Budapest University of Technology and Economics

Department of Structural Engineering

Evaluation of Behavior Factor of Buckling Restrained Braced Steel Frames

Prepared by:

Shadi Fattoum

Supervisor:

Dr. László Gergely Vigh, Associate Professor

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ABSTRACT

Numerous studies were published in the last decades about the hysteresis behavior of buckling restrained brace (BRB). Its behavior was compared with other types of bracings and structural systems in order to demonstrate the advantage of BRB.

Engineers design the structures to resist the seismic forces using linear methods such as Equivalent Lateral Force Method and Modal Response Spectrum analysis. Nevertheless, the structures are responding inelastically. Seismic performance factors are used to estimate the nonlinear seismic demands through linear analyses.

The behavior factor q of BRBFs is not addressed in the European Code EC8. Nonetheless, Vigh, Zsarnóczay and Balogh proposed a design methodology and a value of q for BRBFs. This paper aims to evaluate the behavior factor q of buckling restrained braced steel frames. It will provide a brief summary about two methods of assessment adopted in FEMA P-695 and ATC 19 as well.

Two buildings with different heights are considered in this study. Inverted V is the configuration of bracing system of buildings. Beam-column connections are considered as hinges. The columns are continuous and have fixed support. Therefore, columns and bracings will resist the lateral loads. Results of evaluation in both approaches will be compared to adopt a proper value of q for design.

Keywords: Evaluation of behavior factor, Evaluation of response modification factor, ATC-19, The Methodology FEMA P-695.

1. Introduction

Steel moment-resisting frames suffered critical damages during the 1994 Northridge and 1995 Hyogo-ken Nanbu earthquakes due to their less rigidity. This led the engineers to utilize the conventional bracing systems to enhance the stiffness of structures. Nevertheless, the seismic response was not reliable and convincing because of buckling of bracings. [1]

Conventional steel braces buckle when they subject to compression before yielding stress. The buckling restrained brace (BRB) is the solution to the problem of the buckling and the poor behavior of bracing system. The identical hysteresis behavior of BRB in tension and compression leads to preferable seismic performance. **Fig 1** shows the difference between cyclic behavior of BRB and conventional steel braces behavior. [2]

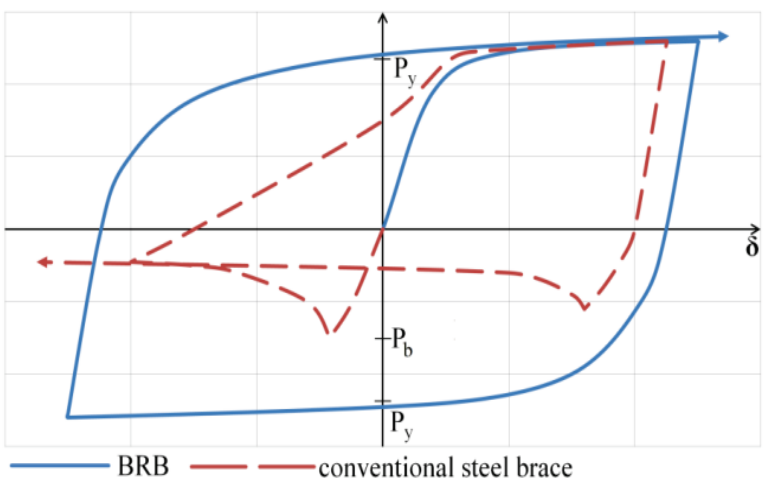

Fig 1 Cyclic behavior of a BRB and a conventional steel brace. [2]

BRB member consists of the steel core surrounded by casing to prevent buckling, transitional zones and elastic zones. The steel core, which is also known as yielding zone, dissipates energy during the inelastic phase. A detailed description of BRB parts is presented in the **Fig 2**. [2]

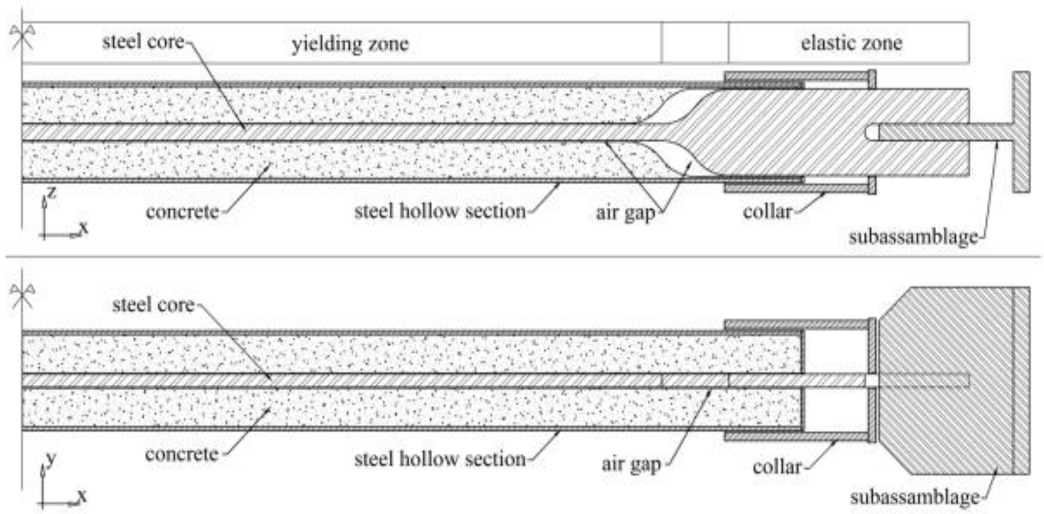

Fig 2 Main parts of BRB member. [2]

2. Seismic Behavior of BRB

Numerous studies were published in the last decades about the hysteresis behavior of BRB. The behavior was compared with other types of bracings and structural systems in order to demonstrate the advantage of BRB.

Mahin et al. [1] carried out an experimental study to evaluate the seismic response of three BRBFs with different bracing configurations. The frames were subject to cyclic loadings and they behaved and dissipated energy well.

Chou and Chen [3] performed an experimental and analytical study of sandwiched buckling restrained braces. 4 Braces were used in the study where one of them less flexural rigidity. They showed a good performance. Moreover, the study presented a design procedure for the proposed subassemblage BRB.

Shen et al. [4] conducted a research to evaluate the seismic response of special concentrically braced frames (SCBFs) with and without brace buckling. 6 structures were used in the study. 3 of them were special concentrically braced frames whereas the other 3 were buckling-controlled braced frames (BCBFs) with chevron configuration. Nonlinear dynamic analyses were carried out to demonstrate the inelastic seismic performance of buildings. It was concluded that the braces and brace-supporting columns of SCBFs got fractures before drifts reached 2%. On other hand, BCBFs showed significantly better response.

