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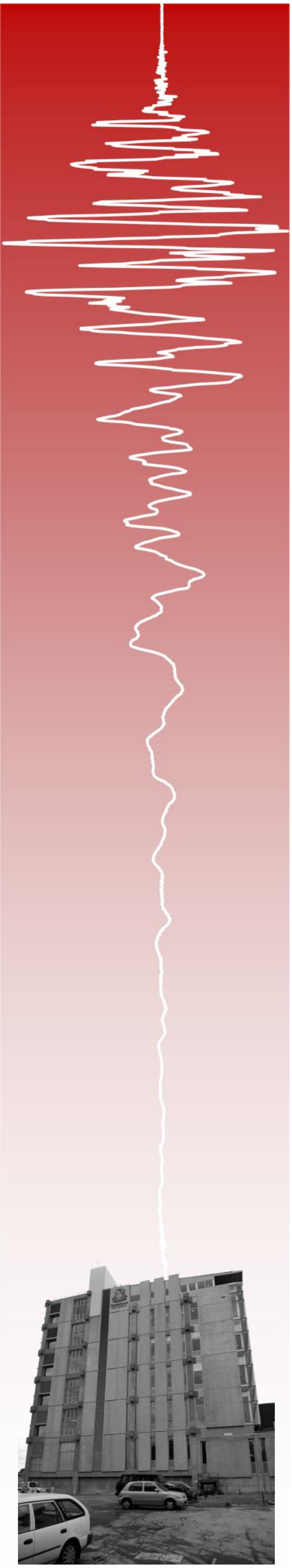

Christchurch Civic Building
53 Hereford Street, Christchurch

INDEPENDENT EARTHQUAKE PERFORMANCE ASSESSMENT

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Limitations

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1. Introduction

Compusoft Engineering Limited have been engaged by the Canterbury Earthquakes Royal Commission to independently assess the performance of specified structures located in the Christchurch central business district (CBD) during the Canterbury Earthquakes of 2010 and 2011. These assessments are required by the Royal Commission to assist in fulfilling the requirements set out for them in their establishing terms of reference [1]. This report presents our independent assessment of the Christchurch Civic Building located at 53 Hereford Street, Christchurch.

This report has been prepared based on documentation and reports provided by the Canterbury Earthquakes Royal Commission. Compusoft Engineering Limited had not inspected the Christchurch Civic Building prior to publication of this report.

2. Location of building

The Christchurch Civic Building is located at 53 Hereford Street, Christchurch as shown in Figure 1. This location places the structure west of the centre of Christchurch and close to the Avon River, which is approximately 90 metres from the structure at its closest approach.

