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DETAILED SEISMIC ASSESSMENT REPORT

RANGITIKEI DISTRICT COUNCIL

90 HAUTAPU STREET, TAIHAPE

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Document Control Record

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Contents

1 Executive Summary

1.1 Background

Resonant Consulting Ltd (Resonant) has been commissioned by the Rangitikei District Council to undertake a Detailed Seismic Assessment (DSA) of the town hall located at 90 Hautapu Street, Taihape. The aim of the assessment is to determine the seismic rating of the building in relation to the New Building Standard (%NBS).

1.2 Building Description

The town hall has the public library attached to the north-west wall. The library is assessed in a separate report.

There is an attached toilet block attached to the auditorium/fly tower. The building appears to have been constructed in stages between 1900 and 1920. Alterations to the roof structure at the front of building carried out in 1945.

The building is currently used as offices. The auditorium and fly tower are used for various events.

The roof structure consists of iron cladding supported on timber purlins and trusses. The front part of the building contains unreinforced brick masonry walls and piers. Alterations to the front façade carried out in 1943 consisted of the construction of a reinforced concrete ring beam at roof level with reinforced concrete parapets on the front elevation of the building. Reinforced concrete roof gussets (four) were also constructed to tie the brick walls together.

1.3 Assess Seismic Rating

The assessment has been completed in accordance with the New Zealand Society of Earthquake Engineering document - Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments, dated July 2017. The seismic rating assumes that Importance Level 2 (IL2), in accordance with the joint Australian/New Zealand Standard – Structural Design Actions Part 0, AS/NZS 1170.0:2002, is appropriate. Refer to Table 1 for a summary of the buildings seismic rating.

Table 1.3.1 - Summary of Seismic Ratings for 90 Hautapu Street

90 Hautapu Street		
Direction	**Seismic Rating (%NBS)**	**Seismic Grade**
Transverse	< 10	E
Longitudinal	20	E

The Seismic Grade has been determined in accordance with the NZSEE grading scheme. The overall building seismic rating for the building is governed by the brick pier (front façade) in-plane bracing capacity in the transverse direction. The longitudinal direction rating is governed by the out of plane capacity of the first-floor brick wall on the north-west elevation. Refer to Section 8 for a summary of the % NBS scores, and commentary, for the various building structure components and to Appendix C for a Technical Summary Report.

1.4 Basis for the Assessment

The assessment has been based on the following information:

- C. G. Talboys and Associates Registered Architects structural drawing no. 328 'Strengthening and Modernizing Taihape Town dated July 29th, 1943.
- Southcombe McClean & Company architectural drawings for Taihape Town Hall & Library Upgrade dated 1995.
- Structural Calculations by Powell Sewell Ltd dated 5th October 1995.
- Taihape Town Hall Structural Report by Kevin O'Connor & Associates dated 22 December 2009.

1.5 Seismic Retrofit Options

A concept strengthening scheme, to achieve a capacity 67 %NBS rating, has been enclosed in Section 11.

The following elements limiting the capacity below 67% NBS:

- Lack of diagonal roof bracing and ceiling diaphragms, particularly in the auditorium and fly tower roofs.
- Inadequate bracing of the timber framed walls in the auditorium and fly tower.
- Brick piers and walls, particularly on the front elevation.

2 Introduction

2.1 Overview

The Rangitikei District Council has engaged Resonant to assess the seismic capacity of the town hall located at 90 Hautapu Street, Taihape. The intention of the assessment is to determine the buildings' ability to withstand earthquake loads in terms of the current New Zealand Building Standards and yield a score for the building expressed as "Percentage New Building Standard" (%NBS).

2.2 Scope of Work

As identified in our proposal dated 31st August 2021, the scope of works to be undertaken as part of the assessment:

- Site Inspection and Information Gathering.
- Analytical Work (Calculations), in which an estimate of the seismic rating (%NBS) is achieved.
- Provide a written report outlining the findings of the assessment.
- Provision of a concept strengthening scheme.

2.3 Sources of Information

The assessment of 90 Hautapu Street is based on the following information:

- Structural drawing No. 326 by E G Talboys & Associates, Registered Architects and dated June 29th, 1945.
- Architectural Drawings by Southcombe McClean & Company tiled 'TAIHAPE TOWN HALL & LIBRARY UPGRADE' numbered WD1 to WD17 and dated 1995.
- On-site inspections completed on 22 November 2021.

All the documents have been obtained from the Rangitikei District Council Property File. No geotechnical report was available.

2.4 Site Investigation

A non-intrusive site investigation was carried out to confirm the information in the available documentation.

2.5 Exclusions

This report does not extend to an assessment of non-structural items such as cladding, ceilings, partitions, other fit-out related items, geotechnical ground conditions and latent defects.

It should be noted that for the purposes of this assessment the %NBS refers to the capacity and performance of the lateral load resisting system only. As Building Codes have evolved it is likely that an older building may not meet current Code requirements for aspects such as access and moisture detailing.

3 Background Regulations

3.1 Building Act 2004 and Earthquake Prone Buildings Amendment Act 2016

Before describing how the seismic analysis was completed, the regulatory requirements and definitions for earthquake prone buildings should be discussed.

The Building (Earthquake-prone Buildings) Amendment Act 2016 introduced major changes to the way earthquake-prone buildings are identified and managed under the Building Act.

Earthquake-prone Buildings

Under section 133AB of the Building Act (2004), the definition of earthquake-prone building is:

- A building or a part of a building is earthquake prone if, having regard to the condition of the building, or part, and to the ground on which the building is built, and because of the construction of the building or part
  - the building or part will have its ultimate capacity exceeded in a moderate earthquake, and
  - if the building or part were to collapse, the collapse would be likely to cause:
    - injury or death to persons in or near the building or on any other property, or
    - damage to any other property
- The above does not apply to a building that is used wholly or mainly for residential purposes unless the building:
  - comprises 2 or more storeys; and
  - contains 3 or more household units

A "moderate earthquake" is defined in Section 7 of the Building Regulations 2005 –

"…moderate earthquake means, in relation to a building, an earthquake that would generate shaking at the site of the building that is of the same duration as, but that is one-third as strong as the earthquake shaking (determined by normal measures of acceleration, velocity, and displacement) that would be used to design a new building at that site."

Whether a building, or part of a building, is earthquake prone is determined by the territorial authority in whose district the building is situated.

For the purposes of the above subsection ultimate capacity and moderate earthquake have the meanings given to them by regulations. To assist with application, both ultimate capacity and moderate earthquake are terms defined in the Building (Specified Systems, Change the Use, and Earthquake-prone Buildings) Regulations 2005 (as amended).

These regulations define ultimate capacity as "The probable capacity to withstand earthquake actions and maintain gravity load support assessed by reference to the building and its individual elements or parts" and moderate earthquake as "In relation to a building, an earthquake that would generate shaking at the site of the building that is of the same duration as, but that is one-third as strong as, the earthquake shaking (determined by normal measures of acceleration, velocity, and displacement) that would be used to design a new building at that site if it were designed on 1 July 2017."

3.2 Ratings

The ratings provided within this report have been generated with respect to New Zealand Society for Earthquake Engineering (NZSEE) guidelines. They are often summarised as "%NBS rating" which reflects the design coefficient for a similar building designed today to current codes, referred to as the New Building Standard (NBS).

Per the NZSEE publication "The Seismic Assessment of Existing Buildings", Section A3.2.4 groups building ratings as follows:

Table 3.2.1 NZSEE Grading Scheme

Percentage of New Building Standard *(%NBS)*	Alpha rating	Approx. risk relative to a new building	Life-safety risk description
>100	A+	Less than or comparable to	Low risk
80-100	A	1-2 times greater	Low risk
67-79	B	2-5 times greater	Low to Medium risk
34-66	C	5-10 times greater	Medium risk
20 to <34	D	10-25 times greater	High risk
<20	E	25 times greater	Very high risk

It should be noted that the demarcation between a C and D rating, 33% NBS, is aligned with the Building Act of 2004. Although these ratings are calculated in a linear manner, they are meant to represent an exponential scale of earthquake risk.

4 Building Descriptions

4.1 General Building Descriptions

The building is located at 90 Hautapu Street, Taihape and was most likely constructed between 1900 and 1920. For the purposes of this report, the building has been divided into four parts as indicated below:

Area 1 – front 2 storey building extending about 5m from the street front.

Area 2 – 2 storey rear area of the part 1 building containing ground floor meeting room and amenities, first floor projector room and seating bleachers.

Area 3 – hall area.

Area 4 – stage and Fly Tower.

Construction of these areas is as follows:

Roof - the roof is constructed from timber framing and is clade with corrugated iron throughout. There are Dutch gables over areas 1 and 4 and with a simple gable roof over Areas 2 and 3 between. Most of the roof to Area 1 is propped off internal walls except for two half trusses spanning from the main frontage of the building back to the first major truss across the hall. Purlins consist of 150 mm x 50mm timber. There are four triangular 6-inch reinforced concrete gussets in the roof plane at the 'wings' of the front of the building which tie in the reinforced concrete parapets.

Ceiling joists span between these half trusses and the internal walls below. The ceiling appears to be fibrous plaster. Throughout Areas 2 & 3, the trusses are at 2.7m spacing and span across the full 15.7m width of the building. The trusses are fixed to the exterior wall top plate with a steel strap. The purlins appear to have no blocking. Ceiling joists span between the truss chords with a slatted timber substrate providing support to the textured fibrous plaster ceiling.

Roof area 4 consists of timber trusses at 2.7m centres spanning 9.3m between the proscenium arch and the rear wall of the stage. There are no straps to fix the trusses to the wall framing. The bottom chord of the trusses has had steel channel strengthening added at some stage. This was presumably to support the inclusion of a loading bridge' within the trusses. No roof plane bracing was observed in any of the four roof areas.

Sloping Floor / Seating Area (Area 2) – It was not possible to ascertain the construction in this area as the linings conceal the structure. Flooring is tongue and groove and there is fixed seating in this area. The floor overhangs the hall area by about 1.5m.

Walls – walls in Area 1 are of unreinforced clay brick construction. The actual construction of the walls could not be determined; however, it is likely that the wall consists of a double thickness brick wall with a veneer brick skin separated by a cavity.

In Area 2 the timber framed walls are clad with timber weatherboard externally and gib board internally.

Walls in Area 3 have a 7.8m stud height and comprise full height 150mm x 50mm studs. Linings are timber weatherboards externally and timber match lining internally. The walls in Area 4 are 150mm x 50mm timber studs at 450 centres. The full height of the fly tower is 13.8m (includes offices below the stage). These studs appear to be joined at approximately mid height above the stage using a bolted lapped splice coinciding with a stage deck/catwalk that extends around these walls (excludes wall to proscenium arch.

Diagonal timber braces of 150mm x 50mm are cut between the studs extending from the stage floor to roof level in a cross pattern to all three walls. The stage deck/catwalk consists of timber joists fixed to the wall framing with braces and handrails. The handrails double as a locking rail for curtain/scene ropes.

The proscenium arch is unsupported by a 400mm x 100mm Oregon lintel spanning 9.0 m. A full height brick chimney has been constructed inside Area 4. It appears to be attached only to the timber framed walls. The exterior linings are timber weatherboard. There are no interior linings to the stage and fly tower.

Floors and Foundations – the ground floor throughout is timber tongue and groove. It was not possible to access the subfloor area of Area 1. The foundation to Area 2 is made up of clay brick perimeter walls transversely and 150mm x 50mm timber jack framing longitudinally.

Intermediate 100m m x 75mm jack piles on 250mm square concrete plinths support the 100mm x 75mm bearers and the 150mm x 50mm joists. Joists are at 450mm centres and bearers at are at 1500mm centres.

Area 3 consists of an in situ concrete perimeter foundation with 150mm 50mm timber jack framing extending to the underside of the hall floor. There is no bracing to the jack framing. The exterior is clad in timber weatherboards to match the linings above.

