GEOLOGICAL CONSTRAINTS ON ACTIVE SEISMICITY IN SOUTHEAST AUSTRALIA

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KEYNOTE SPEAKER

An ARC professorial research fellow in the School of Earth Sciences, University of Melbourne, Mike Sandiford took up the position in 2000 after twelve years lecturing at the University of Adelaide.

His research interests are in the tectonics of continental interiors. Recent work, which will be the subject of his presentation, has focussed on the young (neo-) tectonics of south-eastern Australia, and provides a geological context for understanding earthquake occurrences in this region.

He will discuss constraints on fault slip rates in south-east Australia in the recent geological past (the last few million years) and compare them to the seismic moment release rates associated with the historical earthquake record. This will demonstrate that south-east Australia actually contains a surprisingly rich record of tectonic activity, albeit at very modest rates. He will also attempt to identify the factors that control the unusual pattern of stress in south-east Australia, responsible for this tectonic activity.

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ABSTRACT:

Australia forms part of a stable continental region remote, but not immune, from plate boundary activity. The geological record of young reverse faulting across this region correlates spatially with the intensity of historical seismicity, and geometrically with the *in situ* stress field. The onset of the modern-day tectonic regime at around 6 million years ago relates to changes in the relative velocities of the Australian and Pacific plates. To first order, estimated cumulative fault slip rates (~several 100 m at rates of up to 50 m per million years) are consistent with the seismic strain rates estimates in the range between 10^{-17} - 10^{-16} s^{-1}, and provides a new and independent constraint on the magnitude and frequency of the earthquakes to be expected in southeast Australia, comparable to those deduced from the limited historical seismic record.

Paper No. 1

GEOLOGICAL CONSTRAINTS ON ACTIVE SEISMICITY IN SOUTHEAST AUSTRALIA

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1. INTRODUCTION

The occurrence of widespread, low-level seismicity in stable continental regions (SCR) raises important questions concerning the nature of the intraplate stress field, earthquake mechanisms and seismic hazard within continental interiors (Johnston & Kanter, 1990). Although accounting for less than one percent of the Earth's total seismic energy release, SCR seismicity presents substantial risk because earthquake rupture is general shallow and seismic attenuation is low. The most dramatic historical example of SCR seismicity was the 1811-1812 New Madrid sequence of M_w 7-8 quakes (Johnston & Kanter, 1990). Because repeat times for the largest SCR earthquakes such as the New Madrid sequence are likely to be many thousands of years or more, historical seismic records are likely to be incomplete. This is especially the case in Australia, where the records extend back only ~150 years. Therefore, it is pertinent to ask whether geological observations might provide additional constraints useful for seismic risk assessment.

This paper focuses on active tectonic deformation in southeastern Australia, a SCR region of moderate, but spatially variable seismicity. The main objective is to provide constraints on neotectonic deformation rates, and compare them to estimates of strain rates based on the seismic record. We begin with a brief summary of modelling studies that inform the origin of the *in situ* stress regime and provide a temporal framework for understanding the ongoing deformation.

2. *IN SITU* STRESS IN SOUTHEAST AUSTRALIA

The first order pattern of the Indo-Australian stress field has been explained in terms of the interaction between plate driving forces acting along the southern mid-ocean ridge segment and the northern plate boundary forces (Cloetingh & Wortel, 1986; Coblentz et al., 1995, 1998). The intraplate stress field is characterized by a broad arcuate trend in S_{Hmax} from N-S in India through E-W in the central Indian Ocean to western margin of the continent to NE-SW in northern Australia (Coblentz et al., 1998; Hillis & Reynolds, 2000). This trend implicates the importance of the Himalaya and New Guinea in balancing the driving torques associated with subduction and ocean lithosphere cooling (Coblentz et al., 1995, 1998; Sandiford et al., 1995). The intraplate stress field in several subregions of the plate appears to be influenced by boundary forces other than the ones acting along the northern plate margin. This is particularly true for southeastern continental Australia (Figure 1), which forms a unique and distinctive stress province.

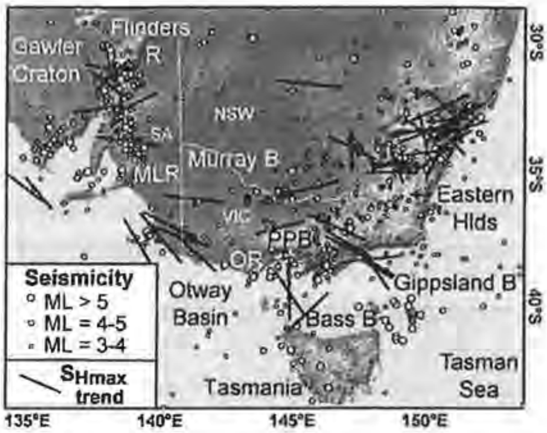

Figure 1,. Distribution of seismicity (Geoscience Australia earthquake database), topography and S_{Hmax} trends (after Hillis and Reynolds, 2000) in southeast Australia. OR = Otway Range; PPB = Port Phillip Bay, MLR= Mount Lofty Range.

As is true throughout most of continental Australia, the S_{Hmax} orientation in this region shows considerable scatter, particularly in comparison to other stable intraplate regions (Zoback, 1992). Information about *in situ* stress field in southeast Australia (Figure 1) is constrained by both earthquake focal mechanisms and borehole breakouts (Denham & Windsor, 1991; Hillis & Reynolds, 2000). Strike-slip and reverse focal mechanisms for earthquakes in the Flinders Ranges yield a principal horizontal compression (S_{Hmax}) of 83±30° (Greenhalgh et al., 1994; Hillis & Reynolds, 2000). In the Eastern Highlands of Victoria, reverse focal mechanisms define a SE-NW azimuth for S_{Hmax} (Gibson et al., 1981). Hillis & Reynolds (2000) summarise borehole breakout data from two basins along the southeast margin where the data are considered sufficient to define a significant trend. In the Otway Basin, the azimuth of S_{Hmax} derived from breakouts is 136°±15°, while in the Gippsland Basin, near the southeast corner of the continent, breakouts yields a S_{Hmax} of 130°±20°.

The tectonic forces controlling the present day regional intraplate stress field in continental Australia have been evaluated through a finite element analysis of the intraplate stresses. Modelling studies show that the stress field in southeast Australia relates to the convergence between the Pacific and Australian plates that have built the Southern Alps of New Zealand (Coblentz et al, 1995, 1998; Reynolds et al, 2002). Figure 2 shows the impact of New Zealand collisional forces on the stress field of southeast Australia. Figure 2a shows the stress field with an effective collisional force acting along the southern part of New Zealand of 3 x 10^{12} Nm^{-1}. As discussed in Reynolds et al., (2002) a force of this magnitude forms part an ensemble of plate boundary forces that minimizes the misfit with the *in situ* stress indicators throughout the Indo-Australian plate. Variations in the magnitude of the force acting along the New Zealand boundary force have a profound influence on the intraplate stress field in southeastern Australia. Figure 2b and 2c show the modelled stress field at 50% and 0% of the Reynolds et al. (2002) best-fit New Zealand boundary force, respectively. Reduction in the boundary force magnitude acting across the New Zealand boundary segment produces a substantial rotation in the S_{Hmax} azimuth (from a dominant NW-SE to a NE-SW orientation), and a slight reduction in the stress magnitudes (from ~ 25 MPa to ~ 20 MPa, averaged over a 100 km thick lithosphere).

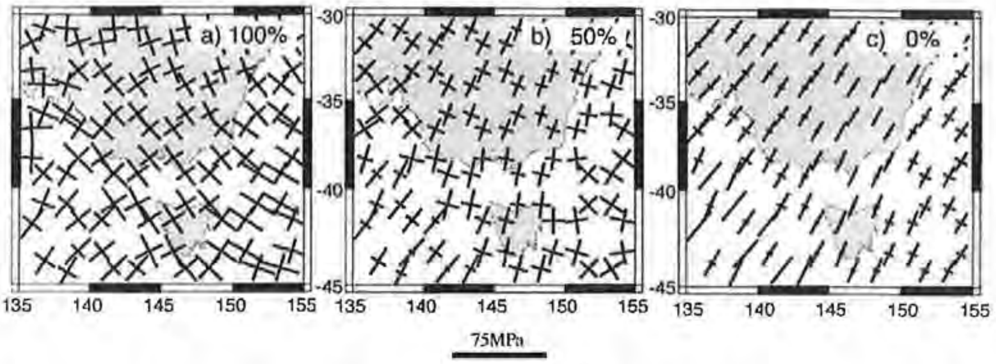

Figure 2. Details of modelled stress in the southeast Australian region. The individual panels show the impact of reducing the magnitude of the New Zealand compressional force from 125% of the Reynolds et al. (2002) present-day "best fit" model (a) to 0% of the "best-fit model" in f). The best-fit solution is shown in panel b). A stress scale of 75 MPa is shown at the bottom centre.

The timing of this change in the New Zealand boundary force is constrained by formation of the Southern Alps of New Zealand (the most obvious geological manifestation of this compression). The southern Alps have been constructed since the late Miocene (about 6 million years ago) due to a change in the relative velocities of the Pacific and Indo-Australian plates (e.g., Walcott, 1998). This places an important temporal constraint on the change in the southeast Australian stress field that is critical for understanding the time frame relevant to neotectonic activity in southeast Australia.

3. NEOTECTONIC FAULTING RECORD IN SOUTHEAST AUSTRALIA

The late Neogene record of southeast Australia contains abundant evidence for faulting. The intensity of this faulting shows marked spatial variation correlating, to a large extent, with the distribution and intensity of seismicity (Sandiford, 2003a). The most extensive faulting record occurs in the Flinders and Mount Lofty Ranges of South Australia, and in southern Victoria, in upland systems such as the Otway and Strezlecki Ranges (in Gippsland) bordering the southern coastline.

The Flinders and Mount Lofty Ranges in South Australia are bounded by N-S to NE-SW trending fault scarps, with morphology of the Mount Lofty Ranges in particular providing dramatic testimony to the role of active faulting and formation of tilt blocks in shaping the landscape (Figure 3a). Exposures of the main range-bounding faults characteristically reflect steep reverse motion with a hanging-wall of ancient (> 500 million year old) rock above a footwall comprising conglomerates shed from the developing upland systems in the last one-two million years (Figure 3b). Fault-slip kinematics are consistent with structures having formed in response to reverse stress regime with S_{Hmax} trending between N080°E and N125°E (Figure 3a) more or less identical to the *in situ* stress field. Slip rates on the major range-bounding faults have been estimated at between 20-100 m/million years. The cumulative vertical displacement on the fault network that forms the western front of the Mount Lofty Ranges is estimated to be ~ 240 m (Sandiford, 2003), with ~80 m offset of early

Quaternary (~1.6 million year old) strata. Assuming that this displacement has accumulated in the last 5-6 million years then the time averaged displacement is ~ 40-50 m/million years.

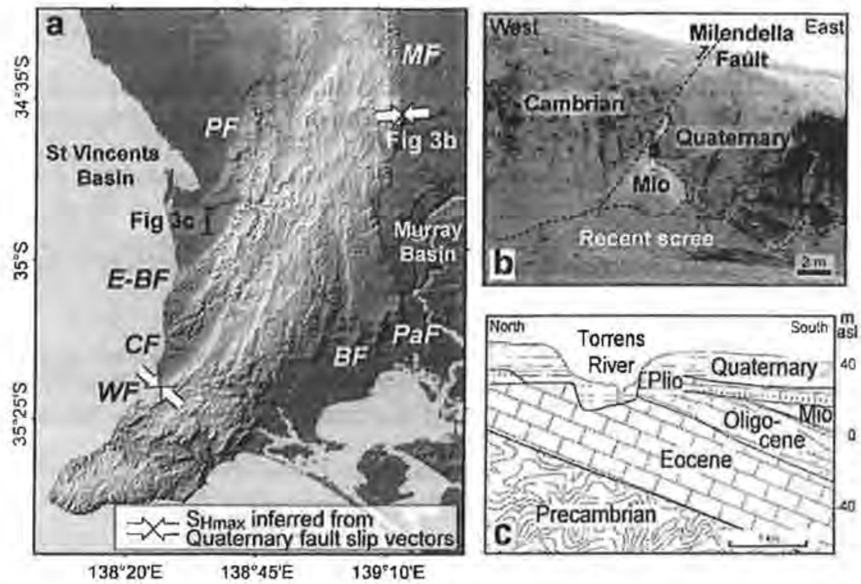

Figure 3. (a) Shaded topography of the Mount Lofty Ranges highlighting the youthful, fault bounded nature of this landscape. The main range bounding faults are the : Para (PF), Eden-Burnside (E-BF), Clarendon (CF), Willunga (WF), Bremer (BF), Palmer (PaF) and Milendella (MF) faults. (b) Outcrop of the Milendella Fault (location as shown in Fig. 3a). Reverse fault movement has thrust Cambrian metasedimentary sequences above Quaternary outwash gravels (~780 ka) and deformed and overturned early Miocene limestone (~ 20 milllion years old, Bourman and Lindsay, 1988). (c) N-S cross section through township of Adelaide modified (location as shown in Fig. 3a). The angular nature of the Miocene(Mio)-Pliocene(Plio) unconformity, implicates significant titling of the Para Fault block at around 6 million years ago.

On the northern flanks of the Otway Range, the remnants of 3-5 million year old beach deposits rise ~120 metres over a series of ENE trending faults and monoclines to elevations of ~ 250 metres. These ancient beach deposits were developed during falling sea levels following a 6 million year old sea stand high approximately 65 m above present day sea level (Brown & Stephenson, 1991) and thus imply almost 200 m of tectonic uplift in the last 3-5 million years due to contemporaneous faulting.

