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Beyond Ductility: The Quest Goes On

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ABSTRACT

Although the precepts of capacity design and detailing for ductile performance are now well established, the end-user community is now demanding more in terms of predictable performance with an expectation that structures should survive earthquakes with minimal and preferably no damage. The paper first explains the shortcomings of present designs from a probabilistic fragility point-of-view, and then goes on to explain how performance can be improved by making a paradigm shift. This paper examines the emerging quest where structural engineering researchers are investigating design alternatives that strive for damage avoidance.

BIOGRAPHICAL NOTE

Professor John B. Mander holds the University of Canterbury's Chair of Structural Engineering. Prior to returning to Canterbury in 2000 to take up the Chair vacated by Prof. Robert Park, from 1988 to 2000 Mander was on the faculty of the State University of New York at Buffalo. There, in addition to teaching in the Department of Civil, Structural and Environmental Engineering he also conducted research on concrete, steel and timber structures. Much of his research was working on contracts to the US Federal Highway Administration in association with the Multidisciplinary Center for Earthquake Engineering Research. Under the supervision of Dr M.J.N. Priestley and Prof. R. Park, Mander completed his PhD in 1984, this was followed by four years in practice with the former New Zealand Railways where he worked on various projects including the electrification of the North Island Main Trunk.

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1 INTRODUCTION

The 1970's saw the widespread emergence of contemporary earthquake engineering. While many investigators, particularly in the United States, concentrated on issues concerned with seismic analysis, it was the pragmatism of the New Zealand structural engineering community that led to a focus on design for ductility. This is attributed to the significant amount of public works being undertaken at that time along with the immediate need to develop dependable seismic resistant solutions.

At the forefront of these events were Profs. Park and Paulay working on reinforced concrete structures. Along with their graduate students, Park and Paulay went on a "quest for ductility". Much of their early successes are summarized in their well-known tome "Reinforced Concrete Structures" (Park and Paulay, 1975). Not only did this book take the research and practitioner world by storm, it immediately became the "required reading" for all University of Canterbury civil engineering students in both their 2nd and 3rd professional years. Thus from 1975, a generation of civil engineers have been conversant with the principles of "ductile design" or "designing for ductility" and "capacity design". Many of these concepts also quickly found their way into the design codes of the day, particularly the concrete design specification NZS 3101 (1984), which at the time was heralded as a world leader.

In spite of this illustrious history, the quest for ductility still goes on. Some work continues to examine components such as beams, columns, walls and cruciform subassemblies that examine performance in either one or two dimensions. However, the emphasis of much recent work at both the University of Auckland and the University of Canterbury has focused on concerns related to the three-dimensional performance of complete building systems through the testing of large super-assemblages. In spite of developing robust ductile frames in accordance with the principles of capacity design and ductile detailing espoused originally in Park and Paulay (1975), the performance of floor-frame interaction still leaves a lot to be desired, as discussed in Matthews et al. (2003).

Moreover, recent earthquakes, such as the 1989 Loma Prieta, 1994 Northridge and 1995 Kobe events demonstrate the societal outrage associated with the loss of post-earthquake serviceability of many buildings and facilities, that although earthquake resistant suffered significant damage. In this paper it is contended that there is now a need to move beyond notions of ductility. First, an examination will be made of the fragility of existing structures that have been designed in accordance with the principles of capacity design and detailed for ductility. The paper will then go on to describe new seismic design paradigms that attempt to overcome one of the significant shortcomings of the large majority of present ductile structures—that ductile structures sustain damage in earthquakes, and although life-safety may be maintained, continued amenity may not.

2 SEISMIC FRAGILITY THEORY FOR STRUCTURAL SYSTEMS

A neat explanation of the expected seismic performance of structural systems can be provided using *fragility curves*. In developing fragility curves for a given type of structure, the theoretical probabilistic formulation takes into account: (*i*) the expected site-specific response characteristics; (*ii*) the inelastic strength and deformation capacity of the structure; (*iii*) damage limit states; and (iv) randomness of ground motion response spectral demand, and uncertainties in modelling structural capacity.

For a given structure, it is possible to predict, deterministically, the level of ground shaking necessary to achieve a target level of response and/or damage state. In addition to assuming material properties and certain other structural attributes that affect the overall *capacity* of a structure, such a deterministic assessment requires that certain assumptions be made about the ground motion and site conditions—both are factors that affect seismic *demand*. Naturally, values of these parameters are not exact—they invariably have a measure of both randomness and uncertainty associated with them. Figure 1 depicts how the inherent uncertainty and randomness of structural capacity versus ground motion demand can be used to establish fragility curves. Figure 1(a) shows a graph of acceleration-displacement spectra for the ground motion. Superimposed with this curve is the pushover capacity of a structure. In a deterministic analysis, the intersection of the two curves gives the expected performance point. However, if probability distributions are drawn over both the capacity and demand curves, these distributions indicate the associated uncertainty and randomness of performance. From this graph it is evident there is a wide range of possible performance outcomes. Although there is no unique outcome, a deterministic approach can be used to assess the expected (median) response. Randomness and uncertainty can then be treated through an appropriate probability distribution.

