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A Procedure for the Assessment of the Behaviour Factor for Steel MRF Systems Based on Pushover Analysis

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Abstract. *To calculate the behaviour factor by means of a re-analysis of a pushover curve, each modern seismic code defines the reference parameters in a different way which results in a wide variety of possible choices to select them. In order to overcome this kind of problem, this paper discusses some possible definitions of the reference overstrength and ductility parameters which can be used for all types of structures. To do so, the influence of different choices of such parameters in the assessment of the behaviour factor will be investigated for three case studies of different composite steel-concrete MRF buildings. Finally, re-analysis of the obtained results is presented and discussed.*

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1 INTRODUCTION

In general, the usual process for the design of a non-statically determined structure requires three steps:

- A pre-design phase, when the engineer, mostly based on his experience and/or by "analogy" with other existing structures of similar dimensions and typology, identifies an initial "size" of the structural load-carrying members.
- A second phase when the structural analysis is carried out, leading to the assessment of the "demand" (in terms of axial load, bending moments, shear and torque and of displacements or rotations in the joints) in the various structural members under the external actions
- A third phase when the "capacity" of the load carrying structural members, of their connections and of the building foundations is verified (under the stresses induced by the internal actions derived by the structural analysis carried out in phase 2) both in terms of "strength" and "ductility". This second property being extremely important and desirable when the structure to be designed is in an earthquake prone area, because it allows for energy dissipation by plastic deformations.

For a building in a seismic area, due to the interaction between gravity (vertical) and seismic (horizontal) loads, the design process, although following more or less the same path, might not be so straightforward because of the nonlinear response of the structure under seismic actions.

The modern seismic design approach accounts for the seismic energy dissipation through inelastic deformations, which however should not exceed the limits related to the local and global ductility of the structure. Based on this idea, in the early 80s, in all the world a new generation of structural design codes were issued, considering two alternative methods for structural analysis:

i) Nonlinear dynamic analysis
ii) Linear elastic response spectrum analysis

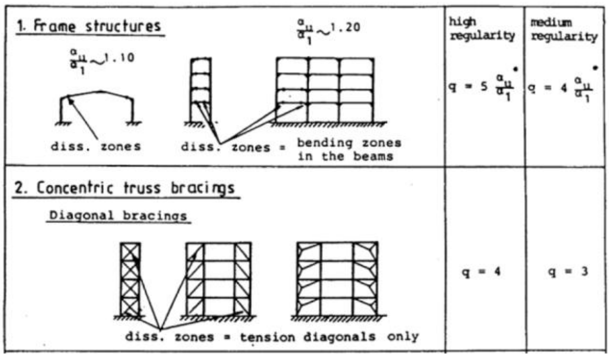
The use of the first method is motivated by particular cases related to the importance of the building or its function, however its application is unusual in everyday practice because of its complexity. The second method allows the design of buildings in seismic areas by means of linear elastic analysis but considering implicitly the nonlinear structural behaviour, making use of a "behaviour factor".

The behaviour factor (so called "q-factor" according to the European Standard [1]) can be defined as the ratio of the peak ground acceleration producing collapse of the structure to that at which first yielding occurs. It represents a factor accounting for both the ductility demand associated with the seismic input and the global and local inelastic resources of the structure which depend on the structural typology, on the ductility of the material [2], [3], on the p-Δ effect and on eventual brittle fracture mechanisms [1], [4], [5].

A precise estimation of the behaviour factor is very complex, since it depends on many factors such as: the accelerogram acting at the base of the structure, material, geometry, degree of redundancy, local and global buckling effects and the types of structural connections. Therefore, in the 80's, generic estimates of the q-factor have been provided by extensive research carried out all over the world [4]–[22], which led, for example, in Europe to the first publication of EuroCode 8 (May 1988). The behaviour factors defined in this edition remain nearly unchanged. Almost all the research performed up-to-date focused on verifying and validating these behaviour factors assessed in 1988. A comparison between two editions of EuroCode 8 is shown in Figure 1 [23].

As shown in Table 1-1 with reference to the MRF, the behaviour factor values vary from code to code (e.g. EC8 [1], FEMA [24], AIJ, etc.), with a large discrepancy, mainly due to the different approaches assumed by various codes for the definition of the q-factor.

This is in general not acceptable from a scientific and practical point of view. The economic implications related to the fact that a same structure, under the same seismic actions, designed by various codes and adopting the same safety factors results in different weight of materials, is barely unacceptable.



(a) EN 1988-1-1: 1998

STRUCTURAL TYPE	Ductility Class DCM	Ductility Class DCH
a) Moment resisting frames	4	$5\alpha_u/\alpha_1$
b) Frame with concentric bracings		
Diagonal bracings	4	4
V-bracings	2	2,5
c) Frame with eccentric bracings	4	$5\alpha_u/\alpha_1$
d) Inverted pendulum	2	$2\alpha_u/\alpha_1$
e) Structures with concrete cores or concrete walls	See section 5	
f) Moment resisting frame with concentric bracing	4	$4\alpha_u/\alpha_1$
g) Moment resisting frames with infills		
Unconnected concrete or masonry infills, in contact with the frame	2	2
Connected reinforced concrete infills	See section 7	
Infills isolated from moment frame (see moment frames)	4	$5\alpha_u/\alpha_1$

(b) EN1998-1-1: 2004

Figure 1-1 q Factors provided in the First and Last Version of EuroCode

Table 1-1 Behaviour Factor's Values in 15 Different Seismic Codes Provision for MRFS [3]

Country	Seismic Code	Year	Design Behaviour Factor
Albania	Earthquake Resistant Design Regulations*	1989	5
Australia	Structural design actions (AS1170.4)	2007	8
Bulgaria	design of buildings and facilities in seismic regions	2014	5
Canada	CAN/CSA-S16-01	2007	4
Greece	Greek code for Seismic Resistant Structures (EAK2000)	2000	4
India	Criteria For Earthquake Resistant Design Of Structures (IS 1893)	2002	5
Iran	Code for Seismic Resistance Design of Buildings (2800)	2014	3.5-7.5
Italy	Technical standards for construction (NTC 08)	2008	4-6.5
Japan	Seismic design of buildings (AIJ)	2000	4
Mexico	Construction Regulations for the Federal District	2003	4
New Zealand	Structural design actions (AS/NZS-1170)	2002	3
Romania	Seismic design code P100-1	2013	5
Spain	Actions on Buildings (NBE-AE-88)	2007	4
Turkey	Specification for Buildings to be Built in Seismic Zones	2007	5, 8
USA	Uniform Building Code (UBC)	1997	12
	NERPH, FEMA P 750	2009	8
European	EuroCode 1998-1	2004	4-8

*No available information on the revision of the code (IISEE)

There are two problems:

1. The available "q" values suggested by the Codes, are valid only for a limited number of structural typologies (those indicated by the codes themselves) but in general they are not suitably applicable (or extendable) to different structural types (e.g. to innovative structural systems not yet encompassed in the Codes). Indeed, several case studies analysing the real ductility of the structures designed with a certain q-factor show that the real ductility is far from the assumed value.

