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ON THE ASSESSMENT OF STRUCTURAL DESIGN FACTORS FOR STEEL STRUCTURES

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SUMMARY

A simplified procedure is presented for assessing rational values of behavior factors for the design of steel structures in seismic areas: the procedure is based on the comparison between the results of linear and non-linear structural analysis and does not require a quantitative modeling of structural damage up to collapse. Non-linear dynamic analysis is based on numerical models which have been calibrated on the basis of extensive experimental testing. Some examples are shown with reference to cantilevered columns and concentric bracing systems.

THE BEHAVIOR FACTOR: DEFINITION AND ASSESSMENT METHODOLOGY

Most of modern codes for design in seismic areas (Refs. 1, 2) prescribe, for the medium and low structural frequency range, a design response spectrum of the type

$$S_a(T,\nu) = a_0\ R(T,\nu)\ /\ q \qquad (1)$$

where $R(T,\nu)$ is a normalized response spectrum, a_0 is the maximum ground acceleration and q (>1) is a coefficient which accounts for structural behavior in the dynamic nonlinear range.

The value of the behavior coefficient is usually stated as the ratio

$$q = a_u /\ a_y \qquad (2)$$

where a_y is the peak ground acceleration leading to the attainment, as detected by linear elastic analysis, of the design resistance, while a_u is the one leading to structural collapse. The a_u acceleration must be regarded as the average upon the values obtained by exciting the structure with a set of independent accelerograms which are compatible, in terms of frequency content, with the elastic response spectrum defined by the code.

Some inconsistency can arise in the application of definition (2) to the case of structures which are subject to significant geometric effects due to vertical loads (P-Δ effects), as is often the case of steel structures.

For example, in the simple case of a cantilevered member bearing a lumped mass and subject to a vertical load, the design bending moment due to seismic excitation can be derived, from (1), as

$$F = \alpha\, a_0 / q + \beta\, a_0 \qquad (3)$$

α and β being coefficients depending on the $R(T,\nu)$ spectrum and on the results of linear elastic analysis. The term $\beta\, a_0$ arises from the P-Δ effect; this is in fact related to the "actual" expected relative displacements, which can be estimated directly on the basis of elastic analysis.

Conversely, if C is the design flexural resistance of the considered structural element sized to resist a peak ground acceleration a_0, the behavior factor which is inherent in the design equals (from (3))

$$q = \alpha\, a_0 / (C - \beta a_0) \qquad (4)$$

Letting $q=1$ and $F=C$ in equation (3) the value of peak ground acceleration a_y can be also computed as

$$a_y = C / (\alpha + \beta) \qquad (5)$$

consistently with the above definition.

To state the maximum acceptable value $\bar{q}$ of the behavior factor equation (4) can be rewritten for $a_0 = a_u$, obtaining

$$\bar{q} = \alpha\, a_u / (C - \beta a_u) \qquad (6)$$

Deriving C from eq. (5) and susbstituting in (6) $\bar{q}$ can be finally expressed as

$$\bar{q} = a_u / a_y \; \frac{\alpha}{\alpha + \beta\,(1 - a_u / a_y)} \qquad (7)$$

It can be observed that expression (7) leads to the definition (2) of behavior factor only in absence of P-Δ effects, i.e. for $\beta=0$; in fact, by using expression (2) in (6) the resistance C is obtained as

$$C = \alpha\, a_y + \beta\, a_u$$

which contrasts with expression (5) and appears to be unnecessarily conservative. On the other hand, definition (2) appears to be consistent with the evaluation of the internal force as

$$F = (\alpha + \beta)\, a_0 / q$$

The last expression, however, appears to be physically questionable, since the benefit of ductile behavoir affects structural accelerations, accounted for by term α, but not relative displacements.

