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SEISMIC PERFORMANCE EVALUATION OF STEEL STORAGE RACKS USING EXPERIMENTAL RESULTS OF BEAM-TO-COLUMN CONNECTION

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ABSTRACT

This paper presents the seismic performance evaluation of the semi-rigid steel storage rack located in Thailand. A numerical model of the structure was created with incorporated nonlinear behavior of semi-rigid beam-to-column connection. To capture the realistic behavior of beam-to-column connections, the connections were tested by the cantilever test and the portal test method according to the international racking design specification. Three different levels of vertical service loadings were conducted in the portal test to investigate the effects of vertical loading levels on strength and stiffness of beam-to-column connections. A capacity spectrum method (CSM) procedure described in ATC-40 was employed to evaluate the seismic response of the studied rack under considered ground motion. Seismic demand which represent the frequent, design and rare earthquake ground motion are having a 50%, 10% and 2% probability of exceeded in 50 years respectively. The ground acceleration is selected according to the average value from 69 provinces in Thailand. The seismic performance criteria in FEMA-356 were applied for evaluating seismic performance in this study.

The results show that the studied storage structure with low vertical loading level has the performance to resist all levels of considered earthquakes without any damages. For the studied structure with medium vertical loading level, the frame has no damage under frequent earthquake but some stories show the inter-story drift ratio beyond the IO limit under design earthquake and rare earthquake. For high vertical loading level, the frame has no damage under frequent earthquake and has low structural damages under design earthquake. The frame is expected to be collapse under rare earthquake. The higher loading levels lead to a considerably poor seismic performance of steel storage racks due to the influence of the gravity moment.

1. INTRODUCTION

In recent years, steel storage racks are extensively used in industry and large public warehouse for storing the product. This type of structure has become a common feature in several countries. The main components of the structure are box beams, beam end connectors, and perforated thin wall cold-formed open sections columns. The beam connects to the column by the steel tabs inserted into the perforated column. According to previous experimantal studies, the connection can be classified as a semi-rigid connection with a low to moderate strength and stiffenss (Bernuzzi and Castiglioni 2001, Bajoria and Talikoti 2006, Prabha et al. 2010). Generaly the storage racks are not braced in the

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longitudinal direction due to the continuously accessible to the product in service. Therefore, the structural response in a longitudinal direction is critical from the seismic induced load and it significantly depends on behaviors of beam-to-column connections (Bernuzzi et al. 2004)

Asawasongkram et al (2013) has experimentally studied about the behavior of beam-to-column connections of steel storage rack by two suggested testing methods in steel storage rack design specification (AS 1993, RMI 2008, FEM 1998) such as cantilever method and portal method. The nonlinear analysis results of a single story frame with moment-rotation of connections obtained by a cantilever test were compared with the experimental results from portal test. It shows that the lateral behavior of semi-rigid analytical frame has well correlated with the experimental results for low to medium vertical loadings whereas it has inaccurate correlation for high to very high vertical loading. It might be the effects of vertical loading which not considered in cantilever testing method.

In Thailand, the steel storage racks are increasingly popular in such application as described above. Before enforcement of seismic design standard for building and other structure issued in 2009 (DPT 2009), many existing steel storage structures may have been designed without any consideration of seismic loading. The lack of seismic consideration in steel storage rack structures results in non-ductile behavior in which the lateral load resistance may be insufficient for even moderate earthquake.

The objective of this study is to evaluate seismic performance of a case study steel storage racks located in Thailand. Seismic performance of a case study steel storage frame is evaluated by a capacity spectrum method (CSM) described in ATC-40 and a seismic assessment is conducted according to FEMA-356 (FEMA 2000). The results of evaluation can be use for appropriate repairing or strengthening for the structure.

2. DESCRIPTION OF A CASE STUDY STEEL STORAGE FRAME

The structural components of a case study steel storage frame compose of box beams, beam end connectors, and perforated thin wall cold-formed open sections columns. The beam connects to the column by the steel tabs inserted into the perforated column. Details of structural components are shown in Fig.1. Dimension shown in the figure are given as an average value of a several measurement. According with FEMA-460 (FEMA 2005), a typical five-story five-bay symmetric steel storage frame with a span length of 2.50 m is considered to be a case study. A story height of 1.50 m was assumed throughout resulting in a total height of 7.5 m. The structure was selected to represent a reference of typical steel storage racks located in Thailand. The structural configuration is shown in Fig.2.

During a service condition, the loading intensity for this kind of structures may be changed frequently. In this study, three different levels of service loading are selected to investigate the effects of loading level in the seismic performance of structures. The first loading case is “no vertical loading”. The second “medium” loading has a total load 2.00 kN/m. The third “high” loading has a total load 4.00 kN/m. These vertical loading levels correspond to 0%, 40% and 80% of the allowable moment of the beam respectively.