If structural capacity and seismic demand are random variables that approximately conform to either a normal or lognormal distribution, then the *central limit theorem* requires the composite performance outcome be distributed log-normally. Therefore, the probabilistic distribution expressed in the form of a so-called *fragility curve* is given by a lognormal cumulative probability density function. Fortunately, only two parameters are needed to define such a curve: a median (the 50th percentile); and a normalized logarithmic standard deviation. Figure 1(b) presents a generic form of a fragility curve. This is a normalised curve and is not only indicative but also relevant for most structural systems and ground motion characteristics. The cumulative probability function is

$$F\left(S_a\right)=\Phi\left[\frac{1}{\beta_c}\ln\left(\frac{S_a}{A_i}\right)\right] \qquad (1)$$

where Φ = standard log-normal cumulative distribution function; S_a= the spectral acceleration amplitude (for a period of T = *1 sec.*); A_i = the median (or expected value) spectral acceleration necessary to cause the i^{th} damage state to occur; and β_c= normalized composite log-normal standard deviation which incorporates aspects of uncertainty and randomness for both capacity and demand.

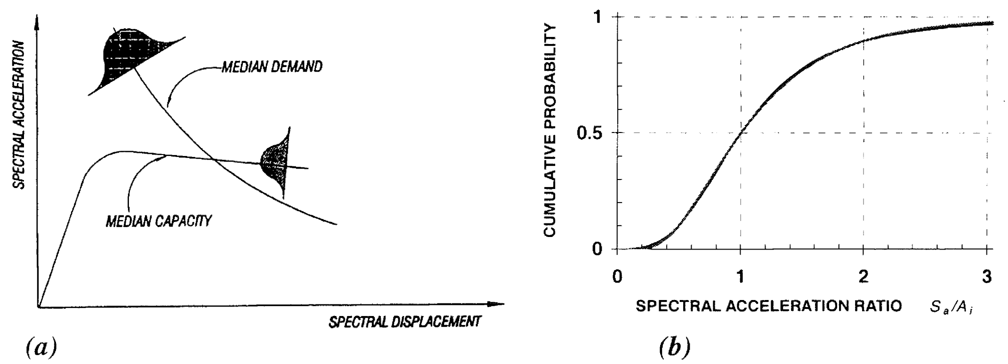


Figure 1. Probabilistic Definition of Uncertainty and Randomness in Establishing Fragility Curves for a Seismic Vulnerability Analysis: (a) Capacity-Demand Acceleration-Displacement Spectra Showing Randomness and/or Uncertainty in Structural Behavior and Ground Motion Response: and (b) Normalized Fragility Curve That Accounts For Uncertainty and Randomness in Both Demand and Capacity.

The latter parameter, sometimes loosely referred to as either the coefficient of variation or the coefficient of dispersion, has been calibrated by Pekcan (1998), Dutta and Mander (1998) and Dutta (1999) from a theoretical perspective. In addition, β_c was validated by Mander and Basöz (1999) against experiential fragility curves obtained from data gathered from the 1994 Northridge and 1989 Loma Prieta earthquakes by Basöz and Kiremidjian (1996, 1998). Based on these previous investigations $\beta_c = 0.6$ is recommended. This value is used in the fragility curve plotted in Figure 1(b).

With the shape of the probability distribution set, the only step that remains in the analysis is assessment of the expected value (50 percentile or median value) of the earthquake needed to provide a required level of performance. For this purpose, nonlinear static procedures that use a capacity or "pushover" analysis suffice. Such a formulation will now be presented for flexible structures with moderately long periods (say T>0.5 sec) to illustrate the simplicity of the approach. Implicit in this approach for "long" period structures is Newark's "equal displacement" principle that elastic displacement demands are similar to the nonlinear response displacements. Note that this assumption needs modification for short-period effects.

Consider the $1/T$ portion of the design spectra, which can be stated, in code-like simplicity as

$$C_d = R\,C_c = \frac{SA}{T_n} \tag{2}$$

in which $C_d =$ normalized elastic base shear demand; $C_c =$ provided normalized base shear strength (capacity); $R =$ force (strength) reduction factor where $R = C_d / C_c$; $A =$ peak ground acceleration (normalized with respect to g); $S =$ soil type factor (note the product $SA =$ the spectral acceleration for a 1.0 sec. period); and T_n = natural (elastic) period of vibration. The latter defined in terms of the structural yield strength and displacement follows:

$$T_n = 2\pi\sqrt{\frac{m_e}{K_e}} = 2\pi\sqrt{\frac{W/g}{F_y/\Delta_y}} = 2\pi\sqrt{\frac{\Delta_y}{C_c\,g}} \tag{3}$$

in which $m_e =$ effective seismic mass of the structure; $W =$ seismic structure weight; $F_y =$ yield strength (base shear force) of the structure; $\Delta_y =$ yield displacement of the structure at the seismic centre of mass; and $g =$ gravitational acceleration. Note the normalized base shear (capacity) is given by $C_c = F_y/W$.

The displacement ductility factor defined as $\mu = \Delta_{max} / \Delta_y$ when coupled with Newmark's equal displacement principle such that $R = \mu$, when substituted into Eqn. (2) along with Eqn. (3) may be rearranged to give a so-called *"expected spectral capacity"* for the i^{th} damage state as follows

$$(SA)_i = 2\pi\sqrt{\frac{C_c\,\Delta_y}{g}}\,\mu_i \tag{4}$$

Here it should be emphasised that both $C_c =$ and $\Delta_y =$ are deterministic values based on expected (or average measured) material properties of an as-built structure—not to be confused with nominal values specified by the original design.

