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BEHAVIOUR COEFFICIENT ASSESSMENT FOR SOFT STOREY STRUCTURES

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SUMMARY

It is the objective of this study to make the assessment of behaviour coefficients (q factors) for soft-storey reinforced concrete structures under seismic loading. For this purpose, a probabilistic safety checking analysis based on damage indices is used.

A methodology for the assessment of q factors using vulnerability functions is followed, computing the probability of failure through the damage vulnerability functions and the probabilistic quantification of both the seismic action (hazard curves) and the structural element capacity.

The vulnerability functions represent the non-linear relationship between the intensity of the actions and the values of the action-effects, quantified in terms of a damage index: the Miner damage index. Having defined the vulnerability functions, the values of the probability of failure, P_f , are computed.

To perform the non-linear dynamic analyses of the soft-storey structures, a *fibre model* element was developed and implemented in Draind-2d, allowing for the use of complex material behaviour laws under cyclic loading and reflecting the section behaviour rather accurately. The geometric non-linearity, i.e. second-order (P-δ) effect in the column model is considered in a simplified way.

In this work, some soft-storey plane frames with different characteristics, designed according to Eurocode specifications (EC2 and EC8), are analysed.

Finally, an analysis is made regarding the admissible q factors for the examined structures.

INTRODUCTION

It is recognised that structural configuration plays an important role on the seismic behaviour of structures. In recent earthquakes, irregular structures have shown an inadequate behaviour when compared with similar regular structures and it can be said that most of the observed collapses are related to some extent to configuration problems or wrong conceptual design.

Different types of configuration irregularities are responsible for a deficient structural behaviour, non-uniformity in terms of stiffness and mass distribution, both in plant and in elevation, being the most important ones.

Stiffness irregularity in elevation may be induced by a sharp transition in the stiffness of the vertical elements, either due to a sudden change on the column's cross sections, or by the interruption of existing shear walls. It can also be due to different lengths of the vertical elements from one storey to the other. Finally, it can also be induced by the existence of stiff non-structural elements in some of the storeys. In some situations, there is even a combination of these negative aspects, such as in the cases where, in the first storey, the columns are

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significantly longer than in the other stories and there is also an absence of infill masonry panels that exist only in the upper stories.

The soft storey concept was originally conceived as a design methodology, which, similarly to the base isolation, envisaged the concentration of all structural deformations in a single storey, thus avoiding the occurrence of structural damage in most of the structure. Obviously, the first experiences were disastrous, as was the case with the Imperial County Services Building, which almost collapsed during the 1979 Imperial Valley Earthquake.

Nevertheless, soft storey buildings still tend to be a common choice in many cases, most of the times due to architectural reasons.

The existing rules for designing such structures do not yet completely reflect the past observations and only some empirical rules have been adopted, namely inducing higher seismic coefficients whenever certain regularity criteria are not satisfied.

An example of these simplified rules is present in Eurocode 8 [CEN, 1994b]. According to these seismic regulations and specifically in what regards criteria for regularity in elevation, it is specified that the regularity criteria is satisfied if "both the lateral stiffness and the mass of the individual storeys remain constant or reduce gradually, without abrupt changes, from the base to the top". If these criteria are not satisfied, the behaviour factor that can be used for design purposes must be reduced by a coefficient k_r, reflecting the regularity in elevation, which is equal to 0.8 in the case of non-regular structures. As the k_r coefficient is equal to 1.0 in the case of regular structures, this means that, in the case of irregular structures, the seismic coefficient or the response spectrum ordinates are affected by a factor equal to 1.25 (1/0.8) regardless of the type and severity of the irregularity. For instance, the behaviour factor is the same regardless of corresponding to a soft storey or a setback structure.

This shows that there is a need for a clearer characterisation of structural irregularity and that different design procedures or design parameters should be used according to the type or severity of the irregularity and that some types of irregularities should even not be allowed.