3. Behavior Factor Assessment

Engineers design the structures to resist the seismic forces using linear methods such as equivalent lateral force method and modal response analysis. Nevertheless, the structures are responding inelastically. Seismic performance factors are used to estimate the nonlinear seismic demands through linear analyses. As a new structural system, researchers are working to adopt the proper values of its seismic performance factors.

Sabelli, Mahin and Chang [5] were working to define new procedure to design buckling restrained braced frames (BRBFs). They use three structures with 3-, 6-floor height in the study. The researcher used response modification factor R=8 and R=6 in their study. They found that the seismic response of the structures did not change significantly when R changed.

Asgarian and Shokrgozar [6] assessed the response modification factors R, overstrength factor and ductility. The assessment was conducted by applying nonlinear static and linear and nonlinear time-history on several buildings. The buildings were representing different heights and bracing configurations and designed according to the

Iranian code. The values of *R* ranged between 7 and 9.4 depending on the bracing configuration.

Kim, Park and Kim [7] evaluated the response modification factors *R*, overstrength factor and ductility. The assessment was carried out on frames and dual systems designed according to IBC 2003 and AISC. Nonlinear dynamic analysis and pushover analyses were applied on buildings with different heights. The researchers concluded that the taller buildings had smaller response modification factors *R* and the value increased as the height decreased. Furthermore, the values adopted in codes were larger than the computed in the study.

Vigh, Zsarnóczay and Balogh [8] proposed some modifications to the capacity design rules of concentrically braced frames design in EC 8 [9]. The purpose of the study is to provide a procedure to design BRBFs in the framework of EC 8 regulations. The method was applied to design a 6-story structure with behavior factor q=7. The researchers explained the modeling of bracing element and the material model required for pushover and nonlinear dynamic analyses. Zsarnóczay and Vigh [10] evaluated the proposed method using the Methodology of FEMA P-695 [11]. 24 Structures having low or high gravity loads and medium or high seismic excitation with different heights were used in the investigation. All models met the acceptance criteria.

Kalapodis, Papagiannopoulos and Beskos [12] carried out a study on 98 plan steel frames in order to define the modal strength reduction factor (behavior factors). The structures used to resist the lateral forces were eccentrically braced frames (EBFs) and buckling restrained braced frames (BRBFs). Four modal strength reduction factors were used to calculate the base shear forces where each factor depended on one mode. Generally, codes and standards use factor for the first mode. The purpose of these factors is to consider the dynamic characteristics of structures, the effects of soil classes and performance. The structures were designed according to EC3 [13] and EC8 [9]. 100 records, represent 4 different soil classes (A, B, C or D), were used to perform nonlinear dynamic analysis. A number of equations was presented to calculate the behavior factors depending on period, soil class and performance.

Zaruma and Fahnestock [14] evaluated the performance of BRBFs by applying the Methodology of FEMA P-695 [11]. The researchers tested the effect of columns oriented in weak axis, gravity columns that did not resist the seismic loads and dual systems such as BRBFs and special moment resisting frame (SMRF). The structures were designed according to US codes with response modification factor R=8. They found that the orientation of columns and the continuity of gravity columns had important effects on inelastic performance. The dual systems proved preferable and better seismic performance.

4. Research Strategy

An analytical study has been conducted using *ETABS 17 Eval* software. Two buildings, which represent different heights and the same seismic zone have been modeled and analyzed.

5. Scope

The behavior factor q of BRBFs is not addressed in the European Code EC 8 [9]. Nonetheless, Vigh, Zsarnóczay and Balogh [8] proposed a design methodology and a value of q for BRBFs. This paper aims to evaluate the behavior factor q of buckling restrained braced steel frames with inverted V configuration. It provides a brief summary about two methods of assessment adopted in FEMA P-695 [11] and ATC 19 [15] and used in the study as well.

6. Methodologies of Assessment

Performance factors are significant for calculating the lateral design loads and lateral displacements. In the past they were proposed by standards depending on the observation and comparison between structural systems. Due to the increasing of structural systems, engineers required reliable values.

Researchers and engineers have developed a number of approaches to evaluate the performance of structural systems and their response factors. Applied Technology Council (ATC) published a report called "Structural Response Modification Factors" ATC 19 [15] in 1995. One of its objectives was to evaluate the response modification factors R. In 2009 Federal Emergency Management Agency (FEMA) published "Quantification of Building Seismic Performance Factors" FEMA P-695 [11]. The so-called Methodology is the approach adopted by FEMA and prepared by ATC to evaluate the performance factors.

6.1 Simplified Approach of ATC-19 [15]

ATC 19 [15] suggest a simple equation (1) to calculate the response modification factors R as the product of three parameters. The steps of computing of response modification factor according to this method is explained in a flowchart in **Fig 3**.

$$R = R_s R_\mu R_R \tag{1}$$

Where: R_s is the period-dependent strength factor to consider the increase in the maximum lateral strength of a structure comparing with the design strength;

R_μ is a time-dependent ductility factor to account for the inelastic deformation; and

R_R is a redundancy factor to calculate the reliability of seismic framing systems constructed with multiple lines of strength.

The period-dependent strength factor R_s is calculated by the equation (2):

$$R_s = \frac{V_o}{V_d} \tag{2}$$

Where: V_o is the base shear corresponding to the maximum nonlinear response; and

V_d is the design base shear.

Several studies were carried out to define an expression to calculate R_μ and mentioned in the report of ATC 19 [15]. One of them is the equation (3) developed by Miranda and Bertero in 1994 will be considered.

$$R_\mu = \frac{\mu - 1}{\Phi} \tag{3}$$

Where: Φ is calculated as in equation (4) for rock sites and as in equation (5) for alluvium sites.

$$\Phi = 1 + \frac{1}{10T - \mu T} - \frac{1}{2T} e^{-1.5(\ln(T) - 0.6)^2} \tag{4}$$

$$\Phi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T} e^{-2(\ln(T) - 0.2)^2} \tag{5}$$

A redundant seismic framing system should be composed of multiple vertical lines of framing, each designed and detailed to transfer seismic-induced inertial forces to foundation. The ATC 19 [15] proposed **Table 1** for the value of R_R.

