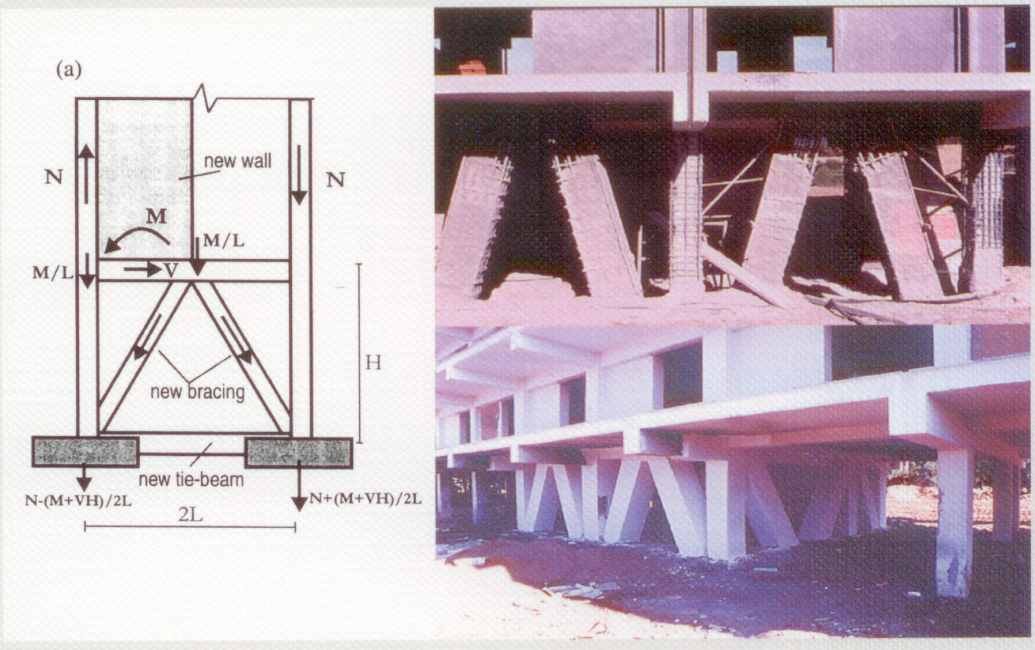
bulletin 24

state-of-art report

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Seismic assessment and retrofit of reinforced concrete buildings

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Seismic assessment and retrofit of reinforced concrete buildings

State-of-art report prepared by

Task Group 7.1

May 2003

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Manual or Guide (to good practice)	approved by the Steering Committee of *fib* or its Publication Board
Recommendation	approved by the Council of *fib*
Model Code	approved by the General Assembly of *fib*

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Cover picture: Foundation of new wall through diagonal concrete braces (see Fig. 5-38)

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Preface

In most parts of the developed world, the building stock and the civil infrastructure are ageing and in constant need of maintenance, repair and upgrading. Moreover, in the light of our current knowledge and of modern codes, the majority of buildings stock and other types of structures in most of the world are substandard and deficient. This is especially so in earthquake-prone regions, as, even there, seismic design of structures is relatively recent. In those regions the major part of the seismic threat to human life and property comes from old buildings.

Due to the infrastructure's increasing decay, frequently combined with the need for structural upgrading to meet more stringent design requirements (especially against seismic loads), structural retrofitting is becoming more and more important and receives today considerable emphasis throughout the world. In response to this need, a major part of the *fib* Model Code 2005, currently under development, is being devoted to structural conservation and maintenance. More importantly, in recognition of the importance of the seismic threat arising from existing substandard buildings, the first standards for structural upgrading to be promoted by the international engineering community and by regulatory authorities alike are for seismic rehabilitation of buildings. This is the case, for example, of Part 3: *Strengthening and Repair of Buildings* of Eurocode 8 (i. e. of the draft European Norm for earthquake-resistant design), and which is the only one among the current (2003) set of 58 Eurocodes attempting to address the problem of structural upgrading. It is also the case of the recent (2001) ASCE draft standard on *Seismic evaluation of existing buildings* and of the 1996 *Law for promotion of seismic strengthening of existing reinforced concrete structures* in Japan.

As noted in Chapter 1 of this Bulletin, ***fib***, and before it CEB and FIP, have placed considerable emphasis on assessment and rehabilitation of existing structures. The present Bulletin is a culmination of this effort in the special but very important field of seismic assessment and rehabilitation. It has been produced over a period of 4 years by a truly international team of experts, representing the expertise and experience of all the important seismic regions of the world. In the course of its work the team had six plenary two-day meetings: in January 1999 in Pavia, Italy; in August 1999 in Raleigh, North Carolina; in February 2000 in Queenstown, New Zealand; in July 2000 in Patras, Greece; in March 2001 in Lausanne, Switzerland; and in August 2001 in Seattle, Washington. In October 2002 the final draft of the Bulletin was presented to public during the 1st ***fib*** Congress in Osaka. It was also there that it was approved by ***fib*** Commission 7 *Seismic Design*.

Patras, February 2003

Michael N. Fardis
Convener of TG 7.1

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1 Introduction

Earthquake engineering is a young field of multidisciplinary applied science, which developed over the last fifty years with remarkable progress. Significant achievements in seismology, in geotechnical and in structural engineering resulted in the accumulation of a broad knowledge base for producing safe structures against earthquake hazards. This knowledge has been successfully converted into well-accepted analysis methods, design procedures and construction techniques that are governed by earthquake resistant codes and standards in earthquake prone countries, sometimes enforced as laws. The point where earthquake engineering stands today is quite satisfactory, in the sense that new structures built in seismic regions possess an in-built margin of safety leading to acceptable performance under design level ground motions.

Such huge progress in earthquake engineering is not yet sufficient to reduce the losses from urban earthquakes to acceptable levels. Recent urban earthquakes in the U.S.A. (1994), Japan (1995), Turkey, Greece and Taiwan (1999) demonstrated the vulnerability of the existing building stock in seismic regions. Older hazardous buildings are responsible for the thousands of lives lost, and significant damage observed in the last decade. Existing substandard buildings perhaps outnumber possibly safe new buildings in the urbanized regions exposed to seismic hazard. Therefore the attention in earthquake engineering should focus more on existing than on new buildings, in order to bring acceptable solutions to earthquake prone societies, as indeed has been the case during the last decade. Seismic rehabilitation has emerged as a major topic in earthquake engineering and has become a prominent research field in countries carrying significant seismic risk.

During the rapid progress of earthquake resistant design and construction of (new) buildings in the last fifty years, the seismic safety profile of the building stock in the earthquake prone regions of the world did not change appreciably, despite higher attrition rates in developing countries. Without any surprise, damage was always observed in these substandard buildings in the past earthquakes, whereas the small portion of buildings possessing post-1980 seismic safety standards displayed highly satisfactory performances. Although the proportion of substandard buildings with acceptably low damage rates was usually higher than those with serious damage under the same earthquake shaking, this fact had usually been disregarded in the final decision process. Considering that the newer buildings complying with the post-1980 seismic codes, which have more or less the same format in all countries, have passed the live tests during the 1985 Mexico City, 1989 Loma Prieta, 1992 Erzincan, 1994 Northridge and 1995 Kobe earthquakes, a silent consensus have been reached for the rehabilitation concept of existing buildings: they must be upgraded to the safety levels of new buildings. The implications of this concept, however, were not rational at all. First, upgrading the individual components of an existing system to the standard of new components is technically impossible, especially in improving the substandard detailing of reinforced concrete members. Second, the cost paid even for achieving the general safety level of new buildings is unacceptably high, comparable in fact with the replacement cost. Third, a large-scale field application is necessary to upgrade a substandard building to the current code level, which would require its evacuation for a considerable length of time.

Social psychology after a damaging earthquake plays a more important role in developing the societal motive for seismic rehabilitation than the concept of preparedness for the next one. Building owners of a community that is freshly shaken by a disturbing earthquake, or who are afraid because of an earthquake which affected a nearby community, build up a strong potential for rehabilitating their building under social pressure. This is a transient motivation however, which fades away as the fear of the past earthquake loses its dominance over the society. On the other hand, long term preparedness efforts are hindered by the negative impact of the code-level upgrading concept stated above, manifested by high costs and long term disruptions in building use. Past experience confirms these observations. Seismic rehabilitation executed by individual building owners is marginal in all earthquake prone societies, and the bulk of rehabilitation work is carried out for public buildings, where cost and service disruptions are not the major concerns.

The demand for seismic safety assessment and rehabilitation of building stocks in large urban settlements is huge. There are hundred thousands of existing vulnerable buildings in large metropolitan cities at risk such as Istanbul, Athens, Los Angeles, Napoli, Tokyo and

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many others. Retrofitting all vulnerable buildings is not a realistic solution that can be applied before the next big earthquake. The highest priority lies in two extremes: Identifying the hazardous buildings which have a high possibility of collapse, and identifying those which ensure life safety despite being substandard. This requires simple, yet effective, assessment procedures that enable handling large number of buildings within a time. Based on the outcome of such simple seismic risk assessment procedures, buildings which do not possess life safety performance must be either demolished or rehabilitated. Social issues usually play a more important role in making such decisions than technical parameters. Strong public policies are required for the implementation of a large scale seismic rehabilitation program, e.g. to enable transfer of property rights for buildings to be demolished and provide financial support for the buildings for which rehabilitation is deemed feasible.

Seismic rehabilitation of large building stocks requires engineering approaches that are dramatically different from the traditional approaches of structural engineering. If seismic rehabilitation is going to become an essential element of earthquake preparedness, building owners should be regarded as the potential victims of a future earthquake, but not as clients. The retrofit solutions offered should focus more on developing simple, standard, cost effective techniques which satisfy minimum requirements for public safety with least disruption of social life. The performance objectives should be well pronounced, to provide clear perception and acceptance by society.

Performance-based seismic rehabilitation is a serious effort with growing appeal for identifying the strengths and weaknesses of existing substandard buildings and upgrading them to performance levels consistent with the objectives discussed herein. It is hoped that the methods presented in this *fib* Bulletin will serve as a strong technical support in overcoming the inherent difficulties of seismic rehabilitation. If so, they may lead to the standardization of assessment procedures and retrofit techniques.

fib, and before it CEB, has placed considerable emphasis on assessment and rehabilitation of existing structures. In the broader, non-seismic field, *fib* has published in 2002 bulletin 17: "Management, maintenance and strengthening of concrete structures", while CEB has produced in 1998 Bulletin 243: "Strategies for testing and assessment of concrete structures – Guidance Report", in 1989 Bulletin 192: "Diagnosis and assessment of concrete structures – State-of-the-art Report" and as early as in 1983, Bulletin 162: "Assessment of concrete structures and design procedures for upgrading (Redesign)". Although it has a general scope and does not mention seismic or cyclic loading in its title, this latter pioneering document places considerable emphasis on this type of loading. Moreover, as it has drawn from the early experience and expertise of Southern European countries, especially Greece and Italy, in seismic upgrading research and application, it has been for long a reference document for seismic assessment and rehabilitation of concrete structures (Among others, it has been a source document for ENV 1998-1-4, i.e. the pre-standard version of the part of Eurocode 8 on (Seismic) "Strengthening and repair of buildings"). In this more narrow field, CEB has also published in 1996 Bulletin 232: "Fastenings for seismic retrofitting – State-of-the-art Report on design and application", another source document for international standardisation in seismic retrofitting.

In this context, it is worth reminding how the relevant terminology has evolved within CEB and *fib*.

CEB Bulletin 163 has introduced the following definitions:

- *Assessment:* Evaluation of the condition and the mechanical characteristics of a structure.
- *Intervention:* Repair, substitution, strengthening.
- *Repair:* Restoring of the initial mechanical characteristics.
- *Substitution:* Demolition and rebuilding of (heavily) damaged elements.
- *Strengthening:* Instating of mechanical characteristics higher than the initial ones.
- *Redesign:* Design procedures concerning interventions.

Later on, the equally seismic-minded CEB Bulletin 232 has re-worded slightly some of the above definitions, and added new ones:

- *Rehabilitation:* reconstruction or renewal of a damaged building to provide the same level of function which the building had prior to the damage.
- *Remodelling:* (term not commonly used in Europe) reconstruction or renewal of any part of an existing building owing to change of usage or occupancy.

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- *Repair:* reconstruction or renewal of any part of a damaged or deteriorated building to provide the same level of strength and/or ductility which the building had prior to the damage.
- *Retrofitting:* concepts including strengthening, repairing and remodeling. (Structural intervention is used in some European countries instead of retrofitting).
- *Restoration:* rehabilitation of buildings in a certain area. (Term used only for historical structures or monuments, it means both repairing and strengthening in Europe).
- *Strengthening:* reconstruction or renewal of any part of an existing building to provide better structural capacity, i.e. higher strength and/or ductility, than the original building.

More recently CEB Bulletin 243, with its more general scope, (re)defined some of these terms:

- *Assessment:* The process of gathering and evaluating information about the form and current condition of a structure or its components, its service environment and general circumstances, whereby its adequacy for future service may be established against specified performance requirements, loadings, durability or other criteria.
- *Evaluation:* As assessment, but may be applied more specifically in respect of suitability against a particular criterion such as a specified loading. The term assessment is often used more commonly in connection with damaged or deteriorated structures.
- *Rehabilitation:* Bringing the structure back to its original or higher level of function, including durability and strength.
- *Remodelling:* Changes or alterations to a structure to meet revised function, performance requirements, usage or occupancy. The term is often employed where changes principally involve appearance, rather than alteration of the structural components.
- *Repair:* Action taken to reinstate to an acceptable level the current functionality of a structure or its components which are either defective, deteriorated, degraded or damaged in some way and without restriction upon the materials or methods employed. The action may not be intended to bring the structure or its components so treated back to its original level of functionality or durability. The work may sometimes be intended simply to reduce the rate of deterioration or degradation, without significantly enhancing the current level of functionality.
- *Retrofitting:* Action to modify the functionality or form of a structure or its components and to improve future performance. It relates particularly to the strengthening of structures against seismic loading as a means of minimizing damage during specified earthquake or to increase load carrying capacity.
- *Strengthening:* Action to increase the strength of a structure or its components, to improve structural stability of the construction.

Considering the already established tradition, as well as the evolution of the terminology within CEB and *fib*, it has been decided in this Bulletin to use mainly the term *Assessment*, over *Evaluation*, and the term *Retrofitting* its commonly used equivalent of *Rehabilitation.*

In addition to the Introduction, the present document comprises the following Chapters:

Chapter 2: Performance objectives and system considerations.

As the contents of the rest of the Bulletin are very quantitative, Chapter 2 adopts a qualitative approach, emphasizing understanding and judgement of the engineer. It comprises an overview of:

- The basic concepts of seismic resistance and response;
- Common reasons/sources of vulnerability of existing buildings (with many pictures of damage as examples);
- Available retrofit options, with their rationale, pros and cons;
- the current philosophy on performance objectives for the assessment or retrofit of existing buildings;
- Socioeconomic issues important for retrofitting (cost, liability, impact on occupancy and environment, historic preservation, etc.).

Chapter 3: Review of seismic assessment procedures

This chapter attempts a comprehensive overview of major and recent procedures for preliminary or detailed assessment of individual existing buildings (force- or displacement-

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based). All procedures are described in a stand-alone manner, independently of national/ regional codes or standards (i.e. in more generic terms). Detailed assessment procedures described in this Chapter are meant to apply (and should be applied) to retrofitted buildings too, to gauge the effect of retrofitting.

Chapter 4: Cyclic behavior of non-seismically detailed components

This chapter represents a behaviour (resistance, stiffness, deformation capacity, etc.) of RC members and connections under cyclic loading, with emphasis on effects of lack of detailing for earthquake resistance. Behaviour is described in qualitative terms, along with several alternative models and comparisons with experimental results (database studies, emphasis placed on scatter).

Chapter 5: Seismic Retrofit Techniques

The chapter attempts a comprehensive coverage of retrofit techniques (conventional or not), including:

- technology of application;
- experimental data on retrofitted members;
- engineering models and expressions for the retrofit design;
- recommendations for the conceptual and the detailed design of the retrofitting.

The chapter also covers retrofit strategies and instructions for selection of the retrofit technique.

Chapter 6: Probabilistic concepts and methods

The chapter overviews modelling of randomness in demand (seismic motion) and (strength or deformation) capacity. It also overviews and applies two "simple" but effective probabilistic assessment procedures, based on simulation of the response by nonlinear dynamic analysis: The 2000 SAC/FEMA 350 method by Cornell and co-workers (with application to a real 4-storey 3D RC building); and the "Efficient Fragility Analysis" method by Giannini and Pinto (with application to a planar 3-storey RC test frame).

Chapter 7: Case studies

Two real buildings from the Ceyhan and Dinar (TR) earthquakes are assessed before and after retrofiting (using the FEMA 273/356 procedure with linear dynamic, nonlinear static "pushover" and – for verification of pushover analysis – nonlinear time-history analysis, and the composite spectrum method of ATC-40). Then, two frames typical of old Italian design, are analysed in great detail with various procedures, from linear dynamic to nonlinear time-history analysis, with "pushover analysis" (FEMA 273/356, ATC-40, N2 method). This is more an in-depth comparative study of analysis procedures applied to existing buildings. Finally, a 4-storey dual building in 2D, typical of old Greek construction, is assessed with the FEMA 273 and 356 linear and nonlinear static procedures and the displacement-based approach of NZ Guidelines.

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2 Performance objectives and system considerations

Evaluation of an existing building and development of a retrofitting scheme benefit from understanding the dynamic response of a structural system and how that response is affected by global and local characteristics of the system. This chapter begins by examining some elementary concepts of structural system dynamic response to earthquake motions. These elementary concepts are illustrated in several examples of the performance of buildings in past earthquakes. A subsequent section describes generic retrofitting strategies that may be considered for rectifying inadequacies. The final sections describe performance levels and objectives as well as some social aspects of seismic retrofitting.

2.1 Concepts of structural system response to earthquakes

2.1.1 Earthquake input

The seismic hazard at a site generally refers to the potential for ground shaking, fault rupture, inundation due to earthquake-induced waves, and other ground failure phenomena such as soil liquefaction and landslide. Each of these hazards can pose a realistic threat to a structure depending on its construction site, and each should be considered as part of a seismic assessment and retrofit. However, in most instances the major seismic hazard is posed by ground shaking associated with energy release from a fault.

For a given earthquake occurrence, the ground motion at a site depends on a number of parameters, including fault rupture mechanism, path from rupture zone to site, and local site conditions. More generally, the hazard level at a site is not represented by a single earthquake occurrence but needs to take into consideration all the earthquake faults that may affect the site as well as the range of earthquakes that can occur on each fault. Seismic assessment at a site needs to recognize rare, large-magnitude earthquakes at near to far distance, as well as more frequent, medium-magnitude earthquakes at close distance.

Studies of the geological strata and historic records indicate that smaller-magnitude earthquakes generally occur more frequently than larger magnitude earthquakes. Consequently, seismic hazard or expected shaking level at a given site depends on the time horizon being considered. This subject is considered in discussions on seismic performance objectives in a later section of this chapter.

Methods for estimating shaking intensity for different hazard levels include both zonation map-based procedures and site-specific approaches. Map-based procedures, such as those in the Uniform Building Code (UBC 97), use maps of peak accelerations or spectral accelerations, along with spectral shapes to define acceleration design spectra for different sites and hazard levels. These procedures were developed for traditional force-based design approaches, and may be unsuitable for displacement-based designs, especially for longer vibration periods.

Past earthquakes have demonstrated the importance of recognizing that shaking intensity depends strongly on the site characteristics. Soft soil sites tend to filter out short periods and amplify longer periods, often resulting in higher spectral ordinates for vibration periods of interest for existing buildings. Procedures for determining site amplification effects have been developed and are widely used in seismic design applications such as those in the National Earthquake Hazard Reduction Program Provisions (BSSC 1998). The 1985 Mexico earthquake demonstrated a pronounced effect of filled land by developing almost harmonic, high-intensity, long-duration motion (Rosenblueth and Meli 1986).

2.1.2 Dynamic response of structures

In the context of this discussion, dynamic response refers primarily to acceleration and displacement response of structural systems subjected to earthquake ground motions. We are interested here in understanding conceptually how properties of a structure, such as stiffness, mass, strength, and damping, influence dynamic response and expected performance. Also, by understanding these parameters, the engineer can be enlightened about how to retrofit a structure to effect changes in response and improvements in performance.

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2.1.2.1 Single-degree-of-freedom oscillators

It is convenient as a starting point to idealize a complex three-dimensional structure into a simpler single-degree-of-freedom (SDOF) oscillator, as shown in Fig. 2-1. As idealized, the oscillator has stiffness, mass, and damping properties equivalent to those of the actual structure vibrating in a predominant mode. The vibration period of the oscillator is defined as $T = 2\pi\sqrt{M/K}$, where M is the mass (weight divided by gravity acceleration) and K is the stiffness. Assuming linear elastic response, the peak absolute acceleration and relative displacement response for an earthquake ground motion can be calculated as a function of the vibration period, T, and viscous damping. Fig. 2-2 a plots linear elastic response spectra for the Beverly Hills (279 deg) station, 1994 Northridge earthquake, for viscous damping ratios of 0.05, 0.10, and 0.20. Fig. 2-2 b plots the same response spectra, this time plotting pairs of acceleration and displacement points for a given period. This latter form of response spectrum has the advantage of showing how change in period or damping affects both the linear acceleration and displacement spectral demands.

