UNDER	**THE COMMISSIONS OF INQUIRY ACT 1908**
IN THE MATTER OF	**ROYAL COMMISSION OF INQUIRY INTO BUILDING FAILURE CAUSED BY CANTERBURY EARTHQUAKES**
	KOMIHANA A TE KARAUNA HEI TIROTIRO I NGA WHARE I HORO I NGA RUWHENUA O WAITAHA
AND IN THE MATTER OF	**THE CTV BUILDING COLLAPSE**

JOINT STATEMENT OF EVIDENCE OF BRIAN E KEHOE AND TERRENCE F PARET IN RELATION TO THE CTV BUILDING

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JOINT STATEMENT OF EVIDENCE OF BRIAN E KEHOE AND TERRENCE F PARET IN RELATION TO THE CTV BUILDING

1. BACKGROUND AND QUALIFICATIONS

1.1 Our full names are Brian Edward Kehoe and Terrence F Paret. We live in California and are, respectively, Associate Principal and Senior Principal of Wiss, Janney, Elstner Associates Inc (**WJE**), a US firm of structural engineers, architects and materials scientists. Copies of our curricula vitae are attached to this statement of evidence.

1.2 WJE undertakes earthquake damage and seismic risk assessments around the world. Its expertise and our experience include emergency assessments of reinforced concrete and other structures undertaken in the aftermath of earthquakes both in the United States and other jurisdictions, including Turkey, Greece, Haiti, Algeria, Japan, China, El Salvador, Taiwan, India, and Guam. Notable reinforced concrete structures that we have assessed include the San Francisco Airport Hyatt Regency Hotel, the Mauna Kea Beach Resort, the Royal Palm Hotel, and the Los Angeles County Civic Center.

1.3 WJE is contracted by the US Department of State to undertake seismic assessment and strengthening designs for its embassies and residential facilities worldwide and its team of earthquake engineers (of which we are part) provides consulting services to the Federal Emergency Management Agency (**FEMA**) and the US National Park Service (**NPS**). Subsequent to the August 2011 “Mineral” earthquake in Virginia, WJE was engaged to performed damage assessment and emergency hazard mitigation for NPS buildings and other buildings in Washington DC including the Washington Monument, Jefferson Memorial, Lincoln Memorial and the National Cathedral. We have developed construction drawings for repair of the Washington Monument and have just completed a performance-based nonlinear assessment of seismic safety for the Monument for a 2,475 year earthquake. Mr Kehoe participated in the initial damage assessment of the Washington Monument and the Lincoln Memorial and Mr Paret is supervising the seismic assessment of the Washington Monument.

1.4 WJE's engineers have developed or contributed to the development of many of the earthquake engineering investigation and analysis procedures in common use today. These include:

a. WJE engineers were part of the project engineering panel that provided overall reviewing guidance for the development of ATC-20 post-earthquake assessment protocols and developed the ATC-20 training materials used to train engineers to conduct post-earthquake safety evaluations throughout the United States. As we discuss below, the New Zealand guidelines for post-earthquake safety procedures are based on ATC-20.[1] WJE engineers (including Brian Kehoe) are regularly engaged by ATC and FEMA as instructors.

b. WJE engineers are credited with developing the Capacity Spectrum Method (**CSM**), a seismic analysis technique that is the source of the nonlinear static pushover analysis methods prescribed by ATC-40, FEMA 356, and ASCE-41.[2] CSM and nonlinear static pushover methods are recognised as innovative methods for the seismic analysis of buildings.[3]

c. WJE engineers developed the acceleration-displacement response spectrum for application in nonlinear static analysis (**ADRS**).[4] ADRS is used world-wide, including in New Zealand.[5] Messrs Paret and Kehoe were primary participants in the development of the ADRS.

d. WJE engineers have contributed to the following projects: FEMA 306, 307 and 308 (Evaluation and Repair of Earthquake Damaged Concrete and Masonry Wall Buildings); FEMA 310 (Handbook for the Seismic Evaluation of

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1 NZSEE "Building Safety Evaluation During a State of Emergency, Guidelines for Territorial Authorities" (August 2009) [ENG.DHB.0004F.2]

2 Sigmund A. Freeman "Review of the Development of the Capacity Spectrum Method" (March 2004) Volume 41, No. 1, ISET Journal of Earthquake Technology, Paper #438 pp. 1-13

3 Associate Professor Rajesh P. Dhakal "Structural Design for Earthquake Resistance: Past, Present and Future" (Report to the Canterbury Earthquake Royal Commission, University of Canterbury, August 2011) p.14-16 [ENG.ACA.0004]; NZSEE Study Group on Earthquake Risk Buildings "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" (April 2012) Section 6

4 James A. Mahaney; Terrence F. Paret; Brian E. Kehoe; Sigmund A. Freeman "The Capacity Spectrum Method for Evaluating Structural Response During the Loma Prieta Earthquake" (National Earthquake Conference: Earthquake Hazard Reduction in the Central and Eastern United States, May 2-5, 1993)

5 NZSEE Study Group on Earthquake Risk Buildings "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" (April 2012) Section 5.4

Buildings: a Prestandard); FEMA 356 (Prestandard and Commentary for the Seismic Rehabilitation of Buildings); ATC-20-2 (Addendum to the ATC-20 Postearthquake Building Safety Evaluation Procedures); ATC-40 (Seismic Evaluation and Retrofit of Concrete Buildings); SAC[6] Steel Moment Frame project; CUREE-Caltech Woodframe Project, ASCE-31 (Seismic Evaluation of Existing Buildings) and ASCE-41 (Seismic Rehabilitation of Existing Buildings). Seminal resource documents that were prepared by WJE engineers include three Naval Facilities Engineering Command seismic technical manuals, Seismic Design for Buildings; Seismic Design Guidelines for Essential Buildings; and Seismic Design Guidelines for Upgrading Existing Buildings; TM5-809-10, TM5-809-10-1, and TM5-809-10-2, respectively.

1.5 WJE was requested by CPG New Zealand Ltd (**CPG**) to review the damage assessment performed by David Coatsworth on the building at 249 Madras St (**the CTV Building**) after the 4 September 2010 earthquake and to provide an expert opinion on:

a. whether the assessment undertaken by Mr Coatsworth was appropriate; and

b. whether, based on his findings from that assessment, his conclusions and recommendations were properly made.

1.6 In relation to the first question, WJE was specifically asked to consider whether Mr Coatsworth should have undertaken a seismic analysis of the as-built condition of the building.

1.7 We undertook this review in conjunction with a Principal from our Chicago office, Conrad Paulson. A copy of Mr Paulson's curriculum vitae is also attached to this statement of evidence. While Brian Kehoe will attend the inquiry to present this statement to the Royal Commission, the other members of the team are able to attend the hearing by videolink if it would be helpful to the Commission for us to do so.

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[6] SAC is a joint venture between the Structural Engineers Association of California (**SEAOC**), the Applied Technology Council (**ATC**) and the Consortium of Universities for Research in Earthquake Engineering (**CUREE**)

1.8 Brian Kehoe was a member of the American Society of Civil Engineers (**ASCE**) reconnaissance team which visited Christchurch after the 4 September 2010 Darfield earthquake and again after the 22 February 2011 event. The team was tasked by the ASCE with assessing the performance of buildings for the purpose of improving procedures for evaluating the seismic performance of existing buildings and met with professors from the University of Canterbury and local structural engineers. After the February event, Mr Kehoe accompanied building damage assessment teams conducting follow-up post-earthquake safety evaluations of buildings and attended briefs for local structural engineers at the emergency operations centre.

1.9 Mr Kehoe is a steering committee member of ASCE-41 (Seismic Evaluation and Rehabilitation of Existing Buildings) and sits on the American Concrete Institute's committee ACI-374 for Performance-Based Seismic Design of Concrete Buildings (among other committees). He has developed training courses and given numerous presentations for FEMA regarding seismic design and evaluation of buildings and non-structural components throughout the United States and is also an instructor for ATC-20, which is discussed in more detail below.

1.10 We have been provided with and have read a copy of the Code of Conduct for Expert Witnesses set out in Schedule 4 of the High Court Rules and we agree to comply with the same.

2. DOCUMENTS AND MATERIALS REVIEWED

2.1 We have reviewed the brief of evidence of David Coatsworth and the documents to which he refers including Mr Coatsworth's email of 24 September 2010 setting out CPG's proposal for the building assessment, Mr Coatsworth's notes, diagrams and photographs concerning the assessment he undertook, his report of his findings and conclusions, and his email concerning his subsequent inspection on 19 October 2010.

2.2 We have also reviewed materials on the secure document access system and the public database set up by the Royal Commission relating to the CTV Building collapse.

2.3 In this statement, we refer to other materials including research and reference materials that we have considered in forming our opinions.

3. POST-EARTHQUAKE ASSESSMENTS

3.1 As outlined above, WJE engineers have undertaken post-earthquake damage assessments in various jurisdictions and have been directly involved in the development of, and provided training for, commonly used post-earthquake procedures. There are a variety of assessment methodologies that can be used. The type of assessment that is performed will depend on a number of factors, which are explained in more detail below. Typically, assessments fall into two categories:

a. Basic rapid assessments conducted by the local jurisdiction (territorial authority).

b. Engineering evaluations undertaken by an engineer engaged by the building owner. In some circumstances, that evaluation may include an assessment of the building's seismic capacity, as designed or constructed.

Basic rapid assessments

3.2 The basic rapid assessment process is initiated by the territorial authority immediately following a significant earthquake. It is a triage process used to establish whether a subsequent damage assessment should be performed. The triage process helps to prioritise buildings that require subsequent damage assessments so that the available resources are best utilised.

Guidelines

3.3 In New Zealand, the primary resource document available in September 2010 to guide the performance of these rapid assessments was a document titled "Building Safety Evaluation During a State of Emergency - Guidelines for Territorial Authorities" prepared by the New Zealand Society for Earthquake Engineering (**NZSEE**) and dated August 2009 (the **NZSEE Guidelines**).[7] The NZSEE Guidelines is an update of the first edition produced in 1998 based on post-earthquake

[7] ENG.DBH.0004F.2

experience gained in both New Zealand and the United States, and draws heavily from the document "ATC-20: Procedures for Post-earthquake Safety Evaluation of Buildings" (**ATC-20**), by the Applied Technology Council (**ATC**).

3.4 The ATC is a non-profit corporation that was founded in 1973 as an initiative of the Structural Engineers Association of California. ATC develops and promotes engineering resources and applications for mitigating the effects of natural and other hazards on the built environment, primarily as they relate to earthquakes. Mr Robert Bruce, who is currently employed by WJE, was a contributor to the development of the ATC-20 series of documents. Brian Kehoe, Robert Bruce and others at WJE are instructors for ATC-20.

Level 1 Rapid Assessment

3.5 Under the NZSEE guidelines, the initial assessment of a building is referred to as a Level 1 Rapid Assessment. Generally, the Level 1 Rapid Assessment is performed by a building official, or volunteer engineer or architect, without interior access to the building and this assessment takes on the order of 10 to 20 minutes per building. The purpose of this initial rapid assessment is to identify buildings with obvious visible indications of severe damage that may present a life safety hazard to the public. The result of this initial assessment is typically a coloured placard placed on the building (also referred to as "tagging") indicating that the building is either Green (Safe to Occupy), Yellow (Restricted Use), or Red (Unsafe).

3.6 In the ATC-20 document, the initial assessment is termed a Rapid Assessment, which corresponds to the Level 1 assessment from the NZSEE Guidelines.

Level 2 Rapid Assessment

3.7 A subsequent evaluation may be performed by the territorial authority to verify the findings from the initial rapid assessment. This subsequent evaluation is termed a Level 2 Rapid Assessment in the NZSEE Guidelines or a Detailed Assessment using the ATC-20 procedure. This evaluation, performed subsequent to the Level 1 evaluation, typically includes visual observations of the accessible interior and exterior portions of the building. In the ATC-20 guidelines, the Detailed Assessment is described as a visual observation of the inside and outside of the structure. These assessments typically last from 1 to 4 hours.

3.8 The criteria used in Christchurch to determine the prioritisation of the Level 2 inspections were:[8]

a. all buildings which had received a Red or Yellow placard in the Level 1 assessment;

b. all Green placarded buildings with more than 4 levels;

c. all Green placarded buildings with high occupancy levels; and

d. all Green placarded buildings where the Level 1 Rapid Assessment form recommended that a Level 2 assessment be carried out.

Subsequent engineering evaluations

3.9 The third level of post-earthquake evaluation is termed a Detailed Engineering Evaluation by the NZSEE Guidelines. The ATC-20 document refers to this level of evaluation as an Engineering Evaluation. In both documents, this level of evaluation is described as one in which an engineer is engaged by the building owner.

When a Detailed Engineering Evaluation is required

3.10 The NZSEE Guidelines, in Figure 2, anticipate that buildings receiving Yellow and Red placards will require a Detailed Engineering Evaluation. However, the NZSEE Guidelines do not require Green placarded buildings to receive a Detailed Engineering Evaluation. With respect to Green placarded buildings, this same figure indicates only that they "… may need further inspection or repairs ([by the] owner's engineer)". A copy of Figure 2 is included below.[9]

[8] NZSEE "Building Safety Evaluation Following the Canterbury Earthquakes" (Report to the Royal Commission of Inquiry into Building Failure Caused by the Canterbury Earthquakes, September 2011) p. 17 [ENG.NZSEE.0001.17]; Christchurch City Council "Report Into Building Safety Evaluation Processes in the Central Business District Following the September 4 2010 Earthquake" p. 14 [ENG.CCC.0002F.14]
[9] ENG.DHB.0004F.14

