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Evaluation of seismic performance factors for tension-only braced frames

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Abstract. The tension-only braced frames (TOBFs) are widely used as a lateral force resisting system (LFRS) in low-rise steel buildings due to their simplicity and economic advantage. However, the system has poor seismic energy dissipation capacity and pinched hysteresis behavior caused by early buckling of slender bracing members. The main concern in utilizing the TOBF system is the determination of appropriate performance factors for seismic design. A formalized approach to quantify the seismic performance factor (SPF) based on determining an acceptable margin of safety against collapse is introduced by FEMA P695. The methodology is applied in this paper to assess the SPFs of the TOBF systems. For this purpose, a trial value of the R factor was first employed to design and model a set of TOBF archetype structures. Afterwards, the level of safety against collapse provided by the assumed R factor was investigated by using the non-linear analysis procedure of FEMA P695 comprising incremental dynamic analysis (IDA) under a set of prescribed ground motions. It was found that the R factor of 3.0 is appropriate for safe design of TOBFs. Also, the system overstrength factor (Ω_0) was estimated as 2.0 by performing non-linear static analyses.

Keywords: tension-only bracing; response modification factor; incremental dynamic analysis; seismic design; overstrength factor

1. Introduction

TOBFs consist of slender members such as single angles, channels or steel rods designed solely for tension under code specified earthquake loading. As a result, these bracing members have negligible buckling strength, which make them incapable of dissipating seismic energy in compression. The result of alternating between tension yielding and compression buckling of the bracing leads to deteriorating and pinched hysteretic behavior in compression cycles, which is the main characteristic of TOBFs. According to seismic provisions of AISC 341-10 (AISC 2010), the use of tension-only bracing is not permitted as special concentrically braced frames (SCBF) and it is only permitted as ordinary concentrically braced frames (OCBF). In the active seismic areas, using the TOBF system is not recommended in medium and high-rise buildings (Filiatrault *et al.* 1998). However, because of inexpensive implementation and design costs, the TOBF system is prevalently used in low-rise steel buildings all over the world. This extensive usage needs to be accompanied by adequate supporting research on their seismic performance. The seismic simulator table test on a half-scale, two-story TOBF specimen was conducted by (Tremblay *et al.* 1996). They compared the experimental hysteretic loops with quasi-static and dynamic monotonic test data. A small but perceptible increment in the tensile strength of the braces was observed, which is caused by the effect of strain rate on the yield strength of steel material. Also, a method was suggested for prediction of this increment in tensile forces at the design stage.

Seismic events have always caused critical challenges in structural design. In this regard, choosing the appropriate building material and system is an important step for designers. Concrete is one of the materials which cannot withstand the tensile and flexural seismic forces alone; hence, reinforcement bars, polymers, and synthetic fibers have been proposed to enhance the ductility and bending flexibility of the concrete structures (Sinaei *et al.* 2011, Toghroli *et al.* 2017, Chuanhua Xu 2019, Li *et al.* 2019, Shariati *et al.* 2019a, Shariati *et al.* 2020). Steel rack structures produced from cold formed steel profiles also have significant deficiencies against seismic forces, where different studies have been performed on mitigating the governing buckling modes under lateral forces (Shariati *et al.* 2019b). The use of partially closed sections or some special reinforcements incorporated within perforated sections has been suggested as a solution for these concerns (Shah *et al.* 2016, Shariati *et al.* 2018, Chen *et al.* 2019, Shariati *et al.* 2019b, Taheri *et al.* 2019, Naghipour *et al.* 2020).

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One of the main issues in utilizing TOBFs in seismic design of buildings is the determination of appropriate response modification factor (R). Current seismic design practice has relied on the capability of structures to dissipate energy by inelastic behavior. Based on the premise that a well-detailed seismic load-carrying system is able to develop a lateral strength more than its design strength and sustain large inelastic deformations without collapse, the seismic design force can be determined by decreasing the expected elastic seismic forces by the R factor. This was first introduced in ATC-3-06 (Council *et al.* 1978). The ratio of the forces is represented by this factor, which would develop under specified ground motions if the structure has to behave entirely elastically to the predetermined design forces at the strength limit state (Kim *et al.* 2005). Utilizing the R factor simplifies the structural design process such that only linear elastic static analysis would be required to design most buildings, especially for those with less complex dynamic behavior.

The R factors proposed in the earlier seismic design codes were selected based on the observed performance of seismic resistant systems during real earthquakes and engineering estimation. Even in the most recent seismic provisions, the values of R factor are sometimes specified by engineering judgment without suitable confirmation of their seismic performance features. Therefore, the necessity of a more reasonable evaluation of the R factor for some LFRSs is demonstrated by the lack of technical basis for the assigned values. Most of the past research efforts in this area have focused on calculating the R factor as a product of overstrength (Ω_0) and ductility (μ) factors. Some literature also mention redundancy in the structure as a separate factor, but it is often considered as a parameter contributing to overstrength. A detailed review of research on the determination of ductility factors for single degree of freedom systems subjected to different types of ground motions was presented by (Miranda *et al.* 1994). A comprehensive list of various R-μ-T relationships that can be used together with an estimation of overstrength factor to determine the value of R factor for different seismic resistant systems was also presented in their article. Based on this approach, (Balendra *et al.* 2003) studied the R factors of moment resisting frames (MRFs), concentrically braced frames (CBFs), and semi-rigid frames. It was found that for inverted chevron bracing and split cross bracing frames, the overstrength and ductility factors are approximately the same. (Kim and Choi 2005) also used the same method to evaluate the R factors of chevron type concentrically braced frames through pushover analyses on model structures with diverse number of stories and span lengths.