Intermediate 200mm square concrete jack piles (1.4m max. height) support the 150mm x 100mm bearers and 150mm x50mm joists. It was not possible to gain access to the area beneath the stage.

There is no subfloor bracing present whatsoever.

Stairs – the internal stairs are timber framed and lead from the ground floor offices in area 1 up to the offices above.

There are 6 sets of external stairs leading up to the 1st floor areas and/or the ground floor (required due to the Kuku Street slope.). These are all timber framed and generally comprise timber support posts with timber stringers and treads.

Seismic Resisting Systems

In longitudinal direction, the lateral earthquake loading is resisted the timber framed walls in the auditorium and fly tower and by the unreinforced brick masonry walls in the front part of the building. In the transverse direction, the lateral earthquake loading is resisted the timber framed walls in the auditorium and the fly tower and by the unreinforced brick piers and spandrel beams on the façade at the front of the building. This assessment covers seismic loading as the only lateral loads and does not address wind loading on the structure.

Longitudinal and Transverse Directions

There is no bracing in the roof structure. There is no bracing provided for the sub-floor piles.

Foundations

The substructure consists of sub-floor piles. The perimeter foundation consists of reinforced concrete foundation walls.

5 Geotechnical Conditions

No geotechnical report for the site was available.

6 Seismic Analysis

6.1 Seismic Parameters

Building Ductility

Ductility is a measure of the ability of a building to resist the earthquake forces/energy by inelastic deformation. Under current design standards the level of ductility is generally determined by:

- Identifying an appropriate mechanism that can sustain inelastic deformations without leading to collapse of a building
- The ability to achieve an appropriate level of structural detailing to ensure that the chosen ductile mechanism is achievable

The ductility factor μ = 1.0 was selected for the unreinforced masonry walls. A ductility factor μ = 3.0 was chosen for the timber walls in the auditorium and fly tower.

Site Geology

The site geology can have a significant impact on the level of loading imparted on a building during an earthquake. Deep, soft soil conditions tend to amplify the ground motions, increasing the forces on a building structure. The assumed subsoil Class is D classification since no geotechnical report is available for this site.

Importance Level

The Importance Level of a building is a classification from NZS1170.0. Increasing importance levels trigger higher factors of safety in design or analysis. The building is designated Importance Level 2 (IL2). The building is a multi-occupancy commercial building, however as the total expected occupancy is less than 5000 people it is not classified as IL3.

TABLE 3.2

IMPORTANCE LEVELS FOR BUILDING TYPES—NEW ZEALAND STRUCTURES

Importance level	Comment	Examples
1	Structures presenting a low degree of hazard to life and other property	Structures with a total floor area of <30 m^2 Farm buildings, isolated structures, towers in rural situations Fences, masts, walls, in-ground swimming pools
2	Normal structures and structures not in other importance levels	Buildings not included in Importance Levels 1, 3 or 4 Single family dwellings Car parking buildings
3	Structures that as a whole may contain people in crowds or contents of high value to the community or pose risks to people in crowds	Buildings and facilities as follows: (a) Where more than 300 people can congregate in one area (b) Day care facilities with a capacity greater than 150 (c) Primary school or secondary school facilities with a capacity greater than 250 (d) Colleges or adult education facilities with a capacity greater than 500 (e) Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities (f) Airport terminals, principal railway stations with a capacity greater than 250 (g) Correctional institutions (h) Multi-occupancy residential, commercial (including shops), industrial, office and retailing buildings designed to accommodate more than 5000 people and with a gross area greater than 10 000 m^2 (i) Public assembly buildings, theatres and cinemas of greater than 1000 m^2 Emergency medical and other emergency facilities not designated as post-disaster Power-generating facilities, water treatment and waste water treatment facilities and other public utilities not designated as post-disaster Buildings and facilities not designated as post-disaster containing hazardous materials capable of causing hazardous conditions that do not extend beyond the property boundaries

The design working life of the structure is 50 years. Combined with the IL2 classification, a Return Period Factor "R" of 1.0 has been used for the analysis.

Elastic Site Spectra

The elastic site spectra (for mu = 1, Sp = 1.0 and for mu = 3, Sp = 0.4) is given by:

C(T) = Ch(T)*Z*R*N(T,D)

Town Hall mu = 1						
Structural System	**T_s**	**C_h(T)**	**Z**	**R**	**N(T,D)**	**C(T)**
X-dir	0.4	2.36	0.13	1.3	1.3	**0.40**
Y-dir	0.4	2.36	0.13	1.3	1.3	**0.40**
Town Hall mu = 3						
Structural System	**T_s**	**C_h(T)**	**Z**	**R**	**N(T,D)**	**C(T)**
X-dir	0.4	2.36	0.13	1.0	1.3	**0.16**
Y-dir	0.4	2.36	0.13	1.0	1.3	**0.16**

6.2 Building Analysis Method

The lateral load resisting systems for the building consists of brick walls in the front part of the building. In the auditorium and fly tower bracing is provided by the timber framed walls. Linear methods are generally appropriate for systems with a nominal ductility of 1.25. Because of the overall low ductility demand on the building, an Equivalent Static Analysis was adopted as recommended by "*The Seismic Assessment of Existing Buildings – Assessment Procedures and Analysis Techniques*" guidelines Part C2 Section 2.6.2 Table C2.1. The assessment was conducted in accordance with Part C8 of guidelines "*The Seismic Assessment of Existing Building - Unreinforced masonry buildings*" and Part C9 of guidelines "*The Seismic Assessment of Existing Building – Timber Buildings*"

Representative 2D frames in the front part of the building and in the auditorium were modelled, for analysis of the existing structure and/or for the strengthening scheme.

6.3 Stairs

There are six stairs constructed in timber frames for the building. Due to the stair's stiffness, relative to the floor diaphragm, they were assessed to not attract any of the floor loading.

6.4 Analysis Assumptions

General Assumptions

- In calculating the self-weight of the structure 24kN/m3 was used for all reinforced concrete elements. Steel weights were calculated from the member sizes. Lightweight roof elements have been assumed to be 0.2kPa. Mezzanine floor self-weight is assumed to be 0.5kPa.
- The following Live Loads & SDLs have been allowed for mezzanine floor:
  - Office Levels = 3.0kPa
- Load combinations used in the analysis are as required by NZS1170.0.
- The building has been designated as an Importance Level 3 (IL2). Post-disaster use - requirements that would necessitate an IL4 rating have not been specified by the client. The design working life of 50 years has been used, giving a return period factor of 1.3.
- The Hazard factor, Z for Taihape is 0.33.
- The Near Fault Factor, N(T,D) is 1.0 as the structure is located more than 20km from any known faults.
- The subsoil class for the site is D – Deep Soil.
- The member capacities have been assessed using the New Zealand Concrete Standard NZS3101:2006 and the guidelines "The Seismic Assessment of Existing Buildings".
- All building materials have been assumed to be in acceptable condition. Allowances for corrosion, spalling or any other latent structural defects has not been considered as part of this assessment.
- Member capacities were calculated per the sizes and dimensions given on the structural drawings and have been verified by field observation or measurement.
- The building has not been checked for wind loads.

Material Properties

Material properties have accounted for the probable strengths. Factors for various materials have been obtained from guidelines "The Seismic Assessment of Existing Buildings". For concrete a probable strength factor of 1.5 has been used while for reinforcing steel a factor 1.3 has been used.

<u>Structural Concrete and Reinforcement</u>

Concrete material strengths vary for different structural components.

- Reinforced Concrete Elements

Probable Compressive Strength	f'_c = 20 MPa – in situ
Probable Yield Strength of Reinforcement	$f_{y,p}$ = 275MPa

- Unreinforced Brick Masonry — f'm = 10.6 MPa F'b =26 MPa Em = 3180 MPa

  ϒ = 18 kN/m^3

7 Seismic Assessment Approach

A discussion on the seismic assessment approach is presented in the sections below, followed by a summary of the building's overall capacity in the Section 8.

7.1 Unreinforced Brick Masonry Walls

For the assessment of buildings with unreinforced brick masonry walls as the primary lateral load resisting systems, the structures have been assessed in accordance with Part C8 – "Unreinforced Masonry Buildings" in the seismic assessment guidelines "The Seismic Assessment of Existing Buildings – Technical Guidelines".

7.2 Timber framed structure

The timber framed structure attached to the front part of the building was assessed using NZS1170.Part 5, NZS3603:1993 Timber structures standard and NZS 3604:2011 Timber-framed buildings, as well as section C9 Timber Buildings.

7.3 Foundations

The subfloor piles were assessed using the above NZ standards.

8 Seismic Assessment Results

The seismic %NBS scores for the lateral structure, gravity structure and secondary structural elements for both directions of loading are summarized in the tables as follows, along with commentary on the results and potential options for strengthening to a higher % NBS:

8.1 Building Capacities

Structural Component	Description	Assessed %NBS Score	Comments about mode of failure, physical consequences, and potential options for strengthening to higher %NBS
Longitudinal-Direction (East – West)			
Roof Bracing	Auditorium	55	Add diagonal roof bracing, upgrade truss connections to top plate
	Front part of building	23	Add diagonal roof bracing
Wall bracing	Auditorium	71	No strengthening required
	First floor brick walls	100	In plane loading
Wall bracing	Ground floor brick walls	27	In plane loading, toe crushing failure
Mezzanine Floor	Flexible diaphragm	14	Tongue and groover flooring on timber joists
Stage floor	Flexible Diaphragm	14	
Sub-floor front part of building	Inadequate pile strength	50	Timber piles
Sub- floor auditorium	Inadequate pile strength	35	Concrete piles
Overall %NBS for Longitudinal Direction Loading		**14%NBS**	**Governed by floor diaphragms.**

Structural Component	Description	Assessed %NBS Score	Comments about mode of failure, physical consequences, and potential options for strengthening to higher %NBS
Transverse-Direction (North - South)			
Roof Bracing	Auditorium	29	Relying on top plate between trusses, add diagonal bracing
	Front part of building	10	Relying on sarking in roof plane, add diagonal bracing
Wall bracing	West wall of auditorium	18	Add plywood lining
	East wall of auditorium	37	Add plywood lining
Wall Bracing	First floor brick walls	35	In plane loading
	First floor north brick wall	23	Out of plane loading
Wall bracing	Ground floor brick walls	22	In plane loading
Wall bracing	Ground floor brick wall (north)	41	Out of plane loading
Mezzanine Floor And stage floor	Flexible diaphragm	10	Add plywood sheeting, add connections
Sub-floor front part of building	Timber piles	50	Inadequate pile strength
Sub-floor auditorium	Concrete piles	35	Inadequate pile strength
Overall %NBS for Transverse Direction Loading		**10% (IL2)**	**Governed by floor diaphragms.**

9 Severe Structural Weaknesses

The general process of the DSA is determining the probable seismic capacity of the structure and relating this to the ULS loading demands. The intention is also to ensure with reasonable satisfaction that the building can withstand higher levels of shaking. This is referred to as the structural resilience and is a necessary aspect of the buildings behaviour if it is to deliver the overall expected seismic performance.

There are potentially some aspects of a buildings behaviour which may not be adequately captured within these general assessment procedures but are likely to have a step change response resulting in sudden (brittle) and / or progressive, but complete collapse of the buildings gravity load support system in shaking greater than that represented by %ULS shaking. These building aspects are referred to as Severe Structural Weaknesses (SSWs). Potential severe structural weaknesses are described in C1 of "The Seismic Assessment of Existing Buildings" and include the following:

- In plane capacity of brick walls and piers.
- Out of plane (face loading) capacity of brick walls and piers.
- Inadequate bracing in the west wall of the auditorium.
- Lack of sub floor bracing.
- Flexible floor diaphragms at 10% NBS.

10 Secondary Structure Considerations

10.1 Stairs

There are six stairs constructed from timber framing to connect the ground floor to the first-floor level, and ground floor from ground surface. Due to the stairs' stiffness, relative to the mezzanine floor diaphragm and lateral load resisting system, and capability to accommodate deformation, they were assessed to not attract any mezzanine floor loading.