These observations are consistent with slip rates approaching ~ 40-60 m/million years on faults bounding the Flinders Ranges and southern Victorian uplands. This can be contrasted with the intervening Murray Basin, where comparative (but not complete) tectonic stability is demonstrated by preservation of 0-5 million year old shorelines (Sandiford, 2003a).

4. SEISMICITY IN SOUTHEAST AUSTRALIA

The southeast is one of the most seismically active parts of the Australian continent (Figure 1) with a broad distribution of earthquakes up to local magnitude (M_L) 6.4 across a zone ~ 1000 km in width from the eastern seaboard to the Eyre Peninsula in the west. Distinct concentrations in seismic activity occur in the Mount Lofty-Flinders Ranges-eastern Eyre Peninsula of South Australia, and in the belt trending from the west coast of Tasmania, through south-central Victoria, northeast through the Eastern Highlands to southern New South Wales (Figure 2). The intensity of seismic activity in these zones contrasts the intervening Murray Basin and the regions further west.

The intensity of seismic activity for any region can be quantified using Gutenberg-Richter relations (Gutenberg & Richter , 1944, 1949):

$$\mathrm{Log}_{10}(\mathrm{N}) = a - b\mathrm{M}$$

where N is the number earthquakes of magnitude M, and *a* and *b* are constants. The *a*-value indicates the background level of seismicity (or activity rate), and the *b*-value characterises the ratio of small magnitude events to large events. In order to evaluate the *a* and *b*-values, the method used here defines N as the number of events of magnitude M or greater (after Gaull et al., 1990).

The nature of observed variability in *b*-values is debated. Some researchers suggest it approaches a global value of 1 (Suzuki, 1959) with departures from unity due to incomplete catalogues. Other researchers suggest *b* varies spatially and temporally (e.g., Utsu, 1999). It is important to note that *a* and *b*-values deduced form historical records depend on the magnitude scale, the magnitude range, and data completeness, with accurate estimates of *a* and *b*-values requiring relatively large sample sizes. As a guide, a magnitude range of at least 2 and a sample size of 200 events is desirable. For Australia, the earthquake catalogues are complete above magnitude 3 for 25-40 years at best (Gaull et al 1990). The low activity rate of most of Australia means obtaining the necessary numbers of earthquakes can be challenging.

In order to ensure statistically significant sample sizes, we have used relatively large regions. In addition, we have employed a composite approach. The historical record of 100 - 150 years is used for characterising the larger events, while the 25 - 40 year instrumental record is used for the smaller events, enabling us to extend the range of magnitudes to greater than 3 units (Figure 3). Results for three regions, normalised to an area of 10,000 km^2 and one year, are summarised in Table 1 and Table 2.

The Flinders Ranges dataset yields an *a*-value of 3 implying a magnitude 5+ earthquake every 100 years per 10000 km^2. Extrapolating this to the entire Flinders Ranges suggests a recurrence interval for magnitude 5+ events of 12 - 15 years. The Murray Basin has an *a*-value of 2, suggesting it has one-tenth the number of earthquakes of the Flinders Ranges. However, the area used to extract the Murray Basin data reflects the most active part of the basin (in order to obtain the requisite number of events), and doubling the area results yields an *a*-value of 1.7, or ~3% of the activity rate of the Flinders ranges.

For Gippsland the *a* and *b*-values calculated for all available data are 1.95 and 0.79, respectively, with the low *b*-value reflecting an anomalously high proportion of magnitude 4.5+ earthquakes in the dataset. These values suggest that the Gippsland region has a higher moment release rate than the Flinders Ranges. However, it is probable that the low *b*-value results from a spurious over representation of large events in the historical time window (see Discussion below). Using only the last 27 years of instrumental data, excluding magnitude 4.5+ events, and assuming a *b*-value of 1, yields activity rate intermediate between the Flinders Ranges and Murray Basin (see Gippsland #1 in Table 2).

Region	Min Mag.	Max Mag	Number of Earthquakes	Number of Years
Flinders Ranges 1	5.6	6.0	1	153
Flinders Ranges 2	4.0	5.5	74	115
Flinders Ranges 3	3.1	3.9	138	40
Murray Basin 1	4.0	5.5	14	104
Murray Basin 2	2.3	3.9	179	29
Gippsland 1	4.5	5.7	19	123
Gippsland 2	2.5	4.4	167	27

Table 1 Completeness intervals adopted for the three seismogenic zones of interest.

Region	a	B b	R^2	5.5+/500yr
Flinders Ranges	3	1.0	0.99	1.6
Murray Basin	2	1.0	0.98	0.16
Gippsland #1	2.6	1.0	0.99	0.6
Gippsland	1.95	0.79	0.98	2.0

Table 2 *a*, *b* and number of earthquakes (per year) M5.5 or larger per 500 years for the 3 regions, normalised to 10,000 km^2. The method of analysis for Gippsland #1 is discussed in the text.

5. SEISMOGENIC STRAIN RATES

Following Johnstone (1994b), we can derive an estimate for notional seismic strain rate, in terms of the total seismic moment:

$$M_O \approx \frac{1}{t}\left(\frac{b\left(10^{(a+d)}\right)}{c-b}\right)\left(10^{(c-b)} M_{max}\right)$$

where t is the timespan of the seismic record, M_{max} is the maximum expected magnitude for earthquakes in the region of interest, and c and d are factors that relate to the conversion of magnitude scale to seismic moment (Hanks & Kanamori, 1979) :

$$\log(M_O) = c\,M + d$$

In terms of the seismic moment, the seismic strain rate is (Kostrov, 1974) :

$$\dot{\varepsilon}_{xx} = \frac{1}{2\mu\, v} M_O$$

where μ is the Young's modulus (8 x10^{10} Pa) and v is the volume of crust in which the seismicity occurs (here we assume the seismogenic zone is 15 km thick, based on the knowledge that the great majority of Australian earthquakes have epicentral depths less than ~15 kms, e.g., Gaullet al., 1990).

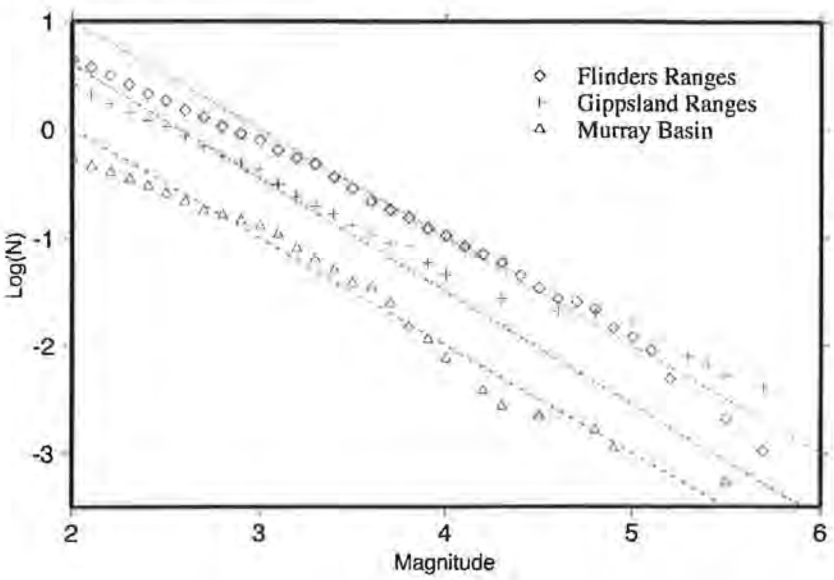

Figure 4. The data used to calculate the *a* and *b*-values for the three regions across southeast Australia.

Since the majority of the moment is carried by the most infrequent, largest earthquakes, the main uncertainty in the calculation of the seismic strain rates is the value of M_{max} (Figure 5). As noted earlier, the historical record can only provide a lower bound on M_{max}. The largest estimated earthquake (pre-instrumentation) in southeast Australia is $M_L \sim 6.4$. Elsewhere in Australia $M_L > 6.5$ quakes have occurred at a number of widely distributed localities, with the largest earthquake of magnitude $M_L = 6.8$ (in 1941, near Mereebie in Western Australia). On longer timescales we would expect somewhat larger earthquakes, and thus the maximum earthquake expected on geological timescales could conceivably be as high as $M_L \sim 7.5$. Assuming $M_{max} = 7$, the calculated seismic strain rate for Flinders Ranges is $\sim 1 \times 10^{-16}$ s^{-1} (Figure 5), with uncertainties in both the M_{max} and the thickness of the seismogenic zone (± 5kms) yielding an uncertainty of at least a factor of 2. The seismic data suggest the Murray Basin is deforming at an order of magnitude lower strain rates, while the preferred value for Gippsland (Gippsland#1) yields a seismic strain rate of about 4×10^{-17} s^{-1} (which is an order of magnitude lower than the rate using the parameter set including all Gippsland seismic data). These estimates are compatible with Johnston's (1994b) estimate of ~2 $\times 10^{-17}$ s^{-1} for bulk Australian seismic strain rates.

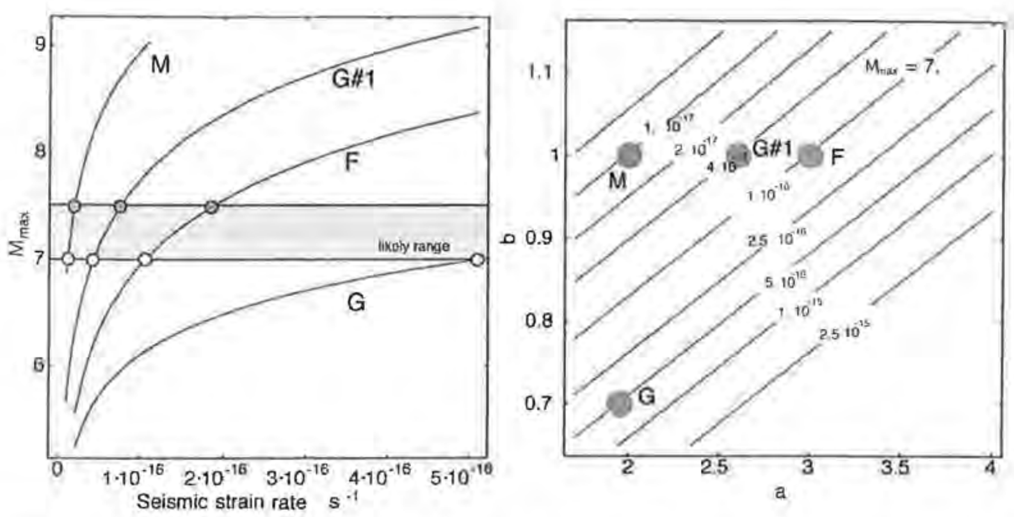

Figure 5. Relationship between, (a) M_{max} and seismic strain rate for the seismic zones listed in Table 2, (b) strain rate as a function of a-b values for M_{max}= 7. Grey shaded region in (a) the preferred range in M_{max}. F : Flinders seismic zone, M: Murray Basin, G: Gippsland, G#1: Gippsland, with analysis as described in text.

Our incomplete knowledge of seismic efficiency (i.e., the relative accommodation of strain by seismic and aseismic mechanisms) imply further uncertainty in translating seismically determined strain rates to bulk crustal strain rates. Nevertheless, these calculations provide a basis for comparison with the fault-slip rates determined from geological observations (see Discussion).

6. DISCUSSION

The geological observations summarised here highlight a spatial correspondence between the young faulting record and the distribution of seismicity (see Sandiford 2003a for further elaboration). Our understanding of the origin of stress field responsible for both the contemporary earthquakes and for the young faulting record suggests that the present tectonic regime developed around 6 million years ago. In the Flinders Ranges and southern Victorian uplands, such as Gippsland, faulting has been responsible for several hundred metres of relief generation in this time, with cumulative vertical slip component on the range-bounding faults systems estimated at about 50 m/million years. For comparison, the seismic strain rates equate to a cumulative vertical slip component on range bounding faults (assuming reverse fault motion on 45° surfaces) of ~95 m/million years for the Flinders Ranges (where the width of the deforming zone is about 100 km wide). A zone of comparable with in Gippsland would be slipping at ~ 35 m/million (Gippsland #1), while in the Murray Basin the cumulative slip rates would be no more than 10 m/million years across a 100 km wide zone. Uncertainties in the thickness of the seismogenic zone or the Young's modulus suggest that these estimates have uncertainties of at least a factor of 2, without considering the uncertainty in the *a* and *b*-values.

While the seismic strain rates should be expected to be lower than geological strain rates, because seismic efficiency is likely to be less than 100%, these estimates are surprisingly close to those deduced from the geological observations (especially in view of the uncertainties). In turn, this correspondence suggests that the historical seismicity is not particularly anomalous in terms of its geological context, at least for the Flinders Ranges and Murray Basin, providing that the maximum expected earthquake magnitude is not much greater than ~ M_w 7. Because seismic moment release is dominated by the largest earthquakes, the calculations summarised here suggest that if $M_w > 7$ earthquakes do occur in southeast Australia, then they must occur much less frequently than expected on the basis of linear extrapolation of historical earthquake statistics (ie., $M < 6.4$).

In Gippsland, the seismic strain inferred from the a and *b*-values derived from all available events ("Gippsland" in Table 2) is an order of magnitude higher than allowed by geological observations, and suggest either (1) an over-representation of larger events in the existing database for Gippsland, or (2) strain rates have increased markedly over the last one million years or so.

