THE STRENGTHENING OF AUCKLAND TOWN HALL

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SUMMARY

Auckland Town Hall, one of New Zealand's premier heritage buildings, was constructed around 1911 to provide Auckland with a world class concert venue and Civic Centre. Constructed of unreinforced masonry the building did not meet current seismic protection standards, particularly as a place of assembly. The owner of the building, Auckland City Council, determined that the building should be strengthened as part of an overall restoration programme. This paper describes the standards of strengthening adopted, the analysis and the strengthening systems utilised.

INTRODUCTION

Auckland Town Hall is located at the junction of Queen Street and the former Greys Avenue, adjacent to Aotea Square and shares an 'entertainment precinct' with the Aotea Centre and the historic Civic Theatre. Built in 1911, the building provides an important civic focus to the city. The main auditorium of the Town Hall was required to serve as a multi-function hall, which placed limitations on its effectiveness for any single activity, but, notwithstanding this, the hall had a reputation for its excellent acoustics for orchestral activities. As well as providing Auckland with a performance hall, the building has, until recently, housed the seat of local government, Mayor's office, council offices and the Council Chamber. This latter activity remains an important part of the building's function.

Built of unreinforced masonry, the building did not meet with current seismic strength requirements, particularly when its function as a place of assembly is considered.

Figure 1: View of the Town Hall from the West.

Further, the building's exterior had been deteriorating at increasing rates and much of the interior was tired, obsolete and no longer adequately met with the requirements of its intended function. The Council therefore resolved that the building should be upgraded and made suitable to serve the community into the 21st century. This upgrade was to include structural strengthening, architectural retrofitting, modern building services and fire protection, heritage conservation and halting the external deterioration.

In 1994 a team of consultants was appointed to design the restoration works and to supervise construction. A construction contract was let in late 1995 for commencement on site in January 1996 with November 1997 as the

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targeted completion date. This paper sets out the structural investigations, analysis and design for the seismic strengthening of the Auckland Town Hall.

BUILDING DESCRIPTION

The West Elevation of the building is shown in Figure 1. The general form can be ascertained from Figure 2 which shows the floor plan at the Queen Street level. The building is wedge shaped in plan and divided into four main zones. These are identified as: the Great Hall, the Concert Chamber, the Apex (which houses the office area and Council Chamber) and the Prow, which includes and supports the clock tower at the north end of the building.

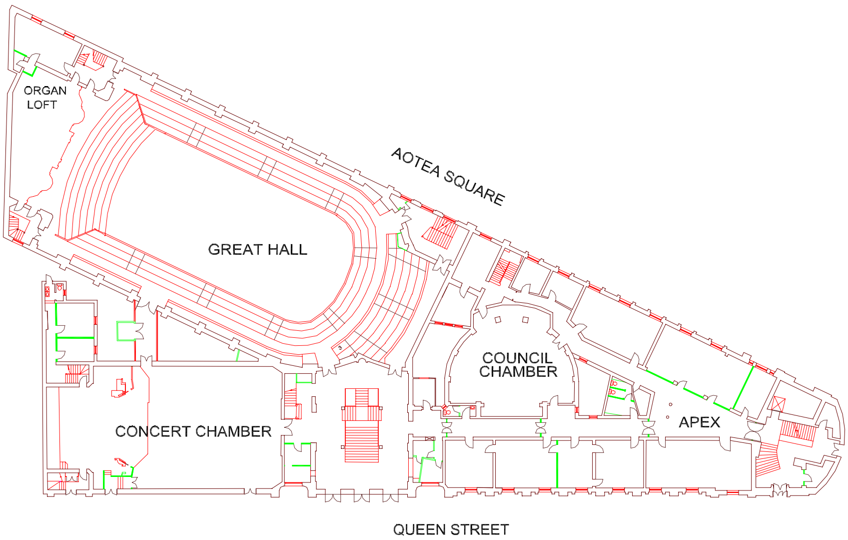


Figure 2: Original Level G Floor Plan. (one level up)

The original site had a significant cross fall and consequently the Queen Street frontage is one floor higher than the Greys Avenue (Aotea Square) frontage.

Figure 3 provides a cross section through the Concert Chamber and Great Hall in its original form and indicates that the building is generally (equivalent to) four stories high.

