MINISTRY OF EDUCATION
Te Tāhuhu o te Mātauranga

Wellington East Girls College

Block 8 – Science

Detailed Seismic Assessment

Template V.1.2

28/01/2016

Prepared By: Opus International Consultants

For the Ministry of Education

Earthquake Resilience Programme

OPUS

Document Control Records

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Revision History

Revision No.	Prepared By	Description	Date
0	9(2)(a) [redacted]	Draft for review	24/09/2015
1		Draft for review	07/10/2015
2		Final	28/01/2016

Document Acceptance

Action	Name	Signed	Date
Site Visit by:	9(2)(a) [redacted]		28/01/2016
Prepared by:			28/01/2016
Reviewed by: CPEng #			28/01/2016
Approved by: CPEng #			28/01/2016

Executive Summary

This building report provides the results of a Detailed Seismic Assessment completed for the following building by the Ministry of Education's Engineering Panel. The report provides a detailed assessment of the building's %NBS seismic capacity, highlights the key seismic risks and presents recommendations for improvements to mitigate potential risks. The table below presents a summary of the assessment findings.

School	Wellington East Girls College
Block No (PMIS).	6549
Block Name/Description	Block 8 - Science
Known Standard Design	Non-standard
Storeys:	2
Year of Design (approx.)	1983
Gross Floor Area (m^2)	833
Construction Type	Reinforced concrete masonry walls
Assessment Type	Detailed
Date Building Inspected	10 September and 17 September 2015
Importance Level	IL3
Structural Assessment Summary	The assessment was based upon a physical internal and external walk around, reviewing drawings and undertaking a detailed structural analysis. The roof space was accessed to review the seismic support of non-structural elements.
Stairs	The two reinforced concrete stairs are tied to reinforced concrete masonry walls; therefore the stairs have low displacement demands and are not expected to be damaged in an earthquake.
Current %NBS estimate	76% NBS
List specific CSWs and life safety hazards	None
Occupancy Considerations	No need to change the building's current occupancy.
Conclusions & Recommendations	Block 8 – Science has an estimated seismic capacity of 76%NBS when assessed as an IL3 building. The governing factor is the pile foundations. Overall, Block 8 is classified as not earthquake prone, as defined in the

	Building Act 2004. The building is classified as a low earthquake risk in accordance with NZSEE guidelines. We recommend that high level glass is checked to confirm if a film or safety glass has been installed. If not, options to strengthen the glass should be explored and implemented to avoid injury if the glass is broken.
Rough order of cost estimate for seismic improvements (where required)	Nil
Timeline for remediation if required	Not applicable

Commentary:

Block 8 – Science is supported by pile foundations which extend through a layer of fill to the rock below. The rock profile is known to slope downwards in the northern direction and so the lengths of the piles vary. The pile lengths vary from 1m to 5m according to the drawings. As the exact rock profile is not known, assumptions have been made regarding the rock profile and pile depths. Therefore the assessment of the piles is an approximate analysis.

The main limiting aspect for the building is the pile foundations loaded in the transverse direction. The piles are governed by their flexural capacity. As the pile plastic hinge zones are well confined a ductility of 3 was assumed.

Lateral Load Resisting System

In both the longitudinal and transverse directions the lateral loads at roof level are distributed through a timber diaphragm to reinforced concrete masonry walls.

The lateral loads at first floor level are distributed through the rigid concrete diaphragm to the reinforced concrete masonry walls.

The lateral loads in the reinforced concrete masonry walls are transferred to the ground through reinforced concrete pile foundations which are founded in rock.

Table of Contents

1. Introduction

This report provides the results of a Detailed Seismic Assessment (DSA) completed for this building by the Ministry of Education's Seismic Assessment Panel. The report provides an assessment of the building's seismic capacity, highlights the key risks and presents recommendations.

Specifically, this report:

- Provides an assessment of the building's capacity in terms of percentage of New Building Standard (%NBS) as defined in New Zealand loading standard NZS 1170.5:2004.
- Identifies any specific Critical Structural Weaknesses (CSWs) or life safety hazards associated with the building and presents recommendations for seismic improvements (if required).

The assessment has involved the following:

- Review of calculations, drawings, specifications and geotechnical information where available.
    - Architectural drawings of Wellington East Girls College Science Building by the Ministry of Works dated 1983. Job Number 5/235/10/7501, sheets 100 to 124.
    - Structural drawings of Wellington East Girls College Science Building
- The roof space was accessed to review the seismic support of non-structural elements.
- Undertaking detailed analysis to determine the seismic strength of the building in accordance with current New Zealand design and material standards to determine the buildings compliance with current building code requirements.
- Where elements of the building have been identified as not meeting acceptable levels of seismic strength, recommendations for seismic improvements are made. Rough order of cost estimates for the structural improvements are included where they are recommended.

For further background information on the Detailed Seismic Assessment (DSA) process please refer to the Ministry of Education website - this includes commentary and relevant context on Building Act compliance requirements.

2. Building and Site Description

Number of Storeys	2
Gross Floor Area (m^2)	833
Year of Design (approximate)	1983
Current use	Teaching Spaces
Structural Alterations	None
Basement	None
Gravity Load Resisting System	Reinforced concrete masonry walls.
Lateral Load Resisting System	Reinforced concrete masonry walls.
Wall/Cladding/Roof System	Plaster clad concrete masonry walls, the roof system is timber roof trusses supporting lightweight cladding.
Floor System	The first floor is a precast interspan concrete rib and timber infill system with a 90mm concrete topping.
Foundation System	Concrete slab on ground beams supported by reinforced concrete piles.
Geotechnical Considerations	Based upon the results of the Opus Geotechnical report dated March 2013, the subsoil classification for the site is considered to be Class B in accordance with NZS 1170.5:2004. Block 8 is underlain by rock. Part of the building is situated on fill, above the rock layer which slopes downwards in the northern direction. The building is founded directly on rock in some areas and on piles extending to the rock in other areas. The liquefaction potential for the site is assessed as nil or low due to rock at shallow depth and the groundwater table being at depth at rock/soil interface at thicker fill areas.

Refer to photos of building in Appendix B and site plan in Appendix C that will assist with understanding building description.

3. Seismic Capacity of the Building

3.1 Analysis Methodology

The building was designed in 1983 by the Ministry of Works. The applicable design code at this time was NZS 4203:1976.

An equivalent static analysis of the building was completed due to the simple geometry and regular layout of the structure. The first floor slab acts as a rigid diaphragm and therefore loads were distributed to the masonry walls based on their relative stiffness, and by tributary areas in some cases.

The elements reviewed were the diaphragm connections, structural walls, foundations and retaining walls. These reinforced concrete and reinforced concrete masonry elements were assessed using NZS 3101:2006, NZS4230:2004 and NZSEE (2006).

The diaphragm connections were analysed using the principle of shear friction.

Longitudinal loads are resisted primarily by the two full height classroom walls. The front wall has multiple openings and was analysed as a frame. The second longitudinal wall and the transverse walls are squat walls with few openings. These walls analysed as shear walls. The transverse walls were checked for a rocking response.

The pile foundations were analysed as being fixed at the rock layer due to the embedment of the piles. The contribution of the fill was neglected. As the exact rock profile is not known, assumptions have been made regarding the rock profile and pile depths. Therefore the assessment of the piles is an approximate analysis.

Part of the rear wall of the building is retaining soil. The seismic capacities of this wall and the reinforced concrete retaining wall beside the building were analysed. The loads acting on these retaining walls were determined using Coulomb sliding wedge theory.

The crib walls behind the building were not reviewed.

There were no historical/original calculations available to assist with the assessment.

3.2 Intrusive Investigations

No intrusive investigations were carried out. Material strengths have been assumed based on the age and condition of the building.

3.3 Assessment Criteria and Building Properties Assumptions

The following table summarises the principal parameters used for the derivation of earthquake loads and the analysis of the building.

Parameter	Value
Design Working Life (remaining)	50 years
Importance Level	3
Return Period Factor (R)	1.3
Site Subsoil Classification	B
Period (seconds)	0.4 seconds (longitudinal direction) 0.4 seconds (transverse direction)
Hazard Factor (Z)	0.40 (Wellington)
Near Fault Factor (N)	1.0
Ductility Factors	1.25 (Diaphragm) 1.25 (Walls in shear) 2.0 (Walls in flexure) 3.0 (Piles)
SP Factor	0.9 (Ductility = 1.25) 0.7 (Ductility ≥ 2)

The following table summarises the probable material strengths utilised in the assessment.

Material	Probable Strength
Concrete – Compressive Strength	f'_c = 30MPa
Concrete Masonry Block Walls – Compressive Strength	f'_m = 12MPa
Steel Reinforcement – Yield Strength	f_y = 325MPa

These material properties have been assumed given the age and condition of the building. There was no information provided on any structural drawings.

3.4 Seismic Capacity Assessment

The following table summarises the %NBS capacity for the various seismic resisting elements in the building based on the detailed seismic analysis.

Element	%NBS Capacity	Commentary
Foundations	76%	Transverse loading, governed by pile flexure
Reinforced concrete masonry walls	>100%	Transverse and longitudinal
Diaphragm connections	>100%	Transverse and longitudinal
Reinforced concrete masonry retaining wall behind Hot Water Room	>100%	Assessed at IL3
Reinforced concrete retaining wall beside building entrance	>100%	Assessed at IL2

The assessment confirms that the building achieves an overall seismic capacity of 76% NBS.

This corresponds to a "Grade B" building as defined by the New Zealand Society for Earthquake Engineering (NZSEE) building grading scheme.

3.4.1 Foundations

The connections between the pile foundations and the ground beams are detailed as pins. The piles, which have a specified minimum embedment into rock of 1m, were assessed as cantilevering from the rock layer. The 76%NBS rating for the foundations is governed by the piles subject to transverse loading. The rock profile is assumed to vary linearly from 1m to 4m depth under the transverse walls. The piles are governed by their flexural capacity. As the pile plastic hinge zones are well confined a ductility of 3 was assumed.

In the longitudinal direction the rock profile is assumed to be flat. Therefore the piles are equally loaded in this direction and the transverse direction is critical.

3.4.2 Masonry Walls

All load resisting masonry walls have an estimated seismic capacity of greater than 100%NBS. The front longitudinal wall which acts as a frame is governed by pier hinging. Therefore a ductility of 2 was assumed. The other walls act as shear walls and a ductility of 1.25 was assumed.

3.4.3 Diaphragm Connections

The diaphragm connections were assessed using the principle of shear friction, assuming a potential crack at the interface of the diaphragm and the masonry walls. The diaphragm connections have an estimated seismic capacity greater than 100%NBS.

3.4.4 Retaining Walls

Part of the external building wall at the southern side of the building is retaining soil (Figure 1). As the wall is part of the building it was assessed as a retaining wall at IL3. This wall has an estimated seismic capacity greater than 100%NBS.

Figure 1: Reinforced concrete masonry retaining wall

There is a reinforced concrete retaining wall beside the building entrance (Figure 2). Sliding or tilting of the retaining wall is not expected to have an effect on the accessibility or structural performance of the Science Block. Therefore the IL3 building rating need not apply to the retaining wall which is considered an IL2 structure in accordance with AS/NZS 1170.0:2002. The retaining wall has an estimated seismic capacity greater than 100%NBS.

Figure 2: Reinforced concrete retaining wall

3.5 Structural Weaknesses & Life Safety Hazards

3.5.1 Potential Critical Structural Weaknesses

None identified.

The first floor concrete diaphragm connections have a seismic capacity greater than 100%NBS and are not considered to be a critical structural weakness.

The seating of the flooring is not considered to be a critical structural weakness. Although the seating length could not be verified on site, based on scaling of the relevant drawing detail the seating length is estimated to be approximately 40mm. The flooring is tied to the masonry walls with L bars.

3.5.2 Specific Critical Structural Weaknesses

None identified.

3.5.3 Stairs

The two reinforced concrete stairs are tied to reinforced concrete masonry walls; therefore the stairs have low displacement demands and are not expected to be damaged in an earthquake.

3.5.4 Secondary Structural Weaknesses & Life Safety Hazards

The ground floor classrooms have heaters that are suspended from the first floor (Figure 3) and the first floor classrooms have heaters that are suspended from the ceiling (Figure 4). These heaters do not appear to be seismically restrained and are suspended a reasonable distance. Therefore these heaters are a *potential* life safety hazard. We recommend that the seismic restraint of these heaters be reviewed in more detail to confirm whether or not the heaters are a life safety hazard.

Figure 3: Heater in ground floor classroom

Figure 4: Heaters in first floor classroom

The mechanical and electrical plant in the roof space is not considered a secondary structural weakness or life safety hazard. The roof space was inspected on September 17 to review the seismic support of this plant.

The rigid ducting in the roof (Figure 5) which spans over the roof truss chords and masonry walls is not positively fixed to the primary structure. The sections of ducting are relatively long, spanning over multiple supports. Although the ducting may attain some damage in a ULS event, loss of support is not possible. Therefore there is no issue with the seismic support of the rigid ducting.

Figure 5: Rigid ducting

The pipes in the roof space are hung from rigid hangers (Figure 6) and propped up by rigid hangers (Figure 7). The pipe hangers are less than 150mm long and therefore the pipes do not require specific seismic restraint in accordance with NZS 4219:2009.

Figure 6: Pipe hangers

Figure 7: Pipe support

The extractor fans in the roof space are positively fixed to the primary structure and the extractor fan exhausts are restrained with steel angle frames (Figure 8). The intakes and exhausts appear to be attached to the extractor fans with flexible joints to allow for differential movement. The extractor fan plant is considered to have adequate seismic support.

Figure 8: Extractor fan and exhaust support

4. Seismic Improvements

4.1 Suggested Improvements

The building achieves the minimum Ministry of Education seismic strength capacity of 67%NBS and no seismic improvements are considered necessary.

4.2 Rough Order of Cost Estimate

Not required if no seismic improvements are suggested.

5. Conclusions & Recommendations

5.1 Conclusions

The building achieves an overall seismic capacity of 76% NBS when considered as an Importance Level 3 building. This meets the Ministry of Education's medium term goal of 67% NBS or above.

There is no need to change the buildings current occupancy.

5.2 Recommendations

The building satisfies the Ministry of Education's desired minimum seismic strength capacity of 67% NBS and no seismic improvements are considered necessary for this building.

6. Explanatory/Limitations Statement

- This report contains the professional opinion of Opus International Consultants as to the matters set out herein, in the light of the information available to it during preparation, using its professional judgment and acting in accordance with the standard of care and skill normally exercised by professional engineers providing similar services in similar circumstances. No other express or implied warranty is made as to the professional advice contained in this report.

- We have prepared this report in accordance with the brief as provided and our terms of engagement. The information contained in this report has been prepared by Opus International Consultants at the request of its client, the Ministry of Education, and is exclusively for its use and reliance. It is not possible to make a proper assessment of this report without a clear understanding of the terms of engagement under which it has been prepared, including the scope of the instructions and directions given to and the assumptions made by Opus International Consultants. The report will not address issues which would need to be considered for another party if that party's particular circumstances, requirements and experience were known and, further, may make assumptions about matters of which a third party is not aware. No responsibility or liability to any third party is accepted for any loss or damage whatsoever arising out of the use of or reliance on this report by any third party.

- The report is also based on information that has been provided to Opus International Consultants from other sources or by other parties. The report has been prepared strictly on the basis that the information that has been provided is accurate, complete and adequate. To the extent that any information is inaccurate, incomplete or inadequate, Opus International Consultants takes no responsibility and disclaims all liability whatsoever for any loss or damage that resulting from any conclusions based on information that has been provided to Opus International Consultants.

Appendix A

Detailed Seismic Assessment Calculations

CALCULATION SHEET

Project/Task/File No: .... Sheet No ..... of .....

Project Description: .... Office: .....

Computed: / /

Check: / /

Contents

Sheets

Introduction

WEGC Science Block is a 2 storey RC Masonry building

- Timber roof
- Rib & timber infill 1st Floor
- 2 RC stairs
- Part of South external wall retaining soil
- Pile foundations founded in rock through fill.

Analysis

Transverse walls and Gridline B walls act as Shear walls.

Front of building acts as a frame.

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CALCULATION SHEET

Project/Task/File No: | Sheet No 2 of
Project Description: | Office:
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Building Weight

Roof

Roof is approx 40m × 14m

Allow 0.4kPa

W_{roof} = 0.4 × 40 × 14 = 224 kN

Halls

Majority of walls are fully grouted 20 series blockwork, with plaster finish or timber framing on either side.

Assume 22 kN/m³

Take wall thickness as 210mm at 22kN/m³ (Allowance of 20mm for finish / timber framing - equivalent weight of block) ⇒ 4.6 kPa
Reduce to 4.0kPa to account for openings.

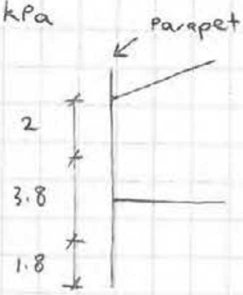
Ground floor RL = 59.6

1st floor RL = 63.225

Ceiling RL = 67.225

1st storey: 3.6 m

2nd storey: 4.0 m



Full Height Masonry (Grid A1 → B6) [4.0kPa]

Gridline	Height (m)	Length (m)	Weight (kN)	Proportion to: Ground	1st	Roof	W1 (kN)	W2 (kN)
A	8.6	40	1582	1.8/8.6	3.8/8.6	3/8.6	608	480
B	8.6	18	712	"	"	"	274	216
	7.8	22	789	1.8/7.8	3.8/7.8	2.2/7.8	335	194

Transverse walls (Total 6)

	7.8	9 × 6	1938	"	"	"	1821	476

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CALCULATION SHEET

Project/Task/File No: | Sheet No 3 of
Project Description: | Office:
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Part Height Masonry (Grid B1 → D6) [4.6kPa]

3.6m walls around stairs

Length = 24m

W_1 = 4.0kPa x 1.8 x 24 = 173kN

Part Height Masonry

Gridline	Height (m)	Length (m)	Weight (kN)	Proportion to 1st fl. Level	W_1 (kN)
C	4.8	5x2	221	(4.8−1.8)/4.8	120
D	4.6	27	571	(4.6−1.8)/4.6	303

Transverse walls

	5.4	2x2	99	(5.4−1.8)/5.4	57
					480kN

Total Wt. of Masonry

ΣW_1 = 2038 + 173 + 480 = 2691kN

ΣW_2 = 1366kN

Timber Walls – Allow 0.4kPa

Refer Sheet 1

Walls around stairs

Length = 24m

Height = 2m on average

W_1 = 0.4 x 24 x 2 = 19kN

Full height (around Toilets)

Length = 13m

Height = 6m

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CALCULATION SHEET

Project/Task/File No: | Sheet No 4 of

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$W_1 = 0.4 \times 13 \times 6 = 31kN$

Ground Floor Timber

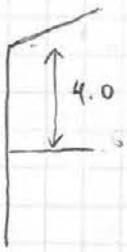
Length = 9 + 4 + 6 = 19m

1.8m contributing

$W_1 = 0.4 \times 19 \times 1.8 = 14kN$

First Floor Timber

Length = 12m



$W_1 = 10kN$

$W_2 = 10kN$

Total Wt. of Timber Walls

$W_1 = 19 + 31 + 14 + 10 = 74kN$

$W_2 = 10kN$

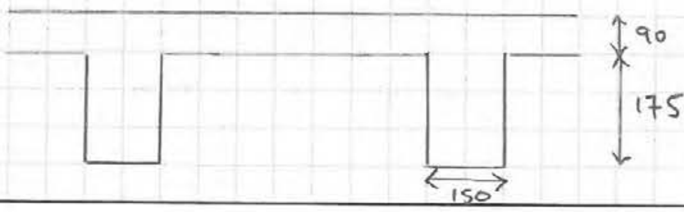
Floor

Lump weight of stairs into floor weight

Refer sheet 1

Floor area = 40 x 11 = 440 m^2

Floor is Stahlton Ribs at 900 centres with 90mm conc infill on 25mm timber.



