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Earthquake ductility and overstrength in residential structures

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Abstract. This paper reviews aspects of current design procedures for seismic design of structures, and specifically examines their relevance to the design of light framed residential buildings under earthquake loading. The significance of the various structural contributions made by the components of cold formed steel framed residential structures subjected to earthquake induced loadings has been investigated. This is a common form of residential construction worldwide. Particular attention is given to aspects related to ductility and overstrength, the latter arising principally from the contributions of the designated "non-structural" components. Based on both analytical and experimental data obtained from research investigations on steel framed residential structures, typical ranges of the ductility reduction factor and overstrength ratios are determined. It is concluded that the latter parameter has a very significant influence on the seismic design of such structures. Although the numerical ranges for the inelastic seismic parameters given in this paper were obtained for Australian houses, the concepts and the highlighted aspects of seismic design methodology are more widely applicable.

Key words: light framed structures; houses; ductility; overstrength; seismic design.

1. Introduction

The overall project described in this paper has been concerned with the dynamic behaviour of cold formed steel framed houses under earthquake-induced loading. This aspect of earthquake

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engineering has received relatively little attention, partly because such residential structures process a high degree of redundancy and overstrength, and therefore have substantially different characteristics compared with conventionally engineered structures. Such structures are however common throughout Australia, North America, Japan, parts of Northern Europe and are currently being extensively introduced throughout South East Asia. These residential structures generally take the form of detached single or double storey houses for a single family occupancy.

The work presented in the present paper forms part of a large research project on the performance of cold formed steel framed residential structures. In this research the behaviour of plasterboard clad, residential steel framed structures has been investigated both experimentally and analytically. The load paths, failure mechanisms and sensitivity of performance of these walls to parameter changes have been examined and understood (Gad 1997 and Gad *et al.* 1995, 1997, 1999). The findings from the experimental programme and complementary analytical models provide a rational approach to the design of these components based on thorough understanding of their behaviour under lateral (earthquake or wind) loading and are therefore considered to produce optimum and safe structures.

This paper examines the various factors used in ultimate strength seismic design of structures, and assesses their relevance to houses and highlights the limitations of this methodology for such structures. Based on previously obtained experimental results and the developed analytical models, a rational method is used to estimate the ductility and overstrength of plasterboard clad walls.

2. Conceptual background

2.1. Ductility and response modification factors in seismic codes

Conventional earthquake design procedures adopted in most earthquake codes including the Australian Earthquake Standard, also known as the Australian earthquake code (Standard Association of Australia AS1170.4-1993) and the U.S.A.'s Uniform Building Code, referred to here as UBC-1994 (International Conference of Building Officials 1994) use elastic analyses to estimate the induced earthquake forces on structures. The elastic forces are reduced to account for the inelastic behaviour of structures using the Structural Response Modification Factor (R_f) which is defined as follows:

$$R_f = \frac{S_e}{S_d} \tag{1}$$

where S_e = Elastic Strength
S_d = Design Strength.

The response modification factor defines the relationship between the elastic and inelastic seismic response of structures and hence is a critical parameter in any seismic design procedure considering ultimate limit state earthquake loading. For ultimate limit state design, earthquakes with a 500 year return period are typically considered. For the serviceability limit state, earthquakes with 20 to 50 year return period are considered, and structures are typically designed to remain in the elastic range.

Buildings are generally classified according to their structural form and correspondingly an R_f value is assigned for design purposes. The values of R_f in the Australian earthquake code are generally based on those of UBC, with modification to account for limit state design rather than working stress design. The R_f factors adopted in the UBC are based largely on Californian research (Uang 1991), along with experience from numerous moderate and severe earthquakes. The code R_f factors broadly account for the following characteristics of the structural system: energy absorbing capacity, expected overstrength, likely degree of redundancy and performance in past earthquakes (Lam *et al.* 1997).

The response modification factor is widely accepted as the product of two components, namely ductility reduction and overstrength. Thus R_f is expressed as:

$$R_f = R_\mu \times \Omega \tag{2}$$

where R_μ = Ductility reduction factor
Ω = Overstrength factor.

R_μ is defined as:

$$R_\mu = \frac{S_e}{S_y} \tag{3}$$

where S_y is the yield strength of the structural system as obtained from a static push-over analysis. The overstrength factor relates the design strength (S_d) and S_y as follows:

$$\Omega = \frac{S_y}{S_d} \tag{4}$$

Fig. 1 depicts the actual, elastic and idealised responses for a system which shows the relationship between the various parameters.

2.2. Theoretical relationships between ductility and R-factors

Newmark and Hall (1982) related the kinematic ductility demand μ to R_μ by the following expressions:

$$R_\mu = \mu \qquad \text{(for } T > 0.5 \text{ sec)} \tag{5}$$

$$R_\mu = \sqrt{2\mu - 1} \qquad \text{(for } 0.1 < T < 0.5 \text{ sec)} \tag{6}$$

$$R_\mu = 1 \qquad \text{(for } T < 0.03 \text{ sec)} \tag{7}$$

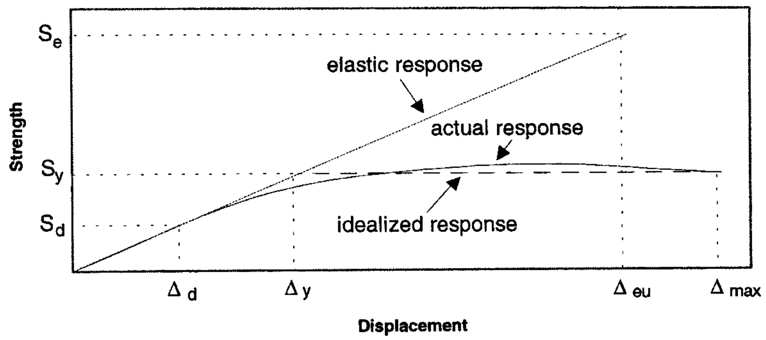


Fig. 1 General structural response

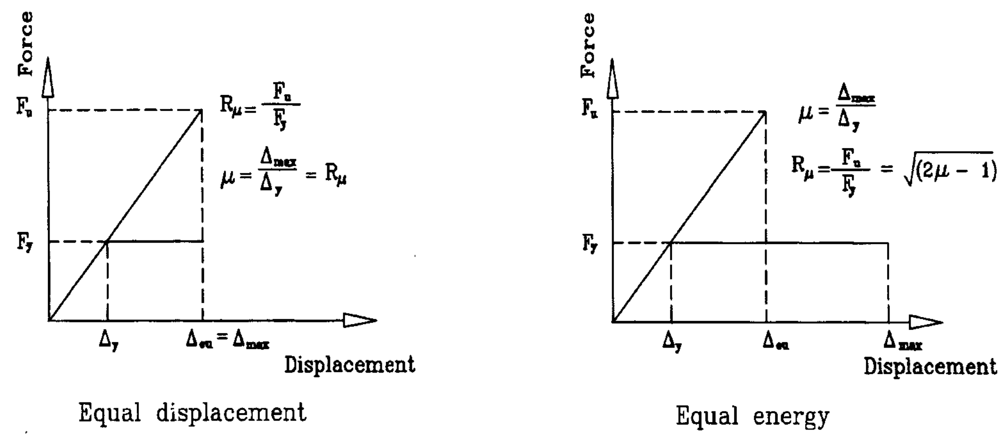


Fig. 2 Definitions of R_μ and μ in terms of equal displacement and equal energy theories

Eqs. (5) to (7) are based on the equal displacement, equal energy and equal acceleration theories, respectively. The latter permits no strength reduction in highly stiff systems which possess limited ductility capacity. The ductility demand factor μ is defined by Eq. (8) and expressed graphically in Fig. 2.

$$\mu = \frac{\Delta_{max}}{\Delta_y} \tag{8}$$

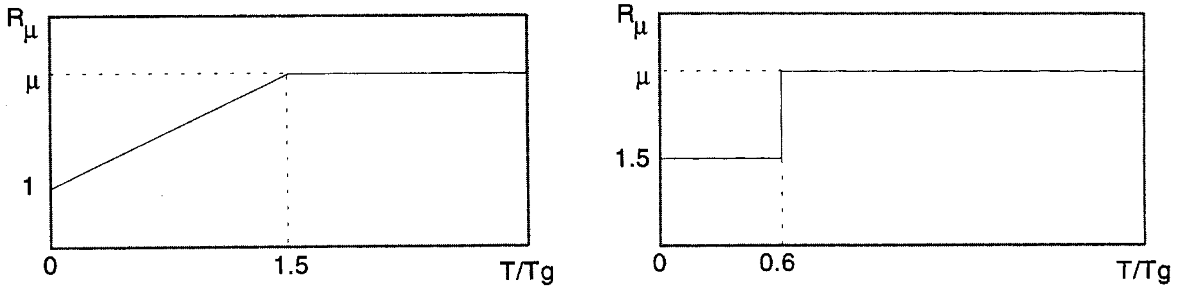
where Δ_{max}=Maximum displacement from a non-linear model
Δ_y = Yield displacement.

