EARTHQUAKES IN AUSTRALIAN CITIES -

Can we ignore the risks?

Proceedings of a seminar held by the

Australian Earthquake Engineering Society

University of Queensland, Brisbane

1997

This work was published by the Australian Earthquake Engineering Society. The views expressed in the papers are those of the author(s) and not necessarily those of the Society.

ISBN 0 7325 1072 4

Editors: R Cuthbertson, W Boyce, C Lynam, S Jaume, B Butler

Publisher: Australian Earthquake Engineering Society

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The front cover depicts a seismogram recorded on the University of Queensland, Brisbane (BRS) seismograph. The event is a magnitude 6.3 earthquake that occurred near Derby, Western Australia, on 10 August, 1997

FOREWORD

The Organising Committee for the Society's 1997 Conference has attempted to combine the theme of the Conference - "Earthquakes in Australian cities" - with a "Learning Cycle". The process of an *Earthquake* and its *Effects* is followed by the *Response* and *Recovery* phases. We can *Learn* by observing this process and therefore *Plan* for the next *Earthquake*. All delegates should see that their area of interest or expertise can be applied at one or more points in the Cycle.

We have also gathered authors from as wide a range of interest areas as possible and we trust this is apparent in the contents of these Proceedings. We would like to thank the two invited speakers whose contributions added considerably to the variety and depth of material presented.

The Organising Committee would like to thank the Society's Secretary, Barbara Butler, for her experience and assistance in producing both these Proceedings and the Conference. We would also like to acknowledge the contributions made by all the sponsors.

Russell Cuthbertson
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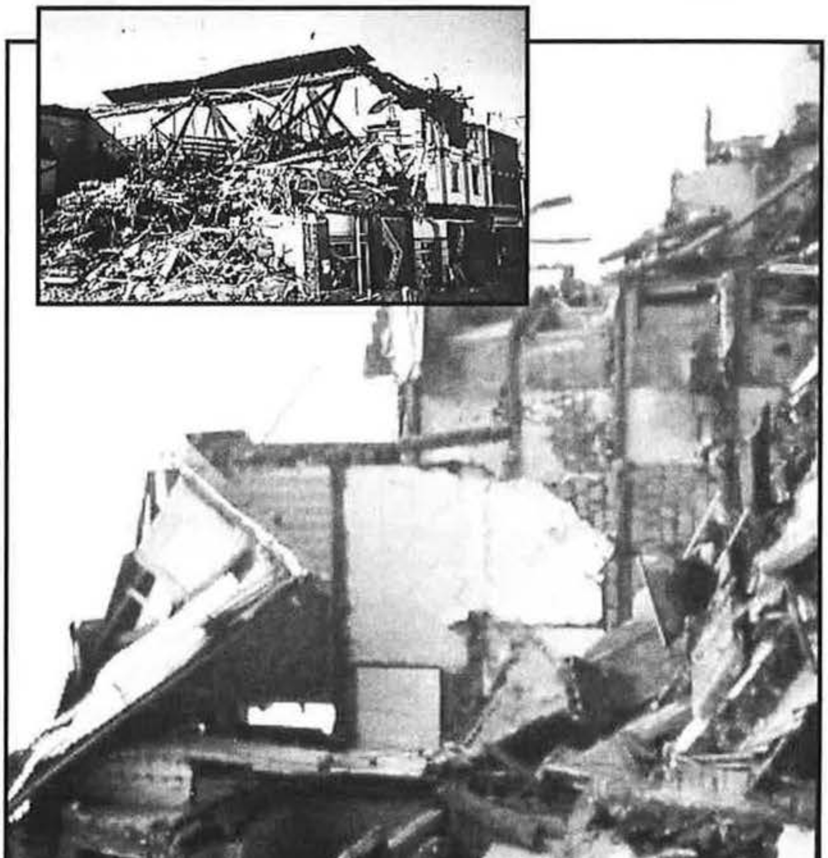

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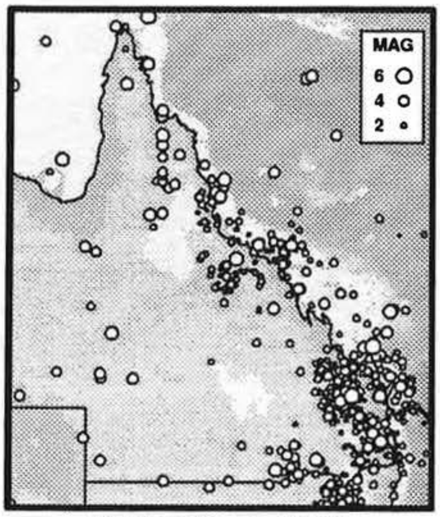

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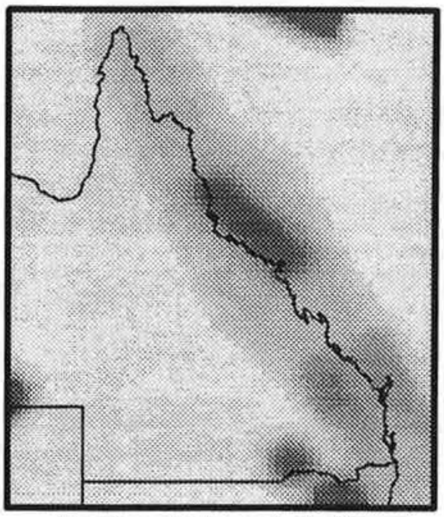
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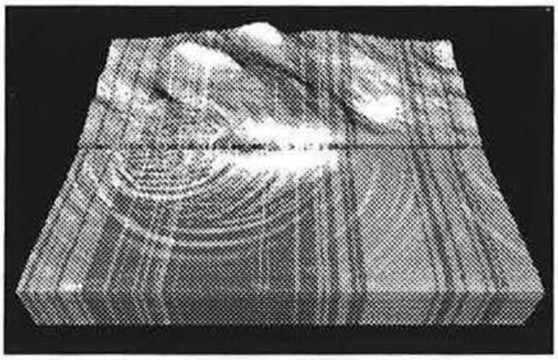
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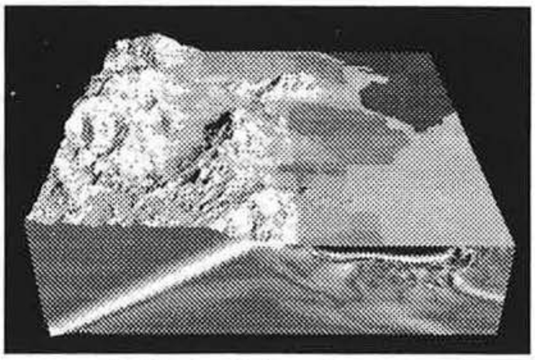
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The University of Queensland, Brisbane, 4072, Qld
Phone: (07) 3365 7418 International: +61 7 3365 7418
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T. PAULAY, EMERITUS PROFESSOR OF CIVIL ENGINEERING
OBE, BE, PH.D, HON.DSC, HON.DR.ENG, FRSNZ, FIPENZ, HON.ACI, HON.IAEE
DEPARTMENT OF CIVIL ENGINEERING, UNIVERSITY OF CANTERBURY,
CHRISTCHURCH, NEW ZEALAND

KEYNOTE ADDRESS

Professor Tom Paulay's structural engineering consulting practice was followed by 28 years of teaching and research relevant to the design of reinforced concrete structures and earthquake engineering. He was a member of the University of Canterbury research team which developed new seismic design strategies, now widely used. Professor Paulay is a past president of the International Association for Earthquake Engineering

Paper No. 1

SIMPLICITY AND CONFIDENCE IN SEISMIC DESIGN

T. Paulay
Emeritus Professor of Civil Engineering
OBE, BE, Ph.D, Hon.DSc, Hon.Dr.Eng, FRSNZ, FIPENZ, Hon.ACI, Hon.IAEE
Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand

Structural engineering consulting practice was followed by 28 years of teaching and research relevant to the design of reinforced concrete structures and earthquake engineering. A member of the University of Canterbury research team which developed new seismic design strategies, now widely used. Past president of the International Association for Earthquake Engineering.

ABSTRACT

It is established that structures can be designed and constructed so as to satisfy various seismic performance criteria, most importantly that of preventing collapse during an exceptionally large earthquake. For most engineers seismic design is synonymous with the complex analysis of elastic or inelastic dynamic response to random ground excitations. This presentation, reflecting the views of a structural designer, attempts to contrast 'analysis' with 'design strategies' that are suited to overcome difficulties that stem from inevitable uncertainties in the prediction of ground motions. Using reinforced concrete buildings as examples, it postulates the precept that the development of energy dissipating mechanisms in structural systems must not be left to randomness of ground motions. Rather a deterministic design philosophy is advocated whereby the designer can, within certain limits, choose the seismic response of a construction. The designer may thus enhance desirable and suppress undesirable features of structural behaviour. In this the vital role of the quality of the design and detailing of critical regions of structural systems is emphasised because this alone can assure the very desirable characteristic of ductile seismic response; 'tolerance' with respect to the inevitable crudeness of predicting earthquake imposed displacements.

1. INTRODUCTION

With emerging interest in Australia in aspects of earthquake phenomena and their consequences, a brief review of the state of the art of structural seismic design should be relevant. Some practitioners, including those with many years of experience, may feel uncomfortable when facing design features not encountered previously. Others may believe that earthquakes to be expected in suspectable regions of Australia need only be treated as a strong breeze. There may also be some apprehension, to some extent justified, that earthquake engineering is associated with complexity and sophistication in analyses and restrictions by codes, that appear to be excessive for Australian conditions. As the title suggests, this presentation attempts to dispel such apprehensions.

Fundamental principles and concepts of a seismic design philosophy, applicable to any region where earthquake effects need to be considered, are stated. Features specific to inelastic seismic response are stressed without delving into details of application. The strategy postulated and also adopted in countries other than New zealand enables the structural designer to "*tell the structure*" how to behave when the predicted ground motions occur.

2. BASIC AIMS IN SEISMIC DESIGN

The widely accepted aims of the seismic design of structural systems are best defined by recalling the *limit states* that are to be satisfied. Boundaries between limit states for seismic design cannot be defined precisely. However, a much larger degree of uncertainty is involved in the recommendations of building codes of various countries to determine the intensities of lateral seismic design forces. The simple design strategy described subsequently is intended to *accommodate* such *uncertainties.*

2.1 Serviceability Limit State

Relatively frequent earthquakes associated with minor intensity of ground shaking should not interfere with functionality, such as the normal operation of a building and the use of its content. *No damage* needing repair is expected. Essentially elastic response with predictable deformations is intended. Therefore the controlling structural property will be *stiffness*. Attention must also be paid to the dynamic response of the content of the building, such as equipment not attached to the structure. The vulnerability of non-structural elements, when subjected to large accelerations within the elastically responding structural system, must be addressed.

For meaningful predictions of the elastic response of reinforced concrete structures, realistic approximations for the evaluation of the effective stiffness of structural members need to be made. Within the domain of elastic response of reinforced concrete systems this will be affected primarily by the degree of cracking of the concrete and the restraint offered by joints where members are connected to each other. Many codes do not draw designers' attention to gross errors that result from a widely used practice, whereby stiffnesses are based on gross uncracked sections of reinforced concrete members.

Figure 1 shows typical load-displacement relationships for reinforced concrete flexural members. It suggests non-linear response, as a consequence of progressive cracking, well before inelastic strains develop. It also shows, what is encountered in laboratory experiments, that after repeated loading not exceeding approximately 75% of the nominal or ideal strength of the member, S_i, approximately linear response is obtained. This then can be used as a realistic estimate of flexural stiffness when examining service performance. The extension of this assumed linear response may also be utilized to define Δ_y, a *reference yield displacement* used subsequently to quantify conveniently the displacement ductility to which the member or the system may be subjected. Recommendations with respect to effective stiffnesses of elements affected by different degrees of cracking are available[1, 2].

2.2 Damage Control Limit State

For ground shaking with an intensity greater than that corresponding to the serviceability limit state, some damage may be expected. However, the damage with a low probability of occurrence during the expected life of the system should be economically repairable and be such as to enable reinstatements of the system to full service.

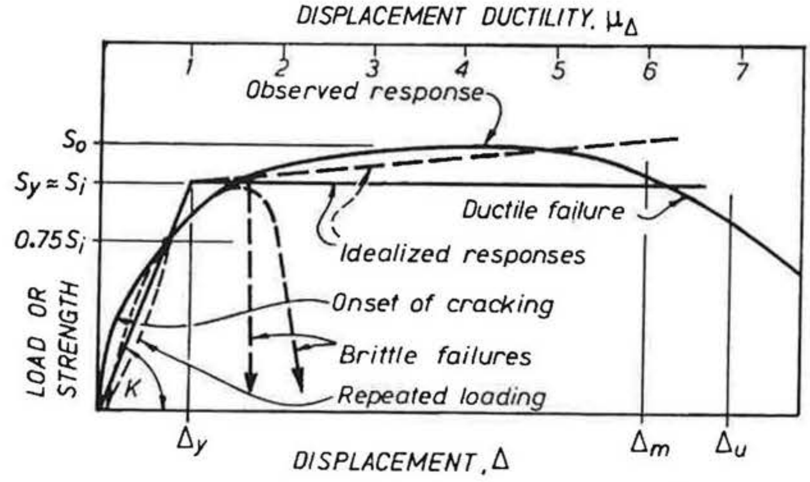

Fig 1. Typical load-displacement relationship for a reinforced concrete member.

If a concrete structure is to be protected against damage during a selected or specified seismic event, inelastic excursions of significance during its dynamic response should be prevented. This means that the structure should have *adequate strength* to resist internal actions generated during the elastic dynamic response of the system. This level of resistance is defined by the *ideal* or *nominal strength*, S_i, shown in Fig. 1.

Because the value at risk, represented by the content of a building or the operation accommodated, may be much larger than that of the building, damage limit states may control seismic design. The principles of performance based earthquake engineering, currently developed[3] address the relevant issues.

