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Seismic Performance Summary
Draft Report

Christchurch Town Hall for the Performing Arts

86 Kilmore Street
Christchurch, New Zealand

April 2012

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DRAFT

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EXECUTIVE SUMMARY

Rutherford & Chekene has been engaged by the Canterbury Earthquakes Royal Commission to summarize the structural performance of the Christchurch Town Hall for the Performing Arts, hereafter referred to as the Christchurch Town Hall, located in Christchurch, New Zealand. As both a historic building and central to the arts and theater community, the Christchurch Town Hall's seismic performance presents a unique case study of earthquake damage and repair issues. The structure suffered significant damage during the Canterbury earthquake sequence with the Lyttelton event of February 22nd 2011 producing by far the greatest effects.

Most of the superstructure damage appears to be caused by widespread liquefaction and lateral spreading that resulted in differential settlement and building separation. Sand boils and the presence of sand and silt in the Christchurch Town Hall's basement are the most obvious evidence of liquefaction while ground cracking near the Avon River suggests extensive lateral spreading. Available reconnaissance reports indicate that no foundation bearing capacity failures were observed. Several portions of the superstructure have tilted, either towards or away from the Avon River, to accommodate the severe ground movement. Structure response due to ground shaking may also explain some superstructure cracking, although it is difficult to identify in the presence of such dramatic settlement damage.

Preliminary retrofit schemes for both the superstructure and foundation, developed for the Christchurch City Council and publically released, are intended to repair earthquake damage and strengthen the building. The Council's Earthquake-Prone, Dangerous and Insanitary Building Policy has generally been interpreted to mean that any building suffering earthquake damage must be repaired and strengthened to a minimum of 67% NBS (New Building Standard). However, retrofit that utilizes a new force-resisting system must meet 100% NBS. Changes to the seismic demand in Christchurch, principally through an increase in the zone factor from 0.22 to 0.30, makes meeting 67% NBS both difficult and potentially costly. Thus superstructure options that achieve 67% and 100% NBS have been proposed to the Council in conjunction with an extensive ground improvement and new foundation system designed to 100% of current code.

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1. INTRODUCTION

1.1 BACKGROUND ON STUDY

Rutherford & Chekene has been engaged by the Canterbury Earthquakes Royal Commission to summarize the structural performance of the Christchurch Town Hall for the Performing Arts, hereafter referred to as the Christchurch Town Hall, located in Christchurch, New Zealand. The Christchurch Town Hall suffered varying levels of damage, primarily due to liquefaction and lateral spreading, during the Canterbury earthquake sequence including the Darfield earthquake of September 4th 2010, the event of December 26th 2010, the Lyttelton earthquake of February 22nd 2011 and the two events of June 13th 2011.

As both a historic building and central to the arts and theater community, the Christchurch Town Hall's seismic performance presents a unique case study of earthquake damage and repair. Review of reconnaissance reports by Tonkin & Taylor and Holmes Consulting Group, in addition to scrutiny of the existing structural drawings, formed the basis of the statements and conclusions found in this report. It is thought that a more detailed description of damage is contained in Holmes Consulting Group's report entitled *Structural Earthquake Damage Assessment* dated May 26th 2011, however this report was not available to Rutherford & Chekene. It must be noted that neither of the authors visited the Christchurch Town Hall in person since it experienced damage. A full list of references can be found at the end of this report.

1.2 BUILDING DESCRIPTION

Overview

The Christchurch Town Hall is located at 86 Kilmore Street within the central business district of Christchurch. See Figures 1 and 2. It is bordered on the east by Colombo Street, on the north by Kilmore Street and on the west by the Crowne Plaza Hotel. The Avon River and Victoria Square sit immediately south of the Christchurch Town Hall.

Forming a T-shape in plan over approximately 6500m^2, the Christchurch Town Hall is divided into several blocks as shown in Figure 2. The north-west portion of the structure comprises the auditorium, capable of seating an audience of 2354, an orchestra of 120 and a choir of 400, and is elliptical in shape [7]. Two levels of tiered seating surround a central floor and orchestral stage with promenade foyers and backstage accommodation encircling the tiered seating.

In the north-east portion of the structure, the James Hay Theatre seats an audience of 1006 and has fan-shaped seating composed of a first floor and partial mezzanine [7]. Above the stage, a fly tower rises to approximately 20m to accommodate the rigging required for performances. North, south and east of the stage are various supporting spaces.

A first floor entrance lobby and mezzanine connect the auditorium to the theater in addition to providing the main entrance to the Christchurch Town Hall. South of the lobby, the structure hosts a restaurant on the first floor that projects over the edge of the Avon River bank and whose upper floor contains a 500 seat banquet hall known as the Limes Room [7]. The portion of the building southeast of the restaurant is composed of a basement, first floor, mezzanine and second floor. It contains the kitchens and a conference room. A 1976 addition to the Christchurch Town Hall to the east of the restaurant and north of the kitchens provides office accommodations, utility rooms and includes the Cambridge Room.

