REINFORCED CONCRETE BUILDING PERFORMANCE IN THE M_w 7.8 1931 HAWKE'S BAY, NEW ZEALAND, EARTHQUAKE

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SUMMARY

This paper examines the seismic performance of over half of the existing low-rise reinforced concrete buildings that survived the 3 February 1931 Hawke's Bay earthquake. Lateral resistance of these buildings is provided by reinforced concrete walls, unreinforced brick masonry infill frames and open reinforced concrete moment-resisting frames. Twenty-five buildings are analysed in both orthogonal directions for the lateral loads estimated to have occurred during the earthquake.

The probable shear and bending strengths of structural members are compared to the maximum calculated seismic shear forces and bending moments. Wall restoring moments are compared to overturning moments. Whereas analyses suggest that most structures should have been severely damaged during the earthquake, in fact they performed well. In most cases no structural damage to reinforced concrete members was reported. Asymmetric buildings performed about as well as symmetric buildings. Possible reasons for these observations are examined and it is recommended how current practice might reflect these findings. The paper also contributes to an approximate assessment procedure, based on ratios of structural cross-sectional area to ground floor area, and reports on the structural areas of buildings that performed well in the earthquake. The excellent seismic performance of reinforced concrete buildings during the 1931 Hawke's Bay earthquake suggests current earthquake engineering analyses of similar pre-1935 low-rise non-domestic reinforced concrete buildings may underrate their seismic performance.

1. INTRODUCTION

1.1 Background to study

The Hawke's Bay New Zealand earthquake occurred at 10.42am on February 3rd, 1931 with a moment-wave magnitude of $M_w = 7.8$ [1]. The fault rupture surface extended from about 30 km depth to within 5 km of the surface adjacent to both Napier and Hastings [2].

The earthquake severely damaged many commercial unreinforced brick masonry buildings, but those constructed from reinforced concrete performed very well [3]. Of the one hundred reinforced concrete buildings that survived the earthquake, some suffered minor structural damage, a few (with moment resisting frames) suffered serious damage, and only one, the Napier Hospital Nurses' Home, collapsed. Construction of the Nurses' Home was not representative of the period prior to the earthquake. Large arches at ground floor created a soft storey, and necessary reinforcing was omitted in structural elements (G. Natusch, pers comm.).

The damage to buildings (still existing in 2000) was reported variously as "*no damage*", "*moderate*", to "*reinforced concrete part ok but brick part collapsed*". In general, damage levels were low. For example, "*...where properly designed and constructed, [reinforced concrete] behaved admirably*" [4]. The same article also describes several different structural systems and how each fared, e.g. "*...(a) The mushroom head columns, flat slab, drop panelled construction survived the earthquake almost without a flaw; (b) Plain beam and column construction is exemplified in the Public Trust Office, Hastings, ...and Dalgety's, Napier, [both] of which and numerous others, were practically unharmed.*"
Another article [5] states "*Most of the damage...occurred in columns, and particularly ground floor columns*", "*...in only one instance was there any evidence of beam or girder failure*" and "*Where integral reinforced-concrete walls had been used very little trouble occurred, but brick partition walls...were damaged.*"

Of over 90 buildings of one to five storeys with significant vertical (and horizontal) reinforced concrete structure that survived the earthquake, about 45 are still in use today. Twenty-five of them are the focus of this study and a list of them, including descriptions of damage, is given in Table 1. All the buildings studied are low-rise, ie. 2-3 storeys. Each building is analysed in both orthogonal directions with the transverse direction normally parallel to the adjoining street. In terms of building configuration, the buildings have various combinations of frames and walls, including unreinforced masonry walls. Two buildings on street corners have two adjacent stiff and two relatively flexible façades, and four

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reinforced concrete frame buildings have ground floor soft storeys.

A summary of the attributes and damage states of the 68 reinforced concrete buildings not analysed in the study is given in Appendix A.

1.2 Sources of information

Visits were made to Hastings and Napier to identify structural systems, determine the extent of reinforced concrete in each building, photograph, and where necessary, to take measurements. Hastings District Council, Napier City Council, Art Deco Trust, Natusch Architects and the Hawke's Bay Museum were approached for construction drawings, photographs and related information. Several informal interviews were held with relatives of building owners and others. A major source of information on damage was the collection of annotated photographs and block plans of buildings (indicating damage levels) of the Insurance Council of NZ Collection held in the Alexander Turnbull Library, Wellington.

Data on each building is compiled and presented succinctly [6]. A typical example of a building summary, together with its analysis results is presented in Appendix B. Building damage information, that focuses on structure rather than architectural elements and contents, ranges from general to precise. Most concrete walls and beams were reported as essentially undamaged. This leaves columns (not surprisingly) as the type of element most frequently incurring damage. The subset of buildings with columns studied here seems not to have had much heavy column damage, though in a few cases this may have been hidden by good repairs. Buildings with heavy column damage not studied here include the three-storeyed Hannah's shoe shop in Napier (demolished in 1932) that was saved by its concrete block infill walls; the four buildings (two and three-storeys) of Dr. Moore's hospital in Napier which were saved from immediate collapse by their brick masonry infills; and the five-storey buildings at Nelson's Freezing Works near Hastings where damage to columns was again mitigated by masonry infill.

A previous study [7] on damage to domestic buildings in the 1931 earthquake provides soil classifications of areas where buildings under study are located. Ground is classified as A (rock), B (firm) and C (soft). All buildings in Hastings are located on Ground Class C, whereas in Napier Classes B and C are applicable. Ground Class is relevant in determining the peak ground acceleration (PGA).

1.3 Structural characteristics of the buildings and their damage

The structural characteristics of the twenty-five buildings studied are summarised in Table 1. Seven buildings have concrete walls in two directions and a further nine have them in one direction. The remaining nine moment-resisting frame buildings all have brick infill in at least one direction. Open moment-resisting frames occur in seven buildings, but only in one direction.

Six buildings are highly asymmetric in plan. Notable among these is the three-storey Hawke's Bay Farmers building with a full masonry infill wall along one boundary. Only two buildings, the Public Trust Offices in Hastings and Napier (Figure 1) are classed as corner buildings. The Napier Public Trust Office was much the most heavily damaged building in the data set, despite its two street frontages appearing outwardly to be undamaged.

Four buildings exhibit soft-storey reinforced concrete construction. Of them, the three-storey Bennett building (Figure 2) is most notable due to its ground floor "columns" in the transverse direction consisting of concrete walls swaying out of plane. Poppelwell's and Villa d'Este have stiffer first floor construction than the ground floor (in one direction), but the upper floor vertical structure is entirely of unreinforced masonry.

The brief descriptions of damage in Table 1 are supplemented by estimates of the damage ratio, D_r, for the concrete structure only, where

$$D_r = \frac{\text{Cost of damage to concrete structure}}{\text{Replacement value of that structure}}$$

The damage ratio for the worst damaged building, the Napier Public Trust Office, is based on actual costs and value, while the remaining damage ratios were assessed on the basis of building damage reports.

