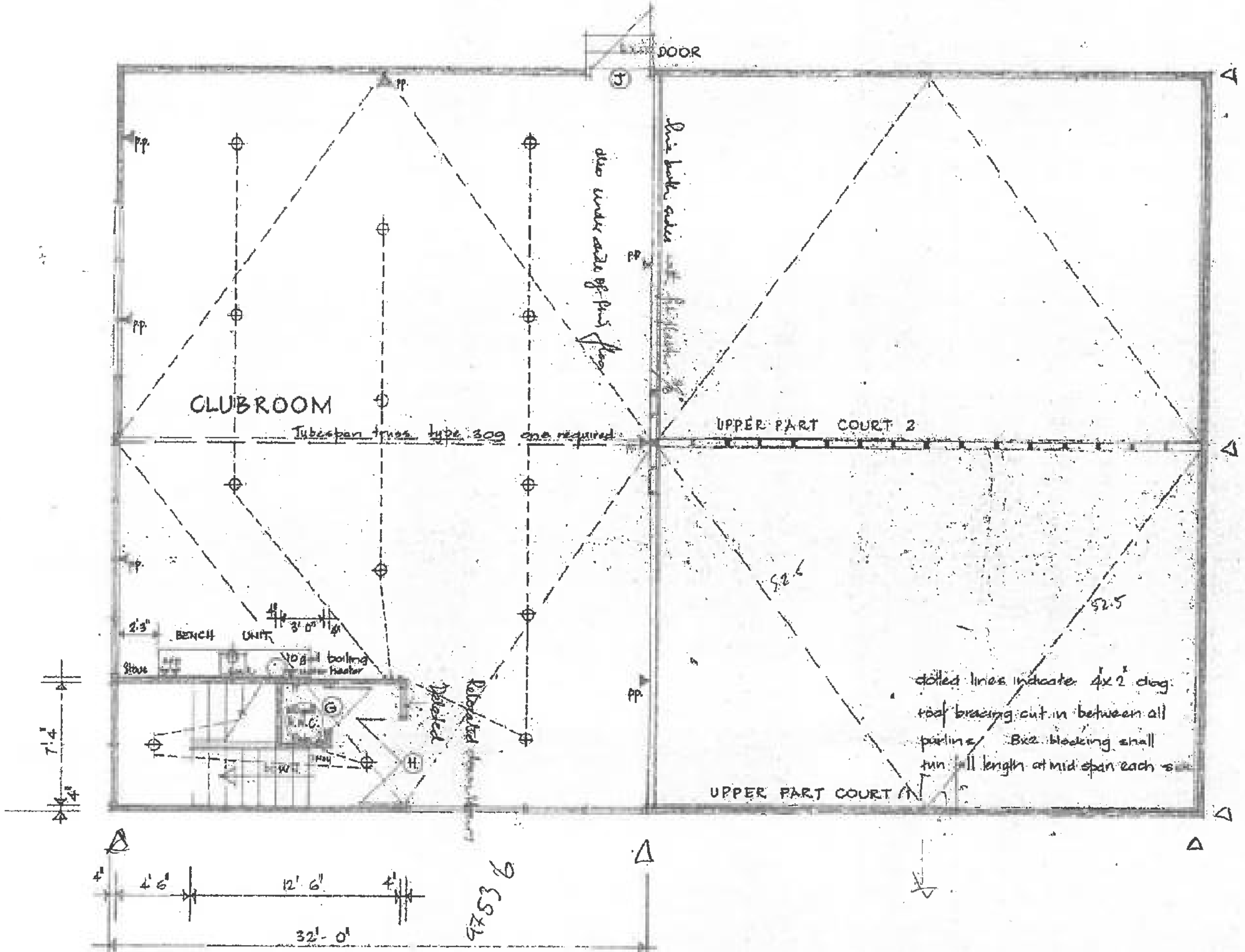
SCOTTISH HARRIERS CLUBHOUSE

MAIN FLOOR PLAN
ELECTRICAL
DOOR SCHEDULE

GRAHAM NAISH/architect
P.O. BOX 2736, WELLINGTON
Phone 557-307

scale ¼" - 1' 0"
date FEB 1970
c/n 405.

DWG NO. 405/1

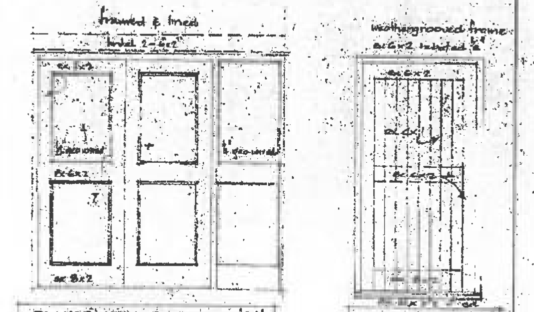


FIRST FLOOR PLAN

scale 1/4" - 1' 0".

ELECTRICAL

- ⊕ : ceiling pendant holders.
- ▶ : 10 amp. power points.

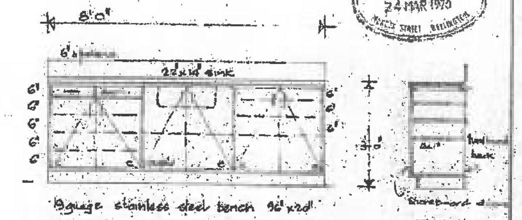


CLUBROOM DOORS 'H'

rebated frame ex 6x2
single action floor spring
scale 1/2" = 1' 0"

EXIT DOOR 'J'

framed ledged & braced – panic bolt



18 gauge stainless steel bench 96"x24"

KITCHEN BENCH scale 1/2" = 1' 0"