This Newmark and Hall method has been widely accepted in the design of ductile structures. However, the relationships have been verified by a limited number of earthquake records. In a study by Mahin and Bertero (1981) the observed scatter of results challenged the accuracy of these relationships between R_μ and μ. Further studies undertaken by Elghadamsi and Mohraz (1987) and by Miranda (1993) highlighted the importance of the ground motion frequency content on the ductility demand, which explained the scatter of results observed by Mahin and Bertero (1981). A new displacement-based seismic assessment procedure developed by Priestley (1997) for existing reinforced concrete buildings which includes the ground natural period has redefined the relationship between R_μ and μ. This new expression is defined by Eq. (9):

$$R_\mu = 1 + (\mu - 1)\frac{T}{1.5T_g} \leq \mu \tag{9}$$

where T = Natural period of an equivalent Single Degree Of Freedom (SDOF) model
T_g = Predominant natural period of site.

The site predominant natural period is the period corresponding to peak spectral response in the elastic response spectrum, also known as the corner period (Lam *et al.* 1995). Eq. (9) assumes that the equal displacement theory (R_μ=μ) applies when T>1.5T_g, and equal acceleration (R_μ=1) applies when T approaches zero. Within these two limits, linear interpolation can be made to determine R_μ. The proposed new relationship is depicted graphically in Fig. 3. T_g is essentially dependent on the soil type, for example, rock and very stiff soils would have a value less than 0.2 seconds, while flexible soils would have a T_g of more than 0.5 seconds.



(a) (b)

Fig. 3 Relationship between R_μ, μ and T/T_g: (a) according to Priestley (1997); (b) according to Lam *et al.* (1997)

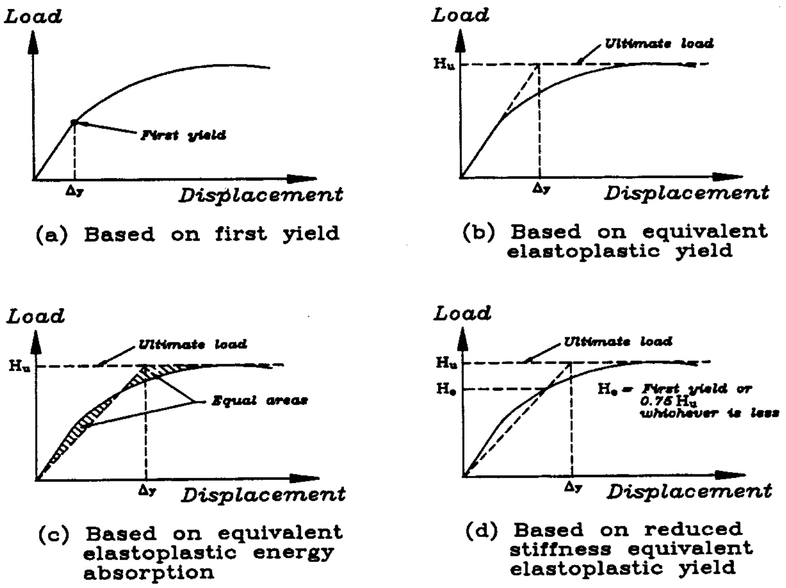


Fig. 4 Alternative definitions for yield displacement, after Park (1989)

This relationship was further refined and verified by numerous artificial and real earthquake records by Lam *et al.* (1997). The relationship between μ and R_μ, according to the latter study, is defined as follows:

$$R_\mu = \mu \qquad T > 0.6\,T_g \tag{10a}$$

$$R_\mu = 1.5 \qquad T \le 0.6\,T_g \tag{10b}$$

Eq. (10a) basically states that the equal displacement approach is suitable as long as the natural period of the structure is more than 0.6 times that of the ground. For short period structures, which would fall under the condition of Eq. (10b), the ductility demand (μ) is irrelevant and R_μ can be approximated to 1.5. Eqs. (10a) and (10b) are illustrated in Fig. 3.

2.3. Determination of ductility factors

Ductility is defined as the ability of a structure to undergo repeated and reversing inelastic deflections beyond the point of first yield while maintaining a significant proportion of its initial load carrying capacity. Ductility is a major concern in earthquake resistant structures. It is desirable to have high ductility, indeed, earthquake codes tend to impose a heavy penalty on non-ductile structures. A philosophy used in most earthquake codes, including the Australian

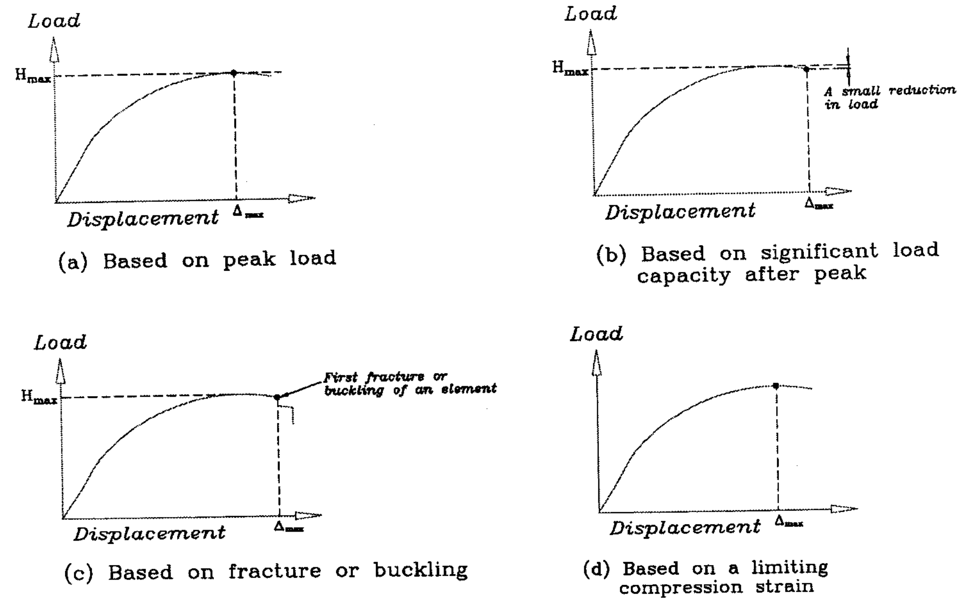


Fig. 5 Alternative definitions for maximum displacement, after Park (1989)

