AUSTRALIA'S FIRST ECCENTRICALLY-BRACED FRAME - SHOULD THERE BE MORE ?

PETER MCBEAN

Peter McBean, BE (Hons) MIE Aust, holds an honours degree in civil engineering from the University of Adelaide and is a member of the Institution of Engineers, Australia. He is currently a Director of Wallbridge & Gilbert consulting engineers, and State Chairman (SA) of the Australian Institute of Steel Construction. Peter has been design director for a number of significant projects including the $570 million Myer Centre, Adelaide, and the $60 million Woolworths Distribution Centre, Pooraka, SA. In the early 1980's he was involved with the design of ASER development which included the Adelaide Convention Centre and Hyatt. His interests include structural analysis and design, seismic design, masts and towers and constructability.

ABSTRACT:

With the aid of a case study, this paper attempts to demonstrate that eccentrically-braced frames present an attractive alternative to conventional lateral bracing systems currently employed in Australian steel framed buildings. Eccentrically-braced frames offer the designer excellent lateral stiffness and control over storey drift, whilst also providing the capacity to absorb large amounts of seismic energy through the formation of stable yielding links at predetermined locations within the frame. The absence of design guidance from local standards presented no significant difficulty to the design process, which applied the well established American design rules to the project.

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Director, Wallbridge & Gilbert, Consulting Engineers

1.0 INTRODUCTION

To the author's knowledge, there are currently no significant buildings in Australia that employ eccentrically-braced frames to provide lateral resistance under seismic loading. The author has recently utilised the structural system on an eight storey open deck carpark currently under construction in Adelaide, South Australia. Eccentrically-braced frames were found to present a cost effective solution for the project following a detailed review of a number of alternative and architecturally acceptable bracing strategies. This paper discusses how the bracing system was chosen for the project, briefly discusses the design basis for eccentrically braced frames, and comments on where the system may be appropriate within the Australian context.

2.0 PROJECT OVERVIEW

The project site is located near the corner of North Terrace and Frome Street, Adelaide. The site is bounded on all sides by existing multi storey buildings which restrict access to a three lane driveway from Frome Street. The client sought to maximise the carpark yield for the site within the (site) planning regulations which included a height limitation of 17.5 metres. In the early planning stages, a number of different structural layouts using both steel and prestressed concrete were reviewed. A steel frame was finally chosen for the superstructure based on cost effectiveness, construction flexibility and speed of erection. Figure 1 illustrates the configuration of a typical level showing the split two way ramping floor, column grid, stair and lift shaft locations and final bracing locations.

The seven suspended levels utilise parking on the 1 in 20 ramps. The columns are set towards the front of the parked cars creating a wide user friendly environment. Composite 530UB beams clear span between the columns which are spaced at 5200 centres. The cladding consists of a mixture of precast and sandwich panels designed to create the appearance of an office building and comply with fire rating provisions of the BCA[1].

3.0 BRACING ALTERNATIVES

Having selected steelwork as the preferred construction material for the primary superstructure, a number of lateral bracing alternatives were reviewed. In seismic design terms, the car park presents itself as a regular Category C structure, and as such required only static lateral analysis in accordance with section 6 of AS 1170.4[2]. Given the relative simplicity of the static analysis method and regular building form, a reasonably thorough comparison of alternative bracing systems was undertaken during the design process.

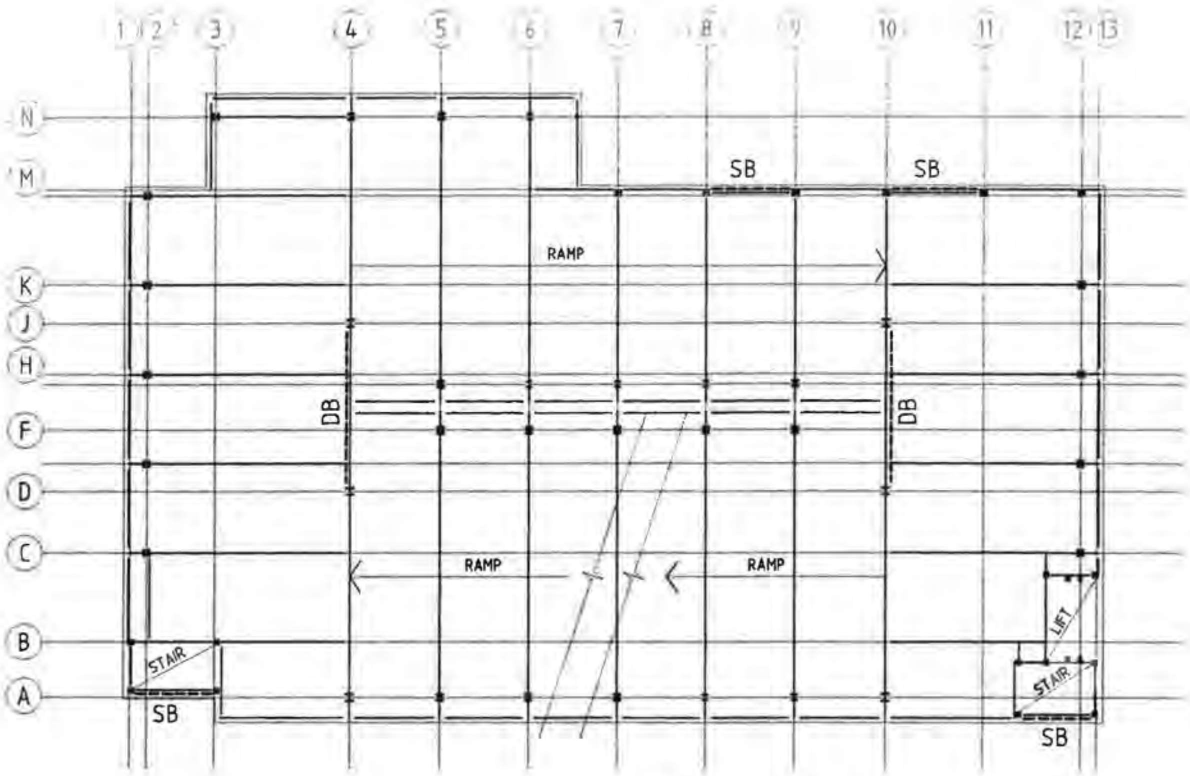

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FIGURE 1
TYPICAL FLOOR PLAN

LEGEND:
SB - SINGLE BAY BRACE
DB - DOUBLE BAY BRACE

3.1 Moment Resisting Frames (MRF)

Preliminary design of the superstructure for gravity loads identified that moment connections would be required between beams and columns in order to obtain the minimum structural floor depth. It seemed logical therefore that lateral forces could be readily accommodated by simply upgrading the moment frame so created. However, a number of problems arose.

Appropriately detailed moment resisting frames, while possessing excellent capacity for energy dissipation, are relatively flexible systems. Strength based solutions for both ordinary MRF (Structural Response Factor $R_f = 4.5$) and intermediate MRF (Structural Response Factor $R_f = 6.5$) resulted in sways that were unacceptably large. In order to restrict sways to within code limits and to account for sway induced P-Δ effects it was found necessary to significantly increase the column size over that required to resist gravity loads, particularly in the lower stories. Special MRF (Structural Response Factor $R_f = 8.0$) were not considered due to the high fabrication cost associated with the additional detailing requirements.

In addition to the sway control issue, limited framing opportunities existed for this particular building in the North-South direction due to site planning issues dictating that the majority of the building frames run East-West.

3.2 Concentrically-braced Frames (CBF)

Potential frame locations compatible with the site planning and architectural intent were identified at stairs, lift shafts, the Western wall, and both ends of the ramps. Various frame combinations were then modelled to obtain a reasonable distribution of stiffness in plan, consistent with the assumptions underlying static seismic analysis.

Concentrically-braced frames provide excellent lateral stiffness and strength under elastic conditions, however, due to their poor inelastic behaviour, AS 1170.4(2) requires that a structural response factor (R_f) of 5.0 be adopted where they are incorporated into a building frame system. This results in a 30% increase in total base shear when compared with an intermediate MRF system. The increased total base shear, coupled with the fact that at some bracing locations architectural constraints required frames to have relatively slender aspect ratios, resulted in an increase to both column and footing sizes at bracing locations. The CBF solution cured the sway problems of the MRF but was materially inefficient for the building.