In order to provide a rational basis for evaluating the SPFs in the context of collapse potential, the FEMA P695 methodology (FEMA P695. 2009) was established in 2009. The SPFs based on this methodology are assigned to provide equivalent safety against collapse under seismic excitation for structures with various seismic force resisting systems (FEMA P695. 2009). Recent investigations on the R factor of various structural systems have been mostly conducted based on the more advanced FEMA P695 approach. The FEMA P695 methodology was utilized by (Zareian *et al.* 2010) to analyze the collapse performance of 1- to 20-story steel special moment resisting frames (SMRFs). It was reported that a satisfactory margin of safety is provided by SMRFs designed with an R factor of 8, except for tall frames designed for high seismic areas using the response spectrum analysis method. It was also found that calculated safety margins did not follow regular patterns as the number of stories increased. (Hsiao *et al.* 2013) assessed collapse and the corresponding R factor for special concentrically braced frames (SCBFs) by conducting the FEMA P695 procedure. A pair of three-story and 20-story SCBFs designed with R factor = 3 were analyzed, and it was found that both systems passed the safety criteria while using a value of 6 for the R factor failed to meet the criteria. The analyses also revealed that the 20-story SCBF has a greater collapse probability than the three-story SCBF. These findings were contrary to popular belief and results of other investigators in which low rise braced frames were considered more vulnerable (Chen *et al.* 2010).

This paper evaluates the seismic performance factors, including the R factor, the overstrength factor (Ω_0), and the deflection amplification factor (C_d) for TOBFs according to the suggested methodology of FEMA P695. The TOBF archetype buildings were initially designed according to ASCE/SEI 7-10 (ASCE. 1994) and AISC Seismic Provisions for Structural Steel Buildings (AISC. 2010). Then, to evaluate the collapse possibility and R factor, the IDAs were carried out that consistently guarantee the intended level of safety against collapse for the archetype structures. The overstrength factor was also derived by conducting non-linear static (pushover) analyses.

2. General scope of collapse safety assessment

The FEMA P695 (FEMA P695. 2009) methodology used in this paper involves different steps that begin with collecting data about the LFRS and characterizing the possible system behavior through representative buildings denoted as archetypes. The procedure continues with non-linear modeling of each archetype and performing non-linear static and dynamic analyses to assess the SPFs, considering uncertainties. The iterative process of this methodology can be outlined as follows:

- Required data about the seismic resistant system are gathered, comprising construction materials, the configuration of the system, mechanisms of energy dissipation, and the intended range of application. The expected range of building sizes, gravity loads, and categories of seismic design are covered by structural system archetypes that are designed in accordance with this information.
- The experimental data and other supporting results characterizing the system behavior are collected. The simulation of non-linear behavior develops the analytical models of the archetypes, and the available experimental data verifies them.

Table 1 Performance groups for assessment of TOBF archetypes

Group No.	Grouping Criteria			Number of Archetypes
	Design Load Level		Period Domain	
	Gravity	Seismic		
PG-1	Typical	SDC D_{max}	Short	3
PG-2			Long	0
PG-3		SDC D_{min}	Short	3
PG-4			Long	0

- To evaluate the Collapse Margin Ratio (CMR) of each model, archetypes are analyzed using the non-linear dynamic procedure. The ratio of the median collapse intensity to the maximum considered earthquake (MCE) intensity defines the CMR. Pushover analyses are also performed to determine the overstrength and ductility factor for each archetype.
- Total uncertainty involved in the evaluation process is estimated through the quality rating of the following four sources of uncertainty: (1) Record-to-record uncertainty, (2) Design requirements uncertainty, (3) Test data uncertainty, and (4) Modeling uncertainty.
- To explain the unique spectral shape influences of rare ground motion, the CMR is adjusted. The Adjusted CMR (ACMR) of each archetype is then compared to acceptable amounts, which depend on the estimated total uncertainty of the process.
- When minimum criteria are achieved, the presumed R factor to design model is deemed appropriate. Otherwise, the R factor value changes and archetypes are designed again until the assumed R factor is able to provide the adequate collapse safety.

3. Development of archetypes

Archetype buildings are a prototypical representation of the seismic resistant system, which are determined by the system information. They represent the system behavior in its various modes and applications, which must be configured broad enough to capture the range of feasible situations according to the design requirements. In this study, different possible building heights were considered by defining 1-, 3- and 5-stories archetypes. Taller buildings associated with long-period region of the design spectrum were not investigated due to the recommendations that limit the application of TOBF to low-rise buildings. The value of 3.2 m was considered for the height of the stories. Also, for all the archetypes, the length of the bay was assumed to be a constant value (5m). It was assumed that the tension-only braces do not receive any gravitational loading and resist all the seismic design loads. Therefore, considering the potential of variation in gravity load level was not required as it does not interact with the response of the LFRS.

