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Behavior versus ductility factors in earthquake resistant design

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ABSTRACT: In the present paper the relationship between natural period, behavior factor (response reduction factor R or q-factor) and the ductility demands imposed upon structures by design level earthquakes is investigated. The problem is studied first for Single-Degree-Of-Freedom (SDOF) systems and subsequently for two types of frames designed in accordance with the UBC provisions. It is found that for SDOF systems there is a strong dependence of the q-factor on the system's period, primarily in the low period range. This dependence becomes weaker for frame structures and disappears completely in the medium and long-period ranges, where code provisions, not affected by the value of q, often control the design. Moreover, frames designed using the static lateral force procedure of UBC exhibit a consistently good seismic behavior over the entire frequency range.

1 INTRODUCTION

Code provisions for earthquake resistant construction are based on the assumption that most structures will sustain inelastic deformations even under the action of moderately strong earthquakes. For this reason, code specified seismic design forces are much lower than the forces generated by design level earthquakes in structures responding elastically. Elastic earthquake actions are reduced to design level forces by dividing them with the so called response reduction factor (R) or behavior factor (q). This factor depends primarily on the capacity of the structure to sustain inelastic deformations, its energy dissipation capacity, its overstrength and the stability of its vertical load carrying system during the maximum induced inelastic deformations.

Behavior factors specified in ATC-3 (1978) and subsequently in UBC (1988) are constants that depend only on the type of structural system. Studies with SDOF systems, however, indicate that if the same level of ductility demand is to be maintained over the entire frequency spectrum, q-factors decreasing with frequency are required for stiff systems (e.g. Anagnostopoulos and Roesset (1973), Bertero (1986), Ridell et al. (1989)). Because of this, Eurocode No. 8 (1989) specifies constant q-factors only for periods $T > T_1$, where $T_1 = 0.2$ sec or 0.3 sec depending on the type of soil, while for $T \leq T_1$ it specifies a design spectrum such that the effective q-factors decrease with period from their maximum values at $T = T_1$ to 1.0 at $T = 0.0$ sec. For a comparison with ATC-3 or UBC, however, one must take into account that these two codes do not reduce the design spectrum in the high frequency region, but instead they extend the horizontal branch up to the zero period, thus reducing, in effect, the q-factors at $T = 0.0$ sec up to 2.75 times. Another difference between EC8 and ATC-3 or UBC is that the tentative values of the q-factors put forward in EC8 are quite lower than the corresponding values in ATC-3 or UBC.

Since the q-factor for which a building is designed has a direct bearing on the ductility demands it will experience in some future earthquake, it is obvious that the relationship between q-factors and ductility demands should be a basic criterion for arriving at appropriate values of the q-factors. In the present paper, relationships between structural period, ductility demands due to design level earthquakes and design q-factors are presented, first for SDOF systems and subsequently for frame structures.

2 SYSTEMS ANALYZED

The systems used for the present study are: (a) Simple, SDOF systems with bilinear force-deformation relationships (Fig. 1-a), (b) Five-story one-bay parametric frames (Fig. 1-b), and (c) Several real, 3-bay, frames with different number of stories (Fig. 1-c). In Fig. 1-a the definitions of the behavior factor q and the ductility factor μ are also given, with the explanation that F_{el} is the maximum force the earthquake would cause if the system were elastic.

The five-story, one-bay frame is a simplified parametric idealization of multistory buildings. Masses are assumed the same in all floors, while