1.4 Research method

The research method is divided into the following phases:

1. Determine the peak structural response of each building during the earthquake.
2. Determine seismic loads and calculate structural actions (shear forces and bending moments for seismic plus gravity) in members using the equivalent static method and linear elastic analysis for each orthogonal direction.
3. Calculate the probable strength of structural members using the Green Book [8].
4. Compare probable strengths with structural actions.

Median Peak Ground Accelerations (PGAs) for the soil classifications of NZS 4203:1992 [7] in both cities, given their different site to fault rupture plane distances, are determined from a recent attenuation model [10] and are presented in Table 2.

Peak structural response, the lateral force coefficient (C) used to calculate the earthquake load on each building, is determined as follows:

$$C = [C_h\,(T_1 = 0, \mu = 1)(NZS\ 4203)] \;x\; \frac{PGA\ of\ 1931\ earthquake}{PGA\ of\ NZS\ 4203\ (T_1 = 0,\ \mu = 1)}$$

Figure 3 shows the assumed response spectrum for Napier sites based on the NZS 4203:1992 spectrum for a site subsoil category (b), equivalent to firm soil. Using the above equation, the value of the acceleration for the plateau between T_1= 0 and T_1= 0.45 seconds is calculated as (0.8 x 0.82 / 0.42) = 1.56

The building base shear in each orthogonal direction is determined in accordance with NZS 4203:1992 except that C, the seismic coefficient, is replaced by a seismic coefficient whose derivation is explained above. As all buildings are low-rise (two and three storeys), the natural period of vibration can be assumed less than 0.45 seconds. A value of $\mu = 1$ (elastically responding structures) is assumed given the lack of reported structural damage. This value gives an upper bound load level that results in the 1931 Earthquake induced loads exceeding those arising from NZS 4203 1992, as shown in Table 3.

Table 1. Summary Table of Damage and Vertical Reinforced Concrete Structure

	% Reinforced Concrete of Gross Floor Area		No. of Storeys	Principal Structural Type		Symmetry	Soft Storey	Top Storey Brick Only	Approx Damage Ratio for Structure (%)	Description of damage
NAPIER	**Trans**	**Long**		**Trans**	**Long**					
Methodist Church Hall	1	0.4	2	BF	BF				0	End walls (brick) unsafe.
Gilberd & Co	1	2	2	F+W	F+W	A			0	No structural damage.
Hawke's Bay County Council	1	1	2	W	W	A			0	2 brick parapets down.
Public Trust Office (N)	1	1	2	W+F	W+F	C			15	Badly cracked and unsafe. Large shear cracks in structural walls. Roof beams damaged.
Murdoch J H, Bakery	3	4	2	W	W	U			0.5	Concrete floor and wall cracked.
Dalgety & Co	1	1	2	F	W	A			0	No structural damage.
Ancient Order of Foresters	4	4	2	W	W+F				0	No structural damage.
Ringland Bros.	0.3	2	2	BF	W+BF	U			0	Brick badly cracked.
Union Hotel	0.4	0.3	2	BF	BF				0	No structural damage.
Harston's Concert Hall	0.4	0.4	2	F	BF				0	No structural damage.
Bennett Building	1	3	3	F	W	P	S		0	No structural damage.
Parker Chambers	1	1	3	W	BF			✓	0	Brick walls on 3rd floor fell.
Williams Buildings (I)	12	4	2	W	F	P	S		0.5	A few minor cracks in concrete
Williams Buildings (II)	1	1	2	BF	F		S		0	End brick gable fell out.
Richardson & Co. Ltd.	1	3	2	W	W				0.5	Slightly damaged.
Howe Bros.	2	1	2	F	BF				0	Tile roof and 2 large chimneys down. Brickwork fell off sidewalls.
West (JS) & Co	1	3	2	F	W				2	Sidewalls damaged at 1st floor.
HASTINGS										
Poppelwells	1	1	2	BF	F			✓	0	End gable fell out.
Dominion Buildings	7	2	2	W	W		S		0	No structural damage.
Public Trust Office (H)	1	2	2	W	W	C			0	No structural damage.
Rainbow, Hobbs & Nesbitt	1	7	2	W+F	W				0	No structural damage.
H.B. Jockey Club's Racecourse	0.3	0.9	2	MBF	W+F	A			2	Stub columns at front sheared through. Sidewalls (brick) cracked. Roof trusses dislodged.
Hawke's Bay Farmer's Co-Op	0.2	0.1	3	F	BF	A			0.5	Hair cracks in concrete on façade (structural damage).
Webber's Buildings	1	0.5	2	BF	BF	A		✓	0	No structural damage.
Villa d'Este	1	1	2	F	BF	U			0	Front façade (brick) fell out. Brickwork badly cracked.

Notes:

Symmetry: A = very asymmetric; C = corner building; P = Moderately asymmetric; U = u-shaped wall layout.
Soft Storeys: S = Soft storey in one direction
Structural Type: F = Moment frame; W = Structural wall; BF = Brick infilled moment resisting frame.

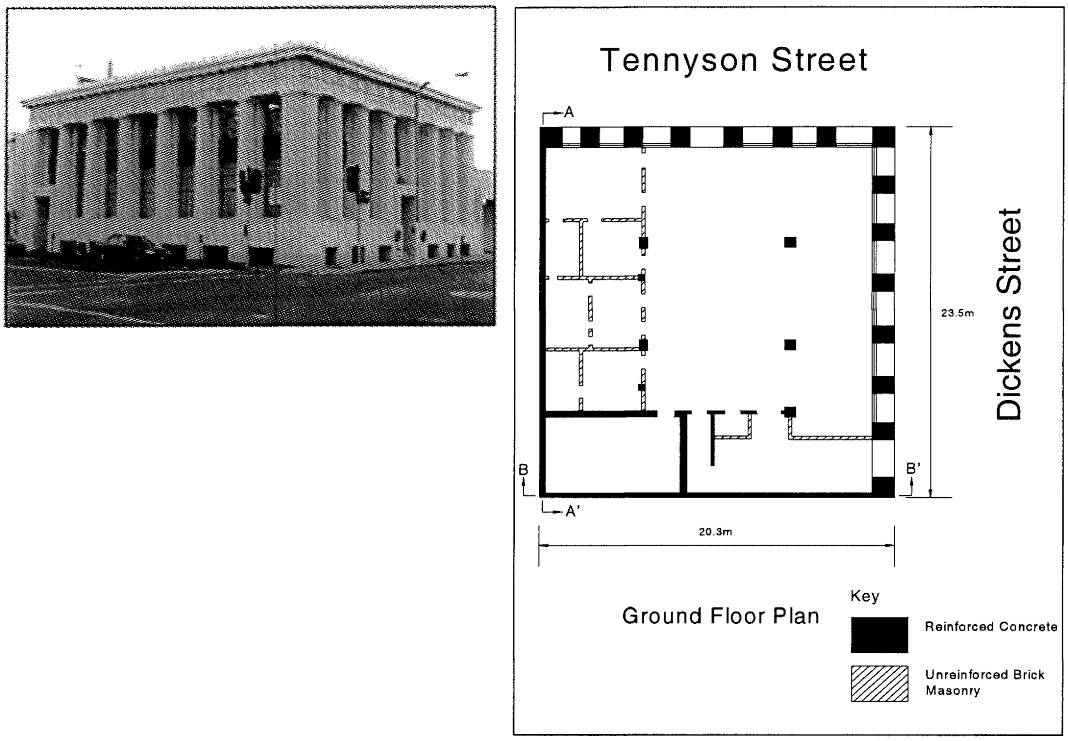

Figure 1. Public Trust Office, Napier, with perimeter walls on two adjacent facades, making it an asymmetric "corner" building.