The AISC Specification for Steel Structures (AISC. 2010) recommends that the slenderness ratio of members designed based on tension should not exceed 300. This limitation was applied using single angle sections as concentric bracing members, which form the only configuration of archetypes. The spectral intensities corresponding to the greatest and lowest values for the highest applicable Seismic Design Category (SDC) are considered by the archetypes in accordance with the FEMA P695 methodology (FEMA P695. 2009). Therefore, they were designed for SDC D_{max} and D_{min} ground motions. Then, the archetypes were collected into performance groups, as shown in Table 1.

The gravity loads were selected similar to those proposed in NIST GCR 10-917-8 report (Kircher *et al.* 2010) for designing steel braced frame archetypes. Floor and roof systems consist of concrete-filled metal decks. The uniformly distributed dead and live loads on each floor were supposed to be 85 psf and 50 psf, respectively. The corresponding values were 67 psf and 20 psf at the roof level, and an average exterior curtain wall weight of 15 psf was also considered in the gravity loading of the archetypes. Assuming no irregularity in plan and elevation, all diaphragms has been considered rigid.

Based on the Table 12-2-1 of ASCE/SEI 7-10 (ASCE. 2010), an R factor of 3.25 is proposed for OCBF systems. Since the TOBF system is likely to be less ductile than the general OCBF system, the trial value of the R factor was initially presumed equal to 2.5. A value of 1.5 was also assumed for the overstrength factor in determining design axial loads on the columns.

The equivalent lateral force method proposed in part 12.8 of ASCE/SEI 7-10 is used to design the archetypes, while the lateral equivalent force mentioned above was conservatively specified by applying the spectra of the design earthquake proposed in FEMA P695. It should be mentioned that the methodology sets the occupancy importance factor equal to 1.0 and uses site coefficients relating to soil Site Class D (stiff soil) uniformly for the design of all archetypes. The key parameters of seismic design for each archetype employed in this study are presented in Table 2. The fundamental periods (T) were calculated by the ASCE/SEI 7-10 equation ($T=C_uT_a$), while the arranged value of the first mode period (T_a) was determined from computer analysis.

Table 2 Design results of each archetype (assuming R=2.5)

Story	Interior Columns	Side Columns	Corner Columns	Columns of Braced Bays		Braces		Beams
				D_{max}	D_{min}	D_{max}	D_{min}	
1-Story Archetype Buildings								
1	IPB100	IPB100	IPB100	IPB120	IPB100	L110×9	L110×6	IPE200
3-Story Archetype Buildings								
1	IPB140	IPB120	IPB100	IPB260	IPB200	L180×20	L120×15	IPE200
2	IPB120	IPB100	IPB100	IPB200	IPB160	L150×18	L120×12	IPE240
3	IPB100	IPB100	IPB100	IPB140	IPB120	L120×13	L110×8	IPE240
5-Story Archetype Buildings								
1	IPB180	IPB140	IPB100	IPB280	IPB280	L150×18	L150×15	IPE200
2	IPB180	IPB140	IPB100	IPB280	IPB280	L150×15	L120×18	IPE240
3	IPB140	IPB120	IPB100	IPB200	IPB180	L150×12	L120×15	IPE240
4	IPB140	IPB120	IPB100	IPB200	IPB180	L120×13	L120×10	IPE240
5	IPB100	IPB100	IPB100	IPB120	IPB100	L110×7	L110×6	IPE240

The design criteria of AISC 360-10 (AISC. 2010) and the associated seismic provisions suggested by AISC 341-10 (AISC. 2010) were applied in the design of steel components. The negligible buckling strength of braces in compression was ignored. Hence, the design was conducted, assuming that all the seismic forces are resisted by braces in tension. Beam to column connections were regarded as simple connections, and the braced frames were supposed to be placed at the building perimeter. Depending on the lateral strength demand and the capacity of available single angle sections, one or two bays were braced as depicted in Figs. 1 and 2 for different archetypes. For all model buildings, the redundancy factor ρ= 1.0 was used, as is commonly the case. Greater values for this factor result in stiffer braces and consequently stiffer structures, which is not guaranteed to exist in all applications of the system.

The designed archetypes were finally checked for maximum permissible story drift defined in ASCE/SEI 7-10 with the assumption that C_d is equal to the R factor. The sections of beam and column members were selected from the standard IPE and IPB parts, respectively. The steel materials for all the members were assumed to be S235 steel with a yield strength of 240 MPa. The column sections changed every 2 stories, while the lightest section satisfying the strength and slenderness requirements were selected for braces in each story to avoid excessive overstrength, which is non-conservative for collapse evaluation. Table 2 shows the design results of the archetypes.

