SECOND EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING AND SEISMOLOGY, ISTANBUL AUG. 25-29, 2014

FULL AND PARTIAL SEISMIC PROTECTION: INFLUENCE OF DESIGN OBJECTIVE ON CONSTRUCTION COST OF BUILDINGS

Grigorios MANOUKAS[1] and Asimina ATHANATOPOULOU[2]

ABSTRACT

Performance Based Design is a seismic design philosophy which has been established in the last twenty years and has already been adopted by almost all modern seismic codes. It consists of a set of provisions, rules, design criteria and methods which aim at a predefined performance of the structure for a specific earthquake hazard level. In accordance with Performance Based Design, the concept of full and partial seismic protection of buildings has been proposed. According to this concept, the building owner, in cooperation with the designer, may decide to define higher seismic performance requirements than the minimum ones specified by codes. It is apparent that this decision influences the construction cost. The scope of the present paper is to quantify this influence. For this purpose a parametric study is conducted comprising the design of multi-storey reinforced concrete buildings for alternative design objectives. The parameters taken into account are the structural type and the seismic hazard level zone. It is shown that the additional cost resulting by adopting a higher design objective is not prohibitive.

INTRODUCTION

As it has been observed in the past after strong earthquakes, the structures are able to overcome seismic loading that exceeds their strength expressed in terms of forces, thanks to their available ductility. This observation has been also confirmed by experimental results and led to the development of a design philosophy which is now adopted by modern seismic codes. According to this philosophy, in case of strong earthquakes the utilization of the available ductility of the structures is desirable in order to absorb energy and withstand the seismic excitation without collapse. Thus, the design seismic demands are reduced and the structure is expected to undergo extensive inelastic deformations under the design earthquake. At the same time a reduction of the construction cost is achieved.

In current practice, although nonlinear response is expected, the structures are usually analyzed by means of a linear analysis procedure. The seismic action, represented by the design spectrum, is divided by the behaviour factor q which depends on the structural system configuration. Based on the calculated member forces, the structural components are detailed in order to dispose a minimum level of strength as well as an available local ductility which ensures a significant energy dissipation capacity of the structure. Ultimate strength and/or serviceability limit states are checked through a number of compliance criteria. If the compliance criteria are satisfied it is considered that the structure meets some fundamental safety requirements. For example, according to Eurocode 8, the structure is able to withstand the specified design seismic action without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events. Also, for a seismic

[1] Dr, Aristotle University, Thessaloniki, Greece, grman7@otenet.gr
[2] Professor, Aristotle University, Thessaloniki, Greece, minak@civil.auth.gr

action having a larger probability of exceedance significant damage and the associated limitations of use are avoided.

It is obvious that this design philosophy implies the occurrence of extensive damage of both structural and nonstructural components in case of a strong earthquake similar to the design earthquake. As a consequence high economic losses are expected, while the structure may remain out of service for much time. Slight damage may occur even for lower intensity earthquakes. Furthermore, in case of a seismic excitation stronger than the design earthquake, non-repairable damage and even the global collapse of the structure cannot be precluded and the fundamental objective for protection of human life is in question. Obviously, this is not consistent to the requirements of the society, which demands from the state and the engineers absolutely safe against earthquakes structures.

On the other hand, the seismic codes in general specify just the minimum requirements for the seimic design. The structure owner, after being informed by the designer, may define highest safety requirements. Hence, the structure can be designed in order to achieve a predefined performance level (higher than the minimum one) for a specific earthquake hazard level (Performance Based Design). Unfortunately, the vast majority of the building owners and occupants are not aware of the current design philosophy as well as of the possible implications of a strong ground motion on their properties. Thus, they are not involved in the definition of the design objective and the engineers usually adopt the minimum performance level (the highest value of q) prescribed in codes.

In accordance with Performance Based Design, the concept of full and partial seismic protection of buildings has been proposed (Anastassiadis et al., 2000). According to this concept three discrete performance levels of the structural components are defined (Table 1). Each level corresponds to a specific value of the behaviour factor q to be applied in linear analysis methods. In particular, to the highest performance level corresponds the value q = 1 (i.e. the structure is expected to remain nearly elastic for the design earthquake), while to the lowest corresponds the value $q = q_{max}$, where q_{max} is the maximum value of q permitted by seismic codes. Analogous performance levels are defined for the nonstructural components too. From the combination of a structural and a nonstructural performance level results the building performance level.

Table 1. Structural Performance Levels

Performance Level	Damage Level	Behaviour Factor q
Full Protection (FP)	Very limited damages	1
Partial Protection 1 (PP1)	Controlled and repairable damages	$\frac{1}{2}\, q_{max}$
Partial Protection 2 (PP2)	Extensive damages	q_{max}

The seismic design objective consists of a target building performance level and an earthquake hazard level. As it has been mentioned above, the seismic codes specify just the minimum requirements concerning the selection of design objective. The building owner, in cooperation with the designer, may decide to choose a more enhanced one. There is no doubt that one of the most important criteria which is taken into account for the selection of the design objective is the construction cost. Obviously, the achievement of a higher performance level requires higher cost, but how much? How important is the influence of the design objective selection on the construction cost? The saving resulting from a selection of a lower objective is enough to justify the reduction of the structural safety against earthquakes? Only few investigations concerning this issue can be found in literature, e.g. (Avramidis et al. 2003), (Lagaros et al. 2006).

The scope of this paper is to further contribute at the quantification of the aforementioned influence. For this purpose a parametric study is conducted. In particular, a series of multi-storey reinforced concrete buildings with different structural systems and plan configurations are designed according to Eurocodes using the Response Spectrum Method of Analysis. The seismic hazard level adopted corresponds to seismic hazard level zones I and II of Greek territory. For each building all the three values of behaviour factor q shown in Table 1 are applied, i.e. three alternative design objectives are examined. For each objective the total construction cost of the load bearing system is calculated. For this calculation real cost data collected from buildings constructed in Greece during the last years is used. It is demonstrated that the increase of the total construction cost resulting from the selection of design objective FP instead of PP1 or PP2 is not prohibitive. Finally, the seismic response of the examined buildings is evaluated by conducting pushover analysis.

