MINISTRY OF EDUCATION
Te Tāhuhu o te Mātauranga

Wellington East Girls College

Block 4 – South Wing

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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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).	6546
Block Name/Description	Block 4- South Wing
Known Standard Design	Non-standard
Storeys:	2
Year of Design (approx.)	1964 (original) and 1999 (additions)
Gross Floor Area (m²)	650m²
Construction Type	Reinforced concrete frame building with partially grouted infill concrete block walls and in-situ concrete walls (on staircases) with timber framing additions to the rear of the building (on east elevation).
Assessment Type	Detailed
Date Building Inspected	10th 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.
Stairs	The in-situ reinforced concrete stairs are integrally connected to the concrete walls (as per original structural drawings). So, any lateral loading will be resisted by the concrete shear walls resulting to no differential movement between stairs and walls. Hence, it is not expected to have any damage to the stairs.
Current %NBS estimate	50% NBS (*)
List specific CSWs and life safety hazards	None

Occupancy Considerations	No need to change the building's current occupancy.
Conclusions & Recommendations Conclusions & Recommendations	(*) The building has an estimated seismic capacity 50%NBS range when assessed as an IL3 building. The governing factors are (in order of importance): • The partially grouted block walls at ground storey on longitudinal direction and the effect on the concrete frame providing an average rating of 50%NBS. • The connection of the first floor slab (diaphragm) to the stair walls has a rating of 63%NBS. It is recommended that the building is strengthened to at least 67%NBS in accordance with current MOE guidelines and NZSEE recommendations. Further detailed design will need to be undertaken to develop the optimum strengthening solution. There has been some minor reinforcement spalling and historic repairs on the West face of the building other than this no significant degradation of the building was observed.
Rough order of cost estimate for seismic improvements (where required)	$200,000-$500,000
Timeline for remediation if required	Medium Priority

Commentary:

The main limiting aspects for the building are the following:

- The partially grouted block walls at ground and first floor in the longitudinal direction and the effect on the concrete frame, and the number of blockwalls on the lower level that are effective for bracing due to additional openings being introduced since construction and the effectiveness of the wall reinforcing bars connected into the concrete frame or concrete slabs.
- Connection of the first floor slab (diaphragm) into the concrete stair walls.

Based on the reinforcing detailing, the maximum ductility of the concrete frames and all wall elements (in-situ concrete and block walls) is taken 1.25 for shear and 2.0 for flexure.

The rear extension of the building, which was constructed in 1999 in lightweight timber framing, is not considered to be a limiting factor in the building performance.

Other Items: **Concrete Masonry Veneer**

The existing concrete masonry veneer on the South Wall of the building is a potential hazard as this is adjacent to escape paths will need to be investigated to confirm the condition of the wall ties and whether any remedial works should be undertaken.

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 drawings and geotechnical information where available.
    - Architectural drawings of proposed classroom block by the Nelson Education Board dated June 1964, sheets 1 to 20.
    - Structural drawings of proposed classroom block by Spencer Hollings and Ferner dated April 1964. Job number 428 sheets 4 to 16.
    - Architectural drawings of South Wing Redevelopment by Re-Design Ltd dated 1999. Sheets w1 to w28.
    - Structural drawings of South Wing Redevelopment by Abuild Consulting Engineers dated 1999. Job Number 1950, sheets S1 to S6.
- 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)	650m^2 (ground) and 630m^2 (first floor)
Year of Design (approximate)	1964 (Original building 2 storey reinforced concrete)
Current use	Classrooms and Teaching Spaces
Structural Alterations	1999 (New 2 storey Timber frame addition + alterations to the original 1964 building)
Basement	None
Gravity Load Resisting System	1964 Portion: Reinforced concrete frames and in-situ concrete walls. 1999 Addition: Timber frame and walls
Lateral Load Resisting System	1964 Portion: Reinforced concrete frames, concrete block walls and in-situ concrete walls. 1999 Addition: Timber framed structure with timber framed wall panels
Wall/Cladding/Roof System	The walls around the staircase area are in-situ concrete with the remaining walls partially filled concrete block walls. Along the perimeter, on east elevation, there are remaining precast concrete panels at ground level only and corrugated steel cladding with timber framing at first floor level. The front west elevation is mainly glazing with short height precast panels. The roof is timber framed supported on concrete frames/concrete walls with a lightweight steel cladding on top for the 1964 portion. For the rear portion the roof is timber supported on timber framed walls.

Floor System	1964 Portion The 1st floor level is reinforced concrete slab (130mm thick) 1999 Portion Timber floor
Foundation System	The foundation system is in general concrete slab on grade with concrete strip footings and pads . Locally at the north end of the building the concrete foundation are on shallow piles over and area of poor ground or fill.
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 NZS1170.5:2004. The report concluded the South Wing is likely to be partially founded on rock and partially on fill more than 3m thick based upon geotechnical investigations around the College. The stability of the slope behind the South Wing, identified in the Geotechnical Report is not any longer an issue as stabilization works recently completed have removed a substantial amount of the weathered rock stepping further back from the east elevation creating a clear path/ access.

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 Building Description

The building was designed in 1964 by Spencer Hollings and Ferner Limited as a concrete frame and wall building (with partially filled concrete masonry infill panels) assumed to be undertaken to NZSS 95:1955 Model Building Bylaw Part IV.

As such the design predates the building code NZSS 1900 Chapter 8:1965 which was the first code with a more modern approach to seismic design

The Redevelopment in 1999 was designed by ABuild Consulting Engineers Limited and was assumed to be undertaken to the design code for this time NZS 4203:1992.

The redevelopment involved some internal alterations to the 1964 building consisting of alteration to masonry infill panels, removal of some of the rear (exterior) precast spandrel panels and cladding, as well as addition of new concrete columns to strengthen the existing 1964 concrete frame building.

At this time (in 1999) a new 2 storey lightweight timber building, which appears to be designed as generally self-supporting for wind and seismic loads, was constructed at the rear of the building to form operationally the complete building in its current form.

3.2 Analysis Methodology

The force based approach method in accordance with the NZSEE assessment guidelines was used to determine the seismic capacity of the building due to the simple geometry and low rise building size.

For the concrete frame analysis on top floor level, a two-dimensional frame model was used based on a tributary area of the timber roof above for the transverse direction.

In the longitudinal direction, the concrete frames have been checked for shear strength both at ground and first floor level due to the effect from the partial infill panels.

The capacity of the wall elements, columns, diaphragm connections and foundations was assessed using guidelines given in NZS4230, NZS3101 and NZSEE 2006.

Hand calculations and structural software (Microstran 2D frame model) were used to calculate the capacity and demands of the building elements. The capacities were then compared against the demands to obtain a rating for the elements.

The walls were checked for rocking stability using a displacement based approach with a limiting drift assumed to be <1% of the building height.

There were no historical/original calculations for the 1964 or 1999 additions available to assist with the assessment.

3.3 Lateral Load Resisting System – Load path

Longitudinal Direction

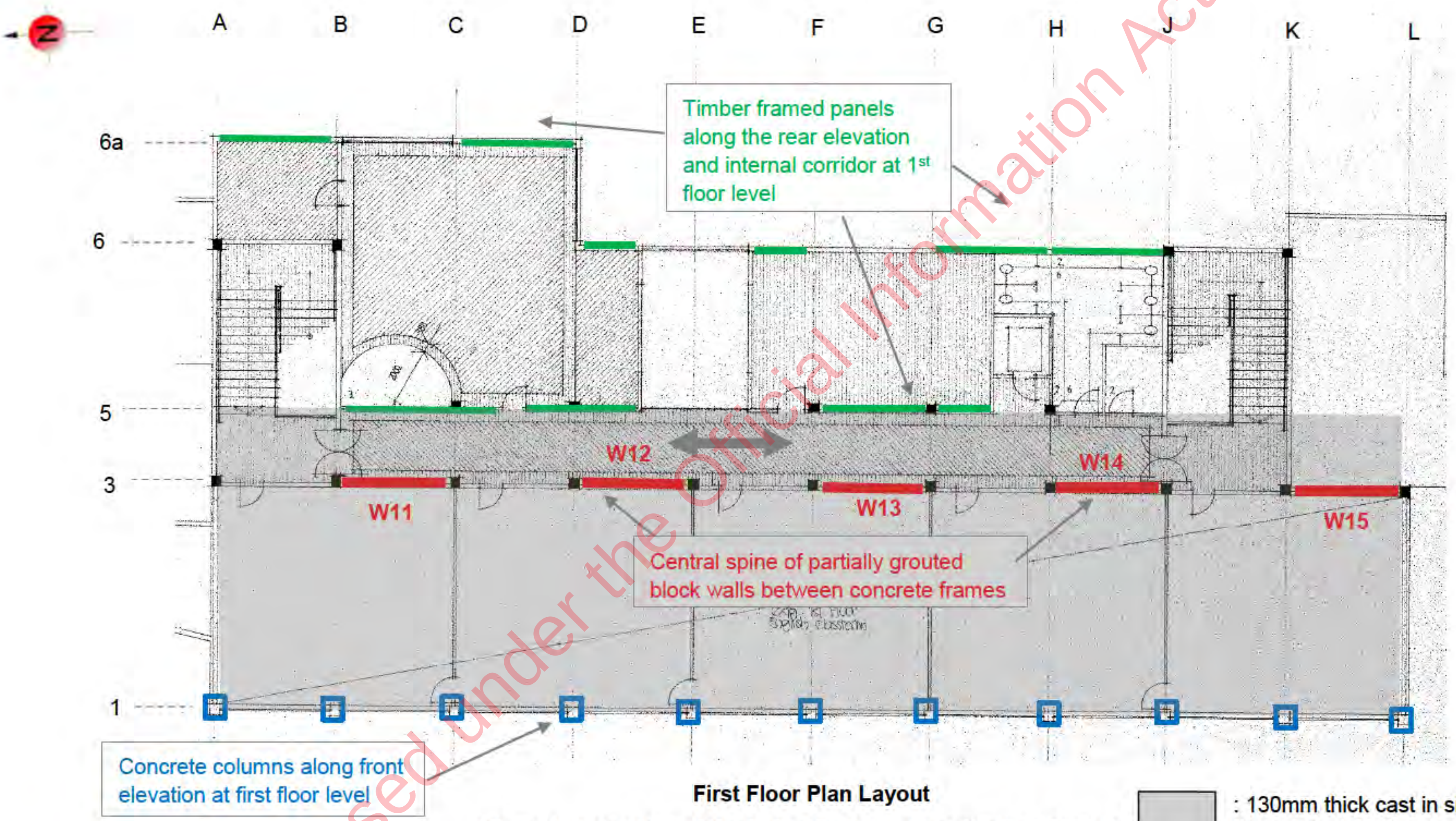
In the longitudinal direction, (approx. North to South), the lateral loads at roof level are distributed based on tributary area between the columns along the front elevation and distributed by steel roof bracing back to the first floor concrete frame (with partially infilled concrete block walls) on the central spine.

The lateral loads at first floor level are distributed through the 5" (130mm thick) in-situ concrete slab into the central spine of concrete frames and partially infilled concrete block walls at ground floor level.

The blockwork walls along the central spine sit on a small foundation beam with larger foundation pads directly under the columns. The ground floor slab is capable of redistribution of forces between the foundations.

The more recent (built 1999) lightweight timber framed 2 storey addition along the rear elevation has been designed to be self-supporting. Loads are generally distributed through flexible timber roof/floor diaphragms to the timber framed wall panels.

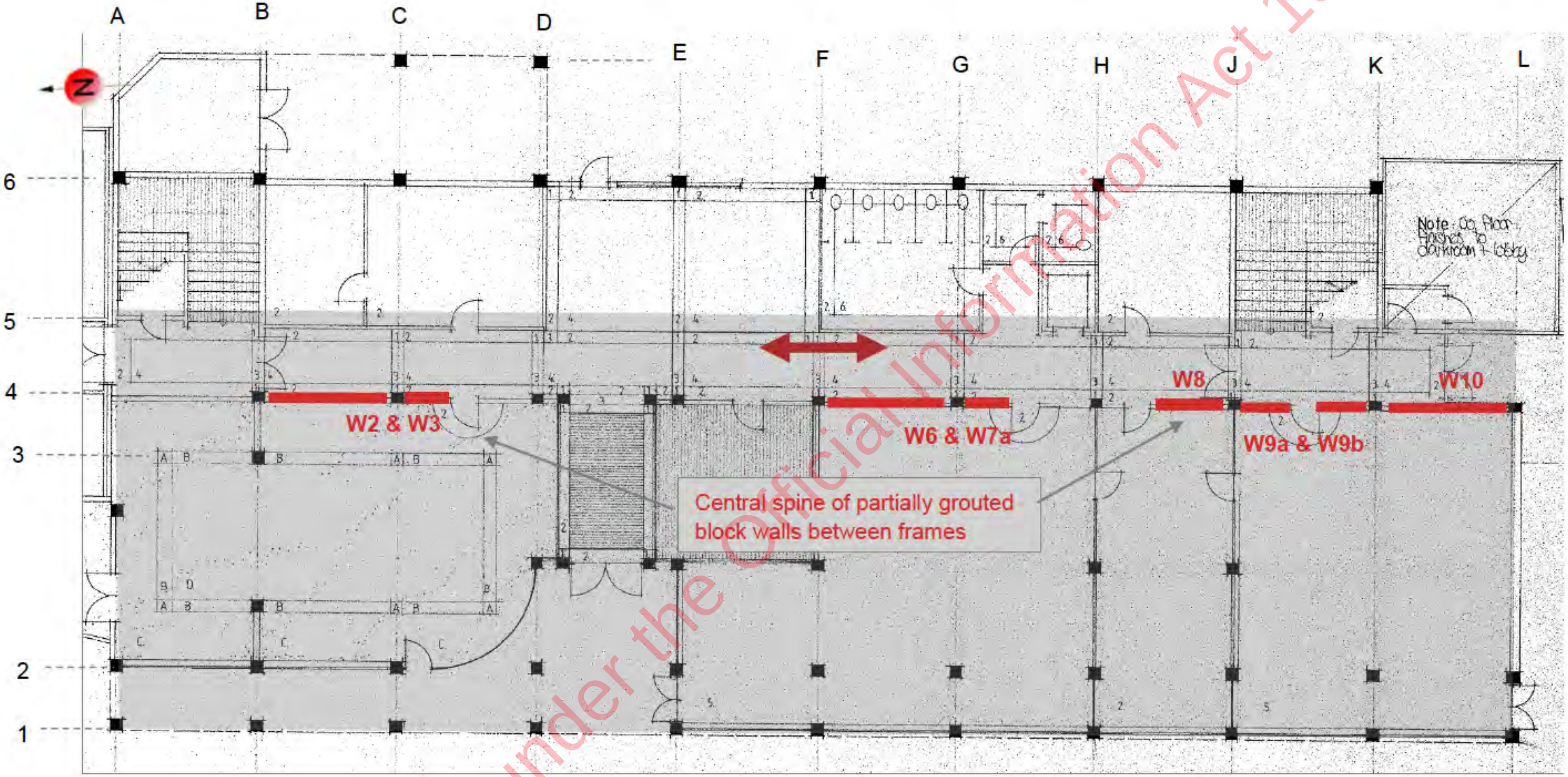
Refer to the over marked drawings Figure 1 to 3 below showing the lateral load resisting elements along the building.



First Floor Plan Layout

Figure1: Lateral Load Resisting System - Longitudinal Direction

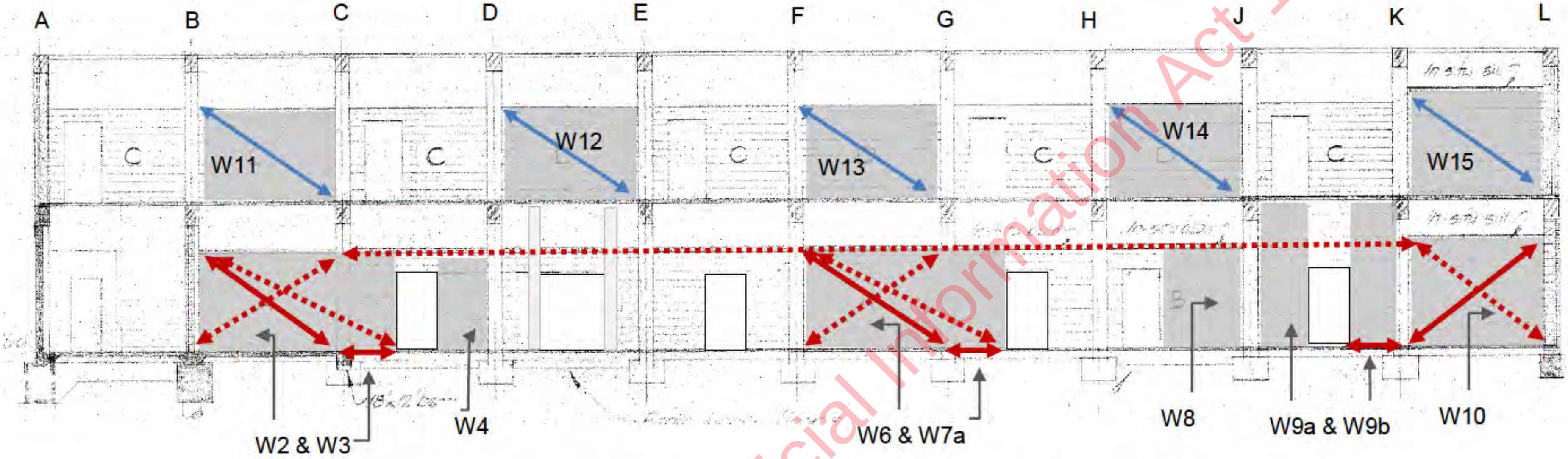
▨ : 130mm thick cast in situ floor slab at 1st floor level



Ground Floor Plan Layout

Figure 2: Lateral Load Resisting System - Longitudinal Direction

▨ : 100mm to 150mm thick cast in situ floor slab at ground floor



***The infill panels used for calculation of lateral resistance are noted above**

Figure 3: Longitudinal Section illustrating the load path from the frame into the infill block wall panels

Along Grid ‘3’ (1^{st} floor) and Grid ‘4’ (Ground floor)

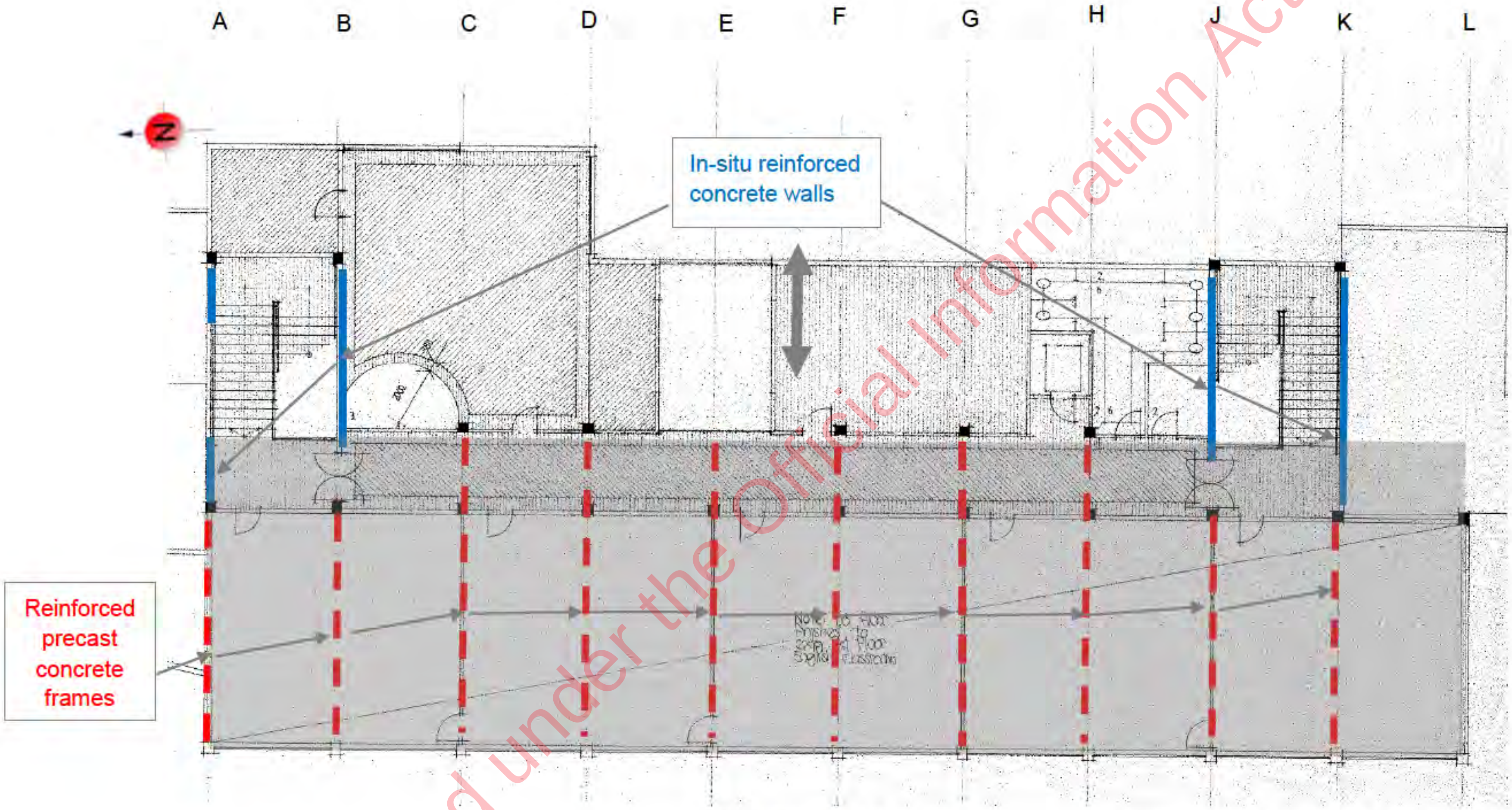
Transverse Direction

In the transverse direction (approx. East to West), the lateral loads at roof level are distributed based on tributary area to the first floor concrete portal frames.

