RESPONSE OF LOW-RISE BUILDINGS TO MODERATE GROUND SHAKING, PARTICULARLY THE MAY 1990 WEBER EARTHQUAKE

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SUMMARY

In the Weber earthquake of 13 May 1990 the stronger component of the ground motions recorded in Dannevirke was similar in strength to the El Centro S00E record from the 1940 Imperial Valley earthquake which underlies the New Zealand loadings code. The Modified Mercalli intensity in Dannevirke however was only about MM7½, whereas the intensity corresponding to the 1984 earthquake code is about MM8½ for the Dannevirke area. This paper compares the strength of the Dannevirke record in terms of spectral accelerations with (i) the above El Centro record, (ii) the Matahina dam record of the 1987 Edgecumbe earthquake, and (iii) the loadings of the 1984 and 1992 New Zealand codes. Also described in the paper are time-history analyses of one- and two-storey buildings subjected to the above ground motions in an attempt to explain why the damage levels were lower than might be expected from the strength of the recorded accelerograms. Comparisons are made of the seismic performance of moment-resisting frames and walled structures. Comments are made on two of the provisions of the 1992 loadings code.

1. INTRODUCTION

The southern Hawkes Bay area experienced a series of earthquakes in 1990. The two biggest earthquakes of that sequence were (1) the magnitude M_L 5.9 ($M_S=M_W=6.3$) event of 19 February 1990, and (2) the M_L 6.2 ($M_S=6.2$, $M_W=6.4$) event of 13 May 1990 [1]. Both events were centred near Weber and for convenience we refer to them as Weber I and Weber II respectively. Weber I had a focal depth of 24 km with estimated centroid of rupture surface 28 km deep while Weber II had its focal depth at 12 km and centroid of rupture surface 13 km deep. The predominant source mechanism of Weber I was strike-slip and of Weber II was reverse.

The closest accelerograph to the fault ruptures of the two events was located in Dannevirke. The first event was recorded at Dannevirke Telecom at a horizontal distance of 26 km from the rupture surface with peak horizontal ground acceleration (PGA) of 0.3g (polar peak) and 5% damped peak spectral acceleration (SA_{max}) of 0.96g (in the S23E direction). The Weber II earthquake was recorded at Dannevirke Post Office at 21 km horizontal distance from the rupture surface. The PGA of the Dannevirke Post Office record was 0.4g (polar peak) with SA_{max} = 1.14g (in the N67E direction). The strength of ground motions recorded, in particular in the Weber II earthquake, imply responses well into the non-linear range for some structures, so it was surprising that relatively little damage occurred near this recording site in Dannevirke. This study sets out to examine and explain this situation.

2. ATTENUATION MODELLING

In a recent paper [1], we have presented plots of the PGA data from these two earthquakes, together with regression analysis curves, with that for Weber II shown here in Figure 1. The comparisons between PGAs and mean regression curves show that there is nothing unusual about the Dannevirke recording in relation to the rest of the PGA data from the Weber events. Similarly Robinson's [2] Weber earthquake isoseismal maps (e.g. Figure 2), fit the Dowrick attenuation model [3] for New Zealand earthquakes reasonably well (e.g. Figure 3), suggesting that there was nothing unusual about the damage levels in Dannevirke in this event.

3. DESCRIPTION OF DAMAGE

The epicentral area around Weber is rural. Dannevirke (population c.5800) is the nearest significant township to the earthquake source. The damage in the Weber and Dannevirke areas was minor in Weber I. There was isolated damage to masonry chimneys and some damage to display windows in the main street of Dannevirke [4]. The isoseismal map of the Weber I event gives MM intensity of 7 for Weber and about 6 for Dannevirke [2].

In the May Weber event, five houses suffered moderate to heavy damage in and around the Weber area. Perrin in his reconnaissance report [5] suggests that the ground shaking at the locations of all of these houses would have been amplified due to either topographical effects or due to the presence of soft material near the ground surface. Two bridges located between Dannevirke and Weber suffered minor cracking. In

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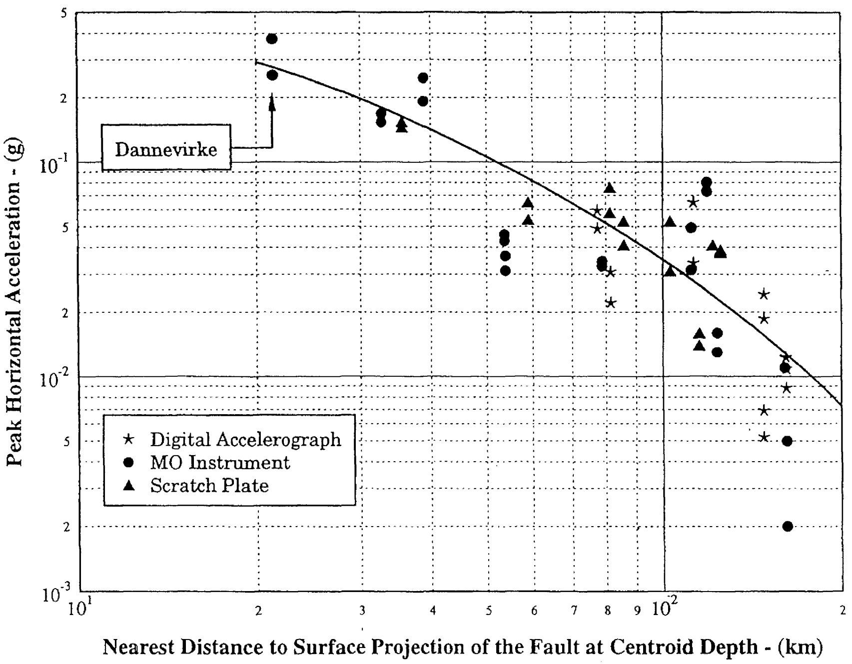


FIGURE 1 *Attenuation plot of PGA data (both horizontal components) for the Weber II earthquake, with mean regression curve for the stronger of the two horizontal components, as discussed elsewhere [1].*

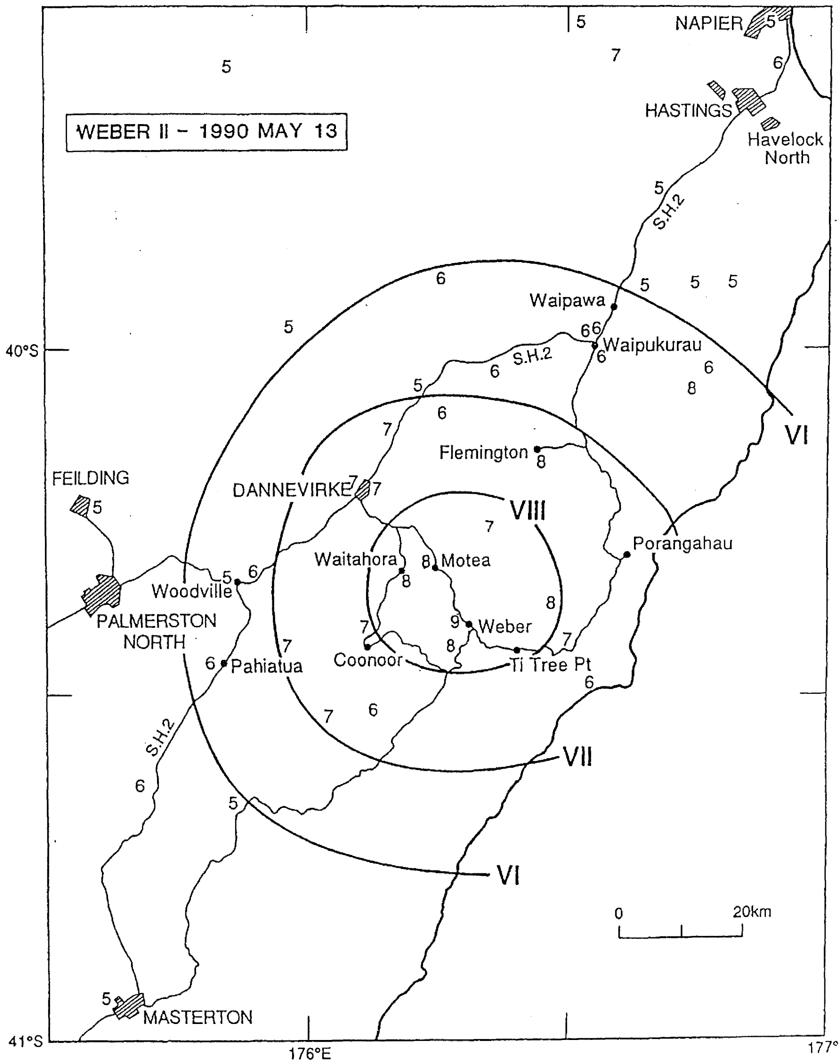


FIGURE 2 *Modified Mercalli intensities and isoseismals of the Weber II earthquake (after Robinson [2]).*

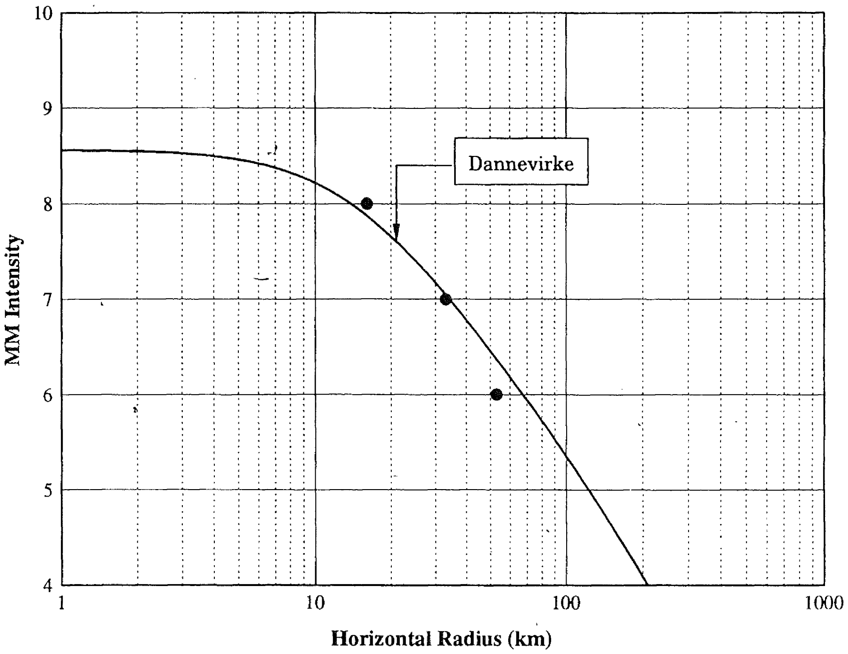


FIGURE 3 Comparison of the isoseismal radius data of Weber II (Figure 2) with prediction of the Dowrick attenuation model.