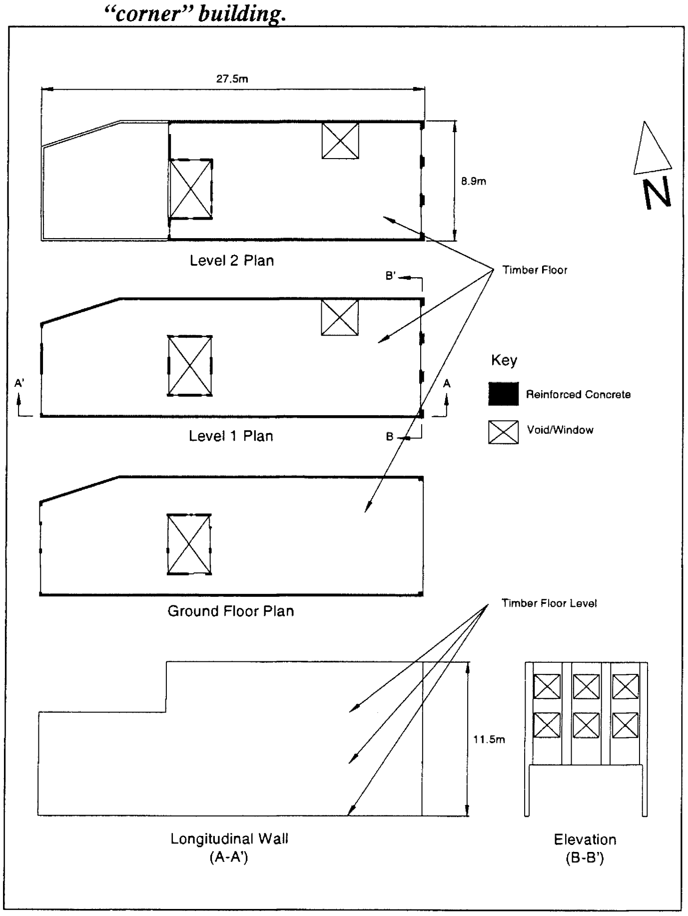

Figure 2. Plans and elevation of the Bennett Building, Napier, showing soft-storey construction.

Table 2. Peak Ground Acceleration for each city and Ground Class, 1931 Hawke's Bay earthquake

Napier Ground Class	**Distance (km)**	**PGA (g)**
A	3.5	0.60
B	3.5	0.82
C	3.5	0.48
Hastings Ground Class	**Distance (km)**	**PGA (g)**
C	6.0	0.45

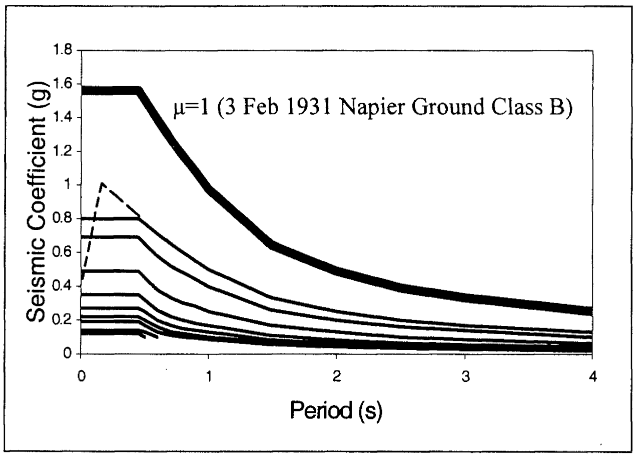

Figure 3. Assumed seismic coefficient of 3 February 1931 Earthquake, Napier, Ground Class B compared with NZS 4203:1992 basic seismic hazard acceleration coefficient.

Table 3. Calculated seismic loads for different ground classes as a percentage of NZS 4203: 1992 loads.

Napier Ground Class	**Calculated seismic load as a percentage of NZS 4203:1992 (%)**
A	No buildings located on this ground
B	243
C	143
Hastings Ground Class	
C	134

After determining the structural system and clarifying a 'structural skeleton', a linear elastic analysis provided structural actions in each member. For very simple structural systems, hand analyses sufficed while for more complex ones a computer analysis was used. Conventional structural modelling assumptions included rigid fixity of columns and walls at foundation level and masonry infill panels as pin-jointed diagonal compression struts.

Calculations of probable member strengths follow the detailed approach (equations) of the Green Book [8] with the following material property assumptions:

- tensile yield stress of reinforcing steel: 280 MPa [Brunsdon, D., 1999, pers. comm.];
- compression strength of masonry infill mortar: 12 MPa, modulus of elasticity of masonry: 18,000 MPa, and tensile strength: 0.4 MPa [11];
- compression strength of concrete: 21 MPa (based upon a 25% increase from 28-day strength of 16.6 MPa [Barnard, D., 1999, pers. comm.]), the modulus of elasticity of concrete: 21,000 MPa; and
- ultimate soil bearing pressure: 300 kPa, for shear wall stability calculations.

Where reinforcing details were unavailable, they were assumed to be similar to those of the 1929 Gilberd & Co. building. Details from buildings in Wellington and Wairoa of the same vintage confirm that these details are representative of those in the buildings studied. In the case of ten buildings, not only were no drawings available dated before or after the earthquake, but the buildings shared one or more boundary walls with adjacent buildings. The uncertainties that this construction brings to an assessment of structural performance, coupled with an inability to model such buildings simply with sufficient accuracy suggested a Results Reliability Rating scheme. Buildings such as those mentioned above received a C rating, meaning that apart from consideration of wall overturning effects, analysis results are not included in any other summaries.

Influences of several assumptions necessary to simplify the analysis, some common practice among structural design engineers, are discussed below:

- Assumed ductility
  Seismic loads are based on $\mu = 1$. If a building had experienced some ductility where μ equalled 1.25 or even 2, structural response would have reduced by14% and 39% respectively.

- Presence of unreinforced masonry (URM) walls
  With the exception of those buildings where URM walls are significant in number and configuration (Harston's Concert Hall and the Methodist Church Hall), URM walls have been ignored in the structural modelling. In some instances they may have resisted a significant proportion of the seismic load, resulting in an overestimation of overturning to restoring moment ratios.

- Adjacent buildings
  Another major assumption involves neglecting the influence of adjacent buildings (close enough to induce hammering), or those that are adjoining. Walls may have overturned in some instances if adjacent or adjoining buildings had not provided increased stability. As most of the buildings in this study are located within the commercial centres of Hastings and Napier, some did have neighbouring buildings with which they shared a common wall or to which they were very close.

- Diaphragm stiffness
  Reinforced concrete suspended floor diaphragms are assumed to be rigid. The flexibility of timber diaphragms is acknowledged by distributing lateral loads to vertical elements on the basis of the tributary floor area resisted by each element.

- Torsional effects
  Torsional forces arising from plan asymmetry were ignored.