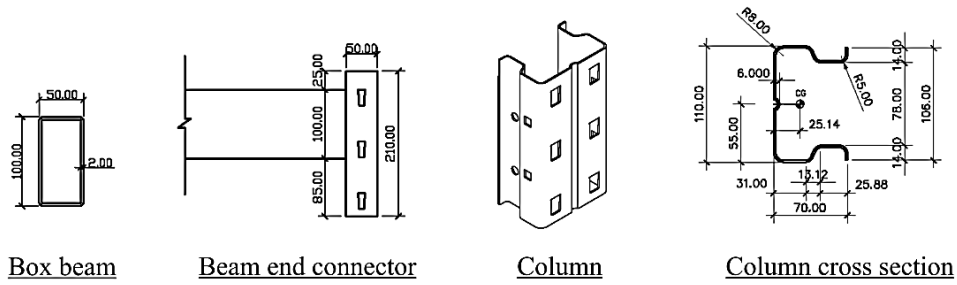


Figure 1. Component of the tested specimens (dimensions in (mm))

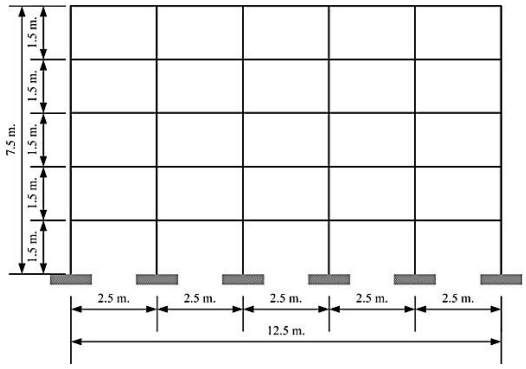


Figure 2. Configuration of the case study frame

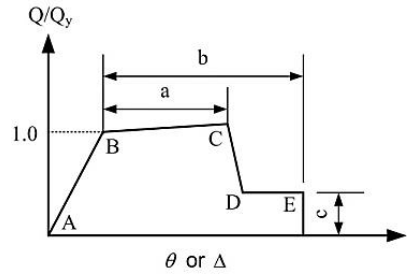


Figure 3. Generalized force-deformation relation for beams and columns

3. MODELLING APPROACH AND ASSUMPTIONS

To evaluate the seismic performance of the structure, a non-linear pushover analysis is performed by mean of computer programs SAP2000 (CSI, 2009) which is a general purpose structural analysis program for static and dynamic analyses of structures. The effects of geometric nonlinearities and material nonlinearities are also taken into account. Material properties of the structural steels were determined by carrying out tensile coupon tests. The average values of the elastic modulus (E) and the yield strength (f_y) are 200×10^5 N/mm^2 and 330 N/mm^2, respectively.

3.1 MODELLING OF BEAM AND COLUMN MEMBERS

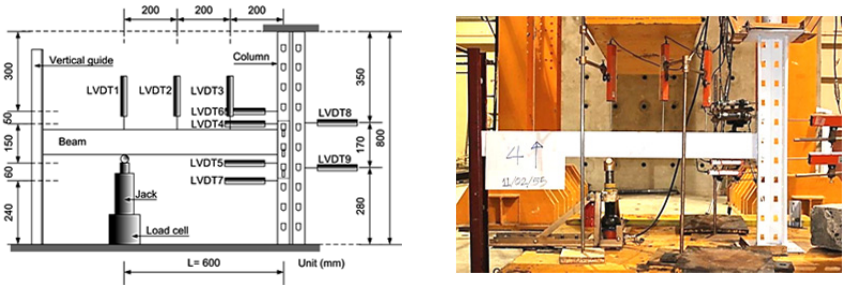
Beams and columns are modelled as elastic frame elements and nonlinear behavior are modelled by lumped plasticity at their ends. SAP2000 implements the plastic hinge properties described in FEMA-356 to capture the inelastic behavior of elements. Generalized force-deformation relation for beams and columns is shown in Fig.3. Five points labeled A, B, C, D, and E define the force-deformation behavior of plastic hinge. The values assigned to each of these points vary depending on the type of element, material properties and the axial load level on the element. The based plated connections of the columns in the structure are considered to be rigid.