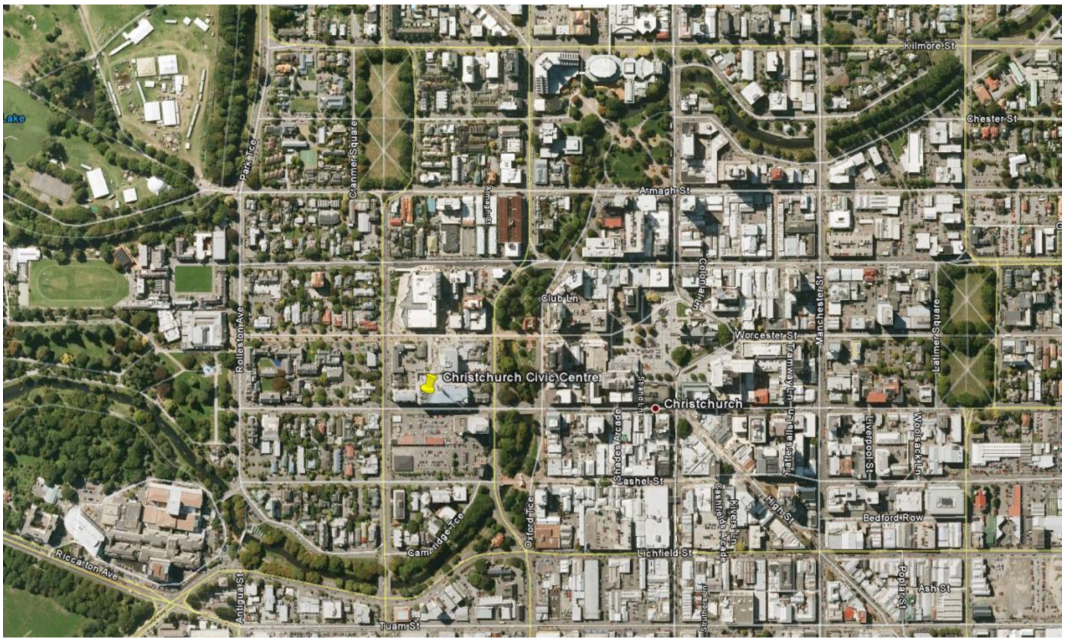

Figure 1: Plan showing location of Christchurch Civic Building

3. Description of building

The Christchurch Civic Building was originally constructed for The New Zealand Post Office in the early 1970s. The design was undertaken by the Ministry of Works who signed the structure off in 1974 [2]. The original building comprised a main six storey structure and a single storey annex structure to the north side of the building. Very large (5791 mm/19') interstorey heights and high floor loads were specified for the original structure due to its intended use as a mail sorting centre. Between 2007 and 2010 the structure was extensively upgraded and extended in order to be recommissioned as civic offices for the Christchurch City Council. The designers for the upgraded structure were Powell Fenwick Consultants Limited. The main features of these alterations comprised the addition of 8.34 m of new structure at the north side of the building and installation of mezzanine floors at several levels. The dimensions of these mezzanine levels are summarised in Table 1. An image of the upgraded structure is shown in Figure 2, while Figure 3 shows a typical floor plan of the upgraded structure.

Table 1: Floor areas of Christchurch Civic Building

Area description	Number of areas	Approximate dimensions	Total area
Original structure typical floor	-	78.0 m×29.3 m	2,283 m^2
Typical extension	-	78.0 m×8.34 m	651 m^2
Level 1 mezzanines	3	various	~175 m^2
Level 2 mezzanines	2	2×14.0 m×9.7 m	273 m^2
Level 3 mezzanines	2	2×19.5 m×19.5 m	761 m^2
Level 4 mezzanines	2	2×19.5 m×19.5 m	761 m^2
Level 5 mezzanines	2	2×19.5 m×19.5 m	761 m^2

Figure 2: Christchurch Civic Building viewed from the north-east

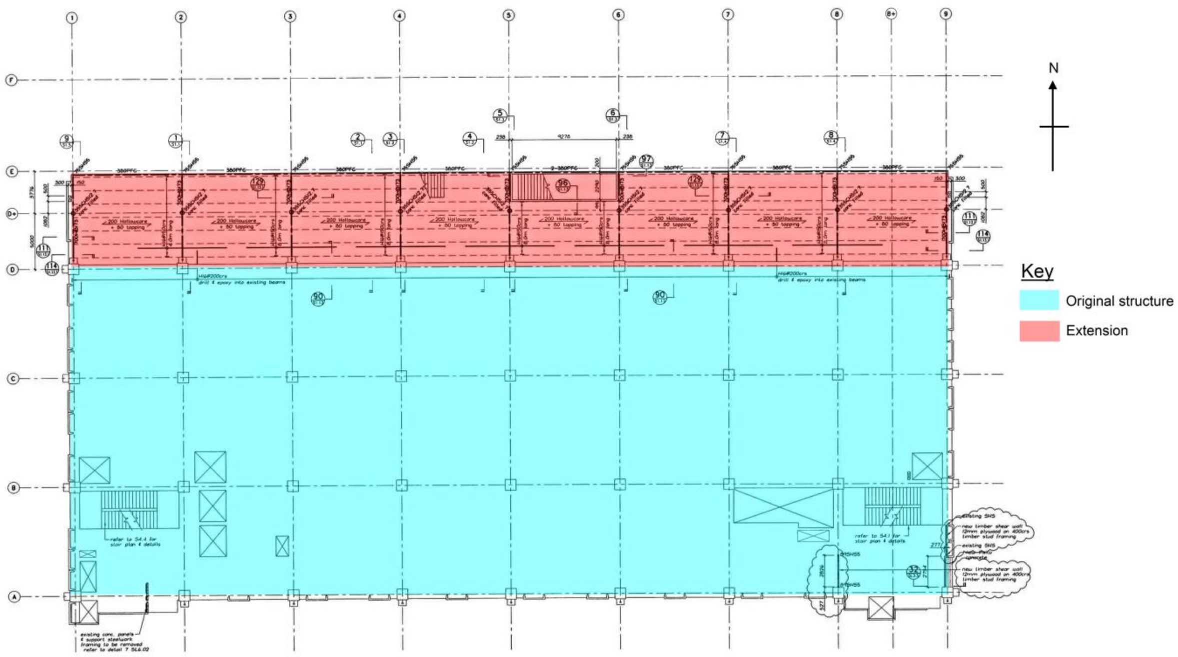

Figure 3: Typical floor plan of the Christchurch Civic Building

The gravity and lateral force resisting systems of the Christchurch Civic Building largely rely on the same structural elements, and hence both are discussed together in this section. Due to the expected significance of the recent additions and upgrades to the seismic response of the structure, the original and extension structural systems are described in separate sections below.