1.5 Analysis uncertainties

We adopted a simple deterministic approach to the calculations, assuming median levels of earthquake loadings and material behaviour. Variability in both loadings and strengths, as well as modelling limitations, could of course explain some of the discrepancies between damage predictions and observations. In terms of its performance during the earthquake, our sample of buildings is known to have performed slightly better than the total population of concrete buildings from Hastings and Napier that generally performed well.

Probably the largest uncertainty in this paper's approach is in the magnitude of the calculated seismic loads. To some degree this uncertainty can be accounted for by applying the mean minus one standard deviation value (m-Φ). Its effect is to reduce load/strength ratios in Appendix B to about 60% of the tabulated values. Ratios between 1.0 and 1.7 then reduce below unity. For example, if (m-Φ) is applied to the Gilberd building, the column earthquake shear/shear strength ratio would no longer predict shear damage (the ratio 1.31 reducing to 0.77), but wall overturning ratios of 20.4 and 4.98 would obviously still greatly exceed unity.

Variation in materials properties is another, but somewhat lesser uncertainty. As has been already mentioned, full working drawings with reinforcing layouts were obtained for only one building, with reinforcement ratios typical of that period assumed elsewhere. A median reinforcing yield stress of 280 MPa was assumed for these pre-1930 buildings, however for post-1930 construction the Green Book [8] considers 280 MPa to be the lower five percentile value. Therefore it is possible the bending and shear strength results we report might be slightly low.

2. RESULTS

Results are expressed as ratios of maximum structural actions to probable shear and bending strengths, and wall resisting moments. A summary of all ratios for ground floor columns, first floor level beams and walls is collated into a single table (Appendix C).

2.1 Overturning of reinforced concrete and masonry infill structural walls

Figure 4 shows the cumulative frequency of overturning to restoring moment ratios for all structural and infill walls using the calculated Hawke's Bay earthquake seismic load. Over 85 percent of all walls should have overturned or at least rocked (ie. ratio of overturing moment to resisting moment > 1). Over 50% of walls have ratios > 3. If earthquake loads are reduced by (m-Φ), all transverse walls and 50% of longitudinal walls should have rocked. Although rocking or overturning of walls is not mentioned in any damage reports it is possible that rocking could have occurred without being apparent afterwards, especially given a probable unawareness of this form of dynamic response in the 1930s.

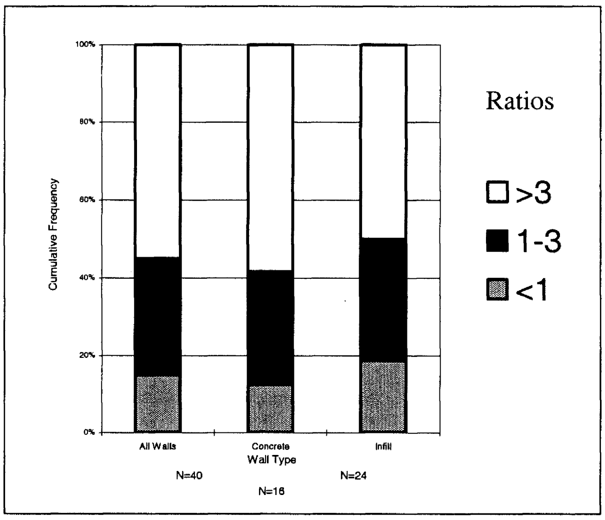

Figure 4. Cumulative frequency chart for ratios of wall overturning to restoring moments

The beneficial effects of wall rocking are well recognised. For example, the following advice from FEMA [11]; "*Check that the calculated resistance* (restoring moment) *is greater than 0.75 times the base moment of the shear wall*", sanctions the acceptability of an overturning moment to restoring moment ratio up to 1.33.

2.2 Shear Strength of Walls

Applied shear force to shear strength ratios for reinforced concrete structural walls and reinforced concrete frame masonry infill walls are summarised in Figure 5. The reinforced concrete walls have shear strengths that in 70% of cases was greater than the maximum shear forces. Assuming a (m-Φ) reduction in load, a total of four walls would have been expected to suffer shear damage, however wall rocking is expected to have occurred well beforehand, reducing load levels.

Masonry infill panels exhibit three possible damage modes; horizontal sliding shear failure of the infill, diagonal tension and compression strut failure or sliding shear failure of adjacent reinforced concrete columns. For all infill walls where damage was expected to occur, five walls should have experienced horizontal sliding shear of the infill panel and three should have failed in diagonal tension. Diagonal tension failure does not necessarily equate to complete failure if tension forces can be redistributed to and resisted by diagonal compression struts. Sliding shear failure of adjacent column tops was found not to be critical.

The fact that both types of wall performed so well can first of all be attributed to several factors relevant to wall overturning that have already been discussed, namely that walls may have had their shear demand reduced by rocking.

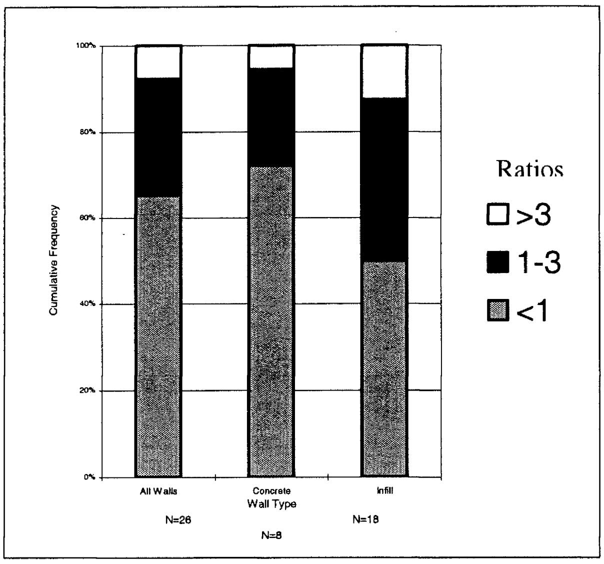

Figure 5. Cumulative frequency chart for ratios of applied shear force to probable wall shear strength.

2.3 Moment-Resisting Frames

The actual performance of frames was better than suggested by the analyses. Figure 6 summarises typical ratios of maximum shear force to probable shear strength, and maximum bending moment to probable bending strength for columns at ground floor level and beams at level 1 in open moment-resisting frames.

Approximately 50 percent of columns had their shear strength exceeded and 63 percent had their bending strength exceeded. Beam shear and bending ratios were exceeded in 60 percent of cases. If the (m-Φ) factor is applied, four column sets would be expected to yield in bending. In two cases, early flexural yielding would have prevented expected shear damage. However, apart from reports of column hair cracks in one frame, no structural damage to columns or beams was reported for these buildings. It must be recalled though, that for *all* frames (including many not in this study) it was observed that *"most of the damage.. ...occurred in columns, and particularly ground floor columns"*. The frames subject to this study clearly performed better during the earthquake than some others (since demolished).

