BRUCE ESPLIN

EMERGENCY SERVICES COMMISSIONER, DEPARTMENT OF JUSTICE, VICTORIA

INVITED SPEAKER

Bruce Esplin was appointed as Victoria's first Emergency Services Commissioner in June 2000. It is a unique role in Australia. The Minister for Police and Emergency Services, André Haermeyer, created the Office to develop a more strategic approach to emergency services resource planning across the State.

Bruce's Office, located within the Department of Justice, provides independent advice to Government on major issues affecting the State's emergency management arrangements and its emergency service organisations.

One of Bruce's key roles is to establish and monitor performance standards across the fire and emergency services. Bruce also plays a central role in whole-of-government coordination of emergency situations affecting Victoria.

With the return of emergency communications operations to the State in September this year, the Office of the Emergency Services Commissioner will have an important role in monitoring the performance of the new body set up to run '000' call taking operations in Victoria.

Bruce has a strong interest in change, particularly in the public safety area, and is committed to ensuring that Victoria remains a leader, both nationally and internationally, and that the State's emergency management arrangements are able to effectively manage the increasingly complex situations confronting governments around the world.

Prior to his appointment as Commissioner Bruce held the position of Director, Emergency Management Policy.

He has represented Victoria on the National Emergency Management Committee, and the National Emergency Management Executive Group for the last 6 years.

He has had some 13 years experience in areas dealing with police, fire and emergency services, and emergency management policy issues in Victoria.

(Full paper not available at time of printing)

Paper No. 1

STRATEGIC PARTNERSHIPS FOR MANAGING EARTHQUAKE RISK

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INVITED SPEAKER

Jonathan Abrahams is Acting Director, Development Group, Emergency Management Australia. This group has responsibility for the EMA Projects Program, Australian Safer Communities Awards and developing National and Commonwealth strategies, with a focus on disaster prevention and mitigation, landuse planning for natural hazards, disaster loss assessment, raising the profile of emergency management and enhancing local government emergency management capability. Jonathan has previously chaired the National Mitigation Working party, and represents the Commonwealth on the Productivity Commission's Emergency Management Performance Measurement Working Group. He holds a Master's Degree in Public Health.

Let's not kid ourselves – the management of earthquake risk is complex. It doesn't matter who we are or what we do, we can't do it on our own. We need people who can provide information on what the risk is. We need people who will make decisions about what needs to be done and we need people who can do something about it. As with many things in life, we need a team to succeed.

1. A COMMON GOAL

A defining feature of a team is that it is working towards a common goal. What are we trying to achieve collectively? It is symptomatic of our compartmentalised society that for many years different professions have shared common goals but have not been working together effectively to achieve them. This is changing as organisations and professions are becoming more global in their approaches to their business and are looking for partners with whom they can work to achieve shared and other outcomes. In the public sector, these outcomes can be expressed in terms of the impact on the status of individuals or a group or community. (SCRCSSP, p.xvi)

Emergency Management Australia's vision is "safer, sustainable communities". This vision is not exclusive to EMA, rather it is one which we share with other emergency managers, other organisations and people working in the field of community safety, and broader still, the vision reflects the aspirations of communities across Australia. Like other emergency management organisations which have adopted similar visions or missions, EMA is stating that we want emergency management to become part of the mainstream strategic and policy agenda and that we have a contribution to make. Furthermore, partnering with organisations which share this objective will be crucial to our future success, and the safety and sustainability of communities.

2. TEAMWORK

Management of the risk from earthquakes (and other hazards, for that matter) requires teamwork and a systemic capability. The capability is provided by a wide range of professions and players which includes seismologists, engineers, risk modellers, social scientists, risk communicators, insurance companies, emergency responders, health professionals, social workers, other public servants, businesses, non-government volunteers and citizens, who contribute to the safety and economic, social and environmental sustainability of communities. How do we make a team out of this "community of interest"? First, we need to respect the professions and what they bring to the management of risk. We are all contributors. Second, we need to provide opportunities for dialogue and broader discussion where people needing information can seek it from others who are willing and able to provide it.

Integration of these contributions is also needed to effectively manage risk. Conferences and forums can help bring people together to share information and knowledge, but it would be desirable if committees and risk management study teams were multi-disciplinary and multi-sectoral to reflect not only community stakeholders but also the people who can help identify, analyse and treat the risk. For example, how does the initial data collected by seismologists become useful for policy and decision-makers, politicians in all spheres of government, business leaders, or individuals? There are some promising trends in this area.

3. A COMMON APPROACH - RISK MANAGEMENT

Perhaps the most significant development in recent years in emergency management has been to consider the business in terms of risk management. Some people suggest that risk management is just window dressing, changing the labels on what we have always done. Risk management does not mean throwing away the past, as there are only so many ways which risk can be managed – measures which have been implemented in the past will continue in the future. While it might look like a small step, risk management is however a paradigm shift in the approach to our business. The big difference is that risk management provides a broader and at the same time more unifying framework which enables more comprehensive and integrated approaches to be applied and helps to break down professional, philosophical and bureaucratic barriers. In principle, risk management provides a common language with which to identify common objectives, analyse problems and develop more effective solutions with a diverse range of partners than ever before.

4. MANAGING EARTHQUAKE RISK IN AUSTRALIA

How much is known about earthquake risk in Australia? What are we – the Australian community - doing about it? How can we do it better? As a nation, as communities, as individuals, where should we be focusing our efforts in the future? And more specifically, how can earthquake engineers and others attending the conference make a more effective contribution to managing earthquake risk? The emergency risk management process can serve as useful guide for identifying the roles of various professions and demonstrating the importance of teamwork.

4.1 Setting the context

The context for earthquake risk management has political, social, financial, environmental dimensions, as well as the paradigm in which philosophy and practice is framed eg risk management. To examine the context, we need to look at the systems, values and trends in Australian communities. The trends which are shaping emergency management philosophy and are relevant to the earthquake risk management context include:

- Increased community participation in risk management decision making
- A greater value placed on data, information, knowledge and research to assist informed decision-making
- Adoption of developments in information technology across emergency management
- Widespread adoption of risk management which values risk assessment, prevention, mitigation, response and recovery as measures to manage risk

Political and community attitudes to risk and decision-making are also important considerations. Part of the political dimension is the reaction to contemporary issues and events. This is evident in the rapid and significant Government response to the terrorist attacks in the United States in September 2001 and the New South Wales Bushfires over December 2001 and January 2002. The message here is that disasters have resulted in changed attitudes, increased funding and enhanced capability development. We need to be ready to put the case to government and the community at the time when they are most receptive to hear them. What can we learn from this? When an earthquake occurs either in Australia or overseas, there is a window of opportunity in which to act and communicate key messages about the earthquake risk in Australia.

4.2 Establishing Evaluation Criteria

In the risk management framework, the common goal can be expressed as managing risk to the safety and sustainability of communities. The earthquake risk management team aims to reduce risk, particularly to life and property, to the lowest level possible, but is also constrained by what is reasonably practicable and acceptable to the community. Constraints include the extent of our knowledge of the risk itself – what do we know about the hazard? How well do we understand the vulnerability and resilience of communities? What has worked in the past – what didn't? Another constraint is the effectiveness of the treatment measures available. Do we have the optimum understanding and technology to make buildings safe or to find survivors from collapsed buildings in rubble piles? What are the best ways to communicate risk to different audiences? How well do we manage the mental trauma of people affected by earthquakes? The third limitation is the extent to which the community is prepared to invest in risk reduction measures. At what cost? The community does not have unlimited resources, so where can we have the most effect on managing risk – or alternatively what are the most cost-effective measures to manage risk?

4.3 Identifying and Assessing Risk

The identification and assessment of risk are the building blocks of risk management – a greater understanding will lead to better targetting of resources to areas of highest risk, and to safer and more sustainable communities. Fundamental to emergency risk management is the description of risk as the interaction between hazards and vulnerability. The current state of Australia's understanding of earthquake hazard is essentially based on the historical record of earthquakes. Geoscience Australia has embarked on a project to map Australia's earthquake hazard based on the technical analysis of geology and topography. Meanwhile social scientists are increasing their understanding of what makes people and communities vulnerable and resilient to disasters. Higher levels of individual and community vulnerability are associated with factors which include poor or declining economic circumstances, being frail aged and very young, seriously ill, poor standards of accommodation, remote location, levels of physical and mental disability, physical and social isolation, higher risk occupations and being on holiday, often living in tents or caravans. On the other hand, resilience factors include resources, knowledge and information, access to services, involvement in decision making process, personal coping capacities, shared community values, and shared community aspirations and plans. (Buckle, Marsh and Smale, pp37-38)

Clearly in earthquakes, the safety of the built infrastructure is a key risk factor for communities. There are opportunities for earthquake engineers to work with health professionals to determine what aspects of buildings causes death and injuries (for example, structural or non-structural factors such as heavy furniture falling down or sources of fire, or building materials which cause respiratory illnesses) and with emergency managers on understanding the circumstances which lead to entrapment and more effective and safe search and rescue operations.

4.4 Risk Treatment

Risk can be managed in a number of ways. Some of these incorporate the comprehensive approach to emergency management: Prevention, Preparedness, Response and Recovery. The community's risk can be reduced by enhancing capability in each of these areas.

Land use planning and development assessments. Planners play an important role in determining the suitable use of land for specific purposes. They need information about the hazard from earthquake scientists upon which to base their recommendations regarding land use for approval by the relevant local or state government authority. At the same time, working with earthquake engineers can help local government to assess the suitability of particular buildings, building types and construction standards for particular areas.

Structural/non-structural and critical infrastructure protection. Given the number of deaths and injuries in earthquakes caused by the collapse of buildings and infrastructure, the role of earthquake engineers and the building industry generally is of paramount importance in preventing structural collapse. In many respects, the standard of building design and construction could be considered the most effective measure for managing risk of earthquake, based yet again on the understanding of the underlying hazard risk. At the same time, the sustainability of communities also depends on certain buildings maintaining their functionality, so that hospitals, emergency operation centres and utilities can continue to provide services to the community, even if the structures experience damage. This has implications for further involvement of engineers in the design and construction of buildings and for the use of materials which reduce the risk of respiratory illness. There are significant opportunities for earthquake engineers to become actively involved in nascent critical infrastructure protection initiatives, because when an earthquake strikes the impact on the infrastructure will have a significant bearing on the risk to communities.

Personal protection. Recent community eduction in structural and bush fire is based on the premise that the most effective level of protection for individuals is action which they take to protect themselves. With respect to earthquake risk, a key question is how can it be taken seriously in Australia? Psychologists, behavioural and social scientists have informed our understanding about changing people's behaviours so that they take action which will protect them from risk. This is a particular challenge for earthquake risk in Australia where the lack of awareness results in limited proactive personal protection behaviour. Nonetheless, there is a need to formulate appropriately tailored messages about risk and personal protective measures which are then delivered effectively and efficiently to, and among, the target audiences. This may include empowering people with information about the earthquake risk in the area where they live or are considering to live, about the security of non-structural elements in the house or at work, protective behaviour during the course of an earthquake and about what to do after the shaking ceases, and if it is safe to do so, where to go after a disaster strikes. It might appear that in the case of earthquake risk, the best time to inform the public will be when an earthquake occurs in the local area or on an anniversary of a significant recent event.

Warnings. The research and further development of systems enabling the short-term prediction of earthquakes is critical for the effectiveness of warning the population of an impending earthquake. If communities and businesses can be given advance notice of an impending earthquake or after-shocks then they can take precautions to reduce their risk. The issuing of warnings requires an understanding of community behaviours, a risk communication strategy, a system by which to deliver warnings and a clear message of what people and industries should do.

Evacuation. Current understanding of earthquake risk indicates that most injuries occur as people as entering or exiting buildings, so the message is that people should stay where they are during an earthquake. However, when it is safe to do so, it is recommended that people move outdoors and move as far away as possible from buildings. (Noji, p160) Structural engineers could provide further insights into determining whether different evacuation behaviours are required for particular circumstances, for example, is there different advice for the person on the 50th floor versus the ground floor of a high rise building, or someone working in a wooden farmhouse.

Response. Australia has a well-established emergency response system based on the strength of State and Territory emergency management arrangements, and cooperation between all spheres of government and with the community. The Australian emergency management arrangements are based on an all-hazards approach so that a common set of arrangements are applied to emergencies irrespective of their cause. The challenges to this framework posed by earthquakes are manifold: the potential for catastrophic losses on a community-wide scale; lack of experience with significant earthquakes unlike other hazards such as bushfire, flooding and cyclones; and unlike these hazards, without the warnings or predictions, it will be a cold start for the response system, initiated by immediate personal experience or on the basis of advice from Geoscience Australia's which monitors earthquake activity and forwards information to EMA which in turn notifies the State and Territory emergency management organisations. An event of any significance would quickly exhaust the resources of most States and Territories to respond, putting greater emphasis on the provision of mutual aid between states, Commonwealth assistance and possibly international assistance. A better understanding of the earthquake risk in Australia would assist emergency managers in determining what further capabilities are required to manage the risk, such as deploying resources to where the most damage is most likely to have occurred and anticipating medical needs due to the number and type of injuries associated with earthquakes. One area where capability is growing is in urban search and rescue (USAR), initiated before 11 September 2001 but given significantly greater impetus as a result of the World

Trade Center collapses. The national approach emphasises the interoperability of equipment, common training standards and protocols for the call-out and deployment of USAR teams. Capabilities include the identification and training of structural engineers who are an integral part of the USAR team. It is proposed that early next year specialised training, based on a package developed in New Zealand and adapted for Australian needs, will be offered to engineers who will be available to provide support in USAR operations.

Recovery. Similarly, Australia's recovery system of psychologists, community development officers, non-government organisations, social workers and engineers has enabled communities to recover effectively from disasters. A key role for engineers in an earthquake will be to assess which buildings are safe and which are not to be entered. This may be of great importance to businesses wanting to re-enter their buildings to recover vital records, utility operators maintaining or resuming services, or for householders wanting to collect treasured possessions.

Cooperation with social workers is paramount in communicating the news that homes need to be abandoned, particularly for people who have lived in the same place for all their lives. Another key challenge which involves engineers will be the provision of suitable temporary shelter for people whose places have been deemed no longer habitable, and then the effective development of permanent housing in such a way as not to repeat the circumstances which led to the first disaster.

5.0 CONCLUSION

The management of earthquake risk in Australia presents particular challenges. People around Australia in different professions are rising to these challenges, by improving the understanding of the earthquake hazards, developing our understanding of vulnerability, risk modelling, risk communication, reviewing the building code and developing urban search and rescue capabilities. In its own way and collectively, this work is making a significant contribution to improving Australia's capability to manage earthquake risk.

For the earthquake engineers there are current and emerging opportunities to increase their influence and contribution to the management of earthquake risk, by getting involved in initiatives addressing critical infrastructure protection, urban search and rescue, and the review of the building standards.

While the Australian community is not focused on earthquakes at this time, it is essentially a sleeping issue for most Australians. I should add it's not the only one. However, experience with other hazards indicates that they will be awakened, both physically and psychologically, when the next tremor or earthquakes comes along. The plan to deal with any damage arising from the event will swing into action, but also, as the earthquake risk management team, we need a plan to communicate key messages at political, business and community levels.

In the meantime, we need to continue to work together as a team to manage the complexity of earthquake and other risks to Australian communities. There should be more forums such as the Australian Earthquake Engineering Society Conference which brings people together from different disciplines and sectors to share and develop their ideas and practices, and work towards our common goal.