Regardless of the adopted definition of the behavior factor, the main difficulty for applying the described methodology lies in the necessity to perform a large number of dynamic structural analyses (experimental or numerical) up to collapse, in order to determine a_u. Experimental analyses, in fact, must be obviously restricted to a relatively small number of prototype structures, while numerical modelling of large-scale structures appears presently able to efficiently describe the overall nonlinear behavior of structural elements but not to account for the complex global and local damage processes which lead to structural collapse. Therefore the collapse of a structural element is usually detected by analyzing "a posteriori" the results of dynamic numerical computations, on the basis of collapse criteria involving response parameters such as dissipated energy or some definition of ductility demand. Even the calibration of such failure criteria, however, is far from well established, at least at the level of reliability that would be necessary for coding purposes.

For all these reasons a simplified procedure for q-factors statement which involves a "conventional" failure definition has been developed and tested for simple structural systems.

THE PROPOSED SIMPLIFIED PROCEDURE

The method (see Ref. 3) is suggested by the observation that for many structural systems an increase in the design q-factor (starting from unity) first corresponds to a decrease of structural nonlinear response with respect to linear; a further increase in the q value, however, leads to the inversion of this trend, until, beyond a value $\bar{q}$, the inelastic response becomes more severe than the elastic one.
According to the proposed procedure $\bar{q}$ is the maximum acceptable value of the behavior factor, under the condition that the corrisponding response be compatible, at least from a qualitative standpoint, with the ductility and energy dissipation capabilities of the structure.

A more detailed description of the procedure can be summarized in the following steps.

a - Choice of a reference peak ground acceleration.

b - Choice of a reference parameter suitable for the comparison between elastic and inelastic response (e.g. interstory drift).

c - Design of a set of structures on the basis of the above peak ground acceleration and of the rules prescribed by the code at study. These structures differ for the design q-factor only; this can be accomplished, for example, by fictitiously varying the yield tension.

d - Computation of the elastic peak response v_e in terms of the parameter chosen at point "b".

For each structure (i.e. for each q value) the following steps are then performed.

e - Computation of the inelastic peak response v_i: this is obtained via averaging upon the responses to a set of independent accelerograms, compatible with the elastic response spectrum defined by the code.

f - Comparison between v_e and v_i.

The value $\bar{q}$ of the behavior factor is finally defined as the maximum value satisfying the condition $v_i < v_e$.

The described procedure suggests the following considerations.

- The choice of modifying the structural resistance in order to vary the behavior factor (point "c") corresponds to decreasing the value of a_y in expression (7) until the structure actually collapses under the action of a previously stated value a_u (point "a") of peak ground acceleration.

- The value of the behavior coefficient is mainly stated on the basis of the dynamic nonlinear behavior of the structure, by assuming as conventional ultimate limit state the corresponding linear elastic response. The actual capacity of structural elements to withstand inelastic deformations is only taken into account by means of a qualitative comparison with the deformations implied by the choice of $\bar{q}$ as behavior factor. This is advantageous until precise quantitative models of damage and collapse will be available, but leads to inherent conservatorism. It must be noted, however, that beyond the $\bar{q}$ value the nonlinear response often increseas, as a function of q, in a very sharp way, making $\bar{q}$ a close estimate of the "true" maximum acceptable behavior factor.

- The results obtained are dependent on design rules other than the behavior factor value. It will be shown, for example, that in the case of cantilever members the $\bar{q}$ value is strongly affected by the way in which P-Δ effects are accounted for in the design.

Some results are presented hereinafter about the application of the proposed procedure to the statement of behavior factors for cantilevered columns and concentric bracing systems: the structural schemes here considered are simple but are deemed of interest in view of the uncertainties related to their dynamic behavior.

EXAMPLES OF APPLICATION

Dynamic non-linear analysis of cantilever columns and concentric bracing systems have been performed according to the following numerical modeling.

Cantilever columns The model is composed by a rigid linear element having a lumped mass on top and connected to the ground by means of a deformable cell: the bending stiffness and elastic limit rotation of the cell are calibrated by equating the natural frequency and elastic limit bending moment of the model to the ones of the real bar. The model accounts for second-order effects due to vertical load and for viscous damping, assumed equal to 3% of critical.

