EXPERIMENTAL EVALUATION OF q FACTOR

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Abstract

Actual seismic design is based on the reduction of the seismic forces obtained from linear analysis, in order to take into account the non-linear response of the structure. The reduction is obtained by multiplying the base shear force with the behavior factor q. Conventional definition of q factor has no direct relation with the internal forces in the structure. q factor is sometimes too conservative, other times too optimistic. European seismic code EN 1998-1 gives just a generic estimation of the q factor, generally related to the typology of the structure and minimum ductility requirements for members and connections. There is always available the dynamic nonlinear analysis, but it operates with sophisticated models that need for calibration. If the problem is new and no benchmark models are available, the reference tests are the only solution. In order to calibrate the q factor, a test-based methodology is proposed. In the first step, the dissipation capacity of the structure is investigated experimentally, and q factor is evaluated at the level of a relevant subassembly. The modeling parameters and acceptance criteria are then defined and calibrated, based on the results of the experimental tests. In the third step, real structural configurations are proposed and tested numerically to confirm the q factor.

1. INTRODUCTION

To design dissipative structure, according to capacity design approach, reduction factors are widely used in design codes to reduce the elastic earthquake spectrum. Structural design stresses considering the earthquake induced forces are lower than those corresponding to the elastic response and are derived from the observation that, most structures are able to survive a major earthquake due to dissipation of energy by plastic excursions and overstrength. Currently, for a given structure, a single reduction factor is used. However, distinction and quantification of different components of the force reduction factor are useful for a better understanding of the seismic response of structures. The simplest and most understandable meaning of what the reduction or behaviour factor can be expressed using the SDOF equivalent model of the [1]. In

*Fig. **1***, the relationship between the base shear force, F and top displacement, D for a SDOF model, is shown.

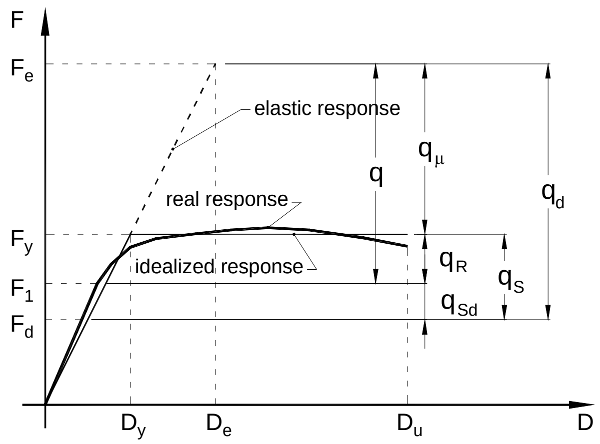


Fig. 1 Definition of force reduction factors [1]

If a bi-linear idealisation of the real response, the structural ductility is defined as:
$$\mu = D_u / D_y \quad (1)$$
where D_u is the ultimate top displacement and D_y is the yield top displacement. Other terms used in that figure are: F_e – elastic base shear; F_y – yield base shear; F_1 – base shear at the first plastic hinge; F_d – design base shear.
The following definitions are used:

- ductility factor : $q_\mu = F_e / F_y$ (2)
- overstrength factor : $q_S = F_y / F_d$ (3)
- redundancy factor : $q_R = V_y / V_1$ (4)
- design overstrength factor : $q_{Sd} = F_1 / F_d$ (5)

to obtain at the end the *total reduction factor:*
$$q_d = q_\mu \cdot q_S = q_\mu \cdot q_{Sd} \cdot q_R \quad (6)$$
The redundancy factor q_R used here in represents the plastic redistribution capacity of the structure (α_u/α_1 ratio in Clause 6 of EN 1998-1).
However, the non-linear dynamic analyses simulating the frame's response, emphasized that, such a *conventional* definition of q-factor – and there are many such definitions in literature - which has no direct relation with the internal forces and stresses in the structures, does not always succeed to predict properly the response of the structure. Sometimes q factor is too conservative, while other times, too optimistic!
In fact, the evaluation of behaviour q factor is a complex problem due to several parameters, among which the following ones can be listed [2]:

- the partial character of the energy dissipation in the structure when a local storey mechanisms occurs;
- the second order geometrical effects, so-called P-4 effects, necessary developed when large energy dissipation is required;
- the structural irregularity of the vertical configuration as well as the plan one;
- the occurrence of local buckling in the dissipative beams reducing their rotation capacity ;
- the buckling risk of columns subject to axial force and bending moment, whose drastic consequence requires to limit the energy dissipation (accepting only a few plastic hinges at the column ends), etc.

In the next section of this paper, a summary review of the different methods proposed in the literature to evaluate the q-factor is presented. These methods [3], [4] show a large scattering of the results, which could be partially explained by the lack of a coherent philosophy for the background of q factor definition, consistent with the given determination procedure. But, since the q-factor based design, still is practical enough, it continues to be applied in current design. However, when some new structural typologies appear, for which specifications are not yet available, the question that rises is what q-factor values could be used?

2. METHODS TO EVALUATE q-FACTOR

Present methods for the evaluation of the q factor can be classified in three main categories, as mentioned hereafter [4].

2.1 Methods based on the inelastic response of SDOF models

The basic method uses the push-over analysis and the ductility factor [5] (see

*Fig. **1***). The method can be also deduced from the dynamic analysis whose results are interpreted by means of an inelastic response spectrum in pseudo-acceleration [6], [7].
However, the q factor values obtained by these simplified methods cannot be easily transposed to real multi-degree of freedom systems (MDOF). Generally, it is required the structure to satisfy the conditions of "structural regularity" and "global plastic mechanism", which means, in case of a Moment Resisting Frame, for instance, to develop plastic hinges in all the beams during the dynamic response under a strong ground motion.
These methods cannot take into account for some local limitations, such as the attainment of rotation capacity at the ends of beams or columns, the risk of buckling in columns subject to high compression and flexural bending, etc.

2.2 Methods based on an energy approach

In multi-story buildings, it is very important to know how the energy input induced by an earthquake is distributed over the entire structure. The energy approach assumes that energy input attributable to the damage of an elastic-plastic system is the same as that producing damage in the relevant elastic system [8]. Damage distribution in each story depends primarily on the strength distribution along the height of the structure. The inelastic strain energy, abbsorbed in story "*i*", $W_{p,i}$, can be expressed in terms of corresponding yiel shear force, displacement and cumulated ductility ratio. The inelastic strain energy absorbed by the entire structure is the sum of the inelastic strain energy, dissipated in stories:

$$W_p = \sum W_{p,i} \qquad (7)$$

To avoid collapse of the i-story:

$$W_{u,i} > W_{p,i} \qquad (8)$$

while for entire structure, in principle:

$$W_u > W_p \qquad (9$$

The method proposed by Como and Lanni [9], is based on the evaluation on the one hand of the elastic strain energy W_e, stored in the state of first yielding, on the other hand of the total energy W_u, stored and dissipated by elastic-plastic deformations up to failure.
But in practice, this evaluation needs to use the approximate concept of equivalent horizontal forces, statically applied and distributed according to a combination of a

selected number of vibration modes. Another method [8] consists in verifying that under the design major seismic action, the capacity of the structure to dissipate energy by cumulated plastic deformations remains greater than the earthquake input energy into the structure; the last one can be obtained by $1/2MS_v^2$, where M is the total mass of the structure and S_v is the spectral elastic response in pseudo-velocity. But the evaluation of the dissipative capacity needs to accept several assumptions, in particular concerning the simplified expression of the hysteretic plastic work at each storey and an optimum distribution of this work between the different storeys (established empirically from parametric studies).
These methods seem to be more attractive because they do not require to satisfy conditions of structural regularity and energy global dissipation (eg. global plastic mechanism collapse).

2.3 Methods based on inelastic dynamic analyses of MDOF models

Using the non-linear dynamic analysis to provide the time-history response of a MDOF structure submitted to natural or artificial ground motions, these methods are really the most precise ones, even though they are laborious enough. The main difficulty arises from the interpretation of the dynamic results.
The well-known approach of Ballio and Setti [10], consists in performing a non-linear dynamic analysis and obtaining the maximum response of the structure during its time-history for different levels of ground motion; practically the considered ground acceleration *a(t)* is multiplied by a factor λ that is step-by-step increased. For each analysis with a fixed value λ, the response of a multi-storey structure is characterised by a significant displacement, generally the top story drift, δ (interstory drift can be also used). As long as the stresses induced in the structure are lower, at most equal to those initiating the plastic deformations, which in terms of accelerogram multiplier means $\lambda \leq \lambda_e$, the response remains elastic (segment OE in

Fig. ***2***); for higher values of λ, the real elastic-plastic displacements δ become generally smaller than the calculated ones assuming an ideal elastic behaviour, so that the curve (δ, λ) determined step-by-step has the position EIU shown in

Fig. ***2***. An interesting reference value λ_u* of the seismic action multiplier can be defined by the intersection point U* between the curve (δ, λ) and the linear elastic line extending the segment OE. At this stage, Ballio & Setti consider that the structure is almost reaching a state of global dynamic instability beyond which the plastic structural dissipation is not enough to offer resistance to important deflections.