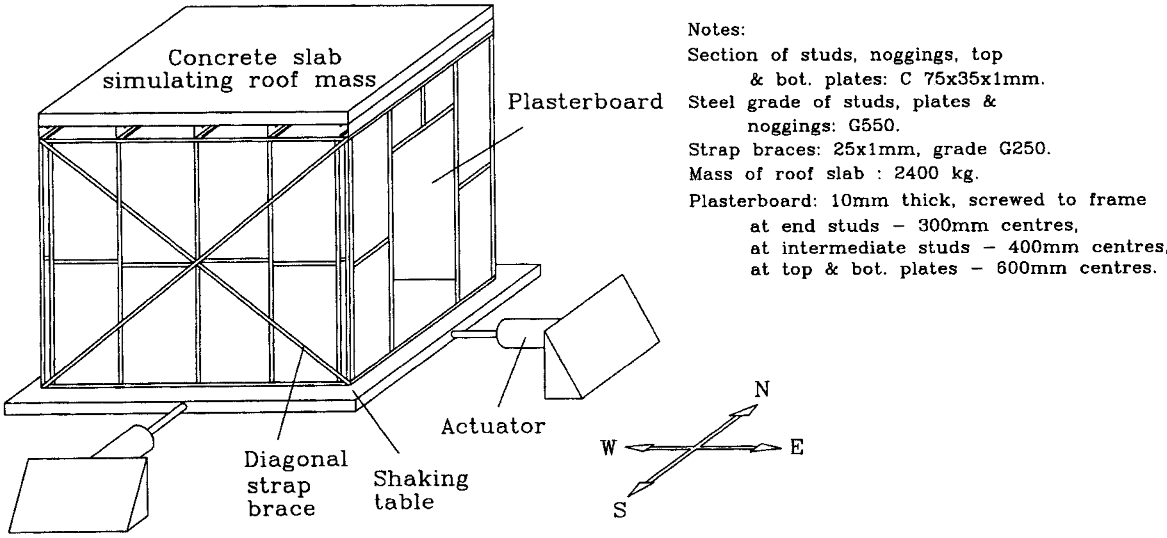


Fig. 6 A diagram of the test house and the shaking table

earthquake loading standard, is that a structure may be damaged but does not collapse in the event of a moderate to large earthquake. Therefore, the structure is expected to deform plastically (in the inelastic range) during a moderate to large earthquake. This ability of the structure to deform plastically is related to the structure's ductility capacity.

In order to evaluate the ductility demand factor μ, the yield and maximum displacements have to be defined, as shown in Fig. 2. However, there are different definitions for the terms yield and ultimate displacements. Park (1989) itemised the possible definitions for yield and ultimate displacements that have gained considerable recognition worldwide. These definitions are presented in Figs. 4 and 5 for the yield and ultimate displacements, respectively. The selection of an appropriate method for evaluating yield and ultimate displacements is a critical feature of seismic design procedures and whilst attention has been focused on these aspects for conventionally engineered structures, there has been limited research into how these procedures relate to the seismic design of light framed residential structures.

2.4. Overstrength factors

The next step towards calculating the response modification factor (R_f) is to estimate the overstrength factor (Ω). The overstrength takes into account all possible sources that may contribute to strength exceeding its nominal or idealised value. These sources have been reasonably identified for reinforced concrete and steel buildings. For example, in steel and reinforced concrete structures, overstrength is attributed to steel strength being greater than the specified yield strength, and additional strength due to strain hardening. For reinforced concrete buildings, overstrength also includes unaccounted for compression strength enhancement of the concrete due to its confinement.

It should be noted that not all earthquake codes specify an overstrength factor. For example, the Uniform Building Code of the U.S.A. (UBC-1994) has both the overstrength factor (Ω) and the ductility reduction factor (R_μ) combined into one parameter which is the working stress Force Reduction Factor (R_w). Similarly, the Australian Earthquake Standard (AS1170.4-1993) does not explicitly split Ω and R_μ, it only offers the structural response factor (R_f). Other earthquake codes do split Ω and R_μ and explicitly define each factor. For example, the New Zealand Earthquake Load Standard NZS4203-1992 (Standards Association of New Zealand 1992) separates the two factors and specifies the overstrength factor as 1.5. This value is applicable to all construction types and structural systems, unless the designer can justify a more appropriate value.

Overstrength factors should not be confused with safety factors. The safety factors remain in the seismic design and are not removed or counteracted by the overstrength factor.

2.5. Current code values

The Australian Earthquake Standard AS1170.4-1993 (Standards Association of Australia, 1993) considers steel and timber framed residential structures to be ductile. In the Australian code, the response modification factor (R_f) is dependent on the structural form and whether the system is load-bearing. There are no specific R_f factors for structural systems used in residential construction, hence, strap braced frames may be classified as concentrically braced frames while plasterboard clad walls may fall under the category of light framed walls with shear panels. For load bearing walls, the strap braced frames have an R_f value of 4.0, while the plasterboard clad panels have an R_f value of 6.0. For non-load-bearing walls, the strap braced frames are assigned an R_f value of 5.0

while the plasterboard clad panels are assigned an R_f value of 7.0. The code does not separate the overstrength and ductility components.

3. Evaluation of μ and R_μ from experimental results using established methods

In determining the response modification factor, the following steps are commonly followed:

- Evaluate the ductility demand factor (μ). This requires an equivalent elasto-plastic system to calculate Δ_y and Δ_{max}.
- Estimate the ductility reduction factor R_μ. This requires a relationship between R_μ and μ.
- Establish the overstrength factor Ω.
- Multiply R_μ and Ω yielding the response modification factor R_f.

This outlined sequence has been used in this paper to assess each key parameter. It is not intended to find a definitive value for the response modification factor for direct code implementation, but rather to introduce a methodology to determine it and then to demonstrate its applicability to cold formed steel framed houses. In order to achieve a reliable design parameter, numerous analyses, configurations and regional differences have to be considered to produce representative values, which are factors outside the scope of the reported project.

3.1. Brief description of the experimental programme

A one-room-house or "test house" was adopted as a test specimen. It measured 2.3m×2.4m× 2.4m high and was constructed from full scale components as shown in Fig. 6. The test house simulates a section of a rectangular house with plan dimensions of 11m×16m. A dead load corresponding to a house plan area of 11m×2.4m was applied to the test house. The mass of the roof tiles, insulation, ceiling cladding, battens, and trusses for that area was found to be 2350 kg. A concrete slab with the same weight was cast and supported on the East-West walls of the test house via steel C sections similar to those used for the bottom cord of typical roof trusses. The two walls in the North-South direction were non-load bearing and had standard 900×2100 mm

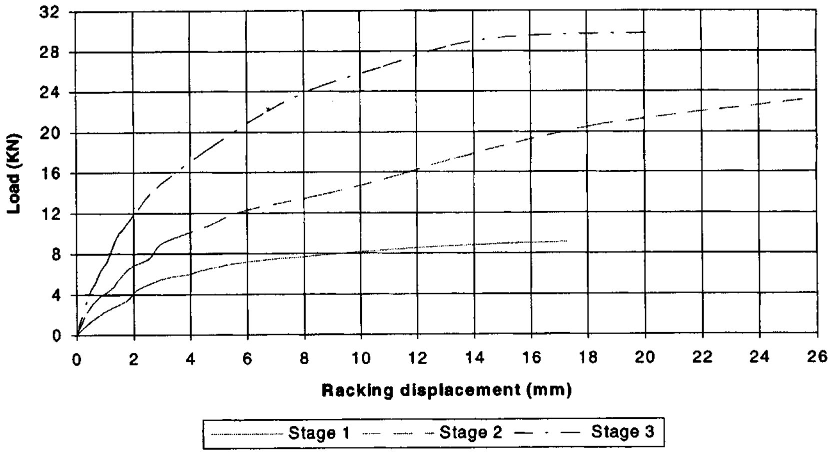


Fig. 7 Load-deflection curves for Stages 1, 2 and 3 in the E-W direction

Table 1 A summary of all experimental stages relating to the test house

Stage	Description
0	Unclad wall frames with no strap bracing.
1	Unclad wall frames with strap bracing on all four walls.
2	Plasterboard clad frames and ceiling with skirting boards and ceiling cornices, without strap bracing.
3	Plasterboard clad frames and ceiling with skirting boards and ceiling cornices, with strap bracing on all four walls.
4	Plasterboard clad frames and ceiling with skirting boards and ceiling cornices, with strap bracing on the East-West walls only. Brick veneer external walls on all four walls but not connected at the corners.

door openings. The test house was built on a two degree of freedom shaking table at The University of Melbourne, Australia. The test house was constructed by professional tradesmen according to detailing recommended by manufacturers.