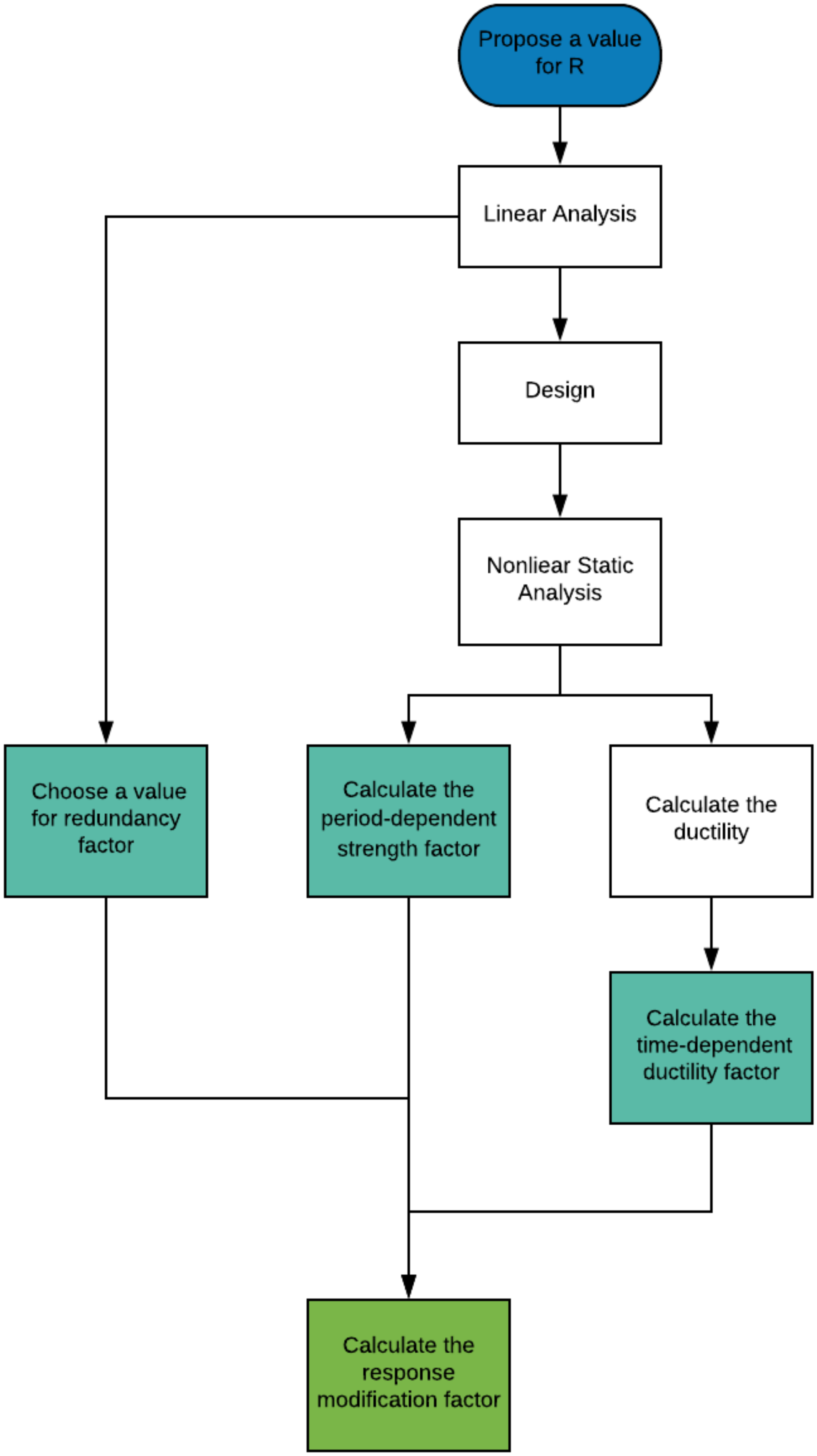

Fig 3 ATC-19 flowchart.

Table 1 Values of R_R [15]

Lines of vertical seismic framing	R_R
2	0.71
3	0.86
4	1

The advantage of this method is that the factors can be calculated after pushover analysis only.

6.2 The Methodology of FEMA P-695 [11]

The Methodology is an innovative approach based on the recent studies on nonlinear performance of structural systems. It is developed to assess and obtain seismic performance factors that guarantee a performance in life safety domain.

The process of assessment comprises the following steps:

1- Develop system concept.
2- Collect the required data and information.
3- Define the archetypes.
4- Modeling.
5- Nonlinear analyses.
6- Evaluating the response results.

6.2.1 Develop System Concept

This is the first step. It includes defining the lateral-force-resisting system, construction materials, system configurations and properties, inelastic dissipation mechanisms, and intended range of application.

6.2.2 Collect the Required Data and Information

Collect the data and the results of tests to describe the properties and behaviors of materials, structural members and connections. The strength, stiffness and ductility of them should be specified. Moreover, other required data are the intended performance and seismic response, gravity loads, methods of constructions and layouts.

6.2.3 Define the Archetypes

The archetypes should represent the behavior of structural system subject to earthquakes. This behavior is defined by the characteristics of materials, structural members, buildings geometric properties and seismic zone data. Then, archetypes are grouped in performance groups which reflect models' heights (periods), loads and seismic intensities.

6.2.4 Modeling

The archetypes should be designed according to Equivalent Lateral Method, Modal Response Spectrum Method or Response History Analysis. They should meet the minimum requirements of seismic design. According to the Methodology, those archetypes are required to be designed to meet the design earthquake (DE) conditions. The design earthquake (DE) is defined as two-thirds of the maximum considered earthquake (MCE) demand which is used in collapse assessment. "Maximum considered earthquake (MCE) for a specific area, is an earthquake that is expected to occur once in approximately 2500 years; that is, it has a 2-percent probability of being exceeded in 50 years."[16]

For response assessment, the Methodology recommends to use values of Site Class D and upper-bound and lower-bound values of Seismic Design Categories B, C and D adopted by ASCE/SEI 7-05 [17]. **Fig 4** shows the response spectra of DE for ground motions records corresponding to those criteria.

