

# AUSTRALIAN DAM SEISMIC PRACTICES - AN UPDATE

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## INVITED SPEAKER

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### ABSTRACT:

Dam safety management has traditionally been based on a deterministic conservative approach with requirements, where life was at risk from dambreak, requiring design for such "no risk" extreme events as the Probable Maximum Flood and the Maximum Credible Earthquake. However, the estimations of these extreme events have changed over the past twenty years, with increasing knowledge and research, and it is apparent that all these extreme event estimates still retain some low but credible risk. In addition, there has been questioning on whether safety standards where thousands of lives are at risk should be higher than those where only a few lives are at risk.

In line with other hazardous industries (ie petro-chemical and nuclear) the dams fraternity over the past ten years have been progressively implementing risk assessment into their decision making practices. This has involved the introduction of an expanded scale of dam failure consequence categories against which to base design and dam safety management decisions, and the expanding use of risk assessment to assist in prioritising decision making. Similarly in seismic design there has been a phased introduction of risk assessment techniques with complementary deterministic guidelines. The definition of the seismic loading event is now probabilistically based with deterministic methods used to analyse dam responses.

The paper outlines these evolving changes in regard to dam safety management with particular reference to seismic practices.

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by

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### 1.0 BACKGROUND

Dam safety management has traditionally been based on deterministic standards-based conservative approaches. They have developed from recognised good practice, which, with experienced judgment, have served the objectives of dam safety well. Where life was at risk from dambreak, dam management systems usually required such matters as:

- Design for such “no risk” extreme events as the Probable Maximum Flood and the Maximum Credible Earthquake and safety factors for other design areas (eg slope stability) based on conservative experience; and
- Daily individual inspection and monitoring of key dam components.

However, with increasing research and updates in technology, the design estimations of these extreme events have changed over time, particularly in the past twenty years. It is apparent that all these extreme event estimates still retain some low but credible risk and there has been questioning on whether safety standards, where thousands of lives are at risk, should be higher than those where only a few lives are at risk. Introduction of automated and telemetered monitoring, and camera surveillance, have also led to questioning of the role of the individual dam inspector/operator.

To take account of these considerations, and in line with other hazardous industries (ie petro-chemical and nuclear), the dams fraternity over the past ten years have been progressively implementing risk assessment practices to assist in dam safety decision making.

### 2.0 CHANGE FROM HAZARD RATING TO CONSEQUENCE CATEGORIES

Traditionally Australian dam safety requirements (eg design, monitoring, reporting) were related to the hazard ratings assigned to a dam. A coarse three level system was usually employed as follows:

- High Hazard-** Loss of life, major property damage expected if the dam failed;
- Significant Hazard-** Loss of life possible, but not expected, with significant property / environmental damage if the dam failed; and
- Low Hazard-** Only minor damage expected if the dam failed.

Difficulties were found in determining the relevant hazard rating and in publicly justifying dam safety requirements across the broad and dissimilar range of dams within each hazard rating.

To address these concerns, and assist in the phasing in of risk assessment practices, the Australian National Committee on Large Dams (ANCOLD), representing the Australian dam safety management community, introduced in 2000 its “Guidelines on Assessment of the Consequences of Dam Failure”. This outlines a broader range of seven dam failure Consequence Categories to replace the previous three levels of dam hazard ratings, as outlined in Table 1.

**Table 1 –Dam Failure Consequence Categories (after ANCOLD 2000)**

<b>Population at Risk</b>	<b>Severity of Damage and Loss (community, property, environmental)</b>			
	<b>Negligible</b>	<b>Minor</b>	<b>Medium</b>	<b>Major</b>
<b>0</b>	<b>Very Low</b>	<b>Very Low</b>	<b>Low</b>	<b>Significant</b>
<b>1 to 10</b>	<b>Low</b>	<b>Low</b>	<b>Significant</b>	<b>High C</b>
<b>11 to 100</b>		<b>Significant</b>	<b>High C</b>	<b>High B</b>
<b>101 to 1000</b>			<b>High A</b>	<b>High A</b>
<b>&gt; 1000</b>				<b>Extreme</b>

The changes have allowed a more reasoned integration of community dam failure concerns into the categorisation process. They have also introduced a broader range of failure categories to facilitate phased levels of dam safety management requirements and integrate better with the continuum approach of risk assessment practices.

### **3.0 INTRODUCTION OF RISK ASSESSMENT PRACTICES**

Traditional standards-based practices, whilst readily applicable to such aspects as flood capacity, earthquake safety and dam stability, do not provide an assessment of the order of safety for economic considerations. They are also less well suited to other dam safety needs such as estimating the reliability of operating systems, and human factors which often play a critical role in failures.

Risk analysis provides a basis to better estimate those risks. Risk assessment techniques have received increasing attention as a means of aiding decision making on dams and several techniques have been developed and are continuing to evolve.

Risk assessment provides a structured, systematic examination of the likelihood of harmful events and their associated consequences, and is the main anticipatory element underpinning any safety management process. The result of a risk analysis on a dam is a transparent construction of the uncertainty in the future performance of the dam (usually in the form of probabilities). In risk assessment, the results of the risk analysis and risk evaluation processes are integrated and recommendations are made concerning the need to reduce risk (see Figure 1).

Figure 1-Representation of the Risk Assessment Process (after ICOLD 2003)



Fundamentally, the purpose of risk assessment is to support risk management, including risk control initiatives. The form and nature of the risk assessment procedure is determined by its end-use, with different forms of risk assessment being used at different stages in the decision-making process.

In the light of these methodologies, the steady improvement in the required data quality, and the demand to demonstrate the cost effectiveness of dam modification works, ANCOLD produced its “Guidelines on Risk Assessment” in 1994. These Guidelines set out the conceptual foundations of risk assessment at that time and are to be superseded in late 2003 by new Guidelines directed to the practical application of risk assessment, as an aid to better dam safety management.

ANCOLD, and the Australian dam safety management community, are now promoting risk assessment practice as a useful tool to extend and enhance the understandings obtained from traditional engineering methods of dam safety evaluation. It is to be applied in conjunction with those traditional approaches, and not as an alternative or sole basis for decision making given that there are still many challenges to be overcome in its full implementation (eg uncertainty in natural event recurrence and failure probability estimates, acceptability to society of tolerable risk levels).

Initially, risk assessment methodologies have been introduced in the ANCOLD Guidelines on “Design of Dams for Earthquakes” (1998), “Selection of Acceptable Flood Capacity for Dams” (2000), “Assessment of the Consequences of Dam Failure” (2000) and “Dam Safety Management Guidelines” (2003 revision).

## **4.0 SEISMIC DESIGN**

The past performance of dams world-wide under earthquake loading has been very good with few dams suffering major damage. Where this has occurred, it has usually been due to liquefaction in the dam or foundation, or failure of a defect in the foundation. Very few of these dams have breached and released a flood wave.

In addition, in Australia, the potential for earthquakes was considered to be less than in more “earthquake-prone” countries situated along tectonic plate boundaries. However, the 1968 Meckering event (M6.9), the 1988 Tennant Creek events (M6.3-6.7) and the 1989 Newcastle event (M5.6) focussed the attentions of dams’ engineers in Australia.

Much work has been done by dams’ engineers and seismologists over the last fifteen years to update dam seismic design methodology from its nominal deterministic based pseudo-static approach. Relative to the situation in overseas countries, seismic design methodology for dams is now reasonably well developed in Australia but there are few highly experienced practitioners. In this regard, recent assessments of earthquake ground motions for some large Australian dams have been based on the seismological estimation that the maximum credible earthquake throughout Australia could be as large as M7.5.

This change in approach has been facilitated by a working group, convened by ANCOLD in 1993 to prepare its “Guidelines for Design of Dams for Earthquakes”, which were released in August 1998. They are not a design code, but give a background to earthquake characteristics and hazard in Australia and provide guidance for the selection of the design earthquake, and the analysis and design of dams and appurtenant structures under seismic loading. The Guidelines recommend the introduction of risk assessment methods for design but reference deterministic alternatives. For detailed considerations, reference to these Guidelines is recommended, however, pertinent aspects of these Guidelines are summarised in the following sub-sections.

### **4.1 Design Loadings**

The approach taken in the Guidelines is for dam designers to consult seismologists for estimates of ground motion and spectra relevant to the specified seismic design frequencies of recurrence. This typically results in 1 in 1000 Annual Exceedance Probability (AEP) peak ground accelerations of 0.15g ranging to 0.35g for 1 in 10,000 and 0.5g for 1 in 100,000.

### **4.2 Design Analysis for Dams**

The Guidelines recommend that the seismic adequacy of significant and higher Consequence Category dams be assessed by risk assessment methods with deterministic methods outlined for lower Consequence Category dams. The proposed risk assessment involves calculating the probability of dam breaching and the expected consequences, and summing these over the range of possible earthquake events. The acceptability of a dam is then tested by comparison of the calculated values, for the full range of hazards (eg earthquakes, floods etc), with appropriate risk criteria.

Various Australian States have also introduced alternative, or complementary, seismic requirements to the recommended risk assessment procedures. These are usually adaptations of the ANCOLD references, as indicated for NSW in Table 2.

**Table 2-NSW Dams Safety Committee Design Earthquake Requirements  
(as alternative to risk assessment)**

<b>Consequence Category</b>	<b>Max. Design Earthquake AEP</b>
Extreme	<1 in 10,000
High A	1 in 10,000
High B	1 in 5,000
High C	1 in 1,000
Significant	1 in 500

Dynamic analysis methods are recommended in the Guidelines along with a staged approach for the assessment of embankment dams. Empirical and pseudo static approaches are also outlined for relevant dam types (eg decked rockfill dams). A range of defensive design measures is also outlined for embankment dams, concrete dams and foundation treatments to assist designers.

#### **4.3 Design Approaches to Liquefaction**

The mechanics of liquefaction of foundations and dam embankments are outlined along with discussion of soil types susceptible to liquefaction. Semi-empirical methods are given for assessing liquefaction potential (eg related to soil properties such as SPT values, shear wave tests or residual undrained strength) along with measures to improve liquefaction resistance (eg densify foundations, grouting etc).

#### **4.4 Design of Appurtenant Structures**

A number of appurtenant structures associated with dams (eg intake tower, outlet, spillway structures, access roads and bridges) are essential for a dam's operation. The Guidelines recommend that these structures be designed according to their importance (including business risk), and provide design performance requirements, such that:

- They maintain their normal operating condition after an operating basis earthquake loading (typically in the range AEP 1 in 200 to 1 in 1,000) relevant to their failure consequences; and
- They are not damaged to an extent where they could allow sudden or uncontrolled loss of storage for earthquakes up to the relevant Maximum Design Earthquake

#### **4.5 Post Earthquake Procedures**

Although a dam may survive an initial earthquake event, there may still be as considerable risk to dam safety for some time after the event. This risk may arise by the weakened dam or its foundations being subject to aftershocks, a following flood event, post-liquefaction deformation, build up of uplift pressure in concrete dams, piping initiation, slope stability initiation or the inability to operate the dam to required standards.

Dam owners are advised to analyse these scenarios, particularly the ability to drawdown the storage to maximise its safety, and ensure that their Dam Safety Emergency Plans effectively cover these contingencies.

## **5.0 CONCLUSION**

In line with other hazardous industries (ie petro-chemical and nuclear) the dams fraternity over the past ten years have been progressively implementing risk assessment into their decision making practices. This has involved the introduction of an expanded scale of dam failure consequence categories against which to base design and dam safety management decisions, and the expanding use of risk assessment to assist in prioritising decision making. Similarly in seismic design there has been a phased introduction of risk assessment techniques to complement deterministic guidelines.

## **ACKNOWLEDGEMENT**

The outstanding efforts of the various seismic, risk assessment and dam safety management working party members of ANCOLD are acknowledged, with the relevant ANCOLD guidelines produced used as a background source material for this paper.

## **REFERENCES**

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# EARTHQUAKE HAZARD IN AUSTRALIA USING A MULTI-TIERED SOURCE MODEL

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## ABSTRACT:

Earthquakes result from tectonic processes, and their distribution is strongly influenced by large-scale geology and the current tectonic stress field. However, earthquake hazard estimates, particularly ground motion recurrence, have traditionally been computed using models based primarily on instrumental and historical seismicity. In areas of low to moderate seismicity such as Australia, large earthquakes commonly occur in areas which have experienced little or no recent activity, making it difficult to develop reliable source models based solely on seismicity.

The seismotectonic model developed for Australia (AUS5) is based on geology, geophysics, tectonics and seismology. The model was developed using a number of tiers of information, so that additional information can easily be incorporated. The information used includes but is not limited to, tectonic provinces, basins and ranges, gravity, magnetics, topography and seismicity, all on a regional scale. On a smaller scale, for a site specific earthquake hazard study active faulting can help define source zone boundaries.

