RATING SEISMIC BRACING ELEMENTS FOR TIMBER BUILDINGS

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SUMMARY

The seismic response of light timber framed systems, like those used in NZS 3604 buildings, is not well represented by the classical elastoplastic hysteresis rule used for time-history analysis. A new simple hysteresis rule has been developed which is easily fitted to reverse-cyclic test results. This rule is compared with the responses from recent pseudo-dynamic tests performed on a series of walls lined with plasterboard. A 'black-box' computer program has been developed to automatically rate the seismic capacity of bracing elements tested using reverse-cyclic loading by simulating their response to a range of design earthquakes. The ratings obtained using the new model and computer program are compared with the ratings calculated using the current New Zealand P21 rating method. A simple revision to the P21 rating method is proposed which gives ratings that are much closer to those obtained experimentally and analytically.

INTRODUCTION

A significant portion of New Zealand buildings are designed and constructed using the Timber Framed Buildings standard, NZS 3604 [SNZ, 1999]. This standard is predominantly used by those from outside the engineering profession so simplified methods are used for seismic design. Seismic design is carried out using the fundamental structural design equation 'resistance ≥ demand'. The bracing demand is estimated from a table of demands per square metre for specified construction styles and geographic regions or earthquake zones. These are multiplied by the floor or roof plan area to obtain the total demand in each of two perpendicular directions.

The required resistance is then provided by bracing elements which are placed into the building until the sums of the bracing element ratings match or exceed the demands in each direction. The bracing elements are distributed around the building in a prescribed manner to reduce the possibility of torsional failure. Bracing demands and ratings are both measured in Bracing Units (20 Bracing Units ≡ 1 kN).

BRANZ recently conducted a review [Herbert and King, 1998] of the 1979 'P21' test and evaluation procedure [Cooney and Collins, 1979] and its 1991 supplement [King and Lim, 1991] that are currently used to assess the resistance provided by bracing elements. A survey of the test and evaluation procedures used in other countries revealed some variations on the P21 test procedure but there is no commonly agreed procedure. The evaluation procedures were limited to those which provided characteristics of the element which were able to be used for time-history analysis using either elastic or elasto-plastic elements.

Revised test and evaluation procedures were developed to provide more accurate ratings using four steps:

1. Conduct a monotonic racking test with one specimen to determine its ultimate strength.
2. Subject three more identically constructed specimens to a reverse-cyclic racking test.
3. Fit an analytical model to the responses of these specimens and use this to determine the mass which can be restrained by the specimen for a suite of design level earthquakes.

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4. Conduct a pseudo-dynamic verification test with a fifth specimen to verify that it is capable of restraining the rated mass. This test is primarily used to ensure that the analytical model characterises the test specimen sufficiently accurately and will be unnecessary once the analytical model is verified.

This paper describes the revised evaluation procedure comprising steps 3 and 4 above [Deam and King, 1996]. The monotonic and cyclic test procedures (steps 1 and 2) are only briefly described.

RATING PROCEDURES

The current 'P21' method of rating the resistance of bracing elements [King and Lim, 1991] uses a reverse-cyclic test. A typical test specimen response is illustrated in Figure 1. Two of the three Figure 1 resistance forces (ie R and S) are used to calculate the earthquake rating, the other is used for a wind resistance rating. All three resistances are averaged over 3 identically constructed specimens to reduce the variability of the rating.

The earthquake rating is derived from the residual resistance, R (see Figure 1), modified to account for the specimen 'ductility' and converted to Bracing Units (1 kN = 20 Bracing Units) using [King and Lim, 1991]:

$$BU_{qu} = 20 \times K_4 \times R \tag{1}$$

The earthquake rating is reduced using K_4 when the specimen 'ductility', μ, is less than $\mu = 4$ which was used to derive the loads tabulated in NZS 3604 [SNZ, 1990]. The ductility is defined as the ratio of the maximum test displacement, y, to the displacement, d, at half of the peak load. The rating is also reduced to a multiple of the service resistance, S, when the ratio of serviceability to ultimate resistance is smaller than the ratio of serviceability to ultimate load assumed when deriving the bracing tables within NZS 3604. The bracing rating is usually published as a rating per unit width of panel, rounded to the nearest 5 Bracing Units.

