THE AUSTRALIAN EARTHQUAKE LOADING STANDARD

3 years on - how is it working and what have we learned?

Australian Earthquake Engineering Society

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Proceedings of a seminar held by the

Australian Earthquake Engineering Society

Adelaide

1996

ISBN 0 7325 1472 X

Editors: Michael Griffith, Barbara Butler

Publisher: Australian Earthquake Engineering Society

Cover: Photographs courtesy of Mines and Energy, South Australia
Designed by Barbara Butler

Ground failure near Robe, S.A. after the 1897 Beachport earthquake in South Australia

Earthquake damage, Beachport, S.A. Post Office after the 1897 earthquake

FOREWORD

This volume of the proceedings of the Society's Technical Seminar, held in Adelaide in 1996, is the fourth in the series published by the Society since it was formed in 1990 following tbe Newcastle Earthquake.

The theme for the 1996 seminar was "The Australian Earthquake Loading Standard: 3 years on - how is it working and what have we learned?". The seminar was well supported with 23 papers being presented. The keynote address on "Performance Based Earthquake Engineering" by Dr. Andrew Whittaker, Associate Director of the Earthquake Engineering Research Center at the University of California, was especially informative and thought provoking.

Indeed, the papers presented in each of the four seminar sessions covering the topics of Earthquake Codes, Insurance, Seismology and Unreinforced Masonry are all well worth reading. I commend them to you and look forward to our next meeting - in Brisbane.

Michael C. Griffith
Seminar Organiser

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KEYNOTE ADDRESS

The keynote speaker, Dr. Andrew Whittaker, is Associate Director of the Earthquake Engineering Research Centre at the University of California at Berkeley. Andrew Whittaker, an honours graduate from the University of Melbourne, worked for seven years as a consulting engineer in Melbourne, Adelaide and Singapore with Connell Wagner before moving to Berkeley where he completed his PhD in 1988. He then worked for three years with Forell Elsesser and Associates in San Francisco before taking up his current position at the University of California, Berkeley. His substantial design experience in Australia and outstanding academic background made him the ideal keynote speaker for the 1996 seminar. This paper discusses the development of the next generation of seismic design codes which is now underway in the US.

Performance-Based Earthquake Engineering

Andrew Whittaker

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University of California, Berkeley

Introduction

Current seismic design codes and practices were written to achieve a loosely defined objective of providing building occupant safety. While this objective has been reasonably well achieved, two major shortcomings are recognized. The first shortcoming lies in the realization, made clear by recent earthquakes in the United States, that buildings designed to provide occupant safety may incur extensive structural and non-structural damage, often resulting in huge economic losses for building owners and the community, and loss of function for weeks or months. While casualties have been relatively small (in comparison with other earthquake-prone areas of the world), tens of billions of dollars in direct losses are associated with recent U.S. earthquakes. The second shortcoming is that our seismic design measures are very unevenly applied; some buildings are subject to costly over-design, while other critical or economically important buildings suffer large losses because designs are not properly related to performance needs and expectations.

Expert design professionals and researchers believe that sufficient knowledge now exists, or can be relatively easily obtained, to enable seismic design to be much more closely geared to real building seismic behavior, thereby meeting in realistic terms the expectations of building owners [ATC, 1995a; FEMA, 1996]. This view of the future of seismic design is termed performance based earthquake engineering. The thrust of this approach is to make building performance in earthquakes predictable, and better related to the owners' and society's needs and resources.

Ongoing projects at the time of this writing are proposing future directions for seismic codes in the United States. The key projects are ATC-33 [ATC, in preparation], EERC-FEMA [FEMA, 1996], and SEAOC Vision 2000 [SEAOC, 1995]. Each project is described in this paper.

The ATC-33 Project

Introduction

The primary purpose of the ATC-33 *Guidelines* [ATC, in preparation] is to provide technically sound and nationally acceptable guidelines for the seismic rehabilitation of buildings. The *Guidelines* are intended to serve as a tool for design professionals, a reference document for building regulatory officials, and a foundation for the future development and implementation of building code provisions and standards. The *Guidelines* are being prepared by ATC for the Building Seismic Safety Council with funding from the Federal Emergency Management Agency (FEMA).

The *Guidelines* constitute the most significant advance in the design practice of earthquake engineering since the introduction of ATC 3-06 [ATC, 1978] in 1978. The significant new features of the Guidelines include:

- introduction of seismic performance levels and objectives
- methods for simplified and systematic rehabilitation
- new displacement-oriented methods of analysis
- quantitative specifications of component behavior
- design procedures for "new technologies"

These features are summarized below together with comments germane to new construction. The reader is referred to the ATC-33 *Guidelines and Commentary* for much additional information.

Seismic Performance Levels and Objectives

The *Guidelines* represent the first systematic effort to both (1) define performance levels and objectives, and (2) present explicit design methods to deliver the intended performance. The four performance levels defined in the *Guidelines* are Collapse Prevention, Life Safety, Immediate Occupancy, and Operational. These levels are discrete points on a scale that describe expected performance. Given that the post-earthquake condition of a building may not fit any of the four specified objectives, two performance ranges are defined. The lower range, Limited Safety, extends below the Collapse Prevention performance level. For new construction, the range of Limited Safety, should terminate at the performance level of Collapse Prevention. The higher performance range, termed Damage Control, includes all levels of performance exceeding that of Life Safety.

Each performance level is composed of a structural performance level and a nonstructural performance level, the latter describing the limiting damage state of the nonstructural systems. Such an holistic approach to seismic design constitutes a paradigm shift in design practice.

The minimum performance objective in the Guidelines is the Basic Safety Objective (BSO). The BSO is achieved when a building can satisfy two criteria: (1) the Life Safety performance level for structural and nonstructural components for the Basic Safety Earthquake 1 (BSE-1), and (2) the Collapse Prevention performance level for structural components only for the Basic Safety Earthquake 2 (BSE-2). For reference, BSE-1 is generally assumed to correspond to an earthquake with a return period of 475 years, and is similar to the design basis earthquake adopted in current seismic codes; BSE-2 is the maximum capable earthquake. The above definition of the BSO is likely appropriate for new construction.

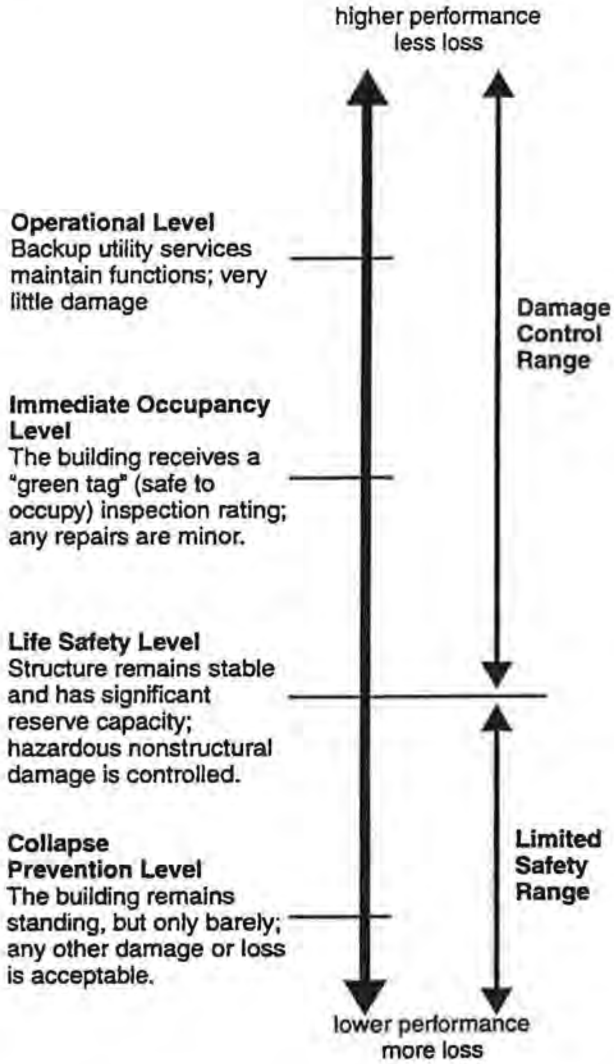

Figure 1 Performance Levels and Ranges [ATC, in preparation]

Methods for Rehabilitation

Two methods for the rehabilitation of buildings are presented in the *Guidelines*: (1) simplified, and (2) systematic. Simplified rehabilitation may be applied to selected types of small buildings with the intent to reduce seismic risk where possible and appropriate by seeking a limited increase in performance. Simple calculations, including the use of the equivalent static force method adopted by current codes for new construction, are the basis of the simplified procedures. Systematic rehabilitation may be applied to any building and involves calculation of likely inelastic displacements, checking of each structural component, and evaluation of each nonstructural component for the drifts associated with the likely displacements. Only systematic procedures are likely appropriate for the design of new construction.

Methods of Analysis

Four analytical procedures are presented in the *Guidelines* for the systematic design of retrofit construction: Linear Static, Linear Dynamic, Nonlinear Static, Nonlinear Dynamic. The choice of analytical method is subject to limitations set on building type, geometry, and degree of expected inelastic response. The linear procedures are new displacement-oriented analysis methods that express displacements in terms of forces for ease of implementation. The Nonlinear Static Procedure (NSP) is a displacement-based procedure which uses simplified nonlinear techniques to estimate earthquake-induced displacements. The Nonlinear Dynamic Procedure (NDP), known to many as nonlinear response (time) history analysis, is the most rigorous of the four procedures, but requires considerable judgment and experience to perform, and will likely not be widely used in the short term.

Nonlinear static analysis, also termed pushover analysis in the literature, is a preferred method of analysis for the evaluation of an existing design. The Nonlinear Static Procedure (NSP) is not a design tool and cannot be used for initial sizing of components. One nonlinear procedure (Coefficient Method) is presented in the *Guidelines*, and two procedures (Coefficient Method and Capacity Spectrum Method) are outlined in the commentary to the *Guidelines*. The end-product of both procedures is an estimate of the maximum likely inelastic displacement at the roof level of a building for a given level of earthquake shaking. Both methods will yield similar answers in most instances. Mathematical models of building frames are then "pushed" until the roof displacement equals or exceeds the maximum displacement (termed the target displacement). The deformations and forces in the structural and nonstructural components corresponding to the maximum roof displacement are used for component checking. Detailed information on the Coefficient Method can be found in the *Guidelines*. The Capacity Spectrum Method is well-documented in the literature [DoD, 1986; ATC, 1995b].

The new linear and nonlinear analysis procedures each represent a substantial improvement in the state-of-the-practice of seismic analysis. All four analytical procedures could be adopted for the design of new construction. A short-term project involving the modification of the *Guidelines* for design of new construction would have an immediate impact on the practice of seismic design in the United States.

Quantitative Specifications of Component Behavior

The introduction of performance levels and performance ranges in the *Guidelines* assumes that performance can be quantified using analytical results such as ductility demands on components and elements, plastic hinge rotations, and story drift ratios. To enable the designer to check components and elements at the selected performance level, stiffness, strength, ductility, and rotation characteristics of common components and elements have been derived from laboratory tests, analytical studies, and expert judgment and put into a standard format in the *Guidelines*. Additional laboratory and analytical studies are needed to verify the values assigned to those component characteristics based on engineering judgment alone.

New Technologies

New technologies are being developed to mitigate the seismic response of buildings. Two important technologies are: (1) seismic isolation, and (2) energy dissipation systems. Both technologies are addressed in the *Guidelines*. Procedures and guidelines for the incorporation of seismic isolators into buildings are well established [ICBO, 1994; BSSC, 1994]. These procedures and guidelines, with a modified format were included in the Guidelines.

Passive energy dissipation devices, also known as supplemental dampers, are being introduced into building frames to reduce seismic response. Draft procedures for the implementation of energy dissipation devices have been developed by the Structural Engineers Association of Northern California (SEAONC) and the Building Seismic Safety Council (BSSC). These draft guidelines were substantially rewritten in the *Guidelines* to facilitate the use of new analysis procedures and new knowledge in this rapidly evolving field.

The EERC-FEMA Action Plan

The development of the EERC-FEMA action plan responds to a need expressed by the Building Seismic Safety Council (BSSC) and others in a number of reports to the FEMA. To meet that need, FEMA organized a preliminary study in which experts from various disciplines and varied geographic regions of the United States examined the issues, possibilities, problems, and opportunities, of performance based seismic design. The preliminary study resulted in an action plan

The action plan recognizes earthquakes as a natural environmental problem that may occur anywhere in the nation, and therefore pose a national threat that will always be present. The passage of time only increases the danger. The plan also recognizes the need to address the problems of existing hazardous construction along with new construction.

The project proposed by the action plan will provide seismic design procedures that are documented in guidelines and commentary that can form the basis of a new generation of seismic codes, and enable designers and owners to expect a high level of reliability in the seismic performance of new construction. In so doing, a new balance between design objectives and design methods can be achieved that will result in overall economies over the lifetime of our building stock while providing building owners and the public with much greater assurance when the earthquake strikes. Improved reliability of the national building inventory will enable lenders, insurers, and re-insurers to better estimate likely earthquake losses, and permit government response agencies such as FEMA to enhance planning for future recovery efforts.

The effort described by the EERC-FEMA action plan covers a span of six years. The following specific products were proposed at project end:

- rational definitions of performance levels
- the technical basis for achieving target seismic performance levels in new and existing buildings, including benefit/cost procedures
- guidelines and commentary for implementation of performance-based seismic design of new and existing buildings
- educational materials and programs for university students, design professionals, and others impacted by the introduction of performance-based seismic design
- a framework for the inclusion of new analysis and hardware technologies
- a national information electronic database to aid planners, policy makers, design professionals, researchers, and others to obtain information and data related to performance-based seismic design.

The action plan is presented in terms of two key elements: Planning, Policy, Management, and Implementation (Element 1); and Technical and Design Requirements (Element 2). Element 1 contains tasks related to overall project management, investigation of appropriate performance objectives, identification of key project milestones, economic and legal issues, and implementation. Element 2 is composed of tasks related to the inventory and synthesis of performance data, identification of design approaches, development of the technical basis for the design approaches, preparation of guidelines and commentary, and benefit/cost studies.

The estimated cost for the effort represented in the action plan is $32.5 million spread over a six-year period. While this represents a substantial sum, this sum is a minute fraction (less than 0.01 percent) of the total annual construction expenditure (approximately $450 billion) in the United States, and a small fraction of the direct losses associated with a moderate or severe earthquake in an urban area.

The successful execution of the EERC-FEMA action plan will require interactions and contributions from all involved in addressing the earthquake problem. The broad constituency to be tapped will include agencies of the federal, state, and local governments; professional trade groups; design professionals; lenders and insurers, and university educators and researchers. Each constituency will benefit from the introduction of performance-based seismic design, and each will have to bear part of the cost for executing the action plan.

SEAOC Vision 2000

The Vision 2000 report [SEAOC, 1995] was prepared by the SEAOC Vision 2000 committee with funding provided by the California Office of Emergency Services. The interim recommendations of the committee are intended primarily for use by design professionals engaged in the design of new and retrofit construction, and to a lesser degree by building officials, public policy makers, regulatory agencies, and building owners and tenants.

The intent of the Vision 2000 recommendations is to:

- define a series of standard performance levels for the construction of buildings
- define a series of reference earthquake hazard and design levels
- specify a series of uniform design performance objectives for buildings of different occupancies and uses
- recommend uniform engineering procedures using currently available technology and design practice

As envisaged by the Vision 2000 committee, performance based seismic engineering constitutes a more holistic approach to seismic design than that currently practiced in the United States, and includes all engineering-related tasks necessary to create building structures with predictable seismic performance. The expanded scope of activity for the design professional includes (1) selection of performance objectives, (2) seismic hazard analysis, (3) seismic design and analysis of structural and nonstructural components, (4) design and construction quality assurance, (5) post-construction maintenance, and (6) monitoring of function and occupancy.

Some of the concepts embodied in the Vision 2000 report have been included in the ATC-33 Guidelines, and will no doubt be incorporated into design practice for new construction before the turn of the century. A key contribution of the Vision 2000 report to advancing the state-of-the-practice is the definition of performance objectives for buildings (see Figure 2). The seismic performance objectives presented in the Vision 2000 report are qualitatively similar to those adopted in the ATC-33 *Guidelines* for retrofit construction. The performance levels are combined with hazard levels to produce performance objectives. The performance levels specified in the Vision 2000 report are:

- *Fully Operational*: building continues in operation with negligible damage
- *Operational*: building continues in operation with minor damage and minor disruption in non-essential services
- *Life Safe*: life-safety is substantially protected, damage is moderate to extensive
- *Near Collapse*: life-safety is at risk, damage is severe, structural collapse is prevented

Much additional information regarding performance-based seismic engineering is presented in the Vision 2000 report. This resource document will play an important role in the future development of seismic engineering codes and guidelines. It is recommended reading for all involved in the design and construction of buildings that are exposed to earthquake shaking.