STRUCTURAL MODELS AND DESIGN

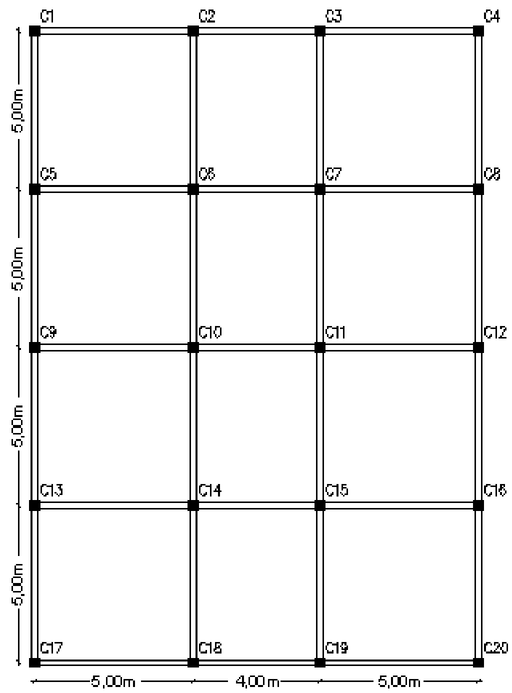
In the framework of the present study, a series of typical multi-storey reinforced concrete buildings are designed according to Eurocodes 2 and 8. In particular, three different structural types according to the classification of Eurocode 8 (Part 1, Section 5.2.2.1) are examined: a 3-storey frame system, a 5-storey ductile wall system and a 5-storey dual wall-equivalent system. All storey heights are 3 m. The floor plans of the analyzed buildings are shown in Fig. 1. All buildings are regular in plan and in elevation and are analyzed applying the Modal Response Spectrum Analysis (Eurocode 8, Part 1, Section 4.3.3.3). For each structural type three alternative values of the behaviour factor, corresponding to the three design objectives given in Table 1, are taken into account. The value of q_{max} is derived from the relevant provisions of Eurocode 8 (Part 1, Section 5.2.2.2). The specific values of the behaviour factor used are tabulated in Table 2. It should be stated that all buildings regardless the q value are designed to meet the Ductility Class High requirements and the capacity design provisions. The seismic hazard level adopted corresponds to seismic hazard level zones I and II of Greek territory possessing Peak Ground Accelerations equal to 0.16g and 0.24g respectively with a probability of exceedance 10% in 50 years. Totally, 18 building models are developed (3 structural types x 3 design objectives x 2 seismic hazard levels = 18 models).

Table 2. Behaviour factor values

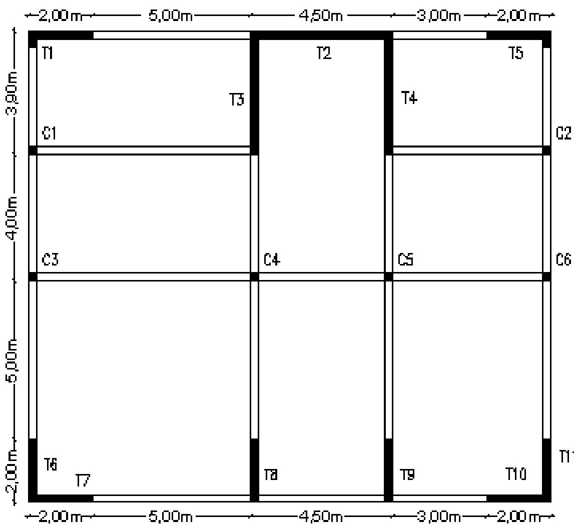
Behaviour Factor q (Performance level)	Frame System	Ductile Wall System	Dual Wall-equivalent System
1 (FP)	1		
$\frac{1}{2}$ q_{max} (PP1)	2.93	2.2	2.7
q_{max} (PP2)	5.85	4.4	5.4

The structural analysis and the detailing of the cross sections are conducted with the aid of appropriate software widely used by engineering practitioners in Greece. The concrete is of class C20/25 (f_{ck} = 20 MPa) and the reinforcement steel bars B500C (f_{yk} = 500 MPa) according to the Greek standards. The slab thickness is equal to 15 cm. In addition to the self weight distributed dead and live loads equal to 1.5 kN/m^2 and 2.0 kN/m^2 respectively are considered. The initially chosen (minimum) height of beams is 50 cm, while their minimum thickness is 20 cm. The dead load of masonry infill is considered equal to 9.0 kN/m for the beams lying on the perimeter and equal to 5.25 kN/m for the others. The columns' cross sections are square shaped with minimum dimension of 30 cm. Finally, the minimum thickness of walls is 25 cm and the minimum length 4.50 m (T2), 3.90 m (T3, T4) or 2.00 m (T1 and T5-T11). When the cross section of a component is not adequate to comply with the code provisions, one or both of its dimensions are successively increased using a 5 cm increment, except the length of the walls where a 10 cm increment is used.

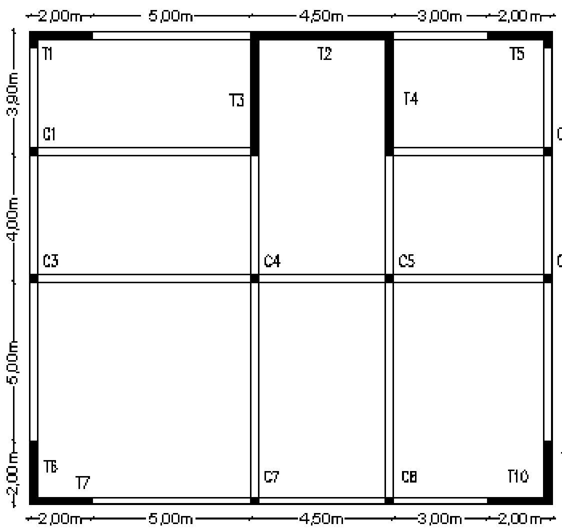
The selected minimum dimensions correspond to minimum dimensions of structural components usually constructed in current practice in Greece. It is obvious that some components, especially beams, could meet the codes' requirements even with smaller cross section dimensions, i.e. the examined buildings, mainly those designed for the lower design objective (PP2), have overstrength. As a consequence the construction cost of these buildings may be overestimated with regard to an ideal building with as small as possible cross section dimensions. However, the main aim of the present study is to compare the construction cost for higher design objectives to the cost of real buildings which almost always are designed for PP2 objective and obviously possess an amount of overstrength. Thus, the adoption of smaller cross sections dimensions would not be representative of the current construction practice and would not permit the derivation of concrete and reliable conclusions.