The test house was tested at various stages of construction, in both directions, to identify the influence of the various structural and non-structural components on the lateral performance. These stages are summarised in Table 1. The results from Stages 1, 2 and 3, in the East-West direction, are used in this paper to evaluate the various seismic parameters. The test house was subjected to cyclic racking lateral loads to destruction in these three Stages. Based on the obtained hysteresis loops from these tests, the backbone load-deflection curve for each Stage was determined, as shown in Fig. 7. Further details regarding the test house and the loading regimes have been presented in Gad *et al.* (1995).

3.2. Evaluation of ductility using methods proposed by Park

Based on the experimental load-deflection curves, the ductility demand factor (μ) is estimated using the proposed methods by Park (1989). The first step is to find the yield displacement (Δy) on the load-deflection curve. The four methods outlined in Fig. 4 have been evaluated to determine whichever is the most suitable. The difficulty with the load-deflection curves for plasterboard clad wall frames is that they do not have a distinct yield point, the curves being highly non-linear, as shown in Fig. 7. The first form of yield occurs at a racking displacement of

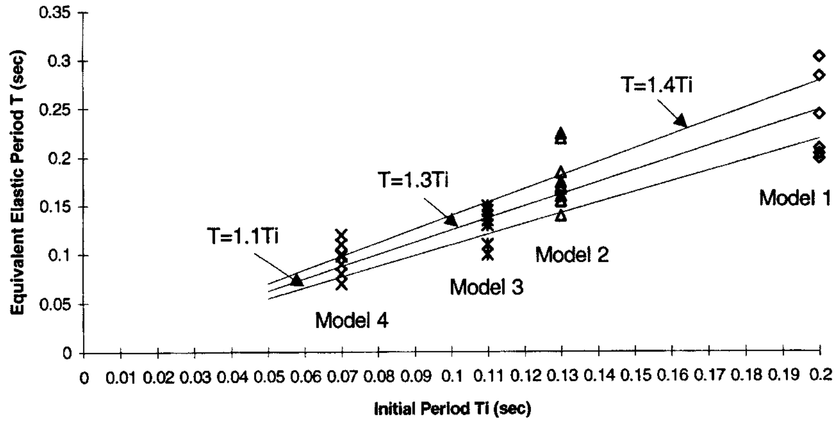


Fig. 8 Initial period versus equivalent elastic period for the four models and 9 earthquake ground motions

less than 5.0 mm. This form of yield takes place in the screw connections between the plasterboard and the frame. Not all the screws yield at the same time, but the yield is progressive, starting from the bottom screws. At some point during loading, there would be screws that have failed, others that are yielding and screws that are still in the elastic range, all before the wall reaches its ultimate load carrying capacity. Even when the crushing of plasterboard edges starts it does not happen suddenly, but again progressively, whereby the very bottom segment crushes first followed by a higher segment and so on. Therefore, the load-deflection curve is a combined result of components that have failed, that are yielding and that are still in the elastic range.

As a result, the so-called first yield concept would not be appropriate because the yield displacement would be so small that may lead to the calculation of unrealistically large ductility demands. Therefore, method (a) in Fig. 4 is considered unsuitable. Method (b) is also not suitable because there is no linear part in the load-deflection curves to define the equivalent elasto-plastic system. Method (d) is recommended for general engineered structures by Park, but again it would not be suitable for light framed residential structures because the yield displacement obtained using this approach should be smaller than or equal to the first yield, which is demonstrably not true in the present case (Fig. 7). However, to investigate the sensitivity of computed ductility values this method has been adopted, along with method (c).

The next step is to define the maximum or ultimate displacement (Δ_{max}). The most suitable method out of those presented in Fig. 5 for Stage 2 of the experimental programme (Table 1) is method (b) as most of the ultimate load is sustained for some displacement. Due to the presence of the strap braces in Stage 3, the maximum load and displacement are more clearly defined, and methods (a) and (c) are applicable and produce the same result. Having obtained the yield and maximum displacements, μ is calculated as in Eq. (8). Consequently, R_μ may be determined using either the equal displacement or equal energy formulae. The results for Stages 2 and 3 for the tests conducted in the East-West direction have been presented in Table 2. It should be noted the equal acceleration approach yields a value of 1 for R_μ.

Similarly, Barton (1997) calculated the R_μ factor for Stage 1 which had the strap braces only. For this case the suitable methods for calculating the yield displacements were methods (b) and (c) of Fig. 4. His results have been reproduced in Table 3.

Table 2 The calculations for μ and R_μ for Stages 2 and 3 of the test house

Stage	Yield displ.[1] Δy (mm)		Maximum displ.	R_μ	
	Method (c)	Method (d)	Δmax (mm)	Equal displ.[2] $R_\mu = \mu$	Equal energy[2] $R_\mu = \sqrt{(2\mu - 1)}$
2	9.7	17.8	35.9	3.7-2.0	2.5-1.7
3	9.0	8.8	20.4	2.3	1.9

1. Methods (c) and (d) are those defined in Fig. 4
2. The range of values for R_μ is determined from the two different yield displacements defined in the Table

Table 3 R_μ for stage 1 of the test house, after Barton (1997)

Stage	R_μ			
	equal displacement		equal energy	
	method (b)	method (c)	method (b)	method(c)
1	5.1	2.9	3.0	2.2

It should be noted that the fundamental period of the test house, and possibly most light framed residential structures, falls under the equal energy condition according to Eq. (6). From Table 3 using the equal energy approach, R_μ ranges between 2.2 and 3.0 for Stage 1. Similarly, from Table 2 and Stage 2, R_μ ranges between 1.7 and 2.5, while for Stage 3 R_μ is 1.9. The variation within Stages 1 and 2 is due entirely to the different definitions of yield displacement. There is no one definition that is ostensibly superior to the others. Most of these definitions are based on experience, particularly on reinforced concrete and steel structures which tend to have more distinct elastic and plastic regions. Whereas, as discussed above the type of residential systems investigated here exhibit high levels of non-linearity with no distinct initial linear behaviour. Although in Stage 1 the bracing was provided by steel strap braces, the apparent yield is a function of the initial tension of the braces (Barton 1997). Hence, R_μ would also depend on the initial tension, increasing its variation even further. It should be noted that the theoretical definitions of yield displacement are not only restricted to those proposed by Park (1989), as illustrated in the following section.

3.3. Evaluation of ductility using methods adopted in New Zealand

Researchers in New Zealand faced the same problem in trying to define the yield displacement for timber framed residential structures. King and Lim (1991) found that the methods presented by Park for defining the yield displacement are difficult to translate to degrading timber systems. The researchers presented a simplified approach to be used in the evaluation of light framed timber walls in conjunction with the New Zealand Timber Framing Code (NZS3604:1990). However, the researchers stated that the simplified approach is an interim measure while investigation is continuing in an attempt to define an equivalent elasto-plastic system which demonstrates performance characteristics similar to those encountered in light timber framed walls.

King and Lim (1991) consider the yield load to be half of the ultimate load. Hence, on the load-deflection curve, the corresponding yield displacement can be found. Consequently, the ductility demand factor μ can be calculated as the ratio of the ultimate displacement to the yield displacement. This approach is used widely in New Zealand and is a supplement to the standard evaluation procedure for light framed walls (BRANZ-P21) (Cooney and Collins 1988). Based on this approach μ has been calculated for the three Stages of the test house, as shown in Table 4. From Table 4 it is obvious that the approach proposed by King and Lim (1991) produces higher ductility demand factors than those proposed by Park (1989) implied by the results in Tables 2 and 3. Although the former approach is widely used in conjunction with experimental walls in New Zealand, its basis is questionable and it is presently used as an interim measure while a more rational methodology is being developed.