11 Concept Strengthening & Investigation

The detailed seismic assessment of the Town Hall at 90 Hautapu Street has found that several components of the building have a seismic score of less than 34%NBS, meaning that the building is deemed to be an earthquake prone building. The following section summarises the deficiencies in the building and provides concept strengthening to achieve at least 67 % NBS score for the structural components.

The detailed seismic assessment identified the following as having a seismic score of 10% NBS. Refer to Sections 8 & 10 for details.

- Roof bracing over the entire roof structure.
- New steel portal brace frames and foundations in the auditorium.
- Plywood sheet bracing on the transverse west wall in the auditorium, steel portal frames across the auditorium.
- Steel brace frames consisting of 310UC198 columns and 310UB40 beams (with foundations) in the front part of the building attached to the unreinforced brick masonry walls.

The conceptual Preliminary Strengthening Scheme is attached in **Appendix A** in this report.

12 Conclusion

RESONANT has been commissioned by the Rangitikei District Council to undertake a Detailed Seismic Assessment (DSA) of buildings, located at 90 Hautapu Street, Taihape. The aim of the assessment is to determine the seismic rating of the building in relation to the New Building Standard (%NBS).

The original building was designed and constructed in stages between 1900 and 1920.

The building is currently used as a library and offices. The auditorium and stage are not used at present due to Covid 19 restrictions. Lateral loads are resisted in the longitudinal direction by unreinforced brick mason walls and timber framed walls lined with weatherboards. Similarly, there are unreinforced brick masonry walls and timber framed walls resisting earthquake loading in the transverse direction. The sub-floor structure consists of timber piles at the front of the building and concrete piles over the remainder of the building. There is no roof bracing present in the roof plane. There is a reinforced concrete perimeter foundation supporting the external walls.

The assessment has been completed in accordance with the Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments, dated July 2017. The seismic rating assumes that Importance Level 2 (IL2), in accordance with the joint Australian/New Zealand Standard – Structural Design Actions Part 0, AS/NZS 1170.0:2002, is appropriate. Refer to the below table for a summary of the buildings seismic rating.

90 Hautapu Street		
Building	**Seismic Rating (%NBS)**	**Seismic Grade**
Town Hall	10%NBS (IL2)	Grade E

The Seismic Grade has been determined in accordance with the NZSEE grading scheme. The overall building seismic rating is governed by the failure of the unreinforced brick masonry walls in the transverse direction. The roof bracing over the front part of the building is also rated at 120%NBS. Refer to Section 8 for a summary of the % NBS scores, and commentary, for the various building structure components and to Appendix D for a Technical Summary Report.

Refer to Section 8 for a summary of the % NBS scores, and commentary, for the various building structure components. The concept strengthening advice, to achieve a greater %NBS rating, is stated in Section 11 and Appendix A.

13 Explanatory Notes

- This assessment contains the professional opinion of Resonant 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.
- The assessment is also based on information that has been provided to Resonant from other sources or by other parties. The assessment 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, Resonant takes no responsibility and disclaims all liability whatsoever for any loss or damage that results from any conclusions based on information that has been provided to Resonant.
- 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 RESONANT at the request of its client, The Rangitikei District Council and is exclusively for its use and reliance. It is not possible to make a proper assessment of this assessment 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 Resonant. The assessment 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 assessment by any third party.

APPENDIX A

PRELIMINARY STRENGTHENING SCHEME

CLIENT	Rangitikei District Council				
SUBJECT	DSA – 90 Hautapu St – Town HALL				
FILE No. 1213690	DATE 3/12/21	PAGE SK1	OF	BY GL	CKD

resonant
MULTI-DISCIPLINARY ENGINEERING & SURVEYING SERVICES
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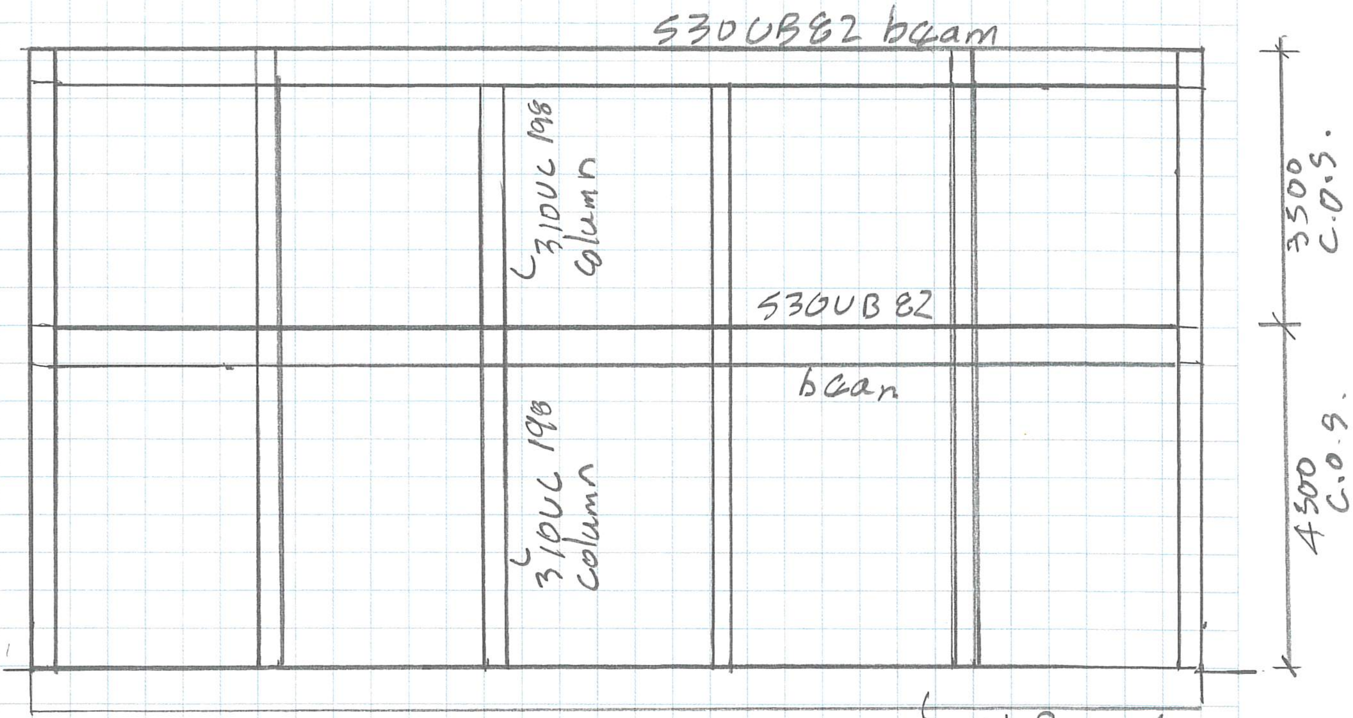