2. The current forced based design procedure is too simple and generalizing, mostly causing over-designed structures.

Recent strong earthquakes showed that steel buildings designed according to the modern codes suffered very little local damages, and did not sufficiently exploit the ductility of their components. Design engineers never tend to optimize their design using more advanced procedures, mainly because of their complexity. Even if an engineer dares to use the current advanced design procedures (such as nonlinear pushover analysis), without "reliable" information on the nonlinear response of each single component of his structure, he cannot achieve "reliable" results.

Academic community agrees on the weakness of the available linear forced-based design code procedures, and have been working on the more complicated procedures to provide tools for the optimized design of structures (such as displacement based design). However, apparently, there are not yet available tools that convince the design engineers to use such methods.

Presently, the linear seismic design procedure requires additional activities to the general linear design procedure defined at the beginning of this introduction that should be modified as follows:

1) After phase 1, a q-factor has to be assumed according to the code; then phase 2 and 3 must be performed.
2) The resulting displacements and rotations have to be amplified by the q-factor, and the capacity of the connection to develop such ductility must be verified.

In order to verify the global and local capacity of the structure in terms of ductility, the easiest way allowed by the modern software tools is to perform a push-over analysis. However, this doesn't assure that the "actual" q (it means the actual ductility of the structure) verified by the pushover analysis is consistent with the "assumed" one (the "assumed" q-factor given by the codes). Of course, for an optimal design these two values should more or less coincide.

This problem can be overcome by performing a pushover analysis after phase 1, and assessing the capacity (in terms of strength and ductility) of the preliminary designed structure as well as its "actual" q-factor based on the results of such analysis. Subsequently, phase 2 and 3 can be performed, but in this case "assumed" and "actual" ductility are more or less coincident, and the design is optimized. This is schematically described in Figure 2.

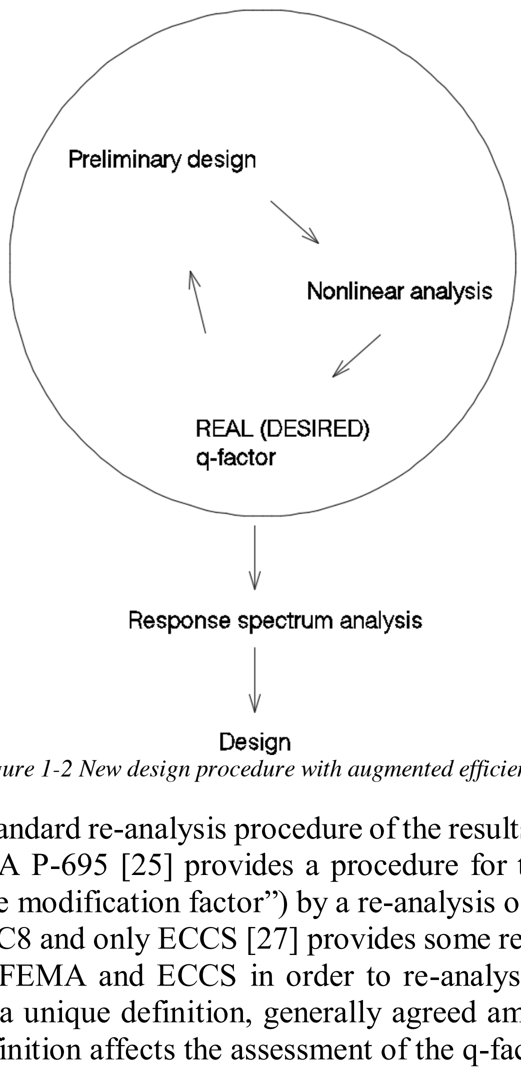


Figure 1-2 New design procedure with augmented efficiency

However, in order to do this, a standard re-analysis procedure of the results of the pushover analysis should be identified. Presently, only FEMA P-695 [25] provides a procedure for the definition of the q-factor (in [24]–[26] referenced to as "response modification factor") by a re-analysis of the push over curve. In Europe, such a method is not proposed by EC8 and only ECCS [27] provides some recommendations. In any case, the main parameters adopted by both FEMA and ECCS in order to re-analyse the push-over curve might be defined in different ways, because a unique definition, generally agreed among the scientific community is presently missing despite such definition affects the assessment of the q-factor.

In this paper, possible definitions of such parameters are considered, resulting in 90 different combinations/methods for the assessment of the q-factor. These methods are hereafter applied to three case studies of MRF systems, designed with an initial value of q=4.0. The 90 values of the q-factors obtained by re-analysing the push-over curves for each case study, are then compared with the initial design value, identifying those methods which lead to q values close to the initial one, and those which lead the values far from it.

In order to identify an optimal method for the definition of the q-factor based on the re-analysis of the push-over curve, incremental dynamic analysis (IDA) [28], [29] will be performed on the same case studies, as part of the EU-RFCS research project INNOSEIS, currently in progress [30]. INNOSEIS project focuses on the dissipative devices suitable for steel structures [31]–[36]. Hereafter, the various definition of the main parameters and a comparison of the achieved results with the initial design value of the q-factor are presented. However, it will be only after comparison with the results of the IDA study that the optimal method can be chosen, being the one giving a q value consistent with the one obtained by the IDA analysis. Such type of analyses, in fact, although being more sophisticated and time consuming, will lead to a more precise assessment of the q-factor than that achievable by means of re-analysis of the pushover curve. However, because of its sophistication, such an approach is not suitable for everyday application in the engineering practice, on the contrary of the one based on re-analysis of the pushover response curve.

2 PROPOSED BEHAVIOUR FACTOR EVALUATION PROCEDURE

According to ECCS [27] and FEMA P-695 [25], the behaviour factor q can be calculated as a product of over-strength q_{Ω}, ductility q_{μ} and redundancy q_{ξ} factors, as follows:

$$q = q_{\Omega} . q_{\mu} . q_{\xi} \tag{1}$$

Where

q_{Ω} is an over-strength dependent factor, function of the non-linear structural response

q_{μ} is a ductility dependent factor, function of the displacement ductility

q_{ξ} is a damping dependent factor, equal to 1.0 when assuming that the same damping ratio holds for both elastic and inelastic analysis

A proper approximation of the over-strength factor "q_{Ω}" may be:

$$q_{\Omega} = \frac{F_y}{F_1} \tag{2}$$

Where

F_y is the strength corresponding to the knee-point of the idealized bilinear elastic-plastic behavior curve.

F_1 is the strength corresponding to the first significant yielding of the structure.