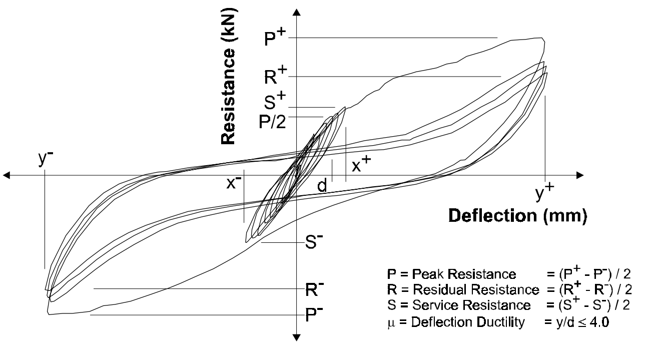

Figure 1. Test specimen response to the 'P21' test procedure [Cooney and Collins, 1979].

The 'P21' test and evaluation procedures [Herbert and King, 1998] have been reassessed recently after a review of test and evaluation procedures used in other countries. As described in the introduction, the test procedure has been modified [King and Lim, 1991] to include an additional specimen. This is used to assess whether there are any brittle failure modes. A second modification includes additional cycles (between x and y in Figure 1) to allow the specimen to be characterised more accurately and to allow the specimen to be rated at a smaller deflection if it fails before y is reached.

The specimen used to assess brittle failure modes is tested using a monotonic racking test. It is also used to evaluate the service displacement and to ensure that the strength of the overturning restraints and other components are not likely to fail during wind loading or a large earthquake acceleration pulse. This test is not necessary when this information is known from previous tests or is able to be calculated. The test procedure has since been amended [Herbert and King, 1998], with additional cycles to large displacements, in order to assess its response with stiff and weak restraints. The test protocol is described elsewhere [King and Deam, 1998] so will not be fully described here.

The proposed evaluation method is considered to be more rigorous than the original [Cooney and Collins, 1979] which rated the specimen from its resistance during the fourth cycle to the estimated displacement (see Figure 1). The proposed method is based upon that developed by Dean [Dean et al, 1987]. This comprised fitting an analytical model and using the model to generate displacement spectra for a number of earthquakes. The mass able to be restrained by that element is then assessed from the least favourable design spectrum. Most of the process is carried out using a computer program which conducts the analysis automatically. The complete procedure and its accompanying computer program are described later.

ANALYTICAL MODEL

Most bracing systems degrade when subjected to reverse cyclic deformation. A number of methods have been used to fit the classic elastoplastic approximation (Figure 2b) to actual hysteresis loops of steel and concrete elements [Park, 1989] since in real elements there is no distinctive point at which the onset of plastic deformation (yielding) occurs. The response of degrading elements (eg Figure 2) is even more poorly characterised by the elastoplastic approximation, particularly at large displacements.

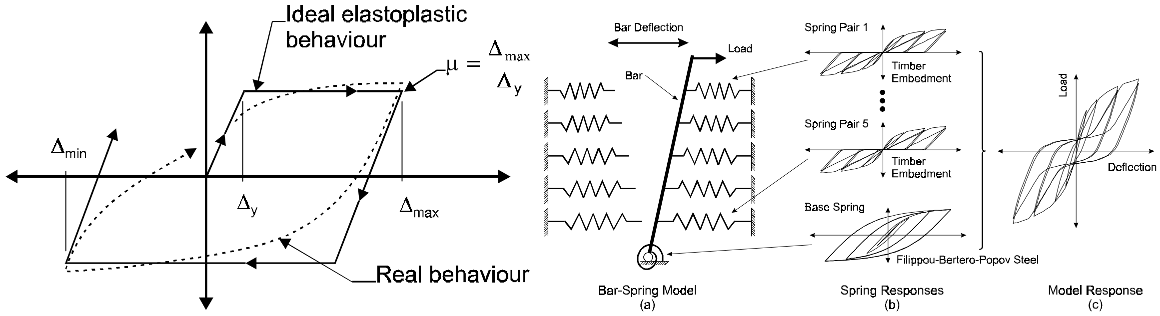

Figure 2 Elasto-plastic hysteretic behaviour approximation [Park, 1989] and Revised bar-spring model.

