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Comparative analysis of R/C building structures (frame and shear wall) designed according to EC8

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ABSTRACT: In this paper, the seismic performance of two structures of different overall typologies, namely frame and shear wall are compared. To this end, two types of earthquake ground motions and various soil conditions are considered in the definition of seismic inputs. The consistency of the Eurocode No. 8 requirements concerning the local ductilities when compared with the real demands is also analysed. In a first phase, single degree of freedom analyses were carried out with the earthquake loading prescribed in EC8. The effect of initial natural frequency and yielding level is highlighted. Having identified the main aspects of the non-linear response, a more refined modelling of a frame and a shear wall was performed in order to verify the validity of the results obtained from the single degree of freedom calculations. The results demonstrated the superior structural performance and the cost benefit of the ductile design for earthquake resistance. Moreover, a good agreement between the results of the single degree of freedom and the refined calculation results was found.

1 INTRODUCTION

Eurocode No.8 (EC8) has reached an advanced stage in its development. A series of design exercises has shown the consistency of the EC8 approach to seismic design as well as the adequacy of the code as a practical design tool. However work is still required in order to calibrate both the parameters defining the seismic input and the level of local ductility required (CEC, 1989; CEC, 1991). Also a need for practical guidelines able to assist the designer toward the most appropriate choice of typology has been identified. In fact, in the code, nothing is suggested concerning the adequacy of each particular structural typology for different characteristics (soil conditions and local seismicity).

The main aim of this paper is to compare the seismic performance of two different structural types, namely frame and shear wall structures with respect to two types of seismic input and different soil conditions. Another aim is to verify the consistency of the EC8 requirements concerning the local ductilities when compared with the actual demands.

In a first phase, a single degree of freedom (SDOF) system was adopted to represent the global structural behaviour. To represent the force/displacement relationships a Takeda-type model including degradation of both stiffness and strength properties was compared with an elasto-plastic model with hardening.

The seismic input was represented by a set of artificial accelerograms fitting the EC8 response spectra and having non-stationary characteristics identified from two real earthquakes. The real earthquakes were chosen to represent the ground motion at near field and far field sites.

As a second step, two R/C structures, namely a frame building and a hybrid (frame/shear wall) building, designed according to EC8 were analysed. A modelling using beam elements with end concentrated inelastic deformations was adopted to represent the structures. To take into account the distributed plasticity at the base of the shear walls, a more refined mesh was adopted here . The seismic input was the one used in the simplified analysis.

2 SIMULATION OF THE GROUND MOTION

As input for the nonlinear analysis a set of accelerograms with non-stationary characteristics both in frequency and time was generated. A previously developed procedure (Pinto and Pegon, 1991) was implemented in the object-oriented type computer code Castem-2000 (CEA, 1990) and was used for the generation of the artificial accelerograms. Following this procedure, it is possible to generate accelerograms from an evolutionary power spectrum obtained from the EC8 response spectrum and a set of modulating functions. These functions can be derived from real earthquake records or from a seismological input model which takes into account the

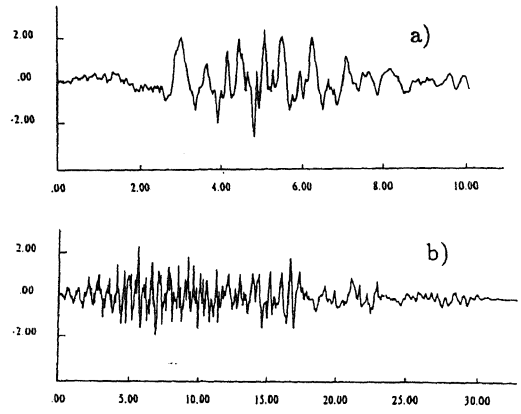

Figure 1: Samples from generated accelerograms: a) Friuli-like (FR), soil B b) Orion-like (OR), soil B.

attenuation and amplification of the soil as well as the source mechanism (Seidel and Reinhorn, 1989). Analytical functions can be also considered for this purpose. In the present case, the Friuli, Italy, 1976 and the Orion, USA, 1971 earthquakes were considered as "master earthquakes" for the generation of EC8 compatible artificial accelerograms representing short and long focal distance earthquakes respectively.

Figure 1 shows examples of the generated accelerograms.

3 SDOF STUDIES

Single degree of freedom oscillators were adopted to represent both the global shear and flexural behaviours. A Takeda-type (TK) model as well as an elasto-plastic model (EP) with hardening (10% of the initial stiffness) were used to represent the force/displacement relationships.