Dannevirke, damage to housing was slight. The commercial buildings are either single or two storeys high. A majority of these buildings are of old pre-code era construction, predominantly of unreinforced brickwork. Some of the brick buildings have reinforced concrete ring beams, but many are entirely unreinforced. Some of the brick buildings were badly damaged; six have subsequently been demolished, while about 12 have required moderate repairs, mainly to parapets. In a recent paper, Johnstone and Potangaroa [6] provided descriptions of the damage suffered by the most badly affected buildings.

Well-built buildings, regardless of age, suffered little or no damage. For instance, the building housing the accelerograph at the time of the May 1990 earthquake, a two-storey 1987 reinforced masonry (concrete block) building, was undamaged. We also note that a c.1930 single-storey brick building adjacent to the Post Office was also undamaged. This building had a concrete ring beam at eaves level. We have included these two buildings in our analysis presented in Section 5.

The isoseismal map for Weber II (Figure 2) and the attenuation model (Figure 3) both place Dannevirke about half way between the MM7 and MM8 isoseismals, so that here we shall refer to the intensity in Dannevirke as MM7½.

4. GROUND MOTIONS IN DANNEVIRKE

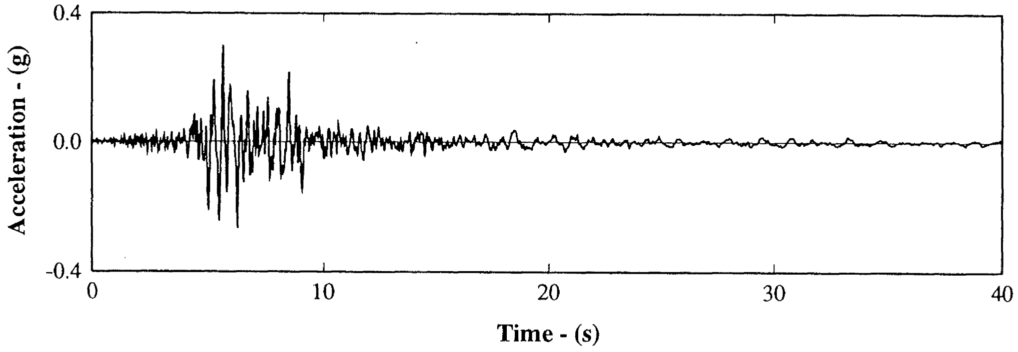
The ground conditions in Dannevirke are generally stiff. Perrin [5] states that "the town is on a wide alluvial terrace with cobbly gravels, sand and silt to a depth of up to 30 m, overlying about 2000 m of conglomerate, sandstone and mudstone of tertiary age. The gravels are dense and the tertiary rocks do not cause large amplification of shaking."

The digitized records of the ground motions recorded in the two Weber earthquakes at various sites by the accelerograph network have been published by Cousins *et. al.* [7, 8]. The earthquake records from Dannevirke were processed in the directions parallel to the instrument axes, i.e. in the N67E and S23E directions. The main street in Dannevirke is aligned with the N67E direction.

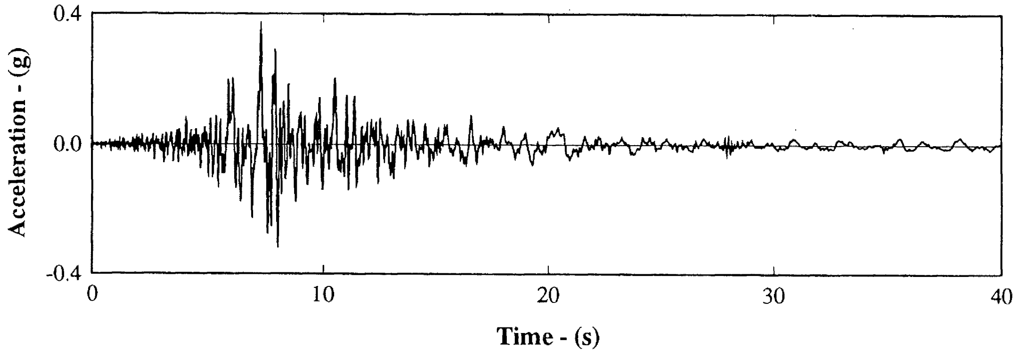
The recorded earthquake motions in Dannevirke were typically directional in the two events. In the Weber I event the S23E component was substantially stronger than the component parallel to the main street, while in the Weber II event the opposite occurred. Figure 4 depicts the stronger of the two horizontal acceleration histories obtained in the two events. Of the four horizontal components in the two events, the N67E component of the Weber II event generally produced the largest spectral accelerations, except in the period range of about 0.3 to 0.5 s and in a narrow period band at very short periods [7, 8]. In this paper we have used both of the Weber II components concentrating on the stronger N67E component, but also briefly discussing the Weber II S23E component because it appears likely to be more representative of the ground motions in Dannevirke. This is suggested by Figure 1, where it is seen that the S23E PGA fits the mean regression curve better than the other component. A more typical PGA for Dannevirke may be lower than the weaker S23E component, as a preliminary New Zealand PGA attenuation model currently under development by Dowrick gives a mean PGA of approximately 0.2g for Dannevirke in a model of the Weber II event.

4.1 Comparison of Elastic Response Spectra

The 5% damped elastic response spectra calculated for the Weber II earthquake are compared to some other spectra. For this purpose, we have chosen the El Centro record from the 1940 Imperial Valley earthquake (which produced damage in El Centro [9] very similar in nature to that in Dannevirke), the record from the base of the Matahina Dam in the 1987 Edgecumbe earthquake (M_S = 6.6, M_W = 6.5 [1]), and the New Zealand loadings code spectra of 1984 [10] and 1992 [11].



(a) S23E component in Weber I



(b) N67E component in Weber II

FIGURE 4 The stronger horizontal components of the acceleration time-histories recorded at Dannevirke in Weber I and Weber II.

Comparison of these three ground motions is also reasonably direct because the ground conditions at the recording sites for the above records are reasonably similar, all being alluvial in the firm to stiff range (see above and Ref. [12]), as evidenced by the broad similarity of the spectral shapes (Figure 5).

The stronger component (N67E) of the Dannevirke recording for Weber II and the widely used S00E component of the El Centro record are close to the directions in which the ground motions gave the largest spectral ordinates over most of the entire period band (0 - 2.0 s). We selected the N53W component of the Matahina record because the spectral accelerations in this direction are close to their maximum values irrespective of the direction, except for short periods up to 0.2 s. The components of the Matahina record used elsewhere were in the N83E and N07W directions, parallel to the axes of the instrument (and those of the Matahina dam) [12, 13].

The 5% damped acceleration response spectra for the N67E and S23E components of Weber II, the S00E component of the El Centro record, and the N53W component of Matahina Dam record are compared in Figure 5. The S23E component of the Dannevirke record contains strong short period peaks, but produces the weakest spectrum for periods above 0.3 s. The strength of the Dannevirke N67E component in Weber II is generally comparable to the El Centro spectrum with prominent peaks appearing at different period bands. The local magnitude of the 1940 Imperial Valley earthquake was re-evaluated by Trifunac and Brune [14] as $M_L = 6.4$ with $M_S = 7.1$. They suggested that the difference of 0.7 of a unit between M_L and M_S is due to the fact that the earthquake was a multiple event which is likely to affect the M_S value determined from long-period waves, while the M_L value calculated from short-period waves will more closely represent the magnitude of a single event in the sequence. For the period range discussed in this study, ie. up to 2.0 s, the spectral ordinates calculated for the El Centro record were governed by the ground motion corresponding to the first 9 s. Trifunac and Brune reported that in the 1940 event there were two sub-events with M_L values in the range of 5.9-6.2 with epicentres approximately 7-23 km from the recording site in this short time interval. These magnitude and distance ranges of the 1940 Imperial Valley earthquake together with the soil conditions at El Centro are similar to those of the Dannevirke record.

The shape of the spectrum calculated for Matahina is different from Dannevirke N67E and El Centro up to about 0.85 s with the predominant spectral peak appearing around 0.4 s. The Edgecumbe earthquake had a focal depth of 8 km [3] and ruptured to the ground surface. The ground motion was recorded 11 km from the fault rupture at the base of the large Matahina earth dam. Although this earthquake caused significant damage in Edgecumbe and other nearby townships, there was very little damage at Te Mahoe village, adjacent to the dam.