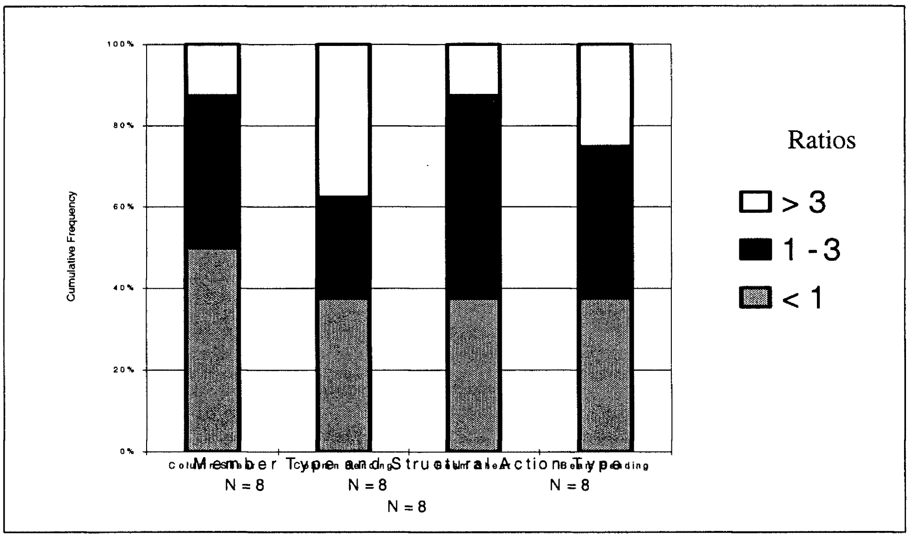

Figure 6. Cumulative frequency chart for ratios of applied shear force to shear strength and bending moment to bending strength of moment resisting frames.

Results for moment-resisting frames are more uncertain than those for walls due to the far greater lateral flexibility of frames when compared to any URM walls whose influence has been neglected. In some cases, frames resist seismic load as part of a reinforced concrete frame-masonry wall hybrid system. Even where frame infill panels are highly penetrated, masonry will usually make some additional contribution to frame strength due to steeply inclined masonry compression struts or composite action.

2.4 Asymmetric buildings

As shown in Table 1, thirteen of the twenty-five buildings are moderately to strongly asymmetric in plan, yet surprisingly only one of these buildings suffered significantly more damage than the rest. Excluding the Napier Public Trust building, the mean damage ratio for the other twelve asymmetric structures is about 0.003; as compared with about 0.002 for symmetric structures. (These damage ratios are for concrete structure damage only, as given in Table 1.) This finding however is consistent with that of Dowrick [14] for corner buildings. Considering all eleven past New Zealand earthquakes with intensity ≥ MM8 that have affected low-rise pre-1976 concrete buildings, corner buildings suffered no more damage than other concrete buildings. The only explanation that we can offer for this curious observation is that buildings of up to three storeys may not be tall enough to fully develop the torsional forces and displacements expected from their asymmetric plan layouts.

3. APPROXIMATE ASSESSMENT PROCEDURE

The final aim of this study was to investigate a simplified building seismic assessment method based on the cross-sectional area of vertical reinforced concrete structure expressed as a percentage of the gross ground floor area. In order to determine the minimum structure required to resist earthquake loads of a given intensity, data from both damaged and undamaged buildings are needed. Since only one building in this data set suffered significant structural damage (in both orthogonal directions), the minimum percentage of structure required for this event to avoid serious structural damage cannot be determined.

The vertical structure area to floor area percentage is a useful means by which to begin to assess seismic structural integrity. Glogau [16], in a study of post-earthquake Japanese buildings, states that the minimum structural footprint should be 0.3n percent, where n is the number of storeys above the level being considered (but not less than 2), which equates to 0.6 and 0.9 percent for two and three–storeyed buildings.

Structural footprint areas expressed as a percentage of gross ground floor areas for all buildings studied as listed in Table 1 are presented graphically in Figure 7. Footprint percentages are calculated for each orthogonal direction separately.

The following observations and comments can be made on these structures given they survived the earthquake.

Reinforced concrete walls

A structural footprint of 1 percent provided excellent structural performance.

- Infill frames

  The building with percentages of 0.1% and 0.2% (in each orthogonal direction, excluding the area of masonry infill) did experience minor structural damage. Infill frames of 0.3% (1.0% if the masonry area is included), performed well.
- Open moment-resisting frames

  Frames with a structural footprint of 0.4 percent or greater performed well.

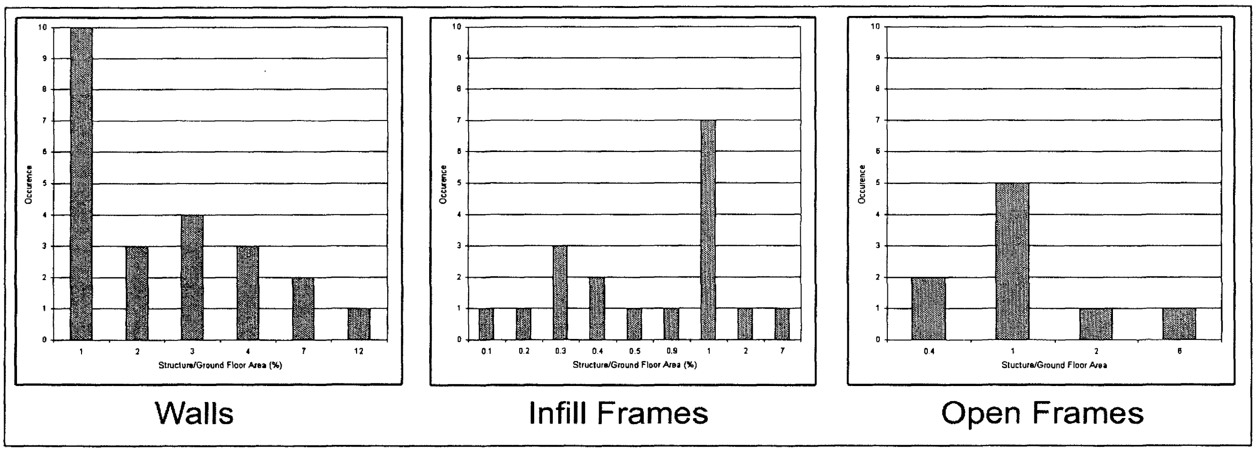

Figure 7. Percentages of structural footprints to ground floor areas.

4. CONCLUSIONS

4.1 General

Of all the twenty-five reinforced concrete buildings in this study (survivors of very strong shaking in the 1931 Hawke's Bay earthquake that still exist), only seven are known to have suffered even slight damage to reinforced concrete structure. Damage to masonry walls, not necessarily surrounded by a reinforced concrete frame, occurred in seven buildings. Given that the level of earthquake load was between 134% and 243% of current code depending on building location, these buildings performed very well. This is surprising, given that detailed structural analyses in accordance with the Green Book [8] indicate significant numbers of structural members in these buildings should have suffered damage under the calculated earthquake loads. Findings suggest that application of current seismic code requirements to reinforced concrete buildings of this type and era could be conservative, unless account is taken of the presence of URM elements, wall rocking, the beneficial effects of adjacent buildings, and low levels of ductility reducing seismic response.

4.2 Asymmetric buildings

Very large eccentricities due to wall layouts made little difference to the response and failure rate of buildings up to three storeys high included in this study.