2.1 Fragility Curve Validation for US Bridges

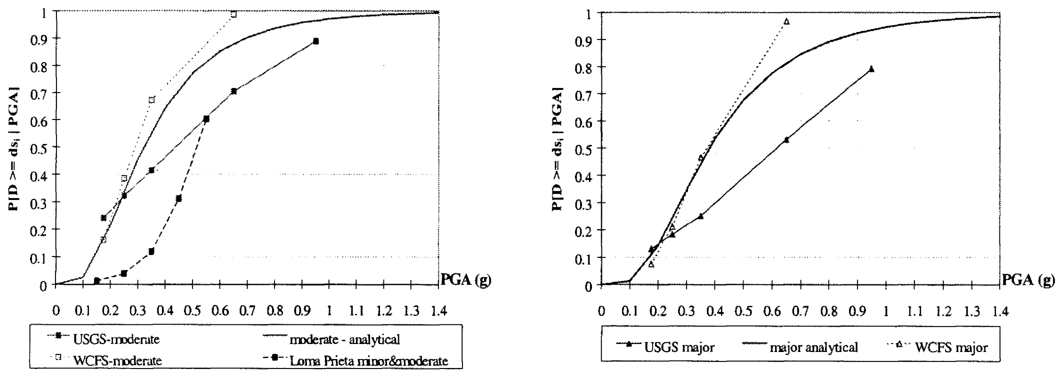
Mander and Basoz (1999) developed fragility curves for several common types of "standard bridge" construction used in the United States. The structural attributes used in formulating fragility curves include number of spans, structure type, year built, skew and structure length. In spite of the wealth of information contained in the National Bridge Inventory (NBI), there is insufficient detail on structural characteristics of each bridge for a detailed analysis to be performed when deriving fragility curves. Therefore, a bridge-specific fragility curve is obtained by modifying the fragility curve of a "standard bridge". A "standard bridge" is assumed as a "long" structure with no appreciable 3D effects present.

Median values of the peak one-second spectral acceleration for five different damage states are assessed using an algorithm that is based on a pushover analysis, as indicated in Figure 1 (a). This displacement-based nonlinear static analysis procedure assumes a standard AASHTO-like earthquake response spectrum shape, which can be adjusted later to account for site-specific spectral ordinates and/or soil types. Table 1 lists the five damage states and their associated performance outcomes.

Table 1. Definition of Damage States and Performance Outcomes.

Damage State	Descriptor for degree of damage	Post-earthquake Utility of structure	Repairs required	Outage Expected	Expected Ductility Factor
1	None (pre-yield)	Normal	None	--	1
2	Minor/slight	Slight damage	Inspect, adjust, patch	< 3 days	2
3	Moderate	Repairable damage	Repair components	< 3 weeks	3
4	Major/Extensive	Irreparable damage	Rebuild components	< 3 months	4
5	Complete/ collapse	Irreparable damage	Rebuild structure	> 3 months	6

Mander and Basoz (1999) compared analytically predicted fragility curves for various different bridge types against empirically derived fragility curves from data gathered for highway bridges damaged in the 1994 Northridge and 1989 Loma Prieta earthquakes. Good agreement was obtained between the theory and field observations for various classes of bridges including: simple span bridges with monolithic and non-monolithic abutments; multi-column bridges with discontinuous spans; multiple span bridges with pier walls and non-monolithic abutments; and multiple span bridges with discontinuous decks with single column per bent and non-monolithic abutments. A sample of the validation for one common class of Californian bridges is given in Figure 2. In spite of the large degree of uncertainty in defining both bridge damage and the spatial distribution of ground motion in the field, it will be noted that both the median damage values [50 percentile] and the shape (for $\beta_c = 0.6$) of the fragility curves agree rather well.



(a) *(b)*

Figure 2. ***Comparison of analytical and empirical fragility curves for discontinuous multi-span bridges with single column bents and non-monolithic abutments for: (a) moderate damage, DS_3; and (b) major damage, DS_4.***

2.2 The Fragility of New Zealand Buildings

Building structures designed and constructed in New Zealand in accordance to the prevailing standards will still have a measure of fragility. To quantify this in an indicative sense, consider the set of fragility curves for a normal reinforced (or precast) concrete building shown in Figure 3(a). Such buildings, based on standard ductile detailing, are generally designed for a ductility factor of $\mu = 4$ for a 10% in 50 year ground motion. There is only a 10% chance of reaching the design limit state ($\mu = 4$) for a standard design ground motion (10% in 50 years, or a return period of approximately 500 years). This is because of the presence of under-capacity (φ) factors in the design process and the fact that the real material properties are stronger (on average) than specified properties. This outcome is also in spite of having a so-called *Sp*-factor (typically *Sp* = *2/3*) inherent in the design to remove some of the conservatism (and over-strength) of the design process. A corollary of the probability of 10% reaching the design limit state is for a structural designer to claim 90 percent confidence in the structure performing in the desired way for the standard design ground motion. Recently, the simplified fragility curve approach has had extensive verification through simulation by Martinez (2003).

Although there is a 90% confidence in adequate performance for the design basis earthquake (DBE = 10% in 50 years), there is still roughly a 3% chance that there will be a structural collapse. Moreover, some 75% of structures designed to the DBE will be expected to sustain some form of structural damage. In addition, earthquakes larger than the DBE are theoretically possible. International best practice is rapidly moving toward the 2% in 50 year event (approximately 2,500 year return period) as the maximum considered earthquake (MCE) for the purposes of survivability.