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THE IMPACT OF 500 MPa REINFORCEMENT TO AS/NZS 4671:2001 ON THE ANALYSIS AND DESIGN OF REINFORCED CONCRETE STRUCTURES TO AS1170.4 – THE IMPACTS AND RISKS

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ABSTRACT:

The introduction of Grade 500 MPa reinforcement and release of AS/NZS 4671 has codified a reduction in the allowable minimum uniform elongation of the standard hot-rolled Class N deformed bar in comparison to the previously produced "Y" Tempcore bar. This has already resulted in several major revisions to the SAA Concrete Code AS 3600, specifically in relation to ductility of flexural cross-sections, moment redistribution, and serviceability requirements. For economically designed reinforced concrete framed structures under seismic base acceleration, safe levels of capacity in the post-yield cycle range is provided through sufficient curvature ductility. This can only be achieved through conventional reinforcement with suitable uniform elongation characteristics. The basis of analysis and design to AS 1170.4, the SAA Earthquake Code, is the selection of the structural response factor, Rf, an empirical value representing the inelastic response of a given lateral force resisting structural system. Is this approach consistent with reinforcement to AS/NZS 4671:2001 ?

Paper No. 3

THE IMPACT OF 500 MPa REINFORCEMENT TO AS/NZS 4671:2001 ON THE ANALYSIS AND DESIGN OF REINFORCED CONCRETE STRUCTURES TO AS1170.4 – THE IMPACTS AND RISKS

1.0 INTRODUCTION

The introduction of Grade 500 MPa reinforcement and release of AS/NZS 4671, the SAA code for Steel reinforcing materials, has been received with much debate and indeed some consternation within various elements of the structural engineering fraternity. The reduction in the allowable minimum uniform elongation of the standard hot-rolled Class N deformed bar in comparison to the previously produced "Y" Tempcore bar of up to 70% has already resulted in several major revisions to the SAA Concrete Code AS 3600, specifically in relation to ductility of flexural cross-sections, moment redistribution, and serviceability requirements. Failure of the main tensile reinforcement prior to ductile crushing of the extreme concrete compression fibre is now a common flexural failure mechanism at ultimate strength Gilbert (2001), resulting from the higher yield strength and lower uniform elongation of the Class N bar.

For economically designed reinforced concrete framed structures under seismic base acceleration, the single most important criteria for safe levels of capacity in the post-yield cycle range is the provision of sufficient curvature ductility to allow the formation of plastic hinges in desirable locations, allowing dissipation of energy. This can only be achieved through conventional reinforcement with suitable uniform elongation characteristics. Analysis and design to AS 1170.4, the SAA Earthquake Code is directly proportional to the inherent and/or design level of ductility and post-yield inelastic response of the structural form, applied in the code through the use of the structural response factor, Rf, and deflection amplification factor, Kd. Empirical values for these factors are contained in Section 6 of AS 1170.4 for the common range of building structural forms. The tabulated values are proposed as an indication of the expected post-yield performance during repetitive load reversal cycles into the inelastic range. When applied with the minimum detailing requirements of AS 3600 it is considered that satisfactory performance can be expected during design seismic ground accelerations and displacements. The response factor Rf is intended as an indication of structure ductility and capacity for dissipation of energy, and is used to reduce the design elastic base shear accordingly. Alternatively, the deflection amplification factor is a direct indicator of the post-yield displacement demand on a frame designed elastically with a reduced base shear relative to the selected response factor.

Although seismic conditions in Australia are considered moderate relative to those experienced in other parts of the world, designers are bound to comply with the requirements of AS 1170.4. As such the most important aspect is to ensure that materials are available which are consistent with the specified design approach. Panagopoulos et al. (1999) performed a detailed non-linear parametric study and dynamic analyses under the El Centro base motion, of the column and beam cross-sections of a typically designed and detailed OMRF and IMRF, varying the yield stress of the reinforcement and the concrete compressive strength. The results of this analysis demonstrated significantly reduced curvature ductility capacities for cross-sections reinforced with Class N & L reinforcement, often governed by reinforcement fracture, relative to 400Y deformed bars manufactured under the requirements of AS1302. The results also demonstrate this reduction to be amplified as the axial load on a cross-section is increased. Although it is conceded that the column and mixed side-sway hinging modes are predominant for reinforced concrete frames designed in accordance with AS 1170.4 (due to the low seismicity relative to gravity loading requirements), the authors concluded that the curvature ductility achieved with class N reinforcement is sufficient for the moderate seismicity of Australia. No recommendations were published regarding the direct effect of the reduced curvature ductility on the design structural response and deflection amplification factors contained in AS 1170.4.

2.0 MECHANICAL PROPERTIES OF REINFORCEMENT TO AS/NZS 4671:2001

As well as allowing an increase in yield strength of deformed bar and welded wire fabric to 500MPa, the introduction of this code has standardised a number of reinforcement classes with varying mechanical properties not recognised under its predecessor AS 1302:1991. These classes are identified by the designations "L", "N", and "E", representing "low", "normal", and "seismic" ductility respectively. The history, introduction, manufacture, and properties of the bars in each of these classes have been discussed in detail by Turner (1999 & 2002). The summary below has been included for information as a comparison of the codified limits for the mechanical properties of the current deformed bars relative to the former "Y" bar.

Deformed Bar	**400Y**	**500N**	**500L**	**500E***	**300E***
Standard	1302	4671	4671	4671	4671
Yield Stress (Mpa): **fy(min)** **fy(max)**	>400 -	>500 ≤ 650	>500 ≤ 750	>500 ≤ 600	>300 ≤ 380
Tensile to yield ratio: fu / fy	1.10	1.08	1.03	1.15	1.15
Uniform elongation: εsu	0.16	0.05	0.015	0.10	0.15

Table 2.1 Code Limits for Mechanical Properties of Reinforcement

* *Currently the availability of Class E deformed bars is confined to New Zealand; the forward to AS/NZS 4671 noting that it is "envisaged" that Class N reinforcement will be adequate for the seismic conditions experienced in Australia.*

Other than the increase and allowable variation in yield stress, the clear difference is the reduction in uniform elongation for 500N and 500L reinforcement – a direct measure of ductility. The values of 0.05 and 0.015 indicate a reduction of 70% and 90% respectively relative to the 400Y bar. Figure 2.1 offers a comparison of approximate static stress-strain curves for each of the above classes of reinforcement. Of most interest is the comparison between the 400Y and 500N bars, the 500N bar being the standard product now commonly available in the Australian market. The use of 500L reinforcement has been limited for use in flexural members subject to the more onerous restrictions of AS 3600:2001 and will not generally be commercially available as a deformed bar product, but designers must be aware that the standard welded wire fabrics currently available in the market (both deformed and round) are Class L.

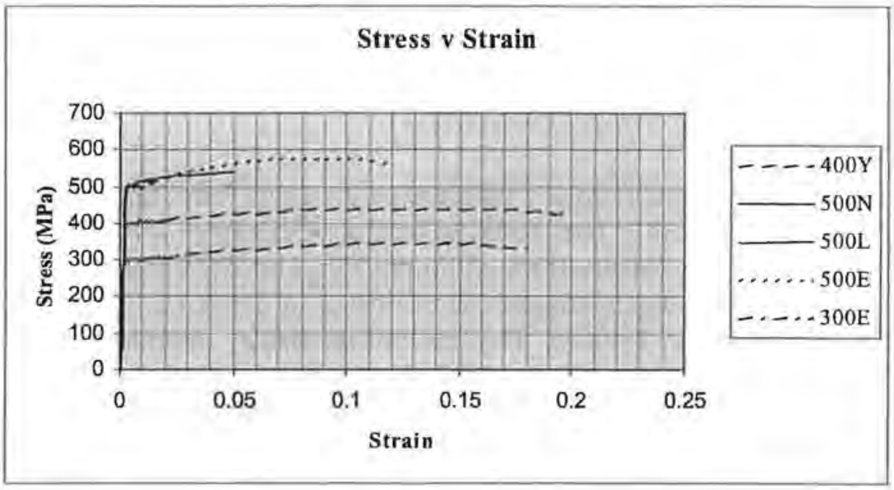

Figure 2.1 Reinforcement Stress – Strain Comparison

3.0 DIRECT IMPACTS ON SEISMIC DESIGN

Each of the properties of reinforcing steel listed in Table 2.1 directly impact the basic accepted concepts of seismic design for reinforced concrete structures regardless of whether they exhibit framed, shear wall, or dual structural lateral resisting systems. The degree of impact is dependant on the type of analysis and design, and the initial assumptions made by the designer as to the degree of ductility that can be expected from the structure during the design event. The discussion below is intended as a brief summary and reference should be made to texts such as Paulay and Priestly (1992) for more detailed discussion.

3.1 Yield Stress Range

Increasing the base yield stress to 500MPa does not itself have any significant undesirable impact for seismic design and in fact may assist in easing congestion at heavily reinforced frame joints and reinforcement ratios in major flexural members. The major drawback is the increase in required development length. The increased crack widths may also warrant further investigation into shear behaviour at cross sections of significant load reversal. The variation of expected yield stress directly influences the onset of plastic hinges in the design of ductile seismic frames, and must be considered to ensure that yielding occurs in the expected frame locations (flexural over-strength). For class "N" bars the maximum limit to yield stress of 650MPa represents a possible 30% increase to a yield moment designed at 500MPa (50% for class "L"). No maximum yield stress was specified in AS 1302 for 400Y bars and the inclusion of a maximum limit may be seen as an extra level of product control providing increased assurance to designers. This is prudent considering the minimum yield stress of bars available in the market has on average been considerably higher than 400MPa for a number of years. The 30% variation is greater than the 20% specified for the 500E bar available to New Zealand, but is in line with the UBC:1997 which provides a limit of 124MPa (30%) to the difference between the minimum specified and actual yield stress for Grade 60 (420MPa) reinforcement to ASTM A 615 when used in seismic frames.

3.2 Tensile to Yield Stress Ratio, (fu/fy)

The tensile to yield stress ratio is a measure of the strain hardening capacity of the bar. For 400Y, 500E, and 300E bars strain softening is exhibited after the ultimate tensile strength has been achieved as shown on Figure 2.1. For 500N and 500L bars Figure 2.1 indicates the ultimate tensile strength as occurring at the failure strain, defined by the onset of necking prior to fracture. The effect of the rate of strain hardening under cyclic loading is discussed below. This ratio is not constant during cyclic loading due as a result of the Bauschinger Effect, where inelastic strain develops below the original yield strain following load excursions exceeding the yield stress of the reinforcement.

3.3 Uniform Elongation, εsu

Uniform elongation of the developed constituent reinforcement is the most direct measure of the level of ductility of a reinforced concrete cross-section. For economical seismically designed members, post-elastic cyclic strain capacity of deformed bars is necessary to achieve the desired strain energy dissipation through curvature ductility while maintaining structural capacity at the location of design flexural plastic hinges. Specified uniform elongations of 0.05 for Class N and 0.015 for Class L reinforcement in AS/NZS 4671 represents reductions in the order of 70% and 90% from the value of 0.16 specified for 400Y bars in AS 1302. This represents reductions in cross-section ductility of the same magnitude. Gilbert (2001 & 2002) has demonstrated the effects of the reduced uniform elongation of 500N and 500L reinforcement on flexural cross sections at both the ultimate and serviceability limit states, and the implications on moment redistribution and plastic design at critical cross-sections under gravity loading. The results of this study are equally applicable to seismic design, and given the design philosophy inherent to the seismic design codes even more directly applicable (refer Section 3.0). The release of AS 3600:2001 has prohibited moment redistribution in flexural members reinforced with Class L reinforcement, but has not tightened the rules for Class N deformed bars relative to the 400Y bar. Plastic design with Class N reinforcement is permitted provided the design rotations at hinge locations can be achieved. Chick et.al (1999) and Patrick et.al (2001) performed full scale tests and computer simulations on single span slab elements and concluded that uniform elongations of 2.5% and 3.5% were necessary to achieve full plastic hinges associated with moment redistribution requirements of 15% and 30% respectively.

The uniform elongation of a deformed bar is also directly proportional to the rate of strain hardening. If εult represents the corresponding value of strain at the ultimate tensile strength fu (MPa), the quasi-static* rate of strain hardening can be represented by Equation 2.1:

$$rsh = (fu - fy) / (\varepsilon ult - \varepsilon y) \qquad \text{Equation 2.1}$$

where:

rsh	=	quasi-static* rate of strain hardening
fu	=	ultimate tensile stress
fy	=	yield stress
εult	=	ultimate tensile strain
εy	=	yield strain

Table 3.1 shows approximate comparative values for the rate of strain hardening (rsh) of each of the reinforcement classes listed in Table 2.1.

Bar	**400Y**	**500N**	**500L**	**500E**	**300E**
rsh	< 500	840	1200	1300	570

Table 3.1 Approximate Quasi-static* Rate of Strain Hardening for Deformed Bars

* *The quasi-static rate of strain hardening is a uniform rate based on mechanical testing in accordance with the code. In practice, under cyclic loading into this range the yield stress (and therefore yield strain) reduces on each cycle under the Bauschinger Effect and promotes premature strain hardening. Although the value of ultimate strain is relatively unaffected, the axial stress of the bar and therefore the corresponding flexural capacity at the location of a plastic hinge increases proportionally at reduced curvatures.*

Table 3.1 demonstrates that the 500N bar exhibits a rate of strain hardening (rhs) significantly higher than that of the 400Y bar. This indicates that as well as resulting in significantly reduced overall curvature ductility of a reinforced concrete cross-section, Class N reinforcement increases the rate of flexural over-strength due to strain hardening. The effects of under-estimating flexural over-strength have been discussed briefly above. For Class L reinforcement the rate of strain hardening is over seven times that of the 400Y bar, a reflection of the cold-forming process used to manufacture this grade. The effects of flexural over-strength in beam cross-sections of seismic frames leading to the formation of column hinges would be considered less severe for frames reinforced with 400Y deformed bars due to the increased curvature ductility provided by the elongation capacity.

It is worth noting that the structural Grade 60 deformed bars to ASTM A 615 as specified by the UBC:1997 for use in seismic frames exhibit minimum uniform elongation values varying from 0.09 to 0.07 with increasing bar diameter.

Testing of 500N deformed bars currently being manufactured to AS/NZS 4671 in Australia has shown average yield stresses of approximately 560MPa and uniform elongation values ranging from 0.07 to 0.12 with an average of approximately 0.11 (courtesy of One Steel Reinforcing).

4.0 IMPACTS ON SAA LOADING CODE AS 1170.4:1993

Given the brief discussion in Section 3.0 above, what impact does the introduction of reinforcement in accordance with AS/NZS 4671 have on design in accordance with AS1170.4:1993?

Building importance aside, it is universally accepted that the design magnitude of the seismic base shear is directly proportional to the selected level of ductility and energy dissipation characteristics of the seismic load resisting system. For reinforced concrete structures these properties are influenced predominantly by the structural system and geometry, the properties of constituent materials, and the detailing, arrangement, and placement of reinforcement. Appendix B of AS 1170.4 defines a ductile structure as one in which "members and connections can sustain a number of load reversing post-yield deflection cycles whilst maintaining a substantial proportion of their initial load carrying capacity", and refers to Appendix C2 where reinforced concrete, prestressed concrete, and reinforced masonry structures are considered to be ductile. Appendix B also notes that structures with limited ductility should be considered non-ductile. The level of ductility of the structural form is then reflected by selection of the appropriate structural response factor, Rf. The magnitude of these values is empirically based and has been chosen to reflect the expected post-yield performance and response under the design earthquake. The higher the response factor, the better the capacity of the structure to dissipate energy and maintain substantial load carrying capacity during and after a seismic event. By dissipating the energy applied through the base acceleration the "real" forces induced in the structure are reduced. The method of analysis is also irrelevant as scaling of the results obtained from dynamic analyses is a function of the response factor. Table 4.1 is an approximate comparison of normalised response factors for similar structural systems as contained in AS 1170.4:1993, UBC:1991, and NZS 4203 Part 4:1992. The UBC was the base model for the development of the Australian code.