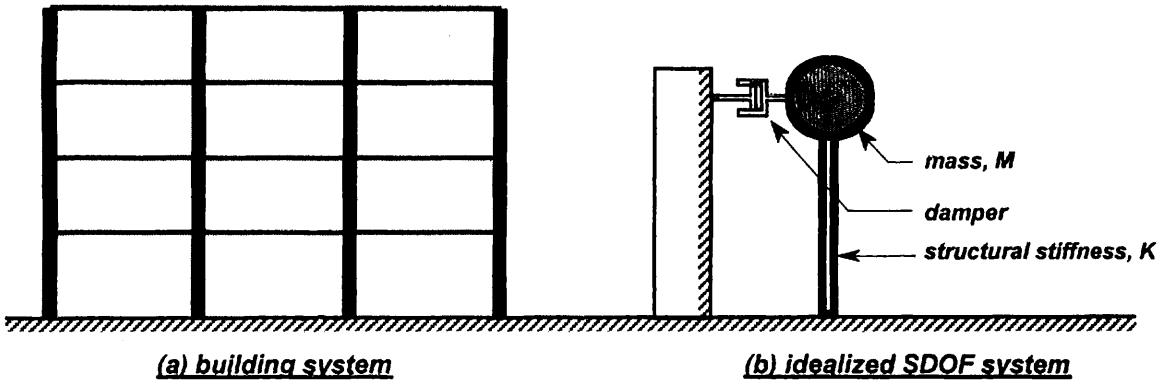


Fig. 2.1: Building system idealized as equivalent single-degree-of-freedom system

Response spectra such as those in Fig. 2-2 demonstrate important information regarding system performance. For example, for the motion represented in Fig. 2-2 a, a building with vibration period of 1 sec and effective damping ratio of 0.05 has spectral acceleration of nearly 1g. Most existing buildings have base shear strengths considerably less than the building weight; therefore, inelastic response can be anticipated. An outcome of inelastic response is that the building base shear is limited by the strength of the building under lateral loading rather than being determined by the linear spectral ordinates. Studies of inelastic response of oscillators show that, for "long" periods, the average peak displacement for inelastic response is approximately equal to that for elastic response. On the basis of this equal-displacement rule, the relative displacement indicated in Fig. 2-2 for the same building is nearly 0.25m. If the building in question is five storeys tall with effective height near 15m, the expected drift ratio is 0.25m/15m = 0.017. Laboratory studies and experience with earthquakes indicate that drift ratio this high is quite likely to correspond to significant inelastic response.

Earthquake ground motions commonly have jagged response spectra similar in nature to those shown in Fig. 2-2, though the details will change from one record to another. It is impossible with current technology to predict the jagged details of spectra for a future earthquake. For design purposes, an alternative approach is to use smoothed design spectra based on the average of spectra for ground motions that are representative of the hazard level at a site. Figure 2-3 shows smoothed response spectra derived using approaches published by the Structural Engineers Association of California (SEAOC 1999).

Smoothed design response spectra can be used to guide decisions about retrofitting. Procedures that increase damping will reduce both the spectral acceleration and spectral displacement demands (this is assuming that the retrofit measure does not modify the mass or stiffness appreciably). Procedures that stiffen a building (decrease the period) tend to reduce displacement demands and thereby can protect drift-sensitive building components. Although

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period reduction may result in an increase in the spectral acceleration, this indication is somewhat misleading; it should be remembered that the response is likely to be inelastic for most structures requiring rehabilitation, so internal forces are limited by the structural strength and not determined directly by the linear acceleration response spectrum ordinates.

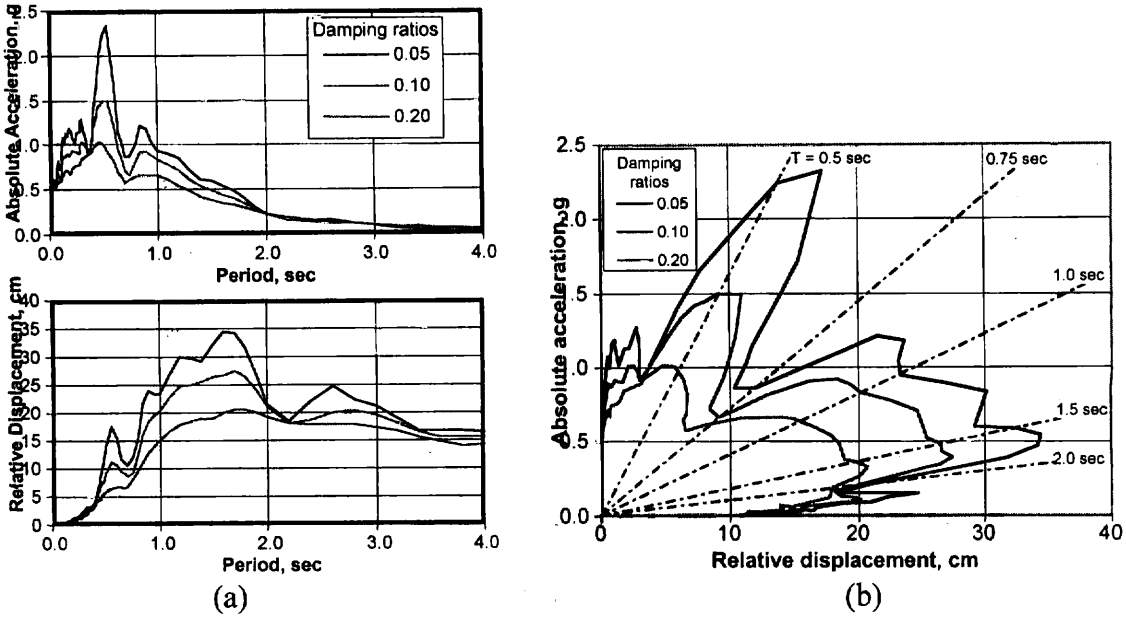


Fig. 2-2: Linear elastic response spectra for Beverly Hills, 279 deg, 1994 Northridge earthquake. Records from the PEER strong ground motion database. (a) Absolute acceleration and relative displacement spectra. (b) Response spectra plotted as acceleration - displacement pairs

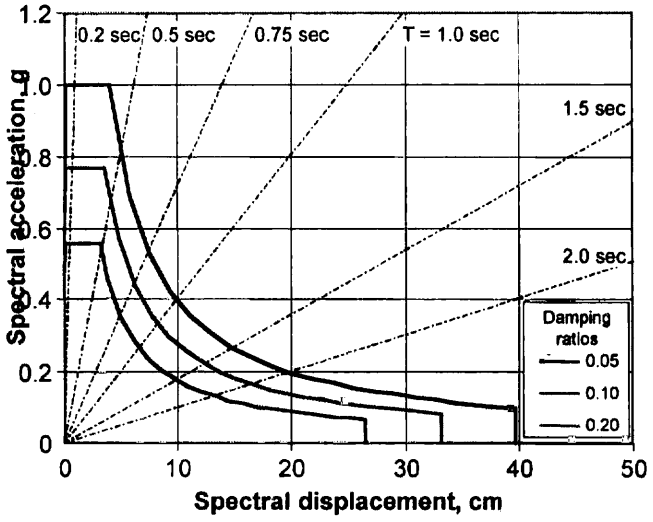


Fig. 2-3: Smoothed design response spectra

2.1.2.2 Multi-degree-of-freedom systems

Insights from study of SDOF oscillators can be applied to understand the behaviour of buildings, which being three-dimensional systems are always multi-degree-of-freedom (MDOF) systems. The majority of buildings that are retrofitted are relatively low-rise systems for which displacement response is dominated by the lowest translational vibration modes. For such buildings, estimates of displacement response for SDOF oscillators can be extended to estimate the overall building drift in each principal direction. A primary concern then is to identify how the drifts are distributed over height.

Figure 2-4 illustrates qualitatively the distribution of drift over height for four common systems. Where walls provide primary lateral force resistance, interstorey drifts tend to be

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relatively higher in the upper floors. A common exception for wall buildings is where the wall foundation is insufficient to prevent rocking, in which case the drifts are likely to be somewhat more uniformly distributed over height. Where frames provide primary lateral force resistance, interstorey drifts tend to be largest in the lower floors and decrease over height. Systems with soft or weak storeys tend to have concentrated drift in the weak or soft storey. The storeys with higher interstorey drifts will tend to have greater damage for most common reinforced concrete framing systems.

Response of buildings may involve significant torsional action in cases where centers of resistance and mass are significantly askew. A common situation for existing buildings is at the corner of a city block, where two adjacent sides of the building at the inside of the block have rigid walls for fire and sound proofing while street-side fa_ades are relatively open and flexible. The resulting torsional action increases displacement demands in the components located furthest from the center of resistance. Resulting damage to those components may weaken them and exacerbate the torsional response. Assessment of existing building performance always should consider three-dimensional aspects related to torsional response.

Figure 2-5 illustrates one example of this type of failure. In that example, there is a plan imbalance between seismic resistance and seismic demand, resulting in torsional response. As the structure translates and twists, the parts of the structure furthest from the center of resistance experience the largest drifts. Those outer portions of the structure may be inherently weaker, and as they are subjected to increased demands they may sustain exaggerated damage that further weakens them. The spiraling increase in deformations and damage may lead to premature failure of those parts of the building located furthest from the center of resistance.

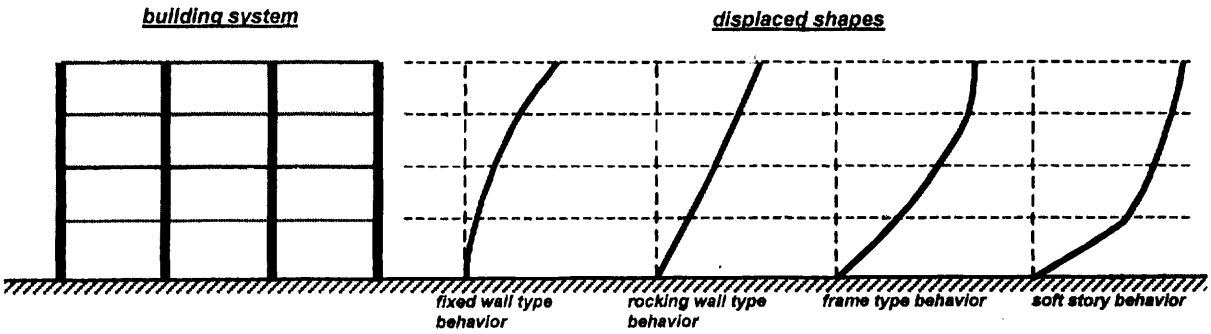


Fig. 2-4: Distribution of drift over height for typical lateral force resisting systems

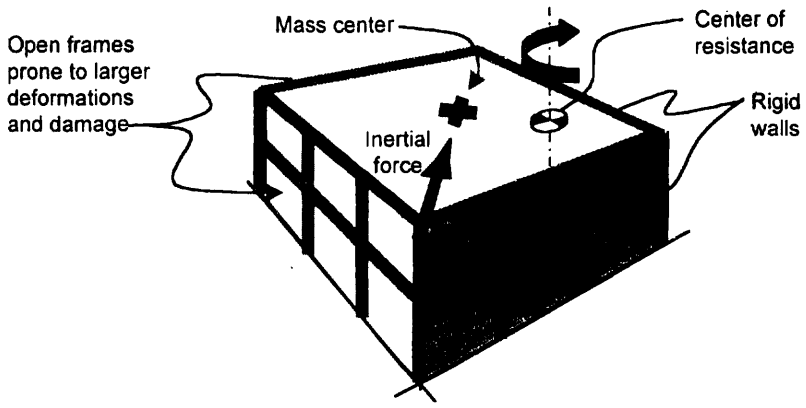


Fig. 2-5: Torsional action on a building where centers of mass and resistance do not match

2.1.3 Localized failure modes

A prevailing philosophy for conventional reinforced concrete construction is to avoid concentrations of deformations and damage in localized areas. The reason is that, if damage is concentrated in a localized portion of a structure, the deformation demands may prematurely exceed the deformation capacities, resulting in failures that can lead to overall poor system

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behaviour. As described in Section 2.2, the earthquake reconnaissance literature is replete with examples of poor performance where this philosophy was not implemented.

In moment-resisting frame buildings this philosophy emphasizes avoiding excessively weak storeys in which inelastic action concentrates in the columns of that storey. Figure 2-6 illustrates three idealized yielding mechanisms for frames. Case A is the least desirable in that all inelastic action occurs in a single weak storey; even if the columns are well detailed this action can result in poor performance because of the large deformations and P-delta actions that may be imposed on the columns during strong ground shaking. This behaviour tends to occur when the column strengths are less than the beam strengths. Case B ideally is the most desirable mechanism in that yielding is well distributed over height and no single component or storey absorbs the brunt of the earthquake demand. To achieve this condition generally requires an unrealistically high column-to-beam strength ratio, so it is uncommon. The yield mechanism of case C tends to predominate if the columns are marginally stronger than the beams; this case is normally acceptable for frame performance.

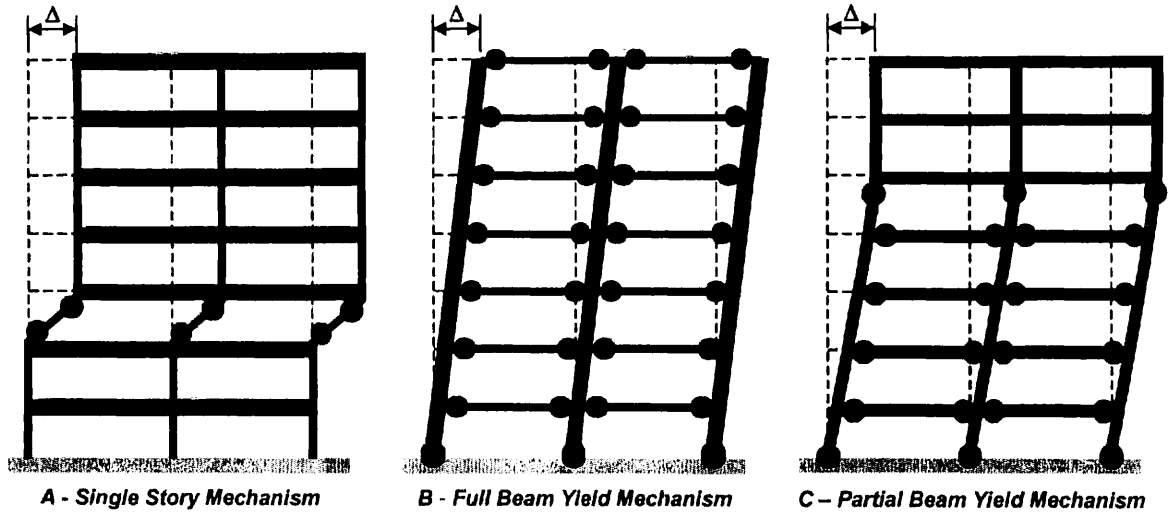


Fig. 2-6: Idealized yield mechanisms for moment-resisting frames

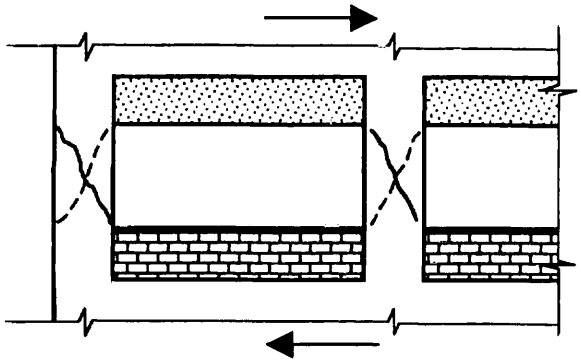


Fig. 2-7: Captive columns

Figure 2.7 illustrates a condition in which architectural concrete or masonry infill elements stiffen and strengthen the beams while restraining deformations of the columns effectively to the clear height between infills. This condition is relatively common in older construction. The shortened columns, commonly referred to as captive columns, may be prone to shear failure before or shortly after development of column flexural strengths, resulting in a relatively brittle weak storey mechanism.

Localized failure modes can occur in buildings with structural walls as well. Figure 2-8 illustrates two possible conditions. In case A, a wall extends from the foundation to a lower or intermediate level of the building. If the wall is very stiff relative to the frame, this can create a weak/soft storey just above the discontinuous wall, with resulting damage concentration in

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the frame. Case B illustrates a case where the wall is placed in every storey except the first storey, as may occur in mixed construction where open spaces are required in the lower floors but room/building partitions are required above. Moments developed in the wall under earthquake loading are transferred to isolated frame columns below the discontinuous wall. These columns may have inadequate axial load capacity, resulting in axial failures as illustrated in the figure.

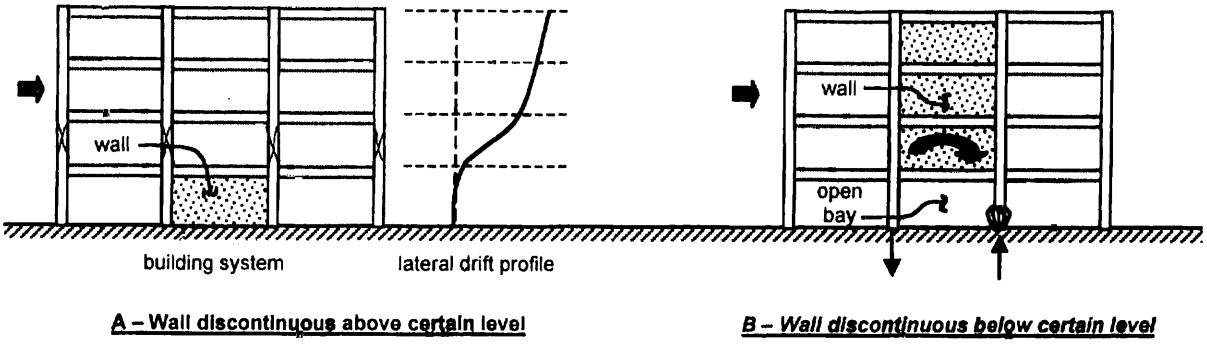


Fig. 2-8: Discontinuous walls

2.1.4 Load paths in structural systems

Dynamic response of structures results in inertial forces that must be equilibrated by a three-dimensional load path through the structural system. Weak and brittle elements along the load path can disrupt the flow of forces and result in failure of the structural system. A structural assessment should address the continuity and strength of the load path from points of force development down to the foundation.

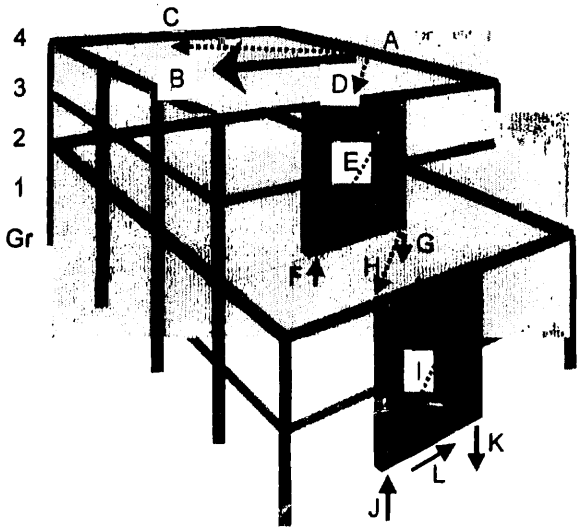


Fig. 2-9: Load path in idealized structure

Figure 2-9 illustrates a partial load path in an idealized structure. Inertial force generated by the fourth floor mass is shown by the arrow oriented from A to B. The diaphragm at that level carries the load to structural walls at points C and D. The floor diaphragm requires adequate collectors and struts to transfer the load to the walls, and also requires adequate shear and flexural strength to span from C to D. The wall at E carries load through shear and bending down to level 2 where the wall in this example is discontinuous. Columns supporting the wall at F and G are required to support axial forces associated with vertical load and overturning moment. The floor diaphragm at level 2 must transfer diaphragm forces introduced by wall E primarily along path H, and the diaphragm at that level needs to have adequate strength for the associated shear, bending, and force transfer. Wall I transfers load through shear and bending to the foundation level Gr, which in turn is required to be adequate for axial tension and compression actions at J and K, as well as shear forces at L.

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Brittle failure of any of the components of the load path can result in poor seismic performance of the overall system. Assessment of the structure should aim to recognize any such deficiencies, and the retrofit should aim to reduce them.

2.1.5 Ductility and deformability requirements

Discussion in Section 2.1.2.1 noted that earthquake demands might exceed reasonable strength capacities of existing structures. Whether inelastic response occurs depends both on the intensity of ground shaking and the properties of the structure. No attempt is made here to quantify the degree of inelastic response, but, on a qualitative level, it is well accepted that inelastic response routinely can be expected during moderate to strong ground shaking for most conventional structures. Furthermore, all other things being equal, the degree of inelastic response in the entire system will tend to increase as the structure strength decreases, and the degree in selected individual components will tend to increase as the response becomes more irregular owing to torsion, soft or weak storeys, and incomplete load paths.

The engineer also should recognize that not only the primary lateral-force-resisting system but also the entire building system must be capable of deforming under earthquake actions while maintaining required performance. In relation to the idealized structure in Fig. 2-9, it is well recognized that wall I must be detailed to enable it to undergo deformation/ductility demands imposed by the earthquake. Equally important from a structural performance and life safety perspective, the adjacent columns, which may be viewed as being only gravity-load-carrying elements, must be capable of deforming along with wall I without losing their capacity to support gravity loads. Likewise, the nonstructural components of the building need to be checked for adequate safety and functionality as they deform with the structure to which they are likely attached.

Subsequent chapters will describe in detail the inelastic deformation capabilities of reinforced concrete components typical of older existing construction. They will illustrate how failure in axial compression, shear, and bond often are relatively brittle in nature, especially with details common in older construction. They also will describe how ductility can be enhanced by reducing shear and axial actions, increasing transverse reinforcement and locating splices away from zones of repeatedly high stress. The same observations also will be apparent in Section 2.2, which describes experiences in the performance of buildings during past earthquakes.

2.2 Performance of vulnerable buildings in earthquakes

The general study of history presumably is driven not so much by a preoccupation with the past, as it is by a desire to learn from our past mistakes so that we might avoid them in the future. So it is with earthquake reconnaissance, which serves to establish a historic record of what worked well in past earthquakes, as well as what did not. Some of reconnaissance reports are listed in Appendix. More so than any laboratory or analytical study, the lessons from past earthquakes have resonated with practice professionals and led to rapid advancement of building regulations that provide greater and more reliable protection.

Observations of past earthquakes have led to the conclusion, contrary to prevalent building code emphasis, that the lateral load strength of the building is not an adequate single index to represent the safety of a building. The earthquake response of a building is influenced by characteristics of earthquake motion, structural configuration, dynamic properties, lateral strength, deformation capability of constituent members, foundation, soil-structure interaction, quality of workmanship, structure age and maintenance, and loading history (Moehle and Mahin 1991). Furthermore, the impact of each of these characteristics will vary depending on whether the acceptable performance is for continued use and function of the building, prevention of collapse, or any performance level or range between these extremes.

This section identifies examples of structural failure caused by poor planning of structural systems, poor detailing, or inadequate implementation or maintenance of critical details. The section begins by consideration of system aspects and concludes by consideration of component aspects. The division is somewhat arbitrary in some parts, but convenient for presentation and understanding.

