Time History Analysis as a Method of Implementing Performance Based Design

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ABSTRACT: The new loadings code, NZS 1170, will replace our current loadings code, NZS4203:1992, and will likely govern structural design and analysis in New Zealand until 2015 and beyond. We can expect that within this time our profession will adopt the technology available to us to evaluate the response of structures to earthquakes much more accurately than is done today. This will move us toward implementing performance based design by using nonlinear time history analysis.

Current codes discourage time history analysis because the scaling of time history records results in larger earthquake response than alternate, linear elastic methods of analysis. The committee developing the draft loadings code is attempting to formulate the provisions so as to remove this impediment to more accurate analysis. This paper presents the results of a study to assess the impact of the proposed earthquake scaling factors on 8 prototype moment frame and shear wall buildings, 6 and 12 stories high, located on sites in Auckland and Wellington. Maximum response predicted by both the response spectrum and time history methods of analysis is compared. The need for different drift criteria, depending on the type of analysis used and the influence of near-fault effects, is highlighted by this study.

1 INTRODUCTION

The current loadings code, NZS4203:1992 (Standards NZ, 1992) permits the use of time history analysis for all structures but the procedures are non-prescriptive and open to varying interpretation, particularly in the selection and scaling of time history records. Scaling of at least three records is required such that "over the period range of interest … the 5% damped spectrum of the earthquake record does not differ significantly from the design spectrum". In practice, this has usually been interpreted so as to scale records to provide approximately the same average spectrum as the design spectrum.

The draft loading code, NZS 1170.5 (Standards NZ, 2004b), is more explicit in that it provides a statistical method based on minimising the least mean square to select a scale factor for each record. A family scale factor is then selected and applied to all records such that at every period within the range of interest the spectral acceleration of at least one record exceeds the target spectrum. The same scale factor is applied to both horizontal components of the earthquake, which are applied simultaneously.

As part of the code development process, some commentators expressed concern that the scaling procedures would result in higher response that would be obtained from response spectrum analysis as a consequence of the record enveloping the spectrum and both components being applied simultaneously. This could act as a disincentive to use the more accurate method of analysis and would also inhibit development of performance based design procedures.

To address these concerns, it was proposed that acceptance criteria, particularly drifts, be a function of the type of analysis. In order to assess appropriate limits, a series of 8 buildings were designed using the response spectrum method of analysis as specified in Draft 8 of the joint standard AS/NZS 1170.4 (Standards NZ, 2004a). The performance was then assessed using the time history method of analysis.

2 PROTOTYPE BUILDINGS

Eight prototype buildings were selected from a matrix of two heights, two locations and two structural types, as listed in Table 1. All buildings were reinforced concrete and located on Site Subsoil Class C. Each building was designed using the response spectrum method of analysis following the requirements of Draft 8 of AS/NZS 1170.4. The response spectrum analysis used the computer program ETABS (Habibullah, 1994).

Table 1 Prototype Buildings

Height	Location	Type
6 Stories	Wellington	Wall / Frame
12 Stories	Auckland	2 Way Frame

The buildings were regular as defined in NZS 1170 but had small variations in wall and frame configuration such that they were not symmetric. Figure 1 shows the analysis models of each building type. The geometry was the same for the corresponding Wellington and Auckland buildings but the element sizes were larger for the former location. The contribution of gravity internal frames was ignored for design.