Consideration was also given to utilising the stair and lift shafts to laterally brace the building. To maximise pedestrian access these elements were located along the Eastern side of the building, thereby creating a large static eccentricity in plan. The resultant torsional amplification generated uneconomic vertical element and footing designs.

3.3 Eccentrically-braced Frames (EBF)

Eccentrically-braced frames contain strategically located deliberate member eccentricities which are intended to yield and absorb large amounts of energy during major earthquakes whilst remaining elastic under wind forces and minor earthquakes. Braces are connected to the beams in such a way that a short length of the beam experiences high transverse loads and yields. This yielding zone is called the link. EBF's combine the stiffness of CBF's with the capacity for inelastic energy dissipation associated with MRF's. In recognition of their excellent seismic characteristics, AS 1170.4(2) assigns EBF's with a structural response factor (R_f) of 7.0. This represents a 30% reduction in total base shear when compared to the CBF solution and suggested the possibility of material savings in both bracing elements and footings. Reference to American EBF detailing practices(3) indicated that only minor fabrication cost penalties would be incurred. Based on this early encouragement it was decided to further pursue the EBF solution for the project. Figures 2 and 3 show in elevation the general bracing arrangements used at single and double bay brace locations. Plan locations of the bracing are shown in Figure 1.

FIGURE 2
SINGLE BAY
BRACE ELEVATION

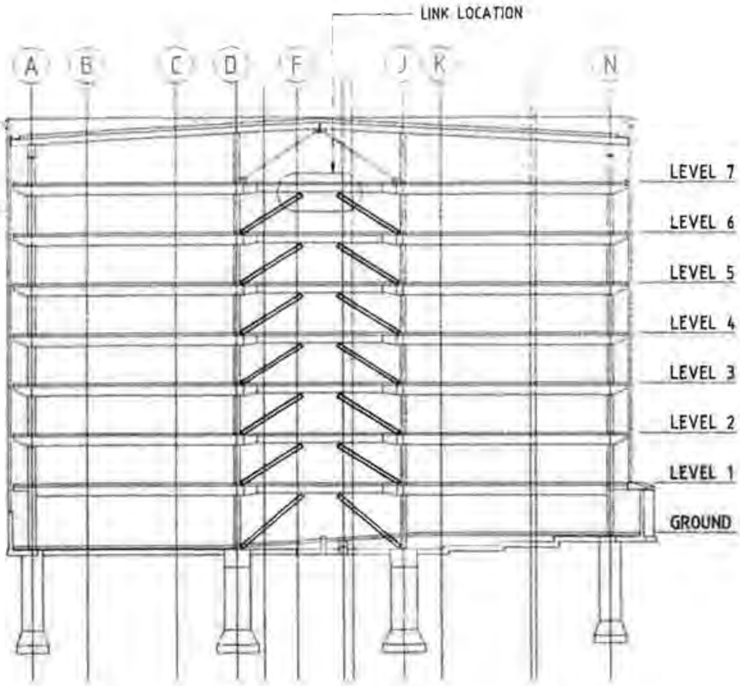

FIGURE 3
DOUBLE BAY
BRACE ELEVATION

4.0 EBF DESIGN BASIS

Unfortunately for the Australian engineer, neither AS 1170.4[2] or AS 4100[4] offer any advice as to how to go about designing an EBF. There is however a considerable pool of published work available, much of which is summarised in the American Institute of Steel Construction "Manual of Steel Construction, Load and Resistance Factor Design - Seismic Provisions for Structural Steel Buildings". The document represents current American practice for the design of EBF's and was used on this project as a basis for determining appropriate frame geometry and detailing.

Whilst space limitations here prevent a full discussion of EBF design methodology, some important issues for consideration are noted:

4.1 The **link design shear strength** is a function of the link length. Short links yield in shear, whilst links with a length greater than twice their plastic moment divided by their yield shear force will form plastic hinges at their ends. Very short links result in structures approaching the lateral stiffness of CBF, but may place an excessive ductility demand on the links. Ductility limits are set in terms of calculated link rotations. Long links on the other hand result in more flexible structures. The use of steep braces and short links was found to provide a more economical solution than shallow brace angles which required heavier braces in order to prevent their buckling. Link geometry adopted for the carpark is detailed in Figures 4 and 5, for single and double bay bracing respectively.

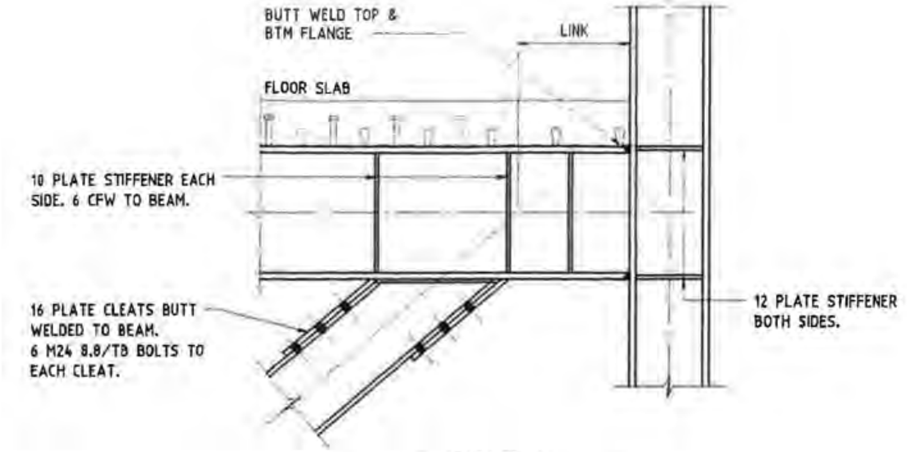

FIGURE 4
SINGLE BAY HINGE DETAIL

4.2 **Link yielding is achieved** by ensuring that the diagonal brace and beam segment outside the link are proportioned such that they can resist the maximum forces generated by the link yielding, having regard for sources of link overstrength such as strain hardening and the actual steel yield strength. To account for link overstrength, the AISC Seismic Provisions require the design strength of adjoining brace and beam elements to exceed 1.25 times the nominal link shear strength. This requirement offsets in part the potential material savings arising from the reduction in total base shear earlier identified.

4.3 Cyclic shear yielding of properly stabilised webs is known to exhibit excellent hysteretic behaviour because yielding occurs simultaneously over much of the web depth, followed by web buckling and the establishment of a stable diagonal tension field. **Detailing requirements to ensure link stability** during cyclic yielding include full depth stiffeners at the link ends and intermediate web stiffeners, the spacing of which is dependant upon the imposed link rotation. Other requirements include the use of full penetration butt welds between link flanges and columns. None of these detailing requirements were seen in practice to add significant cost to the overall frame structure when compared to the CBF design option.

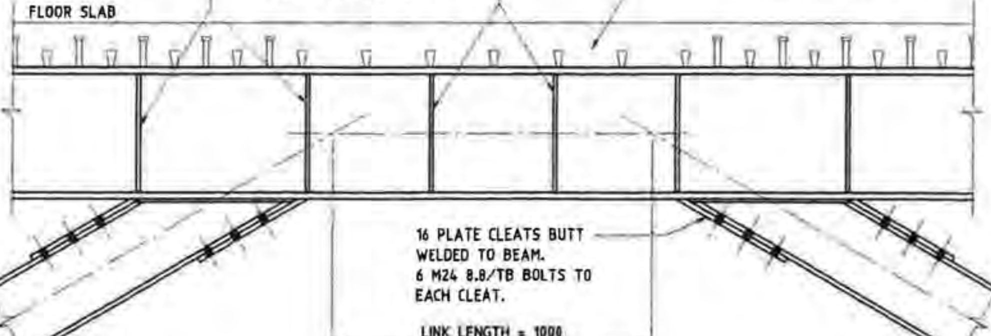

FIGURE 5
DOUBLE BAY HINGE DETAIL

4.4 **Yielding should be confined to the links**. The formation of unwanted hinges, particularly in columns at floor beam level may create a soft storey. Adequate modelling should be undertaken to ensure that potentially harmful hinges do not form elsewhere within the structure at anticipated storey drifts.