3.1. Gravity and lateral force resisting system – original structure

The Christchurch Civic Building is a reinforced concrete moment resisting frame structure. The frames are arranged on a two-way grid, with all primary beam and column members being of similar dimensions and having full moment connections. The moment resisting frame consists of 9.75 m (32 foot) bays in both directions. Secondary beams running along the east-west axis of the structure are also present, spaced at approximately 3.25 m centres. Typical member dimensions are summarised in Table 2.

Table 2: Typical beam and column dimensions

Member type		depth	width
Columns		965 mm	965 mm
Primary beams:	E-W	838 mm	762 mm
	N-S	1016 mm	838 mm
Secondary beams (various types)		762 mm 508 mm 419 mm 419 mm	356 mm 305 mm 356 mm 635 mm

The floors of the Christchurch Civic Building consist of reinforced concrete flat slabs. These slabs are typically 127 mm (5 inches) thick, and there are no thickenings at columns. The slabs are uniformly orthogonally reinforced for positive bending, with reinforcement for negative bending provided only at beams. The reinforced concrete slabs are also required to function as diaphragms.

As noted, the original design for the Christchurch Civic Building was undertaken by the Ministry of Works. Based on the 1974 signoff date it is probable that the structure was designed using the Ministry of Works Code of Practice for Public Buildings [3], with possible reference to other standards used commonly in New Zealand in the early 1970s [4, 5]. The design of the structure occurred after publication of the principles of "capacity design" [6], which had in any case been incorporated to some extent in Ministry of Works guidelines [7] as early as 1968 [8].

3.2. Gravity and lateral force resisting system – recent additions

The extension added to the Christchurch Civic Building between 2007 and 2010 added 8.34 m of additional floor width to the north side of the building, with the new floor comprising 200 Hollowcore planks topped with 80 mm of in-situ concrete reinforced with a layer of 663 mesh and other reinforcement discussed below. The hollowcore planks span in the east-west direction to steel beams (700WB173). As shown in Figure 4 the south end of these beams is connected to columns of the original structure on grid D, while additional support is provided by concrete filled steel columns (355CHS12.7) located on grid D+. The positioning of these new columns results in the beams having a cantilever span of approximately 3.8 m that supports the double skin facade system. Due to the positioning of the new column and the reduced live load allowance required for the new use, the designers of the extension found that the gravity loads in the Grid D columns was lower than allowed for in the original design [9].

Lateral loads from the extension structure are intended to be transmitted back to the original moment resisting frames by diaphragm action through the 80 mm topping. Reinforcement of the topping consisted of 663 mesh and additional HD16 “saddle” bars were placed at 150 mm centres aligned in the east-west direction to resist hogging moments over the supporting steel beams. The design accounted for then-recent considerations related to detailing of precast floors. Each Hollowcore plank had two cores broken out, reinforced with HD12 bars, and then filled with concrete filled. Displacement incompatibility between the moment resisting frame and the Hollowcore planks was mitigated by offsetting the closest Hollowcore plank from the adjacent beam of the moment resisting frame by either 635 mm or 800 mm. This separation was spanned by an 80 mm thick concrete infill slab (see Figure 5), with connection of the topping to the existing structure achieved by drilling and epoxying of HD16 bars into the existing beams on Grid D, typically at 200 mm centres.

The discussion in the preceding two paragraphs reflects the typical construction of the extension structure; in some areas of the extension other construction was used, for example there is an area of 350 double tee flooring at level 2.