It is the objective of this work to analyse the structural response of soft storey structures, aiming at showing that their safety for seismic loading is lower than the safety exhibited by regular structures designed according to similar procedures.

ANALYSIS METHODOLOGY

Damage index as a control parameter

Global failure of building structures is often considered to occur when a global mechanism develops or in the presence of excessive interstorey drift. For soft storey structures, global mechanisms occur due to a "sidesway" mechanism involving local failure of both extremities of all the columns at the soft storey level.

In this work, local failure is characterised by means of a local damage index. The Miner damage index [Bento and Azevedo, 1995] takes into account the number of cycles for different non-linear deformation amplitudes, with failure at a given structural member occurring when a certain limit value for the index (in this case 1.0 or 100%) is reached. It assumes that damage accumulation is related to the process of fatigue for a low number of cycles and is defined by equation (1).

$$I_M = \sum_{i=1}^{L} \frac{n_i}{N_{u,i}} \tag{1}$$

where $N_{u,i}$ is the number of cycles leading to collapse with an amplitude equal to the amplitude of cycle i, n_i is the number of cycles with that same amplitude and L the number of different cycle amplitudes.

Safety checking

To assess design q factors for soft storey reinforced concrete structures under seismic loading, this study follows a non-deterministic approach assuming a probabilistic quantification of both the seismic action (hazard curves) and the structural element capacity (reliability analysis). The methodology is based on the use of vulnerability functions, relating the damage index to the seismic input level. The probabilistic definition of the seismic action, together with these vulnerability functions allows a probabilistic quantification of the structural response in terms of damage and an assessment of the probability of failure.

The safety checking process starts by a preliminary design of the structure, based on linear analysis values and corrected by q factors. In this first step some decisions are made, some related to the structural elements' characteristics, others related to the q factor values and others related to design procedure to be followed in designing the reinforced concrete structure's columns.

A statistical distribution of the action is represented by the $f_a(A)$ function which corresponds to the probability distribution of the seismic motion severity (A, in this study the peak ground acceleration) and which is consistent with the site seismicity. The action effects are quantified by the probability distribution $f_{aE}(D)$ of the earthquake imposed Miner damage index (D).

The vulnerability function, represented in Fig. 1 by V(A), allows the probabilistic transformation of the action into the action effects space, represented in the damage axis (D). It allows the evaluation of the probability density function $f_{aE}(D)$ of the action effects based on the probability density function of the mean peak ground acceleration $f_a(A)$.

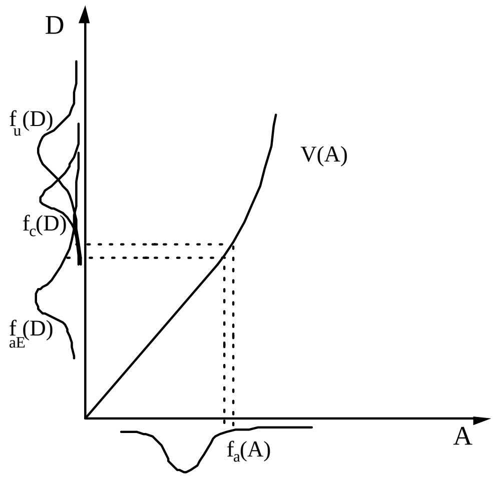

Fig. 1 - Vulnerability Function Methodology.

The $f_u(D)$ function represented in Fig. 1 is the probability density function of the ultimate damage indices and probabilistically quantifies the resistance or capacity, in terms of allowable damage. The $f_c(D)$ function represents the probability density function of failure and thus indicates the damage values, and consequently the seismic action level, for which failure is more likely to occur. The probability that the action effects exceed the capacity, or the probability of failure, is given by the integral of $f_c(D)$. The probability of failure (P_f) can be evaluated by the convolution of $f_{aE}(D)$ with the probability distribution $f_u(D)$ according to equation (2),

$$P_f = \int_0^\infty f_{aE}(D) \int_0^D f_u(D)\, dD \qquad (2)$$

or according to equation (3),

$$P_f = \int_0^\infty \left(1 - F_{aE}(D)\right) f_u(D)\, dD \qquad (3)$$

where $F_{aE}(D)$ represents the cumulative distribution of the action effects.

