MINISTRY OF EDUCATION
Te Tāhuhu o te Mātauranga

Wellington East Girls College

Block 7 – East 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

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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).	6549
Block Name/Description	Block 7 - East Wing
Known Standard Design	Non-standard
Storeys:	3
Year of Design (approx.)	1953 – Lower storeys 2005 – 2nd Floor addition and strengthening
Gross Floor Area (m^2)	1650
Construction Type	Reinforced concrete walls with recent lightweight steel framed top-storey addition.
Assessment Type	Detailed
Date Building Inspected	10 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 stairs at the east end of the building are well protected by the surrounding concrete walls and have a low risk of collapse or significant damage in a large earthquake. The stairs accessing the 2nd floor are part of the Link Building. This block has not been investigated as part of this assessment. However, the 2005 extension that services the 2nd floor is able to tolerate expected building movements with a low risk of collapse.
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	The building has an estimated seismic capacity of 50%NBS when assessed as an IL3 building. The governing factors are: • Lack of tie beam linking the tops of the inner corridor columns in the longitudinal direction, which limits the capacity of the 2nd floor bracing along this line. The rating is governed by the ability of the large concentrically braced frame (CBF) on the front elevation to accommodate the redistributed lateral forces. In order to improve the building's rating to greater than 67%NBS, the following is required. • Retrofit new steel tie beams linking the tops of the columns which will allow the stability system at this level to work as intended. • Improve the connections between the CBF and concrete structure by installing additional fixings at the 1st and 2nd floors. Further detailed design will need to be undertaken to develop the optimum strengthening solution.
Rough order of cost estimate for seismic improvements (where required)	If new tie beams and additional connections were installed, the expected rough-order cost would be approximately $10,000 - $50,000.
Timeline for remediation if required	Medium Priority

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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 and Structural drawings of Wellington East Girls College New Wing by the Ministry of Works dated 1952. Job Number AWDO 20776, sheets 01 to 23.
    - Specifications of Wellington East Girls College New Wing by the Ministry of Works dated 1952. Job Number AWDO 20776.
    - Structural drawings of Wellington East Girls College East Wing Block 7 Structural Strengthening by Connell Wagner Limited 2003. Project Number 7970, sheets S01 to S06.
    - Structural drawings of Wellington East Girls College extension to East Wing Block 7 by Connell Wagner Limited 2005. Project Number 797012, sheets S00 to S26.
    - Architectural drawings of Wellington East Girls College East Block Classroom Additions by Fiona Christeller Architects 2005. Project Number 0501, sheets 100 to 401.
    - Structural drawings and calculations of Wellington East Girls College East Wing Block 7 Seismic Strengthening by Aurecon dated 8 March 2012. Project Number 228499.

- 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	3
Gross Floor Area (m^2)	1650
Year of Design (approximate)	1953
Current use	Teaching Spaces
Structural Alterations	2003 strengthening 2005 addition of a 3rd level
Basement	None
Gravity Load Resisting System	Ground and 1st floors: Reinforced concrete walls and frames. 2nd floor (2005 addition): Steel frames.
Lateral Load Resisting System	Ground and 1st floors: Reinforced concrete walls. 2nd floor (2005 addition): Braced steel frames and portal frames.
Wall/Cladding/Roof System	Painted concrete walls, glazing. The roof system is steel or timber roof purlins supporting lightweight cladding.
Floor System	The first floor is an in-situ concrete slab on downstand beams and walls. The 2nd floor is a lightweight timber floor.
Foundation System	Concrete slab with shallow strip and pad foundations.
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 Block 7 is fully founded on rock based upon geotechnical investigations around the College. 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.

2.1 Structural System

The building is generally of reinforced concrete construction, dating from the 1950s. The lateral stability system comprises concrete shear walls in both directions. In the transverse direction, these are regularly spaced along the building. In the longitudinal direction, the shear walls are concentrated to the rear of the building. Torsional response in the longitudinal direction is resisted by the transverse shear walls. The first floor slab provides a rigid diaphragm. No rigid diaphragm exists at the roof level of the original building.

Around 2005, a new lightweight 'penthouse' storey was added at roof level.

The 2005 addition consists a lightweight steel framed structure, with lightweight timber floor. The lateral stability system for this level comprises bracing in the longitudinal direction and portal frames in the transverse direction. The roof is braced on plan with extensive Reidbar cross-bracing. The timber floor, which provides a flexible diaphragm, is supported by steel beams fixed to the concrete frame below.

The longitudinal bracing is provided by tension-only cross bracing along the corridor at the rear of the building, and by the concentrically braced frame (CBF) on the front of the building. This CBF is connected to the concrete frames below via steel cleats fixed with bolts and chemical anchors. It does not have its own foundations; instead relying on the pads beneath the fin columns to which it is fixed.

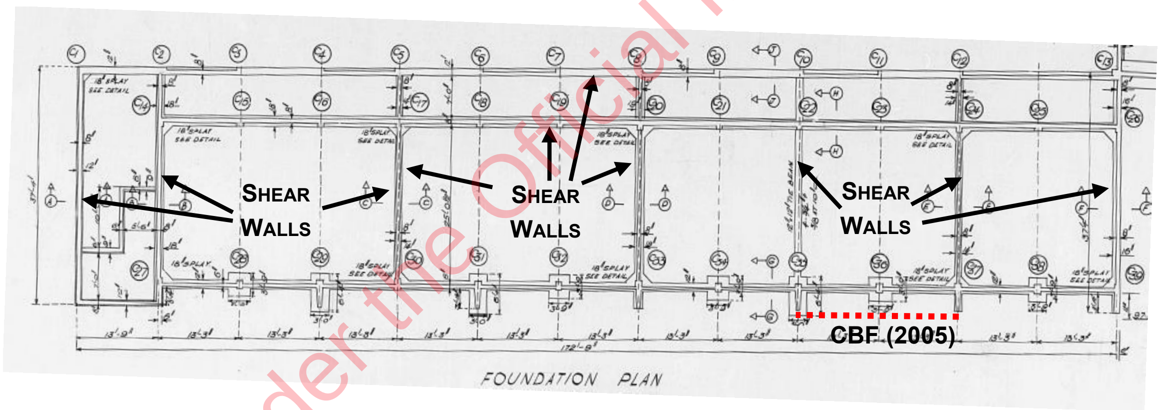

Fig. 1. Foundations and Shear Wall Layout

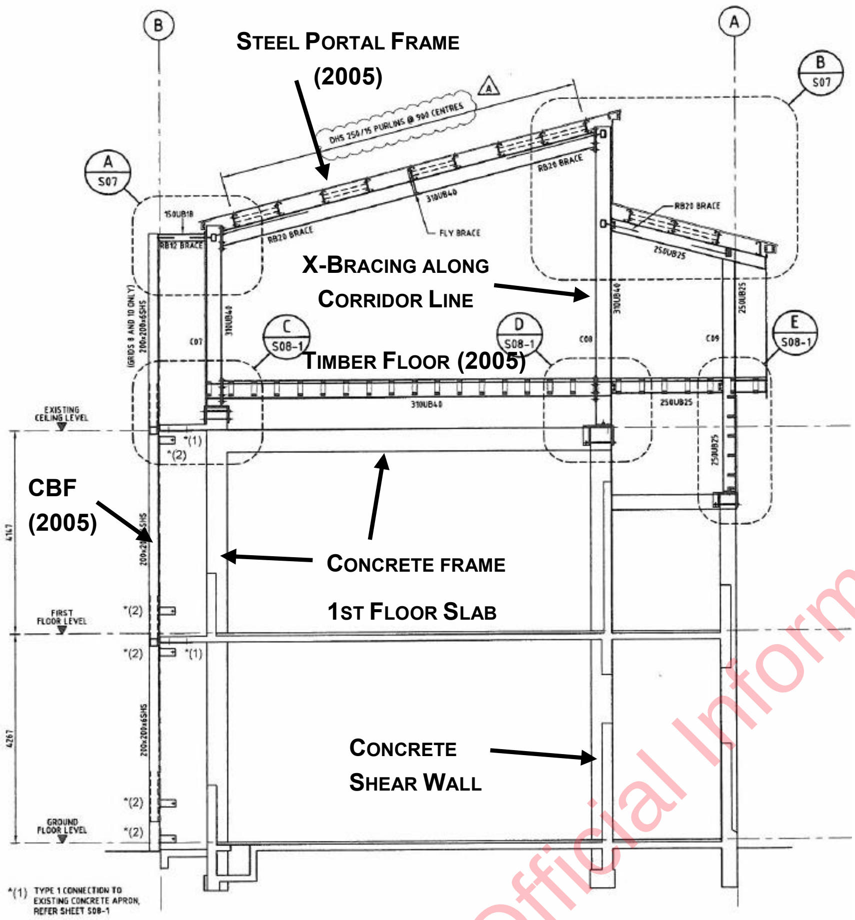

Fig. 2. Cross-section

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 1953 by the Ministry of Works. The design predates building code NZSS1900 Chapter 8 (1965). The redevelopment in 2005 was designed by Connell Wagner Limited. The applicable design code at this time was NZS 4203 (1992).

A force-based method was used to determine the seismic capacity of the building due to its low rise and relatively stiff form. Due to the complexity of the building, which has different structural systems from different eras, the building was modelled using the ETABS 3D building analysis software. Both a modal response spectrum analysis and an equivalent static analysis was carried out in accordance with the current earthquake loading standard NZS1170.5. It was found that the equivalent static analysis gave more conservative results and was generally used to obtain demands. Demands for the lighter top storey were also obtained using NZS1170.5 Section 8 - Parts and Components.

The first floor slab comprises a 5" (127mm) thick concrete slab which provides a rigid diaphragm, distributing lateral forces to the stability elements based on their stiffness. The circa 2005 timber floor was modelled as a flexible diaphragm.

The capacity of the wall elements, columns, diaphragm connections, steel braces and portal frames were assessed using guidelines given in NZS3101, NZS3404 and the NZSEE publication *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes* (NZSEE 2006). Hand calculations and structural software (for 2D frame models) 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.

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

3.2 Intrusive Investigations

None. The main structural elements, such as walls and bracing, are generally exposed. The site investigation generally confirmed that the original construction drawings were accurate.

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	Concrete elements: 1.25 – Shear Concrete elements: 2.0 – Flexure Steel hollow sections: 1.25 Reid Braces: 1.0
SP Factors	$\mu \leq 1.25$: 0.9 $\mu \geq 2$: 0.7

The following material strengths were assumed for analysis. These are based on guidance published by the NZSEE for the original 1950s structure and current materials standards for the recent additions.

Material	**Strength**
Concrete compressive strength f'c	30 MPa
Reinforcement (1950s)	245 MPa
Rolled steel sections	Grade 300
Steel hollow sections	Grade 350
Reid Brace	500 MPa

3.4 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
2nd Floor wall bracing: Reid Braces and connections	75%NBS	Experience from the Canterbury Earthquakes has shown that Reid Brace connections can fail in a brittle manner. The RB25 cross-braces and their connections at this level were therefore assessed for elastic demands.
2nd Floor wall bracing: Cantilevering concrete columns	50%NBS (In combination with the External CBF Frame – see below)	The RB25 cross-braces connect to the top of the original concrete columns. The original drawings do not show a tie beam between the columns at this level. The brace forces must, therefore, be resisted only by individual cantilevering columns. They have some capacity to resist the demands and load can be transferred to the large bracing frame (CBF) on the front elevation via the roof bracing as these columns begin to yield. Therefore there is some redundancy in the system.
External CBF frame Connections	>50%NBS	This frame is the main stability element in the longitudinal direction for the top storey. The main concrete structure at ground and first floors does not rely on the frame for stability. The demands on the CBF are largely dependant on the effectiveness of the cross braces along the corridor wall line (see above). Due to the probable lack of a tie beam linking the bottom of these braces, lateral loads at this level will tend to redistribute to the front CBF. The frame is connected to the concrete structure through bolted fixings on the beams at 1st and 2nd floor and down the columns. The frame does not have its own foundations and instead relies on these connections to transfer forces out into the main concrete structure. The connections between the beams and concrete frame are quite eccentric and have a limited capacity. They are assessed as having a capacity of between 50 and 60%NBS, based on this frame taking the full demands from the penthouse roof above without any contribution from the RB25 cross-bracing along the corridor

		walls, which is somewhat conservative. If the cross braces could be relied on to be fully effective, the demands on the CBF connections would be lower and its rating would be approximately 70-90%NBS.
Concrete walls in-plane shear strength	>85%NBS	The building has a number of concrete shear walls that provide a high level of strength, stiffness and redundancy.
Concrete walls in-plane flexural strength	>90%NBS	The concrete shear walls in both directions are relatively long and squat so are typically shear-governed.
First floor slab	>100%NBS	The in situ slab has a high level of strength to enable it to transfer forces to the stability elements.
Concrete frames	>100%NBS	The stiffness of the building and strong diaphragms will keep displacements low and protect the concrete frames from excessive damage.
Concrete walls out-of-plane strength	>100%NBS	There is only a flexible diaphragm at the original roof/ second floor level, so the concrete walls may need to support themselves in the out-of-plane at this level. The walls have sufficient capacity to resist the demands.
Second floor concrete beams out-of-plane strength	95%NBS	Similarly, the beams at this level have a high capacity to resist out-of-plane demands.
2005 strengthening to rear corridor walls	>100%NBS	It is unlikely that the steel braces added to the original concrete rear corridor walls will attract very much force, due to the stiffness differences between them and long concrete wall elements.
Foundations	>80%NBS	The building is founded on rock using shallow strip and pad foundations. Some rocking of the shear walls may occur, which could cause the allowable bearing pressure to be exceeded in localised areas. However, the building is robust and should be able to tolerate some rocking and moderate settlements without risk to its stability.

The assessment confirms that the building achieves an overall seismic capacity of 50% NBS.

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

3.5 Structural Weaknesses & Life Safety Hazards

3.5.1 Potential Critical Structural Weaknesses

No Critical Structural Weaknesses were identified in this assessment.

3.5.2 Specific Critical Structural Weaknesses

No Critical Structural Weaknesses were identified in this assessment.

3.5.3 Concrete Shear Walls

The original part of the building relies on a number of concrete shear walls for its stability in both directions. These are reasonably strong and well distributed. The concrete shear walls have a minimum seismic rating of 85%NBS due to their shear capacity at the ground floor.

The walls have low aspect ratios and are shear governed. The concrete shear walls were found to have a minimum seismic rating of 90%NBS in flexure.

3.5.1 Diaphragms

The concrete floor diaphragm on the first floor is rigid enough to transfer the total lateral load to the walls in both directions. Generally, the connection between the walls and diaphragm is adequate to transfer the demand forces. On the north elevation, where the slab joins the walls at the east end stairwell, the connection is quite weak. Here, some damage may occur in the floor slab. Alternative load paths are available, however, and there is little risk of the slab losing support.

The roof to the 2005 addition utilises on-plan Reidbraces to transfer lateral forces between the stability elements. These braces all have a minimum seismic rating of 100%NBS.

3.5.2 Concrete Frames - Generally

Concrete frames generally provide gravity support to the building, with stability provided by the shear walls. The stiff first floor diaphragm, along with the large number of shear walls orientated in both directions, resisting any torsion, will keep displacements very low. This will protect the frame elements from excessive damage to enable them to continue to provide gravity support.

The concrete frames have a minimum seismic rating of 100%NBS.

3.5.3 2nd Floor Steel Frames – Tension-only Bracing

In the longitudinal direction, the top storey is stabilised by the large concentrically braced frame on the north elevation and by two braced bays along the line of the inner corridor wall. These braced bays use tension-only RB25 Reidbraces. Due to observations of non-ductile behaviour of Reidbraces in the Canterbury Earthquakes, these were assessed only for elastic (μ=1.0, Sp=0.9) demands. The Reidbraces have a minimum seismic rating of 75%NBS, limited by the capacity of their connections to the main concrete structure.

The braces transfer forces to the main concrete structure below through their connection to the cantilevering concrete columns. The assessment of these elements are described below.

3.5.4 Concrete Cantilevering Columns Supporting Second Floor

On the interior corridor wall, the columns cantilever approximately 1m above the wall line, forming clerestory windows. No tie beam in the longitudinal direction linking the top of these columns is shown on the original construction drawings. However, it appears that one was assumed to be present when designing the 2005 addition, as the Connell Mott MacDonald drawings show an existing tie beam. Site inspections undertaken for this assessment indicated that there is not a tie beam present.

The cross-bracing for the 2005 addition fixes to the tops of four of these columns. A lack of tie beam means that bracing forces are therefore concentrated on these individual columns rather than being shared along the row. These columns are able to provide some resistance through cantilever action to transfer lateral forces from the braces in to the main structure. The capacity of these cantilevering columns is approximately 55%NBS due to the lack of a tie beam. If these columns did begin to yield and deflect, however, lateral load from the top storey would tend to redistribute to the large CBF on the front elevation, which has some spare capacity. Based on this alternative load path being utilised, with some (very limited) resistance still provided by the tension-only braces/ cantilevering columns, the overall system stability in the longitudinal direction is assessed to be at least 50%NBS.

3.5.5 Concentrically Braced Frame on North Elevation

The large concentrically braced frame (CBF) on the north elevation provides stability to the top storey. As the relative stiffness of the main concrete structure is high compared to the CBF, the main concrete structure does not seem to rely on the CBF for any bracing. As part of the analysis, a study was undertaken assessing the effects of disconnecting the frame from the main concrete structure, and it was found that this did not have a large effect on the concrete structure. The CBF is, however, the main stability element for the top storey in the longitudinal direction and does provide some redundancy to the lower concrete structure. The demands on the CBF are largely affected by the effectiveness of the top storey's longitudinal cross-bracing and cantilevering columns along the corridor wall line (see previous section).

The CBF is connected to the concrete structure via steel cleats and anchor bolts. At each level, the horizontal beam is connected to the concrete frame through six cleat connections, which each have 4 Hilti anchors. These connections are eccentric and consequently, only two of the anchors can really be fully engaged. Treating the top storey as a part in order to calculate lateral earthquake demands, and assuming only a minor contribution (less than 10%) from the cross-bracing along the corridor wall line, the seismic rating of the connections at Level 2 and Level 1 is assessed as being between 50% and 60% NBS. Failure of the connections will principally affect the stability of the top storey roof. The 2nd floor itself is fixed to the concrete frames, which are able to act in bending to support lateral demands.

The concentrically braced frame (CBF) has a minimum seismic rating of 50%NBS, based on the capacity of its connections and assuming that it has to provide the majority of support to the top storey in the longitudinal direction. If, however, the cross-braces and cantilevering columns along the corridor wall line were to be effective, then the demands on the CBF would reduce and it would have a rating of around 70%NBS.

3.5.6 Foundations

The building is founded on rock using shallow strip and pad foundations. Analysis indicated that in a large earthquake, some of the shear walls may rock. The rocking action will increase the foundation stresses at the ends of the walls. Since the rock is expected to have a relatively high bearing capacity, this should not lead to excessive settlements. An assessment of a typical wall rocking on its foundation indicated that the ground bearing capacity may be exceeded at demands corresponding to approximately 80%NBS.

In order to assess displacement demands on a rocking wall, an approximate push-over analysis was carried out on a typical wall. This indicated that the displacement demand at 1st floor would be approximately 80mm, which corresponds to a drift of around 1.9%.

The structure has a moderate level of ductility and overall robustness that should allow it to tolerate this level of drift and/ or some localised foundation settlements without its stability being compromised.

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.

3.5.7 Stairs

The main stairs situated at the east end of the building are of in-situ construction and supported by long concrete walls which will provide a high level of protection. The risk of damage to these stairs in an earthquake is considered to be very low.

The stairs servicing the newer second floor are part of the adjacent Link Block. This block has not been investigated as part of this assessment. However, it is understood to partly date from the 1920s, with major modifications made in the 1950s, 1980s, and around 2005 when the stair was extended to provide access to the new East Block second floor.

This extension comprises a lightweight steel and timber structure built above the pre-existing concrete and unreinforced masonry structure. Due to its form of construction and relative low weight, this extension should be able to tolerate moderate building displacements without risk of collapse.

3.5.8 Secondary Structural Weaknesses & Life Safety Hazards

No Secondary Structural Weaknesses or Life Safety Hazards were identified in this assessment.

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:

- In order to make the stability system at the roof level work as the designer intended, new steel tie beams should be installed between the concrete columns. The tops of these columns are reasonably exposed allowing new steel SHS members to be fixed to them.
- Retrofit additional fixings to the CBF beams to improve the connectivity of the CBF to the concrete frames.

4.2 Rough Order of Cost Estimate

A rough order of cost estimate for the suggested physical improvements above is $10,000-$50,000 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 when considered as an Importance Level 3 building. This does meet the Ministry of Education's minimum seismic strength requirements of not being earthquake-prone in the short term, and there is no need to change the building's current occupancy.

5.2 Recommendations

The building is not earthquake prone, and there is no need to change the buildings current occupancy. Seismic improvements have been suggested to achieve a minimum seismic capacity of 67%NBS. A rough order of cost estimate for these improvements has been provided.

A recommended estimate for remediation is to be a medium priority.

Detailed design will need to be undertaken to further develop the suggested seismic improvements and provide more cost certainty. Upon completion of design documentation a building consent application will need to be lodged and approved prior to the installation of the suggested seismic improvements.

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/Reference No: Grey Base Hospital	Sheet No:
Project and Description: Main Building Pile Design	Office:
	Computed: PMO
	Checked:

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These pages contain information and calculations on the analysis undertaken for the Detailed Seismic Assessment of Wellington East Girls' College - East Block.

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2005 2nd floor minor axis

CALCULATION SHEET

L / 01

Project/Task/File No: HTC STAGE 2 | Sheet No | of

Project Description: | Office:

Computed: PMO / 18/07/2015

Check: / /

EAST BLOCK BUILDING WEIGHTS

ROOF

PERMANENT

Cladding	0.1
Steelwork	0.25
Ceiling & Services	0.2 kPa

Σ 0.55 kPa

ROOF – 2nd FL

PERMANENT

Cladding	0.1 kPa	on Elevation
Steelwork	0.1 kPa	" "
Girts	0.2 kPa	" "
Timber	0.1 kPa	" "

Σ 0.5 kPa

2nd FLOOR

PERMANENT

New steel / Timber floor =

Finishes	0.05	
Timber	0.15	0.4 kPa
Steelwork	0.2	
Partitions		0.5 kPa

Original Roof : As above :

Cladding	0.1 kPa
Steelwork	0.25 kPa
Ceiling & Services	0.2 kPa

Concrete Beams = 10" × 12" @ 4.038m c/c / 7.73m long : 0.1 kPa

IMPOSED

Classrooms 3.0 kPa with ψ_E = 0.3

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CALCULATION SHEET

L/02

Project/Task/File No: WEGC EAST BLOCK

Sheet No of

Project Description:

Office:

Computed: PMO 18/09/2015

Check: / /

2nd FLOOR - 1ST FLOOR

PERMANENT LOADS

8" Walls = 5.075 kPa on Elevation

6" Walls = 3.8 kPa

Columns : Normal 16"x12" = 3.1 kN/m

Fin = 8.9 kN/m

Cladding = 0.5 kPa on Elevation

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CALCULATION SHEET

L/03

Project/Task/File No: WEHC EAST BLOCK

Sheet No of

Project Description:

Office:

Computed: PMO / /

Check: / /

1ST FLOOR
PERMANENT

Finishes	0.05
Ceiling & Services	0.25
Slab - 5"	3.23
Beams =	0.15
Partitions	0.5
	4.18 kPa

IMPOSED

Classrooms 3.0 kPa

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BUILDING LOAD TAKEDOWN

6/04a

Total Building Load Takedown

		Load Type	Type	DL (Unit)	LL (Unit)	Wall Length (m)/ Number	Area/ Length	Dead (kN)	Imposed (kN)	SLS (kN)	Imposed Coefficients LL Red.	Seismic Coeff.	LL		Force ULS (kN)	Storey ULS (kN)	Cumulative ULS (kN)	By Type DEAD	Storey ULS (kN)	Cumulative ULS (kN)
Top Roof		LW Roof	DEAD	0.35	0		365	128	0				0		128			128		
		Roof LL	DEAD	0	0.25		365	0	91		1	0	0		0			0		
		0.2 Ceiling & Services	DEAD	0.2	0		365	73	0				0		73			73		
										292			0	0	201		201	201		201
Rise	4	LW Wall Cladding	DEAD	0.5	0	87.506	350	175	0				0		175			175		
										175			0	0	175	376	376	175	376	376
Level	2	Timber Floor	DEAD	0.35	0		365	128	0				0		128			128		
		0.05 Finishes	DEAD	0.05	0		365	18	0				0		18			18		
		0.2 Ceiling & Services	DEAD	0.2	0		601	120	0				0		120			120		
		LW Partitions	DEAD	0.5	0		365	183	0				0		183			183		
		Concrete Beams	DEAD	0.15	0		601	90	0				0		90			90		
		LW Roof	DEAD	0.35	0		176	61	0				0		61			61		
		Roof LL	DEAD	0	0.25		176	0	44		1	0	0		0			0		
		Classroom LL	LL	0	3		365	0	1095		1	0.3	329		329			0		
										1740			329	329	929		1305	600		976
Rise	4.147	8" RC Wall	DEAD	5.1156	0	80	330	1688	0				0		1688			1688		
	4.147	6" RC Wall	DEAD	3.8304	0	55	229	879	0				0		879			879		
	4.147	Fin Columns	DEAD	8.9	0	5	21	185	0				0		185			185		
	4.147	16"x12" Columns	DEAD	3.1	0	17	70	219	0				0		219			219		
	4.147	LW Wall Cladding	DEAD	0.5	0	117	483	242	0				0		242			242		
										3211			0	0	3211	4140	4516	3211	3812	4188
Level	1	5" Slab	DEAD	3.2004	0		601	1923	0				0		1923			1923		
		Concrete Beams	DEAD	0.15	0		601	90	0				0		90			90		
		0.05 Finishes	DEAD	0.05	0		601	30	0				0		30			30		
		0.25 Ceiling & Services	DEAD	0.25	0		601	150	0				0		150			150		
		LW Partitions	DEAD	0.5	0		601	301	0				0		301			301		
		Classroom LL	LL	0	3		601	0	1803		1	0.3	540.9		541			0		
										4297			541	541	3035		7551	2494		6682
Rise	4.267	8" RC Wall	DEAD	5.1156	0	80	339	1737	0				0		1737			1737		
	4.267	6" RC Wall	DEAD	3.8304	0	55	236	904	0				0		904			904		
	4.267	Fin Columns	DEAD	8.9	0	5	21	190	0				0		190			190		
	4.267	16"x12" Columns	DEAD	3.1	0	17	73	225	0				0		225			225		
	4.267	LW Wall Cladding	DEAD	0.5	0	117	497	249	0				0		249			249		
										3304			0	0	3304	6339	10856	3304	5799	9986

L/04 b

CALCULATION SHEET

Project/Task/File No: WEGC EAST BLOCK

Project Description:

Sheet No of

Office:

Computed: AMO / 18/09/2015

Check: / /

QUICK CHECKS

UPPER ROOF

DL ~ 0.5 kPa × $\overbrace{11.379 \times 32.404}^{369}$ = 184

TIMBER FLOOR

DL ~ 1.0 kPa × " " = 369

LL 3.0 × 0.3 × " " = 332

OLD ROOF

DL ~ 2 × $\overbrace{11.379 \times 52.594}^{598}$ = 1197

1ST FLOOR

DL ~ 8 kPa × " " = 4784
- Inc. walls

LL 3 × 0.3 × " " = 532

7404

With walls → ~ 10000 kN looks about right.

Fl. sq metre = $\frac{10000}{(598 + 369)}$ = 10 kPa - looks ~ right

OPUS

ETABS MODEL
– VERTICAL LOAD CHECKS

Story	Load Case/Com	Location	P	VX	VY	T	MX	MY
Story5	DL+SDL+0.3LL	Top	217	0	0	0	1236	-7889
Story5	DL+SDL+0.3LL	Bottom	236	0	0	2	1397	-8561
Story4	DL+SDL+0.3LL	Top	860	0	0	2	5285	-31277
Story4	DL+SDL+0.3LL	Bottom	759	0	2	61	4105	-27604
Story3	DL+SDL+0.3LL	Top	1321	0	2	59	5934	-41739
Story3	DL+SDL+0.3LL	Bottom	714	10	-107	-2913	5624	-17457
Story2	DL+SDL+0.3LL	Top	906	7	-110	-2945	7904	-24354
Story2	DL+SDL+0.3LL	Bottom	2285	7	-110	-2945	20461	-55452
Story1	DL+SDL+0.3LL	Top	6895	0	0	0	40405	-188319
Story1	DL+SDL+0.3LL	Bottom	10095	0	0	0	62933	-268202

Sim numbers to Manual load takedown

LOADCASE = 1.0DL + 1.0SDL + 0.3LL

∴ ETABS Model gives similar loading values to 'manual' load takedown

∴ O.K.

ETABS MODEL

VERTICAL LOAD CHECKS

L/06

Story	Load Case/Com	Location	P	VX	VY	T	MX	MY
Story5	Dead	Top	51	0	0	0	294	-1868
Story5	Dead	Bottom	72	0	0	2	456	-2605
Story4	Dead	Top	133	0	0	2	823	-4821
Story4	Dead	Bottom	116	0	0	9	602	-4222
Story3	Dead	Top	597	0	0	7	2013	-17568
Story3	Dead	Bottom	359	2	-79	-2188	2603	-6362
Story2	Dead	Top	435	3	-80	-2206	3585	-9208
Story2	Dead	Bottom	1814	3	-80	-2206	16051	-40318
Story1	Dead	Top	5097	0	0	0	29724	-134249
Story1	Dead	Bottom	8279	0	0	0	52220	-214095

Dead Load only

4/07

EARTHQUAKE FORCES - BASE SHEAR

- μ = 1.25
Sp = 0.9

OPUS

Job Title: *WEGC East Block*
Job Number:
Member Reference:
Calcs By: *PMO*
Date: *18/09/2015 1:53:00 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) = 0.4 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) = 10000 kN

ULS Results
ULS Return Period of 1/1000
Spectral Shape Factor Ch(T) = 1.891
Return period factor from table 3.5 (Ru) = 1.30
Near Fault Factor N(T,D) = 1.000
Elastic Site Spectrum C(T) = 0.9836
Ductility Factor k(mu) = 1.143
Design Action Coefficient Cd(T) = 0.775
Horizontal Seismic Shear = **7746** kN

SLS1 Results
Return Period of 1/25
Return period factor (Rs) = 0.25
Elastic Site Spectrum C(T) = 0.1891
Ductility Factor k(mu) = 1.000
Design Action Coefficient Cd(T) = 0.132
Horizontal Seismic Shear = **1324** kN

L/09

Story	Load Case/Com	Location	P	VX	VY	T	MX	MY
Story1	Dead	Bottom	8291	0	0	0	52220	-214574
Story1	Live	Bottom	2760	0	0	0	16288	-83245
Story1	SDL	Bottom	988	0	0	0	5827	-29150
Story1	DL+SDL+0.3LL	Bottom	10107	0	0	0	62933	-268697
Story1	EQENV Max	Bottom	10107	1970	1970	98800	109366	-254767
Story1	EQENV Min	Bottom	10107	-6568	-6568	-198380	49003	-315130

L/10

OPUS

Job Title: *WEGC East Block*
Job Number: **Member Reference:**
Calcs By: *PMO* **Date:** *18/09/2015 2:18:47 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) = 0.4 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) = 8480 kN

ULS Results
ULS Return Period of 1/1000
Spectral Shape Factor Ch(T) = 1.891
Return period factor from table 3.5 (Ru) = 1.30
Near Fault Factor N(T,D) = 1.000
Elastic Site Spectrum C(T) = 0.9836
Ductility Factor k(mu) = 1.143
Design Action Coefficient Cd(T) = 0.775
Horizontal Seismic Shear = **6568** kN

SLS1 Results
Return Period of 1/25
Return period factor (Rs) = 0.25
Elastic Site Spectrum C(T) = 0.1891
Ductility Factor k(mu) = 1.000
Design Action Coefficient Cd(T) = 0.132
Horizontal Seismic Shear = **1123** kN

NZS 1170 2004 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQX according to NZS 1170 2004, as calculated by ETABS.

