

## STRENGTHENING OF THE WELLINGTON TOWN HALL

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### SYNOPSIS

*The 'old' Wellington Town Hall has been refurbished and strengthened for earthquakes as part of the Wellington Civic Centre Development. The design outlined made maximum use of the existing brickwalls as shear walls and ensured that a load path was provided between all parts of the building and these. Careful sequencing of the demolition and construction works minimised the extent of temporary support works. In this manner a cost effective solution was achieved which enabled the entire project to be completed within a twelve month period in time for the 1992 International Arts Festival.*

*This paper describes aspects of the design and construction of the project.*

### 1. INTRODUCTION

In 1989 the decision was made to refurbish and strengthen the 'old' Wellington Town Hall as part of this Wellington Civic Centre Development.

The structural strengthening scheme developed by Beca Carter Hollings and Ferner Ltd and outlined in this paper was accepted by the Wellington City Council as an alternative to the originally tendered scheme and as part of a design and build contract put together by Mainzeal Construction Ltd.

Construction commenced in February 1991 and was successfully completed in February 1992 in time for the International Arts Festival.

### 2. HISTORY OF THE BUILDING

The building was constructed between 1902 and 1904 on a site which had been reclaimed approximately sixteen years earlier. The original building was heavily adorned with parapet balustrades, pinnacles and pediments and had a dock tower on the eastern facade [1] which extended to a height of 46 m above pavement level.

Following the 1931 Hawkes Bay earthquake, hazards resulting from unreinforced masonry and in particular inadequately tied back features such as ornate parapets, cornices etc received great attention and in 1934 the building underwent substantial alteration.

This included the removal of the clock tower, down to the building roof level, the removal of the main entrance portico, the cutting back of the main perimeter cornice and the removal of all decorative pediments, parapet balustrades and pinnacles.

During the 1941 Wairarapa earthquakes the building suffered some cracking damage particularly in the upper regions of the masonry walls and in the remaining parapets [1].

Between 1943 and 1945 substantial seismic strengthening in the form of reinforced concrete buttressing of the auditorium walls and concrete banding of the auditorium and concert chamber walls was carried out. The banding was placed on both sides of the walls and on the interior faces replaced brick columns and the cornice bands near the top of the walls.

The construction drawings of these strengthening works indicate that it was originally intended to extend the wall banding through to the foundations, however, for some reason it was only completed to first floor level.

The area of building containing the Council Chamber and Mayoral offices (south east corner) remained unstrengthened.

The 1970's were a time of controversy for the building as plans including demolition were debated [2].

In the late 1980's it was decided that the building should be retained and be refurbished and strengthened as part of the Civic Centre Development

### 3. STRUCTURE OF THE ORIGINAL BUILDING

Construction of the original building was load bearing brick laid in cement mortar. The walls around the main auditorium are approximately 680 mm thick and elsewhere are typically 450 mm thick. In-situ brick shear tests carried out in accordance with the provisions of reference 3 by Works Central Laboratories in 1990 gave failure stress results for the brick/mortar joint ranging from 2.6 to 4.1 Mpa. Bed joint shear tests, also carried out by Central Laboratories gave failure stress results of between 0.6 and

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2.5 MPa. These test results indicated that the brick walls were typically well constructed and in good condition.

The walls stand on an unreinforced concrete strip footing which is itself supported on approximately 730 unreinforced concrete cast in-situ (inferred from investigations carried out during strengthening) piles nominally 350 mm in diameter.

With the exception of the auditorium floor and gallery the floors throughout the building were of unreinforced lightweight "breeze" concrete approximately 350 mm thick supported on steel beams cast with in the breeze concrete and spanning between the brick walls. The auditorium floor and gallery are of timber construction supported on concrete strip footings.

The roof structure across the major spaces comprises roof trusses, at approximately 3 m centres, constructed from massive oregon timbers which are jointed by steel plates and internal keys. The bottom chord members of the trusses which span across the auditorium are 690 deep by 200 wide.

The roofing material is profiled galvanised steel.

The storey heights are 4.25 m on the ground floor and 4.0 m on the first and second floors. The auditorium space extends to a height of 13.7 m.