It should be noted that the ductility reduction factor R_μ is not explicitly defined in the New Zealand Earthquake code NZS4203:1992 (Standards Association of New Zealand 1992). However, the structural ductility demand factor μ is accounted for by the basic seismic hazard acceleration

Table 4 Ductility demand factors based approach proposed by King and Lim (1991)

Test House Stage	μ
1	6.4
2	6.6
3	6.8

coefficient (C_h), which also takes into consideration the different soil conditions and translational periods of vibration. For these reasons, the determination of R_μ from the results presented in Table 4 is not straightforward. However, it is anticipated that the equivalent R_μ will differ only slightly from the values of μ given in the Table.

Given the variation and the uncertainty in evaluating the yield displacement and consequently the ductility reduction factor R_μ, a more reliable technique is required to calculate the response modification factor R_f. For this, a more realistic elasto-plastic model is required to fit the non-linear load deflection curves. The main focus of the present investigation is the plasterboard clad wall frames. A separate study has been presented for walls with strap bracing only (Barton 1997). An attempt has been made in the following section to determine the ductility reduction factor R_μ based on first principles using analytical models of the test house. These models essentially reproduce the same load-deflection curves for Stages 2 and 3 with similar hysteretic behaviour. The analytical models have been constructed using a time history inelastic frame analysis software, RUAUMOKO (Carr 1996). Details of these models are presented in Gad (1997).

4. Analytical approach to evaluate ductility parameters

According to the state-of-the-art knowledge on the relationship between the ductility demand μ and the ductility reduction factor R_μ, as suggested by Priestley (1997) and by Lam *et al.* (1997), the predominant ground period has to be considered. The natural period of framed houses in Australia would tend to fall in the range of short to medium periods, and it is anticipated that most of these structures would have periods less than 0.25 sec (Gad *et al.* 1998). Hence, both conditions presented in Eq. (10) are likely to apply to residential framed structures. A comprehensive analytical investigation therefore necessitates time history analysis to investigate various models with different natural periods under earthquakes from sites with different predominant ground periods.

4.1. Equivalent elastic period

As illustrated earlier, the difficulty in identifying the appropriate equivalent elasto-plastic model for structural systems of the type being studied is fundamentally due to the inability to define the elastic stiffness. In other words, it is difficult to determine the equivalent elastic natural period which could then be used to find the elastic response. Plasterboard clad frames do not only exhibit non-linear behaviour but also stiffness degradation and development of slackness which all lead to changes in the natural period as the frames are loaded into the plastic region. The equivalent elastic period is expected to be higher than the initial period due to this decrease in stiffness.

Using non-linear time history analysis, an attempt has been made to determine the equivalent elastic period. Hence, the equivalent elasto-plastic system can be identified and subsequently the ductility reduction factor determined. The methodology adopted is to scale a particular earthquake record (by amplifying the intensity of the acceleration time history record) to the point where the capacity of a particular wall frame is reached during a non-linear transient dynamic analysis. The corresponding elastic displacement response spectrum for this earthquake is then amplified by the same factor. From the amplified elastic response spectrum the period which corresponds to the maximum displacement of the wall frame is obtained. This period is considered to be the

Table 5 Details of earthquakes used in defining the equivalent elasto-plastic system

Label	M[1]	Peak Acc[2] (m/s/s)	F_g[3] (Hz)	T_g (sec)	Location, recording station, direction, date
EQ1	6.6	3.63	8.30	0.12	Northridge - USA, St Monica City Hall, 360°, 17/1/94
EQ2	6.0	1.04	14.0	0.07	Saguenay - Canada, Chicoutimi-Nord, S34W, 25/11/88
EQ3	6.4	13.2	10.0	0.10	Nahanni - Canada, Iverson, Trans., 23/12/85
EQ4	7.3	7.40	4.17	0.24	Tabas - Iran, Tabas, Long., 13/9/78
EQ5	7.4	2.23	5.56	0.18	Honshu - Japan, Ofunato Harbour Works, 12/6/78
EQ6	5.4	3.80	5.00	0.20	San Salvador, Urban Construction Inst., 90°, 10/10/86
EQ7	6.6	3.06	2.00	0.51	Imperial Valley - USA, El-Centro, NS, 18/5/40
EQ8	5.6	4.98	1.67	0.59	Parkfield - USA, Array No.2, N65E, 28/6/66
EQ9	5.4	3.92	1.25	0.80	San Salvador, National Geographical Inst., 180°, 10/10/86

1. Richter magnitude of earthquake
2. Peak ground acceleration
3. Predominant frequency of ground motion

equivalent elastic period. This approach is based on the equal displacement theory, that is the maximum elastic and plastic displacements are considered equal.

Nine earthquake records and four different wall models were considered in this investigation. It is an attempt to find the trend between the initial period (T_i) and the equivalent elastic period (T). The earthquake records used were selected to cover various possible scenarios. Three records were chosen with low T_g (less than 0.12 sec), three with medium T_g (between 0.18 and 0.24sec) and

Table 6 Initial period and description of models used to determine the relationship between equivalent elastic period and the computed initial period

Model	T_i (sec)	Description
Model 1	0.20	Plasterboard clad walls and tiled roof
Model 2	0.13	Plasterboard and strap braces for walls and tiled roof
Model 3	0.11	Plasterboard clad walls and steel clad roof
Model 4	0.07	Plasterboard and strap braces for walls and steel clad roof

Table 7 The amplification factors and equivalent elastic periods for the four models

EQ record	Model 1		Model 2		Model 3		Model 4	
	Factor	T (sec)	Factor	T (sec)	Factor	T (sec)	Factor	T (sec)
EQ1	3.0	0.205	4.0	0.185	8.0	0.110	10.0	0.098
EQ2	34.0	0.200	38.0	0.140	54.0	0.110	50.0	0.070
EQ3	1.7	0.201	1.7	0.160	3.2	0.130	4.5	0.080
EQ4	1.1	0.210	1.4	0.155	2.9	0.135	3.8	0.120
EQ5	4.2	0.210	5.0	0.168	11.3	0.130	17	0.090
EQ6	3.2	0.195	3.5	0.175	7.8	0.150	11.5	0.110
EQ7	2.5	0.245	3.5	0.170	7.8	0.145	11.0	0.100
EQ8	1.5	0.305	2.3	0.225	5.7	0.150	8.7	0.120
EQ9	2.4	0.285	3.5	0.200	7.8	0.140	11.5	0.110

three records with high T_g (more than 0.5sec). The details of all nine records are listed in Table 5, where EQ1, 2 & 3 have low predominant ground period, EQ4, 5 & 6 have medium ground period and EQ7, 8 & 9 have high ground period. The high ground periods are typical of soft soils, medium ground periods are representative of stiff to intermediate soils, while the low ground periods are commonly associated with rock sites.

The four models adopted have different initial natural periods. Therefore, when these models are combined with the nine earthquakes, the results are not biased towards a particular T/T_g ratio. The four models are based on the test house configuration but with two different roof masses (representing tiled and steel roof clad) and two different bracing systems (plasterboard only, and combined plasterboard and strap braces). The four models have been summarised in Table 6. Each earthquake was run through the four models and amplified to reach the maximum displacement (at full plastic capacity) and then the equivalent elastic period was obtained. The level of amplification and the computed elastic period for the 36 runs have been listed in Table 7.