The ductility factor q_{μ} according to Newmark and Hall [37] can be expressed as a function of the system ductility μ, related to the natural period of vibration T, as follows:

$$\begin{aligned} q_{\mu} &= 1.0 && \text{(for T<0.03s)} \\ q_{\mu} &= \sqrt{2\mu - 1} && \text{(for 0.03s <T<0.5)} \\ q_{\mu} &= \mu && \text{(for T>0.5s)} \end{aligned} \tag{3}$$

with

$$\mu = \frac{d_m}{d_y} \tag{4}$$

Where

d_m is the displacement corresponding to the maximum strength

d_y is the displacement corresponding to the knee-point of the idealized bilinear elastic-plastic behavior curve.

2.1 Definition of Reference Parameters for the Assessment of the q-Factor by Re-Analysis of the Pushover Curve

For the assessment of the behaviour factor by re-analysis of the Push-Over behavior curve, various modern seismic design codes define the reference parameters in a different way. FEMA P-695 [25], for example, defines the overstrength factor q_{Ω} as the ratio of the maximum actual strength of the structure to the design base shear V_{max}/V_{design}, whereas EC8 [1] refers to the first significant yielding α_u/α_1.

In the same way, to define the period based ductility factor q_{μ}, FEMA P-695 [25] defines the maximum displacement δ_u at 20% loss of strength of the structure in post hardening, while, EC8 [1] defines the same parameter as the maximum displacement corresponding to the formation of the plastic mechanism.

Hence, in this section, some possible definitions of these parameters and their combinations will be discussed and the effect of such combination on the assessment of the q-factor will be highlighted.

As explained above, calculation of the behaviour factor is based on 5 main parameters: F_1, F_y, F_m, d_y and d_m. While F_m can be univocally defined, with a general agreement, as the maximum actual strength of the structure, the other parameters might be defined in different ways as shown in the following Tables 2-1, 2-2 and 2-3.

Table 2-1 Possible Definitions for Maximum Horizontal Roof Displacement (d_m) Corresponding to the Maximum Strength or/and in Softening Branch

Identifier	Definition
d_m-1	Horizontal roof displacement corresponding to the maximum structural strength
d_m-2	Horizontal roof displacement corresponding to 5% loss of structural load carrying capacity, in softening branch
d_m-3	Horizontal roof displacement corresponding to 10% loss of structural load carrying capacity, in softening branch
d_m-4	Horizontal roof displacement corresponding to 15% loss of structural load carrying capacity, in softening branch
d_m-5	Horizontal roof displacement corresponding to 20% loss of structural load carrying capacity, in softening branch

The actual deformation energy for all methods explained in Table 2-2 except F_y-d_y-5 is assumed to be equal to the one obtained with reference to the idealized bilinear elastic-plastic curve. By combining the definitions of F_y and F_1 given in Table 2-2 and Table 2-3 respectively, with the five definitions of d_m, given in Table 2-1, 90 possible different definitions of the q-factor obtained by re-analysis of the pushover curve can be identified, as shown in Table 2-4.

The behaviour factor identified by re-analysis of the pushover response curves based on the above combinations of the reference parameters must be further compared with the result of incremental non-linear dynamic analysis (IDA) in order to identify which combination leads to the best consistency between the two methods (push-over re-analysis and IDA).

Table 2-2 Possible Definitions for yielding point (F_y-d_y)

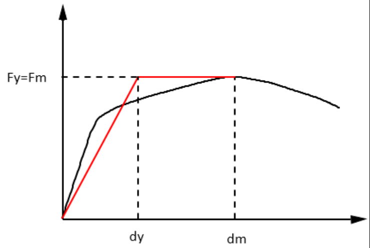
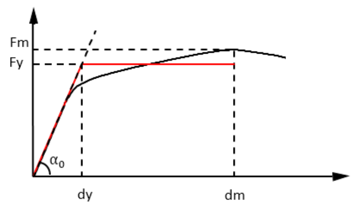
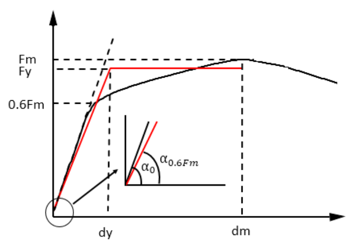
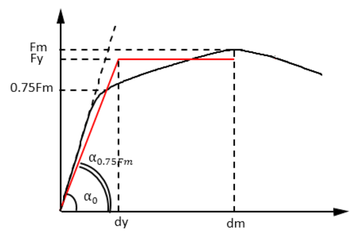
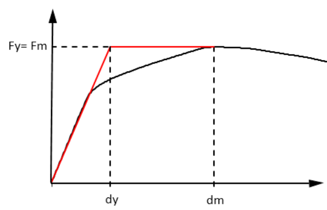
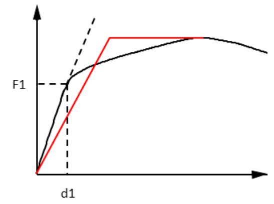
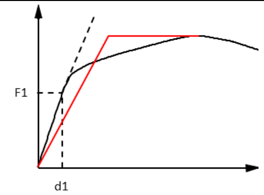
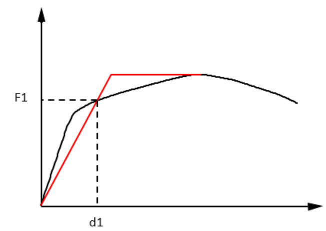
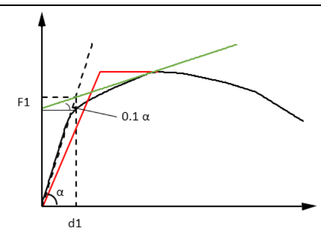
Identifier	Definition	Equations	Schematic View
F_y-d_y-1	Knee point of the idealized bilinear elastic-perfectly plastic curve based on equivalence of the area under both curves (capacity and bilinear curve) up to d_m, with $F_y = F_m$.	1)$F_y = F_m$ 2)$d_y = 2(d_m - E_m/F_y)$ E_m being the area under the capacity curve up to "d_m"	
F_y-d_y-2	Knee point of the idealized bilinear elastic-perfectly plastic curve based on equivalence of the area under both curves (capacity and bilinear curve) up to d_m, where the initial stiffness of the idealized system (F_y/d_y) is equal to the initial stiffness of the capacity curve ($\tan\alpha_0$).	1)$F_y/d_y = \tan\alpha_0$ 2)$d_y = d_m - \sqrt{{d_m}^2 - \frac{2E_m}{\tan\alpha_0}}$ E_m being the area under the capacity curve up to "d_m"	
F_y-d_y-3	Knee point of the idealized bilinear elastic-perfectly plastic curve based on equivalence of the area under both curves (capacity and bilinear curve) up to d_m, where the initial stiffness of the idealized system (F_y/d_y) is equal to the secant stiffness of the capacity curve at $0.6F_m$ ($\tan\alpha_{0.6Fm}$).	1)$F_y/d_y = \tan\alpha_{0.6Fm}$ 2) $d_y = d_m - \sqrt{{d_m}^2 - \frac{2E_m}{\tan\alpha_{0.6Fm}}}$ E_m being the area under the capacity curve up to "d_m"	
F_y-d_y-4	Knee point of the idealized bilinear elastic-perfectly plastic curve obtained by equivalence of the area under curves up to d_m, where the initial stiffness of the idealized system (F_y/d_y) is equal to the secant stiffness of the capacity curve of the structure at $0.75F_m$ ($\tan\alpha_{0.75Fm}$).	1)$F_y/d_y = \tan\alpha_{0.75Fm}$ 2)$d_y = d_m - \sqrt{{d_m}^2 - \frac{2E_m}{\tan\alpha_{0.75Fm}}}$ E_m being the area under the capacity curve up to "d_m"	
F_y-d_y-5	Knee point of the idealized bilinear elastic-perfectly plastic curve with $F_y = F_m$ and the initial stiffness of the idealized system (F_y/d_y) equals to the initial stiffness of the capacity curve ($\tan\alpha_0$).	1)$F_y/d_y = \tan\alpha_0$ 2)$F_y = F_m$	

