COMPARATIVE EARTHQUAKE RISK ASSESSMENT FOR MACKAY, QUEENSLAND

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ABSTRACT:

We have compared the tropical cyclone, flooding and earthquake risks to the Mackay community (population approx. 59,000). Our earthquake risk assessment method investigates a range of building damage scenarios generated by probabilistic spectral ground shaking. The earthquake risk is significant but it ranks below the risk to the community from tropical cyclone and flooding. We estimate that about 750 buildings, or 3.6% of the building stock, would suffer moderate or more severe damage in the average recurrence interval 475-year 'code' earthquake scenario. We estimate a Mean Damage Ratio of 4.4% for timber-framed residences under the ARI = 2,500 year scenario. An overwhelming 96% of critical facilities including electric power substations, bulk food and fuel distributors and transport-related features are located on sediments that potentially will increase damage and reduce the community's capability to respond and recover. Electric power distribution, medical facilities and commercial businesses may be particularly at risk.

1. INTRODUCTION

This paper summarises the earthquake risk assessment component (Jones, in press) of the report 'Community risk in Mackay: a multi-hazard risk assessment' (see the full report for complete results and descriptions of technique). The Mackay report compares the risk to the community from tropical cyclones, floods in the Pioneer River and earthquakes.

2. METHOD

Our general approach to assessing geohazard risk is expressed in the relation:

$$\textbf{Risk}_{\textbf{(Total)}} = \textbf{Hazard x Elements at Risk x Vulnerability} \qquad (1)$$

where

- ***Natural hazard*** *means the probability of occurrence ... of a potentially damaging natural phenomenon;*
- ***Vulnerability*** *means the degree of loss to a given element at risk or set of such elements resulting from the occurrence of a natural phenomenon ...;*
- ***Elements at risk*** *means the population, buildings and civil engineering works, economic activities, public services, utilities and infrastructure, etc., at risk ...;*
- ***Risk*** (i.e. 'total risk') *means the expected number of lives lost, persons injured, damage to property and disruption of economic activity due to a particular natural phenomenon ....*

3. EARTHQUAKE HAZARD SCENARIOS

At least six earthquakes have been felt in Mackay since 1874 but none has caused significant damage. The rock spectrum of Somerville and others (1998) and an appropriate *AS1170.4-1993* acceleration coefficient (Standards Australia, 1993) provided a basis to estimate earthquake hazard in the Mackay region. The earthquake hazard for Mackay is low to moderate by global standards.

An urban earthquake hazard map (or urban earthquake shaking map) for Mackay is shown in Figure 1. This map indicates areas of Mackay where potential earthquake shaking is expected to be relatively weaker or stronger. The site classes are based on those published by the US Building Seismic Safety Council (1997).

Amplification factors describe the relative severity of shaking. The amplification factor for Site Class B is unity and the severity of ground shaking on other site classes is higher in proportion to the amplification factors. The factors are period-dependent and intensity-dependent. As an example, earthquake shaking on the Pioneer River floodplain and on coastal sediments is expected to be 1.6 times as strong as shaking on rock for short period vibration.

Six probabilistic, ground shaking scenarios, with ARIs of 100, 200, 475, 1000, 2000 and 2500 years were generated by developing response spectra for Site Classes B-D.

4. BUILDING DAMAGE MODEL

We used the methods of HAZUS® (FEMA, 1999) to derive building damage scenarios for Mackay. HAZUS® is an earthquake loss assessment software package that can be used to estimate the probabilities of economic and social losses from earthquake.

A comprehensive building database for Mackay (about 20,700 buildings), containing information on building location, age, load bearing type and usage, provided input to the building damage models. The most common construction types of buildings in Mackay are timber frame buildings, domestic and non-domestic (89%) (a positive factor for vulnerability to earthquake); concrete block masonry (5.8%); light steel frame buildings used for small business and factories, etc. (2.5%); and an undifferentiated group of unreinforced masonry buildings and buildings with reinforced concrete frames with unreinforced masonry infills (1.5%).

Building age is also an important determinant of seismic resistance in tropical Queensland. Buildings constructed consequent to the wind loading provisions of the Queensland Building Act (*Post-code* buildings) will be more resistant to earthquake shaking than those built prior to their inclusion in the act (*Pre-code*). Wind loading provisions were introduced in 1975, and the Act included domestic buildings from 1982 (George Walker, verbal communication, 1999).

Figure 1: Earthquake hazard map for Mackay. Rock (Site Class B) outcrops in the northwest of the map. Site Class C is a small area of hard sediments. Large areas of Mackay are underlain by alluvial and marine sediments (Site Class D)

5. BUILDING DAMAGE SCENARIOS

A summary of the outcomes of the building damage scenarios with likelihoods ranging from ARI = 100 years to ARI = 2,500 years is shown in Figure 2. The 'building code' scenario (ARI = 475 years) provides an example. About 3,280 buildings, comprising about 16% of the building stock, are expected to sustain damage, about three quarters of which will be minor. About 750 buildings, comprising about 3.6% of Mackay building stock, will sustain moderate or more severe damage. Most damaged buildings will be residential.

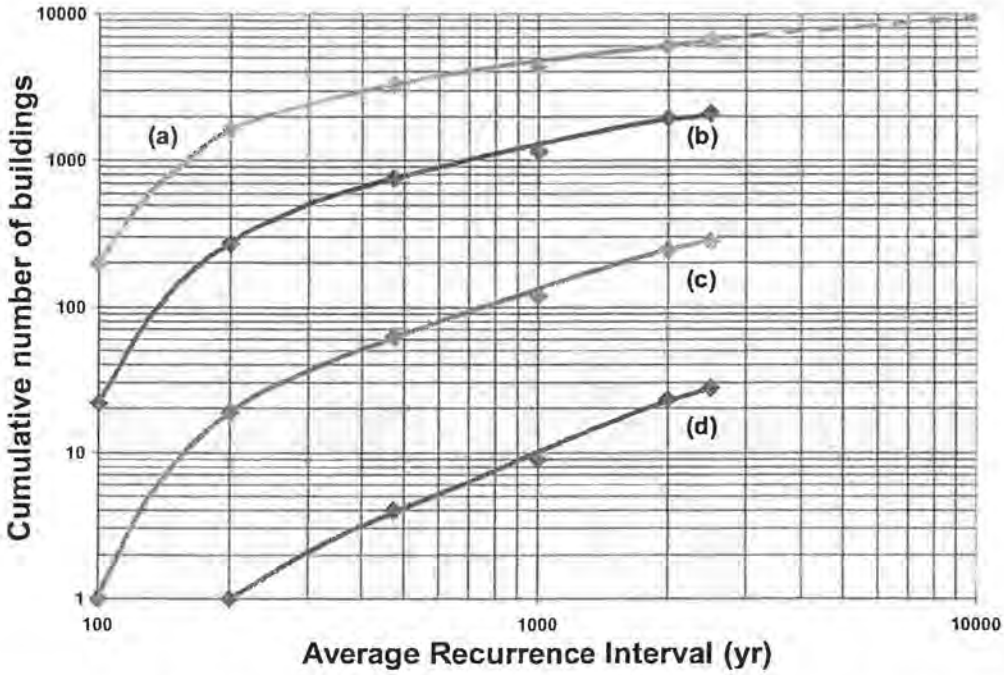

Figure 2: Mackay building damage scenarios with average recurrence intervals in the range 100 years to 2,500 years. Damage states are (a) slight; (b) moderate; (c) extensive; (d) complete. The curves refer to the numbers of buildings in a particular damage state or a more severe state.

6. KEY RESULTS AND IMPLICATIONS FOR MACKAY

6.1 Earthquake risk to Mackay and comments on building vulnerability

Mackay faces a moderate risk from earthquakes. The risk in Mackay is certainly significant and earthquakes should be considered in risk management strategies for the city. Earthquake is the fourth most important geohazard risk to Mackay behind severe wind from tropical cyclone, flooding of the Pioneer River and storm tide.

About two-thirds of the buildings were constructed before the enforcement of wind or earthquake loading standards. We consider *Pre-code* timber framed buildings are at least twice as likely to suffer moderate or more severe damage as their *Post-code* equivalents on the same type of ground. In addition, about 81% of the *Pre-code* buildings are located on the most unfavourable ground conditions in Mackay (Site Class D), which increase the likelihood that buildings will suffer moderate or more severe damage by a factor of about two and one-half compared to rock foundation.

Unreinforced masonry buildings are the most vulnerable buildings to earthquake shaking. We found that they were twice as likely to suffer extensive damage as any other buildings in Mackay. Buildings with a reinforced concrete frame and unreinforced masonry infill panels were more likely to be damaged than any *Post-code* buildings or

Pre-code timber frame buildings in our modelling. Almost all, if not all, of these 300 or so buildings, of both types, are located on the most hazardous ground conditions, Site Class D.

6.2 Scenario Mean Damage Ratio estimate for Mackay timber frame residences
Scenario Mean Damage Ratio (MDR) is defined as:

Scenario MDR = cost of repair or replacement / total value (total replacement cost) (2)

For timber frame residential buildings in Mackay (87% of all buildings), we estimated Scenario MDR of 1.6% for the ARI = 475 year scenario and 4.4% for the ARI = 2500 year scenario. The cost of repair comprises costs to repair both buildings and fittings but not contents. We emphasise the preliminary nature of these results.

6.3 Risk ranking of individual Mackay suburbs to earthquake
There is a very strong contrast in risk between the most at-risk quartile of suburbs (South Mackay, West Mackay, Central Mackay, Andergrove, North Mackay, Slade Point, East Mackay) and the lowest quartile of suburbs. There is also a high association between the suburbs contributing most to overall community vulnerability and those suburbs most at risk from earthquakes. The association can be traced to the history of development of the city. Figure 3 illustrates the ranking.

Figure 3: Mackay earthquake building damage scenarios for ARI = ~ 475 years. Counts indicate the numbers of buildings in a suburb suffering moderate or more severe damage

6.4 Earthquake risk based on building usage
An overwhelming 96% of critical facilities including electric power substations, bulk food and fuel distributors and transport-related features are located on sediments in Mackay, and are particularly at risk from earthquake.

All major electric power distribution facilities are located on sediments. About half of the buildings also may be vulnerable unreinforced masonry and *Pre-code* concrete block. Many other user groups depend on electric power to maintain their own function.

A number of key public safety buildings are constructed of unreinforced masonry and are located on sediments. They include the SES headquarters, the Mackay Police Station, the Mackay Fire Station, and the Mackay Ambulance Station (all in Central Mackay). The integrity of these buildings, and their continued functioning, is critical for emergency response.

Business function in Mackay may be particularly at risk following a strong earthquake. About 97% of businesses are located on sediments. At least 51% of the buildings housing them are *Pre-code*, and about 60% of all unreinforced masonry buildings in Mackay house businesses.

The capability to maintain medical treatment in an emergency may also be threatened. Ninety percent of the hospitals, pharmacies, old people's homes and surgeries are located on sediments. An estimated two-thirds of medical buildings predate wind and earthquake loading provisions.

6.5 Probability of casualties

We cannot exclude the possibility of serious injuries or deaths in Mackay but the probability is low - about 3 in 10,000 in or beside unreinforced masonry or soft storey reinforced concrete buildings when subjected to earthquake shaking approximating the Mackay ARI = 2,500 year scenario (Dowrick, 1998).

7. ACKNOWLEDGEMENTS

Ingo Hartig undertook the GIS analysis and prepared the figures. Miriam Middelmann reviewed the text.

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A FULL RANGE RESPONSE SPECTRUM MODEL FOR ROCK SITES IN THE MELBOURNE METROPOLITAN AREA

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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. He is particularly interested in earthquake hazard analysis, seismic instrumentation and the relationship between earthquakes and dams.