The auditorium connects to the Crowne Plaza Hotel at the west via a pedestrian bridge from the second floor. A second pedestrian bridge spans over Kilmore Street, again from the second floor of the auditorium. See Figure 2.

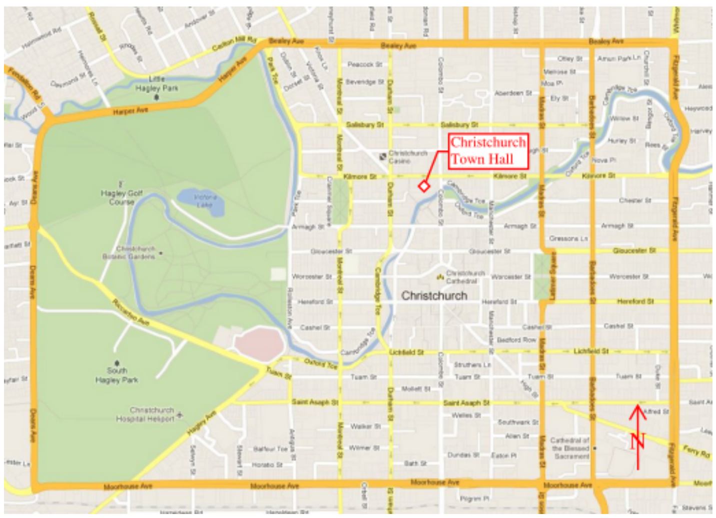

Figure 1: Christchurch Town Hall Location

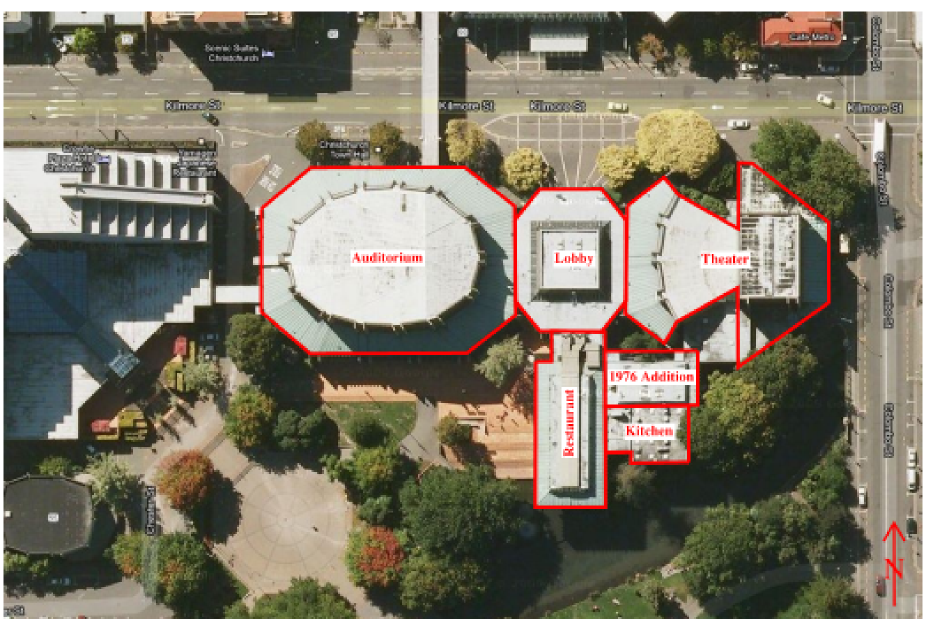

Figure 2: Building Areas

Superstructure Force-Resisting System

Reflecting the building occupancy and use, the lateral force-resisting system varies between portions of the structure. The auditorium relies on reinforced concrete slender piers and walls which form an oval that surrounds the seating and stage. Similar slender piers and walls on the perimeter also contribute to lateral strength and stiffness. See Figure 3. Both the ground floor and seating area slabs use cast-in-place concrete while the promenade foyer floors are composed of precast double-tee units supported on their webs with a cast-in-place topping. The upper roof of the auditorium consists of a lightweight concrete topping on roof slab units which span to built-up steel trusses.