The first floor lateral loads are then distributed through the 5" (130mm thick) in-situ concrete slab which acts as a rigid diaphragm distributing loads to the concrete walls at the staircase and reinforced concrete blockwork walls at the North and South end of the building finally to the foundations.

Therefore, in the transverse direction, the governing elements are the concrete frame at first floor and the connection of the diaphragm to the concrete staircase walls.

Refer to the over marked drawings Figures 4 to 6 below showing the lateral load resisting elements across the building.



First Floor Plan Layout

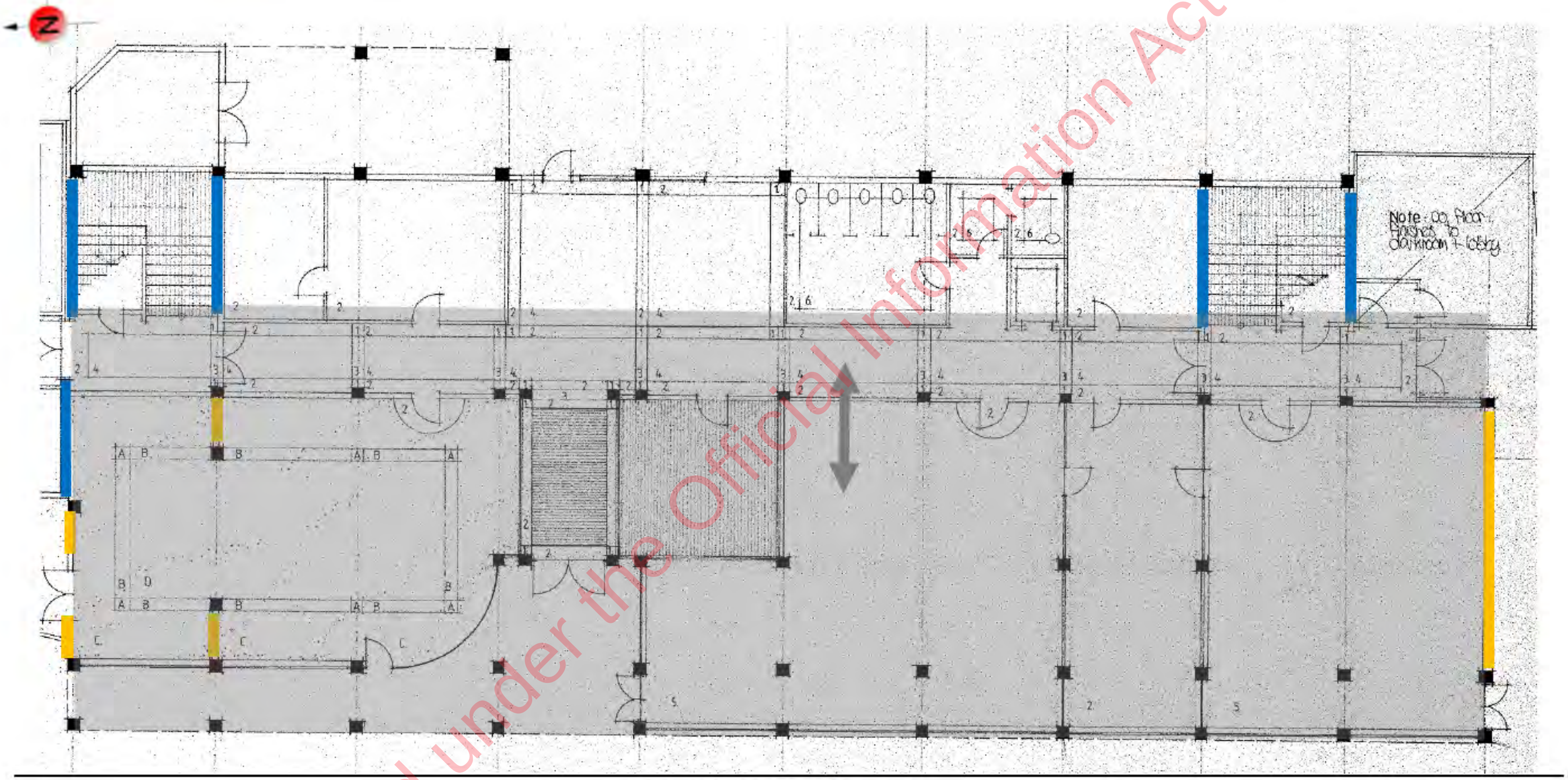
▨ 130mm thick cast in situ floor slab at 1st floor level

Figure 4: Lateral Load Resisting System - Transverse Direction

🟦 In-situ reinforced concrete walls

🟨 Reinforced concrete block walls

▨ 100mm to 150mm thick cast in situ floor slab at



Ground Floor Plan Layout

Figure 5: Lateral Load Resisting System - Transverse Direction

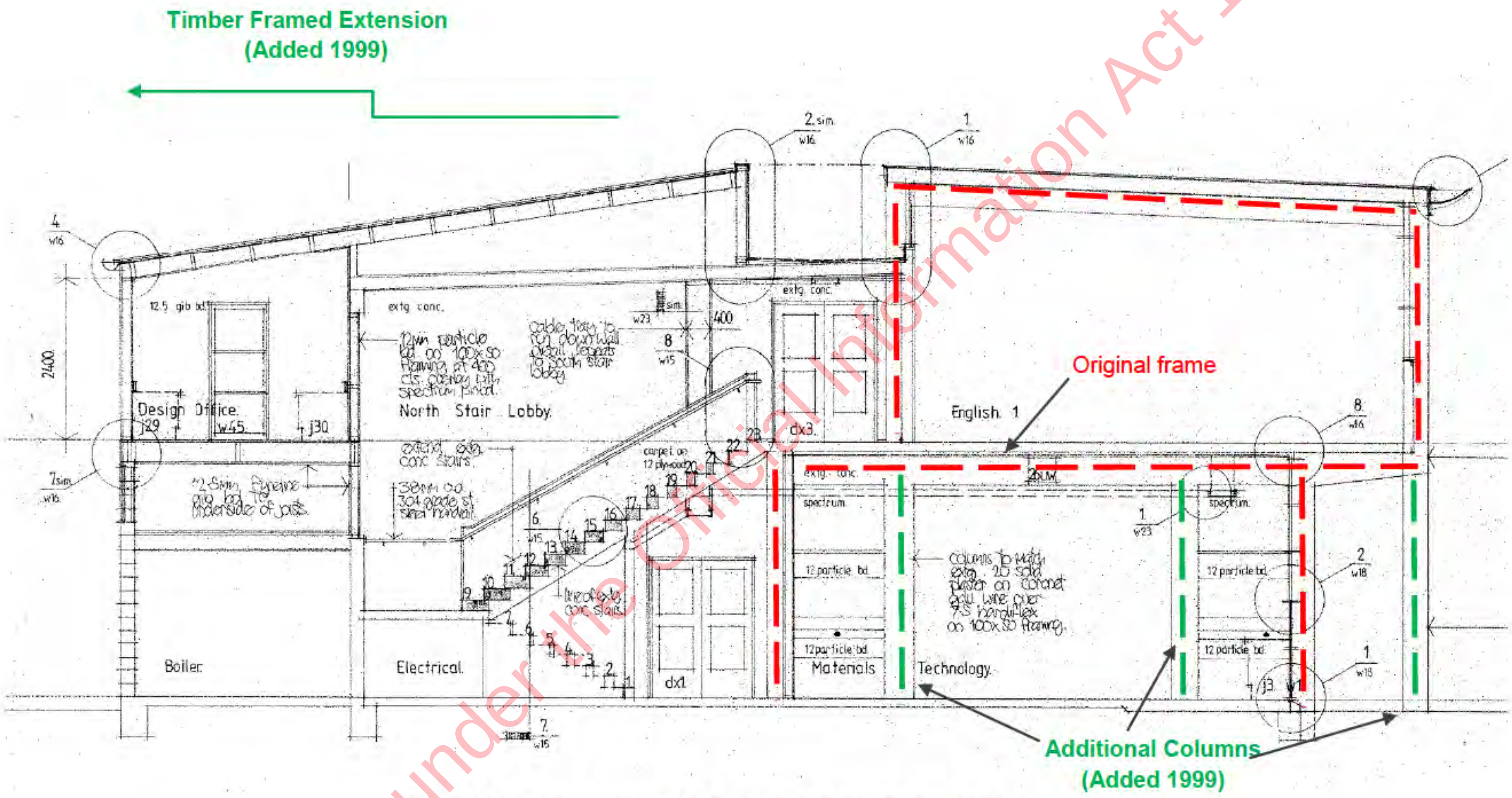


Figure 6: Section across the building (through the staircas

3.4 Intrusive Investigations

There were no intrusive investigations carried out during any of the site inspections of the buildings.

3.5 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 sec. (both directions)
Hazard Factor (Z)	0.40 - Wellington
Near Fault Factor (N)	1.0
Ductility Factors	1.25 - Limitation of reinforcing detailing and shear. 2.0 – Limitation on flexure
SP Factor	0.925 0.70

Probable material strengths are presented below in accordance with NZSEE 2006 guidelines, NZS 3101 and NZS4230:2004.

These values have been used in the analysis.

Material	Nominal Strength
Concrete – Compressive Strength	f'_c = 30MPa
Concrete Masonry Block Walls – Compressive Strength	f'_m = 12MPa
Steel Reinforcement – Yield Strength	f'_y = 300MPa

The material properties have been assumed given the age and condition of the building.

3.6 Seismic Capacity Assessment

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

Element	%NBS Capacity	Commentary
Longitudinal X Direction:		
First Floor Concrete frame/infill panels	≥ 60%	Shear capacity of the columns critical
Ground Floor Concrete frame and partially grouted infill wall panels.	50%	Shear capacity of the columns critical.
Foundation capacity of the frame/wall panels.	98%	Rocking of walls on foundations at drift limitation of 0.01h
Transverse Y Direction:		
First Floor Concrete Frame at top floor.	100%	
Diaphragm Connection to the concrete wall at 1st floor (Transverse direction).	63%	Connection of staircase concrete wall tied back to the concrete slab.
Ground Floor Reinforced Concrete Blockwork walls at ground floor.	100%	
In-situ concrete walls at ground level	>100%	Rocking of walls on foundations at drift limitation of 0.01h

The assessment confirms that the building achieves an overall low range seismic capacity of 50% NBS based on average rating of the concrete columns at ground floor level.

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

3.7 Structural Weaknesses & Life Safety Hazards

3.7.1 Potential Critical Structural Weaknesses

There are no critical Structural Weaknesses.

3.7.2 Diaphragms

Roof Level

The roof is a lightweight timber roof supported on precast concrete frames. The diaphragm at roof level is considered as flexible with the loads from timber roof spanning to the concrete frames on a tributary basis. The loads are transferred to the columns at the front and centre spine wall of the building in out of plane bending of the concrete beams. The columns at the front of the building have partial fixity to the floor beams and can provide some cantilever action to take loads from the roof. However the intended original design load path is through the structural steel tension bracing bays connected to the top of the columns which then transfer the loads back to the concrete columns along the centre spine wall of the building and into the infill walls. This load path provides a rating of 100%NBS for the roof level.

First Floor

The floor diaphragm on the first floor is a robust 5" (130mm) reinforced concrete slab and is rigid enough to transfer the total lateral load to the walls in both directions. However, the critical issue is mainly due to the actual connectivity of the first floor diaphragm into the concrete walls at the staircase in the transverse direction. Substantial forces build up in this connection and the limiting factor will be the capacity of this connection. Our assessment indicates the connection to the walls provides a rating of 63%NBS.

Ground Floor

The loads from the frames and walls are directly distributed to the foundations locally beneath each element. However there is a reinforced concrete slab at ground floor level that will be able to transfer some shear between foundations if required.

3.7.3 Stairs

The stairs are integrally connected into the concrete walls which are very squat and stiff so it is unlikely the stairs will move separately from the walls to cause any significant damage, and are not considered to be a problem for this building.

3.7.4 Precast panels

The most substantial size of the remaining precast panels (after the last alterations dated 1999) are along the rear east elevation of the building at ground floor level. These precast panels are well tied/ connected back to the concrete columns with 6 - ½" dowels and a 2½ " m.s.Angle fixings and there are no concerns with these panels under seismic loads.

Along the front west elevation, there are 1350mm high spandrel precast panels to the underside of the glazing at first floor and ground level.

These panels have been well detailed and designed to accommodate lateral movements in the structure with a 1” (25mm) nominal movement joint at each end, a 1¼” (32mm) sliding dowel connection at the top of the panel and fixed base connection consisting of a 2½ “ m.s. Angle cast into an insitu joint in the column and 12 - $^{3}/_{8}$” starter bars cast into the slab at the base of the panel.

The precast panels are calculated to have a capacity in excess of 100% NBS.

3.7.5 Concrete Frames

Longitudinal Direction

In the longitudinal direction, the shear capacity of the concrete frame columns along the internal spine is affected by the partial infill block wall panels both at ground and first floor level.

We have assessed these using the infilled frame methodology from the NZSEE guidelines with various boundary conditions and the results provide an average overall seismic rating of 50%NBS.

Transverse Direction

In the transverse direction, the concrete frames at roof level have been assessed for carrying seismic forces based on tributary area of the flexible roof diaphragm and are 100%NBS.

The seismic loads at first floor level will be transferred through the rigid diaphragm into the end concrete walls and block walls which are approximately 100%NBS.

The main limiting factor in the transverse direction is the connection to the walls discussed above.

3.7.6 Foundations

The building is supported mainly on shallow foundations (pads and ground beams) and partially only on few piles at the northern side of the building, as per original Drawing No. 24 & 26.

From our analysis of the building, it appears that the structure has insufficient weight on the longitudinal and transverse walls to fully resist earthquake overturning loads without resulting in some uplift or a rocking response mode of structural elements (walls/ footings).

Observed evidence on many new and existing buildings suggests that some local uplift and rocking is not necessarily detrimental to the seismic performance as long as secondary damage is limited, and it may be beneficial in limiting seismic forces transmitted into the structure.

For the purpose of this assessment the likelihood of the uplift or rocking response of the building is described below for each direction.

Longitudinal limiting deflection for damage to the building

In the longitudinal direction, the walls have insufficient weight to prevent uplift occurring. In order to quantify the likely behaviour a displacement method in accordance with the NZSEE assessment guidelines was used to estimate the rocking capacity on a typical single bay (of a full length wall panel) along the internal spine of the frame.

We considered a horizontal displacement demand of 40mm taken at 2/3 of the building height (at which 35mm uplift of the foundation occurs) in our analysis as a limiting displacement. This corresponds to a displacement of approximately 1% drift, a reasonably conservative assessment figure taken to limit damage to the adjacent structure from the wall element rocking.

This provided a capacity a ratio, using the displacement method of at least 98% NBS for this limit.

Transverse limiting deflection

In the transverse direction the walls will also start to rock and a similar analysis to above was undertaken. Although these walls could accept a higher drift limit than the longitudinal walls we reviewed these with the conservative assessment of 1% drift limit and found this provided a capacity of over 100% NBS for this limit.

Walls at the Stair (Grid B) with piles

However, the presence of a series of few piles along Grid line B affects the behaviour of one of the staircase walls. In particular, one end of the wall is supported on a pile and the other end on a shallow footing which is bearing onto a rock, as per Dwg. No. 26.

Hence, in this case, only the end of the wall at the shallow footing will try to uplift and the other end on the pile can only experience some yielding of the reinforcing bars.

We have reviewed the capacity of the wall / pile under the fixed condition and the wall is rated at greater that 100% NBS for nominally elastic loads.

3.7.7 Secondary Structural Weaknesses & Life Safety Hazards

The existing concrete masonry veneer on the South Wall of the building is a potential hazard as this is adjacent to escape paths will need to be investigated to confirm the condition of the wall ties and whether any remedial works should be undertaken.

4. Seismic Improvements

4.1 Suggested Improvements

To increase the seismic %NBS capacity from 50%NBS at IL3 to achieve a minimum 67%NBS at IL3 capacity as recommended by MOE guidelines the following seismic improvements are recommended.

Description of suggested improvements:

- Improve the shear capacity of the RC columns in the longitudinal frame at ground and first floor level.
- Increase the shear capacity of existing partially filled block walls by grouting the unfilled cells.
- Improve the diaphragm connections to the concrete walls providing floor plates or ties.
- Investigate and upgrade if necessary the ties to the concrete masonry veneer on the south wall.

4.2 Rough Order of Cost Estimate

A rough order of cost estimate for the suggested physical improvements above is $200,000-$500,000 Excluding GST.

The above rough order of cost estimate is for the structural improvements only and does not allow for the following:

- Building Consent Fees
- Consultancy fees
- Alterations and making good to architectural and building services components to incorporate the suggested seismic improvements.
- Other costs associated with upgrades that may be considered if a strengthening project was to proceed
- Cost escalations

A more accurate cost estimate should be developed after completing a detailed design for the suggested structural improvements and with the engagement of a qualified builder and/or quantity surveyor.

5. Conclusions & Recommendations

5.1 Conclusions

The building achieves an overall seismic capacity of 50%NBS at Importance Level 3.