5.0 THE FUTURE OF EBF's IN AUSTRALIA

It should be remembered that with Australia's relatively low seismic risk that wind forces often govern lateral systems and that design for seismic resistance often amounts to a quick check for compliance with detailing requirements to ensure adequate ductility. In this particular instance, the building site was well sheltered from high winds by many surrounding structures. Consequently, seismic forces governed the lateral bracing system design. Taller buildings are more likely to be wind critical.

For the eight storey carpark project outlined, the author was satisfied that EBF's provided an economically viable method of resisting seismic induced loading. Lateral stiffnesses similar to that of CBF's were achieved in conjunction with superior levels of ductility. A small nett material saving was achieved in the bracing system for the cost of providing a few strategically placed web stiffeners. By ensuring that links were prefabricated in the workshop, no field welding was required, and critical welds could be inspected and certified prior to delivery to site. Favourable reports have been received during construction from both the steelwork fabricator and erector.

From a consulting engineer's perspective, the design process is made slightly more complex and difficult. This is compounded by the lack of local guidance and the need to access American reference material. However, for regular structures subject to static analysis the additional demand on the structural engineer is warranted.

The author believes that steel framed buildings under 10 storeys in height, located at sites with moderate seismic risk (Adelaide acceleration coefficient equals 0.10), and within the non-cyclonic wind Region A[5], represent likely candidates for future EBF structures.

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EARTHQUAKE MICROZONATION AND THE DEVELOPMENT OF THE AUSTRALIAN EARTHQUAKE LOADING STANDARD

T.D. JONES, M.J. NEVILLE, G. SCOTT AND C. SINADINOVSKI

Trevor Jones B.Sc. M.Sc. Dip.Tch, has worked as an earthquake seismologist for nearly 15 years. He has published papers on earthquake hazard in Australia and a number of Australia's Southwest Pacific neighbours.

More recently Trevor has become a founding member of the new AGSO "Cities" Project. This national project focuses on reducing community vulnerability to many natural hazards including earthquakes.

Michael Neville B.Sc. M.App,Sc. F.G.S., M.G.S.A., has practised for 22 years in engineering geology at the NSW Department of Mineral Resources and then the Department of Public Works and Services. He has had extensive experience in the investigation and construction of dams and tunnels, and is now also responsible for earthquake monitoring in NSW.

Greg Scott has wide experience in cartography in both New Zealand and Australia. For the past six years he has specialised in GIS at the Australian Geological Survey Organisation and is regarded as an expert in this field.

Cvetan Sinadinovski has a doctorate in seismic tomography and geophysical imaging from Flinders University of South Australia. He has worked as visiting fellow in USA and Europe, and as a software specialist in Sydney and Adelaide. Currently he is employed as a professional officer in the Australian Geological Survey Organisation in Canberra. Member of ASEG, AIG and AEES.

ABSTRACT:

Seismic microzonation is becoming an increasingly popular tool in defining urban earthquake hazard. In Australia, it has been employed largely in relation to emergency management (e.g. CERA, 1995; Jones *et al.* 1996) and insurance risk assessments (e.g. Blong, 1995). So far, few studies have been adopted for use with building regulations (e.g. Michael-Leiba, 1995).

We present a case study, the Sydney Pilot Earthquake Microzonation Project (Jones *et al*), 1996) as a catalyst for discussion on the usefulness of the microzoning technique in improving assessment of site-dependent earthquake hazard for the Australian Earthquake Loading Code.

Paper No. 18

EARTHQUAKE MICROZONATION AND THE DEVELOPMENT OF THE AUSTRALIAN EARTHQUAKE LOADING STANDARD

by

T D Jones[1], M J Neville[2], G Scott[1] and C Sinadinovski[1]

[1]Australian Geological Survey Organisation, GPO Box 378, Canberra, ACT, 2601, Australia
[2]New South Wales Department of Public Works and Services, McKell Building, 2-24 Rawson Place, Sydney, NSW 2000, Australia

INTRODUCTION

Seismic microzonation is becoming an increasingly popular tool in defining urban earthquake hazard. Recently in Australia, it has been employed largely in relation to emergency management (e.g., [(1), (2)]) and insurance risk assessments (e.g., [(3)]). So far, few studies have been adopted for use with building regulations (e.g., [(4)]).

We present a case study, the Sydney Pilot Earthquake Microzonation Project[(2)], as a catalyst for discussion on the usefulness of the microzoning technique in improving assessment of site-dependent earthquake hazard for the Australian Earthquake Loading Standard.

CASE STUDY
THE SYDNEY PILOT EARTHQUAKE MICROZONATION PROJECT

Many instances around the world have demonstrated that unconsolidated sediments and uncompacted landfill can intensify the effects of earthquakes on urban areas.

In the 1989 Newcastle earthquake 13 people died and there were damage claims of more than $1 billion to insured structures. There was a strong coincidence between the area of maximum damage and the geographical extent of Quaternary sediments and landfill.

This earthquake motivated many individuals and agencies to try to assess and mitigate the vulnerability of other Australian cities to a moderate or strong, potentially disastrous, earthquake. Our case study describes a pilot project undertaken in Sydney [(2)].

In June 1995 the Engineering Functional Group of the New South Wales State Emergency Management Committee implemented the Sydney Pilot Earthquake Microzonation Project.

The New South Wales Department of Public Works and Services (DPWS) coordinated the pilot project which was carried out by the Australian Geological Survey Organisation (AGSO) and DPWS. The pilot project was a cooperative venture among the State organisations represented on the Engineering Functional Group and the NSW Land Information Centre (LIC).

The Sydney Pilot Earthquake Microzonation Project was undertaken to:

- demonstrate a technique which zones the likely ground shaking response to potential earthquake activity; and
- determine the utility of the technique as an aid to emergency response planning in the Sydney region.

A Pilot Area was chosen in the Homebush Bay-Silverwater-Concord West area. This area was chosen partly because a variety of ground conditions was present, and partly because a large amount of data was available; geological, cultural and spatial. The Pilot Area is about 5 km by 4 km.

We see the preparation of detailed geological maps as a key component of earthquake microzonation studies. In the Pilot Project, data from about 1,300 boreholes and test pits were used to compile a 3-dimensional geological map (Fig. 1; map by M Neville). The map is three dimensional because the thicknesses of the various sediments and fills are included as is geotechnical information on the various geological units. Local geology comprises Triassic shales and sandstones overlain by combinations of stiff residual clays, unconsolidated estuarine sediments and uncompacted landfill. Depths to rock of up to 23 m were present.

A Site Factor map was then prepared using the lithological information and the rock and soil classifications in AS 1170.4-1993 [(5)]. Under this standard, the Site Factor S can have values from 0.67 to 2.0, with larger values representing higher earthquake hazard. The value S in part determines how buildings must be designed and constructed for those site conditions.

This zonation map by itself is useful but we improved on it by using a technique of microtremor analysis developed by Nakamura [(6)].

Three new hazard zonation maps were interpreted from the geological and microtremor information. The two criteria used to distinguish the zones in the three maps were the Site Factor and the vibrational period range over which resonance or amplification may be observed. Earthquake motion is equivalent to that described by AS 1170.4-1993 - i.e., a 10% probability of exceedence in 50 years.

The classification may also be expressed in terms of the natural period of vibration of buildings (e.g., [(5)]; p. 39) and so the maps correspond to zonations for low-, medium-, and high-rise structures.

We incorporated cultural, infrastructure, cadastral and many other data into a GIS. The Land Information Centre and agencies represented on the Engineering Functional Group contributed data for this purpose.

Small demonstrator databases containing attribute data were constructed. Identifiers in these databases allow links to be developed to the large databases of the custodian agencies.

The Pilot Project demonstrated that earthquake microzonation, used in a GIS linked to building stock and infrastructure databases, is a valuable technique for emergency planners to analyse and reduce community vulnerability to earthquake hazard.