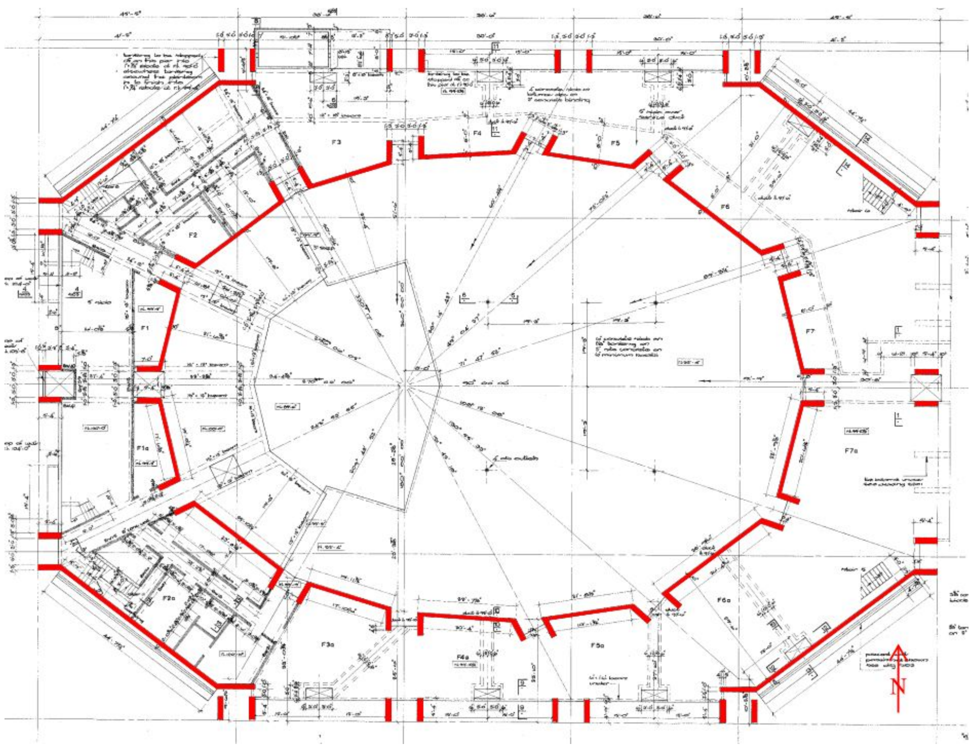

Figure 3: Auditorium Lateral-Force Resisting System

The theater also relies on reinforced concrete slender piers and walls for lateral stability of the seating while the fly tower and adjacent spaces utilize a combination of block and concrete-with-block walls. See Figure 4. Reinforced cast-in-place concrete forms the ground floor slab and mezzanine seating while precast concrete slabs are placed on steel beams or built-up steel trusses for the roof above the theater seating. Timber framing on steel beams provides a roof diaphragm for the supporting spaces adjacent to the fly tower while the fly tower itself utilizes built-up steel roof trusses, timber purlins and horizontal bracing.

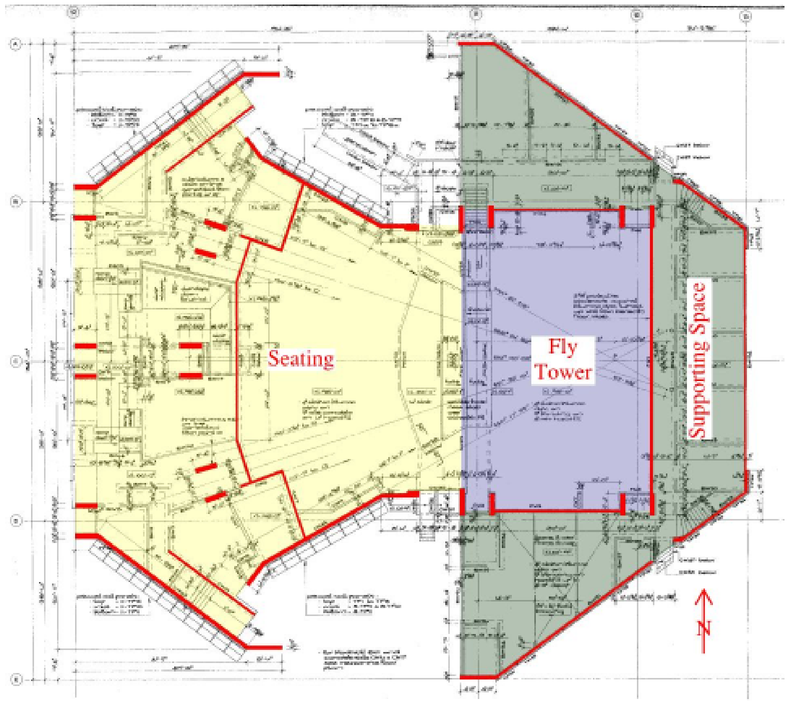

Figure 4: Theater Force-Resisting System

Reinforced concrete slender piers appear to be the only lateral system present for the lobby area's mezzanine and second floor as seen on the left of Figure 5. From the second floor to the roof, reinforced concrete walls are present in place of the slender piers. Precast double-tee units with concrete topping span to reinforced concrete cast-in-place beams to support gravity loads and to form a diaphragm for the mezzanine and second floor. The first floor employs a slab-on-grade while the roof is supported on built-up trusses.