Direction and Eccentricity

Direction = X

Structural Period

Period Calculation Method = User Specified — Conservative

User Period $T = 0.4$ sec

Factors and Coefficients

Return Period Factor, R [NZS Table 3.5] $R = 1.3$

Hazard Factor, Z [NZS Table 3.3] $Z = 0.4$

Structural Performance Factor, Sp [NZS 4.4] $S_p = 0.9$

Structural Ductility Factor, μ [NZS 4.3] $\mu = 1.25$

Near Fault Distance, D [NZS 3.1.6] $D = 10$

Site Sub-soil Class [NZS 3.1.3] = Be - Rock

Equivalent Lateral Forces

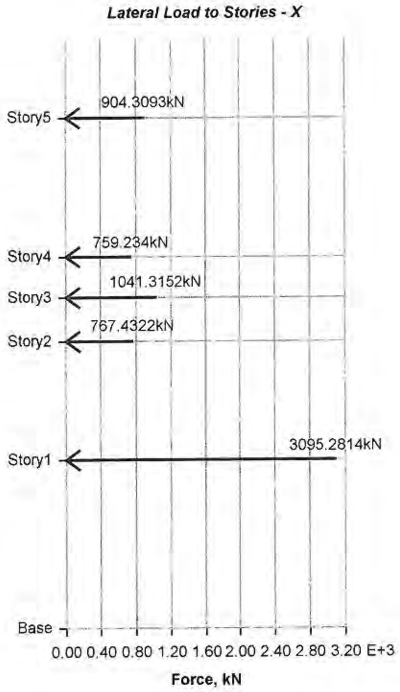
Spectral Shape Factor, $C(T_1)$ [NZS Table 3.1] $C(T_1) = 1.6$

Seismic Design Action Coefficient, $C_d(T_1)$ [NZS 5.2.1] $C_d(T_1) = \frac{C(T_1)\, S_p}{\mu}$

Calculated Base Shear

Direction	Period Used (sec)	$C_{d(T}$	W (kN)	V (kN)	F_t (kN)
X	0.4	0.774562	8479.0744	6567.5721	525.4058

Applied Story Forces



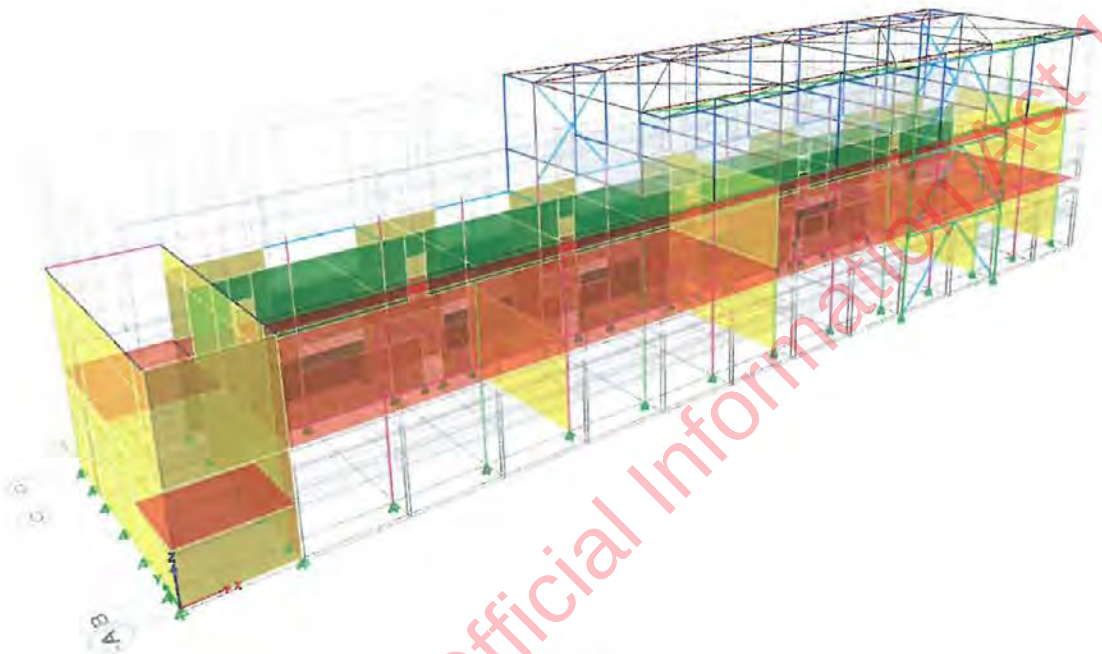
Story	Elevation m	X-Dir kN	Y-Dir kN
Story5	13	904.3093	0
Story4	9.44	759.234	0
Story3	8.414	1041.3152	0
Story2	7.309	767.4322	0
Story1	4.267	3095.2814	0
Base	0	0	0

OPUS

M/01

Calculation Sheet	
Project/Reference No: Wellington East Girls' College East Block DSA	Sheet No:
Project and Description:	Office:
	Computed:
	Checked:

ETABS Analysis



3D View

M/02

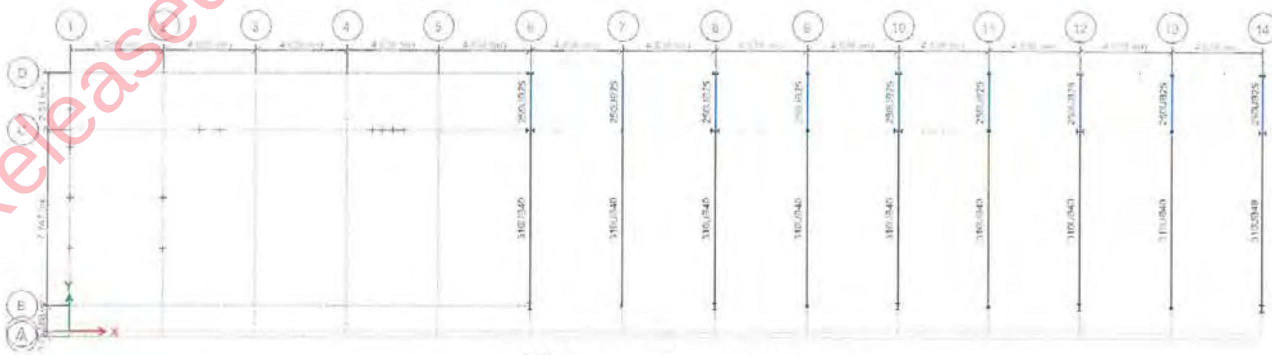
OPUS

Plans

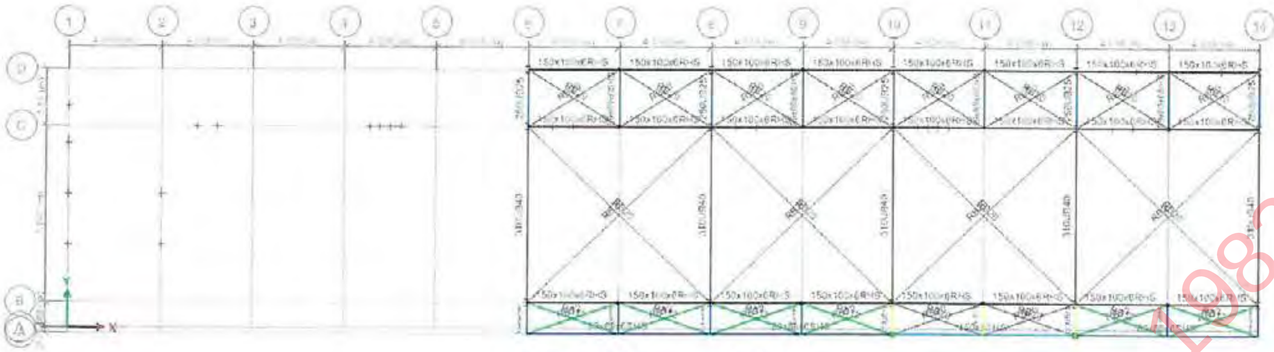
1st Floor

Rear Corridor Top of Wall Level (Storey 2)

Top of Original Building (Storey 3)

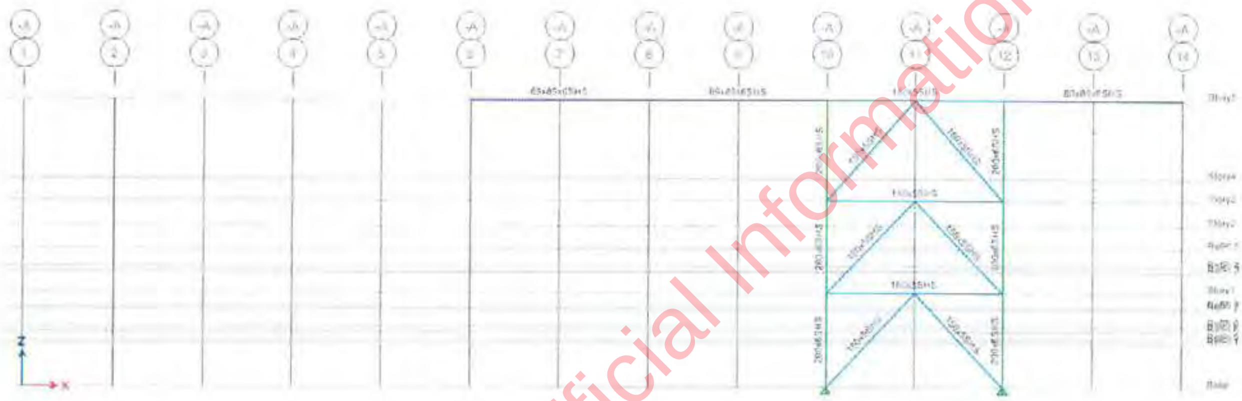


2005 Addition – Floor (Storey 4)

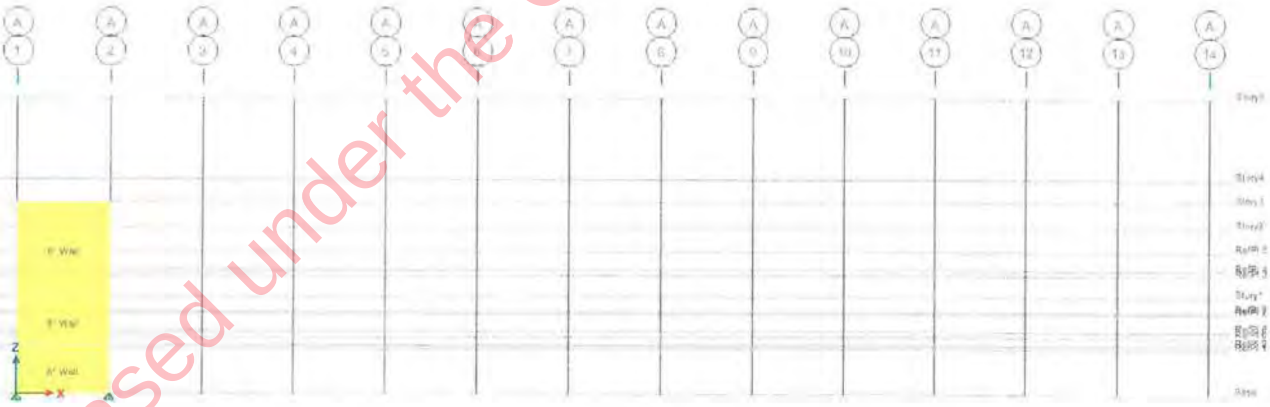


2005 Addition – Roof

Elevations



GLA-



GLA

GLB

GLC

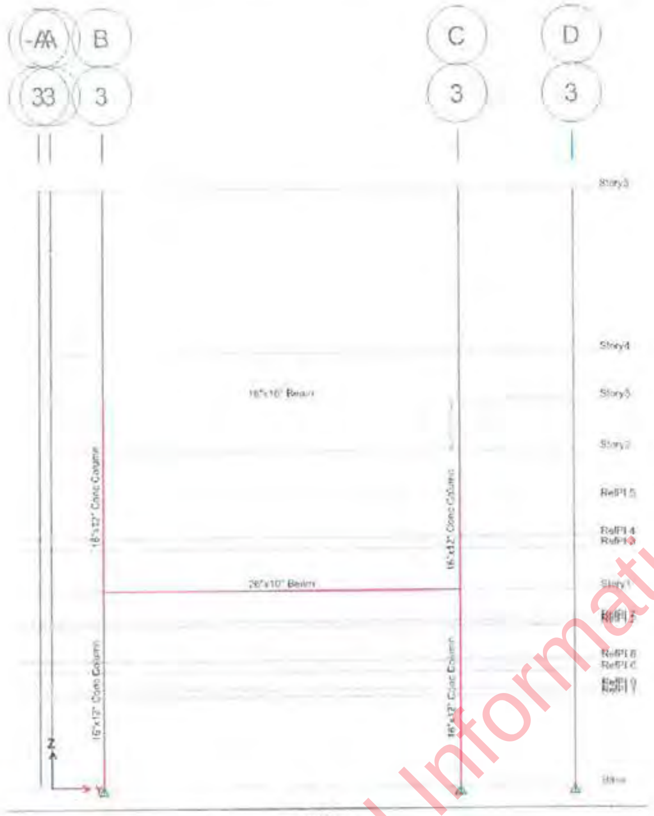
GLD


-A
B
C
D
1
Story5
Story4
Story3
Story2
RefPl 5
Story1
Base
DUMMY
8" Wall
Z
Y

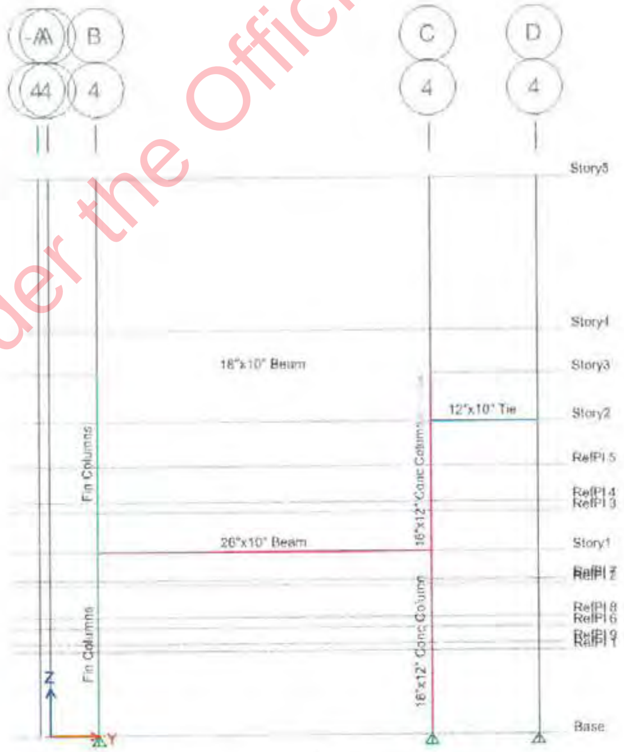
GL1


-A
B
C
D
2
Story5
Story4
Story3
Story2
RefPl 5
Story1
Base
DUMMY
8" Wall on GL2
8" Wall
30"x8" Beam
Z
Y

GL2


-AA
B
C
D
33
3
3
3
18"x10" Beam
26"x10" Beam
16"x12" Conc Column
Story4
Story3
Story2
Story1
Z
Y

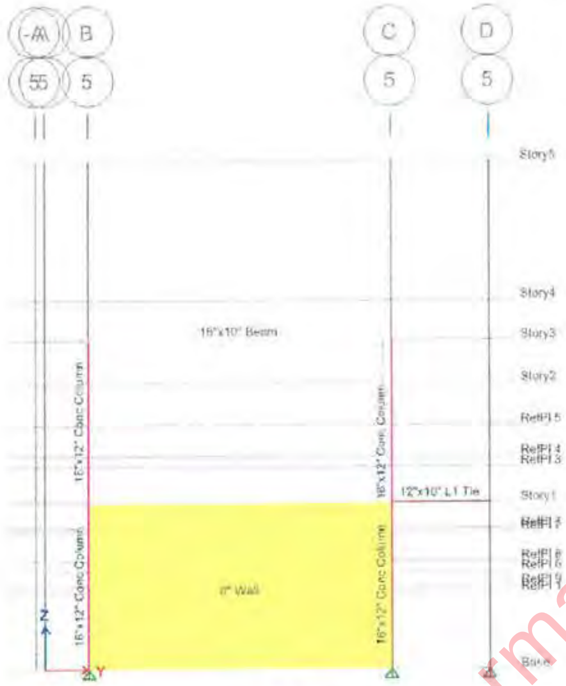
GL3


-AA
B
C
D
44
4
4
4
Story5
Story4
Story3
Story2
RefPl 5
RefPl 4
RefPl 3
Story1
18"x10" Beam
12"x10" Tie
26"x10" Beam
Fin Columns
16"x12" Conc Column
Base
Z
Y

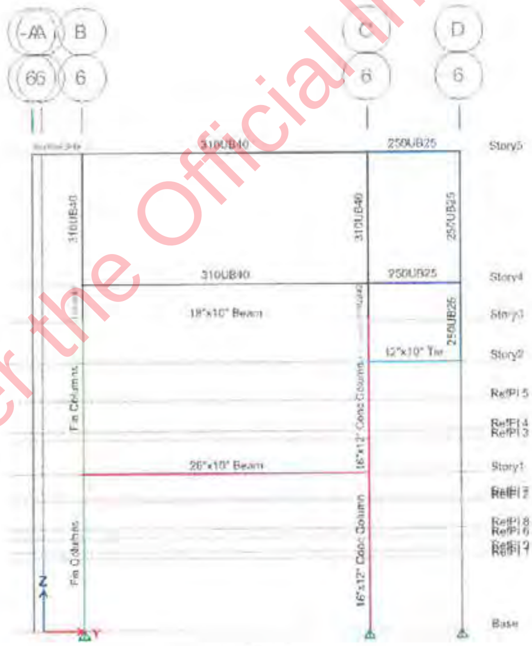
GL4

OPUS

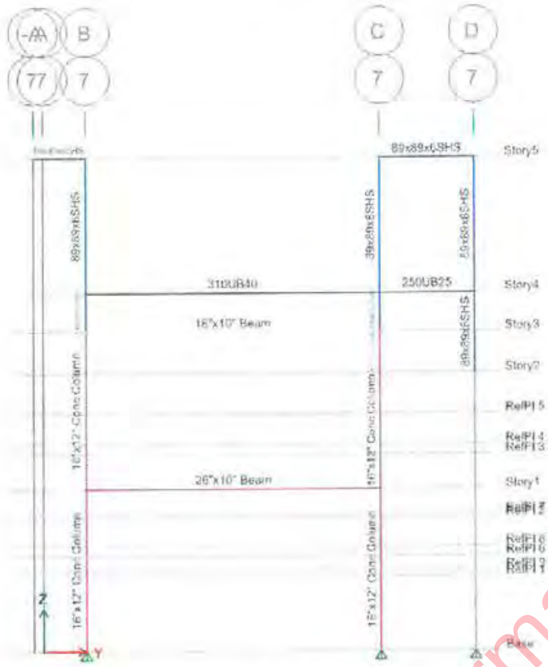
M/07



GL5

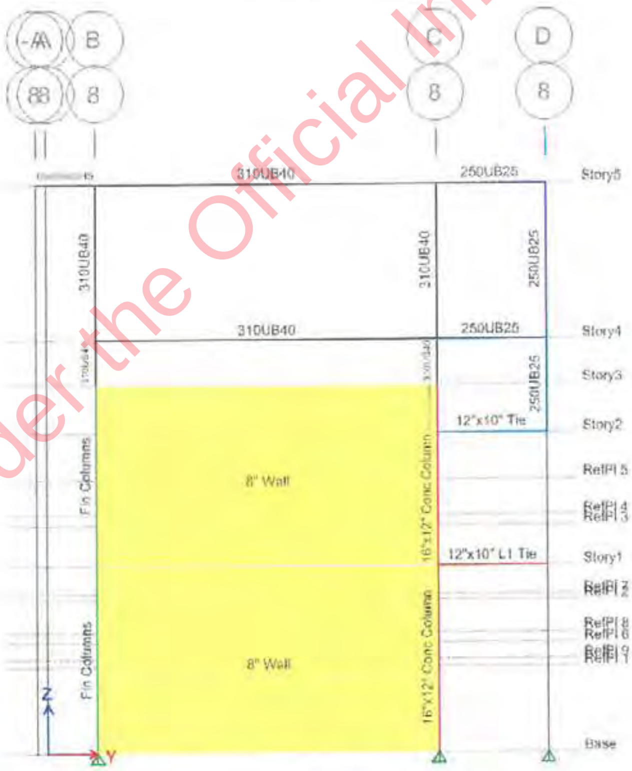


GL6



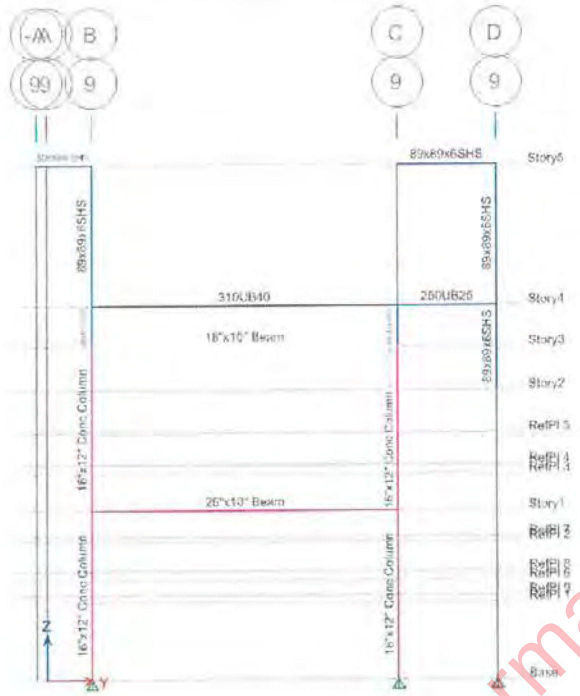
-AA
B
C
D
77
7
7
7
89x89x6SHS
Story5
89x89x6SHS
89x89x6SHS
89x89x6SHS
310UB40
250UB25
Story4
16"x10" Beam
Story3
89x89x6SHS
Story2
16"x12" Conc Column
16"x12" Conc Column
RefPl 5
RefPl 4
RefPl 3
26"x10" Beam
Story1
16"x12" Conc Column
16"x12" Conc Column
Base

GL7



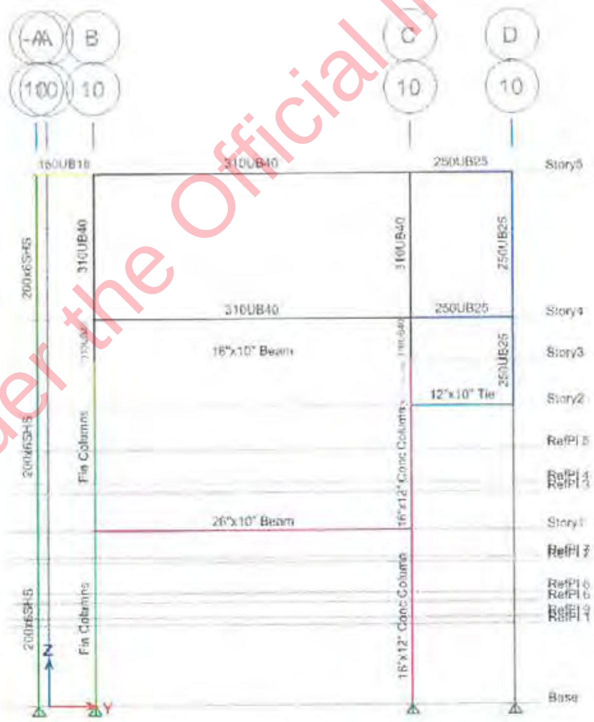
-AA
B
C
D
88
8
8
8
310UB40
250UB25
Story5
310UB40
310UB40
250UB25
310UB40
250UB25
Story4
Story3
250UB25
12"x10" Tie
Story2
Fin Columns
8" Wall
16"x12" Conc Column
RefPl 5
12"x10" L1 Tie
Story1
Fin Columns
8" Wall
16"x12" Conc Column
Base

GL8



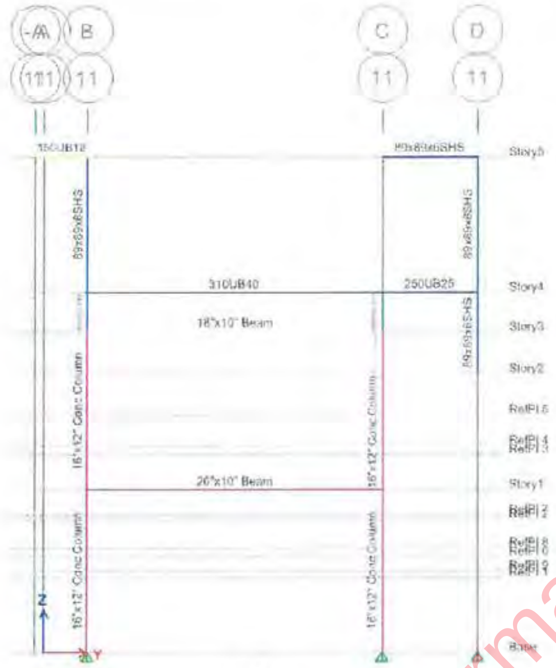
-AA
B
C
D
99
9
89x89x6SHS
Story5
310UB40
250UB25
Story4
18"x10" Beam
Story3
Story2
16"x12" Conc Column
RefPl 5
25"x10" Beam
Story1
Base

GL9

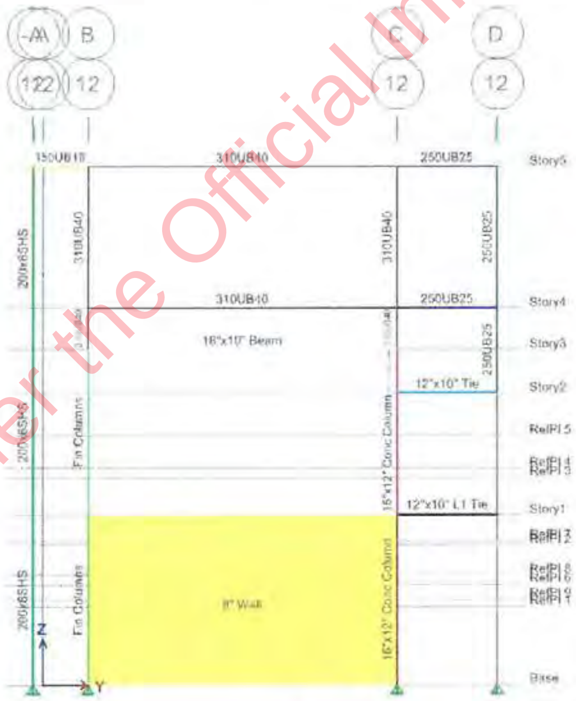


-AA
B
C
D
100
10
160UB18
310UB40
250UB25
Story5
200x6SHS
Story4
16"x10" Beam
Story3
12"x10" Tie
Story2
Fix Columns
16"x12" Conc Column
RefPl 5
26"x10" Beam
Story1
Base

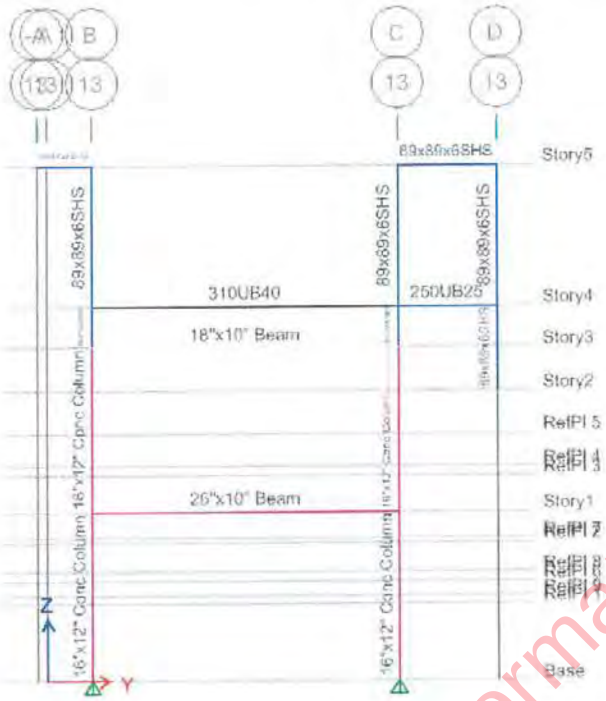
GL10


-AA
B
C
D
111
11
11
11
150UB12
89x89x6SHS
310UB40
250UB25
18"x10" Beam
16"x12" Conc Column
26"x10" Beam
Story5
Story4
Story3
Story2
Story1
Base

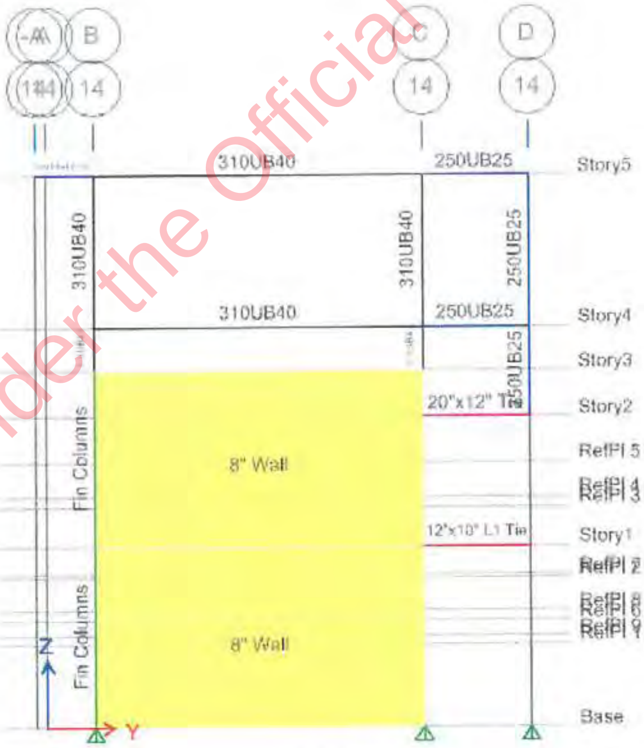
GL11


-AA
B
C
D
122
12
12
12
150UB18
310UB40
250UB25
200x6SHS
18"x10" Beam
12"x10" Tie
12"x10" L1 Tie
16"x12" Conc Column
Fin Columns
8" Wall
Story5
Story4
Story3
Story2
Story1
Base

GL12

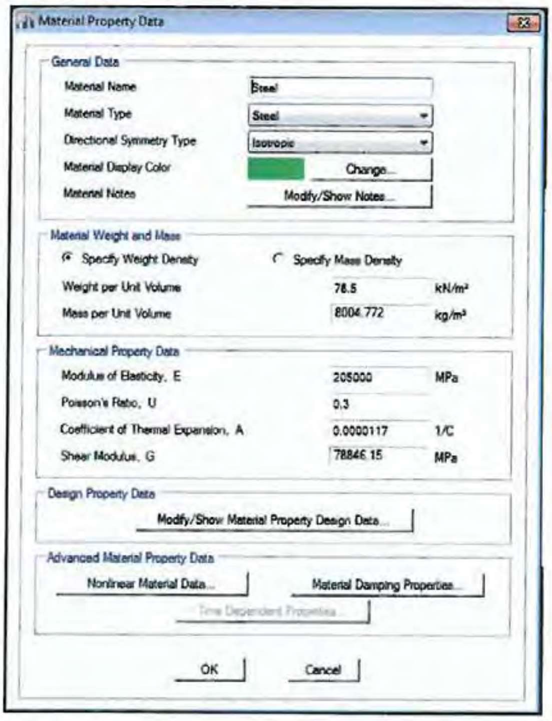

A
B
13
13
C
13
D
13
89x89x6SHS
Story5
89x89x6SHS
89x89x6SHS
89x89x6SHS
310UB40
250UB25
Story4
18"x10" Beam
Story3
Story2
RefPl 5
16"x12" Conc Column 16"x12" Conc Column
16"x12" Conc Column
26"x10" Beam
Story1
Base
Z
Y

GL13

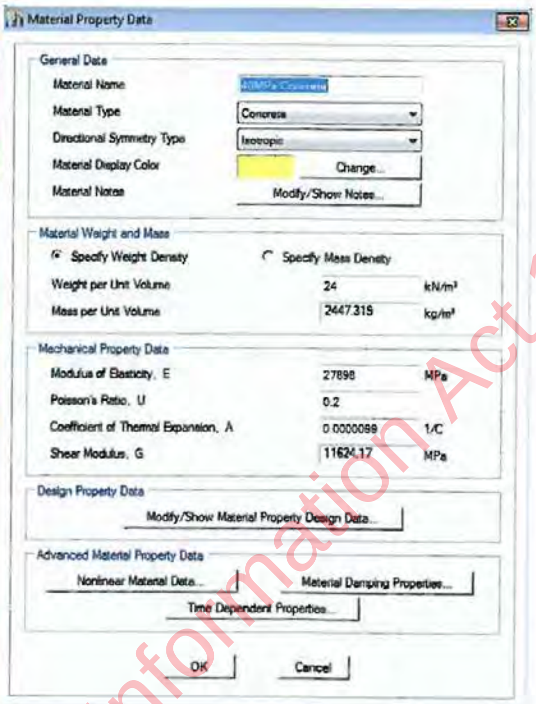

A
B
14
14
C
14
D
14
310UB40
250UB25
Story5
310UB40
310UB40
250UB25
310UB40
250UB25
Story4
Story3
250UB25
20"x12" Tie
Story2
RefPl 5
Fin Columns
8" Wall
12"x10" L1 Tie
Story1
Fin Columns
8" Wall
Base
Z
Y