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CALCULATION SHEET

Project/Task/File No: .... Sheet No 5 of

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Wt. of ribs = $\frac{24 \times 0.15 \times 0.175}{0.9}$ = 0.7 kPa

Wt. of slab = 24 × 0.09 = 2.16 kPa

] 2.9 kPa

W_{floor} = 2.9 kPa × 440 = 1276 kN

Imposed Load

Allow Q = 3 kPa (classrooms)

$\psi_E = 0.3$

$W_{imposed}$ = 3 × 0.3 × 440 = 396 kN

Total Building Weight

W_1 = Masonry + Timber walls + Floor + Imposed

= 2691 + 74 + 1276 + 396 = 4437 kN

W_2 = Roof + Masonry walls + Timber walls

= 224 + 1366 + 10 = 1600 kN

W_t = 6037 kN

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Job Title: *WEGC Science Block DSA*
Job Number: *5-PA010.37*
Calcs By: *MJG*

Member Reference:
Date: *4/09/2015 1:33:32 p.m.*

Report created using Seismic Shear forces to NZS1170.5 Design Tool Version **1.1**

HORIZONTAL SEISMIC SHEAR (V) TO NZS 1170.5

Input Data
Period (T) = 0.2 sec
Site Classification A/B
Equivalent Static Method
Hazard Factor (Z) (See Table 3.3) = 0.4
Importance level of 3
Design Working Life of 50 Years
ULS Ductility (mu) = 1.25
SLS1 Ductility (mu) = 1.0
ULS Structural Performance Factor (Sp) = 0.9
SLS Structural Performance Factor (Sp) = 0.7
Seismic Weight (Wt) = 6037 kN

ULS Results
ULS Return Period of 1/1000
Spectral Shape Factor Ch(T) = 1.890
Return period factor from table 3.5 (Ru) = 1.30
Near Fault Factor N(T,D) = 1.000
Elastic Site Spectrum C(T) = 0.9828
Ductility Factor k(mu) = 1.143
Design Action Coefficient Cd(T) = 0.774
Horizontal Seismic Shear = **4672** kN

SLS1 Results
Return Period of 1/25
Return period factor (Rs) = 0.25
Elastic Site Spectrum C(T) = 0.1890
Ductility Factor k(mu) = 1.000
Design Action Coefficient Cd(T) = 0.132
Horizontal Seismic Shear = **799** kN

Sheet 7

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Job Title: *WEGC Science Block DSA*
Job Number: *5-PA010.37*
Calcs By: *MJG*

Member Reference:
Date: *4/09/2015 1:37:33 p.m.*

Report created using Seismic Shear forces to NZS1170.5 Design Tool Version **1.1**

EQUIVALENT STATIC METHOD TO NZS 1170.5 Cl 6.2.1.3

Level	**Height hi(m)**	**Weight wi(kN)**	**wi * hi**	**Lat Force Fi(kN)**
Level 2	7.6	1600	12160	2232
Level 1	3.6	4437	15973	2440
Sum		**6037**	**28133**	**4672**

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Job Title: *WEGC Science Block DSA*
Job Number: *5-PA010.37*
Calcs By: *MJG*

Member Reference:
Date: *4/09/2015 11:12:07 a.m.*

Report created using Seismic Shear forces to NZS1170.5 Design Tool Version **1.1**

HORIZONTAL SEISMIC SHEAR (V) TO NZS 1170.5

Input Data
Period (T) = 0.2 sec
Site Classification A/B
Equivalent Static Method
Hazard Factor (Z) (See Table 3.3) = 0.4
Importance level of 3
Design Working Life of 50 Years
ULS Ductility (mu) = 2
SLS1 Ductility (mu) = 1.0
ULS Structural Performance Factor (Sp) = 0.7
SLS Structural Performance Factor (Sp) = 0.7
Seismic Weight (Wt) = 6037 kN

ULS Results
ULS Return Period of 1/1000
Spectral Shape Factor Ch(T) = 1.890
Return period factor from table 3.5 (Ru) = 1.30
Near Fault Factor N(T,D) = 1.000
Elastic Site Spectrum C(T) = 0.9828
Ductility Factor k(mu) = 1.571
Design Action Coefficient Cd(T) = 0.438
Horizontal Seismic Shear = **2643** kN

SLS1 Results
Return Period of 1/25
Return period factor (Rs) = 0.25
Elastic Site Spectrum C(T) = 0.1890
Ductility Factor k(mu) = 1.000
Design Action Coefficient Cd(T) = 0.132
Horizontal Seismic Shear = **799** kN

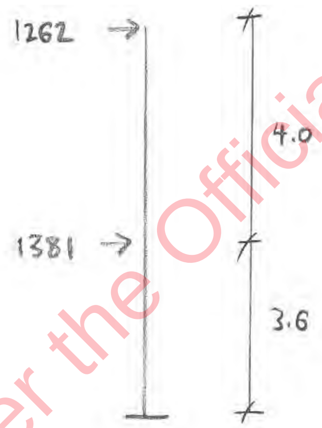
Job Title: *WEGC Science Block DSA*
Job Number: *5-PA010.37*
Calcs By: *MJG*

Member Reference:
Date: *4/09/2015 1:38:28 p.m.*

Report created using Seismic Shear forces to NZS1170.5 Design Tool Version **1.1**

EQUIVALENT STATIC METHOD TO NZS 1170.5 Cl 6.2.1.3

Level	Height hi(m)	Weight wi(kN)	wi * hi	Lat Force Fi(kN)
Level 2	7.6	1600	12160	1262
Level 1	3.6	4437	15973	1381
Sum		**6037**	**28133**	**2643**



Overturning moment = 1262 x 7.6 + 1381 x 3.6 = 14 563 kNm

CALCULATION SHEET

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Project Description: | Office:

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Distribution of Load to Transverse Walls

Use tributary areas based on the extent of the two storey area.

Refer sheet 11.

Total area = $39.4 \times 14.4 - 2(8.6 \times 1.8 + 3.8^2) = 507.5\,m^2$

Gridline	Tributary area (m^2)	% of Load
1	$5 \times 8.8 = 44$	8.6
2	$6.6 \times 14.4 = 95$	18.6
3	$6.5 \times 14.4 = 93$	18.2
4	$8.1 \times 14.4 = 116$	22.7
5	$8.2 \times 14.4 = 118$	23.1
6	$5 \times 8.8 = 44$	8.6
	$510\,m^2$	99.8 %

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Calculation Sheet

Project/Reference No:	Sheet No: 11
Project and Description:	Office:
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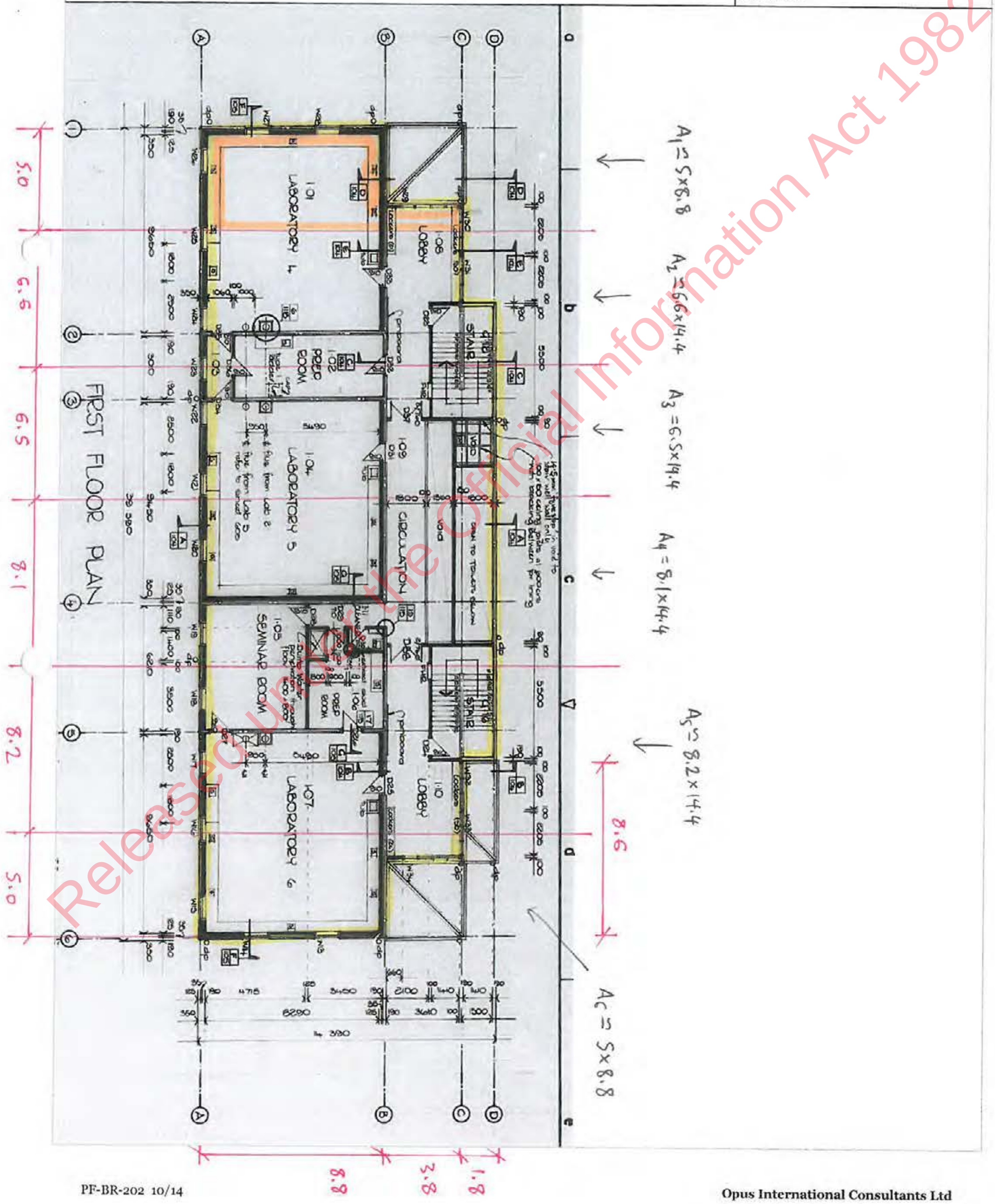

CALCULATION SHEET

Project/Task/File No: Sheet No 12 of

Project Description: Office:

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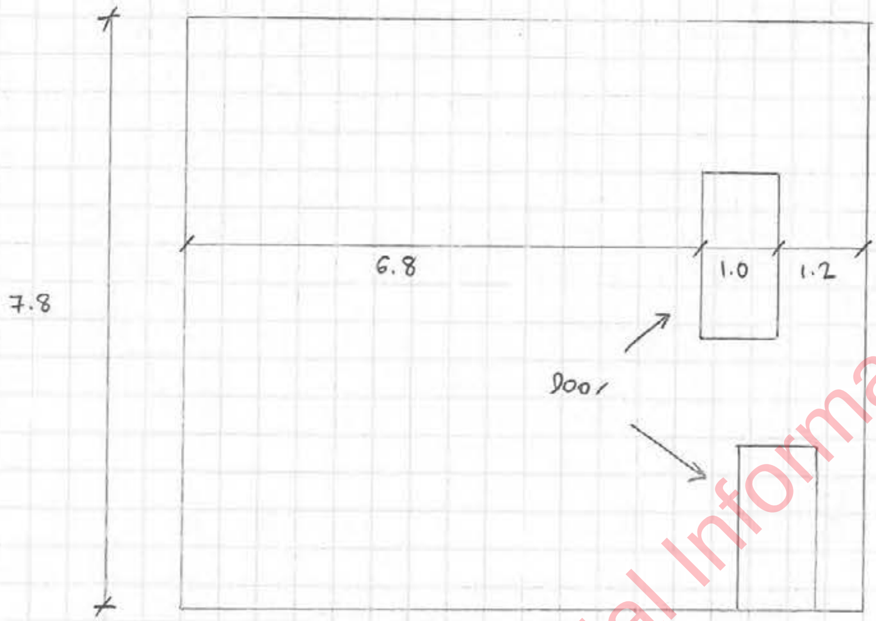
Transverse Internal Walls - Gridlines 2 and 3



D12 @ 400 vertical
D16 @ 400 horizontal

$f'_m = 12$ MPa

$f_y = 300 \times 1.08 = 325$ MPa (NZSEE)

Treat as a 6.8m wide wall.

Shear

$v_{bm} = 0.7$ MPa (Nominally ductile)

$p_w = \frac{A_s}{b_w s} = \frac{\pi \times 12^2/4}{190 \times 400} = 0.0015$

$C_1 = 33 p_w \frac{f_y}{300} = 0.053$

$h_e = 7.8$ m

$L_w = 6.8$ m

$\frac{h_e}{L_w} > 1$

$C_2 = 1$

$v_m = (C_1 + C_2) v_{bm} = 0.74$ MPa

$A_v = \frac{\pi \times 16^2}{4} = 201$ mm²

OPUS

CALCULATION SHEET

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Check: / /

$V_s = 0.8 \frac{A_v f_y}{b_w s} = 0.8 \times \frac{201 \times 325}{190 \times 400} = 0.69 MPa$

Take $V_p = 0$

$V_n = V_m + V_p + V_s = 0.74 + 0 + 0.69 = 1.43 MPa$

$\phi = 0.85$ (NZSEE)

$\phi V_n = \phi v_n \times b_w \times 0.8 L_w = 0.85 \times 1.43 \times 190 \times 0.8 \times 6800 = 1256 kN$

Walls 2 and 3 take 18.6% of overall demand each

$V^* = 0.186 \times 4672 = 869 kN$

$\% NBS_{walls\ 2\ and\ 3} = \frac{1256}{869} = 144\%$

<u>Transverse Internal Wall - Gridline 4</u>

No window or door openings.

$h_e = 7.8m$

$L_w = 9.0m$

D12 @ 400 horizontal
D16 @ 400 vertical

$V_n = 1.43 MPa$

$\phi V_n = \phi v_n b_w 0.8 L_w = 1663 kN$

22.7% of demand on Gridline 4

$V^* = 0.227 \times 4672 = 1061 kN$

$\% NBS_{wall\ 4} = \frac{1663}{1061} = 156\%$

OPUS

CALCULATION SHEET

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Transverse Internal Wall - Gridline 5

Identical wall to gridlines 2 and 3.

Shear

$\phi V_n = 1256 \text{ kN}$

23.1% of demand on Gridline 5.

$V^* = 0.231 \times 4672 = 1079 \text{kN}$

$\% NBS_{wall\ 5} = \frac{1256}{1079} = 116\%$

OPUS

CALCULATION SHEET

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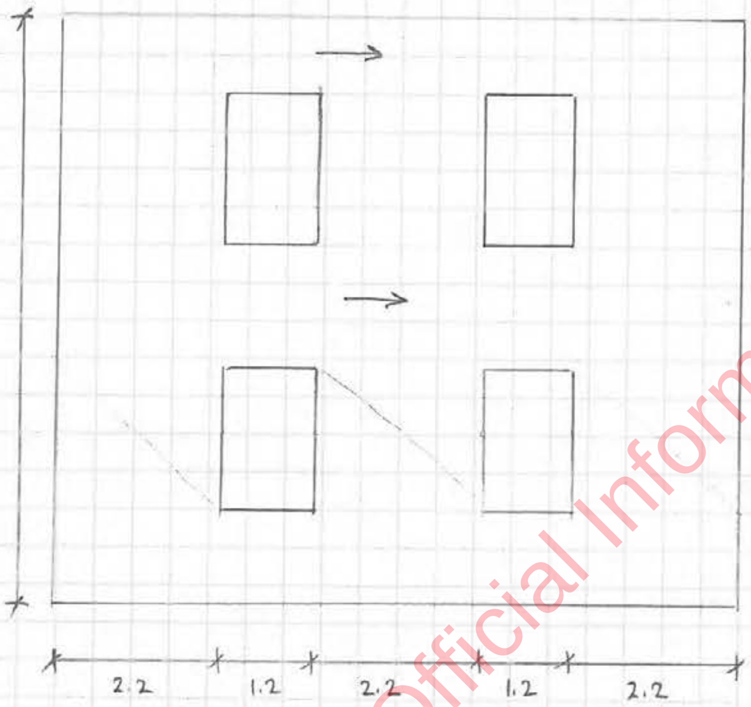
Transverse External Walls - Gridlines 1 and 6



$V_n = 1.43 MPa$

$\phi V_n = \phi v_n \times b_w \times 0.8 L_w = 0.85 \times 1.43 \times 190 \times 0.8 \times 2200 = 406 kN$

8.6% of load on Gridline 1/6

$V^* = 0.086 \times 4672 \times \frac{1}{3} = 134 kN$

% NBS walls 1 and 6 $= \frac{406}{134} = 303\%$

OPUS

CALCULATION SHEET

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Distribution of Load to Longitudinal Walls

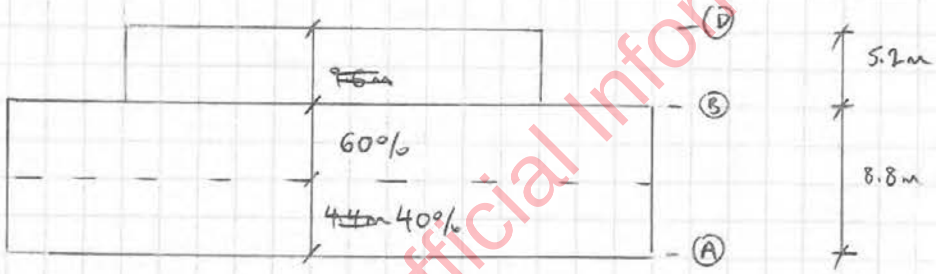
Load is resisted by walls on gridlines A, B and D primarily.

Wall on grid D has little or no tie to the first floor slab

⇒ attracts little load.

Ignore contribution of gridline D.