ABSTRACT:

This paper presents a response spectrum model for rock sites in the Melbourne Metropolitan Area (MMA). The peak ground motion recurrence for the MMA was estimated using the recently developed Component Attenuation Model (CAM) in a probabilistic seismic hazard analysis. CAM has been found to be in good agreement with other published attenuation functions developed for similar tectonic environments. The ground motion parameters predicted by CAM have been used to construct the response spectrum in the MMA. The results are compared with the Australian Earthquake Loading Standard which appears to be conservative, particularly for structures with natural periods exceeding 2 seconds.

1. INTRODUCTION

A response spectrum model for rock sites in The Melbourne Metropolitan Area (MMA) is presented in this paper. The model utilizes the recently developed Component Attenuation Model (CAM) in conjunction with local seismological and geological information related to seismic activity levels and ground motion attenuation behaviour in the region surrounding Melbourne.

Such information has been utilized within the framework of CAM to determine the response spectral attenuation functions. An important consideration in the modelling is the distinction between small magnitude earthquakes (nearby short period, high peak ground motion but short duration events) and infrequent large magnitude earthquakes in stable continental regions (with longer periods and longer durations).

Using the Cornell-McGuire approach, the peak ground acceleration, velocity and displacement (using earthquakes above M_W 5.0) have been determined for the MMA using CAM. These peak ground motion parameters have been used to construct response spectra with periods between 0.1 to 5 seconds, thus covering most of the period range of engineering interests. The developed response spectrum model provides input information for soil response analysis as described in the companion paper entitled: "Displacement Assessment in The Melbourne Metropolitan Area Accounting for Soil Resonance".

2. SEISMIC ACTIVITY IN THE MELBOURNE METROPOLITAN AREA

The earthquake hazard in Australia is low to moderate by world standards reflecting the lower rate of seismic activity compared with interplate regions. Regions that have low seismic hazard may still experience large earthquakes, but at very infrequent intervals. The earthquake hazard in the Melbourne Metropolitan area is about average for southeast Australia.

An area enclosed by 143.0°E, 36.5°S to 146.0°E, 38.5°S (Figure 1 shows seismicity in the Melbourne Metropolitan area from January 1900 to December 1999) was used to determine the seismic parameters needed for a probabilistic seismic hazard assessment (activity and *b* value) of the Melbourne area. The A and b value have been determined for a large area to simplify the probabilistic seismic hazard assessment so that the results given using the computer program FRISK could be compared with manual calculations. Future work will include a detailed model to calculate the seismic hazard in the Melbourne Metropolitan area using the CAM.

The seismic activity for the MMA is $a_5 = 0.50$ and a_5 is the logarithm to base 10 of the total number of earthquakes per 100 years (with $Mw \geq 5.0$) over an area of 100,000km^2, and the *b* value is 0.81. The above parameters were evaluated using the Maximum Likelihood Method.

3. DESCRIPTION OF THE ATTENUATION RELATIONSHIP

The Component Attenuation Model (CAM) used in the seismic hazard analysis was developed by the authors (Lam *et al*, 2000a & b) based on stochastic simulations of the seismological model of Atkinson & Boore (1998). The seismological model was originally developed in the United States to define the average frequency contents of earthquake ground motion in both the Generic Hard Rock and the Generic Rock regions of Eastern North America (ENA) and Western North America (WNA) respectively. CAM effectively utilizes the seismological model to construct response spectra for direct engineering applications in different crustal conditions, and has made a useful contribution to seismic hazard studies in various regions outside North America, particularly in areas lacking local indigenous strong motion data. In this study, CAM has been applied to Victoria where the crustal condition is, with respect to attenuation, similar to the generic rock condition of WNA (Wilkie & Gibson, 1995).

CAM provides estimates for the maximum response spectral velocity Sv_{max} (highest point in the average velocity response spectrum) for a given earthquake magnitude (M) and site-source distance (R) in a generic rock crust as shown by Eqn.1.

$$SV_{max}(mm/sec) = 0.78(93.5)(0.35+0.65(M-5)^{1.8})*G(R,D)*(30/R)^{0.005}*(1.3)(1.15) \qquad (1)$$

The first three terms represent the source effects whilst the fourth term, $G(R,D)$, is the geometrical attenuation factor which takes into account the effects of the crustal wave guide (Somerville, 1999; Atkinson & Boore, 1998) where R is the site distance (km) and D is the crustal thickness (km). For this study the wave guide thickness has been assumed to be 30 km, as the maximum depth of Victorian earthquakes is about 20 km, and there is a higher velocity lower crust to a depth of about 35km. $G(R,D)$ is defined by Eqn.2a – 2c.

$$G(R,D)= 30/R \qquad for\ R<45km \qquad (2a)$$
$$G(R,D)= 30/45 \qquad for\ 45km<R<75km \qquad (2b)$$
$$G(R,D)=(30/45)\sqrt{(75/R)} \qquad for\ R>75km \qquad (2c)$$

The fifth term, $(30/R)^{0.005}$, accounts for anelastic whole path attenuation (energy dissipation along the wave travel path). The sixth term, *1.3*, represents mid-crust amplification whilst the last term, *1.15*, accounts for the wave modification effects of the upper crust which was addressed in Atkinson & Silva (1997) and Boore & Joyner (1997). It has been recommended that the peak ground velocity is of the order of half of the predicted Sv_{max}. (Lam *et al* 2000a&b)

Similar expressions for the prediction of the acceleration and displacement parameters have also been developed so that response spectra can be constructed over the entire period range of interest. (Further details can be found in Lam *et al* (2000a&b)) When compared with the other empirical attenuation functions, CAM predicts higher peak ground acceleration (*PGA*) at short distances, possibly due to higher stress drop, and lower than average *PGA* at longer distances (Figure 2). This observation is consistent with the known crustal properties in Victorian where high frequency wave components are subjected to a high rate of attenuation.

4. RESULTS OF CORNELL-McGUIRE INTEGRATION

The attenuation relationship of CAM (described in Section 3) has been input into the probabilistic seismic hazard analysis program (FRISK) to determine the peak ground acceleration, velocity and displacement recurrence information for the MMA for earthquakes above M_W 5.0. Figure 3 compares the PGA recurrence with other published attenuation functions for similar tectonic environments (Sarma & Free, 1995).

An alternative, simplified, approach to determine the probabilistic ground motion parameter is to determine a series of magnitude-distance (M-R) combinations (Eqn.3) based on considering circular source areas within which a uniform spatial distribution of seismic activity is assumed (Jacob, 1997).

$$M = 5 + \{log_{10}(2\pi R^2 T_{RP})\text{-}7+a_5\} / b \qquad (3)$$

where T_{RP} is the return period (years), R is the site distance (km) and the parameters $a_5 = 0.50$ and b=0.81 has been estimated for the MMA.

The M-R combinations obtained by Eqn.3 have been substituted into the CAM attenuation relationships (Eqn.1, for example) to determine the mean probabilistic ground motion parameters which are shown in Table 1 along with mean results obtained directly from FRISK. The Uniform Seismicity Analyses estimates associated with the chosen M-R combinations (figures in **bold**) are in good agreement with the FRISK analyses mean estimates.

Table 1 Probabilistic Ground Motion Parameters Determined for MMA Rock Sites

Return Period (yrs)	Uniform Seismicity Analyses (Eqn.3)					FRISK Analyses Mean Estimates			
	M	R (km)	PGA (g's)	PGD (mm)	PGV (mm/s)	PGA (g's)	PGD[1] (mm)	PGV (mm/s)	MMI[2]
475	5	30	**0.04**	3	19	0.05	10	23	V
	5.5	50	0.03	3	16				
	6	80	0.03	**7**	**22**				
1000	5	20	**0.07**	4	32	0.07	14	40	VI
	5.9	50	0.05	7	27				
	6.4	80	0.05	**13**	**36**				
2500	5.4	20	**0.10**	6	42	0.11	23	63	VI-VII
	6.4	50	0.08	16	48				
	7	80	0.08	**29**	**63**				

Notes :
1. PGD is based on the peak response displacement at a natural period of 5 seconds.
2. The relationship recommended in Newmark & Rosenbleuth (1971) has been used to determine the Modified Mercalli Intensity (MMI) from the predicted PGV.

It is shown in Figures 4 & 5 that the response spectrum provisions by the Australian Earthquake Loading Standard (AS1170.4, 1993) are comparable to the mean estimates of the seismic hazard analyses for a Return Period between 1000 and 2500 years. In Figure 4, apparent conservatism of Australian Standard 1170.4 is noticeable in the long period range. This meant that the design of larger structures with long periods also considered the motion from relatively infrequent larger earthquakes, rather than just the low magnitude 500 year events. The recommendation by Somerville *et al* (1998) (scaled to the respective PGV of 25, 40 and 65mm/sec) is also shown for comparison in Figure 5. The ratio of the peak responses to the peak ground acceleration, velocity and displacement has been assumed to equal 3, 2 and 1 respectively for 5% damping.

5. DE-AGGREGATION

When carrying out a probabilistic earthquake hazard analysis, the resultant ground motion estimates are not from any one particular event, but rather from a combination of small nearby and larger more distant events. It is important to recognize that structures respond very differently to earthquakes of different magnitudes and distances particularly in the inelastic range. An important consideration in the modelling of seismic hazard is the separation of small and large magnitude earthquakes.

The de-aggregation plot given in FRISK (Figure 6) shows the magnitude-distance contributions to the 1000 years ground motion estimates. For an earthquake of magnitude 6 - 6.5, the most likely distance to produce a PGA of 0.07 g is about 50km - 80km, which is consistent with the M-R combinations obtained from Eqn 3.

6. CONCLUSION

1. The earthquake hazard in the Melbourne Metropolitan area is about average for southeast Australia, and is represented by $a_5 = 0.50$ and $b=0.81$.
2. The Component Attenuation Model (CAM) and its application in Victoria has been briefly described, and verified in a comparative analysis which utilizes numerous published attenuation relationships obtained from research in similar tectonic environments.
3. Probabilistic response spectral parameters for return periods ranging between 475 and 2500 years have been estimated using (i) a simple approach based on circular source areas possessing uniform seismicity (Eqn.3) (ii) Cornell-McGuire Integration using computer program FRISK.
4. The M-R combinations estimated by Eqn.3 and the de-aggregation by FRISK are in good agreement.
5. The response spectrum provisions of the current Australian Earthquake Loading Standard AS1170.4-1993 are comparable to the mean estimates of the seismic hazard analyses for a Return Period between 1000 and 2500 years. The conservatism of the Standard is particularly noticeable in the high period range.