GL14

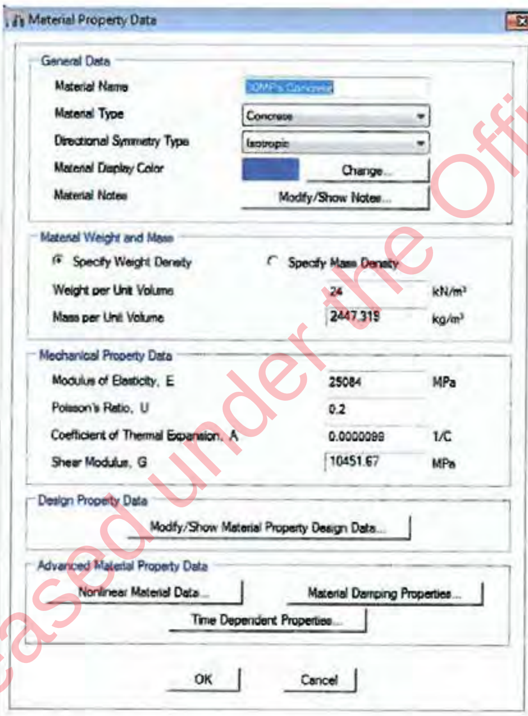
M/12



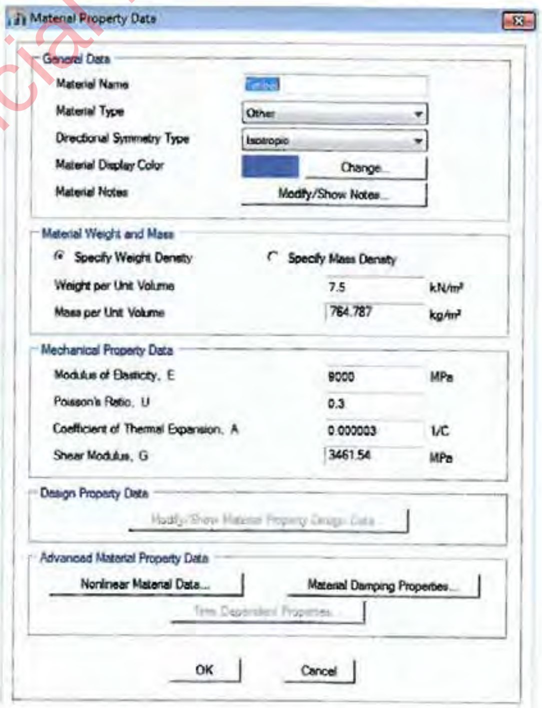
Material Property Data
General Data
Material Name
Steel
Material Type
Steel
Directional Symmetry Type
Isotropic
Material Display Color
Change...
Material Notes
Modify/Show Notes...
Material Weight and Mass
Specify Weight Density
Specify Mass Density
Weight per Unit Volume
78.5
kN/m³
Mass per Unit Volume
8004.772
kg/m³
Mechanical Property Data
Modulus of Elasticity, E
205000
MPa
Poisson's Ratio, U
0.3
Coefficient of Thermal Expansion, A
0.0000117
1/C
Shear Modulus, G
78846.15
MPa
Design Property Data
Modify/Show Material Property Design Data...
Advanced Material Property Data
Nonlinear Material Data...
Material Damping Properties...
Time Dependent Properties...
OK
Cancel



Material Property Data
General Data
Material Name
Material Type
Concrete
Directional Symmetry Type
Isotropic
Material Display Color
Change...
Material Notes
Modify/Show Notes...
Material Weight and Mass
Specify Weight Density
Specify Mass Density
Weight per Unit Volume
24
kN/m³
Mass per Unit Volume
2447.319
kg/m³
Mechanical Property Data
Modulus of Elasticity, E
27898
MPa
Poisson's Ratio, U
0.2
Coefficient of Thermal Expansion, A
0.0000099
1/C
Shear Modulus, G
11624.17
MPa
Design Property Data
Modify/Show Material Property Design Data...
Advanced Material Property Data
Nonlinear Material Data...
Material Damping Properties...
Time Dependent Properties...
OK
Cancel



Material Property Data
General Data
Material Name
Material Type
Concrete
Directional Symmetry Type
Isotropic
Material Display Color
Change...
Material Notes
Modify/Show Notes...
Material Weight and Mass
Specify Weight Density
Specify Mass Density
Weight per Unit Volume
24
kN/m³
Mass per Unit Volume
2447.319
kg/m³
Mechanical Property Data
Modulus of Elasticity, E
25084
MPa
Poisson's Ratio, U
0.2
Coefficient of Thermal Expansion, A
0.0000099
1/C
Shear Modulus, G
10451.67
MPa
Design Property Data
Modify/Show Material Property Design Data...
Advanced Material Property Data
Nonlinear Material Data...
Material Damping Properties...
Time Dependent Properties...
OK
Cancel



Material Property Data
General Data
Material Name
Material Type
Other
Directional Symmetry Type
Isotropic
Material Display Color
Change...
Material Notes
Modify/Show Notes...
Material Weight and Mass
Specify Weight Density
Specify Mass Density
Weight per Unit Volume
7.5
kN/m³
Mass per Unit Volume
764.787
kg/m³
Mechanical Property Data
Modulus of Elasticity, E
8000
MPa
Poisson's Ratio, U
0.3
Coefficient of Thermal Expansion, A
0.000003
1/C
Shear Modulus, G
3461.54
MPa
Design Property Data
Advanced Material Property Data
Nonlinear Material Data...
Material Damping Properties...
OK
Cancel

CALCULATION SHEET

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MODELLING PARAMETERS / MODIFIERS

16" × 12" COLUMNS

Load = 300 kN → $\frac{N^*}{A_g f_c'} = 0.08$

Interpolating : $0.55\ I_g = 0.2$

$0.4\ I_g = 0.0$

∴ $0.46\ I_g$ → Use 0.45

FIN COLUMNS

Area = 354 302 mm² → $\frac{N^*}{A_g f_c'} = 0.03$

∴ $0.42\ I_g$ → Use 0.42

Modifying to an equivalent rectangle:

A_g : 354 302

$I_{xx} = 4.9642 \times 10^{10}\ mm^4$

$I_{yy} = 2.224 \times 10^{9}\ mm^4$

If 1294 × 274 → $A_g = 354\,556$ = Within 1%

$I_{xx} = 4.947 \times 10^{10}$ = ..

$I_{yy} = 2.218 \times 10^{9}$ = ..

∴ 1294 × 274 ≡ FIN COLUMN

FINS COLUMNS INSERTION POINT:

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MODELLING OF FLOOR SLAB AND BEAMS

Alter mass of beam based on % :

For 26" x 10" Beam = 660 x 254

127mm x 254 = 32252 mm² = 19%

↳ Mass mod & Weight = 0.81 = B1

For 12" x 10" Beam = 305 x 254 =

127 x 254 = 42%

∴ Mass & Weight mod : 0.58

For Spandrel Beam = 1400 x 152

127 x 152 = 9%

∴ Mass & weight mod = 0.91

For 30" x 8" Beam = 762 x 203 =

127 x 203 = 17% → Mod = 0.83

For 18" x 16" Beam at roof - 2.75" Slab

= 15% → 0.85

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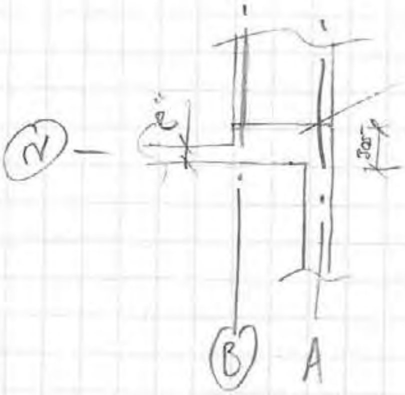
SLABS

Modifiers : Treat diaphragms as walls with N=0

∴ $I_e = 0.32 I_g$

GL 2 : B Corner



Make wall 305 Thick : 12"

Wall will need to act out-of-plane. Alter stiffness for m_{11} & m_{22} to 0.4 (like beam).

Thin or Thick shell ? : Span = ~4 m
Thickness = 305 mm

Use Thick Shell - ETABS guidance.

OTHER WALLS

For out-of-plane, don't want them to attract much load! - m_{11} & m_{22} = 0.2

For 8" wall on GL 2, use m_{11} & m_{22} = 0.4

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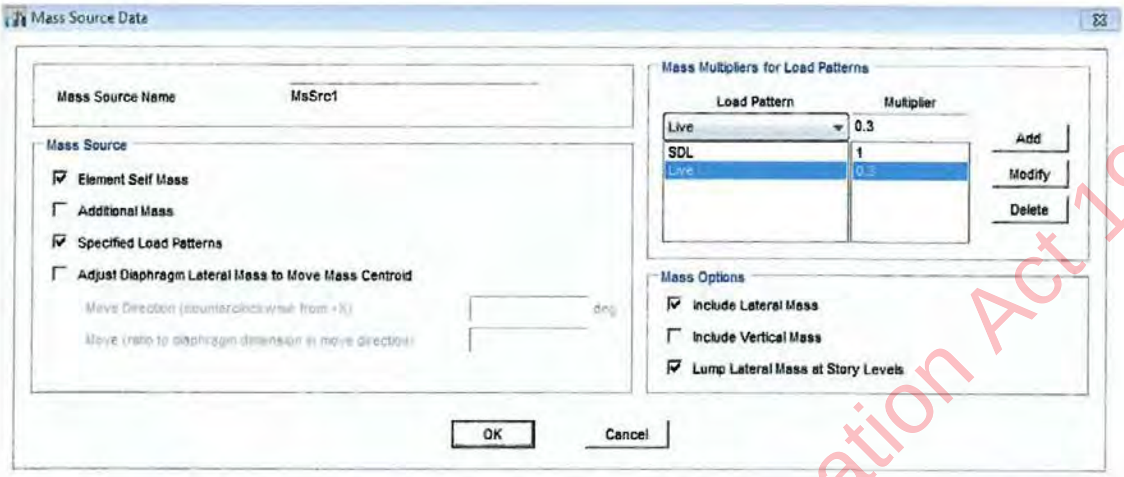
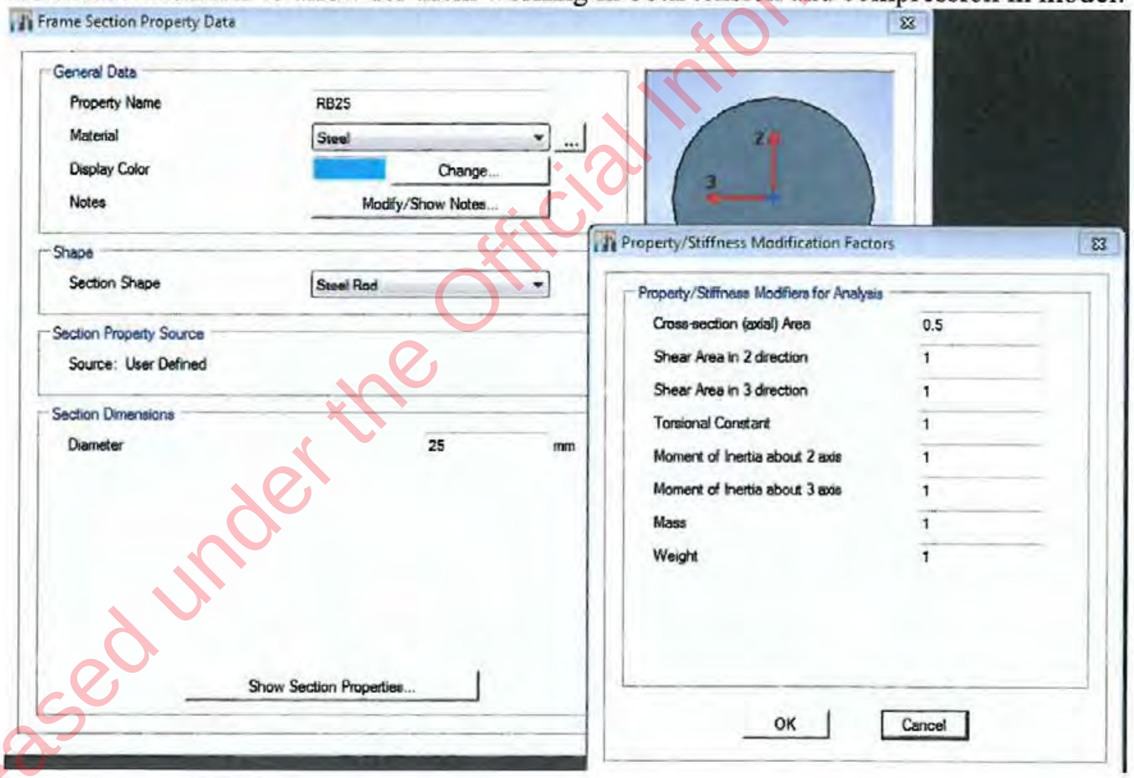
Mass Source:



Reidbrace Modifier to allow for them working in both tension and compression in model:



↳ See M/19

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BRACES

Tension - only RB25 used for top storey. Model can't read as tension-only, so reduce stiffness by 50% to account for them working both in tension & compression in model.

→ Axial cross-sectional area reduced to 0.5.

FLOOR SYSTEM

Floor = 'Posistrut' timber truss joist system

Treat as flexible diaphragm

For modelling purposes, add nominal steel beams between columns ~ Eg. 150UB18 to tie columns & add 11mm thick Ply.

ROOF
APPLICATION OF LOADS

Add roof loads as UDLs to beams.

TOP ROOF:

$DL = \underset{\text{cladding}}{0.1} + \underset{\text{secondary steel}}{0.1} + \underset{\text{ceiling/services}}{0.25} = 0.45\ kPa$

$LL = 0$

For End beams $= 4.038 \times 0.45 = 1.82\ kN/m$

For Int. beams $= 2 \times 4.038 \times 0.45 = 3.63\ kN/m$

LOW ROOF

Beams @ 4.038m generally → 1.82 kN/m

For 2.076m spacing = 3.63 kN/m

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CSi KNOWLEDGE BASE

Thin vs. Thick shells

Added by Mike Abell, last edited by Jessica Napier on Apr 07, 2014

What is the difference between thin and thick shell formulations?

Answer: The inclusion of transverse shear deformation in plate-bending behavior is the main difference between thin and thick shell formulation. Thin-plate formulation follows a Kirchhoff application, which neglects transverse shear deformation, whereas thick-plate formulation follows Mindlin/Reissner, which does account for shear behavior. Thick-plate formulation has no effect upon membrane (in-plane) behavior, only plate-bending (out-of-plane) behavior.

Shear deformation tends to be important when shell thickness is greater than approximately $^1/_5$ to $^1/_{10}$ of the span of plate-bending curvature. Shearing may also become significant in locations of bending-stress concentrations, which occur near sudden changes in thickness or support conditions, and near openings or re-entrant corners. Thick-plate formulation is best for such applications.

Thick-plate formulation is also recommended in general because it tends to be more accurate, though slightly stiffer, even for thin-plate bending problems in which shear deformation is truly negligible. However, the accuracy of thick-plate formulation is sensitive to mesh distortion and large aspect ratios, and therefore should not be used in such cases when shear deformation is known to be small.

In general, the contribution of shear deformation becomes significant when ratio between the span of plate-bending curvature and thickness is approximately 20:1 or 10:1. The formulation itself is adequate for ratio down to 5:1 or 4:1. In that this ratio is dependent upon the projected span of curvature, shell thickness may be greater than the actual plan dimensions of a shell object.

Stiffness for pure-bending deformation

The statement that thick shells tend to be stiffer than thin shells applies only to the bending components of shells, and to models in which meshing is too coarse.

When meshing adequately captures bending deformation, thick-shell elements are more flexible because of the additional shear deformation that is not captured through thin-shell formulation. Given pure-bending deformation, however, the thin-shell element is slightly more accurate, therefore the thick-shell element may be stiffer for coarser meshes. This effect diminishes as the mesh is refined.

Stresses may be of greater concern than deflections. When shear deformation is expected to be important, we recommend the thick-shell element because it will better capture the stress distribution. This is the case not only for thicker shells, but also for regions near openings and other geometric discontinuities in which transverse shear deformation develops.

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LOADCASES

Both Equivalent Static & Modal Response analyses carried out.

EQIVALENT STATIC

Method acceptable - meets all the requirements of NZS1170.5 Section 6.1.3 - see next pages.

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SECTION 6 STRUCTURAL ANALYSIS

6.1 GENERAL

6.1.1 Methods of analysis

A structural analysis to determine the action effects shall be carried out in accordance with one of the following providing the limitations of Clause 6.1.3 are complied with:

(a) A method based on equivalent static forces as outlined in Clause 6.2; or

(b) The modal response spectrum method as outlined in Clause 6.3;

(c) The numerical time history method as outlined Clause 6.4.

6.1.2 P-delta analysis

P-delta effects shall be considered in accordance with Clause 6.5 in analyses of design actions and deflections for the ultimate limit state.

6.1.3 Limitations on the use of methods of analysis

6.1.3.1 *Equivalent static method*

The equivalent static method of analysis shall be used only when at least one of the following criteria is satisfied:

(a) The height between the base and the top of the structure is less than 10 m; or

(b) The largest translational period calculated as specified in Clause 4.1.2 is less than 0.4 s; or

(c) The structure is not classified as irregular under Clause 4.5 and the largest translational period is less than 2.0 seconds.

6.1.3.2 *Modal response spectrum method*

The modal response spectrum method may be used on all structures that fall within the scope of this Standard provided that three-dimensional analyses shall be used when the structure is classified as torsionally sensitive under Clause 4.5.2.3.

6.1.3.3 *Numerical integration time history analyses*

Numerical integration time history analyses may be used on all structures that fall within the scope of this Standard to verify that specific response parameters are within the limits of acceptability assumed during design. Three-dimensional time history analyses shall be used where the structure is classified as torsionally sensitive under Clause 4.5.2.3.

6.1.4 Diaphragm response

6.1.4.1 *Requirement for modelling*

For structures over 15 m in height where the structure is classified as irregular under provisions of Clause 4.5, diaphragms shall be modelled in a three-dimensional modal response spectrum or three-dimensional numerical integration time history analysis. Where diaphragms are not rigid compared to the vertical elements of the vertical action resisting system, the model should include representation of the diaphragm's flexibility.

Elastic diaphragms shall be used in structures and modelled as such.

Inelastic deformations associated with in-plane diaphragm actions resulting from earthquake induced forces shall only be permitted to occur when justified by rational analysis which has been substantiated by experimental data.

∴ Equivalent static method acceptable to use.

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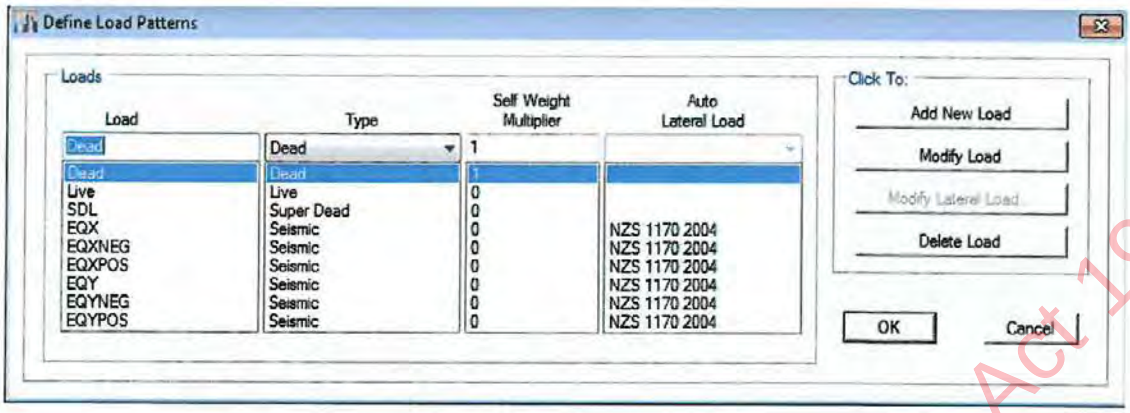
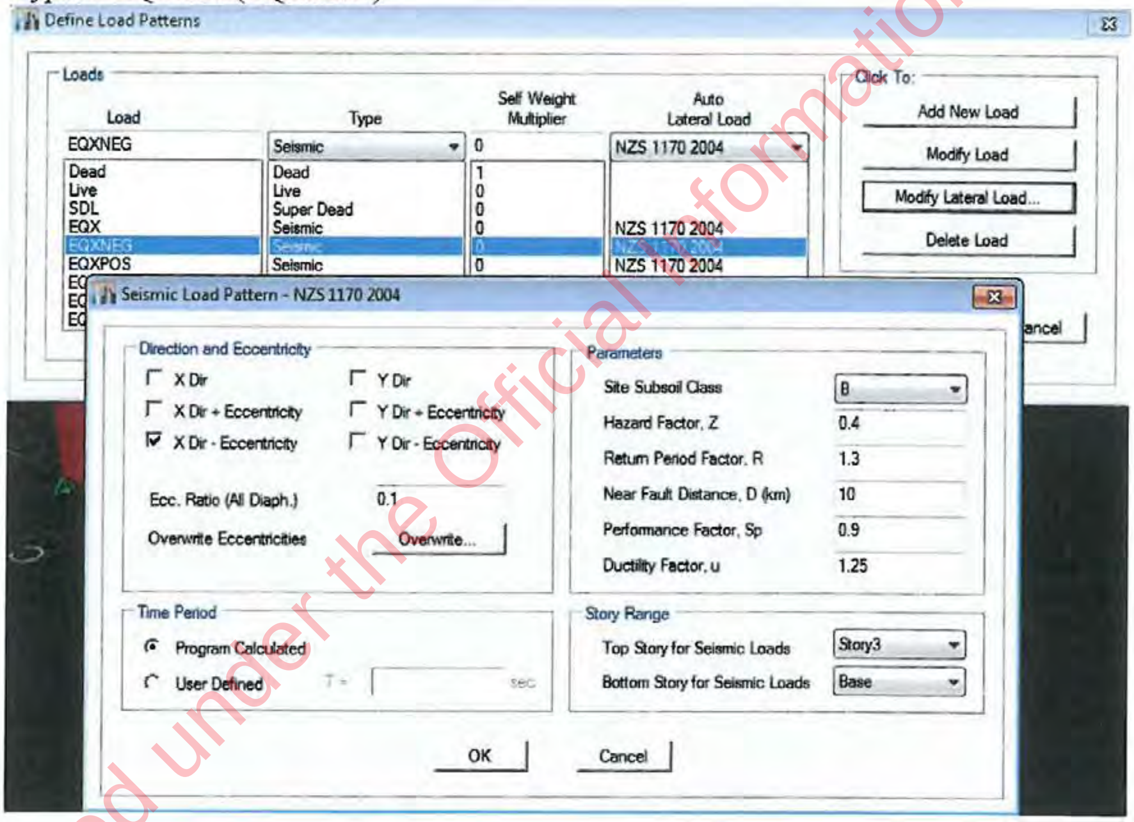


Typical EQ Load (EQXNEG)



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4 Loads

This chapter provides loading information as applied to the model.

4.1 Load Patterns

Table 4.1 - Load Patterns

Name	Type	Self Weight Multiplier	Auto Load
Dead	Dead	1	
Live	Live	0	
SDL	Superimposed Dead	0	
EQX	Seismic	0	NZS 1170 2004
EQXNEG	Seismic	0	NZS 1170 2004
EQXPOS	Seismic	0	NZS 1170 2004
EQY	Seismic	0	NZS 1170 2004
EQYNEG	Seismic	0	NZS 1170 2004
EQYPOS	Seismic	0	NZS 1170 2004

4.2 Auto Seismic Loading

EQUIVALENT STATIC LOAD COMBINATIONS

Loadcases

1	Long	+	0.3 Trans
2	Long	+	0.3 Trans+
3	Long	+	0.3 Trans-
4	Long	-	0.3 Trans
5	Long	-	0.3 Trans+
6	Long	-	0.3 Trans-
7	Long+	+	0.3 Trans
8	Long+	+	0.3 Trans+
9	Long+	+	0.3 Trans-
10	Long+	-	0.3 Trans
11	Long+	-	0.3 Trans+
12	Long+	-	0.3 Trans-
13	Long-	+	0.3 Trans
14	Long-	+	0.3 Trans+
15	Long-	+	0.3 Trans-
16	Long-	-	0.3 Trans
17	Long-	-	0.3 Trans+
18	Long-	-	0.3 Trans-
19	Trans	+	0.3 Long
20	Trans	+	0.3 Long+
21	Trans	+	0.3 Long-
22	Trans	-	0.3 Long
23	Trans	-	0.3 Long+
24	Trans	-	0.3 Long-
25	Trans+	+	0.3 Long
26	Trans+	+	0.3 Long+
27	Trans+	+	0.3 Long-
28	Trans+	-	0.3 Long
29	Trans+	-	0.3 Long+
30	Trans+	-	0.3 Long-
31	Trans-	+	0.3 Long
32	Trans-	+	0.3 Long+
33	Trans-	+	0.3 Long-
34	Trans-	-	0.3 Long
35	Trans-	-	0.3 Long+
36	Trans-	-	0.3 Long-

Actually there are another 36 possible combinations:
-Long +0.3 Trans etc. making 72 in total.
However, not neccessary to input all these since all that will be different is the sign (direction) rather than the magnitude.
∴ 36 combinations adequate.

Table 4.9 - Shell Loads - Uniform

Story	Label	Unique Name	Load Pattern	Direction	Load kN/m²
Story4	F3	7	Live	Gravity	3
Story1	F2	110	Live	Gravity	3
Story1	F4	121	Live	Gravity	3
Story1	F5	112	Live	Gravity	3
Story4	F3	7	SDL	Gravity	0.8
Story1	F2	110	SDL	Gravity	0.8
Story1	F4	121	SDL	Gravity	0.3
Story1	F5	112	SDL	Gravity	0.8

4.4 Load Cases

Table 4.10 - Load Cases - Summary

Name	Type
Dead	Linear Static
Live	Linear Static
SDL	Linear Static
EQX	Linear Static
EQXNEG	Linear Static
EQXPOS	Linear Static
EQY	Linear Static
EQYNEG	Linear Static
EQYPOS	Linear Static

4.5 Load Combinations

Table 4.11 - Load Combinations

Name	Load Case/Combo	Scale Factor	Type	Auto
DL+SDL+0.3LL	Dead	1	Linear Add	No
DL+SDL+0.3LL	SDL	1		No
DL+SDL+0.3LL	Live	0.3		No
1.2DL+1.5LL	Dead	1.2	Linear Add	No
1.2DL+1.5LL	SDL	1.2		No
1.2DL+1.5LL	Live	1.5		No
1.DSL+EQX+0.3EQY	DL+SDL+0.3LL	1	Linear Add	No
1.DSL+EQX+0.3EQY	EQX	1		No
1.DSL+EQX+0.3EQY	EQY	0.3		No
2.DSL+EQX+0.3EQY+	DL+SDL+0.3LL	1	Linear Add	No
2.DSL+EQX+0.3EQY+	EQX	1		No
2.DSL+EQX+0.3EQY+	EQYPOS	0.3		No
3.DSL+EQX+0.3EQY-	DL+SDL+0.3LL	1	Linear Add	No
3.DSL+EQX+0.3EQY-	EQX	1		No
3.DSL+EQX+0.3EQY-	EQYNEG	0.3		No
4.DSL+EQX-0.3EQY	DL+SDL+0.3LL	1	Linear Add	No
4.DSL+EQX-0.3EQY	EQX	1		No
4.DSL+EQX-0.3EQY	EQY	-0.3		No
5.DSL+EQX-0.3EQY+	DL+SDL+0.3LL	1	Linear Add	No
5.DSL+EQX-0.3EQY+	EQX	1		No
5.DSL+EQX-0.3EQY+	EQYPOS	-0.3		No
6.DSL+EQX-0.3EQY-	DL+SDL+0.3LL	1	Linear Add	No

Table 4.11 - Load Combinations (continued)

Name	Load Case/Combo	Scale Factor	Type	Auto
6.DSL+EQX-0.3EQY-	EQX	1		No
6.DSL+EQX-0.3EQY-	EQYNEG	-0.3		No
7.DSL+EQX++0.3EQY	DL+SDL+0.3LL	1	Linear Add	No
7.DSL+EQX++0.3EQY	EQXPOS	1		No
7.DSL+EQX++0.3EQY	EQY	0.3		No
8.DSL+EQX++0.3EQY+	DL+SDL+0.3LL	1	Linear Add	No
8.DSL+EQX++0.3EQY+	EQXPOS	1		No
8.DSL+EQX++0.3EQY+	EQYPOS	0.3		No
9.DSL+EQX++0.3EQY-	DL+SDL+0.3LL	1	Linear Add	No
9.DSL+EQX++0.3EQY-	EQXPOS	1		No
9.DSL+EQX++0.3EQY-	EQYNEG	0.3		No
10.DSL+EQX+-0.3EQY	DL+SDL+0.3LL	1	Linear Add	No
10.DSL+EQX+-0.3EQY	EQXPOS	1		No
10.DSL+EQX+-0.3EQY	EQY	-0.3		No
12.DSL+EQX+-0.3EQY-	DL+SDL+0.3LL	1	Linear Add	No
12.DSL+EQX+-0.3EQY-	EQXPOS	1		No
12.DSL+EQX+-0.3EQY-	EQYNEG	-0.3		No
11.DSL+EQX+-0.3EQY+	DL+SDL+0.3LL	1	Linear Add	No
11.DSL+EQX+-0.3EQY+	EQXPOS	1		No
11.DSL+EQX+-0.3EQY+	EQYPOS	-0.3		No
13.DSL+EQX-+0.3EQY	DL+SDL+0.3LL	1	Linear Add	No
13.DSL+EQX-+0.3EQY	EQXNEG	1		No
13.DSL+EQX-+0.3EQY	EQY	0.3		No
15.DSL+EQX-+0.3EQY-	DL+SDL+0.3LL	1	Linear Add	No
15.DSL+EQX-+0.3EQY-	EQXNEG	1		No
15.DSL+EQX-+0.3EQY-	EQYNEG	0.3		No
14.DSL+EQX-+0.3EQY+	DL+SDL+0.3LL	1	Linear Add	No
14.DSL+EQX-+0.3EQY+	EQXNEG	1		No
14.DSL+EQX-+0.3EQY+	EQYPOS	0.3		No
16.DSL+EQX--0.3EQY	DL+SDL+0.3LL	1	Linear Add	No
16.DSL+EQX--0.3EQY	EQXNEG	1		No
16.DSL+EQX--0.3EQY	EQY	-0.3		No
17.DSL+EQX--0.3EQY+	DL+SDL+0.3LL	1	Linear Add	No
17.DSL+EQX--0.3EQY+	EQXNEG	1		No
17.DSL+EQX--0.3EQY+	EQYPOS	-0.3		No
18.DSL+EQX--0.3EQY-	DL+SDL+0.3LL	1	Linear Add	No
18.DSL+EQX--0.3EQY-	EQXNEG	1		No
18.DSL+EQX--0.3EQY-	EQYNEG	-0.3		No
19.DSL+EQY+0.3EQX	DL+SDL+0.3LL	1	Linear Add	No
19.DSL+EQY+0.3EQX	EQY	1		No
19.DSL+EQY+0.3EQX	EQX	0.3		No
20.DSL+EQY+0.3EQX+	DL+SDL+0.3LL	1	Linear Add	No
20.DSL+EQY+0.3EQX+	EQY	1		No
20.DSL+EQY+0.3EQX+	EQXPOS	0.3		No
21.DSL+EQY+0.3EQX-	DL+SDL+0.3LL	1	Linear Add	No
21.DSL+EQY+0.3EQX-	EQY	1		No
21.DSL+EQY+0.3EQX-	EQXNEG	0.3		No
22.DSL+EQY-0.3EQX	DL+SDL+0.3LL	1	Linear Add	No
22.DSL+EQY-0.3EQX	EQY	1		No

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Table 4.11 - Load Combinations (continued)

