

LATERAL BEHAVIOUR OF LIGHT FRAMED WALLS IN RESIDENTIAL STRUCTURES

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SUMMARY

In attempt to use conventional methods to calculate the ductility demand factor (μ) for light-framed clad residential structures, it was found that this key parameter varies substantially depending on the definition of the yield displacement. The difficulty in defining the yield displacement is due to the fact that there is not a suitable equivalent elasto-plastic model to fit highly non-linear load-deflection curves. Typical plasterboard clad wall frames exhibit such high non-linearity and stiffness degradation.

Based on experimental and analytical results, a strong relationship was found between the initial period (T_i), ground period (T_g), and equivalent elastic period (T). Four different analytical models showed similar linear trend between T_i/T_g and T . As the ratio T_i/T_g increased, T decreased. With a reliable equivalent elastic period 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. Consequently, the ductility reduction factor (R_μ) was calculated for different wall configurations.

For some typical wall design and construction practices, the overstrength was estimated to range between 1.5 and 5 depending on which components of the wall boundary conditions are included in the design process.

Having established the ductility reduction factor and the overstrength factor, the response modification factor, which is used in most seismic codes, can be evaluated. However, presenting the ductility reduction and overstrength factors separately may be more appealing as the designer can better predict the behaviour of the structure and more appropriate structural optimisation can be achieved.

INTRODUCTION

Light framed structures are very common in residential construction in many parts of the world including Australia, North America and Japan. These residential structures generally take the form of detached single or double story houses for single family dwelling. The frames are made from either timber or cold formed steel. In Australia, the exterior wall cladding is brick veneer and the interior is plasterboard lining. The roof cladding is either cold formed steel sheeting or tiles (concrete or terracotta).

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 *et al.* 1999a & b). 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.

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This paper focuses on two significant parameters used in ultimate strength seismic design of structures, namely ductility and overstrength. Current methods for determining the ductility of conventional structures are briefly reviewed and their applicability to clad residential wall frames are examined. 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.

DESIGN FACTORS

Response modification factor

Conventional earthquake design procedures adopted in most earthquake standards 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:

$$R_f = \frac{S_e}{S_d} \quad (1)$$

where S_e = Elastic Strength & S_d = Design Strength.

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, along with experience from numerous moderate and severe earthquakes. There are no specific R_f factors for structural systems used in residential construction in the Australian earthquake code (Standards Association of Australia, 1993), 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 response modification factor is widely accepted as the product of two components, namely ductility reduction (R_μ) and overstrength (Ω). Thus R_f is expressed as:

$$R_f = R_\mu \times \Omega \quad (2)$$

R_μ is defined as:

$$R_\mu = \frac{S_e}{S_y} \quad (3)$$

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:

$$\Omega = \frac{S_d}{S_y} \quad (4)$$

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

Ductility reduction factor

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

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

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

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

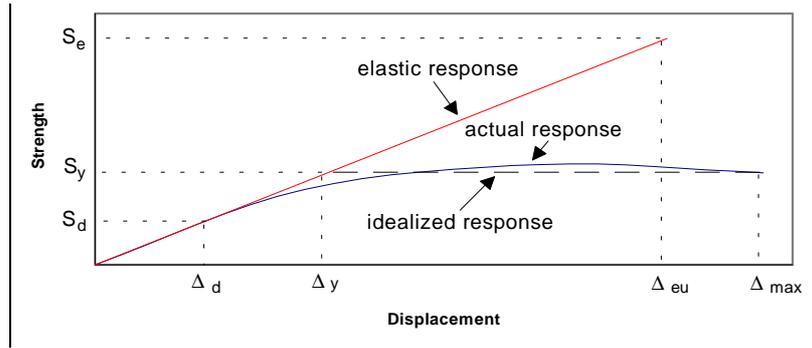


Figure 1: General structural response.

Eq's (5)-(7) are based on the equal displacement, equal energy and equal acceleration theories, respectively. The ductility demand factor μ is defined by Eq. (8):

$$\mu = \frac{\Delta_{\max}}{\Delta_y} \quad (8)$$

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

These Newmark and Hall expressions have been widely accepted in the design of ductile structures. However, the relationships do not take into account ground motion frequency content which was found to be significant in the determination of the ductility demand. 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):

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

where T is the natural period of an equivalent Single Degree Of Freedom (SDOF) model and T_g is the 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. 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. 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:

$$R_\mu = \mu \quad T > 0.6 T_g \quad (10a)$$

$$R_\mu = 1.5 \quad T \leq 0.6 T_g \quad (10b)$$

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.

Ductility demand factor

In order to evaluate the ductility demand factor μ , the yield and maximum displacements have to be defined. 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 world-wide. These definitions are presented in Figs. 2a and 2b 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.