Earthquake hazard estimates for a region in southeastern Australia, and capital cities in Australia have been calculated using the AUS5 model. The results of this earthquake hazard analysis are compared to previous studies.



## **1. INTRODUCTION**

Even in areas with low to moderate seismicity, such as Australia, there is a risk that an earthquake could cause significant damage to a major city. The likelihood of this event occurring is much lower than in more active parts of the world, but if we do not plan for these events, the consequences can be just as devastating. The production of earthquake hazard maps can help identify the character and level of ground motion with a given return period. These earthquake hazard maps can be used by engineers, builders and government to mitigate the risk an earthquake poses to urban areas.

In areas like Australia where earthquakes tend to cluster in time and space, it is difficult to determine the earthquake hazard with any certainty, especially when the models used to calculate the earthquake hazard are based only on the distribution of very recent past seismicity. A new model for use in earthquake hazard analysis in Australia based on geology and geophysics, AUS5, is presented. The peak ground acceleration with a 10% chance of exceedence in 50 years was determined and compared to that used in the current Australian earthquake loading code.

## **2. SEISMICITY IN AUSTRALIA**

Seismicity in Australia is intra-plate, the earthquakes occur entirely within the plate and result from a build up of stress. The earthquakes differ in several characteristics with those at the plate boundaries that surround Australia. It may be because Australia is located so close to one of the most active plate margins in the world (Indonesia through to the Kermadec Islands and New Zealand), that Australia is one of the most active intra-plate regions of the world.

Figure 1 shows all earthquakes above magnitude 6 that have occurred in Australia between 1872 and 2002, and also the large earthquakes that have occurred in Indonesia during that time. It is obvious from figure 1 that Australia has a much lower rate of seismicity than Indonesia. The level of seismicity in western Australia appears to be higher than that in eastern Australia.

The catalogue contains only 22 earthquakes above magnitude 6, most of which occurred before modern seismographs were installed in Australia in about 1960. Most of these large earthquakes have occurred in remote parts of Australia and have caused little damage. The two most damaging earthquakes in Australia, the 1954 Adelaide ML 5.6 earthquake and the 1989 Newcastle ML 5.6 earthquake, were the most destructive because they occurred so close to large urban centres, rather than for their size.

Figure 2 shows all earthquakes above magnitude 4 that have been recorded between 1872 and 2002. The earthquake catalogue should be complete for magnitude 4 and above events since 1960. Figure 2 also shows the clustered nature of Australian seismicity. We cannot be sure that future earthquakes will follow the same pattern. The long-term pattern of seismicity in Australia is not yet clear, making it difficult to model future activity based only on the distribution of past activity.

### **3. SEISMOTECTONIC MODEL, AUS5**

One of the most essential elements in a probabilistic earthquake hazard study is the source model. The model describes the spatial distribution of the seismic activity and incorporates the b value (ratio of small to large earthquakes) and the rate of earthquake activity for use in probabilistic earthquake hazard studies (Cornell 1968).

In areas like Australia that have a low rate of earthquake activity relative to the rest of the world, and where earthquakes appear to cluster in time and space, the short catalogue of earthquakes that covers just over 200 years is probably not a good sample of the true long-term distribution of the seismicity.

Past studies of earthquake hazard in Australia have used models based on the distribution of past seismicity (Gaul and others 1990, Australian Standards AS1170.4 1993). Given the nature of Australian seismicity it is very difficult for these models to anticipate where the next cluster of earthquake activity may occur.

The AUS5 seismotectonic model assumes that the current seismicity is related to geology and tectonics extending back in geological time for the period during which the current tectonic stress field has existed. The model was developed using geology, geophysics and seismicity. The aim of developing a model using geology, geophysics and seismicity is to improve the model's performance in the low-level clustered seismicity that is present in Australia.

#### **3.1 USE OF GEOLOGY AND GEOPHYSICS**

The AUS5 model was developed using geological and geophysical data to define the boundaries of the source zones. The process was an iterative one with five versions of the model being produced. The fifth and current model, AUS5, was the first generation of the model to organise the information into layers to allow a greater flexibility (Brown and others, in Press).

The seismotectonic model was first drawn using the major geological regions of Australia. These are large structural zones in Australia that are defined by age of formation. Geological regions include; the Lachlan fold belt, Eucla Basin, Canning Basin and the Eromanga Basin. These first zones were crude with little detail. The zones drawn using the geological regions were compared to the spatial distribution of Australian earthquakes (figure 3). The correlation was promising with decreasing activity in the basins. There were still many areas where the seismicity was not well described by the source zones so more information was sought.

The next two sources of broad scale geological information used were a map of gravity anomalies and a map of magnetic anomalies for Australia. The correlation between the first set of source zones drawn and the geophysics was also promising. There were many areas where the source zone boundaries could be altered slightly to better fit both the geological boundaries and changes in gravity and magnetics. There

were also areas where the gravity or magnetics would not fit with the source zones drawn. This implied that there was probably further information that could be added and that the source zones were not governed by any single factor (figures 4 and 5).

The use of geology and geophysics helped define a model that was useful for large scale or regional evaluations of earthquake hazard in Australia. However, for the model to be useful in site specific studies it would need additional detail. Active faults and zones based on seismicity (only in areas where the seismograph coverage was good) were added to the model. The active faults were identified using geological information rather than seismological information (Brown and others, in press).

#### **4. EARTHQUAKE MAGNITUDE RECURRENCE**

In order to evaluate the earthquake hazard in an area using a probabilistic approach, each source zone in the model must have the earthquake activity quantified. The seismicity is characterised in terms of an activity level, the relationship between the number of small to large earthquakes (b value) and a maximum magnitude earthquake (Gutenberg and Richter 1944).

##### **4.1 b VALUE AND ACTIVITY**

Australian b values tend to be below world average, ranging from 0.75 to 1.00 (Doyle and others 1968). A low b value can be an indication of an incomplete catalogue, or an indication of high stress, and in Australia we conclude that it is due to high stress. A maximum likelihood method (Aki 1965) was used to determine the activity and b value for each source zone, and their uncertainties.

The b value is a regional indicator of seismicity so Australia was broken into large regions incorporating several source zones, and the regional b value was calculated for each region (Figure 6). The activity for each source zone in the region was then determined using the regional b value. There were a number of source zones with sufficient activity that they could have their b values calculated individually. All of these zones are areas with b values close to or equal to 1.0.

When determining the activity values, several remote source zones with few earthquakes were grouped together and given a common activity value. In areas with no recorded seismicity the source zones were assigned an activity based on the activity of its neighbours.

##### **4.2 MAXIMUM AND MINIMUM MAGNITUDE**

When dealing with an earthquake catalogue that is far shorter than the average return period of large earthquakes, the activity over a large area must be considered when estimating a maximum credible earthquake. For this study the area considered was taken as all of Australia.

The largest recorded earthquake to occur within Australia was in Meeberrie in 1941, with an estimated magnitude of Ms 7.2 (Doyle and others 1968). A larger earthquake was recorded outside the Australian continent, 800 km northwest of Carnarvon in 1906 with an assigned magnitude of 7.4 (Doyle and others 1968).

The maximum magnitude estimated for Australia is MW 7.5, based on previous large earthquakes that have occurred in Australia and in areas overseas with similar tectonic environments. Most seismologists in Australia agree that an even larger earthquake may be possible, but would be extremely rare, (Gaul and others 1990). The assigned maximum magnitude will usually make little difference to most probabilistic hazard estimates, especially for short return periods of less than 1,000 years.

Australian earthquakes are mainly on reverse faults, so tend to release a high proportion of seismic wave energy in high frequency motion, producing high peak ground accelerations. A well engineered structure is not likely to experience damage from the high frequency, short duration strong ground motion commonly experienced in earthquakes up to magnitude ML 4.0 or even ML 5.0. The choice of minimum magnitude is quite arbitrary, and depends on the purpose of the ground motion recurrence calculations. For this study, the minimum magnitude adopted was ML 4.0.

## **5. GROUND MOTION ATTENUATION**

Earthquake hazard analysis using the Cornell (1968) method requires the selection of an attenuation function. In areas like Australia, where local peak ground velocity, peak ground acceleration and spectral attenuation functions are limited, attenuation functions from areas that have a similar tectonic environment are used.

The tectonic environment in southeastern Australia has reverse faulting due to horizontal compression, and attenuation that is average or above average due to the predominance of Palaeozoic and younger sedimentary geology. Typically most earthquake hazard analysis undertaken in Eastern Australia use functions derived in Western North America (Brown and others 2001). The ground motion recurrence estimates presented in this paper were calculated using the attenuation relationship developed by Sadigh and others (1997) to simulate the attenuation in eastern Australia.

The tectonic environment in Central and Western Australia has reverse faulting due to horizontal compression, and attenuation that is below average because most of the area being made up of Archaean aged cratons. These old hard rocks have extremely low attenuation and are markedly different from the attenuation in eastern Australia. Studies undertaken on the isoseismal maps of past earthquakes highlight this difference (Gaul and others 1990 and Everingham 1968). Typically most earthquake hazard analysis undertaken in Central and Western Australia uses functions derived in Eastern North America. The ground motion recurrence estimates presented in this paper for central and western Australia were calculated using the attenuation relationship developed by Toro and others (1997).

## 6. GROUND MOTION RECURRENCE

The AUS5 Seismotectonic model was used to calculate the peak ground acceleration with a 10% chance of exceedence in 50 years for Victoria and all the capital cities. Both the regional hazard map of Victoria and the site-specific hazard studies of the capital cities were calculated using EZ-Frisk V5.05 (created by Risk Engineering).

The source zones used in the model could either incorporate active fault activity within the zone, or have separate fault source zones. The total number of earthquakes in an area source zone is adjusted depending on whether separate faults are used. When determining the ground motion recurrence for the regional study of Victoria, active faults were not included and their contribution to the earthquake hazard was incorporated into the source zones that contained the faults. The faults were not included because in most cases they produce little noticeable difference in the low-resolution regional hazard map.

For the determination of ground motion for a specific site, the location of nearby faults can have a significant influence on the level of estimated ground motion. In this case, the activity assigned to the faults was included. Only active faults within 100 km of the site need to be included. A sensitivity study concluded that the contribution to the level of ground motion of faults at distances greater than 100 km is negligibly different to estimates made incorporating the fault activity into the area source zones.

The ground motion estimates made using the AUS5 seismotectonic model are similar to those used in the current Australian earthquake loading code (Table 1). The estimates of ground motion are similar between the two models with two exceptions; Adelaide and Hobart. The earthquake hazard in Adelaide was lower using the AUS5 model and the earthquake hazard in Hobart was higher. The variation between the two models is due to both the source model used and the choice of attenuation functions.

City	Longitude	Latitude	AS1170.4 1993	AUS5 2003
	°E	°S	(g)	(g)
Adelaide	138°36'	34°56'	0.10	0.08
Brisbane	153°01'	27°28'	0.06	0.05
Canberra	149°13'	35°17'	0.08	0.09
Sydney	151°13'	33°53'	0.08	0.09
Melbourne	144°58'	37°49'	0.08	0.09
Perth	115°50'	31°55'	0.09	0.09
Darwin	130°50'	12°27'	0.08	0.07
Hobart	147°15'	42°50'	0.05	0.07

Table 1: PGA with a 10% chance of exceedence in 50 years

The AUS5 model was also used to evaluate regional hazard, as shown in figure 7. On a regional scale, the estimates of peak ground acceleration determined using the AUS5 model are in good agreement with the current earthquake loading code (AS1170.4 shown in figure 8). If AUS5 is used to evaluate hazard at particular locations, using quantified active faults, the higher resolution gives greater variations from site to site. In particular, sites on the upthrown (hanging-wall) block have considerably higher ground motion than on the downthrown (foot-wall) blocks.

## 7. CONCLUSIONS

A new model for use in earthquake hazard studies in Australia, based on geology and geophysics, AUS5, was presented as an alternative to models whose source zones are based only on historical seismicity. The boundaries of the source zones in the AUS5 model are based on the boundaries of major geological structures in Australia (figure 3).

The peak ground acceleration with a 10% chance of exceedence in 50 years was calculated using the AUS5 model. These estimates were compared to the estimates made in the current Australian Earthquake Loading Code, AS1170.4 with comparable results.

The true value of the AUS5 model is that it is more likely to incorporate the next cluster of seismic activity in Australia. The hazard estimates produced using this model should be valid before and after the next large earthquake has occurred due to the model being based on long-term tectonic and geological processes that underlie the occurrence of earthquakes in Australia, rather than where a few earthquakes have occurred in the very recent past.

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Figure 1: Australian earthquakes above M6 (January 1872 to December 2002). Catalogue courtesy of the Seismological Research Centre.



Figure 2: Australian earthquakes above M4 (January 1872 to December 2002). Catalogue courtesy of the Seismological Research Centre.