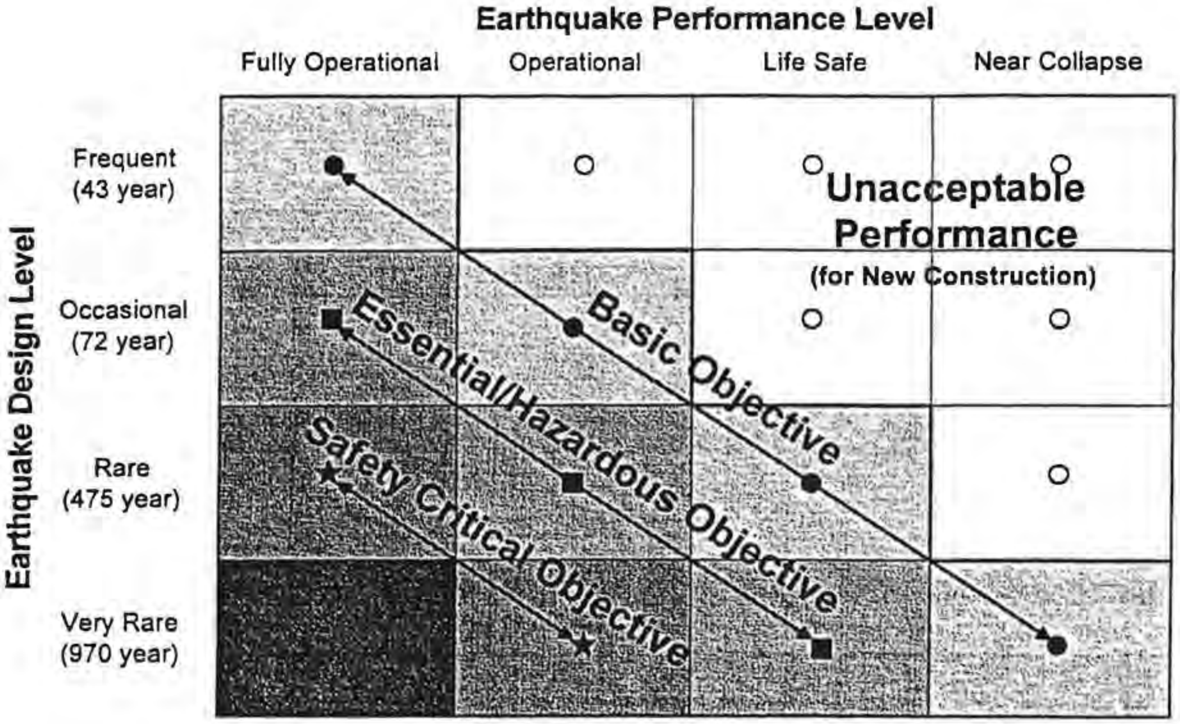

Figure 2 Seismic Performance Objectives for Buildings [SEAOC, 1995]

Summary

Recent earthquakes in the United States (1989 Loma Prieta; 1994 Northridge) and Japan (1995 Hyogoken-Nanbu) have made it clear that buildings designed to provide occupant safety (life safety) may suffer significant structural and non-structural damage in moderate and severe earthquakes, resulting in tens of billions of dollars in direct losses, and loss of function for weeks or months. In part due to these direct and indirect losses, the earthquake engineering community is seeking to develop seismic design procedures that are better able to predict building response, thereby addressing the expectations of building owners and the public-at-large.

Three projects in the United States that focus on performance based earthquake engineering have been described in this paper: ATC-33, EERC-FEMA, and SEAOC Vision 2000. Each project advances the practice of earthquake engineering in the United States, brings performance based earthquake engineering closer to fruition, and likely reduces direct and indirect losses in future earthquakes.

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THE DYNAMIC RESPONSE OF A BRIDGE DURING SIMULATED INTRA-PLATE EARTHQUAKES

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Kevin McCue, FIEAust, has worked as an Earthquake Seismologist for some 20 years and now leads the Earthquake Information and Hazard Assessment Project at AGSO. Kevin has contributed to both earthquake codes AS2121 and AS1170.4, and has participated in engineering projects in Australia, Papua New Guinea and Europe. (Australian Seismological Centre, AGSO, GPO 378, Canberra, ACT 2601)

ABSTRACT:

Strong motion records of Australian intra-plate earthquakes show different characteristics such as frequency content, peak acceleration and duration, when compared with events from inter-plate regions. The lack of quality strong motion records of intra-plate earthquakes at short distances demands the use of synthetic seismograms for testing of structural behaviour. In this study, the near-field synthetic records of likely intra-plate earthquakes are considered, with a strong ground motion duration of several seconds. The dynamic analysis is performed on a model of a bridge structutre, using simulated intra-plate earthquakes with magnitudes similar to that of the 1989 Newcastle earthquake. The Design Spectrum defined in the Australian Loading code is compared with that from potential seismic events. The results of the study are presented in terms of recommendations for earthquake design of bridge structures.

NOTE: The findings presented in this paper are extracted from a large R&D project, coordinated and funded by The Roads and Traffic Authority of NSW. The results of the research should provide some preliminary parameters for modification of sections from the current AUSTROADS, Bridge Design Code 1992, relevant to Earthquake Design of Bridges.

Paper No. 2

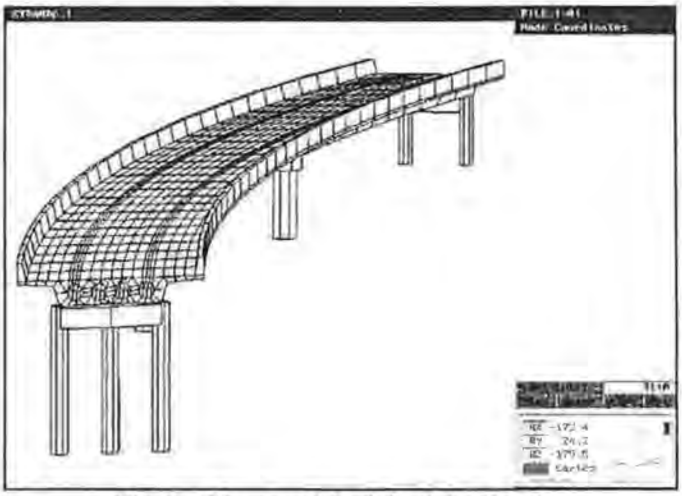
BACKGROUND

The 1989 Newcastle Earthquake, with an estimated magnitude of 5.6 on the Richter Scale, vastly increased the public awareness of the level of seismic activity in Australia [Ref. 1,2]. In the following years a detailed research of the seismic hazard in Australia, was undertaken. To date no significant bridges have suffered earthquake damage in Australia; one of the abutments of the Stockton Bridge settled during the 1989 Newcastle earthquake and a footbridge over the Merri River in Victoria was destroyed during the 1903 Warrnambool earthquake. Bridges in the Wide Bay-Burnett region of Queensland and Perth WA survived intact the large 1918 and 1968 earthquakes. Also, a review of the established design practice was considered. Consequently, in 1993 a completely revised Earthquake Loading Code, AS1170.4 [Ref. 3], was introduced. The '92 AUSTROADS Bridge Design Code [Ref. 4] was prepared in the knowledge that the earthquake provisions may need modification in the light of revisions to AS1170.4.

METHODOLOGY AND BRIDGE MODEL

The aim of this study is to examine the characteristics of the dynamic response of bridge structures due to intra-plate earthquake excitation and to estimate the most probable level of seismic-induced force. Typical earthquakes in continental interiors are thought to be associated with high local stress drop. The duration of higher intensity ground motion is shorter than that for large magnitude inter-plate earthquakes and the frequency content is shifted towards shorter periods [Ref. 5]. The dynamic characteristics of a bridge were examined by extracting the values of the natural periods and associated mode shapes. The mathematical model of the bridges was made by using a finite element software package.

The total seismic forces in all three orthogonal directions, based on the Static Method from AS1170.4, were calculated. The input parameters were selected to simulate the most common design situation in an urban area with a medium to high level of seismicity.



Finite Element Model of the Bridge

Then, the bridge was analysed by the Response Spectral method using curves adopted from AS1170.4. Some additional Spectral analyses were performed using spectral curves derived from some recorded earthquakes.

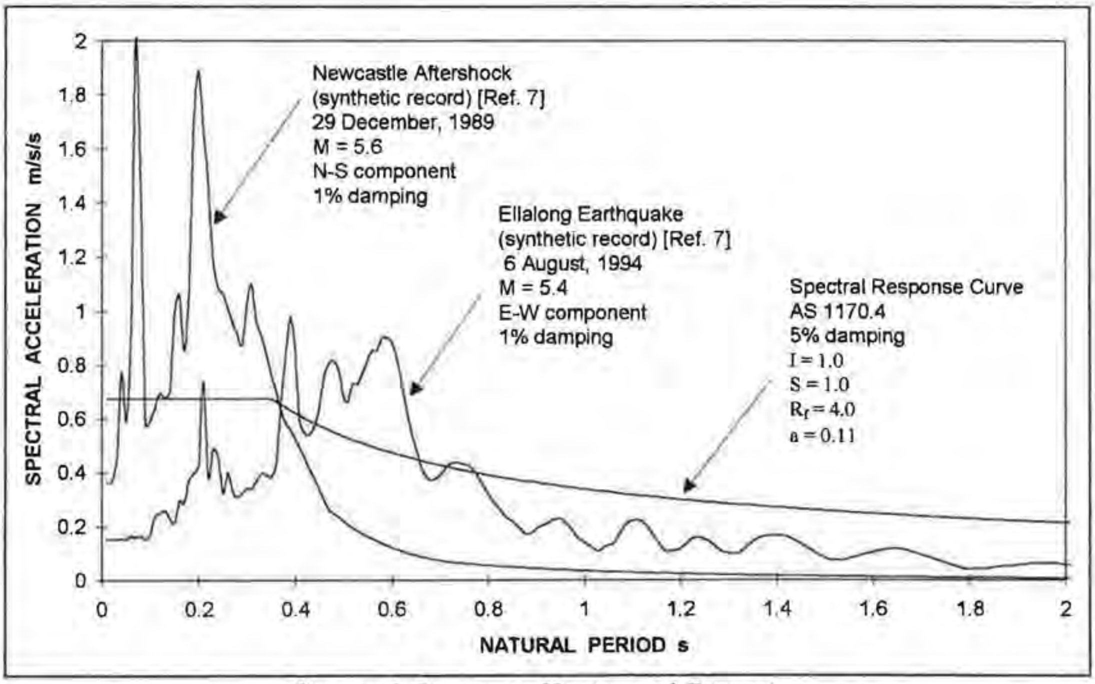
The basic information about the bridge is given below:

Location: Eastbound Ramp over State Highway No 9, on Main Road, near Newcastle
Stricture: Reinforced Concrete Continuous Voided Slab, Span: 33.0 + 43.5 + 33.0 m
FE Model: 16 Beam elements, 1680 Brick elements, 3110 nodes, STRAND6, [Ref 6]
Mass: 2,722 t

SEISMIC INPUT DATA

The seismic input data were taken as typical Response Spectral Curves, which were used as input data in the Dynamic Response analysis. In this study six Spectral curves were used. Two Design Spectral curves, one in the vertical and one in the horizontal direction, were adopted from AS 1170.4 [Ref. 3, Fig. 7.2]. The assumed value for the Site Factor is 1.0. The normalised values of the curves were scaled with the factor of $(a{*}I/R_f)g = 0.27$. (where: Acceleration Coefficient a = 0.11, Importance Factor I = 1.0, Gravity Acceleration $g = 9.81\ m/s^2$, Structural Response Factor $R_f = 4.0$)

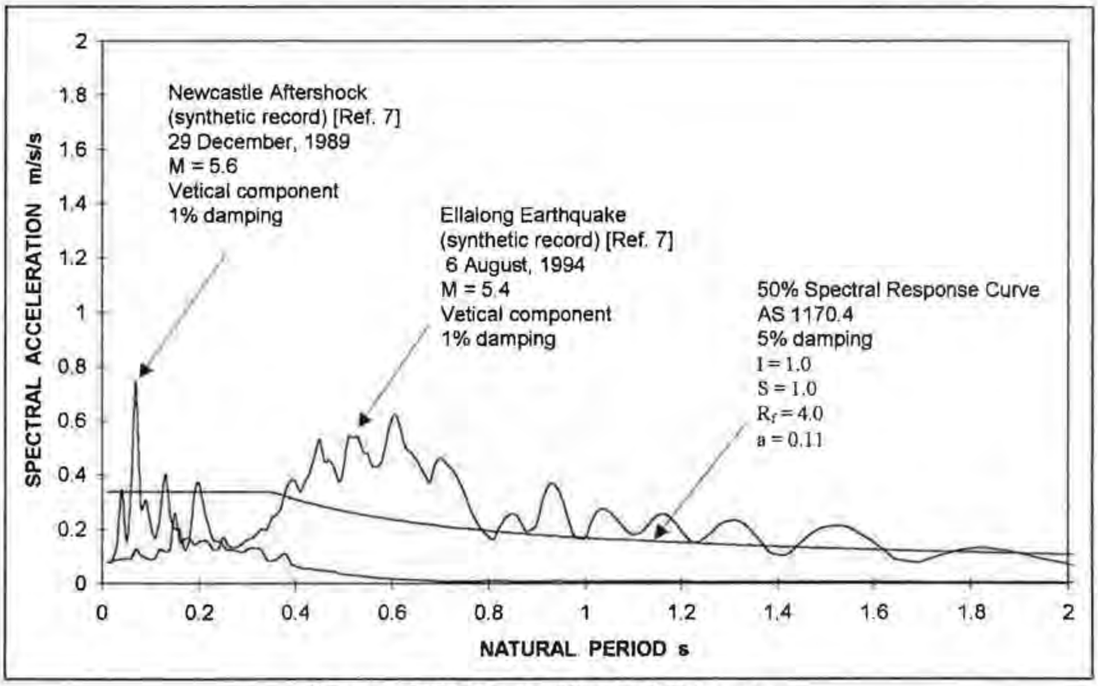
The aftershock of 29 December 1989 earthquake with magnitude 2.3 was used as an input during the simulation of larger strong motion with the Green's Function method [Ref. 8]. The original accelerogram was recorded at the site of the University, about 20km from the epicenter, where a free-field instrument was installed on rock. The sub-events were summed gradually and magnitude increased to 5.6. In the other example, accelerograms from the Ellalong event of 6 August 1994 were used, when a magnitude 5.4 earthquake struck central NSW. For this purpose the record from the closest site - North Lambton, 39km away, was selected. Two spectral curves in vertical and horizontal direction were derived for each case. Spectral curves in the vertical direction are shown in the figures below:



Spectral Curves in Horizontal Direction

The Spectral curve from the code is for a damping ratio of 5%, which is more appropriate for building structures. The Newcastle and Ellalong Spectra are defined for a damping ratio of 1%, which is considered to be more appropriate for bridge structures. The AS 1170.4 curves provide some indication of the earthquake force required by the design code. The Newcastle Earthquake curves provide a more realistic value of the earthquake, in terms of frequency content and intensity.

A typical value of the damping ratio for concrete bridge structures is 2%, as given in the American Bridge Design Specification [Ref 9, C4.7.1.4]. But, in this study a damping ratio of 1% is assumed. This value is considered to be more appropriate for Australian conditions, where the seismic forces are at a lower level. It is expected that the structure will remain in the elastic range, with very limited energy dissipation capabilities.



Spectral Curves in Vertical Direction

FREQUENCY ANALYSIS RESULTS

The frequency analysis results and corresponding Mass Participation Factors [Ref. 7, page 308], for the first 10 natural periods of the bridge model, are given in the Table 1, below.

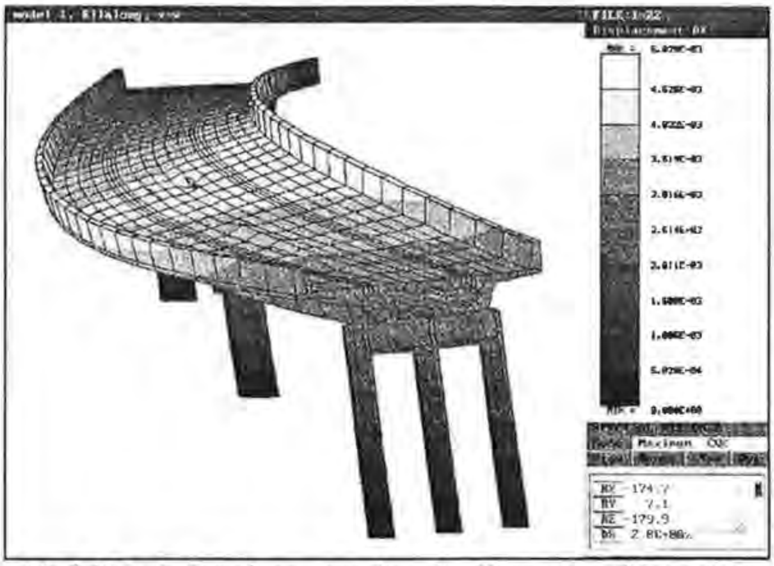
Frequency analysis results Table 1.

Mode No	Freq. [Hz]	Period [s]	Mass Participation Factor [%] Longitudinal	Transversal	Vertical
1	2.02	0.496	46.51	42.46	0.00
2	2.07	0.484	51.03	33.48	0.00
3	2.25	0.444	0.20	0.00	1.92
4	2.82	0.355	1.12	0.31	0.00
5	3.60	0.278	0.03	0.01	0.01
6	4.19	0.239	0.03	0.57	65.76
7	4.39	0.228	0.11	10.65	1.32
8	5.37	0.186	0.58	10.90	0.45
9	6.53	0.153	0.04	0.45	0.00
10	7.86	0.127	0.01	0.05	0.00

In the Table 1, it can be observed that there is not a single dominant mode shape in any direction. For instance, in the transverse direction about 75% of the dynamic response is contributed by the 1st and the 2nd modes, and the additional 20% is contributed by the 7th and 8th modes. The dynamic response is determined by at least two or more mode shapes. One mode shape alone is not sufficient to model the dynamic behaviour of the bridge.

Also, some of the modes have similar values of natural period. This indicates a need to implement CQC (Complete Quadratic Combination) methods for combinations of modal response, instead of the commonly used SQSS (Square Root of the Sum of Squares) [Ref. 7, page: 650]. CQC considers the coupling of modes shape with close values of natural period.

For the bridge models there is not a single dominant mode in any direction. This observation emphasise the need for considering multiple modes when calculating the total earthquake force.



Model No 1, Displacement Results, Transversal Direction

STATIC SEISMIC FORCE

The value of the total Base Shear in each orthogonal direction was selected as the most important parameter, determining the total value of the earthquake force. The value of the Base Shear gives the earthquake force defined as a percentage of the total gravity load. The Base Shear, as defined in AS 1170.7, is a linear function of several parameters. The values of all input parameters, used in this study, are given in Table 2.

Base Shear S_b, by Static Method Table 2.

		Longitud.	Transver.	Vertical
Bridge Model	T [s]	0.49	0.50	0.24
	S_b	5.5%	5.5%	3.4%

DYNAMIC SPECTRAL ANALYSIS

The results for the Base Shear from all analyses are summarised in Table 3. It can be observed that the static method from AS1170.4 provides slightly lower earthquake forces than the earthquake records. The Spectral curves from AS1170.4 demanded about 50% lower forces than the Static Method and earthquake records.

Base Shear [%] by Dynamic Analysis — Table 3.

Spectrum	Longitud.	Transfer	Vertical
AS 1170.4	4.3%	3.8%	2.2%
Newcastle N-S	1.9%	2.3%	1.0%
Ellalong E-W	6.3%	4.8%	0.8%

RECOMMENDATIONS AND DISCUSSION

The total dynamic response of a bridge structures is a combination of more than one mode shape. It is recommended that 10 to 20 mode shapes should be examined, in order to identify the most significant modes in each direction. Mass Participation Factor can be used to identify the contribution of each mode to the response of the structure in a particular direction.