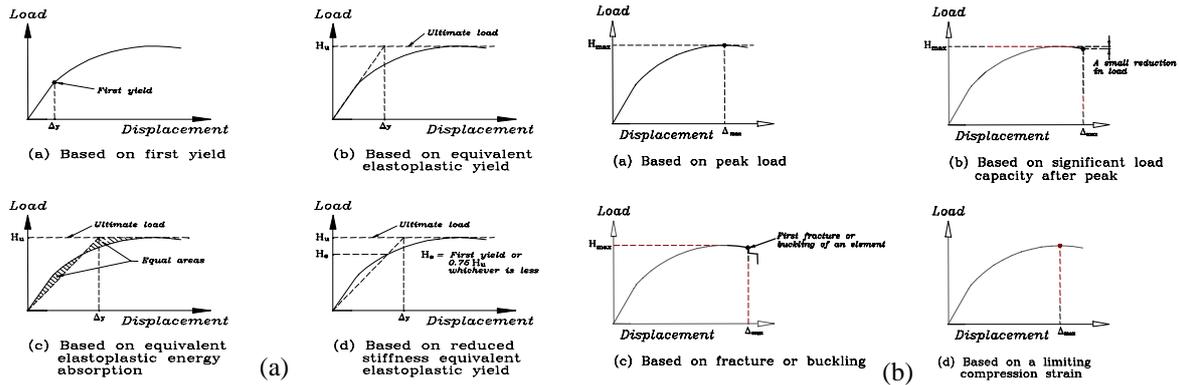


Figure 2: Alternative definitions according to Park (1989) for; (a) yield displacement and (b) maximum displacement.

Overstrength factor

The overstrength factor is intended to take into account possible sources that may contribute to strength exceeding its nominal or idealised value. 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. 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.

EVALUATION OF DUCTILITY REDUCTION FACTOR BASED ON ESTABLISHED METHODS

Brief description of the experimental programme

A one-room-house “test house” was adopted as a test specimen. It measured 2.3m x 2.4m x 2.4m high and was constructed from full scale components as shown in Fig. 3. The test house simulates a section of a rectangular house. The test house was built on a two degree of freedom shaking table at The University of Melbourne, Australia. It 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. 4. Further details regarding the test house and the loading regimes have been presented in Gad et al. (1999a).

Table 1: A summary of all experimental stages related 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.

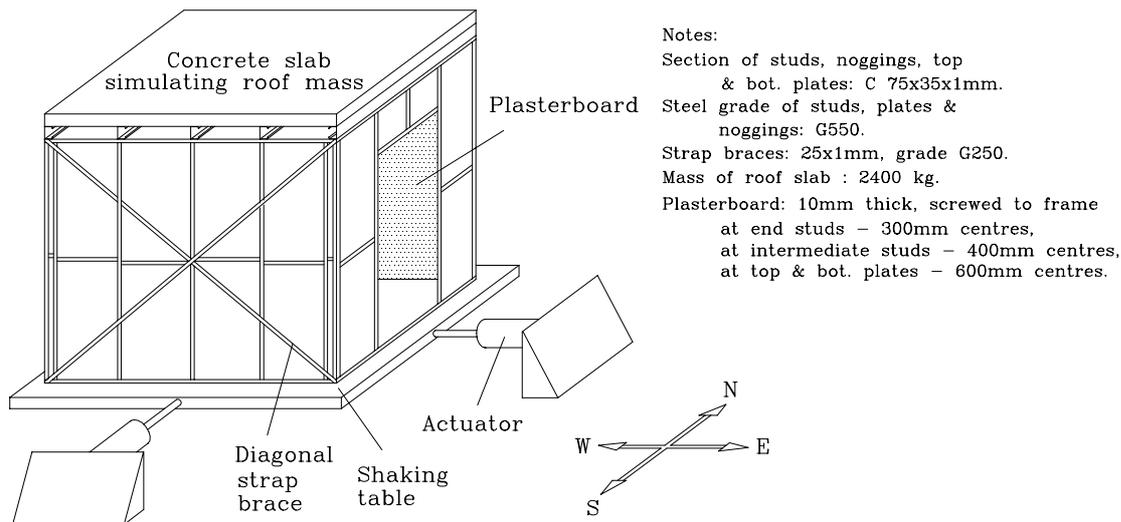


Figure 3: Diagram of test house and shaking table.