Figure 3: Geological Regions of Australia, with earthquakes from January 1900 to March 2002. Base map courtesy of Geoscience Australia.



Figure 4: Australian Gravity Anomalies and AUS5 Seismotectonic Model. Base map courtesy of Geoscience Australia.



Figure 5: Australian Magnetic Anomalies and AUS5 Seismotectonic Model. Base map courtesy of Geoscience Australia.



Figure 6: b values for AUS5 source zones in Australia. Using earthquakes located by the Seismology Research Centre. Earthquakes to March 2000





Figure 7: Earthquake Hazard Map of Victoria showing the 10% chance of exceedence in 50 years. Using the AUS5 Seismotectonic Model. The calculations were performed using EZ-Frisk. Sadigh and others (1997) attenuation function was used to estimate the peak ground acceleration.



Figure 8: AS1170.4-1993 Seismic Hazard Map for southeastern Australia showing the acceleration coefficient with a 10% chance of exceedence in 50 years.

# BOND AND ANCHORAGE OF COGGED BAR WITHIN CONCRETE-FILLED CHS COLUMN-I BEAM JOINT

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## **ABSTRACT.**

This paper presents some results from an ongoing investigation into the behaviour of an innovative connection between concrete filled steel tubes and steel beams. The connection utilizes blind bolts with extensions which can be bent to form cogs. This paper focuses on developing a finite element model describing the bond behaviour between the deformed reinforcement bars and the concrete. The three-dimensional finite element model is evaluated using previous experimental tests that investigated the bond behaviour of deformed bars with confined concrete reported in the literature. The finite element model can be employed to predict the bond and anchorage performance of coggled bars within the confined concrete restrained by a circular hollow section. Effects of the confinement of tube wall, cog extension, bar diameter, geometry of coggled bar, and concrete strength on the behaviour of coggled bars will be investigated in an experimental program outlined in this paper.

## 1. Introduction

Circular hollow section (CHS) steel tubes have the advantage of restraining local buckling and confining filled concrete. The increase in the use of circular hollow sections in main stream steel structures coupled with the economics of prefabrication have highlighted the need for economical and reliable connections between the steel I beam and the CHS column.

Many full-penetration welded connections at beam ends fractured in most of the damaged steel buildings under the Hyogo-ken Nanbu earthquake in 1995 and the Northridge earthquake in 1994. This damage has highlighted the need for a complete review of the connection design and welding procedure. One of the most effective ways to prevent the fracture at beam-to-column connection is to adopt non-welded connections at beam ends.

An innovative moment-resisting beam-to-circular column connection with blind bolts and extensions is being developed by Gardner and Goldsworthy to suit loading conditions for regions of low to medium seismicity (Gardner and Goldsworthy 2003). The understanding of the behaviour of each component of the connection is essential to the evaluation of the capacity of ductility and energy dissipation for the connection. The research on the bond and anchorage of reinforcing bar in the concrete-filled circular steel tubular columns is a key point.

## 2. Beam-to-column connection

The connection detail proposed is shown in Figure 1 and Figure 2. The beam flanges are connected to the circular hollow section (CHS) column by a built-up T-section consisting of a curved endplate and a flared horizontal web-plate cut into a curved shape at the end to fit with the endplate. The T-sections are attached to the tube wall by blind bolts, and are anchored back into the concrete within the tube by extensions welded on to the bolt heads. The bolt extensions consist of Grade 500 reinforcing bars with a cog.

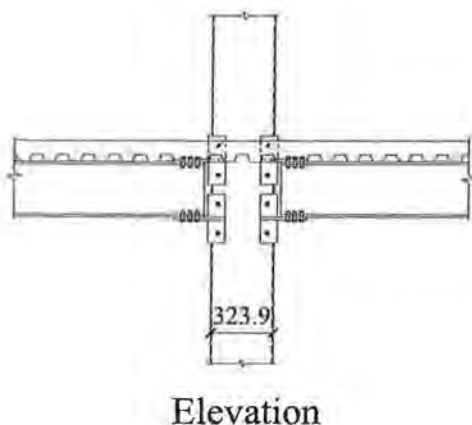


Figure 1: Composite structure

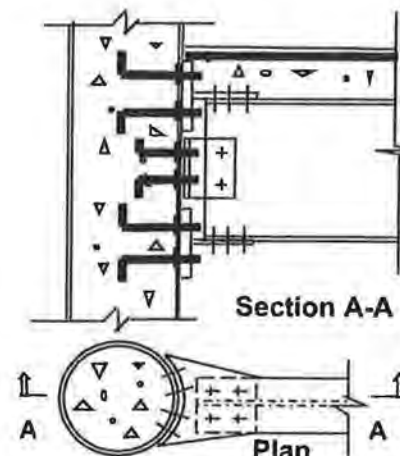


Figure 2: Beam-Column Connection

A series of initial tests on T-stub connections with Grade 400 reinforcing bars of various configurations (long, short, straight, cogged) welded to the head of the blind bolts were executed by Gardner and Goldsworthy in 2000 (Gardner and Goldsworthy 2003). These tests had shown that provision of extensions onto blind bolts into the concrete-filled CHS had increased the load at failure by 20%. The extensions also increased the initial stiffness of the connection element up to 5.2 times the value of the initial stiffness without the extensions.

### 3. Finite Element Analysis

#### 3.1 Element types and material properties

SOLID 45 is used for the 3-D modeling of the solid steel bar. It is defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions. The geometry and node locations for this element type are shown in Figure 3. The element has plasticity, hyperelasticity, stress stiffening, creep, large deflection, and large strain capabilities. The steel for the finite element models is assumed to be an elastic-perfectly plastic material and identical in tension and compression. Poisson's ratio of 0.3 is used for the steel reinforcement in this study. Figure 4 shows the stress-strain relationship used in this study.

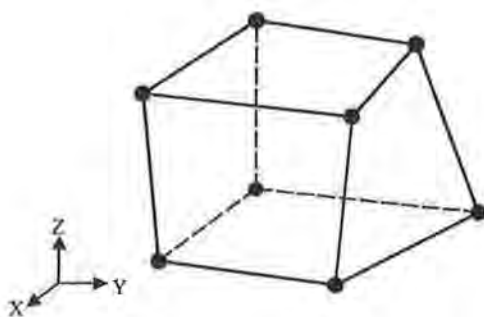


Figure 3: Solid 45 & 65 – 3-D solid (ANSYS)

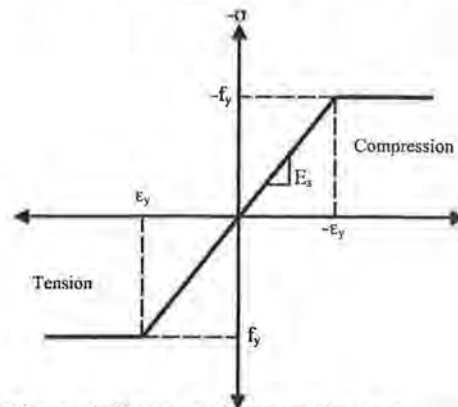


Figure 4: Stress-strain curve for steel

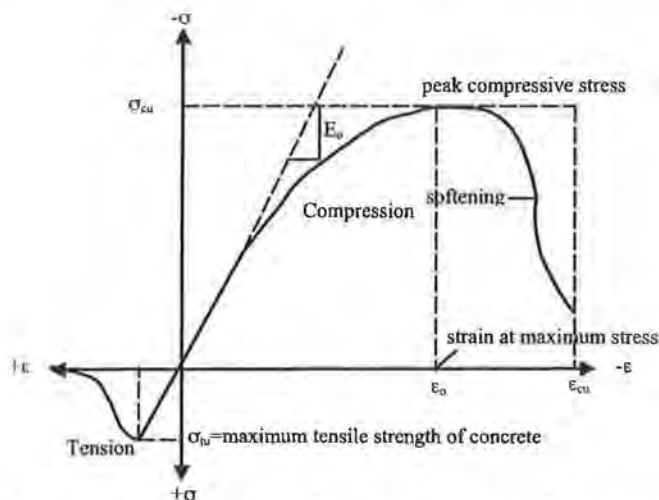


Figure 5: Typical uniaxial compressive and tensile stress-strain curve for concrete (Bangash 1989)

SOLID65 is used to model the concrete in 3-D environment. It is capable of cracking in tension and crushing in compression. The element is defined by eight nodes having three degrees of freedom at each node. In addition, translations in the nodal x, y, and z directions are included. The geometry and node locations are shown in Figure 3.

Concrete is a quasi brittle material and has different behavior in compression and tension. Development of a model for the behavior of concrete is a challenging task. Figure 5 shows a typical stress-strain curve for normal weight concrete (Bangash 1989).

The ANSYS program requires the uniaxial stress-strain relationship for concrete in compression. Figure 6 shows the simplified compressive uniaxial stress-strain relationship that is used in this study. The simplified stress-strain curve for each model is constructed from six points connected by straight lines. The curve starts at zero stress and strain. Poisson's ratio for concrete is assumed to be 0.2.

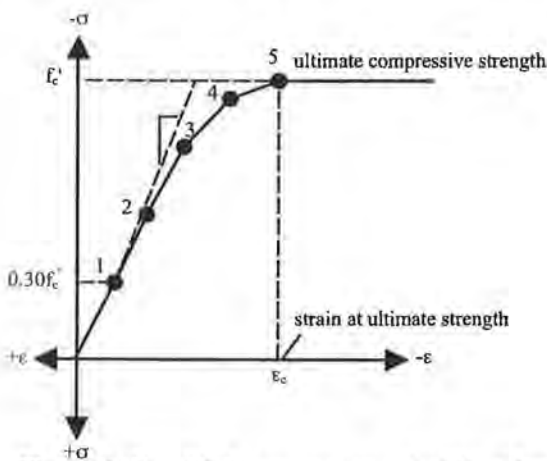


Figure 6: Simplified compressive uniaxial stress-strain curve for concrete

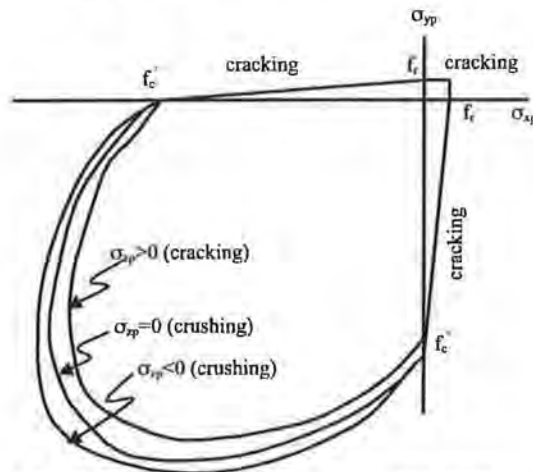


Figure 7: 3-D failure surface for concrete (ANSYS 2002)

A three-dimensional failure surface for concrete is shown in Figure 7. The most significant nonzero principal stresses are in the x and y directions, represented by  $\sigma_{xp}$  and  $\sigma_{yp}$ , respectively. Three failure surfaces are shown as projections on the  $\sigma_{xp}$ - $\sigma_{yp}$  plane. The mode of failure is a function of the sign of  $\sigma_{zp}$  (principal stress in the z direction).

Surface-to-surface contact elements are employed to model rigid-flexible and flexible-flexible contact between reinforcing bar and surrounding concrete. 3-D contact pairs are established with contact element TARGE 170 (assigned to steel) and CONTA173 (assigned to concrete). Each contact element includes several KEYOPTS controlling various aspects of contact behavior.

A small gap 0.01 unit is introduced between steel and concrete for the sake of easy identification of contact interfaces. Initial contact conditions are adjusted using PMIN, PMAX, CNOF, ICONT, KEYOPT(5) and KEYOPT(9).

### 3.2 Finite element discretization

The analysis models for bond-slip behavior of steel bar and concrete over a short segment are constructed as a quarter model because of two symmetric XY and YZ

planes available in the geometry, boundary condition, and applied load. By taking advantage of the symmetry, computational time and computer memory requirements are reduced significantly. The quarter of the entire model is shown in Figure 8. Generating the proper mesh is an important step in finite element modeling.

### 3.3 Nonlinear solution

In nonlinear analysis, the total load applied to a finite element model is divided into a series of load increments called load steps. At the completion of each incremental solution, the stiffness matrix of the model is adjusted to reflect nonlinear changes in structural stiffness before proceeding to the next load increment. The ANSYS program (ANSYS 2002) uses Newton-Raphson equilibrium iterations for updating the model stiffness. If convergence criteria are not satisfied, the out-of-balance load vector is re-evaluated, the stiffness matrix is updated, and a new solution is attained. This iterative procedure continues until the problem converges (ANSYS 2002).

