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DEVELOPMENT AND APPLICATION OF A MECHANICAL MODEL OF BEAM-TO-COLUMN CONNECTIONS OF STEEL STORAGE RACKS

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ABSTRACT

This paper focuses on the development of a mechanical model of beam-to-column connections of steel storage racks for predicting the initial rotational stiffness and flexural strength of the connection. An application of the proposed mechanical model for predicting the lateral behavior of steel storage racks is also evaluated. The connection model is developed based on the concept of component method suggested by Eurocode 3. A new practical methodology for evaluation of the flexural strength based on the actual failure mode is introduced. To validate the proposed mechanical model, extensive studies on experimental testing by cantilever and the portal tests specified in steel storage rack design specification are conducted. The validation shows that the performance of the proposed mechanical model is satisfactory for predicting the response of connection. Moreover, it reveals that a modification factor of initial rotational stiffness and initial looseness should be incorporated into a rack's connection model to improve the accuracy in predicting lateral behavior of steel storage frames.

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Steel storage racks

1. Introduction

In recent years, steel storage racks are extensively utilized in industries and large warehouses for storing products. Storage racks are simple structures consisting of open-section columns and horizontal beams. The column is punched with slotted holes along its length, and the beam is prefabricated with the so-called beam-end connectors on both ends. Assembly of the racks can be achieved by simply inserting the tabs on the beam-end connectors and sliding the connectors down into the slotted holes on the column. Fig. 1 illustrates a typical beam-to-column connection of steel storage racks.

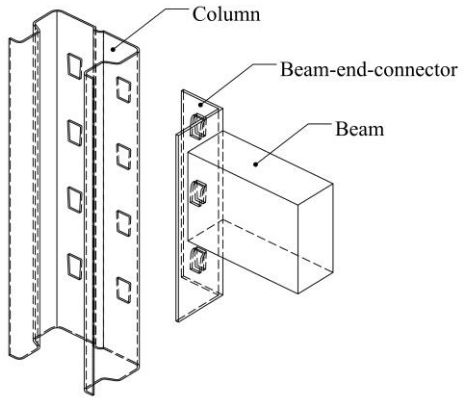


Fig. 1 Typical beam-to-column connection of steel storage racks

Despite being a simple structure, steel storage rack belies its complex connection's mechanical behavior. Accurate design and strength evaluation of the steel storage frame rely on the modeling of the moment-to-rotation $(M-\phi)$ behavior of the beam-to-column connections. For estimating the connection behavior, three modelling options are available. These are empirical models based on experimental testing, finite element models, and mechanical models. Previously, most research on the estimation of connection response for the rack structures had been concentrated only on experimental testing models [1-8] and finite element models [5-6] [8]. The development of the mechanical model for this type of connection is very limited.

While experimental testing and finite element modelling yield satisfactory results, they can be costly, and time-consuming. The major drawback of these two methods is that the results are only applicable to the specific connection typologies, geometries, and material strength that undergo the testing and analysis. The mechanical models derive the connection's behavior directly from its geometries and material strength and hence adaptable to geometry and material-strength variations of the connections. Recently, a theoretical approach for a mechanical model of storage rack connections has been proposed [9] by the concept of component method suggested by Eurocode 3 [10].

In this study, further development of the mechanical model of the beam-to-column connection of steel storage racks formulated by Kozlowski [9] has been conducted. Improvements of the model based on experimental findings from the authors [11] are proposed for a more simplistic moment-resistance calculation. Moreover, the mechanical models are developed for both increasing moment (hogging moment) and reversal moment (sagging moment) to emulate behavior under the two distinct loading conditions. Finally, the proposed model has been validated by comparing it to the results obtained from our testing. An application of the proposed model to predict lateral behavior of steel storage frames is also a highlight of this paper. In addition to the customary cantilever tests called for in design specifications, the portal tests is also conducted in this study in order to evaluate the performance for predicting lateral behavior of steel storage frames under various vertical loads.

2. Experimental study

This section presents extensive testing of the beam-to-column connections of steel storage racks. Two experimental testing methods in compliance with the international standards for design of steel storage racks are conducted for evaluation of the performance of the proposed mechanical model. Complete information of the experimental work is presented in previous research published by the authors [11].

The first experimental method is the cantilever test. This method provides a simple methodology for establishing the stiffness and strength of the connections. Moreover, the behavior of the connection under increasing moment and reversal moment, which may have different characteristics, can be distinguished by this testing method. Many researchers have conducted this test as shown the literature [1-2] [5-7]. The connection properties, which are the initial rotational stiffness and flexural strength from the cantilever test, will be used for evaluation of the proposed mechanical model in section 4.

The cantilever test cannot characterize the influences of the vertical service loading under real usage conditions, though it offers a simple approach for identifying the stiffness and strength of the connections. Thus, the portal test is conducted based on the international storage rack design specifications in this study. Beam-to-column connections are exposed to shear force, bending moment, and axial force in a portal test, which can represent the actual behavior of the connections. The results of the portal test are used to evaluate an application in which the proposed mechanical model is used to predict lateral behavior of steel storage racks. From literature, only a small number of portal tests have been reported [12-13] due to difficulty of the experiment setup. In this study, the portal testing is conducted on 6 testing frames, with varying vertical loading in 2 levels. Each portal test involves 4 beam-to-column connections, resulting in 24 testing connections. In addition, the cantilever tests are carried out for 10 beam-to-column connections. Thus, the test specimens comprised 34 beam-to-column connections in total.

2.1. Specimen details

The test specimens of selected racks are chosen from a commercial manufacture where their configurations are commonly used. In particular, each specimen consists of a beam-end-connector welded to the end of a box beam. The beam-end connector comprised a 3 mm thickness of hot rolled angle section. The column and beam are fabricated from cold formed steel. The thickness of the column and beam is 2.5 mm and 2.0 mm respectively. Fig. 2 gives the detail for the beam-to-column connection and the column cross section.

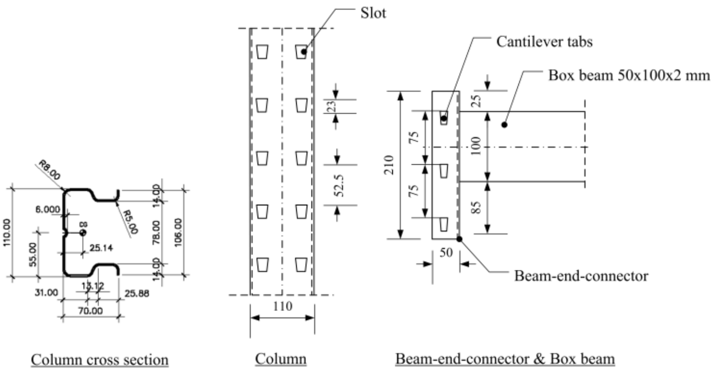


Fig. 2 Dimension of the specimen (Unit: mm)

Tensile tests on coupons extracted from the column, beam, and beam-end-connector are conducted for the purpose of indicating the actual mechanical properties of steel. The tensile tests are performed according to JIS [14] yielding the results shown in Table 1. Steel Material properties for the connection can be classified using the following ASTM standard: A611 [15], A283 [16] and A36 [17] for column, beam-end-connector, and beam, respectively.

