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SEISMOLOGICAL INFORMATION FOR DISPLACEMENT-BASED DESIGN -A STRUCTURAL ENGINEER'S WISH LIST

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SUMMARY

With conventional force-based seismic design, the most important information for the structural engineer has been the effective peak ground acceleration. This number is comparatively insignificant in displacement-based or performance-based seismic design, where the key data are peak spectral response displacement, and the "corner" period at which this occurs. There appears to be a disagreement between seismologists on opposite sides of the Atlantic about these data, which the structural engineer views with concern, since ductile structural response will often be in this region of the corner period. Dependable information on the reduction of displacement response with damping or ductility is also needed for design approaches utilizing secant stiffness characterization. Other areas needing clarification include the issue of how to develop accelerograms for time-history analysis compatible with the design seismicity using an acceptably small number of accelerograms, and how to reconcile the statistical nature of seismicity characterization with the structural engineer's preference for deterministic analysis. There is some evidence that a consequence of this is invalid averaging of response characteristics by structural engineers.

1. INTRODUCTION

Seismologists and structural engineers see things differently. This is immediately apparent when we look at the way the fundamental information describing the design intensity – namely the response spectrum – is presented. Seismologists, for some reason unfathomable by structural engineers prefer to use a log scale for the period axis while structural engineers prefer a linear axis. The alternatives are compared for an acceleration response spectrum based on a peak ground acceleration of 0.4g, firm ground, and a causative earthquake of about $M_W = 7$.

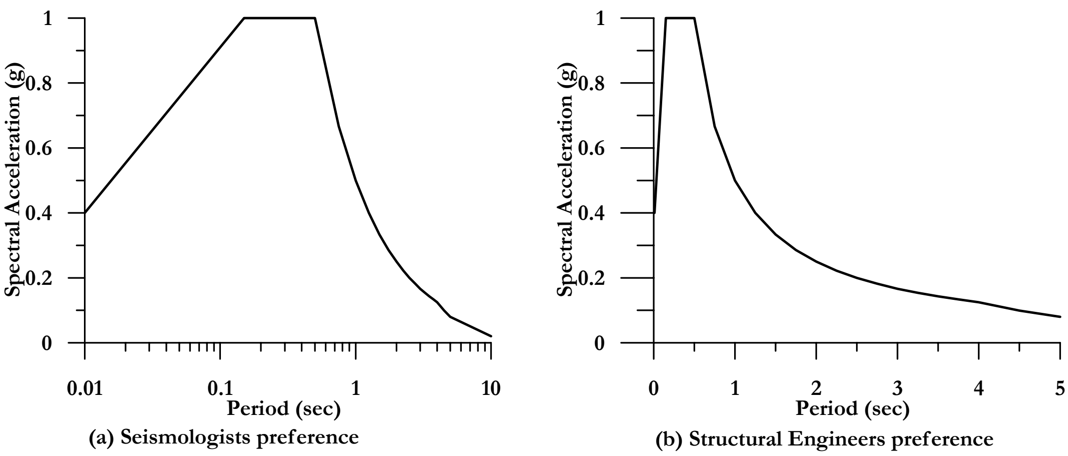

Fig.1 Different presentations of acceleration response spectra (PGA = 0.4g, $M_W \approx 7$, $\xi = 5\%$)

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The logarithmic presentation provides a great deal of information about the response at very low periods, but condenses the information above 0.5 seconds, which is of primary interest to the structural engineer to be almost unreadable. The natural period scale of Fig.1(b) enables the structural engineer to visualize the response variation with period over the relevant range with much greater clarity.

The current move towards performance-based seismic design (Priestley, 2000) implies greater emphasis on displacements rather than accelerations, particularly when design is based on secant representation of effective stiffness (Priestley, 2006). If we present the same information provided in Fig.1 in displacement terms, rather than accelerations, Fig.2 results:

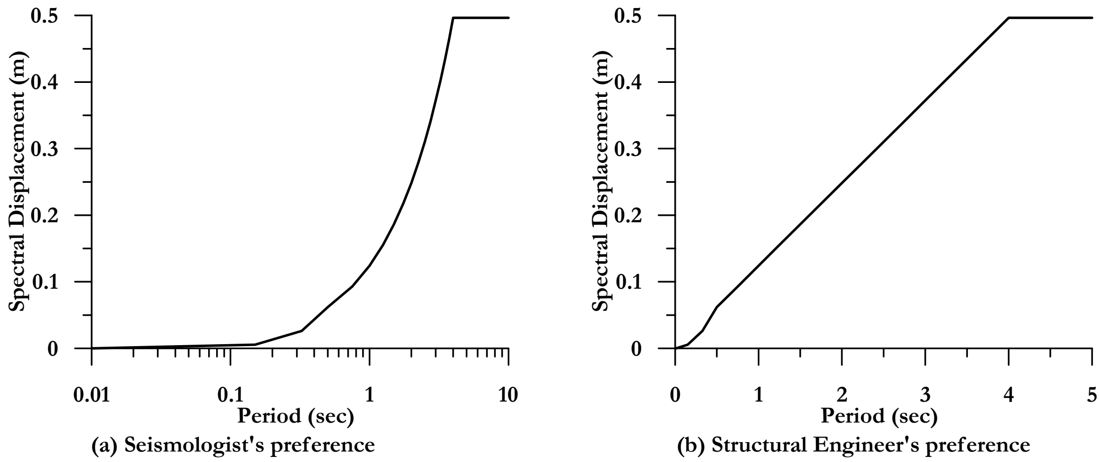


Fig.2 Different presentations of elastic displacement response spectra (PGA = 0.4g, $M_W \approx 7$, $\xi = 5\%$)