2.3 Ultimate Limit State

The single most important design criterion is the *preservation of life*. Because of the severe but rare seismic events that are considered, this is also referred to as a *survival limit state*. With judicious detailing of critical regions, structural damage to members may be minimized even after exceptional seismic events. However, large residual plastic deformations that could occur may make repair impractical. The principal aim is to ensure that collapse will not occur. Therefore the designer will need to address structural qualities which will ensure that for the expected duration of such a rare earthquake, relatively *large displacements can be accommodated* without significant loss of lateral force resistance, and that the integrity of the structure to support gravity loads is maintained.

The important property associated with this survival limit state is *ductility*, that is, ability to accommodate large inelastic deformations without significant loss of resistance. The exploitation of this property is a relatively recent and challenging feature in the evolution of structural engineering. For this and other reasons, emphasis in this presentation is placed on *inelastic structural response.*

The *ductility capacity* of a reinforced concrete building system is conveniently quantified by the ratio of the lateral displacement at a suitable level, such as the roof, to the yield displacement at the same level; that is, the *displacement ductility factor*. Because the transition from elastic to inelastic response is non-linear, acceptable simplifications need to be made, particularly with respect to the definition of the displacement at first yield (Section 2.1), such as shown in Fig. 1. While such global ductility is indicative of inelastic response of the entire system, the designer must also pay attention to repeated ductility demands that arise within critical potential plastic regions of the structure.

3. A DETERMINISTIC LIMIT STATE DESIGN STRATEGY

3.1 Concepts of a Philosophy

It must be recognized that current estimates of seismic activity at any given location are extremely crude. For common situations, the strength of the structure with respect to the resistance of lateral forces is chosen to be a fraction of the strength that corresponds to elastic dynamic response to a crudely predicted seismic event. Therefore building codes make approximations in estimating the reduced intensity of acceptable lateral design forces. Consideration in design of *elastic dynamic response* characteristics, such as the contribution of the higher mode shapes of vibration to internal structural actions, is often emphasised and recommended. However, in the ultimate limit state, *inelastic response*, relying on ductility and ability to dissipate seismic input energy, will primarily govern structural response. Because of the *crudeness* resulting from predictions of ground motions, the results of the analyses of models of elastic structures and code estimates relevant to ductility capacities, do not justify the often perceived accuracy aimed at, and claimed, in the design for an ultimate limit state.

These considerations suggest that gross approximations, particularly when they simplify routine design processes, are both attractive and justified. This is particularly the case when a structural system is rationally and deterministically *chosen* so as to be able to efficiently mobilise energy dissipating regions, examined in Section 3.2. Such systems should have ample reserve deformation capacity to accommodate significant departures from displacements associated with initial estimates for ductility. Such an approach avoids the need for sophisticated techniques of analysis to evaluate the development of numerous possible plastic mechanisms in a complex structural system. Instead, the strategy invites the designer to "*tell the structure*" where plastic hinges are desirable or convenient and practicable at the ultimate limit state, and to *proscribe plastification* in all other regions. The strategy leads to the establishment of a suitable strength or capacity hierarchy between components of the total system.

In structures so designed for earthquake resistance, distinct elements of the primary lateral force resisting system are chosen and suitably designed and detailed for energy dissipation under severe imposed deformations. All other elements are then to be protected against actions that could cause failure by providing them with strength greater than that corresponding to the maximum feasible strength in the potential plastic hinge regions.

The strategy requires the maximum probable values of *displacement-induced forces* to be estimated. Such forces are associated with the development of the *overstrength* of potential plastic hinges. This is examined in Section 3.3. For this, the strength properties of components *as built*, including strength enhancement of both steel and concrete under large imposed strains, need to be evaluated. The contribution to internal tension forces of all reinforcement, irrespective of its intended purpose, such as temperature or shrinkage control or to satisfy code and construction requirements, including those in floor slabs, must be included wherever such bars can be subjected to earthquake-induced tensile strains. When, in critical regions, excess reinforcement is provided, this must not be interpreted as a feature resulting in increased safety. In such cases, which are not uncommon, excessive overstrength may be developed and, as a consequence, all elements of the system intended to remain elastic must be designed for correspondingly increased resistance.

By now concepts of this philosophy have been incorporated in building code proposals in Europe[4] and Japan[5] and also for bridges[6].

3.2 The Choice of Plastic Mechanisms

In conformity with widely accepted principles, with very few exceptions, plastic mechanisms in reinforced concrete structures must rely on flexure as the source of energy dissipation. Therefore, with few exceptions, mechanisms associated with inelastic deformations originating from shear, transfer of bond between reinforcement and concrete, sliding and instability of members, must be *definitively suppressed*. The choice of the designer involves thus the selection of plastic hinges in beams, columns or walls that enable a kinematically admissible complete mechanism in the given structural system to be developed . An important aim in this selection is that for a given global or system displacement ductility demand,

μ_Δ, the associated curvature ductilities at plastic hinges remain within proven limits.

The above considerations are illustrated in a series of diagrams in Fig. 2. These show desirable or acceptable mechanisms, and those that are to be avoided. The same ultimate displacement, Δ_u, has been assumed for all the example systems shown. The advantages of *strong column - weak beam mechanisms* in ductile multistorey frames are well established. When columns are provided with sufficient strength, plastic hinge formation in them can be avoided in all storeys above level 2, as shown in Fig. 2(a). When columns are adequately detailed for hinge formation, the widely used system shown in Fig. 2(b) may also be adopted. It is to be noted, however, that the possibility of simultaneous hinge formation at both ends of columns in any story, such as seen in Fig. 2(c), generally termed a *soft storey*, must, with the exception of low-rise buildings, not be permitted. It is evident that rotational ductility demands in the hinging columns, such as shown in Fig. 2(c), may become excessive.

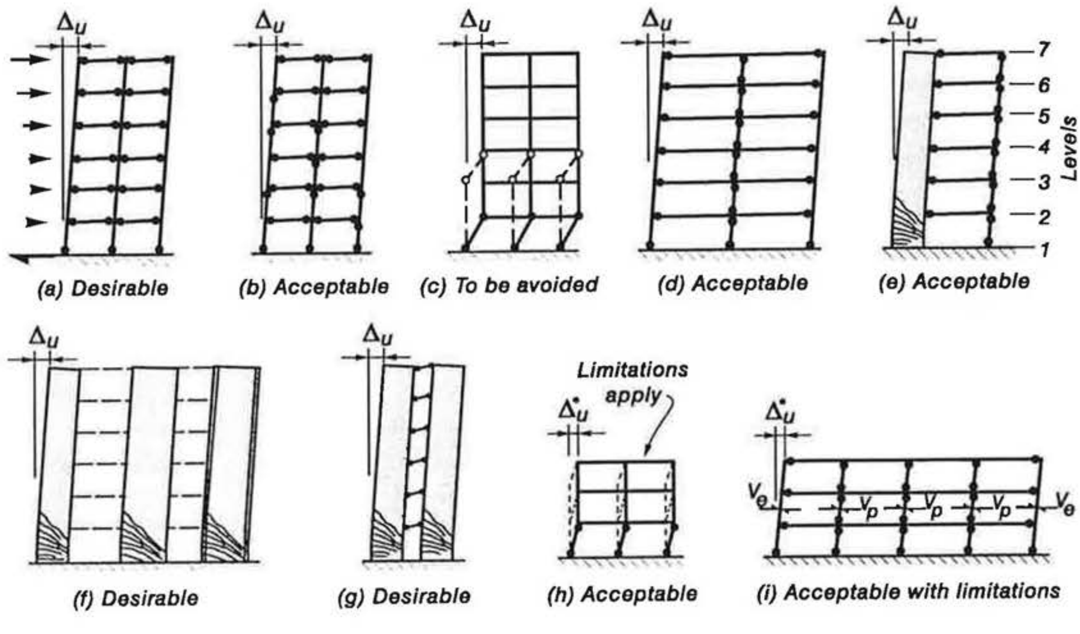

Fig. 2. Plastic mechanisms in multistorey buildings.

The plastic mechanism in frames, shown in Fig. 2(b), is implied in the seismic design philosophy practised and codified in the United States and in many other countries where similar recommendations have been adopted. In terms of the detailing of the columns for ductile response, this mechanism requires in the end regions of columns, transverse reinforcement in the form of stirrups, hoops, ties or spirals, in addition to those used in similar columns constructed in regions not affected by seismic considerations. This is to ensure that the rotational ductility capacity of a potential plastic hinge at the bottom *or* the top end of the column in a story is adequate. Moreover, the location of lapped splices of the principal column reinforcement is required to be in the region of midstorey height. Under reversing cyclic inelastic straining of the reinforcement, the capacity of lapped splices is known to deteriorate rapidly, unless heavy transverse reinforcement providing adequate clamping of each pair of spliced bars, is provided. Another reason for avoiding lapped splices in potential plastic hinge regions, even if adequately reinforced, is the drastic reduction of the length over which reinforcing bars can yield. Thereby for a given hinge rotation, correspondingly increased and perhaps excessive tensile steel strains will be developed.

The system illustrated in Fig. 2(a) allows both the reduction of the amount of transverse reinforcement in the end regions of columns above level 2 and the placing of lapped splices immediately above the top of a floor. This convenient concession is justified because, with adequate precautions, the formation of plastic hinges with significant curvature ductility demands need not be expected in such columns.

When the exterior columns of a frame, absorbing actions from one beam only, are made strong enough to ensure that a *soft storey* cannot develop, the formation of simultaneous plastic hinges at both ends of all interior columns (Fig. 2(d)) is acceptable, provided that all potential plastic hinges in these columns are detailed accordingly. A similar situation, usually encountered when long span beams need to be used, arises when walls (Fig. 2(e)) ensure that a soft storey cannot develop. With certain limitations frames relying mainly on energy dissipation by columns, shown in Fig. 2(i), are also acceptable.

When structural walls, which are interconnected by adequately designed diaphragms, are used to provide the necessary lateral force resistance, it is preferable to restrict the formation of plastic hinges to the base of the walls, acting as cantilevers, as shown in Fig. 2(f). This can be readily achieved when the walls above the plastic zone are provided with sufficient reserve flexural capacity. The precaution should ensure that, irrespective of the ground motions, essentially elastic response of such walls will occur at levels above the potential plastic region. This will allow, in the elastic regions of the walls, to dispense with the more onerous requirements of the detailing for ductility.

Figure 2(g) shows the desirable and readily achievable plastic mechanism in a pair of structural walls interconnected by coupling beams which are specially reinforced to enable large member ductilities to be developed. This system is capable of delivering excellent energy dissipating capacity in a very stable manner.

Frames in which *soft storeys* can develop, as shown in Fig. 2(h), should only be used when the displacement ductility demand assigned to them is restricted. Moreover, for an assumed overall displacement ductility demand, μ_Δ, it is necessary to evaluate the rotational ductility demand that may be imposed on the plastic hinges that are to be expected to form at both ends of all columns in any of the soft storeys. The plastic hinges of columns of such frames, designed for severely restricted ductility demands, are likely to be required to be detailed for large curvature ductility demands. The structure shown in Fig. 2(h) is an example to illustrate the need for the evaluation of *local ductility demand* as a function of the overall displacement ductility associated with the deflection, Δ_u , at the ultimate limit state.

When admissible plastic mechanisms, such as the examples shown in Fig. 2, are chosen, it becomes evident *which members*, or parts of members, are to remain elastic at all times. All that needs to be done, is to estimate the *overstrength* of the selected plastic hinges, *as detailed and hence constructed*. This is reviewed in the next section. The resulting actions due to the development of ductility, when combined with those due to gravity, lead then to the design actions for members or regions to be protected against inelastic deformations.

3.3 Overstrength

The term overstrength was introduced in the late 1970s with the development in New Zealand of the philosophy of capacity design. It was necessitated by the realization of the need to know the probable maximum actions that could be developed in materials, structural members and the entire structural system, as a result of large ductility demands imposed by earthquakes. This enables then the desired hierarchy of strengths within the system to be quantified.

Although the designers deal with forces, it is to be noted that *earthquakes impose deformations*. These in turn generate forces corresponding to the *capacity of members as built*. It is thus essential that the maximum likely actions, i.e., the overstrength, so developed in potential plastic hinges are estimated. This strength estimate must be based on the properties of the structure as constructed, on the maximum likely strength properties of constituent materials, including some strain-hardening of reinforcing steel, and the contribution of all sources, including those normally neglected in routine structural design.

Regions intended to remain elastic and hence not requiring special detailing, can then be protected against 'overloading'. The simplest example is the design of a member for shear, an action associated with a brittle failure mode, where the critical value must be based on the flexural overstrength of the plastic hinges that can develop at predetermined locations of that member. This implies a *knowledge of the real strength* properties of the materials, particularly that of steel, to be used.

4. STRUCTURES WITH RESTRICTED DUCTILITY

4.1 Conditions For Restricted Ductility Demands

In many situations earthquakes are unlikely to impose large ductility demands on structures. This will be commonly the case in Australia. Ranges of limit states in terms of ductility that are generally considered in seismic design, are illustrated in Fig. 3. It shows the approximate bands that may be used to define fully ductile, or restricted ductile and essentially elastic response corresponding with displacement ductility factors of the order of 6, 2.5 and 1.25, respectively. In this example a single structure with a given stiffness, to define yield displacement, Δ_{yi}, was used. The dashed lines in Fig. 3 indicate the likely non-linear response.

Restricted ductility demands may arise in typical situations shown in Fig. 4.

(1) Inherently a structure may possess strength considerably in excess of the strength corresponding to fully-ductile seismic response. The causes of this may be moderate seismicity in the region, or that other design requirements for strength, such as gravity loading and wind forces, are more critical. The tall structure shown in Fig. 4(e) may be one for which design actions resulting from wind forces would have been greater than those derived for code specified seismic forces corresponding with full ductility demand. Because earthquake-induced displacements in the structure shown in Fig. 4(c) will be controlled by the perforated wall, ductility demands on members of the attached frames will be limited.