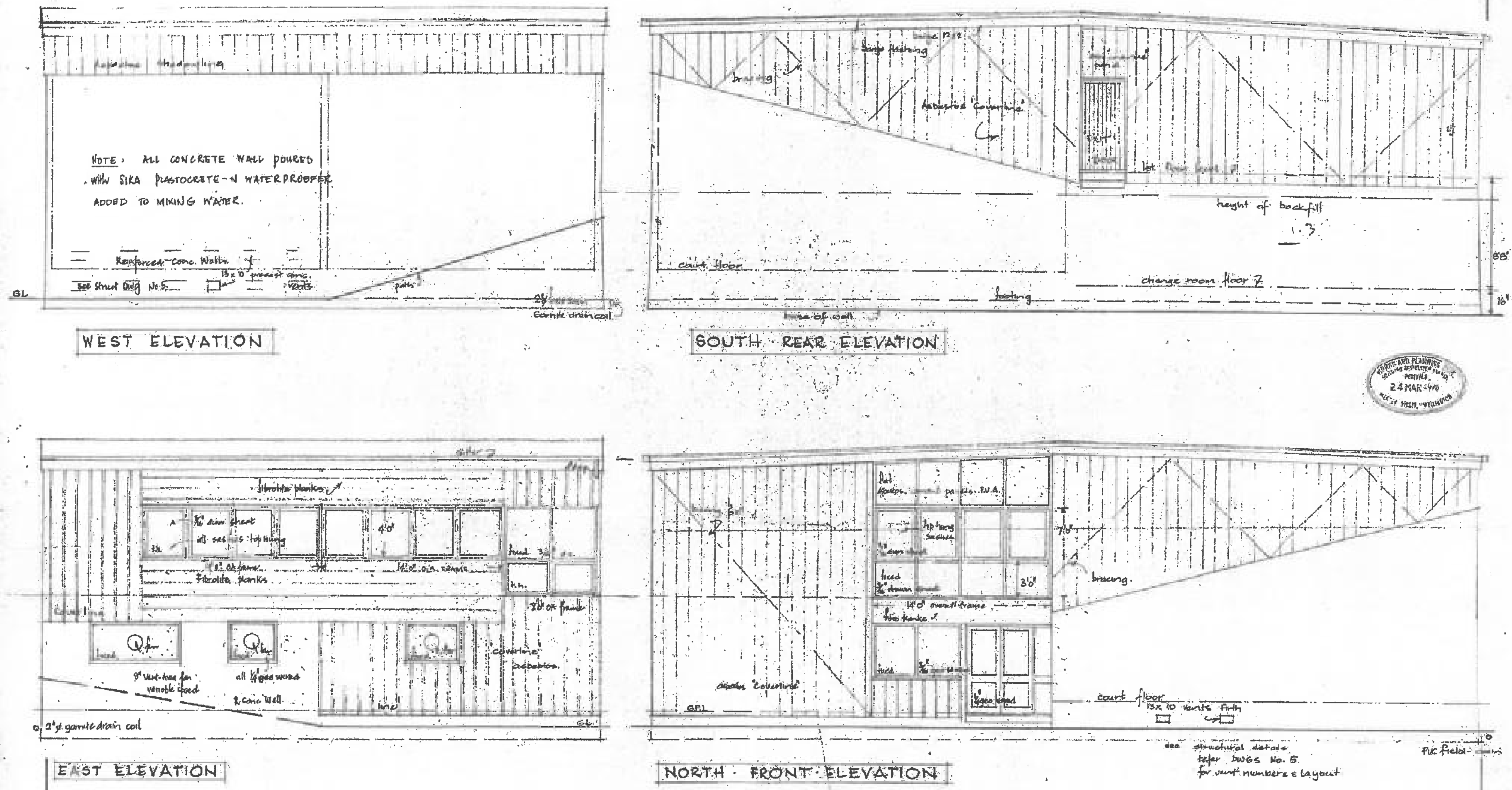
SCOTTISH HARRIERS CLUBHOUSE

FIRST FLOOR PLAN DETAILS

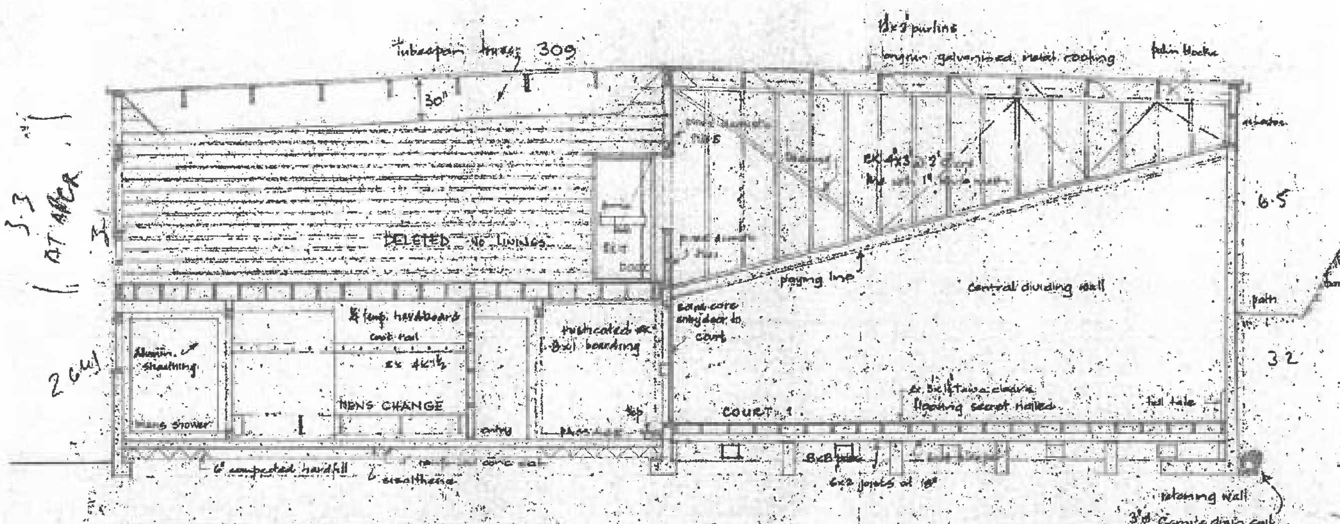
GRAHAM NAISH/architect
P.O. BOX 2736 WELLINGTON
Phone 557-307

scale	1/4" = 1' 0"
date	FEB 1970
c/n	405

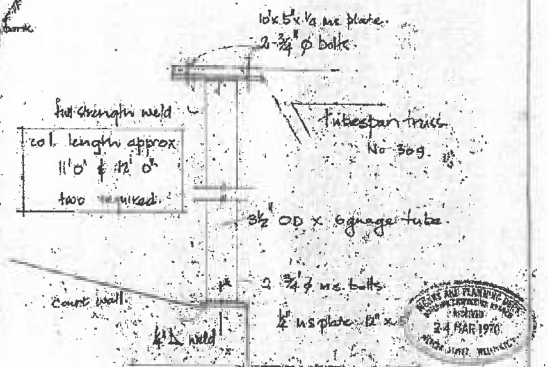
DWG NO. 405/2



SCOTTISH HARRIERS CLUBHOUSE	ELEVATIONS	GRAHAM NAISH/architect P.O. BOX 2736 WELLINGTON Phone 557·307	scale : 1/8" - 1'0" date : FEB. 1970 c/u : 405	DWG NO. 405/3