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2.2.1 System aspects

Failure of a reinforced concrete building is normally initiated by the failure of vertical load carrying members at relatively small deformation. As described in Section 2.1.3, it is generally desired to avoid the concentration of structural deformation to limited localities or members, because RC members with conventional details do not have large deformation capacity. Furthermore, brittle modes of failure should be prevented in vertical load carrying members by the application of design requirements and reinforcement detailing.

2.2.1.1 Lack of strength and deformation capacity

Failure of a structure during a strong earthquake motion occurs when the deformation capacity is reached, normally by failure of vertical load carrying members, before or after formation of the plastic mechanism under lateral loading, or when the second order (P-delta) effect causes instability after a significant deformation. Numerous failures after earthquakes have demonstrated the collapse of buildings by reaching the strength and deformation capacity.

The ability of a structure to withstand given earthquake shaking depends on a complex interrelation between the ground motion, the building resistance in terms of stiffness and strength, and the building deformability. Some aspects of this interaction are described in Section 2.1. This complex relation commonly is simplified for design and assessment purposes into the dictum that a building with low deformation capacity requires high stiffness and strength while one with larger deformation capacity can perform well with lower stiffness and strength.

Recent performance-based seismic engineering requires, for some types of buildings, the prevention of damage from frequent earthquake motions in addition to the traditional goal of providing safety in rare events. The structural damage of a building with high building resistance (stiffness and strength) is likely to be smaller than that of a building with low resistance, regardless of the deformation capacity. Therefore, a certain minimum resistance is desired where enhanced performance is sought for frequent events.

2.2.1.2 Vertical irregularities

One of the more common risky conditions of a building has been noted where the stiffness and associated strength are abruptly reduced in a storey along the height (Figs. 2-6, 2-8, 2-9). Earthquake-induced deformations tend to concentrate at the flexible and/or weak storey. If such a soft storey consists of less-ductile columns, the damage often leads to collapse of the storey.

Photo 2-1 shows damage to the Imperial County Services Building after the 1979 Imperial Valley earthquake. The end walls in the building were discontinued in the first storey. Interior walls and the floor diaphragm connecting those walls to the end walls provided an adequate storey shear load path, but overturning moments under the end walls led to column failures. The failure was made more likely by detailing of the columns – closely spaced hoop reinforcement required by the code had been buried underground beneath the slab on grade, resulting in large demands above grade where details were inadequate.

Soft/weak first storeys are especially common in multi-storey residential buildings in urban areas, in which case the first storey often is used for open space, commercial facilities, or garages. Structural walls that separate residential units in levels above commonly may be discontinued in the ground storey to meet the change in use, resulting in a soft/weak storey there. Similar conditions are not uncommon in commercial buildings. The ground-storey columns during strong earthquake shaking must resist a large base shear, inevitably leading to large storey drift concentrated in that storey. If the ground-storey columns have not been well detailed, or if the axial forces are large, the columns may be unable to follow the large storey drift without failure. Furthermore, it should be noted that exterior columns may be subjected to large variation of axial forces induced by overturning moment of lateral forces acting on a building. This additional axial force further reduces the deformation capacity of the columns.

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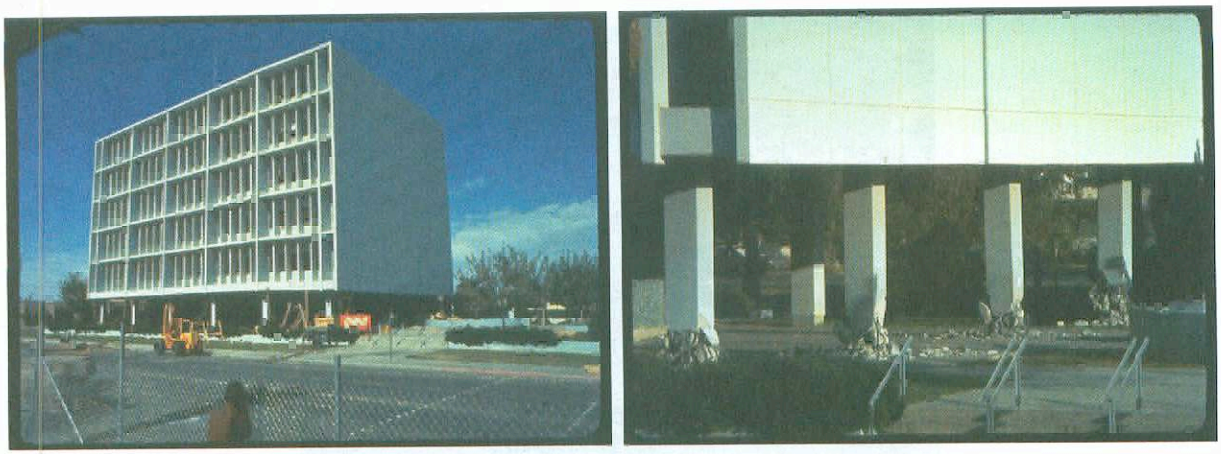


Photo 2-1: Imperial County Services Building damaged in the 1979 Imperial Valley earthquake. Source: National Information Service for Earthquake Engineering (NISEE), Univ. of California, Berkeley.



Photo 2- 2: Olive View Hospital, heavily damaged in the ground storey in 1971 San Fernando Earthquake, CA.

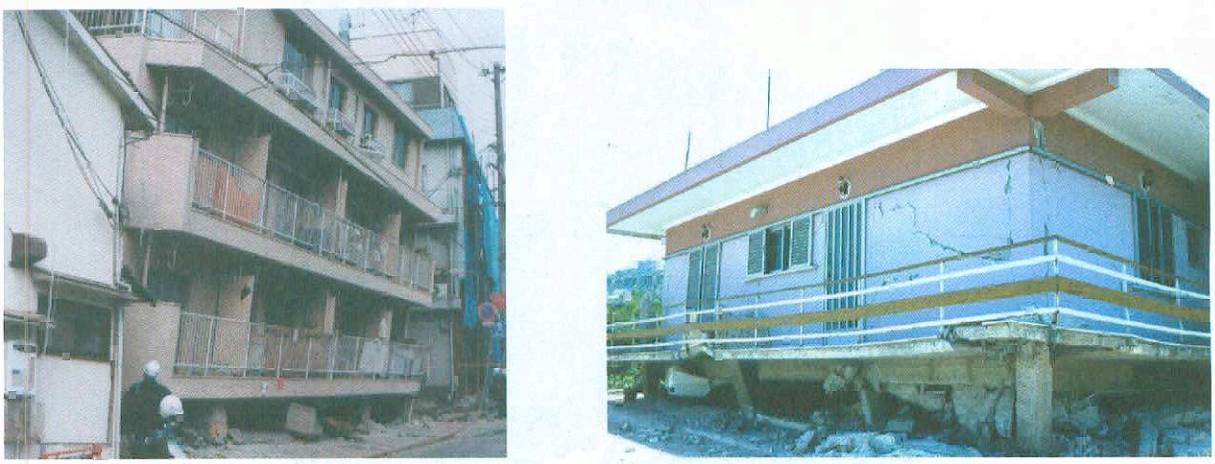


Photo 2-3: (Left) Rokko Building, Kobe, collapsed in the first storey in the 1995 Hyogo-ken Nanbu (Kobe) Earthquake, Japan. (Right) Two-storey building collapsed in the open ground storey in the 1995 earthquake in Aegion, Greece.

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Photo 2-4: Condominium building collapsed in the third storey in the 1995 Hyogo-ken Nanbu (Kobe) Earthquake.

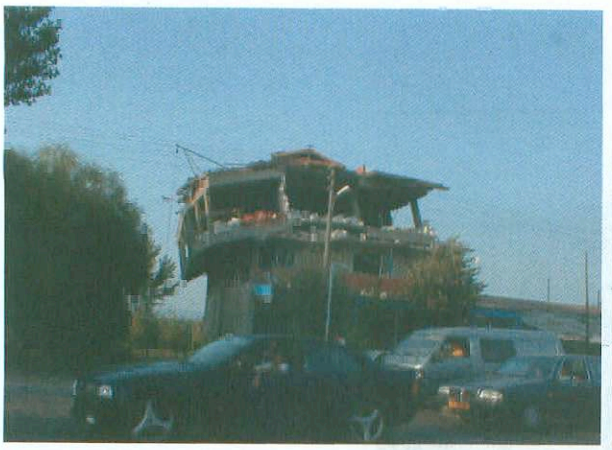




Photo 2-5: (Left) Collapse at third storey in 4-storey building in the 1999 Kocaeli earthquake in Turkey. (Right) Penthouse collapse in 5-storey building in the 1986 Kalamata earthquake in Greece.



Photo 2-6: First-storey failure of corner building due to torsion in the 1999 Chi Chi Taiwan earthquake. Note open front and wall at the back . Source: NISEE

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Photo 2-2 shows the Olive View Hospital after the 1971 San Fernando earthquake in California – discontinuous walls resulted in large lateral drifts in the ground storey. Other examples from the 1995 Kobe earthquake, Japan and the 1995 earthquake in Aegion, Greece, are shown in Photo 2-3.

Middle storey collapses (Photo 2-4) have been observed in relatively old construction, where the code requirement about shear reinforcement was not sufficient to prevent brittle shear failure. A relatively brittle failure of a column and accompanying loss of lateral and vertical load resistance redistributed forces to other columns and caused successive failure of columns in the storey. Similar failures were not observed in newer buildings, apparently because of improved detailing of columns to delay shear failure. Upper storey failures also have been observed in older steel-reinforced-concrete buildings in which structural steel sections were discontinued in an upper storey. The reduction on resistance apparently resulted in increased demands and subsequent member failures above the point of discontinuity.

Another form of vertical discontinuity occurs with setback buildings, in which the structural width is reduced toward the top of the building. Such configuration can tend to excite the second mode of vibration during an earthquake and cause large deformation just above the setback (Photo 2-5). Large diaphragm transfer forces can exist at the setback level, as lateral forces accumulated in upper levels are transferred through the floor diaphragm to adjacent frames below. An extreme example of setback building is where a small appendage, such as a penthouse, occurs at the upper levels of the building. In this case, it is possible that the base of the building will provide input to the appendage, which, if in resonance with the base, can be set into exaggerated amplitudes of motion.

2.2.1.3 Horizontal irregularities

The eccentricity between the centres of mass and resistance causes torsional vibration during an earthquake and results in larger damage in members away from the centre of resistance (that is, on the "flexible side"). This problem is especially common in buildings at the corner of a block, where common walls at the back of the building provide large resistance while the street sides provide less resistance (Fig. 2-5 and Photo 2-6). Another example occurs in office buildings in which an elevator hall surrounded by structural walls may be placed on one-side of the floor to leave large open office area in the remainder of the floor (Photo 2-7). In designing a retrofit for a torsionally sensitive structure, an objective should be to balance the resistance and mass and provide substantial torsional resistance for accidental torsional effects.

2.2.1.4 Inadequate diaphragms

Structural diaphragms are required to span between vertical elements of the lateral force-resisting system, and thus transfer forces in the horizontal plane (Fig. 2-9). Buildings having diaphragms that span large distances between vertical elements of the lateral force-resisting system may become overstressed in moment or shear, leading to inelastic behaviour in the diaphragm. Such inelastic behaviour normally is not considered in the design of a structural system and can lead to unintended system behaviour. Failures of precast diaphragms with cast-in-place topping slabs were observed in precast parking structures after the 1994 Northridge earthquake (Photo 2-8). The diaphragms were weaker than the vertical elements of the lateral force-resisting system, making them the weak link of the system. Contributing to the failures were the thin topping slabs compounded by the long spans between vertical elements of the system. Where vertical elements of the lateral force-resisting system are discontinuous, for example, in setback buildings, diaphragms also must serve to transmit forces from one vertical plane of the building to the other near the discontinuity.

2.2.1.5 Secondary (gravity) framing

A common practice in some regions is to designate some parts of the structural system as being part of the primary lateral force-resisting system, with other parts designated as secondary or gravity-only elements. In the extreme, only a few elements are designated as being lateral force-resisting, with the majority of the system designated as secondary. In many cases, these secondary systems have not been adequately proportioned and detailed to sustain the deformations that are imposed when the building sways during an earthquake. Several examples of failures of this type of system were observed in the 1994 Northridge earthquake.

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The failure of these elements can be associated with localized loss of gravity load-carrying capacity (Photo 2-9 left), which may in turn lead to a cascading or progressive collapse of the building (Photo 2-9 right). The ability of the framing system to support gravity loads in the event of a localized failure should be examined to determine consequences for the entire building system. The transfer must be achieved by structural walls, adjacent girders, and slabs at and above the failing component.

2.2.1.6 Interaction with nonstructural elements

Interaction with nonstructural components can pose a problem both for the system as a whole or for individual components. Aspects of the system as a whole are considered here.

From a systems perspective, important nonstructural components include infill walls of masonry or concrete and stairways. These elements can impart significant stiffness to the framing system where they are located, and because that additional stiffness was unintended, it may result in adverse system behaviour. One example is where fire walls are constructed along shared perimeter lines and open framing is used along the street side, resulting in eccentricity between stiffness and mass. Another example is where infills initially are very stiff, but their sometimes weak and brittle behaviour results in failure at a single storey, thereby creating a soft storey (Photo 2-10). Still another common example is where infills are used in upper floors of a building, leaving a weak and flexible first floor that is vulnerable to excessive drift and failure.

2.2.1.7 Previous modifications and damage

In examining an existing building, the estimated seismic resistance of the system should take into account any modifications to the structural system over time. In some cases, building occupants/owners may modify the structural system for functional reasons; such modifications may result in adverse seismic performance. Both addition and removal of structural/ nonstructural elements are commonly observed. Changes in occupancy can result in significant increases in dead and live loads which may overstress the system and reduce seismic resistance.

Deterioration due to aging and aggressive environmental conditions also can have an adverse effect on the seismic performance potential of a building. Included in this category is the effect of prior earthquake damage, which may have been overlooked or patched without adequate attention to structural restoration. Aspects to consider include existing cracks, excessive deflection, uneven settlement caused by foundation deformation, concrete deterioration, and reinforcement corrosion.

The existence of previous modifications, damage, or other deterioration can only be observed by field inspection of the building. Destructive or non-destructive testing may be advisable. A field inspection also will provide information on the quality of construction, especially the concrete quality, which should be suspect in many older buildings. Evaluation of an existing building solely by examination of the structural drawings generally is insufficient.

Photo 2-11 shows the collapse of a department store building in the 1948 Fukui Earthquake, Japan. The collapse is believed to be due in part to previous fire damages and soil settlement.

2.2.1.8 Pounding of adjacent buildings

Pounding between adjacent buildings can result in poor building performance. The condition is particularly adverse when the floor slabs of adjacent buildings do not line up horizontally, in which case the top slab of the shorter building can severely damage the columns of the adjacent building. Photo 2-12 shows one of several examples from the 1985 Mexico City earthquake.

2.2.1.9 Pancake failure

Pancake failures (Photo 2-13) has been observed in flat-slab and waffle-slab construction, as well as in frames. This type of failure should be avoided for life safety, because no space is left for survival of people between the floors above and below. The use of deep beams or spandrel beams can reserve some space between floors even after the collapse of a building.

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Photo 2-7: (Left) The damage occurred on the right side where the structural wall was discontinued in the first level on the right, while the structural walls were continued to the ground on the left (the 1978 Miyagi-ken Oki earthquake, Japan). (Right) Collapse of frames due to elevator shaft and staircase at one corner of the building plan by the 1999 Athens earthquake





Photo 2-8: (Left) Buckled topping slab on precast diaphragm during the 1994 Northridge Earthquake. The mild tension reinforcement apparently yielded due to diaphragm flexure, then buckled as the unbonded prestressing reinforcement compressed the elongated mild reinforcement. (Right) Fractured diaphragm reinforcement at connection with structural wall. Source: W.G. Corley





Photo 2-9: Consequence of column failures; (Left) Failure of interior "gravity" columns apparently led to (Right) progressive collapse of perimeter lateral force resisting system. 1994 Northridge earthquake. Source: NISEE

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Photo 2-10: Twelve identical buildings collapsed when first storey stiff brittle masonry walls failed thus causing the reinforced concrete frame columns to fail in shear during the Popayan, Colombia, Earthquake of March 31, 1983.

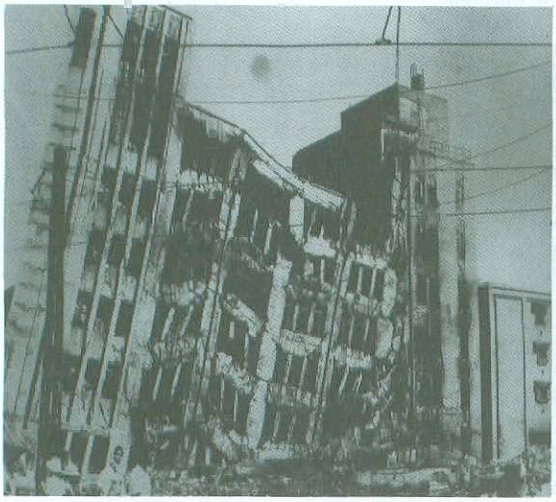


Photo 2-11: Daiwa department store building collapsed in the 1948 Fukui Earthquake. Building suffered fire before the earthquake. Soft soil condition and inadequate reinforcement details also may have contributed to the failure.



Photo 2-12: Pounding of adjacent buildings in the 1985 Mexico Earthquake



Photo 2-13: Failure of a school building in Mexico City during the 1985 Mexico earthquake. Many buildings failed in the pancake failure mode.

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Photo 2-14: (Left) Damage of an entrance door and non-structural concrete partition walls due to large deformation in the 1995 Hyogo-ken Nanbu (Kobe) Earthquake, Japan. (Right) Fallen heavy precast concrete curtain walls in a building, Kobe City, in the 1995 Hyogo-ken Nanbu (Kobe) Earthquake

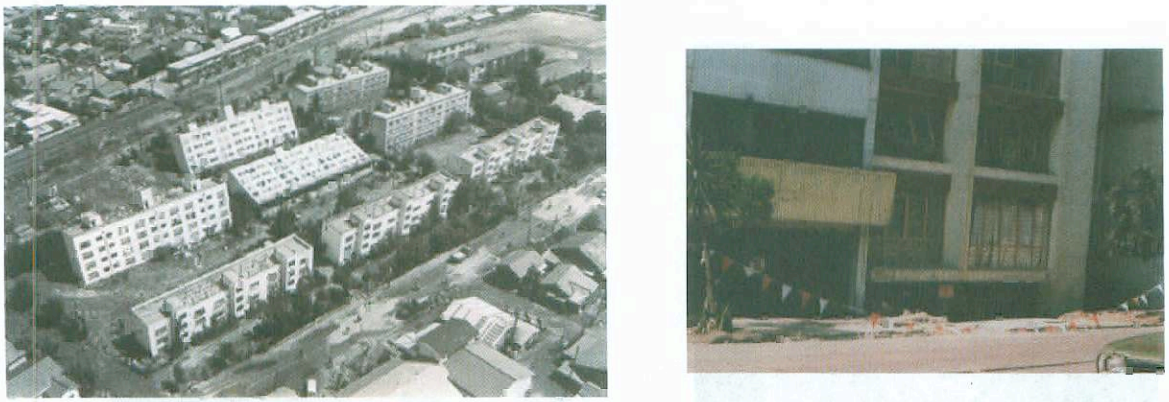


Photo 2-15: (Left) Overturned apartment buildings, Niigata City, due to liquefaction of sandy soil in 1964 Niigata Earthquake, Japan. (Right) Settlement of buildings due to liquefaction of soil in Mexico City during the 1985 Mexico Earthquake, Mexico.



Photo 2-16: Fault rupture at building during 1999 Chi-Chi, Taiwan earthquake resulting in foundation compression, forcing columns inward. Source: NISEE



Photo 2-17: Compaction under low-rise building in 1999 Chi-Chi earthquake. Lack of evidence of liquefaction and presence of large cobbles in subgrade suggests damage due to settlement. Source: NISEE

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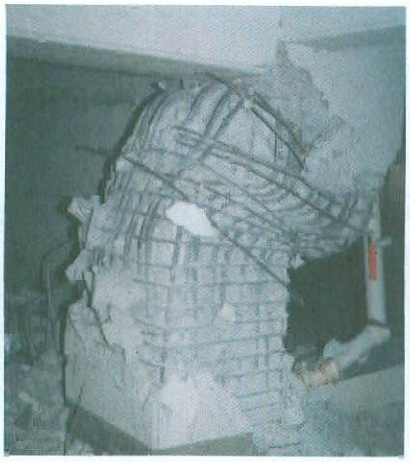


Photo. 2-18: Flexural compression failure of columns in 1995 Kobe earthquake. Column had peripheral ties at 100mm centres in Motoyama Royal Condominium Building.



Photo 2-19: Collapse due to excessive sway deformation during 1997 Iran-Qayen Earthquake. Failure was due to pull-out of column longitudinal reinforcement from beam column connection.





Photo 2-20: Thin widely-spaced plain bars were used as lateral reinforcement in columns failing in shear. (Left): in Baguio, the 1990 Luzon Earthquake, Philippines, (Right): in 1986 Kalamata earthquake, Greece.



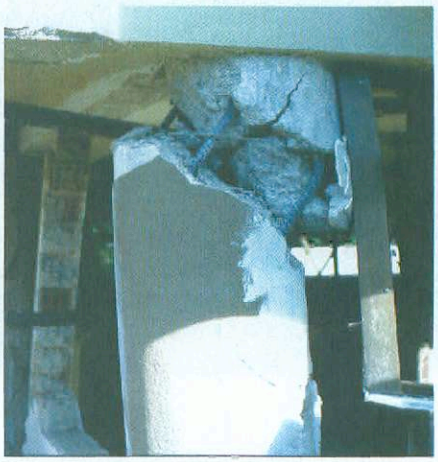


Photo 2-21: Column shear failures: (Left) in school building in Lazaro Cardenas, Mexico, during 1985 Mexico Earthquake. Spacing of the large-diameter ties was almost as wide as width of the column. Spalling of concrete was observed along longitudinal bars. (Right) in a building in 1986 Kalamata Earthquake, Greece; stirrups of plain bars were open and had no hooks or bents at the ends.