~~Distribute load onto A and B by tributary area.~~



$\frac{4.4}{14}$ ~~= 30% on GL A~~

~~70% on GL B~~

Gridline B (shear walls) stiffness > Gridline A (frame) stiffness

Assume Gridline A takes 40% of 8.8m

% Load GL A = $\frac{0.4 \times 8.8}{14}$ = 25%

% Load GL B = $\frac{5.2 + 0.6 \times 8.8}{14}$ = 75%

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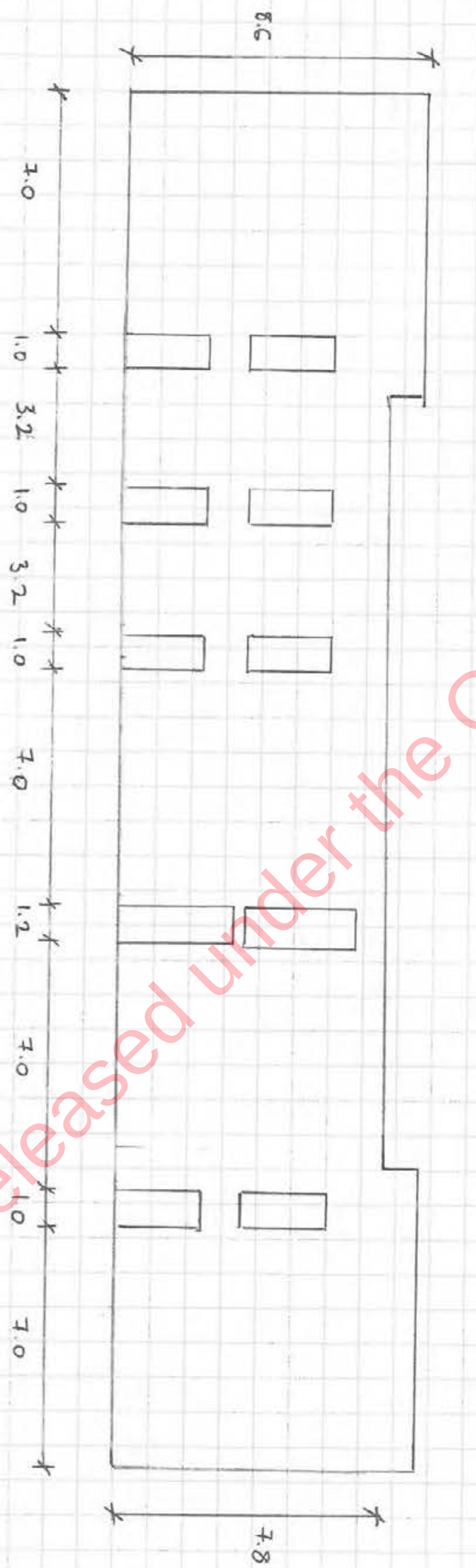
Longitudinal Wall – Gridline B



D12 at 400 centres vertical, D16 around openings

D12 at 400 centres horizontal

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CALCULATION SHEET

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Analyse as 6 walls.

Distribute load according to their stiffness.

Using cracked sections, the load distribution is obtained from L^2.

$\Sigma L^2 = 4 \times 7^2 + 2 \times 3.2^2 = 216.48$

% Load on 7m wall = $7^2 / 216.48 = 22.6\%$

% Load on 3.2m wall = $3.2^2 / 216.48 = 4.7\%$

<u>7m walls</u>

<u>Shear</u>

$V_{bm} = 0.7MPa$ (Nominally ductile)

$V_m = V_{bm}$ conservatively

$A_v = \frac{\pi \times 12^2}{4} = 113mm^2$

$V_s = 0.8 \frac{A_v f_y}{b_w s} = 0.8 \times \frac{113 \times 325}{190 \times 400} = 0.39MPa$

Take $v_p = 0$

$v_n = 0.7 + 0.39 = 1.09MPa$

$\phi V_n = \phi v_n \times b_w \times 0.8 L_w = 0.85 \times 1.09 \times 190 \times 0.8 \times 7000 = 986kN$

Gridline B takes 75%

$V^* = 4672 \ (\mu=1.25) \times 0.75 \times 0.226 = 792 kN$

$\%NBS_{7m\ walls} = \frac{986}{792} = 124\%$ NBS

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CALCULATION SHEET

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Check: / /

3.2m walls

$\phi V_n = 0.85 \times 1.09 \times 190 \times 0.8 \times 3200 = 451\,kN$

$V^* = 4672 \times 75\% \times 4.7\% = 165\,kN$

$\%NBS_{3.2m\ walls} = \frac{451}{165} = 273\%$

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CALCULATION SHEET

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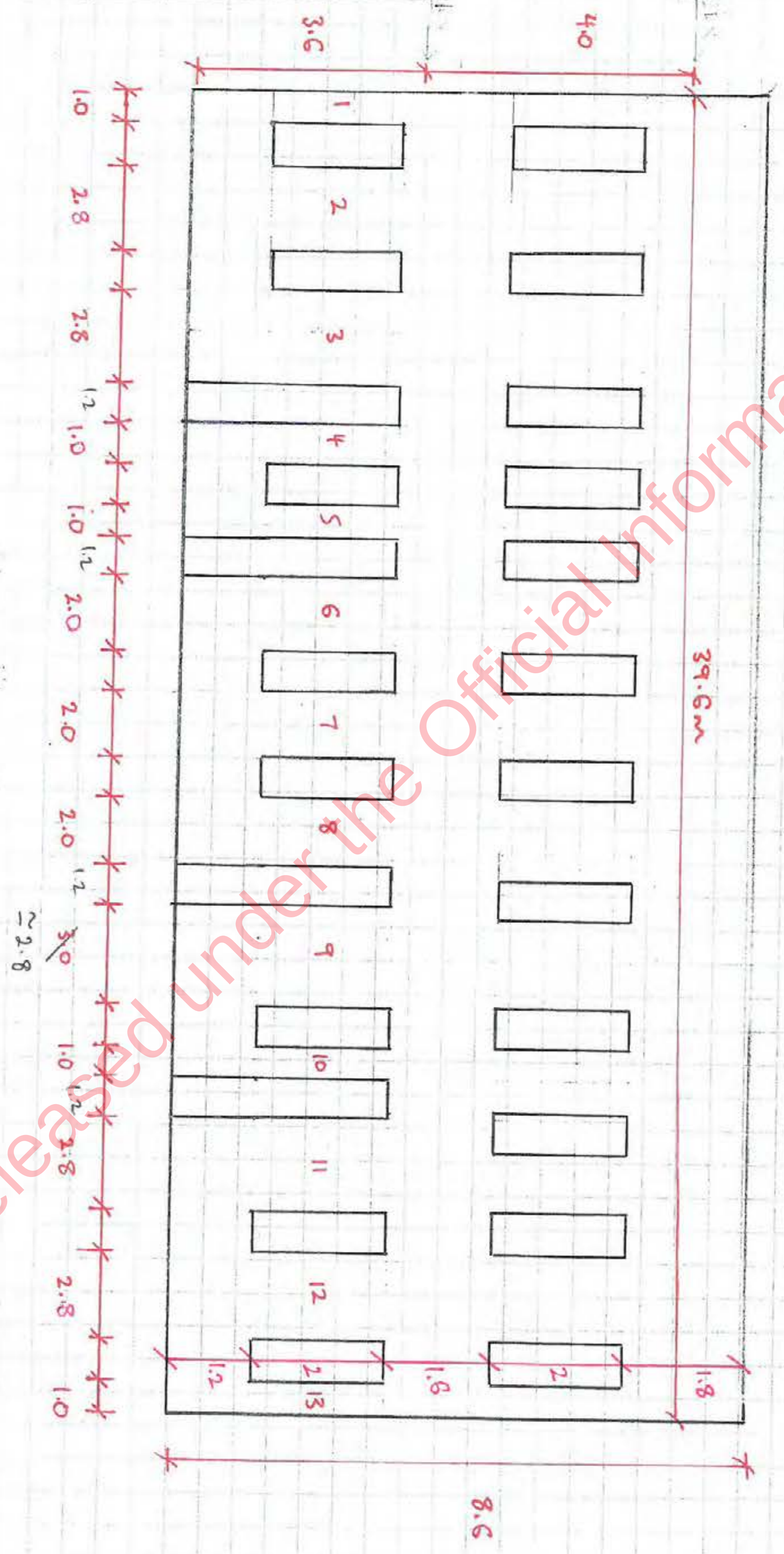
Longitudinal Wall - Gridline A



Note: Height drawn to double scale.

Loads

$F_2 = 25\% \times 2232 = 558kN$

$F_1 = 25\% \times 2440 = 610kN$

CALCULATION SHEET

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Check: / /

Top Spandrel Checks (1.8m deep)

D12 @ 400 vert
D16 @ 400 horizontal

Shear

$A_v = \frac{\pi \times 12^2}{4} = 113 mm^2$

$V_s = 0.8 \frac{A_v f_y}{b_w s} = 0.8 \times \frac{113 \times 325}{190 \times 400} = 0.39 MPa$

$V_m \approx V_{bm} = 0.7 MPa$

$V_n = V_m + V_p + V_s = 0.7 + 0 + 0.39 = 1.09 MPa$

$\phi V_n = \phi v_n b_w \times 0.8 L_w = 0.85 \times 1.09 \times 190 \times 0.8 \times 1800 = 253 kN$

$V^* = 71 kN$

%NBS top spandrel shear $= \frac{253}{71} = 356\%$

Flexure

$\phi M_n = M_n = 271 kNm$

$M^* = 128 kNm$ conservatively $(\mu = 1.25)$

%NBS top spandrel flexure $= \frac{271}{128}$

CALCULATION SHEET

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Check: / /

Bottom Spandrel (First Floor) (1.6m deep)

D12 @ 400 vert
D16 @ 400 horiz

Shear

$\phi V_n = \phi v_n b_w 0.8 L_w = 0.85 \times 1.09 \times 190 \times 0.8 \times 1600 = 225$ kN

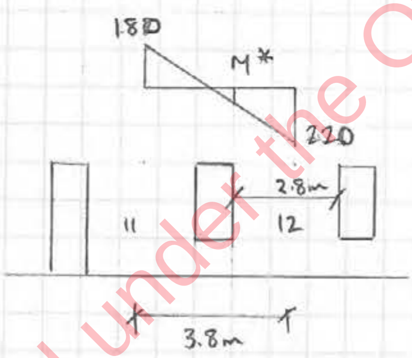
$V^* = 121$ kN

$\%NBS_{\text{bottom spandrel shear}} = \frac{225}{121} = 186\%$ NBS

Flexure

$\phi M_n = M_n = 166$ kNm

$M^* = 220$ kNm. Reduce to face of window.



$M^* = 220 - \left[(220 + 180) \times \frac{1.4}{3.8}\right] = 73$ kNm

$\%NBS_{\text{bottom spandrel flexure}} = \frac{166}{73} = 227\%$

CALCULATION SHEET

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2.8m Pier Checks

D12 @ 400 both ways

Shear

$v_n = 1.09 \text{ MPa}$

$\phi V_n = \phi v_n b_w \times 0.8 L_w = 0.85 \times 1.09 \times 190 \times 0.8 \times 3800 = 535 \text{ kN}$

$V^* = 307 \text{ kN}$

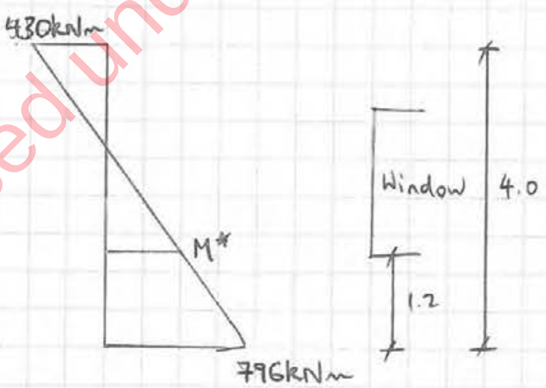
$\% NBS_{2.8m \text{ pier shear}} = \frac{535}{307} = 174\%$

Flexure

Since the wall is 40m long the axial forces due to lateral load are not large - of a similar magnitude to the gravity load.

The effect of axial load on flexural strength of piers will be insignificant.

$\phi M_n = M_n = 318 \text{ kNm}$



$M^* = 796 - \left[(796+430) \times \frac{1.2}{4}\right] = 428 \text{ kNm}$ (If $\mu = 1.25$)

Pro rata M^* for $\mu = 2$

$\mu = 1.25$: $\frac{S_p}{k_\mu} = \frac{0.9}{1.143} = 0.787$

$\mu = 2$: $\frac{S_p}{k_\mu} = \frac{0.7}{1.57} = 0.445$

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CALCULATION SHEET

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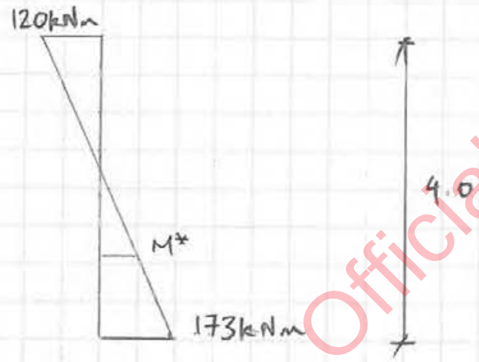
$M^* = 428 \times \frac{0.445}{0.787} = 242 kNm$

$\% NBS_{2.8m}$ pier flexure $= \frac{318}{242} = 131\%$

2.0m Pier Checks

D12 @ 400 both ways

$\phi M_n = M_n = 156 kNm$



Refer sheet 23

$M^* = 173 - \left[(173+120) \times \frac{1.2}{4}\right] = 85 kNm$ (If $\mu = 1.25$)

By inspection 2.8m piers are critical (based on elastic stiffnesses)

Conservative as pier loads could be distributed using cracked properties - L^2

>100%NBS using elastic stiffnesses ✓ok

CALCULATION SHEET

Project/Task/File No: .... Sheet No 25 of

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Check: / /

Check of Detailing Requirements for Ductile Walls to NZS4230

NZS4230, Section 7.4 sets out the requirements for ductile wall design.

Check Science Block walls meet these requirements.

7.4.4.1

Less than 3 stories ✓ok

7.4.4.2

Wall thickness = 190mm > 140mm ✓ok

190mm > $0.05 L_n$ = 0.05 × 3600 = 180 mm ✓ok

7.4.4.3

Shortest wall L_w = 1000mm > L_w = 790mm ✓ok

7.4.5.1 Vert Reo

D12 vert bars ✓ok

400mm spacing ✓ok

Min. 4 vert bars per wall

- 1000mm walls have 3 bars ✗NG
- All other walls have >4 bars ✓ok

7.4.5.2 Horiz Reo

400mm spacing ✓ok

7.4.5.4 Lap Splices in PHR

f_y = 300MPa

D12

$60 d_b$ = 720 mm

Lap length = 500 mm ✗ NG

CALCULATION SHEET

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Summary

Walls meet the majority of detailing requirements for ductile design.

Considered adequate for assessment purposes.

Main issue is lap length in PHR.

Consider basic development length (6.3.7.3 NZS 4230)

$L_{db} = 40 d_b = 40 \times 12 = 480mm < 500mm$ ✓ OK.

⇒ Assess as 'ductile' walls

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CALCULATION SHEET

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Section B Check: / /

Section B

Note on foundation assessment:

Variations from the assumed pile configuration are unlikely to result in the rating going below the current rating of 76% NBS.

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CALCULATION SHEET

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Check: / /

Foundations

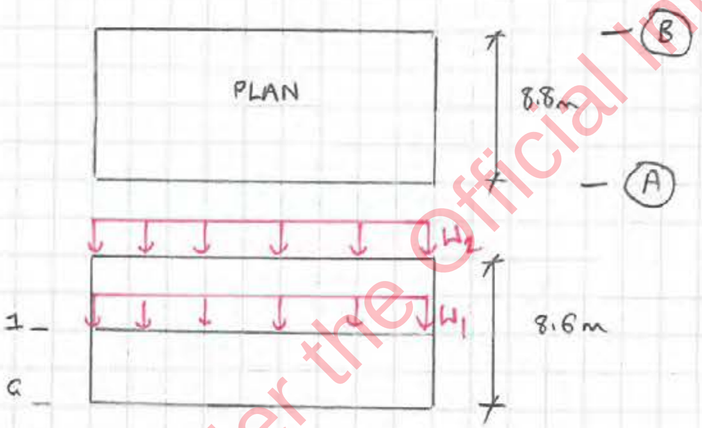
Longitudinal Loading - Gridline A

Assume that the ground profile at GL A is 4m of fill underlain by rock.

This is the worst case as the piles are up to 5m long with min. 1m embedment into rock.

Assume that the rock profile is level in the long direction in the absence of information.

Axial load on piles



w_2 = roof weight = 0.4 kPa × 4.4m = 1.8 kN/m

w_1 = floor weight = (2.9 + 0.3 × 3) kPa × 4.4 = 16.7 kN/m

Self weight of wall = 22kN/m³ × 8.6 × 0.19 = 35.9 kN/m

Sum = 54.4 kN/m

Pile spacing = 3.2m

Gravity load = 54.4 × 3.2 = 174 kN

Worst case seismic load on pier = 55 kN tension

Apply to one pile conservatively.

$N^* = 174 - 55 = 119$ kN

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CALCULATION SHEET

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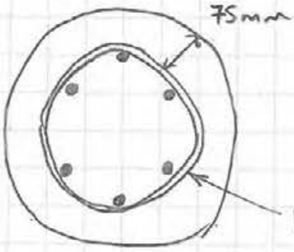
Shear capacity



$d'' = 500 - 75 \times 2 - 12 = 338mm$

$A_{sp} = \frac{\pi \times 12^2}{4} = 113 mm^2$

$V_s = \frac{\pi}{2} \frac{A_{sp} f_{yt} d''}{s} \cot 30° = \frac{\pi}{2} \times \frac{113 \times 325 \times 338}{150} \times \cot 30 = 225 kN$

$\alpha = 1$ (M = 0 at top)

Longitudinal reo is 8-D24 bars

$\rho_l = \frac{A_s}{A_g} = \frac{(8 \times \pi \times 24^2/4)}{(\pi \times 500^2/4)} = 0.018$

$\beta = 0.5 + 20\rho_l = 0.87$

$k = 0.29 \alpha \beta = 0.25$

$V_c = k \sqrt{f'_c}\, 0.8 A_g = 0.25\sqrt{30} \times 0.8 \times \frac{\pi \times 500^2}{4} = 217 kN$

Take $V_n = 0$

$V_p = 0.72 (V_c + V_s + V_n) = 318 kN$

OPUS

```
General Information:
====================
  File Name: p:\projects\5-pa010.00 moe seismic panel agreement to assess public buildings...\pile.col
  Project:   WEGC Science Block
  Column:    Pile                               Engineer: MJG
  Code:      ACI 318-11                         Units: Metric

  Run Option: Investigation                     Slenderness: Not considered
  Run Axis:   X-axis                            Column Type: Structural

Material Properties:
====================
  f'c    = 30 MPa                               fy    = 325 MPa
  Ec     = 25084 MPa                            Es    = 200000 MPa
  Ultimate strain = 0.003 mm/mm
  Beta1 = 0.85

Section:
========
  Circular:    Diameter = 500 mm

  Gross section area, Ag = 196350 mm^2
  Ix =  3.06796e+009 mm^4                       Iy =  3.06796e+009 mm^4
  rx =  125 mm                                  ry =  125 mm
  Xo =  0 mm                                    Yo =  0 mm

Reinforcement:
==============
  ar Set: ASTM A615
  Size Diam (mm) Area (mm^2)   Size Diam (mm) Area (mm^2)   Size Diam (mm) Area (mm^2)
  ---- --------- -----------   ---- --------- -----------   ---- --------- -----------
  #  3       10          71    #  4       13         129    #  5       16         200
  #  6       19         284    #  7       22         387    #  8       25         510
  #  9       29         645    # 10       32         819    # 11       36        1006
  # 14       43        1452    # 18       57        2581

  Confinement: Other; #4 ties with #8 bars,  #4 with larger bars.
  phi(a) = 0.85,  phi(b) = 1,  phi(c) = 0.75

  Layout: Circular
  Pattern: All Sides Equal  (Cover to transverse reinforcement)
  Total steel area: As = 4077 mm^2 at rho = 2.08%
  Minimum clear spacing = 89 mm

  8 #8    Cover = 75 mm

Axial Load and Corresponding Moment Capacities:
===============================================
  Load       PhiPn      PhiMnx  NA depth Dt depth    eps_t    Phi
   No.          kN         kNm        mm       mm
  ---- ------------ ------------ --------- --------- -------- ------
     1       119.0      238.51       130      400  0.00625  1.000
                       -238.51       130      400  0.00625  1.000

 ** End of output ***
```

$\phi M_n = M_n = 239$ kNm

CALCULATION SHEET

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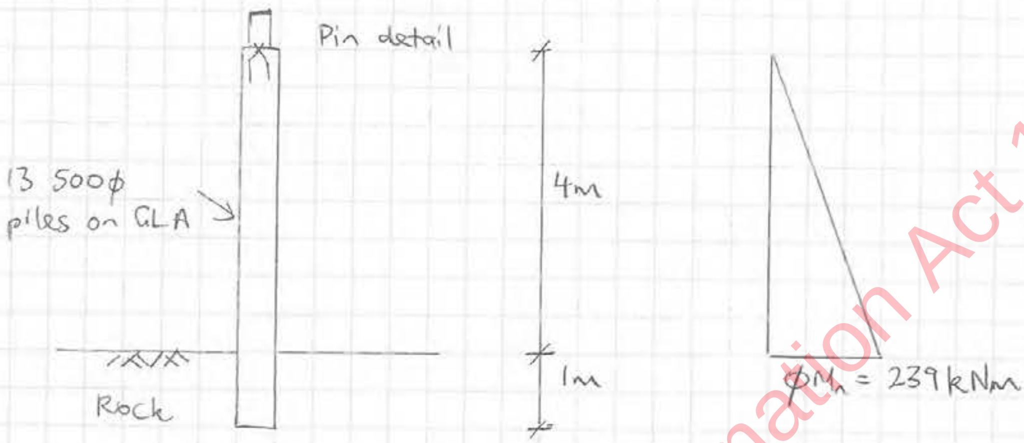
Check: / /



Lateral capacity in flexure = $\phi M_n / 4$ = 60 kN

Lateral capacity in shear = V_p = 318 kN

⇒ Flexure governs.