Table 2-3 Possible Definitions for first significant yielding (F_1) and d_1

Identifier	Definition	Schematic View
F_1-d_1-1	Point on the capacity curve corresponding to the first global plasticization of the structure.	
F_1-d_1-2	Point on the capacity curve corresponding to the first yielding of any elements of the structure (Local Plasticization).	
F_1-d_1-3*	Intersection point between the capacity curve and the initial stiffness of the idealized elastic-perfectly plastic system.	
F_1-d_1-4	Intersection point between the straight line with a slope equal to the initial stiffness of the capacity curve ($\tan\alpha_0$) and the tangent to the same curve with slope equal to 10% of $\tan\alpha_0$.	

* F_1-$D_1$3 cannot be generated with F_y-d_y-2 and F_y-d_y-5 as there is no intersection point in these methods

Table 2-4 All Possible q Factor Evaluation Methods

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	F1-d1-1	**F1-d1-2**	**F1-d1-3**	**F1-d1-4**	**F1-d1-1**	**F1-d1-2**	**F1-d1-3**	**F1-d1-4**	**F1-d1-1**	**F1-d1-2**	**F1-d1-3**	**F1-d1-4**	**F1-d1-1**	**F1-d1-2**	**F1-d1-3**	**F1-d1-4**	**F1-d1-1**	**F1-d1-2**	**F1-d1-3**	**F1-d1-4**

3 CASE STUDIES

In order to evaluate the behaviour factor of a structure using the above definitions, three composite steel-concrete MR Frames with different height are examined. All buildings have a composite steel concrete slab and secondary beams which transfer the loads to the main frames, where dissipative bolted beam splices are employed [38], [39]. The three buildings are regular both in height and plan and considered as a general office (class-B) buildings. The assumed design peak ground acceleration is 0.3g and the design value of the behaviour factor was assumed equal to 4.0. All the structures are those used in the framework of the EU-

RFCS project INNOSEIS [30]. Figure 3-1 displays the plan and elevation view of the 2, 4 and 8 storey-buildings.

As the considered case studies are “non-conventional” structural types, proposed values of the q factors are not present in EC8 [1]. This is one of the typical applications of the proposed assessment procedure, allowing to overcome the codes.

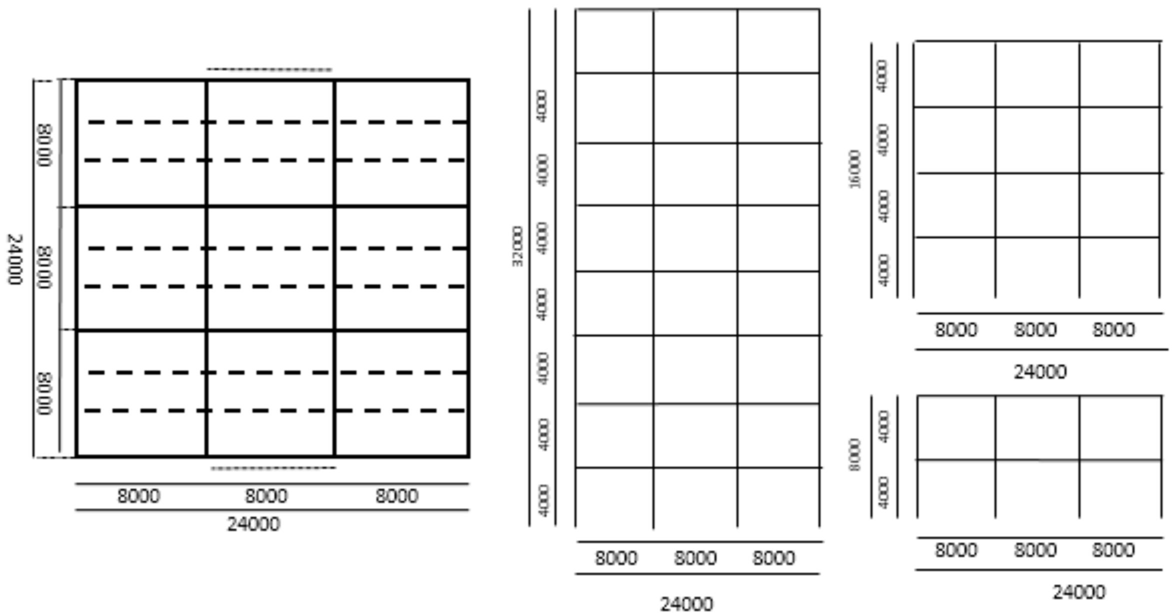


Figure 3-1 Plan and Elevation View of the 2/4/8-story archetype structures (diameter in mm)

3.1 Dissipative FUSEIS Bolted Beam Splice

The dissipative “FUSEIS” bolted beam splices [38], [39] are located in a cross-section close to the beam ends at a certain distance from beam-to-column connections, introducing a discontinuity on the composite beams and splicing the two parts through steel plates bolted to the web and flange of the beam. These devices are designed to act as fuses, forcing the plastic hinges to develop within the fuse devices, preventing the spreading of damage into the main structural members (beams and columns).

Repair work after an earthquake is limited to replacing the splice plates with new ones, a low-cost and rather quick operation. Figure 3-2 shows the schematic representation of the bolted beam splice. More information can be found in [40], [41].

3.2 Design and Numerical Modeling

The modelling of the buildings was performed by the commercial finite element software SAP2000.Ve.19. All buildings are designed according to EN1993-1 [42], EN1998-1 [1], EN1994-1 [43] and to the specific design guideline of the bolted beam splice as a dissipative system [39] . Columns sections vary along the height and change for each the building (depending on the numbers of total floors). The assumed sections are given in Table 3-1. For all floors and buildings, IPE450 has been as primary composite beams.

