FIELD INVESTIGATION ON THE EFFECT OF BLAST VIBRATIONS ON RESIDENTIAL STRUCTURES

EMAD F. GAD[1] AND JOHN L. WILSON[2]

SWINBURNE UNIVERSITY OF TECHNOLOGY[1] AND UNIVERSITY OF MELBOURNE[1] [2]

AUTHORS:

Emad Gad BE, PhD. Senior Lecturer, School of Engineering and Science, Swinburne University of Technology and Senior Research Fellow, Department of Civil and Environmental Engineering, University of Melbourne.

Emad completed his PhD in 1998 and since then has been researching and consulting in the areas of structural performance of residential construction, vibration analysis and finite element modelling.

John Wilson, BE, MSc, PhD. Associate Professor, Department of Civil and Environmental Engineering, University of Melbourne.

John was with Ove Arup and Partners and SECV before joining the University of Melbourne in 1991. He is actively involved in research and consulting in the earthquake engineering and structural dynamics fields.

ABSTRACT:

Blasting is common in the coal industry to remove rock overburden so that the exposed coal can be mechanically excavated. The ground vibrations and air blast produced by blasting are often felt by residents surrounding the mines. There has been a trend for regulatory authorities, especially those concerned with the environment, to impose low limits on blast vibration levels in response to community pressure, based on human perception and response to vibration. This paper reports the findings of an extensive study on a house which was located adjacent to a coal mine. The house was monitored for over one year and was subjected to ground peak particle velocity (PPV) ranging from 2mm/s to 222mm/s. The house was instrumented with accelerometers to measure its dynamic response from blasting and was also monitored for cracks before and after each blast. Based on this study, ground motion amplification factors as a function of structure height have been established. A simplified methodology has been developed to estimate the ground PPV at which cracking of plasterboard is likely and is presented in this paper.

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1. INTRODUCTION

Blasting is common in the coal industry to remove rock overburden so that the exposed coal can be mechanically excavated. Explosives used in opencut coal mines are loaded into blast holes which have been drilled downwards through the overburden rock. The blast holes are then detonated in sequence and a portion of the energy released is converted to wave energy with compression (P) waves, shear (S) waves and surface Rayleigh (R) waves transmitted in all directions from the blast source.

The ground vibrations and air blast produced by blasting are often felt by residents surrounding the mines. There has been a trend for regulatory authorities, especially those concerned with the environment, to impose low limits on blast vibration levels in response to community pressure, based on human perception and response to vibration. This has a particular impact on the mining industry as mines move closer to towns and cities to extract the adjacent natural resources from the ground.

This paper reports findings from an on-going study investigating the effects of blast vibrations on residential structures. The research team includes Terrock Consulting Engineers, Newcastle University and University of Melbourne. As part of this investigation several houses were monitored in the Hunter Valley region in New South Wales (NSW) where a number of open-cut coal mines are operating close to townships. The houses monitored were selected to represent different types of construction, vintage and distance from mines. This paper focuses on results from field monitoring of a specific test house which was monitored from December 1999 to January 2001.

2. THE TEST HOUSE

The house is adjacent to the Rix's Creek open-cut mine adjacent to the township of Singleton in NSW. It is of conventional brick veneer construction, with a timber frame, 10mm plasterboard lining for the walls and ceiling, tiled roof and wooden floor boards. The house was constructed in the early 1970's. The brickwork is supported by strip footings whilst the wooden floor is supported by a series of floor joists, bearers and masonry piers. A floor plan of the house is shown in Figure 1.

The condition of the house at the beginning of the monitoring appeared reasonable with little evidence of deterioration from environmental effects. The plasterboard and brickwork had a number of cracks ranging in size from fine (<1mm) to noticeable but easily filled (<5mm), scattered throughout the house.

Compressive and bond wrench tests were conducted on the brickwork by Newcastle University. The characteristic compressive strength was found to be 19.1MPa. The mean flexural tensile strength of the masonry was 0.25 MPa with a large CV of 66% translating to an unrealistically low characteristic tensile strength of 0.03MPa. The bond strength is considered low but not necessarily atypical for domestic construction where the standard of workmanship is highly variable.

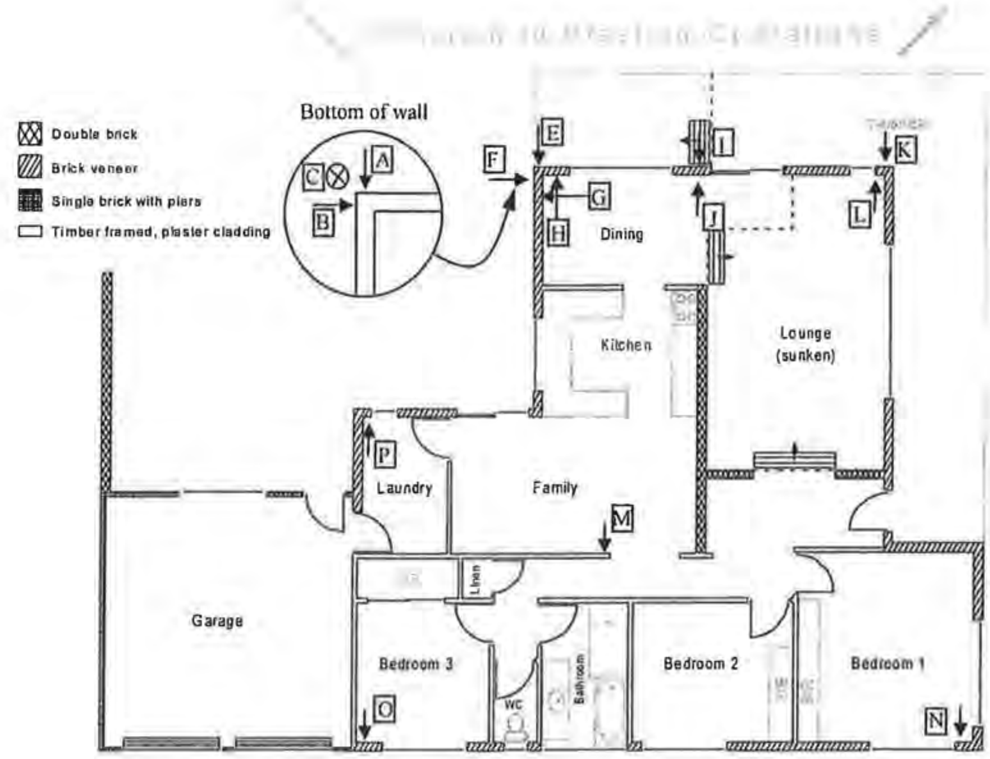

Figure 1: Floor plan of the Rix's Creek house (arrows labelled A to P indicate locations of accelerometers mounted on the house to measure the dynamic response).

Soil and geotechnical investigations were carried out by Newcastle University. Based on the soil investigation it was found that the site can be classified as Class M, or moderately reactive in accordance with AS2870 (1996). The foundation of the structure was examined from an excavation which revealed a strip footing of between 370 and 400mm.

3. MONITORING

3.1 Blasts

During the monitoring period (December 1999 to January 2001) the test house experienced some 44 blasts with charge masses varying from 50kg to 1300kg at distances between 50 and 1000 metres. The peak particle velocity (PPV) measured on the ground adjacent to the site varied between 1.5mm/sec and 220mm/sec.

3.2 Crack and Level Surveys

The crack lengths in all the rooms were monitored, in addition, the width of some cracks were monitored using Demac gauges. All cracks were monitored before and after each blast and any changes noted.

Level surveys were undertaken with the relative levels of the house foundations were measured twice some five months apart. The difference in levels between the two surveys indicated a maximum settlement of 12 mm and a maximum heave of 10mm during the five months period. These measured movements were consistent with the predictions from the geotechnical investigation.

3.3 Structural Monitoring

Fifteen accelerometers were used to measure vibrations in different locations in the house as shown in Figure 1. Three accelerometers were located at ground level (A, B, C) to measure the two horizontal and one vertical components of acceleration. The remaining twelve accelerometers were orientated in the horizontal direction at ceiling level (approximately 2.4m above floor level) with four on the external brick veneer walls (E, F, I, K) and eight on the internal plasterboard (G, H, J, L, P, M, N, O).

4. STRUCTURAL RESPONSE

The overall damage in a residential structure due to blasting is directly correlated with the in-plane distortion of the walls between the ceiling and floor. The in-plane distortion is often measured in terms of the drift ratio (γ) which is defined by the horizontal displacement (Δ_1) of the wall at the ceiling level divided by the wall height (H). The ceiling displacement can be estimated from the ground peak component velocity (V_g), the amplification (λ) of the velocity between the ground and ceiling and the dominant frequency (f) of the structure as shown in Eq [1].

$$\gamma = \frac{\Delta_1}{H} \qquad [1a]$$

$$\Delta_1 = \frac{V_g}{2\pi f}\lambda \qquad [1b]$$

$$\gamma = \frac{V_g}{2\pi f}\frac{\lambda}{H} \qquad [1c]$$

The drift ratio (γ) provides an estimate of the gross shear strain in a wall. However, damage occurs when the principle tensile strain of the material is exceeded and hence rupture occurs. The average principal tensile strain (ε) can be simply estimated from the gross shear strain using basic mechanics of solids principles as follows:

$$\varepsilon = 0.5\gamma \qquad [2]$$

The bases for selecting the amplification and frequency values are described in the following sections.

4.1 Amplification Effects

The ratios of the peak component velocity (V_1) at ceiling level to ground level were calculated to estimate the likely vibration amplification effects with height in the structure. The ratios were calculated for both the in-plane and out-of-plane directions and for both the frame and brick veneer walls. The in-plane measurements are of vital importance from damage level perspective, whilst the out-of-plane records are less structurally significant but do contribute to the overall vibration and noise perception of the occupants. This paper reports the in-plane results only.

The resulting amplification values varied significantly depending on the level of ground vibration, Figure 2 plots the in-plane amplification for the framed walls and brick veneer walls versus the ground PPV measured adjacent to the house. It should be noted that the PPV is always greater than the V_g, with the ratio of PPV to V_g typically in the range of 1-2. Most blast related regulations worldwide, including Australia, are based on PPV rather than V_g, and hence the values plotted are conservative.

An upper bound envelope has been fitted to the data, so that an approximate and conservative estimate of the amplification effects can be obtained. The amplification envelope can be described by a step function as follows:

$\lambda = 4.0$ for PPV $\leq$ 5 mm/sec [3a]

$\lambda = 2.0$ for 5< PPV $\leq$100 mm/sec [3b]

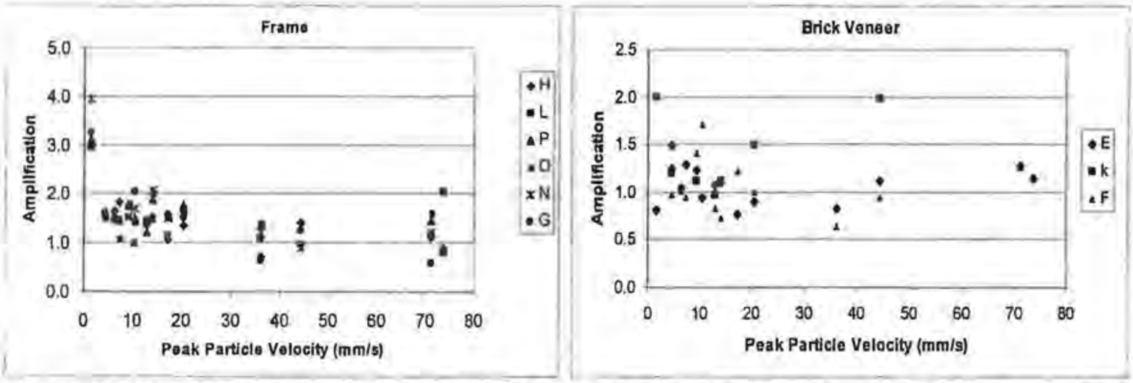

Figure 2: Amplification of velocity in the in-plane direction for the frame and brick veneer.

4.2 Dominant Frequency

For the measuring locations shown in Figure 1, the acceleration records were integrated and double integrated to obtain the peak velocity (V_1) and peak displacement at the ceiling level (Δ_1). The dominant frequency (f) was calculated assuming a simple single degree of freedom response as follows:

$$f = \frac{V_1}{2\pi\Delta_1} \qquad [4]$$

It is noted that this is a major simplification, however, the method enables a realistic estimate of Δ_1 and hence the drift to be made. The dominant frequency tends to vary with V_g and hence PPV and was found to be in the range of 6-10Hz. A lower bound frequency figure of 6Hz can be used to estimate conservative values for the displacement and drift.

4.3 Quantification of Damage

The conservative values for the amplification (λ) and frequency (f) developed above have been used to estimate the ceiling displacements (Δ_1) and average principal tensile strain (ε) for a single storey house (ceiling height of 2.4m) subject to different levels of ground vibration expressed in terms of PPV as shown in Table 1.

Table 1: Estimation of displacements and average principal strains in a single storey house subjected to various levels of blast vibrations.

PPV (mm/s)	λ	F (Hz)	Δ_1 (mm)	γ	$\mu\varepsilon$
5	4	6	0.5	1/4500	111
10	2	6	0.5	1/4500	111
25	2	6	1.3	1/1800	276
50	2	6	2.7	1/900	553
75	2	6	4.0	1/600	829
100	2	6	5.3	1/450	1105

Most codes of practice around the world recommend maximum drift ratio in the order of 1/300 to 1/500 at the serviceability limit state to prevent damage from wind and earthquake loading. These drift ratios would correspond to blast vibrations in the order of PPV=100 mm/s.

The principal tensile failure strain associated with plasterboard is in the order of 1000$\mu\varepsilon$ which corresponds to ground vibration in the order of 100mm/sec. For masonry construction, such correlations are more difficult to establish due to the isotropic properties of this composite material (bricks and mortar bed joints). The tensile strength of masonry is always quoted in terms of the tensile stress needed to rupture the bond between the bricks and the mortar (the associated tensile strain with rupture is typically in the order of 100-300$\mu\varepsilon$). In contrast, a blast loading which induces racking displacements in a masonry wall would result in shear stress and strains (and not principle tensile stresses) at the bricks and mortar interface. The shear strength at this interface is typically stronger than the corresponding tensile strength.

The strain levels presented in Table 2 are all dynamic strains and must be added to any residual or existing strains in the structure. The residual strains could arise from a number of sources including foundation and thermal movements. In order to establish an acceptable level of dynamic strain, an understating of the level of residual strains in the structure is required. For example, if it is estimated that existing strains are in the order of 90% of the material rupture strain, then the dynamic strain would need to be limited to 10% of the rupture strain to prevent cracking. This would translate to a conservative limiting PPV of 10mm/sec for plasterboard assuming that the residual strains are in the order of 900$\mu\varepsilon$. In the Rix's Creek house, no new damage from blasting was observed for PPV less than 75mm/sec. This suggests that the residual strain in this house were relatively small and in the order of 100$\mu\varepsilon$ or 10% of the plasterboard rupture strain.

It is recognised that for fatigue to be an issue, the dynamic strain need to be greater than some limiting threshold value. The dynamic strain associated with the normal PPV limits of blasting are generally small and less than the threshold value to cause fatigue cracking.