3.2 BEHAVIOR OF BEAM-TO-COLUMN CONNECTIONS

As explained earlier, the seismic responses of the storage rack in the longitudinal direction significantly depends on behaviors of beam-to-column connections. To obtain a reliable of moment-rotation of beam-to-column connections, the experimental testing of beam-to-column connections are performed in this study. There are two testing method specified in the internatinal standards for design of steel storage rack to determine the connection behaviors: the cantilever test and the portal test. The cantilever test provides a simple method of determining the connection strength and stiffness.The characteristic of moment-rotation relations and modes of failure of the beam-to-column connections are investigated. Although the cantilever test provides the simple method of determining the moment-rotation relation of the connections, it cannot represent the effects of vertical service loading in a connection. In order to get the moment-rotation relation of a beam-to-column connection under an actual condition, the portal test method are performed in this study. Under the portal test, beam-to-column connections are subjected to shear force, bending moment and axial force thus representing the actual field conditions. The portal frame test gives an average of moment-rotation relation of the connections for a given vertical loading. The moment-rotation relation results from cantilever tests are indirectly compared with the portal test results to investigate the effects of vertical loading in the connections. The strength and stiffness of beam-to-column connection obtained from the experiments are used for modelling in an analytical model to evaluate a seismic performance of a case study steel storage rack. The test specimens are selected from a commercial Thailand manufacture.

3.2.1 CANTILEVER TEST

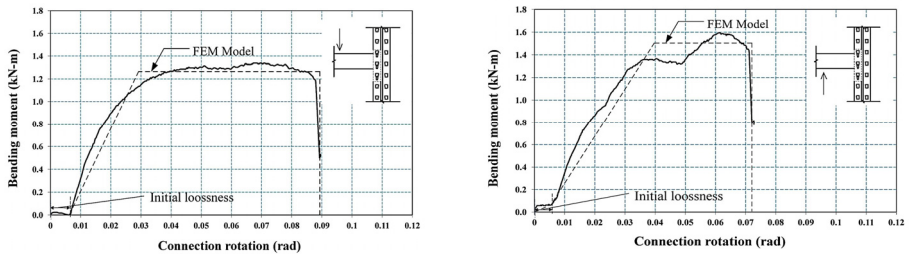
The cantilever test setup consists of a short cantilever of pallet beam connected to the center of a short length of a column. Both ends of the column are rigidly supported. The load is applied monotonically by a hydraulic jack placed on a load cell. The free end of the beam is constrained by a vertical guide to prevent an undesirable out-of-plane movement of the beam. The applied loads are recorded at each increment of loading until the failure is occurred at the connection. Displacement transducers are mounted to measure beam and column deflections. The rotation of the connection is calculated from the deflections for each load step assuming that the beam is rigid. Fig.4 shows the general layout of the experimental setups and the arrangement of the transducers. Full details of the experimental results are available in an earlier paper by the authors (Asawasongkram and Premthamkorn 2012).



(a) Transducer arrangement (b) Actual test setup

Figure 4. Cantilever test setup

The cantilever tests are conducted of six connection samples which consist of three for hogging moment testing and three for sagging moment testing. The samples of experimental results of moment-rotation curve are shown in Fig.5. The experimental results show that, the connection exhibited looseness at the initial stage because the steel tabs are not fit with the column perforation. The looseness is overcome when the rotation reached an approximately 0.006 rad. The initial looseness of the connection is also found in the previous of experimental studies (Bernuzzi and Castiglioni 2001, Bajoria and Talikoti 2006, Prabha et al. 2010). The strength and stiffness of the connections are different under hogging and sagging moments due to the asymmetric of the connection configuration. The failure took place when the tab in the tension side was cut by the column perforation. The connection stiffness and the ultimate moment capacity are tabulated in Table.1. In this study, the connection stiffness is determined by the concept suggested by FEM (FEM 1998). The FEM uses an iterative graphical procedure that approximately balances the areas below the actual and ideal curves up to the failure point.



(a) Hogging moment (b) Sagging moment

Figure 5. Example of moment-rotation curve from cantilever test

Table 1. Beam-to-column connections test results from cantilever test

Test no.	Type of loading	Stiffness (kN-m/rad)	Average stiffness (kN-m/rad)	Ultimate moment capacity (kN-m)	Average of ultimate moment capacity (kN-m)
1	Hogging moment	51.42	49.50	1.44	1.37
2	Hogging moment	51.54		1.34	
3	Hogging moment	45.52		1.32	
4	Sagging moment	45.33	44.87	1.36	1.53
5	Sagging moment	48.54		1.59	
6	Sagging moment	40.75		1.63	

3.2.2 PORTAL TEST

Under a portal test, two portal frames are mounted on hinges supports which are clamped to strong floors. Three bracing elements are connected between two portal frames by channel sections to prevent any displacement in a transverse direction. The portal test setup is shown in Fig.6. A vertical distance between centers of a hinge to centers of a beam is 700 mm. A horizontal distance between column center lines is 2500 mm. A distance between two portal frames is 1000 mm. A horizontal rigid transfer beam, used to distribute lateral loads to portal frames equally, is bolted to a perforated column. Lateral loads are applied at the level of portal beams by a hydraulic jack placed on a load cell attached to a very rigid steel support and connected to a strong floor. Four displacement transducers, LVDT1-LVDT4, are placed on the columns at a level of center lines of the portal beam to monitor the lateral displacements of the portal frame. Additional displacement transducers, LVDT5-LVDT8, are placed at the base of the columns to check the sliding of the columns. The lateral loads and the lateral displacements are recorded at each increment of loading until failures at the connection is occurred.