The behavior of the cell in the non linear hysteretic range is modeled (see Ref. 4) by discretizing the column section in a number of finite areas: each area follows a constitutive law which accounts for non linearity, Bauschinger effect, isotropic and kinematic hardening. Damage modelling includes low-cycle fatigue and fracture as well as local buckling. All columns were designed first by neglecting P-Δ effects and then by taking account of it, both in terms of deformability and of resistance.

Concentric bracing systems Single D.O.F. bracing systems have been considered: these are modeled as rigid pinned frames having two cross diagonal bars connected at midspan. Inertia is concentrated on top of the frame, while viscous damping is taken equal to 3% of critical. For each diagonal bar the model described in Ref. 5 and 6 has been adopted: this consists of two rigid elements connected by a deformable cell. The flexural elastic stiffness and elastic limit rotation of the cell have been obtained by equating the Euler load and the bending moment at elastic limit of the model to the ones of the real bar. The behavior of the deformable cell in the inelastic range is modeled according to the same criteria as quoted in the previous section. All bracing systems were designed by taking account of a single diagonal bar, both in terms of resistance and of stiffness.

Discussion of results The obtained results are summarized in figures 1 and 2. Fig. 1 refers to columns having slenderness λ=100, but with different section shape, natural period and axial to critical load ratio. Fig. 2 refer to concentric cross bracing systems having different section shape (back to back channels or angles) and slenderness.

Each curve represents, as a function of the design q factor, the ratio of the peak inelastic response v_i to the elastic response $v_y = v_e / q$: the ratio v_i / v_y can be regarded as the actual ductility demand imposed by the seismic action, while q represents the ductility implicitly assumed in the design. From the above definition of v_y it can be also noted that points on the bisecant of each graph satisfy the condition $v_i = v_e$. The maximum acceptable value $\bar{q}$ of the behavior factor can thus be obtained as the abscissa of the point of intersection between the bisecant and the numerical simulation curve.

As concerns the behavior of the cantilevered columns we can observe that, when P-Δ effect is considered in the design the $\bar{q}$ value (equal to about 5.5 for the HEA sections and to about 7 for the IPE sections) is not significantly affected by the axial load. Neglecting the geometric effect in the design gives rise to much lower acceptable design factors which, in addition, vary almost linearly as a funcion of the axial load value.

From fig. 2, finally, it can be noted how the behavior of bracing systems is strongly affected by the section shape. As concerns the braces slenderness it seems to play a more significant role in the case of back-to-back channels ($\bar{q}$=7.2 for λ=50 and $\bar{q}$=4.8 for λ=125) than in the case of angles ($\bar{q}$=3.2 for λ=50 and $\bar{q}$=2.4 for λ=125). This can be explained by observing that in the latter case the inelastic behavior is much more affected by damage due to local buckling and fracture, which tend to overshadow the effect of global buckling of the compressed bar.

CONCLUSIONS

The procedure described need to be applied to a much larger number of cases in order to state reliable conclusions about the behavior of the structures at study. The few results here shown, however, suggest the following considerations.

- Geometric effects strongly affect the behavior of framed structures. If these effects are properly accounted for in the design, however, it seems possible to state q factors values which are independent of vertical loads.
- Width to thickness ratios have a significant influence on the behavior of steel bent members. The results shown here, in fact, point out the better performance of IPE sections (b/t = 14) with respect to HEA sections (b/t = 20).
- The slenderness and section shape of the diagonal bars appear to be critical aspects for the behavior of single-storey concentric bracing systems .