a) 3-storey frame system



b) 5-storey ductile wall system



c) 5-storey wall-equivalent dual system

Figure 1. Floor plans of the analyzed buildings

ECONOMICAL ANALYSIS

The total concrete volume and the total reinforcement steel mass of each building are shown in Figs. 2 to 7. It is obvious that no remarkable differences between buildings designed for the two levels of partial seismic protection (q_{max} - PP2 and $\frac{1}{2}$ q_{max} - PP1) are observed, i.e. the buildings designed for PP2 design objective have significant overstrength. This is mainly due to:

- the minimum cross section dimensions selected, especially for beams (for the reasons explained in the previous paragraph) and
- the limitation imposed by Eurocode 8 (Part 1, Section 5.5.3) to the normalized axial force of columns and walls, which implies a surplus of flexural strength.

On the contrary, considerable increase of the material quantities results for FP design objective. In particular, the concrete volume of the frame system is increased with regard to PP2 objective by 32% and 61% for seismic hazard level zones I and II respectively, while the reinforcement steel mass by 90% and 146%. Concerning the ductile wall system, the concrete volume is increased by 22% and 29% and the reinforcement steel mass by 55% and 73% for zones I and II respectively. Finally, concerning the dual wall-equivalent system, the concrete volume is increased by 12% and 21% and the reinforcement steel mass by 57% and 87% for zones I and II respectively. It should be noticed that the increase of material quantities is more important for the frame system and for seismic hazard level zone II. In addition, it is obvious that the reinforcement steel mass is influenced much more than the concrete volume.