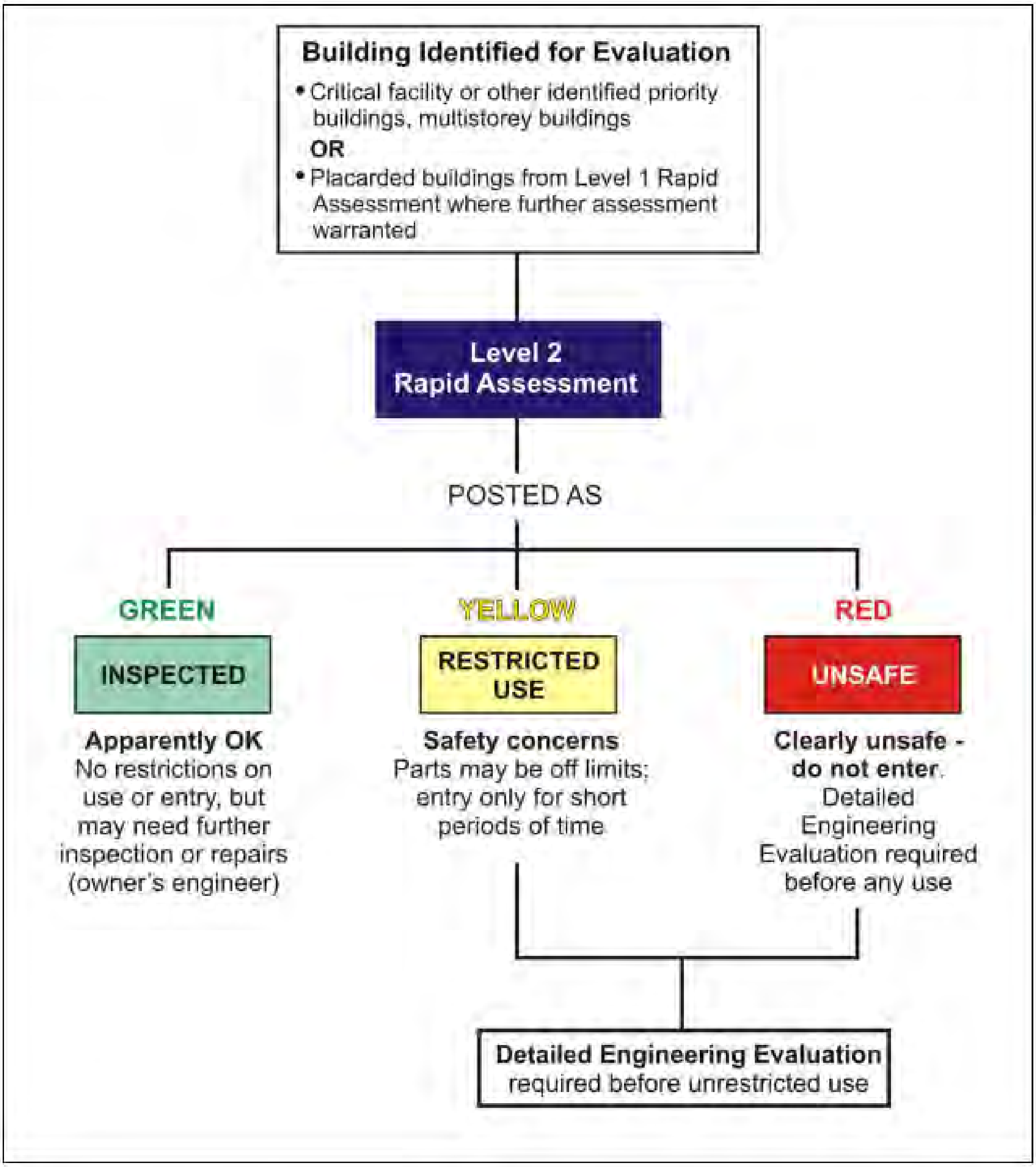

Figure 2: Flow Chart Showing the Level 2 Rapid Assessment and Posting Process

(Adapted from ATC-20)

3.11 We understand that buildings that received Yellow or Red placards following the September 2010 Darfield Earthquake were considered by the Christchurch City Council to be Earthquake Prone Buildings under s122 of the Building Act 2004 once the State of Emergency expired on September 16, 2010. These buildings were required to have a Detailed Engineering Evaluation and possibly remedial work prior to resuming unrestricted occupancy. Buildings, such as the CTV building, that received Green placards did not require any further action by the Christchurch City Council and the Green placard could remain on the building at the discretion of the owner.[10]

3.12 In our experience, there can be a number of reasons why an owner engages an engineer, such as Mr Coatsworth, to perform a visual assessment of a building which has been given a Green placard under the basic rapid assessment process and which accordingly does not require further assessment under the NZSEE guidelines (i.e. falls outside the Figure 2 flow chart, illustrated above). These reasons include, among others:

a. confirming or refuting the post-earthquake posting of the building performed by the territorial authority;

b. developing estimates of repair costs for insurance claims;

c. providing an initial assessment of the condition of the building for the purpose of determining the need for follow-up damage assessments; and

d. designing repairs or other remedial measures.

3.13 Following an earthquake, building owners and tenants often have heightened awareness of the condition of the building. Engineers are engaged to assess visually the conditions identified by building owners and tenants to help distinguish cosmetic damage from structural damage. This may be needed when the building owner or tenant perceives the observable damage to be more significant than that indicated by

[10] Christchurch City Council "Resumption of occupancy and use of earthquake-damaged buildings Section 1 Buildings included in the Scope of the Building Act 2004" (Version 1, 18 October 2010). Found as Attachment 9 to Christchurch City Council Building Evaluation Transition Team "Processes Used and Lessons Learnt following the Darfield Earthquake of 4 September 2010" (19 January 2011) [ENG.CCC.0001.98 to ENG.CCC.0001.103]

the post-earthquake rapid assessments (colour placard tagging) performed by the territorial authority.

Scope of evaluation

3.14 The NZSEE Guidelines do not provide detailed guidance for the implementation of the Detailed Engineering Evaluation of Red and Yellow placarded buildings, or for any "further inspection" of Green placarded buildings outside the scope of the NZSEE Guidelines.[11] The ATC-20 document also does not provide guidance regarding the scope or implementation of the Engineering Evaluation.

3.15 The specifics of the Detailed Engineering Evaluation of a Yellow or Red placarded building or further inspection of a Green placarded building would be expected to vary depending on the type of damage and the type of building. The decision on the specifics of the evaluation is usually made based on agreement between the engineer and building owner, typically with consideration given to the amount of effort required and the cost associated with the evaluation compared to the perceived need for the evaluation.

3.16 An evaluation of a Green placarded building (that is, a building that has not been identified in the Level 1 and Level 2 Rapid Assessment process as being hazardous or unsafe) will always involve a visual inspection, at least in the first instance. Implicit in the methodology of post-earthquake safety evaluations is that imminent hazards and unsafe conditions are visible physical conditions as opposed to numerically calculated conditions. Particularly with respect to reinforced concrete buildings, structural damage significant enough to compromise safety is normally expected to be visible. This is the case because the concrete has to crack before the strength of the steel reinforcing embedded within the structural concrete members can be mobilised, and full mobilisation of the steel reinforcing normally necessitates the formation of wide cracks.

[11] Engineering Advisory Group "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury. Part 2 Evaluation Procedure" (draft document, Revision 5, 19 July 2011) p 1 [ENG.EAG.0001.5]; NZSEE "Building Safety Evaluation Following the Canterbury Earthquakes" (Report to the Royal Commission of Inquiry into Building Failure Caused by The Canterbury Earthquakes, New Zealand Society for Earthquake Engineering, September 2011) p.37 [ENG.NZSEE.0001.37]

3.17 By way of clarification, in a post-earthquake environment, a finding that an imminent hazard or unsafe condition exists is generally indicated when a part or portion of a building might fall or collapse either spontaneously or in the event of an aftershock, where aftershocks are commonly understood to be smaller than the main shock.[12] The 22 February 2011 aftershock was unusual; although it was of a lesser magnitude on the Richter scale than the 4 September 2010 event, the depth and the proximity of its epicentre relative to the Central Business District meant that the seismic demands experienced in the centre of Christchurch exceeded both those of the main 4 September 2010 earthquake shock[13] and the theoretical 475-year "design event" that new commercial office buildings must be designed to withstand.[14] In our opinion, the occurrence of an aftershock with an intensity meeting or exceeding a 1 in 2,475 year event (such as the 22 February 2011 event) would not have been anticipated by engineers undertaking assessments in the aftermath of the 4 September 2010 earthquake.[15]

3.18 While ATC-20 and the NZSEE Guidelines are intended to guide post-earthquake assessments by professional volunteers as organised by a territorial authority, or by qualified employees of the territorial authority itself, the conceptual framework established in these documents for recognising imminent hazards and unsafe conditions is generally also applicable to post-earthquake assessments conducted by engineers under contract to an owner. This applies both to Detailed Engineering Examinations required under the NZSEE Guidelines and to the assessment of Green placarded buildings, which are not required under those guidelines.

3.19 An engineer conducting a post-earthquake assessment under contract to an owner would normally focus more on-site attention on the structure of interest than would volunteers or civil authorities utilising either the NZSEE Guidelines or ATC-20. Examples of examinations that might be done by an engineer engaged by a building owner beyond that done during the Level 2 Evaluation are:

[12] Ibid. p.38 [ENG.NZSEE.0001.38]
[13] Dr Clark Hyland and Ashley Smith "CTV Building Collapse Investigation" (Report for Department of Building and Housing, 25 January 2012) p. 20 [BUI.MAD249.0189.50]
[14] Dr. Weng Yuen Kam and Associate Professor Stefano Pampanin, "General Building Performance in the Christchurch CBD: a contextual report" (report prepared for the Department of Building and Housing, November 2011) p.18-22
[15] Ibid; NZSEE "Building Safety Evaluation Following the Canterbury Earthquakes" (Report to the Royal Commission of Inquiry into Building Failure Caused by The Canterbury Earthquakes, New Zealand Society for Earthquake Engineering, September 2011) p.38 [ENG.NZSEE.0001.38]

a. mapping the location of cracks;

b. measuring the width of cracks;

c. observing concealed conditions, such as by removing ceiling tiles at strategic locations; and

d. measuring selected dimensions of the building and structural elements.

3.20 Engineers have recourse to a limited number of available documents for guidance with respect to assessing the condition of a building affected by an earthquake. The selection of the documents that are relied upon is at the judgement of the engineer performing the evaluation.

3.21 One document that provides a detailed procedure for quantitatively evaluating buildings constructed with concrete or masonry walls for seismic lateral resistance is FEMA 306 "Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings". This document provides guidance for the evaluation of earthquake damage to concrete and masonry shear walls in terms of the effect of the observed damage on the future performance of the building.

3.22 FEMA 306 includes component classification guides that provide recommendations for the characterisation of various types of damage to the lateral force resisting elements of the building. FEMA 306 guides can be used to evaluate the structural significance of cracks of various widths in concrete shear walls. However, FEMA 306 does not provide guidance for the evaluation of the other structural elements in a building such as the gravity load framing.

3.23 Other general guidance about what may or may not constitute structural damage can be obtained from structural engineering textbooks and available professional papers. We note, however, that documents published in New Zealand following the February 2011 earthquake point out the lack of guidance for engineers performing post-earthquake damage assessments and provide some additional guidelines.[16]

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[16] NZSEE "Building Safety Evaluation Following the Canterbury Earthquakes" (Report to the Royal Commission of Inquiry into Building Failure Caused by The Canterbury Earthquakes, New Zealand Society for Earthquake Engineering, September 2011), p. 37 [ENG.NZSEE.0001.37]

Design drawings

3.24 There is no post-earthquake evaluation standard of which we are aware that requires the review of design drawings as part of the post-earthquake damage assessment. The NZSEE Guidelines indicate only under the category "Detailed Engineering Evaluation and Remedial Work" that such evaluations "are likely to involve review of construction documentation…"[17] As outlined above, a Detailed Engineering Evaluation is only required if a building has already been inspected and identified as hazardous (placarded Yellow or Red) during the basic rapid assessment process.

3.25 It is sensible for an engineer to request the design drawings to obtain information about the building structure. In reality, however, post-earthquake damage assessments, including further assessments and Detailed Engineering Evaluations carried out by engineers engaged by the building owner, are often performed for older buildings for which original structural design drawings are no longer in existence and so the drawings are not available.

3.26 Where original design drawings are not available, an engineer performing a post-earthquake evaluation may obtain an architectural floor plan that shows the general layout of the permanent building elements, or the engineer may create such a floor plan. This type of plan may enable the engineer to understand the location and quantity of the structural elements of the lateral force resisting system, such as concrete shear walls, among other elements. The general information about the layout of structural elements can be used by the engineer to attempt to identify the location of the elements that would be more likely to have been damaged by the earthquake. This information could be used by the engineer performing an engineering evaluation to direct the engineer's efforts towards potentially important structural elements.

3.27 For buildings that were previously identified as earthquake prone or are suspected to be earthquake prone following an assessment of their seismic capacity using a simplified calculation process such as the Initial Evaluation Procedure (**IEP**)[18], it would be reasonable for an engineer to review the structural drawings, if they exist,

[17] NZSEE "Building Safety Evaluation During a State of Emergency, Guidelines for Territorial Authorities" (August 2009) p.9 Table 1 [ENG.DHB.0004F.10]
[18] The IEP is described in detail in Section 3 of NZSEE Study Group on Earthquake Risk Buildings "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" (June 2006) [ENG.DHB.0004E-A.47]

to make an assessment of whether the damage to the building has affected the critical deficiencies identified by the IEP.

3.28 An engineer would also generally recommend reviewing available drawings for buildings that have unusual configurations or that have structural framing that is not readily observable. This would provide the engineer with confidence that the important structural details would be identified and observed in detail if warranted by the available evidence of damage. If the engineer feels that the pattern and extent of damage to the building are not consistent with the engineer's expectations based on knowledge of the age of the building, ground motion at the site, and damage to nearby buildings, then the engineer may review drawings or perform in-situ testing to explain the observed damage.

3.29 An engineer would generally not obtain and review structural drawings (or undertake an IEP) for modern buildings that do not exhibit unusual or excessive structural damage. This is because modern buildings are generally considered to be designed to current earthquake design standards. In the City of San Francisco, for example, code triggers that require strengthening of existing buildings are not applicable to buildings designed after 21 May 1973, when the first San Francisco code which incorporated significant improvements to ensure ductile response of reinforced concrete structures was adopted.[19] Similarly, ASCE-31 "Seismic Evaluation of Existing Buildings" includes an explicit definition of "benchmark buildings" which sets forth that a seismic evaluation need not be performed when the design and construction of the buildings is in accordance with modern building code provisions identified as benchmark provisions.[20] These modern buildings would generally be expected to perform well during a design earthquake, even though building codes have evolved and become more stringent since that time. A similar philosophy exists within the New Zealand earthquake engineering profession, where post-1976 buildings are credited with superior seismic resistance by virtue of their design date.[21]

[19] 2010 San Francisco Building Code, Chapter 16, Section 1604.11.1
[20] ASCE-31-03, Seismic Evaluation of Existing Buildings, American Society of Civil Engineers, 2003
[21] Department of Building and Housing "Earthquake Prone Building Provisions of the Building Act 2004: Policy Guidance for Territorial Authorities" (June 2005) p.22 Table 1 [ENG.DBH.0005D.25]. NZSEE Study Group on Earthquake Risk Buildings "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" (June 2006) p. 2-13 [ENG.DHB.0004E-A.39]. In these tables, buildings designed according to "NZS 4203:1976 or better" are assumed to be capable of 80%NBS to greater than 100%NBS.

Assessment of Expected Seismic Capacity

3.30 The primary goal of a post-earthquake assessment is to identify if the building exhibits visible physical evidence of having its capacity diminished by the earthquake. In the absence of such evidence, the building can be assumed to be capable of withstanding another earthquake of equivalent force to the earthquake that resulted in the inspection taking place.

3.31 Under the circumstances discussed below, an engineer conducting a post-earthquake assessment under contract with an owner might determine (that is, numerically calculate) the expected seismic capacity of a building as designed or as constructed or both. Generally, such a determination, if it were to be done, would be done with the authorisation of the owner of the building who would have agreed in advance to the scope and cost of this kind of assessment.