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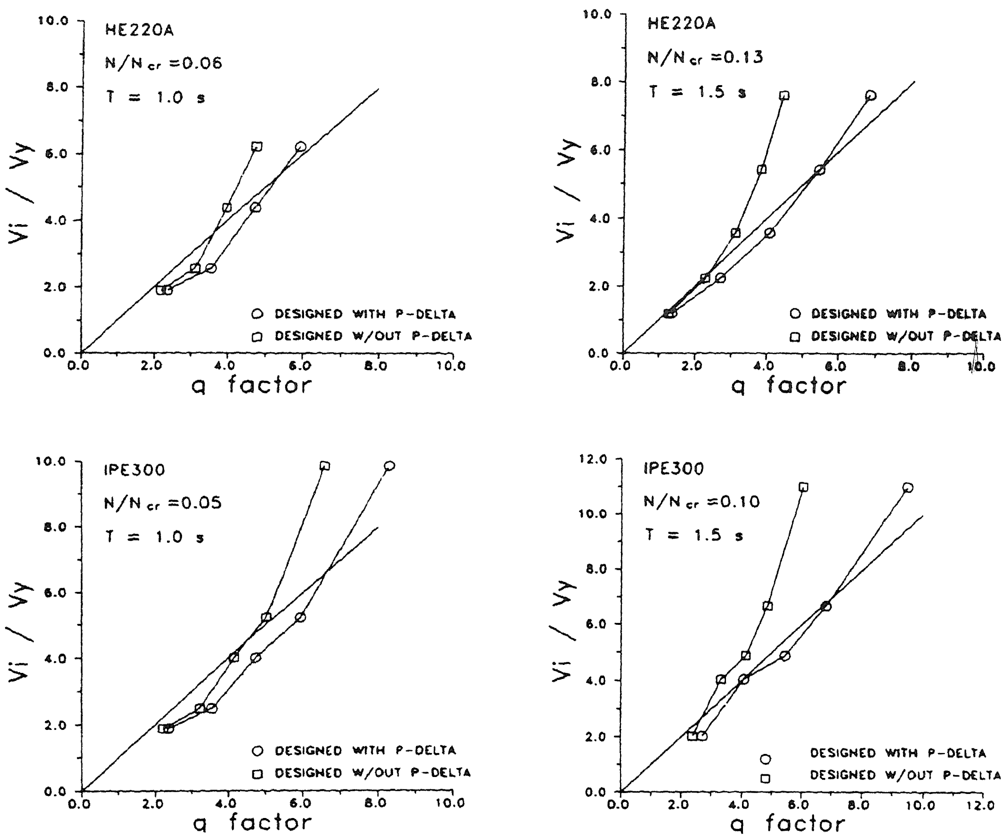

Figure 1. Determination of the behavior factor for cantilevered columns.

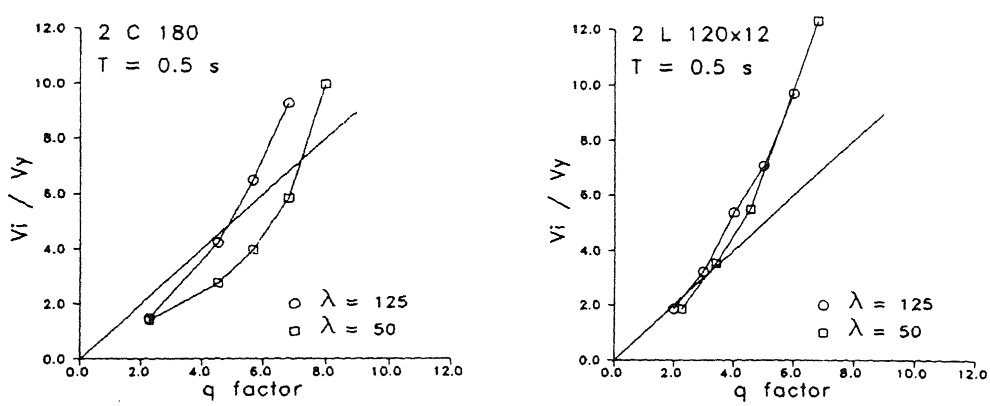

Figure 2. Determination of the behavior factor for bracing systems.