The building meets the Ministry of Education's minimum seismic strength requirements of not being earthquake-prone or >34% NBS in the short term, but does not meet the Ministry of Educations medium term goal of achieving 67% NBS or above for their building stock.

5.2 Recommendations

Seismic Improvements

The building is not earthquake prone, and there is no need to change the buildings current occupancy, but we recommend the Ministry consider undertaking the suggested improvements to the building to achieve a minimum seismic capacity of 67%NBS in the medium to long term.

These seismic improvements have a rough order of cost estimated as $200,000 to $500,000 excluding GST.

Other Items: Concrete Masonry Veneer

The existing concrete masonry veneer on the South Wall of the building is a potential hazard as this is adjacent to escape paths will need to be investigated to confirm the condition of the wall ties and whether any remedial works should be undertaken.

A recommended time for remediation is to be a medium priority in view of the overall rating.

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: 5-PA010.37 **Sheet No** 0 of 80

Project Description: Wellington East Girls College, Block 4 - South Wing **Office:** Wellington

Computed: MX

REVISED Nov. 2015

Contents

* It is recommended to carry out a site investigation for the blockwalls and masonry veneer for health & safety reasons to pedestrians.

References

1. NZS1170.5:2004 Earthquake Actions
2. NZS 4230:2004 Design of Reinforced Concrete Masonry Structures
3. NZS3101:2006 Design of Concrete Structures
4. NZSEE guidelines 2006

CALCULATION SHEET

Project/Task/File No: 5-PA010.37

Project Description: Wellington East Girls College, Block 4 - South Wing

Sheet No 01 of

Office: 0

Computed: 0

Check: 0

1.0 Introduction

1.1 Summary/ brief description of revisions made and alterations

- Original Mechanism adopted: → Partially Reinforced Masonry Blockwalls.

Based on limitted number of effective wall panels

- Shear Capacity : ~ 50%.
- Flexure Capacity : ~ 36%.

} Longitudinal direction was critical.

- Final Mechanism adopted: → Effect of partial infills on Concrete Frame.

- Ground level : ~(43-62) ≃ 50% Average.
- 1st floor level : ≥ 60%

} - Shear Critical on Longitudinal direction

- Additional Calculations have been added for:

1. Foundations / wall rocking response
2. Roof diaphragm
3. Out-of-plane check of precast panel at 1st floor level

CALCULATION SHEET

Project/Task/File No: 5-PA010.37

Project Description: Wellington East Girls College, Block 4 - South Wing

Sheet No 02 **of**

Office: 0

Computed: 0

Check: 0

1.2 ***Ductility of frames and walls***

Concrete frame (Top) – As per NZSEE 2006, Sect.7.

- Spacing of stirrups : s = 9" crs = 230 mm
- For effective depth : d = 330 mm

$\Big\}\ s > d/2$

→ An available $\mu = 2$ may be assumed.

Given the detailing of Top frame column at the corners and the connection ties into the precast frame below, a ductility of $\mu = 1.25$ has been adopted.

Walls.

- Due to the partially grouted block walls and the reinforcing detailing of in-situ concrete walls, a ductility of $\mu = 1.25$ has been adopted for shear capacity checks & $\mu = 2.0$ for flexure capacity checks.

CALCULATION SHEET

Project/Task/File No: Sheet No 03 of

Project Description: Office:

2. DESIGN PARAMETERS.

2.1 Material Properties.

Computed: / /

Check: / /

1. CONCRETE BLOCKWALL - TYPE B - Masonry Construction to NZS 4229.

> DESIGN STRENGTH USED:

- Compression stress of masonry : $f_m = 12$ MPa.
- Shear provided by masonry of limited ductile structures. $(\mu = 1.0 - 1.25)$: $v_{bm} = 0.2\sqrt{f_m} = 0.7$ MPa.

> STRENGTH REDUCTION FACTOR :

- Shear & shear friction : $\phi = 1.0$
- Flexure : $\phi = 1.0$

} for assesment on existing.

2. CONCRETE

> Compressive Strength : $f'_c = 30$ MPa

3. STEEL REINFORCEMENT

> Yield Strength : $f_y = 300$ MPa

CALCULATION SHEET

Project/Task/File No:

Project Description:

Sheet No 4 **of**

Office:

Computed: / /

Check: / /

2.2 Building Weights

ROOF : –

- Dead = ~~0.6~~ 0.4 kN/m² (light weight timber framed) & plaster tiles

WALLS :

- Timber Framed = 0.3 kN/m².
- 6" thk. blockwall (150 mm thk.)

$22 \frac{kN}{m^3} \times 0.15m = 3.3\ kN/m^2$

- 8" thk. blockwall (200 mm thk.)

$22 \frac{kN}{m^3} \times 0.2m = 4.4\ kN/m^2$

- 8" thk. In-situ (200 mm thk.)

$24 \frac{kN}{m^3} \times 0.2m = 4.8\ kN/m^2.$

COLUMNS

- 15" × 15" ~ (380 × 380) :

$24 \frac{kN}{m^3} \times 0.38^m \times 0.38^m = 3.46 \frac{kN}{m}$

1st FLOOR SLAB (5" thk. = 127mm thk.)

$24 \frac{kN}{m^3} \times 0.13^m = 3.12\ kN/m^2.$

CONCRETE PANELS (6" thk. = 150mm thk.)

$24\ kN/m^3 \times 0.15^m = 3.6\ kN/m^2.$

Have been removed (on east elevation) @ 1st floor due to extension.

OPUS

CALCULATION SHEET

Project/Task/File No: Sheet No 5 of

Project Description: Office:

WEIGHT OF BUILDING Computed: / /

- LUMPED AT 1st FLOOR LEVEL. - Check: / /

(kN)

Beams @ 1st FL.

$24\,kN/m^3 \times 0.61 \times 0.38 \times (7.695 + 1.68) \times 9$ No. = 469.4

Walls

* Timber Studs

\- DL = $0.3\,kN/m^2 \times \left[\left(7{,}315^m \times \frac{3.8^m}{2}\right) \times 4$ No. $+ \left(7.315^m \times \frac{3.8^m}{2}\right) \times 3^{No.}\right]$ = 45.9

* RC. 200mm thk :

\- 1st fl. : $4.8\,kN/m^2 \times \left[2 \times (7.54 \times 4.1)^{m^2} + 2 \times (5.715 \times 4.1)^{m^2}\right]$ = 521.7

\- Gr. fl. : $4.8\,kN/m^2 \times \left[\left(4.4 \times \frac{4.1}{2}\right) \times 7_2 + \left(4.2 \times \frac{4.1}{2}\right) \times 2\right]$ = 255.8

* Block 200mm thk.

1st fl + GR/2 : $4.4\,\frac{kN}{m^2} \times \left[2^{No.} \times (7.315 \times 3.353) + (7.315 + 4) \times \frac{3.25}{2}\right]$ = 296.7

* Block 150mm thk.

\- 1st floor : $3.3\,kN/m^2 \times 2.44^m \times (3.6^m \times 5^{No.} + 2^m \times 5^{No.})$ = 225.5

\- Gr. floor : $3.3 \times \left[\frac{2.44}{2} \times (3.6^m \times 6\text{ No} + 2 \times 3) + \frac{3.2}{2} \times 7.315\right]$ = 149.7

Columns

(@GRID 6:) - DL = $3.46\,\frac{kN}{m} \times \left(4.1^m + 3.81^m/2\right) \times 4$ No. = 83.1

@GRID 5 : - DL = $(3.46\,kN/m \times 2.44^m \times 6$ No. = 50.6

@GRID 4 : - DL = $3.46 \times 3.81/2 \times 10$ No = 65.9

@GRID 3) : - DL = $\left(3.46\,\frac{kN}{m} \times 4.115^m\right) \times 11$ No. = 156.6

@GRID 2 : - DL = $3.46 \times 3.81/2 \times 11$ No. = 72.5

(@GRID 1) : - DL = $\left(3.46\,\frac{kN}{m^2} \times 3.8^m\right) \times 11$ No. = 144.6

RC Slab (1st Floor)

\- DL = $3.12\,\frac{kN}{m^2} \times (40.7 \times 9.775)\,m^2$ = 1241.3

Additional Timber floor @ 1st floor

\- DL = $0.6\,kN/m^2 \times \left[(4 \times 5.7)^{m^2} \times 6\text{ No} + (4 \times 4)^{m^2} \times 3\text{ No}\right]$ = 110.9

\- LL = $\left[3\,\frac{kN}{m^2} \times (184.8 + 397.8)\,m^2\right] \times 0.3$ = 524.3

ΣW_i = 4414.5 kN.

CALCULATION SHEET

Project/Task/File No:
Sheet No 6 of
Project Description:
Office:
// BUILDING WEIGHT
Computed: / /
REVIEWED - SUMMARISED //
Check: / /

Roof : (Area = 16 × 41 = 656 m^2)

(kN)

- Dead = 0.4 kN/m^2 × 656 m^2 = 262.4
- Concrete Beams : 24 kN/m^3 × (7.7 m × 0.457 × 0.38) × 10 No = 320.9

ΣW_{ROOF} = 583.3

1st Floor Level (Area_SLAB = 400 m^2)
(Area_timber floor = 78 m^2)

- RC slab : 3.12 kN/m^2 × (41×8 + 2×36) = 1248
  (5" thk)
  130 mm thk.
- Timber floor :
  Dead = 0.6 kN/m^2 × 78 m^2 = 46.8
  Live = 3 kN/m^2 × 478 m^2 × 0.3 = 430.2
  tot area
- RC Beams : 24 × 0.61 × 0.38 × (7.7 + 1.68) × 10 = 521.8
- RC walls (200mm thk) :
  4.8 kN/m^2 × 5.7 × 3.6 × 4 No. = 394
  long high
- Blockwalls (6" thk = 150 mm thk)
  3.3 kN/m^2 × 2.6 × 3.835 × 6No = 197.4
  high long
- Blockwalls (8" thk = 200 mm thk)
  4.4 kN/m^2 × 3.3 × 7.3 × 2 No = 211.9
- Columns (380 × 380)
  3.46 kN/m × 3.6 m × 22 No. = 274

3324.

CALCULATION SHEET

Project/Task/File No:

Sheet No 7 of

Project Description:

Office:

Computed: / /

Check: / /

GROUND FLOOR LEVEL

(kN.)

- Blockwalls (150 mm thk.)

$3.3 \text{ kN/m}^2 \times \underset{\text{high}}{2.45} \times 3.835 \times 3\text{No.}$ = 93.

$3.3 \times \left[\left(2.45 \times 1.5 \times 8\text{No.} \right) + \left(\begin{array}{l} 2.45 \times 2.3 \\ + 3.2 \times 7.3 \end{array} \right) \right]$ = 192.7

- Blockwalls (200 mm thk)

$4.4 \text{ kN/m}^2 \times 3.2 \times (7.315 + 3.3)$ = 149.5

- RC walls (200 mm thk)

$4.8 \text{ kN/m}^2 \times 3.2 \times 3.9 \times 5\text{No.}$ = 299.5

ΣW_{GF} = 734.7

ΣW_{tot} = 4640 kN

REVISED

CALCULATION SHEET

Project/Task/File No:
Sheet No 8 of
Project Description:
Office:
2.3 Seismic Load Assumptions
Computed: / /
& Hor. Design Coef.
Check: / /

SEISMIC LOADING ASSUMPTIONS

For Wellington : $Z = 0.4$.

Soil Class : B , $C_h(T) = 1.89$.

Importance Level 3 : $R_u = 1.3$

$T_0 \leq 0.4$ sec : $N(T_0, D) = 1.0$

HOR. DESIGN ACTION COEFFICIENT

- For $\mu = 1.25 \rightarrow S_p = 0.925$ & $k_\mu = 1.14$.

& $C_d(T) = C_h(T) \cdot Z \times R_u \times N(T,D) \times \frac{S_p}{k_\mu}$

$\therefore C_d(T) = 1.89 \times 0.4 \times 1.3 \times 1.0 \times \frac{0.925}{1.14}$

$\therefore C_d(T) = 0.797 \simeq \underline{\underline{0.8}}$ — 1st floor level

---

- For $\mu = 2$

$\rightarrow S_p = 0.7$ & $k_\mu = \frac{0.4}{0.7} + 1 = 1.57$

& $C_d(T) = C_h(T) \times Z \times R_u \times N \times S_p / k_\mu$

$C_d(T) = 1.89 \times 0.4 \times 1.3 \times 1.0 \times \frac{0.7}{1.57}$

$C_d(T) = \underline{\underline{0.438}}$ — Used for flexure check ← (Rev. 15/10/15)
& foundation check.

CALCULATION SHEET

Project/Task/File No:

Sheet No 9 of

Project Description:

Office:

Computed: / /

Check: / /

2.4 Total Base Shear Demand

Total Shear : V_b

$V_b = C_d(T) \times W_t$

$\therefore V_b = 0.8 \times 4415$

$\therefore V_b = \underline{\underline{3532 \text{ kN.}}}$ — for $\mu = 1.25$ (for frames and all walls)

AND.

$V_b = 0.438 \times 4415$

$\therefore V_b = \underline{\underline{1934 \text{ kN.}}}$ — for $\mu = 2.0$ (foundation check)

CALCULATION SHEET

Project/Task/File No: 5-PA010.37 Sheet No 10 of

Project Description: WEGC - SOUTH WING Office:

3. CAPACITY / DESIGN CHECKS Computed: / /

3.1 Reinforced Masonry Walls. Check: / /

Based on NZS 4230:2004

1. Shear Capacity:

$\phi V_n \geq V^*$ $(\phi = \overset{1.0}{0.\cancel{75}})$

Where: V^* _ shear demand.

V_n _ nominal shear strength

$\therefore V_n = v_n b_w d$

$\therefore V_n = (v_m + v_s) b_w d$

ϕ $v_n = v_m + v_s + v_p \rightarrow 0$

With: a) v_m = shear strength provided by masonry.

b) v_s = shear stress provided by shear reinforcement

& c) v_p = shear stress provided by masonry under axial load

Therefore,

$\star$ $\boxed{v_m = (C_1 + C_2) v_{bm}}$

With: • $C_1 = 33 \times \rho_w \frac{f_y}{300}$

- $f_y = 300$ MPa (Reinforcement)
- $\rho_w = 3.8 \times 10^{-3}$ - for 6" thk. block.
- $\rho_w = 6.7 \times 10^{-3}$ - for 8" thk. block.

$\therefore \underset{6'' thk.}{C_1} = \underline{\underline{0.125}}.$

& $\underset{8'' thk.}{C_1} = \underline{\underline{0.221}}$

• $C_2 = 1.5$ _ for $h_e/L_w < 0.25$

or $C_2 = 0.42[4 - 1.75(h_e/L_w)]$ _ for $0.25 \leq h_e/L_w \leq 1.0$

or $C_2 = 1.0$ _ for $h_e/L_w > 1.0$.

& ϕ $\boxed{v_s = C_3 \frac{A_v f_y}{b_w s}}$

Where: • $C_3 = 0.8$ — for walls

• $A_v = \frac{\pi D^2}{4} <$ 126.7 mm² (6" thk.) / 253.4 mm² (8" thk.)

• $f_y = 300$ MPa

• $b_w = <$ 70 mm (6" thk) / 80 mm (8" thk.)

• $s = 600$ mm

OPUS

REFER TO SPREADSHEET TABLE 1

CALCULATION SHEET

Project/Task/File No:

Sheet No 11 of

Project Description:

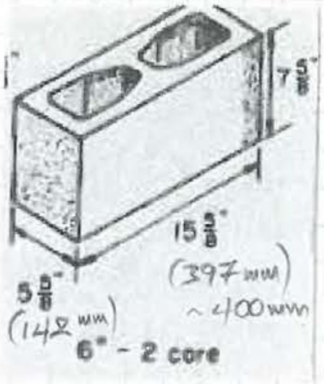
Office:

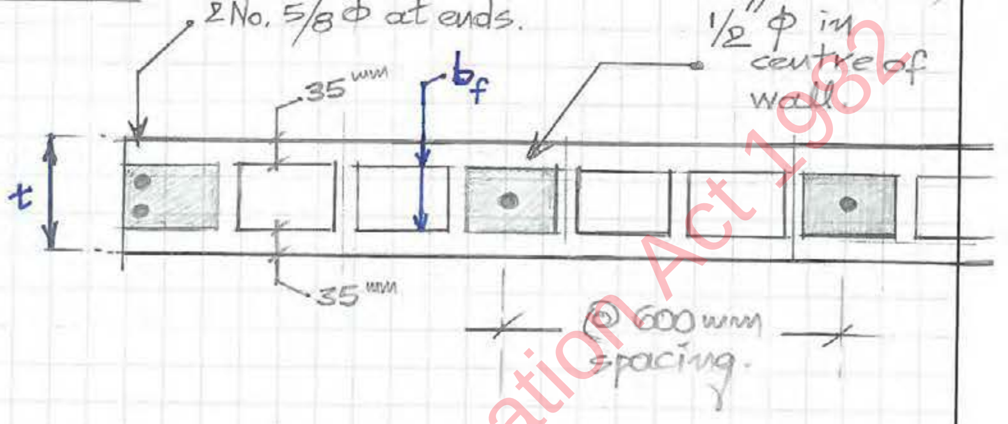
CONCRETE BLOCKS

Computed: / /

Check: / /

6" BLOCK WALL ~ (152 mm)





For partially grouted wall - in plane: (Fig. 10.1, NZS 4230:2004)

- Length: $d = 0.8 L_w$
- Eff. width: $b_w = t - b_f = 70\text{mm}$
- Area of steel reinf.: $A_s = \frac{\pi D^2}{4} = \frac{\pi \times 12.7^2}{4} = 126.7\text{mm}^2$

$\therefore \frac{A_s}{s} = \frac{126.7\text{ mm}^2}{600\text{ mm}} = 0.211\text{ mm}$

- Volumetric ratio: $\rho_w = \frac{A_s}{b_w d} = \frac{(A_s/s) \times L}{b_w \times 0.8 L}$

$\therefore \rho_w = \frac{0.211\text{ mm}}{70\text{ mm} \times 0.8} = 3.8 \times 10^{-3}$

OPUS

CALCULATION SHEET

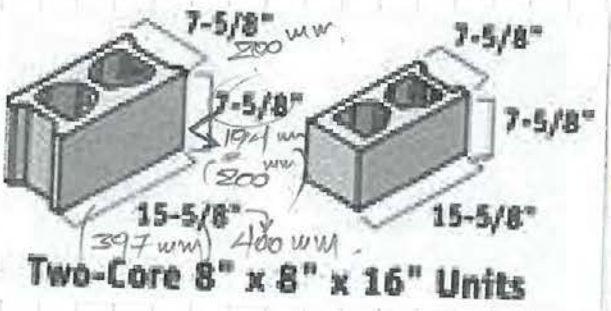
Project/Task/File No:　　　　Sheet No 12　of

Project Description:　　　　Office:

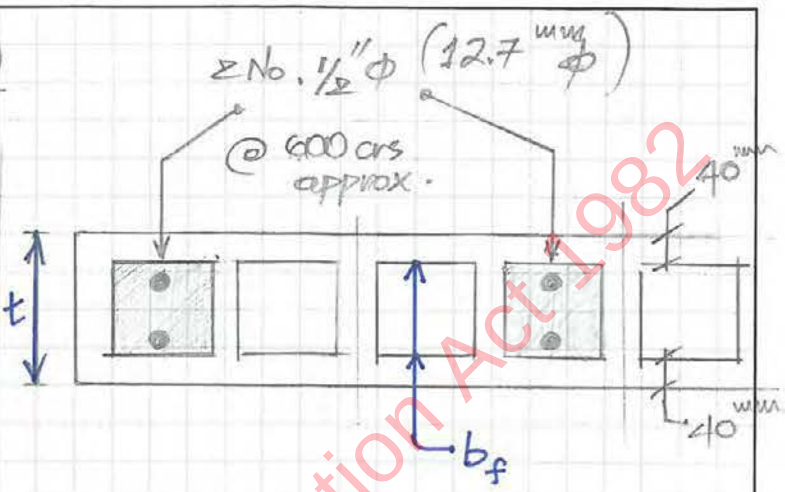
Computed:　　/　　/

Check:　　/　　/

8" Block Wall – (203 mm)



Two-Core 8" x 8" x 16" Units



For partially grouted wall - in plane

- Length : $d = 0.8 L_w$.
- Eff. width : $b_w = t - b_f = \underline{\underline{80\,mm}}$.
- Area of steel reinf. : $A_s = \frac{\pi D^2}{4} \times 2 = \frac{\pi \times 12.7^2}{2} = 253.4\,mm^2$

$$\therefore \frac{A_s}{s} = \frac{253.4\,mm^2}{600\,mm} = 0.422\,mm$$

- Volumetric ratio : $\rho_w = \frac{A_s}{b_w d} = \frac{(A_s/s) \times \cancel{L}}{b_w \cdot 0.8 \cancel{L}}$

$$\therefore \rho_w = \frac{0.422\,mm}{80\,mm \times 0.8} = \underline{\underline{6.7 \times 10^{-3}}}$$

OPUS

TABLE 1 SHEAR CAPACITY CHECKS - BLOCK WALLS

X DIRECTION at Ground Storey

~~SUPERSEDED.~~

SUPERSEDED

↓

DUE TO REVISED
RE-DISTRIBUTION OF
FORCES.