3.32 An engineering recommendation to make a determination of expected capacity of a building would not normally be made prior to completion of an on-site post-earthquake visual assessment. Only after determining during the on-site assessment that the building exhibits visible physical evidence of its capacity having been diminished by the earthquake would an engineer conducting a post-earthquake assessment normally recommend that calculation of expected capacity of the building as designed, as constructed, or both, and calculation of diminution of capacity, be performed.

3.33 In the absence of visible physical evidence of some diminution of capacity, there would not normally be cause to recommend calculation of expected capacity as part of a post-earthquake assessment. The absence of visible physical evidence of some diminution of capacity would be all the more significant if the intensity of earthquake shaking that prompted the assessment was near that of a design level event (such as the 4 September 2010 earthquake), because exposure to actual earthquake shaking at design-level intensity is a better test of capacity than any calculation or analysis.

3.34 Determination of the expected capacity of a building can be done in a multitude of ways and with varying levels of accuracy. The extent of professional services involved in making such determinations might vary from approximate calculations (such as the IEP) to highly complex computer simulations. The level of effort involved would normally be the subject of discussion and negotiation between the engineer and the building owner, and based on the physical evidence of damage exhibited by the building after the earthquake. Lack of visually significant damage would not warrant further analysis since the extent of professional services undertaken should be commensurate with the damage observed.

3.35 Without available drawings, a "back of the envelope" type of determination would likely be of limited utility. For example, without structural drawings, the strength of a shear wall building might be relatively easy to approximate using the "back of the envelope" approach, but the vulnerability of the associated beam and column framing to go undergo significant lateral displacements without being structurally compromised would not be readily assessable in this manner.

Closure of whole or part building

3.36 Under appropriate circumstances, it would be prudent and generally accepted practice for an engineer to recommend closure of the whole or part of a building while at the site in response to conditions observed when conducting a post-earthquake damage assessment. In particular, it would be prudent and generally accepted practice for an engineer to recommend closure of the whole or part of a building which exhibits an imminent hazard or unsafe condition that jeopardizes the safety of either occupants or passersby.

3.37 It is not prudent or generally accepted practice to close parts or all of a structure that do not exhibit an imminent hazard or unsafe condition, although closures for inappropriate reasons do occur. Guidance set forth within the NZSEE Guidelines and ATC-20, and also within other documents,[22] explains that improper closures are unduly burdensome to building owners and to the community as a whole and should therefore be avoided. Prudence does not therefore justify being overly conservative and closing buildings unduly.

Reassessments

3.38 All buildings affected by an earthquake and that have been given Green, Yellow, or Red placards need to be re-evaluated following any significant aftershock using the

[22] NZSEE "Building Safety Evaluation Following the Canterbury Earthquakes" (Report to the Royal Commission of Inquiry into Building Failure Caused by The Canterbury Earthquakes, September 2011) p.19 [ENG.NZSEE.0001.19]; Building Evaluation Transition Team, "Processes Used and Lessons Learnt Following the Darfield Earthquake of 4 September 2010" [ENG.CCC.0001.28]

same triage procedure with Level 1 Rapid Assessments to assess changes in the condition of the building. This is because the conditions may have changed during subsequent events, such as the Boxing Day earthquake of 26 December 2010. Any evaluation made following an aftershock supersedes the evaluations made prior to that event.

4. POST-EARTHQUAKE ASSESSMENT UNDERTAKEN BY DAVID COATSWORTH

4.1 We have reviewed the statement of evidence of David Coatsworth and the supporting documents referred to in that statement. The opinions we express are based on those materials.

Proposal

4.2 Mr Coatsworth of CPG conducted a post-earthquake assessment at the request of and under agreement with the owner of the property. Mr Coatsworth's proposal was drafted, as is usual and customary, at a time when he had only a limited understanding of the conditions at the building, and as such was general in nature.

4.3 At the time of Mr Coatsworth's instruction, Level 1 and Level 2 Rapid Assessments had already been conducted by post-earthquake inspection teams operating under the direction of the Christchurch City Council. Since the CTV Building was not placarded as either Yellow or Red by a Level 2 Rapid Assessment, a Detailed Engineering Evaluation was not required by the NZSEE Guidelines. Nonetheless, the owner of the building appears to have decided to obtain a separate evaluation.

4.4 In his proposal, Mr Coatsworth appropriately requested both the structural and architectural drawings as having such drawings would assist in his developing an understanding of the structural systems in the building. We understand that his evidence is that he was informed that they were not available.

4.5 Mr Coatsworth proceeded with the post-earthquake assessment without benefit of the information that the original structural drawings might have provided. As we have explained above, this is usual and customary when drawings are not available.

4.6 In lieu of original drawings for the building, Mr Coatsworth made use of interior layout plans for the first and second floors (also known as the ground floor and first floor) provided to him by the tenant of those floors, CTV. Use of such plans, where original drawings are unavailable, is accepted engineering practice.

4.7 Mr Coatsworth's proposal explains that his recommendation is to examine the exterior of the building from whatever vantage points are available and to inspect all visible interior finishes. In addition, he proposes to look selectively above ceiling tiles, but reserves judgment on removing wall finishes until the condition of those finishes is examined. We agree with his recommended scope of work. To recommend more on-site activity at the time the proposal was written would not have been appropriate prior to viewing the condition of the building.

4.8 Mr Coatsworth's proposal states that at the time the proposal was written he was not including either analysis or development of repairs in his recommended scope of work. Importantly, and, in our view, quite correctly, his proposal indicates that analysis of the structure might ultimately be necessary if significant structural damage is identified. This is the correct hierarchy in which to place analysis in a post-earthquake inspection. As we have explained at paragraphs 3.32-3.33 above, it is usual and customary to conduct the on-site assessment first to determine if significant structural damage is exhibited, and then, if called for by the presence of damage, recommend that structural analysis be performed.

Inspection

4.9 We understand that Mr Coatsworth's evidence is that on 29 September 2010 he spent approximately four hours performing visual observations at the CTV Building. His field notes from this site inspection include visual observations undertaken at every level of the interior, including the mechanical plant room at the top of the building, and also visual observations of the building exterior. He includes sketches of cracking on the walls of the north core stairwell and w.c. room, and on the south shear wall. He took photographs of numerous conditions at the interior and exterior of the building, including photographs taken at locations appearing to be above the ceiling, presumably accessed by lifting up of some ceiling tiles. For the size of the CTV Building and its type of construction, the actions of Mr Coatsworth are consistent with our understanding of accepted engineering practice.

4.10 His field notes also indicate that Mr Coatsworth systematically examined and then documented locations of readily-observed cracks in all of the readily-accessible reinforced concrete shear walls in the building. We understand that Mr Coatsworth's evidence is that he made numbered notes during his inspection and then, on October 6, returned to the building and made a detailed record of crack width measurements. This kind of record is what we would expect to see in an assessment of this nature.

4.11 The largest crack width measurement recorded by Mr Coatsworth as of October 6 2010 was 0.30 mm for inclined or diagonal cracks, and 0.35 mm for horizontal cracks. Based on our own review of Mr Coatsworth's crack measurement field notes and previous experience in evaluating concrete buildings, we would interpret the vast majority of the horizontal "cracks" to be movement along construction joints that in all likelihood would have already existed as open cracks before the earthquake of 4 September 2010. Mr Coatsworth characterises these cracks as "minor structural damage" and did not raise any alarm, immediate or delayed, concerning these cracks. We agree with his assessment in that we consider these cracks to be damage to a structural element that is not of structural consequence. Cracks of this size may be considered minor damage to a structural element (damage of a cosmetic nature).

4.12 Mr Coatsworth's records indicate that, while preparing his report, Mr Coatsworth sought the opinion of his colleagues and others in the engineering profession regarding the size of the cracks in the concrete shear walls and what, if anything, should be done about them. Seeking counsel from peers is not unusual in the engineering profession and, in our opinion, amounts to good practice. In many engineering firms, this type of action is encouraged.

4.13 Mr Coatsworth did not undertake any numerical calculation of the expected seismic capacity of the CTV Building. This is consistent with his proposal to the building owner. As we have explained, we would not expect any such calculations to be done in the absence of any indication that significant structural damage had occurred. Such damage ought to be apparent on a visual inspection of the building.

4.14 Following the aftershock of 19 October 2010, Mr Coatsworth says that he returned to the CTV Building that same day to re-examine the building. He spoke with several building occupants and re-examined distressed areas he previously observed. His only notable observation was the possibility that two horizontal cracks in the shear walls, likely pre-existing construction joint cracks, may have enlarged slightly, with the maximum horizontal crack width now reported as 0.4 to 0.5 mm. According to his records, diagonal cracks did not increase in width. In our opinion, cracks of this size in a modern reinforced concrete structural element do not constitute structurally substantive damage to the structure, whether or not they are at construction joints.

4.15 In summary, it is our opinion that Mr Coatsworth's inspection is consistent with what we would expect an engineer to do when assessing a building like the CTV Building for post-earthquake damage.

The earthquake damage report, its conclusions and advice

4.16 On the basis of Mr Coatsworth's field notes, photographs, and the other records related to his earthquake damage assessment, we agree with his conclusion that the damage he observed to the structure of the building was minor damage at worst and for the most part did not warrant structural repairs. In those circumstances, we do not believe that a structural analysis or that further detailed inspection of the CTV Building was indicated.

4.17 As discussed above, the largest crack width measurement recorded by Mr Coatsworth on 6 October 2010 was 0.35 mm and, following the 19 October 2010 aftershock, the maximum crack width recorded was 0.4 to 0.5 mm, and these maximum crack widths were all recorded at apparent horizontal construction joints. His opinion that cracks of this size represent minor damage is consistent with available guidance on the significance of cracks in modern reinforced concrete shear walls based on a number of sources, as follows.

4.18 Professors Park and Paulay, on page 476 of their textbook, characterise cracks of the sizes reported by Mr Coatsworth as an "aesthetic consideration", specifically stating: "The maximum crack width that will neither impair a structure's appearance nor create a public alarm is probably in the range 0.010 to 0.015 in (0.25 to 0.38 mm), but larger crack widths may be tolerated."[23] In this passage, Professors Park and Paulay clearly indicate that these maximum sizes are in the context of "aesthetic consideration", and not with regard to structural impairment. Similarly, the New Zealand Standard for the Design of Concrete Structures allows crack widths of 0.4 mm for reinforced concrete subjected to "permanent load plus infrequent

[23] R. Park and T. Paulay, Department of Civil Engineering, University of Canterbury *Reinforced Concrete Structures*, John Wiley & Sons, New York 1975

combinations of transient load…"[24] as would apply to seismic loading. Accordingly, their further comment that "larger crack widths may be tolerated" means that even Mr Coatsworth's largest recorded crack size of 0.4 to 0.5 mm remains only an aesthetic consideration, may be aesthetically acceptable, and is not of any structural concern.

4.19 FEMA 306 describes various damage states for concrete shear walls. In all cases, cracks less than 1/16 inch (1.5 mm) in width are considered "Insignificant Damage" for which repairs may be necessary for restoration of non-structural characteristics.[25] Mr Coatsworth's largest recorded crack size of 0.4 to 0.5 mm clearly qualifies as "Insignificant Damage" according to the FEMA 306 criteria.

4.20 In Mr Coatsworth's notes and in his emails, he indicates that he has based his opinion upon discussions he had with his colleagues at CPG and with others in the engineering profession regarding the observed size of the cracks in the concrete shear walls. In our view, seeking a second opinion is consistent with good engineering practice. We agree with the advice that he received.

4.21 In his report, Mr Coatsworth advises that cracks larger than 0.2 mm be repaired by epoxy injection. In his notes and emails summarising his consultations with others, it appears that the basis of this specific recommendation is "for peace of mind and weathering."[26] There is also discussion that some of the stiffness of the wall may be restored by the recommended repairs. His recommendation to repair these cracks is, in our opinion, appropriate not only as a measure against the weather but also for partial restoration of stiffness of the wall, and is in accordance with accepted engineering practice.

4.22 In his report, Mr Coatsworth also comments on other damage he observed, in addition to the cracking damage in the concrete shear walls. The described damage is itemised under the following categories: Columns, Beams and Spandrel Panels; Flooring; Non-Load Bearing Concrete Block Walls; Internal Framing and Linings; and Windows.

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[24] Standards Association of New Zealand "Code of practice for The Design of Concrete Structures" Table 4.3, NZS 3101 Part 1: 1982

[25] FEMA 306 "Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings - Basic Procedures Manual" (May 1999)

[26] Email from David Coatsworth, dated Wednesday, 6 October 2010 3.21 p.m., to Tony Crang and Jerry Kearney, subject "Cracked Shear Walls"

4.23 From a structural perspective, all of the damage described under these added observations is characterised as minor damage at worst, for the most part not indicating structural damage and not warranting structural repairs. However, several repairs for protection against water intrusion are again recommended. Additionally, loose plaster on the exterior of the building is identified as a falling hazard, and Mr Coatsworth recommends taking measures to mitigate this. We agree with Mr Coatsworth's conclusions on these added observations and his recommendations for repair or mitigation.

4.24 Finally, in his email dated 19 October 2010, recounting his follow-up visual inspection of the CTV Building following the aftershock in the morning of that day, Mr Coatsworth concluded that the building was "still structurally sound". To an engineer, use of the phrase "structurally sound" in the context of a post-earthquake assessment means that the building had not been structurally compromised and would be capable of withstanding another earthquake of equivalent intensity, including such considerations as the acceleration, velocity, and displacement caused by the shaking. On the basis of the observations noted in Mr Coatsworth's email we agree with the assessment that the condition of the structure had not deteriorated as a result of the 19 October aftershock.

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REVIEW OF THE DEVELOPMENT OF THE CAPACITY SPECTRUM METHOD

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ABSTRACT

The Capacity Spectrum Method (CSM), a performance-based seismic analysis technique, can be used for a variety of purposes such as rapid evaluation of a large inventory of buildings, design verification for new construction of individual buildings, evaluation of an existing structure to identify damage states, and correlation of damage states of buildings to various amplitudes of ground motion. The procedure compares the capacity of the structure (in the form of a pushover curve) with the demands on the structure (in the form of response spectra). The graphical intersection of the two curves approximates the response of the structure. In order to account for non-linear inelastic behavior of the structural system, effective viscous damping values are applied to linear-elastic response spectra similar to inelastic response spectra. The paper summarizes the development of the CSM from the 1970s to the present and includes discussions on modifications presented by other researchers, as well as recommendations by the author.

KEYWORDS: Capacity Spectrum Method, Response Spectra, Damping, Ductility, Push-Over Curve

INTRODUCTION TO PBSD AND THE CSM

The purpose of Performance-Based Seismic Design (PBSD) is to give a realistic assessment of how a structure will perform when subjected to either particular or generalized earthquake ground motion. While the code design provides a pseudo-capacity to resist a prescribed lateral force, this force level is substantially less than that to which a building may be subjected during a postulated major earthquake. It is assumed that the structure will be able to withstand the major earthquake ground motion by components yielding into the inelastic range, absorbing energy, and acting in a ductile manner as well as by a multitude of other actions and effects not explicitly considered in code applications (Freeman, 1992). Although the code requires special ductile detailing, it does not provide a means to determine how the structure will actually perform under severe earthquake conditions. This is the role of PBSD (Freeman et al., 2004).