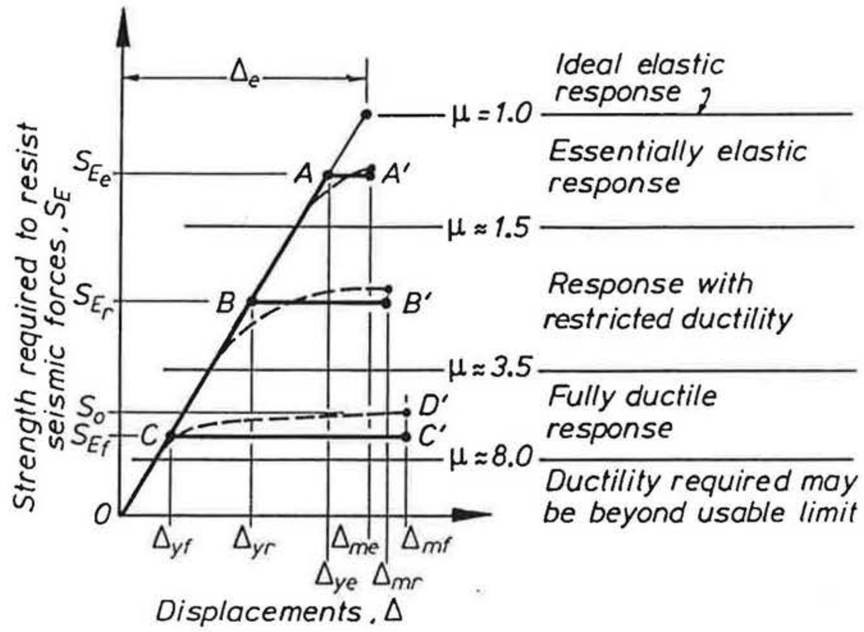

Fig. 3. Relationships between strength and ductility.

(2) In certain circumstances detailing for full ductility may be found to be difficult or too costly. The adoption of larger seismic design forces, to reduce ductility demands during earthquakes, may be a more attractive proposition. Greater economic benefits may well be derived from consequent simplifications in the detailing for reduced ductility and perhaps from adopting less than optimal locations for plastic hinges (Fig. 4(d)).

(3) There are moderate sized structures, the configurations of which do not readily permit a clear classification in terms of structural types. The precise modelling of such structures may often be difficult. Consequently, the prediction of their inelastic seismic response is likely to be rather crude. However, without incurring economic penalty, a more conservative design approach, relying on increased lateral force resistance and consequent reduction in ductility demands, may be more promising. Examples of such structures are seen in Figs. 4(b) and (c).

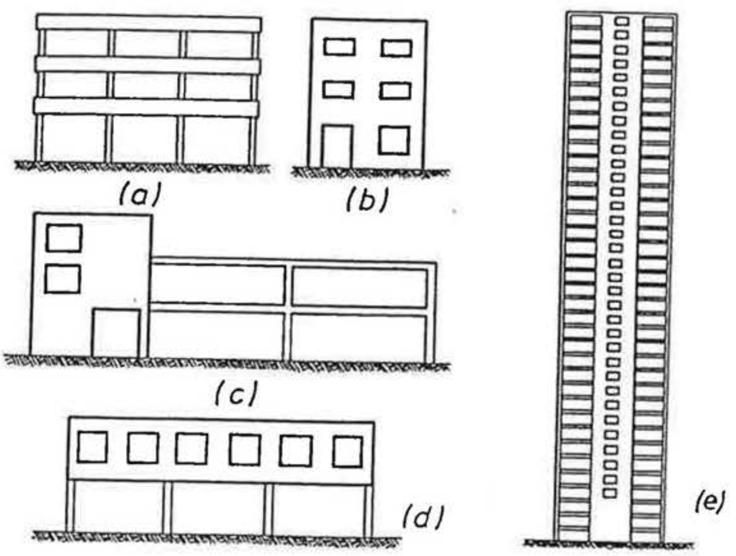

Fig. 4. Examples of structures with restricted ductility.

(4) The overall ductility demand on certain structural systems may need to be reduced in order to protect certain elements from excessive local ductility demands. An example is shown in Figs. 4(a).

(5) Restriction on ductility will arise when, for the sake of protecting invaluable building content, inelastic displacement must be limited.

4.2 The Applicable Design Strategy

In terms of the *choice of seismic design procedures*, there are only two classes of structures, those responding elastically and ductile systems. In essentially elastic structures, shown in Fig. 3, inelastic deformations of any significance are not expected. Hence no special detailing for any region, other than that adapted for gravity load conditions, may be required. In ductile systems, as Fig. 3 implies, the magnitudes of ductility demands and hence the need for improved detailing, will vary.

Structures, typically used for buildings, the design of which is intentionally based on *elastic response*, may in fact enter the inelastic range when the seismic event is larger than anticipated. Therefore structures, even in this class, should have at least some *reserve ductility capacity*.

In structures of restricted ductility, which will be commonly encountered in Australia, the development of plastic regions within the structural system *must be allowed for*, as in fully-ductile structures. Therefore, the establishment of a kinematically admissible plastic mechanism in structural systems, even if it is exposed only to limited displacement ductility demands, is essential. The location of these regions should be clearly *identified* to enable the necessary detailing to be provided. It has been often suggested that structures with limited ductility should be considered as a separate group for which simplifications in both analysis and detailing of the reinforcement should be adopted. The justification for this is claimed to be that in terms of seismic effects, such structures are seldom critical and hence a high degree of sophistication in their design is not warranted. When the design process for ductile systems with a transparent rationale is made simple, it can be applied with equal ease to this group of structures. The only difference in the design process that is suggested, is a *quantified relaxation* for systems with restricted ductility of the requirements established for the detailing of fully ductile reinforced concrete structures. It is necessary to ensure that no serious underestimates, resulting from oversimplified and often arbitrary rules of thumb, will be made of likely local ductility demands in systems which are expected to be exposed only to restricted overall ductility.

5. CONCLUSIONS

(1) In the context of the state of the art in structural engineering, current predictions of the probable characteristics of earthquake-generated ground motions are *crude*. Under these circumstances an aim to achieve a degree of *precision* in analytical techniques, comparable to those developed for structures to satisfy serviceability and 'hypothetical' ultimate limit states, to predict both earthquake induced actions and deformations within the structure, is *not justified*.

(2) Provided that a reasonable level of resistance to lateral forces, such as prescribed by relevant national building codes, is chosen, then inevitable errors arising from crude estimations of the characteristics of ground motions will manifest themselves only in *erroneous predictions* of earthquake imposed displacements, that is *ductility demands*. Thus *deformation capacity reserve* is an extremely important structural property in areas of high seismic risk.

(3) Types and localities of energy dissipation mechanisms within a system need to be chosen as part of an *effective seismic design procedure*, in which a unique hierarchy of strengths is established. All weak and necessarily ductile links must satisfy requirement of the stipulated level of lateral force

resistance. In such an inelastic system the *maximum resistance that may be developed* during an earthquake can be predicted with a relatively high degree of precision.

(4) As a general rule, *rationally detailed* reinforced concrete structures can be made very ductile with relative ease and little, if any, additional cost. Thereby a considerable reserve for potential inelastic deformations, that is *ductility capacity*, can be imparted to structural systems. Detailing of reinforced concrete structures deserves at least as much attention as the analytical work used to estimate design actions. Faults in detailing are the first that will be revealed during earthquakes. They are predominant causes of structural distress.

(5) Various steps in the description in previous sections of the design procedure were intended to emphasise the designer's *determination* to simply 'tell the structure what to do'. It is in this respect that the design strategy is *deterministic*. It inhibits the activation of mechanisms other than *those chosen*. Construction practice must manifest unambiguously the *goodness of detailing*. Thereby reinforced concrete buildings can be made very *tolerant* to a wide range of ductility demands. Hence they can be expected with *confidence* to perform 'as they were told to'.

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A MULTI-DISCIPLINARY APPROACH TO EARTHQUAKE HAZARDS MITIGATION

DANIEL P. ABRAMS
PROFESSOR OF CIVIL ENGINEERING
UNIVERSITY OF ILLINOIS AT URBANA-CHAMPAIGN

KEYNOTE ADDRESS

Dan Abrams is a structural engineering professor, and is noted for his research in earthquake response of building structures. He has chaired a number of national technical committees in the United States and has served on the Research Committee of the National Center for Earthquake Engineering Research.

ABSTRACT:

The paper provides a summary of a coordinated research program supported by the U.S. National Center for Earthquake Engineering Research known as the "Loss Assessment of Memphis Buildings" (LAMB) project. The paper provides a general background to the methodology for assessing economic losses for a scenario earthquake in the Memphis region. The multi-disciplinary study consisted of investigations by researchers in the areas of structural engineering, risk/reliability, seismology and economics. Fragility curves for typical buildings constructed with concrete frames, unreinforced masonry shear walls or masonry infill-frame systems were generated from representative nonlinear force-deflection relations of structural elements, and synthetic ground motions. These curves were then used to estimate economic losses resulting from damage to these building types. The emphasis of the presentation is on the viability of the multi-disciplinary approach for investigating the impact of future earthquake hazards on the built environment.

Paper No. 2

1. INTRODUCTION

The central emphasis of the U.S. National Center for Earthquake Engineering Research (NCEER) Building Project for the last ten years has been evaluation and rehabilitation of existing gravity-load designed buildings in the eastern United States. Numerous research investigations were done to study hysteretic force-deflection relations for concrete and masonry structural elements not necessarily designed to resist seismic action, and dynamic response of building systems comprised of them. Significant contributions were made to the development of national guidelines for seismic evaluation and rehabilitation of existing buildings from this research. However, over the last two years, application of NCEER building research has extended beyond the conventional context of structural engineering codes or computational procedures. Information on cyclic behavior of building structures, and newly developed computational models for nonlinear dynamic response, were used to help assess urban losses for a scenario earthquake in the Memphis region. Fragility curves for concrete and masonry buildings were generated using these computational models, and economic losses were estimated from these fragility curves. Researchers with combined talents from structural engineering, seismology, risk/reliability and socio-economic disciplines comprised a new coordinated program which was named the Loss Assessment of Memphis Buildings (LAMB) project, and is the subject of this presentation.

2. OBJECTIVE

The immediate goal of the LAMB project was to assess probable losses and risks for specific building types in Memphis. The overall purpose of the exercise, however, was to develop a unified methodology that could be applied to other building types in other geographical regions. The LAMB project was an experiment to study how various disciplines could be coordinated to research a common theme in earthquake engineering.

3. OVERVIEW OF PROGRAM

A brief overview of the tasks involved with the LAMB project is given to illustrate scope and integration. Information obtained from past laboratory experiments on nonlinear behavior and response of: (a) lightly reinforced concrete frame members and systems, (b) unreinforced masonry infill panels confined in frame systems, and (c) unreinforced masonry bearing wall systems were used to develop and verify nonlinear dynamic response computational models. These models were then used by risk/reliability investigators to generate a series of fragility curves for these three structural types which represented damage levels for a series of earthquake motions that were synthesized by seismological researchers. Loss estimates were then deduced using these fragility curves with data collected from building inventory studies.

3.1 Structural Behavior and Response Computation Models

Deterministic estimates of the strength and ductility of building structural components and dynamic response of building systems have been fundamental aspects of NCEER research since the start of the Center. Numerous laboratory investigations were done to define response and behavior of reinforced concrete frame systems[(7)], unreinforced masonry bearing wall systems[(2)], and concrete frames with masonry infill panels[(1)]. Computational models were developed for estimating nonlinear dynamic response of such building systems to earthquake excitation. The models were contrived using experimental data on component force-deflection relations, and confirmed with measured dynamic response of shaking table test structures. The experimental studies concentrated on seismic response of buildings that were not necessarily designed to resist lateral seismic forces as is typical of much of the building stock in the eastern United States.

3.2 Synthetic Earthquake Motions

The Seismic Hazards program was relied on for developing synthetic ground motions for the Memphis region. Synthetic ground motions were based on different stress drops, distances from the epicenter and soil types. Procedures for developing synthetic ground motions[(4)] were adapted from previous NCEER seismological research. These motions were based on a scenario earthquake of magnitude 7.5 at Marked Tree, Arkansas along a strike-slip focal mechanism on a northeast striking, vertical fault. Epicentral distances ranged from 40 to 90 km to give the full range of distances to locations in Shelby County. Synthetic records were developed for a sampling of twenty acceleration histories for each of ten peak ground accelerations ranging from 0.05 to 0.50g. To obtain these records, moment magnitude was varied from 5.0 to 7.5 and stress drop was varied from 100 to 500 bars.

3.3 Fragility Analyses

Fragility curves were determined for three categories of damage: Insignificant, Moderate and High. These categories were analogous with those used in a recent study (ATC-38) on performance of structures near strong-motion recording sites. Each damage category was correlated quantitatively with a structural parameter such as lateral drift percentage for each type of building system (concrete frame, masonry wall, or infill-frame system).

Fragility curves were developed based on procedures developed through the NCEER Risk and Reliability program[(5)]. Probability of failure was plotted versus the peak ground acceleration for a scenario earthquake for each of three damage categories. Parameters other than peak ground acceleration were explored for expressing seismic intensity including spectral acceleration and modified Mercali intensity.

Substudies were done examining the sensitivity of fragility curves to different modeling assumptions, and different building configurations. Information developed from these investigations was used to assess the precision of the overall loss estimation procedure, and to confirm that a sufficient variance in configurations had been selected.

3.4 Building Inventory

The LAMB project developed information on the inventory of buildings within the Memphis region. Information obtained previously with the LAMB project regarding the inventory of masonry and concrete buildings in the Memphis region[(6)] was relied on for selection of the sample building systems, and for extrapolation of the results. This study made direct estimates of the distribution of buildings in Memphis and their square footage by structural type and use through an inventory of the buildings in Shelby County using records of tax assessors. According to this study, the inventory of concrete (10.8%) and masonry buildings (18.2%) in Memphis represents nearly a third of the total floor area for all buildings. Comparisons of these distributions with those in Wichita, Los Angeles and Salt Lake County revealed conforming patterns that were useful in extrapolating results of the LAMB project nation wide.