The stronger N67E Dannevirke elastic response spectrum is compared in Figure 6 with spectra from the 1984 and 1992 loadings codes. The 1984 code elastic spectrum was derived by applying combined structural (S) and material (M) factors of SM = 4 to the basic spectrum for "rigid and intermediate" subsoils. SM = 4 is appropriate for elastically responding reinforced

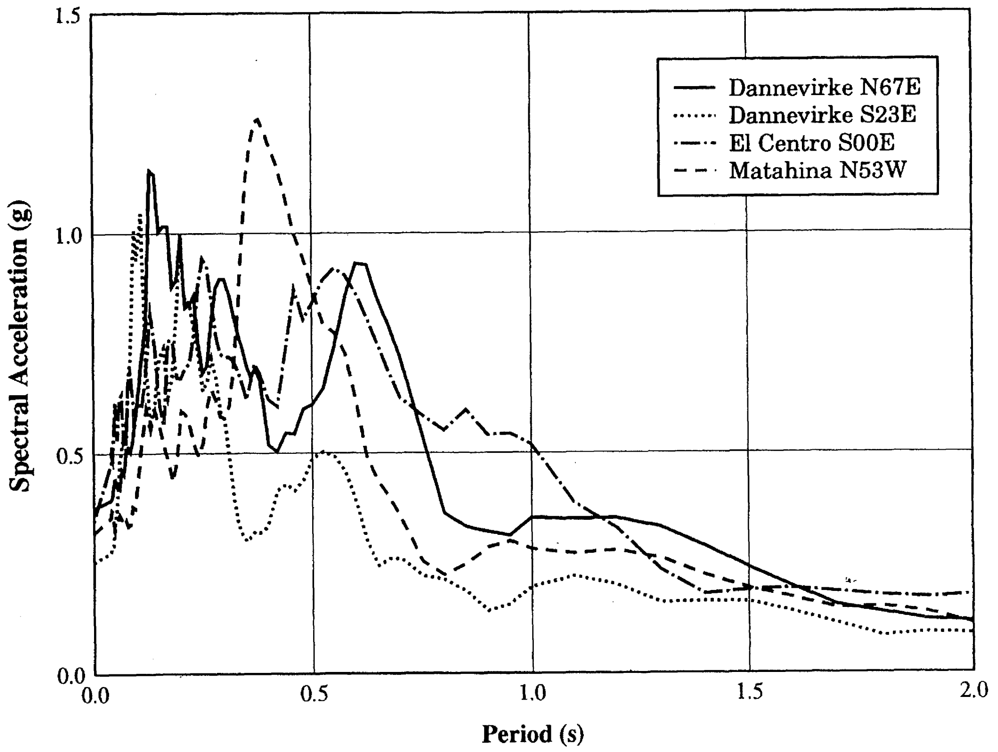


FIGURE 5 *Comparison of the 5% damped elastic response spectra for Dannevirke (N67E and S23E components of Weber II), Matahina (N53W) and El Centro (S00E).*

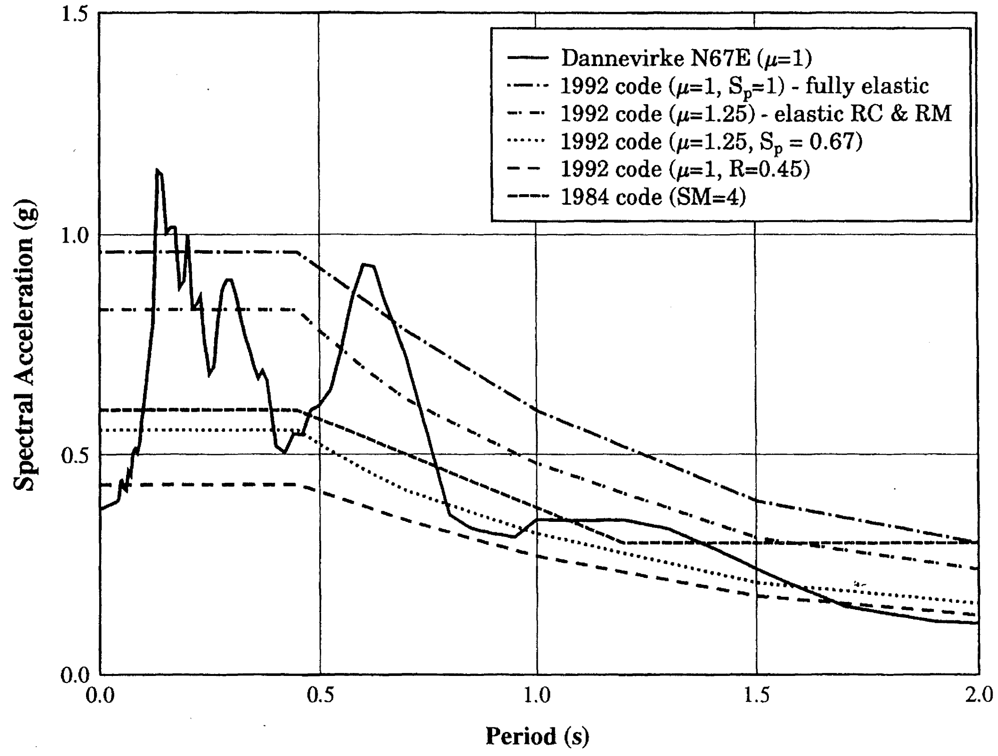


FIGURE 6 *Comparison of the 5% damped elastic response spectrum for the Weber II Dannevirke N67E record with various loadings spectra from the 1984 and 1992 New Zealand codes.*

concrete (RC) and reinforced masonry (RM) structures. For the 1992 code, four spectra are plotted in Figure 6, derived from the basic spectrum for "intermediate soils". The three strongest of these 1992 code spectra use different combinations of ductility factor (μ) and structural performance factor (S_p), as follows:

- "Fully" elastic: $\mu = 1.0$, $S_p = 1.0$
- Elastic RC and RM: $\mu = 1.25$, $S_p = 1.0$
- Elastic RC and RM (design): $\mu = 1.25$, $S_p = 0.67$

Comparing the Dannevirke elastic response spectrum (N67E) with those of the codes for low rise buildings ($T \leq 0.7$ s), it is seen that the Dannevirke spectrum is slightly stronger than the fully elastic 1992 code spectrum at $T \approx 0.2$s and $T \approx 0.6$s, but is a lot weaker at $T \approx 0.4$s. Over the period range $T \leq 0.7$s the Dannevirke spectrum has an average strength equal to that of $\mu = 1.25$, $S_P = 1.0$ curve (the latter would be appropriate for RC or RM structures if $S_P = 1.0$).

The third 1992 code spectrum in Figure 6 is that recommended for design for RC and RM structures using elastic model analysis ($\mu = 1.25$, $S_P = 0.67$). This spectrum is about 10 percent weaker than the equivalent spectrum from the 1984 code, while the Dannevirke spectrum is substantially stronger than both these code design curves over most of the period range $T \leq 0.7$s. Thus the question arises: Does the Dannevirke spectrum really match the 1992 code which is meant to represent a shaking strength for 450 years return period (about MM8½), or is the Dannevirke spectrum representative of the MM7½ which was typical of Dannevirke in the earthquake?

As part of this study, it was found that since 1840 Dannevirke has experienced intensity MM7 at average intervals of about 20 years. This matches the MM7 return period predicted by an unpublished seismic hazard model using a slightly revised version of the Dowrick attenuation model [3]. For MM7½ this hazard model predicts a return periods of 50 years. The 1992 code spectrum may be scaled from 450 years to 50 years return period by multiplying it by the risk factor R = 0.45 (Figure C4.6.1 of Ref [11]). As shown in Figure 6, the 50 year code spectrum falls well below the Dannevirke spectrum. Thus we need to evaluate the strength of this earthquake by comparing theoretical and actual structural responses.

4.2 Comparison of Inelastic Response Spectra

The next step is to compare the inelastic response spectra of the stronger components of the above three earthquake records and those of the 1984 and 1992 loading codes. When calculating the inelastic spectra, the stiffness characteristics of the oscillators were taken as bi-linear with a strain hardening ratio of 2.5 percent. The viscous damping of the SDOF systems was 5% of critical and the damping coefficients were calculated from elastic stiffness of the oscillators.