The Capacity Spectrum Method (CSM) is a procedure that can be applied to PBSD. The CSM was first introduced in the 1970s as a rapid evaluation procedure in a pilot project for assessing seismic vulnerability of buildings at the Puget Sound Naval Shipyard (Freeman et al., 1975). In the 1980s, it was used as a procedure to find a correlation between earthquake ground motion and building performance (ATC, 1982). The method was also developed into a design verification procedure for the Tri-services (Army, Navy, and Air Force) "Seismic Design Guidelines for Essential Buildings" manual (Freeman et al., 1984; Army, 1986). The procedure compares the capacity of the structure (in the form of a pushover curve) with the demands on the structure (in the form of a response spectrum). The graphical intersection of the two curves approximates the response of the structure. In order to account for non-linear inelastic behavior of the structural system, effective viscous damping values are applied to the linear-elastic response spectrum similar to an inelastic response spectrum. In the mid 1990s, the Tri-services manual was updated (WJE, 1996).

By converting the base shears and roof displacements from a non-linear pushover to equivalent spectral accelerations and displacements and superimposing an earthquake demand curve, the non-linear pushover becomes a capacity spectrum. The earthquake demand curve is represented by response spectra, plotted with different levels of "effective" or "surrogate" viscous damping (e.g. 5%, 10%, 15%, 20% and sometimes 30% to approximate the reduction in structural response due to the increasing levels of damage). By determining the point, where this capacity spectrum "breaks through" the earthquake demand, engineers can develop an estimate of the spectral acceleration, displacement, and damage that may occur for specific structure responding to a given earthquake. A number of changes have been proposed to the capacity spectrum method that increase the complexity and computational effort associated with this method, usually requiring iteration to find the "exact" point where the capacity spectrum intersects the "correct" level of damping. The author believes that iteration is unnecessarily complex and clumsy for the intended use of this procedure. Rather, the author views the capacity spectrum method as a tool for estimating and visualizing the likely behavior of the structure under a given earthquake in a simple graphical manner. By formatting the results in the acceleration-displacement-response-spectrum format (Mahaney et al., 1993) in lieu of the traditional spectral acceleration (Sa) versus period (T) format, the graphical and intuitive nature of the capacity spectrum method become even more apparent.

ORIGIN OF THE CSM

The CSM can trace its roots to John A Blume's Reserve Energy Technique (RET) (Blume et al., 1961), which estimated the inelastic displacement by equating elastic energy (or work) with inelastic energy (or work) as illustrated in Figure 1. In other words, the area within the green trapezoid is equated to the area in the red triangle. The green line plateau is equal to the peak of the triangle divided by R. The ductility, μ (mu), is equal to the displacement at the end of the green line divided by the displacement at the bend in the green line. In the example shown in Figure 1, the elastic period is 0.70 sec and the inelastic secant period is 1.4 sec. The μ is equal to 4.0 and R is equal to 2.65. It should be noted that this procedure is consistent with the force/acceleration reduction factor $R = (2\mu - 1)^{½}$ associated with Newmark's equation for the constant acceleration range of response spectra (Newmark and Hall, 1982).

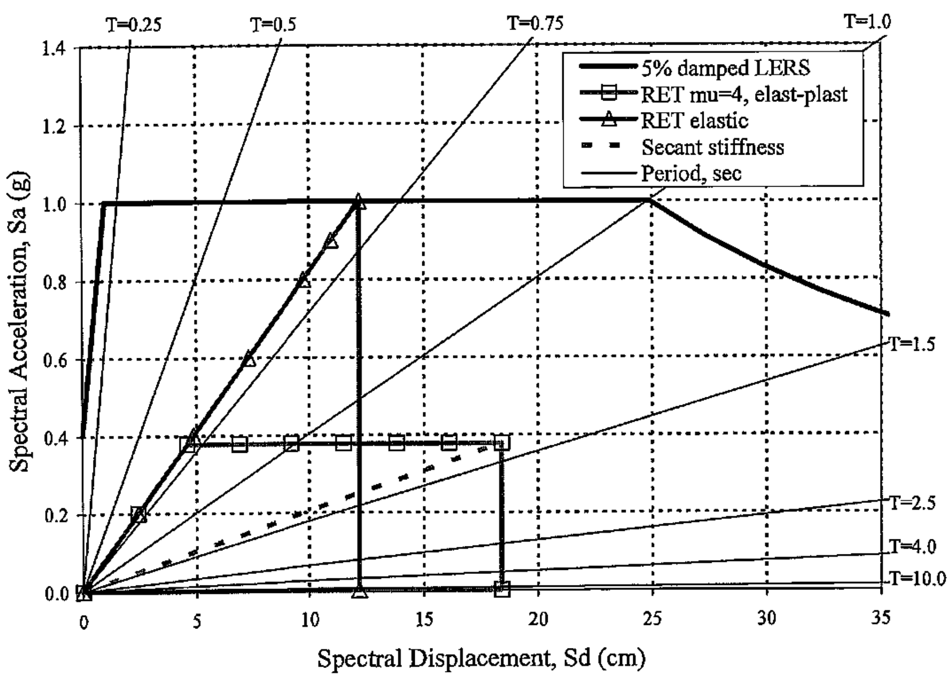

Fig. 1 John A. Blume's reserve energy technique (RET)

1. Puget Sound Naval Shipyard (PSNSY) Project

In the early 1970s, while working for John A. Blume Associates, the author was assigned the task of evaluating 80 buildings for a pilot program on establishing the seismic vulnerability of the PSNSY (Freeman et al., 1975). The time allotted for each building was in the neighborhood of an average of six hours and the proposed procedure was to be the RET. Most of the buildings were built between the beginning of the 20th century and the early 1940s. There was a wide variety of building types including brick, concrete, steel, wood, moment frames, braced frames, shear walls, and combinations thereof.

Seismic design was most likely not considered for many of the buildings and in some cases there did not appear a definable lateral force resisting system. Many buildings had dual or multi-lateral force resisting systems, some acting in parallel and others acting sequentially.

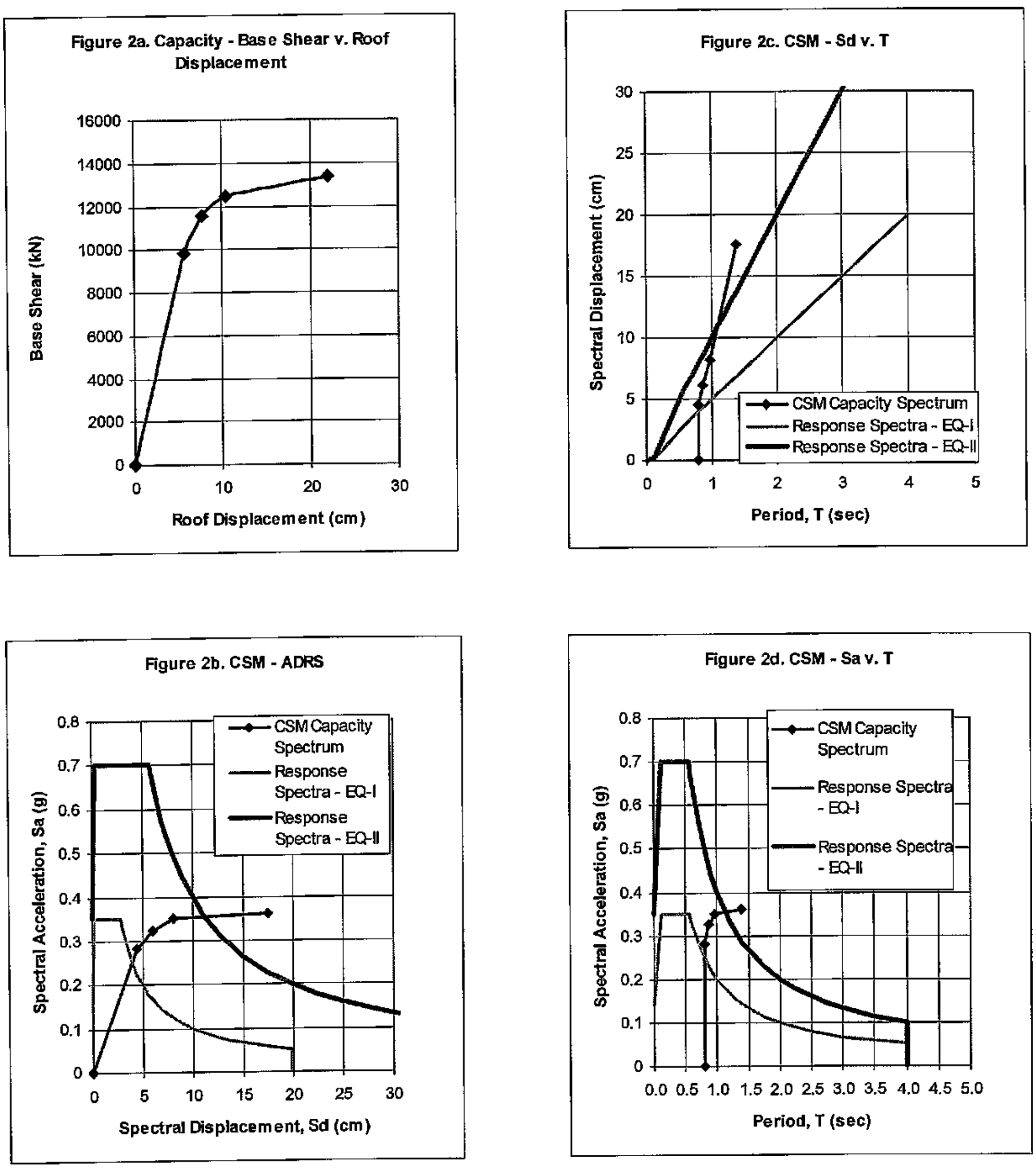

Fig. 2 Capacities and demand spectra

Capacities of the structures were determined on the basis of on-site observations, review of available drawings, calculations to approximate force-displacement relationships, and some engineering judgment. The capacity was defined by three points on a graph: origin, yield limit (incipient damage), and ultimate limit. This was essentially a rough approximation of what is now referred to as a pushover curve, which had been well illustrated in the 1961 PCA publication (Blume et al., 1961). For the PSNSY project, the yield limit was defined as the base shear represented by the force required to reach the capacity of the most rigid lateral force resisting element of the building systems. The ultimate limit was defined as the base shear causing the most flexible lateral force resisting elements to yield after the more rigid elements yielded or failed. It was possible for the ultimate base shear capacity to be lower than the yield limit capacity. However, the ultimate base shear capacity would represent a more flexible structure that would have a longer fundamental period and could be in a more favorable position in the response spectrum. A sample capacity curve is shown in Figure 2a, where the force is measured by base shear and the displacement is measured at the roof. For the PSNSY project, the capacity was represented by a bi-linear plot connecting the yield point and ultimate limit by a straight line.

Table 1: Return Periods and Probability of Exceedance

Probability of Exceedance for a Mean Return Period

Mean Return Period, Pr (yrs)	Probability of Exceedance in "x" Number of Years, P(ex)				
	Number of Years, x				
	131	50	**25**	10	1
2475	**5%**	2%	1.0%	0.4%	0.04%
475	24%	10%	5%	2%	0.2%
238	42%	19%	**10%**	4%	0.4%
87	78%	44%	**25%**	11%	1.1%
50	93%	63%	39%	18%	2%
25	99.5%	86%	63%	33%	4%
10	99.9998%	99%	92%	63%	10%
Equation: $P(ex) = 1 - \exp[-x/Pr]$					

Mean Return Period for a Probability of Exceedance

Probability of Exceedance, P(ex) (% in "x" years)	Mean Return Period in "x" Number of Years, Pr				
	Number of Years, x				
	1	10	**25**	50	**131**
1%	100	995	2488	4975	13035
2%	50.0	495	1238	2475	6485
5%	20.0	195	488	975	2554
10%	10.0	95.4	**238**	475	1244
25%	4.0	35.3	**87**	174	456
50%	2.0	14.9	36.6	72.6	189
85%	1.2	5.8	13.7	26.9	69.6
99%	1.0	2.7	5.9	11.4	28.9
Equation: $Pr = [1 - (1 - P(ex))^{1/x}]^{-1}$					

The demands on the structures were based on site specific probabilistic response spectra. The site had been subjected to ground motion from two moderate earthquakes from sources near Olympia, Washington in 1949 and 1965. Peak ground accelerations (PGA) at PSNSY were estimated to have been in the neighborhood of 0.07g. Results of a seismicity study concluded that (1) the maximum postulated PGA at the site is 0.20g, (2) the PGA for a 25-year period with a 10% probability of being exceeded is 0.07g, and (3) the PGA for a 25-year period with a 25% probability of being exceeded is 0.04g. The relationship between percent probability of exceedance in a number of years (e.g. 10%/25-yrs) can be translated into average return period (e.g. Pr = 238 years) by referring to Table 1. Similarly, for 25%/25-yrs, Pr = 87 years. Response spectra were given to represent these three PGAs. An estimated relationship between frequency of occurrence during a 131-year period and peak horizontal ground acceleration was also determined for the site. This was later used to compare average annual cost of earthquake damage to costs of seismic upgrades.

Because of the nature of the variety of building characteristics, it was determined that the RET methodology would not be applicable and that a different approach was needed. An intuitive approach, that seemed rational, was to graphically compare the demand to the capacity. By converting the capacity pushover curve from force-displacement to spectral acceleration (Sa)-spectral displacement (Sd), the capacity curve (i.e., a capacity spectrum) could be plotted on the same graph as the response spectrum (i.e., the demand). The conversion process to form the capacity spectrum is illustrated in Table 2 and the transformation from Figure 2a to Figure 2b will be discussed in more detail later.

Table 2: Capacity Spectrum

V Base Shear (kips)	d_R Roof Displ. (cm)	C_B V/W*	d_R/S_d Roof PF	C_B/Sa EMMR	S_a (g)	S_d (cm)	T Period (sec)
9786	5.79	0.22	1.30	0.78	0.282	4.45	0.80
11565	7.75	0.26	1.28	0.80	0.325	6.05	0.87
12455	10.41	0.28	1.28	0.80	0.350	8.14	0.97
13345	22.07	0.30	1.26	0.83	0.361	17.52	1.40

* Weight, W, equals 10,000 kips or 44,482 kN

PF is modal participation factor

EMMR is effective modal mass ratio

Table 3: Damping Values for Structural Systems

Structural System	Freeman et al. (1975)		Freeman et al. (1984)	
	Yield Limit	Ultimate Limit	Elastic-Linear	Post-Yield
Structural Steel	2%	5%	3%	7%
Reinforced Concrete	5%	10%	5%	10%
Masonry Shear Walls	2%	10%	7%	12%
Wood	5%	10%	10%	15%

For linear-elastic analysis, the pre-yield capacity is compared to the linear-elastic response spectra (LERS) viscously damped (e.g. 5% of critical damping). During excursions into the non-linear-inelastic range of response, energy will be absorbed by hysteretic damping. In addition, the non-linear behavior will prevent the resonant amplification assumed in the development of the LERS. Thus, there is a rational basis for reducing the response spectrum for inelastic response. How much of a reduction is a topic of debate that will be discussed later in this paper. For the PSNSY project, a very conservative approach was taken by using a modest increase in damping to represent the inelastic demand (e.g. increasing damping from 5% to 10% for a concrete structure, refer to Table 3, under the heading "Freeman et al. (1975)". The loss of stiffness that results from inelastic response will lengthen the effective period (T) that generally reduces the acceleration but increases the displacement. The process of reducing response spectra and taking into account period lengthening to represent inelastic response was based on observations and studies of buildings and recorded building response (Czarnicki et al., 1975; Freeman, 1978; Freeman et al., 1977).