Seismic Load -Resisting System	**Structural Response Factor, Rf**		
	AS 1170.4:1993	**UBC:1991#**	**NZS 4203:1992***
RC Shear Walls	6.0	5.7	2.0 to 9.0
RC OMRF	4.0	3.6	3.0
RC IMRF	6.0	5.7	3.0 to 9.0
RC SMRF	8.0	8.6	6.0
Dual System RC Shear Walls plus RC IMRF	6.0	6.4	2.0 to 9.0
Dual System RC Shear Walls plus RC SMRF	8.0	8.6	2.0 to 9.0

Table 4.1 Comparison of Structural Response Factors

* *Direct comparison with the NZS 4203 is complicated by the response factor being represented by the combination of a structural performance factor, Sp, and the structural ductility factor,* μ *. The seismic acceleration coefficient used is also*

proportional to the structural ductility factor as well as the fundamental period. This standard tabulates values for the ductility factor based on three tiers of structure only; elastically responding, limited ductility, & ductile. The ductility factor for shear walls is a function of wall geometry and seismic base overturning moment.

 # Response factors have been listed from UBC:1991 as a relevant comparison. Some values have been modified in UBC:1997 following the 1994 Northridge Event.

Due to the relatively low seismicity of most regions of Australia, very few commercial structures are designed to resist seismic loads on the basis of detailed plastic theory. In the majority of instances, elastic analysis and design using the ductility response factors contained in Table 4.1 and the deemed to comply detailing requirements contained in AS 1170.4 and AS 3600 the SAA Concrete structures code is deemed sufficient by designers. Therefore, reliance is placed directly on the response factor, Rf, to adequately define an acceptable level of post-yield inelastic performance when default plastic hinges form in the structure. Elastically calculated inter-storey drifts are simply amplified by the deflection factor, Kd.

Potger et.al (2001) proposed a non-linear dynamic time history analysis technique for the calculation of response factors under Australian conditions. The authors' conclusions from this study indicated the current response factors of AS 1170.4 to be conservative, and ductility demand to be negligible under ground accelerations typical in Australia, however cautioned the accuracy of the analytical methods used. No results were provided of curvature and ductility demands exhibited by the frames analysed during the study.

The release of AS 3600:2001 in line with the introduction of AS 4671:2001 has prohibited the use of Class L reinforcement in seismic resisting moment frames due to the low strain capacity of the bar, but has allowed its use in shear walls, subject to the requirements within the relevant section of the standard. For seismic frames designed to AS 1170.4 that may exhibit large deformation under the design earthquake, Clause 1.1.2 of AS 3600:2001 would also prohibit the use of Class L reinforcement in shear walls. The results from the research of Chick et.al (1999) and Patrick et.al (2001) discussed in Section 3.0 would indicate that under a displacement demand of up to 5.5 times the elastically calculated static deflection (for Special Moment Resisting Frames Kd = 5.5) the curvature demand may require reinforcement with uniform elongation characteristics well in excess of 3.5%.

If a consistent approach to seismic analysis and design in Australia is desired, ***is there adequate uniform elongation in the codified limits for Class N reinforcement in accordance with AS/NZS 4671:2001 to allow consistent use of the response and deflection amplification factors of AS 1170.4:1993?***

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DYNAMIC ANALYSIS OF BEAM TYPE STRUCTURES USING ARTIFICIAL RESPONSE SPECTRA

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ABSTRACT

In the seismic analysis of a structure the common practice is to use a site dependent design response spectrum (e.g. response spectrum given in AS 1170.4) as the basic formulation of the ground motion.

More realistic assessment involves the transient dynamic analysis of the structure subject to acceleration time histories. The earthquake ground motions representing Australian conditions, in the form of synthetic acceleration time histories have been made available by Geoscience Australia for assessment. The synthetic waveforms were derived using Empirical Green's Function Method and the records of the 29 December 1989 aftershock in the Newcastle NSW region.

Two categories of dynamic analyses of a beam type structure were performed – transient dynamic analysis, using the synthetic acceleration time history and spectral analysis based on AS 1170.4. The results of both analyses were compared.

Keywords: earthquake; response spectrum; spectral analysis, transient dynamic analysis

1. Introduction

A single earthquake can be characterised in terms of its ground motion waveforms (e.g. acceleration time histories in the three orthogonal directions), which are often available as a result of direct records. Due to a variety of parameters influencing the earthquake process, a single record cannot characterise seismic hazard at one site. The seismic hazard evaluation is usually based on information from a few sources: the recorded ground motion, the history of seismic events in the vicinity of the site and the geological data and fault activities of the region [5]. For most regions of the world this information is very limited and may not be sufficient to predict the size and recurrence intervals of future earthquakes [5]. Nevertheless, seismologists are able to predict the character of typical earthquakes using synthetic seismic waveforms. The calculations of the synthetic seismograph are based on aftershock waveforms and are derived using Empirical Green's Function Method. Such synthetic earthquake waveforms were produced by Geoscience Australia, from the 29 December 1989 aftershock waveforms in the Newcastle NSW region [6]. The Newcastle earthquake synthetic data were used to predict the behaviour of a beam structure (fig. 1) under earthquake loads. Two types of dynamic analyses of the beam structure were performed – (i) transient dynamic analysis, using the synthetic acceleration time history and (ii') spectral analysis based on the derived response spectrum curve from the synthetic data and (ii'') spectral analysis based on AS 1170.4.

2. Data

2.1 *The beam.* The basic dimensions of the beam are shown in fig.1.
The beam consists of internal stiffening beams and skin plates of various thicknesses with stiffeners.

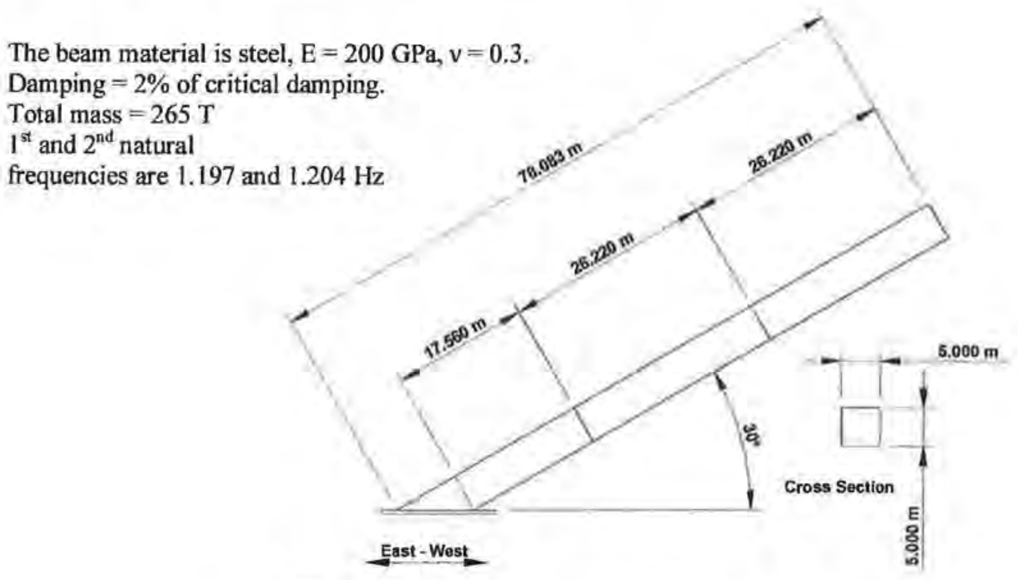

Fig.1. The beam structure geometry

2.2 *Synthetic Waveforms.* The accelerographs shown in figure 2 are based on recordings on rock of the magnitude M_L 2.3 aftershock of the 29 December 1989 Newcastle earthquake and simulate the main shock of M_L 5.6 [6].

The computed maximum ground motion of about 0.25 g is in accordance with a moderate earthquake with an intensity of VIII on the modified Mercalli scale [6].

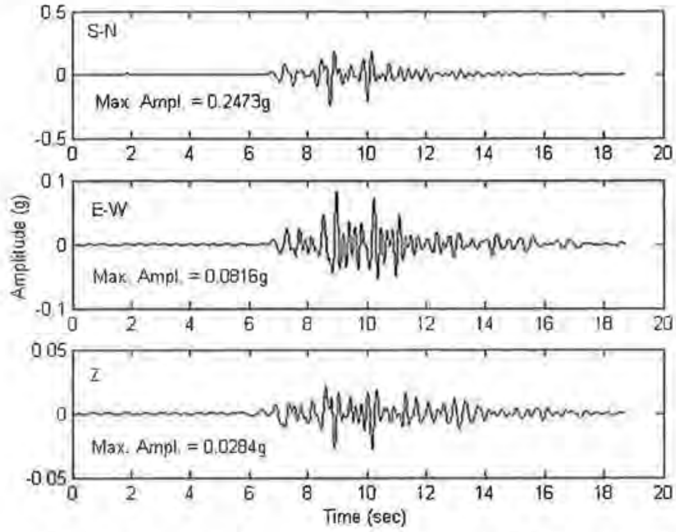

Fig. 2. The synthetic data (Newcastle earthquake)

2.3 *Response Spectra.* The synthetic acceleration time history was converted to a form suitable for the evaluation of maximum actions on the structure. The structure dynamic behaviour was divided into single oscillators, each fully described by its frequency and damping. The synthetic waveform was applied to each of these simple systems and the maximum response (in terms of acceleration) was calculated. The ordinary differential equation describing the behaviour of a single degree of freedom system was solved numerically for 5% damping and periods ranging from 0 to 5 seconds. The synthetic waveform normalised response spectra together with the smoothed response spectrum are given in fig. 3. The AS 1170.4 normalised response spectrum for site factor S = 1 and 5% damping is also shown in fig. 3.

The smoothed response spectrum curves shown below (fig. 3) are defined with the following expressions:

Synthetic response spectrum:	for $0\ s < T <= 0.41468\ s$	$Sa = 3.5$
	for $0.41468 < T <= 3\ s$	$Sa = 0.75/T^{1.75}$
	for $T > 3\ s$	$Sa = 0.10967$
AS 1170.4 response spectrum:	for $0\ s < T <= 0.35355\ s$	$Sa = 2.5$
	for $0.35355\ s <= T < 3\ s$	$Sa = 1.25/T^{2/3}$
	for $T > 3\ s$	$Sa = 0.60094$

2.4*Site and structure factors according to AS 1170.4* The normalised response spectra (the synthetic spectrum and the spectrum given in AS 1170.4) were multiplied by the following scaling factor:

$$\text{Scaling Factor} = gaI/R_f = 9.81*0.08*1/1 = 0.7848$$

Acceleration Coefficient	: a = 0.08	(AS 1170.4, Table 2.3)
Site factor for a rock material	: S = 1	(AS 1170.4, Clause 2.4)

Importance factor	: $I = 1$	(AS 1170.4, Clause 2.5)
Structural response factor	: $R_f = 1$	(AS 1170.4, Table 6.2.6)
Gravitational constant	: $g = 9.18\ m/s^2$	

Note that the R_f factor was not used in the transient dynamic analysis, therefore here it equals 1.

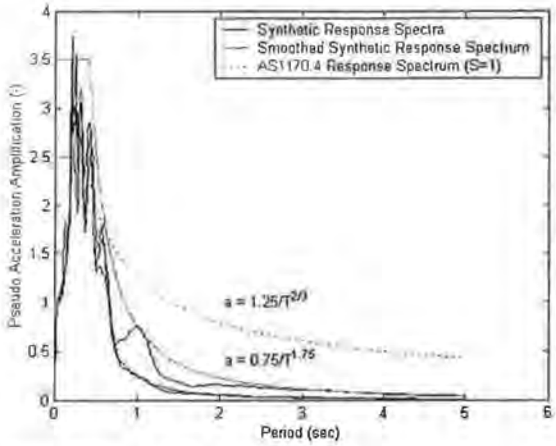

Fig. 3. The normalised response spectra for the synthetic waveforms and 5% damping, and the AS 1170.4 normalised response spectrum for S = 1 and 5% damping

3. METHODOLOGY

The purpose of this assessment is to compare results of dynamic analyses of the beam structure using the synthetic earthquake data and AS 1170.4 approach. Transient dynamic and two spectrum response analyses were carried out.
The maximum bending moments of four internal beam frames of the structure, in the two principal directions have been chosen as the comparison parameter. The bending moments of the vertical sections of the frames (fig. 4) were directly compared. The total load applied to the structure consisted only of the earthquake base excitation F_{eq}. For clarity, the gravity load was not included in the analysis.

3.1 *Transient Dynamic Analysis (i).* The transient dynamic analysis was used to calculate the time history of the dynamic response of the structure subjected to loads in a form of acceleration waves. The acceleration waves as shown in fig. 2 were applied to the base of the beam in the North-South, East-West and Z directions respectively. The maximum bending moments of the beams as shown in fig. 4 were calculated.

3.2 *Spectral Response Analysis (ii').* In this analysis the translational excitation of the base of the beam, in a form of the normalised response spectrum multiplied by the site and structure specific factors (refer to section 2.4), was applied equally in the North-South, East-West and Z directions. The response spectrum is a smoothed curve, produced using the synthetic acceleration waveforms (fig. 3). The maximum response in a form of bending moments of the chosen beams was calculated using both CQC (Complete Quadratic Combination) and SRSS (Square Root of the Sum of the Squares).

3.3 *Spectral Response Analysis (ii'').* This analysis is similar to the previous one. The only difference is that the response spectrum was taken from AS 1170.4 – 1993.

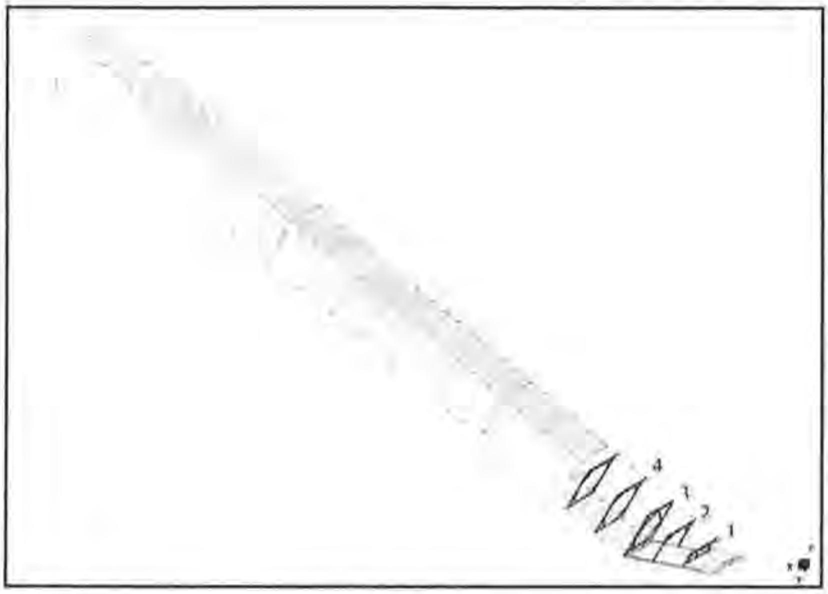

Fig. 4. The FEA model of the beam structure. Analysed internal beam frames are visible.