5. CONCLUSIONS

This paper describes part of an on-going investigation into the effects of blast vibrations on houses. The results from a typical single storey brick veneer house, which was monitored for over a year, have been presented. During the monitoring period, the house was subjected to 44 blasts with ground peak particle velocity (PPV) ranging from 1.5mm/s to 220mm/s. Crack surveys were performed before and after each blast. No additional damage due to blasting was observed for blasts with PPV below 75mm/s.

Based on the measured dynamic response of the house, it was found that the amplification factor of the ground PPV at ceiling level was a maximum of four for the in-plane walls (racking motion) and for PPV of 5mm/s and less. For higher ground PPV, it was found that the amplification factor was a maximum of 2.

Using a simple degree of freedom analogy, and adopting the obtained conservative amplification factors with conservative estimates of the natural frequency of the house, the maximum strains in the plasterboard in the house were estimated for different ground PPV values. These results suggest that the plasterboard is unlikely to crack at low ground PPV levels (less than 10mm/s) unless the residua strains are extremely high (more than 90% of the failure strains).

6. ACKNOWLEDGEMENTS

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AN INNOVATIVE APPROACH TO THE SEISMIC ASSESSMENT OF NON-STRUCTURAL COMPONENTS IN BUILDINGS

NELSON LAM AND EMAD GAD
UNIVERSITY OF MELBOURNE

AUTHORS:

Nelson Lam is senior lecturer at University of Melbourne.

Emad Gad is senior lecturer at University of Melbourne and at Swinburne University of Technology

ABSTRACT:

The analysis and design of structural systems to comply with life safety requirements have been the main thrust of earthquake engineering research. As community expectations increase, there has been increasing attention to address potential economic losses resulted from earthquakes. The economical impact associated with damage to non-structural building components in recent earthquake events has been found to exceed those associated with structural damage. In this paper, the economic significance of earthquake induced non-structural damage and the limitations of current design methodology is discussed. An alternative assessment approach based on considering acceleration, velocity and displacement is introduced and explained by relating to fundamental principles. The merit of this approach in making simple and realistic assessments for regions of low and moderate seismic activity is demonstrated. Whilst the new methodology is yet to be fully developed, the innovative concept presented is original and has never been published previously.

Paper No. 10

1. INTRODUCTION

Research into the seismic performance of non-structural (NS) components is guided by two principal objectives. The first objective is to reduce potential casualties and injuries resulted directly from the failure of NS components. The second objective is to mitigate the ensuing economic costs associated with (i) loss of function both during and after the event, (ii) repair and replacement work and (iii) collateral damage.

NS components is a generic term which encompasses a diversity of building items which can be grouped into the following categories: (a) interior components including partitions and ceilings; (b) exterior components such as building facades; and (c) building services components. Refer Figure 1.

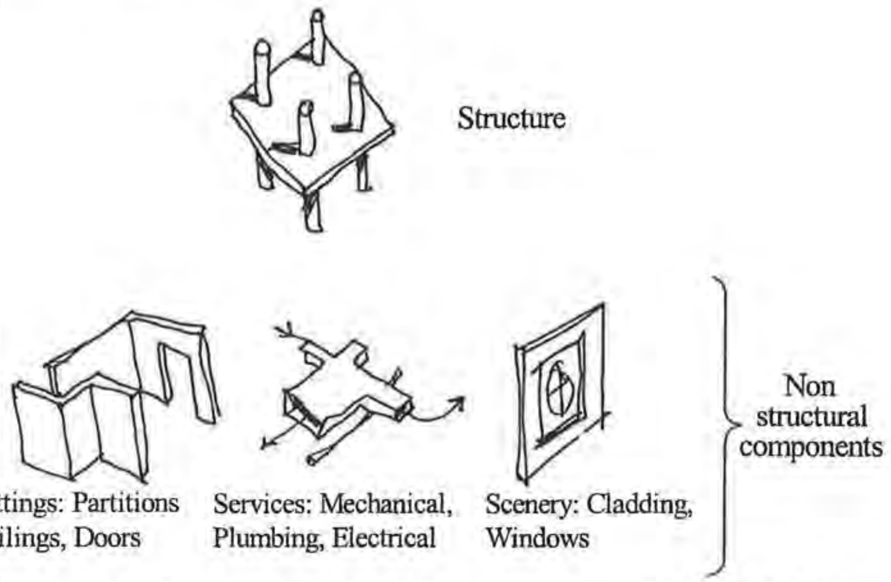

Figure 1 - Structural and non-structural building components (Hira *et al*, 2002)

The percentage breakdown of individual NS items in a typical Australian office building is listed in Table 1 to demonstrate the significance of NS components in economic terms (Rawlinson, 2000).

Table 1. Percentage breakdown of cost items for office buildings in Australia.

Element **storey nos:**	7-20	21-35	36-50
Builder's preliminaries (plant, scaffolding, insurance, etc.)	17.5	20.0	22.1
Substructure (excavation, foundations, service tunnels, etc.)	1.4	1.1	0.9
Superstructure			
- Columns, upper floors, staircases & roof	12.7	11.0	10.2
- External walls and windows (facades)	14.1	18.1	17.7
- Internal walls and doors	4.1	4.7	4.4
Finishes (wall, floor and ceiling)	7.9	6.9	6.7
Fitting (fitments and special equipment)	0.8	0.6	0.6
Services (plumbing, mechanical, fire, electrical, transportation)	39.0	35.1	34.9
Contingency	2.5	2.5	2.5

The combined value for facades and services is shown in Table 1 to account for over 50% of the total construction cost. The demonstrated significance of NS components is consistent with the result of a post-disaster survey for the 1971 San Fernando earthquake in California involving 355 high-rise buildings. The survey showed that 79% of the damage costs in dollar terms was non-structural (Arnold, 1987). Direct replacement costs of NS components can be only a small part of the total costs which account for the loss of function and lost of inventory. Such indirect costs can be two to three times the cost of replacing the damaged structure (Phan and Taylor, 1996). NS components in low-moderate seismic regions deserves engineering attention even in situations where the risk of structural damage seem very low. For example, buildings responding elastically can produce highly periodic floor motions which can be very damaging to certain NS components even at very low intensity.

In regions of low and moderate seismic activities such as Australia, most of these NS items have not been fully engineered for satisfactory seismic performance. When seismic actions are calculated, adequate structural engineering input is not always provided to check the installations for compliance, particularly for items installed during the service life of the building (Beattie, 2001). The complex interactions between certain NS components with the structural system which might have high performance implications are often not considered. For example, heavyweight cladding systems have been shown to contribute to additional lateral stiffness and might also alter the dynamic properties of the structure (Henry & Roll, 1986). Partitions made of plasterboard-clad steel-framed walls have also been found to contribute to 10-20% of the lateral stiffness of the structure (Freeman, 1977). In addition, such partitions add significantly to damping, particularly for low intensity responses. Overall, there is generally little control over the seismic protection of NS components and building contents in Australia.

The limitations of existing methodologies developed for assessing NS components are discussed in Section 2 and the alternative innovative methodology introduced in Section 3. Due to space limitations, the rest of the paper only addresses components that can be considered as an isolated component excited by the floor motions. Thus, the effect of structural deformation is not considered.

2. REVIEW OF EXISTING ASSESSMENT METHODS

The modelling methodology adopted in contemporary seismic loading codes (e.g. AS1170.4, 1993) is based on estimating the transmission and amplification of the peak horizontal acceleration from the ground to the individual floor, and eventually to the centre of inertia of the component, in order that the seismic inertia force can be calculated (Lam, 1998). Interestingly, a comparative study by Phan and Taylor (1996) revealed up to five-fold discrepancies in the recommended amplification factors from several major code of practice, suggesting significant uncertainties in the estimated amplification factors. A recent study by Rodriguez (2002) made recommendations for the peak floor accelerations based on rigorous non-linear dynamic analysis of building structures ranging between three and twelve stories. An independent study by Yao (2001) developed floor spectra from floor accelerograms recorded in 19 buildings during the 1999 Chi-Chi earthquake (Taiwan). These recent studies also revealed considerable uncertainties in the prediction of peak accelerations. Importantly, none of the studies addresses the mechanism of damage leading to failure of a component.

Isolated studies have identified the direct link between floor velocity (and displacement) and the overturning vulnerability of uniform objects (Ishiyama, 1984; Clark, 1993). However, the absence of reliable information on these floor motion parameters has limited further development of such a model. Full-scale dynamic testing of physical models of suspended ceiling modules (Yao, 2001), fire sprinklers and air-conditioning ducts (Beattie, 2001) have also been reported. Observations from these tests provide valuable insights into the vulnerability of the tested components, but further research efforts are required to generalize these test results for practical applications. A generic probabilistic procedure for quantifying damage cost has been proposed recently by Porter (2000). Whilst the philosophy is sound, its implementation is only possible when reliable vulnerability models are available.

3. PROPOSED ASSESSMENT METHOD

The assessment method proposed in this paper is to be developed from results obtained by a combination of dynamic testing and finite element analysis of calibrated computer models. The observed non-linear behaviour is approximated by a linearised system possessing 2-5% critical damping (these limits are currently reviewed based on calibration with experimental observations). The recent review by Miranda (2002) shows that displacement estimated by the linearisation methodology applied to simple building models has been found to be in reasonable agreement with non-linear analysis results in terms of ensemble average, provided the effective period and damping of the linearised system has been suitably selected. A recent achievement in modelling the out-of-plane overturning behaviour of masonry walls based on linearisation is described in Doherty (2002). The linearisation of NS component behaviour which is characterized by abrupt change in the "dynamic stiffness" represents a new challenge.

Linearisation enables the seismic response of the component to be approximated by an elastic floor spectrum. The velocity floor spectrum in the tri-partite logarithmic form is bounded by 3 straight lines representing the peak response acceleration (A_o), velocity (V_o) and displacement (D_o) responses, respectively, as shown in Figure 2.

Taking the suspended ceiling as an example (refer 2nd row of Table 2), the initiation of damage to the ceiling is evidently represented by the force required to separate the edge of the ceiling from the wall supports, and this force is bounded by the peak response acceleration (A_o). As the ceiling becomes disengaged from the initial restraints and pounds against the wall and neighbouring objects, the damage associated with the pounding is best represented by the peak response spectral velocity (V_o). As the ceiling is subject to significant drift, the deformation of the fire-sprinklers and air ducts, etc, (which are in contact with the ceiling) is related directly to the peak response spectral displacement (D_o). This modelling approach allows the progressive deterioration of the component to be tracked.

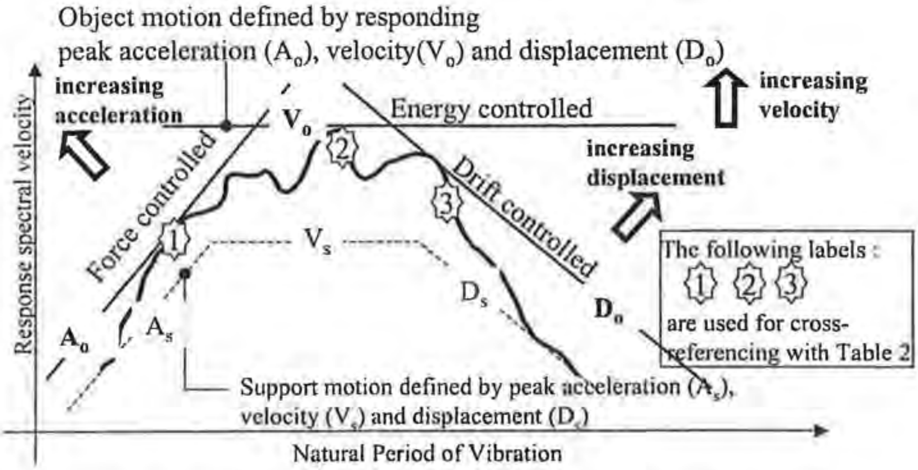

Fig.2 Proposed Generalised Floor Spectrum Model

Table 2: Floor motion parameters and component vulnerability

Basic floor motion parameters	A_o	V_o	D_o
Ceiling suspended components	pull-out of angle-section *Acceleration (force) initiates "pendulum" action*	$D \le D_o$ ceiling panels dislodged by shocks during pounding *Velocity controls damage caused by pounding*	straining of HVAC ducts and sprinkler heads *Displacement controls damage to drift-sensitive items (e.g. fire sprinklers)*
Floor mounted components	*Acceleration (force) initiates yielding of base restraints*	$D \le D_o$ *Velocity controls damage to restraints*	*Displacement of centre of gravity controls overturning (when freed from restraints)*
Components susceptible to sliding	*Acceleration (force) initiates sliding*	*Velocity controls damage to colliding objects*	*Displacement controls objects sliding off their support*

The damage mechanisms of floor mounted components and unrestrained components are similarly linked to the A_o-V_o-D_o parameters that can be articulated to operate a fast-track "scanning" procedure to determine the component vulnerability. As shown in Table 2 (3rd row), floor mounted components are deemed safe from overturning if either D_o is insufficient to move its centre of gravity (c.g.) far enough to its edge or if A_o is insufficient to result in static instability. For components located at a lower level of a building, or on a rock site where the shaking is characterised by high frequency and low displacement (low D_o), the displacement approach is clearly preferred. For components located at the upper levels of a tall building where the shaking is characterised by low frequency and low acceleration (low A_o) the force approach is preferred instead. In the latter situation, overturning is deemed unlikely since damage to the restraints cannot be initiated.

The A_o-V_o-D_o parameters can also be combined to obtain refined estimates in situations where initial scanning shows non-compliance. For example, direct displacement based (DB) assessment is particularly convenient in checking drift related damage, as drift is upper bounded by D_o irrespective of the component mass, natural period and the complex interaction with other components possessing incompatible stiffness properties. Should the initial DB assessment based on D_o show non-compliance, drift may be re-calculated based on V_o using energy principles. Should the first two assessments show non-compliance, drift may be re-calculated again based on A_o using conventional principles of force and stiffness. Interestingly, the component is deemed satisfactory should any one of the three assessments show compliance. This is justified by the fact that each of the A_o-V_o-D_o lines making up the tri-linear envelope represents an upper bound estimate of the seismic demand (see Figure 2).

Refined estimates for the peak response velocity and acceleration could be obtained similarly using the "3-steps" approach described above. The new modelling framework is in significant contrast with the contemporary acceleration based approach which often requires the natural period of both the component and the building to be determined, or else the "A_o envelope" would be overly conservative. Flexibility and versatility is clearly lacking in such an approach.

4. CONCLUSIONS

This paper highlights the significance of non-structural components under seismic actions. Traditionally, the vulnerability of non-structural components has been higher than that associated with the structural elements. In order to minimise the damage to non-structural components and reduce the potential cost of earthquakes, an integrated system approach should be adopted to ensure compatibility between the non-structural and structural components in the building.

The presented methodology employing a three-step scanning procedure is based on representing seismic demand in terms of acceleration, velocity and displacement. The concept of combining acceleration, velocity and displacement demand to predict the seismic response behaviour of structures possessing different natural periods of vibration has been employed for a long time. The use of response spectra in the tri-partite format is also standard in engineering seismology. However, the articulation of these basic principles for rapid assessment and for tracking the deterioration of non-structural components is new and innovative, and has distinct advantages over conventional methods. Whilst the new methodology is yet to be fully developed, the presented innovation in the context of modelling NS component behaviour is original and has never been published previously.