Table 1
Steel mechanical properties

Part of the connection	Yield stress (Mpa)	Ultimate stress (Mpa)
Column	383	422
Beam-end-connector	296	374
Beam	381	419

2.2. The cantilever tests

For evaluation of the proposed mechanical model, the cantilever test setups are performed according to the international design standard for steel storage racks [18-20]. A cantilever of pallet beam joined to the midpoint of a short length of a column comprises the test setup. The column is firmly fixed at its top and bottom ends. Using a hydraulic jack placed on a load cell, the applied load is operated monotonically. To prevent any unwanted out-of-plane beam movement, the free end of the beam is restrained by a vertical guide made from a channel section. The applied loads, as well as beam and column deflections, are recorded at each increment of loading until failure takes place at the connection. Measurement of beam and column deflections is done using displacement transducers, LVDT1-LVDT9. The deflections for each load step are used to calculate the rotation of the connection. The overall layout of the experimental setups and the layout of the transducers are shown in Fig. 3. Utilizing ten connection samples comprising five hogging moment tests and five sagging tests, the cantilever tests are completed.

Fig. 4 shows the experimental results of the moment-rotation curve. The results show that the moment-rotation curve under hogging and sagging moments testing are quite different due to asymmetry of the connection configuration. The experimental curve expresses nonlinearity due to yielding of the connection parts, specifically steel tab and column perforation. The connection is considered to have failed when the steel tab in the tension side located farthest from the center of rotation is cut by the column perforation. At this point, the load carrying capacity drops suddenly. During the test, it was found that some of the tested connections exhibited looseness, indicated by a short line at the beginning of the curve. This looseness occurred due to the steel tabs not fitting snugly in the column holes. When the rotation reaches about 0.002-0.006 rad, looseness is eliminated. In prior experimental studies, looseness of the connection was also uncovered [2] [5-6]. After the looseness ceases, the initial rotational stiffness of the connections is calculated [11]. Some of the tested connections do not show looseness because those connections are preloaded to prevent unwanted infirm effects at the beginning of the test setup. The averages of the initial rotational stiffness and flexural strength of the connections from the cantilever tests are shown in Table 2.

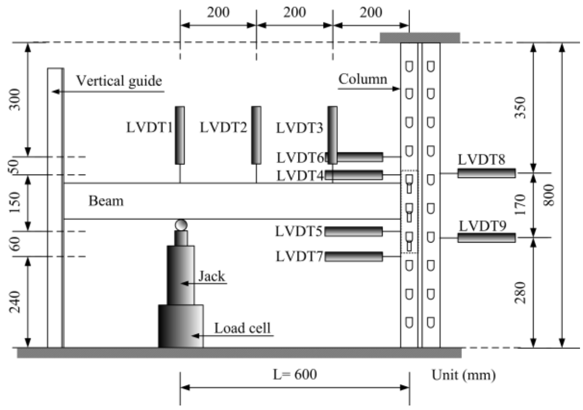
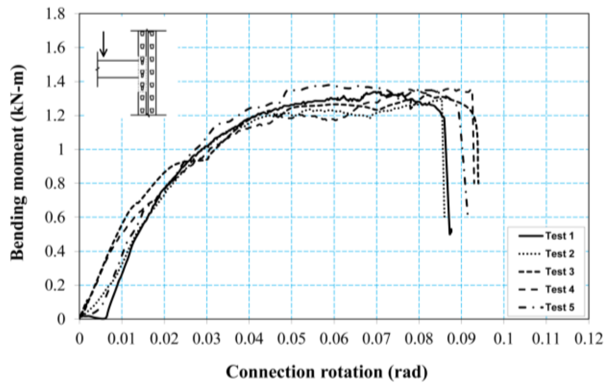


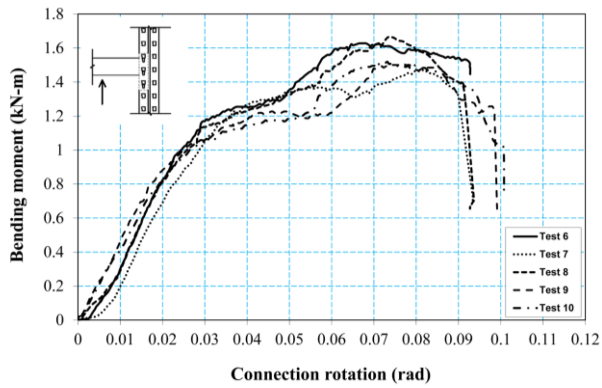
Fig. 3 Details of cantilever test setup

Table 2
Average value of the initial rotational stiffness and flexural strength of beam-to-column connections obtained from the cantilever tests

	Hogging moment	Sagging moment
Initial rotation stiffness (kN-m/rad)	$S_{j,ini} = 62.50$	$S_{j,ini} = 60.10$
Flexural strength (kN-m)	$M_u = 1.33$	$M_u = 1.56$



(a) Hogging moment



(b) Sagging moment

Fig. 4 Moment-rotation curves of the connections from the cantilever tests

2.3. The portal tests

In this study, the portal test [18-20] is performed according to the standard testing method to study steel storage frame behavior under lateral loads. Two portal frames fixed to hinge supports, which are fastened to a very stiff concrete floor, comprise the testing frame. Light lip channel beams are used to connect between the two portal frames in transverse directions. Fig. 5 illustrates the portal test setup. The length of portal frame is 2500 mm between column center lines. The height of portal frame is 700 mm, measured from each hinge support to the center of each horizontal beam. The space between the two portal frames is 1000 mm. For the purpose of transferring lateral loads to each portal frame equally, the horizontal rigid transfer beam is bolted to the portal frame. Lateral loads are applied to the transfer beam by a hydraulic jack mounted on a load cell fixed to a very stiff steel support connected to the concrete floor. The lateral displacement of the portal frame is measured by four displacement transducers (LVDT1-LVDT4) which are set on the columns level to the center line of the portal beam. To measure the sliding of the columns, other displacement transducers (LVDT5-LVDT8) are set at the bottom of the columns. The lateral loads and corresponding lateral displacements are recorded at each increment of loading until failure takes place at the connection.