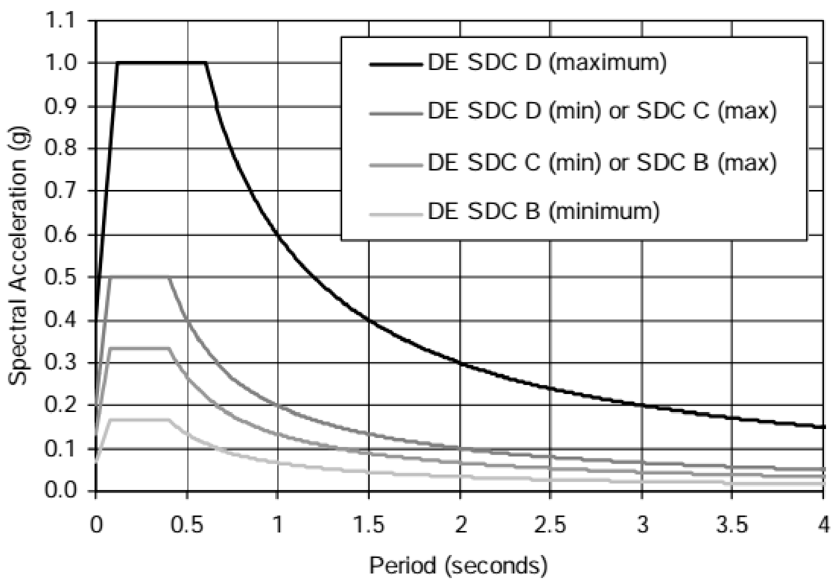

Fig 4 DE response spectral accelerations used for design of Seismic Design Category D, C and B structure. [11]

The Methodology recommends to calculate the approximate fundamental period T using the equation (6) which is adopted by ASCE/SEI 7-05 [17]. The fundamental period is important to calculate the base shear force in Equivalent Lateral Method and collapse margin ratio *CMR*.

$$T = C_u T_a = C_u C_T h_n^x \tag{6}$$

The structural model should represent the properties, configurations and nonlinear behavior of system and its members. Archetypes can be modeled using the nonlinear continuum finite element models, nonlinear springs, 2D or 3D models. Type of model depends on the features of structure. The most significant issue is to simulate the inelastic response of structural members. Each modeling method has advantages and disadvantages in simulating the collapse and behavior. Thus, the Methodology introduced a factor to represent the uncertainty of modeling β_{MDL}.

6.2.5 Nonlinear Analyses

The first step of nonlinear analyses is carrying out a nonlinear analysis of gravity loads. The Methodology suggests a combination of dead and live loads as in equation (7). P-Delta effects should be considered in analyzing.

$$1.05\ Dead\ loads + 0.25\ Live\ loads \qquad (7)$$

Nonlinear static analyses (pushover) are carried out to estimate system overstrength Ω and period-based ductility μ_T. The load pattern used in pushing models is proportional to the first mode shape of the analysed model. In order to obtain the system overstrength Ω and the period-based ductility μ_T of an archetype, V_{max} and δ_u should be calculated as they are defined in **Fig 5**. V_{max} is the maximum base shear capacity and δ_u is the ultimate roof displacement corresponding to 20% loss of the maximum base shear capacity (0.8 V_{max}).

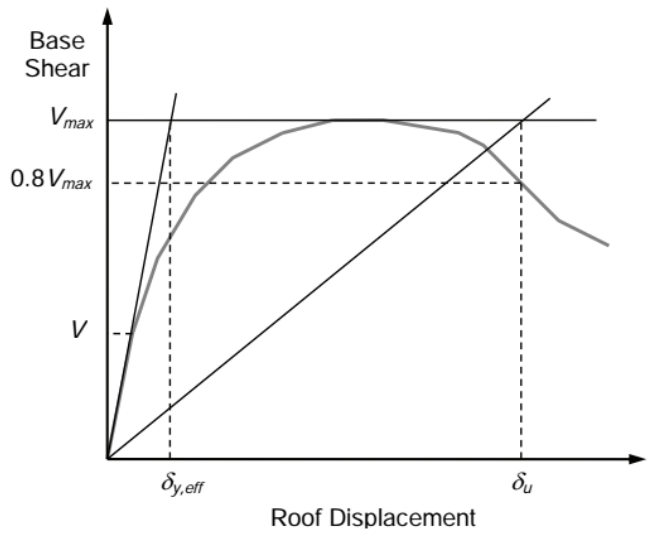

Fig 5 Pushover curve and the essential values. [11]

The overstrength factor Ω is ratio of the maximum base shear strength V_{max} to the design base shear V as in equation (8).

$$\Omega = \frac{V_{max}}{V} \tag{8}$$

The period-based ductility μ_T is introduced as the ratio of ultimate roof displacement δ_u to the effective yield roof drift displacement $\delta_{y,eff}$ as in equation (9).

$$\mu_T = \frac{\delta_u}{\delta_{y,eff}} \tag{9}$$

The effective yield roof drift displacement $\delta_{y,eff}$ is computed by equation (10):

$$\delta_{y,eff} = C_0 \frac{V_{max}}{W} \left[\frac{g}{4\pi}\right] (\max(T, T_1))^2 \tag{10}$$

Where: C_0 is a factor relates fundamental-mode (SDOF) displacement to roof displacement as in equation (11);

V_{max}/W is the maximum base shear normalized by building weight;

g is the gravity constant;

T is the approximate fundamental period as in equation (6); and

T_1 is the fundamental period of the model estimated by eigenvalue analysis.

$$C_0 = \Phi_{1,r} \frac{\sum_1^N m_x \Phi_{1,x}}{\sum_1^N m_x \Phi_{1,x}^2} \tag{11}$$

Where: m_x is the mass at story level x;

$\Phi_{1,x}$ is the ordinate of the fundamental mode at story level x and $\Phi_{1,r}$ is the roof's; and

N is the number of stories.

After pushover analyses, the incremental dynamic analyses (IDA) are conducted to obtain median collapse capacities $\hat{S}_{CT}$ and collapse margin ratios CMR. "Median collapse capacity $\hat{S}_{CT}$ is defined as the ground motion intensity where half of the ground motions in the record set cause collapse of an index archetype model" [11]. Collapse margin ratio CMR is the ratio of median collapse capacities $\hat{S}_{CT}$ to the intensity of maximum considered earthquake (MCE) ground motion intensity S_{MT} as in equation (12).

$$CMR = \frac{\hat{S}_{CT}}{S_{MT}} \tag{12}$$

MCE ground motion intensity S_{MT} is computed from the response spectrum of MCE ground motions at the fundamental period T. It can be calculated for short-period models ($T \leq Ts$) as in equation (13) and for long-period models ($T > T_s$) as in equation (14).

$$S_{MT} = S_{MS} \tag{13}$$

$$S_{MT} = \frac{S_{M1}}{T} \tag{14}$$

IDA are performed by 22 pairs of records suggested by FEMA P695 [11] and can be found in PEER NGA database [18]. The records are scaled until the collapse of the structure occurs. In case of 3D models, the 22 pairs are applied twice to each model to consider both directions. Furthermore, the median collapse intensity $\hat{S}_{CT}$ should be multiplied by a factor of 1.2 due to the conservative results. In case of 2D models, the component of each pair of records will be applied in the considered direction as an independent event. At least 5 analyses for each component are required to reach the collapse and draw the curve. Consequently, 220 analyses are required for each archetype to estimate $\hat{S}_{CT}$ and *CMR*. The plot of analyses results represents the relationship between the intensities of records and the maximum story drift ratio