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2.2.1.10 Inadequate stiffness and resulting damage to nonstructural elements

Protection of building function in minor and frequent earthquakes may be an important performance objective. To maintain reasonable functionality, adequate performance of nonstructural or architectural elements in addition to structural elements must be assured. Experience demonstrates that residents of a building may be frightened by the damage of non-structural elements, such as partitions, windows, doors and mechanical facilities (Photo 2-14 left). In such cases, the building may not be occupied until the damaged nonstructural elements are repaired or replaced. The cost of repair work is often governed by replacement of the damaged nonstructural elements, rather than repair work on structural elements. Non-structural elements must be protected from damage to reduce financial burden on the owner.

Nonstructural elements must be also protected from damage, as damaged elements may create a falling hazard for people in, or escaping from, the building (Photo 2-14 right). Furthermore, fallen elements may block evacuation routes in a severely damaged building. Consideration also might be given to nonstructural elements such as doors, which may become locked in position after an earthquake, owing to residual deformations of the structure.

The damage level of structural and non-structural elements is known to be closely related to storey drift (inter-storey deformation) of a building. A number of damage investigations reported the effectiveness of structural walls in reducing the damage in structural members as well as nonstructural elements. Controlling inter-storey drift by the use of structural walls, or improving connection and bracing systems for nonstructural elements, can improve nonstructural element performance.

Stiff, weak and brittle brick walls, filled in a flexible moment-resisting frame, may fail at an early stage even during medium-intensity earthquakes. Providing some gap on both side of a column could reduce such damage, provided that infills are secured against overturning in the out-of-plane direction.

The response (acceleration or velocity) of a structure must be controlled to prevent heavy furniture and equipment from overturning on the floor and heavy equipment from falling from shelves. Alternatively, the contents of a building should be properly fastened to the structure.

2.2.1.11 Foundation inadequacies

The failure of foundation is caused by: a) liquefaction (Photo 2-15) and loss of bearing or tension capacity, b) liquefaction and spreading, c) fault rupture (Photo 2-16), d) compaction of soils (Photo 2-17), and e) differential settlement at cut and fill locations. Liquefaction of soil causes large deformation demand on pile foundations. Failure of piles was reported after the 1995 Hyogo-ken Nanbu (Kobe) Earthquake. The cost of repair work is extremely high, if failure occurs in the foundation. However, experience suggests that foundation failure normally does not pose a life-safety threat. Exceptions include overturning failures due to soil liquefaction or other generally rare behaviours, and failure of adjacent gravity-load-carrying elements due to excessive drift resulting from foundations that provide inadequate rocking restraint to walls or braced frames.

2.2.2 Component aspects

Failure types of members may be different for columns, beams, walls and beam-column joints. It is important to consider the consequence of member failure to the structural performance. In this part, the failure of members is described according to failure modes, placing emphasis on columns as column failure may lead to total collapse of a building.

2.2.2.1 Flexural failure of columns

The deformation capacity of a column in flexure is influenced by the level of axial force in the column and the amount of lateral reinforcement provided in the region of plastic deformation. Exterior columns, especially corner ones, are subjected to varying axial force due to the overturning moment of a structure. The axial force level in these columns may become extremely high in compression, leading to flexural compression failure followed potentially by the loss of gravity load carrying capacity. It is often difficult to distinguish shear compression failure and flexural compression failure, as both failures takes place near the column ends and involves concrete crushing (Photo 2-18).

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A building seldom falls due to the P – Δ effect after excessive deformation. A rare example is shown in Photo 2-19, in which yielding hinges formed at the ends of columns. The collapse of this type may occur in a building with slender columns.

2.2.2.2 Shear failure of columns

The most brittle mode of member failure is shear. Shear failure is caused by the lack of lateral reinforcement (size, spacing and strength of shear reinforcement). Photo 2-20 shows the shear failure caused by the use of small diameter shear reinforcement at wide spacing. The tensile stresses carried by the concrete before onset of significant shear cracking cannot be resisted by shear reinforcement once shear cracks open, leading to diagonal tension failure. The spacing of shear reinforcement should be sufficiently narrow so that at least one or two ties or stirrups should cross a shear crack (Photo 2-21). Closer spacing may be necessary in cases where shear forces are relatively high.

The ends of rectilinear lateral reinforcement should be anchored in the core concrete with a bend of at least 135°, or they should be welded together. The ends of lateral reinforcement was not properly bent in old construction, leading to shear failure of columns by pull-out of lateral reinforcement from the anchorage zone (Photo 2-22 left).

When a reinforcing bar is bent, permanent plastic deformation takes place at the bent and the region becomes less ductile. Fracture of reinforcement is observed especially when the steel does not develop high toughness and ductility before fracture (Photo 2-22 right).

Nonstructural elements commonly are neglected in modeling and analysis for design calculations, but are placed for the purpose of building function, for example, partition walls. When stiff and strong nonstructural elements are placed in contact with structural elements, the interaction can result in nonstructural and structural element damage. Photo 2-23 shows a case in which concrete nonstructural walls reduced the deformable length of columns, leading to shear failure of the captive column.

2.2.2.3 Bond splitting failure

The bond stress acting on deformed bars cause ring tension to the surrounding concrete. High flexural bond stresses may exist in members with steep moment gradients along their lengths. If the longitudinal reinforcement of a beam or column is not supported by closely spaced stirrups or ties, splitting cracks may develop along the longitudinal reinforcement, especially when the strength of concrete is low (Photo 2-24), when large diameter longitudinal bars with high strength are used, or when the concrete cover on the deformed bars is thin. These splitting cracks result in loss of bond stress, weakening the column and potentially resulting in various system failures that can be associated with weak storey systems (Section 2.1.1.2).

2.2.2.4 Splice failure of longitudinal reinforcement

Longitudinal reinforcement is spliced in various ways, including lap splices, mechanical splices, welded splices, and others. Splices are best located in a region where tensile stress is low. However, splices in older buildings commonly were located in regions of higher stress as a matter of convenience and because the implications for earthquake performance were inadequately understood. Examples of failure of welded splice and lap splices are shown in Photo 2-25.

2.2.2.5 Anchorage failure

The force in the longitudinal reinforcement in beams and columns must be anchored within a beam-column connection. Connections of older building construction may be without joint transverse reinforcement, in which case the column and beam reinforcement is anchored in essentially plain concrete. The resulting high anchorage bond stresses may cause bond splitting failure (Photo 2-26 Left). If the beam longitudinal reinforcement is not fully anchored in a beam-column joint, for example with a hook, the bar may pull out from the joint (Photo 2-26 Right). A common example of inadequate embedment occurs with beam bottom reinforcement that may be embedded a short distance into the beam-column joint.

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Photo 2-22: Shear failures in columns during the 1995 Hyogo-ken Nanbu earthquake. (Left) Opening of ties at a 90° bend. (Right) Fracture of lateral reinforcement at the corner in a column.

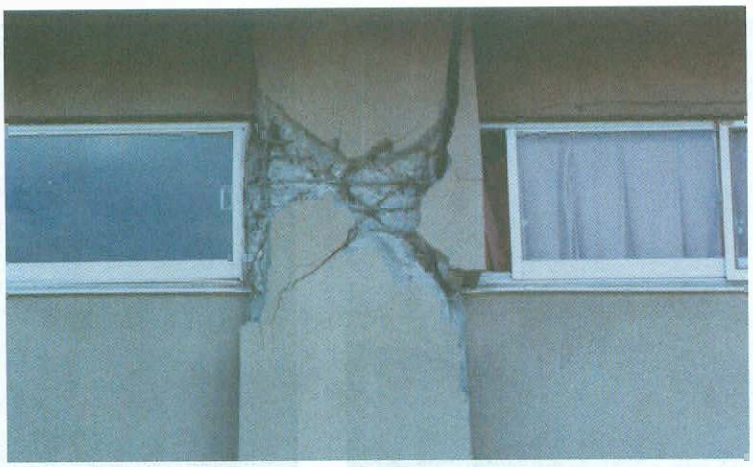


Photo 2-23: In Railway Technical High-school during the 1978 Miyagi-ken Oki Earthquake; non-structural but monolithically cast concrete walls restrained the deformation of a slender reinforced concrete column and caused the lateral deformation to concentrate in a short length



Photo 2-24: Bond splitting cracks were developed along the longitudinal reinforcement in columns of Namioka Hospital, Aomori, in the 1983 Mid-Japan Sea Earthquake; deformed bars were used with round-bar lateral reinforcement; concrete strength was low in this area due to poor coarse aggregates.

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Photo 2-25: (Left) Gas pressure welding splice was fractured during the 1995 Hyogo-ken Nanbu Earthquake, Japan. (Right) Failure at lap splice in a building in Baguio during the 1990 Luzon Earthquake.

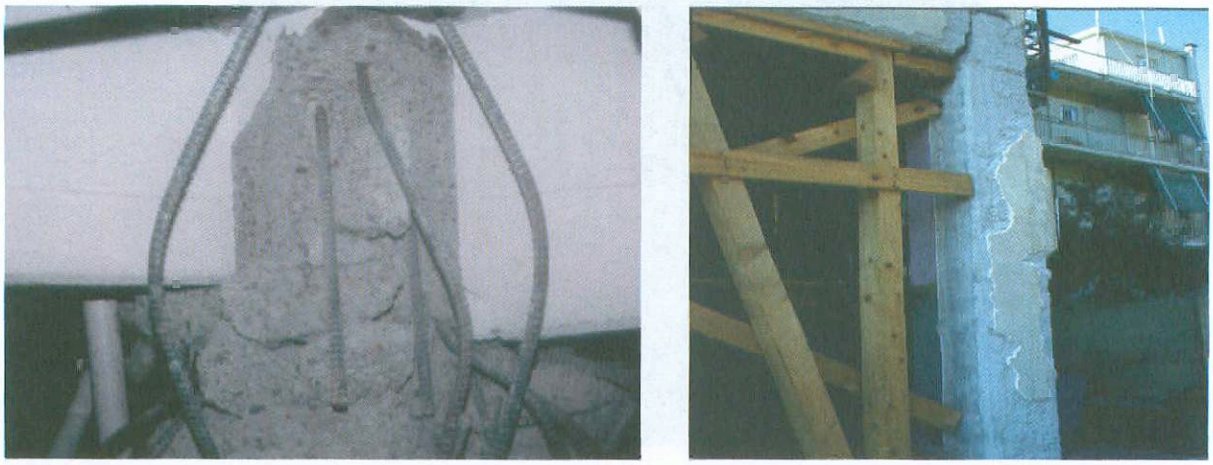


Photo 2-26: (Left) No lateral reinforcement was placed in a beam-column joint; the column longitudinal reinforcement failed in splitting of plain concrete in the anchorage region due to lack of confinement. (Right) Separation of beam from corner joint in the 1986 Kalamata earthquake, Greece, due to short straight anchorage of beam bars in the joint.



Photo 2-27: (Left) Beam-column joints of weak-beam strong-column building were heavily cracked in shear during the 1995 Hyogo-ken Nanbu, Japan earthquake. (Centre) Joint failures in older concrete frame in 1994 Northridge earthquake. Source: NISEE (Right) Diagonal failure of unreinforced corner column in 7-storey building in the 1986 Kalamata earthquake, Greece.

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2.2.2.6 Joint failure

When a moment-resisting frame is designed for weak-beam strong-column behaviour, the beam-column joint may be heavily stressed after beam yielding and diagonal cracking may be formed in the connection (Photo 2-27 left). Such shear cracking may reduce the stiffness of a building. Also wide flexural cracks will be developed at the beam end, partially attributable to the slip of beam reinforcement within the connection. Failure of interior beam-column joints normally is not found to result in building failure. In contrast, it is possible that failure of an exterior joint could result in collapse of building, if the degradation of the joint is sufficiently severe that the concrete spalling results in loss of column support (Photo 2-27 centre and right).

2.2.2.7 Failure of precast/prestressed elements

Experience with precast concrete buildings is relatively limited; that experience suggests that connections commonly are the weak link in the structural system. In the past, buildings composed of precast concrete elements were built in regions of high seismicity without due consideration to earthquake resistance. In the 1988 Spitak earthquake in Armenia, severe damage and collapse were caused by the use of construction practices not tested for earthquake resistance (Photo 2-28).

Some precast concrete parking garages collapsed during the 1994 Northridge earthquake. A paucity of lateral force-resisting elements resulted to overstressing of the cast-in-place topping slabs. In other cases, excessive lateral deformations of columns that had been designed for gravity loads only resulted in column failures that progressed to nearly complete collapse of the building. Unseating of girders also is possible; if large lateral forces or (especially) deformations develop; seat widths should be checked to ensure that the girders have adequate seating after the connections undergo rotations associated with the expected lateral deformations.

Tilt-up construction is a special form of precast construction whose performance is likely controlled by connection performance. The weak link commonly is the connection between the precast walls and the roof diaphragms, which may fail resulting in out-of-plane failure of the walls and loss of vertical support for the roof.



Photo 2-28: Examples of collapse of precast concrete buildings in 1988 Spitak earthquake in Armenia: (Left) Collapse of frame under construction in Leninakan. (Centre) Collapse of building in Spitak composed of wall panels with connections of plain cast-in-situ concrete and hollow-core slabs. (Right) Collapse of building in Leninakan, built of precast and in-situ infilled frame panels

2.2.2.8 Failure of piles

The lateral forces that develop during an earthquake in a building must be resisted by the foundation structure. High bending moments combined with axial forces acting at the top of a pile can cause crushing of concrete. Such failures were observed in the 1978 Miyagi-ken Oki

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earthquake and the 1995 Hyogo-ken Nanbu earthquakes in Japan (Photo 2-29). Such damage in the foundation structure is difficult to identify after an earthquake, unless apparent inclination of a building is detected as a result of permanent foundation deformation.

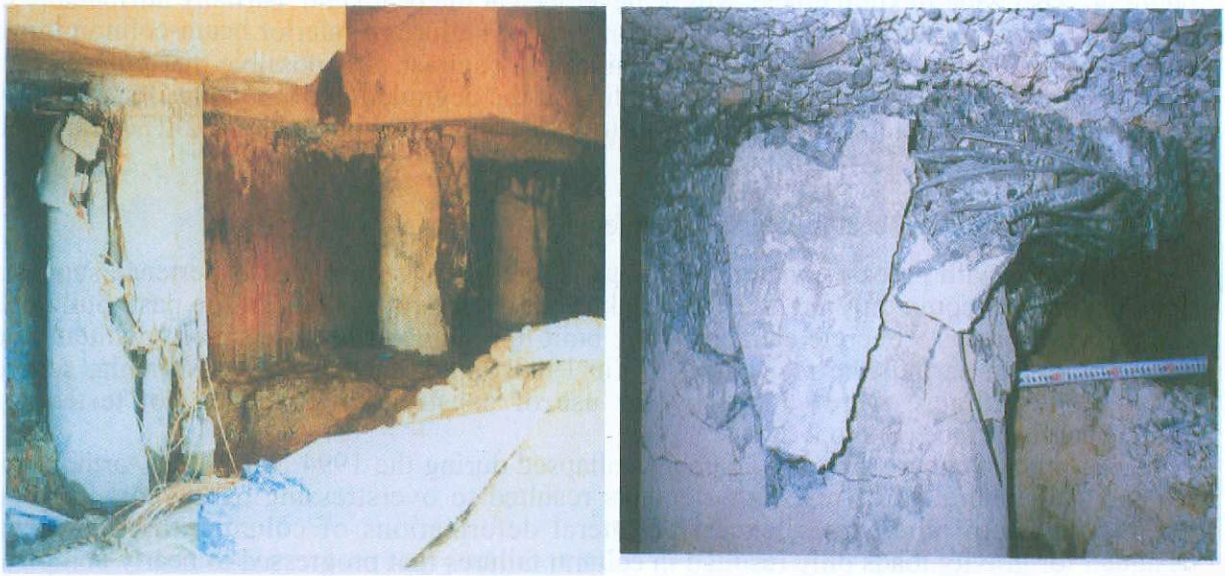


Photo 2-29: Failure of piles in the 1995 Kobe earthquake disaster. (Left) Failure of piles in reclaimed land. (Right) Failure of pile by lateral spreading of liquefied soil.

2.3 Retrofitting strategies

Earthquake-resistant design technology has progressed significantly in the last few decades. Damage investigations have demonstrated the poor performance of older buildings designed using out-dated technology. Although the photographic evidence of poor past performance is extensive (Section 2.2), damage statistics after major earthquakes indicate that the percentage of severely damaged buildings is relatively low (5 to 25 percent even in the heavily damaged area after the 1985 Mexico earthquake, the 1990 North Luzon, Philippines, earthquake, the 1992 Erzincan, Turkey, earthquake, and the 1995 Hyogo-ken Nanbu earthquake, Otani, 1999). Therefore, an efficient and reliable seismic assessment procedure should be employed to identify probably deficient buildings rather than aiming to upgrade all older buildings. A simple procedure is desirable to "screen out" the majority of safe buildings. Thereafter, a more detailed and sophisticated procedure may be utilized to further refine decisions about whether retrofitting is warranted on the basis of the safety and economic risk. Various procedures for seismic assessment are described in other chapters in this document.

In any design endeavour it is appropriate first to consider broad concepts or strategies to accomplish performance objectives, then to define systems that are suitable to the strategies, and finally to define details for those systems that seem most suitable. We follow that approach in the following text. Retrofit strategies are defined broadly as basic approaches that are targeted to improve seismic performance, such as increasing strength, increasing deformability, or reducing deformation demands. Retrofit systems are specific approaches that might be used such as jacketing or addition of new structural elements. Details of such retrofit systems are not covered here, though some details can be found in subsequent chapters.

The paragraphs that follow describe various retrofit strategies and systems in conveniently separated sections. In many cases of actual retrofit it will be effective to use two or more strategies or systems in combination to meet performance objectives.

2.3.1 Decreasing earthquake demands

Existing buildings designed by out-dated requirements are often significantly damaged by brittle failure of members before the members develop flexural yielding, or after developing flexural yielding but at a small deformation. A basic strategy for such structures is to limit the deformation or force demands on the brittle components by adding lateral stiffness or

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otherwise reducing the earthquake input. As shown in Section 2.1.2.1, increasing stiffness tends to reduce deformation demands. The strategy is illustrated in Fig. 2-10.

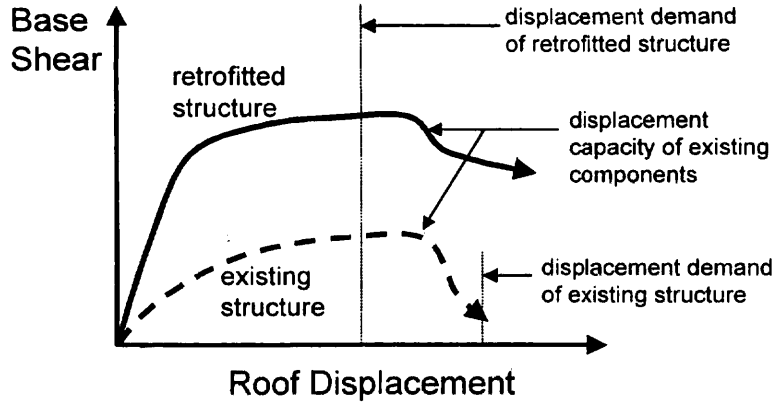


Fig. 2-10: Strategy to protect brittle structural components by increasing stiffness and reducing drift

2.3.1.1 Increasing lateral stiffness and strength

The lateral stiffness can be increased by placement of structural walls, steel bracing, or new moment frames. This method is effective when the structure has many existing structural members known to fail at a small lateral deflection, such as existing columns whose details are insufficient to prevent shear failure at a small lateral deflection. Column jacketing in this case can be very expensive and disruptive to building function. In contrast, it may be possible to introduce new lateral bracing elements around the perimeter of the building, minimizing intrusion in the building interior. This solution also can be effective to reduce the likelihood of pounding between adjacent buildings, or to improve functionality of buildings with drift-sensitive structural and nonstructural components.

In designing new bracing systems for an existing building, care must be taken to produce a final design that is structurally well balanced in plan and regular in height. In fact, one advantage of adding new bracing systems is that existing plan or vertical irregularities can be effectively reduced. In many buildings, architectural and functional requirements may restrict where new bracing elements are located. Good communications between building owner and engineer are important so that functional and structural requirements can both be achieved. If the rehabilitation work is carried out in several phases due to a financial reasons or continued use of the building, the placement of bracing systems must be carefully planned to minimize eccentricity in stiffness in plan during the retrofit work.

Some new bracing systems introduce considerable new weight into the building. The foundation must be examined to avoid overstress by the addition of dead loads associated with the rehabilitation work. Steel bracing sometimes is found preferable to structural walls because of their lower weight.

Structural stiffening by addition of bracing elements may attract larger seismic forces into a building during an earthquake, but it normally adds strength to the building to resist the additional forces. Load paths must be examined to ensure that seismic inertial forces developed within the retrofitted building can reach the new bracing elements and be carried by those elements to the foundation. Existing diaphragms may require strengthening, commonly by addition of new collectors. If existing columns are used as boundary elements of new walls or braces, the adequacy of existing reinforcement splices needs to be checked. The capacity of the foundation to resist shear and especially overturning actions introduced by the new elements also must be checked. Foundations may need to be strengthened by addition of new footings or piers. An alternative strategy is to add large grade beams to spread overturning actions to adjacent columns that can resist overturning actions through the gravity loads that they carry.

2.3.1.2 Seismic isolation

Seismic isolation involves the insertion of flexible or sliding bearings at one level of a building, typically though not always at the foundation level. The bearings reduce the stiffness of the system at that level, and thereby limit the seismic forces that can be introduced into the

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building. This approach can be highly effective in protecting brittle structural elements as well as acceleration-sensitive building contents. System displacements may increase associated with the increase in period, but the displacements are concentrated in the bearings and therefore do not adversely affect the drift-sensitive components of the building. Seismic isolation has become especially popular for buildings of historic significance not only because of the seismic protection provided by isolation but also because isolation retrofit can be implemented without significant disturbance of the historic architectural features.

For the isolation devices to be effective, the period of vibration must be shifted to the descending part of the acceleration spectrum. For this reason, light structures may not be suited for the rehabilitation with isolation devices. Also, widely available isolation devices cannot resist high tension forces. Therefore, isolation devices have limited application for slender structures where overturning actions can be high.