It is seen here that the logarithmic presentation of Fig.2(a) is difficult to interpret (at least for structural engineers), and masks the fact that the displacement increases linearly with period in the period range of interest from 0.5 sec to the peak response displacement, which in this case occurs at T = 4 sec. The linearity of this section of the response in this range is a consequence of the common assumption that it is a region of constant peak velocity response, and that displacement may be calculated from acceleration assuming sinusoidal response. These assumptions become increasingly approximate as the period increases. As discussed subsequently, the period at which the displacement plateau initiates is of key concern to structural engineers, and appears to be a matter of controversy amongst seismologists.

It will be noted that in both Figs. 1 and 2, the seismological presentation of data has extended to 10 sec, but the structural presentation has stopped at 5 sec, since no further information of value is available by extending the constant displacement plateau to 10 sec. However, it is perhaps relevant to view the form of the entire elastic displacement spectrum, as presented in Fig.3.

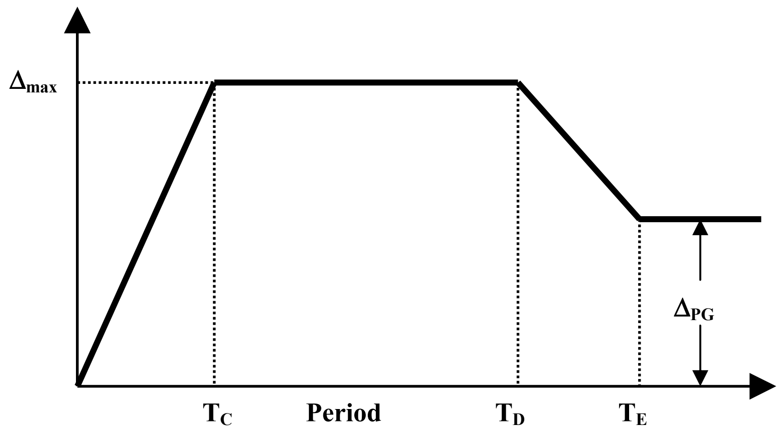


Fig.3 General characteristics of elastic displacement response spectra (CEN-EC8 (2003))

Apart from an initial non-linear range from about 0 – 0.5 sec, which is apparent in Fig. 2(b), but is of little interest to the structural engineer when displacement response is the key issue, the response is essentially linear up to a peak response displacement of $\mathbf{\Delta_{max}}$ at a corner period $\mathbf{T_C}$. This is followed by a region of essentially constant response displacement up to a period of $\mathbf{T_D}$ whereafter the displacement decreases to a value close to the peak ground displacement for periods greater than $\mathbf{T_E}$. Although the behaviour at periods greater than $\mathbf{T_D}$ may be of interest for structures of very long periods, such as suspension bridges, and large diameter tanks containing fluids, it is unlikely that structural engineers would design for reduced response from the plateau. Thus it is the initial linearly increasing range, and the plateau that are of particular interest to the structural engineer. As discussed subsequently, the current data for these are less than robust.

Despite the good efforts of experts (e.g. Pinto et al, 2004), structural engineers tend to have at best a tenuous grasp of probability and reliability theory as related to seismic engineering. We prefer deterministic approaches, and as a consequence may sometimes misuse data that require statistical interpretation. There are many areas that could be cited here, but primarily they relate to the problem of design verification using inelastic time-history analysis. Of particular importance is the choice and number of accelerograms used to represent the design seismic intensity, and the way in which the results are interpreted to determine the expected displacement response. We believe that seismologists, who appear to swim in probability theory as naturally as fish in water, could greatly assist structural engineers in this area.

To summarize this introduction, there are a number of topics, relevant to seismology, in which structural engineers could benefit from seismologist's advice. The method of presentation, discussed with relation to Figs. 1 and 2 is of course trivial, and included mainly for entertainment value. The important issues are seen to be:

- The corner period $\mathbf{T_C}$ (see Fig.3) of the displacement spectrum
- The displacement $\mathbf{\Delta_{max}}$ at $\mathbf{T_C}$.
- The influence of damping or ductility in reducing response displacements.
- The influence on duration of strong motion on the damping or ductility reduction to displacement
- Choice of accelerograms for design verification using inelastic time-history analysis.
- Determination of probable design response from time-history analysis using multiple accelerograms.

These issues are briefly discussed in this paper.

