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EARTHQUAKE RESISTANCE CONSIDERATIONS FOR DEGRADING SYSTEMS

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ABSTRACT

Following the January 17th, 1994 Northridge earthquake, a reconnaissance team was dispatched from New Zealand to observe damage and performance with a view to its application to New Zealand. Of particular interest to BRANZ was the behaviour of low-rise buildings. While damage was widespread within this class of building, collapse was relatively infrequent and in most cases the cause could be reasonably clearly ascertained. An exception was the Meadows Apartment collapse where tragically 13 lives were lost. This building was of particular relevance to New Zealand because the construction of the lateral load resisting system comprised gypsum based plaster board lining panels fastened to timber framing which is commonly used in New Zealand and has been found to perform well.

This paper details an assessment undertaken at BRANZ of the lateral shear load carrying capacity of such systems based on an integrated experimental and computer simulated time history analysis technique. It shows that it is likely the level of load experienced exceeded that prescirbed by the design codes of the day. The performance of an alternative, enhanced gypsum wall board systems commonly used for bracing in New Zealand was also investigated. This demonstrated a much superior structural performance.

KEYWORDS

Timber, Earthquake Design, Degrading Systems, Experimental Verification, Pinched Hysteretic Response, Computer Simulation

INTRODUCTION

Modern, western style timber (2x4) framed buildings are well capable of surviving severe earthquakes without collapse (Shephard *et al.,* 1990). Often significant damage occurs to secondary elements, brittle cladding and lining materials, but collapse is rare. This can be attributed to a several factors, including the intrinsically lighter mass of timber structures (which reduces lateral forces), the common presence of numerous sheet lined wall elements (interconnection of these shares load, making some redundant) and a structural form which can sustain large inelastic deformations without instability or collapse.

In general the performance of timber framed buildings during the 17th January 1994 at Northridge earthquake confirmed these findings (Norton *et al.,* 1994). Damage to this class of building was generally confined to brittle cladding systems (particularly cement based stucco exterior claddings and terracotta roofing tiles), to secondary elements (such as chimneys, masonry fences, etc.) and to internal linings. Such damage was widespread within the epicentral area. Collapses which were observed were usually the result of soft storey side-sway mechanisms developing within basement carparks which were torsionally very irregular. The one exception to the above observed by the NZNSEE reconnaissance team was the Meadows Apartment (see

Figure 1 & Figure 2) where 7 of the 13 three storey timber framed blocks within the complex collapsed in a similar manner. The remaining 6 blocks were severely damaged and on the verge of collapse.

The Meadows Apartment was of particular interest to BRANZ because the lateral load resisting system (gypsum based wall board) was similar to that in widespread use within New Zealand. The collapse mechanisms observed at the Meadows had never been observed during extensive laboratory testing and evaluation programmes undertaken at BRANZ to provide ratings for a variety of wall panels systems, allowed within the NZ standard for timber framed houses (SNZ, 1990). An experimental programme was conducted to replicate the observed collapse mechanism and to use a recently developed advanced computer based evaluation technique to establish the seismic resistance of the building.

THE MEADOWS APARTMENTS, NORTHRIDGE

The three storey timber framed complex was constructed in the early '70s and comprised 13 individual blocks linked together at each floor with landings, access passageways and external stairs. The buildings had a stucco exterior cladding and a near flat bitumen over concrete roof. Each floor had a light concrete topping cast over timber slats supported from timber joists. Of vital significance to its earthquake performance (and to BRANZ) was that the lateral load resistance at each level was primarily provided by (12 mm) gypsum wall board panels with cut-in diagonal timber braces. Although some carparking was provided within each block, there were many accommodation units at ground level. The distribution of these bracing panels was such that several unbroken transverse walls were present between apartments and a central passageway along the length of each block provided internal access to each unit. These were broken only by access doors at approximately 4.8 m centers *(Note: these walls were replicated in the experimental programme described later*). The external walls at the front or rear of each apartment were predominantly range-slider doors and windows which contributed little to the lateral load resisting system.