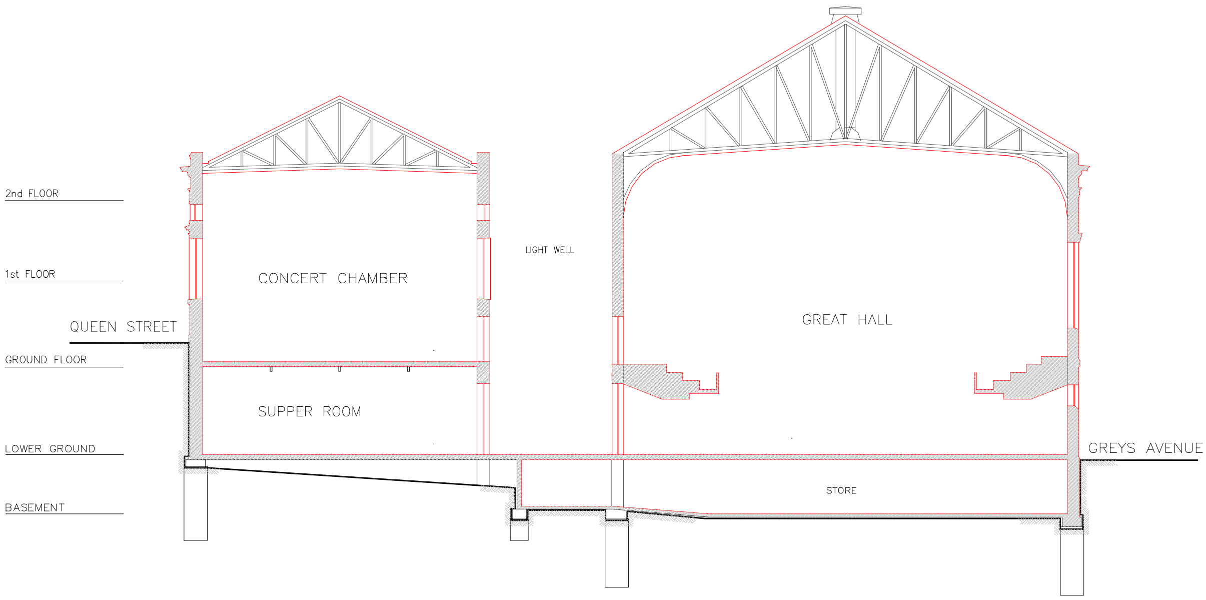


Figure 3: Cross Section Through Great Hall and Concert Chamber

The Great Hall, however, is a single volume space over all four levels, but also with a partial basement to the south. The Concert Chamber is significantly smaller with its auditorium floor one storey up, level with Queen Street. The Apex is well compartmented by internal walls but the cross walls have a slender aspect ratio due to the presence of two longitudinal corridors on each level.

The building is generally constructed of unreinforced masonry. The exposed external walls to the east, north and west are Oamaru Stone backed internally with brickwork, except at the lowest level where they are backed by unreinforced concrete. The south external wall is brickwork. Internal walls are generally brickwork, though there are a few concrete walls associated with previous strong rooms and for the lowest level of the tower. The Queen Street retaining wall is concrete reinforced with steel lath.

All internal floors are timber, with simple tongue & groove flooring on joists which are simply socketed into the supporting walls.

The building is founded on mass concrete piers that extend down to the Waitamata series rock, approximately 5 to 6 metres below Lower Ground level. Consequently the building has not suffered significant settlement over its lifetime.

Over the past 87 years the building has suffered little alteration except in the Apex area where isolated walls had been deleted at various times to create bigger rooms and two rather incongruous mansard rooms were added at roof level.

ARCHITECTURAL MODIFICATIONS

There were a number of significant alterations included within the current restoration, particularly within the Apex area. Here the mansard additions were removed and the original roofline largely restored. However, the original top level was modified to include a large reception venue, together with new lifts and egress stairs. At lower levels a further number of walls were removed to create more functional spaces and the opportunity was taken to install some new walls and replace some previously removed.

The Great Hall was architecturally largely unaltered other than restoration to the original decor, removal of the obtrusive 1950s acoustic reflector and installation of new acoustic sidewalls at the lowest level. Back stage facilities were substantially modified and improved, as were public toilet facilities.