The installation of seismic isolators into an existing structure requires careful construction planning, as the existing structure needs to be supported while the isolation devices are inserted. Often the insertion of isolation devices at the foundation level will require extensive new construction work to provide an appropriate interface between the structure and the isolation devices. It is important that piping systems and elevator shafts should be able to follow the large deformations anticipated at the level of isolation devices.

2.3.1.3 Energy dissipation devices

Energy dissipation devices can be inserted in an existing structure to reduce dynamic response through increased damping. Different types of energy dissipating devices such as visco-elastic fluid dampers, visco-elastic solid dampers, hysteretic energy dissipating dampers and friction dampers have been used in seismic retrofitting. The devices usually are mounted on supplementary vertical braced frames, which results also in an increase in stiffness of the building system.

It should be noted that hysteretic energy dissipating devices become effective in dissipating energy after the devices are deformed beyond their yield deformation. Therefore, the yield deformation must be much smaller than the deformation capacity of structural members. If existing members fail in brittle modes at small deformation, such energy dissipation devices may be ineffective.

2.3.1.4 Mass reduction

Mass reduction can be an effective retrofitting technique for some existing buildings. By reducing effective mass, the vibration period of the structure is shortened, inertial forces are reduced, and displacement demands are reduced. Mass reduction can be achieved by removing heavy nonstructural elements (such as cladding, water tanks, heavy contents such as equipment and storage, and soil used as part of landscape architectural features.) In the extreme, mass reduction can involve the removal of one or more storeys in the existing building. Although mass reduction can be a very effective technique in some cases, in most cases it is of marginal value.

2.3.2 Increasing deformation capacity

As described in Section 2.3.1, performance of existing buildings is limited by the poor deformation capacity of relatively brittle older construction. Several techniques exist for decreasing deformation demands, such as adding additional stiffness or using seismic isolation or energy dissipating devices as described previously. Sometimes these techniques are unsuitable to the building function or they may be structurally ineffective. In such cases, various approaches for increasing deformation capacity may be an attractive. Figure 2-11 illustrates the strategy of increasing deformation capacity. In the case shown, deformation capacity is increased without appreciably modifying overall system stiffness or strength. Therefore, displacement demands of the earthquake are not changed. In some cases it will be appropriate both to increase deformation capacity and to modify demands using one of the procedures identified in Section 2.3.1.

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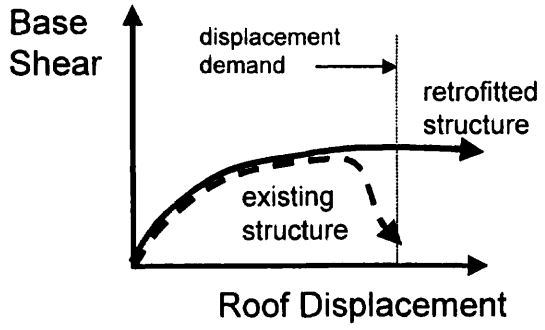


Figure 2.11 – Strategy of increasing deformation capacity

The deformation capacity of structural members can be increased by shifting brittle failure modes to ductile failure modes; e.g., a column may be jacketed by concrete or steel plates, or wrapped by FRP (fibre reinforced plastic) sheets to increase shear resistance, or a captive column may be separated from adjacent spandrel walls to lengthen deformable height of the column. Detailed approaches are described in other chapters of this report.

A structural member can fail in a variety of different modes, but it fails in the mode having lowest strength under seismic loading. Normally, an undesirable failure mode is avoided by strengthening the member against failure in that mode. An alternative approach is to lower the strength of some desired mode so that the desired mode becomes the weak link in the system. For example, a captive column having spandrel walls on both sides may be susceptible to brittle failure because the stiff captive column attracts large lateral force and the short length makes it prone to shear failure before flexural yield. By weakening the spandrel beams (severing reinforcement or cutting slots between the spandrel and the column) the yielding mode may be transformed to ductile spandrel or column flexural yielding.

2.3.3 System completion

Some existing buildings will be found to have most of the ingredients of an adequate lateral-force-resisting system, including vertical bracing elements, structural diaphragms, and vertical-load-bearing elements with adequate lateral deformation capacity, but they may be lacking some details to complete the load path. Seismic retrofit in these cases may be limited to providing the missing connectivity or force-transfer capacity. In other cases, these connectivity or force-transfer inadequacies will be coupled with other system needs such as described in Sections 2.3.1 and 2.3.2, in which case a more comprehensive retrofit may be required.

A common shortcoming of existing buildings is inadequate chords, collectors, and drags in existing diaphragms. New components can be added by jacketing existing floor beams or slabs or by adding flush-mounted steel straps or plates.

Precast concrete buildings often require modifications to improve seat bearing widths and otherwise tie the precast elements together.

Buildings with vertical irregularities such as discontinuous structural walls may require more significant modifications to complete the lateral-force-resisting system, such as jacketing columns supporting discontinuous walls, or constructing new walls to complete the discontinuous elements.

2.4 Performance levels and objectives

Recent advances in computational software and hardware, new information on earthquake hazards and structural behaviour, and new thinking about earthquake performance have encouraged development of performance-based seismic assessment and retrofitting methods. These methods aim to guide design decisions on the basis of anticipated performance, balancing short-term costs and disruption against the calculable benefits of retrofitting. While prescriptive procedures continue to play a role, performance-based approaches appear to have improved decision-making about seismic retrofitting, potentially resulting in more reliable and cost-effective retrofit decisions.

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Several important performance-based design guidelines and codes provide sources of reference for performance-based seismic retrofitting. Among these are various guidelines for seismic design and retrofitting of buildings (ATC, 1996; FEMA, 1997). In 2000 the Japan Building Standard Law was the first to introduce performance-based design in a legal document governing building design and construction (Otani, et al., 2002).

In the lexicon of performance-based earthquake engineering, the term *performance level* has come to mean the physical condition of the building, its ability to function and protect occupants and contents, and cost impacts of functionality loss and repair or replacement. The term *performance objective* expresses what performance levels are expected to be satisfied given the occurrence of a specific event or given the passage of a specified period of time. As an example, an owner may select a performance objective that the structure will be heavily damaged but not collapse during earthquake shaking intensity corresponding to a rare earthquake.

It has long been believed that it is not economically feasible to design a building to remain undamaged under intense ground motions. Recognizing this, engineers have long adopted performance objectives that accept the possibility of damage for severe levels of shaking. Since the 1960s, the Structural Engineers Association of California (SEAOC 1968) has adopted the structures designed in conformance with their recommendations "should, in general be able to:

1. Resist a minor level of earthquake ground motion without damage.
2. Resist a moderate level of earthquake ground motion without structural damage, but possibly experience some nonstructural damage.
3. Resist a major level of earthquake ground motion having an intensity equal to the strongest either experienced or forecast for the building site, without collapse, but possibly with some structural as well as nonstructural damage."

The concept of rehabilitation objectives, performance levels and seismic hazard levels have been well developed in the 1997 NEHRP Guidelines for Seismic Rehabilitation of Existing Buildings (BSSC, 1998), as described in more detail in Chapter 3 of this report. The following text introduces concepts and expectations regarding performance levels and performance objectives for seismic rehabilitation of buildings.

2.4.1 Performance levels

Building performance can be described by the extent of damage sustained by the building, which influences the safety of building occupants during and after the event; the cost and feasibility of restoring the building to pre-earthquake condition; the length of time the building is removed from service to effect repairs; and economic, architectural, and historic impacts on the larger community. Building performance is a combination of the performance of the structure, the nonstructural elements and systems, and the contents. Structural performance usually is the primary consideration for seismic rehabilitation because of the potentially extreme consequences of structural failure on occupant safety. An alternative approach is to select life safety performance level for the structural system, and a hazards-mitigated approach involving anchoring heavy items for the nonstructural components (FEMA, 1997). In special circumstances, such as critical facilities or facilities where financial solvency depends on continued operations, retrofitting of structural and nonstructural components for enhanced performance is considered.

A wide range of performance can be targeted in building retrofit design, ranging from *damage onset* to *collapse*. Intermediate performance levels that commonly are targeted in performance-based design are *operational*, *immediate occupancy*, *repairable*, and *life safe*. Each of these terms may be applied separately to the structural and nonstructural parts of the building, or the structural and nonstructural performance levels can be combined to express building performance levels.

Figure 2-12 identifies approximate positions of these performance levels for a building having relatively strong, brittle behaviour and for a building having less strong but ductile behaviour. The positions shown are intended to be illustrative only; actual points corresponding to performance of a specific building will depend on the details of that building. The following text describes each of the performance points as they relate to structural and nonstructural building parts.

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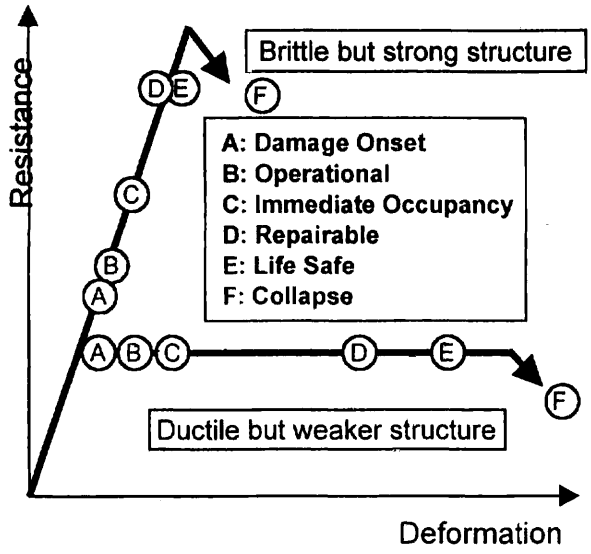


Fig. 2-12: Illustration of possible locations of performance levels for different structures

The six performance levels have definitions below:

Damage onset – This performance level corresponds to a state in which damage requiring repair initiates in either the structural or nonstructural systems. Some damage may be acceptable up to this state, but the damage should be such that it does not require repair for aesthetic, functional, or safety reasons. For ductile structures, this limit may occur sometime around or after the onset of yielding and be apparent in residual cracks. For more brittle structures, this damage state may occur at higher force levels and may be associated with damage to nonstructural components because of earthquake-induced accelerations.

Operational – A building meeting this performance level is expected to sustain little or no damage to the structural or nonstructural systems. The building is suitable for its original occupancy and use with minimal disruption required for repair or restoration. This performance level assumes that power, water, transportation, and other systems external to the building either are available to support the building function or else supplementary sources are provided in the building. This performance level also assumes that the degree of damage does not require intrusive repair that would indirectly disrupt the function of the facility. The ability to perform repair work "after working hours" may need to be considered in defining this performance level. For a ductile structure this performance point probably is beyond the point of overall structural yielding, and may be associated with structural or nonstructural repairs. For a stronger structure, this limit may well be controlled by nonstructural and contents performance.

Immediate Occupancy – Buildings at this performance level are expected to have minor to no damage to structural components with only minor damage to the nonstructural components and systems. While the building can be occupied soon after the earthquake, its function may be impaired until cleanup is completed and power and other systems are restored. It is expected that the degree of structural damage is fully repairable, and that the ability to sustain aftershocks or future earthquakes is not impaired. The risk to life safety during the earthquake should be minimal.

The preceding paragraph defines this performance level in terms of the building being sufficiently functional and comfortable for occupancy. An alternative definition is that the building is safe for occupancy, in that the degree of damage has not impaired its ability to resist aftershocks and future earthquakes. This latter definition envisions significantly more damage than does the earlier definition.

Repairable – Ductile buildings at this performance level have been damaged and will require repair to provide future safety, occupancy, and function. Furthermore, the repair is economically and physically feasible. While this performance level does not directly address life safety, in general it is expected that buildings performing at this level will provide substantial life safety. Because brittle structures do not sustain significant inelastic deformation before onset of serious and perhaps irreparable damage, this point may well lie

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below the strength capacity of the system.

Repairability is recognized to be an important limit state in determining the rehabilitation strategy. The cost of repairing or upgrading structural members after an infrequent earthquake may reach replacement cost of the building. A structural engineer should advise a building owner about the possible cost of the intervention and the loss associated with the disruption of the building operation during the repair work. It is worth noting that technically significant damage can be repaired using present state of construction technology. The technical repairability is not necessarily dependent on the damage level, but is rather governed by the needs of the building owner.

Life safe – Buildings at this performance level are expected to present low risk of life-threatening injury to building inhabitants. Therefore, the structural system should have suitable margin against collapse to accommodate the anticipated randomness and uncertainty inherent in earthquakes and buildings responses. Falling hazards associated with the nonstructural system should be addressed. Egress may or may not be considered part of this performance level. The level of damage to structural and nonstructural systems may be extensive, and repair costs may be prohibitively high, so that repair may not be feasible and the building may have to be demolished.

Seismic retrofit practice in the western United States sometimes adopts a hazards-reduced performance level for nonstructural components. Extensive damage may occur to nonstructural components, but large or heavy items that pose a falling or overturning hazard to a number of people are retrofitted to prevent life-threatening behaviour.

Collapse – It is uncommon to design buildings for the collapse limit state, but instead to design them for a state just preceding collapse. Thus, this performance level may be referred to as the *collapse prevention* level or the *structural stability* level. Buildings at this limit state are on the verge of collapse, and therefore should provide substantial but not complete life safety (falling hazards may pose some risk). Repair may or may not be feasible. This performance level can be subdivided into two sub-levels, one referring to the onset of local collapse and another referring to complete system collapse. Local collapse refers to the loss of one or more vertical-load-carrying elements resulting in localized collapse of a portion of a storey in a part of the building not exceeding one or a few bays. System collapse refers to the condition where simultaneous failure of several components, or progressive collapse emanating from one or a few failures, leads to collapse of a significant portion of the building. This could involve collapse of a single weak storey or collapse of the entire building. Buildings at or near the collapse limit state generally have sustained substantial structural damage, potentially including significant degradation in stiffness and strength lateral-force-resisting system, and large permanent lateral deformation to the structure. The structure is not safe for re-occupancy, as aftershock activity could cause collapse.

2.4.2 Seismic hazard levels

Seismic hazard levels should be selected considering the building function, its targeted performance, and its expected service life. Seismic hazard levels can be stated in terms of the probability that specified shaking levels will be exceeded over an established time. A common reference time is 50 years, which relates conveniently to both the service life of a conventional building structure as well as the adult life of a typical occupant or investor, and hence relates both to property risk and life risk.

Some performance objectives aimed at providing enhanced performance such as immediate occupancy will consider a seismic hazard defined for example by mean return period as short as 25 years, or 87% probability of exceedance in 50 years (SEAOC, 1999). Conventional building design for life safety has considered a 10% probability of exceedance level in 50 years, or a return period of approximately 475 years.

In regions of higher seismicity close to active faults, ground motion representations at the level of 10% probability of exceedance in 50 years may provide a reasonable approximation of the maximum ground motions expected at a site. For regions of low to moderate seismicity, the 10% in 50-year hazard level may provide a relatively low measure of maximum expected ground motions, and therefore provide inadequate protection against collapse. Retrofit design for the longer-return-period shaking hazard level should be considered in such locations.

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2.4.3 Rehabilitation objectives

The rehabilitation or retrofit objectives define the desired performance of a rehabilitated building when the building is subjected to seismic hazard of specified intensity. Such objectives are normally selected after considering the cost of the work and the loss estimate, in addition to the benefits derived from rehabilitation, such as the improved safety, the reduction of property loss and the continued use during and after the seismic event.

The rehabilitation objective of buildings may vary from country to country, because each country has different levels of seismic risk, risk tolerance, economic background, and technical development. Some jurisdictions impose minimum performance objectives that relate to the minimum code requirements for new buildings, whereas others allow retrofitting to lower performance objectives. In some jurisdictions, as an encouragement to begin retrofitting work, it is allowable to retrofit to any performance level provided the retrofit work does not degrade the expected future performance of the facility. Such approaches are useful to enable phased retrofitting that aims eventually to achieve a higher performance objective.

In addition, rehabilitation objectives may vary depending on the function or importance of a building, with higher performance objectives for essential and large-occupancy buildings. Examples of essential buildings include hospitals, emergency information or disaster management centers, law enforcement officials, and fire-fighting facilities. New technologies, such as base-isolation, energy-dissipating devices, and auto-adaptive media, may be used in such rehabilitation work to achieve enhanced performance.

2.5 Socio-economic considerations in seismic rehabilitation

2.5.1 Liability considerations

Depending on the legal provisions applying in a country regarding the responsibility and liability of designers and contractors, there may be a serious issue regarding the share of responsibility/liability against losses (property damage or casualties) in future earthquakes, between the designer or contractor of the original building and those employed by the owner for the retrofitting. Within the usual constraints of budget, architectural configuration, disturbance or interruption of occupancy, etc., the retrofit design cannot fully reverse a poor structural configuration of the original design and eliminate its adverse effects on performance in future strong earthquake. Similarly, serious flaws in the quality of the materials of the original building and/or in the amount or detailing of its reinforcement may remain undetected, especially as the cost and time needed for a complete survey and documentation of the as-built structure may be prohibitive. As a result the designer and the contractor of the retrofit work may feel that are exposed to more liability against future possible losses than is warranted. This may be more so for relatively old buildings, the original designer and contractor of which may not be known, or may not be available anymore for the purposes of sharing liability. Such considerations are disincentives for engineers to undertake rehabilitation work, and if they do, they may be led to an unwarranted increase in retrofit costs (owing to more extensive survey of the as built structure or to more conservative retrofit decisions) in order to reduce their own liability. A fair and efficient legal/regulatory system for the liability of designers and contractors of retrofitting projects seems to be a pre-requisite for the successful implementation of any targeted seismic rehabilitation program at the local, regional or national level.

An additional liability issue emanates from the use of performance-based design. As noted in FEMA 273 (FEMA, 1997), the concepts and terminology of performance-based earthquake engineering are relatively new to engineers and building stakeholders. All parties should be aware that performance objectives are targets, and not guarantees. Incomplete knowledge regarding earthquakes, existing conditions and seismic analysis and design procedures makes guarantees regarding performance impossible and irresponsible. The building owners and the engineer should work together to come to a complete understanding of the nature of performance-based earthquake engineering and the fact that there is no guarantee implied regarding future performance.

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2.5.2 Operational impact and continued use during rehabilitation

Rehabilitation work causes noise, vibration, dust and other types of pollution and disturbance. The extent of disruption during seismic retrofitting will vary depending on the nature of the work. Retrofit work involving interior work, such as diaphragm strengthening and column jacketing, is often sufficiently intrusive that building occupants need to be evacuated during the work. Work around the exterior of a building, such as addition of infill walls or seismic bracing, sometimes can be accomplished while the building is occupied. Some special retrofitting techniques have been devised that allow work to be done after hours with minimal disruption to the building fabric.

Retrofitting concrete columns normally requires removal of mortar and other finishing materials (e.g. tiles) from the concrete surface to effectively confine the concrete. In many occasions, additional preparation work lengthens the duration of work and increases cost. Therefore, the development of retrofit work without removing existing finishing materials is highly desired.

Obviously, the retrofitting work should be planned with the building owners/tenants to ensure that cost aspects associated with disruption are included in the overall decision.

2.5.3 Construction cost

Seismic assessment and rehabilitation commonly involve the removal of existing finishes to expose structural materials underneath. Such action invariably results in costs beyond the structural retrofit costs that might initially be envisioned. In addition to replacement of removed finishes and other materials, retrofit work may trigger other work such as upgraded electrical and fire-suppression systems and handicapped access. Some displaced occupants may opt to abandon leases, resulting in additional costs to building owners. Still other displaced occupants require interim facilities that may result in some costs to building owner. Because retrofit work may result in a general improvement in the quality of a facility, it may also lead to increased rents; the impacts on lower-income groups need to be considered in planning retrofit programs.

2.5.4 Considerations for historic buildings

Some jurisdictions have special requirements for historic buildings. In such cases, existing laws may require that the seismic retrofitting take into consideration the historic fabric of the building. Special procedures may be required to retain the essential architectural features of the facility. These may include not only special restrictions on retrofitting systems so that they leave the architecture intact but also special restrictions on performance objectives so that the historic fabric is preserved during future earthquakes. Seismic rehabilitation may attempt to hide the new elements behind the existing architecture, may reconstruct or replicate existing architectural elements to simulate the historic architecture, or may instead expose the new structural elements as obvious additions to the original construction. The latter approach sometimes is preferable especially when executed so that the additions can be modified or "undone" in the future without disturbing the historic elements.

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3 Review of seismic assessment procedures

3.1 Introduction

A detailed seismic assessment (or evaluation) of an individual building not only determines the need for seismic retrofitting or not, but identifies also the particular weaknesses and deficiencies to be corrected through retrofitting. For this reason recent years have seen a worldwide shift from rapid screening and empirical evaluation methods, to fundamental assessment procedures based on a direct or indirect comparison of the inelastic deformation demands to the corresponding deformation capacities.

During the past two decades considerable work has been done in the direction of developing seismic assessment methodologies, usually under the auspices of national or international organizations. In Europe a UNDP/UNIDO project produced in the early-to-mid 80's state-of-the-art reports (UNIDO, 1983), which, although focusing on post-earthquake assessment and repair, cover also pre-earthquake evaluation and strengthening. In Japan the main evaluation tool has been – and still is – a detailed multilevel semi-empirical procedure, published in 1977 by the Japan Building Disaster Prevention Association and revised in 1990. In the U.S.A. the Applied Technology Council (ATC) has taken the lead in the late '70s-early '80s, first including Sections 13 and 14, "Systematic abatement of seismic hazards in existing buildings" and "Guidelines for repair and strengthening of existing buildings", in its ATC (1978) tentative provisions for seismic regulations and continuing with the ATC (1987) methods for evaluating existing buildings. In the mid-'80s and within its role as the lead agency for the NEHRP project, the Federal Emergency Management Agency (FEMA) developed an action plan for reducing earthquake hazards of existing buildings (FEMA Report 90). The part of the plan which materialized first includes the ATC (1988) Handbook on rapid screening, and the NEHRP Handbooks for seismic evaluation and rehabilitation of existing buildings (BSSC 1992a and b).