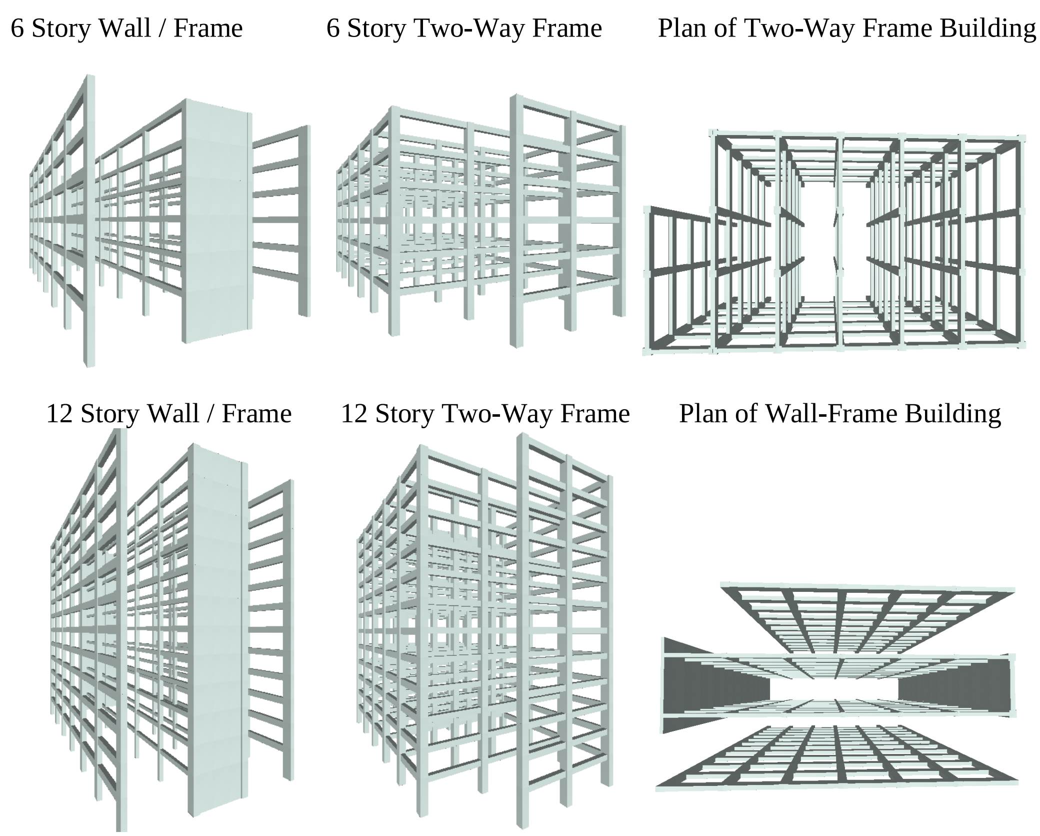

Figure 1 Prototype Buildings

3 SEISMIC INPUT

The input acceleration records were these recorded from the three earthquakes listed in Table 2 for each location, based on recommendations from (McVerry 2004). The scale factors are a function of the translational periods, as listed in Table 3. The buildings were designed using minimum member sizes, as is normal practice, and so the Auckland buildings were more flexible than those located in Wellington. This is reflected in the longer natural periods for Auckland buildings than their Wellington counterparts.

Table 2 Earthquake Records

Location	File	Records
Auckland	BO1	Bovino Campano Lucano (Italy) 1980
	DEL1	Delta Imperial Valley (USA) 1979
	MA1	Matahina Dam D (bottom centre) Edgecumbe 1987
Wellington	LU1	La Union Michoacan (Mexico) 1985
	TB1	Tabas (Iran) 1978 N74E
	TBP1	Tabas (Iran) 1978 N00E (Forward Directivity Near Fault Effects)

Note from Table 3 how flexible modern buildings are compared to those 30 years ago when periods were often estimated as N/10, where N is the number of stories. In Auckland, ductile frame periods can exceed N/3. These longer periods are a function of the optimised section sizes used in current structural design practice and also the effective stiffness factors specified by material codes. In Wellington the periods tend toward a maximum value of approximately N/5.

Table 3 Building Periods

	6 Story Wall/Fr	6 Story Fr/Fr	12 Story Wall/Fr	12 Story Fr/Fr
Auckland				
X	1.76	2.18	3.06	3.34
Y	1.38	1.82	3.51	3.25
ROT	0.55	1.44	1.64	2.48
Wellington				
X	1.31	1.26	1.90	1.95
Y	0.80	1.29	1.93	2.05
ROT	0.44	0.99	1.05	1.57

Table 4 lists the scaling factors calculated for each of the three earthquakes for each prototype building. There was not a great variation in scaling factors for the same earthquake for different buildings, with the maximum and minimum values generally being within about ±15% of the mean value, even though the periods vary by a factor of more than 2.0. The scaling was performed over the range $0.4T_1$ to $1.5T_1$. The upper limit has been reduced to $1.3T_1$ in later drafts.