The initial periods versus the equivalent elastic periods are depicted in Fig. 8. To determine an approximate relationship between the initial period (T_i) and the equivalent elastic period (T), the nine values of T from each model have been averaged. Hence an average elastic period is obtained from each model. The ratios of T to T_i are found to be 1.1, 1.3, 1.2 and 1.4 for models 1,

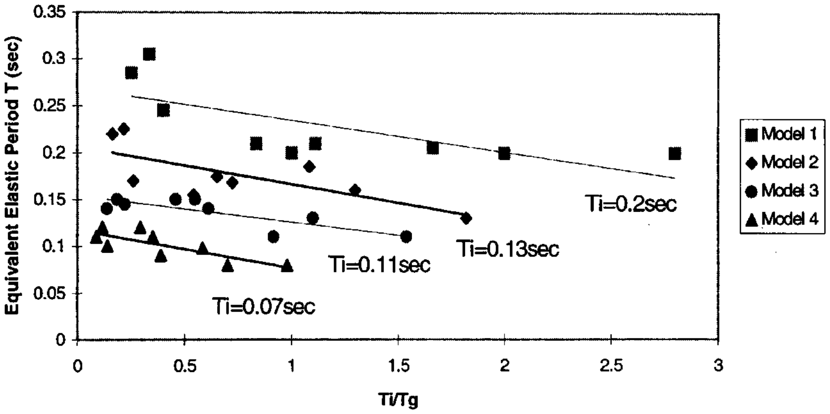


Fig. 9 Results from time history analyses and fitted linear functions for the four models

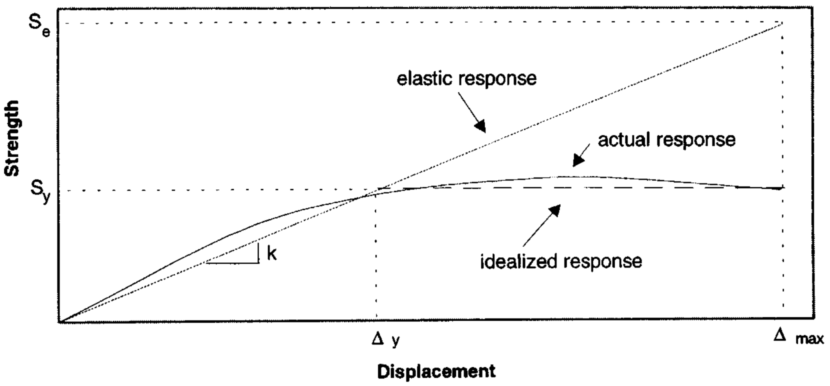


Fig. 10 Model used to determine Δ_y and μ

2, 3 and 4, respectively. These ratios suggest that the equivalent elastic period is expected to be 1.1 to 1.4 times the initial period, with a typical overall ratio being approximately 1.3, refer to Fig. 8. Although 36 substantially different combinations of models and earthquakes were used to achieve this ratio, more combinations would need to be considered to provide a higher level of confidence in the results.

Due to the rather large scatter of results for Model 1, another method to relate T to T_i has been developed. This is a more detailed approach which takes into account the predominant period of the ground motion (T_g). The ratio of T_i to T_g has been plotted versus the elastic period for each model, as shown in Fig. 9. The four models show a consistent relationship between T_i/T_g and T. As T_i/T_g increases, T decreases. Fitting a linear function to the results of each model reveals that the resulting lines have similar slopes, as shown in Fig. 9. The fitted lines have a satisfactory accuracy, yielding coefficients of correlation above 0.6. These linear regression relationships are considered to give reliable estimates of the equivalent elastic periods based on given initial and ground periods. Based on the four linear functions an overall relationship is estimated and presented as follows:

$$T = -0.05\,\frac{T_i}{T_g} + 1.5T_i \tag{11}$$

Therefore, for design purposes a set of curves could be developed in a similar fashion based on the analysis of more models and earthquakes. Alternatively, Eq. (11) can be further verified for other combinations and refined as necessary. This approach would ensure more reliable estimates of the equivalent elastic periods based on analyses conducted on typical behaviour of residential clad frames rather than using models which were developed for reinforced concrete frames or other construction types which exhibit different characteristics.

4.2. Determining μ and R_μ

Having established an equivalent elastic period (T), it is now possible to estimate the ductility demand factor μ. Given the mass (m) of each model and the period T, the corresponding elastic

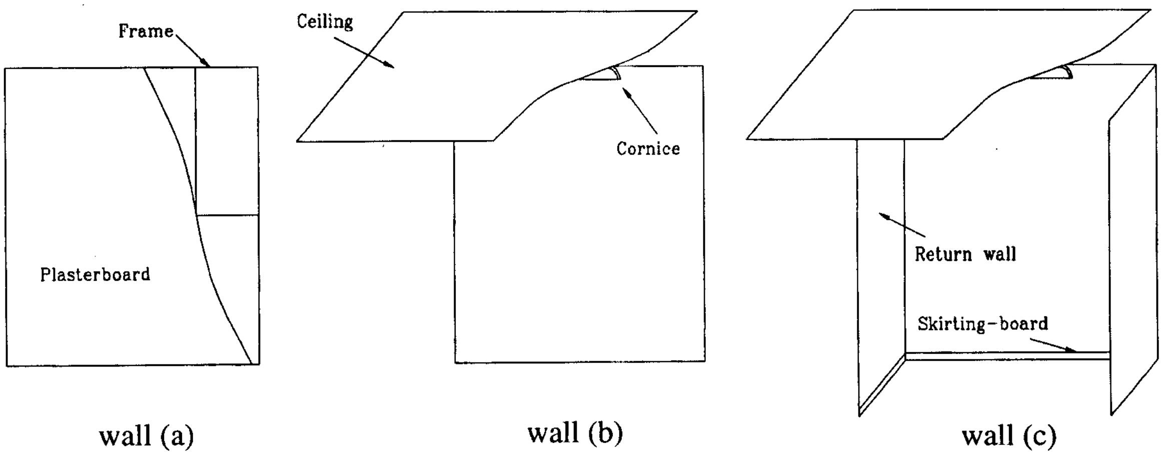


Fig. 11 Plasterboard clad walls with different boundary conditions

Table 8 Calculated R_μ for low, medium and high predominant ground periods

Relation between T and T_g	R_μ			
	Model 1	Model 2	Model 3	Model 4
Low T_g[1]: $T/T_g \geq 1.0$	3.5	3.0	2.8	2.3
Medium T_g[1]: $0.6 < T/T_g < 1.0$	2.6	2.0	1.6	1.5[3]
High T_g[2]: $T/T_g \leq 0.6$	1.5	1.5	1.5	1.5

1. For this range Eq. (10a) was used
2. For this range Eq. (10b) was used
3. Model 4 had the lowest natural period, hence, the earthquakes with medium period produced values of T/T_g less than 0.6. Therefore, Eq. (10b) was applied to this case

stiffness (k) can be found using an equivalent SDOF model as presented in Eqs. (12) and (13).

$$f = \frac{1}{T} = \frac{1}{2\pi}\sqrt{\frac{k}{m}} \tag{12}$$

$$k = \frac{4m\pi^2}{T^2} \tag{13}$$

The yield displacement (Δy) can be calculated based on the elastic stiffness (k). The yield load (S_y) is assumed to be the same as the ultimate load of the non-linear model as shown in Fig. 10 (where $S_y = k\Delta_y$). Consequently, μ is calculated as the ratio of Δ_{max}/Δ_y where Δ_{max} is the maximum displacement from either the non-linear or elastic analyses.

Using the above procedure, the ductility demand factor (μ) has been calculated for the four models and for each of the nine selected earthquakes. Generally, for the four models used, the earthquakes with low ground periods produced a ratio of T/T_g more than 1, those with medium periods produced T/T_g between 0.6 and 1, while the earthquakes with high periods resulted in T/T_g below 0.6. For each model, the calculated μ values were found to be consistent for each of those three ground period categories, hence μ has been averaged for each category. In order to determine R_μ, the relationships presented in Eqs. (10a) and (10b) were adopted. The resulting R_μ values for the 4 models are summarised in Table 8.

It should be noted that Model 1 is the same as the experimental Stage 2, and Model 2 is identical to Stage 3. Models 3 and 4 have the same bracing as Models 1, 2, respectively, but with lighter roof mass (steel cladding rather than roof tiles). Push-over tests hence give the same load-deflection curves for Models 1 and 3 (assuming there are no P-Δ effects) which would also be the same as that of Stage 2. Similarly, the push-over load-deflection curves for Models 2 and 4 are the same as for Stage 3. Hence, a comparison between the analytical results presented in Table 8 and the experimental results in Table 2 (based on Park's methods) can be made.