4. RESULTS

Table 1 – Total Load = F_{eq}

			FRAME 1	FRAME 2	FRAME 3	FRAME 4
TDA (i)	BM1 (Nm)	(-)	115.7 @ 8.75s	115.8 @ 9.00s	1143.8 @ 9.00s	66.4 @10.15s
	BM2 (Nm)	(-)	556.9 @ 10.05s	674.1 @ 10.15s	1220.6 @ 10.15s	951.9 @ 10.15s
SRA (ii')	BM1 (Nm)	S R S S	218.3	343.8	3327.9	200.1
		C Q C	275.4	419.6	3363.6	215.0
	BM2 (Nm)	S R S S	702.2	1134.3	2489.8	3023.5
		C Q C	739.8	1159.9	2667.2	3075.2
SRA (ii'')	BM1 (Nm)	S R S S	276.3	448.6	4283.5	242.3
		C Q C	358.1	551.4	4338.5	245.1
	BM2 (Nm)	S R S S	570.2	1152.3	2858.6	3697.4
		C Q C	629.4	1191.7	3064.6	3775.7

where:

F_{eq}	:	Earthquake load
TDA(i)	:	Transient dynamic analysis results
SRA(ii')	:	Spectral response analysis results (synthetic response spectrum)
SRA(ii'')	:	Spectral response analysis results (AS1170.4 spectrum)
BM1	:	Bending moment in principal direction 1
BM2	:	Bending moment in principal direction 2

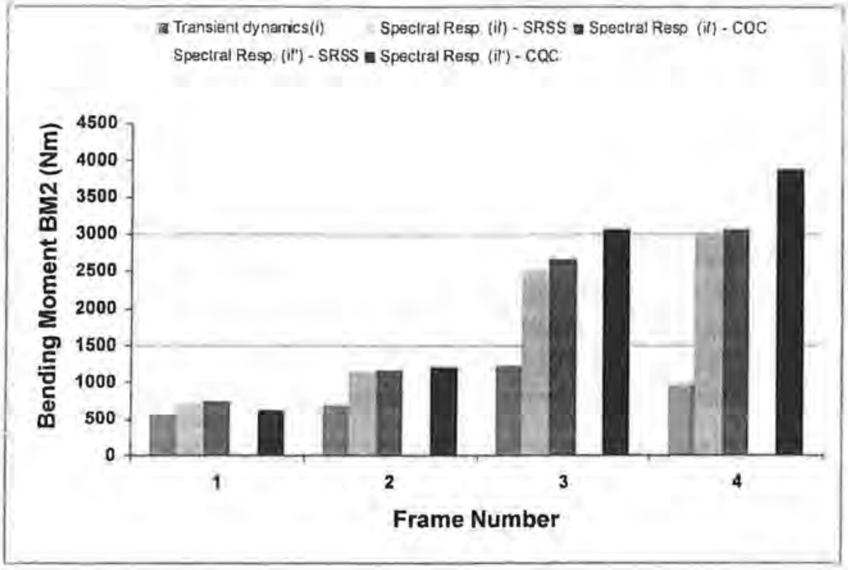

Fig. 5. Bending moment BM1 – comparison of the calculation results.

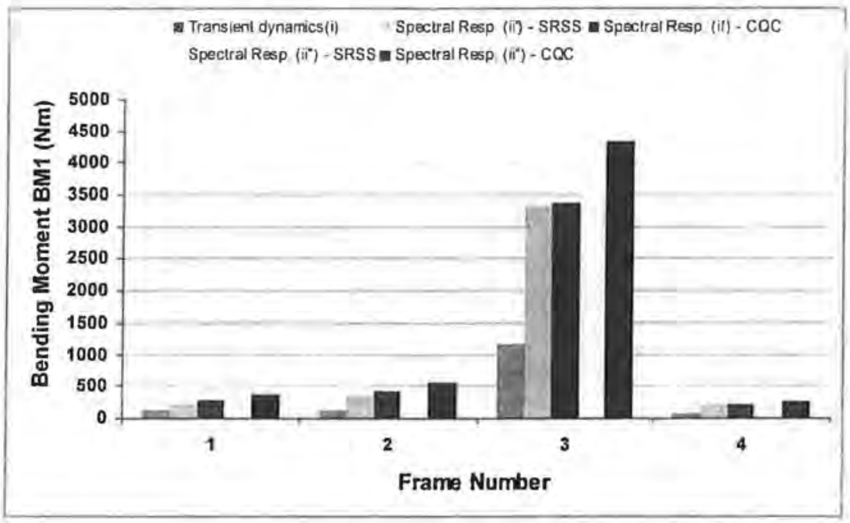

Fig. 6. Bending moment BM2 – comparison of the calculation results.

5. CONCLUSIONS

- Transient dynamic analysis is considered as the most realistic. The results obtained with this approach are treated here as a benchmark for the spectral response analyses. The level of conservatism obtained using the spectral response method is measured as the deviation from the transient dynamic results.

- The analysis of the beam structure illustrates that using the synthetic earthquake data in a form of the response spectrum conservative results can be obtained.

- The synthetic response spectrum has higher magnification factor than the spectrum given in AS 1170.4 at frequencies above approximately 1.6 Hz. For the frequencies below 1.6 Hz the synthetic response spectrum lies below the standard spectrum. The beam natural frequencies (analysed) cover the range between 1.197 Hz to approx. 40 Hz (100 modes). The synthetic spectrum has relatively high magnification at frequencies above 1.6 Hz. The modes below 1.6 Hz are subject to higher amplification by the AS 1170.4 spectrum, therefore, the difference between the results of both analyses is small. Although the results obtained using the synthetic data are similar to the AS 1170.4 analysis, it cannot be concluded with absolute certainty that the frequency content in the synthetic spectrum is sufficient.

- The low frequencies deficiency in the synthetic spectrum may become significant for the structures with low natural frequencies.

- The synthetic response spectrum and the acceleration time history should be further validated analytically and against real earthquake data to confirm or dismiss the need for higher content of low frequencies in the spectrum.

- The differences in the results of the transient dynamic and spectral response analyses are due to the method (SRSS and CQC methods) of calculating the structure maximum response. Both, the SRSS and CQC give approximate results and calculate the maximum response of each member that may correspond to a different point in time. Moreover, the terms computed by SRSS and CQC methods do not retain their signs.

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Acknowledgment: The authors would like to express their gratitude to Mr Kevin McCue for reading the manuscript and valuable suggestions.

SOME KEY EMERGENCY MANAGEMENT ISSUES RELATING TO EARTHQUAKES

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ABSTRACT:

The immediacy and broad-ranging consequences of earthquake events pose significant emergency management problems in terms of preparation for, response to and recovery from impact. The key factor that has the potential to turn a seismic event into a disaster is its impact upon the community. Since the extent of impact on the community is a function of that community's vulnerability, effective emergency management, with its focus on community safety, aims to raise the resilience of the community, before, during and after impact under an overarching focus on mitigatory activities.

This paper raises some of the issues that need to be addressed in order to make communities more resilient to earthquakes.

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Paper No. 5

1. INTRODUCTION

History testifies that the risk of devastating earthquake in San Francisco is greater than that in Australia. Those living in San Francisco, however, probably know to instinctively *drop, cover, hold on* and most likely inhabit buildings constructed in accordance with a rigorous earthquake code. By contrast, an Australian might do little more than pause whilst mesmerised by the rumbling and may frequent buildings of questionable structural integrity. Who is more vulnerable, one might ask?

2. WHO SHOULD BE TOLD?

Early in 2001, a seismologist from the China Seismological Bureau and another from the University of Queensland both predicted that during the latter half of 2001, or early 2002, South Australia would experience an earthquake approaching Magnitude 6, and that this earthquake had the potential to impact settled areas of the State, including metropolitan Adelaide. This prediction was, in confidence, initially revealed to the Seismology Section of Primary Industries and Resources South Australia. Not surprisingly, the question which immediately arose from receipt of such a prediction focused on who else should be told.

As with all hazards which have the potential to have horrific impacts, the motive behind informing members of the public should be to empower them to take appropriate action to prevent or minimise the risk of the hazard impacting on them and to respond appropriately to the impact should it occur.

It could be argued that to inform people about horrific hazard impacts without empowering them to undertake appropriate personal intervention strategies is to do them a great disservice. The void that marks the absence of personal empowerment may instead be filled by fear and panic. Should this happen, not only might the safety of the public be directly threatened, but as well, other adverse consequences may follow. It is reasonable to speculate that at no time throughout history has panic ever been an effective emergency management strategy.

In response to the above earthquake prediction, the following course of action was adopted:

1. The prediction was considered confidential and the public was not told.
2. Appropriate key personnel in Government and the State Disaster Organisation were informed concerning the prediction.
3. Shortly after receiving the prediction, the State Disaster Organisation practised its arrangements to deal with hazard impacts, in particular, earthquakes, in order to determine its readiness and to rectify shortfalls.
4. In an attempt to assess how credible the prediction was, the opinions of several leading seismologists of international standing in the field of earthquake prediction were obtained.

From time to time, when an occasional noteworthy tremor does occur in South Australia, it gains coverage in the media and may prompt excited anecdotes over morning tea, but after a few days earth tremors drop below the horizon of public interest. Encouraging communities to undertake what ought to be routine mitigatory activities relating to bushfires is difficult enough even though, in South Australia, the bushfire threat recurs every summer. How much more difficult it is to encourage the public to be pro-active about protection from earthquake. Any attempt to persuade the South Australian community to undertake mitigatory activities (for example, strapping the domestic water heater to its adjacent wall) is most likely to be ignored and, as in the case of the installation of smoke detectors, would need the coercive muscle of legislation for appropriate measures to be implemented.

3. EFFECTIVE LEGISLATIVE FRAMEWORK

Clear delineation of responsibilities and accountabilities needs to be in place, and practised, before the event. This applies not only at the level of individual organisations, but also at Government level. To use, as an exemplar, the events that followed the Kobe earthquake; "Japan's Disaster Relief Law, empowering the national Government to intervene in cases where 'abnormal and massive disasters gravely affect the national economy and public welfare', could not be invoked because it required a vote of the Diet (Parliament), and the Diet was not sitting.
Five days after the quake, no national or prefectural emergency headquarters had been established in Kobe. More than 1000 local government and private organisations were still milling around in uncoordinated chaos. No one was in charge. Nearly a week later, hospitals were still watching patients die because they had no water or drugs and the operating theatres were closed.
This paralysis of leadership in an emergency cost many hundreds of people their lives. While Tokyo-based bureaucrats from 23 different ministries and agencies fought vicious turf battles over the responsibility, people who should have been saved were dying in the ruins." (1)

Clearly, there is a need for workable and flexible legislation and consequent planning to provide a mandate for appropriate emergency response activities to be undertaken without the need to jump bureaucratic hurdles and battle conflicting personalities during disaster response.

4. THE NEED TO EXERCISE

"Disaster management" involves a much broader range of issues and strategies than those generally linked with the management of "day-to-day emergencies". Consequently, disaster management must involve a wider and more diverse range of agencies and interests, e.g. Treasury, Health and Welfare (Human Services), engineering, agriculture, transport, community-based service agencies, various support programmes and industry and commerce.

Although many diverse agencies are required to cooperate and work together in a coordinated manner during a disaster, on a normal day-to-day basis they may rarely interact. One cannot expect agencies to interact effectively during times of disaster if they rarely or never practice doing so beforehand. If mutual assistance and interaction are to be effective in the strained environment arising from earthquake impact, then agencies must practise exercising together. Experience shows that at least some sectors of private enterprise have a hard-nosed "time is money" attitude and show an unwillingness to "waste" time participating in exercises. Sir John Harvey-Jones, a former chairman of ICI is reported to have said, "Planning is an unnatural process: it is much more fun to do something. The nicest thing about ***not*** planning is that failure comes as a complete surprise, rather than being preceded by a period of worry and depression."

Thorough planning is essential, and exercising is part of the planning process. Participation in exercises needs to be perceived as an integral part of an agency's core business rather than merely an annoying optional extra.

5. DEFINING THE SCOPE OF THE IMPACT

Like it or not, disaster response always operates within a "fog" of confusion. That "fog" may be dense or sparse, and may vary significantly from emergency to emergency, but there is no avoiding the reality that there ***will*** be "fog". The delay in knowing what is going on depends on a range of factors including the effectiveness and resilience of communication infrastructure, the time of day and the weather, etc.

Those whose responsibility it is to coordinate disaster response are initially faced with determining, as quickly as possible, what the scope of the problem is.

It is reported that the emergency manager in charge of the response to the Kobe earthquake was asked what he would do differently if he could wind the clock back and start all over again. Allegedly, he admitted that he had allowed his resources to be exhausted before the scope of the problem was realised and added that he wouldn't make the same mistake again.

Because of the immediacy of earthquake impact, a blanket of "fog" instantaneously shrouds unfolding events, heralding the arrival of a time of confusion and, to some extent at least, chaos. The challenge for Emergency Managers is to pursue the development and implementation of resilient information gathering, interpreting and dissemination networks to enable that "fog" and chaos to be pared back to a minimum.

6. PRESERVATION OF CORPORATE MEMORY

Like other organisations throughout the community, many of those responsible for responding to disasters experience staff reductions, with consequent loss of "corporate memory."

Corporate memory refers to the technical and procedural experiences developed and shared informally by an organisation's personnel. These include the experience acquired, sometimes at considerable cost, regarding which processes and methods work in the "real world" and which don't. [2]

The preservation of corporate memory is a challenge to any organisation and lifeline (critical infrastructure) organisations in particular. Earthquakes may damage part of a lifeline infrastructure not "visited" for many years and the people who worked on it and knew the nuances associated with it may have long since retired. However challenging its retention may be, corporate memory nevertheless is a vital resource which needs to be able to be readily tapped during a disaster, and therefore organisations need to implement strategies to ensure it is captured for possible future use.

7. MITIGATION AS THE WAY AHEAD

Figure (1) outlines the now superseded strategy for managing emergencies which involved a heavy emphasis on HAZARDOUS EVENTS occurring. As a direct consequence of such an emphasis, acquisition of resources to provide a comprehensive emergency response capability occurred at the expense of thorough risk management and risk prevention. As a further consequence, because risk management was not conducted as well as it might have been, the actual level of risk resulting from hazard impacts increased.

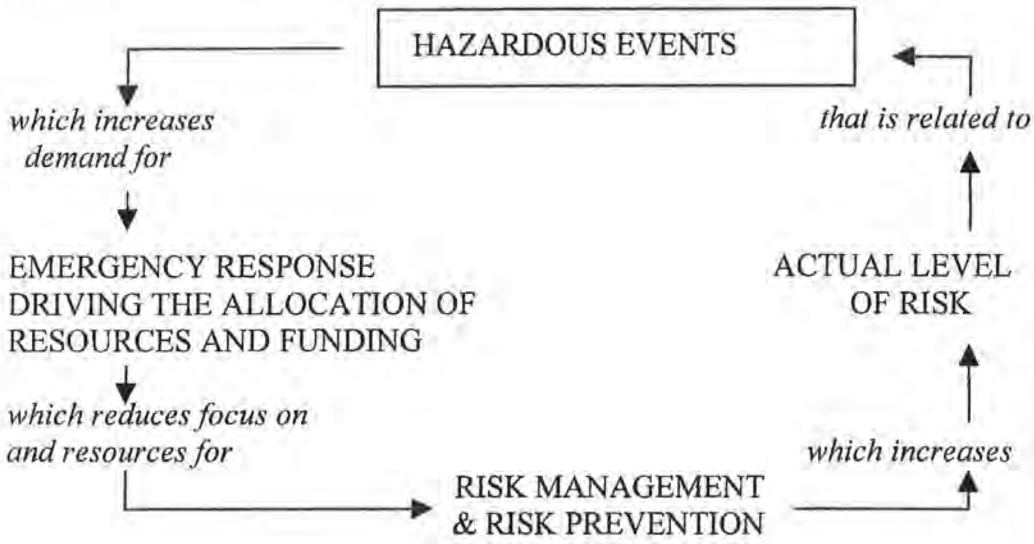

Figure 1

Nowadays, not only in South Australia, but nationally as well, the emergency management emphasis is on mitigation (stopping or minimising the hazard impact). In order to do this effectively, the current strategy for dealing with emergencies puts the focus on Risk Management and Risk Prevention (Figure 2).