Many models have been developed to approximate the pinched response of timber elements. A simple method of generating realistic responses was developed by Dean [Deam, 1997] to eliminate the need for a complex set of mathematical rules. Dean's bar and spring model was revised as part of the current study (Figure 2) to simulate the response of a nail bearing on a timber substrate.

The revised model uses five springs (Figure 2b) to model the embedment of the nail into a timber substrate. These springs develop slackness and their reloading stiffness decreases as they deform to give a more realistic response at small displacements. The nail is represented as a rigid bar but has an inelastic hinge at the base (modelled using the Fillipou-Bertero-Popov steel model [Filippou et al., 1983]). The model produces a very realistic response (Figure 2c) with only seven characteristic parameters.

AUTOMATED RATING PROCEDURE

Software has been developed to automate the process of characterising the test specimen responses and hence assessing specimen response to the level of earthquake ground motion required by the NZ Loadings Standard, NZS 4203 [SNZ, 1992]. Non-linear time-history analyses are performed within the software, named BraceRate, using acceleration records from a suite of earthquakes. Natural earthquake records were modified to generate acceleration spectra similar to the uniform risk spectrum for normal soils given in NZS 4203. The time-history analysis is carried out for a single degree-of-freedom oscillator using the constant acceleration step-by-step method [Clough and Penzien, 1975]. A relatively small time-step (0.005 sec) is required for the analysis because the stiffness of the bar-spring model (Figure 2) is highly non-linear. Hysteresis loops, load-time records and displacement-time records are able to be plotted on the screen.

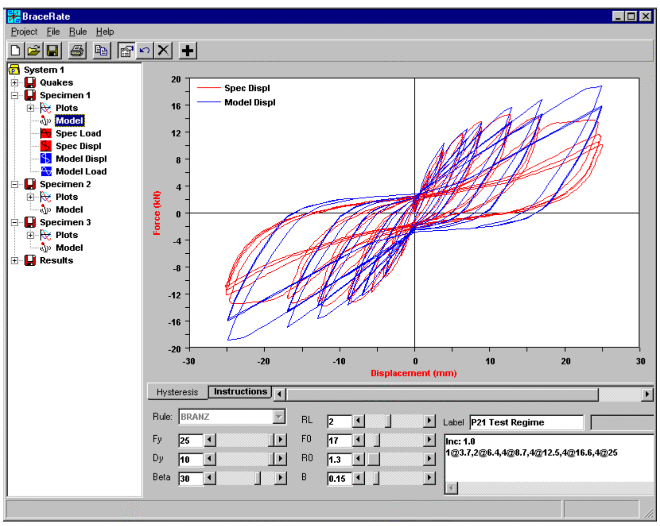

Figure 3. Fitting the bar-spring model to a test record using the BraceRate software.

Once the model has been fitted to each of the three specimens, a displacement spectrum is generated for each specimen subjected to each earthquake. The spectra are generated by incrementally increasing the mass in the single degree-of-freedom oscillator until the maximum displacement recorded during the time-history analysis exceeds a predefined displacement. These are plotted (Figure 4) with the displacements on the x-axis and the mass (in place of the period) on the y-axis once the analysis is completed. The mass able to be restrained by the specimen is then read off the plot at the maximum reliable test displacement. Further investigation is required to establish whether the lowest or the average mass is most appropriate. BraceRate uses a suite of five earthquake accelerograms which were modified [Clow et al., 1995] so their elastic response spectra were similar to the elastic design response spectrum given in NZS 4203 for normal soils.