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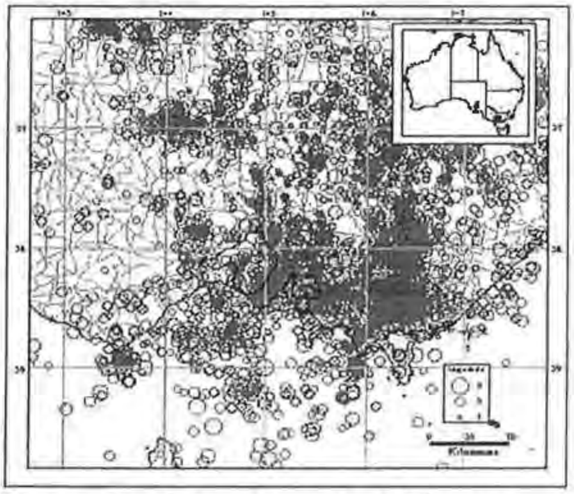

Figure 1. Seismicity of Melbourne Metropolitan Area (Period 1900 to 1999)

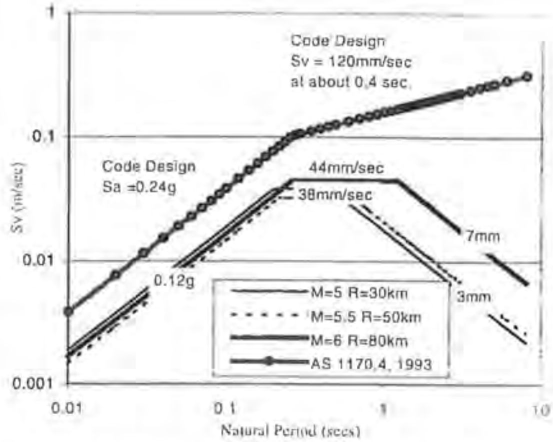

Figure 4. Comparison with AS 1170.4 spectrum for 475 yrs return period

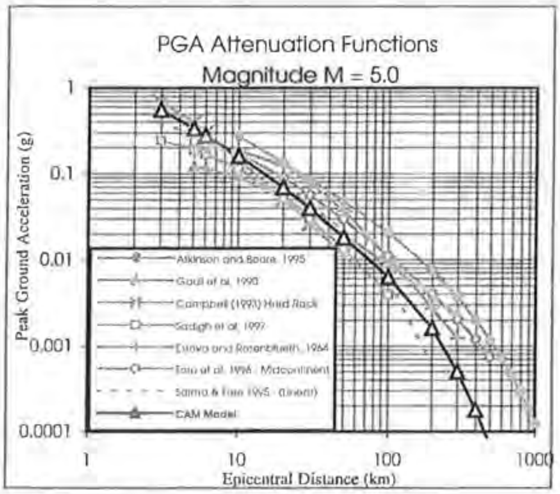

Figure 2. Comparison of different PGA attenuation functions.

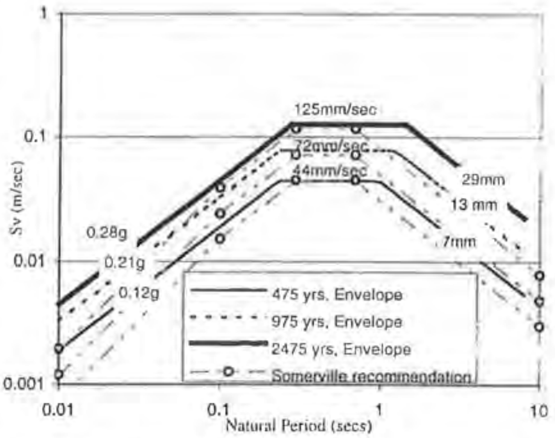

Figure 5. Comparison with Somerville recommendation for different yrs r. p.

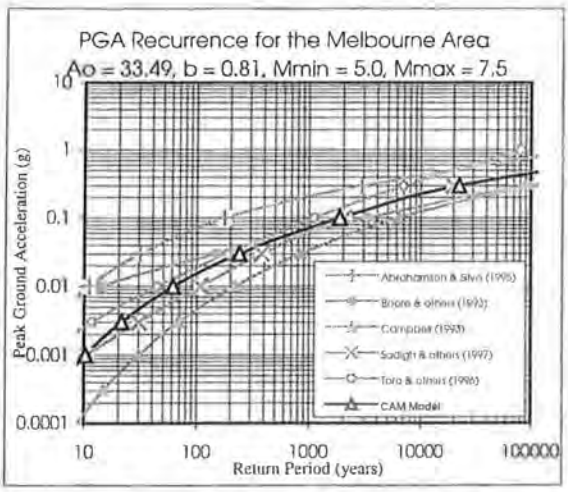

Figure 3. PGA recurrence for Melbourne area

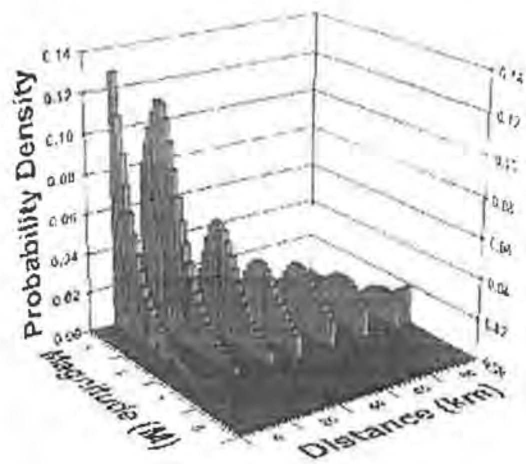

Figure 6. Magnitude-Distance Deaggregation Plot (1000 yrs)

DISPLACEMENT ASSESSMENT IN THE MELBOURNE METROPOLITAN AREA ACCOUNTING FOR SOIL RESONANCE

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Michael Cheng is a Master of Engineering student at the University of Melbourne, where he completed his undergraduate degrees of Civil Engineering and Science. He has been researching on soil dynamic modelling in earthquakes.

Graham Hutchinson is Professor and Head of the Department of Civil and Environmental Engineering at The University of Melbourne. He has written two books and over 100 papers on earthquake engineering and structural dynamics. He is also specialist consultant for earthquake engineering related projects all over the world.

ABSTRACT:

The seismic hazard model for the Melbourne Metropolitan Area (MMA) has been developed in the companion paper. The full range response spectrum obtained from the modelling will provide realistic predictions for the displacement demand of both short and long period systems founded on average rock sites. The objective of this paper is to extend this prediction for structures founded on 1 second period soft soil sites, taking into account the effect of resonance in both the soil and the structure. The prediction utilises a rational manual procedure, known as the Frame Analogy Soil Amplification (FASA) model in which the response spectrum of the broad band bedrock excitation is scaled to obtain the response spectrum for the periodic ground motions transmitted to the soil surface.

1. INTRODUCTION

Shear wave analyses of soft soil columns show narrow band periodic waves arising from resonance of seismic waves trapped within the soil column. The periodic waves result in significant amplification of the soil response spectrum at the site natural period. Such resonance effects are most pronounced when the soil is underlain by a distinct soil-rock interface with a very high impedance contrast. The periodicity of the soil surface motion is directly related to the time taken for the incident, or reflected, wavefront to propagate through the soil medium. Consequently, the site natural period T_g (defined as the period corresponding to the peak of the soil response spectrum) correlates very well with soil depth H. This T_g-H relationship has been demonstrated directly by field recordings of earthquake tremors in both Australia and Singapore [1-2].

Analytical models have been developed to model soil resonance together with a combination of wave modification mechanisms[3]. Alternatively, simple manual methods, such as the code response spectrum model[4] , may be used for general seismic hazard analyses. Such code models have been developed from statistical analyses of large volumes of empirical data[5-6]. Contemporary empirical models typically parameterize soil dynamic properties based on the shear wave velocity averaged over a certain soil depth. Thus, information related to the soil layering and discontinuities, which induce resonance in the soil, are usually not parameterized. In empirical modelling, measurements taken on soil columns of different depths and layering conditions are usually averaged so that dynamic amplification associated with periodicity are shown on a period band rather than focused on distinct site periods.

Thus, some codes specify amplification of the soil response spectrum in the low period range even for soft deep soil columns in which low period (high frequency) waves are expected to be attenuated. Similarly, amplification of the soil response spectrum in the high period range is often specified for shallow soil columns.

A simple, and reliable, scaling procedure which addresses soil resonance is clearly advantageous, particularly for the assessment of displacement demand since the soil surface displacement is highly dependent on the site natural period (T_g). The Frame Analogy Soil Amplification (FASA) model has been developed to address these issues by scaling the response spectrum of the bedrock excitation to obtain the soil response spectrum based on a pre-determined site period. The theoretical basis of the procedure facilitates its application in low and moderate seismicity regions where only limited field data is available for verification and calibration. Early development of FASA was based on modelling accelerations[7]. In contrast, this paper describes a revised, and a more effective, approach in which velocity and displacement (as opposed to acceleration) have been used as the basis for modelling.

FASA is described herein in three parts. In Part One (Section 2), the peak ground velocity (PGV) and peak ground displacement (PGD) at the soil surface is predicted in accordance with the response spectral velocity of the bedrock motion. In Part Two (Section 3), soil damping and the associated damping factors are predicted in accordance with the soil deformation which is in turn related to the PGD. In Part Three (Section 4), the response spectral velocity and displacement for structures founded on soil is determined in accordance with bedrock motions predicted for the Melbourne Metropolitan Area (MMA)[8].

2. BEDROCK MOTION AND SOIL PEAK GROUND VELOCITY

The response spectral velocity of the bedrock motion at the period of the site can be related to the soil PGV by Eqn.1. This is based on the modelling of a soil column as a shear beam or moment resisting frame (MRF) which has further been simplified into a single-degree-of-freedom system (Figure 1),

$$PGV = \sqrt{(1.2\beta Sv(T_g))^2 + PRV^2} \tag{1}$$

where β = damping factor which has been normalised to unity at 5% damping
1.2 = the adopted participation factor (Figure 1),
$Sv(T_g)$ = response spectral velocity of the bedrock motion at T_g for 5% damping,
PRV= peak velocity of the bedrock motion.

The first term in Eqn.1 accounts for the motion associated with the shear vibration of the soil whilst the second term accounts for the direct contribution of the bedrock motion. For T_g which is very close to the highest point on the rock spectrum (bottom diagram in Figure 1), PRV can be approximated as $Sv(T_g)/2$ and hence Eqn.1 can be rewritten as follows :

$$PGV = \sqrt{(1.2\beta Sv(T_g))^2 + \left(\frac{Sv(T_g)}{2}\right)^2} \tag{2}$$

3. SOIL DEFORMATION AND DAMPING

The average shear strain in the soil (γ_{ave}) at maximum displacement can be estimated in accordance with $Sv(T_g)$ and soil depth (H) using Eqn. 3.

$$\gamma_{avg} = \frac{1.2\beta Sv(T_g)\left(T_g/2\pi\right)}{0.6H} \tag{3}$$

T_g can be related to H and V_s (average shear wave velocity of the soil) by Eqn.4.

$$T_g = 4H/V_s \tag{4}$$

Substituting Eqn. 4 into Eqn. 3 followed by some manipulations leads to Eqn. 5 ;

$$\gamma_{ave}(\%) \sim \beta\, Sv(T_g)(\text{mm/sec}) / 8V_s\,(\text{m/sec}) \tag{5}$$

The average shear strain (γ_{ave}) determined by Eqn.5 (assuming β=1 in the 1st iteration) is translated to soil damping (ζ%) based on established shear strain-damping relationships (Figure 2). The damping factor (β) has been modelled by parametric studies and the results show general agreement with the Newmark-Hall model[9] (Figure 3 and Eqn.6).

$$\beta = \sqrt{7/(\zeta+2)} \tag{6}$$

The correlation of $Sv(T_g)$ with γ_{avg}, ζ% and β is shown in Table 1 for cohesionless soil. The values of β obtained in the 1st iteration (Column 4 of Table 1) have been back substituted into Eqn.5 & 6 in the 2nd iteration to obtain improved estimates of the same parameter (Column 5). It is shown that β converges in two iterations.