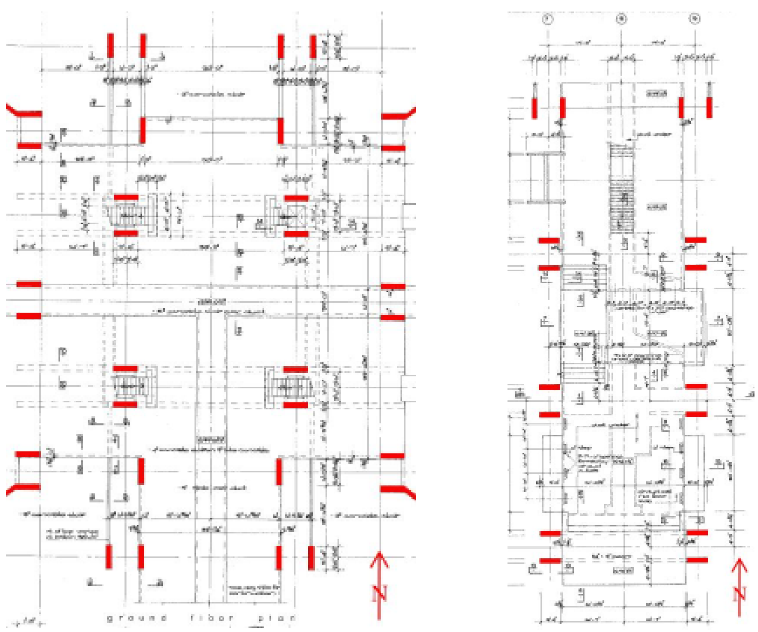

Figure 5: Lobby (left) and Restaurant (right) Lateral Force-Resisting Systems

The restaurant relies solely on reinforced concrete slender piers for lateral resistance with these piers acting about their weak axis for earthquake excitation in the north-south direction. See Figure 5 right. Several significant reinforced concrete beams span east-west between slender piers. Precast double-tee units are supported on their webs on the aforementioned beams and are overlain by a reinforced concrete topping slab for the second floor. The roof relies on built-up trusses while the first floor is made up of a reinforced concrete slab-on-grade.

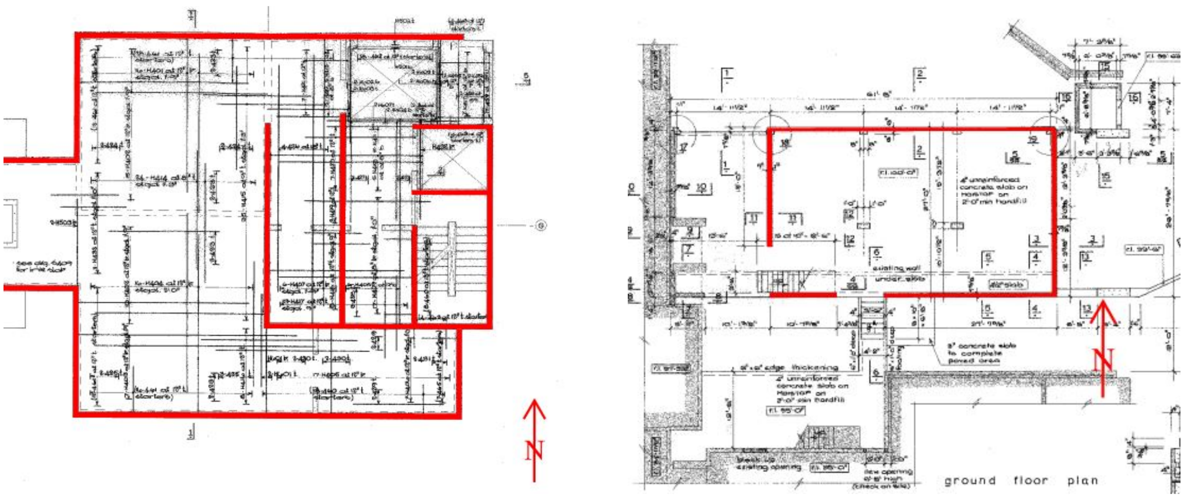

Figure 6: Kitchen (left) and 1976 Addition (right) Lateral Force-Resisting Systems

Both the kitchen and 1976 addition rely on reinforced concrete and block walls for lateral stability as shown in Figure 6. They both additionally utilize cast-in-place concrete suspended slabs except that the kitchen's roof is composed of precast double-tee units with topping. The roof of the 1976 addition is supported by built-up steel frames. A rendering of the entire complex's superstructure can be seen in Figure 7.