Design Parameters

6" thk part. grouted concrete blocks

μ	1.25
ϕ	1.00
t (mm)	70
v_{bm} (MPa)	0.70
f_y (MPa)	300
ρ_m (kN/m^3)	22
Av/ s (mm^2/mm)	0.211

$V_{o,\mu=1}$ (kN)	3532

X Direction

	Wall Ref.	H (m)	L_{xx} (m)	L_{yy} (m)	$(L_{xx}/H)^3$	a_{xx}	V^*_{Demand} (kN)	ρ_w	C_1	C_2	v_m (MPa)	$C_{3\text{ - for walls}}$	v_s (MPa)	v_n (MPa)	$V_{n,Capacity}$ (kN)	$\phi V_{n,Capacity}$ (kN)	%NBS
Concrete block walls	W2	2.44	3.660	-	3.38	0.299	1055	0.0038	0.124	1.190	0.920	0.800	0.724	1.644	336.95	336.95	32
	W3	2.44	1.350	-	0.17	0.015	53	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	114.23	114.23	216
	W4	2.44	1.340	-	0.17	0.015	52	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	113.38	113.38	219
	W5a	2.44	1.350	-	0.17	0.015	53	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	114.23	114.23	216
	W5b	2.44	1.350		0.17	0.015	53	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	114.23	114.23	216
	W6	2.44	3.660	-	3.38	0.299	1055	0.0038	0.124	1.190	0.920	0.800	0.724	1.644	336.95	336.95	32
	W7a	2.44	1.350	-	0.17	0.015	53	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	114.23	114.23	216
	W7b	2.44	1.350		0.17	0.015	53	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	114.23	114.23	216
	W8	2.44	2.250	-	0.78	0.069	245	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	190.38	190.38	78
	W9a	3.66	1.350		0.05	0.004	16	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	114.23	114.23	728
	W9b	3.66	1.350	-	0.05	0.004	16	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	114.23	114.23	728
	W10	2.65	3.660	-	2.65	0.235	828	0.0038	0.124	1.149	0.891	0.800	0.724	1.615	331.04	331.04	40
		TOTAL	24.020		11.30	1.00	3532									2108.29	60

REV.A **TABLE 1**

SHEAR CAPACITY CHECKS - BLOCK WALLS

X DIRECTION at Ground Storey

Design Parameters

6" thk part. grouted concrete blocks	
μ	1.25
ϕ	1.00
t (mm)	70
v_{bm} (MPa)	0.70
f_y (MPa)	300
ρ_m (kN/m^3)	22
Av/ s (mm^2/mm)	0.211

$V_{o,\mu=1.25}$ (kN)	3532

X Direction

	Wall Ref.	H (m)	L_{xx} (m)	t (m)	Cross Sect. Area L x t (m^2)	a_{xx}	V^*_{Demand} (kN)	ρ_w	C_1	C_2	v_m (MPa)	C_3 - for walls	v_s (MPa)	v_n (MPa)	$V_{n,Capacity}$ (kN)	$\phi V_{n,Capacity}$ (kN)	%NBS
Concrete block walls	W2	2.44	3.660	0.152	0.56	0.152	538	0.0038	0.124	1.190	0.920	0.800	0.724	1.644	336.95	336.95	63
	W3	2.44	1.350	0.152	0.21	0.056	199	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	114.23	114.23	58
	W4	2.44	1.340	0.152	0.20	0.056	197	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	113.38	113.38	58
	W5a	2.44	1.350	0.152	0.21	0.056	199	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	114.23	114.23	58
	W5b	2.44	1.350	0.152	0.21	0.056	199	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	114.23	114.23	58
	W6	2.44	3.660	0.152	0.56	0.152	538	0.0038	0.124	1.190	0.920	0.800	0.724	1.644	336.95	336.95	63
	W7a	2.44	1.350	0.152	0.21	0.056	199	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	114.23	114.23	58
	W7b	2.44	1.350	0.152	0.21	0.056	199	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	114.23	114.23	58
	W8	2.44	2.250	0.152	0.34	0.094	331	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	190.38	190.38	58
	W9a	3.66	1.350	0.152	0.21	0.056	199	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	114.23	114.23	58
	W9b	3.66	1.350	0.152	0.21	0.056	199	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	114.23	114.23	58
	W10	2.65	3.660	0.152	0.56	0.152	538	0.0038	0.124	1.149	0.891	0.800	0.724	1.615	331.04	331.04	62
	TOTAL		24.020		3.65	1.00	3532										
																2108.29	**58.7**

* SUPERSEDED

↓

MAKE USE OF LESSER NUMBER OF EFFECTIVE WALLS.

NOTE:

* RE-DISTRIBUTED SHEAR FORCE BASED ON CROSS-SECTIONAL AREA OF WALLS.

* KEPT ORIGINAL No. OF WALLS, AS PER ORIGINAL CALCS.

-15

REV. * 8/10/15.

TABLE 1 **SHEAR CAPACITY CHECKS - BLOCK WALLS**

REV B. **X DIRECTION at Ground Storey**

Design Parameters

6" thk part. grouted concrete blocks	
μ	1.25
ϕ	1.00
t (mm)	70
v_{bm} (MPa)	0.70
f_y (MPa)	300
ρ_m (kN/m^3)	22
Av/ s (mm^2/mm)	0.211

$V_{o,\mu=1.25}$ (kN)	3532

X Direction

	Wall Ref.	H (m)	L_{xx} (m)	t (m)	Cross Sect. Area L x t (m^2)	a_{xx}	V^*_{Demand} (kN)	ρ_w	C_1	C_2	v_m (MPa)	C_3 - for walls	v_s (MPa)	v_n (MPa)	$V_{n,Capacity}$ (kN)	$\phi V_{n,Capacity}$ (kN)	%NBS
Concrete block walls	W2	2.44	3.835	0.152	0.58	0.195	687	0.0038	0.124	1.212	0.936	0.800	0.724	1.660	356.42	356.42	52
	W3	2.44	1.468	0.152	0.22	0.074	263	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	124.21	124.21	47
	W4	2.44	1.468	0.152	0.22	0.074	263	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	124.21	124.21	47
	W6	2.44	3.835	0.152	0.58	0.195	687	0.0038	0.124	1.212	0.936	0.800	0.724	1.660	356.42	356.42	52
	W8	2.44	2.335	0.152	0.35	0.118	418	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	197.57	197.57	47
	W9a	3.66	1.468	0.152	0.22	0.074	263	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	124.21	124.21	47
	W9b	3.66	1.468	0.152	0.22	0.074	263	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	124.21	124.21	47
	W10	2.65	3.835	0.152	0.58	0.195	687	0.0038	0.124	1.173	0.908	0.800	0.724	1.632	350.51	350.51	51
		TOTAL	19.712		3.00	1.00	3532									1757.77	**48.9**

~ 49% → 50%.

NOTES :

* ALLOWED LESS NUMBER OF WALLS. (SEE ELEVATION DWG.)

* DISTRIBUTED SHEAR FORCE BASED ON CROSS - SECTIONAL AREA.

TABLE SHEAR FORCE DISTRIBUTION & BLOCK WALL SHEAR CAPACITY CHECKS
Y DIRECTION at Ground Storey

Design Parameters

6" thk part. grouted concrete blocks

μ	1.25
φ	1.00
t (mm)	70
v_{bm} (MPa)	0.70
f_y (MPa)	300
ρ_m (kN/m^3)	22
Av/ s (mm^2/mm)	0.211

8" thk part. grouted concrete blocks

μ	1.25
φ	1.00
t (mm)	80
v_{bm} (MPa)	0.70
f_y (MPa)	300
ρ_m (kN/m^3)	22
Av/ s (mm^2/mr	0.422

Modulus of Elasticity	E (MPa)	E (kPa)
Concrete walls	25000	25000000
Masonry walls	15000	15000000

$V_{o,\mu}$ (kN)	3532

	Wall Ref.	H (m)	Thick. t (m)	L_{yy} (m)	I (m^4)	E*I	EI/ΣEI	V^*_{Demand} (kN)	ρ_w	C_1	C_2	v_m (MPa)	C_3 - for walls	v_s (MPa)	v_n (MPa)	$V_{n,Capacity}$ (kN)	$\phi V_{n,Capacity}$ (kN)	%NBS
Concrete block walls	W1b	3.25	0.150	1.700	0.061	921188	0.004	14.91	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	143.84	143.84	965
	W1a	3.25	0.150	1.700	0.061	921188	0.004	14.91	0.0038	0.124	1.000	0.787	0.800	0.724	1.511	143.84	143.84	965
	W11	3.20	0.200	7.315	6.524	97854995	0.449	1584.14	0.0038	0.124	1.358	1.038	0.800	1.266	2.304	1078.65	1078.65	68
	W12a	3.20	0.150	1.200	0.022	324000	0.001	5.25	0.0038	0.124	1.000	0.787	0.800	1.266	2.053	157.68	157.68	3006
	W12b	3.20	0.150	1.800	0.073	1093500	0.005	17.70	0.0038	0.124	1.000	0.787	0.800	1.266	2.053	236.52	236.52	1336
							TOTAL	1636.9								TOTAL	1760.5	100 %.
In-situ concrete walls	W12	3.80	0.180	3.910	0.897	22416177	0.103	362.89										
	W13	3.80	0.180	3.910	0.897	22416177	0.103	362.89										
	W14	3.80	0.200	3.910	0.996	24906863	0.114	403.21										
	W15	3.80	0.200	3.910	0.996	24906863	0.114	403.21										
	W16	3.80	0.180	3.910	0.897	22416177	0.103	362.89										
			TOTAL	33.265		218177126	1.000	3532.00										

BLOCK WALLS

Note:

1. Distribution of forces was based on effective stiffness of walls (blockwalls + in-situ concrete)
2. For Concrete wall capacities, refer to TABLE 2.

Design Parameters

TABLE 1B

IN PLANE FLEXURE CHECK - X DIRECTION @ GROUND FLOOR

μ	2.00	
φ	1.00	
t (mm)	150	6"thk blockwall
ρ (kN/m³)	22	
v_{bm} (MPa)	0.70	
f_y (MPa)	300	
f_m (MPa)	12	
A_s (mm²/m)	211	

$V_{o,\mu=2.0}$ (kN)	1934.00

$$a = \frac{N + A_s f_y}{0.85 \cdot f_m \times b}$$

X_Direction	Wall Ref.	H (m)	L_{xx} (m)	t (m)	Nn- Axial SW (kN)	Cross Sect. Area L x t (m²)	a_{xx}	V^*_{Demand} (kN)	M^*_{Demand} (kN.m)	a-depth compr. Block (mm)	L-a (mm)	d eff.depth (mm)	L/2 (mm)	a/2 (mm)	$M_{n,Capacity}$ (kN.m/m)	$M_{n,Capacity}$ (kN.m)	%NBS
	W2	2.44	3.835	0.152	8.16	0.58	0.28	535.90	1307.60	46.71	3788	1941	1918	23	136.83	524.75	40
	W6	2.44	3.835	0.152	8.16	0.58	0.28	535.90	1307.60	46.71	3788	1941	1918	23	136.83	524.75	40
	W8	2.44	2.335	0.152	8.16	0.35	0.17	326.29	796.15	46.71	2288	1191	1168	23	83.24	194.36	24
	W10	2.65	3.835	0.152	8.84	0.58	0.28	535.90	1417.46	47.15	3788	1941	1918	24	138.13	529.73	37
		TOTAL	13.840			2.10	1.000	1934.00									36

OVERWRITTEN BY ADOPTING ANOTHER MECHANISM WITH INFILL PANELS AND CONCRETE FRAME.

$$\phi M_n = \phi (N_n + A_s f_y)\left(d - \frac{a}{2}\right)$$

$\phi = 1.0$.

Design Parameters

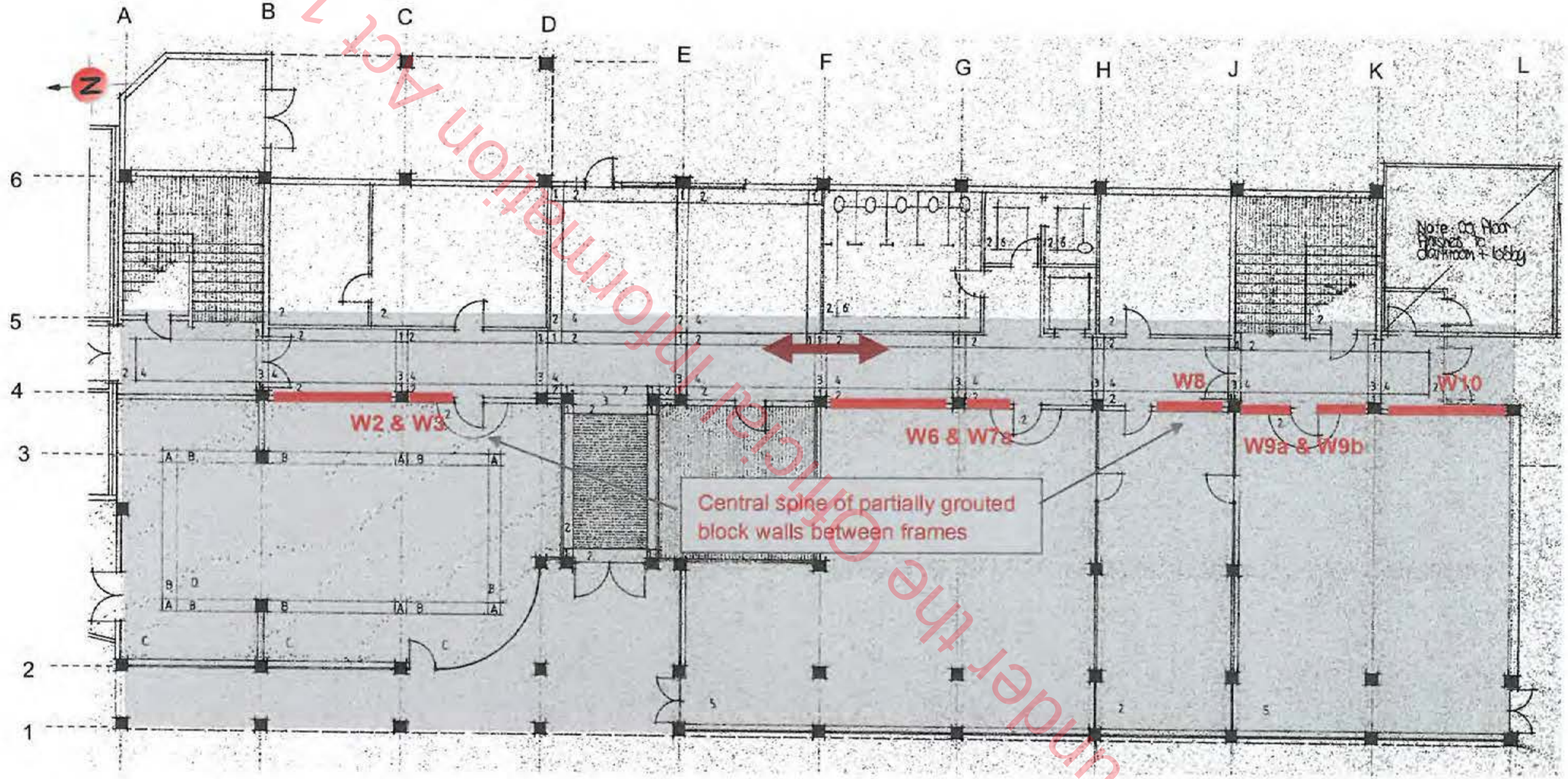
TABLE 1C

IN PLANE FLEXURE CHECK - X DIRECTION @ TOP FLOOR

μ	2.00	
ϕ	1.00	
t (mm)	150	6"thk blockwall
ρ (kN/m^3)	22	
v_{bm} (MPa)	0.70	
f_y (MPa)	300	
f_m (MPa)	12	
A_s (mm^2/m)	211	

$V_{o,\mu=2.0}$ (kN)	220.00

X_Direction	Wall Ref.	H (m)	L_{xx} (m)	t (m)	Nn- Axial SW (kN)	Cross Sect. Area L x t (m^2)	a_{xx}	V^*_{Demand} (kN)	M^*_{Demand} (kN.m)	a-depth compr. Block (mm)	L-a (mm)	d eff.depth (mm)	L/2 (mm)	a/2 (mm)	$M_{n,Capacity}$ (kN.m/m)	$M_{n,Capacity}$ (kN.m)	%NBS
	W11	2.44	3.835	0.152	8.16	0.58	0.20	44.00	107.36	46.71	3788	1941	1918	23	136.83	524.75	489
	W12	2.44	3.835	0.152	8.16	0.58	0.20	44.00	107.36	46.71	3788	1941	1918	23	136.83	524.75	489
	W13	2.44	3.835	0.152	8.16	0.58	0.20	44.00	107.36	46.71	3788	1941	1918	23	136.83	524.75	489
	W14	2.44	3.835	0.152	8.16	0.58	0.20	44.00	107.36	46.71	3788	1941	1918	23	136.83	524.75	489
	W15	2.65	3.835	0.152	8.84	0.58	0.20	44.00	116.38	47.15	3788	1941	1918	24	138.13	529.73	455
		TOTAL	19.175			2.91	1.000	220.00									
																	482