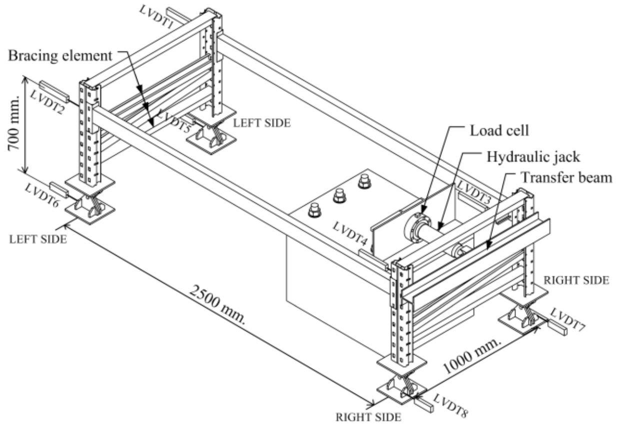
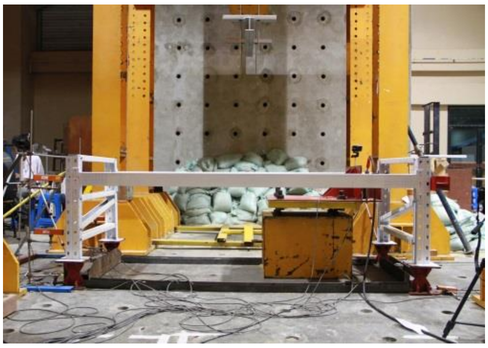


Fig. 5 Details of portal test setup



(a) No loading

(b) Normal loading (2 kN/m)

Fig. 6 Loading intensity for portal test setup

The loading intensity on steel storage racks may change frequently under service conditions. Thus, two distinctive levels of vertical loading were chosen in order to investigate the connection behavior under actual usage conditions. In this study, the two vertical loading levels are "no loading" and "normal loading". The normal loading level has a total load of 2.00 kN/m. This loading level corresponds to 40% of the allowable moment of the beam, which is assumed to represent the normal service loading condition on the structure. As shown in Fig. 6, sand bags resting on ordinary wood pallets are used to simulate the vertical loads. Portal tests are carried out on six frames, comprised of three testing frames for each loading level.

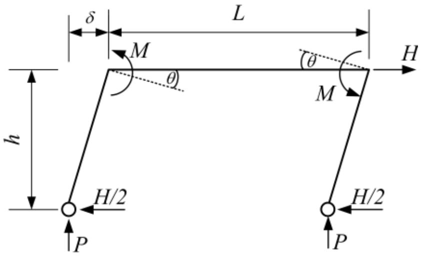


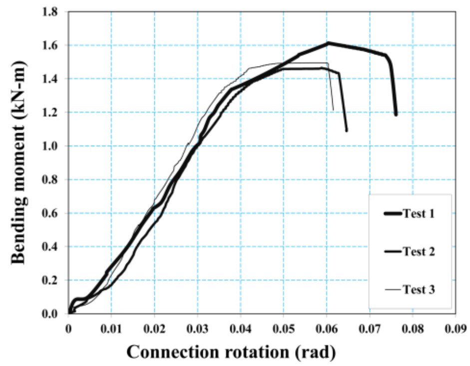
Fig. 7 Deformation of portal frame under lateral loading

The moments (M) and rotation (θ) of the beam-to-column connection in the portal frame are given as Eq. 1 and Eq. 2, respectively, illustrated in Fig. 7 [12].

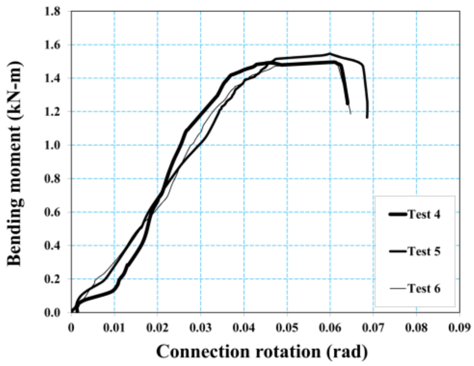
$$M = \frac{H}{2}h + P\delta \tag{1}$$

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

where H is the lateral load applied to one portal frame, h is the height of portal frames, P is the axial force in a column due to a vertical load, and δ is the lateral displacement of the portal frame. δ is taken as the average recorded displacement obtained from displacement transducers LVDT1 to LVDT4, E is a Young's modulus, I_c and I_b are moment of inertia of the column and the portal beam respectively, and L is the horizontal distance between the centerlines of the columns. Fig. 8 shows the test results for the portal tests with two distinct vertical loading levels.



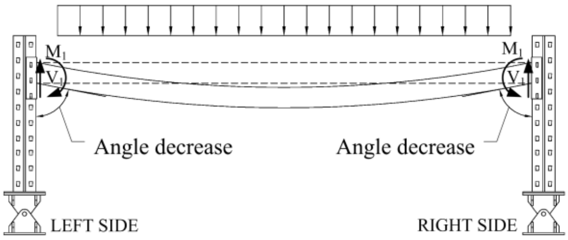
(a) Hogging moment



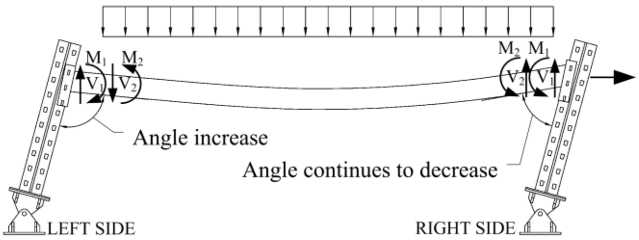
(b) Sagging moment

Fig. 8 Moment-rotation curves for the portal tests with two distinct vertical loading levels

The following observations can be made based on the portal test results. A failure occurs with the connection at the right side of the portal frame when the steel tab in the tension side placed farthest from the center of rotation is cut by the column perforation, like the cantilever test results. This can be explained by the force transferring mechanism of the connection as follows. At the initial stage, before the lateral load is applied to the portal frame, both sides of the beam-to-column connections have an initial hogging moment and vertical shear force (M1, V1) due to vertical load, as can be seen in Fig. 9 (a). The vertical load comes from sand bags, wood pallets and the weight of the horizontal beam itself. Subsequently, when the lateral load is applied, the connection at the left side of the portal frame is subjected to a sagging moment, whereas the connection at the right side undergoes a hogging moment, as shown in Fig. 9 (b). This hogging moment and the shear force caused by the lateral load (M2, V2) at the right side connection will combine with the initial hogging moment and shear force caused by the vertical load (M1, V1). This force combination makes the connection at the right side of the portal frame reach its moment capacity before that on the left side. Another observation from the experimental results is that the moment-rotation curves exhibit looseness at the initial stage of both vertical load levels because the steel tabs do not fit with the column perforation. As shown in Fig. 8, the looseness is represented by a short line with a low slope value at the beginning of the curve.



(a) Deflected shapes under vertical load



(b) Deflected shapes after applying of lateral load

Fig. 9 Illustration of portal frame deflected shapes

3. Mechanical model of beam-to-column connections by component method

Eurocode 3 [10] outlines an analytical procedure so-called component method for determination of the design characteristics of semi-rigid beam-to-column connections. The code suggests that non-linear characteristics of the connection can be represented by linearized approximations such as bi-linear and tri-linear relationships. The design characteristics shall determine two main properties of the connections: moment resistance and rotational stiffness. Fig. 10 illustrates the concept of the analytical procedure outlined in Eurocode 3 for an unstiffened welded beam-to-column connection. The main properties depend on resistance of the components in three critical zones: tension zone, compression zone, and shear zone of the connection.

