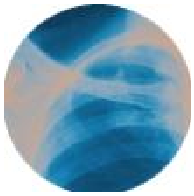
MICHAEL FOWLER CENTRE

DETAILED SEISMIC ASSESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS



MICHAEL FOWLER CENTRE

DETAILED SEISMIC ASSESSMENT REPORT

PREPARED FOR

WELLINGTON CITY COUNCIL

107303

NOVEMBER 2014

MICHAEL FOWLER CENTRE

DETAILED SEISMIC ASSESSMENT (DSA) – USING NLTHA

Prepared For: Wellington City Council

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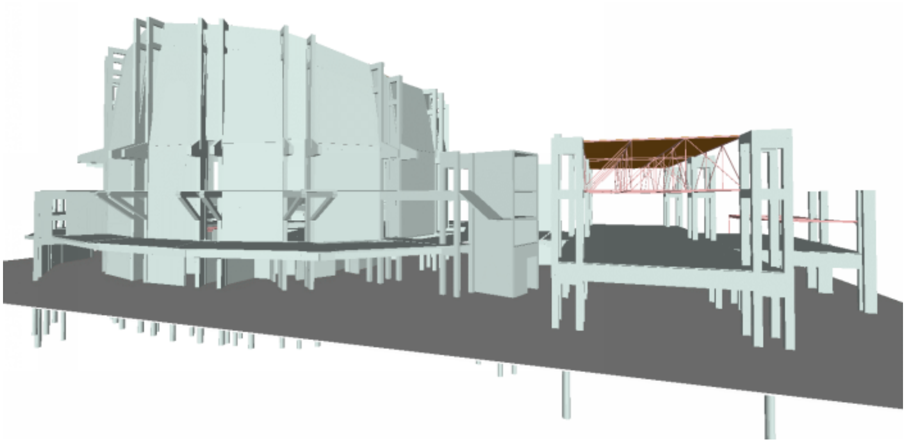
The Michael Fowler Centre was designed in 1980 to seismic loadings that are between one third and half of the seismic loadings that would be required under current seismic design codes. A Detailed Seismic Assessment (DSA) reported here used the nonlinear time history analysis (NLTHA) procedure to assess the seismic performance of the building.

Computer Model of Complex



Summary of Ground/Foundation Performance

A Geotechnical Desktop Assessment has been undertaken for this site by Tonkin & Taylor (T&T) as outlined in their 15 August 2014 correspondence. This assessment indicates liquefaction and lateral spread conditions exist for the near surface reclamation fills on this site, under moderate levels of earthquake ground shaking, increasing in severity as earthquake accelerations increase. Building foundations are predominantly (relatively) slender "Franki" piles extending down through the reclamation to the underlying alluviums. Piles frame into relatively substantial ground beams or pile caps, however these do not tie across separate buildings (e.g. no foundation beams connect the Auditorium to Stair Blocks, nor Stair Blocks to Foyer, nor between the Stair Blocks).

Assessment of piles indicates lateral displacement capacity in the order of 200mm and even beyond those displacements, contribution of the foundation beams will help limit gross building displacements to some extent. Based on T&T "expected" lateral displacements being less than 200mm at 34% seismic load levels, we do not expect the ground conditions to render the building earthquake prone. However, at higher load levels, and if building "strengthening" is proposed, this will need to carefully consider the effects of lateral spread with the likely need to provide ground improvement measures to reduce likely lateral spread displacements. We have noted that any such works may provide best "value" if undertaken on the lagoon side of Jervois Quay, in order to enhance resilience of both the main Jervois Quay roadway and City to Sea Bridge.

Summary of Building Performance

The Michael Fowler Centre comprises structurally separate structures (Auditorium, Stair Blocks and Foyer) which were included in a single model but the evaluation was performed separately on the three buildings. This is because the structures have varying levels of seismic resistance and excessive displacements in any one building would terminate the analysis if all buildings were included. The physical connections between the buildings are not robust and so it is considered appropriate to model them separately in the as-is condition. If strengthening is to be implemented, it may be better to consider all buildings as a single unit.

Our assessment estimates that the three main structures have an ultimate strength DBE (ULS) between 67% and 83% of the load specified in NZS1170 for new buildings. However, all three buildings have lower levels of ductility and resilience than required for new buildings to avoid collapse and so the overall global building rating is approximately 50-60 %NBS.

In addition, detailed seismic assessment of some specific building components has shown that several of these items have seismic capacity less than this 50-60 %NBS rating, some of which fall below the 34 %NBS Earthquake Prone threshold. Specific component performance of note is summarised as follows;

- Stairs 7 and 8 (external Fire Egress to side of main Stair Blocks) – rigid connection between levels and across Stair Block to Auditorium structures and foundations. Susceptible to both inter-storey displacement, relative displacement between independent buildings and differential foundation movement (lateral spread). Remediation necessary – and currently considered Earthquake Prone.
- Stairs 9 and 10 (external Fire Egress from Renouf Foyer) – poor detailing around the top flight sliding connection at Foyer floor level. Whilst this independent stair might not be considered Earthquake Prone the detailing of the top flight connection warrants remediation.
- Stairs 15 and 16 (high level stairs at northern end of building connecting function rooms) – rigidly connected across three floor levels (two major stair flights). ULS capacity as low at 15 %NBS (ULS). As such, deemed Earthquake Prone and remediation is recommended.
- Auditorium structure adjacent Stair Blocks (Bays 6/7, 6a/7a) – have unconnected foundations and are prone to differential foundation movement (lateral spread). Ground floor column remediation recommended. Capacity is subject to degree of differential lateral ground movement (can tolerate up to 100mm lateral differential movement). Assuming "expected" lateral displacement (as reported by T&T) and 50% differential displacement, capacity of these two towers will be greater than 34% NBS. However, strengthening is recommended.
- Auditorium Roof – hollowcore units are supported on steel trusses with minimal seating. Building finishes (soffit insulation and top surface waterproofing) limit the access for inspection. Our assessment concludes the roof capacity is not less than the overall Auditorium structure. However, additional inspection of hollowcore unit soffit is recommended, as is some supplementary hollowcore support, in particular along the main roof ridge line.

Overall building seismic ratings are estimated as outlined in Table ES-1 below. In considering variations in S_p factor and a desire to maintain some margin to Collapse Limit State (CLS) an approximate 50-60 %NBS rating across all three structures is deemed appropriate, excluding the specific vulnerabilities outlined above.

Table ES-1: Seismic Performance as IL3 Buildings

	Fraction of NZS1170 Load at which limit state is reached		Margin to DBE(CLS)	Nominal Overall Building Rating
	DBE (ULS) S_p=0.7	DBE (ULS) S_p=1.0		
Auditorium	67%	55%	1.35	≈55-60 %NBS
Entrance Foyer	83%	70%	1.0	≈50-55 %NBS
Stair Blocks	70%	60%	1.15	≈50-55 %NBS

The seismic evaluation has been restricted to the seismic performance of the Michael Fowler Centre. Continued operability of a building after an earthquake is not assured in the absence of structural damage as damage to secondary components including, façade systems, glazing, building services and other non-structural components and contents may impair functionality. The seismic resistance of these items has not been assessed.

1. INTRODUCTION

1.1 THE MICHAEL FOWLER CENTRE

The Michael Fowler Centre was opened in 1983 as a new concert hall and conference venue for the city. Warren & Mahoney were the project Architects and Holmes Wood Poole & Johnstone the project Structural Engineers. Originally intended to replace the existing Wellington Town Hall, this building draws heavily on the Christchurch Town Hall for Performing Arts, completed by the same consultants some 10 years earlier, with similar structural and architectural forms.

Figure 1-1 shows an aerial view of the Michael Fowler Centre and adjacent buildings. The building is located within the Civic Square complex of buildings, including the Central Library, Wellington Art Gallery, City Council office buildings and the Wellington Town Hall. The Michael Fowler Centre is located immediately to the east of the existing Wellington Town Hall building, bounded to the east by Jervois Quay (Figure 1-2), to the south by Wakefield Street (Figure 1-3), and the Capital E building to the north.

Figure 1-1 Aerial Photo of Michael Fowler Centre and Surrounding Buildings

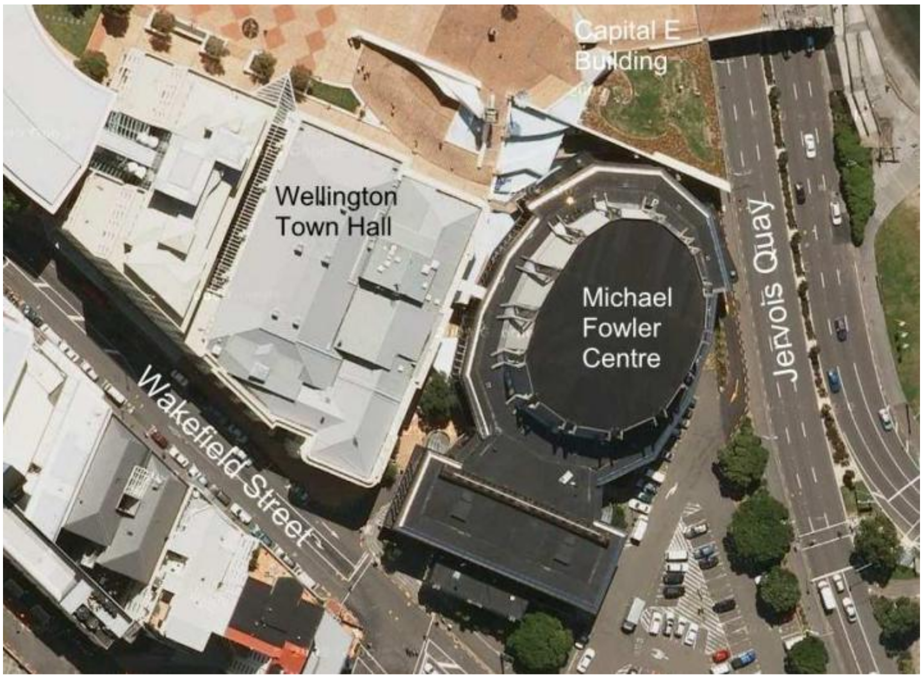


Figure 1-2 Michael Fowler Centre from Jervois Quay, showing Auditorium

Figure 1-3 Michael Fowler Centre from Wakefield Street, showing Foyer Building

1.2 PREVIOUS SEISMIC EVALUATIONS

A qualitative seismic review was performed in November, 2011 [1]. In this review, a detailed analysis of the building was not performed, with the seismic rating based on a comparison of original design loads to current code values along with a review of the building drawings and details. This review indicated that the original building was designed for between 34% and 48% of full current code level loads that would be applicable for the building if designed today.

This includes the requirement to design a Public Building, containing people in crowds, for a 1000 year return period earthquake.

The review also noted that there are several building components that may exhibit less than desirable performance in a large earthquake. These details may need to be addressed as part of any seismic upgrade work to the building so that they did not limit the overall performance of the building. Some of the components that warrant further investigation in this regard include;

- Seismic gap clearances.
- Stair detailing, particularly within the Auditorium.
- Upper level promenade floor diaphragm details.
- Detailing and performance of the roof diaphragm.
- Detailing of the main lateral components (columns and walls) and their ability to act in a "ductile" manner.

This quantitative review is intended to provide a more detailed evaluation of the seismic performance relative to current New Building Standard (NBS) and also address the component issues identified above as far as practical.

1.3 SCOPE OF WORK

The evaluation is restricted to an assessment of the resistance to earthquake loads of the structural system and does not consider gravity load capacity or the performance of non-structural components and contents. (This may be considered separately as the design phase proceeds).

The scope of work for this evaluation is defined as:

1. Source all available documentation and drawings of the existing building and any alterations / works that have been undertaken since construction.

2. Carry out site visits to identify variations from the documentation.

3. Perform a detailed non-linear time history of the existing auditorium, foyer and stair buildings to validate and quantify the seismic performance.

4. Prepare a Detailed Seismic Assessment (DSA) report, summarising the overall building performance and discussing in concept any strengthening required.

1.4 INFORMATION USED FOR THE EVALUATION

The information used for the analysis was a complete set of structural drawings dated between 1979 and 1980 for the original buildings plus kitchen extension drawings dated 1989.

1.5 EVALUATION DATA AND COMPUTER FILES

The nature of the time history method of analysis used for this evaluation is such as to produce a large amount of data. A summary of this data is provided in this report and if more detail is required it can be provided in digital format as follows:

1. All input is contained in Excel© workbooks. These contain geometric, material, section and strength data.

2. A summary of all output is also contained in Excel© workbooks. These contain maximum forces and deformations in every component and also the evaluation of these components against project criteria.

3. Damage files are produced from each output workbook. These display damage on a rendered image of the model.

4. Time history output is produced from every analysis. This includes time histories of accelerations and/or displacements plus animations of structural response showing damage. Because of the volume of data within these files, and the relative ease to re-create them, these files are not saved once processed.

5. Similarly, animated mode shapes are not saved but can be simply re-produced if required.

The damage files and animations are in a program specific format but a copy of the program used to display these (Showtime) can be provided on request.

1.6 LIMITATIONS

Findings presented as a part of this project are for the sole use of Wellington City Council. The findings are not intended for use by other parties, and may not contain sufficient information for the purposes of other parties or other uses. Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practising in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

Conclusions relate to the structural performance of the building under earthquake loads. We have not assessed the live load capacity of the floors, nor have we assessed the performance of secondary components including, façade systems, glazing, building services and other non-structural components and contents. The seismic resistance of these items has not been assessed.

2.1 PERFORMANCE LEVELS

The amplitude of seismic loads in NZS1170.5 [2] is a function of the use of the structure, as listed in Table 2-1. The seismic load level of "normal" buildings is defined by an R factor of unity, and seismic loads are increased by 30% for structures with crowds or of high value and by 80% for structures with special post-disaster functions.

The Building Act also defines a building as earthquake prone if it is unsafe at a level of earthquake one-third that required for new buildings.

The Michael Fowler Centre is defined as an Importance Level 3 structure, with an R factor of 1.3. The NZS1170.5 seismic input is interpreted as representing the Ultimate Limit State (ULS) loads. A further limit state, the Collapse Limit State (CLS) which has higher acceptance criteria is defined as 1.5 times the ULS. This is based on the NZS1170.5 Commentary which states that a margin of at least 1.5 to 1.8 ULS against collapse will be available.

Table 2-1 Performance Levels

Importance Level	Earthquake Annual Probability of Exceedance	Comment	Examples
2 IL2	1/500 R = 1.0	Normal structures and structures not in other importance levels.	Hotels, offices, apartments
3 IL3	1/1000 R = 1.3	Structures that may contain people in crowds or contents of high value to the community.	Emergency medical and other emergency facilities not designated as post-disaster.
4 IL4	1/2500 R = 1.8	Structures with special post-disaster functions.	Designated civilian emergency facilities, medical emergency facilities.

2.2 SEISMIC LOADS

Seismic loads were based on the requirements the current loading code NZS1170.5:2005 which locates Wellington in a high seismic zone with a zone factor of 0.40.

Based on foundation investigations at the time the structure was designed, the site was classified as a 'flexible' subsoil site to NZS4203:1976. A current Wellington CBD site subsoil classification map shows that the Michael Fowler Centre is located in transition zone between a site subsoil "C" or 'Shallow soil" and a site subsoil "D" or 'Deep/soft soil" classification to NZS1170.5:2004. For the purposes of this assessment a site subsoil "D" or 'Deep/soft soil" classification has been assumed. The seismic parameters used for the site were as listed in Table 2-2.

Table 2-2 Seismic Parameters

Design Code :	NZS1170
Soil Category :	D
R :	1.3
Z :	0.40
L_u :	1.0
S_p:	0.70 – 1.0 (varies as below)
D :	2 km

The structural performance factors, S_p, has been defined as 0.70 which is the appropriate value for structures of ductility 2 or higher. The capacity curves presented later (see Figures 5-1, 5-2 and 5-3) show that the Auditorium does not have a clearly defined yield point so it is difficult to assess the ductility. The value of S_p for non-ductile structures is 1.0. If this value were used the scaling factors below would be higher by a factor of 1 / 0.85. In this case, the assessed structural capacity would be 0.85 times that reported here.

2.3 NZS1170 TIME HISTORY RECORDS

Table 2-3 lists the three earthquake records used, together with the scaling factors calculated for each building in the complex. The NZS1170 scaling procedure is based on the fundamental period of the structure in each orthogonal horizontal direction, with the value taken as not less than 0.40 seconds. The scale factors were calculated separately using the periods for the three separate structures as listed in Section 5 of this report.

Table 2-3 lists the scaling factors for the three selected records. Figure 2-1 shows the envelope spectra of the scaled records, compared to the design spectrum. Figures 2-2 to 2-4 plot the individual scaled records for the T = 0.40 second option. The tabulated factors and plots are for R = 1.3. As this building contains crowds, the appropriate R factor is 1.3 and this factor is applied to the listed scale factors.

Of the three selected records, the last two contain forward directivity effects which are appropriate for a site located near an active fault.

Table 2-3 Earthquake scaling factors R =1.3

Earthquake	Auditorium	Foyer	Stair	Forward Directivity?
USA, El Centro Imperial Valley (USA) 1940	2.08	2.07	1.89	NO
USA, El Centro Imperial Valley 1979, Array 6	1.95	1.94	1.69	YES
Yarimka YPT, Kocaeli, Turkey 1999	2.23	2.17	2.01	YES

Figure 2-1 Envelope of Scaled Earthquake Records

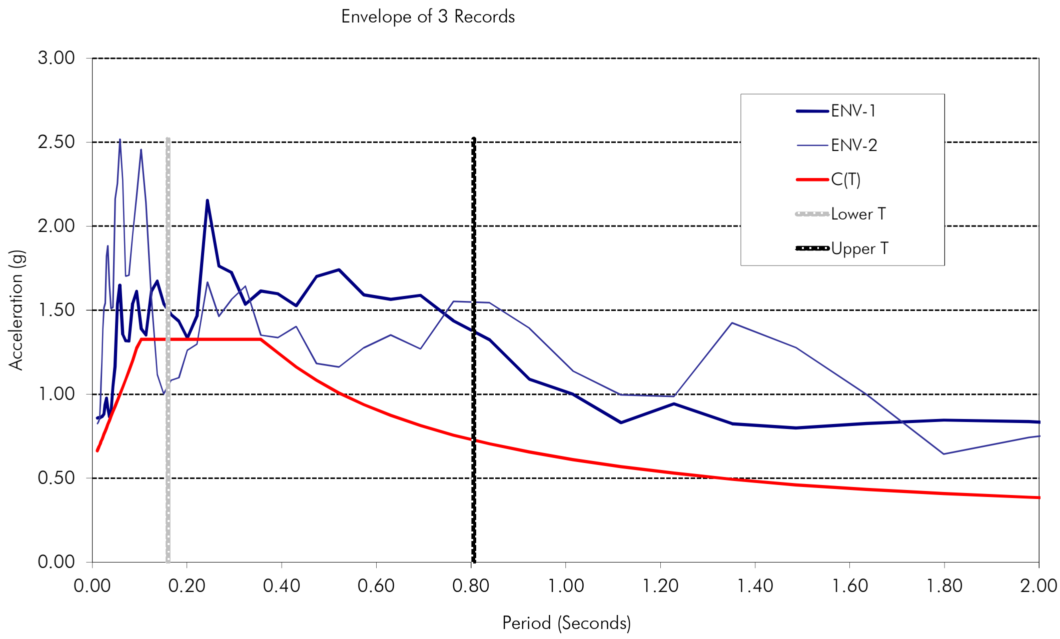


Figure 2-2 Scaled Earthquake Record 1

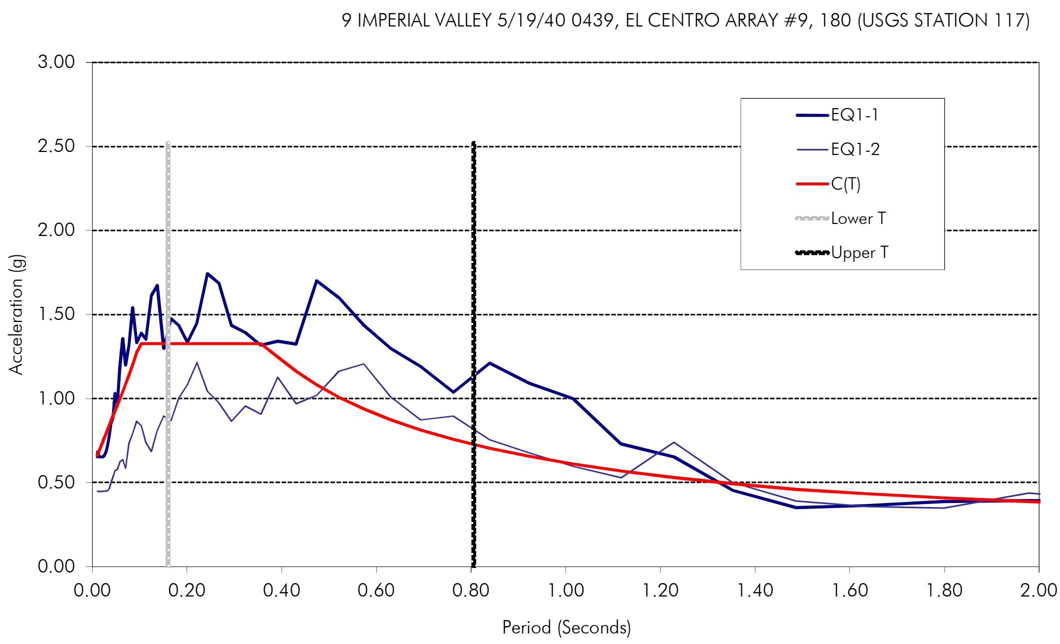


Figure 2-3 Scaled Earthquake Record 2

10 IMPERIAL VALLEY 10/15/79 2316, EL CENTRO ARRAY #6, 140 (CDMG STATION 942)

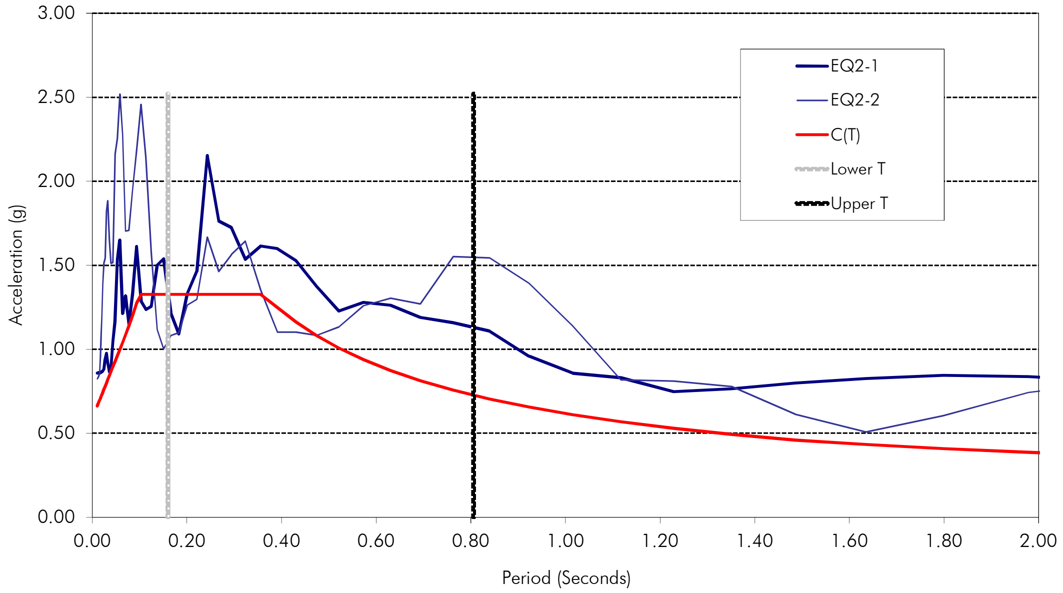
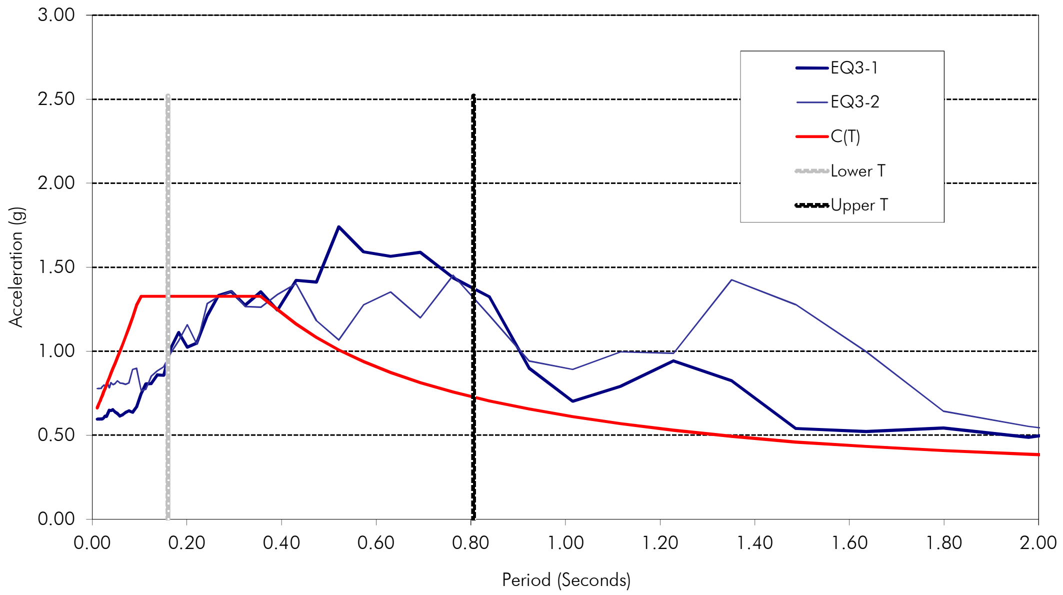


Figure 2-4 Scaled Earthquake Record 3

41 Yarimka YPT, Kocaeli, Turkey 17/08/1999



3. EVALUATION PROCEDURE

The structure was evaluated using the HCG nonlinear analysis procedure. This is based on a linked series of modules:

1. MODELA, an Excel spreadsheet for preparing input data and writing input files

2. ANSR-II, a general purpose nonlinear analysis program to analyse the structure

3. PROCESSA, an Excel spreadsheet to process output files and import envelope results.

These modules implement the time history method of analysis, as specified in NZS1170.