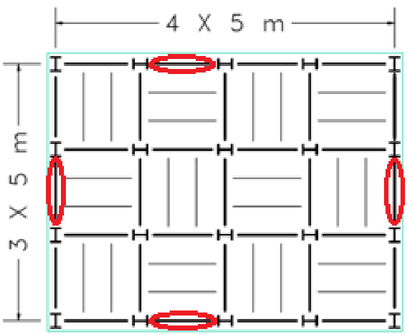

Fig. 1 Plan of archetype buildings except for the 5-story archetype located in SDC Dmax

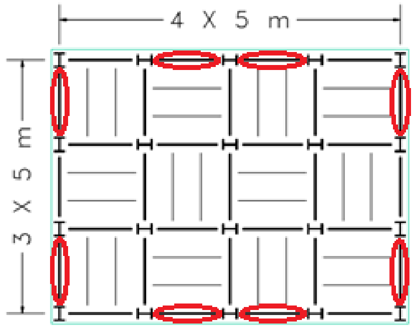

Fig. 2 Plan of the 5-story archetype located in SDC Dmax.

4. Non-linear modeling of archetypes

Implementation of the FEMA P695 methodology involves non-linear modeling of the archetypes. A phenomenological modeling approach was considered in this research, using the non-linear modeling features of SAP2000 v17.3 software. Components were modeled as elastic elements, with all inelastic behavior concentrated in braces, which were simulated as elastoplastic springs.

4.1 Non-linear simulation of tension-only braces

The hysteretic behavior of tension-only braces was modeled through multilinear plastic link elements in SAP2000. In this method, the force-deformation response of each brace is characterized based on the stress-strain curve of the steel material and the dimensions of bracing members. The hysteretic loops of the link elements were assumed to be of the Kinematic Model type, which is deemed appropriate for ductile materials like structural steel. The idealized multilinear diagram which was applied to simulate the non-linear behavior of tension-only braces is shown in Fig. 3. The points A through D on the diagram were defined in accordance with the properties of S235 steel, as summarized in Table 3.

Table 3 Properties of S235 steel

Properties	Value
E	2.0×10^{5} MPa
σ_y (Point A)	240 MPa
ε_y (Point A)	0.00112
ε_{sh} (Point B)	0.015
σ_u (Point C)	360
ε_u (Point C)	0.11
σ_{rup} (Point D)	350
ε_{rup} (Point D)	0.17

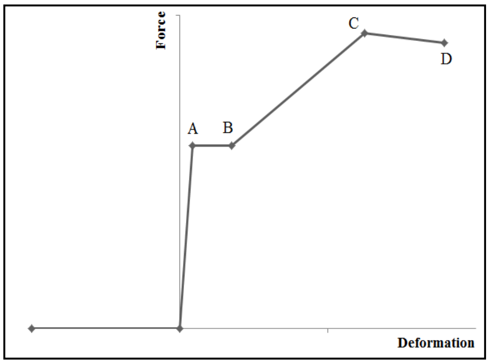

Fig. 3 The general form of tension-only brace force-deformation diagram

4.2 Model validation

The ability of the aforementioned simulation to capture the non-linear behavior of tension-only braces was examined through a comparison of analytical results and the experimental data obtained by (Tremblay and Filiatrault 1996).

According to the seismic simulator table test conducted in their study, a two-dimensional analytical model of the TOBF test frame was prepared in SAP2000. The two pin-ended beams of the test frame were fabricated from HSS 127×76×5 sections, while the section of columns was W250×58.

The link elements were defined based on the properties of bracing members, which consisted of grade 300W steel round tie-rods with a diameter of 12.7 mm. The mass at each story was applied through two equal 15.12 kN point loads, simulating the loading conditions of the test frame. The analytical model is shown in Fig. 4.

Non-linear time history analyses were carried out on the computational model using the Puget Sound and El Centro ground motion records applied during the seismic simulator table experiment (Tremblay and Filiatrault 1996). The ground motion records for analyses were scaled by factors of 1.3 and 1.2 as employed in the seismic simulator table test for the Puget Sound and El Centro accelerograms, respectively. The hysteretic loops of the first floor link elements are plotted and compared to those presented in (Tremblay and Filiatrault 1996), as demonstrated in Fig. 5.

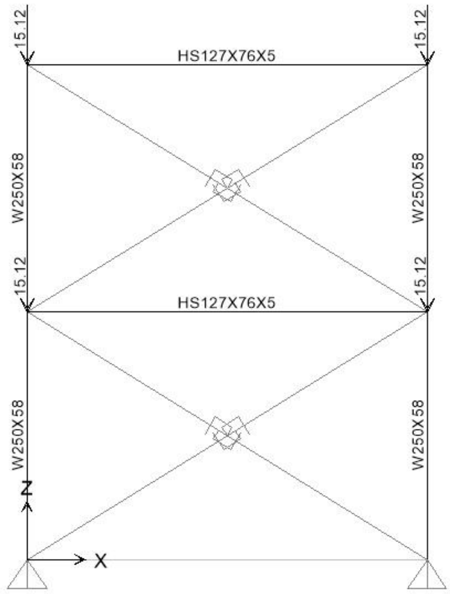

Fig. 4 Two-dimensional analytical model of the test frame

Note that the stress values on the curves have been normalized to the steel yield strength. As can be seen in Fig. 5, the maximum plastic deformations obtained from the time history analyses are acceptably compliant with the corresponding deformations recorded in the seismic simulator table test.