The Concert Chamber had previous modifications to the stage area reversed and egress provisions improved but otherwise underwent only conservation restoration.

Between the two halls, a previously unused wedge shaped lightwell was incorporated into the public circulation areas serving both halls, which has greatly improved patron comfort during interval periods. At Lower Ground (Aotea Square) level a previous truck dock and plant room were also incorporated into the public circulation spaces to provide a dramatic new public entrance and bar area.

The basement was partially taken over for plant space and a new services corridor tunnelled under the Great Hall extending from the basement to the Apex area.

EARLIER STUDIES AND STRENGTHENING

The Council had carried out a number of earlier studies from which it was confirmed that the building was indeed a significant earthquake risk and that strengthening would be required. Broad-brush cost comparisons were made of various alternative approaches from which the most significant conclusion was that the building did not suit a base isolation concept, which additionally would have cost substantially more than a more conventional strengthening approach. This conclusion was significantly influenced by the difference in street level on the two principal sides.

In 1992 consultants were engaged for a preliminary investigation [1] into the seismic strength of the existing building and the feasibility of strengthening for life safety and reasonable repairability after a Maximum Credible Earthquake (MCE) event. This study concentrated primarily on the Great Hall and the Prow and Tower. It was assumed in that study that the Concert Chamber would follow similar, but lesser behaviour to the Great Hall and that the Apex was well compartmented and hence did not present any peculiar strengthening problems. The study concluded that generally the Great Hall had sufficient shear capacity to meet the requirements of the brief providing face loading deficiencies of the external walls were attended to by means of bracing at Circle and Roof levels.

In 1993 a separate consultant was engaged to assess and report on the site seismicity [2]. This study provided site specific spectra for a Design Basis Earthquake (DBE) and MCE. These events were respectively lesser and greater than the spectrum determined from NZS 4203[4]. These spectra are shown in Figure 4.

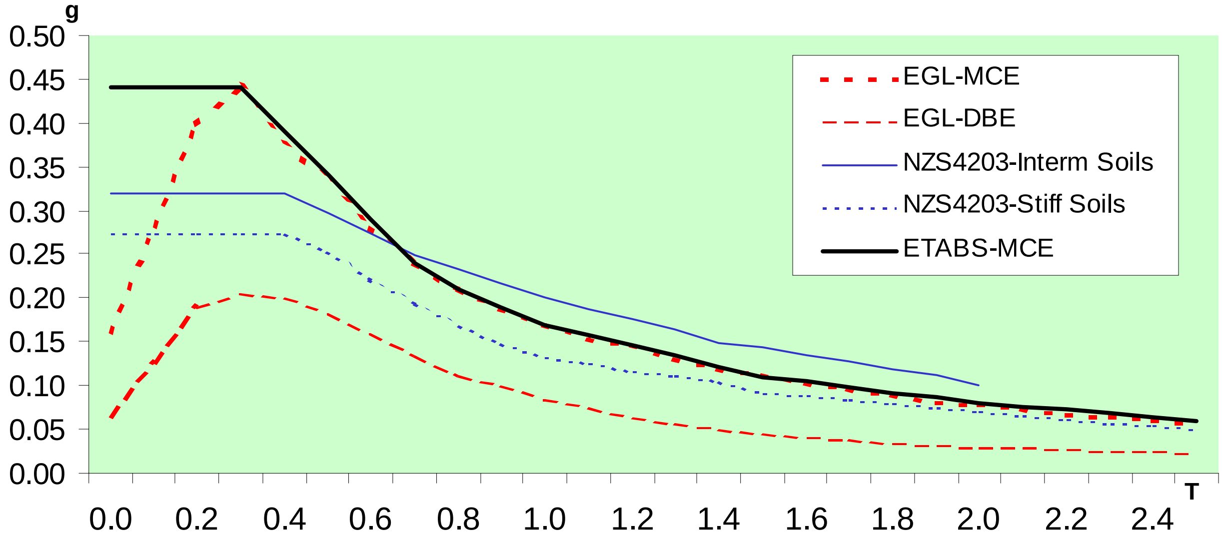


Figure 4: Project Response Spectra

An important conclusion of the evaluation study was that the Tower was grossly deficient in lateral strength and that strengthening of this element should be carried out as a matter of some urgency. Design work on a tower strengthening system proceeded and in 1993 a contract was let to Mainzeal Construction Limited to carry out this and other refurbishment work on the Tower. As at that time there was some uncertainty as to the architectural requirements of the future design of the Apex area, particularly with respect to lift requirements, the Tower strengthening was installed only down to general roof level other than for some tension straps within the Prow stair that were installed full height.