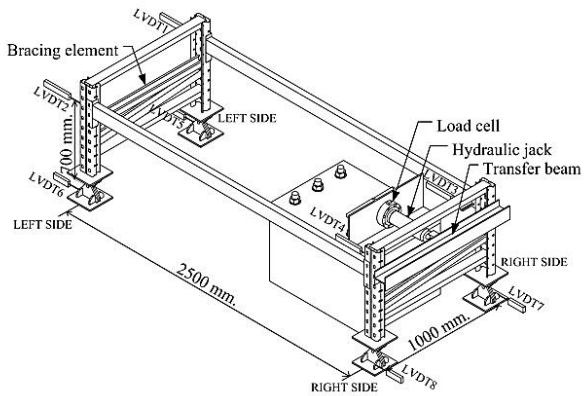


Figure 6. Portal test setup

Three types of the vertical loading level as explanined in section 2.0 are used for the test series to investigate its effects on strength and stiffness of beam-to-column connection. Vertical loads are simulated by sand bags resting on standard wood pallet as shown in Fig.7. The portal tests are conducted for nine portal frames which consist of three frames for each loading level.

Figure 7. Portal test setup

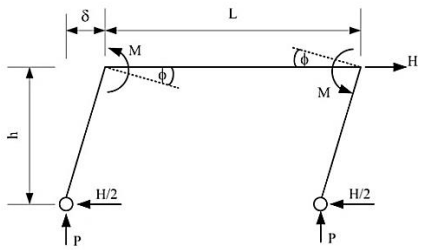


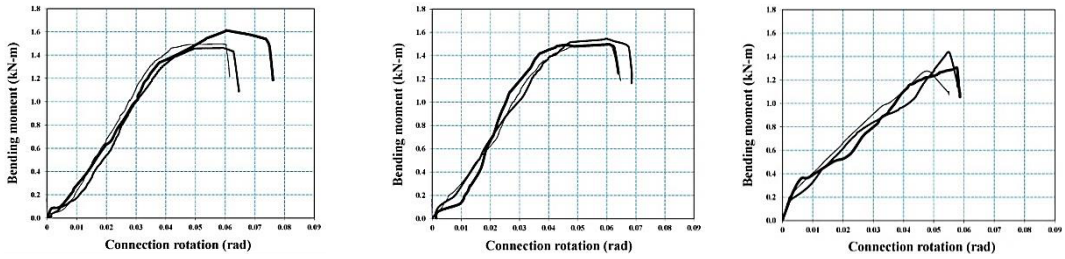
Figure 8. Portal frame deformation

The moment (M) and rotation (ϕ) of the beam-to-column connection in the portal frame are shown in Fig.8. The moments and rotation of the connection are expressed as Eq.(1) and Eq.(2) respectively (Krawinkler et al. 1979).

$$M = \frac{H}{2}h + P\delta \tag{1}$$

$$\theta = \frac{\delta}{h} - \left(\frac{Mh}{3EI_c} + \frac{ML}{6EI_b} \right) \tag{2}$$

where H is the lateral load applied to one portal frame, h is the vertical distance from center of hinge support to center of the portal beam, P is the axial force in the column due to vertical loads, δ is the lateral displacement at the center of the portal beam. δ is taken as the average recorded displacement obtain from LVDT1-LVDT4, E is the Young's modulus, I_c and I_b are the moment of inertia of the column and portal beam respectively, and L is the horizontal distance between the centerline of the columns. The experimental moment-rotation curves are given in Fig.9. The main characteristics of the experimental moment-rotation curves are shown in Table 2.



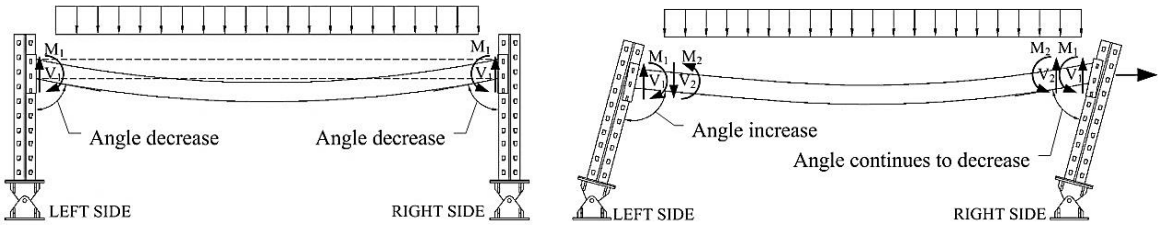
(a) No vertical loading (b) Medium vertical loading (c) High vertical loading