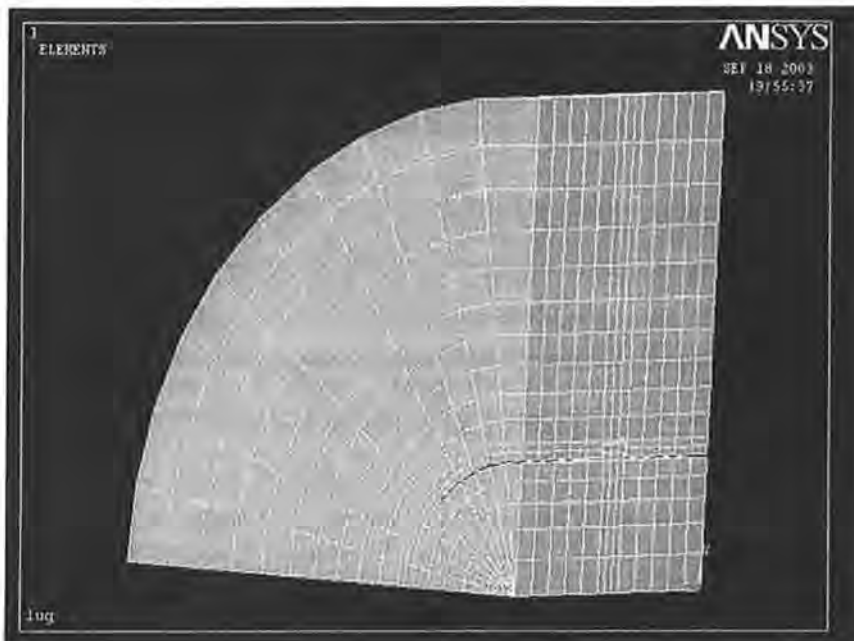


Figure 8: Finite element model

In this study, for the reinforced concrete solid elements, convergence criteria are based on force and displacement, and the convergence tolerance limits are initially selected by the ANSYS program. It is found that convergence of solutions for the models is difficult to achieve due to the nonlinear behavior of reinforced concrete and contact status. Therefore, the convergence tolerance limits are increased to a maximum of 5 times the default tolerance limits (0.5% for force checking and 5% for displacement checking) in order to obtain convergence of the solutions.

### 3.4 Load stepping and failure definition for FE models

For the nonlinear analysis, automatic time stepping in the ANSYS program predicts and controls load step sizes. If the convergence behavior is abrupt, automatic time stepping will bisect the load increment until it is equal to a selected minimum load step size. In

this study, the convergence behavior of the models depends on behavior of the reinforced concrete and contact elements.

### 3.5 Finite element result

A series of pull out tests (Eligehausen et al. 1983) had been executed to investigate local bond stress-slip relationship of deformed reinforcing bars subjected to generalized excitation. Each tested specimen represented the confined region of a beam-column joint. Only a short length (5 times the bar diameter  $d_b$ ) of a Grade 60 deformed reinforcing bar was embedded in confined concrete. Series 1.3 and Series 1.4 tests studied the influence of the confining reinforcement on the bond behaviour.

Figure 9 represents the bar pull out test under monotonic loading. Figure 10 and Figure 11 show the comparison between FEA analysis and test result, which match quite well at the ascending part of bond stress-slip relationship, whereas FEA models encounter convergence problem right after the peak. There is distinctively different behaviour for tests with or without confining reinforcement. Specimen having no confinement failed at a rather small bond stress.

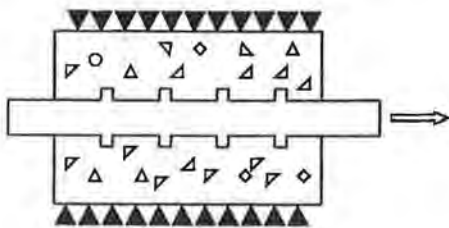


Figure 9: Bar pull out

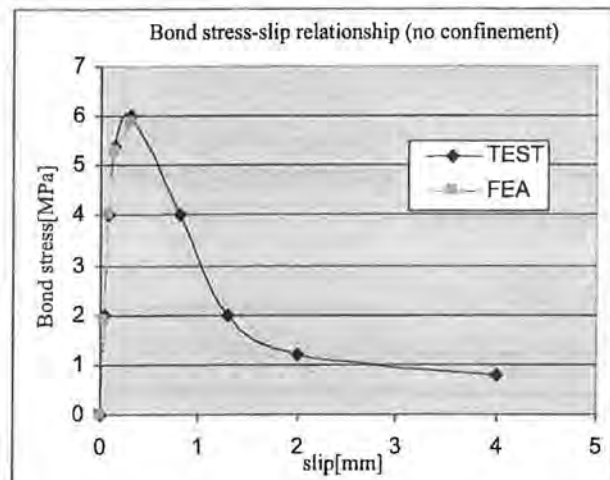
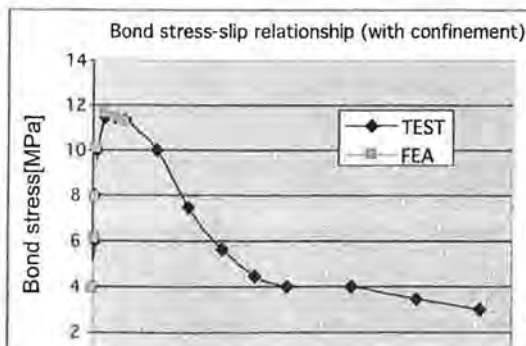
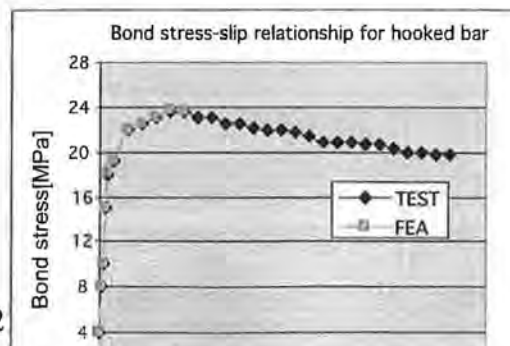


Figure 10: Bond stress-slip relationship for test series 1.4

Seven specimens simulating the behaviour of beam reinforcement anchored by 90-degree standard hooks inside an exterior beam-column connection were tested under pullout forces to determine the hook pullout force-slip relationship (Soroushian et al. 1988). Figure 12 shows the comparison between FEA and test results for the hooked bar pull out test. Hooked extension improves the bond and anchorage behaviour dramatically.



ge 2



**Figure 11:** Bond stress-slip relationship for test series 1.3

**Figure 12:** Bond stress-slip relationship for hooked bar

#### 4. Proposed experimental program

The experiments proposed will simulate an interior beam-column connection. In the test program, two group tests are designated to examine the bond and anchorage characteristics of the cogged deformed N-type reinforcing bars embedded in the infilled concrete of circular steel tubes. Each of the important parameters, cog extension and confinement of the tube, will be varied in turn in order to determine their influence on the monotonic load versus deformation relationship. Based on the experimental data, the finite element model will be evaluated and calibrated.

##### 4.1 Details of tests

The first group will investigate the effect of confinement on the bonding of the cogged bar. The configuration of the cogged bar will remain constant while the D/t ratio of the steel tube will vary.

Details of Group 1:

Constant parameter of cogged bar:	Variable D/t ratio:
N type reinforcing bar Diameter=16mm Horizontal portion= $3d_b=48\text{mm}$ Bend radius= $4d_b=64\text{mm}$ Vertical cog= $12d_b=192\text{mm}$	Circular steel tube (constant size, variable thickness) Diameter (D) =323.9mm Thickness(t) =6.0mm, 8.0mm, 10.0mm D/t ratio =54.0, 40.5, 32.4

The second group will investigate the effect of the length of the vertical cog on the anchorage provided by the cogged bar. The parameter of the steel tube and the horizontal part of the cogged bar will be kept constant while the vertical length of the cogged bar will be varied.

Details of Group 2:

Constant parameter of steel tube and cogged bar:	Variable length of vertical portion:
Diameter(D) =323.9mm Thickness(t) =6.0mm	Vertical cog= $8d_b$ (128mm), $12d_b$ (192mm)



D/t ratio =54.0 N type reinforcing bar diameter=16mm Bend radius=4d <sub>b</sub> =64mm Horizontal portion= 3d <sub>b</sub> =48mm	
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Three identical specimens will be tested for each category. The configuration is shown in Figure 13.

### 4.2 Specimen features

#### (a) Reinforcement

N type steel bar with 500MPa yield strength will be used in the test. This steel bar is manufactured in accordance with AS/NZS4671. The characteristic value of the ratio of tensile strength to yield strength is to be at least 1.08, and the characteristic value of the uniform elongation at maximum load in a tensile test is to be at least 5%, which is slightly lower than that that of 400Y steel.

#### (b) Concrete

The materials used in the preparation of the concrete are ordinary Portland cement, natural Lyndhurst sand and graded gravel of 20-mm maximum size. The mixed proportion (by weight) for 1 cubic meter concrete are 198.9kg water, 327.6kg cement, 351kg of 10mm aggregate and 819kg of 20mm aggregate. This will result in an average cylinder compressive strength of 40MPa (after 14 days) and initial slump of 100mm. A trial mix was tested in 2002.

#### (c) Steel tube

Grade 350 Circular Hollow Section (CHS) manufactured in accordance with AS1163 will be used in this test. The yield stress  $f_y=350\text{MPa}$  and tensile strength  $f_u=430\text{MPa}$ . The actual tensile strength will be determined by a coupon test.



Figure 13: Cogged Bar Pull Out Test

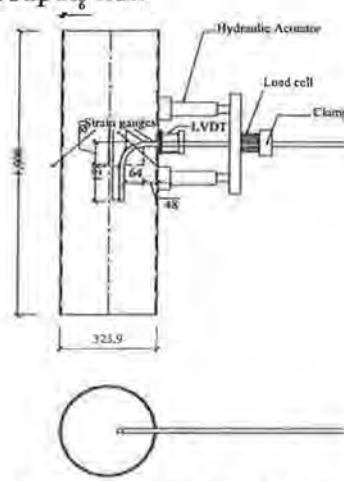


Figure 14: Measurement and loading

### 4.3 Measurement: load, slip, strain in reinforcing bar, stain in steel tube

#### (a) Measure strain in reinforcing bar

To measure the strain in the cogged reinforcing bars, five strain gauges (5mm long) are to be mounted on each bar. Gauges are located at the horizontal and vertical tangency points of the bend portion of the cog, middle of vertical cog and on the horizontal portion of the cog adjacent to the steel tube wall.

(b) Instrumentation for slip

One of the most common methods for comparing pullout tests is through the use of load-slip curves where the slip of the bar relative to the concrete is plotted against the stress or load on the bar. In order to measure the total slip at the edge of circular steel tube, a pin or a flat strip is attached to the bar at the edge of the tube, and a LVDT set on an independent reference object is connected to the pin or the flat strip.

#### **4.4 Loading procedure**

The cogged bar is loaded in increments of roughly 15MPa at 1 minute intervals by a computer-controlled actuator. The test is to be terminated when either the cogged bar is pulled out of the steel tube or the cogged bar yields. Figure 14 shows the configuration for measurement and loading.

#### **5. Conclusion**

Comparisons made with published pull out test data in the literature and finite element modeling prediction show the finite element model is reasonably accurate in ascending part, and can be used to determine the peak value of bond-slip behavior for the cog bar utilized in a concrete-filled CHS column-I beam joint. The presence of adequate confinement inhibits the propagation of splitting cracks and results in substantially higher bond stress. Significant improvement on the performance of anchorages can be achieved when a cog is provided at the end of the bar. The finite element models developed shall be verified and calibrated in accordance with experimental results of the proposed tests detailed in this paper.

#### **Acknowledgements**

We greatly appreciate the generous support of Smorgon Steel in providing steel tube specimens for the experimental program.

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# SEISMIC MICROZONATION OF THE GREATER HOBART AREA, TASMANIA

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## **ABSTRACT:**

This paper summarises the work completed by Darren Andrews as part of his B.Sc. Honours degree at the University of Tasmania, supervised by Dr. Michael Roach. The work assessed the local earthquake hazard present in the greater Hobart area, Tasmania, in two ways. The first method involved analysing how the June 2002 ( $M_L$  4.4) Lake Crescent earthquake (whose epicentre was approximately 80km north of Hobart) was felt across the area. The second method assessed the likely hazard posed to buildings in the area from ground motion amplification within unconsolidated materials during an earthquake, by means of a microtremor survey. Several areas in the greater Hobart region are built upon unconsolidated Cainozoic materials. Darren received one of the 2002 AEES research scholarships based on his proposal for this research, and was awarded first class honours for the research.