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SECURING CRITICAL INFRASTRUCTURE

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INVITED SPEAKER

AUTHOR:

Athol Yates, BEng, GradDip Soviet Studies, M Public Policy, is the Associate Director-Public Policy at Engineers Australia. His specialities include infrastructure security and planning, defence policy, government procurement and disaster analysis. He project managed the *2003 NSW Infrastructure Report* and *2001 Australian Infrastructure Report Card.* Other notable publications have been *Government as an Informed Buyer*, *Risk Allocation in Major Projects* and *Engineering Skills for Rail Sector Growth.*

ABSTRACT:

Engineering safe and secure infrastructure means more than just applying world's best engineering practice. Engineering excellence cannot overcome poor security policies, procedures and practices. Only when engineering is supported by a sound security environment can the engineering contribution deliver its full potential.

Currently such a security environment, consisting of supporting government and business policies and culture, does not exist.

This paper highlights what needs to be done to create a sound security environment that leads to making security sustainable for infrastructure organisations.

The paper also focuses on what improvements are required by engineering and the profession to enable it to contribute effectively to countering terrorism and other hazards.

1 Introduction

In June 2003, Engineers Australia released the report, *Engineering a Safer Australia: Securing critical infrastructure and the built environment.* It analysed developments in critical infrastructure protection and identified a series of weaknesses in Australia's existing approach. This paper highlights some of its major findings and recommendations for moving forward.

The paper is divided into 3 parts of context, making security sustainable and securing engineering excellence.

2 Context

Threats from international terrorism have dramatically transformed the international security environment. The attacks in the US and Indonesia have changed the global environment in which Australians live and will effect the Australia's domestic security agenda for decades. In response to these new threats, Australia requires a detailed, long-term strategy to address the security of critical infrastructure and the built environment in a holistic fashion.

For this to occur, government, industry and the professions will need to shift their focus from reactive, tactical activities to proactive, strategic ones. One of the main consequences of this is will be to shift of attention from critical infrastructure protection to built environment security. Another will be to integrate terrorism into the management arrangements used to treat all other man-made and natural hazards.

Australia's three spheres of government have done and are doing a great deal to improve critical infrastructure and built environment security. However, the Commonwealth's response to threats to critical infrastructure and the built environment have been lacking. Particular deficiencies included the slow speed of policy development and implementation, inadequate whole of government coordination and a failure to adopt an all-hazards approach. Also notable is the Commonwealth's failure to focus on built environment elements outside of the critical infrastructure areas, facilitate industry self-improvement, and capitalise of the contribution that science, engineers and technology can make to enhancing national security.

Over the next few years as the threat level changes from acute to chronic, an increased effort will need to be directed at 'rear-end' challenges, such as integrating business and security practices, engineering-in security rather than adding it on, and creating structures to rapidly transfer engineering security best practice knowledge throughout the built environment.

Underpinning the recommendations to achieve a safe and secure infrastructure is the following principles:

- **All-hazards approach**: Critical infrastructure and the built environment face a series of hazards which can all be a source of potential harm. These include natural hazards such as cyclones, floods and earthquakes, and man-made hazards such as vandalism, arson and terrorism. While every hazard is different and requires specific counter-measures, they should all be treated under a single set of management arrangements so that resources are allocated on a comparative hazard basis that reflects the risk, probability and consequences of each hazard. This is called an all-hazards approach.
- **Risk management**: Risk management is the systematic application of management policies, procedures and practices to the tasks of identifying, analysing, evaluating,

treating and monitoring risk. It provides a standardised way to address all hazards and determine risk mitigation treatment based on a comparative analysis.

- **Comprehensive approach**: A security strategy must address all aspects of security including physical and cyber security as well as security policy, practices and procedures. It must also address the four elements of security consisting of prevention, preparedness, response and recovery.
- **Internal and external integration**: Security strategies will be effective if they are integrated into all other activities of an organisation, and integrated with the activities of external stakeholders. Internally, the development of a security culture that permeates throughout the organisation is required. Externally, a partnership with government, industry and the community so that all partners can mutually support the others is needed.
- **Best practice**: Best practice in security is rapidly evolving as security experiences increase and practices adapt to new threats. Due to the speed of these changes, documented best practice embodied in codes and standards invariably lag behind practitioners' best practice. This means that organisations need to do more than simply implement relevant security codes and standards. Instead they need to continually improve their security by identifying and implementing relevant world best practice.
- **Security is everyone's business:** There is no one organisation, profession or group that is solely responsible for security. Everyone should contribute to security both within and outside their organisation, by following security procedures and sharing experiences with other businesses.

3 *Making security sustainable*

Threats from international terrorism, weapons of mass destruction and rogue States have transformed the global security environment. For Australia, the attacks on the US and Indonesia have fundamentally changed the global environment in which Australians live. The impact of this change will be felt for years to come.

To ensure that the new security consciousness of Australia remains at a heightened level in the coming years will require that the security focus becomes sustainable. This means embedding security into business and government values in much the same way that environmental considerations have been embedded.

Factors that may prevent the security focus from becoming sustainable include:

- The return on the security investment does not justify additional security expenditure.
- The security systems themselves impose a significant reduction in the efficiency and effectiveness of staff completing their work.
- Many common business practices and paradigms actually undermine security and continuity of supply. Examples of these are early to market products, commercial pressure to lower design and documentation cost, outsourcing of maintenance, break up of infrastructure organisations into units to stimulate competition and just in time supply.
- A failure to integrate security considerations into strategic land-use planning and infrastructure development.
- A lack of coordination in security policy and programs across government portfolios.
- The lack of a security culture and experience in engineering education and practice.

- A lack of resources to update engineering codes and standards during the window of opportunity that follows a disaster.

The key to making security sustainable is to ensure that all stakeholders get a dividend for their security investment. Governments are seeking a security dividend that generates political, economic and social benefits. Businesses are seeking a dividend that generates a financial advantage. This return on investment occurs because it:
- Increases the businesses' efficiency for three reasons. Firstly, the focus of systems shift from correcting errors after they occur to preventing them. Secondly, processes become more traceable and individual's accountability is enhanced. Thirdly, the response to abnormal incidents are improved and restricted to a limited area without causing disruption throughout the business.
- Increases the appeal of the business to the customers. This is because the business will be viewed as more reliable and resilience to unforeseen events. This is particularly important if the business is in a supply chain and continuity of supply is critical. For those businesses in global supply chains, demonstrating this is critical to keeping existing business, as many purchasers have given preference to suppliers that are geographically closer to them since the 11 September 2001 attacks.
- Increases the businesses' employees retention rate as most employees prefer to work in a safer workplace.
- Demonstrates that the business is a good corporate citizen as it takes on security issues, thereby reducing the demand on police resources. This in turn increases stakeholder loyalty.
- Reduces the likelihood of the business being liable for incident: Building owners and designers failing to provide a safe environment, such as secure parking lots and security lighting, may have failed in their duty of care requirement and hence are liable for incidents that occur.

3.1 Whole of nation domestic security strategy

Australian needs to move beyond self-evident platitudes about the importance of protecting critical infrastructure and vague policy statements about future security improvements. The nation requires a detailed, long-term strategy addressing the security of the built environment in a holistic fashion.

The elements of such a strategy are a:
- Whole of nation perspective.
- Ten year time frame.
- Partnership between Commonwealth, State/Territory and local governments, industry and the professions.
- Sophisticated level of government intervention.

The most important are the last two.

Partnership between Commonwealth, State/Territory and local governments, industry and the professions

A genuine partnership between Commonwealth, State/Territory and local governments, industry and the professions is essential to rapidly advance a domestic security strategy. This is because all organisations and groups can potentially make substantial contribution in their areas of responsibilities but this will only occur if it is harnessed effectively under a partnership arrangement.

The traditional Commonwealth approach to addressing national security issues will need to be discarded if a domestic security strategy is to be effectively implemented. This is because the command and control model use to employ the Australian Defence Forces and direct foreign policy to achieve national security is not applicable to domestic security. A domestic security strategy requires that the Commonwealth contributes but not dominate.

Politically, it will be difficult for the Commonwealth to take a back seat. The government of the day perceives (correctly) that the public will hold it responsible for all terrorist actions against Australians, even when it had little ability to influence them. Consequently, the Commonwealth will seek to protect its political interests by directly controlling the domestic security agenda. This can be seen in the Commonwealth's approach to the Critical Infrastructure Advisory Council (CIAC). and the Trusted Information Sharing Network (TISN). It appears that the chair of the Council will be a representative of the Commonwealth Attorney-General's Department, and the chair of each sectoral Infrastructure Assurance Advisory Groups in the TISN will be a representative of the Commonwealth's lead agency for that Group.

Sophisticated level of government intervention

Establishing the correct policy, regulatory and administrative settings to deliver sustainable security will not be simple. This is because of the radically different agendas, capability and capacity of different stakeholders, and the complexity and variability of the built environment.

The role of governments in the domestic security strategy is two fold. They include establishing a macro level environment that is conducive to creating a sustainable security and micro level intervention that corrects market failures which are resulting in an under or over provision of security.

Looking just at the Commonwealth, there is little evidence of a whole of Commonwealth government approach to critical infrastructure protection, despite its claim that its response is based on an 'all agencies' approach. While a number of Commonwealth agencies have been involved, there is a lack of engagement in several essential areas, notably in regulation, taxation and planning. The lack of whole of Commonwealth government approach is undermining the objectives of the Government and industry. Examples of a lack of a whole of Commonwealth government approach are:

- Failure for security to flow into policy planning. It is essential that security issues flow into all major infrastructure planning initiatives but this does not seem to be occurring. An example of this is the November 2002 Commonwealth green paper on land transport infrastructure reform, *AusLink: Towards the National Land Transport Plan*. AusLink aims to improve the national land transport network by integrating the separate rail and road networks and ad hoc intermodal developments. Nowhere in the document were issues of security considered.
- Failure to encourage economic regulators of infrastructure to consider issues relating to security investment. Currently, the majority of the economic regulators do not have a process which can quickly advise an organisation if its proposed security related investments can be recouped via consumer price rises. Typically, an agreement on the price charged to consumers will be determined by an economic regulator with input from the infrastructure supplier and other interested parties such as major customers. It may run for up to five years and have a provision for price rises based on some CPI minus X factor (that is Consumer Price Index minus *X*, where *X* is an annual operating cost reduction).

Applications to vary an agreement will normally take about six months to assess, as it requires significant public consultation and regulator scrutiny. To approve the variation, the regulators will have to be convinced that the investment is prudent. If a variation is not approved, the supplier cannot pass on the costs to its customers so are faced with the choice of not spending money or internalising the costs. The existing process is unsuitable for expenditure that is urgently required, such as following a security alert by government, or after a disaster, such as the 2003 Canberra bushfires. Consequently, a more streamlined variation process as needed which allows any necessary security investment to be undertaken quickly.

- A failure for security to flow into tax policy. Private investments in infrastructure is being held up by various tax issues. In particular, there is a need to change two sections of the Income Tax Assessment Act – Section 51AD and Division 16D – which deny normal business tax deductions, such as depreciation, to the legal owner of assets where those assets are used to benefit the public. Two of the consequences of poor tax law are that it is uneconomic to develop interconnections between systems and duplicate infrastructure, both of which increase system redundancy.

4 Securing engineering excellence

Engineering excellence cannot overcome poor policies, procedures and practices. Only when a sound security system exists can the engineering contribution deliver to its full potential. The engineering contribution can take many forms including the efficient organisation of resources, technical management and physical design. In the short term, the greatest contribution from engineers will come from their assistance in the development and improvement of policies, procedures and practices, particularly those that have a technological dimension. Over the longer term, a significant contribution will come from developing designs that engineer-in security by designing-out the potential for human error, reducing opportunities for malicious attacks, and aiding the response and recovery effort following a disaster.

The secret to maximising the engineering contribution is to make engineers involved from the early stages of any potential project. By including engineering professionals in multi-disciplinary teams alongside, for example, emergency services, human resource, finance personnel, far better outcomes will be generated than if they were only involved after the project has been defined in detail. This reflects the fact that for a typical structure, about 80% of its costs are fixed after it leaves the concept stage.

A multi-disciplinary approach is also essential in identifying potential problems that may prevent a solution from working effectively in a system. For example, while a biometric security device may be effective as a stand-alone technology, it will fail to work as a system if there are no procedures in place to protect the privacy of the data or if there is adequate technology to ensure the data's integrity.

The prerequisites for an engineer to effectively contribute to engineering a safer and secure built environment include:

- Engineers moving from a culture of compliance to an intuitive culture. Currently much engineering, like other professions, is based on compliance. Compliance with quality systems, performance systems and codes. Just as can be observed with corporate governance compliance, too much attention given to complying with processes will result in little added value and reduced focus on outcomes. Given that hazards are continually evolving, engineers need to develop a more intuitive attitude that focuses on delivering better outcomes for clients while meeting compliance requirements.

- Engineers need to employ leading edge engineering practices rather than waiting for them to be incorporated into codes and standards. For example, if new peer-reviewed research identifies weaknesses in the existing high strength concrete design rules, an engineer designing a high rise tower should incorporate these finding into their work without waiting for them to appear in standards, which would take many years. In addition, because much best practice does not appear in codes (partly due to not wanting to provide terrorists with a manual of vulnerabilities), engineers should constantly seek out new developments and maintain current proficiency.
- Clients allowing more time for engineers to deliver secure structures. The last two decades has seen a reduction in time and money spent on the design of structures. This has resulted in increased flaws, errors and omissions during the design process. It has also resulted in more template designs where a standard design is simply duplicated with little allowance for customising it for the environment. These tendencies are anathema to designing secure structures. Security can be built-in at relatively little cost as it invariably means providing more time for analysis, detailing and construction oversight. Spending an additional 1% on a structure could result in a building becoming significantly more secure and resilient to catastrophic collapse.
- Increasing confidence in clients of the security capabilities of engineers. One way to do this may be for the registration of competent engineers working in security areas. Registration already occurs in many areas critical to public safety such as the Victorian Building Practitioners Board which prescribes the qualification and experience of fire safety engineers and the Federal Government's Construction and Equipment Committee which approves security engineering consultants in the areas of physical protective security. Areas where registration may be appropriate include disaster assessment of built facilities, blast protection, and systems engineering security. The National Professional Engineers Register may be an appropriate vehicle for the registration of security engineers.
- Engineers utilising new design methodologies such as *design for non-failure*. Most engineered systems are designed without a detailed safety and operational analysis on the consequences of a component failing. Applying *design for non-failure* analysis can be difficult even given today's simulation tools but is increasingly becoming necessary and demanded by clients.