3.1 CURRENT STATUS OF EVALUATION CRITERIA

Following the recent Canterbury earthquake sequence, discussions within the engineering profession are underway with respect to revising the New Zealand guidelines for the assessment of existing buildings. The current guidelines are known to contain a number of errors and additional scope to cover building resilience will likely be required.

In order to continue evaluation while this development continues, HCG have issued three practice notes which incorporate the current state of practice.

- PN30.1 Evaluation of the Seismic Performance of Existing Buildings
- PN30.2 Linear Analysis Methods and Acceptance Criteria
- PN30.3 Non-linear Analysis Methods and Acceptance Criteria

These practice notes represents current knowledge at the time of writing. However, it is expected that they will be updated in due course to reflect any amendments to the New Zealand guidelines.   The performance objectives and acceptance criteria are generally based on these practice notes, in so far as they can be implemented with the non-linear analysis tools currently available.   Where the currently available New Zealand guidelines do not give adequate guidance, references has been made to other relevant sources, primarily the US guidance ASCE41-06.

3.2 BUILDING PERFORMANCE OBJECTIVES

Recommended building performance objectives have been summarised in Table 3-1. These performance objectives are based on the recommendations of the NZSEE (NZSEE, 2006) and have been further developed following the Canterbury earthquakes (EAG, 2011).

Table 3-1 Building Performance Objectives

Building Performance Level	Earthquake Hazard Level	
	Performance Limit State	
	Ultimate Limit State	Collapse Limit State
New Building - Minimum Legal Standard	100% DBE	150%+ DBE [1]
Existing Building - Recommended Minimum	67% DBE	100% DBE
Existing Building - Legal Minimum	34% DBE [2]	– [3]

Notes:

1. There is no specific requirement to check the CLS when designing new buildings. However, it is implicit in the relevant standards that new buildings should achieve in excess of 150% DBE at the CLS

2. Based on the definition of an EPB. This only forms a minimum legal requirement when required by the Territorial Authority

3. There is currently no legal requirement to consider the CLS in the assessment of existing buildings. However, the definition of an EPB as 33% NBS at ULS is equivalent to 50% DBE at the CLS

Earthquake hazard levels at the ULS recommended in Table 3-1 for existing buildings are in accordance with the New Zealand Society for Earthquake Engineering recommendations (NZSEE, 2006).

Earthquake hazard levels at the CLS have been determined by providing a margin of 1.5 above that used for the ULS. This margin has been adopted on the basis that it is at the lower end of the range that would be expected for a new building designed in accordance with NZS1170.5:2004.

The earthquake hazard levels are expressed in terms of percentage of the ULS design load (also known as the Design Basis Earthquake, DBE) and as such, are independent of building Importance Level.

3.3 TIME HISTORY ANALYSIS

The time history analysis is based on the provisions of NZS1170. A finite element model of the building is developed, including the strength of all elements. The response of the building is then evaluated under the actions of the three earthquakes described above. Each analysis involves 12,000 or more steps. At the end of the analysis, maximum forces and deformations in the structure and in every element are printed. The envelope values from all analysis variations are accumulated and used to evaluate performance.

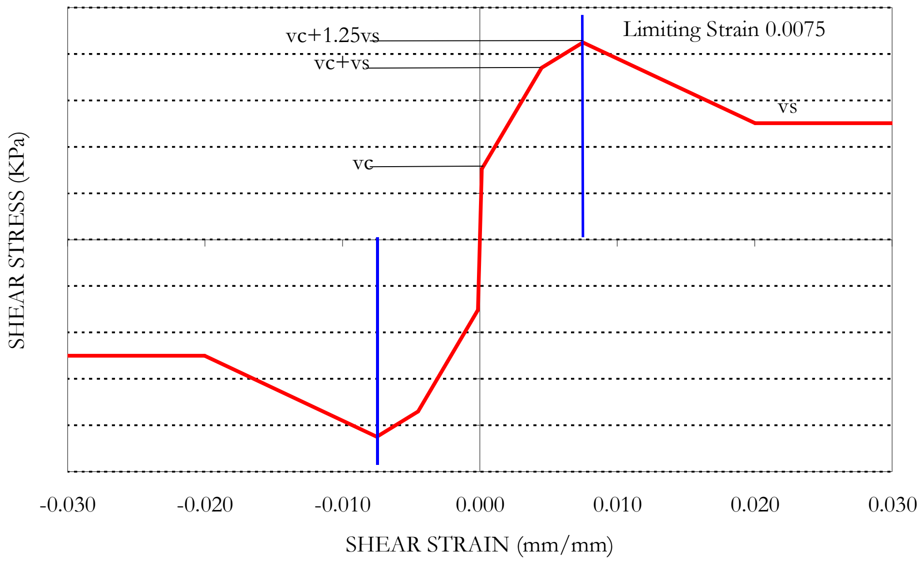
3.4 COMPONENT MODELLING

3.4.1 Concrete Wall Elements

The shear stiffness of the concrete foundation shear wall elements are modelled using plane stress elements with a thickness corresponding to the values specified on the drawings.

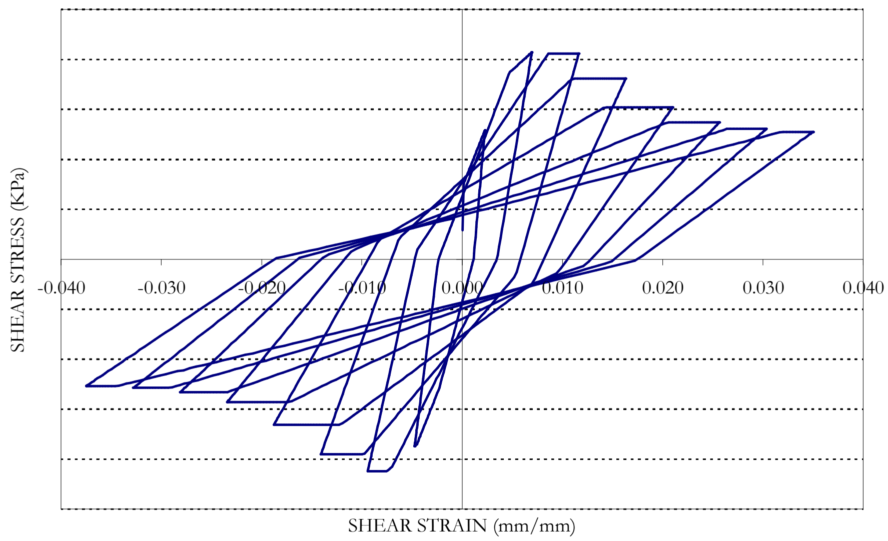
The shear stiffness of the element may include degradation in strength and stiffness, depending on the level of shear stress. The wall panel yield function is shear controlled. A strength envelope is developed, with three regions defined by the strength provided respectively by the concrete, the nominal strength of the shear reinforcing (strain of 0.0045) and the over-strength of the reinforcing (strain of 0.0075).

Figure 3-1 Wall Shear Strength Envelope



When the shear stress reaches the strength envelope the stiffness reduces to the secant stiffness for unloading and this reduced stiffness is maintained for reversed loading, as shown in Figure 3-2. The stiffness degradation is non-recoverable.

Figure 3-2 Shear Hysteresis



The panel element models the shear degradation of walls. For slender walls, flexural yielding may occur at the base before the shear strength is reached. To model flexural yielding, gap elements are placed at each node across the length of the wall at the elevation at which yielding is expected to occur. Each gap element contains two elements in parallel, a concrete element which is elastic in compression but with zero tensile strength and a reinforcing bar element which is bi-linear, with yielding in both tension and compression.

The gap elements are pre-loaded by gravity loads on the structure. The concrete area and steel area at each gap is taken as the tributary areas of all panels incident to each gap.

3.4.2 Beam and Column Elements

For beam and column elements the strength is modelled as a bi-linear yield function. The elastic stiffness is based on effective properties up to the calculated nominal yield moment. Properties are defined by axial area, shear area and moment of inertia about each axis.

Beams have a separate yield moment specified for positive and negative bending at each end of the beam. Once the yield moment is attained, the flexural stiffness is reduced to the initial stiffness times the specified strain hardening ratio.

Columns are represented by a flexural element similar to beams. However, the yield moments about each axis and the axial load are coupled. An interaction diagram is calculated based on nominal material strengths. The interaction between bending moments and axial load is defined by:

$$\sqrt{\left(\frac{M_y}{M_{yu}}\right)^2 + \left(\frac{M_z}{M_{Zu}}\right)^2} + \left(\frac{F - F_o}{F_u}\right) = 1.0$$

where

$$F_o = \frac{1}{2}(F_{ut} - F_{uc})$$

$$F_u = \frac{1}{2}(F_{ut} + F_{uc})$$

M_y, M_z and F denote bending moments about the element y and z axes and the axial force respectively. Subscript u denotes ultimate. F_{ut} and F_{uc} are axial ultimate strengths in tension and compression. As for the beams, a bilinear strain hardening yield function is generally used in the model.

The flexural element used for beams and columns permits degrading strength and/or stiffness characteristics to be specified. The ASCE41-06 guidelines provide limiting plastic rotations beyond which strength degradation is assumed to occur and these limits are incorporated in the analysis procedure.

3.5 PERFORMANCE EVALUATION CRITERIA

3.5.1 Global Performance Criteria

NZS1170 provides drift limits of 2.5% for all structures evaluated using the time history method. However, specific element limits as described in the following sections provide restrictions on the element deformations which act to reduce the effective allowable drifts.

For concrete shear walls controlled by shear capacity the ASCE41-06 limits used restrict maximum drift to 0.75%. For shear walls controlled by flexure the drift limit is based on a maximum drift after yield of between 0.8% and 1.5%, which provides a total drift limit of approximately 1% to 1.7% depending on the drift level at yield. The wall segments in these buildings are typically relatively squat such that they are shear controlled and so the walls are governed by the lower limit, 0.75%.

Provided the overall drifts are within this level, it is generally possibly to use the existing structural system to provide seismic resistance, although individual elements with deficiencies may need remedial strengthening. If the drift levels exceed these limits then it is likely that the existing structural system will need to be augmented with additional strengthening elements.

3.5.2 Evaluation of Structural Components

The primary evaluation criteria are for the ULS loads, where components are required to meet the ASCE-41 Life Safety (LS) requirements. Some judgement has been used in deciding strengthening requirements based on the CLS, where components are ranked as failing if they exceed the Collapse Prevention (CP) limit state. It would likely be too restrictive to strengthen every component > CP, and so they will be assessed as to the extent they exceed CP and also the consequences of failure of the component.

For both limit states, the ASCE-41 secondary criteria will be used as the elements include both stiffness and strength degradation.

All structural components were evaluated and their response characterised into one of the four categories listed in Table 3-2. The first category, serviceability, defines the ability of the structure to remain operational immediately post-earthquake. The second and third categories, ultimate and collapse limit states, define the ability of the structure to preserve life safety and avoid total failure respectively. Any component in the last limit state, failure, presents a collapse hazard.

Table 3-2 Limit States

Rating	Limit State	ACSE 41
1	Serviceability, SLS2	< IO
2	Ultimate, ULS	< LS
3	Collapse, CLS	< CP
4	Failure, FAIL	> CP

At each level of seismic input (ULS and CLS) each component is assessed to determine whether it is deficient and, if so, whether the deficiency is non-critical or critical:

1. A non-critical deficiency (NCD) indicates damage which is unlikely to lead to collapse.

2. A critical deficiency (CD) indicates loss of capacity which may lead to instability of the structure or partial or full collapse.

A component with a NCD does not need strengthening for the building to meet either the ULS or CLS. A component with a CD usually requires strengthening. Table 3-3 lists the criteria used to assess the overall structure and also each of the types of structural elements in the complex (beams, columns and walls).

Table 3-3 Critical Deficiencies Used to Assess Limit States

Component Type	Action	Serviceability SLS2	Ultimate ULS	Collapse CLS
Structure	Drift	-	0.025 0.037 FD(1)	0.0375 0.056 FD
Beams	Flexure	CD(2) if θ_{PL} (4)> IO	NCD (3)	NCD
	Shear	CD if $A_V < A_{VREQD}$	NCD	NCD
	Confinement	NCD	NCD	NCD
Beams Supporting Precast Floors	Flexure	CD(2) if θ_{PL} (4)> IO	CD (8) $\theta_{PL} > LS$	CD $\theta_{PL} > CP$
	Shear	CD if $A_V < A_{VREQD}$	CD if $A_V < A_{VREQD}$	CD if $A_V < A_{VREQD}$
	Confinement	NCD	CD	CD
Columns	Flexure	CD $\theta_{PL} > IO$	CD $\theta_{PL} > LS$	CD $\theta_{PL} > CP$
	Shear $P < 0.05\ f'_C$	CD if $A_V < A_{VREQD}$	CD if $A_V < A_{VREQ}$ & $\theta_{PL} > 0.0075$	CD if $A_V < 0.85A_{VREQ}$ & $\theta_{PL} > 0.0100$
	Shear $0.05\ f'_C < P < 0.15\ f'_C$	CD if $A_V < A_{VREQD}$	CD if $A_V < A_{VREQ}$ & $\theta_{PL} > 0.0055$	CD if $A_V < 0.85A_{VREQ}$ & $\theta_{PL} > 0.0075$
	Shear $P > 0.15\ f'_C$	CD if $A_V < A_{VREQD}$	CD if $A_V < A_{VREQ}$ & $\theta_{PL} > 0.0000$	CD if $A_V < 0.85A_{VREQ}$ & $\theta_{PL} > 0.0000$
	Confinement	CD if $A_{SH} < A_{SHREQD}$	CD if $A_{SH} < A_{SHREQ}$ & $\theta_{PL} > LS_P$	CD if $A_{SH} < A_{SHREQ}$ & $\theta_{PL} > CP_P$
Wall Panels	Flexure(4) $P < 0.10\ f'_C$	NCD $\theta_{PL} > IO$	NCD $\theta_{PL} > LS$	NCD $\theta_{PL} > CP$
	Flexure(4) $P > 0.10\ f'_C$	CD $\theta_{PL} > IO$	CD $\theta_{PL} > LS$	CD $\theta_{PL} > CP$
	Shear $P < 0.05f'_C$	NCD if $\gamma > 0.004$	NCD if $\gamma > 0.015$	NCD if $\gamma > 0.020$
	Shear $0.05\ f'c < P < 0.15\ f'_C$	CD if $\gamma > 0.004$	CD if $\gamma > 0.0075$	CD if $\gamma > 0.010$
	Shear $0.15\ f'c < P$	CD if $\gamma > 0.004$	CD if $\gamma > 0.0045$	CD if $\gamma > 0.0045$

NOTES:

[1] FD indicates the results from an input record which includes forward directivity effects.
[2] CD indicates a critical deficiency for the appropriate limit state.
[3] NCD indicates a non-critical deficiency for the appropriate limit state.
[4] θ_{PL} is the envelope plastic rotation in an element.
[5] γ indicates the envelope shear strain in a panel element.
[6] A_V is the area of shear steel in a section and A_{VREQD} is the area required for maximum envelope imposed actions.
[7] A_{SH} is the area of confining steel in a section and A_{SHREQD} is the area of confining required for maximum envelope imposed actions.
[8] Failure modes of reinforced concrete beams that support precast floor units shall be classified as critical if the failure could lead to progressive collapse of floor units below.

Note that the criteria do not necessarily define insufficient shear reinforcing as a critical deficiency unless axial stresses are high or plastic rotations are high. The criteria are based on permitting a shear strain in the column (assumed equal to the plastic rotation) equivalent to the shear strain permitted in wall panels controlled by shear.

3.5.3 Evaluation of Concrete Wall Performance

For walls dominated by shear, the drift limits are as listed in Table 3-4. The peak shear strain for the LS and CP limit states equals or exceeds the strain at which strength degradation occurs, 0.0075 (Figure 3-1). An exception is where the axial load exceeds $0.15A_g f'_c$, in which case the element is force controlled and the limiting strain is that at which the elastic strength of the wall is reached, a strain of 0.0045 for both LS and CP limits.

The criteria in Table 3-4 relate to acceptable drift over the height of the wall. These are generally applied to the shear strain in each individual shear panel in the model. In some configurations there may be local strain increases when small elements are used. The shear strain in these elements may exceed the drift limit even though the global drift is within the limit. In these cases, an assessment of the full extent of the wall may result in local over-limit strains being acceptable.

Table 3-4 Acceptance Criteria for Shear Walls: Shear

Condition	Acceptable Drift (%)		
	IO	LS	CP
Axial stress ≤ 0.05f'c	0.004	0.015	0.020
0.05 f'c < Axial Stress ≤ 0.15f'c	0.004	0.0075	0.010
Axial Stress > 0.15f'c	0.004	0.0045	0.0045

For shear walls dominated by flexure, acceptance criteria relating to the maximum plastic rotation in the wall are listed in Table 3-5. The limits are a function of the axial stress, shear stress and whether or not boundaries are confined. A confined boundary has closed stirrups at less than d/2 which are capable of resisting the total shear in the boundary element. The plastic rotation is calculated as the width of the crack opening in the gap element used to define flexural yielding divided by the length of the shear wall.

Table 3-5 Acceptance Criteria for Shear Walls: Flexure

			Plastic Rotation (Radians)		
$P/A_g f'_c$	Confined Boundary	$V / b_w d\sqrt{f'_c}$	IO	LS	CP
≤ 0.1	Yes	≤ 0.25	0.005	0.015	0.020
≤ 0.1	Yes	≥ 0.50	0.004	0.010	0.015
≥ 0.25	Yes	≤ 0.25	0.003	0.009	0.012
≥ 0.25	Yes	≥ 0.50	0.0015	0.005	0.010
≤ 0.1	No	≤ 0.25	0.002	0.008	0.015
≤ 0.1	No	≥ 0.50	0.002	0.006	0.010
≥ 0.25	No	≤ 0.25	0.001	0.003	0.005
≥ 0.25	No	≥ 0.50	0.001	0.002	0.004

3.5.4 Concrete Beam and Column Evaluation

Table 3-6 lists the acceptance criteria for beam plastic rotations and Table 3-7 the equivalent values for columns. The procedure for evaluating each beam element is:

1. Assess whether transverse reinforcing is conforming.
2. Interpolate between values in the table depending on the reinforcing ratio and applied shear stress.
3. Check whether the envelope plastic rotation exceeds the allowable value.

Table 3-6 Criteria for Beams

			Plastic Rotation (Radians)		
$\frac{\rho-\rho'}{\rho_{bal}}$	Trans. Reinforcement	$V / b_w d\sqrt{f'_c}$	IO	LS	CP
≤ 0.0	C	≤ 0.25	0.010	0.020	0.050
≤ 0.0	C	≥ 0.50	0.005	0.020	0.040
≥ 0.5	C	≤ 0.25	0.005	0.020	0.030
≥ 0.5	C	≥ 0.50	0.005	0.015	0.020
≤ 0.0	NC	≤ 0.25	0.005	0.020	0.030
≤ 0.0	NC	≥ 0.50	0.0015	0.010	0.015
≥ 0.5	NC	≤ 0.25	0.005	0.010	0.015
≥ 0.5	NC	≥ 0.50	0.0015	0.005	0.010

For columns, the HCG procedure has recently been modified to incorporate the ASCE-41 modelling procedures and acceptance criteria for columns, as listed in Table 3-7. This is implemented as part of the time history analysis. At each time step, the axial and shear stress are evaluated and the parameters modified depending on the current conditions. The maximum ratio of demand to capacity is retained and output at the end of each analysis. This is then imported to the processing workbook.

3.6 FLOORS

The Michael Fowler Centre complex has a complex flooring system, comprising mainly precast floor panels with a cast-in-place topping. Some floor units are sloping as they form seating and galleries. These floors were explicitly modelled using plane stress panel elements as for the shear wall segments described above. In general, the structural thickness of these panels was defined as the thickness of the topping alone.

Table 3-7 Criteria for Reinforced Concrete Columns

$\frac{P}{A_g f_c'}$	$\frac{A_v}{b_w s}$	$\frac{V}{b_w d \sqrt{f'_c}}$	a	b	c	IO	LS	CP
Condition i. Columns controlled by flexure mode[1,2]								
≤0.1	≥0.006		0.027	0.034	0.2	0.005	0.020	0.027
≥0.6	≥0.006		0.035	0.060	0.2	0.005	0.026	0.035
≤0.1	=0.002		0.005	0.005	0.0	0.002	0.003	0.004
≥0.6	=0.002		0.010	0.010	0.0	0.003	0.008	0.009
Condition ii. Columns controlled by shear-flexure mode[1,2]								
≤0.1	≤0.0005	≤ 0.25	0.012	0.012	0.2	0.005	0.009	0.010
≤0.1	≤0.0005	≥ 0.50	0.006	0.006	0.2	0.004	0.005	0.005
≥0.6	≤0.0005	≤ 0.25	0.004	0.004	0.0	0.002	0.003	0.003
≥0.6	≤0.0005	≥ 0.50	0.000	0.000	0.0	0.000	0.000	0.000
≤0.1	≥0.006	≤ 0.25	0.032	0.060	0.2	0.005	0.024	0.032
≤0.1	≥0.006	≥ 0.50	0.025	0.060	0.2	0.005	0.019	0.025
≥0.6	≥0.006	≤ 0.25	0.010	0.010	0.0	0.003	0.008	0.009
≥0.6	≥0.006	≥ 0.50	0.008	0.008	0.0	0.003	0.006	0.007
Condition iii. Columns controlled by shear mode[1,2]								
≤0.1	≥0.006		0.000	0.006	0.0	0.000	0.000	0.000
≥0.6	≥0.006		0.000	0.060	0.0	0.000	0.000	0.000
≤0.1	≤0.0005		0.000	0.000	0.0	0.000	0.000	0.000
≥0.6	≤0.0005		0.000	0.008	0.0	0.000	0.000	0.000
Condition iv. Columns controlled by inadequate development[1,2]								
≤0.1	≥0.006		0.000	0.006	0.2	0.000	0.000	0.000
≥0.6	≥0.006		0.000	0.060	0.4	0.000	0.000	0.000
≤0.1	≤0.0005		0.000	0.000	0.0	0.000	0.000	0.000
≥0.6	≤0.0005		0.000	0.008	0.4	0.000	0.000	0.000
Condition v. NZS3101:2006 conforming columns[5]								
≤0.5	Ductile		$18\phi_y l_p$	$27\phi_y l_p$	0.2	0.01	$18\phi_y l_p$	$27\phi_y l_p$
>0.5			0.0	0.0	0.0	0.0	0.0	0.0
≤0.5	Limited Ductility		$10\phi_y l_p$	$15\phi_y l_p$	0.2	0.01	$10\phi_y l_p$	$15\phi_y l_p$
>0.5			0.0	0.0	0.0	0.0	0.0	0.0

[1] Refer to Section 6.4.2.2.2 ASCE 41-06 Supplement 1 for definition of conditions i, ii, and iii. Columns will be considered to be controlled by inadequate development or splices when the calculated steel stress at the splice exceeds the steel stress specified by Equation 6-2. Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.

[2] Where P > 0.7Agf'c, the column is assumed to be force controlled all performance levels unless columns have transverse reinforcement consisting of hoops with 135 degree hooks spaced at ≤ d/3 and the strength provided by the hoops (Vs) is at least three-fourths of the design shear. P is the design axial force in the member. Alternatively, use of axial loads determined based on a limit state analysis shall be permitted.

[3] Linear interpolation between values listed in the table for conditions (i) to (iv) shall be permitted.

[4] V is the design shear force calculated using limit-state analysis procedures in accordance with Section 6.4.2.4.1 ASCE 41-06 Sup.1.

[5] Detailing consistent with requirements of Section 10.4, NZS3101:2006. Values provided are for reversing plastic hinges. Deformation capacity of unidirectional plastic hinges may be taken as twice the value given.

3.7 FOUNDATIONS

Due to the variable and weak reclamation soils overlying the site, shallow foundation systems were not feasible for the Michael Fowler Centre. The building is supported on a series of enlarged base piles – cast in-situ reinforced concrete driven piles, with an enlarged base (Franki piles). A large foundation ring beam encircles the Auditorium supporting the main shear walls and piers to the Auditorium building. The Foyer building is similarly founded on enlarged base piles beneath each column group with foundation beams tying the column group foundations together. The piles were modelled individually with beam and wall elements representing the foundation components.

A desktop geotechnical assessment has been completed by Tonkin & Taylor [7] and highlights there is liquefaction and lateral spread potential at this site.

Liquefaction affects piled foundation capacities which has been accounted for in part by reducing the tension strength of the piles, as described in Section 4.3.6.1 of this report.