2. ELASTIC DISPLACEMENT RESPONSE SPECTRUM

With conventional force-based design there has been a tendency to erroneously believe that the spectral acceleration response at periods greater than about 2 seconds is of little relevance to design of "normal" structures. In fact is has been not uncommon in the past to specify design elastic acceleration spectra with a constant minimum acceleration applying for periods larger than (say) 2 seconds. The intent has been to ensure that at least an acceptable minimum lateral strength is provided, but if interpreted literally, and used to determine design response displacements using the sinusoidal relationship

$$S_{D(T)} = \frac{T^2}{4\pi^2} \cdot S_{A(T)} \tag{1}$$

where $S_{A(T)}$ and $S_{D(T)}$ are spectral accelerations and displacements respectively at period T, the results are alarming.

In part the lack of interest in periods greater than about 2 seconds is a result of the unrealistically low estimates of fundamental periods specified in seismic design codes. Typically these are only 30-50% of the true values applicable at first-yield of the structure. A second reason is that designs have been based on the initial stiffness, rather than the secant stiffness applying at maximum response (see Priestley, 2006 for a discussion on this). Recent analyses (Crowley et al, 2005) have indicated that for frame buildings of typical proportions in moderate seismic zones the elastic period can be approximated by

$$T = 0.1H \tag{2}$$

where H is the building height in metres. For displacement-based designs using secant stiffness, the relevant period is likely to be 50-100% larger than this, depending on the ductility demand. Thus, for a ten-storey frame building with height of 35m, the period range of interest might be 3.5 to 7.0 seconds. Note that with initial-stiffness designs the same period range is of interest, in that the displacements must be interpreted from the degraded stiffness. This is implicit if an approximate rule such as the "equal-displacement" rule is used to determine displacements based on the elastic period. The characteristics of the elastic displacement response spectrum, particularly related to the corner period and peak displacement response are thus (or at least should be) of critical interest to designers.

The current EC8 (CEN-EC8, 2003) regulations specify that the corner period $\boldsymbol{T_C}$ (Fig.3) is taken as 1.2 sec. for causative earthquakes with $M_W < 5.5$, and 2.0 sec. for $M_W \geq 5.5$. This would imply elastic response for most buildings with more than about 8 stories. It has been suggested (Boore and Bommer, 2005) that this low corner period is a result of interpretation of data from analogue records which have been processed with low-order filters set at periods that make the interpretation unreliable at periods above 2 seconds.

Recent work by Faccioli et al (Faccioli et al, 2004) analyzing a large number of recent high quality digital records can be interpreted (Priestley et al, 2006) to imply that a conservative relationship between moment magnitude and corner period can be expressed as

$$T_C = 1.0 + 2.5(M_W - 5.7) \tag{3}$$

for $M_W \geq 6.0$.

NEHRP recommendations (NEHRP, 2003), based primarily on seismology theory include the following relationship between corner period and moment magnitude:

$$\log_{10} T_C = -1.25 + 0.3 M_W \tag{4}$$

The three different relationships (EC8, Faccioli et al, NEHRP) are compared in Fig.4. The large differences between the different models are disturbing to the structural engineer, particularly since all models assume a constant velocity range up to the corner period. This implies that the displacement at the corner period will be in proportion to the corner period, if the constant-velocity segment of the acceleration response commences at the same period (say, 0.5 seconds), and same spectral acceleration response. Thus for an earthquake of $M_W = 7$, the peak spectral displacement response predicted by Eqs.(3) and (4) would be 3.54 and 2.13 times the value predicted by EC8, respectively. This is a totally unacceptable situation for the structural engineer, who would appreciate urgent clarification from seismologists. It should be noted that this difference in peak spectral displacement response is equally of importance to designs based on initial-stiffness force-based principles, and secant-stiffness displacement-based principles, provided realistic estimates of stiffness are used in both cases.

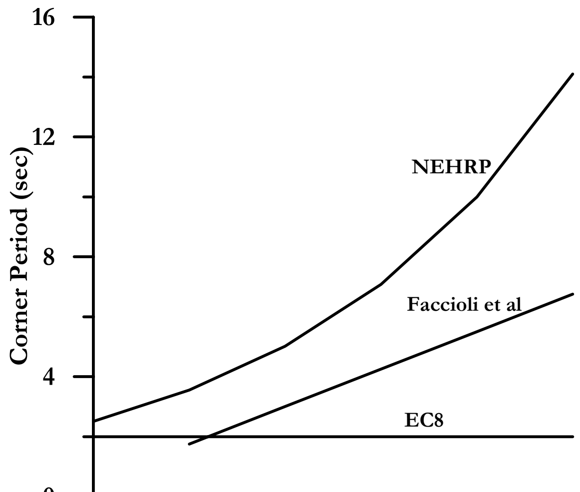
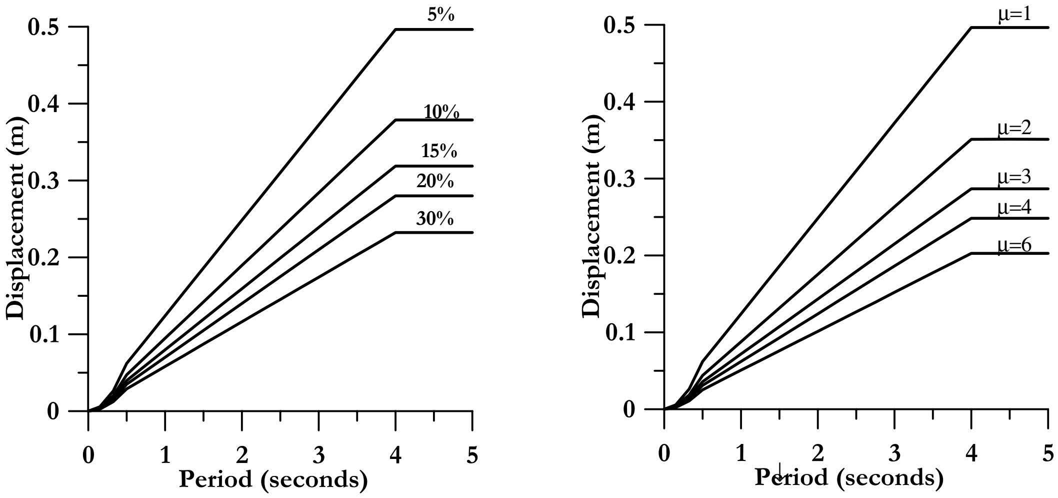