3.5 Loss Estimates

Loss estimates were based on research within the NCEER Socio-Economic program. Direct economic losses were considered initially and followed by inclusion of indirect losses. Economic losses were estimated based on probabilities of attaining the three levels of damage (Insignificant, Moderate and High) for specific earthquake intensities. Procedures for estimating losses from fragility curves[(3)] were used to estimate losses associated with damage to existing building systems. Losses were determined for various damage levels by relating estimated costs of repair with each damage category. Repair cost models were based on the ATC Northridge field survey database (ATC-38) and on a mail survey to building owners conducted by EQE following the Northridge earthquake. In the ATC-38 database, each building was evaluated in terms of both the general damage state and quantitative repair costs for structural, nonstructural, equipment and contents damage or loss, respectively. Repair cost was defined in terms of damage factors or percentages of replacement value.

4. CONCLUDING REMARKS

The LAMB study was an initial effort to coordinate research between structural engineers, seismologists, risk/reliability researchers and socio-economic researchers. The research program was conceived after much of the basic research had been completed, and attempted to provide a post-integration of research results from these various disciplines. As a consequence, economic losses were limited to those resulting from seismic damage to concrete frame, unreinforced masonry wall or infill-frame construction since these were the three building systems that were investigated with prior NCEER research. More meaningful estimates, particularly for indirect losses, would need to include possible damage to the entire building stock in Memphis as well as to non-building structures and lifelines. However, since the overall objective of the exercise was to explore development of a methodology that could be expanded to more diverse building stocks and locales, such future loss estimates could rely on knowledge gained from the LAMB project.

Memphis was used as a case study to complement previous NCEER research on lifeline vulnerability in the region. Furthermore, data was available on the Memphis building stock and local ground motions. The loss assessment methodology could be used to study other urban areas if additional investigations are done to define building inventories and expected local ground motions.

In general, the LAMB project focused on damage to existing buildings in the Memphis area that have not been retrofitted. The methodology as developed could also be applied to a group of rehabilitated buildings to study the probable reduction in losses that retrofit measures could provide. By comparing rehabilitation costs with probable loss estimates, retrofit strategies could be developed.

5. ACKNOWLEDGMENTS

The research reported on in this report was funded by the National Center for Earthquake Engineering Research at the State University of New York at Buffalo which is supported by the National Science Foundation. Research results were coordinated from several individual projects supported by the Center in Years 9 and 10. More detailed information on any one project can be found in individual research project reports.

Any opinions, findings and statements made in the individual chapters are those of the authors, and do not necessarily reflect those of NCEER or NSF.

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NATURAL DISASTERS IN AUSTRALIA - DO EARTHQUAKES DOMINATE THE LOSS POTENTIAL ?

ERNST RAUCH, GEOPHYSICIST
MUNICH REINSURANCE CO., MUNICH, GERMANY

AUTHOR:

ABSTRACT:

The moderate M 5.6 event in Newcastle in 1989 highlighted the loss potential from earthquake disasters in Australia for the insurance industry. A similar event close to the center of one of Australia's "megacities" - with possible losses in the multi billion A$ order - would shake both the local economy and the insurance market.

However other events can also produce enormous damage, as Cyclone Tracy in Darwin in 1974 or the Sydney hailstorm of 1990 have shown. The paper looks at the potential losses from earthquakes and a number of different natural disaster scenarios and compares the earthquake risk (risk = hazard x vulnerability) in Australia with the risk from other types of event.

Paper No. 3

THE BURRA ML5.1 EARTHQUAKE

HUGH MOUNTFORD, DAVID LOVE AND CVETAN SINADINOVSKI

AUTHOR:

Hugh Mountford has extensive experience in investigating and reporting on building failures of many kinds. He commenced work as a structural draftsman in the former Public Buildings Department but branched into consulting work in 1970 in the offices of Hosking, Fargher & Oborn, where under the guidance of Philip Fargher achieved significant experience in aspects of soil technology and of building behaviour. From 1980-1985 with R.M. Heriot & Associates, he gained further experience in relation to building inspection and the then new concept of rational design procedures for domestic footings.

From 1985 to the present, Hugh Mountford has been increasingly involved in technical reporting on building behaviour with particular emphasis on reporting to insurance companies on damage sustained by buildings from various causes, including earthquakes.

ABSTRACT:

On 5th March 97 at 4:45pm Adelaide experienced its worst shaking since the infamous 1954 earthquake. There was no major damage and no injuries, although intensities in the epicentral area 130Km north of Adelaide reached MM6. An aftershock survey recorded only a few events, but located the epicentre 15km SSE of Burra, about 15km deep, and revealed a compressive focal mechanism. Detailed inspection of over 100 properties near the epicentre for insurance purposes has revealed a north-south elongation of the strongest intensities, generally following the line of the mountain range under which the event occurred. Few cases of totally new cracking were observed but a number of cases of extended cracking, and a number who found existing cracks for the first time. The strong motion network (JUMP) produced good data with four out of five instruments triggerring. They reveal that high frequencies transmitted very well in a southerly direction, but poorly in a north-westerly direction. The Adelaide soft-rock accelerometer showed clear horizontal amplification at around 1Hz in agreement with the results of a recent microtremor survey.

INTRODUCTION

On 5th March 1997 at 4:45pm, Adelaide experienced its worst shaking since the 1954 earthquake. It was fortunate that the epicentre was 130 km north of Adelaide, between Burra and Robertstown in the North Mount Lofty Ranges, and not directly under a major town. There was no major damage and no injuries, however there were many insurance claims. The area experiences a moderate level of activity with a similar sized event in 1889. The shock was well recorded by four of the five digital accelerometers installed as part of the Joint Urban Monitoring Project. One of the authors (Mountford) has inspected over 160 properties, mainly near the epicentre, as a result of insurance claims and this enabled the collection of a valuable isoseismal data set to compare with a wide questionnaire survey.

SOURCE PARAMETERS

The main shock was of Richter magnitude 5.1. There was no discernible increase or decrease in activity locally in the previous two years. There were three aftershocks of magnitudes 2.8 to 3.5 in the two hours following and two aftershocks of magnitude 2.3 about 12 hours later. In the following 8 weeks there were only two events over magnitude 2. The first portable seismographs were installed in the area 26 hours after the event, and they were removed a week later. Two events of magnitudes 1.6 and 1.8 were recorded on all five instruments. From these the best estimate of the main shock epicentre is at 138.984° S and 33.816° E.

For the main shock, depth phases from five stations in the distance range 64° to 136° gave depths of 16.8 to 24.6 km with an average of 20.4 km. Depths calculated from an aftershock were 14.1 to 16.0 km. This is relatively deep for a moderately large Australian event and would explain the small number of aftershocks.

First motions from 25 stations in South Australia, Victoria and New South Wales were used to produce a focal mechanism (Figure 1). They were complemented by five compressional readings from the located aftershock. The mechanism is one of almost pure compression in a direction close to east-west. One of the nodal planes (357°, dipping 52° west) is very close to the direction of the ranges in the area (345°).

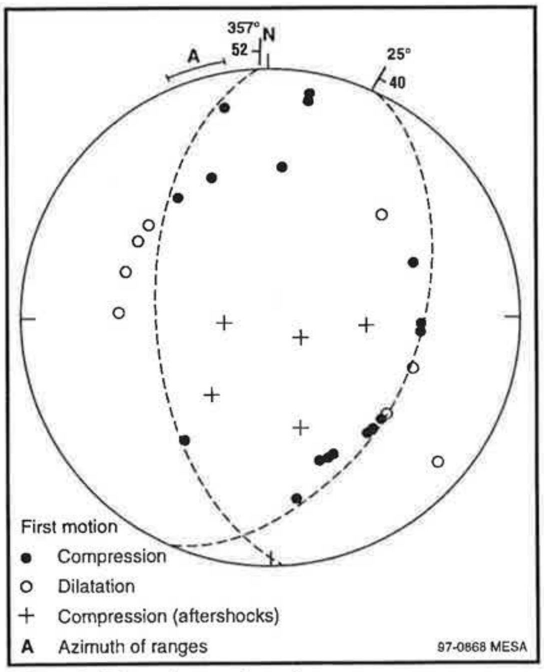

Fig. 1 Focal mechanism

ACCELERATION RECORDINGS

Four digital accelerograms were recorded at Government House (GHS, Adelaide,soil) Mt Osmond (TUK, near Adelaide, rock), Napperby (NAP, near Port Pirie, rock), and Whyalla (WHY, rock) (Figure 2). Epicentral distances were 127, 131, 105, and 155 km respectively. Respective peak accelerations were 0.0047g, 0.0010g, 0.0021g (calibration uncertain), and 0.0009g. Of the four stations, GHS and TUK are both SSW of the event and NAP and WHY are both NW of the event. There is a significant difference between the two pairs of spectra. They are shown normalised in figure 3.

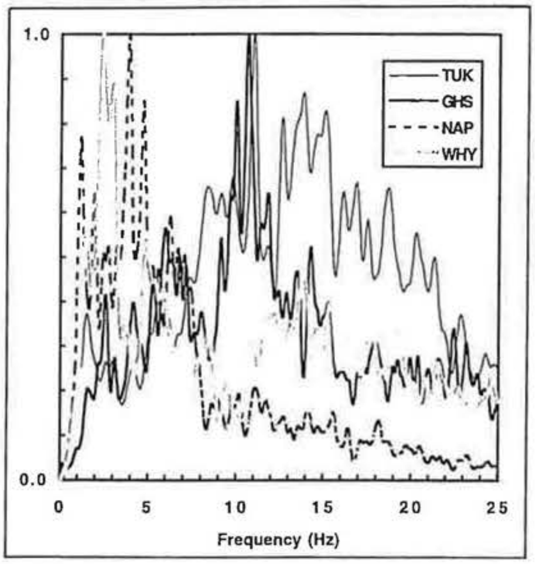

Fig. 2 Normalised acceleration spectra

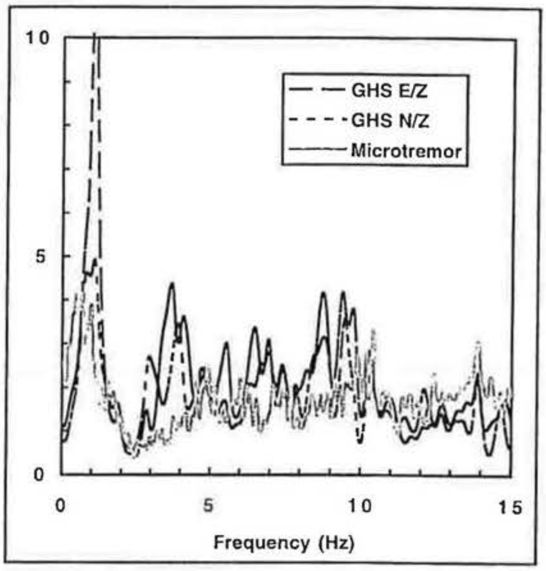

Fig. 3 Earthquake vs microtremor

In the SSW direction most of the energy is in the frequency range 7 to 20Hz while for the NW direction it is in the range 2 to 7 Hz. Spectral amplification of the Adelaide soil site is in good agreement with a microtremor reading near GHS (Figure 4).

INTENSITY ASSESSMENT ENQUIRY

For the description of ground motion, Mountford used the Modified Mercalli Scale of Earthquake Intensity (MMI) as a basis for the assessments. The MMI was included with the superseded Australian Standard AS2121-1979, and no other intensity scale is now referred to in Australian Standards. The MSK scale was considered, but early comparisons showed no specific advantage, as the intensity very soon became defined as not exceeding MM6 and it was not seen as appropriate to refer to the percentage based quantitative aspects defined in the MSK scale, because of the generally sparsely populated area involved.

At each site the person(s) present within the house at the time of the tremor was interviewed. Questions extracted from the MMI were asked initially, and the responses noted. Following this, each person was provided with a copy of the MMI and asked to select an intensity from it, consistent with their experiences and observations. The MMI descriptors fall into two categories, firstly, aspects of human and animal perceptions, and secondly, the response of inanimate objects within or around the building. The human perception was frequently selected at levels higher than that which would be indicated from the inanimate response for that level. Many of those interviewed selected MM6 as appropriate in the first instance, because of the references contained within that level to cracking of masonry. However, there was generally a lack of supporting response from the inanimate objects within particular houses that could be related to level MM6 - the descriptors from level MM5 were those most referred to.

INTENSITY ASSESSMENT INSPECTION

The selection of the appropriate superstructure classification was made taking due account of each building's age and nature. Many had been built in stages and had mixed classifications. Structures classified as Masonry D included some old miners' cottages of stone and pug (clay core) construction, well constructed stone masonry laid in ashlar coursing, and buildings incorporating lime mortar. More recent buildings (say post 1930) were regarded as masonry C because of the probable use of cement mortars.

A detailed inspection of the walling of the house was then made to determine, in particular, whether any two dimensional diagonal shear failures in masonry could be found, particularly around window openings, or in other areas where racking movements were regarded as likely to occur. The discovery of such cracking would lead to the selection of MM6; if all that was found was obviously recurrent cracking (commonly caused by reactive clay soils), the selection of MM5 would follow. The extent of damage was variable, but closest to the epicentre several buildings sustained minor damage, of fresh, two dimensional diagonal shear failure form. In most (but not all) cases where MM6 was determined by way of the cracking form, there were other confirming inanimate descriptors from MM6, the most common of these being pictures falling from walls.