Table 4 Earthquake Scaling Factors

		6 Story Wall/Fr	6 Story Fr/Fr	12 Story Wall/Fr	12 Story Fr/Fr	Max	Min	Mean
Auckland								
BO1	X	4.56	4.49	3.59	3.42	4.56	3.42	3.95
	Y	4.03	4.56	3.47	3.47			
DEL1	X	0.40	0.41	0.42	0.43	0.43	0.36	0.41
	Y	0.36	0.40	0.43	0.43			
MA1	X	0.85	0.84	0.58	0.57	0.85	0.57	0.69
	Y	0.69	0.85	0.57	0.57			
Wellington								
LU1	X	2.55	2.55	2.59	3.03	3.03	2.21	2.65
	Y	2.21	2.55	2.69	3.03			
TB1	X	0.65	0.65	0.65	0.77	0.77	0.63	0.68
	Y	0.63	0.65	0.65	0.77			
TBP1	X	0.68	0.68	0.68	0.80	0.80	0.64	0.71
	Y	0.64	0.68	0.68	0.80			

Two earthquakes (Bovino, Italy for the Auckland site and La Union, Mexico for the Wellington site) were scaled up by a relatively large number, with factors in the range of 2.2 to 4.6. In practice, records with scale factors greater than 3 would be replaced by another record which required a lower scale factor. However, as these were the three recommended records available at this time these earthquakes were included in the study.

4 COMPARISON OF RESPONSE

The structures are all designed for ductility under seismic loads and so deformations are more meaningful than forces. The parameter used to compare the response spectrum and the time history methods is the peak inter-story drift. For the response spectrum methods the drifts are based on elastic drifts modified by $\mu/S_{p,}$ as was required by Draft 8 of AS/NZS 1170.4. For the time history analysis, peak drifts were the maximum values occurring at any time step of any of the three earthquakes. All time history analyses used the computer program ANSR-II (Mondkar, 1979).

The peak drifts from each method of analysis are listed in Table 5, together with the ratio of time history drift to response spectrum drift.

Table 5 Peak Drifts

	Time History (TH)		Response Spectrum (RS)		Ratio TH/RS	
	X	Y	X	Y	X	Y
Wellington						
6 St. Wall/Frame	1.67%	1.72%	2.23%	1.11%	0.75	1.54
6 St Frame/Frame	1.87%	2.23%	2.01%	2.21%	0.93	1.01
12 St. Wall/Frame	2.57%	2.33%	2.10%	2.26%	1.22	1.03
12 St Frame/Frame	3.85%	3.99%	2.16%	2.47%	1.78	1.61
Auckland						
6 St. Wall/Frame	0.87%	0.65%	0.68%	0.69%	1.28	0.94
6 St Frame/Frame	1.49%	1.65%	0.86%	0.84%	1.73	1.97
12 St. Wall/Frame	0.95%	0.54%	0.65%	1.16%	1.44	0.47
12 St Frame/Frame	1.00%	1.10%	0.82%	0.86%	1.22	1.28

Of the 16 structural systems (2 in each of 8 buildings), the time history method provided the higher drifts in 12 cases and the response spectrum method in the other 4 cases. Figure 2 plots the maximum drifts from each method of analysis.

For both 12 story structures in Wellington, the time history drifts exceeded the limit of 2.5%, even though the design using the response spectrum method met this criterion.

Figure 3 plots the drift ratios versus building period for the two sites. There is no apparent correlation between building period and ratio of time history drift to response spectrum drift.

The ratio of time history drift to response spectrum drift ranged from 0.47 to 1.97, with a mean value of 1.26. The mean ratio was similar for the two sites (1.24 Wellington, 1.29 Auckland). There appear to be no general trends in Figure 3, although in Auckland the 6 story frames had the highest ratios and in Wellington the 12 story frame ratios were highest.