4.3 Non-linear dynamic analyses

The non-linear analyses were performed according to the FEMA P695 provisions with the far-field record set applied to assess the dynamic response of the archetypes. The set includes 22 pairs of horizontal ground motions from sites that were placed greater than or equal to 10 km from fault rupture. The investigations were intended to determine the Median Collapse Capacity ($\hat{S}_{CT}$) in addition to the CMR, which is the principal parameter employed to describe the collapse safety of each archetype. The $\hat{S}_{CT}$ was defined as the intensity of ground motion in which half of the applied records result in the collapse of an archetype model. This intensity can be determined by the concept of IDA in which the ground motions are scaled increasingly until a collapse limit state occurs. It should be mentioned that the FEMA P695 approach does not require a complete IDA since the purpose is just to find the $\hat{S}_{CT}$.

The gravity loads according to the methodology should indicate the values of median service, in contrast to the design gravity loads. Both nonlinear dynamic and static analyses were conducted under the combination of following loads, based on the FEMA suggestion:

$$1.05D+0.25L \tag{1}$$

Where D is the nominal dead load of the structure plus the superimposed dead load and L is the nominal live load.

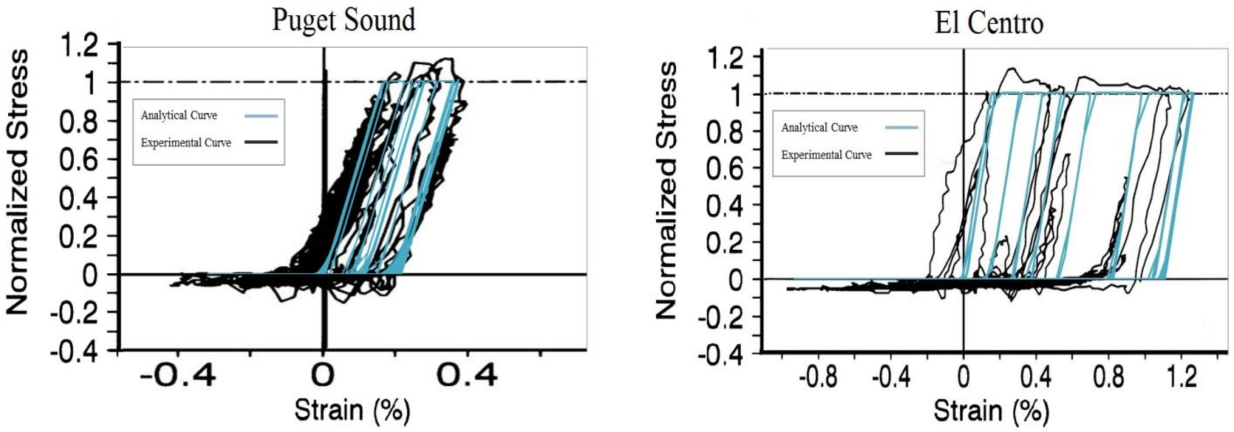

Fig. 5 Analytical and experimental response curves of the test frame subjected to Puget Sound and El Centro ground motions

5. Collapse limit state

If the direct simulation of all deterioration modes is not possible, the collapse behavior can be assessed with the use of alternative limit state control of structural response quantities, which were measured throughout the analyses. Non-simulated limit state checks were considered in accordance with the assessment approach of ASCE/SEI 41-13 (ASCE. 2014) that proposes component acceptance criteria for evaluation of performance targets based on demand quantities. However, this approach improves the uncertainty of analytical results and causes conservative estimations of collapse capacity.

Tension-only braces are ductile when yielding in tension. However, the available test data are not sufficient to validate the behavior of the TOBF system at large deformations. In fact, after intense plastic deformation, failure of connections or other components partly due to second order effects may occur and prompt the collapse of the structure. Thereby, in this paper, the collapse limit was chosen according to the acceptance criteria presented in Table 9-7 of ASCE/SEI 41-13, in which the plastic deformation of single angle braces in tension is limited to $10\Delta_T$ for a Collapse Prevention (CP) performance level, where Δ_T shows the axial deformation at expected tensile yielding load. So, the ductility of all archetypes at collapse was the constant value of $\mu=10$. Considering the bracing frame geometry depicted in Fig.4, the story drift ratio corresponding to the defined collapse point was calculated approximately equal to 2.5% as follows:

$$10\Delta_T = 10 \times 0.00112 \times \sqrt{320^2 + 500^2} = 6.65\, cm$$

$$dx = \frac{6.65}{\cos(\alpha)} = 7.89\, cm \approx 0.025h$$

5.1 Calculation of median collapse intensity

Implementation of the FEMA P695 methodology necessitated two steps of scaling the Far Field record set. The normalization of the individual record was first determined by their peak ground velocity. This process was performed to remove the unwarranted variability between records because of natural variations in the event magnitudes distance to the source, type of source and site conditions, without influencing their frequency content (FEMA P695. 2009). Then, the $\hat{S}_{CT}$ is determined through these normalized records which were integrally scaled to various ground motion intensities. Target scale factors for the set of 44 ground motion records were chosen and altered until 22 collapse events were observed. That scale factor multiplied by the median spectral acceleration of the unscaled records at the fundamental period ($T=C_uT_a$) of the archetype model, gives the median collapse intensity ($\hat{S}_{CT}$). The CMR is described by Eq. (2) as the ratio of median collapse intensity to the Maximum Considered Earthquake (MCE) intensity.

$$CMR = \frac{\hat{S}_{CT}}{S_{MT}} \tag{2}$$

MCE intensity (S_{MT}) is supposed to be equal to the average of acceleration spectral values of MCE ground motions at the fundamental period of each archetype. Table 4 summarizes the obtained results of the nonlinear dynamic analyses.

