THE THOMSON RESERVOIR-TRIGGERED EARTHQUAKES

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ABSTRACT:

There are few better documented examples of reservoir triggered earthquakes than those recorded about Thomson Reservoir, east of Melbourne.. In 1977, the Melbourne and Metropolitan Board of Works (MMBW), predecessor to the current Melbourne Water Corporation, commissioned the Seismology Research Centre to install a network of seismographs to monitor seismicity around the Thomson dam area during the initial filling cycle of the reservoir.

About three years following the commencement of filling in 1983, shallow earthquakes began occurring immediately under the reservoir, with magnitudes up to ML 3.0, at rates of up to 5 events per week. These events were almost certainly reservoir-induced and coincided with steadily increasing water levels.

The reservoir reached, and was consistently near capacity during the period from 1990 to 1996. During this time, the majority of seismic activity near the Thomson Reservoir had migrated to greater depths and away from the reservoir, with decreasing activity levels.

On 25 September 1996, a magnitude ML 5.0 earthquake occurred at a depth of 12 km adjacent to the reservoir. The mechanism of this event showed southeast to northwest compression, consistent with it occurring on the nearby Yallourn Fault. This was followed by a period of intense seismicity with approximately 200 aftershocks recorded over more than a year.

Recent records indicate that the area seems to be approaching the end of the period when triggered earthquakes are likely with activity levels now similar to those existing prior to commencement of filling. The decreasing earthquake activity rate since 1996 however, coincides with steadily decreasing water levels at the reservoir.

Paper No. 8

1. RESERVOIR-TRIGGERED SEISMICITY

Since Carder (1945) first reported on the apparent increase in seismicity at Lake Mead in the United States, the number of cited cases of reservoir-triggered earthquakes has steadily increased worldwide.

Reservoir-triggered earthquakes are triggered by several mechanisms: (1) elastic stress increase following the filling of the reservoir, (2) an increase in pore water pressure in saturated rocks due to the decrease in pore volume caused by compaction in response to the surface water load, and (3) an increase in pore fluid pressure associated with water diffusion (Bell and Nur, 1978; Gupta, 1992).

Given the deepest reservoirs only generate surface loads of approximately 2 MPa, it seems that direct activation from the water load is unlikely (Bell and Nur, 1978).

2. THE THOMSON RESERVOIR

Background: The Thomson reservoir is located on the Thomson River, east of Melbourne, and north of the Latrobe Valley. The reservoir is bounded to the west by the Baw Baw Plateau and to the east by a narrow ridge separating the Thomson River from the parallel flowing Aberfeldy River (MMBW, 1975).

In 1973, the Victorian State Government in association with the MMBW announced the approval of the Third Stage in the Thomson River Development. Construction of the 165 m high embankment dam, one kilometre south of the Talbot Creek inlet commenced in 1976, and was completed by 1983. The dam now creates a reservoir extending 23 km upstream, with a capacity of approximately 1.1 million megalitres, and inundates approximately 2,200 hectares (MMBW, 1975).

Geology: The Thomson Reservoir region possesses two main formations of Paleozoic rocks – the Upper Devonian Baw Baw Granite, and a conformable folded sequence of Silurian and Lower Devonian sedimentary rocks which are intruded by the granite. The eastern margin of the granite lies west of and sub-parallel to the Thomson River and dips approximately 70° east under the reservoir (MMBW, 1975). The sedimentary rocks possess numerous folds with north to south striking axial planes, but tend to curve around the granite mass. Faults in the region are commonly associated with these axial fold plains (MMBW, 1975).

The dominant structural feature of the area is the Yallourn Fault which trends southwest to northeast, outcropping about 25 km southeast of the Thomson Dam. The fault dips northwest approximately 12 to 14 km under the reservoir and its fault scarp is readily apparent north of the Latrobe Valley, with cumulative vertical movement of hundreds of metres. In Australia, reverse faults often dip about 30° to 40° under the up-thrown block (Gibson and Wesson, 1979).

3. THOMSON RESERVOIR SEISMOGRAPH NETWORK

In 1977, the MMBW commissioned the Seismology Research Centre (SRC) to install a network of seismographs to monitor the seismicity around the Thomson Dam area. The aim of the project was to locate the earthquake occurrences before, during, and after the initial filling of the reservoir. Other aims of the project were to: (1) study the spatial distribution of earthquakes, (2) determine earthquake magnitudes, (3) investigate reservoir-triggered seismicity, (4) obtain an indication of earthquake focal mechanisms, (5) monitor the response of the dam and outlet towers to vibratory motion from these earthquakes and (6) if possible, predict future patterns of earthquake occurrence (Gibson, 1976).

The initial installation of the Thomson Dam seismograph network consisted of five digital seismographs, installed between May 1977 and February 1978 (Gibson, 1978).

4. HISTORY OF SEISMICITY

The Thomson Reservoir and surrounding region is relatively seismically active by Australian standards. Figure 1 shows the entire catalogue of earthquake activity about the Thomson Reservoir since 1976. The seismicity in the region has been divided into several time periods to demonstrate the spatial and temporal migration of earthquake activity. Figure 2 shows the epicentral distribution of the earthquakes over the designated time periods.

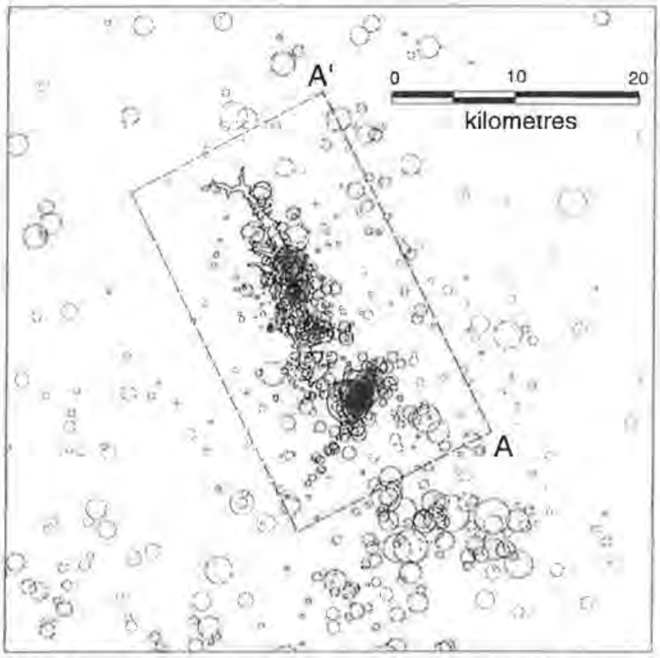

Fig. 1. Total earthquake activity about the Thomson Reservoir since 1976 with respect to the reservoir.

Pre 1976: Few accurate earthquake locations in the area were known before reasonable seismograph coverage in Victoria commenced in 1960, although many reports of earthquakes being felt in the Walhalla, Erica and Moondarra areas were made prior to the advent of this network. In the period from 1891 to 1907, several strong ground motions were felt in the Walhalla-Moondarra areas. One event in 1901 is reported to have destroyed several buildings and was estimated to have a maximum epicentral intensity of 7 on the Modified Mercalli scale (Gibson, 1977). A long period of quiescence ensued during which there was no significant seismological reporting. No major earthquakes occurred in the area from 1960 to 1976, however the new seismographs operated by the Bureau of Mineral Resources and Australian National

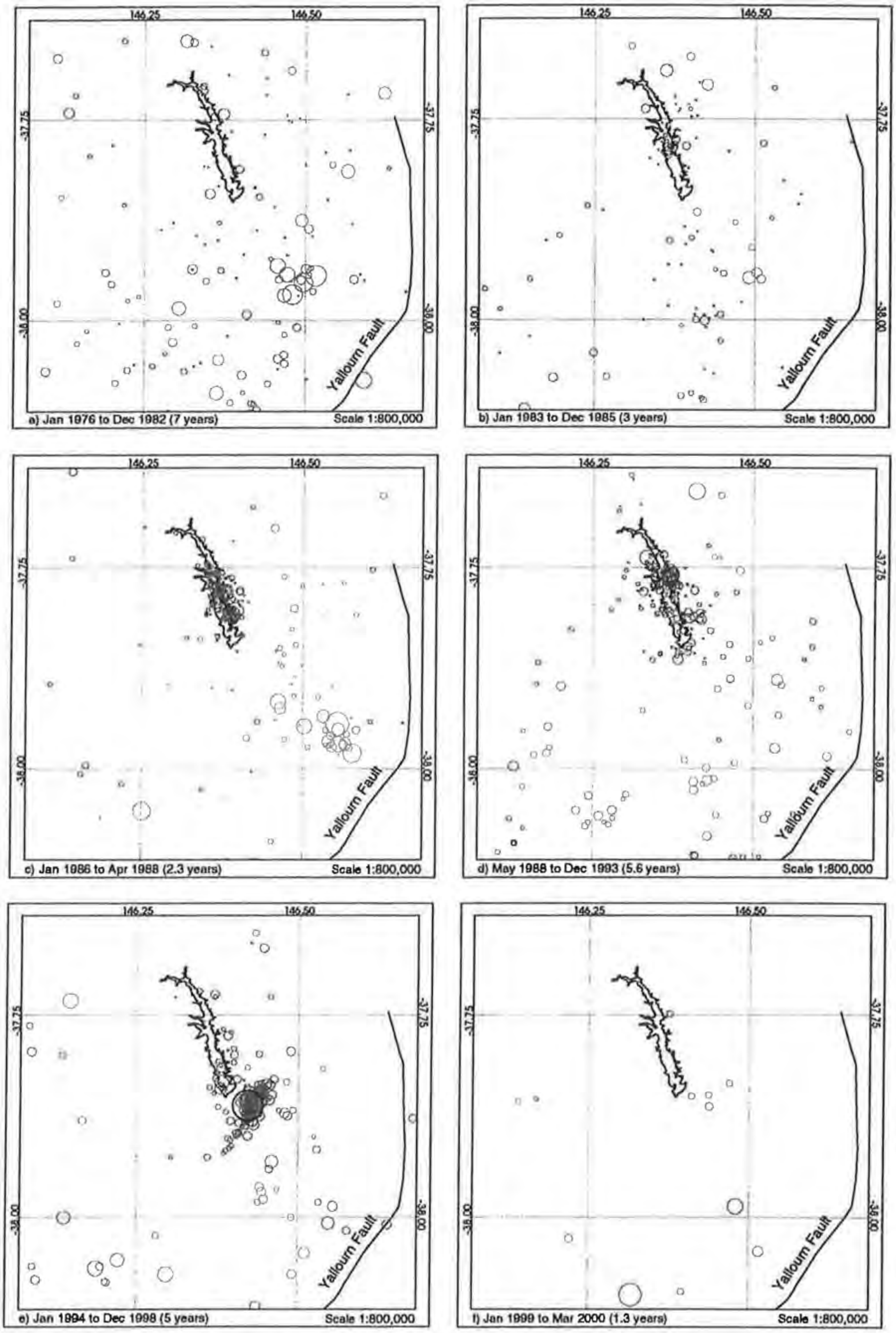

Fig. 2. Seismicity about the Thomson Reservoir since 1976. Circles indicate earthquake location with respect to the reservoir (diameter in mm is equivalent to local magnitude ML).

University respectively, did permit the location of smaller events recorded instrumentally. Five earthquakes with Richter magnitudes of around 3.5 were documented during this period.

January 1976 to December 1982: (Figure 2a) Since the beginning of 1976, the SRC has operated and maintained a number of microearthquake seismographs throughout Victoria, permitting the location of even smaller events. In 1976, this allowed the location of three earthquakes with Richter magnitudes greater than 2 in the Moondarra area (Gibson, 1977). With the completion of the Thomson Reservoir network in 1978, it became clear that the Gippsland area possessed a higher level of background seismicity than average for Victoria (Gibson, 1978).