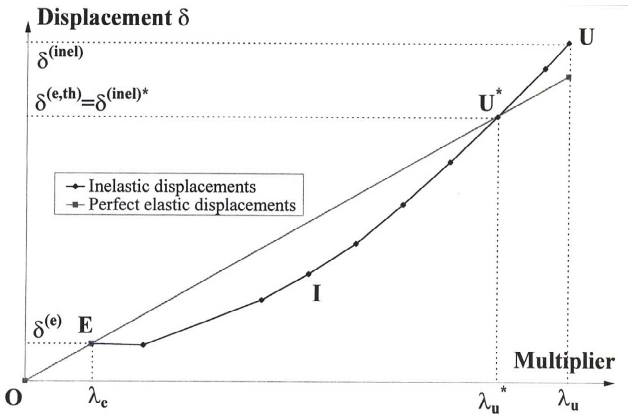


Fig. 2 Maximum displacement vs. accelerogram multiplier [4]

Usually, the behaviour factor q is evaluated as the ratio:

$$q_B = \lambda_u^* / \lambda_e \tag{10}$$

which means the ratio between the ground acceleration close to the structural collapse and that corresponding to the first yielding.
The main merit of their definition is to be consistent with the ductility factor definition, which from

*Fig. **1*** , expressed in terms of accelerogram multipliers is $\lambda_u^* / \lambda_e = \delta^{(inel)*} / \delta^{(e)}$.
However, there are some criticism in the literature regarding the suitability of the above formula [4]:

- it keeps up some confusion of the external ground acceleration with the inelastic spectral response in acceleration of the structure, whose consequence may be the more significant as the structure is governed by several degrees of freedom. Similarly, the relevant definition is not connected efficiently to the internal forces and moments in the structure, neither to the forces applied to the foundation.
- depending on the accelerogram type and the fundamental period T of the structure, there are cases where the definition of point U* is not clear (due to quasi-parallelism of the (δ, λ) curve and OE curves), also cases where there is no real intersection (due to reduced rotation capacity of some elements, beams and columns). There are also cases where the (δ, λ) curve is above the OE line as soon as $\lambda > \lambda_e$.

In order to improve the Ballio & Setti method and also to introduce a more general definition of *q*, Sedlacek & Kuck [11] proposed to change the definition of the straight line in

*Fig. **2*** with another straight line defined by the equation:

$$\delta = k \frac{\delta^{(e)}}{\lambda_e} \lambda \tag{11}$$

where the k factor may have different conventional values, for instance k=1.5 (this value seems to cover a more realistic domain of application).
However, both methods still raise many criticisms, and therefore they cannot be adopted as a general method in the seismic design codes. Among them, the most used is the one of Ballio & Setti.

2.4 Base shear force approach

Aribert & Grecea [2] have proposed a new method for the evaluation of the q factor, which is based on the ratio between the elastic theoretical base shear force $V^{(e,th)}$ corresponding to an elastic theoretical behaviour at the ultimate value λ_u of the multiplier and the real inelastic base shear force $V^{(inel)}$.

$$q = \frac{V^{(e,th)}}{V^{(inel)}} \tag{12}$$

or, more exactly:

$$q = \frac{\left(V^{(e)} / \lambda_e\right)}{\left(V^{(inel)} / \lambda_u\right)} \tag{13}$$

where:
- λ_e is the accelerogram multiplier at first yielding stage and λ_u is the one corresponding to the failure criteria.
The main advantage of this definition is the more suitable evaluation of the internal forces and moments in the structure. The method allows the evaluation of the q factor as a function of the level of performance required for the structure [12]. Therefore, one may apply this method to evaluate the q factor when a performance based design is used, giving the possibility to implement the multiple performance design in the actual code methodology. A parametric study on three steel moment resisting frames (MRF) was performed (**Error! Reference source not found.**).

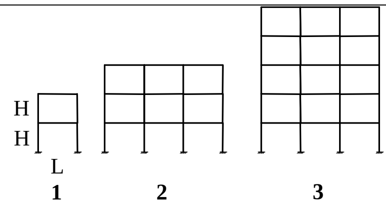
Frame type				
Frame	L(m)	H(m)	Beams	Columns
1	5	3	IPE300	HEB180
2	4	4	IPE330	HEB240
3	4	3	IPE360	HEB280

Table 1. Geometric properties of the frames under consideration

Three performance levels were associated with corresponding levels of seismic intensity:
- Serviceability limit state SLS
- Damageability limit state DLS
- Ultimate limit state ULS.

Fig. 3 shows the values of the behaviour factor q for each frame. According to the definition of the q factor, the values accounts for the contribution of the ductility, only.

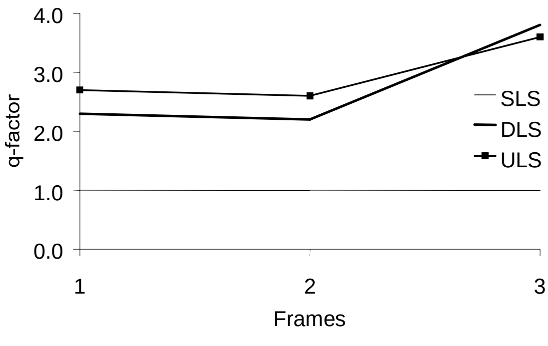


Fig. 3 q factors for frames

3. EXPERIMENTAL BASED APPROACH FOR CALIBRATION OF q-FACTOR

All methods presented in the previous section are based on numerical analysis, and therefore are sensitive to the definitions of the modelling parameters of the elements and also to the levels of damage that can be accepted in the elements. EN 1998-1 gives just a generic estimate of the q-factor, generally related to the typology of the structure (moment frames, braced frames, etc.) and minimum ductility requirements for elements and connections. Of course, there is always available the dynamic nonlinear analysis, but it operates with models, and they need for calibration. As already stated, if the problem is new, and no benchmark models are available, the reference tests are the only solution. Therefore, in order to have a better estimation of the behaviour factor q, the following procedure can be applied:

- first step: the dissipation capacity of the structure is investigated experimentally, and the q factor at the level of the subassembly is evaluated.
- second step: the modeling parameters and acceptance criteria are then defined, based on the results of the experimental tests.
- third step: real structural configurations are proposed and tested numerically for confirming the behaviour factors.

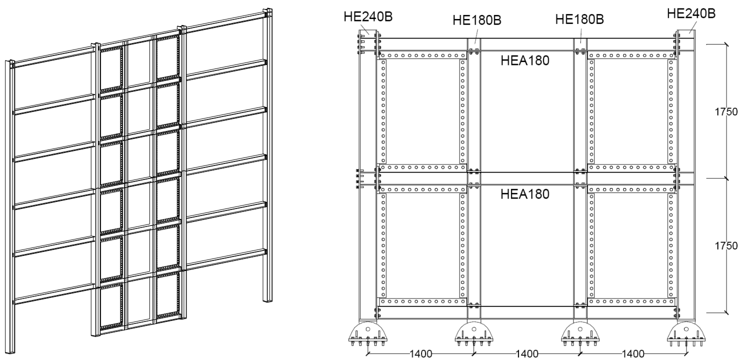
In the next sections, this methodology is applied to three different types of structures, for which EN 1998-1 gives no indications regarding the selection of q factor.