STRUCTURAL DESIGN PHILOSOPHY

The toughest 'call' of the client requirements was that of making maximum use of the existing structure in order to minimise the cost of strengthening works. Strengthening by concrete encapsulation is intrinsically easier. This approach required in-depth analysis of a structure that does not lend itself to analytical form.

Seismic strengthening targeted life safety as a priority over property protection, though the latter will have been achieved as a consequence of the former. The basic premise for the seismic resistance was to remove or correct deficiencies in the original structure in order to mobilise the inherent toughness of the massive walls of the existing building. Where this was not achievable, loads were transferred to new or enhanced lateral load resisting elements by means of diaphragms or trusses. A consequence of this approach was an acceptance that property damage may occur during a major seismic event, primarily in the form of cracking in the masonry walls but to an extent that will leave the building repairable.

Because life safety was the priority concern, and as lateral resistance is still dependant to some extent on brickwork that could behave in a brittle manner in the event of overload, analysis was carried out based on the MCE for the site. This event is significantly greater than the design spectrum for an elastic response required by the Loadings Code NZS 4203 for new construction for short period structures.

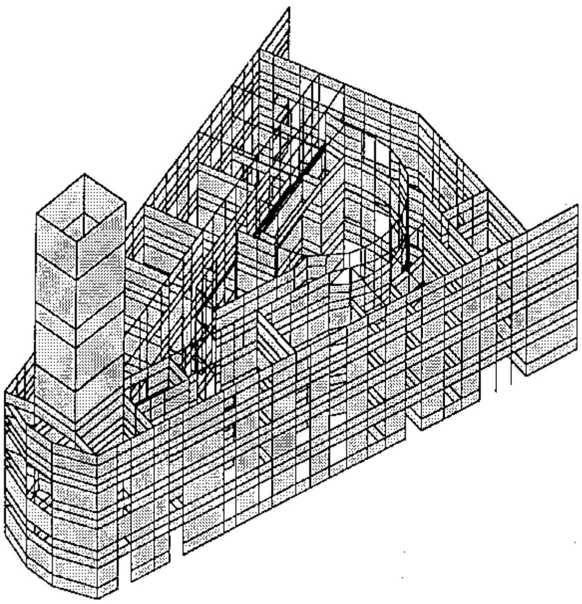
Dynamic analysis of the structure, in response to the MCE event, was carried out elastically. The stresses so determined were compared with defined failure criteria and the consequence of resultant cracking was assessed by spreadsheet methods. Where the extent of potential cracking was greater than identified limits then localised strengthening has been provided.
The flexibility of the timber diaphragms was included within the dynamic analysis, which was particularly important in order to model the building's earthquake response realistically. While the effects of overstress may be redistributed within any one common wall, the flexibility of the diaphragms means that there is little ability

for the diaphragm to redistribute loads to distant walls and also that the structure has little ability to redistribute the effects of torsion brought about by major differences in mass location from stiffness centre. This was particularly relevant with respect to the triangular shape of the building and the effect of the clock tower.

ANALYSIS

The principal means of dynamic analysis was by use of ETABS 6.0, the then most recent release of ETABS. Version 6.0 had a number of significant new features over previous versions that made it particularly suited to analysis of old structures with flexible timber diaphragms. One advancement was that ETABS could now accept flexible floor panels. These, however, could not have assigned seismic mass, which had to still be applied to infinitely stiff diaphragms. This seeming impasse was overcome by a second new feature which allowed any number of (infinitely stiff) diaphragms in any one plane. Previously they would have had to be separated by a nominal vertical distance. Flexible diaphragms were therefore modelled by means of a 'chequer board' of multiple diaphragms with mass and flexible floors. Particular care was taken to ensure that the sections of stiff diaphragm did not have common corners where there would be direct load transfer.