January 1983 to December 1985: (Figure 2b) In 1983, Stage Three of the Thomson River Development was complete, and filling of the reservoir commenced that year. In November of 1983, a swarm of 37 earthquakes occurred in a small volume about 6 km northwest of the Thomson Dam at a depth of about 11 km. They were all very small events, the largest magnitude being about ML 1.5. The occurrence of the swarm so soon after the commencement of filling suggests that there may be some relationship. However, the 11 km depth of these events is much greater than would normally be expected for reservoir-triggered seismicity caused by increased pore fluid pressure. Similarly, a modification of the regional stress field appears to be an unlikely mechanism given the large depth and the small magnitude of the events. In addition, the reservoir was only at 3 per cent of its capacity at that time (Gibson and Wesson, 1983). Hence it follows that the location of the swarm may just have been coincidental. Apart from the cluster, few events occurred in the three years subsequent to commencement of filling, although seismicity was marginally higher than that of pre-filling with an activity rate of a little over one event per week (Gibson and Wesson, 1986). The reservoir level at the end of this period had increased regularly to approximately 40 m below capacity.

January 1986 to April 1988: (Figure 2c) From 1986, shallow earthquakes within 3 km of the surface began occurring immediately under the reservoir, with magnitudes to ML 3, at rates of up to 5 events per week. These events were almost certainly reservoir-triggered, resulting from water diffusion increasing pore fluid pressure beneath the reservoir.

Earthquakes located within the rectangular polygon (see Figure 1) were employed to quantitatively examine earthquake focus migration with depth and are illustrated in cross-sections along the line A to A′ (Figure 3). Figure 3a shows earthquake migration with respect to depth for the period of January 1986 to April 1988.

May 1988 to December 1993: (Figure 2d) During the period from 1988 until December 1993, the seismicity near the Thomson Reservoir area migrated to greater depths, down to about 12 km, and away from the reservoir, with decreasing activity levels (Figure 3b). This behavior is typical of reservoirs in their first ten to twenty years, and can be explained by the increasing pore water pressure in the system migrating to greater depths at a relatively slow rate (depending on rock permeability), until arriving at a state of equilibrium.

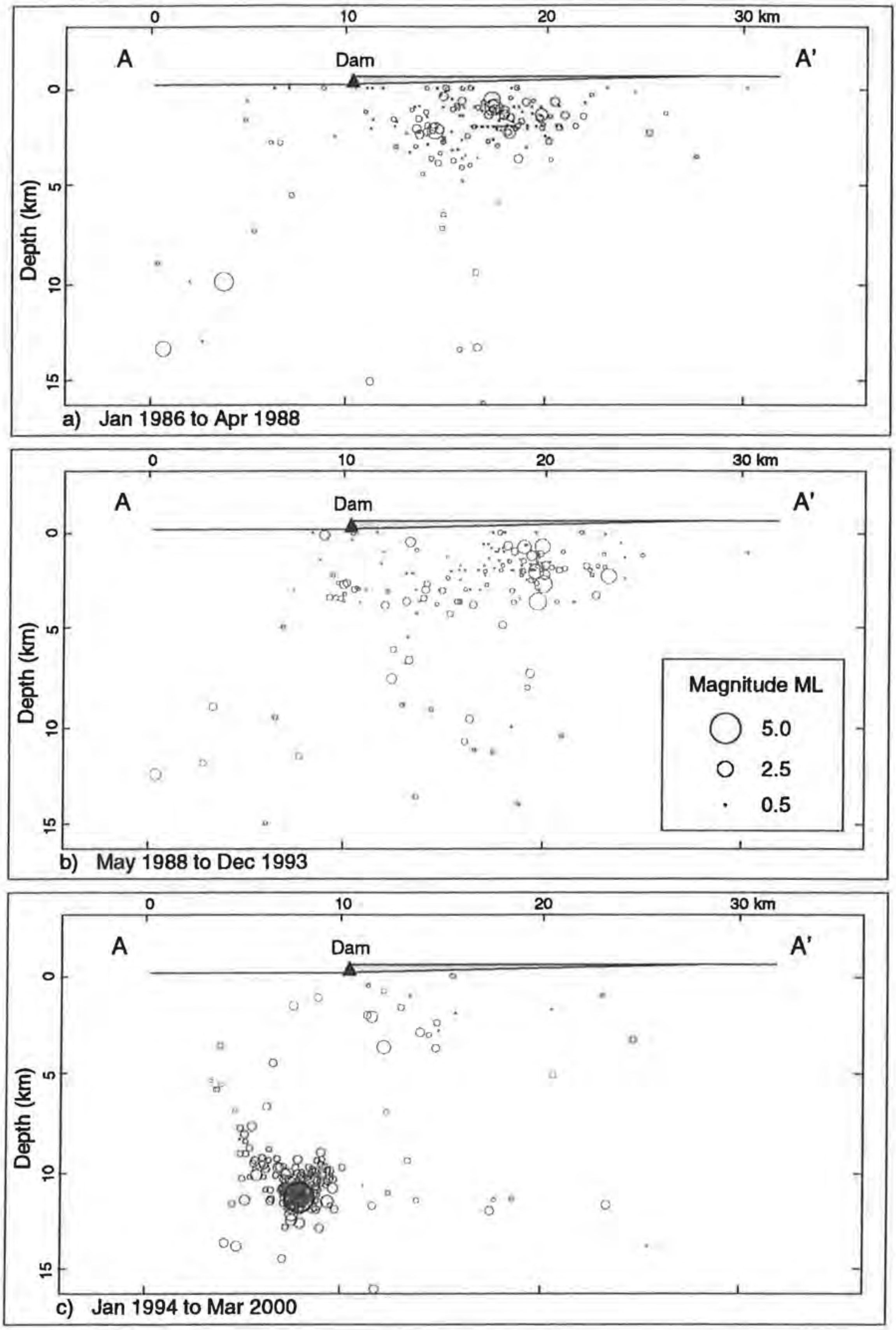

Fig. 3. Earthquake migration beneath the Thomson Reservoir with respect to time. The dam wall and reservoir level have been exaggerated.

January 1994 to December 1998: (Figure 2e) From 1994, earthquake activity at the Thomson Reservoir was consistently at depths between 9 and 12 km with further decreasing activity levels.

On 25th September 1996 at 05:49 pm EST, an ML 5.0 earthquake occurred at a depth of 12 km adjacent to the reservoir. The earthquake was felt strongly in the Thomson area, and as far away as Melbourne. Several strong motion accelerograms were recorded. The earthquake was preceded at 02:53 pm the same day by a foreshock of magnitude ML 3.3 at the same location. Many aftershocks followed for several months. In the first quarter (1 October to 31 December 1996) following the main shock, 99 aftershocks were recorded. Several of these exceeded magnitude ML 2.0. During the next quarter 21 aftershocks were recorded. The number of aftershocks continued to decrease each quarter for two years until no events were recorded at the reservoir for the quarter ending December 1998.

The location of the cluster suggests that the earthquakes occurred on the Yallourn Fault at depths between 9 to 12 km adjacent to the reservoir (Figure 3c). These earthquakes can be explained by an increase in water pore pressure in a highly prestressed environment.

It should also be noted that from 1990 until late 1996, the reservoir reached and was consistently near capacity.

Since 1998: (Figure 2f) Relatively few earthquakes have been recorded at the Thomson Reservoir since the abatement of aftershocks. Coincidentally, water levels at the reservoir have also decreased significantly since 1996 to a capacity similar to that of the first cycle of filling in 1986.

5. THE 1996 THOMSON EARTHQUAKE

The location of the ML 5.0 Thomson earthquake suggests that it occurred on the Yallourn Fault. Plotting the first motions of the direct seismic waveforms on a lower hemispherical projection, the mechanism for the earthquake was observed to possess southeast to northwest compression, consistent with reverse faulting of the Yallourn Fault (Figure 4).

Determinations of the mechanisms for the aftershocks were attempted via moment tensor inversion, but limitations in the software prevented the successful computation of focal mechanism solutions. The moment tensor is an important parameter in earthquake seismology as it describes the seismic source in three dimensions and gives

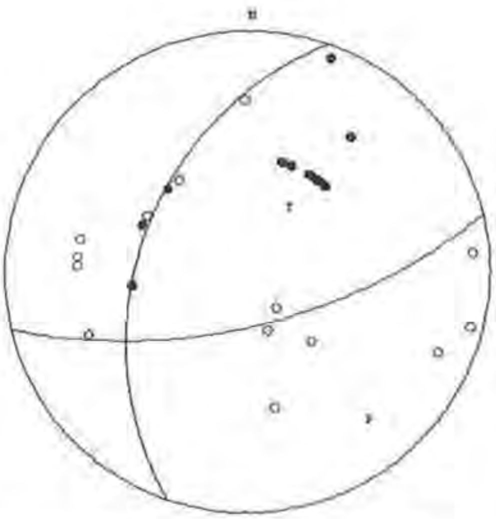

Fig. 4. Focal mechanism for the ML 5.0 Thomson Earthquake plotted on a lower hemispherical projection. The open and filled circles represent dilatational and compressive first motions respectively, illustrating southeast to northwest compression.

an indication of the local stress regime. The deficiency of the inversion techniques however may result from the presence of the Baw Baw Granite affecting the take-off angles of the elastic seismic waves. Therefore, in order to achieve a successful inversion, a 3-dimensional velocity model must be developed.

6. CONCLUSION

There is little doubt that the earthquakes occurring about Thomson Reservoir following commencement of filling in 1983 were reservoir-triggered. Records acquired for the Melbourne Water Corporation by the Seismology Research Centre provide one of the best documented examples of triggered seismicity in the world, and demonstrate the importance of water pore pressure, permeability and diffusion in this process.

On the 25th September 1996, the magnitude ML 5.0 earthquake occurred at a depth of 12 km near the reservoir. The focal mechanism of the main shock showed southeast to northwest compression, consistent with the reverse faulting expected for the Yallourn Fault. The mechanisms of the aftershocks are more complicated. However, their locations correspond to also having occurred on the Yallourn Fault. In order to explain the apparent complex nature of the focal mechanisms of the aftershocks, a comprehensive 3-dimensional velocity model for the area should be developed.

The area now appears to be approaching the end of the period when triggered earthquakes are likely, and the earthquake activity is trending towards pre-filling levels. This assumes that increased pore pressure is the driving mechanism. However, given water level information from the reservoir, it could be argued that the mechanism for earthquakes occurring between 1990 and 1996 was an effect of a stress increase due to the surface water load rather than an increase in pore water pressure. Consequently, how the reservoir responds to refilling is unknown and will be of great interest.

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FAULTING IN THE NEWCASTLE AREA AND ITS RELATIONSHIP TO THE 1989 M5.6 NEWCASTLE EARTHQUAKE

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ABSTRACT:

The mechanisms responsible for the 1989 M_L5.6 Newcastle earthquake, including the fault responsible for the event, are still not understood, Either northeast or southwest-dipping faults could be responsible for the earthquake. This research tests both hypotheses by acquisition and interpretation of high-resolution and multi-channel seismic reflection data. All areas of the Newcastle-Hunter Valley region with Pleistocene or younger strata have been examined for evidence of active faulting. New single-channel marine seismic relection data has been acquired in the Newcastle area to fill gaps in the existing seismic data coverage and constrain the faulting identified offshore. No evidence for Quaternary faulting has been found in the Lake Macquarie to Port Stephens region. Thus, the offshore northwest-trending, southwest-dipping reverse fault located southeast of Newcastle represents the only young deformation and as such the most likely candidate for the causative fault responsible for the earthquake. This fault is here referred to as the Newcastle fault. Total area and slip on the fault may represent hundreds of Quaternary earthquakes, Seismicity in the region over the last 150 years has not been sufficient to rupture the entire fault.