4.3 Reinforced concrete structural walls

Most walls analysed in this study should have overturned or rocked on their foundations at a (m-Φ) load level. As mentioned previously, there was no evidence of this happening, but it is possible that some rocking could have occurred without leaving any visible record. In spite of high values of overturning to restoring moment ratios, these buildings exhibited excellent structural performance during the earthquake. It would therefore appear reasonable that in assessing and strengthening similar types of buildings, explicit recognition of the acceptability of wall rocking be given.

4.4 URM infill frames

Masonry was damaged in five out of twelve buildings with infill panels. In some cases, infill panels made a very positive (vital) contribution by preventing collapse.

4.5 Moment-resisting frames

In many cases columns and beams, at ground floor and Level 1 respectively, were estimated to have exceeded bending and shear strengths, however no such damage occurred. If the earthquake load is reduced by (m-Φ) to about 60% of the median value, flexural damage to 50% of beams and columns would be expected and one set of beams would fail in shear. The influence of minor URM infill partition walls, not taken into account in this study, together with low levels of ductility demand may have reduced seismic actions enough to have avoided visible damage.

4.6 Approximate Assessment Procedure

An assessment method based on the percentage of structural footprint to ground floor area could not be completed due to insufficient data. However, the following structural footprint percentages (for each orthogonal direction) performed with acceptable structural damage in this event:- structural walls 1%, reinforced concrete frame infill walls 0.3% (or 1.0% including masonry infill area) and open moment resisting frames 0.4%.

5. ACKNOWLEDGEMENTS

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APPENDIX A Reinforced concrete buildings not analysed in this study:

In addition to the 25 buildings analysed in this study (Table 1), a further 68 reinforced concrete buildings in Napier and Hastings in 1931 were identified, as listed here in Table A1. These buildings were not analysed partly because there was insufficient information available on their structural properties.

Enough information was available about them to determine that most of them performed well in the earthquake. Of the 68 buildings 24 were one-storey, 30 were two-storeys and 14 were three to five-storeys. One of these buildings collapsed completely, and one partially. Of the remaining 66 buildings 20 exist today, 30 more were slightly damaged and reinstated, 10 were intact after being burnt out, and the other six were damaged enough not to be reinstated. One of the latter, Hannah's 3-storey building is illustrated in Reference 1 (Figure 11(b)), and the Nurses Home which collapsed is shown in Figure 11(a) of that paper.

Table A1. **List of 68 reinforced concrete buildings and their damage states in Napier and Hastings in 1931 that are additional to those analysed in this paper.**

Building	Status (1)	No of storeys	Walls (2)	Damage State
Napier				
Cranby & Co: warehouse	G	2	C	In use for many years after earthquake.
The Tribune: offices	G	1	C	Undamaged.
Bate & Bell: garage	G	2	C	Columns cracked. Reinstated.
Ball & Co: printer	E	1	C	Window and ceiling damaged.
Peach & Co: garage	E	2	C	Shear crack in side wall.
Anderson & Hansen: garage	G	2	CB	Shell standing after fire.
HB Farmers: garage	E	2,1	CB	Some damage, but reoccupied immediately after earthquake.
HB EPB Bldg 1	G	2	CB	Moderate damage repaired.
HB EPB Bldg 2	G	2	CB	Moderate damage repaired.
Golding Bros: offices	G	2	C	Slight damage.
F Neal's shop	G	2,1	CM	Slight damage. Replaced in 1988.
Briasco's shop	E	2	CB	Shell ok after fire. Reinstated.
Humphrey's shop	G	2	C	Shell ok after fire. Reinstated.
Hurst Building	G	2	C	Ok before fire. Reinstated.
E&D Building	G	3	C	Virtually undamaged by shaking. Replaced 1984.
Central Hotel	G	2	C	Ground floor MRF ok after fire.
HB Farmers: shop	G	1	C	Shell intact after fire.
Cosmopolitan Club	G	2	C	Shell intact after fire.
C Pointon: shop	G	2	C	Front ok before fire.
HB Friendly Society: shop	G	2	CB	Walls intact after fire.
Napier Power House	E	1	C	Any damage repaired. Buttresses added.
Borough Engineers Office	E	1	C	Any damage repaired.
Napier Tech School Workshop	E	1	C	Any damage slight.

Blythes: shop	G	2	CB	Safe to reoccupy after earthquake. Replaced 1997.
Napier Post Office	E	3	C	Intact after fire. Reinstated.
Caledonian Hotel	E	3	C	Small repairs to south and east walls.
Hannah & Co: shop	G	3	CBl	Columns shattered. Concrete blocks ok.
Thorp & Co: shop	G	2	C	Frontage ok after fire.
HB DSA: shop	G	2	CS	Frontage ok before fire.
Style & Styles: shop	G	2	C	Walls intact after fire.
Carlile, McLean: office	G	2	C	Windows and partitions cracked.
W Anderson: shop	E	1	C	Slight damage.
H Williams & Sons: warehouse	G	1	CB	Partial collapse.
Soldiers Club	E	1	C	Slight damage.
Dr Moore's Hospital (S)	G	3	CB	Intact.
Dr Moore's Hospital (E)	G	3	CB	Tilted. Earthquake occurred during underpinning.
Dr Moore's Hospital (N)	G	3	CB	Near collapse.
Dr Moore's Hospital (W)	G	2	CB	Intact.
Municipal Baths	G	1,2	CB	Some brickwork fell. Reinstated.
Hukarere School (1)	?	1	C	Ok.
Hukarere School (2)	E	3,4	C	Moderate cracking of walls. Reinstated.
Acetone: factory	E	1	C	Bad cracks.
Nurses Home	G	3,4	C	Soft storey. Collapsed.
Empire Hotel	E	3	CB	RC end ok, brick end fell. Reinstated.
Rayment's garage	E	1	CB	Walls standing after fire. Reinstated.
McCulloch et al. Office	E	1	CB	Walls ok after fire. Reinstated.
HB Steam Co	G	1	C	No visible damage. Operational.
Napier Waterworks	G	1	CB	Brick gable fell.
Williams & Creagh: warehouse	E	1	CB	Damage slight.