Demand on GL A ($\mu = 2$) = 25% x 2643 = 661 kN

Add demand from additional weight not included in equivalent static distribution

No amplification → PGA

C(0) = 0.52 (Sheet B13)

1st storey height = 3.6 m

Extra weight (GL A) = 22 kN/m² x $\frac{3.6}{2}$ x 39.4 x 0.2 = 312 kN

0.52 x 312 = 162 kN

Total demand = 661 + 162 = 823 kN

Demand per pile = $\frac{823}{13}$ = 63 kN

M^* = 63 x 4 = 252 kNm

% NBS piles gridline A = $\frac{239}{252}$ = 94%

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Check degradation of shear strength in PHR

Fig 7.7(b) NZSEE 2006

Take $k = 0.10$ conservatively

$V_p = \frac{0.1}{0.29} \times 318 = 110\text{kN} > V^* = 63\text{ kN}$ ✓OK

Longitudinal Loading – Gridline B

The rock is shallower under GL B than GL A.

Assume 13 piles. Assume level rock profile.

If governed by flexure, >100% NBS.

Could be shear governed.

$V^* = \sout{75}\% \times \frac{4672}{13} = 269\text{ kN}$ per pile

$V_p = 318\text{kN}$

% NBS piles gridline B >100%

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CALCULATION SHEET

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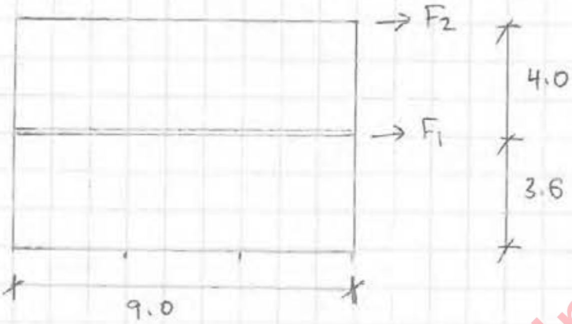
Transverse Loading

Check heaviest loaded wall which is Gridline 5. 23.1% of load.

Wall Rocking Capacity



Overturning moment = 23.1% x (2232 x 7.6 + 2440 x 3.6) = 5 948 kNm

Resisted by self weight that is concentrated at the load bearing longitudinal wall on Gridline B, and by piles on Gridline A bearing (or reverse).

⇒ Lever arm = 9m

$T = C = \frac{5948}{9} = 661\ kN$

Check bearing:

Resisted by 3 piles.

$A = 3 \times \frac{\pi \times 0.5^2}{4} = 0.589\ m^2$

Bearing pressure $= \frac{661}{0.589} = 1100\ kPa$

Considered OK for pile end bearing onto rock.

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Check: / /

Check self weight:

Refer sheet B1

9.1m (trib width) of longitudinal wall resisting, 54.4 kN/m

$54.4 \times 9.1 = 495 \text{ kN}$

Add weight of 3 piles

$3 \times \frac{\pi \times 0.5^2}{4} \times 5 \times 24 = 71 \text{ kN}$

$\% NBS_{\text{transverse wall rocking}} = \frac{(495+71)}{661} = 86\%$

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CALCULATION SHEET

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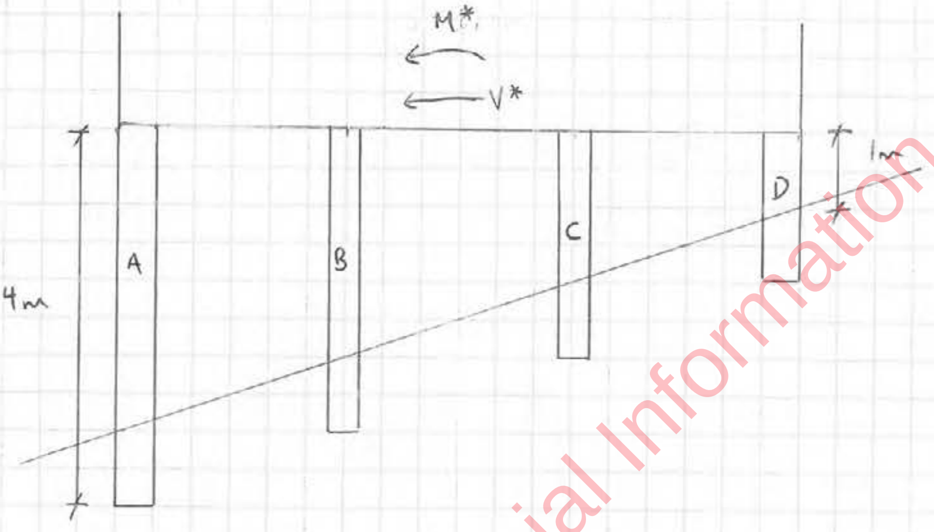
Check: / /

Transvese Loading - Pile Capacity   Gridline 5.

Assume a linear rock profile from 1m to 4m depth below wall



V_p = 318 kN

ϕM_n in the order of 239 kNm (Sheet 83)

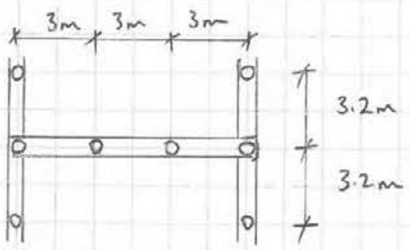
Lever arm = 1m

⇒ Flexure governs. Adopt $\mu = 2$

V* = 23.1% × 2 643 = 611 kN

M* = 23.1% × 14 563 = 3 364 kNm   (Sheet 9)

Axial load



CALCULATION SHEET

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Check: / /

Grav load piles B and C = weight of transverse wall

$= 22 kN/m^3 \times 7.6 \times 3 \times 0.2 = 75 kN$

Grav load piles A and D = wt of transverse + wt. from longitudinal

= 75/2 + 174 (Sheet B2)

= 212 kN

Seismic axial load

Refer sheet B6

$T = C = \frac{3364}{9} = 374 kN$

Resisted by 3 piles.

$T = C$ $374/3 = 125 kN$

Net axial load

Pile A : 212 + 125 = 337 kN

Pile B : 75 kN

Pile C : 75 kN

Pile D : 212 − 125 = 87 kN

General Information:
====================
File Name: p:\projects\5-pa010.00 moe seismic panel agreement to assess public buildings...\pile.col
Project: WEGC Science Block
Column: Pile   Engineer: MJG
Code: ACI 318-11   Units: Metric

Run Option: Investigation   Slenderness: Not considered
Run Axis: X-axis   Column Type: Structural

Material Properties:
====================
f'c = 30 MPa   fy = 325 MPa
Ec = 25084 MPa   Es = 200000 MPa
Ultimate strain = 0.003 mm/mm
Beta1 = 0.85

Section:
========
Circular: Diameter = 500 mm

Gross section area, Ag = 196350 mm^2
Ix = 3.06796e+009 mm^4   Iy = 3.06796e+009 mm^4
rx = 125 mm   ry = 125 mm
Xo = 0 mm   Yo = 0 mm

Reinforcement:
==============
Bar Set: ASTM A615

Size	Diam (mm)	Area (mm^2)	Size	Diam (mm)	Area (mm^2)	Size	Diam (mm)	Area (mm^2)
# 3	10	71	# 4	13	129	# 5	16	200
# 6	19	284	# 7	22	387	# 8	25	510
# 9	29	645	# 10	32	819	# 11	36	1006
# 14	43	1452	# 18	57	2581			

Confinement: Other; #4 ties with #8 bars, #4 with larger bars.
phi(a) = 0.85, phi(b) = 1, phi(c) = 0.75

Layout: Circular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area: As = 4077 mm^2 at rho = 2.08%
Minimum clear spacing = 89 mm

8 #8 Cover = 75 mm

Axial Load and Corresponding Moment Capacities:
===

Load No.	PhiPn kN	PhiMnx kNm	NA depth mm	Dt depth mm	eps_t	Phi
1	75.0	232.73	127	400	0.00643	1.000
		-232.73	127	400	0.00643	1.000
2	87.0	234.31	128	400	0.00638	1.000
		-234.31	128	400	0.00638	1.000
3	337.0	266.40	142	400	0.00543	1.000
		-266.40	142	400	0.00543	1.000

*** End of output ***

CALCULATION SHEET

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$k_{pile} = \frac{3EI}{L^3}$

$\Sigma k = \left(\frac{1}{1^3} + \frac{1}{2^3} + \frac{1}{3^3} + \frac{1}{4^3}\right) 3EI = (1.18)\, 3EI$

<u>Distribution of load - elastic</u>

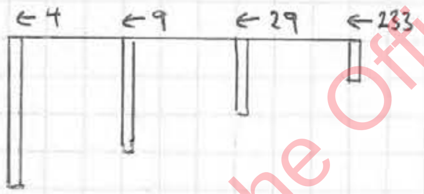
Pile A $\frac{1/4^3}{1.18} = 1.3\%$

B 3.1%

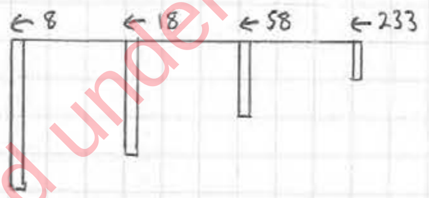
C 10.6%

D 84.7%

Pile D yields. $\phi M_n = 233\,kNm$. Total elastic capacity $= \frac{233}{0.847} = 275\,kN$



Elastic



$\mu = 2$

Total capacity = 8 + 18 + 58 + 233 = 317 kN

$\frac{317}{611} = 52\%$

Consider a ductility capacity of $\mu > 2$.

Check NZSEE 2006, 7-15.

Spiral spacing = 150mm

d = 373 mm

d/2 = 187mm > 150mm ✓OK

OPUS

CALCULATION SHEET

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$16 d_b = 16 \times 24 = 384$ mm > 150 mm ✓OK

⇒ μ can be taken as >2 but less than 6 as $\frac{d}{4} = 93$mm ≯ 150mm

Set μ = 3

Pro rata demand: $611 \times \frac{1.571}{2.143} = 448$ kN

Capacity = $(4+9+29) \times 3 + 233 = 359$ kN

% NBS piles transverse $= \frac{359}{448} =$ ~~80% NBS~~ Add demand from retaining wall

Additional demand = 70kN elastic (sheet B19)

Total demand $= 70 \times \frac{0.7}{2.143} + 448 = 471$ kN

% NBS piles transverse $= \frac{359}{471} = 76$ % NBS

OPUS

CALCULATION SHEET

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Diaphragm Connection Checks

Floor Diaphragm

Connection to Transverse Walls

Connection detail is given by section K-K of sheet 204.

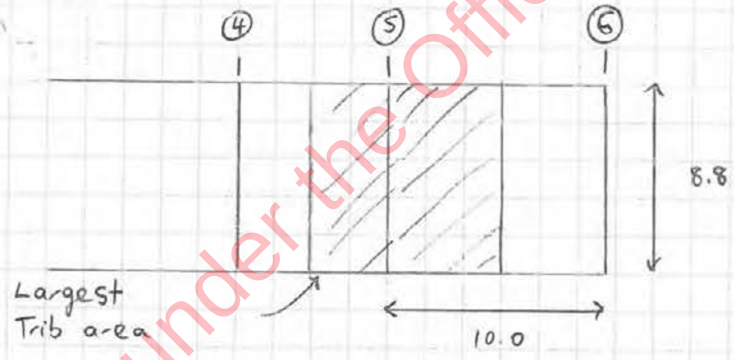
90mm thick slab

D12 L bars at 200 centres from slab to wall, alternating

(400 centres each side of wall)

Reo is anchored with adequate development (500mm)

Analyse by shear friction.



$A_{vf} = \frac{\pi \times 12^2}{4} \times \frac{8800}{400} = 2488 \text{ mm}^2$

$V_n = (A_{vf} f_y + N^*) \mu = (2488 \times 325 + 0) \times 1.4 = 1132 \text{ kN}$

$\phi = 0.85$ (NZSEE)

$\phi V_n = 962$ kN per side

$\phi V_n = 1924$ kN overall

Conservatively apply the shear demand which includes whole weight of structure, not floor only.

$V^* = 1071$ kN (Sheet 13) ✓OK

OPUS

CALCULATION SHEET

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Or compare with wall shear strength

ϕV_n wall = 1256 kN (Sheet 17)

ϕV_n diaphragm connection = 1924 kN ✓OK

Connection to Longitudinal Walls

Connection detail is given by sections J-J and L-L on Sheet 204.

D12 at 400 centres (Line A)

D12 at 200 centres (Line B) (400 centres each side of wall)

Wall Line A

$A_{vf} = \frac{\pi \times 12^2}{4} \times \frac{39400}{400} = 11\,140 \text{ mm}^2$

$\phi V_n = 5\,068$ kN

$V^* = 30\% \times 4672 = 1402$ kN $\ll \phi V_n$ ✓OK

Wall Line B diaphragm connection OK by inspection.

Roof Diaphragm

Connection detail not known, however ceiling is within the height of the bond beams.

If diaphragm connection is inadequate, load can be transferred indirectly to in-plane walls by bond beam acting out of plane.

Retaining Walls

Retaining Wall (Blockwork)

External wall along Gridline D of Science Block, between Gridlines 5 & 6, is retaining soil.

Check for seismic actions.

2.2m blockwork on 0.1m concrete upstand.

Site visit confirmed that wall is retaining ~2.3m of soil.

Flexural Strength

Self weight = 22kN/m³ × 2.3 × 0.19 = 9.6kN/m

Refer section G-G, Sheet 203

Reo is D20 at 400 centres, 50mm cover.

2.5 bars per metre

$\phi = 1$

$\phi M_n = M_n = 31$ kNm/m (Sheet C2)

Check development length

Actual L_{db} = 800mm

Required L_{db} = $40d_b$ = 800mm ✓OK

Analysis

Wall has a good sized footing that is tied into the floor slab.

Expected failure mechanism is wall flexure.

Assume $\gamma = 18$kN/m³

$\phi = 30°$

CALCULATION SHEET

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Wall has free draining backfill and Novaflo drain - no water pressure.

No surcharge in seismic case.

REFERENCE

Reference has been made to Opus "T-CEP 702 Retaining Wall Design Notes"

Seismic loads are derived from NZS1170.5 ~~and NZTA Bridge Manual Edition 3 (SP/M/022)~~

Loads

IL3

$R_u = 1.3$

Soil class B

$C_h(0) = 1.00$

$Z = 0.4$

$N(T,D) = 1$

$C(0) = C_h(0) Z R_u N(T,D) = 0.52$

Analysis

Analyse as a rigid wall since wall is restrained at top by bldg roof.

Then the earthquake pressure is given by

$$\Delta P_E = C(0)\,\gamma H^2$$

Applied at approximately 0.6 H above the base.

For slope stability of vertical slopes the critical curved failure surface gives a result very similar to the critical planar failure surface.

⇒ Reasonable to assume a planar failure surface.

CALCULATION SHEET

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Assume wall angle of friction, $\delta = \phi = 30°$

Wall angle, $\alpha = 0°$ (vertical)

Backfill angle, $i = 0°$ (Flat)

$$K_A = \frac{\cos^2(\phi - \alpha)}{\cos^2\alpha \cos(\alpha + \delta)\left[1 + \sqrt{\frac{\sin(\phi+\delta)\sin(\phi - i)}{\cos(\alpha+\delta)\cos(i-\alpha)}}\right]^2}$$

$K_A = 0.297$

$P_A = \frac{1}{2} K_A \gamma H^2 = \frac{1}{2} \times 0.297 \times 18 \times 2.3^2 = 14.1\ kN/m$

$\Delta P_E = C(0)\gamma H^2 = 0.52 \times 18 \times 2.3^2 = 49.5\ kN/m$

$M^* = \frac{2.3}{3} \times 14.1 + 0.6 \times 2.3 \times 49.5 = 79.1\ kNm/m$ as a cantilever

$\phi M_n = 31.0\ kNm/m$

However, the wall is restrained by transverse walls and by roof.

Consider the wall as a propped cantilever.

$M^* = 25.1\ kNm/m$ (Sheet C23)

% NBS blockwork ret wall $= \frac{31.0}{25.1} = 123\%$ NBS

OPUS

CALCULATION SHEET

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Check transverse walls for additional load from retaining wall:

Approx. 6m of wall retaining.

Top of wall reaction = 23.4 kN/m (sheet C21)

Demand = 23.4 x 7 = 140 kN

Taken by walls on Grids 5 and 6

Say 100%, 140kN taken by grid 5 conservatively

$V^* = 1079 + 140 = 1219$ kN

$\phi V_n = 1256$ kN ✓ok

OPUS

CALCULATION SHEET

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Concrete Retaining Wall

Up to 2.3m high beside building. Joins to blockwork wall.

Flexural Strength

Reo is D20 at 400 centres, 50mm cover.

Assume $f'_c = 30MPa$

Self weight = $24kN/m^3 \times 2.3 \times 0.19 = 10.5kN/m$

Check development length

Actual $L_{db} = 700mm$

Required $L_{db} = \frac{0.5 \alpha_1 \alpha_2 f_y}{\sqrt{f'_c}} d_b = \frac{0.5 \times 1 \times 325}{\sqrt{30}} \times 20 = 593mm$ ✓OK

$\phi = 1$

$\phi M_n = M_n = 33kNm/m$ (Sheet C4)

Analysis

$c(0) = 0.52$

Wall is cantilevering - 'flexible' wall

⇒ Analyse by Coulomb sliding wedge theory.

