Seismic Performance Evaluation and Retrofit of Fixed Jacket Offshore Platform Structures

Mohamed Nour El-Din[1] and Jinkoo Kim[2]

Abstract: In this study the seismic performances of the fixed-base jacket structures with various bracing configurations were evaluated by nonlinear static and dynamic methods, and the effects of various retrofit schemes were compared. It was observed that the conventional retrofit methods of increasing member size were somewhat effective in increasing the strength of fixed steel jacket platform structures but were not so effective in increasing ductility. However, the ductility of the structures retrofitted with buckling-restrained braces turned out to be increased significantly. In the structures with conventional bracing, plastic hinges or buckling generally occurred at the upper half of the jacket structures. On the contrary, they were more uniformly distributed along the structure height in the structure retrofitted using buckling restrained braces. The capacity curves obtained from incremental dynamic analysis generally corresponded well with those obtained from nonlinear static analysis using a lateral load pattern proportional to the first mode shape of the jacket structure. **DOI: 10.1061/(ASCE)CF.1943-5509.0000576.**

Author keywords: Seismic performance; BRB; Offshore structures; Incremental dynamic analyses.

Introduction

Recently, many offshore platforms have been built in seismically active regions worldwide. Especially the fixed steel jacket offshore platforms (FSJOPs) have been extensively employed to support offshore platforms in different locations of the world (Rodrigues and Jacob 2005; Ali et al. 2009). The steel jacket structures were originated in the Gulf of Mexico and spread worldwide as typical shapes of fixed offshore platforms. There are more than 6,700 platforms in operation worldwide, and 30% of them have been in operation for more than 20 years (Nabavian and Morshed 2010). Many of these platforms are operating beyond their original design life. As more oil and gas reservoirs have been discovered, the operation life of the existing offshore platforms should be extended by retrofit. FSJOPs have been generally designed for harsh marine environments including strong tidal forces, hurricanes, storms, ice load, etc., and in many cases the seismic force has not been considered as a primary design load. In order to reduce the possible damage and to improve the ductile behavior of FSJOPs not designed against strong earthquakes, it is of importance to retrofit existing offshore structures.

This paper compared the performance of an operational jacket structure retrofitted with buckling restrained braces (BRBs) and with the conventional retrofitting techniques commonly used in practice. Nonlinear static analysis and incremental dynamic analysis were conducted to predict the seismic performance of the structures. Parametric studies were carried out through nonlinear static and dynamic analyses to identify the different behavior and responses of the structures analyzed.

[1]Lead Engineer, Samsung Engineering Co., Sangil-dong, Gangdong-gu, Seoul 134-090, Korea.

[2]Professor, Dept. Civil and Architectural Engineering, Sungkyunkwan Univ., Chunchun-dong, Jangan-gu, Suwon 440-746, Korea (corresponding author). E-mail: jkim12@skku.edu

Note. This manuscript was submitted on September 3, 2013; approved on January 7, 2014; published online on January 9, 2014. Discussion period open until January 26, 2015; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Performance of Constructed Facilities***, © ASCE, ISSN 0887-3828/04014099(10)/$25.00.

Design Philosophy and Acceptance Criteria of Offshore Structures

Studies on the response assessment of fixed jacket structures are relatively rare and are focused primarily on jacket structures with conventional braces. Some of these studies were concerned with describing the assessment process of existing platforms (Krieger et al. 1994). Other studies assessed performance of structure and foundation of jacket platforms against metocean loads (Petrauskas et al. 1994). Assessment criteria for various loading conditions were investigated in Ali et al. (2009). Komachi et al. (2011) investigated the effect of friction damper devices for vibration control of an existing steel jacket platform under seismic excitation.

One of the main seismic design requirements for fixed offshore structures, according to API RP 2A code, is to allow structural damage while the total platform collapse is prevented under strong earthquakes. This is dictated primarily by economic considerations, since an elastic response under severe earthquakes would be excessive in cost (McClelland and Reifel 1996). Current American Petroleum Institute recommended practices (API RPs) (2000) are intended to provide a platform that is adequately designed with enough strength and stiffness to ensure no significant structural damage for the level of earthquake shaking which has a reasonable likelihood of not being exceeded during the life of the structure. The ductility requirements are intended to ensure that the platform has sufficient reserve capacity to prevent its collapse during strong earthquake motions, although structural damage may occur (API 2000).

The API RP 2A is one of the most important and useful standards for the design and assessment of offshore structures. Section 17 of this standard has recommendations for the assessment of offshore structures. The assessment criteria required to satisfy the collapse prevention limit state of the structure under extreme earthquake conditions. This standard describes global rehabilitation objectives and does not present a routine methodology for seismic retrofit. The API is currently developing recommendations for the assessment of existing platforms including requirements for platforms subjected to hurricanes, storms, earthquakes, and ice load. These recommendations will likely focus on a demonstration of

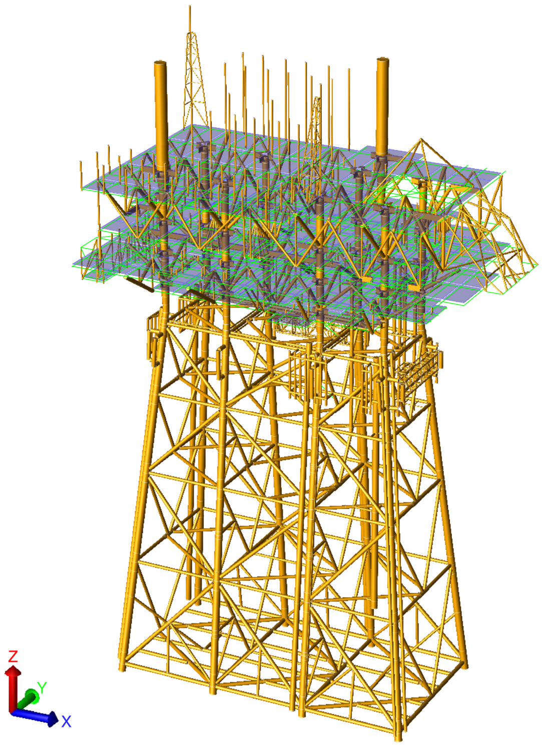

Fig. 1. Fixed steel jacket offshore platform structure

adequate ductility for platforms located in earthquake-dominated regions (O'Connor et al. 2005). The focus toward ductility under extreme earthquake conditions is based on the objective of prevention of loss of life and pollution. The performance criterion for assessment is essentially identical to that of the design level earthquake (DLE) requirement for new designs. The structures need to match one of two sets of global structural performance criteria, depending on the platform's exposure category. In addition, local structural performance requirements for topside equipment and appurtenances must be checked independently of the platform's exposure category classification. In the case of high-exposure platforms, the structures need to be verified by nonlinear analysis procedures (pushover or time history procedures) to ensure good seismic behavior under median ground motions representative of an earthquake with a return period of 1,000 years. For lower-exposure platforms located in areas with high seismic activity, a return period of 500 years must be selected. The ASCE 41-06 (ASCE 2007) provides modeling parameters and numerical acceptance criteria for beams, columns, and braces as a function of parameters such as the diameter-to-thickness ratio. Since API-RP2A does not provide detailed guidelines for the failure and acceptance criteria for individual elements, the ASCE 41-06 criteria were used for braces with circular hollow section. Fig. 1 shows the typical configuration of a fixed steel jacket offshore platform structure in the Gulf of Thailand.