6. Characterization of uncertainty

Many sources of uncertainty exist in the evaluation process of a seismic resistant system. A system with less uncertainty can attain the safety intended level with a lower safety margin against collapse. The uncertainty of Design Requirement (DR), Modeling uncertainty (MDL), the uncertainty of Test Data (TD), and Record to Record uncertainty (RTR), according to FEMA P695 (FEMA P695. 2009) are the uncertainty sources that are considered in this paper. The parameter of lognormal standard deviation (β) shows the impact of each uncertainty source.

Table 4 Summary of collapse results for for each archetype (assuming R=2.5)

Archetype Name	No. of Stories	SDC	S_{MT} (g)	S_{CT} (g)	CMR
		Performance Group PG-1			
1AngleDmax	1	D_{max}	1.5	2.62	1.75
3AngleDmax	3	D_{max}	1.5	2.97	1.98
5AngleDmax	5	D_{max}	1.5	3.40	2.27
		Performance Group PG-3			
1AngleDmin	1	D_{min}	0.75	1.52	2.03
3AngleDmin	3	D_{min}	0.75	1.82	2.43
5AngleDmin	5	D_{min}	0.512	1.71	3.34

Table 5 System Quality Rating (FEMA P695. 2009)

Completeness, Robustness, and Representation of Characteristics	Confidence and Accuracy		
	High	Medium	Low
High	(A) Superior β=0.10	(B) Good β=0.20	(C) Fair β=0.35
Medium	(B) Good β=0.20	(C) Fair β=0.35	(D) Poor β=0.50
Low	(C) Fair β=0.35	(D) Poor β=0.50	--

The changeability in the archetypes response is explained by the Record to Record uncertainty. The methodology proposes a constant value of β_{RTR}=0.4 for this uncertainty unless a limited ductility exists in the system. Other parameters were obtained based on the quality ratings assigned to each source, as summarized in Table 5.

6.1 Quality rating for design requirements

Since the latest editions of reliable design codes and seismic provisions were considered in this research in the design of archetypes, a rational safety margin was deemed to be provided against unanticipated failure modes. Considering that quality assurance requirements do not adequately regard all the essential aspects of manufacturing and final construction, the completeness and robustness characteristics were rated as Medium.

Utilizing conventional materials with specified properties, strength criteria, stiffness parameters, and design equations supported by experimental data, a high level of confidence was assumed, meaning that designed components will perform as intended. Therefore, the confidence in design requirements was rated as High. In accordance with Table 5, the quality of design requirements is (B) Good, and the quantitative value of β_{DR}=0.2 was assigned to the system as a result.

6.2 Quality rating of test data

The only available experimental data on the behavior of the TOBF system are the seismic simulator table test results carried out by (Tremblay and Filiatrault 1996), and there is a lack of test results on the inelastic behavior of tension-only braces particularly those made from single angles. Nevertheless, using standard materials and conventional components with reliable experimental information and history of use, other important behavior aspects are generally understood; hence, a rating of Medium was assigned to the completeness and robustness characteristics.

The provided experimental information that affects design requirements and analytical modeling was considered moderately reliable as the available test results of the TOBF system are supported by the basic principles of mechanics. The confidence in test results was rated as Medium, so the quality of test data is (C) Fair, and the corresponding value is β_{TD}=0.35, according to Table 5.

6.3 Quality rating of archetype models

Applying the design provisions and code-based limitations for tension-only bracing, the models of archetype support a broad range of building heights, configurations of the system, and design alternatives that are allowed by the design requirements. Thus, the quality of representing collapse characteristics is rated as High.

The onset of yielding and subsequent plastic deformation was captured by the nonlinear model of the TOBF system, but the degrading response was not directly simulated. The collapse point was also evaluated using a non-simulated limit state check that reduces the accuracy and robustness of the nonlinear models to a Low level. Based on these ratings, the quality of archetype models was considered Fair (C) with a value of β_{MDL}=0.35 assigned to the system.

Table 6 Summary of Collapse Performance Evaluation assuming R=2.5

Archetype Name	No. of Stories	SDC	CMR	μ_T	SSF	ACMR	Acceptable ACMR	Pass/Fail
Performance Group PG-1								
1AngleDmax	1	D_{max}	1.75	10	1.33	2.33	1.76	Pass
3AngleDmax	3	D_{max}	1.98	10	1.33	2.63	1.76	Pass
5AngleDmax	5	D_{max}	2.27	10	1.345	3.05	1.76	Pass
Average of Performance Group:						2.67	2.38	Pass
Performance Group PG-3								
1AngleDmin	1	D_{min}	2.03	10	1.14	2.31	1.76	Pass
3AngleDmin	3	D_{min}	2.43	10	1.14	2.77	1.76	Pass
5AngleDmin	5	D_{min}	3.34	10	1.16	3.87	1.76	Pass
Mean of Performance Group:						2.98	2.38	Pass