Lateral spreading imposes demands on the piles to accommodate additional lateral displacements. The lateral displacement capacity of piles has been considered independently of our analysis, as discussed in Section 7.2. We have estimated that piles have lateral displacement capacity in the order of 150-200mm.

4. DEVELOPMENT OF FINITE ELEMENT MODEL

4.1 BUILDING CONFIGURATION

The Michael Fowler Centre comprises two main structurally separated buildings that form the complex, the main Auditorium and the Foyer Building fronting Wakefield Street. Stair Block structures, providing stair access between the Foyer and Auditorium, are also structurally separated and are intended to respond independently from the two main building components.

The Auditorium is elliptical in plan, formed with reinforced concrete shear walls and column elements around the perimeter, supporting two tiers of seating around the stage. Two levels of promenade foyers and backstage accommodation are located around the perimeter of the Auditorium. The reinforced concrete shear walls support large precast cantilever beams off either side which support the upper tier of seating and promenades around the Auditorium.

The floors are generally made up of reinforced concrete slabs or precast double tee units supported on reinforced concrete beams and columns. The roof over the Auditorium is a concrete hollowcore slab supported in long span steel trusses, tied into the perimeter concrete walls. The building is founded on a foundation ring beam beneath the walls and a series of reinforced concrete enlarged base piles.

The Foyer Building is seismically separated from the Auditorium building and stair structures providing access to upper level promenades and the upper seating tier within the Auditorium. The original footprint of the Foyer was extended in 1989, toward the Wakefield Street frontage, to provide for additional kitchen facilities in this area of the complex. This extension is tied into the original building with additional reinforced concrete columns and foundations to provide gravity and lateral support to the extended floor area.

The Foyer Building has one suspended concrete level with a precast double tee flooring system supported on reinforced concrete beams. In the N-S direction, the group of four close coupled columns act as a one-way concrete frame in conjunction with the reinforced concrete beams supporting the floor slab. The two end frames and the central frame column cantilever above the suspended level to form a double height space. These columns support the steel roof trusses and a lightweight roof. The intermediate frames extend to the suspended floor only.

In the E-W direction, the close coupled columns are linked together with a reinforced concrete slab at floor level (and part-way up their height for the end and central frames) and act as mini frames to provide lateral resistance in this direction. Column groups are linked together with foundation beams, supported on reinforced concrete enlarged base piles.

Figure 4-1 Auditorium and Stair Blocks Plan

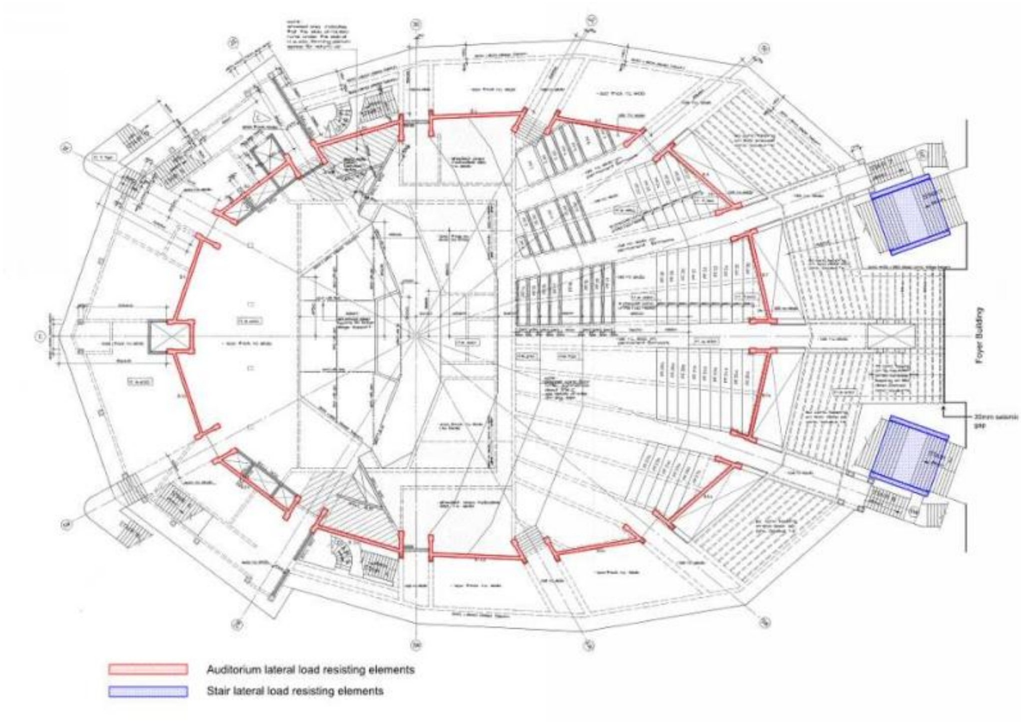
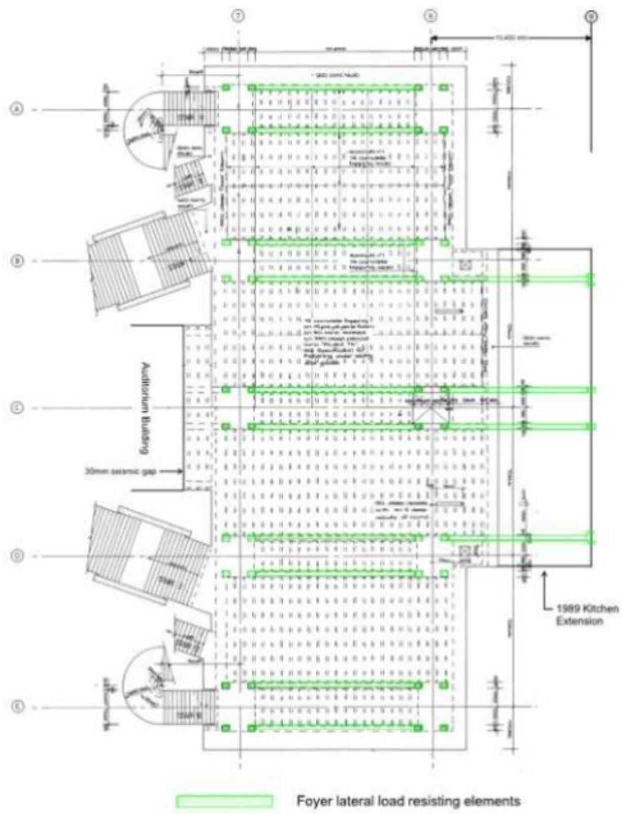


Figure 4-2 Foyer Plan, including 1989 Kitchen Extension



4.2 COMPUTER MODEL CONCEPT

The computer model defined each independent structure in the complex by a pre-defined series of column line numbers. This permits either the full or partial models to be generated by limiting the number range used to generate the model. Ranges used are listed in Table 4-1.

Table 4-1 Column Numbering Definition

Model Number	Column Line Range	Part of Complex
1	1-1999	Auditorium
2	2000-2999	Piles
3	2500-2999	Stair Blocks
4	3000-3999	Entrance Foyer

Although the Auditorium, Stair Blocks and Foyer were all included in a single model, at this stage the evaluation was performed separately on the three buildings. This is because the structures have varying levels of seismic resistance and excessive displacements in any one building would terminate the analysis if all buildings were included. The physical connections between the buildings are not robust and so it is considered appropriate to model them separately in the as-is condition. If strengthening is to be implemented, it may be better to consider all buildings as a single unit.

4.3 MODEL FEATURES COMMON TO ALL STRUCTURES

The buildings, especially the Auditorium, are of a complex geometry. As such, they are non-typical of concrete structures in that they do not all have clearly defined floor levels or rigid diaphragms. The structural model was developed based on the drawings of the existing building and the general principles for this the type of component:

1. All columns were modelled as three dimensional flexural elements. Columns were modelled as flexural elements with yield a function of bi-axial moments and axial load. Effective moments of inertia for the columns were calculated as $0.3I_g$. A strain hardening of ratio of 0.01 was used for all yielding columns. All strengths were based on the probable strengths of material with $\varphi = 1.0$.

2. Shear wall segments were modelled using degrading plane stress elements. The strength of the wall elements was based on the reinforcing ratios, both horizontal and vertical, as specified on the drawings.

3. Flexural elements are line elements which use rigid links where they connect to other line elements (e.g. beam and columns). However, some of the walls connect to alternate sides of the blade columns. Because of the dimensions of the pairs of columns rigid links were used over the height of the column where other building components connected.

4. Beams were modelled as flexural elements with yield a function of major axis moment and axial load. Effective moments of inertia for the beams were defined as $0.3I_g$. A strain hardening of ratio of 0.01 was used, as for columns, and strengths were based on the probable strengths of material with $\varphi = 1.0$.

5. Floors were modelled using plane stress elements, using a similar hysteresis model as for walls.

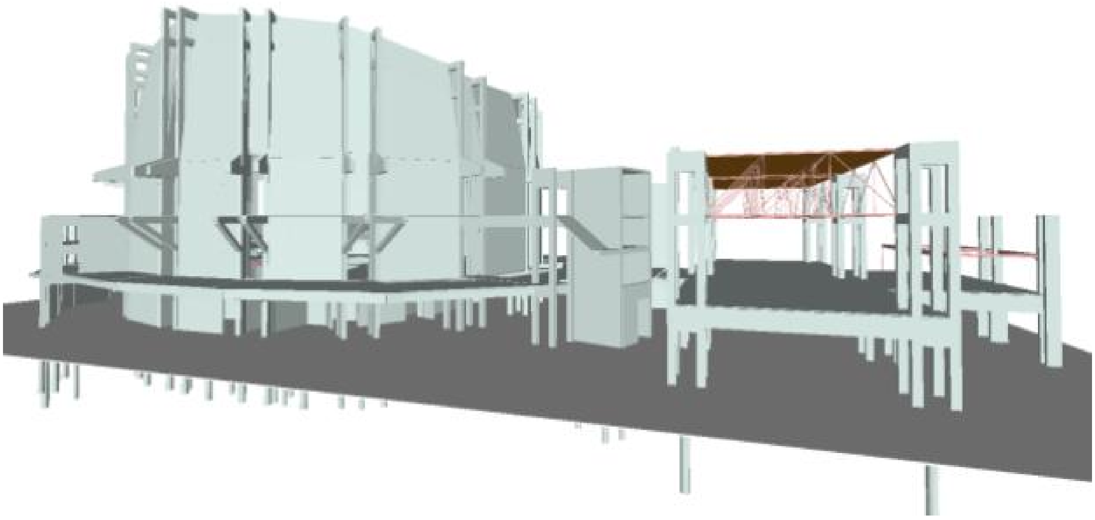
6. Self weight and floor load corresponding to dead plus seismic live load was applied prior to the dynamic analysis.

7. Vertical fixity was assumed at the base of the piles and horizontal fixity at the Level 1 floor.

8. Gap elements, some with a predefined initial gap opening, were used to model seismic joints at the Auditorium Level 4 seating and at the interface between the Auditorium / Stair Blocks and the Auditorium / Entrance Foyer.

Figure 4-3 shows a rendered view of the complete complex. Development of this model is described in subsequent sections.

Figure 4-3 View of Model of Complete Complex



4.3.1 Geometry

The geometry was described by a series of column numbers to identify plan locations and elevations to identify sections in the vertical plane. Column locations were defined at each grid line intersection in each building and at intermediate locations as required to define wall openings, which required a total of approximately 1100 column lines. Figure 4-4 shows the plan layout of all column lines. More detail for individual buildings in the complex is provided in the sections below.

The coordinate system was defined with the X axis oriented in the N-S direction, the Y axis vertical and the Z axis in the E-W direction, as indicated in Figure 4-4. The origin of the axis (0, 0) was defined as the focus of the Auditorium ellipse, at the intersection of Grids 3E/3W and C.

A total of 48 levels, as listed in Table 4-1, were used to define all elevations. Levels were defined at each floor diaphragm of each building plus at intermediate elevations to define wall openings and other vertical features which required a node definition. The main structure is defined by the lower 21 levels and above that the additional levels are required to model the changing elevation of the beams within the precast columns (P levels) and the top of the precast columns (C levels).

The elevations used the base line of the structural drawings, defined so Level 9 of 1.900 m corresponded to the top of the pile cap. Level 8 at 1.000 defined the bottom of the pile caps and the levels below this defined the founding levels of the varying length piles.

Figure 4-4 Column Line Locations

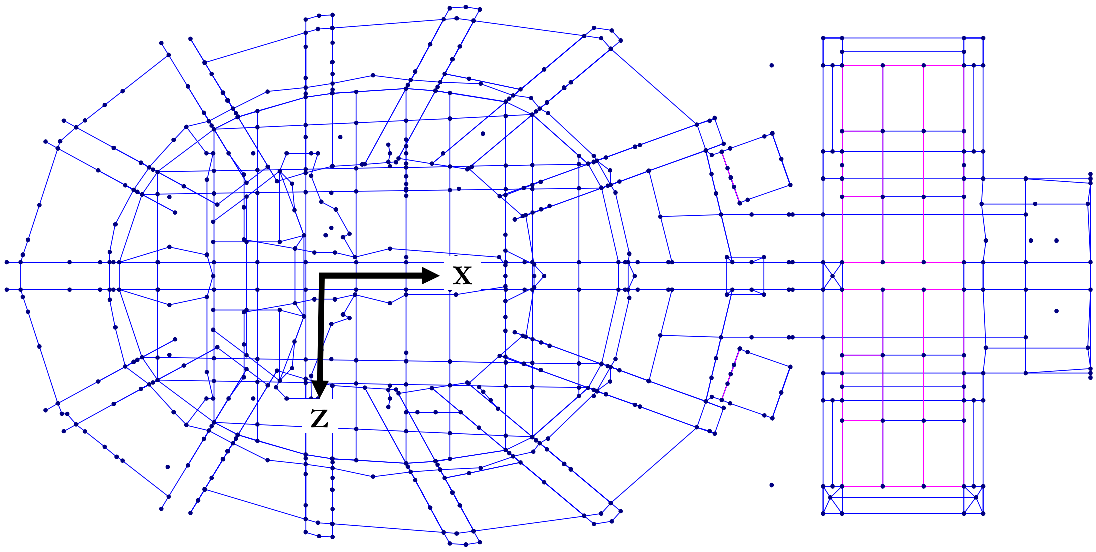


Table 4-2 Elevations

Level Number	Name	Elevation
1	Tip	-6.000
2		-5.000
3		-4.000
4		-3.000
5		-2.000
6		-1.000
7	Grade	0.000
8		1.000
9	1st	1.900
10		5.320
11		6.000
12	2nd	6.620
13		7.740
14	3rd	9.340
15		9.720
16		10.620
17	4th	12.060
18		15.760
19		16.370
20		16.930
21		17.470
22	P801-804-704-901	22.335

Level Number	Name	Elevation
23	P701-904	23.322
24	P104-101	23.544
25	P201-1404	24.208
26	P204-1401	24.583
27	C801-804-704-901	24.935
28	P604-1001	25.235
29	C701-904	25.922
30	P601-1004	26.021
31	C104-101	26.144
32	P301-1304	26.237
33	P304-1301	26.797
34	C201-1404	26.808
35	P504-1101	26.999
36	C204-1401	27.183
37	P501-1104	27.345
38	C604-1001	27.835
39	P404-1201	28.391
40	C601-1004	28.621
41	P401-1204	28.691
42	C301-1304	28.837
43	C304-1301	29.397
44	C504-1101	29.599
45	C501-1104	29.945
46	T5	30.655
47	C404-1201	30.991
48	C401-1204	31.291

4.3.2 Material Properties

Table 4-3 reproduces the properties of the seven material types defined for the evaluation.

Table 4-3 Material Definitions

Material ID	Material Type	Strength f'_m Masonry f'_c Concrete F_y Steel	E	G	ρ	Weight Density γ_w	Mass Density γ_m
1	CONCRETE	30000	25084389	10451829	0.200	25.00	2.55
2	STEEL (REINFORCING)	297000	200000000	76923077	0.300	78.00	7.95
3	STEEL (STRUCTURAL)	200000	200000000	76923077	0.300	78.00	7.95
4	CONCRETE	12000	18400817	7667007	0.200	25.00	2.55
5	CONCRETE	30000	25084389	10451829	0.200	0.00	0.00
6	USER DEFINED	0	200000000	76923077	0.300	0.00	0.00
7	STEEL (REINFORCING)	447000	200000000	76923077	0.300	78.00	7.95

1. Type 1 was the typical concrete material used throughout the complex. The probable strength was defined as 30 MPa, 1.5 times as assumed minimum specified strength of 20 MPa.

2. Type 2 was mild steel reinforcing, defined with a probable yield strength of 297 MPa.

3. Type 3 was structural steel, as used for roof trusses etc. The steel strength was defined as 200 MPa.

4. Type 4 was a low strength concrete material used to model the concrete block walls. The probable strength was defined as 12 MPa.

5. Type 5 has the same properties as the default concrete (Type 1) but zero weight and mass density. This is used where more than one element occupies the same space.

6. Type 6 has the same properties as the default steel (Type 3) but zero weight and mass density. As for Type 5, this is used where more than one element occupies the same space.

7. Type 7 is high strength steel reinforcing, defined with a probable yield strength of 447 MPa and used where the drawings indicated H bars.

4.3.3 Panel Section Properties

Panel section properties are reproduced in Table 4-4. These definitions are used to define all plane stress elements (walls, floor and roof). A description of these properties is as follows:

1. A description of the location at which each element type is used is provided.

2. The section identification number generally defines the type of component (10 to 29 are concrete walls, 30 to 39 masonry walls, 40 to 99 floor slabs).

3. The material identification numbers correspond to those in Table 4-3. Most plates are Type 1, concrete, as they are used for walls and floors.

4. The structural thickness corresponds to the thickness on the drawings, or the thickness of the topping for precast floors.

5. The thickness used for weight is equal to the structural thickness for vertical panels (walls) but increased to allow for superimposed loads for floor panels. These increased are described below.

6. The reinforcing material and volume per unit area is listed. The reinforcing content was calculated from the layouts on the drawings and specified separately for the horizontal and vertical directions. Material Type 2 was used for panels unless the drawings specified H bars, in which case Material Type 7 was used.

For the panel sections used to model floors (53 through 69) the thickness used to calculate the seismic weight was increased so as to apply superimposed dead load plus seismic live load to all floors. A seismic live load of 2.20 kPa was applied to the floors by the following procedure:

1. For flat slabs the total thickness was taken as (1.2 x structural thickness) + 2.2 / 25, where 25 is the weight density of the concrete material Table 4-3).

2. For double tee slabs the total thickness was taken as (3.5 x structural (topping) thickness) + 2.2 / 25. This applies to Sections 54, 65 and 67.

Table 4-5 lists the effective distributed weight of the floor panel sections, calculated on the basis above.

Section types 71 and 72 are used to model the Auditorium and Foyer roof respectively. For these panels, the thickness for weight was defined to provide a total seismic weight of 5 kPa in the Auditorium and 2 kPa in the Foyer.

Table 4-4 Panel Section Definitions

	Plate ID	Material ID	Structural Thickness	Thickness for Weight/Mass	Reinforced Sections Material ID	Vertical ρ_V	Horizontal ρ_H	Shear Strength, v_c	Reinforcing Shear Strength v_s
W16W18	10	1	0.250	0.250	7	0.0080	0.0080	910	3576
W18,W1	11	1	0.300	0.300	7	0.0080	0.0080	910	3576
W8	12	1	0.200	0.200	2	0.0032	0.0032	910	960
Stair Walls	13	1	0.400	0.400	2	0.0040	0.0025	910	746
W7	14	1	0.200	0.200	2	0.0045	0.0045	910	1344
W21 L8-10	16	1	0.600	0.600	7	0.0126	0.0126	910	5617
W21 L10-12	17	1	0.150	0.150	7	0.0025	0.0025	910	1118
Bays to L12	18	1	0.250	0.250	2	0.0026	0.0080	910	2389
Bays to L14	19	1	0.250	0.250	2	0.0026	0.0054	910	1592
Bays to L20	20	1	0.250	0.250	2	0.0026	0.0040	910	1194
Bays above L20	21	1	0.250	0.250	2	2.0000	0.0026	910	768
W17 & W60	22	1	0.200	0.200	2	0.0057	0.0057	910	1679
W1	23	1	0.200	0.200	7	0.0073	0.0073	910	3268
W6	24	1	0.200	0.200	7	0.0080	0.0080	910	3595
W11	25	1	0.200	0.200	7	0.0101	0.0101	910	4494
W14	26	1	0.200	0.200	7	0.0134	0.0134	910	5992
Block Walls	31	4	0.190	0.190	2	0.0010	0.0010	575	297
Level 1/2 Lower	41	1	0.200	0.200	2	0.0057	0.0057	910	1696
L1/2 Beams	51	1	0.150	0.150	2	0.0021	0.0060	910	1791
L1/2 Beams	52	1	0.300	0.300	2	0.0121	0.0121	910	3583
L1/2 Upper Slab	53	1	0.050	0.130	2	0.0026	0.0026	910	778
Foyer L2 Slab TT	54	1	0.125	0.526	2	0.0021	0.0021	910	622
L2 125	61	1	0.125	0.238	2	0.0040	0.0040	910	1194
L2 200	62	1	0.200	0.328	2	0.0025	0.0025	910	746
L2 225	63	1	0.225	0.358	2	0.0040	0.0040	910	1194
L2 400	64	1	0.400	0.568	2	0.0040	0.0040	910	1194
L2 tees	65	1	0.050	0.263	2	0.0026	0.0026	910	778
L3 225	66	1	0.225	0.358	2	0.0051	0.0051	910	1517
L4 tees	67	1	0.050	0.263	2	0.0026	0.0026	910	778
L4Seating	68	1	0.050	0.148	2	0.0105	0.0105	910	3110
L5	69	1	0.150	0.268	2	0.0107	0.0107	910	3185
L5Beams	70	1	0.400	0.400	2	0.0100	0.0100	910	2970
Roof	71	1	0.050	0.200	2	0.0050	0.0050	910	1485
Foyer Roof	72	1	0.050	0.080	2	0.0050	0.0050	910	1485
Stair slab	73	1	0.400	0.400	2	0.0050	0.0050	910	1485
Ground	81	1	0.100	0.100				910	0

Table 4-5 Seismic Weights of Floor Panels

Plate ID	Location	Weight (Kpa)
53	L1/2 Upper Slab	3.25
54	Foyer L2 Slab TT	13.14
61	L2 125	5.95
62	L2 200	8.20
63	L2 225	8.95
64	L2 400	14.20
65	L2 tees	6.58
66	L3 225	8.95
67	L4 tees	6.58
68	L4Seating	3.70
69	L5	6.70

4.3.4 Flexural Section Properties

Tables 4-6 to 4-7 lists the flexural section definitions used in the model for column, miscellaneous and beam sections respectively. The properties used to define each section are:

1. A description of the location at which each element is used.

2. The section identification number generally defines the type of component. Section numbers 101-199 are piles, 201-299 concrete columns, 301-399 concrete means and 401-599 secondary and miscellaneous sections.

3. The material identification numbers correspond to those in Table 4-3. As for the plate components, most sections are Type 1, concrete.

4. The section shape. Most are rectangular, although there are circular and I-shapes in some locations. Where steel sections are used these are identified by their designation.

5. These section dimensions, in metres, as used in the model.

The concrete sections have properties and evaluation criteria as defined by ASCE-41. The columns and beams are defined respectively as:

130 ASCE CONCRETE Columns - Calculate Condition Type
123 ASCE CONCRETE Beams Conforming Transverse Reinforcement

All concrete sections used analysis properties based on the gross concrete area and 0.3 times the gross moment of inertia.