Results from calculations with the two models are presented in Figure 2 showing a small difference between the responses. The Takeda model gives larger responses essentially for high strength reduction factors (behaviour factor (q) in EC8). However, it appears that the model adopted for the global constitutive relations has only a moderate influence on the calculated responses (short focal distance earthquakes). An aim of these SDOF studies is to extend the results to real buildings in which global and storey behaviours can be represented by either of the constitutive models. In practice, the behaviour would be between the prediction of those models.

The curves shown in Figure 2 are the mean values (ten samples used) of the ductility demand of SDOF systems for different values of q when subjected to Friuli-like excitations fitting the EC8 response spectrum corresponding to a soil type B.

In general, it appears that the ductility demand increases with the initial natural frequency of the structure especially for high values of behaviour factor. However, for structures with natural frequencies less than 2-3 Hz the value of the ductility demand is of the same order as the q factor. This is not inconsistent with the results obtained from ideal elasto-plastic models (Newmark and Hall, 1982).

The curves of Figure 2 confirm the EC8 design philosophy. In practice, for structures with natural periods less than a certain value (T_1 in EC8 nomenclature) a smaller behaviour factor (q_{eff}) is used. However the actual EC8 suggestions for T_1, 0.2 seconds for soils A and B and 0.3 seconds for soil C (frequencies 5 and 3.33 Hz), should be increased in order to limit the ductility demands of stiff structures to acceptacble values. An explicit indication of the frequency dependent behaviour factor ($q(\omega)$), in high frequency range, could be done.

The philosophy for the design of structures to resist strong earthquakes is that of accommodating large inelastic deformations without collapse. Such philosophy is adopted because, in principle, it implies a more economical construction (reduction of design earthquake loadings); however, a certain safety level must be guaranteed. Thus, appropriate ductility in the structure must be provided in order to support the real demands without collapse.

Actually, EC8 considers three levels of ductility, namely: low (L), medium (M) and high (H). In addition, a set of corresponding behaviour factors is suggested for different structural types. The present studies intend to assess the adequacy of such behaviour factors for structures designed for different soil conditions and subjected to different earthquake excitations. Moreover, it is intended to identify the design options leading to the most economical construction whilst guaranteeing the appropriate safety levels. This guarantee of safety requires an upper bound for the ductility demand. Thus, in the current calculations, it is assumed that the structures have a constant ductility factor (μ) which is directly related to the ductility class adopted for the structure.

3.1 Allowable behaviour factors (q)

Following the lines suggested above, the information contained in Figure 2 was reploted in terms of constant ductility factors (μ) thus giving direct guidelines on the maximum behaviour factors that can be adopted in design calculations. Figure 3 shows these curves were obtained using a linear interpolation algorithm in the inversion of the functions plotted in Figure 2.

In the same figure, curves are presented for three different soil conditions and the two earthquake types identified as representative of near-field and

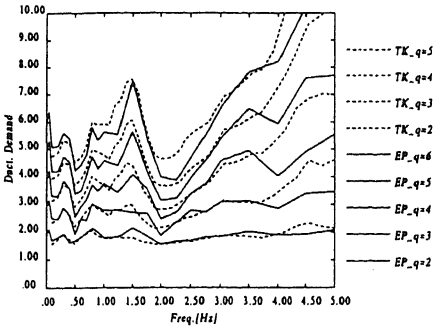

Figure 2: Maximum ductility demands of SDOF systems with constitutive relations given by an elasto-plastic model and a Takeda-type model (q = 2,3,4,5 and 6), Friuli-like excitations.

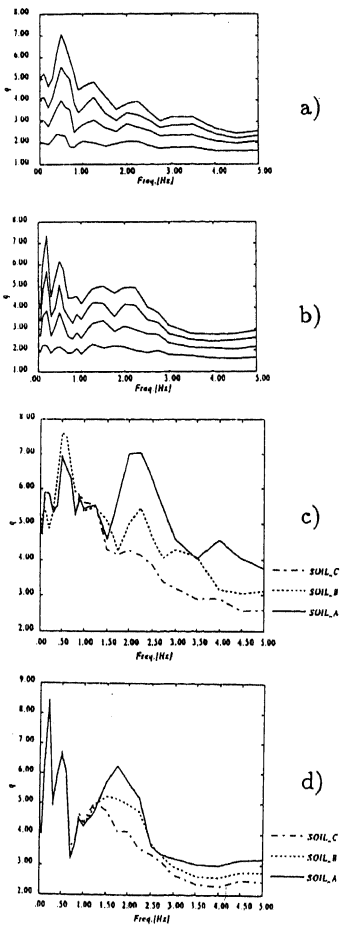

Figure 3: Allowable behaviour factors for constant ductility factors (μ = 2,3,4 and 5): a) Friuli-like, Soil B, b) Orion-like, soil B, c) Friuli-like μ=5 Soils A,B and C d) Orion-like, μ=5, Soils: A,B and C.

far-field ground motions.