Name	Load Case/Combo	Scale Factor	Type	Auto
22.DSL+EQY-0.3EQX	EQX	-0.3		No
23.DSL+EQY-0.3EQX+	DL+SDL+0.3LL	1	Linear Add	No
23.DSL+EQY-0.3EQX+	EQY	1		No
23.DSL+EQY-0.3EQX+	EQXPOS	-0.3		No
24.DSL+EQY-0.3EQX-	DL+SDL+0.3LL	1	Linear Add	No
24.DSL+EQY-0.3EQX-	EQY	1		No
24.DSL+EQY-0.3EQX-	EQXNEG	-0.3		No
25.DSL+EQY++0.3EQX	DL+SDL+0.3LL	1	Linear Add	No
25.DSL+EQY++0.3EQX	EQYPOS	1		No
25.DSL+EQY++0.3EQX	EQX	0.3		No
26.DSL+EQY++0.3EQX+	DL+SDL+0.3LL	1	Linear Add	No
26.DSL+EQY++0.3EQX+	EQYPOS	1		No
26.DSL+EQY++0.3EQX+	EQXPOS	0.3		No
27.DSL+EQY++0.3EQX-	DL+SDL+0.3LL	1	Linear Add	No
27.DSL+EQY++0.3EQX-	EQYPOS	1		No
27.DSL+EQY++0.3EQX-	EQXNEG	0.3		No
28.DSL+EQY+-0.3EQX	DL+SDL+0.3LL	1	Linear Add	No
28.DSL+EQY+-0.3EQX	EQYPOS	1		No
28.DSL+EQY+-0.3EQX	EQX	-0.3		No
29.DSL+EQY+-0.3EQX+	DL+SDL+0.3LL	1	Linear Add	No
29.DSL+EQY+-0.3EQX+	EQYPOS	1		No
29.DSL+EQY+-0.3EQX+	EQXPOS	-0.3		No
30.DSL+EQY+-0.3EQX-	DL+SDL+0.3LL	1	Linear Add	No
30.DSL+EQY+-0.3EQX-	EQYPOS	1		No
30.DSL+EQY+-0.3EQX-	EQXNEG	-0.3		No
31.DSL+EQY-+0.3EQX	DL+SDL+0.3LL	1	Linear Add	No
31.DSL+EQY-+0.3EQX	EQYNEG	1		No
31.DSL+EQY-+0.3EQX	EQX	0.3		No
32.DSL+EQY-+0.3EQX+	DL+SDL+0.3LL	1	Linear Add	No
32.DSL+EQY-+0.3EQX+	EQYNEG	1		No
32.DSL+EQY-+0.3EQX+	EQXPOS	0.3		No
33.DSL+EQY-+0.3EQX-	DL+SDL+0.3LL	1	Linear Add	No
33.DSL+EQY-+0.3EQX-	EQYNEG	1		No
33.DSL+EQY-+0.3EQX-	EQXNEG	0.3		No
34.DSL+EQY–0.3EQX	DL+SDL+0.3LL	1	Linear Add	No
34.DSL+EQY–0.3EQX	EQYNEG	1		No
34.DSL+EQY–0.3EQX	EQX	-0.3		No
35.DSL+EQY–0.3EQX+	DL+SDL+0.3LL	1	Linear Add	No
35.DSL+EQY–0.3EQX+	EQYNEG	1		No
35.DSL+EQY–0.3EQX+	EQXPOS	-0.3		No

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Table 4.11 - Load Combinations (continued)

Name	Load Case/Combo	Scale Factor	Type	Auto
EQENV	7.DSL+EQX++0.3EQY	1		No
EQENV	6.DSL+EQX++0.3EQY+	1		No
EQENV	9.DSL+EQX++0.3EQY-	1		No
EQENV	10.DSL+EQX+-0.3EQY	1		No
EQENV	11.DSL+EQX+-0.3EQY+	1		No
EQENV	12.DSL+EQX+-0.3EQY-	1		No
EQENV	13.DSL+EQX-+0.3EQY	1		No
EQENV	14.DSL+EQX-+0.3EQY+	1		No
EQENV	15.DSL+EQX-+0.3EQY-	1		No
EQENV	16.DSL+EQX--0.3EQY	1		No
EQENV	17.DSL+EQX--0.3EQY+	1		No
EQENV	18.DSL+EQX--0.3EQY-	1		No
EQENV	19.DSL+EQY+0.3EQX	1		No
EQENV	20.DSL+EQY+0.3EQX+	1		No
EQENV	21.DSL+EQY+0.3EQX-	1		No
EQENV	22.DSL+EQY-0.3EQX	1		No
EQENV	23.DSL+EQY-0.3EQX+	1		No
EQENV	24.DSL+EQY-0.3EQX-	1		No
EQENV	25.DSL+EQY++0.3EQX	1		No
EQENV	26.DSL+EQY++0.3EQX+	1		No
EQENV	27.DSL+EQY++0.3EQX-	1		No
EQENV	28.DSL+EQY+-0.3EQX	1		No
EQENV	29.DSL+EQY+-0.3EQX+	1		No
EQENV	30.DSL+EQY+-0.3EQX-	1		No
EQENV	31.DSL+EQY-+0.3EQX	1		No
EQENV	32.DSL+EQY-+0.3EQX+	1		No
EQENV	33.DSL+EQY-+0.3EQX-	1		No
EQENV	34.DSL+EQY--0.3EQX	1		No
EQENV	35.DSL+EQY--0.3EQX+	1		No
EQENV				

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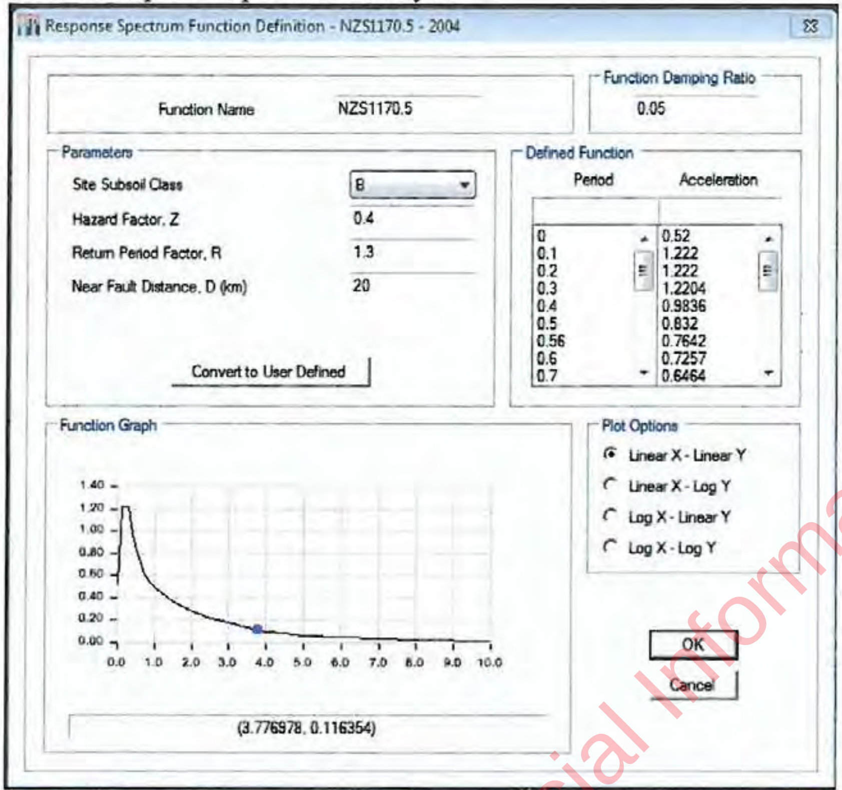
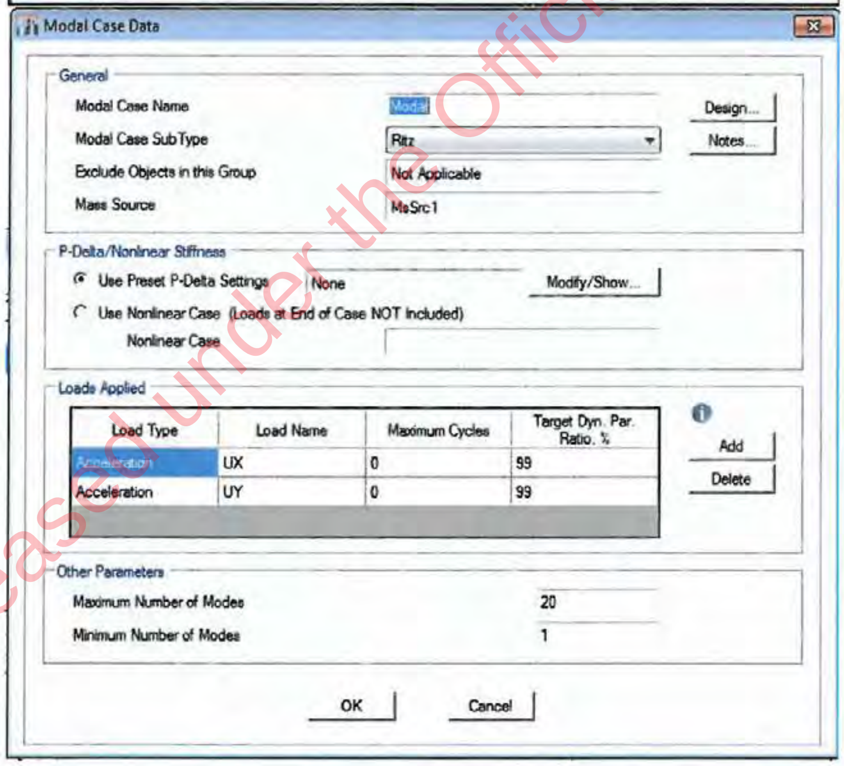
Modal Response Spectrum Analysis





4 Loads

This chapter provides loading information as applied to the model.

4.1 Load Patterns

Table 4.1 - Load Patterns

Name	Type	Self Weight Multiplier	Auto Load
Dead	Dead	1	
Live	Live	0	
SDL	Superimposed Dead	0	
EQX	Seismic	0	NZS 1170 2004
EQXNEG	Seismic	0	NZS 1170 2004
EQXPOS	Seismic	0	NZS 1170 2004
EQY	Seismic	0	NZS 1170 2004
EQYNEG	Seismic	0	NZS 1170 2004
EQYPOS	Seismic	0	NZS 1170 2004

4.2 Auto Seismic Loading

Table 4.9 - Shell Loads - Uniform

Story	Label	Unique Name	Load Pattern	Direction	Load kN/m²
Story4	F3	7	Live	Gravity	3
Story1	F2	110	Live	Gravity	3
Story1	F4	121	Live	Gravity	3
Story1	F5	112	Live	Gravity	3
Story4	F3	7	SDL	Gravity	0.8
Story1	F2	110	SDL	Gravity	0.8
Story1	F4	121	SDL	Gravity	0.3
Story1	F5	112	SDL	Gravity	0.8

4.4 Functions

4.4.1 Response Spectrum Functions

Table 4.10 - Response Spectrum Function - NZS 1170:2004

Name	Period sec	Acceleration	Damping	Site Class	Z	R	D km
NZS1170.5	0	0.52	5	B	0.4	1.3	20
NZS1170.5	0.1	1.222					
NZS1170.5	0.2	1.222					
NZS1170.5	0.3	1.220422					
NZS1170.5	0.4	0.983571					
NZS1170.5	0.5	0.832					
NZS1170.5	0.56	0.764205					
NZS1170.5	0.6	0.725667					
NZS1170.5	0.7	0.646439					
NZS1170.5	0.8	0.584835					
NZS1170.5	0.9	0.535388					
NZS1170.5	1	0.49471					
NZS1170.5	1.5	0.364991					
NZS1170.5	2	0.273					
NZS1170.5	2.5	0.2184					
NZS1170.5	3	0.182					
NZS1170.5	3.5	0.133714					
NZS1170.5	4	0.102375					
NZS1170.5	4.5	0.080889					
NZS1170.5	5	0.06552					
NZS1170.5	6	0.0455					
NZS1170.5	8	0.025594					
NZS1170.5	10	0.01638					

4.5 Load Cases

Table 4.11 - Load Cases - Summary

Name	Type
Dead	Linear Static
Live	Linear Static
SDL	Linear Static
EQX	Linear Static
EQXNEG	Linear Static
EQXPOS	Linear Static
EQY	Linear Static

Table 4.11 - Load Cases - Summary (continued)

Name	Type
EQYNEG	Linear Static
EQYPOS	Linear Static
RSEQX	Response Spectrum
RSEQX-	Response Spectrum
RSEQX+	Response Spectrum
RSEQY	Response Spectrum
RSEQY-	Response Spectrum
RSEQY+	Response Spectrum
~TorsionRSEQX-	Linear Static
~TorsionRSEQX+	Linear Static
~TorsionRSEQY-	Linear Static
~TorsionRSEQY+	Linear Static

4.6 Load Combinations

Table 4.12 - Load Combinations

Name	Load Case/Combo	Scale Factor	Type	Auto
DL+SDL+0.3LL	Dead	1	Linear Add	No
DL+SDL+0.3LL	SDL	1		No
DL+SDL+0.3LL	Live	0.3		No
1.2DL+1.5LL	Dead	1.2	Linear Add	No
1.2DL+1.5LL	SDL	1.2		No
1.2DL+1.5LL	Live	1.5		No
1.DSL+EQX+0.3EQY	DL+SDL+0.3LL	1	Linear Add	No
1.DSL+EQX+0.3EQY	RSEQX	1		No
1.DSL+EQX+0.3EQY	RSEQY	0.3		No
2.DSL+EQX+0.3EQY+	DL+SDL+0.3LL	1	Linear Add	No
2.DSL+EQX+0.3EQY+	RSEQX	1		No
2.DSL+EQX+0.3EQY+	RSEQY+	0.3		No
3.DSL+EQX+0.3EQY-	DL+SDL+0.3LL	1	Linear Add	No
3.DSL+EQX+0.3EQY-	RSEQX	1		No
3.DSL+EQX+0.3EQY-	RSEQY-	0.3		No
4.DSL+EQX-0.3EQY	DL+SDL+0.3LL	1	Linear Add	No
4.DSL+EQX-0.3EQY	RSEQX	1		No
4.DSL+EQX-0.3EQY	RSEQY	-0.3		No
5.DSL+EQX-0.3EQY+	DL+SDL+0.3LL	1	Linear Add	No
5.DSL+EQX-0.3EQY+	RSEQX	1		No
5.DSL+EQX-0.3EQY+	RSEQY+	-0.3		No
6.DSL+EQX-0.3EQY-	DL+SDL+0.3LL	1	Linear Add	No
6.DSL+EQX-0.3EQY-	RSEQX	1		No
6.DSL+EQX-0.3EQY-	RSEQY-	-0.3		No
7.DSL+EQX++0.3EQY	DL+SDL+0.3LL	1	Linear Add	No
7.DSL+EQX++0.3EQY	RSEQX+	1		No
7.DSL+EQX++0.3EQY	RSEQY	0.3		No
8.DSL+EQX++0.3EQY+	DL+SDL+0.3LL	1	Linear Add	No
8.DSL+EQX++0.3EQY+	RSEQX+	1		No
8.DSL+EQX++0.3EQY+	RSEQY	0.3		No
9.DSL+EQX++0.3EQY-	DL+SDL+0.3LL	1	Linear Add	No
9.DSL+EQX++0.3EQY-	RSEQX+	1		No

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Table 4.12 - Load Combinations (continued)

Name	Load Case/Combo	Scale Factor	Type	Auto
9.DSL+EQX++0.3EQY-	RSEQY-	0.3		No
10.DSL+EQX+-0.3EQY	DL+SDL+0.3LL	1	Linear Add	No
10.DSL+EQX+-0.3EQY	RSEQX+	1		No
10.DSL+EQX+-0.3EQY	RSEQY	-0.3		No
12.DSL+EQX+-0.3EQY-	DL+SDL+0.3LL	1	Linear Add	No
12.DSL+EQX+-0.3EQY-	RSEQX+	1		No
12.DSL+EQX+-0.3EQY-	RSEQY-	-0.3		No
11.DSL+EQX+-0.3EQY+	DL+SDL+0.3LL	1	Linear Add	No
11.DSL+EQX+-0.3EQY+	RSEQX+	1		No
11.DSL+EQX+-0.3EQY+	RSEQY+	-0.3		No
13.DSL+EQX-+0.3EQY	DL+SDL+0.3LL	1	Linear Add	No
13.DSL+EQX-+0.3EQY	RSEQX-	1		No
13.DSL+EQX-+0.3EQY	RSEQY	0.3		No
15.DSL+EQX-+0.3EQY-	DL+SDL+0.3LL	1	Linear Add	No
15.DSL+EQX-+0.3EQY-	RSEQX-	1		No
15.DSL+EQX-+0.3EQY-	RSEQY-	0.3		No
14.DSL+EQX-+0.3EQY+	DL+SDL+0.3LL	1	Linear Add	No
14.DSL+EQX-+0.3EQY+	RSEQX-	1		No
14.DSL+EQX-+0.3EQY+	RSEQY+	0.3		No
16.DSL+EQX--0.3EQY	DL+SDL+0.3LL	1	Linear Add	No
16.DSL+EQX--0.3EQY	RSEQX-	1		No
16.DSL+EQX--0.3EQY	RSEQY	-0.3		No
17.DSL+EQX--0.3EQY+	DL+SDL+0.3LL	1	Linear Add	No
17.DSL+EQX--0.3EQY+	RSEQX-	1		No
17.DSL+EQX--0.3EQY+	RSEQY+	-0.3		No
18.DSL+EQX--0.3EQY-	DL+SDL+0.3LL	1	Linear Add	No
18.DSL+EQX--0.3EQY-	RSEQX-	1		No
18.DSL+EQX--0.3EQY-	RSEQY-	-0.3		No
19.DSL+EQY+0.3EQX	DL+SDL+0.3LL	1	Linear Add	No
19.DSL+EQY+0.3EQX	RSEQY	1		No
19.DSL+EQY+0.3EQX	RSEQX	0.3		No
20.DSL+EQY+0.3EQX+	DL+SDL+0.3LL	1	Linear Add	No
20.DSL+EQY+0.3EQX+	RSEQY	1		No
20.DSL+EQY+0.3EQX+	RSEQX+	0.3		No
21.DSL+EQY+0.3EQX-	DL+SDL+0.3LL	1	Linear Add	No
21.DSL+EQY+0.3EQX-	RSEQY	1		No
21.DSL+EQY+0.3EQX-	RSEQX-	0.3		No
22.DSL+EQY-0.3EQX	DL+SDL+0.3LL	1	Linear Add	No
22.DSL+EQY-0.3EQX	RSEQY	1		No
22.DSL+EQY-0.3EQX	RSEQX	-0.3		No
23.DSL+EQY-0.3EQX+	DL+SDL+0.3LL	1	Linear Add	No
23.DSL+EQY-0.3EQX+	RSEQY	1		No
23.DSL+EQY-0.3EQX+	RSEQX+	-0.3		No
24.DSL+EQY-0.3EQX-	DL+SDL+0.3LL	1	Linear Add	No
24.DSL+EQY-0.3EQX-	RSEQY	1		No
24.DSL+EQY-0.3EQX-	RSEQX-	-0.3		No
25.DSL+EQY++0.3EQX	DL+SDL+0.3LL	1	Linear Add	No
25.DSL+EQY++0.3EQX	RSEQY+	1		No
25.DSL+EQY++0.3EQX	RSEQX	0.3		No

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Table 4.12 - Load Combinations (continued)

Name	Load Case/Combo	Scale Factor	Type	Auto
26.DSL+EQY++0.3EQX+	DL+SDL+0.3LL	1	Linear Add	No
26.DSL+EQY++0.3EQX+	RSEQY+	1		No
26.DSL+EQY++0.3EQX+	RSEQX+	0.3		No
27.DSL+EQY++0.3EQX-	DL+SDL+0.3LL	1	Linear Add	No
27.DSL+EQY++0.3EQX-	RSEQY+	1		No
27.DSL+EQY++0.3EQX-	RSEQX-	0.3		No
28.DSL+EQY+-0.3EQX	DL+SDL+0.3LL	1	Linear Add	No
28.DSL+EQY+-0.3EQX	RSEQY+	1		No
28.DSL+EQY+-0.3EQX	RSEQX	-0.3		No
29.DSL+EQY+-0.3EQX+	DL+SDL+0.3LL	1	Linear Add	No
29.DSL+EQY+-0.3EQX+	RSEQY+	1		No
29.DSL+EQY+-0.3EQX+	RSEQX+	-0.3		No
30.DSL+EQY+-0.3EQX-	DL+SDL+0.3LL	1	Linear Add	No
30.DSL+EQY+-0.3EQX-	RSEQY+	1		No
30.DSL+EQY+-0.3EQX-	RSEQX-	-0.3		No
31.DSL+EQY-+0.3EQX	DL+SDL+0.3LL	1	Linear Add	No
31.DSL+EQY-+0.3EQX	RSEQY-	1		No
31.DSL+EQY-+0.3EQX	RSEQX	0.3		No
32.DSL+EQY-+0.3EQX+	DL+SDL+0.3LL	1	Linear Add	No
32.DSL+EQY-+0.3EQX+	RSEQY-	1		No
32.DSL+EQY-+0.3EQX+	RSEQX+	0.3		No
33.DSL+EQY-+0.3EQX-	DL+SDL+0.3LL	1	Linear Add	No
33.DSL+EQY-+0.3EQX-	RSEQY-	1		No
33.DSL+EQY-+0.3EQX-	RSEQX-	0.3		No
34.DSL+EQY--0.3EQX	DL+SDL+0.3LL	1	Linear Add	No
34.DSL+EQY--0.3EQX	RSEQY-	1		No
34.DSL+EQY--0.3EQX	RSEQX	-0.3		No
35.DSL+EQY--0.3EQX+	DL+SDL+0.3LL	1	Linear Add	No
35.DSL+EQY--0.3EQX+	RSEQY-	1		No
35.DSL+EQY--0.3EQX+	RSEQX+	-0.3		No
36.DSL+EQY--0.3EQX-	DL+SDL+0.3LL	1	Linear Add	No
36.DSL+EQY--0.3EQX-	RSEQY-	1		No
36.DSL+EQY--0.3EQX-	RSEQX-	-0.3		No
EQENV	1.DSL+EQX+0.3EQY	1	Envelope	No
EQENV	2.DSL+EQX+0.3EQY+	1		No
EQENV	3.DSL+EQX+0.3EQY-	1		No
EQENV	4.DSL+EQX-0.3EQY	1		No
EQENV	5.DSL+EQX-0.3EQY+	1		No
EQENV	6.DSL+EQX-0.3EQY-	1		No
EQENV	7.DSL+EQX++0.3EQY	1		No
EQENV	8.DSL+EQX++0.3EQY+	1		No
EQENV	9.DSL+EQX++0.3EQY-	1		No
EQENV	10.DSL+EQX+-0.3EQY	1		No
EQENV	11.DSL+EQX+-0.3EQY+	1		No
EQENV	12.DSL+EQX+-0.3EQY-	1		No
EQENV	13.DSL+EQX-+0.3EQY	1		No
EQENV	14.DSL+EQX-+0.3EQY+	1		No
EQENV	15.DSL+EQX-+0.3EQY-	1		No
EQENV	16.DSL+EQX--0.3EQY	1		No

Table 4.12 - Load Combinations (continued)

Name	Load Case/Combo	Scale Factor	Type	Auto
EQENV	17.DSL+EQX--0.3EQY+	1		No
EQENV	18.DSL+EQX--0.3EQY-	1		No
EQENV	19.DSL+EQY+0.3EQX	1		No
EQENV	20.DSL+EQY+0.3EQX+	1		No
EQENV	21.DSL+EQY+0.3EQX-	1		No
EQENV	22.DSL+EQY-0.3EQX	1		No
EQENV	23.DSL+EQY-0.3EQX+	1		No
EQENV	24.DSL+EQY-0.3EQX-	1		No
EQENV	25.DSL+EQY++0.3EQX	1		No
EQENV	26.DSL+EQY++0.3EQX+	1		No
EQENV	27.DSL+EQY++0.3EQX-	1		No
EQENV	28.DSL+EQY+-0.3EQX	1		No
EQENV	29.DSL+EQY+-0.3EQX+	1		No
EQENV	30.DSL+EQY+-0.3EQX-	1		No
EQENV	31.DSL+EQY-+0.3EQX	1		No
EQENV	32.DSL+EQY-+0.3EQX+	1		No
EQENV	33.DSL+EQY-+0.3EQX-	1		No
EQENV	34.DSL+EQY--0.3EQX	1		No
EQENV	35.DSL+EQY--0.3EQX+	1		No
EQENV	36.DSL+EQY--0.3EQX-	1		No
EQENV	DL+SDL+0.3LL	1		No

CALCULATION SHEET

Project/Task/File No: WEGC EAST BLOCK DSA | Sheet No of

Project Description: | Office:

Computed: PMO1 8/10/2015

Check: / /

MODAL RESPONSE VERSUS EQUIVALENT STATIC COMPARISON OF RESULTS

Equivalent static analysis gives more conservative building forces than the Modal Response Analysis that was also carried out:

Base shear = 6571 kN - Eq. Static

4527/5358 kN - Modal Response

∴ Equivalent Static Method adopted for obtaining element demands.

MODEL V.3.0 – MODAL RESPONSE STOREY FORCES

Story	Load Case/Com	Location	P	VX	VY	T	MX	MY
Story5	EQENV Max	Bottom	278	217	513	18756	3240	-6857
Story5	EQENV Min	Bottom	193	-217	-513	-18752	-446	-10265
Story4	EQENV Max	Bottom	932	598	967	36859	4442	-21474
Story4	EQENV Min	Bottom	586	-598	-963	-36738	3768	-33735
Story3	EQENV Max	Bottom	919	608	187	3515	7496	-9172
Story3	EQENV Min	Bottom	509	-587	-401	-9342	3753	-25743
Story2	EQENV Max	Bottom	2761	1512	676	14616	26307	-38984
Story2	EQENV Min	Bottom	1809	-1499	-896	-20506	14615	-71921
Story1	EQENV Max	Bottom	10134	4527	5358	168680	91998	-242236
Story1	EQENV Min	Bottom	10055	-4527	-5358	-168680	33868	-294167

Equivalent Static Base Shear = 6571 kN

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MODEL 2.2 EQUIVALENT STATIC STOREY FORCES

Story	Load Case/Com	Location	P	VX	VY	T	MX	MY
Story5	EQENV Max	Bottom	294	186	275	10283	4709	-8225
Story5	EQENV Min	Bottom	208	-57	-917	-33701	285	-10735
Story4	EQENV Max	Bottom	849	166	510	23947	4645	-15011
Story4	EQENV Min	Bottom	413	-549	-1693	-63334	3815	-32615
Story3	EQENV Max	Bottom	1060	292	-34	7075	8609	-1907
Story3	EQENV Min	Bottom	343	-925	-489	-11581	2084	-28787
Story2	EQENV Max	Bottom	3314	557	138	19114	33053	-36770
Story2	EQENV Min	Bottom	1636	-1843	-1125	-25227	13906	-88702
Story1	EQENV Max	Bottom	10111	1971	1971	98752	109423	-254894
Story1	EQENV Min	Bottom	10111	-6571	-6571	-198498	48977	-315340

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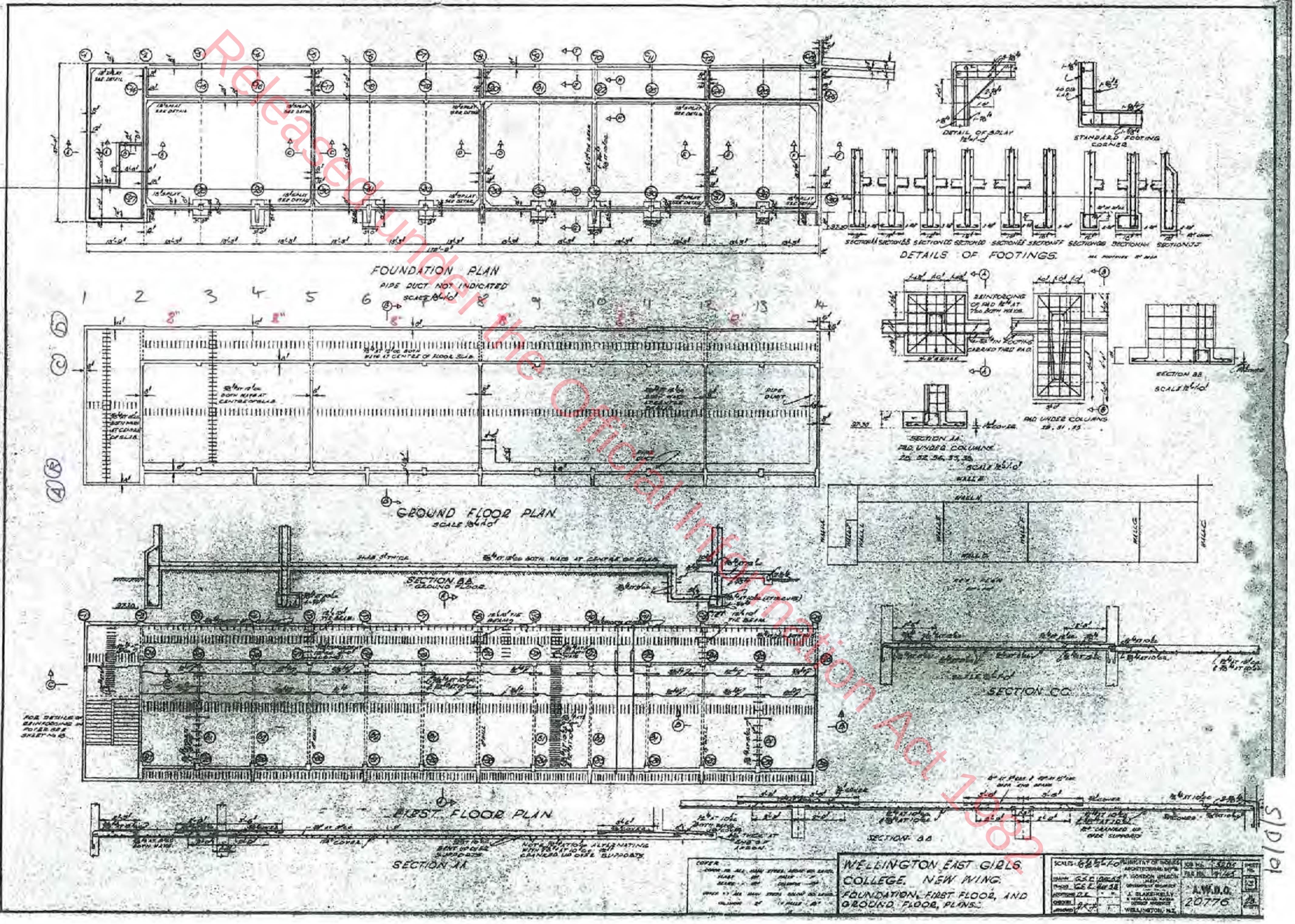
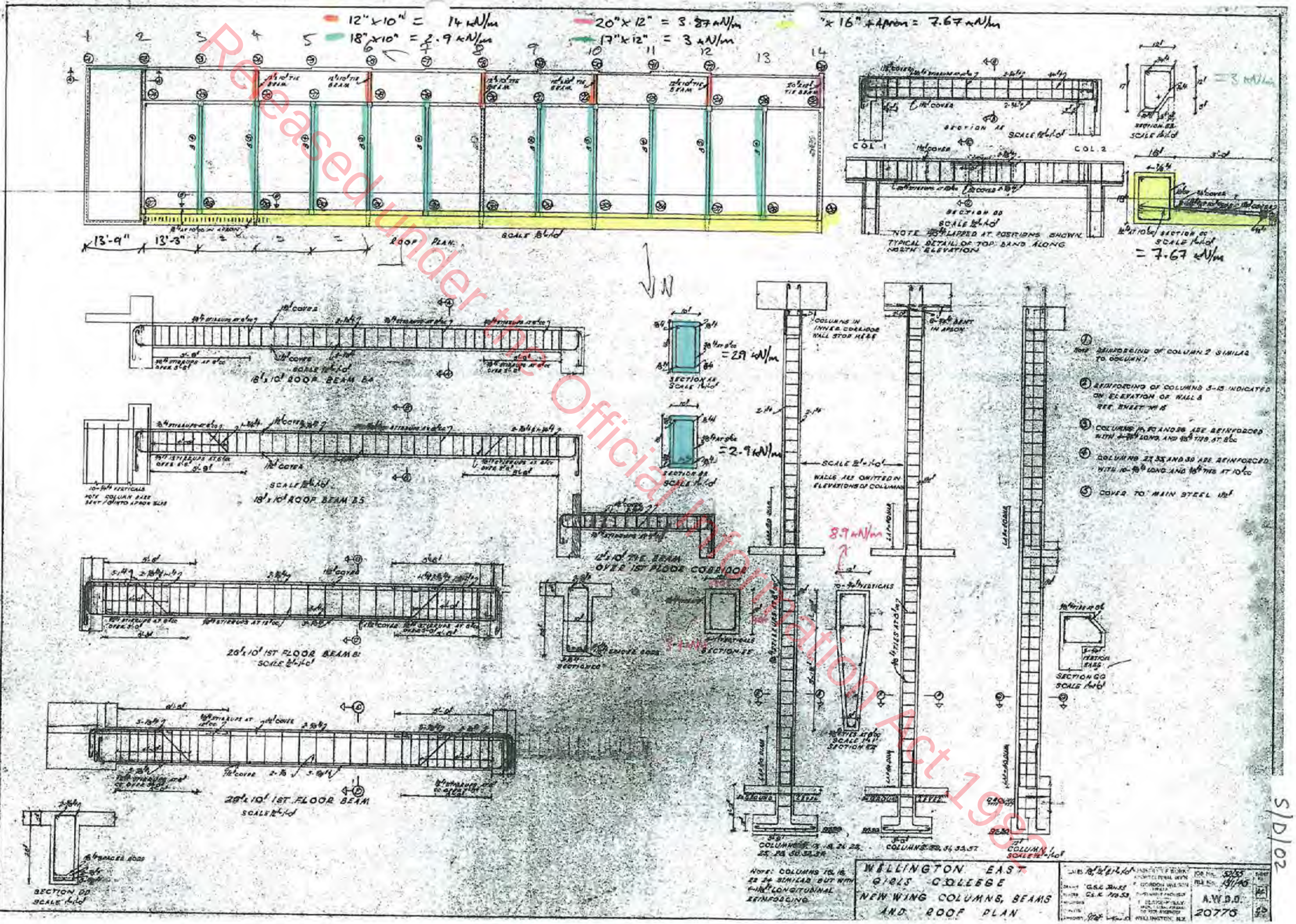
Model Register