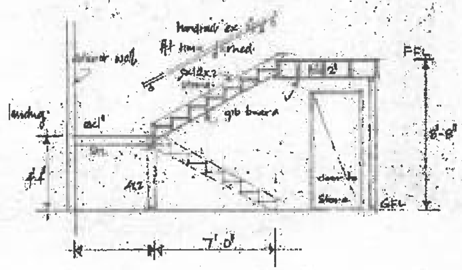


CROSS SECTION A-A



TRUSS SUPPORT COLUMNS

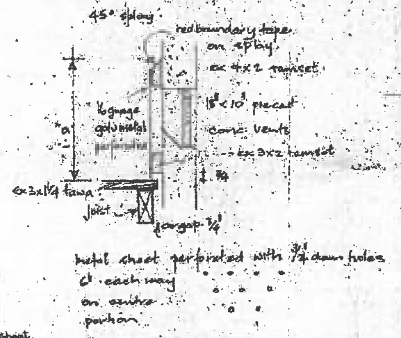
1/8 FS



STAIRS DETAILS

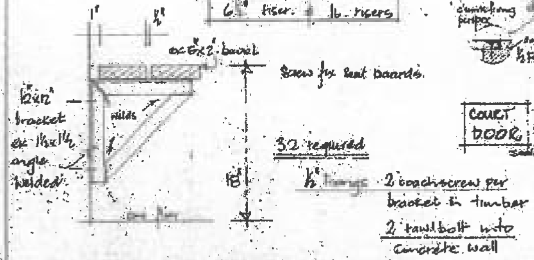
¼" = 1' 0"

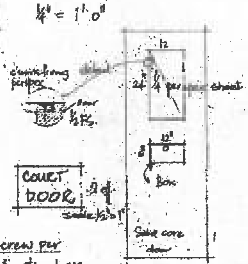
10" tread	
6½" riser & 16 risers	

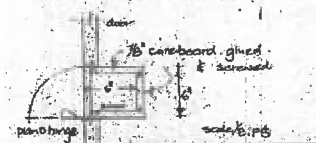


DETAIL OF TELLTALE

Scale 1" = 1' 0"