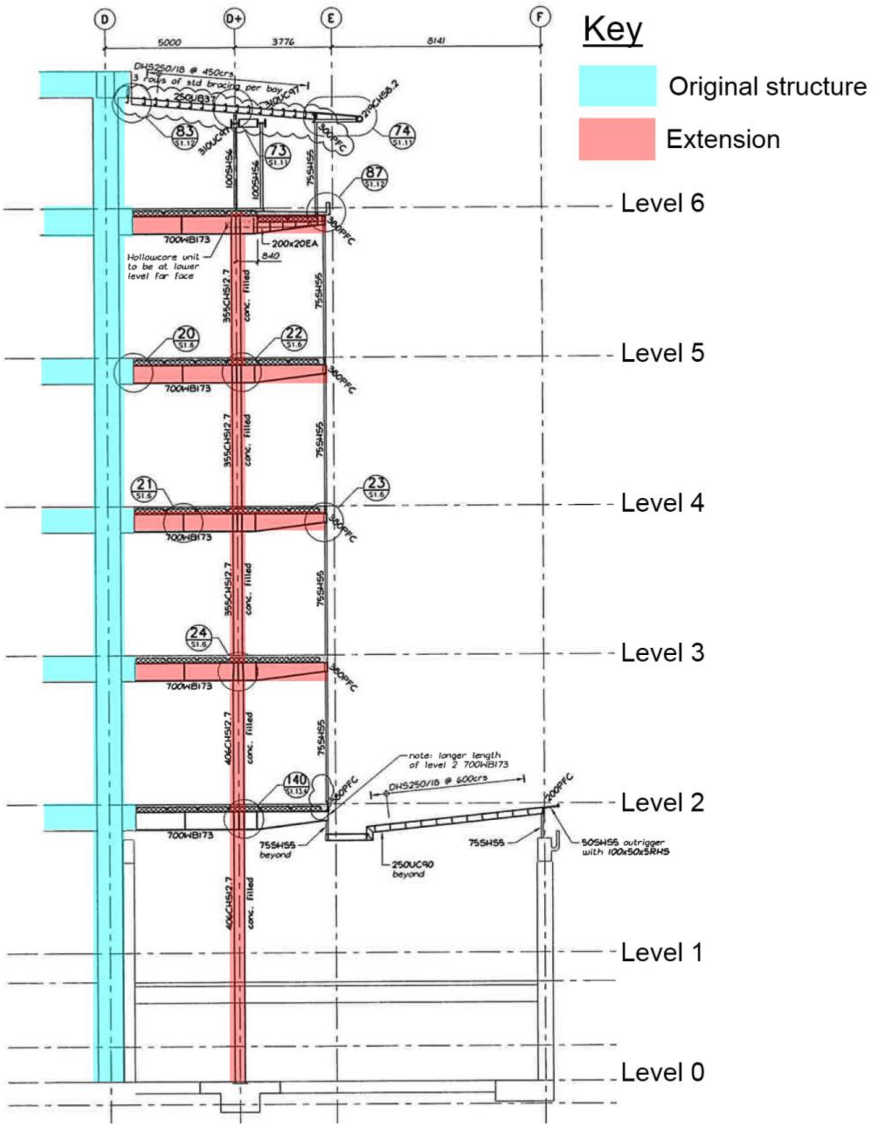

Figure 4: Typical section through extension structure

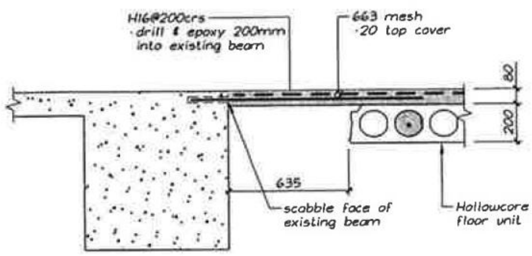

Figure 5: Section at interface of original and extension structure

The upgrade of the Christchurch Civic Building entailed some other structural works in addition to the extension described above. These included addition of the mezzanine floors previously summarised in Table 1, the alteration of various beams and floor areas in the main structure, and the removal of the original roof from the annex structure. These aspects of the work are relatively minor and not considered likely to have an impact on the structural behaviour of the building.

At the time of this upgrade it was verified that the structure was able to resist 100% of then-current design actions [10]. The frame was assumed to respond with ductility $\mu = 3$ [11], although its detailing was considered to be equivalent to that of a "fully ductile" structure [9]. It is important to note that this verification was undertaken on the basis of the structure being Importance Level 2 (IL2) rather than Importance Level 4 (IL4). There is no indication that the original moment resisting frame required retrofit work to achieve this level of compliance.

3.3. Foundation system

The foundations of the original part of the Christchurch Civic Building consist of a reinforced concrete cellular raft system with a total depth of approximately 3556 mm (140 inches). The base slab of the raft is 1270 mm (50 inches) deep, and the top slab (also the Level 1 floor slab) is 305 mm (12 inches) deep. Support between the two slabs is provided by a grillage of 1200 mm (48 inch) wide reinforced concrete beams. Many of these beams contain 1200 mm wide by 1830 mm high penetrations.

The foundations for the extension structure consist of three foundation beams and an isolated pad foundation. The foundation beams are 1200 mm deep and 1500 mm wide, with two having 2500 mm wide thickenings at their ends to accommodate a high voltage cable duct. The pad foundation is a 3600 mm wide square with a depth of 1200 mm.

3.4. Secondary elements

While not necessarily significant with regards to structural performance, it is noted that a raised floor system was installed as part of the recent upgrades of the Christchurch Civic Building. This has a typical height of 420 mm at levels 2 to 5, but larger heights of 600 mm to 1744 mm at level 1.