The results can be summarized as follows:

- The design of structures for high ductile capacity having high natural frequencies should be performed using small values of behaviour factor. For example, from Figure 3 a) it is possible to see that the design of a structure with a ductility factor (μ = 5) and natural frequency 5 Hz to be constructed in a medium stiff soil (Soil type B) in a near source zone (Friuli-like earthquakes) should be performed using a behaviour factor of about (q = 3) which is in close agreement with the theoretical results (Newmark and Hall, 1982) obtained from the elastic perfectly plastic model. In such a frequency region the energy conservation between elastic and elasto-plastic models holds, given the well known relation ($q = (2\mu - 1)^{(1/2)}$). This relation which is normally assumed for frequencies greater than 2 Hz gives, in the present case, conservative results.
- The equality between q and μ for values less then 2 Hz can generally be applied giving also conservative results.
- In the transition zone (2 to 3 Hz) a smooth transition curve should be assumed.
- In general, it is true that for the same ductile capacity the design of structures in near source zones and on stiff soils can be performed using larger q factors. On the other hand, for structures to be constructed in soft soils (type C) smaller q factors should be used in order to limit the maximum ductility demands to acceptable values.

3.2 The "economy factor" (EF)

As noted earlier a ductile design of earthquake resistant structures, in general leads to a more economical construction. However cases exist where this can not be attained without a reduction of safety levels. Identification of such cases is important and design guidelines leading to the more economical construction should be made available. In fact, the behaviour factor is a measure of the construction economy of earthquake resisting ductile structures. For example, in R/C structures it determines the allowable reduction of the amount of reinforcing steel in critical regions. In addition, it allows design with smaller sections which is also an advantage. On the contrary, it does not include repair costs and the increase of design and construction expenses associated with the more complicated procedures and the necessary construction quality control.

Figure 3 shows the allowable q factors corresponding to different levels of ductility. From these curves it can be seen that, in general it is most economical to build highly ductile structures with rather low natural frequencies.

The q factor, as currently defined can be used to make comparisons between structures on the same soil type, but comparisons between different soil conditions are not possible.

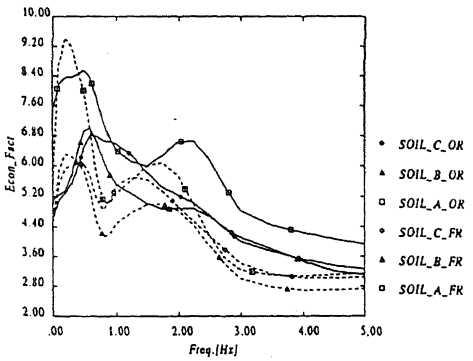

Figure 4: Relative "economy factor" (smoothed curves); Friuli-like and Orion-like excitations.

In order to make possible such comparisons, an "economy factor" (EF) is proposed. This is defined as the product of the q factor for a certain ductility level (μ) by a coefficent which depend on the elastic design response spectrum. Referring to EC8, the correction to be introduced is the ratio between spectra of the different soil types. As the highest values are for soil B, the EF should be computed by

$$EF(\mu) = q(\mu).(RS_B/RS_i)$$

where RS_B is the value of the EC8 elastic response spectrum corresponding to the soil type B and RS_i is the corresponding value for the other soil types (i = A, C).

Figure 4 shows the curves obtained from such correction and the following comments can be made:

- Structures should be designed to have low natural frequencies (less than 2-3 Hz), in order to best exploit the advantages of a ductile design.
- The "economy factor" for such a structure increases if it is to be constructed on stiff soils.
- However, stiff structures designed for high ductility are only economical if to be built on stiff soil in near field zones.