## **ABSTRACT**

Tasmania has relatively high seismic risk for an intraplate setting, with numerous large ( $\geq M_L$  4) earthquakes occurring in the region in the last 150 years. Seismic risk in the greater Hobart area was highlighted in June 2002 by an  $M_L$  4.4 earthquake that occurred near Lake Crescent, 75-80km north of Hobart. This event was felt extensively throughout the Hobart area at a predominant Modified Mercalli Intensity (MMI) of 4. The earthquake was experienced more intensely (MMI 4-6) at sites located on Cainozoic, Triassic and Permian sedimentary rocks than at sites located on Jurassic and Tertiary igneous rocks.

Microtremor measurements were conducted at 362 sites across the greater Hobart area to assess the earthquake hazard posed by frequency dependent amplification of ground motion. The resonant frequency of the material at these sites was determined using the Nakamura spectral ratio method. The resultant Nakamura spectra indicated that resonance occurred at many sites and that across the whole area buildings above 12 storeys are at greatest hazard. Within the inner city area of Hobart however, 5-8 storey buildings are most susceptible to resonance. Resonance was most apparent on unconsolidated Cainozoic materials, but was also observed on a number of solid rock sites. Resonance on solid rock substrates has rarely been documented in previous microzonation studies.

## **1. INTRODUCTION**

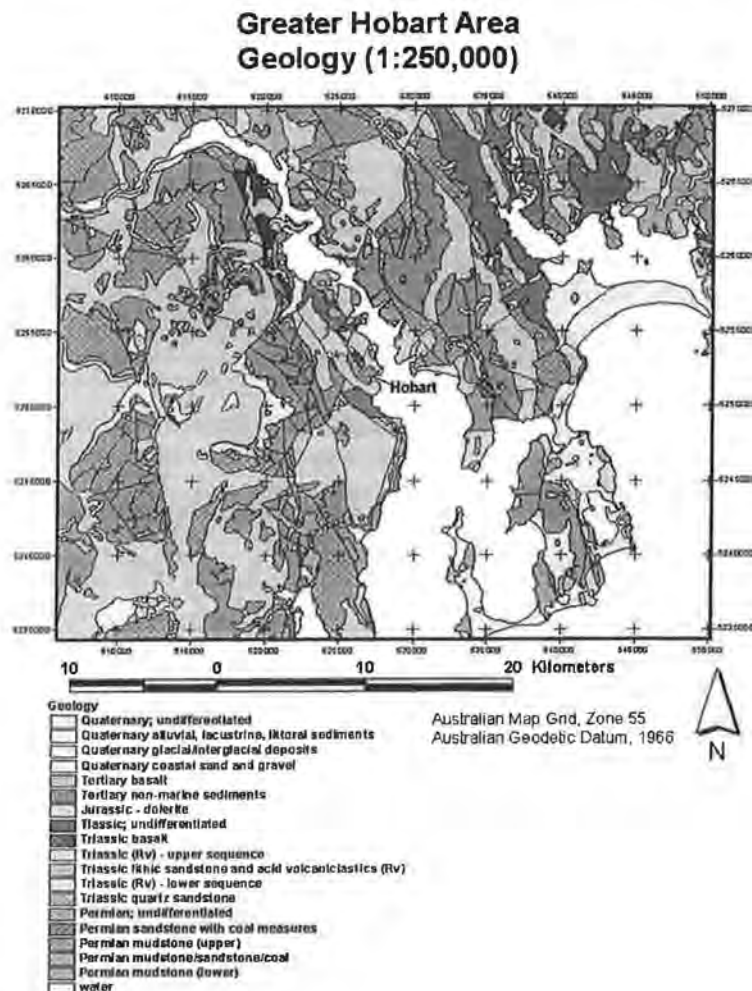
The 1989 Newcastle earthquake highlighted the potential threat to life and infrastructure in major Australian urban centres posed by earthquakes. Since then numerous microzonation studies have been conducted in major Australian urban centres (e.g. Gaul et al., 1995; Michael-Leiba & Jensen, 1996) to define the areas that are most susceptible to frequency dependent ground motion amplification during an earthquake, in doing so indicating the potential for damage to existing and future buildings and infrastructure.

Ground motion amplification on unconsolidated materials has been observed in numerous empirical and theoretical studies (e.g. Anderson et al., 1986; Boatwright et al., 1991). Amplification is frequency dependent, a function of both the thickness and the elastic properties of the material. The frequency at which peak amplification occurs in a material is generally referred to as the resonant frequency.

### **1.1 Geology of the Greater Hobart Area**

The greater Hobart area is situated within the Cainozoic Derwent graben, a setting conducive to the development of thick packages of poorly to unconsolidated sediments. Several suburbs in the area are at least partly built on such sediments, and are therefore likely to be susceptible to amplified ground motion during an earthquake. These areas consist of two main types of material; the first being fine grained "rift fill" materials such as clays, silts and sands related to the Tertiary course of the Derwent River. The second type of material is poorly sorted alluvial material developed on and to the east of the Cascade Fault Zone, the west-bounding fault system of the Derwent graben. The

thickness of these materials has been estimated to range up to >300m, particularly in the case of the rift fill materials. Other rocks that outcrop in the greater Hobart area include the Upper Carboniferous to Triassic Parmeener Supergroup sedimentary rocks, Jurassic dolerite and Tertiary basalt (Figure 1).



**Figure 1:** Geology of the greater Hobart area.

## 1.2 Tasmanian Seismicity

Tasmania has a relatively high seismicity for an intraplate environment. Numerous large ( $\geq M_L 4$ ) earthquakes have occurred in the last 150 years. Tasmania has two major seismic zones; of these the western Tasmanian seismic zone is most relevant to the greater Hobart area, given its proximity (~80km west of Hobart). The recurrence interval for an  $\geq M_L 6$  earthquake in this zone is approximately 290 years (Michael-Leiba & Gaul, 1989).

Seismic hazard in the greater Hobart area is therefore a relevant issue, given the seismicity of the surrounding region and the susceptible geological elements within the area. The risk posed to the area from earthquakes was highlighted in June 2002, with the occurrence of an  $M_L$  4.4 earthquake near Lake Crescent, 75-80km north of Hobart. This study analysed seismic hazard in the greater Hobart area in two forms. Firstly an assessment of how the Lake Crescent earthquake was felt across the area was undertaken, by means of an isoseismal survey. Secondly, the microzonation of the area was analysed by conducting a microtremor survey. The working hypothesis taken into this study was that sites located on Cainozoic unconsolidated materials would, in the case of the Lake Crescent earthquake, correlate with sites of high felt intensity, and in the case of the microtremor study, sites located on these materials would show resonance preferentially to sites located on solid rock sites in the area.

## **2. THE JUNE 2002 LAKE CRESCENT EARTHQUAKE**

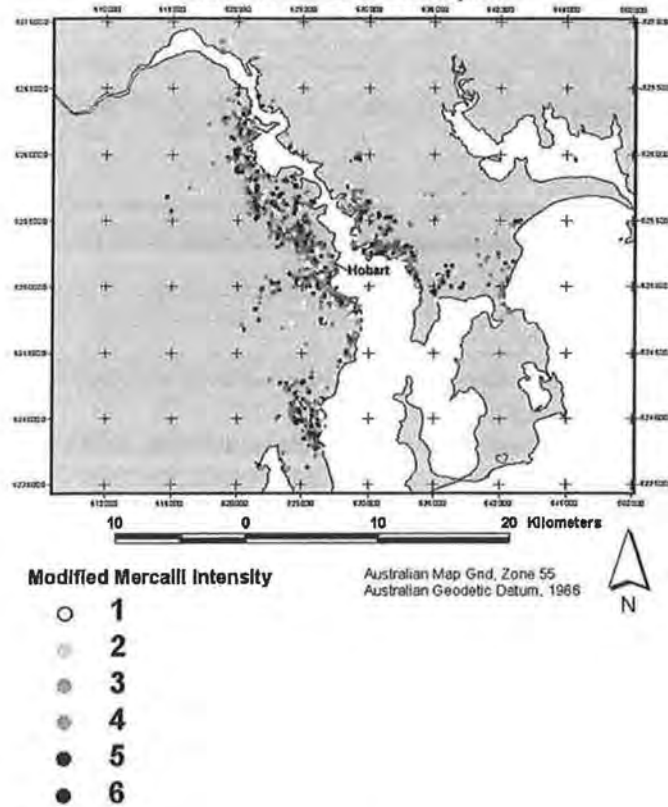
The felt effects of the Lake Crescent earthquake across the greater Hobart area were analysed by means of an isoseismal survey published in *The Mercury* newspaper (June 17, 2002), and on the School of Earth Sciences website. 1280 survey forms were received back from across the area. A Modified Mercalli Intensity (MMI) was assigned to each survey based upon the 1996 New Zealand MMI scale (after Dowrick, 1996).

### **2.1 Results and Discussion**

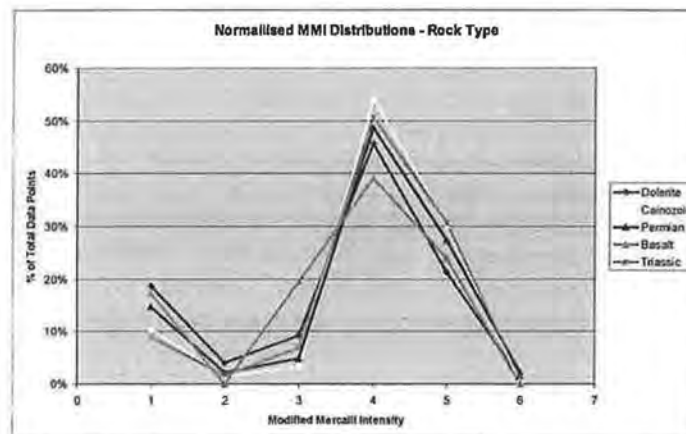
The Lake Crescent earthquake was felt extensively through the greater Hobart area at an intensity range of MMI 1 to 6, with a predominant intensity of MMI 4 (Figure 3). Statistical analysis was then conducted on the data to determine if any correlations existed between high (4-6) MMI values and site parameters such as slope, aspect, elevation and geology. No clear correlations were found between slope, aspect and elevation and MMI distribution. Normalised frequency distributions of MMI created for each of the rock types present in the area (Figure 4) indicate that the earthquake was experienced more intensely at sites located on unconsolidated Cainozoic materials and Triassic and Permian sedimentary rocks than at sites located on Jurassic and Tertiary igneous rocks.

Chi-Squared tests conducted on these distributions confirmed that the differences between the MMI frequency distributions were statistically significant. However the chi-squared tests also indicated that no significant differences occurred between the Cainozoic material and Permian and Triassic rock MMI distributions. Therefore high felt intensities were not restricted to unconsolidated materials in the greater Hobart area as anticipated. Possible reasons for this are given in section 3.1 (points 1-3).

**Isoseismal Map of the  
Greater Hobart Area -  
2002 (ML 4.4)  
Lake Crescent Earthquake**



**Figure 2:** Isoseismal map of the greater Hobart Area for the June 16, 2002 Lake Crescent Earthquake.



**Figure 3:** Normalised MMI distributions for each of the rock types in the greater Hobart area, June 16, 2002 Lake Crescent earthquake.



### 3. SEISMIC MICROZONATION OF THE GREATER HOBART AREA

Microtremor measurements were taken at 362 sites across the greater Hobart area, in order to delineate zones of ground motion amplification. Ground motion amplification was determined at each site using the Nakamura spectral ratio method; a method commonly used in microzonation studies. The Nakamura method calculates the spectral ratio of the horizontal to vertical components of ground motion, assuming that the vertical component of incident seismic energy does not change when travelling between the basement (bedrock) and surface layers. Numerous studies (e.g. Lermo & Chavez-Garcia, 1994; Clitheroe & Taber, 1995) have shown that the Nakamura method provides an accurate estimate of the resonant frequency of a site material, whereas the method has been demonstrated to be inadequate in estimating the amplification factor at a given site (e.g. Field & Jacob, 1995; Coutel & Mora, 1998).

Microtremor measurements were taken using a tri-axial seismometer with a natural frequency of 1Hz. 400 seconds of data was recorded at each site, at 100 samples per second. Spectral ratios were calculated using a data processing program developed by the Urban Geohazards group at Geoscience Australia. Quality values were applied to the spectral ratio from each site, according to the height and width of the resonant peak compared to the background ratio, according to the following quality criteria developed by Geoscience Australia (C. Sinadinovski, Pers. Comm., 2002):

Quality	"A"	"B"	"C"	"D"
Height of Resonant Peak Above Background Ratio	>3×	2 to 3×	<2×	No Resonant Peak
Width of Resonant Peak (Hz)	<0.6	0.6 to 1.2	>1.2	No Resonant Peak

**Table 1:** Quality criteria applied to the Nakamura spectral ratio data.

Only "A" and "B" quality spectral ratios were incorporated in this study, reducing the number of data points in the dataset to 187.