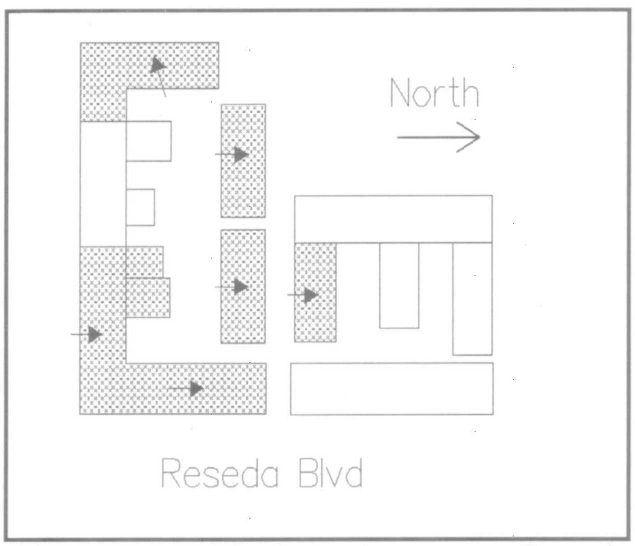


Figure 1. Meadows Apartment: General plan layout Shaded blocks collapsed in direction of arrows

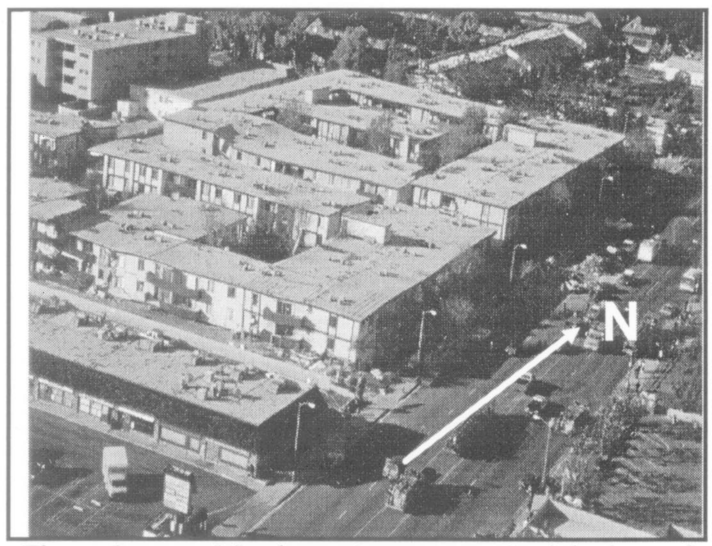


Figure 2. Meadows Apartment: Aerial view of front blocks

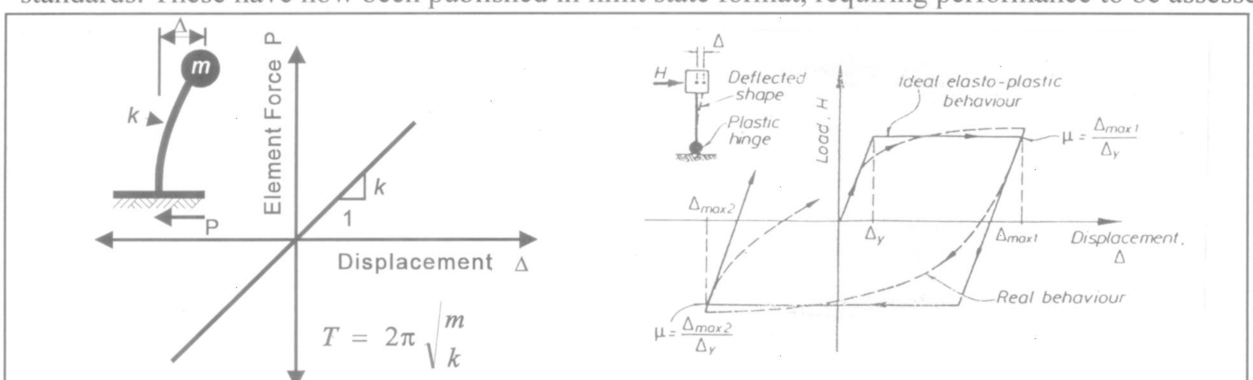
Figure 1 and Figure 2 show the plan layout and the collapse directions (mainly northerly) observed during the site inspection. Even blocks which survived were noted to have experienced very large lateral drift (600 mm) within the ground floor storey height. No seismic separation provisions were observed between blocks and the collapse of some attached transverse blocks was attributed to them being dragged down by adjacent units. The characteristics of the bracing panel collapse was that of nail-head pull-through primarily along either the top or bottom framing member (plate) causing a complete loss of shear resistance. Large interstorey drifts followed, in some cases resulting in P-Δ shake-down collapse (refer Figure 3.). . Cut-in timber diagonal braces failed even within the surviving blocks (refer Figure 4).