Seismic Performance of a Steel Jacket Offshore Platform Structure

Design of Model Structures

In this section, seismic performance of steel jacket structures was investigated using nonlinear static analysis. Four case studies representing the most common jacket types (Chakrabarti 2005) were considered for comparison. The structures with V- and X- bracing and two types of diagonal bracing (K and N types) are shown in Fig. 2, while Fig. 3 shows the dimensions of the model structures. Structural analysis and design of the platform structures were carried out using the software program *SAP2000* (2005) according to the API-RB2A WSD-2000. The design loads are gravity, wind, wave, and current loads, which are generally used to design offshore structures in the Gulf of Thailand. The region is considered to be a low seismic region; therefore, seismic load was not included in the design of the model structures. The platform structural model includes the deck, jacket, and its appurtenances. The platform has topside with four stories and a four-story jacket with total mass of about 13,800 tons located in the main nodes of the jacket. The appurtenances include the nonstructural members such as flooding system, centralizer, pad-eyes, plates and stiffeners, etc. Only the major structural components were included within the model, and the contribution of conductors to the platforms' stiffness and strength were neglected. The jacket horizontal members are frame elements rigidly connected at the ends. The added mass, the fluid enclosed within the structure, and the marine growth, have been considered in the design along with the gravity and metocean loads. Table 1 shows the dimensions and mass of the model structure.

The model structures were designed with three different configurations. In the first case conventional steel braces were used to carry all lateral loads such as wind, wave, and current loads according to the API-RB2A. In the second case all the bracing members were designed with BRBs. Fig. 4 shows the typical configuration of a BRB in which a core element is enclosed in a buckling restrainer and therefore yields in both tension and compression (Xie 2005). In the third design scheme, the conventional braces

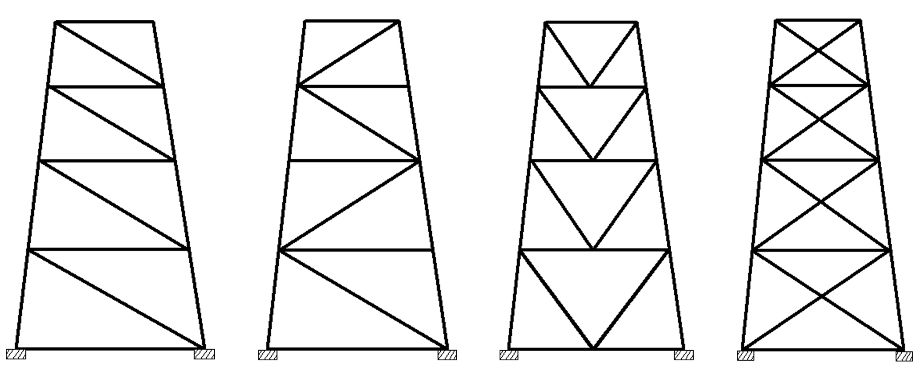

Fig. 2. Model structures used for the seismic performance evaluation: (a) diagonal (N) bracing; (b) diagonal (K) bracing; (c) V-bracing; (d) X-bracing

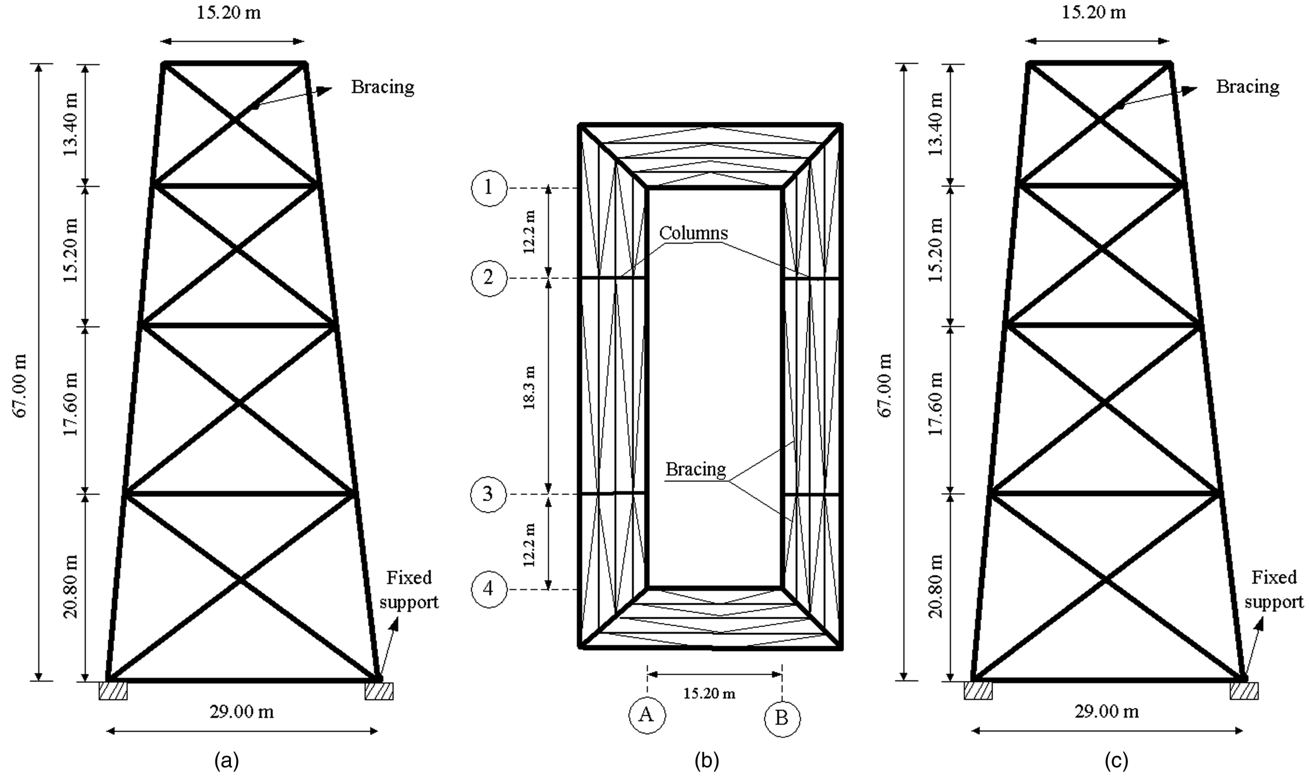

Fig. 3. Dimensions of the model structures: (a) elevation (X bracing); (b) plan view; (c) elevation (V bracing)

located in the upper two stories of the model structures were reinforced (wrapped) with buckling-restraining elements to prevent global buckling of the conventional braces. This scheme was shown to be effective by Lee and Kyriakides (2004) who demonstrated that collapse resistance of steel pipe could be enhanced by using slip-on buckle arrestors. In all schemes the braces are made of tubular sections that are commonly used in practice for corrosion resistance in an ocean environment. In this case the third design scheme may only be applicable for tubular sections with small width–thickness ratio and thus with high resistance against local buckling. The cross sections of the structural members are shown in Table 2, and Table 3 shows the natural periods of the two-dimensional (2D) model platform structures with V-type bracing.

FSOJPs are commonly constructed or fabricated on onshore yards like typical modular steel structures. Therefore, it is feasible to use BRBs in off-shore steel structures. The typical configurations of BRB using pipe sections are presented in Uang et al. (2004) and Xie (2005). Also, there are some new BRB configurations that may be suitable for tubular elements of offshore jacket structures such as proposed by Yin et al. (2009). They proposed a double-tube buckling restrained brace with contact ring. In this configuration some discourteous steel rings are set between the inner tube and the outer tube, namely, the contact rings. The lateral confinements

Table 1. Geometrical Dimension of the Platform

Item	Description
Water depth	62.92 m
Jacket height	67 m
Jacket plan dimensions	15.2 × 42.7 m
Total numbers of jacket legs	8
Total mass	13,800 ton

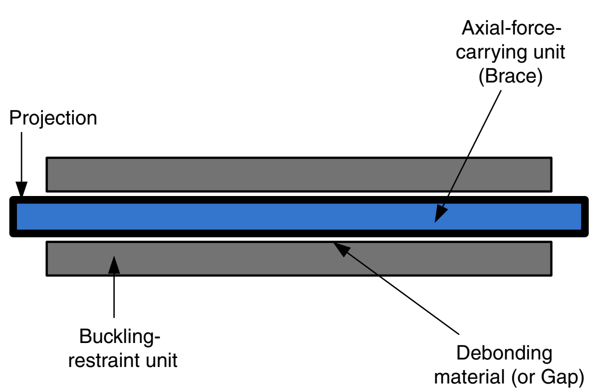

Fig. 4. Composition of typical buckling-restrained brace

Table 2. Cross Sections of Bracings in Model Structures (unit: in.)

