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Investigation on the monotonic behavior of the steel rack upright-beam column connection

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Abstract. The cold-formed steel storage racks are extensively employed in various industries applications such as storing products in reliable places and storehouses before distribution to the market. Racking systems lose their stability under lateral loads, such as seismic actions due to the slenderness of elements and low ductility. This justifies a need for more investigation on methods to improve their behavior and increase their capacity to survive medium to severe loads. A standardized connection could be obtained through investigation on the moment resistance, value of original rotational stiffness, ductility, and failure mode of the connection. A total of six monotonic tests were carried out to determine the behavior of the connection of straight 2.0 mm, and 2.6 mm thickness connects to 5 lug end connectors. Then, the obtained results are benched mark as the original data. Furthermore, an extreme learning machine (ELM) technique has been employed to verify and predict both moment and rotation results. Out of 4 connections, increase the ultimate moment resistance of connection by 13% and 18% for 2.0 mm and 2.6 mm upright connection, respectively.

Keywords: steel storage rack structures; upright; cyclic loading; moment-rotation

1. Introduction

Since the various industries systems are developing increasingly, the well-suited warehouse system should be designed and optimized to provide a safe and reliable place for products. Therefore, steel storage rack structures have been proposed and developed in various applications. "Products in warehouses are stored in rack systems which are load-bearing structures. These products are stored on pallets or in box-containers. Racks are produced from cold-framed steel elements such as upright frames, beams, and decking. Special beam to column (upright) connections and bracing systems are used to provide a three-dimensional steel structure with "aisles" passage for industrial trucks, order pickers and stacker cranes to reach the storage positions (CEN 2009, Shariati *et al*. 2019e, Taheri *et al*. 2019).

Fig. 1 indicates a typical rack system containing upright, beam, frame bracing members, base plate connections, and boltless beam to upright connections.

The closed rectangular boxed configuration or open "C" sections are the typical shapes of rack systems in the industry. On the other hand, the thin-walled open sections with perforations are the uprights that provide beam end connections to the uprights by either bolts or hooks. Similar to other structural setups, rack systems should be connected to the floor employing unique upright base (base plate) connections.

Due to their slenderness, rack structures can be vulnerable to lateral loads such as seismic actions or side-impact, and for this purpose to ensure stability of the system, moment (or dual) resisting frame system is used in longitude (i.e., Down Aisle), and bracing system is mainly used in transverse (i.e., Cross Aisle) direction. (See Fig. 2).

In comparison to conventional structural systems, very few investigations have been conducted to evaluate the performance of cold-formed rack structures under static and particularly dynamic reversal loads. Therefore, the current racking codes and specifications for static and seismic design of the rack structures are not yet well-consolidated based on thorough research and investigations on different rack structures. Hence, there is an urgent need for further research on different aspects of the rack structures behaviour and possible improvements.

The application of standard methods of analysis and design of conventional structural systems for the design of storage racks is yet acceptable in the rack industry since there is no more accurate alternative method for analysis and design of rack structures in particular. However, using the analysis and design rules and specifications of other steel structural frames for designing a rack system may not be reasonable because of some fundamental differences

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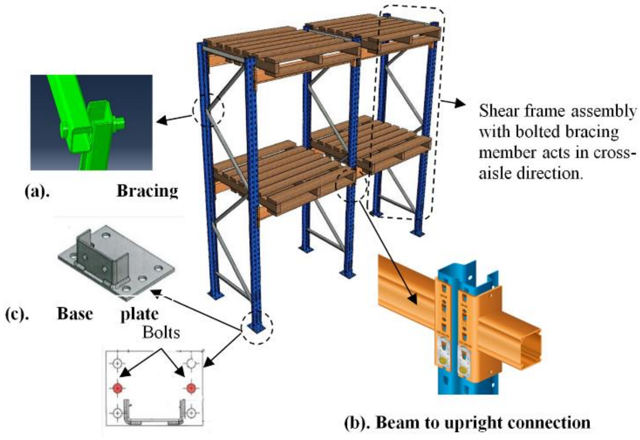
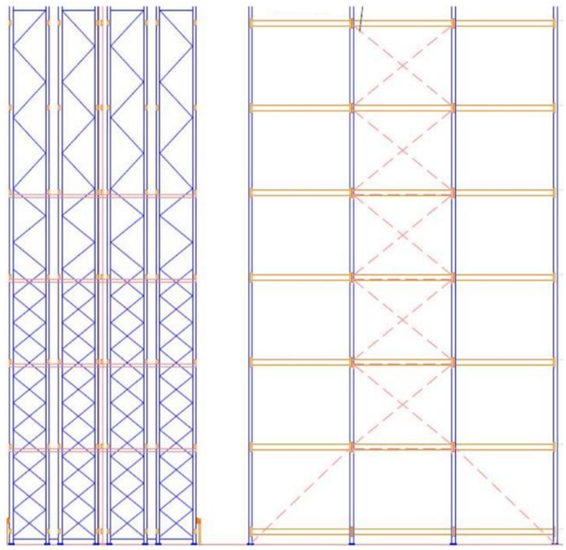


Fig. 1 Typical rack system (Firouzianhaji 2016)