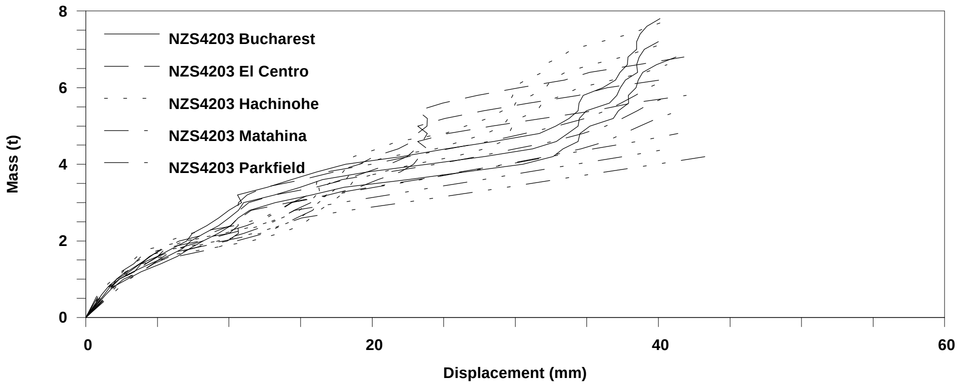

Figure 4. 'Displacement spectra' generated by the BraceRate software for the Figure 3 specimen.

Earthquake design loads in NZS 3604 [SNZ, 1999] were derived from a draft version of the loadings standard NZS 4203 [SNZ, 1992] for a building with a period of 0.4 seconds and ductility $\mu = 4$ [Thurston, 1994]. The seismic design force, V, on a building of mass M is given by (from equation 4.6.2 (a) of NZS 4203):

$$V = C_h(T,\mu) \times S_p \times R \times Z \times L_u \times M \times g \quad (2)$$

The other symbols used above are defined in equation 4.6.2 (a) of NZS 4203. Setting these to the values for NZS 3604 buildings, namely $R = L_u = 1$, $S_p = 0.67$, $g = 9.81$ m/sec^2, $Z = 1.2$ (Wellington) and $C_h(T,\mu) = 0.27$ (for $T = 0.4$ seconds, $\mu = 4$ and "intermediate" soil), gives:

$$V = 2.13\ M \quad (3)$$

If the seismic design force is distributed among the bracing elements in proportion to their strength (ie they are rated at the same displacement), the building force V and mass M in equation 3 can be replaced by the bracing element force, v, and a mass, m, that is able to be restrained by that element. Modifying equation 3 and converting the force, v, from kN to BU (Bracing Units) using 1 kN = 20 BU as defined previously, allows the bracing rating to be calculated from the mass m generated by BraceRate (ie from Figure 4) using:

$$v = 43\ m \quad (4)$$

These simplistic assumptions are adequate for investigating the differences between the original and revised rating methods. They need further investigation because the individual element responses will not be the same as the response of the whole system. Moreover, no account is made of the torsional response of the building.

PSEUDO-DYNAMIC TESTING

Testing is acknowledged as the most accurate method of establishing how a structural system performs during an earthquake. Pseudo-static reverse cyclic tests have been used for many years to assess the deformation capacity of structural systems ranging from sub-assemblages to complete buildings. Shake-table tests have been used to both observe and verify system response under more realistic conditions. On-line, computer controlled or pseudo-dynamic testing is increasingly being used in Japanese and US laboratories to simulate the inertial response of a shake-table test.

In a pseudo-dynamic test, the test specimen is used in place of a numerical model in a dynamic time history analysis. The test equipment is similar to that used for a reverse cyclic test, with the addition of an interface between the physical specimen and the numerical analysis. The pseudo-dynamic test offers several significant advantages over the physically equivalent shake table-test:

- the duration of the test may be extended to allow more detailed observation of the specimen;
- the mass is simulated which makes it simpler to vary and reduces the danger for those observing the test;
- the damping is simulated so it may be used to model damping from sources external to the specimen;
- the fidelity of the reproduction of the ground motion is improved and specimens may be physically much larger because there is no shake-table to move at the dynamic rate; and
- the specimen may be tested so that it responds as though it is within a complete structure. The remainder of the structure may be modelled analytically or even simultaneously tested in another laboratory.