Table 1 Damping De-amplification Factor for Cohesionless Soil (V_s=200-400m/sec)

$Sv(T_g)$ (mm/sec)	γ_{avg} (%) Eqn.5	ζ (%) Figure 2	β (1st iteration) Eqn.6	β (2nd iteration) Eqn.6
15	0.005-0.010	4 - 6	1.0	1.0
30	0.010-0.020	6 - 8	0.9	0.9
70	0.020-0.050	8 - 12	0.8	0.8
160	0.050-0.100	12 - 15	0.7	0.7
320	0.100-0.200	15 - 20	0.6	0.65

4. SOIL RESPONSE SPECTRAL VELOCITY AND DISPLACEMENT

The maximum response spectral velocity (Sv_{max}) of a single-degree-of-freedom structure founded on soil (in which T_g is close to the highest point on the rock spectrum) can be obtained using Eqn.7 which is supported by Figure 4.

$$Sv_{max} = 5\ PGV \qquad \text{(assuming 5\% damping in the structure)} \tag{7}$$

The predicted Sv_{max} can be used to predict the peak displacement demand (Sd_{max}) of the structure possessing a natural period equal to the site period (i.e. $T = T_g$) using Eqn.8.

$$Sd_{max} = Sv_{max}\,(T_g/2\pi) = 5\ PGV\,(T_g/2\pi) \tag{8}$$

The β factors listed in Table 1 can be combined with Eqn. 2 & 8 to determine both the Sv_{max} and Sd_{max} for any given $Sv(T_g)$ values.

A PRV of 25 and 65mm/sec (i.e. $Sv(T_g)$=50 and 130mm/sec respectively) has been estimated for MMA for Return Periods of 475 and 2500 years respectively[8]. The corresponding values of Sv_{max} and Sd_{max} calculated in accordance with Eqn.7 & 8 are shown in Table 2 and the recommended displacement spectrum in Figure 5. Note, the response spectrum amplification factor (Column 6 of Table 2) is in the order of about 5-6, which is significantly higher than the existing code provisions[4] for similar ground motion intensities.

Table 2 Maximum Response Spectral Velocity and Displacement for Cohesionless Soil

$Sv(T_g)$	β	PGV(soil)	Sv_{max}(soil)	Sd_{max}(soil)	Sv_{max} (Soil)/$Sv_{(Tg)}$ (Rock)
50mm/s	0.85	57mm/s	285mm/s	45mm	5.7
130mm/s	0.75	135mm/s	670mm/s	110mm	5.2

The relationship between Sv_{max} and $Sv(T_g)$ has been obtained for both cohesionless and cohesive soils (with plasticity index of 50%) based on the damping relationships defined in Figure 2 & 3 and the FASA procedure as outlined in this paper. The Sv_{max}-$Sv(T_g)$ relationships calculated by FASA has been verified by comparison with SHAKE analyses in Figure 6.

5. CONCLUSIONS

A rational procedure (known as FASA) to predict soil surface displacement, damping and response spectra for soil sites possessing resonance behaviour is presented. The response spectrum amplification factor (soil/rock) obtained from FASA is significantly higher than existing code provisions for similar ground motion intensities. The FASA model predicts a moderate response spectral displacement of about 45mm and 110mm for a 1 second period soil site based on the seismic hazard information of the Melbourne Metropolitan Area for Return Periods of 475 and 2500 years respectively.

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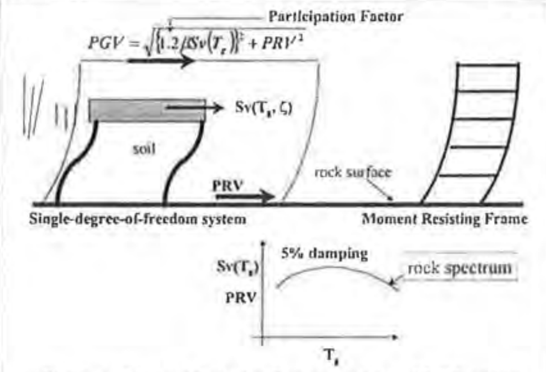

Figure 1 Moment Resisting Frame Analogy of Soil Response

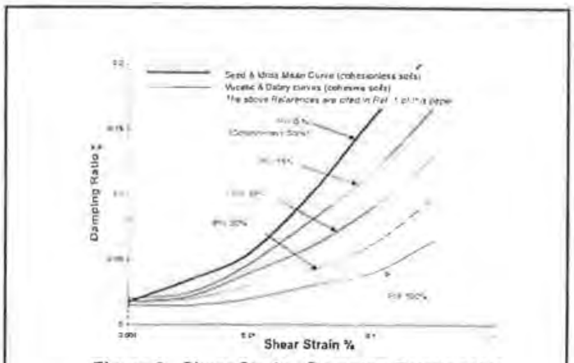

Figure 2 Shear Strain - Damping Relationship

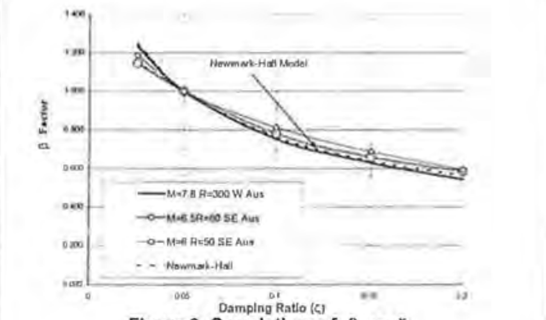

Figure 3 Correlation of β vs ζ

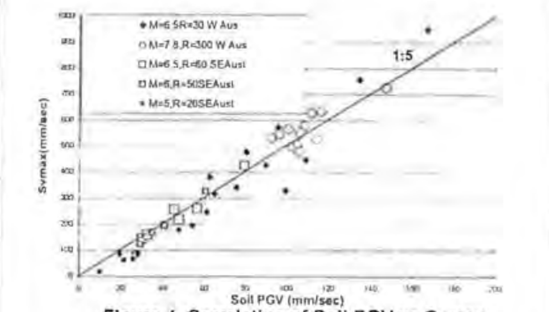

Figure 4 Correlation of Soil PGV vs Svmax

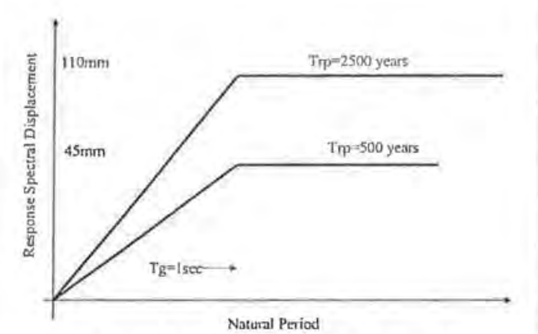

Figure 5 Displacement Response Spectra for 1 sec Period Cohesionless Soil Site

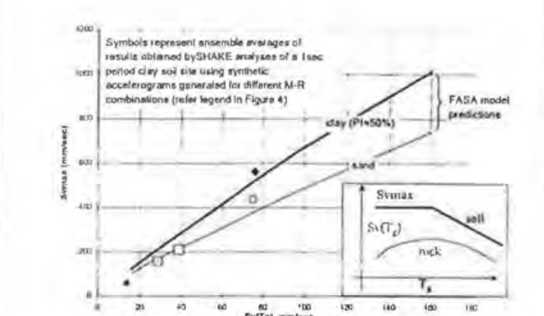

Figure 6 Sv(Tg) - Svmax Relationships

EARTHQUAKE RESISTANCE OF BUILDINGS WITH HIGH-STRENGTH REINFORCED CONCRETE COLUMNS

JL VAN DER MOLEN CPENG

AUTHOR:

Mr. van der Molen has been in engineering practice for the last 55 years. He spent roughly 20 years in the supervision of civil engineering construction, and another 20-odd in design, mostly leading project teams for industrial and public infrastructure projects. The last 12 years he was first lecturer, then senior lecturer in the Department of Civil and Environmental Engineering at the University of Melbourne. Presently he is engaged in consulting activities and based in Hobart.

ABSTRACT:

An experimental programme has been carried out at the University of Melbourne, to investigate the behaviour of high-strength reinforced concrete (HSRC) columns in multi-storey buildings, where these columns are interspersed with normal-strength (NS) concrete slabs. The failure mode of this construction is somewhat unexpected and particularly brittle. The reasons for this and the consequences with regard to earthquake-resistant construction are discussed.

Earthquake Resistance of Buildings with High-Strength Reinforced Concrete Columns

JL van der Molen CPEng

Synopsis: An experimental programme has been carried out at the University of Melbourne, to investigate the behaviour of high-strength reinforced concrete (HSRC) columns in multi-story buildings, where these columns are interspersed with normal-strength concrete (NSC) slabs. The failure mode of this construction is somewhat unexpected and particularly brittle. The reasons for this and the consequences with regard to earthquake-resistant construction are discussed.

1. Introduction

HSRC Columns in multi-story buildings have gained acceptance on the ground that column cross-sections may be reduced, saving floor space and so improving their economic performance. In the normal course of events, the construction techniques are not modified: columns are cast up to the soffit level of each floor, the floor is constructed and the next lift of columns is cast on top of the new floor.

This causes the concrete subject to column actions to consist of column sections of HSRC, interspersed with sections of NSC slab, nominally the thickness of the floor in height and the cross-section of the column in plan. Refer Figure 1.

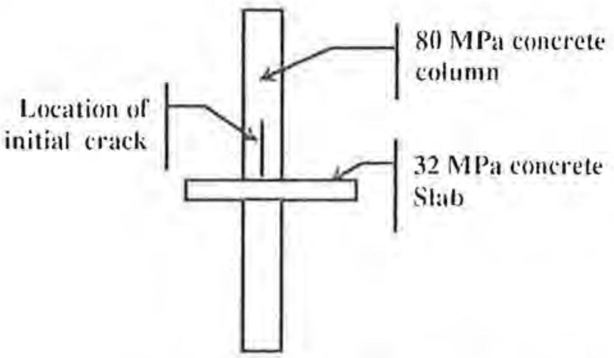

Figure 1 - View of Column-Slab Combination

Intuitively, the strength of this composite column will lie somewhere between that of the HSRC of the column and the compressive strength of the slab concrete. The argument for this is that the slab concrete is confined between the upper face of the lower column and the lower face of the upper column. The two faces will provide a lateral restraint of the slab concrete, causing this concrete to be subjected to a tri-axial compressive stress condition. This will lift the compressive ultimate stress of this concrete to anything up to 2f'c, this figure being arrived at from experience in the design of Freyssinet hinges. The research has been aimed at obtaining accurate experimental data of the strength of such a composite column.

2. Observed Failure Mode

The failure mode postulated in the above is that of crushing of the restrained slab concrete at a load corresponding to its compressive strength f'c, multiplied by a shape factor, γ_s, where probably, $1.0 < \gamma_s \leq 2.0$. γ_s will be a function of such factors as the efficiency of the lateral restraint, the size of the column, the slab thickness and the location of the column in relation to the edge(s) of the slab.

However, the observed failure mode is entirely different: what occurs is that the column stub, either immediately above or below the slab concrete, suddenly develops a longitudinal crack, as indicated in Figure 1. This crack is tapered: it is wide at the column-slab interface and reduces in width as it extends up (or down) the column. The crack causes spalling of the concrete cover, generally at one of the corners or one of the sides of the column. This spall is followed immediately by crushing of the slab concrete, which is now no longer restrained to the same extent, i.e. the shape factor γ_s is being suddenly reduced due to the crack and the spalling occurring in the column concrete. In turn, this reduces the load capacity of slab portion of the column, which is now exceeded by the actual load applied. Failure follows immediately.

This failure mode is brittle and catastrophic.

3. MaterialsTechnology

It is well to compare the two materials involved in this investigation. They are normal-strength concrete (f'c $\cong$ 32 MPa), which contains the customary materials and has the expected elastic properties, significantly a Poisson's ratio $\mu \cong 0.2$ at working stress. This ratio has the property that it increases at increased stress levels, reaching close to 0.5 at the time of failure, at some 27 MPa.