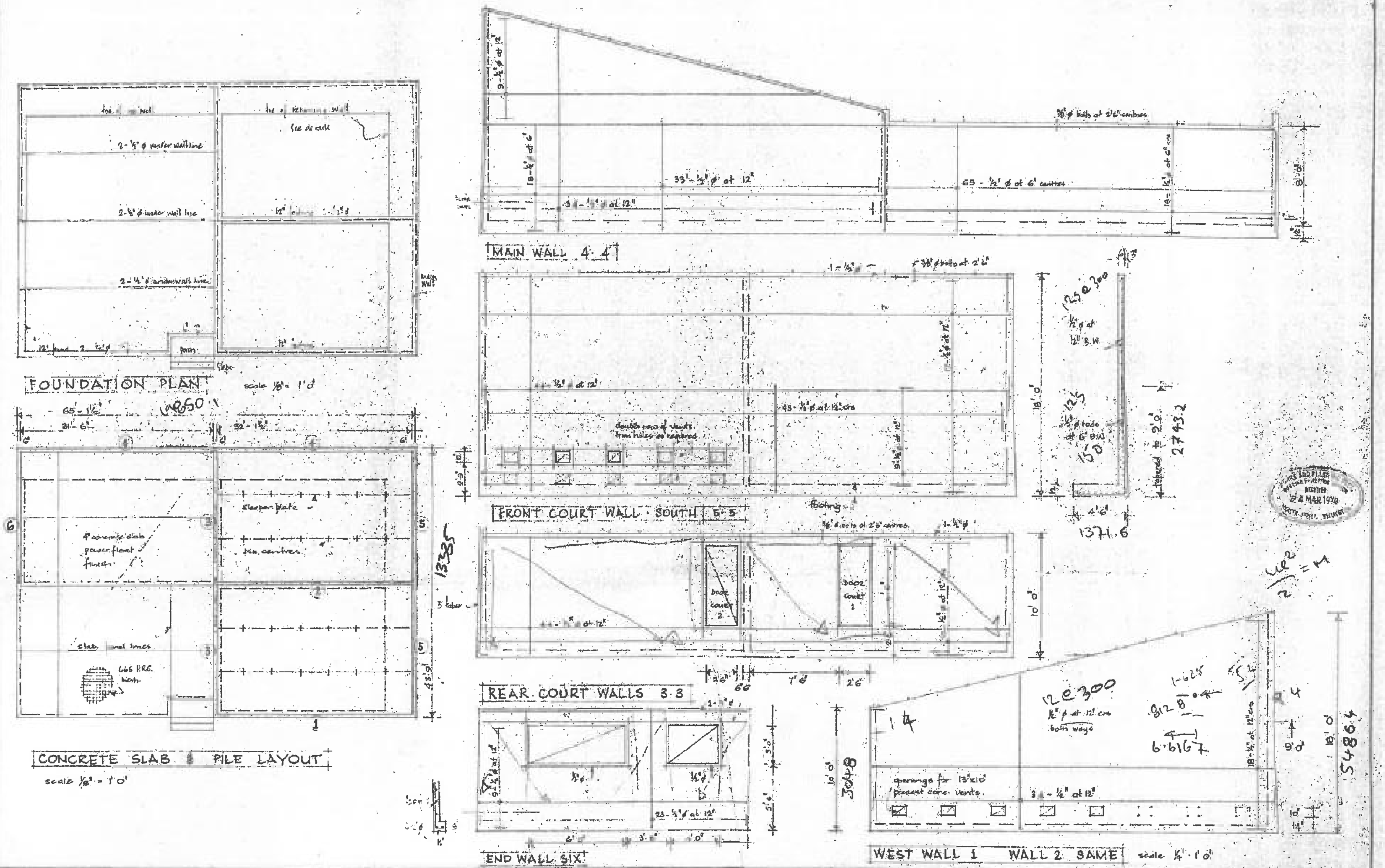


WALLET BOX + DOOR

SCHEDULE OF FINISHES

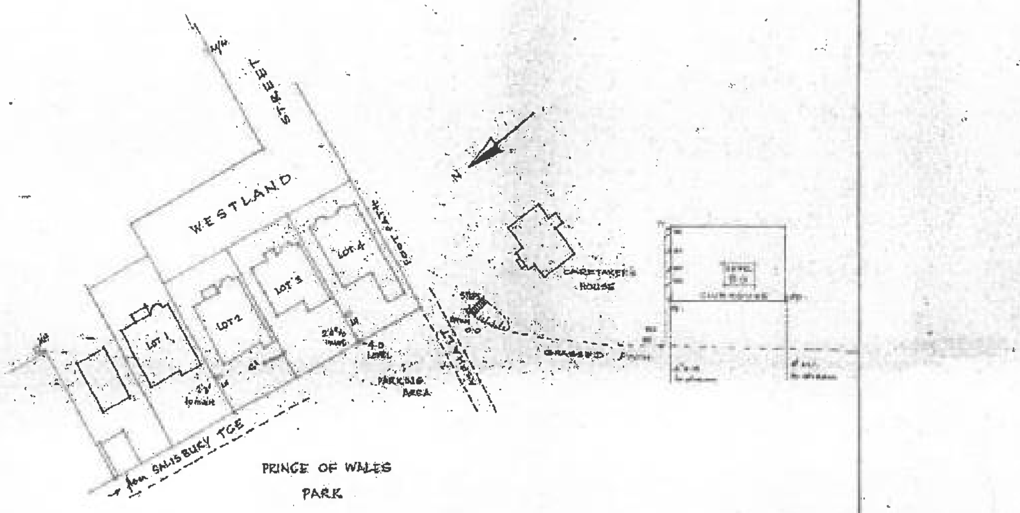
ROOM	WALLS N	S	E	W	CEILING	FLOOR	PAINTING	NOTES
FIRST FLOOR								
CLUBROOM	NIL	NIL	¼ hardboard [illegible]	rusticated boarding	NIL	sanded twice	varnish to [illegible]	
STAIRWELL	vertical [illegible] boarding all round				gib bd [illegible]		varnish to [illegible]	
COURTS	white plaster 3 coat work (speci)				NIL	sanded twice	no floor finish	Floor and [illegible] & secret nailed
GROUND FLOOR								
FOYER	Horizontal boarding varnished			NIL	gib bd. [illegible]	conc. slab	walls varnish	
STAIRWELL	vert [illegible] boarding			NIL	gib bd. [illegible]	Kelly float	varnish	
WOMEN CHANGE	Hardboard	→	→	→	" "	finish	enamel	
TOILET	Plymax and al[illegible] sheeting				gib board	NIL - K float	gib board	
MEN - CHANGE	Hardboard	→	→	→	gib bd [illegible]	NIL K float	enamel	
TOILET	Hardboard	→	→	→	" " "	NIL - K float	enamel	
SHOWERS	Alum[illegible]	→	→	→	gib bd + hd bd	Stainless Steel	[illegible]	
COURTS	white plaster 3 coat [illegible] (speci)				NIL	wood - nat. [illegible]	[illegible]	

SCOTTISH HARRIER'S CLUBHOUSE	CROSS SECTIONS DETAILS SCHEDULE & FINISHES	GRAHAM NAISH/architect P.O. BOX 2736, WELLINGTON Phone 557-307.	scale: ¼" = 1'0" date: FEB.1970 j/n: 405.	DWG NO. 405/4



SCOTTISH HARRIERS CLUBHOUSE

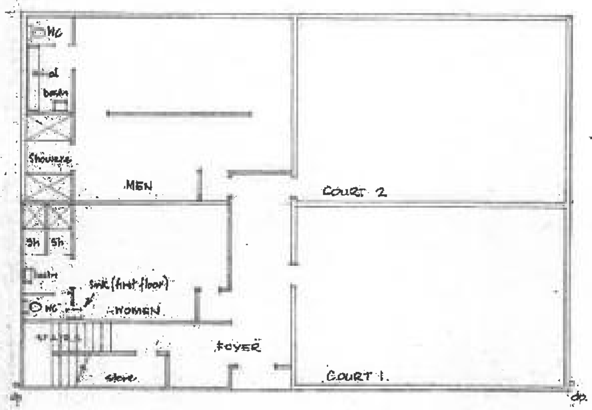
STRUCTURAL DETAILS EDWARDS & CLENDON STRUCTURAL ENGINEERS	GRAHAM NAISH/architect P.O. BOX 2736, 68 NAIRN ST Phone 557-307 W'GTON	scale : 1/4" = 1'0" date : JAN 1970 O/N 405	DWG NO. 405/5



SITE & LOCATION PLAN

scale 1/40" = 1' 0"

KEY.



DRAINAGE & FLOOR PLAN

1/8" = 1'0"

"SCOTTISH HARRIERS CLUBHOUSE"	SITE, LOCATION & DRAINAGE PLAN	GRAHAM NAISH/architect P.O. BOX 2736, WELLINGTON Phone - 557-507	scale : 1/8" = 1'0" date : FEB 1970 c/n : 405

DWG NO 405/6

24 MAR 1970

APPENDIX 2

Supporting Calculations

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BRIEF

DSA OF SCOTTISH HARRIERS CLUBHOUSE DESIGNED IN 1970 + EXTENDED IN 1978

GENERALLY LIGHT WEIGHT TIMBER FRAMED STRUCTURE WITH CONCRETE RETAINING AT REAR AND SIDES

TIMBER ROOF + FLOOR WITH SLAB ON GRADE AT FRONT AND TIMBER FLOOR AT REAR.

LOADS

ROOF

G = 0.4 kPa
Q = 0.25 kPa

WALLS

G = 0.4 kPa (TIMBER WALLS)
γ_{conc} = 23.5 kN/m³ (CONCRETE WALLS)

FLOOR

G = 0.5 kPa (TIMBER FLOOR)
Q = 4.0

HAZARD FACTOR = 0.4 WELLINGTON
SOILTYPE = B (ROCK)

COMBINED HAZARD = LOW TO MOD
GROUND SHAKING = LOW
LIQUEFACTION = OUTSIDE DESIGNATED ZONE ⇒ UNLIKELY
SLOPE FAILURE = LOW

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ROOF

ROOF area = 13.3 × 19.85 = 264.7 m²

Wt = 0.4 × 264.7 = 105.9 kN

TOTAL = ROOF + WALLS
= 167.6

25.6 kN

$V^* = 0.76 \times 167.6 \times 0.809$
= 103 kN

⇒ 103 / 19.850 = 5.2 kN/m
13.335 = 7.7 kN/m

FROM TABLE 9.3 φV = 6 kN/m = 100%
= 78% NBS

⇒ CEILING ACHIEVES 78% NBS.