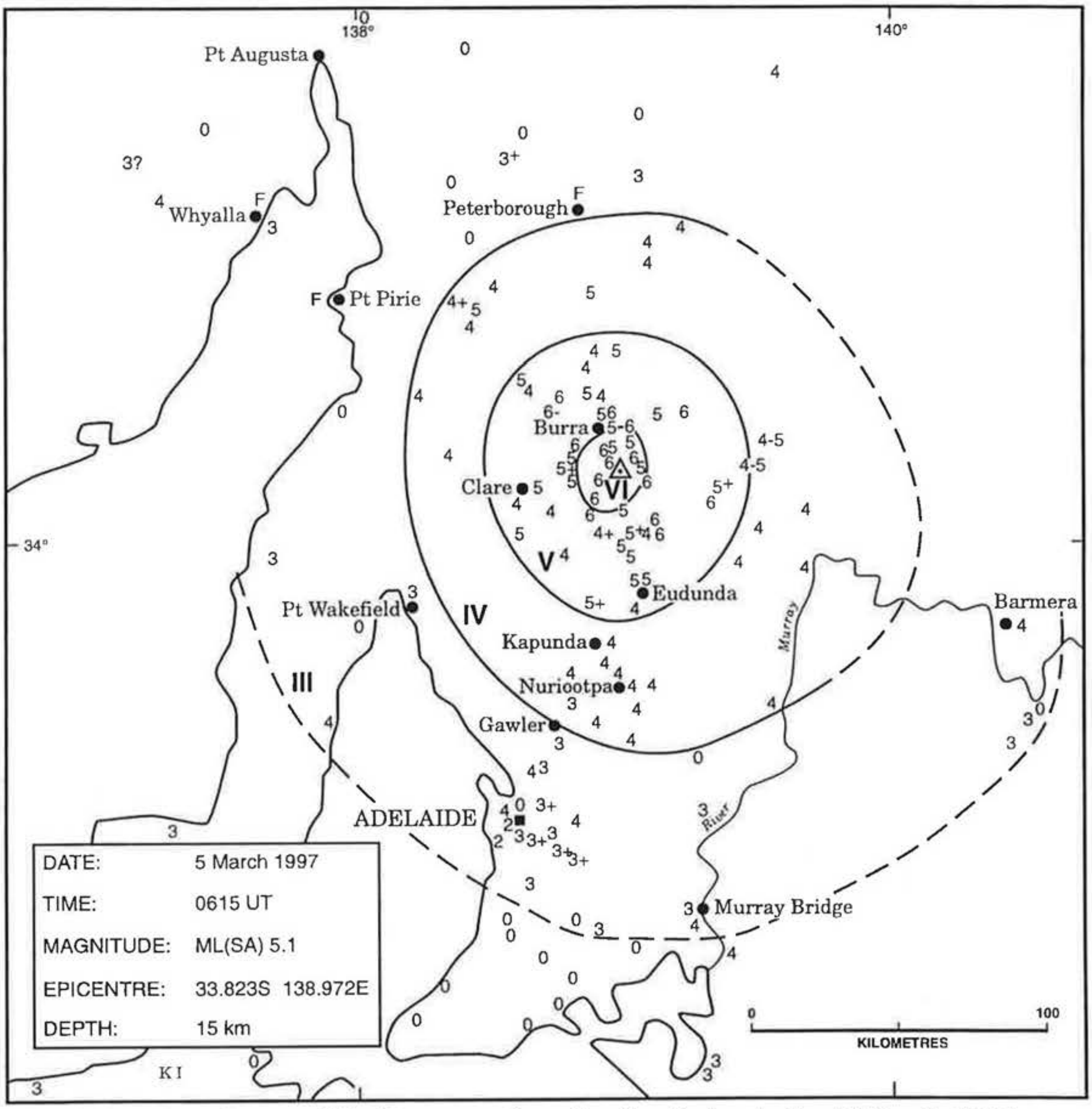

Fig. 4 Isoseismal map of the Burra earthquake, South Australia, 5 March 1997

A number of instances of movements of cornice to wall junctions of quite specific serrated appearance was found, in dwellings where ceiling cornices had been changed - original ceiling cornices had been removed, and replaced with modern plasterboard cornices, these being affixed to the old plastered wall surfaces with modern cornice adhesives. In houses where this had been done, minor racking movements that occurred as the tremor passed (but, which were insufficient to initiate cracking in the masonry walling), have been sufficient to cause delamination of the original wall plaster and/or render from the underlying wall, this junction being the "weakest link in the chain". The high strength of modern cornice adhesives has maintained the junction of the wall plaster with the cornice, with the resultant fairly deep serrated fractures at the plaster - render interface being a fresh movement induced directly by the passage of the earthquake. This aspect of building damage occurs at intensity MM5. It was seen to have occurred in houses where only some rooms had new cornices and exhibited the serrated cracking, but where the older style cornices have different fixture methods and less competent cornice adhesives did not show signs of recent movement.

There were frequent assertions made by homeowners that old cornices had separated from walls, but in virtually all instances these movements could be seen to have their origins in older movements. There was no serrated failure, probably because the older cornices had lost their adhesion to the wall before the tremor had occurred.

ISOSEISMAL MAP AND INTENSITY ATTENUATION

The results of a standard questionnaire are displayed in figure 5. In figure 6 the intensity values from the questionnaire by Love and the detailed assessments by Mountford are displayed against hypocentral distance. The detailed assessments show that most MM6 reports on the isoseismal map are probably too high, and it is doubtful if a MM6 contour is warranted. Of the three attenuation formulae displayed, the Gaull 1986 attenuation for south-east Australia is the least accurate for this earthquake.

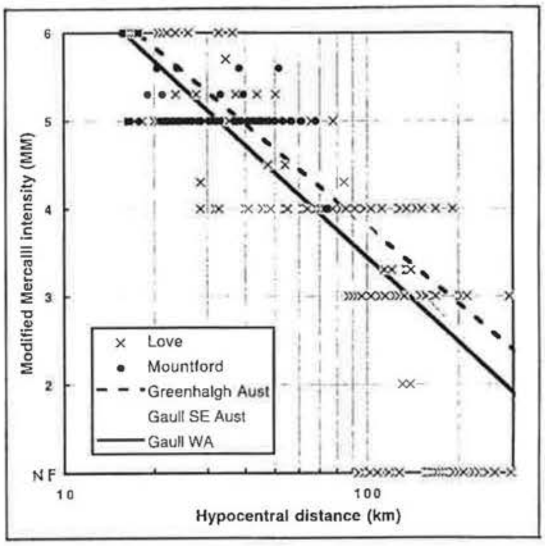

Fig. 5 Intensity attenuation

CONCLUSIONS

This earthquake was particularly interesting, producing much valuable information without being a disaster. There were no obvious cases of strong amplification near the epicentre, but spectral amplification in Adelaide was confirmed. The cost to insurance companies is estimated at $150,000 to $200,000, mainly in processing and assessment, however the large number of detailed assessments would have reduced the total cost. Strong motion readings revealed significant variations in frequency transmission, probably due to geology at depth.

ACKNOWLEDGEMENTS

We wish to thank the SA State Emergency Services for distributing part of the isoseismal questionnaire and the Seismology Research Centre, RMIT for providing assistance at the time of the event, and focal mechanism information.

THE LAKE EDGAR FAULT: EVIDENCE FOR REPEATED QUATERNARY DISPLACEMENT ON AN ACTIVE FAULT IN SOUTHWESTERN TASMANIA

RUSS VAN DISSEN[1], KEVIN MCCUE[2], GARY GIBSON[3], VAGN JENSEN[4], MALCOLM SOMERVILLE[2], BRUCE BOREHAM[5], BYRON MCKAVANAGH[5] AND ALBERT GOEDE[4]

[1]INSTITUTE OF GEOLOGICAL AND NUCLEAR SCIENCES, [2],AUSTRALIAN GEOLOGICAL SURVEY ORGANISATION, [3]SEISMOLOGY RESEARCH CENTRE, RMIT, [4]UNIVERSITY OF TASMANIA, [5]CENTRAL QUEENSLAND UNIVERSITY, ROCKHAMPTON

AUTHOR:

Russ Van Dissen has over 10 years of professional experience in earthquake geology, encompassing studies in active tectonics, fault characterisation and earthquake hazard evaluation. He has published 45 research papers and client reports, and is currently a member of the management committee of the New Zealand National Society for Earthquake Engineering.

ABSTRACT:

The Lake Edgar Fault is an active, earthquake generating fault located about 70 km inland from Tasmania's western and southern coastlines. It has a northerly trend, and dips to the west. The active trace of the fault displaces geologically young river and glacial deposits, is at least 30 km long, and appears to follow in part, a pre-existing "old" fault. The fault has ruptured the ground surface at least twice, possibly more within the Quaternary (the last ca 2 Ma); the most recent rupture probably occurred since the most recent glacial (within the last 5 ca 20,000 years). These ground ruptures are evidence of significant earthquakes on the fault. Along the central portion of the fault, the two most recent surface-faulting earthquakes have resulted in about 2.5 m of vertical displacement each (western side up). The Lake Edgar fault is considered capable of generating earthquakes in the order of magnitude 6-3/4 - 7-1/4.

1. INTRODUCTION

In the past 30 years, five of Australia's largest earthquakes have ruptured the ground surface[1,2]. These five earthquakes comprise nearly half of all known historical surface rupture earthquakes in stable continental regions throughout the globe[2]. The fault scarps produced by these earthquakes are, in some instances, quite impressive features in the landscape - extending for some tens of kilometres in length and up to several metres in height. So, long after the shaking stops, and the dust settles, these scarps provide testimony of the earthquake that was, and, in many cases, preserve information related to the earthquake's location, size, and timing.

There are over a dozen reported "recent" fault scarps in Australia[2], so there is little doubt that large pre-historic earthquakes in Australia have also ruptured the ground surface. The study of these fault scarps, and related geology, will provide critical information relevant to understanding the longer-term occurrence of large earthquakes, and associated hazard in Australia.

The purpose of this paper is to present results, to date, regarding our field studies of the pre-historic fault scarp of the Lake Edgar fault, southwestern Tasmania (Fig. 1). We first describe general features of the fault, and then present evidence that we consider documents repeated surface-faulting earthquakes on the Lake Edgar fault within the Quaternary (the last ca 2 Ma). We conclude with some speculative comments regarding possible spatial and temporal relations between surface-faulting earthquakes on the Lake Edgar fault and on a second active fault approximately 40 km to the north, the Gell River fault.

2. LAKE EDGAR FAULT

2.1 General Features

The Lake Edgar fault, located about 70 km inland from Tasmania's western and southern coastlines in the Tasmania Wilderness World Heritage Area, has a northerly trend, and dips about 60°-70° to the west[3,4]. The active trace of the Lake Edgar fault is at least 30 km long; is upthrown to the west, indicating a reverse sense of displacement; and follows, at least in part, a pre-existing "old" fault of Cambrian age[4] (ca 540 Ma).

2.2 Evidence for Repeated Quaternary Displacement

Geomorphology: Deposits and landforms from four distinct glaciations have been identified in the Lake Edgar region, and are thought to date from the latest Pleistocene (youngest glaciation, ca 20,000 years), middle Pleistocene (middle two glaciations), and early Pleistocene or older (oldest glaciation, ca 2 Ma)[5]. The Lake Edgar fault displaces till of the oldest glaciation; the scarp height across these highly weathered deposits exceeds 5 m. The fault also cuts younger alluvial fans that presumably correlate to the younger glaciations. Along the central portion of the fault, the scarp height across the youngest extensive fan, which probably relates to the youngest glaciation, is about 2.5 m. The scarp height across the next oldest fan is about 5 m. Older fans have higher scarps which indicates that the fault has generated at least two surface-faulting earthquakes within the Quaternary; the youngest being within the last ca 20,000 years.

Trench exposure: In the search for a suitable reservoir outlet for part of the Gordon River Power Project, a trench was excavated across a local topographic high. The local high was the Lake Edgar fault scarp, and the trench was located on the central portion of the fault, on the youngest extensive fan that we consider latest Pleistocene in age, or younger (≤ ca 20,000 years). More than two decades after the trench was dug, we cleaned-down its walls and were rewarded with beautiful stratigraphic evidence of at least two surface-faulting earthquakes.

Gravels, sands, and peats/lignites comprise the main units exposed in the trench (Fig. 2). The oldest unit is a dark peat/lignite which contains diagenetic quartz laminae. These laminae, which probably formed parallel to the once horizontal bedding in the peat, are now deformed and locally attain dips of 45°-80°. Near the fault, a fine white sand appears to have "intruded" into the peat/lignite. We interpret this sand unit as a sand-blow[6,7] - a liquefaction feature resulting from strong earthquake ground shaking. The peat/lignite and the fine white sand are overlain by alluvial gravel. These gravels comprise, in part, the youngest extensive fan, and are warped over the scarp which, at this locality, is about 2.5 m high (we take this to represent the amount of vertical displacement associated with the most recent surface-faulting earthquake). Two lines of evidence indicate older faulting. 1) The white sand, interpreted as a sand-blow, is truncated by, and thus older than, the overlying gravel. The shaking that resulted in the emplacement of the sand-blow must be older than both the gravel and the surface-faulting that later deformed the gravel. 2) The quartz laminae in the peat/lignite are more deformed than the overlying gravel, suggesting that the laminae were already deformed by at least one earthquake prior to the deposition, and subsequent deformation, of the younger gravel.

2.3 Earthquake Magnitude Estimate

The Lake Edgar fault is considered capable of generating earthquakes in the order of magnitude 6¾-7¼. This estimate assumes a surface rupture length and displacement of 30 km, and 2.5-3 m, respectively, and is based on empirical relations that relate surface rupture parameters with earthquake magnitude[8]. Calculations based on seismic moment considerations also yield magnitude estimates in the order of M 7.

3. THE GELL RIVER FAULT AND POSSIBLE RELATIONS WITH THE LAKE EDGAR FAULT

About 40 km north of the Lake Edgar fault lies another active fault, the Gell River fault which has a north-northeast trend and is ca 10 km long. The active trace of the Gell River fault appears to be more subdued than that of the Lake Edgar fault; this could suggest that the most recent displacement of the Gell River fault is older than that on the Lake Edgar fault. Separating the two faults are two large bodies of ultramafic rock, including serpentinite and talc. The ultramafic rocks appear to be bounded by the northern extension of the Lake Edgar fault[4]. The Gell River fault is parallel to local geological structure, though based on existing mapping, it is not clear whether it is part of the same structure that controls the location of the active trace of the Lake Edgar fault.

We speculate that the recent activity on the Lake Edgar and Gell River faults, and their geographic position relative to each other, is influenced by the occurrence of relatively weak ultramafic rocks along a major pre-existing fault. The more easily deformed talc and serpentinite between the two faults may, in essence, act to concentrate strain in the Lake Edgar and Gell River regions. This strain may be released by "continuous" deformation in the weak ultramafic rocks, but to the north and south, where the rocks are presumably stronger, strain is released episodically as earthquake rupture along the Gell River and Lake Edgar faults respectively.