The component method consists of three main steps: dividing the connection into its basic components in which each one is represented by an elastic spring, evaluation of both stiffness and strength of each component, and finally all components being assembled into one mechanical model to determine the strength and stiffness capacity of the whole connection.

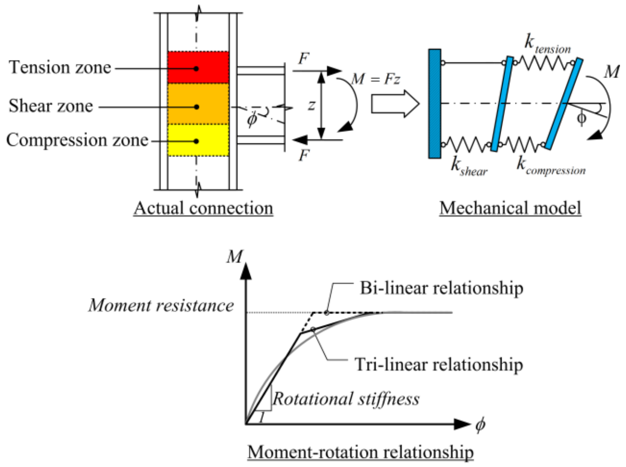


Fig. 10 Systematic model of beam-to-column connection according to Eurocode 3

In accordance with Eurocode 3, the initial rotational stiffness of the connection can be computed as:

$$S_{j,ini} = \frac{M}{\phi} = \frac{Ez^2}{\sum \frac{1}{K_i}} \tag{3}$$

where E is elastic modulus, z is the lever arm between tension and compression springs, and K_i is the stiffness of each component. The connection moment resistance $M_{j,Rd}$ is governed by the weakest component as given by:

$$F_{Rd} = min[F_{Rd,i}] \tag{4}$$

$$M_{j,Rd} = F_{Rd} z \tag{5}$$

where $F_{Rd,i}$ is the strength of component i.

Various mechanical models have been proposed in the past for several connection typologies using the component method [21-25]. However, there is no specific provision for the evaluation of beam-to-column connections for steel storage racks in those contexts. The lack of code specification means that there is considerable scope for development of an analytical procedure for predicting the behavior of beam-to-column connections of steel storage racks. This section provides procedure for determination of the initial stiffness and moment resistance of the rack's connections.

The following components are considered to be the basic components for evaluating stiffness of the beam-to-column connections in steel storage racks as illustrated in Fig. 11.

Column web in tearing ($k_{cw,tear}$)
Column web in bearing ($k_{cw,b}$)
Column web in tension or compression ($k_{cw,t}$ or $k_{cw,c}$)
Column web in shear ($k_{cw,s}$)
Tabs in shear ($k_{t,s}$)
Connector in bending and shear (k_{co})
Connector web in tension or compression (k_{cow})
Beam flange in tension or compression (k_{bf})

The column web in shear component is not a major contributor to the initial rotational stiffness, according to the study of Zhao [26]. Moreover, the shear deformation of the column webs cancels each other out for the internal connection, upon which a two-sided bending moment is acted from the beam. Therefore, in this study, the column web in shear is neglected. The last component is only considered in the determination of the connection flexural strength. The full derivation of the basic components contributing to the mechanical model is outside the scope of this study, however available methodologies can be found in the companion paper [27]. Table 3 provides a summary of the initial stiffness and strength of each basic component.

A mechanical model adopted to predict the initial rotational stiffness of beam-to-column connections in steel storage racks is shown in Fig. 12. Each spring component in a mechanical model would be assembled in parallel or series, depending on the relevant geometries.

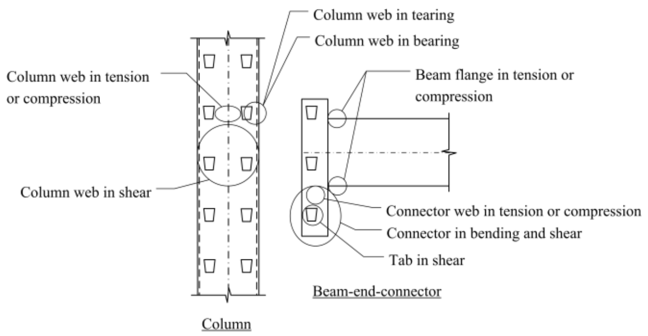
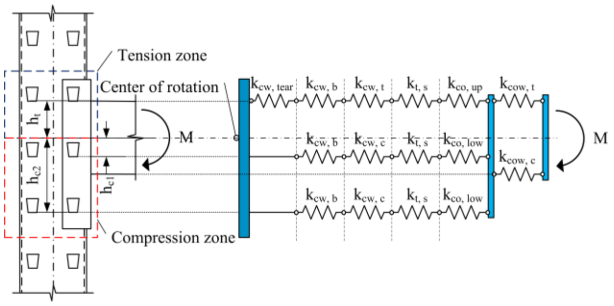
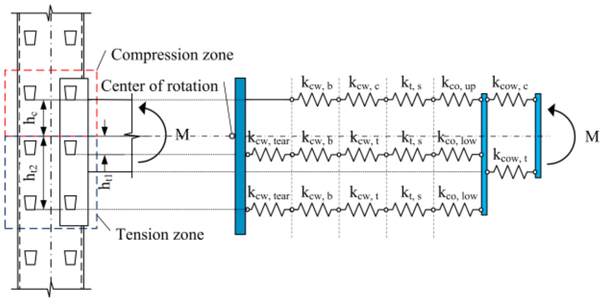


Fig. 11 Basic components of beam-to-column connections in steel storage racks



(a) Hogging moment



(b) Sagging moment

Fig. 12 Mechanical model for determination of the initial rotational stiffness

For determining the initial rotational stiffness, the center of rotation is assumed to be the center of the beam, due to the simplicity of the method, and this assumption agrees with the connection behavior during the initial stage of the experimental testing performed in this study. The initial rotational stiffness of the connection $S_{j,ini}$ can be computed by combining the stiffness of the components given by:

$$\frac{1}{S_{j,ini}} = \frac{1}{h_t^2 K_{eqt}} + \frac{1}{h_c^2 K_{eqc}} \tag{6}$$

where K_{eqt} and K_{eqc} are the stiffness of the equivalent spring in tension zone and compression zone respectively. h_t and h_c are distance from the center of rotation to the location of the tension or compression spring.