LONGITUDINAL TIMBER FRAMED WALLS (TFW)

ON GRID 2 $N^*_{BRACE} = 25.1$ kN

FROM TABLE C9.2 $\phi N_{6\times1} = 6 \times 2.5 = 15$ kN $\approx 60\%$NBS

REACTION ON CONCRETE WALL

$R = 27.07 + 29.81 + 20.29 + 0.05$
$= 77.22$ kN $\Rightarrow 7.723$ kN/m

M12 @ 800 = 8.9kN/m > 7.723 OK

$\Rightarrow$ TIMBER BRACES IN LONG DIRECTION ACHIEVES 77 %

TRANSVERSE WALLS

ON GRID 6 $N^* = 36.5$kN $\equiv$ 730 BU's

WALLS = 40× 7.8 = 312 BU's = 43 %

3 $N^* = 65.9$kN $\approx$ 1318 BU's

WALLS = 40× 11.1 = 444 BU's = 34 %

Table C9.2: Probable strength values for existing timber framed wall bracing systems (based on 2.4 m wall height)

Bracing type	Probable strength values
150 x 25 mm let-in brace at 45°	2.0 kN
150 x 25 mm let-in brace at 45° and sheet material* one face	2.5 kN
150 x 25 mm let-in brace at 45° and sheet material* both faces	3.7 kN
90 x 45 mm fitted brace both ways at 45°	2.0 kN
90 x 45 mm fitted brace both ways at 45° and sheet material* one face	2.5 kN
90 x 45 mm fitted brace both ways at 45° and sheet material* both faces	3.7 kN
90 x 45 mm dog leg brace (600 mm wall length)	0.75 kN
Timber framed stud walls with wood or metal lath and plaster	1.5 kN/m each side
Timber framed stud walls with diagonal braces and wood or metal lath and plaster	2.8 kN/m
Gypsum plasterboard one side, and fixed at 300 mm centres (no diagonal timber braces included)	1.0 kN/m
Gypsum plasterboard one side, and fixed at 150 mm centres (no diagonal timber braces included)	2.5 kN/m
Gypsum plasterboard two sides, and fixed at 300 mm centres (no diagonal timber braces included)	2.0 kN/m
Gypsum plasterboard two sides, and fixed at 150 mm centres (no diagonal timber braces included)	3.0 kN/m
Match lining on one or both faces (no diagonal timber braces included)	1.25 kN/m
3.2 mm tempered hardboard fixed with clouts at 200 mm centres	3.0 kN/m
Horizontal board sheathing	1.0 kN/m
Horizontally oriented corrugated steel sheets	2.0 kN/m
Vertically oriented corrugated steel sheets	1.50 kN/m
140 x 20 mm bevel back weatherboard	0.30 kN/m

Note:

*Sheet material is defined as having a density of not less than 450 kg/m^3. It may be a wood-based material not less than 4.5 mm thick or a gypsum-based material not less than 8 mm thick, both fixed to framing members not closer than 10 mm from sheet edges.

When determining the probable wall bracing capacity using the values in Table C9.2 the capacity of each bracing element should be calculated by multiplying by the length of the bracing element and adjusting for height in accordance with the following equation:

$$\frac{2.4}{\text{element height in metres}}$$

This equation is applicable for framing with sheet bracing products attached (and therefore it is not applicable for bracing systems such as horizontal sarking). Elements less than 2.4 m in height should be rated as if they are 2.4 m high. Walls of varying height should have their bracing capacity adjusted using the average height.

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FLOOR DIAPHRAGM

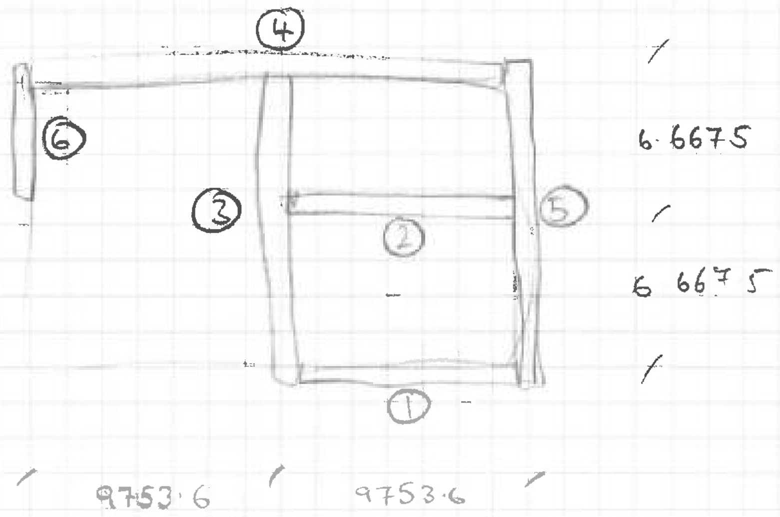
V = 13·3 × 9·925 × (0·5 + 0·3 × 3) × 0·76
= 140·45 kN

V/2 = 70·2 kN ⇒ 7·1 kN/m
⇒ 5·3 kN/m

ϕV = 4 + 1·5 = 5·5 kN/m ⇒ 5·5/7·1 = 77%
3 + 1·5 = 4·5 kN/m ⇒ 4·5/5·3 = 85%

⇒ achieve 77% NBS.

WALLS INPLANE



CONNECTION TO TIMBER BOTTOM PLATE

3/8" φ BOLTS @ 2'6" C/S => M12 @ 762mm crs

φV = 6.8 kN / BOLT => 8.9 kN/m

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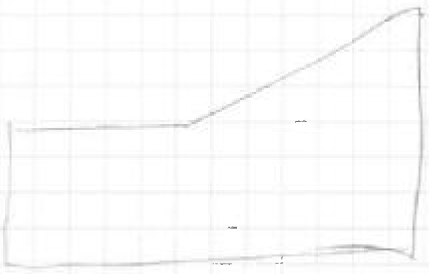
CONCRETE WALLS



Moment

M^*_{WALLS}	1 = 1895 kNm	ϕM = 13587 kNm	
	2 = 3123 kNm	ϕM = 3397 kNm	
	4 = 2882 kNm	ϕM = 13587 kNm	
	3 = 2209 kNm	ϕM = 6317 kNm	
	5 = 3116 kNm	= 6317 kNm	
	6 = 1426 kNm	ϕM = 1351 kNm	= 95% NBS

V^*_{WALLS}	1 = 365.6 kN	ϕV = 2170 kN	
	2 = 610.8 kN	ϕV = 2170 kN	
	4 = 538.5 kN	ϕV = 4340 kN	
	3 = 724.4 kN	ϕV = 2959 kN	
	5 = 552.4 kN	ϕV = 2959 kN	
	6 = 479.1 kN	ϕV = 1368 kN	OK