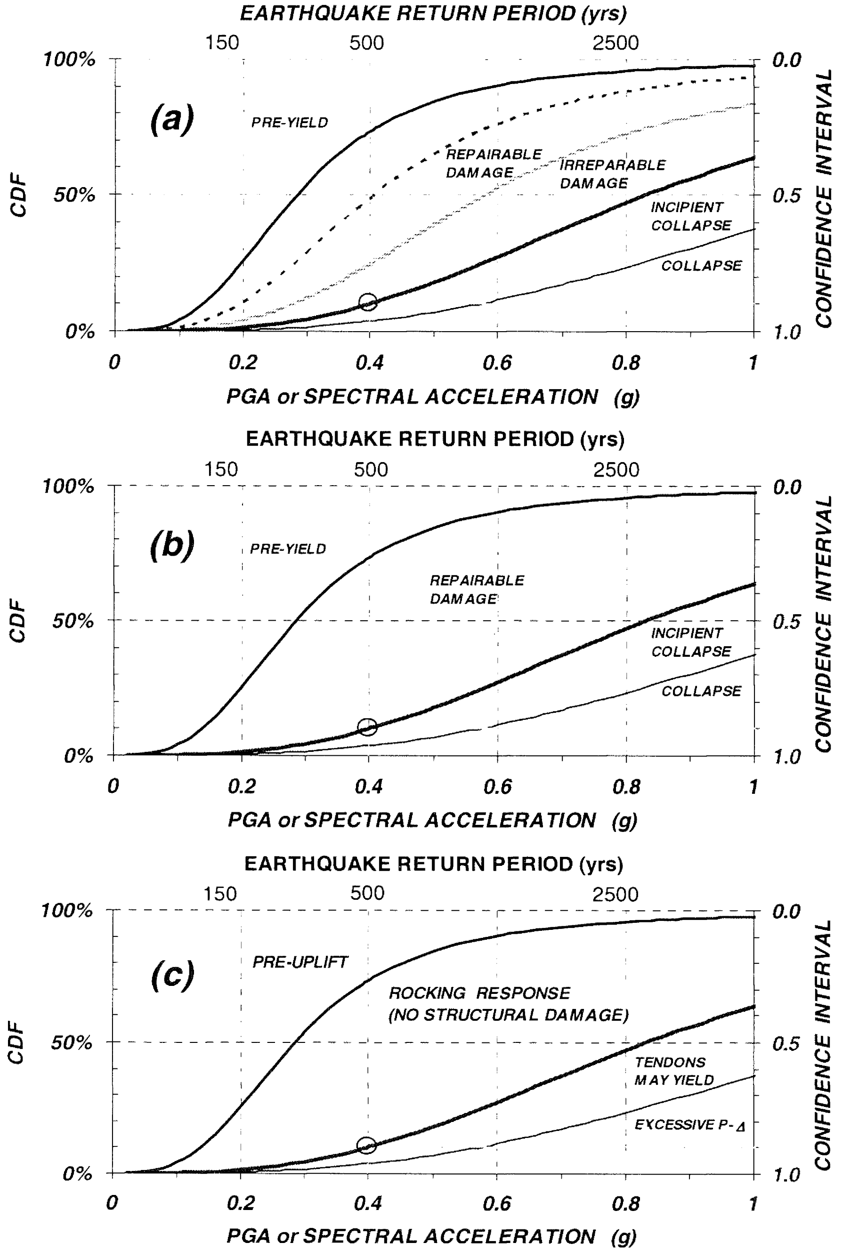


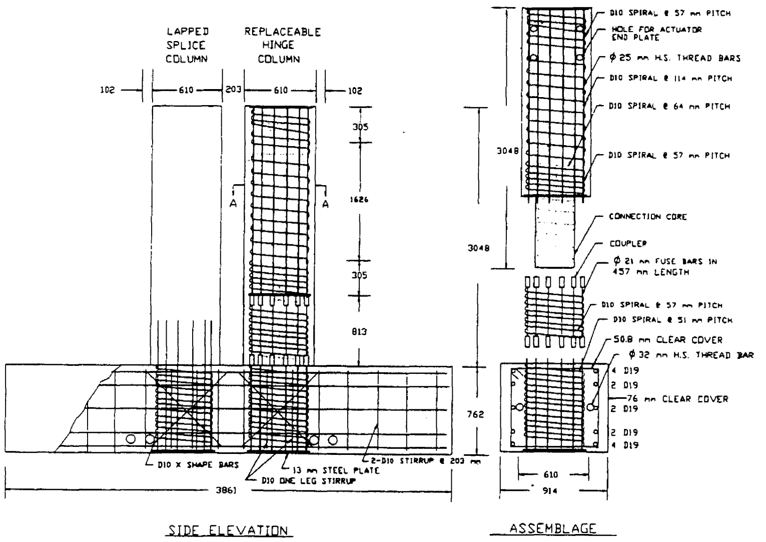
Figure 3. Fragility curves for different design philosophies: (a) Ductile design; (b) design for Control and Repairability of Damage; and (c) Damage Avoidance Design.