The HSRC has a strength f'c = 80 MPa and contains significant amounts of condensed silica fume (CSF). This would significantly increase the shear modulus G, and decrease Poisson's ratio to about 0.15 at its working stress. This working stress would be similar to the ultimate strength of the slab concrete.

The result of this situation is, that at the slab-column interface there are two concretes which at the time of failure have considerably different Poisson's ratios, ~0.45 for the slab concrete against ~0.15 for the column concrete. This means that the lateral strain of the slab concrete is approximately three times that of the column concrete. Across this interface, therefore, the two materials are incompatible, precipitating the failure mechanism observed.

4. Consequences

Obviously, by application of adequate safety factors the combination of HSRC columns and NSC slabs in multi-story buildings may be made to comply with relevant Code provisions. However, two aspects of this construction require close investigation:

- The failure mode of this detail is by its nature brittle. This property is contrary to the basic tenet of present-day design philosophy, which assumes ductile failure mechanisms.

- As a consequence, the use of this construction detail without modification aimed at making its failure mode more ductile would seem contra-indicated in areas of seismic activity.

Designers would have to draw their own conclusions as to the desirability of this detail in any given circumstances. My personal preference would be not to use it at all, unless modification to induce ductility has been proved feasible.

5. Remedial Measures

Given the unsatisfactory situation noted above, the question presenting itself is whether it would be possible to modify the brittle behaviour of this joint by suitable addition of reinforcing steel. Obviously, this reinforcement should be aimed at preventing the column from splitting, i.e. absorbing the radial tensile stresses developing in the column. The situation is unusual because the stress is induced at the column-slab interface by a transverse tensile strain. This strain causes a local fracture, which propagates upwards (or downwards). The task is, therefore, to provide reinforcement across the crack's path which will arrest its propagation.

In the following simplistic model it is assumed that the reinforcement so provided should be able to absorb the tensile force existing in the concrete just prior to rupture. The tensile strain in the concrete is a maximum at the column face. It is therefore assumed that any column ligature near the column-slab interface should be able to resist a tensile force equal to that existing in a concrete area of a width equal to twice the concrete cover plus the diameter of the ligature, and a height equal to the ligature spacing. This leads to the following assumptions:

- the ultimate tensile stress, f_{ct}, of the column concrete is of the order of 8 MPa,
- Cover to ligatures is 30 mm, spacing is 40 mm, material 450W wire, 10 mm dia.
- The stressed area per ligature has a width of 2x30+10 = 70 mm and a height of 40 mm, refer Fig 2,
- A safety factor of 1.5 is required.

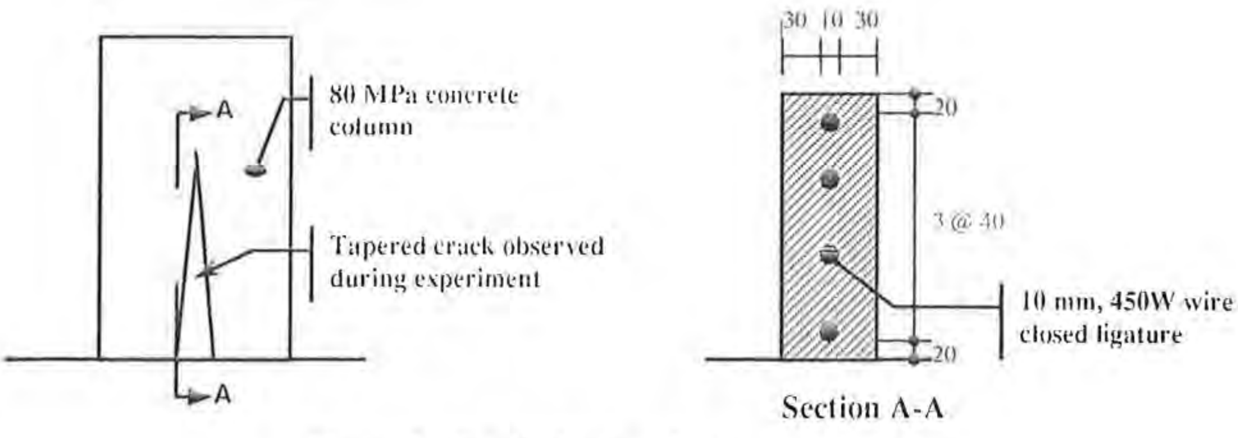

Figure 2 - Detail of stressed area

The force per ligature is then 70x40x8 = 22.4 kN, and the required steel area:

$$A_{sf} = \frac{22400 \times 1.5}{450} = 74.7\,mm^2$$

Available $A_{sf} = 78.5\ mm^2$

The ligatures are not stressed until the longitudinal crack appears. The function of the ligatures is to limit the crack width sufficiently to induce a measure of ductility into the failure mechanism. As the crack originates at the interface of the column and the slab, only a few stirrups of the type and spacing shown are required, typically not more than, say, four. They have to be spaced as closely as possible, with the lower (or upper) ligature positioned as closely as possible to the slab.

The above suggestions still need to be confirmed by test. It has been postulated that the initiation of the failure mechanism is by the formation of a wide longitudinal crack. The design of the ligature system is aimed at limiting the crack width. Therefore, the major question to be settled by test is that limiting this crack width will indeed slow down the collapse of the slab concrete.

6. Acknowledgments

For the development of the above I am indebted to Joanna Portella and Ris Lee, both post-graduate students in the Department of Civil and Environmental Engineering of the University of Melbourne who invited me to watch their experimental programme and who discussed results with me, and to Mr. Robert F Viles, Technology Manager - Cement and Corrosion, Fosroc Construction Development Group, for our discussion of compatibility issues with different concretes.

UPGRADING THE PHILIPPINES NATIONAL SEISMIC NETWORK

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ABSTRACT:

During January to March this year (2000) the Philippine national seismic network was significantly upgraded by the installation of digital seismic recording systems, telephone communications and a near real-time automated earthquake location and alarm generation system.

The software at each field station performs automatic P arrival picking and sends this information via FTP to the head office. There, all arrivals are automatically placed in a database and event association performed. An initial location and magnitude are computed automatically. Any location may be improved at a later time with additional data received via radio or other means. Location information may be transmitted to interested parties via a worldwide web page, email or via pager.

The automated system should provide assistance in responding to the significant hazard caused by earthquakes, volcanoes and tsunami in the Philippines.

Background

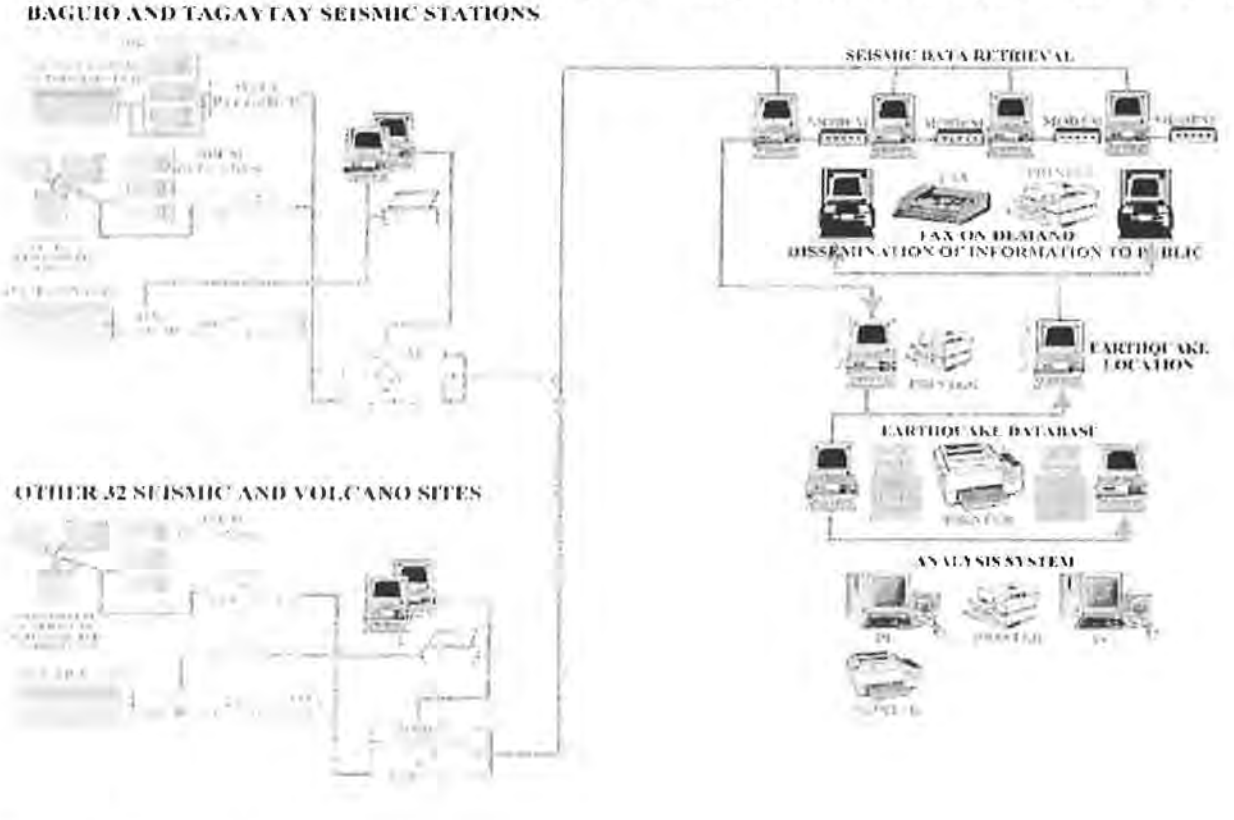
Following the catastrophic eruption of Mt Pinatubo in 1991 the Philippine Institute of Volcanology and Seismology (PHIVOLCS) applied to the Japanese International Cooperation Agency (JICA) for funding to upgrade the volcano and seismic monitoring capability of the Philippines. This application was ultimately successful.

PHIVOLCS had been running the national seismic and volcano monitoring networks in the Philippines under various names since 1952. This network consisted of approximately 35 seismic monitoring stations spread across the 1500km length of the country. Most stations used single component short period seismometers recording on helical drum recorders. A few stations had digital recorders provided for special projects. Communication with the head office in Manila was by a short wave radio network.

The perceived need was for a more modern digital seismic network with multi-channel recording and an automated system running in Manila for automatic determination of earthquake epicentres and magnitudes. The telecommunications infrastructure of the country and budget of the relevant department precluded continuous telemetry from each station. Therefore, an event based system using dial-up communications was proposed by the Japan Weather Association (JWA), the technical consultants for the project.

The project was put out to tender in early 1999. Mitsubishi Corporation was the successful bidder. They used a largely Australian consortium including the Australian Bureau of Meteorology (BoM), the Australian Geological Survey Organisation (AGSO), SRC/Mindata Australia and Delairco. The tender was let in August 1999 with all equipment to be supplied and installation complete by March 2000.

SCHEMATIC OF UPGRADED PHIVOLCS SEISMIC NETWORK



System Overview

The system specified in the tender was for the upgrading of 33 regional stations, the station in Quezon City, Manila and the installation of an automated analysis system in Manila.