The value of the total seismic force, as define by the Static Method in AS 1170.4, correlates very well with the expected level of seismic activity in Australia. The Static Method in AS1170.4 can be used for bridge structures without any significant modifications. However, it should be noted that the Structural Response Factor R_f has a great influence on the value of the total force. In this study an average value of 4.0 for R_f was used. It is recommended that this factor shall not vary significantly from the average value. The recommended range is from 3.0 to 5.0, depending on construction type [Ref. 12].

The bridge structures shall remain in the elastic range for a total horizontal force of about 5% of the total gravity load. For location with a very low seismicity the total force may be as low as 2%. But, a nominal seismic force above 8% is now justified. All structural elements shall be designed and detailed to resist this force. The Strength Limit States of any section, as defined by the relevant material codes, shall not be exceeded when the nominal seismic force is applied.

The earthquake effects in the vertical direction are generally very low, 1% to 2% of the gravity load. The vertical effects should be considered only for bridges with larger spans. For bridges with spans less than 40 to 60 m, it is expected that the design truck load will be more critical.

The Spectral Curves from AS 1170.4 provide about 50% lower total force than the scaled recorded earthquake events. In order to increase the forces up to the level provided by the Static Methos, the scaling procedure for the Spectral Method, , is recommended. Another alternative is to device a new set of Spectral curves for bridges. Also, a set of carefully selected Spectral curves derived from earthquake records, for a damping ratio of about 1%, may be used.

Spectral method is considered the preferred method of analysis. It provides a more realistic distribution of the internal forces and reactions. A single static force applied at the mass centroid of the deck is considered a too simplistic approach. Since some of the modes are expected to have closely spaced values of natural period, the CQC method is recommended for combining of the modal responses.

The horizontal seismic force shall not be resisted by a single pier or column. The seismic force shall be uniformly distributed over all columns, in both longitudinal and transversal direction.

The load bearing capacity of the bearings shall not rely on friction. The friction coefficient between any two materials shall be assumed to be equal to zero.

It is very likely that in most cases, the earthquake forces will not be critical for the design of the columns and deck. The strength of all the columns in a model bridge, considered in this study, was at least five times larger than the strength required by the earthquakes.

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EARTHQUAKE HAZARD IN QUEENSLAND: A REASSESSMENT

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ABSTRACT:

Earthquake hazard in the State of Queensland has been reassessed using data from additional earthquake monitoring and a new ground motion attenuation relationship. The additional monitoring data suggests the entire east coast of the State is seismically active. New earthquake ground motion data shows the spread between predicted and observed ground motions is greater than that used in earlier studies. Taken together, this new information suggests earthquake hazard in Queensland is greater than previous estimates. We estimate, at a 10% probability of exceedance, ground motions of Modified Mercalli Intensity V, Peak Horizontal Ground Acceleration of 10% *g*, and Peak Horizontal Ground Velocity of 20 cm/sec or more can be expected in the eastern portion of the State during the next 50 years. If the proposed acceleration coefficients are adopted in Queensland, many more structures along eastern coastal Queensland will require design for earthquake loads.

1. INTRODUCTION

Since 1984, almost 900 earthquakes (600 $M_L \geq 1.0$) have been recorded in Queensland and adjacent areas, nearly doubling the number of known earthquakes in the state. In addition, new quantitative strong ground motion prediction relationships have been computed using earthquake ground motion data collected in Australia. This increased knowledge of earthquake locations, rate of occurrence, and ground motion has prompted a reassessment of earthquake hazard in Queensland. Earlier efforts to quantify earthquake hazard in Queensland used earthquake data recorded up through 1984[1,2]. These estimates were subsequently incorporated into the Australian Loading Code, 1170.4[3].

2. METHODS

In this study, we use a probabilistic method of seismic hazard estimation, generally termed the Cornell-McGuire method[4,5]. For the quantitative assessment of the seismic hazard we use the FORTRAN program EQRISK[5], which has been adapted to run on the Unix based computer system at QUAKES.

2.1 Source zones and recurrence parameters

Most seismic activity in Queensland is concentrated in a broad zone along the east coast with local areas of increased activity. The source zones assigned in this study (Figure 1) reflect this observation. A broad source zone of low level activity extends along the entire east coast of Queensland together with a number of smaller zones that attempt to define the areas of increased activity. The resultant hazard map will reflect these observations, obviating the need for the smoothing used in the construction of the maps adopted in AS1170.4. We also use some zones (Simpson Desert and in Papua New Guinea) in areas adjacent to Queensland that have the potential to produce damaging ground motions in the State. These zones are the same as those used in the earlier Australian seismic hazard study[2].

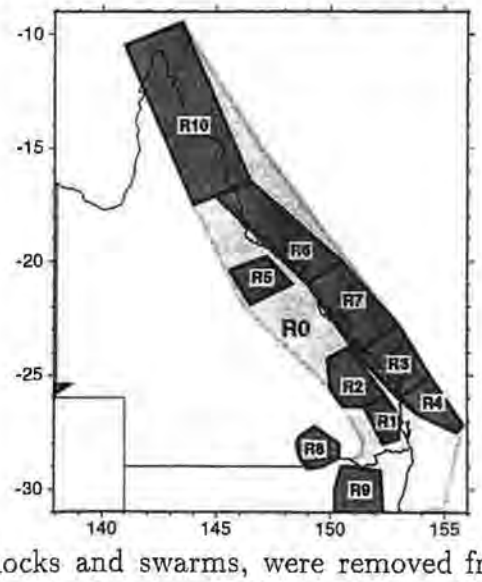

Figure 1: Source zones used in this study. Zone R0 is to take account of scattered seismicity throughout the east coast of Queensland and northern New South Wales. Zones R1 through R10 account for local variations in seismicity rates.

Dependent events, i.e., foreshocks, aftershocks and swarms, were removed from the catalogue using the event codes assigned in the database. Although the criteria for assigning these codes are not rigorous, it is considered that any incorrect assignments will only have a minor effect on the resultant hazard determinations. The distribution of the number of earthquakes as a function of magnitude is assumed to follow the Gutenberg-Richter recurrence relation:

$$\log(N) = A - BM \tag{1}$$

where N is the number of earthquakes with a magnitude greater than or equal to M, and A and B are constants.

For this study we used a modified version of the Stepp Test[6] to estimate the earthquake rate at a particular magnitude. This was used in conjunction with a series of "detectability" maps, i.e., maps showing the estimated detectability threshold of the network, to determine magnitude completeness levels in the source zones. For each zone we estimated the period of uniform detectability at each magnitude level and then plotted the rate of earthquake activity on a magnitude-frequency graph, together with error bars estimated assuming a Poisson process with a given rate. It was found that a line with a B-value of 0.7 could be fitted to the magnitude-frequency plots of almost all zones. It was decided to calculate the earthquake hazard using this average B-value, except for the one zone (R2) where the B-value was constrained by the data to be 0.8.

The rate for the broad Zone R0 was calculated by analysing all earthquakes enclosed within the boundaries of the zone (i.e., including those smaller zones within R0). The rate was calculated in this manner in an attempt to cover the possibility of future activity outside the currently defined areas of increased activity. The rates used for the background level and for the zones in the Simpson Desert and Papua New Guinea regions were taken from an earlier study[2].

Given the short historical observation period and relatively low earthquake occurrence rate, we believe that basing the maximum allowable magnitude on the largest historical magnitude is fraught with danger. Therefore we used a constant value of 7.0 for all zones and for the background. This is based on the observation that $M \sim 7.0$ earthquakes have occurred throughout much of Australia, including three, large (M = 6.3, 6.4, and 6.7) earthquakes that occurred near Tennant Creek, Northern Territory in 1988 in an area that had not previously experienced significant activity[7].

2.2 Ground motion parameters and attenuation

We estimated seismic hazard in Queensland using three parameters, peak horizontal ground acceleration (PGA), peak horizontal ground velocity (PGV), and Modified Mercalli Intensity (MMI). We also review the appropriate standard deviation (σ) to use with the attenuation functions.

We adopted a PGA relationship derived from an inversion of Australian PGA data[8]. This relationship was judged superior to the relationship used in the previous seismic hazard study in predicting PGA in eastern Australia (Figure 2). Unfortunately, there was only one PGA measurement in Queensland available for the present study; therefore we compiled PGA data from the larger database of earthquakes in New South Wales, which has similar geology to Queensland. We selected data recorded at rock sites at distances less than 300 km for earthquakes $M_L > 2.5$ located north of 35° S latitude. The new relationship better predicts the mean values and shows less systematic misfit with distance. What is important to note is that the σ estimated from this data set is larger than that estimated by the authors of the relationships. We used the σ from Figure 2 in our "best estimate" PGA maps.

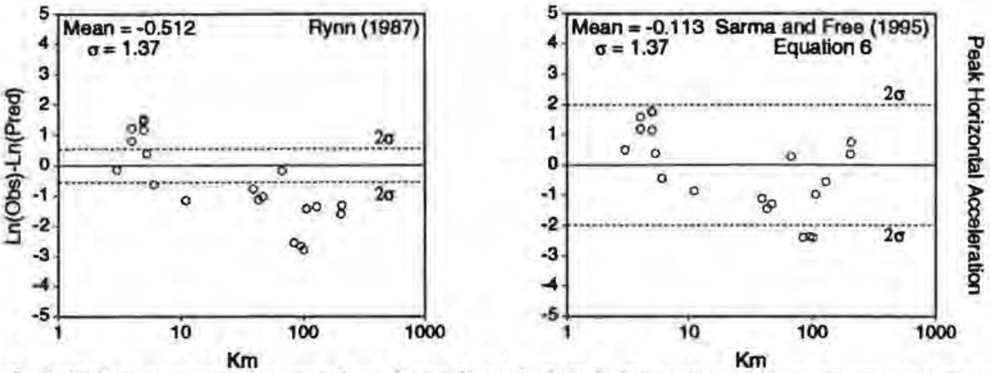

Figure 2: Misfit between observed and predicted PGA. Positive values represent cases where PGA has been underestimated; negative values represent overestimated PGA. The 2σ errors in the individual studies are shown as the dashed lines ($\sigma = 0.28^{(1)}; \sigma = 0.99^{(8)}$).

A relationship between MMI, earthquake magnitude and distance from the earthquake was derived for Queensland as part of a previous hazard study[(1)]. We examined contoured intensity maps for several recent earthquakes and find no reason to revise this relationship. Therefore, we adopt the same MMI relationship and σ used to produce the most recent earthquake hazard map of Australia[(2)].

To our knowledge, a general relationship between PGV, earthquake magnitude, and distance has not been derived using Australian data. Therefore, the only available relationship for PGV for Queensland is the one derived as part of the previous hazard study[(1)]. Given that the spread between observations and predictions in the PGA relationship was much larger than that given, we have decided to adopt this PGV relationship but increase σ. Based upon the results of Figure 2, the true spread between PGA data and observations is about 3 times that assumed in the earlier study. Therefore we increase the σ used in the earlier study (0.22) by an equivalent factor, giving us a value of 1.31.

3. RESULTS

3.1 Probabilistic ground motions

In Figure 3, we present our estimate of PGA with a 10% probability of exceedance in the next 50 years. The maximum exceedance values are in the order of 20-25% g. The eastern coastal region is nearly covered by values of 10% g or greater. Our maximum exceedance values are two to three times as large as those in previous studies[(1,2)]. This is largely due to our adoption of a larger spread for the predicted ground motion values. MMI and PGV results parallel those for PGA. The 5% g and greater regions correspond to values of MMI V and greater and of PGV 15 cm/sec and greater. The maximum MMI value is VI and PGV is 30 cm/sec.

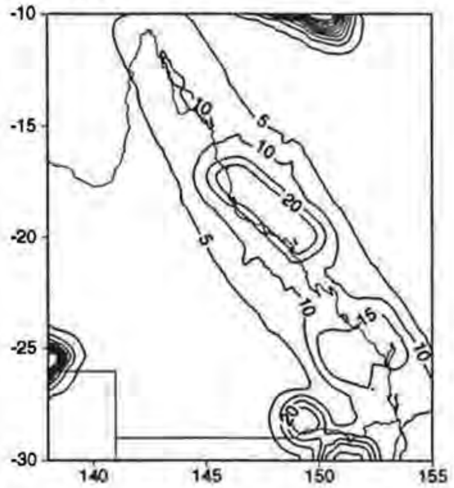

Figure 3: Peak Horizontal Ground Acceleration with a 10% probability of exceedance in 50 years (rock sites). Accelerations are in % g. Contour interval is 5% g.

3.2 Uncertainties

Our best estimate ground motion exceedance values are dependent upon a number of parameters, each of which have some uncertainty. We focus on two inputs: the source zones and their associated earthquake rates, and the σ used for the probabilistic ground motion attenuation calculations. These parameters are dependent upon direct observation of the earthquake process; i.e., our ability to locate earthquakes and record the resulting ground motion.

One major difference between our results and earlier studies lies in the earthquake source zones. Instead of a set of discrete source zones, we define a broad seismic zone that extends along the east coast of Queensland with zones of higher seismicity (Figure 1). Our zones of higher seismicity generally overlap with source zones used in earlier studies. We also computed PGA maps using estimated minimum and maximum earthquake rates in zones R1 through R10, holding the rate in the broad zone R0 constant. The change in computed PGA is in the order of 10% or less.

One of the most important parameters controlling the computed ground motions is σ. It is the difference between the value of σ we use (i.e., 1.37) and the σ used in earlier studies that largely governs the global difference in the magnitude of the PGA values in Figure 3 relative to earlier studies. We also compute PGA using the σ (0.99) given by the authors of the attenuation relationship. This is to take into account that the small amount of data in Figure 2 may lead to an over-estimate of σ. The resulting PGA values are approximately 2/3 those in Figure 3.

4. IMPACT ON BUILDING DESIGN IN QUEENSLAND

In terms of the Australian Standard for Earthquake Loads (AS 1170.4), the extent to which any building needs to be specifically designed or detailed for earthquake effects is determined by the design category into which it falls. The design category, in turn, is a function of the structure's classification and the product of an acceleration coefficient (a) and site factor (S).

Further discussion in this paper is limited to Domestic Structures and Type I and Type II regular General Structures. It is the authors' opinion that Type III structures and irregular structures should always receive specific and very careful earthquake design consideration.

4.1 Acceleration coefficient in current AS 1170.4

The largest acceleration coefficient obtained from AS1170.4, Figure 2.3 (g), is 0.12 near Bundaberg in the Wide-Bay Burnett region. For a location with a site factor of 2, the product aS is then 0.24 and the design categories become:

- Domestic structures ———— H3
- General structures - Type II —— D
- General structures - Type I —— C

This applies to only a small region surrounding Bundaberg. Nowhere else in the state does the acceleration coefficient exceed 0.10. In the Bundaberg region, domestic structures where S has a value of 2 require specific design for earthquake force if they are non-ductile. Elsewhere in Queensland no specific force design is required for domestic structures. Specific detailing requirements apply only to non-ductile domestic structures in category H2. It is considered good practice to provide continuity throughout the structure and in high wind regions this has been recognized as a necessity. Thus the AS 1170.4 requirements have minimal impact on domestic structures in Queensland.

For regular general structures the situation may be summarized as follows[10]:

- Dynamic analysis is never required
- Static analysis will be required for many Type II structures
- When design for earthquake force is required this will dictate the design of upper levels of buildings even in high wind regions
- Most buildings will not require specific design for earthquake force

4.2 Proposed Acceleration Coefficients

The coefficients proposed in this paper are up to 2 to 3 times as large as those in AS1170.4. With this increase and with AS1170 Table 2.6 'Earthquake Design Category' unchanged, many more domestic structures would be caught in category H3, requiring specific design for earthquake force if they are non-ductile. More would also be caught in category H2, requiring more severe detailing and anchorage requirements.

For regular general structures, static analysis will be required for many more Type II structures and for many Type I structures. The earthquake force would dictate the design of many buildings even in high wind areas.

5. CONCLUSIONS

New earthquake hazards maps computed for Queensland suggests that a moderate level of earthquake hazard exists in the State. This earthquake hazard is largely concentrated in the eastern coastal region of the State. The results of this study suggest that, at a 10% probability level, MMI of V and greater, PGA of 10% g and greater, and PGV of 20 cm/sec and greater can be expected in the eastern portion of the State during the next 50 years.

Whereas design for wind loading now dominates on the coast, the proposed coefficients will cause design for earthquakes to become a much more dominant consideration. The same would apply for the rest of Australia because the application of the approach adopted in this paper would cause an upward revision of acceleration coefficients for the whole country. The corollary to this is that the cost of construction will rise. If this is too unpalatable for the public and the legislators, then a different approach will need to be taken in considering earthquake design.

Acknowledgments
André Herrero adapted EQRISK for use on the UNIX system. Peter Mora reviewed the manuscript and provided useful comments. This work was funded by the State of Queensland and the sponsors of QUAKES.

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A PROPOSED SEISMIC DESIGN PROCEDURE FOR HIGH VOLTAGE ELECTRICAL EQUIPMENT TO AS 1170.4 - 1993

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ABSTRACT:

This paper proposes a seismic design procedure to AS 1170.4 - 1993 for HV electrical equipment in Australia, taking into account damping, ductility and risk. The paper recommends seismic design coefficients which are simple to use for the seismic design of HV electrical equipment anywhere in Australia. Examples are used to illustrate the recommended design procedure.

Paper No. 4

1. INTRODUCTION

Experience has shown that High Voltage (HV) Electrical Equipment is very vulnerable to earthquake damage. The major contributing factors to this vulnerability are the low level of damping/ductility inherent in electrical equipment and the brittle nature of porcelain insulators used to support electrical equipment.

Unfortunately, design standards such as AS1170.4 and NZS4203: 1992 have been written to deal primarily with building structures where ductility plays an important part in the design. The application of these design codes to the seismic design of HV electrical equipment, if not interpreted correctly, may lead to the adoption of a design which has inadequate seismic strength.

This paper proposes a seismic design procedure to AS1170.4 - 1993 for HV Electrical Equipment. It is based on current practice in New Zealand and overseas as well as material prepared for the Electricity Supply Association of Australia (ESAA). The procedure as outlined in this paper would be useful for consulting engineers, power engineers and manufacturers of HV electrical equipment.