The rest of this section describes the priorities by which engineering excellence can be implemented and fostered. These are:

- Implementing a 3-year program to improve relevant codes, standards and guidelines.
- Improving knowledge sharing.
- Enhancing security skills for engineers and other built environment professionals.

4.1 Implementing a 3-year program to improve relevant codes, standards and guidelines

The 11 September 2001 attacks have highlighted the need to update many *standards documents*. These include those relating to physical, electronic, biological, chemical and radiological hazards.

While most infrastructure sectors have generally updated their sector specific codes, many multi-sector codes which industry relies on have not been updated. An example of this is structural design codes. Previously, the hazard that presented the most concern was inadequate strength of structural members under loads such as winds or earthquakes to prevent displacements causing catastrophic failure. With the advent of aircraft being used as a weapon, an additional hazard must now be considered. Structures must now be ductile enough (ie stretched without breaking) to withstand large energy shocks (such as very high blast pressures) without catastrophic failure.

Structure codes are central to the security of buildings and many other structures which house critical infrastructure assets. It will not be simple or quick to incorporate the latest security engineering knowledge in any individual *standards document* because most take several years to revise due to the mechanism of drafting, reviewing and accepting changes. For example, the *Loading Code* and *Concrete Code* will not reflect the experience from the World Trade Centre for at least another two years. The revision of the *Loading Code* is yet to start, while it is anticipated that the *Concrete Code* will be available for public comment in early 2004, it will not be released in final form until early 2005.

4.2 Enhance security skills for engineers and other built environment professionals

Australia has a limited base of engineers and other built environment professionals who have high level security skills. This is only to be expected where security has not been a fundamental concern in the past.

Currently, this issue is not being comprehensively addressed by tertiary institutions at either undergraduate or post-graduate level. With few exceptions, security does not appear to be a major or even a minor component of undergraduate engineering courses outside of the IT areas.

For practicing engineers and other built environment professionals, there are few continuing professional development opportunities to improve their security knowledge. Where there is an opportunity the content is related mostly to IT security or physical security in a defence context. There is almost nothing that addresses biological, chemical and radiological hazards.

To address these deficiencies, the following options should be considered:

- Introducing security units into all undergraduate engineering courses. To facilitate this, a standard module should be developed and made freely available to all tertiary institutions and other education providers. The Australian Council of Engineering Deans could play a significant role in implementing this.
- Continuing professional development activities on security issues should be developed. Engineering Education Australia could play a significant role in providing development activities online and/or by distance education.
- A post-graduate security infrastructure and built environment course should be developed.

Programs targeted at small groups of professionals should also be considered. For example, the most efficient way to raise the skills of the dozen or so engineers who design skyscrapers in Australia is to organise an overseas study tour for them to gain knowledge of international best practice. Within this group, selected engineers should be additionally qualified to treat extreme conditions such as blast effects. These people would then form the cadre of a national continuing education program for the training of Australia's engineers, officials and builders in multi-hazard design and construction. Notably areas where specialist skill development activities are urgently needed include:

- Post-disaster assessment of built facilities.
- Disaster relief engineering.
- Risk management dimensions of extreme events with low probability.

5 Conclusion

Currently Australia is about 18 months behind the US in its work to protect critical infrastructure and the built environment. The recommendations below can rapidly reduce this gap by allowing the entire nation, and the engineering sector in particular, to contribute under a domestic security strategy. Without these reforms, the current slow pace and uncoordinated efforts will continue.

1. **Australia should develop a whole of nation, 10 year domestic security strategy that encompasses all elements of the built environment and is based on a true partnership between Commonwealth, State/Territory and local governments, industry and the professions.**
2. **Australia should produce a biannual *State of Australian Domestic Security Report* which ensures a focus on strategic issues and provides a benchmark to measure performance.**
3. **Governments should broaden the security agenda to encompass all elements of the built environment not just critical infrastructure.**
4. **Governments, industry and the professions should broaden their focus from the current dominant hazards and risks to encompass all hazards and risks.**
5. **A three year program should be instigated to accelerate the incorporation of security best practice into all relevant codes, standards and guidelines or develop new ones if required.**
6. **A range of knowledge sharing networks be established to facilitate the rapid dissemination of engineering security best practice.**
7. **Governments, industry and the professions should implement a program of security skills enhancement for engineers and other built environment professionals.**
8. **Governments, industry and the professions should accelerate the quantum of innovation, research and development of security technologies and practices, and their dissemination throughout the built environment.**

SEISMICITY IN THE SOUTHWEST OF WESTERN AUSTRALIA

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ABSTRACT

As part of the Cities Project, an earthquake hazard and risk model is being developed for the Perth metropolitan area. Development of the hazard model requires information on historical earthquakes, underlying geology and local soil characteristics. In December 2002 a workshop was held in Geoscience Australia with the purpose of defining an appropriate model of seismicity, in particular to formulate geological and seismological parameters for the Southwest Seismic Zone.

Utilising knowledge of seismicity and tectonic settings, the seismic zones were determined and their seismic parameters used to develop hazard models. These models are used to calculate spectral accelerations and to produce maps for specific return periods.

This study has found different acceleration values to the current earthquake hazard map. The most significant factors likely to dominate the results include the selection of the *a* coefficient, choice of attenuation function, and maximum magnitude cut-offs. In this approach we investigated all geological and seismological data currently available and found the optimal values of the seismic parameters which influence the hazard modelling process in the Perth region.

Paper No. 3

1. INTRODUCTION

As part of the Cities Project, an earthquake hazard and risk model is being developed for the Perth metropolitan area. Development of the hazard model requires information on historical earthquakes, underlying geology and local soil characteristics. In December 2002 a workshop was held at Geoscience Australia. Utilising the workshop participants knowledge of seismicity and tectonic settings, the seismic zones were determined. The seismic zones and their parameters were used to calculate spectral accelerations and to produce hazard maps for specific return periods.

The exact boundaries of the Southwest seismic zone are not precisely defined, but is generally referred to as the Yilgarn Block within the Archaean Shield structure. The geology of the wider Perth region is presented in Figure 1. Three large earthquakes have caused considerable destruction and surface ruptures in the Southwest WA (SWWA). They are the Meckering 1968 earthquake M_L6.9, the Calingiri event of 1970 M_L5.9 and the Cadoux earthquake of 1979 M_L6.2.

During 2001 and 2002, a series of earthquakes occurred near Burakin which exemplified the continual process of release of accumulated stress. The fault-plane solutions are usually represented with compression-dilatation planes popularly called "beachballs". The current ones from the GA's database are also shown on Figure 1.

2. MODELLING

A review of intra-plate seismicity models identified a number of potential causes of intra-plate seismicity, based on areas which have a similar seismicity to SW WA. They include:

- I – Zone of weakness/resurgent tectonics;
- II – Lateral density contrasts;
- III – Contrasts in elastic properties; and
- IV – Increased heatflow.

The geophysical measurements and the spatial distribution of epicentres were used to define a new model of seismicity for the SW WA (Figure 2), comprising:

a) Cadoux-Meckering zone, or Zone 1, modified from Gaull *et al.* (1990) to include the latest series of Burakin events;

b) Larger zone east of Darling Fault, or Zone 2, with modified borders from the Gaull *et al.* (1990) to follow closely the Darling Fault and the structural orientation;

c) Yilgarn zone covering the whole Yilgarn craton;

d) Offshore zone, or Zone 3, as part of the continental margin, modified from Gaull *et al.* (1990); and

e) Perth basin, or background zone, as part of the continental shelf, modified from the EPRI 1994 report and Gibson and Brown AUS5 model (2002).

Note: The model had two versions: the first one when epicentres in Zone 1 are excluded from Zone 2, and Zone 2 epicentres further excluded from the Yilgarn zone during seismic parameters calculation, and second version when Zone 2 and the Yilgarn zone are combined as one zone.