The process of plotting the capacity spectrum with varying damped response spectra later became known as the Capacity Spectrum Method (CSM). The procedure is illustrated in Figure 2. Figure 2a shows a sample capacity curve. The curve represents the fundamental mode of vibration, which was appropriate for the buildings in the PSNSY study. The capacity curve is converted to a capacity spectrum (Figure 2b, Sa versus Sd) by means of dynamic modal participation factors as illustrated in Table 2. The capacity spectrum can also be expressed in terms of Sd versus T (Figure 2c) and Sa versus T (Figure 2d). The Sa versus T format was used for PSNSY because that was the common format for response spectra at that time. The response spectra in Figure 2 represent two earthquake demands. The smaller curve (lime green) represents a smaller earthquake (EQ-I) for which the building is expected to remain elastic (i.e., the response spectrum intersects the capacity spectrum below the yield point. The larger curve (red) represents the major earthquake (EQ-II), which has been adjusted by an appropriate damping factor to represent inelastic response (e.g. 10% damped instead of the 5% damped used for EQ-I). The fact that the capacity spectrum crosses the EQ-II spectrum satisfies the criteria (i.e., the capacity spectrum punches through the demand envelope to survive the earthquake demands). The point of crossing indicates the degree of estimated damage.

Sample values for reducing 5% damped spectra are shown in Table 4 and relationships between ductility, μ (mu), and damping, β (beta), are shown in Table 5 (adapted from Freeman (1998a)). A process for interpolating between damped response spectra and the intersection of the capacity spectrum will be discussed in detail later in this paper. More information on the PSNSY project is found in Freeman et al. (1975).

Table 4: Spectral Reduction Factors versus Damping (β)

Effective Damping (β), %	Newmark-Hall One-Sigma SRA	Newmark-Hall One-Sigma SRV	Newmark-Hall Median SRA	Newmark-Hall Median SRV	*ECS-94 European*
5	1.00	1.00	1.00	1.00	*1.00*
7	0.86	0.90	0.91	0.93	*0.88*
10	0.72	0.80	0.78	0.83	*0.76*
15	0.58	0.70	0.64	0.73	*0.64*
20	0.46	0.60	0.55	0.66	*0.56*
25	0.38	0.53	0.48	0.60	*0.51*
30	0.32	0.47	0.42	0.55	*0.47*
35	.--	.--	0.38	0.52	*0.43*
40	.--	.--	0.33	0.50	*0.41*

Table 5: Effective Damping (β) versus Ductility (μ)

Effective Ductility, μ	WJE (1996) One-Sigma	WJE (1996) Median	ATC (1996) Type A 0	ATC (1996) Type A 0.1	ATC (1996) Type B 0	ATC (1996) Type B 0.1	ATC (1996) Type C 0	ATC (1996) Type C 0.1	From Figure 1 of Priestley et al. (1996)	New-mark Hall
	Effective Percent of Critical Damping, β		ATC (1996) (r = 0 and 0.10)							
1	5	5	5	5	5	5	5	5	5	5
1.25	7.5	8.5	18	16	13	12	9	9	--	8.3
1.5	10	12	24	23	18	17	11	11	--	11
2	14	16	33	29	25	22	16	14	13	17.5
3	21	26	39	33	29	25	19	16	17	27.5
4	26	35	40	34	29	25	20	16	19	35.5
6			40	33	29	25	20	16	21	46
8			40	31	29	24	20	15	22	54

2. Sample Buildings

After the PSNSY project was completed, the process was used for several case-studies where buildings had experienced ground motion that had been recorded. Included are two full scale four-story reinforced concrete test structures, built on the Nevada test site (NTS), that were subjected to ground motion from underground nuclear explosions (Freeman et al., 1976, 1977). Data available from the 1971 San Fernando earthquake was used for a CSM study of two reinforced concrete 7-story hotel buildings (Freeman, 1978). Variations of the CSM procedure were used in an Applied Technology Council (ATC) study on the correlation between earthquake ground motion and building performance (ATC, 1982). The CSM was applied to an example steel frame structure to compare varying design considerations to performance capabilities (Freeman, 1987). After the 1989 Loma Prieta earthquake, several buildings were evaluated using CSM (Mahaney et al., 1993). It was in this paper that Sa versus Sd format for response spectra (ADRS) was introduced as an alternative to the Sa versus T format.

3. Giving the Procedure a Name

Circa 1983, the process was being developed as a procedure for the seismic design of essential buildings as a supplement to the Tri-services (i.e., U.S. Army, Navy, and Air Force) Seismic Design manual. It was during this time that Dr. Peter Gergely had been reviewing the graphical concept described in the then available reference documents. In a telephone conversation, he expressed interest in the procedure, but suggested that to be successful it needed a name. He suggested "capacity spectrum method", which this author accepted, and included it to describe the procedure in the upcoming manual discussed below. The late Professor Gergely had encouraged research at Cornell University on the validity of the CSM procedure. One example of such research is included in a thesis by G.R. Searer (Searer, 1994).

TRI-SERVICES DESIGN MANUALS

Over the years, the Departments of the Army, the Navy, and the Air Force have published technical manuals for design guidelines for new construction. The seismic design for buildings had generally followed the Uniform Building Code provisions as recommended by the Structural Engineers Association of California and modified by the Tri-services Committee. In the early 1980s, it was decided to provide guidelines for dynamic analysis provisions for essential buildings (e.g., hospitals, fire stations, etc.). The resulting "Seismic Design Guidelines for Essential Buildings" (Freeman et al., 1984; Army, 1986) provides a two-level dynamic analysis approach to design buildings. First, the building is designed to resist the lower level of earthquake motion by elastic behavior. Then the building is evaluated for its ability to resist the higher level earthquake with allowances for inelastic behavior. Two acceptable methods are presented, one being an updated and expanded version of the CSM approach initially developed for PSNSY. Recommended damping values were revised as shown on the right hand side of Table 3 (Freeman et al., 1984). The values of the primary, or more rigid, structural system governs; or if dual systems participate significantly, a weighted average can be used. Illustrative guidelines, such as a table similar to Table 2 and diagrams similar to Figure 2, are given. Detailed design examples are included and guidelines are given for nonstructural components as well as the structural system. The final document was published in 1986 (Army, 1986). It should be noted that the guidelines can also be applied to non-essential buildings as an alternative dynamic analysis procedure.

As building codes were being upgraded, it was decided circa 1993 to upgrade the 1986 document. At about midway into its development, the 1994 Northridge earthquake occurred. Advances were being made in the field of probabilistic and deterministic earthquake ground motion postulation and there was a greater interest in performance-based design procedures. In a quest to establish a rationale for surrogate damping values to represent inelastic response spectra, Nathan Newmark's approach (Newmark and Hall, 1982) was reviewed. Using Newmark's equations for determining damping reduction factors and those used for inelastic ductility reduction factors, an approximate relationship between percent damping (β) and ductility (μ) was established. Examples of these relationships are shown in Table 5, under the headings "Effective Ductility", "WJE 1996", and "Newmark-Hall". For example, on the basis of a 5% damped LERS, an inelastic response spectrum for a ductility of 3 may be approximated by using roughly 27% damping. These values are substantially more liberal than those used for PSNSY and proposed in the earlier guidelines as shown in Table 3. Due to a delay in completing the updated guidelines and a change in Department of the Army policy of publishing technical manual, the final manuscript of the guidelines were not published. The final manuscript is presently being modified for possible distribution to interested parties (WJE, 1996).

ATC 40, SEISMIC EVALUATION AND RETROFIT OF CONCRETE BUILDINGS

Under the direction of the State of California, the Applied Technology Council (ATC) was assigned the task of developing guidelines for the seismic evaluation and retrofit of concrete buildings designed and constructed by earlier seismic design standards (ATC, 1996). As a result of a previous study, the CSM was selected as a recommended procedure. Initial drafts of the CSM procedure generally followed the format established in the Tri-services update (WJE, 1996). As the ATC document was being developed, the question arose regarding use of damped spectra representing inelastic spectra. There had been a school of thought that felt the surrogate damping values should be directly linked to hysteretic damping based on energy loss due to hysteretic cyclic behavior. When this method is used, the resulting inelastic reduction factors appear to most researchers to be too large (i.e., unconservative). In order to compensate for this concern, it was decided to identify three categories of reduction factors: Type A at 100%, Type B at two-thirds, and Type C at one-third the hysteretic reduction. It was generally accepted, at least by this author, that Types **B** and C would apply to existing concrete buildings and that the Type A 100% solution would not apply and could be resolved at a later date. The resulting proposed damping values are listed in Table 5 under the heading "ATC 1996". Although the ATC document went through a detailed review process and was the subject of a number of workshops, its publication has created many ensuing interesting discussions and debates.

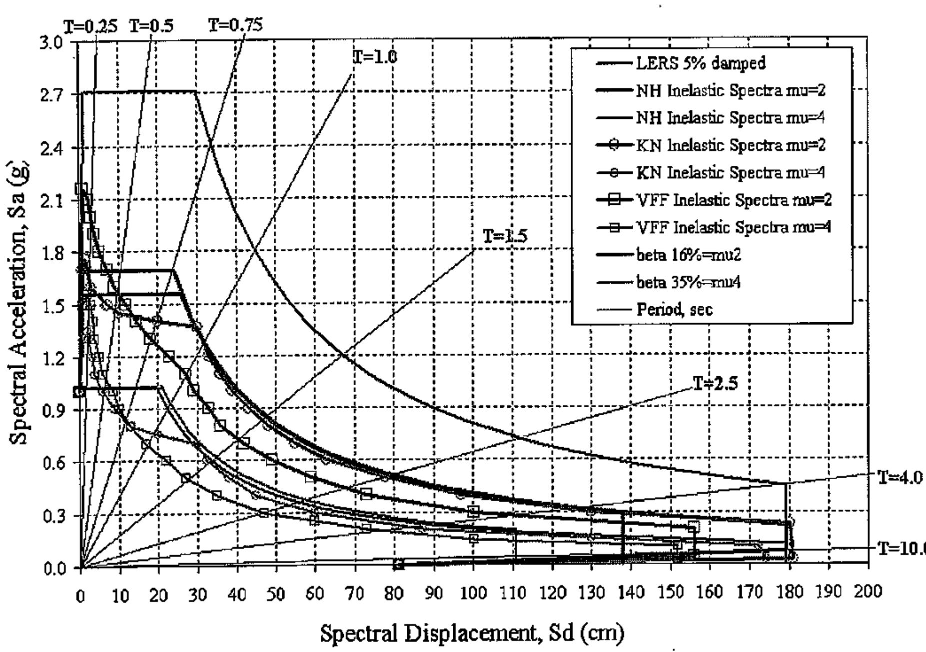

Fig. 3 Inelastic response spectra and damped spectra

CRITIQUES AND ALTERNATIVES

This author has watched with interest as groups of researchers discuss the fine points about which PBSD procedures give the "best" results. In recent years, there has been substantial research and discussion on the merits of inelastic response spectra and equivalent (surrogate) damped spectra and on the appropriateness of using damped spectra to represent inelastic response (e.g., Chopra and Goel, 1999; Fajfar, 1998; Judi et al., 2002). Although the conclusions of these researchers are not wholly consistent with each other, it has been claimed by some (Chopra and Goel, 1999) that use of damped spectra may lead to less conservative results as compared to inelastic spectra. The comparisons, in general, are based on the ATC 40 Type A damped spectra as discussed above. When comparisons are made with other surrogate damping procedures, such as those used in the PSNSY study and Tri-services manuals (Tables 3 and 5), claims of non-conservative results tend to disappear. An example is given in Figure 3. Two examples of inelastic response spectra (KN and VFF, from Chopra and Goel (1999)) are compared to Newmark-Hall ductility reductions and WJE 1996 equivalent damping, for ductility ratios of $\mu = 2$ and 4. As shown in Figure 3, the variations are not great (e.g. a 10% to 15% spread).

In many studies, researchers sometimes define an "exact" solution and compare other "approximate" solutions to this "exact" solution. Often, a non-linear dynamic time history is deemed to be the baseline "exact" solution. This author finds it difficult, however, to understand how any such solution is "exact." Every computer analysis requires development of a computer model of the building's structural system. In the computer, the model is then subjected to a digitized version of each of the ground motions. At best, the computer model is an idealized mathematical model that is based on a mélange of assumptions regarding the strength, behavior, and configuration of the component structural materials and assemblages. Therefore, the computer analysis is, at best, able to generate an exact numerical solution to a reasonable but inexact set of assumptions.

To anoint non-linear time histories as the benchmark against which all other non-linear methods should be compared is without basis. It is yet to be shown that any published non-linear time history evaluation accurately describes how a building has actually performed in a past earthquake, even though such an analysis is "after the fact" and benefits from prior knowledge of the building's actual performance. Recognizing that "after the fact" non-linear time histories are – more often than not – liberally tweaked in order to arrive at a reasonable facsimile of actual performance, it is dubious that a "before the fact" non-linear time history can be relied upon to correctly predict the response of a building to a yet-to-occur earthquake with unknown characteristics. To accurately predict how a building did or will perform, one must have accurate data on both demand and capacity; however, the accuracy of the answers cannot be greater than the accuracy of the input data. This is not to say that non-linear time history analysis does not provide important information about structural response; but it is not "exact".

It is important to try to understand why procedures, that appear to be rational, do not always give consistent results. In the case of inelastic response spectra, the peak displacements often include inelastic baseline shifts that can be very sensitive to the hysteretic model used in the analysis (Freeman et al., 2004). Interesting research on this topic, as well the use of equivalent damping, has been done by Hayder Judi and his associates (Judi et al., 2002 and correspondence by email).