For evaluation of the flexural strength, it is assumed that the failure of the connection initially occurs at the outermost row of tabs on the tension side. This assumption is valid in case of connections with a low ratio of beam height to beam-end-connector height; it has been confirmed by many experimental investigations [2] [5-7] and by the author's the experimental testing [11]. The strength of the outermost tab is calculated from the minimum value of the strength of the basic components in a row. Then, the linear distribution of force inside the connection is applied. Finally the strength of the connection is calculated from the equilibrium equation. This procedure is rather simple and gives a more conservative flexural strength than the study of Kozlowski [9]. The center of rotation is assumed to be located at the bottom surface of the beam for the connection subjected to a hogging moment. For the connection subjected to a sagging moment, it is assumed that the center of rotation is located at the top surface of the beam. Mechanical models for evaluation of flexural strength are shown on Fig. 13.

Table 3
Initial stiffness and strength of each basic component [9-10] [22]

Component	Strength	Initial stiffness
Column web in tearing	$F_{cw,tear} = \frac{f_{y,cw} A_{v,cw}}{\sqrt{3}}$	$k_{cw,tear} = \frac{F_{cw,tear}}{\delta_{cw,tear}}$
Column web in bearing	$F_{cw,b} = 2.5\alpha\, f_{u,cw}\, d\, t_{cw}$	$k_{cw,b} = 24 k_b\, k_t\, d\, f_{u,cw}$
Column web in tension	$F_{cw,t} = \omega b_{eff,cw,t}\, t_{cw}\, f_{y,cw}$	$k_{cw,t} = E_{cw} \frac{b'_{eff,cw,t}\, t_{cw}}{d_{wc}}$
Column web in compression	$F_{cw,c} = 107661 \frac{t_{cw}^{3}}{d_{wc}} \sqrt{f_{y,cw}}$	$k_{cw,c} = E_{cw} \frac{b'_{eff,cw,c}\, t_{cw}}{d_{wc}}$
Tabs in shear	$F_{t,s} = \frac{f_{u,co} A_{v,t}}{\sqrt{3}}$	$k_{t,s} = \frac{1}{\left(\frac{l_t^3}{3E_t I_t} + \frac{1.2 l_t}{G_t A_{v,t}} \right)}$
Connector in bending and shear	$F_{co} = \frac{f_{y,co} A_{v,co}}{\sqrt{3}}$	$k_{co} = \frac{1}{\left(\frac{l_{co}^3}{3E_{co} I_{co}} + \frac{1.2 l_{co}}{G_{co} A_{v,co}} \right)}$
Connector web in tension and compression	$F_{cow} = \omega b_{eff,co}\, t_{co}\, f_{y,co}$	$k_{cow} = E_{co} \frac{b'_{eff,co}\, t_{co}}{d_{wc,co}}$
Beam flange in tension or compression	$F_{bf} = b_{eff,bf}\, t_{bf}\, f_{y,bf}$	$k_{bf} = \infty$

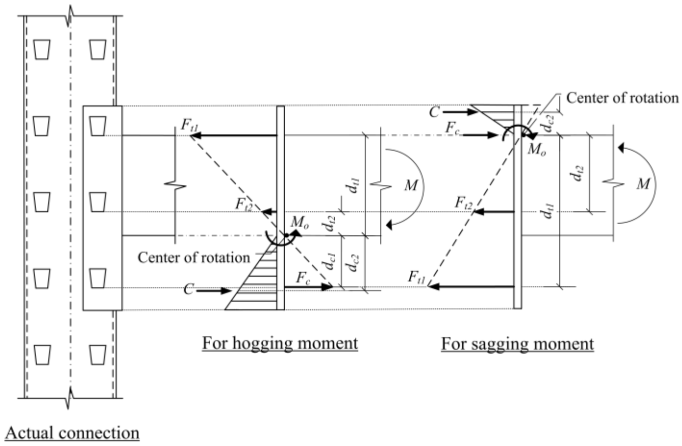


Fig. 13 Mechanical model for flexural strength evaluation

The flexural strength of the connection M_u is given by using the equilibrium equation as follow:

$$M_u = F_{ti} \sum_{i=1}^{2} d_{ti} + M_o \tag{7}$$

where F_{ti} is tension force in the steel tab of i component from the center of rotation, and d_{ti} is the distance of i component from the center of rotation. M_o is the internal moment of the connector at the center of rotation, which can be obtained by using the following expression.

$$M_o = F_c d_{c1} + C d_{c2} \le M_{co,p} \tag{8}$$

where d_{c1} is the distance from the center of rotation to a compression force F_c. d_{c2} is the distance from the center of rotation to compression force C, and $M_{co,p}$ is a plastic moment of the connector. C is the compression force due to the bearing stress between the connector and the column.

The bearing stress is assumed to be a triangle distribution. Using the equilibrium condition, the compression force can be obtained as follows:

$$F_{t1} + F_{t2} = C + F_c \tag{9}$$

where F_{t1} is tension force in the steel tab on the tension side at the outer most point from the center of rotation. F_{t2} is tension force in the steel tab at the second row on the tension side. F_c is compression force in the steel tab on the compression side. F_{t1} is taken as the minimum value of the strength of the basic components in a row as follows:

$$F_{t1} = min(F_{cw,tear}, F_{cw,b}, F_{cw,t}, F_{t,s}, F_{co}) \tag{10}$$

For simplicity, elastic force distribution of internal forces inside the connection is applied. The following equation can be obtained to calculate F_{t2} and F_c.

$$\frac{F_{t1}}{d_{t1}} = \frac{F_{t2}}{d_{t2}} = \frac{F_c}{d_{c1}} \tag{11}$$

where d_{t1} is the distance from the center of rotation to the tension force F_{t1}, and d_{t2} is the distance from the center of rotation to the tension force F_{t2}.

4. Evaluation of mechanical model through experimental testing

In this section, the initial rotational stiffness $S_{j,ini}$ and the flexural strength of the connection M_u are obtained from the proposed mechanical model, and are compared with the experimental testing data to evaluate the proposed mechanical model. Geometrical data and material properties of beam-to-column connections given in section 2.1 are used as the input data for calculation of initial rotational stiffness and flexural strength of the connection by using the proposed mechanical model as described in section 3. Table 4 shows numerical values of the initial stiffness and strength of each basic component for the studied beam-to-column connection. The comparisons of the initial rotational stiffness and flexural strength of the connection from the proposed mechanical model and the cantilever test are shown in Table 5.

It can be seen that the mechanical model closely predicts the flexural strength, and that the difference is on the conservative side, while giving a moderate overestimation of the initial rotational stiffness. In particular, the flexural strength is found to be 5-11 % lower than the experimental values. This may be due to the impact of lack of fit between steel tab and column perforation, and residual stress that usually affects experimental setup [28].

The experimental testing shows that the moment-rotation relation for rack connections exhibits a non-linear behavior, which comes from many factors, such as the yielding of the steel tabs, beam-end connector, or perforated column, which the mechanical model does not take into account in the formulation. This causes the mechanical model to overestimate the initial rotational stiffness by 15-19 %.