Table 4-6 Column Section Definitions

	Section ID	Material ID	Type (Double Click for Selection)	X Dim (Beam Depth Below)	Z Dim (Beam Width)	Beam Depth Above	Flange Thickness Tfl	Web Thickness Tw
Piles	101	1	CIRC	0.250				
	102	1	CIRC	0.450				
	103	1	CIRC	0.450				
	104	1	CIRC	0.560				
	105	1	CIRC	0.560				
	106	1	CIRC	0.900				
Pile cap	107	1	RECT	0.900	0.900			
Main I Columns	201	1	I-SECT	2.200	0.400		0.500	0.250
Columns no Web	202	1	RECT	0.500	0.400			
Main Column Rigid Zone	203	1	RECT	2.200	0.400			
Colum Type C1	204	1	RECT	0.400	0.300			
	205	1	RECT	0.400	0.400			
Stair Walls	220	5	RECT	2.500	0.400			
Foyer Columns	230	1	RECT	0.800	0.300			
	231	1	RECT	1.150	0.400			
	232	1	RECT	0.600	0.400			
	233	1	RECT	0.500	0.400			

Table 4-7 Miscellaneous Flexural Sections

	Section ID	Material ID	Type (Double Click for Selection)	X Dim (Beam Depth Below)	Z Dim (Beam Width)	Beam Depth Above	Flange Thickness Tfl	Web Thickness Tw
Impact element	501	6	RECT	0.050	0.050			
Auditorium Roof Truss	502	3	I-SECT	3.500	0.150		0.038	0.003
Truss wall beam (ledger)	503	1	RECT	3.500	0.300			
Roof stiffener	504	3	150PFC	0.150	0.075		0.010	0.006
Wall Beam	505	5	RECT	1.000	0.250			
Stair Slab Beam	506	5	RECT	1.000	0.250			
Foyer Slab Beam	507	5	RECT	0.350	1.000			
Foyer Roof Truss	508	3	100X100X4.0SHS	0.100	0.100		0.004	0.004
	509	3	50X50X4.0SHS	0.050	0.050		0.004	0.004
Foyer Kitchen	510	3	89X89X5.0SHS	0.089	0.089		0.005	0.005
	511	3	310UB40.4	0.304	0.165		0.010	0.006
	512	3	200PFC	0.200	0.075		0.012	0.006
Dummy	520	6	RECT	0.001	0.001			
Ledger	521	6	RECT	0.001	3.000			

Table 4-8 Beam Section Definitions

	Section ID	Material ID	Type	Beam Depth Below	Beam Width) Width)	Beam Depth Above	Flange Thickness Tfl	Web Thickness Tw
Beams Between Columns	301	1	RECT	0.600	0.400			
	302	1	RECT	0.800	0.400			
	303	1	RECT	0.900	0.400			
	304	1	RECT	1.480	0.400			
	305	1	RECT	0.900	0.400			
Foundation beams	306	1	RECT	1.200	0.200			
	307	1	RECT	2.000	1.500			
	308	1	RECT	2.000	0.300			
	309	1	RECT	0.900	0.900			
	310	1	RECT	1.500	1.000			
	311	1	RECT	1.500	0.600			
	312	1	RECT	1.070	3.270			
	313	1	RECT	2.000	0.900			
L2 C edge beam	314	5	RECT	0.650	2.000			
Foyer beams	315	1	RECT	1.500	0.400			
	316	1	RECT	0.700	0.400			
	317	1	RECT	1.200	0.400			
	318	1	RECT	0.900	0.400			
	319	1	RECT	0.350	0.650			
	320	1	RECT	0.550	0.450			
Foyer Truss	321	1	RECT	0.600	0.500			
L2 TT beam	322	1	RECT	0.550	0.500			
L4 TT beam	323	1	RECT	0.500	0.500			
Not used	325	5	RECT	0.900	0.400			
Stair Floor (1/2)	326	5	RECT	0.200	2.400			
Links	401	6	RECT	1.000	1.000			
Stage Beams	410	1	RECT	0.450	0.300			
	411	3	410UB53.7	0.403	0.178		0.011	0.008
	412	3	310UB40.4	0.304	0.165		0.010	0.006
	413	3	250UB31.4	0.252	0.146		0.009	0.006
Precast Beams PB1-PB6 Top	420	1	RECT	0.800	0.400	-0.200		
Diagonal	421	1	RECT	0.400	0.400			
L2 Slab Beam	422	5	RECT	0.200	0.800			
L2 Ring beam	423	1	RECT	0.800	0.300			
Slab beams	424	1	RECT	0.800	0.400			
(pinned)	425	4	RECT	0.800	0.180			
Precast Beams Level 4	426	1	RECT	0.600	0.400			
Diagonal	427	1	RECT	0.400	0.400			

4.3.5 Column Strengths

As described above, Sections 201 to 233 were reinforced concrete columns. The yield function for these components is a function of the biaxial moments and concurrent axial load. An interaction diagram about each axis was generated for each variation of reinforcing layout of each column section. Examples of the interaction diagram as shown in Figure 4-5.

The strengths calculated for each section are listed in Table 4-9. Some sections have multiple strengths. For example, Section 201 which is the main I-section column, has seven reinforcing variations and so seven strength types used at different locations.

The column elements were defined with both stiffness and strength degradation, in accordance with the provisions of ASCE-41. Implementation of degradation allows secondary component criteria to be used to evaluation these columns.

Figure 4-5 Column Interaction Diagram

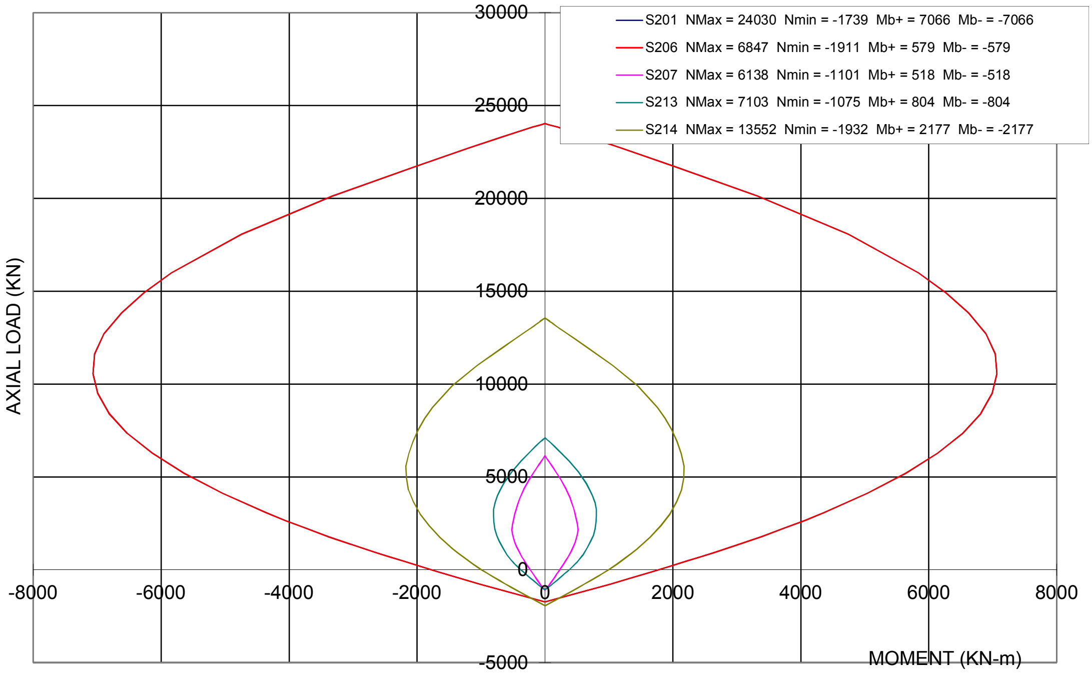


Table 4-9 Column Strengths

Section ID	Strength ID	Compression Strength	Tension Strength	X Axis Axis for Columns +ve Moment Strength	Z Axis Axis for Columns -ve Moment Strength	Stiffness and Strength Degradation
201	201	24030	1739	7066	1220	Both
202	202	6847	1911	579	448	Both
202	203	6138	1101	518	403	Both
204	204	3572	560	204	166	Both
205	205	5417	1463	354	354	Both
206	206	5063	1075	315	315	Both
201	211	24514	2269	7185	1253	Both
201	212	24030	1739	7066	1220	Both
201	213	24269	2000	6989	1239	Both
201	214	23784	1470	6860	1208	Both
201	220	27547	2239	9089	1575	Both
201	221	26810	1433	8637	1462	Both
230	230	7103	1075	804	313	Both
231	231	13552	1932	2177	821	Both
232	232	7594	1612	715	453	Both
233	233	6901	1910	577	477	Both
233	234	6392	1371	505	418	Both

4.3.6 Pile Strengths

The pile yield strength was defined as a function of the bi-axial moment and axial load using limits derived from interaction diagrams, in a similar fashion to the columns. Each of the five pile sections was defined with seven strength types, corresponding to reinforcing cage types A to G. Not all reinforcing cage types were used for all pile sections but the definition of all types for all sections simplified look-up functions in the model development.

Although the pile strengths were calculated from interaction diagrams as for the columns, the ultimate compression and tension strengths were replaced by the ultimate load capacity listed on the drawings for each pile type. Some combinations of pile type and cage type did not have a tension capacity and so a nominal strength of 1 kN was assigned. Table 4-10 lists the strength values for each pile and reinforcing variation.

The piles were modelled as bi-linear elements, with no strength or stiffness degradation. This was because the piles function primarily as axial elements with axial strengths based on sub-soil properties rather than structural properties.

4.3.6.1 Reduction in Pile Tension Strength

As noted above, the pile axial strength was based on the ultimate compression and tension strength listed on the drawings for each pile type for the 450 mm and 560 mm diameter single and double bulb types listed in Table 4-10.

Based on the assessment from the geotechnical consultants (Tonkin and Taylor) the piles will likely have little to no tension capacity due to potential liquefaction occurring at the site. Some risk remains, due to the potential liquefaction, that the piles will achieve an ultimate compression capacity less than that noted on the drawings. This has not been included in the time history analysis, through further reduction of the pile compression capacities, but pile ultimate compression demands have been reviewed against reduced capacities separately.

The final pile type listed in Table 4-10 is a drilled type pile, with a 900 mm diameter shaft and 2000 mm diameter bell. On best advice from the geotechnical consultant they probably only have the bell portion of the pile embedded 1 m into alluvium, below the liquefiable material. It is difficult to determine the geotechnical tensile capacity of these piles as there are a couple of potentially different failure mechanisms due to the limited strength of the liquefiable material above the alluvium layer. It was therefore considered conservative to assign zero tension capacity to these piles also.

Preliminary analyses had used the tension strengths listed in Table 4-10. This was modified by inserting a gap element at the base of every pile. The effect on the response of the structures was relatively slight. This is because the lateral load resisting systems are widely distributed and have a relatively small height to width ratio, resulting in low net tension forces on the piles.

Table 4-10 Pile Strengths

Pile Type	Reinforcing Cage	ID	Compression Strength	Tension Strength	X Axis Axis for Columns +ve Moment Strength	Z Axis Axis for Columns -ve Moment Strength	Stiffness and Strength Degradation
450 single	A	110	2011	1	217	217	None
	B	111	2011	1	217	217	None
	C	112	2011	1	230	230	None
	D	113	2011	1	230	230	None
	E	114	2011	1	204	204	None
	F	115	2011	1	238	238	None
	G	116	2011	1	293	293	None
450 double	A	120	2011	1177	217	217	None
	B	121	2011	1177	217	217	None
	C	122	2011	1668	230	230	None
	D	123	2011	1668	230	230	None
	E	124	2011	1668	204	204	None
	F	125	2011	1668	238	238	None
	G	126	2011	1668	293	293	None
560 single	A	130	4415	1	408	408	None
	B	131	4415	1	408	408	None
	C	132	4415	1	429	429	None
	D	133	4415	1	429	429	None
	E	134	4415	1	388	388	None
	F	135	4415	1	441	441	None
	G	136	4415	1	522	522	None
560 double	A	140	4415	1177	408	408	None
	B	141	4415	1177	408	408	None
	C	142	4415	1668	429	429	None
	D	143	4415	1668	429	429	None
	E	144	4415	1668	388	388	None
	F	145	4415	1668	441	441	None
	G	146	4415	1668	522	522	None
900 with 2000 bell	A	150	4954	3826	1610	1610	None
	B	151	4954	3826	1610	1610	None
	C	152	4954	3826	1654	1654	None
	D	153	4954	3826	1654	1654	None
	E	154	4954	3826	1567	1567	None
	F	155	4954	3826	1679	1679	None
	G	156	4954	3826	1837	1837	None

4.3.7 Beam Strengths

Beams are typically modelled with a yield function dependent on applied moment only, not axial load. This is because beams within rigid diaphragms have zero axial load. For this structure, especially the Auditorium, all floors are flexible and a number of beams have a non-horizontal orientation. This results in non-trivial axial loads in a number of beams.

To account for this, the beam's yield was defined as a function of major axis bending and axial load. Interaction diagrams were derived for each beam reinforcing layout variation. The interaction diagrams differed from the column diagrams in that they were not necessarily symmetrical due to differing top and bottom reinforcing. Figure 4-6 shows example diagrams.

Figure 4-6 Beam Interaction Diagram

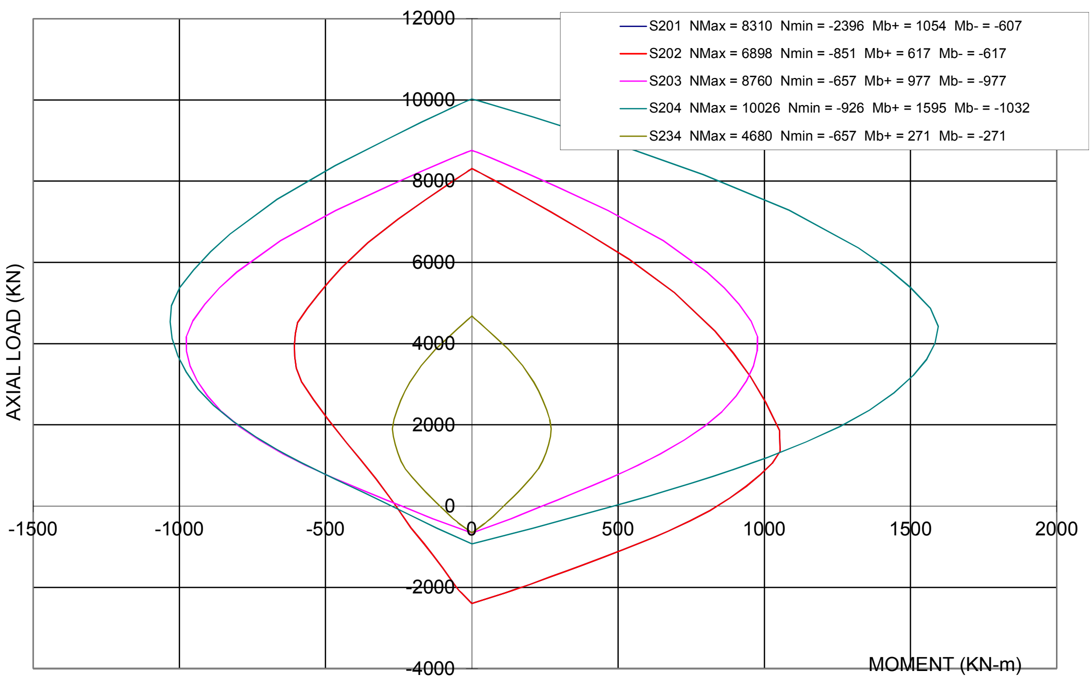


Table 4-11 lists all beam strengths. Although strength degradation is usually modelled for beams, this was not implemented for beams where axial load dependency was incorporated. This is because the gravity loads were significant for some beams. A reduction in strength due to applied axial loads together with strength loss due to plastic rotation would result in moment capacity lower than required to support gravity loads. In this situation, equilibrium cannot be satisfied and the analysis terminates.

Table 4-11 Beam Strengths

Section ID	Strength ID	Compression Strength	Tension Strength	X Axis Axis for Columns +ve Moment Strength	Z Axis Axis for Columns -ve Moment Strength	Stiffness and Strength Degradation
301	1301	8310	2396	196	920	Stiffness Only
	1330	6898	851	196	196	Stiffness Only
302	1302	8760	657	200	200	Stiffness Only
303	1303	10026	926	227	450	Stiffness Only
304	1304	15888	866	532	532	Stiffness Only
305	1305	10272	1194	450	450	Stiffness Only
	1331	9972	866	314	314	Stiffness Only
315	1315	16405	1209	657	915	Stiffness Only
316	1316	7986	926	257	257	Stiffness Only
317	1317	13086	926	461	461	Stiffness Only
	1332	13175	1023	308	723	Stiffness Only
319	1319	6337	586	54	79	Stiffness Only
1320	1320	7461	1258	225	318	Stiffness Only
1321	1321	8837	1299	312	312	Stiffness Only
1323	1323	7221	926	174	174	Stiffness Only
326	326			42	42	
420	1420	10125	2150	439	1021	Stiffness Only
421	1421	4680	657	90	90	Stiffness Only
423	1423	7594	1612	396	693	Stiffness Only
426	1426	7116	1090	258	258	Stiffness Only
	1431	7785	1821	442	442	Stiffness Only
	1433	7232	1217	196	383	Stiffness Only
	1435	7273	1261	258	346	Stiffness Only
	1437	7048	1015	129	346	Stiffness Only
	1439	7225	1209	129	442	Stiffness Only
	1441	7184	1164	208	346	Stiffness Only
	1443	7608	1627	346	442	Stiffness Only
	1445	7273	1261	258	346	Stiffness Only
	1447	7608	1627	346	442	Stiffness Only
427	1427	4899	896	109	109	Stiffness Only
	1432	4899	896	109	109	Stiffness Only
	1434	5192	1217	120	226	Stiffness Only
	1436	4680	657	90	90	Stiffness Only
	1438	4680	657	90	90	Stiffness Only
	1440	4680	657	90	90	Stiffness Only
	1442	4680	657	90	90	Stiffness Only
	1444	4680	657	90	90	Stiffness Only
	1446	4680	657	90	90	Stiffness Only
	1448	4680	657	90	90	Stiffness Only

4.3.8 Foundation Model

The total gravity support for the model is on the concrete piles. The piles are not all located along grid lines, but rather are incorporated into the foundation by the use of pile caps and wide foundation beams. As the model foundation beams are line elements with zero width they cannot capture pile tops within a finite width. To incorporate the piles, the beams follow a zigzag pattern under parts of the Auditorium, as shown in the plan in Figure 4-7 and rendered view in Figure 4-8.

The subterranean structural elements are in a complex configuration of pile caps, walls and beams and so a full evaluation of these components was not included within this scope. The approximate dimensions of the elements were included but they were assumed to remain elastic. Pile strengths were modelled and it was assumed that this would be sufficient to identify any major foundation issues.

Figure 4-7 Plan of Foundation Model

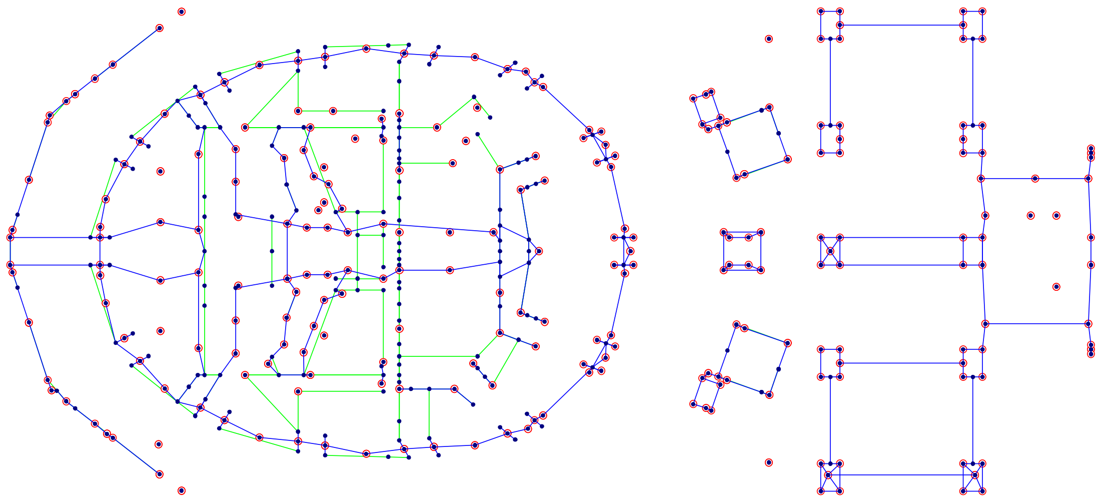
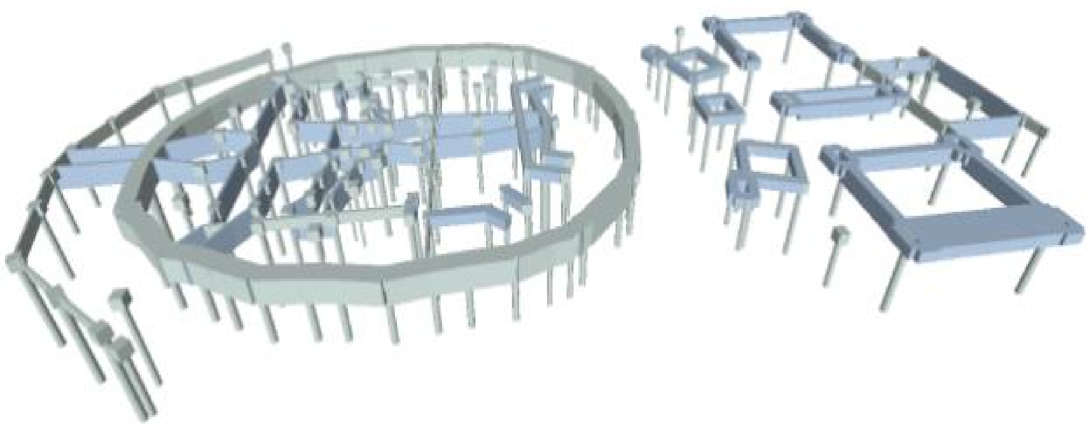


Figure 4-8 Rendered View of Foundation Model



4.3.9 Mass & Weight

The mass and weight were assembled from the element self-weights, including increased floor element self weight to account for floor and roof loads. The analysis model has weight and mass specified separately, with the seismic weight defined as the mass time the gravitational constant. These are tabulated by building in Table 4-12.

Table 4-12 Total Complex Weight

Structure	Building Weight		Seismic Weight	
	kN	Fraction	kN	Fraction
Auditorium			79,392	79%
Entrance Foyer			16,961	17%
Stair Blocks			3,520	4%
Total	132,100	100%	99,873	100%

The building weight, the total of applied gravity loads, is greater than the seismic weight. This is because all buildings have gravity support at the footing level but translational restraint provided by slabs on grade is at a higher level. The difference between the two reflects the difference in elevation between gravity and lateral load supports.

The Auditorium is the largest and tallest building in the complex and accounts for about one-half the total weight. The Christchurch Town Hall Auditorium model seismic weight was 70,063 KN, about 10% less than for this structure.

4.3.10 Stairs

Main Egress Stairs have not been explicitly modelled. Stair assessment has been undertaken separately to the time history analysis, and discussed in Section 7.1.

4.4 AUDITORIUM MODEL

The auditorium geometry is relatively complex with multiple levels and types of floor units. Figures 4-9 and 4-10 show sections through the model along each ellipse axis. These illustrate the combinations of elements used to model the lateral load system.

1. The model is fixed against translation from RL 1.900 and below. This is at the bottom of the two wall panels shown in Figure 4-9. Vertical fixity is at the base of the piles only.

2. There are no rigid diaphragms in the model, all floors are modelled explicitly.

3. All floors are included but the roof is included over the Auditorium and the Foyer only. It is assumed that any other roofing is lightweight and non-structural.

4. The pairs of blade columns are modelled using flexural elements with link elements acting as outriggers to define the correct spatial dimensions. Where the column web portion was removed (e.g. 6E/6W between Level 1 and Level 2), pairs of columns were used to represent each blade column, as shown in the section along Grid C in Figure 4-10.

5. Precast beams supporting stepped seating are generally modelled with two components, a horizontal and an inclined beam as shown in Figures 4-9 and 4-10.

6. The Auditorium roof truss is modelled as a single horizontal pinned beam member with I-section properties to approximate the chords spanning east to west (Figure 4-9). This is on the assumption that the trusses are not major elements for seismic resistance. The roof is defined with plane stress elements, the properties of which are based on a 50 mm thick concrete topping (Section ID 71 in Table 4-4).

7. There are sets of double nodes at the Level 4 floor to allow for the 30 mm structural joints between sections. These joints are totally unconnected until the 30 mm gap closes.