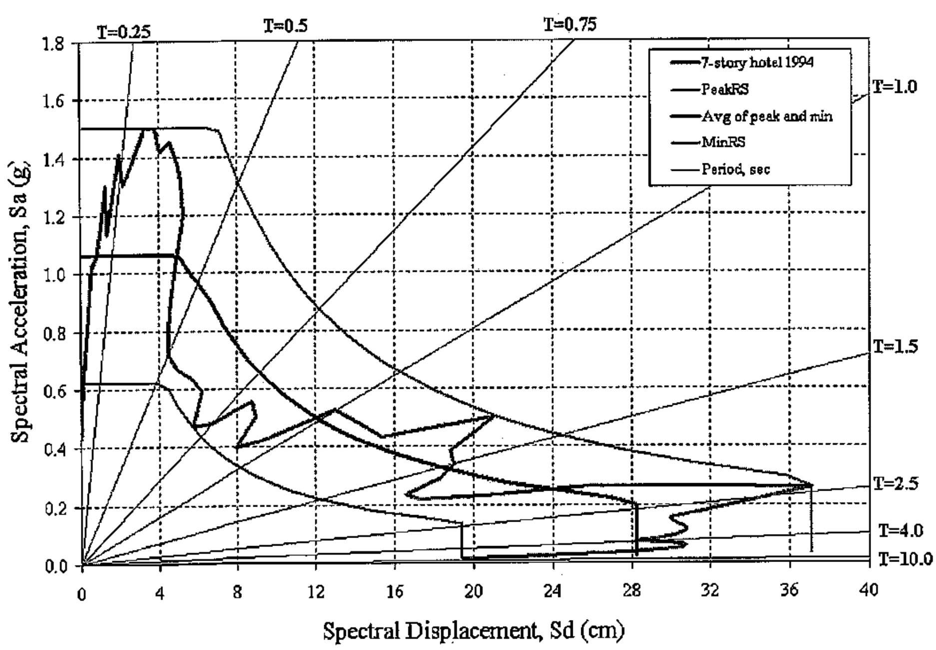

Fig. 4 7-story hotel: smoothing spectrum - Northridge 1994

As PBSD procedures become common, engineers must also be aware of the approximate nature of using smooth design response spectra as a basis for design of a structure to perform in a future earthquake. An example is given in Figure 4, where the green jagged 5% damped response spectrum represents the ground motion experienced in the 1994 Northridge earthquake. The upper (red) curve represents a code type smooth spectrum that envelopes the spikes of the measured spectrum, while the lower (purple) curve represents a lower limit defined by the valleys. The average between the maximum and minimum envelopes is represented by the heavy (black) curve. It can be seen that there can be vast differences between the potential response and a code projected response, depending on the period of the structure. The approximate nature of any PBSD procedure must be taken into account when designing buildings.

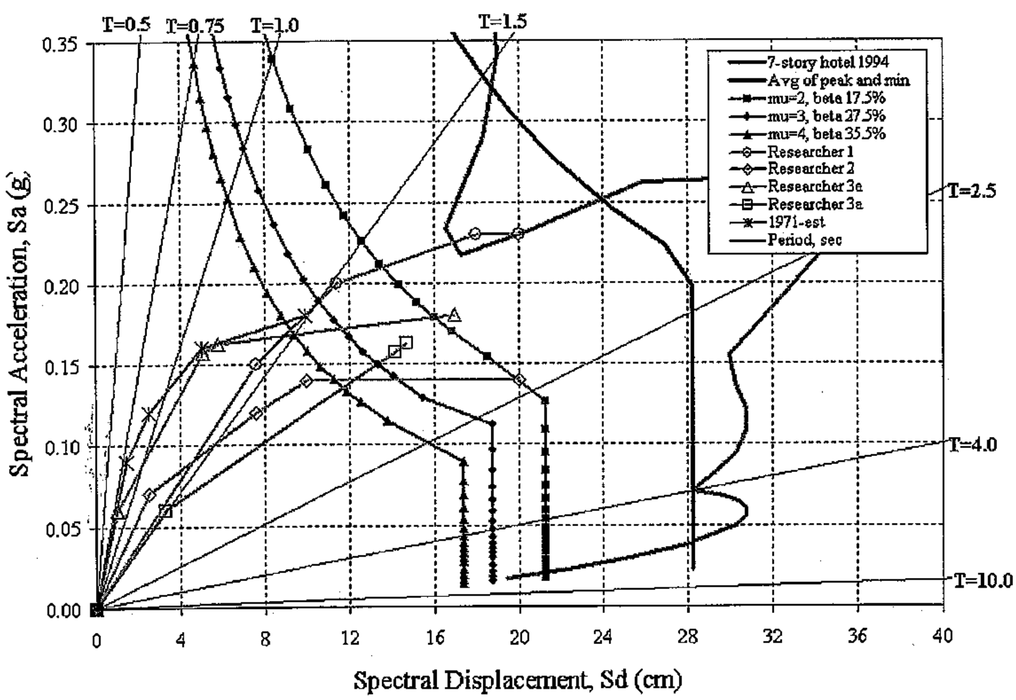

Fig. 5 7-story hotel: CSM study - Northridge 1994

An example of how calculated pushover curves can vary is illustrated in Figure 5. The graph is a close up of Figure 4, where the irregular (green) curve represents the earthquake spectrum and the heavy smooth (black) curve represents the average smooth spectrum. As a part of the CSM procedure, damped spectra at β = 17.5%, 27.5% and 35.5% are shown to represent ductility ratios of μ = 2, 3 and 4, respectively. Also shown are representations of capacity spectra developed by several researchers for the subject 7-story building that was damaged by the 1994 Northridge earthquake (Gilmartin et al., 1998). Although there appears to be substantial differences in the calculated capacity spectra, there is general agreement that the building was subjected to damaging ground motion and would have to exhibit ductility ratios of 3 or 4 to survive. Figure 6 illustrates a graphical procedure for estimating inelastic displacements by CSM (Freeman, 2000). By matching ductility ratio markings on the capacity spectrum with the closest effective damped spectrum, the ductility demand of 2.5 and the displacement of 16 cm can be estimated.

Alternatives to CSM include Priestley's direct displacement-based design method (DDBD) and Aschheim's yield point spectra (YPS). In a sense, DDBD (Priestley et al., 1996) is a reverse process of CSM. The target displacement and ductility ratio is selected to determine the required elastic period and strength required for the design. Equivalent viscous damping is used as shown in Table 5. They appear to be most consistent with ATC 40 Type C. The YPS procedure (Aschheim, 1999) is an extension of CSM, but it makes use of standardized inelastic spectra. In a sense, it is a cross between CSM and DDBD. The inelastic spectral displacements are reduced by the ductility ratios to form an elastic design spectrum.

PBSD procedures are generally limited to fundamental modes of vibration. For tall buildings, the participation of higher modes can be significant. Procedures for including higher mode effects have been presented that are based on the CSM concept (Paret et al., 1996; Sasaki et al., 1998). Other researchers are addressing this important issue.

Another resource for developing and verifying PBSD procedures is the use of building response records (Gilmartin et al., 1998). By carefully studying recorded building motion records, modal responses can be filtered out and pushover characteristics can be identified.

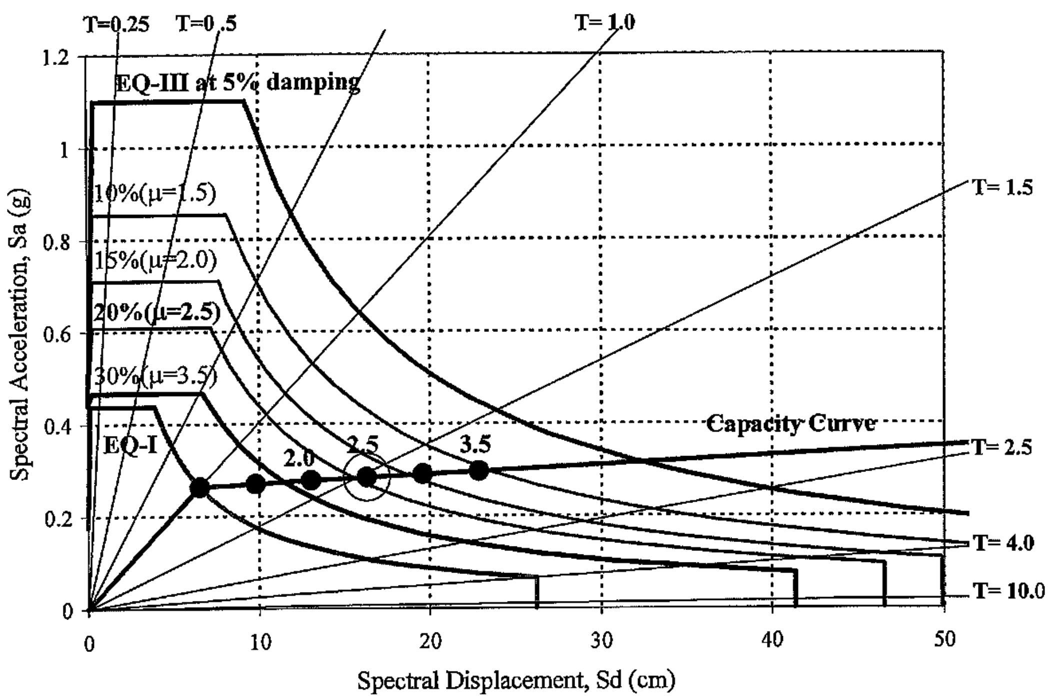

Fig. 6 Capacity spectrum method (CSM)

CONCLUSIONS

Over the years, PBSD procedures have evolved significantly from their humble beginnings; however, there has been a push to develop increasingly complex, codified PBSE procedures. In the author's opinion, by codifying PBSD, the very essence of PBSD, i.e. the focus on the attributes and behavior of an individual building, is destroyed. Engineers must be given sufficient latitude to arrive at the best estimate of a building's capacity (Freeman et al., 2004).

PBSD can be a useful tool for design and to estimate the performance characteristics of buildings subjected to strong earthquake ground motion. There is no "magic bullet" single procedure. It takes a combination of analytical procedures, data evaluation, judgment, experience and peer review to get a credible approximation of how a building works in the inelastic range of lateral motion (Freeman and Paret, 2000).

The CSM stands up well when compared to other PBSD procedures and has the added advantage of giving the engineer the opportunity to visualize the relationship between demand and capacity. Differences between the various methodologies have more to do with unknowns in material behavior and quantification of energy dissipation than in the methods of analysis (Freeman, 1998b).

Research on PBSD procedures should be encouraged to close the gap between researchers and practicing structural engineers. There should be more interaction between the researchers and practicing structural engineers to resolve controversial issues and to form consensus.

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1993 NATIONAL EARTHQUAKE CONFERENCE
Earthquake Hazard Reduction in the Central and Eastern United States:
A Time for Examination and Action

THE CAPACITY SPECTRUM METHOD FOR EVALUATING STRUCTURAL RESPONSE DURING THE LOMA PRIETA EARTHQUAKE

James A. Mahaney[I], Terrence F. Paret[II], Brian E. Kehoe[II], Sigmund A. Freeman[III]

ABSTRACT

In regions of the central and eastern United States, where maximum credible earthquake force levels are predicted to be significantly larger than building code design force levels, the need to predict structural performance and survivability is great. Following the 1989 Loma Prieta earthquake in California, the performance of many buildings were evaluated. This paper presents three case studies which were analyzed using the Capacity Spectrum Method (CSM): wood- framed residences; a modem reinforced concrete shearwall building; several framed buildings with brick infilled walls. The CSM is an approximate nonlinear analysis procedure for estimating building load-deformation characteristics and for predicting earthquake damage and structure survivability. The results of these analyses are plotted in an Acceleration-Displacement-Response Spectrum (ADRS) format and they correlate well with the damage observed after the earthquake. The method is recommended for predicting the performance of buildings under postulated earthquake scenarios and is a very useful analytical tool for evaluating buildings in the eastern and central United States.

INTRODUCTION

Assurance of adequate structure performance in response to very strong ground motion is a design goal implicit within all modem building codes, but because regional seismicity is defined on a probabilistic basis, this goal is not equally attainable by code-based designs in all seismic regions. Given a maximum credible event, structures in areas of low to moderate code-defined seismic zones may actually be at the most risk.

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In regions of central and eastern United States, the maximum credible earthquake is predicted to be significantly larger than the design level earthquake anticipated by the design methodologies contained in the current building codes. In these regions the necessity of predicting structure performance and survivability in response to a maximum credible event is greatest.

The Capacity Spectrum Method (CSM) is an approximate nonlinear analysis procedure for estimating the ability of a structure to resist the forces and deformations caused by earthquake ground motion and for predicting structure survivability. The procedure involves computation of a force-displacement or acceleration-displacement curve for a structure of interest and comparison of this curve with the spectral demands computed for any particular earthquake. This procedure was originally developed as a rapid evaluation method for a pilot seismic risk project for the Puget Sound Naval Shipyard (Freeman, et al. 1975) and has been incorporated into the U.S. Army, Navy, and Air Force Manual on Seismic Design Guidelines for Essential Buildings (Army 1986).

During the Loma Prieta earthquake in California on October 17, 1989, a great number of buildings experienced peak horizontal ground motion in the range of 10 percent to 20 percent of gravity. A wide variety of building types were evaluated using the CSM. Building types included multistory frame structures with brick infill (typical of many central and eastern United States buildings), reinforced concrete structures built since the 1960?s, wood frame construction, and unreinforced brick masonry bearing wall buildings (also typical of central and eastern United States). In conjunction with data generated by seismic damage condition surveys and numerical analysis of the lateral force resisting systems of various buildings, response spectra derived from Loma Prieta ground motions were used to correlate observed structural and nonstructural damage with anticipated damage as predicted by the capacity spectrum procedure. In this paper, the CSM is described along with three case studies which are presented to demonstrate the usefulness of the method for predicting the response of building structures to earthquake ground motion.

CAPACITY SPECTRUM METHOD

The CSM requires constructing a global force-displacement capacity curve of a structure and comparing it to the assumed site-specific response spectrum earthquake demands. The CSM is not purported to be a theoretically accurate procedure, but one that provides a reasonable graphical description of how buildings have responded to earthquake ground motion. It can also be used for predicting how a structure might perform in response to different ground motion scenarios. Additional references for the CSM include ATC 1982, Deirlein & Hsieh 1990, and Freeman 1978, 1987.

The capacity curve is determined by statically loading the structure with lateral forces and calculating the roof displacements and base shear coefficients that define first significant yielding of structural elements. The yielding elements are then relaxed to form plastic hinges. Additional incremental lateral loading is applied until a nonlinear static capacity curve is generated. The structure is assumed to withstand cycling and behave in a hysteretic manner. The roof displacements and base shear coefficients are converted to spectral displacements (S_d) and spectral accelerations (S_a), respectively, by use of roof modal participation factors and effective modal weights as determined from dynamic characteristics of the fundamental mode of the structure. S_d is generally equal to about 70 to 80 percent of the roof displacement and S_a is roughly 1.1 to 1.3 times the base shear coefficient (Army 1986). The equivalent inelastic period of vibration (T) at various points along the capacity curve are calculated by use of the secant modulus

$$i.e.,\ T_i = 2\pi\ (S_{di}/S_{ai}\ g)^{1/2}$$

The demand curves are represented by earthquake response spectra and are presented at various levels of equivalent viscous damping. For example, the 5 percent damped curve may be used to represent the demand when the structure is responding elastically. The 10 percent and 20 percent damped curves may be used to represent the reduced demand in the inelastic range to account for hysteretic damping and anti-resonant nonlinear effects. The higher damped curves are used as a tool for reducing the spectrum and are not presented as necessarily being theoretically correct.