Fig.4 Different estimates of relationship between corner period and moment magnitude

3. INFLUENCE OF DAMPING AND DUCTILITY ON SPECTRAL DISPLACEMENT RESPONSE

Displacement-based seismic design (Priestley, 2000) using a secant stiffness representation of structural response requires a modification to the elastic displacement response spectrum to account for ductile response. The influence of ductility can be represented either by equivalent viscous damping (Fig. 5(a)), or directly by inelastic displacement spectra for different ductility levels (Fig.5(b)). The use of spectra modified by different levels of damping requires relationships between ductility and damping to be developed for different structural hysteretic characteristics (e.g. Grant et al, 2005), but enables a single design spectrum to be used for all structures (e.g. Fig.5(a). The use of spectra modified by different levels of ductility is perhaps more direct, but requires the ductility modifiers to be determined for each hysteretic rule considered.



(a) Modified by Equivalent Viscous Damping (b) Modified by Displacement Ductility

Fig. 5 Representation of inelastic response by displacement response spectra

If a relationship between initial-period elastic displacement and inelastic displacement such as the equal-displacement approximation is assumed, the inelastic spectra of Fig.5(b) can be directly computed. Assuming that the skeleton force-displacement response can be represented by a bilinear approximation with a ratio of post-yield to elastic stiffness equal to **r**, the secant period $\boldsymbol{T_{eff}}$ is related to the elastic period $\boldsymbol{T_i}$ by the relationship

$$T_{eff} = T_i\left(\frac{\mu}{1+r(\mu-1)}\right)^{0.5} \tag{5}$$

Since the inelastic displacement at $\boldsymbol{T_{eff}}$ must equal the elastic displacement at $\boldsymbol{T_i}$ the modification factor $\boldsymbol{R_\mu}$ to be applied to the elastic spectrum is

$$R_\mu = \left(\frac{1+r(\mu-1)}{\mu}\right)^{0.5} \tag{6}$$

The data in Fig.5(b) have been calculated for an elasto-plastic response (**r** = 0). Different relationships apply for different post-yield stiffnesses, but not to different hysteretic energy absorption within the loop, provided that the equal-displacement approximation is assumed to be valid. However, recent research (Priestley and Grant, 2005) indicates that the equal-displacement approximation is in fact non-conservative. The reasons for this are presented in another paper to this conference (Priestley, 2006). The consequence is that inelastic spectra based on displacement ductility (e.g. Fig.5(b)) must be calibrated for each different hysteretic rule.

Because of these reasons, the equivalent viscous damping approach, presented in Fig.5(a) are preferred (at least by this writer), since codified specification is simpler. However, there appears to be still some controversy amongst seismologists as to the appropriate form of the damping modifier $\boldsymbol{R_\xi}$ to be applied to the elastic displacement spectrum for different levels of damping $\boldsymbol{\xi}$. The 1998 EC8 expression was

$$R_{\xi} = \left(\frac{7}{2+\xi} \right)^{0.5} \tag{7}$$

In the 2003 revision to EC8, Eq.(7) was replaced by

$$R_{\xi} = \left(\frac{10}{5+\xi} \right)^{0.5} \tag{8}$$

In both Eq.(7) and (8), $\boldsymbol{\xi}$ is expressed as a percentage of critical damping. An alternative expression has been proposed by Newmark and Hall (Newmark and Hall, 1987):

$$R_{\xi} = (1.31 - \ln \xi) \tag{9}$$

The three expressions are compared in Fig.6 for different levels of damping. Again structural engineers would appreciate clarification as to the most suitable expression to use, since the required design force for a given displacement limit state is essentially proportional to the square of $\boldsymbol{R_{\xi}}$.