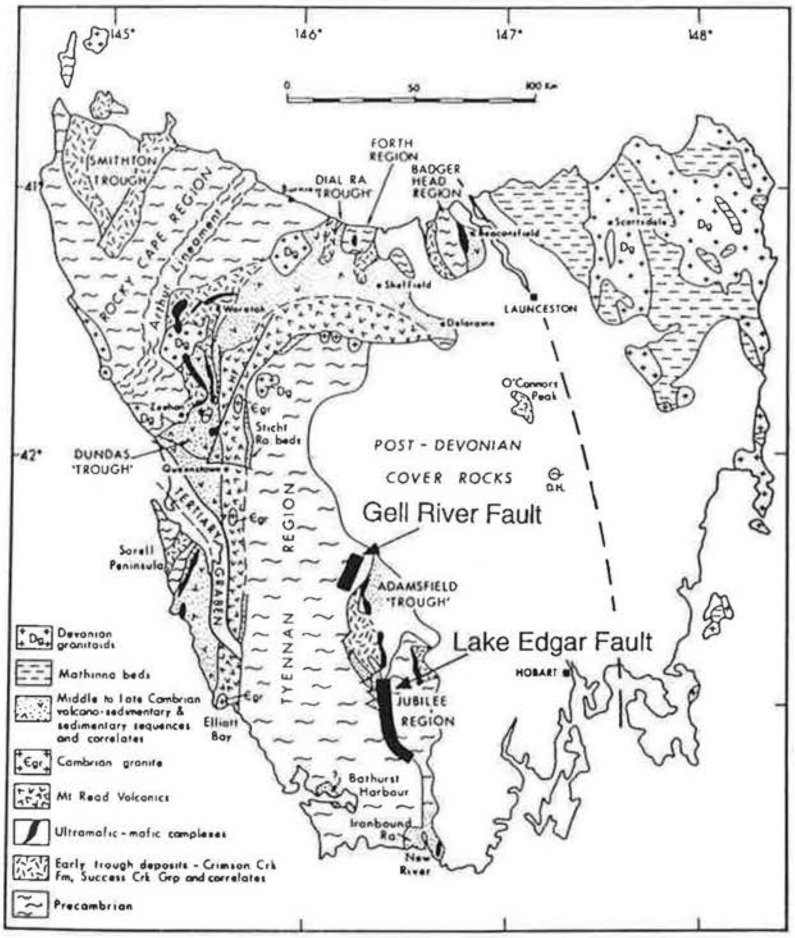

Figure 1. Simplified geological map(4) showing the distribution of major early Palaeozoic tectonic elements of western Tasmania, and the location of the active Lake Edgar and Gell River faults.

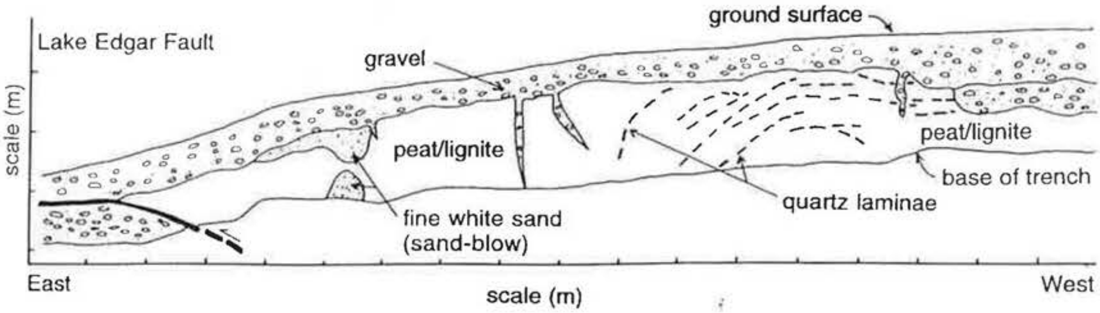

Figure 2. Simplified log of trench excavated across active trace of Lake Edgar fault.

4. CONCLUSIONS

The Lake Edgar fault has generated at least two surface-faulting earthquakes within the last ca 2 Ma, the most recent of these being younger than ca 20,000 years. These earthquake were probably in the order of M 6¾-7¼ in size, and produced, along the central portion of the fault, at least 2.5 m of vertical displacement. The spatial relationship between the Lake Edgar and Gell River faults, and temporal relationships between surface-faulting earthquakes on these two faults, are probably influenced by one, or both, of the following: 1) the existence of a pre-existing fault which is oriented favourably, both with respect to strike and dip, for movement in the contemporary stress field; and 2) the existence of large bodies of ultramafic rocks, including easily deformed talc and serpentinite, along the pre-existing fault which may concentrate strain in the Lake Edgar and Gell River areas.

5. ACKNOWLEDGMENTS

We thank Des Darby, Kathleen Hodgkinson, and Mark Rattenbury (all from the Institute of Geological & Nuclear Sciences) for their reviews, and consequent improvement of this manuscript. We also thank Dr Hodgkinson for her fruitful input towards the modelling of possible strain concentration in the Lake Edgar region. This study benefited from discussions with numerous people; we would specifically like to acknowledge those with Glyn Roberts (formerly of the Hydro-Electric Corporation), Tom Laudon (University of Wisconsin), and Ruth and David Wilson (Hydro-Electric Corporation). We also wish to thank the Hydro-Electric Corporation, Tasmanian Parks and Wildlife Service, and the then Department of Industry, Technology and Commerce for their logistical and/or financial support of this project. This study was, in part, commissioned by the Hydro-Electric Corporation as part of an ongoing program of assessing the seismic risk for their dams.

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CONTEMPORARY ASSESSMENT OF TSUNAMI RISK AND IMPLICATIONS FOR EARLY WARNINGS FOR AUSTRALIA AND ITS ISLAND TERRITORIES

J. DAVIDSON[1] AND DR. J. RYNN[2]

[1]COMMONWEALTH BUREAU OF METEOROLOGY, BRISBANE, QUEENSLAND
[2]CENTRE FOR EARTHQUAKE RESEARCH IN AUSTRALIA, BRISBANE, QUEENSLAND

AUTHOR:

Jim Davidson has a B.Sc and Graduate Diplomas in Meteorology and Management. He is employed by the Commonwealth Bureau of Meteorology. Currently he is Supervising Meteorologist of the Queensland Forecast and Warning Cente in Brisbane.

ABSTRACT:

An Australian IDNDR project undertook a contemporary (post-European settlement in 1788) assessment of tsunami risk. A specific methodology was developed in terms of the definition

TSUNAMI RISK = HAZARD X VULNERABILITY

This involved a multidisciplinary approach with multi-institutional collaboration both in Australia, and internationally through NOAA and the Pacific Tsunami Warning Centre (PTWC) in the USA and the International Tsunami Commission (ITC). Reference was made to the Australian IDNDR projects on earthquake zonation and storm surge, NOAA tsunami studies and a broad literature survey.

Paper No. 6

CONTEMPORARY ASSESSMENT OF TSUNAMI RISK AND IMPLICATIONS FOR EARLY WARNINGS FOR AUSTRALIA AND ITS ISLAND TERRITORIES

J. DAVIDSON[1] and DR J. RYNN[2]

[1]Commonwealth Bureau of Meteorology, Brisbane, Queensland, Australia
[2]Centre for Earthquake Research in Australia, Brisbane, Queensland, Australia

The International Decade for Natural Disaster Reduction 1990-2000 (IDNDR) has identified the need to seriously consider the risk of tsunami for those countries bordering the major oceans and seas of the world. Recent catastrophic tsunamis have emphasised this need. For Australia and its island territories, which are surrounded by the Pacific, Indian and Southern Oceans, little has been known about the tsunami hazard - although several major tsunamis have impacted their shores. Recent studies have sought to quantify the tsunami risk for Australia and so provide an information resource for the proposed Australian tsunami warning system and in emergency services responses.

An Australian IDNDR project undertook a contemporary (post-European settlement in 1788) assessment of tsunami risk. A specific methodology was developed in terms of the definition **TSUNAMI RISK = HAZARD X VULNERABILITY.**

This involved a multidisciplinary approach with multi-institutional collaboration both in Australia, and internationally through NOAA and the Pacific Tsunami Warning Centre (PTWC) in the USA and the International Tsunami Commission (ITC). Reference was made to the Australian IDNDR projects on earthquake zonation and storm surge, NOAA tsunami studies and a broad literature survey.

The quantification of HAZARD is based primarily on actual observational data by analyses of both instrumental tide-gauge records and anecdotal information. The data set was compiled from known tsunami occurrences and supplemented by the systematic search of tide-gauge records for more than 350 additional possible tsunamigenic sources (earthquakes, volcanos, landslides) from global and Australian catalogues. Analyses of these data determined tsunami parameters and their empirical relationships. A comprehensive catalogue was compiled. While computer modelling of specific parameters was not a premise of this project, previously documented results therefrom (such as PTWC tsunami travel time charts) were included. VULNERABILITY has been delineated in terms of the coastal and island built environments and human resources at potential risk. This also includes offshore petroleum and gas facilities.

Integration of these hazard and vulnerability results provided the quantitative and qualitative RISK assessment in terms of the tsunami zonation map and commentary. Potential tsunamigenic sources were identified and travel time charts obtained for each source. It was evident from this pilot that various aspects require more detailed analyses and recommendations for future projects are given. A "Source Document on Tsunami Risk" was thus prepared to provide information for mitigation planning by emergency response agencies and Bureau of Meteorology procedures for tsunami warnings.

EARTHQUAKES AND THE COMMUNITY

JOHN M.W. RYNN AND PETER R. HUGHES
CENTRE FOR EARTHQUAKE RESEARCH IN AUSTRALIA

AUTHOR:

ABSTRACT:

During this current decade, community awareness as to the peril of earthquakes has been on the increase - a clear result of recent disastrous earthquakes (e.g. 1994 Northridge, USA; 1995 Kobe, Japan) and then implementation of the United Nations IDNDR program. This has occurred in both known earthquake prone areas along the plate margins and in low seismicity regions like the continents. In Australia, the needs were starkly realised with the 1989 Newcastle earthquake. Although earthquakes are rare herein, the consequences for potentially severe disruption to the community are becoming well-recognised. The key to reducing community losses is preparedness through the earthquake mitigation process. This demonstrates a multidisciplinary approach with multiagency cooperation. Earthquake mitigation is the understanding of the risk through its elements of hazard and vulnerability and the practical application of the information resources available from the sciences and humanities. Caution must be exercised in such application by recognising the limitations and uncertainties in the earthquake parameters, engineering relationships and theoretical modelling. A series of case studies illustrating the essential elements of earthquake mitigation and application for community needs in Australia (urban areas of Sydney, Newcastle, Southeast Queensland, Melbourne; rural areas of Cessnock and Toowoomba) and Fiji (Suva City) will be presented. These represent the practical application of earth science, engineering and socio-economics studies through integration with emergency services authorities and community service organisations.

Paper No. 7

EARTHQUAKES AND DAMS IN AUSTRALIA

GARY GIBSON, SEISMOLOGY RESEARCH CENTRE

AUTHOR:

Gary wrote his first earthquake location computer program in 1971, and has been trying to learn more about earthquakes since then. He is particularly interested in earthquake hazard analysis, seismic instrumentation and the relationship between earthquakes and dams.

ABSTRACT:

Large new reservoirs can trigger earthquakes. This is due to either a change in the stress because of the weight of water, or more commonly due to weakening of fractures and faults under the reservoir by increased water pore pressure. The energy released in a reservoir induced earthquake is normal tectonic strain energy, released prematurely. Study on induced earthquakes provides useful insight into the mechanism of earthquakes in general, because of the effects of pore pressure and because they are likely to be well recorded.

Several Australian reservoirs have experienced reservoir induced seismicity (Talbingo, Thomson and Pindarri), and there are several cases of probable induced seismicity (Eucumbene, Warragamba, Gordon and Argyle). The proportion of reservoirs which experience induced seismicity in Australia is much higher than the world average.

Paper No. 8

EARTHQUAKES AND DAMS IN AUSTRALIA

Gary Gibson, Seismology Research Centre, RMIT, Melbourne

1. EARTHQUAKES AND THE EARTHQUAKE CYCLE

An earthquake is the motion produced when stress within the earth builds up over a long period until it exceeds the strength of the rock, which then fails by breaking along a fault. Earthquake motion includes both transient vibrations and permanent rupture displacement.

The stress is associated with strain, or deformation, and it is convenient to think of the earthquake in terms of strain energy. It may take tens or thousands of years for the elastic strain energy to accumulate, and most of this energy may be released in seconds.

The earthquake cycle at a location may be considered in the following phases:

- quiescence, a period of build up of strain energy that may take tens of years in tectonically active regions, or very much longer in areas of low tectonic movement.
- precursory activity, with increasing seismicity in the highly stressed rocks, perhaps over a period of months or years. These events are sometimes called pre-shocks.
- foreshocks may occur minutes, hours or days before the major energy release.
- a main shock is the largest event in the cycle. In some cases almost all energy is released in a single main shock, or there may be multiple large shocks, or the energy may be released in a swarm of small events.
- aftershocks occur during the days following the main shock in numbers varying from few to many. The difference in magnitude between the mainshock and the largest aftershock can vary widely.
- adjustment activity occurs within the area for years after the main shock. This may occur at increasing distances from the main shock, and decreasing rates with time. It decays much slower than the aftershock activity.
- the next quiescence begins, lasting much longer than the total of the other periods.

The magnitude of an earthquake depends primarily on the volume of rock that is under stress, or on the area of fault plane that ruptures in the earthquake. The area of rupture in an earthquake of magnitude 4 is about 1 km^2 (1 x 1 km), magnitude 5 is 10 km^2 (3 x 3 km), magnitude 6 is 100 km^2 (10 x 10 km) and so on. The size of the stress volume or fault rupture area considered in a seismic cycle is not clearly defined, and for a particular point must be considered on a range of scales. The granularity of the volume must account for the observed variation in earthquake magnitudes.