Figure 3. Within the 500 mm residual ground floor crawl space following collapse.

Figure 4 Compression brace failure and loss of sheet lining nearing collapse.

THE ENGINEERING IMPLICATIONS

New Zealand has long been aware of the beneficial seismic behaviour of timber structures. Widespread use is made of this abundant natural resource in domestic construction, predominantly as single and two storey timber framed units. The recent introduction of the New Zealand Building Code, or NZBC (NZ Government, 1992), and the inherent performance requirements has resulted in a major revision to most of our structural standards. These have now been published in limit state format, requiring performance to be assessed at both



(a) Elastic

(b) Elastoplastic (Park, 1989)

(c) Slackness (Dean et al., 1986)

(d) Pinched (Stewart, 1987)

Figure 5. Elemental modelling of Inelastically Structural Systems

serviceability limit state (where the onset of damage under moderate more frequent loading conditions), and for ultimate limit state conditions (where collapse under extreme events is to avoided).

Earthquake design is covered by Part 4 of NZS 4203 "General Structural Design Requirements and Design Loadings for Buildings" (SNZ, 1992). Pseudo-static, modal and integrated time history analysis techniques are all permitted, although the pseudo-static approach remains dominant. Elastic response spectra with 5% damping for 3 soil conditions are given along with suites of inelastic response spectra. (derived from an elasto-plastic single degree of freedom oscillator). While the assumed bilinear response is appropriate for many structural materials (eg reinforced concrete and steel), an alternative model is required to represent the inelastic response of many other systems, including timber. Although the option remains to design such systems to remain elastic at all load levels, this is unrealistic both with regards cost and real field response. Dowrick (1977) proposed that, provided such systems are designed to ensure they have sufficient inelastic deformation capacity, then collapse mechanisms can be avoided and satisfactory ultimate limit state performance assured, this being consistent with the generally good performance of such systems in the field.

Four alternative response models are indicated shown in Figure 5. The elastic response (insert a) is always available, although caution is required to ensure that the design event is indeed an upper bound loading regime or brittle failure may occur when the system has no inelastic capacity. The elasto-plastic model of insert (b) is commonly used to evaluate inelastic response for many structural systems. Dean *et al.* (1986) developed the slack model (insert c) and used this to demonstrated that system slackness resulted in a very significant increase of the natural period of the structure, which, with most design spectra results in a reduction in the lateral acceleration coefficient and hence the base shear of such systems. This force reduction occurs without "fat" hysteretic loops normally associated with absorption of seismic energy. Slackness is also thought to isolate the mass from high ground acceleration pulses of short duration.

Later work by Stewart (1987) and Dean (1994) culminated in the development of a bar and spring model, which replicated observed degrading system response. The hysteretic model comprised a rigid bar with a number of nominally bi-linear elastoplastic springs attached to it as shown in Figure 6. This model generated a smoothed hysteresis loop similar to that observed in the laboratory. The parameters required to describe the response were however complicated and Deam (1996) developed a windows based computer simulation procedure, Phylmas (Pinched Hysteretic Loop Matching and Analysis System) which visually matched the elemental response to the mathematically 'equivalent' element. This procedure was used to determine the probable response of the failed Meadows Apartment longitudinal wall panel system.