6.2.6 Evaluating the Response Results

The median collapse capacities $\hat{S}_{CT}$ and collapse margin ratios *CMR* are significantly affected by the frequency content of the ground motion records. In order to consider this influence, the *CMR* is modified to adjusted collapse margin ratio *ACMR* by multiplying the *CMR* by spectral shape factor *SSF* as in equation (15). *SSF* relies on the approximate fundamental period, period-based ductility μ_T and the applicable Seismic Design Category.

$$ACMR_i = CMR_{\,i} \times SSF_i \tag{15}$$

The next step is to define the acceptable values of $ACMR_{10\%}$ and $ACMR_{20\%}$. Both values depend on the total collapse system uncertainty β_{TOT}. The sources of uncertainty are record-to-record collapse uncertainty, design requirements-related collapse uncertainty, test data-related collapse uncertainty and modeling-related collapse uncertainty. β_{TOT} can be computed using the equation (16).

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \tag{16}$$

Whereβ_{RTR}: is the record-to-record collapse uncertainty;

β_{DR} is the design requirements-related collapse uncertainty;

β_{TD} is the test data-related collapse uncertainty; and

β_{MDL} is the modeling-related collapse uncertainty.

FEMA P695 [11] provides 4 tables to estimate β_{TOT} for models with $\mu_T \geq 3$ depending on model quality, quality of design requirements and quality of test data, provided that, $\beta_{RTR} = 0.4$ and the other uncertainties are 0.1 for superior, 0.2 for good, 0.35 for fair and 0.5 for poor. Then, $ACMR_{10\%}$ and $ACMR_{20\%}$ can be estimated using table provided by FEMA P695 [11].

Finally, the value of proposed behavior factor or response modification factor is accepted if the average value of $ACMR$ for each performance group $\overline{ACMR_\iota}$ is larger than $ACMR_{10\%}$ and value of $ACMR$ for each archetype is larger than $ACMR_{20\%}$. In other words, the performance is acceptable. The equations (17) and (18) represent the former conditions.

$$\overline{ACMR_\iota} \geq ACMR_{10\%} \tag{17}$$

$$ACMR_i \geq ACMR_{20\%} \tag{18}$$

In order to evaluate the value of system overstrength Ω_0, the average value of system overstrength for each performance group should be calculated first. Then, the value of system overstrength Ω_0 is the largest value of the average values. However, it should not exceed 1.5 times of response modification factor and for practical design it should not exceed 3 according to ASCE/SEI 7-05 [17].

The deflection amplification factor C_d depends on response modification factor and effective damping. Generally speaking, it is equal to response modification factor.

Fig 6 shows a flowchart explain the steps of evaluation of proposed response modification factor according to this method.

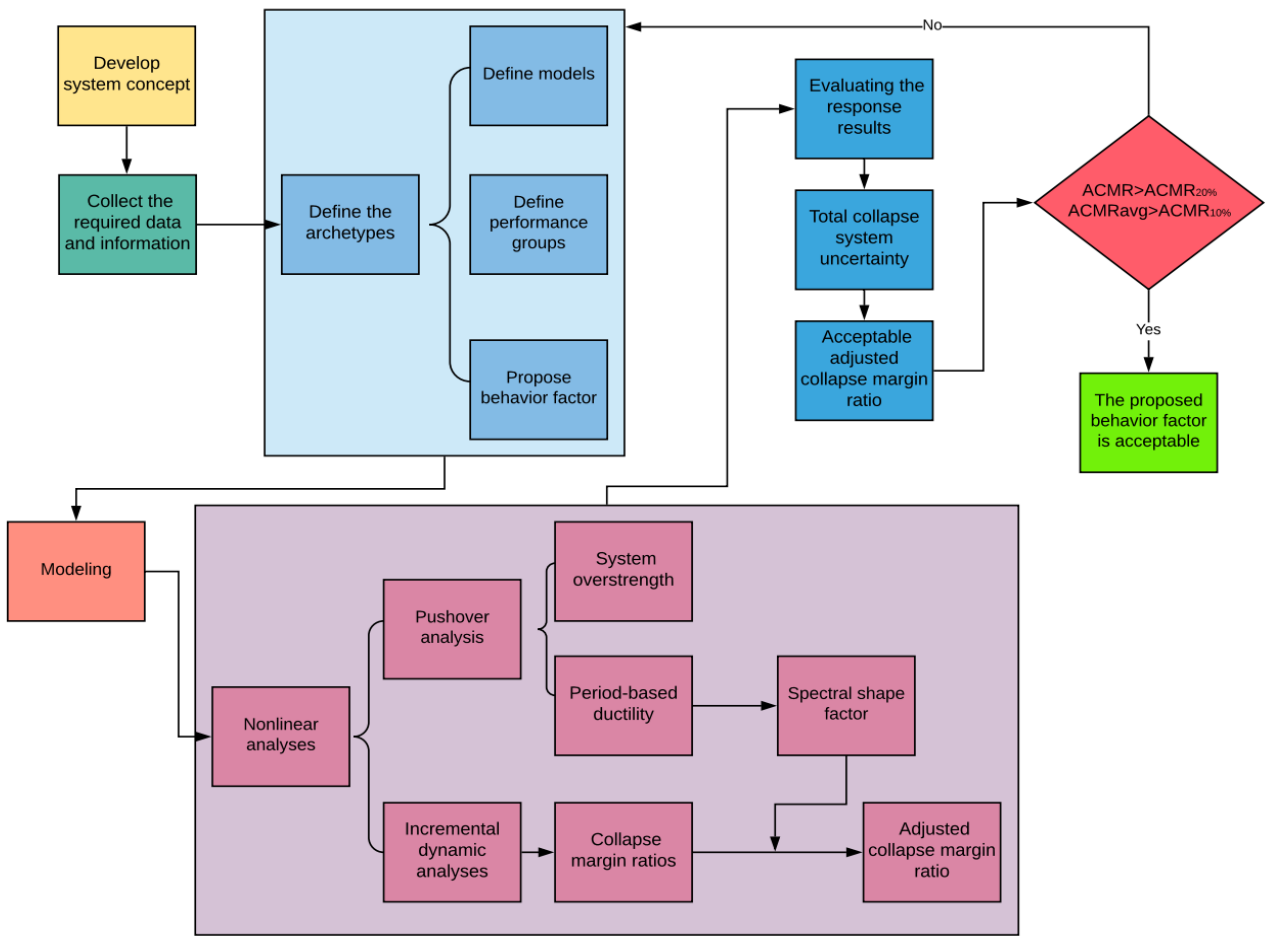

Fig 6 The Methodology FEMA P-695 flowchart.