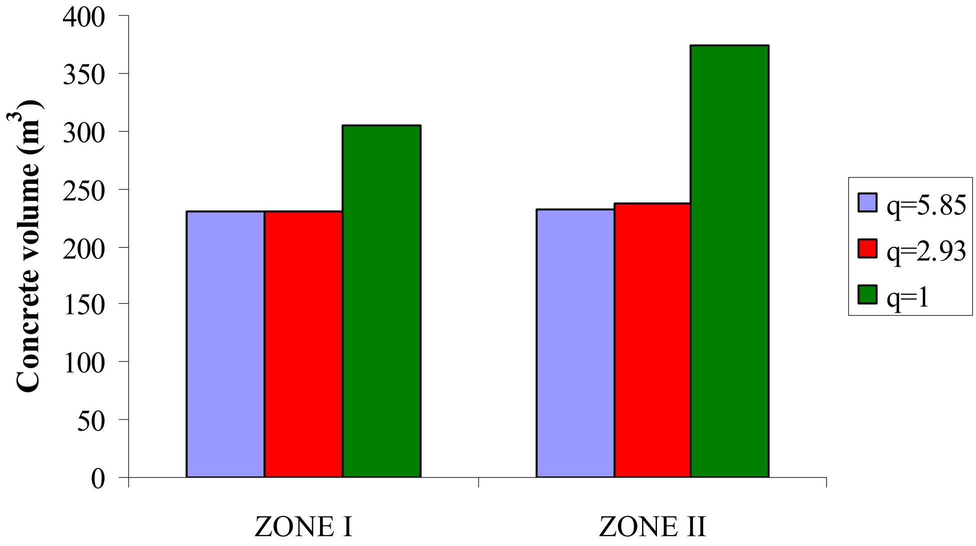


Figure 2. Concrete volume – frame systems

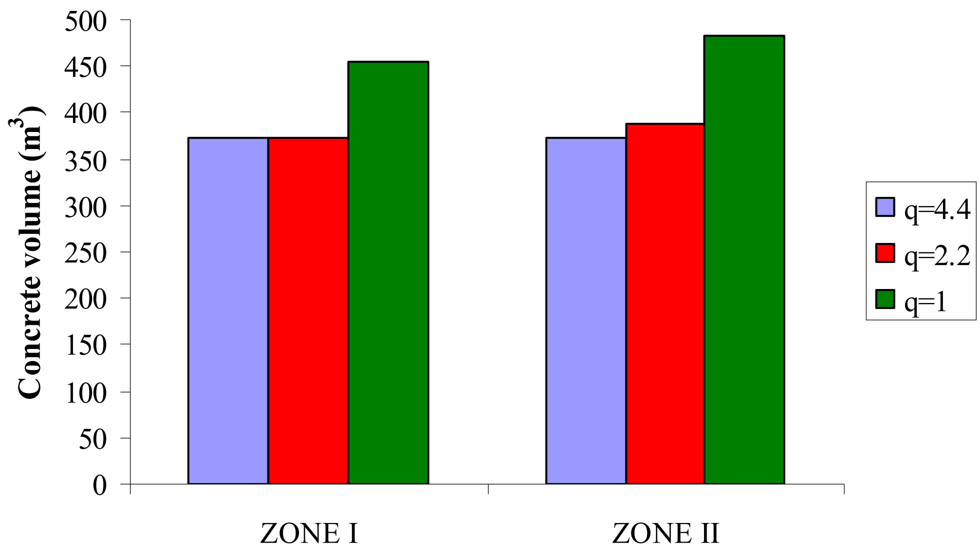


Figure 3. Concrete volume – ductile wall systems

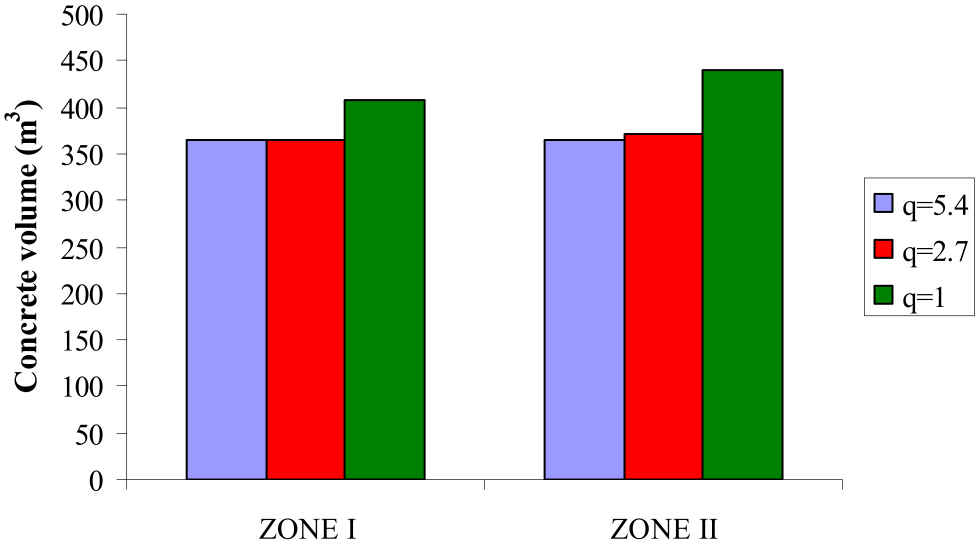


Figure 4. Concrete volume – dual wall-equivalent systems

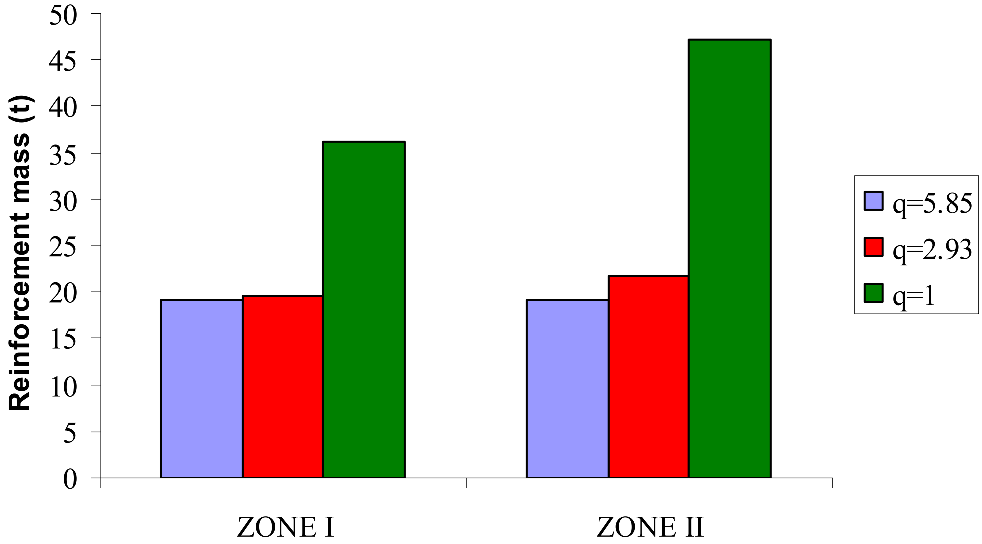


Figure 5. Reinforcement steel mass – frame systems

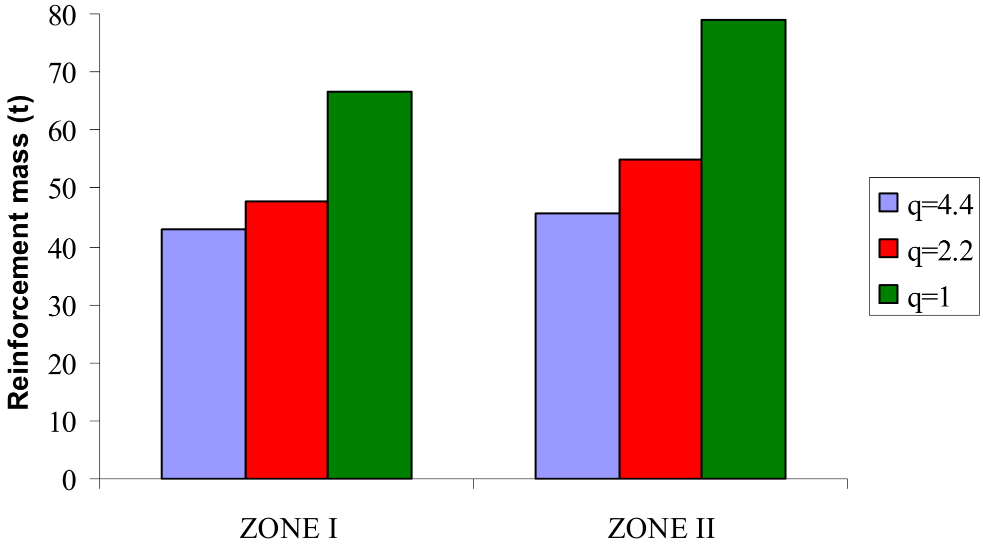


Figure 6. Reinforcement steel mass – ductile wall systems

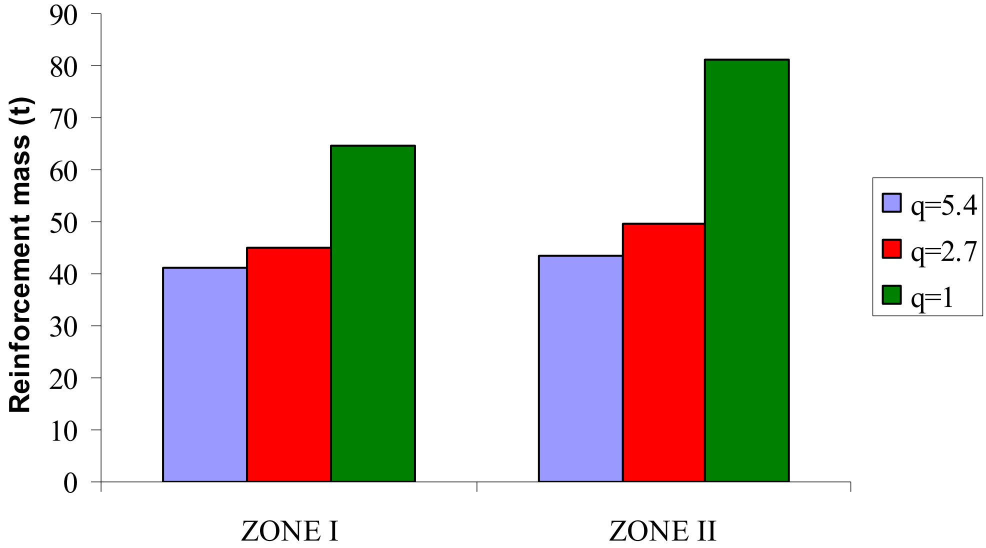


Figure 7. Reinforcement steel mass – dual wall-equivalent systems

The construction cost of the buildings' structural system is calculated using real cost data collected from buildings constructed in Greece during the last years. In particular, it is considered that the cost of concrete is 150.3 €/m^3 and of reinforcement steel 875.25 €/t. These prices include the costs of materials, the remuneration of workers and the social security contributions. In Figs. 8 to 10, the construction cost of the structural system of all buildings is shown, while in Table 3 the same costs normalized to the cost for PP2 design objective are tabulated. The adoption of the PP1 instead of the PP2 design objective causes a small increase in the cost of the structural system ranging between 1% and 5% for the seismic hazard level