FE studies and tests have shown that the increment of EQ force acts at approx 0.33H above the base of the wall. (Opus Ret Wall Manual)

(Same location as static pressure)

Horizontal accel., $k_h = c(0) = 0.52$

Vert accel, k_v taken as 0.

$\theta = \tan^{-1}\left(\frac{k_h}{1-k_v}\right) = 27.5° = 0.48 rad$

OPUS

CALCULATION SHEET

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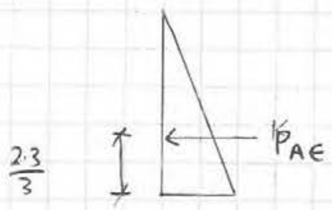
$$K_{AE} = \frac{\cos^2(\phi - \theta - \alpha)}{\cos\theta \cos^2\alpha \cos(\alpha + \delta + \theta)\left[1 + \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta-i)}{\cos(\alpha+\delta+\theta)\cos(i-\alpha)}}\right]^2}$$

Where ϕ, α and i are same as given on sheet B14

$K_{AE} = 1.049$

This includes the static thrust.



$P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 = \frac{1}{2} \times 1.049 \times 18 \times 2.3^2 = 49.9$ kN/m

$M^* = \frac{2.3}{3} \times 49.9 = 38.3$ kNm/m

$\phi M_n = 33.0$ kNm/m

% NBS$_{\text{concrete ret wall}}$ $= \frac{33}{38.3} = 86\%$ NBS at IL3

Since failure of this wall will not affect the building, it could be considered IL2, in which case it would be >100% NBS.

OPUS

WELCOME TO CONPROP(V 1.8) ** AN EXCEL SPREADSHEET FOR ANALYSING CONCRETE SECTIONS FOR FLEXURE UNDER UNCRACKED, CRACKED AND ULTIMATE CONDITIONS, IN ACCORDANCE WITH NZS 3101.

OPUS

Project:	**WEGC Science Block**		
Computed:	**M Geddes**		
		Date: 17-Sep-15	Time: 16:03

STEP 1 ..... Describe the Uncracked Section...
(use consistent units.. e.g. N and mm through out the spreadsheet)

Total Section depth (d) = ...	190	
Web width (w) = ...	1000	
Top flange width **excluding** web (b1) = ...	0	<----
Top flange thickness (t) = ...	0	THESE
Bottom flange width **excluding** web (b2) = ...	0	6 values
Bottom flange thickness (b) = ...	0	may
Axial compressive load (P) and, ...	9,600	be
Depth from top surface of this load (di)	95	zero
Assumed tensile cracking stress (f't)	0	<-----
Steel Elastic Modulus (Es)	200,000	

STEP 2 ..... Describe steel sizes and locations.................
describe location of the centroid of up to 10 bar bundles from either the top or the bottom surface. Describe Location of each bundle from only one surface.

Modular ratio (n=Es*(1+Ct)/Ec) =	11

TOP BARS			BOTTOM BARS		
No. Bars	Bar Diam	Distance From Top Surface	No of Bars	Bar Diam	Distance From Bottom Surface
3	20.00	60.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00

STEP 4 ..... Describe section properties for ULTIMATE conditions..........

(values entered in steps 1&2 may be varied for this part of the analysis.)

Concrete ultimate strain (e)	**0.003**
Ratio of (stress block)/(N.A.) depths....................	**0.85**
Axial compressive load (P) and, ...	**9,600**
depth from top surface of this load (di)	**95**
Crack root tensile stress (say 0.5f't)	**0.0**
Concrete Elastic Modulus (Ec)	**18,401**
Concrete compressive strength (f'c)....................	**12**
Steel Elastic Modulus (Es)....................	**200,000**
Steel Yield Stress (Fy)....................	**325**

Analysis results shown below correspond to the conditions that exist when the peak compression strain equals (e) given above. A rectangular stress block with average stress=0.85f'c is assumed.

RESULTS FOR ULTIMATE MOMENT SECTION ANALYSIS:

(a) CRACK PROPAGATING FROM BOTTOM

Depth to N.A.(zero stress) from top (c)....................	3.05E+01	Ratio T/C = 1.000 (=1.0 for iteration convergence)
Steel Stress (Maximum Tension)....................	3.25E+02	
Crack Depth	1.59E+02	
Total Tension Force (including P)....................	2.65E+05	
Total Compression Force -incl. comp steel....................	2.65E+05	
Nominal Flex strength (Mn)---SEE NOTE 2....................	1.28E+07	
Section Curvature (from curv = e/c)....................	9.82E-05	

(b) CRACK PROPAGATING FROM TOP

Depth to N.A.(zero stress) from bottom (c)....................	3.05E+01	Ratio T/C = 1.000 (=1.0 for iteration convergence)
Steel Stress (Maximum Tension)....................	3.25E+02	
Crack Depth	1.59E+02	
Total Tension Force (including P)....................	2.65E+05	
Total Compression Force -incl. comp steel....................	2.65E+05	
Nominal Flex strength (Mn)---SEE NOTE 2....................	3.07E+07	
Section Curvature (from curv = e/c)....................	9.82E-05	

WELCOME TO CONPROP(V 1.8) ** AN EXCEL SPREADSHEET FOR ANALYSING CONCRETE SECTIONS FOR FLEXURE UNDER UNCRACKED, CRACKED AND ULTIMATE CONDITIONS, IN ACCORDANCE WITH NZS 3101.

OPUS

Project:	**WEGC Science Block**	
Computed:	**M Geddes**	
	Date: 17-Sep-15	Time: 16:03

STEP 1 ..... Describe the Uncracked Section...
(use consistent units.. e.g. N and mm through out the spreadsheet)

Total Section depth (d) = ...	**190**	
Web width (w) = ...	**1000**	
Top flange width **excluding** web (b1) = ...	**0**	<----
Top flange thickness (t) = ...	**0**	THESE
Bottom flange width **excluding** web (b2) = ...	**0**	6 values
Bottom flange thickness (b) = ...	**0**	may
Axial compressive load (P) and, ...	**10,500**	be
Depth from top surface of this load (di)	**95**	zero
Assumed tensile cracking stress (f't)	**0**	<-----
Steel Elastic Modulus (Es)	**200,000**	

STEP 2 ..... Describe steel sizes and locations.................
describe location of the centroid of up to 10 bar bundles from either the top or the bottom surface. Describe Location of each bundle from only one surface.

Modular ratio (n=Es*(1+Ct)/Ec) =	8

TOP BARS			BOTTOM BARS		
No. Bars	Bar Diam	Distance From Top Surface	No of Bars	Bar Diam	Distance From Bottom Surface
3	20.00	60.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00

STEP 4 ..... Describe section properties for ULTIMATE conditions..........

(values entered in steps 1&2 may be varied for this part of the analysis.)

Concrete ultimate strain (e)	**0.003**
Ratio of (stress block)/(N.A.) depths........	**0.85**
Axial compressive load (P) and, ...	**10,500**
depth from top surface of this load (di)	**95**
Crack root tensile stress (say 0.5f't)	**0.0**
Concrete Elastic Modulus (Ec)	**25,084**
Concrete compressive strength (f'c)........	**30**
Steel Elastic Modulus (Es)........	**200,000**
Steel Yield Stress (Fy)........	**325**

Analysis results shown below correspond to the conditions that exist when the peak compression strain equals (e) given above. A rectangular stress block with average stress=0.85f'c is assumed.

RESULTS FOR ULTIMATE MOMENT SECTION ANALYSIS:

(a) CRACK PROPAGATING FROM BOTTOM

Depth to N.A.(zero stress) from top (c)........	1.23E+01	Ratio T/C = 1.000 (=1.0 for iteration convergence)
Steel Stress (Maximum Tension)........	3.25E+02	
Crack Depth	1.78E+02	
Total Tension Force (including P)........	2.66E+05	
Total Compression Force -incl. comp steel........	2.66E+05	
Nominal Flex strength (Mn)---SEE NOTE 2........	1.49E+07	
Section Curvature (from curv = e/c)........	2.45E-04	

(b) CRACK PROPAGATING FROM TOP

Depth to N.A.(zero stress) from bottom (c)........	1.23E+01	Ratio T/C = 1.000 (=1.0 for iteration convergence)
Steel Stress (Maximum Tension)........	3.25E+02	
Crack Depth	1.78E+02	
Total Tension Force (including P)........	2.66E+05	
Total Compression Force -incl. comp steel........	2.66E+05	
Nominal Flex strength (Mn)---SEE NOTE 2........	3.28E+07	
Section Curvature (from curv = e/c)........	2.45E-04	

WELCOME TO CONPROP(V 1.8) ** AN EXCEL SPREADSHEET FOR ANALYSING CONCRETE SECTIONS FOR FLEXURE UNDER UNCRACKED, CRACKED AND ULTIMATE CONDITIONS, IN ACCORDANCE WITH NZS 3101.

OPUS

Project:	**WEGC Science Block**	
Computed:	**M Geddes**	
	Date: 23-Sep-15	Time: 10:11

STEP 1 ..... Describe the Uncracked Section...
(use consistent units.. e.g. N and mm through out the spreadsheet)

Total Section depth (d) = ...	**2000**	
Web width (w) = ...	**190**	
Top flange width **excluding** web (b1) = ...	**0**	<----
Top flange thickness (t) = ...	**0**	THESE
Bottom flange width **excluding** web (b2) = ...	**0**	6 values
Bottom flange thickness (b) = ...	**0**	may
Axial compressive load (P) and, ...	**0**	be
Depth from top surface of this load (di)	**0**	zero
Assumed tensile cracking stress (f't)	**0**	<-----
Steel Elastic Modulus (Es)	**200,000**	

STEP 2 ..... Describe steel sizes and locations..................
describe location of the centroid of up to 10 bar bundles from either the top or the bottom surface. Describe Location of each bundle from only one surface.

Modular ratio (n=Es*(1+Ct)/Ec) =	**11**

TOP BARS			BOTTOM BARS		
No. Bars	Bar Diam	Distance From Top Surface	No of Bars	Bar Diam	Distance From Bottom Surface
1	16.00	100.00	0	0.00	0.00
1	12.00	500.00	0	0.00	0.00
1	12.00	900.00	0	0.00	0.00
1	12.00	1300.00	0	0.00	0.00
1	12.00	1700.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00

STEP 4 ..... Describe section properties for ULTIMATE conditions..........

(values entered in steps 1&2 may be varied for this part of the analysis.)

Concrete ultimate strain (e)	**0.003**
Ratio of (stress block)/(N.A.) depths........................	**0.85**
Axial compressive load (P) and, ...	**0**
depth from top surface of this load (di)	**0**
Crack root tensile stress (say 0.5f't)	**0.0**
Concrete Elastic Modulus (Ec)	**18,401**
Concrete compressive strength (f'c)...	**12**
Steel Elastic Modulus (Es)...	**200,000**
Steel Yield Stress (Fy)..	**325**

Analysis results shown below correspond to the conditions that exist when the peak compression strain equals (e) given above. A rectangular stress block with average stress=0.85f'c is assumed.

RESULTS FOR ULTIMATE MOMENT SECTION ANALYSIS:

(a) CRACK PROPAGATING FROM BOTTOM

Depth to N.A.(zero stress) from top (c)......................................	9.40E+01	
Steel Stress (Maximum Tension)..	3.25E+02	
Crack Depth	1.91E+03	
Total Tension Force (including P)...	1.55E+05	Ratio T/C =
Total Compression Force -incl. comp steel..................................	1.55E+05	1.000
Nominal Flex strength (Mn)---SEE NOTE 2..................................	1.56E+08	(=1.0 for iteration
Section Curvature (from curv = e/c)...	3.19E-05	convergence)

(b) CRACK PROPAGATING FROM TOP

Depth to N.A.(zero stress) from bottom (c)................................	1.29E+02	
Steel Stress (Maximum Tension)..	3.25E+02	
Crack Depth	1.87E+03	
Total Tension Force (including P)...	2.12E+05	Ratio T/C =
Total Compression Force -incl. comp steel..................................	2.12E+05	1.000
Nominal Flex strength (Mn)---SEE NOTE 2..................................	2.45E+08	(=1.0 for iteration
Section Curvature (from curv = e/c)...	2.33E-05	convergence)

WELCOME TO CONPROP(V 1.8) ** AN EXCEL SPREADSHEET FOR ANALYSING CONCRETE SECTIONS FOR FLEXURE UNDER UNCRACKED, CRACKED AND ULTIMATE CONDITIONS, IN ACCORDANCE WITH NZS 3101.

OPUS

Project:	**WEGC Science Block**	
Computed:	**M Geddes**	
	Date: 23-Sep-15	Time: 10:11

STEP 1 ..... Describe the Uncracked Section...
(use consistent units.. e.g. N and mm through out the spreadsheet)

Total Section depth (d) = ...	**2800**	
Web width (w) = ...	**190**	
Top flange width **excluding** web (b1) = ...	**0**	<----
Top flange thickness (t) = ...	**0**	THESE
Bottom flange width **excluding** web (b2) = ...	**0**	6 values
Bottom flange thickness (b) = ...	**0**	may
Axial compressive load (P) and, ...	**0**	be
Depth from top surface of this load (di)	**0**	zero
Assumed tensile cracking stress (f't)	**0**	<-----
Steel Elastic Modulus (Es)	**200,000**	

STEP 2 ..... Describe steel sizes and locations..................
describe location of the centroid of up to 10 bar bundles from either the top or the bottom surface. Describe Location of each bundle from only one surface.

Modular ratio (n=Es*(1+Ct)/Ec) =	**11**

TOP BARS		
No. Bars	Bar Diam	Distance From Top Surface
1	**16.00**	**100.00**
1	**12.00**	**500.00**
1	**12.00**	**900.00**
1	**12.00**	**1300.00**
1	**12.00**	**1700.00**
1	**12.00**	**2100.00**
1	**12.00**	**2500.00**
0	**0.00**	**0.00**
0	**0.00**	**0.00**
0	**0.00**	**0.00**

BOTTOM BARS		
No of Bars	Bar Diam	Distance From Bottom Surface
0	**0.00**	**0.00**
0	**0.00**	**0.00**
0	**0.00**	**0.00**
0	**0.00**	**0.00**
0	**0.00**	**0.00**
0	**0.00**	**0.00**
0	**0.00**	**0.00**
0	**0.00**	**0.00**
0	**0.00**	**0.00**
0	**0.00**	**0.00**

STEP 4 ..... Describe section properties for ULTIMATE conditions..........

(values entered in steps 1&2 may be varied for this part of the analysis.)

Concrete ultimate strain (e)	**0.003**
Ratio of (stress block)/(N.A.) depths.......................	**0.85**
Axial compressive load (P) and, ...	**0**
depth from top surface of this load (di)	**0**
Crack root tensile stress (say 0.5f't)	**0.0**
Concrete Elastic Modulus (Ec)	**18,401**
Concrete compressive strength (f'c)...	**12**
Steel Elastic Modulus (Es)...	**200,000**
Steel Yield Stress (Fy)..	**325**

Analysis results shown below correspond to the conditions that exist when the peak compression strain equals (e) given above. A rectangular stress block with average stress=0.85f'c is assumed.

RESULTS FOR ULTIMATE MOMENT SECTION ANALYSIS:

(a) CRACK PROPAGATING FROM BOTTOM

Depth to N.A.(zero stress) from top (c)......................................	1.22E+02	
Steel Stress (Maximum Tension)...	3.25E+02	
Crack Depth	2.68E+03	
Total Tension Force (including P)..	2.21E+05	Ratio T/C =
Total Compression Force -incl. comp steel..................................	2.21E+05	1.000
Nominal Flex strength (Mn)---SEE NOTE 2..................................	3.18E+08	(=1.0 for iteration
Section Curvature (from curv = e/c)...	2.46E-05	convergence)

(b) CRACK PROPAGATING FROM TOP

Depth to N.A.(zero stress) from bottom (c)..................................	1.74E+02	
Steel Stress (Maximum Tension)...	3.25E+02	
Crack Depth	2.63E+03	
Total Tension Force (including P)..	2.86E+05	Ratio T/C =
Total Compression Force -incl. comp steel..................................	2.86E+05	1.000
Nominal Flex strength (Mn)---SEE NOTE 2..................................	4.42E+08	(=1.0 for iteration
Section Curvature (from curv = e/c)...	1.73E-05	convergence)

WELCOME TO CONPROP(V 1.8) ** AN EXCEL SPREADSHEET FOR ANALYSING CONCRETE SECTIONS FOR FLEXURE UNDER UNCRACKED, CRACKED AND ULTIMATE CONDITIONS, IN ACCORDANCE WITH NZS 3101.

OPUS

Project:	**WEGC Science Block**
Computed:	**M Geddes**
Date: 23-Sep-15	Time: 10:11

STEP 1 ..... Describe the Uncracked Section...
(use consistent units.. e.g. N and mm through out the spreadsheet)

Total Section depth (d) =	1600	
Web width (w) =	190	
Top flange width **excluding** web (b1) =	0	<----
Top flange thickness (t) =	0	THESE
Bottom flange width **excluding** web (b2) =	0	6 values
Bottom flange thickness (b) =	0	may
Axial compressive load (P) and,	0	be
Depth from top surface of this load (di)	0	zero
Assumed tensile cracking stress (f't)	0	<-----
Steel Elastic Modulus (Es)	200,000	

STEP 2 ..... Describe steel sizes and locations..................
describe location of the centroid of up to 10 bar bundles from either the top or the bottom surface. Describe Location of each bundle from only one surface.

Modular ratio (n=Es*(1+Ct)/Ec) =	11

TOP BARS			BOTTOM BARS		
No. Bars	Bar Diam	Distance From Top Surface	No of Bars	Bar Diam	Distance From Bottom Surface
1	16.00	100.00	0	0.00	0.00
1	16.00	500.00	0	0.00	0.00
1	16.00	900.00	0	0.00	0.00
1	16.00	1300.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00

Bottom Spandrel Gridline A

STEP 4 ..... Describe section properties for ULTIMATE conditions..........

(values entered in steps 1&2 may be varied for this part of the analysis.)

Concrete ultimate strain (e)	0.003
Ratio of (stress block)/(N.A.) depths.......................	0.85
Axial compressive load (P) and, ...	0
depth from top surface of this load (di)	0
Crack root tensile stress (say 0.5f't)	0.0
Concrete Elastic Modulus (Ec)	18,401
Concrete compressive strength (f'c)...	12
Steel Elastic Modulus (Es)...	200,000
Steel Yield Stress (Fy)..	325

Analysis results shown below correspond to the conditions that exist when the peak compression strain equals (e) given above. A rectangular stress block with average stress=0.85f'c is assumed.

RESULTS FOR ULTIMATE MOMENT SECTION ANALYSIS:

(a) CRACK PROPAGATING FROM BOTTOM

Depth to N.A.(zero stress) from top (c).....................................	1.12E+02	
Steel Stress (Maximum Tension)..	3.25E+02	
Crack Depth	1.49E+03	
Total Tension Force (including P)...	1.96E+05	Ratio T/C =
Total Compression Force -incl. comp steel................................	1.96E+05	1.000
Nominal Flex strength (Mn)---SEE NOTE 2..................................	1.66E+08	(=1.0 for iteration
Section Curvature (from curv = e/c)...	2.68E-05	convergence)

(b) CRACK PROPAGATING FROM TOP

Depth to N.A.(zero stress) from bottom (c).................................	1.59E+02	
Steel Stress (Maximum Tension)..	3.25E+02	
Crack Depth	1.44E+03	
Total Tension Force (including P)...	2.61E+05	Ratio T/C =
Total Compression Force -incl. comp steel................................	2.61E+05	1.000
Nominal Flex strength (Mn)---SEE NOTE 2..................................	2.18E+08	(=1.0 for iteration
Section Curvature (from curv = e/c)...	1.89E-05	convergence)

WELCOME TO CONPROP(V 1.8) ** AN EXCEL SPREADSHEET FOR ANALYSING CONCRETE SECTIONS FOR FLEXURE UNDER UNCRACKED, CRACKED AND ULTIMATE CONDITIONS, IN ACCORDANCE WITH NZS 3101.