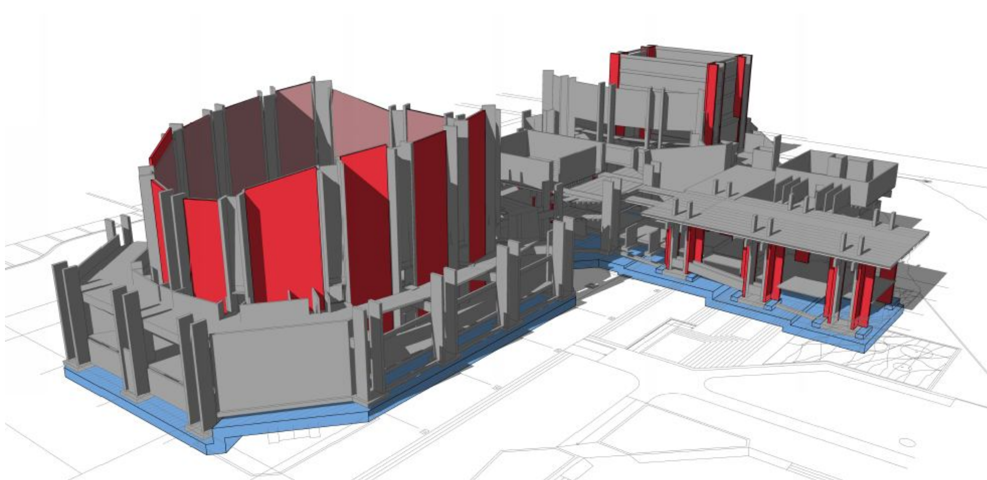

Figure 7: Superstructure 3D Model [1]

Note that elements in red or blue indicate proposed strengthening

Foundation Force-Resisting System

Generally, the Christchurch Town Hall's foundation system consists of shallow foundations such as strip or rectangular footings except deep foundations exist under the 1976 addition. In the auditorium, the slender piers are supported by pad footings at both the interior oval and the exterior. Foundations for the slender piers at the interior oval are further connected to each other by strip footings under the reinforced concrete walls. At the exterior they are tied together by small grade beams. The theater employs similar foundation elements with slender piers supported by pad footings, reinforced concrete walls by strip footings and several small grade beams interconnecting various elements.

Lobby slender piers are also supported on pad footings with small grade beams. Additionally several significant concrete ducts run under the slab-on-grade. One of these ducts continues to the restaurant where slender piers are again supported on pad footings. More significant tie beams exist between pad footings in the north-south direction of the restaurant with smaller grade beams running east-west. The kitchen block rests on a reinforced concrete mat foundation which is thickened near its center where an interior reinforced concrete wall is supported by it. In contrast to the original structure, the 1976 addition uses reinforced concrete piles rather than shallow foundations.

1.3 Relevant Codes & Compliance

Original Design Codes

Constructed in the late 1960s and early 1970s, the Christchurch Town Hall was most likely designed with reference to NZS 1900:1965 [1]. The architect was Warren & Mahoney, the structural engineer Holmes & Wood Consulting Engineers and the main contractor C.S. Luney Limited [7].

Current Design Codes

For their evaluation of repair and retrofit concepts, both Holmes Consulting Group and Tonkin & Taylor use NZS 1170:2004 with amendments that came into effect May 18th 2011. These amendments principally increased the zone factor, Z, from 0.22 to 0.30 for the Christchurch area.

Compliance & Alterations

Rutherford & Chekene did not receive any information pertaining to compliance documents. However, an addition to the Christchurch Town Hall was constructed around 1976 as shown in Figure 2.

2. Damage Description

2.1 Overview

The Christchurch Town Hall suffered damage to its superstructure, foundation and surrounding land as a result of the Canterbury earthquake sequence. Only minor cosmetic disturbances were observed after the Darfield event while significant damage was recorded after the Lyttelton event [1]. This damage appears to accumulate with aftershocks of the Lyttelton earthquake but is relatively minor compared to the initial effects. It is thought that most of the observed damage resulted from both vertical settlement and horizontal movement in response to liquefaction and lateral spreading rather than due to ground shaking, although it is difficult to separate the two causes in some cases.

2.2 Superstructure Damage

Movement & Separation

Total and differential settlement due to liquefaction and lateral spreading was significant after the Lyttelton event. A survey undertaken in April 2011 indicates foundation settlement in the range of 70mm to 460mm but more typically between 200mm and 350mm and is attributed mostly to liquefaction-induced settlement plus some limited strain as a result of building loads [5]. These measurements match well with those estimated by Tonkin & Taylor as described under Liquefaction of the Foundation & Land Damage section of this report. The auditorium, lobby and restaurant have tilted toward the southeast, south and northwest, respectively, as a result of differential settlement [2]. Put another way, the auditorium and lobby tilted towards the river while the restaurant tilted away. Such tilting, in addition to lateral spreading, has resulted in lateral separation between areas of the building and at seismic gaps for pedestrian bridges [1]. Figure 8 shows an example of separation damage where a building joint widened due to settlement. The restaurant suffered additional movement towards the Avon River after the June earthquakes and it is inferred that this resulted from reoccurrence of liquefaction and lateral spreading [5].