Table 4

Characterization of component of the studied beam-to-column connection evaluated by the component method

Component	Strength (kN)	Initial stiffness (kN/cm)
Column web in tearing	$F_{cw,tear}$ = 8.66	$k_{cw,tear}$ = 11123
Column web in bearing	$F_{cw,b}$ = 7.91	$k_{cw,b}$ = 79.11
Column web in tension	$F_{cw,t}$ = 35.19	$k_{cw,t}$ = 790.32
Column web in compreession	$F_{cw,c}$ = 88.98	$k_{cw,c}$ = 675.68
Tabs in shear	$F_{t,s}$ = 6.8	$k_{t,s}$ = 1838
Connector in bending and shear	F_{co} = 18.29	$k_{co,up}$ = 1967 $k_{co,low}$ = 350
Connector web in tension and compression	F_{cow} = 16.87	k_{cow} = 1794
Beam flange in tension or compression	F_{bf} = 16.15	$k_{bf} = \infty$

Table 5

Comparisons of the initial rotational stiffness and flexural strength of beam-to-column connections obtained from the mechanical model and the cantilever tests

		Hogging moment	Sagging moment
Initial rotational stiffness ($S_{j,ini}$)	Experimental result (Average value) (kN-m/rad)	62.50	60.10
	Mechanical model (kN-m/rad)	71.71	71.98
	Mechanical model/ Experiment	1.15	1.19
Flexural strength (M_u)	Experimental result (Average value) (kN-m)	1.33	1.56
	Mechanical model (kN-m)	1.18	1.48
	Mechanical model/ Experiment	0.89	0.95

5. An application of the proposed mechanical model to predict lateral behavior of steel storage frames

5.1. Analysis of lateral behavior of steel storage frames

The proposed mechanical model discussed in section 4 is used in this section to predict lateral behavior of a single story steel storage frame. The predictions are compared with the experimental results from the portal test in terms of base shear-top displacement relation. The portal test results give the mean values of the initial stiffness and flexural strength, whereas the proposed mechanical model distinguishes both connection properties between the hogging and the sagging moment. To indirectly compare the results, a nonlinear static analysis [29], so-called pushover analysis, is applied for this study. Under the pushover analysis, a structure is subjected to gravity loadings as an initial condition. Then the lateral load is applied monotonically in a step-by-step nonlinear static analysis. From the analysis, the numerical results can be present in the form of base-shear top displacement relation that will be referred to as a “pushover curve” in this paper.

The pushover curve of a single story frame using beam-to-column connection properties obtained from the proposed mechanical model is compared with the experimental results from the portal tests. The finite element computer program SAP2000 [30] is used to carry out the pushover analysis in this study. The structural configuration and section properties of the finite element model are calibrated against the portal frame tests. The base plate is modelled as a pined support. Uniformly distributed vertical forces are applied on the beam element to simulate the pallet load. Non-linear behavior of the beam-to-column connection is modelled by a non-linear rotational link element connected between the ends of beams and the ends of columns. The geometric nonlinearities and material nonlinearities are also considered in the finite element model.

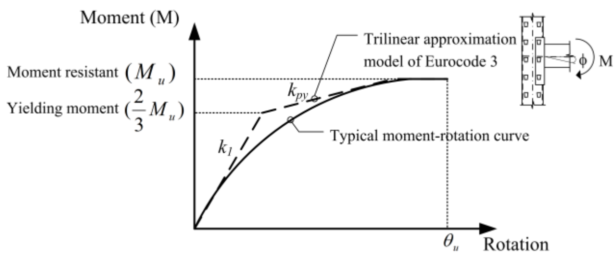
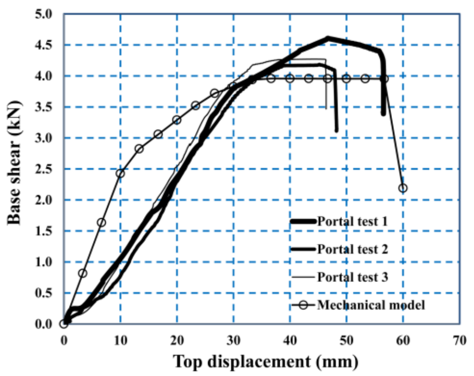
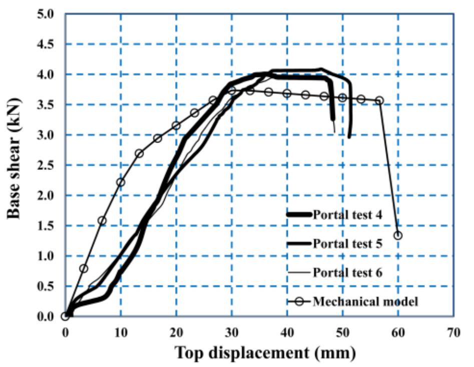


Fig. 14 Typical moment-rotation curve for rack connections and approximation of Eurocode 3

A typical moment-rotation curve for a rack connection is shown in Fig. 14. The moment-rotation relation exhibits non-linear behavior as shown in the section of experimental testing. In order to represent the non-linear behavior of the connection and simplify the analysis, a tri-linear approximation model, in accordance with Eurocode 3, is used in this study. This approximation is widely used for the development of mechanical models for hot roll section. A first yielding moment is assumed to be $2M_u/3$ [11] [22] where M_u is flexural strength; k_1 is initial rotational stiffness; k_{py} is post-yielding stiffness; θ_u is plastic rotation capacity. It should be noted that the value of θ_u and k_{py} are chosen to give the best-fit to the experimental results from the cantilever test. The prediction of steel storage frame behavior under lateral loads by using an application of the proposed mechanical model is shown in Fig. 15 (a) -15 (b) for no loading and normal loading respectively. The figures present the comparison between base shear-top displacement relation from the portal test and pushover curves of portal frame with a trilinear approximation of moment-rotation relation for the proposed mechanical model.



(a) No loading



(b) Normal loading

Fig. 15 Comparison of the portal test results and pushover analysis of portal frame with approximation of rack connections

From the resulting curves, it can be seen that the mechanical model give a conservative prediction of maximum base shear for all testing frames. On the other hand, the predicted displacements of the proposed mechanical model underestimate the displacement compared with the portal test results. The predicted displacement of the frame is greatly influenced by the initial rotational stiffness of the connection. It can be seen from the cantilever test in section 2.2 that the stiffness of rack connections start to deviate from the initial rotational stiffness at low value of moment. The stiffness is increasingly softened at moments approaching the ultimate moment. Therefore using the initial rotational stiffness for the rack connection model may underestimate the displacement response of the steel storage frame.

5.2. Improvement of rack's connection model

It has been shown in the previous section that the lateral behavior of steel storage frames is significantly influenced by the rack connection model. This section presents the further improvement of the rack connection model for a better prediction of the lateral behavior.

It is evident from the comparison of the portal test and the pushover analysis that the mechanical models were slightly too stiff in comparison to the actual frame, thus resulting in too low a prediction of the lateral displacement. The two sources of the problem that were addressed are the accuracy of initial stiffness to represent the stiffness of the connections as the loading increased, and the presence of the initial looseness of the actual connections.