1. INTRODUCTION

The identification and characterisation of the causative fault for the 1989 $M_L5.6$ Newcastle earthquake is critical to the understanding of processes responsible for the event and in determining the seismic hazard in the Newcastle-Hunter Valley region. At present we have limited knowledge of the location of the active fault(s), their location at seismogenic depths, how fast they are slipping, their recurrence intervals, the slip on the fault per earthquake event, and whether the previous earthquakes have released the total strain on the system. The identification of faults with the potential to have caused the 1989 earthquake is one step in the investigation of these factors. The 1989 event did not deform the surface onshore so imaging of the subsurface is required to characterise the fault.

The Newcastle-Hunter Valley region has an extensive history of moderately sized earthquakes, with six ~M5 earthquakes felt or recorded since 1841 (McCue et al., 1990). Over this period the typical earthquake for the Newcastle-Hunter region based on the measured and derived records appears to be in the range of Richter equivalent magnitude (M_L) 5.2 to 5.6 based on the modified Mercalli Intensity scale (McCue et al., 1990).

Based on the focal mechanism solution of the aftershock, the earthquake occurred on a broadly NW-SE striking reverse fault, with either a steep northeast or a shallow southwest dip. A steeply northeast-dipping fault is favoured by Gibson et al. (1990), based on the regional geology of the Newcastle area, while Huftile et al. (1999) and Van Arsdale and Cox (1998) suggest that a shallow southwest-dipping fault is more likely. Huftile et al. (1999) based this hypothesis on the identification of a monocline (Figure 1) in presumed Pleistocene sediments offshore of Newcastle, interpreted to be the result of movement on a shallowly southwest-dipping reverse fault. However, Huftile et al. (1999) suggested that this fault cannot be the causative fault for the 1989 earthquake because it terminates approximately 20 km southeast of Newcastle.

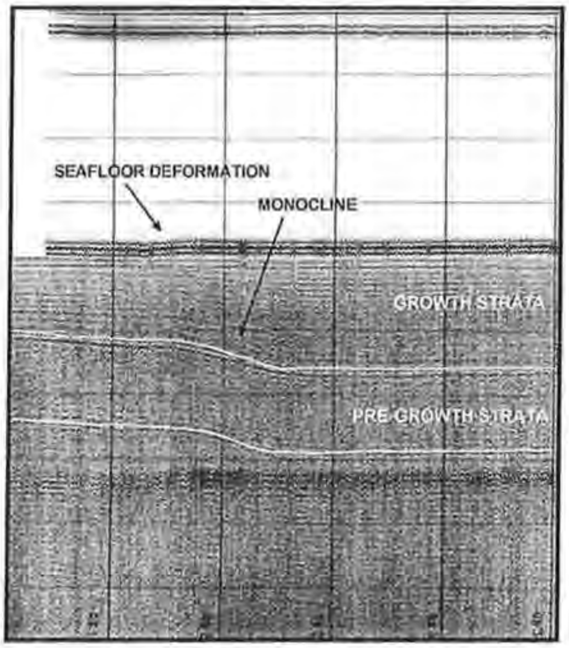

Figure 1. Seismic line LM 5 showing the monocline identified offshore. Growth strata represents those stratum deposited during uplift of the monocline. Pregrowth stratum were deposited prior to deformation. Minor seafloor deformation can be seen above the monocline and possibly represents flexure of the seafloor during folding. Water depth is about 135 m and the section is 3.75 km long.

The non-unique solution provided by the earthquake focal mechanism restricts the possibilities for the near surface location of the causative fault. If a second fault was responsible for the 1989 event, evidence for its presence should be in Pleistocene and younger strata onshore or near shore some point along the strike of the fault. A steeply east-dipping fault favoured by Gibson et al. (1990) should affect Pleistocene sediments in Lake Macquarie. A shallowly southwest-dipping en echelon fault, related to the offshore monocline, should pass through the Newcastle Bight embayment between the Newcastle CBD and Port Stephens. Both of these hypotheses have been tested as part of this study. The third or "null" hypothesis is the fault present offshore of Newcastle is the only active fault currently present in the Newcastle-Hunter Valley region and is therefore the causative fault for the 1989 earthquake.

The Newcastle-Hunter Valley region lies within the Permo-Triassic Sydney basin, the southern extension of the larger structurally controlled Sydney-Gunnedah-Bowen basin system (Bembrick et al., 1973). Permian age rocks dominate the area, with an extensive cover of Quaternary fluvio-estuarine and beach deposits present both in the Newcastle region and in the vicinity of the Hunter River extending up to Singleton (figure 2). A number of major structural features are also present, with the shallowly northeast-dipping Hunter-Mooki thrust fault the dominant structure in the area. The structural features of the area become more subdued and difficult to identify in the Newcastle and lower Hunter Valley regions as a result of the overlying Quaternary cover. Structures in the region record an extensive and complex tectonic history, from the Late Carboniferous continuing to the present, as made evident by the recent occurrence of earthquakes in the region.

2. METHODOLOGY

Acquisition and interpretation of seismic reflection data from throughout the onshore and offshore Newcastle-Hunter Valley region were the major methodologies employed for this investigation. In the absence of any significant paleoseismologic indicators or aftershocks and because of the failure of the causative fault to rupture the surface, seismic reflection profiles provide the best means of identifying the near-surface location of active faulting that is the result of repeated earthquakes. While discrete ruptures may not usually be obvious on seismic reflection profiles, areas of active deformation in Pleistocene sediments may be identified, and may appear as anticlinal or monoclinal features over which sediment is found to thin and reflectors disappear (e.g. Huftile et al., 1999).

Data sets used for this investigation include onshore and offshore multi-channel seismic reflection data, onshore and offshore high-resolution seismic reflection data, and additional high-resolution marine seismic reflection data acquired specifically for this investigation (Figure 2). These data sets provide extensive coverage of the area from Port Stephens to the southern portion of Lake Macquarie and west to Maitland including all areas of Pleistocene and younger deposition such as Lake Macquarie, Port Stephens, offshore Newcastle, the Hunter Valley, and Hunter River. The quality of these data sets is variable, ranging from poor to very good, with the results largely dependent on the conditions in the sediments and the presence of geophysical artefacts in the record. Most of these data were acquired to test sedimentological problems and were reanalysed here for neotectonic signatures.

A total of approximately 43 km of new high-resolution seismic reflection data were acquired in the north and south arms of the Hunter River and in the northern section of Lake Macquarie (Figure 2). These data were acquired to test both the southwest- and northeast-dipping hypotheses and to enhance the resolution of the available pre-existing data by targeting the Quaternary section displayed by these records.

3. RESULTS

Analysis of the pre-existing offshore seismic reflection data has revealed a Quaternary section that is relatively undeformed. The only resolvable Quaternary deformation revealed by the seismic data is the Newcastle fault, the northwest-striking and southeast-dipping fault located 20 km southeast and offshore of Newcastle (Van Arsdale and Cox, 1998; Huftile et al., 1999). Minor seafloor deformation can be seen above the monocline on seismic line LM 5 (Figure 1), which gives further indication of the young age of this feature. The top of the pregrowth strata has been correlated to the S1 unconformity that juxtaposes Miocene and Pleistocene strata onshore (Davies, 1979). No evidence of a westward extension of this fault was found between the western termination of this fault and the coast between Port Stephens and Swansea (Figure 2). This result therefore invalidates the hypotheses that the fault either steps to the south and passes through to Lake Macquarie or that the fault steps north and continues through the Newcastle Bight embayment. The pre-existing and newly acquired seismic reflection data from Lake Macquarie, the Hunter River and the data from Port Stephens reveal no Quaternary deformation despite the presence of faults in the bedrock. Analysis of the limited and essentially poor quality data from onshore areas west and northwest of Newcastle, while showing some significant faults in bedrock, do not appear to display any deformation of the overlying Quaternary sediments.

Interpretation of the seismic data sets suggest the presence of only one fault deforming Quaternary sediments in the Newcastle- Hunter Valley region. As such, this result, along with the error associated with the location of the earthquake mainshock (McCue et al., 1990), suggests that the Newcastle fault is the most likely candidate for the causative fault responsible for the 1989 earthquake.

4. DISCUSSION

According to Wells and Coppersmith (1994) the 1989 event (M_L5.6) would have ruptured an area of approximately 28 km^2 based on a world wide data set, representing only a small fraction of the area of Quaternary faulting currently mapped in the region (Figure 1; Huftile et al., 1999). The total area of rupture during six ~M5.6 events would be less than 170 km^2. The fault mapped by Chaytor (2000) is a minimum of 30 km long with part of the fault unmapped on the continental slope. The greatest amount of uplift is at the shelf-slope break (Figure 2). Assuming a dip of 39°, and the upper 15 km of the crust being brittle, we estimate a fault area greater than 700 km^2. This therefore suggests that the Newcastle fault, if responsible for the earthquakes in the

Newcastle region, has not completely ruptured in recorded history. The maximum uplift of approximately 30 m as displayed by the southern end of the monocline (Figure 2) indicates a maximum slip on the fault of about 40 m. According to Wells & Coppersmith (1994) a maximum displacement of approximately 0.025 m per M5.6 earthquake can be expected, and as such the uplift on the southern section of the Newcastle fault could represent as many as 1200 M5.6 earthquakes and a cumulative seismic moment of 5.32×10^{27} dyne.cm. This calculation provides an estimate of these factors and is done only to show that earthquakes on the Newcastle fault may be an ongoing process.

This result should be of interest to engineers in the Newcastle region. It suggests that any future events should occur south of Newcastle. Slip on a reverse fault striking approximately 330° and dipping southwest would result in a first motion of a combination of vertical and northeast directed accelerations. Shear waves (S-waves) generated by an earthquake on this fault are likely to propagate northwest towards Newcastle with strong ground movement vibrating roughly east-west, and as such represent a significant hazard.

Future constraints on the seismic hazard in the region are needed. The extent of the monocline to the southeast on the continental slope must be determined with further seismic reflection data or with detailed bathymetric analysis. Movement rates on the monocline can best be found by drilling wells on both sides of the monocline and the detailed correlation of recovered Quaternary sediments. In addition, a tsunami threat to the Sydney-Central Coast region of New South Wales posed by the possibility of future earthquakes on Newcastle fault requires consideration.