If, however, a structure has been designed for a lesser event, the DBE, then a greater level of damage may be expected for the MCE. From Figure 3(a) it follows that there is a confidence interval of only 75% against collapse prevention, and a probability of 75% that the structure will be irreparably damaged. Such a high probability of failure is untenable, in terms of both life safety and continued amenity of the structure.

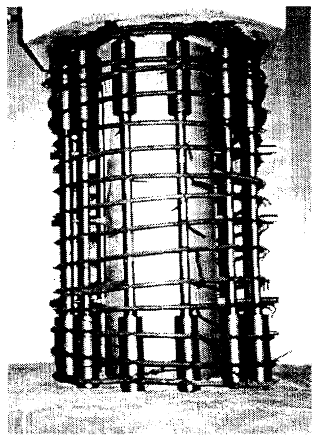
The question remains: Without dramatically increasing the cost of construction by increasing structural strength to offset poor performance, what design and construction measures can be taken to improve the current state of affairs?

3 WHERE TO BEYOND DUCTILITY?

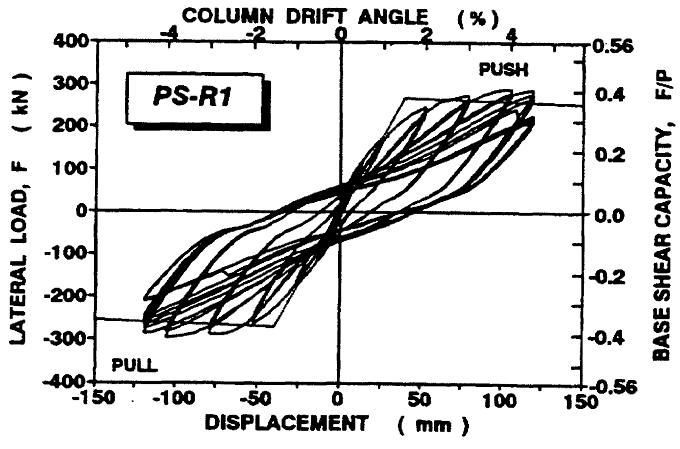
It is evident that the next step in earthquake resistant design is to develop structural systems that are at least repairable after an earthquake, or preferably damage resistant. This paper will describe two seismic design paradigms referred to as: (i) Control and repairability of damage (CARD); and (ii) Damage avoidance design (DAD). These emerging design paradigms described in what follows, are summarized in Figures 4-8.



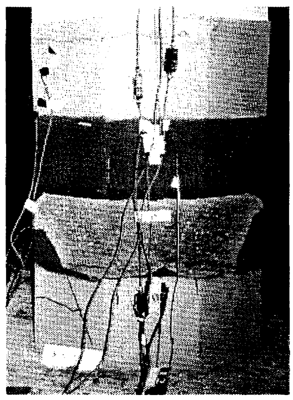
(a) Details of specimen with repairable hinge zone



(b) Replaceable fuse bars In plastic hinge zone



(c) Hysteretic response of column



(d) Damaged hinge zone at conclusion of testing and prior to repair

Figure 4. Results of a precast concrete column with a replaceable plastic hinge zone.

3.1 Control and Repairability of Damage (CARD)

Based on research work sponsored by the US Federal Highway Administration (FHWA), Mander and Cheng (1999) proposed a modification to the usual ductile design philosophy. By installing replaceable fuse bars in the plastic hinge zone of structures, it is possible to repair damaged components after an earthquake. Figure 4 presents a bridge structure showing the application of the concept for a bridge pier.

Proof of concepts experiments were conducted by Cheng and Mander (1997) on one-third scale specimens, as well as near full size columns. In the case of the former, one specimen was repaired and retested some 19 times. At all times damage was restricted to the hinge zone that could be repaired. Figure 4 presents results for a near full size specimen that has been constructed in the form of a precast concrete column. The connection may be defined as a "wet joint". Within the hinge zone the column has a smaller diameter stub (or "connection core" as seen in Fig. 4(a)), which is used during construction. The columns are seated, plumbed, and then the "fuse bars" are fitted by joining into couplers. The "fuse" portion of these bars locates the plastic hinge zone (see Fig. 4(b)) and has an optimal length of about two-thirds of the column diameter. Figure 4(c) presents the hysteretic response following one of the repairs. The seismic behaviour, in terms of energy dissipation, strength, and fatigue-life was essentially identical to the original construction. The performance at the end of testing, prior to a repair, is shown in Figure 4(d).

Dutta *et al* (1999) have developed this approach further to embrace seismic retrofitting of structures.

Figure 3(b) presents fragility curves for this class of construction. Although the curves for damage states 4 and 5 are fundamentally the same as for regular ductile design, there is a decided improvement in the post-earthquake amenity of the structure due to its repairability capability. All structures that meet the design objective (or better) for the DBE, that is 90%, can theoretically be repaired and returned to reuse.

One might concede that in spite of the improved performance, this system still has the disadvantage of requiring post-earthquake repairs. This may be especially difficult if there is a moderate level of residual displacement. If this becomes an important design and performance issue from the owners' standpoint, then the following dry-joint re-centering system maybe adopted as the preferred alternative.