3.1 Dual steel frames of dissipative shear walls

First structural system that was investigated is relatively new in Europe, but with many applications outside Europe, mostly in USA and Japan. The system is considered ductile and, where is available in the seismic provisions (eg. AISC 2005), has seismic reduction factors (equivalent to q factor) comparable to moment resisting frames. In order to cope with the lack of such information in EN 1998-1, a research program has been developed at the Politehnica University of Timisoara, Laboratory of Steel Structures [13], [14]. An objective was the evaluation of q factor for such structures. Four specimens were tested experimentally, then, a numerical program was performed. The steel plate shear wall specimens were extracted from a six story frame structure (

Fig. *4*a). The two actuators used for the tests have 360mm stroke and 1000 kN and 500 kN capacity, respectively. Due to the stroke limitation, the specimens were half-scaled. The infill plates had thickness of 2mm and 3mm, respectively. The frames measured 3500 mm high and 4200 mm wide between member centerlines (

Fig. *4*b). Slender shear walls have been used, with ratio L/t_w amounting 595 for 2 mm panels and 397 for 3 mm panels, while the aspect ratio L/h was 0.8. Two types of beam-to-column connections were used. First one was a flush end plate bolted connection, and the other one was extended end plate bolted connection. Based on EN 1993-1-8 classification, the flush end plate connection is semi-rigid and weak partial strength (M_j, R_d=$0.4M_{b,Rd}$) (further refereed as semi-rigid) and the extended end plate connection is rigid and with a capacity almost equal to that of the connected beam ($M_{j,Rd} = 0.9M_{b,Rd}$) (further refereed as rigid).



(a) (b)

Fig. 4 a) Six story frame structure; b) half-scale tested frame

Specimens have been tested cyclically using ECCS procedure. Testing set-up is shown in

Fig.5. In

Fig.6 is shown the deformed shape after the test.

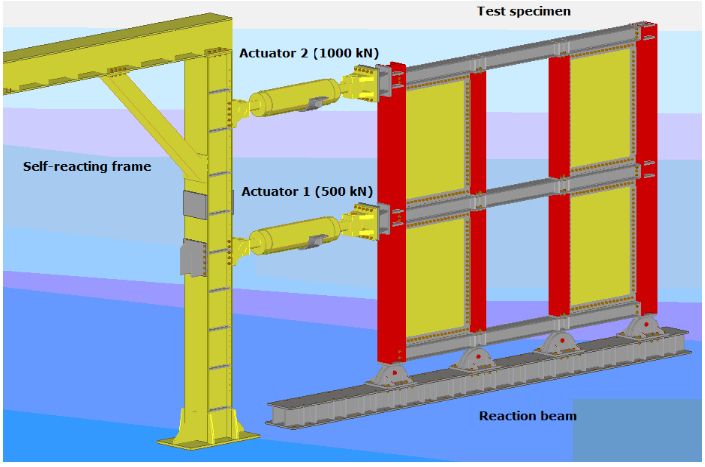


Fig.5 Test set-up

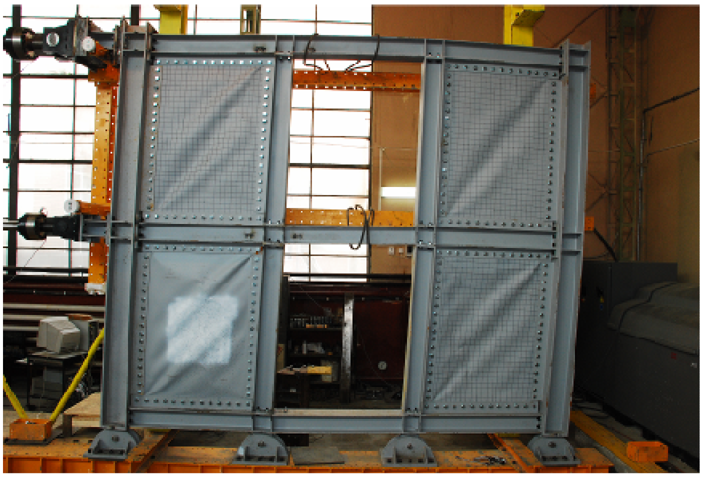


Fig.6 Deformed shape after the test

Fig.7 plots the hysteresis of rigid and semi-rigid specimens. As the initial stiffness is mainly attributed to the panels, differences between rigid and semi-rigid specimens in terms of initial stiffness are not as important as differences in terms of strength.

The behaviour factor q can be expressed as a product of the ductility factor, q_{μ}, and the overstrength factor, q_s. The overstrength may vary significantly and is affected by the contribution of gravity loads, material overstrength, structural redundancy, etc. However, since for such a system the major component of the behavior factor q is the ductility factor, q_{μ}, it is more important to focus on the ductility component, which can be taken equal to the displacement ductility factor μ. The ductility reduction factor q_{μ} is therefore defined as the ratio of the ultimate displacement D_u and the yield displacement D_y, where D_u corresponds to a reduction of the load carrying capacity of 10% compared to the maximum one. Yielding displacement D_y has no standardized or at least harmonized definition for steel plate shear wall systems. Therefore, the evaluation of the yielding displacement, D_y, was based on the ECCS methodology. The series of values in **Error! Reference source not found.** for q factor correspond to ductility factor, because the tested subassemblies that have no static redundancy and, in this case, the design overstrength is not active.

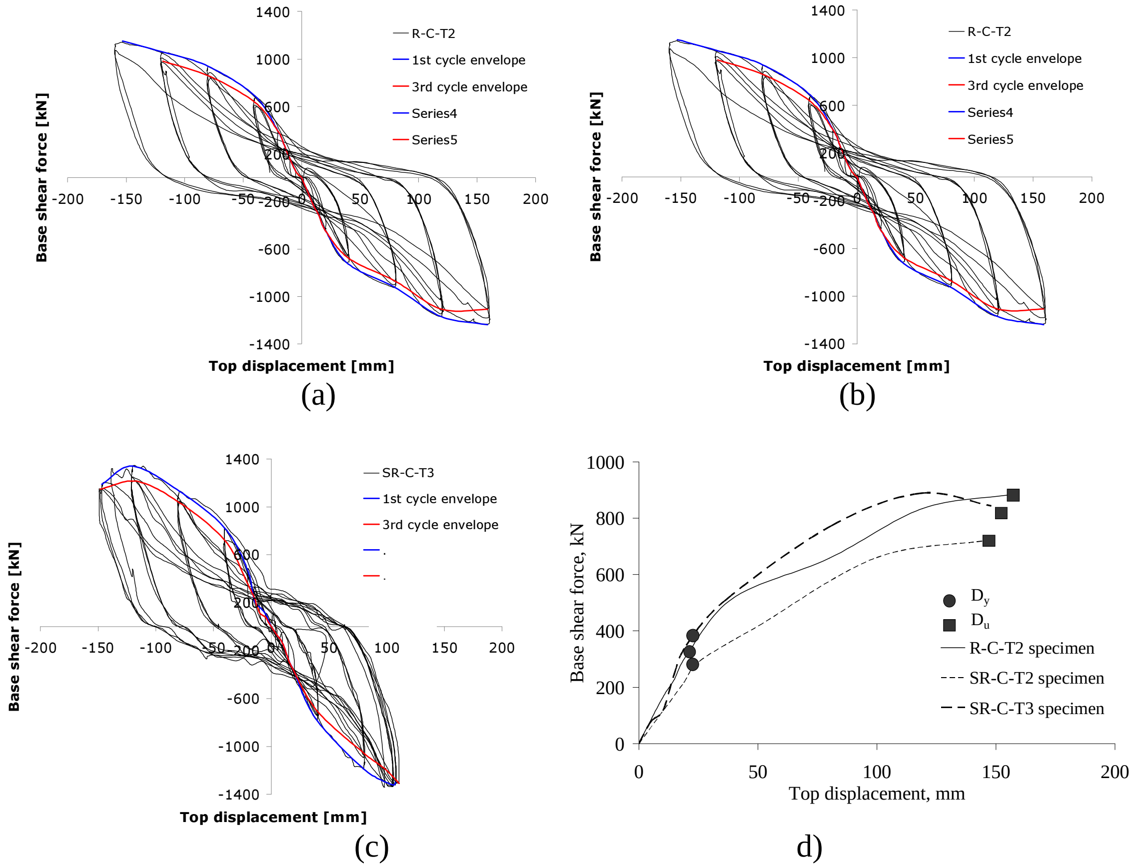


Fig.7 Results of the cyclic tests: a) hysteresis curve for R-C-T2 specimen; b) hysteresis curve for SR-C-T2 specimen; c) hysteresis curve for SR-C-T3 specimen; d) envelopes of hysteresis curves, 1st and 3rd cycle

Structure	D_y	D_u	q_u
R-C-T2	38	153	4.0
SR-C-T2	33	163	4.9
SR-C-T3	40	147	3.7
Average value			4.2

Table 2. q factor values

In the second step, the FEM has been calibrated, together with relevant acceptability criteria.

Fig. 8 shows the comparison of the FEM with the test results.