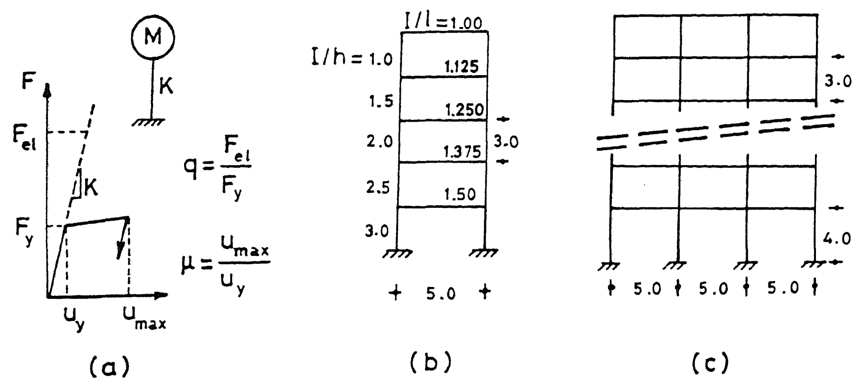

Fig. 1 Systems analyzed

member areas and moments of inertia are constant multiples of the first floor column properties, as indicated in Fig. 1-b. In this manner, the frame stiffness becomes function of a single parameter, the variation of which gives frames with different periods. Design forces for these frames are obtained from combinations of gravity and earthquake loadings, the first arising from the floor masses and the second computed in accordance with the UBC design spectrum for soil type 2 and peak ground acceleration 0.4 g. In order to make the computational work manageable, elastic behavior was assumed for all but the first and last story columns, based on results from analyses of real frames designed in accordance with UBC. In addition, the following relations were assumed for the section properties of columns:

$$\left.\begin{aligned} W &= 0.885\ z \\ A &= 0.0625\ z \end{aligned}\right\} \qquad (1)$$

where A = section area (cm^2), W = section modulus (cm^3) and z = plastic section modulus (cm^3). A last set of assumptions involved allowable stresses: it was assumed that $\sigma_y/\sigma_{al} = 1.5$ for flexure, $\sigma_y/\sigma_{al} = 1.67$ for compression, where σ_y and σ_{al} are yield and allowable stresses, respectively, and that allowable stresses increase by 33% for the loadings that include earthquake. Under the foregoing simplifications, the following design equations are obtained for yield moments M_y and axial forces N_y

Beams:

$$\left.\begin{aligned} M_y &\geq 1.7\ M_G \\ M_y &\geq 1.275\ M_{G+E} \end{aligned}\right\} \qquad (2)$$

Columns (length unit cm):

$$\left.\begin{aligned} M_y &\geq 1.7\ (16 N_G + M_G) \\ M_y &\geq 1.275\ (16 N_{G+E} + M_{G+E}) \\ N_y &= 0.0625\ M_y \end{aligned}\right\} \qquad (3)$$

where M_G, N_G = bending moment and axial force due to gravity loads and M_{G+E}, N_{G+E} = bending moment and axial force due to the combination of gravity and earthquake loads. Besides the above, columns were also designed to resist an axial force $N = N_G + 0.375\ qN_E$ and at the same time the condition $N_{G+E} \leq 0.4\ A\sigma_y$ was also met. Finally, the drift limitation $\delta \leq (0.03\ h)/q$, where δ = interstory drift, h = story height and q = behavior factor, was always satisfied. The above design procedure permits member strength determination independent of its stiffness, so that the strength variation resulting from different q-factors does not change the period of the frame (same as for the SDOF systems).

The third group of structures is a series of 3-bay, steel frames, with number of floors varying from 2 to 20 (Fig. 1-c). These frames were designed according to the UBC code with the equivalent static method for soil type 2, z = 0.4 and for different values of the q-factor. It is noted here that the equivalent static method, compared to the dynamic response spectrum method used to design the parametric group of 1-bay, 5-story frames, results in substantially higher seismic design forces due to differences between actual and design periods.

The systems described above were each subjected to 10 synthetic accelerograms compatible with the UBC design spectrum for soil type 2. The mean response spectrum of the ten motions is shown in Fig. 2, where it is compared with the target design spectrum. In addition to the artificial motions, five historical accelerograms were also used to analyze the SDOF systems. These are : El Centro-NS (1940), Taft-S69E (1952), Eureka-N79E (1954), Olympia-N86E (1949) and Parkfield-Array No.2-N65E (1966).

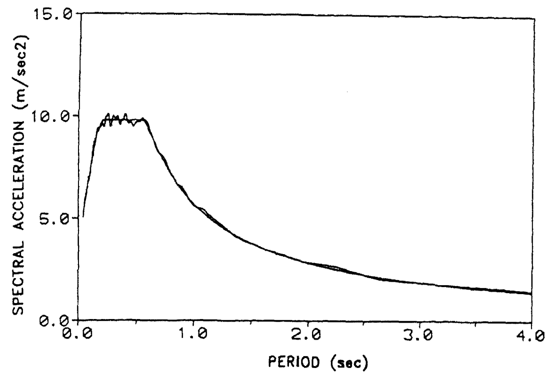

Fig. 2 Design spectrum and mean spectrum of 10 artificial earthquakes

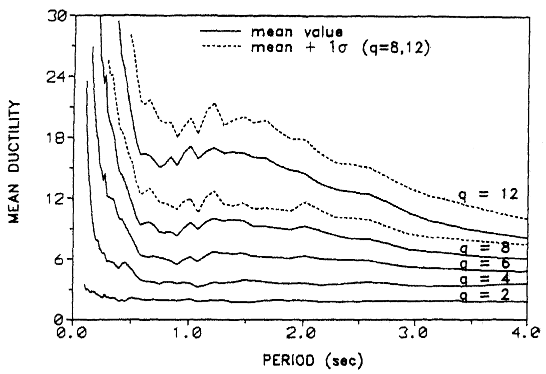

Fig. 3 Ductility versus period for SDOF systems (10 artificial earthquakes)