In this paper earthquake response spectrum demand curves are not presented in the standard S, vs. T format. Instead, the Acceleration-Displacement Response Spectrum (ADRS) format plots the spectral accelerations (S3) vs. the spectral displacements (Sd) with the period (T) represented by radial lines. An advantage of the ADRS format is that an engineer can simultaneously present both the force (i.e., acceleration times mass) and displacement demand characteristics of the earthquake and the capacity of the building. A structure can be shown to survive a particular earthquake if its capacity curve can extend beyond the envelope of the site-specific demand curve. In Figure 1, three site-specific, 5 percent damped, ADRS generated from Loma Prieta ground motion time histories are plotted. It is clear from these plots that each site experienced a unique ground motion, consistent with local soil conditions and epicentral distances.

Case Study 1: 1 & 2 Story Wood Framed Residential Buildings

The structures are located in Gilroy, California, approximately 16 miles from the epicenter. Their vertical load carrying systems are composed of prefabricated roof trusses, 2-inch wide (i.e., 2x) joist framing on the second floor, 2x interior and exterior stud walls, a first floor concrete slab-on-grade, and reinforced concrete foundations. The lateral systems for the one-story and two-story structures are composed of plywood diaphragms, wall let-in bracing and gypsum board stud walls. The structures are considered conventionally framed. Participation of the exterior lap/siding was not included in our analysis of the lateral system due to its relatively weak and flexible load/deformation characteristics.

During the Loma Prieta earthquake, the one-story structure sustained minor damage. Cracks in the gypsum board finishes radiating from the corners of doors and windows were observed. At selected locations, minor distress was noted at the taped gypsum board joints. The two-story structure sustained major first floor wall damage that consisted of let-in brace failures, exterior lap-siding, nail popping with minor splitting, buckling of the siding, and significant first floor gypsum board damage. On the second floor only minor cracks in the gypsum board finishes radiating from the corners of some doors and windows were observed.

Site-specific 5 and 15 percent ADRS were developed considering seismic records in the Gilroy area, site location with respect to the Loma Prieta epicenter, and local soil and rock profiles (Figure 2). Load-deformation curves were developed for each shearwall type and story masses were determined. Building capacity curves were developed considering the dynamic characteristics of the buildings, shearwall capacities and the level of damage observed at each floor level and are plotted in Figure 2 for a one-story building (Curve 1), a two-story building without plywood (Curve 2), and a hypothetical two-story building with supplemental plywood on the 1st level (Curve 3).

The ADRS capacity spectrum plots in Figure 2 correlate well with observed damage. For the one-story building (Curve 1), the building remains essentially elastic through the 15 percent damped curve. Since wood structures are typically expected to respond with 10 to 15 percent damping (Army 1986), only minor damage would be expected. The inelastic capacities of the 2-story buildings represented by Curves 2 & 3 (with and without 1st floor plywood shearwalls) are similar and exceed the 15 percent damped demands. Despite the similarity of their capacity curves, the 2-story buildings are predicted to experience dissimilar damage with the Curve 2 structure yielding at the 1st story and the strengthened Curve 3 structure yielding in the 2nd story.

If the elastic portion of Curves 1 and 3 were to be projected to intersect the 5 percent demand curve, the elastic displacements would be somewhat similar to the inelastic displacements. For Curve 2 the inelastic displacements are slightly less than the projected elastic displacement.

Case Study 2

The subject building is located in the San Francisco Bay Area and was designed under the 1982 Uniform Building Code (UBC). It is an eleven-story, square-shaped building. The roof and floors are post-tensioned concrete slabs. They are supported by cast-in-place reinforced concrete columns and eight walls which also act as shearwalls. The shearwalls are either 18-inches or 24-inches thick. Each column is supported by a group of prestressed concrete piles, and each shearwall is supported on two pile groups that are interconnected with a grade beam. The centerline of each of these pile groups is located near the end of each shearwall.

The building was severely damaged during the Loma Prieta earthquake. An elevation of one of the damaged shearwalls is shown in Figure 3. Superimposed on the elevation is a damage survey indicating crack extent, crack width, spalled concrete, and exposed and buckled reinforcing. The structural damage experienced by this shearwall is typical of the damage observed in the four walls comprising the lateral system for east-west earthquake loading, except the pattern of damage varied somewhat reflecting the variations of size and location of the openings in each wall. In each case, damage was concentrated in and appeared to have initiated in an area below the 2nd floor where there is a shearwall transition.

The building was instrumented on three floors, including the ground level, and Loma Prieta motion records were obtained for the ground level and for the floor below the roof. Unfortunately, the instrument in the building mid-height had been unplugged.

ADRS generated from the ground level records for 5 percent and 20 percent damping, are plotted in Figure 4. Superimposed on this plot is a representation of the path the building followed in the course of surviving the earthquake. To compute this path, the slope of the elastic portion of the curve was determined by, reading from the 9th floor time histories the approximate elastic periods of vibration which were in the range of 0.6 to 0.8 seconds. The elastic limits were computed on the basis of code strengths of the critical regions of the walls.

The maximum inelastic response of the building was also derived from the 9th floor records. The time histories clearly indicate an abrupt transition from an elastic period of vibration of 0.6 to 0.8 seconds to a period between 1.2 and 1.4 seconds. The maximum spectral acceleration experienced by the building was computed from the 9th floor record using the analytically derived dynamic building characteristics.

At least three important observations can be made on the basis of this plot. First, the curves indicate that the building achieved an equivalent viscous damping level between 10 and 15 percent of critical, which greatly reduced the demands on the building relative to the 5 percent damped response spectrum. Second, the curves indicate that the building experienced greater displacements than would have been predicted by the elastic analysis of the building responding to the 5 percent damped Loma Prieta earthquake spectrum. Finally, it is useful to observe that for this particular earthquake on this particular site, the period lengthening precipitated by localized failure of the critical regions of the shearwalls forced the building to move into, rather than out of, the period range of maximum response.

<u>Case Study 3</u>

Case Study 3 represents several 6 to 8-story buildings with steel or concrete frames and infill brick walls. Floor systems were either concrete or wood with lath and plaster ceilings. In some of these buildings the first story is relatively soft compared to the upper stories. In others, the stiffness distribution is more uniform with a weak story above the second floor. The buildings were analyzed taking into account the lateral capacity of the frames and the brick infill. Under earthquake loading, the brick initially resists the lateral forces until it cracks and the load is incrementally transferred to the frames. The cracking pattern varied from building to building and included X-cracking in the brick piers, and/or flexural cracking in the brick spandrels. Interior partitions consisting of hollow clay tile and/or lath and plaster walls also experienced cracking, indicating their participation in the lateral force resisting system. For the soft first story buildings, interior partitions on the first story experienced greater damage than upper story partitions. For other buildings, upper story damage to partitions and ceilings was greater.

The 5 and 20 percent damped ADRS in Figure 5 were developed from downtown Oakland seismic records. The 5 percent ADRS curve has a large demand peak in the period range of 1.0 to 1.5 seconds, which corresponds roughly to the estimated periods of the Case Study 3 buildings. If the buildings had remained essentially elastic, they would have been required to withstand spectral displacements of about 9 inches. Using the CSM, however, it was determined that the buildings only experienced spectral displacements of approximately 4 inches. For the soft first story buildings, most of the displacement (2-3 inches) occurred in the first story while the remainder (1-2 inches) occurred in the upper stories. Thus, interstory drifts at the upper stories were less than 0.5 inch. For the other buildings, interstory drifts approached 1 inch.

Brick and partition damage was consistent with the deformations predicted using the CSM. Fortunately, for this particular earthquake at this particular site, spectral displacement demands diminish with period lengthening beyond 1.5 seconds. If larger displacements had occurred, there would have been more degradation of the brick infill, with possible excessive displacement and vertical instability occurring in the soft first stories. By contrast, the standard UBC spectrum postulates spectral displacement demands that increase with period lengthening. If this characteristic had been present in downtown Oakland for the Loma Prieta earthquake, survival of the soft story buildings may have been questionable.

CONCLUSIONS

The CSM is shown to be a useful tool for predicting structural performance and survivability. For the three case studies presented, observed damage resulting from the Loma Prieta earthquake correlates well with damage predicted by the CSM.

The ADRS format provides an especially visual means for understanding the response of buildings to earthquake induced acceleration and displacement demands.

These case studies indicate that the 5 percent damped elastic earthquake displacement demands do not necessarily equal the actual inelastic displacement demands as is sometimes assumed.

It is important to correlate actual building performance with site-specific response spectra as well as to predict performance of buildings for postulated site-specific earthquake scenarios.

The use of the ADRS and the CSM is recommended for predicting response to maximum credible earthquakes, especially in the eastern and central United States.

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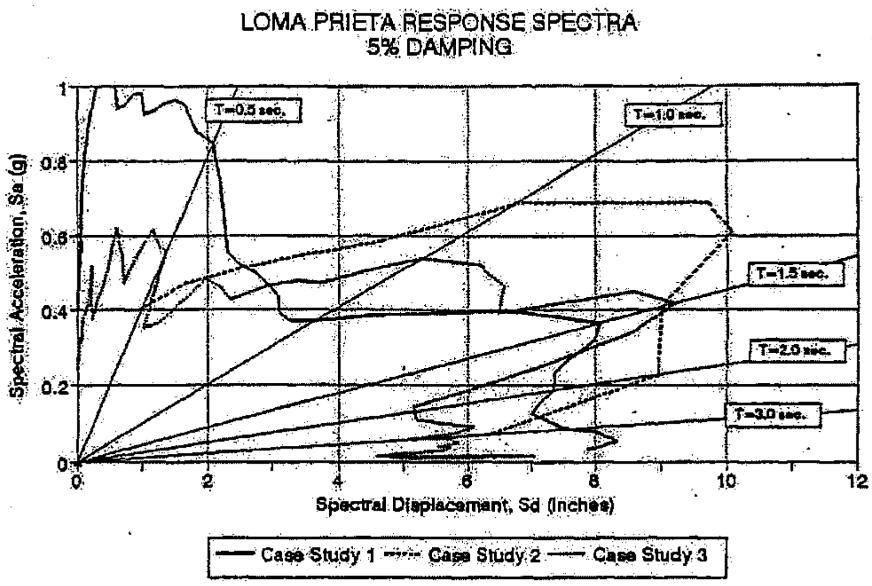


FIGURE 1

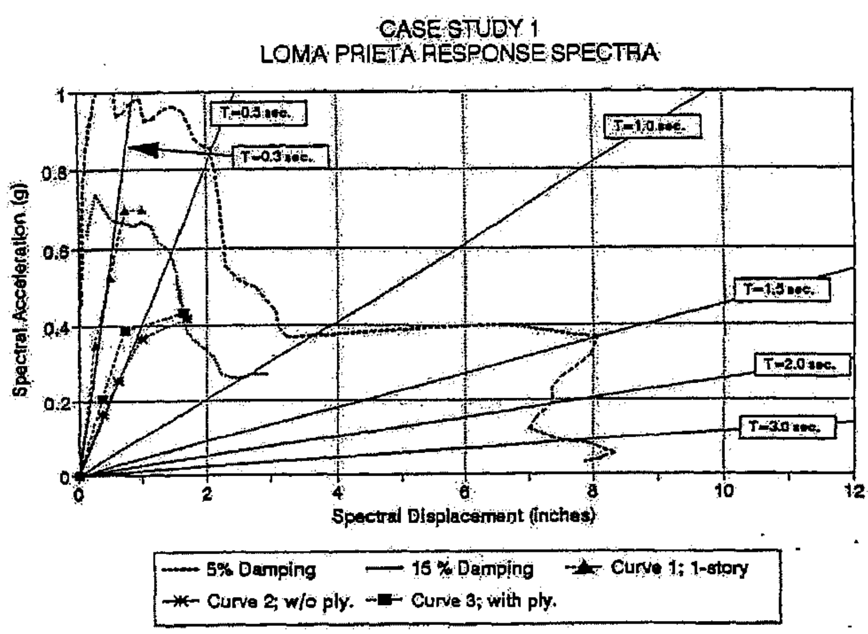


FIGURE 2

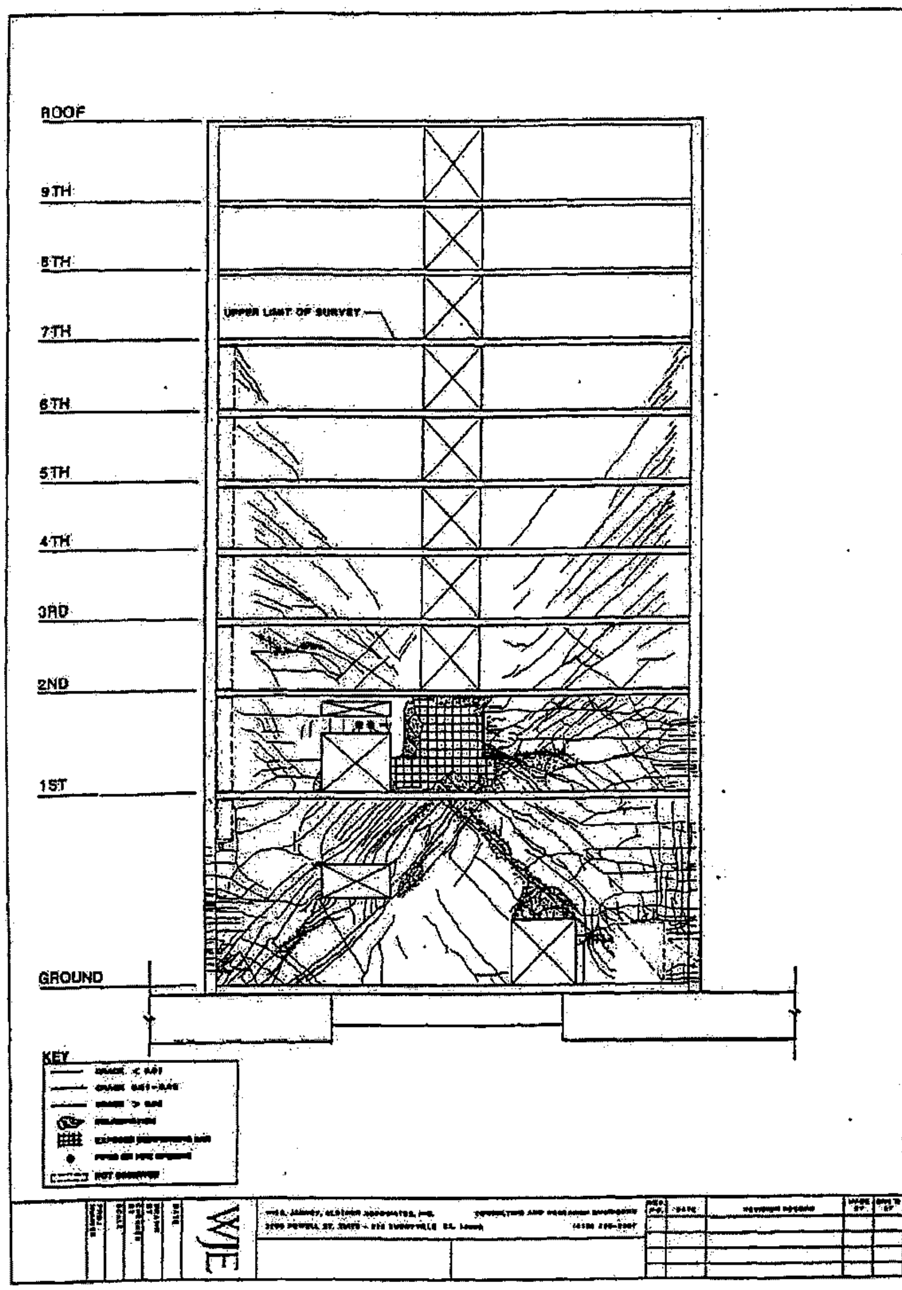

ROOF
9TH
8TH
UPPER LIMIT OF SURVEY
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
KEY
WJE