An adaptation of the pseudo-dynamic test has been developed at BRANZ to test degrading systems. The BRANZ test moves the specimen continuously throughout the test to avoid relaxation. The new method has been implemented for single test specimens and is shown to correlate very well with analytical results for linear systems.

MODEL VERIFICATION BY PSEUDO-DYNAMIC TESTING

Pseudo-dynamic tests were conducted on three full scale 6.4 m long wall specimens to assess the proposed rating system and the BRANZ analytical model. One wall was typical of internal wall construction and the other two of exterior wall construction. The internal wall specimen had two internal doorways, a segment of exterior wall at one end and segments of interior walls at the other end and adjacent to the central doorway. The exterior wall specimens each had two window openings, exterior wall segments at each end and an interior wall segment. The test specimens were identical to those tested by Thurston [Thurston, 1993] except the internal wall had additional metal straps attaching the door trimmer studs to the foundation beam to make the wall strength in the weak direction similar to its strength in its strong direction.

An early version [Deam, 1997] of the BraceRate software was used to assess the seismic masses that the test specimens were capable of restraining. This matched Dean's bar-and-spring [Deam, 1997] model to the test specimen responses reported by Thurston [Thurston, 1993]. The mass was then assessed at a displacement of 16 mm (Table 1) using two earthquake records.

Table 1 Specimen Ratings based on reverse cyclic tests.

Specimen	Ultimate Strength (kN)	Initial Stiffness (kN/mm)	Earthquake Record	Rated Mass (kg)
Interior	27	5.7	1.25 × El-Centro 1940	5000
Exterior	14	3.0	1.25 × El-Centro 1940 NZS 4203 Matahina	3500 3500

The BRANZ analytical model was matched to the cyclic test specimen responses using the BraceRate software. The analytical model matched the cyclic response of the interior wall reasonably accurately up to 16 mm displacement (Figure 4) but was unable to match the exterior wall response for negative displacements or beyond 8 mm positive displacement.

The displacement-time response of the pseudo-dynamic test specimen was very closely matched by a time-history analysis using the BRANZ model as shown in Figure 5.

The excellent agreement between the pseudo-dynamic test specimen responses (shown as points) and the mass-displacement responses predicted by the BRANZ model is illustrated in Figure 6.