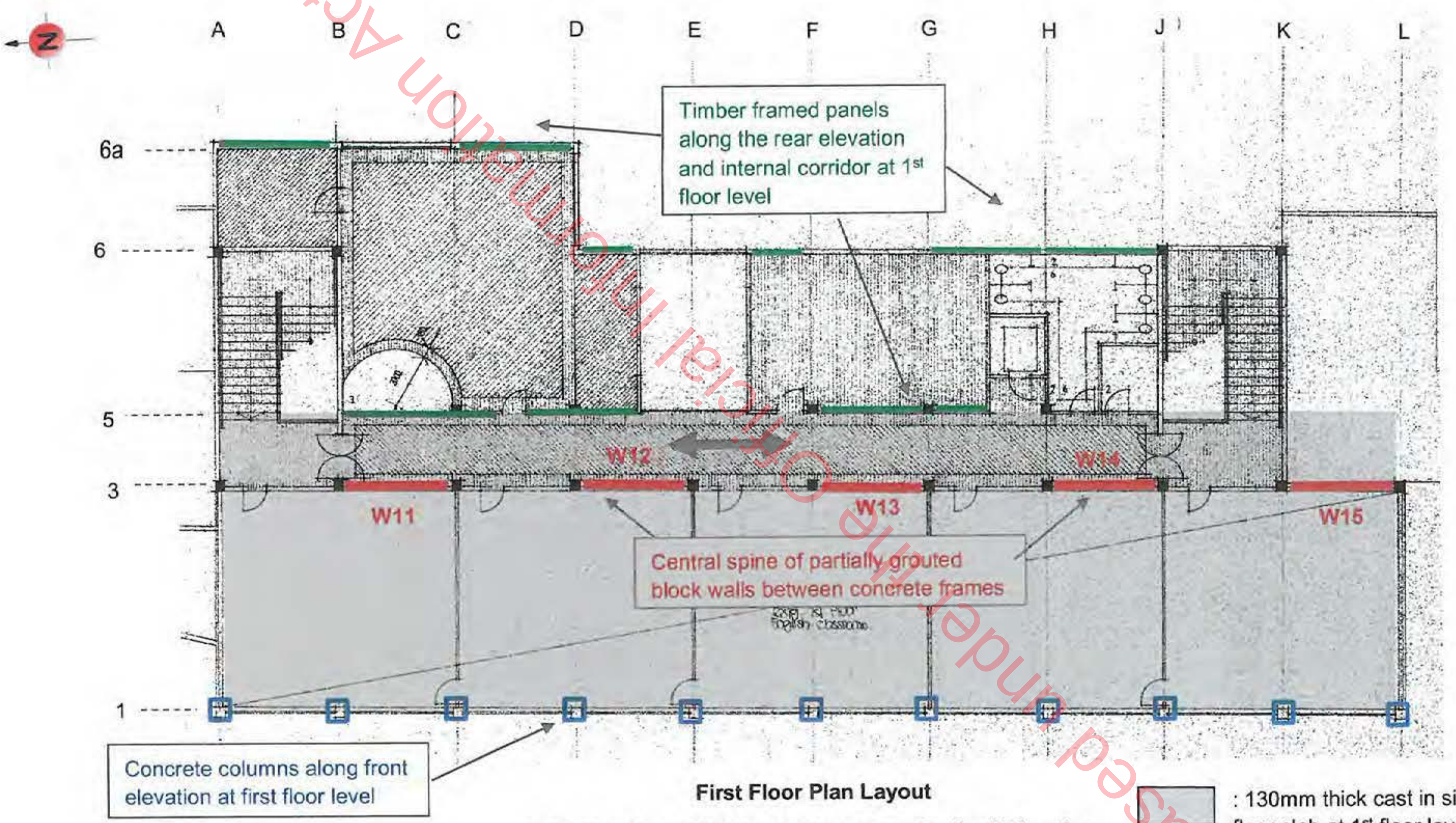


Ground Floor Plan Layout

Lateral Load Resisting System - Longitudinal Direction

▨ : 130mm thick cast in situ floor slab at 1st floor level

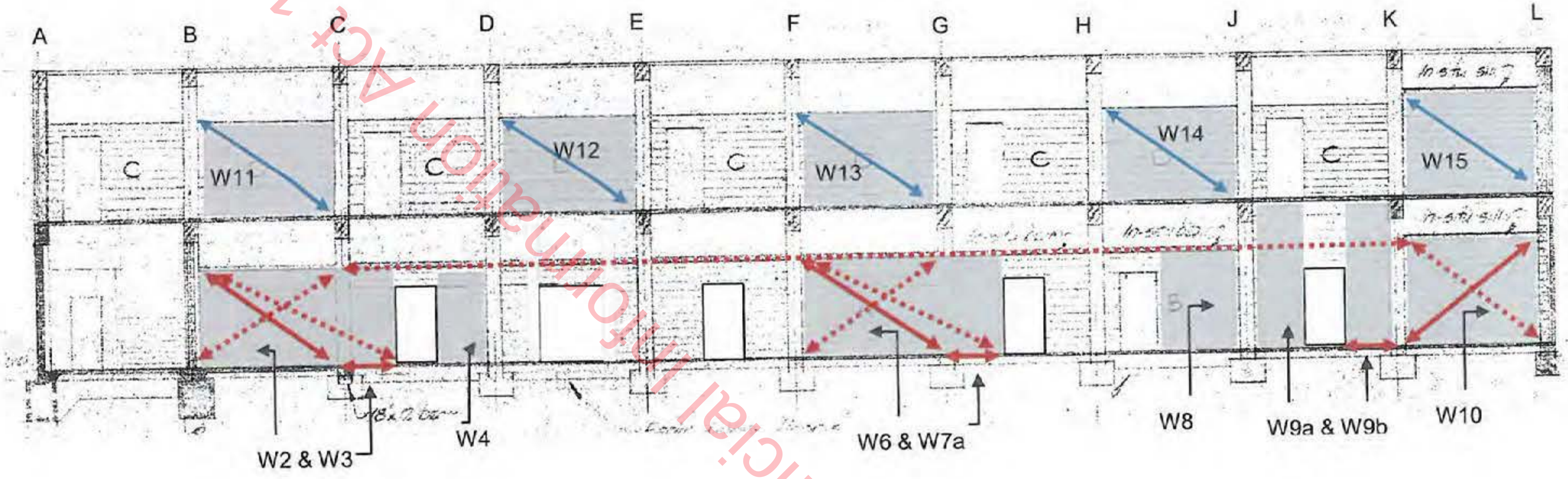
OZ -



First Floor Plan Layout

Lateral Load Resisting System - Longitudinal Direction

▨ : 130mm thick cast in situ floor slab at 1st floor level



Longitudinal Section illustrating the load path from the frame into the infill block wall panels

The infill panels used for lateral resistance are noted above

Along Grid '3' (1st floor) and Grid '4' (Ground floor)

CALCULATION SHEET

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3.2 1st FLOOR BLOCKWALL CONNECTIONS.

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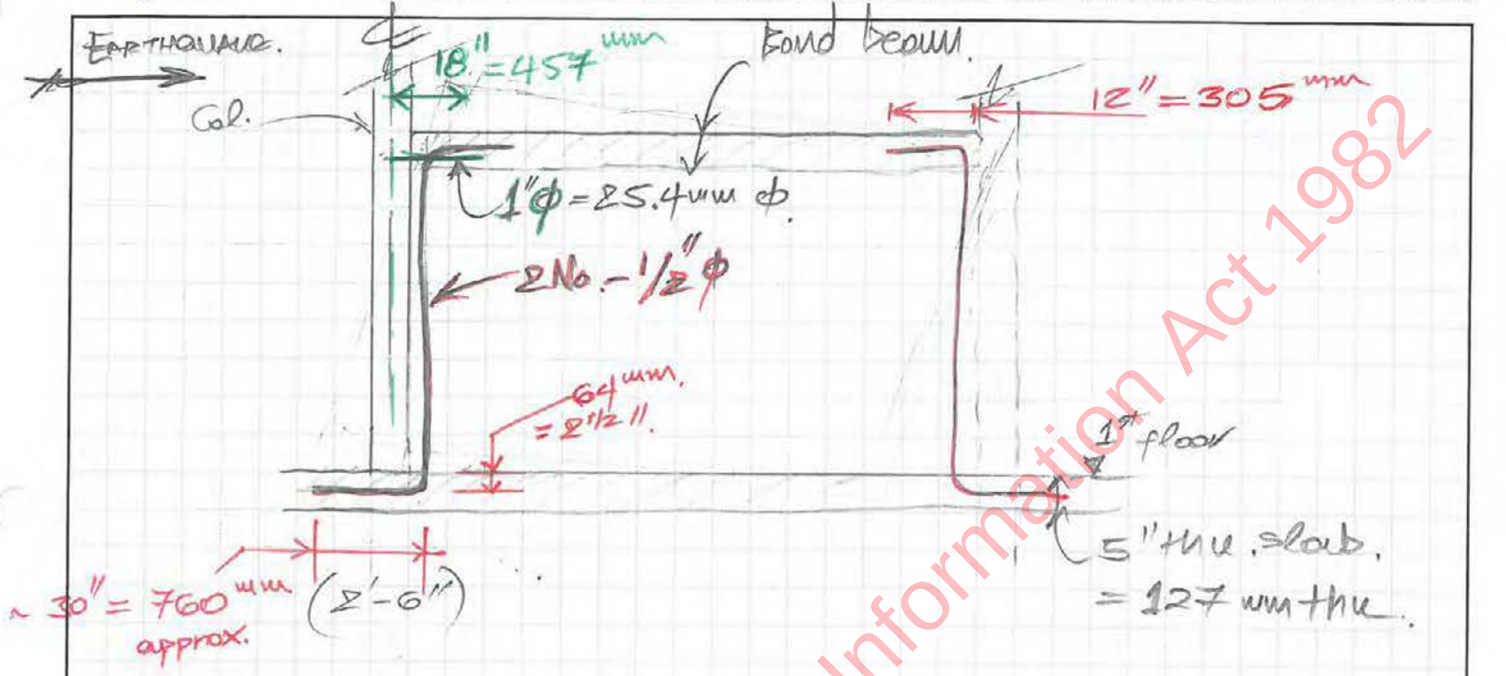
Check: / /



Tie Bars into slab

- 2 No. 1/2" rods. = 12.7 mm ϕ. → $A_v = 2 \times 126.7$ mm² = 253.3 mm².
- $f_y = 300$ MPa

Tension Capacity : $T_y = f_y A_v$

∴ $T_y = 300\ N/mm^2 \times 253.3\ mm^2 \times 10^{-3}$.

∴ $T_y = 76$ kN > 45 kN = V^*

✓ ok

Tie Bars into Column.

- 1" ϕ. = 25.4 mm ϕ. → $A_v = 506.7$ mm²

∴ $T_y = f_y A_v = 300 \times 506.7 \times 10^{-3}$

∴ $T_y = 152$ kN >> $V^* = 45$ kN

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Development Length of 90° hook bar. $(1/2'' \phi)$

$$L_{dh} = 0.24 a_b \cdot a_1 \cdot a_2 \frac{f_y d_b}{\sqrt{f_c}} \geq 8d_b = 101.6 \text{ mm}$$

Minimum

Where:

- $a_b = 1.0$.
- $a_1 = 1.0$.
- $a_2 = 1.0$.

$$\rightarrow \quad L_{dh} = 0.24 \times 1 \times \frac{\overset{N/mm^2}{300} \times \overset{mm}{12.7}}{\sqrt{30}}$$

$$\therefore L_{dh} = 167 \text{ mm}.$$

✓ ok

- Provided length = 760mm

CALCULATION SHEET

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Development length in tension

- For straight bar @ top into column.

$$L_{db} = \frac{(0.5 \times \alpha_a \cdot f_y)}{\sqrt{f_c}} \times d_b$$

$$\therefore L_{db} = \frac{0.5 \times 1.0 \times 300 \ ^{N/mm^2}}{\sqrt{30} \ \frac{N}{mm^2}} \times 25.4^{mm} = 695.6 \text{ mm}$$

$$\& \quad L_d = \frac{\alpha_b}{\alpha_c \alpha_d} L_{db} = 0.6 L_{db}.$$

Where:

- $\alpha_b = 1.0$
- $\alpha_c = 1 + 0.5 \left(\frac{\overset{70^{mm}}{c_m}}{\underset{25.4}{d_b}} - 1.5 \right)$

$$\therefore \alpha_c = 1 + 0.5 \times \left(\frac{70}{25.4} - 1.5 \right) = 1.63$$

Adopt $\alpha_c = 1.5$.

- $\alpha_d = 1.0$.

$$\Rightarrow \boxed{L_d = 417 \text{ mm}}$$

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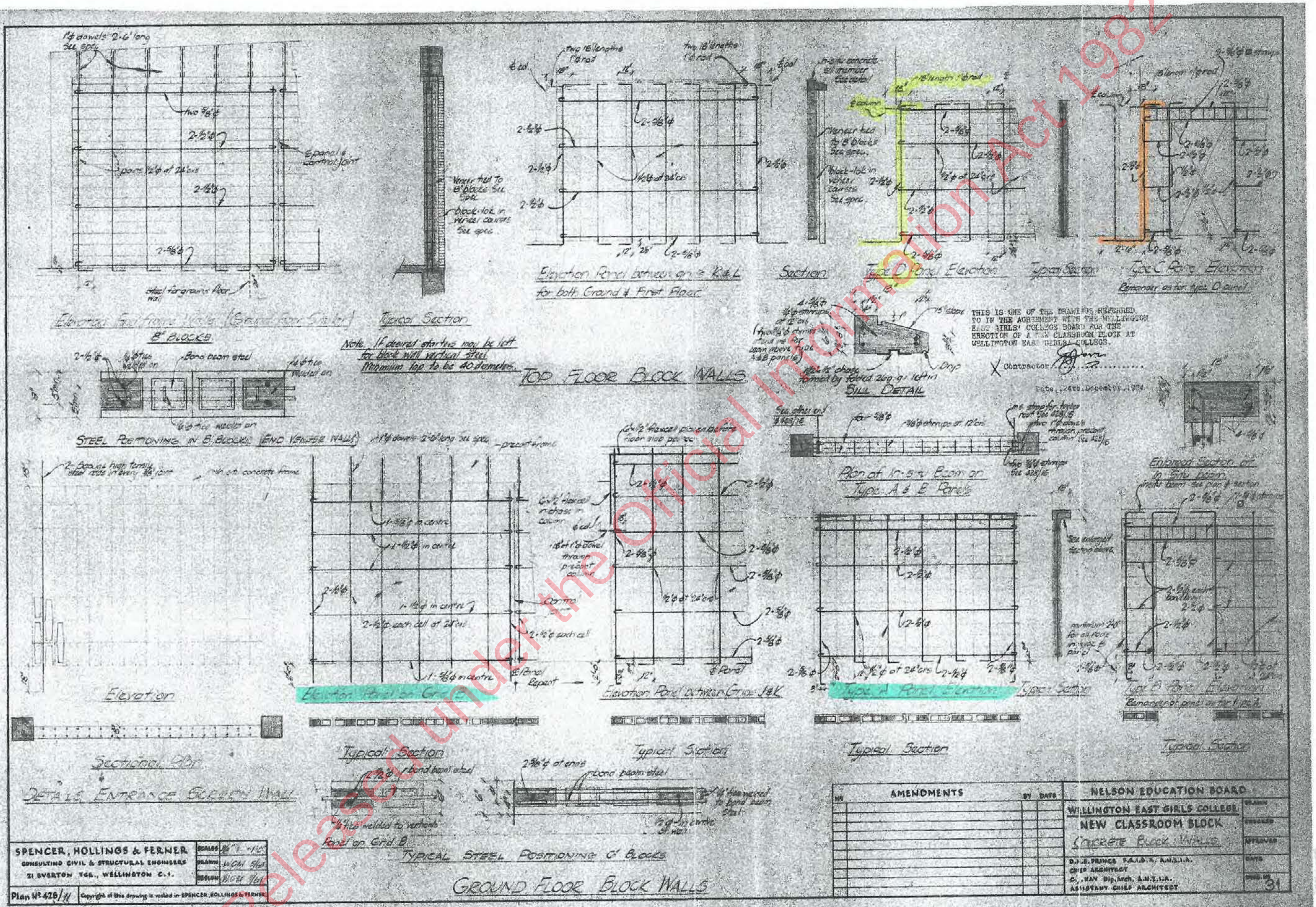
-25
1



THIS IS ONE OF THE DRAWINGS REFERRED TO IN THE AGREEMENT WITH THE WELLINGTON EAST GIRLS' COLLEGE BOARD FOR THE ERECTION OF A NEW CLASSROOM BLOCK AT WELLINGTON EAST GIRLS' COLLEGE.

X Contractor..........................

Date..24th December 1964

SPENCER, HOLLINGS & FERNER
CONSULTING CIVIL & STRUCTURAL ENGINEERS
21 EVERTON TCE., WELLINGTON C.1.

SCALES	3/4" 1'0"
DRAWN	W.C.M. 5/64
DESIGN	B.C.M. 7/64

Plan Nº 426/11

Nº	AMENDMENTS	BY	DATE

NELSON EDUCATION BOARD
WELLINGTON EAST GIRLS COLLEGE
NEW CLASSROOM BLOCK
Concrete Block Walls

D.J.S. PRINCE F.R.I.B.A. A.N.Z.I.A.
CHIEF ARCHITECT
G. KAY Dip.Arch. A.N.Z.I.A.
ASSISTANT CHIEF ARCHITECT

DRAWN
CHECKED
APPROVED
DATE
DRG. No. 31

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3.3 Reinforced Concrete Walls

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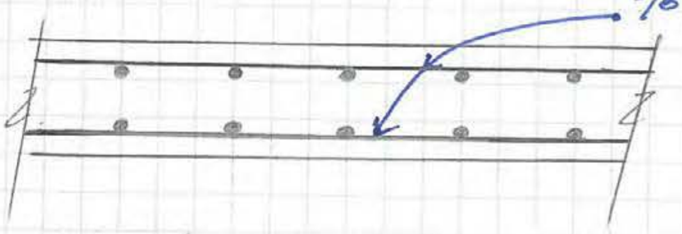
8" thk. WALL = (200mm)



5/8" ϕ @ 9" crs $(= 16^{mm} \phi$ @ 230 mm spacing$)$.

- both faces & both ways.

Probable Shear Strength

$$\boxed{V_p = 0.75 (V_c + V_s)}$$

Where : $\phi\ V_c = v_c \times 0.8 A_g$.

$\therefore V_c = 1.28 \times 0.8 \times 0.2 \times 10^3$ (m^2/m)

$\therefore \underline{\underline{V_c = 205 \text{ kN.}}}$ / m run

With : - $A_g = 1 \times 0.2 = 0.2 m^2/m$

- $v_c = (5 - \mu) \times \sqrt{f_c + N^*/A_g} / 16$.

for $N^* = 0$.

$f_c = 30$ MPa. (N/mm^2)

$\mu = 1.25$

$\Rightarrow$ $\Rightarrow \underline{v_c = 1.28}$

& $\phi\ V_s = 2 \times A_v f_y \frac{d}{s}$

2 layers

With

- $A_v = \pi R^2 = 201 mm^2$

$\therefore V_s = 2 \times 201 \times 300 \times \frac{800}{230} \times 10^{-3}$ (mm^2, N/mm^2)

- $f_y = 300$ MPa (N/mm^2)

- $d = 0.8 \times$ Length $= 0.8 \times 1000 = 800$ mm

$\therefore \underline{\underline{V_s = 419.5 \text{ kN}}}$ / m run.

- $s = 230$ mm.