The techniques used in the Pilot Project could be applied in the County of Cumberland to produce an earthquake microzonation map for Sydney. High priority would be placed on defining the hazard and, ultimately, the risk in the areas of sediment and landfill. Fortunately the engineering foundation in many parts of Sydney is on competent rock, either sandstone or shale.

DISCUSSION

Microtremor analysis, particularly using the method described by Nakamura [(6)], is an economical and effective technique for microzonation. Microtremor studies have been undertaken in a number of Australian cities: Sydney [(2)], Perth [(7)], Newcastle [(8)], Launceston [(4)], and Rockhampton [(9)].

AGSO is working with its partners to reduce the vulnerability of Australian urban communities to earthquake hazard. A reduction in vulnerability can be achieved partly by improving the assessment of site-dependent earthquake hazard for application in building regulations. Key steps in achieving this objective include:

- providing urban hazard assessments through zonation maps. These maps should be compatible with AS 1170.4-1993 and related standards (as we have attempted to do with the zonations in the Sydney Pilot Project), and any future developments of them which are called up by the Australian Building Code;
- adopting and enforcing the standards, and the zonation maps, at local government level.

ACKNOWLEDGEMENTS

We thank the New South Wales Land Information Centre for permission to use their data on the map in this paper and David Denham for a constructive review of the paper.

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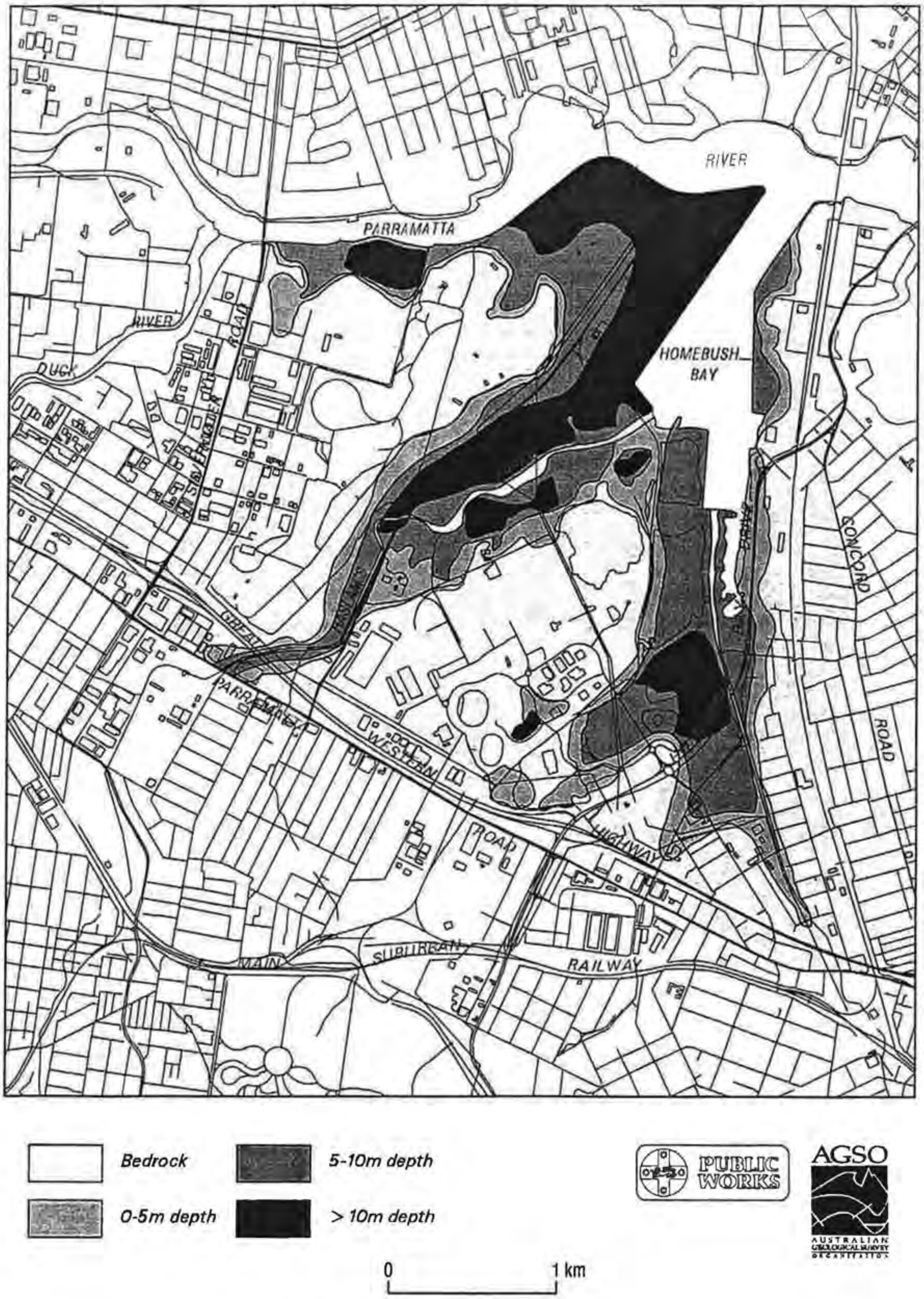

Figure 1. Geological map of the Sydney Pilot area. Cultural and drainage data ©Land Information Centre, Panorama Avenue, Bathurst, 2795.

ATTENUATION OF EARTHQUAKE GROUND MOTION IN AUSTRALIA

GARY GIBSON

Paper not available

ABSTRACT:

Recent improvements in seismic instrumentation in Southeast Australia have confirmed that attenuation of earthquake wave amplitudes with distance is a little above world average, and similar to that experienced in western USA.

The shallow depths and reverse or thrust faulting of most Australian earthquakes give higher amplitudes, more energy at higher frequencies and shorter durations than world average, affecting response spectra. There have been few moderate magnitude Australian earthquakes since Newcastle, 1989, and no large events. Most Australian data are from small nearby earthquakes, so spectral attenuation functions derived from this data alone are still highly uncertain.

This suggests that use of Californian response spectrum attenuation will be appropriate for design purposes in Southeast Australia for some time yet. For time series analysis it may be appropriate to use selected records from the 17 January 1994 Northridge earthquake, a shallow reverse fault earthquake of magnitude MW 6.7.

Variations in attenuation affect magnitude determinations, which are made at distances of up to 600 km or more, much more than they affect the strong motion near to a large earthquake. There are greater uncertainties in current earthquake hazard estimates from magnitude uncertainties than from strong motion attenuation functions.

JOHN SANDLAND
JOHN SANDLAND & ASSOCIATES PTY LTD

Paper not available

The Author's firm currently designs for more than 2,500 houses in Adelaide per year.

ABSTRACT:

The paper covers the current common practice among engineers designing for earthquake loads for domestic structures in Adelaide and how the relevant codes apply.

Paper No. 20

THE SEISMIC BEHAVIOUR OF REINFORCED SEGMENTAL RETAINING WALLS

CLAUDIA TAPIA

ABSTRACT:

Reinforced Segmental Retaining Walls (RSRW) are widely believed to perform well during seismic activity. This paper is a review of the analysis that has been performed to date on the response of soil RSRW under seismic loading. This analysis includes the numerical approaches currently adopted as well as a summary of the observed performance of walls which have been subjected to horizontal groumd accelerations.

Paper No. 21

The Seismic Behaviour of Reinforced Segmental Retaining Walls

C. A. Tapia
BE Grad IE Aust
Marketing Engineer, Boral Masonry NSW, Australia

Abstract: Reinforced Segmental Retaining Walls (RSRW) are widely believed to perform well during seismic activity. This paper is a review of the analysis that has been performed to date on the response of soil RSRW under seismic loading. This analysis includes the numerical approaches currently adopted as well as a summary of the observed performance of walls which have been subjected to horizontal ground accelerations.

1. INTRODUCTION

Soil RSRW have shown considerable resistance to seismic activity over the last decade. Many of the walls observed over this period were not designed for seismic loading but have demonstrated an ability to withstand large ground motions due to their inherent flexible nature.