The resonant frequency of a site material can be used to quantify earthquake hazard in terms of the size of building at most hazard on that material. The building at greatest hazard on a given material is the building whose resonant frequency matches the resonant frequency of the underlying material. The resonant frequency of a building is a function of its height and mass. Michael-Leiba and Jensen (1996) give a simple formula that approximates the resonant frequency of a building with N stories:

$$F_N \text{ (Hz)} = 10/N$$

Using this formula, building hazard classes were created for this study as follows:

<b>Building Size (No. of Stories)</b>	<b>Calculated Resonant Frequency Range (Hz)</b>	<b>Hazard Classes used In Hobart Study (Hz)</b>
1 to 4	10 to 2.5	$\geq 2.5$
5 to 8	2 to 1.25	2.5 to 1.25
9 to 12	1.11 to 0.83	1.25 to 0.8
Above 12	$\leq 0.83$	0.8 to 0.15

**Table 2:** Building hazard classes used for the Hobart microzonation survey.

### 3.1 Results & Discussion

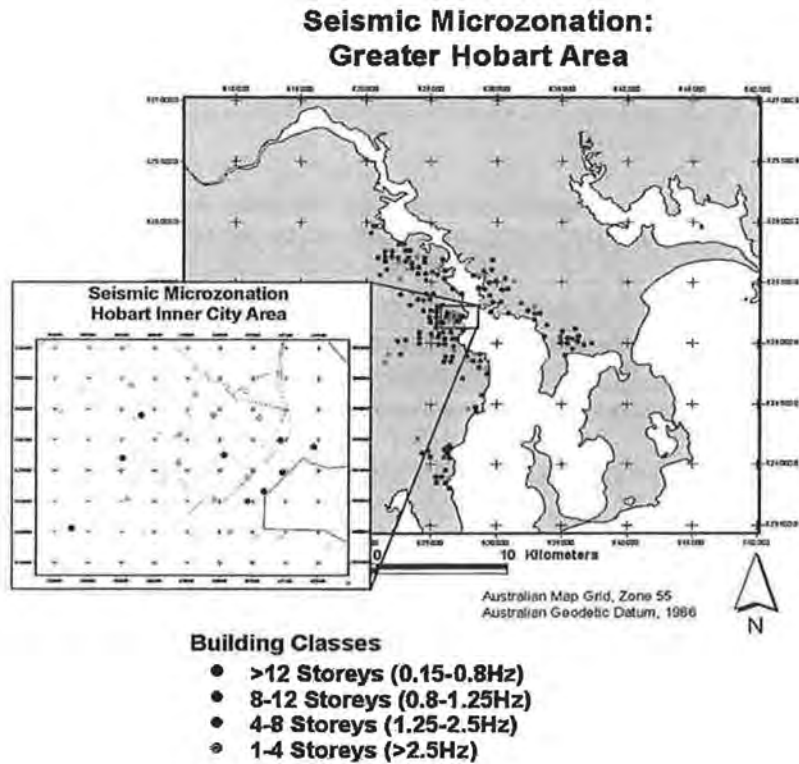
The resultant microzonation map is shown in Figure 5. Across the greater Hobart area, the potential hazard of resonance during an earthquake is greatest for buildings of 12 storeys or more, while hazard is extremely low for 1-4 storey buildings. Within the inner city area of Hobart, the greatest hazard from resonance occurs for 5-8 storey buildings.

Some interesting results were obtained when comparing the microtremor data to the geology of the field area (1:25000 scale). It was found that while the potential hazard from resonance was most apparent on sites located on unconsolidated Cainozoic materials, resonance was also measured on the various solid rock types (Figure 6). Resonance on solid rock sites is very rarely observed in microzonation studies. Several factors may account for this phenomenon:

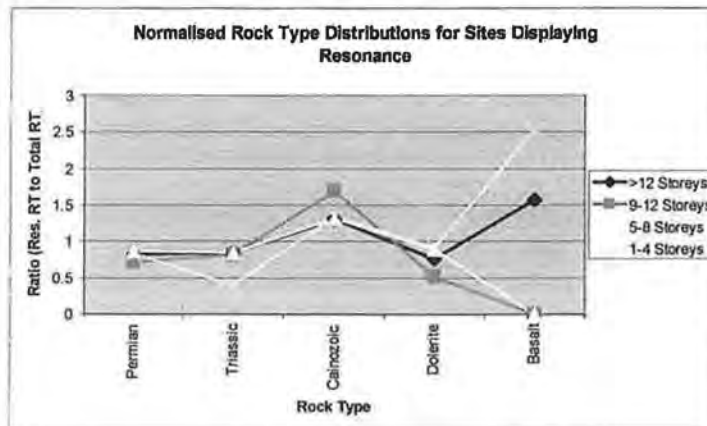
1. Variable development of unconsolidated or weathered material on top of what has been previously mapped as solid rock. Heavily fractured and extensively weathered rocks such as basalt and dolerite may be susceptible to this process, and may also have been misinterpreted as solid rock sites during mapping.
2. Development of thick deposits of unconsolidated 'excavation fill' on sites previously mapped as solid rock. This may be particularly applicable to the older, inner city areas of Hobart that have had a long development history.
3. The available geology maps (1:25000) may not approximate site conditions accurately enough for microzonation purposes.
4. Propagation of cultural-source microtremors containing Love waves on solid rock sites, contributing to increased horizontal ground motion at specific frequencies.

### CONCLUSIONS

Both the isoseismal and microzonation data indicate that a real hazard exists within the Hobart region from the effects of earthquakes. However heightened intensity and resonance occurred not only on unconsolidated sediment sites as anticipated, but also on solid rock sites in the area. Possible reasons for this are given in section 3.1. Further work should include characterising materials at measurement sites to determine their exact nature. This would validate the microtremor and isoseismal measurements, and provide useful information for further earthquake engineering studies in the area.



**Figure 4:** Seismic microzonation of the greater Hobart area, and (inset) the Hobart inner city area.



**Figure 5:** Ratio of rock type distributions on which resonance occurred (“Res. RT”) to total rock type distribution (Total RT), based upon 1:25000 scale site geology values.

## ACKNOWLEDGEMENTS

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# IMPROVED VULNERABILITY MODELS FOR REGIONAL EARTHQUAKE RISK ASSESSMENT

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The first four authors are students in their final year of study for the BE degree at The University of Adelaide. This paper presents the results of their final year (honours) research project. Mike Griffith was their supervisor on this project. He has been at The University of Adelaide since 1988. His main research and professional interests are in the areas of structural dynamics and earthquake engineering, focusing in particular on seismic effects on unreinforced masonry and non-seismic reinforced concrete buildings.

## **ABSTRACT:**

This project involves studying existing methods of earthquake risk assessment, on a regional basis rather than on a building by building basis. Current methods typically use a HAZUS type model where  $RISK = HAZARD * ELEMENTS AT RISK * VULNERABILITY OF ELEMENTS AT RISK$  to assign estimates of likely damage in various sized seismic events for cities or regions. It is an extremely useful emergency management tool, and also of great interest to insurers who foot the bill. The problem is that existing models don't seem to give very good correlation with the damage reported in the Newcastle region after the 1989 earthquake. It has been suggested by researchers from Geoscience Australia and The University of Melbourne that the problem is most likely that the VULNERABILITY part of the model does not account for building degradation, quality of construction, and local out-of-plane failure mechanisms in URM buildings. In this project, a team of honours research students have developed a two-step vulnerability assessment procedure for URM buildings that considers both the global in-plane and local out-of-plane URM building response. Data from the Newcastle earthquake was used to verify the accuracy of this new approach. The results are very promising as a similar level of accuracy in the overall damage estimate was achieved but with a much better correlation with the observed distribution of damage on a suburb-by-suburb basis.

## 1. INTRODUCTION

The need for earthquake risk assessment in Australia was realised in the wake of the Newcastle Earthquake of 1989, which resulted in twelve deaths and caused more than one million dollars (1990 AUD\$) of damage. Risk assessment enables the prediction of the financial loss resulting from a natural disaster in terms of structural, non-structural and contents damage and loss-of-business. It is a valuable tool for insurance companies, emergency services and government agencies. Towards that end, Geoscience Australia (GA) developed a MatLab-based software program called "Earthquake Risk Management" (EQRM) for regional earthquake risk assessment in Australian cities, based on the HAZUS (Hazards United States) model that is widely used for risk assessment purposes around the world.

GA has adapted the HAZUS model to better represent Australian conditions where URM is a much more common form of building construction. While changes have been made to the way URM is represented in EQRM, it is thought that further improvements are still possible (Edwards et al, 2002). In particular, the recorded and predicted damage distributions for the 1989 Newcastle earthquake event still differ significantly.

This paper presents a brief overview of the methodology before describing potential improvements for the treatment of URM construction. Assessment of the correlation between the predicted and observed distribution of damage during the 1989 Newcastle earthquake is then used to test the feasibility of some of these improvements.

## 2. GA METHODOLOGY

The GA loss estimation methodology is implemented in EQRM which calculates loss estimation on a regional basis. The methodology consists of four modules and is represented schematically in Figure 1.

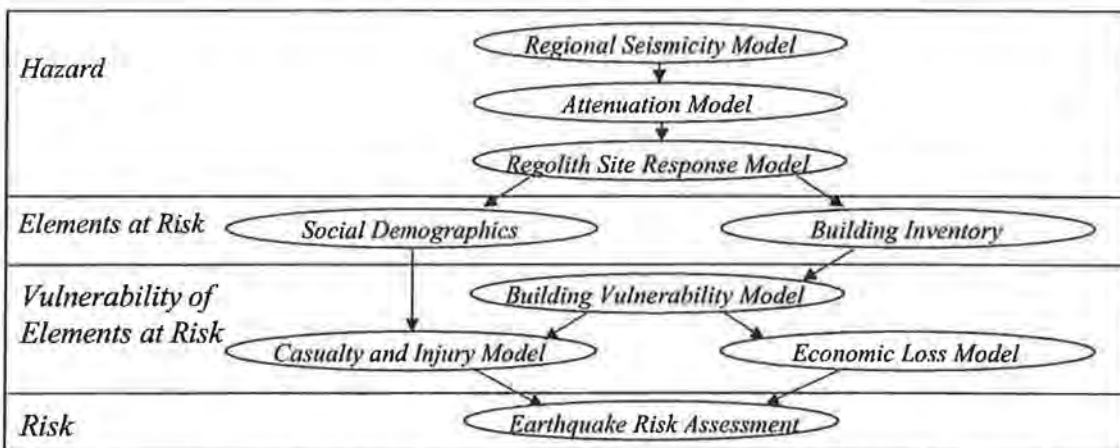


Figure 1. The GA earthquake risk assessment methodology (Geoscience Australia 2002)

The hazard module quantifies the seismic hazard, that is, the ground motion that a seismic event will cause. The most important output of the hazard module for this study is the quantification of the earthquake load (demand) on potential structures in the study region which is represented graphically as the earthquake demand curve (refer Fig. 2). The elements-at-risk module contains input data regarding the social demographics and structures subjected to the demand. The vulnerability module calculates the building

capacity, in the form of a capacity curve (Fig.3), based on its construction type, form and use. This is defined in EQRM in terms of two points; the yield point ( $D_y, A_y$ ) and the ultimate point ( $D_u, A_u$ ) as shown in Fig.3. The specific input parameters used for URM buildings to define these two points are discussed in the next section of this paper. Importantly, the vulnerability module then calculates the response of each building type to a given earthquake event by finding the displacement corresponding to the point where the earthquake demand curve intersects the building capacity curve (refer Fig.4). This module then calculates the predicted damage to building and contents as a function of the lateral displacement response. The number of casualties and injuries and the economic loss expected, given the damage calculated by the building vulnerability model are also calculated here. Finally, the risk module presents the results of the assessment in terms of economic loss and casualties.

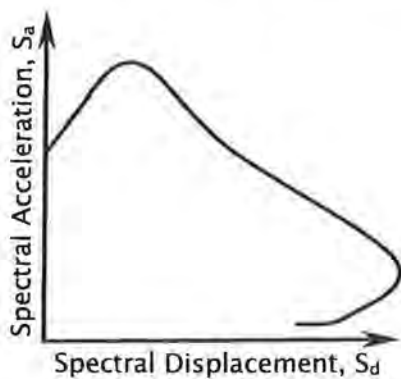


Figure 1. Spectrum Demand Curve.