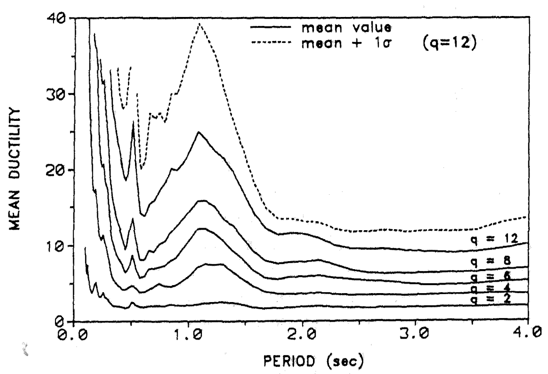

Fig. 4 Ductility versus period for SDOF systems (5 real earthquakes)

3 SINGLE-DEGREE-OF-FREEDOM SYSTEMS

The SDOF systems were generated by changing their stiffness K, while keeping their mass constant. For each system having period T and for each motion it is possible to compute either the ductility factor μ for a given value of q or the value of q required for a given value of μ. In both cases, the maximum elastic force F_{el} is first computed, from which the yield level $F_y = F_{el}/q$ corresponding to a specified value of q can be found. Thus, the non-linear system is completely defined and its response to any base motion can be computed by numerical integration to obtain the ductility demand factor μ. Computation of the q-factor, (i.e. of the yield level F_y) required to produce a specific value of μ can be accomplished through a procedure of successive approximations.

The variation of $\bar{\mu}$, the mean value of μ for a group of motions, with period T under constant values of q is presented in Figures 3 and 4, while the variation of $\bar{q}$ with period T, under constant values of μ, is given in Figures 5 and 6 for the synthetic and historical earthquakes, respectively. The variability of the results due to different motions can be assessed from the dashed lines giving mean plus one sigma values. From these Figures, the strong dependence of the q and μ factors on the period T for values of T $\leq$ ~0.6 sec becomes obvious, thus suggesting a need to use decreasing values of q as the system becomes stiffer.

4 FIVE - STORY , ONE - BAY , PARAMETRIC FRAMES

These frames have fundamental periods T = 0.1 sec, 0.2 sec, 0.3 sec etc. up to T = 1.8 sec. The end period T = 1.8 sec results from the interstory drift limitation $\delta \leq (0.03\ h)/q$ or $q\delta = \delta_{el} \leq 0.03\ h$, which obviously does not depend on q but only on the period T. For each value of T five frames were designed, with q = 2, 4, 6, 8, 12, and subsequently analyzed for the 10 artificial motions. The analyses were carried out with the program DRAIN-2D (Kanaan and Powell (1973)), which can perform non-linear dynamic analyses using a plastic hinge model. As measure of ductility demands, the maximum rotational ductility factor (μ) of all the frame members was used.

Figure 7 shows the variation of the mean ductility demands (for the 10 motions) with period T under constant values of q. We observe that for T $\leq$ ~0.7 sec there is a rapid increase in ductility demands with decreasing T and increasing q, while for T $\geq$ ~0.7 sec, ductility demands stay at reasonable levels even for q = 12. Moreover, we see that as T increases the influence of q on ductility is reduced, to the extent that for T = 1.8 sec, a reduction of q from 12 to 6, i.e. doubling of the seismic design base shear, results in a reduction of μ only by 13%.