Immediately north of the Great Hall there is a full height lightwell extending across most of the width of the building. This also lines up with a stairwell that continues the dissection of the structure. This arrangement results in little diaphragm continuity between the northern and southern sections of the building, which greatly influences the lateral dynamic behaviour of the building. Advantage was taken of this to split the building into two sections for the purpose of analysis so that the analysis file was of manageable proportions.



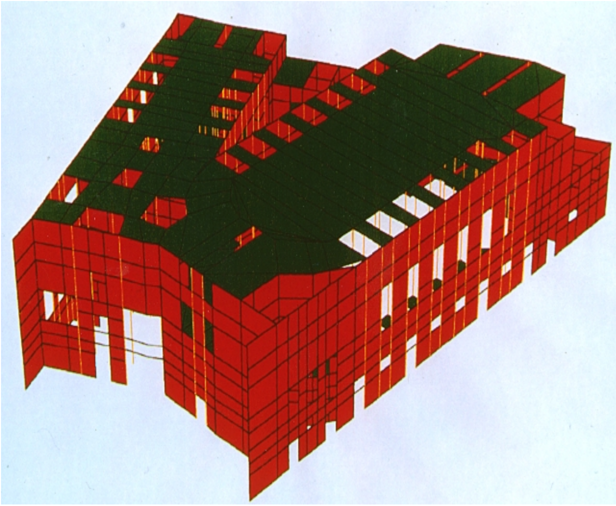
For longitudinal analysis additional panels were included in the external walls to simulate the boundary effects of the 'other half' of the building, but these were proportioned specifically to carry little shear. By this means each of the analysis models took care of its own shears even though in reality there would be some load sharing. The longitudinal stiffness of each half was approximately in proportion to its relative mass and hence this approach was considered reasonable.

The northern Apex analysis model included the Tower, in order to model the distribution of its seismic reactions below roofline. The eccentricity of the Tower meant that modelling the flexibility of the diaphragms was particularly important otherwise, with stiff diaphragms, the Tower reactions would be analysed as distributed over a far wider distance than reality. All significant openings such as windows and doors were included in the model, as was the double storey height Council Chamber. Figure 5 shows the Apex model. (Floors and diaphragms not shown).

Figure 5: Apex ETABS Model

The southern analysis model incorporated both the performance halls, together with the Entry Foyer. While the halls have no connection between them at their southern ends, they are 'joined at the hip' in the vicinity of the Foyer with a major common wall. Further, the Foyer floors are concrete and hence act as relatively stiff diaphragm elements, even though they have large stair penetrations.

Quite significant interaction between the halls was expected at their common wall and this was confirmed by the analysis. Figure 6 shows the ETABS model for the halls, showing also the flexible floor layout at ceiling level. The diaphragm "patches" are not shown but fill the spaces between the floors.



On the basis of the preliminary studies described above it was apparent that the Great Hall external walls would require some form of bracing at Circle and Roof levels. During the developed design phase the bracing elements, as described in the next section, were assessed and these elements were included in the dynamic analysis model.

Face loading analyses of different wall elements were carried out by the technique commonly referred to as the Priestley Method [3]. For this project this technique was developed into an automated, macro driven, spreadsheet which permitted efficient multiple analysis of all the various different wall sections and arrangements. Analyses included external walls to the Great Hall sides, Great Hall rear, Concert Chamber, Apex and Foyer as well as various different combinations of internal walls.

Figure 6: Halls ETABS Model, showing flexible "floor" panels with cut outs for mass locating diaphragms

GREAT HALL STRENGTHENING

The Great Hall is the focal space of the building and is essentially a large four storey high box. Its principal seismic hazards were the tall slender sidewalls, tall slender end walls and its lack of diaphragm at either Circle or ceiling levels.

The principal strengthening elements in the Great Hall are:

- Wall stiffening trusses at Circle level,
- Plywood diaphragm at ceiling level together with steel truss diaphragms at each end,
- Concrete shear wall at the north end of the hall,
- Concrete 'stitching' of two sections of the south wall so that it will act as one wall with improved aspect ratio,
- An external horizontal bracing truss on the south wall to reduce its height to thickness ratio.

