DUCTILITY REQUIREMENTS FOR SHEAR
WALL STRUCTURES IN SEISMIC AREAS

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SUMMARY

In several studies required displacement ductility factors for single degree of freedom systems in seismic areas are derived. In the present paper a possibility to use them for the determination of ductility requirements for multistory shear wall structures is established. The seismic base shear of the inelastic structure as well as the influence of the damping hypothesis on the recommended numerical values is also investigated.

INTRODUCTION

The need to insure a satisfactory inelastic behaviour of antiseismic structures is often expressed by the demand to attain an adequate displacement ductility factor μ of the structural system. As shown in Ref. 1, in the design of multistory shear wall structures this requirement leads to the demand of a curvature ductility factor

$$\mu_\phi = 1 + \frac{(\mu-1)\ h^2}{3 l_p (h-0.5\ l_p)} \qquad (1)$$

of the shear walls, where h is the height of the structure and l_p the length of the plastic hinges at the base of the shear walls. In several studies (e.g. in Ref. 2) required displacement ductility factors μ_1 for single degree of freedom systems in seismic areas are derived. However, their introduction in Eq. (1) can be nonconservative because of the influence of higher vibrational modes, occuring at multi degree of freedom systems, on ductility demands. In the present paper this influence is investigated, establishing a possibility to use required displacement ductility factors of single degree of freedom systems for the determination of ductility requirements of multistory shear wall structures. Simultaneously the influence of higher vibrational modes on the base shear of inelastic shear walls is also investigated.

STRUCTURAL MODEL

For the derivation of ductility demands of multistory shear wall structures nonlinear dynamic analyses are performed for slender cantilever shear walls with n = 5, n = 10 and n = 20 lumped story masses on their height, as shown in Fig. 1 a. The yielding moment of a shear wall is given by the relation

$$M_y = c\ M_1\ g/S_a(T_1) \qquad (2)$$

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with

c - yielding level factor;
$S_a(T_1)$ - design value of the acceleration response spectrum for the fundamental vibrational period T_1 of the structure;
M_1 - overturning moment at the base of the shear wall, due to the design seismic load, corresponding to the fundamental vibrational mode;
g - acceleration of gravity.

For a system with n = 1 (cantilever column with a single lumped mass on its end), having the weight G and the yielding moment $M_y = h\ F_y$, where F_y represents the yielding force of the system, Eq. (2) turns into

$$F_y = c\ G. \tag{3}$$

For each number of story masses 3 different yielding levels (3 values of c) and 4 different fundamental vibrational periods (T_1 = 0.2 s, T_1 = 0.4 s, T_1 = 0.8 s and T_1 = 1.6 s) are considered, as shown in Fig. 1 b. The yielding level 2 corresponds to the design forces of the German Seismic Code DIN 4149.

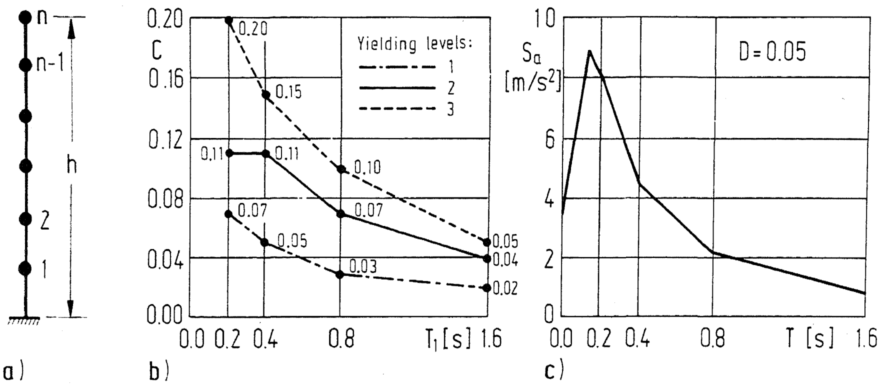

Fig. 1 a) Structural system, b) Yielding levels,
c) Mean acceleration response spectrum