Table 7 Summary of Collapse Performance Evaluation assuming R=3.0

Archetype Name	No. of Stories	SDC	CMR	μ_T	SSF	ACMR	Acceptable ACMR	Pass/Fail
Performance Group PG-1								
1AngleDmax	1	D_{max}	1.53	10	1.33	2.03	1.76	Pass
3AngleDmax	3	D_{max}	1.76	10	1.33	2.34	1.76	Pass
5AngleDmax	5	D_{max}	2.09	10	1.345	2.81	1.76	Pass
Mean of Performance Group:						2.40	2.38	Pass
Performance Group PG-3								
1AngleDmin	1	D_{min}	2.03	10	1.14	2.31	1.76	Pass
3AngleDmin	3	D_{min}	2.18	10	1.14	2.49	1.76	Pass
5AngleDmin	5	D_{min}	3.14	10	1.16	3.64	1.76	Pass
Mean of Performance Group:						2.81	2.38	Pass

6.4 Uncertainty of total system collapse

The uncertainty of total collapse is determined by the combination of RTR, DR, TD, and MDL uncertainties. Eq. (3) shows the parameter of lognormal standard deviation (β_{TOT}) and provides a value of 0.675 for this research.

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \cong 0.675 \tag{3}$$

7. Performance evaluation

The frequency content of the ground motion record set can considerably affect the collapse capacity. To explain unique attributes of strong ground motions that can affect the seismic response of a structure, the spectral shape factor (SSF) multiplies the CMR to provide an Adjusted Collapse Margin Ratio (ACMR) for all archetypes.

$$ACMR_i = CMR_i \times SSF_i \tag{4}$$

SSF depends on the fundamental period (T), the period-based ductility (μ_T), and the seismic design category. Values of SSF were extracted from Table 7(1a) and Table 7(1b) of FEMA P695 (FEMA P695. 2009), considering a constant value of μ_T =10 for all archetypes.

The main purpose of the performance evaluation process is establishing an acceptably low, yet reasonable, probability of collapse. The methodology suggests limiting the collapse probability due to MCE ground motions to 10% for each performance group, while a limit of 20% should be considered for the acceptability of each archetype. This purpose is achieved when the ACMR of archetypes meet the following criteria:

- the average value of ACMR for each performance group exceeds the acceptable ACMR of 10% collapse probability.

$$\overline{ACMR_i} \geq ACMR_{10\%} \tag{5}$$

- individual values of ACMR for each archetype exceeds the acceptable ACMR of 20% collapse probability

$$ACMR_i \geq ACMR_{20\%} \tag{6}$$

The acceptable ACMRs relying on the uncertainty of total system collapse (β_{TOT}) are prepared by Tables 7-3 of FEMA P695. The values of $ACMR_{10\%}$ and $ACMR_{20\%}$ in this study are equal to 2.38 and 1.76, respectively. Table 6 shows the results of the performance evaluation. The results