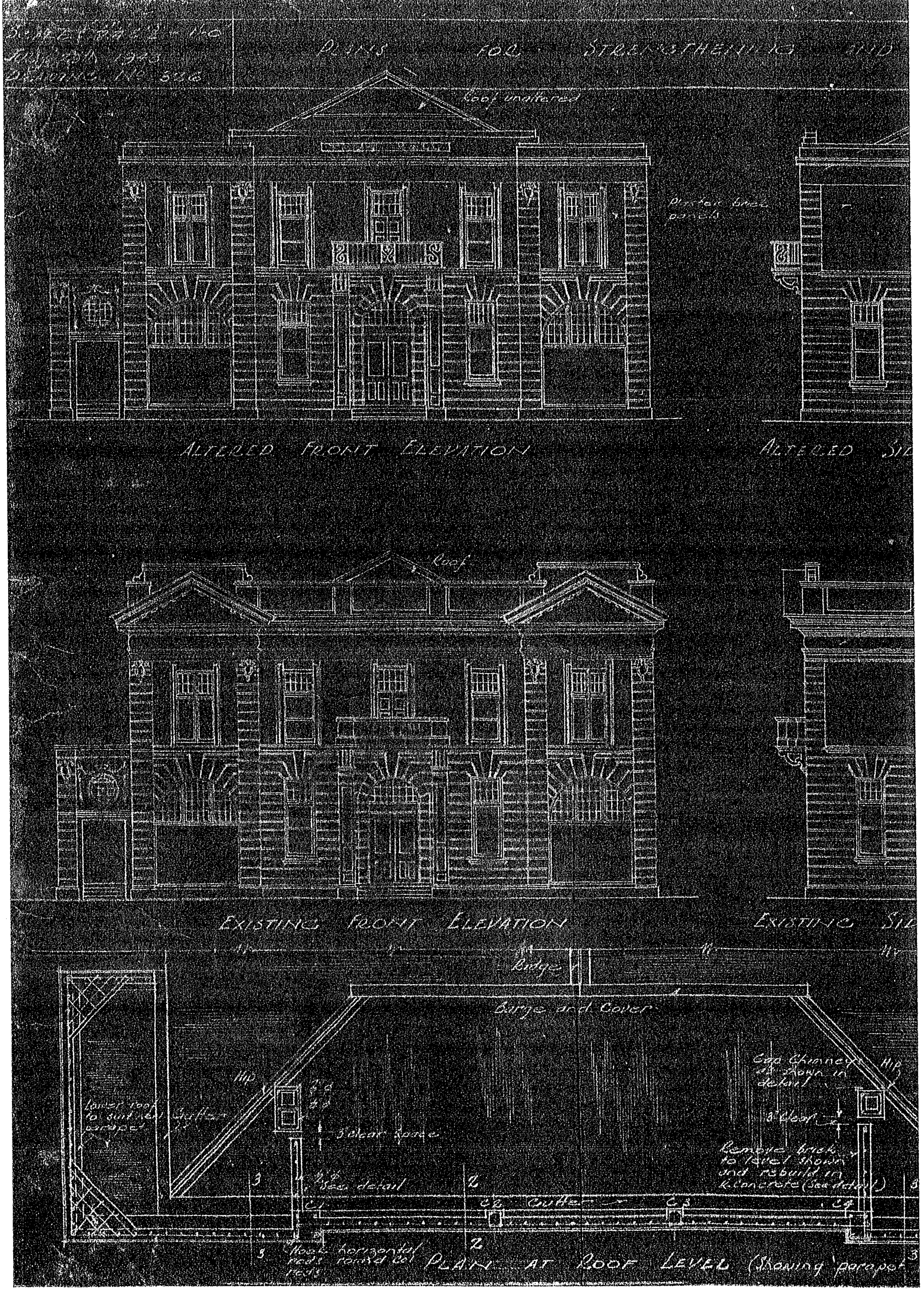
FRONT STEEL FRAME

Note: Dimensions of foundation beam to be confirmed

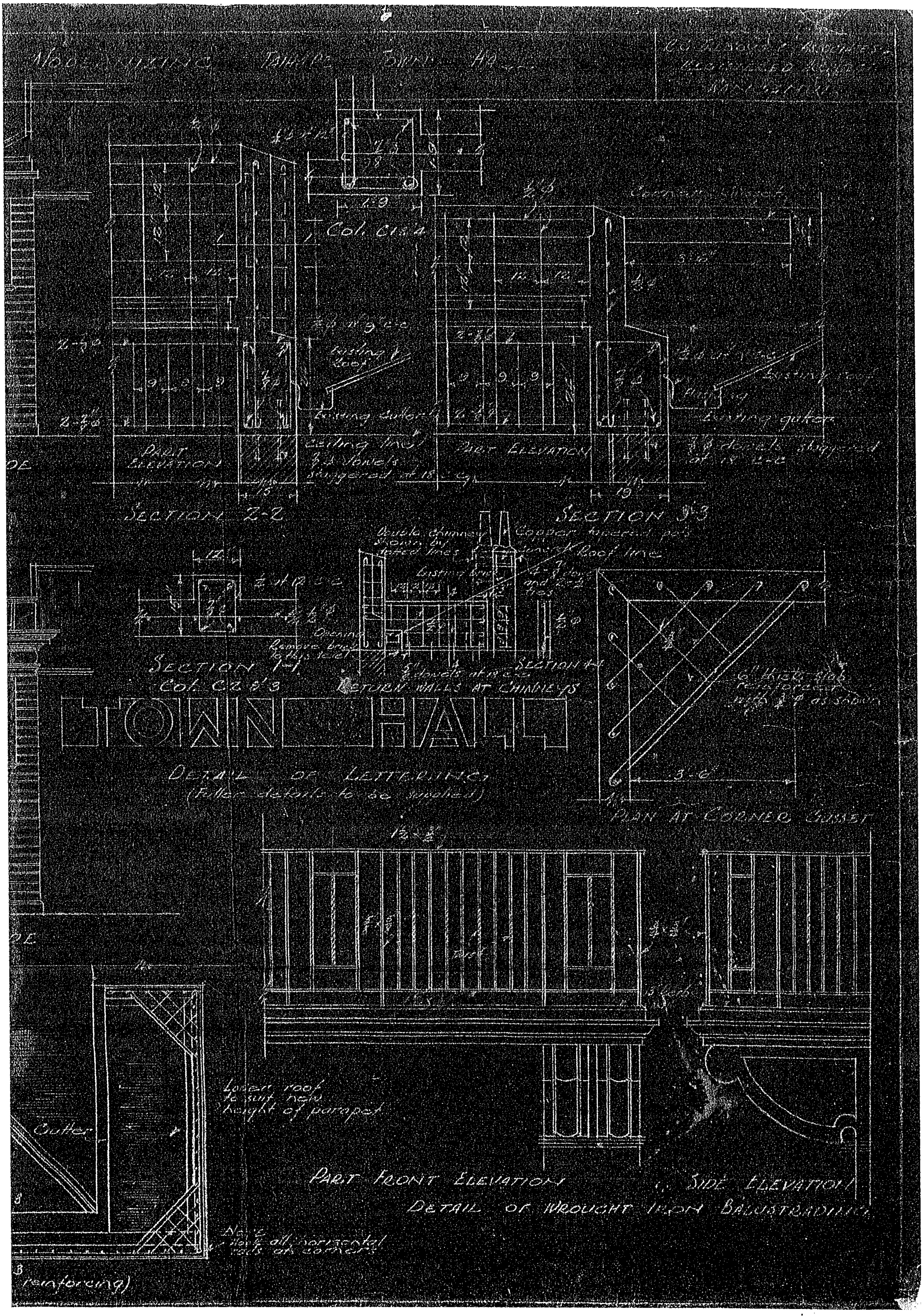
All dimensions to be checked on site

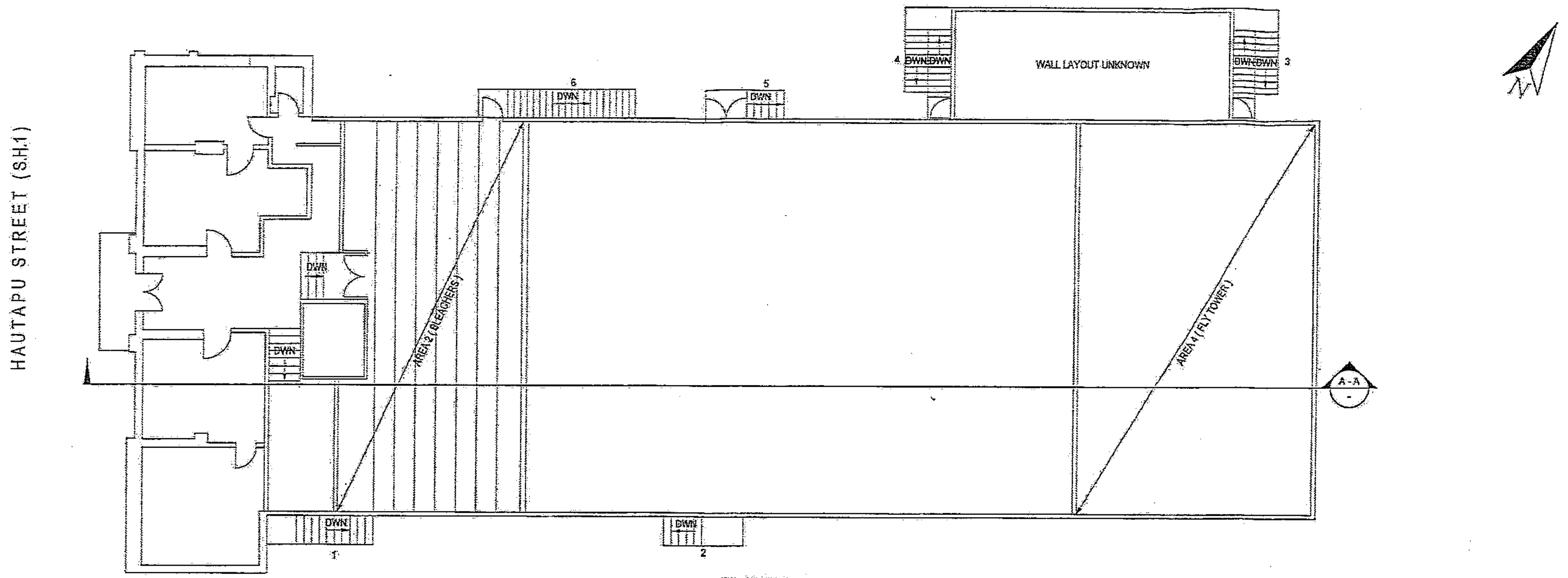
APPENDIX B

STRUCTURAL DRAWINGS

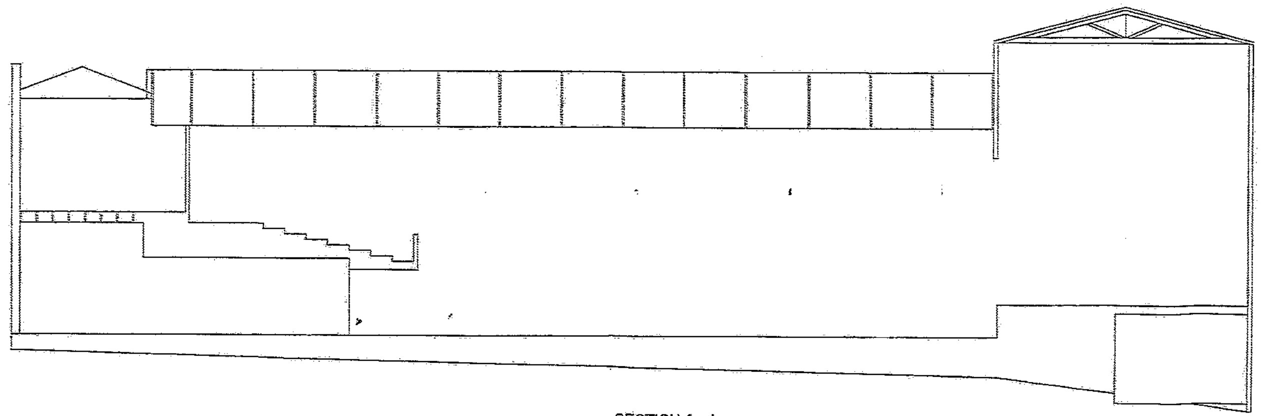


JULY 20th 1943
PLANS FOR STRENGTHENING AND
Roof unaltered
Plaster brick panels
ALTERED FRONT ELEVATION
ALTERED SI
Roof
EXISTING FRONT ELEVATION
EXISTING SI
Ridge
Barge and Cover
Hip
Lower roof to suit new gutter parapet
3' clear space
See detail
Gutter
C1
C2
C3
C4
Cap Chimney as shown in detail
3' clear
Remove brick to level shown and rebuild in R.Concrete (See detail)
PLAN AT ROOF LEVEL (Showing parapet


SECTION 2-2
SECTION 3-3
Col. C1 & 4
PART ELEVATION
PART ELEVATION
Existing roof
Existing gutter
Ceiling line
SECTION 1-1
Col. C2 & 3
RETURN WALLS AT CHIMNEYS
Roof line
TOWN HALL
DETAIL OF LETTERING
(Fuller details to be supplied)
PLAN AT CORNER GUSSET
Lower roof to suit new height of parapet
Gutter
(reinforcing)
PART FRONT ELEVATION
SIDE ELEVATION
DETAIL OF WROUGHT IRON BALUSTRADING



FIRST FLOOR PLAN



SECTION A - A

Rev	Details	Chk	Date
A	ISSUED FOR STRUCTURAL REPORT		10-12-09

KEVIN O'CONNOR & ASSOCIATES LTD
CONSULTING ENGINEERS, SURVEYORS & PLANNERS

The Contractor shall check all dimensions on site.

7 Pitt Street, PO Box 600 Palmerston North.
Ph (06) 356-7000 Fax (06) 356-7007 Email reception@koa.co.nz

TOWN HALL
TAIHAPE

FIRST FLOOR & SECTION A-A
RANGITIKEI DISTRICT COUNCIL

	Initials	Date
Surveyed		
Designed		
Drawn	JCR	DEC-2009
Checked		
Approved		

DO NOT SCALE. If in doubt ask for dimensions

Scales A1	Scales A3	Job No.
1:100	1:200	109 403

CAD Path/File	Sheet	of	Sheets	Rev.
J1/100/109/109403/03/01/02/Plans.Dgn	S2			A

APPENDIX C

ASSESSMENT SUMMARY REPORT

LONGITUDINAL DIRECTION (EAST-WEST)

ITEM	%NBS	COMMENT
Roof Bracing front part of building	23	Timber sarking
Roof bracing auditorium	55	Timber sarking
Timber wall bracing	71	Auditorium & fly tower cut in timber bracing
First floo121389r brick walls	100	In plane loading
Ground floor brick walls	27	In plane loading
First floor diaphragm	14	Tongue & groove flooring on timber joists
Stage diaphragm	14	Tongue & groove flooring on timber joists
Sub-floor front part of building	50	Timber piles
Sub – floor auditorium	35	Concrete piles

TRANSVERSE DIRECTION (NORTH-SOUTH)

ITEM	%NBS	COMMENT
Roof bracing front part of building	10	Timber sarking
Roof bracing auditorium	29	Timber sarking
Timber wall bracing west wall of auditorium	18	Gib. lining
Timber wall east wall mod auditorium	37	Pair of cut in 150x50 timber diagonals
First floor brick piers	35	In plane loading
First floor brick wall (north)	23	Out of plane loading
Ground floor brick piers	22	In plane loading
Ground floor brick wall (north)	41	Out of plane loading
First floor diaphragm	10	Tongue & groove flooring on timber joists
Sub-floor front part of building	50	Timber piles
Sub-floor auditorium	35	Concrete piles

APPENDIX D

CALCULATIONS

November 2021
Ref: 121389

90 HAUTAPU ST, TAIHAPE

DETAILED SEISMIC ASSESSMENT FOR THE TOWN HALL

STRUCTURAL CALCULATIONS

Resonant Consulting Limited Reference: 121389

November 2021
Ref: 121389

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90 HAUTAPU ST, TAI HAPE
DSA FOR THE TOWN HALL

Introduction

A Detailed Seismic assessment for the town Hall has been carried out. Structural calculations are presented in this document.

The assessment was carried out in accordance with the following New Zealand Standards and documents:

NZS 3603:1993 Timber Structures Standard
NZS 3604:2011 Timber-framed buildings
NZS 3101:2006 Concrete structures standard

Detailed Seismic Assessment Part C8 Unreinforced Masonry Buildings

Brief

A DSA is required for the Town Hall is required. A concept strengthening scheme to achieve 67%NBS is also required.

The building was constructed in stages between approximately 1900 and 1920.