#### 4. PLANNING CHANGES

The architectural scheme for the refurbishment required the following changes to the building:

1. The removal of the miscellaneous structures along the western side of the building and the construction of a two level ambulatory walkway structure (Figure 2).
2. The relocation of the concert chamber originally in the north east corner of the building to the south east basement area. The two storey height space for the new concert chamber was created by removing the ground floor slab in this area and spanning the new first floor slab across the chamber, a distance of 14 m. (Figure 2).
3. Turning the vacated concert chamber space into seminar rooms and mayoral offices by the addition of a new second floor (Figure 4).
4. Filling in of the light wells to provide additional floor area.
5. Excavation of additional basement area for a kitchen and access way to the new concert chamber (Figure 1).
6. Provision for three new plant room floors, two adjacent and to the north of the main entrance way and the third at the south end of the auditorium (Figure 5).
7. Provision of a new roof over the main entrance way and a new floor around the lantern light to the main entrance way at second floor level (Figure 4).
8. Extension of the south stairs down to the basement and up to the second floor level.
9. Construction of an access tunnel from the north wall of the auditorium to a lift at the front of the auditorium stage. The lift pit extends approximately 2.8 m below sea level while the tunnel floor is approximately 1 m below sea level (Figure 1).
10. The closing off of numerous original openings and the provision of new openings in the brickwork.