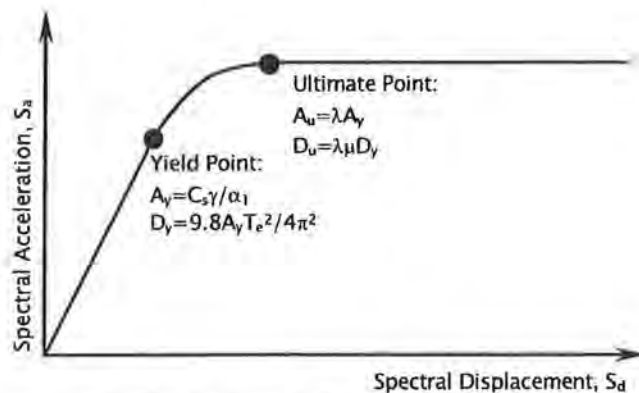


Figure 2. Generic Building Capacity Curve.

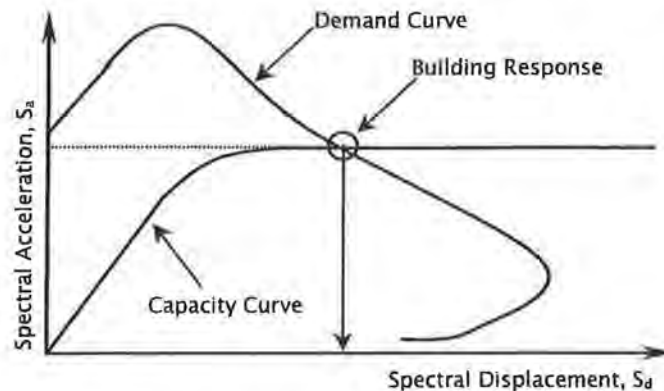


Figure 4. Capacity spectrum calculation of building response.

### 3. IMPROVED PROCEDURE FOR URM

Reports from the Newcastle earthquake have shown that most of the damage to URM occurred due to out-of-plane failure. Widespread incidence of corrosion of wall ties led to facade failure and parapets and suspended awnings were a major source of damage due to inadequate anchorage as is detailed in a report published by The Institution of Engineers, Australia (Melchers, 1990). A footpath survey conducted by Geoscience Australia immediately after the event further confirmed this result.

To investigate the feasibility of accounting for out-of-plane response as part of the earthquake risk assessment procedure, it was decided to trial a parallel method of “capacity spectrum” assessment using both in-plane and out-of-plane capacity curves. The following two paragraphs present a brief summary of the evidence in the literature upon which the parameters for defining the URM building capacity curves were based.

For in-plane wall response, the values for Dy and Du taken from the literature have been normalised by dividing the lateral displacements by the height of the test specimens so that the test results on walls of vastly differing geometry can still be compared directly on the basis of storey drift. The values typically reported in the literature correspond to the drift at which cracking occurs (and usually also ultimate strength) and the maximum displacement at which a wall’s loss of strength becomes significant. Values are given in Table 1 where the Dy and Du values correspond to “cracking” and “maximum displacement”, respectively. The values in the last line of the table were used to produce the initial values for use by the authors in their subsequent analyses.

Table1. Summary of in-plane capacity curve drift values.

Dy (% h)	Du (% h)	Source
0.1		Gallipoli et al (2003)
0.1	1.0	Fardis & Calvi (1995)
0.3		Zhao et al (2003)
0.2 – 0.3	2.0	Zarnic (1995)
0.2 – 0.3		Valiasis & Stylianidis (1989)
0.05 – 0.10	0.6 – 0.9	Bosiljkov et al (2003)
0.1 – 0.2	1 - 2	Authors’ (initial) EQRM values

For out-of-plane wall response, the results of this survey are summarised below in Table 2 where the drift values for Dy correspond to first cracking and the values for Du correspond to drifts equal to about 2/3 of the wall thickness. For out-of-plane walls, the concept of drift is taken as the wall deformation (relative to its supports) divided by the wall span. For example, a maximum wall displacement of 73 mm (= (2/3)\*110mm) corresponds to a value for Du of approximately 2.4% (= (73/3000)\*100%) for a 110 mm thick, 3m tall wall.

Table 2. Summary of out-of--plane capacity curve drift values.

Dy (% h)	Du (% h)	Source
0.1	1.5%	Lawrence (1983)
0.03	0.68	Doherty (2000; et al 2002)
0.1	2.5	Griffith (2000)
0.39		Hamoush et al (2001)
0.1	2.0	Authors’ (initial) EQRM values

### 3.1 Capacity Curves

Since the economic loss predicted by EQRM depends upon the displacement response (corresponding to the intersection point in Fig.4), appropriate values for the definition of building capacity curves are important in producing an accurate output. Determination of drift ratios to be used for the in- and out-of-plane capacity curves for URM



construction was therefore a focus of the current research. Appropriate values, based on the available evidence in the literature, were used as a starting point in analysis. These values are listed in Tables 3 and 4 as “Authors (initial)” values for URM buildings with tile roof and metal roof. It should also be noted that the values for  $D_y$  and  $D_u$  in Tables 3 and 4 are in terms of displacements (mm), not drift ratios (%). The displacement values were determined by multiplying the corresponding drift ratios by the corresponding building height. The low- and mid-rise building heights were taken as 4.5m and 10.7m, respectively (corresponding to the values of 15 feet and 35 feet in HAZUS).

Table 3. Values for capacity curve definition – URM with tile roof.

		$A_y$ (g)		$D_y$ (mm)		$A_u$ (g)		$D_u$ (mm)	
		low-rise	mid-rise	low-rise	mid-rise	low-rise	mid-rise	low-rise	mid-rise
HAZUS		0.20	0.11	6.1	6.9	0.40	0.22	61	46
GA		0.30	0.20	1.7	3.9	0.60	0.40	6.7	15.6
Authors (initial)	in-plane	0.11	0.074	4.5	4.5	0.22	0.146	70	70
	out-of-pl.	same as in-plane values							
Authors (final)	in-plane	0.128	0.086	5.0	12.0	0.257	0.171	82	190
	out-of-pl.	0.133	0.089	6.7	15.6	0.267	0.178	26.8	62.3

Table 4. Values for capacity curve definition – URM with metal roof.

		$A_y$ (g)		$D_y$ (mm)		$A_u$ (g)		$D_u$ (mm)	
		low-rise	mid-rise	low-rise	mid-rise	low-rise	mid-rise	low-rise	mid-rise
HAZUS		0.20	0.11	6.1	6.9	0.40	0.22	24.4	27.6
GA		0.4	0.3	1.7	3.9	0.8	0.6	6.7	15.8
Authors (initial)	in-plane	0.146	0.11	4.5	10.5	0.292	0.22	70	164
	out-of-pl.	same as in-plane values							
Authors (final)	in-plane	0.171	0.128	5.0	12.0	0.342	0.257	82	192
	out-of-pl.	0.178	0.133	6.7	15.8	0.356	0.267	26.8	63.0

A perceived major drawback of the current capacity curve for URM in EQRm is due to its shaped – it does not allow for strength loss beyond the point of ultimate strength such as is expected for non-ductile URM construction. The consequence of using a horizontal curve to extend beyond  $D_u$  is that the displacement response given from the capacity spectrum intersection point will underestimate the likely displacement response when the intersection point occurs at points along the capacity curve beyond  $D_y$ . This task was beyond the scope of the current project but in order to more accurately model URM construction in EQRm, it will be necessary to change the shape of the current capacity curve for URM and allow for post-peak softening.

### 3.2 Threshold drift values for URM buildings

HAZUS specifies four threshold drift ratios to which are assigned corresponding damage ratios of 9.0%, 17.0%, 43.0% and 100% (FEMA 1999). Bommer et al. (2002), in their development of a risk assessment model based on the 1999 Kocaeli earthquake, assumed damage ratios of 2%, 10%, 50% and 100% for slight, moderate, extensive and complete damage. The ratios used by Geoscience Australia are noticeably different to

those used in HAZUS and Bommer, being 25% for slight, 40% for moderate, 60% for extensive and 100% for complete damage. Therefore, values for the four threshold drift ratios corresponding to the four levels of damage were also investigated.

The drift ratios used in the original EQRM model are presented in Table 5 below along with the original HAZUS values and those recommended by participants at a workshop convened by Geoscience Australia (Stehle et al. 2001). It is noted that the HAZUS and workshop values vary significantly from the GA values, but that the HAZUS and workshop values are of the same order and are reasonably consistent with the values from the literature for in-plane behaviour (refer Table 1). It was found from the literature that the smaller threshold values used in the original EQRM model are probably more applicable to the out-of-plane behaviour of URM (refer Table 2). These values have been used as a reference in defining new drift values.

Table 5. URM damage threshold drifts.

		Damage State			
		Slight	Moderate	Extensive	Complete
HAZUS	in-plane	0.2%	0.4%	1.0%	2.4%
GA	in-plane	0.05%	0.08%	0.12%	0.2%
Workshop	in-plane	0.1%	0.8%	1.5%	3.0%
Authors (initial)	in-plane	0.5%	0.8%	1.2%	2.0%
	out-of-plane	0.05%	0.08%	0.12%	0.2%
Authors (final)	in-plane	0.2%	0.6%	1.2%	2.0%
	out-of-plane	0.05%	0.08%	0.12%	2.0%

### 3.3 Sensitivity Analyses

In order to determine whether improved correlation between the observed and predicted distribution of damage (on a suburb-by-suburb basis) could be obtained using the EQRM model, a sensitivity analysis was performed. The input parameters used in EQRM to define the displacement values for the in-plane and out-of-plane capacity curves and the drift-damage threshold limits were studied. As shown in Fig.6, the results were most sensitive to changes in the parameters,  $C_s$ ,  $T_e$  and  $\lambda$ .

The EQRM program was then used to compute the economic loss, on a suburb-by-suburb basis, for the Newcastle earthquake simulated event. The loss values were then compared to insurance data provided by the NRMA insurance company. However, only suburbs included in both databases were used in the comparisons.

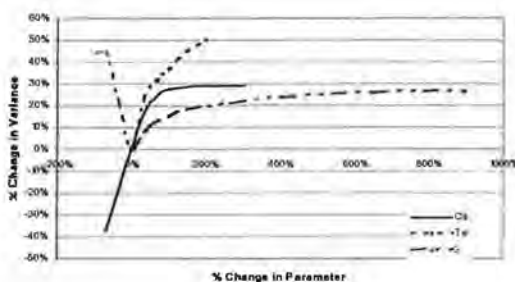


Figure 6. Sensitivity analysis results.

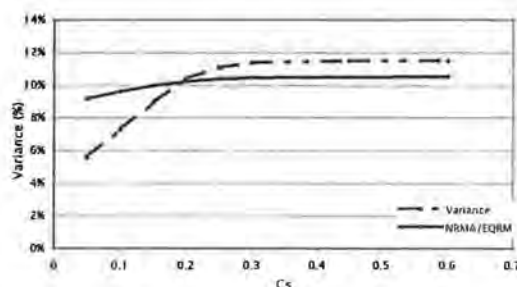


Figure 7. Minimisation of variance for  $C_s$ .

The assessment criteria for establishing the optimal values for the input parameters consisted of minimising the variance ( $\sigma^2$ ) given by

$$\sigma^2 = \frac{\sum_{i=1}^n (x_i - \mu)^2}{n-1} \text{ where}$$

$x_i$  = the (NRMA loss ratio) divided by the (EQRM calculated loss ratio) for suburb “ $i$ ”;

$\mu$  = the mean of  $x_i$ ; and

$n$  = the total number of suburbs in the region.

In addition to minimising the variance, the total loss ratio was also required to remain at approximately 10% (GA, 2002). A typical plot is shown in Figure 7 where it can be seen that the optimal value  $C_s$  (while maintaining an EQRM estimate of 10% for the total loss ratio for the region was approximately 0.2.

After extensive modelling, it was found that the values in the last two rows of Tables 3 – 5 (Authors (final)) gave the best correlation with the pattern of damage distribution observed in Newcastle during the 1989 earthquake. With the final Author values the overall variance was reduced to 7.25% from the 8.89% value obtained using the original GA capacity curves and threshold drifts. From these results it can also be seen that the final values used to specify the in-plane and out-of-plane capacity curves are much more realistic and consistent with the values reported in the literature. Not surprisingly, the original GA values correspond more closely to values corresponding to out-of-plane wall behaviour. Since much of the damage was associated with out-of-plane response, the overall GA loss estimates were quite good.

It should be noted that while these analyses were performed using the parallel capacity spectrum evaluation method (for in-plane plus out-of-plane curves), almost every assessment indicated that out-of-plane damage was significantly more than that due to in-plane response.

#### **4. SUMMARY AND CONCLUDING REMARKS**

A review of the current literature has provided strong evidence for further modifications of the building capacity curves for URM buildings in the EQRM/HAZUS model. Using a parallel capacity spectrum assessment procedure with modified in-plane and out-of-plane capacity curves for URM buildings, further improvements in the correlation between the observed and predicted distribution of damage have been achieved. However, adjustments of the model to account for post-peak strength softening of the capacity curves could still further improve the accuracy. Other issues such as building degradation (function of age) could be accounted for fairly easily where this information is available as part of the building inventory but workmanship, a significant issue in the Newcastle event (Swiss Re, 1990), is probably a local characteristic that could be very difficult to quantify in terms of a capacity curve. It may be better accounted for by adjustment of loss percentages corresponding to the threshold drift values.