The works required at each station included:

- Supply of new mains electricity cable from outside power pole, power distribution board with lightning protection and so on. Provision of new power points for all equipment.
- A power rack with battery chargers and sealed lead acid batteries.
- A seismic rack with two digital recorders, three drum recorders and an Intensity display.
- A set of tri-axial short period seismometers and a tri-axial accelerometer
- Two desks and chairs
- Two PC's for data analysis and storage
- Two modems and a facsimile for communication with head office
- Two printers for printing out seismic waveforms and general use
- A UPS for each station
- All associated cabling
- Consumables for two years

For the head office in Manila, a complete computer based analysis and archiving system was required. This included:

- Approximately twelve computers and associated network cabling and switches. A UPS for each computer
- A number of printers and facsimile machines and four modems for dial-in data reception
- Software for automatic reception of data, automatic location of events and storage in a database
- Interactive programs for displaying seismic waveforms, event location and database browsing
- A GIS system for displaying maps of earthquakes and other information

Installation

The seismic equipment was completed at the beginning of January 2000 and shipped to the Philippines. Installation commenced in the middle of January. The first "demonstration" installation was performed at Tagaytay, about an hour or two south of Manila. This initial installation took three days with various members of SRC/Mindata performing the installation. It was watched by personnel from PHIVOLCS, JWA, Mitsubishi, AGSO and BoM. This was a stressful time for all concerned. It was stressful for PHIVOLCS staff because up until this time, they did not really know what sort of a system they were going to get. It was stressful for the Australian consortium because due to time constraints this was the first time they had been able to put all of the pieces of the system together, and it was being performed in a confined area overseen by up to thirty observers.

Once this installation was complete, SRC/Mindata used five teams to perform the installation at the remaining twenty eight sites. Five sites in Mindanao were considered by the Japanese consulate to be unsafe for foreigners. Each of the five teams consisted of one SRC/Mindata engineer and one or more PHIVOLCS staff member. The PHIVOLCS person was given on-site training so that they would be able to install the remaining five sites.

An AGSO staff member visited each site after installation was complete to check that all systems were installed and operating satisfactorily. They were also accompanied by a PHIVOLCS employee.

Overall, twenty nine stations were installed by the five teams in a seven week period. Including travel, this amounts to about five days per station. It was quite a difficult logistical exercise to support this installation work. We had between two and four people in Manila supporting the installation work.

Before the installation started in the field, the computer systems and their network had been installed in head office. This involved organising the desks, physically unpacking all the computers, printers, UPS's etc., running hundreds of metres of networking cable around the room, configuration of the computers, switches and so on. This took a couple of person weeks in total.

The seismic analysis system then had to be installed and set up on the various computers. This was about another person's week of effort.

Overall, the partners in the project are very pleased to have been able to install the system within the very tight time frame imposed by the Japanese financial system and end of Japanese financial year.

Operation

The system has been in operation to a lesser or greater extent since mid March 2000. Over that time hundreds of earthquakes have occurred within the Philippines and been detected by the system, including a magnitude 6.2 event in July this year just off northern Luzon.

It is fair to say that a number of problems have been encountered, as you would expect of a project such as this. Of the 33 remote stations 22 have standard land line telephones installed. Of these, two have lines of such poor quality that they cannot be used for modem based communications. This leaves only 20 of the sites to automatically dial in information following an earthquake. Of these a number are poorly sited being close to main roads or a similar source of significant cultural noise. This makes the system less reliable than it would otherwise be in automatically detecting or locating earthquakes.

There have been some equipment problems, particularly with the seismic sensors, but also with the seismic recording equipment and computers. It seems that the computers (particularly their hard discs) do not like operating in the hot humid conditions present in the Philippines. One or two sites have suffered lightning damage to the seismic equipment in the first six months of operation.

The seismic analysis software has undergone some fine tuning to adapt it to the conditions in the Philippines and the way in which PHIVOLCS wish to operate it. This has involved many discussions between PHIVOLCS staff and the various members of the Australian consortium.

PHIVOLCS now has a modern digital seismic network installed which is recording high quality multi-component continuous digital data which can be used for a range of seismic and related studies within the Philippines and surrounding areas. This will also facilitate communication of data between the Philippines and neighbouring seismic observatories.

THE TASMANIA SEISMIC NET

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ABSTRACT:

The Tasmania Seismic Net is the oldest telemetered seismic network in the world. It currently operates a combination of 3 analogue and 5 digital telemetered stations, together with 4 Kelunji Classic three or six component dial-up seismographs plus an international IRIS/IDA station (Fig. 1). To improve the quality of earthquake (EQ) data and the reliability of the network, the remaining analogue stations are in the process of being replaced by digital ones. The EQ database accumulated over the last 43 years is used for risk management analysis by HYDRO Tasmania (Hydro). It is also included in the Australian Geological Survey Organisation's national EQ database from which hazard assessments are made.

Until recently all data from the telemetered network were recorded on a large ten-pen analogue drum. This has now been changed to a three-channel recorder. All digital data are recorded on a dedicated MAC computer using the SEISMONITOR software developed by the Seismology Research Centre (SRC), Melbourne. Compact Discs are used for permanent storage of digital data. Over the last three years communications channels have been completely upgraded, with new digital microwave equipment being installed. Accurate timing is achieved using GPS receivers. As from July 1, 2000, data interpretation is contracted out to SRC.

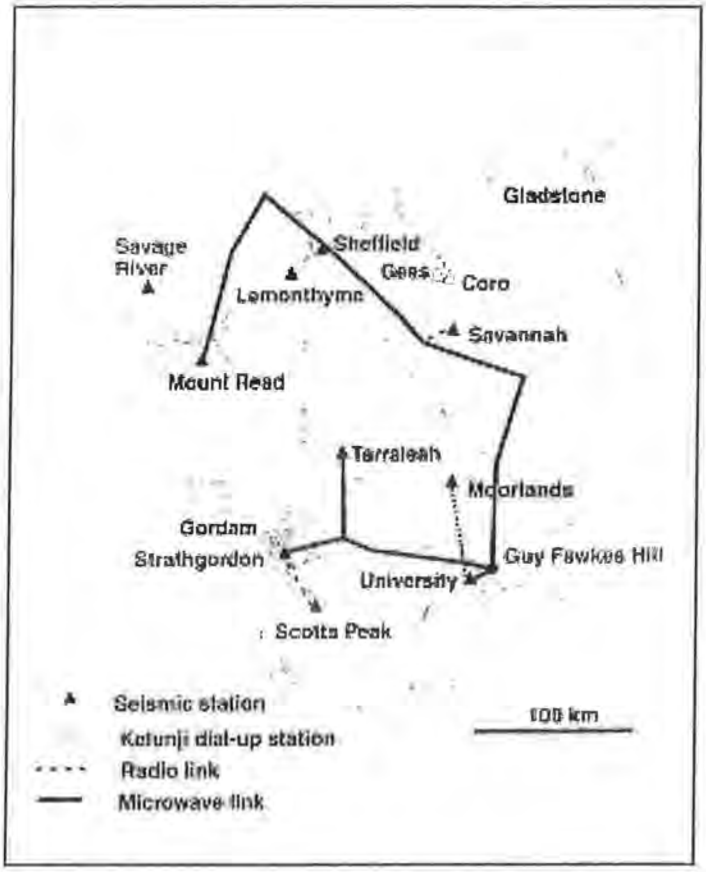

Fig. 1. The Tasmania Seismic Net (Sept. 2000)

2. TELEMETERED NETWORK

2.1 Stations

The University of Tasmania established its first seismic station (FNT) November 1957 at Fort Nelson, originally part of the defences of the Port of Hobart, in the suburb of Sandy Bay. FNT was closed down in 1962. The equipment was reinstalled in a newly constructed vault (TAU, fig. 1) at Mt. Nelson, above the University, together with a World Wide Standard Seismograph Network Station (WWSSN).

Because new geological evidence had been discovered (Lake Edgar Fault Scarp) suggesting an appreciably higher risk to existing and proposed hydro-electric installations than had hitherto been assumed, it was considered prudent to establish a larger telemetered network of stations (Carey, 1960). A search of local newspapers had also revealed a sequence of over 2500 tremors during 1883-86 with epicentres in the East Tasman Sea, west of Flinders Island. The tremors were felt in northeastern Tasmania, especially in Launceston, where some buildings were damaged.

In September 1960 an array of three telemetered stations (TRR, SAV and MOO) were installed jointly by the Geology and Engineering Departments of the University of

Tasmania. Further stations were later added: SFF and LMT 1969, SPK, STG in the southwest 1972 and SVR 1979. SVR was shut down 13 March 1991 and the equipment moved to Mount Read (MTRD), western Tasmania, where it was reinstalled 25 March 1991. STG was closed down in 1997 due to problems with providing a communications channel from Strathgordon to Twelvetrees Range. A DD1 Digitiser with a S13 seismometer was installed at TAU on 27th November the same year.

2.2 Ten pen drum

A 10 pen recording drum, designed and constructed by the Hydro, has been recording the 8 analogue seismic signals since 1960. The 3.6m long paper record needed to be changed every day. This drum has been modified (Jan. 2000) to record three analogue channels (SAV, MOO and TAU). The paper is now replaced every 3-4 days, thus eliminating the previous need for changing paper on weekends. This drum is the largest seismic drum recorder in the world. A few years ago the Australian National University, Canberra, made a copy of it for their own use but reversed the direction of rotation.

2.3 JUMP stations

Following the 1989 Newcastle earthquake, the Commonwealth Government made funding available for the Joint Urban Monitoring Program (JUMP) to upgrade EQ monitoring facilities in 16 Australian cities. Through this initiative Tasmania was allocated three Kelunji Classic seismographs. Two were installed in Launceston. One, GEES (installed 6th March, 1995), is situated on a solid dolerite outcrop above the CBD and the other, CORO, in Coronation Park (5 Jan., 1995), on deep sediments, near the centre of the city. The third seismograph (GLAD) was installed just south of Gladstone, northeastern Tasmania, (April 1997), to monitor the West Tasman Sea earthquake zone.

A fourth Kelunji Classic seismograph with a three component accelerometer and a Willmore MK2 single component vertical seismometer was purchased by HYDRO Tasmania and installed in the footings of the 140m high Gordon dam in southwest Tasmania. It is also maintained by the Tasmania Seismic Net.

2.4 International seismographs

April/May 1962 the United States Coast and Geodetic Survey established a station of the World Wide Network of Standardised Seismographs (WWSSN) at the TAU vault, Mt. Nelson. On January 14, 1994 this was replaced with the next generation of seismographs in the newly established digital IRIS/IDA network, using Streckeisen three component broadband seismometers. The data from this seismograph are accumulated on a SUN SPARC computer and downloaded via Internet to the IDA data centre in San Diego, California, approximately every 5 minutes.

3. COMMUNICATIONS

3.1 History

From 1960 to 1996 seismic data was transmitted to the Geology Department at the University of Tasmania using a combination of Hydro Power Line Carrier Systems, VHF and UHF radio links and hired TELSTRA telephone lines. A multipair cable transmitted the WWSSN, TAU and MOO signals from Mt. Nelson to the Seismology Laboratory. This cable was destroyed for the second time by a bush fire in February 1996 (the first time during 1967 bush fires). Replacing the damaged cable would have cost over $40,000. The insurance money from the latest fire was used to purchase and install a digital microwave link.

3.2 Recent Upgrading

A MITEC 2048 kb/s PCM ultra low power 10.5 GHz U Link was purchased and installed and has functioned very satisfactorily since 1996. A second MITEC microwave link was installed March 1998 between the Hydro communications centre at Guy Fawkes Hill (on the Eastern Shore of the river Derwent) and TAU, a distance of about 10 km, eliminating the need for hiring expensive TELSTRA lines. Both links use AWA-PLESSEY multiplexers and provide 30 analogue or digital circuits. This upgrade has resulted in substantial savings in yearly running costs together with greatly improved communications lines.

4. DIGITISATION OF DATA AND REDUNDANCY

A fourth generation upgrade of all electronics at the field stations is now being implemented. Three analogue stations have been replaced with SRC Kelunji D series seismographs and new three component seismometers (TRR 1999, SPK and SFF 2000) and a further two with DD1 Remote Digitisers (TAU and MTRD 1999). These upgrades have been funded by a special grant from HYDRO Tasmania.