EARTHQUAKE EXCITATION

The shear wall structures, represented by the model shown in Fig. 1 a, are analysed under the acceleration time histories of a set of 10 strong motion records, belonging to 5 real earthquakes: San Francisco Golden Gate Park 1957.3.22 (I = 7, M = 5.3), Helena (Montana) 1935.10.31 (I = 8, M = 6.0), Hollister (Calif.) 1949.3.9 (I = 7, M = 5.2), Ferndale (Calif.) 1975.6.7 (I = 7, M = 5.7) and Oroville (Calif.) 1975.8.1 (I = 7, M = 5.8). The aforementioned earthquakes have been chosen in the magnitude range of M = 5 to M = 6, which is the characteristic range for the seismicity of Central Europe. The purpose of the paper being to analyse the influence of higher vibrational modes on ductility requirements, the consideration of this magnitude range, with predominant higher frequencies of the ground shaking, is conservative. The acceleration time histories have been scaled for the MSK intensity level of I = 8, using the properties of Fourier amplitude spectra of accelerations, as shown in Ref. 2. The mean response spectrum of the scaled accelerograms, corresponding to the damping ratio D = 0,05, is given in Fig. 1 c.

REQUIRED CURVATURE DUCTILITY

In order to establish relations between the inelastic behaviour of multistory shear wall buildings and that of single degree of freedom systems, the computed curvature ductility factor of each shear wall is compared with that calculated for a cantilever column with a single lumped mass on its end, under the same earthquake excitation. The considered yielding level factor c of both systems is the same, but the vibrational periods of the simplified system equals the fundamental vibrational period T_1 of the shear wall. Expressing the required curvature ductility factor of the shear wall by the relation

$$\mu_\phi = 1 + \frac{(\lambda \mu_1 - 1) h^2}{3 l_p (h - 0.5\, l_p)} \qquad (4)$$

analogous to Eq. (1), a correctional factor λ is derived, introducing the influence of higher vibrational modes. Performing the nonlinear dynamic analyses of the shear walls for the 10 accelerograms, mentioned before, by considering the hypothesis of Rayleigh damping with the first two damping ratios $D_1 = D_2 = 0.05$, the mean values $\bar{\lambda}$, shown in Fig. 2, and the values $\lambda = \bar{\lambda} + s$ (with s = standard deviation), shown in Fig. 3, are obtained. They can be expressed in the range $T_1 \leq 1.6$ s, $5 \leq n \leq 20$ by the approximate relations

$$\bar{\lambda} = 1 + 0.25\, T_1, \qquad (5)$$

$$\lambda = 1 + 0.60\, T_1, \qquad (6)$$

with T_1 in sec. The aspect of the λ-curves denotes an increasing importance of higher modes with increasing fundamental vibrational period.