7. Buildings Description

Two buildings are considered in the study and formed a performance group. The same plan and elevation (inverted V braces in 2 spans) are used for both buildings. **Fig 7** shows the plan and elevation of R4H and. The plan is 5 spans in each direction. The typical span length is 6m and the typical story height is 3m. 4-Story and 8-story buildings will be used in this study. The dead loads are 8 KN/m^2 and live loads are 2 KN/m^2. PGA is 0.35g.

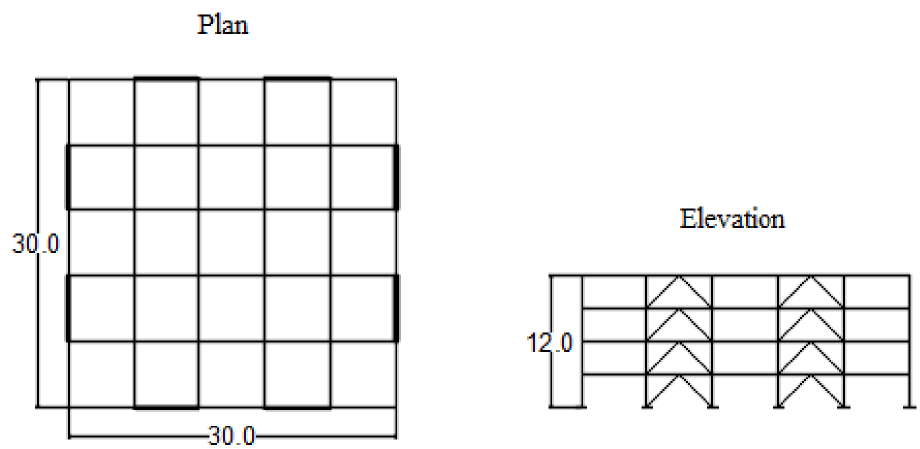

Fig 7 Plan and elevation of archetype R4H.

8. Assumptions of Modeling, Linear and Nonlinear Analyses

The buildings are analysed and designed according to EC 8 [9] using the modal response spectrum analysis. The modal analyses are carried out by eigenvalue analysis considering a number of modes enough to achieve more than 90% of modal participating mass ratio. Importance factor is considered as 1, Ψ_2 is 0.3, soil class is D and the proposed behavior factor q is **8**. The maximum inelastic drift is 0.02. Bracing systems (inverted V configuration) and columns resist the seismic loads. The beam-column connections are considered as hinges. P-Δ effects are considered in the study.

Since the building is symmetric, a simplified 2D model will be used in the study. The 2D model represent half of a structure and the leaning column carry the half of the mass. The weight of the model is calculated as the combination of total dead loads and 0.3 of live loads in case of linear analysis. For nonlinear calculations, the weight or mass was calculated using the equation (7).

Material grades are S355 for beams and columns and S235 with 1.25 hardening factor for BRB (Star seismic BRB). Modulus of elasticity is 210 GPa.

Area of BRB yield zone cross-section is calculated using the nominal value of steel S235 whereas the force of BRB is calculated by mean value 245 MPa according to Star Seismic report [19].

Plastic hinges are defined and assigned to columns (axial load - flexural moment) and BRB (axial load) according EC8 part 3 [20] regulations. The positions of plastic hinges in columns are located at 150 mm (5% of column length) over the base of columns and at 150 mm (5% of column length) under the bottom of beams. The proposed length of plastic hinges is 300 mm (10% of column length). The length of yield zone of BRB is 2.97 m (70% of brace length) according to Star Seismic report [19].

A nonlinear analysis of gravity loads (called GR) according to the combination of dead and live loads as in equation (7) is carried out before any nonlinear static analysis. The pushover analyses are carried out using load pattern proportional to the first mode for each model. The pushover analysis should be done until reach 20% loss of the maximum base shear capacity.

In order to perform Nonlinear Modal Time-History Analysis, plastic hinges are modeled as multilinear plastic links. The columns plastic hinge is axial load - flexural moment. The axial behavior is modeled as linear with effective stiffness equals to EA/L. Here L is the length of the link 30 cm (the same as the plastic hinge). The behavior of link is nonlinear with Kinematic Hysteresis Model. The values of multilinear moment-rotation curve are adopted from the backbone of plastic hinge. The braces have axial load plastic hinges which will be modeled by multilinear plastic links using a BRB Hardening Hysteresis Model. The effective stiffness of each brace will be taken from the section properties in ETABS and the nonlinear behavior will be similar to the backbone curve of the brace. The same hardening factor and deformation values are adopted as well.

Modal analyses using Ritz vector should be carried out in order to perform Nonlinear Modal Time-History Analysis (FNA). Then, a nonlinear analysis of gravity loads according to the combination of dead and live loads as in equation (7) should be conducted. This could be done by carrying out FNA which can simulate the same results and effects of nonlinear static analysis of gravity loads GR using the gravity loads with proper scale factor.

Incremental dynamic analyses are conducted using the 22 pairs of records which were normalized by normalization factors provide by FEMA P-695 [11]. Records must be scaled until reaching the collapse.

According to ASCE/SEI 7-05 [17] MCE is 1.5 times the design earthquake (DE). This means in EC8 1.5 times the elastic response spectrum (before dividing it by behavior factor). S_{MT} for each model is calculated depending on the first mode period computed by modal analysis instead of approximate fundamental period because the EC8 allows to use the period from modal analysis.

9. Results of Analyses

9.1 Results of Analyses and Design

The results of linear analyses of R4H and R8H are show in **Table 2**. In both models HEA 340 is the steel profile used for beams. **Table 3** and **Table 4** show the sections in R4H and R8H respectively.