zone I and between 6% and 11% for the seismic hazard level zone II. The adoption of the design objective FP leads to an increase of 30% to 51% for zone I and of 47% to 89% for zone II. However, these cost augmentations are not excessive with regard to the total construction cost of the buildings (Figs. 11 to 13, Table 4). Indeed, the increase of total costs for the design objective PP1 is negligible (≈1%), while for FP is not prohibitive, ranging between 3% and 4% for zone I and between 5% and 8% for zone II. These estimations are based on the fact that the total construction cost of a R/C building in zone I for the design objective PP2 is about 700 €/m^2.

Table 3. Normalized construction cost of the structural system (%)

Structural Type	Seismic Hazard Level Zone	q_{max} (PP2)	$\frac{1}{2}$ q_{max} (PP1)	1 (FP)
Frame System	I	100.0	100.8	150.7
	II	100.0	106.2	189.1
Ductile Wall System	I	100.0	104.7	135.3
	II	100.0	111.1	147.4
Dual Wall-equivalent System	I	100.0	103.6	129.8
	II	100.0	107.0	147.9

Table 4. Normalized total construction cost (%)

Structural Type	Seismic Hazard Level Zone	q_{max} (PP2)	$\frac{1}{2}$ q_{max} (PP1)	1 (FP)
Frame System	I	100.0	100.1	104.4
	II	100.0	100.5	107.8
Ductile Wall System	I	100.0	100.5	103.8
	II	100.0	101.2	105.3
Dual Wall-equivalent System	I	100.0	100.4	103.1
	II	100.0	100.8	105.2