Figure 7 compares the $\mu = 6$ inelastic spectra calculated for the Dannevirke, Matahina and El Centro records with the 1992 and 1984 code spectra. The code spectra are for the soil types as noted in the previous section. The 1992 code spectra are those given for $\mu = 6$ with $S_P = 1.0$ and $S_P = 0.67$, whereas the 1984 code spectrum is obtained for material factor M = 0.8 (of reinforced non-prestressed concrete) and for structural type factor S = 0.8 (of ductile frames). The inelastic spectra of the Dannevirke and El Centro records are similar, crossing over

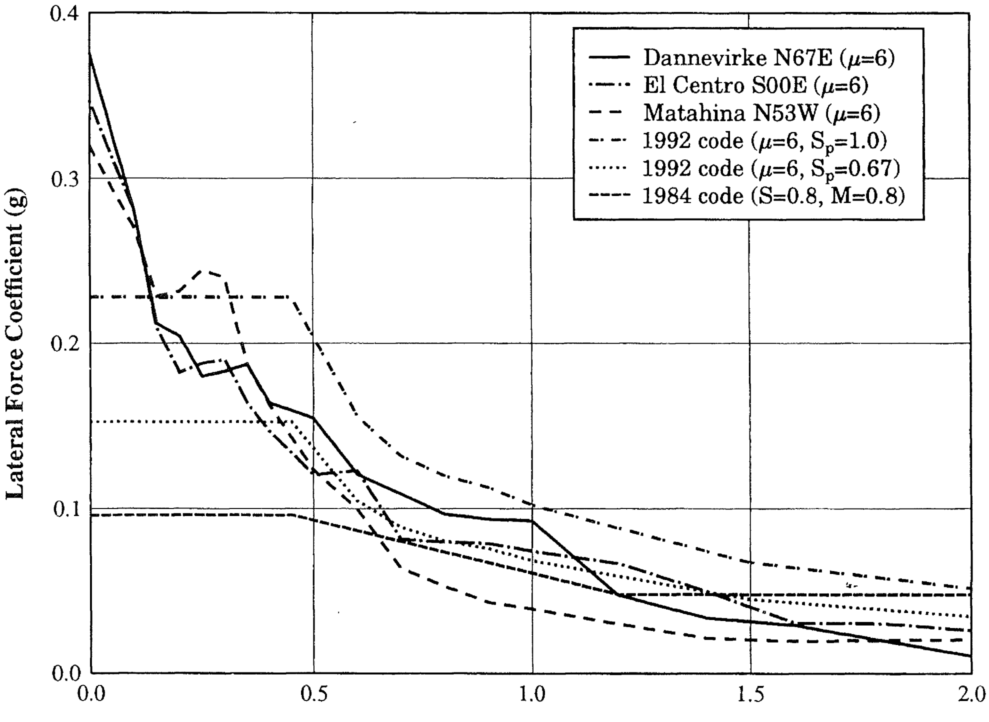


FIGURE 7 Comparison of the inelastic spectra for $\mu = 6$ for the stronger components of the Dannevirke (Weber II), El Centro and Matahina records. Also shown are the appropriate inelastic spectra of the 1984 and 1992 codes.

each other six times throughout the plotted period range. The Matahina inelastic spectrum shows stronger values between 0.15 and 0.35 s than the Dannevirke and El Centro spectra, but the former gives considerably smaller spectral values for periods from 0.5 to 1.75 s. The inelastic spectral ordinates of the three earthquakes fall below the 1992 code spectrum for $S_P = 1$, except at short periods for which a constant value was introduced in the code. The short period plateau of the 1984 code is 0.6 times that of the 1992 code spectrum for $S_P = 0.67$, and falls well below the spectra calculated for the three records up to about 0.5 s. Here the 1992 code *design* spectrum ($\mu = 6$, $S_P = 0.67$) is stronger than the 1984 code spectrum, which is the reverse of the situation for the elastic spectra (Figure 6).

The significantly large differences between the inelastic spectral accelerations of the Dannevirke record and 1984 code spectrum in the short period range (< 0.7 s) need to be investigated further in terms of the response of real structures. Three of the buildings discussed in Section 5 were designed to the 1984 code, which allows this issue to be investigated.

5. DYNAMIC ANALYSES OF BUILDINGS

Dynamic responses of four buildings to the ground motion recorded at Dannevirke in the Weber II event are discussed in this section. The results are compared with those obtained for the El Centro and Matahina records.

The buildings chosen for this study were (1) a pre-code era single-storey unreinforced brick masonry building (UBM); (2) the Dannevirke Post Office which is a two-storey reinforced concrete block masonry building (RM); (3) a two-storey bare frame moment-resisting frame (MRF) building, and (4) a building with the same frame as (3) but with infill block walls added. The two-storeyed buildings were designed in accordance with the 1984 loadings code [10], and should respond in a reasonably ductile manner under strong earthquake attack. The seismic responses of these two-storey buildings are important in that the majority of New Zealand commercial buildings are low-rise.

The UBM and RM buildings survived the earthquake ground motion in Dannevirke with no signs of damage. The third and fourth buildings modelled were not actually located in Dannevirke but were representative of 1980's New Zealand low-rise commercial buildings, and were buildings for which we were able to obtain all the design data.

As noted earlier, the main street in Dannevirke is in the N67E direction in which the ground motion was relatively strong and hence the structural elements in the lateral direction of most of the buildings would have experienced stronger shaking than those in the longitudinal direction. The buildings were modelled in two dimensions, and the responses of structural elements in the lateral direction were examined for the ground motion recorded in the N67E direction. The stronger components of the recordings at El Centro (S00E) and Matahina (N53W) were also used to study the responses of the buildings in the lateral direction. A simple model was used to assess the response of the single-storey UBM building, while the two-storey buildings were analyzed using the DRAIN-2DX [15] computer program. The first 20 s segments of the Dannevirke and Matahina records and 30 s segment of the El Centro record were used in the time-history analyses.

5.1 Single-Storey UBM Building

This pre-code era single-storey brick masonry building, which is located a few metres from the Dannevirke Post Office, showed no damage due to the Weber earthquakes. This building has a floor area of 5.25 m x 11.6 m with a light-weight roof. The height of the 0.28 m thick masonry wall is 2.6 m. The brick walls are unreinforced, but they are capped with a concrete ring beam which may be reinforced.

The back wall of the structure would have been the most highly stressed in the Weber II event, so the behaviour of this wall was studied under in-plane seismic loadings. The response of this short wall can be satisfactorily calculated taking into account the shear deformation only. Neglecting the openings in the wall, the dynamic characteristics of the wall can be approximately estimated assuming a uniformly distributed mass and stiffness along the height of the wall. By analogy to an approach of Clough and Penzien [16], the horizontal displacement u(x) at height x is

$$u(x) = \sum Z_n \psi_n(x) \qquad (1)$$

where Z_n is an amplitude term and $\psi_n(x)$ is a shape function of the n^{th} mode. It can be shown that the natural frequency and the mode shape of the n^{th} mode of the wall are respectively

$$\omega_n = \frac{(2n-1)\pi}{2h}\sqrt{\frac{GA}{\bar{m}}} \qquad (2)$$

and

$$\psi_n(x) = \sin\frac{(2n-1)\pi x}{2h} \qquad (3)$$

where G is shear modulus, A is shear area, $\bar{m}$ is mass per unit height and h is height of the wall.

Assuming the density of brick as 1900 kg/m^3 [17] and a G value of 0.4 GPa gives the fundamental period of the wall as 0.023 s, corresponding to a frequency outside the filter bands of earthquake records. If the contributions of the higher modes of equation (2) are ignored, the maximum stress developed in the wall (on its own) can be shown to be 15 kPa for the N67E component of the Dannevirke record. Alternatively, for this short wall responding at high frequency, the maximum stress is expected to be close to a static case in which the uniformly distributed load on the wall is taken as the mass times the peak ground acceleration. The base shear of the static analysis of the wall on its own is 18 kPa, which is very close to the value given above. However, when a portion of the inertia load associated with the side walls and the roof structure is considered to be acting on this wall and an allowance was made for the openings, the maximum shear stress induced in the wall increased from 18 kPa to 37 kPa. These calculated shear stress responses are substantially less than the lower bound shear strength of 69 kPa for unreinforced solid brick masonry derived from Amrhein [17]. This is consistent with the absence of earthquake damage to this wall.

The PGAs of the El Centro and Matahina records are similar to the PGA of the Dannevirke record. Therefore the maximum stress developed in the wall would have been similar to those calculated for the Dannevirke record if this building had experienced the stronger component of the El Centro or Matahina records.

5.2 Two-Storey RM Building

The closest accelerograph to the source of the May Weber earthquake was located on the ground floor of this building, the Dannevirke Post Office. Despite the PGA at this site reaching 0.4g, the building showed no sign of distress. No cracks, which could have been caused by the earthquake, were seen in the structural elements of the building.

This is a 2-storey reinforced concrete masonry structure (Figure 8(a)) which was designed in 1987. This building has a floor area of 10.7 m x 30.0 m. In the lateral direction, the structure consists of a light wall frame in the front, a wall with moderate sized openings at the rear, and three short internal walls in the lateral direction. The length of the short walls (which run from ground floor to first floor) are about one third of the width of the building, and are built into one of the side walls. The designers assumed that no seismic lateral forces would be carried by the light frame in the front and designed each of the remaining element as a ductile shear wall.

In the present study, the structure was modelled using beam-column elements with rigid ends representing the joints, and rigid link connections between the front wall frame and the other walls were adopted (Figure 8(b)). The back wall was taken as two separate walls coupled together with deep beams, allowing for openings. A single element was used to represent all three short internal walls. For each member, appropriate flanges were used to calculate section properties which were based on gross sectional areas.

The floor diaphragms were assumed to be horizontally rigid and the mass associated with the lateral motion was lumped at floor levels. The estimated masses at the first floor and roof level were $256\text{x}10^3$ kg and $82\text{x}10^3$ kg respectively. The properties of concrete were taken from the New Zealand masonry code [18] as used by the designers. The code recommends using a compressive strength, f'_m, of 8 MPa and an elastic modulus, E_m, of 25 GPa. The E_m value was, however, considered to be large compared to the values reported in the literature. We adopted two values, ie. E_m = 10 and 20 GPa, for the analyses with Poisson's ratio of 0.25. Given the age of the building and quality of the construction, the E_m value of 20 GPa may be regarded as the "best estimate". However, it is of interest to examine the response of the building for a lower value of 10 GPa. As suggested by Priestley and Calvi [19], the "best estimate" of the yield strength of the steel was taken as $1.1f_y$, where f_y is the nominal yield strength. The elastic modulus of steel, E_s, was taken as 200 GPa. Since the building showed no visible cracks after the earthquake, it was decided to perform linear time-history analyses assuming 2% of critical damping for the two translational modes of vibration. The damping in the dynamic analyses was taken into account in the form of Rayleigh damping, a combination of the mass and the initial stiffness matrices representing the damping matrix. The seismic time-history analyses were carried out in the presence of gravity loads, and the bending moment calculated for each member was compared with the probable yield strength to assess the possible need for non-linear analyses.