Figure 8: Building Joint Damage [1]

Concrete Shear Walls and Block Walls

The majority of damage to these elements appears to have occurred during the Lyttelton earthquake as a result of differential settlement caused by liquefaction and lateral spreading [1]. Detailing of the concrete shear walls seems to have anticipated some inelastic response, which is thought to have occurred in limited form, and damage reflects response beyond yield especially in the north wall of the 1976 addition's Cambridge Room. Other than limited shaking damage, the concrete shear walls and block walls primarily cracked and spalled in order to accommodate foundation movement. Typical block wall damage, most apparent in the auditorium, and concrete shear wall damage are shown in Figure 9.

Figure 9: Typical Block Wall Damage (left) and Concrete Shear Wall Cracking (right) [1]

Concrete Beams and Columns

Moderate damage of beams and columns during the Lyttelton earthquake appears to have resulted from foundation settlement although some cracking may have been caused by shaking. Flexural cracking of beams in the auditorium's promenade were measured between 0.2mm and 2.0mm while shear cracks up to 5.0mm were observed in the Lobby columns [1]. Figure 10 depicts the damage just described.

Figure 10: Column Cracking (left) and Beam Cracking (right) [1]

Slabs-on-grade, Service Tunnels and Basement Walls

All of these reinforced concrete structural elements suffered damage during the Lyttelton event due to differential vertical and horizontal movements brought upon by liquefaction and lateral spreading [1]. Much of the basement became flooded with water and a mixture of sand and silt [2]. As the underlying soils liquefied, the more highly loaded strip and pad footings settled to a greater extent than their adjacent slabs-on-grade thus causing forces to build up within the service tunnels and basement walls. Fracturing of the concrete members then occurred to alleviate these incompatibilities [3]. Additionally, the sand and silt ejecta pushed the slabs-on-grade of the auditorium and theater upward which, in combination with settling of the footings, exacerbated cracking and damage in them. Lateral movement of adjacent structures also contributed to damage of slabs-on-grade. Representative damage to slabs-on-grade is shown in Figure 11.

Figure 11: Slab-on-grade Damage [1]

Timber Floors, Walls and Beams

The auditorium and theater employ timber framing above the concrete slabs-on-grade. This timber flooring became sloped and uneven after the Lyttelton event because the slab-on-grade on which it was supported suffered significant differential settlement [1]. For the same reason, timber-framed walls in the auditorium were damaged in the form of fractured framing and distorted wall linings. Photos of representative damage can be found in Figure 12. Accompanying the building tilting and separation previously discussed, connections of glue-laminated timber beams in the lobby and restaurant tore out while the members themselves distorted as seen in Figure 13.

Figure 12: Timber Floor Damage (left) and Timber Wall Damage (right) [1]

Figure 13: Glue-Laminated Beam Connection Tear-Out [1]

Double-Tee Units & Floor Slabs

Precast double-tee units were found cracked and spalled after the Lyttelton earthquake, especially at their ends, as seen in Figure 14, and in their webs near points of seating [1]. It is unclear whether this results from differential settlement, shaking, poor detailing or some other unintended action although it is thought to be a result of lateral separation of the structure.

Figure 14: Double-Tee Cracking [1]

Holmes Consulting Group notes that tearing of floor slabs was anticipated based on the type of detailing used [1]. The affected diaphragms tended to be cold-drawn wire mesh, known to be non-ductile, within a reinforced concrete topping slab over precast double-tee units. Cracking and spalling appears to be concentrated in the auditorium's promenade, the lobby and the 1976 addition's Cambridge Room. It tends to manifest adjacent to columns and at re-entrant corners of diaphragms.

Cladding & Glazing

Although no specific locations have been identified in any of the reports submitted to Rutherford & Chekene, there is mention that the Christchurch Town Hall's cladding and glazing was damaged for much of the structure's exterior during the Lyttelton earthquake [1].

2.3 Foundation & Land Damage

Foundations

Although significant loss of support occurred during the Lyttelton event as liquefaction and lateral spreading progressed, the foundation system of the Christchurch Town Hall performed quite well in the context of ultimate limit state design [2]. This is exemplified by consistent observations that no bearing failures occurred and by the fact that the structure was able to tolerate the differential settlements without total or partial collapse.