Figure 4-9 Section Through Auditorium Grid 3E/3W

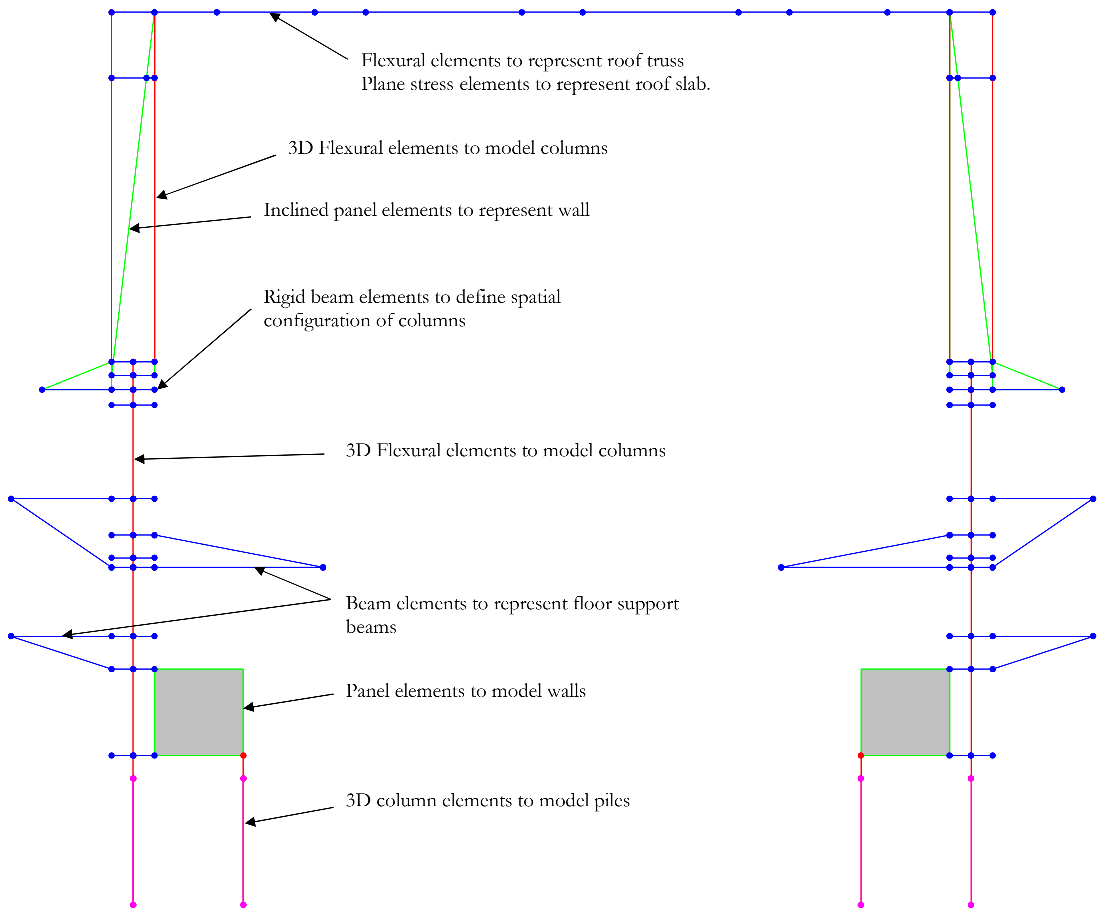
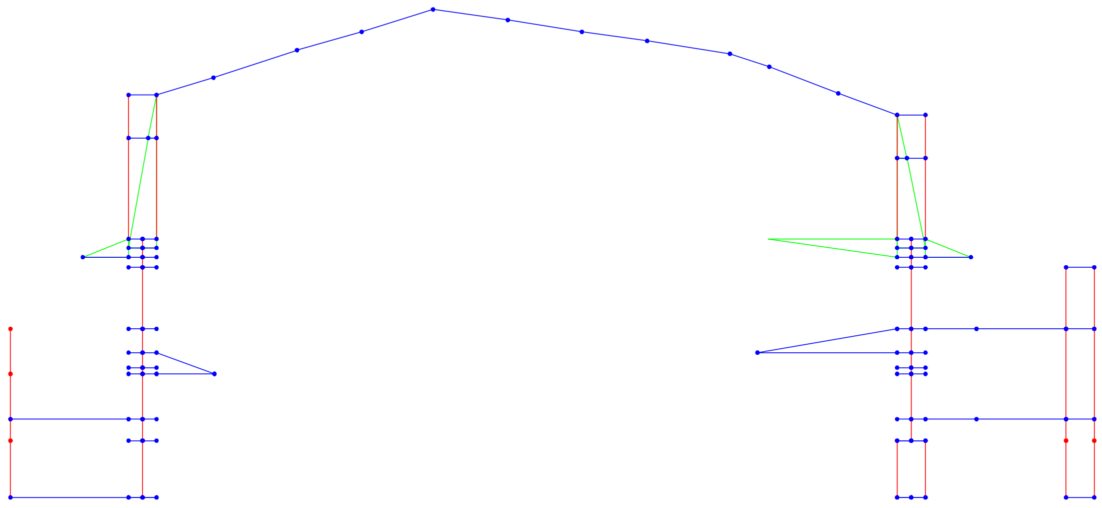
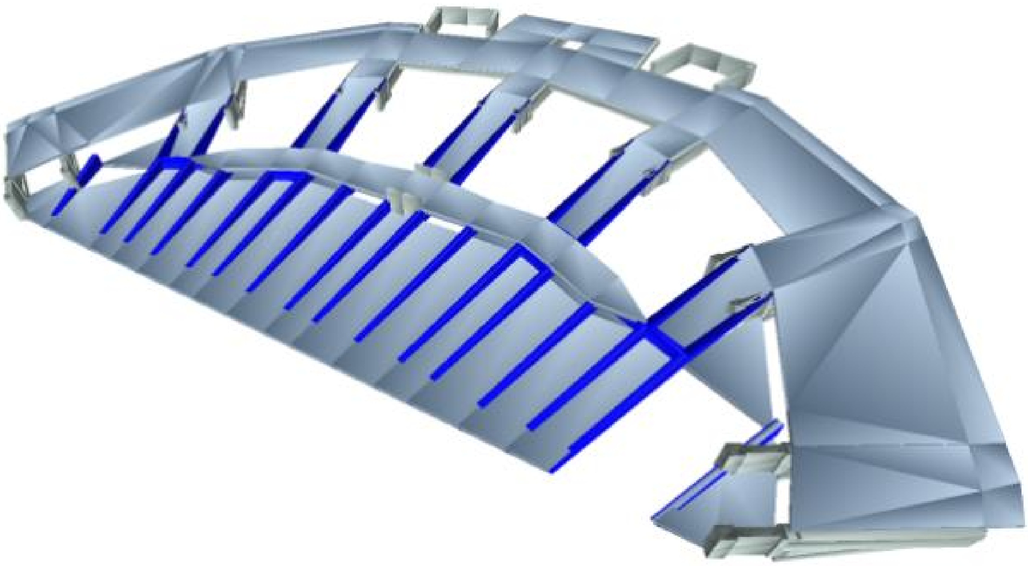


Figure 4-10 Section Through Auditorium Grid C



The seating between Levels 1 and 2 has a lower slab with tapered precast beams supporting the seating above. This has been modelled using two grids of plane stress elements, a lower slab and an upper slab which represents the precast concrete seating units above. Because of the complex geometry, the beams are actually modelled as plane stress elements rather than beam elements. These are shaded blue in the model portion shown in Figure 4-11.

Figure 4-11 Level 1/2 Floor (Upper slab removed for clarity)



The rendered views in Figures 4-12 to 4-14 illustrate the manner in which the components are assembled into the full model:

1. Figure 4-12 shows a rendered view of the complete Auditorium model.

2. Figure 4-13 shows the grid configuration used to model the roof of the structure.

3. In Figure 4-14 the roof elements are removed to show the configuration of the internal floors of the model.

4. Figure 4-15 shows the concrete columns and walls which form the primary seismic load resisting elements of the building.

Figure 4-12 Rendered View of Auditorium Model

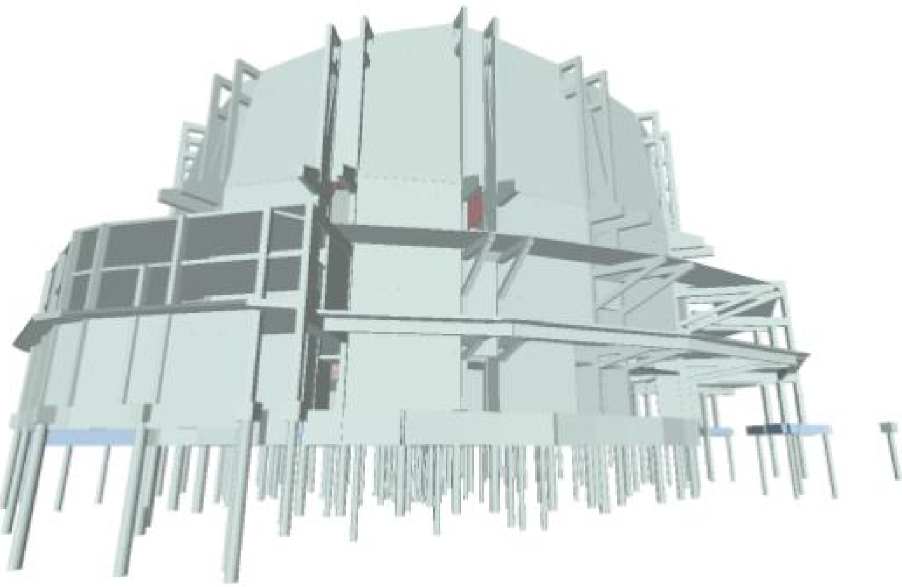


Figure 4-13 Auditorium Roof Elements

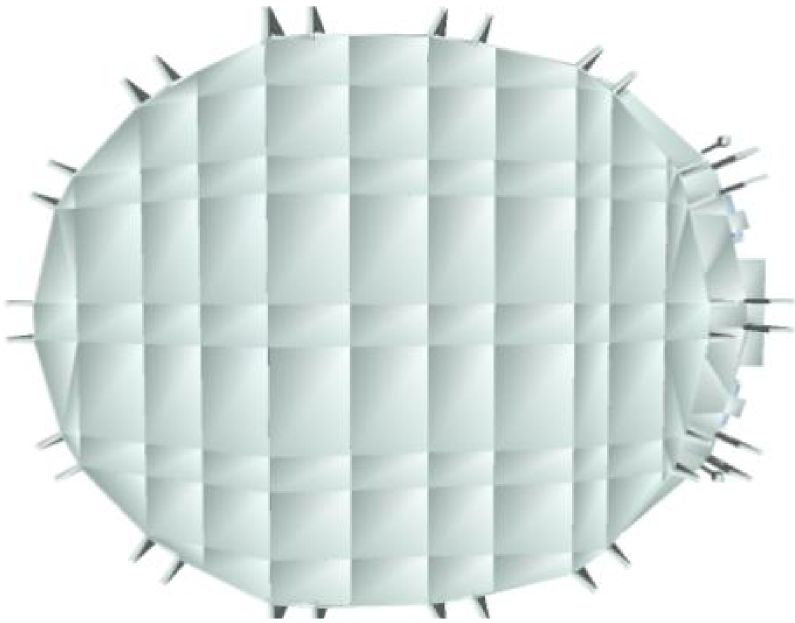


Figure 4-14 Auditorium Floor Modelling

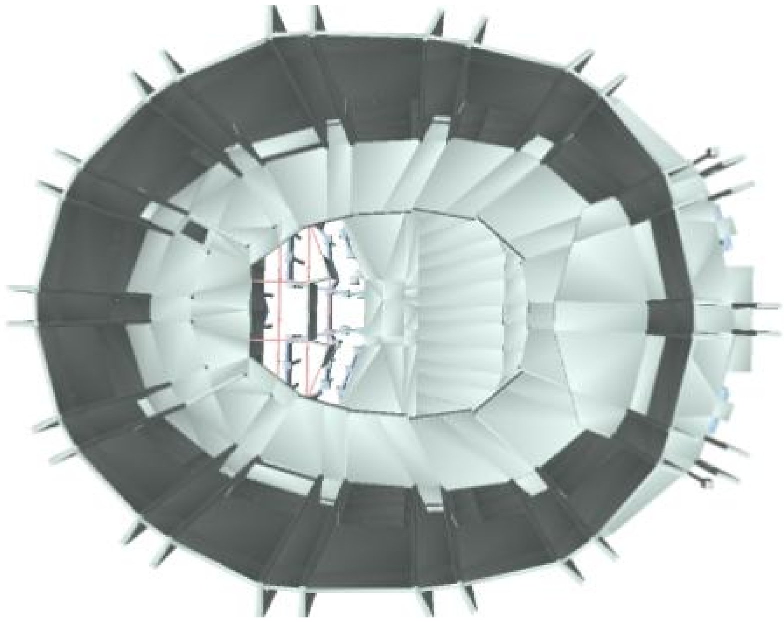
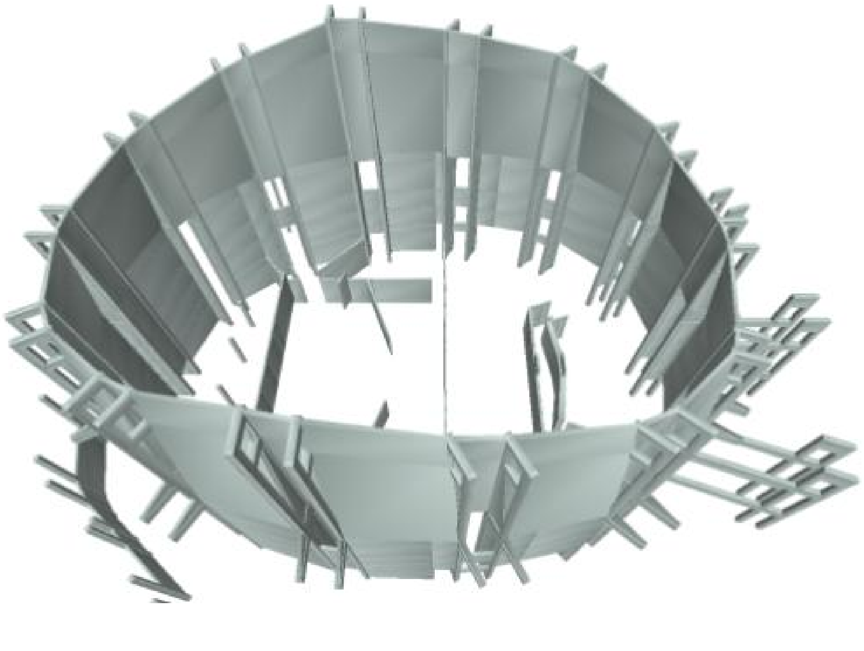


Figure 4-15 Auditorium Lateral Load Resisting Elements



4.5 ENTRANCE FOYER MODEL

Figure 4-16 shows a plan view of the Entrance Foyer model. The Foyer Building has one suspended concrete level with a precast double tee flooring system supported on reinforced concrete beams. The model has similar support conditions as the Auditorium with fixity against translation from RL 1.900 and below and vertical fixity at the base of the piles only. As for the Auditorium, the model does not include rigid diaphragms as all floor and roof elements are explicitly modelled.

Figure 4-16 Plan View of Foyer Model

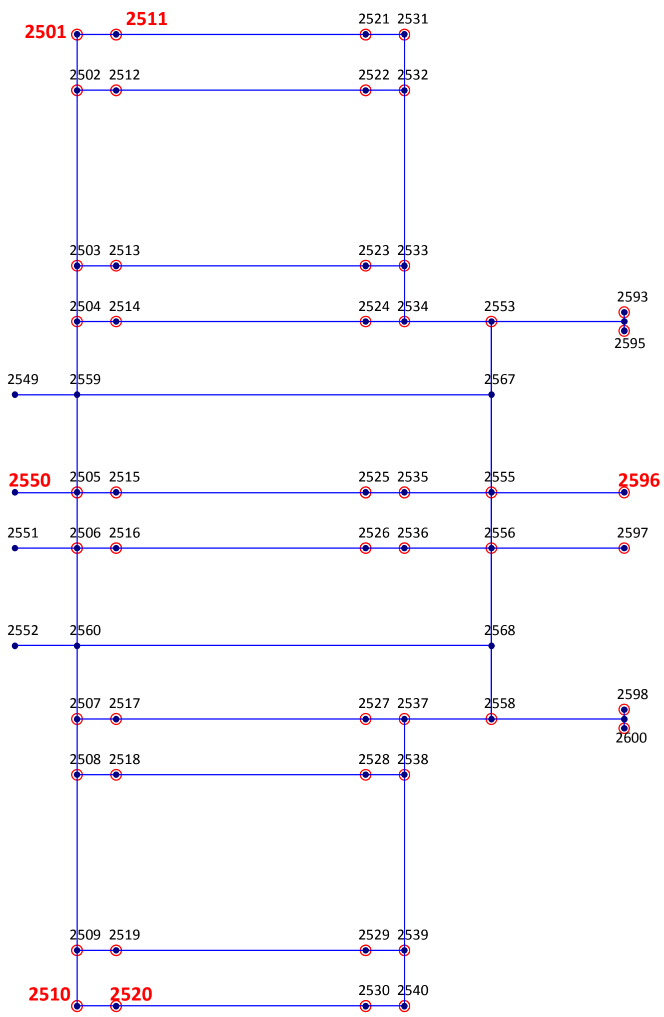
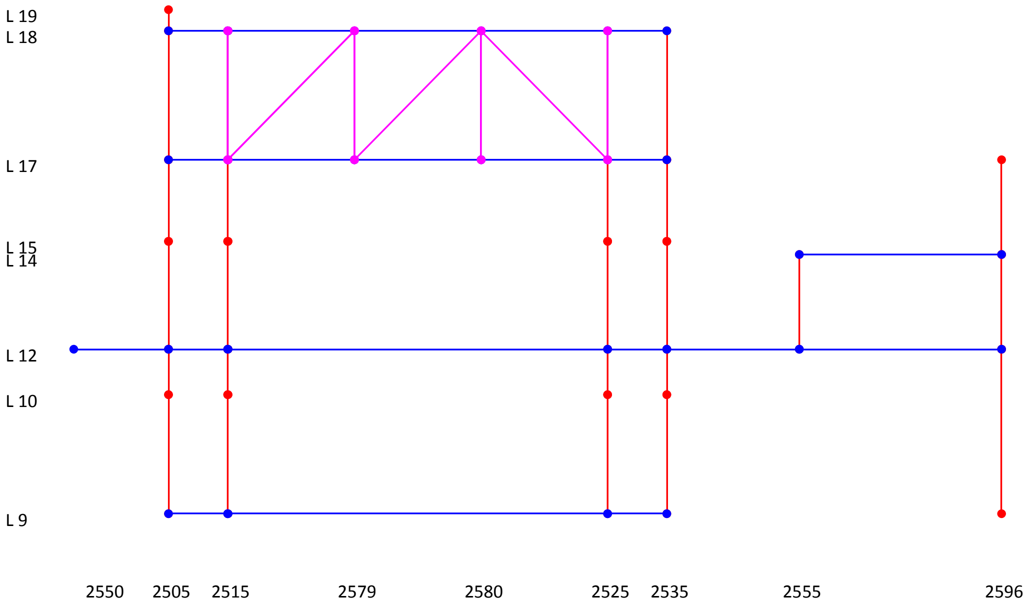


Figure 4-17 is a section showing structural elements in the N-S direction. This elevation connects column lines 2550 to 2596 (see Figure 4-16). There are ten frames each of four close coupled columns which act as one-way concrete frames in conjunction with the reinforced concrete beams supporting the floor slab. The columns of the two end frames and the central frame (Figure 4-17 is a central frame) cantilever above the suspended level to form a double height space. These columns support the steel roof trusses and a lightweight roof. The intermediate frames extend to the suspended floor (L12 in the model) only.

Figure 4-17 North-South Section of Entrance Foyer Model



In the E-W direction, the close coupled columns are linked together with a reinforced concrete slab at floor level (and part-way up their height for the end and central frames) to act as narrow frames to provide lateral resistance in this direction. Column groups are linked together with foundation beams, supported on reinforced concrete enlarged base piles. Figure 4-18 shows the outer frames (column lines 2510 to 2501) and Figure 4-19 shows the inner frame (column lines 2520 to 2511). The roof trusses span between columns in the inner frames.

Figure 4-20 shows a rendered view of the complete Entrance Foyer model. Details of the roof construction above the trusses were not available from the drawings. The model assumes a roof equivalent to 50 mm concrete with a seismic weight of 2 kPa (Section 72 in Table 4-4). The roof does not need to act as a transfer diaphragm as all lateral load elements are symmetrically laid out and so the stiffness and strength of the roof is probably not an important parameter.

Figure 4-18 East-West Section of Entrance Foyer Model (Outer)

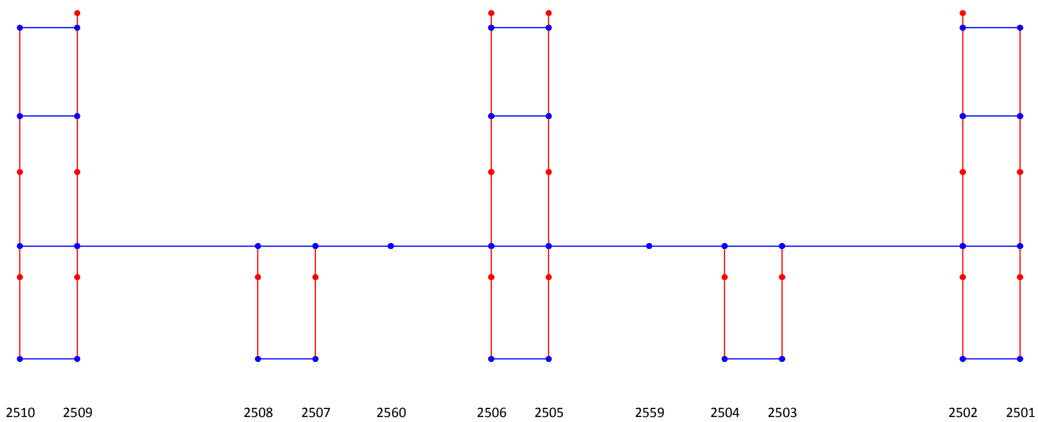


Figure 4-19 East-West Section of Entrance Foyer Model (Inner)

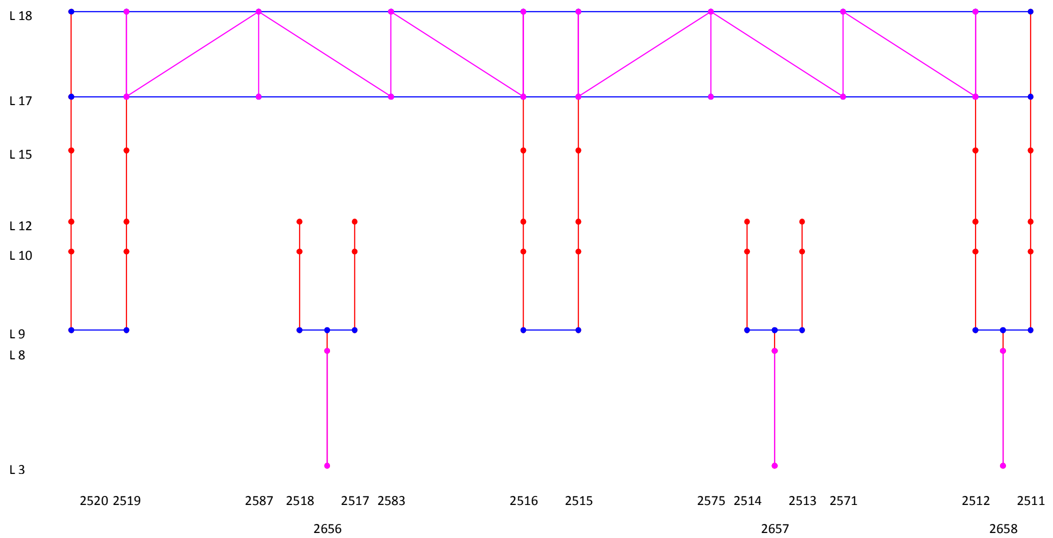
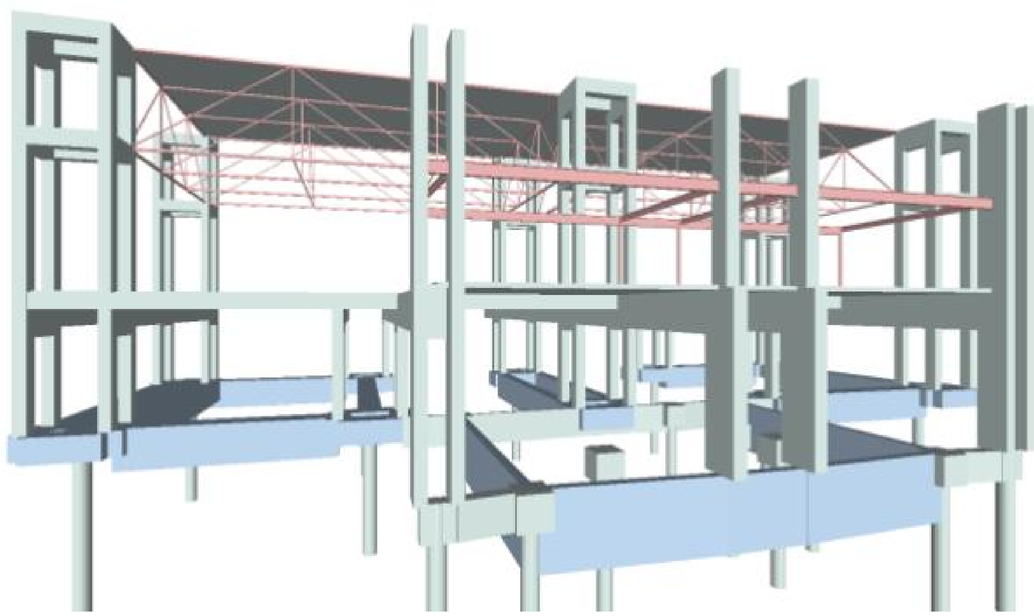


Figure 4-20 Rendered View of Entrance Foyer Model



4.6 STAIR BLOCK MODEL

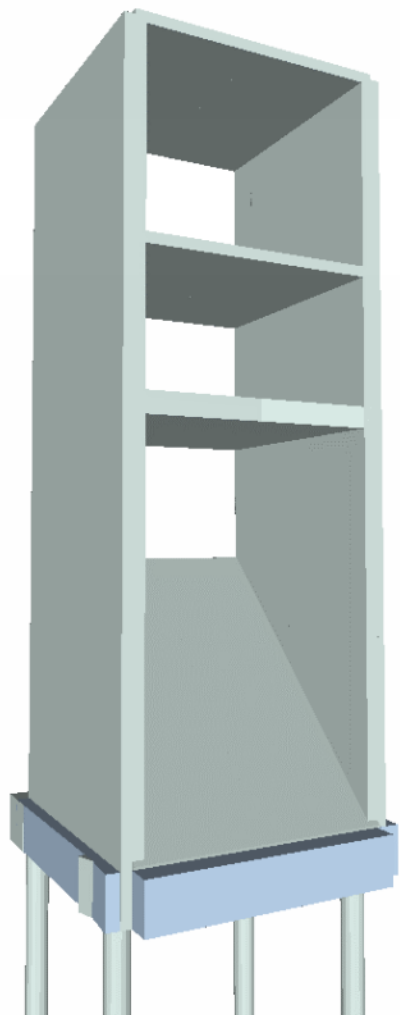
The structural system of the Stair Block structures comprises parallel cantilever wall structures in the N-S direction and a "frame" structure in the E-W direction. The frame is formed by the floor slabs in the Stair acting as beam elements with plastic hinges forming at the ends within the slabs. Figure 4-21 shows a rendered view of one of the stair blocks.

There are connections between the stair blocks and the Auditorium at Level 2 and Level 4. However, the configuration and effectiveness of these is uncertain. Because of this, the Stair

Blocks were modelled and evaluated as "stand alone" structures. The model does include the connections so that the Stair Blocks can be connected as part of the Auditorium model if required.