The two natural periods of vibration of the model were 0.077 s and 0.028 s for E_m = 10 GPa, and 0.055 s and 0.020 s for E_m = 20 GPa. Most of the lateral resistance was provided by the bigger wall W1 at the rear end and by the short internal walls. The analyses showed that the forces developed in the members of the front wall frame were small and within the linear range. Similarly the forces determined for the deep beams and wall W2 representing part of the back wall were well within the yield levels. The forces calculated for wall W1 and one of the short walls under three different loadings are compared in Table 1 with the estimated probable yield strengths and shear resistance.

The estimated yield moments listed in Table 1 are the smaller of the positive and negative values corresponding to the maximum axial loads developed in the walls during the response of the model to the Dannevirke record. These axial loads are similar to those calculated for the El Centro record, and in both cases are close to the axial load levels at the time when the maximum bending moments were induced in the walls. It was assumed in the calculation that there is no tension carried by the masonry. The shear resistance for each wall was obtained using the code recommended shear stress of 0.24 MPa for the concrete of the masonry (alone). The flanges of the walls were not considered in determining the shear resistance provided by the masonry. The calculated element forces reported in Table 1 for different records are the maximum of the positive and negative values. For the Dannevirke record the maximum positive and negative moments are nearly the same.

For all three records comparison between the shear resistance and calculated shear forces indicates that shear failure is unlikely to occur in the walls. For the Dannevirke record, in the model using E_m = 10 GPa, the maximum calculated shear response equals the estimated shear resistance in the short wall. The fact that there was no sign of distress in the short walls after the earthquake may be partly because E_m = 10 GPa is the lower bound of the elastic modulus and E_m = 20 GPa is more likely to have been achieved. For E_m = 20 GPa, the shear force is reduced by about 30 percent in the two walls for the Dannevirke record. The calculated shear forces may actually have been slightly greater than determined here, due to torsion (from asymmetry in the longitudinal direction) which we ignored in the analyses. However most of the torsion will be resisted by the side walls. Of the structural elements in the lateral direction, the short internal wall which is located closest to the front of the building is likely to be affected most by torsion. This aspect was considered in the design and additional shear reinforcement was provided in this wall.

For the Dannevirke record, the calculated maximum bending moments in the walls for E_m = 10 GPa are larger than the corresponding probable yield moments, but for the most likely E_m value of 20 GPa for the walls, the calculated response moments fall well short of yield. However, the bending moments calculated for the two E_m values were large enough to produce flexural cracks. For E_m = 20 GPa, the ratio between the calculated bending moment to the probable yield moment is about 0.65 for wall W1 and 0.75 for the internal wall. Despite that, there were no visible cracks in the walls.

The walls in this building were designed using SM = 2, which is approximately equivalent to a ductility factor of $\mu = 3$. The $\mu = 3$ inelastic response spectrum for the Dannevirke N67E record, the code spectra for the Dannevirke region for SM = 2 (1984 code) and $\mu = 3$, $S_p = 0.67$ (1992 code) are shown in Figure 9. Noting again that the fundamental period of the building was ≤ 0.077 s and that the walls reached only about 70 percent of the yield strength when subjected to the stronger of the Dannevirke components, it appears from Figure 9 that both the short period truncation and the use of $S_p = 0.67$ in the 1992 code are supported by this particular case study.

As is to be expected from the elastic response spectra (Figure 5) and the periods of vibration given above, the maximum member

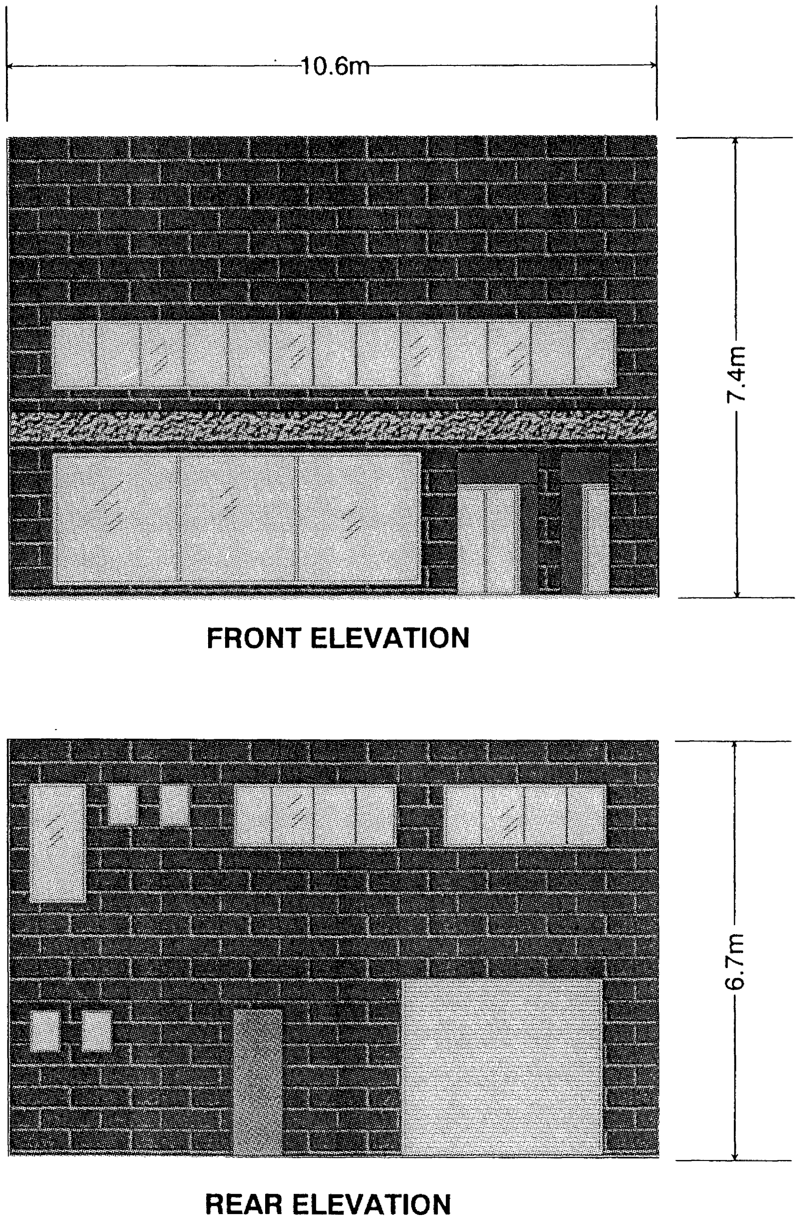


FIGURE 8(a) *Front and rear elevations of Dannevirke Post Office, made of r.c. blocks.*

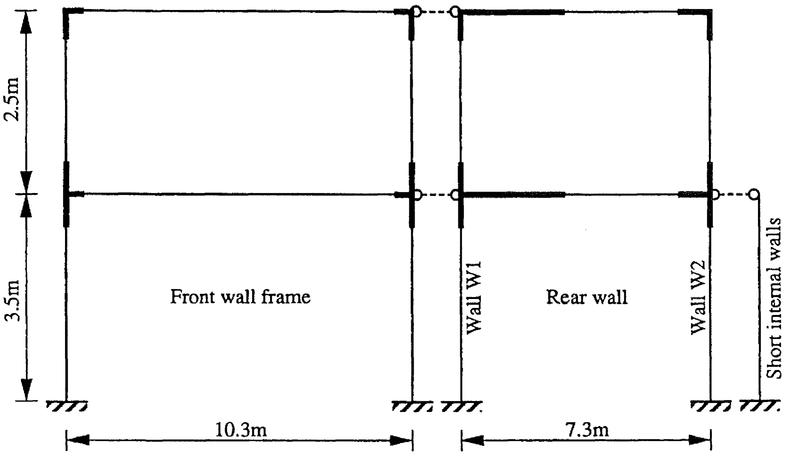


FIGURE 8(b) *Model of the two-storey reinforced concrete masonry (RM) Post Office building in Dannevirke (see also Figure 8(a)).*

Table 1 : Comparison of the calculated forces with the estimated resistance in two walls of the RM building (Figure 8). Calculated values are those corresponding to a damping value ζ = 2% of critical.

	Wall W_1	Short wall(s)
Estimated resistance (probable strength)		
Shear resistance (kN)	1025	357
Yield moment for E_m = 10 GPa (kNm)	3093	964
Yield moment for E_m = 20 GPa (kNm)	3147	967
Dannevirke N67E (E_m = 10 GPa)		
Shear (kN)	775	340
Max. bending moment (kNm)	3138	1092
Dannevirke N67E (E_m = 20 GPa)		
Shear (kN)	541	242
Max. bending moment (kNm)	2070	749
El Centro S00E (E_m = 10 GPa)		
Shear (kN)	937	328
Max. bending moment (kNm)	3774	1317
El Centro S00E (E_m = 20 GPa)		
Shear (kN)	836	285
Max. bending moment (kNm)	3286	1166
Matahina N53W (E_m = 10 GPa)		
Shear (kN)	485	212
Max. bending moment (kNm)	1810	675
Matahina N53W (E_m = 20 GPa)		
Shear (kN)	526	222
Max. bending moment (kNm)	1988	734

forces for the Matahina record are generally less than those for the Dannevirke record, while those for El Centro are greater. In the latter case the strongest response is about 1.05 times the probable yield resistance (in the E_m = 20 MPa model).