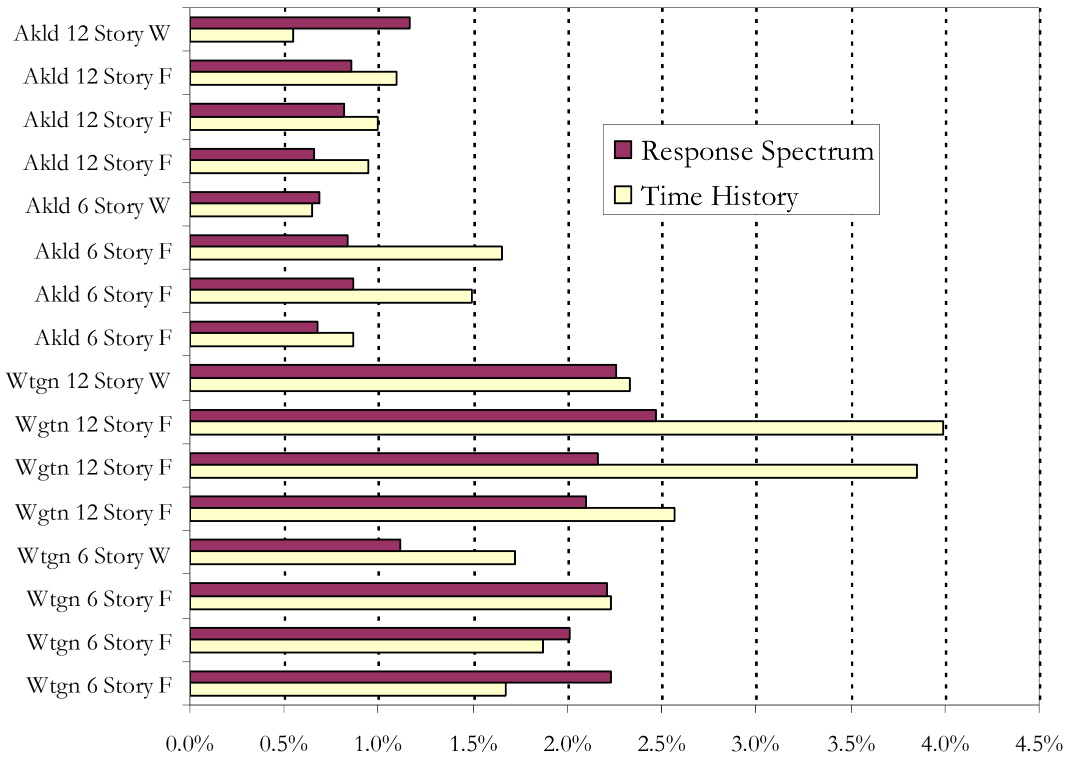

Figure 2 Maximum Drifts

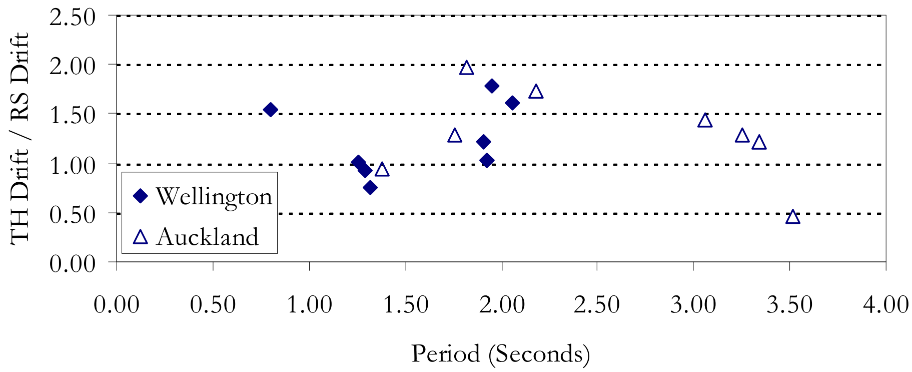

Figure 3 Drift Ratio Versus Period

5 EFFECT OF TIME HISTORY

The peak drifts in Table 5 and Figure 2 are envelope results from the suite of three scaled time history records. Table 6 lists the peak time history drifts for the individual earthquake records and identifies the record which produced the maximum drift.

Figure 4 plots the acceleration and displacement spectra for the Auckland and Wellington sites and corresponding code defined target spectra. In these plots, each earthquake has been scaled by the maximum scale factor for that particular record.

The plots in Figure 4 show the large difference in maximum spectral accelerations for the two sites.

At Auckland, the maximum displacement imparted to any structure will be 330 mm with peak values occurring at periods of 4.0 seconds. At Wellington, the near fault record (TBP1) dominates spectral displacements which continue to increase to 1920 mm for periods of 5 seconds or longer. For this record, the spectral accelerations also increase as the period exceeds 3 seconds. For most buildings, the spectral displacements approximate the displacements at 2/3H and so roof displacements will be about 50% higher than the spectral displacements.