3.2 Damage Avoidance Design (DAD)

Motivated by the need to provide immediate post-earthquake serviceability for the nations major transportation arteries, the US Federal Highway Administration (FHWA) has supported research work on developing structures that avoid earthquake-induced damage. Mander and Cheng (1997) developed a dry-jointed precast concrete pier system for bridge structures. Being precast, the system has the advantage of rapid construction—this is an important by-product of the research—the construction approach is also potentially attractive for post-earthquake response and recovery needs. Bridge piers have historically been cast in-situ, and tend to be one of the activities that stall on-site rapid construction progress. The proposed dry-jointed system can be off-site or factory precast, with only a few days needed to erect piers at most sites.

Figure 5 presents a bridge structure showing the application of the dry-jointed precast concrete concept for a pier. The columns in the pier are post-tensioned prestressed between the cap beam and pile cap (or footing). Under lateral loading the columns, being dry jointed, are free to open and close. Thus a rocking structural system is formed. At the ends of each column, resistance moments are provided by the eccentric nature of the axial load path that include the combined effects of gravity, prestress and dissipater forces (if any).

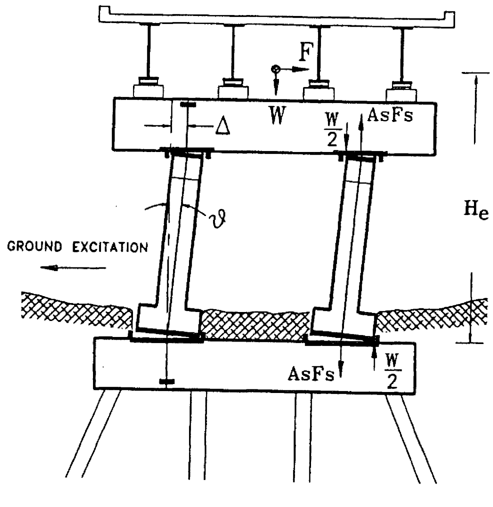


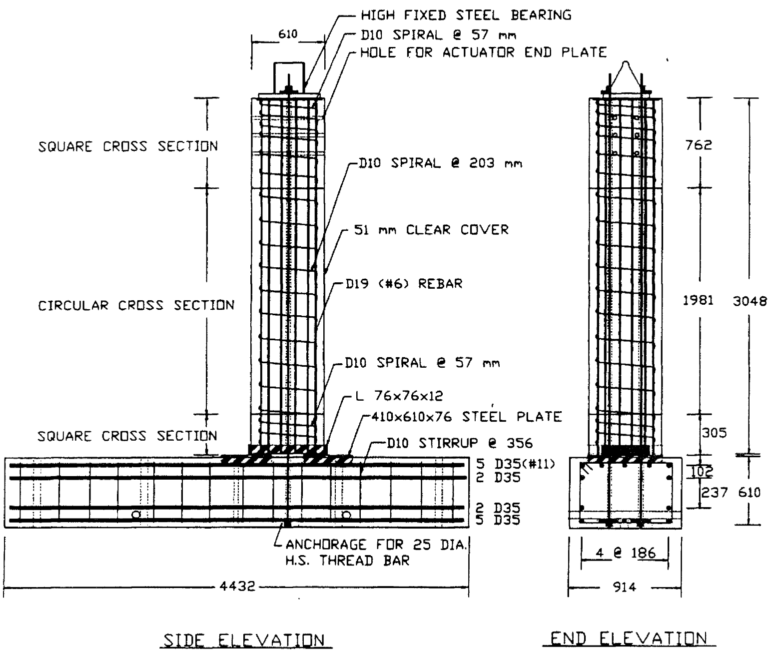
Figure 5. Precast concrete bridge pier with armored dry-jointed rocking connections.

Figure 6 presents construction and performance details of a rocking column experiment. Detailing of the armored column-end dry-joint connections for a near full-size bridge column specimen are presented in Figure 6(a). The longitudinal reinforcing bars, within the column, are welded to a thick end plate. The end plate is seated on steel plates that are cast into the foundation/cap beam. When the column rocks, the steel end plate pivots on the foundation seating plate. Although there are very high contact stresses, no damage to the concrete occurs as is evident in the photograph of Figure 6(b)—behaviour is essentially bi-linear elastic. Figure 6(c) shows the performance of the rocking column specimen under reversed cyclic loading. As the displacement amplitude increases, there is a tendency to yield the longitudinal column prestressing thread-bars. Either this could be overcome by using higher strength prestressing strand (thread-bars with a strength of 1,100 MPa were used in the experiments), or the thread-bars can easily be re-stressed after an earthquake. The slight yielding and plastic elongation of the bars is the sole reason for the modest amount of hysteresis shown in Figure 6(c). The photographic enlargement of the column base, shown in Figure 6(d), is at a column drift angle of 4%. For a regular reinforced concrete column at this level of drift, one would expect substantial permanent irreparable damage.

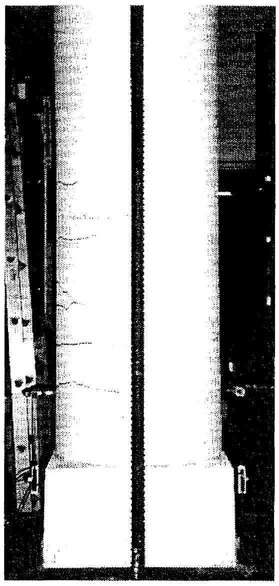
Being mostly elastic, one of the problems with rocking systems is the inherent structural damping is low. Moreover, there is no significant hysteretic damping as in classic ductile structures. The little inherent damping that rocking structures possess is mostly derived from radiation damping which is part of the rocking process. Performance can be markedly improved by the addition of mechanical energy dissipaters. Holden *et al* (2003) conducted experiments on two wall specimens: (a) a conventionally designed ductile reinforced concrete wall; and (b) a rocking wall with steel armored ends that was post-tensioned and possessed supplemental metal yielding energy dissipaters. Following the tests, the condition of the former was irreparable, while the latter remained in good condition at all times—only the dissipaters sustained damage. Research on rocking walls is presently ongoing investigating ways of improving the effectiveness of the dissipaters while keeping them cost effective as well as renewable.