5.3 Two-Storey MRF Building

This is an office building which was designed in 1984 as a moment-resisting ductile frame, adopting the code provision that capacity design was not required for two-storey buildings [20]. The building is 12.6 m x 25.6 m in plan, and consists of 2 bays in the lateral direction and 5 bays in the longitudinal direction. In the lateral direction, only the end frames are capable of resisting significant seismic loads. For analysis in the lateral direction, a 2-dimensional model was formed by assuming rigid link connections between the two end lateral frames (Figure 10). The response of the building model was studied for two cases as described below.

The dimensions of the columns are 600 mm x 400 mm and of the beams are 900 mm x 400 mm. Allowance was made for the effect of flexural cracking when estimating the flexural stiffness of the columns and beams. The "best estimate" of the strength of concrete was taken as 1.5 times the nominal design strength (f'_c = 25 MPa) and $1.1f_y$ was used as the yield strength for the reinforcement [19]. An elastic modulus of 29 GPa was used for the concrete, with Poisson's ratio of 0.25.

The strength of ground motions considered in the study is likely to cause extensive flexural cracking in the beams and possibly in columns. To make realistic prediction of the response of the structural model, it is imperative that we make appropriate allowance for the effect of cracking when estimating the stiffness of the beams and columns. To allow for the stiffness reduction, the material code NZS 3101 [20] recommends that the effective moment of inertia (I_e) of a beam section may be taken as 50% of the value obtained based on the gross concrete

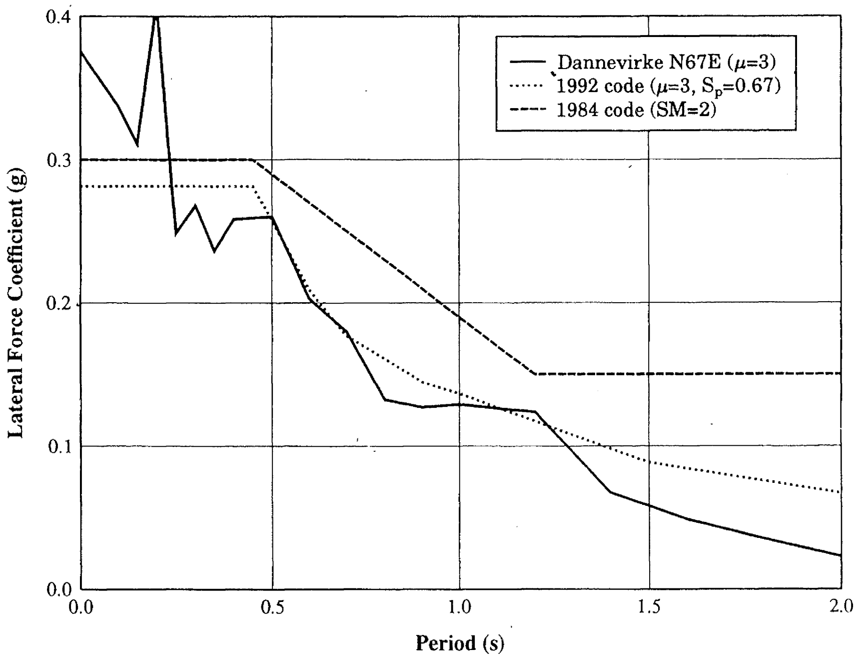


FIGURE 9 *Comparison of the μ = 3 inelastic response spectrum for the Weber II Dannevirke N67E component with the code spectra for the Dannevirke region for SM = 2 (1984 code) and μ = 3, S_p = 0.67 (1992 code).*

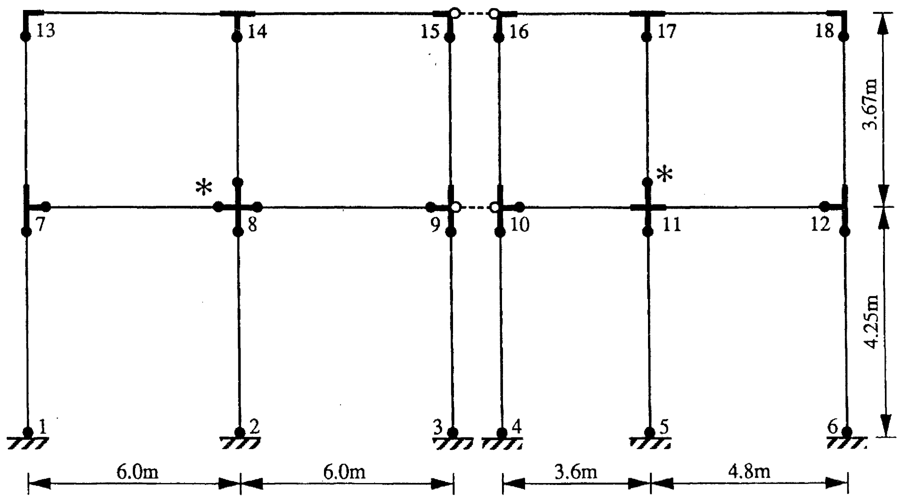


FIGURE 10 *The locations of the plastic hinges (denoted by solid circles) formed in the bare frame model of the two-storey moment-resisting frame building (Model 1) when subjected to the Weber II Dannevirke N67E ground motion. For Model 2 the pattern was the same, except that hinging did not occur at the two locations marked *.*

area (I_g) and 100% of the gross section value for a column section sustaining significant axial compression. Paulay and Priestley [21] recommend that I_e may be taken as 30-50% of I_g for beams and 30-90% of I_g for columns depending on the axial load level. In view of the uncertainties involved in choosing appropriate stiffnesses, we investigated responses of two models.

Model 1: In the first model the flexural stiffness was estimated using the recommendation given in the material code, ie. I_e = 0.5I_g for beams and I_e = I_g for columns. (Values directly from [20], but this model is probably more appropriate to beam-hinging structures).

Model 2: In the second model, the I_e value was taken as 0.8I_g for the first floor beams, and 0.5I_g for the ground floor beams and for all the columns. (Based on [21], these values were judged to be appropriate to the column-hinging behaviour of the structure being studied).

The response of the building model did not demand formation of plastic hinges in the first floor beams under the suite of ground motions considered in this study. Hence a larger proportion of the I_g value was thought to be appropriate for the first floor beams compared to the ground floor beams.

For each model the seismic mass was lumped at each floor level with 342×10^3 kg at the first floor and 312×10^3 kg at the roof level. The acceleration time-history analyses were performed in the presence of gravity loads. Viscous damping in the dynamic analyses was taken into account in the form of Rayleigh damping. Five percent of critical damping was assumed for the two translational modes because the structural models responded non-linearly for the earthquake loadings imposed by the various ground motions that were applied.

The beams and columns of the bare frame structure were modelled using beam-column elements with rigid zones representing the joints. The moment-curvature relationship of these elements was taken as bi-linear. Yielding was permitted in the members by defining appropriate yield interaction surfaces conforming to the reinforcement detail. The DRAIN-2DX program [15] assumes that yielding takes place in concentrated plastic hinge zones at the element ends. This assumption makes it difficult to assign suitable strain hardening ratios, α, for the members. Researchers have adopted different methods for determining α. It has been reported [22] that analyses carried out using nominal values for strain hardening ratios produced satisfactory results. We adopted an approach somewhat similar to that was used by Fenwick and Davidson [23]. This involved performing a "push-over" analysis on the models in the presence of gravity loads, in which the applied load pattern was similar to that obtained from the 1984 loadings code for an equivalent static analyses. For the end of each beam-column element an α value was calculated to produce a displacement strain hardening ratio of 2½ percent. This implies that once all the plastic hinges have formed, for a unit displacement at the roof level, the bending moment at each plastic hinge will increase by 2½ percent of the moment corresponding to a unit displacement at the roof level before yielding occurred in the structure. Where different α values were found for the two ends of a member the average value was used.

Model 1

The periods of vibration of this model were found to be 0.43 s and 0.12 s for the two translational modes. Step-by-step time-history analyses were performed for the stronger components of the Dannevirke, El Centro and Matahina records. In all three computer runs, yielding occurred in each member except in the beams at the roof level. At the end of 6.0 s of the N67E component of the Dannevirke ground motion, plastic hinges formed at the top and bottom of all the ground floor columns and at the top of all the first floor columns. In the next 1.5 s, the bottom ends of the intermediate columns in the first floor and the beams at the first floor level developed plastic hinges.

Figure 10 depicts the locations of the plastic hinges formed when the model was subjected to the Dannevirke record. The yielding occurred at similar locations for the El Centro and Matahina records except at Node 8. No hinges were formed in the beam end located at the left hand side of Node 8 in the latter records. Although hinges formed in the beam ends located either side of this nodal point for the Dannevirke record, the accumulated plastic deformations at these locations were relatively small.