The ductility reduction factor (R_μ) based on the conventional methods (from Table 2) fails to recognise the influence of the structural period and the site soil type (conveyed by its predominant period). In other words, using a particular definition for yield displacement would yield a single value for R_μ. However, from Table 8, it is clear that R_μ decreases as the period of the structure decreases. R_μ is considered to be a minimum for a soft soil site and a maximum for a site on very stiff soil or rock. It should be noted that the experimental results agree with the analytical trends in that the system which has both the plasterboard and strap braces (Stage 3) has a lower value R_μ than that with plasterboard only (Stage 2). An investigation into the ductility of frames with strap braces only, has been presented by Barton (1997).

Thus, a single value for the ductility reduction factor to represent all scenarios may not be appropriate. The natural period of the structure and the ground period should both be considered in assessing the ductility reduction factor R_μ. A minimum value of 1.5 could be assumed as recommended by Lam *et al.* (1997), and increased as the natural period of the structure increases.

5. Overstrength

5.1. Components of overstrength

In light framed residential structures, lateral strength and stiffness are provided by a number of walls which are designed specifically to perform this function. However, there are other walls which are placed to function as partitions and not taken into account in the design process, but which still provide some lateral resistance. Hence, overstrength should be considered at two levels, an element level and system level. The element level refers to individual walls which may have an overstrength component plus a higher capacity due to the presence of boundary conditions such as corner wall returns and cornices. The system level evaluates the potential overstrength due to consideration of partition walls which are not considered in the design process. Hence, the overall overstrength factor (Ω) can be expressed as follows:

$$\Omega = \Omega_e \times \Omega_s \tag{14}$$

where Ω_e = Element overstrength
Ω_s = System overstrength.

Based on analytical modelling of the test house a range of values for the element overstrength factor is presented in this paper. This range covers possible construction scenarios for plasterboard lined residential steel wall frames.

5.2. Element overstrength

To estimate an element overstrength, consideration has been given to a typical isolated 2.4m long $\times$ 2.4m high, plasterboard clad steel frame, with no strap braces, namely wall (a) in Fig. 11. The lateral capacity of this wall was found to be 3.6 kN (Gad 1997). According to the current practice in determining design loads based on experimental results, a factor of safety of 2 is used when five or more walls are tested (Experimental Building Station, 1978). Hence, the nominated design load for this wall may be taken as 1.8 kN.

However, the plasterboard industry, in Australia, does not utilise the whole capacity of walls which are not intended as bracing walls. Plasterboard literature specifies a value of 0.5 kN per metre as the design load for walls clad on one side with standard 10 mm plasterboard fixed as a non-structural component. Hence, the adopted design capacity for this wall is 1.2 kN (based on 2.4m length). Therefore, the element overstrength factor for this wall equals 1.5 (1.8 divided by 1.2).

The value of 1.5 is considered to be the lower bound for element overstrength factor, because the contribution from the cornices and returns are not considered. The second scenario is to consider the contribution of the connection between the ceiling lining and the wall plasterboard via the cornice, wall (b) in Fig. 11. This connection was found to increase the capacity by approximately 10% (Gad 1997). However, the design capacity still does not change, remaining at 1.2 kN. Therefore, the element overstrength factor for this configuration is 1.65 (1.5 multiplied by 1.1).

The next scenario is to consider the possible contribution from return walls, wall (c) in Fig. 11. Including return walls was found to increase the capacity by approximately a factor of three (Gad 1997). Therefore, the element overstrength factor for this wall is approximately 5 (1.65 multiplied by 3).

Hence, the element overstrength factor is found to be highly sensitive to the wall configuration and what is considered as the design load. The problem lies in that the contributions from the various boundary conditions are not ordinarily considered. Therefore, according to current practice, the element overstrength factor could range from 1.5 to 5 depending on the wall configuration.

5.3. System overstrength

System overstrength incorporates strength contributions from items which have not been considered in the design process. These include the following:

- strength provided by clad wall sections above and below window openings,
- possible strength contribution from out-of-plane walls,
- strength from walls which are ignored in the design process, such as short partition walls. It should be noted that walls with length as small as 600 mm were found to have significant load carrying capacity (Gad 1997).

There is very little data on the system overstrength. Only a limited number of full scale tests have been conducted on houses, which give some appreciation for the system overstrength. Racking tests on a full scale timber house measuring 9.8×4.9 m revealed that the walls perpendicular to the load direction could carry as much as 25% of the applied load (Phillips *et al.* 1990). Tests on a full scale two storey Japanese wooden house suggested that the application of wall cladding to the areas above and below window and door openings in all the frames increased the lateral stiffness by 10 to 15% (Sugiyama *et al.* 1988). The researchers also found that the racking resistance measured in the full-scale house test was about 1.5 times that estimated by adding the contribution from all in-plane shear walls.

It is beyond the scope of this paper to quantify the system overstrength. However, it is anticipated that the element overstrength factor may be more significant since it incorporates the contributions of the main structural and non-structural components. As design procedures for walls are improved by including the boundary condition effects, the system overstrength factor would decrease. Ultimately, as the design of houses is refined, the over-capacity, that is historically assumed, is likely to decrease.

6. Response modification factor: Results and discussion

The structural response factor can be estimated by multiplying the ductility reduction factor (R_μ) by the overstrength factor (Ω). R_μ was analytically estimated to be between 1.5-3.5 depending on the natural period of the structure and the soil type (defined by the predominant site period). On the other hand, element overstrength (Ω_e) varied considerably, ranging from 1.5 to 5, depending on what is considered to be the design load. If the system overstrength factor (Ω_s) is conservatively given a value of 1.0 (assuming there is no system overstrength), R_f would range approximately from 2.3 to 17.5. In other words, to calculate the induced earthquake forces on a typical house, the elastic forces may be decreased by factor ranging between 2.3 and 17.5 to estimate the base shear for the corresponding non-linear system.

Obviously the above range of R_f is impractical and misleading. The variation in R_f primarily lies within the uncertainty of the actual strength of walls. Therefore, the values of R_μ and Ω should be distinguished and clearly specified. The ductility of the walls should not be confused with their strength. A system may have a high value of R_f but not necessarily high ductility. A large R_f value may only reflect a high degree of redundancy and consequently high overstrength. In other words, R_f may not reflect the level of ductility as commonly believed.

If boundary conditions (such as return walls, skirting boards and cornices) are considered in the design of walls, the overstrength factor should be decreased accordingly, hence, R_f would be smaller. Reducing R_f would result in higher imposed earthquake loads. But, because the boundary conditions are included, the structure would have higher design capacity. So, while R_f is reduced, the design capacity is increased, and vice versa. It is vital to understand the components of R_f so that overstrength is not considered twice. In other words, determining R_μ and Ω separately would avoid situations when the boundary conditions (or non-structural components) are included in the design process and then a high overstrength factor is also assumed.

It is therefore recommended that R_μ and Ω should be separately determined, which would lead to more reliable estimates of R_f for incorporation into design codes. The methodology provided in this paper provides rational values for R_μ. The overstrength factor may vary as the design of residential structures is refined and more of the so-called non-structural components are included directly in the design procedure. As the architecture of houses changes, adapting to new trends of life styles and building regulations, so the function of structural and non-structural elements alters accordingly. As the industry and building codes opt for more refined design procedures and more efficient use of resources, the over-capacities which have been present and evident in the past will be substantially reduced to produce more cost efficient structures. This reduction in over-capacity is particularly true for houses as their design progresses towards engineered structures. Design refinement is also true for other structures evidenced by observations post Australia's 1989 Newcastle earthquake that reinforced concrete structures have lower over-capacity than historically assumed, EEFIT (1991).

To reflect the practice of refining designs it is recommended that the overstrength and ductility reduction factors should be separated in earthquake codes. The codes could additionally provide values for R_μ dependent on the natural period of the structure and the type of soil. The overstrength factor should reflect the current state of design practice and should decrease as the so-called non-structural components are included in the design process to increase the design capacity.