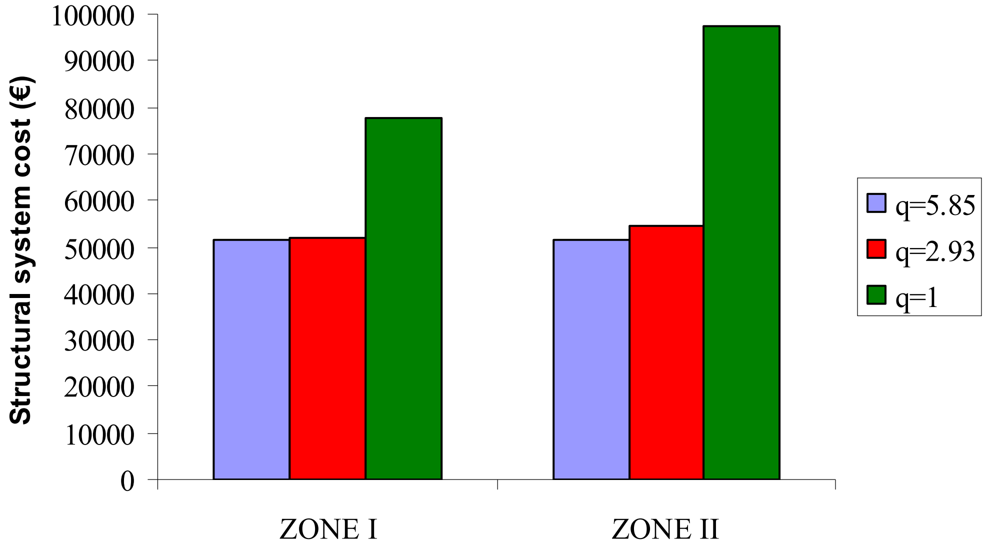


Figure 8. Construction cost of the structural system – frame systems

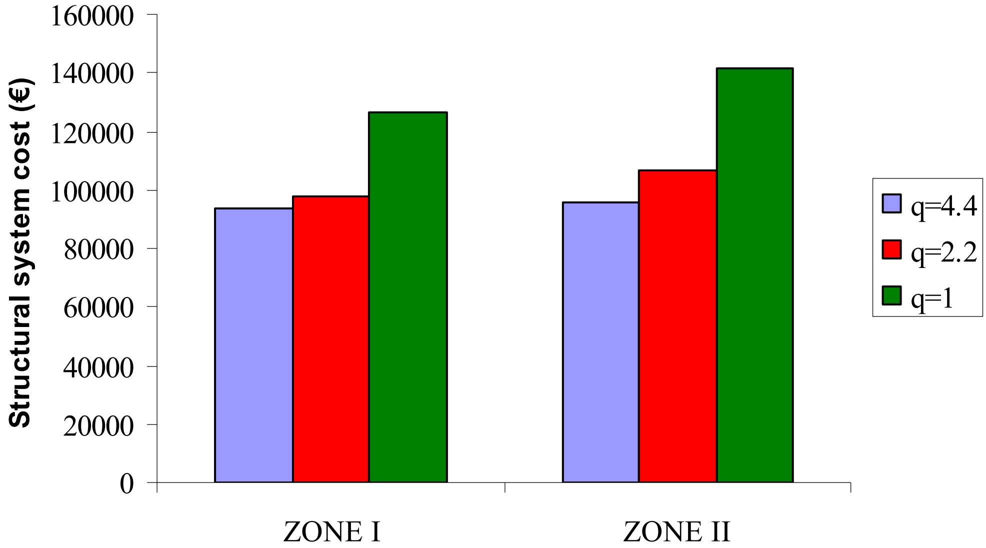


Figure 9. Construction cost of the structural system – ductile wall systems

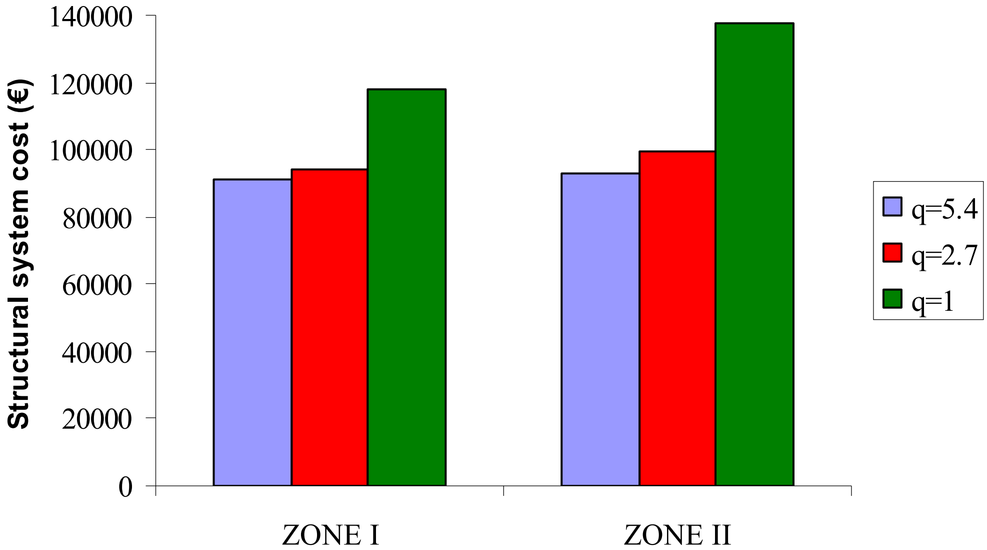


Figure 10. Construction cost of the structural system – dual wall-equivalent systems