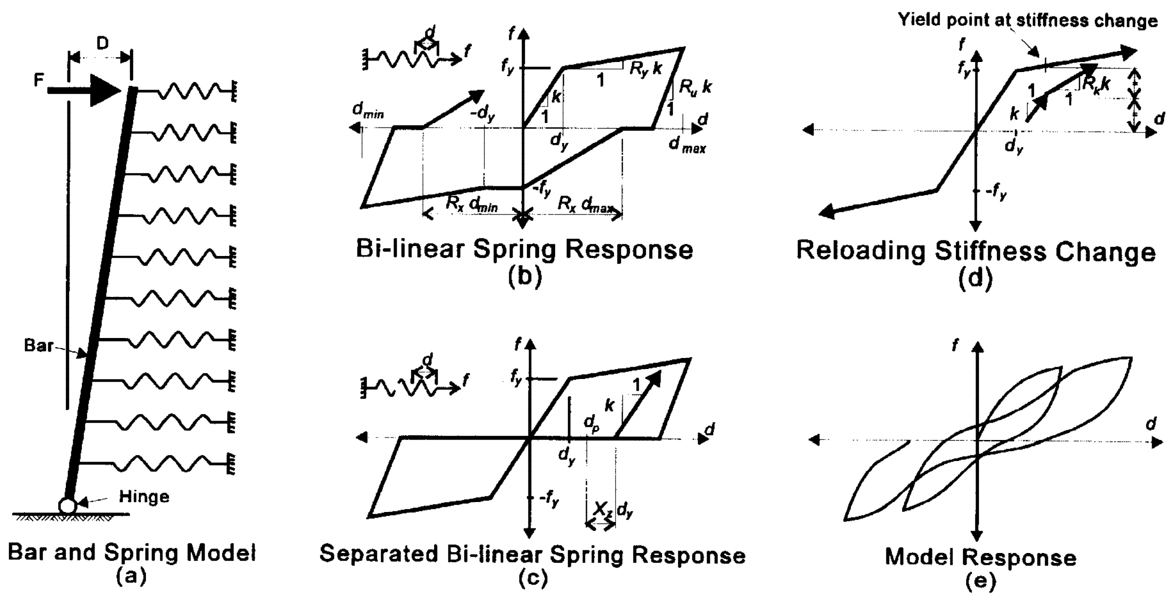


Figure 6. **Bar & Spring Model (Deam 1994)**

THE EXPERIMENTAL PROGRAMME

During the post collapse inspection of the Meadows Apartments at Northridge, the primary mechanism observed, and considered to be fundamental to the collapse of the building, was the loss of lateral load resisting capacity of the gypsum based wall bracing panels. The mechanism observed (see Figure 3) was the failure of the sheet lining fasteners along the top or bottom plate. Shear transfer was then only possible by flexure within the studs. The panels most obviously deficient in this regard were approximately 4.8 m long. The sheet linings were nailed to the framing timber with 8d nails at approx 6 inch centres. The lining sheets were installed horizontally, with paper reinforcing tape over the horizontal seam. The panels were assessed as supporting an axial load of not less than 500 kg/metre length.

Four test specimens were prepared. Each was 4.8 m long by 2.4 m high and formed from Ex 100 by 50 mm radiata timber framing members (cf the 3 by 4 studs observed). Specimens 1 and 4 had sheets horizontally fixed with perimeter nails at 150 and 200 mm respectively. In line with New Zealand practice, specimen 2 was similar in length but with the gypsum boards in a vertical orientation. Specimen 3 used an enhanced gypsum bracing board (Braceline® by Winstones Wallboard Ltd) also installed with the sheets vertically orientated. In addition to a reinforcing fibre within the body of the sheets, 12 mm diameter washers are used beneath each perimeter nail to preclude nailhead pull-through with specimen 3.