HENCE :

$V_p = 0.75 \times (205 + 419.5)^{kN.} = \underline{\underline{468.4 \text{ kN}}}$ per m run

(Refer to Table 2)

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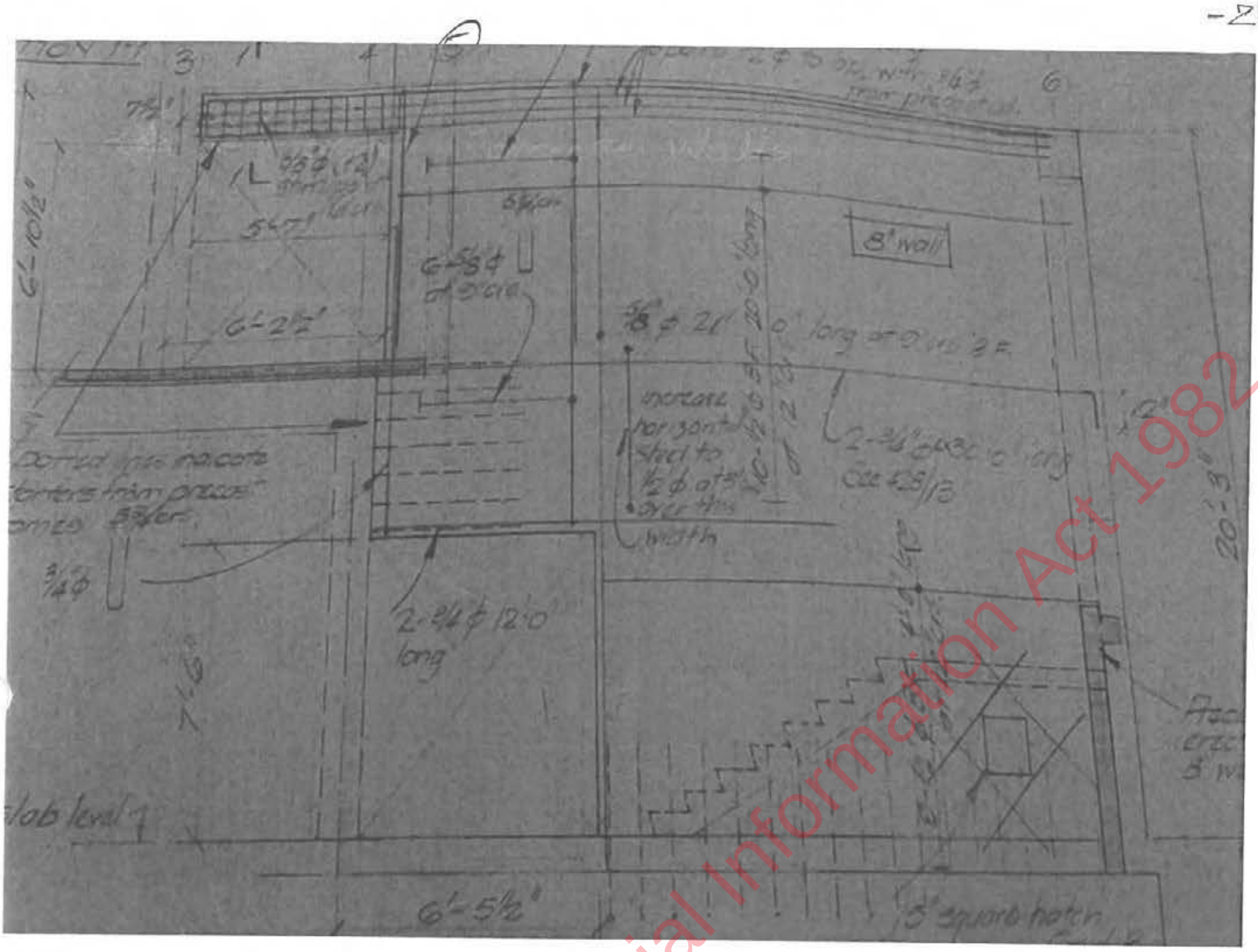
TABLE 2

RC walls -Shear Capacity			μ=1.25	Wall dimensions								
Story	Pier	Dir.	V*max	L(m)	t(m)	Vc (kN)	Layers	spacing (m)	Vs	V_{prob} (kN)	maxV* (kN)	Check
Ground	W12	YY	363	3.91	0.18	619	2	0.23	1640	1627	363	ok
	W13	YY	363	3.91	0.18	619	2	0.23	1640	1627	363	ok
	W14	YY	403	3.91	0.2	688	2	0.23	1640	1676	403	ok
	W15	YY	403	3.91	0.2	688	2	0.23	1640	1676	403	ok
	W16	YY	363	3.91	0.18	619	2	0.23	1640	1627	363	ok


TION 1-1
3
4
5
6
7½"
⅝ ϕ (12)
5'-7"
6'-2½"
6'-10½"
6-⅝ ϕ
@ 9" c/c
6¾ c/c
⅝ ϕ 21'-0" long at 9" c/c
8" wall
increase
horizontal
steel to
½ ϕ at 9"
over this
width
2-¾" ϕ 22'-0" long
See 428/13
20'-3"
2-¾ ϕ 12'-0"
long
¾ ϕ
5¾ c/c
7'-6"
slab level
6'-5½"
3" square hatch

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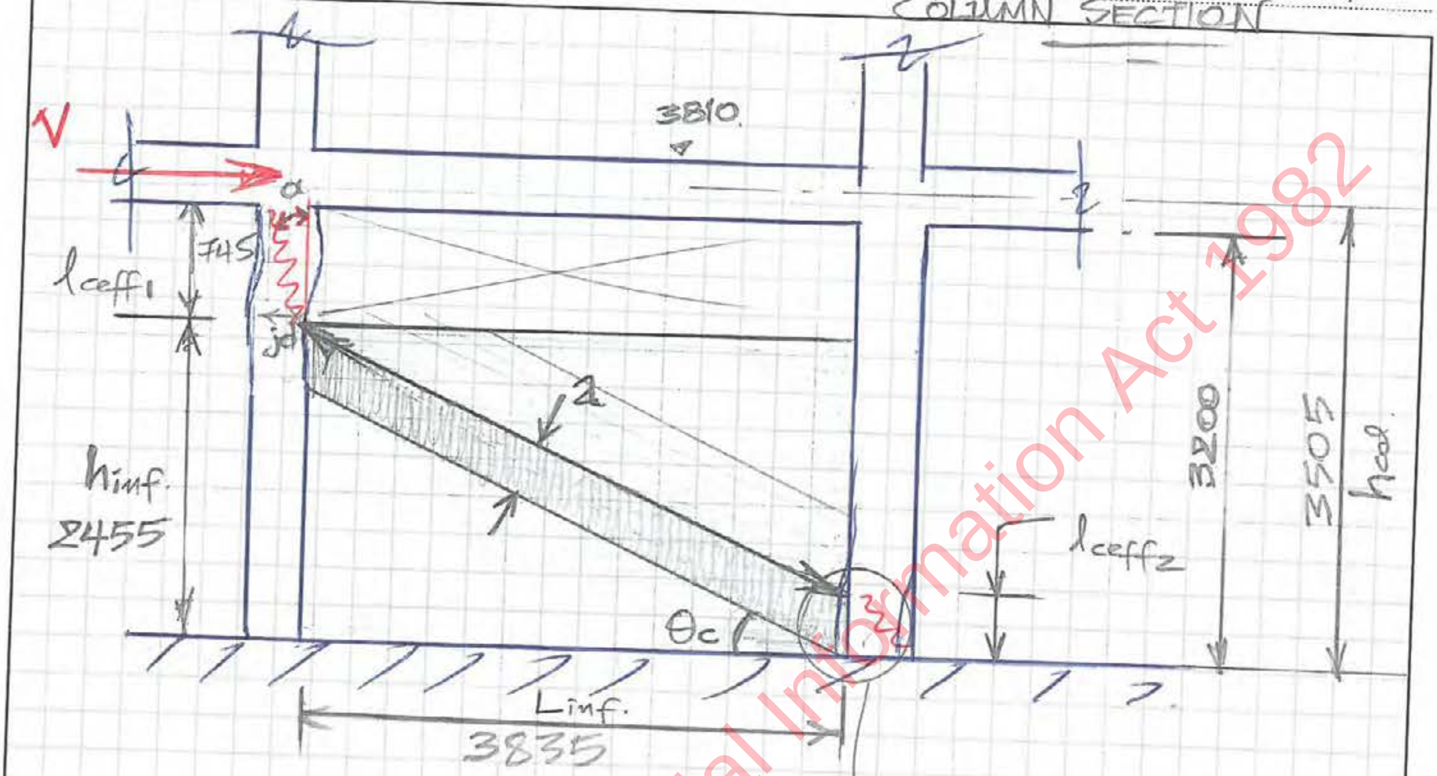
EFFECT OF PARTIAL INFILLS ON FRAME PERFORMANCE. FOR BOTTOM COLUMN SECTION



SHEAR DEMAND ON FRAME @ BOTTOM.

$$V^*_{col.} = \frac{\Sigma M_p^{col}}{l_{ceff2.}}$$

Where: • M_p^{col} = Plastic Moment Capacity

Nom. Flexural Strength.

- $M_o = 1.25\, M_n$
- $M_o = 1.25 \times 185$
- $\therefore M_o = 231.25\,kNm$

or

"USED" → • $M_n = 185\ kNm$

• $l_{ceff.2} = \frac{a}{\cos\theta_c}$

- a : effective width of longitudinal compression strut'
- θ_c = diagonal strut angle

• $\tan\theta_c = \frac{h_{inf.} - \left(\frac{a}{\cos\theta_c}\right)}{L_{inf.}}$

1. Solving the above equation, θ_c can be found.
2. Then calculate a : width of compression strut.

$a = 0.175 \times (\lambda_1 \cdot h_{col})^{-0.4}\, r_{inf.}$

3. Derive $l_{ceff.}$ from above formula
4. Calculate shear demand

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Where:

- $$\lambda_1 = \left[\frac{E_{me} \cdot t_{inf} \times \sin 2\theta}{4 E_{fe} I_{col} . h_{inf}}\right]^{1/4}$$

With:

- $E_{me} = 4700\sqrt{f_{ce}} = 4700\sqrt{12} = \underline{16281 \text{ MPa}}$

  ↳ Youngs Modulus of reinf. Concrete Infill

- $E_{fe} = E_c = 3320\sqrt{f_c} + 6900 = \underline{25084} \text{ MPa}$

  $(f_c = 30 \text{ MPa})$

- $\theta = \tan^{-1}\left(\frac{h_{inf}}{L_{inf}}\right) = \tan^{-1}\left(\frac{2455}{3835}\right) = \underline{0.56}$ (rad.)

  $\therefore \sin 2\theta = 0.9$

- $t_{inf} = 0.15 \text{ m}$

- $I_{col} = \frac{bd^3}{12} = \frac{0.38 \times 0.38^3}{12} = 0.001738 \text{ m}^4$

$$\Rightarrow \lambda_1 = \left[\frac{16281 \times 0.15 \times 0.9}{4 \times 25084 \times 0.001738 \times 2{,}455}\right]^{1/4} = 1.5$$

THEREFORE:

$$\lambda = 0.175 \cdot (\lambda_1 \cdot h_{col})^{-0.4} \cdot r_{inf}$$

$$\therefore \lambda = 0.175 \times (1.5 \times 3.5)^{-0.4} \times 4.55.$$

$$\therefore \underline{\lambda = 0.410 \text{ m.}}$$

- $h_{col} = 3.5 \text{ m}$
- r_{inf} = Diagonal length of infill panel!

$\therefore r_{inf} = \sqrt{3.835^2 + 2455^2}$

$\therefore r_{inf} = 4.55 \text{ m}$

HENCE – Using spreadsheet iteration $\theta_c = 0.48$. $= l_{eff} = \frac{0.410}{\cos 0.48}$

$\therefore \underline{l_{eff} = 0.46 \text{ m}}$

$$\rightarrow \text{Shear DEMAND } V^* = \frac{2 \times 185}{0.46} = \underline{804 \text{ kN}}$$

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MODIFIED SHEAR CAPACITY OF RC FRAME

- Crack angle : $\alpha_c = \tan^{-1} \frac{jd}{l_{ceff2}}$, $\boxed{20^\circ < \alpha_c \leq 45^\circ}$

Where : • $jd = 0.8 \times 380 = 304$ mm.

• $l_{ceff_1} = 460$mm

$\therefore \quad \alpha_c = \tan^{-1} \frac{304}{460} = \underline{\underline{33.5^\circ}}$

- Shear Resistance given by :

$$V_r = V_s + V_n + V_c$$

→ V_s : Shear carried by the transverse reinf.

$$V_s = A_{sh} f_{yhe} \frac{jd}{s} \cot\alpha_c$$

$\therefore \quad V_s = 127 \times 300^{N/mm^2} \times \frac{304}{230} \times 1.73$

$\therefore \quad \underline{\underline{V_s = 87 \text{ kN}}}$ ✓

• $A_{sh} = 127$ mm^2 (1 hook set)

• $f_{yhe} = 300$ MPa.

• $jd = 304$ mm

• $s = 9''$ crs $= 230$mm

• $\cot\alpha_c = \cot 30^\circ = \frac{1}{\tan 30^\circ} = 1.73$

→ V_n : Shear carried by axial load (strut action)

$$V_n = N \times \tan\alpha_c$$

$\therefore \quad V_n = 100^{kN} \times \tan 30^\circ$

$\therefore \quad \underline{\underline{V_n = 58 \text{ kN}}}$ ✓

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→ V_c : Shear carried by the concrete

$$V_c = k\sqrt{f'_{ce}} \cdot b_w \times d$$

Where :

- $f'_{ce} = 30$ MPa (N/mm²)
- $b_w = 380$ mm.
- $d = 330$ mm
- $K = 0.33 - 0.06\Lambda^2 \dfrac{E_s}{f_{ye}} \tan\alpha_c \, \theta_p$

With :
- $\Lambda = 2.0$ – for fixed-fixed (2 hinges)
- $E_s = 200 \times 10^3$ MPa
- $f_{ye} = 300$ MPa
- $\tan\alpha_c = \dfrac{jd}{l_{eff}} = \dfrac{304}{1050} = 0.29$

- θ_p : plastic hinge rotation in the column

$$\theta_p = (\phi_u - \phi_y) L_p$$

$$\therefore \theta_p = (0.028 - 0.009) \times 0.19$$

$$\therefore \theta_p = 3.6 \times 10^{-3}.$$

- $\phi_u = \dfrac{\varepsilon_{cu}}{c} = \dfrac{0.003}{0.94} = 0.003$

c ← NA depth. (from Gen-Col)

- $\phi_y = \dfrac{2.12 \times \varepsilon_y}{h} = \dfrac{2.12 \times (1.08 \times 300)/200 \times 10^3}{0.38}$

h ← Col depth.

$$\therefore \phi_y = 9.04 \times 10^{-3}$$

- $L_p = 0.5 \times h = 0.5 \times 380 = 190$ mm

From the above:

$$K = 0.33 - 0.06 \times 1.0^2 \times \frac{200 \times 10^3}{300} \times 0.29 \times 3.6 \times 10^{-3}$$

$$\therefore K = 0.288 \simeq \underline{\underline{0.29}}. \checkmark$$

ADOPT $k = 0.29$ ✓

$\mu \leq 2$.

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CALCULATION SHEET

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$\Rightarrow V_c = k\sqrt{f_c'}\, b_w d$: Shear by the Concrete.

$\therefore V_c = 0.29 \times \sqrt{30} \times 380 \times 330$

$\therefore \underline{\underline{V_c = 199 \text{ kN.}}}$ ✓

HENCE :

$V_r = V_s + V_m + V_c$

$\therefore V_{r_c} = 87 + 58 + 199 = \underline{\underline{344 \text{ kN}}}$ ✓

$\%NBS = \frac{\text{Capacity}}{\text{Demand}} = \frac{344}{804} = \underline{\underline{43\%}}$

NOTE :

Additional shear can be utilised from adjacent panel. Refer to the calculations on next page.

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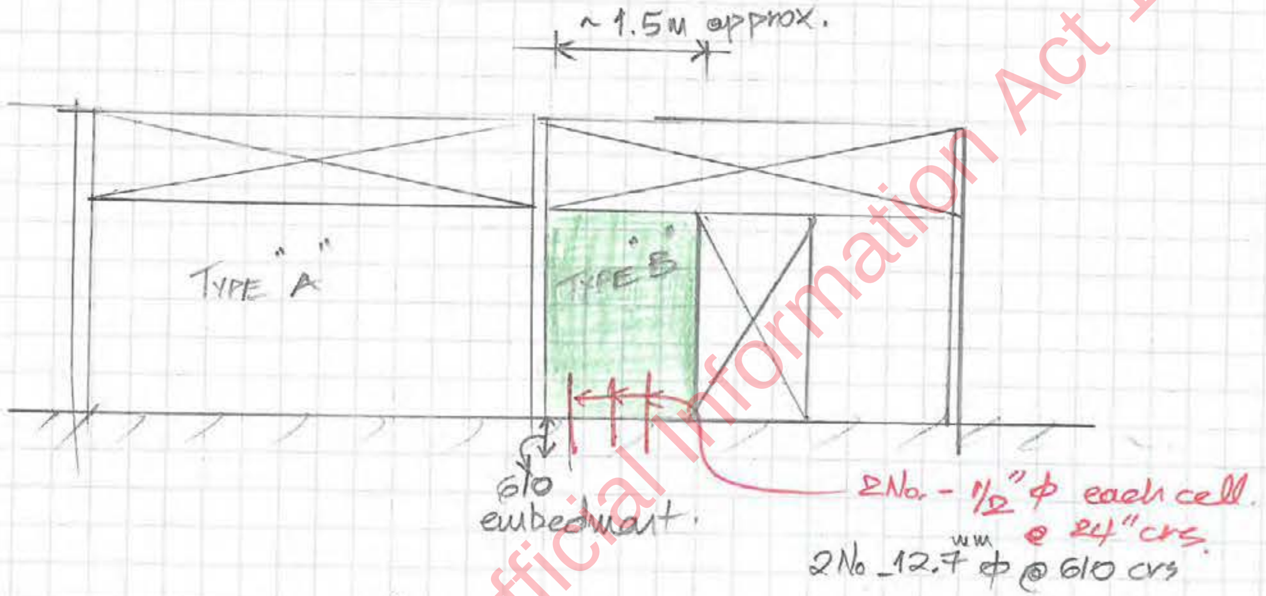
GROUND STOREY

CHECK shear friction on a typical panel "B" based on rebar connecting wall & ground beam – Refer to Dwg. No. 31. –



- Shear Friction : V_n (As per NZS 3101, § 7.7)

$$V_n = (A_v f_y + N^{\circ})\mu$$

Where : $\ast$ $A_v = 2$ No. Bars $\times 2 \times 127 = 508.\text{mm}^2$ (per cell, mm²)

$\ast$ $f_y = 300\text{MPa}$

$\bullet$ $\mu = 1.0 \times \lambda = 1.0.$

$$\Rightarrow V_n = 508 \times 300 \times 10^{-3} = \underline{\underline{152.4\ \text{kN}}}.$$

HENCE : Total Shear Resistance : (V_r)

$$V_r = V_{col.} + V_{wall}$$

$$\therefore V_r = 344 + 152 = \underline{\underline{496\ \text{kN}}}.$$

$$\%\text{NBS} = \frac{496}{804} = \underline{\underline{62\%}}$$

CALCULATION SHEET

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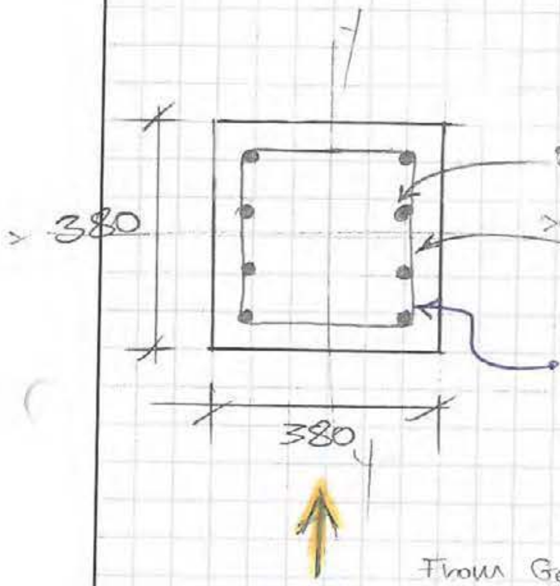
RC COLUMN CAPACITY - GROUND STOREY

Computed: / /

Check: / /

AT GRID 4 - (BOTTOM)

- $E = 200 \times 10^3$ MPa
- $f_y = 300$ MPa



8No.-1"φ = 8No.- 25 φ mm

1/2"φ stirrups @ 9"crs (12.7 mm φ @ 230 crs)

($A_{sv} = 127$ mm²)

Cover = 26 mm.