Each side wall was modelled as a plane stress elements with a thickness equal to the specified thickness of 400 mm (Section 13 in Table 4-4). As the plane stress elements have in-plane stiffness only, the out of plane stiffness was modelled by adding columns at each end of the wall. These columns, which are 400 mm deep and have a width equal to one-half the wall width of 5.000 m, are Section 220 in Table 4-6. The column elements framed into beams representing the floor slabs. The beams which are 200 mm deep and have a width equal to one-half the slab width of 4.800 m, are Section 326 in Table 4-8.

Figure 4-21 Rendered Views of Stair Block Model



5. SEISMIC RESPONSE OF BUILDING

The finite element models as defined in the MODELA spreadsheet were used to develop series of input files for the analysis programs:

1. Linear elastic models for the extraction of periods and mode shapes.

2. Nonlinear models for pushover analysis using the ANSR-II program. A separate input file was generated for each translational direction.

3. Nonlinear model sfor time history analyses. ANSR-II input files were generated for various earthquakes, each at 2 orientations.

These models were used to define the seismic response characteristics of the building and evaluate seismic response.

All nonlinear analysis results are based on the model with gap elements installed at the base of every pile.

5.1 DYNAMIC CHARACTERISTICS

The periods and mode shapes were extracted for each of the buildings in the complex. As there are no rigid diaphragms, each model typically has a large number of local modes. A total of 50 modes were extracted for each configuration. Table 5-1 summarizes the periods for each model. For each of the models the following results are provided:

1. The dominant mode in each direction, defined as the period with the greatest effective mass factor.

2. The effective mass associated with this dominant mode is listed.

3. The period for the 50th mode is listed, plus the cumulative total effective mass in each direction after 50 modes.

5.1.1 Auditorium

The Auditorium has fundamental periods in the X (N-S) and Z (E-W) directions of 0.27 seconds and 0.35 seconds respectively. The fundamental modes have relatively small effective mass, which is a function of the lack of rigid diaphragms and the open seismic joints between portions of the structure.

The periods are defined as short (less than 0.40 seconds) and as such, their seismic response would tend to be dominated by accelerations rather than displacements and secondary effects (P-delta) would be expected to be relatively minor.

There are a large number of local modes such that Mode 50 still has a relatively long period of 0.138 seconds and effective mass is less than 70%.

5.1.2 Entrance Foyer

The Entrance Foyer modes are extracted from a model which is disconnected from the Auditorium. This structure is very flexible for the height of the structure, with periods of 0.64 and 0.89 seconds. The fundamental modes have effective mass of 50% and 89% in the X and Z direction respectively. As there are fewer local modes, the first 50 modes account for almost 100% effective mass.

5.1.3 Stair Blocks

As for the Entrance Foyer, the Stair Block modes are extracted from a model which is disconnected from the Auditorium. This structure is stiff in the X (shear wall) direction with a period of 0.12 seconds but is very flexible in the Z (frame) direction, with a periods of 0.65 seconds. The fundamental modes have effective mass of 50% and 60% in the X and Z direction respectively. Again, there are fewer local modes and the first 50 modes account for almost 100% effective mass.

Table 5-1 Building Periods

	Auditorium	Entrance Foyer	Stair Blocks
X (N-S) Direction			
Period (Seconds)	0.267	0.637	0.123
Effective Mass	56.4%	50.1%	49.4%
Z (E-W) Direction			
Period (Seconds)	0.345	0.885	0.648
Effective Mass	25.0%	89.2%	60.4%
After 50 Modes			
Period (Seconds)	0.138	0.058	0.004
X Effective Mass	69.4%	100.0%	99.9%
Z Effective Mass	62.1%	99.9%	100.0%

5.2 CAPACITY CURVES

Prior to the time history analysis, it is usual to perform a nonlinear static analysis to quantify the overall strength of the building. In this type of analysis, often termed a pushover analysis, a lateral load distribution corresponding to the equivalent static load is applied to the building in small increments and the roof displacement recorded at each step. This provides a plot of applied load versus displacement, termed the capacity curve.

In this procedure the load vector is proportional to the seismic weight at every node of the analysis model. When there are no rigid diaphragms then premature failure in a local mode may terminate the analysis. As a general principle, a pushover analysis is of greatest utility for rigid diaphragm buildings.

Notwithstanding this, a lateral load was applied to each of the models in 1000 steps at an increment of 0.010W, a total load of 1.00W. Although the lateral loads were specified to a maximum of 1.0W, the analyses in most usually terminated at a lower level of load due to numerical instability when equilibrium could not be achieved.

The capacity curves in the following sub-sections plot the displacement versus lateral load up to the point at which each analysis terminated. This does not imply that the buildings are safe up to the level plotted as many of the elements have increasing levels of damage and a detailed examination of component response is required to check adequacy. This is done using the results from the time history analyses described later.

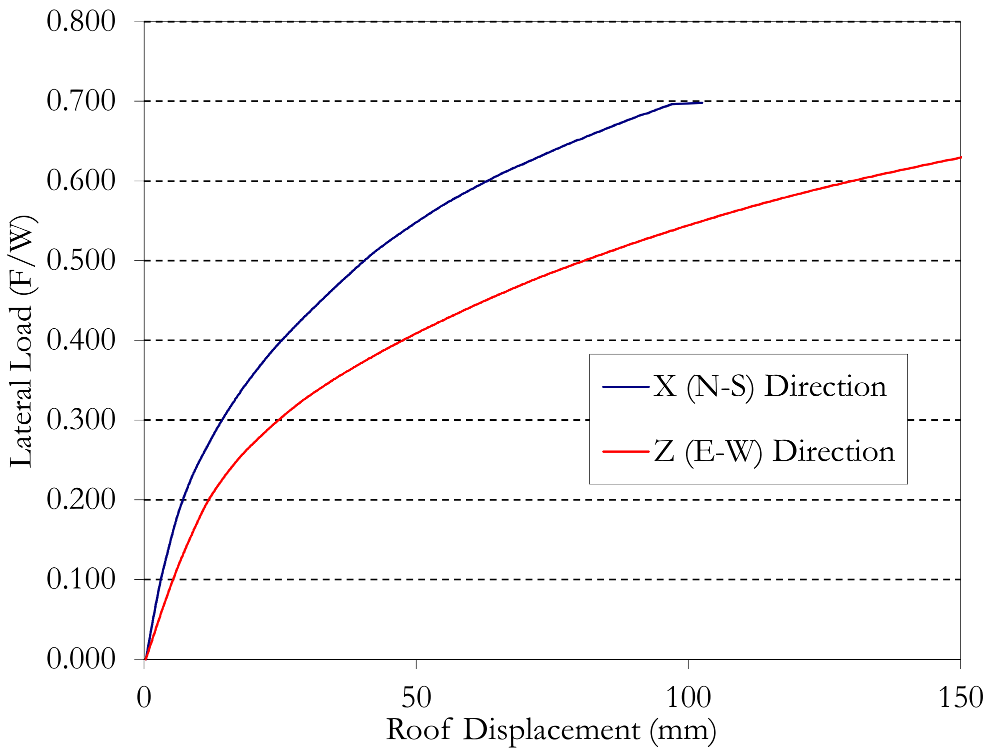
5.2.1 Auditorium

Figure 5-1 plots the Auditorium capacity curve. This curve shows a relatively high level of ultimate strength, 0.7 W in the N-S direction, and over 0.6W in the N-S direction. The difference in the two directions reflects the approximately elliptical shape of the lateral load resisting system.

Deflections are relatively small given the height of the structure, with maximum deformations of about 150 mm, which represents a drift ratio of about 0.5% over the 29.4 m height from the support to the point where displacements were measured. This is expected given that the short periods (Table 5-1) indicate a stiff structure.

The curves do not show a definite yield point but rather indicate progressive softening up to X displacements of 100 mm and X displacements of 150 mm. After that point the onset of instability is rapid.

Figure 5-1 Auditorium Capacity Curve

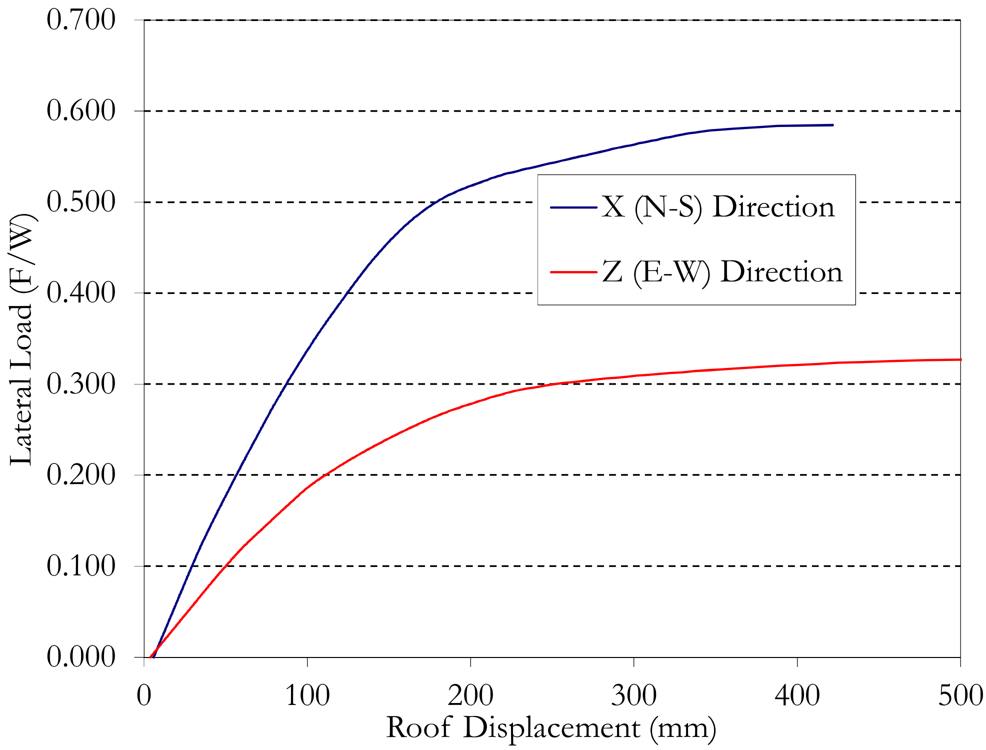


5.2.2 Entrance Foyer

Figure 5-2 shows that the Entrance Foyer has a definite yield point in the X direction at about 0.5W and the analysis terminates at 0.6W. This is in the transverse frame direction (Figure 4-17). In the Z direction where lateral load resistance is provided by the narrow frames formed of floor slabs (Figures 4-18 and 4-19) the frame is more flexible and has a lower strength, with a yield level of about 0.30W and ultimate strength of about 0.35W.

The height at which displacements are measured for Figure 5-2 is 13.90 m above fixity and so a 100 mm displacement corresponds to a drift of 0.7%.

Figure 5-2 Entrance Foyer Capacity Curve

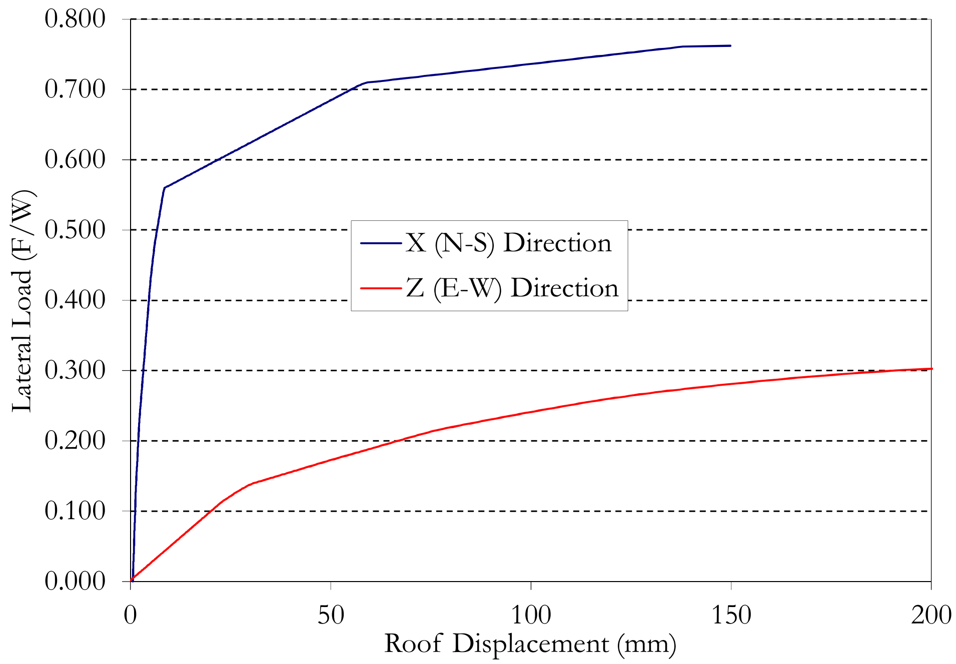


5.2.3 Stair Blocks

The Stair Block capacity curves are plotted in Figure 5-3. These curves show a large difference between the X (wall) and Z (frame) direction. The walls are stiff and relatively strong in-plane and are able to resist the maximum applied lateral load of over 0.75W. In this direction the resistance is essentially elastic up to 0.55W at 10 mm displacement after which rocking commences and the stiffness reduces.

In the "frame" direction the floor slabs acting as beams provide low levels of strength and stiffness and this is reflected in the capacity curve. In this direction, yield occurs at a load level of 0.15W and a displacement of about 30 mm. The ultimate strength is approximately 0.30W.

Figure 5-3 Stair Block Capacity Curve



6. TIME HISTORY ANALYSIS

6.1 SCOPE OF ANALYSES

The time history analyses were performed on three separate models, the Auditorium, the Entrance Foyer and a Stair Block.

All time history analyses used the Newmark beta method. Damping was applied using Rayleigh coefficients. These were calculated to provide 5% viscous damping at both the longest period and the period at which 90% effective mass was achieved. For all periods between these limits the effective damping would be less than 5%.

The analysis time step is generally defined as not greater than 1/100th of the fundamental period, which requires maximum time steps of 0.0025 seconds for the Auditorium, 0.005 seconds for the Entrance Foyer and 0.00125 seconds for the Stair Block. A value of 0.005 seconds was used except for the stairs, where a value of 0.00125 seconds was selected.

The response under NZS1170 input was assessed using the three earthquakes described earlier scaled to the requirements of an R = 1.3 structure. The two components of each record were applied simultaneously, first with the dominant component in the X direction and then with the dominant component in the Z direction.

The sets of analyses were repeated with different scale factors to quantify earthquake damage versus seismic amplitude to assess the overall performance of the structure.

6.2 AUDITORIUM

6.2.1 Assessment of Seismic Performance

As discussed in Section 2.1, performance equivalent to that of a new building requires meeting two criteria:

1. Performance within ULS limit state at NZS1170 loads (R = 1.3 for this structure).

2. Performance within CLS limit state at 150% of NZS1170 loads.

The global results of sets of analyses of the Auditorium at 67%, 90%, 100% and 125% NBS are summarized in Table 6-1.

1. Table 6-1 shows that at 67% of NZS1170 loads drifts are low and all components are within the ULS limit.

2. At 90% of NZS1170 loads drifts are much higher but still well within the NZS1170 limit of 2.5%. There is one panel which appears to exceed the CLS limit. A more detailed assessment would show that the shear strains in this panels is not likely to lead to significant risk of collapse. On this basis, the performance at this level of load is assessed as meeting the CLS limit state.

3. At 100% of NZS1170 loads only 5 of the 6 analyses completed. The remaining analysis terminated prematurely because displacements exceeded the pre-defined termination level of 5.000 m. At this level of load, there were multiple panels, columns and beams which exceeded the CLS and so the performance was assessed as failing.

4. Similarly, at 125% of NZS1170 loads only 3 of the 6 analyses completed, there were multiple panels, columns and beams which exceeded the CLS and so the performance was assessed as failing.

Based on these results, the seismic performance of the Auditorium would be assessed as 67% NBS as the ULS is reached at 67% of NZS1170 design loads and the CLS at 1.5 times this, equivalent to 100% of NZS1170 design loads.

Table 6-1 Auditorium Global Response

	Seismic Input (%NZS1170 R =1 .3 Input)			
	67%	90%	100%	125%
Completed Runs (of 6)	6	6	5	3
Drifts				
X	0.36%	0.58%	0.68%	1.92%
Z	0.32%	1.01%	1.07%	1.91%
Panel Deficiencies				
> ULS	0	6	9	18
> CLS	0	1	6	17
Column Deficiencies				
> ULS	0	1	2	9
> CLS	0	0	4	14
Beam Deficiencies				
> ULS	0	0	4	4
> CLS	0	0	5	7
Global Rating	ULS	CLS	FAIL	FAIL

6.2.2 Failure Mechanism

As listed in Table 6-1, one analysis at 100% and three analyses at 125% of NZS1170 loads terminated due to excessive displacements. There were deficiencies in panels, column and beams as shown in Figure 6-1 (refer to Table 6-2 for the colour key). Figure 6-2 shows that the damage was concentrated at the South end of the Auditorium in the lower levels of the building, although there were isolated failing components in other regions.

Figure 6-2 plots the deformed shape of the Auditorium model at the point at which the El Centro analysis terminated, approximately 12 seconds. This shows that there is apparent buckling of a column at Grid 5W, with consequent failure of components adjacent to this.

Table 6-2 Legend for Damage Plots

(green)	< ULS
(blue)	> ULS
(red)	FAILING

Figure 6-1 Auditorium Damage at 125% NBS

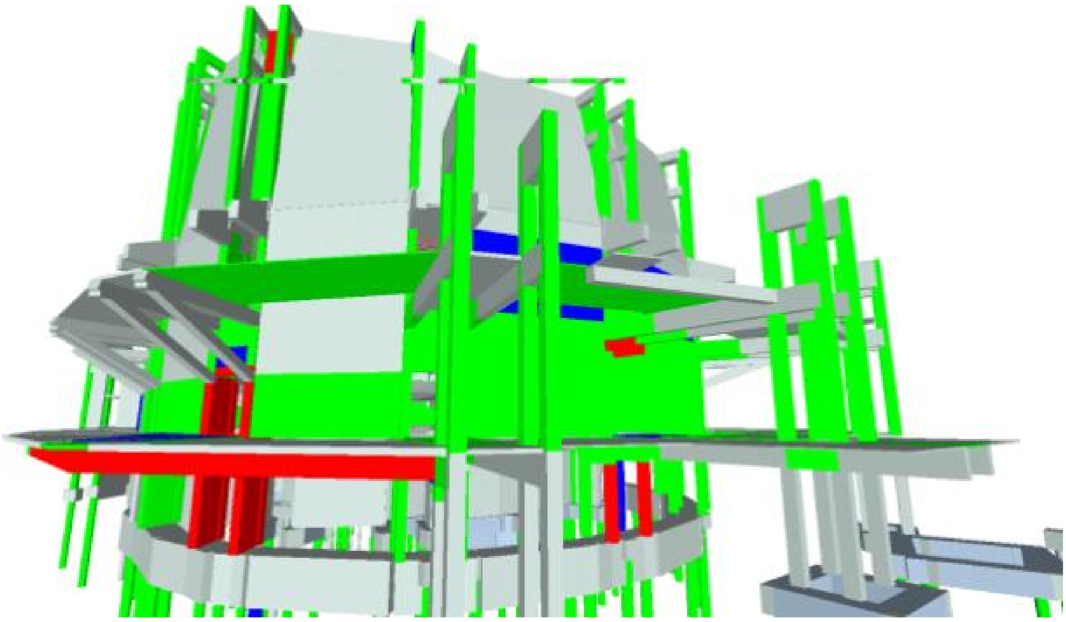


Figure 6-2 Auditorium Deformed Shape at 12 Seconds of El Centro 1940 x 2.60

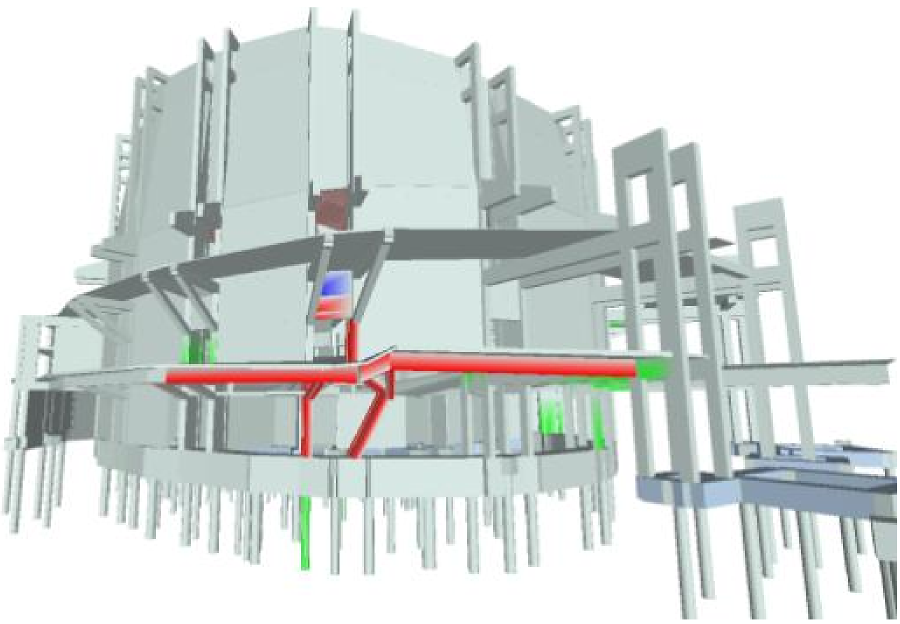
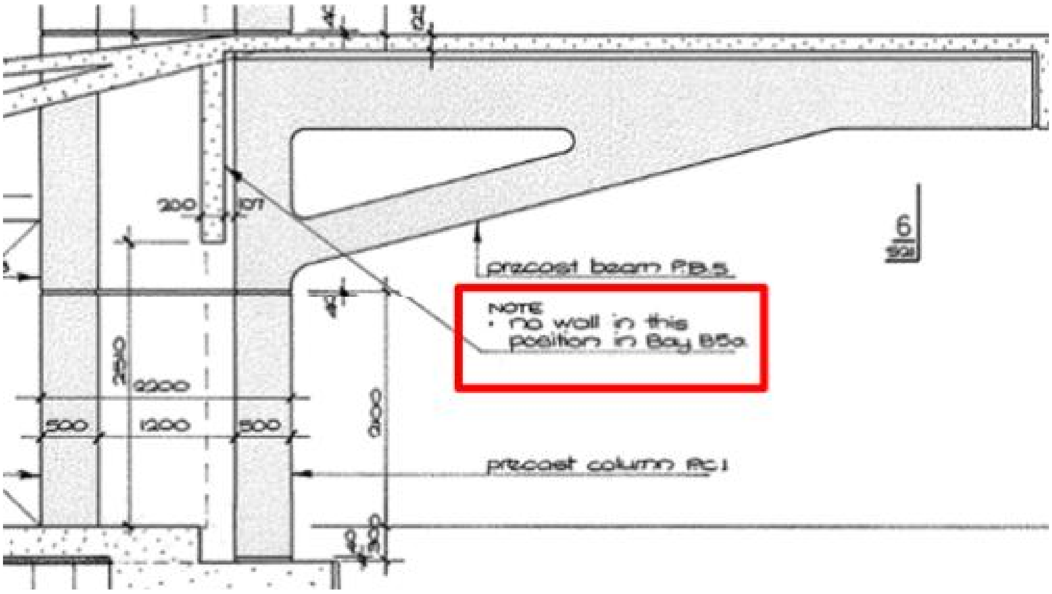


Figure 6-3 extracted from the original structural drawings set (Drawing S20) shows that there is a wall segment missing from this column, which results in excessive unbraced length in the minor axis direction. This appears to be the cause of the sudden failure as loads increase from 100% to 125% NZS1170.

Figure 6-3 Auditorium Missing Wall at Bay 5a



6.2.3 Auditorium Roof Potential Understrength

The Auditorium Roof was modelled using plane stress elements with properties based on a concrete material with a thickness of 50 mm, equal to the thickness of the topping on the hollowcore precast panels.

The properties of the roof panels are listed in Table 4-4 as Section 71. The panel has a concrete shear strength of 910 kPa and a steel shear strength of 1485 kPa based on steel content of 0.0050 (average of the two mesh sizes used, 661 and 663) time a steel strength of 297 MPa. The ultimate strength at a 1.25 steel overstrength is 2766 kPa, which corresponds to 138 kN/m for the 50 mm thickness.

Using these properties, at the CLS of 90% NZS 1170 loads, a number of panels reached the strength limit and so responded nonlinearly in shear. However, the maximum strain of 0.010 was well within the criteria limit of 0.020 for concrete panels governed by shear and so the roof panels did not form a limit state governing the building capacity.

There are some uncertainties as to the integrity of the connections and their ability to transfer this level of shear. To assess sensitivity to this, the concrete thickness was reduced by a factor of 4, which effectively reduced the stiffness and strength by this ratio. The maximum shear strength reduced from 138 kN/m to 35 kN/m.

The analyses were repeated with this modification. Excluding the roof elements, the response was slightly improved compared to that reported in Table 6-1 in that collapse level damage occurred later, 120% of NZS1170 compared to 100% NZS 1170. However, severe roof damage occurred relatively early. At 67% NZS 1170 seismic input, 4 panel elements (of the 95 elements representing the roof) has shear strains exceeding the CLS limit of 0.020 (maximum strain of 0.028) and at 90% NZS 1170 30 of the panels exceeded the CLS limit, with a maximum strain of 0.040, twice the CLS limit.