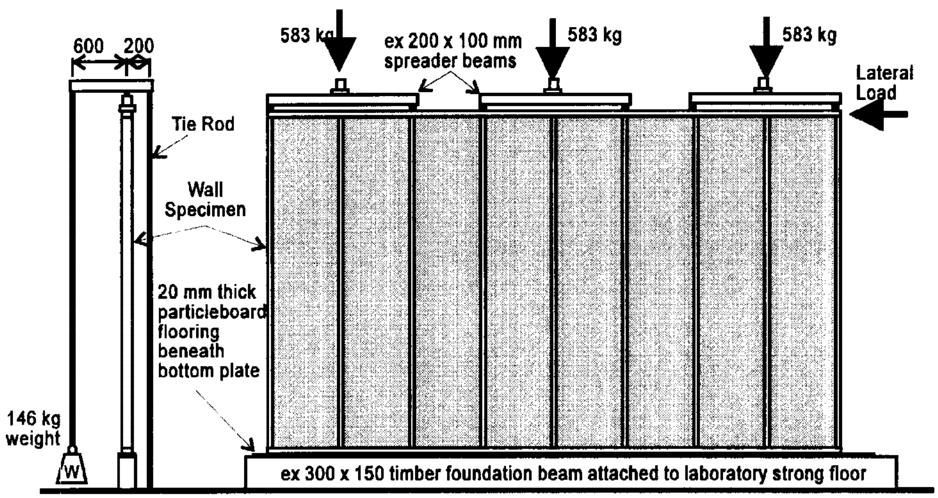


Figure 7. Experimental Test Configuration which Included Axial Self-Weight Provisions

Each specimen was subjected to the require vertical load as indicated in Figure 7, then subjected to three reverse displacement cycles, which were incremented by 5 mm following each set.

Specimens 1 and 4 exhibited collapse mechanisms similar to those observed at Northridge with the plate nails being pulled through the sheet material in both cases. Specimen 2 failed in a similar manner and at similar displacements. The superior quality of this record has resulted in this being as the basis for assessing the performance of all of these systems below. The enhanced Braceline® panel was much stiffer (5 kN/mm cf 2.6 kN/mm) and the washers were found to inhibit the onset of nail-head pull-through. Loss of strength was similarly delayed and the system considered to perform in a significantly superior manner.

MEADOWS APARTMENT ASSESSMENT PROCEDURE

The procedure used to ascertain the dynamic response which could be expected from the Meadows Apartment complex during the Northridge earthquake involved the following steps:

1. Determine the inelastic dynamic response characteristics of the degrading being considered. The most satisfactory means of achieving this is to subject a full scale element to increasing cyclic displacement until significant loss of lateral load carrying capacity (50% maximum) is observed.

2. Define a simulation element which replicates the observed behaviour by visually matching the hysteresis loops from the experimental programme with the computer generated element by adjusting the array of pinched hysteresis response parameters. The match may be seen in Figure 9.

3. Generate displacement (or acceleration) response spectra for a range of earthquake records. This includes NZA, a synthetic acceleration record generated to match the elastic uniform risk acceleration response spectrum published in NZS 4203 (SNZ, 1992), as well as actual time history records recorded during the January 17th Northridge from Sherman Oaks (9 km South of Meadows Apartment) and the Nordoff/Arleta Fire Station site (8 km East) and strong motion seismographs as recorded earthquake. Norton *et al.*, (1994) indicated that the Sherman Oaks spectrum is similar in character to that implicit within UBC S2 (1991) and to the uniform risk NZS 4203 design level while the Arleta/Nordoff record is significantly below code design response For comparison the displacement spectra were also generated from El Centro (N/S) 1940 and Bucharest 1977 ground motion records. The resulting displacement response spectra are shown in Figure 10.

4. The maximum natural period, T_{max}, at which the calculated displacement coincides with that at which the test specimen experienced significant loss of lateral strength is assessed directly from the displacement spectra.

5. The lateral mass M (in kg) able to be restrained by the element without exceeding Tmax is then calculated, using the initial stiffness k (in kN/mm) derived during the hysteresis loop match and Equation 1 below

$$M = 25300\,k\,T^2 \qquad (1)$$

Equation 1 was derived from Figure 5(a) dynamic response equation.