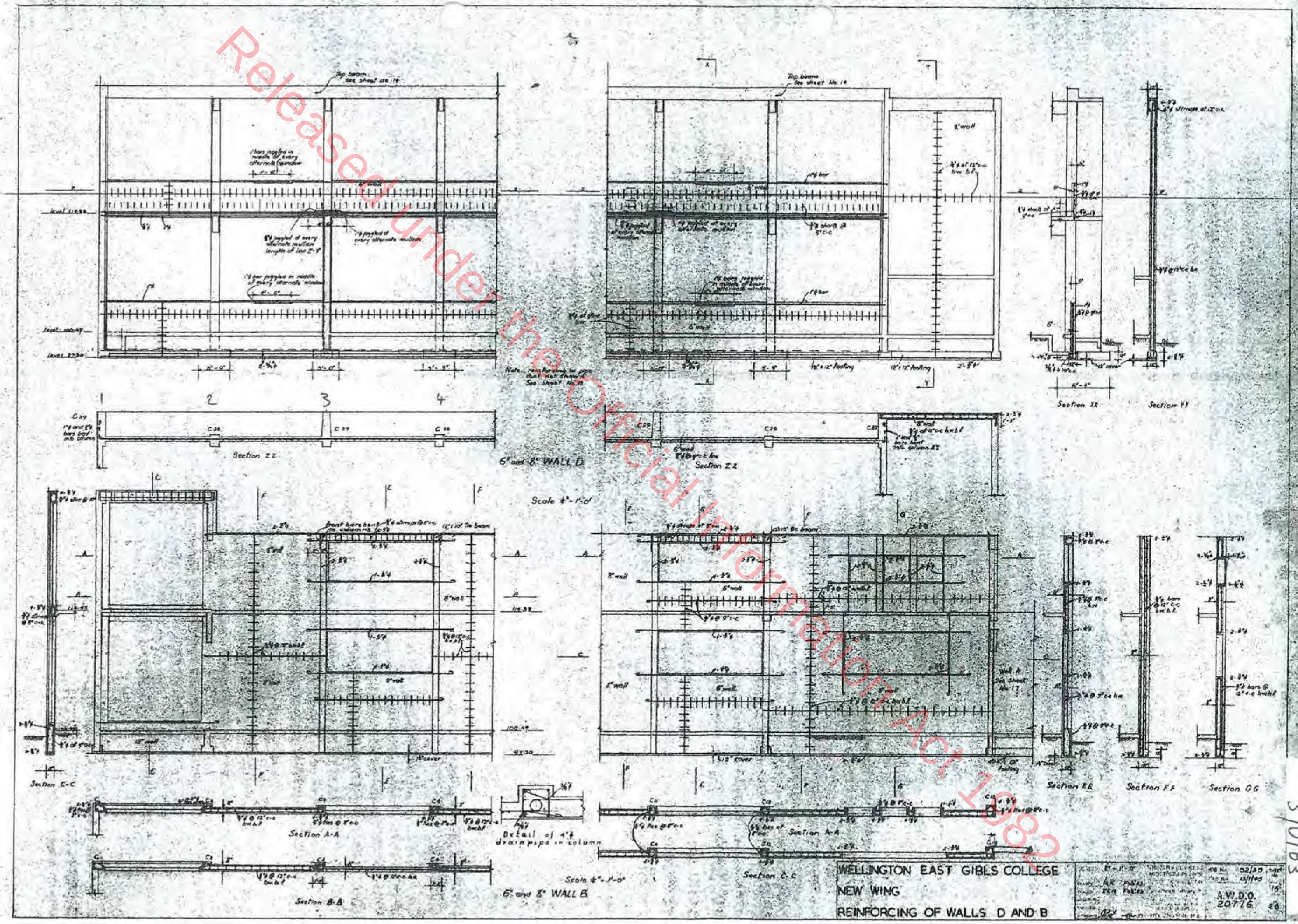
	Developed from	Description
East Block v1.0	N/A	Basic model. No strengthening, no top storey.
East Block v1.1	East Block v1.0	Diaphragms added. Load cases added.
East Block v2.0	East Block v1.1	Top storey added. Roof loads added as UDLs on beams.
East Block v2.1	East Block v2.0	Period specified to 0.4s.
East Block v2.2	East Block v2.1	Disconnected CBF on GLA L1 and L2 by offsetting CBF by 250mm. Top storey roof still connected.
East Block v2.3	East Block v2.2	Wall on GLA:1-2 stiffness reduced so that it won't attract very much in-plane shear. To model effects of it becoming disconnected from the floor diaphragm.
East Block v2.4	East Block v2.2	Vertical braces on GLC removed
East Block v2.5	East Block v2.2	Model v2.2 with mu=2, Sp=0.7 demands
East Block v2.6	East Block v2.2	Model v2.2 with mu=4, Sp=0.7 demands

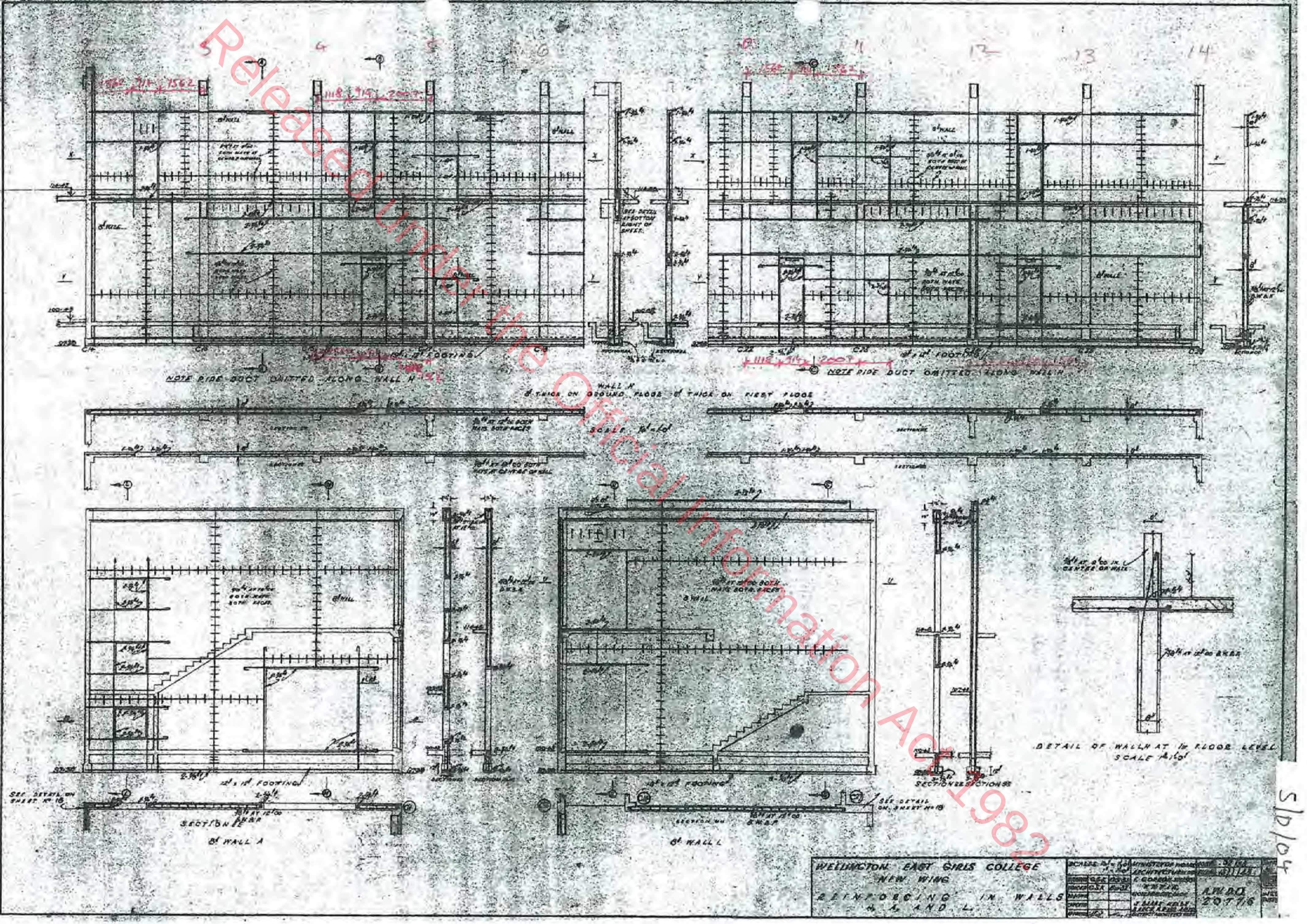
East Block v3.0	East Block v2.0	Modal Response Spectrum method. Used to compare to Equivalent Static model.



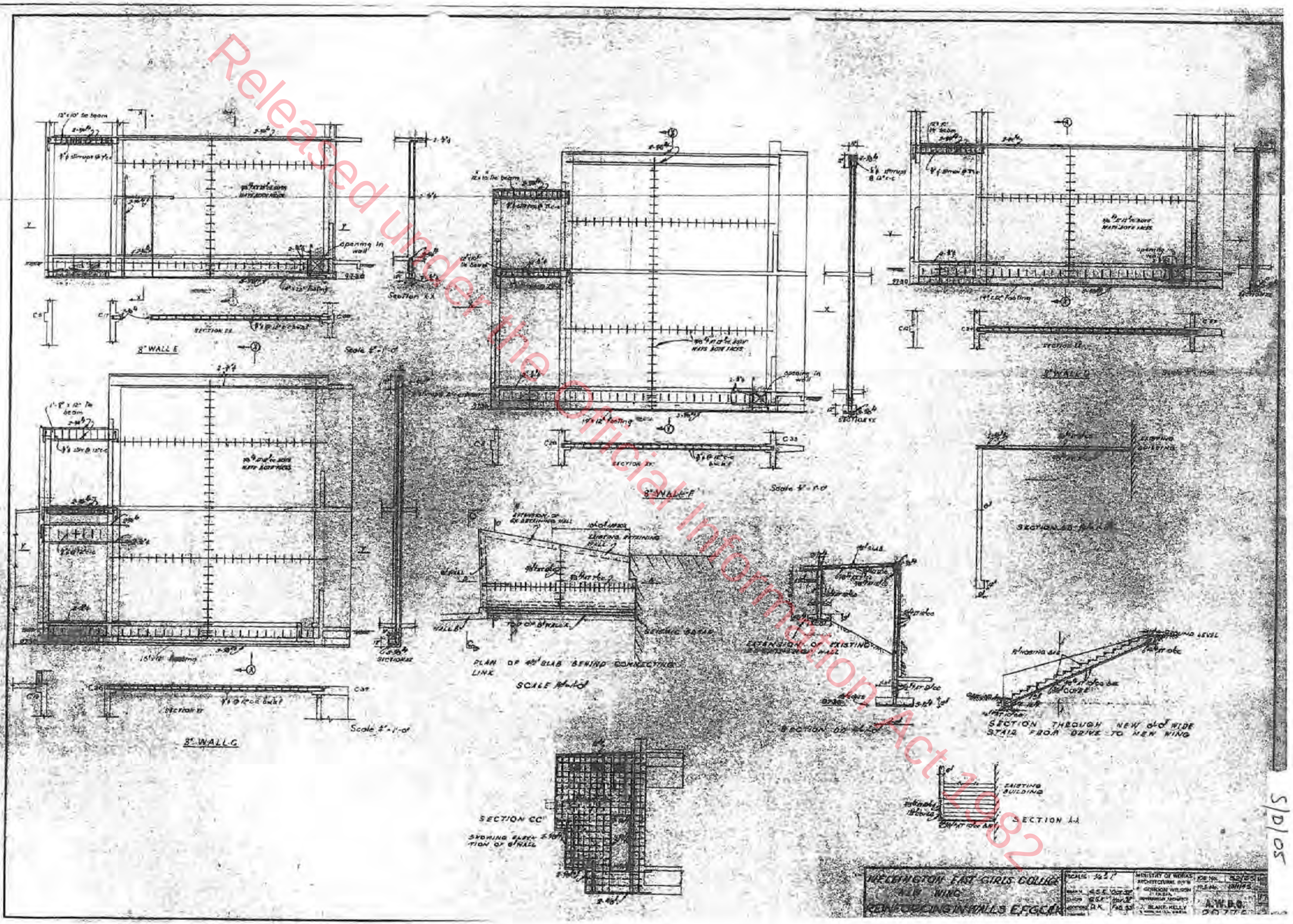
FOUNDATION PLAN
PIPE DUCT NOT INDICATED
DETAILS OF FOOTINGS
DETAIL OF SPLAY
STANDARD FOOTING CORNER
SECTION BB
SECTION AA
PAD UNDER COLUMNS
GROUND FLOOR PLAN
SECTION BB GROUND FLOOR
SECTION CC
FIRST FLOOR PLAN
SECTION BB
SECTION AA
WELLINGTON EAST GIRLS COLLEGE. NEW WING.
FOUNDATION, FIRST FLOOR, AND GROUND FLOOR, PLANS.
A.W.D.O.
20776
5/D/01

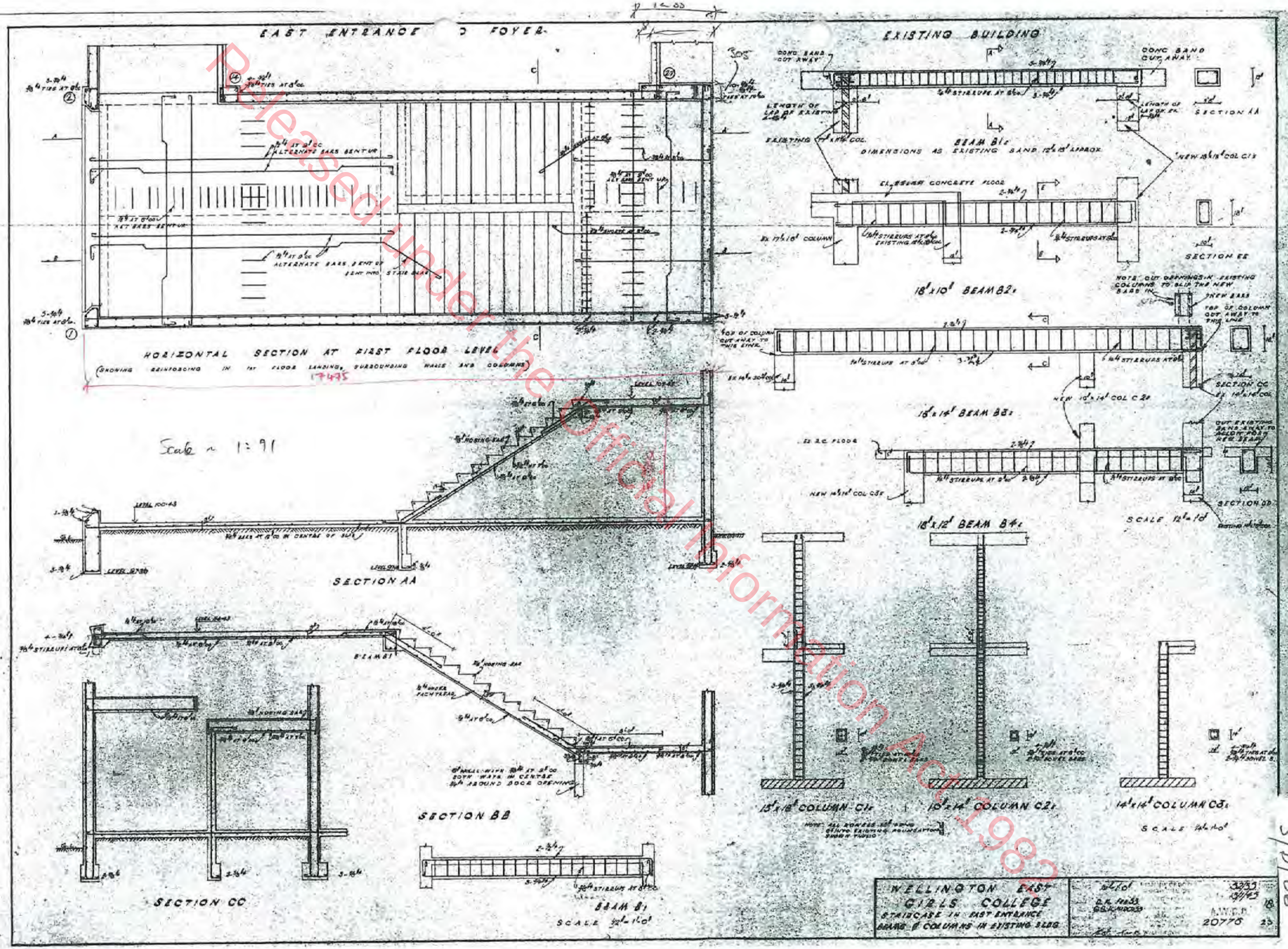

12"×10" = 14 kN/m
18"×10" = 2.9 kN/m
20"×12" = 3.87 kN/m
17"×12" = 3 kN/m
"×16" + Apron = 7.67 kN/m
ROOF PLAN
SCALE 1/8"=1'0"
13'-9"
13'-3"
COL. 1
COL. 2
= 3 kN/m
= 7.67 kN/m
18'×10' ROOF BEAM B4
18'×10' ROOF BEAM B5
= 2.9 kN/m
= 2.9 kN/m
12'×10' TIE BEAM OVER 1ST FLOOR CORRIDOR
20'×10' 1ST FLOOR BEAM
20'×10' 1ST FLOOR BEAM
8.9 kN/m
COVER TO MAIN STEEL 1½"
NOTE: COLUMNS 16, 18, 22, 24 SIMILAR BUT WITH 4-1" LONGITUDINAL REINFORCING
WELLINGTON EAST GIRLS COLLEGE
NEW WING COLUMNS, BEAMS AND ROOF PLAN
A.W.D.O.
20776
S/D/02


S/0103
Top beam see sheet No. 14
6" and 8" WALL D
Scale ¼"-1'-0"
Section ZZ
Section XX
Section YY
Section C-C
Section A-A
Section B-B
Section EE
Section FF
Section GG
Detail of 4" drain pipe in column
6" and 8" WALL B
WELLINGTON EAST GIRLS COLLEGE
NEW WING
REINFORCING OF WALLS D AND B
A.W.D.O. 20776

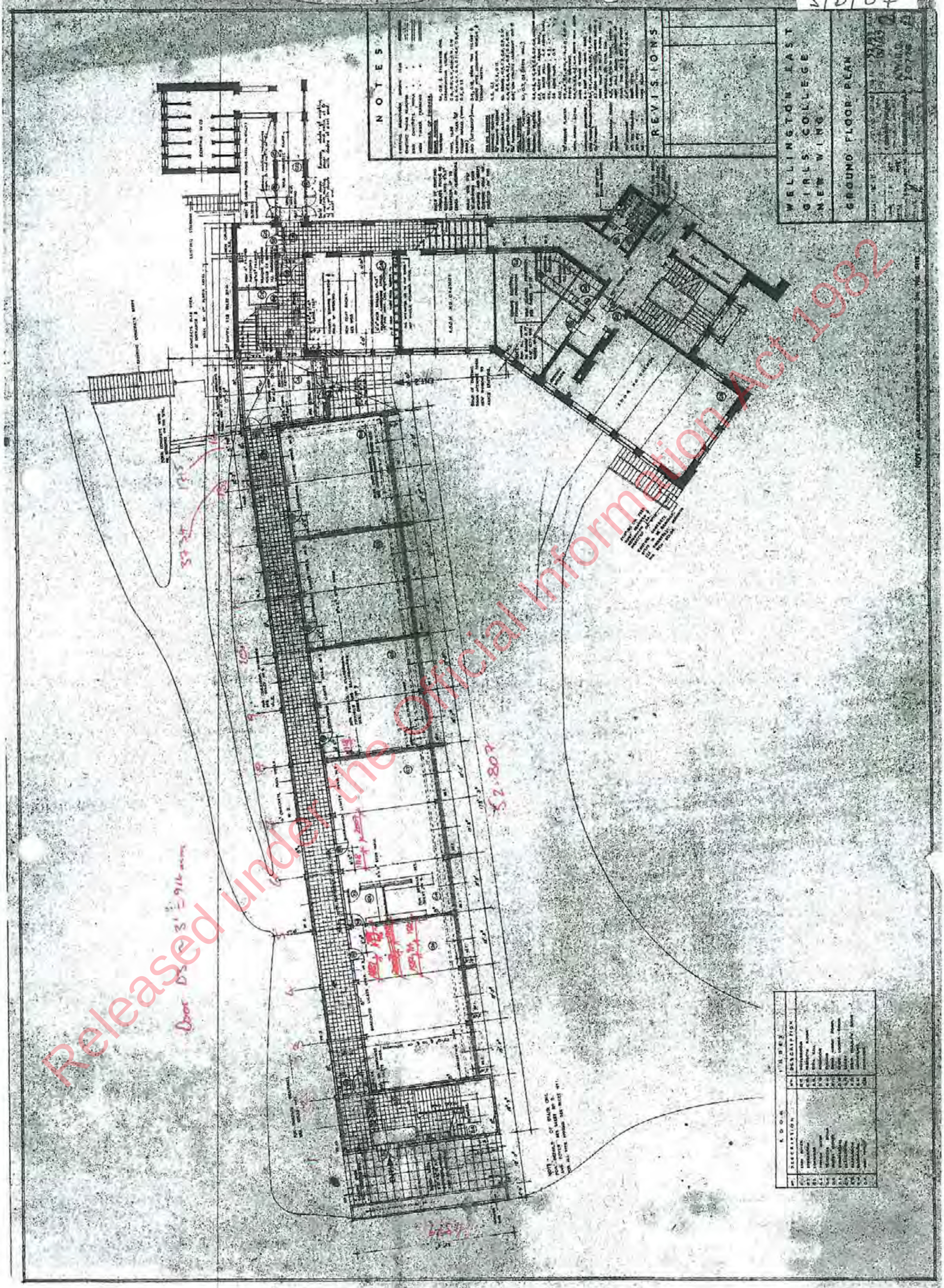

WALL H
8" THICK ON GROUND FLOOR 10" THICK ON FIRST FLOOR
NOTE PIPE DUCT OMITTED ALONG WALL H
SECTION
8" WALL A
8" WALL L
DETAIL OF WALL H AT 1st FLOOR LEVEL
WELLINGTON EAST GIRLS COLLEGE
NEW WING
REINFORCING IN WALLS
H, A AND L
20776
5/D/04

S/D/05


8" WALL E
Section X-X
8" WALL F
8" WALL G
Scale 1/2" = 1'-0"
PLAN OF 4" SLAB BEHIND CONNECTING LINK
EXTENSION OF EXISTING RETAINING WALL
SECTION THROUGH NEW 6'0" WIDE STAIR FROM DRIVE TO NEW WING
SECTION CC
SHOWING ELEVATION OF 8" WALL
EXISTING BUILDING
SECTION A-A
WELLINGTON EAST GIRLS COLLEGE
A1B WING
REINFORCING IN WALLS E.F.G.C.A.A.
MINISTRY OF WORKS
ARCHITECTURAL DIVN
GORDON WILSON
J. BLAKE-KELLY
A.W.B.O.

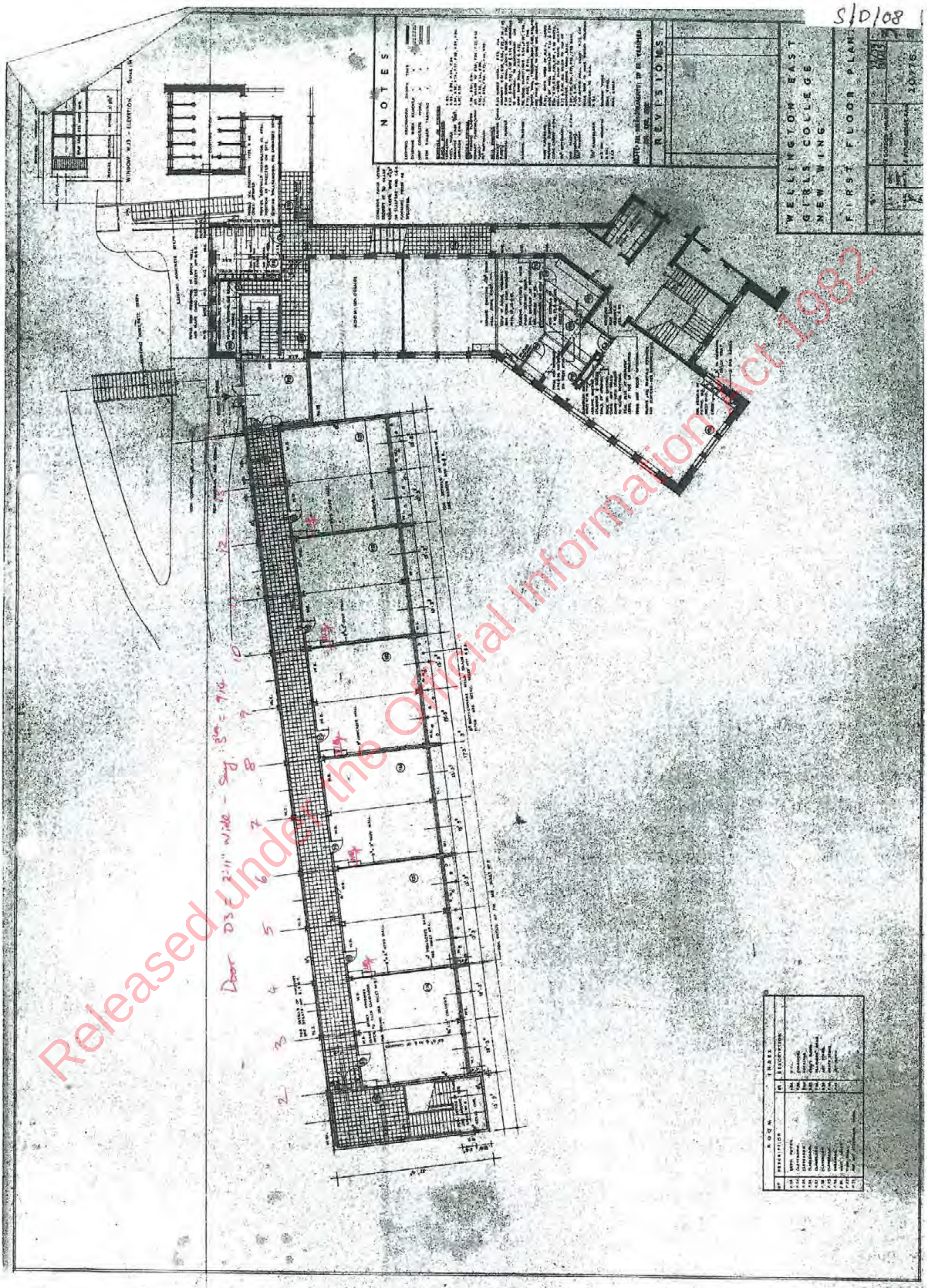

S/D/06
EAST ENTRANCE FOYER
EXISTING BUILDING
HORIZONTAL SECTION AT FIRST FLOOR LEVEL
(SHOWING REINFORCING IN 1ST FLOOR LANDING, SURROUNDING WALLS AND COLUMNS)
17475
Scale ~ 1:91
SECTION AA
SECTION BB
SECTION CC
BEAM B1
SCALE
BEAM B1:
DIMENSIONS AS EXISTING BAND
SECTION AA
SECTION EE
18" x 10" BEAM B2:
SECTION CC
18" x 14" BEAM B3:
SECTION DD
18" x 12" BEAM B4:
SCALE
15" x 18" COLUMN C1:
10" x 14" COLUMN C2:
14" x 14" COLUMN C3:
WELLINGTON EAST GIRLS COLLEGE
STAIRCASE IN EAST ENTRANCE
BEAMS & COLUMNS IN EXISTING BLDG
20776