2. SEISMIC DESIGN STANDARDS

The scope of AS 1170.4 - 1993 excludes nuclear reactors, dams, transmission towers, bridges, piers and wharves. [1] It is intended to apply to structures, particularly buildings, non-building structures, fixings and non-structural components, including building services and architectural elements.

For the seismic design of HV electrical equipment in substations or power stations, other than IEEE Std 693-1984 [2], there are no published standards in Australia and New Zealand other than documents prepared by the Electricity Supply Association of Australia [3],[4] and Trans Power New Zealand Ltd [5],[6]. References (2), (5) and (6) are very useful documents but are based on seismic design loads which are not the same as those applicable to Australia. References (3) and (4) provide useful background information and, when used in conjunction with this paper, would provide a practical means for the seismic design of HV electrical equipment.

3. AS 1170.4 - 1993 - EARTHQUAKE LOADS FOR MECHANICAL & ELECTRICAL EQUIPMENT

Given the scope of AS 1170.4 - 1993 as described above, the question is, can it be used to design HV electrical equipment? The answer, as detailed in this paper is yes, with care.

Clause C5.1.1 of AS 1170.4 Supplement 1 - 1993 states that 'although the standard refers to equipment mounted on structures and portions thereof, the standard should also be applied to equipment mounted on the ground such as high voltage circuit breakers, switchgear, transformers and large horizontally mounted tanks'. The horizontal earthquake load for electrical and mechanical components is given by Clause 5.3.1 of AS 1170.4 - 1993 as:

$$Fp = aSa_c a_x C_{c2} I G_c \leq 0.5\ G_c$$

3.1 **Example 1** - *Determine Fp for a metal-clad switchgear mounted on a foundation pad*

a = acceleration coefficient = 0.1 (for Adelaide)
S = site factor = 1.25 (say, the ground is 30m of firm, stiff clay)

a_c	=	attachment amplification factor = 1 (switchgear bolted rigidly to the foundation)
a_x	=	height amplification factor = 1 since h_x = 0, mounted on the ground
C_{c2}	=	earthquake coefficient = 2 (from Table 5.1.5(b) of AS 1170.4 1993)
I	=	Important factor = 1.25 (structure type III, Table 2.5)
Fp	=	$(0.1)(1.25)(1)(1)(2)(1.25)G_c$
Fp	=	$0.312\ G_c \leq 0.5\ G_c$ (where G_c = weight of the switchgear in kN)

If this switchgear unit is mounted on top of the building then

a_x = $1 + (h_x = h_n)/h_n = 2$
and Fp = $0.624\ G_c > 0.5\ G_c$
therefore Fp = $0.5\ G_c$ should be used

The same process can be used to derive the earthquake design loads for:

- small size transformers
- dead tank circuit breakers
- battery racks
- control panels
- boilers, furnaces
- other items as given in Table 5.1.5(b) of AS 1170.4 - 1993

It should be noted that C_{c2} is given as 0.7 for electrical panel boards and dimmers. However, for electrical switchboards controlling equipment essential for the safe shut down of the substation or power station C_{c2} = 2 should be used.

3.2 **Example 2** - *Determine Fp for an outdoor live tank circuit breaker (say 220kV) mounted on a foundation pad.*

Fp as given in example 1 is based on equipment with an assumed damping of 5% and possesses some degree of ductility. However, live tank circuit breakers, surge arresters, instrument transformers and reactors have little damping (typically between 1 and 2 percent). Furthermore, these items often use porcelain insulators for the main structural support elements which, being brittle, contribute no ductility at all to the equipment during an earthquake event.

To assess how Fp for this example is determined, the seismic design response spectra should be briefly discussed.

4. AS 1170.4 - 1993 RESPONSE SPECTRA

The response spectra as given in Figure 7.2 of AS 1170.4 - 1993 are normalised by the acceleration coefficient, a, and the ratio I/R_f in order to account for structural importance and ductility where I is the important factor and R_f is the structural response factor. The damping value associated with this spectra is 5% of the critical damping. It should be noted that R_f is an empirical response reduction factor intended to account for both damping and the ductility inherent in the structural system. Thus, for a lightly damping building structure of brittle material, R_f would be close to 1 (clause C6.2.6 of AS 1170.4 - Supplement 1 - 1993). Based on I = 1.25, a = 0.1 (for Adelaide) the maximum seismic coefficients for the derivation of base shear V are

for minimum R_f (Table 6.2.6(a)), $V_{max} = I(2.5a/R_f)\ G_g) = 0.208\ G_g$
for $R_f = 1$, $V_{max} = I\ (2.5a/R_f = 0.312\ G_g$

For the purpose of this paper, the response spectra associated with $R_f = 1$ are considered to be the linear elastic response spectra where the ductility level is about 1. It should be noted that R_f is not a displacement ductility (Appendix B, AS 1170.4 - 1993). To make adjustment for different damping values, it is proposed that the relationship as given in Clause 12.2.9 of NZS 3404: 1992 [8] be used.

Scale factor for x damping = $[0.5+1.5/(0.4\ x + 1)]$

for x (percentage of critical damping)	= 2,	5,	10
Scale factor	= 1.33,	1,	0.8

Returning to example 2, earthquake load Fp can now be estimated by:

a) Fp (2%) = Fp (5%) x scale factor (2%) G_c
= $(0.312)(1.33)\ G_c$
= $0.415\ G_c$
where G_c = weight of the equipment in kN

It should be noted that Fpmax should also be scaled by the same factor:

i.e. Fpmax = $0.5\ G_c$ (5%)
Fpmax = $0.665\ G_c$ (2%)

b) Alternatively Fp (2%) can be estimated from the elastic response spectra at $R_f = 1$. The maximum spectral acceleration is 0.312g for periods between 0 and 0.5 sec (Clause 7.2 of AS 1170.4 - 1993). Since the items of equipment in example 2 are expected to have natural periods in this range, it should be designed at an acceleration of (1.33)(0.312g) = 0.415g.

It should be noted that for flexible items such as live tank circuit breakers, a dynamic analysis for the earthquake design would be more appropriate than the static method as outlined above.

5. **EXAMPLE 3** - *Determine the earthquake design load Fp for a 3 phase 100 MVA transformer mounted on a foundation pad.*

Example 1 shows how Fp is derived for small-size transformers and where Fp = $0.312\ G_c$. For large transformers, it would be too conservative to apply the same design earthquake load. Large transformers (typically weighing between 20 and 200 tons) are very rigid items when bolted to the foundation pad. Their fundamental periods have been analysed to be about 0.03 sec or less. Because of this rigidity, they tend to move with the ground during an earthquake event without any amplification of the ground motion. On the other hand, small transformers (or control panels, metal-clad switchgear, battery racks etc) are more flexible and are likely to experience amplified motion.

Unfortunately, AS 1170.4 - 1993 does not appear to provide a peak ground acceleration at zero period (refer to Example 4). The normalised response spectra shows a peak acceleration response from 0 to 0.5 sec (for S = 1.25) of 0.312g (for $R_f = 1$, I = 1.25, a = 0.1, damping = 5%) but this is not the same as the zero period acceleration. To establish a peak ground acceleration, in the absence of other information, the normalised response spectra of NZS 4203: 1992 [7] for intermediate soils (to match S = 1.25) is proposed as a guide.

Using NZS 4203: 1992, Table 4.6.1(b)

Peak response	=	1g at 0.2 sec normalised to 2.38
Peak ground acceleration	=	0.42g at 0 sec normalised to 1
Truncated acceleration	=	0.8g (for μ = 1 at 0.45 sec) normalised to 1.9

If the same normalised ratios are used for AS 1170.4 - 1993 normalised spectra, the following values are obtained:

Peak response acceleration	=	0.39g
Peak ground acceleration	=	0.164g
Truncated acceleration	=	0.312g

Figure 1 shows the results obtained from this method. An elastic response spectra for HV electrical equipment in Adelaide on S = 1.25 is also proposed as curve GHIK. It should be noted that the reason for the truncation of the response spectra, according to NZS 4203: 1992 is that low displacement at low periods is unlikely to cause collapse, thus a reduced strength is permissible. This is primarily for buildings. For HV electrical equipment, it could be argued that the truncation should not be used and the peak above the truncation should be included in the response spectra (refer Figure 1).

By a linear interpolation, the acceleration at 0.03 sec is 0.198g. So for large, rigid transformers at Adelaide, the following earthquake load Fp should be used

Fp = 0.198 G_c say 0.2 G_c

where G_c = total weight in kN

However, to allow for any likelihood of lengthening in natural period, the seismic design load for large transformers should be set at 50% higher than the peak ground acceleration,

i.e. Fp = (1.5)(0.164) G_c

Fp = 0.246 G_c

6. **EXAMPLE 4** - *Determine Fp for a computer panel which is critical to the operation of the electrical network.*

Acceleration coefficients as defined by AS 1170.4 - 1993 is 'an index related to the expected severity of earthquake ground motion'. It is obtained by 'dividing peak ground velocity, in mm/s by 750'. So, it is not a peak ground acceleration. The acceleration coefficient maps as per AS 1170.4 - 1993 depict contours of the acceleration coefficient with a 10% chance of being exceeded in 50 years. This gives an annual probability of exceedance of 0.2% (10% divided by 50). By definition, the return period is the inverse of the annual probability of exceedance or 500 years.

There are items in a substation or a power station where a 500 year return period earthquake may not represent adequate risk assurance and a 1000 or 5000 year return period event would be more appropriate. The panel in this example, for the purpose of this paper, should be designed to withstand a 1000 year return period earthquake event. To take into account the more severe earthquake event, this paper proposes that the relationship between the risk factor and return period as per NZS 4203: 1992[7], Volume 2 be used. Based on Figure C4.6.1 of NZS 4203: 1992, Vol 2

Return period = 500 years, Risk factor R(500) = 1.03

Return period = 1000 years, Risk factor R(1000) = 1.3

The ratio between R(1000)/R(500) is 1.26.

So for this example, the panel should be designed to:

Fp (1000)	=	1.26 Fp (500)
	=	(1.26)(0.312) G_c
	=	0.393 G_c
say	=	0.4 G_c
where G_c	=	weight of the panel

7. **EXAMPLE 5** - *Determine earthquake loads for a substation control building.*

The structure type is III (for a substation or power station control building).

a	=	0.1
S	=	1.25 as before
I	=	1.25
aS	=	(0.1)(1.25)
	=	0.125

Hence earthquake design category is D. Assume the building is a load bearing masonry, the earthquake base shear V is

$$V = I(CS/R_f)G_g$$

within the limits $V \geq 0.01\ G_g$ and $V \leq I\left(\frac{2.5a}{R_f}\right) G_g$

assume

	h_n	=	4m
	T	=	0.086 sec
	C	=	0.631
	R_f	=	4 for reinforced masonry shear walls
hence	V	=	0.25 G_g

which is greater than $(1.25)(2.5)(0.1)/4)G_g = 0.078\ G_g$.

So V = 0.078 G_g should be used.

It should be noted that equipment shown in example 2 is designed to an earthquake load at 0.312g is at least 4 times the design earthquake load for this building. If these items of equipment are inside this building then, should the design earthquake load be exceeded, the building may collapse onto the equipment. To avoid this situation, care should be taken in the selection of a suitable structural response factor R_f.

8. **SUMMARY**

To enable the results presented in this paper to be used for other soil conditions in different parts of Australia, the following table 1 is prepared.

Example 6, a = 0.05 (Hobart), S = 2 (soft soils), G_c = 100 kN, Fp = 2.5aS G_c = (2.5)(0.05)(2)(100) = 25 kN where Fp is the horizontal design earthquake load.

HV ELECTRICAL EQUIPMENT	PROPOSED SEISMIC DESIGN COEFFICIENT, C
<u>Flexible Items with 5% Damping</u> Metal-clad switchgear, control panels Battery racks, small transformers Boilers, battery chargers	C = 2.5 aS
<u>Flexible Items with 2% Damping</u> Live tank circuit breakers, surge arresters Instrument transformers, reactors Capacitor racks, line traps Disconnectors	C = 3.3 aS
<u>Rigid Items with 5% Damping</u> Large transformers, steam turbines Gas turbines	C = 1.6 aS or C = 2 aS

Table 1 - Proposed Seismic Design Coefficients
for Ground-Mounted HV Electrical Equipment in Australia
a = acceleration coefficient, S = site factor, I = 1.25, R_f = 1,
static analysis only, Fp = CG_c where Fp and G_c are in kN

9. CONCLUSIONS

This paper outlines how AS 1170.4 - 1993 in conjunction with NZS 4203: 1992 can be used to derive the seismic design loads for HV electrical equipment. The results presented would provide a useful starting point for design engineers to use to ensure that HV electrical equipment is adequately protected against earthquake damage in Australia.

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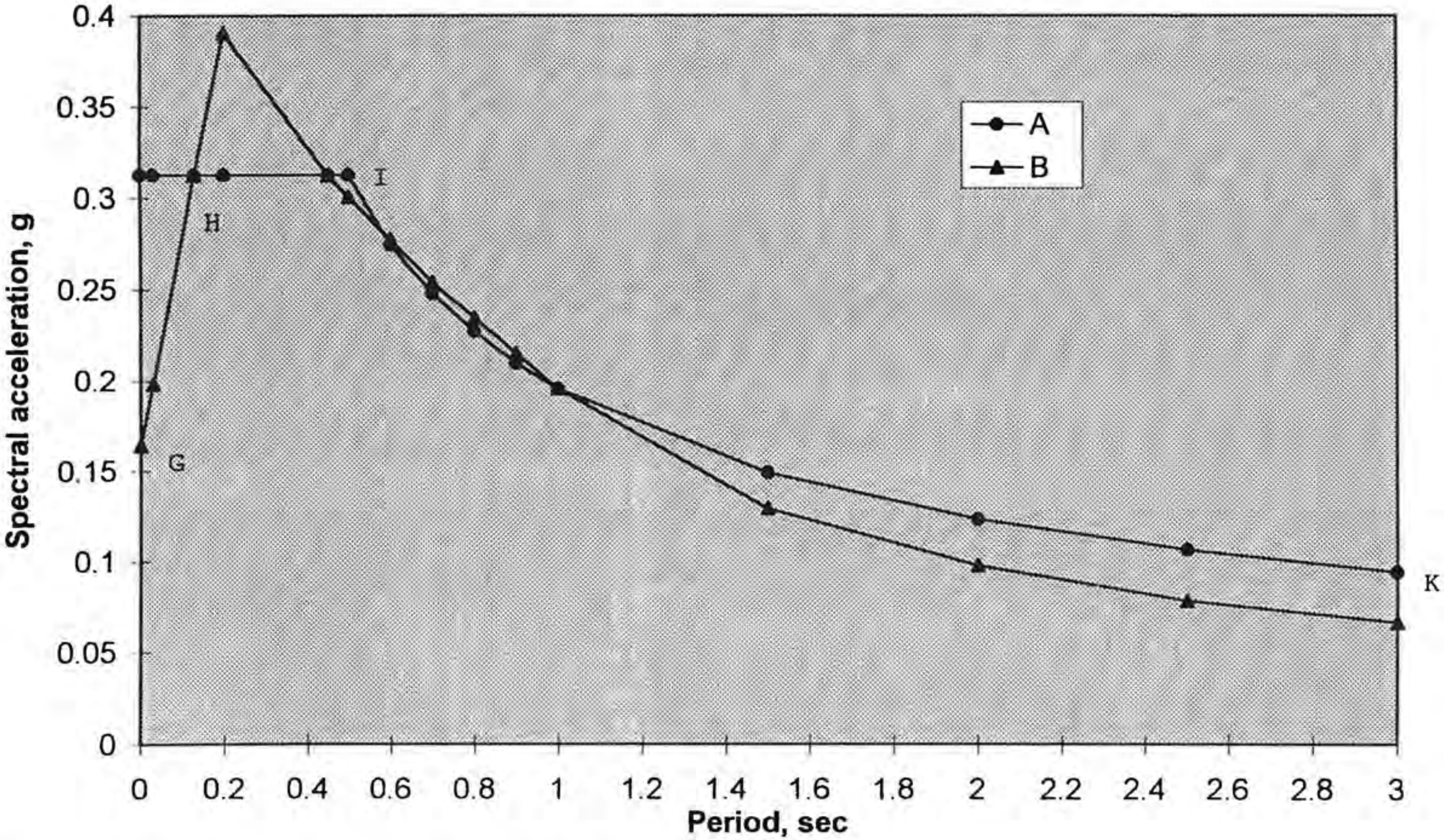
(8) NZS 3404: Part 1: 1992 - Steel Structures Standard - published by Standards New Zealand.

Fig. 1 - Proposed Elastic Response Spectra for Adelaide a=0.1, I=1.25, Rf=1, S=1.25, damping=5% (curve GHIK)



A = based on normalised spectra in AS1170.4-1993
B = based on normalised spectra in NZS 4203:1992 for intermediate soils, ductility factor Mu=1

STRUCTURAL RESPONSE UNDER INTRAPLATE CONDITIONS

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ABSTRACT:

This paper considers each stage in the seismic evaluation procedure and highlights factors contributing to differences between intraplate and interplate structural responses as identified from recent research. Discussions cover bedrock excitations, soil amplification, inelastic structural responses, overstrength and ductility.

1. INTRODUCTION

The vulnerability of buildings and infrastructure to earthquakes is a concern in most countries, including low seismicity intraplate countries such as Australia(1). The seismic resistance of existing buildings can be evaluated by comparing the expected seismically induced base shear force with the ultimate capacity of the lateral load resisting system. In the absence of well established information on intraplate seismically induced loads, the earthquake loading standard for Australia(2), (like many other earthquake loading standards around the world), are based on codes and recommendations developed in the USA, such as the Uniform Building Code(3). In order to take into account the seismicity level of the area, design earthquake forces are adjusted using coefficients specified in local seismic hazard maps . Such seismic hazard maps have been developed from small near field or larger far field recordings and historical macroseismic data, and not from near field strong motion accelerogram recordings(4).

The primary objective of intraplate earthquake engineering research is to develop means of ascertaining the seismic risks in an intraplate area. Whilst some progress has been made in understanding intraplate earthquakes in a seismological context(5-7), far less is known from the engineering context concerning structural response behaviour. The lack of representative data and well documented experience has made it difficult to realistically predict the impact of future intraplate earthquakes on structures.