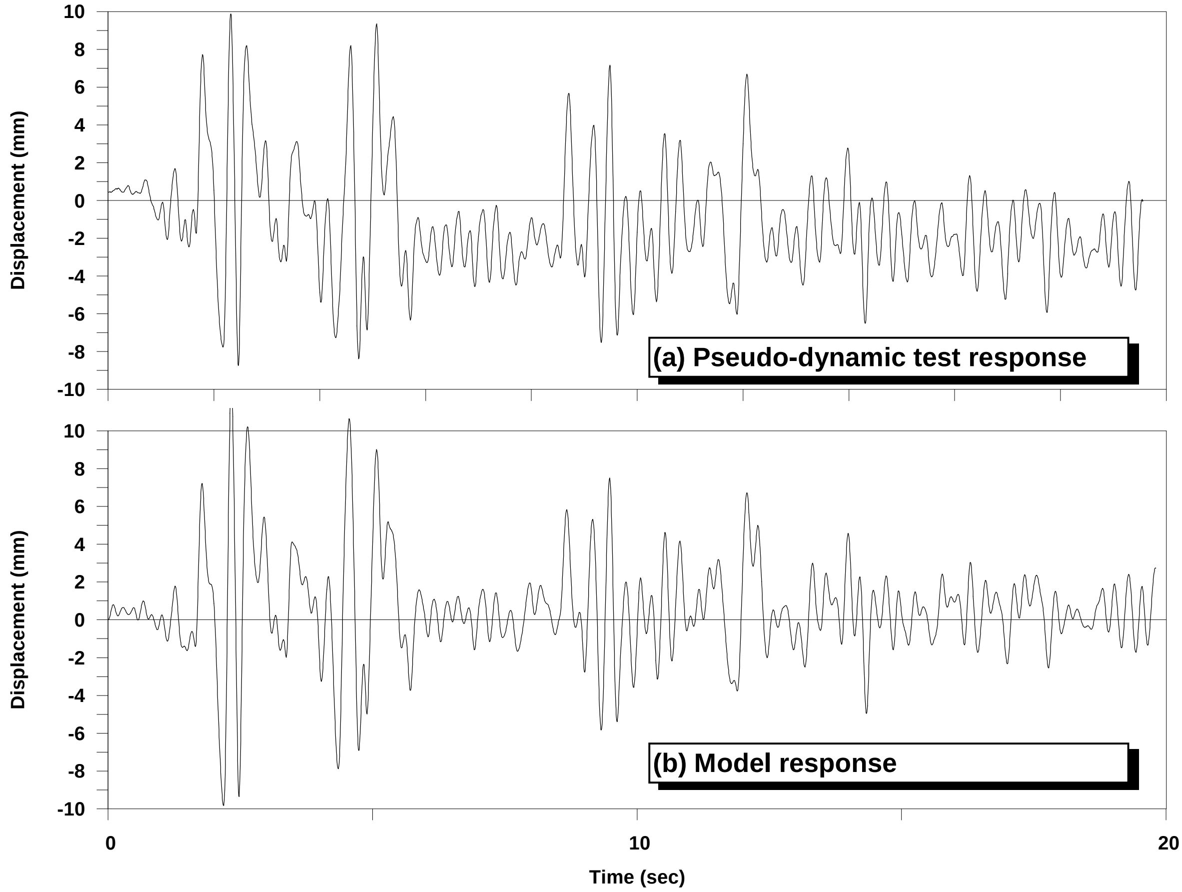

Figure 5. Interior wall responses with a mass of 5000 kg and subjected to 1.25 × El-Centro 1940 earthquake record (3.8 % damping).

COMPARISON WITH CURRENT P21 EVALUATION PROCEDURE

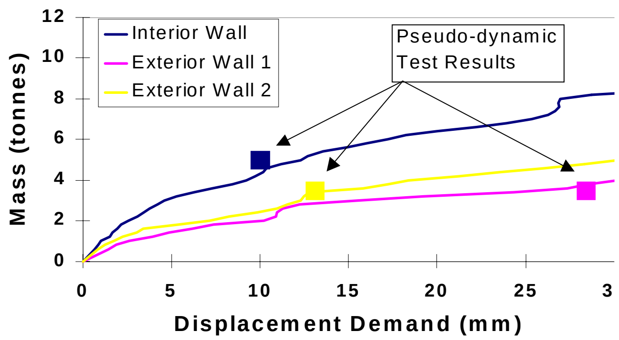

Figure 6. Mass-displacement demand plots for the three walls

The newly developed rating method was used to check the ratings assigned by the current P21 evaluation procedure [King and Lim, 1991] described above. To do this, the mass that the plasterboard lined test walls are capable of restraining was evaluated for the suite of design earthquakes using BraceRate. The mass able to be restrained by the two specimens is plotted in Figure 7 for all five earthquakes and for displacement demands of up to 30 mm. The equivalent rating in Bracing Units is indicated on the right axis. This was calculated from the mass using Equation 4 (ie for a building with T = 0.4 seconds, a specimen ductility of μ = 4 and "intermediate" soil).

The ratings evaluated using the current P21 evaluation procedure are almost twice those evaluated using the new method. The ratings, evaluated using the current method at the 16 mm displacement cycles, are shown as points in Figure 7.

The significant factors which lead to this major difference between ratings produced by the two methods are:

1. The current method assumes a natural building period of 0.4 seconds which imposes a displacement of about 32 mm upon an elastically responding building. The inelastic displacement demand is normally greater than this but most bracing elements are not even capable of resisting a displacement demand of 32 mm.