The overall satisfactory performance of RSRW under seismic conditions has resulted in minimal advancement in the currently adopted seismic design approaches since those proposed by Mononobe and Okabe in the 1920's. Whilst it is generally accepted that this pseudo-static approach is somewhat conservative, given the magnitude of the horizontal ground accelerations during seismic activity often exceeds design assumptions, few have pursued the development of more pertinent methods.

Bathurst and Cai (1995) have pioneered the study into alternative analysis methods and are currently assessing the virtues and shortcomings of each. The field studies performed by the likes of Collin et al (1992) and Tatsuoka et al (1995) have been instrumental in demonstrating the ductility of RSRW.

As the segmental retaining wall market becomes more competitive with the advent of an increasing number of products, it is imperative to continue the evolution of appropriate seismic design methods tailored to these wall systems.

Reinforced segmental retaining wall systems have been gaining worldwide popularity for aesthetics, cost and speed of installation. Their ability to withstand the effects of dynamic loading is proving to be another advantage over traditional gravity and cantilever retaining walls.

2. NUMERICAL ANALYSIS APPROACHES

The numerical methods of analysing RSRW under seismic conditions are summarised below.

2.1 MONONOBE - OKABE PSEUDO STATIC METHOD

The simplified design approach proposed by Mononabe-Okabe represents dynamic earth forces as pseudo-static forces in both the external and internal analysis. These pseudo-static forces are added to the calculated forces under static conditions and then compared against reduced safety factors during the seismic event. The seismic analysis includes increased earth pressure and the inertial force of the masses involved due to the seismic event.

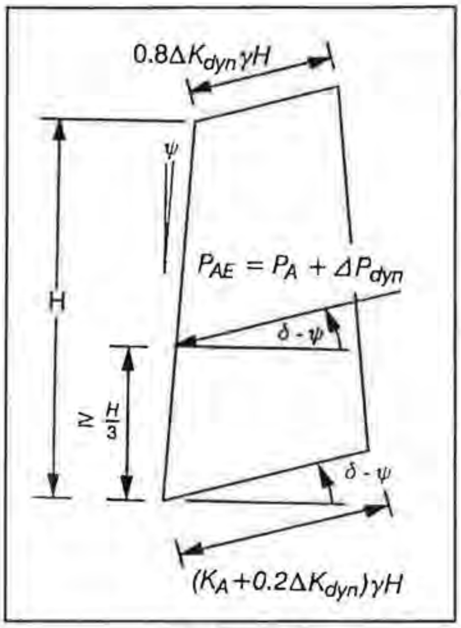

Figure 1.

Dynamic earth pressure distribution under seismic loading

K_A	=	static earth pressure coefficient
ΔK_{dyn}	=	dynamic earth pressure coefficient increment
δ	=	angle of friction at interface of soil and mass
γ	=	wall batter from vertical

The total dynamic earth distribution as proposed by Bathurst and Cai (1995) is illustrated in Figure 1. This illustration clearly shows that the triangular distribution due to soil self-weight under static load conditions becomes a trapezoidal distribution under static and dynamic load conditions.

The effect of designing with the pseudo-static rigid body approach is that more geogrid reinforcement layers may be required in the uppermost sections of the wall than normally required for static loaded structures.

Other practical consequences are greater connection loads at the interface of geogrid reinforcement to wall facing and possibly longer geogrid lengths towards the top of the wall as the internal failure plane becomes shallower with increasing horizontal ground acceleration, as illustrated in Figure 2.

The Mononobe-Okabe method is a limit-equilibrium method which does not allow for the analysis of wall deformations. However, it remains the most accepted form of seismic loading analysis.

2.2 DISPLACEMENT METHOD

Displacement methods such as Newmark sliding block theory can adequately address the serviceability criteria related to wall deformations. There are typically three areas of concern when evaluating the connection and interface shear properties of segmental retaining walls. These are:

a) external sliding failure along the base of the reinforced soil structure
b) internal sliding failure along a geogrid reinforcement through the wall facing
c) shear failure between segmental wall facing units.

Presently, an empirical Newmark sliding block theory can be adopted which relates predicted displacements to seismic parameters such as critical acceleration ratio and peak ground acceleration (Cai and Bathurst, 1996). This approach is particularly recommended when a range of different segmental facing units are being considered which have different interface shear capacities.

Some examples of different shear connectors are shown in Figure 3.

K_N = static earth pressure coefficient

L_{sta} = static soil reinforcement length

L_{dyn} = dynamic soil reinforcement length

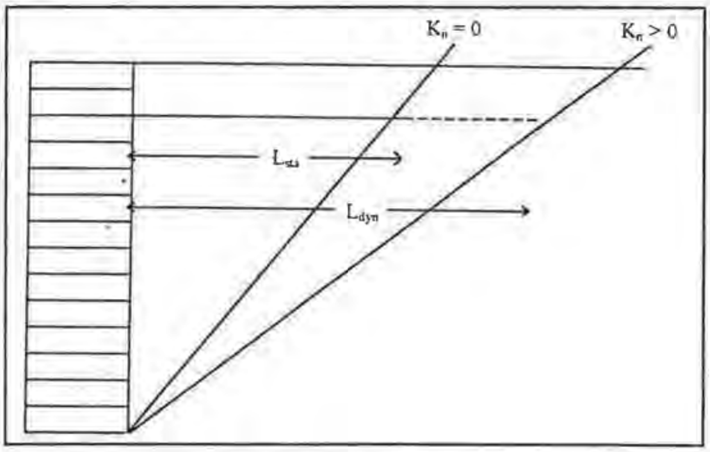

Figure 2.

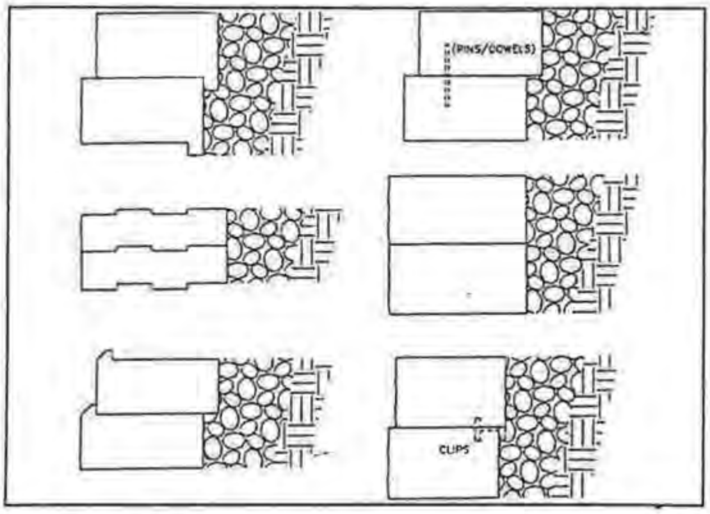

Figure 3 - Shear Connectors

Shaking table tests have been performed (Bathurst and Cai, 1996) which confirm the importance of interface shear properties of the facing column stability in simulated RSRW.

2.3 FINITE ELEMENT METHOD

Dynamic finite element modelling of geosynthetic RSRW has been carried out by Cai and Bathurst (1995). Geosynthetics are extensible polymeric reinforcing materials which can provide an additional factor of safety under earthquake loading due to the long term creep factors incorporated in any design, and due to the reduction in overall stiffness of the structure resulting from the extensibility of the reinforcement.

The development of appropriate complex finite element models has involved cyclic load testing of geogrid reinforcement materials (Bathurst and Cai, 1994).

The inherent advantage of finite element analysis is that non linear and hysteretic properties of soils and reinforcement materials can be carefully modelled. The main disadvantage is the difficulty in accessing such properties.