5. ACKNOWLEDGEMENTS

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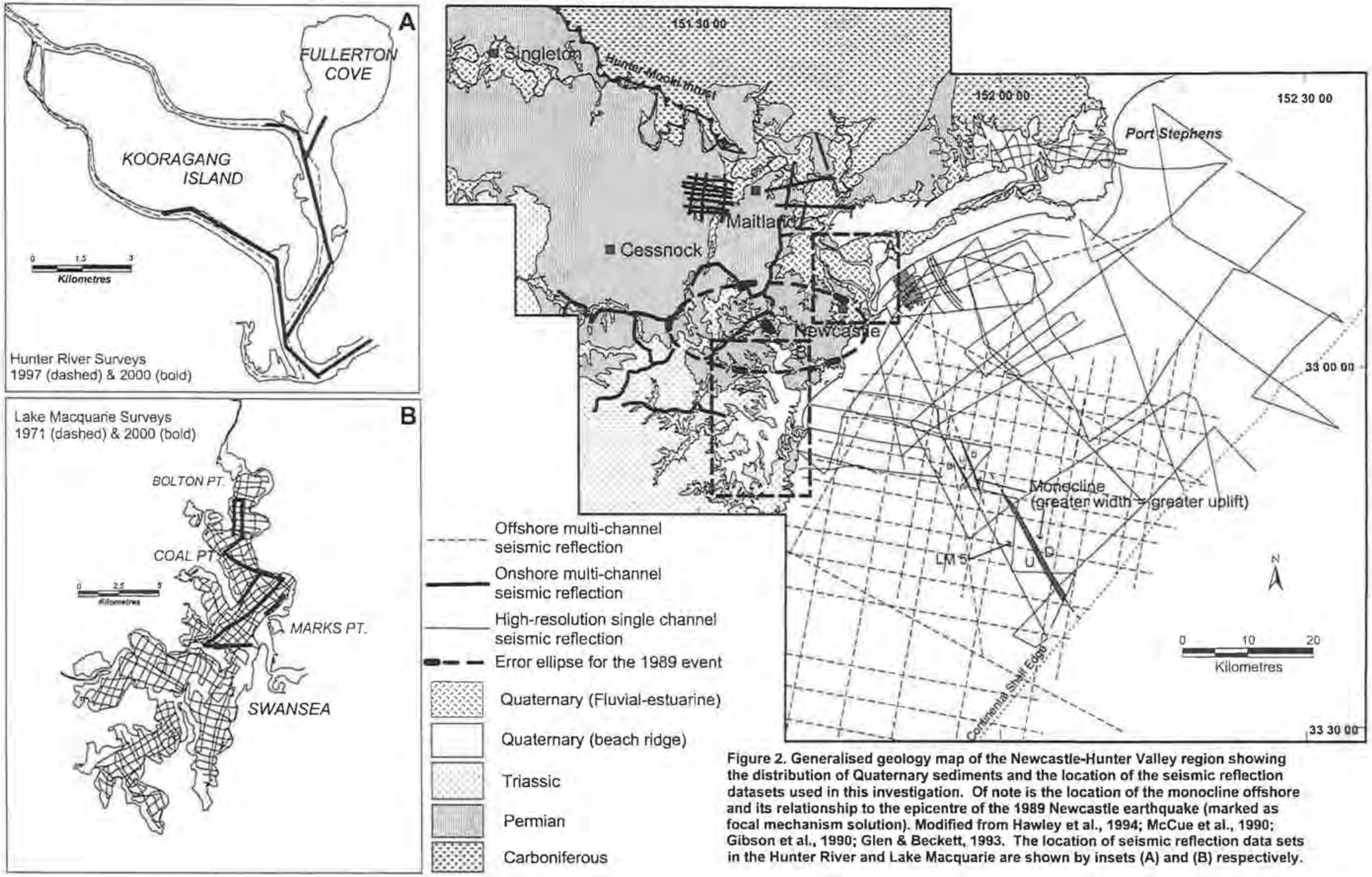

Figure 2. Generalised geology map of the Newcastle-Hunter Valley region showing the distribution of Quaternary sediments and the location of the seismic reflection datasets used in this investigation. Of note is the location of the monocline offshore and its relationship to the epicentre of the 1989 Newcastle earthquake (marked as focal mechanism solution). Modified from Hawley et al., 1994; McCue et al., 1990; Gibson et al., 1990; Glen & Beckett, 1993. The location of seismic reflection data sets in the Hunter River and Lake Macquarie are shown by insets (A) and (B) respectively.

SEISMIC MICROZONATION OF BUNDABERG, QUEENSLAND

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ABSTRACT:

Ground velocity microseismograms have been collected from 183 test sites within the Bundaberg city area. Response spectra have been calculated for those sites using the Nakamura technique and a seismic risk microzonation has been carried out for Bundaberg. The results can be used to supplement the general recommendations given in Australian Standard AS 1170.4-1993 for earthquake loadings within the city.

Using the site response spectra in conjunction with a shear wave velocity determined from Standard Penetration Tests sedimentary depths within the Bundaberg city area have been estimated. Measurements of the sedimentary depths in boreholes adjacent to nine of the test sites have been obtained and compared to the estimations. The results of the comparison show that the calculated estimations give a reasonable indication of the sedimentary depth.

Paper No. 10

MICROZONATION OF BUNDABERG CITY
M.L.Turnbull

1. DATA COLLECTION

During January of 1998 and 2000, 183 sites were occupied in Bundaberg city area. The sites were chosen to be as near as possible to the intersection of the 1000 m grid lines on the Hema Bundaberg street map. Data were collected using a combination of Kelunji Classic seismograph and Sprengnether S6000 triaxial sensor. The sensor was placed on bitumen road surface, levelled, and orientated according to the manufacturer's recommendation (i.e. arrow pointing west). The seismograph was manually triggered for a period of 120 seconds, recording at 100 samples per second.

2. DATA ANALYSIS

The seismograms recorded at each of the data collection sites were analyzed in accordance with a microzonation methodology developed by the author based on site response spectra calculated using the Nakamura technique.

For the purposes of microzonation it was assumed that the generalised natural shaking frequency (f_N), of a building with N stories, is approximated by Eq 1.

$$f_N \approx 10 / N \text{ (Hz)} \ldots \text{Eq 1}$$

Table 1 lists the range of shaking frequencies relevant to the proposed building classes.

Building Class	Range of number of storeys	Calculated frequency range	Frequency range used for analysis
Low-rise	1 to 3	10 Hz to 3.3 Hz	$2.9 \leq$ Hz ≤ 10
Medium-rise	4 to 9	2.5 Hz to 1.1 Hz	$1.1 \leq$ Hz < 2.9
High-rise	10+	≤ 1.0 Hz	$0.5 \leq$ Hz < 1.1

Table 1 : Shaking frequency ranges for building classes.

In conformity with the AS1170.4 – 1993 site factor allocations, and following analysis of the site response spectra to determine the individual average, and maximum average response gains for the resonant frequency range of each building category, microzone site factors (S_M) were allocated according to the normalisation method indicated in Table 2. This resulted in microzone site factors for low-rise (1 to 3 storeys), medium-rise (4 to 9 storeys) and high-rise buildings (10 to 20 storeys), depending on the relative horizontal ground movement amplification exhibited at that site.

Relationship of average gain to maximum average gain for Normalisation of MicrozonationSite Factor	Site factor (S_M)
$\mathbb{G} < 1.0$	0.67
$1.0 \leq \mathbb{G} < (\mathbb{G}_{Max} * 0.25)$	1.00
$(\mathbb{G}_{Max} * 0.25) \leq \mathbb{G} < (\mathbb{G}_{Max} * 0.50)$	1.25
$(\mathbb{G}_{Max} * 0.50) \leq \mathbb{G} < (\mathbb{G}_{Max} * 0.75)$	1.50
$(\mathbb{G}_{Max} * 0.75) \leq \mathbb{G} \leq \mathbb{G}_{Max}$	2.00

Table 2 : Normalisation of Site Factors.

3. MICROZONATION MAPS

The author has developed software that will accept any number of Kelunji classic seismograms and perform microzonation analysis in accordance with the above methodology. By interpolation of the site factors produced by the microzonation software three contour maps (Figures 1, 2 & 3) were drawn indicating the relative earthquake risk zones in Bundaberg.

All three maps display several well-defined zones of S_M values superimposed on a background of 1.25.

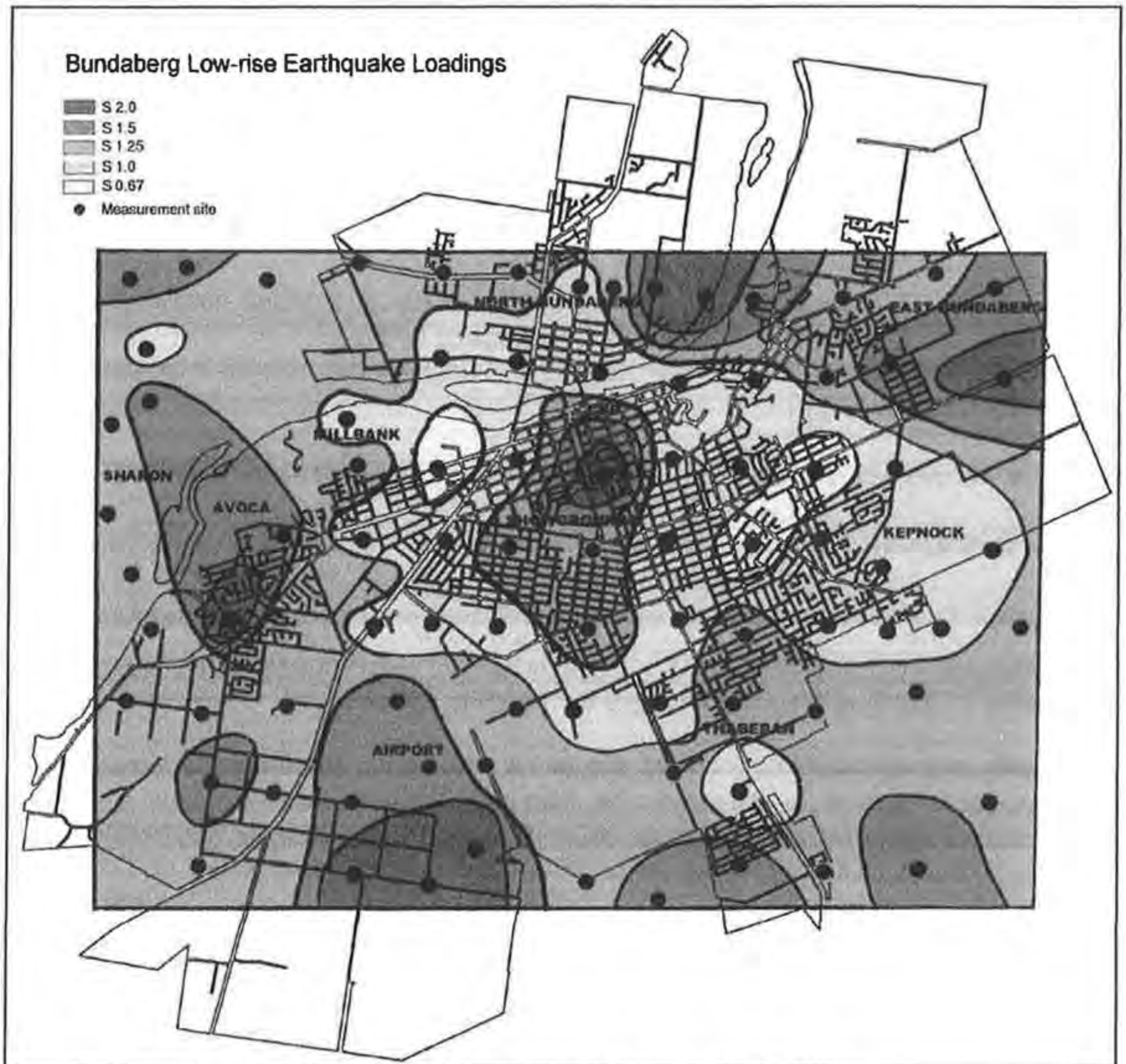

Figure 1: Earthquake site loading factors for Low-rise buildings

In the center of the map for low-rise buildings (Figure 1) is a large 1.0 zone extending east to west from Kepnock to Millbank, and north to south from North Bundaberg to Thabeban. This *central zone* encloses an oblong *central island* of higher values that separates two zones of 0.67 to the east and west. The *central island* contains a 2.0 core just south of the CBD. The oblong *central island* of high values has apparent extensions in the areas of the Airport and the eastern part of North Bundaberg. The *central zone* is framed by a series of high value zones to the north, west and south. The high value zones in the areas of Avoca/Sharon, the Airport, East Bundaberg and the east section of North Bundaberg exhibit core values of 2.0.