7. Conclusions

Using conventional methods for calculating the ductility demand factor μ, it was found that this key parameter varies substantially depending on the definition of the yield displacement for the same load-deflection curve. The difficulty in defining the yield displacement is due to the fact that there is not a suitable equivalent elasto-plastic model to fit the non-linear load-deflection curves. Plasterboard clad frames exhibit highly non-linear behaviour with stiffness degradation and slack development which all lead to a change in period during an earthquake event.

Based on nine earthquakes and four wall models, a strong relationship was found between the initial period (T_i), ground period (T_g), and equivalent elastic period (T). The four models showed similar linear trend between T_i/T_g and T. As T_i/T_g increased, T decreased. Based on the combinations of models and earthquake records used in this study an approximate relationship was formulated to predict T.

With a reliable equivalent elastic period for each model and soil category, μ was calculated based on the equal displacement approach. Thus, the observed variation in μ using this approach can be explained in rational engineering terms, unlike the situations when conventional methods are used.

R_μ was consequently estimated for plasterboard clad wall frames using the recommendation by Lam *et al.* (1997). For the four models adopted in this investigation, representing different forms of residential construction, R_μ was found to vary between 1.5 and 3.5 depending on the initial period of the structure and the ground period.

For residential structures, the overstrength factor has been divided into two components, namely those relating to element and system. The element overstrength reflects the overstrength in a wall due to its boundary conditions, and system overstrength is introduced to account for walls and sections not considered in the design process. If the boundary conditions (such as return walls and ceiling cornices) are taken into account in the design phase, then there would be losses and gains. The element overstrength factor would be low and hence the reduction of the elastic forces would be small. But the system would gain higher design capacity. Similarly, if the boundary conditions are not included, the design capacity would be less, but the overstrength would be high. However, including the boundary conditions does not only increase the design capacity for earthquakes but wind loads as well which do not rely on an overstrength factor. It should be clearly highlighted to designers not to take advantage of the boundary conditions twice, that is having high design capacity and high overstrength factor.

For typical Australian design and construction practices, the element overstrength may range between 1.5 and 5 depending on which components of the boundary conditions are included in design. The system overstrength was not quantified due to lack of data. However, it is anticipated that the element overstrength may be the more critical of the two factors as it includes most of structural and non-structural contributions.

With regard to codified seismic design practice referred to above, it is recommended to separate the ductility reduction factor and the overstrength factor to avoid misconception about the system performance. As the design of residential structures is refined, the over-capacity that is historically recognised will be reduced. Hence, the ductility and overstrength components should be identified separately so that designers can predict the behaviour of the structure and optimise the design accordingly.

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References

Barton, A.D. (1997), "Performance of cold formed steel frames when subjected to earthquake loading", Ph.D thesis, Department of Civil and Environmental Engineering, The University of Melbourne, Australia.

Carr, A.J. (1996), *RUAUMOKO*. Dept. of Civil Engineering, Canterbury University. New Zealand.

Cooney, R.C. and Collins, M.J. (1988), *A Wall Bracing Test and Evaluation Procedure*, Building Research Association of New Zealand (BRANZ). Technical Paper P21. Judgeford.

EEFIT (Earthquake Engineering Field Investigation Team) (1991), *The Newcastle, Australia Earthquake of December 1989*, ed. Pappin J.W., Chandler, A.M. and Couburn, A., Institution of Structural Engineers, London.

Elghadamsi, E.F. and Mohraz, B. (1987), "Inelastic earthquake spectra", *J. Earthquake Eng. Struct. Dyn.*, **15**, 91-104.

Experimental Building Station. (1978), *Guidelines for testing and evaluation of products for cyclone-prone areas*. Technical record 440, Recommendations of a workshop held during July 1977, Department of Construction, James Cook University, Australia.

Gad, E.F. (1997), "Performance of Brick-Veneer cold-formed steel-framed domestic structures when subjected to earthquake loading", Ph.D thesis, Department of Civil and Environmental Engineering, The University of Melbourne, Australia.

Gad, E.F., Duffield, C.F., Stark, G. and Pham, L. (1995), "Contribution of non-structural components to the dynamic performance of domestic framed structures", *Proceedings of the Pacific Conference on Earthquake Engineering, The University of Melbourne*, Australia, 177-186.

Gad, E.F., Duffield, C.F., Mansell, D.S. and Stark, G. (1997), "Modelling of plasterboard lined domestic steel frames when subjected to lateral loads", *Proceedings of the 15th Australasian Conference on the Mechanics of Structures and Materials, University of Melbourne*, Australia, 331-337.

Gad, E.F., Duffield, C.F., Chandler, A.M. and Stark, G. (1998), "Testing of cold-formed steel-framed domestic structures", *Proceedings of the 11th European Conference on Earthquake Engineering*, Paris, France.

Gad, E.F., Duffield, C.F., Hutchinson, G.L., Mansell, D.S. and Stark, G. (1999), "Lateral performance of cold-formed steel-framed domestic structures", *Eng. Struct.*, **21**(1), 83-95.

International Conference of Building Officials (1994), *Uniform Building Code*. Whittier. California.

King, A.B. and Lim, K.Y.S. (1991), Supplement to P21: An evaluation method of P21 test results for use with NZS 3604:1990, Building Research Association of New Zealand (BRANZ), Technical Report, Judgeford.

Lam, N.T.K., Wilson, J.L. and Hutchinson, G.L. (1995), "The independence of ductility demand on the frequency characteristics of earthquake ground motion", *Proceedings of the 14th Australasian Conference on the Mechanics of Structures and Materials*, University of Tasmania, Hobart, 290-295.

Lam, N.T.K., Wilson, J.L. and Hutchinson, G.L. (1997), "Seismic strength and ductility provisions for building structures in Australia", *Proceedings of the Technical Conference of the New Zealand National Society for Earthquake Engineering*, Wairakei, New Zealand, 96-103.

Mahin, S.A. and Bertero, V.V. (1981), "An evaluation of inelastic seismic design spectra", *J. Struc. Eng. ASCE*, **107**(9), 1777-1795.

Miranda, E. (1993), "Evaluation of site-dependant inelastic seismic design spectra", *J. Struc. Eng., ASCE*, **119**(5), 1319-1338.

Newmark, N.M. and Hall, W.J. (1982), *Earthquake Spectra and Design.* Engineering Monograph, Earthquake Engineering Research Institute, Berkeley, California.

Park, R. (1989), "Evaluation of ductility of structures and structural assemblages from laboratory testing", *Bulletin NZ National Society Earthquake Eng.,* **22**(3), 155-166.

Phillips, T.L., Itani, R.Y. and McLean, D.I. (1993), "Lateral load shearing by diaphragms in wood framed buildings", *J. Struc. Eng., ASCE,* **119**(5), 1556-1571.

Priestley, M.J.N. (1997), "Displacement-based seismic assessment of reinforced concrete buildings", *J. Earthquake Eng.,* **1**, 157-192.

Standards Association of Australia (1993), *Minimum Design Loads on Structures, Part 4: Earthquake Loads, AS1170.4.* Sydney, Australia.

Standards Association of New Zealand (1990), *Code of Practice for Light Timber Frame Buildings Not Requiring Specific Design, NZS3604.* Wellington, New Zealand.

Standards Association of New Zealand (1992). *Code of Practice for General Structural Design and Design Loadings for Buildings, NZS4203.* Wellington, New Zealand.

Uang, C.M. (1991), "Establishing R and C_d factors for building seismic provisions", *J. Struct. Eng. ASCE,* **117**(1), 19-28.

Sugiyama, H., Uchisako, T., Andoh, N., Arima, T., Hirano, S. and Nakamura, N. (1988), "Comparison of Lateral Stiffness of Frame Obtained From Full-Scale Test and That Estimated by Racking Tests in Japanese Type of Wooden Frame Construction", *Proceedings of the International Conference on Timber Engineering,* Forest Products Research Society, Madison, Wis., 804-810.