OPUS

Project:	**WEGC Science Block**	
Computed:	**M Geddes**	
	Date: 23-Sep-15	Time: 10:12

STEP 1 ..... Describe the Uncracked Section...
(use consistent units.. e.g. N and mm through out the spreadsheet)

Total Section depth (d) = ...	1800	
Web width (w) = ...	190	
Top flange width **excluding** web (b1) = ...	0	<----
Top flange thickness (t) = ...	0	THESE
Bottom flange width **excluding** web (b2) = ...	0	6 values
Bottom flange thickness (b) = ...	0	may
Axial compressive load (P) and, ...	0	be
Depth from top surface of this load (di)	0	zero
Assumed tensile cracking stress (f't)	0	<-----
Steel Elastic Modulus (Es)	200,000	

STEP 2 ..... Describe steel sizes and locations..................
describe location of the centroid of up to 10 bar bundles from either the top or the bottom surface. Describe Location of each bundle from only one surface.

Modular ratio (n=Es*(1+Ct)/Ec) =	11

TOP BARS			BOTTOM BARS		
No. Bars	Bar Diam	Distance From Top Surface	No of Bars	Bar Diam	Distance From Bottom Surface
1	16.00	100.00	0	0.00	0.00
1	16.00	500.00	0	0.00	0.00
1	16.00	900.00	0	0.00	0.00
1	16.00	1300.00	0	0.00	0.00
1	16.00	1700.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00
0	0.00	0.00	0	0.00	0.00

STEP 4 ..... Describe section properties for ULTIMATE conditions..........

(values entered in steps 1&2 may be varied for this part of the analysis.)

Concrete ultimate strain (e)	**0.003**
Ratio of (stress block)/(N.A.) depths.......................	**0.85**
Axial compressive load (P) and, ...	**0**
depth from top surface of this load (di)	**0**
Crack root tensile stress (say 0.5f't)	**0.0**
Concrete Elastic Modulus (Ec)	**18,401**
Concrete compressive strength (f'c)...	**12**
Steel Elastic Modulus (Es)..	**200,000**
Steel Yield Stress (Fy)...	**325**

Analysis results shown below correspond to the conditions that exist when the peak compression strain equals (e) given above. A rectangular stress block with average stress=0.85f'c is assumed.

RESULTS FOR ULTIMATE MOMENT SECTION ANALYSIS:

(a) CRACK PROPAGATING FROM BOTTOM

Depth to N.A.(zero stress) from top (c)......................................	1.39E+02	
Steel Stress (Maximum Tension)..	3.25E+02	
Crack Depth	1.66E+03	
Total Tension Force (including P)..	2.61E+05	Ratio T/C =
Total Compression Force -incl. comp steel..................................	2.61E+05	1.000
Nominal Flex strength (Mn)---SEE NOTE 2....................................	2.71E+08	(=1.0 for iteration
Section Curvature (from curv = e/c)...	2.15E-05	convergence)

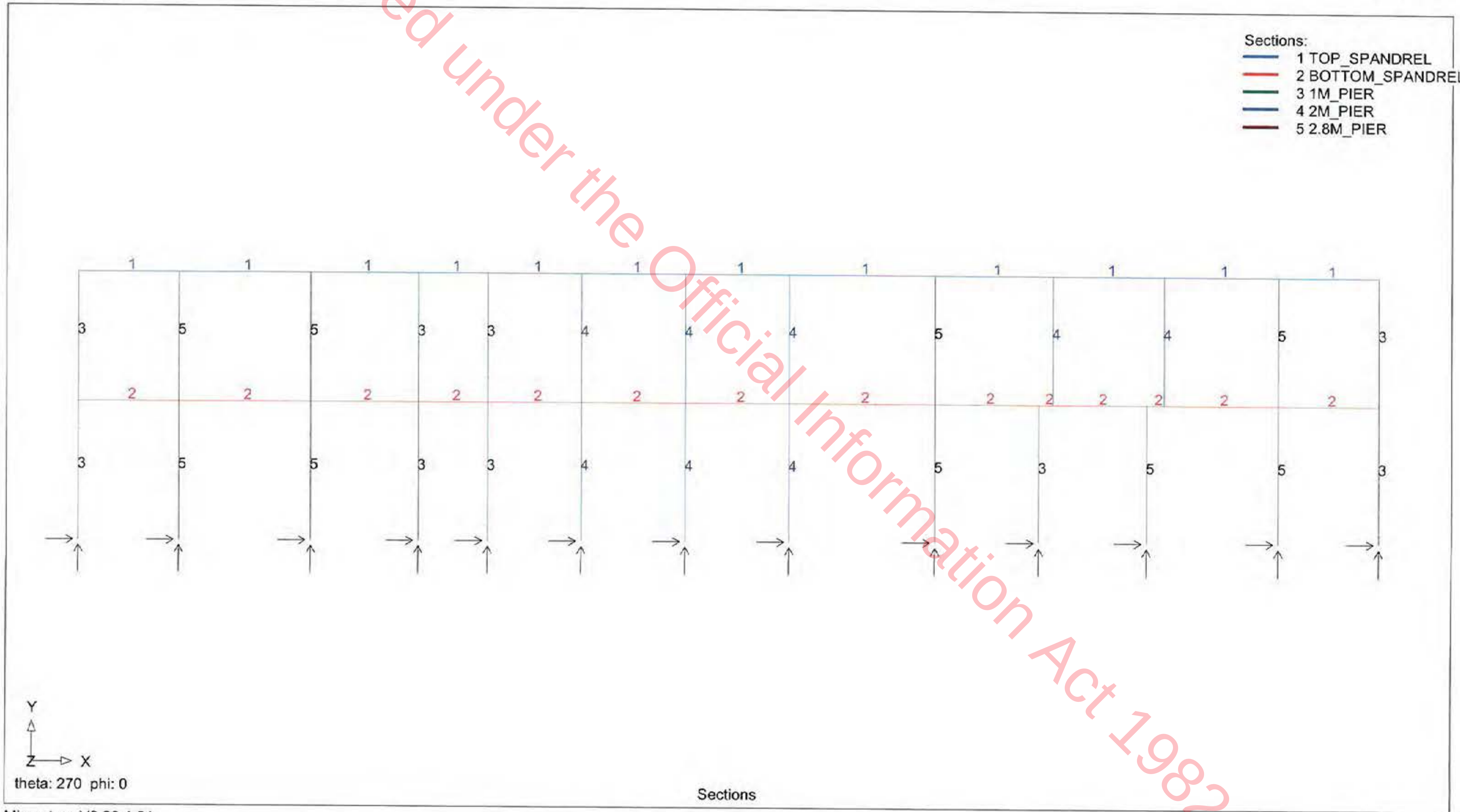
(b) CRACK PROPAGATING FROM TOP

Depth to N.A.(zero stress) from bottom (c).................................	1.39E+02	
Steel Stress (Maximum Tension)..	3.25E+02	
Crack Depth	1.66E+03	
Total Tension Force (including P)..	2.61E+05	Ratio T/C =
Total Compression Force -incl. comp steel..................................	2.61E+05	1.000
Nominal Flex strength (Mn)---SEE NOTE 2....................................	2.71E+08	(=1.0 for iteration
Section Curvature (from curv = e/c)...	2.15E-05	convergence)

Microstran V9

wemjg0
Job: Gridline A frame
Frame

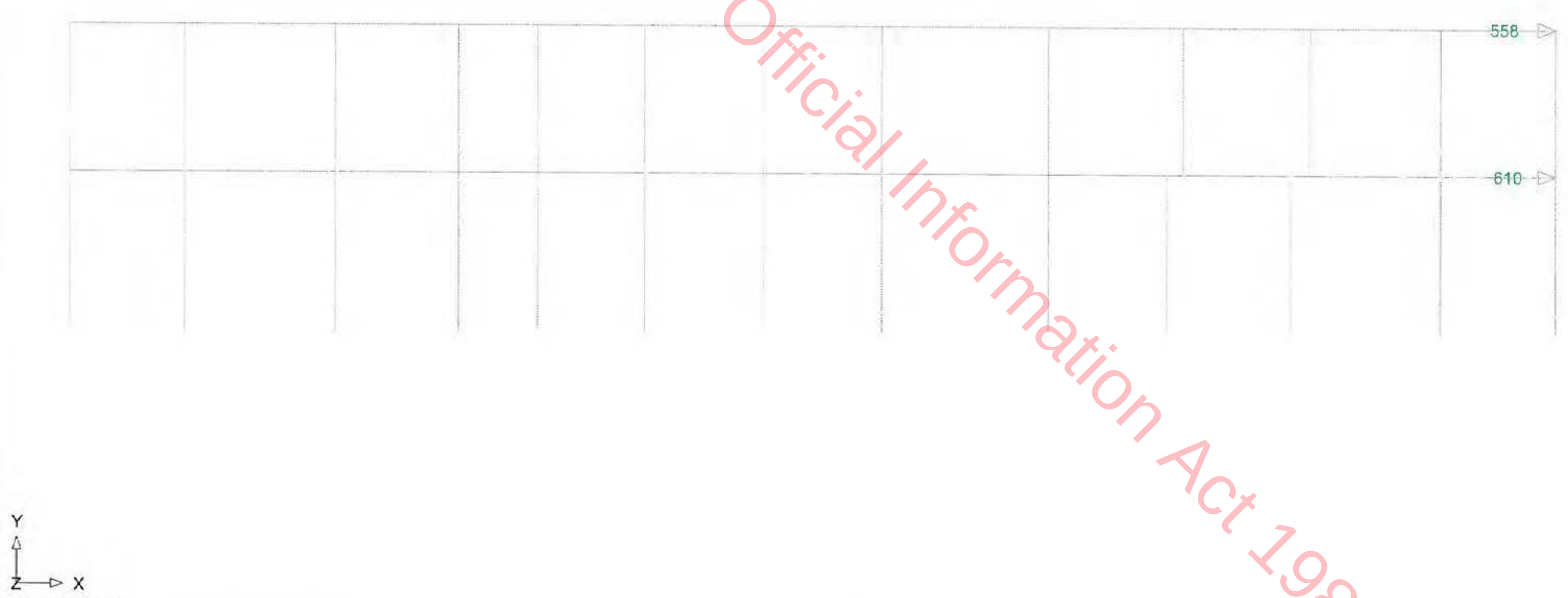
23/09/2015
10:16:11 a.



Sections

C13

Load Cases:
2 P mu = 1.25 left to right



theta: 270 phi: 0

Microstran V9

wemjg0
Job: Gridline A frame
Frame

23/09/2015
10:17:12 a.

Load Cases:
3 P mu = 1.25 right to left

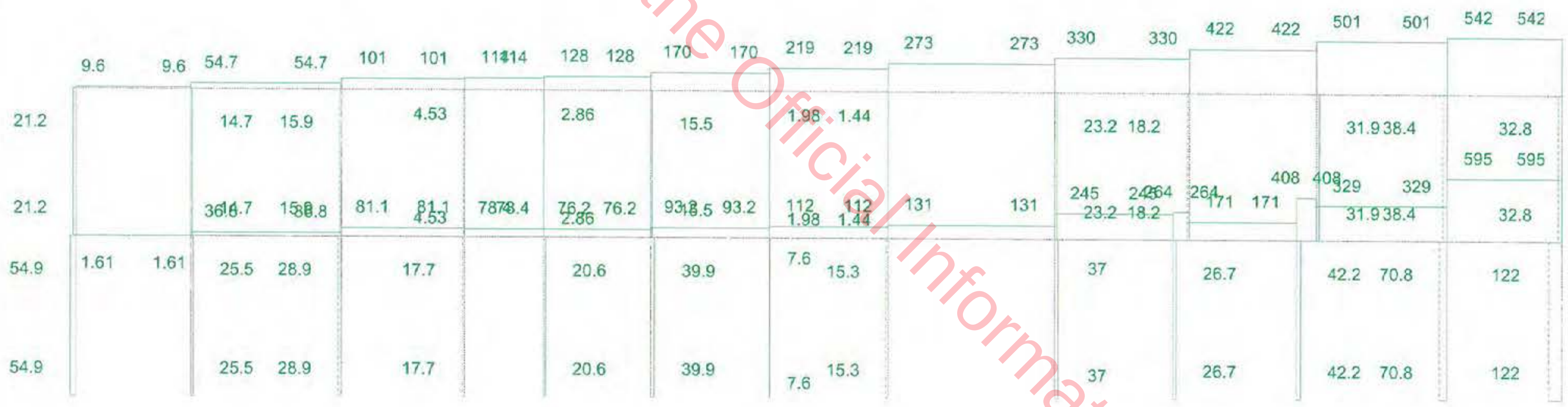
theta: 270 phi: 0

Microstran V9

wemjg0
Job: Gridline A frame
Frame

23/09/2015
10:16:41 a.

Load Cases:
2 P mu = 1.25 left to right



Y
Z X
theta: 270 phi: 0

Axial Force, Fx

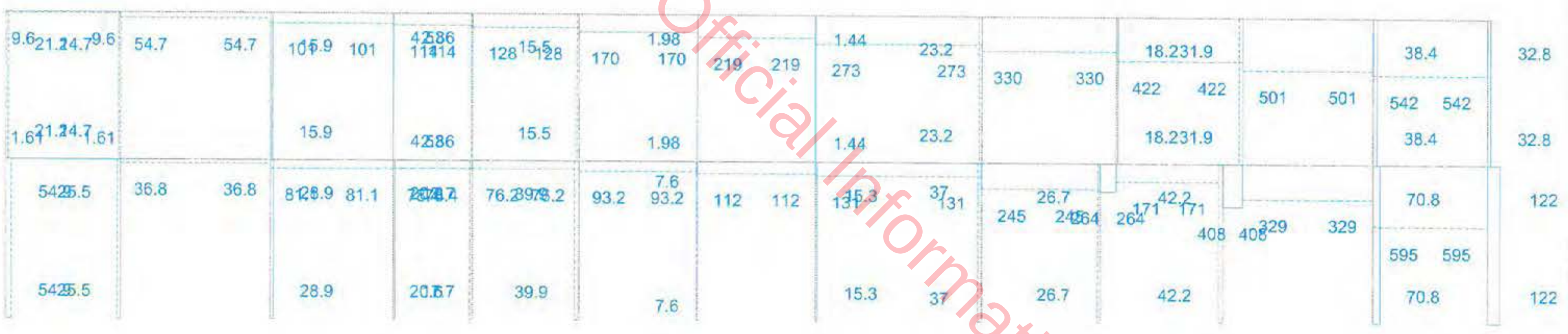
C16

Microstran V9

wemjg0
Job: Gridline A frame
Frame

23/09/2015
10:17:26 a.

Load Cases:
3 P mu = 1.25 right to left



theta: 270 phi: 0

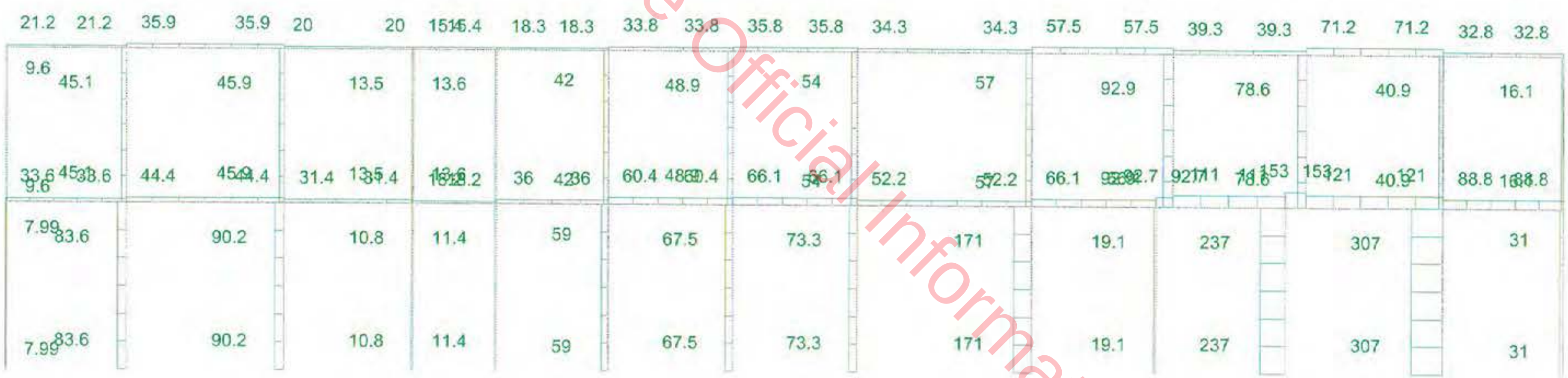
Axial Force, Fx

Microstran V9

wemjg0
Job: Gridline A frame
Frame

23/09/2015
10:16:48 a.

Load Cases:
2 P mu = 1.25 left to right



theta: 270 phi: 0

Shear Force, Fy

Microstran V9.20.1.21

...\Gridline A frame

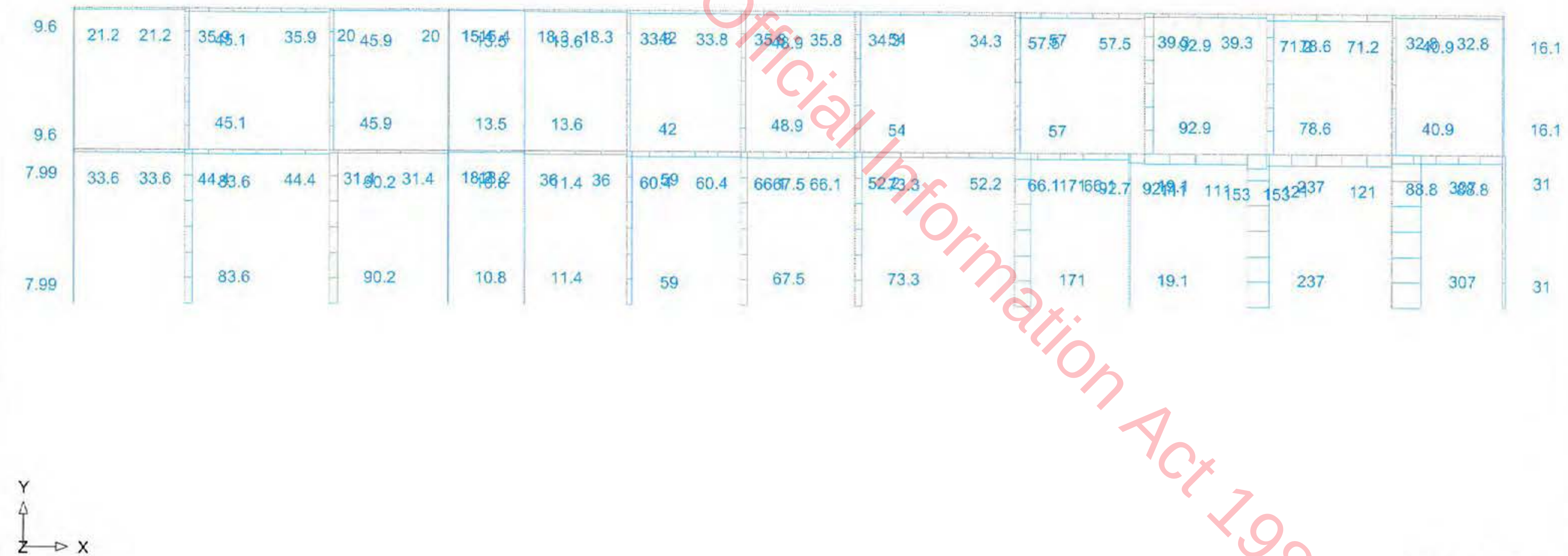
C18

Microstran V9

wemjg0
Job: Gridline A frame
Frame

23/09/2015
10:17:34 a.