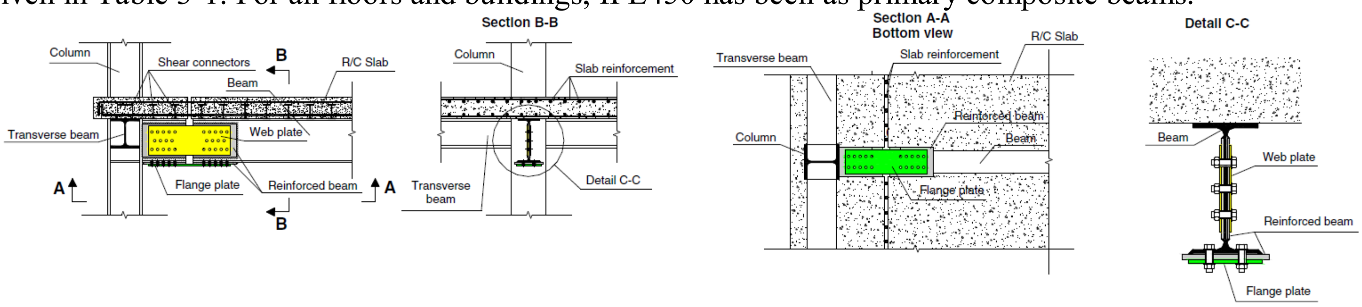


Figure 3-2 Schematic Representation of the Bolted Beam Splices [40]

Table 3-1: Columns Section for the 8/4 and 2 Storey Buildings

	8-Storey		4-Storey		2-Storey	
Storey	Centre	Perimeter	Centre	Perimeter	Centre	Perimeter
1-2	HEM550	HEB550	HEM450	HEB450	HEM360	HEB360
3-4	HEM500	HEB500	HEM360	HEB360	----	----
5-6	HEM450	HEB450	----	----	----	----
7-8	HEM360	HEB360	----	----	----	----

Table 3-2 Dimension of the Beam Splices and Their Distribution in Height

Storey	8-Storey	4-Storey	2-Storey
1-2	170x12 (mm)	170x10 (mm)	170x8 (mm)
3-4	170x12 (mm)	170x8 (mm)	----
5-6	170x10 (mm)	----	----
7-8	170x8 (mm)	----	----

4 RESULTS AND DISCUSSIONS

Dissipative bolted beam splices [39] introduced in this model were able to dissipate the energy during strong seismic ground motions protecting the main structural members from inelastic deformations and damages [30]. Table 4-1 shows the fundamental period of vibration and participating mass for each building.

Table 4-1 Fundamental Period of Vibration and Mass Participations for 2, 4 and 8 Storey Building

Mode	8-Storey			4-Storey			2-Storey		
	Period [s]	UX [%]	SUM UX [%]	Period [s]	UX [%]	SUM UX [%]	Period [s]	UX [%]	SUM UX [%]
1	2.04	77	77	1.06	78	78	0.55	86	86
2	0.67	11	88	0.32	13	91	0.15	13.7	99.7
3	0.36	4	92	0.16	5	96	0.08	0.1	99.8

4.1 Behaviour Factor Calculations

Figure 4-1 displays the response of the case-study buildings, in terms of capacity (push-over) curves, under a lateral force distribution according to the first modal shape [1], [25], [27].

These curves were re-analysed based on the combinations of the possible definitions of the reference parameters previously presented in Table 2-4 and 90 different values of the q-factor were derived for each building, as shown in Table 4-2, Table 4-3, Table 4-4 and Figure 4-2.