5. ACKNOWLEDGEMENTS

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EARTHQUAKE RISK ANALYSIS IN NEWCASTLE AND LAKE MACQUARIE

GLEN FULFORD[1] TREVOR JONES[1] MARK EDWARDS[2] DAVID ROBINSON[1] AND JOHN SCHNEIDER[1]
GEOSCIENCE AUSTRALIA[1] AND UNIVERSITY OF MELBOURNE[2]

AUTHORS:

Glenn Fulford (PhD) is currently a software engineer at Geoscience Australia. Glenn has been a lecturer in applied mathematics at the Australian Defence Force Academy in Canberra and at La Trobe University and has worked as a research associate for AgResearch in New Zealand.

Trevor Jones (BSc., MSc., Dip.Tch.) Trevor is the Leader of Geoscience Australia's Cities Project. His research interests are primarily in earthquake risk. He is currently active in assessing the natural hazard risks to Perth.

Mark Edwards (B.E., M.E.) Mark is a Research Fellow at the University of Melbourne. He has 14 years industry experience with an on-going interest in earthquake engineering. He is currently researching improved methods for predicting maximum earthquake induced building displacements.

David Robinson (BSc. Hons) is a geophysicist with the Urban Risk Research Group at Geoscience Australia. He has also worked at the Woods Hole Oceanographic Institute and the CRC for Soil and Land Management. David has majors in mathematics and geophysics from Flinders University of South Australia.

John Schneider (PhD) is the research Group Leader for Geohazards and Risk at Geoscience Australia. John's primary research interests are in the development of natural hazards risk assessment methods for application to urban centres, with a particular emphasis on earthquake hazards.

ABSTRACT:

This paper overviews the earthquake risk methodology for building damage, and economic losses, in the Newcastle and Lake Macquarie region. A comprehensive field survey in the study region was conducted to document the vulnerability characteristics of a sample of more than 6,300 buildings. Australian damage models based on the capacity spectrum method were prepared and a HAZUS economic loss model was used. Damage, economic loss and casualties were calculated in a stratified Monte Carlo simulation of 1,200 events. Ground motion response spectra calculated for each earthquake-site pair were used to estimate building damage for all scenarios. Results show that the annualised loss for the study region is of the order of 0.04%, or around $12 million per year. The 1989 Newcastle earthquake had an economic impact with a return period of the order of 1,500 years. Differences in the regolith distribution and thickness cause strong local variations in the risk.

1 INTRODUCTION

Geoscience Australia has developed a probabilistic risk modelling software package based around a stratified Monte Carlo simulation and written in `Matlab`. This paper describes the vulnerability module of the software and its application to the Newcastle/Lake Macquarie region (Fulford *et al.*, 2002). Companion papers, Robinson *et al.* (2002); Dhu *et al.* (2002), describe the regional earthquake hazard module and the module for amplification of seismic energy by regolith. In these modules, a group of earthquake events are assigned random magnitudes and positions for which the hazard module produces a random ground-shaking (see Figure 1). The ground-shaking is then input to a building damage module which uses a random building capacity curve (representing variability in building response) and a random classification of the damage state of the building. The output is used to calculate an economic loss for each building corresponding to each random earthquake event.

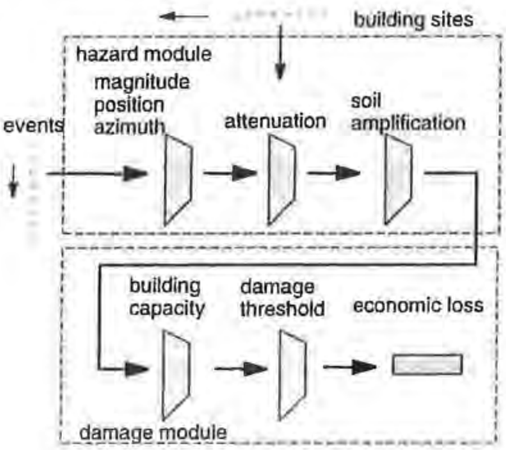


Figure 1: Illustration of the Monte Carlo simulation for earthquake risk showing the various random components.

2 VULNERABILITY MODEL DESCRIPTION

Building damage is calculated according to the Capacity Spectrum Method (Kircher, 1997; National Institute of Building Science, 1999; Freeman, 1998). It is based upon finding the intersection of a response curve (associated with a response spectrum which is rescaled for appropriate building damping) with a building capacity curve (commonly called a push-over curve). The intersection gives the peak spectral displacement and acceleration acting on the structures. Each different building construction type has a different capacity curve that depends on certain parameters. These parameters, together with the damping parameters, are:

C_s = design strength coefficient (fraction of the building weight),

T_e = natural elastic building period (seconds),

α_1 = fraction of building weight participating in the first mode,

α_2 = fraction of the effective building height to building displacement,

γ = over-strength factor—yield to design strength ratio,

λ = over-strength factor—ultimate to yield strength ratio,

μ = ductility factor,

$\kappa_{1,2,3}$ = hysteretic damping for short, medium and long duration earthquakes, and

B_e = elastic damping ratio.

In February 2001, a workshop was held at the University of Melbourne to assess the applicability of the HAZUS building capacity parameters to Australian building construction types. It was recommended that some HAZUS construction types be divided into subtypes depending on wall and roof materials and their building parameters adjusted to better suit Australian construction standards. Subsequently, Geoscience Australia contracted the University of Melbourne to further look at the building capacity parameter values for timber frame buildings (W1 types) and concrete moment frame buildings (C1 types) and to provide new values where necessary (Edwards *et al.*, 2002). These improved values are listed in Table 1 together with values for the unreinforced masonry types (URM types) suggested in the workshop.

Table 1: Capacity spectrum building parameters and damping parameters for Australian building types: W1 (timber frame, wall type and roof type); C1 (concrete moment frame, low-rise or mid-rise or high-rise, soft or non-soft storey); URM (unreinforced masonry, low-rise or mid-rise, roof type). Units are dimensionless, except where indicated.

Type	C_s	h feet	T secs	α_1	α_2	γ	λ	μ	κ_1 %	κ_2 %	κ_3 %	B_e %
W1MEAN	0.077	13	0.275	0.9	0.7	1.75	2	7	0.1	0.1	0.1	8
W1BVTILE	0.063	13	0.32	0.9	0.7	1.75	2	7	0.1	0.1	0.1	8
W1BVMETAL	0.082	13	0.28	0.9	0.7	1.75	2	7	0.1	0.1	0.1	8
W1TIMBTILE	0.069	13	0.3	0.9	0.7	1.75	2	7	0.1	0.1	0.1	8
W1TIMBMETAL	0.094	13	0.26	0.9	0.7	1.75	2	7	0.1	0.1	0.1	8
C1LMEAN	0.2	20	0.45	0.975	0.795	1.5	1.38	3.75	12	9	9	7
C1LSOFT	0.2	20	0.45	1	0.79	1.5	1.25	3.5	12	9	9	7
C1LNOSOFT	0.2	20	0.45	0.95	0.8	1.5	1.5	4	12	9	9	7
C1MMEAN	0.1	50	0.85	0.95	0.6	1.5	1.38	2.5	10.5	8	8	7
C1MSOFT	0.1	50	0.85	1	0.55	1.5	1.25	2	10.5	8	8	7
C1MNOSOFT	0.1	50	0.85	0.9	0.65	1.5	1.5	3	10.5	8	8	7
C1HMEAN	0.05	120	1.6	0.925	0.55	1.5	1.38	2	9.5	7	7	7
C1HSOFT	0.05	120	1.6	1	0.5	1.5	1.25	1.5	9.5	7	7	7
C1HNOSOFT	0.05	120	1.6	0.85	0.6	1.5	1.5	2.5	9.5	7	7	7
URMLMEAN	0.15	15	0.15	0.75	0.75	1.5	2	2	0.1	0.1	0.1	5
URMLTILE	0.15	15	0.15	0.75	0.75	1.5	2	2	0.1	0.1	0.1	5
URMLMETAL	0.2	15	0.13	0.75	0.75	1.5	2	2	0.1	0.1	0.1	5
URMMMEAN	0.1	35	0.28	0.75	0.75	1.5	2	2	0.1	0.1	0.1	5
URMMTILE	0.1	35	0.28	0.75	0.75	1.5	2	2	0.1	0.1	0.1	5
URMMMETAL	0.15	35	0.23	0.75	0.75	1.5	2	2	0.1	0.1	0.1	5

A key difference in our implementation of the Capacity Spectrum Method to that used by HAZUS is that we do not use design spectra based on only a few building periods. For example, the HAZUS approach only uses periods 0.3 and 1.0 seconds. Our approach has the advantage that the full shape of the response spectrum and all the available information for the soil amplification factors, at all periods, is taken into account, rather than at only two periods.

An inventory of more than 6,300 buildings was used in the simulation, based on a field data acquisition survey in Newcastle and Lake Macquarie, carried out by Geoscience Australia. In the simulation, each building may be in one of five damage states for each of three different types of damage: structural damage, drift sensitive non-structural damage, and acceleration sensitive non-structural damage. Fragility curves give probabilities for a given building to be in a given damage state for each of the three types of damage, depending on the peak displacement or acceleration. The curves are based on a cumulative log-normal distribution with a median threshold value and a given threshold variability value,

The economic loss is calculated according to the HAZUS methodology, (National Institute of Building Science, 1999). Buildings are classified into usage-types, where the costs of replacement (per square metre) are specified. The economic loss for each of the three damage types is a weighted sum of the probability of being in each of the five damage states with the cost for that damage state. The loss due to contents damage is calculated from the loss due to acceleration sensitive non-structural damage. To obtain the total aggregated loss (for a single earthquake) the loss for each building site is multiplied by the appropriate survey factor and summed over all the sites in the building inventory.

3 THE 1989 NEWCASTLE EARTHQUAKE

The 1989 earthquake in Newcastle gives us an opportunity to compare our simulation model with a major historical event. Using the database of 6,300 buildings, the computer model was run for 1000 earthquakes of identical magnitude and location, giving a histogram of losses, (see Figure 2). Multiple simulations were run to sample the variability in attenuation, regolith amplification, and building damage. The results are expressed as a percentage of the total estimated value of buildings and contents in the study region. The median value is approximately 7% with a likely range from 6.5–8%, corresponding to $1.1 billion (1989 dollars). This value is compared with estimates from data in Section 5.

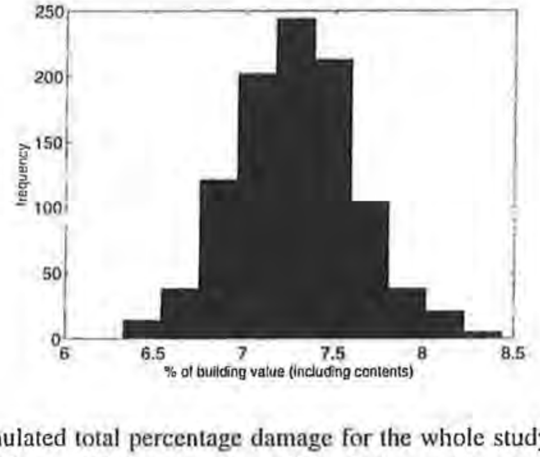
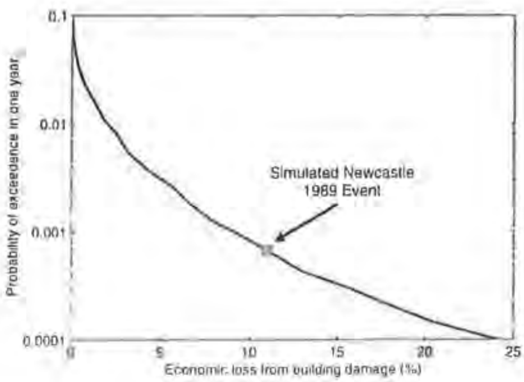


Figure 2: Histogram of simulated total percentage damage for the whole study region for Newcastle and Lake Macquarie, corresponding to 1 000 earthquakes of identical magnitude and location. The percentages are ratios of the simulated economic loss for all buildings and contents to the total value of all buildings and contents for the study region.

4 RISK RESULTS

To investigate future risk for the Newcastle and Lake Macquarie region the computer model was run for 1,200 randomly distributed earthquakes with varying magnitude and location, as discussed in Robinson *et al.* (2002). A curve showing the estimated economic loss due to building damage for the entire study region is given in Figure 3. This curve describes the probability of the study region incurring various minimum levels of economic loss within a single year. Economic loss is expressed as a percentage of the total value of all buildings and their contents in the study region.



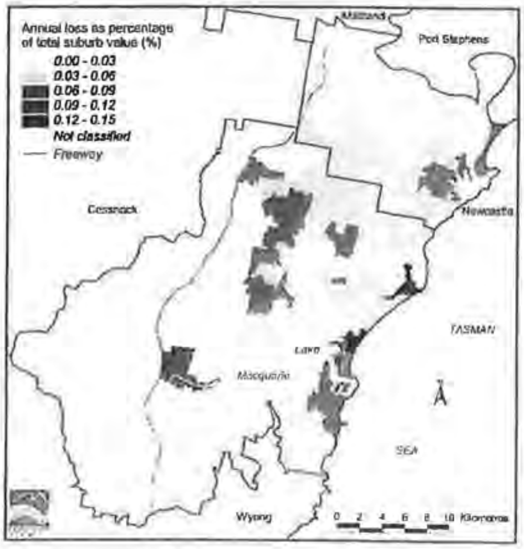


Figure 3: Probable maximum loss curve curve for the Newcastle and Lake Macquarie region (left) together with the spatial distribution of the annualised risk for each suburb (right).

Annualised economic loss can be calculated from an integration of the area under the probable maximum loss curve. Annualised economic loss can be determined for any one or group of building sites, depending on the interests of the stake-holder. The annualised risk for the whole study region in Newcastle was calculated as approximately 0.04% of the total value of buildings and contents.

A measure of the economic risk to a suburb is the annualised loss for the suburb expressed as a percentage of the estimated total value of buildings in that suburb. A map of the spatial distribution of annualised risk for each suburb in the study area is given in Figure 3. This clearly demonstrates that risk varies spatially across the study region. This variation in loss can be partially attributed to differences in building stock across the study region. However, the underlying regolith also affects the annualised losses, with areas that are built on substantial thicknesses of regolith, such as parts of the Newcastle municipality, having noticeably higher annualised risk.

5 DISCUSSION

The simulation model takes account of the natural uncertainties that occur by using random variables with assumed probability distributions. These uncertainties include uncertainties in the earthquake location and magnitude, variation due to the degree of ground shaking observed between two adjacent sites, and natural variation of building response for identical building types at the same location. However, there are also further uncertainties associated with the choice of the probability distributions, along with particular model chosen. For example, the choice of attenuation model used, and the choice of the building capacity method, also contribute to overall uncertainty. Taking account of these may tend to increase the risk estimated by the simulation. Conversely, further research that can improve the models may reduce the estimated risk.

The simulated estimate for economic loss due to building damage from the 1989 earthquake was $1.1 billion (1989 dollars) or 7% of total building value including contents. This compares well to the estimates by Melchers (1990) of $0.9 billion and that obtained from NRMA data (9% of total building value including contents).

There is a need to perform a detailed sensitivity analysis on the model, to determine whether changes in inputs such as earthquake magnitude and frequency parameters, attenuation parameters or building damage parameters contribute the most to changes in economic loss estimates. This analysis can be used to prioritise future research.