The front part of the building consists of a reinforced concrete ring beam at roof level supporting reinforced concrete parapets on the front and side elevations The are also four triangular reinforced concrete gussets at roof beam level which tie the parapets together. The wall and piers consist of unreinforced brick masonry. There are also timber framed walls lined in fibrous plaster board. The first floor consists of timber tongue and groove flooring on timber joists.

The auditorium and fly tower/stage make up the rest of the building. The walls are timber framed with clad externally with timber weatherboards. The stage and auditorium floor consist of timber tongue and groove flooring supported on timber joists and bearers.

The subfloor for the front part of the building comprises timber piles. The Subfloor for the rest of the building consists of concrete piles. There is a reinforced concrete perimeter foundation beam around the whole building.

The roof structure consists of corrugated iron cladding on timber purlins and trusses. There is no roof bracing in the auditorium and fly tower.

Scope of Calculations

Refer to table of contents on the next page.

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INDEX

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1. Loads

Roof $G = 0.5$ kPa timber trusses/purlins
$Q = 0.25$ kPa

There are triangular reinforced concrete roof gussets (4) at the front of the building

$G = 24 \times 0.154 = 3.7$ kPa
$Q = 0.25$ kPa

timber floor $G = 0.5$ kPa
$Q = 3.0$ kPa offices

1st floor, dress circle $G = 0.5$ kPa
$Q = 4.0$ kPa (fixed seating)

Stage $G = 0.5$ kPa
$Q = 7.5$ kPa

auditorium floor $G = 0.5$ kPa
$Q = 4.0$ kPa

brick walls, with plaster:

$G \approx 0.381 \times 18$
$= 6.86$ kPa

CLIENT Rangitikei

SUBJECT DSA 90 Hautapu St, Town Hall

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2. Seismic Load Take-Off

1) Front Facade and wings.
a) at roof level:

roof = 0.5 × 5 × 19.8
= 49.5 kN

Concrete gussets: 4x 24×0.154 × 1.7²/2
6"
= 21.4 kN

parapet 15" = 381mm × 1.1
= 24×1.1× 0.381 kN/m
= 10.05 kN/m have 10.8 m
= 108.65 kN

low parapet 15" × 0.7m high
= 24×0.7×0.381
= 6.4 kPa
have 5+5+4+4+4
= 18.8 kN
= 18.8× 6.4
= 120.3 kN

concrete roof beam at roof level

24" × 15" = 24×0.61×0.381
= 5.58 kN/m

have 5+5+19.8
= 29.8m

= 5.58×29.8
= 166.2 kN

Brickwalls :- 3×0.090×18 + 0.01×20 = 5.06 kPa
plaster
ht = 3.6/2 = 1.8 m

wt = 1.8 × 5.06 = 9.108 kN/m

length = 29.8 + 1.8 + 1.8 + 2 + 7.7 - openings

openings = 2.3+1.6+1.1 + 1.3+1.1 + 1.5 = 9.2m

wt brickwalls = 28.9 × 9.108 = 263.2 kN

glazing = 0.5×1.8×9.2 = 8.3 kN

Columns = 2×24[(0.305× 0.381) + (0.457×0.457)]×3.6/2 = 28.1 kN

timber walls = 0.5×3.6/2 (15+2.8+3.5+6.8+8.2+3.6+4.6) = 40 kN = 805.5kN
int

Wind

$M_{z,\,cat} = 1.025$ cat 2 $z = 12.5m$! tall walls assumed

ult $V_{sit\,\beta} = 45 \times 1.0\,(1.025 \times 1 \times 1)$
$= 46.13\ m/s$

$q_u = 0.6 \times (46.13)^2 \times 10^{-3}$
$= 1.277\ kPa$

serv $V_{sit\,\beta} = 37 \times 1.025 = 37.93\ m/s$

$q_s = 0.6\,(37.93)^2 \times 10^{-3}$
$= 0.863\ kPa$

Seismic

The building (original) was constructed in stages, circa 1900s to 1920s. Importance Level 2

Brick walls ⇒ $\mu = 1.0$ (unreinforced) $Z = 0.33$ $R = 1.0$
$S_p = 1.0$

assume class D soils - soils do not appear to be soft, as there are no signs of settlement having occurred in the foundation beams.

$C_h(T) = 3.00$ class D, period 0.4s

$C(T_1) = C_h(T)\,Z\,R\,N(T,D)$
$= 3.0 \times 0.33 \times 1.0$
$= 0.99$

$C_d(T_1) = \dfrac{C(T_1)\,S_p}{k_\mu}$ $k_\mu = \dfrac{(\mu-1)\,T_1}{0.7} + 1 = \dfrac{0 \times 0.4}{0.7} + 1$
$= 1.0$

$C_d(T_1) = \dfrac{0.99 \times 1.0}{1} = 0.99$

for $\mu = 3$ $k_\mu = \dfrac{(3-1)\,0.4}{0.7} + 1 = 2.143$

$S_p = 1.3 - 0.3 \times 3 = 0.4$

$\mu = 3$ $C_d(T_1) = \dfrac{0.99 \times 0.4}{2.143}$

$= 0.185$

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Seismic Load Take off cont

1) Front Facade & wings

b) first floor

upper walls = 303.2
" glazing = 8.2
upper columns = 28.0 kN

Floor : area = 19.8 × 7.2 ~~1.1 × 1.6 × 2.4~~
= 138.7

$\gamma_a = 0.3 + \frac{2.7}{\sqrt{138.7}} = 0.53$

wt = $(0.5 + \underset{\gamma_a}{0.53} \times \underset{\gamma_u}{0.3} \times 3)$ 138.7
= 135.5 kN

Dress circle:- area = 15 × 3.6
= 54 m^2

$\gamma_a = 0.3 + \frac{2.7}{\sqrt{54}} = 0.67$

wt = (0.5 + 0.3 × 0.67 × 4.0) 54
= 70.2 kN

lower walls: brick t = 5.06 × 4.6/2 ~~- 3~~ × 11.64 kN/m
glazing = 0.5 × 2.3 = 1.15 kN/m

parimeter = 2.9 + 1.7 + 5 + 6.5 + 19.8 + 2.9 + 1.5
= 40.3 m

openings = 0.8 + 0.9 + 2.8 + 1.2 + 1.8 + 1.0 + 2.9 + 1.1
= 12.5 m

⇒ wall = (40.3 − 12.5) × 11.64
= 323.6 kN

glazing/timber framed walls = 12.5 × 1.15
= 14.4 kN

brick + plaster = 5.06 kPa
area = $2\left[\left((2.6 \times 2.8) - (\frac{\pi}{2} \times 1.4^2)\right)\right] + (2.2 \times 2.8) - (\frac{\pi}{2} \times 1.4^2)$

b) first floor cont.

brick + plaster arches = 5.06 (8.40 + 3.08)
= 58.1 kN

timber walls = 0.5 x 2.3 (15.3 + 7.2 + 5.1 + 6.4 + 6.4 + 6.4 + 8.8
+ 4.4 + 4.4 + 7.0)
= 82.1 kN

Distribution

at first floor Σ = 1096.3

level	w	h	wh	wh/Σwh	0.92 wh/Σwh	V* kN
roof	805.5	8.0	6444.00	0.5611	0.5182	1126.3
1st floor	1096	4.6	5041.60	0.4309	0.4038	760.2
	1901.5		11,485.60			Σ = 1882.5 kN

OK

$V^* = 1901.5 \times 0.99$
= 1882.5 kN

sub floor:

timber floor $G = 0.5$ kPa
$Q = 3.0$ kPa

area = 5 x 1.6 + 9.3 x 18.1 + 4.4 x 15.3
= 243.65 m²

$\psi_a = 0.3 + \frac{2.7}{\sqrt{243.65}} = 0.473$

$G + Q = (0.5 + 0.3 \times 0.473 \times 3)\ 243.65$

= 225.5 kN

$V^* = 1.287 \times 225.5$
= 290.2 kN

2) auditorium

a) roof level

roof area = 15.3 × 30
= 459 m²

wt = 0.5 × 459
= 229.5 kN

walls = 0.5 × $\frac{9.6}{2}$ (2×30 + 15.3)
= 180.7 kN

at roof level Σ = 410.2 kN

V^* = 0.99 × 410.2 kN   μ = 1
= 406.1 kN

3) Fly tower

a) at roof (11.2 m high)

roof = 0.5 × 9.6 × 15.3
= 73.4 kN

walls = 0.5 × $\frac{9.8}{2}$ (15.3 + 2×9.6) + (0.5 × 4.2 × 15.3)
(procanium)

= 116.7 kN   at roof 151.2 kN

cat walk / gallery 0.5 × 2 × (2 × 9.6 + 15.3)
= 34.5 kN

b) stage ht = 1.5 m above floor

area = 15.3 × 9.6
= 146.88 m²

ψ_a = 0.3 + $\frac{2.7}{\sqrt{146.88}}$
= 0.52

wt = (0.5 + 0.3 × 0.52 × 7.5) 146.88
= 233.8 kN

walls above = 116.7 kN   stage Σ = 390.7

walls below = 0.5 × $\frac{1.5}{2}$ × 15.3 + 0.5 × $\frac{4}{2}$ × (2 × 9.6 + 15.3) = 40.2 kN

3 b) continued

distribution

level	w	h	wh	wh/Σwh	0.92wh/Σwh	V* kN
roof	151.2	11.2	1693.44	0.5201	0.4785	~~2~~299.6
stage	390.7	4.0	1562.80	0.4799	0.4415	236.9
	541.9		3256.24			Σ = 536.5
						OK.

$V^* = 0.99 \times 541.9$
$= 536.5$ kN

4. attached toilet block.

internal walls – layout unknown, use 0.5 kPa

a) roof h = 4.2 (average)
roof = 0.5 x 11 x 4.4
= 24.2 kN

external walls = $0.5 \times \frac{2.4}{2} (2 \times 4.4 + 11)$

= 11.9 kN

Partions = 24.2 kN

at roof Σ = 36.1 kN

Sub floor floor $\gamma a = 0.3 + \frac{2.7}{\sqrt{46.4}} = 0.7$

G + γQ = (0.5 + 0.3 x 0.7 x 3) 4.4 x 11
= 54.7 kN
upper walls = 11.7 kN
Partions = 24.2 kN
lower walls = 0.9 x 0.5 x 19.8
= 8.9 kN

at floor Σ = 99.5 kN

distribution

level	w	h	wh	wh/Σwh	0.92wh/Σwh	V* kN
roof	36.1	4.2	151.62	0.4585	0.4218	57.3
floor	99.5	1.8	179.10	0.5415	0.4982	66.9
	135.6		330.72			134.2 kN
					OK.	

$V^* = 0.99 \times 135.6$
$= 134.2$ kN

Distribution to walls

The diaphragms, where available, will be flexible. Apportion loads on a tributary width basis.

For timber framed walls use $\mu = 3$

Revised $C_d(T_1)$: $\mu = 3$ $Z = 0.33$ $R = 1.3$

$$C(T_1) = 3 \times 0.33 \times 1.0 \times 1$$
$$= 0.99$$

$$S_p = 1.3 - 0.3\mu$$
$$= 0.4$$

$$k_\mu = \frac{(3-1)0.4}{0.7} + 1 = 2.143$$

$$\Rightarrow C_d(T_1) = \frac{\cancel{1.0}\,0.99 \times 0.4}{2.143}$$
$$= 0.185$$

$$\text{ratio } \frac{\mu = 3}{\mu = 1} = \frac{0.185}{0.99}$$

$$\text{ratio} = 0.187$$

3. Front Building

At first floor have unreinforced brick masonry walls.