Bracing type		"Conventional" and "BRB retrofit" cases			
		X-bracing	V-bracing	N-bracing	K-bracing
Story	1	24 × 0.725	24 × 0.725	24 × 1.5	24 × 1.5
	2	24 × 0.625	24 × 0.625	24 × 1.25	24 × 1.25
	3[a]	24 × 0.5	24 × 0.5	24 × 1.0	24 × 1.0
	4[a]	24 × 0.5	24 × 0.5	24 × 1.0	24 × 1.0
		"BRB" case[b]			
	1	24 × 0.5	24 × 0.5	24 × 1.0	24 × 1.0
	2	24 × 0.5	24 × 0.5	24 × 1.0	24 × 1.0
	3	24 × 0.5	24 × 0.5	24 × 1.0	24 × 1.0
	4	24 × 0.5	24 × 0.5	24 × 1.0	24 × 1.0

Note: In all types of model structures, all horizontal elements and columns have the tubular cross section of 24 × 0.625 and 54 × 1.0 in., respectively.
[a]In the "BRB Retrofit" case, BRB replaces normal bracing with the same section.
[b]In "BRB" case, BRBs replace all normal bracings.

Table 3. First Four Modal Periods and Characteristics of the Conventional *V*-Type 2D Model Extracted from the Platform Structure

Mode	Modal period (s)	Modal participating mass (for each mode) (%)	Modal participating mass (cumulative) (%)
1	1.77	97.0	97.0
2	0.24	0.0	97.0
3	0.141	1.7	98.7
4	0.072	0.2	98.9

are provided by the contact rings to improve the behavior of the component under compression. Such a scheme may easily be applied to both new and existing offshore structures for providing restraint against buckling. Retrofitting a conventional pipe section into a BRB can happen using pipeline buckle arrestors (Lee and Kyriakides 2004) which are devices locally protecting an offshore pipeline and safeguard it against the effects of a potential buckling. They can play the role of the buckling-restraining unit with some modifications to be suitable for tubular braces in jacket platform structures. A separation unit between brace and buckling-restraining unit should be maintained to ensure that the brace can slide freely inside the sleeve and that the lateral deformation of the brace can take place when the brace yields in compression. This typically requires some debonding materials to be employed as filler, or a gap should be kept between the two units.

Modeling of the Structures

The platform structural model selected for analysis includes the deck, the jacket, and its appurtenances and eight piles. Member lengths were measured from node to node. Design of the platform elements was carried out according to the API-RP2A considering the added mass, the fluid enclosed within the structure, and the marine growth along with the dynamic mass using the program SACS (2009). The nonlinear analyses were carried out using *SAP2000* based on a 2D frame model structure extracted from the original three-dimensional (3D) platform.

In the nonlinear static analysis the structure stiffness is updated every time there is a significant change in the stiffness of any element. Fig. 5 shows the force-deformation relationship of the conventional braces and buckling-restrained braces. The postyield stiffness of structural elements was assumed to be 3% of the initial stiffness. The structural steel with minimum yield stress of 345 MPa was used in the design, which is commonly used in practice for fixed steel jacket platforms.

Nonlinear Static Analysis Results

The seismic performance of the model structures for a given lateral load pattern was evaluated by a nonlinear static analysis procedure carried out using the *SAP2000* software with the lateral load pattern proportional to the fundamental vibration mode shape. The first modal mass participation factor of the 2D model structure was

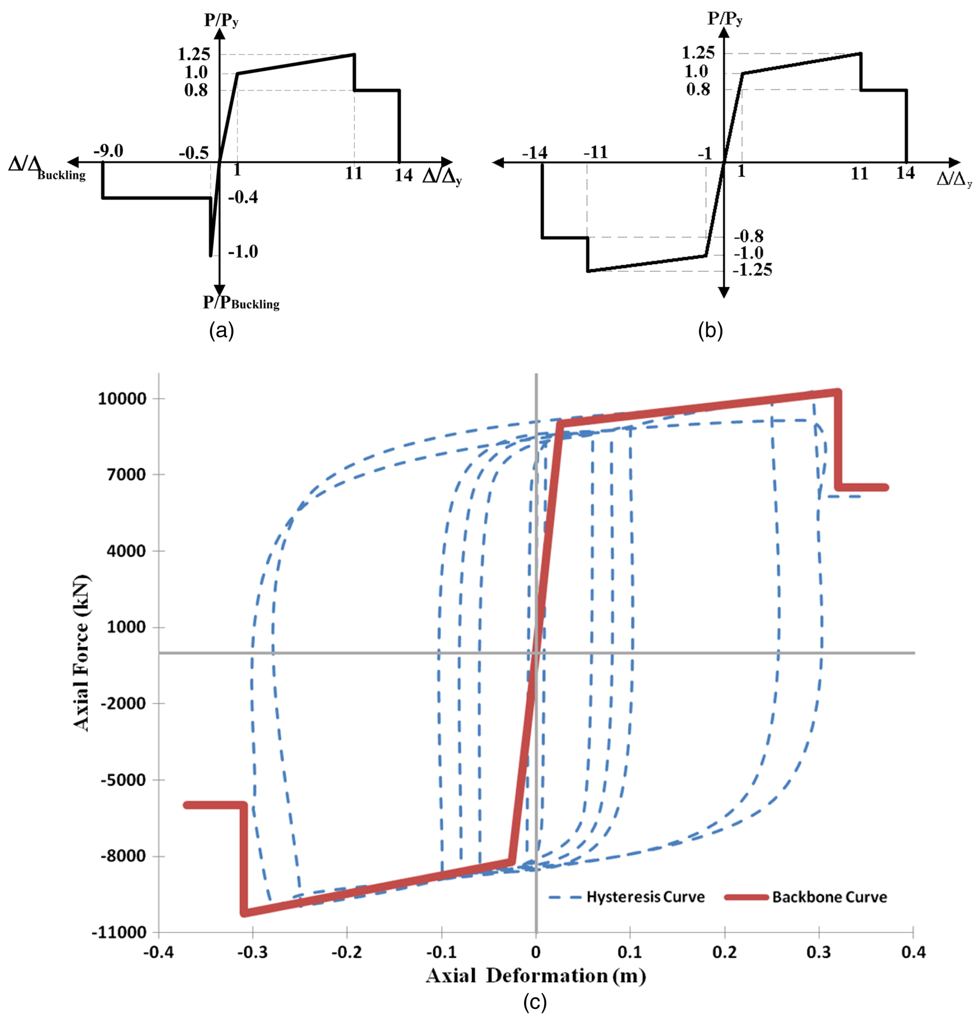

Fig. 5. Force-deformation relationship of a plastic hinge defined in *SAP2000*: (a) conventional brace; (b) BRB; (c) hysteresis curve for BRB