A second notable feature added to the structure during the upgrades is a double skin facade installed on the north side of the building. This facade is intended to act as a thermal and solar buffer zone, and is supported from by the cantilever extensions of the steel beams discussed in section 3.2 and shown in Figure 4.

The original structure as built for the New Zealand Post Office featured an access ramp for trucks entering the structure from Worcester Street (i.e. at the north side of the structure). During the upgrade of the structure this ramp was converted to provide pedestrian access to the building. The ramp structure was designed to be seismically separated from the building [11], and thus not to affect the earthquake behaviour of the structure.

4. Geotechnical site assessment

A geotechnical report [12] for the Christchurch Civic Building has been provided to Compusoft Engineering Limited as an appendix to the report describing the structural performance and repairs for the structure [10]. The geotechnical report was based on a total of fourteen bore logs. Twelve of these were those used by the Ministry of Works during the original design of the structure. The remaining two bore logs were from holes bored during May 2011 to investigate the post-earthquake ground conditions.

The geotechnical report found that the Christchurch Civic Building was founded on a typical Christhchurch CBD soil profile consisting of a shallow (6-8 m) gravel layer, sands and silts at depths of 8-22 m, and Riccarton gravels below 22 m. The water table for the site is approximately 2.5 m below the surface. It was reported that there was potential for significant liquefaction at the site. Typically this would be at depths between 9 m and 15 m, although some areas of the site could liquefy at depths as shallow as 5 m.

The geotechnical report is consistent with a widely published subsurface cross section of Hereford Street [13, 14], reproduced here as Figure 6.

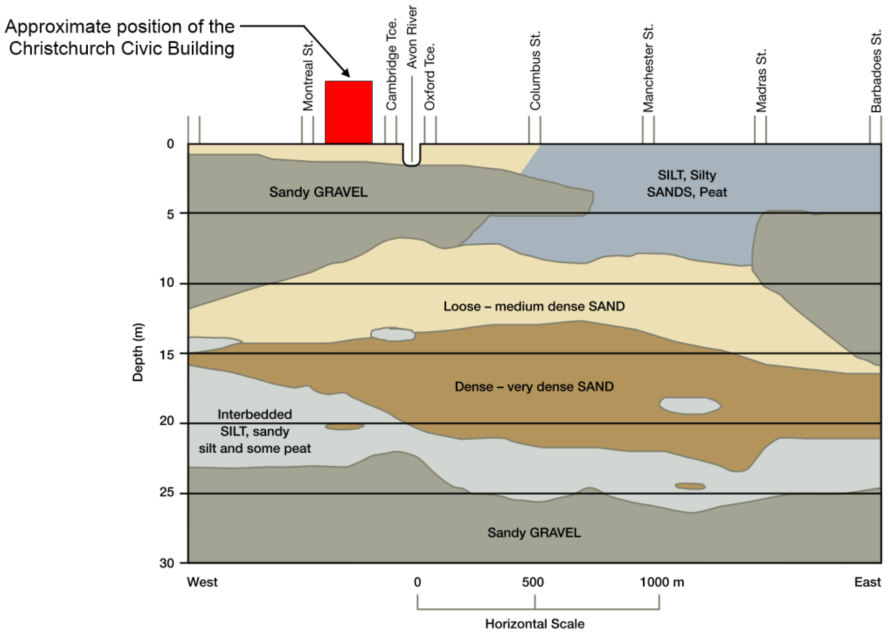

Figure 6: Subsurface cross section of Christchurch CBD along Hereford Street (adapted from [13])

5. Compliance

The building at 53 Hereford Street was originally designed in the early 1970s by the Ministry of Works to be used by the New Zealand Post Office. As it was owned by the Crown there was no requirement to obtain a building permit from the Council.

The structure of this building remained largely unaltered until 2008, when a programme of works commenced to convert it to offices for the Christchurch City Council, including a substantial extension to the north, an additional floor, and the addition of mezzanines to levels 3, 4 and 5. These alterations and extensions were covered by building consent applications made in multiple stages. The work was completed and final code compliance certificates were issued on 12 August 2010.

Subsequent to the September 2010 earthquake, repairs were undertaken but there is no record of any building consent having been obtained.

The building was further damaged by the February earthquake. Repairs were commenced, and building consents obtained in a series of stages. No code compliance certificate has been issued as of 13 February 2012.