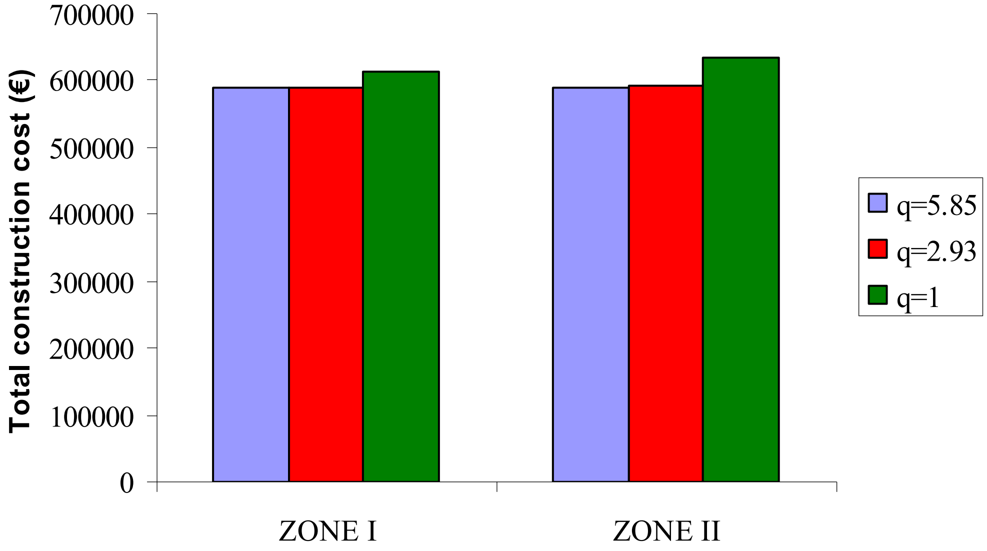


Figure 11. Total construction cost – frame systems

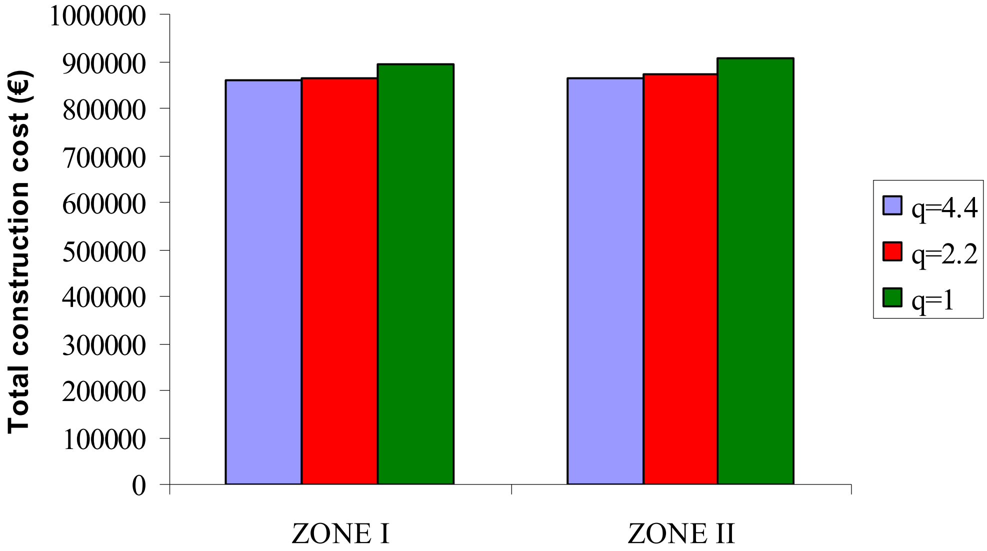


Figure 12. Total construction cost – ductile wall systems

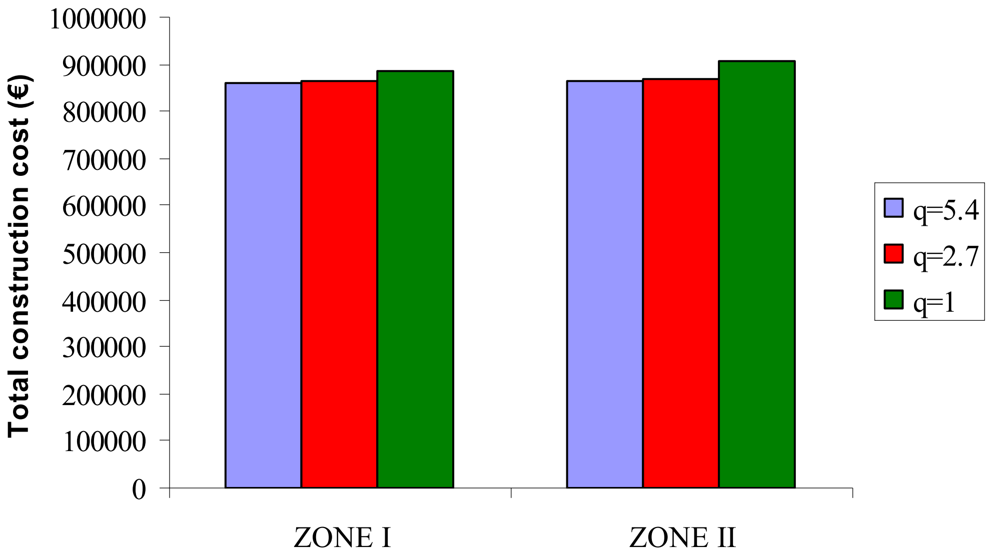


Figure 13. Total construction cost – dual wall-equivalent systems

ASSESSMENT OF STRUCTURAL RESPONSE

The seismic response of the examined buildings is evaluated by conducting pushover analysis according to the provisions of Eurocode 8 (Part 1, Section 4.3.3.3.4.2). The analysis is performed using the program SAP 2000 v14.0.0. The modeling of the inelastic behaviour is based on the following assumptions:

- Shear failure is precluded.
- The inelastic deformations are concentrated at the critical sections, i.e. at the ends of the structural components (plastic hinges).
- Plastic hinges are modeled by bilinear elastic-perfectly plastic moments-rotations diagrams derived from the relevant Eurocode 8 provisions (Part 3, Annex A.3).
- The moment-axial force interaction is taken into account by appropriate interaction surface incorporated in SAP 2000.

Each building is analyzed for each direction of excitation independently applying a “modal” load pattern (Eurocode 8, Part 1, Section 4.3.3.3.4.2.2). The pushover curves are idealised to bilinear curves and the target roof displacements are determined applying the relevant Eurocode 8 procedure (Part 1, Annex B).

In Figs. 14 to 16 the number of plastic hinges formulated for each building and each direction of excitation is shown. It should be noticed that some hinges occur even at the buildings designed for the objective FP (q=1), i.e. for nearly elastic response. This is mainly due to the aforementioned procedure of Eurocode 8 for the idealisation of the pushover curve which - regardless the earthquake intensity - always leads to a period of the equivalent single degree of freedom system higher than the elastic one and as a consequence to roof displacements greater than those corresponding to linear elastic response. As it is expected, all plastic rotations comply with the acceptance criteria of Life Safety Performance level (Eurocode 8, Part 3, Annex A.3.1). From Figs. 14 to 16 becomes clear that the buildings designed for partial seismic protection experience extensive inelastic response for the design earthquake which implies significant repair costs. Thus, the economic benefits achieved by the adoption of a lower design objective are in question if the total life - cycle cost of the buildings is considered. Furthermore, in case of a ground motion with higher intensity than the design earthquake, the buildings designed for lower performance levels may experience non-repairable damage or global collapse with significant economic losses and other social consequences. Even the protection of human life is in question. It is obvious that this lack of safety against strong earthquakes cannot be accepted by the society. Thus, the fundamental design philosophy adopted by modern seismic codes should probably be revised. This issue requires further investigation, which is beyond the scope of the present paper.