MASS RATING FOR GYPSUM BASED WALL SYSTEMS

The displacement response spectra for both the standard gypsum based wall (as with the Meadows Apartment) and an enhanced gypsum based bracing panel of the same configuration are shown for various earthquake records in Figure 10. In each case, the reliable maximum displacement observed during the experimental phase is noted and used as an upper bound for the system response within the respective displacement response spectra. The two local records shown are for the Arleta-Nordhoff Ave Fire Station and Sherman Oaks.

The results indicated in Figure 8 detail the mass which could be supported by each systems when subject to the Sherman Oaks (NZ Code level) earthquake and the Arleta/Nordoff record for comparison. These apply to each lining face of each 4.8 m long panels. In the case of the Meadows Apartment, there were two double lined longitudinal wall systems. Other tests show that second lining face contributes an additional 50%. Therefore 3 effective wall linings were considered to be present present. The mass sustainable without exceeding the reliable displacement when subjected to the Sherman Oaks record was (2250x3=) 6,750 kg per panel (cf Arleta excitation equivalent of 3800x3 = 11,400 kg). The assessed total mass present per metre of building length was 1840 kg (including self weight plus nominal live load) or 10,000 kg per 4.8 m panel (with allowance for door opening). Thus the lateral resistance is sufficient to resist an equivalent of 67% g for standard gypsum board of 114% g for enhanced board when subjected to code intensity earthquake attack without exceeding the reliable displacement limits as demonstrated by test.

	Sherman Oaks/NZ Code		Arleta/Nordoff	
	Period (Sec)	Mass (kg)	Period (Sec)	Mass (kg)
Std Gypsum	0.185	2250	0.24	3800
Enhanced Gypsum	0.225	6400	0.30	11,400

Figure 8. Lateral Mass Sustainable By Alterative Systems Under Various Earthquake Records