The assessment methodologies proposed in these older U.S. reports are of the multi-stage type, with a first screening on the basis of checklists and subsequent evaluations of individual buildings at two levels of increasing sophistication. As in the UNDP/UNIDO Balkan studies, these evaluations are of the capacity-demand-comparison type, with both demand and capacity basically determined in terms of forces according to code provisions for the seismic design of new buildings. This is also the case for the ENV version of the part of Eurocode 8 on "Strengthening and repair of buildings" (CEN, 1996), which is essentially the first official standard internationally on seismic assessment and redesign.

New approaches have been proposed recently for the seismic assessment of existing RC buildings, which are partly or fully displacement-based. The rationale behind displacement-based approaches is well known: The earthquake does not represent for the structure a set of given lateral forces to be resisted, as considered in forced-based seismic design or assessment, but a demand for accommodation of a given energy input or of given imposed dynamic (ground) displacements. Therefore displacements, rather than forces, represent a much more rational basis for the seismic design or assessment of structures. After all, structures collapse not due to the earthquake lateral loads per se, but due to gravity loads, acting through the lateral displacements caused by the earthquake (P-Δ effects). For these reasons displacement-based seismic design has been proposed in the early '90s by Moehle (1992) and Priestley (1993) as a more rational alternative to the current forced-based design approach.

For new structures, procedures for direct proportioning of RC members on the basis of given deformation demands are not fully developed and accepted yet, hence in displacement-based design the problem of member proportioning is still reduced in most procedures proposed so far to conventional force-based proportioning. In assessment, though, which is an analysis rather than a synthesis problem, deformation capacities of the members and of the system can be easily computed for given dimensions, reinforcement and material properties. Therefore, assessment of existing structures provides a better ground than the design of new ones for the application of deformation- and displacement-based concepts. As a logical extension, a strengthening intervention is easier to design if it is considered as a means to reduce the global seismic displacement demands on the existing members, to levels below the corresponding deformation capacities. In other words, the detailing of old members does not need to be upgraded to the level required by modern standards for new members on ductility

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grounds, provided that the demands imposed on them are not beyond their ultimate deformation capacity and do no impair their resistance against gravity loads.

Recent assessment and strengthening guidelines which are clearly and explicitly displacement-based, are:

- The draft guidelines of the New Zealand National Society for Earthquake Engineering, first drafted in 1996 for the NZ Building Industry Authority and being under development until the time of this writing (June 2002);
- The "1997 NEHRP Guidelines for the Seismic Rehabilitation of Buildings" developed in the US by the Applied Technology Council for the Building Seismic Safety Council and FEMA (ATC, 1997a, b) - a parallel effort within ATC (ATC, 1996) has resulted in a document with a more limited scope;
- The draft EN of Part 3 of Eurocode 8 on "Strengthening and repair of buildings" (CEN, 2002).

The 1997 NEHRP Guidelines – widely known as FEMA 273/274 – have evolved into an ASCE "Prestandard for Seismic Rehabilitation" (ASCE, 2000). They are also been supplemented with an "ASCE draft Standard for Seismic Evaluation of Existing Buildings" (ASCE, 2001), which is currently evolving from a FEMA "1998 Handbook for Seismic Evaluation" (ASCE, 1998).

This part of the report contains a (critical) review of the main regulatory-type documents for seismic assessment or evaluation of existing RC building structures, namely of: a) Eurocode 8 (ENV1998-1-4:1996); b) the Japan BDPA 1977/1990 Guidelines; c) the New Zealand 1996 and 2002 draft Guidelines; and d) the most recent US documents, namely the 1997 NHERP Guidelines for rehabilitation of existing buildings (FEMA 273/274), as these have developed into an ASCE pre-standard (FEMA 356), along with the 1998 Handbook for evaluation of existing buildings (FEMA 310), as this is developing into an ASCE draft-Standard. All procedures are presented with a few comments, explanations, or even extensions and in enough detail to allow the reader to apply them to ordinary and simple RC buildings, without having to resort to the original sources.

The presentation is organised around the distinction between the traditional force-based approaches and the most recent but currently prevailing displacement-based ones.

For reasons of uniformity the same units (SI) and notation (that of the ISO/Eurocode system) are used in the presentation of the various approaches, which may be different from those used in the original documents. For the same reason, the well known rules of the CEB/FIP Model Code 1990 and of Eurocode 2 are used as the reference for member flexural and shear resistances, including the associated (material) partial safety factors.

3.2 Force-based procedures for seismic assessment

3.2.1 Conventional assessment procedures and their limitations: the case of the ENV-Eurocode 8, Part 1-4

In conventional procedures for the detailed seismic assessment of individual existing RC buildings, evaluation is done at the member level in the form of a capacity-demand-comparison, with demand and capacity expressed in terms of forces (member seismic internal forces for the demand and member resistances for the capacity) and determined according to code provisions for the seismic design of new buildings ("Force-based" approach). This is also the case of the ENV version of the part of Eurocode 8 on "Strengthening and Repair of Buildings" (CEN, 1996). To distinguish it from the EN version currently under development (draft prEN1998-3), the ENV version is termed in the following as ENV1998-1-4:1996.

According to ENV1998-1-4:1996, the assessment is based on satisfaction or not of the provisions of Part 1-3, Section 2 of Eurocode 8 (ENV1998-1-3:1994) for the design of new RC buildings according to one of three different Ductility Classes (DCs). For buildings of DC High (H) or Medium (M), ENV1998-1-3 requires capacity design of columns and walls in bending (at beam-column joints for the columns, above the base region for walls) and in shear. For DC H, capacity design in shear is required for beams as well. Detailing and minimum or maximum reinforcement provisions within regions of (potential) plastic hinges become less strict, if one selects DC M instead of H, or DC L (Low) instead of M, with the detailing requirements on DC L structures being only slightly more demanding than those

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applying to structures designed to Eurocode 2 alone (i.e. without any requirement for earthquake resistance). It is noteworthy that the ENV version of Eurocode 8 does not have a distinction between "primary" and "secondary" members, that might exempt those members characterised as "secondary" from full satisfaction of the requirements of the corresponding Ductility Class for detailing, capacity design, etc. The behaviour factor q (i.e. the European counterpart of the US force-reduction factor R) in DC H structures is double that of DC L ones, or 4/3 of that applying to DC M. For DC H, in heightwise-regular frames and coupled-wall structures the q-factor is equal to 5.0, in (uncoupled) wall systems it is equal to 4.0 and to 4.5 in dual systems with (uncoupled) walls resisting between 50% and 65% of the base shear. In heightwise-irregular structures these q-factor values are reduced by 20%.

According to the ENV1998-1-4:1996 approach all members are first examined for satisfaction of the detailing and minimum/maximum reinforcement requirements of the three alternative DCs. If they satisfy those applying to one of the two upper DCs, they are checked for fulfillment of the corresponding capacity design rules. After the structure is classified in one of the three DCs, the value of the q-factor for which it qualifies is determined as in a new structure, i.e. according to its structural system and regularity. If not even the requirements of DC L are met, the structure may be considered as a Eurocode 2 one and enjoy a (not clearly specified in ENV1998-1) q-factor value of 1.0.

With the value of the behaviour factor q known, the design spectrum can be entered and linear elastic analysis, equivalent static or (modal) dynamic, may be employed to determine the design internal forces, E_d, of members (including any P-Δ effects and the necessary capacity-design modifications). Design values of member resistances, R_d, are also determined, according to the relevant rules of Eurocode 2, as modified by Eurocode 8 (ENV1998-1-3:1994). If the criterion $E_d \leq R_d$ is not satisfied, strengthening of the building is required.

More sophisticated nonlinear analysis, static or dynamic, is explicitly allowed for the seismic evaluation. Nevertheless, as such methods of analysis are not promoted by the general Part 1-2 of Eurocode 8 for new buildings (ENV1998-1-2:1994), their application is not encouraged for old ones either, in the sense that no detailed guidance is given for them in ENV1998-1-4:1996.

Assessment of existing structures by checking compliance with a standard for the design of new ones is neither rational nor practical, as it is extremely unlikely that an old structure meets the very stringent requirements of modern codes for structural regularity, ductility at the local level (member detailing) and at the structural level (control of inelastic response through capacity design), continuity of the load path, etc. In this way all old structures, with the possible exception of low-rise ones with structural walls, may be found to be inadequate and in need of retrofitting. Moreover, to comply with a current code for new structures, practically all structural elements will need to be upgraded to meet all the resistance and detailing requirements of this code, increasing the cost of retrofitting so much, that demolition or the "do-nothing" alternative will be the most likely outcomes of the evaluation.

In view of the difficulty of conventional approaches, recent developments are in the direction of adopting different performance requirements and criteria for existing or retrofitted buildings, relative to those implicit in current codes for new buildings. The basis for this pragmatic attitude is not the presumably shorter remaining service life of an existing building (on such a basis a building could be evaluated as adequate for a future life of a few years, after the end of which the evaluation might be renewed for another period, and so on and so forth), but the recognition of the much higher total cost of seismic retrofitting (including the indirect cost of disruption of occupancy) in comparison to new construction. The differentiation is effected mainly in two ways. First, by explicitly taking into account sources of earthquake resistance and energy dissipation in the existing and in the retrofitted structure, which are normally neglected in the design of new buildings, such as the positive effects of non-structural elements (e.g. masonry infills) and the redistribution and reduction of seismic demands due to nonlinearities in structural elements and in the foundation. Second, by explicitly allowing certain structural elements to develop large and permanent post-ultimate-strength deformations, provided that their gravity-load bearing capacity is not impaired. The first point requires modelling and analysis at a higher level of sophistication than provided by current codes for the seismic design of new structures. The second implies that poor detailing and insufficient strength in many elements is not a problem, provided that global stability is assured by a few lateral-load-resisting elements, old or new. Both these points represent a significant change in the philosophy that prevailed earthquake-resistant design for the past

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decades, namely: a) the use of relatively simple, yet conservative, modelling and analysis, and b) the requirement for universal proportioning and detailing of members for strength and ductility, regardless of whether it is essential for the global seismic performance. As this new trend becomes established through successful application in assessment and retrofitting projects, it is starting to affect the codes for earthquake-resistant design of new structures as well. This represents a reversal over the past tradition, in which procedures and codification efforts for existing structures followed and emulated those for new ones.

3.2.2 Guidelines of the Japan Building Disaster Prevention Association

3.2.2.1 Introduction

Guidelines for the seismic assessment and retrofit of RC buildings were developed in 1977 by the Japan Building Disaster Prevention Association (BDPA) under the auspices of the Ministry of Construction, Building Administration Division. Guidelines for steel buildings were added in 1978. Between 1983 and 1985 guidelines for the seismic assessment (but not retrofit) of composite (steel-concrete) and timber buildings were developed.

Unlike the seismic design code for new buildings, the 1977/1990 guidelines have not been enforced so far as a law. Instead, their application has been in the jurisdiction of local authorities. Nevertheless they have been extensively applied since 1977, especially to low- and medium-rise buildings and schools. As a matter of fact, the initial development and calibration of the guidelines was based on the performance of buildings subjected to the Tokachi-oki 1968 earthquake. The guidelines were further applied for verification purposes to buildings subjected to the Izu Ohshima Kinkai 1978 and to the Miyagiken-oki 1978 earthquakes.

The part of the guidelines referring to RC buildings was revised in 1990. The revised guidelines were applied after 1995 to buildings in the Kobe area. On the basis of the experience gained in those applications, in late 1995 the Japanese Ministry of Construction issued a technical notification, which was enforced then as part of a law for the promotion of strengthening of vulnerable concrete buildings. This law requires owners of large buildings (more than 2 storeys and 1000m^2 in total floor area) with large occupancies (public assembly facilities, rental appartment buildings, office and business buildings, hotels, shops, restaurants, industrial plants, etc.) to apply to their property the assessment procedure specified in the law. If required by the outcome of this assessment, owners should then strengthen their property at the first opportunity.

3.2.2.2 The three levels of assessment

The Japan BDPA 1977/1990 Guidelines provide for three levels (or tiers) of assessment. Accuracy and reliability of the assessment increases with the level; at the same time, cost and time requirements increase also: cost of application per m^2 of floor area increases by a factor of about 2.5 from level 1 to level 2 and by a further factor of about 2.5 from level 2 to level 3.

The assessment is recommended to start at the lowest level, i.e. at level 1. If there is a clear-cut answer that the building is seismically adequate, one may proceed with normal use of the building. Otherwise the next level of assessment should be applied. Similarly at level 2. The decision to strengthen or demolish the building may be reached only after the level 3 assessment gives a negative result for the adequacy of the building.

The rationale of the three-level assessment is based on the observation that the large majority of existing and most likely substandard RC buildings can remain operational after earthquakes as strong as those of Mexico 1985, Luzon Philippines 1990, Erzincan 1992, Kobe 1995, or Kocaeli 1999. Then level 1 of the procedure aims at screening out those RC buildings which can resist such strong earthquakes by virtue of their overstrength alone, rather than of their ductility (examples of such buildings are those with shear walls of low aspect ratio). Then the more time-consuming but less conservative level 2 and 3 assessment may focus on the more vulnerable part of the building stock.

At the two lower levels beams are considered as stronger than columns; so the resistance of the building to lateral loads is estimated on the basis of the vertical elements alone. Level 3 assessment may cover also weak-beam/strong-column buildings, as the possibility of beam failure is explicitly considered.

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3.2.2.3 Storey shear demands

According to the 1977/1990 guidelines at all three levels the assessment is based on a comparison of the elastic storey shear demand, V_S, with the corresponding supply (resistance), separately for each storey and for both horizontal directions. The elastic storey shear demands V_S are computed on the basis of the elastic base shear coefficient and an assumed inverted triangular first mode shape. In the 1977/1990 guidelines the fundamental period of the building is not calculated and the base shear coefficient is taken equal to the product of the 5%-damped elastic spectral acceleration (in g's) at the acceleration-controlled part of the spectrum, times the importance factor and any other factors which apply for the design of new buildings (having to do with local soil conditions, or topography, etc.). At level 3 the elastic storey shear is taken equal to this base shear coefficient, times the weight of the overlying storeys times $1.5(n_{st}+i)/(2n_{st}+1)$ (n_{st} = total number of storeys and i = storey number, equal to 1 at ground level), which is the exact result for a linear first mode shape and uniform heightwise distribution of storey masses (including the first mode participation factor). Such a mode shape is common in weak-beam/strong-column buildings, which are best addressed by the level 3 assessment. In the two lower levels this factor is replaced by the more conservative factor: $(n_{st}+i)/(n_{st}+1)$, which assumes higher values than the factor used in level 3, especially for medium- or high-rise buildings, which are likely to develop a nonlinear first mode shape. Moreover, as the accuracy of the assessment is lowest at level 1, at that level elastic storey shears are further increased by a model factor with the value of 4/3.

3.2.2.4 Capacity modification due to building configuration and deterioration

The elastic storey shear V_S is compared to the product of the corresponding storey shear strength, V_R, times an appropriate value of the behaviour factor q and times two more modification factors, S_D and T, which account respectively for the configuration of the structure and for its condition of deterioration.

On the basis of the performance of structures during the Tokachi-Oki 1968 earthquake, in the 1977/1990 guidelines the modification factor S_D for structural configuration is taken equal to the product of nine factors for level 1 of assessment, or of eleven for levels 2 or 3. The first five and the tenth among these factors refer to irregularities of the structure in plan; the rest refer to irregularities in elevation. At level 1 only the first nine factors apply; they are the following:

- The first factor refers to the overall plan configuration of the structure; it is equal to $1-0.1g_1$, with: a) $g_1=0$ for approximately rectangular plans, in which any projections of the structure beyond re-entrant corners are less than 10% of the plan dimension in the parallel direction; b) $g_1=1$ for structures with an L-, T-, or U-shape in plan, in which projections beyond re-entrant corners are between 10% and 30% of the length of the structure in the given direction; and c) $g_1=2$ for more complex and irregular shapes in plan.
- The second factor is for the aspect ratio of the plan of the structure; it is equal to $1-0.05g_2$, in which: a) $g_2=0$ if the aspect ratio is less than 5; b) $g_2=1$ if the aspect ratio is between 5 and 8; and c) $g_2=2$ if the aspect ratio exceeds 8.
- The third factor is due to any constriction or pinching at the centre of the plan; it is equal to $1-0.05g_3$, with: a) $g_3=0$ if the dimension of the most narrow part is at least 80% of the overall plan dimension in the given direction; b) $g_3=1$ if this most narrow part is between 50% and 80% of the overall dimension; and c) $g_3=2$ if it is less than 50% of the latter.
- The fourth factor refers to seismic joints; it is equal to: $1-0.05g_4$, with: a) $g_4=0$ if at the storey of interest the joint is wider than 1% of the elevation from ground level; b) $g_4=1$ if the joint width is between 0.5% and 1% of the elevation of the given storey; and c) $g_4=2$ if the joint is more narrow than 0.5% of the elevation from ground level.
- The fifth factor refers to the size of an open court (atrium) inside the plan; it is equal to $1-0.05g_5$, with: a) $g_5=0$ if such an open court occupies less than 10% of the overall surface area in plan (or if there is no open court); b) $g_5=1$ if the surface area of the open court is between 10% and 30% of the overall surface area; and c) $g_5=2$ if this surface area is more than 30% of the overall area.
- The sixth factor refers to the location of an open court that may exist within the plan; it is equal to $1-0.025g_6$, with: a) $g_6=0$ if the open court is eccentric with respect to the

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overall plan by less than 10% of the longer of the two overall plan dimensions and 40% of the shorter (including the case of no open court); b) g_6=1 if the eccentricity of the open court is less than 40% of the shorter overall dimension in plan, but between 10% and 30% of the longer; and c) g_6=2 if the eccentricity exceeds 40% of the shorter overall dimension or 30% of the longer.

- The seventh factor covers the beneficial effects of a basement, possibly smaller in plan than the ground storey; it is equal to 1.2-0.1g_7, with: a) g_7=0 if the basement occupies in plan the same area as the ground storey; b) g_7=1 if the basement surface area is between 50% and 100% of that of the ground storey; c) g_7=2 if the basement takes up less than 50% of the ground storey area or if there is no basement.
- The eighth factor is due to irregularities in storey height; it is equal to 1-0.05g_8, with: a) g_8=0 if the storey of interest is not more than 20% shorter than the one directly above (or below, if we are talking about the top storey); b) g_8=1 if the storey in question is by 20% to 30% shorter than the afore-mentioned adjacent storey; c) g_8=2 if it is shorter by more than 30%.
- The last factor for level 1 refers to the possible presence of shear walls above the ground storey which are discontinued at the ground storey itself; it is equal to 1-0.05g_9, with: a) g_9=0 if there are no such discontinuous walls; b) g_9=1 if such shear walls are symmetrically arranged throughout the plan; and c) g_9=2 if the discontinuous shear walls are eccentric in plan.

At levels 2 and 3 all modification factors above apply but: a) with half the above values of g_i for i=1 to 5 and 8 or 9 (i.e. 0.5 instead of 1.0 and 1.0 instead of 2.0); b) always with g_6=0, regardless of the eccentricity of an open court in plan; and c) with the same values of g_7 as in level 1. Moreover two additional reduction factors are applied, both referring to irregularities of stiffness and mass in plan and in elevation:

- The first additional factor refers to the eccentricity, e, of the storey centre of mass with respect to the storey centre of stiffness; it is equal to 1-0.1g_{10}, with: a) g_{10}=0 if the value of e is less than 10% of the sum B+L of the plan dimensions; b) g_{10}=1 if e is between 0.1(B+L) and 0.15(B+L); and c) g_{10}=2 if the value of e exceeds 0.15(B+L).
- The second factor is equal to 1-0.1g_{11}, with: a) g_{11}=0 if the stiffness-to-mass ratio, K/M, of the storey above does not exceed that of the storey of interest by more than 20%; b) g_{11}=1 if the K/M ratio in the storey above is between 1.2-times and 1.7-times that of the storey in question; and c) g_{11}=2 otherwise. The sum of the cross-sectional areas A_c of the vertical elements of the storey may be taken as a measure of the storey stiffness K in this calculation; the mass M is the total mass in the overlying storeys.

The reduction factor T for deterioration assumes a single value for the entire building and accounts for its age and condition in the following way:

- At level 1 the value of T is equal to the minimum value determined at any storey according to the following factors:
- Age, with T=0.8 in buildings more than 30 years old, T=1.0 for those with less than 20 years of age and T=0.8 to 0.9 for the intermediate cases.
- The use of the building, with T=0.8 if it involves aggressive chemical compounds.
- The possible damage due to past fires, with T=0.8 if such damage has been repaired or T=0.7 otherwise.
- The deformations of the building, with T=0.7 if the building is out-of-plumb or if uneven settlements have developed, T=0.9 if there is visible deformation of beams or columns, or if the building is founded on reclaimed land.
- The state of cracking and deterioration of structural and non-structural elements, with T=0.9 if the finishes are sevely deteriorated or if structural members or partition walls have visible cracks or even leaks but without signs of reinforcement corrosion, or T=0.8 if there are signs of corrosion.

At levels 2 and 3 the value of T for the building is equal to the mean of the values determined in the individual storeys.

3.2.2.5 Storey capacity

The calculation of storey shear strength V_R and the determination of the storey behaviour factor q differ among the three levels of assessment. At level 1 an average shear strength (in terms of shear stress) and a single representative value of q is considered for each type of vertical element. At level 2 the shear strength V_{Ri} of each vertical element is more accurately

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computed on the basis of its reinforcement and of its most likely failure mode, discounting the possibility of beams failing before columns. This possibility is considered at level 3. Details of the calculations are given in the following.

At all three levels of assessment torsional effects due to irregularity or unbalance of the distribution of resistance in plan are normally neglected in the calculation of V_R. Such effects are considered empirically through the structural configuration factor S_D that multiplies qV_R. When the eccentricity in plan exceeds a certain limit, the level 2 and 3 assessment procedures estimate qV_R as the minimum of the following: a) the value computed neglecting the elements causing eccentricity, and b) the value determined neglecting any element with (local) q_i-factor smaller than those of the elements causing the eccentricity.