So the logic goes, if these tasks are done thoroughly then the risk from hazard impacts will reduce. As a consequence, with fewer hazard impacts, it can be argued that there is justification in reducing emergency response resources, (whilst also recognising that there will always be a need to have some capability to respond to hazard impacts, because no matter how effective risk management strategies are, there will always be some occasions on which hazard impacts occur).

The overall reduced expenditure on response activities enables more funds to be released for risk management.

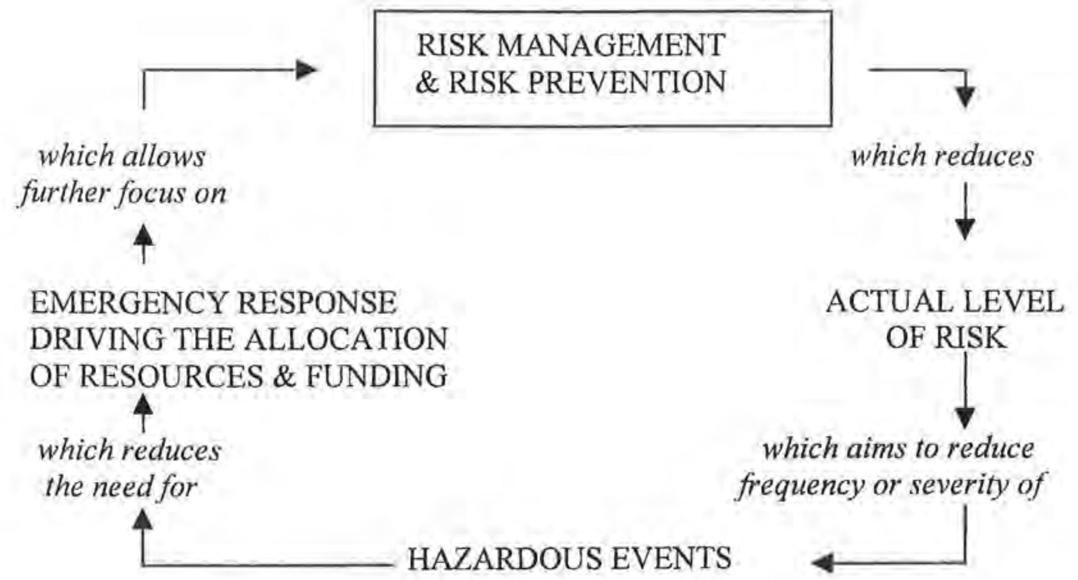

Figure 2.

Effective response to an earthquake impact may demand the speedy deployment of an abnormally large quantity of diverse resources. The emergency risk management process, properly applied, will include the development of strategies to ensure that such required resources, which include those held by Police, Emergency Services, critical infrastructure agencies, and those organisations addressing health and welfare, can readily be acquired and deployed. Such strategies, of necessity, would involve an ongoing commitment to the planning process and could consolidate mutual aid agreements with other Australian States as well as with, to some extent at least, the international community.

8. CONCLUDING REMARKS

The infrequency of significant earthquakes in those regions of the world that are commonly viewed as seismically safer areas may give rise to a latent community vulnerability. Evidence for such vulnerability can be found in apathy, ignorance and a lack of will to plan for earthquake impacts. Faced with such overall disinterest, implementing strategies to reduce such vulnerability presents significant challenges to emergency managers.

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COMPARING THE PREDICTIONS OF THE COMPONENT ATTENUATION MODEL WITH REAL AUSTRALIAN EARTHQUAKES RECORDS

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ABSTRACT:

The lack of instrumented earthquake data in Australia means that a representative response spectrum attenuation model cannot be developed from strong motion accelerogram data by conventional regression analysis. There are also inadequate seismogram records in Australia to develop a seismological model, or stochastic model. Consequently, attenuation models developed from overseas have been used in Australia. When choosing a "representative" model, regional conditions must be taken into account. However, this could have been handled in a more systematic and transparent manner. The Component Attenuation Model (CAM) provides a systematic way of combining generic information from overseas models with local information. CAM expresses the response spectrum in terms of the product of a source function and numerous path functions. The source function is generic but could be modified in accordance with observations from local earthquakes. Path functions should account fully for local conditions. For example, the Quality factor (Q) has been incorporated into the 2002 version of CAM as an input parameter for the path attenuation factor. In this paper, the response spectra predicted using CAM are compared with both instrumented and/or macro-seismic field measurements from the Ellalong earthquake, Newcastle earthquake and the Tennant Creek aftershock. All these events were in the magnitude range of 5 to 5.5. Further comparisons are also made with the well known attenuation model of Toro and Sadigh. Significantly, the scaling relationships built into CAM allows realistic predictions for events of much larger magnitude.

Paper No. 6

1. INTRODUCTION

In intraplate regions such as Australia, developing response spectrum models by the regression of representative strong motion data recorded locally is not infeasible due to the paucity of data. Consequently, response spectrum models for these regions are typically based on overseas codified models (refer commentary in AS1170.4: 1993) or attenuation models such as those developed by Toro (1997) and Sadigh (1997). The choice of a "suitable" model requires experienced seismologists to exercise judgement based partly on the comparison with data recorded locally (typically from small earthquake events and tremors). However, there are difficulties in extrapolating ground motion properties for a large earthquake event from observations of small events (Gibson, 1995). Intensity information from iso-seismal maps of major local historical events has been used extensively in developing local attenuation models (eg. Gaull, 1990), although it is recognised that there are considerable scatters associated with intensity information due to uncertainties in site effects and building vulnerability.

This suggests that intraplate response spectrum models have been developed in a somewhat *ad-hoc* and often non-transparent manner with a significant amount of professional judgement. Recommendations developed from a non-transparent process can lead to problems in code development and implementation (Wilson, 2001). Addressing this, code recommendations for Australia have been presented recently in a more transparent format (Lam, 2001a). The global trend in earthquake engineering towards a performance-based approach is associated with the need for a better understanding of both seismic hazard and system behaviour.

The seismological model which was pioneered by eminent seismologists in the US in the early 80's defines earthquake ground motion properties in terms of the Fourier spectrum (eg. Boore, 1983). The model which was originally based on a simple theoretical framework has been further developed based on the analysis of a large volume of representative seismological data (eg. Atkinson, 1993,2000). This semi-empirical approach relies mainly on low intensity far-field measurement and waives the need to regress strong motion accelerogram data. Thus, it has had special appeals for applications in the low seismicity region of Central and Eastern North America (CENA). Importantly, the model provides a platform from which seemingly complex earthquake processes can be modelled and interpreted in relatively simple terms.

Lam and Wilson have recognised the potential of the seismological model for a worldwide application and transformed the Atkinson version of the model developed for the CENA region from the original Fourier spectrum format into the engineering response spectrum format (Lam, 2000a-c). Whilst Australia could develop a seismological model of its own from local seismological data, a more viable approach is to adapt and verify the already developed models.

This seismological-engineering model, known as the Component Attenuation Model (CAM, as outlined in Section 2) represents various source and path effects by separate component factors so that variations in regional conditions can be readily accomodated. Predictions from CAM are compared with both Australian field observations in Section 3 & 4 and with the well known attenuation models of Toro (1997) and Sadigh (1997) in Section 5.

2. OVERVIEW OF CAM

The Component Attenuation Model (CAM) provides attenuation relationships for three response parameters which can be used to construct response spectrum in either the response spectral acceleration (RSA), response spectral velocity (RSV) or response spectral displacement (RSD) formats (Lam, 2000b). Due to length limitations, this paper considers only the $\boldsymbol{RSV_{max}}$ parameter which defines the highest point on the velocity response spectrum. $\boldsymbol{RSV_{max}}$ controls the response spectrum behaviour in the period range of greatest engineering interests and relates directly to the peak ground velocity (***PGV***). The scaling of the design response spectra in AS1170.4 (1993) is effectively based on these two parameters.

RSV_{max} (mm/sec) is defined in CAM by Eq.1 as the product of numerous component factors:

$$RSV_{max} = \alpha(M).G(R,D).\beta(R,Q,M).\gamma_{mc}.\gamma_{uc} \qquad \text{(soil factors not shown)} \qquad (1)$$

where $\alpha(M)$ represents the effect of earthquake magnitude as shown in equation (2).

$$\alpha(M) = 70\left(0.35 + 0.65(M-5)^{1.8}\right) \qquad (2)$$

Although Eq.2 models the effect of magnitude only, the α factor could be extended to account for variations in the type of faulting and regional stress-drop characteristics. The equation was developed for the magnitude range M5-M7.5 for distances exceeding 10km although good correlation has been obtained for larger earthquake magnitudes (Bala,2002; Lam,2002b). M is defined as the moment magnitude as opposed to the usual local magnitude which has historically been used to characterise Australian earthquakes. Disparities between the two magnitude scales arising from saturation are only significant for magnitudes well in excess of 6.

$G(R,D)$ represents the effect of geometrical attenuation where R is the nearest distance between the site and the surface of fault rupture and D is the depth of the Moho discontinuity. The model is not intended for handling near-fault conditions associated with directivity effects (which are manifested in the form of elliptical isoseismal contours) and hence R should preferably be at least 10km for M>5 and 20km for M>6. G can be simplified as follows for 10km<R<50km:

$$G(R) = {}^{30}\!/_{R} \qquad (3)$$

The anelastic attenuation factor β which represents energy dissipation effects incorporates the Quality factor (Q) as an input parameter to account for regional variations in the wave transmission properties of the earth crust. Long distance attenuation of up to 500km can be modelled by the full expression for the β factor as presented in Lam (2002a) and reproduced diagrammatically in Figure 1. The effect of Q is shown to be critical only when R>100km. β can be approximated to unity for distances up to 50km. For greater distances, information on Q as obtained by studies such as that undertaken by Wilkie (1995) can be critical to modelling accuracy.

The mid-crustal factor γ_{mc} is given by Eq.4 which has been simplified from wave theory.

$$\gamma_{mc} = \sqrt[3]{{}^{3.8}\!/_{V_{source}}} \qquad (4)$$

where V_{source} is the estimated shear wave velocity at the depth of rupture which is typically less than 10km in Australia. The values of γ_{mc} for the "hard rock" conditions of CENA and the "rock" condition of WNA are 1.0 and 1.3 respectively based on average crustal properties (Boore,1997).

The upper-crustal factor γ_{uc} which can be calibrated in accordance with the regional crustal shear wave velocity gradient is estimated at 1.0 and 1.25 for the generic shear wave velocity profiles identified for CENA and WNA respectively. A full expression to allow for intermediate regional conditions has yet to be developed. Special crustal effects such as basin edge effects have not been considered. Additional site factors representing the effects of soil resonance have been developed (Lam, 2001b) but is not shown herein. Further details on the modelling of site effects could also be found in the companion paper (Srikanth, 2002). Full details for each of the equations listed in this section and their derivations can be found in Lam (2000b&c, 2002a).

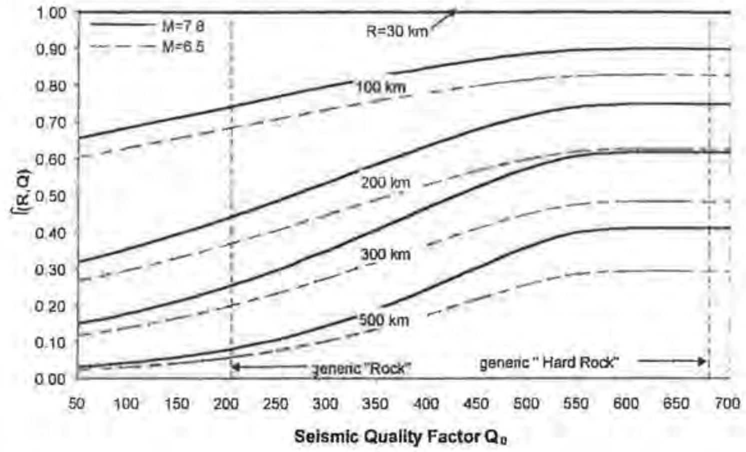

Figure 1 β factor representing effects of energy dissipation (Chandler, *submitted*)

RSV_{max} obtained from Eq.1 can be related directly to the peak ground velocity (***PGV***) by Eq.5. The inferred ***PGV*** values can then be compared directly with seismogram measurements and with recorded Intensity information using the "Newmark-Rosenblueth" approximate expression of Eq.6.

$$PGV \cup 0.5 RSV_{max} \qquad (5)$$

$$2^{MMI} \cup {}^{7}\!/_{5}\, PGV \qquad (6)$$

3. COMPARISON WITH PRE 1984 ISOSEISMAL RECORDS

Information from iso-seismal maps collected from a number of Australian earthquakes including the M6.9 Meckering earthquake of 1969 and the M6.2 Cadoux earthquake of 1979 have been used to study earthquake attenuation behaviour (Gaull, 1990). In view of variations in crustal properties across the continent, separate attenuation relationships were developed for (i) Western Australia (WAus) which is characterised by hard rock conditions typical of mid-continental regions like CENA and (ii) Southeastern Australia (SEAus) which is characterised by softer rock conditions similar to WNA. The developed relationships are shown by Eqs. 7a & 7b respectively.

$$MMI = 1.5M_L - 3.2\log R + 2.2 \qquad \text{(WAus)} \qquad (7a)$$

$$MMI = 1.5M_L - 3.9\log R + 3.9 \qquad \text{(SEAus)} \qquad (7b)$$

In Table 1, the ***PGV's*** predicted from Eqs.6,7a&7b (Gaull) and Eqs.1-4 (CAM) are compared.

Table 1 Predicted PGV's (in mm/sec) from the relationship of Gaull and CAM

M-R(km)	Average Australian sites (Gaull,1990)			USA hard rock and rock sites (CAM)		
	WAUS	SEAUS	**crustal factor**	CENA	WNA	**crustal factor**
M5 R30	22	36	**1.64**	12	20	**1.67**
M5.5 R30	38	60	**1.58**	20	30	**1.50**
M6 R30	64	100	**1.56**	35	56	**1.60**

PGV values for hard rock ("WAus" and "CENA" columns) and rock ("SEAus" and "WNA" columns) by the two independent sets of relationships each differ by a factor of 1.8-2.0. This difference is considered to reflect the site amplification effects inherently associated with the models presented by Gaull. Interestingly, the implied site factor is very consistent across different crustal conditions and earthquake scenarios. The crustal factor which has been normalised to unity for hard rock conditions represent the amplification of motion intensity in softer crustal conditions. The factors as inferred from the Gaull's relationship for Australian earthquakes (ratio of ***PGV's*** for Rock : Hard Rock) were also highly consistent with similar factors inferred from CAM based originally on North American earthquakes. Furthermore, the rate of increase in ***PGV*** with increasing magnitude was highly consistent between the two sets of relationships. For example, the ***PGV*** predicted for a M5.5 event was consistently 1.5-1.7 times that of the same predicted for a M5 event. The ***PGV*** ratio for a M6:M5 event was also consistently 2.8-2.9. In summary, the source factor (Eq.2) and the crustal factors (γ_{mc} and γ_{uc}) of CAM appear to be very consistent with the intensity observed from historical earthquakes in Australia.