Table A1 Continued

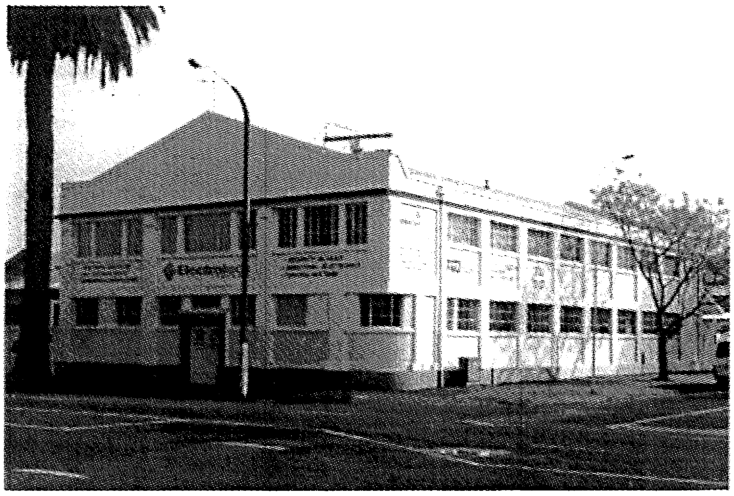
Building	Status (1)	No of storeys	Walls (2)	Damage State
Napier				
Ellison & Duncan: warehouse	G	2,1	C	Walls badly cracked after fire.
Ellison & Duncan	G	1	C	Concrete walls needed some repairs.
Selby: shop	G	1	CB	Brick parapet fell through roof.
Hastings				
Ritz Tearooms et al.	G	2	CB	Brick parapet and gable fell.
Commerce Buildings	G	2	C	Cracked but repairable.
Pacific Hotel	G	2	C	Bad cracks. Reinstated. Replaced c. 1985.
Heretaunga Buildings	G	2	C	Several big cracks. Reinstated?
McKenzie's shop	E	2	CB	Frontage intact. Continued in use.
Hennah's buildings	G	1	CB	Brick parapet, otherwise ok.
Carberry et al: shop	G	1	C	0.3 m sidesway of front columns.
Farmers et al: shop	G	1	C	Frontage ok.
Racecourse grandstand	G	1	CB	Minor damage.
Heretaunga Co-op Dairy	G	1	C	Ok including plant.
Ebbett & Gifford: offices	E	2	C	Several cracks. Reinstated.
Tonkin's: offices	G	2	CB	Slight damage.
Jesmond House: flats	E	1	C	Small cracks. Reinstated.
Nelson's Freezing Works	G	3	CB	No visible damage.
Nelson's Freezing Works	G	4,5	C	Small cracks, windows broken.
Nelson's Freezing Works	G	3	CB	Brick gables fell.

Notes: (1) E = Building Exists in 1999; G = Building Gone in 1999 or earlier.

(2) C = Reinforced Concrete; B = Brick; Bl = Concrete blocks; S = steel; M = Brick or Concrete

APPENDIX B Example of building description, structural analysis results and structural skeleton drawing:

Building Description



Building Name: Gilberd & Co
Address: 47 Kennedy Rd, Napier
Date of construction: 1927

Primary lateral load resisting structural systems:
Transverse – Two reinforced concrete frames with reinforced concrete spandrels and two internal single storey reinforced concrete walls.
Longitudinal – Two reinforced concrete frames with reinforced concrete spandrels. **Construction:** The intermediate floor is timber, supported by steel joists and steel columns. The roof is of lightweight construction (timber and corrugated steel sheeting).
Recorded notes on damage: No visible damage (Dowrick notes, 1998).

Assumptions made in modelling:

X All spandrels are of reinforced concrete and are modelled as beams;

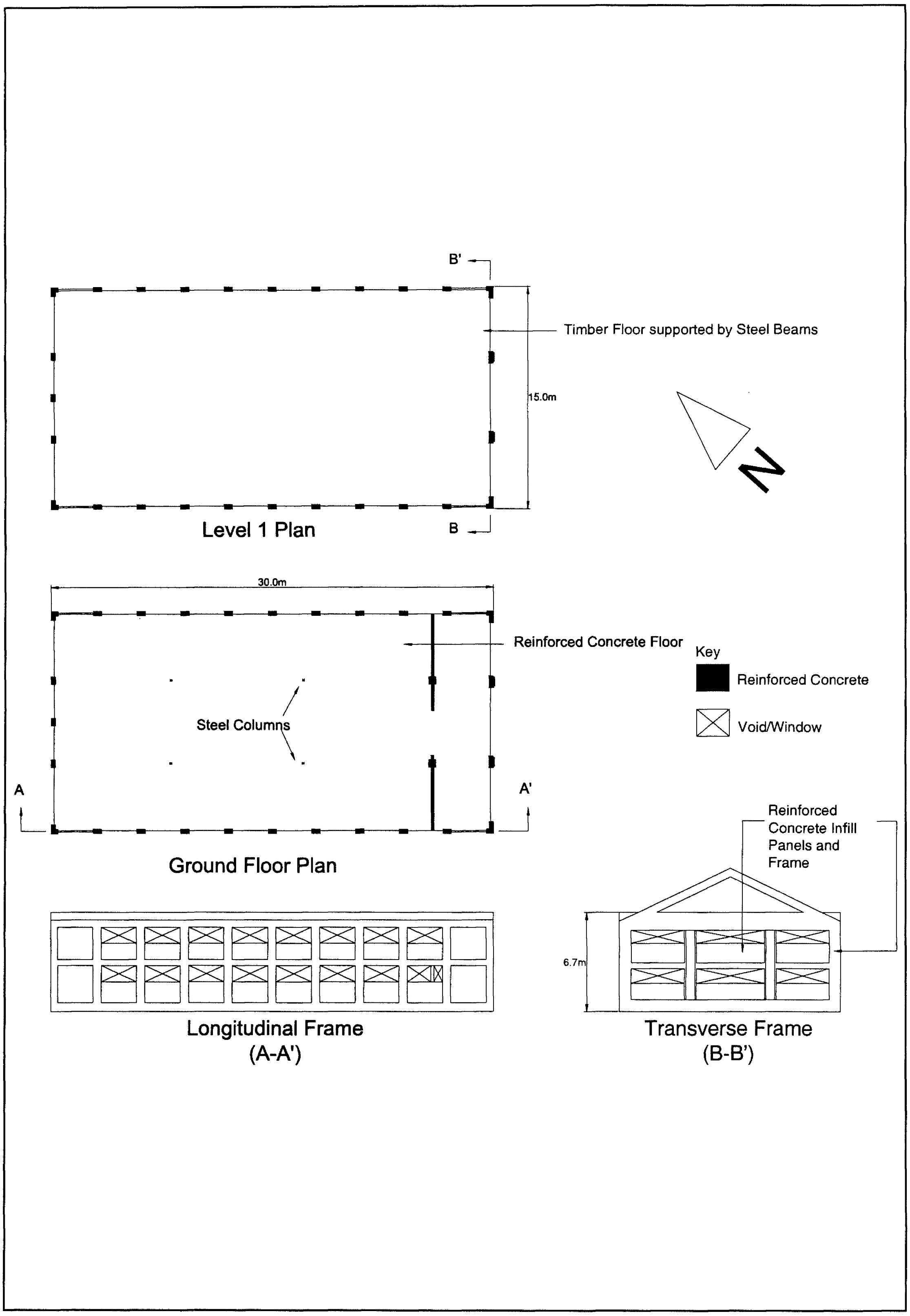
X The transverse facade has a gable end of 125mm reinforced concrete. The gable is modelled as beams of increasing depth. All beams and columns have appropriate rigid offsets from the joints; and

X The longitudinal facade is modelled as a frame that includes walls in the end bays.

<u>Seismic Loads:</u>

The variables for the seismic loads for the transverse and longitudinal directions are summarised in the table below.

μ	C	W_t (kN)	V (kN)
1	1.14	3527	4021


B'
Timber Floor supported by Steel Beams
15.0m
N
Level 1 Plan
B
30.0m
Reinforced Concrete Floor
Key
Reinforced Concrete
Void/Window
Steel Columns
A
A'
Ground Floor Plan
Reinforced Concrete Infill Panels and Frame
6.7m
Longitudinal Frame (A-A')
Transverse Frame (B-B')

Gilberd & Co.