EQ across use trib. width approach,

at roof level $V^* = 1126.3$ kN

have $1.8 + 1.6 + 1.6 + 2.6 + 2.2 + 2.4 + 2.7 + 1.6$
$= 16.5$ m

EQ $= 1126.3 / 16.5$
$= 68.3$ kN/m

EQ along have $2.2 + 1.5 + 3.0 + 2.3 + 2.3 + 2.3 + 0.9 + 0.8$
$= 15.3$ m

$= 73.6$ kN/m

will need to anaylse as piers & spandrels.
south & front of building.

across. typical wall 1.8m long
$= 122.9$ kN

In plane: $V^* = 159.2$ kN use Section C8 of MBIE guidance

Material Properties

Brick – medium hardness $f'_b = 26$ MPa $f_{bt} = 3.1$ MPa

mortar strength, assume $f'_j = 2$ MPa $\Rightarrow f'_m = 10.6$ MPa for $f'_b = 26$

mortar hardness, medium assumed $\Rightarrow f'_j = 2$ MPa, $C = 0.5$ MPa, $\mu_f = 0.6$

rupture of clay bricks $f'_r = 0.12\, f'_b = 0.12 \times 26$
$= 3.12$ MPa

$E_m = 300 f'_m = 300 \times 10.6$
$= 3180$ MPa

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In plane capacity of first floor walls cont.

1) diagonal tensile capacity

$$V_{dt} = f_{dt} A_n \beta \sqrt{1 + \frac{f_a}{f_{dt}}} \qquad \beta = 0.67.$$

$h_{eff}/l = 3600/1800$
$= 2.0 > 1.5 \Rightarrow$ slender pier

from table C8:13 $\beta = 0.67$

$A_n = 1800 \times 3 \times 100 = 540000$
(includes mortar)

$$f_a = \frac{1.8 \times 0.270 \times 3.6 \times 18 \times 10^3}{1.8 \times 0.270} = 64,800 \frac{N}{m^2}$$

$= 0.065$ MPa $\qquad f_{dt} =$

$f_{dt} = 0.5C + f_a \mu f$ $\qquad$ Eqn C8.3

$= 0.5 \times 0.54 + 0.6 \times 0.065$

$= 0.289$ MPa

$\Rightarrow V_{dt} = 0.289 \times 540,000 \sqrt{1 + \frac{0.065}{0.289}} \times 10^{-3}$ kN

$= 191.16$ kN > 122.93 kN

diagonal tensile cap. OK

2) Toe crushing capacity

$$V_{tc} = (\alpha P + 0.5 P_w)\left(\frac{l_w}{h_{eff}}\right)\left(1 - \frac{f_a}{0.7 f_m}\right)$$

$\alpha = 0.5$ (cantilever)

$P_r = 3.696 \times \frac{1.22^2}{2 \times 2}$ $\qquad$ roof gusset

$= 1.36$ kN

$P_w = 18 \times 1.8 \times 0.3 \times 3.6 = 35.0$ kN

$f_a = 0.065$ MPa
$f_m = 10.6$ MPa

Toe Crushing cont.

$$V_{tc} = (0.5 \times 1.36 + 0.5 \times 35)\left(\frac{1800}{3600}\right)\left(1 - \frac{0.065}{0.7 \times 10.6}\right) \times 10^3$$

$$= 9010 \text{ kN} > 122.9 \text{ kN}$$

Toe Crushing OK

3). Rocking capacity

$$V_r = 0.9(\alpha P + 0.5 P_w)\frac{l_w}{h_{eff}}$$

$$= 0.9(0.5 \times 1.36 + 0.5 \times 35)\left(\frac{1800}{3600}\right) \times 10^3$$

$$= 8181 \text{ kN} \quad \text{OK}$$

Rocking Capacity OK

Face Loading (out of plane)

Upper wall between townhall and library:-
use Simplifications for regular walls page C8-98 of C8 Unreinforced Masonry Buildings

divide wall in two $W = 3.6 \times 1.8 \times 0.381 \times 18 = 44.4 \text{ kN}$ (15")

$t_{nom} = 381$

$$t = t_{nom}(0.975 - 0.025 \, P/W)$$

$$= 381(0.975 - 0.025 \times \frac{1.36}{44.4})$$

$$= 371.2 \text{ mm}$$

from Table C8B.1 Case 0 is appropriate

$e_p = 0 \qquad b = (W/2 + P)t$

$e_b = 0 \qquad a = (W/2 + P)h$

$$\Delta_i = \frac{bh}{2a} = t/2 = 185.6 \text{ mm}$$

$$\Delta_m = 0.6\,\Delta_i = 111.36 \text{ mm}$$

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$$T_p = \sqrt{\frac{0.28h}{1+2P/w}}$$

$$= \sqrt{\frac{0.28 \times 3.6}{1+ 2\times1.36/44.4}}$$

$= 0.97$ Second

$C_i(T_p) = 2(0.5/0.97)^{0.75}$. $0.5 < T_p < 1s$
$= 1.212$

$\gamma = Wh^2/8Jg$ use max. value of 1.5

$D_{ph} = \gamma (T_p/2\pi)^2 C_p T_p R_p . g$

$\approx 1.5 \times \frac{0.97}{3} \times 1.0.212$

0.485 m $= 485$ mm !!

$\% NBS = 100 \times \frac{\Delta_m}{D_{ph}} = 60 \frac{\Delta_i}{D_{ph}}$

$= \frac{60 \times 185.6}{485}$ %

$= 23\%$ NBS Upper Wall @ 23% out of plane

A. Ground Floor Walls - At front

In plane:-

walls across $= 1.8 + 1.0 + 1.0 + 1.7 + 1.2 + 1.7 + 1.7 + 2.0 + 1.0$
$= 13.1m$

$V^* = 1882.5kN$
$= 143.7$ kN/m

walls along : $= 1.8 + 2.3 + 3.3 + 1.9 + 1.9$
$= 11.2$

$= 168.1$ kN/m · governs

wall ① see sketch length $= 3.3m$

$V^* = 554.7$ kN

i) diagonal tensile capacity

$$V_{dt} = f_{dt} A_n \beta \sqrt{1 + \frac{f_a}{f_{dt}}}$$

$h_{eff}/l = 4600/3300$
$1.39 < 1.5 \Rightarrow \beta = 1.0$

$A_n = 3300 \times 381 = 1,257,300$

$G = 1.36 + 18 \times 0.381 \times 3.6 \times 1.7 + 24 \times (0.381 \times 0.61 + 0.56 \times 0.7) \times 2.5$
gusset · wall above · ring beam/parapet

$+ 18 \times 0.381 \times 3.3 \times 4.6$
wall

$= 167.85$ kN $= 167.85 \times 10^3$ N

$$f_a = \frac{167.85 \times 10^3}{3300 \times 381}$$

$= 0.133$ MPa

$f_{dt} = 0.5C + f_a \mu f$
$= 0.5 \times 0.5 + 0.6 \times 0.133$
$= 0.330$ MPa

$$V_{dt} = 0.33 \times 1,257,300 \times 1.0 \sqrt{1 + \frac{0.133}{0.33}} \times 10^3$$

$= 491.7$ kN < 554.7 kN N/G diagonal tensile cap. 89% NBS

Ground Floor walls In Plane cont

2) Toe crushing Capacity $\alpha = 1$

$$V_{tc} = (\alpha P + 0.5 P_w)\left(\frac{l_w}{h_{eff}}\right)\left(1 - \frac{f_a}{0.7 f_m}\right)$$

$P = 1.36$ gusset $+ 18 \times 0.381 \times 3.6 \times 1.7 + 1.36$ gusset $+ 24(0.381 \times 0.61 + 0.154 \times 0.7) 3.3$
$= 106.4$ kN

$P_w = 18 \times 0.381 \times 4.6 \times 3.3$
$= 104.1$ kN

$f_a = 0.133$ MPa see sheet 13

$f_m = 10.6$ MPa

$$V_{tc} = (1.0 \times 106.1 + 0.5 \times 104.1)\left(\frac{3300}{3600}\right)\left(1 - \frac{0.133}{0.7 \times 10.6}\right) \times 10^3$$

$= 142.4$

@ 27 % NBS toe crushing.

toe crushing 27% NBS

Out of plane loading

$W = 3.3 \times 4.6 \times 0.381 \times 18 = 104.1$ kN

$P = 79.4$ kN

$t_{nom} = 0.381$ m

$$t = 381\left(0.975 - 0.025 \times \frac{79.4}{104.1}\right)$$

$= 364$ mm

$$\Delta_i = \frac{bh}{2a} = \frac{t}{2} = 182\text{mm}$$

$\Delta_m = 0.6\Delta_i = 109.2$ mm

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SUBJECT	DSA 90 Hautapu St - Town Hall			
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$$T_p = \sqrt{\frac{0.28h}{1+2P/w}}$$

$$= \sqrt{\frac{0.28 \times 4.6}{1 + 2 \times 794/104.1}}$$

$$= 0.8 \text{ s}$$

$$C_i(T_p) = 2(0.5/0.8)^{0.75} \quad 0.5 < T_p < 1$$

$$= 1.4 \text{ s}$$

$$D_{ph} = \gamma (T_p/2\pi)^2 C_p T_p R_p g \qquad \gamma = \text{(use max value)}$$

$$= 1.5 (0.8/2\pi)^2 1.4 \times 0.8 \times 1.0 \times 9.81$$

$$= 0.267 \text{ m} = 267 \text{ mm}$$

$$\%NBS = \frac{60 \Delta_i}{D_{ph}} = \frac{60 \times 182.4}{267} = 41\%$$

lower wall @ 41%NBS
(between library & hall) under face loading

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G. Auditorium / Fly Tower

Timber $\mu = 3$

at roof level $V^* = 705.7$ kN $\mu = 1$
$V^* = 130.6$ kN $\mu = 3$

Fly tower (at stage) $V^* = 236.9$ kN $\mu = 1$
$V^* = 43.8$ kN $\mu = 3$

$\Sigma \approx 174.4$ kN $\mu = 3$

Side walls (west and east walls)
= 87.2 kN / wall

east wall, have X 150x50 timber braces.

$N_t^* = \sqrt{2} \times 87.2$ kN
$= 123.3$ kN

NZS 3603 $\phi N_t = \phi k_1 \times A \times f_t$
$0.8 \times 1 \times 150 \times 50 \times 6 \times 10^{-3}$
$= 45$ kN

@ 37% NBS

North wall = 87.2 kN $N_t^* = 123.3$ @ 71%

South wall = 87.2 kN $N_t^* = 123.3$ @ 71%

west wall have gib lining:
capping BU $= 60 \times \frac{2.4}{3.6} = 40$ BUs/m
$= 2$ kN/m

have 13.6 m of wall

V^* from front building $= 1882.5 \times \frac{1.4}{8.0} = 329.4$ kN $\mu = 1$
$= 61$ kN $\mu = 3$

$\Sigma = 87.2 + 61 = 148.2$ kN

have 13.6 m of wall $= 10.9$ kN/m

west wall @ 18% NBS

Auditorium/Fly tower bracing
$V^* = 130.6$ kN $= 2612$ BUs $=$ along 87.1 BUs/m
$= 163.3$ BUs/m across