Figure 3(c) presents a set of fragility curves for the class of construction designed in accordance with the principles of damage avoidance. When compared to conventional seismic design and ductile detailing, it is evident that there is a marked improvement in performance. To achieve this, the structural elements need be no stronger than with conventional design. Rather, the detailing needs to be more advanced, along with some unbonded post-tensioning to enable structural re-cantering. According to the fragility curves in Figure 3(c), the designer can be 90 percent confident that the structure will be damage-free for the DBE (10% in 50 years event). This is in contrast to the expectation that some 40 percent of structures may be damaged in the case of conventional design.

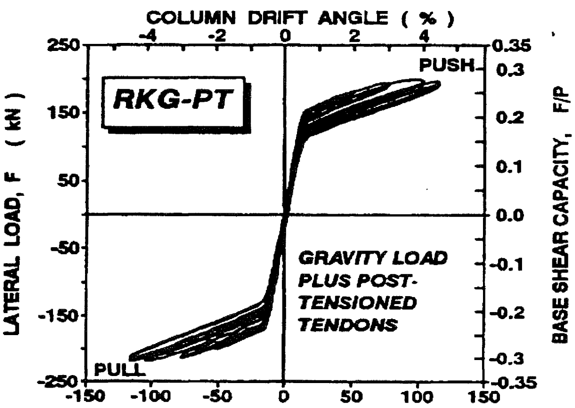
Under the MCE (2% in 50 years event), although some prestressing tendons may yield (but can easily be re-stressed), structural collapse is prevented. The main limiting factor is excessive drift; this may potentially lead to non-structural damage. If this becomes a performance issue for the MCE, as for important structures such as hospitals or manufacturing plants, then the structure may need to be stiffened and more highly damped.



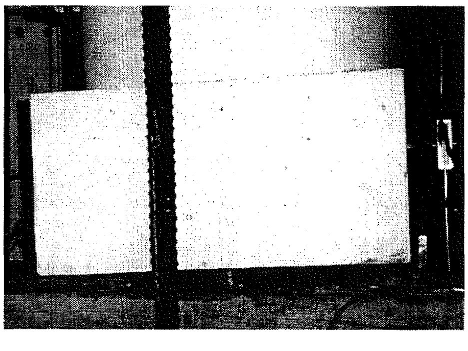
(a) Rocking Column Specimen Dimensions



(b) undamaged specimen at end of testing



(c) Hysteretic response of rocking column



(d) Uplift of base at a column drift of 4%

Figure 6. Experimental results of a precast concrete rocking column.

4 FUTURE WORK

The aforementioned details have been for structures where the axial (gravity) force in the column (or the wall) member has been a significant factor in providing the seismic resistance. The next challenge is to prefect the system whereby horizontal members, such as beams, can be post-tensioned to columns and the dry-jointed connection can be constructed in a straightforward manner. The dry-jointed connections must also possess good shear and torsional resistance characteristics.

An example of a dry-jointed beam-column experimental connection is presented in Figure 7.