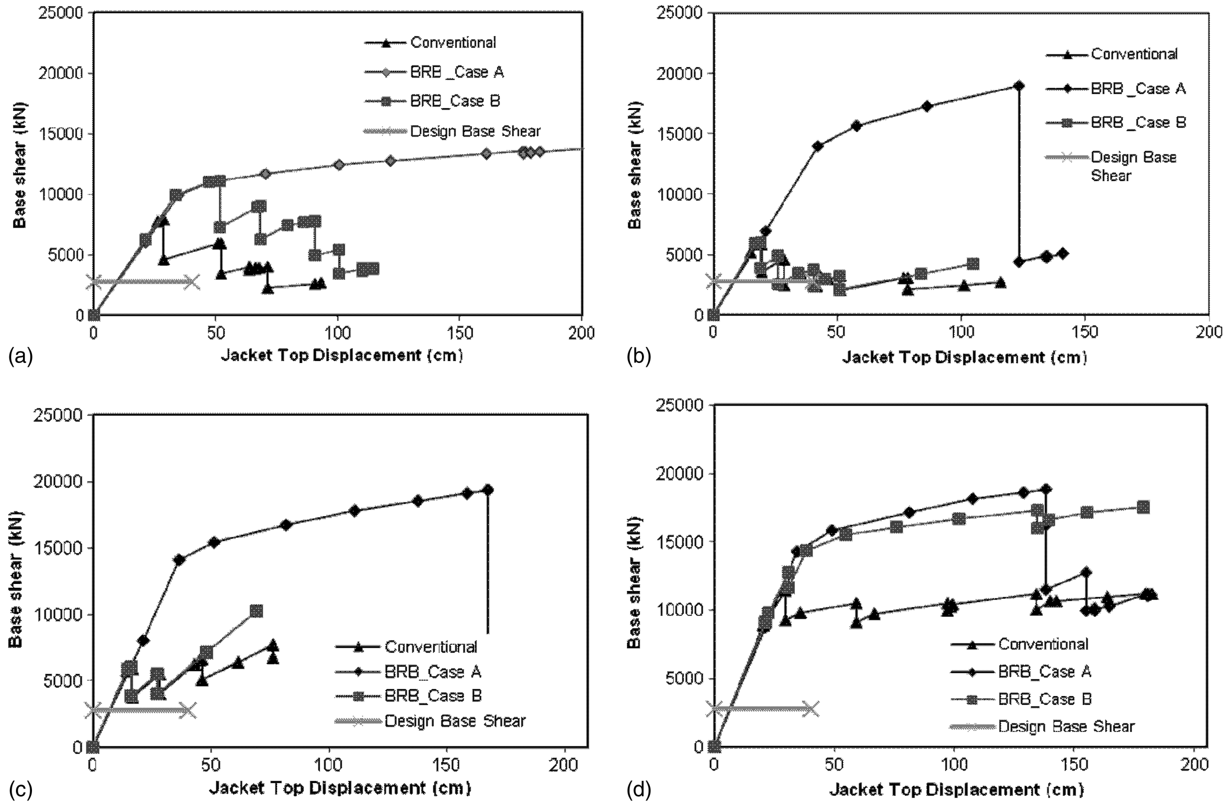

Fig. 6. Pushover curves of the model structures: (a) jacket with V-bracing; (b) jacket with N-bracing; (c) jacket with K-bracing; (d) jacket with X-bracing

found to be 0.97, which justifies the use of fundamental vibration mode shape, which has a fundamental natural period of 1.77 s, as a basis for the lateral load pattern in the nonlinear static analysis. The gravity loads of the top side structure were applied to the upper two joints of the jacket. The maximum top displacement was different for each jacket structure depending on the bracing type. Fig. 6 depicts the pushover curves of the four model structures. The maximum strengths of the structures turned out to be higher than the design base shear required by the API-RB2A. For each type of bracing configuration, there are three pushover curves obtained. The curve denoted as "Conventional" depicts the behavior of the jacket using normal bracing as designed according to the API-RP2A. The curve denoted by "BRB_Case A" denotes the behavior of the same jacket structure but designed with BRBs instead of conventional steel braces. The curve denoted as "BRB_Case B" shows the force-displacement relationship of the structure retrofitted with the BRBs having the same cross-sectional area as the conventional braces for the third- and the fourth-story braces only. In the case of the jacket structures with V-bracing, shown in Fig. 6(a), the initial stiffnesses of the structures with three different types of design schemes were almost the same until the top displacement reached 33 cm. After that point, the pushover curves of the BRB_Case A and the BRB_Case B are almost the same until the top displacement reached 52 cm, where the BRB_Case B curve dropped suddenly and the BRB_Case A curve continued to increase until 3% of the jacket height was reached. In Fig. 6 it can be observed that the maximum strength was obtained in the structures with X-type bracing, while the strength was lowest in the structures with two types of diagonal bracing. The increase in the strength due to using BRB (BRB_Case A) instead of using conventional braces was found to be highest in the structures with K-type and N-type bracing with almost threefold increase in strength. For structures with V-type and X-type bracing, the increase in strength was found to be 41 and 61%, respectively. It also can be observed that the model structures with conventional bracing showed poor ductile behavior when compared to those of the structures with BRB. In the current study, the ductile behavior was measured by using the ductility factor which was defined as the ratio of the maximum lateral displacement at the top of the jacket to the lateral displacement at which yielding started ($\mu = \Delta_{\max}/\Delta_y$). In the jacket structures with BRB, the ductility factors turned out to be significantly higher than those of the structures with conventional braces. Maximum ductility was observed in the structures with BRB_Case A bracing. The maximum ductility factor (μ) was found to be $\mu = 6.15$ in the V-type jacket, while for X, N, and K types the ductility factors were 4, 2.9, and 4.6, respectively. The V-type model showed the largest ductile behavior before a local failure mechanism occurred at the top story. Also, it is found that the plastic hinges were distributed uniformly over the members of the upper three stories. This means that both the number and the distribution of the plastic hinges play an important role in maintaining ductile behavior for the jacket structure. However, in the case of the N-type jacket, there was extensive concentration of damage in the brace elements before local failure mechanism occurred in the top story.

Fig. 7 shows the capacity curves of the structures with V-type bracing with indications of various stages of plastic deformation, and Fig. 8 depicts the formation of plastic hinges/buckled members at a level of displacement equal to 1.5% of the jacket total height (IO, immediate occupancy; LS, life safety; CP, collapse prevention). In the structure with conventional bracing, strength abruptly dropped after buckling of some braces. The strength further decreased due to formation of plastic hinges at horizontal members. The upper two braces under compression buckled and the upper three horizontal elements yielded. No yielding of tension braces

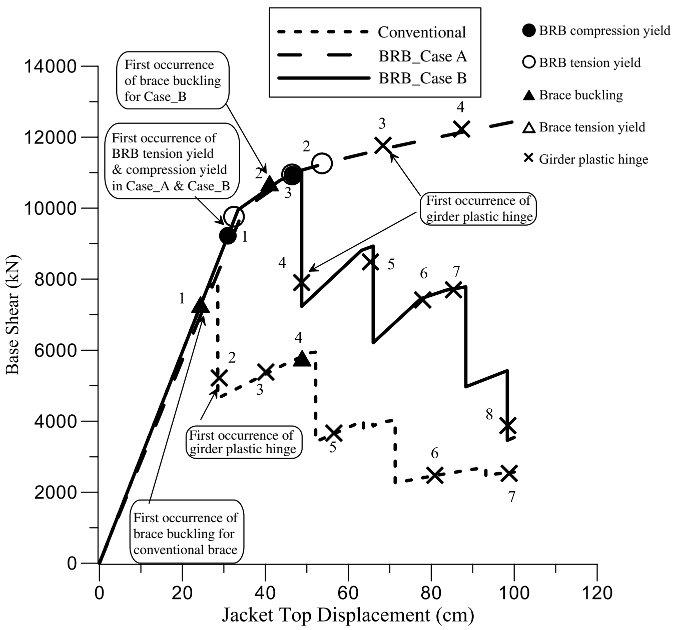

Fig. 7. Capacity curves of the structures with V-type bracing showing various stages of plastic deformation