6. Effects of earthquakes on building

The level of damage inflicted on the Christchurch Civic Building by the 4th September 2010 22nd February 2011, and June 2011 earthquakes was relatively low. Extensive investigations of the building were undertaken, resulting in a detailed catalogue of damage being prepared [10]. The following sections describe key aspects of the damage incurred. Damage is generally discussed without reference to the specific earthquake that caused the damage; while it can reasonably be stated that the majority of damage occurred during the 22nd February Lyttelton earthquake, further information about the occurrence of each type of damage during the September 2010, February 2011, and June 2011 earthquakes is summarised in Table 3.

Table 3: Summary of damage occurrence during 2010 and 2011 earthquakes

Structural aspect	Earthquake		
	September 2010	**February 2011**	**June 2011**
Original frames	-	Spalling of concrete in columns adjacent joints. Shear cracking in beams	Some cracking.
Extension structure	Yielding where steel beams connect to existing structure. Crushing and spalling of concrete in the infill slab.	No apparent movement at steel beam connections. Cracking of concrete at edge of infill slab.	Movement where steel beams connect to existing structure. Cracking and spalling at double tee seating.
Stairs – general	Cracks in topping concrete at stair landings.	Spalling to edges, cracking through stairs in places. Stairs safe to use.	Cracks to landings at level 3 and 4, alongside previous repair
Stairs – Level 1 to 2	Cracking and spalling of top conection, cracking of “sliding” base connection.	Stairs jammed and considered unsafe to use.	-
Foundation	-	Moderate liquefaction at east end of structure.	-

6.1. Original moment resisting frame structure

Damage to the moment resisting frames of the original structure was reported to be minor [10], with most cracks having widths of 0.5 mm or less and a maximum crack width of approximately 2.0 mm [15]. Additional damage taking the form of spalling of cover concrete from the column corners below beam-column joints occurred in a number of locations. An example of this damage is shown in Figure 7.

Figure 7: Concrete spalling at column corner under beam-column joint (from [10])

It has been reported that hardness testing of reinforcement in the structure has been undertaken by Holmes Solutions to assess the level of residual strain in the bars [10]. While Compusoft Engineering Limited have not seen the results of this testing, the results of the testing apparently indicated that the residual strains were generally less than 10% of the ultimate strain and that yielding of the reinforcement has not significantly affected the residual capacity of the structure [15].

6.2. Extension structure

As noted in previously in this report, the extension structure added to the building during the recent upgrades relies on the original moment frame structure to resist lateral forces. As a result of this reliance, damage to the extension structure during the 2010 and 2011 earthquakes mostly occurred at the interface between the two structures. This damage took two forms, namely damage to the steel support beams and damage to the concrete infill strip.

Damage to the steel support beams in the vicinity of their connection to the existing structure was reported to have occurred. It was proposed that this was due to their being designed as pinned shear connections while in reality having some moment resistance. The most significant damage seems to have occurred during the September 2010 earthquake.

The damage to the infill strip between the original frame and the extension precast floor units comprised crushing and spalling of concrete. This was attributed to the concrete at the connection being thinner than intended. As with the steel member damage discussed above it appears from reports to have been most significant during the September 2010 earthquake.

6.3. Stairs

It is apparent that the designers paid attention to support and isolation of the stairs of the Christchurch Civic Building during the recent upgrade of the structure (see for example Figure 8). As a result, the damage incurred by the stairs was generally minor. Through the main part of the structure damage was typically limited to cracking and minor spalling at the landings where topping concrete concrete cracked and spalled where precast units met. A set of stairs spanning from level 1 to level 2 was more significantly damaged during the February earthquake. It was reported that this damage was due to the performance of the sliding joint at the stair base being impeded by the installation of tiles [10].

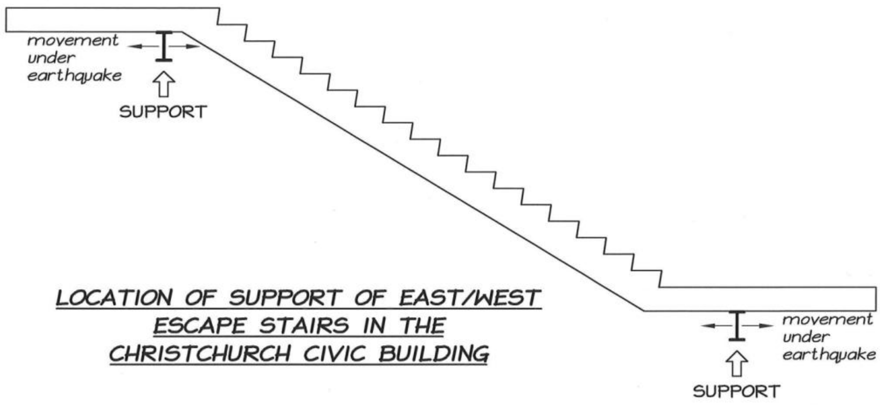

Figure 8: Schematic of stair support used at Christchurch Civic Building (from [10])

The performance of the egress stairwells was somewhat impeded by the performance of the wall linings used, with significant "non-structural" damage occurring. This issue is discussed further in section 8.1.