Despite adequate performance, the existing foundations are experiencing additional forces as they help to resist differential settlement [2]. As such, they cannot be expected to be in the same condition as before the Lyttelton event. Nonetheless, it is believed that once pore water pressures dissipate, the static and seismic bearing capacities will have returned to their prior values.

Liquefaction

Widespread liquefaction was observed after the Lyttelton earthquake both within and around the building footprint and resulted in differential settlement of the building, heaving of slabs-on-grade, and flooding of the basement [2]. Sand boils on the ground surface around the Christchurch Town Hall and on the roads and properties immediately surrounding it were one form of evidence. Additionally, approximately 70 cubic meters of ejected sand and silt was removed from the Christchurch Town Hall's basement in addition to daily pumping of water. Immediately following the June earthquakes, additional sand and silt flowed into the Christchurch Town Hall's basement accompanied by more sand boils in the general vicinity. Figure 15 shows a mixture of sand and silt discovered in the basement.

Figure 15: Evidence of Liquefaction in Basement [1]

Tonkin & Taylor conducted a liquefaction analysis using data from the Resthaven Station, the closest strong-motion recording station to the Christchurch Town Hall [2]. Based on existing information and additional testing, an idealized soil profile was constructed as shown in Figure 16 with groundwater levels estimated between 1.5m and 2.5m down [5]. Their analyses predicted that layers 2, 3 and 5 were likely to have liquefied under the shaking present during the Lyttelton event and resulted in between 210mm and 350mm of settlement. There is a high risk that liquefaction will occur under a future Ultimate Limit State earthquake and a low to moderate risk under a future Serviceability Limit State event if no ground remediation is conducted [2].

Layer Number	Depth to top of layer (m)	Depth to bottom of layer (m)	Description	Geological formation	Typical SPT N	Typical q_c (MPa)
1	0	1.5	Fill	-	-	-
2	1.5	6	Interbedded silt, sandy silt, silty sand and sand.	Yaldhurst member, Springston	4 – 20	1 – 7
3	6	13	Interbedded sands, sandy gravel and gravels.	Yaldhurst member, Springston	15 – 50	5 – 30
4	13	14	Interbedded organic silts and peat.	Yaldhurst member, Springston	1 – 5	N/A
5	14	20	Silty sand and sand	Christchurch	25 – 50	N/A
6	20	21	Silt (aquaclude)	Christchurch	5 – 20	N/A
7	21	Unknown	Interbedded sands, sandy gravels and gravels.	Riccarton	30 – 50+	N/A

Figure 16: Idealized Soil Profile [5]

Lateral Spreading

Liquefaction often results in a phenomenon known as lateral spreading when the terrain is sloping such as exists for the Christchurch Town Hall due its proximity with the Avon River. Evidence of liquefaction-induced lateral spreading was observed following the Lyttelton event all around the Christchurch Town Hall but most prominently on the north bank of the Avon River as seen in Figure 17 [2]. It manifested as ground cracking, slumping, and cracks in pavements and is thought to be associated with an old river terrace. Lateral spreading was most pronounced nearest the Avon River and decreased as you moved away [5]. Tonkin & Taylor's analyses tended to overestimate the lateral displacements under the Lyttelton event but were of the correct order of magnitude. Summation of crack widths yielded the following lateral spread displacements:

- 350mm lateral displacement within 20m of the Avon River bank
- 100mm lateral displacement at the south side of the auditorium
- 50mm lateral displacement at the north side of the auditorium
- Null lateral displacement at the north side of Kilmore Street

It is thought that additional lateral spreading movement occurred during the June earthquakes, yet measurement instruments installed after the June earthquakes did not record any displacement in October nor November of 2011 [5]. A high risk of lateral spreading during a future Ultimate Limit State earthquake exists if no ground remediation is employed.

Figure 17: Lateral Spreading Map [5]

3. Repair & Retrofit

3.1 Overview

Both Holmes Consulting Group and Tonkin & Taylor have created proposed retrofit schemes in pursuit of repairing the damage caused by the earthquakes and bringing the structure up to an acceptable percentage of current code requirements. It is clear that for any superstructure retrofit to be effective, the ground conditions at the site must be improved. Tonkin & Taylor suggest two alternatives consisting of either deep or shallow foundations. Additionally, Holmes Consulting Group has developed a superstructure strengthening scheme that meets either 67% or 100% of the NBS (New Building Standard).