⇒ WALLS ACHIEVE 95% NBS

Restoring Moment

WALL	SW	OW		%NBS
1	931	339	1270 kNm	67%
2	997	1128	2125 kNm	68%
4	2684	1600	4284 kNm	149%
3	478	1745	2222 kNm	101%
5	1515	1483	2998 kNm	96%
6	833	312	1144 kNm	80%

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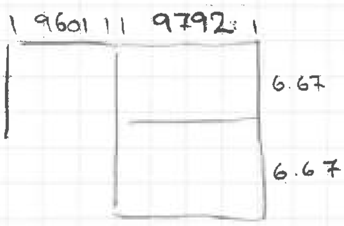
CONCRETE WALLS OUT OF PLANE



$C_p T_p = 0.4 \times 1.0 \times 1.0 \times 2.0 \times (1 + \frac{5.5}{6})$
$= 1.53$

$V^* = 0.15 \times 23.5 \times 1.53$
$= 5.405$ kPa

$M^*_{OOP} = 9.792^2/8 \times 5.4$
$= 64.8$ kNm /m width

1/2" @ " crs $\phi m = 0.85 \times 113/500 \times 324 \times (75 - 4.28/2)$
$= 7.5$ kNm /m

1/2" @ 6" crs $\phi m = 14.6$ kNm

WALLS ACT VERTICALLY

$M^* = 0.76 \times 0.15 \times 23.5 \times 5.5^2/2$
$= 40.5$ kNm

FROM SPACE GASS ANALYSIS $M^*_{plate} = 14.32$ kNm/m

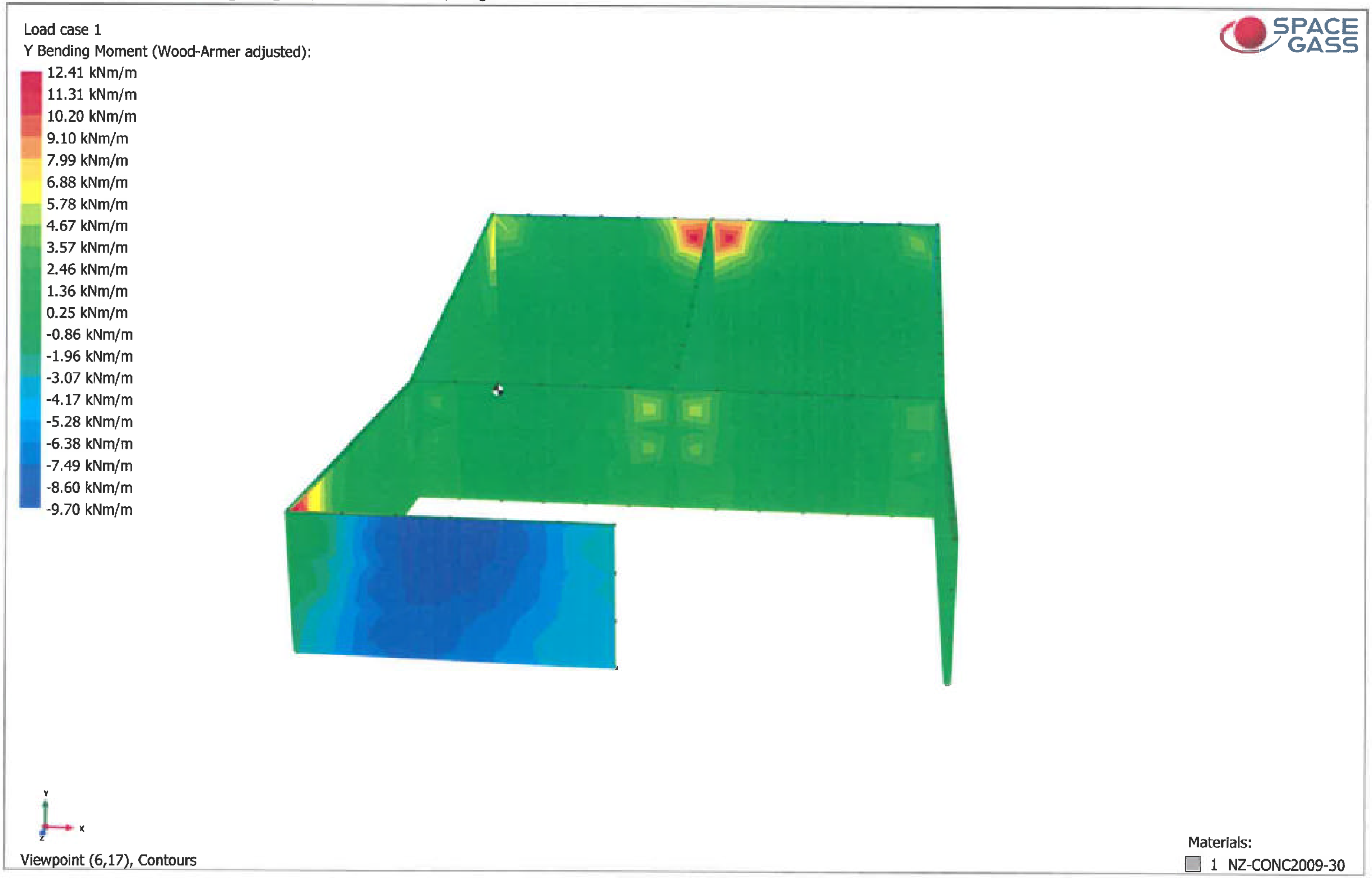
$\Rightarrow 7.5/14.3 = 52\%$ NBS

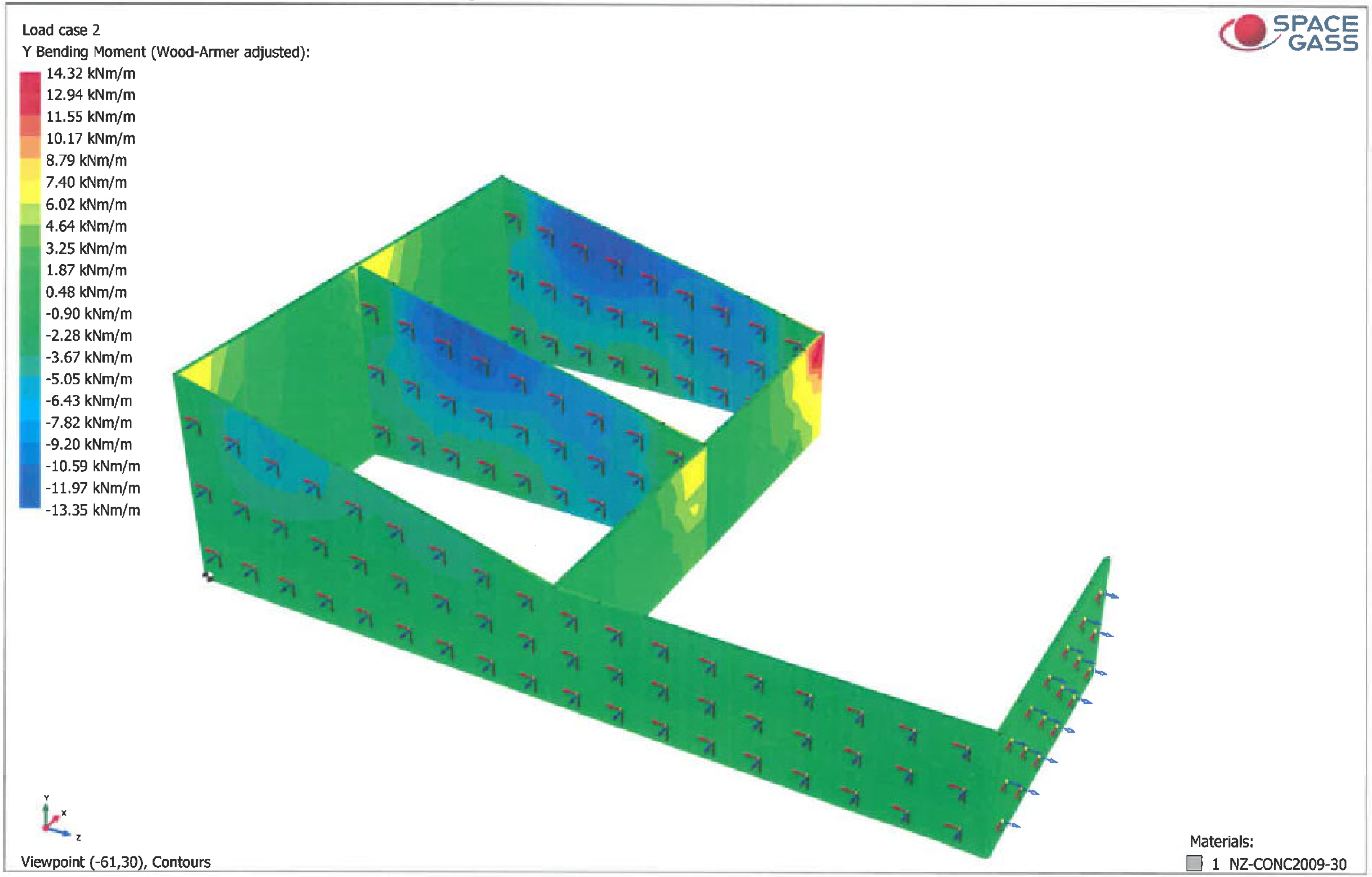
ADDING SEISMIC SOIL LOADS

$M^*_{plate} = 95$ kNm

$\phi m = 41$ kNm $= 43\%$ NBS

SPACE GASS 12.85 - SPENCER HOLMES LIMITED
Path: G:\2022jobs\220740-59\220750\CALCULATIONS\Walls OOP Mesh
Designer: Date: Wednesday, May 31, 2023 12:37 PM, Page: 1




Load case 2
Y Bending Moment (Wood-Armer adjusted):
14.32 kNm/m
12.94 kNm/m
11.55 kNm/m
10.17 kNm/m
8.79 kNm/m
7.40 kNm/m
6.02 kNm/m
4.64 kNm/m
3.25 kNm/m
1.87 kNm/m
0.48 kNm/m
-0.90 kNm/m
-2.28 kNm/m
-3.67 kNm/m
-5.05 kNm/m
-6.43 kNm/m
-7.82 kNm/m
-9.20 kNm/m
-10.59 kNm/m
-11.97 kNm/m
-13.35 kNm/m
SPACE GASS
Y
X
Z
Viewpoint (-61,30), Contours
Materials:
1 NZ-CONC2009-30

x.4 - Applied Wall Forces

x.4.1 - At-Rest Pressure Coefficient

Where effect of backfill is taken from NZGS Module 6, Figure 6.1.

Increase in pressure due to backslope		Ω	1
At rest pressure coefficient	(1 - sin [Φ]) × Ω =	k_0	0.5

x.4.2 - Active Pressure Coefficient