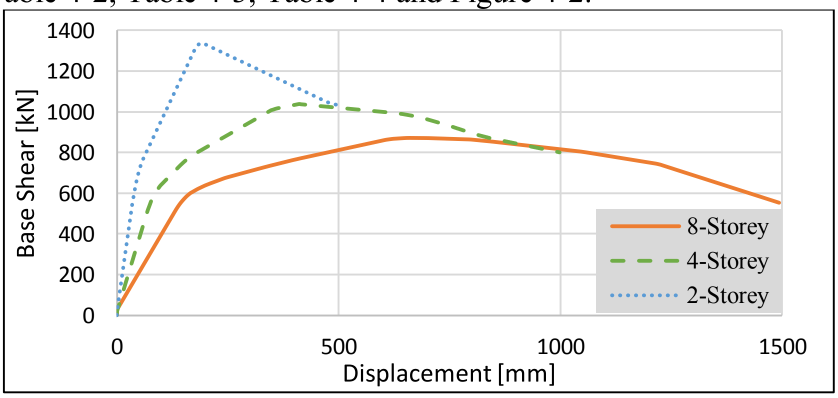


Figure 4-1 Push-Over Curves

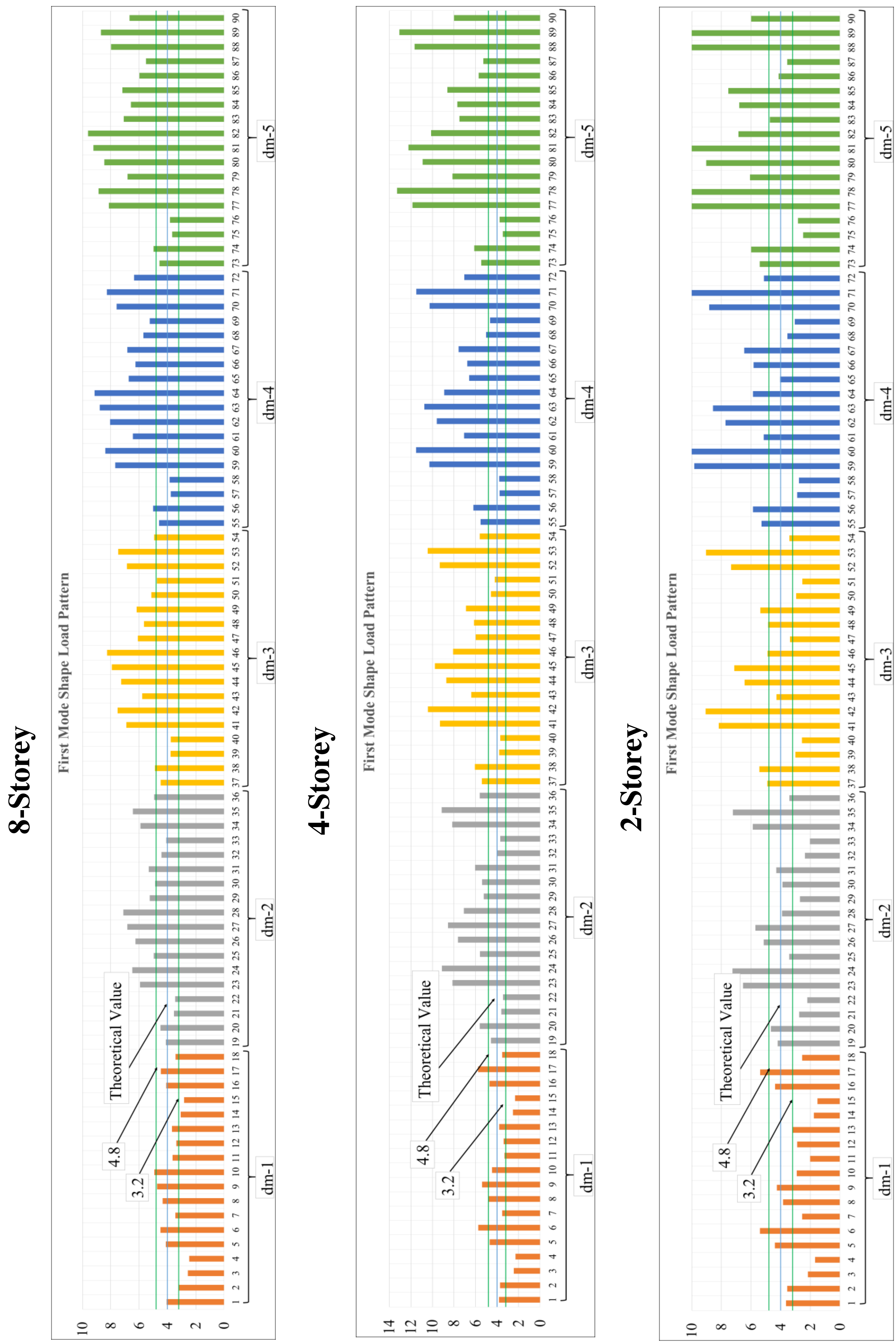


Figure 4-2 Behaviour Factor Calculated Based on the Different Possible Definitions of the Reference Parameters

The large scatter of the obtained results in terms of q-factor is evident. For this reason, while waiting for the results of the IDA's (presently ongoing at NTUA within the INNOSEIS Project), in order to identify which pushover response curve re-analysis method gives a q-factor consistent with the IDA analysis, hereafter attention was focused on those methods giving results ranging within ±20% respect to the assumed design q-factor value of 4.0.

The "shortlisted" values (and the related combination of definitions of the relevant parameters) are highlighted in Table 4-2, Table 4-3 and Table 4-4, respectively for the 2, 4 and 8 storey building.

Table 4-2 Behaviour Factor Calculated Based on the Different Possible Definitions of the Reference Parameters for 8-Storey Building

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.4	3.2	2.6	2.5	4.1	4.5	---	3.5	4.3	4.7	4.9	3.6	3.4	3.7	3.1	2.8	4.1	4.5	---	3.4
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.1	4.5	3.5	3.5	5.9	6.5	---	5.0	6.3	6.8	7.1	5.2	4.9	5.3	4.4	4.1	5.9	6.4	---	4.9
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.5	4.9	3.8	3.8	6.9	7.5	---	5.8	7.3	7.9	8.3	6.1	5.7	6.2	5.1	4.7	6.9	7.5	---	5.7
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.6	5.0	3.8	3.8	7.7	8.4	---	6.4	8.0	8.8	9.1	6.7	6.3	6.8	5.7	5.3	7.6	8.3	---	6.4
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.6	5.0	3.7	3.8	8.1	8.9	---	6.8	8.5	9.2	9.6	7.1	6.6	7.2	6.0	5.5	8.0	8.7	---	6.7
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 4-2 Behaviour Factor Calculated Based on the Different Possible Definitions of the Reference Parameters for 4-Storey Building

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.7	3.7	2.4	2.3	4.7	5.8	---	3.5	4.8	5.4	4.5	3.3	3.4	3.8	2.5	2.3	4.7	5.8	---	3.5
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.6	5.6	3.6	3.4	8.1	9.1	---	5.6	7.6	8.5	7.1	5.2	5.4	6.0	4.0	3.7	8.2	9.1	---	5.6
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	5.4	6.1	3.8	3.7	9.3	10.4	---	6.4	8.7	9.8	8.1	6.0	6.1	6.9	4.6	4.2	9.3	10.4	---	6.4
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	5.5	6.2	3.8	3.8	10.3	11.5	---	7.1	9.6	10.7	8.9	6.6	6.8	7.6	5.0	4.6	10.3	11.5	---	7.0
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	5.5	6.1	3.5	3.8	11.8	13.3	---	8.1	10.9	12.2	10.1	7.5	7.7	8.6	5.7	5.3	11.7	13.1	---	8.0
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 4-2 Behaviour Factor Calculated Based on the Different Possible Definitions of the Reference Parameters for 2-Storey Building

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.6	3.6	2.2	1.7	4.4	5.4	---	2.6	3.8	4.3	2.9	2.0	2.9	3.2	1.8	1.5	4.4	5.4	---	2.5
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.2	4.7	2.8	2.2	6.5	7.2	---	3.4	5.1	5.7	3.9	2.7	3.9	4.3	2.4	2.0	5.9	7.2	---	3.4
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.9	5.4	3.0	2.6	8.2	9.1	---	4.3	6.4	7.1	4.9	3.4	4.9	5.4	2.9	2.5	7.4	9.0	---	4.3
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	5.3	5.9	2.9	2.8	9.8	10.9	---	5.2	7.7	8.6	5.9	4.0	5.8	6.5	3.5	3.1	8.8	10.9	---	5.1
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	5.4	6.0	2.5	2.8	11.6	12.9	---	6.1	9.0	10.0	6.9	4.7	6.8	7.5	4.1	3.6	10.3	12.7	---	6.0
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

4.2 The Optimal Methods

Among all the possible combinations of definitions of the reference parameters given in section 2.1 for the assessment of the behaviour factor, the six methods summarized in Table 4-2 seem to give the best correlation with respect to the target value q=4.0 (see Figure 4-2 and Table 4-2, Table 4-3 and Table 4-4). The deformation energy, for all methods except method 16, is calculated with reference to the bilinear elastic- plastic response.