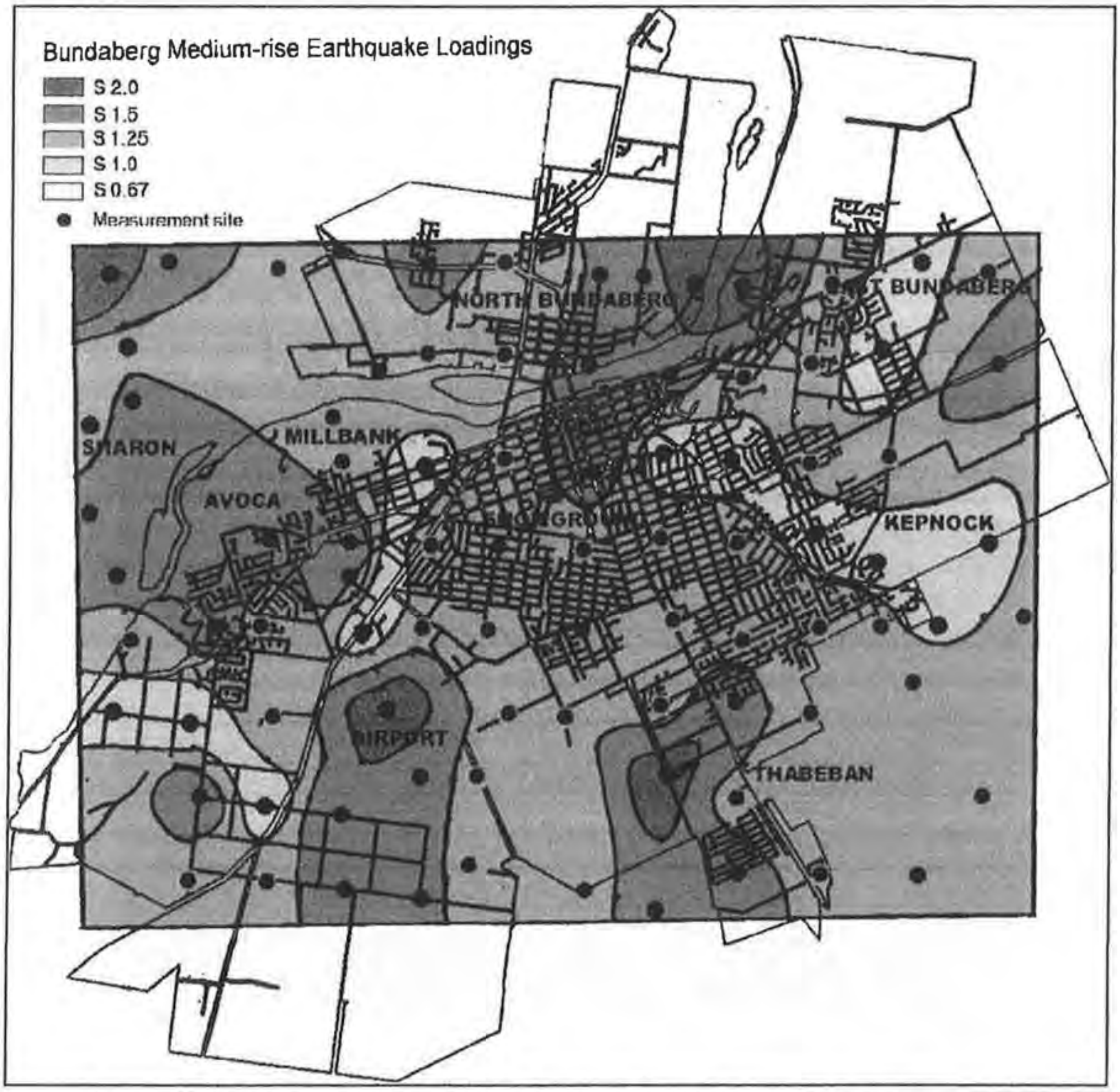

Figure 2: Earthquake site loading factors for Medium-rise buildings

Distinctive features of the map for medium-rise buildings (Figure 2) are the two high value zones thrusting into the central area of the map from the NNE and SSW. In the center is a broad background area that separates one oblong 1.0 zone (from Kepnock through almost to the CBD) in the east from another (from Millbank down to Sugarlands) in the west. The perimeter is flanked by a series of high value zones to the north, west and south. The high value zones in the areas of the Airport, south Thabeban, the east section of North Bundaberg and north Sharon exhibit core values of 2.0.

In the center of the map for high-rise buildings (Figure 3) are two 1.0 *central zones* extending east to west from Kepnock to Millbank, and north to south from North Bundaberg to Thabeban separated by a background *central corridor*. The *central zones* are framed by a series of high value zones to the north, west and south. The high value zones in the areas of the Airport, south Thabeban, East Bundaberg and the east section of North Bundaberg exhibit core values of 2.0.

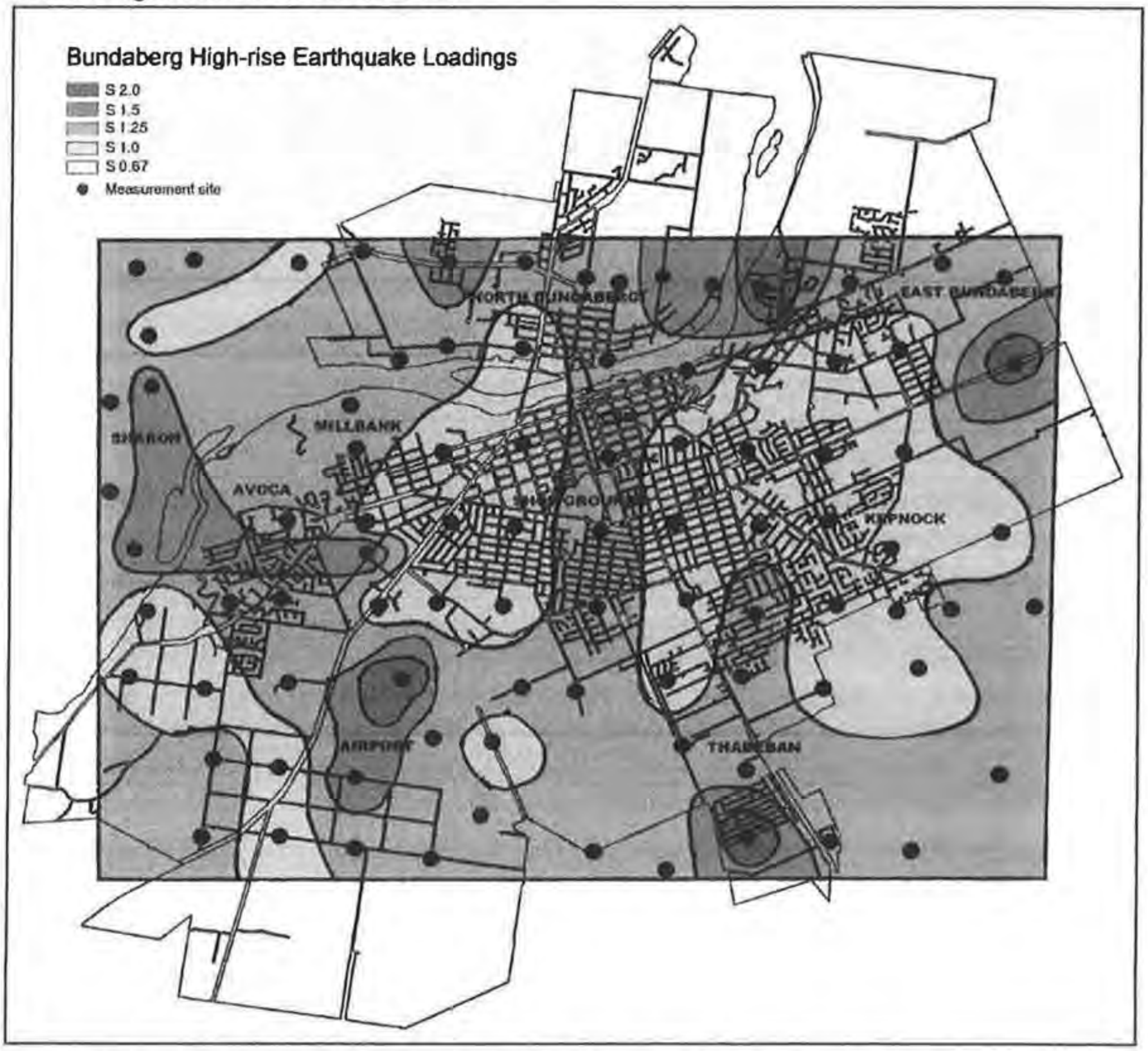

Figure 3: Earthquake site loading factors for High-rise buildings

4. RELATIONSHIP BETWEEN SPT AND SHEAR WAVE VELOCITY

Ganev et.al. provide a set of measured Standard Penetration Test (SPT) with associated measured shear wave velocities. The author has used these data to derive a linear correlation between SPT and shear wave velocity (v_s) (See Figure 4 and Eq 2)

$$v_s\,(ms^{-1}) \approx 3.9SPT + 182.9 \quad \ldots \quad \text{Eq. 2}$$

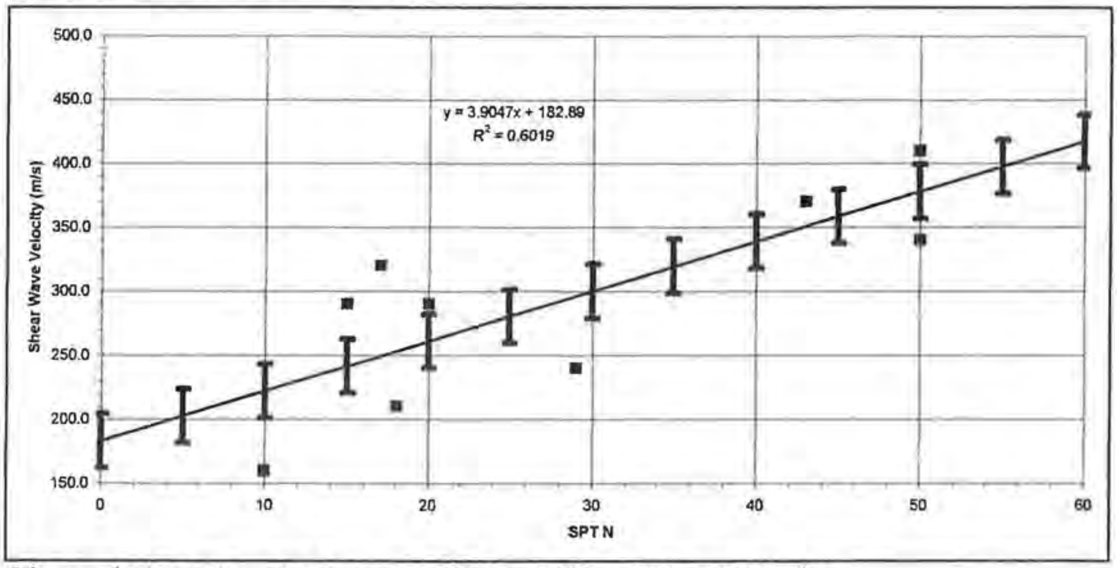

Figure 4 : Relationship between SPT and Shear Wave Velocity

5. DETERMINATION OF SHEAR WAVE VELOCITY.

Three SPTs obtained from a building site in the Bundaberg CBD (Cameron MCNamara) were converted to shear wave velocities using Eq. 2 (See Table 3).

Site and depth	Measured SPT N	Calculated Shear Wave Velocity V_s (ms^{-1})
WB1 (6 m)	32	308
WB2 (7.7 m)	34	316
WB3 (8.55 m)	26	284
	Average velocity	303

Table 3 : Shear wave velocities in Bundaberg CBD

From the reported distribution and nature of the sediments in the Bundaberg city area the average value of the shear wave velocity was assumed for all sedimentary sites within the microzonation area.