Results:

Summary of results of transverse frame for Gilberd & Co. building:

Member Type	Prob Shear Strength (kN)	Prob Bending Strength (kNm)	Wall Restoring Moment (kNm)	EQ + Gravity Shear Force (kN)	EQ + Gravity Bending Moment (kNm)	EQ + Gravity Overturning Moment (kNm)	EQ + Gravity Shear Force/Prob Shear Strength	EQ + Gravity Bending Moment/Prob Bending Strength	EQ + Gravity Overturning Moment/ Wall Restoring Moment
Column, ground	109	163		143	150		1.31	0.92	
Beam, level 1	241	378		82	166		0.34	0.44	
Column, level 1	84	163		253	212		3.01	1.30	
Beam, roof	602	736		97	185		0.16	0.25	
Structural walls, ground	1670		232	1610		4730	0.96		20.4

Summary of results of longitudinal frame and walls for Gilberd & Co. building:

Member Type	Prob Shear Strength (kN)	Prob Bending Strength (kNm)	Wall Restoring Moment (kNm)	EQ + Gravity Shear Force (kN)	EQ + Gravity Bending Moment (kNm)	EQ + Gravity Overturning Moment (kNm)	EQ + Gravity Shear Force/Prob Shear Strength	EQ + Gravity Bending Moment/Prob Bending Strength	EQ + Gravity Overturning Moment/Wall Restoring Moment
Column, ground	225	288		170	99		0.76	0.34	
Beam, level 1	295	511		127	165		0.43	0.32	
Column, level 1	214	288		76	62		0.36	0.22	
Beam, roof	108	80		25	21		0.23	0.26	
Structural wall, end bay	904		133	255		662	0.28		4.98

Discussion:

Transverse: Structural walls resisted most of the seismic loads at ground floor. The overturning moments of both walls exceeded the probable restoring moments indicating that the walls may have rocked. There are no records to verify this mechanism. The ratios for the structural walls are unusually high because there is little floor weight supported by them, significantly reducing the restoring moments. The columns at Level 1 may have suffered some shear cracking though again there is no recorded evidence to support this.

Results Reliability Rating: A

Longitudinal: The majority of the seismic load was resisted by end bay walls stabilised by reinforced concrete spandrels.

Results Reliability Rating: B

APPENDIX C Results Table (For Ground Floor Members):

NAPIER					Columns				Beams				Walls					
	No. of Storeys	Gross Ground Floor Area (m^2)	Results Reliability Rating		EQ + Gravity Shear Force/ Column Prob Shear Strength Ratio		EQ + Gravity Bending Moment/ Column Prob Bending Strength Ratio		EQ + Gravity Shear Force/ Beam Prob Shear Strength Ratio		EQ + Gravity Bending Moment/ Beam Prob Bending Strength Ratio		EQ + Gravity Wall Shear Force/ Prob Wall Shear Strength Ratio		EQ + Gravity Wall Overturning Moment/ Wall Restoring Moment Ratio		% Reinforced Concrete Vertical Structure of Gross Ground Floor Area	
			Trans	Long	Trans	Long	Trans	Long	Trans	Long	Trans	Long	Trans	Long	Trans	Long	Trans	Long
Methodist Church Hall	2	256	A	A	*	*	*	*	*	*	*	*	0.77	3.62	2.21	5.18	1	0.4
Gilberd & Co	2	276	A	B	1.31	0.76	0.92	0.34	0.34	0.43	0.44	0.32	0.96	0.28	20.4	4.98	1	2
Hawke's Bay County Council	2+B	277	B	B	*	*	*	*	*	*	*	*	2.58	0.70	24.00	3.18	1	5
Public Trust Office (N)	2	502	B	B	*	*	*	*	*	*	*	*	1.90	1.66	4.41	2.88	1	1
Murdoch J H, Bakery	2/3	213	B	B	*	*	*	*	*	*	*	*	0.48	1.14	3.93	6.83	3	4
Dalgety & Co	2	456	B	B	1.66	*	5.52	*	2.70	*	2.75	*	*	0.71	*	2.55	1	1
Ancient Order of Foresters	2	599	B	B	*	*	*	*	*	*	*	*	0.14	0.39	2.43	20.8	4	4
Ringland Bros.	2	183	B	B	0.26	*	0.85	*	1.15	*	1.20	*	2.97	0.69	23.2	3.23	0.3	2
Union Hotel	2	453	B	B	*	*	*	*	*	*	*	*	2.01	2.85	5.11	23.2	0.4	0.3
Harston's Concert Hall	2	464	B	B	2.79	*	9.41	*	3.53	*	7.14	*	*	0.70	*	1.32	0.4	0.4
Bennett Building	3+B	265	B	B	3.42	*	11.27	*	1.87	*	1.19	*	*	0.35	*	1.08	1	3
Parker Chambers	3	594	C	C	*	*	*	*	*	*	*	*	2.89	2.29	7.00	0.64	1	1
Williams Buildings (I)	2	109	C	C	*	8.54	*	4.16	*	2.51	*	2.29	0.35	*	6.61	*	12	4
Williams Buildings (II)	2	195	C	C	*	3.05	*	6.55	*	3.98	*	4.96	1.18	*	2.93	*	1	1
Richardson & Co. Ltd.	2	465	C	C	2.07	*	3.33	*	0.94	*	0.96	*	0.72	0.50	2.27	1.50	1	3
Howe Bros.	2	172	C	C	1.79	*	5.17	*	2.95	*	7.84	*	*	1.64	*	1.69	2	1
West (JS) & Co	2	257	C	C	3.26	*	8.11	*	1.68	*	2.22	*	*	0.33	*	0.92	1	3
HASTINGS																		
Poppelwells	2	368	B	B	*	0.90	*	0.93	*	0.93	*	0.74	0.64	*	1.79	*	1	1
Dominion Buildings	2	416	B	B	*	*	*	*	*	*	*	*	4.47	0.31	14.9	0.71	7	2
Public Trust Office (H)	2	514	B	B	*	*	*	*	*	*	*	*	1.99	0.77	3.26	1.14	1	2
Rainbow, Hobbs & Nesbitt	2	117	B	B	0.54	*	2.66	*	1.10	*	3.16	*	0.86	0.25	3.49	0.93	1	7
H.B. Jockey Club's Racecourse	2	929	C	C	9.42	*	*	*	*	*	33.0	*	*	11.8	*	30.5	0.3	0.9
Hawke's Bay Farmer's Co-Op	3+B	944	C	C	2.17	*	3.94	*	1.81	*	2.18	*	*	0.55	*	0.52	0.2	0.1
Webber's Buildings	2	234	C	C	*	*	*	*	*	*	*	*	0.94	4.37	6.41	3.40	1	0.5
Villa d'Este	2	455	C	C	0.94	*	1.99	*	0.56	*	1.06	*	*	2.01	*	1.16	1	1
Mean Values					**1.66**	**0.83**	**5.11**	**0.64**	**1.78**	**0.68**	**2.65**	**0.53**	**1.65**	**1.03**	**7.55**	**4.93**		