was observed. These resulted in brittle behavior of the structure. In the model structure designed with BRB (Case A), the stiffness started to decrease after the BRB at the top story yielded followed by the yielding of the BRB located in the third story. Finally, plastic hinges occurred at horizontal elements located in the third and the fourth stories, respectively. In this case, the distribution pattern for plastic hinges changed as a result of the use of BRBs (i.e., the inelastic deformation spread uniformly throughout the upper half of the jacket compared with the case with conventional braces). When the bracing in the upper part of the structure was retrofitted into BRBs (BRB_Case B), the behavior was similar to that of the structure designed with BRBs (BRB_Case A) until the top displacement reached 50 cm, except that the stiffness and strength were slightly higher. This is reasonable considering that the cross-sectional areas of the bracing members retrofitted into BRB are higher than those of the structure originally designed with BRBs. Also, it is noticed that the plastic hinges were found to be uniformly distributed through the height of the jacket when compared to the plastic hinge formation in the structure with conventional braces. BRB_Case A showed a more ductile behavior associated with yielding of braces and horizontal members when compared to BRB_Case B or the conventional brace case.

Fig. 9 shows the interstory drifts of the model structures at a point on the pushover curve corresponding to the top lateral displacement of 1% of the jacket height which is 67 cm. Even though no distinct pattern was observed, the interstory drifts of the structure with BRB_Case A were relatively uniformly distributed compared with those of the structure with conventional braces. In the structures with V and X bracing, the maximum interstory drifts generally occurred at the top story, whereas in the structures with N and K bracing, the maximum drift occurred at the second story and the first story, respectively. In all cases the maximum interstory drift occurred in the structure with conventional braces except for the K-bracing case where the maximum occurred in the BRB_Case B.

Comparison of Different Retrofit Schemes

Nonlinear Static Analysis Results

In this section the performance of various retrofit schemes applied to the structure with V-type bracing was investigated by pushover analysis using the lateral load pattern proportional to the fundamental mode shape of the structure. The analysis model structure

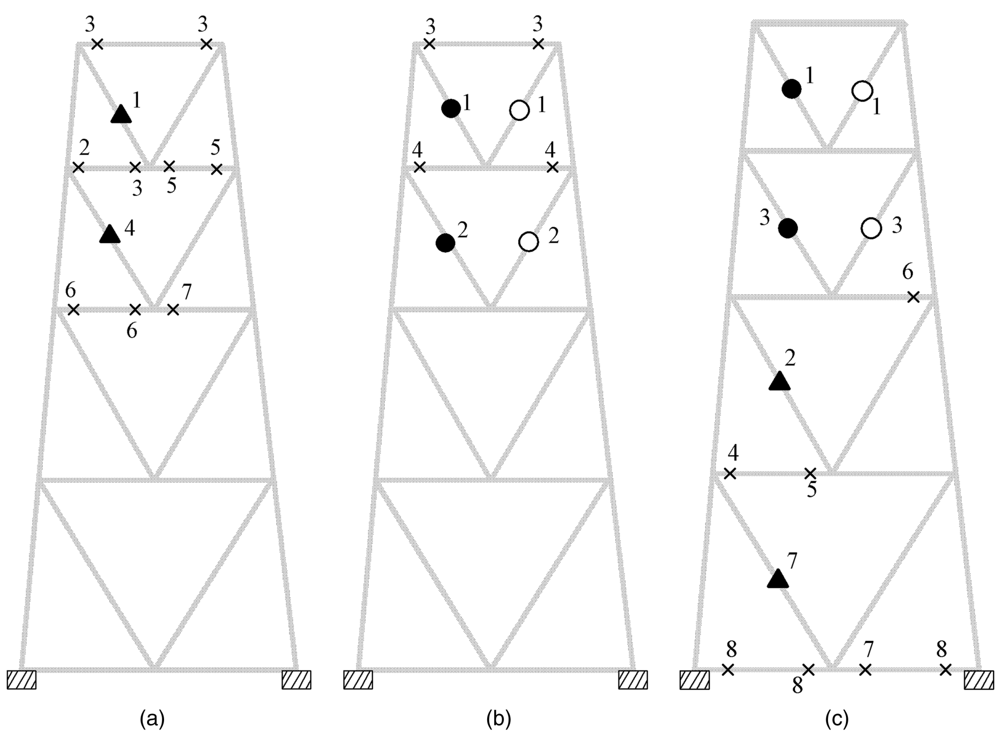

Fig. 8. Plastic hinge formation in the structures with V-type bracing (at the maximum displacement of 1.5% of the jacket height): (a) conventional; (b) BRB Case A; (c) BRB Case B

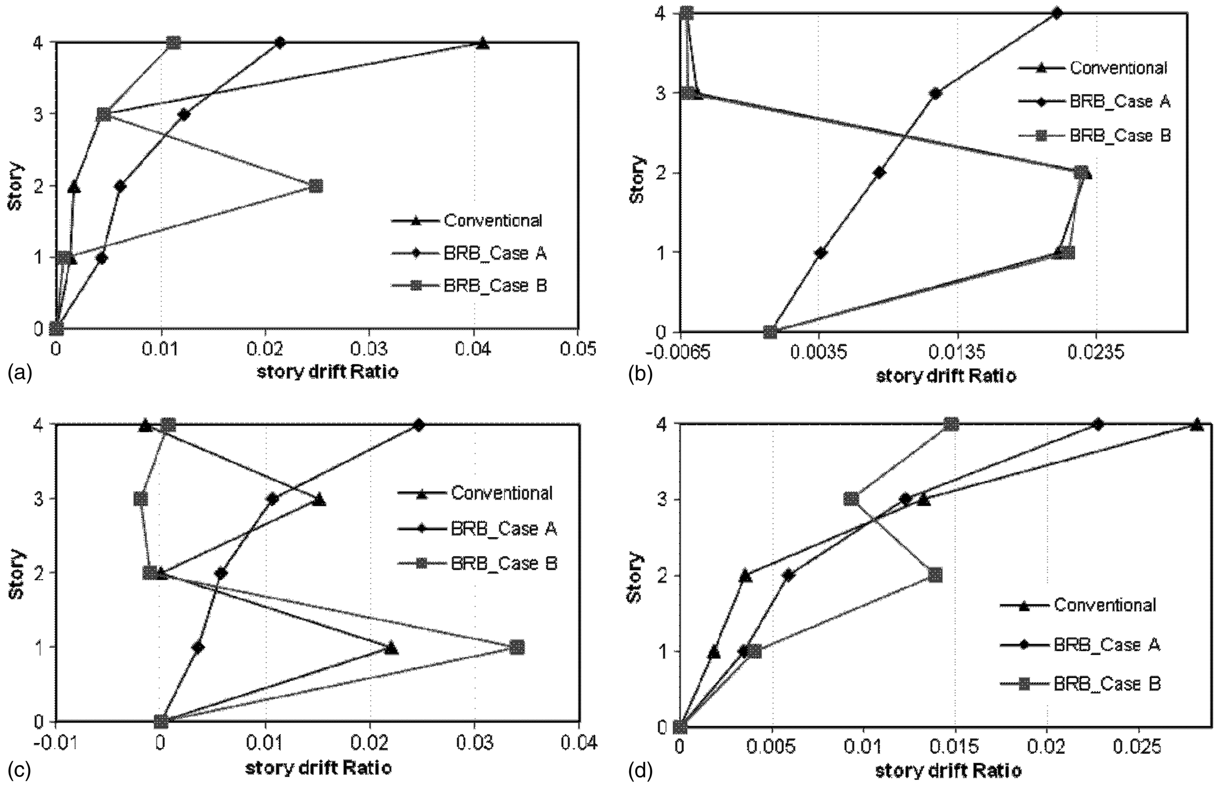

Fig. 9. Interstory drift ratios for model structures (at the maximum displacement of 1% of the jacket height): (a) jacket with *V*-bracing; (b) jacket with *N*-bracing; (c) jacket with *K*-bracing; (d) jacket with *X*-bracing