3.2 In order to extend the investigation to real structural systems, in the thirst step numerical studies were conducted using a nonlinear dynamic procedure. The geometry of the structures is presented in

Fig.9. For the preliminary design, a q factor of 6 resulting from the experimental program was used. A 4 kN/m^2 dead load on the typical floor and 3.5kN/m^2 for the roof were

considered, while the live load amounts 2.0kN/m^2. The buildings are located in a high seismic area (i.e. the Romanian capital, Bucharest), which is characterized by a design peak ground acceleration 0.24g for a returning period of 100 years, and soft soil conditions, with T_C=1.6sec.

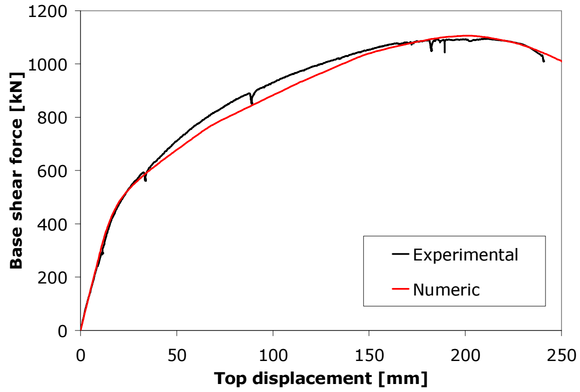
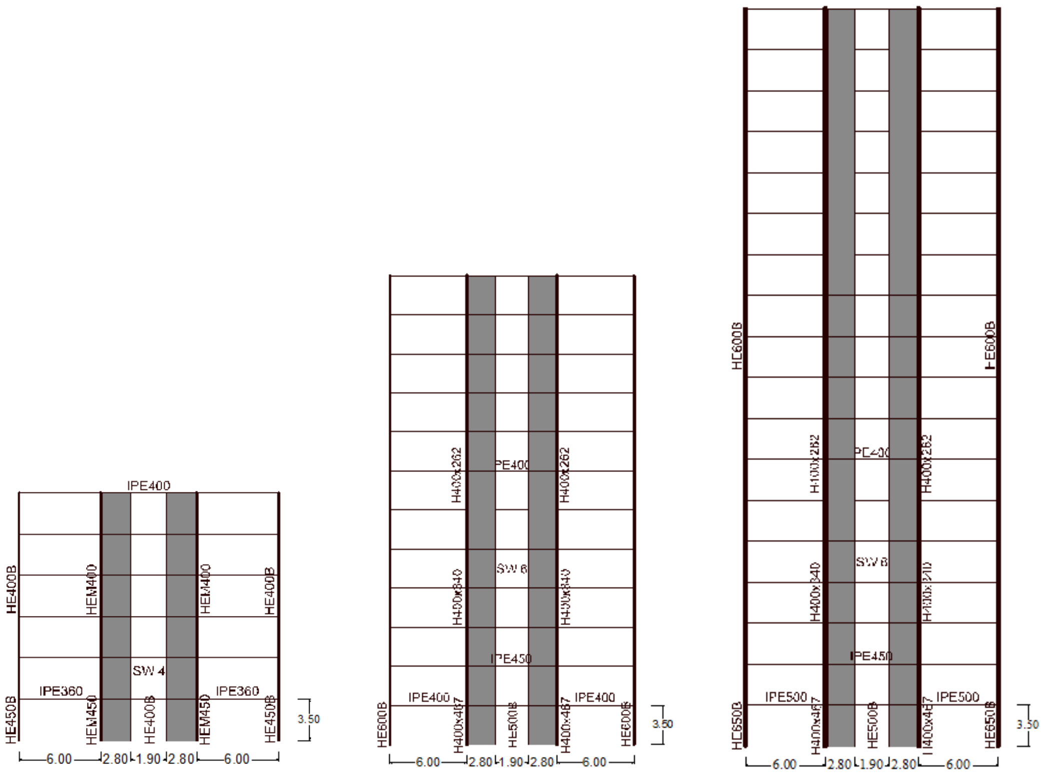


Fig. 8 FEM model vs. test results



6 story 12 story 18 story

Fig.9 Geometry and members of the structural systems considered in the study

A set of six ground motions was used. Spectral characteristics of the ground motions were modified by scaling Fourier amplitudes to match the target elastic spectrum from P100-1, see

Fig. 10. This results in a group of semi-artificial records representative to the seismic source affecting the building site and soft soil conditions in Bucharest. The procedure was

based on the SIMQKE-1 program [15].
Fig. 11 shows the maximum interstory drift ratio vs. spectral acceleration S_a for all structures and records. **Error! Reference source not found.** shows the values of the behaviour factor q, defined as the ratio between the acceleration leading to collapse and the acceleration leading to first yielding. One observes, if the values of q factor from **Error! Reference source not found.** would be multiplied with redundancy factor (1.3) and the design overstrength (say 1.1), the results will be closed to the ones presented in **Error! Reference source not found.**.

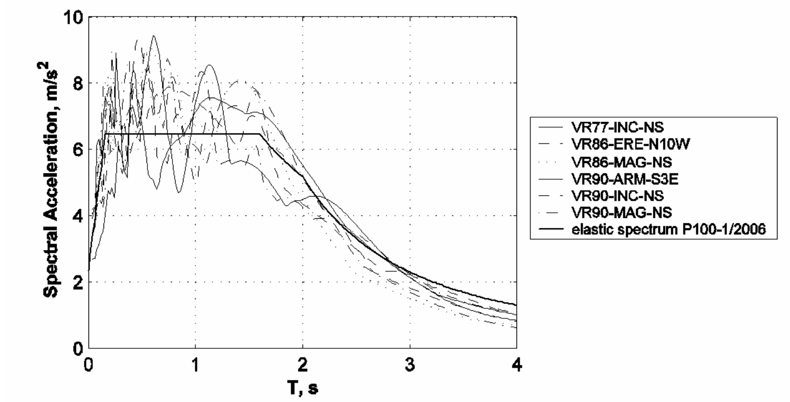


Fig. 10 Elastic response spectra of semi artificial records and P100-1/2006 elastic spectrum

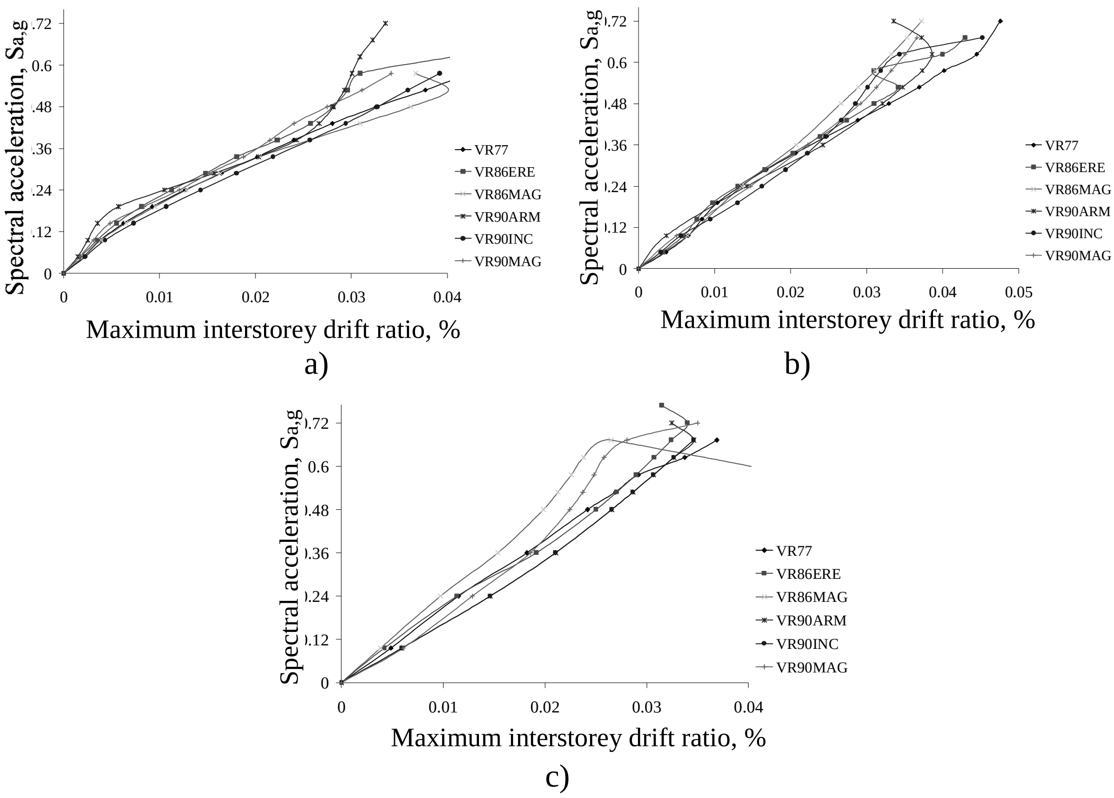


Fig. 11 IDA curves: maximum interstory drift ratio vs. spectral acceleration $S_a(g)$ for all records for: a) 6 story structure; b) 12 story structure; c) 18 story structure