Table 6 Maximum Drift by Earthquake

	X (Frame) Direction				**Z (Wall / Frame) Direction**			
Wellington	**LU1**	**TB1**	**TBP1**	**Max**	**LU1**	**TB1**	**TBP1**	**Max**
6 St. Wall/Frame	1.27%	1.67%	1.48%	TB1	0.81%	0.89%	1.72%	TBP1
6 St. Frame/Frame	1.55%	1.66%	1.87%	TBP1	1.64%	1.89%	2.23%	TBP1
12 St. Wall/Frame	1.30%	1.44%	2.57%	TBP1	1.40%	1.28%	2.33%	TBP1
12 St. Frame/Frame	1.79%	1.95%	3.85%	TBP1	1.92%	2.20%	3.99%	TBP1
Auckland	**BO1**	**DEL1**	**MA1**		**BO1**	**DEL1**	**MA1**	
6 St. Wall/Frame	0.67%	0.84%	0.87%	MA1	0.42%	0.39%	0.65%	MA1
6 St. Frame/Frame	1.20%	0.85%	1.49%	MA1	1.50%	1.17%	1.65%	MA1
12 St. Wall/Frame	0.95%	0.73%	0.89%	BO1	0.52%	0.54%	0.54%	MA1
12 St. Frame/Frame	1.00%	0.72%	0.86%	BO1	1.10%	0.84%	1.01%	BO1

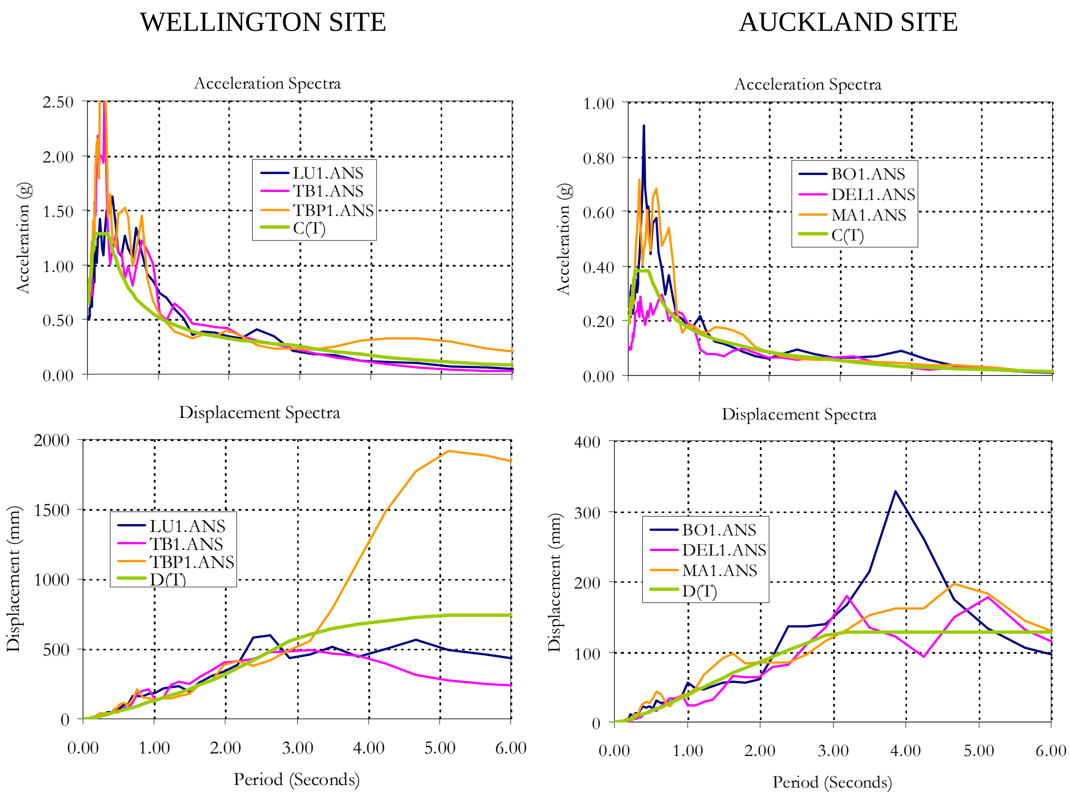

Figure 4 Acceleration and Displacement 5% Damped Response Spectra

5.1 **Wellington Records**

With the exception of one frame, the peak drifts in all Wellington buildings occurred under the TBP1 record, which is the Tabas, Iran earthquake with forward directivity near fault effects.