6. ESTIMATION OF BUNDABERG CITY SEDIMENT DEPTHS FROM SPECTRAL RESULTS

In a simple two layer model it can be shown (Seht & Wohlenberg) that the approximate relationship between fundamental resonant frequency (f_r), shear wave velocity (v_s), and sediment depth (d) is given by:

$$f_r \approx v_s / 4d \qquad \ldots \qquad \text{Eq. 3}$$

Using the estimated average shear wave velocity and the fundamental resonant frequency peaks of the site response spectra, the sediment depths at the microzonation measurement sites were calculated. The calculated depths ranged from 14 m to 97 m, with an average of 44 m and a sample standard deviation of 14 m.

7. COMPARISON OF ESTIMATED DEPTHS TO MEASURED DEPTHS

Test borehole logs from the Geo-Eng Bundaberg groundwater investigation for boreholes located within the limits of the microzonation boundary were examined, and those that intersected the Burrum Coal Measures were identified from comments in the borehole logs. Sediment depths for nine boreholes were plotted against calculated depths for adjacent microzonation measurement sites (See Figure 5). The black diagonal line represents the position of data points that have ideal coincidence. The dashed lines indicate the extremities of the spread of the estimations from the ideal. The dotted line is the line of best fit calculated from linear regression of the data points, forced through the origin. Whilst the correlation coefficient for the best-fit line is very low (0.2), its proximity to the ideal line and the well-defined nature of the data spread indicates a favourable result for the comparison.

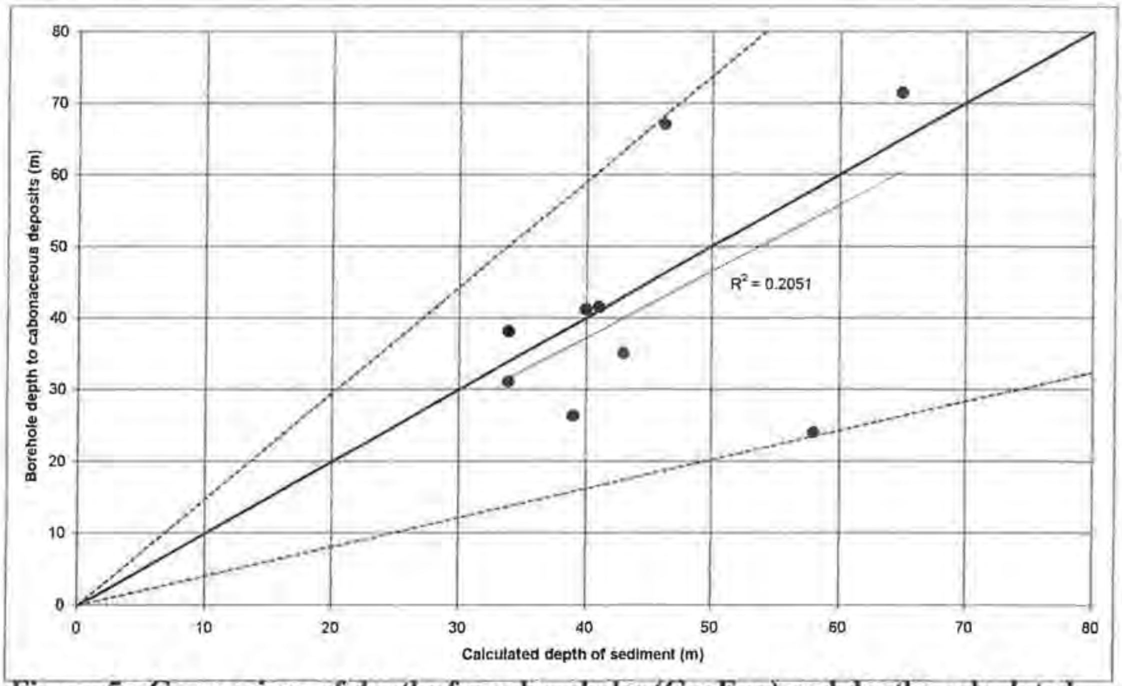

Figure 5 : Comparison of depths from boreholes (GeoEng) and depths calculated from microtremor sites.

Elimination of the two outliers improves the correlation coefficient to 0.75; however, in the absence of further investigation into the heterogeneity or otherwise of the geology within the area, removal of the outliers could not be supported. Whilst the calculated depths seem to be capable of providing very good approximations of the actual depths, in the absence of supporting criteria they should be interpreted with caution.

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BUILDING TO RESIST EARTHQUAKES - THREE GENERATIONS OF CODES IN AUSTRALIA

CHARLES BUBB, GERHARD HOROSCHUN, DAVID LOVE AND KEVIN MCCUE

AUTHORS:

Charles Bubb was the founding President of AEES and its first life member. He was until 1987 Director of Engineering for the Federal Dept of Housing and Construction (aka CommWorks). He chaired the first Standards Australia Earthquake Code Committee when AS2121 was published. He is a Fellow of IEAust and Emeritus Member of the College of Civil Engineers.

Gerhard Horoschun is a lecturer at the Australian Defence Force Academy. He was for many years a senior engineer with Australian Construction Services and during this period contributed to the development of the first two generations of the Australian Earthquake Code.

David Love heads the seismology group at PIRSA and was a member of the Standards working groups for the last two iterations of the Loading Code.

Kevin McCue is a seismologist at AGSO, a Fellow of IEAust and scribe of three of the Standards working groups for the latest revision of the Loading Code.

ABSTRACT:

The new draft joint Australia/New Zealand Loading code soon to go out for public comment will be the third generation to codify earthquake resistant design recommendations for use in Australia. Significant changes have accompanied each revision including compliance with ISO standards but the changes have not necessarily all been seen to be progressive.

Building to Resist Earthquakes - Three Generations of Codes in Australia

Charles Bubb, Gerhard Horoschun, David Love and Kevin McCue

1. Introduction

For nearly 200 years building practitioners in Australia saw no need to protect against lateral loads other than wind, until a large earthquake at Meckering on 14 October 1968 wrecked the small Western Australian wheat belt town and caused significant damage in Perth. As a consequence engineers wrote Australia's first earthquake code following reviews of earthquake engineering practice worldwide, and after numerous meetings and seminars and public consultation. AS2121-1979 was a self sufficient code incorporating all minimum requirements for a structural design code in that it incorporated the loading, structural response, material behaviour and commentary in a single volume.

Larger earthquakes had occurred in Australia in 1941 (M6.9) and 1906 (M7.2), both in Western Australia, but sufficiently far from populated centres that little or no damage resulted. After 1946 there was a population explosion with a strong demand for and growth in supporting infrastructure such as dams, railways and roads, power stations and grid, gas pipelines and tall buildings. The lessons of Meckering were not lost on the engineering profession that set in train the processes leading to publication of AS2121 in 1979.

Revision of AS2121 commenced in 1988 and the process was fast tracked following the 1989 Newcastle earthquake. In the meantime though it was decided that separate loading and materials codes and a separate commentary would replace the holistic earthquake code. Thus was AS1170.4-1993 developed. This separation of design and loading was strongly opposed at the time by one of us.

Political agendas that encouraged economic agreement across the Tasman were the stimulus for the third generation of codes by harmonising Australian and New Zealand Standards. This latest version will go out for public comment soon. It had necessarily to grapple with a wider range of loadings than either individual code, catering for low to moderate intraplate and moderate to high interplate seismicity.

Our paper comments on some of the changes in the codes and compliance practice and how those might impact on earthquake engineering practice, building resilience and safety.

2. The Three Generations

2.1 A Self Contained Code. AS2121-1979 was a self sufficient code incorporating all minimum requirements for a structural design code in that it incorporated the loading, structural response, material behaviour and commentary in a single volume. The zoning requirement was unique, based not on the ground motion at a single return period but for the worst case in 6000 years (Figure 3.1 in AS2121).

2.2 A Loading Code. AS1170.4-1993 was developed as a loading code with a separate commentary. Materials related provisions were transferred to the relevant materials codes.

The difficulty with this practice is that the committee members of the basic materials codes typically have little experience or interest in the response of structures to earthquake and so the relevant clauses are delegated to a subcommittee with limited representation. There is thus the risk that the provisions developed are not entirely appropriate for the level of the risk assumed by the loading code. In addition there is the danger that such an approach implies that earthquake resistant design is simply divisible into two phases;

1. assessment of earthquake loads
2. design of structure for the assessed loads

As the nature of the load experienced depends on the structure, structural materials and its form this is not so. The earthquake loading process is, in reality, more akin to a feedback loop – and that as modifications are made to the structural concept, there is an impact on the structural loads. There is a danger with the current division of responsibility that this aspect is not fully appreciated by all designers.

The zones of AS2121 were replaced by a contour map of acceleration coefficient defined at a single return period of 475 years (nominally 500 years) interpolated at 0.01 g intervals. Whilst this removed the difficulties associated with the sudden jumps in design requirements at the zone boundaries it led to an intricate hazard map which implies an unwarranted understanding of the earthquake process.

2.3 A Performance-Based Loading Code. The new draft harmonised code for New Zealand and Australia follows the basic philosophy of AS1170.4-1993 and its NZ counterpart NZ4203:1992, being a Loading Code and expanded to include performance demand ie design for different return periods or exceedance probabilities in the structure's lifetime. For the first time the code is not being written by the Standards committee directly, the process was outsourced to a private company responsible for interpreting the working group outputs and translating them into a document which satisfies the Building Code Board (and the working groups). Whilst this provides the opportunity for a more rapid development of the code it is clear that without careful monitoring by the relevant working groups, there is the risk that the document may simply represent the interpretation of an individual.

3. Hazard mapping

3.1 Hazard assessment Although a low gain seismograph network was in place in 1906, detection capability remained near magnitude 6 until the International Geophysical Year in 1957 when various bodies began operating high gain stations or networks. By the time AS2121-1979 was being composed, the network was capable of monitoring the continent to magnitude 4 and the first seismic hazard maps for parts of Australia had been published (Underwood, 1973; McCue, 1973; McEwin et al, 1976) and these were incorporated into the code.

By the late 1980's when a new committee was drafting AS1170.4, earthquake monitoring coverage of the continent was almost complete to magnitude 3 and studies of the historical record were well advanced with publication of the first 2 volumes of the 3 volume of the Atlas of Isoseismal Maps (Everingham and others, 1982; Rynn and others, 1987).

3.2 Spectrum At the time AS2121-1979 was written no local accelerograms had been recorded. Although there are now more accelerographs, and records of near source data are increasing, the new joint Standard uses spectra developed from overseas data (Somerville and others, 1998). The limited local data seem to fit these spectra well.

4. Site Factor S

In AS2121-1979 the default soil factor, called the *site-structure resonance factor* was defined to be S=1.5 (page 23, AS2121) or calculated using a sophisticated method based on the ratio of the building period and the natural period of the soil layer underneath the structure - just as theory would demand. Whilst there were difficulties with this approach the factor accounted in an approximate manner for an effect commonly experienced. In AS1170.4 this factor was replaced with a table giving the site factor S in terms of the soil profile without even a mention of building period. This was an unfortunate outcome in that it implied that a stiff site, such as a rock site, is always good and a soft site is always bad. The evidence from a number of earthquakes indicates that the severity of damage caused by an earthquake is often linked to the degree of resonance of the building and site periods. The approach to the determination of the site factor adopted by AS1170.4 is retained by the current draft code.