Coulomb eqaution, as modified by Müller Breslau and Mayniel

Active pressure coefficient

$$K_a = \frac{cos^2(\phi-\alpha)}{cos^2(\alpha)\cos(\alpha+\delta)\left(1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\alpha+\delta)\cos(\alpha-\beta)}}\right)^2}$$

K_a 0.30

x.4.3 - Stiff Wall Seismic Pressure Coefficient

NZGS Module 6, section 6.6.2.

ULS seismic wall deflection - found by iteration		Δ	N/A mm
Deflection as a fraction of wall height		Δ/H	- %
Normalised wall force (Fig. 6.2)		$\Delta P_E / K_h \gamma H^2$ =	-
Height of centre of pressure force component (Fig 6.3)		h_0 / H =	-
Increase in stiff wall pressure component due to backslope (Fig 6.4)		Ω	-
Rigid seismic component coefficient	$(\Delta P_E / k_h \gamma H^2) \times kh \times \Omega$ =	K_{RE}	-

x.4.3 - Flexible Seismic Earth Pressure Coefficient

Mononobe-Okabe equation, where negative square problem is addressed as per Eurocode EN1998-5.

ß ≤ (Φ - θ)

$$if\ \beta \le (\phi-\theta),\quad D = 1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta-\theta)}{\sin(\psi-\theta-\delta)\sin(\psi+\beta)}}\quad otherwise\ D = 1.0\quad therefore:$$

D 1.5

Seismic pressure coefficient

$$K_{aE} = \frac{sin^2(\psi+\phi-\theta)}{\cos(\theta)\, sin^2(\psi)\sin(\psi-\theta-\delta)\,(D)^2}$$

K_{aE} 0.68

x.4.4 - Applied Forces

			k_0	k_A	K_{RE}	K_{aE}	
Max soil pressure horizontal	0.5 K γ H² cos(δ) =	qh	28	17		39	kN/m
Max soil pressure, vertical	0.5 K γ H² sin(δ) =	qv	16	10		22	kN/m
Surcharge pressure, horizontal	K ωG H cos(δ) =	qsh	6	3		6	kN/m
Surcharge pressure, vertical	K ωG H sin(δ) =	qsV	3	2		4	kN/m

x.4.5 - Wall Weight and Seismic Inertia

Seismic inertia due to wall self weight kh γw H th = qE 3.8 kN/m

x.4.6 - Deflection Assessment

Calculating curvature deflection only. Allowance for sliding and global rotation estimated.