This simulation model is work-in-progress, so some caution should be exercised in using the results discussed in this paper. Further feedback from the engineering community is necessary to further refine the model so that it can become a valuable planning tool for earthquake risk mitigation.

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NEW ESTIMATES FOR A FREQUENCY DEPENDENT SEISMIC QUALITY FACTOR $Q(F)$ FOR SOUTHEASTERN AUSTRALIA

TREVOR ALLEN(1), GARY GIBSON(2), DAVID LOVE(3) AND JIM CULL(1)
MONASH UNIVERSITY(1), SEISMOLOGY RESEARCH CENTRE(2)
AND PRIMARY INDUSTRIES AND RESOURCES SOUTH AUSTRALIA(3)

AUTHORS:

Trevor Allen is a PhD candidate within the School of Geosciences, Monash University, Melbourne. He is currently investigating seismic attenuation of southeastern Australia and its effects on source parameters for local and regional earthquakes.

Gary Gibson established the Seismology Research Centre in 1976, and has worked there since. He has worked at Phillip Institute of Technology and RMIT University since 1968. His interests lie in observational seismology and its practical applications.

David Love works in the Office of Minerals and Energy Resources, a division of Primary Industries and Resources South Australia, and has managed the seismograph network in South Australia since 1986.

Jim Cull is the Head of the School of Geosciences, Monash University. His interests include the development and application of geophysics involving electrical and EM methods, rheology, crustal heat flow, and mineral exploration.

ABSTRACT:

Since previous attenuation studies in southeastern Australia, new data from well-constrained earthquakes (up to about magnitude 5.2) are now available. The present study utilises these new data to provide more robust estimates of $Q(f)$ for the Victorian, New South Wales and South Australian regions, respectively.

The spectral content of 38 local earthquakes are examined. Increases in spectral amplitude, especially at high frequencies, between the 80 to 200 km range are attributed to critical reflections of *SmS*-waves from the Moho. The earthquakes are categorised into five groups representing different geological terrains of southeastern Australia. $Q(f)$ in the 0.5 to 25 Hz band was typically found to fit the relationship $Q = Q_0 f^{\eta}$. The relations for these terrains are given by;

Victoria (Mesozoic)	$46 f^{0.87}$
Victoria (Palaeozoic)	$53 f^{1.02}$
New South Wales (Mesozoic)	$32 f^{1.00}$
New South Wales (Palaeozoic)	$78 f^{0.79}$
South Australia (Palaeozoic)	$96 f^{0.75}$

Overall, the values derived using earthquakes with epicentres in Palaeozoic terrains of southeastern Australia were observed to possess higher values of Q_0 resulting in comparatively higher values of $Q(f)$ than for those in the younger Mesozoic terrains. These new data better describe the properties of seismic attenuation in southeastern Australia and will be employed for future earthquake source parameter computations.

1. INTRODUCTION:

The degree of seismic attenuation resulting from a complex rheological response is one of the fundamental challenges facing earthquake seismologists. Elastic waves that propagate through the earth are attenuated by geometric spreading, and absorption through both anelastic and scattering processes. Anelastic attenuation is the loss of elastic seismic energy through the fracturing and/or permanent deformation of rock, and through the generation of heat. In contrast, scattering is defined as the deflection and/or mode conversion of seismic energy due to transmission path inhomogeneities (Dainty, 1981). Hence, it is more pragmatic to describe scattering as a redistribution of energy rather than energy loss (Kvamme and Havskov, 1989). The associated attenuation factor, or seismic quality factor Q, is an intrinsic property of the material through which seismic waves pass and it summarises the characteristics that cause loss of wave amplitude with distance for reasons other than geometric spreading (Wilkie and Gibson, 1994).

However, we find that only considering Q may result in an over simplification in attenuation studies since near-surface rocks (from about 1 to 4 km in depth) typically have much lower values of Q than the rest of the travel path. In particular, high frequency energy (f > 10-20 Hz) is filtered more severely than lower frequencies (Havskov, 2001). Consequently, local crustal earthquake studies must model attenuation in the upper crust differently from functions based on Q alone. It should also be noted that the effects from near-surface attenuation are site dependant and rely heavily on local geology. Similarly, external factors including large-amplitude Moho reflections (*SmS*-phase) may add additional complexity to, and play a crucial role in attenuation studies. Previous international studies have shown that these reflections can increase overall ground motion by factors up to 2-3 (Somerville and Yoshimura 1990; Mori and Helmberger, 1996; Boztepe-Güney and Horasan, 2002).

In the present study, we attempt to provide improved estimates of the frequency dependent seismic quality factor in southeastern Australia. Data used for this study were recorded by the Seismology Research Centre (SRC), Melbourne, and Primary Industries and Resources, South Australia (PIRSA), Adelaide.

2. THE SEISMIC QUALITY FACTOR:

To determine source parameters from earthquake spectra once instrumental corrections have been made, we must first estimate and compensate for the anelastic losses resulting from waveform propagation. The dimensionless quality factor Q allows us to quantify these anelastic losses and is defined as the ratio of stored energy to the energy dissipated per cycle of a harmonic excitation (Knopoff, 1964).

The standard relation for the attenuation of seismic spectral amplitude $A(f,t)$ is given by

$$A(f,t) = A_0 e^{-\pi f \frac{t}{Q(f)}}, \tag{1}$$

where A_0 is the initial amplitude at the source, t is the source-receiver travel time and f is frequency.

Rocks possessing high values of Q are indicative of low anelastic attenuation, whereas a medium having a low Q will attenuate elastic wave amplitudes significantly more with distance. Q values greater than 500 are considered high, while values less than 50 are low (Wilkie and Gibson, 1994).

It is now generally accepted that observed Q increases with frequency in the 0.1 to 30 Hz band (Aki and Chouet, 1975; Aki, 1980; Dainty, 1981; Kvamme and Havskov, 1989). The scattering of seismic waves is believed to be the dominant mechanism for this increase (Kvamme and Havskov, 1989).

The frequency dependence of Q is commonly expressed in the following form (Aki, 1980; Kvamme and Havskov, 1989)

$$Q(f) = Q_0 f^{\eta}, \tag{2}$$

where Q_0 is the intrinsic value of Q at 1 Hz and η is a numerical constant. Values for η typically range from about 0.7 to 1.1, however values down to 0.3 have been observed for eastern Canada (Woodgold, 1990) and as high as 1.4 in Canadian underground mining environments (Feustel, 1998).

3. PRIOR ESTIMATES OF *Q* FOR AUSTRALIA:

Most estimates of Q for Australia have been derived from low frequency *Lg* coda recorded at teleseismic distances (Bowman and Kennett, 1991; Gudmundsson *et al.*, 1994; Mitchell *et al.*, 1998; Cheng and Kennett, 2002), and consequently estimate seismic attenuation in the upper mantle rather than the attenuation of higher frequency seismic energy generated from earthquakes in the upper few kilometres of the crust. Although giving an insight to the mantle structure beneath Australia and facilitating more precise magnitude determinations for teleseismic earthquakes, these estimates are not appropriate for the determination of source parameters for local earthquakes.

With advances in earthquake data acquisition and processing techniques, the need to develop a local attenuation correction to determine source parameters from the analysis of earthquake spectra became apparent. In 1994, Wilkie and Gibson adopted a simple two-layer model for the attenuation of local, high frequency Victorian earthquakes. Using the limited digital seismogram data then available, they observed that attenuation in the upper 4 km of the crust was much greater than that of the lower crust. Consequently, they suggested that $Q(f)$ of the upper (above 4 km) and lower (below 4 km) crust beneath Victoria could be characterised with a frequency dependence following the relations $20 f^{0.50}$ and $100 f^{0.85}$, respectively.

4. METHOD OF ANALYSIS:

In the present study we have derived $Q(f)$ using the two-station *spectral ratio method.* The spectral ratio method is the most widely used method in the calculation of Q and uses the spectral ratio of seismic amplitudes A_i recorded at two seismograph sites of different hypocentral distances R_i for the same event. If we assume the azimuths of the two sites are approximately equal (i.e. $\theta_1 \sim \theta_2$), the amplitude ratio of the two stations at each discrete frequency can subsequently be used to evaluate $Q(f)$ following (Kvamme and Havskov, 1989)

$$\frac{A_2(f,R_2)}{A_1(f,R_1)} = \left(\frac{G(R_1)}{G(R_2)}\right)^{\beta} e^{-\frac{\pi f(t_2-t_1)}{Q(f)}}, \tag{3}$$

where β is the geometric spreading exponent, assumed to be 1.0 for the spherical spreading of body waves in a whole space (Kvamme and Havskov, 1989) and the geometric spreading coefficient $G(R_i)$ is given by (Herrmann and Kijko, 1983)

$$G(R_i) = \begin{cases} R_i & R_i \le R_0 \\ (R_0 R_i)^{1/2} & R_i > R_0 \end{cases}, \tag{4}$$

where R_0 is typically taken to be 100 km, about twice the crustal thickness in the source region. Taking the logarithm of relation (3), we can then solve for $Q(f)$

$$\ln[A_2(f,R_2)] - \ln[A_1(f,R_1)] - \beta \ln\left(\frac{G(R_1)}{G(R_2)}\right) = -\frac{\pi f(t_2-t_1)}{Q(f)}. \tag{5}$$

All seismograms used in the analysis were digitally recorded on vertical component seismographs at a rate of 100 samples per second with an anti-alias filter at 25 Hz. An interval of 1024 samples (10.23 seconds) of the *S*-wave coda was used for Fourier analysis. Each spectrum was instrumentally corrected and subjected to multiple iterations of smoothing. Spectral amplitudes were subsequently calculated at 24 discrete frequencies logarithmically distributed from 0.5 to 25.0 Hz. $Q(f)$ was calculated following relation (5) for each site pair of similar azimuth ±15° from the earthquake epicentre. Regression analysis to determine a power-law relationship was subsequently performed to calculate the values for Q_0 and η for the aggregate site pairs of each event. Additional analysis was performed to calculate an overall attenuation function using the combined data for the Victorian, New South Wales and South Australian regions, respectively.

These data were further characterised into two geological categories; Palaeozoic and Mesozoic, grouping data from earthquakes whose epicentres occurred in terrains of these ages. The categories refer to the surface geology near the epicentre of each earthquake. Earthquakes assigned as Mesozoic may, however, occur in older rocks beneath the Mesozoic cover. Furthermore, the category refers to the conditions at the earthquake epicentre only, which may be recorded by seismographs on other rock types.

5. RESULTS AND DATA ANALYSIS:

Data used in this analysis were recorded digitally on seismographs in both the SRC and PIRSA networks. Calculations of $Q(f)$ were performed employing software developed for this study. A list of events used is given in Table 1.

Table 1. Earthquakes used for $Q(f)$ calculations. Event types M and P represent earthquakes occurring in or beneath Mesozoic and Palaeozoic terrains, respectively.

Place	State	Type	Date	HHMM	Long.	Lat.	Depth km	M_L
Katoomba	NSW	M	1993-03-22	0806	150.346	-33.826	17.4	2.5
Fish Creek	VIC	M	1994-06-27	0152	145.962	-38.722	11.6	2.9
Ellalong	NSW	M	1994-08-06	1104	151.292	-32.917	0.6	5.2
Boolarra South	VIC	M	1995-02-01	1021	146.259	-38.462	10.1	3.3
Dora Dora	NSW	P	1995-03-26	0653	147.233	-35.958	3.7	3.2
Boolarra South	VIC	M	1995-05-03	1748	146.283	-38.472	15.0	3.3
Glendon Brook	NSW	P	1995-05-28	2313	151.549	-32.543	13.7	4.0
Thomson Reservoir	VIC	P	1996-09-25	0453	146.425	-37.859	10.8	3.5
Thomson Reservoir	VIC	P	1996-09-25	0749	146.422	-37.863	11.4	5.0
Thomson Reservoir	VIC	P	1996-09-25	0756	146.438	-37.855	11.5	2.5
Thomson Reservoir	VIC	P	1996-09-25	1920	146.421	-37.856	10.6	2.4
Katoomba	NSW	M	1996-10-01	2142	150.393	-33.830	8.7	3.0
Thirlmere	NSW	M	1996-12-10	1254	150.501	-34.153	12.3	3.0
Thomson Reservoir	VIC	P	1996-12-24	0844	146.440	-37.857	11.7	2.4
Thomson Reservoir	VIC	P	1997-02-03	2314	146.427	-37.866	10.6	2.4
Burra	SA	P	1997-03-05	0615	138.979	-33.817	21.7	5.0
Burra	SA	P	1997-03-15	1507	139.077	-33.778	17.5	3.0
Tatong	VIC	P	1997-06-27	0320	146.094	-36.781	8.6	4.3
Port Augusta	SA	P	1997-07-21	1510	137.769	-32.725	14.0	3.5
Clare	SA	P	1998-01-28	1554	138.601	-33.558	11.7	2.5
Brindabella	NSW	P	1998-02-14	1823	148.702	-35.375	8.5	4.5
Cleve	SA	P	1998-02-26	1413	136.621	-33.347	27.7	4.1
Fish Creek	VIC	M	1998-04-04	0955	146.124	-38.709	16.2	3.7
Corryong	VIC	P	1998-07-17	0122	148.005	-36.441	20.3	4.6
Peterborough	SA	P	1998-09-10	2028	138.788	-33.090	3.7	2.5
Thomson Reservoir	VIC	P	1998-09-19	0548	146.381	-37.835	3.6	2.4
Sedan	SA	P	1998-11-15	0939	139.400	-34.126	10.0	2.8
Yallourn North	VIC	P	1999-01-13	0940	146.317	-38.097	12.3	3.9
West Wyalong	NSW	P	1999-03-14	0013	147.070	-34.009	12.2	4.7
Appin	NSW	M	1999-03-17	0158	150.748	-34.253	5.3	4.3
Lake Mountain	VIC	P	1999-04-16	1051	145.984	-37.413	3.6	3.3
Frogmore	NSW	P	1999-07-13	0142	148.976	-34.286	3.7	3.5
Clare	SA	P	2000-03-11	0329	138.443	-33.727	21.8	2.9
Boolarra South	VIC	M	2000-06-11	1840	146.302	-38.425	16.6	2.6
Fish Creek	VIC	M	2000-08-03	1129	145.987	-38.752	16.2	3.4
Gladstone	SA	P	2000-08-18	1101	138.464	-33.274	19.5	4.1
Boolarra South	VIC	M	2000-08-29	1205	146.245	-38.402	15.1	4.6
Boolarra South	VIC	M	2000-08-29	1020	146.236	-38.391	14.6	2.8

Prior to estimating $Q(f)$, the authors first investigated the amplitude dependency of each of the 24 frequencies with hypocentral distance for each event. These observations indicate that spectral amplitudes decrease with distance, as predicted by geometric spreading. Moreover, many events suggest an increase in ground motion at seismographs within a range of approximately 80 to 200 km from the source. An example of this occurrence is shown in Figure 1. Increases in spectral amplitude and consequently stronger ground motion in this range could be due to critical reflections of *SmS*-waves from the Moho. For this reason, the authors have avoided calculating $Q(f)$ for site pairs at dissimilar distances according to the assumed zone of *SmS*-wave

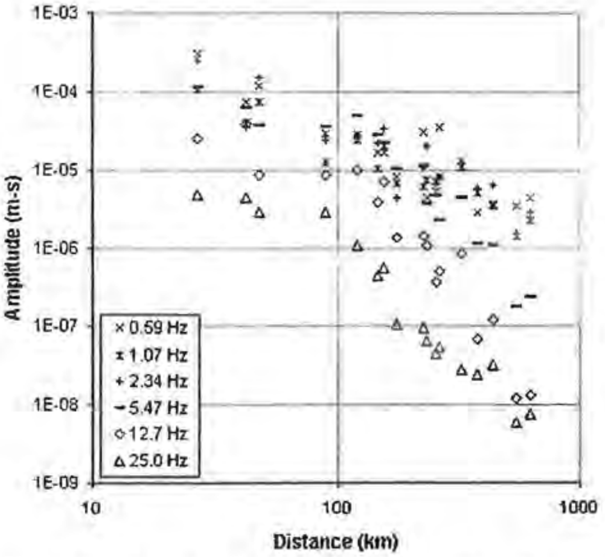

Figure 1. The spectral amplitude vs. hypocentral distance response of the M_L 4.6 29 August 2000 Boolarra South earthquake for frequencies sampled at 0.59, 1.07, 2.34, 5.47, 12.7 and 25.0 Hz, respectively. Stronger ground motion at distances from 80 to 200 km are due to critical reflection of *SmS*-waves in this range. Note that high frequency motion attenuates more quickly with distance than low frequency motion. It should also be noted that effects due to differences in site azimuth and site condition were ignored in this plot.

amplification (i.e. $R_1 \leq 80$ km and $80 < R_2 \leq 200$ km, or $80 < R_1 \leq 200$ km and $R_2 > 200$ km).