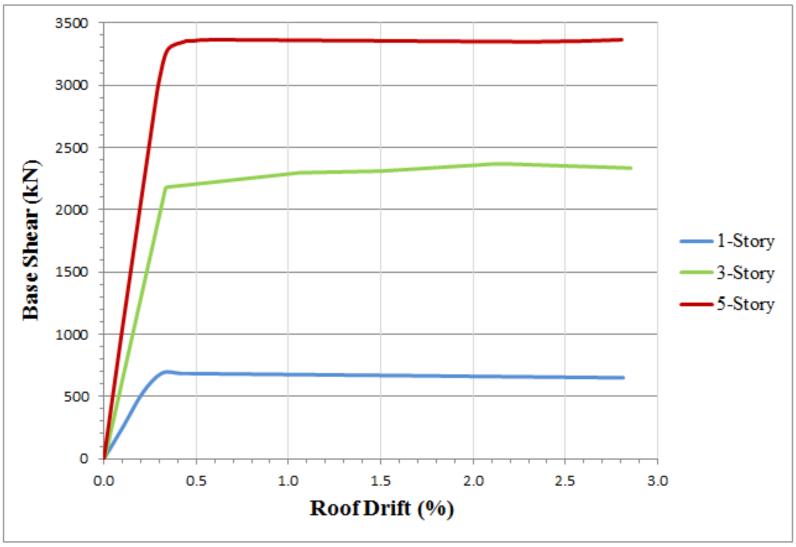

Fig. 6 Pushover Curves for the archetypes of Performance Group PG-1

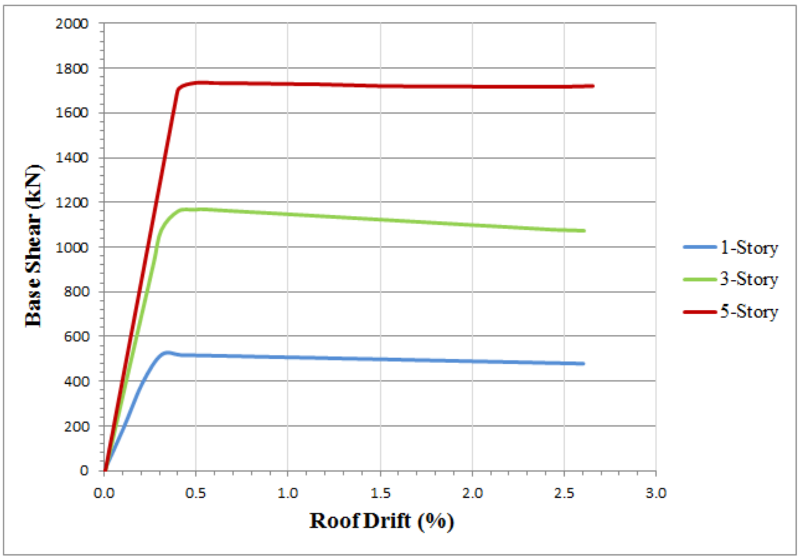

Fig. 7 Pushover Curves for the archetypes of Performance Group PG-3

demonstrate that the acceptance criteria are obtained by all the archetypes and performance groups. Therefore, the presumed factor (R=2.5) could be utilized for TOBF systems; however, the differences between the values of calculated and acceptable ACMRs proposed that the archetypes had been designed conservatively. Thus, the performance of the TOBF system was reevaluated by assuming a higher value of the R factor to achieve a more optimal design. The evaluation process was repeated by assuming an R factor of 3.0 to redesign the archetype buildings. ACMR values of the new archetypes are calculated, and the results of acceptability checks are presented in Table 7.

Since the average ACMR of the governing performance group has been obtained a bit greater than the acceptable ACMR, it can be inferred that the assumed R factor of 3.0 is optimally appropriate for designing TOBF systems.

8. Evaluation of overstrength factor

Nonlinear static (pushover) analyses were conducted on the archetypes designed by assuming R=3.0. First, the factored gravity load combination was applied to the model, according to Eq. (1). Next, the nonlinear static procedure (NSP) of Section 7.4.3 of ASCE/SEI 41-13 was employed to perform pushover analyses. The vertical distribution of the lateral forces at story levels was considered proportional to the fundamental mode shape of the archetype model. The ultimate roof displacement was evaluated by considering predefined non-simulated collapse mode corresponding to 2.5% drift. Pushover curves for index archetypes are illustrated in Figs. 7 and 8. The overstrength factor for a given archetype model was determined as the ratio of the maximum base shear capacity (V_{max}) from the pushover curve divided to the design base shear (V).

Table 8 Summary of collapse results for each archetype (assuming R=2.5)

Archetype Name	No. of Stories	SDC	V_{max} (kN)	V (kN)	Ω_0
Performance Group PG-1					
1AngleDmax	1	D_{max}	687.6	474.8	1.45
3AngleDmax	3	D_{max}	2373.6	1758.9	1.35
5AngleDmax	5	D_{max}	3338.4	2342.4	1.42
Mean of Performance Group:					1.41
Performance Group PG-3					
1AngleDmin	1	D_{min}	519	238.6	2.17
3AngleDmin	3	D_{min}	879.8	1166.3	1.32
5AngleDmin	5	D_{min}	1030.9	1770.5	1.72
Mean of Performance Group:					1.73

$$\Omega_0 = V_{max} / V \tag{7}$$

The methodology submits the greatest mean value of the overstrength factors (Ω_0) computed for each performance group as system overstrength factor (Ω_0). This factor should be conservatively rounded to half unit intervals (FEMA P695. 2009).

Table 8 shows the overstrength values for individual archetypes and average values for various performance groups. The resulting value of Ω_0 is equal to 1.74 which can be rounded to 2.0.

9. Conclusions

Seismic performance factors for tension-only braced frames were evaluated in this study, based on the FEMA-P695 methodology (FEMA P695. 2009). A set of 6 archetype buildings were investigated, including 1-, 3- and 5-story structures designed for seismic design categories of D_{max} and D_{min} to cover the possible range of TOBF applications.

The margin of safety against collapse was determined for each archetype under a set of predefined ground motions proposed by FEMA-P695. The CMRs were then compared to the acceptable values depending on the target collapse probability and the estimated total system uncertainty. The described iterative process was performed twice to assess different R factors of 2.5 and 3.0 as appropriate for all archetypes. Finally, nonlinear static analyses were performed to calculate the system overstrength factor. The following concluding remarks are made from the results:

1) The results of performance evaluation indicate that an R factor of 3.0 is appropriate to achieve an optimal design of Tension-Only Concentrically Braced Frames while providing an acceptable safety margin against collapse.
2) The CMRs had an upward trend as the number of stories increases, revealing that low rise TOBFs is more vulnerable. 5-Story archetypes in each performance group demonstrated the largest CMR values, which make them dominant in the calculation of average CMR.
3) The deflection amplification factor (C_d) is estimated according to the accepted value of being equal to R factor. Table 18.6-1 of ASCE/SEI 7-10 indicates a relation of C_d=R, which can be used when the inherent damping is assumed equal to 5%. Therefore, the deflection amplification factor is suggested to be equal to 3.0.
4) A value of 2.0 should be considered as the system overstrength factor (Ω_0), which is governed by the archetypes designed for SDC D_{min}. The design results of the 1-story archetype located in SDC D_{min} did not change when R was increased from 2.5 to 3.0. The reason is that the slenderness limitation is governing the brace design of this archetype, which has the largest value of the overstrength factor.

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