Earthquake	No of story	Acceleration		q	No of story	Acceleration		q	No of story	Acceleration		q
		a_{gy}	a_{gu}			a_{gy}	a_{gu}			a_{gy}	a_{gu}	
VR77INC	6	0.10	0.58	6.0	12	0.10	0.53	5.5	18	0.10	0.48	5.0

VR86ERE		0.12	0.62	5.2		0.10	0.58	6.0		0.10	0.77	8.0
VR86MAG		0.10	0.58	6.0		0.10	0.72	7.5		0.13	0.67	5.1
VR90ARM		0.12	0.72	6.0		0.10	0.62	6.5		0.10	0.72	7.5
VR90INC		0.10	0.58	6.0		0.10	0.53	5.5		0.10	0.53	5.5
VR90MAG		0.12	0.62	5.2		0.10	0.62	6.5		0.10	0.67	7.0
AVERAGE				5.7				6.3				6.3

Table 3. q factors for structures with rigid connection

3.3 MR reinforced concrete frames strengthened of steel buckling restrained braces

The system with braces prevented from buckling (BRB) is also relatively new, and there are no provisions in European seismic codes regarding the selection of the q factor. In the North-American code AISC (2005), there are specific provisions, in that case, the BRB systems are considered in terms of ductility similar with Moment Resisting Frames and ccentric Braced Frames. But for the case of Reinforced Concrete Frames strengthened with BRBs, practically there are no design recommendations, from this point of view. The system gained much interest in the recent years and there are many applications in the seismic areas, because the system has a good ductility and can be used both for new structures and for rehabilitation of existing ones.
An extended experimental and numerical study that aimed at evaluating the seismic vulnerability of reinforced concrete frame buildings designed for gravity loads was carried-out at the "Politehnica" University of Timisoara [16], [17]. In the first part, a reinforced concrete frames retrofitted with BRB was extracted from a multi-story concrete frame building and then tested experimentally to evaluate its ductility and consequently the q factor. In the second part, the original multi-story structure was investigated numerically in order to check the results of the experimental program.
A modified ECCS loading protocol was applied in the cyclic tests. This modified procedure is characterized by a single loading at $D_y/4$, $2D_y/4$, $3D_y/4$ and D_y, followed by three repetitions of the cycles increased by $0.5D_y$ ($1.5D_y$, $2D_y$).

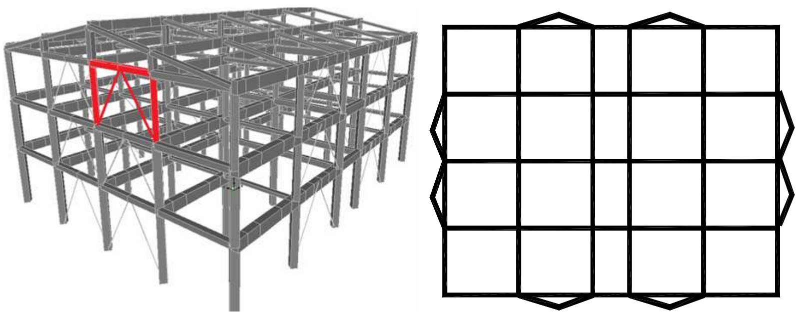


Fig. 12. RC building model, with location of brace system

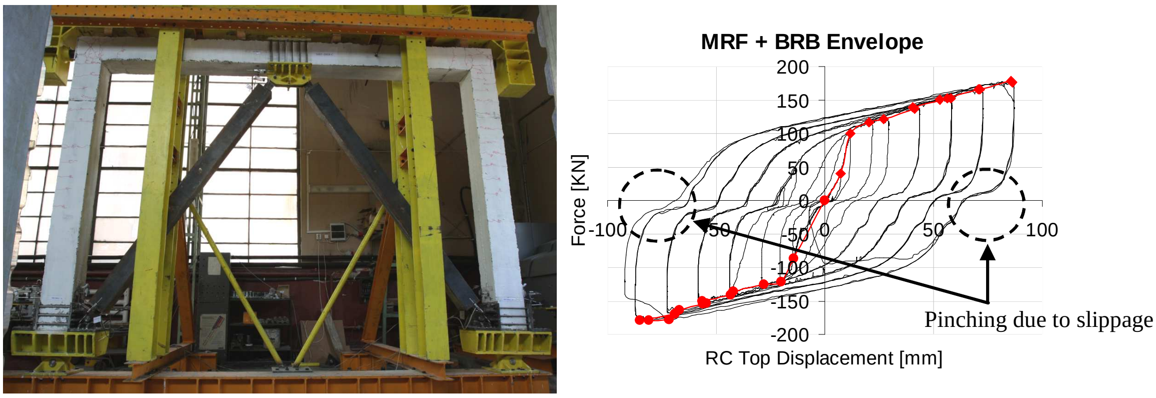


Fig. 13. Test set-up for the reinforced concrete frame strengthened with BRB (left) and envelope of the cyclic test (right)

For the evaluation of the behaviour factor, only the contribution of the ductility was considered. The value of the yield displacement D_y is 16.3 mm and the ultimate displacement D_u is 71 mm, which leads to a behavior factor q equal to 4.3.

In the second step, the inelastic behavior model of BRB considered the concentrated tri-linear plasticity curve with strain hardening and strength degradation of 0.8 from maximum capacity and was calibrated based on experimental tests summarized above. Acceptance criteria for BRB were also based on the results of the experimental tests.

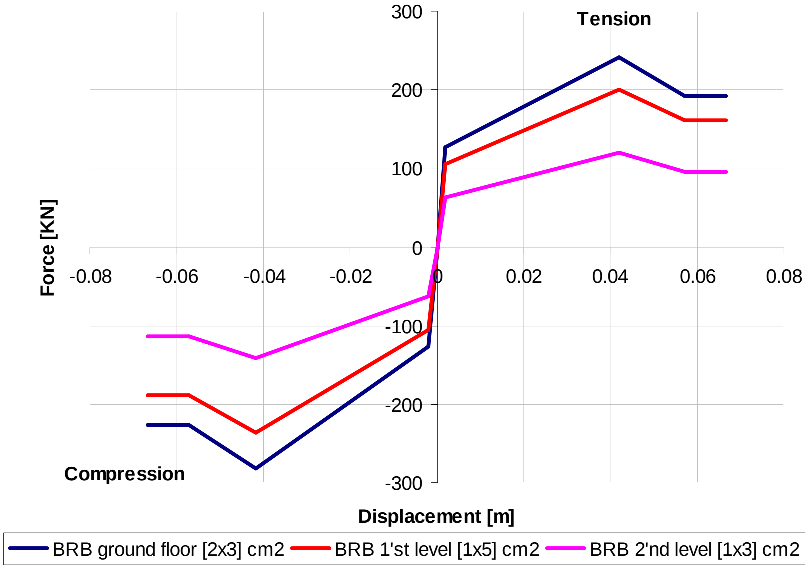


Fig. 14. BRB tri-linear model

In the third step, in order to extend the investigation on the behavior factor q, a nonlinear dynamic analysis was employed on the structure strengthened with BRB (see
Fig. 12). Seven artificial accelerograms were generated, whose response spectra are compatible with the design spectra (PGA = 0.23g and Tc=0.5s) (
Fig. 15). Each accelerogram was scaled up to the attainment of collapse (
Fig. 16). The q factor was calculated as the ratio between the acceleration multiplier for collapse and the acceleration multiplier for first yielding.