Load Cases:
3 P mu = 1.25 right to left



theta: 270 phi: 0

Shear Force, Fy

C19

Microstran V9

wemjg0
Job: Gridline A frame
Frame

23/09/2015
10:16:57 a.

Load Cases:
2 P mu = 1.25 left to right

theta: 270 phi: 0

Bending Moment, Mz

C20

Microstran V9

wemjg0
Job: Gridline A frame
Frame

23/09/2015
10:17:41 a.

Load Cases:
3 P mu = 1.25 right to left

Y
X
Z
theta: 270 phi: 0

Bending Moment, Mz

Microstran V9.20.1.21

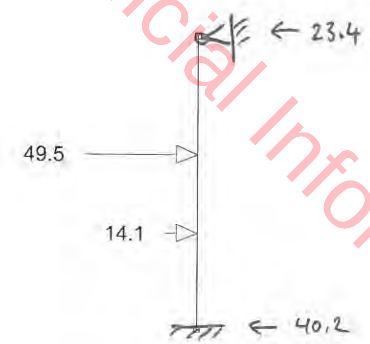
...\Gridline A frame

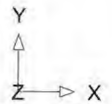
Microstran V9

wemjg0
Job: Retaining Wall
2.3m Blockwork

17/09/2015
04:15:26 p.

Load Cases:
1 P G + Eu





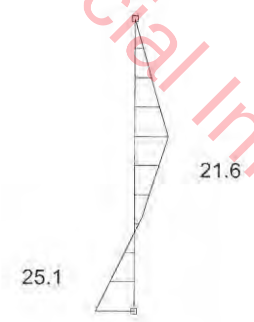
theta: 270 phi: 0

Microstran V9

wemjg0
Job: Retaining Wall
2.3m Blockwork

17/09/2015
04:15:12 p.

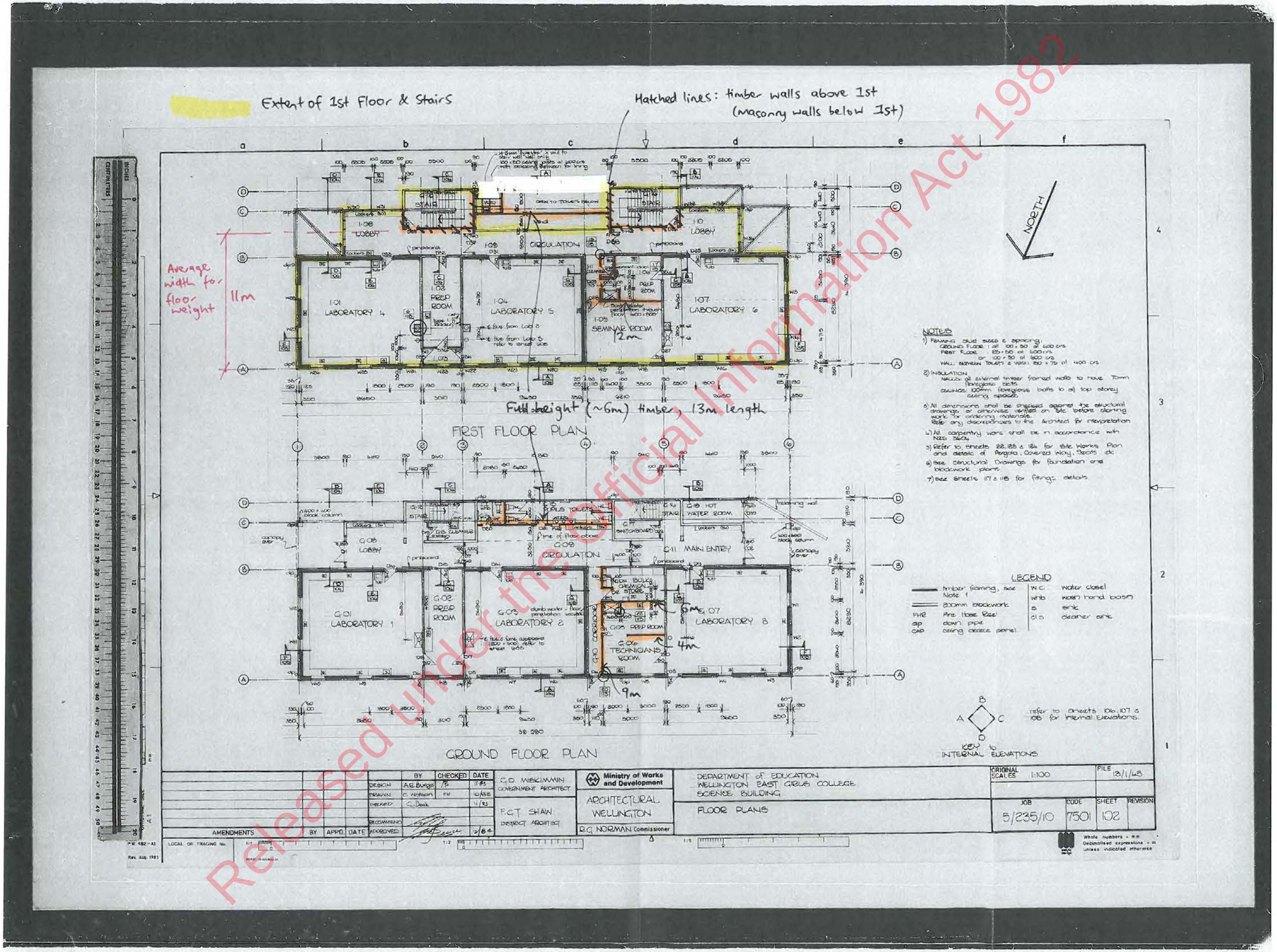
Load Cases:
1 P G + Eu



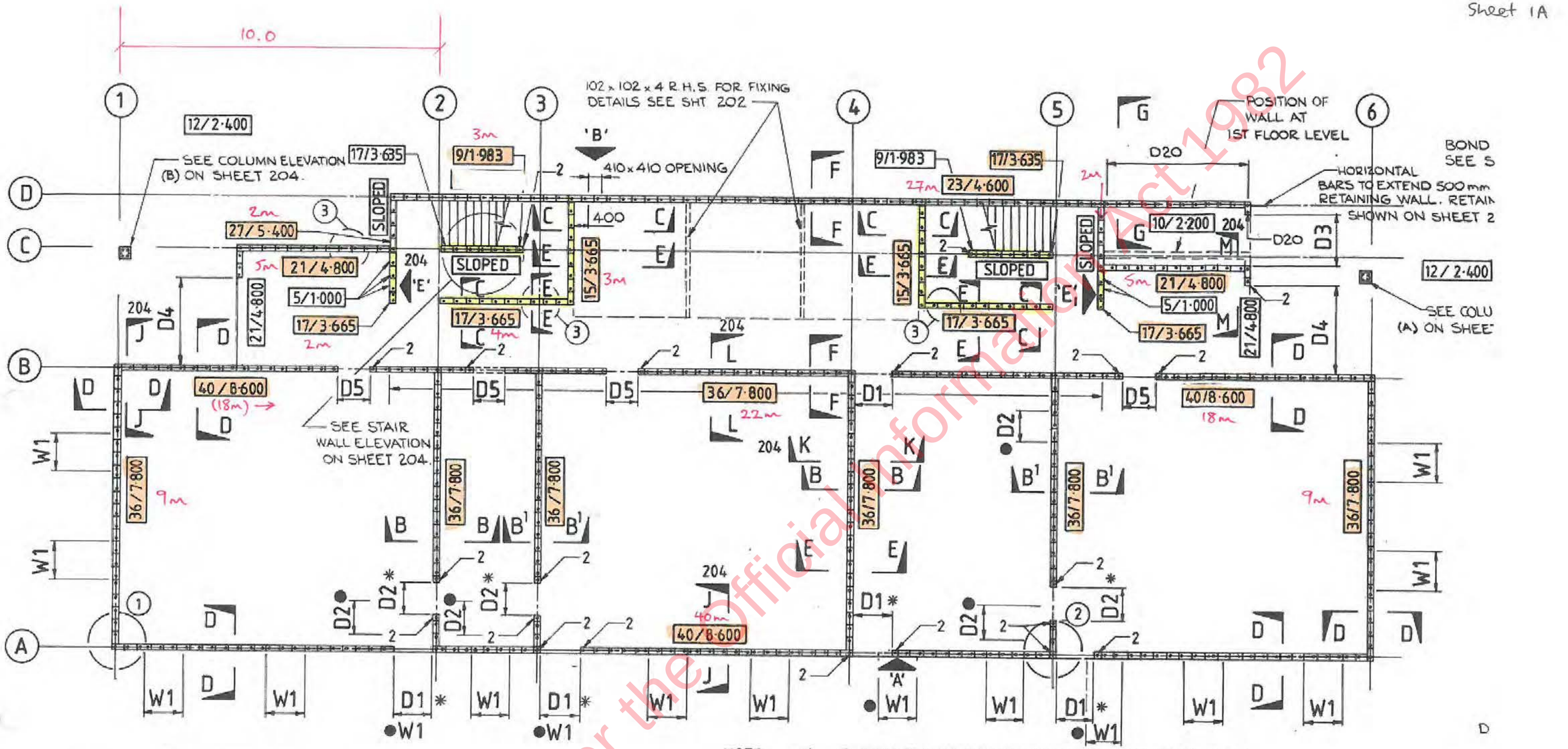
Y
Z X
theta: 270 phi: 0

Bending Moment, My
Bending Moment, Mz

Sheet 1



Extent of 1st Floor & Stairs
Hatched lines: timber walls above 1st (masonry walls below 1st)
Average width for floor weight
11m
12m
Full height (~6m) timber, 13m length
6m
4m
9m
FIRST FLOOR PLAN
GROUND FLOOR PLAN
NORTH
1.01 LABORATORY 4
1.02 PREP ROOM
1.04 LABORATORY 5
1.05 SEMINAR ROOM
1.06 PREP ROOM
1.07 LABORATORY 6
1.08 LOBBY
1.09 CIRCULATION
1.10 LOBBY
STAIR
G.01 LABORATORY 1
G.02 PREP ROOM
G.03 LABORATORY 2
G.05 PREP ROOM
G.06 TECHNICIANS ROOM
G.07 LABORATORY 3
G.08 LOBBY
G.09 CIRCULATION
G.11 MAIN ENTRY
G.18 HOT WATER ROOM
NOTES
LEGEND
KEY to INTERNAL ELEVATIONS
Ministry of Works and Development
ARCHITECTURAL WELLINGTON
DEPARTMENT of EDUCATION
WELLINGTON EAST GIRLS COLLEGE
SCIENCE BUILDING
FLOOR PLANS
JOB 5/235/10
CODE 7501
SHEET 102



DOOR OR WINDOW TYPE	SIZE (W x H)	SILL R.L.
W1	1·210 x 2·010	60·750 & 64·350
D1	1·210 x 3·160	

BLOCK LAYOUT PLAN

1:100

NOTE

a) ● INDICATES WINDOW OR DOOR AT FIRST FLOOR ONLY.

b) ✱ INDICATES WINDOW OR DOOR AT GROUND FLOOR ONLY.

c) REMAINING WINDOWS AND DOORS OCCUR AT BOTH GROUND AND FIRST FLOOR.

Masonry to 1st floor height

Total length of wall = 2+4+3+3 = 12m per stairwell
= 24m overall

40 / 8.6 — No. of blocks / wall height
40m — Wall length

Appendix B

Photos of Building

West Elevation

East Elevation

North Elevation

Rear of building (South Elevation)

South Elevation

Ground Floor Classroom

Appendix C

Plans of Building

THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH 200/1, 202, 203 & 204.

NOTES

1. COVER - Shall be in accordance with section 8 sheet 200/1, except as noted.
2. SPLICES/DEVELOPMENT LENGTHS - D24 - 900mm; D20 - 700 mm; D16 - 600 mm; D12 - 450 mm; D10 - 400mm. NO SPLICES OVER PILES
3. Where ground beams cannot be founded on rock, piles shall be placed at locations approved by the Architect.
4. All cable ducts to contain draw wires.
5. All cable ducts to fall to outside of building.

AMENDMENTS	BY	APPD.	DATE

	BY	CHECKED	DATE
DRAWN		R. Matias	11/83
SUPER'VD			12/83
DESIGN			12/83
RECOM'D			12/83

STRUCTURAL DESIGN OFFICE

J.B.S. HUIZING, CHIEF CIVIL ENGINEER

G.H.F. McKENZIE, CHIEF STRUCTURAL ENGINEER

APPROVED 12/83

Ministry of Works and Development

CIVIL ENGINEERING HEAD OFFICE

R.G. NORMAN Commissioner

DEPARTMENT OF EDUCATION
WELLINGTON EAST GIRLS COLLEGE
SCIENCE BUILDING

REINFORCED CONCRETE
GROUND FLOOR SLAB AND FOUNDATIONS

ORIGINAL SCALES 1:20 & AS SHOWN

FILE

JOB	CODE	SHEET	REVISION
5-235-10	7001	201	

ORIGINAL SIZE A1

LOCAL OR TRACING No.

1:1 1:2 1:5

Whole numbers = mm
Decimalised expressions = m
unless indicated otherwise

	BY	CHECKED	DATE
DRAWN			11/83
SUPER'VD			12/83
DESIGN			12/83
RECOM'D			12/83

AMENDMENTS | BY | APPD. | DATE

STRUCTURAL DESIGN OFFICE

J.B.S. HUIZING
CHIEF CIVIL ENGINEER
G.H.F. McKENZIE
CHIEF
STRUCTURAL ENGINEER
APPROVED

Ministry of Works and Development
CIVIL ENGINEERING HEAD OFFICE
R.G. NORMAN Commissioner

DEPARTMENT OF EDUCATION
WELLINGTON EAST GIRLS COLLEGE
SCIENCE BUILDING

REINFORCED CONCRETE - STAIRS
STEELWORK

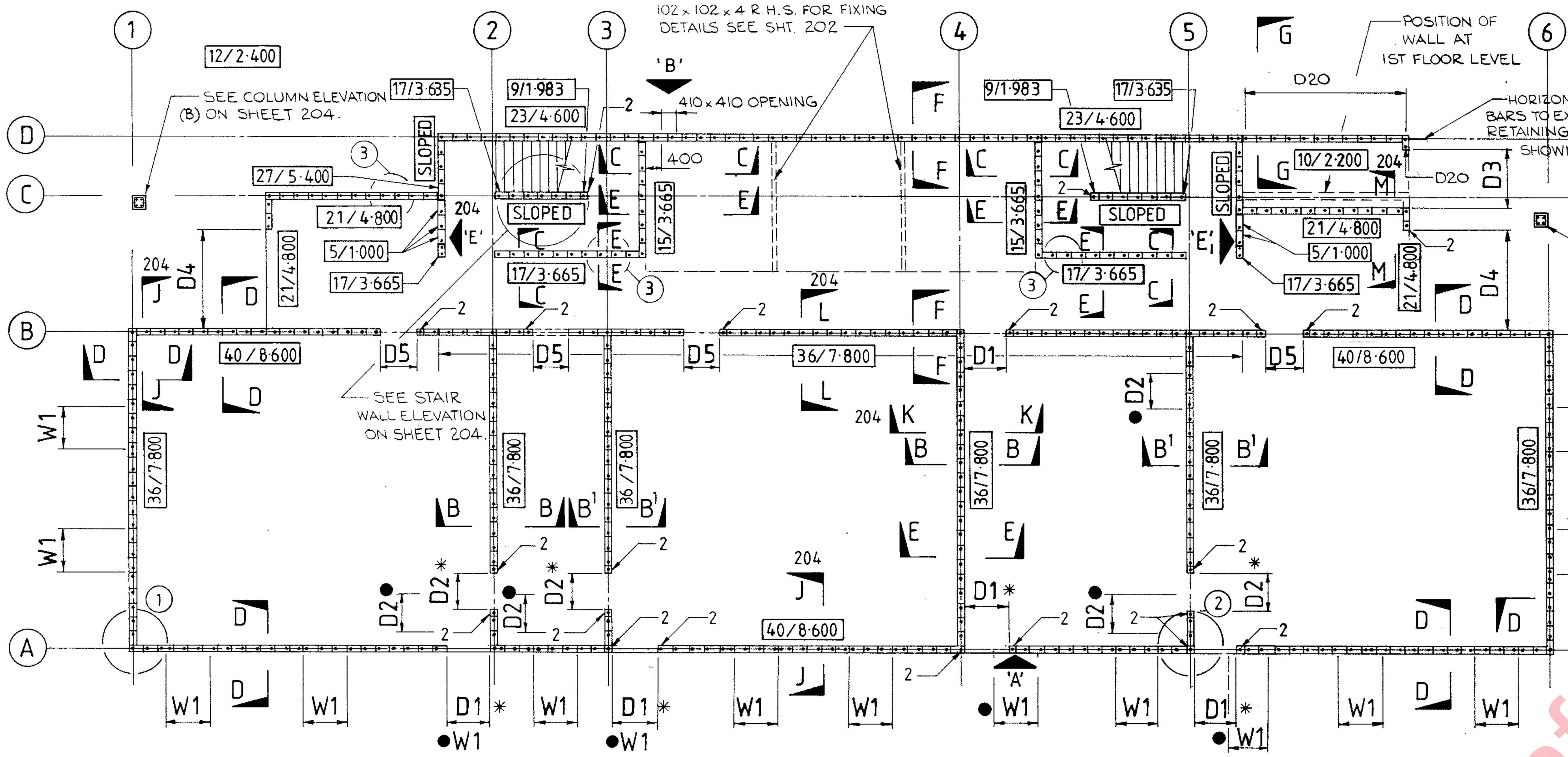
ORIGINAL SCALES 1:25 & AS SHOWN

FILE

JOB	CODE	SHEET	REVISION
5-235-10	7001	202	

TRACING No.