4. COMPARISONS WITH RECORDS FROM MORE RECENT EARTHQUAKES

Intensity information recorded from the M5.6 Newcastle earthquake event was more precise due to proximity to built-up areas and the destructive nature of the earthquake. Significantly, ***MMI*** observed on alluvial sites and rock sites could be distinguished for that event with rock sites typically showing a ***MMI*** value of VI-VII at an epicentral distance of 15km (Map no. 3 of Melchers, 1990). The $\boldsymbol{RSV_{max}}$ value calculated using CAM for the earthquake at an epicentral distance of 15km for "rock" conditions (ie. γ_{mc}=1.3 and γ_{uc}=1.25) is 136mm/sec from Eqs.1-4. The inferred ***PGV*** and ***MMI*** is accordingly 68mm/sec (Eq.5) and VI-VII respectively (Eq.6) which is very consistent with the field observations. Further, the higher ***MMI*** value of VIII observed on alluvial sites was also very consistent with a site magnification factor of 2 inferred from the comparative study with Gaull as described in Section 3.

Even more precise ground motion information was obtained from the M5.3 Ellalong earthquake of 1994 where some five seismographs located on rock sites within 40-50km of the epicentre were activated with a ***PGV*** averaging around 10mm/sec recorded (McCue, 1995). The PGV calculated from CAM was 15mm/sec (assuming the same crustal condition as for the Newcastle earthquake). The slightly lower recorded ***PGV*** is believed to be due to the geology of the Sydney basin which is characterised by a thick layer of soft rock sediments with high energy absorption properties. The significant of this energy dissipation effect increases with increasing distance.

In contrast to the Newcastle earthquake and the Ellalong earthquake, the Tennant Creek earthquake of 1998 occurred in the intercontinental region of Central Australia which is characterised by hard rock conditions similar to CENA. The velocity response spectrum recorded from the M4.9 aftershock at an epicentral distance of 10km was also highly consistent with the predictions by CAM as shown in Figure 2 (and reported in Lam, 2000c). Note, the difference in the corner period (T_1) between Figure 2 for hard rock and the illustrations in Koo (2000) and Lam (2001a) for rock.

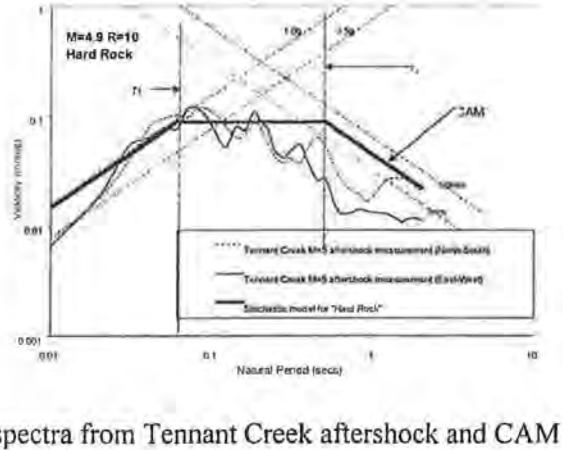

Figure 2 Response spectra from Tennant Creek aftershock and CAM predictions(Lam, 2000c)

5. COMPARISONS WITH PREDICTIVE MODELS BY TORO AND SADIGH

Further comparisons are made between CAM and the predictive models of : (i) Toro (1997, model developed for mid-continental earthquakes within CENA and representing hard rock conditions and (ii) Sadigh (1997, model developed for shallow Californian earthquakes and representing rock conditions. Predictions are listed in Table 2 under separate groupings for "hard rock" and "rock" conditions, assuming reverse faulting.

Table 2 $\boldsymbol{RSV_{max}}$ (and $\boldsymbol{PGV}$) predictions in mm/sec

Earthquake Scenarios	Hard Rock		Rock	
	CAM (hard rock)	Toro	CAM (rock)	Sadigh
M5 R=30km	25(13)	25(13)	39(20)	26(13)
M5.5 R=30km	38(19)	41(20)	60(30)	46(23)
M6 R=30km	70(35)	77(38)	112(56)	82(41)

A reference distance of 30km has been adopted in the comparison to circumvent complications arising from near fault effects. Predictions for other distances could be made using Eq.3 and the full expression for β where necessary (refer Section 2). Excellent correlation was observed between the CAM (hard rock) model and the model of Toro for similar crustal conditions. However, predictions by Sadigh (97) for Californian earthquakes of $\boldsymbol{M}$=5 are some 35% lower than predictions by CAM for similar crustal conditions. The lower values associated with Californian earthquakes is postulated to have resulted from stress drop which is generally lower than what was implicit in the CENA database (which is the basis of the Atkinson model and CAM). Interestingly, this regional variation seems to diminish with increasing magnitude. For example, predictions by the two models differ only by 25% at M=6, and show little differences at M7.5 (Lam, 2000c). Refer Figure 3 for a 3D diagrammatic illustration.

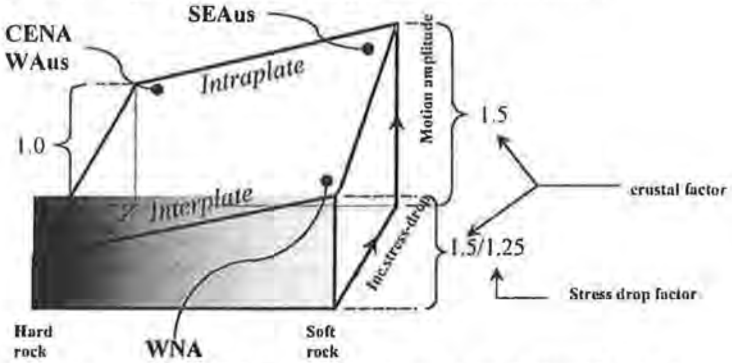

Figure 3 Crustal factor and stress drop factor (M=5.5)

6. CLOSING REMARKS

The comparative study presented in this paper confirms the robustness of the CAM relationships. Improved estimates for the path effects could be achieved through studying crustal conditions and combining regional seismological information with geological information. Source effects are less predictable but useful trends could be revealed by collating and analyzing near-field information collected worldwide. Difficulties associated with the collection of adequate earthquake data in intraplate regions could be alleviated through such research efforts. CAM is conceptually different to most existing attenuation models in that it is not based on a defined database of earthquake data. CAM instead provides a convenient platform on which relevant research contributions to ground motion modeling from different sources could be used and presented in a transparent format.

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MICROTREMOR SURVEY DESIGN OPTIMISED FOR APPLICATION TO SITE AMPLIFICATION AND RESONANCE MODELLING

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ABSTRACT

Microtremor surveys at their simplest level of measurement (single three-component geophones) provide the natural period of site resonances which may be used as a first-order tool in zonation of earthquake risk. At a second level of measurement, the use of a circular array of geophones enables calculation of the phase velocity of propagation of microtremor energy, from which estimates of the shear-velocity profile and thickness of unconsolidated sediments can be derived without recourse to invasive site testing such as drilling or cone-penetrometer measurements.

The shear-velocity profile and thickness of sediments obtained from microtremor data are suitable as direct inputs into computation of site amplification and resonances using SHAKE software. This paper shows that site-resonance data alone is insufficient for adequate calculation of site-amplification effects, but site-resonance data combined with microtremor velocity information is a potentially powerful micro-zonation methodology.

1. INTRODUCTION

The use of high-frequency background seismic noise, known as microtremors, for site resonance classification is well established; see the companion paper in this volume (Asten and Dhu, 2002) for a review of information available from single-station measurements of horizontal and vertical microtremor spectra.

Traditional micro-zonation methodology based on the Nakamura (1989) method uses the fundamental site resonance period (T_s) to characterize the potential site hazard levels. Site resonance can result in significant amplification of the structural response but it can be suppressed very effectively by damping or other energy dissipation mechanisms in the structure. For this reason, aseismic design of ductile construction in active regions does not normally include resonance considerations in their design calculations. In contrast, non-ductile or limited ductile structures typical of low and moderate seismicity regions such as Australia are particularly vulnerable to site resonance due to the poor energy absorption capacity in these structures. For such conditions, T_s seems to be particularly important since the higher the value of T_s the higher the displacement demand of structures found on the soil surface.

However, T_s on its own is not fully indicative of the extent of resonance related amplification which also depends on period shift, hysteretic damping and radiation damping at the soil-rock interface, amongst other factors. Period shift and hysteretic damping are the result of non-linear behaviour in the soil layers. The extent of such behaviour depends on soil depth and properties such as the plasticity index. The presence of a very soft soil layer could also result in anomalous response behaviour of the site. Radiation damping at the interface between two wave transmission media (ie rock and soil) depends on their wave velocity contrasts. For example, severe resonance is often resulted from waves trapped within a very soft (low shear-wave velocity) soil medium that overlies a very hard (high shear-wave velocity) bedrock medium. In contrast, resonance is muted when substantial amount of wave energy is dissipated into "soft" bedrock.

The multitude of factors influencing soil responses as outlined above suggests a major drawback in the traditional approach of using T_s as the sole parameter to define the potential site hazard. There are further obvious drawbacks where, for example, the upper soil layers are to be excavated for basement construction. Information related to soil depth, soil properties and shear wave velocity of individual soil layers and bedrock is traditionally obtained by site drilling and testing of soil samples. Whilst multiple site drilling may be justified for major construction projects, employing the same approach for micro-zonation studies (which typically cover a very large areas) would be too costly and is generally not practical.

As pointed out earlier, mapping of the T_s from single-station micro-tremor monitoring is not on its own sufficiently indicative of the soil profile and hence cannot be used as effective substitutes for conventional site drilling. The multiple-station non-invasive micro-tremor monitoring method demonstrated in this paper offers a viable solution to this problem. This new monitoring procedure produces the shear wave velocity profile and not just T_s. Such profiling can assist in identifying the entire geological formation of the site when properly integrated with existing borehole information.

This paper considers three soil sites which possess similar fundamental site period but with very different soil depths and shear wave velocities. It is first demonstrated in Section 2 that single-station measurements of the H/V ratios (based on the conventional Nakamura approach) could not distinguish between the three sites. In contrast, the coherency measurements obtained from a circular array of six geophones as presented in Section 3 managed to identify the site shear wave velocity profile with good resolution. The engineering significance of distinguishing between the sites is demonstrated in Section 4 which compares soil response spectra associated with the individual shear wave velocity profiles.

2. NATURAL PERIOD METHODS

The seismic energy associated with microtremors propagates principally in surface-wave modes. Measurement of vertical-component seismic noise excludes Love modes and allows study of the data in terms of Rayleigh modes of propagation. Rayleigh-wave energy propagates with elliptical particle motion, where the ratio of horizontal to vertical-motion particle motion is dependent on wave period and the compressional and shear elastic parameters of the earth. At shear-wave resonance frequencies the particle-motion ellipse tend to degenerate into dominantly horizontal motion, hence the use of H/V spectral ratios is a very useful guide to the *period* of such resonances. However as demonstrated by Lachet and Bard (1994), the spectral ratio is a poor indicator of *amplification* at the resonance period, since amplification is also a function of Poisson's ratio.

Figure 1 shows the H/V spectrum from a recording over Barrier Sand at Blacksmith, south of Newcastle (NSW). The thickness of sand and weathered bedrock is a minimum of 30 m (SCPT test) and may be up to 60 m. The spectrum shows a strong maximum at period 0.85 s, with a lesser secondary peak around period 0.3 s.

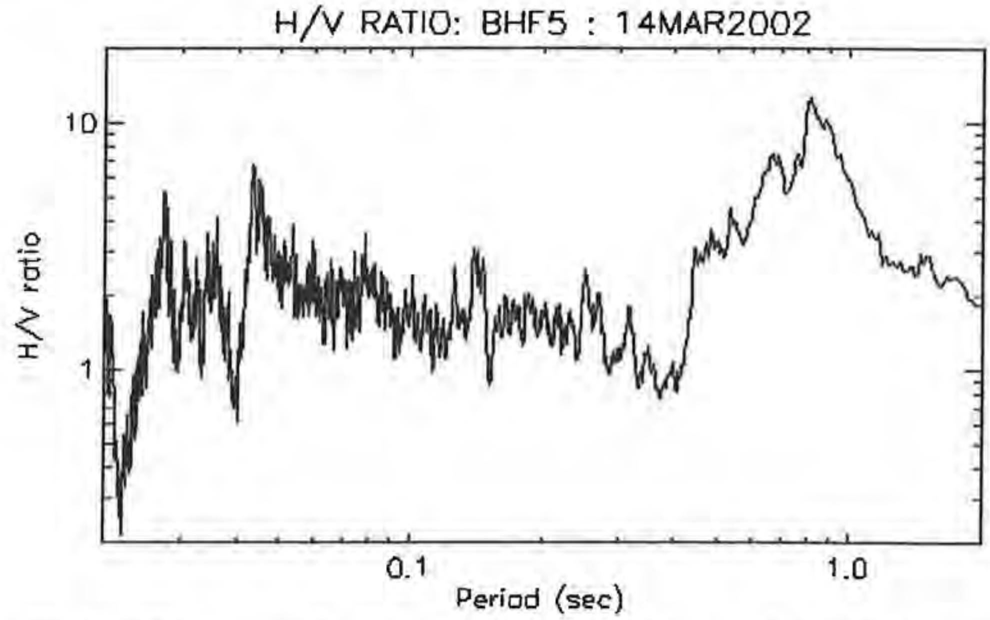


Fig.1. H/V spectrum for 100 s of microtremor data recorded at Blacksmith.

Table 1 shows a useful generic layered-earth elastic parameter model for the Blacksmith site. The model consists of two unconsolidated layers of sand overlying a sandstone rock basement (also underlain at 1000 m depth by granite). Shear velocities for the upper two layers are derived from a SCPT measurement in the locality, about 1 km to the east (Dhu et al, 2002). Table 1 also shows two additional models derived from the first by respectively halving and doubling the thickness of unconsolidated material while maintaining the (shear velocity x thickness) product constant. The three models have equivalent resonance period for the fundamental shear-wave resonance, and Figure 2 shows that the modelled H/V particle motion ellipse for the first three Rayleigh wave modes shows maxima at similar periods (0.85 and 0.3 s), where we have modelled the wave motion using the method of Herrmann (2001). The similarity in period of the H/V maximum demonstrates that simple resonance studies cannot distinguish between these three models, and hence in the absence of independent information, the shear modulus of the site for the purpose of foundation design is similarly poorly defined.

Table 1 Soil profiles of resonance-equivalent sites

	Model 1 (Blacksmith)		Model 2 (thick regolith)		Model 3 (thin regolith)	
Soil layers	Thickness (m)	V_s (m/sec)	Thickness (m)	V_s (m/sec)	Thickness (m)	V_s (m/sec)
1st layer	6.5	140	13	280	3.25	70
2nd layer	45.5	250	91	500	22.25	125
bedrock	1000*	1700	1000*	1700	1000*	1700

The sandstone bedrock is underlain by granite (Vs=3490m/sec) at a depth of 1km.
The density of the soil layers and sandstone is taken as 1.78tonne/cum and 2.39tonne/cum respectively.