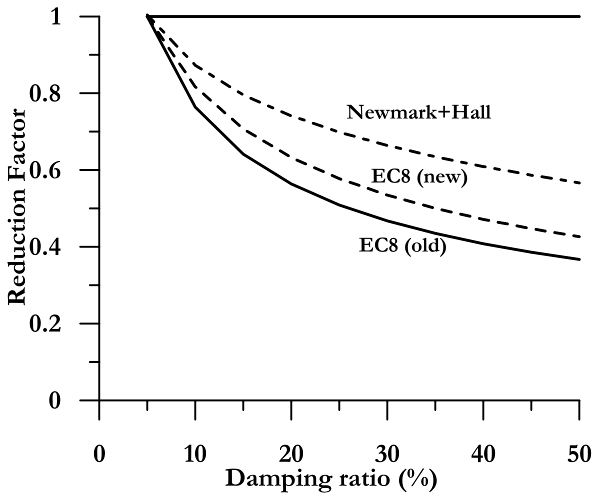


Fig.6 Damping modifiers to elastic spectral displacements

Our analyses using spectral compatible earthquake records support the 1993 EC8 expression for accelerograms without forward directivity velocity pulse characteristics. It would also be desirable to have an equivalent expression for sites where forward directivity velocity pulse characteristics might be expected. It has been suggested (Priestley, 2003), based on extremely limited data, that a modification to the 2003 EC8 expression given by

$$R_{\xi} = \left(\frac{10}{5+\xi} \right)^{0.25} \tag{10}$$

might be appropriate. The effect of this modification is to increase the value of $\boldsymbol{R_{\xi}}$ compared with the value applying for "normal accelerograms. Figure 7 compares the dimensionless displacement modifiers for the current EC8 expression (Eq.(8)) and the suggested expression (Eq.(10)). Some qualified support for this expression in available in (Bommer and Mendis, 2005) who provide additional discussion of this topic. Their work indicates that the scaling factors may be period-dependent, which is not currently considered in design.

Thus, structural engineers would appreciate more definitive expressions for damping modification for accelerograms with normal and velocity pulse characteristics. It will be noted that all the expressions above are independent of period. Confirmation that this is a reasonable approximation would be desirable.

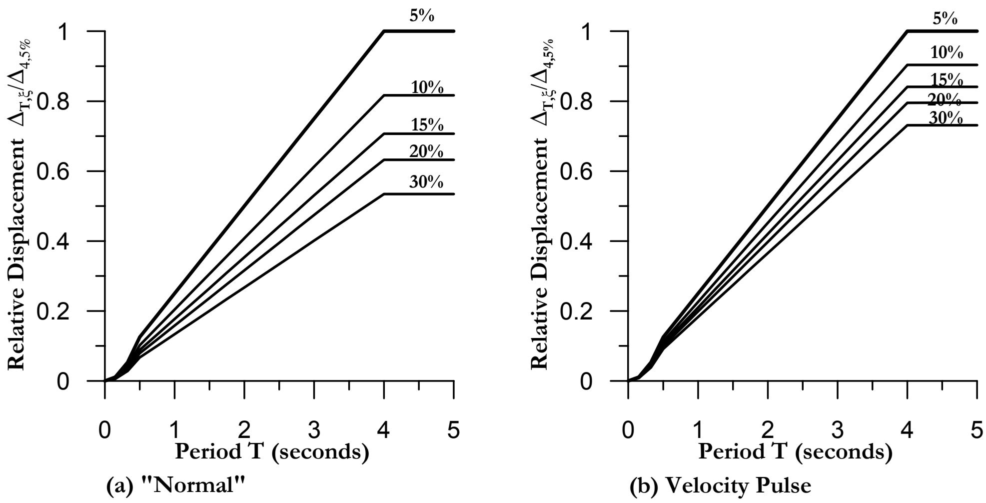


Fig.7 EC8 (2004) damping reduction factor, and tentative factor for forward directivity effects

4. ACCELEROGRAMS FOR DESIGN VERIFICATION

There are a number of issues to be decided when completed designs are to be verified by inelastic time-history analyses. These relate to the number and selection of accelerograms to be used, and what should be the characteristics of the accelerograms when multi-axis excitation is required. These aspects are briefly discussed in turn.

4.1 Number of Records

Two alternatives are generally defined by codes for the number of accelerograms to be used in design verification. The first involves using three spectrum-compatible records, with the design response being taken as the maximum from the three records, for the given response parameter (displacement, shear force etc) investigated. The second uses a minimum of seven spectrum-compatible records, with the average value being adopted for the response parameter considered. Because of the simplicity of inelastic time-history analysis (ITHA) with modern computing power, the latter approach is now almost always adopted, and there appears to be a tendency to increase the number of records above the minimum of seven, to ensure a more representative average. However, it is our experience that structural engineers frequently forget the statistical basis of the design response spectrum, and think of the average result from the ITHA in deterministic terms: a displacement 1% above the design limit means failure; a displacement 1% below the design limit means acceptable response. In fact, most structural engineers are unaware as to whether the design spectrum is based on the mean, mean + 1σ, mean + 2σ etc. More transparent statements about the basis of the design spectra (in language which the structural engineer can understand) might help. Also some statistical description of the significance of choosing a different number of records for ITHA might help: the pass/fail criterion when one is using the average of a small number of records is clearly problematic.