To study the response of structures to intraplate earthquakes, the following steps in a conventional seismic evaluation procedure must be considered. Initially, seismic hazard maps provide the acceleration coefficients to define the level of seismicity. The 'Design Response Spectrum' is then scaled according to the acceleration coefficients to predict the elastic structural response on a rock site. The effect of soil overlying bedrock is then taken into account by the 'Site Factor'. Finally, the 'Structure Response Factor' is used to predict the inelastic responses taking into account the overstrength and ductility of the structure.

The objective of the paper is to review each step in the procedure and highlight differences between intraplate and interplate conditions as identified from recent research.

2. ELASTIC RESPONSE ON ROCK

The majority of recorded intraplate earthquakes are relatively shallow and small and associated with a compressive fault mechanism(4,8). Such earthquakes generate very high frequency, and possibly damaging, vibrations in the near field. In contrast, lower frequency damaging vibrations are typically associated with larger, and deeper, interplate earthquakes in the both near and far field(9). However, both large intraplate earthquakes and shallow thrust faulting interplate earthquakes can occur although these occurences are less common.

Further, significant variation of frequency content amongst intraplate earthquakes is common. For example, Figure 1 shows the normalised response spectra of bedrock accelerograms recorded from the magnitude 6.9 earthquake* in Nahanni, Central Canada, in 1985(10). Although the accelerograms are recorded within 25km of each other on rock, very different response spectral shapes are found. Hence, the frequency content of intraplate ground motions can be highly variable.

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* The Surface Wave Magnitude of the mainshock was 6.9. The Body Wave Magnitude of the mainshock and the aftershocks are shown in the legend of Figure 1.

3. ELASTIC SOIL AMPLIFICATION

Soil amplification is traditionally considered to be primarily dependent on the geology of the site.The UBC[3] and AS1170.4 [2] introduces the Site Factor 'S' to take into account the effect of soil overlying bedrock on the base shear of the building. The value of S varies from 0.67 for rock to 2.0 for deep soft soil. Importantly, the peak spectral acceleration associated with the flat part of the soil spectrum is assumed to be always equal to that of the bedrock spectrum. Effectively, the spectrum "Corner Period" increases with the Site Factor.

Recent research in the United States has identified that the peak spectral acceleration in the soil spectrum can be very much higher than the corresponding bedrock spectrum[11]. Further, the degree of amplification depends very much on the intensity of the ground motion. Lower intensity bedrock excitations generally result in larger amplifications. As the excitation intensity increases, amplification is gradually suppressed due to the non-linear softening behaviour of the soil.

Recent studies by the authors have further confirmed that the frequency characteristics of bedrock excitation has a very significant effect on soil amplification behaviour[12]. Amplification is enhanced if the soil natural frequency is close to the excitation frequency of the ground motion at bedrock level. In contrast, a deep soft soil with a long natural period will attenuate high frequency seismic waves rather than amplify them. Clearly, the response of soil to intraplate ground motions is very dependent on frequency content which is quite variable and difficult to predict.

4. INELASTIC RESPONSE OF STRUCTURES

Most structures built of ductile materials such as well detailed steel and concrete are normally designed to yield and deform significantly beyond the elastic limit when subjected to ultimate earthquake loading. It is assumed in the widely used 'Force-Based Assessment Procedure'[13] that the maximum displacement of the structure is approximately equal to the corresponding elastic displacement. However, this 'Equal-Displacement' assumption is only reliable if the natural period of the structure is greater than the predominant soil natural period[14]. Much larger displacements are expected for low rise structures on rock, or for taller structures founded on soft soils[15]. Further, structures which respond in the inelastic range without a significant degradation in strength respond similarly to earthquake ground motions with similar frequent content regardless of their origins, phase-angle characteristics and durations. The excitation frequency content has been identifed as the most influential factor governing the response behaviour of linearly elastic perfectly plastic structures.

The effect of regional seismicity on inelastic displacement has also been considered (publications are currently under preparation). For example, if two identical structures are each located in an intraplate and an interplate area with the same design acceleration (based on 10% exceedence in fifty years), the average ductility demand of the structure located in the intraplate area is expected to be less than that in the interplate area due to the probabilistic distribution characteristics of the earthquake intensity. Conventional seismic design procedure ignores such differences.

5. OVERSTRENGTH AND DUCTILITY

The seismic performance of a structure depends on both the system's lateral strength S_r and the allowable system ductility μ. For satisfactory performance, the ultimate displacement Δ_u should satisfy : -

$$\Delta_u \leq \mu \cdot \Delta_r \qquad \text{Eq(1)}$$

where :

$$\Delta_r = \frac{S_r}{K_r} \qquad \text{Eq(2)}$$

$$S_r = \Omega_{os} \cdot S_d \qquad \text{Eq(3)}$$

and where Ω_{os} is the Overstrength Factor and the other variables are defined in Figure 2 .

The Overstrength Factor takes into account the increase from the design strength to the ultimate strength of a member(16) (refer Figure 2). For a highly redundant structure where a mechanism is formed well beyond the strength associated with first yield, significant system overstrength is produced. It is considered that structures generally have system overstrength of about 2 (17). However, designers must ensure that the structure has the necessary ductility to mobilise any overstrength assumed in design. Overstrength is **not** an elastic consideration. Premature failure by shear, bond or buckling or failure at the connections must also be identified. Further, non-structural components in a structure must not be assumed to contribute to overstrength if their behaviour under earthquake is uncertain. Estimations of overstrength can be evaluated using a static push-over analysis.

The allowable system ductility depends on the kinematics of the system undergoing displacement and the strength degradation properties of individual members(16). The kinematic effects relate the system ductility to the individual rotational ductility of a plastic hinge. Strength degradation depends on the design and detailing of individual members. The New Zealand Standard stipulates that strength degradation must not exceed 20% following four full reversed load cycles(17). Similar criterion for intraplate conditions has not yet been established.

The quantification of overstrength and ductility of structures in intraplate areas are currently being investigated by the authors. Due to differences in the design and construction practices between interplate and intraplate countries, significant differences in overstrength and ductility are possible.

6. CONCLUSIONS

1. The frequency content of intraplate earthquake excitations are highly variable.

2. Intraplate soil amplification is also highly variable as it depends not only on the soil properties of the site but also on the intensity and frequency characteristics of the bedrock excitations.

3. The inelastic displacement of a structure may significantly exceed the equivalent elastic displacement depending on the natural period of the structure in relation to the soil natural period.

4. The probabilistic distribution of intraplate earthquake intensity contributes to some reduction in the ductility demand.

5. The seismic performance of a structure depends on the overstrength and ductility which in turn depends on the structure's design and detailing at both system and member level. Thus,

overstrength and ductility factors associated with intraplate regions could be different from interplate regions.

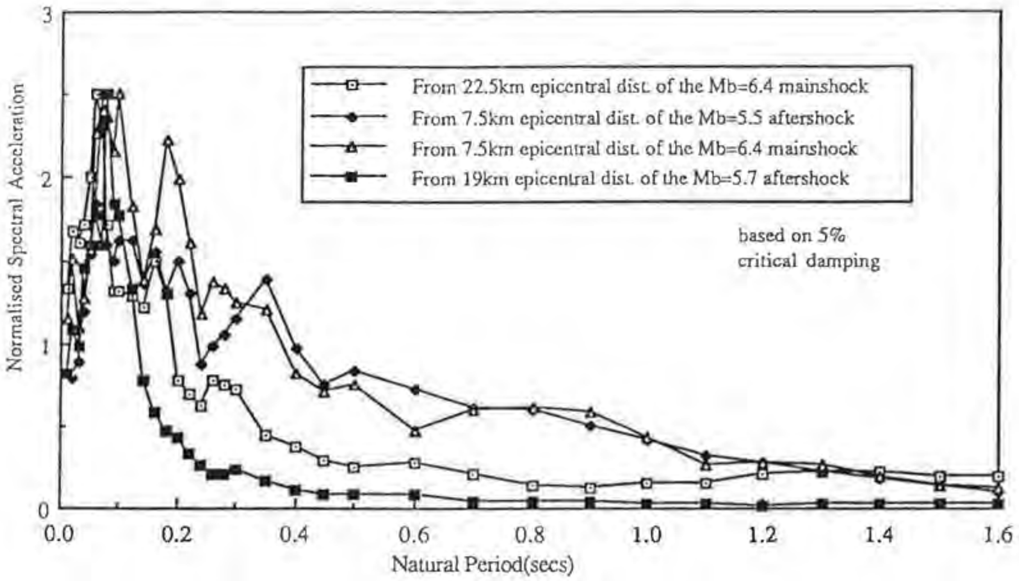

Figure 1 - Normalised Response Spectra of Accelerograms Recorded at Nahanni, Central Canada, in December, 1985.

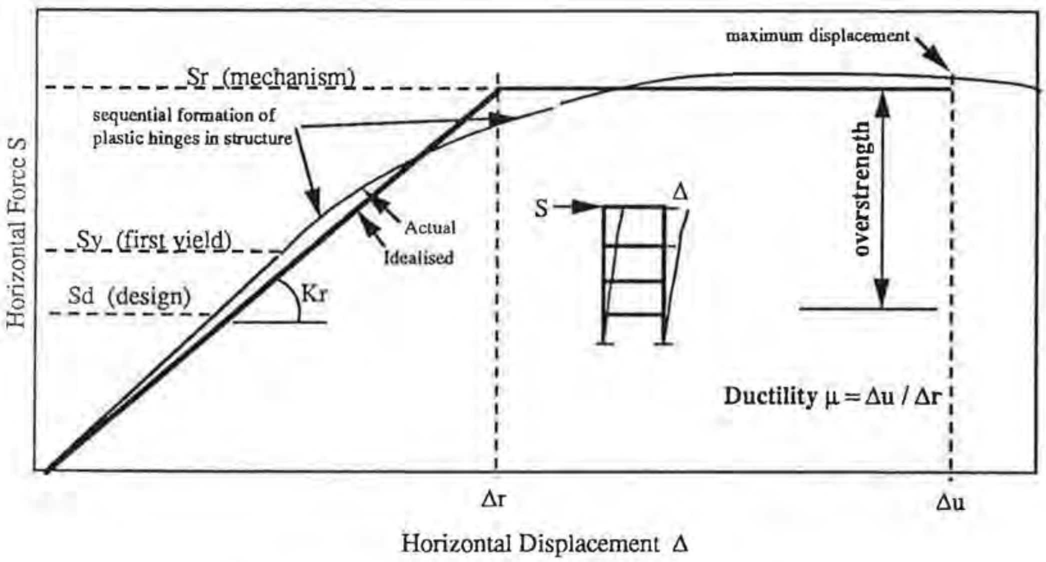

Figure 2 - System Overstrength and Ductility

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A REINSURER'S VIEW OF EARTHQUAKE RISK ASSESSMENT

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Eric Durand joined Swiss Re in Zurich in 1990 starting in their natural catastrophes' R&D group. After specialising in storms and climatological hazards he was sent on secondment to Swiss Re Australia. There, as a research scientist and underwriter, he has worked on all natural perils relevant to the Australian insurance business.

ABSTRACT:

A proper risk assessment is of utmost importance for the insurance and reinsurance companies offering cover against earthquake damages. Due considerations must be given to all elements of the peril. The seismic activity, the sensitivity to damages and the insurance conditions must be carefully ascertained to ensure the long-term viability of the insurance cover. We will show in our presentation how Swiss Re, one of the leading reinsurance companies, has integrated its risk assessment approach into a software package. This paper, in conjunction with the accompanying demonstration, shall shed some light onto the process of rendering insurable a risk which, half a century ago, was mostly considered uninsurable.

Paper No. 6

Introduction

The global cost of natural catastrophes is constantly increasing. The following graph[1] quite clearly shows this trend in the insured costs of such events. Since 1989 the yearly loss burden for the insurance community has constantly been over US$ 11 billion, whereas it never exceeded US$ 8 billion before 1988. It peaked in 1992 at around US$ 22 billion with hurricane Andrew as major loss (US$ 16 billion) and in 1994 at over US$ 15 billion with the Northridge earthquake (US$ 12 billion).

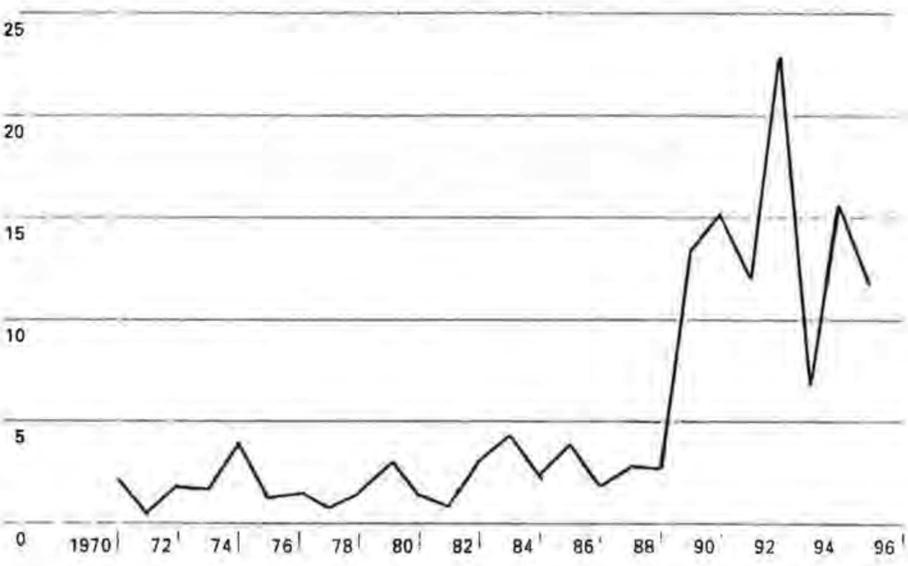

Figure 1: Increasing insured losses due to natural catastrophes (US$ billion at 1992 prices).

Looking back at that development, and still suffering from these past losses, the insurance and reinsurance industry is wondering what the future might have in store. Whilst in the field of hurricanes and other types of atmospheric hazards climate change might be playing a role, the increase in the cost of earthquake catastrophes must rather be found in the ever increasing development of wealth, especially in zones at risk from major tremors. The trend in the insured loss is further enhanced by the widening in the scope of cover offered by the insurance industry and by the development of covers for indirect losses like Business Interruption and Loss of Rents or for secondary effects (e.g. tsunami, EQ-induced landslides etc.).

The 1906 San Francisco earthquake had a major impact on the insurance and reinsurance market and, although direct earthquake damages were excluded from reinsurance contracts, fires caused by the earthquake were not explicitly excluded. This brought about the largest ever pay out hitherto from the reinsurance community.

Some 15 years ago Swiss Re, one of the major reinsurance companies world-wide, set up a Research & Development group in order to better understand the threat of natural perils to its business. The technical assessment methodology developed and used by Swiss Re is summarised in the following paragraph.

The technical approach

Our approach combines the different elements of a computer aided risk analysis to offer a comprehensive solution[2]. Two main parts emerge, a scientific/technical assessment of the hazard on the one hand and an insurance related evaluation on the other hand (Figure 2).

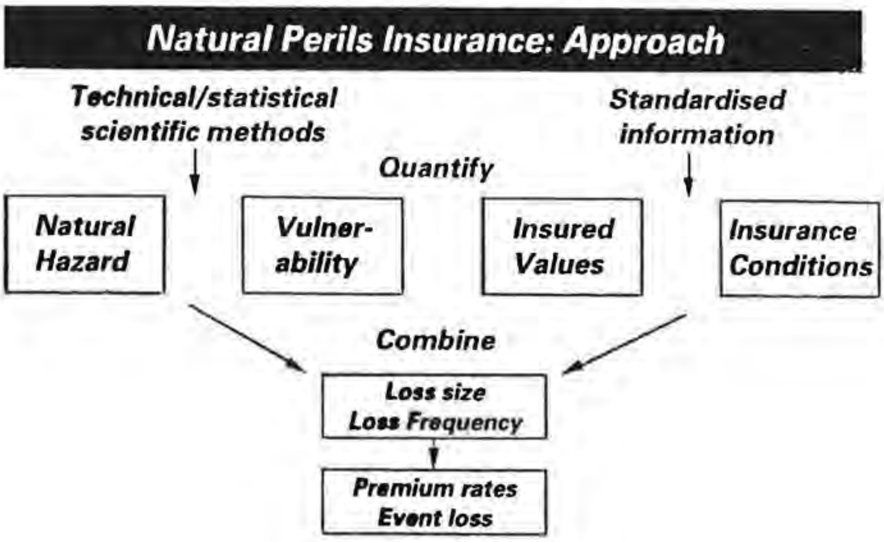

Figure 2: Four Box concept.

The hazard box looks at the frequency and intensity (intensity here in a very general sense, not limited to the seismological meaning of the word) of the events in the region of interest. This includes the gathering of the information, such as catalogues of earthquakes, and also some extrapolation for larger than recorded events in those regions. This allows us to somewhat compensate for the short observation period for which reliable measurements are available. The earthquake activity is then geographically spread over a regular grid (mesh size of 1/2 degree) to alleviate the need to use individually defined seismic zones[3]. Each grid point, considered to be a source point, contains as seismic parameters the following elements: the magnitude-frequency relationship, the maximum possible magnitude, the focal depth distribution and factors for the attenuation relationship.

The vulnerability is certainly a most crucial element. Here the interest is in the propensity for damage in relation to an intensity parameter. The simplest form being a Mean Damage Ratio (i.e. cost of repair in percent of Total Sum Insured in the region of interest) for given (Mercalli) Intensities. The relationship is a function of the building construction type, its age and its shape[4]. The subsoil quality plays a further role in our definition of the vulnerability curves. Fire Following EQ is also taken into account as an aggravating factor using four different levels. These levels have to be set according to prior knowledge of the construction type, the use of gas in domestic buildings (cooking, heating) and the density of the building environment.

The insurance data The portfolio entered for analysis should be as detailed as possible. Splits between Buildings and Contents for different business classes is advantageous. Typical business types used are Homeowners/Houseowners (in other words the Domestic risks), Commercial and/or Industrial risks. The number of risks in the portfolio to be studied is also an important parameter, influencing the probabilistic spread of the vulnerability function. Analysing a single risk (e.g. a so-called facultative risk) requires a much more in depth analysis of the risk factors (siting, vulnerability, etc.). Insurance parameters such as the level of primary deductibles, co-reinsurance and limits of indemnity can also be taken into consideration by the system.