Figure 4 clearly shows why this record will dominate medium to long period buildings as the spectral acceleration increases for periods increasing beyond 3 seconds. This produces correspondingly high increases in spectral velocity and displacement.

5.2 **Auckland Records**

The spectrum of the Bovino, Italy record (BO1) has a secondary acceleration peak at about 3.8 seconds, which produces a high spectral velocity and displacement at periods of about 4 seconds. As shown in Table 5, this record produces the maximum response in the 12 story buildings, which have an elastic period exceeding 3 seconds.

The Matahina Dam record of the 1987 Edgecumbe earthquake (MA1) has a secondary peak at approximately 1.5 seconds and this has the greatest effect on the shorter period buildings, the 6 story buildings which have periods of 1.4 to 2.2 seconds.

6 **CHANGES TO NZS1170.5**

The results from the evaluation of these prototype buildings showed that:

1. The time history method produced higher drifts than the response spectrum method in at least one direction in all 8 buildings, and in both directions for 4 of the 8 buildings.
2. Near faults effects are much more severe for the time history method than for the response spectrum method. This is because the spectral acceleration increases with increasing period for the record used whereas the response spectrum with near fault effects still has spectral accelerations reducing with increasing period.

The Code Committee considered these results, and other data, to implement a number of changes into the current draft code. The main changes which would affect this study were:

1. The structural performance factor, S_p, for time history analysis was reduced from 1.0 to 0.85 (compared to 0.7 for response spectrum analysis).
2. The response spectrum drifts are not divided by the S_p factor.
3. Response spectrum drifts are scaled by factors ranging from 1.2 to 1.5, depending on building height.
4. Time history drifts are scaled by 0.67 for records which include forward directivity (near fault) effects, 1.0 otherwise.

The net effect of these changes is that all buildings included in this study are likely to comply with draft code requirements for both the response spectrum and time history methods of analysis.

For near fault analyses the maximum permitted drift limit is 2.5%/0.67 = 3.73%. This is likely to be close to the collapse limit for some types of structure. Protection against collapse will be enforced by requirements to explicitly consider P-delta effects in the analysis and by limits on inelastic deformations supplied by the material codes.

7 **COMPARISON WITH FEMA 356**

FEMA-356 (FEMA, 2000) is a widely used guideline for the seismic evaluation of buildings. This document specifies a procedure for selecting scale factors for time histories which differs in two main respects from the procedure of NZS1170:

1. The individual FEMA scaling factors are based on the ratio of the SRSS of the two components to 1.4 C(T), rather then the ratio of the primary component to C(T) as for NZS1170.
2. The FEMA family scaling factor is calculated so that the average of the three records exceeds the target spectrum at each period point. The NZS1170 family scaling factor if selected so that at least one record exceeds the target.

The net effect of these two differences is that the FEMA-356 scaling factors are 50% to 100% higher than the NZS1170 factors, as shown in Table 7 for the 12 Story Wall/Frame buildings at each site. As the response spectrum and time history methods have the same acceptance criteria under FEMA-356, it is unlikely that the time history procedure would be used for projects evaluated to these guidelines.

Table 7 Comparison of Scaling Factors

	Wellington Site			**Auckland Site**	
Record	**NZS 1170**	**FEMA 356**	**Record**	**NZS 1170**	**FEMA 356**
LU1	2.59	5.22	BO1	3.59	7.29
TB1	0.65	1.18	DEL1	0.42	1.00
TBP1	0.68	1.10	MA1	0.58	1.08

FEMA-356 permits use of mean time history results rather than maximum values provided 7 or more records are used. This may be an option worth assessing for our code to reduce the effects of variability between records.

8 CONCLUSIONS

The nonlinear time history method has huge potential to improve seismic performance in that dynamic amplification effects due to yielding are explicitly included in the evaluation, a major advantage compared to the highly empirical ω factors currently used. Earlier drafts of NZS 1170 raised impediments to the use of this technique in that structures which performed satisfactorily using response spectrum analysis did not meet code criteria when the time history method was used. This was a factor the code had in common with FEMA 356, where very large scaling factors actively discourage use of the nonlinear dynamic procedure.

Changes implemented by the Code Committee in the current draft have adjusted the procedures such that the relative accuracy of each analysis method is recognised and there is no penalty for using more advanced analysis techniques. This will help speed the adoption of performance based design procedures during the lifetime of the new code.

9 ACKNOWLEDGMENTS

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