The maximum responses found from analyses of the three ground motions are given in Table 2 as follows: (1) the base shear, (2) the summation of the column bending moments at the base, and (3) the interstorey deflections. Also given in Table 2 are the base shear and the summation of the column moments at the base obtained when applying the lateral loadings (Figure 7) of the code equivalent static load procedure, and the allowable interstorey deflections recommended in the codes. A

Table 2 : Comparison of the time-history results with those determined using the code spectra for the MRF structure (Model 1). The forces and deflections given for the time-history analyses are the peak responses.

	Base shear (kN)	Base moment[1] (kNm)	Drift (mm)	
			Ground Floor	First Floor
Resistance provided (probable strength)	2160[2]	1940	-	-
NZS 4203 : 1984	616	1323	42.5[4]	36.7[4]
NZS 4203 : 1992[3]	981	2110	106.3[4]	91.8[4]
Dannevirke N67E	1051	2004	44.6[5]	5.4[5]
El Centro S00E	949	1813	32.6[5]	8.8[5]
Matahina N53W	986	1886	35.8[5]	8.2[5]

1 $\sum$Column moment at base
2 Calculated based on the shear reinforcement at mid-height (not in potential plastic hinge region)
3 1992 code loadings without adjustments for capacity design
4 Allowable drift (1984 code - based on 1% of storey height; 1992 code - 2.5% of storey height)
5 Calculated drift

performance factor $S_p = 0.67$ was used for determining the lateral forces from the 1992 loadings code. Allowable interstorey drift given in the 1992 loadings code depends on the method of analysis used in determining the member forces. The values given in Table 2 are those corresponding to the numerical integration time-history method. The response contributions of the gravity loads do not affect the comparison of the base shear, nor do they have any significant influence on the other two responses given in Table 2.

The base shears and moments obtained from the three earthquake records and from the 1992 code spectrum are all very similar, and on average exceed by a factor of about 1.5 times those obtained using the 1984 code spectrum. However, the shear resistance provided by the columns was found to be about 3.5 times higher than that required by the 1984 code spectrum, because the design of the shear reinforcement was controlled by the minimum detailing requirements given in the material code [20].

When the base moments reached the maximum value of 2004 kNm for the Dannevirke ground motion, the bending moments at the top and bottom of all the ground floor columns slightly exceeded the probable (ultimate) bending strengths. The bending moments in the remaining elements never exceeded the probable values, but in some cases reached close to the probable strengths of the members. In the El Centro load case the ultimate moment was never exceeded, while in the Matahina load case the ultimate moment was exceeded in one member only. The increment of the bending moment at the end of each element beyond yielding is sensitive to the strain hardening ratios chosen for the structural members. Thus, in a sensitivity test, using half the α values established for the members from the "push-over analysis", the bending moment never exceeded the probable ultimate strength in any member under the Dannevirke earthquake loading. The reduction in bending moment obviously reduces the curvature ductility demand, but the reduced α values had no significant influence on the interstorey deflections (Table 2), maximum plastic rotations and accumulated plastic rotations (Table 3).

The Dannevirke ground motion produces a maximum horizontal deflection in the ground floor (bottom storey) of the building which slightly exceeds the allowable drift recommended in the 1984 loadings code (ignoring the provision regarding unseparated non-structural elements). The maximum interstorey drift estimated for the El Centro and Matahina records are both about 25 percent less demanding than the Dannevirke ground motion. The large drifts permitted by the 1992 code (Table 2) appear to be over generous in relation to the demands of the three records studied, and they imply a very high level of damage to structural and non-structural elements.

The maximum plastic rotation, curvature ductility and accumulated plastic rotation which occurred in three element groups are compared in Table 3 for the three ground motions. The maximum plastic rotation and accumulated plastic rotations correspond to the largest values obtained in a particular element group, while curvature ductility is the value for the element which produced the largest ratio between the curvature ductility demand and available curvature ductility calculated for the section based on the reinforcement detail. In determining the available curvature ductilities of the columns the presence of axial load was ignored, as the load levels seemed to have no significant influence on the ductility ratios. The ductility demand was then estimated from the bending moment calculated from the analyses using the moment-curvature relationship established for each end of the members. The Dannevirke

Table 3 : Comparison of the maximum plastic deformations which occurred in the beam-column elements when the MRF bare frame model (Model 1) was subjected to three earthquake records.

	Dannevirke N67E	El Centro S00E	Matahina N53W
<u>Max. plastic rotation (rad)</u>			
Ground floor column	0.0112	0.0077	0.0090
First floor column	0.0018	0.0031	0.0027
First floor beam	0.0014	0.0026	0.0020
<u>Curvature ductility demand</u>			
Ground floor column	17.0 (11.3)*	8.0 (10.5)	12.6 (11.3)
First floor column	4.6 (17.7)	7.9 (17.7)	10.3 (11.3)
first floor beam	10.0 (24.7)	18.2 (24.7)	21.4 (24.7)
<u>Accum. plastic rotation (rad)</u>			
Ground floor column	0.0579	0.1073	0.0410
First floor column	0.0554	0.0992	0.0415
First floor beam	0.0064	0.0093	0.0075

* bracketed values are those determined for the reinforcement detail.

motion demanded the largest single excursion plastic rotation at the base. However, the accumulated rotations obtained for the three ground motions indicate that the cumulative damage is likely to be higher for El Centro than for the other records.

Model 2

This model was more flexible than Model 1, such that its periods of vibration were 0.54 s and 0.15 s. Time-history analyses were performed for the three ground motions as described above for Model 1. The responses of Model 2 are generally similar to those of Model 1 for the Dannevirke, El Centro and Matahina ground motion components.

Tables 4 and 5 summarise the responses of MRF Model 2 subjected to the three earthquake loadings. As the flexibility of the building increased from Model 1 to Model 2, the base shear and the base moment reduced, but the interstorey deflections increased. The plastic hinge locations under the Dannevirke loading were almost identical for the two models. The maximum values of the plastic hinge rotation and accumulated plastic hinge rotation in each element group also increased, except for the maximum accumulated plastic rotation in the first floor column for the El centro and Matahina records. The end moments induced in the beam-column elements slightly exceeded the probable ultimate strengths in one ground floor column for the Dannevirke record, in one first floor beam for the El Centro record and in none of the members for the Matahina record.

The differences between the responses of Model 1 and Model 2 are sufficiently large to underline the importance of choosing the appropriate stiffnesses for the yielding members. As the MRF building that we have studied was not located in Dannevirke it was not possible to infer from field results which of the two models is more realistic, but we consider that Model 2 is likely to be more realistic for the column-hinging structure being modelled.

Table 4 : Comparison of the time-history results obtained for the MRF Model 2 under three earthquake loadings. The forces and deflections given for the time-history analyses are the peak responses.

	Base shear (kN)	Base moment[1] (kNm)	Drift (mm)	
			Ground Floor	First Floor
Dannevirke N67E	967	1842	53.1[2]	6.5[2]
El Centro S00E	919	1756	47.7[2]	10.1[2]
Matahina N53W	928	1773	42.6[2]	8.0[2]

1 $\sum$Column moment at base
2 Calculated drift

Table 5 : Comparison of the maximum plastic deformations which occurred in the beam-column elements when MRF Model 2 was subjected to three earthquake records.

	Dannevirke N67E	El Centro S00E	Matahina N53W
Max. plastic rotation (rad)			
Ground floor column	0.0127	0.0110	0.0096
First floor column	0.0023	0.0037	0.0028
First floor beam	0.0024	0.0037	0.0020
Curvature ductility demand			
Ground floor column	11.4 (11.3)*	8.0 (11.3)	8.8 (11.3)
First floor column	3.9 (17.7)	6.2 (17.7)	4.7 (17.7)
first floor beam	21.4 (24.7)	32.9 (24.7)	18.4 (24.7)
Accum. plastic rotation (rad)			
Ground floor column	0.0756	0.1163	0.0421
First floor column	0.0590	0.0843	0.0399
First floor beam	0.0098	0.0187	0.0088

* bracketed values are those determined for the reinforcement detail.

The code design strengths corresponding to Model 2 are smaller than those required by Model 1 because 0.54 s fundamental period of Model 2 falls outside the constant acceleration plateaus used in the code spectra. The design values of the base shear and base moment of Model 2 are 0.94 and 0.81 times those of Model 1 (Table 2) for the 1984 and 1992 codes respectively.

From Table 2 for Model 1 and Table 4 for Model 2, it is seen that Dannevirke gives the largest values for base shear and base moment, followed by Matahina and then El Centro. This sequence cannot be inferred from the elastic response spectrum (Figure 5) at the fundamental periods $T = 0.43$ s for Model 1 and $T = 0.54$ s for Model 2 or at the longer periods as produced by non-linearity (say $0.43 \leq T \leq 0.7$ s). The inelastic spectra corresponding to $\mu = 6$ (Figure 7) indicates that the Dannevirke ground motion is stronger than the other two records for periods of vibration in the range $0.4 \leq T \leq 0.58$ s.

From Table 3 for Model 1 and Table 5 for Model 2 it is seen that for cumulative plastic rotation in the ground floor columns, El Centro gives the strongest response, followed by Dannevirke and then Matahina. This sequence cannot be inferred from either the elastic or the inelastic spectrum in the period range $0.4 \leq T \leq 0.7$ s. The cumulative plastic rotation is determined by the duration of *strong shaking*, which is not taken account of by either the elastic or the inelastic spectrum.

From the DRAIN-2DX analyses it is predicted that the bare frame MRF structure, designed to the 1984 code, would have suffered considerable damage to its columns, if subjected to any of the three moderate ground motions considered here. (This also assumes that the building is truly "bare frame" in behaviour, with negligible stiffness contribution from cladding or partitions.) It may also be inferred that a similar building designed to the loadings of the 1992 code, but without capacity design, would perform better because the design strength required by the 1992 code is over 35 percent higher than that required by the 1984 code for Models 1 and 2.