The analysis Once all the necessary data has been entered, the system is in a position to compute the risk premium needed to cover the long-term expected earthquake loss burden of the portfolio. The insurance industry is also interested in the calculation of the potential losses for very rare events, trying to set up EMLs and/or PMLs (Estimated Maximum Losses, Probable Maximum Losses). These values are critical for an insurance company in view of purchasing adequate reinsurance protection. The loss frequency curve, the final output of the system, provides the answers to such questions (see figure 3). On such a curve one may read yearly annual probabilities of reaching or exceeding given thresholds of losses, calculate the pure risk premium (the integral of the curve) for whole portfolios and price reinsurance treaties (non-proportional catastrophe covers). It is therefore an invaluable tool in the assessment and rating of the earthquake hazard.

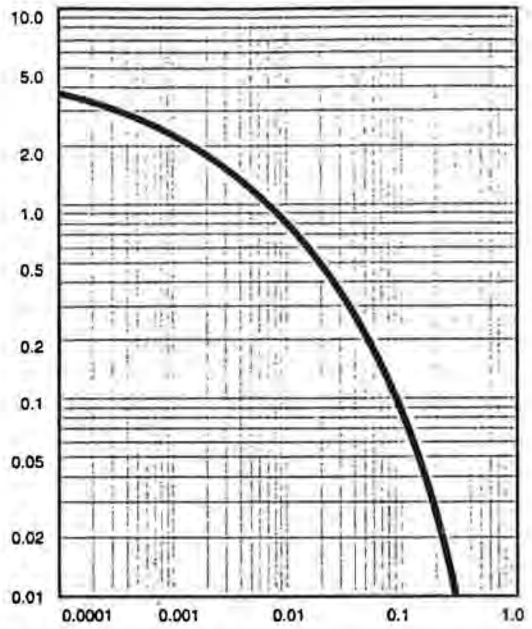

Figure 3: **Example of a loss frequency curve: Loss (% of value) in function of annual exceedence frequency.**

Conclusion

Cover for earthquake damage is no simple matter for the insurance industry. There are still many open questions with regard to: **a)** the seismic hazard and the fluctuations inherent with rare events, **b)** the response of building stock to tremors and **c)** the influence/adequacy of building codes. Earthquake tariffs have to consider all individual risk factors but, at the same time, be simple enough to be usable. The scope of cover and the loss reserves necessary to cater for future disasters are other important topics of discussion. We are confident that, as knowledge grows and communication between the experts in the different fields improves, insurance for earthquake hazard will become less of the shaky business than it has been up to now.

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EARTHQUAKE RESPONSE OF UNREINFORCED MASONRY

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Dr Corderoy studied engineering at the University of NSW where he received his PhD in Structural Engineering. Shortly after graduation he joined Ove Arup and Partners working in both their Sydney and London offices over a five year period. During this time he designed several multi-storey buildings in Sydney and London. He joined the NSW Institute of Technology in 1972 as a Lecturer in Structural Design and became Sub-dean of the Faculty of Engineering in 1982. Dr Corderoy was admitted to the Bar of the Supreme Court of NSW in 1978. He practised full-tine at the bar in 1979 and parttime for a year after that, specialising in cases involving building defects and professional liability.

He joined the Queensland University of Technology in 1984 as Dean of the Faculty of Engineering., In 1991 he became the Dean of the Faculty of Built Environment and Engineering. Dr Corderoy has published seven monographs and over thirty papers on the behaviour of reinforced and prestressed concrete, the design of tall buildings, engineering law and engineering education. He is currently a Professor of Engineering and the Pro-Vice-Chancellor (Research and Advancement) at QUT.

Paper No. 7

EARTHQUAKE RESPONSE OF UNREINFORCED MASONRY

Yan Zhuge, David Thambiratnam and John Corderoy

1. INTRODUCTION

Masonry is commonly used throughout Australia for the construction of low rise buildings, eventhough masonry buildings suffered extensive damage during recent earthquakes. It was evident from the 1989 Newcastle earthquake, that eventhough unreinforced masonry (URM) was extremely vulnerable to earthquake forces, with proper design, it could withstand moderate earthquake loading satisfactorily.

In Engineering literature, reference to seismic/dynamic analysis of masonry has been rare. with most of the existing studies being experimental in nature[(1), (2)]. In analytical investigations, a single degree of freedom model has been commonly used to study the global behaviour of a building system[(3)]. Local effects of masonry walls, such as cracking, crack propagation, crushing etc. cannot be represented by this model. A two-dimensional finite element model has been used by some investigators to study the cyclic behaviour of masonry [(4)], under pseudo dynamic loads. There is no reference to time history analysis in the literature.

Masonry is a two-phase material and its properties are therefore dependent upon the properties of its constituents, the brick and the mortar. The influence of mortar joint as a plane of weakness is a significant feature and this makes the numerical modelling of URM very difficult. In order to predict the complex behaviour of URM walls under in-plane earthquake excitation, a comprehensive finite element analytical model has been developed by the authors[(5)]. The model is capable of performing both static and time history analyses of masonry structures and has been calibrated by using results from experimental testing. In this paper the response of URM to earthquakes is treated, using the analytical model, and the effects of important parameters are studied. It has been found that URM has substantial deformation capacity after cracking, if it is designed to function under appropriate compressive stresses and that it can adequately withstand moderate earthquakes.

2. MATERIAL MODEL FOR MASONRY UNDER DYNAMIC LOADS

2.1 Orthotropic Material Model for Brick Masonry

The non-linear behaviour of URM under in-plane loading is caused by two major effects: cracking/crushing of the material and the non-linear deformation characteristics of the masonry constituents. All these effects are considered in the orthotropic constitutive relations developed by the authors[(6)] and can be used both before and after failure of the material. The proposed model uses the concept of "equivalent uniaxial strain" where the elastic moduli for the constitutive laws are the elastic moduli in the two principal stress directions determined from the uniaxial stress-strain curve[(4)]. An exponential equation, derived from experimental data[(7)], is used to describe the non-linear stress-strain relation.

2.2 Simplified Failure Envelopes for Masonry

Masonry is a non-homogeneous material in which it is not possible to attribute failure to a single cause. Many types of failure are possible and the one that gives the lower bound is the critical one. A failure envelope has been developed which is capable of predicting both joint sliding and cracking and/or crushing type of failure for a homogeneous material model. The effect of bed joint orientation has been considered in the model (Figure 1)

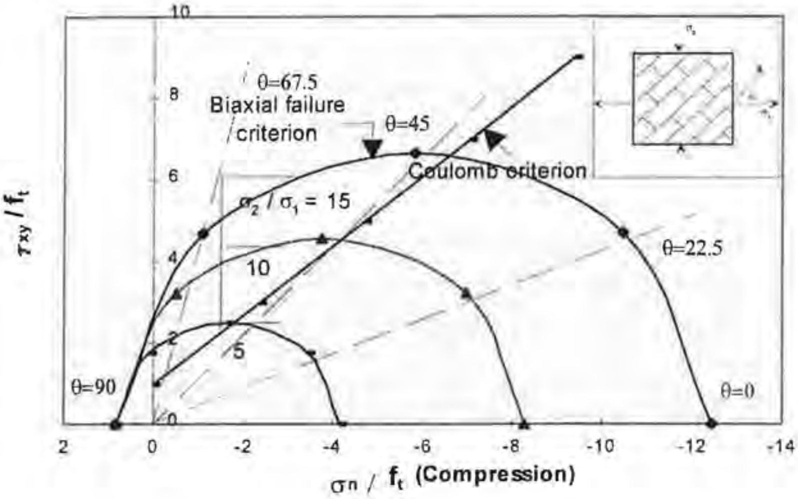

Figure 1 Failure envelopes of brick masonry

3. NONLINEAR DYNAMIC ANALYSIS

In order to perform non-linear dynamic analysis, the constitutive law developed for monotonic loads is extended to allow for reversed cyclic loads and the analytical model developed for carrying out time history analysis of URM under seismic loads.

3.1 Solution of dynamic equilibrium equations

The governing equation for the dynamic analysis of a structure is given by:

$$[M]\{\ddot{U}\}+[C]\{\dot{U}\}+[K]\{U\}=\{P\} \tag{1}$$

where [M], [C] and [K] are the mass, damping, and stiffness matrices; {P} is the load vector, derived by multiplying the mass matrix by the appropriate ground acceleration, and $\{U\}$, $\{\dot{U}\}$ and $\{\ddot{U}\}$ are the displacement, velocity, and acceleration vectors. The mass and stiffness matrices can be derived by using Lagrange's equations(5). Non-linear dynamic analysis of equation (1) is carried out with the modified Newton-Raphson iteration scheme in conjunction with the Newmark direct time integration algorithm.

3.2 Tensile Unloading/Reloading Curves for Masonry

For cyclic loading, Figure 2 shows typical elastic and secant type of unloading or reloading options. For elastic unloading, the crack closes immediately upon a strain reversal, whereafter further strain-decomposition is terminated and elastic behaviour is resumed (line B→E). For secant unloading (line B→O), the stress follows a straight line back to the origin. The crack normal strain is reversible and upon reaching the origin of the diagram, the crack truly closes and ε_n=0, whereafter elastic behaviour is recovered [(8)].

The tensile unloading/reloading constitutive relations can be defined as follows: before the peak ($\varepsilon_{iu} \le \varepsilon_{cr}$), the elastic modulus of masonry is assumed for the linear unloading/reloading path; the secant unloading/reloading model is used beyond the peak, as it provides a better approximation to reality. The secant model accounts for the decrease of stiffness with increasing crack opening strain. It is assumed that static tensile stress-strain curve provides an envelope of cyclic curve (see Figure 2). If the strain exceeds ε_{cr}, the crack starts to open and the stress drops along the envelope curve. Assume that unloading starts at point B, where $\varepsilon=\beta\varepsilon_{cr}$, and β ($1 \le \beta \le a$) is defined as an unloading parameter which can be determined by calibrating the finite element model with experimental results. The reloading path follows the same secant modulus as unloading, back to point B and proceeds along A-C up to the point of next unloading.

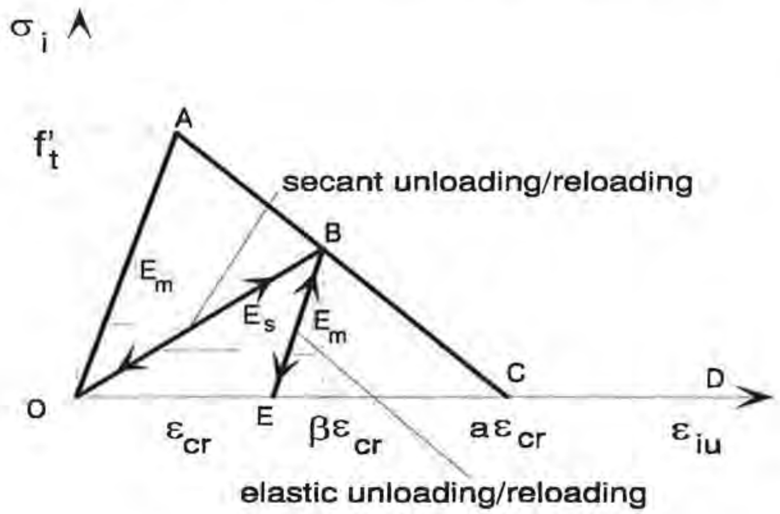

Figure 2 Tensile cyclic softening model

The smeared crack model has been used and the possibility of crack closing and opening and the formation of secondary cracks has been considered. Cyclic tests carried out elsewhere[(5)] have shown that under reversed loading, reopening and closing of cracks may take place repeatedly and this feature is incorporated into the analytical model. When unloading starts after cracking, the constitutive matrix $[D_f]$ can be replaced by:

$$[D_f]=\begin{bmatrix} E_S & 0 & 0 \\ 0 & E & 0 \\ 0 & 0 & \beta G \end{bmatrix} \qquad (2)$$

where E_s is the secant modulus of the unloading branch, as shown in Figure 2 and E and G are the Young's modulus and the shear modulus respectively. It is also assumed that (a) during unloading if σ_i becomes zero, the crack is considered to be closed and elastic behaviour is recovered and (b) during reloading a crack reopens and follows the same path as unloading when the stress in the direction normal to the crack becomes positive.

4. EXPERIMENTAL VERIFICATION

The proposed analytical model was implemented in a finite element program and verified by comparing the results with those obtained from experiments on URM (single-story) walls conducted at the University of Adelaide[9].

Sinusoidal base motion was used to simulate earthquake excitation. The frequency of the base motion was kept constant while the amplitude was increased until the specimen was deemed to have failed. In all tests, failure occurred by rocking of the panels when they separated from the reinforced concrete bases. Testing was stopped at this point[2]. The material properties are summarised in Table 1. Detailed comparison of analytical and experimental results can be found in[5] and in the present paper, results from the analytical model are compared only with the test results of wall 5. The fundamental period of vibration of the uncracked wall is 0.081 sec with an aspect ratio of 1.58.

Table 1 Material properties of the wall

f'_m (MPa)		f'_t (MPa)		E_0 (MPa)		ρ (kg/m^3)
Brickwork	Mortar	Brickwork	Mortar	Masonry	Mortar	Masonry
8.8	2.9	0.0	0.0	1404	1000	1937

Damping was assumed to be 7% of critical damping, based on existing information and a time step of 0.001 sec was chosen in the time history analysis. Predicted and experimental results for wall 5 are presented and compared in Table 2 for different values of tensile strength and the unloading parameter β.

Table 2 Comparison of results for wall 5

Max. base acceleration (g)	Absolute Max. top displacement (mm)						
	Experiment	Analysis					
		f_t'=0.3MPa		f_t'=0.4MPa		f_t'=0.5MPa	
		β=2	β=4	β=2	β=4	β=2	β=4
0.343	0.024	0.020	0.020	0.020	0.020	0.020	0.020
0.616*	0.103	0.422 failed	0.673 failed	0.137	0.221 failed	0.143	0.509

* The wall failed immediately after the acceleration increased beyond 0.616g.

It can be seen from Table 2 that the present analysis provides good results with the combination of f'_t=0.4MPa and β=2. Linear and non-linear (for f'_t= 0.4MPa) time histories of the horizontal displacement at the top of the wall are shown in Figure 3.

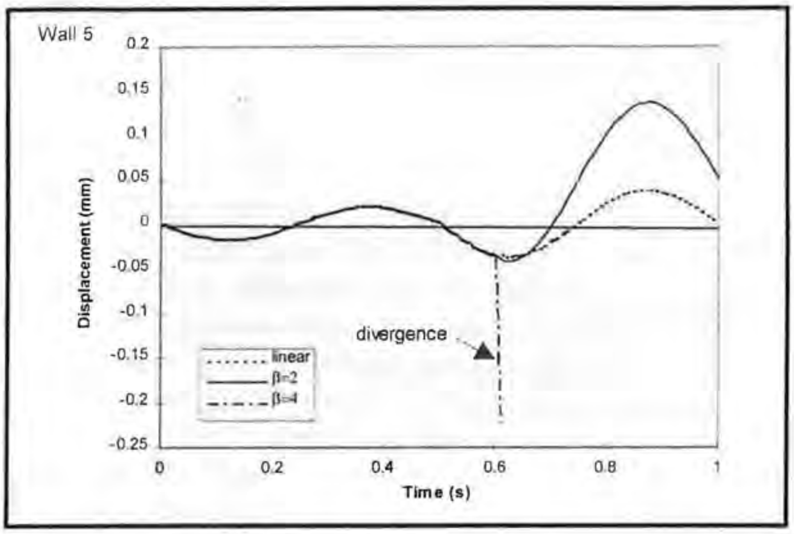

Figure 3 Comparison between linear and non-linear analyses

It can be seen that after cracking initiated at 0.59sec, the maximum top displacement increased significantly for the non-linear analysis (β=2). When β=4 was used, more cracks developed quickly causing large displacements and failure at 0.62 sec. When β was set to 2, the wall had substantial deformation capacity after cracking and did not fail until 1.08 sec, indicating the importance of the unloading parameter β.

The horizontal displacements at the top of the wall with different values of tensile strength are shown in Figure 4. It can be seen that, as expected, the tensile strength has a significant effect on the structural behaviour of masonry as the failure is dominated by the cracking of the material. When f'_t= 0.3MPa, the wall failed a few seconds after cracking. At f'_t= 0.4MPa and 0.5MPa, the wall had substantial deformation capacity after cracking.

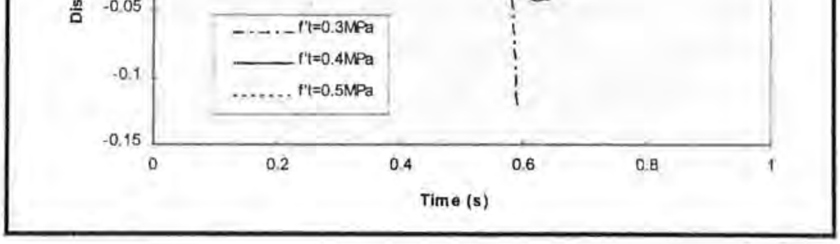

Figure 4 Effect of tensile strength

The cracks initiated at both ends of the wall and developed along the interface between the bottom mortar and the concrete base. The cracks grew gradually and final failure was by rocking, agreeing with the experimental observation[5].

6. CONCLUSIONS

An analytical model developed by the authors has been used to study the behaviour of URM under in-plane seismic loads. A simple secant type unloading/reloading curve is adopted for masonry under tension and the unloading parameter β was determined by calibrating the finite element model against experimental results. A smeared crack model was used for the crack modelling and the possibility of crack closing and opening and the formation of secondary cracks was also considered. Analytical results were compared with those from experiments and showed good agreement. It has been found that the unloading parameter β has a significant effect on both the crack development and the ultimate strength of the URM wall and that a linear analysis underestimates the deflections of the structure after cracking occurs and cannot predict the true dynamic response of URM.

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THE FUNDAMENTALS OF AN EARTHQUAKE STANDARD

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ABSTRACT:

This paper proposes a rational framework for an earthquake design standard which could be adopted and applied internationally. Initially the standard is expected to be used within New Zealand and Australia as one component within a suite of joint structural standards which would enable design and construction practices to be applied either side of the Tasman. The framework could be easily adopted by other countries because it addresses both inter- and intra- plate earthquakes. Allowance is made for national variations to address varying levels of seismicity and different national performance expectations. The author outlines his views on the objectives and expected contents of the essential elements of an earthquake loading standard which are currently being examined in detail by a Standards Australia and Standards New Zealand working party.