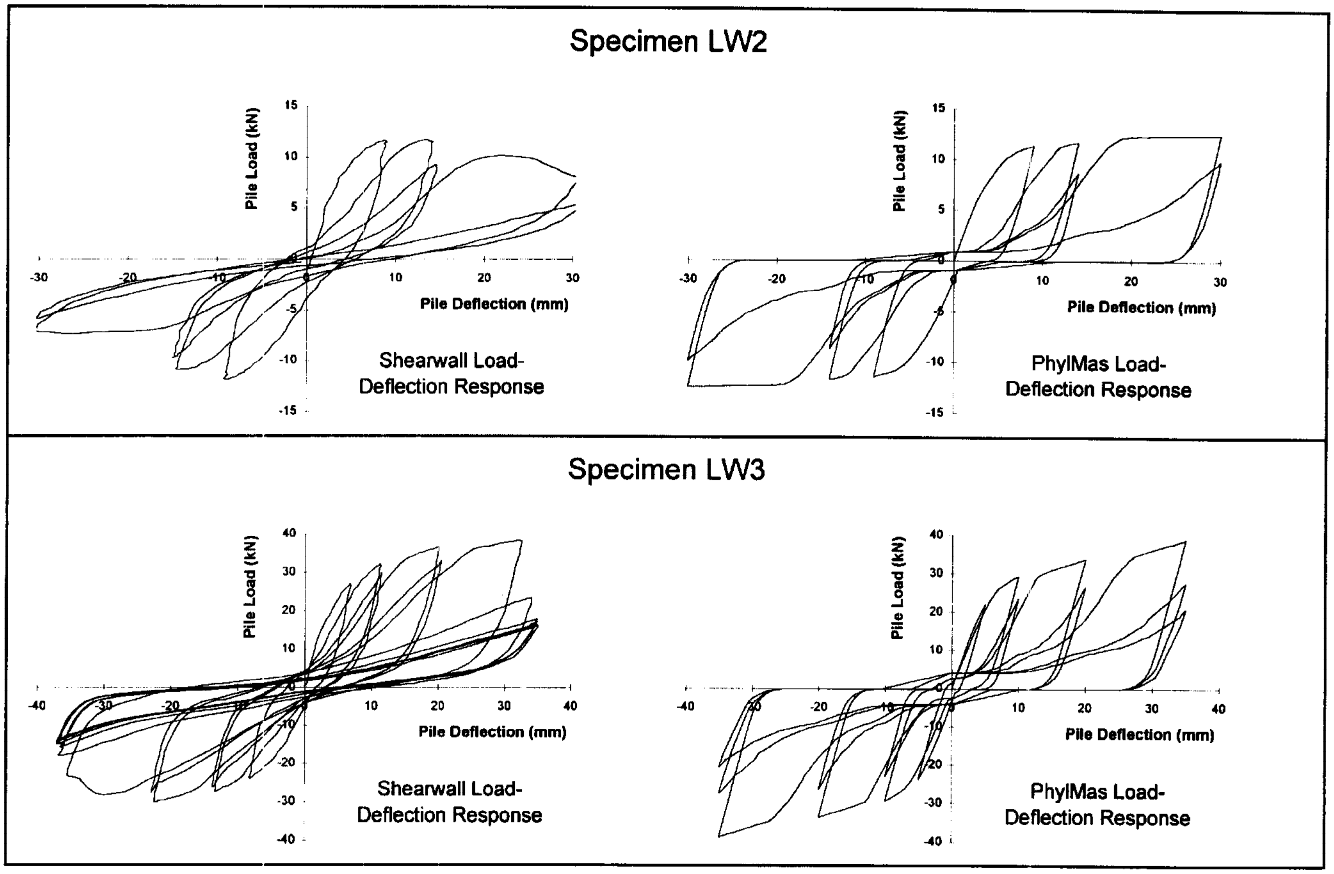
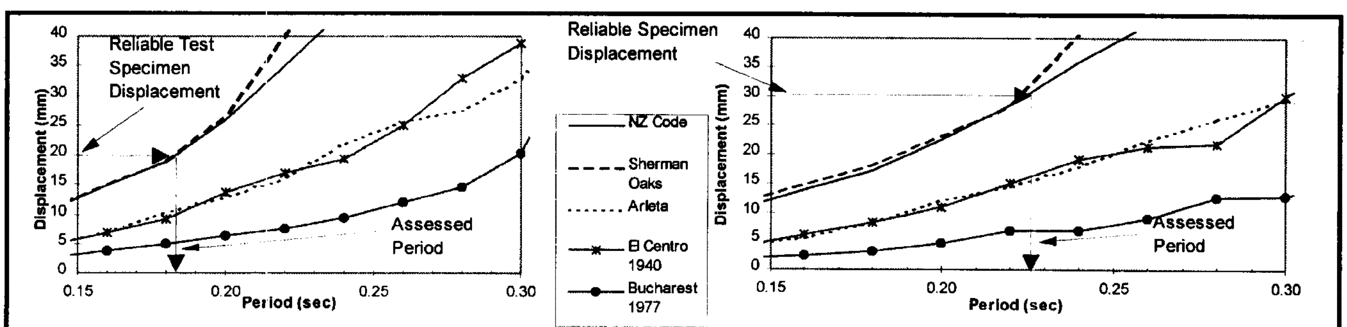


Figure 9. **Phylmas Matched Comparison with Experimental Response**



(a) Standard Gypsum Board (1 Side only)

(b) Enhanced Gypsum Board (With Nail Washers & fibreglass reinforcing)

Figure 10. **Gypsum Wall Panel Displacement Response**

SUMMARY

1. The earthquake generated displacements which resulted in the collapse of several blocks within the Meadows Apartment block during the 17 January 1994 Northridge earthquake can be replicated within the laboratory and can be used as the basis for specific engineering design of degrading systems

2. The good behaviour of timber framed structures observed in the field is a function of the large change of stiffness which accompanies degradation rather than the more traditional energy dissipation applied to other structural materials.

3. Provided inelastic displacement can be controlled, degrading systems, such as timber framed buildings, can be reliably designed to resist earthquakes without collapse.

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