5. Disaggregation of the Code

Mud brick or Adobe The Loading Codes give no guidance on materials. In choosing an ancient/modern building material like mud brick it would not be normal practice for a builder/owner to look into the material code to check for its earthquake response/suitability nor to consult historical records which testify that urban dwellers in towns and cities built from mud brick have perished repeatedly during earthquakes. Yet modern Australian designers and builders have embraced mud brick, and homes and government buildings such as tourist visitors centres are now being built, with no thought to earthquakes.

A special zone category, category A, was introduced in AS2121-1979 to restrict load-bearing brick construction, specifically to restrict the practice in Perth where several such buildings up to 10 storeys had already been erected. During the 1968 Meckering earthquake the deflections were so large that several of these buildings pounded together although subsequent inspection revealed no obvious damage.

The construction of such buildings was still possible with AS2121 (page 39, table 11.2) but required a K value of 3.2 practically prevented their construction in Adelaide and Perth. Under AS1170.4, detailing was required under design category A and a height restriction of 3 storeys was introduced for design category B, the lowest two categories.

6. Compliance practice

Prior to States and Territories adopting the Australian Building Code (ABC) in or about 1990, each Government was responsible for producing its own building code which was based on the Australian Model Uniform Building Code but with variations; some did call up AS2121-1979 and some did not. The Earthquake Code was adopted and used by the Commonwealth Government (Bubb, 1999) and South Australian and Western Australian Governments. The Commonwealth and Western Australian Governments did so in response to the Meckering WA earthquake. Likewise in South Australia where the memory of the 1954 Adelaide has somehow survived. Some Local Government Councils also adopted the Earthquake Code including the Wollongong NSW Council after their experience of the damaging earthquakes of 1961 and 1973.

Today all Governments including the Commonwealth Government require compliance with the ABC so the question of compliance is interesting. Take the AGSO Building, Canberra as an example. This is a modern building completed in December 1997 for occupation the following March.

In the planning phase in the early 1990's the architect noted concerns that the building ought to comply with the yet-to-be-published Loading Code AS1170.4. What eventuated was that one quadrant of the building was certified design category C, the other three quadrants design category B. The difference is in the detailing, extra steel was included in the three floor diaphragms and three stairwells of the southwest quadrant, stiffening this part of the building. Such detailing has the potential, particularly under earthquake overload, to induce significant torsional actions to the potential detriment of the building. How can this happen? Who now oversees and approves the plans on which this intention was clearly noted?

7. Displacement towards the Future

An earthquake load is not like a wind load! In an earthquake the building foundation is forced to move with the ground whereas the upper parts of the building attempt, through their inertia, to remain in place. Relative displacements between the ground and structure masses therefore develop which generate internal forces that accelerate the structure in an attempt to cause the structure to catch up to the ground motion. As earthquake ground motions are erratic the structure can never quite catch the ground motion and in its attempts may often overshoot the ground displacement. From some perspectives therefore, structural displacement can be seen to be the physical parameter driving the earthquake response.

The new approach to design buildings for this displacement and its consequences (Priestley, 1995) rather than the implied, as opposed to applied, force or acceleration should give designers a better feel for reality than do the current codes.

8. Discussion

Kircher (2000) points out that the same problem exists in the US as we are faced with due to harmonisation. One code (UBC) is used where earthquake design is routinely required while other codes (NBC and SBC) are used where seismic design is typically the exception rather than the rule. Also the Western US region (WUS) is dominated by activity on faults while the Central and Eastern US region (CEUS) has relatively few active faults and activity is distributed in *background* zones.

Kircher's hazard curves for US cities illustrates a significant difference in the slopes of WUS and CEUS hazard curves (his Figure 4). As he says, the traditional 500 year RP suits high seismicity regions but does not adequately define ground shaking in regions of low seismicity. In regions of low to moderate seismicity ground shaking is governed by the 2500 year RP MCE [Maximum Credible Earthquake]. Deterministic limits are used to effectively bound the ground shaking at sites close to highly active faults to avoid undue conservatism in high seismicity regions.

The results of this methodology are exhibited in Kircher (Figure 8) with design spectra for selected US cities. A very wide variation between cities can be seen at all periods. For example, the spectral acceleration for 1 sec period varies from approximately 0.1g for Chicago to 0.75g for Salt Lake City to an upper bound of almost 1.5g for the San Francisco Peninsula along the San Andreas Fault.

Kircher describes a parallel path approach between earth scientists and engineers. This division of effort and then recombination to produce final useable results is a sound and necessary basis. This is what we must strive for in Australasia if harmonisation is to succeed in practice. What we are about is engineering seismology after all. Not one or the other but both together.

The separation of an Earthquake Code into a pure loading code on the one hand with all the real engineering design and construction happening elsewhere creates severe problems for engineers whose normal practice is not dominated by earthquake resistant design. On the other hand earthquake design codes which are fully inclusive can appear highly prescriptive and limiting to experienced earthquake engineers.

One solution is to have both available to all and for the choice to be made at the beginning. The objection is the amount of work required to produce both.

The managerial solution is to have only a loading code and leave everything else to the market place and to market forces. This has the appearance of providing earthquake resistant buildings, structures and facilities for the community. In regions of high seismicity and therefore normal practice of earthquake resistant design, appearance and reality will match. However in regions of low to moderate seismicity and lack of familiarity with earthquake resistant design there will be a mismatch. Consequently many supposedly earthquake resistant facilities will not in reality be capable of resisting a damaging earthquake. Of course these are rare. But should we depend on the appearance or the reality?

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DIGITAL ISOSEISMAL MAPPING

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ABSTRACT:

The future of isoseismal mapping in Australia lies in two areas. Firstly there is the traditional isoseismal map of a large event, where accumulated results are used for attenuation. Secondly there is the small detailed map of a populated area, where results can be used to outline amplification. This division will be reflected in the reporting mechanism, with internet usage and automated processing for the populated areas and mailed questionnaires elsewhere.

In the last two years, this department has attempted to produce detailed isoseismal maps in digital form. While good maps have been produced, it has cost a considerable amount of time to get good locations. The ability to produce a detailed map quickly, either to post with a questionnaire, or to display an internet enquirer, will save time and deliver results..

Current attenuation functions are usually produced from isoseismal contours. The availability of point data will result in different functions

1. INTRODUCTION

Despite the number of published isoseismal maps in Australia, the number of moderate to large events with dense consistent reporting is quite few. We have only a handful of events with intensity MM7 combined with good locations and magnitudes. Given the importance of these in predicting damage from large events there is a need to focus efforts to produce results of value. Even in the US with its dense instrumentation the use of intensity reports is increasing.

2. INTERNET REPORTING

The rise of the internet has led to the heavy use of this method for intensity reporting in densely populated areas in USA. There is the potential to be totally swamped by the feedback in this method. Already automated methods have been arranged to handle the incoming data. The Gladstone SA event of 1999 produced 8 email replies via the Victorian website. The recent Boolarra South Vic event resulted in over 350 without advertisement. We need to prepare to produce the best from a limited number of events with limited resources.

I propose that isoseismal data should deliberately be handled in two segments: densely populated areas with good internet access, and other areas. Densely populated areas should be handled by internet access with questionnaires that can be automated. Locations should be accurate to better than postcode scale, but with fall back to postcode accuracy if necessary. The seismological target should be to delineate areas of amplification. Other areas should continue to be handled by the traditional questionnaire method, with an attempt at better locations, and greater density for a similar cost.

3. THE QUESTIONNAIRE

Despite interest in recent years, we have not settled on a standard questionnaire in Australia. With the addition of the internet option, the variability could rapidly escalate. A guiding factor for the internet scenario is the vast amount of data that may eventuate. Probably the best option is to follow the lead begun by the Humboldt Earthquake Education Centre (Dengler and Dewey 1998). Although this was a phone survey intended to mimic previous MMI surveys, the analysis points directly to a smaller group of questions with set replies. This has led to automated processing (Wald et al 1999). There should be some additional questions so that a standard MMI assessment can be made if necessary.

For areas of less population density, a more detailed questionnaire with room for plenty of extra comments is still a better alternative. There are advantages in a standard questionnaire and this should still be pursued. While a standard evaluation of replies would be ideal, this is difficult. Comments, particularly comparisons with previous events are valuable and cannot be systematised.

4. INTENSITY SCALES

It has been suggested that we should move from the Modified Mercalli scale to the European Macroseismic Scale (Grunthal, 1998). This may have some benefits, particularly for larger events where a significant amount of damage occurs. However for most other events it is unlikely to make much difference. In rural areas the quantity descriptions of EMS-98 are of little value for single buildings. We particularly need to grapple with the use of noise descriptions. We have continued to use the Modified Mercalli for the present.

For dense and automatically processed data, a number of new scales sporting acronyms are already arising (Wald et al 1999). While the authors warn against equating these directly with MMI assessments from questionnaires, indications are that the types will mesh well. With the density of data, averaging has produced stable enough data that decimal values are used.

It would appear that the practice in Canada is to send out considerably more than here. Cajka and Halchuk (1998) suggest that 1000 would normally have been sent for the M5.1 Cap-Rouge, Quebec event. With a greater number of returns there is less necessity to carefully consider individual reports. This should also result in less reports such as 3-4, 4+, 6- and so on.

5. QUESTIONNAIRE DISTRIBUTION

The distribution of questionnaires typically takes considerable time. We have used three main means. Firstly, by simply using the phone book to dial out and get addresses to mail to. This is very time consuming, but in very sparse areas is likely to get the best results. Secondly, we have kept a list of names and addresses of all past questionnaires mailed. Questionnaires are numbered so that if one is returned without a location, or with some item of particular interest, we are in a better position to follow up. The entry of the questionnaire number onto the database produces an easy to way to find an address later. The depopulation of rural Australia means that this method is only useful if not too many years have gone by. Thirdly, we have had good response in moderately populated areas by using 'Unaddressed mail', currently 10.9c per letter. We contacted the local post office to find out what delivery runs were available, then sent sufficient items with a request to deliver every second, third, or fourth delivery point. This service has recently been centralised, but hopefully will work well on the next event.

6. LOCATION ACCURACY

Since 1989 we have included a map on the back of the questionnaire. With the specific mail out, we now supply more detailed maps. These have been photocopied from printed maps, but with the ease of use of GIS software, we may produce our own in future. In the questionnaire we have requested section and hundred, which was for many years the standard way to describe land parcels in SA. We attempted to use this method to accurately locate all replies for the Padthaway (1998) and Gladstone (1999) earthquakes, using an on-line system, but a large number of locations have been renamed by a new system, leaving about 40% of replies to locate manually on old maps, which was particularly slow. The remainder were located using the less detailed map on the back. When we next use the 'unaddressed mail' option we may produce detailed maps for the particular postcode. This will not result in the high accuracy of the section and hundred, but should be quick. Given the substantial organisational effort in a mail out, a doubling of returns probably only adds 20 or 30 percent to the total effort if locations are easy.

For internet replies, various location options are becoming available. Postcode is the easiest but greater accuracy is desirable. This should be a fallback option. Other methods may not yet be quick and convenient enough, but are likely to improve.

7. ATTENUATION FUNCTIONS

The use of point data (i.e. one data value for one report) will change the calculation of attenuation functions. Traditionally this has been done using distances to manual contours based on the point data. The contours are normally drawn to enclose most of a particular intensity. With point data the use of scatter may change PML calculation. Figures 1 to 3 show the results of the first three surveys recorded digitally by this department. Surprisingly the Gaull attenuation function (Gaull et al 1990) underestimates the reported values in most cases. It is also less than the values obtained for the contours. This may be the result of estimating intensity values, different attenuation or the magnitude equation. There is a significant difference between the two magnitude 4.1 events, although these are very different geological terrains, and the Eyre Peninsula event was at night. The data are all plotted against hypocentral distance. In this case all the events were moderately deep (19 - 28 km). Assuming a shallower depth affects the closer points, predominantly higher intensities, to a greater extent.