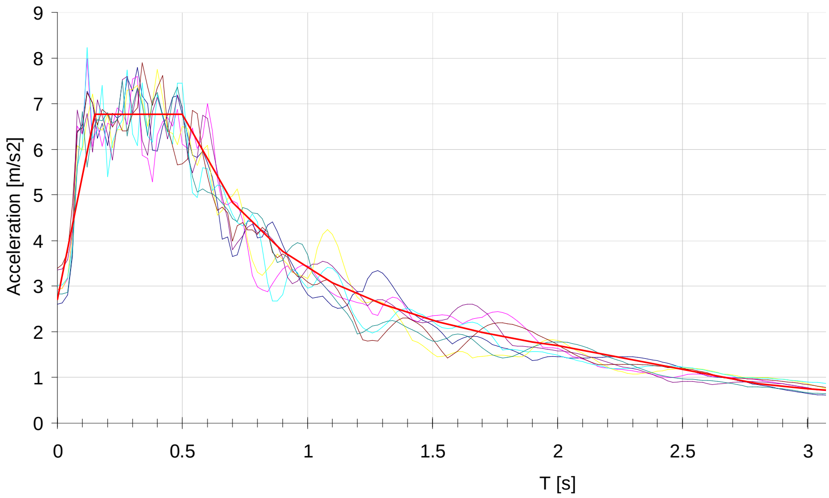


Fig. 15. Elastic response spectra of artificial accelerograms vs. elastic design spectrum, 5% damping

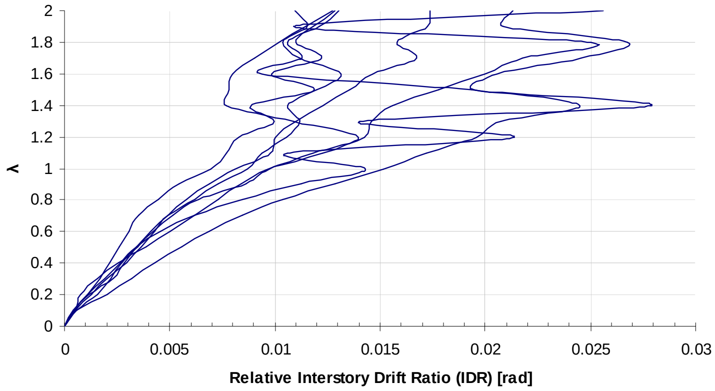


Fig. 16 Relative interstory drift vs. seismic multiplication factor λ in X direction

Values of q factors presented in **Error! Reference source not found.** represent the contribution of ductility, only. It is expected that these values can be larger if the overstrength (eg. design and redundancy) of the system is large. However, the value of this overstrength can be very different from one structure to another, and therefore the values of q factor may be different.

Accelerograms	X direction			Y direction		
	λ_e	λ_u	q	λ_e	λ_u	q
1	0.3	1.3	4.3	0.4	1.6	4.0
2	0.3	1.2	4.0	0.4	1.6	4.0
3	0.3	1.3	4.3	0.4	1.6	4.0
4	0.2	1.0	5.0	0.3	1.3	4.3
5	0.3	1.2	4.0	0.47	1.5	3.8
6	0.2	1.1	5.5	0.4	1.4	3.5
7	0.3	0.9	3.0	0.4	1.4	3.5
	Average on X		4.3	Average on Y		3.9

Table 4. q factor values from time-history analysis

3.4 Light gauge wall stud framing of corrugated sheathing

Steel-framed houses are usually built of light thin-walled load bearing structures having different solutions for interior and exterior cladding. This technology is popular and accounts for an important and increasing market share in the US, Japan, Australia and Europe. The same method is used for buildings, of small dimensions, of other purposes (offices, schools, manufacturing premises, etc.), that are referred to as small industrial buildings (SIB). Even if widely used in practice, the behaviour of shear walls subjected to earthquake is not fully understood and in recent years an important effort has been made to clarify certain aspects related to shear wall strength, stiffness and ductility, as main parameters governing seismic behaviour. A large experimental and numerical program has been undertaken at the "Politehnica" University of Timisoara, in order to investigate the shear behaviour of some of the most popular wall-panel typologies characteristics in an attempt to provide evidence on the possible values of behaviour factors q [18], [19]. The program was based on six series of full-scale wall tests with different cladding arrangements based on common practical solutions in housing and SIB (**Error! Reference source not found.**). Cyclic testing methodology followed ECCS Recommendation, consisting of cycled of ¼ Δ_{el}, ½ Δ_{el}, ¾ Δ_{el}, 1 Δ_{el}, 2 Δ_{el}, 2 Δ_{el}, 2 Δ_{el}, 4 Δ_{el}, 4 Δ_{el}, 4 Δ_{el}, 6 Δ_{el}, 6 Δ_{el}, 6 Δ_{el},…, until failure or a significant decrease of load bearing capacity. Testing set-up is shown in
Fig.17. In

Fig.18 are displayed one panel with corrugated sheet (without opening) and one panel with door opening.

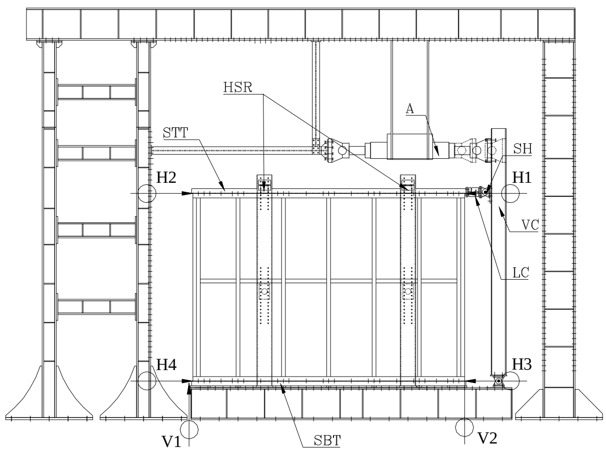


Fig.17 Test set-up

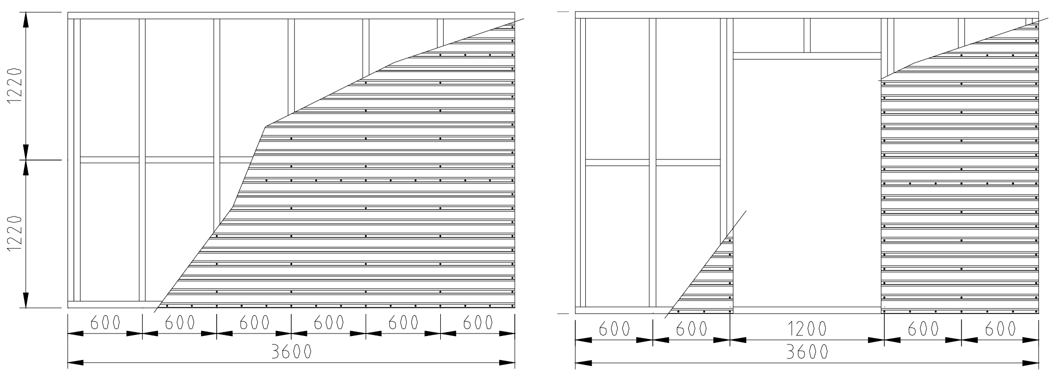


Fig.18 Panel with corrugated sheet (left) and panel with door opening (right)

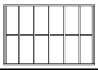
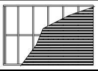
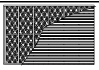
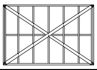
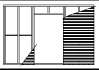
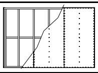
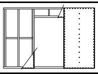
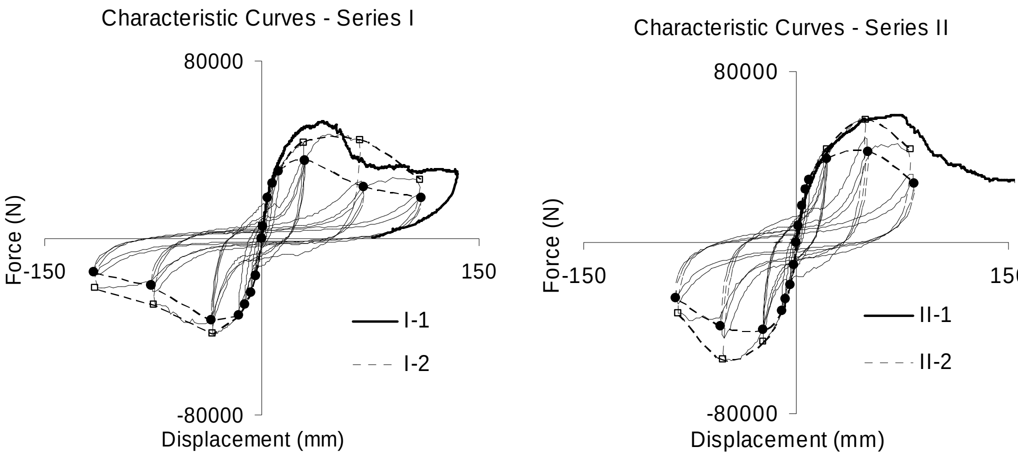
Series	Opening	Bracing	Exterior Cladding	Interior Cladding	Testing Method	Loading Velocity	No. Test
O		-	-	-	Monotonic	1	1
I		-	Corrugated Sheet LTP20/0.5	-	Monotonic	1	1
					Cyclic	6 – 3	2
II		-	Corrugated Sheet LTP20/0.5	Gypsum Board	Monotonic	1	1
					Cyclic	6 – 3	2
III		-	-	-	Monotonic	1	1
					Cyclic	3	1
IV		Door	Corrugated Sheet LTP20/0.5	-	Monotonic	1	1
					Cyclic	6 - 3	2
OSB I		-	10 mm OSB	-	Monotonic	1	1
					Cyclic	3	1
OSB II		Door	10 mm OSB	-	Monotonic	1	1
					Cyclic	3	1
Total Number of Specimens							15