The Circle truss, Figure 7, was installed partially under and partially within the original Circle floor construction. It was conservationally important that the trusses were not visible in the finished building and that the soffit line of the Circle was not lowered. The function of the trusses is to resist seismic face loading forces from the perimeter walls and thus reduce their height to thickness ratio. The span to depth ratio of the trusses is necessarily quite large in order to fit within these restrictions and also the chord dimensions were limited to 200 mm depth in order to fit within the existing Circle framing. This limited the stiffness that could be achieved with these trusses. The original masonry walls are some 1200 mm thick at their buttresses and hence have some significant stiffness in their own right. Consequently, the walls and trusses were of comparable stiffness, leading to complex interaction and variable levels of support along the length of the wall. By means of localised (artificial) diaphragms, the face loading mass of the walls was included at mid-height for the ETABS analysis.

A plywood diaphragm was selected for ceiling level support of the walls in lieu of steel trusses. The diaphragm was determined to have greater stiffness than sensible sized steel trusses and was also lighter and would be easier to install. As the diaphragm creates a cross tie between the opposite walls, out-of-phase responses tend to cancel out such that it is only in-phase responses that are required to be transferred the length of the hall. The cost saving was, however, smaller than anticipated. Additional benefits of plywood are that it provides enhanced acoustic isolation for the hall and also provides a walkable surface within the ceiling void.

Seismic stresses within the plywood are quite high and for ordinary nailing the nail spacing was in the realm of being impractical. However, as the diaphragm is to remain elastic even under an MCE loading, it was decided to use elastomeric adhesive in conjunction with nails which allowed a more practical nail spacing of 100 mm. At each end of the hall where the ceiling is narrower and the loads higher, plywood would be too highly stressed and so steel grillage trusses have been used in these areas.

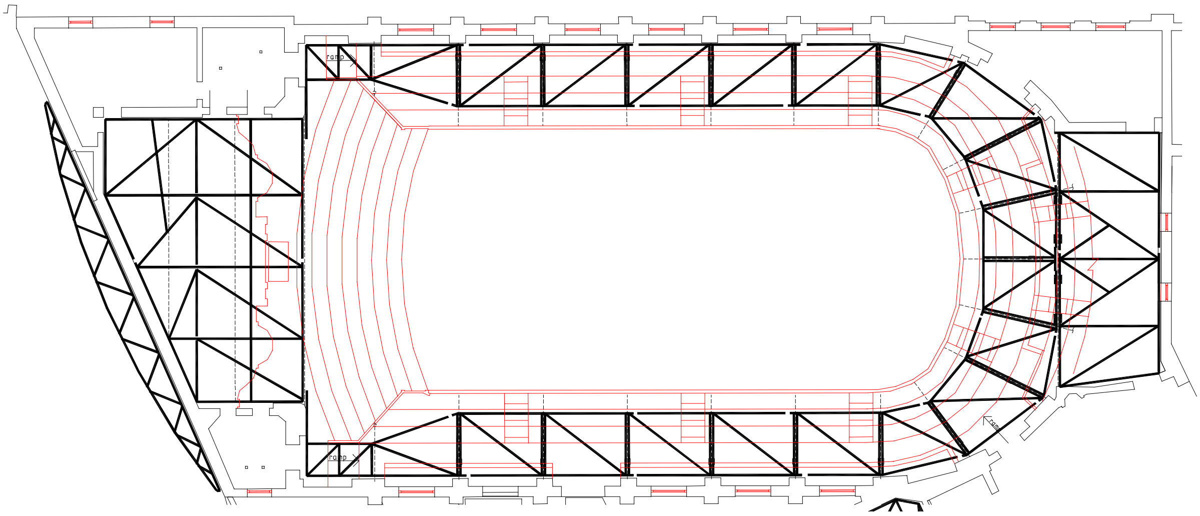


Figure 7: Great Hall Circle Truss

The analysis clearly identified response magnification. due to the flexibility of the diaphragms, as anticipated by theory . The south (rear) wall of the Great Hall is very slender and is also quite heavily loaded under seismic actions. As this wall provides essential support to both the Circle trusses and the ceiling diaphragm its integrity is critical to the whole of the Great Hall. Stabilising this wall presented a significant challenge. The problem was that the wall forms the rear wall of the Organ Loft, which contains some 2000 individual organ pipes, many being quite delicate. The major pipes are mounted against this wall and the loft is so full of pipes that it is difficult to move within the space. It was decided, therefore, that no work should take place within the organ loft. Fortunately, the Council also owned the adjacent rear property, which permitted installation of an external bracing truss overhanging the boundary. This truss is generally not visible as it is to be hidden behind the roof of the neighbouring ACC Garage. In the event that the garage site be developed in the future it may be necessary to reformat the method of support to the wall.