5.4 Two-Storey MRF Building with Infill Panels

The building used here was the same as the bare MRF structure discussed in the preceding section except that the frames were filled with reinforced concrete masonry panels. The infill panels used for the structural model were of 200 mm thick reinforced concrete masonry, for which it was assumed that the elastic modulus was in the range of E_m = 10-20 GPa and Poisson's ratio was 0.25. The latter value was fairly arbitrarily assigned, partly because the actual value of Poisson's ratio has negligible influence on the results. It is noted that a value of 0.20 has been adopted for concrete by most codes internationally, while a value of 0.25 would be more appropriate for concrete blocks in compression (R.C. Fenwick, Auckland University, pers. comm., 1994). Assigning a suitable value for the shear strength of masonry walls is a difficult task. The maximum shear strength of masonry walls can vary by an order of magnitude depending on the type of construction material, mortar strength and quality of construction. The New Zealand masonry code [18] postulates a maximum *total* shear stress of 1.6 MPa for high quality reinforced masonry construction. Higher shear stress values of up to 2.4 MPa (1.8 MPa for seismic loads) are allowed where higher design strength is established in laboratory tests. Average total shear strengths of about 2 MPa or more have been reported from experimental investigations on masonry walls subjected to cyclic loadings [24, 25], and we adopted a shear strength of 2 MPa for the panels considered in our structural model.

Simple beam theory can be applied to infilled frames to estimate the elastic responses satisfactorily [26, 27]. Separation between the frame and infills may occur at 50-70% of the ultimate lateral shear strength of the panels, and behaviour of the frame and infills becomes more complicated thereafter [21, 27]. Hence, if linear analyses produce maximum average shear stresses of around 1 MPa in the panels, it can be regarded that the whole system responds linearly.

The beams and columns of the building were modelled using beam-column elements as described for the bare frame structure. The effective moment of inertia I_e was taken as equal to I_g for the beams and columns. Each infill panel was modelled using a modified beam-column element [15]. The panel element was assumed to have shear resistance only, and no provisions were made for the interaction between the columns and panels. The time-history analyses were performed in the presence of gravity loads. The viscous damping of 2% critical was adopted for the linear analyses.

For E_m = 20 GPa for the infill panels, the periods of vibration of the model were found to be 0.081 s and 0.026 s. For the N67E component of the Dannevirke record, the frame structure with infills gave a maximum average shear stress of 0.88 MPa in a panel at the ground floor level when 2% of critical damping was considered. The forces in the beams and columns were small enough to keep the members well below their yield strengths. Similarly for the El Centro and Matahina records, the responses of the model were within the linear range with maximum average shear stress in any one panel reaching 1.06 and 0.54 MPa respectively. The base shears induced in the structure were 3530 kN for the Dannevirke, 4150 kN for the El Centro, and 2160 kN for the Matahina records. These shears are up to four times greater than those for the bare frame models (Tables 2 and 4), showing that while the addition of infill walls attracts more load, the walled structure is better able to resist its applied forces than the moment-resisting frame.

For E_m = 10 GPa for the infill, the periods of the model lengthened to 0.10 and 0.035 s. The time-history analyses of this model for the three records produced maximum average shear stress of 0.77-1.15 MPa in the panels. The forces induced in the beams and columns did not exceed the yield strengths.

6. RESPONSES OF BUILDING MODELS TO THE WEAKER S23E COMPONENT OF THE WEBER II DANNEVIRKE RECORD

The PGA of the Weber II Dannevirke record is 0.26g in the S23E direction compared to 0.38g in the N67E direction, and the 5% damped elastic spectral ordinates of the S23E component are generally smaller than those of its orthogonal component (Figure 5). The responses of some of the building models to the weaker component of the Dannevirke record were examined because the PGA of the S23E component seems more likely to be representative of the PGAs in Dannevirke in the Weber II event (see beginning of Section 4). The key results obtained for the two components of the Dannevirke record are compared in the following paragraphs.

The response of the structural model of the Dannevirke Post Office to the S23E component of the Dannevirke record induced forces in the principal elements (ie. Wall W1 and short walls) in the range of about 80-85% of those obtained for the N67E component when E_m = 20 GPa was used (Table 1). For E_m = 10 GPa, the S23E component of the Dannevirke record induced similar or larger forces in the principal elements than the N67E component because the weaker component has a narrow dominant spectral peak at a period close to the fundamental period of this model.

The two models of the MRF bare frame responded non-linearly to the S23E component of the Dannevirke record. The maximum values of base shear and base moment calculated for the models during the time-history analysis were 80-85% of those obtained for the stronger component (Tables 2 and 4). A much greater reduction was found for the interstorey drifts at the ground floor level, the S23E component demanded drifts of 12 mm for Model 1 and 20 mm for Model 2 compared to 45 mm and 53 mm for the N67E component. The drifts for the S23E component are both within the limits required by the 1984 code.

The MRF with infill panels responded linearly to the S23E component of the Dannevirke record. The maximum average shear stress induced in any one panel is about 83% of that calculated for the N67E component when E_m = 20 GPa was used for the panels. The maximum value of the average shear stress was nearly the same for the two components when E_m = 10 GPa was used.

7. CONCLUSIONS

The following conclusions have been reached:

1. Structural Responses - Theory and Practice

1.1 Both of the Dannevirke buildings studied, the *pre-code era UBM building and the two-storey RM Post Office (designed to the 1984 code), had very short fundamental periods ($T \leq 0.077$ s). Thus the responses of both buildings were those of nearly rigid structures, and the high spectral accelerations obtained for the Dannevirke spectrum at slightly greater periods ($T \geq 0.13$ s) were irrelevant to their responses.

1.2 For the single storey brick building (UBM), in the Weber II earthquake theory matched practice to the extent that no damage was modelled or observed.

1.3 For two representative ground floor walls in the reinforced masonry building (RM) the ratios of maximum calculated response moments to yield moments were sufficient (at 0.65 and 0.75) to produce flexural cracks. If such cracks occurred they are likely to have closed up again more or less immediately, which is consistent with the fact that no cracks have been observed in the building since the earthquake.

1.4 The third structure analysed, a 1984 code two-storey bare moment-resisting frame building (MRF Models 1 and 2) showed highly non-linear response to the N67E Dannevirke record. The ground floor interstorey drift for both models exceeded the value permitted by the 1984 code.

1.5 The inclusion of infill panels in the frames of the MRF resulted in a considerably reduced fundamental period compared with the bare frame, so this model was a nearly rigid structure and responded elastically, similar to the UBM and RM buildings. Because of the stress- and deformation-reducing attributes of the panels, the building performs much better with infill panels than without, despite the base shear being up to 4 times larger when infill is present.

2. Analytical Modelling

2.1 The significant differences in the predicted responses of these two MRF structural models demonstrates the need for reliable stiffness values for non-linearly responding reinforced concrete members.

2.2 The damage potential of the three principal earthquake records studied, for the MRF building, was compared using three different analysis methods, (1) elastic response spectra, (2) inelastic response spectra, and (3) inelastic time-history analysis. Neither method (1) or (2) consistently predicted the same order of severity for the three records as method (3). This highlights the familiar problem of producing reliable response analysis methods that are reasonably simple to use.

3. Design Observation

3.1 As is now well recognised in earthquake engineering, it is again shown that buildings with structural walls or tough infill panels are far less prone to earthquake damage than those with bare moment-resisting frames.

4. Strength of Ground Shaking

4.1 From the theoretical responses of the four buildings modelled, it was found overall that the damage potential for low-rise buildings was similar for the three principal strong-motion records studied (namely Dannevirke N67E, El Centro S00E, and Matahina N53W). This is consistent with the similar magnitudes of the events concerned, and the similar local ground conditions and observed effects at the three recording sites.

4.2 The weaker component (S23E) of the Dannevirke record of the Weber II event may be considered to represent the strength of ground shaking in Dannevirke at the observed intensity of MM7½ better than the N67E component. The S23E record generally induces maximum forces of about 80-85% of those calculated for the N67E component in the buildings studied, while the interstorey deflections in the ground floor of the bare MRF building were less than 40% of those produced by the N67E component.

5. Code Implications

5.1 Considering the 1992 code spectra for $\mu = 1.25$ and $\mu = 6$, $S_p = 0.67$, the Dannevirke N67E record appears to be similar in strength to the design loadings implied by the code for low-rise buildings for the Dannevirke area (about MM8½). However, being from an earthquake of only moderate magnitude, it is deficient in duration of strong shaking for reaching the damage potential appropriate for time-history analyses modelling of 450 year events for this part of New Zealand. However, it would be appropriate as part of a suite of ground motions for such analyses.

5.2 The case study of the response of the Dannevirke Post Office supports the adoption in the 1992 code of design spectra which are truncated in the short-period range and which have a structural performance factor of 0.67.

5.3 The interstorey deflections (drift) permitted by the 1992 code appear to be over generous in relation to the demands of design earthquakes on MRF's without capacity design, and they imply very high levels of damage to structure and non-structural elements, which may not be compatible with structural survival or with the growing demands for damage limitation.

ACKNOWLEDGEMENTS

We would like thank the consulting engineers Ormond Stock & Associates and Connell Wagner for providing the design details of the Dannevirke Post Office building and two- storey moment-resisting structure respectively. We are grateful to our colleagues G. McVerry and J. Zhao for their constructive in-house reviews.

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