Digitally recorded events are stored on a dedicated Power Macintosh 7220/200 computer at the Seismology Laboratory and continuous data recorded on a hardisk and then transferred to a CD for permanent storage. Another computer at the Guy Fawkes Hill Hydro communication's centre will record events in parallel with the above mentioned one. Event data are also stored in the memory of D series Kelunji field seismographs, thus providing three fold redundancies in case of failures of any part of the seismic network.

5. PAGING

September 2000 an automatic paging system was installed on the computer as part of a new SEISMONITOR software upgrade. This pages personnel in the event of a local earthquake occurring. The data are downloaded to a mobile telephone and enable a quick preliminary determination of epicentre and amplitude.

6. EPARS

Digital earthquake data are also used in Hydro's 'Earthquake Preparation, Alarm and Response System' (EPARS), developed by SRC. This uses automated event location and magnitude determination together with a database of all HYDRO Tasmania assets and their vulnerability to provide an alarm that alerts Hydro personnel of the earthquake and assigns inspection priorities.

7. INTERPRETATION

About 2200 local and teleseismic earthquakes together with numerous quarry blasts have been recorded instrumentally during the last 43 years. From 1957-2000 interpretation of seismic data has been carried out by Lesley Hodgson initially and then June Pongratz. Under a recent contractual agreement (1st July 2000), SRC will do all interpretation and alarm functions from their new premises at the La Trobe University Enterprise Centre, Melbourne. This change has become necessary due to a lack of staff and funding at the School of Earth Sciences.

8. RESERVOIR INDUCED EQ's

After the commencement (February 1972) of the filling of Lake Pedder and Lake Gordon in southwest Tasmania, many reservoir induced earthquakes (Shirley, 1980) were recorded with quite a close correlation between the level and volume of water in storage and the cumulative number of events. However over the years the number of earthquakes in the area has reverted to the background number evident before the filling of the lakes.

9. FUNDING

Funding of the Tasmania Seismic Net is currently being provided jointly by the Australian Geological Survey Organisation (AGSO), HYDRO Tasmania and the University of Tasmania. In the past the Tasmanian Government supplied funds through Mineral Resources Tasmania. It withdrew its support in 1991.

10. CONCLUSION

The Tasmania Seismic Net has undergone many changes since its inception in 1957. It has proved a robust useful tool to assess the seismicity of Tasmania and has also contributed much to the national database. Little is known of the focal mechanism of Tasmanian earthquakes. The replacement of single component analogue stations with three component digital ones should provide data capable of determining focal mechanisms. The data accumulated over the last 43 years have been used in many theses and papers and are listed at the end of this paper.

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THE VICTORIAN WATER INDUSTRY SEISMIC NETWORK

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ABSTRACT.

The Victorian Water Industry Seismic Network was substantially upgraded in 1999. This paper will look at the design and outcomes of the seismic network from a risk management and emergency management perspective. Funding issues for a diversified network providing benefits to a range of clients within the one industry group will also be discussed.
Prior to 1999 the Victorian seismic network had been developed on an ad hoc basis resulting in an incomplete level of seismic coverage throughout the state. The upgraded network now provides sufficient coverage to provide an intensity based alarm service for all contributing Victorian Water Authorities.

KEYWORDS.

Risk management, earthquake alarm, earthquake response, emergency simulation.

Seismic Networks

The major outcomes from seismograph networks may be categorised as:

- Long term estimation of seismic activity parameters *ie* rate of activity, the relative number of small to large earthquakes (b value), identification of active seismogenic structures.
- Determination of attenuation of earthquake motion with distance and measurement of structural response.
- Earthquake alarm or warning functions.

The mix of outcomes required from a particular seismograph network will determine the characteristics of the network.

The first two categories of outcomes require long term monitoring but do not require real-time telemetry of data.

The third category requires real-time telemetry of seismic data, 24 hours per day response and rapid response. The requirement for rapid response often implies a need for some degree of automation. This paper will primarily concentrate on the third category of outcomes.

A seismograph uses precise timing to determine the distance to an earthquake. A network of at least three seismographs is normally required to precisely locate an earthquake epicentre and focal depth for events within the network. The more recordings used, the more precise the location.

Earthquake depths can only be precisely determined if the distance to the *nearest* seismograph is not greater than about twice the earthquake depth or about 10 kilometres, whichever is the larger. East Australian earthquakes usually occur at depths from just beneath the surface to 20 kilometres.

The scale of operation for a seismograph network can vary from tens or hundreds of metres for mining related seismology to thousands of kilometres for global seismological studies. Smaller scale seismograph networks usually record higher frequency motion with higher timing precision and accuracy that results in higher precision for earthquake locations.

The scale of network required for a real time earthquake information system therefore depends primarily on the earthquake location precision required and the minimum magnitude earthquake about which you are likely to require information.

For example to determine the epicentre of an earthquake to a location precision of plus or minus a few kilometres, the earthquake must be inside a network of three or more seismographs all within 100 kilometres of the epicentre. To determine the depth of the event requires that one seismograph is near the epicentre.

Victorian Water Industry Seismic Network Pre 1999

Since 1976 the water industry has been the primary sponsor of earthquake monitoring in Victoria.

For many years the SRC operated instruments for the Rural Water Corporation at the Grampians, Cairn Curran reservoir. and Dartmouth reservoir (later for Wimmera-Mallee and Goulburn-Murray Water).

Networks of instruments were also operated for the MMBW (later Melbourne Water) around Sugarloaf, Thomson and Upper Yarra reservoirs. In 1996 the Melbourne Water seismic network was re-organised to provide an earthquake alarm service for a broader range of Melbourne Water assets.

Aside from the water industry a network of three recorders is operated in the Latrobe Valley for the electricity generation industry. Following the Newcastle earthquake, the state government funded the operation of urban monitoring instruments in Melbourne and Geelong.

The Australian Geological Survey Organisation also has seismographs at Toolangi and Bellfield.

Figure 1 is a map showing the distribution of Victorian water industry funded seismographs in 1998. Continuously telemetered sites are indicated by a black diamond, non-telemetered sites by a grey diamond.

It should be noted that there are other non-water industry funded instruments located in the Latrobe Valley, Melbourne and Geelong as well as a number of unfunded recorders operated by the SRC, however the ongoing operation of the unfunded recorders can not be guaranteed.

The map shows that continuously telemetered data, which is necessary for the provision of an earthquake alarm, was only available from central Victoria, east of Melbourne, and Gippsland.

The map shows large gaps in the seismograph coverage in both northern and southern west Victoria, far east Gippsland, south Gippsland, central Victoria and northern Victoria.

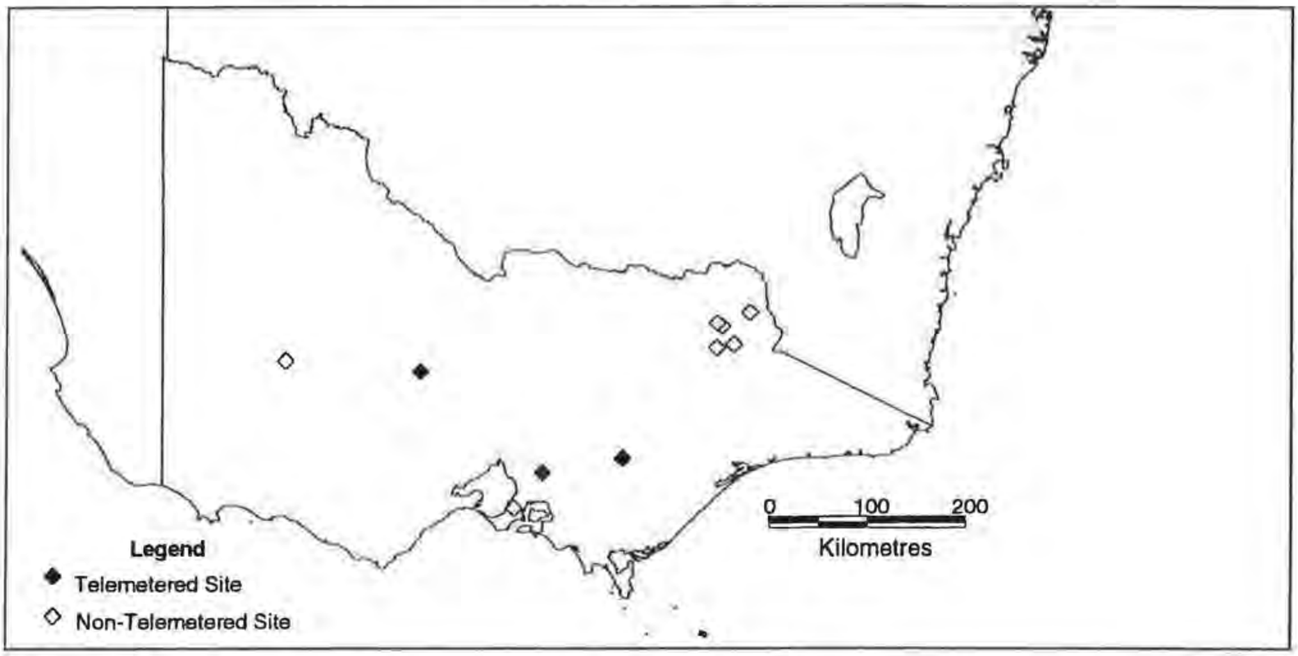

Figure 1: Victorian water industry funded seismographs, 1998

VICTORIAN WATER INDUSTRY SEISMIC NETWORK POST 1999

Following extensive discussions between the Water Bureau at the Victorian Department of Natural Resources and Environment, the Victorian water industry dams working group and the SRC, the Victorian seismic network was extensively upgraded in late 1998 and early 1999.

Figure 2 shows the location of the recorders that make up the upgraded seismic network.

The Victorian Water Industry seismic monitoring network operated by the Seismology Research Centre now comprises five pre-existing seismographs and seven new or upgraded seismographs. One channel of seismic data is continuously telemetered to the SRC laboratory from the recorders at Cardinia, Cairn Curran, Mt Macedon, Molesworth, Rowsley and Thomson. The remaining recorders all have dial up telephone connections to provide additional data following a significant earthquake.

The tighter instrument spacing of the upgraded network allows for higher accuracy locations to be computed, particularly in western and central Victoria.

The additional telemetered sites mean that more accurate rapid locations can be computed to raise alarms. Furthermore two of the new telemetered sites use spread spectrum radios to send data to the SRC so some data will still be received at the laboratory even if the telephone system fails during a significant earthquake.

There are still significant gaps in the seismic network, particularly in south Gippsland, the north-west, far east and north of the state.