(Fig. 3) is a 2D braced frame located in the short direction of the steel jacket. The first retrofitting scheme is to increase the thickness of the horizontal elements at the bracing joints. This technique is commonly used in practice for newly designed platforms to satisfy the punching shear strength requirement of design codes. In this technique, the diameter or the thickness of the horizontal element is increased over a specific length starting from the intersection of the bracing and the horizontal element. In the model structure, the length with increased section is 3.4 m as shown in Fig. 10. According to API-RP2A-2000 (Item 4.3.1.c) design practice, the increased wall thickness in the chord at the joint should be extended over the outside edge of the bracing by considering the greatest value between one-quarter of the chord diameter or 12 in. (305 mm) including taper. This criterion was used to evaluate the length of the increased cross section. In this region the diameter of the horizontal members was increased from 24″ to 30″. To increase cross-section thickness, an arbitrary thickness is first assumed, then the joint allowable capacity is calculated using equation 4.3-4 in API-RP2A-2005. Another retrofit scheme is to add a diaphragm at each story of

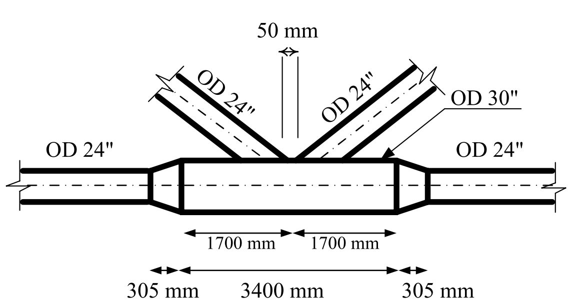

Fig. 10. Jacket structure with *V*-type bracing

the jacket. This can be achieved in practice by introducing horizontal bracing between columns at each story level. To simulate such an effect in the 2D model in *SAP2000*, a special type of constraint for the joints located at the same level was assigned. This constraint causes all of its constrained joints to move together as a planar diaphragm that is rigid against membrane deformation. Effectively, all constrained joints are connected to each other by links that are rigid in the plane but do not affect the out-of-plane deformation. The third technique is to increase the cross section of the critical brace elements. In particular, the sections of the brace elements located in the upper two stories were increased by 25 and 45% of the original cross-sectional area, because deformation in these stories contributes to the seismic behavior of the platform significantly. The performances of the structures retrofitted with the above three conventional retrofitting schemes were compared with the performance of the structure with BRB.

Fig. 11 shows the pushover curves of the structures retrofitted according to the different methodologies described above. It can be observed that both the "adding diaphragm" and the "increasing horizontal elements" schemes did not improve the seismic performance of the jacket significantly. As the cross-sectional area increased, the strength also increased, and the increase in the cross-sectional area of braces turned out to be effective in increasing the strength of the structure to some extent. However, the improvements in the deformation capacity were not significant. When the braces were replaced by BRB the ductility increased significantly the dynamic analysis results.

Fig. 12 shows the plastic hinge formation of the model structure retrofitted with the four methods. The order of plastic hinge formation is also indicated on the curves, and the corresponding loading stages are shown in Fig. 11. It was observed that both the "adding diaphragm" and the "increasing horizontal elements" schemes resulted in the same plastic hinge distribution as the structure without

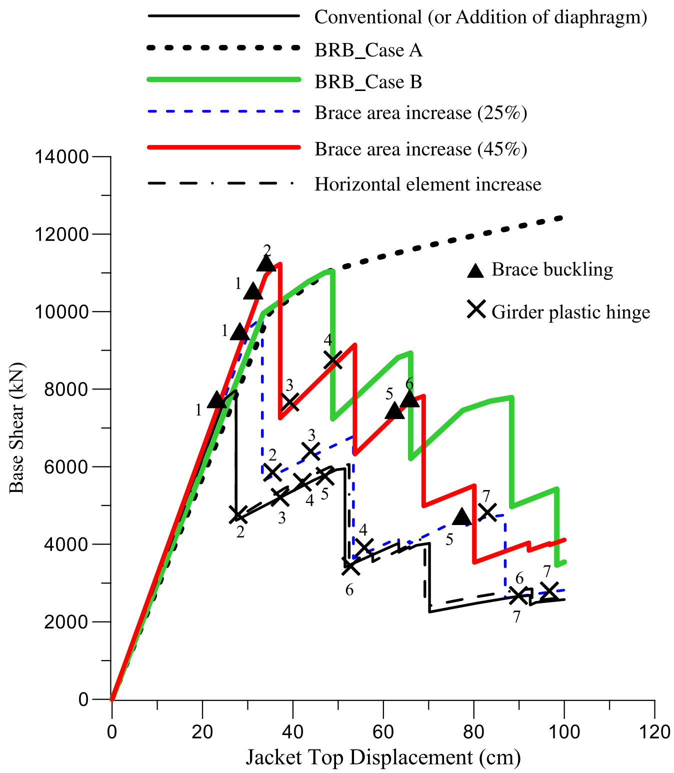

Fig. 11. Capacity curves of the model structures with different retrofit schemes

retrofit. The increase in the cross-sectional area of bracing members by 45% resulted in a more uniform plastic hinge distribution all over the jacket structure without tension yielding of braces. This distribution pattern of plastic hinges is similar to that of the structure with BRB_Case B as shown in Fig. 8. However, the amount of steel tonnage required to retrofit with BRB is significantly smaller than that required for increasing the cross-sectional area of bracing members.

Incremental Dynamic Analysis Results

Incremental dynamic analysis (IDA) is a parametric analysis method to estimate more deeply the dynamic structural performance under seismic loads. The method is also used to assess the applicability of pushover analysis in predicting the overall response of structure. It consists of subjecting a structural model to a ground motion record scaled to multiple levels of intensity, which is the peak ground acceleration in the current study, thus producing a curve of response parameterized versus intensity level (Vamvatsikos and Cornell 2002; Dolsek 2009). Recently, IDA has been implemented as an effective tool for evaluation of seismic performance of steel moment resisting frames (Asgarian et al. 2010), reinforced concrete structures (Kim and Kim 2007), and jacket-type offshore platforms (Asgarian et al. 2008).

In this section, the incremental dynamic analyses were carried out with the jacket structure with V-type conventional and buckling-restrained braces utilizing a suit of ground motion records such as El Centro, Kobe, Northridge, Loma Prieta, and Imperial Valley earthquakes. Table 4 shows the main characteristics of each ground motion record, including the peak ground acceleration (PGA) and duration of the vibration. These records were obtained from the strong motion database of the Pacific Earthquake Engineering Research Center (PEER) at University of California, Berkley. Fig. 13 shows the pseudo-acceleration spectra for all earthquake records assuming 5% damping. Nonlinear time-history analysis was carried out using the aforementioned suit of earthquake records scaled up to different levels to predict the structural responses in both linear and nonlinear range. The force-deformation relationship of plastic hinges defined for steel braces and buckling-restrained braces in this section is the same as those used for nonlinear static analyses. The same modeling parameters have been used for both