The frequency dependence of the quality factor for the different regions in southeastern Australia is evaluated in Table 2 and illustrated in Figure 2.

Table 2. Estimates of $Q(f)$ for the different regions of southeastern Australia.

Region	Q_0	η
Victoria (Mesozoic)	46	0.87
Victoria (Palaeozoic)	69	1.02
New South Wales (Mesozoic)	32	1.00
New South Wales (Palaeozoic)	78	0.79
South Australia (Palaeozoic)	96	0.75

6. DISCUSSION AND CONCLUSIONS:

Using the spectral ratio method, we have estimated the quality factor $Q(f)$ for five regional geological settings within the states of Victoria, New South Wales and South Australia. Earthquakes occurring in the Palaeozoic terrains of Victoria yielded the highest values of Q; increasing to about 1400 at 25 Hz, indicative of lower spectral attenuation in the medium to high frequency band (Figure 2). Although higher than the other terrains, the Q data are comparable to Wilkie and Gibson's (1994) estimates for the lower crust in Victoria.

South Australia gave the next highest values of Q, increasing to about 1070 at 25 Hz. Furthermore, the $Q(f)$ function of South Australia is also coupled with a high Q_0 of 96, indicative of lower attenuation at low frequencies. The precision of these data, however, is probably somewhat less than the Victorian and New South Wales estimates as data could only be utilised from three sites for most of the South Australian events cited in this study.

Overall, the Palaeozoic terrains of southeastern Australia were observed to possess higher values of Q_0 resulting in comparatively higher values of $Q(f)$ than for the younger, warmer and less dense Mesozoic terrains. The quality factor functions derived from Palaeozoic New South Wales and South Australia indicate consistent levels of attenuation for both terrains. The quality factor functions calculated for the Mesozoic terrains of Victoria and New South Wales are also similar.

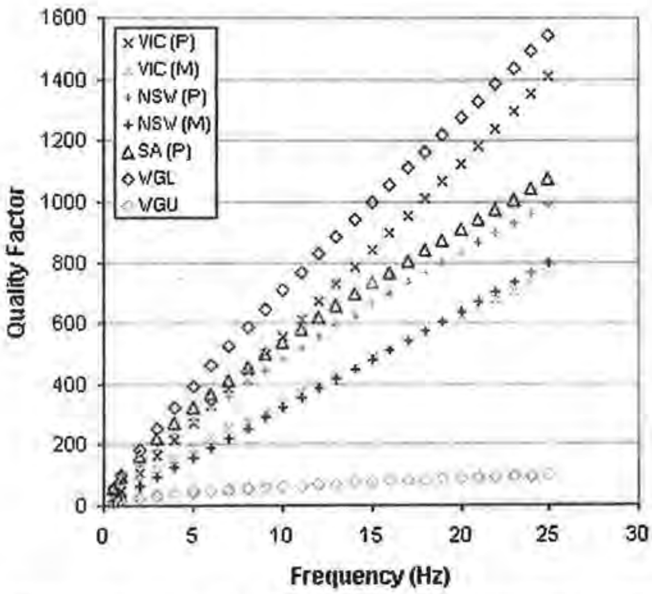

Figure 2. The frequency dependence of Q for the five designated geological settings within southeastern Australia. Again, M and P represent earthquakes occurring in Mesozoic and Palaeozoic terrains, respectively. The curves WGL and WGU refer to Wilkie and Gibson's (1994) estimates for the lower and upper crust, respectively.

These new estimates of $Q\ (f)$ have provided valuable information for future attenuation corrections to earthquake spectra, and consequently will assist in more robust estimates of earthquake source parameters and magnitudes. More work is required to better constrain the effects of site conditions, near-surface attenuation and increased ground motion from the critical reflection of *SmS*-waves in the 80 to 200 km range.

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EARTHQUAKE HAZARD ASSESSMENT OF NEWCASTLE AND LAKE MACQUARIE

DAVID ROBINSON, CVETAN SINADINOVSKI, TREVOR DHU, ANDRES MENDEZ, GLENN FULFORD, TREVOR JONES AND JOHN SCHNEIDER

AUTHORS:

Mr David Robinson[1], BSc. (Hon.) is a geophysicist with the Urban Risk Research Group at Geoscience Australia. He has also worked at the Woods Hole Oceanographic Institute and the CRC for Soil and Land Management. David has majors in mathematics and geophysics from Flinders University of South Australia.

Dr Cvetan Sinadinovski[1] PhD in geotomography, has worked as a visiting fellow in US A and Europe, and as a software specialist in Sydney and Adelaide. He is currently employed as a seismologist at Geoscience Australia in Canberra and is a member of AIG, Association of Physicists of Macedonia and AEES.

Mr Trevor Dhu[1] BSc. Hon., is a geophysicist with the Minerals and Geohazards Division of Geoscience Australia. Trevor's current research is focussed on the calculation of site amplification factors for Australian conditions. He is also interested in the use of microtremor data for the delineation of site classes in regions with limited geotechnical and geological data.

Dr Andres Nendez[2] PhD, is Principal Scientist at Impact Forecasting, where he leads the development and implementation of the seismic hazard component of Aon's catastrophe loss modelling and risk assessment services. Aon Corporation, headquartered in Chicago, USA, is a risk management, financial services, retail reinsurance and wholesale brokerage firm.

Dr Glenn Fulford[1] PhD, is currently a software engineer at Geoscience Australia., He was a lecturer in applied mathematics at the Australian Defence Forces Academy in Canberra, and has worked as a lecturer at LaTrobe University. He has also worked as a research associate for AgResearch in New Zealand.

Mr Trevor Jones[1] BSc., MSc., Dip.Tch is the Leader of Geoscience Australia's Cities Project. His research interests are primarily in earthquake risk. He is currently active in assessing the natural hazard risks to Perth.

Dr John Schneide[1] PhD is the Group Leader for the Urban Risk Research Group in the Minerals and Geohazards Division of Geoscience Australia. John's primary research interests are in the development of natural hazards risk assessment methods for application to urban centres, with a particular emphasis on earthquake hazards.

Affiliations:
(1) Minerals and Geohazards Division, Geoscience Australia
(2) Aon/Impact Forecasting, Chicago, USA

SUMMARY

The 1989 Newcastle earthquake measured 5.6 on the Richter scale, claimed 13 lives and caused an estimated insured loss of approximately $1.2 billion in 2002 dollars. Historical seismicity suggests that there is a potential for similar events in the future. This paper is the first of three companion papers that examine in detail the earthquake risk for the Newcastle and Lake Macquarie region.

Technical experts in seismology, tectonics and structural geology estimated the parameters describing the probability of occurrence of earthquakes for the Newcastle and Lake Macquarie region. Computer software was developed to simulate earthquakes as virtual ruptures; and attenuate the associated earthquake energy to locations on the Earth's surface. Spectral acceleration hazard maps were produced for periods of 0s or PGA, 0.3s and 1s at probabilities of exceedance of 2% and 10% in 50 years. The results indicate higher hazard for the region than that estimated by the Australian earthquake loading standard.

1. INTRODUCTION

Earthquake hazard can be measured by considering *the level of ground shaking that has a certain probability of being exceeded in a period of time.* A common way to represent earthquake hazard, and the method used in the Australian earthquake loading standard AS1170.4-1993, is to produce maps of peak ground accelerations (PGA) that have a 10% chance of being exceeded in 50 years. In order to calculate earthquake hazard on bedrock, Geoscience Australia has adopted a probabilistic modelling approach which:

- simulates numerous earthquakes using an *earthquake source model*; and
- estimates how ground shaking decreases with increasing distance from the source using an *attenuation model.*

This approach has been applied to the Newcastle and Lake Macquarie region as part of a broader study designed to analyse the hazard and associated risk of natural hazards to Australia's urban communities. The results of the Newcastle and Lake Macquarie study are summarised.

2. EARTHQUAKE SOURCE MODEL

The earthquake source model details the probability of occurrence, location and magnitude of earthquakes that could affect the study area. It can be separated into two components: the *earthquake source zones* and the *simulation of earthquakes.*

2.1 Earthquake Source Zones

An *earthquake source zone* is an identifiable region of the Earth that has a consistent level of seismicity. That is, earthquakes of a given magnitude are assumed to have an equal probability of occurrence anywhere within an earthquake source zone. The selection of the earthquake source zones used in this study was guided by a panel of expert geologists and seismologists who met at a workshop convened by Geoscience Australia in December 2000. The source zones identified during the workshop include the Tasman Sea Margin Zone (TSMZ) covering South Eastern Australia, the Newcastle

Triangle Zone (NTZ) located below Newcastle, Maitland, Singleton and Wyong and the Newcastle Fault Zone (NFZ) extending offshore from the Newcastle city. Dhu *et al.* (2002a) provide a detailed description of the three source zones. The cumulative *Gutenberg-Richter Recurrence Relationship* was used to characterise the seismicity in each of the source zones.

Two different configurations of the three earthquake source zones were used to simulate earthquakes in the Newcastle and Lake Macquarie study (Figure 1). The first configuration (Figure 1a) was used to generate earthquakes with moment magnitudes between 3.3 and 5.4. It consists of the NTZ and two portions of the TSMZ (labelled as TSMZ1 and TSMZ2). The second configuration (Figure 1b) was used to generate earthquakes with moment magnitudes between 5.41 and 6.5. This configuration consists of the NFZ and two different portions of the TSMZ (labelled as TSMZ3 and TSMZ4). The separation of the region into the two configurations was required because the different geological structures are believed to give rise to earthquakes with different magnitudes. The relevant seismological parameters for each of the zones are given in Table 1.

Table 1: Seismological parameters for the earthquake source zones. The parameters a and b are the coefficients of the cumulative Gutenberg-Richter relationship, a_{min} is the number of earthquakes occurring per year within each source zone that have a moment magnitude greater than or equal to Mw_{min} and Mw_{max} represents the maximum magnitude that is likely to occur in the source zone. Note that the a parameter values have been normalised to 100,000 km^2 for comparison between zones. Contrastingly, the a_{min} values have not been normalised and are the actual values used in the simulation of earthquakes.

zone	Area (km^2)	b	a	a_{min}	Mw_{min}	Mw_{max}
TSMZ1	57731	1.14	4.40	2.53	3.3	5.4
TSMZ2	56703	1.14	4.40	2.48	3.3	5.4
NTZ	5054	1.0	4.35	0.568	3.3	5.4
TSMZ3	72205	1.118	4.33	0.014	5.41	6.5
TSMZ4	44149	1.118	4.33	0.0086	5.41	6.5
NFZ	3047	1.0	4.13	0.0016	5.41	6.5

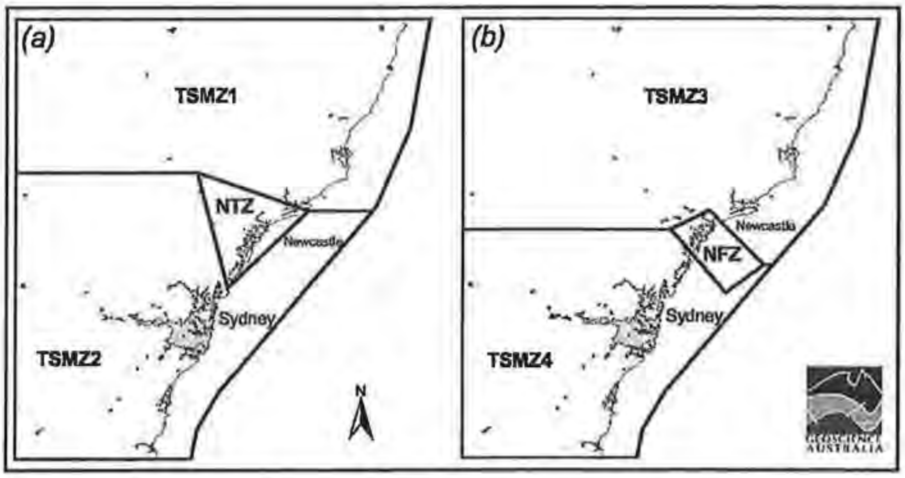

Figure 1: Two configurations of the Earthquake Source Zones. The two configurations are believed to generate events with moment magnitude between (a) 3.3 and 5.4, and (b) 5.41 and 6.5 respectively.

2.2 Simulation of Earthquakes

The *simulation of earthquakes* involves the creation of a catalogue (or database) of representative earthquakes that are likely to contribute to the hazard in the study region. The earthquakes were conceptualised as 'virtual' rupture planes within the Earth. The simulation incorporated assumptions that reflect our current understanding of earthquakes in the region. The main assumptions include:

- each earthquake is treated as an independent event;
- the earthquake magnitude follows a bounded Gutenberg-Richter distribution (Kramer, 1996);
- each simulated earthquake is a rectangular rupture, with width, depth and length assumed to be deterministic functions of the magnitude; and
- the azimuth of each virtual rupture was randomly assigned.

3. ATTENUATION MODEL

Attenuation models describe how the intensity of ground shaking decreases with increasing distance from an earthquake. The nature of earthquake attenuation in the study region is poorly understood due to a lack of strong ground motion data. This study adopted an attenuation model developed for central and eastern North America. The Toro *et al.* (1997) model was selected because:

- the tectonic environment in central and eastern North America is generally thought to be a similar intraplate environment to southeastern Australia;
- it describes the attenuation of response spectral acceleration as well as PGA; and
- it includes both a median attenuation model and a measure of the model variability due to the randomness inherent in natural processes.

The response spectral acceleration describes the maximum acceleration experienced by a single degree of freedom (SDOF) system due to a particular ground motion and it is a function of the natural period and damping ratio of the SDOF system. It should be noted that for simplicity building damage is modelled by assuming that buildings behave as SDOF systems in the Newcastle and Lake Macquarie earthquake risk assessment (Fulford *et al.*, 2002).