Figure 9. Moment-rotation curve for the portal test with three different vertical loading level

From the portal test results, the following observation can be made. The portal frame with no vertical loading has the ultimate moment capacity of the connection approximately equal with the portal frame with medium vertical loading case. For the portal frame with high vertical loading, the ultimate moment capacity of the connection is equal to 87% of the portal frame with no vertical loading. It can be shown from the experimantal results that the structure with the high vertical loading level has a lower connection capacity than the structure with the lower vertical loading level. This can be explained by the force transfer mechanism at the connection as follows. In the high vertical loading levels, both sides of the beam-to-column connections have a high initial hogging moment (M_1) and a high shear force (V_1) acting on a positive direction as shown in Fig.10 (a) . When the lateral load is subsequently applied, the left side connection would be a sagging moment and a shear force acting on a negative direction whereas the right side connection would be a hogging moment (M_2) and a shear force (V_2) acting on a positive direction. The hogging moment and shear force due to the lateral load (M_2, V_2) will combine with the high initial hogging moment and shear force due to the vertical load (M_1, V_1) at the right side connection as shown in Fig.10 (b) and make them reaches the ultimate moment capacity prior to the other lower vertical loading levels. The failure of the connection occurred at the steel tab in the tension side placed farther from the neutral axis was cut by the column perforation.

Table 2. Beam-to-column connections test results from portal test

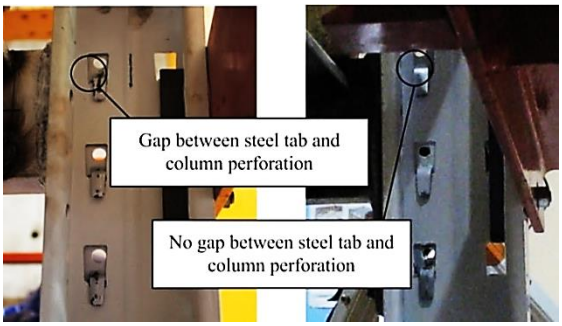
Test no.	Type of vertical loading	Ultimate moment capacity, (kN-m)	Average of ultimate moment capacity (kN-m)
1	No	1.612	1.523
2	No	1.464	
3	No	1.493	
4	Medium	1.511	1.517
5	Medium	1.545	
6	Medium	1.495	
7	High	1.301	1.333
8	High	1.431	
9	High	1.267	



(a) Application of vertical load (b) Application of lateral load

Figure 10. Portal frame deflected shapes

In case of the portal frame with medium vertical loading levels, the force transfer mechanism at the connection are as follows. At the initial stage, the portal beam deform with a small deflection under the medium vertical loading. The steel tab above the neutral axis of the beam end connector move with a small amout and not attach with the column perforation as shown in Fig.11 (a). The hogging moment due to the vertical load occuring at the connection will be resist by the bearing force acting between the contact area of the steel angle of the beam end connector and the flange of the perforated column. Thus the vertical loading only produce the vertical shear force at the steel tab. This force transfer mechanism are the same with in the case of no vertical loading. Therefore the ultimate moment capacity of the connection for that two cases are approximately equal. In contrast to the high vertical loading case, the portal beam deform with a large deflection at the initial stage. The steel tab above the neutral axis of the beam end connector move a lot and attach with the column perforation as shown in Fig.11 (b). Thus in the high vertical loading case, the vertical loading produce both the vertical shear force and horizontal shear force at the steel tab. Subsequently when the lateral loading are applied, the shear force due to the horizontal load will combine with the shear force due to the vertical load and make the steel tab reach its ultimate capacity before the other two cases as explain earlier.



(a) Medium vertical loading level (b) High vertical loading load level

Figure 11. Right side connection at the initial stage

Another observation from the experimantal results is that in the case of no vertical loading and medium vertical loading, the moment-rotation curve show the “slip” at the initial stage. The slip represent by a short line with a low slope value at the initial stage as shown in Fig.9(a) and Fig.9(b).
The slip come from the steel tabs are not perfectly fit with the column slot. Under the lateral load the steel tab move in the column slot rapidly until it attach with the edge of the column slot. However there is no slip in the initial range for the high vertical loading case because there is a large friction between the steel tab and column slot due to a high vertical loading. The friction force cause the steel tab move slowly under the lateral load therefore there is no “slip” line at the initial range as shown in Fig.9(c).