S/D/07

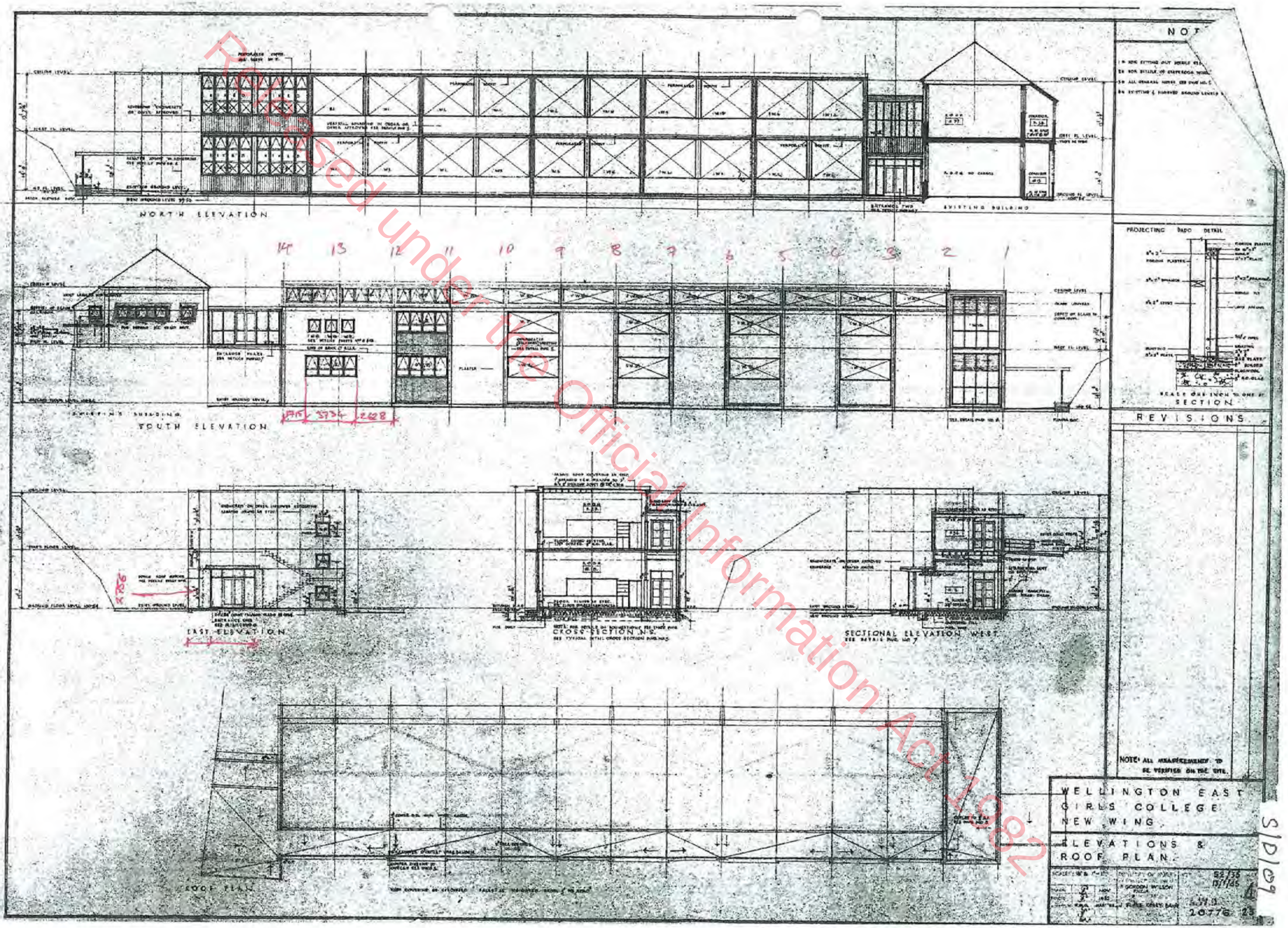


NOTES
REVISIONS
WELLINGTON EAST
GIRLS COLLEGE
NEW WING
GROUND FLOOR PLAN
INDEX
Door D5 ~ 3' = 914 mm

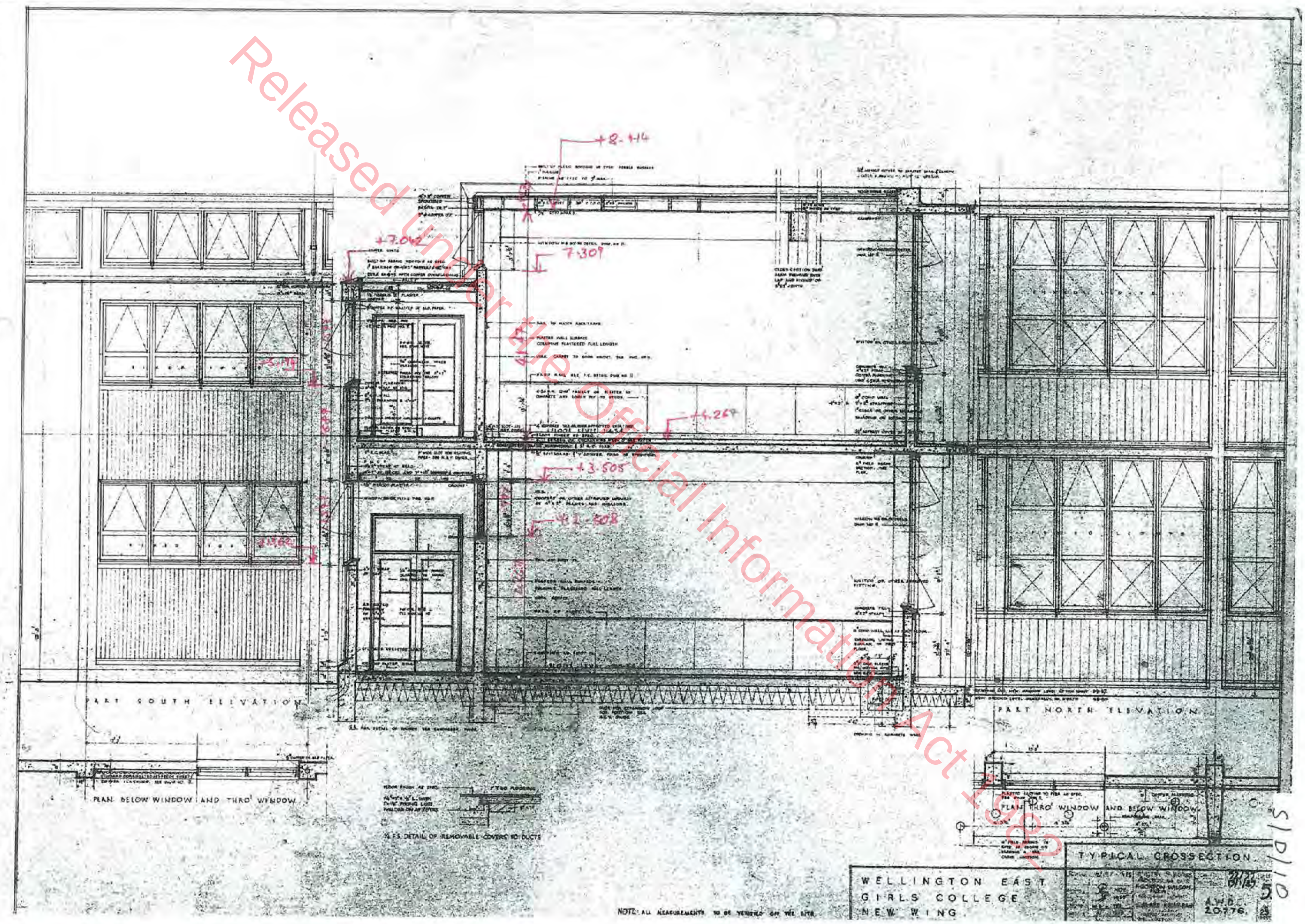
S/D/08



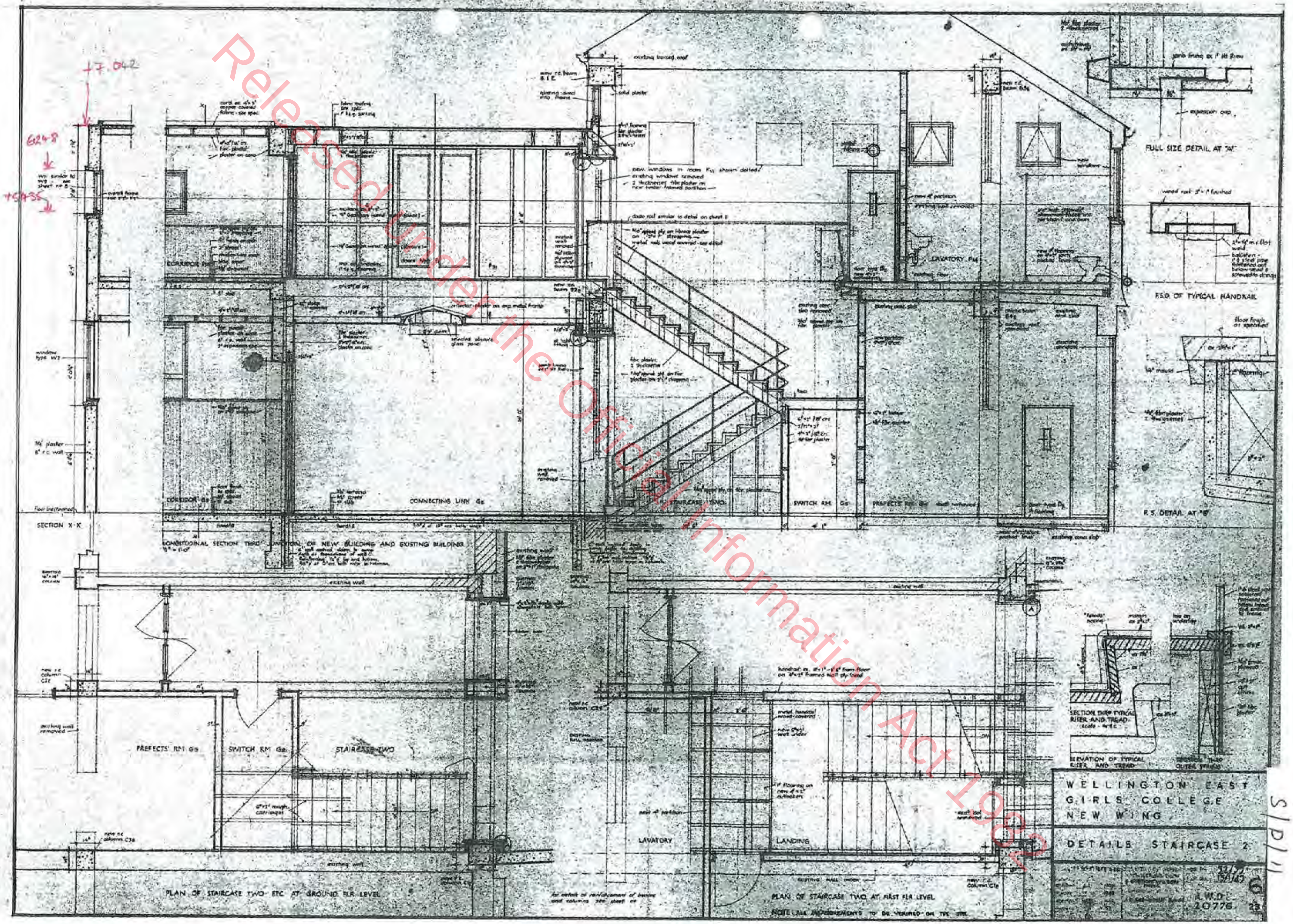
NOTES
REVISIONS
WELLINGTON EAST GIRLS COLLEGE NEW WING
FIRST FLOOR PLAN
ROOM INDEX



NORTH ELEVATION
EXISTING BUILDING
SOUTH ELEVATION
14 13 12 11 10 9 8 7 6 5 4 3 2 1
1715 5734 2628
EAST ELEVATION
2756
CROSS SECTION
SECTIONAL ELEVATION WEST
ROOF PLAN
PROJECTING DADO DETAIL
SECTION
REVISIONS
NOTE: ALL MEASUREMENTS TO BE VERIFIED ON THE SITE.
WELLINGTON EAST GIRLS COLLEGE NEW WING
ELEVATIONS & ROOF PLAN
20776
S/D/09



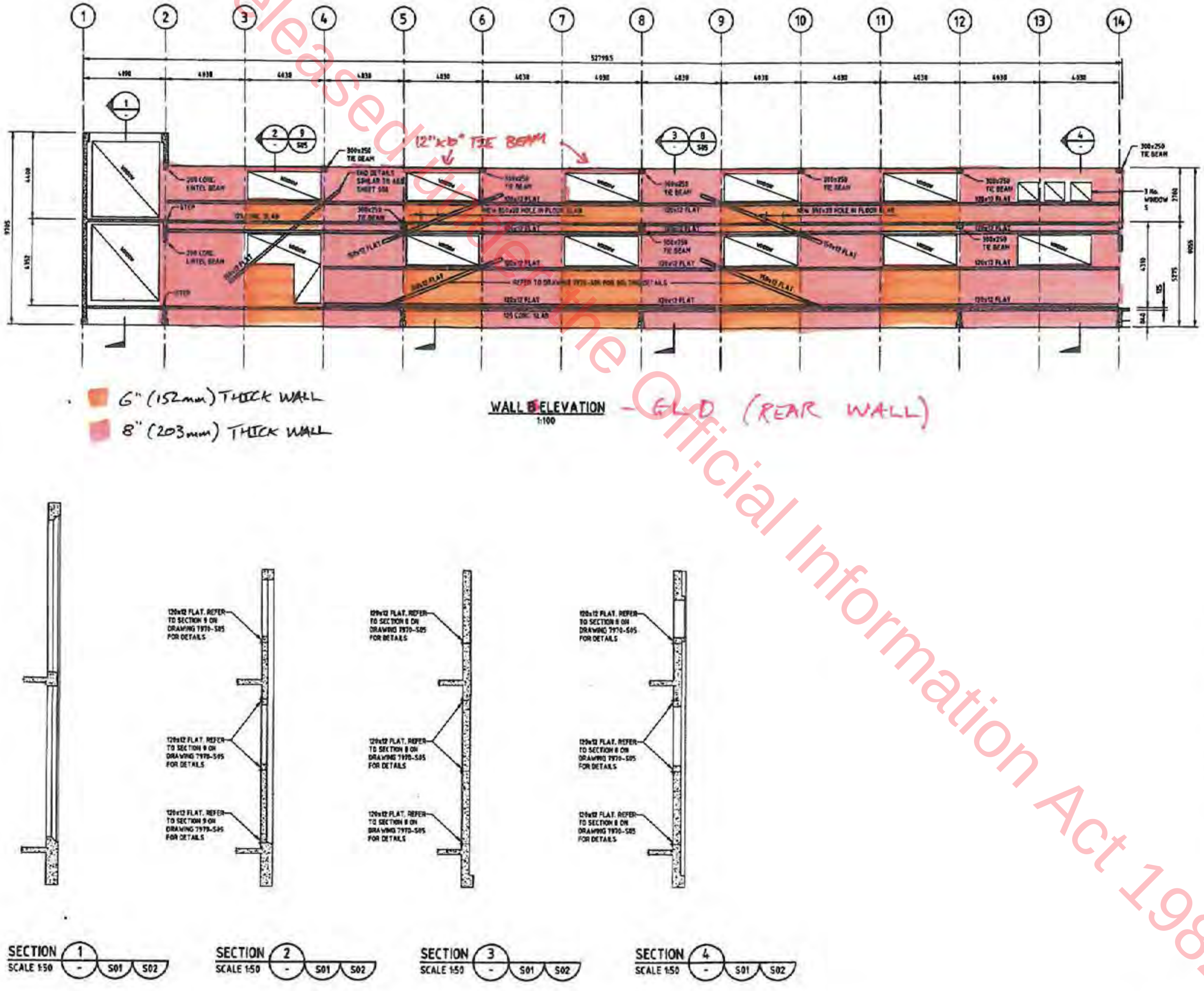
+8.414
+7.042
7.309
+4.269
+3.505
+1.808
PART SOUTH ELEVATION
PART NORTH ELEVATION
PLAN BELOW WINDOW AND THRO' WINDOW
PLAN THRO' WINDOW AND BELOW WINDOW
F.S. DETAIL OF REMOVABLE COVERS TO DUCTS
TYPICAL CROSSECTION
WELLINGTON EAST GIRLS COLLEGE NEW WING
NOTE: ALL MEASUREMENTS TO BE VERIFIED ON THE SITE
20776
5
S10/10



S/D/11
WELLINGTON EAST GIRLS COLLEGE NEW WING
DETAILS STAIRCASE 2
L.W.D. 20776
6
SECTION X-X
LONGITUDINAL SECTION THRO' JUNCTION OF NEW BUILDING AND EXISTING BUILDING
CONNECTING LINK
CORRIDOR
STAIRCASE TWO
SWITCH RM
PREFECTS RM
LAVATORY
LANDING
PLAN OF STAIRCASE TWO ETC AT GROUND FLR LEVEL
PLAN OF STAIRCASE TWO AT FIRST FLR LEVEL
NOTE: ALL MEASUREMENTS TO BE VERIFIED ON THE SITE
FULL SIZE DETAIL AT "A"
F.S.D. OF TYPICAL HANDRAIL
F.S. DETAIL AT "B"
SECTION THRO' TYPICAL RISER AND TREAD
ELEVATION OF TYPICAL RISER AND TREAD
existing trussed roof
existing wall
+7.042
6248
+5435

ALL DIMENSIONS TO BE CONFIRMED ON SITE

LONGITUDINAL SHEAR WALLS - GL D



WALL B ELEVATION - GL D (REAR WALL)
1:100

SECTION 1 - S01 S02
SCALE 1:50

SECTION 2 - S01 S02
SCALE 1:50

SECTION 3 - S01 S02
SCALE 1:50

SECTION 4 - S01 S02
SCALE 1:50

Rev	Date	Revision Details	By	Ver.	App.
2	24/10/03	ISSUE FOR BUILDING CONSENT	GKS		
01	18.07.03	ISSUE FOR INFORMATION	GKS		

Connell Mott MacDonald
...building the future

Project:
WELLINGTON EAST GIRLS COLLEGE
EAST WING BLOCK 7
STRUCTURAL STRENGTHENING

Drawing Title:
WALL B ELEVATION AND TYPICAL SECTIONS

WELLINGTON CITY COUNCIL ENVIRONMENT
24 OCT 2003
RECEIVED
WAKEFIELD ST, WELLINGTON

Drawn	Signed	Date	Verified	Signed	Date
JK		24/10/03			
Designed GKS		24/10/03	Approved		

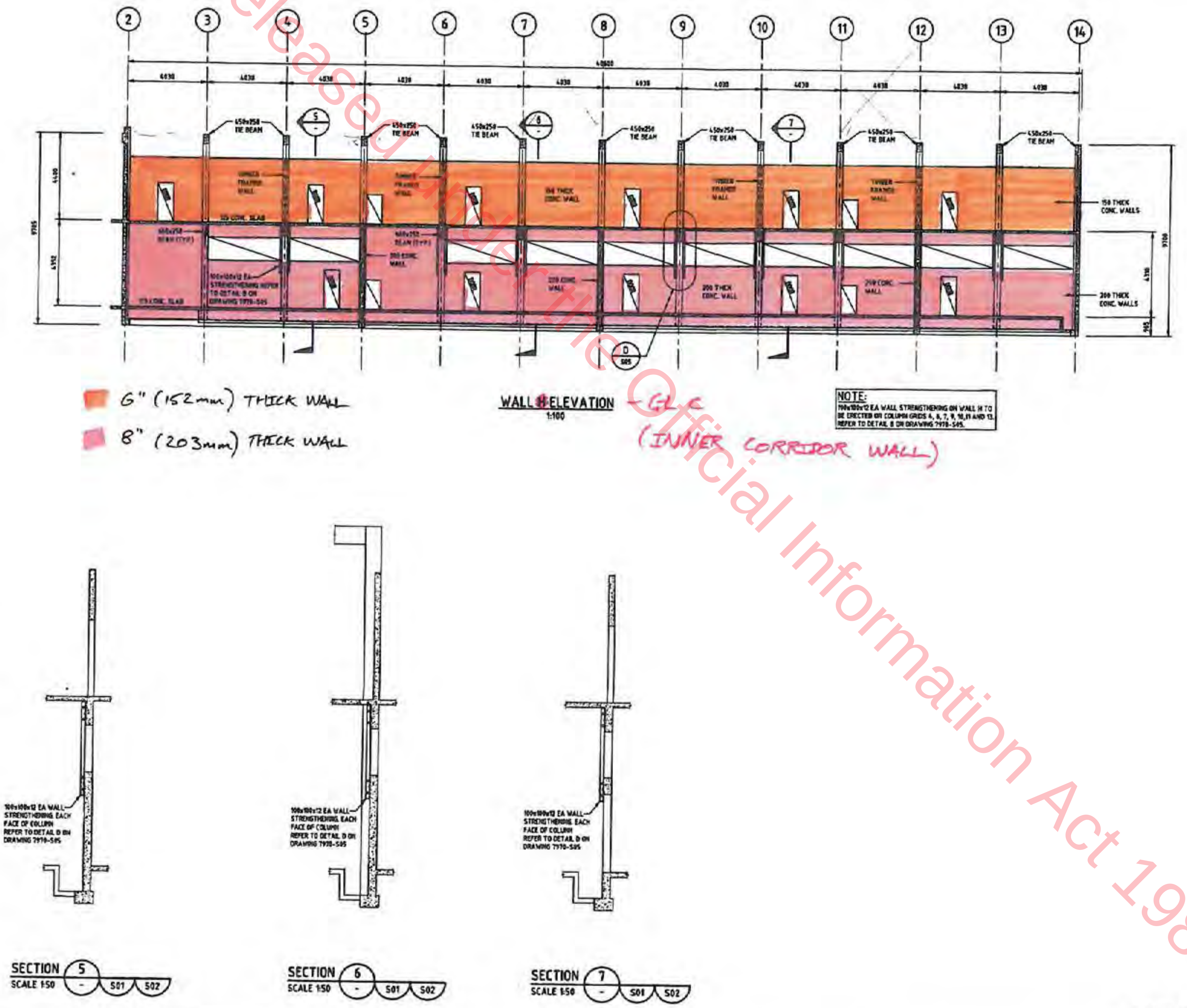
CW Project No. 7970
Scale 1:100 1:50 (A1) 1:200 1:100 (A3)
Drawing No. S03
Revision 2

PRELIMINARY

5/D/12

A1

ALL DIMENSIONS TO BE CONFIRMED ON SITE



LONG. SHEAR WALLS - GLC

Rev	Date	Revision Details	By	Ver.	App.
2	24/10/03	ISSUE FOR BUILDING CONSENT			
01	18.07.03	ISSUE FOR INFORMATION	GKS		

Connell Mott MacDonald ...building the future

Connell Wagner Limited
Client:

Project: WELLINGTON EAST GIRLS COLLEGE EAST WING BLOCK 7 STRUCTURAL STRENGTHENING

Drawing Title: WALL H ELEVATION AND TYPICAL SECTIONS

WELLINGTON CITY COUNCIL 24 OCT 2003

Drawn	Signed	Date	Verified	Date
JK		24/10/03		
Designed	Signed	Date	Approved	Signed	Date
GKS		24/10/03			

CW Project No. 7970
Scale 1:100 1:50 (A1) 1:200 1:100 (A3)
Drawing No. S04
Revision 2

PRELIMINARY

3/D/13

S/001

CALCULATION SHEET

Project/Task/File No: WEGC EAST BLOCK DSA

Project Description:

Sheet No of

Office:

Computed: AMO / 6/10/2015

Check: / /

SUMMARY OF RESULTS

TRANS SHEAR WALLS

Ground Floor > 88%

1st Floor > 100%

LONGITUDINAL SHEAR WALLS

Ground Floor > 100%

1st Floor > 100%

CBF ON FRONT ELEVATION

Ground Floor > 100%

1st Floor > 100%

2nd Floor > ~70% – If other x braces become ineffectual.

>100% If other x braces work.

X-BRACES ON 2nd FLOOR

- No tie beam at bottom of braces – limits their ability.

Cantilevering Columns > 55%* – But CBF on front elevation carries more force at the columns yield.....

Braces & Connections > 75%

2nd Floor CBF > 70% If x braces lose strength/stiffness.

TRANSVERSE PORTAL FRAMES ON 2nd FLOOR

2nd Floor > 100% – LTB may occur but fairly low ductility demands – $\mu < 2$

S/00ii

CALCULATION SHEET

Project/Task/File No: WEGC EAST BLOCK DSA

Project Description:

Sheet No of

Office:

Computed: PMol 6/10/2015

Check: / /

DIAPHRAGMS

1ST Floor (Concrete) > 100% — Some damage around GL 2:B corner, but not a life-safety issue

2nd Floor (Timber) > 100%

ROOF - (Reid braces) > 100%

2nd Floor - Concrete Beams Out of Plane > 95%

OPUS

S/01

OPUS Opus International Consultants PO Box 12003 Wellington 6144	Project Wall Shear to NZS3101				Job Ref.	
	Section 8" Thick Wall on GLA				Sheet no./rev. 1	
	Calc. by PMO	Date 21/09/2015	Chk'd by	Date	App'd by	Date

Shear Capacity of Concrete Wall

Axial compressive load	$N = 1$ kN
Shear action	$V = 1$ kN
Concrete compressive strength	$f_c = 30$MPa
Length of wall	$L_w = 4.191$m
Wall thickness	$t = 203$ mm
Capacity reduction factor	$\phi = 0.75$
Area of shear reinforcement within	$A_v = 143\text{mm}^2$ (2 layer of 3/8" diameter bars)
a distance	$s_2 = 305$ mm
Yield strength of the shear reinforcement	$f_{yt} = 245$MPa (33,000 psi x 1.08)
Effective depth of wall	$d = 0.8 \times L_w = \mathbf{3.353}$m
Gross area of section	$A_g = L_w \times t = \mathbf{850773.000}\text{mm}^2$
Shear area	$A_{cv} = d \times t = \mathbf{680618.400}\text{mm}^2$
Shear resisted by concrete	$v_c = 0.17 \times (\sqrt{(f_c)} \times 1\ \text{kg}^{0.5} \times 1\ \text{m}^{-0.5} \times 1\ \text{s}^{-1} \times 1000 + N/\ A_g) = \mathbf{0.931}$ MPa
Concrete shear strength	$V_c = v_c \times A_{cv} = \mathbf{633.879}$ kN
Shear resisted by reinforcement	$V_s = A_v \times f_{yt} \times d\ /\ s_2 = \mathbf{385.132}$kN
Probable shear resistance	$V_p = \phi \times (V_c + V_s) = \mathbf{764.259}$ kN

Minimum Requirements

Minimum reinforcement	$A_{v\_min} = 0.7 \times 1000\text{kg} / 1\text{mm} / 1\text{s}^2 \times t \times s_2 / f_{yt} = \mathbf{177}\ \text{mm}^2$

Max Allowable Shear Stress

Shear stress	$v = V / (L_w \times t) = \mathbf{0.001}$ MPa
Max nominal shear stress	$v_{max} = \min(0.2 \times f_c, 8\text{MPa}) = \mathbf{6.000}$MPa

S/02


A
1
A
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6
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A
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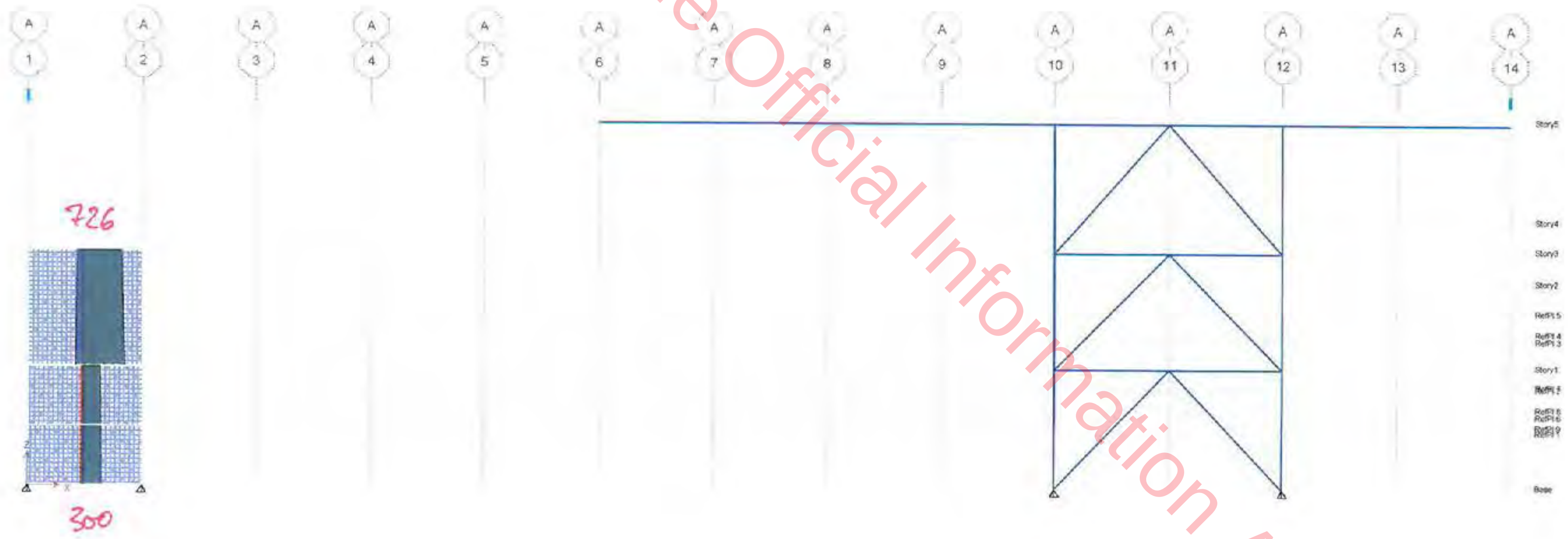
A
11
A
12
A
13
A
14
Story5
Story4
Story3
Story2
Story1
Base
8" Wall
8" Wall
8" Wall

S/03



East Block v2.1.EDB Elevation View - A Shear Force 2-2 Diagram (EQENV) [kN]

S/04

MODEL V2.2 - CBF DISCONNECTED

GLA IN-PLANE SHEAR

CAPACITY = 764 kN - O.K.

S/05

OPUS Opus International Consultants PO Box 12003 Wellington 6144	Project Wall Shear to NZS3101				Job Ref.	
	Section Typical 6" Thick Wall				Sheet no./rev. 1	
	Calc. by PMO	Date 8/09/2015	Chk'd by	Date	App'd by	Date

Shear Capacity of Concrete Wall – GL C2 GL D

Axial compressive load	N = 1 kN
Shear action	V = 1 kN
Concrete compressive strength	f_c = 30MPa
Length of wall	L_w = 4.343m
Wall thickness	t = 152 mm
Capacity reduction factor	ϕ = 0.75
Area of shear reinforcement within	A_v = 71mm^2 (1 layer of 3/8" diameter bars)
a distance	s_2 = 229 mm
Yield strength of the shear reinforcement	f_{yt} = 245MPa (33,000 psi x 1.08)
Effective depth of wall	$d = 0.8 \times L_w =$ **3.474**m
Gross area of section	$A_g = L_w \times t =$ **660136.000**mm^2
Shear area	$A_{cv} = d \times t =$ **528108.800**mm^2
Shear resisted by concrete	$v_c = 0.17 \times (\sqrt{(f_c)} \times 1\ kg^{0.5} \times 1\ m^{-0.5} \times 1\ s^{-1} \times 1000 + N/\ A_g) =$ **0.931** MPa
Concrete shear strength	$V_c = v_c \times A_{cv} =$ **491.873** kN
Shear resisted by reinforcement	$V_s = A_v \times f_{yt} \times d / s_2 =$ **263.918**kN
Probable shear resistance	$V_p = \phi \times (V_c + V_s) =$ **566.843** kN

Minimum Requirements

Minimum reinforcement	$A_{v\_min} = 0.7 \times 1000kg / 1mm / 1s^2 \times t \times s_2 / f_{yt} =$ **99** mm^2

Max Allowable Shear Stress

Shear stress	$v = V / (L_w \times t) =$ **0.002** MPa
Max nominal shear stress	$v_{max} = \min(0.2 \times f_c, 8MPa) =$ **6.000**MPa

S/06

OPUS Opus International Consultants PO Box 12003 Wellington 6144	Project Wall Shear to NZS3101				Job Ref.	
	Section Typical 8" Thick Wall				Sheet no./rev. 1	
	Calc. by PMO	Date 8/09/2015	Chk'd by	Date	App'd by	Date

Shear Capacity of Concrete Wall GL C& GLD

Axial compressive load	N = 1 kN
Shear action	V = 1 kN
Concrete compressive strength	f_c = 30MPa
Length of wall	L_w = 4.343m
Wall thickness	t = 203 mm
Capacity reduction factor	ϕ = 0.75
Area of shear reinforcement within	A_v = 143mm^2 (2 layer of 3/8" diameter bars)
a distance	s_2 = 305 mm
Yield strength of the shear reinforcement	f_{yt} = 245MPa (33,000 psi x 1.08)
Effective depth of wall	$d = 0.8 \times L_w$ = **3.474**m
Gross area of section	$A_g = L_w \times t$ = **881629.000**mm^2
Shear area	$A_{cv} = d \times t$ = **705303.200**mm^2
Shear resisted by concrete	$v_c = 0.17 \times (\sqrt{(f_c)} \times 1\ kg^{0.5} \times 1\ m^{-0.5} \times 1\ s^{-1} \times 1000 + N/ A_g)$ = **0.931** MPa
Concrete shear strength	$V_c = v_c \times A_{cv}$ = **656.864** kN
Shear resisted by reinforcement	$V_s = A_v \times f_{yt} \times d / s_2$ = **399.100**kN
Probable shear resistance	$V_p = \phi \times (V_c + V_s)$ = **791.973** kN