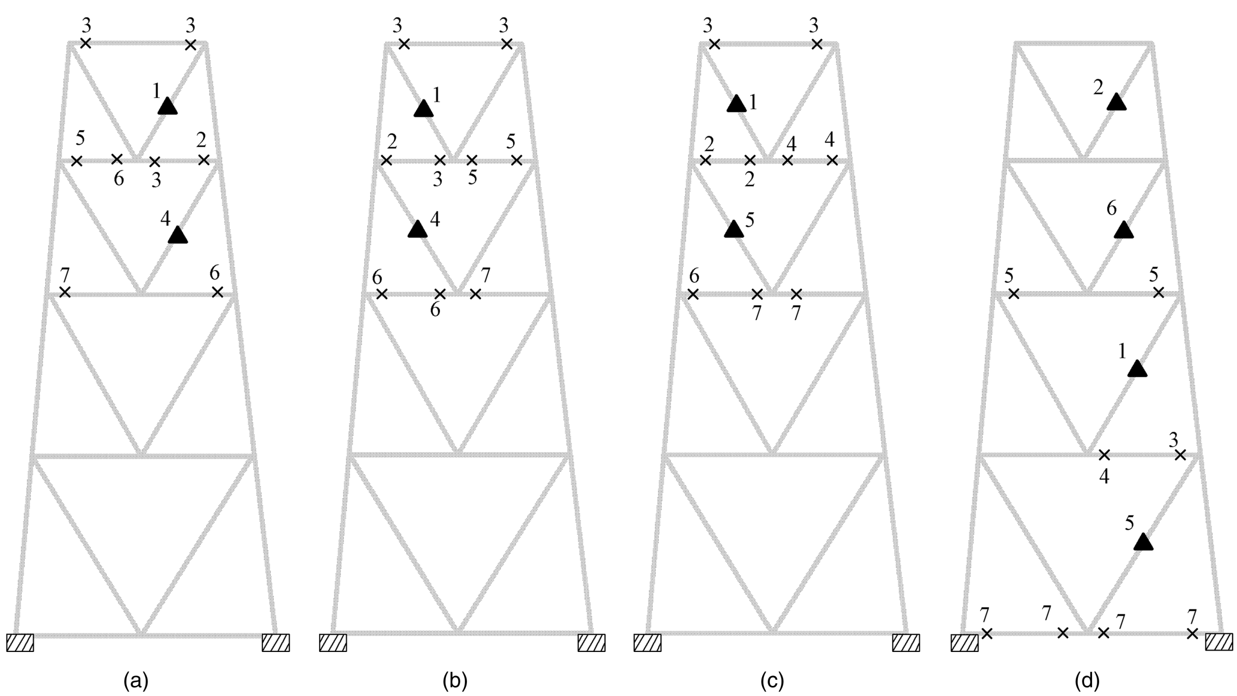

Fig. 12. Plastic hinge formation in the retrofitted model structure (at the maximum displacement of 1.5% of the jacket height): (a) increase of horizontal elements; (b) addition of horizontal bracing; (c) 25% increase in bracing size; (d) 45% increase in bracing size

Table 4. Characteristics of Earthquake Records Considered in Nonlinear Time-History Analysis

Characteristics	El Centro	Kobe	Northridge	Loma Prieta	Imperial Valley
Year	1940	1995	1994	1989	1979
Country	United States	Japan	United States	United States	United States
PGA (g)	0.32	0.34	0.22	0.48	0.36
Recorded points	222	4,096	2,000	2,775	3,930
Time step	0.050	0.010	0.020	0.005	0.005
Duration (s)	12.11	40.95	39.98	13.87	19.65
Component	I-ELC180	090	ORR-UP	CYC285	A-E06230
Magnitude	M(7.0)	M(6.9)	M(6.7)	M(6.9)	M(5.2)
Fault distance(km)	8.3	26.4	22.6	21.8	—
Station	117 El Centro Array #9	Kakogawa	24278 Castaic—Old Ridge Route	57217 Coyote Lake Dam (SW Abut)	942 El Centro Array #6

nonlinear static and dynamic analyses. Fig. 5(c) shows the hysteretic curve of the "24 × 0.5 in." BRB used in the current study. The *SAP2000* program was used for conducting the nonlinear time history analyses.

The jacket structures retrofitted with various retrofit schemes were subjected to the five earthquake records to obtain the nonlinear base shear–roof displacement relationship of the structure. To obtain each data point in the capacity curve, a nonlinear time-history analysis was carried out using an earthquake record scaled up using different scaling factors and then the maximum top-story displacement was plotted against the corresponding base shear. The same procedure was repeated using the same earthquake record scaled up to a different level until a failure state of the structure was reached. Regression analysis was performed using polynomial functions to establish the best-fit curve for the dynamic results, which can be referred to as dynamic pushover curves. The number of nonlinear time-history analyses required for each case differed from one case to another depending on the number of trials needed to identify the maximum response demanded by the earthquake. In this study generally 15–20 nonlinear time-history analyses were required for each earthquake record to identify the intensity levels and the responses in both the linear and nonlinear ranges. Fig. 14

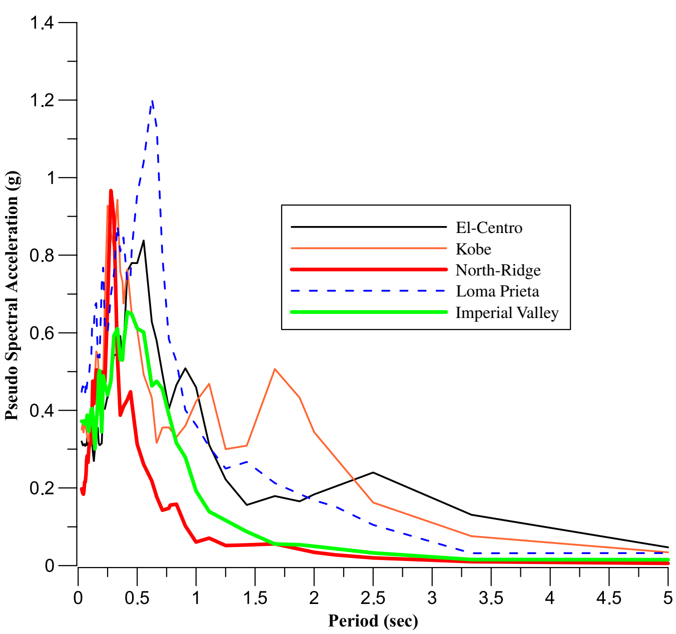

Fig. 13. Pseudo-acceleration response spectra of earthquake records used in the analysis