4.2 Selection of Records

There are three basic choices for the means of obtaining spectrum-compatible accelerograms:

- Amplitude scaling of acceleration records from real earthquakes to provide a "best fit" to the design spectrum over the period range of interest
- Generating artificial spectrum compatible records using special purpose programs
- Manipulating existing "real" records to match the design spectrum over the full range of periods.

The following is this structural engineer's view of the advantages and disadvantages of each. Confirmation (or correction) by seismologists would be appreciated.

When the records are obtained by amplitude scaling of existing records, the scatter between records is likely to be large, and hence a larger number of records might be needed to obtain a reliable average. Care has to be

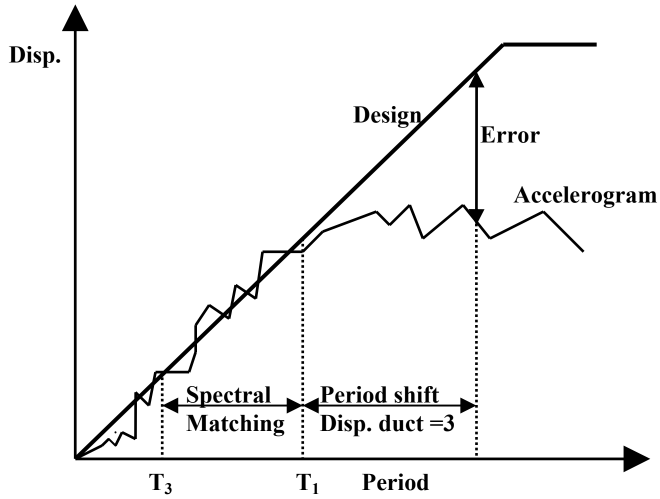


Fig.8 Displacement spectrum matching for inelastic time-history analysis

exercised in selecting the period range over which the spectrum matching is obtained. In Fig.8, which shows the matching referred to the 5% damping elastic displacement spectrum, the matching has been reasonable achieved over the period range encompassing the first three elastic periods, which might be considered adequate. However, it is seen that at periods longer than the fundamental period, displacements of the scaled spectrum are significantly lower than the design spectrum. It is important to match the spectrum for a period range that includes the period shift expected as the structure responds inelastically. This is shown in Fig.8 for a displacement ductility of $\mu = 3$. In this example, the scaled displacement spectrum becomes increasingly deficient in displacement demand, compared with the design spectrum, as the period increases above the elastic period, resulting in an unconservative estimate of the inelastic displacement from the time-history analysis. Of course, this argument applies to a single record. If a large number of records are used such that the average of the displacement spectra over the full range from elastic to inelastic period matches the design spectrum, valid average results can be expected. Unfortunately this is unlikely to be the case for longer period structures, since the large majority of records used in the analyses tend to have peak spectral displacements at periods from 1.0-2.0 seconds. In this case there is likely to be a consistent deficit in displacement demand at the degraded (inelastic) fundamental period, regardless of how many records are used. Thus conclusions about structural response for structures with expected displacement ductilities of (say) $\mu = 4$ and elastic periods of $T > 0.75$ seconds can be expected to be suspect, unless the earthquake records are very carefully chosen.

The second alternative involves artificially generated accelerograms, using programs. These can be matched to the design spectrum for the full period range with comparatively small error. A lesser number of records are required to obtain a meaningful average using artificial records. An objection commonly voiced about artificial records is that they are too severe, in that real records are not spectrum compatible over the full period range, and that artificial records typically have a longer duration than real records. The first of these arguments cannot be accepted, since the larger number of real records needed to obtain a full spectrum matching should produce essential identical results, and the artificial records can be considered as a more efficient means of obtaining the same results. The second objection – excessive duration – is unlikely to be of concern when the analyses are being carried out for verification of a new design.

Recently, the third option, where existing recorded accelerograms are scaled to obtain full spectrum matching has become more common. This method has the advantage over pure artificial records that the essential character of the original record is preserved. Thus, records that conform to the type of source characteristics expected (e.g. strike/ slip, subduction, near-field forward directivity etc.) can be selected. This is important, since the spectrum matching will normally be done at 5% damping, and the characteristics at different levels of effective damping will depend on the source characteristics and distance from the fault plane. However, to obtain the required spectrum matching, the duration of the record typically has to be extended, opening the method to the same objection as directed against artificial records.

It is not clear that the choice amongst the above alternatives should lie entirely with seismologists, but their opinion and guidance would be appreciated by structural engineers.

4.3 Multi-axis Excitation

When structural response is likely to be influenced by 2-D or 3-D effects – that is, when the response in orthogonal directions cannot reasonably be de-coupled, the accelerograms will have to include two or three orthogonal components. The generation of the characteristics of the various components needs careful consideration. Examination of recorded three-component accelerograms reveals the following characteristics – The two horizontal components have different spectral shapes and intensities, but it does not seem that there is a consistent difference when decomposed to fault-normal and fault-parallel directions, except, perhaps in near-field response. Thus our impression is that choice of a predominant, or principal direction of attack does not seem feasible at this stage of knowledge. Site-specific design spectra are generally generated using the average of fault-normal and fault-parallel attenuation relationships. In the event that a code-specified spectrum is adopted, this will almost certainly be independent of orientation.