3. PHASE VELOCITIES FROM MICROTREMORS

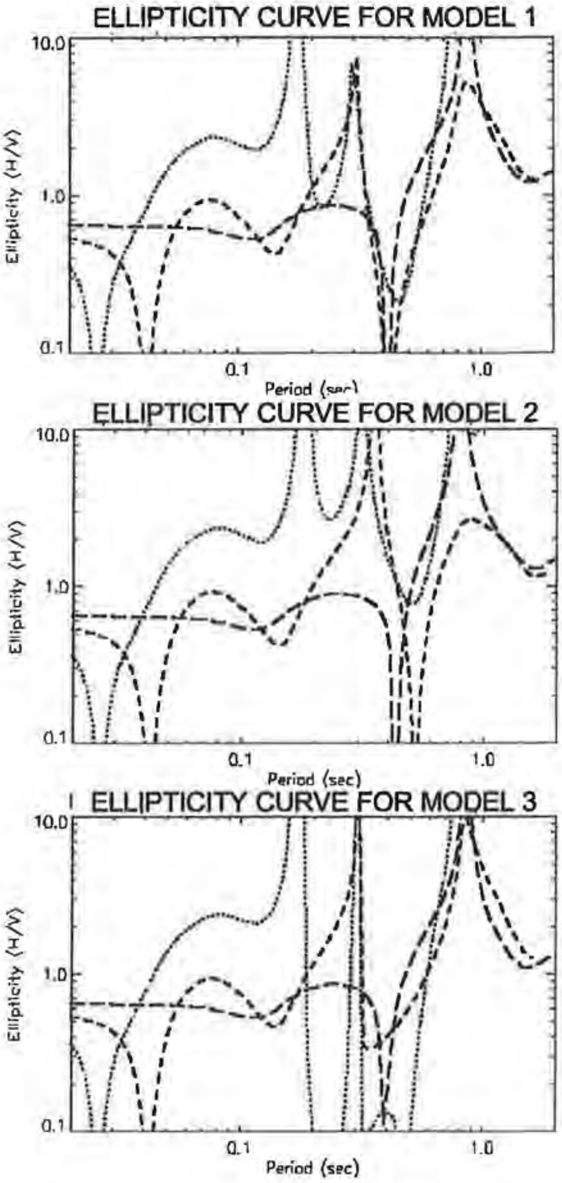
Phase velocities of microtremors may be estimated using beam-forming techniques (eg Liu et al, 2000) or azimuthal averaging. We use the latter for reasons summarised in Okada (1997) and Asten (2002). Use of a circular array allows sampling of the propagating microtremors over a range of azimuthal angles. For each pair of stations the coherency may be computed by standard spectral analysis methods (eg Koopmans, 1974). The circular array thus samples coherency over a range of azimuths. The averaging of these coherencies over azimuth provides a new parameter which, provided wave energy is confined to a single scalar velocity at each frequency, can be shown (Aki, 1957; Asten, 1976, 2001; Okada, 1997 and references therein) to take the form

$$ave\ c(f) = J_0\,(kr) = J_0\,(2\,\pi\, f\, r\, /\, V(f)\,), \qquad \text{-----}(1)$$

where *ave c(f)* is azimuthally-averaged coherency,
f = frequency, J_0 is the Bessel function of zero order,
k is the scalar wavenumber, $V(f)$ is the required phase velocity dispersion curve, and
r is the station separation in the circular array.

We now compare field and modelled *ave c(f)* for the sample of field data whose spectrum appears in Figure 1. A seismic array of six geophones arranged in a circle (hexagon) about a central geophone (following Asten, 2001) allows computation of *ave c(f)* for the field data, shown in Figure 3. Using the generic layered-earth model given in Table 1, we calculate the theoretical Rayleigh-wave phase-velocity vs frequency dispersion curves (not shown in this paper, but see Asten and Dhu, 2002, this volume, for examples). The theoretical velocity-frequency data is then substituted into equ(1) above (with r=50 m as for the field seismic array) to obtain a theoretical curve of *ave c(f)* vs frequency.

Figure 3 shows three examples of the modelled curve for *ave c(f)*, computed for each of the three regolith models shown in Table 1. It is obvious that the modelled average coherency curves for the thick-regolith and thin-regolith "resonance-equivalent models" bear no relation to averaged coherencies for the field data. Study of sensitivity of fitting of the theoretical and model curves indicates that the four parameters important to foundation design, namely thickness and shear velocity of the two unconsolidated layers, can be resolved to within about 20% by interactive visual fitting of the field data with varying



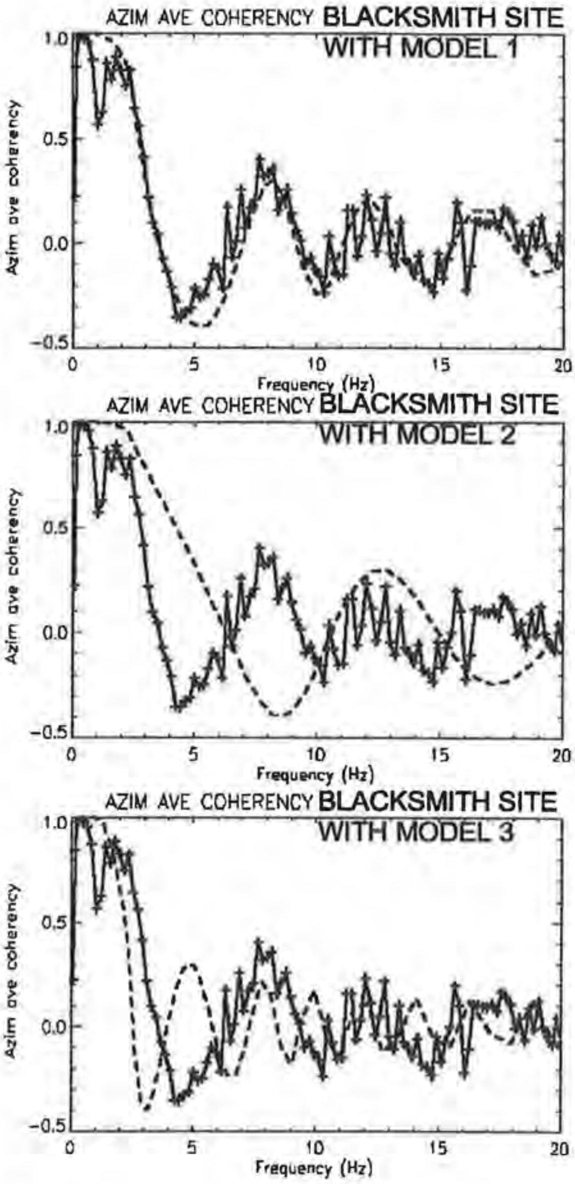


Fig.2. Computed H/V ellipticity vs period for three layered-earth models shown in Table 1.
Dashes: Fundamental Rayleigh mode.
Short dashes: 1st higher mode.
Dots: 2nd higher mode.

Fig.3. Fit of averaged coherency of field data (solid line) with modelled coherency (dashed line), for the three models given in Table 1.

model data. Formal inversion methods under development are expected to bring higher resolution.

4. COMPUTATION OF RESONANCES FOR THREE SITE MODELS

We now consider how the constraints on soil shear velocity, obtained from microtremor velocity measurements in the previous section, impact on resonances as calculated for building design.

One dimensional non-linear shear wave analyses (using program SHAKE) have been undertaken to determine the soil response spectra for the three "resonance-equivalent" models for which details of the soil layers have been summarized in Table 1. The rate of modulus reduction with increasing soil strain assumed for these analyses is based on the model defined by Lam and Wilson (1999) for cohesionless soils. It is shown that the adopted rate is very consistent with the "mean" recommendations by Seed & Idriss (1970). As shown in earlier sections of this paper, all three models possess similar natural period of about 0.9 secs but very different soil profiles. Model no.1 is the original profile recorded near the Blacksmith site using SCPT measurements, whereas Model no.2 has been modified to include deep, stiff soil layers totaling 100m in thickness. In contrast, Model no.3 has a total thickness reduced to 25m, but with soils of proportionately lower shear strength. The bedrock excitation employed in the analyses possesses a PGV of 50-60 mm/sec which is generally consistent with the ground motion intensity level stipulated for Australian capital cities for the 500 year return period.

The velocity response spectra of the three soil sites are similar in shape, with the highest point of the spectrum at the resonant peaks being consistently about 4 times higher than the corresponding level in the bedrock spectrum (Figure 4a). This observed amplification is in agreement with the prediction by a simple manual method developed very recently for modelling the effects soil resonance (Lam et al, 2001). Significantly, the period at resonance is well above the notional 0.9sec and varies between 1-1.5secs depending on the soil profile. The resonance period variation results in very different peak displacement demand as shown by the corresponding displacement response spectra (Figure 4b). The significance of period shift is thus well demonstrated. The notably higher period shift in the thin-regolith model (no. 3) is considered to be the result of non-linear behaviour in the soil. Non-linearity is clearly most pronounced in the very soft shallow soil site due to the higher angle of soil shear deformation compared to a stiff deep soil site. The peak displacement demand is shown to differ by as much as 50% between the resonance-equivalent sites.

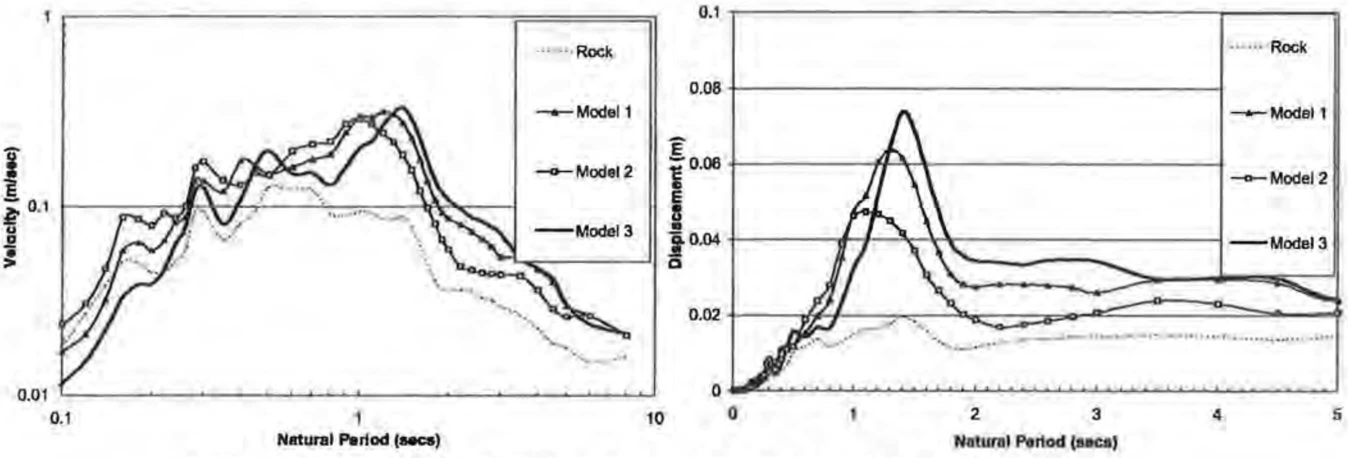


Figure 4 Velocity and Displacement Response Spectra for soil sites and bedrock

5. CONCLUSIONS

While site classification by natural period using observations of microtremors is a useful aid in assessing risk of damage from earthquakes, it is also subject to some ambiguity unless either the shear velocity or thickness of unconsolidated soil and sediments are known independently (such as with SCPT measurements). However the measurement of phase velocities of microtremors using a small circular array of seismometers, instead of a single station, provides sufficient information to determine the soil shear-velocity profile to within about 20%, thus allowing independent estimates of shear velocity, or extrapolation of existing drilling or SCPT measurements.

SHAKE modelling, using the shear-velocity profile derived from microtremor data, demonstrates that the ambiguities inherent in the use of natural-period measurements alone, may result in errors or order 50% in calculations of displacement demand. Such modelling quantifies the resonance period and site amplification which are of prime importance. It also shows the importance of non-linear behaviour of soils, which perturbs the frequency of maximum amplification to longer periods than that indicated by simple microtremor period measurements. We note that all analysis considered in this paper is restricted to layered-earth models; additional provisions may need to be incorporated in site classification studies to account for non-horizontal stratification of the soil layers, where the potential benefits justify the additional costs.

We conclude from studies thus far that microtremor velocity measurements have a significant role in improving inputs to site-amplification studies.

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ENHANCED INTERPRETATION OF MICROTREMOR SPECTRAL RATIOS USING MULTIMODE RAYLEIGH-WAVE PARTICLE-MOTION COMPUTATIONS.

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ABSTRACT:

Detailed modelling of microtremor propagation, using shear velocities acquired from seismic cone penetrometer tests (SCPT), shows that multiple peaks in the measured H/V spectral ratio of field data can be clearly correlated with maxima in particle-motion ellipticity as computed for a chosen layered-earth regolith model. This provides additional physical understanding of why the use of peaks in H/V ratios is effective for regolith site classification.

The resolution of spectral peaks is sufficient to allow the H/V ratios to constrain estimates of depth to bedrock. The use of modelled ellipticities in conjunction with observed spectral ratio data complements SCPT data, and can be used to estimate depths where the SCPT penetration stops short of bedrock, or to map laterally-changing thickness of unconsolidated sediments away from the SCPT site.

In some cases the period of the modelled 1st higher mode ellipticity maximum can be identified as a secondary maximum in H/V ratios. This wave mode and period can provide a constraint on P-wave velocity. Such additional information has applications in estimating depth to the water table, and may also be useful in estimating tendency to liquefaction for slope-stability studies

1. INTRODUCTION

The use of horizontal/vertical (H/V) spectral ratios of seismic microtremors is a standard tool for calculating natural site period, which can then be used in regolith site classification for earthquake hazard and risk studies. Plots of spectral ratio against period frequently show multiple peaks. This paper shows that the periods of the multiple peaks are closely related to particle-motion profiles for multi-mode Rayleigh wave propagation of microtremor energy; these profiles in turn are related to resonance properties (both shear and compressional) of the regolith.

Background seismic noise known as microtremors are generated by meteorological and cultural sources, with machinery and vehicle traffic being the principal sources at periods of interest in this project (1 sec and shorter). This seismic energy propagates primarily as surface waves, with the majority of the energy in the fundamental mode. Vertical-component geophones will detect only Rayleigh waves, while horizontal-component instruments detect both Rayleigh and Love modes. See Asten (1976, 2001) and Okada (1997) for reviews of studies of the modes of microtremors

The fundamental Rayleigh mode generally dominates high-frequency microtremor energy, and the basic spectral ratio method of site classification, as well as more sophisticated interpretation tools such as the Spatial Autocorrelation (SPAC) method, assume energy is confined to this single mode. However, this restriction is not always the case, and higher modes can be identified, both in the spectra shown in this paper, and in measured propagation velocities (eg Asten, 2001, and references therein). Higher modes of surface-wave propagation have the potential to influence both measured microtremor spectra and measured propagation velocities, both of which affect the interpretation of regolith thickness and geotechnical properties.

2. RAYLEIGH-WAVE MODELLING

Rayleigh-wave seismic energy propagates as a surface wave, with the particle-motion being elliptical in a plane parallel with the direction of propagation (waves on the surface of water are a close analogy). Rayleigh waves consist of an interaction of both shear (S) and compressional (P) waves, and at certain resonances, the particle-motion ellipse degenerates into pure horizontal or pure vertical motion. At a period close to the fundamental S-wave resonance of the regolith, Rayleigh-wave particle-motion degenerates to pure horizontal motion. This is why spectra of the H/V amplitude ratio are effective for basic mapping of the lowest natural S-wave site resonance. However, since surface waves can propagate in multiple modes, and each mode is a function of both compressional and shear elastic modulii, the relation between spectral H/V ratios and resonances is generally very complex.

For this modelling study we use a layered-earth seismic model developed for the Newcastle (NSW) area, with layer thicknesses specific to the H/V spectrum shown in Figure 2. Dhu et al (2002) provide a set of generic S-velocity models for different site classes in the Newcastle area, which are based on a compilation of geotechnical data for the Newcastle area. To the selected generic model we add a basement layer representing basement below the Sydney basin sediments (although this has no effect on modelling at the periods of interest), and we also use specific S-velocities for the upper two unconsolidated layers which are derived from a seismic cone penetrometer test (SCPT) about 100 m from the site of data acquisition for data shown in Figure 2. Tables 1(a) and (b) show the generic model and the specific SCPT model used subsequently in this

TABLE 1

Alternative layered-earth models for Rayleigh-wave modelling.