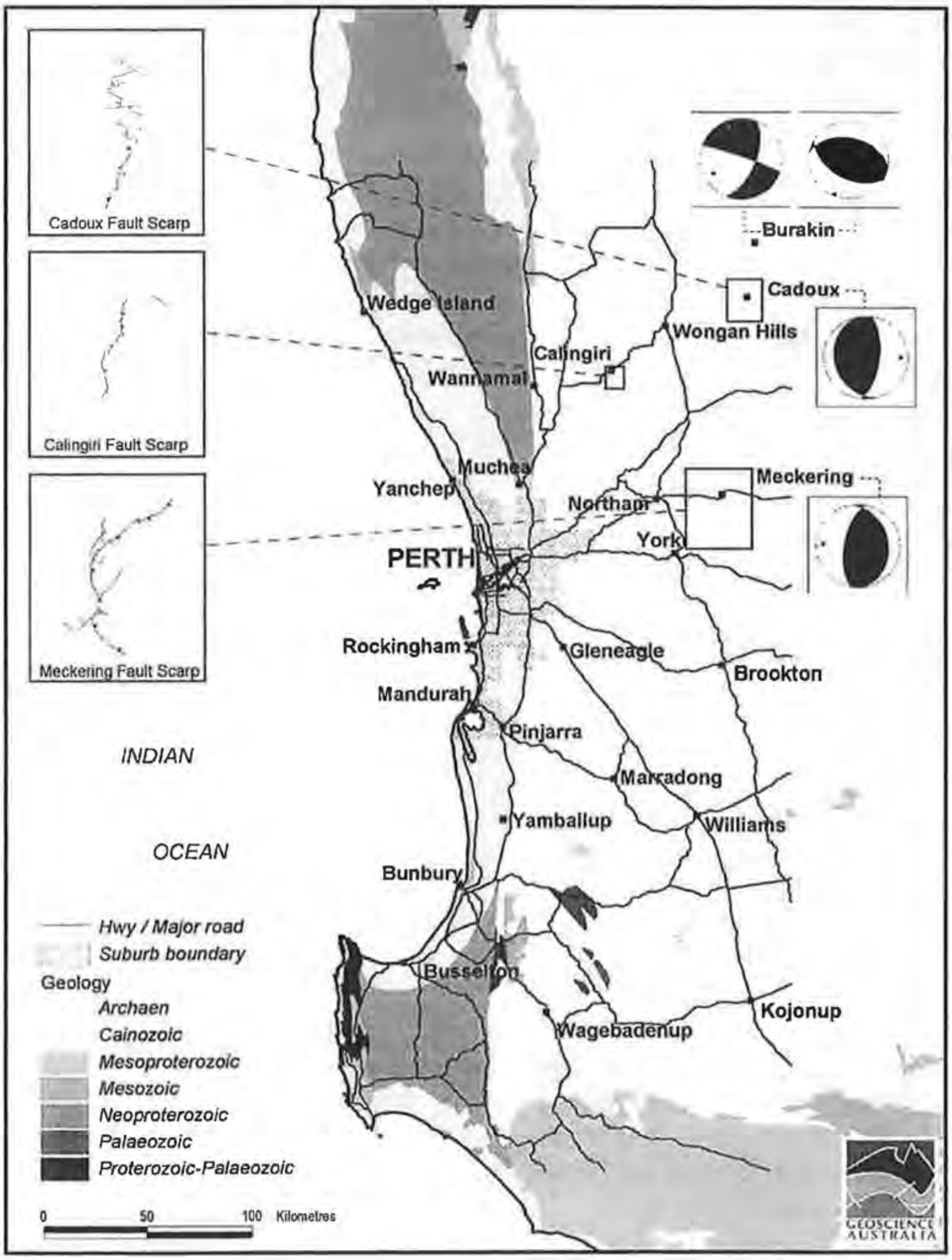


Figure 1: Geology of the SW WA with indication of the three largest earthquakes

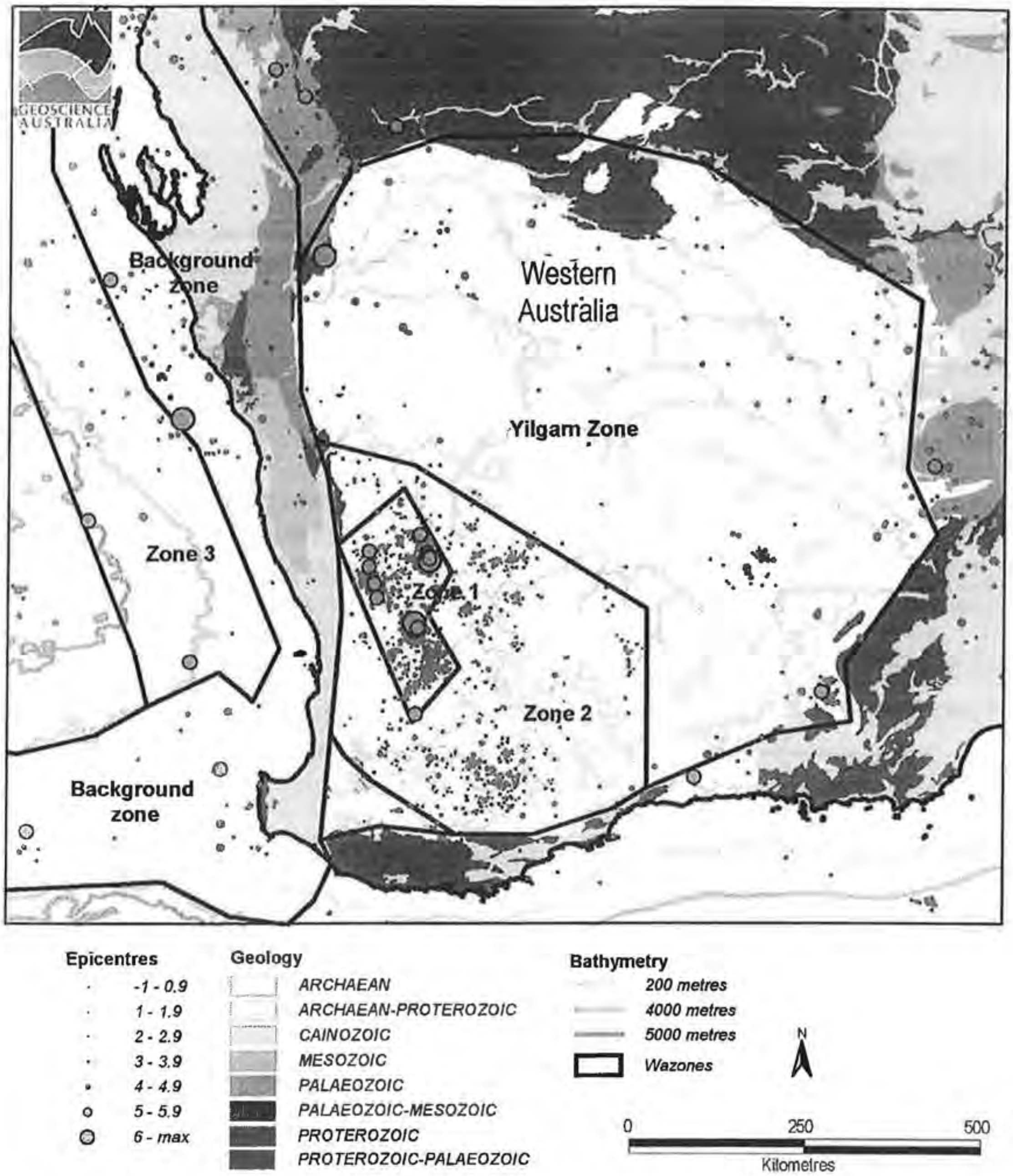


Figure 2: Seismic source zones of Southwest WA

Maximum magnitudes of 7.5 or 8.0 were assigned to the zones. The background had seismic parameters defined for the large continental shelf zone. The distributions of earthquake hypocentres were in the top 20 km. The boundaries in zones 1 and 2 were designed to reflect the general structural trend NNW-SSE. The mechanisms were anticipated to be mainly aligned with it and partially strike-slip varying around the principal horizontal stress trend. The likely position of dipping faults were set to 35° equally proportional to East and West.

3. EARTHQUAKE RECURRENCE RELATIONSHIP

The relation between the number of earthquakes and their magnitude is routinely approximated by the Gutenberg and Richter (1949) empirical formula represented by a single straight line in log-linear coordinates

$$\log N = a - bM$$

where N is usually the cumulative number of earthquakes per year, M is the local or Richter magnitude and a, b are constants related to the level and the slope. The data used for this study comprise a subset of the GA earthquake database for Australia. Analysis was restricted to only those time intervals in which the seismic network was able to consistently record all earthquakes of the specified magnitude in the Australian continent, with a minimum magnitude threshold of 3. Numbers of earthquakes were counted for the declustered dataset when identifiable foreshocks or aftershocks were removed in a procedure described by Sinadinovski, 2000.

The coefficient b usually takes a value around 1. In general this relationship fits the data well on global scale, although deviates for some inter-plate tectonic regions (Kárník, 1971). We have adopted the b coefficient of 1 for our study, while the values of a coefficients vary. The recurrence relationship for our model for earthquakes in southwest Western Australia normalised to 100,000 km^2 and with data up to 2002 is shown in Figure 3.

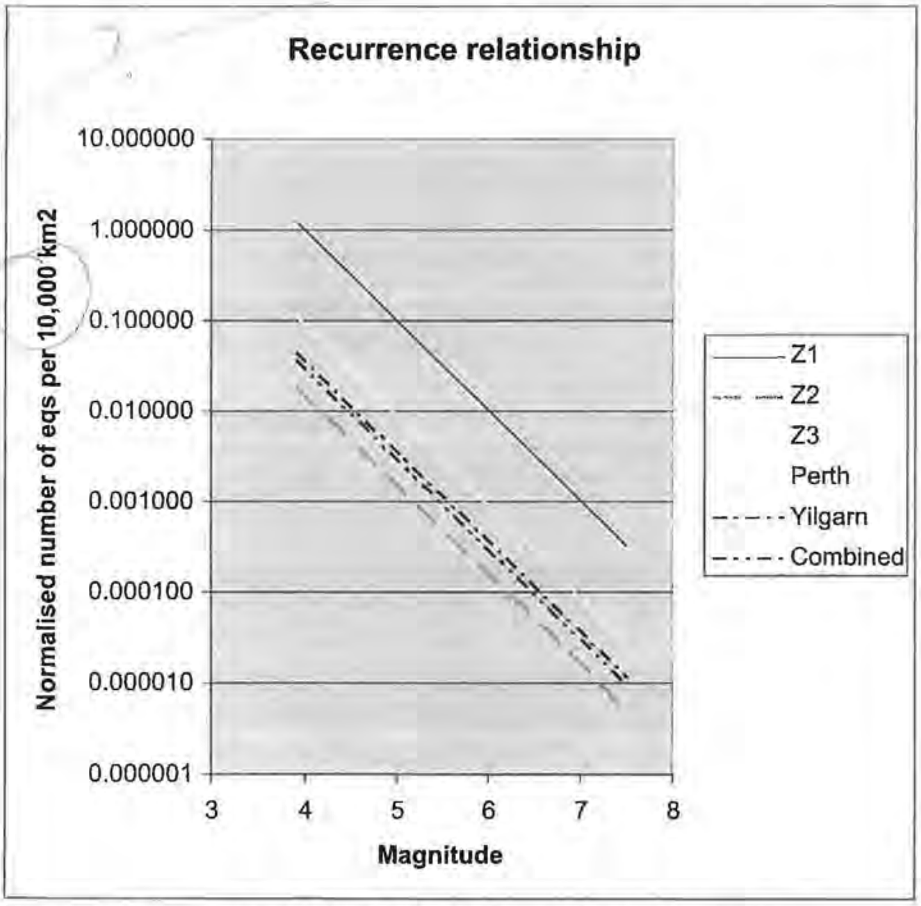


Figure 3: Recurrence relationship for the areal seismic zones

4. RESULTS

In our program we considered different empirical relationship for intensity, peak ground acceleration (PGA) and typical spectral amplitudes (between 0.3 and 1.0 second). Numerous attenuation relationships have been developed for intra-plate regions. For example Lam *et al.*, (2002) has recently completed a comparison between many of these intra-plate attenuation models and Australian data.

Here we show the preliminary results for PGA using Toro *et al.* (1997) and Gaull *et al.* (1990) attenuation formulas to compare it with the earthquake hazard model map of Australia - 1993 AS 1170.4 (Figure 4). The complete analysis of the workshop of December 2002 will be published separately.

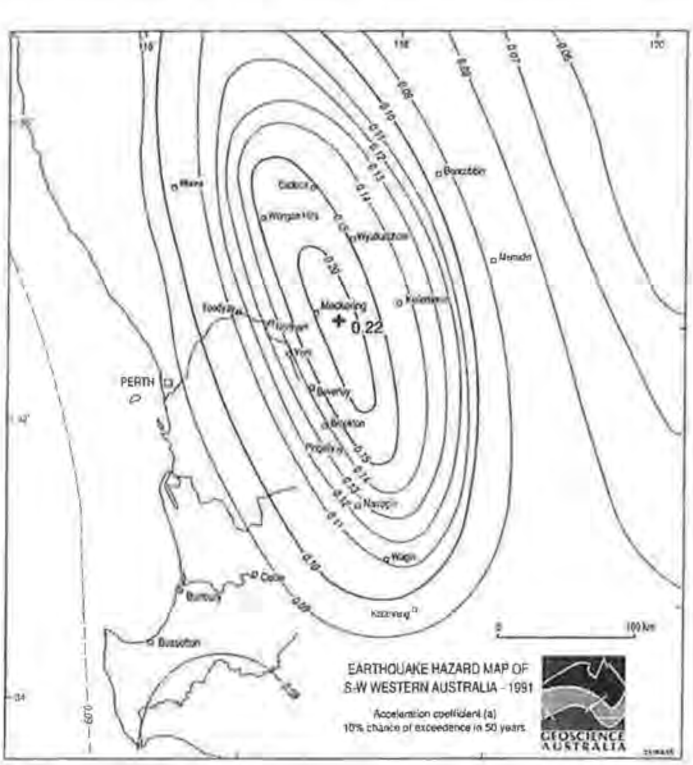
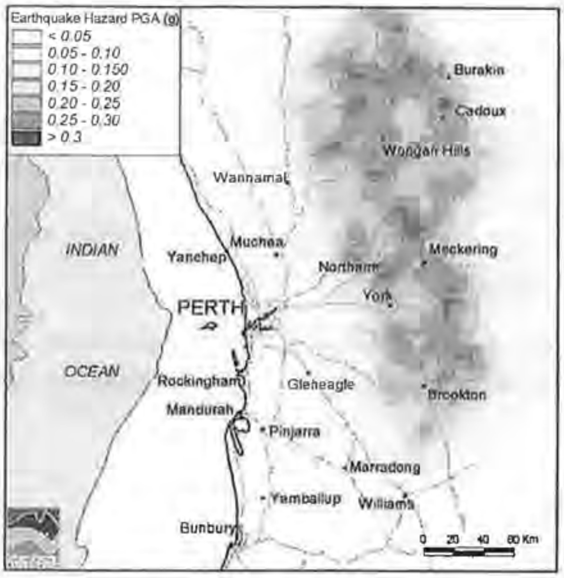
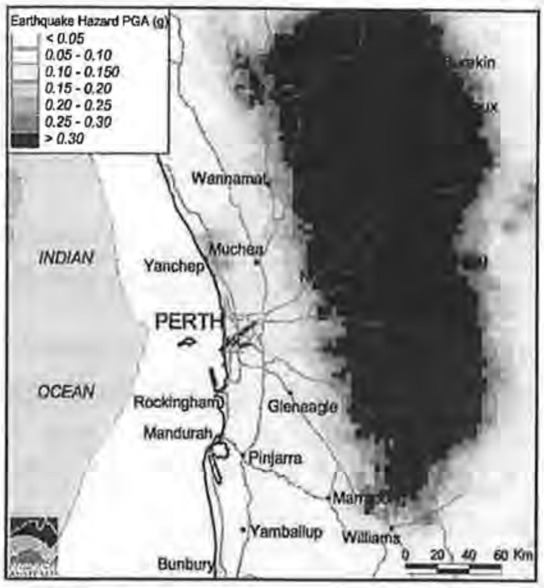


Figure 4: Earthquake hazard map of Western Australia

The differences between Fig. 5a and Fig. 5b is the result of a difference in the level of the mean and the variability associated with the Gaull *et al.* (1990) and Toro *et al.* (1997) attenuation functions. That is, Toro *et al.* formula predicts a slightly higher mean values and a larger variability with its estimates of ground motion. Gaull (pers. discussion, 2002) has indicated that the uncertainty in the Gaull *et al.* (1990) formula has been under estimated.



a) PGA using Gaull *et al.* (1990)



b) PGA using Toro *et al.* (1997)

Figure 5: Acceleration coefficients for the Perth region for return period of 475 years

5. SUMMARY

The different acceleration values between Fig. 5 and Fig. 4 could be the result of a method obtained to produce them (intensity in case of the Australian Standard) and our program selection. Most likely factors to dominate in our program are the variation of the Gutenberg-Richter *a* coefficient, the choice of attenuation function and its associated variability, and the range of maximum magnitude in the zones.

Our study identified the major geological and seismological components in the Perth region which might influence the hazard modelling process. A more detailed sensitivity analysis is to be subsequently performed.

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SEISMIC GROUND MOTION PREDICTION AND RC FRAME STRUCTURE FAILURE PROBABILITY ESTIMATION IN LOW SEISMIC SINGAPORE

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Amy Kirke graduated from the School of Civil and Resource Engineering, at the University of Western Australia in 2002 with first class honors. She also received the best woman engineering graduate award. Currently she works at Wood & Grieve Engineers as a structural engineer.

ABSTRACT:

See over

Seismic Ground Motion Prediction and RC Frame Structure Failure Probability Estimation in Low Seismic Singapore

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ABSTRACT

This paper presents simulation results of ground motions on soft soil sites in Singapore generated from a M_w=9.0 event occurring at 550 km away in Sumatran Subduction Zone in Indonesia, and the dynamic responses of low- medium- and high-rise non-ductile RC frame structures. The ground motion time histories on rock site are stochastically simulated first based on a 95% confidence level response spectrum corresponding to such an event. The simulated time histories are then used as input to various soft soil sites to estimate ground motions on soil surface. The input motions are assumed consisting of SH wave or combined P and SV wave in the calculation. Statistical variations of soil properties are considered. The maximum credible ground motion time histories with a 95% confidence level on soil site are determined. Those time histories are then used as input to calculate structural responses. Soil structure interaction effect and statistical variations of structural parameters are considered in the analysis. Rosenblueth's Point Estimator method is employed for statistical calculation. The estimated statistical storey drifts are compared with the SEAOC recommended storey drift limits for various structural performance levels. The failure probabilities of the RC frame structures in Singapore corresponding to a worst-case scenario Sumatran Subduction Zone event are estimated.