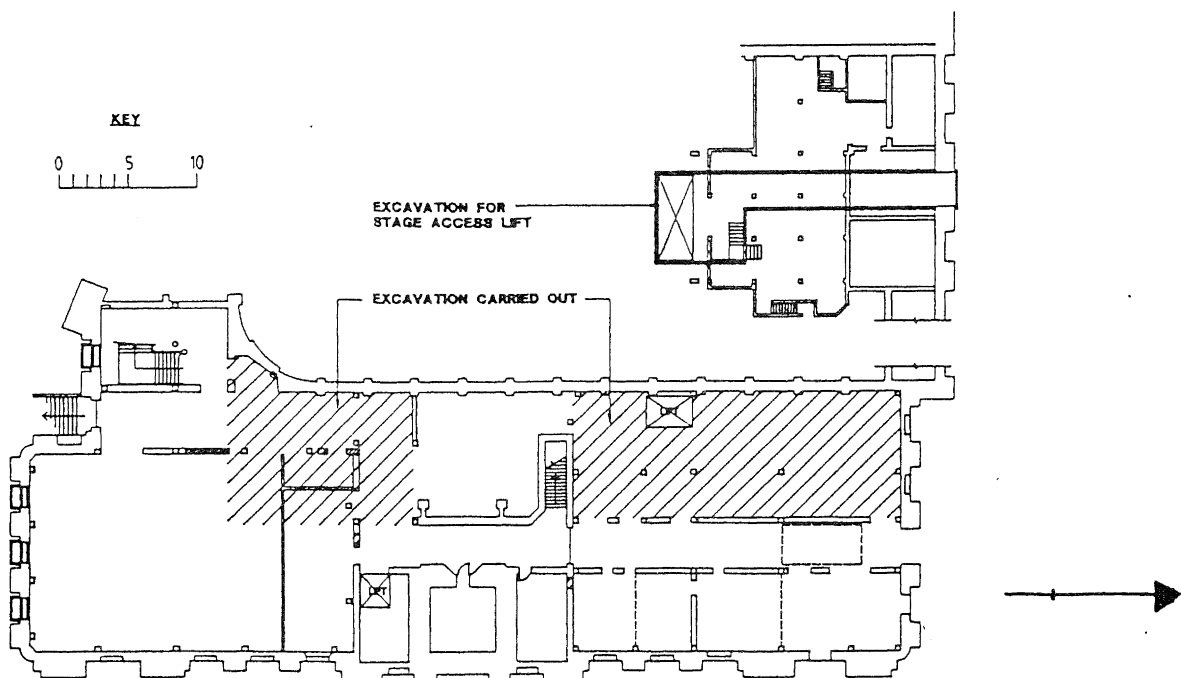


Figure 1 Basement plan

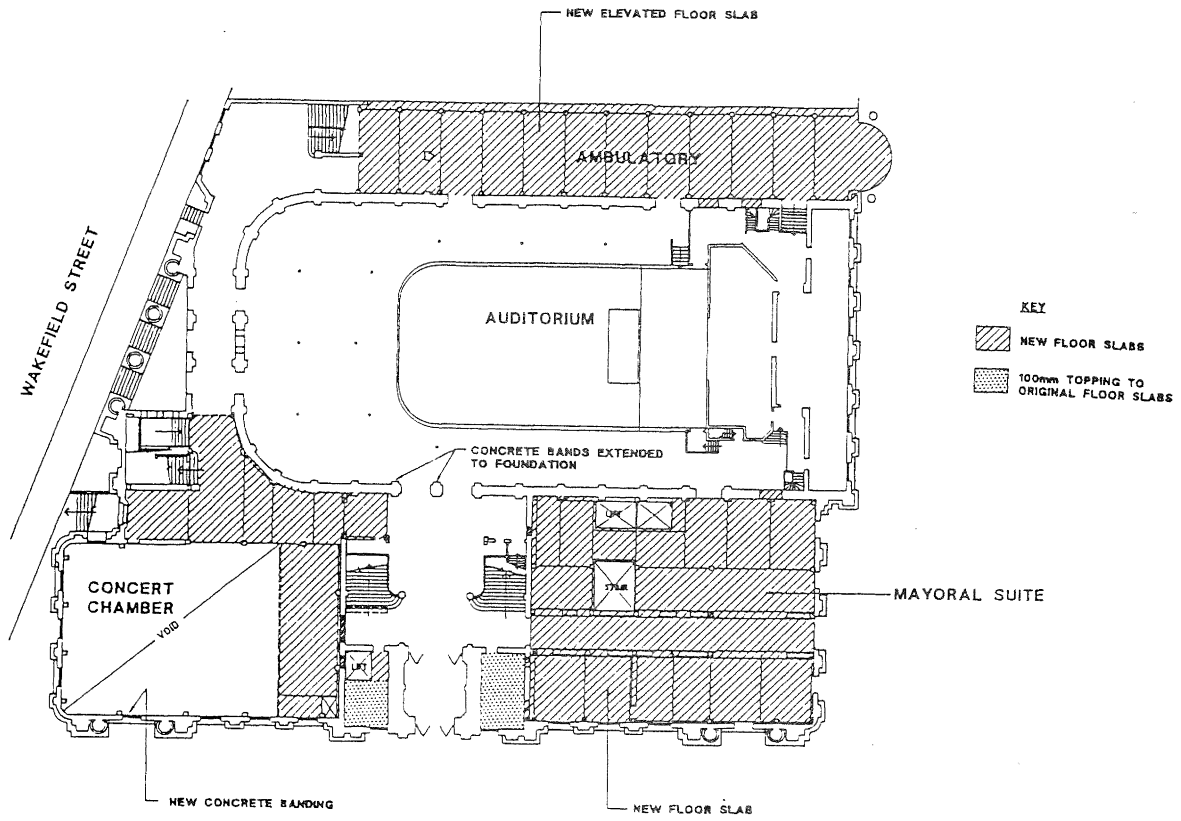


Figure 2 Ground floor plan

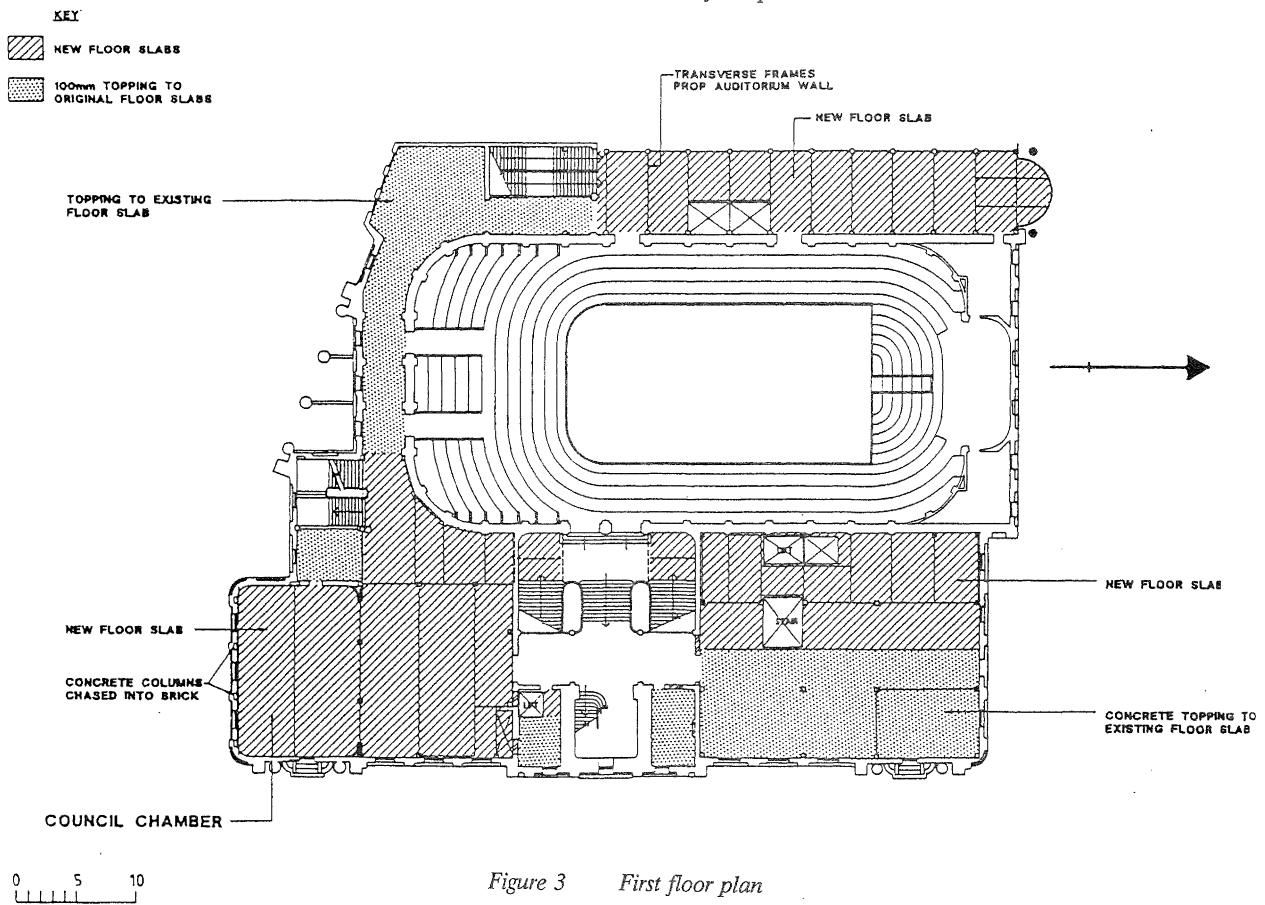


Figure 3 First floor plan

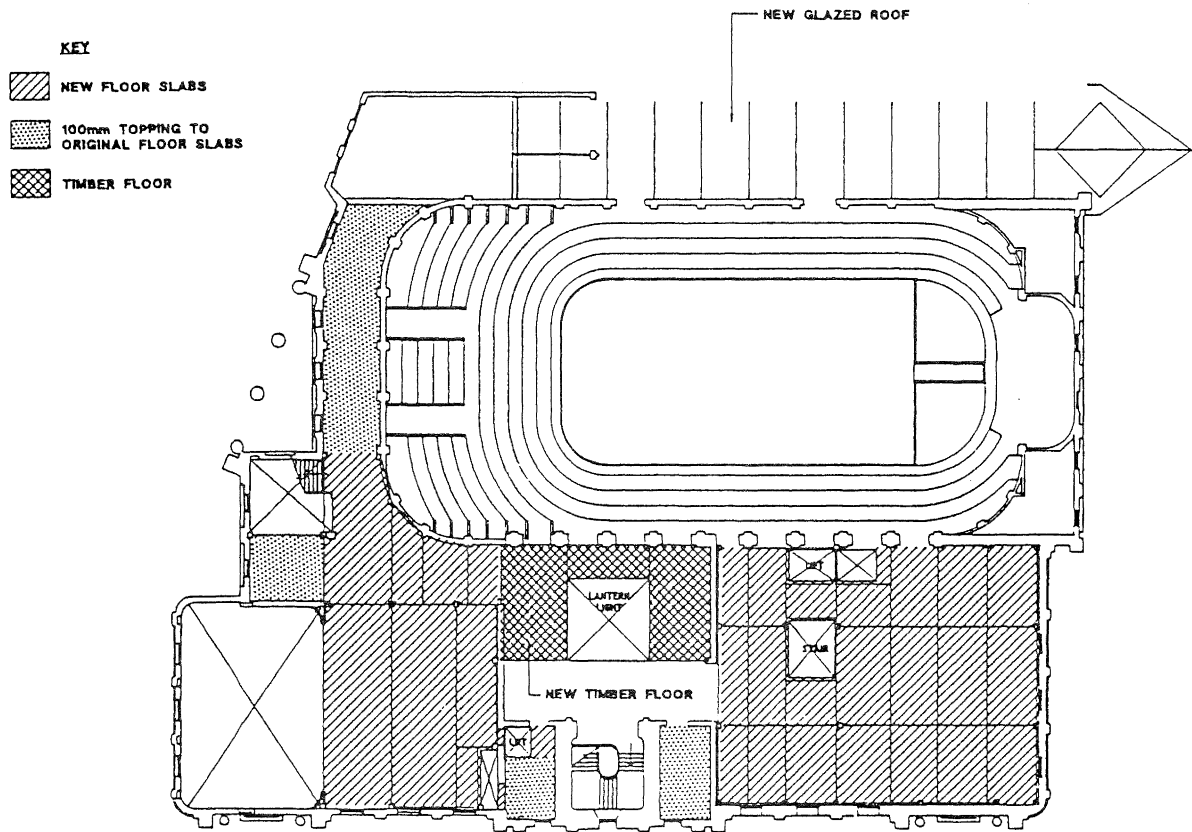


Figure 4 Second floor plan

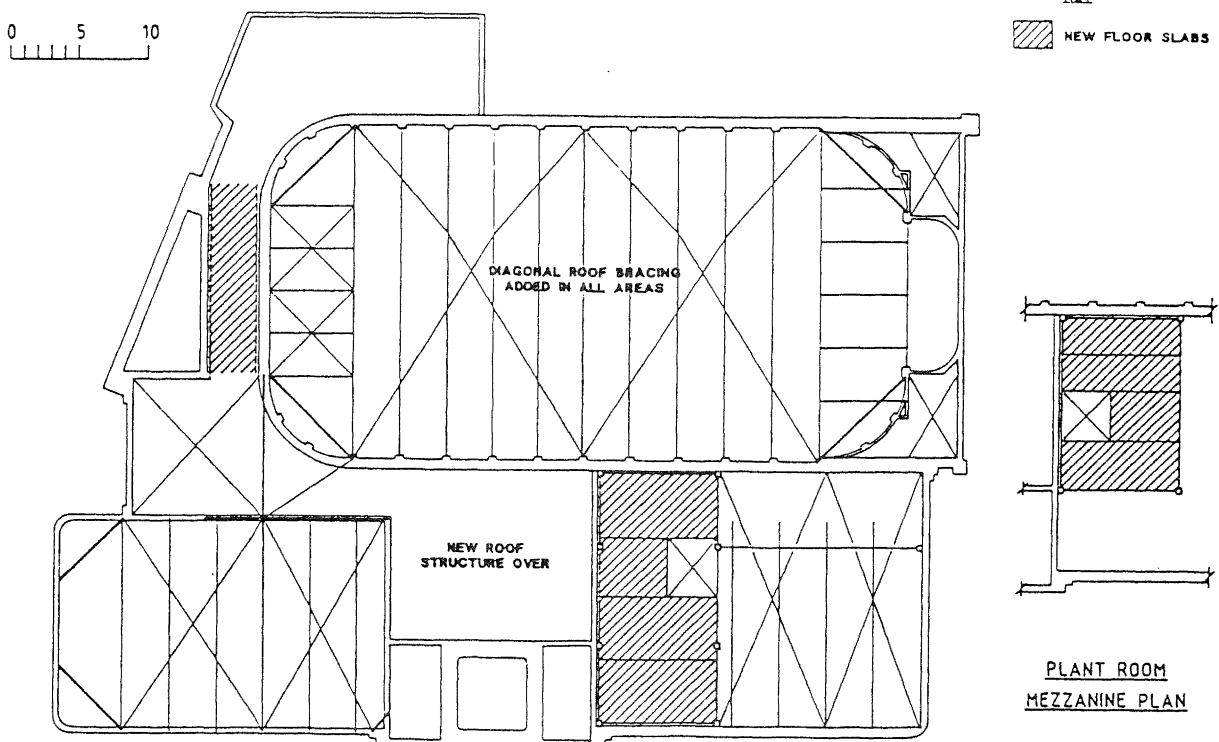


Figure 5 Roof plan and plant room levels

## 5. STRENGTHENING PHILOSOPHY

### 5.1 Structural Concept

The main features of the strengthening concept are:

1. The original substantial masonry walls are utilised as shearwalls.
2. A reinforced concrete frame is provided to carry gravity floor and roof loads.
3. New reinforced concrete floor diaphragms tie the walls together and provide the ability to distribute seismic floor loads to the walls.
4. New steel bracing is provided at roof level to stabilise the tops of the walls and distribute seismic roof loads.
5. A new, fully ductile, concrete frame provided along the west side of the building which stabilises the west wall of the auditorium under face loading.