3.3 COMPARISON OF THE EXPERIMENTAL RESULTS

The experimental results obtained from the cantilever test and the portal test are compared to determine the appropriate value of strengths and stiffnesses of a beam-to-column connection for modelling in an analytical model. The cantilever test distinguish the experimental results between the hogging and the sagging moment-rotation relation of the connections. On the other hand the portal test give a mean value of the moment-rotation of the connections. To compare the experimental results indirectly, a non-linear pushover analysis of a single story frame is performed by means of a computer program SAP2000 (CSI 2009). A pushover analysis of a single story frame using beam-to-column connection properties obtained from the cantilever test are compared with the experimental results from the portal test. Non-linear behavior of the beam-to-column connection is modeled by a non-linear rotational link element. The link element connected to the end of beam and the end of column.

The effects of geometric nonlinearities and material nonlinearities are also taken into account in the numerical model. A nonlinear moment-rotation relation of a connection is modeled by a tri-linear model to account for the effects of the initial looseness of the connection as explained in section 3.2.1. The tri-linear model is illustrated in Fig.12. The value of k_1 is an average stiffness of a connection in the initial looseness stage. The value of k_2 is an average stiffness of a connection after the initial looseness is terminated obtained by the concept suggested by FEM (FEM 1998). The parameters of the tri-linear model $(\theta_i, \theta_u, M_i, M_u)$ are obtained from the experimental results of the cantilever test.

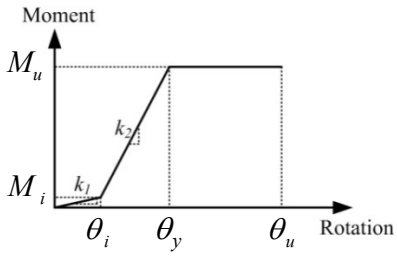
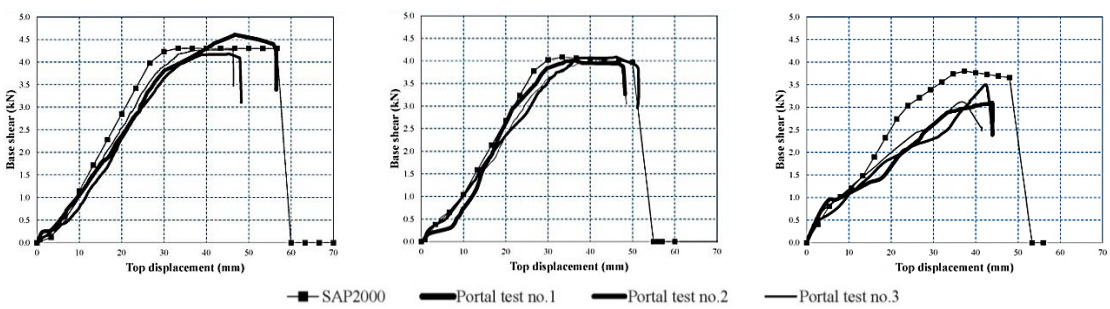


Figure 12. Tri-linear model for moment-rotation of beam-to-column connections

The pushover analysis results compare with the experimental results from the portal test as illustrated in Fig.13. The analytical results show a good correlation with the experimental results from the portal test for the case of no vertical loadings and medium vertical loadings. For a high vertical loadings, the analytical results do not produce a good correlation with the experimental results from the portal tests. The portal tests results produce a lower strength than the analytical results. The portal tests results produce a lower value of connection stiffness than the analytical results. These results show a limitation of a cantilever test for a high vertical loadings. As explained in the earlier section, the cantilever tests can not represent the effects of a tightening at the connection due to the present of a vertical service load. Morever it can not represent the effects of a combination of force at the connection due to the vertical loading and lateral loading. These effects lead to a non-correlation of the analytical results compared to the experimental results from portal tests for high vertical loading case.



(a) No vertical loading (b) Medium vertical loading (c) High vertical loading

Figure 13. Relation between experimental results from portal tests and numerical results using beam-to-column connection from cantilever tests

3.4 MODELLING OF BEAM-TO-COLUMN CONNECTION

In this study, to capture behaviors of the beam-to-column connection correctly, the experimental results from both testing method are used to model the beam-to-column connection for seismic performance evaluation. For no vertical loading and medium vertical loading, the strength and stiffness of the connection are selected from the cantilever tests which give different connection properties for hogging moments and sagging moments. For the high vertical loading, the strength and stiffness of the connection are selected from the portal tests which give the same connection properties for both hogging moments and sagging moments. The average of connection properties are shown in Table.3. The plastic rotation capacity (θ_u) is 0.07 rad for no and medium vertical loading and 0.06 for high vertical loading. To simplify the analysis, a nonlinear moment-rotation relation of the connection is modeled by a tri-linear model as illustrated in Fig.12. A beam-to-column connection is modeled by a non-linear rotational link element. The link element connected to the end of beams and the end of columns.