INTRODUCTION

Singapore is located on the southern tip of the Malay Peninsula, an area of low seismicity. The closest identified seismic sources, the Sumatran fault are more than 300 km, and the Sumatran Subduction Zone more than 500 km from Singapore. Many Sumatran seismic events have been felt in Singapore, especially in the last three decades after construction of many high-rise buildings. The magnitudes of those events range between 5.5 and 7.6, and epicentral distances are from 500 km to more than 700 km. They did not cause any structural damage in Singapore, but disturbances to high-rise building residents. As such the current design code does not take into consideration seismic loading, besides a nominal lateral load of 1.5% of the total building weight.

A recent study of historical Sumatran events revealed that the largest possible earthquake in the Sumatran fault is about M_w=7.5, and that in the subduction zone is 9.0. The study also indicated that the return period of the magnitude 9.0 event, which occurred last time in 1833 with an epicentral distance of 550 km to Singapore, is 250 years [Sieh and Natawidjaja 2001].

A previous study indicated that a magnitude 8 epicentral distance 300 km event occurring in Sumatran fault might cause damage to high-rise RC structures on soft soil sites in Singapore [Duan and Hao 1994]. The present study investigates the effect of a magnitude 9.0 and epicentral distance 550 km event on RC non-ductile frames on soft soil sites in Singapore. Stochastic ground motion time history on rock site is simulated according to a predicted 95% confidence level response spectrum [Megawati and Pan 2002]. Random field method is employed to calculate site amplifications of the seismic wave propagating through a random soil site [Wang and Hao 2002]. The worst-case scenario, namely the 95% confidence level maximum credible ground motion on soil surface is estimated.

The estimated ground motion time history on soil surface is then used as input to analyze nonlinear inelastic responses of RC non-ductile low-rise, medium-rise and high-rise frame structures. The effects of uncertain structural parameters, such as the material properties and structural member dimensions, are included in the analysis. The Rosenblueth's Point Estimates Method is used to estimate the mean and standard deviation of the various response quantities. The accuracy of the numerical results is verified by Monte Carlo simulation method. The estimated statistical storey drifts are compared with the SEAOC recommended storey drift limits for various structural performance levels [SEAOC 1995]. The failure probabilities of the RC frame structures in Singapore corresponding to a worst-case scenario Sumatran Subduction Zone event are estimated.

BEDROCK GROUND MOTION SIMULATION

Using an extended reflectivity method with random but seismologically possible rupture model, the maximum credible (95% confidence level) response spectra of horizontal and vertical component of bedrock ground motion in Singapore corresponding to a M_w=9.0 and epicentral distance 550 km event were generated [Megawati and Pan 2002]. These response spectra are used to simulate bedrock ground motion time histories. In simulation, the duration of the ground motion is estimated by an empirical formula derived from long distance earthquake records [Reinoso and Ordaz 2001],

$$T = 0.01e^{M} + (0.036M - 0.07)R + (4.8M - 16) \times (T_s - 0.5) \quad (1)$$

in which M is earthquake magnitude, R is epicentral distance in km and $T_s=0.5$ for rock sites. The ground motion time history as simulated as a modulated stationary process as

$$a(t) = \xi(t)s(t) \quad (2)$$

where $\xi(t)$ is a Bogdanoff envelope function for earthquake ground motion [Bogdanoff 1961], and $s(t)$ is a stationary time history, estimated from

$$s(t) = \sum_{i=1}^{n} A(\omega_i)\cos(\omega_i t + \phi_i) \quad (3)$$

in which the amplitude $A(\omega_i)$ at frequency ω_i is determined from the response spectra, and ϕ_i is a random angle uniformly distributed between 0 and 2π [Hao 1989].

Figure 1 shows the simulated bedrock motion time histories and their response spectra as compared with the maximum credible response spectra for bedrock motions in Singapore. As shown, the strong motion duration is about 220 s for such an event, and the vertical component is larger than that of the horizontal component. The PGA for horizontal and vertical component is respectively 39.36 cm/s^2 and 70.70 cm/s^2.

SITE AMPLIFICATION ESTIMATION

A few methods have been developed to calculate soil site amplification to incoming seismic waves from bedrock. In this study, wave propagation method developed by Wolf [1985] is used. Many studies indicated significant variations of soil properties. For example, it was found that the coefficients of variation for cohesion and undrained strength of clay and sand are about 10% to 60%, and the vertical correlation distance is between 1-5 m, and the horizontal correlation distance is in the order of 50 m [Phoon and Kulhawy 1996].

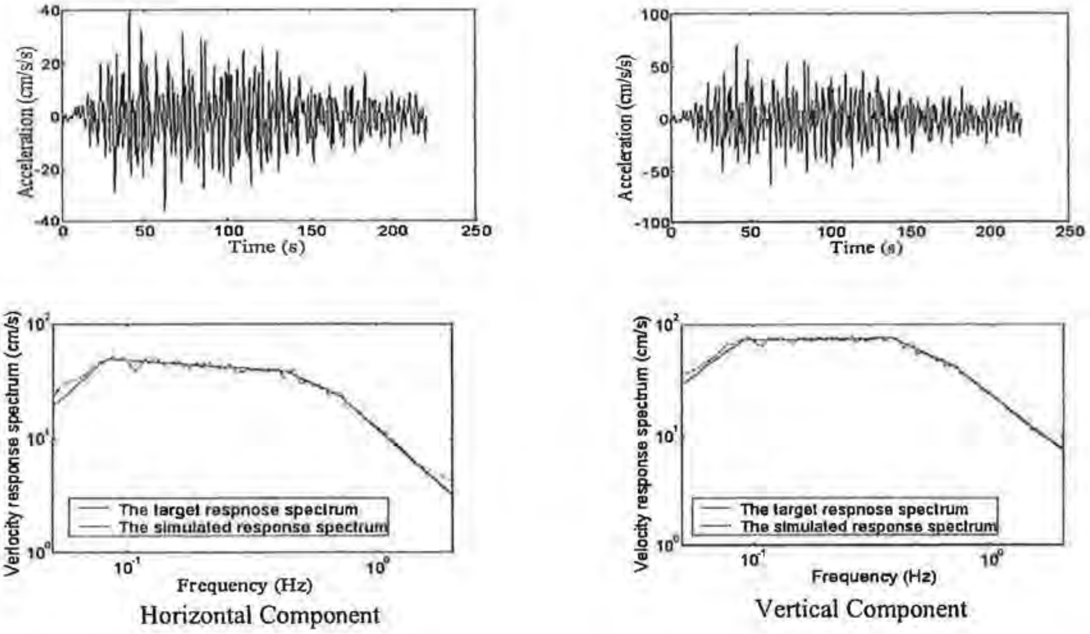

Fig 1. Simulated bedrock ground acceleration time histories and response spectra

Recent study also indicated that water saturation degree of a site has a pronounced effect on P-wave propagation [Yang and Sato 2000]. When the saturation degree changes from 100% to 98%, the site amplification spectrum on P-wave is completely different. In view of this, the wave propagation method developed by Wolf [1985] was modified to account for the effects of statistical variations of soil properties and ground water level on site amplification [Wang and Hao 2002].

In the study, 6 soft soil sites are analysed. The sites were selected from the four major geological formations in Singapore. Here only the results from a deep stiff soil site are presented. It should be noted that this site resulted in the largest horizontal ground motion on ground surface among the 6 sites. Figure 2 shows the layered site. The mean values of the soil properties of each layer are given in the figure. The mean damping ratio is 5% for all the layers. In numerical calculations, the coefficients of variation of shear modulus and damping ratio are assumed as 60%, while that of mass density 5%. The vertical correlation distance is assumed to be 4 m and the horizontal correlation distance 50 m. The S-wave incident angle is assumed to be 60°. The mean ground water level is 2 m below the surface and varies between ±2 m with uniform distribution. The saturation level is assumed as 98% at the ground water level and varies linearly to 100% at 2 m below the ground water level. Figure 2 also shows the simulated largest credible (95% confidence level) acceleration time histories on ground surface. Figure 3 shows their response spectra. It should be noted that the y-component is simulated with SH wave assumption, and x- and z- component with the combined P and SV wave assumption. The PGA for y-, x- and vertical component is 128.52 cm/s^2, 132.56 cm/s^2 and 71.65 cm/s^2, respectively. The estimated time histories in the x- and vertical direction are used in the subsequent 2-dimensional structural response analysis.

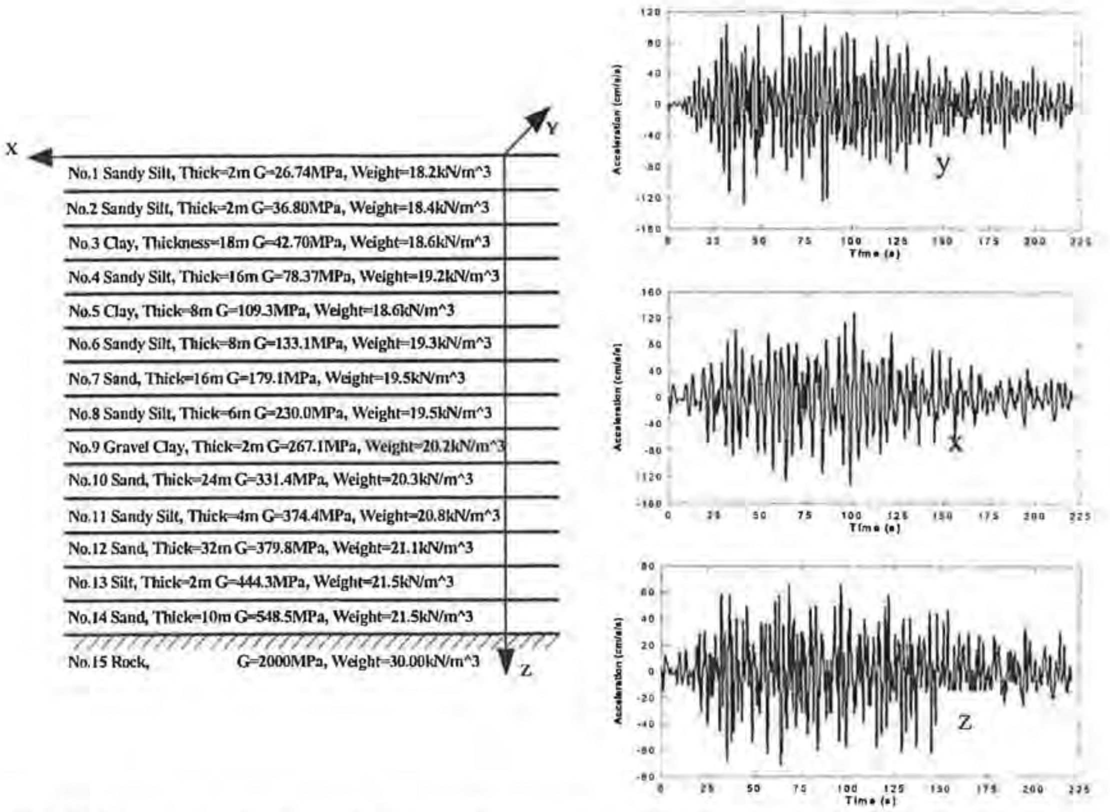

Fig. 2. Layered soil site and estimated ground acceleration time histories on site surface

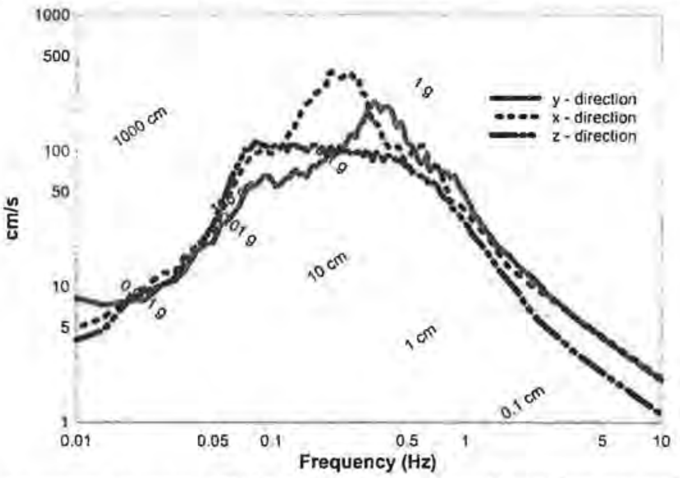

Fig. 3. Response spectra of the simulated ground motion on soil surface

RC FRAME MODEL

Three RC structures, namely a 2-storey single-bay, a 10-storey 2-bay and a 24-storey 2-bay frame are considered. The structures were designed according to BS8110 [British Standard Institution 1985] without considering seismic loading. The storey height is 3.5 m for the first storey and 3 m for the other storeys, and span length is 5 m for all the three structures. The design concrete material properties are: elastic modulus 27 GPa, uniaxial compressive strength 30 MPa, steel confinement ratio 0.0055, strain at maximum stress 0.002. The design steel material properties are: elastic modulus 200 GPa, yield stress 460 MPa, strain hardening ratio 0.08, and the ultimate strain 0.1. 5% Rayleigh mass and stiffness proportional damping is used. The total floor loading is 10 kN/m^2, and the floor area per frame is 30 m^2. All the beams were designed as 230mmx400mm with 2.5% tensile and 1.5% compressive reinforcement. The floor is 150 mm thick and all the beams are considered as T-beam in the analysis with inclusion of the floor stiffness. The external column size is 200mmx300mm for the 2-storey and 10-storey frame with 2% reinforcement; the internal column size for the 10-storey frame is 250mmx400mm from the ground to the fifth storey, and 250mmx300mm for the columns above, all with 2% reinforcement. For the 24-storey frame, the external column and internal column sizes and reinforcement ratios are: 1st to 4th storey, 250mmx600mm and 250mmx1000mm, 5% reinforcement; 5th to 7th storey, 250mmx500mm and 250mmx900mm, 4% reinforcement; 8th to 10th storey, 250mmx400mm and 250mmx800mm, 3% reinforcement; 11th to 13th storey, 250mmx350mm and 250mmx700mm, 3% reinforcement; 14th to 16th storey, 200mmx250mm and 250mmx600mm, 2% reinforcement. From 17th to 24th storey, the external column size remains unchanged as 200mmx350mm with 2% reinforcement, while the internal column size is 250mmx400mm from 17th to 19th storey and 250mmx300mm from 20th to 24th storey, all with 2% reinforcement.