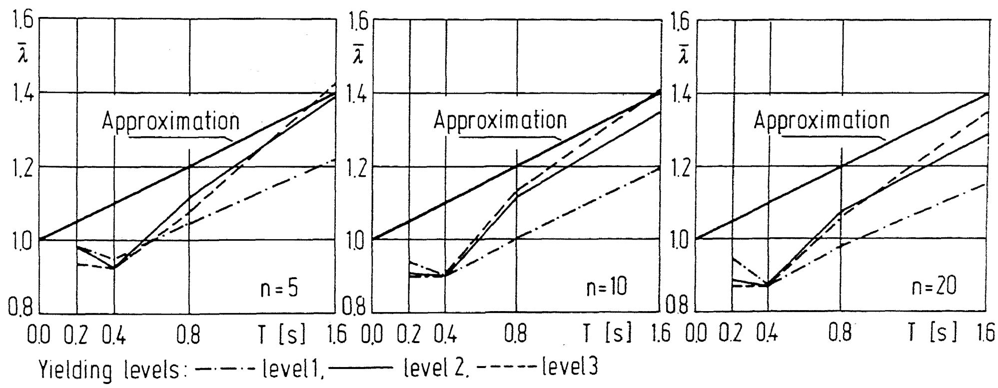

Fig. 2 Correctional factors $\bar{\lambda}$ for n = 5, n = 10 and n = 20

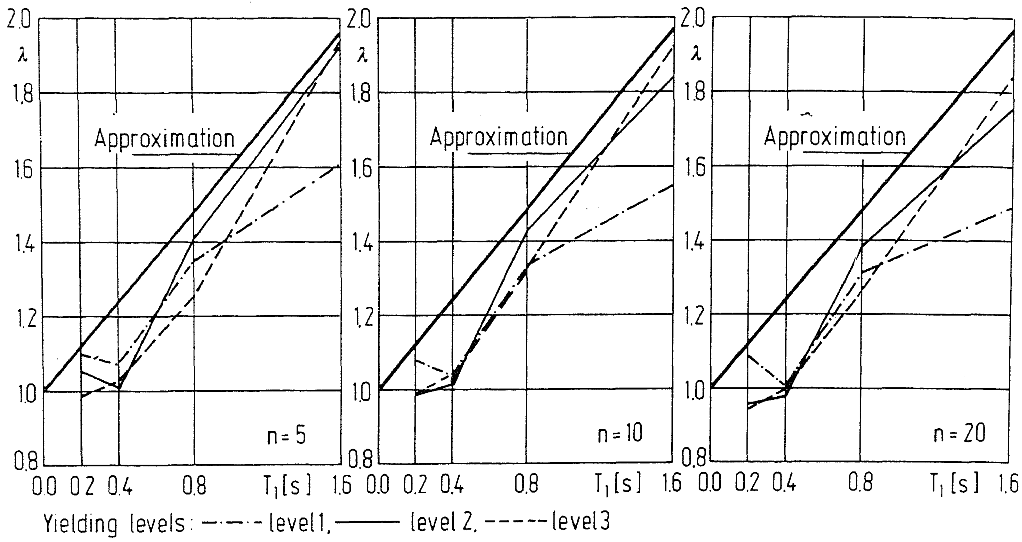

Fig. 3 Correctional factors $\lambda = \bar{\lambda} + s$ for n = 5, n = 10 and n = 20

BASE SHEAR

The nonlinear dynamic analyses of the shear walls lead to values of the base shear Q, much greater than the design value Q_1, corresponding to the fundamental vibrational mode. Considering for M_1 and Q_1 the design values of the German Seismic Code (yielding level 2), the ratios $\bar{Q}/Q_1$, shown in Fig. 4, and Q/Q_1, shown in Fig. 5, are derived, where $\bar{Q}$ is the mean value of the computed base shears, Q being defined as $Q = \bar{Q} + s$ (with s = standard deviation). Approximate expressions for these ratios, analogous to Eq. (5) and (6), are

$$\bar{Q}/Q_1 = 2(1+T_1)\,(M_y/M_1)^r \leq 2\,(2+T_1), \tag{7}$$

$$Q/Q_1 = 3(1+T_1)\,(M_y/M_1)^r \leq 3\,(2+T_1), \tag{8}$$

with T_1 in sec and r = 0.2 - 0.5. The upper bound of the expressions (7) and (8) corresponds for $0.4\ s \leq T_1 \leq 1.6\ s$ to the elastic behaviour of the structure, occuring for great values of M_y. For $T_1 = 0.2$ s it was found $\bar{Q}/Q_1 \leq 8$ and $Q/Q_1 \leq 13$. Comparing the relation (7) with the recommendations given in Ref. 3, p. 47, it seems that there, at least for the considered range of yield levels and earthquake magnitudes, the influence of higher vibrational modes on the base shear is underestimated, but the influence of the yielding level is overestimated.