Paper No. 8

2. INTRODUCTION

Over recent years, since the New Zealand economic revival, New Zealand industry has accepted the challenge of international trade across all sectors of our economy. Nowhere more so than in the building industry where building products, design and construction technology, and New Zealand technical expertise are being applied within Australia, Asia and the Pacific Rim. Advances in telecommunication and computer technology make New Zealand less isolated and more able to participate, and favourably influence, development of sound building control codes and standards. This is occurring at a time when both the technical and financial resources required to develop new, or even maintain existing, structural design standards have become increasing difficult to obtain within New Zealand.

The logical solution to this dilemma is to be influentially involved in the development of common standards which can be used both within New Zealand and within the borders of our trading partners. It is important that this involvement is active to ensure that the high performance expectations contained within our existing structural design standards are not compromised.

Earthquakes dominate the structural design process within New Zealand. Consideration of both post-elastic, ductile response and capacity design principles are commonplace. This reflects New Zealand's moderate to high seismicity. This is in distinct contrast to structural design in Australia where most regions are of moderate to low seismicity and do not merit such considerations. While most Australian buildings provide lateral resistance for wind loads, post-elastic response and structural ductility are seldom specifically considered or provided.

Nowhere is New Zealand's technical expertise more widely acknowledged than in the area of earthquake engineering. This paper proposes a rational framework for an earthquake design standard which could be adopted and applied internationally. Initially, and in accordance with the New Zealand - Australia free trade agreement, the standard is expected to be used within New Zealand and Australia as one component within a suite of structural standards which would enable design and construction practices to be applied either side of the Tasman. The framework is sufficiently robust that it could be adopted by other countries because it provides for both inter- and intra- continental earthquakes. National variations would be required to account for varying levels of seismicity and different national performance expectations.

In this paper, the author outlines his views on the objectives and expected contents of the essential elements of an earthquake loading standard. Three major issues within the earthquake design process are identified and discussed in detail in the following sections. The first issue is the basis for determining the ground motion expected during an event with an appropriate recurrence interval (as required by each national building code) for each region within the coverage region of the standard. The second issue is the adjustments to the ground motion which are required for the structure to meet the performance expectations of the community. The third issue is the means of translating the adjusted ground motion into the dynamic response of the structure.

The elements described in this paper are currently being examined in detail by Standards Australia and Standards New Zealand working party BD6/4. It is expected that the conclusions of the working party will be presented in the Bulletin in the near future.

3. DEVELOPING A JOINT EARTHQUAKE DESIGN STANDARD.

Many key issues need to be considered regardless of the level of seismicity present. By focusing specifically on New Zealand and Australian requirements, aspects have been identified which require national variations. These variations reflect differences in the performance expectations the two societies have of their building stock along with differences in construction and design practice and the reliability of the structures which result. Ideally, national variance factors would be provided as a means of attaining stated target safety indices under each load combination which needs to be considered. As such they may differ with different load cases and different load ratios, but should be similar for constructions of different materials and structural forms.

A schematic representation of the earthquake design process is given in Figure 1. The items within this schematic are numbered in accordance with the subsequent sections of this paper. For reference, the key elements of each of these items are summarised in Appendix A for several international earthquake standards.

Each item in the following sections of this paper identifies its objective, (typeset in *italics*), discusses the issues involved (in normal text) and anticipates the outcome which may evolve from detailed considerations ***(in bold italics).***

4. PRESCRIPTION OF THE DESIGN EARTHQUAKE GROUND MOTION

{Objective: To prescribe the dynamic response characteristics of any site to earthquake ground motion such that the hazard resulting from that motion is uniform and has a probability of excedence consistent with that demanded by the national building code.}

4.1. Site Response

{Objective: To prescribe the dynamic response characteristics of rock sites anywhere within each seismologically unique region that the design standard is to cover.}

Earthquake induced ground motion is usually codified either as a response spectra, being a plot of one parameter of motion (ie. displacement, velocity or acceleration) against the response period, or as a plot of actual motion against time. Response spectra are the most convenient form and acceleration response spectra the most common form for publication within earthquake design standards because lateral forces can be directly derived from acceleration by multiplying it by the seismic mass present. There is however, an increasing demand that the overall building performance be considered as the basis for design. Force based design procedures are not particularly suitable for establishing either the onset of damage nor other displacement related performance criteria. There is thus an increasing trend towards using displacement spectra as the basis for design. Thus it may be expected that in the future, both acceleration and displacement design spectra will be published within the earthquake design standard. It is also possible that a suitable time history record (or suite of records) will be published that represents the same ground motion which formed the basis of the design spectra. Any of the above methods of representing the site ground motion are legitimate, but any methods used should be consistent and transparent between each method.

Priestly (1993) outlined a displacement design method which can be used as the basis of codified design. Figure 2 indicates the character of typical acceleration and displacement

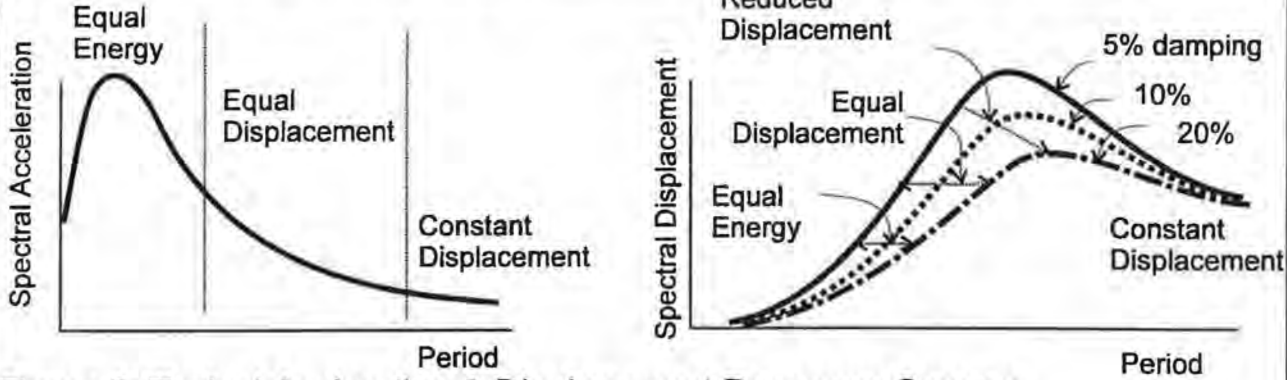

Figure 2: Typical Accleration & Displacement Response Spectra (from Priestley 1993)

response spectra prepared in this manner.

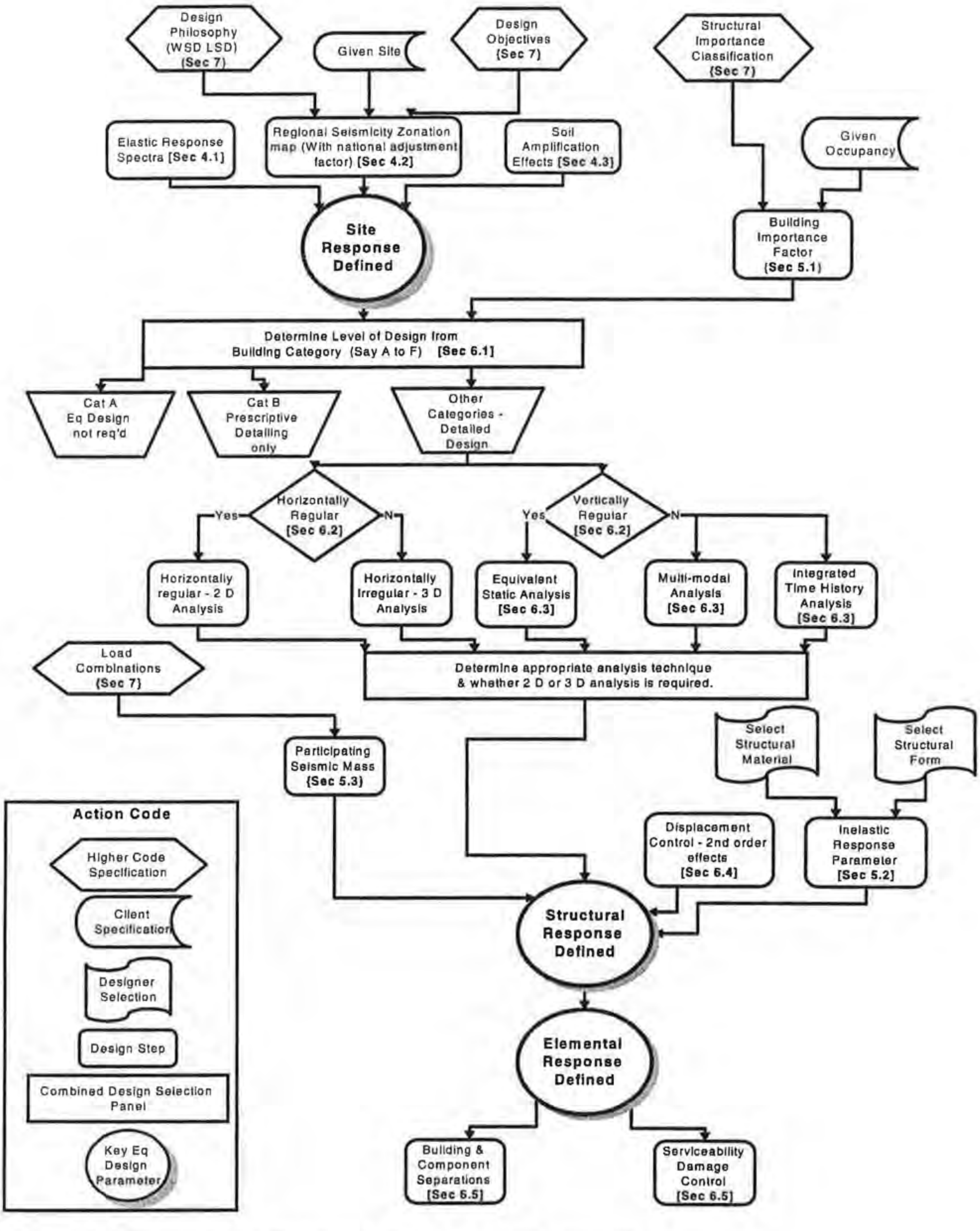

Figure 1: A Structure for a Common Earthquake Design Standard

Alternative approaches, either being developed or in current practice, include using a combination of acceleration and velocity spectra as the basis for design and for assessing seismic hazard. Canada uses this approach to determine their seismic hazard potential across the country from which the level of earthquake design is determined (refer section 6.1) as outlined by Heidebrecht (1995).

When developing a common earthquake design standard for both interplate (moderate to high seismicity) zones such as New Zealand, and intraplate (low to moderate seismicity) zones such as Australia, response spectra could be published for each seismologically similar region. The basis upon which each is developed should be common. The significance of the nominated return period used to normalise the intensity of the 'design event' will be of greater significance within zones of low seismicity where the ratio of design intensity to maximum credible ground motion may be in the range of 2 to 3, (cf 1.5 to 2 for zones of high seismicity). Thus the design parameters may be highly sensitive to the return period nominated as the 'design intensity' within such zones, yet insensitive within areas of greater seismicity.

Building damage is also acknowledged as being a function of the duration of high intensity excitation. The large magnitude design event associated with zones of high seismicity, can be expected to generate high intensity shaking which will continue for between 20 to 45 seconds (cf 5 to 10 seconds duration for the design event within zone of low seismicity). This relates directly to the number of inelastic excursions (and thus accumulated damage) the building may be expected to experience. Thus the duration of intense ground motion is an important parameter when assessing building damage but is yet to be satisfactorily codified.

Markedly different ground motion attenuation relationships may be expected between New Zealand (with its highly disturbed rock strata of relatively low shear modulus and shear wave velocity) and Australia (where the rock strata is largely undisturbed and the shear wave velocity is relatively high). Such effects will need to be incorporated into the development of the uniform hazard spectra for each country if this approach is to be used. There is currently a dearth of data which may preclude the use of this approach in the short to medium term. A rational basis for deriving a design spectra for vertical response will also be required for both New Zealand and Australia to enable the dynamic response of horizontally spanning elements (cantilevers, floor span members etc.) to be assessed.

[Anticipated Outcome: Within a future earthquake design standard, the nominal site ground motion is likely to include both an acceleration and a displacement design spectra for each seismological zone. The peak intensity of any such spectra will probably be based on a uniform risk approach where the intensity of ground shaking relates to an event with a return period of that nominated for the specific limit state under consideration.]

4.2. Regional Seismicity (Seismic Zonation Maps)

{Objective: To prescribe the a suite of scaling constants by which the normalised design spectra can be related to expected ground motion within zones of differing seismicity}

A method is required by which the nominal elastic response spectra prescribed in section 4.1 can be modified to reflect regional variations in seismicity. While it would be preferable to apply a single factor across the full period range, this may distort the design spectra and two or more period-related scale factors may be required. Modification factors may be presented as seismic zonation maps (which prescribe scale factors to various locations based on isoseismal style maps of each region). Alternatively one or more spectral ordinate maps may be used, with the resulting period dependant functions be used to adjust a generalised spectral shape.

[Anticipated Outcome: A suite of seismic zonation maps is expected. These will extend over all regions within coverage of the standard. They would need to be prepared on a common basis and be based upon a common recurrence interval. National risk adjustment factors may be introduced

where the consumer expectation, as prescribed by the national building control system, requires a departure from the nominated recurrence interval of the design event.]

4.3. Soil Amplification Effects

{Objective: To prescribe a method whereby the modifying influence of near-surface soil conditions on the nominal response can be ascertained.}

The influence that near surface soil strata have in modifying the transmission of the energy waves released from the rupture plane must now be considered. Such effects are usually local to each site and are intended to reflect modifications which may occur to the energy shear waves released from the rupture surface as they pass through the surface layers. Such effects may either result in the amplification or attenuation of ground motion with factors of between 0.67 and 2.5 being typical.

Two common means of allowing for such effects are found within the mainstream overseas standards. A soil amplification factor is used in the Uniform Building Code (ICBO 1992) and Australia which has the effect of linearly scaling the normalised response spectra across the full period range. The alternative approach, adopted within the current New Zealand standard and the draft of European standard, assigns period dependant reference points that modify the overall spectra shape. This latter approach provides greater opportunity to realistically represent the actual site response.

Typically rock or stiff soil sites have greater short period response (ie greatest energy within the 0 to 0.4 second period range), but experience more rapid decay and lower long period response (refer Figure 3). Conversely, soft soils may filter (attenuate) some short period response, resonate and therefore amplify excitation when the basement excitation is sympathetic with their own natural period, and have little effect on longer period response. Thus a broad elevated response plateau should be expected about this natural period of the subsoils beneath a specific site. Unfortunately, within the broad parameters used to prescribe the site conditions, it is seldom practical to precisely identify this natural period of the site and therefore the period range whereby amplification can be expected. In such cases, an extended spectral plateau has been introduced to cover the amplification zone. This may often be narrowed with site specific evaluation.

In any regard, it is essential that the end-user (ie the structural engineer) is able to accurately determine which soil conditions should be applied to any given site. Thus, although shear wave velocity may be the actual site parameter which needs to be considered, a fuller description of geotechnical properties (such as depth, density, cohesion etc.) may be necessary to minimise the designer uncertainty.

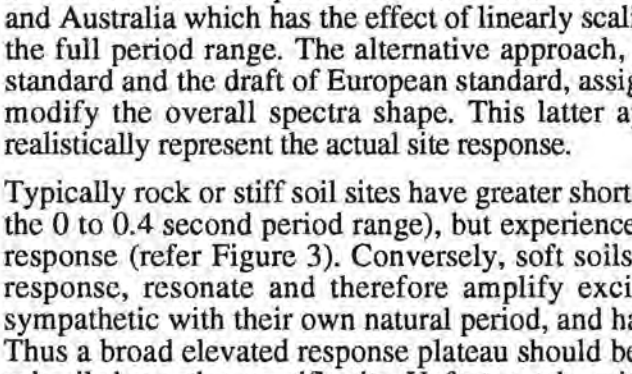

Figure 3: Site influence on spectral response

Microzonation effects are caused by a combination of unfavourable, often subterranean, features which focus (or disperse) energy as it is transmitted to the surface from the rupture plane. Since such features are particularly site specific, there is at this time no realistic means by which these effects can be included within design standards. Specific site investigations are required to for the such features to be identified and quantify such effects.

[Anticipated Outcome: Soil modification effects will need to be included. A simple soil related response magnification factor would be preferred, but only if there is confidence that

the response spectra is unaffected by the soil effects. Publication of soil specific elastic response would otherwise be acceptable.]

5. STRUCTURAL RESPONSE TO EARTHQUAKE MOTION

{Objective: To prescribe procedures and principles by which the response of a specific structure can be assessed from the elastic site response spectra prescribed above.}

The dynamic response at a given site has been prescribed within section 4. Rules must now be developed to enable these characteristics to be translated into a design response spectra for a particular building which is to be founded on that site.

The proposed use and occupancy class of the building will affect the mass distribution within the building and often its layout. The selection of the structural form, the materials to be used and the acceptable level of inelastic response all combine to determine the dynamic response of the building.

5.1. Building/Occupancy Importance (Risk Adjustments)

{Objective: To adjust the design response to allow for differing performance expectations associated with the type and importance of end-use}.

Modern building codes require that collapse be avoided under severe earthquake attack and that functionality (ie before the threshold of damage requiring repair) be maintained when subjected to an intermediate (or serviceability) intensity earthquake. These requirements apply to all buildings, although they may differ from one society to another (and could be adjusted by a National Performance Factor if required). It is common practice however to provide a greater level of security for 'special' buildings. These may be key post-disaster facilities that are required to remain operational following earthquake attack (eg hospitals, emergency management facilities, prisons, etc), or buildings which house national treasures or large crowds (whereby collapse is to be prevented and the contents protected, but subsequent occupancy may not be essential). Heritage buildings require protection against damage to their fabric and provide a special challenge. There is also an increasing trend for businesses to be aware of economic losses associated with post disaster decommissioning of plant and facilities and incorporating preventative measures within the plant design.

The simple approach of scaling the design response spectra according to the building importance is the method adopted within the current New Zealand and Australian earthquake codes. Such an approach, while providing greater capacity against collapse and perhaps postponing the onset of damage, does not address the particular performance expectations which may be appropriate for each building. There is an increasing trend overseas to provide base-isolation for essential facilities as a means of actively controlling the level of earthquake excitation which can be reasonably imparted to a building. Where internal damage control is the essential parameter, deformation control or the provisions of securing high elevation elements against collapse may be more appropriate. There is little doubt that the simple magnification of the design spectra, although a traditionally convenient tool, is due for review.