Effects of soil-structure interaction are approximately modeled by using equivalent springs at the supports. The two-storey frame is assumed resting on a shallow foundation, and the 10-storey and 24 storey frames are supported on pile foundation at each support. The equivalent dynamic spring stiffness is estimated following the method proposed by Novak and El-Sharnouby [1983] and Gazatas [1990]. The dimension of the square shallow foundation is assumed as B=0.5 m, and the diameter of RC pile is d=0.226 m, soil elastic modulus 88.6 GPa, pile elastic modulus 250 GPa, and floating pile length 40 m.

The computer program DRAIND2DX [Prakash and Powell 1992] is used in the analysis. All the mass of the structure is lumped to various beam-column joints. P-Δ effect is considered in the calculation. The fundamental vibration frequencies of the three frames are estimated to be 2.83 Hz, 0.78 Hz and 0.42 Hz, respectively. Figure 4 shows the displacement, shear force, bending moment and storey drift envelope, estimated using the design values given above. It should be noted that the increase in storey drift in the 24-storey frame always occurs at floors with changes in the column sizes.

The Structural Engineers Association of California (SEAOC) has defined the performance level of RC frame structures with respect to the storey drift ratios [SEAOC 1995]. It specifies that if the storey drift ratio is less than 0.2%, no damage will occur; 0.2% to 0.5%, light (repairable) damage; 0.5% to 1.5%, moderate (irreparable) damage; 1.5% to 2.5%, severe damage and structure will collapse if storey drift is larger than 2.5%. These criteria indicate that the 2-storey frame will not experience any damage, whereas the 10-storey and 24-storey structures in Singapore, under the worst Sumatran earthquake scenario, will suffer moderate but reparable damage. The damage to the high-rise 24-storey structure is most pronounced because its fundamental vibration frequency falls in the dominant ground motion frequency range.

PROBABILISITC STRUCTURAL DAMAGE ESITMATION

The above results were derived using design values of the structural parameters. In reality, structural properties such as dimensions, material strength and modulus inevitably vary from the design value owing to construction quality control, deterioration and fatigue. A more accurate assessment of structural response and damage should take into consideration the structural property fluctuations.

In this study, the variation in elastic modulus, concrete and steel strength and structural dimensions are considered. The design value of each parameter given above is considered as the respective mean value. Based on literature review [Low and Hao 2001], the coefficient of variation (COV) of concrete modulus is 0.1, that of steel 0.0. The COV of concrete compressive strength is 0.11, and that of the reinforcing steel strength 0.08. The COV of the member dimension is 0.03. All the variables have normal distribution.

Rosenblueth Point Estimator method is used to estimate the mean and standard deviation of the maximum storey drift. The validity of the method was proved using Monte Carlo simulation. It also found that the maximum storey drift has a normal distribution. Using the SEAOC recommended storey drift ratio, it is estimated that the 2-storey frame has a 99.99% chance experiencing no damage, while the 10-storey frame has a 99.34% probability of suffering moderate damage, and the 24-storey frame has a 75.04% probability of suffering irreparable damage and 24.79% probability of severe damage.

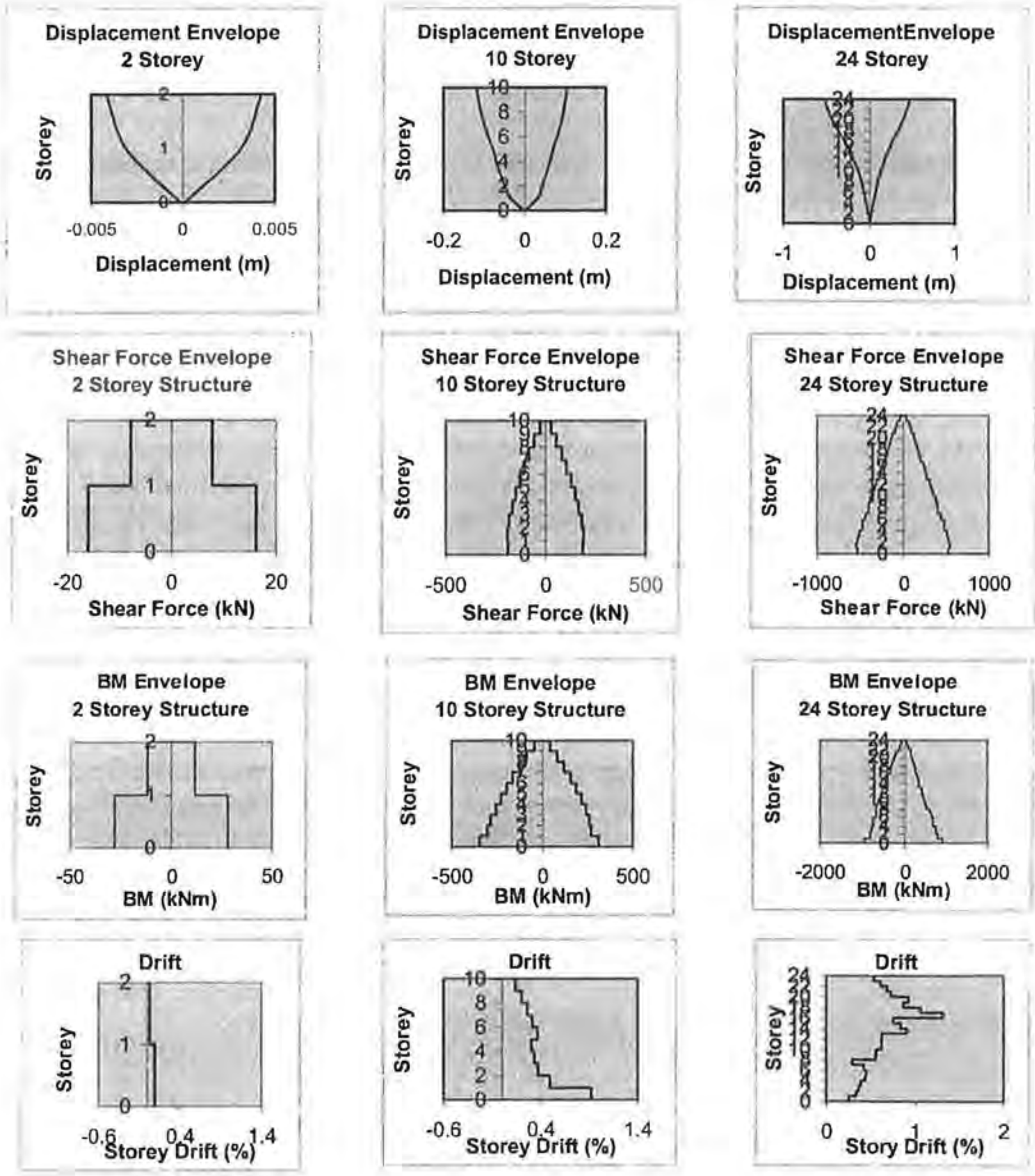

Fig. 4. Response envelopes of the three structures

CONCLUSIONS

This paper presents the simulated largest credible (95% confidence level) ground motion time histories on soft soil site in Singapore. Dynamic response and damage of three non-ductile RC frames designed according to BS8110 and subjected to the simulated ground motions are estimated. Using the SEAOC recommended storey drifts and performance levels, and taking into consideration the statistical variations of structural properties, it is founded that the low-rise RC structures in Singapore is very unlikely to experience any damage, but the medium-rise and high-rise RC frame structures are very likely to suffer irreparable to severe damage to the worst case scenario earthquake ground motions.

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NEAR SOURCE EARTHQUAKE GROUND MOTION SIMULATION AND A PRELIMINARY ANALYSIS OF THEIR EFFECTS ON STRUCTURAL POUNDING RESPONSES

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ABSTRACT:

See over

Paper No. 5

Near-Source Earthquake Ground Motion Simulation and a Preliminary Analysis of Their Effects on Structural Pounding Responses

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ABSTRACT

This paper presents results of a preliminary study of near-source ground motion effects on pounding responses of a simple bridge model. Although both horizontal and vertical components of ground motion time histories at the distance of 0 km, 5 km and 10 km on soft soil sites from a $M_s = 7.5$ event are simulated, in this study, only horizontal ground motions are considered in the bridge response analysis with soil-structure interaction and pounding between adjacent spans. The simulated time histories are compatible with the 5% damped pseudo-acceleration response spectra of near-source ground motions derived from hundreds of recently recorded motions by other researchers. Pounding responses of the bridge model resting on soft soil sites are calculated. Effects of soil-structure interaction and pounding between adjacent bridge spans are discussed.

1. INTRODUCTION

In the past, because of lack of recorded strong ground motion data in areas close to earthquake centers, most empirical relations of earthquake ground motions were derived based on data recorded in intermediate and far-field earthquakes. Studies of earthquake ground motion characteristics were also made primarily based on the strong motion data recorded in the intermediate and long distances.

Recently, a few major earthquakes, such as the Northridge, Kobe and Taiwan events, occurred near the metropolitan areas. Not only abundant data of near-source earthquake ground motions become available, but also the uniqueness of those ground motions on structural responses was observed. For example, some structural damage modes such as tensile cracks in concrete bridge piers and buckling of steel columns were observed, and those damage modes, which had not usually been seen in far-field earthquake events, are believed associated with a strong vertical component of the ground motion.

A few studies have been devoted to analyze near-source earthquake ground motion characteristics and their effects on structures. Those analyses were limited to using some recorded data in specific seismic events. The structural model used was single degree of freedom, and no soil-structure interaction effect is considered. It has been found that structural response characteristics to near-source earthquake ground motions could be very different as compared to those to far-field earthquake motions.

In this paper, near-source earthquake ground motions are stochastically simulated according to an empirical response spectrum for near-source motions derived from 186 strong motion time histories recorded within 15 km from the surface projection of the rupture plane. The simulated ground motions are used as input to analyze structural responses. Both pounding and soil structure interaction effect are considered in the analysis.

2. GROUND MOTION MODELING

After analyzing 186 strong motion time histories recorded worldwide within 15 km of the surface projection of the rupture plane of earthquakes between $M_s = 5.8$ and 7.8, Ambraseys and Douglas [2003] proposed empirical relations of acceleration response spectra of horizontal and vertical components of near-source earthquake ground motions. The empirical acceleration response spectrum is given as a function of the surface-wave magnitude M_s, the distance d to the surface projection of the rupture plane, and local site conditions. It has the form

$$\log y = b_1 + b_2 M_s + b_3 d + b_4 S_A + b_5 S_s \tag{1}$$

in which b_i, $i = 1 - 5$ are frequency-dependent constants; S_A and S_S are correction factors for different site conditions. It has $S_S = 1$ and $S_A = 0$ for soft soil sites. The constant b_i is given in the paper by Ambraseys and Douglas [2003] as a function of vibration period. It should be noted that the above relation does not include the focal depth as a parameter. Therefore the effect of focal depth of near-source earthquakes cannot be investigated.

In this study, the above empirical relation is used to estimate the acceleration response spectra of near-source ground motions on a soft soil site with d = 0km, 5km, and 10km. The estimated acceleration response spectra are used as target spectra in stochastic simulations of ground motion time histories generated from a $M_s = 7.5$ event.

Since most empirical relations for estimating earthquake strong ground motion duration were derived from recorded time histories far more than 10km, they might not be suitable for predicting the duration of near-source motions. For this reason, the semi-analytical formula for estimating the source duration is used [Boatwright and Choy 1992]. The duration is estimated by

$$T = 1/(2f_a) \tag{2}$$

where f_a is the corner frequency of earthquake source spectrum model. For a large event, it has

$$\log f_a = 2.41 - 0.533 M_b \tag{3}$$

in which M_b is the earthquake moment magnitude. Based on a previous study, at M_s = 7.5, M_s is approximately equal to M_b [Campbell 1985]. Thus the duration of the ground motion is estimated to be 19.34s. In actual simulation, a time increment dt of 0.005s, and a total duration of 20.48s is adopted.

The nonstationarity of the ground motion in the time domain is modeled by a shape function, whereas the motion is assumed stationary in the frequency domain. The Bogdanoff [Bogdanoff 1961] type of shape function is used, and it is assumed that the peak ground acceleration occurs at 6s.