Table 4-2 Optimal Methods to Define the Behaviour Factor

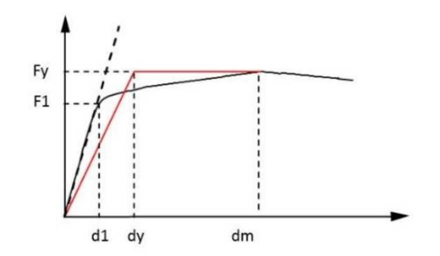
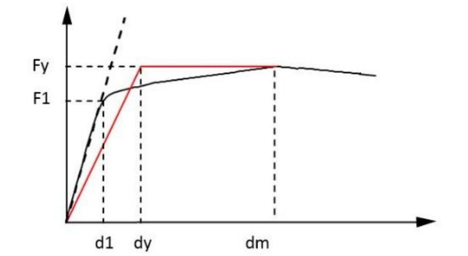
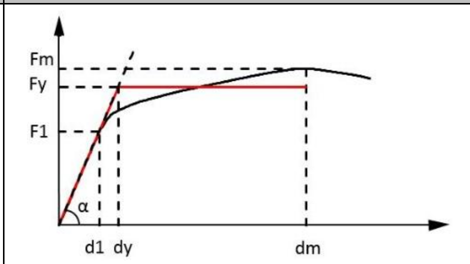
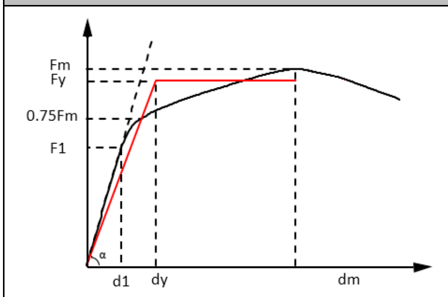
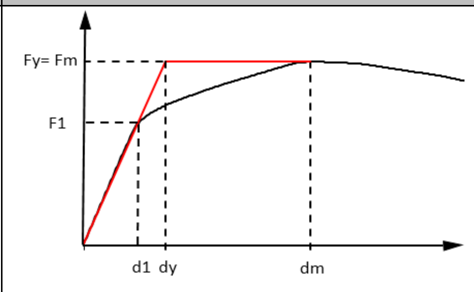
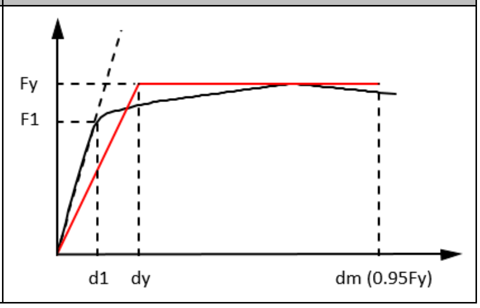
Method 1	Method 2	Method 5
		
Method 13	**Method 16**	**Method 19**
		

Table 4-3, Table 4-4 and Table 4-5 summarize, respectively for the 2, 4 and 8 storey-building, the reference parameters adopted for the re-analysis of the pushover response curve, as well as the derived values of $q_{\Omega}.q_{\mu}.q_{\xi}$ leading to the assessment of the q-factor for each building.

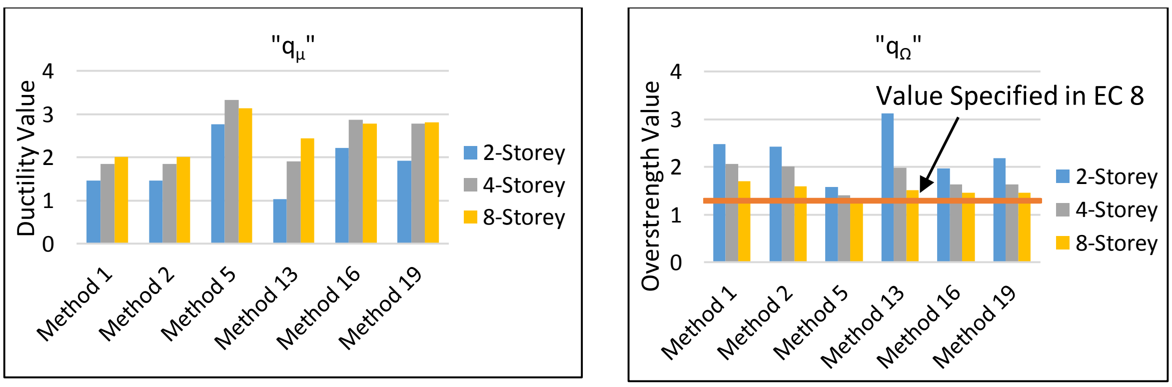
Figure 4-3 represents the ductility q_μ and overstrength q_Ω factors for 2, 4 and 8 storey-building obtained adopting the "short listed" methods. Results show that the ductility factor (Figure 4-3-a), in the majority of the cases, increases with the number of stories. Whereas, it is the opposite for the overstrength factor (Figure 4-3-b). Method 5 allows for all the three case-study buildings a definition of q_Ω in good agreement with the value specified in EC8 (q_Ω =1.3), but results in larger values of q_μ when compared to the other "shortlisted" methods. Method 13 seems to overestimate q_Ω for 2 storey building (q_Ω = 3.1).

Table 4-3 Behaviour Factor Calculation for 2-Storey Building

	2-Storey						
	Method 1	Method 2	Method 5	Method 13	Method 16	Method 19	
dm (mm)	193.2	193.2	193.2	193.2	193.2	259.2	
dy (mm)	131.9	131.9	69.9	188.2	87.2	135.1	
d1(mm)	40.0	35.1	40.0	35.1	40.0	40.0	
Fm (kN)	1335.9	1335.9	1335.9	1335.9	1335.9	1335.9	
Fy (kN)	1335.9	1335.9	1074.2	1716.0	1335.9	1335.9	
F1(kN)	675.7	549.5	675.7	549.5	675.7	675.7	**Average**
μ	1.5	1.5	2.8	1.0	2.2	1.9	1.81
Ω	2.5	2.4	1.6	3.1	2.0	2.2	2.30
q	3.6	3.6	4.4	3.2	4.4	4.2	3.90

Table 4-4 Behaviour Factor Calculation for 4-Storey Building

	4-Storey						
	Method 1	Method 2	Method 5	Method 13	Method 16	Method 19	
dm (mm)	412.4	412.4	412.4	412.4	412.4	653.5	
dy (mm)	222.7	222.7	123.9	215.5	143.9	234.5	
d1(mm)	80.2	69.4	80.2	69.4	80.2	80.2	
Fm (kN)	1037.5	1037.5	1037.5	1037.5	1037.5	1037.5	
Fy (kN)	1037.5	1037.5	891.3	1025.3	1037.5	1037.5	
F1(kN)	632.7	515.6	632.7	515.6	632.7	632.7	**Average**
μ	1.9	1.9	3.3	1.9	2.9	2.8	2.43
Ω	2.1	2.0	1.4	2.0	1.6	1.6	1.79
q	3.8	3.7	4.7	3.8	4.7	4.6	4.22



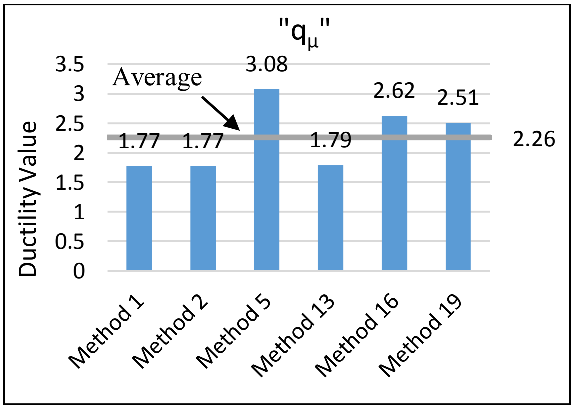
a) b)

Figure 4-3 a) Ductility Factor b) Overstrength Factor for 2, 4 and 8 Storey-building Based on Different Possible Methods