Sarking ≈ 48 BUs/m

Auditorium roof bracing = 55% NBS
across = 29% NBS

Fly tower V^* =

Pier/Spandrel – at front facade

Loads

G members 5 to 13
$= 24 \times 0.435$
$= 10.44$ kN/m

G members 15 to 24
$= 24 \times 0.762$
$= 15.29$ kN/m

$Q = 3.0 \times \frac{0.4}{2}$
$= 0.6$ kN/m
members 15 to 24

Ex. roof ... stretch
$V^* = \frac{1126.3}{}$

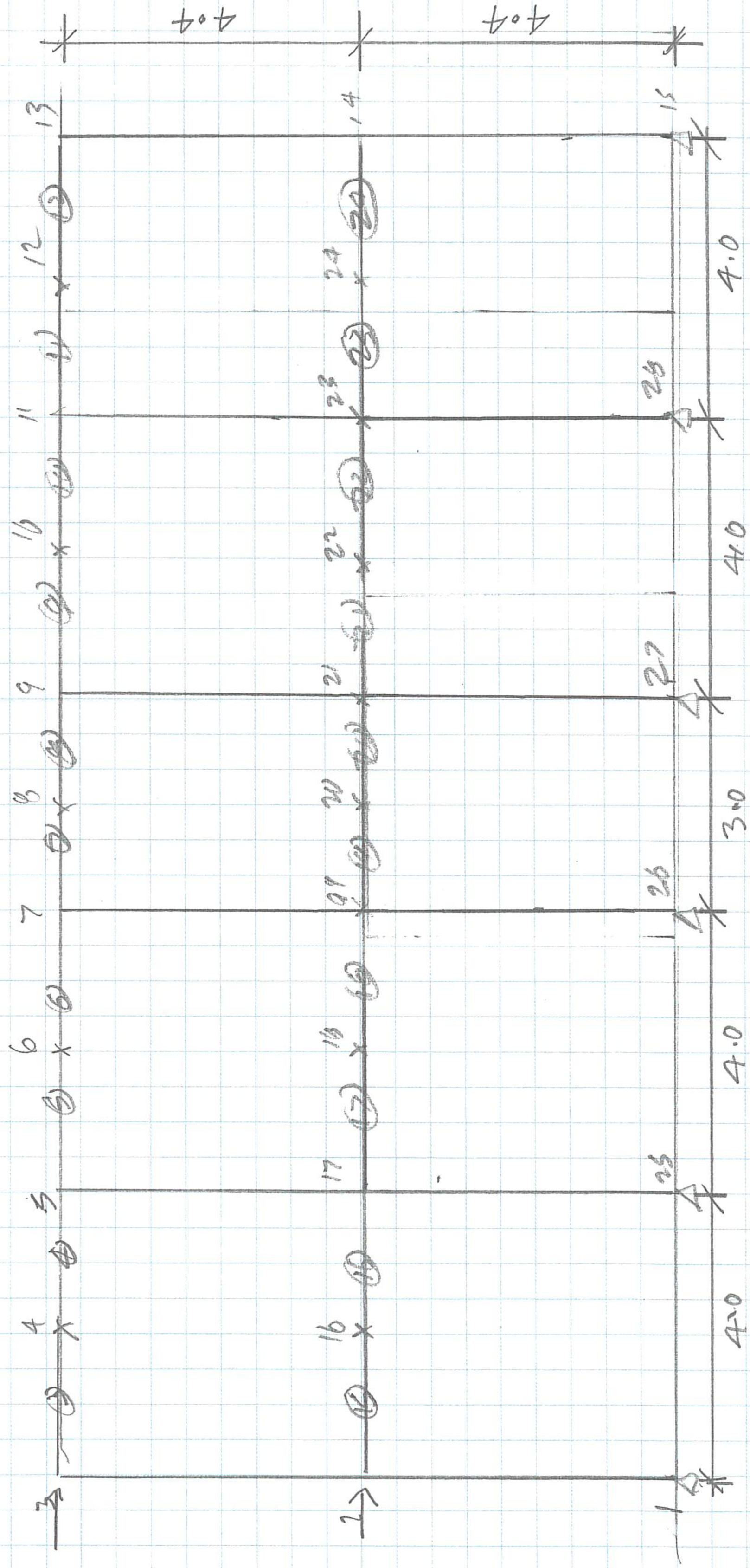
$= \frac{1126.3}{16.7}$

$= 67.4$ kN/m

first floor
$V^* = 760.2$ kN

$= \frac{760.2 \text{ kN}}{14}$

$= 54.3$ kN/m



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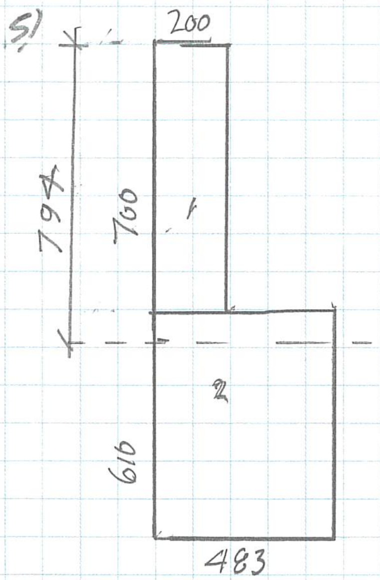
Parapet / Ring Beam



Section	area	y	Ay	h	Ah^2	Iself $\times 10^6$	Itotal
1	140,000	350	49,000,000	444	27,599.04	5,716.67	33,315.7
2	294,630	1005	296,103,150	311	28,496.91	9,135.99	37,632.9
	434,630		345,103,150				$70,949.6 \times 10^6$ mm^4

$\bar{y} = 794$

cracked $I \approx 0.4 \times 70,949.6 \times 10^6$
$= 28,379 \times 10^6\ mm^4$

Section	area	x	Ax	h	Ah^2	Iself $\times 10^6$	Itotal
1	140,000	100	14,000,000	96	1290.24	466.66	1756.9
2	294,630	241.5	71,153,145	45.5	609.96	5727.83	6337.8
	434,630		85,153,145				8094.7×10^6 mm^4

$\bar{x} = 196mm$

$0.4 I_g = 3237.9 \times 10^6 mm$

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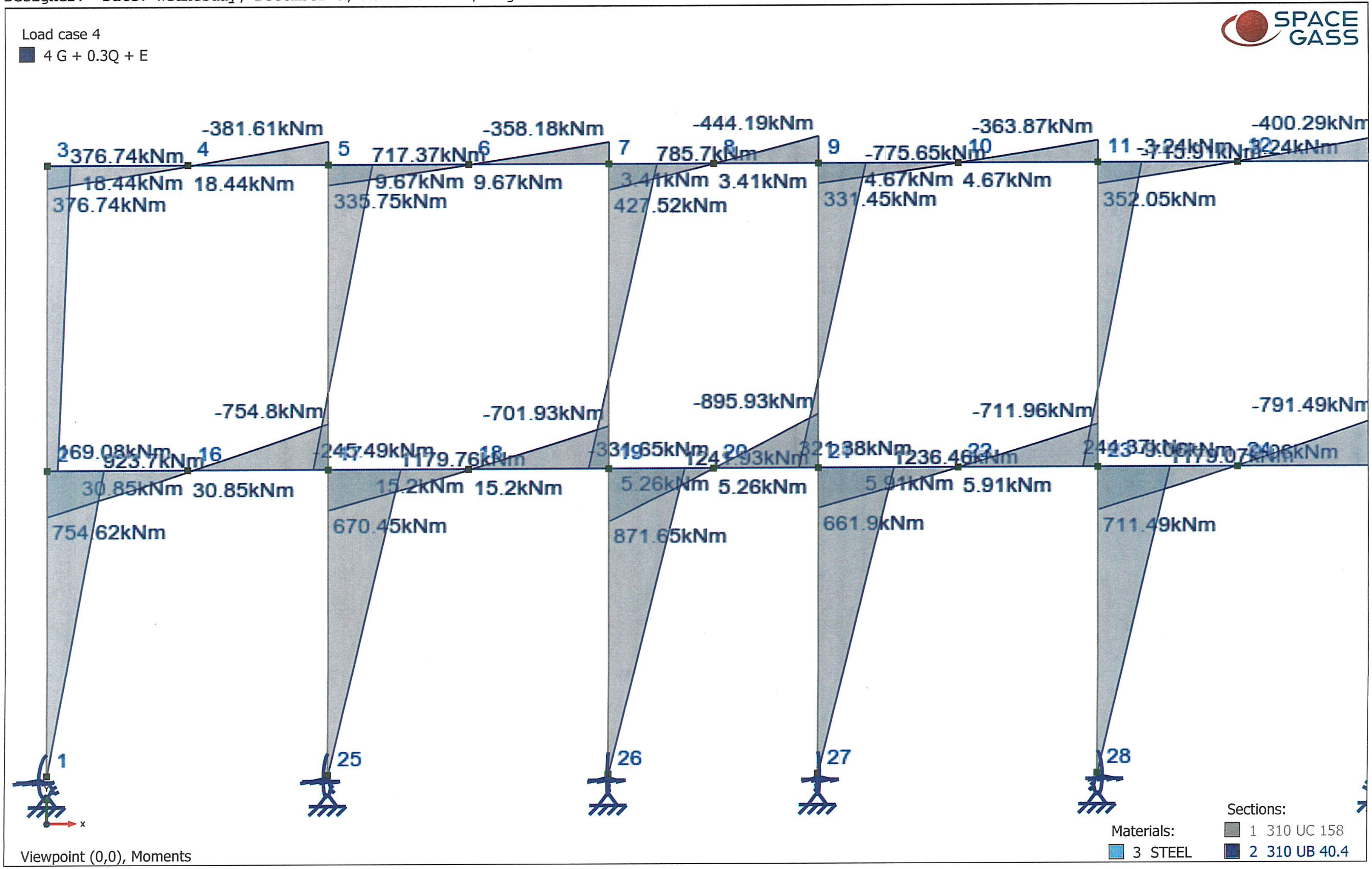
Designer: Date: Wednesday, December 8, 2021 1:00 PM, Page: 1



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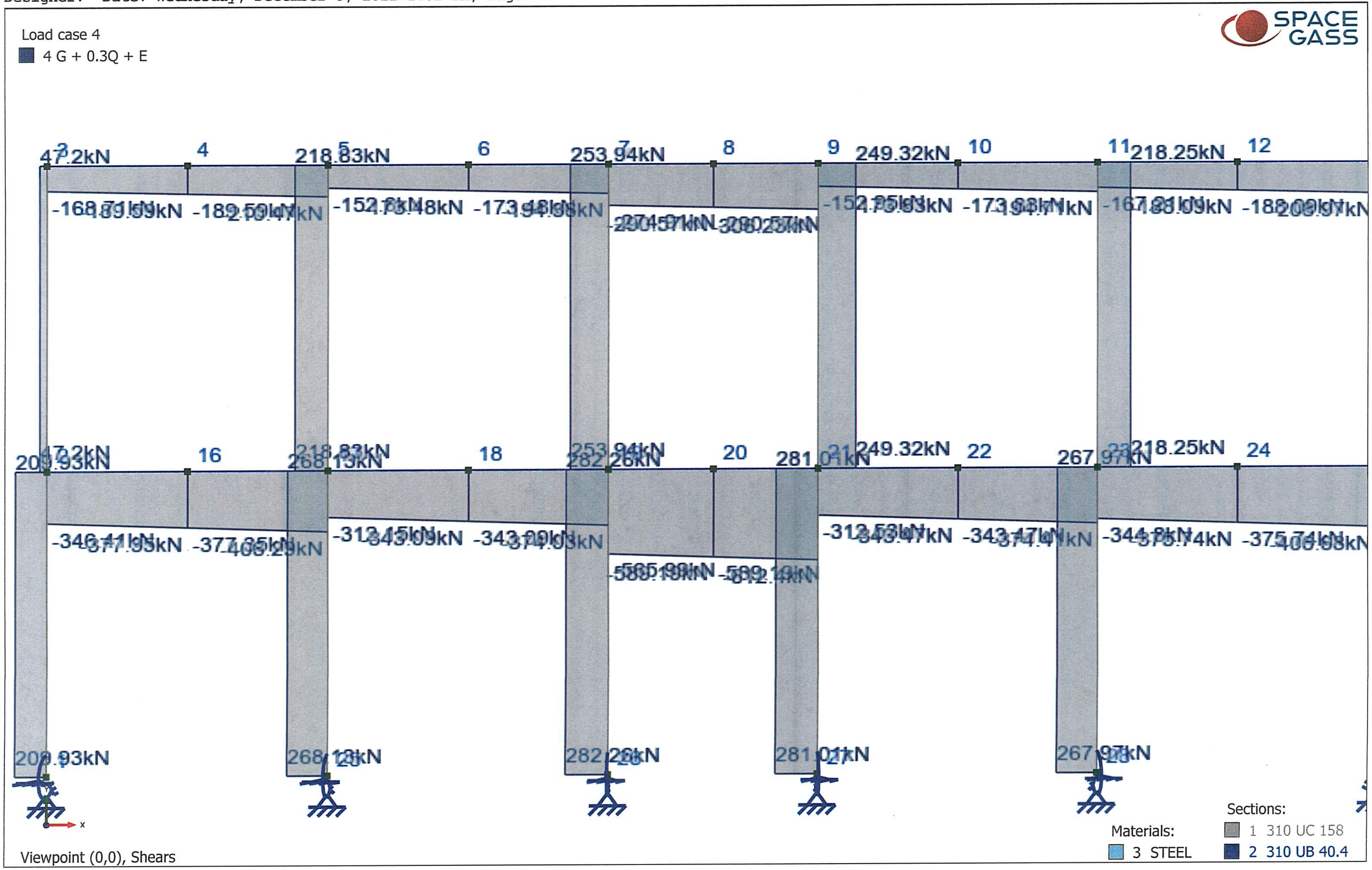