The linear idealization of the moment-rotation characteristic of the beam-to-column connections based on the use of the secant stiffness is applied for improvement, as shown in Fig. 16. This approach is recommended by Eurocode 3 [10]. The secant stiffness is defined as $S_{j,ini}/n$. The stiffness modification factor *(n)* results from the non-linearity of the connection. Generally, a stiffness modification factor of 2.0-3.5 is suggested by Eurocode 3, depending on connection types. Based on a comprehensive experimental study of beam-to-column connections under bending moment in this study, a numerical value of 1.5-2.0 is proposed for the stiffness modification factor.

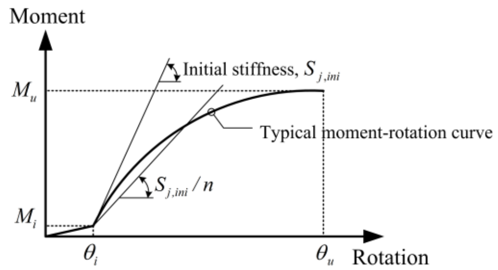
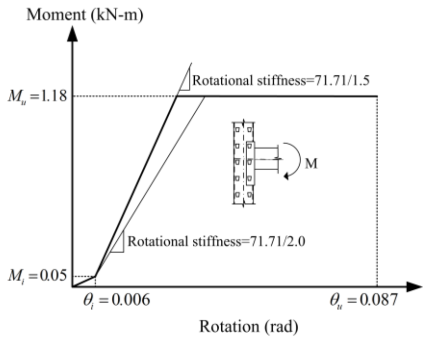
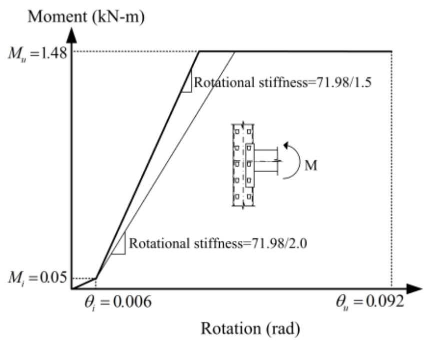


Fig. 16 Improvement of rack connection model

It is also shown in the experimental testing of rack connections that some of the moment-rotation curves show a looseness of the connection at the initial stage. The looseness contributes to the overall deformation of the portal frame; hence, it needs to be incorporated into the connection model. A flat slope of linear line is introduced in the connection model to represent looseness of the connection, as shown in Fig. 16. It should be noted that M_i and θ_i are moment and rotation, where the initial looseness is terminated. θ_i can be calculated from the gap between steel tabs and the column perforation, or can be determined by the conventional cantilever test. In this study, M_i and θ_i are taken from the cantilever test as 0.05 kN-m and 0.006 radian respectively. These numerical values, M_i and θ_i, produce a low value of stiffness in the initial looseness stage, which is about 10% of the initial rotational stiffness. The numerical values of the connection model for hogging moment and sagging moment of the connection for frame analysis are shown in Fig. 17 (a)-17 (b). In Fig. 17, the initial stiffness $(S_{j,ini})$ and flexural strength (M_u) of the connection are calculated from the proposed mechanical model. θ_u is rotation, where the steel tab is cut by the column perforation, which is taken from cantilever test. The pushover analysis is conducted for the portal frame with the connection model presented in Fig. 17. The comparison between the frame analysis results and the experimental results of the portal test are shown in Fig. 18 (a)-18 (b).

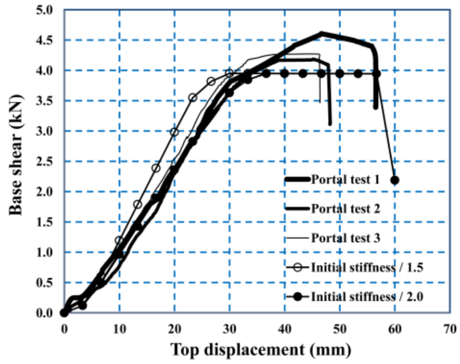


(a) Hogging moment

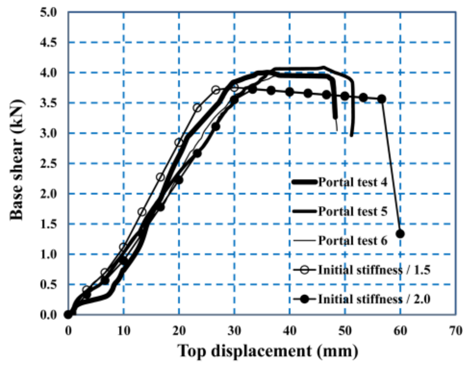


(b) Sagging moment

Fig. 17 Linearization of moment-rotation characteristic of rack connections



(a) No loading



(b) Normal loading

Fig. 18 Comparison of the portal test results and pushover analysis of portal frame with modification of rack connection model

As expected, introducing the stiffness modification factor *(n)* and the initial looseness to the mechanical model improves the accuracy of the lateral-displacement prediction from frame analysis, and the improved pushover curves agree well with the portal frame testing results, as shown in Fig. 18. To this end, modification of initial rotational stiffness and inclusion of the initial looseness to the modeling are deemed necessary for a better prediction of lateral behavior of steel storage frames.

6. Summary and conclusions

This research presents the development of a mechanical model for beam-to-column connections of steel storage racks using a component method recommended in Eurocode 3. A proposed mechanical model of the rack's connections is derived from the combination of the characteristics of the basic structural components of the connections. Initial rotational stiffness and flexural strength yielded from the mechanical model, along with simple linearized moment-rotation relationship, can be used as the estimated characteristics of any steel rack connection under both hogging and sagging moments. The study proposes a simple methodology for evaluation of the flexural strength based on the actual failure mode.

The initial mechanical model adequately predicts the flexural strength with acceptable degree of accuracy. However, the simple linearized moment-rotation relationship based the initial stiffness and moment resistance underestimates the lateral displacement when it is applied to the pushover analysis. Consequently, the study proposes further refinement of the moment-rotation relationship by introducing initial looseness and replacing the initial stiffness with secant stiffness. It is recommended that the modification factor for the initial stiffness to secant stiffness be in the range between 1.5 and 2.

The proposed mechanical model can be used to predict the response of a connection, as well as the structural response, subjected to both vertical loads, and horizontal loads, with a fraction of testing costs and time. Also, the proposed model can be easily modified to predict the connection behavior for other geometry of connections for these types of structures.