The value of the probability of failure P_f can then be compared with a reference value P_{f0}, which represents an acceptable probability of failure.

Non linear model

To obtain the damage vulnerability functions it is necessary to develop numerous non-linear dynamic time-history analyses. The structural model used to evaluate the inelastic response is a *member-type model*. This type of model allows a close enough representation of the key features of the seismic behaviour of reinforced concrete elements and is able to describe the distribution of inelasticity and damage all over the structure. The non-linear dynamic analyses of the structures are carried out using a modified version, Drain2D/ICIST [Guerreiro and Bento, 1998], of the well-known Drain-2D program [Kannan and Powell, 1975], that uses a new finite element developed to model the reinforced concrete structural elements (beams and columns). This element is a *one-component model*, which consists of a linear-elastic beam element with a non-linear rotational spring at each end. The elastic element models the elastic deformations of the structural element, whereas all the non-linear deformations are concentrated in the two end springs or plastic hinges.

The non-linear hinge behaviour is analysed by means of a fibre model, where the section is composed of a number of element fibres describing separately the concrete and steel behaviour. To model the reinforcing steel a relationship proposed by Giuffrè-Menegoto-Pinto [Giuffrè and Pinto, 1970] and modified by Filippou, Popov and Bertero [1983] is used. No strength degradation with cycling was considered. Low cycle fatigue was accounted for assuming that steel failure occurs when a single plastic deformation exceeds 20% or when accumulated plastic deformations exceed 90%. The stress-strain curves adopted to model the confined and the unconfined concrete are the ones proposed by Scott, Park and Priestley [1982]. A modified version of the model proposed by Thompson and Park [1980] is adopted, assuming that the unloading and loading paths are coincident, even for concrete strain values higher than the concrete strain at maximum stress. The developed model accounts for important characteristics of the behaviour of reinforced concrete elements both under static and cyclic loading. Under static loading it simulates the cracked stiffness of the element and under cyclic loading pinching, strength and stiffness degradation and the effect of axial load variation.

Procedure for q factors assessment

To evaluate the adequate q factor for a specific structure, this one is designed according to the adopted design rules assuming a chosen q factor. The non-linear response of the designed structure is analysed and the probability of failure is evaluated based on the probabilistic definition of the seismic action and of the structural capacity. If the probability of failure is within admissible values, the chosen q factor can be assumed as adequate for the seismic design of structures similar to the analysed one, if subjected to a comparable seismic action and designed according to the same design rules.

If the probability of failure is larger than acceptable, a smaller q factor must be chosen and the process repeated. If, on the contrary, the computed probability of failure is significantly smaller than a reference value, a larger q factor may be chosen and the process repeated likewise until the chosen q factor leads to a failure probability which is comparable to the one previously assumed as acceptable.

CASE STUDIES

The assessment of "behaviour factors" q, for two different irregular structures was made. These structures, displayed in Fig. 2, represent typical types of configuration irregularity and correspond to two different soft-stories, with the consideration of the effects of infill masonry walls.

The selected structures have a fundamental natural frequency about 1.6 Hz and were designed according to Eurocodes 2 [CEN, 1991] and 8. A design procedure corresponding to a medium ductility class according to Eurocode 8 classification was considered. The adopted capacity design procedure was not exactly the one prescribed in Eurocode 8 but a simplified one, dropping the relaxation criterion (δ factor). The structures were designed assuming three different choices of q factor values, respectively q=2, 3 and 4.