To obtain the variation of q with T for constant values of μ, it becomes necessary to find the relations $\bar{\mu}$-q at each period T. These relations, found by successive analyses, are shown in Fig. 8, where the ordinate is the mean value of μ (maximum for

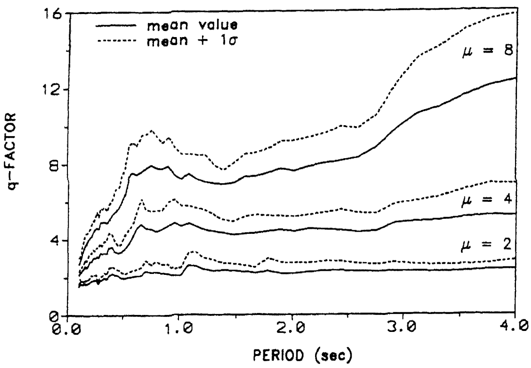

Fig. 5 q-factor versus period for SDOF systems (10 artificial earthquakes)

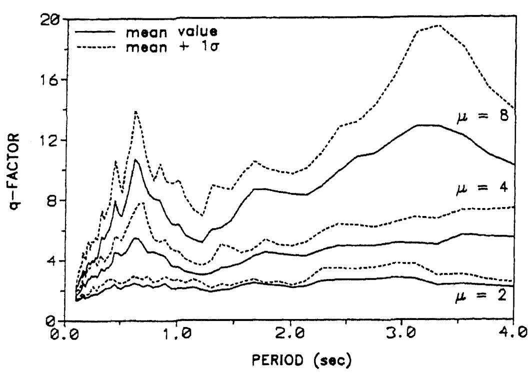

Fig. 6 q-factor versus period for SDOF systems (5 real earthquakes)

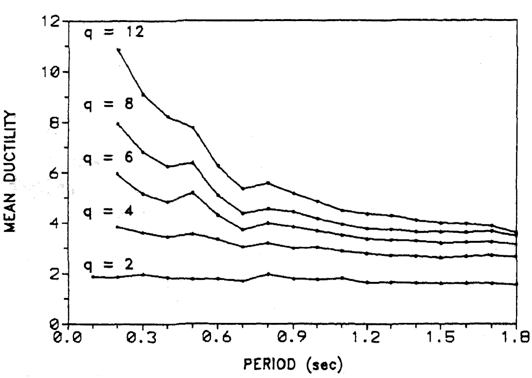

Fig. 7 Ductility versus period for 5-story parametric frames

each frame) from the 10 synthetic motions. We observe that as the period and the q factor increase, the corresponding portions of the μ-q curves tend to become horizontal, which indicates that the ductility

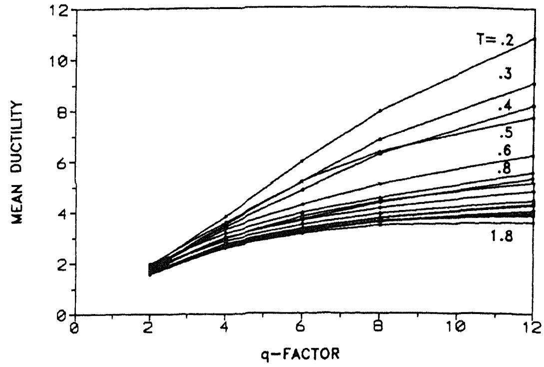

Fig. 8 Ductility versus q-factor for 5-story parametric frames

demands become quite insensitive to changes in the values of q. Under such conditions, estimates of q for specified values of μ cannot be very reliable. For this reason, q-T relations were computed only up to T = 0.5 sec and are shown in Fig. 9. Qualitatively, these relations are similar to those derived for the SDOF systems (Fig. 5), but the values of q for the same ductility factor are now higher.

5 REAL FRAMES WITH 2 TO 20 STORIES

Use of actual frames in the study of our problem complicates matters considerably, mainly because member strength cannot be specified independent of stiffness and consequently the frame period T becomes function of the behavior factor q. Therefore, a change of q leads to a different period, so a simultaneous variation of the two basic parameters of the problem becomes inevitable (this was not the case with the SDOF systems or with the parametric one-bay frames). The period variation was effected by varying the number of floors and for this reason frames with 2, 3, 5, 7, 10, 12, 14, 18 and 20 stories were designed, each for five different values of q : 2, 4, 6, 8 and 12. The design periods of these frames, according to the approximate UBC formula for moment resisting steel frames, are between 0.37 sec and 1.86 sec, while their actual periods are between 0.43 sec (2-story with q=2) and 3.22 sec (20-story with q = 12). For q = 12, which is the code specified value, the 2-story frame has an actual period of 0.63 sec while the design period according to the code formula is only 0.37 sec. One - story frames have somewhat lower periods but were not included in the study because their design was controlled by gravity loads.