### 5.2 Design Standard

The building has been strengthened to be capable of resisting earthquake forces at least two thirds those specified in NZS 1900, Chapter 8 [3] for public buildings. This is the minimum standard for strengthening of earthquake risk buildings currently acceptable to the Wellington City Council.

This design standard leads to design forces much less than required by modern codes. However, the actual strength of the refurbished building is expected to exceed this minimum level by a considerable margin.

### 5.3 Foundations

The existing foundations comprise unreinforced concrete strip footings supported on cast-in-situ unreinforced concrete piles. It is expected that the piles are bearing in the underlying alluvium although this has not been fully confirmed.

During design it was necessary to consider two aspects of the foundation performance. They were:

1. The likely performance under additional gravity loading imposed by the rearrangement of structural support, and
2. The likely performance during seismic shaking.

The loads imposed on the piles/foundation beams by the refurbished structure are generally less than the calculated loads in the original configuration. However, in some local areas the refurbished building is expected to impose design loadings up to 20% greater than the most heavily loaded sections of the foundation in the original building, or up to 300 kN/pile.

It was impracticable at the time of design to confirm the founding depth of the piles and therefore their likely load capacity. The additional pile loading was therefore justified on the following basis:

1. There was no evidence of differential settlement in the structure.
2. The fill materials are likely to be less stiff than the pile foundations and are therefore likely to be carrying the majority of the foundation load.
3. The past performance of the building/foundation was considered to represent a "full scale" foundation test loading (the majority of the gravity load results from dead load) and in the absence of any evidence of deflection the additional load, locally in some piles, was considered acceptable.

During construction, the opportunity arose to load test several piles by cutting off the top of the piles and jacking against the underside of the footing. These tests indicated that the capacities and stiffnesses of the piles vary but that typical ultimate pile capacities are in the order of 350 - 450 kN.

There have been no signs of settlement since the refurbishment works have been completed.

The piles are unreinforced. In a severe seismic event, shear failure in the piles adjacent to the foundation footing could be expected. In the event of such failure, it has been assumed that the support for the building would then be provided by the foundation acting as a spread footing albeit with some settlement estimated to be in the order of 50 mm. The building would be likely to suffer some damage due to foundation movements of this order but the risk of collapse was considered to be low. The cost of fully upgrading the foundation was prohibitive and the above philosophy was accepted as reasonable by the Wellington City Council.

### 5.4 Method of Analysis

The following assumptions were made:

1. Each wall would resist the inertial forces resulting from its own mass.
2. The floor and roof inertial forces would be distributed to each wall in accordance with the contributing tributary areas.
3. The accidental eccentricity for floor and roof inertial forces was 5% of the appropriate tributary area dimensions.
4. The seismic forces would be distributed over the height of each wall in accordance with NZS1900, Chapter 8.

A rigorous torsional rigidity analysis was considered inappropriate for the building for the following reasons:

1. In excess of 80% of the mass of the building is in the walls themselves and, notwithstanding that slab diaphragms were provided, it was considered inappropriate to redistribute in-plane inertial forces resulting from a wall's self weight to other walls.
2. The building is effectively divided in two by the auditorium and any redistribution of forces from the east to the west side of the building would be influenced by the stiffness of the interconnecting diaphragms.

3. The building is very stiff.
4. The actual stiffnesses of the heavily penetrated walls could not be estimated to a significant accuracy to justify a more detailed analysis than that outlined above.

Using the above method, the walls were found to have a lateral capacity typically 50 to 75% greater than required by the design standard described in section 5.2.

The stability of the unreinforced brickwalls spanning vertically between the lines of restraint provided by the diaphragms was checked using the method proposed by Priestley [4]. This method indicated that all walls could be expected to remain stable under an inertial load considerably in excess of the requirements of the design standard (ie 0.269).

### 5.5 Stresses In Brickwork

The design ultimate capacity of the unreinforced brickwalls was derived in accordance with Figure A.3 of reference 5 and was typically in the range 300 - 315 kPa.

The design capacity compares with failure stresses obtained from on site tests of 2.6 to >4 MPa and 0.9 to 2 MPa for in plane mortar shear tests and bed joint shear tests respectively, carried out in accordance with Appendix A of reference 5.