6.4. Foundations

Moderate liquefaction was reported to have occurred at the east end of the site during the February 2011 earthquake [12]. No liquefaction was reported to have occurred at the site during the September 2010 or June 2011 earthquakes. Despite the occurrence of liquefaction during the February 2011 earthquake the performance of the foundations was considered to be satisfactory, with no significant impact on the structure.

7. Structural performance

Compusoft Engineering Limited have not independently analysed the behaviour of the Christchurch Civic Building. However, we consider the performance of the structure during the 2010 and 2011 earthquakes to be commensurate with the performance that would be expected of the structure. This judgement is made with reference to the findings of a recent assessment by Compusoft Engineering Limited of the the Christchurch Central Police Station [16], a structure of similar age, form, and designer (see Table 4).

The assessment of the Christchurch Central Police Station found that the demands placed on that structure by the earthquakes were approximately equal to the code design earthquake actions, and that the earthquake response of the Police Station structure was similar to that predicted based on analysis results. Both the Christchurch Central Police Station and the Christchurch Civic Building had capacities of the order of 100%NBS [10, 15, 16], although the actual capacity of the Christchurch Civic Building was reported only to exceed 100%NBS without a margin of exceedence being provided. The ductility assumed during assessment was similar for both structures; thus it seems reasonable to conclude that the capacity of the Christchurch Civic Building is greater than that of the Christchurch Central Police Station, and would therefore be likely to be less damaged. This conclusion agrees with reported damage levels for the Christchurch Civic Building, which appeared to be similar to or slightly lower than those reported for the Christchurch Central Police Station.

7.1. Connection of extension structure to existing building

It was noted in section 6.2 that damage occurred at the interface between the existing and extension structures. Damage to the concrete infill slab was attributed by others [17] to the infill slab being thinner than intended. While this may have contributed to the damage, it seems probable that damage would have occurred to the infill slab even if it had been constructed in accordance with the drawings. While heavily reinforced, the purpose of the infill slab is to span between elements deforming in an incompatible manner, and would thus have experienced significant out of plane deformations during the 2010 and 2011 earthquakes.

Table 4: Comparison of Christchurch Civic Building and Christchurch Central Police Station

Structural characteristic	Christchurch Civic Building (from [10, 11, 15])	Christchurch Central Police Station (from [16])
Constructed	~1974	~1969
Lateral load resisting system	two-way moment resisting frame	two-way moment resisting frame
Designer	Ministry of Works	Ministry of Works
Structural periods	2.30 and 2.45 seconds	2.0 and 2.15 seconds
Ductility assumed for assessment	3.0	2.25 – 3.0
Capacity (%NBS, Z = 0.3)	> 100%	≈ 100%
Estimated ductility demand during 22nd February earthquake	-	≈ 2.0

While the damage resulting from displacement incompatibility noted above was not due to any deficiency in the design process, review of the limited information available regarding the design of the diaphragm connection between the extension and the original structure suggests that the design was not rigourous. The seismic design for the connection was based forces determined as the product of the extension weight and the design action action coefficient for the building, $C_d(T_1)$, with no account made for amplification of accelerations that typically occurs above ground level. Some idea of the magnitude of this amplification can be gained by considering the design actions for parts, F_{ph}/W_p. For an elastic effectively rigid part (such as the extension diaphragm component), F_{ph}/W_{ph} could be as high as 1.5. This compares to the value of $C_d(T_1) = 0.05$ used for the design of the diaphragm. While it is debatable whether it would be appropriate to design the extension structure diaphragms as a part, the fact that the part coefficient could be 30 times higher than the design coefficient for the structure gives an indication of the magnitude of the non-conservatism of using the structural design coefficient.

8. Issues arising from review

The largely satisfactory performance of the Christchurch Civic Building means that there are few issues that have arisen from the review reported in this document. However, three issues did occur to the authors while writing the report. These are outlined in the following sections.

8.1. Detailing of non-structural items in high-importance areas of structures

As noted previously, the seismic performance of the stairs in the Christchurch Civic Building was relatively good. However, the post-earthquake usability of the stairs was to some extent impeded by the performance of the non-structural walls, which were at risk of falling into the stairwells after being severely damaged during the February earthquake. Retrofit measures mitigated the risk of blocking the stairwells during the June earthquakes.