Vs, Vp are S and P-wave velocities. H, RHO are layer thickness and density respectively.

(a) GENERIC MODEL from Dhu et al(2002) CLASS WITH F/K water table at 10 m

H	Vp	Vs	RHO
(m)	(m/s)	(m/s)	(t/m^3)
10	445	222.7	1.78
6.5	1700	138.5	1.82
6.5	1700	180.5	1.98
20	1800	900	2.14
1000	2940	1700	2.39

(b) HOLE ISO-02 with water table at 3 m

H	Vp	Vs	RHO
(m)	(m/s)	(m/s)	(t/m^3)
3.0	240	120	1.80
25.4	1700	195	2.00
20.0	1800	900	2.14
1000.0	2940	1700	2.39
0.0	6040	3490	2.80

(d) WATER TABLE AT 18 m DEPTH

H	Vp	Vs	RHO
3.0	240	120	1.80
17.0	400	195	2.00
20.0	1800	900	2.14
1000.0	2940	1700	2.39
0.0	6040	3490	2.80

(c) MATCH SPECTRUM AT ISLINGTON Pk SITE

H	Vp	Vs	RHO
3.0	240	120	1.80
15.4	1700	195	2.00
20.0	1800	900	2.14
1000.0	2940	1700	2.39
0.0	6040	3490	2.80

(P-wave velocities Vp are derived using a rule-of -thumb of Vp/Vs~2 for unconsolidated sediments, Vp/Vs=1.73 for consolidated rock, and Vp~1700 m/s for saturated unconsolidated sediments from Asten, 1976).

paper. Figure 1 shows plots of modelled phase and group velocity for one model (Table 1(b)), computed using the methodology and forward-modelling code of Herrmann (2001).

The following points assist in understanding significance of the curves shown in Figure 1:

- The phase velocity is the velocity of a wavefront crossing the survey area; these are the velocities measured by an array of geophones (provided the array diameter is less than about a wavelength).
- The group velocity is the velocity of propagation of wave energy; these velocities are important because group velocity minima (GVM; also known as the "Airy phase" to seismologists) tend to carry the bulk of the energy associated with a wave packet (Hudson and Douglas, 1975). These group velocity minima tend to occur at P-wave resonances in a layered earth.
- The particle-motion ellipse is important because when the ellipse is flat (H/V is large), then the horizontal-component energy dominates over vertical-component energy. These elongated H/V ratios tend to occur at layer resonances, which in turn tend to occur at group velocity minima. This is why simple resonance concepts are often useful in understanding surface-wave propagation phenomena. However, surface waves, with their multiple modes and two classes (Rayleigh and Love waves) are much more complex than simple S- or P-wave resonance phenomena can describe, which contributes to limitations of the Nakamura method, especially when multiple-layer models are considered. Note also that particle-motion ellipses apply only to Rayleigh modes; Love modes consist of pure horizontal motion (in the absence of elastic anisotropy) and thus will add only to the power spectrum of the horizontal components.

Study of the curves of Figure 1 shows that GVM in the fundamental Rayleigh mode, R_0, can be identified at periods broadly corresponding to resonances in the sedimentary column above crystalline basement (at T=1.5 s), S-wave resonances in the unconsolidated surficial layers (at T=0.45 s), and P-wave resonances in the unconsolidated surficial layers (at T=0.21 s). We need not concern ourselves further with the first of these (other than to note that a thickness of 90 m of unconsolidated material could give a similar period of 1.5 s, indicating that knowledge of basement rocks at kilometres of depth may be relevant when analysing microtremor data at sites having unconsolidated sediments of thickness in the hundreds of metres).

The GVM at T=0.45 s corresponds with a high H/V ellipticity in particle motion for all Rayleigh modes, and thus we expect that it is this period which will dominate classical spectral ratio plots.

The GVM at 0.16 s in the 1st higher mode coincides in period with high H/V ellipticity for 1st, 2nd and 3rd higher modes (R_1 to R_3) and we may expect to see another spectral maximum at this frequency if higher modes are present in the microtremor energy. This GVM appears to be associated with P-wave resonances and is sensitive to choice of P-wave (Vp) velocities in the model.

3. COMPARISON OF RAYLEIGH WAVE VELOCITIES AND ELLIPTICITIES WITH SPECTRAL RATIOS

The single-station spectral ratio method of regolith site classification was first outlined by Nakamura (1989) and has been further tested and described by Lermo and Chavez-Garcia (1994), Field and Jacobs (1995), Ibs-von Seht and Wohlenburg (1999), Sato et al (2001) and Bodin et al (2001). It is routinely used in Australia for regolith site classification for earthquake hazard and risk studies (see eg. Michael-Leiba,1995; Jensen, 2000; Jones, 2000; Turnbull, 2000; Dhu et al, 2002). The importance of considering ellipticity of particle motion in interpretation of H/V ratios has been recognised by Lermo and Chavez-Garcia (1994), Yamanaka(1994), Lachet and Bard (1994) and Satoh et al (2001).

Figure 2 shows the H/V spectrum for a sample of field data acquired at Islington Park, Newcastle. The site is within 100 m of a SCPT test hole. The principal maximum in the observed spectrum is at period 0.4 s, which is shifted from the maximum of 0.62 s which might otherwise be predicted from the model constructed from the nearby SCPT hole (Table 1(b) and Figure 1).

We can use the difference between periods of the maxima in observed and model data to estimate directly the depth of unconsolidated layers below the field site. Since the computed curves are plotted on a logarithmic scale, the translation in the x-axis corresponds to a constant multiplication factor. Scaling laws applicable to seismic models allow us to interpret this shift as equivalent to either a proportional change in acoustic velocity parameters, or a proportional change in layer thicknesses, since

Velocity . Period / thickness

is a dimensionless constant. In the case of these models, S-velocities are well established from the SCPT investigations, and the thickness may then be varied in order to match model data with field observations. The scaling required to match the field data peak to the fundamental-mode ellipticity peak in Figure 1 is *0.65*. The corresponding reduction in thickness of sediments is applied to give the layered earth model shown in Table 1(c). As a check, the new model is used to compute ellipticity vs period shown in Figure 2(b), and it is apparent that there is an acceptable match in period between the field data H/V peak and the peak in ellipticity.

In the absence of S-velocity information (such as from SCPT measurements), the nature of the dimensionless constant means that estimation of layer thicknesses from spectral maxima of single-geophone measurements alone, is very poorly defined. However, where microtremor phase velocities are measured with an array of geophones this ambiguity is greatly reduced, as discussed in another paper in this volume (Asten et al, 2002).

4. INTERPRETATION OF A SECONDARY PEAK IN H/V AS A HIGHER MODE

The field data shows secondary peaks in H/V, at periods 1.1 to 1.7 s. These peaks are present in "noisy" samples of field data, but is less evident in "quiet" data samples. It is self-evident from field observation that "noisy" data arises when nearby seismic sources exist, such as traffic on an adjacent road. A useful hypothesis is that nearby sources create a higher content of higher-mode energy than distant sources. Modelling shows that the peak in H/V for the 1st higher mode at period 0.14 s is sensitive to the P-wave velocity of the unconsolidated sediments, ie this peak is associated with P-wave resonances in the model. Since P-wave velocity is strongly influenced by water saturation, identification of such a 1st higher mode peak in field data has the potential to yield information on depth to the water table.

Figure 2 (c) shows ellipticity vs period for the layered model of Table 1(d), which has a water table at 20 m depth (instead of at the 3 m depth used in previous models). While the ellipticity maximum for the fundamental mode is unaffected, the lower P-wave velocity used in the new model shifts the 1st higher mode peak from 0.14 s to 0.2 s. There is no indication of a peak in field data at this longer period, and it is concluded that the secondary peak in the field data H/V spectrum is likely to be due to a shallow water table which has affected the propagation of higher-mode energy. The Islington Park site is adjacent to a creek which supports the conclusion of a shallow rather than deep water table.

5. CONCLUSIONS

Modelling studies of Rayleigh-wave velocities and ellipticities provide a physical explanation of the H/V spectral peaks used for regolith site classifications. Where representative S-velocities are available from SCPT measurements, the period of the H/V spectral peak can be used to estimate laterally-changing thickness of unconsolidated sediments. Secondary peaks in H/V spectra can be correlated with ellipticities of higher-mode Rayleigh waves which provides the possibility of at least a qualitative estimation of depth to the water table. Detection of depth to the water table by these non-invasive methods may have further potential in mapping the likelihood of liquifaction of sediments.

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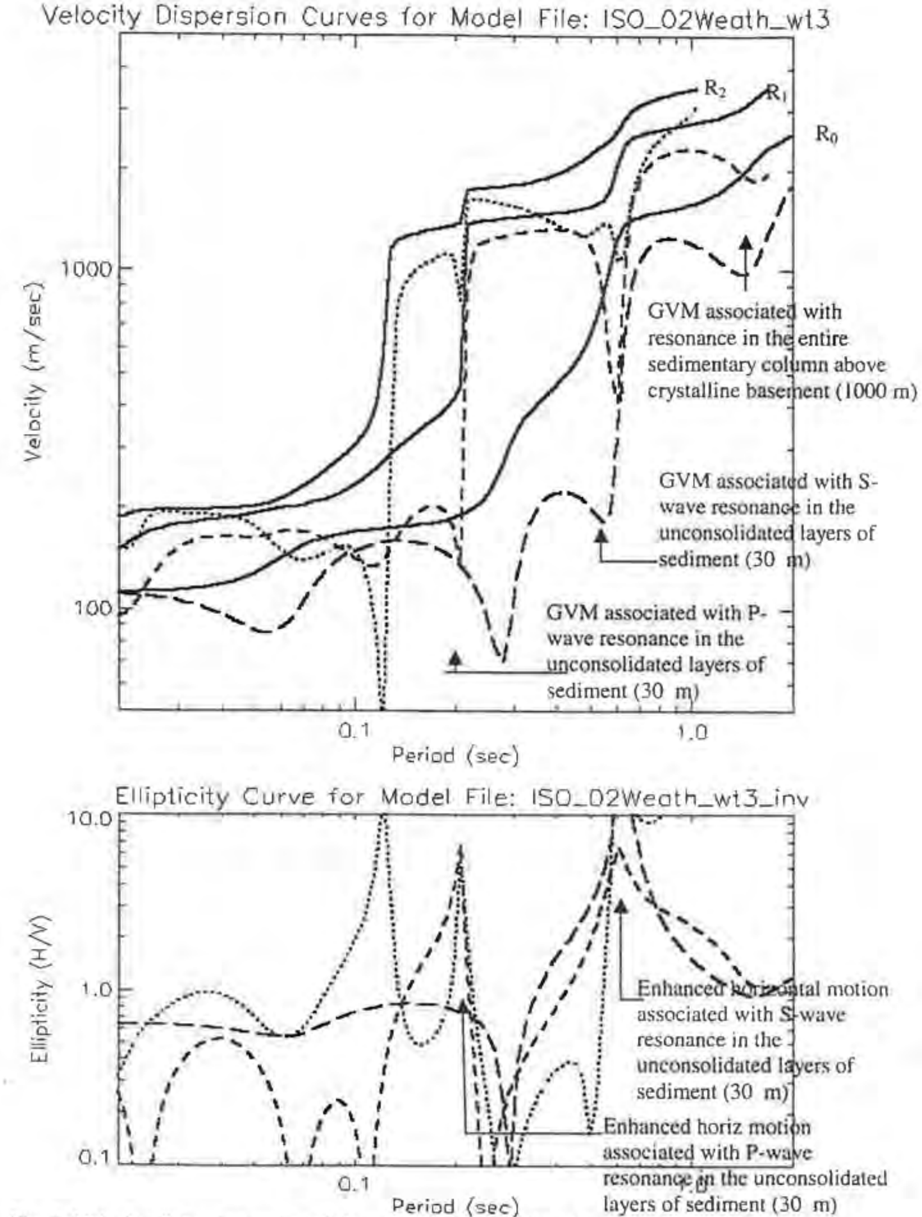

Fig. 1. Velocity dispersion and particle-motion ellipticity for a layered-earth model of Table 1(b)
Solid lines: Phase velocities for Rayleigh modes 0, 1, 2, 3, 4.
Long dash, dash and dotted lines: group velocities and ellipticity for modes 0, 1, 2.

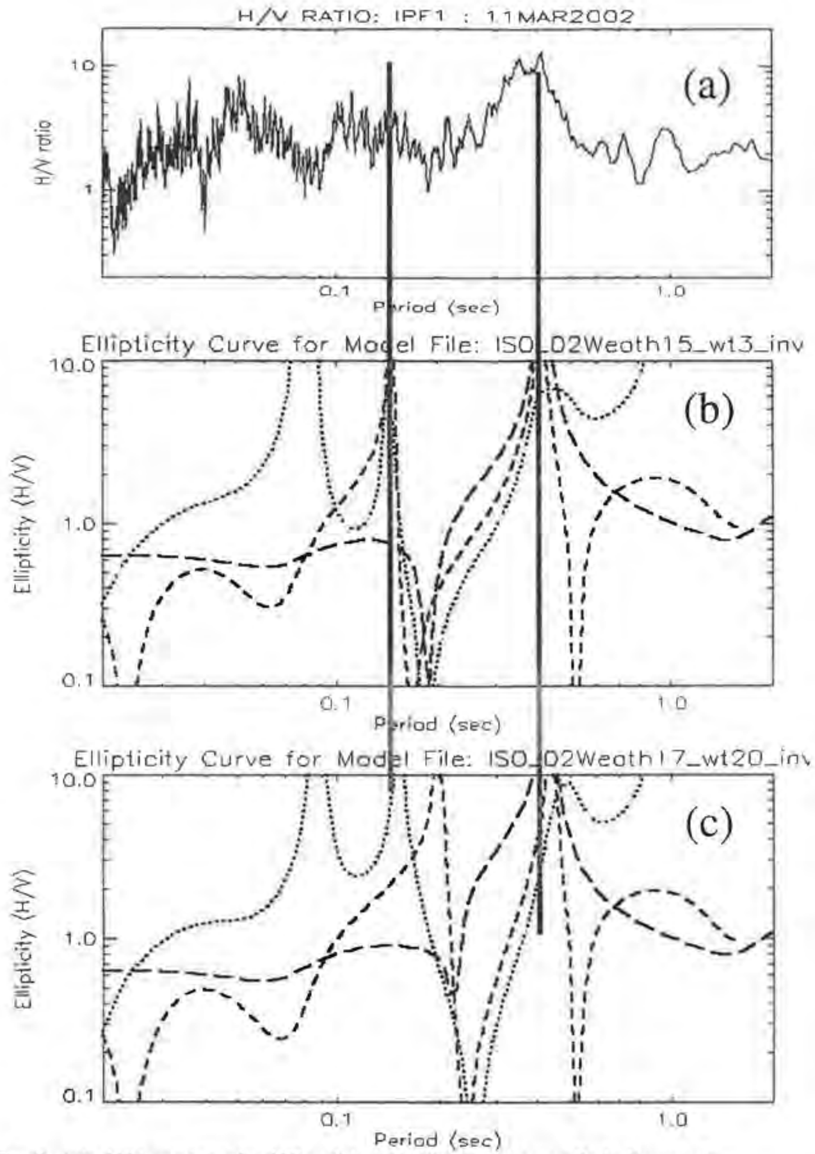

Fig. 2. (a): H/V spectrum for 100 s of data acquired at Islington Park, Newcastle.
(b): Computed ellipticity curves for three Rayleigh surface wave modes, for layered earth model of Table 1(c).
(c) Computed ellipticity curves for three Rayleigh surface wave modes, for layered earth model of Table 1(d). Long dash, dash and dotted lines correspond to fundamental, 1st and 2nd higher modes respectively.