4. CALCULATION OF EARTHQUAKE HAZARD

12000 earthquakes were simulated using the earthquake source model. A 250m uniformly spaced grid of sample points was created. For each earthquake - sample point combination, the intensity of ground shaking was calculated by choosing a random variation from the median attenuation model. This stochastic approach thus incorporates the documented randomness inherent in the attenuation process of seismic energy. A given level of hazard was defined and the maximum response spectral acceleration that has at least that chance of being exceeded in the specified time frame identified. The identified response spectral acceleration is then taken as the measure of the hazard at each sample point.

5. RESULTS

Figure 2 illustrates the peak ground acceleration that has a 10% chance of being exceeded in 50 years on a rock outcrop. The results illustrate a north easterly trend of increasing hazard and an average acceleration of around 0.2g. It should be noted that the buildings in Newcastle and Lake Macquarie are not built on rock but on varying thicknesses of regolith. A discussion of regolith hazard in Newcastle and Lake Macquarie is provided in the second of this series of papers (Dhu *et al.*, 2002b). The Australian earthquake standard AS1170.4-1993 indicates a similar trend of increasing hazard towards the northeast of the study region. However, the bedrock hazard calculated within this study is typically greater than the hazard suggested by the Australian earthquake loading standard which has an average of around 0.12g . This change is attributable to the use of different earthquake source and attenuation models.

6. DISCUSSION

The damage that is experienced by buildings is often influenced not only by the peak ground acceleration, but also by the level of ground shaking at a specific period of vibration. For example, low- to medium-rise structures are typically more vulnerable to ground shaking that has a period of vibration of approximately 0.3s than they are to peak ground acceleration. The approach adopted by Geoscience Australia can be used to determine earthquake hazard for buildings with a number of different natural periods. It can also be used to consider the hazard for a range of different probability-time combinations.

The earthquake hazard results in this work are based on numerous assumptions and idealisations ranging from the empirical relationships used to determine rupture dimension through to the use of the Toro *et al.* attenuation model. There are some assumptions and uncertainties that are believed to strongly influence the results that have been presented. The two key issues relating to the earthquake source zones that have had a significant impact on the earthquake hazard results are outlined below:

1. The Gutenberg-Richter relationships defined for the NTZ and NFZ are based on data that have a great deal of uncertainty associated with them. The Gutenberg-Richter relationship for the NTZ is based on an historical record of seismicity that is short and generally incomplete. The Gutenberg-Richter relationship for the NFZ is based on poorly constrained estimates of rupture age and total slip for a coupled fault system that may or may not be currently active. Variations in the Gutenberg-Richter relationships for either of these source zones would have a significant affect on the estimated hazard.
2. The definition of the source zones in the region has been partly based on an interpretation of the local geology. Variations in the interpretations of regional tectonics, and the definition of the source zones would influence the Gutenberg-Richter relationships for the region and consequently change estimates of earthquake hazard.

A change in the attenuation model also has the potential to significantly change the estimated hazard. To date there has been no detailed analysis of the applicability of this model to Australian conditions, and consequently there is still some question as to the appropriateness of the attenuation model. Geoscience Australia is in the process of conducting a complete sensitivity analysis to identify the effect that each of these uncertainties has on the predicted hazard.

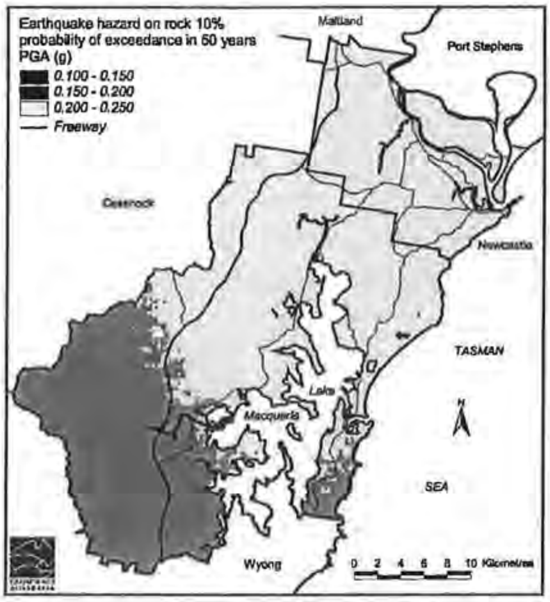

Figure 2: Earthquake hazard on rock in Newcastle and Lake Macquarie predicted by the hazard assessment conducted for this study. Earthquake hazard is defined as the peak ground acceleration that has a 10% chance of being exceeded in 50 years.

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SITE RESPONSE ANALYSIS – A COMPARISON OF METHODS AND SCALES

SRIKANTH VENKATESAN, TREVOR DHU, DAVID ROBINSON
NELSON LAM, JOHN WILSON AND ANDREW JONES

AUTHORS:

Srikanth Venkatesan is a graduate student at University of Melbourne

Trevor Dhu, David Robinson and Andrew Jones are research scientists at Geoscience Australia.

Nelson Lam is a senior lecturer at University of Melbourne.

John Wilson is an associate professor at University of Melbourne.

ABSTRACT:

The presence of sedimentary soil layers (regolith) can cause dramatic variations in localised earthquake hazard. Consequently, the response of regolith to earthquake ground shaking is an important research issue in both earthquake hazard as well as structural engineering. Earthquake hazard assessments are often carried out on a regional scale, as opposed to structural engineering problems that tend to deal with smaller scale or site-specific problems. The difference in scale between these two applications causes subtle but very important differences in the approach to site response analysis.

In order to emphasise the differences between these two scales of site response analysis, seismic cone penetrometer (SCPT) data from Newcastle has been modelled by two different techniques. At a broad scale, the SCPT data was combined to form a single site class that was then modelled using equivalent-linear techniques. This approach provides amplification factors that are generally applicable to the entire region, but lack in specific details. Contrastingly, individual SCPT holes were modelled using a technique based on total regolith thickness and shear wave velocity. This approach provides more detailed estimates of site response at a specific location, but lacks the ability to be applied across a broad region.

The objective of this paper is to examine the subtle and important differences between the two modelling approaches and to discuss their engineering implications.

1. INTRODUCTION

Developing realistic site amplification provisions for seismic design and assessment applications requires a reasonable and workable balance between over-simplicity and unwarranted complexity (Martin, 1994). A typical approach adopted by current earthquake codes of practice, is to categorize sites into some four to five generic site classes. For example, IBC2000 and UBC1997 of the United States specify site classes ranging between "hard rock" and "soft soil" based mainly on the average soil shear wave velocity. Whilst the model was developed for simplicity in application, important factors such as site period, soil-rock impedance contrasts and the dependence on earthquake source properties have not been explicitly parameterised.

As the world community becomes increasingly conscious of intraplate seismicity, seismic code provisions are required for a diversity of seismo-tectonic and geological environments. US soil classification schemes have been adopted in the codes of practice of countries, such as Australia, that do not have locally derived amplification provisions. When adopting amplification provisions from other regions of the world, it is imperative that the underlying implicit assumptions be reviewed. The use of these generic site classifications across the US has often led to concerns about ambiguities (Whitman, 1991). It is expected that the use of these classifications is far more problematic in applications outside the US.

In addressing the limitations of generic site classes, Geoscience Australia (GA) has developed regolith amplification factors for Newcastle and Lake Macquarie based on local conditions. Seismic cone penetrometer tests (SCPTs) obtained locally were used to develop a representative "average" soil profile, which was then used as input to a stochastic procedure to generate a median site amplification function. Detail of GA procedure can be referred to Dhu (2002a) published in this volume. A summary is outlined in Section 2.

The amplification factors of GA, which are based on the "average" soil profile, are generally applicable to the entire area in estimating the total cost of damage for any given earthquake scenario and is ideal for situations where detailed information on individual sites is not required. To compliment the area-specific modelling procedure of GA, this paper presents a site-specific modelling methodology, which was developed at University of Melbourne (MU) to include site period and soil depth as modelling parameters. The importance of addressing site resonance in intraplate countries characterised by non-ductile construction offers significant advantages in explicitly parameterising site period. This can be demonstrated effectively using soil displacement response spectra. Salient details of the MU methodology are presented in Section 3.

The objective of this paper is to present the area-specific and site-specific methodologies as complimentary tools.

2. AREA-SPECIFIC PROVISIONS

2.1. Geotechnical Datasets

As part of its earthquake risk assessment of Newcastle and Lake Macquarie, GA, in collaboration with the University of Newcastle, acquired approximately 100 SCPTs across the Newcastle and Lake Macquarie region (Dhu and Jones, 2002a). These SCPTs were used as the basis of GA's classification of the regolith in the region as described in Dhu et al. (2002b). Regolith site class E, which consists of sands, silts and clays overlying weathered rock, has been used as a test case in this study. This site class has been defined on the basis of ten SCPTs distributed over the study region. The velocity data from these SCPTs is presented in Figure 1.

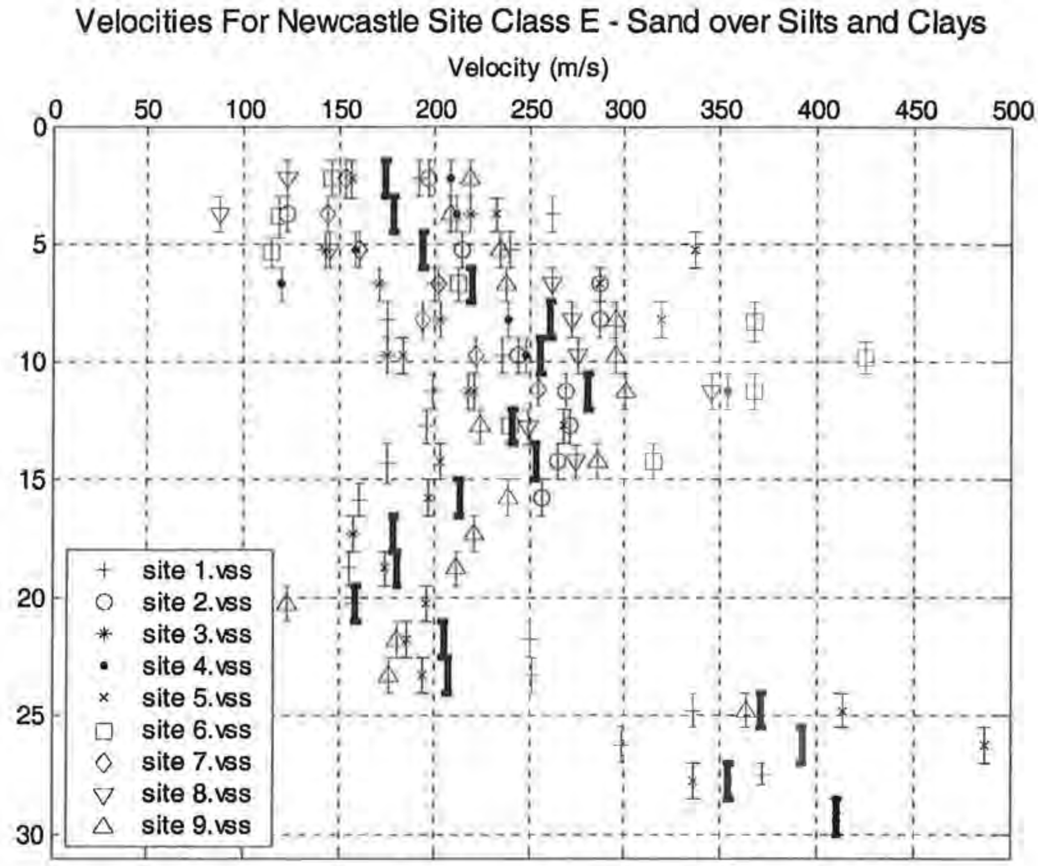

Figure 1: Velocity data from SCPTs in regolith class E (Dhu et al., 2002b). Note the thick, dark lines are the mean velocity at each depth.

2.2. Regional Risk Assessment Approach

For the purposes of the regional earthquake risk assessments carried out by GA, it is necessary to define amplification factors for every building or point of interest in the study region. It is not practical to define individual amplification factors for each building or point of interest. Consequently, it is necessary to classify the study region into areas where the regolith is thought to behave consistently during an earthquake.

After dividing the study region into these regolith site classes, it is necessary to create amplification factors that can be used when calculating the earthquake risk or hazard at each point. Natural processes are inherently variable, and consequently it is not realistic to assume that a single geotechnical model will accurately represent the entire region classified by a single site class. Hence, calculating the site response of a single representative velocity profile will not adequately capture the response of an entire site class. Consequently, a series of 50 velocity profiles were statistically generated for this site class. The mean velocity profile presented in Figure 1 was used as a typical profile for each class. Fifty velocity profiles were then generated from lognormal distributions based on variability observed in North America. Examples of the randomised velocity profiles are displayed in Figure 2. The total regolith thickness and strain dependent material properties were also randomised for each of the velocity profiles.

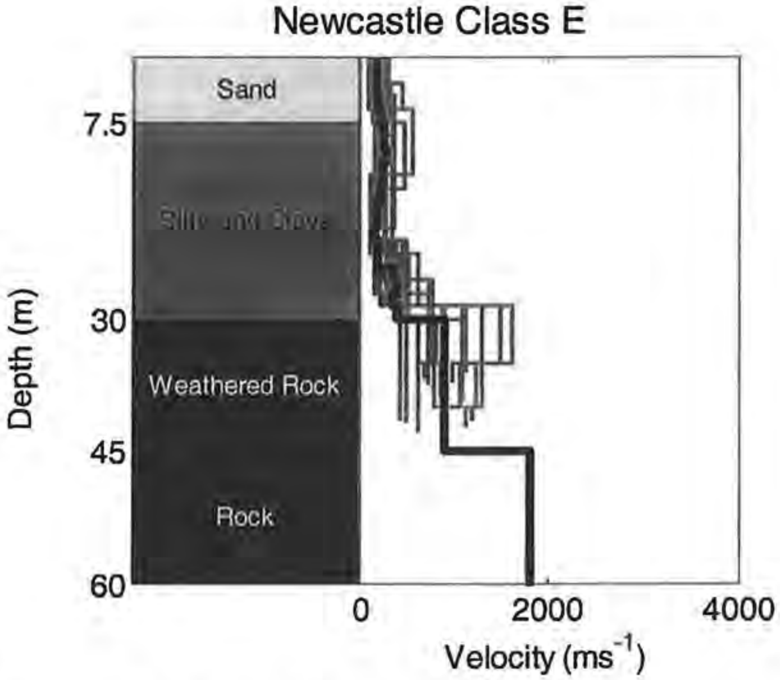

Figure 2: Examples of the randomised velocity profiles (light) calculated from site class E's median profile (dark)

Amplification factors were calculated for all 50 of the randomised profiles, using an equivalent-linear methodology (Electric Power Research Institute, 1993; Dhu et al., 2002b). These amplification factors are log-normally distributed, and hence an appropriate median and standard deviation are calculated from the data. This "distribution" of amplification factors is then used to randomly select amplification factors when calculating earthquake risk or hazard for any point within this site class (Dhu et al., 2002c).