The question then arises of how the orthogonal components of real records should be scaled. In the following it is assumed that no preferential direction for the design spectrum has been identified. There are a number of possibilities:

- Systematically rotate the axes of the two horizontal records, to determine the principal direction, and generate new major and minor principal-direction accelerograms. Scale the major principal record to the design spectrum, and use the same scaling factor for the minor horizontal and the vertical components.
- Scale the larger of the records to the design spectrum, without determining the principal direction, and use this scaling factor for the other two components.
- Scale both records independently to the design spectrum, and scale the vertical spectrum to the vertical design spectrum.

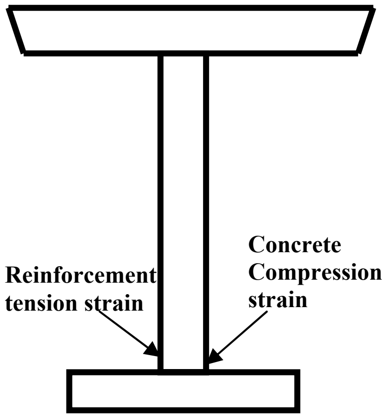
It might appear that the first, or possibly the second alternative is the most satisfactory. However, without a knowledge of the direction in which the major component should be applied with respect to the structure axes (which appears to be the current state of the science), multiple analyses would be required, with the direction of the horizontal components rotated by (say) 15^{o} between successive analyses to capture the maximum response. This approach also has the disadvantage of not recognizing how the design spectrum is obtained. As noted above, attenuation relationships used to generate site-specific design spectra normally are based on the average of fault-normal and fault-parallel response. Thus no specific principal direction is implied by the design spectrum. It thus appears that both records should be scaled to the design spectrum. Since the records will have low cross-correlation, it is expected that the elastic response displacement in any direction will be similar, when the two horizontal components are applied simultaneously in any two orthogonal directions.

Again, this is a structural engineer's interpretation of the problem, and its most appropriate current solution. Comments from seismologists would be welcome.

5. AVERAGING RESULTS FROM MULTIPLE ANALYSES

It is our opinion that the way in which results from multiple ITHAs are averaged by structural engineers may often be incorrect. The problem seems trivial, but in fact some consideration of rather simple probabilistic concepts seems necessary. Consider the response displacements of the simple bridge pier shown in Fig.9, subjected to uniaxial seismic excitation. The table indicates the maxima positive and negative displacements obtained from each of seven spectrum-compatible accelerograms, and the averages for the positive and negative directions. The maximum of the positive and negative displacement for each record is highlighted with bold text. What is the correct average response displacement? If the absolute value of the response displacement is of interest, then the average of the highlighted values should be used, giving a value of 7.023, which is 18% and 22% higher than the averages in the positive and negative directions respectively. However the absolute maximum displacement, regardless of sign, is unlikely to be a significant design parameter. If longitudinal response is considered, the displacement may be required to determine whether a movement joint closes up, causing impact, or opens up sufficiently to cause unseating. In this case since the critical displacements will be different in the different directions, the sign of the displacement is important.

Under transverse response, the displacements may be critical to determine whether reinforcement and concrete strain limits are exceeded in the plastic hinge region. The critical locations for positive displacement are indicated in Fig. 4.40. For these two locations and design parameters (concrete compression, and reinforcement tension strain), it would clearly be inappropriate to include the negative displacements in the averaging,



Maxima Displacements

Record	Δ Positive	Δ Negative
1	6.341	**-6.347**
2	4.491	**-7.945**
3	**8.948**	-3.179
4	4.486	**-6.543**
5	**6.148**	-4.543
6	**6.198**	-4.710
7	4.667	**-7.032**
Average	**5.954**	**-5.761**

Fig.9 Bridge pier response displacements from seven spectrum-compatible accelerograms

since these provide strains of the incorrect signs in the critical region.

The argument supporting the inclusion of the larger negative displacements is to note that the polarity of the excitation for a given record is arbitrary, and hence the displacement signs are arbitrary, indicating that averaging the larger of the positive and negative displacements is justified. This argument appear to us to be invalid, however, as it implies considering the response for both negative and positive polarities, (which essentially implies consideration of 14 rather than 7 accelerograms), selecting the response of the seven largest, and discarding the response of the seven smallest. From a probabilistic viewpoint it would appear equally valid to select the seven smallest and discard the seven largest. In fact it is obvious that no selection can be justified, and if both positive and negative polarities are considered, the results of all 14 records should be averaged. Using the data of Fig.9 this would result in a design displacement of 5.858, 17% lower than the value from averaging the peak displacement magnitudes.

The illogicality of selecting the maximum of the positive and negative displacements becomes even more apparent when 2-D or 3-D excitation is considered. If the critical polarity of each component is to be adopted, then four possible combinations of the 2-D components (+ve/+ve; -ve/–ve; +ve/-ve; -ve/+ve) and eight possible combinations of the 3-D components would need to be considered.