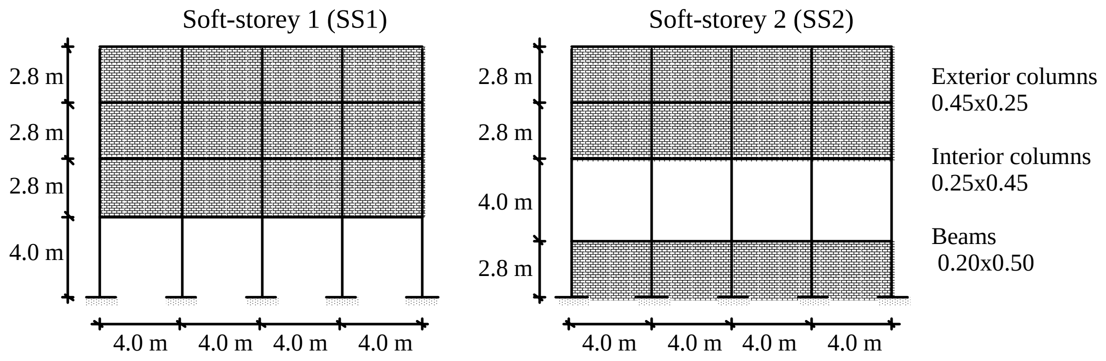

Fig. 2 - Structural configuration and element' s dimensions.

The structures were designed according to Eurocode 8 for a soil type C (soft soil) response spectrum, for a given design q factor and assuming a medium ductility class. A linear dynamic analysis of the structure was carried out, and the design action effects were calculated adding the gravity and the seismic effects divided by the assumed q factor. The cracking of the reinforced concrete members was considered by an appropriate reduction of the member rigidities. An increase in the beam elements inertia was adopted to account for the slab contribution. To decrease the probability of plastic hinge formation in the columns, the acting bending moments were corrected as indicated in Eurocode 8, and for each considered q factor value, the reinforcement was evaluated observing the design rules presented on Eurocodes 2 and 8, namely those regarding minimum reinforcement.

In the design procedure dead and live loads were considered and the seismic action was represented by the acceleration response spectrum. The gravity and the seismic loads were evaluated and combined according to Eurocode 1 [CEN, 1994a].

The second order effects (P-δ effects) were accounted for in the columns by means of a linearised geometric stiffness matrix of the element. An approximation was made considering, in each column, a constant value of the axial force due to gravity loads.

Figures 3 and 4 display the vulnerability functions corresponding to the first (SS1) and the second (SS2) structures, for the three assumed q factors. The vulnerability functions represent the value of the maximum obtained Miner damage index as a function of a normalised peak ground acceleration level (A_F). The amplification factor A_F, represents the relationship between the peak ground acceleration level and the one adopted for design.

As can be seen in both figures, for all amplification factor values, the higher the adopted q factor the higher is the corresponding damage index value.

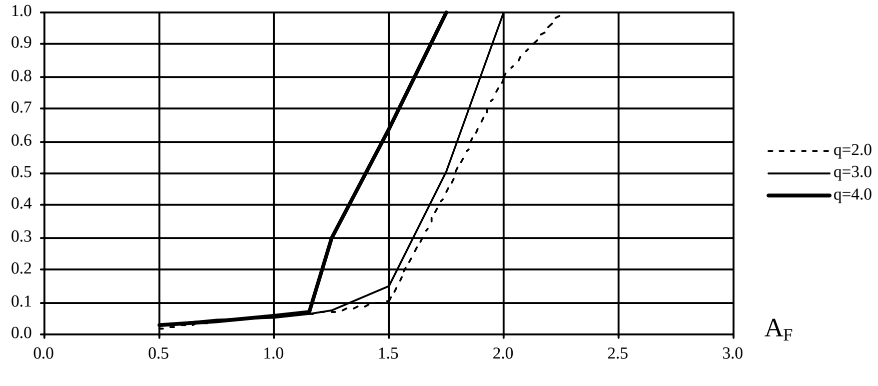

Fig. 3 - Vulnerability functions for soft storey structure SS1, q=2, 3 and 4.