The earthquake activity at a particular location is complicated by the surrounding area. The stress will be affected by surrounding earthquake activity. The average period for the earthquake cycle at particular locations will vary depending on local geological conditions. The effect of activity in the surrounding area will introduce considerable scatter into the actual interval between main shocks, so the earthquake cycle will not display periodicity. If adjacent areas approach high stress together, a single large main shock may result.

Reservoirs may induce, or trigger, seismicity by two mechanisms. Either the weight of the water may change the stress field under the reservoir, or the increased ground water pore pressure may decrease the stress required to cause an earthquake. In either case, **reservoir induced seismicity** (RIS) will only occur if the area under the reservoir is in the precursory activity phase of the cycle, or perhaps late in the quiescence phase. If the stress has been relieved by a recent large earthquake, say in the last few hundred years for low seismicity areas like Australia, then RIS is unlikely to occur.

2. STRESS DIRECTION AND WATER WEIGHT

Tectonic stress is considered in three orthogonal principal stress directions, ranked as maximum, middle and minimum. These may be oriented in any direction, but geological processes and the free surface of the earth often constrain one or other to be near vertical.

It is the difference between maximum and minimum principal stress that causes an earthquake. The fault plane will usually be at right angles to the plane containing the maximum and minimum principal stresses, and at an angle less than 45° (often 30° to 35°) to the maximum principal stress, depending on friction properties of the fault material.

If the minimum principal stress is vertical then horizontal compression gives reverse faulting. If the middle principal stress is vertical then, with both minimum and maximum stresses horizontal, strike slip faulting results. If the maximum principal stress is vertical, the resulting normal fault motion is equivalent to that from horizontal tension.

The effect of the weight of water on the stress field about the reservoir is quite complex. There will be compression under the reservoir with an increase in the vertical principal stress. The magnitude and direction of the change in stress in the area around the reservoir can be estimated using finite element methods or other techniques. The changes decrease rapidly with distance from the reservoir.

For reverse faulting the stress change increases the minimum principal stress under the reservoir, reduces the difference between maximum and minimum, and will tend to repress an imminent earthquake. This has been called reservoir induced aseismicity. In some cases earthquakes could then be induced by later releasing water from the reservoir.

For strike-slip faulting, the weight of the water will primarily affect the middle principal stress, so should neither repress nor trigger activity.

For normal faulting the weight of water will increase the vertical maximum principal stress, the difference between maximum and minimum may exceed the strength of the rocks, and a reservoir induced earthquake may be triggered.

It was originally thought that RIS occurs with normal faulting or strike-slip faulting, but rarely with reverse faulting. This is contrary to Australian experience, where the clear cases of RIS (Talbingo, Thomson and Pindari) and all cases of suspected RIS (Eucumbene, Warragamba, Gordon, and Argyle) are in areas where reverse faulting dominates.

3. GROUND WATER PORE PRESSURE

Ground water plays a large part in earthquake activity. Fluid injection into wells in USA, Japan and elsewhere has triggered earthquakes. Water pore pressure reduces the normal stress within a rock while not changing the shear stress. Increased pore pressure is due to:

1. the decrease in pore volume caused by compaction under the weight of a reservoir.
2. diffusion of reservoir water through porous rock under the reservoir. The rate of flow depends on the permeability of the rock, so this effect is not instantaneous but takes increasing time with distance from the reservoir. It may take years for the pore pressure to increase at depths of kilometres beneath a reservoir.

These occur under any reservoir, whether there is reverse, strike-slip or normal faulting. The first occurs near instantly, while the second is delayed depending on permeability. Any increase in water pore pressure means that a failure is more likely. The critical value of the shearing stress may be made arbitrarily low by increasing the pore pressure. It is now believed that for RIS, increased pore pressure is more important than stress changes.

4. CHARACTERISTICS OF RESERVOIR INDUCED EARTHQUAKES

If there is a **major fault** near the reservoir, RIS can produce earthquakes exceeding magnitude 6.0 (Xinfengjiang, China, 1962, M 6.1; Koyna, India, 1967, M 6.3). Several reservoirs have triggered earthquakes exceeding magnitude 5.0 (Eucumbene, 1959, M 5.0; Warragamba, 1973, M 5.4; Thomson, 1996, M 5.0). A larger magnitude RIS event only occurs if there is an existing nearby fault of sufficient dimension that is late in its earthquake cycle (the stress is already approaching the strength of the fault).

A **series of small shallow earthquakes** is a more common form of RIS (Talbingo, 1973 to 1975; Thomson, 1986 to 1995). These events possibly occur on joints rather than established faults, so are limited in size, with magnitudes up to ML 3 or 4. There is little or no hazard from such small earthquakes, even if they occur in large numbers. They are much smaller than the maximum credible magnitude for normal tectonic earthquakes, which may vary from 6.0 to over 8.0 depending on the local geological situation. It seems that RIS with many small events is more likely in areas with near-surface jointed crystalline rocks like granite, rather than sedimentary rocks.

RIS has been observed for over 100 reservoirs throughout the world, and small shallow induced events have probably occurred under many others. With limited knowledge of background seismicity, in space or in time, it can be very difficult to determine whether earthquakes near a reservoir have been induced. A high proportion of reservoirs with RIS seismograph networks record such activity.

A high proportion of RIS occurs in low-seismicity intraplate areas, especially regions that are close to an active plate boundary. Above average rates of RIS are found in China, Australia, Africa, Brazil and India.

The **onset** of RIS may be **immediate** after filling **or delayed** a few years, depending on whether it is due to stress or pore pressure. Any delay depends on the permeability of the rock. All large or moderate magnitude induced earthquakes have occurred within ten years of the first filling of the reservoir. Aswan Reservoir began filling in 1964, filled in 1975, and experienced a relatively deep induced earthquake in 1981. It is unlikely that reservoir induced seismicity will start beyond twenty years after first filling.

RIS activity usually **decays** within a few years of first occurrence. As years pass after first filling, pore pressure increases permeate to greater depths and distances, and the events may occur further from the reservoir.

In most cases RIS is a **transient** phenomenon which will cease once the stress and pore pressure fields have stabilised at new values. Earthquake hazard will then revert to similar levels that would have existed if the reservoir had not been filled. The Koyna earthquakes in India have continued for over 30 years. Even for those reservoirs showing a correlation between earthquakes and water level, RIS probably does not continue indefinitely.

The **depth** of RIS events, especially those occurring immediately after filling, is usually very shallow under or near the reservoir, within a few kilometres of the surface. Their shallow depth means that they may often be felt or heard. Induced earthquakes at reservoirs experiencing delayed triggering may be deeper, perhaps ten to fifteen km. These may occur from ten to twenty years after filling of the reservoir. Examples include Aswan in Egypt, Thomson in Victoria and possibly Warragamba in NSW. These suggest a diffusion rate of something like one kilometre per year.

It is not easy to **predict** whether a future reservoir will experience RIS, because the state of stress and the rock strength at earthquake depths are not easily measured. For the same reason, prediction of normal non-RIS earthquakes has usually been unsuccessful.

5. RESERVOIR INDUCED SEISMICITY IN AUSTRALIA

Eucumbene Reservoir began filling in 1958, with a capacity of 4.8 km^3 and height of 116 m. An earthquake of magnitude **ML 5.0** occurred about 10 kilometres south of the dam on 1959 May 18. Over 270 events occurred between Eucumbene Reservoir and Jindabyne Reservoir over the next 40 years.

Warragamba Dam was completed in 1960, with a capacity of 2.06 km^3 and height of 137 m. The dam is about 3 km west of the Lapstone Fault outcrop, and the reservoir extends to about 30 km further west. This is probably a reverse fault, dipping to the west under the reservoir at about 35°. On 1973 March 9, an earthquake of magnitude **ML 5.5** occurred at the south end of the reservoir, about 18 km west of the Lapstone Fault at a poorly constrained depth possibly about 12 km. It was followed by over 300 aftershocks.

Talbingo Dam was completed in 1971 with a capacity of 0.92 km^3 and depth of 162 m. Over 200 small shallow events occurred under the reservoir over the next four years, the largest of magnitude **ML 3.5** in 1973. Many of these were felt or heard in the town. This is regarded as a classic example of a swarm of shallow induced earthquakes.

Lake Argyle in the north of Western Australia was completed in 1971. It has a capacity of 5.72 km^3 and a depth of 99 m. To 1984, about 30 events occurred under or near the reservoir from about ML 2.0 to **ML 3.5**, well above the activity of the surrounding area. Seismograph coverage is poor, both before and since impounding. A seismograph installed nearby from 1995 to 1996 recorded several small events under the reservoir.

Lake Gordon in Tasmania began filling in 1972. This reservoir and Lake Pedder are connected by a short canal, and have a total capacity of 13.5 km^3 with a maximum depth of 140 m at Gordon Dam. A three to four-fold increase in earthquake activity under the two lakes was detected from July 1974. The increased level of activity ended about 1984 after some 120 events, the largest being only of magnitude **ML 2.5**.

Thomson Reservoir began filling in July 1983, with a capacity of 1.1 km^3 and depth of 166 m. It is about 25 km northwest of the outcrop of the Yallourn Fault. A swarm of very small events occurred at a depth of 11 km under the reservoir in November 1983. There was no significant activity until February 1986, when a series of events began under the reservoir at depths to about 3 km. For the next couple of years there were about 2 to 5 events per week up to ML 2.5. Activity from 1988 to 1996 spread away from the reservoir, north, south and deeper, with maximum magnitude of ML 3.1, and at a decreasing rate. An earthquake of **ML 5.0** occurred on the Yallourn Fault on 1996 September 25, with epicentre about 2 km from the dam, at a depth of 11 km, and at the nearest slant distance from the reservoir to the fault.

Pindari Dam in northeast NSW was built in 1969, doubled in height to 85 m in 1994, and began to re-fill in early 1995. In March 1995, a swarm of over 30 very shallow earthquakes occurred under the reservoir. The largest was of magnitude **ML 2.3** on March 27, within 3 km of the seismograph at the dam, giving a peak acceleration of 0.09 g. None of these events was reported felt. Further activity occurred in the Pindari area over the next two years.

6. CONCLUSIONS

Reservoir induced earthquakes are normal earthquakes that have been triggered prematurely. Their study provides useful insight into the mechanism of earthquakes in general, because of the effects of pore pressure and because they are likely to be well recorded. Relating induced seismicity to the earthquake cycle suggests that the problems of predicting induced events are similar to those for predicting normal earthquakes.

EARTHQUAKE GROUND MOTION MODELLING FOR AUSTRALIAN BEDROCK CONDITIONS

NELSON LAM, JOHN WILSON, AND GRAHAM HUTCHINSON

AUTHOR:

ABSTRACT:

This paper reviews research that has been carried out in the past by both seismologists and engineers to derive the elastic response spectrum for rock sites in interplate and intraplate regions. Factors influencing the frequency content of the bedrock excitations are identified, and response spectra suitable for Australia are derived based on the latest information available and compared with the current provisions. Problems in specifying seismic hazard for rock sites in Australia are discussed with recommendations.

Paper No. 9

EARTHQUAKE GROUND MOTION MODELLING FOR AUSTRALIAN BEDROCK CONDITIONS

NELSON LAM[1], JOHN WILSON[1] and GRAHAM HUTCHINSON[1]

1. INTRODUCTION

Although most earthquakes occur along tectonic plate boundaries, destructive intraplate earthquakes can occur in unexpected areas. Earthquakes exceeding magnitude 6 occur on average once every five years in Australia which is completely within the Indo-Australian plate.

Like many earthquake loading standards and codes of practices around the world, the current Australian Earthquake Loading Standard AS1170.4[1] has been based on the Uniform Building Code[2](UBC) from the United States. However, the code provisions have principally been derived from empirical information and experience in California.

Research into intraplate earthquake hazard in recent years has focused on the frequency and duration characteristics of the earthquake ground motion. Studies have also been carried out reviewing both the intraplate site responses and the behaviour of limited ductility structures. These studies have typically been carried out within the conventional boundaries dividing the seismological and engineering disciplines.

The following extended abstract briefly reviews research progress and highlights current challenges in the following areas of earthquake engineering : (i) **Bedrock motion modelling** (ii) **Seismic hazard modelling**.

2. BEDROCK MOTION MODELLING

The design response spectra specified by many earthquake loading standards is based on, or related to, the well known response spectrum model developed by Newmark-Hall[3]. Peak ground velocity (PGV) is often used to scale the spectrum to take into account the seismicity of the site. However, the frequency content represented by this spectrum does not vary to reflect regional variations in the seismicity and the attenuation characteristics of rock.

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[1] Dept. of Civil & Environmental Engineering, The University of Melbourne, Parkville, Vic 3052, Australia.

In contrast, the **Uniform Hazard Spectra** (UHS) introduced recently in the United States by Algermissen & Leyendecker[4] reflect differences in the average frequency content of earthquakes across the country. However, the probabilistic response spectra cannot distinguish the frequency content of a small earthquake at close range from that of a larger and more distant earthquake. The effect this bedrock motion frequency content has upon site amplifications can be quite significant.

Empirical models have been used extensively in interplate areas such as the Western United States to model the frequency content of earthquakes. However, such models cannot be developed easily in intraplate areas due to the paucity of near field strong motion data. Semi-empirical methods such as the use of **Empirical Green Functions** can be employed to model the waveform of future earthquakes. The model is valid for a particular faulting arrangement and geological conditions surrounding the transmission path of the earthquake waves. However, many major intraplate earthquakes occured in unexpected areas and on previously unknown faults.

A **Geophysical Model** has been developed in the United States since the early 1980's to predict the frequency content of an earthquake based on only the moment magnitude, focal distance, stress drop and attenuation of waves through rock. Intraplate earthquakes have been distinguished from interplate earthquakes by assuming a higher stress drop associated with the characteristics of the reverse faulting mechanism. The geophysical model has been modified recently by Atkinson[5] to match field measurements taken during major intraplate earthquake events that have occured in recent years in the intraplate regions of Middle and Eastern North America.