[Anticipated Outcome: Guidance is required as to how particular performance objectives are best achieved, if necessary by limiting the acceptable design solutions for specific 'special' buildings. While a simple building importance multiplier may be appropriate, a more refined method of matching specific performance expectations with particular control measures or constraints would seem timely.]

5.2. Structural Form and Inelastic Response

{Objective: To establish the inelastic dynamic response of the structure following selection of the structural materials and structural form to be used.}

The elastic response spectra provides dynamic response characteristics for a particular site. Building damage in rare, major events, is regarded as acceptable provided collapse is avoided. This allows the designer to select an appropriate levels of ductility (inelastic response) for each structure. While this decreases the lateral shear force which is required to be applied for design, it increases the interstorey inelastic displacement. This places greater demands on clearances necessary for secondary components and building contents. The level of damage expected from highly ductile frames is expected to be greater, with the consequence of large post-disaster disruption being expected.

Once the principle of inelastic behaviour has been adopted (usually as an expedient means of reducing construction cost and permitting greater design freedom), a method must be instigated by which the inelastic design spectra for each particular structure can be derived from the site response spectra (as prescribed in section 4 above). The two most common methods are to use a combination of structural ductility and redundancy (the $\mu \sim S_p$ combination within the New Zealand code (SNZ 1992) and the structural behaviour factor, q of Eurocode 8 (ENV 1994)) or alternatively to use a structural response factor, R_f (the UBC (ICBO 1992)/ Australian (SAA 1993) approach). Both q and R_f are period independent and are therefore inverse scaling factors of the site response spectra. The various inelastic response spectra published within the New Zealand standard introduce period dependency with equal energy concepts being applied to short period structure and equal displacement to long period ones (with a transition between).

In each case, the ability of the structure to sustain levels of inelastic response depends upon the material and the detailing employed. They are material parameters and as such should be prescribed within the seismic provisions of the material design standards together with the detailing requirements which are needed to attain the level of ductility assumed.

An important issue is the ability to rationally verify any such reduction factor by test (or other means) to ensure consistency between materials and to allow new systems to be introduced. To facilitate this, there is an increasing trend internationally to derive the inelastic response reduction factor from several material related variables. In New Zealand the ductility factor, μ, is used to reflect the structural ductility present and the structural performance factor, Sp (= 2/3), is used to adjust the inelastic structural response spectra to reflect the building damage associated with several inelastic excursions. SEAOC have proposed changes to the UBC (Phillips and Hamburger 1995) for the 1997 revision which, if adopted, will split the R factor into two parts, namely a ductility reduction component, R_d, and an overstrength component, R_o. EC 8 (ENV 1994) goes further with a basic behaviour factor, q_o, (ranging from 2.5 to 5) and three modification factors associated with the ductility class, k_d (1 to 0.5), a regularity function, k_r, (1 or 0.8), and a wall modifier, k_w, which is used to address the prevailing failure mode of wall systems.

[Anticipated Outcome: A quantifiable inelastic response reduction factor will be required. This will be material and structural form dependent and is likely to be composite in character. The structural behaviour factor contained within EC8 offers a good starting point for committee considerations, although the emphasis on energy dissipation may require further consideration.]

5.3. Seismic Mass

[Objective: To prescribe the mass of the building and its contents which should be considered as being present and active during an earthquake attack.]

The use and occupancy characteristics of a building will usually be used as the basis for determining the live load, Q, which should be considered as present within that structure at various periods during its intended life. The seismic mass assumed present at the time of an earthquake attack is generally the self weight of the structure plus the arbitrary-point-in-time occupancy load, Q_{APT} (ie that intensity of live load which has a 5% probability of being exceeded at any time during that expected life). Q_{APT} should be prescribed within the general section of the loading standard. For NZ (SNZ 1992) and Australia (SAA 1989) this is set at 40% of the nominal live load specified.

[Anticipated Outcome: The participating seismic mass, and its relative distribution within the structure will need to be prescribed. The intensity of this load may be prescribed within the general loading provisions rather than specifically for earthquake considerations. The reduction factor which relates the probability that mass of this intensity will be applied over all of the contributing area may be specific for earthquake considerations.]

6. TECHNICAL PROCEDURAL ISSUES (ANALYSIS AND DESIGN LIMITATIONS)

Clauses within this part focus on the design and analysis procedures which are appropriate for earthquake design within regions of differing seismicity and the attainment of particular earthquake related performance expectations. It sets some limits within which simplified procedures may be applied and beyond which the implicit assumptions made in preparing such alternative methods are no longer valid.

6.1. Design Application (From 'doing nothing' to capacity procedures and beyond)

[Objective: To prescribe a means by which design simplification procedures may be confidently applied while retaining an appropriate level of confidence that building performance expectations will be attained.]

The necessity to design for earthquake loadings within areas of moderate to high seismicity is seldom challenged. Traditionally, such a consideration has been ignored in areas of low seismicity. Well designed buildings have been shown to perform satisfactorily in both environments. The challenge in this section is to define appropriate limits which will direct the designer to make provisions for, or to require specific earthquake design to be executed, whilst not demanding that complex earthquake design or construction procedures be imposed unnecessarily.

Such limits should be identified early in the design process so as to avoid unnecessary design effort. The Australian earthquake code sets four levels of design, namely making no specific provisions, imposing simple detailing requirements, using simple equivalent static design and using dynamic design. The boundaries of each level are based upon the building category (which is a function of the peak ground response expected at the site including soil amplification effects and the structural performance expectations) and the inherent resilience (ductility) implicit within the structural system being employed. The implication that dynamic design procedures will result in better building performance are open to question, and it may be expected that in future standards the limitations will be directed more to achieving the particular performance expectations which are prescribed.

[Anticipated Outcome: Guidance will be provided which relate the site seismicity to the level of performance expected with a view to ensuring that specific earthquake resistant provisions are in place where necessary but can be avoided where the inherent capacity of the building is in excess of that prescribed for the nominated design return period.]

6.2. Distribution of Earthquake Induced Lateral Loads

[Objective: To provide a means whereby the base shear induced by the inertial effect on the seismic mass is rationally and appropriately distributed up the height of each of the vertical lateral load resisting systems and between these systems so as to allow for torsional effects].

This section addresses the effects of building irregularity (both vertical and horizontal). Limits are required to specify where the simplifying assumptions are used as the basis of the equivalent static design procedure, and where the application of two dimensional design is no longer valid.

Higher mode effects are often caused by abrupt changes in structural stiffness up the height of the structure. Considerable simplifications can be made within the design process if the structure is **vertically regular** in which case the first mode can be reasonably assumed to dominate. Vertically irregular structures, particularly those of moderate or greater height, must specifically consider higher mode effects (using multi-modal or integrated time history analysis techniques — see section 6.3).

The distribution of total base shear between vertical lateral load resisting system and the influence of torsional twist within the structure is a function of its horizontal regularity. When **horizontally regular**, accidental eccentricity need only be considered to amplify the lateral loads of the outside lateral load resisting systems. The analysis can be confined to two dimensions in such cases. For horizontally irregular structures, three dimensional analysis or a greater torsional allowance is required. Preparing clear and unambiguous guidelines to access acceptable levels of horizontal regularity is difficult.

[Anticipated Outcome: Regularity, both vertical and horizontal, remains a key issue in accurately predicting the earthquake response of a structure. Guidance will be required as to acceptable regularity limits for which simplifying assumptions with respect to the vertical distribution of load apply, and when 3 dimensional analysis is required to address torsion induced by horizontal irregularity.]

6.3. Methods of Analysis

{Objective: To prescribe limits within which various methods of analysis can be used to ascertain the earthquake load effects within the structure and to give guidance as to the various analysis methods which should be applied and their results interpreted.}

6.3.1 Equivalent Static Method

The equivalent static method of analysis may only be used to analyse simple regular buildings. The dynamic response of such systems is only used to distribute the total base shear within each frame (usually with an inverse triangulated load pattern), together with an amplification factor to allow for torsional effects. The system is then designed to resist the static loads induced from this distributed load pattern superimposed upon those arising from self weight and long term occupancy loads. Members may be sized and detailed according to the resulting actions, or, if capacity design is required, as a basis for determining the flexural member capacity and the overstrength capacity which is used to ensure the appropriate hierarchy of strength attained.

[Anticipated Outcome: The equivalent static method of design should still be the preferred option for simple structures. The transparent nature of the analysis provides less likelihood of gross errors being introduced. Guidance will be required as to the appropriate sectional properties (stiffness and member end zone allowance), how accidental torsional effects are to be incorporated, and how deformation compatibility can be assured.]

6.3.2 Multi-Modal Analysis Method

When more accurate representations of the actual excitation modes of the structure are required, either because of vertical irregularity or for other reasons, the multi-modal method of analysis can be employed. This method determines the actual period and mode shapes of the building which are used, in conjunction with the inelastic response spectra, as the basis for distributing base shear. The contribution from each mode is combined usually using a sum of squares combination technique. The actions derived more realistically represent the building response. Since the contribution of each mode will not occur simultaneously, the means by which they are combined requires careful attention. (Refer Carr 1994)

[Anticipated Outcome: Multi-modal analysis procedures will need to be specified. Guidance is required as to methods of combining the resulting

modal effects, with particular reference to the time dependant nature of the resulting action envelopes. Methods by which post-elastic response can be assessed would be helpful.]

6.3.3 Integrated Time History Analysis

Integrated Time History analysis techniques, although still only being used for major structures, are more widely available with the advent of more powerful, cheaper computing. The process involves developing a computer model of the structure, usually including both the elastically responding characteristics and the post-elastic characteristics of each element. The resulting model, together with the appropriately distributed seismic mass, is subjected to the synthesised ground motion for each time step of the ground motion being considered.

Issues which require inclusion in any such analysis are: the dynamic character and scaling effects of the ground motion, the dynamic interaction between interconnected lateral load resisting elements, and the actual inelastic response of the structural components being modelled. Techniques such as these are most commonly used as checking tools once the structural form has been proportioned in accordance with other methods.

[Anticipated Outcome: Integrated time history analysis techniques have become widespread in their use a means of establishing the onset of damage and thus control within a structure. As such, guidance will be needed as to how to select appropriate input records and scaling factors to be applied for a specific site and structural form. Guidance is required as to how to interpret and apply the data derived from such analyses. The resulting data should be incorporated into the design process.]

6.4. Post-elastic Displacement Control

[Objective: To ensure displacements are appropriately controlled, that the onset of damage is appropriately deferred and that building and component separation distances are sufficient to prevent collision.]

The deformation profile of a building, derived from the force based analysis methods discussed in sections 6.3.1 and 6.3.2, provides little useful guidance as to the post-elastic deformations which should be expected during severe earthquake attack. It is usual to merely scale the elastic profile, normally by a factor which relates to the structural ductility assumed. The true post-elastic deformation profile is strongly influenced by the structural form of the building. Framed structures tend to concentrate their post-elastic drift within the lower third of the building (the remainder following an unmodified elastic profile) in contrast with shear wall or cantilever systems in which the amplified elastic deformation approach is more realistic.

Drift control is required under post-elastic conditions to ensure the actions of second-order effects, such as displaced gravity actions (P-Δ effects), remain small. Accurately assessing such drift is notoriously difficult and the imposed limits are somewhat arbitrary. Typical limits of between 1 and 2% of interstorey height have been used.

Separation between buildings and between elements within a building provide a control upon which building clearance and boundary separation are assessed. The potential exists for dynamically unsympathetic responding systems to clash and hammer each other when excited. Impacts of this nature have been observed during recent earthquakes (Mexico 1985) and have resulted in premature structural collapse. Realistic controls on both building and component separation distances will be required within future earthquake codes.

[Anticipated Outcome: Both elastic (serviceability) and post-elastic drift limits are needed. The rationale for the imposition of such limits should clearly identified, even if the technical basis for such controls may be somewhat sparse. Guidance will be required to ensure that magnification of

the elastic displacement profile is correctly applied to obtain the post-elastic displacement profile.]

6.5. Serviceability Displacement Controls

[Objective: To provide earthquake induced dynamic actions from which the design of building components and non-structural elements can be designed to attain their required performance.

Damage to non-structural components and elements has been found to be the most common cause of building closure following an earthquake attack. Avoidance of the premature onset of such damage is becoming an increasingly common performance criterion. The implication of this applies both to the elements themselves and to their interaction with, and connection to, the structure and to each other.

The complexity of defining the dynamic response of the structure and the interaction between the structure and the element or supported component becomes much greater and the uncertainty is much more pronounced when the component is elevated within the building. In such cases, the structural response, within both elastic and inelastic range, will be highly influential on the loading pattern applicable to the component, and thus the dynamic response of that element to such loading.

[Anticipated Outcome: A rational method is needed to enable the design of non-structural building components to both serviceability intensity events (after which they should continue to operate) and ultimate limit state events where damage <u>may</u> be acceptable.]

7. HIGHER LEVEL ISSUES

[Objective: To identify issues which must be specified to enable consistency of approach across the suite of structural design standards but have application or implications beyond the earthquake part. Such issues would be addressed within either the national building codes or the parent loading standard.

The following issues are fundamental to all structural standards within the suite:

- The proposed objective of the design (protection from hazard, neighbouring buildings, national economic loss, etc.);
- The acceptable design philosophies (Working Stress or Limit State Design);
- Definition of structural and occupancy classes; and
- Load combinations will be required for each design state.

8. THE CURRENT STATUS OF THE JOINT EARTHQUAKE CODE

In November 1995, prior to the Pacific Earthquake Engineering Conference in Melbourne, Standards Australia and Standards New Zealand convened a meeting of interested parties to discuss the development of a joint earthquake design standard for New Zealand and Australia. Concern was expressed that too many changes had been imposed on the design profession over recent years and a time of consolidation was required. It was agreed by both Standards organisations that the existing suite of structural codes would remain and be supported for the next 10 years. It was however agreed that the time was right for work to begin on developing a new suite of structural codes, with the earthquake section of the joint loading standard being perhaps one of the more difficult to bring together.

A working party, operating under the name BD6/4, had been established some time previously and was called together in March '96 prior to the Annual NZNSEE conference in New Plymouth. The author was appointed chairman of that working party and, working under the above structure, persuaded a number of participants from both New Zealand and

Australia to form working groups to prepare briefing statements and identify the key issues on each of the topic areas discussed above. This process in now well advanced, and will be reported in the Bulletin in the near future. The timetable for executing the work has yet to be finalised, but is expected to extend over the next three years. Funding remains a considerable impediment and there still appears to be no solution in sight.

9. CONCLUSIONS:

It is quite feasible to prepare a common earthquake design standard for New Zealand and Australia which would be able to accommodate the wide range of seismicity within each country and the level of design refinement present within the respective design communities. Any such standard could be readily adopted by other regions regardless of the level of their seismicity or their stage of social or economic development.

The essential features of most international earthquake engineering design standards have been identified and discussed. Although there are differences in detail, the overall approach is common and with refinements could be readily applied to any future code development.

Earthquake engineering design continues to become more refined and focused on assuring the required structural performance is attained. Several new international initiatives have been identified and should be considered for any future standard development.

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Appendix A

Comparison of issues covered in several international earthquake standards

	ISSUES	AUS[1]	NZ[2]	EC8[3]	UBC[4]	ISO[5]
	SITE PARAMETERS					
4.1	Elastic Response Spectra	normalised $(1.25S/T^{2/3}) <$ 2.5 eqns and graphs	3 (soil category) 2 segments graphs & tables	parametric 4 segments eqns & tables	parametric 3 segments graphs & tables	parametric $\rho_0(T_c/T)^{\eta} < \rho_0$ graphs & tables
4.2	Seismic Zonation map return period	10% in 50 years	10% In 50 years	475 yrs	475 yrs	not specified
4.3	Soil Classification	5 classes	3 classes	3 classes	5 classes	not specified
	PROCEDURES					
5.1	Important factor	1-1.25	1-1.3	not specified	1-1.25	not specified
5.2	Inelastic Structural Response (Seismic resisting system)	R_f (1.5-8.0)	μ and S_p (1.25-6.0, 0.67)	$q = q_0 k_d k_r k_w$ q_0=2-5, k_d=0.5-1 k_r=0.8-1 k_w=0.4-1	$R = R_d R_0$ R_d=1.2-3.4 R_0=1.8-2.5	δ 1/5-1/3 ductile 1/2-1 non-ductile
5.3	Seismic mass	$G + \psi c\, Q$	$G + \psi u\, Q$	$G + \varphi\, \psi 2I\, Q$	$G + x\, Q$	
	DESIGN APPLICATIONS					
6.1	Levels of design Do nothing details only conventional design capacity design displacement design	 fn(aS, bld. type) fn(aS, bld. type) fn(aS, bld. type) not required not covered	 not allowed not allowed μ related μ related ?	 a < 0.04g a < 0.1g a > 0.1g ? ?	 zone 0 if wind control required not required ?	not specified
6.2	Structural regularity criteria	Yes	Yes	Yes	Yes	
6.3	Methods of analysis Equivalent static Modal Time history	 Cat B, irregular Cat C or above, regular Cat D or above, irregular optional	 H<15m regular T<2.0sec irregular T<0.45 s outside equivalent static limits outside equivalent static limits	 (i) regular (ii) limited class with T< 2 outside equivalent static limits outside equivalent static limits		
6.4	Post-elastic Displacement Control	Δ_e (3R/8)	$\mu \Delta_e$	$q_d \Delta_e \gamma_i$	0.7 $R_d R_0$	
6.5	Serviceability control P-δ Damage limit	 m>0.1 0.015 h	 not required for T<0.45s H<15m and T<0.8s μ < 1.5 0.02h for H< 15m 0.015h for H<30m	 m >0.1 0.015h to 0.008h	 m >0.1 0.025h for T<0.7s 0.020h for T>0.7s	
7	Design methods	WSD & LSD	LSD	LSD	WSD & LSD	LSD

[1] Australian Standard AS1170.4-1993. Earthquake Loads

[2] New Zealand Standard NZS 4203:1992 Part 4 : Earthquake Provisions

[3] European Prestandard EuroCode 8 - Design Provisions for earthquake resistance of structures

[4] Uniform Building Code-SEAOC - including the latest changes as approved by ICBO

[5] International Standards Organisation ISO-3010 Basis for design of Structures- Seismic Actions