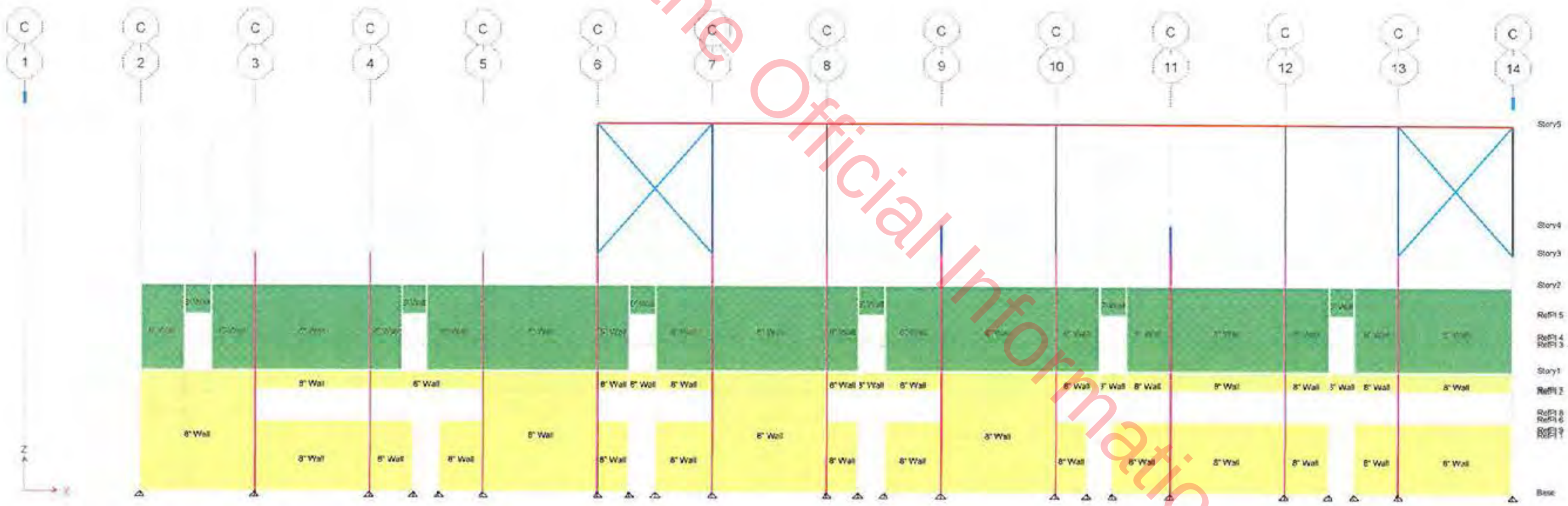
Minimum Requirements

Minimum reinforcement	$A_{v\_min} = 0.7 \times 1000kg / 1mm / 1s^2 \times t \times s_2 / f_{yt}$ = **177** mm^2

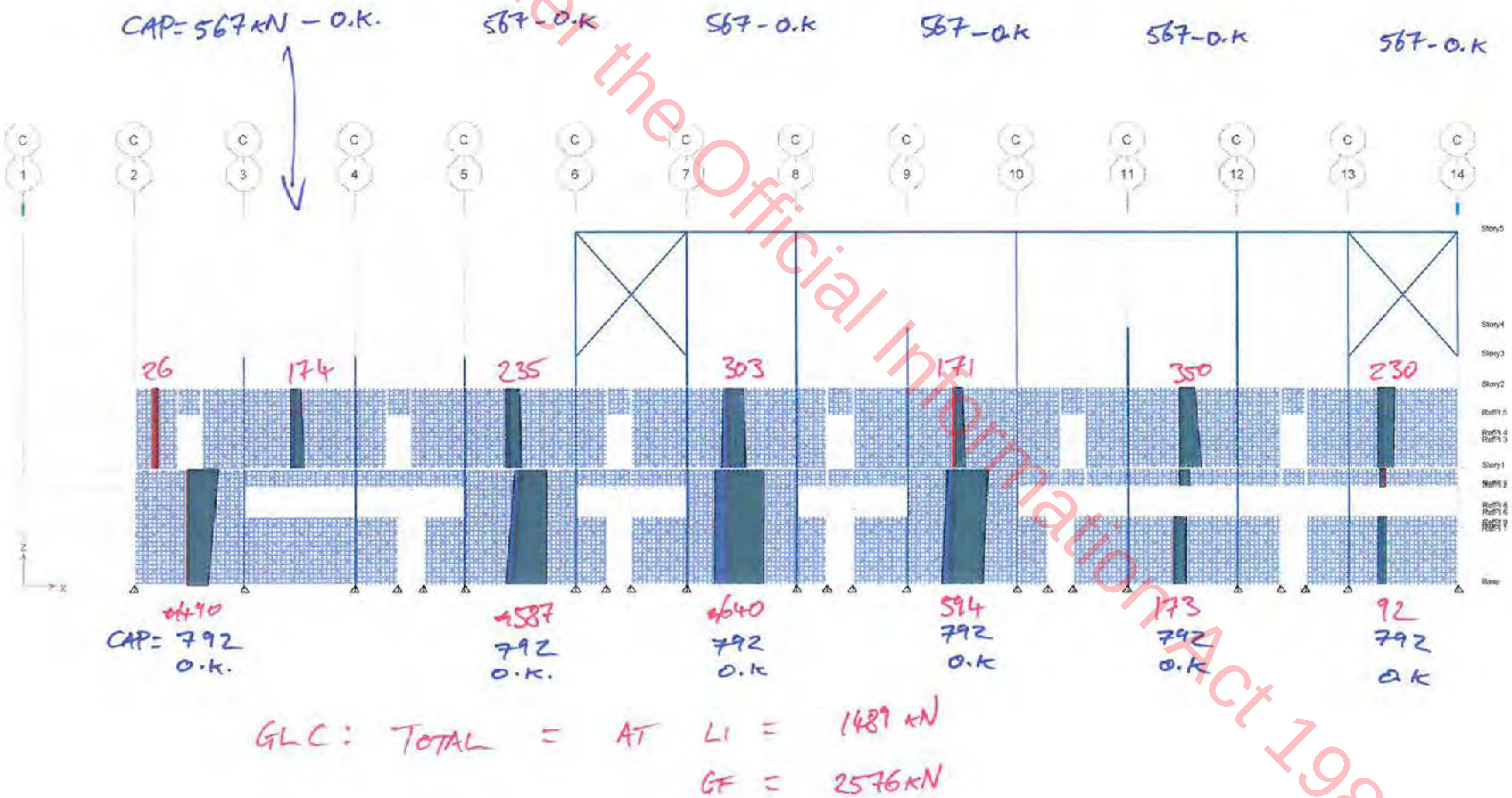
Max Allowable Shear Stress

Shear stress	$v = V / (L_w \times t)$ = **0.001** MPa
Max nominal shear stress	$v_{max} = \min(0.2 \times f_c, 8MPa)$ = **6.000**MPa

8/07



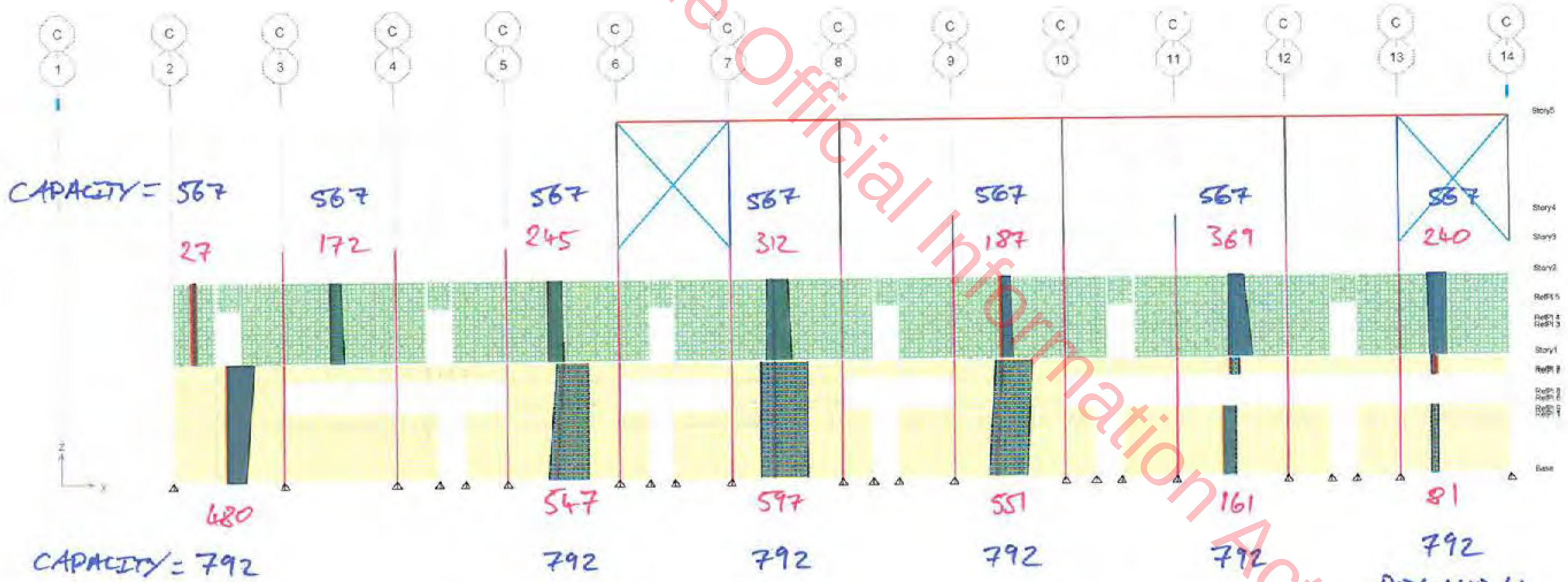
8/08



Elevation View - C Shear Force 2-2 Diagram (EQENV) [kN]

MODEL V2.2 - EBF ON GLA DISCONNECTED

~~GLA~~ IN-PLANE SHEARS



∴ CAPACITY OF WALLS ON THIS LINE EXCEED ~~CAPACITY~~ DEMAND !!

S/09

Elevation View - C Shear Force 2-2 Diagram (EQENV) [kN]

5/10


D
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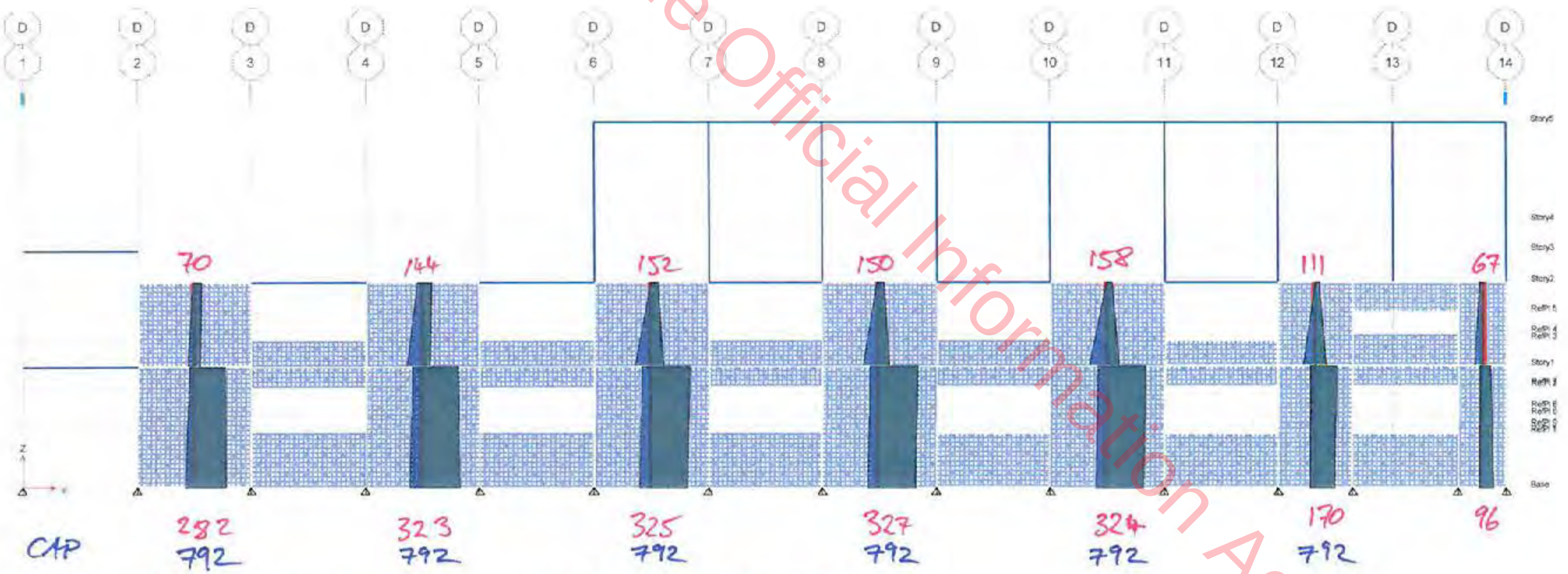
14
Story5
Story4
Story3
Story2
RefPl 5
RefPl 4
RefPl 3
Story1
Base
8" Wall

S/11



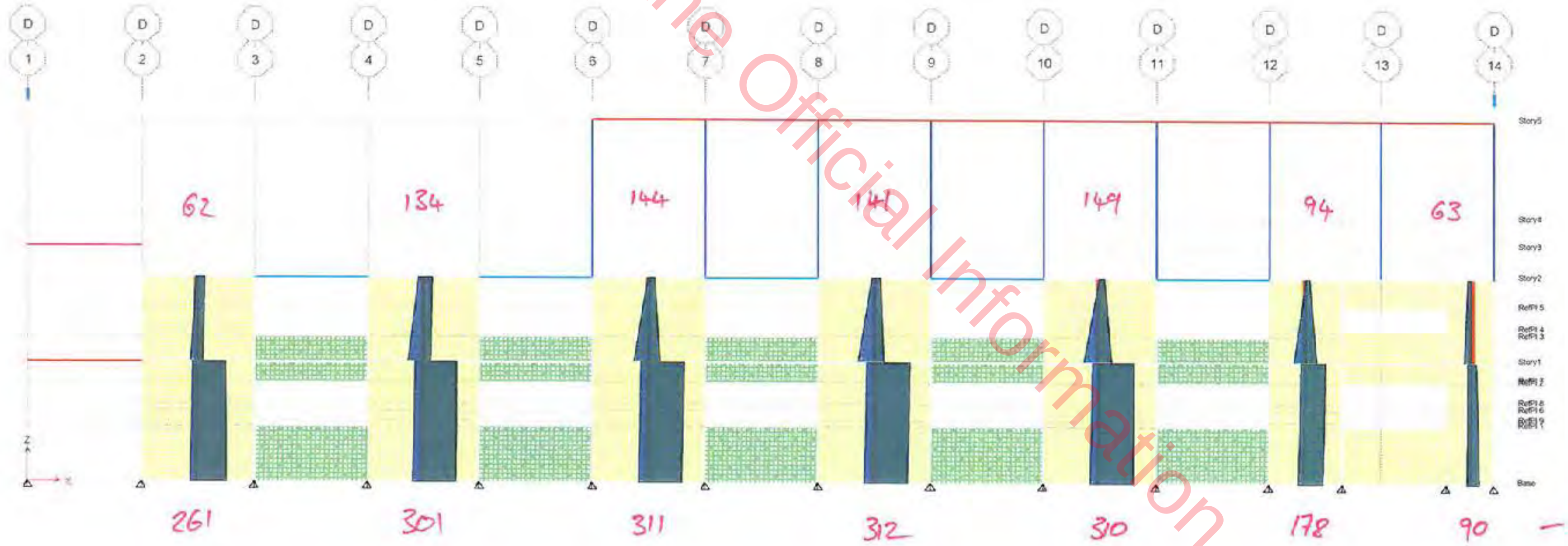
GLD TOTALS : L1 = 852 kN

GF = 1847 kN

All PIERS > 100%

MODEL v 2.2 - CBF DISCONNECTED

GL D IN-PLANE SHEARS



– Σ = 1763 kN

S/12

S/13

OPUS Opus International Consultants PO Box 12003 Wellington 6144	Project Wall Shear to NZS3101			Job Ref.	
	Section GL1 8" Thick Wall			Sheet no./rev. 1	
	Calc. by PMO	Date 18/09/2015	Chk'd by	Date	App'd by	Date

Shear Capacity of Concrete Wall

Axial compressive load	$N = 1$ kN
Shear action	$V = 1$ kN
Concrete compressive strength	$f_c = 30$MPa
Length of wall	$L_w = 11.379$m
Wall thickness	$t = 203$ mm
Capacity reduction factor	$\phi = 0.75$
Area of shear reinforcement within	$A_v = 143\text{mm}^2$ (2 layer of 3/8" diameter bars)
a distance	$s_2 = 305$ mm
Yield strength of the shear reinforcement	f_{yt} = 245MPa (33,000 psi x 1.08)
Effective depth of wall	$d = 0.8 \times L_w = \mathbf{9.103}$m
Gross area of section	$A_g = L_w \times t = \mathbf{2309937.000}\text{mm}^2$
Shear area	$A_{cv} = d \times t = \mathbf{1847949.600}\text{mm}^2$
Shear resisted by concrete	$v_c = 0.17 \times (\sqrt{(f_c)} \times 1\ \text{kg}^{0.5} \times 1\ \text{m}^{-0.5} \times 1\ \text{s}^{-1} \times 1000 + N/A_g) = \mathbf{0.931}$ MPa
Concrete shear strength	$V_c = v_c \times A_{cv} = \mathbf{1720.814}$ kN
Shear resisted by reinforcement	$V_s = A_v \times f_{yt} \times d / s_2 = \mathbf{1045.674}$kN
Probable shear resistance	$V_p = \phi \times (V_c + V_s) = \mathbf{2074.866}$ kN

Minimum Requirements

Minimum reinforcement	$A_{v\_min} = 0.7 \times 1000\text{kg} / 1\text{mm} / 1\text{s}^2 \times t \times s_2 / f_{yt} = \mathbf{177}\ \text{mm}^2$

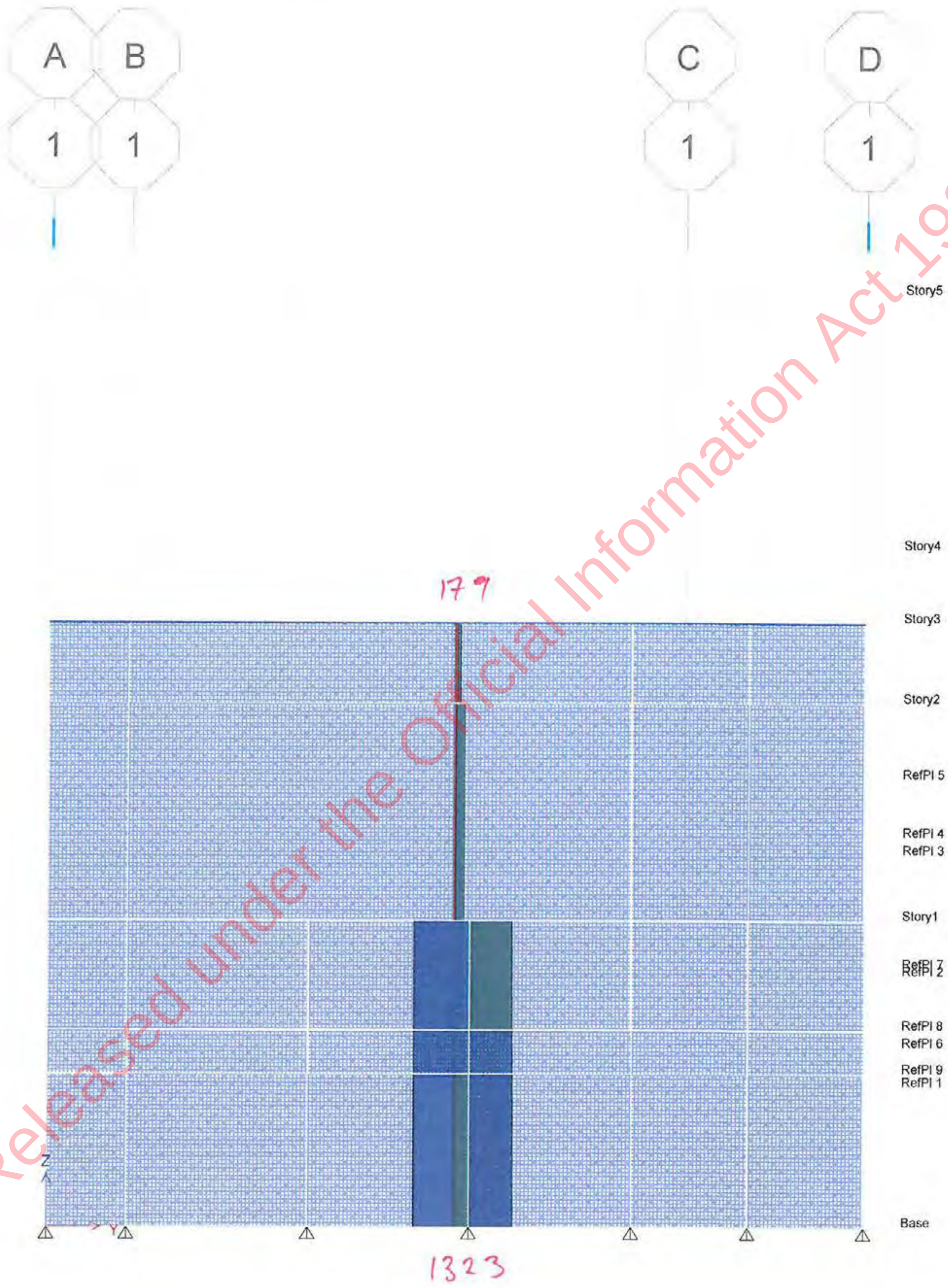
Max Allowable Shear Stress

Shear stress	$v = V / (L_w \times t) = \mathbf{0.000}$ MPa
Max nominal shear stress	$v_{max} = \min(0.2 \times f_c, 8\text{MPa}) = \mathbf{6.000}$MPa

TRANSVERSE WALLS

S/15

GL 1



CAPACITY = 2075 kN > 100%

8/16

OPUS Opus International Consultants PO Box 12003 Wellington 6144	Project: Wall Shear to NZS3101			Job Ref.	
	Section: GL2 8" Thick Wall			Sheet no./rev. 1	
	Calc. by: PMO	Date: 18/09/2015	Chk'd by	Date	App'd by	Date

Shear Capacity of Concrete Wall

Axial compressive load	$N = 1$ kN
Shear action	$V = 1$ kN
Concrete compressive strength	$f_c = 30$MPa
Length of wall	$L_w = 8.869$m
Wall thickness	$t = 203$ mm
Capacity reduction factor	$\phi = 0.75$
Area of shear reinforcement within	$A_v = 143\text{mm}^2$ (2 layer of 3/8" diameter bars)
a distance	$s_2 = 305$ mm
Yield strength of the shear reinforcement	$f_{yt} = 245$MPa (33,000 psi x 1.08)
Effective depth of wall	$d = 0.8 \times L_w = \mathbf{7.095}$m
Gross area of section	$A_g = L_w \times t = \mathbf{1800407.000}\text{mm}^2$
Shear area	$A_{cv} = d \times t = \mathbf{1440325.600}\text{mm}^2$
Shear resisted by concrete	$v_c = 0.17 \times (\sqrt{(f_c)} \times 1\ \text{kg}^{0.5} \times 1\ \text{m}^{-0.5} \times 1\ \text{s}^{-1} \times 1000 + N/A_g) = \mathbf{0.931}$ MPa
Concrete shear strength	$V_c = v_c \times A_{cv} = \mathbf{1341.264}$ kN
Shear resisted by reinforcement	$V_s = A_v \times f_{yt} \times d / s_2 = \mathbf{815.017}$kN
Probable shear resistance	$V_p = \phi \times (V_c + V_s) = \mathbf{1617.211}$ kN

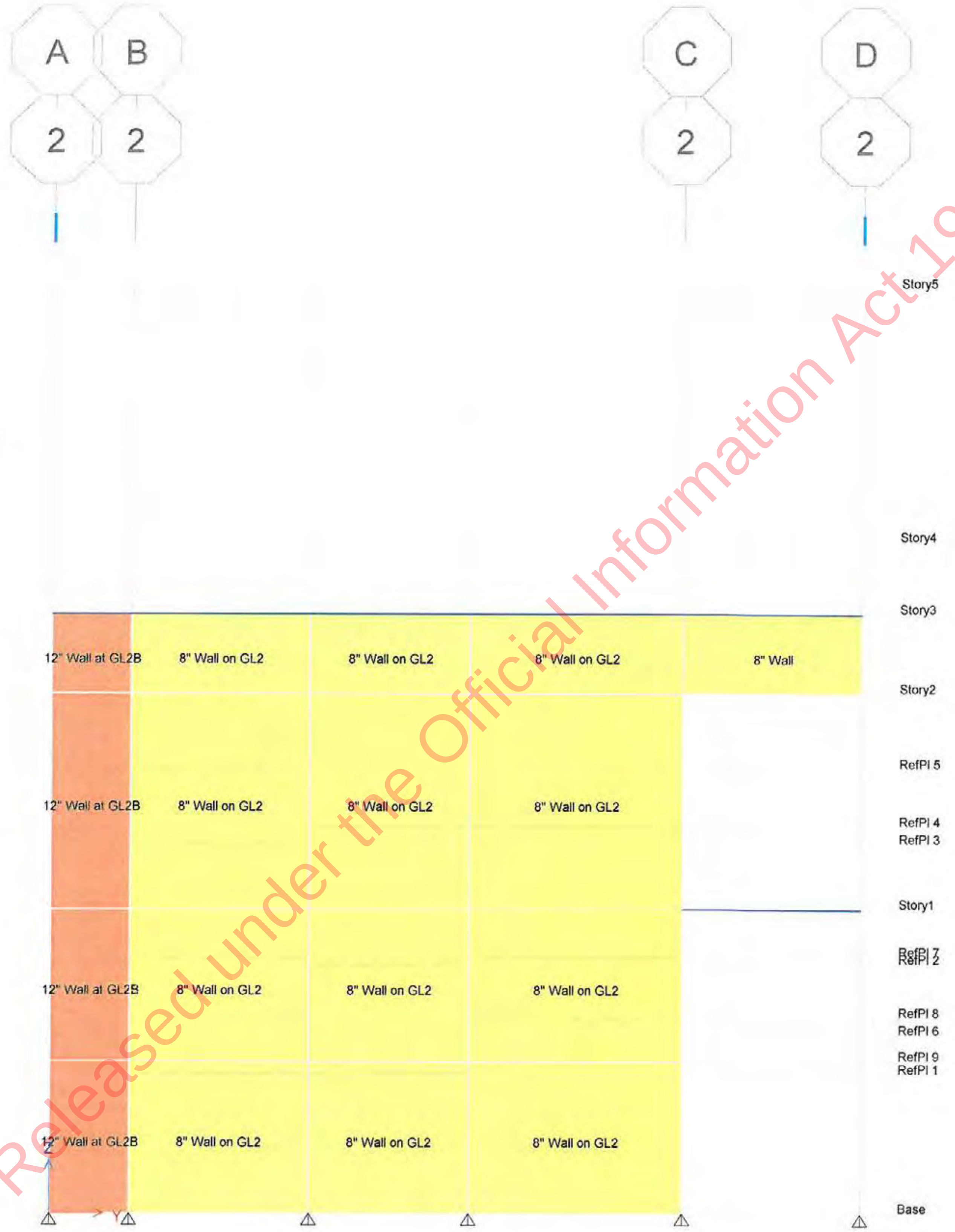
Minimum Requirements

Minimum reinforcement	$A_{v\_min} = 0.7 \times 1000\text{kg} / 1\text{mm} / 1\text{s}^2 \times t \times s_2 / f_{yt} = \mathbf{177}\ \text{mm}^2$

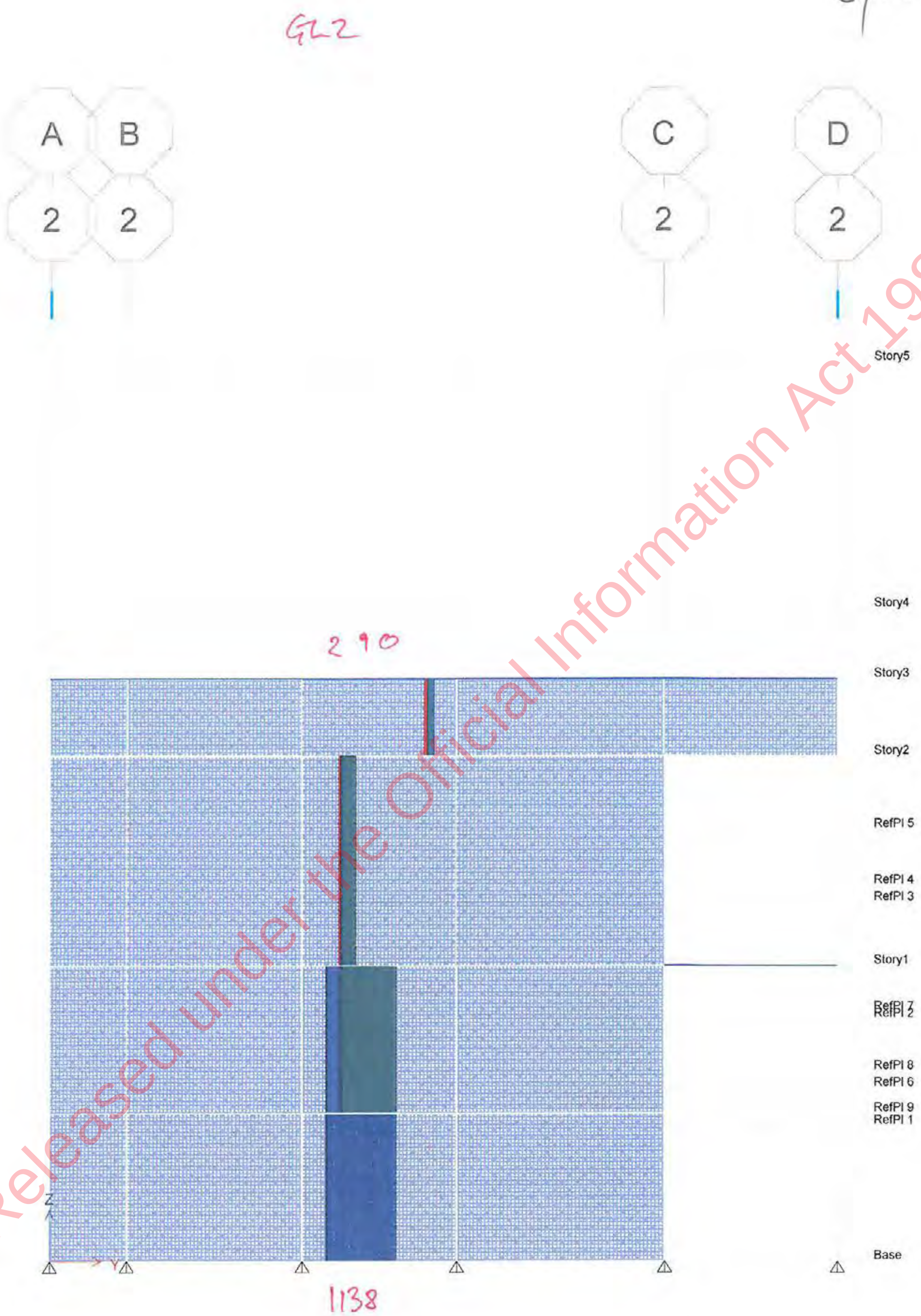
Max Allowable Shear Stress

Shear stress	$v = V / (L_w \times t) = \mathbf{0.001}$ MPa
Max nominal shear stress	$v_{max} = \min(0.2 \times f_c, 8\text{MPa}) = \mathbf{6.000}$MPa

S/17


A
B
C
D
2
2
2
2
Story5
Story4
Story3
12" Wall at GL2B
8" Wall on GL2
8" Wall on GL2
8" Wall on GL2
8" Wall
Story2
RefPl 5
12" Wall at GL2B
8" Wall on GL2
8" Wall on GL2
8" Wall on GL2
RefPl 4
RefPl 3
Story1
RefPl 7
RefPl 2
12" Wall at GL2B
8" Wall on GL2
8" Wall on GL2
8" Wall on GL2
RefPl 8
RefPl 6
RefPl 9
RefPl 1
12" Wall at GL2B
8" Wall on GL2
8" Wall on GL2
8" Wall on GL2
Z
Y
Base

5/18



CAPACITY = 1617 kN > 100%

8/19

Opus International Consultants PO Box 12003 Wellington 6144	Project Wall Shear to NZS3101			Job Ref.	
	Section GL5 8" Thick Wall			Sheet no./rev. 1	
	Calc. by PMO	Date 18/09/2015	Chk'd by	Date	App'd by	Date

Shear Capacity of Concrete Wall – APPLICABLE FOR GL5, GL8, GL12, GL14

Axial compressive load	N = 1 kN
Shear action	V = 1 kN
Concrete compressive strength	f_c = 30MPa
Length of wall	L_w = 7.747m
Wall thickness	t = 203 mm
Capacity reduction factor	ϕ = 0.75
Area of shear reinforcement within	A_v = 143mm^2 (2 layer of 3/8" diameter bars)
a distance	s_2 = 305 mm
Yield strength of the shear reinforcement	f_{yt} = 245MPa (33,000 psi x 1.08)
Effective depth of wall	$d = 0.8 \times L_w$ = **6.198**m
Gross area of section	$A_g = L_w \times t$ = **1572641.000**mm^2
Shear area	$A_{cv} = d \times t$ = **1258112.800**mm^2
Shear resisted by concrete	$v_c = 0.17 \times (\sqrt{(f_c)} \times 1\ kg^{0.5} \times 1\ m^{-0.5} \times 1\ s^{-1} \times 1000 + N/A_g)$ = **0.931** MPa
Concrete shear strength	$V_c = v_c \times A_{cv}$ = **1171.600** kN
Shear resisted by reinforcement	$V_s = A_v \times f_{yt} \times d / s_2$ = **711.911**kN
Probable shear resistance	$V_p = \phi \times (V_c + V_s)$ = **1412.634** kN