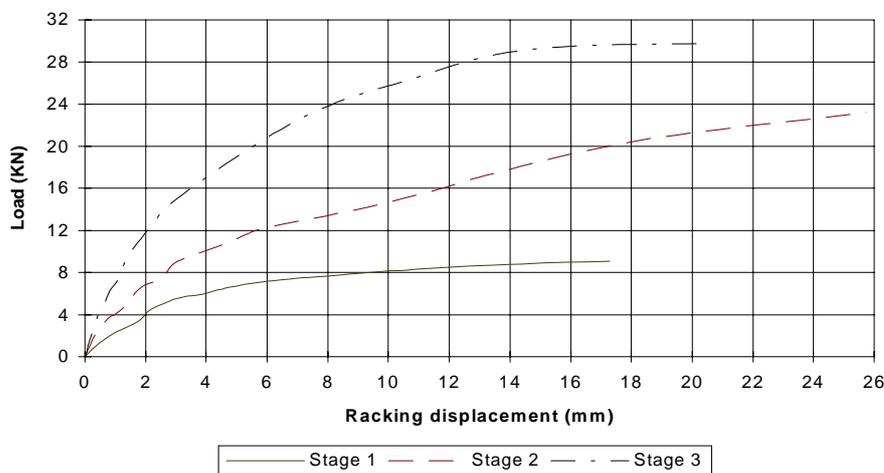


Figure 4: Load-deflection curves for different stages of the test house.

Evaluation of ductility using conventional methods

As shown in Fig. 4, the load deflection curves for framed residential structures are highly non-linear with no distinct yield point. For plasterboard clad walls, the first form of yield occurs at a racking displacement of less than 5.0 mm. This form of yield takes place in the screw connections between the plasterboard and the frame. Not all the connections yield at the same time, but the yield is progressive, starting from the bottom screws. For frames with diagonal strap bracing, it was found that the apparent kink in the load deflection is not due to yielding of the braces but is directly related to the initial tension in the strap which is arbitrary (Barton, 1997).

It should be noted that the theoretical definitions of yield displacement are not only restricted to those proposed by Park (1989). 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 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. It should be noted that researchers in New Zealand are currently investigating the use of system identification with time history analyses and displacement based spectra to evaluate wall capacity without the need to establish the ductility demand (Deam 1999).

Using the various techniques presented by Park (1989) and King and Lim (1991) the calculated R_{μ} varies significantly. For Stages 1, 2 and 3, the ranges are 2.2-6.4, 1.7-6.6 and 1.9-6.8, respectively. Given this 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, 1998). Details of these models are presented in Gad (1997).

ANALYTICAL APPROACH TO EVALUATE DUCTILITY PARAMETERS

Equivalent elastic period

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

In attempt to find the trend between the initial period (T_i) and the equivalent elastic period (T), nine earthquake records and four different wall models were considered in this investigation. The earthquake records used were selected to cover various possible scenarios. Three records (EQ1, 2, &3) were chosen with low T_g (less than 0.12 sec, rock sites), three (EQ4, 5, &6) with medium T_g (between 0.18 and 0.24sec, stiff to intermediate soils) and three records (EQ7, 8, & 9) with high T_g (more than 0.5sec, soft soils). The details of all nine records are listed in Table 3.

Table 3: Details of earthquakes used in defining the equivalent elasto-plastic system.

Label	Richter Magnitude	Peak ground Acceleration (m/s/s)	T_g (sec)	Location, recording station, direction, date
EQ1	6.6	3.63	0.12	Northridge - USA, St Monica City Hall, 360°, 1994
EQ2	6.0	1.04	0.07	Saguenay - Canada, Chicoutimi-Nord, S34W, 1988
EQ3	6.4	13.2	0.10	Nahanni - Canada, Iverson, Trans., 1985
EQ4	7.3	7.40	0.24	Tabas - Iran, Tabas, Long., 1978
EQ5	7.4	2.23	0.18	Honshu - Japan, Ofunato Harbour Works, 1978
EQ6	5.4	3.80	0.20	San Salvador, Urban Construction Inst., 90°, 1986
EQ7	6.6	3.06	0.51	Imperial Valley - USA, El-Centro, NS, 1940
EQ8	5.6	4.98	0.59	Parkfield - USA, Array No.2, N65E, 1966
EQ9	5.4	3.92	0.80	San Salvador, National Geographical Inst, 180°, 1986

The four analytical 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 4. 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 periods for the 36 runs are listed in Table 5.

Table 4: 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 5: The amplification factors and equivalent elastic periods for the four models.

EQ record	Model 1		Model 2		Model 3		Model 4	
	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

Based on these results the ratio of T_i to T_g has been plotted versus the elastic period for each model, as shown in Fig. 5. 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. 5. 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. Therefore, for design purposes a set of curves could be developed in a similar fashion based on the analysis of more models and earthquakes.