From these results, it can be concluded that the roof does not perform a significant transfer function because the response of the major structural elements does not increase due to the excessive roof deformations. However, the roof is required to distribute inertia loads arising from its own self weight (5kPa in the model) to the structural elements around the perimeter. It appears that if the roof capacity is significantly lower than assumed then this may form a limiting state to the overall rating of the building.

Further, seating of the hollowcore roof panels onto the steel roof trusses is not ideal and further review and detailed inspection to the roof and roof trusses and concrete connections is warranted - refer to further discussion in Section 7.4.

6.3 ENTRACE FOYER

6.3.1 Assessment of Seismic Performance

Table 6-3 summarizes the seismic performance of the Entrance Foyer for a series of analyses with the amplitude ranging from 50% of NZS 1170 input to 100% NZS 1170 input. Each amplitude is assessed a rating of ULS (performance within the ultimate limit state), CLS (performance exceeding the ultimate limit state but within the collapse limit state) or FAIL (performance exceeding the collapse limit state).

Table 6-3 Entry Foyer Global Response

	Seismic Input (%NZS1170 R =1 .3 Input)			
	50%	67%	83%	100%
Completed Runs (of 6)	6	6	6	6
Drifts				
X	1.27%	1.69%	2.08%	3.07%
Z	2.03%	2.79%	3.60%	7.32%
Column Deficiencies				
> ULS	0	0	0	0
> CLS	1	1	1	51
Beam Deficiencies				
> ULS	2	3	0	10
> CLS	0	3	6	9
Global Rating	ULS	ULS	ULS	FAIL

For each amplitude, the full set of six time histories completed without numerical instability due to excessive displacements.

1. The drift limit for time history analysis are 2.5% (increased to 3.73% for records which include forward directivity effects (FD) for ULS and 3.75% (increased to 5.6% for FD) for the CLS. At 67% input, the peak drift for the non-FD record is 1.95% and for the two FD records 2.79%. At 83% input, the peak drift for the non-FD record is 2.33%

and for the two FD records 3.60%. These are within the appropriate ULS limits. At 100% input the drifts exceed both the ULS and CLS limits.

2. There is a single column deficiency tabulated for input up to 83% and multiple column deficiencies at 100%. The single column deficiency is in a pile due to excessive axial loads. As discussed below, this deficiency is not rated as critical.

3. There are beam deficiencies exceeding ULS for input of 67% and above. Reinforced concrete beam deficiencies are non-critical unless they are low redundancy transfer elements or the beams support precast concrete floors and failure could lead to progressive collapse of floor units below. As this Entrance Foyer has a single suspended floor, these deficiencies are ranked as non-critical.

Based on the drift and component assessment, the Entrance Foyer is ranked as meeting both the ULS and CLS limits for seismic input up to 83% of NZS 1170 loads but fails for higher loads.

6.3.2 Failure Mechanism

Figure 6-4 identifies the components identified as exceeding the CLS at 83% NZS 1170 (one column and six beams, shaded red in the figure).

The single failing column is shown in Figure 6-4. In the analysis model, this pile is loaded to 93% of the ultimate compressive capacity under gravity alone. The pile layout is non-symmetrical and the location is an area where the subterranean structure is complex with tunnels and foundation walls. The model representation in this area is greatly simplified and this likely results in excessive axial load assigned to this component. For this reason, the pile overload is not assessed as a critical deficiency.

The other deficiencies identified in Figure 6-4 are in six beams at first floor level. As noted above, these are non-critical deficiencies as the damage will not lead to progressive collapse of floors below.

At 100% of NZ S1170 loads, the level identified as failing in Table 6-3, the failing components are identified in Figure 6-5. These components are all columns at the ground story of the Entrance Foyer. As shown in the displaced shape in Figure 6-6, a soft-story type column hinging mechanism forms and all deformations are concentrated in this story.

Figure 6-4 Entrance Foyer Damage at 83% NBS

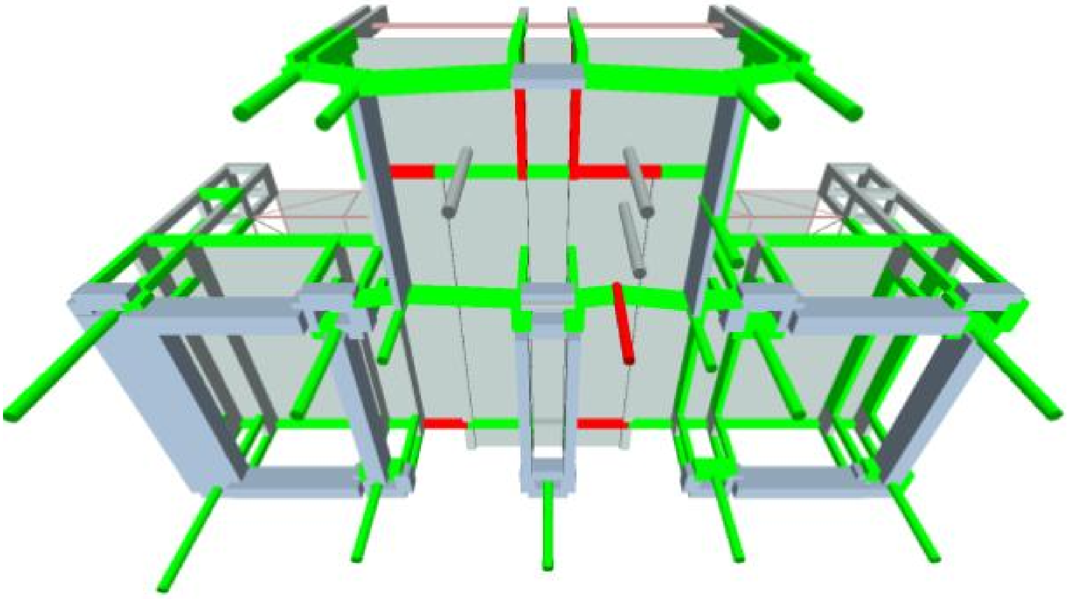


Figure 6-5 Entrance Foyer Damage at 100% NBS

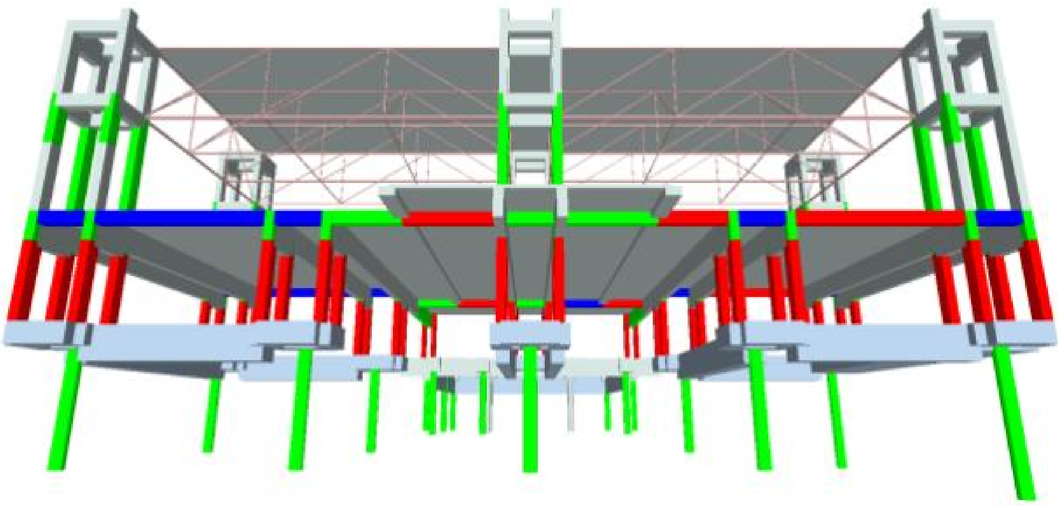
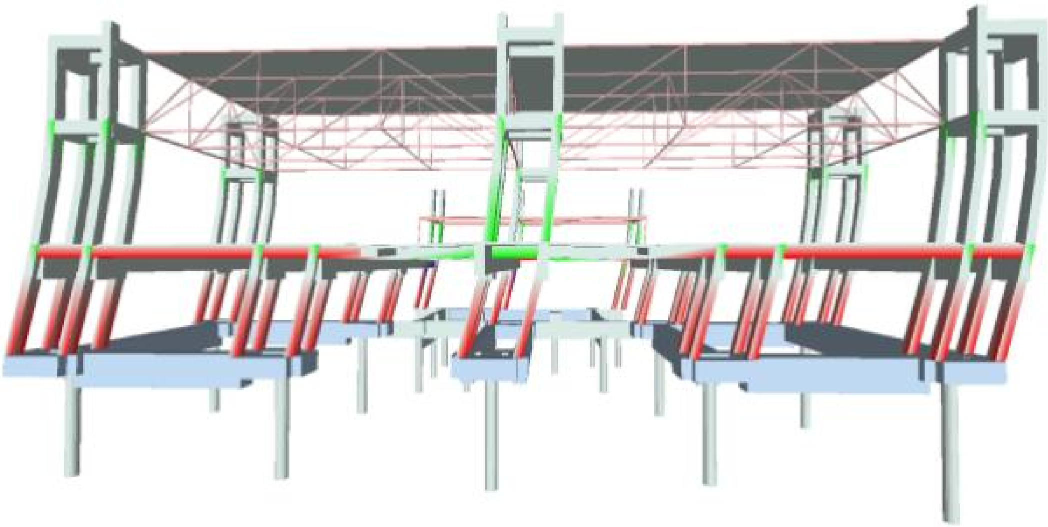


Figure 6-6 Entrance Foyer Deformed Shape at 7.21 Seconds of El Centro 1979 x 1.69



6.4 STAIR BLOCK

6.4.1 Assessment of Seismic Performance

Table 6-4 summarizes the seismic performance of the Stair Blocks for a series of analyses with the amplitude ranging from 33% of NZS 1170 input to 100% NZS 1170 input. Each amplitude is assessed a rating of ULS (performance within the ultimate limit state), CLS (performance exceeding the ultimate limit state but within the collapse limit state) or FAIL (performance exceeding the collapse limit state).

1. At 33% and 67% amplitude the full set of six time histories completed but for 100% only one analysis completed, the remainder terminating from numerical instability due to excessive displacements.

2. There are no beam or column deficiencies up to 67% NZS 1170.

3. There are column and beam deficiencies at 100% NZS 1170.

It is apparent that the defining deficiency in the stair block is excessive drifts in the X (frame) direction. As discussed above, drift limits from NZS 1170 for time history analysis are:

a) ULS drift limit are 2.5% (increased to 3.73% for records which include forward directivity effects (FD).

b) CLS drift limits of 1.50 times these, corresponding to 3.75% (increased to 5.6% for FD).

In order to assess compliance with these between 67% and 100%, the set of analyses was repeated for amplitudes from 65% to 100% at a 5% increment. Peak drifts from these analyses are summarized in Table 6-5. These show that the drifts are within ULS up to 70%, within CLS up to 80% and exceeding CLS above 80%.

Table 6-4 Stair Blocks Global Response

	Seismic Input (%NZS1170 R =1 .3 Input)		
	33%	67%	100%
Completed Runs (of 6)	6	6	1
X	0.02%	0.06%	0.40%
Z	1.23%	2.90%	112.00%
Column Deficiencies			
> ULS	0	0	0
> CLS	0	0	23
Beam Deficiencies			
> ULS	0	0	0
> CLS	0	0	8
Global Rating	ULS	ULS	FAIL

Table 6-5 Stair Block Drifts

Input	Maximum X Drift	Maximum Z Drift	Maximum of Non-FD Records	Maximum of FD Records	Rating
100%	0.40%	112.00%	85.%	112%	FAIL
95%	0.82%	205.00%	155%	205%	FAIL
90%	0.39%	110.00%	7.13%	110%	FAIL
85%	0.68%	219.00%	3.99%	219%	FAIL
80%	0.11%	4.52%	3.02%	4.52%	CLS
75%	0.09%	3.45%	3.07%	3.45%	CLS
70%	0.07%	3.01%	2.38%	3.01%	ULS
65%	0.06%	2.81%	2.07%	2.81%	ULS

6.4.2 Failure Mechanism

The failure mechanism for the Stair Blocks is excessive plastic rotations in the stair walls bent about their weak axis, caused when they function as the frame columns. Figure 6-7 identifies the deficient components and Figure 6-8 plots the deformed shape which gives rise to these deficiencies.

Figure 6-7 Stair Foyer Damage at 100% NBS

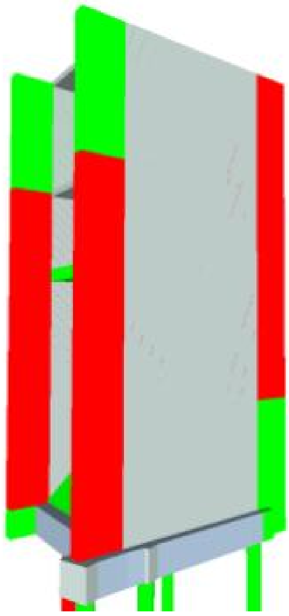
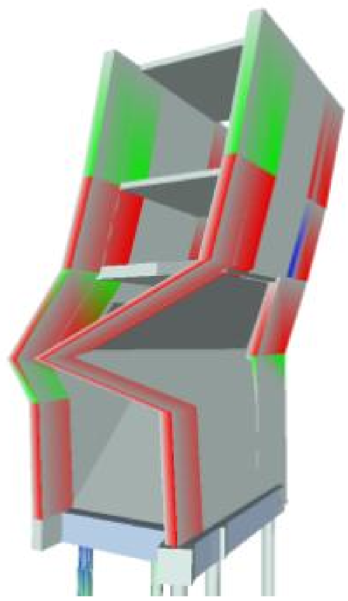


Figure 6-8 Stair Deformed Shape at 8.7 Seconds of El Centro 1979 x 1.94



7. LOCAL ELEMENT PERFORMANCE

7.1 STAIRS AND EGRESS

Due to the number and complexity of stairs in the Michael Fowler Centre, they have not been explicitly modelled as part of the time history analysis, generally, as they are not expected to alter the global response of the whole building. Where stairs are expected to contribute significantly to the response of the building as a whole, or influence the response of major lateral load resisting elements, they have been incorporated into the time history model e.g. stair flight within the Stair Blocks.

Due to the function of stairs to provide for public access and importantly emergency egress following a seismic event, review of stair construction details and a separate assessment of their performance is undertaken separate to the time history analysis.

Stairs that have been judged to be the most critical for emergency egress have been assessed by inspection of construction detailing in the original structural drawings, review of the intended mechanisms to accommodate movements across the stair between adjacent floors and reviewed on site to confirm as-built details against the intentions in the structural drawings. Figure 7-1 shows the plan locations of each stair and identification numbering from the original structural drawings. Where stair numbering from the original structural drawings differs from signage within the building, this has been noted in the discussion.

All stairs are generally reinforced concrete cast-insitu or precast with throat thickness that varies from 150mm to 200mm.

Secondary access stairs to the basement or to ceiling and roof spaces within the Auditorium have not been considered as part of this assessment work.

7.1.1 Stairs 1 and 2 (stairs 2 and 1, respectively as per building signage)

Stairs 1 and 2 provide main public access, and emergency egress, between the ground floor entrance and the main Foyer at Level 2.

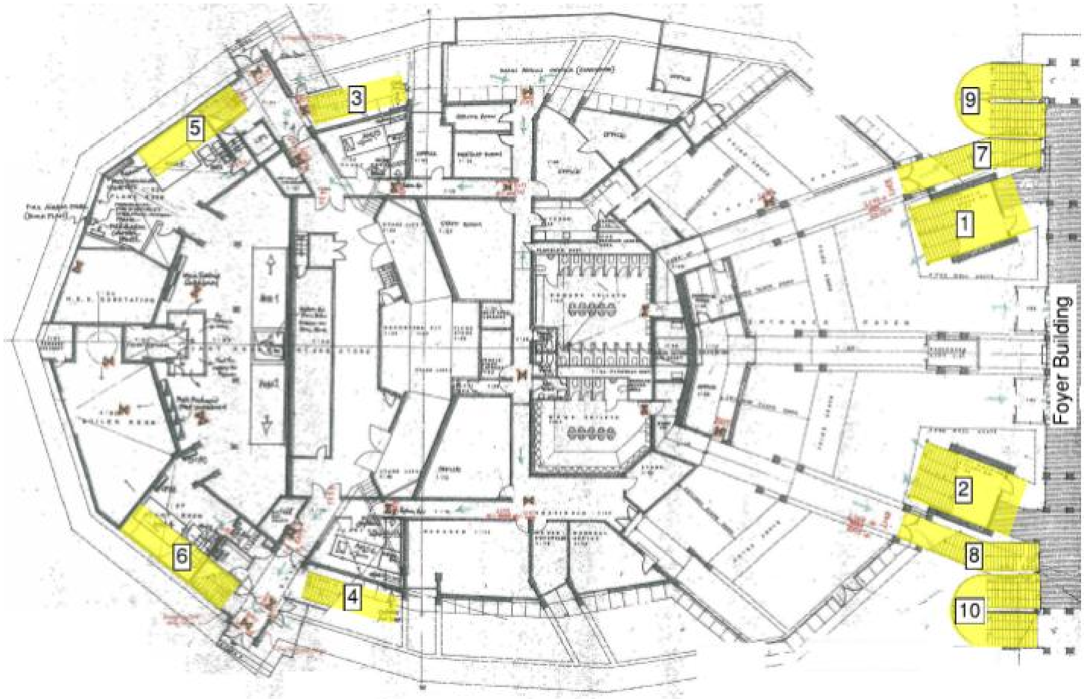
The stair flights are fixed at the base and are cast insitu into the stair block walls on either side. The top flight of this stair appears to have been designed to cantilever beyond the mid-height landing and is provided with a sliding movement joint detail at the underside of the Foyer slab at Level 2. This isolates the stair from any potential for damage during a seismic event due to relative movement between the Foyer and the Stair Block.

7.1.2 Stairs 3 and 4

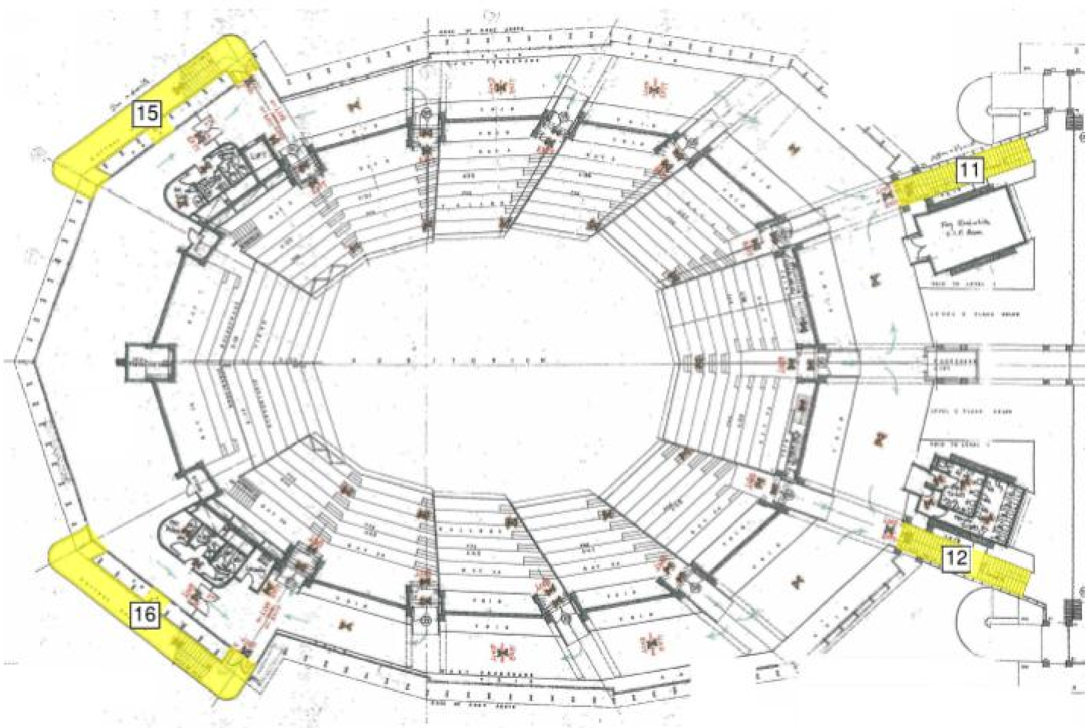
Stairs 3 and 4 provide internal access from the Auditorium ground floor (Level 1) to the lower promenade at Level 2 toward the northern end of the building. These stairs are connected along one longitudinal edge with cast in-situ reinforcement projecting from the adjacent structural shear wall and are fixed top and bottom into the floor slabs.

Seismic performance of these stairs is not expected to be an issue, due to the relatively small interstorey displacements and construction of the stair tying it into the adjoining concrete shear wall.

Figure 7-1: Locations of stair wells reviewed



Reviewed Flights of Stairs - L1 to L2



Reviewed Flights of Stairs - L2 to L4

Figure 7-2: Stair 2 (Stair 1 as per building signage)

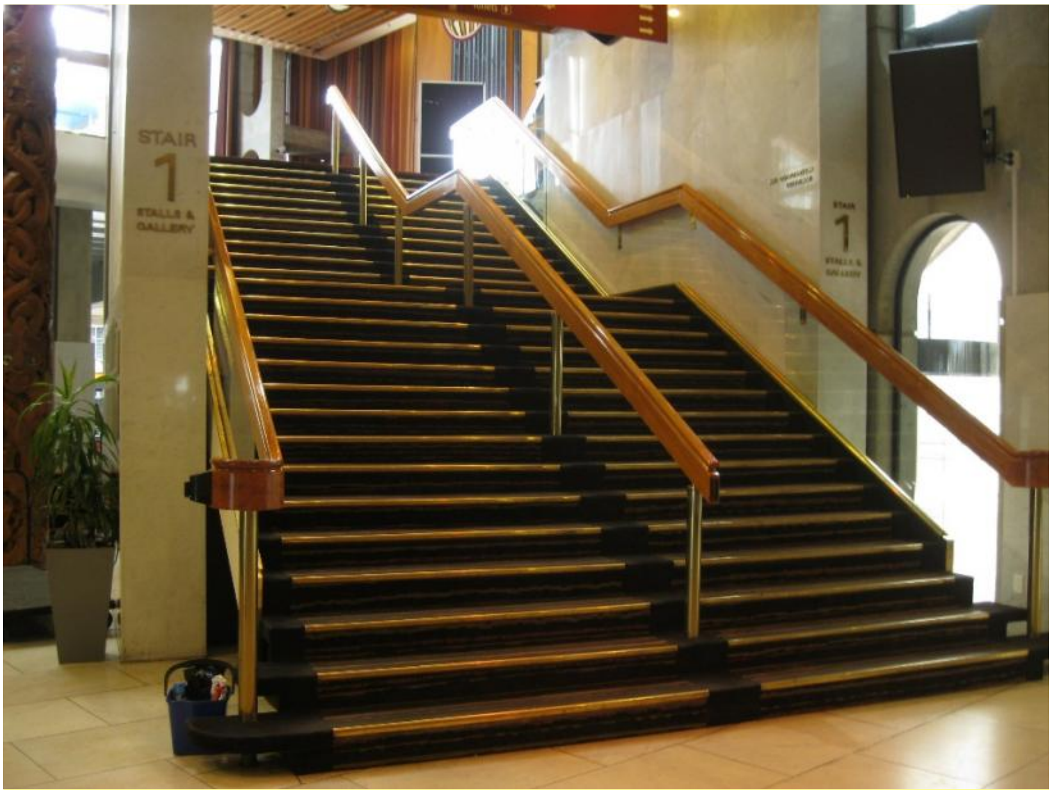


Figure 7-3: Stair 4

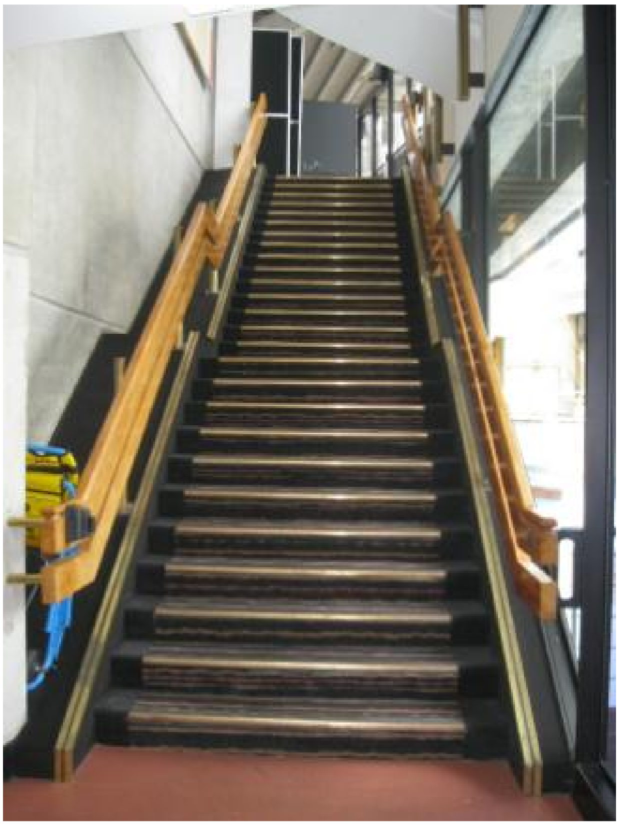


Figure 7-4: Stair 6

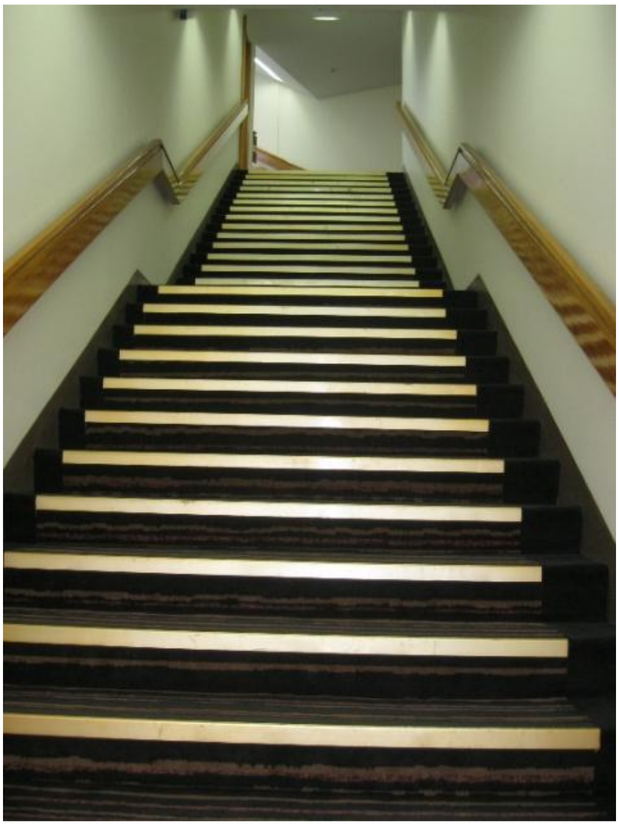


Figure 7-5: Stair 8

7.1.3 Stairs 5 and 6

Similar to Stairs 3 and 4, Stairs 5 and 6 provide access from Level 1 to 2 into areas behind the Auditorium stage and dressing rooms (Frank Taplin Room). The stair flights are fixed top and bottom into the floor slabs and are also built into concrete shear walls along both sides of the stair. Again, interstorey displacements are small and the stair is restrained by connection into the shear walls, so significant damage is not expected in a seismic event.