Each of the 45 frames was analyzed with DRAIN-2D for the 10 artificial motions. Figure 10 shows the variation of mean ductility demands (from the 10 motions) with period under constant values of q. The important observation here is that for each value of q, the mean ductility demands are practically constant, independent of period. Moreover, it is seen

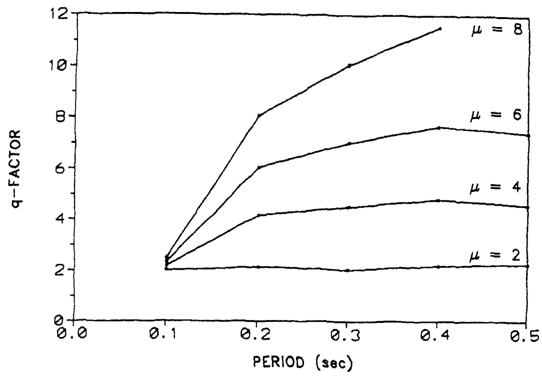

Fig. 9 q-factor versus period for 5-story parametric frames

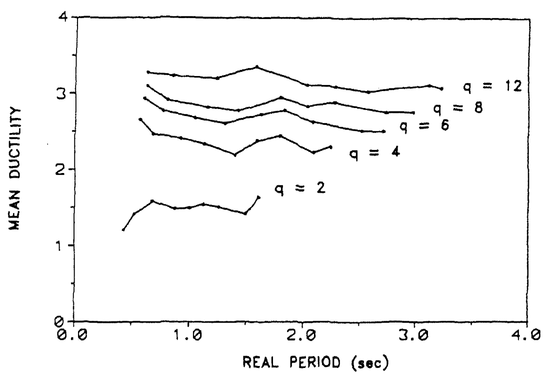

Fig. 10 Ductility versus period for real frames

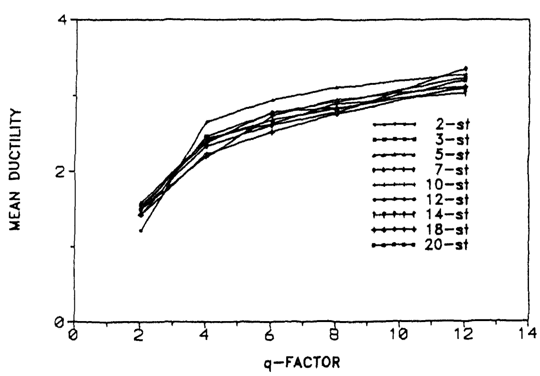

Fig. 11 Ductility versus q-factor for real frames

that for q = 12, i.e. the value specified by UBC for special moment resisting frames, ductility demands are between 3.0 and 3.35, which are values indicative of very good behavior. These demands are substantially lower than those found for the parametric frames of comparable periods, mainly because of the higher lateral forces for which the real frames are designed due to the lower period resulting from the approximate code formula. It must also be mentioned that for the frames with 10 stories or more, drift limitations control the design, for $q > 8$.

The μ-q relations for the same frames are shown in Fig. 11, indicating a good and consistent behavior under design level earthquakes, both for stiff and flexible buildings. The closeness of these curves and their nearly horizontal slope as q increases, renders the calculation of q-T curves for different values of μ meaningless. Moreover, Figures 10 and 11 justify fully a constant value of q for steel frame structures, independent of period T.

6 CONCLUSIONS

The conclusions from this study may be summarized as follows:

1. The constant value of the behavior factor $q = 12$ specified by UBC (where it is called response reduction factor R_w) for special moment resisting steel frames, together with other code provisions, lead to uniformly low ductility demands irrespective of the frames' periods. (maximum rotational ductilities $\mu \sim 3.0$ to 3.4). The periods of such frames are typically greater than ~ 0.4 sec, i.e. outside the spectral region where demands for lower values of q appear.
2. For stiff structures ($T \leq \sim 0.5$ sec), a reduction of q with decreasing period is required to keep ductility demands at acceptably low levels.
3. A frame structure designed with the equivalent static procedure of UBC, possesses significant amounts of overstrength, which reduce the ductility demands imposed by design level earthquakes. Therefore, estimates of ductility demands for buildings based on SDOF systems can be grossly misleading.
4. As structural periods increase, the sensitivity of ductility demands to changes in the design behavior factor decreases. For example, at T = 1.8 sec, a reduction of q to half its value, i.e. a 100% increase in the design base shear, decreased the ductility demands due to design level earthquakes only by 13%. This confirms the well known fact that very often, increasing the ductility capacity of a structure provides better protection against earthquakes than the increase of the seismic design forces.

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