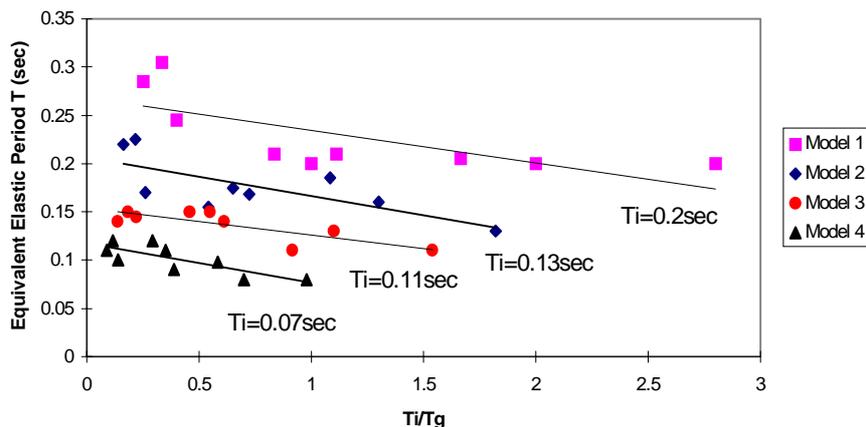


Fig. 5: Results from time history analyses and fitted linear functions for the four analytical models.

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

$$k = \frac{4m\pi^2}{T^2} \quad (12)$$

The yield displacement (Δ_y) 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. 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 6.

Table 6: 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 : $T/T_g \geq 1.0$	3.5	3.0	2.8	2.3
Medium T_g : $0.6 < T/T_g < 1.0$	2.6	2.0	1.6	1.5 ³
High T_g : $T/T_g \leq 0.6$	1.5	1.5	1.5	1.5

EVALUATION OF OVERSTRENGTH

Overstrength in residential structures may be considered at two levels, an element level and system level. The element level refers to individual walls which may have an overstrength due to the over capacity provided by boundary conditions such as corner wall returns and cornices which are not considered in the design process. The system level evaluates the potential overstrength due to consideration of partition walls which are not considered to be non-structural elements. Hence, the overall overstrength factor (Ω) can be expressed as the product of element strength (Ω_e) and system overstrength (Ω_s). In this paper only the element overstrength is evaluated and discussed in detail due to limitations of paper length. Based on analytical modelling of the test house a range of values for the element overstrength factor is presented in this paper.

To estimate an element overstrength, consideration has been given to a typical isolated 2.4m long x 2.4m high, plasterboard clad steel frame, with no strap braces, namely wall (a) in Fig. 6. The lateral capacity of this wall was found to be 3.6 kN (Gad et al. 1999b). 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. 6. This connection was found to increase the capacity by approximately 10% (Gad et al. 1999b). 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. 6. Including return walls was found to increase the capacity by approximately a factor of three (Gad et al. 1999b). Therefore, the element overstrength factor for this wall is approximately 5 (1.65 multiplied by 3).

This demonstrates that the element overstrength factor is 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

which 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.

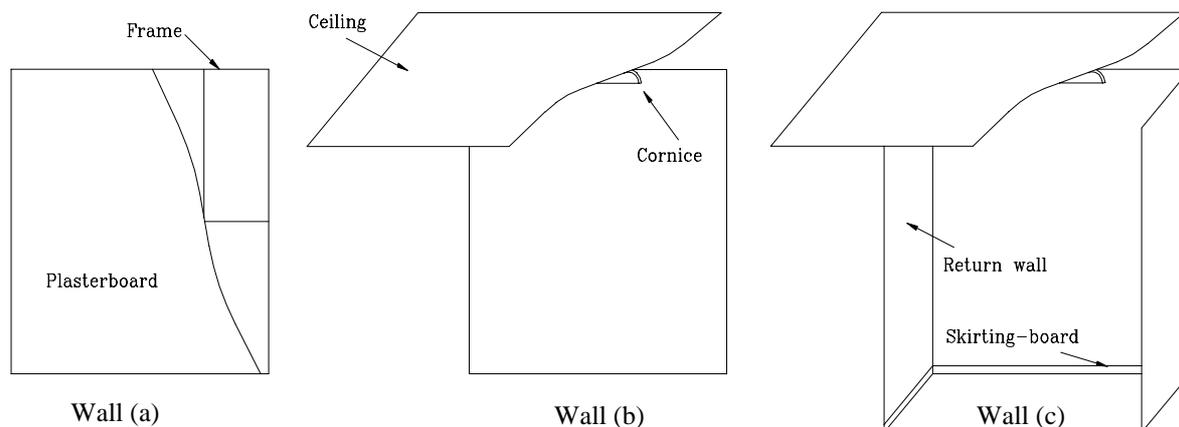


Fig. 6: Plasterboard clad walls with different boundary conditions.

CONCLUSIONS

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.

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.

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 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.

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