Minimum Requirements

Minimum reinforcement — $A_{v\_min} = 0.7 \times 1000kg / 1mm / 1s^2 \times t \times s_2 / f_{yt}$ = **177** mm^2

Max Allowable Shear Stress

Shear stress — $v = V / (L_w \times t)$ = **0.001** MPa

Max nominal shear stress — $v_{max} = \min(0.2 \times f_c, 8MPa)$ = **6.000**MPa

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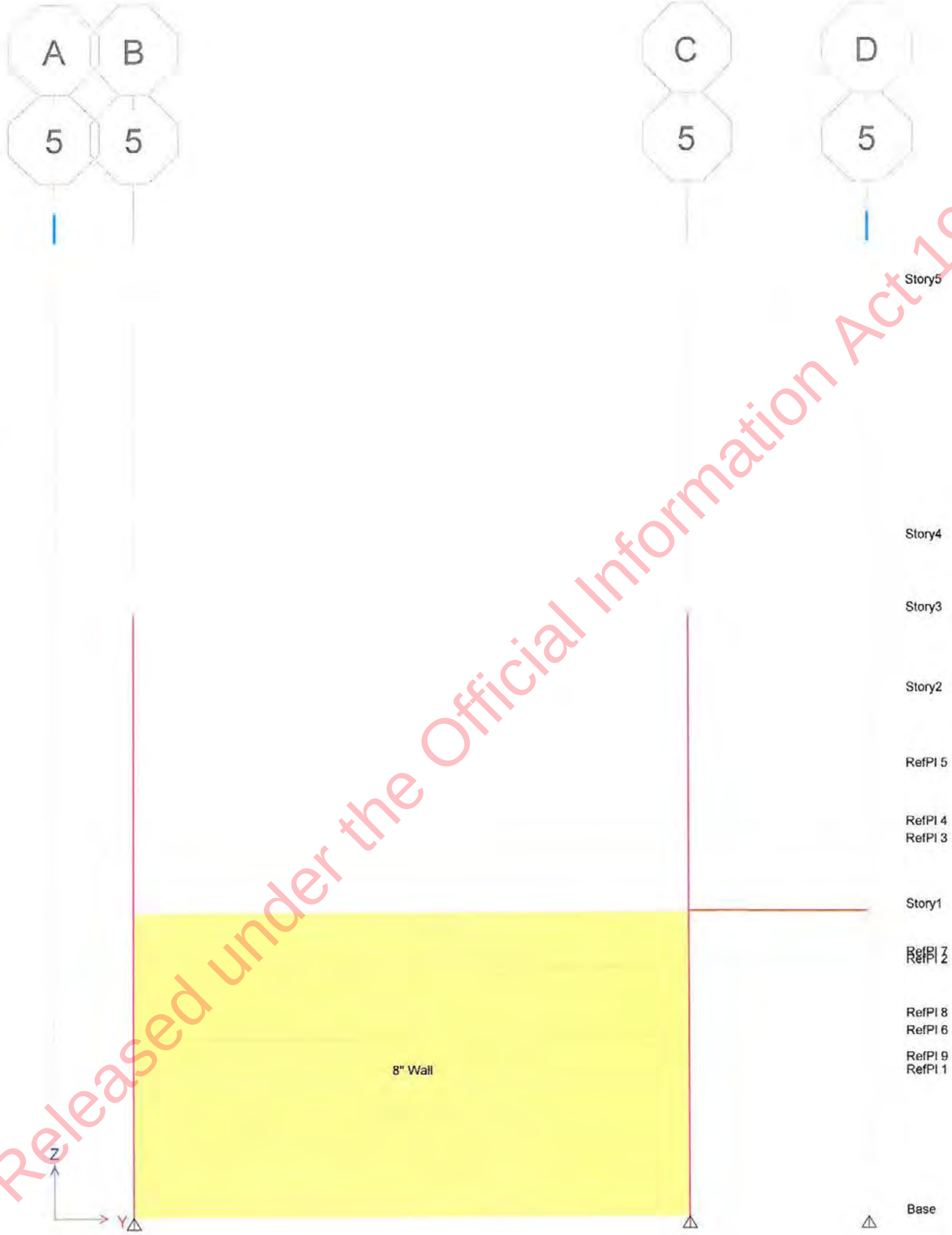

A
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Story5
Story4
Story3
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RefPl 5
RefPl 4
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Story1
RefPl 7
RefPl 2
RefPl 8
RefPl 6
RefPl 9
RefPl 1
8" Wall
Z
Y
Base

GL5

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1113

CAPACITY = 1413 kN > 100%

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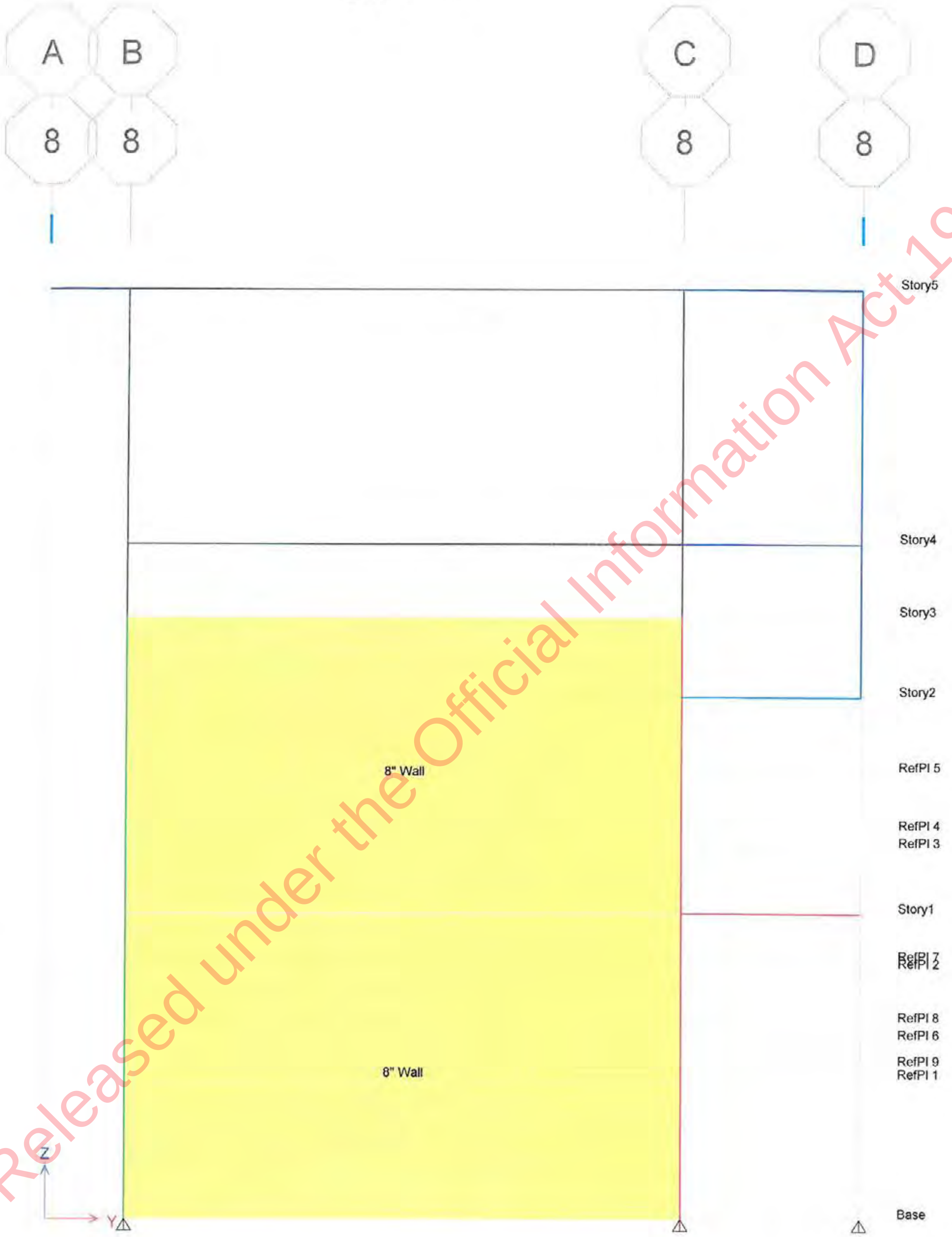
GL 8



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Story5
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Story2
RefPl 5
8" Wall
RefPl 4
RefPl 3
Story1
RefPl 7
RefPl 2
RefPl 8
RefPl 6
RefPl 9
RefPl 1
8" Wall
Z
Y
Base

GL 8

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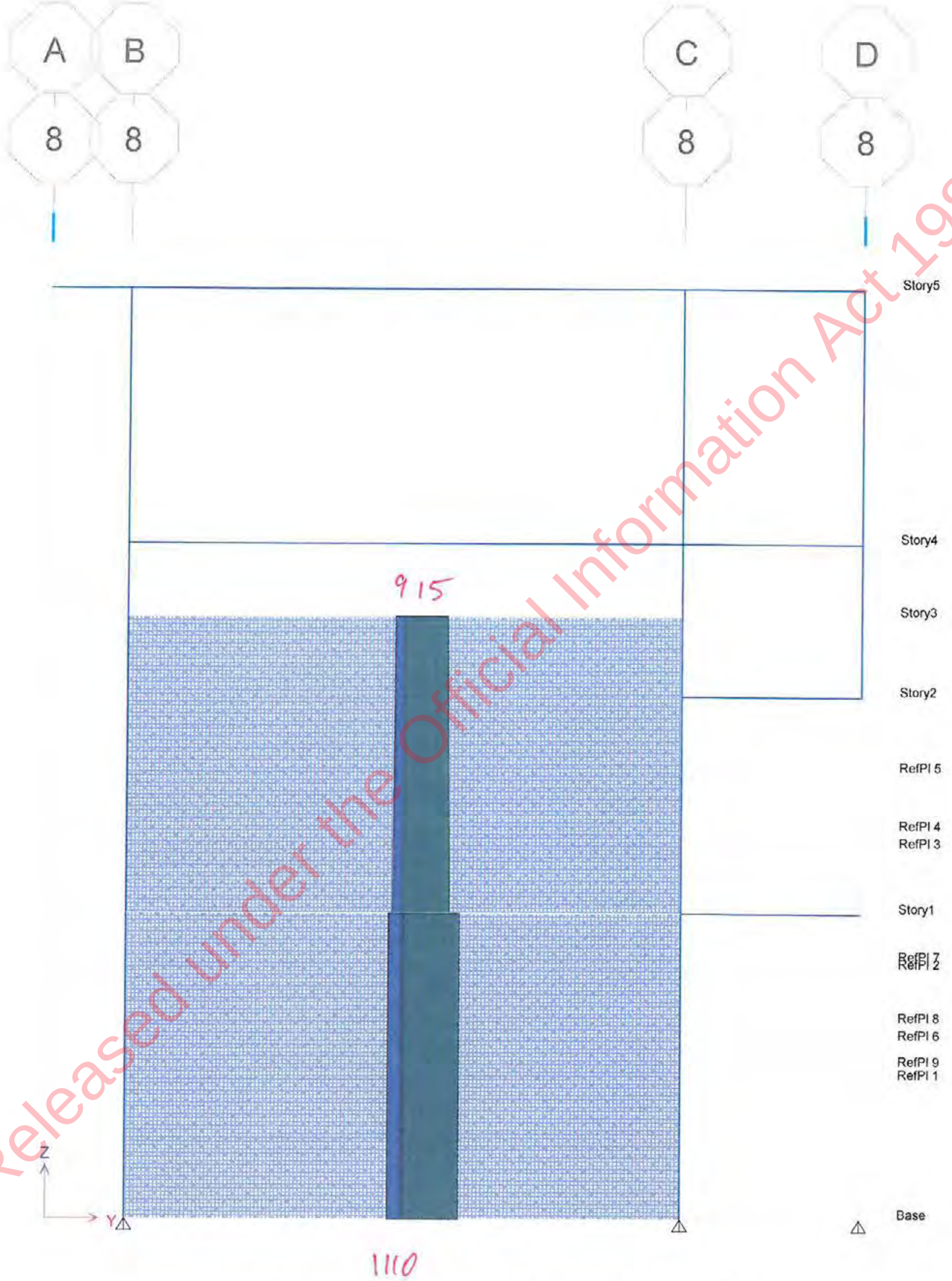

CAPACITY = 1413 kN > 100%

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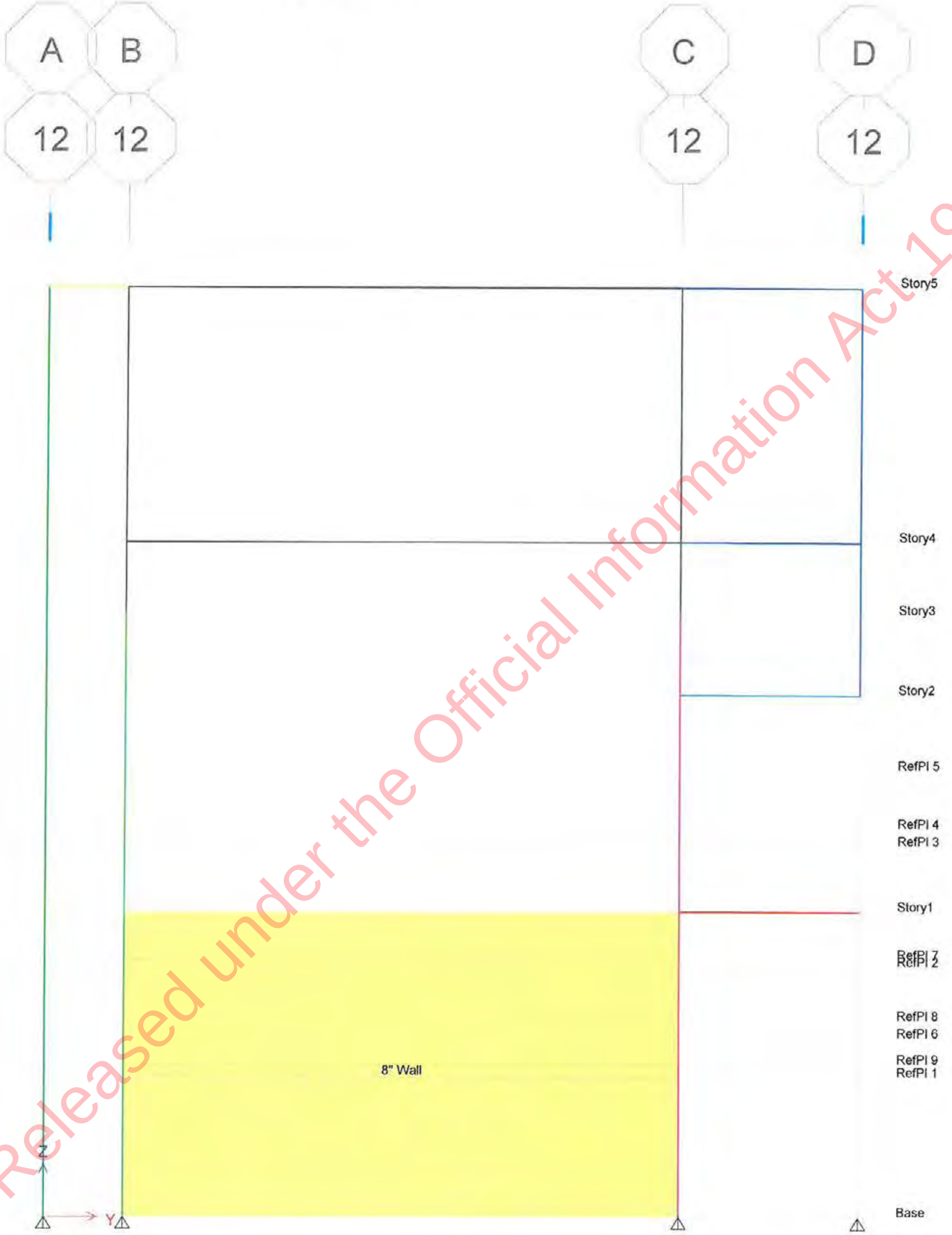
GL 12


A
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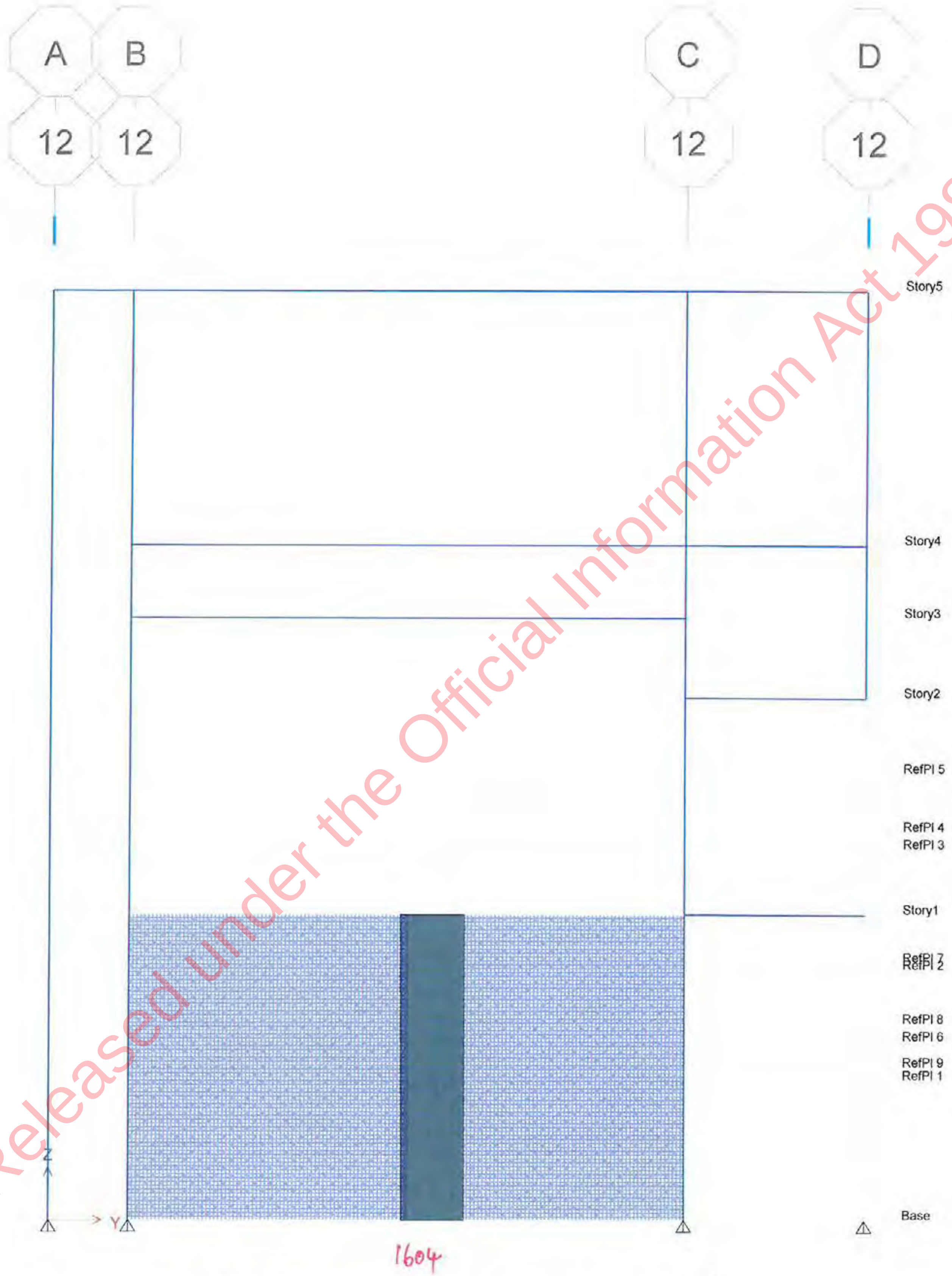
Story5
Story4
Story3
Story2
RefPl 5
RefPl 4
RefPl 3
Story1
RefPl 7
RefPl 2
RefPl 8
RefPl 6
RefPl 9
RefPl 1
8" Wall
Z
Y
Base

GL12 S/25



CAPACITY = 1413 kN → 88%

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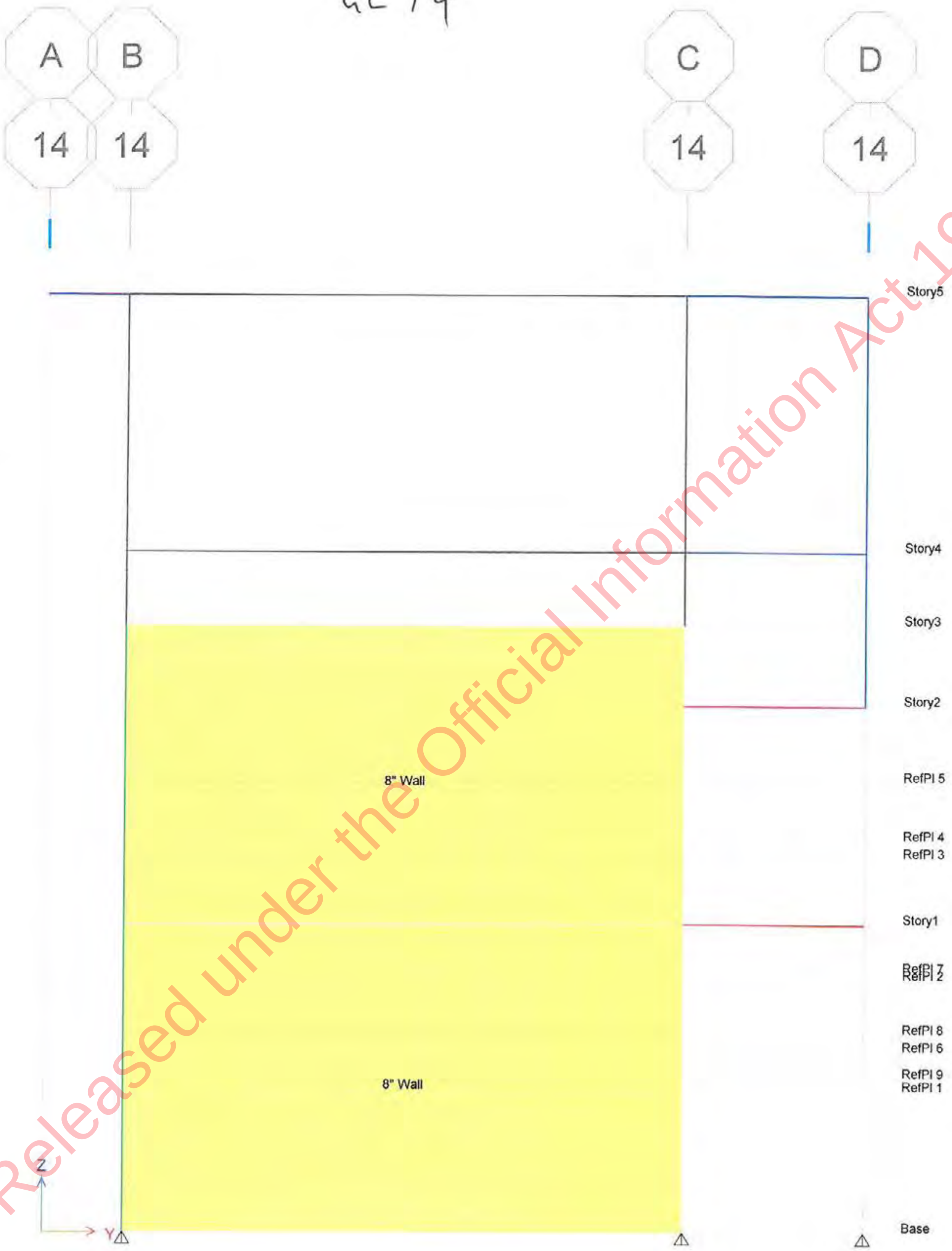
GL 14



A
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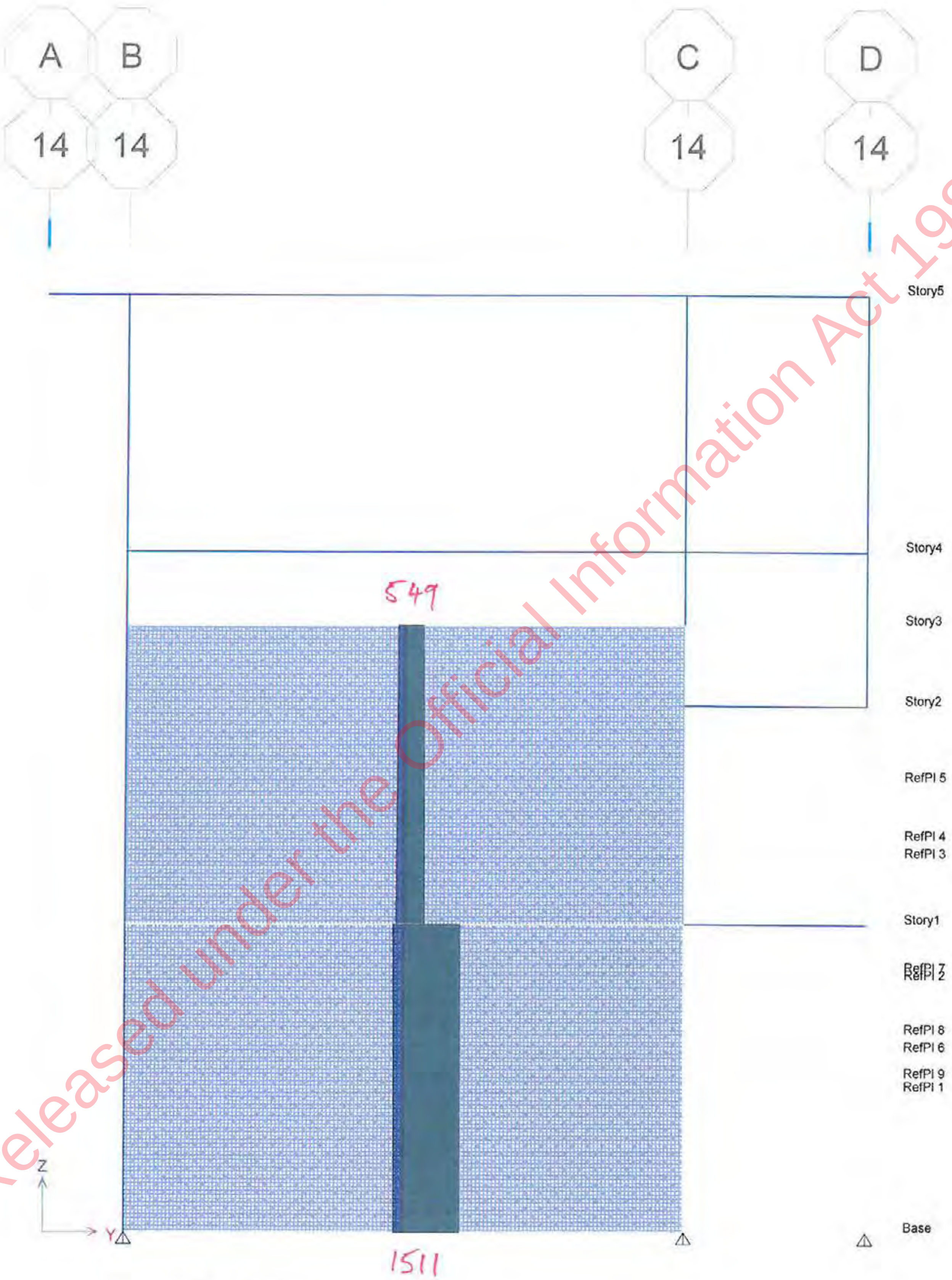
Story5
Story4
Story3
Story2
RefPl 5
RefPl 4
RefPl 3
Story1
RefPl 7
RefPl 2
RefPl 8
RefPl 6
RefPl 9
RefPl 1
Base
8" Wall
8" Wall
Z
Y

S/27

GL14



CAPACITY = 1413 kN → 94%

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CALCULATION SHEET

Project/Task/File No: WEGC EAST BLOCK DSA | Sheet No | of
Project Description: | Office:
Computed: PMO 18/10/2015
Check: / /

CONCRETE WALLS IN FLEXURE

The concrete walls are all relatively long and squat, with aspect ratios:

Transverse wall: 2-storey =
Height = 8.414m
Length = 11.379m
→ Aspect ratio: 0.74

Incidentally, more force 'inputted' into the wall at 1st floor level, so aspect ratio actually more like 0.4.

↳ By inspection, shear governed.

But check moment demand vs. capacity for typical wall:

TYPICAL WALL CHECK
WALL ON GL 12 = 8" THICK

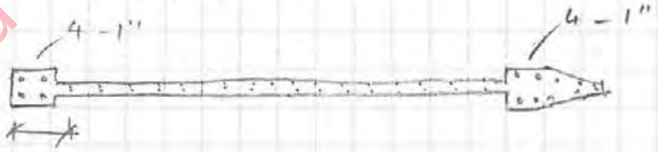
V^* = 1604 kN → S/25 (88% NBS based on shear capacity)
h = 4.267 m

∴ M^* = 6844 kNm → μ = 1.25, S_p = 0.9

Original drawings = Wall G

2-layers 3/8" Bars @12" c/c each way, plus end columns

Length = 7.747m



Axial load = 307 kN min → 414 kN max
↳ Assume 350 kN

M_n = 6240 kN

∴ 91% based on μ = 1.25 loads. (Better than for shear ↳ 88%.)

Allowing some yielding, probably > 100%

OPUS

S/27c

GL 12 WALL
FLEXURAL CHECK

WELCOME TO CONPROP(V 1.7) ** AN EXCEL SPREADSHEET FOR CALCULATING
GEOMETRIC PROPERTIES & MOMENTS FOR CONCRETE SECTIONS UNDER
UNCRACKED, CRACKED & ULTIMATE CONDITIONS.

[Flanges (top and/or bottom) and axial load are optional. For walls/columns see note 2 or the MANUAL.]

Checked : ________ /____/____

Project:		
	Date:	Time:
	08-Oct-15	13:26

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

Total Section depth (d) = ...	**7747**		mm
Web width (w) = ...	**203**		mm
Top flange width **excluding** web (b1) = ...	**102**	<----	mm
Top flange thickness (t) = ...	**406**	THESE	mm
Bottom flange width **excluding** web (b2) = ...	**102**	6 values	mm
Bottom flange thickness (b) = ...	**406**	may	mm
Axial compressive load (P) and, ...	**350,000**	be	N
Depth from top surface of this load (di)	**3873.5**	zero	mm
Assumed tensile cracking stress (f't)	**0**	<-----	N/mm2
Steel Elastic Modulus (Es)	**200,000**		N/mm2

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	$= E_s/E_c$

TOP BARS

No. Bars	Bar Diam	Distance From Top Surface
4	25.40	203.00
2	9.53	558.00
2	9.53	1160.00
2	9.53	2064.00
2	9.53	2968.00
2	9.53	3873.00
0	0.00	0.00
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
4	25.40	203.00
2	9.53	558.00
2	9.53	1160.00
2	9.53	2064.00
2	9.53	2968.00
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

mm

Checked : ________ /____ /____

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)	**0.03**
Ratio of (stress block)/(N.A.) depths........	**0.85**
Axial compressive load (P) and, ...	**350,000**
depth from top surface of this load (di)	**3873.5**
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)........	**245**

=3320(f_c)

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)........	2.02E+02		
Steel Stress (Maximum Tension)........	2.45E+02		
Crack Depth	7.55E+03		
Total Tension Force (including P)........	1.25E+06	Ratio T/C =	
Total Compression Force -incl. comp steel........	1.33E+06	0.937	Mn
Ideal Flex strength (Mi)---SEE NOTE 2........	6.24E+09	(=1.0 for iteration	**6240**
Section Curvature (from curv = e/c)........	1.49E-04	convergence)	θMn
			5304
			KNm

(b) CRACK PROPAGATING FROM TOP

Depth to N.A.(zero stress) from bottom (c)........	2.02E+02		
Steel Stress (Maximum Tension)........	2.45E+02		
Crack Depth	7.55E+03		
Total Tension Force (including P)........	1.25E+06	Ratio T/C =	
Total Compression Force -incl. comp steel........	1.33E+06	0.937	Mn
Ideal Flex strength (Mi)---SEE NOTE 2........	6.24E+09	(=1.0 for iteration	**6240**
Section Curvature (from curv = e/c)........	1.49E-04	convergence)	θMn
			5304
			KNm