Table 5. Description of wall specimens

Fig.19 shows the load versus lateral displacement curves. Initial stiffness was determined as secant stiffness to the load level of 0.4 F_{max}. The evaluation of the conventional yield limit was based on ECCS Recommendation, at the intersection point of the elastic line (K_o) to a line of $0.1K_o$ rigidity, tangent to the experimental curve. Based on this conventional elastic limit, the ultimate point (F_u, D_u) results at the intersection of the horizontal yield line to the experimental curve in the downloading branch. **Error! Reference source not found.** presents the results of the tests. Values of ultimate displacements and ultimate force are derived based on the 3rd envelope curve (stabilized envelope), positive and negative.



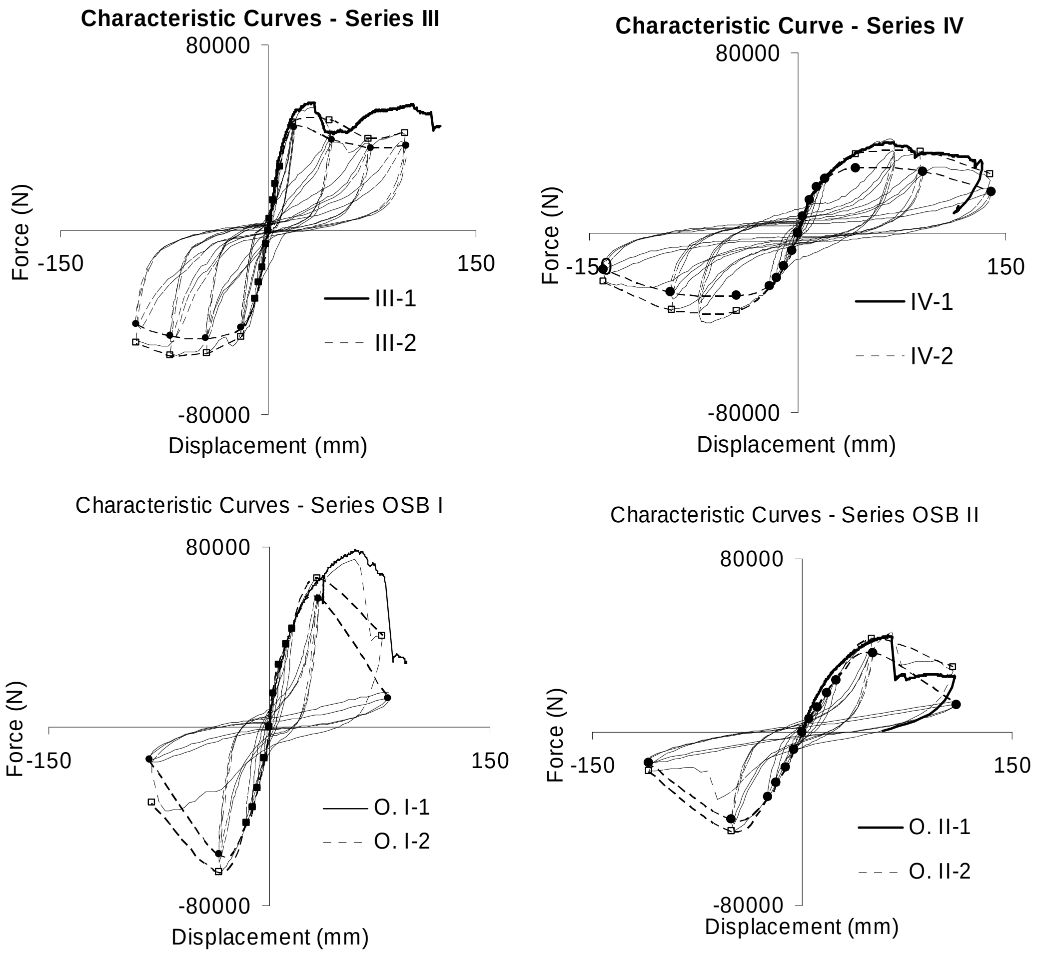


Fig.19 Experimental curves

Series – Curve	F_{el} (N)	D_{curv} (mm)	F_{max} (N)	F_u (N)	q_μ
I-2	17355.2	12.90	44061.0	31333.3	6.53
I-3	17362.8	13.57	44077.2	30666.7	5.47
II-2	22654.0	12.77	57796.0	35000.0	7.48
II-3	22161.9	13.35	56820.6	36444.4	6.09
III-2	19875.7	14.39	52682.9	41666. 7	3.00
IV-2	13962.6	32.54	34981.1	31444.4	5.45
IV-3	15626.1	24.89	40843.0	26444.4	5.07
OSB I-2	25426.1	19.84	64972.3	51666.7	3.12
OSB II-2	17717.1	36.17	46049. 7	35666.7	1.54

Table 6. Experimental results from stabilized 3rd envelope curve, positive and negative

In the second step, a tri-linear model was built starting from the proposal of Della Corte et al. [20], based on a Richard-Abbott type curve. The model has a very good capability in characterizing all aspects of the panel behaviour. The model was calibrated based on experimental results.

In the third step, in order to extend the results of the experimental program, the structures were tested numerically. For the purpose of earthquake analysis, five earthquake records have been selected. Normalized elastic spectra with a damping ratio of 5% form the critical have been compared to EN 1998 elastic spectra for A, B and C subsoil conditions (

Fig.20).

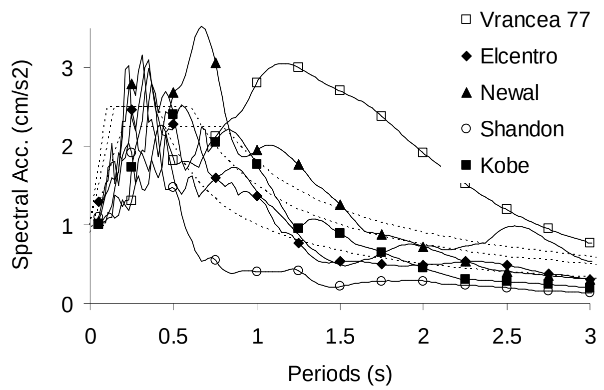


Fig.20 Elastic spectra of records, damping factor 5%

Fig.21 plots the seismic intensity vs. maximum story drift, for different inertial acting masses. Records were scaled up to 2.0g. Based on the displacement values, corresponding earthquake Intensity Measure levels (IM) have been identified for the different panel configurations and earthquake records. The three limit states correspond to the following states for the wall panel under consideration: D_{el} – elastic design limit of the panel up to which behaviour can be considered elastic and it is the conventional capacity to be used in design; D_{yield} – yield limit of the wall panel, where the panel lost its load bearing capacity, but it is still capable of deforming under constant load, D_{ult} – ultimate state, the panel is not capable of sustaining a constant load level, and its capacity is decreasing.