(1) Calculation of storey capacity at level 1

Storey failure is considered to take place when the most brittle of the storey elements fails. If in the storey and direction of interest there are columns with shear span ratio L_s=M/Vh less than 1.0 ("short columns" – L/h<2), the value of q is taken equal to 0.8 for the entire storey and the storey shear strength V_R is taken equal to the total shear strength of these short columns plus the fraction α_i of the shear strength of the slender columns and walls attained at the ultimate strength and displacement of the short columns. If the short columns are not essential for gravity load capacity, in the sense that their gravity loads can be safely transferred to and carried by neighbouring elements, the short columns may be neglected. In the absence of short columns the q-factor of the entire storey is taken equal to 1.0 and the storey V_R is taken equal to the shear strength of (any) walls plus the fraction α_i of the column shear strength attained at failure of the walls. If there are no walls in the storey, V_R is the sum of column shear strengths. The storey drift at failure of short columns is assumed to be equal to that at attainment of 70% of the ultimate strength of shear walls (α_i=0.7 for walls, in the presence of short columns). The storey drift at ultimate strength of walls is taken equal to 70% of the drift ratio at column ultimate strength (α_i=0.7 for columns in the presence of walls and α_i=0.7x0.7=0.5 in the presence of short columns). The shear strength V_{Ri} normalized to the gross concrete section A_c is taken equal to $0.075f_c$ for short columns (L/h<2), to $0.05f_c$ for columns with L/h between 2 and 6 and to $0.035f_c$ for those with L/h>6. For walls the – normalized to A_c – shear strength V_{Ri} is taken equal to $0.05f_c$ if the wall has no column-like boundary elements (or zones) at the ends of its cross-section, to $0.1f_c$ if it has such an element at one end of the section, or to $0.15f_c$ if it has such elements at both ends. These shear strength values are consistent with member dimensions and reinforcement ratios common in Japanese RC buildings before 1971, i.e. at a time when the contribution of transverse steel to shear strength was neglected and shear resistance was considered to be provided by concrete alone.

It is clear that level 1 relies on strength alone and not on ductility (q is taken equal to 1.0 or less) and that it favours buildings with walls over those with frames. Frame buildings are penalized by being evaluated too conservatively.

(2) Storey capacity at level 2

Level 2 differs from level 1 regarding the determination of the shear strength V_{Ri} and of the behaviour factor q_i of the individual vertical elements of a storey, as described in detail below:

For every vertical element the flexural and shear capacities M_u and V_u are computed, on the basis of the information available regarding their reinforcement and material strengths. Flexural capacities M_u are converted into associated shear forces, $V_{Mu}=M_u/L_s$. This may be done assuming attainment of flexural capacity at both ends for the columns (shear span L_s equal to half the clear storey height), or a shear span L_s of walls equal to half the distance from the base of the storey of interest to the top of the building (or the full storey height at the top storey). If $V_{Mu}=M_u/L_s$ is found less than the shear capacity per se, V_u, V_{Mu} is taken as the shear strength V_{Ri} and a flexural failure mode is considered as most likely; the element behaviour factor q_i is taken then greater than 1.0. Otherwise, the member shear strength V_{Ri} is considered equal to V_u and q_i is taken equal to 1.0.

More specifically, for walls we take q_i=1 if $V_u/V_{Mu}=V_uL_s/M_u \leq 1.2$ and q_i=2 if $V_u/V_{Mu}=V_uL_s/M_u \geq 1.3$ (with interpolation in-between).

Columns with either: a) peak axial compression above the balance point (axial load ratio $\nu=N/A_cf_c>0.4$); or b) with shear force at flexural failure, $V_{Mu}=M_u/L_s$, greater than $0.2f_cb_wz$

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(b_w and $z \approx 0.9d$ are the width and the internal lever arm of the column section); or with c) ratio of tension reinforcement above 1%, are considered to be brittle and their q_i–value is taken equal to 1.0. In all other columns, the available displacement ductility factor is estimated from the empirical relation: $\mu=10(V_u L_s/M_u-1)-30(V_u/b_w z f_c-0.1)-2$, in which the last term (i.e. the 2) is omitted if the stirrup spacing s_h is less than eight times the longitudinal bar diameter d_b. The value of μ used thereafter is limited between 1 and 5. From this value of μ the column behaviour factor q_i is computed, using the relation: $q=\sqrt{2\mu-1}\ /(0.75(1+0.05\mu))$, which yields results higher than those of the expression: $q=\sqrt{2\mu-1}$ often considered to apply in the acceleration-controlled region of the spectrum. The relation used was empirically derived as a lower bound (conservative) envelope - over the period and ductility ranges of common interest - from the results of nonlinear dynamic analyses of a SDOF system which followed a stiffness degrading hysteresis model with a trilinear monotonic curve and was subjected to the El Centro 1940 (NS), Taft 1952 (EW) and Hachinohe 1968 (EW and NS) records.

If there are columns in the storey which have slenderness ratio $L/h<2.0$ (short) and are essential as gravity load-bearing elements (i.e. their share of gravity load cannot be safely undertaken by neighbouring elements), then the value $q=0.8$ applies to the entire storey and the storey shear strength is determined as in level 1 above, the only difference being that the more accurate values for the shear strengths of the individual vertical elements are used.

If in the storey there are only columns with $L/h>2$ and/or shear walls, then the storey vertical elements are grouped in (up to) three classes of elements with similar values of q_i. Each class is assigned the lowest value of q_i of its members. One class may comprise those elements likely to fail in shear, for which $q_i=1$. The other classes may include shear walls likely to fail in bending with values of q_i between 1.0 and 2.0 and separately those columns which are critical in flexure and have q_i-values (well) above 2.0. The deformations (storey drifts) at ultimate strength of these (up to) three classes of elements may differ significantly, implying that at ultimate strength of one class of elements those of another class may not have reached ultimate strength, or may be at their post-ultimate falling branch. If one of the three classes consists of shear-critical elements with $q_i=1$, failure of the storey is considered to take place at failure of these brittle elements and the shear strength of the storey is computed from a linear combination of the shear strengths of its vertical elements similar to that used in level 1, without considering any beneficial effects of ductility. If the two or three groups of elements all have values of q_i greater than 1, then the product of a q-factor times the ultimate strength, qV_R, is computed for the storey as the maximum of two values: a) the linear combination of element ultimate strengths mentioned at the beginning of this paragraph and used also in level 1, times the lowest q_i value of the (up to three) groups; and b) an SRSS-type of combination of the products $q_i V_{Ri}$ of the (at most) three classes, as $qV_R=(\Sigma(q_i V_{Ri})^2)^{1/2}$. This rule was developed empirically from the results of nonlinear dynamic analyses of a parallel system consisting of: 1) an element with bilinear monotonic curve and origin-centred hysteresis, considered to fail at a ductility demand of 2; and 2) a ductile element with trilinear monotonic curve and stiffness degrading hysteresis. This system was subjected to the Taft 1952 record, for various combinations of strengths of its constituent elements. The rule above is an approximation to the combination of element strengths leading to either failure of (the less ductile) element listed under 1, or to constant ductility demand in (the more ductile) element listed under 2.

(3) Storey capacity at level 3

At level 3 shear strengths V_{Ri} and behaviour factors q_i of vertical elements are determined taking into account the possibility of pre-emptive failure (in shear or in flexure) of the beams framing into them. Moreover, for shear walls their behaviour as a whole is considered, from the foundation to the top of the building.

To determine whether the beams or the columns fail first, at the beam-column joints at the top and bottom of a column, column and beam moment capacities are compared, considered in pairs acting in the same sense on the joint. These beam capacities are denoted here as M_{Rb1}^- and M_{Rb2}^+, when they cause tension at the top of the beam on one side of the joint and at the bottom on the other, and as M_{Rb1}^+ and M_{Rb2}^- when they act in the opposite sense. Normally

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these capacities are controlled by flexure and are equal to the beam flexural capacity M_u at the face of the column (approximately equal to the tension reinforcement area, A_{sl}, times its yield strength, f_y and the internal lever arm $z=0.9d$). In some cases, they may be controlled by shear. It is unlikely that this will be the case at the beam end which is in positive bending, as there the shear due to gravity loads, V_G, counteracts the shear due to the bending moments at the beam ends. If at the beam end which is in negative bending (indexed here with i) the beam shear strength, V_u, is less than the maximum shear force that can develop there, $V_{Gi}+(M_{ubi}^-+M_{ubj}^+)/L_{bn}$, then M_{Rbi}^- should be taken equal to $M_{Vu}=(V_u-V_{Gi})L_{bn}-M_{ubj}^+$ (where L_{bn} is the beam clear span and j indexes the other end of the beam, considered in positive bending). So $M_{Rb}^- = \min(M_u, M_{Vu})$.

Similarly the pairs of column moment capacities above and below a joint are computed: M_{Rc1}^+ and M_{Rc2}^- is the first pair, or M_{Rc1}^- and M_{Rc2}^+ the second one (typically $M_{Rc1}^+=M_{Rc1}^-$, $M_{Rc2}^+=M_{Rc2}^-$). The column moment capacity M_{Rc} is the smaller of the column flexural capacity, M_u, and of the product of its shear strength, V_u , times the shear span, L_s (taken for simplicity as half the storey clear height). The likely failure mode of the column is also determined: the column fails in flexure if $M_u < M_{Vu}=V_uL_s$, or in shear if $M_u > M_{Vu}=V_uL_s$. The sums $M_{Rb1}^-+M_{Rb2}^+$ and $M_{Rb1}^+ + M_{Rb2}^-$ at the joint are compared to the corresponding sums for the columns: $M_{Rc1}^-+M_{Rc2}^+$ and $M_{Rc1}^+ + M_{Rc2}^-$. (Transfer of all these sums to the centre of the joint may be needed, if the ratio of the joint horizontal dimension to the beam span differs significantly from that of the joint vertical dimension to the storey height). If $M_{Rc1}^-+M_{Rc2}^+ > M_{Rb1}^-+M_{Rb2}^+$, the beams control the magnitude of the moments around the joint and the mean of M_{Rb1}^- and M_{Rb2}^+ (or, more generally, the values of M_{Rc1}^- and M_{Rc2}^+ multiplied by the ratio: $(M_{Rb1}^-+M_{Rb2}^+)/(M_{Rc1}^-+M_{Rc2}^+)$) is taken as the column moment capacities instead of M_{Rc1}^- and M_{Rc2}^+. Similarly for the opposite sense of action of moments around the joint, i.e. for M_{Rc1}^+ and M_{Rc2}^-.

The column shear strength, V_{Ri}, is taken as the sum of the column moment capacities at storey top and bottom, computed according to the two paragraphs above, divided by the column clear height. The value of the column behaviour factor depends on the likely failure mode. Similarly to level 2, q_i is taken equal to1, if at beam-column joints the columns are more critical than the beams and they fail in shear (i.e., if $M_u > M_{Vu}$); if they are controlled by flexure, then $q_i=\sqrt{2\mu-1}/(0.75(1+0.05\mu))$, with the column available ductility factor μ determined as in level 2. If beams fail before the columns at the joint and one of them fails in shear, (i.e. if $M_{Rb}^-=M_{Vu}$), $q_i=1.5$ applies for the column; for flexural failure of the beam, $q_i=3$ applies. The minimum (rather than the average) q_i value should be used for a column, if the likely failure modes at its top and bottom are different.

The storey shear strength in shear walls, V_{Ri}, is the value of the storey shear at attainment of the ultimate resistance of the wall as a whole, subjected to lateral forces with an inverted triangular heightwise distribution. This ultimate resistance may be reached due to: a) attainment of flexural capacity at any section of the wall from the foundation to the top; b) exhaustion of the shear strength of the wall at any storey; or c) attainment of the overturning moment of the wall (product of the wall axial force times the distance between the edge and the centre of the wall footing) at the interface of the footing with the soil, whichever happens first. For failure modes a) or b), the q_i–value of the wall is taken equal to the values quoted for level 1. For mode c), q_i is taken equal to 3.0. In the calculation of the wall ultimate resistance to lateral forces with inverted triangular heightwise distribution, the bending moments of beams framing into the wall at storey levels are taken equal to the corresponding capacities, as these are determined by shear or flexure in the beam. The sense of action of these moments is opposite to that of the bending moments in the wall itself.

The combination of the values of V_{Ri} and q_i of the individual vertical elements into a single qV_R value for the storey is done as in level 2.

In general level 3 produces lower storey shear strengths V_R than level 2, but higher q-factor estimates. The value of the product qV_R is then higher. This result, along with the use in level 3 of $1.5(n_{st}+i)/(2n_{st}+1)$ instead of $(n_{st}+i)/(n_{st}+1)$, makes level 3 assessments less conservative than level 2 ones.

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(4) Consideration of joint shear failure at level 3

The 1977/1990 Guidelines do not consider beam/column joints as separate components to be assessed. This may be due to the fact that the members of existing RC buildings are typically more critical than the joints they frame into (especially in Japan, where prior to 1971 only the concrete contribution to shear resistance was considered, leading to large member cross-sections and hence to voluminous joints). If members are heavier reinforced and cross-sectional dimensions are small, (unreinforced) joints may be more critical than the members themselves. Then, calculation of V_{Ri} at level 3 for each beam-column subassembly should include the determination of the joint shear strength itself and a check of whether the joint controls the value of V_{Ri}. For completeness, and as it will be referred to in the description of other assessments procedures, one approach that may be used to this end (Fardis, 2001, Panagiotakos et al, 2002) is given here.

The shear force that develops in a joint by bond along the extreme beam or column bars passing through, is related to the moments developing in the beams and columns framing into it. If the sum of beam moments at opposite faces of the joint, ΣM_b, governs the shear input in the joint, the horizontal shear force V_{jh} in the joint is:

$$V_{jh} = \sum M_b \left(\frac{1}{z_b} - \frac{1}{h_{st}} \frac{L_b}{L_{bn}} \right) \tag{3-1}$$

where h_{st} is storey height, L_b and L_{bn} the mean theoretical and clear span of the beams on either side of the joint and $z_b=d-d_1\approx 0.9d$ the beam internal lever arm. The shear stress demand in the joint is:

$$v_j = \frac{V_{jh}}{b_j h_c} \tag{3-2}$$

with h_c: column cross-sectional depth in the horizontal direction in which the joint is checked and b_j: width of the joint in the transverse horizontal direction, usually taken as:

$$b_j = \min\left(\max(b_c, b_w), 0.5h_c + \min(b_c, b_w)\right) \tag{3-3}$$

with b_c and b_w denoting the width of the column and the beam in the direction normal to h_c.

Alternatively, the sum of column moments at opposite faces of the joint, ΣM_c, may control the shear input therein. Usually column vertical bars are the same above and below the joint. If the total cross-sectional area of all vertical bars at the two extremes of the column section and of the joint core is denoted by $A_{sc,tot}$, the vertical shear force in the joint core is:

$$V_{jv} = \sigma_s A_{sc,tot} + N_{top} - V_{b,\min} \tag{3-4}$$

where N_{top}: axial force in the column above, $\sigma_s \leq f_y$: average (absolute value of the) stress in the column reinforcement above and below the joint, and $V_{b,min}$: minimum beam shear force on either side of the joint, about equal to:

$$V_{b,\min} \approx \min\left(\frac{\sum M_c}{L_b} \frac{h_{st}}{h_{st,n}} - V_{g+\psi_2 q,b} \right) \tag{3-5}$$

$V_{b,min}$ may be positive or negative. In eq. (3-5) $h_{st,n}$ is the clear storey height – average value – and $V_{g+\psi 2q,b}$ the shear force at the beam end due to gravity loads alone.

As:

$$\sum M_c \approx \sigma_s A_{s,tot} z_c + 0.5 h_c \left(N_{top} \left(1 - \nu_{top}\right) + N_{bot} \left(1 - \nu_{bot}\right) \right) \tag{3-6}$$

where $\nu = N/A_c f_c$, *top* and *bot* index the column section above and below the joint and z_c (internal lever arm of the column) $\approx 0.9d \approx 0.8h_c$, eq. (3-4) yields:

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$$V_{jv} \approx \sum M_c \left(\frac{1}{z_c} - \frac{1}{L_b} \frac{h_{sl}}{h_{sl,n}} \right) + \max V_{g+\psi_2 q,b} \tag{3-7}$$

The shear stress in the joint core is (h_b: beam depth):

$$v_j = \frac{V_{jv}}{b_j h_b} \tag{3-8}$$

Diagonal tension cracking of the joint core will take place when the principal tensile stress under the combination of v_j and of the mean vertical compressive stress in the joint, $\nu_{top} f_c$, exceeds the tensile strength of concrete, f_{ct}. This takes place when:

$$v_j \geq v_c = f_{ct} \sqrt{1 + \frac{\nu_{top} f_c}{f_{ct}}} \tag{3-9}$$

(cf. eq. (4-36) in Chapter 4). According to Priestley (1997), in exterior joints with bars bent vertically towards the joint core (instead of outwards into the column above and below) confinement due to the bent bars increases the joint shear stress at diagonal cracking by 50% over the value v_c in the right-hand-side of eq. (3-9).

Diagonal cracking of the joint core rarely has catastrophic consequences, especially if beams of significant cross-section frame into the joint from more than two sides. The ultimate threat is crushing of the unreinforced joint core due to diagonal compression. This may be considered to take place if v_j exceeds the limit:

$$v_j \geq v_{ju} = n f_c \sqrt{1 - \frac{\nu_{top}}{n}} \tag{3-10}$$

where: $n=0.7-f_c(\text{MPa})/200$: reduction factor on f_c due to simultaneous transverse tensile strains. Nonetheless, in the absense of horizontal hoops in the joint, it is doubtful that eq. (3-10) applies after diagonal cracking of the joint. Therefore, in such joints the conservative limit of eq. (3-9) should be considered to control the joint shear strength.

On the basis of eqs. (3-1), (3-2), the limit of eq. (3-9) gives the following upper limit for the sum of beam moments at opposite faces of the joint:

$$\sum M_b \leq \frac{v_c b_j h_b}{\frac{1}{z_b} - \frac{1}{h_{sl}} \frac{L_b}{L_{bn}}} \tag{3-11}$$

The second term in the denominator of eq. (3-11) is neglected at the top storey. The right-hand-side of eq. (3-11) is the upper limit of the sum of bending moments: $M_{Rb1}^- + M_{Rb2}^+$ or $M_{Rb1}^+ + M_{Rb2}^-$, that can develop in the beams framing into the joint. If the value of this right-hand-side is less than the corresponding sum(s), ΣM_b, determined on the basis of the flexural and shear capacities of the beams themselves, M_u and V_u, it is this value that should be compared with the sum ΣM_{Rc} of the columns. If the right-hand-side of eq. (3-11) is also less than the sum ΣM_{Rc} for the columns, the column shear strength, V_{Ri}, should be computed on the basis of column end moments equal to the corresponding value of M_{Rc1} or M_{Rc2} multiplied by the ratio of this right-hand-side to the value of $M_{Rc1}+M_{Rc2}$ at the joint. The corresponding behaviour factor may be taken equal to $q_i=1.5$, as, due to (partial) confinement by the surrounding members, a shear failure in a joint is not more brittle or catastrophic than in a beam.

Similarly, eqs. (3-7)-(3-9) provide the following limit to the sum of column moments above and below the joint:

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$$\sum M_b \le \frac{v_c b_j h_c - \max V_{g+\psi_2 q,b}}{\frac{1}{z_c} - \frac{1}{L_b}\frac{h_{st}}{h_{st,n}}} \qquad (3\text{-}12)$$

If the value of the right-hand-side of eq. (3-12) is less than the corresponding sum, ΣM_c, determined on the basis of the flexural and shear capacities of the columns themselves, M_u and V_u, it is this value that should be compared with the sum ΣM_{Rb} of the beams. If this right-hand-side is also less than ΣM_{Rb}, the column shear strength, V_{Ri}, should be computed on the basis of column end moments equal to the corresponding value of M_{Rc1} or M_{Rc2} multiplied by the ratio of this right-hand-side to the value of $M_{Rc1}+M_{Rc2}$ at the joint and the behaviour factor may also be taken equal to $q_i=1.5$.

3.2.2.6 Concluding remarks

At first sight the 1977/1990 Guidelines seem to follow a force-based approach. Nonetheless, in reality they fall in between force- and displacement-based assessment. As a matter of fact, they may be closer to a displacement-based approach, as they employ a comparison between the elastic storey shear demand (which according to the "equal-displacement" rule is roughly proportional to the inelastic interstorey drift demand) to a quantity qV_R that attempts to approximate, albeit in a rough way, the deformation capacity of the storey as a whole, as this is determined either by the deformability of its most brittle element(s), or by the deformation capacity of its ensemble of ductile elements, whichever is critical.

The 1977/1990 BDPA Guidelines were implicitly based on the assumption that the building assessed has been designed for elastic response and working stress design to a base shear coefficient of 0.2 (like most existing RC buildings in Japan). This affects mainly the applicability of level 1, which is based on assumed member shear strengths representative of pre-1977 Japanese design and construction practice. This factor does not seem to adversely affect the applicability of the level 2 and 3 approaches under more general conditions, especially with the generalisations made in the presentation above. It is also claimed that the validity of these guidelines is limited to buildings with less than seven storeys, because the demand base shear coefficient is always taken from the constant spectral acceleration part of the spectrum. For taller buildings this drawback may be overcome if the base shear demand is based on the estimated fundamental period of the building. Indeed this is done in Notification 2089 of the Ministry of Construction issued with the 1995 law for the promotion of strengthening, as outlined below.

The law introduced at the end of 1995 for the promotion of vulnerability assessment of RC buildings adopts the level 3 approach of the 1977/1990 BDPA Guidelines, with some modifications aiming at increasing its accuracy and bringing it in line with the code for the seismic design of new buildings.

One modification is the estimation of the elastic base shear demand using the full acceleration spectrum specified for new buildings, with the fundamental period (conservatively) estimated as T=0.02H(m).

Another modification is the replacement of factor $1.5(n_{st}+i)/(2n_{st}+1)$ for the conversion of the base shear coefficient to one for storey i, by the factor $A_i=1+2T(\frac{1}{\sqrt{\alpha_i}}-\alpha_i)/(1+3T)$ (with α_i denoting the ratio of building weight above storey i to the total) of the current Japanese seismic code for new buildings.

A third modification is the replacement of the structural configuration factor S_D, which reduces the storey shear capacity for heightwise and planwise irregularities, by an increase of the storey shear demand, through multiplicative factors specified for this purpose in the Japanese code for new buildings. The value of these factors may differ from storey to storey; it depends: a) for heightwise irregularity, on the ratio of a measure of storey stiffness (inverse of storey drift under equivalent static lateral forces) to the mean value of this measure over all storeys; and b) for planwise irregularity, on the ratio of the eccentricity between centres of stiffness and mass to the torsional radius of the storey in the horizontal direction of interest.