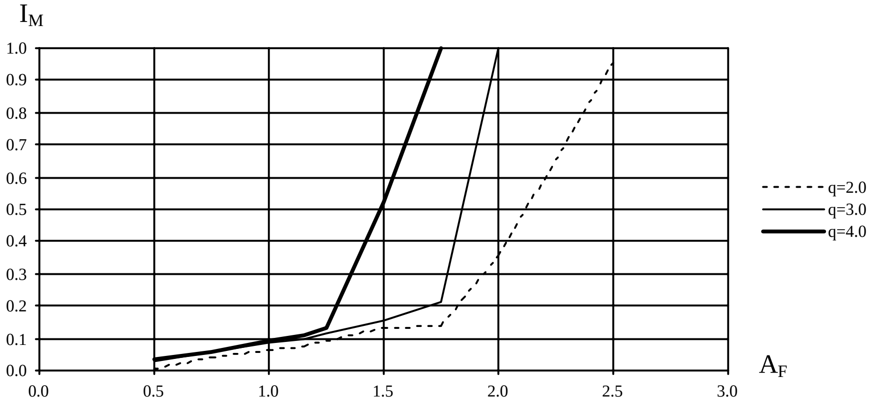

Fig. 4 - Vulnerability functions for soft storey structure SS2, q=2, 3 and 4.

In both cases, for q equal to 4, there is a rapid increase in the vulnerability function values for an amplification factor approximately equal to 1.5. This means that the structures, if designed using this q factor value, are not able to resist seismic actions larger than about 1.5 times the one assumed in design. For q equal to 3 and 2, this value respectively increases to about 1.8 and 2.0.

Also comparing both figures it can be seen that the SS1 structure is more vulnerable than the SS2 one because, irrespectively of the assumed q factor, the damage index values are always larger in the SS1 case.

The pattern of maximum horizontal displacements, δ_H, at each storey level can be seen in figures 5 and 6, respectively for structure SS1 and SS2.

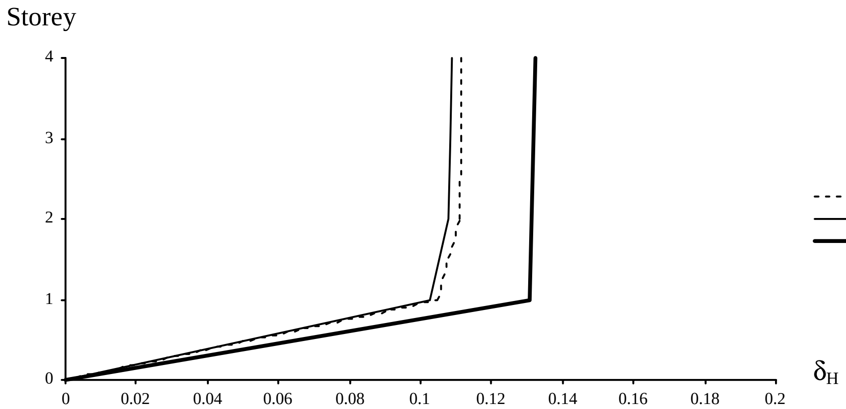

Fig. 5 - Maximum storey displacements for soft storey structure SS1, q=2, 3 and 4.