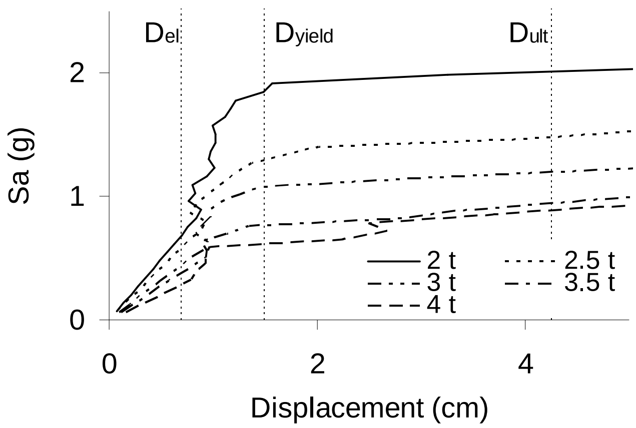


Fig.21 IDA curve example

Wall panel behaviour is characterized by important strength reserve over the accepted allowable design strength and it can be expected that this over-strength plays an important role in the post-elastic performance and consequently, in the value of q factor. Following this assumption, the two main contributors of the q factor, which are the reserve in strength (q_s) and the ductility (q_μ), have been accounted separately.

	Series I	Series II	Series IV	Series OSB I	Series OSB II

Inertial mass [in to]	2	2.5	3	3.5	4	2	2.5	3	3.5	4	2	2.5	3	3.5	4	2	2.5	3	3.5	4	2	2.5	3	3.5	4
Sa_{el} (g)	0.62	0.47	0.41	0.39	0.36	0.97	0.60	0.51	0.42	0.39	0.36	0.29	0.21	0.22	0.19	0.97	0.61	0.47	0.45	0.43	0.34	0.29	0.19	0.20	0.14
Sa_{yield} (g)	1.63	1.15	0.97	0.85	0.79	1.75	1.39	1.08	0.97	0.81	1.09	0.87	0.78	0.69	0.60	2.01	1.74	1.26	1.18	1.04	1.11	0.94	0.67	0.72	0.58
Sa_{ult} (g)	2.06	1.65	1.53	1.31	1.14	2.64	1.99	1.83	1.67	1.49	2.04	1.84	1.75	1.61	1.58	2.84	2.09	1.77	1.69	1.43	1.87	1.75	1.36	1.18	1.19
q_s	2.84	2.57	2.42	2.25	2.39	1.89	2.35	2.17	2.38	2.26	3.04	3.12	3.80	3.09	3.37	2.14	2.97	3.04	2.70	2.47	3.46	3.43	3.80	3.95	4.29
q_μ	1.25	1.49	1.59	1.53	1.45	1.51	1.44	1.74	1.72	1.83	1.92	2.12	2.40	2.54	2.82	1.42	1.21	1.44	1.43	1.37	1.73	1.85	2.02	1.65	2.15
$q = q_s \times q_\mu$	3.58	3.64	3.92	3.46	3.48	2.90	3.36	3.60	4.10	4.09	5.72	6.46	8.71	7.44	9.90	3.11	3.62	4.23	3.99	3.39	5.86	6.42	7.60	6.24	8.66

Average	q_s	q_μ	q	q_s	q_μ	q	q_s	q_μ	q	q_s	q_μ	q	q_s	q_μ	q
	2.50	*1.46*	*3.62*	*2.21*	*1.65*	*3.61*	*3.28*	*2.36*	*7.65*	*2.66*	*1.38*	*3.67*	*3.78*	*1.88*	*6.96*

Table 7. Performance parameters and q factor values

One leaves also the overstrength, as safety margins, and considers ductility only. One excludes the values to large, obtained for walls with door opening. It is for sure, from ductility point of view, these structures can be considered *low dissipative*, according to EN1998-1, which values $q < 1.5 – 2.0$.

4. CONCLUDING REMARKS

Full scale or scaled testing can be used to calibrate q factors for seismic design of structures for which no relevant recommendations are met in design codes. The tests are in fact used to obtain reference results for structural sub-assemblies or structural macro-components, and to offer benchmarks for calibration of numerical models. The calibrated numerical models are afterwards used for evaluating the q factors for real design structures. Incremental dynamic analysis (IDA) is recommended on this purpose. The three applications presented in the paper, i.e. dual steel frames of dissipative shear walls, reinforced concrete frames strengthened with steel BRB and cold formed steel stud shear walls can be considered relevant for the proposed procedure.

5. REFERENCES

[1] Peter Fajfar: "General definition and basic relations", Moment resistant connections of steel frames in seismic areas. Design and reliability, Ed. F.M. Mazzolani, E&FN Spon, London, p. 565-578, 2000.

[2] Jean-Marie Aribert, Daniel Grecea: A new method to evaluate the q-factor from elastic-plastic dynamic analysis and its application to steel frames, Proc. of the Second International Conference STESSA'97, Kyoto, Ed. F.M. Mazzolani & H. Akyiama, p. 382-394.

[3] Vincenzo Piluso, Federico M. Mazzolani: “Theory and Design of Seismic Resistant Steel

Frames”, E&FN Spon, An imprint of Chapman & Hall, First Edition, 1996, ISBN 0-419-18760-X.
[4] Jean-Marie Aribert, Daniel Grecea: "The base shear force approach", Moment resistant connections of steel frames in seismic areas. Design and reliability, Ed. F.M. Mazzolani, E&FN Spon, London, p. 597-617, 2000.
[5] Cosenza E, De Luca A, Faella C, Mazzolani FM.: "Una proposta per la definizione del coefficiente di struttura nei Telai Metallici. Roma: 3 convegno Nazionale l’ingegneria sismica in Italia; 1987 [in Italian].
[6] Giuffre, Giannini: La risposta non lineare delle strutture in cemento armato.IN: Progetazione e particolari construttivi in zona sismica, ed. G,Grandori, pp.175-238, Rome:Edilstampa, 1982.
[7] Palazzo, Fraternali: "The use of collapse spectra in the seismic analysis. A proposal for a new formulation of the structural factor", 3rd AIDIS Congress, Roma, 1987.
[8] Hiroshi Akiyama: "Earthquake-resistant limit-state design for buildings", University of Tokyo Press, Technology & Engineering, 372 pages, 1985.
[9] M. Como, G. Lanni: "Aseismic toughness of structures", Meccanica 18 (1983), pp. 107–114, 1983.
[10] Giulio Ballio, Setti P.: "A method to compute the behaviour factor for construction in seismic zones", Costruzioni Metalliche 37 (3), p. 128–139, 1985.
[11] G Sedlacek, J. Kuck: "Determination of *q*-factors for Eurocode 8", Aechen den 31.8.1993, 1993.
[12] Daniel Grecea, Florea Dinu, Dan Dubina: "Performance criteria for MR steel frames in seismic zones", Journal of Constructional Steel Research 60 (2004) 739–749, 2004.
[13] Calin Neagu, Florea Dinu, Dan Dubina: "Design of steel frames of dissipative shear walls", SDSS’Rio 2010 Stability and ductility of steel structures, E. Batista, P. Vellasco, L. de Lima (Eds.), Rio de Janeiro, Brazil, September 8 - 10, 2010.
[14] Calin Neagu, Florea Dinu, Dan Dubina: Seismic performance of steel plate shear walls structures, Pollack Periodica Journal, Volume 6, Number 1, p. 47-58, April 2011.
[15] D. A. Gasparini, E. H. Vanmarcke, Simulated earthquake motions compatible with prescribed response spectra, Research Report R76-4. Massachusetts Institute of Technology, Department of Civil Engineering, Cambridge, Massachusetts, 1976.
[16] Dan Dubina, Sorin Bordea: Experimental investigation of non-seismic RC frames strengthened with BRB steel braces, Proc. of International Symposium “Steel Structures:Culture & Sustainability 2010”, 21-23 September 2010, Istanbul, Turkey.
[17] Dan Dubina, Sorin Bordea, Florea Dinu: Experimental and numerical investigation of non-seismic reinforced concrete frames strengthened with concentric steel braces, COMPDYN 2011 III ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering M. Papadrakakis, M. Fragiadakis, V. Plevris (eds.), Corfu, Greece, 26–28 May 2011.
[18] L.A. Fülöp, Dan Dubina: Performance of Wall-Stud Cold-Formed Shear Panels under Monotonic and Cyclic Loading. Part I: Experimental Research, Thin Walled Structures, 42(2), 2004, p. 321-338.
[19] L.A. Fülöp, Dan Dubina: Performance of Wall-Stud Cold-Formed Shear Panels under Monotonic and Cyclic Loading. Part II: Numerical Modelling and Performance Analysis, Thin Walled Structures, 42(1), 2004, p. 339-349.
[20] Della Corte G, De Matteis G, Landolfo R, Influence of Connection Modeling on the Seismic Response of Moment Resisting Steel Frames, In F.M. Mazzolani (ed.) Moment Resistant Connections of Steel Frames in Seismic Areas, E & FN Spoon, 2000.