FIGURE 3

CASE STUDY 2
LOMA PRIETA RESPONSE SPECTRA

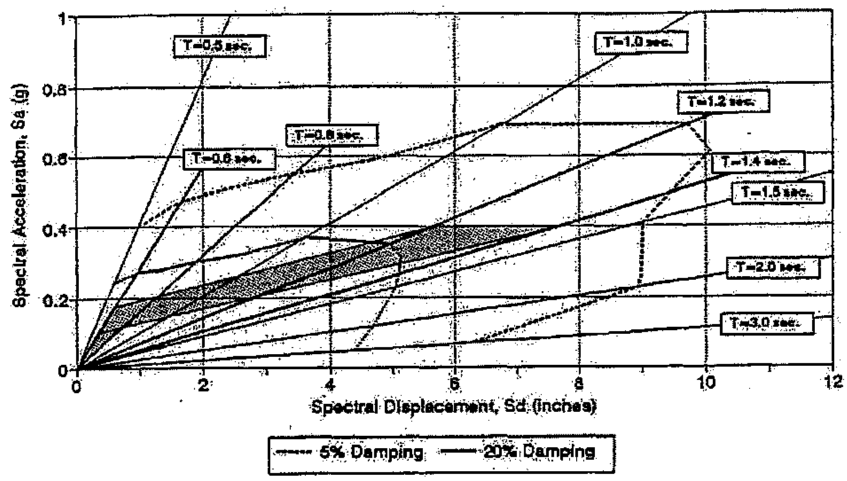


FIGURE 4

CASE STUDY 3
LOMA PRIETA RESPONSE SPECTRA

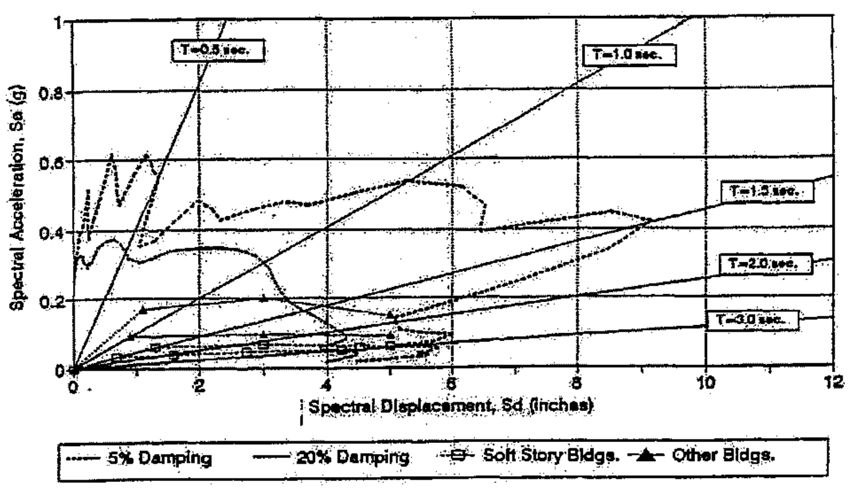


FIGURE 5

"C"

FEMA 306

EVALUATION OF EARTHQUAKE DAMAGED CONCRETE AND MASONRY WALL BUILDINGS

Basic Procedures Manual

Prepared by:

ATC

Applied Technology Council (ATC-43 Project)

555 Twin Dolphin Drive, Suite 550
Redwood City, California 94065

Prepared for:

The Partnership for Response and Recovery

Washington, D.C.

Funded by:

Federal Emergency Management Agency

1998

RC1B	COMPONENT DAMAGE CLASSIFICATION GUIDE	System: **Reinforced Concrete**
		Component Type: **Isolated Wall or Stronger Pier**
		Behavior Mode: **Flexure/Diagonal Tension**

How to distinguish behavior mode:

By observation:
For insignificant to moderate levels of damage, indications will be similar to those for RC1A, although shear cracking may begin at lower ductility levels. At higher levels of damage, one or more wide shear cracks hegin to form.

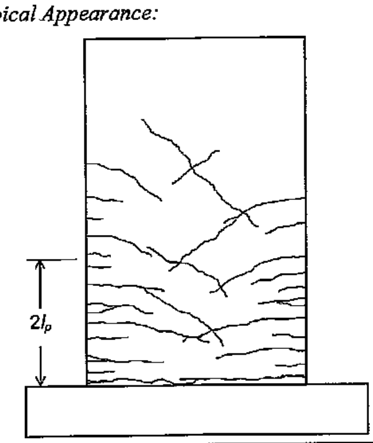
Typically occurs in walls that have a low-to-moderate amount of horizontal reinforcement, and which may have heavy vertical (flexural) reinforcement. May be most prevalent in walls with intermediate aspect ratios, $M/Vl_w \approx 2$, but depending on the reinforcement, can occur over a wide range of aspect ratios.

By analysis:
Shear strength calculated for conditions of low ductility exceeds flexural capacity, but shear strength calculated for conditions of high ductility is less than the flexural capacity.

Foundation rocking strength exceeds moment strength. Boundary ties are sufficient to prevent buckling of longitudinal bars and loss of confinement prior to shear failure. Wall thickness is sufficient to prevent overall buckling prior to shear failure. Sliding shear strength is not exceeded.

Refer to Evaluation Procedures for:

- Identifying plastic hinge locations and extent.
- Identifying flexural versus shear cracks.
- Calculation of moment, diagonal tension, web-crushing, sliding-shear, lap splice, and foundation rocking strength.
- Required boundary ties and wall thickness.

Severity	Description of Damage	Performance Restoration Measures
Insignificant $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* • Shear crack widths do not exceed 1/16 in., and • Flexural crack widths do not exceed 3/16 in., and • No significant spalling or vertical cracking. *Typical Appearance:*  Note: *l_p is length of plastic hinge. See Section 5.3.3*	(Repairs may be necessary for restoration of nonstructural characteristics.)

"D"

9-320

ASCE/SEI 31-03

American Society of Civil Engineers

Seismic Evaluation of Existing Buildings

This document uses both the International System of Units (SI) and customary units.

ASCE

SEI Structural Engineering Institute of the American Society of Civil Engineers

Published by the American Society of Civil Engineers

Table 3-1. Benchmark Buildings

Building Type[1, 2]	Model Building Seismic Design Provisions: NBC[ls]	SBC[ls]	UBC[ls]	IBC[ls]	NEHRP[ls]	FEMA 178[ls]	FEMA 310[ls, io]	CBC[io]
Wood Frame, Wood Shear Panels (Type W1 & W2)	1993	1994	1976	2000	1985	*	1998	1973
Wood Frame, Wood Shear Panels (Type W1A)	*	*	1997	2000	1997	*	1998	1973
Steel Moment-Resisting Frame (Type S1 & S1A)	*	*	1994[4]	2000	**	*	1998	1995
Steel Braced Frame (Type S2 & S2A)	1993	1994	1988	2000	1991	1992	1998	1973
Light Metal Frame (Type S3)	*	*	*	2000	*	1992	1998	1973
Steel Frame w/ Concrete Shear Walls (Type S4)	1993	1994	1976	2000	1985	1992	1998	1973
Reinforced Concrete Moment-Resisting Frame (Type C1)[3]	1993	1994	1976	2000	1985	*	1998	1973
Reinforced Concrete Shear Walls (Type C2 & C2A)	1993	1994	1976	2000	1985	*	1998	1973
Steel Frame with URM Infill (Type S5, S5A)	*	*	*	2000	*	*	1998	*
Concrete Frame with URM Infill (Type C3 & C3A)	*	*	*	2000	*	*	1998	*
Tilt-up Concrete (Type PC1 & PC1A)	*	*	1997	2000	*	*	1998	*
Precast Concrete Frame (Type PC2 & PC2A)	*	*	*	2000	*	1992	1998	1973
Reinforced Masonry (Type RM1)	*	*	1997	2000	*	*	1998	*
Reinforced Masonry (Type RM2)	1993	1994	1976	2000	1985	*	1998	*
Unreinforced Masonry (Type URM)[5]	*	*	1991[6]	2000	*	1992	*	*
Unreinforced Masonry (Type URMA)	*	*	*	2000	*	*	1998	*

[1] "Building Type" refers to one of the Common Building Types defined in Table 2-2.
[2] Buildings on hillside sites shall not be considered Benchmark Buildings.
[3] Flat Slab Buildings shall not be considered Benchmark Buildings.
[4] Steel Moment-Resisting Frames shall comply with the 1994 UBC Emergency Provisions, published September/October 1994, or subsequent requirements.
[5] URM buildings evaluated using the ABK Methodology (ABK, 1984) may be considered benchmark buildings.
[6] Refers to the GSREB or its predecessor, the Uniform Code of Building Conservation (UCBC).

[ls] Only buildings designed and constructed or evaluated in accordance with these documents and being evaluated to the Life Safety (LS) Performance Level may be considered Benchmark Buildings.
[io] Buildings designed and constructed or evaluated in accordance with these documents and being evaluated to either the Life Safety or Immediate Occupancy (IO) Performance Level may be considered Benchmark Buildings.

* No benchmark year; buildings shall be evaluated using this standard.
** Local provisions shall be compared with the UBC.

NBC = *National Building Code* (BOCA, 1993).
SBC = *Standard Building Code* (SBCC, 1994).
UBC = *Uniform Building Code* (ICBO, 1997)
GSREB = *Guidelines for Seismic Retrofit of Existing Buildings* (ICBO, 2001).
IBC = *International Building Code* (ICC, 2000).
NEHRP = FEMA 368 and 369, *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* (BSSC, 2000)
FEMA 178 (*See* BSSC, 1992a)
FEMA 310 (*See* FEMA, 1998)
CBC = *California Building Code, California Code of Regulations, Title 24* (CBSC, 1995).

CITY AND COUNTY OF SAN FRANCISCO

BUILDING CODE 2010 Edition

[Includes legislation adopted by the Board of Supervisors through November 30, 2011, and Administrative Bulletins approved through January 1, 2011.]

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Chapter 16
STRUCTURAL DESIGN

SECTION 1604 – GENERAL DESIGN REQUIREMENTS

1604.11. Add the following section:

1604.11 Minimum lateral force for existing buildings.

1604.11.1 General. This section is applicable to existing buildings when invoked by Section 3401.8. This section may be used as a standard for voluntary upgrades.

An existing building or structure which has been brought into compliance with the lateral force resistance requirements of the San Francisco Building Code in effect on or after May 21, 1973, shall be deemed to comply with this section except when a vertical extension or other alterations are to be made which would increase the mass or reduce the seismic resistance capacity of the building or structure.

1604.11.2 Wind forces. Buildings and structures shall be capable of resisting wind forces as prescribed in Section 1609.

1604.11.3 Seismic forces. Buildings and structures shall comply with the applicable provisious of Sections 1613, except that, when compliance with this section is required by Section 3401.8, then structures and elements may be designed for seismic forces of not less than 75 percent of those given in Section 1613, and the building separation limitations of Section 1613.8 do not apply.

When upper floors are exempted from compliance by Section 3401.8, the lateral forces generated by their masses shall be included in the analysis and design of the lateral force resisting systems for the strengthened floor. Such forces may be applied to the floor level immediately above the topmost streugthened floor and distributed in that floor in a manner consistent with the construction and layout of the exempted floor.

In lieu of meeting the specific requirements of this section, an alternative lateral analysis procedure incorporating inelastic behavior may be submitted and approved in accordance with rules and regulations adopted by the Building Official pursuant to Section 104A.2.1.

1604.11.4 Design values for existing materials. The incorporation of existing materials, construction and detailing into the designed lateral force system shall be permitted when approved by the Building Official. Minimum quality levels and maximum load and stress values shall comply with Table 16C-D of this code, Tables 8-8-A and 8-8-B of the State Historical Building Code, or with other rules, regulations and standards adopted by the Building Official pursuant to Section 104A.2.1.

1604.12. Add the following section:

1604.12 Earthquake recording instrumentation. This section is adopted by the City and County of San Francisco for the purpose of evaluating the performance of instrumented building in earthquakes.

1604.12.1 General. Every building over six stories in height with an aggregate floor area of 60,000 square feet (5574 m^2) or more, and every building over 10 stories in height regardless of floor area, shall be provided with not less than three approved recording accelerographs. The accelerographs shall be interconnected for common start and common timing.

1604.12.2 Location. The instruments shall be located in the basement, midportion, and near the top of the building. Each instrument shall be located so that access is maintained at all times and is unobstructed by room contents. A sign stating MAINTAIN CLEAR ACCESS TO THIS INSTRUMENT shall be posted in a conspicuous location.

Reinforced Concrete Structures

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New Zealand

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10.4 CONTROL OF CRACKING

10.4.1 The Need for Crack Control

The occurrence of cracks in reinforced concrete structures is inevitable because of the low tensile strength of concrete. The tensile resistance of concrete is normally neglected in design. Structures designed with low steel stresses at the service load serve their intended function with very limited cracking. In many cases no cracking is visible at all because many members are not subjected to their full service load and the concrete has some tensile strength. However with high service load steel stresses, particularly as a result of the use of high-strength steel, some cracking must be expected at the service load. The cracking of a reinforced concrete structure at the service load should not be such as to spoil the appearance of the structure or to lead to corrosion of the reinforcement. These two requirements are considered next.

Aesthetic Considerations

The maximum size of a crack that may be considered nondetrimental to the appearance of a member, or nonconducive to feelings of alarm, depends on the position, length, width, illumination, and surface texture of the crack. The social background of the users and the type of structure also exert an influence. The limits on aesthetic acceptability are difficult to set because of the variability of personal opinion. The maximum crack width that will neither impair a structure's appearance nor create public alarm is probably in the range 0.010 to 0.015 in (0.25 to 0.38 mm), but larger crack widths may be tolerated.

Protection Against Corrosion

Portland cement concrete usually provides good protection for embedded reinforcing steel against corrosion. The protective value of the concrete is due mainly to its high alkalinity. If chemical agents such as carbon dioxide (producing carbonic acid) penetrate to the concrete surrounding the steel, the alkalinity is neutralized and the corrosion-inhibiting properties are reduced. Chlorides from deicing salts, sea spray, and so on, are also extremely active corrosion agents. Concrete of low permeability resists the penetration of corrosion agents. The main factors affecting the rate of diffusion of corrosion agents to the steel are the permeability of the concrete, the thickness of the concrete cover, the width, shape, and length of cracks, and the period of time the cracks are open.