8. THE PROBLEM OF NO REPLIES

The radius of perceptibility remains a difficult issue, as people are less likely to respond if they did not feel the event. This is particularly likely to be the case with internet replies. Rapid display of results on the internet may help overcome this problem, and requests for such replies may also be helpful. We did note that there were sufficient 'not felt' replies in the 'unaddressed mail'. By using median and percentile values for intensity 3 replies, the lack of 'not felt' replies becomes less of an issue. It is still desirable to have a fair number of 'not felt' points to have assurance that the intensity MM3 zone has been fully covered.

9. OLD ISOSEISMAL MAPS

We have begun the process of digitising old isoseismal maps. The maps are firstly scanned to the tif format. These are projected to decimal degrees by Smartimage, an extension to ArcView. The projected image can then be digitised (using ArcView) on screen with true ditigital culture also showing.

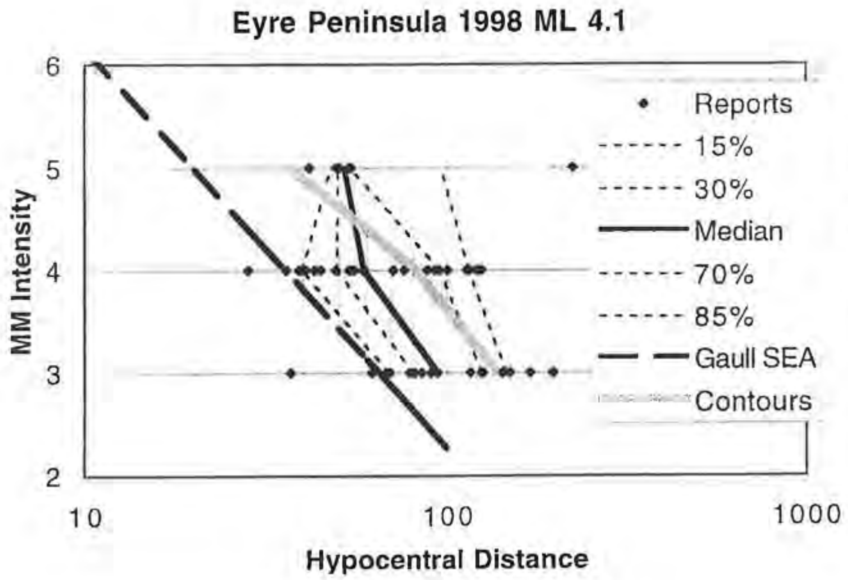

Figure 1

Intensity attenuation

Eyre Peninsula SA
26 February 1998
magnitude 4.1

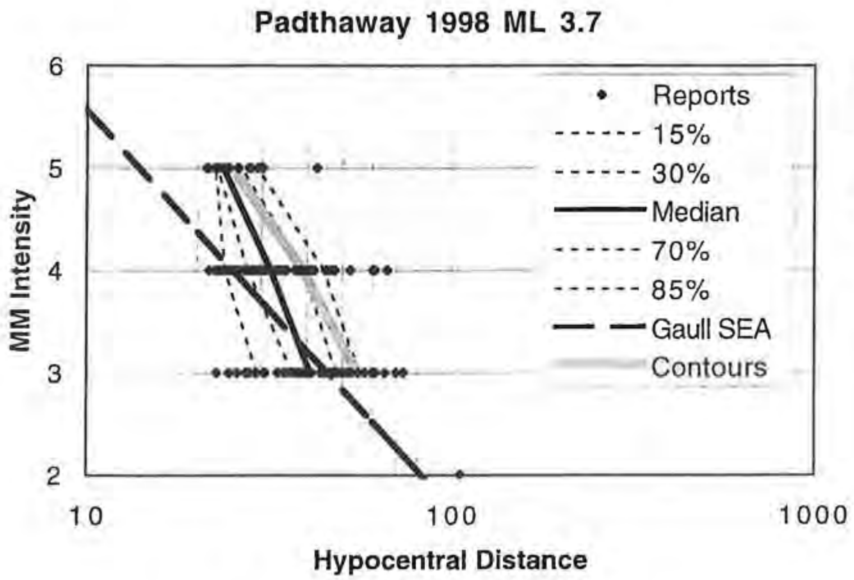

Figure 2

Intensity attenuation

Padthaway SA
11 March 1998
magnitude 3.7

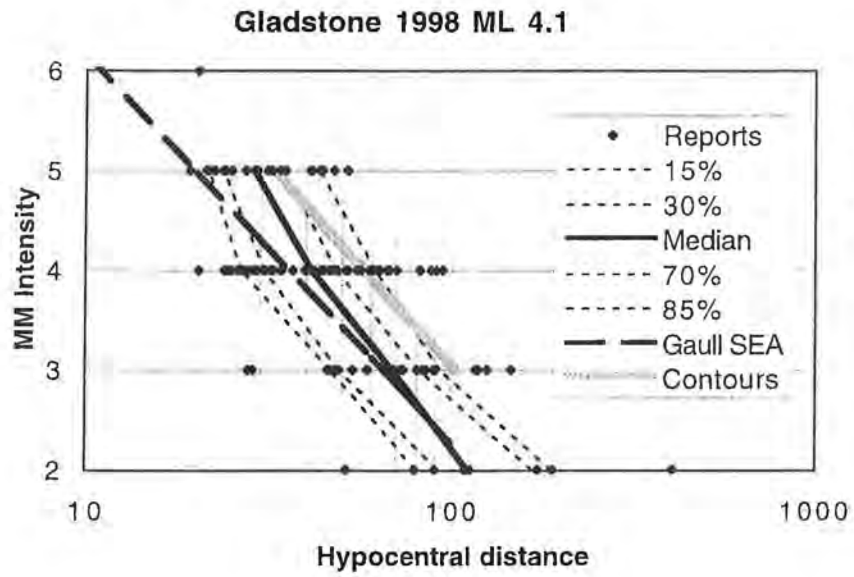

Figure 3

Intensity attenuation

Gladstone SA
18 August 1999
magnitude 4.1

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THE 17 MARCH 1999 APPIN ML 4.5 EARTHQUAKE

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AUTHORS:

Amy Brown studied geological engineering at RMIT, and has been working in earthquake hazard analysis at the Seismology Research Centre for several years. She is working on a revised earthquake hazard map of Australia that is more heavily based on regional geology than previous seismicity based maps.

Wayne Peck has more than ten years experience in field seismology, operating seismograph networks and installation of aftershock networks. He has also been involved in a range of seismological consulting projects, particularly in earthquake hazard analysis, and has just submitted a Masters thesis on earthquake lifelines.

Gary Gibson wrote his first earthquake location computer program in 1971, and has been trying to learn more about earthquakes since then. He is Vice-President of the Asian Seismological Commission, and is particularly interested in earthquake hazard analysis, seismic instrumentation and the relationship between earthquakes and dams.

Ian Landon-Jones is responsible for managing the safety of the dams owned by the Sydney Catchment Authority (SCA). He was instrumental in establishing the modern seismic network for the SCA and is particularly interested in earthquake hazards and their impacts on dams. He has been involved in many risk assessment studies involving earthquake loading on dams.

ABSTRACT: *(Full paper not available at time of printing)*

On March 17 1999 at 12:58 pm a magnitude ML 4.5 earthquake occurred near Appin, NSW. This was the largest event to occur within the Sydney Catchment Authority Seismic Network since it was upgraded in 1992. The event was recorded on 15 bedrock seismographs within a 100 kilometre radius, providing valuable attenuation information for the Sydney Basin.

The density of seismographs in the Sydney Catchment Authority Seismic Network made it possible to accurately determine the location of the event, with uncertainties less than two kilometres. With this location accuracy the depth of the earthquake was determined to be well below the current mining activities in the area, making it very unlikely that this event was mining induced event. This accuracy would not have been possible with the pre 1992 seismograph network.

The Appin earthquake triggered the Seismology Research Centre's (SRC) real time Earthquake Preparation, Alarm and Response (EPAR) system. The SRC was able to notify the relevant authorities that a moderate earthquake had occurred and provide a location, magnitude and estimated intensities at key facilities less than twenty minutes after the event.

EARTHQUAKE SCENARIO GROUND MOTION HAZARD MAPS FOR THE SAN FRANCISCO BAY REGION

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Dr. John F. Schneider - John joined AGSO in June 2000 as the Geohazards Research Group Leader and Project Leader for the Cities Project, which is developing geohazards risk models in key urban areas around Australia. John came to AGSO from the US, where for the past 5 years he was the Chief Scientist for Impact Forecasting in Chicago, a subsidiary to Aon Corporation devoted to developing catastrophe risk models for the insurance and financial industry. John has a Ph.D. in Geophysics from the University of Wisconsin.

Dr. Walter J. Silva - Walter is the Principal of Pacific Engineering and Analysis, a seismic hazard consulting company based in El Cerrito, California, USA. Walter has been working in the field of strong motion earthquake ground motion modeling for the past 30 years, developing approaches for earthquake engineering and risk modeling applications for critical facilities and local and regional hazard assessments worldwide. Walter has a Ph.D. in Geophysics from the University of California, Berkeley.

ABSTRACT: *(Full paper not available at time of printing)*

We have developed seismic hazard maps for two major earthquake scenarios in the San Francisco Bay Region. The maps incorporate the effects of source rupture directivity, crustal wave propagation and near-surface soil response, as well as the associated uncertainty. The maps will be useful to a broad audience of earthquake risk analysts to provide improved estimates of ground motion hazard for input to probable maximum loss analysis, structural designs and building codes, and urban disaster planning.

The scenario hazard maps present estimates of ground motion hazard for two key earthquakes identified by the US Geological Survey's Working Group on California Earthquake Probabilities: a M 7.9 rupturing the four northern segments of the San Andreas (repeat of 1906) fault; and a M 7.1 on the northern and southern segments of the Hayward fault. To generate rock motions, the approach combines the results of a numerical ground motion simulation model with empirical attenuation relations. In the numerical simulation model, the earthquake rupture and crustal wave propagation are represented using a stochastic finite-fault simulation method with random vibration theory (RVT) to generate suites of response spectra for each scenario. Four alternative empirical attenuation models are used that are appropriate and commonly used to estimate ground motions in California. To accommodate the effects of near-surface amplification in soil (or soft rock), frequency- and amplitude-dependent amplification factors were developed. These factors take into account depth to bedrock (for alluvial sites), soil type (based on near-surface geology), and nonlinear amplification effects at high strains. The amplification factors are applied to rock motions across a dense geographic grid, then averaged across models at several discrete spectral periods. The results are contoured to yield hazard maps for 5% damped response spectra at five periods (0, 0.2, 0.3, 1.0 and 3.0 sec) and for median and 1-sigma levels.

The results show that while predicted ground motions are on average similar to those predicted by empirical relations alone, there is significantly more variability due to the combination of source, path and site effects modeled in this study. The largest median peak ground accelerations are nearly 1 g for the Hayward fault scenario and ~1.2 g for the San Andreas, with 1-sigma motions ranging from 50% to 120% higher than median motions. Due to source slip and directivity effects, median motions are lower at the ends of the fault relative to sites located elsewhere at similar fault distances, but show significantly greater variability (higher sigma). While crustal wave propagation is dominated by the general decay of ground motion with distance, post-critical Moho reflections do contribute to slight increases in median and 1-sigma motions at 50-80 km distance. Variations in soil amplification contribute significant variability in ground motions at all periods. Soft soils show strong relative de-amplification at short periods and close distances (Fig. 5.21-5.22). The range of soil depths modeled clearly shows that amplification significantly increases with period as soil depth increases.