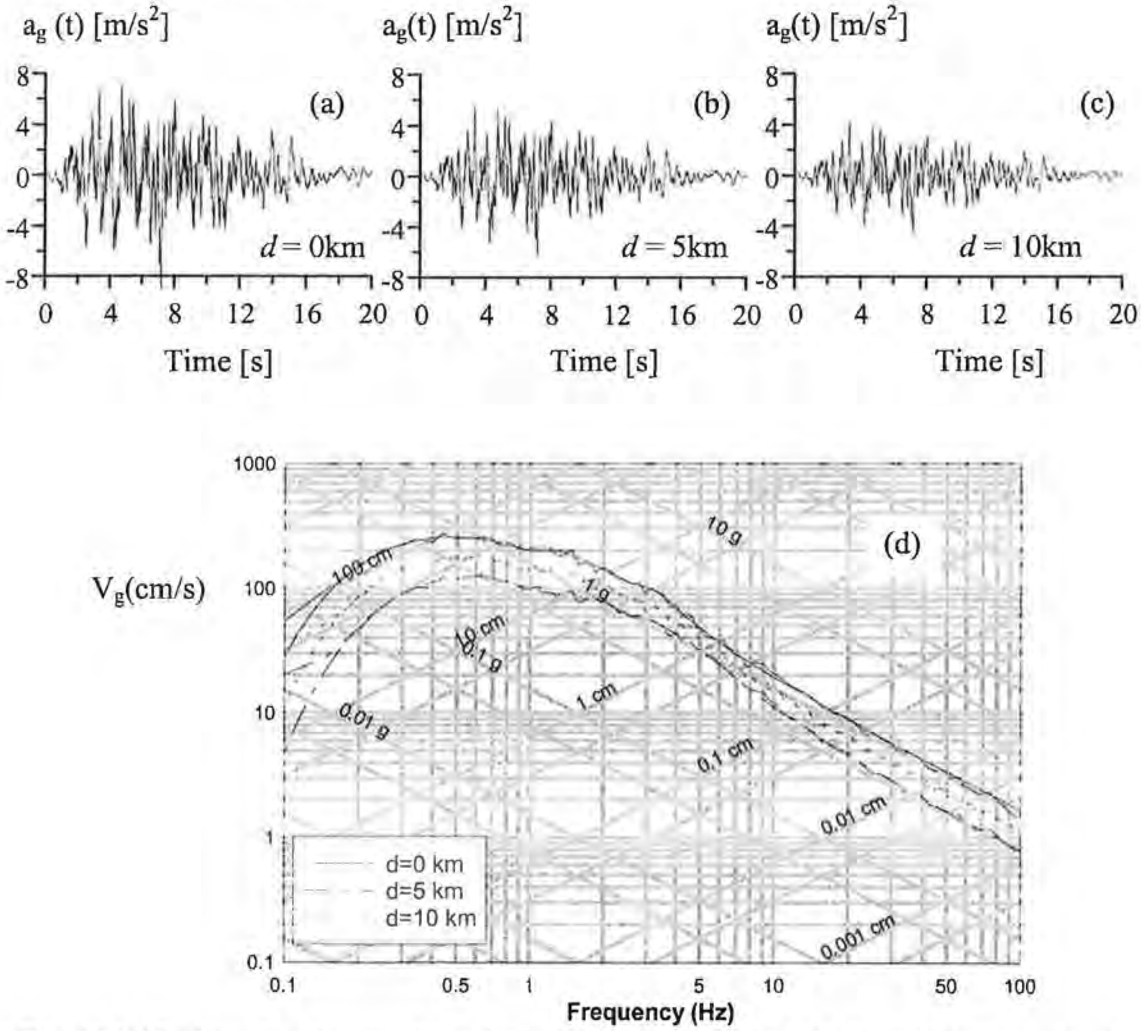

Fig. 1(a)-(d). Simulated ground motions at the distance d of (a) 0km, (b) 5km, (c) 10km, and (d) their corresponding response spectra with the empirical spectra.

The Bogdanoff shape function has the form as

$$\alpha(t) = a\, t\, e^{-b t^2} \tag{4}$$

The constants a and b can be determined by using the condition that α(t) reaches the maximum value when t is 6s. The simulation is performed in the frequency domain. More detailed information on stochastic simulation of earthquake ground motions can be found in the reference [Hao 1989]. Fig. 1(a)-(c) show the simulated time histories of the horizontal ground acceleration. The response spectra of the simulated time histories are compared with the empirical response spectra in Fig. 1(d). As shown, the response spectra of the simulated motion are compatible well with the empirical ones obtained from recorded data.

3. POUNDING RESPONSE ANALYSIS OF SOIL-STRUCTURE SYSTEM

The considered bridge in Fig. 2 is adopted from the work performed by DesRoches and Muthukumar [2002]. While in their investigation each of the adjacent bridge frames is modeled as a single-degree-of-freedom (SDOF) system with an assumed fixed base, in this study the multiple-pier bridge frames are idealized as single-pier bridge frames on subsoil. The material data of the bridge frames is given in Table 1. The damping of the bridge structures is considered by a complex Young's modulus [Hashimoto and Chouw 2002]. The real and imaginary parts of the modulus are a function of the Kelvin-chain parameters E_1 of 0.1 and E_n of 10^{28}. For these chosen parameters the equivalent damping ratio is 1.4%.

It is assumed that the subsoil is a half-space with a shear wave velocity of 100m/s, a density of 2000kg/m^3, and the Poisson's ratio of 0.33. Since the simulated ground motions are too strong for the considered structures, only 50% of the magnitude is taken into account. Both pier foundations are about 100m apart. Depending on the apparent wave propagation velocity the ground motions at the foundations can be different, and can therefore affect the adjacent structures differently, as indicated by Zanardo et al. [2002]. In this preliminary investigation, however, it is assumed that both pier foundations experience the same ground excitation.

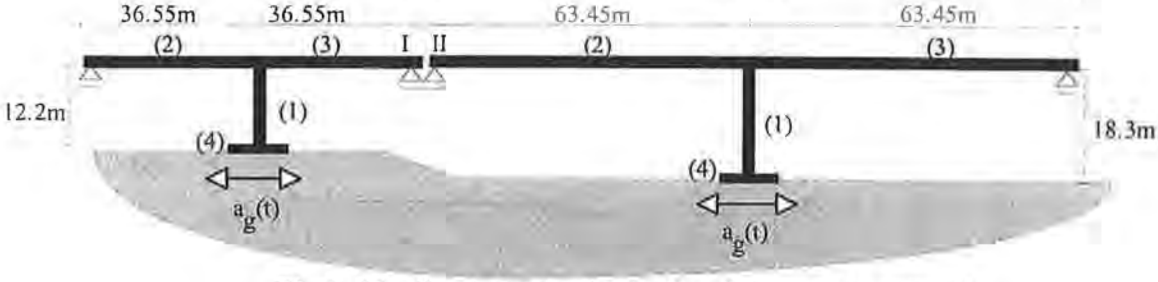

Fig.2. Idealized single-pier bridge frames with subsoil

Table 1. Material data

	Left bridge frame				Right bridge frame			
No	Mass [t/m]	EA [10^8kN]	EI [10^8 kNm^2]	Length [m]	Mass [t/m]	EA [10^8kN]	EI [10^8 kNm^2]	Length [m]
(1)	5.26	1.407	1.546	12.2	7.89	2.111	2.32	18.3
(2)	75.5	63.42	50.49	36.55	108.75	63.42	50.49	63.45
(3)	75.5	63.42	50.49	36.55	108.75	63.42	50.49	63.45
(4)	91.5	768.6	1024.8	9.0	91.5	768.6	1024.8	9.0

Pounding response of adjacent structures is often investigated without a consideration of the soil-structure interaction effect. Even if the subsoil influence is taken into account, it is normally only approximated by using frequency independent dynamic soil stiffness [e.g. Rahman 1999]. In this study the bridge frames with foundations are modeled by finite elements, and the subsoil by boundary elements in the Laplace domain. By using the obtained dynamic stiffness of the soil-structure system the linear response of the adjacent structures can be calculated. If the two girders collide at time t_j, we define the difference value at the pounding location, and condense the dynamic stiffness of one of the frame-soil system. Since the two structures are now in contact, we add the condensed stiffness into the stiffness of the uncondensed neighbouring frame-soil system, and obtain the corrective results. After transforming these results into the time domain, the pounding effect can be incorporated in the previous linear result from the time t_j. We examine the corrected result, and define the time t_k when the bridge girders separate. The unbalanced forces due to the separation are equal to the contact forces. By using the governing equation of the uncoupled system we obtain the corrective results. The influence of the girder separation can be considered in the time domain. We examine the response again for poundings. The analysis is complete when no more pounding or separation. The nonlinear procedure in the time and Laplace domain is described for a SDOF system by Chouw [2002].

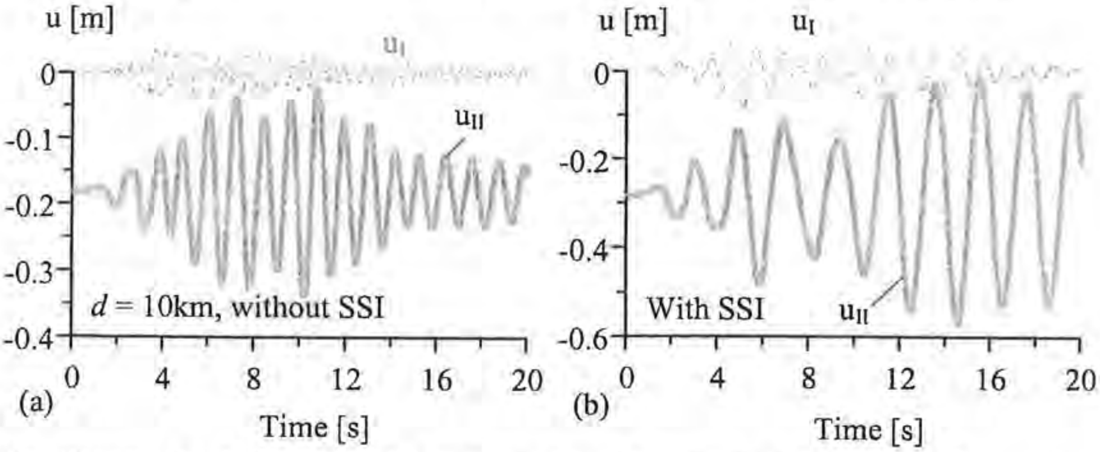

Fig. 3(a) and (b). Influence of the soil-structure interaction on the structural response

Fig. 3(a) and (b) show the linear response u_I of the left girder and u_{II} of the right girder without and with the consideration of the soil-structure interaction, respectively. It is assumed that the bridge is located 10km from the surface projection of the rupture plane. The result shows that the soil-structure interaction does not only cause larger response amplitude, but also different relative vibration between the adjacent girders. A consideration of a fixed base cannot produce the vibration behaviour of the system *structures with subsoil*. Since poundings are strongly influenced by the out-of-phase vibration of the two girders and the amplitude of the vibrations, in the considered case the necessary distance d_g to avoid girder pounding of the fixed-base bridge is 18cm, while the required distance is 28cm when the soil-structure interaction is considered. In fixed base case the first pounding will occur at 10.7s, and in subsoil case pounding occurs much later at 15.25s.

Fig. 4 shows the response of the bridge girders with soil-structure interaction effect due to the ground motions at the distance d of 0km, if the existing gap between the girders is assumed to be 30cm. Because of the larger vibration amplitude and the out-of-phase vibration the bridge girders with soil-structure interaction will experience a larger number of poundings, while the bridge with the assumed fixed base experience only two time poundings. The result shows that neglecting soil-structure interaction can lead to an underestimation of deck damage due to poundings. Table 2 confirms the necessity of including the soil-structure interaction effect in the analysis of damage of bridge girders in near-source regions.

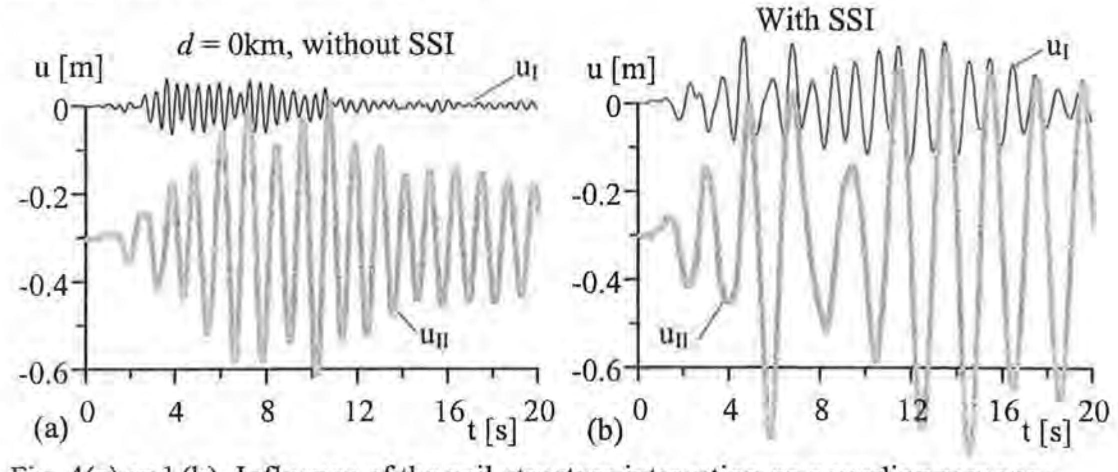

Fig. 4(a) and (b). Influence of the soil-structure interaction on pounding response

Table 2. Effect of soil-structure interaction and the distance d on the required distance d_g to avoid pounding

Distance d [km]		0	5	10
d_g [cm]	without SSI	35	26	18
	with SSI	60	41	28

4. CONCLUSIONS

In the analysis of structural pounding responses with soil-structure interaction effects the calculation is performed in the Laplace and time domain. The nonlinear behaviour due to pounding and separation of the bridge girders is approximated by a sequence of successively changing linear behaviour. It reveals that in near-source regions neglecting the soft soil effect can underestimate the necessary distance between the bridge girders to avoid pounding, and consequently, underestimate the damage potential of bridge decks. For a general conclusion, however, further investigations with non-uniform near-source ground excitations of the adjacent structures are necessary.

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