7.1.4 Stairs 7 and 8

Stairs 7 and 8 are external egress stairs between Level 2 of the Auditorium down to ground level. The stairs are cast-into the floor slab at Level 2 of the Auditorium and span down to a mid-height landing, which is cast into a reinforced concrete beam projecting from the exterior of the Stair Block wall. The base of the stair is supported on a strip footing detail buried below the finished ground surface. In addition the stair is kinked in plan, at the mid height landing.

These stairs are fixed at the top and at the mid height landing, between the Auditorium and the Stair Block structures, with no provision for accommodating any relative movement between the two structures. There is some evidence of past cracking and existing repairs at the top of both stairwells (underside of Level 2) likely due to relative movement from past minor earthquakes or other environmental effects.

Fixity at the base is of the stair is of minor concern, where no sliding detail is provided, however displacements of the Stair Block shear wall structure are minimal and it is expected that this can be accommodated by the lower stair flight without any structural distress.

No specific performance rating has been determined for this stair as it spans between floor levels and across two seismically independent structures. Displacement demands on this stair are likely to exceed the ultimate limit state capacity of the stair at levels of load well below 34 %NBS and these stairs are considered Earthquake Prone.

It is recommended that a structural separation is made in the stair flight, at the underside of the Auditorium Level 2 slab, to allow for seismic displacements to occur between the Auditorium and Stair Blocks while also maintaining gravity support to the stair flight.

7.1.5 Stairs 9 and 10

Stairs 9 and 10 are external stairs to provide emergency egress from the Foyer to the outside of the building. The stairs are stand-alone structures with a mid height landing that turns through 180°. They are fixed at the base, with a sliding joint detail provided at the top through a recessed pocket into the side of the Foyer floor slab.

The original structural drawings show the upper flight of the stair is supported on a corbel formed within the depth of the Foyer Level 2 floor slab with a 50mm closing gap and 125mm overlap. No provision within the corbel detail appears to be available for relative displacements perpendicular to the stair.

The corbel detail supporting the stair appears to be unreinforced, with no reinforcing details indicated on the structural drawings. While unseating of the stair flight is not expected below 34 %DBE, this supporting corbel detail is vulnerable to any damage that occurs and this may compromise gravity support of the upper stair flight. It is recommended that supplementary details are installed to secure this stair in the event the supporting corbel is damaged.

Figure 7-6: Stair 10

7.1.6 Stairs 11 and 12 (stairs 4 and 3, respectively as per building signage)

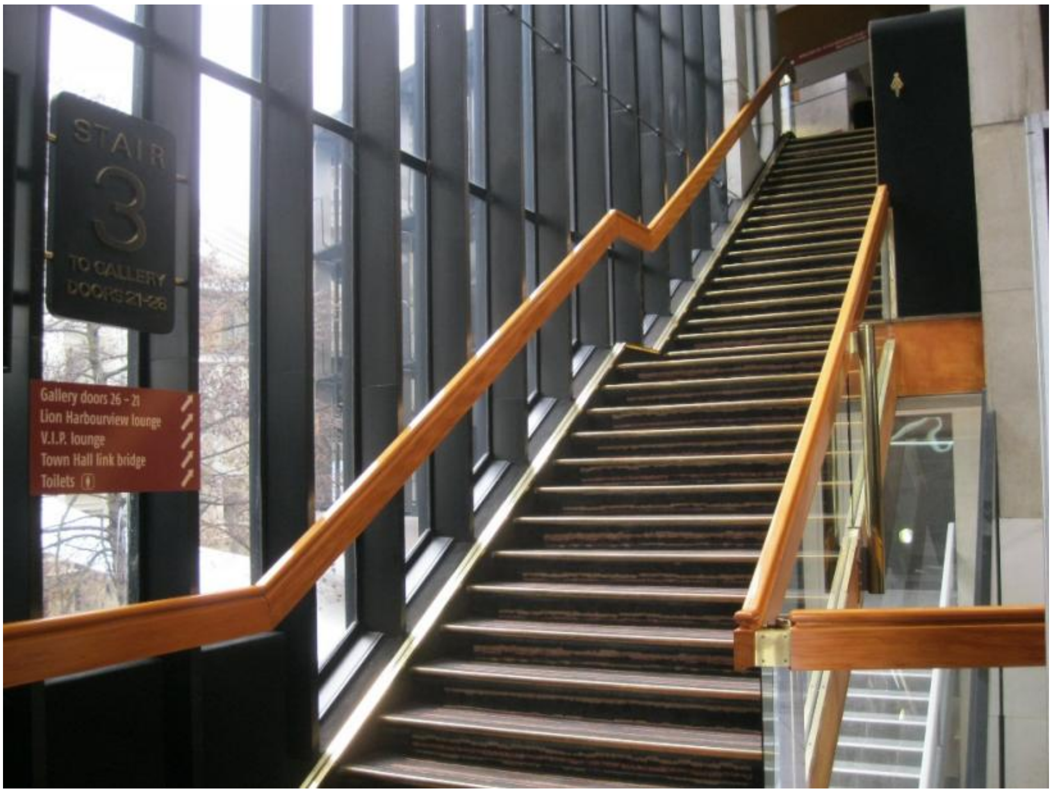
Stairs 11 and 12 (Stairs 4 and 3, respectively as per building signage) are enclosed stairs that lead from the Level 2 Foyer to the promenade at Level 4 of the Auditorium.

The stairs are formed as two separate flights with sliding joints detailed at the base of each flight (the mid-height landing and Level 2 slab). The upper flight is cast into the Auditorium floor slab at Level 4 and spans to a sliding detail at the mid-height landing which is formed by a cast in-situ slab projecting from the Stair Block wall. The lower flight is fixed at the mid-height landing, and spans to a sliding support on the Foyer building Level 2 slab.

The sliding joints will allow the Auditorium, Stair Block and Foyer to move independently, without restraint from the stair flights, during a seismic event. Some past spalling is evident at the underside of the stair flight at the mid-height sliding joint detail, but this has been repaired in the past and is considered as cosmetic only.

Assessed interstorey displacement demands which would exceed the stair support overlap distance, including the DBH recommendations for a factor of 2 margin, are greater than those expected at the 50-55% NBS assessed level of performance of the underlying building structures.

Figure 7-7: Stair 12 (Stair 3 as per building signage)



7.1.7 Stairs 13 and 14

Stairs 13 and 14 are circular spiral stairs that provide access from Level 2 to dressing rooms and choir assembly area on Level 3 at the northern end of the Auditorium. The stairs are reinforced concrete slabs, tied into the in-situ floor slabs at both levels. The flights contain top and bottom flexural reinforcing and transverse bars at close centres.

Based on the limited seismic displacements between floors in the Auditorium and a well reinforced concrete slab forming the stair, it is considered that any structural damage to this stair due to interstorey displacements will be minimal. Additionally, it is unlikely that this stair would be used as a major emergency egress route.

7.1.8 Stairs 15 and 16

Stairs 15 and 16 are enclosed stairs, exterior to the main building. These stairs provide access between the northern function rooms (Lion Harbour View Room) on Level 4, and back of stage areas on Levels 3 and 2.

The structural drawings indicate that the stair flight is fixed at each floor level, through a cast in-situ landing projecting from the main floor slab at each level. No provision exists to accommodate relative displacements between the three levels.

Assessment of the stairs to accommodate the effects of relative movement between the floor levels indicates that the ultimate limit state flexural capacity of the stair flights would be reached at about 15%NBS level of displacements. The stair flights can be expected to sustain displacements beyond this point but significant damage at each landing will occur at higher levels of earthquake loading which may hamper use of the stairs for egress following an earthquake. It is recommended that the stairs are structurally separated at each floor level, to allow for interstorey seismic displacements to occur without leading to damage to the stair flights.

Figure 7-8: Stairwell 16

7.1.9 Town Hall Air Bridge

An access air bridge is provided from Level 4 (upper promenade), across to the Town Hall on the eastern end of the Michael Fowler Centre. The air bridge is fixed, cantilevering from the Town Hall, with no physical connection at the Michael Fowler Centre side. The air bridge penetrates through the glazed façade of the Michael Fowler Centre and non-structural damage to glazing, mullions and transoms is likely as combined seismic displacements of both buildings will exceed the available clearances.

The air bridge is not considered as a primary emergency egress route from the Michael Fowler Centre. Any damage to the air bridge is considered likely to be confined to non-structural damage and will not affect the assessed structural performance of the Auditorium structure.

The proposed Town Hall base isolation strengthening scheme intends to increase the available clearances around the air bridge on the Michael Fowler Centre side, to accommodate the larger seismic displacements that would occur should the proposed strengthening works be implemented.

Figure 7-9: Town Hall Air Bridge

7.2 FOUNDATIONS

Tonkin & Taylor (T&T) have been engaged to undertake a desktop review of geotechnical conditions and liquefaction/lateral spread potential at the Michael Fowler Centre site [7]. A review of the effects on the building structural performance, due potential liquefaction and lateral spreading at the site under earthquake events, has been undertaken separate to the time history analysis.

A majority of the driven 'Franki' type piles, supporting the buildings are founded at a depth between 4 and 6 metres. The piles connect to foundation beams/pilecaps that are between 1 and 2 metres deep along with a series of basement tunnels beneath the Auditorium area.

Reclamation fill and loose beach sand deposits, across the Michael Fowler Centre site are assessed as susceptible to liquefaction at levels between 20 to 40 %NBS. The dense alluvium material, underlying these materials are the founding level for the driven piles and are not expected to liquefy.

Pile lengths indicate the majority of piles will have minimal embedment into the dense alluvium layer and these will be "pinned" but not fixed against rotation at the base. Any lateral spread displacements will induce deflections over the length of the piles passing through the overlying liquefiable material.

Liquefaction of the reclamation fill and beach sand could result in lateral spreading of the land toward the sea. For a 34 %NBS earthquake, it is considered that less than 200mm of lateral spread beneath the building can be expected.

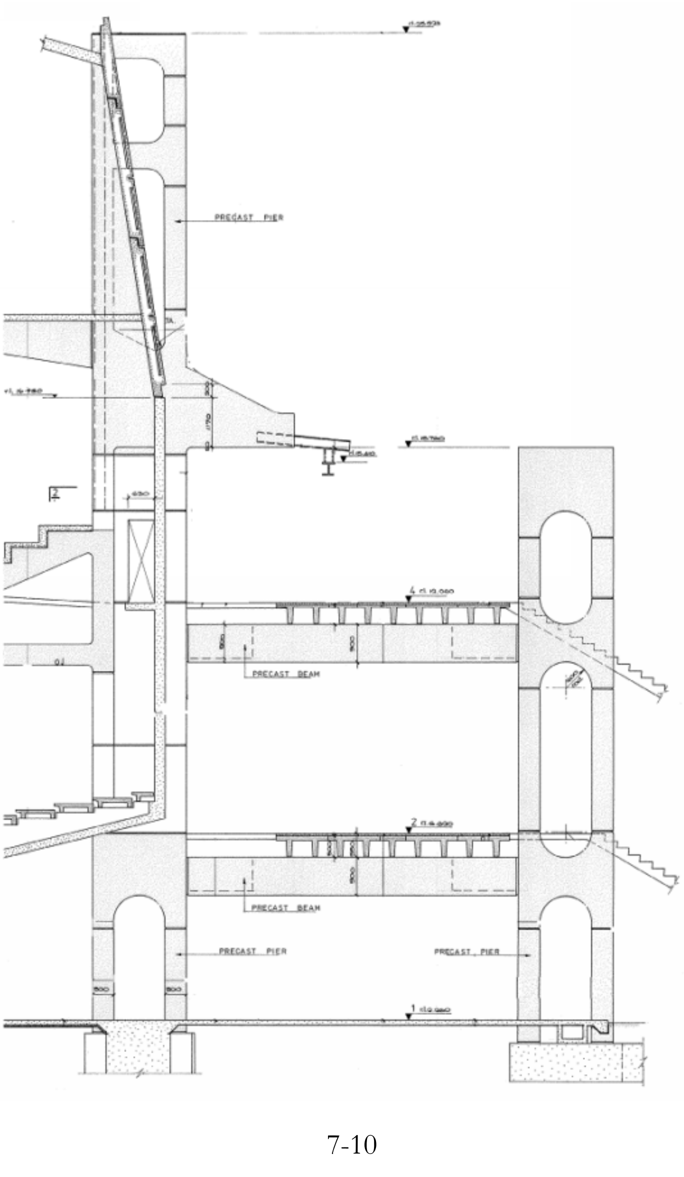
Checks on the lateral displacement capacities of typical piles indicate these are controlled by the flexural capacity of the section and could accommodate lateral displacements in the order of

200 mm at ultimate limit state. Beyond this point, loss of support from the piles may be possible, leading to the possibly large vertical displacements as the building attempts to re-distribute vertical loads onto the shallow pilecaps and foundation beams. As the upper soil layers are liquefiable, vertical settlements may be large and lead to secondary structural damage occurring through the building.

The main Auditorium walls are founded on a continuous concrete ring beam surrounding the building. Bays 6 and 7, 6a and 7a, either side of the main entry foyer to the south of the building, contain a group of columns with foundations shared with the Stair Block structures. These foundations are separate to the main Auditorium building and there exists a potential for differential displacements between the two foundation groups due to lateral spreading across the site – refer Figure 7-10.

Differential lateral displacements between the two foundations will lead to a column hinging mechanism forming in the lower storey. The columns have been assessed as being capable of sustaining differential lateral displacements between the foundations of up to 100mm at Ultimate Limit State, with some margin beyond that before the collapse limit state is reached.

Figure 7-10: Section through Bay 7



Assessment of the amount of differential lateral spread that may occur across the site is difficult. The effect of these differential movements on the foundations will also be dependent on the direction of lateral spread movements. Broadly estimating the differential lateral spread displacements across the site as being up to half the total lateral spread displacements, this would indicate the columns should perform at least up to the earthquake prone threshold limit of 34 %NBS.

Decreasing the lateral spread displacements, through installation of ground improvement measures and/or reducing the potential differential movements of the foundations, through the ground improvements installed or tying the foundation elements together, would appear to be the best means of improving the performance of this part of the building, if higher %NBS performance is sought.

7.3 AUDITORIUM UPPER WALL PANELS

The upper walls within the Auditorium (above approximately Level 4) were modelled in the time history as insitu wall panel elements. A later review of the drawings showed that these panels were constructed as a series of horizontal precast panels connected with cast insitu concrete along the vertical edges to the column frame members (external to the panels). Horizontal connections between each panel were via two cast-in weldplate details welded to a steel packer plate.

Testing of a sub-assembly model, comparing a cast in-situ wall and a slotted wall panel with no effective horizontal joint, showed that the global response of the two sub assembly models were essentially identical and that the performance of the structural model would be unchanged. Shear force transfer through the slotted wall panel model relied on shear force transferred through the boundary elements to the walls.

Shear strain and shear force demands in the upper wall elements are not considered high, even under higher levels of earthquake loading. The existing horizontal connections between the panels (plus mobilisation of any shear capacity in the boundary elements) is not considered to be a limiting factor on the panel performance at levels of earthquake loading less than that of the main Auditorium structure.

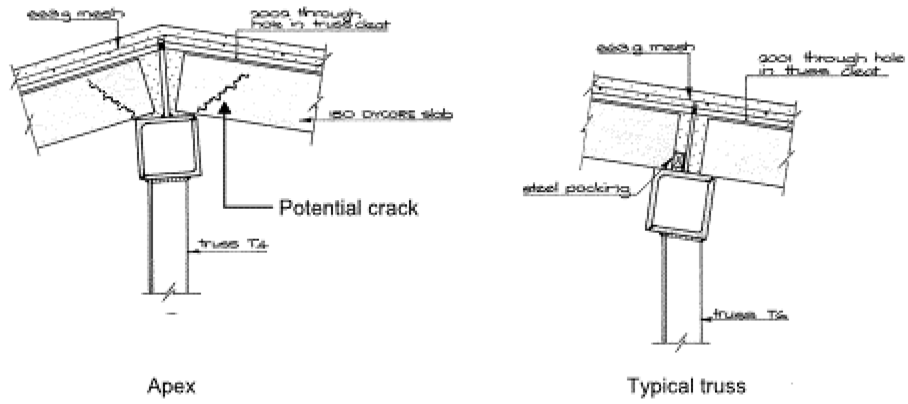
7.4 AUDITORIUM ROOF PANELS

The roof to the Auditorium is constructed from precast hollowcore units with a thin cast in-situ topping slab, tied into the walls around the exterior of the building. The precast hollowcore units are supported on the top chord of steel roof trusses spanning across the building. A flat steel plate cleat “shear connector”, at 1200mm centres (nominally one per precast hollowcore unit) is located between the joint between the precast units and filled with cast in-situ concrete to tie this to the topping concrete.

Typically the top chord to the roof truss is rotated to match the slope of the precast roof units and provide even seating ot the ends of the panels. Displacements at roof level, and typically through the whole building, are low due to the stiff perimeter wall structural arrangement. Section 6.2.3 discusses sensitivity of the building response to the strength assigned to the roof diaphragm and concluded that the roof does not perform a significant transfer function.

Observation of the roof panel connections to the roof truss top chords was attempted, but not visible due to insulation installed to the underside of the roof slab. We recommend that a section of the roof insulation is removed in order to enable observation of the connection of the roof panels, in particular to the truss along the apex of the roof.

Figure 7-11: Roof Panel support on roof trusses



At the roof apex, running east-west across the building, the precast roof units are installed at a sharp angle to the top chord member (refer Figure 7-11) and end support to the units will be concentrated over a small area. This detail does not appear as ideal, as there is some concern that this could lead to a crack developing in the precast hollowcore roof unit and greater risk of loss of support to the roof panels. Performance of this detail is difficult to assess and consideration should be given to installation of supplementary support details either side of the roof truss as an additional securing measure.

The buildings in the Michael Fowler Centre were designed and constructed over 30 years ago. Although seismic design procedures were deemed "modern" at that stage, there have been developments in the intervening years which have resulted in a better understanding of seismic performance, increased levels of design load and also more restrictive requirements for the detailing of reinforced concrete components – in particular for buildings of such complex form and geometry.

A quantitative evaluation reported here used the nonlinear time history analysis (NLTHA) procedure to assess the seismic performance of the building in terms of the current new building standard (%NBS).

The Michel Fowler Centre comprises structurally separate structures (Auditorium, Stair Blocks and Foyer) which were included in a single model but the evaluation was performed separately on the three buildings. This is because the structures have varying levels of seismic resistance and excessive displacements in any one building would terminate the analysis if all buildings were included. The physical connections between the buildings are not robust and so it is considered appropriate to model them separately in the as-is condition. If strengthening is to be implemented, it may be better to consider all buildings as a single unit.

Analysis results indicated that the buildings have seismic performance ratings in the range 50-60 %NBS. The following elements are critical to the overall performance of each building.

- The Auditorium has a relatively high elastic strength but is susceptible to sudden partial collapse of Southern portions of the structure for loads exceeding 90% of NZS1170 loads.
- The Entrance Foyer forms a soft story column hinge mechanism which leads to failure due to excessive column plastic rotations once significant yielding occurs.
- The walls of the Stair Blocks form plastic hinges out-of-plane and, as for the Foyer, failure occurs due to excessive plastic rotations.

Performance of local components within the building have been reviewed and performance levels assessed against that of the building(s) as a whole. This has shown that several of these items have seismic capacity less than this 50-60 %NBS rating, some of which fall below the 34 %NBS Earthquake Prone threshold. Specific component performance of note is summarised as follows;

- Stairs 7 and 8 (external Fire Egress to side of main Stair Blocks) – rigid connection between levels and across Stair Block to Auditorium structures and foundations. Susceptible to both inter-storey displacement, relative displacement between independent buildings and differential foundation movement (lateral spread). Remediation necessary – and currently considered Earthquake Prone.
- Stairs 9 and 10 (external Fire Egress from Renouf Foyer) – poor detailing around the top flight sliding connection at Foyer floor level. Whilst this independent stair might not be considered Earthquake Prone the detailing of the top flight connection warrants remediation.

- Stairs 15 and 16 (high level stairs at northern end of building connecting function rooms) – rigidly connected across three floor levels (two major stair flights). ULS capacity as low at 15 %NBS (ULS). As such, deemed Earthquake Prone and remediation is recommended.
- Auditorium structure adjacent Stair Blocks (Bays 6/7, 6a/7a) – have unconnected foundations and are prone to differential foundation movement (lateral spread). Ground floor column remediation recommended. Capacity is subject to degree of differential lateral ground movement (can tolerate up to 100mm lateral differential movement). Assuming "expected" lateral displacement (as reported by T&T) and 50% differential displacement, capacity of these two towers will be greater than 34% NBS. However, strengthening is recommended.
- Auditorium Roof – hollowcore units are supported on steel trusses with minimal seating. Building finishes (soffit insulation and top surface waterproofing) limit the access for inspection. Our assessment concludes the roof capacity is not less than the overall Auditorium structure. However, additional inspection of hollowcore unit soffit is recommended, as is some supplementary hollowcore support, in particular along the main roof ridge line.

Geotechnical assessment indicates liquefaction and lateral spread conditions exist for the near surface reclamation fills on this site, under moderate levels of earthquake ground shaking, increasing in severity as earthquake accelerations increase. Building pile foundations extend down through the reclamation to the underlying alluviums and frame into relatively substantial ground beams or pile caps. However these foundation beams do not tie across separate buildings (e.g. no foundation beams connect the Auditorium to Stair Towers, nor Stair Towers to Foyer, nor across Stair Towers).

Assessment of piles indicates lateral displacement capacity in the order of 200mm and even beyond those displacements, contribution of the foundation beams will help limit gross building displacements to some extent. Based on T&T's "expected" lateral displacements being less than 200mm at 34 %NBS seismic load levels, we do not expect the ground conditions to render the building earthquake prone. However, at higher load levels, and if building "strengthening" is proposed, this will need to carefully consider the effects of lateral spread with the likely need to provide ground improvement measures to reduce likely lateral spread displacements. We have noted that any such works may provide best "value" if undertaken on the lagoon side of Jervois Quay, in order to enhance resilience of both the main Jervois Quay roadway and City to Sea Bridge.

[1]. Holmes Consulting Group, *Michael Fowler Centre, Qualitative Seismic Review*, Report 107303, November 2011.

[2]. Standards New Zealand, (SNZ), 2005, *NZS 1170.5 Standard Structural Design Actions – Part 5 Earthquake Actions,* Wellington, New Zealand.

[3]. *Concrete Structures Standard: Part 1 - The Design of Concrete Structures*, NZS 3101:Part 1:1995, Standards New Zealand, 1992.

[4]. American Society of Civil Engineers, *Seismic Rehabilitation of Existing Buildings,* ASCE41-06, ASCE, 2006, including Supplement 1, 2007.

[5]. Mondkar, D.P. and Powell, G.H., 1979, *ANSR II Analysis of Non-linear Structural Response User's Manual*, EERC 79/17, University of California, Berkeley, July.

[6]. Holmes Consulting Group, *Performance Based Evaluation of Existing Buildings: Nonlinear Pushover and Time History Analysis: Reference Manual.* Revision 5, November, 2000 available on www.holmesgroup.com

[7]. Tonkin & Taylor, *Michael Fowler, 111 Wakefield St, Wellington, Geotechnical Desktop Assessment.* T&T Ref: 85841, August 2014