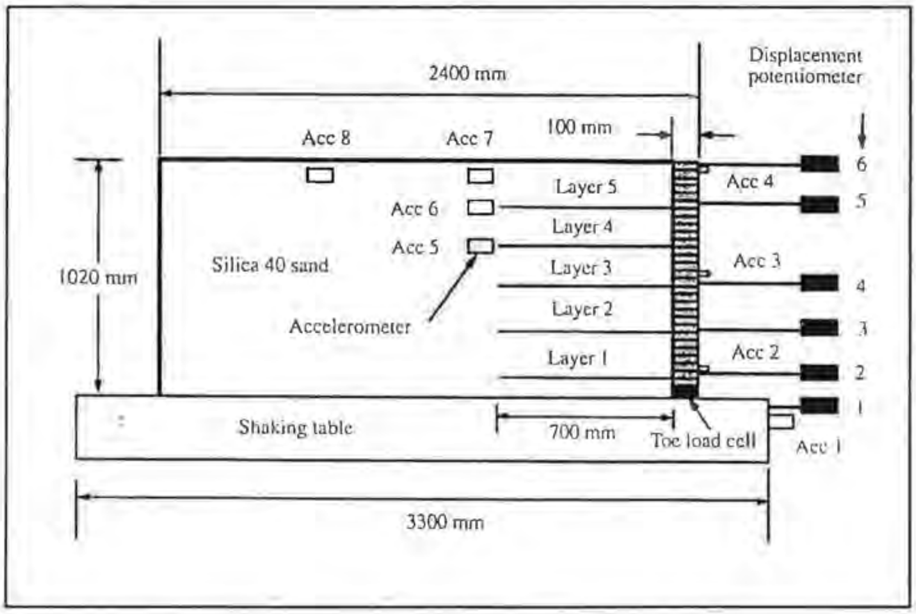

Figure 4 - Example of a shaking table test set up

3. OBSERVATIONAL APPROACHES

Segmental retaining wall systems were first introduced in North America in 1986. Since then they have been readily accepted worldwide, and are often preferred over conventional gravity type and cantilevered retaining walls.

During this period there have been two major earthquakes which have had minimal impact on existing RSRW. These are reviewed separately.

3.1 FIELD OBSERVATIONS - NORTHRIDGE, CALIFORNIA

On January 17, 1994 at approximately 4.30 AM, an earthquake with its epicentre near Northridge, California, shook the greater Los Angeles area with a magnitude of 6.7 on the Richter scale. It has been established that the duration of strong shaking lasted between 10-15 seconds.

Sandri (1994) examined eleven reinforced segmental retaining wall structures with a wall height exceeding 15 feet and within a 70 mile radius of the epicentre. His results are summarised in Table 1.

Evaluation of the response of the walls under seismic loading included visual inspection of the structure for alignment, relative facia movements, facia bulging, facia cracking, disconnection of geogrid to wall face and tension cracking of soil or pavement near the wall area.

All of the structures examined had been designed using different methodologies and only a few had been designed for seismic loading using pseudo-static methods.

It is interesting to note that the wall closest to the epicentre was designed for a horizontal acceleration of 0.3g using pseudo-static analysis. Subsequent Joyner-Boore attenuation relationships for the Northridge earthquake indicated peak horizontal accelerations of about 0.6g for this site.

Observation of the Valencia Water Treatment Plant walls established relatively minor signs of movement despite being subjected to horizontal ground accelerations of at least 2 times those for which they were designed.

Sandri (1994) concluded that the relatively minor movements exhibited by the RSRW he observed confirmed the general belief that these structures perform well under seismic conditions. In particular, the wall facia blocks maintained their serviceability criteria, regardless of the level of design conservatism applied.

3.2 FIELD OBSERVATIONS - KOBE, JAPAN

Exactly a year after the Northridge Earthquake in California, USA, another earthquake with a magnitude of 7.2 on the Richter scale devastated Kobe City in Japan. At approximately 5.46 AM on January 17, 1995 the Great Hanshin Earthquake claimed close to 5,500 lives and caused widespread damage.

Several papers have been produced on the observed effects of the earthquake of reinforced soil walls and they all indicate that these types of walls performed considerably well. Minimal outward displacements were observed on walls which were in areas where the damage to conventional reinforced concrete structures was severe. (Tatsuoka et al, 1995)

Name & Location	Structure Type & Description	Distance from epicentre (kms)	Wall facing/ slope surfacing	Soil reinforcement	Year	Performance based on visual observation
Congregate Care Facility, Dana Point	31.5m high 1.5:1 slope with a 7m wall midway up slope	112	Slope vegetated wall (Keystone)	Slope and wall (Miragrid)	1992	Excellent
Mission Viejo Hospital, Mission Viejo	9m max height wall, constructed at crest of slope	104	Keystone	Miragrid	1988	Excellent
Cameo Highlands, Newport Beach	7.9m max height wall	88	Keystone (modified)	Tensar	1993	Excellent
Pelican Hills, Irvine	15m max height wall supported at crest of 7.9m un-reinforced 2:1 slope	93	Loffel	Miragrid	1993	Excellent
Rodeo Ridge, City of Walnut	3, 4m max height walls tiered into 15.8m 1.5:1 un-reinforced soil slope	64	Keystone	Miragrid	1993	Excellent
Tennis court pad Diamond Bar	7.9m max height walls	72	Diamond	Miragrid	1993	Excellent
Driveway support Diamond Bar	7.9m max height walls supporting driveway	72	Keystone	Miragrid	1992	Excellent
Festival Centre Anaheim Hills	6m max height walls	72	Loffel	Miragrid	1991	Excellent
Residence Diamond Bar	6.7m high tiered wall at crest of 15M slope	72	Keystone	Miragrid	1991	Excellent
Water Treatment Plant, Valencia	8.3m high walls along Santa Clara river	22	Keystone	Miragrid	1993	Excellent
Gould Tank, Pasdadena	7.5m high wall built at crest of slop	35	Keystone	Miragrid	1991	Excellent

Table 1

4 CONCLUSIONS

The obvious advantages of RSRWs are cost efficiency, ease of construction and aesthetic appeal. Clearly, their ability to tolerate dynamic loading during seismic activity is an advantage designers are not yet exploiting.

Numerical analysis of seismically loaded RSRWs and observations of walls which have been subjected to actual earthquake conditions clearly indicate that the flexible nature of the walls lends them to applications where more conventional walling systems are often inadequate.

Further development of more realistic design approaches needs to be pursued in light of the conservation exhibited by current pseudo-static methods.

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MICROTREMOR SURVEY AND SEISMIC MICROZONATION OF LAUNCESTON, TASMANIA

MARION MICHAEL-LEIBA AND VAGN JENSEN

Marion Michael-Leiba has a BSc degree from University of Tasmania and MS and PhD degrees from University of Hawaii. Her interests include earthquake hazard assessment, historic Tasmanian earthquakes, earthquake activity in the Canberra region and lecturing on Australian earthquake hazard to the emergency services, schools and the general public.

Vagn Jensen has since 1968 worked at the University of Tasmania in the design and maintenance of its 11 station telemetered seismic network. Earlier this year he spent five weeks in the United States and Europe studying "Understanding earthquakes and mitigating their effects" with the United States Geological Survey and visiting observatories involved in seismological research. He has represented Tasmania at a number of national seismological seminars and conferences and is a member of the Australian Earthquake Engineering Society.

ABSTRACT:

During the period 1884-1995, Launceston has been damaged by five earthquakes with epicentres up to 240 km away in the Tasman Sea. As this damage was thought to be due to amplified site response by underlying sediments, the Launceston City Council requested AGSO to prepare a seismic zoning map of Launceston. We carried out a microtremor survey, and ratios of the spectra of the horizontal and vertical components of microtremors were compuited to determine the fundamental site period.

Our microtremor measurements suggest amplified responses of sediments at 46 out of 53 sites. The three dolerite bedrock sites showed no site resonance. Based on the microtremor measurements and soils, drillhole and gravity data, we have prepared zoning maps of Launceston for use by council personnel and practising engineers. The zones have been entered on a GIS together with geological, street and lifeline information.

INTRODUCTION

During the period 1884-1995, the city of Launceston in northern Tasmania has been damaged by five earthquakes with epicentres up to 200 km away in the west Tasman Sea near Flinders Island. As this damage was thought to be due to amplified site response caused by local superficial sediments (Michael-Leiba and Gaull, 1989; Michael-Leiba and Jensen, 1993), the Launceston City Council requested AGSO to prepare a zoning map of Launceston with zones related to the requirements of Australian Standard AS 1170.4-1993 (the SAA Loading Code, Part 4: Earthquake loads).