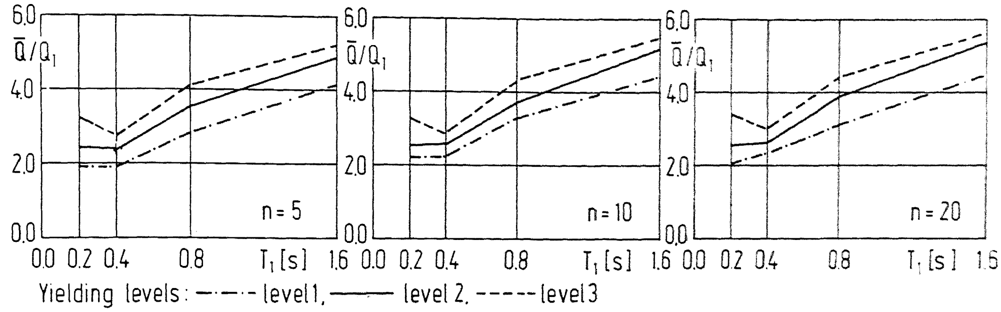

Fig. 4 Base shear ratios $\bar{Q}/Q_1$ for n = 5, n = 10 and n = 20

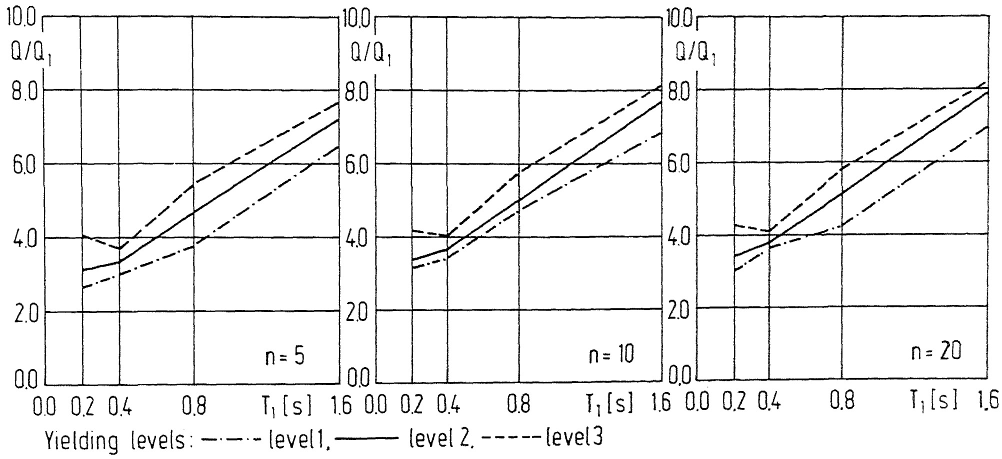

Fig. 5 Base shear ratios Q/Q_1 for n = 5, n = 10 and n = 20

INFLUENCE OF THE DAMPING HYPOTHESIS

In order to estimate the influence of the damping hypothesis on the resulting correctional factor λ and on the computed base shear, beyond the investigations described in the above, considering the hypothesis of Rayleigh damping, computations are carried out also for the hypothesis of mass proportional damping. The results, depicted in the Figs. 6 – 8, lead to the approximate relations

$$\bar{\lambda} = 1 + 0.35\, T_1, \tag{9}$$

$$\lambda = 1 + 0.80\, T_1, \tag{10}$$

$$\bar{Q}/Q_1 = 2 + 3.0\, T_1, \tag{11}$$

$$Q/Q_1 = 3 + 4.5\, T_1, \tag{12}$$

with T_1 in sec. They show the order of magnitude of the influence of the damping hypothesis upon the considered quantities, highly dependent on the higher vibrational modes.