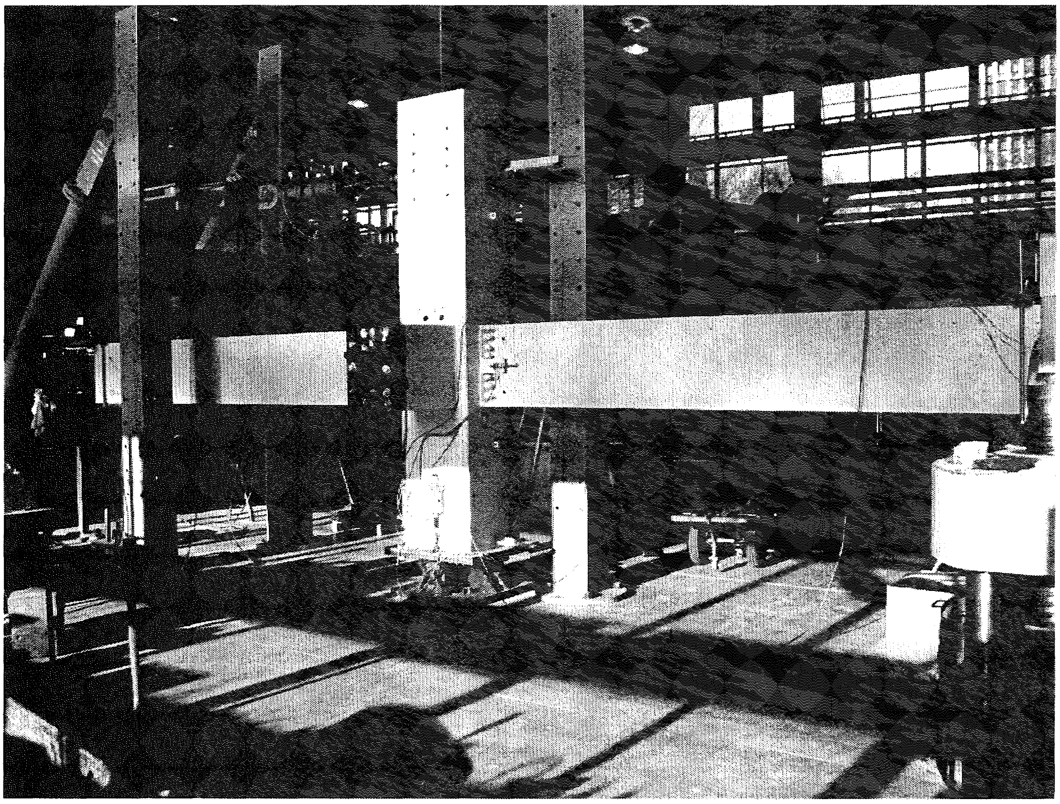


Figure 7. Beam-column subassemblage with armoured beam-end dry-joint connections designed in accordance with the principles of damage avoidance.

5 CONCLUSIONS

The reasons for the emerging quest to move beyond ductility are summarised in the following conclusions:

- The precepts of capacity design and detailing for ductile performance are now well established. Their roots go back to the 1970's where many of the pioneering ideas stemmed from the early work summarised in Park and Paulay (1975). Ductile structural systems have performed well in earthquakes, and in general, the life-safety expectation has been achieved.
- The end-user community is now demanding more in terms of predictable performance with an expectation that structures should survive earthquakes with minimal and preferably no damage.
- From a probabilistic fragility point-of-view, seismic performance can be improved by making a paradigm shift to a damage avoidance design philosophy.
- Considerably more research work needs to be done to bring a completely damage avoidance design philosophy to fruition. Of particular importance will be developing techniques that handle the complex three-dimensional interactions of seismic behaviour of floor, frame, wall and cladding systems.

6 REFERENCES

Basöz, N. and Kiremidjian, A. S., (1996). "Risk Assessment for Highway Transportation Systems", Technical Report No. 118. *John A. Blume Earthquake Engineering Center,* Civil Engineering Department, Stanford University, Stanford, California.

Basöz, N. and Kiremidjian, A. S., (1998). "Evaluation of Bridge Damage Data from the Loma Prieta and Northridge, CA Earthquakes", *MCEER* Technical Report MCEER 98-004, State University of New York at Buffalo, Buffalo, NY.

Cheng, C.-T. and Mander, J.B., (1997). "Seismic Design of Bridge Columns Based on Control and Repairability of Damage", *NCEER*, Technical Report NCEER-97-0013, December 8, 1997.

Dutta, A., (1999). "On Energy-Based Seismic Analysis and Design of Highway Bridges". Ph.D. Dissertation, *Science and Engineering Library, State University of New York at Buffalo,* Buffalo, NY.

Dutta, A., and Mander, J. B., (1998). "Seismic Fragility Analysis of Highway Bridges, INCEDE-MCEER Center-to-Center", *Workshop on Earthquake Engineering Frontiers in Transportation Systems*, June, Tokyo, Japan.

Dutta, A., Mander, J.B. and Kokorina, T., (1999). "Retrofit for Control and Repairability of Damage", *Earthquake Spectra*, Vol. **15**, No. 4, November, pp. 657-679.

Holden, T., Restrepo, J. and Mander J.B., (2003). "Seismic performance of Precast Reinforced and Prestressed Concrete Walls", *Journal of Structural Engineering*, ASCE, Vol. **129** (3), March, pp 286-296.

HAZUS. 1997. Earthquake Loss Estimation Methodology. Technical Manual. *Prepared by the National Institute of Building Sciences for Federal Emergency Management Agency.*

Mander J.B. and Basöz N., (1999). "Seismic Fragility curve Theory for highway bridges in Transportation Lifeline Loss Estimation", in Optimizing Post-Earthquake Lifeline System Reliability, *TCLEE* Monograph No. 16. Eds Elliot and McDonough, pp 31-40.

Mander, J.B. and Cheng, C-T., (1997). "Seismic Resistance of Bridges Based on Damage Avoidance Design", *NCEER*, Technical Report NCEER-97-0014, December 10, 1997.

Mander, J. B. and Cheng, C-T., (1999). "Replaceable Hinge Detailing for Bridge Columns", *American Concrete Institute*, Special Publication SP-187 Seismic Response of Concrete Bridges. July 15.

Matthews, J.G., Bull D.K. and Mander J.B., (2003). "Preliminary Experimental Results and Seismic Performance Implications of Precast Floor Systems with Detailing and Load Path Deficiencies," 2003 Pacific Conference on Earthquake Engineering, *NZSEE,* 13-15 February, Christchurch, NZ. CD ROM paper 077.

NZS3101 (1982). "Code of Practice for the Design of Concrete Structures", *Standards Association of NZ,* Part 1 Code, Part 2 Commentary.

Pekcan, G., (1998). "Design of Seismic Energy Dissipation Systems for Concrete and Steel Structures". Ph.D. Dissertation, *Science and Engineering Library, State University of New York at Buffalo*, Buffalo, NY.

Park, R. and Paulay, T., (1975). Reinforced Concrete Structures, *Wiley*, NY. 769pp.