It is apparent that the actual first cycle shear strength of the building is likely to be much greater than that assumed for design purposes.

Consideration of the 'piers' between windows indicated that these were likely to rock prior to exhibiting a shear failure and therefore satisfied the criterion for acceptable performance given in reference 4.

## 6. STRUCTURAL ASPECTS

### 6.1 Temporary Support

By careful sequencing of the removal of floors and internal walls and the reconstruction works, a minimum lateral capacity of 0.19 was maintained at all times without the need to provide temporary falsework.

The wall banding provided during the 1940's strengthening works greatly improved the horizontal spanning action of the otherwise unreinforced brick wall.

### 6.2 Banding of the Brickwalls

Extensive banding of the auditorium and north east areas of the building were completed during the 1940's strengthening works. The banding comprises reinforced concrete sections typically 400 x 150 on each side of the brickwall which are connected together by bars passing through the brickwork. The bands run both horizontally and vertically and form a 'frame' in the place of the brickwalls.

As part of the latest strengthening works, these bands were extended down to foundation level (the 1940's work stopped at first floor level) and the horizontal bands were tied into the new floor slabs or new toppings to the retained slabs.

On the east facade, several of the large circular columns are of reinforced concrete tied through the walls to concrete bands on the inner face.

The 'frame' created by the banding provides a means of vertical support to the floor slabs as well as additional stability to the brickwalls. The risk of collapse of the floor slabs or of the roof structure in the event of localised failure of the brickwalls is therefore greatly reduced.

### 6.3 Roof Bracing

The roof bracing consists of steel flats which pass between and are attached to the bottom chords of the timber trusses. In this manner the truss chords act as an integral part of the roof bracing system.

Above the auditorium space the connections between the roof trusses and the brickwalls had been improved during previous strengthening works. These were further improved as part of the latest refurbishment works by passing tie bolts through the brick wall and anchoring them in the concrete band behind.

### 6.4 Mayoral Suite Area

With the exception of a section of floor slab on the first floor level (retained to provide temporary support to the facade) all of the original floors were removed and replaced with new concrete floor slabs. A concrete topping was placed over the retained section of floor to complete the diaphragm.

A new floor was constructed within the original concert chamber space and two new floors were constructed at high level immediately to the north of the main entrance to accommodate the mechanical plant.

Also in the Mayoral Suite area a section of original roof structure 915 m x 14 m had to be temporarily supported 17 m above ground level after one supporting wall had been removed to enable construction of the new floor slabs below.

This was achieved by casting a concrete beam in the roof space using the trusses to support the formwork while the roof was still supported on the wall, bringing the supporting concrete columns up through the structure to each end of the beam and then removing the wall.

### 6.5 Concert and Council Chambers

A new concert chamber was constructed in the basement in the south eastern corner of the building. A double height space was created by removing the first floor. Reinforced concrete beams span (14 m) across the space to support the council chamber floor above.

The council chamber has been left essentially unaltered. New concrete 'columns' to provide independent support to the roof were chased into the brickwork and are undetectable in the completed structure.

### 6.6 Tunnel and Stage Lift

Construction of the stage lift pit and connecting tunnel required excavation to 2.8 m below ground water level. This was achieved by draining down the water level locally using pumps and using a standard shoring system of driven steel

piles and timber lagging. In the completed structure, the steel piles provide restraint against the uplift forces.

The tunnel was constructed under the pipe organ and extends under the north wall of the auditorium to the adjacent Wellington City Council garage.

#### **6.7 Remedial Works to Foundations to the Auditorium North Wall**

During removal of sheet piling on an adjacent site, it was noted that in a localised area several piles under the north wall had separated from the foundation beam and cracks had appeared in the walls above due to foundation rotation.

Extensive investigations of these piles were carried out including:

1. Load testing by jacking against the weight of the structure above to determine pile capacity, and
2. Sonic testing in an attempt to determine pile lengths.

Following investigation, the piles were jacked down until the required resistance had been achieved and then the connection between the pile top and foundation beam was reinstated.

#### **7. IN CONCLUSION**

The structural scheme outlined above provided a cost effective means of strengthening the building. A standard of strengthening was achieved which, although not meeting the requirements of current codes for new buildings, should provide an acceptable level of occupant safety in the event of a major earthquake.

The scheme adopted met conservation aims and enabled the contractor to meet a very tight construction programme.

#### **REFERENCES**

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