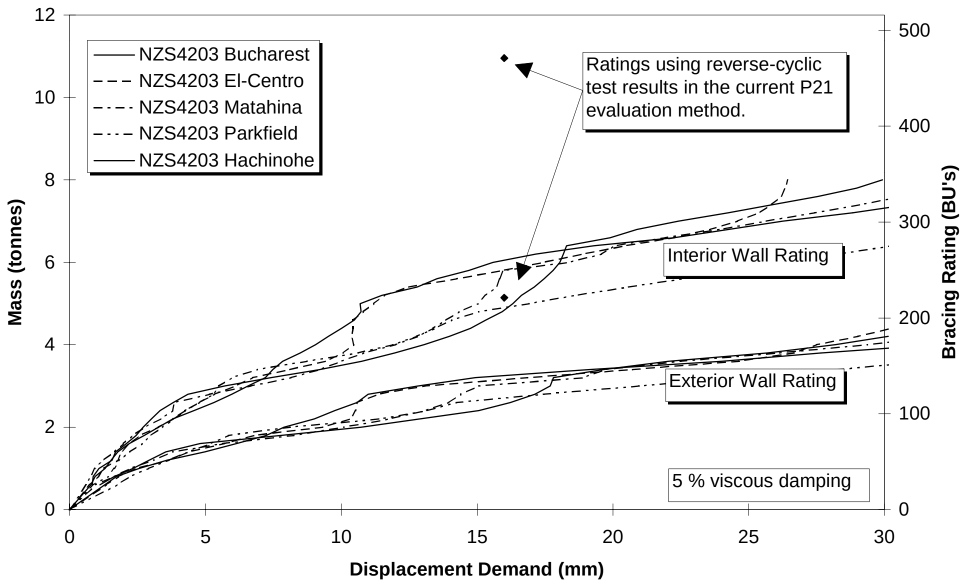

Figure 7. Mass-Displacement ratings for NZS 3604 Zone A.

2. The current [King and Lim, 1991] method assumes a structural ductility factor of $\mu = 4$. The method of calculating this based upon a "yield" displacement at half of the peak load is open to question. This is discussed further below.
3. The proposed method rates the specimen at a "maximum reliable displacement" rather than the maximum test displacement. This may be over conservative but the reserve displacement capacity is needed to provide life-safety protection for a "maximum credible earthquake" is not currently quantified in the Loadings Standard, NZS 4203.
4. The viscous damping used with the proposed method (5 percent of critical, based on the initial stiffness) may be insufficient to account for damping provided by the additional "non-structural" walls that are not specifically detailed or counted as bracing elements.

The definition of yield given in the second factor above needs further examination because a modification to this definition could provide better correlation between the King and Lim evaluation method and the results obtained using the proposed method. This form of modification would simplify the re-evaluation of existing test results and allows it to be used with the inelastic NZS 4203 spectra with their implicit elastoplastic "ductility".

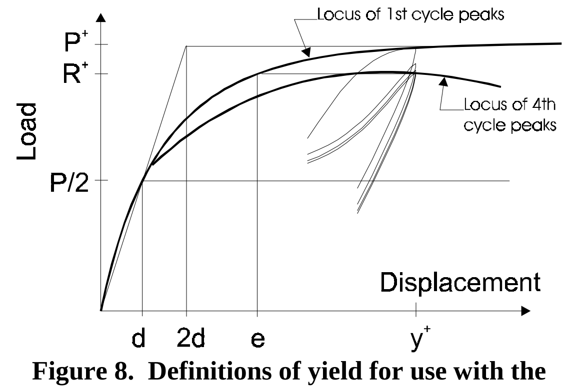

Figure 8. Definitions of yield for use with the current 'P21' rating procedure

The current definition of $\mu = y^+/d$ (Figure 8) was "… adopted as an interim measure while further investigation is continuing …" [King and Lim, 1991]. A "plastic" strength of P/2 may have been more appropriate than R for use with an elasto-plastic response spectra but this would appear to be excessively conservative because the residual strength, R, is always greater than P/2.

The King and Lim method could be adapted to use a ductility of $\mu = y^+/2d$ or $\mu = y^+/e$ (Figure 8) without significantly increasing the complexity of re-evaluating ratings for existing systems. Bracing ratings for the two walls calculated using the three different methods of calculating ductility are compared with the Figure 7 time-history ratings (ie, the average rating for the 5 earthquakes) in the following table:

Table 2 Bracing Ratings from the different evaluation methods

Wall	$\mu = y^+/d$	$\mu = y^+/2d$	$\mu = y^+/e$	Time-History
Interior	470	445	327	250
Exterior	220	210	160	130

The ratings calculated for $\mu = y^+/e$ are 25 to 30 percent higher than those produced by time-history analysis. This more modest difference is more likely to represented by the strength difference between that of an isolated test element and the same element attached to and strengthened by its surrounding walls.