4 ANALYSIS OF TWO R/C STRUCTURES

The results from the SDOF calculations can be extended to multi degree of freedom systems and consequently to real structures if the appropriate analogies are used. In general, the ductility factor (μ) obtained from simplified modelling can be identified as the overall ductility of the structure if it is designed to exhibit a uniform distribution of non-linear deformations. In particular, for buildings, ductility can be defined as the storey ductility factor. However, real structures respond to earthquake excitations in a very complicated manner. From the point of view of dynamics it is important, for example, to include the higher modes in the response (essentially for base shear calculations). In addition, designing for high ductility levels implies a series of design procedures and detailing leading to a uniform distribution of inelastic deformations in the structure (dissipation mechanism). This is only possible if detailed modelling is utilized.

Therefore, two building structures designed according to the Eurocodes No. 2 and No. 8 were analysed. The first structure is a R/C framed building (FA) with 8 storeys. The second structure is a hybrid (frame/shear wall) building (HB) whose plan view is presented in Figure 5. Analysis of this structure was performed for excitation in direction y, i.e. in the plane which contains all the types of structural elements of interest.

The two structures under analysis have natural frequencies of (f_{FA} = 1.4 Hz) and (f_{HB} = 1.9 Hz) respectively. Thus, having in mind the SDOF results, it should be economical to design such structures for high ductile capacity. Indeed, the structures had been designed as belonging to the ductility class H and q values of 5 and 4.5 had been assumed respectively for the FA and the HB structures. Steel S400 and concrete C25/30 were assumed for the structural materials.

4.1 Modelling of the structures

The frame structure was idealized as a planar association of beam elements.

Because of symmetry only one half of the hybrid structure was considered and it was taken as a planar association of the three bays, each containing floors assumed to be indeformable in their own plane, and connected by hinged rigid links.

The following assumptions were made in the modelling of the structural elements:

- beams and columns were each represented by beam elements with plastic hinges at their ends governed by Takeda-type relationships and the beam/column joints were rigid;
- walls were represented by the same type of beam element but adopting a discretization between floors in order to simulate the distributed plasticity which actually exists. This was intended to minimize the limitations of the concentrated plasticity model sometimes used to describe the behaviour of walls with both flexural and shear nonlinearities (Emori *et al*, 1981). The walls of the building under analysis can be regarded as slender (ratio between height and length greater than four); consequently only the flexural nonlinearities need to be taken into account, although both shear and bending elastic deformations were considered. Fictitious rigid links between the centre of the walls and the adjacent beams were included to maintain the true geometrical relationship.
- coupling beams normally play a very important role in the nonlinear behaviour of structures and a large ductility and energy dissipation capacity is required of them. Because such beams have a low slenderness ratio, a flexural mode of failure is not guar-

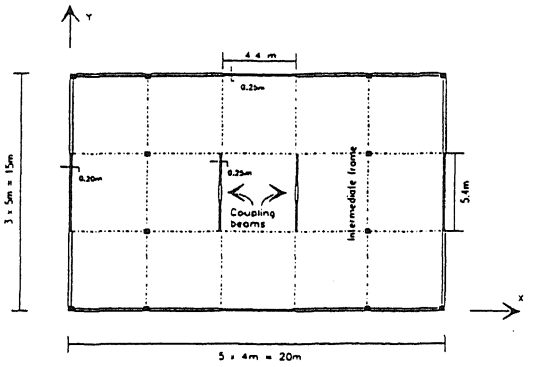

Figure 5: Plan view of the hybrid (frame/shear-wall) structure.

anteed and a shear or diagonal splitting or a mixed failure mode is also possible (Subedi, 1991). In the absence of an appropriate model to consider the coupling of the flexural and shear nonlinearities the flexural mode was assumed. This assumption seems to be appropriate because, in the design, the coupling beams had been specified as having a 'strong' diagonal reinforcement.

The hysteretic behaviour in the plastic zone of the R/C elements is described by a Takeda-type model implemented in the computer code ANSR-I (Mondkar and Powell, 1975). The control parameters of the model moment/curvature relations for several values of superimposed axial force were obtained using a "fibre-type" model of the section. This incorporated a steel stress/strain curve which shows, after yielding, a plateau followed by a hardening region. The concrete curve is the model suggested in EC8 which accounts for the amount and type of confinement. The analysis as usual assumed that plane sections remain plane during deformation.

Detailed analysis of the HB structure can be found in (Pinto and Jones, 1991).

4.2 Basic calculations and ductility demands

Both the frame and the hybrid structures were subjected to the two sets of artificial accelerograms representing the near-field and far-field excitations and fitting the design response spectrum (Soil B, peak ground acceleration of $.25g$). The earthquake ground motions were among the ones utilized in the SDOF calculations. In addition, the structures designed to be constructed in a soil B zone were subjected to excitations corresponding to the other soil conditions maintaining the remaining design variables.