3. SITE-SPECIFIC PROVISIONS

The stochastic procedure presented in Section 2 is designed to provide amplification factors for a statistical or probabilistic hazard assessment of a region. These amplification factors are not designed to precisely describe the dynamic properties of individual soil columns. Significantly, site specific details such as the total thickness of the soil layers which controls the period of the trapped shear waves (ie the site natural period) has not been accurately represented by the stochastic model. To assess the implications, results obtained from the stochastic procedure based on site category Class E are compared with those obtained directly from equivalent-linear shear wave analysis [using program SHAKE (Schnabel et al., 1972)] of the original SCPT records from Newcastle pertaining to the same site class. The analyses employed synthetic bedrock motion records that were generated by the computer program GENQKE (Lam et al., 2000).

The response spectral amplification factors obtained from the SHAKE analysis of individual soil columns are shown in Figure 3 along with the amplification functions predicted by the stochastic procedure. The variability of the locations of the individual amplification peaks is important to note. Whilst the amplification factors at resonance were consistently in the order of 3-5, the period at resonance (or site period) varied between 0.5 and 1sec. This has significant implications since the peak response spectral displacement demand (Δ) at resonance is proportional to the site period when the response spectral velocity is held constant. For example, in Figure 3 Site-2 & Site-8 have almost the same response spectral velocity but very different response spectral displacement, as demonstrated in Figure 4. The storey drift behaviour on the two sites are consequently very different.

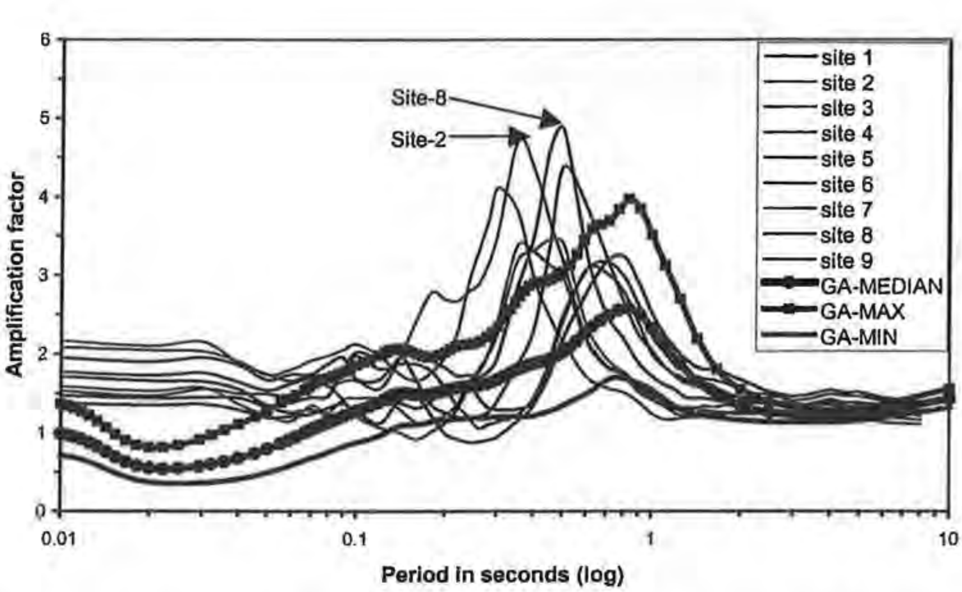

Figure 3. Comparison of amplification factors between GA & MU approach (log normal distribution)

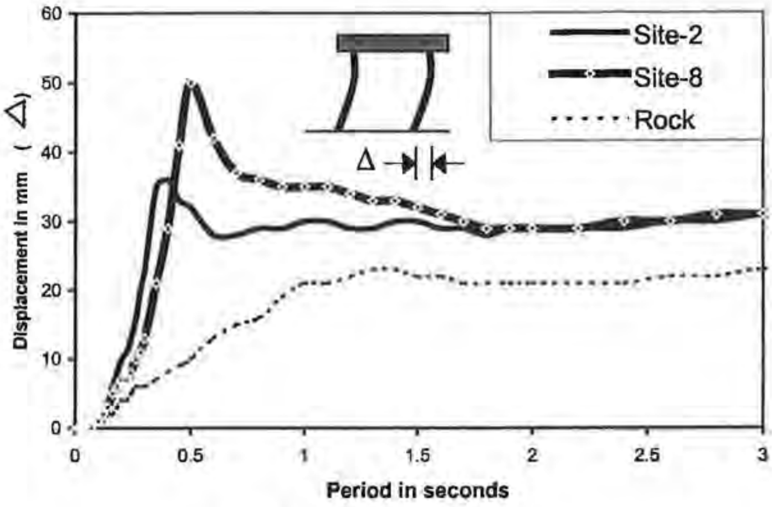

Figure 4. Soil and Rock Response Spectral Displacement

A simple manual procedure known as the Frame Analogy Soil Amplification (FASA) model developed in recent years at the University of Melbourne (MU) facilitates the incorporation of the site period in the construction of the site specific displacement response spectrum as shown in Figure 4 (Lam et al., 2001). Application of the MU procedure in the recent study by Chandler et al. (2002) shows excellent linear correlation between Δ and the site period. Consequently, Δ is well correlated with the total thickness of the soil layers.

4. CLOSING REMARKS

The comparative analysis presented in this paper shows that the soil amplification factors developed from the GA stochastic procedure are generally consistent with results obtained from the SHAKE analysis of individual SCPT records. However, more representative predictions of the displacement demand could be obtained by the analysis of individual SCPT or borehole records when input data, which accurately represents site-specific and scenario-specific information, are available.

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FAULT-CONTROLLED STRONG GROUND MOTION AND SEISMIC HAZARD

BRUCE A. BOLT
DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING
UNIVERSITY OF CALIFORNIA, BERKELEY

AUTHOR:

Professor of Seismology, 1963-1992
Director, Seismographic Stations, University of California, 1963-1989
Member, U.S. National Academy of Engineering, 1978
George W. Housner Medal, Earthquake Engineering Research Institute, 2000
President, Seismological Society of America, 1974-75
Chairman, California Seismic Safety Commission, 1984-1986
President, Consortium of Organisations for Strong-Motion Observation Systems (COSMOS), 1999-2002
Consultant on seismic hazard to private and government agencies.

ABSTRACT:

Recent damaging earthquakes, including the 1989 Loma Prieta, 1994 Northridge, and 1992 Landers, California, the 1995 Kobe, Japan, and particularly the1999 Chi-Chi, Taiwan, earthquakes have been well instrumented along the ruptured causative faults. The free-field wave recordings and GPS measurements have allowed major reformulations of the estimation of seismic motions and ground deformations in the highest-intensity zones. In particular, near-fault directivity fling-pulse contributions in velocity time histories and response spectra are now accepted as critical for engineering design in relevant hazard zones. Examples will be discussed for large bridges in the San Francisco Bay region.

Paper No. 15

1. INTRODUCTION

This paper considers properties of strong seismic shaking near its causative fault. Predictions of basic seismological source mechanism theory have now been observationally verified in the patterns of recorded time histories. Analysis and interpretation have developed in the last few years using recordings obtained from several recent earthquakes with magnitudes about 7.0 to 7.5, particularly the 1994 Northridge, California, earthquake and the 1999 Chi-Chi, Taiwan, earthquake. These earthquakes provided crucial new instrumental measurements of strong ground shaking that verified and extended earlier explanations of the "fling" pulse recorded at the Pacoima site in the 1971 San Fernando and 1992 Landers, California, earthquakes. These pulse-like ground velocities and displacement, in both the fault-normal and fault-parallel directions, have significant consequences for structural engineering design.

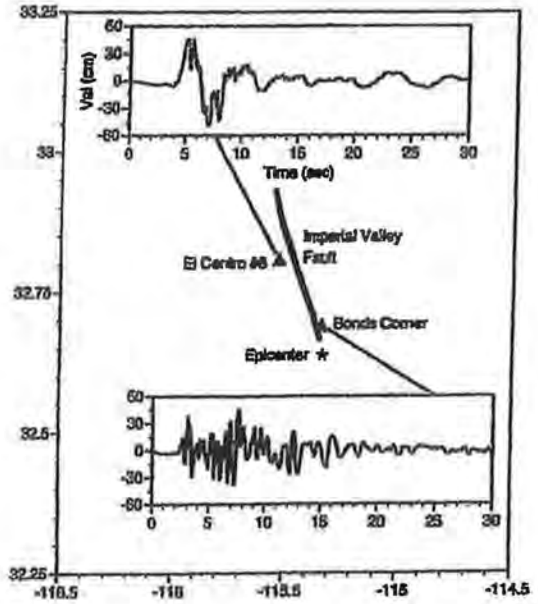
2. SEISMIC GROUND MOTIONS NEAR TO A FAULT SOURCE

Instrumental recordings show that particular directional components of near-fault ground motions contain large long-period (1-4) wave pulses. There are two causes of these long-period pulses; first, constructive interference of the dynamic waves due to directivity focusing of the fault rupture; second, strain of the ground associated with the permanent elastic offset. These pulses are thus an aspect of the elastic rebound of the strained rocks along the rupturing fault (see e.g., Bolt, 1996). These two types of long-period ground pulse are superimposed for arbitrary directions but attenuate differently from one another. For reference to separate these two effects, the terms "directivity pulse" and "fling-step" are used for the rupture directivity and elastic rebound ground displacements, respectively (see Bolt & Abrahamson, 2002).

Rupture directivity occurs when the fault ruptures toward the site and the slip direction (on the fault plane) is aligned with the rupture direction (e.g., Somerville et al., 1997). As described below, it is strongest on the component of motion perpendicular to the strike of the fault (fault normal component) and has been incorporated in seismic hazard estimations for critical large structures for several decades. Fling-step effects have been more recently confirmed and, as yet, are not as familiar in earthquake engineering. They occur when the site is located close to the fault with significant surface rupture and they are present on the component parallel to the slip direction as a permanent ground displacement. Thus, for strike-slip earthquakes, such as the 1940 and 1979 Imperial Valley, California, earthquakes, the rupture directivity pulse and the fling-step pulse will naturally separate themselves on the two horizontal components. For dip-slip earthquakes, such as the 1999 Chi-chi, Taiwan, earthquake, the vectorial resolution is more complicated. The rupture directivity effect will be strongest on the fault normal component at a location directly updip from the hypocenter. A component of the fling step will also be observed on the horizontal component perpendicular to the strike of the fault depending on its dip. Thus for dip-slip faults, directivity-pulse effects and fling-step effects occur on the same component (see Somerville & Abrahamson, 1995).

The horizontal recordings of stations in the 1966 Parkfield, California, and the Pacoima station in the 1971 San Fernando, California (Bolt, 1975), earthquake were the first to be discussed in the literature as showing near-fault velocity pulses. These pulses, with maximum amplitudes of 78 and 113 cm/sec, respectively, consisted predominantly of horizontally polarized SH wave motion and were relatively long period (about 2-3 sec).

Additional recordings (see Figure 1) in the near field of large sources have confirmed the presence of energetic pulses of this type (Bolt, 1997), and they are now included routinely in synthetic ground motions for seismic design purposes. Most recently, the availability of instrumented measured ground motion close to the sources of the 1994 Northridge earthquake (Heaton et al., 1995) the 1995 Kobe earthquake, and the 1999 Chi-Chi earthquake (see Uzarski and Arnold, 2001) provided important recordings of the velocity pulse under different conditions.



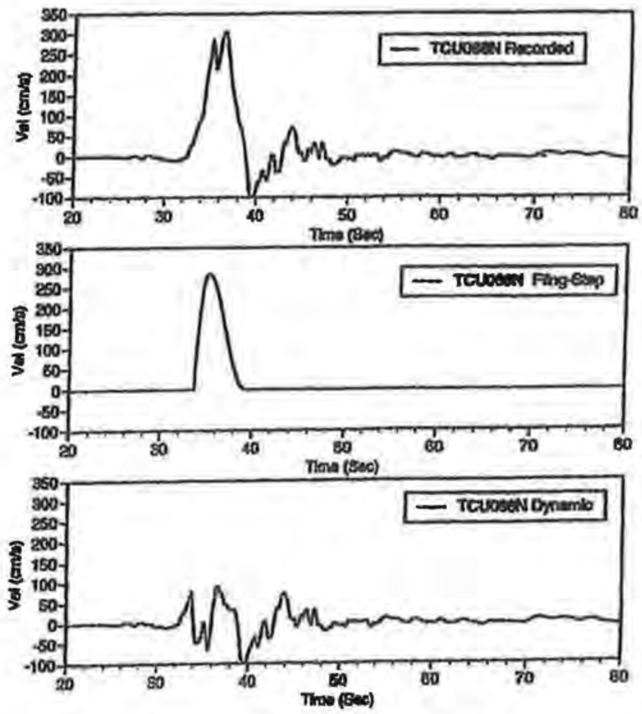


Figure 1 Directivity effects in the fault-normal (230° comp) velocity time histories from the 1979 Imperial Valley, California, earthquake.

Figure 2 Mainly fault-parallel velocity pulses at Station TCU068 separated into a dynamic seismic-pulse and a fling-step component (after N. Abrahamson) in the 1999 Chi-Chi, Taiwan, earthquake.

3. THE EFFECT OF RUPTURE DIRECTIVITY

In the case of a fault rupture toward a site at a more or less constant velocity (almost as large as the local shear wave velocity), most of the seismic energy from the extended fault rupture arrives in a short time interval resulting in a single large long-period pulse of motion, which occurs near the beginning of the record. This wave pulse represents the cumulative effect of almost all of the seismic radiation from the moving dislocation. In addition, the radiation pattern of the shear dislocation causes this large pulse of motion to be oriented mostly in the direction perpendicular to the fault. Coincidence of the radiation-pattern maximum for tangential motion and the wave focusing due to the rupture propagation direction toward the site produces a large displacement pulse normal to the fault strike (see Bullen and Bolt, 1985, pg. 441).

The directivity of the fault source rupture causes spatial variations in ground motion amplitude and duration in various directions around faults and produces systematic differences between the strike-normal and strike-parallel components of horizontal ground motion amplitudes. These variations become significant for periods greater than 0.6 sec and generally grow in size with increasing period. Modifications to empirical strong ground motion attenuation relations have been suggested (Somerville et al., 1997) to account for the effects of rupture directivity on strong motion amplitudes and durations based on an empirical analysis of near-fault recordings. The ground motion parameters that are modified include the average horizontal response spectral acceleration, the duration of the acceleration time history, and the ratio of strike-normal to strike-parallel spectral acceleration.

As in acoustics, the amplitude and frequency of the directivity pulse in both ground velocity and displacement have a geometrical focusing factor, which depends on the angle between the direction of wave propagation from the source and the direction of the source velocity. Instrumental measurements show that such directivity focusing, at least for strike-slip faults, can modify the amplitude velocity pulses by a factor of up to 10, while reducing the duration by a factor of 2. For engineering purposes, it is important when considering design time-histories to realize that the pulse may be single or multiple, with variations in the *impetus* nature of its onset and in its half-width period. It has been found in design studies of large arch dams (such as the Boulder Dam in the United States) and large bridges (such as the San Francisco Bay Bridge) that the form of the pulse onset can be critical. A clear illustration is the recorded ground velocity of the 15 October 1979 Imperial Valley, California, earthquake generated by a strike-slip fault source (Figure 1). The main rupture front moved toward El Centro and away from Bonds Corner.