Table 3. Average of strength and stiffness of the beam-to-column connections for tri-linear model

Type of vertical loading	Type of loading	M_i (kN-m)	M_u (kN-m)	θ_i (rad)	θ_y (rad)
No and Medium	Hogging moment	0.05	1.37	0.006	0.034
	Sagging moment	0.05	1.53	0.006	0.040
High	Hogging moment, Sagging moment	0.36	1.33	0.006	0.048

4. SEISMIC PERFORMANCE EVALUATION OF STRUCTURES

Seismic performance evaluation of the case study storage frame is performed according to FEMA-356 guidelines. The performance objective is defined as the acceptable of damage expected in a structure (performance level) under a particular intensity of ground motion (seismic demand). Three structural performance levels, i.e. immediate occupancy (IO), life safety (LS) and collapse prevention (CP) are used for the assessment in this study. For immediate occupancy, the structure is expected to sustain a minimum or no damage to the structural elements under a frequent earthquake ground motion. For life safety, the structure is expected to have a low or repairable structural damage during a design earthquake ground motion. For collapse prevention, the structure may have an irreparable structural damage but no collapse under a rare earthquake ground motion.

4.1 SEISMIC DEMAND

Seismic demand, which represent the frequent, design and rare earthquakes ground motion are defined as having follows (ATC-40, FEMA356): (i) frequent earthquake is defined a minor earthquake which have a 50% probability of being exceeded in 50 years, (ii) design earthquake is defined as a moderate earthquake which have a 10% probability of being exceeded in 50 years and (iii) rare earthquake is defined as a major earthquake which have a 2% probability of being exceeded in 50 years.

In this evaluation, the seismic demand is identified assuming that the structures are situated in Thailand which at the northern or western area has been a number of moderate-size shallow-depth earthquake events during a past few decades (Ornthammarath et al. 2011). For spectrum construction, the 5% damping design spectrum being scaled by 0.5 and 1.5 to obtain the frequent and rare spectra (ATC-40) as shown in Fig. 14. The peak ground acceleration is selected according to the average value from 69 provinces in Thailand. A class D soil is used, which corresponds to a stiff soil having the average shear wave velocity of 180-360 m/s (DPT 2009).

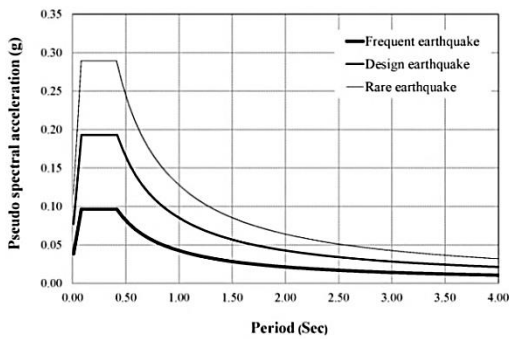


Figure 14. Site-specific response spectra Figure

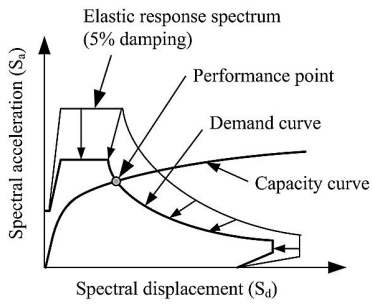


Figure 15. Performance evaluation by capacity spectrum method

4.2 CAPACITY SPECTRUM METHOD (CSM)

The seismic performance of a case study storage frame is carried out by using a capacity spectrum method (CSM) as recommended by ATC-40 as illustrated in Fig. 15. The method requires construction of a structural capacity curve and its comparison with the demand response spectrum. Both curves are represented in Acceleration-Displacement Response Spectrum (ADRS) format. In order to account for the inelastic response, the spectral reduction factors are calculated using the Newmark and Hall (1982) ductility-damping relationships. The performance point that intersected on both the capacity spectrum and the reduced demand spectrum is obtained for evaluation of performances of the structure.

In this study, the non-linear static pushover analysis is conducted to present the lateral capacity of the structure. The pushover analysis consists of the application of gravity loads and a representative lateral load pattern. The lateral loads are applied monotonically in a step-by-step nonlinear static analysis. In this study, the applied lateral loads are proportional to the product of mass and the first mode shape of the structure. This load pattern gives the reliable results especially for the structure dominated by the fundamental mode like this case study frame (Chopra and Goel 2002). P-Delta effects were also taken into account in the analysis.