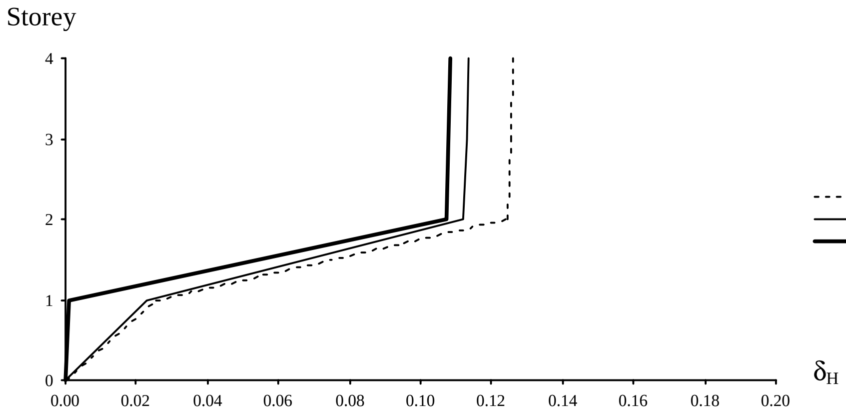

Fig. 6 - Maximum storey displacements for soft storey structure SS2, q=2, 3 and 4.

In both cases it can be seen that, irrespectively of the adopted q factor, the inter-storey displacements are totally concentrated at the soft storey level with values about 2.5 to 3% of the storey height, exceeding what is commonly accepted as collapse limit inter-storey drift values (≈2%) [Penelis & Kappos, 1997].

The obtained values would eventually be larger if a non simplified consideration of the second-order (P-δ) effects had been made.

An also worth noting observation is that the maximum displacements are not sensitive to the adopted q factor. This is consistent with the previous observations regarding the evaluation of maximum non-linear displacements and which have led to the adoption of q factors equal to 1 for displacement assessment.

It can thus be concluded that, independently of the adopted q factor, the structural safety is not guaranteed due to excessive interstorey drift.

Regarding the damage distribution along the structure, it could be shown that, similarly to what was observed for the interstorey drifts, damage is totally concentrated in the columns at the soft storey level, showing that non-linear behaviour and energy dissipation takes place only at that level.

A final observation can be made regarding the probabilities of failure for the two analysed structures designed for the three q factor values. Table 1 displays these probabilities of failure. From that table it can be seen that the probabilities of failure increase for higher q factor values and are always larger for structure SS1.

Table 1. Probabilities of failure based on the Miner damage index.

	q=2	q=3	q=4
SS1	$5.11x10^{-5}$	$5.35x10^{-5}$	$1.30x10^{-4}$
SS2	$2.71x10^{-5}$	$2.90x10^{-5}$	$1.08x10^{-4}$

As an example, let us assume that an admissible probability of failure between 10^{-5} and 10^{-4} is adopted. Say $5.0x10^{-5}$. This would mean that for structure SS1 one should adopt a q factor value lower than 2 and that for structure SS2 one should adopt a q factor value about 3.

Other studies [Bento and Azevedo, 1998; Bento, 1999] have shown that the probabilities of failure obtained for regular structures are consistently lower than the ones obtained for soft storey structures. This finding, for itself and not considering what was observed in terms of interstorey drifts, would suggest that lower q factors should be adopted for soft storey structures as compared to regular ones.

The fact that the inter-storey drifts are considerably higher and concentrated in a single storey in the case of soft storey structures further highlights the need for lower q factors for soft storey structures. This is true even if one thinks that, adequately considering the P-δ effects, one is indirectly accounting for this effect in the evaluation of the damage indices and thus in the probabilities of failure and admissible q factors.

CONCLUSIONS

The proposed methodology for safety assessment and q factor evaluation has shown to lead to adequate results for the analysed irregular structures.

Based on the results obtained for the analysed soft storey structures it can be confirmed that this kind of structures exhibit a less safe behaviour than do similar regular structures. This is due to the concentration of damage at the soft storey level and to the corresponding excessive interstorey drift.

The importance of a correct consideration of the P-δ effects is clear, due to the fact that this type of structures tend to have very large inter-storey drifts concentrated at a single storey.

Also relevant is the need for a probabilistic analysis to assess the structural safety of this kind of structures. This is due to the fact that the differences between the responses of soft storey and regular structures tend to increase with the increasing of the seismic loading levels, as could be seen comparing the corresponding vulnerability functions.

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