Whole numbers = mm
Decimalised expressions = m
unless indicated otherwise

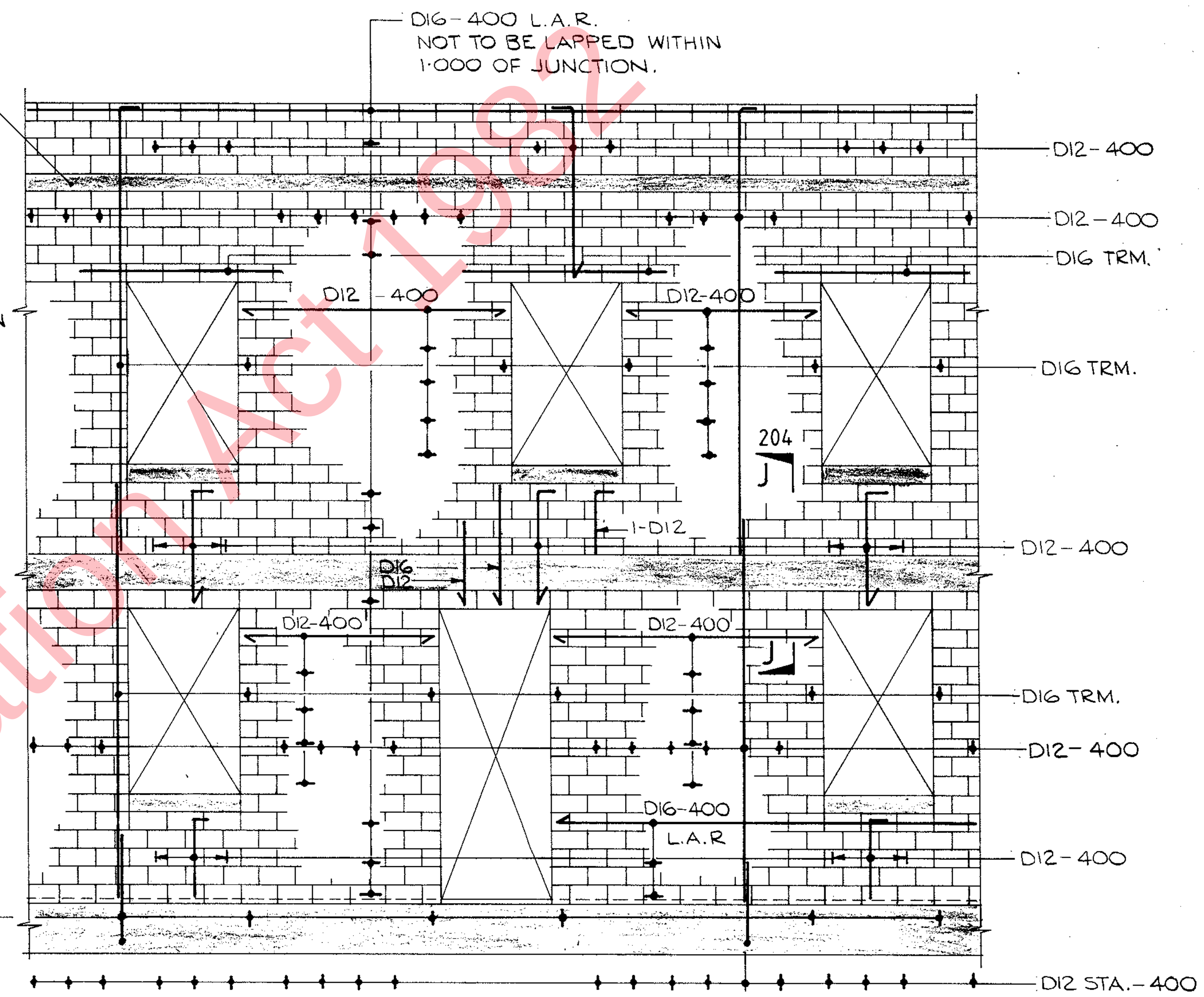


BLOCK LAYOUT PLAN

1:100

NOTE
a) ● INDICATES WINDOW OR DOOR AT FIRST FLOOR ONLY.
b) ✱ INDICATES WINDOW OR DOOR AT GROUND FLOOR ONLY.
c) REMAINING WINDOWS AND DOORS OCCUR AT BOTH GROUND AND FIRST FLOOR.

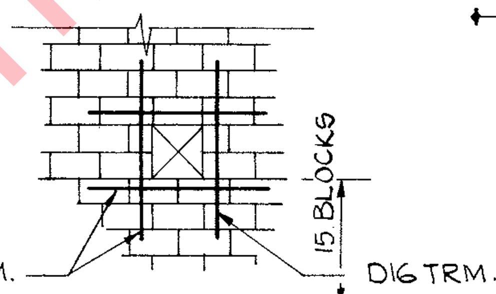
DOOR OR WINDOW TYPE	SIZE (W x H)	SILL R.L.
W1	1·210 x 2·010	60·750 & 64·350
D1	1·210 x 3·160	
D2	1·010 x 2·160	
D3	1·610 x 2·160	
D4	2·610 x 2·160	
D5	1·010 x 2·360	



NOTE:- TRANSVERSE WALLS TO HAVE D12 - 400 (VERT.) AND D16 - 400 (HORIZ.)

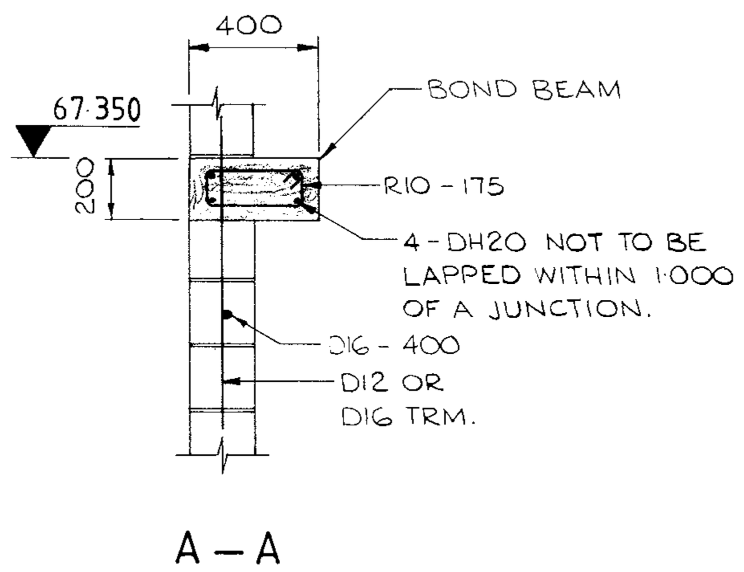
VIEW 'A'

1:50

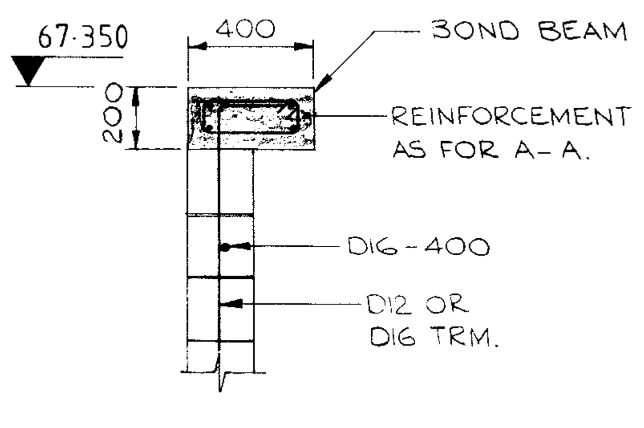


VIEW 'B'

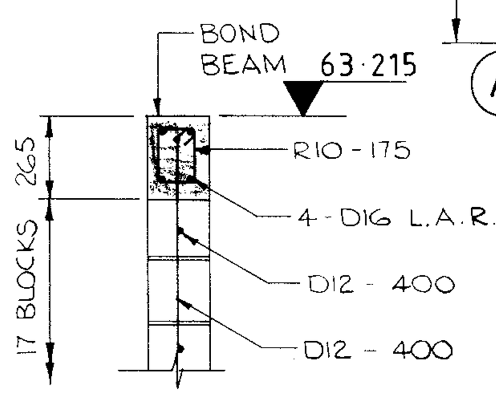
1:50



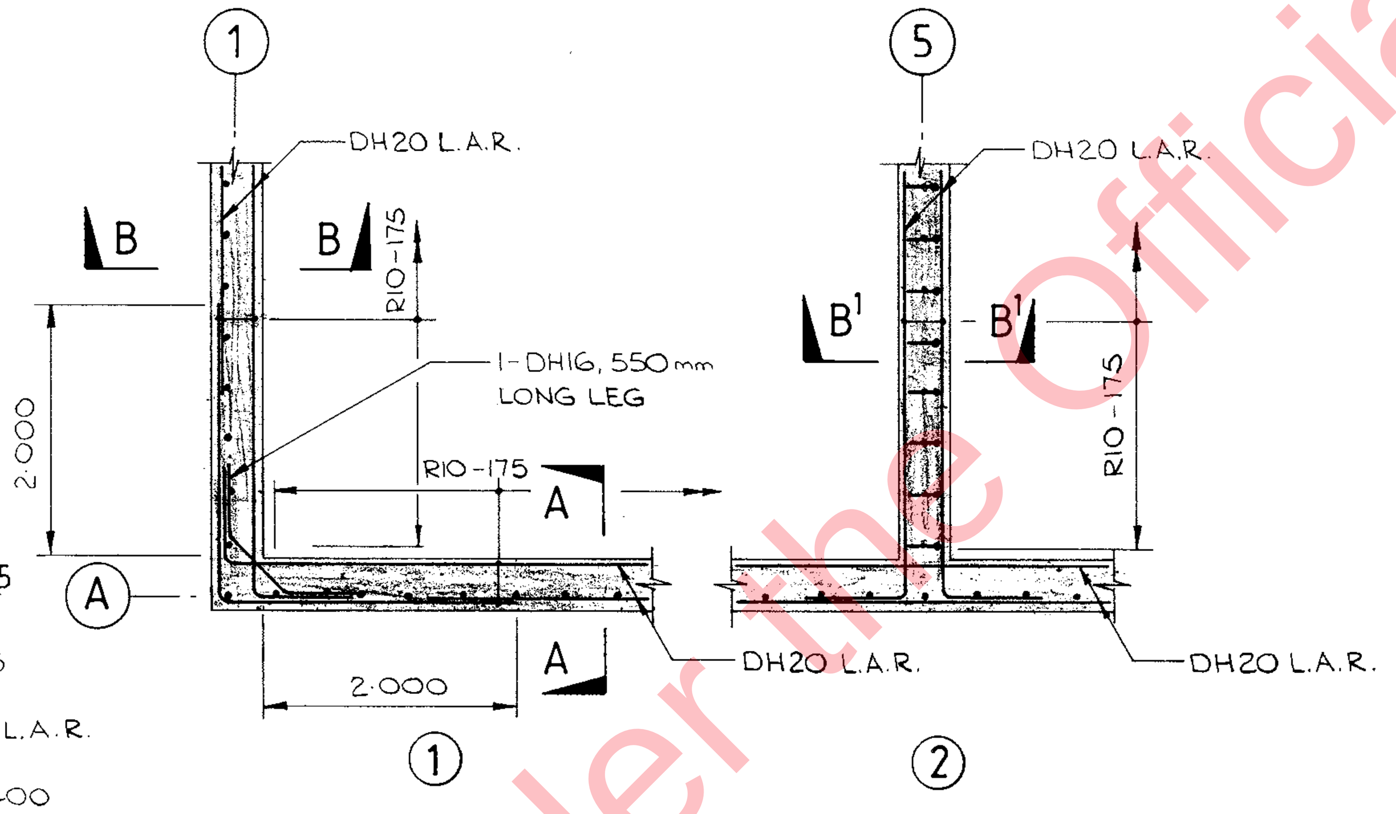
A – A
1:20



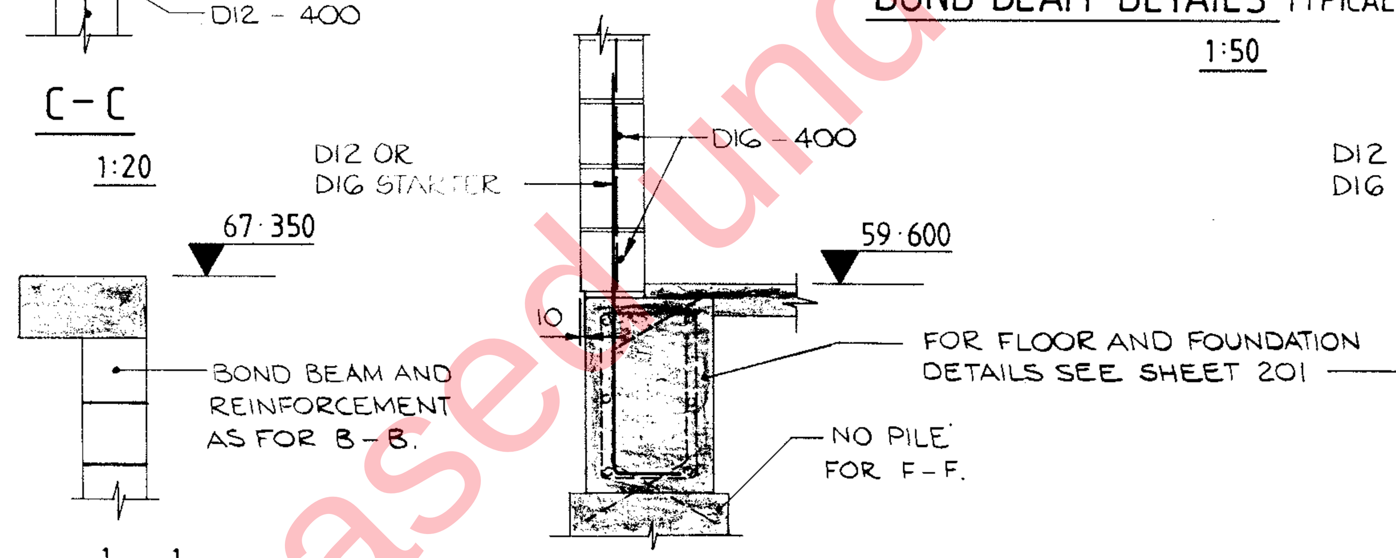
B – B
1:20



C – C
1:20

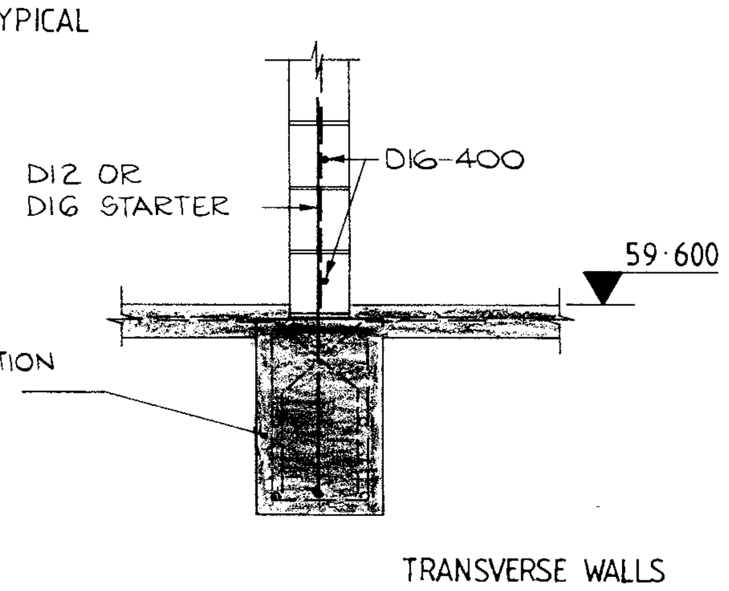


BOND BEAM DETAILS TYPICAL
1:50

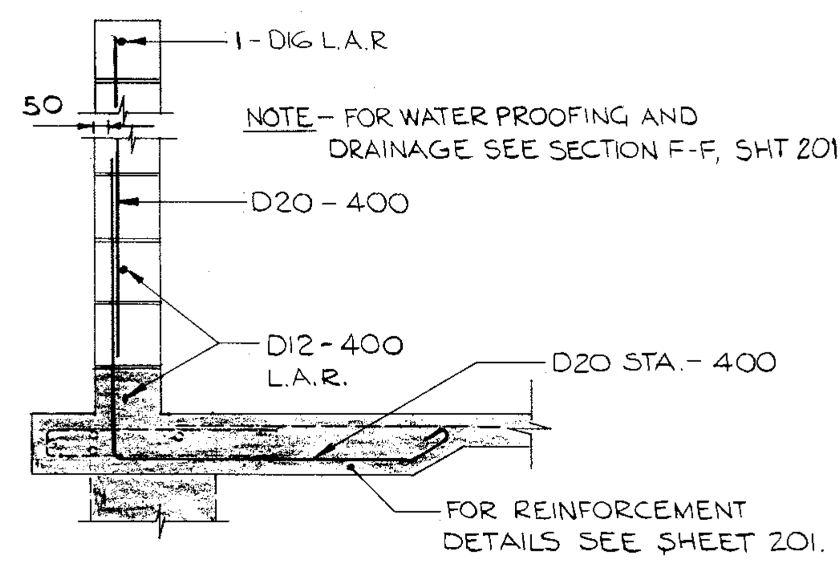


B¹ – B¹
1:20

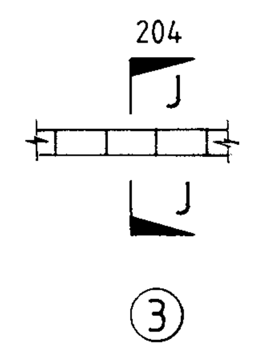
D – D F – F
1:20



E – E TRANSVERSE WALLS TYPICAL
1:20



G – G
1:20



N.T.S.

NOTES

1. REINFORCEMENT TO BE D12 - 400 (VERT.) & D16 - 400 (HORIZ.) UNLESS SHOWN OTHERWISE.
2. FOR STRUCTURAL DETAILS OF FIRST FLOOR SEE 'STAHLTON' DRAWING.
3. BLOCKS TO BE 20 SERIES, TYPE 16. SEE (1a), SHEET 200/2.
4. UNLESS SHOWN OTHERWISE, LAP AND ANCHORAGE LENGTHS ARE TO BE:- 500 mm FOR D12; 650 mm FOR D16; 800 mm FOR D20; 900 mm FOR DH20
5. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH 200/1, 200/2, 201, 202, 204.
6. FOR GEOMETRY DETAILS SEE ARCHITECTS DRAWINGS.
7. 40 / 8·000 DENOTES:- NUMBER OF COURSES / WALL HEIGHT. WHERE THE NUMBER OF COURSES DOES NOT CORRESPOND WITH THE WALL HEIGHT THE BALANCE IS MADE UP OF INSITU CONCRETE AT FIRST FLOOR, LANDINGS, GROUND FLOOR NIB AND BOND BEAMS OR PRECAST SILLS. WALL HEIGHTS ARE FROM LEVEL 59·550.

AMENDMENTS	BY	APPD.	DATE

	BY	CHECKED	DATE
Drawn	R. Matkin		11/83
Dsg. sup'v'r			12/83
Designed			12/83
Rec'mnd			12/83

STRUCTURAL DESIGN OFFICE

J.B.S. HUIZING, CHIEF CIVIL ENGINEER
G.H.F. McKENZIE, CHIEF STRUCTURAL ENGINEER
APPROVED 12/83

Ministry of Works and Development
CIVIL ENGINEERING HEAD OFFICE
R.G. NORMAN Commissioner

DEPARTMENT OF EDUCATION
WELLINGTON EAST GIRLS COLLEGE
SCIENCE BUILDING

MASONRY
PLAN AND DETAILS

ORIGINAL SCALES: AS SHOWN
FILE

JOB	CODE	SHEET	REVISION
5 / 235 / 10	7001	203	

Whole numbers = mm. Decimalised expressions = m unless indicated otherwise

LOCAL OR TRACING No.
W 482 - A1
ORIGINAL SIZE A1

	BY	CHECKED	DATE
Drawn			11/83
Des. check'd			12/83
Designed			12/83
Rec'm'd			12/83

AMENDMENTS | BY | APPD. | DATE

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W. 482-A1 Rev. Aug. 1981

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Wellington East Girls College: Source (LINZ Data Service)