Table 2 Linear analysis results

Model	**R4H**	**R8H**
Base Shear (KN)	1936.66	1940.37
Weight (KN)	15480	30960
$T_{Dynamic}$(sec)	0.77	1.529

Table 3 Sections of structural elements (R4H)

Floor	**Column**	**BRB**
1st	HEM220	StarBRB_3 (19.4 cm^2)
2nd	HEM200	StarBRB_4 (25.8 cm^2)
3rd	HEM200	StarBRB_3 (19.4 cm^2)
4th	HEM200	StarBRB_2 (12.9 cm^2)

Table 4 Sections of structural elements (R8H)

Floor	Column	BRB
1st	HEM340	StarBRB_3 (19.4 cm^2)
2nd	HEM300	StarBRB_5 (32.3 cm^2)
3rd	HEM280	StarBRB_4.5 (29 cm^2)
4th	HEM220	StarBRB_4 (25.8 cm^2)
5th	HEM200	StarBRB_3.5 (22.6 cm^2)
6th	HEM200	StarBRB_3 (19.4 cm^2)
7th	HEM200	StarBRB_2.5 (16.1 cm^2)
8th	HEM200	StarBRB_1.5 (9.7 cm^2)

Pushover analyses are carried out using load pattern proportional to the first mode for each model. Pushover curve of R4H is shown in **Fig 8**. Then, IDA are carried out for each model. After 220 nonlinear time-history analyses, the relationship between spectral intensity of the ground motion and maximum story drift ratio for R4H is drawn in a chart in Fig 9.

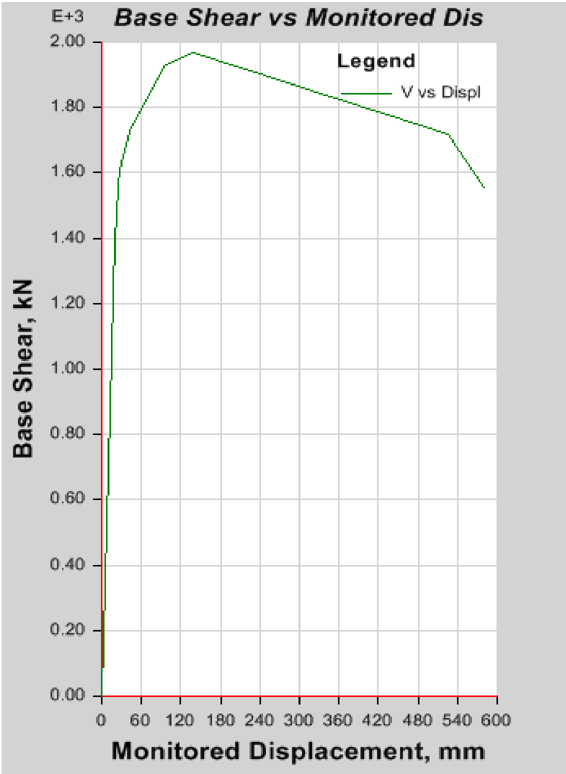

Fig 8 Pushover curve of archetype R4H.

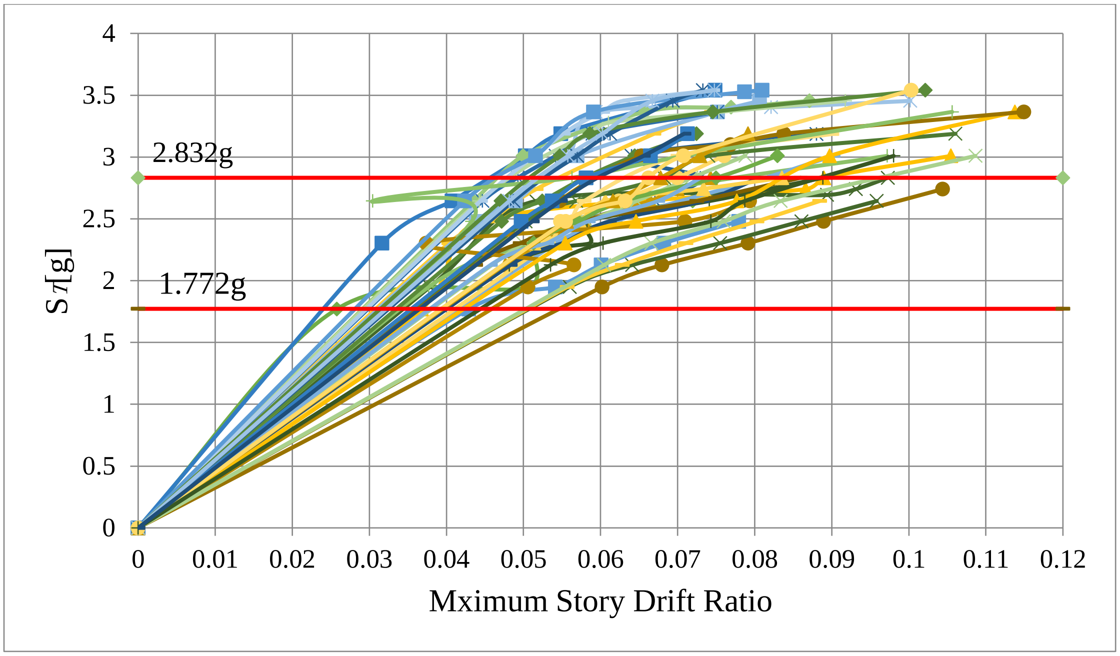

Fig 9 Incremental dynamic analysis plot of archetype R4H.

9.2 Results of Simplified Approach of ATC-19

Results of simplified approach of ATC-19 is provided in **Table 5**. For R4H, the behavior factor q is 6.5 and for R8H the behavior factor q is 8.58. The difference between the results is due to the difference in ductility.

Table 5 Results of simplified approach of ATC-19

Model	R4H	R8H
V_d	1936.66	1940.37
V_o	1969.46	2042.96
Δ_m mm	193.2	507.69
Δ_y mm	28.3	56.04
μ	6.83	9.06
Φ	0.912	0.986
R_s	1.017	1.05
R_μ	6.39	8.17
R_R	1	1
q	6.5	8.58

The computation of response modification factors R (or behavior factor q) according to ATC-19 method is provided here for R4H.

The period-dependent strength factor R_s is:

$$R_s = \frac{V_o}{V_d} = \frac{1969.46}{1936.66} = 1.017$$

In order to calculate the ductility, the maximum inelastic displacement should be computed first which is done according to Target Displacement method of EC8 $\Delta_m = 193.2\ mm$ and $\Delta_y = 28.3\ mm$.

$$\mu = \frac{\Delta_m}{\Delta_y} = 6.83$$

For alluvium sites:

$$\Phi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T} e^{-2(\ln(T) - 0.2)^2} = 0.912$$

The time-dependent ductility factor R_μ is:

$$R_\mu = \frac{\mu - 1}{\Phi} = 6.39$$

Redundancy factor $R_R = 1$

$$q = R_s R_\mu R_R = 1.017 \times 6.39 = 6.5$$

9.3 Results of the Methodology of FEMA P-695

Results of Methodology of FEMA P-695 is provided in **Table 6**. The MCE response spectrum of EC8 (PGA=0.35g and Soil class D) is close to response spectrum of Seismic Design Category D_{max} in FEMA P-695. Therefore, Spectral shape factor SSF can be computed according to SDC D_{max} table. A Comparison between MCE response spectrum of EC8 and MCE response spectrum SDC D_{max} is shown in **Fig 10**.