Figure 6 shows the distributions of the curvature ductility demands and Table I presents the results obtained from several calculations. The following comments can be made:

- The results are, in general agreement with the predictions from SDOF studies.

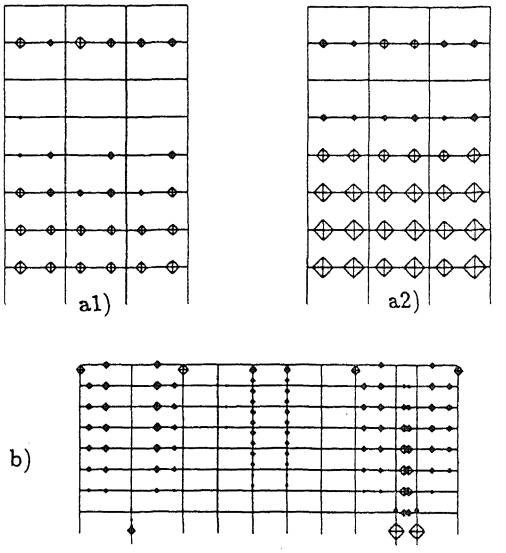

Figure 6: Maximum curvature ductility demands (CDD) in the inelastic zones (diagonal of lozenges are proportional to CDD values): a) Frame structure; soil A (a1)) and soil B (a2)) b) Hybrid structure soil B.

- The values of ductility demands in both structures are well within the capacity of R/C members designed for high ductility levels. Moreover, the distribution of ductility can be considered rather uniform. In this respect, it is to be noted that the more uniform distribution is obtained in the HB structure; the walls have an important role in this.
- The comparison between the demands of the frame structure when analysed on different soil conditions confirms that structures, on stiffer soils are generally more economical.
- Still referring to the frame structure, a decrease of demands in the beams from the bottom to the top of the structure is noted which is consistent with other studies on low frequency frame structures. The desired dissipation mechanism (strong columns weak beams) was evidenced for all excitations used. The damage found on B and C soil sites is greater than the damage found on soil type A; however, the demands in the upper storeys are similar, in magnitude and distribution because they are essentialy due to the contribution of higher mode vibrations.
- The hybrid structure behaves as expected: nonlinear deformations (damage) in the walls (uncoupled and coupled) are restricted to the first two storeys, the critical zones considered by the design code.
- The same structure was subjected to both Friuli-like and Orion-like ground motions. The results for far-field excitations lead to larger demands in the structure for the same peak ground acceleration.

Table 1: Maximum curvature ductility demands (CDD)

	Friuli-like excitations		
	Soil A	Soil B	Soil C
FR Structure	5.5	9.7	10.6
HB Structure	7.6	7.9	9.0
	Orion-like excitations		
	Soil A	Soil B	Soil C
HB Structure	8.1	8.5	9.9

SUMMARY AND CONCLUSIONS

The response of two different types of R/C building structure to various seismic loadings have been analysed to compare their performance with each other and with the EC8 design philosophy. The structures chosen were a frame and a hybrid (frame/shear wall) already designed for a previous EC8 design exercise. The seismic inputs were derived from the EC8 spectra with characteristics obtained from real earthquake records. Simple nonlinear SDOF simulations were first performed to establish the general characteristics of the response as functions of the natural frequencies of the structure. Detailled calculations of the two specific structures were then performed for comparison.

The results of the SDOF calculations indicated that:

- structures designed for high ductility levels are more economical if they have low natural frequencies (less then 2-3 Hz)
- soil conditions and the local seismicity play an important role,
- definition of a relative "economy factor" facilitates comparisions between the costs of ductile structures to be built on different soil conditions and in zones near and far from the earthquake sources.

The analysis of two multi-storey R/C buildings confirmed the results obtained from the SDOF calculations. In fact, both structures have rather low natural frequencies and were designed for high ductility. The ductility demands were all found to be within the ductile capacity assumed, particularly for the hybrid structure.

The efficacy of the capacity design procedure required in EC8 for frame structures was demonstrated.

For the hybrid structure a uniform distribution of damage along over the height of the structure in the non-wall elements was observed and the damage in the walls was found to be concentrated in the first storeys as assumed in EC8.

The fact that both analysed structures had rather low natural frequencies inhibited further exploration of the SDOF predictions of ductility demand for stiff structures (low rise buidings).

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