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# A SPATIAL REGOLITH DATABASE FOR AUSTRALIAN CITIES – HOW COULD IT AID EARTHQUAKE DESIGN?

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## ABSTRACT

Since 1995 the Risk Research Group of Geoscience Australia (GA) has been assessing seismic risk in Australian cities. As part of this work GA has collected a large quantity of layer thickness and shear wave velocity information on the regolith (i.e. soils, geological sediments and weathered rock) of a number of Australian cities. This spatial data may serve as a valuable supplement for assessing sub-soil classes where the scale of a building development and its locality effectively limit the information available on deeper regolith layers. To maximise the usefulness of this information if made available GA needs input from geotechnical engineers and structural designers who potentially will use it. This paper aims to draw the attention of Australian engineers to this potential resource and to initiate some discussion on how this could best be made accessible. The feedback will assist GA in expending resources so as to achieve the maximum benefit from what could be a progressively expanding data source.

## **1 INTRODUCTION**

The presence of soils, geological sediments and weathered rock (collectively referred to as regolith) can significantly increase the intensity of earthquake related ground shaking. Tangible evidence of this effect was observed in the damage sustained during the Newcastle earthquake where the degree of damage and the regolith depth showed a correlation (Institution of Engineers 1990). For this reason site information on parameters known to influence the response of regolith during an earthquake is valuable in any process attempting to estimate or mitigate earthquake effects. Included in this information are regolith type, thickness and shear wave velocity. The site factor in the Australian earthquake loading standard (Standards Australia 1993) used to assess the earthquake hazard at a specific site is selected based on information of this type. Similarly, more detailed probabilistic earthquake hazard assessments for regolith sites invariably use this information when incorporating the effect of local regolith.

Over the last nine years GA has been conducting detailed, localised, multihazard risk assessments for a number of Australian communities. As part of this work GA has collected an extensive range of regolith data in regions varying from Perth to Newcastle. Furthermore, GA is commencing a process of broader scale nationwide risk assessments that will require the acquisition and compilation of additional regolith information. Proposed participation by GA in the Disaster Mitigation Australia Package (DMAP) announced in the recent budget will lead to a more extensive regolith database across the nation.

Several groups have expressed an interest in this dataset, including the Newcastle branch of the Institution of Engineers (Dhu 2003) and Standards Australia Working Group 20 (WG20 2003). While GA is amenable to releasing this data for public use this would require a significant investment of time and resources. GA is seeking input from the engineering community relating to how this data could best be released in order for the work to be justified and efficiently implemented. More specifically, GA is interested in understanding how the engineering community would most likely use this data and how they would prefer it to be made available.

## **2 DESIGN CODE METHODOLOGIES FOR SEISMIC HAZARD ASSESSMENT**

Seismic hazard assessment is typically the first step in calculating earthquake loads for design purposes. Determining an appropriate hazard spectrum using code based procedures requires the categorisation of the regolith at the construction site. While design standards differ on the precise methods used and on certain site category details they do have common features. More recent code revisions also show a shift in emphasis on how sites can best be classified by the practitioner. These aspects are discussed below to provide a context for ascertaining the information most useful to geotechnical engineers and structural designers.

The regolith depth considered in the selection of site category varies between code documents. The IBC (2000) fixes the depth of interest to 30 m for most site classes. The current Australian loadings standard uses 30 m as a demarcation depth but requires

knowledge of the sub-soil layers at greater depths to differentiate softer regolith profiles. In a similar manner the currently proposed amendments to AS 1170.4 (Standards New Zealand 2003) use 30 m as a demarcation depth but in some cases categorises sites using information from depths in excess of 60 m. Clearly the geotechnical investigation for a proposed low-rise structure having pad foundations would be unlikely to furnish information for depths in excess of 15 to 20 m. In recognition of this practical limitation design standards permit some heuristic judgement by the geotechnical engineer based on previous deeper regolith investigations in the area and a broader knowledge of the local geology (AS1170.4 1993 Table 2.4 Footnote #1, IBC 2000 Cl 1615.1.1). This is justified in part by the generally stiffer nature of regolith layers at greater depths which consequently have a lesser influence on overall site behaviour. However, the inclusion of total regolith depth information in a proposed database may be a useful supplement where the total profile depth has not been sounded as part of the geotechnical investigation.

Quantitative regolith properties are valuable in making site classifications. While qualitative borelog descriptions and assessments of layer stiffness or density are useful, physical measurements of soil parameters are more reliable. Empirical relationships from the literature permit the conversion of in-situ measurements of undrained shear strength and SPT values into moduli of rigidity that relate directly to the site response. Direct measurement of shear wave velocities is required by the IBC (2000) for a hard rock site classification and can be used for softer rock. The increasing use of shear wave velocity values in site classification suggests that field measurements of this parameter from test locations across urban areas may also be a useful component of a regolith database.

The approaches for aggregating the behaviour of a layered deposit also have a range of sophistications. The current AS 1170.4 (Standards Australia 1993) and its draft replacement (Standards New Zealand 2003) present scenarios of soil layer type combinations that permit a more qualitative assessment of site classification. However, proposed code revisions imply or express a preference for aggregating the flexibilities of all layers to obtain an overall shear wave velocity that can be converted into a site period. A procedure for carrying this out is described by Lam and Wilson (1999) that includes the adjustment of the low amplitude site period to that associated with the higher shear strains induced in the soils during an earthquake. Field observations of layer shear wave velocities and the low amplitude site period based on the Nakamura (1989) spectral ratio method may be a further useful addition to the database.

A review of code documents suggests that there is a general move away from a descriptive site classification (UBC 94) to one based on the low strain shear wave velocity of the layers (IBC 2000, EC8 1996). Site periods calculated from the same shear wave velocity data are becoming key parameters for differentiating between site classes as in the proposed AS1170.4 draft document (Standards New Zealand 2003).

### **3 REGOLITH DATA COLLECTION AND CHARACTERISATION**

The methodology and quality of the earthquake risk assessments conducted by GA have evolved with time. Consequently, the quantity and type of regolith information

available from study area to study area varies. For example, in GA's most recent work, the approach used has been based on a combination of published boreholes, seismic cone penetrometer tests, natural period measurements from passive seismology, digital terrain models and surface geology maps (Dhu and Jones, 2002). In other work, less detailed datasets have been acquired. For example in work in the Botany Bay region of Sydney, the only available datasets were natural period measurements and some published boreholes (Dhu *et al*, 2003).

GA has compiled and interpreted this field data in order to classify the regolith within each study area. Each regolith classification has been designed to describe regions that are expected to respond in a consistent fashion during an earthquake. To date, a different classification regime has been used in each study area, and consequently the classes are very region specific. For example, within the Newcastle and Lake Macquarie regions GA has defined six classes varying from approximately 30 m of sand through to approximately 15 m of regolith consisting of clays and silts overlying sand. In contrast, the site classes in Perth include classes containing inter-bedded limestone as well as other significantly thicker classes. One possible approach to making this information more useful would be to reclassify the regolith in these regions according to the descriptions within the anticipated update of AS 1170.4 .

In addition to the classification of regolith, GA has calculated period dependant amplification factors for most of the regolith classes. These amplification factors have been created for a range of earthquake scenarios and have incorporated some of the variability inherent in nature. The rock motions used for this work are generally different to the rock spectra prescribed by the Australian Standard. Nonetheless, it should still be possible to use the derived amplification factors with the Standard's rock spectra.

#### **4 A FUTURE DATA BASE**

As previously discussed regolith data has been collated for a number of urban areas. Data can be supplied on the total regolith depth, overall shear wave velocities, and the low amplitude site periods for specific areas. In applying this information to the current procedures in AS 1170.4 (Standards Australia 1993) total regolith depth appears immediately useful. Table 2.4(a) uses depths of both 20 m and 30 m to assign the site factor S. Shear wave velocities are also applicable but care needs to be exercised where a layer or regolith sub-thickness is considered in the standard rather than the total profile. Proposed changes to AS 1170.4 (Standards New Zealand 2003) in some instances utilise velocity values for the top 30m of regolith and elsewhere the values for individual soft layers. Low amplitude site period data measured in the field may also be useful. Periods can be adjusted for the increases in damping and decreases in velocity associated with higher shear strains. These adjusted periods can then be compared to the corner periods implied by the current design standard spectra using an approach similar to that proposed by Lam and Wilson (1999). Period adjustments will not be necessary with proposed changes to the current standard as the regolith classification will utilise the low response amplitude natural period as measured in the field by GA.



Ease of accessibility is vital if this regolith information to be used by Australia's engineering community. The future database may best be web based ([www.ga.gov.au](http://www.ga.gov.au)) with downloadable information presented spatially in 3D space. The latitude/longitude of the region could form the horizontal plane of the image with the parameter information presented as the vertical elevation of the surface formed. Alternatively a fully topographical surface could be presented using the elevation above sea level and the parameter presented could be represented by gradations in colour. An ability to interrogate the surface by mouse click at the position of the proposed structure may also be useful. Supplementary data such as borehole logs or cone penetrometer soundings could potentially be presented which are accessed by mouse clicking on an icon on the surface.

Establishing a web based information site of the form described with the programmed features proposed will take time and resources. Information could be made available on a priority basis (e.g. regolith depths and average regolith shear wave velocities first), with supplementary data (site periods and available borelogs) provided later. The database would grow as historical work is incorporated and as ongoing work is completed. Site classification in terms of the current earthquake code may be a future inclusion through a reworking of data. The research associated with the DMAP initiative will potentially furnish significant quantities of additional site information as part of a comprehensive coverage of all major urban areas in the country. The envisaged information facility could be expected to widen in its Australian coverage and improve in its range and quality of site data with time.

## 5 CONCLUSION

Earthquake design of buildings aims to detail a structure that will possess resistant characteristics compatible with the site earthquake hazard. Knowledge of site conditions is an essential component of this process. Historical earthquake damage has highlighted the increased demands imposed on structures where they are situated on deep regolith deposits. Spectral displacement demands are particularly accentuated which can have a significant impact upon some forms of Australian.

Over several years GA has accumulated a large body of regolith data for several urban areas in Australia. Furthermore, this database is expected to grow with GA involvement in the DMAP initiative that in part seeks to define seismic risk nationwide. Design standard procedures suggest that the types of information collected could serve as a valuable resource to practitioners to supplement soil investigation information obtained for each project. In making this information freely available GA would wish it to be in a form that best serves the practitioners that will use it. Moreover, for GA to justify the use of its resources to develop this database there would need to be significant demand from the Australian engineering community. This feedback would help ensure that the significant deployment of resources will bring the greatest benefit in minimising earthquake risk. GA welcomes comment from the profession as to how this can be best achieved.

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# EARTHQUAKE RISK IN AUSTRALIA AND INSURANCE

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## INVITED SPEAKER

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Dr George Walker is Senior Risk Analyst at Aon Re Australia, the Australian reinsurance subsidiary of the global Aon insurance and reinsurance broking group. He began his involvement in earthquake engineering over 40 years ago in 1961 when he commenced a PhD in earthquake engineering at the University of Auckland. Prior to his present position he spent over 20 years as an academic and five years in government research. He has been in his current position for nine years, his primary role being the development and application of catastrophe loss models in the insurance industry. In recent years he has become a leading specialist on the design of disaster insurance schemes and pricing of catastrophe insurance risk. His primary area of work is Australia, New Zealand and Southeast Asia.

### ABSTRACT

Insurance against earthquake risk is primarily based on the risk as perceived by the insurance industry. Property in Australia is better insured against financial losses arising from earthquake damage than property in most other countries of the world. How this has come about will be outlined. A key factor was that prior to the Meckering earthquake the earthquake risk was perceived as negligible. This situation changed dramatically after the Newcastle earthquake. Nowadays earthquake insurance risk analysis dominates the determination of the amount of reinsurance which Australian companies need to buy to protect them against a catastrophic loss from property damage from any cause. This analysis is based on the use of models that are dependent on the perceived risk as estimated by experts. The paper will describe how this perceived risk translates into insurance premiums, and its relative importance in this respect relative to other hazards.

Currently there are large differences in the perceived risk as estimated by different models representing the opinions of different groups of experts. This is a matter of considerable concern to the insurance industry. One solution to this problem would be the development of a national consensus on the best assumptions to use in relation to modelling the occurrence of earthquakes, attenuation, and vulnerability of property to insured loss from earthquakes. An outline of how this could be achieved through a national cooperative effort involving researchers, the insurance industry and government agencies concerned with emergency management and building regulations who have similar needs, will be proposed.

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