Table 6 Results of Methodology of FEMA P-695

Model	**R4H**	**R8H**
$\boldsymbol{V}$ **KN**	1936.66	1940.37
$\boldsymbol{V_{max}}$ **KN**	1969.46	2042.96
$\boldsymbol{\Omega_0}$	1.017	1.05
$\boldsymbol{C_0}$	1.313	1.403
$\boldsymbol{\delta_{y,eff}}$ **mm**	74.4	161.45
$\boldsymbol{\delta_u}$ **mm**	573.41	842
$\boldsymbol{\mu_T}$	7.676	5.22
$\boldsymbol{\widehat{S}_{CT}}$	2.832 g	1.575 g
$\boldsymbol{S_{MT}}$	1.772 g	0.927 g
$\boldsymbol{CMR}$	1.598	1.7
$\boldsymbol{SSF}$	1.391	1.467
$\boldsymbol{ACMR}$	2.223	2.494

The evaluation of response modification factors R (or behavior factor q) according to Methodology of FEMA P-695 is provided here for R4H.

The overstrength factor Ω is:

$$\Omega_0 = \frac{V_{max}}{V} = \frac{1969.46}{1936.66} = 1.017$$

$$C_0 = \Phi_{1,r} \frac{\sum_1^N m_x \Phi_{1,x}}{\sum_1^N m_x \Phi_{1,x}^2} = 1.313$$

The effective yield roof drift displacement $\delta_{y,eff}$ is:

$$\delta_{y,eff} = C_0 \frac{V_{max}}{W} \left[\frac{g}{4\pi}\right] (\max(T, T_1))^2 = 74.7\ mm$$

δ_u which is the ultimate roof displacement corresponding to 20% loss of the maximum base shear capacity is:

$$\delta_u = 573.41\ mm$$

The period-based ductility μ_T is:

$$\mu_T = \frac{\delta_u}{\delta_{y,eff}} = 7.676$$

Collapse margin ratio *CMR* is:

$$CMR = \frac{\hat{S}_{CT}}{S_{MT}} = 1.598$$

for T=0.77 sec, $\mu_T = 7.676$ and Seismic Design Category D_{max} $SSF = 1.391$.

Then, the adjusted collapse margin ratio *ACMR* is:

$$ACMR_i = CMR_{\ i} \times SSF_i = 1.598 \times 1.391 = 2.223$$

The sources of uncertainty are record-to-record collapse uncertainty, design requirements-related collapse uncertainty, test data-related collapse uncertainty and modeling-related collapse uncertainty. Due to the numerous researches about the behavior of BRB, its design requirements are clear in research and international standards as well as the ability of ETABS to simulate the behavior in different ways, the following considerations will be taken into account. The design requirements used in this study are considered good. The quality of test data is considered good. Modeling is also considered good representing the structure very well. Consequently, the total collapse system uncertainty is $\beta_{TOT} = 0.525$.

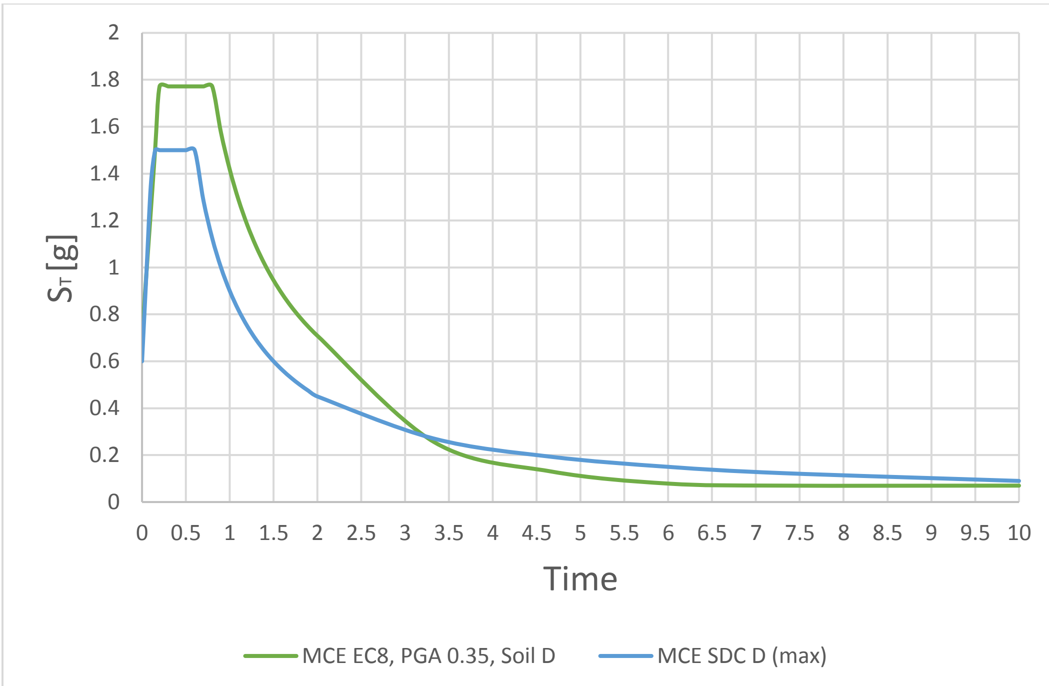

Fig 10 Comparison between MCE response spectrum of EC8 and MCE response spectrum SDC D_{max}.

According to FEMA P-695 [11] for $\beta_{TOT} = 0.525$, $ACMR_{10\%} = 1.96$ and $ACMR_{20\%} = 1.56$. For each model $ACMR_i \geq ACMR_{20\%}$ and the average $\overline{ACMR_i} = 2.359 \geq ACMR_{10\%}$ this means that both models meet the requirements and the proposed behavior factor *q*=8 is appropriate.

10. Comparison and Discussion

Simplified approach of ATC-19 is a simpler method to calculate the behavior factor than the Methodology of FEMA P-695. It only depends on the pushover analysis which does not consume much time. On other hand, the Methodology of FEMA P-695 requires both pushover analysis and nonlinear time history analyses. Thus, it is considered more accurate and efficient in evaluation the performance of structures. Simplified approach of ATC-19 provides two different values for the models due to the difference in ductility and other parameters and the behavior factor of R8H is close to the proposed behavior factor q=8. Nonetheless, according to the Methodology of FEMA P-695 the proposed behavior factor q=8 is appropriate.

Further studies are recommended for adopting an accurate value for behavior factor.

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