However, applying this model to Australia, for example, has the following difficulties:
(i) Existing local seismicity data is expressed in terms of the Local Magnitude, instead of the Moment Magnitude.
(ii) The expected stress-drop, which depends on the size of the earthquake, is still uncertain due to lack of local data.
(iii) The directivity effect has not been taken into account in the model.

Once these problems have been overcome, then the design properties associated with intraplate earthquakes will be able to be predicted with reasonable accuracy and reliability.

3. SEISMIC HAZARD MODELLING

The seismic hazard of a site is normally quantified in terms of a probabilistic ground motion parameter (eg. peak ground velocity) which is determined by integrating contributions from all surrounding earthquake sources, based on the predicted level of activity in each source zone and the attenuation function in the region. Although the parameter is easy to interpret for engineering applications, the amount of computation and editing required to produce and revise seismic hazard maps can be costly and time consuming. Further, seismic hazard maps need frequent updating in intraplate areas.

With the **Geophysical Model**, the design earthquake ground motion can be predicted readily based on combinations of magnitude and epicentral distance for a given return period assuming the spatial distribution of earthquakes is random. For a given probability of occurrence, the larger the epicentral distance, the larger the magnitude of the earthquake since the source area is larger. Preliminary analysis shows that the 1 in 500 years seismic hazard of major coastal cities in Australia is dominated by a Magnitude 5 to 6 event at an epicentral distance of about 30 to 50km. Earthquakes within 20km epicentral distance generally have a small probability of exceeding Magnitude 5. On the other hand, the maximum credible earthquake is around 7 [6].

4. CONCLUSION

(i) A geophysical model has been developed to predict the frequency content of the bedrock excitation based on the moment magnitude, epicentral distance and stress-drop of the earthquake. However, further research is required to determine suitable moment magnitude and stress drop for input into the model for Australian conditions.

(ii)With the geophysical model, the design earthquake ground motion can be predicted readily based on combinations of magnitude and epicentral distance for a given return period. The seismic hazard level of a site can be represented by such combinations.

5. ACKNOWLEDGEMENT

Financial support by the Australian Research Council is gratefully acknowledged.

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SEISMIC GROUND RESPONSE ANALYSIS FOR PORT MELBOURNE

SRI T. SRITHAR,
GOLDER ASSOCIATES PTY LTD.

AUTHOR:

The author's experience in seismic design considerations include involvements in the development of a seismic design procedure for major structures, seismic retrofit of an LNG storage facility and seismic upgrade of major bridges in B.C. Canada, and seismic ground amplification and liquefaction assessment of a number of sites.

ABSTRACT:

A seismic site amplification study was carried for a typical soil profile in Port Melbourne. The soil profile considered consists of very loose to medium dense sand and soft to firm clay deposits to a depth of about 15 m, overlying stiff to very stiff clays and dense to very dense sands and gravels. Siltstone bed rock at the site is expected to be at a depth of about 40m.

The results of the seismic response analysis are discussed in terms of amplification of acceleration through the soil profile, surface acceleration levels, fundamental period of the soil profile and acceleration response spectra at the surface. The results are compared with the general recommendations given in Australian Standard AS 1170.4-1993, where appropriate. The liquefaction potential of the loose sand deposits and its general effects are also discussed.

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1.0 INTRODUCTION

Amplification of ground motions on deep soil sites can cause severe structural damages. Notable examples of this are the 1985 Mexico earthquake, the 1988 Armenian earthquake, and the 1989 Loma Prieta earthquake in California. A seismic site amplification study was carried out for a typical soil profile in Port Melbourne area. The purpose of the study was to provide a preliminary assessment regarding the necessity for a site specific seismic ground response analysis in view of the general guidelines given in the Australian Standard AS 1170.4-1993.

2.0 GROUND RESPONSE ANALYSIS

2.1 Method of Analysis

The program SHAKE[1] was used to carry out the seismic ground response analysis of the site. SHAKE calculates the seismic site response based on vertical propagation of the shear waves through a one dimensional soil column. The nonlinear and hysteretic stress-strain behaviour of the soil is modelled as linear visco-elastic using strain dependent moduli and damping. Equivalence is achieved by an iterative procedure such that the moduli and damping values used are compatible with computed strains.

2.2 Soil Profile and Input Parameters

A typical soil profile in the Webb Dock area[2] in Port Melbourne was considered for the seismic ground response analysis. The soil profile consists of fill at the surface to about 2 m depth, underlain by very loose to medium dense sands (Port Melbourne Sands) to about 7 m depth, underlain by soft to firm clay deposits (Coode Island Silt) to about 15 m depth, underlain by stiff to very stiff clays (Fishermens Bend Silt) to about 19.5 m depth, underlain by medium dense to very dense sands and gravels (Moray Street Gravels and Tertiary age deposits). Siltstone bed rock at the site is expected to be at a depth of about 40 m. The soil profile and the assumed maximum (small-strain) shear modulus values based on the available geotechnical information are shown in Figure 1.

The variation of shear modulus and damping values with shear strain considered in the analyses were based on Seed et al.[3] for sands and Idriss[4] for clays.

2.3 Earthquake Records

Analysis of seismic data[5] indicates that most of the Victorian earthquakes occur due to compression from south-east to north-west, with movement on reverse fault striking at right angles. However, in the absence of an actual acceleration-time history of a moderate earthquake in Victoria, the following Loma Prieta earthquake records were used in the analyses:

- EQ 1 - Gilroy, San Ysidro, peak ground acceleration = 0.169 g
- EQ 2 - San Francisco, Diamond Heights, peak ground acceleration = 0.090 g
- EQ 2 - San Francisco, Rincon Hill, peak ground acceleration = 0.113 g

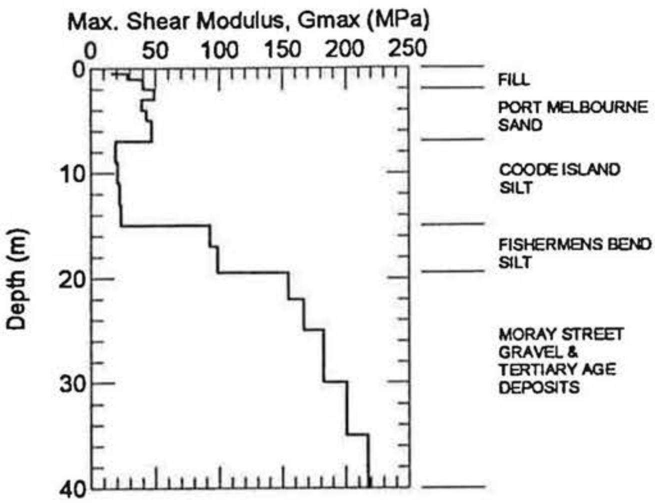

Figure 1. Subsoil and Maximum Shear Modulus Profile

The acceleration coefficient map for Victoria presented in the Australian Standard AS 1170.4-1993 indicates an acceleration coefficient of 0.09 g to 0.1 g for the site. Hence, the selected earthquake records were scaled to a peak acceleration of 0.1 g and given as input motions at the bed rock level in the analyses.

2.4 Results

The results in terms of maximum acceleration with depth for the earthquake records considered are shown in Figure 2. Maximum accelerations in the range of 0.20 g to 0.25 g are predicted at the surface. This implies an site amplification factor of 2 to 2.5. The site factor presented in the Australian Standard AS 1170.4-1993, which is an indicator of the site amplification, suggest a site factor of 2 for this site.

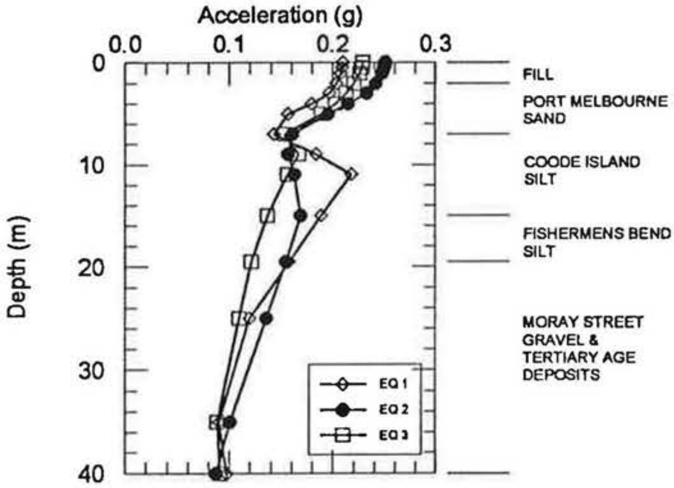

Figure 2. Variation of Maximum Acceleration with depth

The peak ground accelerations assumed at the rock surface and calculated at the ground surface are shown in Figure 3. It can be seen that the accelerations calculated at the surface are within the range of site observations made during the 1989 Loma Prieta earthquake and slightly higher than the median relationship proposed by Idriss[4].

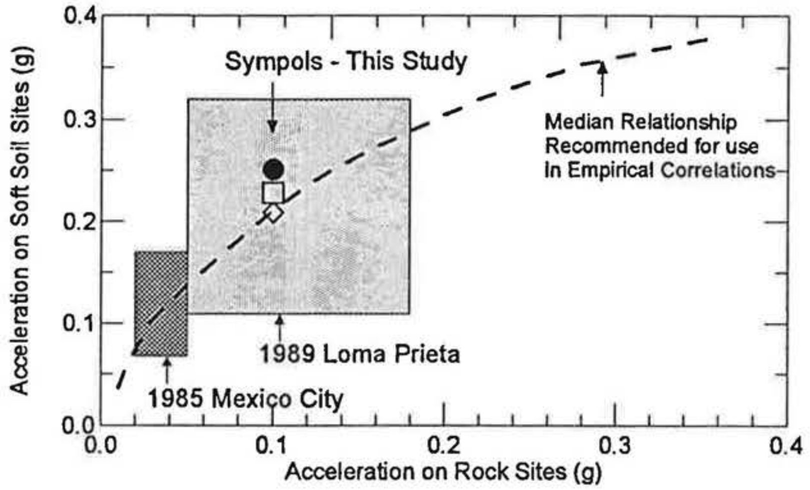

Figure 3. Variation of Accelerations on Soft Soil Sites vs Rock Sites

The normalised acceleration response spectra (for 5% damping) of the input motion at the rock surface and of the calculated motion at the ground surface are shown in Figures 4 and 5. The acceleration values in these plots have been normalised to the assumed acceleration coefficient of 0.1 g. Also shown in the figures are the design response spectra given in Australian Standard AS 1170.4-1993 for site factors of 1.5 and 2.

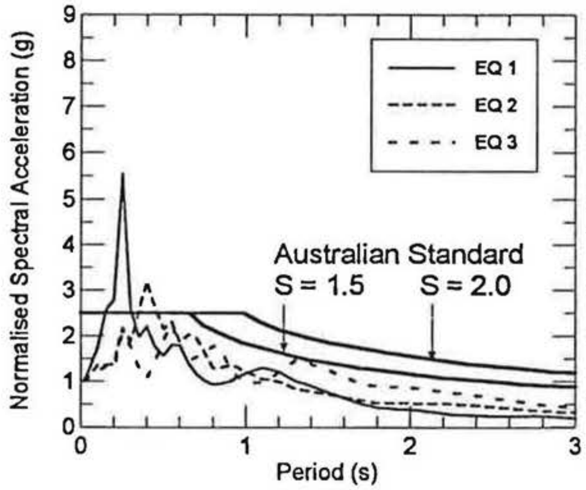

Figure 4. Acceleration Response Spectra of the Input Motions

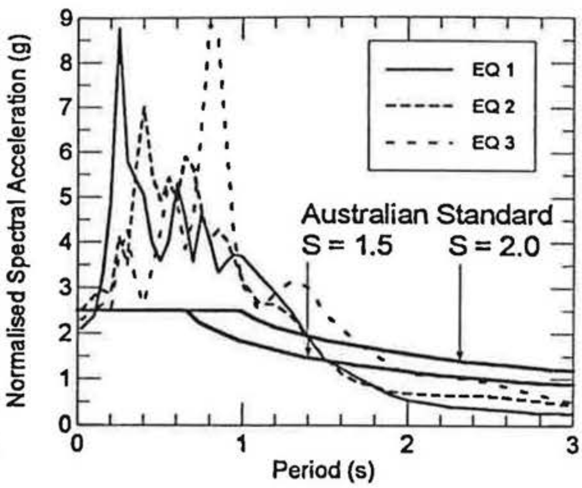

Figure 5. Acceleration Response Spectra of Calculated Motions at Ground Surface

The spectral acceleration values of the ground motion calculated at the ground surface generally indicate higher values than the design spectra for periods less than about 1.4 s. The results of the analysis indicate that the natural period of the site is about 0.8 s. The acceleration spectra show higher amplifications around that period.

3.0 DISCUSSION

It should be noted that the results of the ground response analysis will depend on the assumed input earthquake motions, soil properties, and the modulus and damping variation with strain. Especially the acceleration response spectra will depend on the frequency content of the input earthquake motion.

Liquefaction in Port Melbourne Sand

The potential for liquefaction in the Port Melbourne Sands has been assessed based on Seed's[6] procedure and using the results of the SHAKE analyses. The assessment indicates that a normalised Standard Penetration Test (SPT) blow count, $(N_1)_{60}$ of more than 16 will be required for no liquefaction.

The uncorrected SPT blow counts in Port Melbourne Sands varies from less than 1 up to about 10 to 20[2]. At the particular site considered for the study, the measured normalised SPT blow counts vary from less than 1 to 14 indicating a high potential for liquefaction.

If liquefaction of the sands occurs at the site, most buildings on shallow footings will suffer some damage as a result of uneven settlement due to liquefaction. The layer of fill above the sand is not expected to liquefy but it may crack and sand boils can appear on the surface. Severe damage will result where the cracks in the ground occur beneath the buildings. Taller buildings which are generally supported on piles may suffer damage if the soil surrounding the pile liquefies and if the piles are not designed to withstand the lateral forces exerted by the soils above the liquefied layer. Lateral displacement of the ground can also occur due to liquefaction and its magnitude will depend on the slope of the existing ground and the proximity of the site to the river bank.

ACKNOWLEDGMENT

The author is grateful to Dr. Marcis Kurzeme for his review of this paper.

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