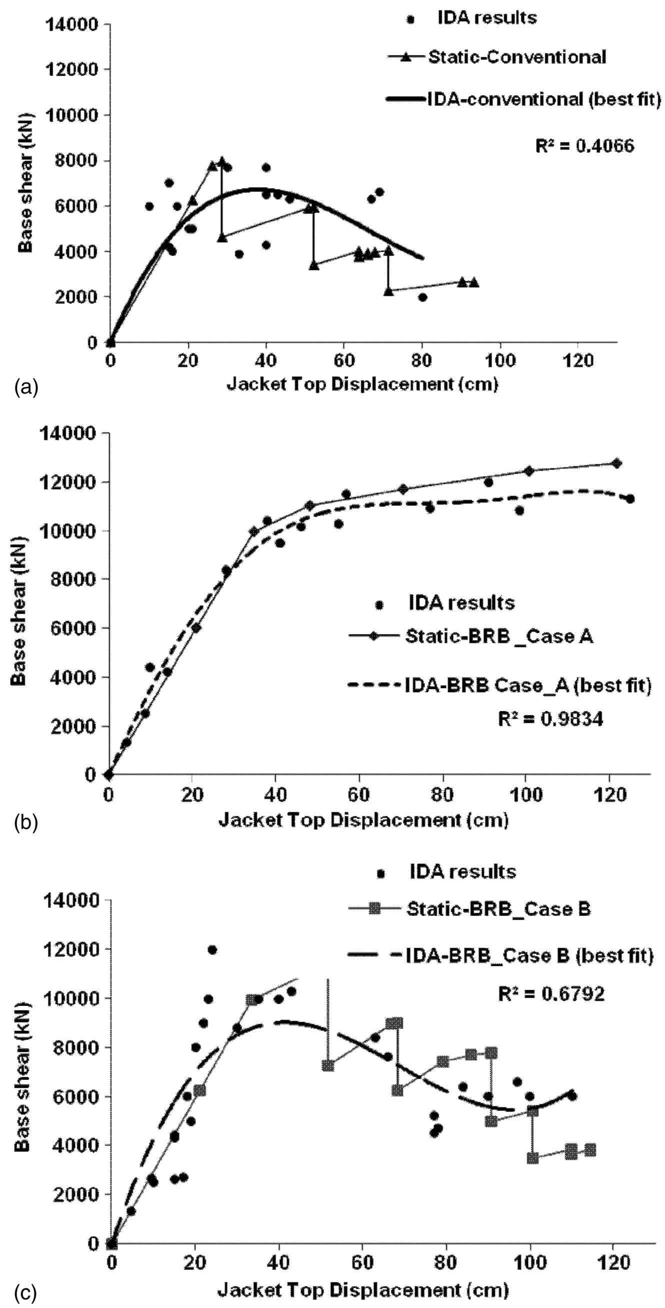

Fig. 14. Best-fit dynamic pushover curves for the *V*-type jacket structure compared to nonlinear static pushover curves: (a) conventional; (b) BRB_Case A; (c) BRB_Case B

compares the capacity curves of the model structures obtained by nonlinear static analysis and the incremental dynamic analysis. It is found that the dynamic analysis results of BRB_Case A are close to each other. However, the results are scattered around the best-fit curve in the Conventional and the BRB_Case B cases. It can be noticed that the dynamic pushover curves match the nonlinear static pushover curves obtained using a lateral load pattern proportional to the first mode shape of the structure.

Summary

In this study the seismic performance of a typical fixed-base jacket structure was evaluated by nonlinear static analysis and incremental dynamic analyses. The behaviors of the jacket structures with various bracing configurations were compared, and the effects of various structural retrofit methods were also evaluated.

According to the analysis results, the jacket structures with X-type and V-type bracing showed larger strengths than those of the N-type and K-type bracing. It was observed that the conventional retrofit methods of increasing member sizes were somewhat effective in increasing the strength of fixed steel jacket platform structures but were not so effective in increasing ductility. However, the structures retrofitted with buckling-restrained braces turned out to have enough strength and ductility required by the API. It was also noticed that the effect of BRB depended on the configuration of the bracing: the strength increase in the BRB-retrofitted jacket structures was found to be three times that of the conventional structure with K-bracing while it was only 41% in the structure with V-type bracing. The ductility capacities of the jacket structures retrofitted with BRB ranged between 2.9 and 6.15 times the yield displacement, while those of the structures with conventional bracing were much smaller. In the structures with conventional bracing plastic hinges generally formed at the upper half of the jacket structures. However, they were more uniformly distributed along the structure height in case of retrofit with BRB. The interstory drifts of the structures with BRB were also more uniformly distributed in the structures with BRB. The capacity curves obtained from incremental dynamic analysis generally matched those obtained from nonlinear static analysis using a lateral load pattern proportional to the fundamental mode shape of the jacket structure.

Acknowledgments

This research was supported by a grant (13AUDP-B066083-01) from the Architecture and Urban Development Research Program funded by Ministry of Land, Infrastructure and Transport of Korean government, and by the Samsung Engineering Co., Ltd.

References

Ali, A. G., Mohammad, R., Tabeshpour, B., and Younes, K. (2009). "FEMA approaches in seismic assessment of jacket platforms (case study: Ressalat jacket of Persian Gulf)." *Constr. Steel Res.*, 65(10–11), 1979–1986.

American Petroleum Institute (API). (2000). "Recommended practice for planning, design and constructing fixed offshore platforms." *API RP-2A*, 21st Ed., Washington, DC.

ASCE. (2007). "Seismic rehabilitation of existing buildings." 41–06, Reston, VA.

Asgarian, B., Aghakouchak, A. A., Alanjari, P., and Assareh, M. A. (2008). "Incremental dynamic analysis of jacket type offshore platforms considering soil-pile interaction." *Proc., of 14th World Conference on Earthquake Engineering (WCEE)*, Beijing, China, 1–11.

Asgarian, B., Arezoo, S., and Pejman, A. (2010). "Seismic performance evaluation of steel moment resisting frames through incremental dynamic analysis." *J. Constr. Steel Res.*, 66(2), 178–190.

Chakrabarti, S. K. (2005). *Handbook of offshore engineering*, Elsevier, Oxford, U.K.

Dolsek, M. (2009). "Incremental dynamic analysis with consideration of modeling uncertainties." *Earthquake Eng. Struct. Dyn.*, 38(6), 805–825.

Kim, T., and Kim, J. (2007). "Seismic performance evaluation of a RC special moment frame." *Struct. Eng. Mech.*, 27(6), 671–682.

Komachi, Y., Tabeshpour, M. R., Golafshani, A. A., and Mualla, I. (2011). "Retrofit of Ressalat jacket platform (Persian Gulf) using friction damper device." *J. Appl. Phys. Eng.*, 12(9), 680–691.

Krieger, W. F., et al. (1994). "Process for assessment of existing platforms to determine their fitness for purpose." *Proc., Offshore Technology Conf.*, Houston, TX.

Lee, L. H., and Kyriakides, S. (2004). "On the arresting efficiency of slip-on buckle arrestors for offshore pipelines." *Int. J. Mech. Sci.*, 46(7), 1035–1055.

McClelland, B., and Reifel, M. D. (1996). *Planning and design of fixed offshore platforms*, Van Nostrand Reinhold, New York, NY.

Nabavian, M., and Morshed, A. (2010). "Extending life of fixed offshore installations by integrity management: A structural overview." *Abu Dhabi Int. Petroleum Exhibition and Conf.*, Society of Petroleum Engineers, Abu Dhabi, UAE, 1–6.

O'Connor, P. E., Versowsky, P., Day, M., Westalke, H., and Bucknell, J. (2005). "Platform assessment: Recent section 17 updates and future API/industry developments." *Offshore Technology Conf.*, Houston, TX.

Petrauskas, C., Finnigan, T. D., Heideman, J. C., Vogel, M., Santala, M., and Berek, G. P. (1994). "Metocean criteria/loads for use in assessment of existing offshore platforms." *Proc., Offshore Technology Conf.*, Houston, TX.

Rodrigues, P. F. N., and Jacob, B. P. (2005). "Collapse analysis of steel jacket structures for offshore oil exploitation." *J. Constr. Steel Res.*, 61(8), 1147–1171.

SAP2000, Structural Analysis Program, Version 10. (2005). *Analysis reference manual*, Berkeley, CA.

Structural Analysis Computer System (SACS). (2009). *User's Manual, Release 6, version 0*, Engineering Dynamics, Kenner, LA.

Uang, C. M., Nakashima, M., and Tsai, K. C. (2004). "Research and application of buckling-restrained braced frames." *Steel Struct.*, 4(1), 301–313.

Vamvatsikos, D., and Cornell, C. A. (2002). "Incremental dynamic analysis." *Earthquake Eng. Struct. Dyn.*, 31(3), 491–514.

Xie, Q. (2005). "State of the art of buckling-restrained braces in Asia." *Constr. Steel Res.*, 61(6), 727–748.

Yin, Z., Wang, X., and Li, X. (2009). "Hysteretic response and energy dissipation of double-tube buckling restrained braces with contact ring." *Computational structural engineering*, Springer, Netherlands, 173–179.