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As far as the capacity side of the equation is concerned, the modification in the level 3 procedure consists in the discretisation of the q_i-factors of those flexure-critical columns that are more critical than the beams they are connected to, into three values: a) a value of 3.2 for highly ductile columns impossible to fail in shear, b) a value of 2.2 for ductile columns unlikely to fail in shear, and c) a value of 1.3 for less ductile columns still unlikely to fail in shear.

The building is assessed as "unlikely to collapse" if at every storey and in both horizontal directions: a) the total storey shear strength V_R (calculated by summing the shear resistances V_{Ri} of the individual vertical members with q_i=1) exceeds the (inelastic) storey shear force demand according to the code for new buildings, amplified for the effects of irregularities in plan and elevation; and b) the value of qV_R of the storey exceeds the elastic storey shear demand, taken as twice the (inelastic) shear force demand in (a) above. If one of these two conditions is violated at any storey, collapse of the building is considered possible. If, finally, one of the conditions (a) or (b) is violated at any single storey by a factor of more than 2.0, the building is assessed as "likely to collapse". Owners of buildings of this last category are required to strengthen their property to the level of "unlikely to collapse" at the earliest opportunity.

3.2.3 The forced-based approach in the 1996 New Zealand (draft) guidelines

3.2.3.1 Introduction

Draft guidelines for seismic assessment and retrofitting were first released in 1996 by the New Zealand (NZ) National Society for Earthquake Engineering. They were prepared for the NZ Building Industry Authority and referred to detailed assessment of individual buildings (mainly concrete frame and wall buildings, but to a certain extent steel moment frames as well).

As the document has developed since 1996 up to the time of this writing (early 2002), its aim is to be nominated – after completion – in the NZ Building Code Handbook as a means of compliance with the - revised - Section 66 of the NZ Building Act, which addresses buildings not capable of adequate seismic performance. The intent is to oblige competent local or regional authorities to consider the risk posed by hazardous buildings and provide them with an opportunity and the means to take appropriate action. Further, the intent is to empower competent authorities – through the amended Sections 64 and 65 of the NZ Building Act – to require buildings not deemed seismically adequate to be made to comply with the performance requirements applicable to new buildings as closely as practicable and within a reasonable timeframe. It is envisaged that this may be accomplished through either active or passive programmes. In active programmes, competent authorities will set target performance levels for existing buildings, identify and prioritize the hazardous ones, and serve notice to their owners to assess them in detail and retrofit them as necessary, at their own cost and within a given time-frame. In passive programmes owners will be required to assess their buildings and retrofit them, only when they apply for a alteration or change in use; moreover, a target date (e.g. of 2020) may be set for all existing buildings to be assessed in detail and retrofitted if necessary, regardless of alteration or change in use.

Similarly to the Japanese Guidelines and to the ENV versions of EC8, the NZ draft guidelines address only the life safety performance level under the hazard level intended for the design seismic action of new buildings.

In their 2002 form, the draft guidelines include also an "initial evaluation procedure", for use by competent authorities engaged in active programmes, to identify and prioritize high risk buildings, in order to ask their owner to proceed further with a detailed assessment. An "initial evaluation" is essentially a visual assessment, supplemented with general knowledge of the building and information from previous assessments. An experienced professional engineer is meant to need between 2 and 4 hours to complete one for an ordinary multistorey building. Competent authorities may also subject buildings to a preliminary "initial evaluation", focusing on vulnerability features and obvious structural deficiencies, in order to identify priority buildings to be subjected further to the full "initial evaluation".

In their 2002 version, the draft Guidelines recognize that it is totally impractical to try to bring (all) existing buildings up to the seismic performance standard currently applicable to new buildings. Therefore, the draft Guidelines consider existing buildings acceptable, if they have satisfactory performance under one-third of the design seismic action of the

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corresponding new building (i.e. of similar importance and occupancy, at the same site and over the same type of soil profile). If they don't, they are characterized as high risk buildings and ultimately – i.e. if such a characterization is confirmed by a detailed assessment – they are required to be retrofitted, so that they have satisfactory performance under two-thirds (not 100%) of the design seismic action of the corresponding new building. This reduction of performance requirements reflects society's presumed acceptance of higher risk for existing buildings and its readiness to adopt a pragmatic approach in its pursuit of reduced seismic risk for the current building stock.

The ratio of the seismic action for which seismic performance of a building is found to be adequate, to the design seismic action of the corresponding new building is expressed in the draft NZ Guidelines in percent and termed "seismic performance score" (SPS). A priority and grading index is derived for each building, by reducing the value of SPS through factors which collectively range from about 1.3 to 0.7 and account for the consequences of collapse on loss of life (maximum number of occupants in normal operation, occupants per total floor area, weekly duration of normal occupancy and risk to people outside the building). The recommended time-frame for retrofitting (in years) is 20% of the so-derived grade of the building.

3.2.3.2 The initial evaluation procedure

During an initial evaluation procedure the value of SPS is determined in two steps. In the first step, the SPS is estimated as the elastic base shear capacity of the building, expressed as percentage of the design elastic base shear of the corresponding new building according to current NZ standards, assuming that the building has no significant vulnerability features or deficiencies (termed "critical structural weaknesses"). In the second step, the SPS value is reduced due to such features and deficiencies.

It is assumed in the first step that the building complies fully with the code that applied at the time of its construction. Then its SPS value (in percent) is estimated from the ratio of elastic base shear coefficients at the time of construction and at present, taking into account:

- Date of construction (and hence applicable code, including whether ULS or Working Stress Design applied at the time).
- Soil type (as this affects the design base shear coefficient now and then).
- Seismic zone (for the same reason as above).
- Fundamental period (estimated as 0.1 or 0.06 times the number of storeys in frame or wall buildings, respectively, also as it affects the design base shear coefficient now and then).
- Importance and occupancy of the building (as it affects the importance factor).
- Available global ductility factor of the building (value ranging between 1.25 and 2, depending on code applied for its design and construction).

Quantitative details are not given here, as these are closely related to the rules of design codes applying in New Zealand currently and in the past.

"Critical structural weaknesses" considered in the second step are:

1. Irregularity in plan.
2. Irregularity in elevation.
3. Short columns.
4. Site and soil issues (potetial for landslide, liquefaction, etc.).
5. Pounding with adjacent buildings.

The effect of each one of these five types of "Critical structural weaknesses" on the seismic performance of the building may be classified as:

- Severe (implying probable total collapse under the design seismic action due to this particular "weakness"),
- Significant (implying possible partial collapse under the design seismic action due to the "weakness"), or
- Insignificant.

Irregularities in plan with "severe" effect are the following: At least one wing in a non-rectangular plan (T-, L-, U- or E-shaped plan, etc.) with aspect ratio above 3, or at least two wings with aspect ratio more than 2; stiff structural or non-structural elements, such as ramps, stairwells, walls or heavy partitions, grouped near a corner or side of the plan; aspect ratio in plan above 5; eccentricity between centres of mass and stiffness greater than 50% of the parallel plan dimension. Irregularities in plan with "insignificant" effect are: Wings in a non-

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rectangular plan (T-, L-, U- or E-shaped, etc.) with aspect ratio less or equal to 2; aspect ratio in plan not exceeding 4; eccentricity between centres of mass and stiffness not greater than 40% of the parallel plan dimension. Plan irregularities with "significant" effect are those in-between the two extremes above, including one wing in a non-rectangular plan (T-, L-, U- or E-shaped, etc.) with aspect ratio between 2 and 3 and some collective influence of stiff structural or non-structural elements.

As irregularities in elevation with "severe" effect are considered any difference in mass or lateral stiffness between adjacent storeys above 150%, or any vertically discontinued structural elements that account, in the storeys they exist, for at least 50% of lateral stiffness. Differences in storey mass or lateral stiffness below 100%, or vertically discontinued structural elements accounting for not more than 30% of lateral stiffness, are considered as irregularities in elevation with "insignificant" effect. Intermediate cases are considered to have an "insignificant" effect.

Columns are classified as "short" not on the basis of their shear-span ratio, but on whether their clear height (between confining infill walls and/or horizontal structural elements) is less than 70% of the storey height. Their effect is deemed "severe", if at least 60% of all columns in a storey or on a side of the plan are classified as "short", while it is considered "significant" if they account for 40% of all columns in a storey or on two adjacent sides in plan. In all other cases, their effect is considered "insignificant".

Sites or soils with "severe" effects are considered as those with probable liquefaction, instability, or extensive landslide. If there is just potential for liquefaction or instability, or for a landslide above the building, sites or soil effects are deemed as "significant".

The penalty applied (as a multiplicative factor) on the SPS estimated from the first step due to "weaknesses" of categories 1 to 4 above is equal to 0.7, if the "weakness" is "significant", and to 0.4 or 0.5, if the "weakness" is of category 1 to 3 (irregularities or short columns) or 4 (site and soil), respectively.

Regarding category 5 (pounding), two sub-categories are defined: 5-a is pounding per se and 5-b is the effect of height differences between the adjacent buildings. For 5-a, two factors are considered, having to do with the local (impact) effects of pounding: the magnitude of the gap between the buildings and the (maximum) misalignment of their floors. If the gap exceeds 1% of the building height and floor misalignment is more than 20% of storey height, or if the gap is from 0.5 to 1% of building height and floor misalignment is less than 20% of storey height, the penalty factor is 0.8. If the gap is less than 0.5% of the building height and floor misalignment is less than 20% of storey height, or if the gap is from 0.5 to 1% of building height and floor misalignment exceeds 20% of storey height, the penalty factor is 0.7. The penalty factor is 0.4, when the gap is less than 0.5% of the building height and floor misalignment is more than 20% of storey height.

The above penalty factors for 5-a apply to RC frame buildings. In buildings with shear walls the penalty factors quoted apply for the next range of gaps downwards (e.g. for a gap from 0.5 to 1% the building height, the factors quoted for gaps above 1% of the height apply).

For 5-b (effect of height difference), if the gap is from 0.5 to 1% of the building height and there is a difference of 2 to 4 storeys between the two buildings, the penalty factor is 0.9. If the gap is less than 0.5% of building height and the difference in number of storeys is from 2 to 4, or if the gap is from 0.5 to 1% of the building height and the difference in storeys is 5 or more, the penalty factor is 0.7. If the gap is less than 0.5% of building height and there is a difference of more than 5 storeys, the penalty factor is 0.4.

The lesser of the two values resulting from 5-a and 5-b is taken as the penalty factor due to pounding.

Penalty factors due to the five possible "critical structural weaknesses" are determined separately for each of the two principal horizontal directions, and multiplied together to yield a single penalty factor to be applied on the SPS-value from the first step of the procedure. The minimum of the two products for these directions is adopted as the final SPS value of the building according to the "initial evaluation procedure". This final value may be increased up to 50%, if the engineer considers that the building possesses

In closing the description of the initial evaluation procedure of the draft NZ Guidelines, it is noted that, although evaluation of the SPS in the first step is rational, the correction of its value in the second step to account for "critical structural weaknesses" is subjective and sometimes excessive. Penalisation of buildings with "critical structural weaknesses" categorised as "severe" or "significant" reflects the common conviction that what makes the

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difference for building performance in strong earthquakes is not the adequacy of global strength and ductility, but the presence or not of irregularities and other vulnerability features, such as those considered as "critical structural weaknesses" in the NZ draft Guidelines. Nonetheless, more often than not, the difference between the values of penalty factors applied to cases with a "severe" or an "insignificant" "critical structural weakness" seems disproportionate. This is, for example, the case of wings in a non-rectangular plan (T-, L-, U- or E-shaped) with aspect ratio above 3 or below 2; or the cases of aspect ratio in plan above 5 or below 4 and of eccentricity between centres of mass and stiffness above 50% or below 40% of the parallel plan dimension; or the cases of differences in mass or lateral stiffness between adjacent storeys above 150% or below 100%; etc. It is noteworthy that the penalty factors provided, through the modification factor S_D for structural configuration, by the 1977/1990 Japanese Guidelines for the same structural deficiencies are much closer to 1.0 (this does not mean, though, that the values in the Japanese Guidelines are much more realistic than those in the NZ draft Guidelines). What is more important is that a single "severe" "critical structural weaknesses" may be enough for characterisation of a building as seismically inadequate, without this characterisation been confirmed later by the detailed assessment to follow.

3.2.3.3 The detailed assessment procedure

(1) The seismic action

As far as RC buildings are concerned, the detailed assessment procedure in the 2002 draft of the NZ Guidelines differs little from the one described in the 1996 draft. A major difference is that a complete procedure is now given for the determination of the seismic action for use in the assessment – and in the retrofit design – that allows for considerable flexibility in setting the (remaining) lifetime of the building and selecting the target exceedance probability of the "design seismic action" during that lifetime. The ratio of these two parameters gives the mean return period of the "design seismic action" (e.g. a mean return period of 500 yrs. for a 10% exceedance probability in 50 yrs.). Although a lifetime of 50 yrs. is recommended as the standard (implying an unlimited life, as for new buildings), shorter lifetimes are conceivable, under the condition that the building will be demolished at the expiration of the lifetime considered in the assessment! Values are given for the exceedance probability, other than the standard of 10% adopted for ordinary buildings, depending on building occupancy and its importance for loss of life and for the post-earthquake period. Finally, the factor by which the elastic response spectrum with a 500yr return period should be multiplied is given, as function of the return period (for return periods from 2 to 2000 yrs.). This factor ("importance factor" in European terminology) depends on the seismic hazard environment and should be developed for the particular region (if not site) of interest.

The elastic response spectrum is given for a value of damping ξ=5% and the correction factor of the ENV version of Eurocode 8 is adopted for $\xi \neq 5\%$ (elastic spectrum equal to the 5%-damped one times $\sqrt{7/(\xi+2)}$. For the inelastic spectrum, the equal displacement approximation is adopted, meaning that the inelastic displacement spectrum is taken the same as the elastic one and the inelastic acceleration spectrum is taken equal to the elastic divided by the displacement ductility factor, μ.

Due to various sources of overstrength and overcapacity not accounted for in usual analysis and assessment procedures (material overstrengths, contribution of non-structural elements, energy dissipation in diaphragms and in secondary elements, force redistribution to adjacent members upon local member failure, effect of inelastic action on global stiffness and damping, radiation damping, etc.), analysis and assessment should be for only two-thirds of the seismic action corresponding to the return period of interest. This reduction of the action by one-third should not be applied, if most of the aforementioned sources of overstrength and overcapacity are accounted for, through more advanced modelling and analysis (e.g. nonlinear time-history analysis).

The reduction of the seismic action by one-third due to overstrength, etc., according to the previous paragraph is additional to the afore-mentioned reduction of the action by two-thirds

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for assessment and by one-third for retrofit, for reasons of relaxation of performance requirements for existing buildings.

(2) Analysis and assessment procedures

The 2002 draft of the NZ Guidelines quotes five types of analysis procedures: Linear (equivalent) static, linear response spectrum (modal), nonlinear static (pushover), nonlinear dynamic (time-history) and "simple lateral mechanism" analysis.

The two linear procedures are well-known to the average engineer with some experience in seismic design, but are considered possibly very inaccurate for strongly nonlinear response and/or irregular buildings. So, they are restricted to the case of elastic response, or, alternatively, up to a member ductility demand of 2, provided that there are no in- or out-of-plane discontinuities in the lateral-force-resisting system, or significant irregularities of strength vertically (weak storey) or in plan (torsional unbalance). Moreover, linear (equivalent) static analysis is restricted to buildings of up to 30m, with an orthogonal lateral-force resisting system and without significant irregularities of mass and lateral stiffness in elevation or of torsional stiffness in plan (cf. definitions of "significant" "critical structural weaknesses" of these types for the initial evaluation procedure).

Pushover analysis is recommended only if the fundamental period is less than 1 sec and the (SRSS) base shear due to all modes accounting for at least 90% of total mass does not exceed 1.3 times that due to the 1st mode alone. The same restriction applies to the use of the "simple lateral mechanism" analysis, plus the additional requirement of no significant irregularity of torsional stiffness. Pushover analysis requires the use of a (simple) nonlinear force-deformation model for every location in the structure where formation of a plastic hinge or brittle (shear) failure is possible. It may be performed with an inverted triangular or uniform distribution of monotonically and proportionally increasing lateral loads, with simultaneous action of gravity loads. The NZ draft Guidelines recommend continuous updating of the distribution of lateral forces in this analysis, to follow the instantaneous pattern of (inelastic) lateral displacements of the structure. Final results may not be too different, though, from those obtained from an inverted triangular heightwise pattern of lateral forces in 2D, or from a 1st- mode one for torsionally unbalanced structures analysed in 3D.

The "simple lateral mechanism" analysis is the major novelty in the NZ draft Guidelines. It is a manual procedure to determine the likely collapse mechanism (with the possibility of flexural and/or shear hinging) and its lateral strength and displacement capacity. It involves reduction of the building to a SDOF system and approximate estimation of its yield displacement and – therefrom – its elastic stiffness. It has strong similarities with pushover analysis, but cannot trace the development of plastic hinging and inelasticity in the system, nor can it provide accurate estimates of its ultimate deformation capacity. For RC buildings, it constitutes also the main focus of the detailed assessment procedure, as detailed guidance for assessment of RC buildings on the basis of the results of the other methods of analysis is not given.

For RC buildings the NZ draft Guidelines include a force-based assessment procedure on the basis of a "simple lateral mechanism" analysis, as well as a displacement-based approach. The force-based version is more familiar to engineers. In the following, the steps of force-based assessment, on the basis of a "simple lateral mechanism" analysis, are described. Some steps are common with the displacement-based approach.

(3) Step 1: Capacities of members and joints; likely failure modes and location of plastic hinges.

Step 1 in both force-based and displacement-based assessment comprises calculation of the flexural and shear capacities of members and joints and identification of the likely failure modes and locations of plastic hinges. For beams, columns and joints, this may be done as described in Section 3.2.2.5(3) and (4) for level 3 of the Japan BDPA guidelines. In shear walls, flexural and shear capacities, M_u and V_u, are determined for each storey. It is decided then which one of these two capacities controls, on the basis of the value of the shear span, L_s, resulting for that storey of the wall under lateral loading with an inverted triangular distribution: if $M_u < M_{Vu}=V_uL_s$, then flexure controls; shear controls if $M_u > M_{Vu}=V_uL_s$.

In addition to identifying the likely failure mode of members and joints, the potential for soft-storey development is also examined at each storey. In frames and dual systems

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dominated by frames, this possibility is examined through the «sway-potential» index: $\Sigma(\Sigma M_{Rb})/\Sigma(\Sigma M_{Rc})$, first introduced by Priestley and Calvi (1991). The outer sums in this index refer to all top nodes in the storey of interest. ΣM_{Rb} is the sum of moment capacities at the end sections of the beams framing in a joint, $M_{Rb1}^{-}+M_{Rb2}^{+}$ or $M_{Rb1}^{+}+M_{Rb2}^{-}$ (the value of M_{Rb} may be controlled by the associated beam shear strength, V_u, if the value of $V_u L_s$ is less than the flexural strength, M_u, of the beam end section); similarly for ΣM_{Rc} (i.e. for $M_{Rc1}^{-}+M_{Rc2}^{+}$ or $M_{Rc1}^{+}+M_{Rc2}^{-}$) at the column sections above and below the joint. The concept underlying this index is that for a soft-storey to develop, plastic hinges need to be formed not just in few, but in most columns of the storey. In the NZ draft Guidelines the limit of $\Sigma(\Sigma M_{Rb})/\Sigma(\Sigma M_{Rc})$ for soft-storey formation is conservatively taken equal to 0.85 instead of 1.0.

An important end result of Step 1 is the shear resistance V_{Ri} at each storey of every vertical element.

(4) Step 2: Ultimate base shear

In Step 2 of the force-based approach, the ultimate base shear is estimated in each of the two principal horizontal directions for a postulated inverted triangular heightwise distribution of lateral forces. Three different approaches are suggested for this estimation: a) a conservative one based on linear static analysis, with the ultimate base shear taken as that at first yielding anywhere in the structure; b) an (unconservative) limit analysis of the most likely (column or beam side-sway) failure mechanism identified in Step 1; or c) a more accurate incremental nonlinear static (pushover) analysis.

In the first approach a linear elastic analysis under lateral storey loads with inverted triangular distribution and a base shear of unity, may be employed to determine the value of the seismic base shear at first attainment of a member flexural or shear capacity anywhere in the structure, with simultaneous action of the gravity loads. This value is a lower limit to the ultimate base shear.

In the second approach the shear resistances, V_{Ri}, of the individual vertical elements determined in Step 1 are summed up to provide an estimate of the storey ultimate shear, V_R. It is determined then at which storey this ultimate shear is first attained, under lateral loads with inverted triangular heightwise distribution. The corresponding value of the base shear is an upper limit to the ultimate base shear.

The pushover analysis of the third approach requires the use of a (simple) nonlinear force-deformation model for every location in the structure where formation of a plastic hinge or brittle (shear) failure is possible. This model may be linear-elastic until the corresponding force capacity is reached, with almost perfectly plastic behaviour thereafter for plastic hinging in members with medium-high shear ratio (M/Vh>2), or with shedding of the load in members or joints failing in shear. The ultimate base shear is identified from the peak of the base shear – top displacement diagramme and is typically lower than the upper limit estimated through the second approach. The base shear at the point where this diagramme first departs from linearity corresponds to first plastic hinge formation or failure in the structure and coincides with the outcome of the first approach.

The third approach requires more advanced computational tools and significantly more effort than the other two. Nonetheless, its results are more accurate and reliable. Moreover it can take into account the effects of torsion due to eccentricities in plan, provided that it is performed in 3D under lateral loads with a planwise distribution in each storey which is representative of torsional phenomena.

Unless the structure is symmetric with respect to a plane perpendicular to the direction of the seismic loading, in any of the three approaches the ultimate base shear should be estimated for both senses of action (positive or negative). The lower of the two so-determined values should be adopted.

The ultimate base shear may be divided by the total weight of the building to give the corresponding base shear coefficient (in g's).

(5) Step 3: Fundamental translational period of elastic structure

In Step 3 the fundamental translational period of the elastic structure in each one of the two principal horizontal directions, T_{el}, is computed, considering the members as cracked. The NZ Guidelines refer for this purpose to the NZ code for seismic design of new structures,