Comparisons need to be undertaken for a range of bracing materials and systems because plasterboard is generally stiffer but weaker than wood- and cement- based lining materials.

CONCLUSIONS

A new method of rating the resistance of bracing elements has been proposed for use with NZS 3604:1999. A new analytical model has been developed which accurately models the load-displacement responses of these bracing elements. A continuous pseudo-dynamic test method has been developed and implemented. Time-displacement responses of pseudo-dynamic tests on three specimens have been accurately predicted by the analytical model.

The current P21 rating procedure [King and Lim, 1991] has been shown to be dangerously unconservative when compared with the new method. A simple modification to the current method has been proposed but still needs to be verified by the more rigorous method for a wider range of specimens.

ACKNOWLEDGEMENTS AND REFERENCES

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Clow, K., Davidson, B. and Matthews, J. (1995) A methodology for calculating the optimal bilinear seismic isolation systems. NZNSEE Annual Technical Conference, Rotorua, New Zealand.

Clough, R.W. and Penzien, J. (1975) Dynamics of Structures. McGraw-Hill. New York.

Cooney, R.C. and Collins, M.J. (1979, revised 1982, 1987, 1988) A wall bracing test and evaluation procedure. Building Research Association of New Zealand Technical Paper P21. Judgeford.

Deam, B.L. and King, A.B. (1996) Pseudo-dynamic seismic testing of structural timber elements. Proceedings, International Wood Engineering Conference, New Orleans, Lousiana, 1:53-59.

Deam, B.L. (1997) Seismic Ratings for Residential Timber Buildings. BRANZ Study Report 73, Judgeford.

Dean, J.A., Stewart, W.G. and Carr, A.J. (1987) The Seismic Design of Plywood Sheathed Timber Frame Shearwalls. Pacific Conference on Earthquake Engineering, Wairakei, New Zealand. 2:165-175.

Filippou, F.C., Bertero, V.V. and Popov, E.P. (1983) Effects of bond deterioration on hysteretic behaviour of reinforced concrete joints. EERC Report No. 83-19, University of California, Berkeley.

Herbert, P.D. and King A.B. (1998) Racking Resistance of Bracing Walls in Low-rise Buildings Subject to Earthquake Attack. Building Research Association of New Zealand, Study Report 78. Judgeford.

King, A.B. and Lim, K.Y. (1991) An Evaluation Method of P21 Test Results For Use With NZS 3604:1990, Building Research Association of New Zealand, Technical Recommendation No 10, Judgeford.

King A.B. and Deam B.L. (1998) A Rational Engineering Basis for Assessing Timber Framed Bracing Panels under Earthquake Attack. Proc. Australasian Structural Eng. Conference, Auckland, New Zealand. 2:949-955.

Park, R. (1989) Evaluation of Ductility of Structures and Structural Assemblages from Laboratory Testing. Bulletin, New Zealand National Society for Earthquake Engineering 22(3): 155-166.

Standards Association of New Zealand (SANZ) (1990). Code of Practice For Light Timber Frame Buildings Not Requiring Specific Design , Standards New Zealand, NZS 3604, Wellington.

Standards New Zealand (SNZ) (1992) Code of Practice for General Structural Design and Design Loadings for Buildings, Standards New Zealand, NZS 4203, Wellington.

Standards New Zealand (SNZ) (1999). Timber Framed Buildings, NZS 3604, Wellington.

Thurston, S.J. (1994) Field Testing of House Pile Foundations Under Lateral Loading. Building Research Association of New Zealand. Study Report SR58. Judgeford.

Thurston, S.J. (1993) Report on Racking Resistance of Long Sheathed Timber Framed Walls With Openings, Building Research Association of New Zealand, Study Report 54, Judgeford.