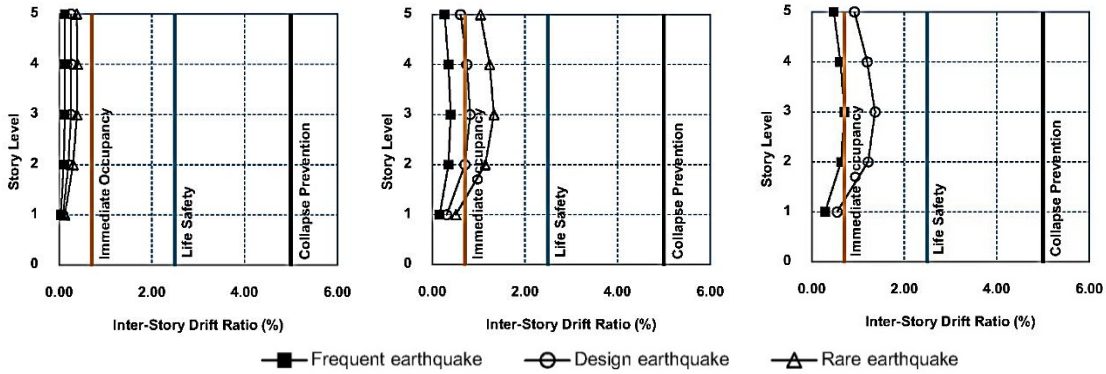
5. RESULT AND DISCUSSION

For evaluating the seismic performance of steel storage racks, FEMA 460 suggests that the total weight should be modified by 0.67 to represent the fraction on the total weight systems contributing to the effective horizontal seismic weight. After factored the total weight, the seismic responses of the studied steel storage rack were determined by the CSM. The inter-story drift ratios (ratio of relative displacement to story height) were compared to the limit value of each performance level. The limit of inter-story drift ratios suggested by FEMA 365 are 0.7%, 2.5% and 5% for the IO, LS and CP performance levels respectively. Fig.16 shows the inter-story drift ratio of the studied storage rack under three considered earthquakes.

The case study frame has no damage under frequent earthquake, design earthquake and rare earthquake for low vertical loading level. The inter-story drift ratios are below all the limits value in every story. For medium vertical loading level, the frame has no damage under frequent earthquake and has low structural damages under design earthquake and rare earthquake. For high vertical loading level, the frame has no damage under frequent earthquake and has low structural damages under design earthquake. The frame is expected to be collapse under rare earthquake.

According to the results, the frame with higher loading level has an undesirable seismic performance than the lower loading level due to the influence of gravity moments. In the higher loading level, the connection has a higher initial sagging moment at beam-ends from gravity loads. When the structure is subjected to an increasing lateral load from pushover analysis, one side of beam-ends will be a hogging moment and the other will be a sagging moment. The sagging moment will be

combined with the high value of the initial gravity sagging moment. This moment combination will produce a larger moment at the beam-to-column connection and will reach their ultimate moment capacity before the lower loading levels.



(a) No vertical loading (b) Medium vertical loading (c) High vertical loading

Figure 16. Inter-story drift ratio of the case study storage racks.

6. SUMMARY AND CONCLUSIONS

This paper presents the seismic performance evaluation of the semi-rigid steel storage rack located in Thailand. A numerical model of the structure was created with incorporated nonlinear behavior of semi-rigid beam-to-column connection. To capture the realistic behavior of beam-to-column connections, the connections were tested by the cantilever test and the portal test method according to the international racking design specification. A capacity spectrum method (CSM) procedure described in ATC-40 was employed to evaluate the seismic response of the studied rack under considered ground motion. The seismic performance criteria in FEMA-356 were applied for evaluating seismic performance in this study. The following conclusions were made based on the results of this study. The seismic performance evaluation of the case study structure using the inter-story drift criteria according to FEMA-356 gave the following results.

1. The case study frame has no damage under frequent earthquake, design earthquake and rare earthquake for low vertical loading level. The inter-story drift ratios are below all the limits value in every story. For medium vertical loading level, the frame has no damage under frequent earthquake and has low structural damages under design earthquake and rare earthquake. For high vertical loading level, the frame has no damage under frequent earthquake and has low structural damages under design earthquake. The frame is expected to be collapse under rare earthquake.
2. The higher loading levels lead to a considerably poor seismic performance of steel storage racks due to the influence of the gravity moment. The high gravity load will produce a larger moment combination at the beam-to-column connection. The connections will reach their ultimate moment capacity prior to the lower loading levels.
3. Although the cantilever test is a simple testing method of determining the strength and stiffness of a beam-to-column connection of a steel storage rack, their results have a good correlation with the results from the more complex methods, the portal test, under no vertical service loading to medium vertical service loading.
4. It is suggested that for a seismic analysis of a steel storage rack under a medium to high vertical service loading, the experimental results from the portal tests is recommended to get the conservative of the moment-rotation of the beam-to-column of the connection.

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