References

[1] Markazi F.D., Beale R.G. and Godley M.H.R., "Experimental analysis of semi-rigid boltless connectors", Journal of Constructional Steel Research, 28(1), 57-87, 1997.
[2] Bernuzzi C. and Castiglioni C.A., "Experimental analysis on the cyclic behaviour of beam-to-column joints in steel storage pallet racks", Thin- Walled Structures, 39(10), 841-859, 2001.
[3] Aguirre C., "Seismic behavior of rack structures", Journal of Constructional Steel Research, 61(5), 607-624, 2005.
[4] Abdel-Jaber M., Beale R.G. and Godley M.H.R., "A theoretical and experimental investigation of pallet rack structures under sway", Journal of Constructional Steel Research, 62(1), 68-80, 2006.
[5] Bajoria K.M. and Talikoti R.S., "Determination of flexibility of beam-to-column connectors used in thin walled cold-formed steel pallet racking systems", Thin- Walled Structures, 44(3), 372-380, 2006.
[6] Prabha P., Marimuthu V., Saravanan M. and Jayachandran A.S., "Evaluation of connection flexibility in cold formed steel racks", Journal of Constructional Steel Research, 66(7), 863-872, 2010.
[7] Zhao X., Wang T., Chen Y. and Sivakumaran K.S., "Flexural behavior of steel storage rack beam-to-upright connections", Journal of Constructional Steel Research, 99(8), 161-175, 2014.
[8] Shah S.N.R., Sulong N.H.R., Khan R. and Jumaat M.Z., "Structural performance of boltless beam end connectors", Advanced Steel Construction, 13(2), 144-159, 2017.
[9] Kozlowski A., Rzeszow and Sleczka L., "Preliminary component method model of storage rack joint", Proceeding of the fifth international workshop on connections in steel structures, Delft University of Technology, Netherlands, June, 253-262, 2004.
[10] CEN, "Eurocode 3 - Design of steel structures Part 1-8: Design of joints", CEN European Committee for Standardization, Brussels, Belgium, 2005.
[11] Asawasongkram N., Chomchuen P. and Premthamkorn P., "Experimental analysis of beam-to-column connection in steel storage racks using cantilever test and portal test method", Proceeding of the 13th East Asia-Pacific Conference on Structural Engineering and Construction, Sapporo, Japan, Paper No.268, 2013.
[12] Krawinkler H., Cofie N.G., Astiz M.A. and Kircher C.A., "Experimental study on the seismic behavior of industrial storage racks (Report no. 41)", The John A. Blume Earthquake Engineering Center, Department of Civil Engineering, Stanford University, Stanford, 1979.
[13] Harris E., "Sway behavior of high rise steel storage racks", Ph. D. Thesis, University of Sydney, Sydney, Australia, 2006.
[14] JIS, "JIS Z2241: Method of tensile test for metallic materials", Japanese Industrial Standards Committee, Tokyo, Japan, 1998.
[15] ASTM, "A611: Standard Specification for Structural Steel (SS), Sheet, Carbon, Cold-Rolled", ASTM American Society for Testing and Materials, PA, USA, 1997.
[16] ASTM, "A283: Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates", ASTM American Society for Testing and Materials, PA, USA, 2003.
[17] ASTM, "A36: Standard Specification for Carbon Structural Steel", ASTM American Society for Testing and Materials, PA, USA, 2014.
[18] AS, "AS 4084: Steel storage racking", Australian Standards, Sydney, Australia, 1993.
[19] FEM, "FEM 10.2.02: Recommendation for the design of steel pallet racking and shelving", European Racking Federation, Birmingham, UK, 1998.
[20] RMI, "MH 16.1 - Specification for the design, testing and utilization of industrial steel storage rack", RMI - Rack Manufactures Institute, Charlotte, USA, 2008.
[21] Huber G. and Tschemmernegg F., "Modeling of beam-to-column joints - Test evaluation and practical", Journal of Constructional Steel Research, 45(2), 199-216, 1998.
[22] Faella C., Piluso V. and Rizzano G., "Structural steel semi-rigid connections – Theory, design and software", CRC Press LLC, Boca Raton, Florida, USA, 2000.
[23] Pucinotti R., "Top-and-seat and web angle connections: prediction via mechanical model", Journal of Constructional Steel Research, 57(6), 661-694, 2001.
[24] Silva L.S. and Coelho A.G., "A ductility model for steel connections", Journal of Constructional Steel Research, 57(1), 45-70, 2001.
[25] Beg D., Zupancic E. and Vayas I., "On the rotation capacity of moment connections", Journal of Constructional Steel Research, 60(3-5), 601-620, 2004.
[26] Zhao X., Dai L., Wang T., Sivakumaran K.S. and Chen Y., "A theoretical model for the rotational stiffness of storage rack beam-to-upright connections", Journal of Constructional Steel Research, 133(6), 269-281, 2017.
[27] Asawasongkram N., Chomchuen P. and Premthamkorn P., "Mechanical modelling of beam-to-column connections of steel storage racks", Journal of Constructional Steel Research, (to be submitted).
[28] Bursi O.S. and Jaspart J.P., "Calibration of a finite element model for isolated bolted end-plate steel connections", Journal of Construction Steel Research, 44(3), 225-262, 1997.
[29] Applied Technology Council, "ATC-40 Seismic evaluation and retrofit of concrete buildings", Redwood City, 1996.
[30] CSI, "SAP2000 - Linear and nonlinear static and dynamic analysis of three-dimensional structures", Computer & Structures Inc., Berkeley, CA, USA, 2009.

List of notation

$b_{eff,bf}$	Effective width of beam flange
$b_{eff,co}$	Effective width of connector web
$b'_{eff,co}$	Effective width of connector web for stiffness calculation
$b_{eff,cw,t}$	Effective width of column web in tension zone
$b'_{eff,cw,c}$	Effective width of column web in compression zone for stiffness calculation
$b'_{eff,cw,t}$	Effective width of column web in tension zone for stiffness calculation
d	Thickness of steel tab
d_{wc}	Depth of column web
$d_{wc,co}$	Depth of connector web
$f_{y,bf}$	Yield stress of beam flanges
$f_{y,co}$	Yield stress of connector web
$f_{u,co}$	Ultimate tensile strength of connector web
$f_{y,cw}$	Yield stress of column web
$f_{u,cw}$	Ultimate tensile strength of column web
l_t	The distance between the concentrated shear force and fixed end of the tab
l_{co}	The distance between the concentrated shear force and

	fixed end of the connector web
t_{bf}	Thickness of beam flanges
t_{co}	Thickness of connector web
t_{cw}	Thickness of column web
$A_{v,co}$	Shear area of the connector web
$A_{v,cw}$	Shear area of column web
$A_{v,t}$	Shear area of the tab
E_{co}	Modulus of elasticity of connector web
E_{cw}	Modulus of elasticity of column web
E_t	Modulus of elasticity of the tab
G_{co}	Tangential modulus of elasticity of connector web
G_t	Tangential modulus of elasticity of the tab
I_{co}	Moment of inertia of connector web
I_t	Moment of inertia of the tab
α	Ratio between ultimate tensile strength of connector web and ultimate tensile strength of column web
$\delta_{cw,tear}$	Deformation of column web under strength of column web in tearing
ω	Reduction factor to allow for the possible effects of shear in the column web panel