For the relatively high-frequency, maximum acceleration, the results are that when rupture propagates toward a site, the spectral acceleration is larger for periods longer than 0.6 seconds. For sites located close to faults, the strike-normal spectral acceleration is larger than the strike-parallel spectral acceleration at periods longer than 0.6 seconds in a manner that depends on magnitude, distance, and azimuth.

4. FAULT PARALLEL FLING AND DISPLACEMENT

Prior to the 1999 Turkey and Taiwan earthquakes, nearly all of the observed large long period pulses in near-fault ground motions could be explained by rupture directivity effects and were observed to be dominant on the fault-normal component. However, not only did recordings from the two above earthquakes contain a directivity pulse on the fault normal component but also a long-period pulse ("fling-step") on the fault parallel component.

A clear example, taken from Bolt and Abrahamson (2002), can be observed on the east-west components of ground velocity from several recordings near the source of the Chi-Chi, Taiwan, earthquake. This 7.6 magnitude earthquake was caused by thrust rupture of the Chelungpu fault, with a maximum vertical scarp offset of 4-8 m and horizontal slip of 2-3 m. In Figure 2, ground motions at station TCU068, located on the hanging wall near the northern end of the rupture, had the largest peak velocities ever recorded (on the north-south component 300 cm/s). The velocity pulse from the fling-step effect is one-sided. Most of the very large velocity at TCU068 is a result of the static displacement rebound (strain release). If the fling-step is separated from the dynamic shaking, the peak velocity of the dynamic component of shaking is reduced to about 100 cm/s (Figure 2).

There is evidence, seen the 1994 Northridge, California, and 1995 Kobe, Japan, earthquake, that the fling-step velocity pulses do not attenuate at the same rate as the directivity pulses. This entails that the two pulse components should not be simply aggregated with strong motion data consisting of purely dynamic shaking. Observational statistics or elastic dislocation modeling are not yet adequate to estimate the peak velocity from a fling-step for various fault sources. In displacement, one approach, suggested by Abrahamson is to model the fling-step by adding the amplitudes of the static tectonic deformation and the rupture rise-time (time it takes for the fault to slip at a point). For present engineering needs the lack of a representative library, such as the COSMOS Virtual Data Center, of strong ground motions and spectra from appropriate earthquakes requires judgmental extrapolation from available records of large seismic motions.

5. FAULT MECHANISM EFFECTS

More detailed description of the fling-step time-history patterns in which relative ground velocities (strains) are critical, requires computation of the actual rupture mechanisms of the relevant faults (see, e.g., Takei & Kanamori, 1997). For the time history of ground strain, the analysis is relatively simple. For example, assume a North-South fault source having a strike-slip mechanism. Suppose right-lateral rebound and a rupture beginning to the North of the structure. The earthquake mechanism then produces a directivity pulse near the site that consists mainly of a polarized SH wave. The fault mechanism entails that the first cycle amplitude would be towards the East, and, as a consequence, there would be a compressive strain at a seismic joint in a bridge, say, aligned perpendicular to the fault. In practice, the actual position of the initiation of the fault rupture and its evolutionary history cannot be predicted deterministically so that asymmetric and multiple pulse shapes and polarities and, hence, strains must be considered.

Another factor has now entered the problem of recording accurately the near source ground motions. Recorded displacement time histories can in some seismic regions be checked against precisely mapped co-seismic ground deformations directly measured by networks of GPS instruments. These correlations help greatly in the understanding of fault source effects, including fling steps and pulses. For example, the correction to strong-motion displacement records was calibrated by near-fault source GPS recordings in the Chi-Chi earthquake along the Chelungpu fault. It is clear that additional instrumentation to record strong-motion time histories remains a crucial need in earthquake countries around the world (see e.g., Bolt, 1999b. A broad collection of standardized strong-motion time histories is now being accumulated in virtual libraries for easy access on the Internet (e.g., the COSMOS Virtual Data Center at http:www.cosmos-eq.org). Such records are essential to allow greater confidence in making seismologically sound selection of design time histories and spectra.

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EARTHQUAKE INSURANCE AND TOTAL RISK MANAGEMENT

GEORGE WALKER
AON RE AUSTRALIA

AUTHOR:

George Walker is Head of Strategic Development 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 eight years, his primary role being the development and application of catastrophe loss models in the insurance industry.

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ABSTRACT:

Insurance is a significant component in the overall management of risks arising from earthquakes adopted by individuals, businesses, corporations, and governments. In its simplest form it is a case of each entity being individually responsible for its own actions, and separately insuring its own property against possible losses from earthquakes, and leaving the insurance industry through reinsurance to cope with the resulting aggregation of losses. However this simplistic picture overlooks the much more complex nature of the interaction of insurance with other aspects of total risk management. If individuals do not insure, then governments can find themselves under major pressure to provide funds for relief and reconstruction – a pressure that is driving an increasing interest in national disaster insurance schemes. If governments do not enforce earthquake resistent building standards then insurers may decide that earthquake damage is not insurable – or set premiums that have the same effect. If the infrastructure is vulnerable then the resulting business interruption insurance risk to corporations and businesses may dwarf the risk from property damage. The paper looks at the role of earthquake insurance for different groups and these interactions, and the challenges these pose to the development of a rational approach to total risk management embodying earthquake insurance.

1 INTRODUCTION

Earthquake insurance has its origins in fire insurance and the conflagration resulting from the 1906 San Francisco earthquake. At that time specific insurance against losses from earthquakes was not generally available, but the insurance industry ended up paying about two thirds of the estimated US$330 million dollars worth of property damage because much of the property was covered for fire insurance, and the fires following the earthquake were one of the major causes of loss. This resulted in earthquake losses being taken seriously by the insurance industry, with essentially three different approaches emerging – earthquake losses being covered as a voluntary addition to normal fire policies, earthquake being included as a standard peril within a fire policy, and earthquake losses being covered by special schemes under government control. (Walker,2000)

In Australia, earthquake has been included as a standard peril in conjunction with normal fire policies since 1927 for home and contents insurance, and is one of the few countries in the world where this is the case. Commercial earthquake insurance is available as a voluntary addition to fire policies for the buildings, contents including stock, and losses arising from business interruption. Because most commercial property is covered this way, and all domestic property is covered, Australia has a very high proportion of its buildings and contents covered for earthquake damage, and also probably has a reasonable level of cover against losses due to business interruption.

Indirectly insurance is also provided for loss of life in earthquakes through normal life insurance policies, for medical costs arising out of injuries through normal health insurance policies, and for injuries sustained at work through normal Workers Compensation policies. In Australia at least some governments also use insurance to cover themselves against some infrastructure losses. Insurance may also be called upon to pay for losses occurring during an earthquake which are deemed to be the result of negligence. Such losses will generally include substantial legal costs, adjustments for inflation between the time of the earthquake and the payment of costs, and may include punitive damages, and future inflationary effects if an injury giving rise to permanent medical care is involved.

2 TOTAL RISK MANAGEMENT

It is less easy to define the term 'total risk management' than it is to define the term 'earthquake insurance'. This is because it depends what is meant by 'total'. In general it probably means management in a holistic manner of all the risks within a specified system. In the business world the current term used for it is 'enterprise risk management' or 'ERM'. However businesses are only one type of system, and are themselves part of larger systems, to the risks of which they are also exposed. One of the consequences of information technology has been to increase this interdependency between systems. Such risks are often described as 'environmental risks' – not to be confused with those due to the natural environment which are also commonly referred to as environmental risks. The term 'total' could imply that these should be included explicitly by considering the larger system. Unfortunately such a process can lead to the conclusion that the only way to describe total risk management is as the holistic management of all risks in the universe! This may be a very good issue for intellectual debate, but it is not a very practical approach to business, politics and every day living. However the interactions between systems are important. Consequently a somewhat middle line has been adopted of looking at the way different systems are impacted by earthquakes, the role played by insurance in each of them, and identifying the most important factors underlying its application to the overall risk management of these systems.

The different systems considered have been based on different groups in society on the basis that the practice of 'total risk management' will be dependent on what is being managed.

3 EARTHQUAKE INSURANCE AND RISK MANAGEMENT

3.1 Personal Risk Management

The world is a risky place for individuals. The most significant are probably the risks associated with health and income generating activities, followed by the risk of a serious accident or theft of valuable possessions. The home is often an individual's most valuable possession and the risks of serious damage to it are also a matter of concern.

The primary perceived risks from earthquake are probably damage to the home and contents, the risk of death or injury arising from the earthquake, and disruption due to loss of services such as power, water and communications. There are also risks to health from poor living conditions resulting from the earthquake, and to income generating activities as a result of the impact on businesses and the economy. The primary factor affecting the impact in most cases will be construction details, with infrastructure failures and business interruption, which will generally themselves be often the consequence of construction failures, being important secondary factors. The primary factors mitigating the impact of any losses sustained at the personal level will be savings and loans, insurance, social services and charitable donations.

In Australia, with the possible exception of people in the Newcastle area who suffered loss from the 1989 earthquake, these risks will generally be perceived as very small relative to all the other risks to which a person is exposed. The primary reason for this is the low risk of serious earthquakes occurring. The automatic coverage for damage to buildings and contents in home insurance policies is probably also a factor, although experience overseas suggests that if earthquake insurance was voluntary for homeowners, few would probably purchase it, even if it was cheap.

3.2 Commercial and Industrial Risk Management

For industry the primary risks are those to financial performance. Of these risks the most significant for shareholders are those associated with management decisions, sales, financial services – particularly interest rates – and staff performance. Also significant are risks that can disrupt operations such as power, water or communication failures, equipment failures, or interruption of the supply of essential basic materials or outsourced services.

The primary concern of earthquakes is probably related to their potential to disrupt business, with the direct costs of repair and reconstruction of buildings and replacement of damaged stock and contents also of concern. Building construction and equipment failures are the major underlying factors affecting the impact, not only on the direct damage to the business, but also in respect of their influence on infrastructure failures and supplier failures, which can be the primary cause of business interruption costs. It is not unusual for the latter to be many times the direct cost of repair and reconstruction of facilities, and replacement of damaged stock and contents.

The primary factors utilised by industry to mitigate the financial impact of losses resulting from earthquakes are financial reserves, insurance, legal processes if it is believed that negligence contributed to the loss, loans, and, generally as a last resort, government relief. Reserves and insurance will generally be the principal factors, with reserves being used to meet losses up to a certain level, and insurance being used to meet losses above this level.

In Australia, industry generally perceives the risks from earthquakes to be small, but not negligible. Hence the relatively high level of insurance, which will greatly aid the recovery process in respect of repair and reconstruction, but also have a big influence on ensuring business continuity.

3.3 Government Risk Management

The primary risks for government are political risks arising from their role in managing the various aspects of community life for which it is deemed responsible. It manages these risks largely through regulations and the allocation of financial resources funded by taxation. Government operations are largely built around a steady state or gradually changing macro-environment.

Earthquakes create problems because they can cause a sudden spike which can place significant demands on government above those normally experienced in day-to-day operations. These may be in the form of emergency and social services, but also in the form of financial aid. The performance of infrastructure poses special risks. That which it owns and is damaged will need repair or reconstruction with the associated costs. However, if any of it suffers significant failure resulting in considerable community disruption, then it will create demands on government for action with attendant political risks, irrespective of the ownership.

The primary means of meeting the losses are normal government funds, if the losses are relatively small, and loans, possibly in association with increased taxation, if available funds are not sufficient. In Australia, in contrast to many other countries, the governments also use insurance to mitigate the impact of major disasters. This is primarily to cover losses to government owned facilities and infrastructure, and the risk of major legal liability for damage and injury. Concern about these issues results in governments perceiving earthquakes as a moderate risk.

3.4 Insurance Industry Risk Management

The insurance industry is different from the rest of industry since its business is risk. It collects risks, the management of which is one of its primary activities. Because the insurance industry accumulates risk, risks that may be small and unimportant to individuals can be of major significance to the insurance industry. Earthquake risks are in this category. Some risks like building fires occur relatively regularly, but are independent of each other, and the probability of any particular property being damaged in one year is small. Risks such as these are relatively easily handled by the insurance industry because they obey the Central Limit Theorem and annual losses are relatively predictable. Not so earthquakes. The risk of a significant earthquake event occurring may be very small, but if it does occur the loss to the insurance industry can be very large due to aggregation of losses. This makes management of them very difficult, particularly for publicly owned companies whose shareholders expect them to perform like ordinary companies.

The primary risks are those arising directly from building damage, and business interruption losses which are largely due to either building or equipment damage or infrastructure failures – which are also generally the result of construction or equipment failures.

The primary manner by which the insurance industry mitigates the impact of such losses is through the establishment of reserves and purchase of reinsurance. The reserves are largely invested in a mixture of equities, bonds and property, each of which have their own risks, and this adds to the problems of total risk management. In Australia, potential losses from earthquakes dominates the purchase of property catastrophe reinsurance, and is a major factor in the assessment of solvency for regulatory purposes. As a consequence, despite the low general risk of earthquakes, the perceived risk to the insurance industry is very high.

3.5 The Overall System

These individual systems do not operate independent of each other. Although the perceptions of earthquake risk vary greatly from one particular group to another, there is much in common, and some strong interactions.

The most common aspect is the dominance of failure of constructed facilities and to a lesser extent equipment on the risks of all groups. Its direct influence is large, and it is also the primary cause of infrastructure failures with their flow on in regard to business interruption losses, disruption of normal activities, and pressures on government.

The most significant interaction is the influence of government on the outcome for all groups. This is because the performance of constructed facilities is almost solely a consequence of the earthquake resistance of constructed facilities and equipment, and this in turn is primarily a consequence of the level of building regulation and control exercised by governments. Experience around the world demonstrates a clear relationship between the level of building regulations and control, and the level of earthquake damage. Poor building performance results in high losses, leading to high insurance premiums, leading to a low level of insurance, leading to greater reliance on government for relief, leading to high national social and economic consequences – and vice versa.

4 BUILDING PERFORMANCE AND INSURANCE

Since building performance has such a significant effect on the insurance industry it might be thought that insurance would have a significant effect on building regulations. However this is not true. In Australia the insurance industry is not even represented on the Australian Building Codes Board.

The reason for this appears to be that insurance is a downstream operation. Insurers interact directly with financiers, occupiers, developers, owners and builders, but not directly, apart from lobbying, with government, in respect of buildings. It is the builders, owners and developers who interact with government through regulations, but, because of a temporary association in the case of builders and developers, and insurance combined with a low perception of the risk in the case of owners, they have little interest in the risks from earthquakes. And those who have the largest influence on regulations such as researchers and designers are quite removed from the orbit of the insurance industry. As a result insurance is largely retroactive – it accepts the situation and sets its premiums and policies in accordance with the risk as it perceives them.

5 CLOSING COMMENTS

All losses are ultimately paid for by individuals. Insurance, government aid, and charity are primarily means of sharing loss, not paying for loss. Government aid and charity is paid for retrospectively, with the associated bureaucratic delays and procedures. Insurance is paid for prospectively in association with an organised system for receiving and paying claims with a minimum of delay and bureaucracy. That is its strength. Its weakness is that it is dependent on a reasonably affluent society and reasonably strong government in terms of building control to maintain a worthwhile level of cover at an affordable price, particularly in areas of high risk. Australia is fortunate in being a relatively affluent society with a relatively high level of building control living in an area of relatively low earthquake risk. As a consequence insurance penetration for earthquake cover is high, and if a major loss occurs the impact on society will be much less than would be the case in many other countries.

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