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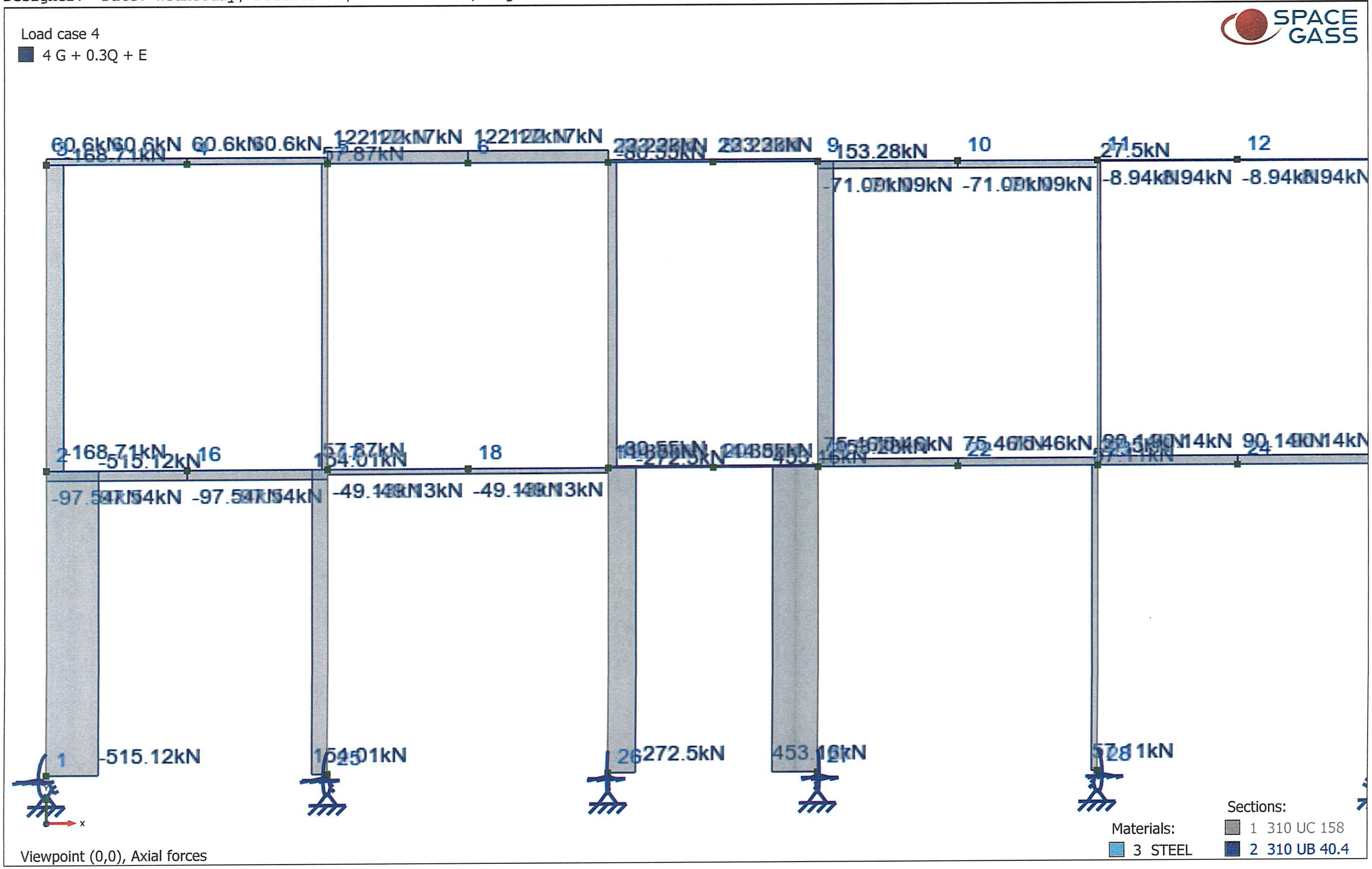

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Results SCA Sheets 19 to 21

1) Piers at ground floor

$V^* = 82.3$ kN member 22

i) diagonal tensile capacity

$V_{dt} = f_{dt} A_n \beta \sqrt{1 + \frac{f_a}{f_{dt}}}$ $h_{eff}/l = \frac{4600}{2700} = 1.7 \Rightarrow \beta = 0.67$

$A_n = 1{,}028{,}700\ mm^2$

$$f_a = \frac{20.37 \times 10^3 + (18 \times 1.0287 \times \frac{4.6}{2})10^3}{1{,}028{,}700}$$

$= 0.0612$ MPa

$f_{dt} = 0.5c + f_{a,up}$
$= 0.5 \times 0.5 + 0.6 \times 0.0612$
$= 0.287$ MPa

$\Rightarrow V_{dt} = 0.287 \times 1{,}028{,}700 \times 10^{-3} \sqrt{1 + \frac{0.0612}{0.287}}$
$= 325.2$ kN

@ 100% NBS

ii Toe crushing capacity

$V_{tc} = (\alpha P + 0.5 P_w) \frac{L_w}{h_{eff}} \left(1 - \frac{f_a}{0.7 f_m}\right)$

$P = 91.63$ kN $P_w = 18 \times 1.0287 \times 2.7 = 49.99$

$V_{tc} = (1 \times 91.63 + 0.5 \times 49.99)\left(\frac{2700}{4600}\right)\left(1 - \frac{0.0612}{0.7 \times 10.6}\right)$

$= 67.9$ kN

@ 24% NBS !!

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iii) Rocking capacity

$V_r = 0.9(\alpha P + 0.5 P_w)\frac{L_w}{h_{eff}}$

$= 0.9(1 \times 91.0 + 0.5 \times 49.99) \times \frac{2700}{4600}$

$= 61.7 kN$

@ 22.3 % NBS !

1st floor piers

$V^* = 253.9 kN$ member 28

$P = (10.43 + 2)2.8$ kN
Parapet roof $= 34.8$

$P_w = 18 \times 0.381 \times 2.3 \times 3.6$
$= 56.78 kN$

$h_{eff} = 3600$
$L_w = 2300$

$A_n = 876,300$

i) diagonal tensile capacity

$V_{dt} = f_{dt} A_n \beta \sqrt{1 + \frac{f_a}{f_{dt}}}$

$f_a = \frac{2.22 \times 10^3 + (18 \times 2.3 \times 0.381 \times \frac{3.6}{2}) \times 10^3}{381 \times 2300}$

$= 0.035 MPa$

$f_{dt} = 0.5 \times 0.5 + 0.6 \times 0.035$
$= 0.271 MPa$

$V_{dt} = 0.271 \times 876,300 \times 10^{-3} \times 0.67 \sqrt{1 + \frac{0.035}{0.271}}$

$= 131.02 kN$

@ 52 % NBS

ii) toe crushing capacity

$$V_{tc} = (\alpha P + 0.5 P_w)\frac{l_w}{h_{eff}}\left(1 - \frac{f_a}{0.7 f_m}\right)$$

$= P = 34.82$ kN

$P_w = 18 \times 0.381 \times 2.3 \times 3.6$
$= 56.78$ kN

$$V_{tc} = (1 \times 34.0 + 0.5 \times 56.78)\left(\frac{2300}{3600}\right)\left(1 - \frac{0.035}{0.7 \times 10.6}\right)$$

$= 16.4$ kN

@ 66% NBS !

iii) Rocking Capacity

$$V_r = 0.9(\alpha P + 0.5 P_w)\frac{l_w}{h_{eff}}$$

$$= 0.9(1 \times 34.0 + 0.5 \times 56.78)\left(\frac{2.3}{3.6}\right)$$

$= 17.6$ kN

@ 4.7 @ 35% NBS.

7. Parapet - Face Loading

Use parts & components - Section 8 of NZS 1170.5:2004

Cat. P1 ⇒ $R_p = 1.0$

$= C_{Hi}\ C_i(T_p)$ $h_i = 8.2$m
$h_n = 8.9$m

$C_{Hi} = \left(1 + \frac{8.2}{6}\right) = 2.37$

$C_i(T_p) = 2.0$ $T_p \le 0.75$ s.

$C(0) = 1.12$ Table 3.1 for $T = 0$

$C_p(T_p) = 1.12 \times 2.37 \times 2$
$= 5.30$

$F_{ph} = C_p(T_p)\ C_{ph}\ R_p\ W_p \le 3.6\ W_p$
$= 5.30 \times 1 \times 1 \times W_p$

use $3.6\ W_p$

Parapet continued

$W_p = 10.5$ kN/m /m

$F = 10.5 \times 3 = 31.5$ kN/m

$M^* = 31.5 \times \frac{1.1^2}{2}$

$= 19.05$ kNm /m run.

have $\frac{1}{2}''$ bars @ $12''$ crs $A_s = 415\ mm^2/m$

$d = 300mm$

$\phi M_u = 0.85 \times 415 \times 275 \left(300 - \frac{0.59 \times 415 \times 275}{20 \times 1000}\right) \times 10^{-6}$

$= 28.8$ kNm OK

Tall parapet @ 100% NBS under face loading

Small parapet

$W_p = 6.4$ kN/m

$F_p = 3 \times 6.4$

$= 19.2$ kN/m

$M^* = 19.2 \times \frac{0.7^2}{2}$

$= 4.7$ kNm /m

have $\frac{1}{2}''$ bars @ $12''$ crs centrally placed

$A_s = 415\ mm^2$ /m $d = 100$

$\phi M_u = 0.85 \times 415 \times 275 \left(100 - \frac{0.59 \times 415 \times 275}{20 \times 1000}\right) \times 10^{-6}$

$= 9.4$ kNm OK

Low parapet @ 100% NBS under face loading

Sub floor bracing – Front Part of Building

Floor area = 8 × 19.6 = 156.8 m²

$\gamma_a = 0.3 + \frac{2.7}{\sqrt{156.8}}$
= 0.52

floor supports seismic floor load only, as there is a perimeter foundation wall/strip footings.
There are also strip footings under the brick walls/piers.

wt = (0.5 + 3×0.3×0.52) 156.8
= 151.8 kN

Eu = 0.99 × 151.8 = 150.1 kN

piles :- have 3 × 9 = 27 piles
= 5.57 kN /pile

have 140×140 timber piles – cantilevers but depth of embedment unknown

height – assumed 1 m

M^* = 1 × 5.57 = 5.57 kNm

$\phi M_n = \phi k_1 k_4 k_5 k_8 f_b Z$
= 0.8×1×1×1×1×14× 0.457
= 5.1 kNm @ 92% NBS

but depth of pile embedment unknown

assume 50% NBS

auditorium have 200×200 piles

area 16 × 30 = 480 m²
γ_a = 0.5

Eu = 0.99(0.5 + 4×0.3×0.5) 480
= 522.7 kN

piles 2×2 grid = 7×14 = 98 piles = 5.33 kN/pile

Piles 1.7 high M^* = 5.33 × 1.7 = 9.1 kNm assume single bar

ϕM_n = 3.25 kNm

concrete piles @ 35%