From Gen.CoP

$M_{cx} = 190$ kNm (N = 150 kN)

$M_{cx} = 185$ kNm ← → (N = 100 kN)

$M_{cx} = 174$ kNm (N = 0 kN)

- Eff. depth : $d = 380 - 50 = 330$ mm
- Spacing of stirrups : $s = 230$mm. $> d/_2 = 165$mm.

Probable Shear Capacity = As per NZSEE 2006.

$V_p = 0.75 \times (V_c + V_s)$

$\therefore V_p = 0.75 \times (126.5 + 94.7)$

$\therefore V_p = 165.9 \simeq \underline{\underline{166 \text{ kN}}}$

- $V_c = v_c \times 0.8 A_g$.

$\therefore V_c = k\sqrt{f_c} \times 0.8 \times A_g$

$\therefore V_c = 0.2 \times \sqrt{30} \times 0.8 \times 380 \times 380$

$\therefore \underline{\underline{V_c = 126.5 \text{ kN}}}$

- $V_s = \frac{A_v f_y d}{s} \times \cot 30°$

$\therefore V_s = \frac{127 \times 300 \times 330}{230} \times \cot 30° \times 10^{-3}$

$\therefore \underline{V_s = 94.7 \text{ kN}}$

- 36

Job number (or name):
Column number:

User name : wescc0

Concrete properties:

Rectangular stress block as defined by NZS 3101:1995.
Concrete cylindrical compressive strength = 30.0 MPa
Concrete compression stress coefficient, a1 = 0.85
Compression zone depth coefficient, B1 = 0.85
Concrete maximum strain = 0.0030

Steel properties:

Steel modulus of elasticity = 200 000 MPa
Steel yield strength = 300.0 MPa

Dimensions of the column section:

Rectangular section.
Height of the column section = 380.0 mm
Width of the column section = 380.0 mm
Clear cover to ties parallel to the y-axis = 26.0 mm
Clear cover to ties parallel to the x-axis = 26.0 mm

Results:

Load combination number 1 :
Axial load = 152.8 kN, Mx = 190.2 kNm, My = 0.0 kNm
Strength reduction factor, Phi = 1.00
Required reinforcement ratio = 0.02718, Required reinforcement area = 3924.8 mm2
Initial reinforcement ratio = 0.02718, Initial reinforcement area = 3924.8 mm2
Initial reinforcement ratio scaled by = 1.0000
Moment ratio = 0.00000, Target moment ratio = N/A
Skew angle = 0.0 degrees, NA depth = 93.5 mm
Force (unfactored) carried by concrete = 769.8 kN
Force (unfactored) carried by reinforcement = -617.0 kN
Axial load eccentricity: ex = 0.0 mm, ey = 1244.8 mm

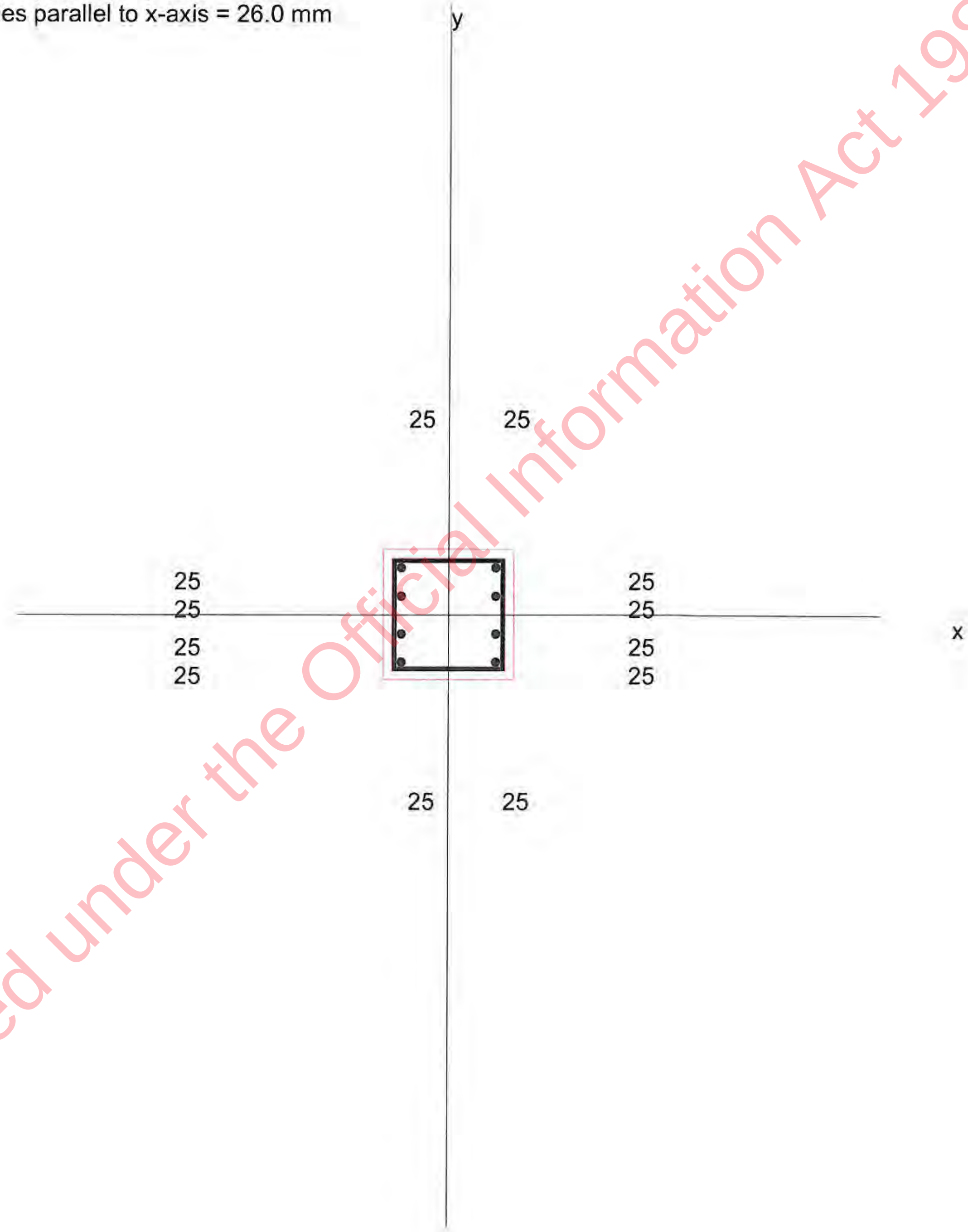
The analysis has been finished.

-37

Job number (or name):
Column number:

Column area = 144400 mm2
Reinforcement area = 3927 mm2
Reinforcement ratio = 0.02720

Drawing scale : 1 / 20
Column section height = 380.0 mm
Column section width = 380.0 mm
Clear cover to ties parallel to y-axis = 26.0 mm
Clear cover to ties parallel to x-axis = 26.0 mm



CALCULATION SHEET

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Project Description:

GROUND STOREY
(TOP SECTION)

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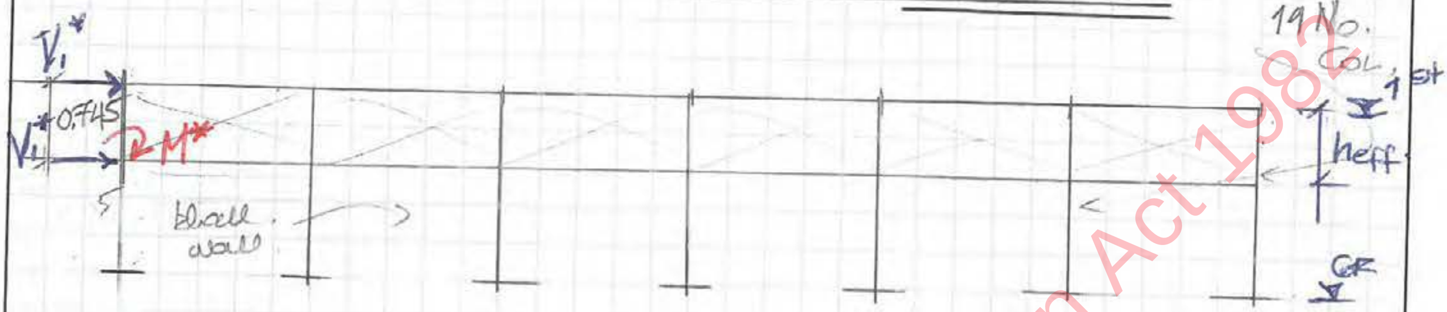
Check: / /

ELEVATION ALONG GRID LINE "4" or "D" @ GROUND FL.



* Max. Shear Demand $F_v^* = \frac{2 M_n}{h_{eff}}$

Where: $M_n = 240$ kNm (Nominal flex. strength From Gen-Col.)

$h_{eff} = 0.745$ m

$\therefore F_v^* = \frac{2 \times 240}{0.745} = \underline{\underline{644}}$ kN per column

HENCE: $\frac{\text{Capacity}}{\text{Demand}} = \frac{V_p}{F_v^*} = \frac{530.4}{644} = 82$ % NBS [Top Section COLUMN].

Prob. Shear Cap. (See over)

ALSO. Based on the applied force per column/bay

$V_1^* = 320$ kN

& $M^* = V^* \times h_{eff} = 320 \times 0.745 = \underline{\underline{238.4}}$ kNm

$\therefore \frac{M_n}{M^*} = \frac{240}{238.4} = \underline{\underline{100\%}}$ — ✓ flexure OK

OPUS

CALCULATION SHEET

Project/Task/File No:
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RC COLUMN CAPACITY
GROUND STOREY
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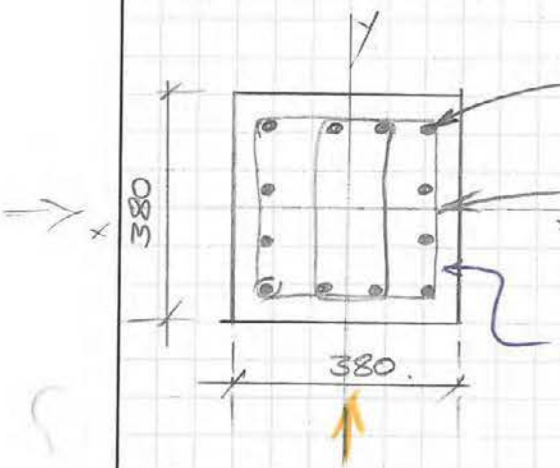
AT GRID 4 - (TOP) - LONG DIRECTION



1" φ rods = 25 mm φ. (As = 491 mm² per bar)

½" φ @ 3" crs (12.7 mm φ @ 75 mm crs)
(Av = 2 legs/sets × 127 = 254 mm²)

Cover = 26 mm.

(N = 150 kN. Axial).

- Eff. depth : $d = 380 - 50 = 330$ mm.
- Spacing of stirrups : $s = 75\text{ mm} < 330/2 = 165$ mm.

Prob. Shear Capacity: - As per NZSEE 2006.

$V_p = 0.75 (V_c + V_s)$

Where : • $V_c = v_c \times 0.8A_g$.

$= k\sqrt{f_c} \times 0.8 \times A_g = 0.2 \times \sqrt{30} \times 0.8 \times 380 \times 380 = \underline{126.5 \text{ kN}}$

• $V_s = \dfrac{A_v f_y d}{s} \times \cot 30°$

$= \dfrac{254 \times 300 \times 330}{75} \times \cot 30° \times 10^{-3} = \underline{580.7 \text{ kN}}$

$\Rightarrow V_p = 0.75 \times (126.5 + 580.7)$

$\rightarrow \underline{\underline{V_p = 530.4 \text{ kN}}}$

OPUS

Gen-Col
Analysis of Reinforced Concrete Columns

Job number (or name): WEGC
Column number: Grid 4

User name : wescc0

Concrete properties:
Rectangular stress block as defined by NZS 3101:1995.
Concrete cylindrical compressive strength = 30.0 MPa
Concrete compression stress coefficient, a1 = 0.85
Compression zone depth coefficient, B1 = 0.85
Concrete maximum strain = 0.0030

Steel properties:
Steel modulus of elasticity = 200 000 MPa
Steel yield strength = 300.0 MPa

Dimensions of the column section:
Rectangular section.
Height of the column section = 380.0 mm
Width of the column section = 380.0 mm
Clear cover to ties parallel to the y-axis = 50.0 mm
Clear cover to ties parallel to the x-axis = 50.0 mm

Results:
Load combination number 1 :
Axial load = 0.1 kN, Mx = 228.7 kNm, My = 0.0 kNm
Strength reduction factor, Phi = 1.00
Required reinforcement ratio = 0.04077, Required reinforcement area = 5887.2 mm2
Initial reinforcement ratio = 0.04077, Initial reinforcement area = 5887.2 mm2
Initial reinforcement ratio scaled by = 1.0000
Moment ratio = 0.00000, Target moment ratio = N/A
Skew angle = 0.0 degrees, NA depth = 104.1 mm
Force (unfactored) carried by concrete = 857.2 kN
Force (unfactored) carried by reinforcement = -857.1 kN
Axial load eccentricity: ex = 0.0 mm, ey = 2287000.0 mm

Nominal Flexural Strength

Load combination number 2 :
Axial load = 147.1 kN, Mx = 241.1 kNm, My = 0.0 kNm
Strength reduction factor, Phi = 1.00
Required reinforcement ratio = 0.04077, Required reinforcement area = 5887.2 mm2
Initial reinforcement ratio = 0.04077, Initial reinforcement area = 5887.2 mm2
Initial reinforcement ratio scaled by = 1.0000
Moment ratio = 0.00000, Target moment ratio = N/A
Skew angle = 0.0 degrees, NA depth = 110.3 mm
Force (unfactored) carried by concrete = 908.6 kN
Force (unfactored) carried by reinforcement = -761.5 kN
Axial load eccentricity: ex = 0.0 mm, ey = 1639.0 mm

The analysis for all load combinations have been finished.

Project/Task/File No:

Project Description: 3.4 CONTINUE.

EFFECT OF PARTIAL INFILL ON FRAME

@ 1^{st} FLOOR ~ BOTTOM SECTION

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Office:

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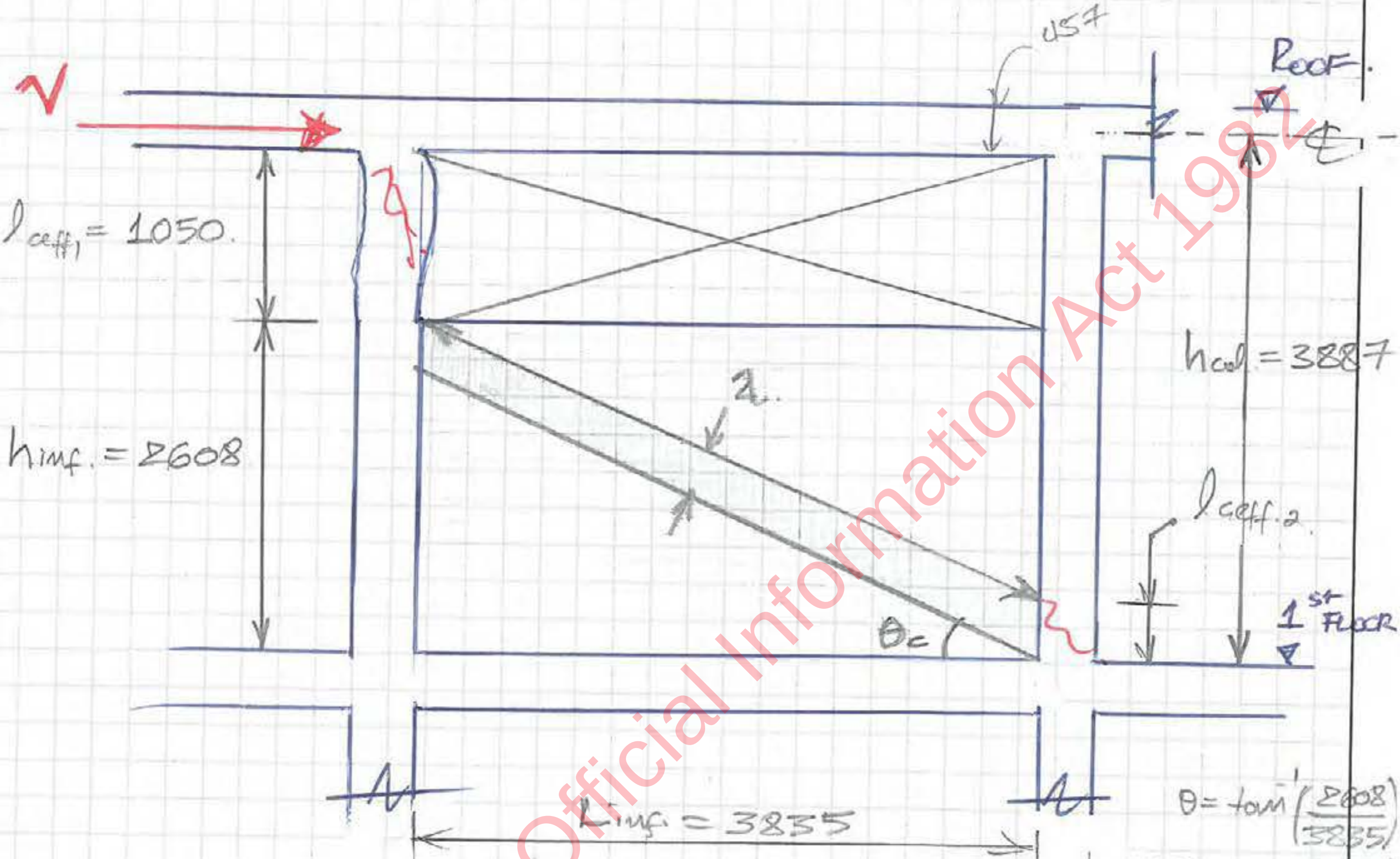
Check: / /



$\theta = \tan^{-1}\left(\frac{2608}{3835}\right)$

$\therefore \theta = 0.597$ rad

$2 \sin 2\theta = 0.93$

SHEAR DEMAND OF FRAME @ BOTTOM PART.