There are, however, additional problems with determining the correct average for response under multi-axial excitation. Consider the case of the simple wharf segment shown in plan view in Fig.10 which has considerable eccentricity between the centre of mass and the centre of strength (effective stiffness), as a result of short piers on the landward edge and long piers on the seaward edge. Because of the significant torsional response under longitudinal (X direction) excitation the transverse and longitudinal response cannot be decoupled, and simultaneous excitation in the orthogonal (X and Y) directions must be modelled.

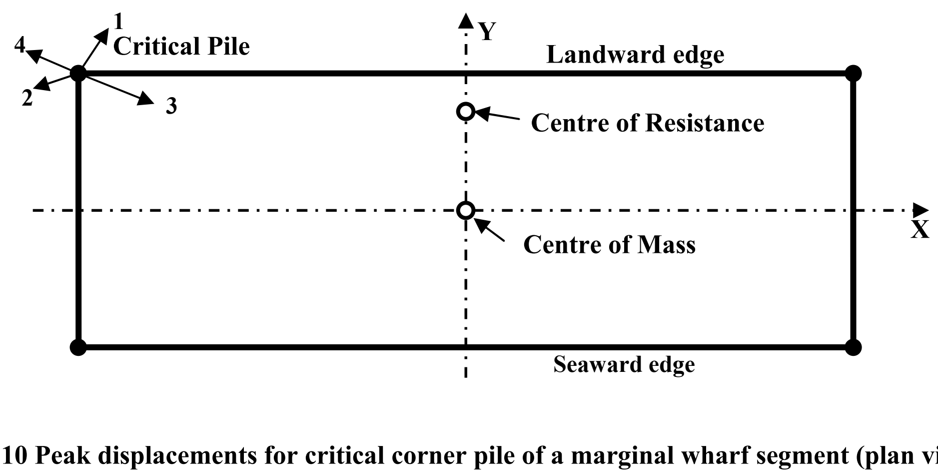


Fig.10 Peak displacements for critical corner pile of a marginal wharf segment (plan view)

The critical design parameter will be the average displacement from the seven (or more) pairs of accelerograms (record pairs) of a corner pile. As with the previous example of the bridge pier, the average displacement is of interest because it can be directly correlated to the pile limit strains.

Output from the ITHA will be in the form of X and Y displacements (as well as member forces etc). Normally the peak values $\boldsymbol{\Delta_{max,X}}$ and $\boldsymbol{\Delta_{max,Y}}$ from the analysis will also be listed in a summary table. However, it is not possible to combine these vectorially to obtain the peak response displacement for the record under consideration, since $\Delta_{max,X}$ and $\boldsymbol{\Delta_{max,Y}}$ are unlikely to occur at the same time. Instead, at each time interval **i**, the vectorial displacement $\boldsymbol{\Delta_i}$ must be calculated according to:

$$\Delta_i = \sqrt{(\Delta_{i,X}^2 + \Delta_{i,Y}^2} \tag{11}$$

This peak displacement will occur in a specific direction. In Fig.10, the critical displacements and directions for the first four of the record pairs are indicated by vectors. It will be clear that these cannot be averaged as scalars to obtain the design response displacement, as they occur in different directions. The correct method of averaging becomes apparent when one considers the condition that defines the limit strains. Let us first consider the peak response in a specified direction (say 15^o from the X axis). The average displacement response **in this direction** will define whether the limit strains on the diagonal defined by the direction are satisfactory. To obtain this average, the displacement **in this direction** must be obtained for each record pair, and for each time step. The full time-history of response displacements in this direction is searched for each record pair to obtain the maximum value, and the average of the maxima **in this direction** from the seven (or more) record pairs is found.

This procedure must be carried out for a series of directions around the full 360^o. Normally directions at 15^o intervals provide sufficient accuracy. The average response displacements in the different directions are then searched to find the critical direction, and thus the critical displacement.

It is noted that this procedure represents a departure from what has been customary practice in the past, but we believe it represents the only logical interpretation of the averaging process for response under a number of accelerograms. Comments from seismologists would be appreciated.

6. CONCLUSIONS

The recent refinements in seismic structural design place greater emphasis on the long period (1.5 – 7 sec) range of structural response than has been the case in the past. It was explained that this is not a function of significant changes to structural forms, but to a better appreciation of structural characteristics at maximum displacement response. It was pointed out that the long-period portion of design spectra are not adequately defined at present, and it is to be hoped that the major discrepancies between alternative representations of the period/displacement/ damping relationships can be resolved in the near future.

In our experience, structural engineers have a poor appreciation of the probabilistic basis of seismic design spectra and treat them as deterministic. The actual risk associated with small violations in design verification is not well appreciated by structural engineers. Although it is desirable that structural designs be prepared based on full reliability analysis, this is a desire unlikely to be realized in the near future. Help with meshing the deterministic design approach of structural engineers with the probabilistic basis of design spectra is needed.

A number of problems associated with determining structural response by inelastic time-history design verification were identified, and solutions to these problems were suggested that are in variance with current analysis procedures. The view of scientists more versed in probability theory on these suggested solutions would be greatly appreciated.

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