To accomplish this, we carried out a microtremor survey of Launceston (Michael-Leiba and Jensen, 1995, 1996). Recordings were made at 53 sites on sediment and three on dolerite, and analysed using the Nakamura (1989) technique. The Nakamura method involves computing the spectral ratios of each of the horizontal components (N-S and E-W) of ground motion relative to the vertical component, This technique was found by Lermo & Chavez-Garcia (1994) and Field *et al* (1995) to give the best results in estimating the dominant natural period of a site from microtremors, when compared with the results from earthquakes.

As the geology of Launceston was not well understood, a detailed gravity survey was carried out by Tasmanian Industry, Safety and Mines. It revealed a complex geology involving at least two deep NNW-SSE trending valleys filled with variably consolidated sediments (Leaman, 1994).

RESULTS

Ratios of the spectra of the horizontal and vertical components of microtremors suggest amplified responses of Tertiary and Quaternary sediments at 46 of the 53 sediment sites at periods ranging from 0.1 to more than 1 second. As expected, the three dolerite (bedrock) sites showed no site resonance. It is assumed for the purposes of the microzonation that a single storey building has a period of 0.1 seconds, a two storey 0.2 seconds, a three storey 0.3 seconds, and so on. Sediments in the deep valleys showed amplification at periods from around 0.7 to greater than 1 second (Figure 1), which would be expected to most affect medium and high rise buildings. Ceilings of two churches in this zone have been damaged by earthquakes. Natural periods of 0.1-0.5 seconds were measured on Quaternary and Tertiary sediments overlying shallow dolerite basement in the eastern part of the Launceston Central Business District (Figure 2). The buildings which would be most affected by site resonance in this area would be low or medium rise buildings. There have been several reported cases of earthquake damage to these types of structures in this area.

At five out of six sites south of the Tamar River, where microtremor measurements were made near to Leaman's (1994) modelled cross sections and sediment thicknesses of 30-230 m were estimated, measured natural periods overlapped the theoretical range of periods calculated assuming a uniform elastic layer on a rigid base. At the sixth site, the lowest measured period was 13% longer than the upper limit calculated. This could be because the sediment thickness is greater than and/or the S wave velocity lower than expected, or the geology deviated too greatly from the simple model. Also, on the 5-10 metre thick sediments of the river flood plain in the old railway yard, anomalously long variable natural periods of 0.3 or 0.4 to more than 1 second were measured, relative to the 0.1-0.2 second periods calculated. This may be due to the non-uniform nature of the sediments, which in some parts are mixed with wood and/or water saturated.

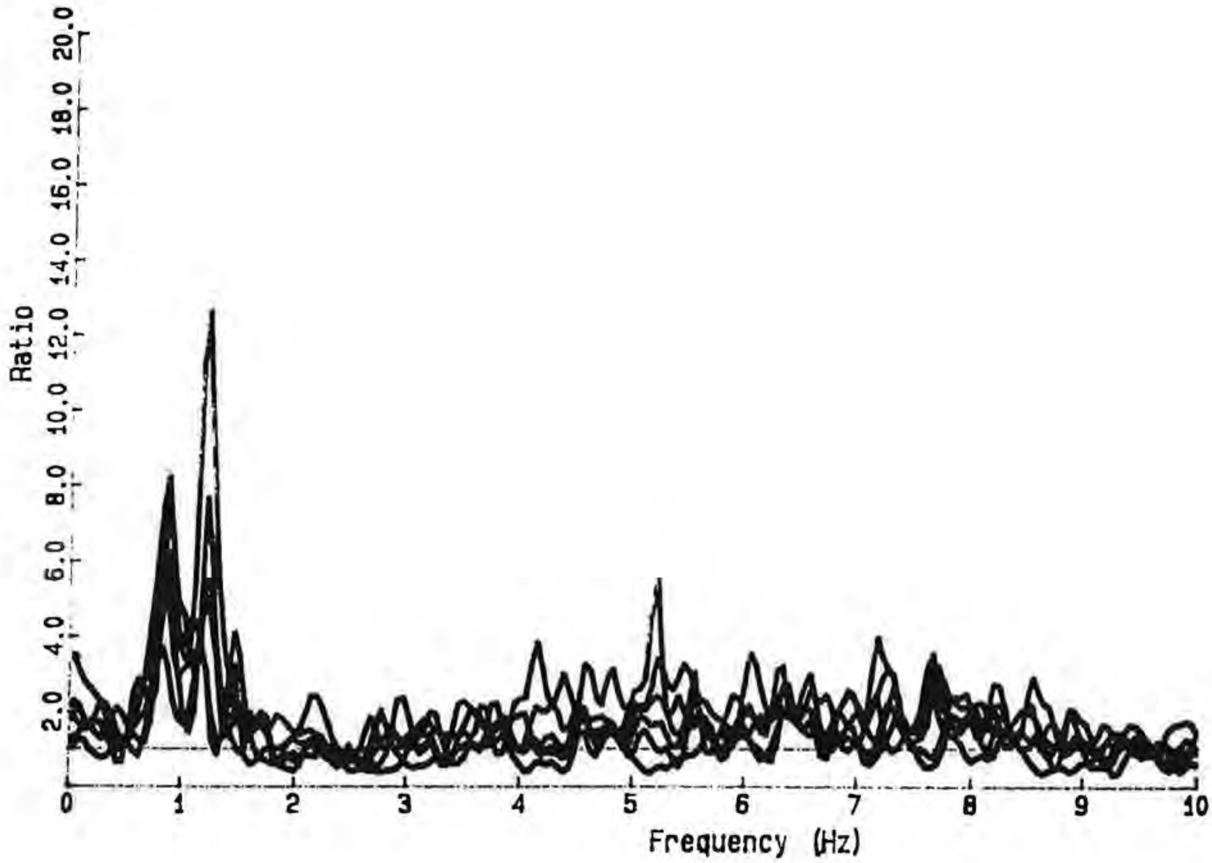

Figure 1. Spectral ratios of the horizontal to the vertical component.
Thick sediments, Tamar axis valley, Brickfields Reserve.