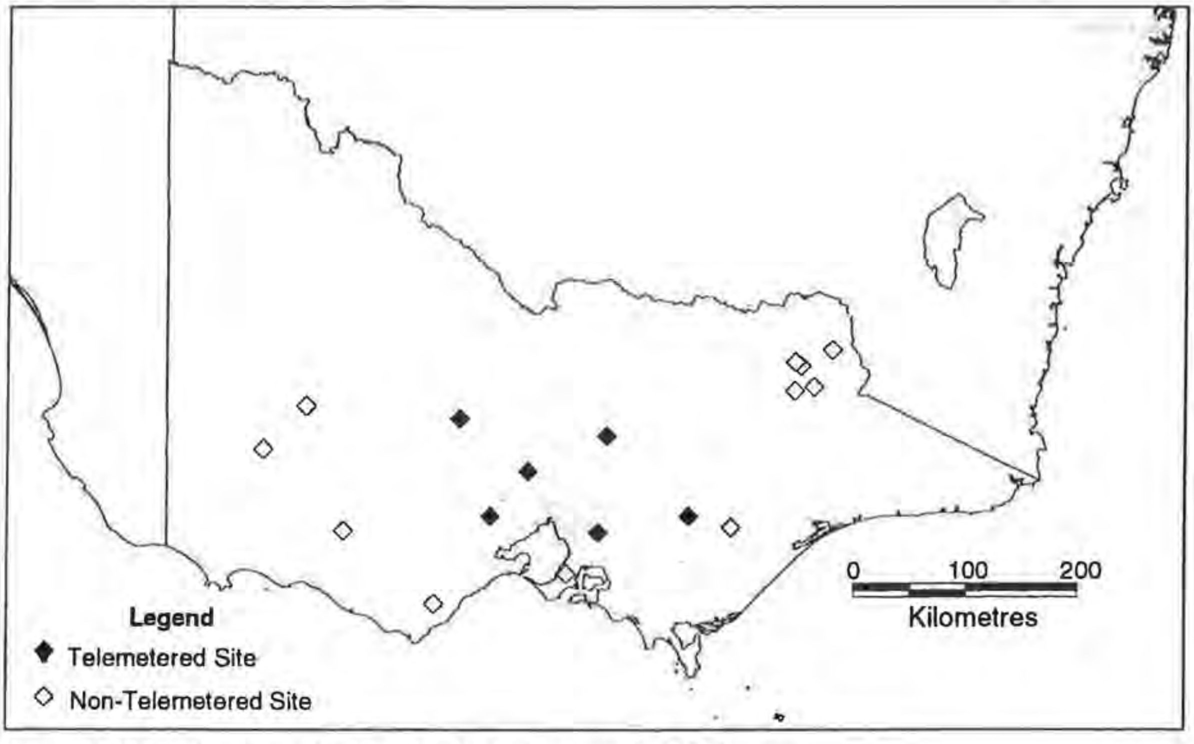

Figure 2: Victorian water industry funded seismographs, 2000

Figure 3 is a map showing the distribution of high hazard classification dams overlaying the seismic recorder network and shows that there is good coverage of the high hazard dams within Victoria.

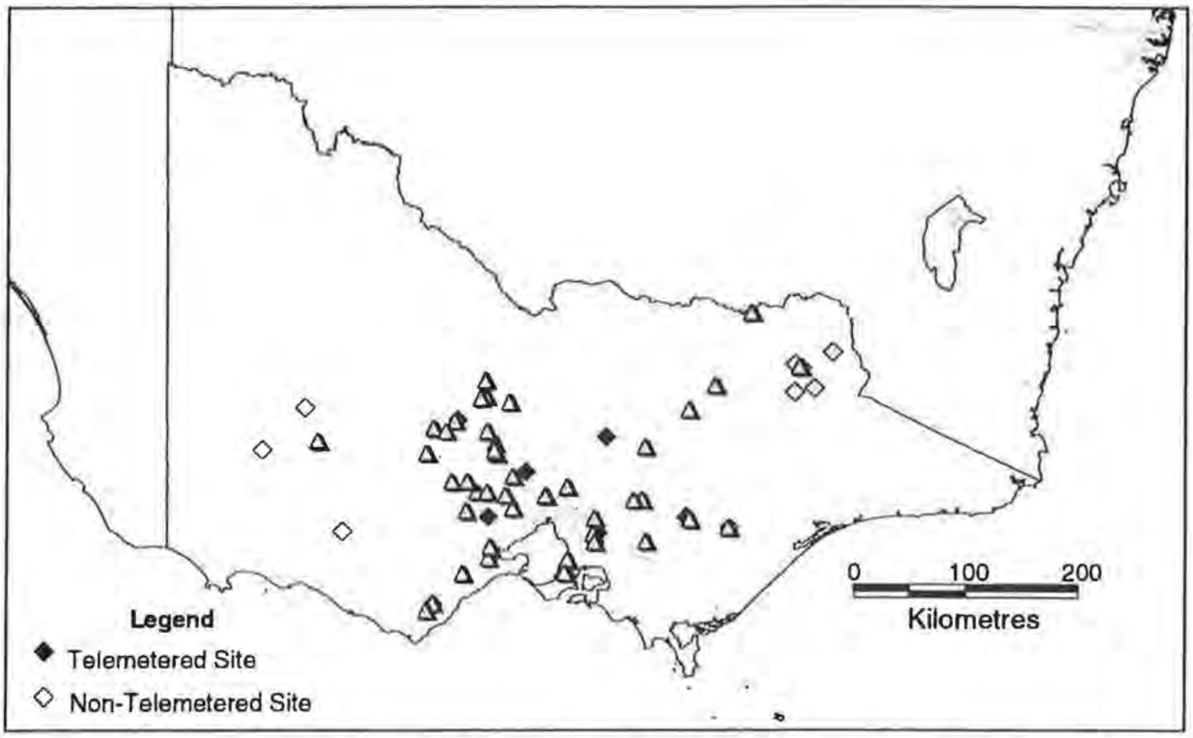

Figure 3: Victorian water industry funded seismographs and High hazard classification dams, 2000

RESULTS FROM THE UPGRADED NETWORK

There are two levels of alarm service provided by the SRC Earthquake, Preparation. Alarm and Response (EPAR) system to water industry stakeholders. The premium level of service provides information about intensities at the authority's assets within 30 minutes of an earthquake occurring during normal working hours, or 60 minutes outside of normal working hours. The standard level of service provides earthquake location and magnitude information as soon as possible, and certainly within 2 hours of the event occurring.

Since January 1999 there have been seven earthquakes of significant magnitude to alert water industry personnel. Even though most of the seven events occurred at night, in every case all relevant water industry personnel were informed of the event within one hour of the earthquake origin time, exceeding the contractual requirements of the network.

The two largest earthquakes that have occurred within Victoria since January 1999 are described below.

2000 March 12, Gaffneys Creek, ML 3.7.

A magnitude ML 3.7 event occurred at a depth of eight kilometres near Gaffneys Creek, about 100 kilometres east of Melbourne on 12 March, 2000 at 12:26 am AEDT. Reports of the earthquake being felt were received from Yarra Junction, Lilydale, Belgrave and as far west as Preston and Greensborough in the northern suburbs of Melbourne. The maximum reported intensity in the epicentral area was MMI 4. A magnitude 3.7 earthquake would normally be felt over a radius of about 70 to 80 kilometres.

No reports of any structural damage were received, and none would be expected from an earthquake of this magnitude and depth. Even minor damage would not be expected unless the earthquake depth was very shallow, within a couple of kilometres of the surface.

There were two moderate aftershocks in the 24 hours following the main shock, one of magnitude 2.3 at 12:39 am (thirteen minutes after the main shock), and another of magnitude 1.9 at 7:26 pm.

The preliminary location for this earthquake was computed by the SRC duty seismologist within 15 minutes of the origin time and personnel from five different water authorities were contacted and informed of the event. The first notification was at 12:51 am AEDT and the last at 1:08 am AEDT

2000 August 29, Boolarra, ML 4.8.

On 29 August, 2000 at 11:05 pm AEDT an earthquake of magnitude ML 4.8 occurred just west of Boolarra in Gippsland, about 22 kilometres south-west of Morwell and 130 kilometres south-east of Melbourne.

This earthquake was the largest to occur in Victoria since the ML 5.0 Mt Baw Baw earthquake of September 1996.

The Boolarra earthquake was strongly felt in Gippsland and was felt throughout the suburbs of Melbourne particularly in the east, but as far west as Sunshine.

Two foreshocks occurred in the hours before the mainshock, the larger being of magnitude ML 2.6 and occurring at 9:20 pm AEDT.

Because the SRC duty seismologist had completed a location of the foreshock, water industry personnel who felt the event and contacted the SRC were able to be given a preliminary estimate of the location and magnitude of the earthquake within minutes of the origin time.

Formal notification of water industry personnel began at 11:34 pm and personnel from six different water authorities were informed of the event details by 11:50 pm.

FUNDING ISSUES

Figure 4 is a map showing the rural, metropolitan and non-metropolitan urban water authority stakeholders contributing towards the Victorian water industry seismic network. Not shown on this map is the contribution of the Victorian Department of Natural Resources and Environment (DNRE).

The DNRE contributed substantially to the capital cost of the upgraded network. The remaining capital costs were apportioned based upon a formula related to the number of high or significant hazard structures owned by the various water authorities.

Ongoing operating costs of the seismic network and the earthquake alarm service are likewise apportioned using the same formula based on the number of high or significant hazard structures.

This funding structure allows water authorities that would otherwise be unable to fund a network on their own to contribute towards an extensive seismic network, the benefits of which are available to all stakeholders.

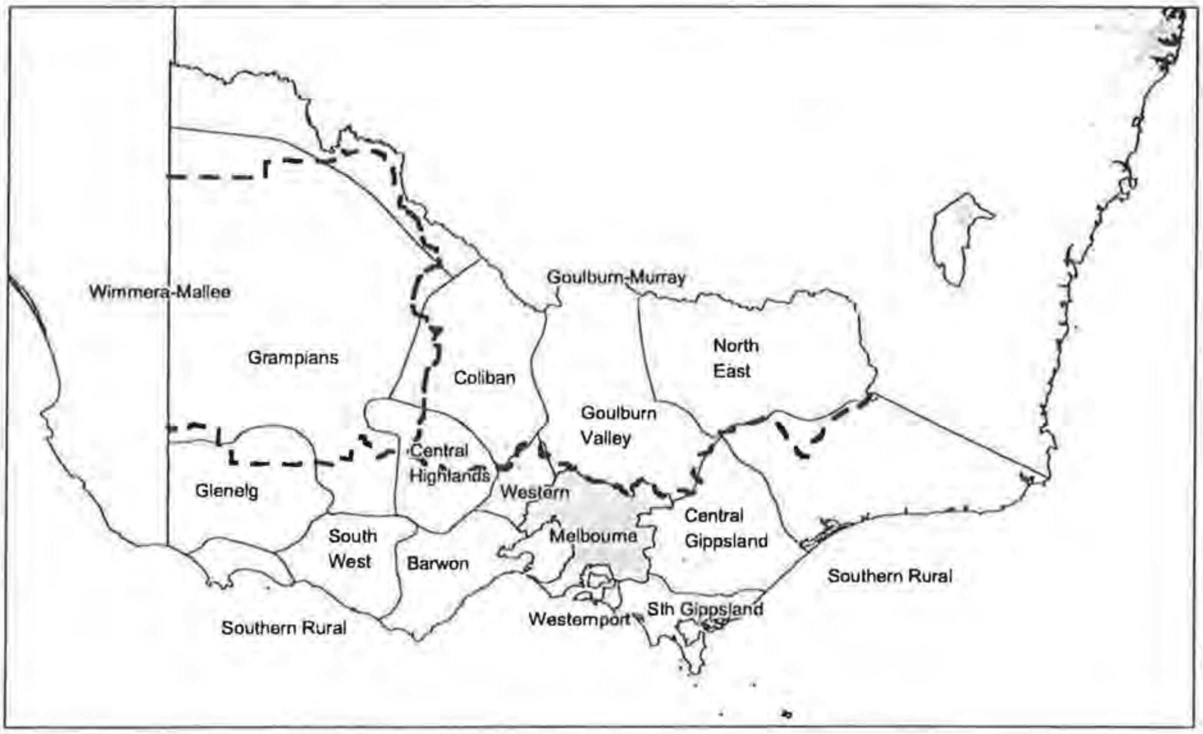

Figure 4: Victorian water industry seismic network stakeholders

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