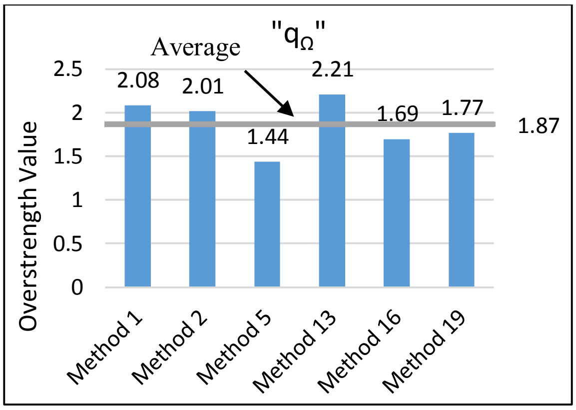
Table 4-5 Behaviour Factor Calculation for 8-Storey Building

	8-Storey						
	Method 1	Method 2	Method 5	Method 13	Method 16	Method 19	
dm (mm)	660.5	660.5	660.5	660.5	660.5	953.0	
dy (mm)	329.1	329.1	210.2	271.0	236.8	339.1	
d1(mm)	162.0	140.9	162.0	140.9	162.0	162.0	
Fm (kN)	870.7	870.7	870.7	870.7	870.7	870.7	
Fy (kN)	870.7	870.7	777.5	822.6	870.7	870.7	
F1(kN)	593.5	544.1	593.5	544.1	593.5	593.5	**Average**
μ	2.0	2.0	3.1	2.4	2.8	2.8	2.53
Ω	1.7	1.6	1.3	1.5	1.5	1.5	1.51
q	3.4	3.2	4.1	3.7	4.1	4.1	3.77

Figure 4-4 shows the average values of the ductility factor a) and of the overstrength factor b), obtained for the 2, 4 and 8 storey-building by application of each one of the "shortlisted" methods. In the same figures, the average of the values obtained by all the "shortlisted" methods is shown (q_μ = 2.26 and q_Ω = 1.87).



a)



b)

Figure 4-4 Average Calculated a) Ductility Factor b) Overstrength Factor for 2, 4 and 8 Storey-Building Based on Different Methods

It can be noticed that method 5 gives an estimate of q_μ higher than the average of all "shortlisted" methods, but an estimate for q_Ω lower than the average of all the same methods, while method 19 leads to estimates which are rather close to the average values obtained for all six methods for both q_μ and q_Ω.

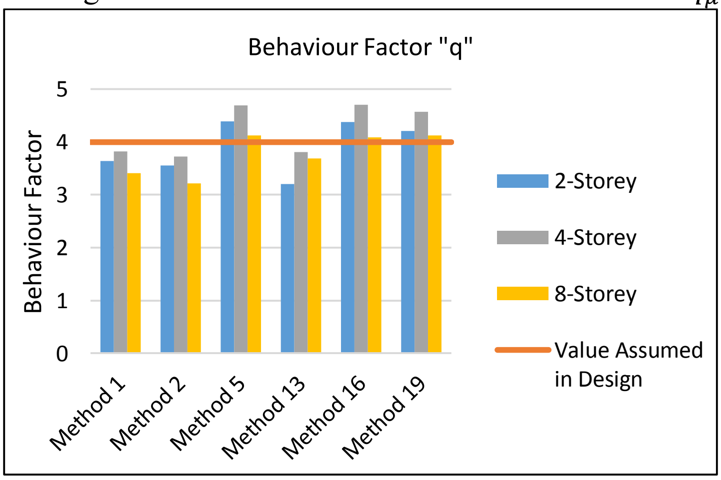


Figure 4-5 Behaviour Factor Calculated for 2, 4 and 8 Storey-Buildings Based on Different Methods

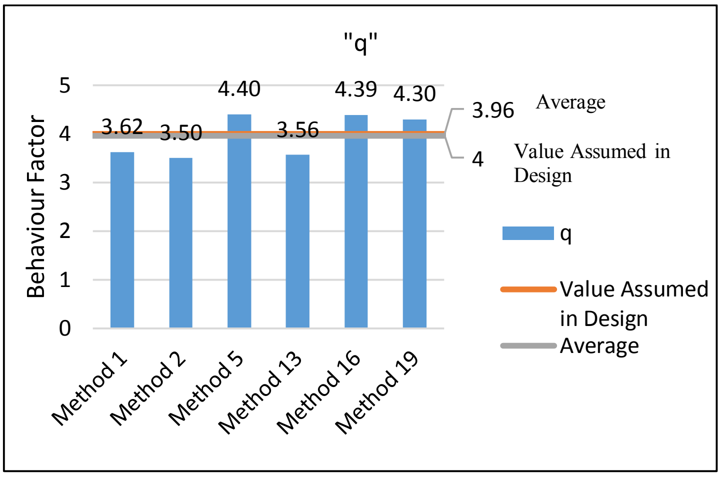


Figure 4-6 Average Behaviour Factor Obtained for 2, 4 and 8 Storey-Building Based on Different Methods

Figure 4-5 presents the behaviour factor estimated for 2, 4 and 8 storey-building by means of the six "shortlisted" methods, while Figure 4-6 shows the averages of the values estimated by each method for the three case-study buildings. In the same figure, the average value (q= 3.96) calculated for the six shortlisted methods is shown, very close to the assumed design value of the q-factor =4.0.

Behaviour factor obtained by method 19 is in most close agreement with the assumed design one, while methods 5 and 16 result in estimated values slightly higher than the assumed design one.

5 CONCLUSION

In this paper, first, some possible definitions of the reference parameters needed to define the behaviour factor by re-analysis of the pushover curve are presented and discussed. Then, the influence of different choices and combination of these reference parameters on the assessment of q-factor is shown for three case-study buildings, respectively with 2, 4 and 8 storeys. In order to do so, the overstrength "q_Ω" and the ductility "q_μ" factor were identified and the behaviour factor "q" calculated for the three case-study buildings, by performing for each building a nonlinear pushover analysis and re-analysing the results according to 90 different possible combinations of the different definitions of the reference parameters. Among the 90 methods, six were "shortlisted", being those giving results in closest agreement with the initially assumed design value of the q-factor (q=4.0). As a preliminary result, it was possible to identify a combination of definitions of reference parameters (Method 19) leading to an estimate of the q-factor in close agreement with the design one.

The same methodology will be applied to some other types of structures in the framework of European project (INNOSEIS). Within the same project, nonlinear Incremental Dynamic Analysis will be performed by Vamvatsikos at NTUA [30] on the same case study buildings. Such type of analyses, more sophisticated and time consuming, will lead to a more precise assessment of the q-factor than that achievable by means of re-analysis of the pushover curve. However, because of its sophistication, such an approach is not suitable for everyday application in the engineering practice, on the contrary of the one based on re-analysis of the pushover response curve.

A comparison between the q-factor values obtained by means of the two procedures (the one based on re-analysis of the pushover response curve, and the one based on non-linear IDA) will allow to identify which of the 90 methods introduced in this paper will actually lead to an estimate of the q-factor in closest agreement with the IDA analyses, eventually confirming (or not) the suitability of Method 19, identified so far as the "best one".

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