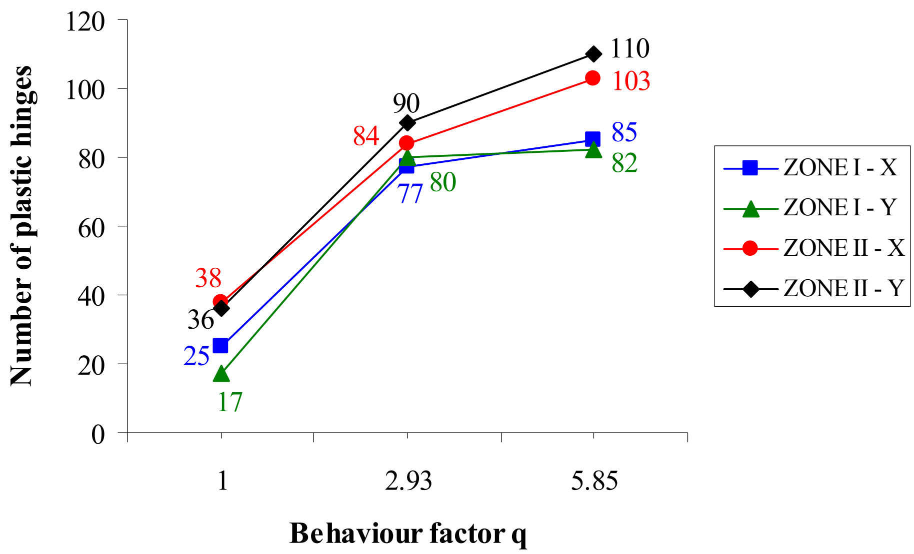


Figure 14. Number of plastic hinges – frame systems

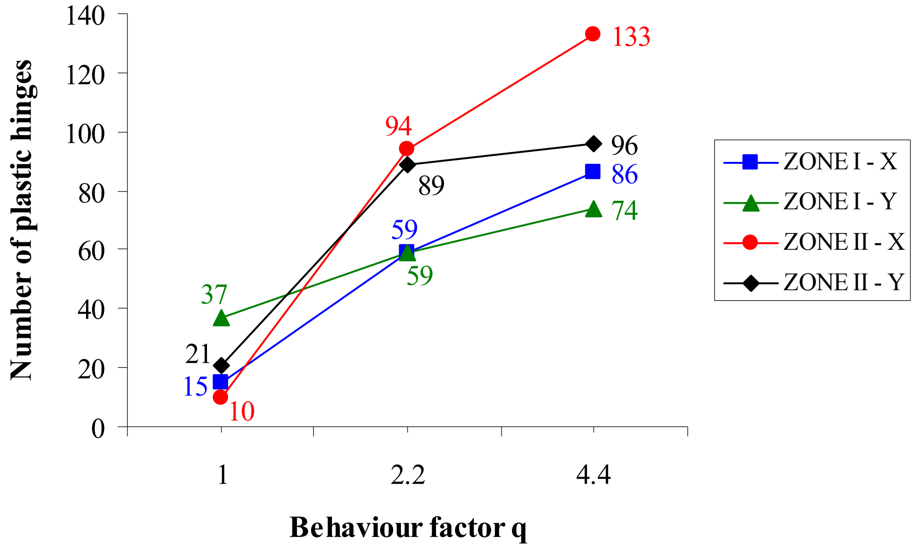


Figure 15. Number of plastic hinges – ductile wall systems

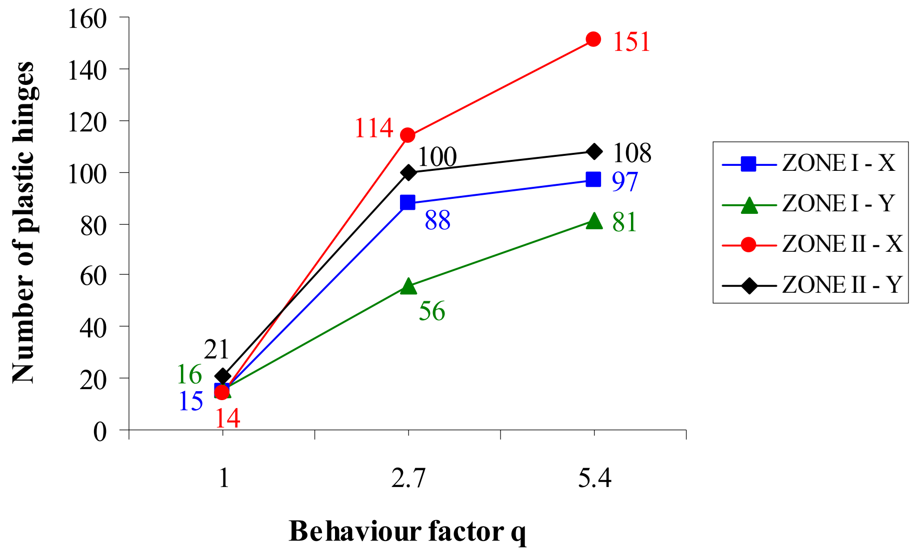


Figure 16. Number of plastic hinges – dual wall-equivalent systems

CONCLUSIONS

The scope of this paper is the estimation of the influence of the design objective selection to the construction cost of buildings. For this purpose a parametric study is conducted comprising the design of a series of multi-storey reinforced concrete buildings with different structural systems and plan configurations for alternative design objectives and seismic action levels. The main conclusions derived are as follows:

- The increase of the construction cost resulting from the adoption of the PP1 ($q = \frac{1}{2} q_{max}$) instead of the PP2 ($q = q_{max}$) design objective is negligible, due to the overstrength that almost always possess the conventional buildings.
- The increase of the construction cost resulting from the adoption of the FP ($q = 1$) instead of the PP2 ($q = q_{max}$) design objective ranges between 3% and 4% for seismic hazard level zone I and between 5% and 8% for seismic hazard level zone II.
- The adoption of a higher design objective influences much more the construction cost of frame systems in comparison with the ductile wall or the dual wall-equivalent systems.

- The buildings designed for lower design objectives experience extensive inelastic deformations for the design seismic action. Thus, the economic benefits from the reduction of construction cost may be eliminated due to the increased repair cost in case of strong earthquakes.

Conclusively, the aforementioned findings, which are consistent to the findings of previous investigations (Avramidis et al. 2003), (Lagaros et al. 2006), indicate that the additional cost for full protection of buildings against the design earthquake is not prohibitive.

REFERENCES

Anastassiadis K, Avramidis IE and Morfidis K (2000) "Full and partial seismic protection of buildings – Proposition of a new design philosophy," *Bulletin of Technical Chamber of Greece*, 2094

Avramidis IE, Anastassiadis K, Athanatopoulou A and Katavelos A (2003) "The Myth of the Excessive Cost of Seismic Resistant Structures Designed for Elastic Behaviour for the Design Earthquake", *Proceedings of the 14th National Congress on RC Structures*, Kos, Greece, 15-17 October

European Committee for Standardization (2002) Eurocode 2: Design of Concrete Structures, EUS: Brussels

European Committee for Standardization (2004) Eurocode 8: Design of Structures for Earthquake Resistance, EUS: Brussels

Lagaros N, Fotis A and Krikos S (2006) "Assessment of seismic design procedures based on the total cost," *Earthquake Engineering and Structural Dynamics*, 35:1381–1401