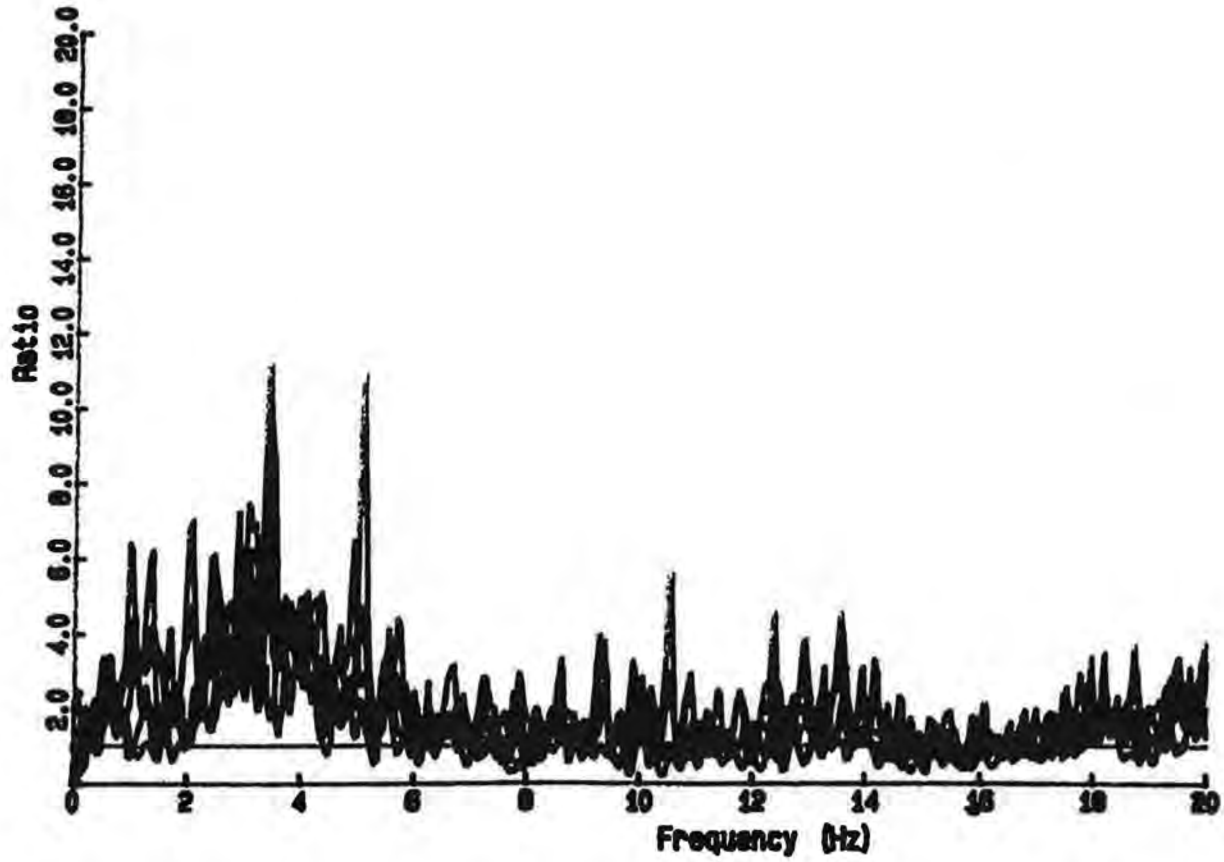

Figure 2. Spectral ratios of the horizontal to the vertical component.
Shallow sediments over dolerite, St Andrews Church, St Johns Street.

ZONING MAPS

Based on the microtremor measurements, the soils map (Forsyth, 1995), the gravity results (Leaman, 1994), and unpublished drill hole information, we have prepared 1:10 000 zoning maps of the northern and southern parts of Launceston, showing areas where resonance may occur for low, medium or high rise buildings in various parts of the city (Figure 3). The zones are consistent with the microtremor measurements and with most reports of damage in earthquakes. We have also drawn a 1:25 000 map of the site factor defined in AS 1170.4-1993 (Standards Australia, 1993). These maps have been incorporated, along with lifeline information, into a GIS for planning purposes.

Our maps are generalised and do not take into account very small scale variations in geology, or topographic or seismic wave focussing effects. They also do not take into account landslip, settlement or liquefaction potential which have already been mapped by Ingles (1991) or ground motion strong enough to cause the soils to behave inelastically. However, they should provide a reasonable indication of site response in another earthquake off northeastern Tasmania, the zone from which Launceston has been damaged by earthquakes in 1884, 1885, 1892, 1929 and 1946. We recommend that specific site studies be carried out prior to the erection of a new structure near a zone boundary or on a site where no specific information is available.

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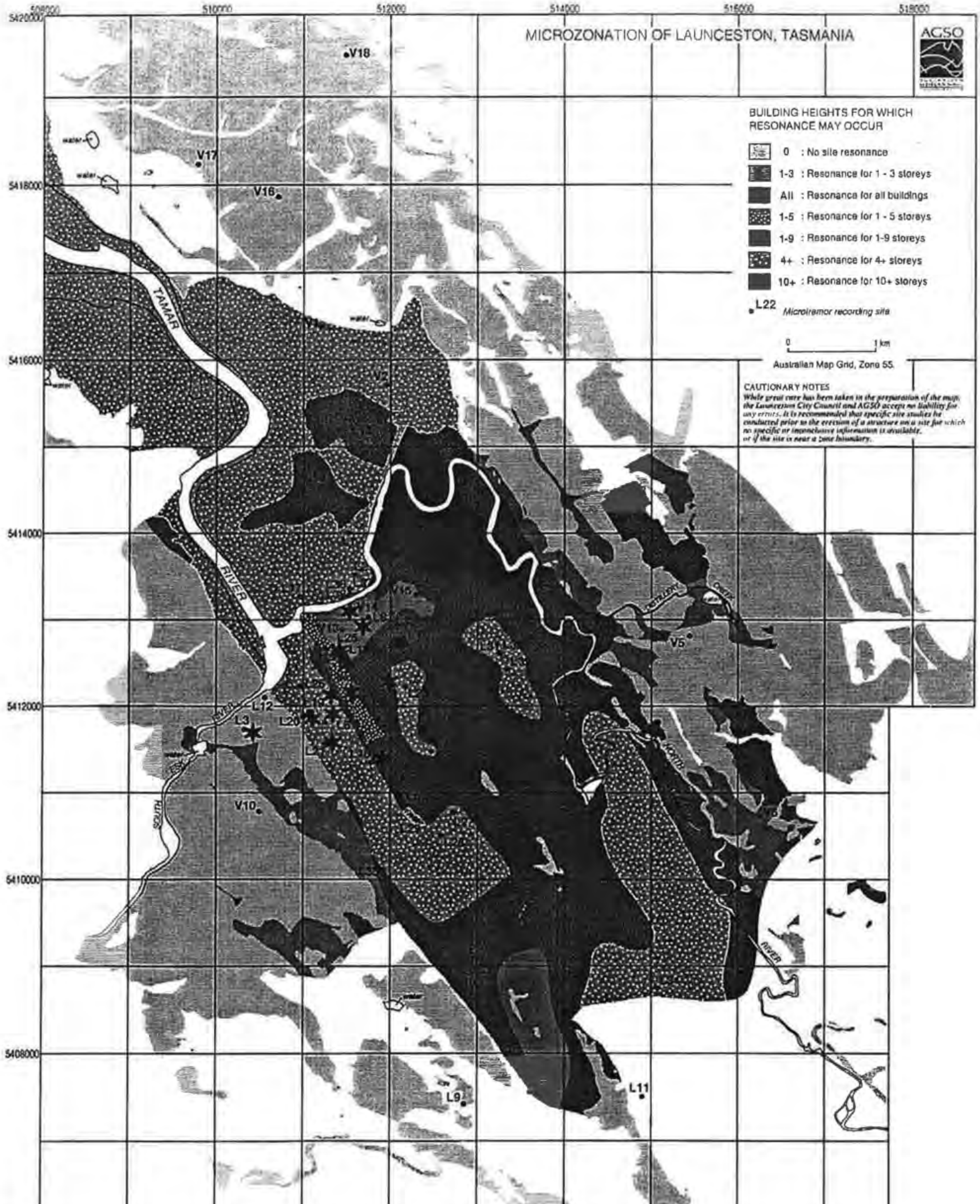

Figure 3. Microzonation map of Launceston showing building heights for which resonance may occur. The stars mark sites at which there has been earthquake damage.

A NOTE ON THE SHEAR CAPACITY OF MEMBRANE TYPE DAMP-PROOF COURSES

ADRIAN PAGE AND ROGER TAGGART

Paper not available

ABSTRACT:

Membrane type damp-proof courses (dpc's) are commonly used as a means of preventing moisture movement in masonry walls. Moisture movement is prevented by the damp-proof course acting as an impervious membrane, usually incorporated into a mortar joint. The most commonly used membranes are bitumen and/or plastic coated aluminium or embossed plastic. The damp-proof course can be sandwiched within a mortar joint, or more commonly, laid directly onto the masonry units of the course below and the mortar then laid on top.

By its very nature, the damp-proof course creates a plane of weakness, particularly if it is not sandwiched in a mortar joint. If the damp-proof course is placed directly on the masonry below there will obviously be no bond at the interface. This reduces the flexural and shear capacity of the joint, with the precompression from the loads above the plane under consideration providing the only contribution to the tensile and shear resistance. There is thus a need to evaluate the flexural and shear capacities of masonry joints containing a damp-proof course and subjected to various levels of compression to enable realistic wall designs to be carried out. This need is compounded by the fact that in the absence of suitable test data, the SAA Masonry Code AS3700 requires that the bond strength and frictional capacity of a joint containing a damp-proof course be taken as zero. This has serious ramifications, particularly for earthquake design.

This paper describes an experimental investigation of the shear capacity of the commonly used Australian damp-proof course materials, and suggests appropriate design procedures for use in conjunction with AS3700.

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