The Christchurch City Council adopted their Earthquake-Prone, Dangerous and Insanitary Building Policy in 2006 and made an amendment in 2010 that requires earthquake-prone buildings be strengthened to at least 67% of current code [1]. The 2010 amendment additionally covers buildings damaged by an earthquake. This section is interpreted to mean that applications for building consent for repairs will only be issued if the retrofit scheme brings the structure to at least 67% of current code. Therefore some damaged buildings may need to be strengthened to a higher standard than they met before suffering damage. Adopting the current New Zealand Building Code, including the increased zone factor, the Christchurch Town Hall would not have met the 67% cutoff before the Canterbury earthquakes. As such, superstructure strengthening includes not only damage repair but new and strengthened elements. In summary, the Christchurch City Council's minimum policy as it relates to the Christchurch Town Hall is interpreted as [6]:

- 67% New Building Standard for superstructure strengthening
- 100% New Building Standard for new foundations

3.2 Superstructure

Differences in superstructure strengthening between 67% and 100% NBS appears relatively minor with small additions of strength and stiffness for 100% compared to 67%. Recommendations have been based off nonlinear response history analysis in accordance with NZS 1170 [6]. Several critical deficiencies were identified during the quantitative analysis as shown in Figure 18 and listed below [4].

1. Some concrete shear walls in the auditorium have excessive shear deformations
2. The slender piers and block walls of the theater's fly tower become severely damaged
3. The lobby and restaurant's slender piers experience excessive flexural deformations
4. The kitchen's concrete walls suffer severe shear deformations

In addition to the new foundation discussed next, it has been proposed to use hydraulic jacks to re-level the structure so as to return it to a serviceable condition.

Figure 18: Critical Superstructure Deficiencies [4]

3.3 Foundation & Land

Two alternate foundation systems have been proposed. Both include repair or replacement of the damaged slabs-on-grade. The first system consists of piled foundations extending to the Riccarton gravels to support static and seismic loads that would surpass the liquefiable soil layers and thus prevent liquefaction-induced settlement from damaging the superstructure [5]. It additionally includes a lateral spreading treatment zone between the Christchurch Town Hall and the Avon River which arrests any horizontal soil movement tending to accompany liquefaction.

The second alternative consists of a mat foundation or raft slab under the entire Christchurch Town Hall that would serve to tie all of the structural elements together and would be capable of bridging over areas of differential settlement [5]. In conjunction with the mat foundation, ground improvement in the form of jet grouting would attempt to reduce the likelihood of liquefaction triggering.

4. CONCLUSION

4.1 SUMMARY

The Christchurch Town Hall suffered significant damage during the Canterbury earthquake sequence with the Lyttelton event producing by far the greatest effects. Most of the superstructure damage appears to be caused by widespread liquefaction and lateral spreading that resulted in differential settlement and building separation. Sand boils and the presence of sand and silt in the Christchurch Town Hall's basement are the most obvious evidence of liquefaction while ground cracking near the Avon River suggests extensive lateral spreading. Available reconnaissance reports indicate that no foundation bearing capacity failures were observed. Several portions of the superstructure have tilted, either towards or away from the Avon River, to accommodate the severe ground movement. Structural response due to ground shaking may also explain some superstructure cracking, although it is difficult to identify in the presence of such dramatic settlement damage.

Preliminary retrofit schemes for both the superstructure and foundation, developed for the Christchurch City Council and publically released, are intended to repair earthquake damage and strengthen the building. The Christchurch City Council's Earthquake-Prone, Dangerous and Insanitary Building Policy has generally been interpreted to mean that any building suffering earthquake damage must be repaired and strengthened to a minimum of 67% New Building Standard. However, retrofit that utilizes a new force-resisting system must meet 100% of current code. Thus superstructure options that achieve 67% and 100% of the New Building Standard have been proposed to the Council in conjunction with an extensive ground improvement and new foundation system designed to 100% of current code.

4.2 FINAL THOUGHTS

The following points stood out during Rutherford & Chekene's review of the Christchurch Town Hall:

- Why did no liquefaction or lateral spreading occur during the Darfield event but did during the June earthquakes? Is this predictable with current methods? Should the methods be improved?
- Although liquefaction is not generally considered to create life safety risks, should the building code require mitigation for new buildings? How should the "trigger" for such mitigation be measured? Probability of liquefaction occurrence? Total potential differential settlement?
- Is it possible that liquefaction essentially isolated the structure from damaging shaking? Should research in this area be recommended?
- Due to the various occupancies, the Christchurch Town Hall possessed a complex network of diaphragms with relatively poor connections. This damage and damage to diaphragms in other buildings suggests building code requirements for diaphragms should be clarified or strengthened.

- During their nonlinear response history analysis, Holmes Consulting Group noted the lack of New Zealand specific procedures for evaluating performance of buildings constructed prior to the advent of modern seismic engineering [4]. Should New Zealand develop such a document that addresses critical versus non-critical failure modes?
- Damage and repair costs associated with this building highlight a policy issue on the balance between provision for good future performance and the preservation of heritage structures. Could less strengthening be accepted in order to enable preservation and re-occupancy if the cost of full repair and retrofit is too great for the Christchurch Town Hall?

5. REFERENCES

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