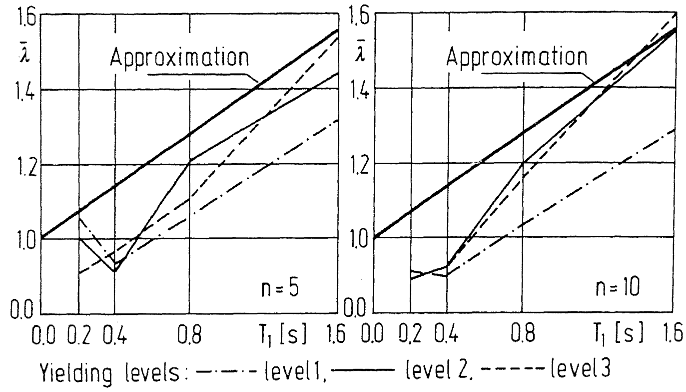

Fig. 6 Correctional factors $\overline{\lambda}$ for n = 5 and n = 10, mass proportional damping

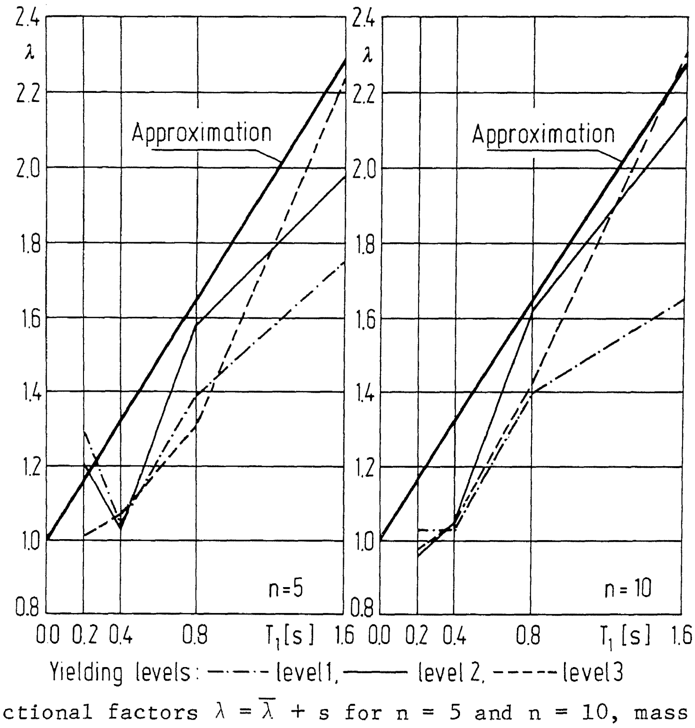

Fig. 7 Correctional factors $\lambda = \overline{\lambda} + s$ for n = 5 and n = 10, mass proportional damping

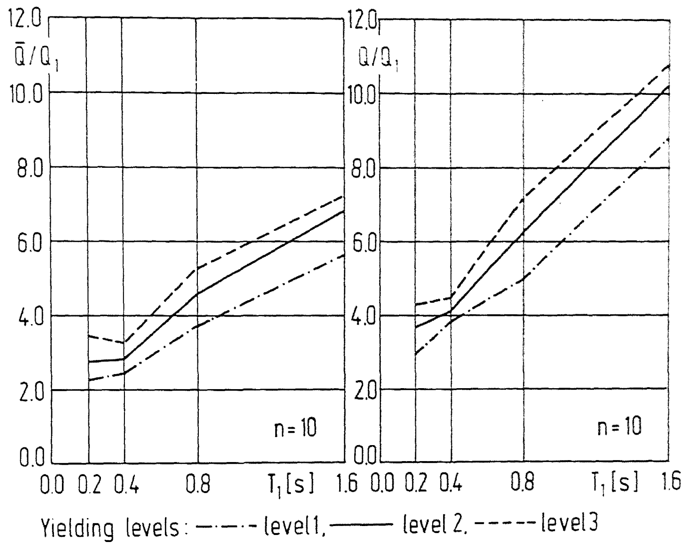

Fig. 8 Base shear ratios $\overline{Q}/Q_1$ and Q/Q_1 for n = 10, mass proportional damping

CONCLUSIONS

In the paper the influence of higher vibrational modes on the required curvature ductility factors of shear walls and of their inelastic base shear under seismic excitation is investigated. The approxitate relations derived may be used in the design of multistory antiseismic shear wall structures.

REFERENCES

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