Wall material			Masonry
Elastic modulus		E	15000 MPa
Second moment of area (gross)	$1.0m \times L_{stem}^3 \div 12$ =	I_{xx}	1E-03 m^4
Second moment of area (cracked)	$0.4 \times I_{xx}$ =	I_{cr}	4E-04 m^4

	Load P (kN)	Load height b (m)	Deflection Δ (mm)	
At Rest soil, horiz.	28	0.9	4.6	
At Rest surcharge, horiz.	6	1.35	1.9	
Additional deflection allowance			4.0	
Sum of At-Rest Case Deflections			**10.5**	**ΣΔ ÷ H = 0.39%**

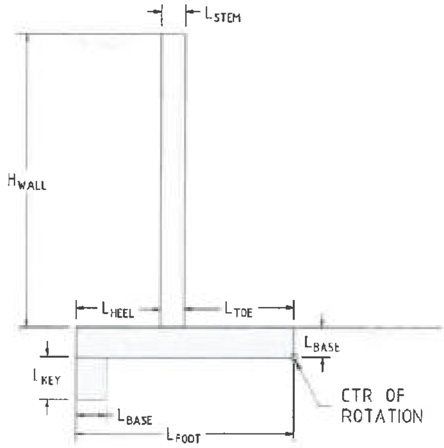
Intermediate pressure applies for Gravity case.

Inertia of wall				
Inertia of soil above heel				
Intermediate soil, horiz.				
Intermediate surcharge, horiz.				
Rigid seismic soil component				
Rigid seismic surcharge component				
Additional deflection allowance			4.0	
Sum of Rigid Seismic Case Deflections				**ΣΔ ÷ H = N/A**

Flexible pressure applies for Seismic Case

x.3 - Wall Dimensions

x.3.1 - Dimensions



	H_w	2.70	m
	L_{stem}	0.229	m
	L_{toe}	1.143	m
	L_{base}	0.300	m
	L_{heel}	0.000	m
	L_{key}	0.000	m
$L_{stem} + L_{toe} + L_{heel} =$	L_{foot}	1.37	m
Wall density =	γ_w	18	kN/m³
Footing density =	γ_f	24	kN/m³

x.3.2 - Wall Reinforcing

Breadth	b	1000 mm	Compression strength parameter 1	α	0.850
Depth	h	229 mm	Compression strength parameter 2	β	0.850
Concrete strength	f_c	30 MPa	Compression strain in extreme fibre	ε_c	0.003
Steel strength	f_y	324 MPa	Minimum reinforcing	ρ_{min}	0.0042
Steel elastic modulus	E_s	200 GPa	Maximum reinforcing	ρ_{max}	0.0326
Strength reduction	ϕ	1	Balanced failure reinforcing ratio	ρ_{bal}	0.0434

Dia.	sp.	n	Cover	Area	d_{eff}	A × d_{eff}
mm	*mm*	*#*	*mm*	*mm²*	*mm*	*mm³*
12	150	6.667	50	754	172.6	130137
0	400	2.5	0	0	0	0
			Σ =	754	Σ =	130137

Member effective depth	$\Sigma(A.d_{eff}) \div \Sigma A =$	d	172.6 mm	
Depth of flexural compression zone	$\Sigma A.f_y \div \alpha.f_c.b =$	a	9.6 mm	
Reinforcing ratio, actual	$\Sigma A \div b.d =$	ρ_{actual}	0.004	OK
Bending strength of wall stem		ϕM_n	41 kN.m	

reo needs to be symmetrical.

Compression strength parameter 1	α	0.850
Compression strength parameter 2	β	0.850
Compression strain in extreme fibre	ε_c	0.003
Minimum reinforcing	ρ_{min}	0.0027
Maximum reinforcing	ρ_{max}	0.0177
Balanced failure reinforcing ratio	ρ_{bal}	0.0236

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EXTENSION

TWO LEVEL TIMBER FRAMED STRUCTURE

ROOF WT = 25.6 kN

WALL ABOVE = (2×9 + 2×7.1) × 0.4 × 1.5 = 19.32 kN
BELOW = 19.32 kN

FLOOR = 7.1 × 9 × (0.5 + 0.3 × 3.0) = 89.5 kN

WALL = (2×5 + 2×7.1) × 0.4 × 1.2 = 11.62 kN

TOTAL = 165.4 kN

V = 165.4 × 0.76
= 125.7 kN

EQUIV STATIC

			$w_i h_i$	$V_{\mu=1.0}$	$V_{\mu=3.5}$
ROOF	44.92	5.4	242.568	57.4 kN	16.6 kN = 332 BU's
FFL	120.44	2.4	289.056	68.3 kN	19.8 kN = 396 BU's
			531.624	125.7 kN	36.4 kN = 728 BU's

CONVERTING TO μ = 3.5 FOR SHEET BRACING SYSTEMS

$\mu_{3.5} / \mu_{1.0} = 0.22 / 0.76 = 0.289$

WALLS AVAILABLE IN EACH DIRECTION

FFL

L ⇒ (0.5 + 2.1 + 0.9) + 3.8 = 7.3 m ⇒ 332 / 7.3 = 45 BU/m

T ⇒ 7 + 1.1 + 2.5 = 10.6 m ⇒ 332 / 10.6 = 31 BU/m

GF

L ⇒ 3 + 3 + 4.8 + 0.8 + 0.8 = 12.4 m ⇒ 728 / 12.4 = 58.7 BU/m

T ⇒ 7 + 1.1 + 1.4 + 7 = 19 m ⇒ 728 / 19 = 38 BU/m

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AT FFL LININGS ARE BOARDS AND GIB WITH CEMENT EXTERIOR SHEETING

$\phi V = 21 + 18$ TO 41
$= 39$ BU/M TO 41 BU/M

⇒ BRACING ACHIEVES 87% TO 100% AT FFL

AT GF FDNS L/H 7/3 = 2.3 ⇒ 100 BU/M ALLOWABLE

SHEET LINING DEMAND REDUCES TO:

L 728 − 6×100 = 128/6.4 = 20 BU/M < ϕV ABOVE

T 728 − 700 = 28/9.5 = 2.9 BU/M < ϕV ABOVE

⇒ BRACING ACHIEVES 87% NBS