$V^*_{col.} = \frac{\Sigma M_p^{col.}}{l_{ceff.2}}$ ‖ • $M_p = M_n = 99$ kNm (from Gen Col)

• $l_{ceff.2} = ?$

• Calculate a : width of compression strut

$a = 0.175 \times (\lambda_1 \times h_{col})^{-0.4} \times r_{inf}.$ — Equation (1)

Where : • $\lambda_1 = \left[\frac{E_{me} \times t_{inf} \times \sin 2\theta}{4 E_{fe} \times I_{col.} \times h_{inf}}\right]^{1/4}.$

$\therefore \lambda_1 = \left[\frac{16281 \times 0.15 \times 0.93}{4 \times 25084 \times 0.001738 \times 2.608}\right]^{1/4} = 1.49$

• $r_{inf} = \sqrt{3.835^2 + 2.608^2} = 4.64$ m

From Equation (1) => a = 0.4 m

CALCULATION SHEET

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DEMAND &/ CAPACITY @ TOP FRAME - BOTTOM PART

Computed: / /

Check: / /

Using a spreadsheet iteration → $\theta_c = 0.51$

$$\therefore l_{ceff,2} = \frac{a_1}{\cos\theta_c} = \frac{0.4}{\cos 0.51} = \underline{0.458}$$

HENCE.

$$V^*_{col} = \frac{2 \times 99}{0.458} = \underline{\underline{432 \text{ kN}}}$$

SHEAR DEMAND

MODIFIED SHEAR CAPACITY OF FRAME $(V_r = V_s + V_N + V_c)$

- $\alpha_c = \tan^{-1}\frac{jd}{l_{ceff2}} = \tan^{-1}\frac{304}{458} = \underline{33.6°}$: Crack Angle

- $V_s = A_{sh} \cdot f_y \frac{jd}{s} \cot\alpha_c \leftarrow \left(\frac{1}{\tan 33.6}\right)$

$\therefore V_s = 70.9 \times 300 \times \frac{304}{230} \times 1.5$

$\therefore \underline{V_s = 42 \text{ kN}}$ — shear by reinf.

- $V_N = N \times \tan\alpha_c$

$\therefore V_N = 35 \times \tan 33.6° = \underline{23 \text{ kN}}$ ~ shear by strut action

- $V_c = k\sqrt{f_c}\, b_w d$

$\therefore V_c = 0.29 \times \sqrt{30} \times 380 \times 330 = \underline{199 \text{ kN}}$ ~ shear by Concrete

HENCE: $V_{cap} = V_s + V_N + V_c$

$\therefore V_{cap} = 42 + 23 + 199 = \underline{264 \text{ kN}}$

$\% NBS = \frac{264}{432} = \underline{\underline{61\ \%}}$

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CALCULATION SHEET

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Sheet No 43 **of**

Project Description:
RC COLUMN CAPACITIES.

Office:

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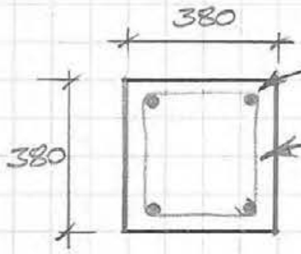
TYPICAL COLUMN AT TOP FRAME .(GL B→K)



- 4No. - 1"ϕ. (25^{mm} ϕ)
- 3/8"ϕ @ 9" crs stirrups (typically) or 4½" crs (top)
- 9.5 mm ϕ @ 230mm crs.
- (A_v = 70.9 mm²)

(· Cover = 1" = 25 mm.)
· Eff depth : d = 380 − 50 = 330 mm.

Prob. Shear Capacity

$$V_p = 0.75 \times (V_c + V_s)$$

Where :

- $V_c = \nu_c \times 0.8 A_g = k\sqrt{f'c} \times 0.8 \times A_g.$

$$\therefore V_c = 0.2 \times \sqrt{30} \times 0.8 \times 380 \times 380 = \underline{126.5 \text{ kN}}$$

- $V_s = \dfrac{A_v f_y d}{s} \times \cot 30^\circ = \dfrac{70.9 \times 300 \times 330}{230} \times \cot 30^\circ \times 10^{-3} = \underline{\underline{52.9}} \text{ kN}$

$$\Rightarrow V_p = 0.75 \times (126.5 + 52.9) = \boxed{\underline{179.4 \text{ kN.}}}$$

* REVISED *

Nominal Flexural Strength

M_n = 99.3 kNm. − from Gen-Col. (shown overleaf)

Gen-Col
Analysis of Reinforced Concrete Columns

Job number (or name): WEGC
Column number: Top frame column 380x380

User name : wemxx0

Concrete properties:

Rectangular stress block as defined by NZS 3101:1995.
Concrete cylindrical compressive strength = 30.0 MPa
Concrete compression stress coefficient, a1 = 0.85
Compression zone depth coefficient, B1 = 0.85
Concrete maximum strain = 0.0030

Steel properties:

Steel modulus of elasticity = 200 000 MPa
Steel yield strength = 300.0 MPa

Dimensions of the column section:

Rectangular section.
Height of the column section = 380.0 mm
Width of the column section = 380.0 mm
Clear cover to ties parallel to the y-axis = 25.0 mm
Clear cover to ties parallel to the x-axis = 25.0 mm

Results:

Load combination number 1 :
Axial load = 35.3 kN, Mx = 99.3 kNm, My = 0.0 kNm
Strength reduction factor, Phi = 1.00
Required reinforcement ratio = 0.01359, Required reinforcement area = 1962.4 mm2
Initial reinforcement ratio = 0.01359, Initial reinforcement area = 1962.4 mm2
Initial reinforcement ratio scaled by = 1.0000
Moment ratio = 0.00000, Target moment ratio = N/A
Skew angle = 0.0 degrees, NA depth = 45.0 mm
Force (unfactored) carried by concrete = 371.0 kN
Force (unfactored) carried by reinforcement = -335.8 kN
Axial load eccentricity: ex = 0.0 mm, ey = 2813.0 mm

The analysis has been finished.

CALCULATION SHEET

Project/Task/File No:

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Project Description:

1 st FLOOR STOREY.

Office:

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Check: / /

AT TOP PART OF COLUMN

- Shear Demand $V^*_{top} = \frac{2 M_n}{h_{eff.}}$

$\therefore V^*_{top} = \frac{2 \times 99}{1.05} = \underline{\underline{188.6 \text{ kN}}}$

- Prob. Shear Capacity : $\underline{\underline{V_p = 179 \text{ kN}}}$

$\therefore \% NBS = \frac{179}{189} = \underline{\underline{95\%}}$

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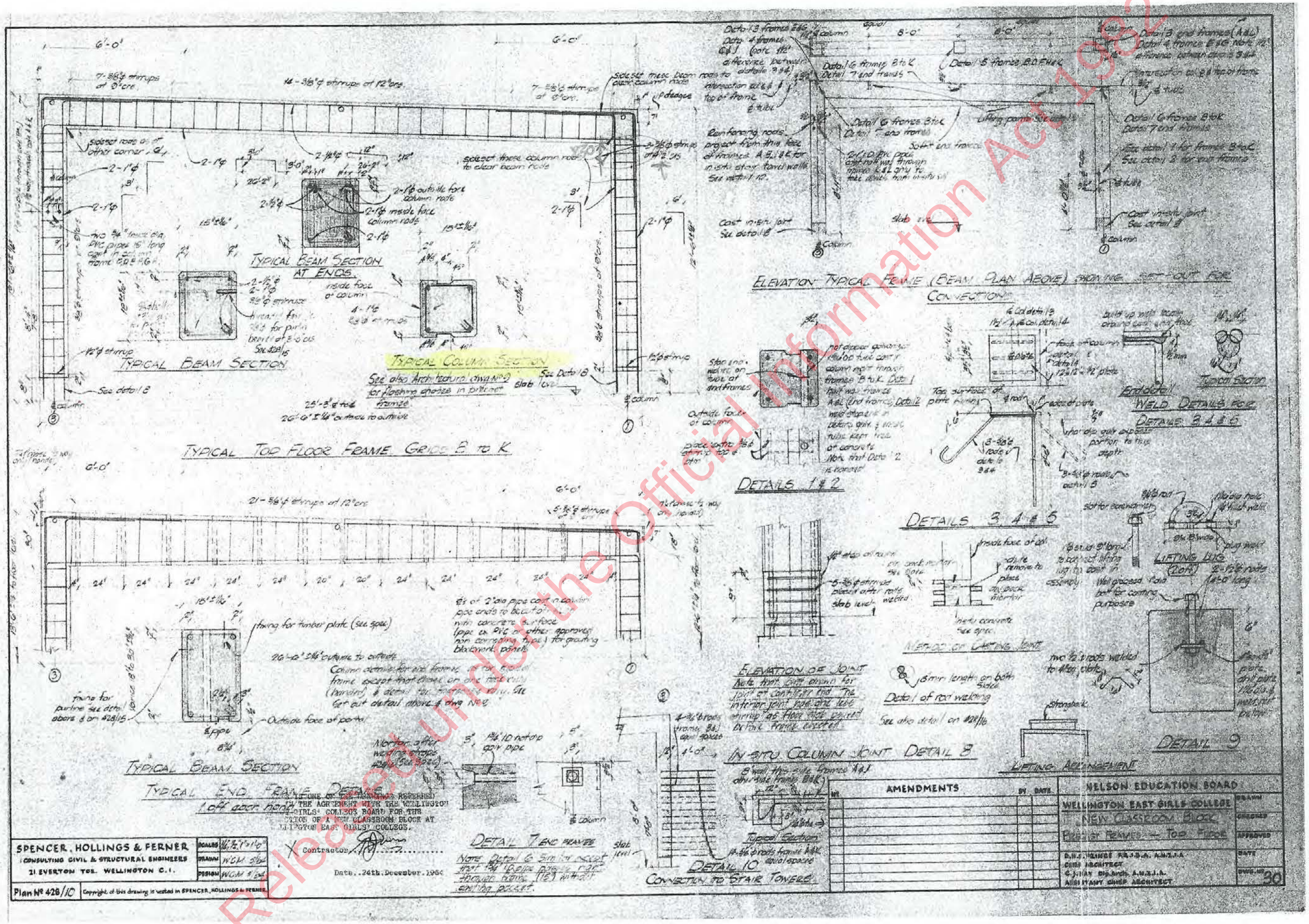

THIS IS ONE OF THE DRAWINGS REFERRED TO IN THE AGREEMENT WITH THE WELLINGTON GIRLS' COLLEGE BOARD FOR THE ERECTION OF A NEW CLASSROOM BLOCK AT WELLINGTON EAST GIRLS' COLLEGE.

Contractor.

Date..24th.December.1964

SPENCER, HOLLINGS & FERNER
CONSULTING CIVIL & STRUCTURAL ENGINEERS
21 EVERTON TCE. WELLINGTON C.1.

SCALES	1/2", 1", 1'-0"
DRAWN	WGM
DESIGN	WGM

Plan No 428/10

No	AMENDMENTS	BY	DATE

NELSON EDUCATION BOARD
WELLINGTON EAST GIRLS COLLEGE
NEW CLASSROOM BLOCK
Precast Frames – Top Floor

D.H.J. HEINDS F.R.I.B.A. A.N.Z.I.A.
CHIEF ARCHITECT
C.J. HAY Dip.Arch. A.N.Z.I.A.
ASSISTANT CHIEF ARCHITECT

DRAWN
CHECKED
APPROVED
DATE
DWG. No 30

CALCULATION SHEET

Project/Task/File No: 5-PA010.37

Project Description: Wellington East Girls College, Block 4 - South Wing

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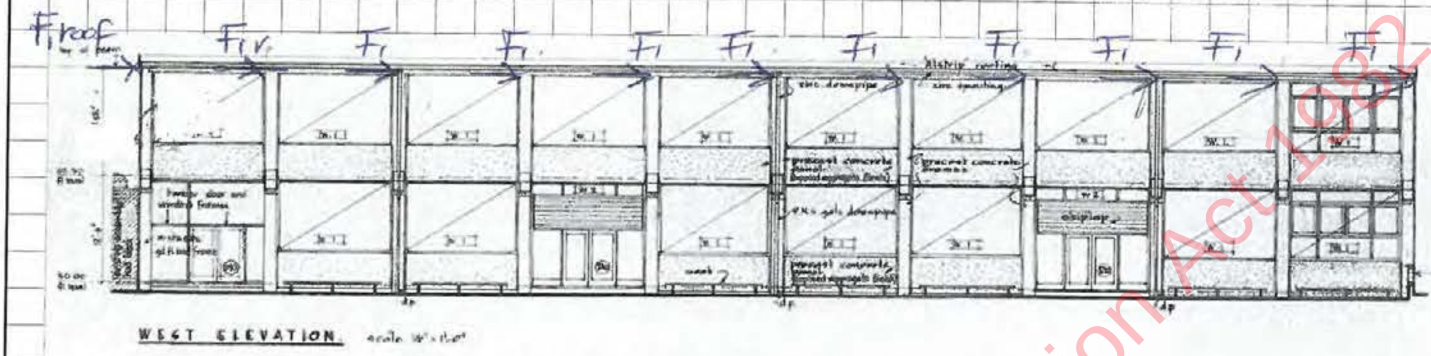
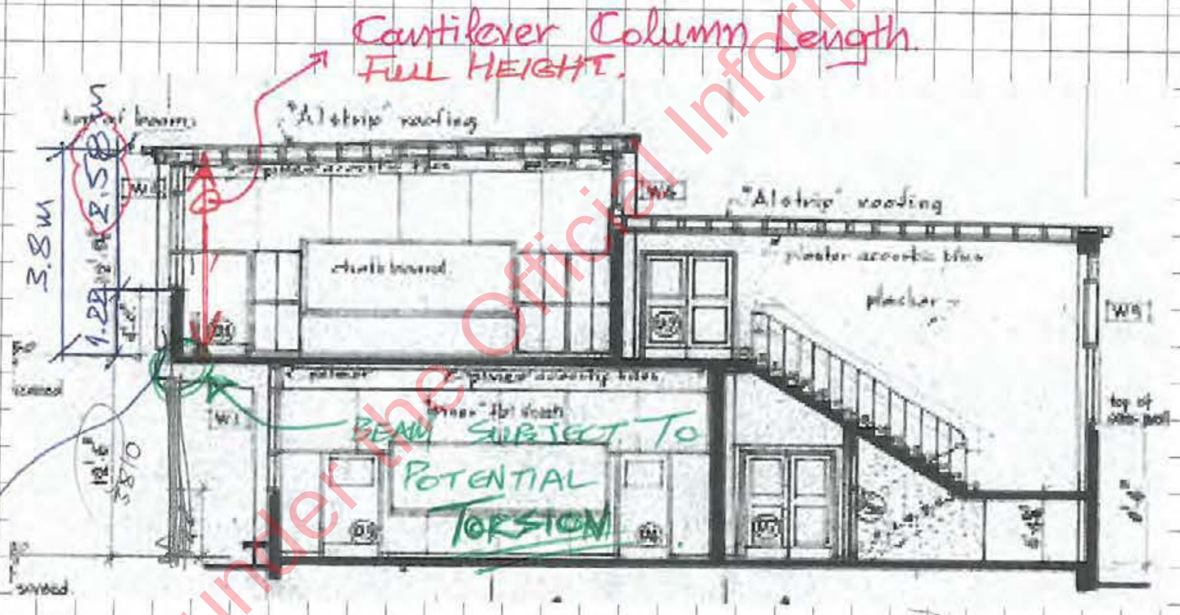
LONGITUDINAL DIRECTION

3.5/ CONCRETE FRAME.

 # Concrete Columns along front elevation - top floor



Front elevation of the building



Typical Section

Precast panel at front elevation is not taken into account for any lateral resistance. For fixing details refer to Dwg. No. 29.

- This type of panel has been checked below for out-of-plane.

CALCULATION SHEET

Project/Task/File No:

Project Description:

CHECK Cantilevered Columns at top floor – Longitudinal direction.

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Check: Rev. A 10 Oct 2015

WEIGHT along front elevation per column — kN

- Roof – Dead $= 0.6\ kN/m^2 \times \frac{8.1}{2}\ m \times 4\ m$ = 9.72

(width) (spacing between columns)

- Concrete rafter : $25\ kN/m^3 \times \underbrace{0.38^{m} \times 0.457^{m} \times \frac{7.7}{2}}_{\text{Dimensions of rafter}}$ = 16.71

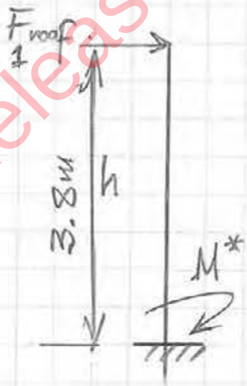
W_{tot} per Column = 26.43 kN

Hor. Loading $F_{,Roof}$

$F_{,roof} = C_d(T) \times W_{tot}$

$F_{,roof} = 0.8 \times 26.43^{kN} = \underline{\underline{21.14\ kN}}$ per Column on top.

Based on extract of drawings shown below, the top front columns are cantilevered on long direction full height with no restraint from precast panels.



Flexure Demand : $M^* = F_{,roof} \times h$

$\therefore M^* = 21.14 \times 3.8 = \underline{\underline{80.3\ kN}}$

$\therefore \frac{M_n}{M^*} = \frac{99.3}{80.3} > 100\%$.

∴ COLUMN CAPACITY IS OK. ✓

CALCULATION SHEET

Project/Task/File No:

Sheet No 480a **of**

Project Description:

Office:

3.5 CONTINUE CHECK @ 1^{st} FLOOR.
FRONT PRECAST PANELS.

Computed: / /

Check: / /

CHECK PC WALLS FOR OUT OF PLANE

Wall Height
H= 1.35m

- Horizontal Design Actions "by Parts".

$F_{ph} = C_p(T_p) C_{ph} R_p W_p$ — (1)

Where:

- $C_p(T_p) = C(0)\, C_{Hi}\, C_i(T_p)$ — (2)
- $C_{ph} = 0.85$ for $\mu = 1.25$
- $R_p = 1.0$
- $W_p = 24\ kN/m^3 \times 0.15^m \times 1.35 = 4.86\ kN/m$

With:

- $C(0) = C_h(0) \cdot Z \cdot R_u \cdot N(T,D)$.

$\therefore C(0) = 1.0 \times 0.4 \times 1.3 \times 1.0 = 0.52$

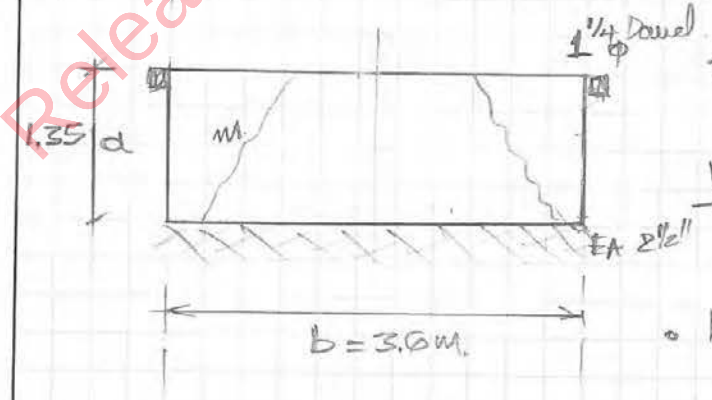
- $C_{Hi} = 1 + \frac{h_i}{6} = 1 + \frac{4.48}{6} = 1.74$
- $C_i(T_p) = 2.0$ for $T_p < 0.75 sec$

From (2) => $C_p(T_p) = 0.52 \times 1.74 \times 2 = 1.8$

From (1) => $F_{ph} = \overbrace{1.8 \times 0.85}^{1.53} \times 1.0 \times W_p$.

=> $F_{ph} = 7.43\ kN/m$: Face Load,

- Flexure Demand



$m = \frac{q a^2}{K}$ — (3)

Where:

- $q = 7.43 kN/m$.
- $a = 1.35$
- $K^2 + 2\left(3 - 4\frac{a^2}{b^2}\right)K - 192\frac{a^2}{b^2} = 0$.

for $b > 1.8a$
$3.6 > 2.16$ ✓

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