It is not clear from the information seen by Compusoft Engineering Limited why such severe damage to the non-structural partitions occurred. Irrespective of the detailed reason, it is reasonable to assume that the damage was due to the non-structural components not being detailed to accommodate lateral deformation of the structure. As far as Compusoft Engineering Limited are aware the current New Zealand Building Code does not require designers to ensure that the post-earthquake useability of emergency exits remain uncompromised by the performance of non-structural components, although there are requirements for access ways and fire escape routes [18-21]. While the the New Zealand Design Action Standard [22] does require structures to be designed so that:

> *"damage to non-structural systems necessary for the building evacuation procedures that renders them inoperative"*

is avoided, it seems probable that this aspect of structural performance is often overlooked. This is likely due to partition walls often being considered to be the responsibility of the architect rather than the engineer. Adding explicit requirements for earthquake egress to the building code would ensure that this was considered by engineers. It would be simple to require adoption of a methodology in which "maximum credible" displacements are considered in a similar manner to that proposed for stair units themselves [23].

8.2. Design of diaphragms

As discussed in section 7.1, review of information related to the diaphragm connection between the extension and original structure showed that the accelerations used during the design of this connection were incorrectly assessed. While the mistake is not felt to have affected the performance of the building during the 2010 and 2011 earthquakes, Compusoft Engineering Limited consider that it reflects a lack of thought or understanding about diaphragm design that is widespread within the structural engineering profession in New

Zealand. While Compusoft Engineering Limited are aware that this is an issue that has been raised numerous times previously in various forums, we feel its importance warrants its re-mentioning.

8.3. Importance level of large office structures

It was noted earlier that the upgrades for the Christchurch Civic Building were designed assuming that the structure was of Importance Level 2, i.e. that it was a conventional office structure. Assuming no post-disaster function for the structure, this Importance Level is in accordance with the New Zealand Design Actions Standard [22]. However, item (h) under examples of Importance Level 3 structures in Table 3.2 of the Design Actions Standard states:

> *(h) Multi-occupancy residential, commercial (including shops), industrial, office and retailing buildings designed to accommodate more than 5,000 people* ***and*** *with a gross area greater than 10,000 m^2.*

The emphasis placed on "and" in the definition has been added here. The use of the word "and" seems unusual for two reasons:

1. It is difficult to envisage a structure of less than 10,000 m^2 that is designed to accommodate more than 5,000 people, thus making the word "and" redundant.
2. Even if a structure could be envisaged with a floor area less than 10,000 m^2 that accommodated more than 5,000 people, it would seem reasonable that this structure should be considered to be of Importance Level 3.

A similar, but possibly less convincing argument to that made in point 2 above could be made for the inverse, i.e. a large structure intended to accommodate less than 5,000 people. This type of structure would include the Christchurch Civic Building, which has a floor area of significantly more than 10,000 m^2 but accommodates less than 5,000 people [24, 25].

Aside from issues discussed above, it is not clear on what basis 5,000 people was selected as the cutoff for a structure being considered important, and in comparison to the numbers required for educational, health, and transport facilities to be considered important (see Table 5) the number seems very high. Lowering the threshold at which a structure must be considered to be of Importance Level 3 would inherently enhance the robustness of large structures.

It is suggested that the items discussed above represent inconsistencies in the New Zealand Design Actions Standard [22] that could lead to structures being considered less important than might rationally be assumed. Revision of these requirements is recommended.

Table 5: Capacities of various structural types corresponding with Importance Level 3 (adapted from [22])

Structure type	Capacity for structure to be Importance Level 3
Day care facilities	150 people
Schools	250 people
Tertiary education facilities	500 people
Non-emergency healthcare facilities	50 people
Airport and railway terminals	250 people
Residential, commercial, industrial, office, and retail structures	5,000 people

9. Conclusions

The Christchurch Civic Building was originally built in the 1970s by the Ministry of Works for the New Zealand Post Office. The recent conversion for use in its current role involved addition of a substantial extension to the structure, but no retrofit strengthening of the original moment resisting frame structure. The performance of the building during the 2010 and 2011 Canterbury earthquakes was largely satisfactory, with primarily non-structural damage occurring. The performance of the structure was considered to be commensurate with the performance that would be expected based on the design of the structure.

Two issues were identified for attention during the review. The first of these is an apparent oversight in the current New Zealand Building Code not specifically requiring non-structural items to be designed so that their performance does not impede the post-earthquake use of emergency exit ways. The second issue identified related to the Importance Level assigned to large buildings. It was suggested that the threshold at which “general” structures are required to be considered to be of Importance Level 3 should be revised, which would result in the resilience of large structures in New Zealand being enhanced.

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