Figure 8: Circle Truss Inner Chord within Circle Bleachers

CONCERT CHAMBER STRENGTHENING

Strengthening of the Concert Chamber follows similar principals to the Great Hall, except as follows.

Steel trusses at Queen Street level were installed under the auditorium floor and thus did not suffer the same restrictions as the Great Hall Circle truss and hence were able to be made much stiffer. This was advantageous due to the additional loads arising from the earth retention along this frontage.

The perimeter walls between the steel trusses and roof level were still unacceptably slender but further trussing was not possible due to the windows. Internal brick piers were therefore removed and replaced with reinforced concrete piers to the same dimension in order to brace each wall panel over the height of the Concert Chamber.

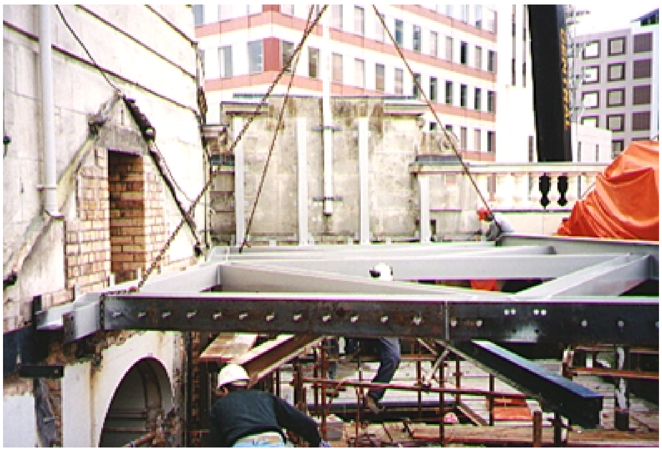
A roof level plywood diaphragm was installed over the top of the pitched roof rather than at ceiling level as has done in the Great Hall. This decision was based primarily on the geometry of the roof and on limitations of access.

APEX STRENGTHENING

Strengthening the Apex followed a more conventional approach than the two halls. All floors were upgraded as diaphragms by glue/nailed plywood fixed to the soffits of the existing floor joists. This overhead application was chosen in preference to the more usual floor overlay approach so as not to upset the riser heights at stairs, to avoid the need to trim heritage doors and to comply with the requirement that certain heritage tiled floor areas are to remain undisturbed. On the other hand, the ceilings had to be disturbed in any case in order to upgrade inter-floor fire rating and to install new services. Heritage cornices and the like were relatively easy to reproduce from moulds taken from the originals.

Longitudinally the Apex had sufficient shear walls of squat aspect ratio that therefore required little strengthening. Laterally there was more of a problem. Although there were sufficient walls to meet shear demands, the aspect ratio of many of these walls resulted in them having insufficient capacity under overturning moments. There were also some walls that also required shear enhancement. Consequently there are a significant number of cross walls that were strengthened either by a concrete skin or by insitu concrete replacement. Whole body overturning was also a problem in some instances. Every effort was made to maximise hold-down by means of the mass of adjacent walls, but in a number of locations, primarily at the Prow, ground anchors were used to enhance resistance to overturning.

TOWER STRENGTHENING



The Tower itself had been strengthened under a separate contract several years earlier. This strengthening consisted of a steel grillage installed inside the tower extending down to roof level. Below roof level tension straps were installed over the remaining height of the building, buried within the brick walls, to resist overturning. Within the main strengthening contract a massive steel truss was installed at roof level just behind the tower and laced to it. This receives the tower shears at this level and distributes them to the Apex sidewalls as well as controlling torsional response

Figure 9: Tower Restraint Truss

CONCLUSION

The works described in this paper, together with other minor strengthening works not discussed has provided the Auckland Town Hall with sufficient integrity for life safety and reasonable repairability when subjected to the Maximum Credible Earthquake for the site.

REFERENCES

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3 Priestley. M.J.N. (1985), “Seismic Behaviour of Unreinforced Masonry Walls”, *New Zealand National Society for Earthquake Engineering Bulletin*, Vol. 18, No. 2. also, “Discussion” Vol. 19, No. 1.

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