(a) Cross aisle direction (b) Down aisle direction

Fig. 2 Cross aisle and down aisle direction views of typical racking systems (Firouzianhaji 2016)

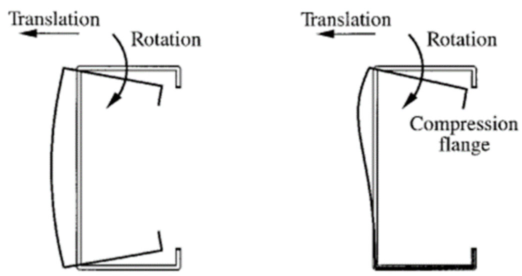
between them. Some of the most critical differences are outlined in the following:

- Aspect ratio: despite structural systems which usually have a ratio between 1 to 3 between their two load-bearing directions in-plane, racking structures dimension in down aisle direction is much more than the cross-aisle direction, sometimes up to 100 times.
- Dead load to Live load ratio: this is less than other conventional structures and is very important in seismic design.
- Slenderness: slenderness of thin-walled open perforated sections and stiffness of base plate and beam to upright connection are low, which makes the racking structure very susceptible under lateral load.
- Low ductility: the typical connections (beam to upright/base plate/frame bracing) in current racking system connections result in low ductility in these structures.
- Free pallets: pallets are not clamped to the beams and can move under seismic actions that may end up having less seismic loads exerted to racking structures due to frictional damping effects.

Considering the above points, racking systems lose their stability under lateral loads such as seismic actions due to the slenderness and low ductility. These create an urgent need for more investigation on a more reliable design improvement to increase the capacity of the rack structures to survive medium to severe earthquakes and heavy loadings.

Constructed from thin gauged (less than 3.5 mm) steel profiles, cold-formed structures in general, and racking systems in particular, are subjected to loss of stability due to different buckling modes taking place in beam or column profiles. The particular design obstacles are created due to using thinner material and cold-framing process, whereas these problems are not available in hot-rolled constructions.

Generally, experimental studies have been performed to investigate an innovative and novel issue of some structural elements (Arabnejad Khanouki *et al*. 2016, Khorami *et al*. 2017b, Khorramian *et al*. 2017, Mansouri *et al*. 2017, Shariati *et al*. 2017, 2019c, Toghroli *et al*. 2017, 2018b, c, Heydari *et al*. 2018, Hosseinpour *et al*. 2018, Ismail *et al*. 2018, Nosrati *et al*. 2018, Zandi *et al*. 2018, Ziaei-Nia *et al*. 2018, Chen *et al*. 2019, Davoodnabi *et al*. 2019, Li *et al*. 2019, Luo *et al*. 2019, Milovancevic *et al*. 2019, Sajedi and Shariati 2019, Cao *et al*. 2020, Zhu *et al*. 2020, Suhatril *et al*. 2019, Xie *et al*. 2019). In the past, scholars have studied various buckling modes of commonly used cold-formed steel sections (Zhao *et al*. 2014, Shah *et al*. 2016a, b, c, Nguyen-Thoi *et al*. 2017, Shariati *et al*. 2018, Chen *et al*. 2019, Mehrmashhadi *et al*. 2019). The local buckling may happen prior to section yielding when the thicknesses of individual plate elements of cold-formed sections are usually small in comparison with their widths. Though, the local buckling presence of an element does not necessarily mean that its load capacity has been obtained. If such an element is stiffened by other elements on its edges, it possesses still greater strength, called "post-buckling strength. In most cold-framed parts, the local buckling is expected to happen, and often it has a superb economy compared to another part that does not buckle locally. As indicated in Fig. 3(b), a mode of buckling which is known as distortion buckling occurs in sections that are braced



(a) (b)

Fig. 3 Distortional buckling modes: (a) Compression; (b) flexure (Hancock *et al*. 2001) against lateral or torsional-flexural buckling. This condition can take place for members in flexure or compression (Hancock *et al*. 2001).

Different types of shear connectors have been introduced to eliminate the lack of ductility and the interlocking strength between the I-beam and concrete slab. Using C-shaped connectors enhanced both ductility and shear strength especially in the steel-concrete boundaries. On the other hand, exposing higher temperatures changes the normal status of the connector performances. Hence, various approaches have been proposed to mitigate the strength loss at elevated temperatures (Shariati 2013, Shariati *et al*. 2010, 2012a, d, 2014a, b, 2015b, 2020a, c, g, Khorramian *et al*. 2016, Shahabi *et al*. 2016b, Tahmasbi *et al*. 2016, Nasrollahi *et al*. 2018, Paknahad *et al*. 2018, Wei *et al*. 2018, Safa *et al*. 2020).

Non-destructive techniques have been performed to investigate the different structural performances such as compressive strength and flexural strength of concrete. Pulse velocity and Schmidt hammer are the two old fashioned techniques of non-destructive assessment where could be used for investigation in rack systems (Shariati 2008, Hamidian *et al*. 2011, Shariati *et al*. 2011a, Alaska *et al*. 2020a, b).

Different investigations have studied the dynamic response of the mid-rise building, and composite structures under seismic events. Also, various types of loading scenarios such as full-cyclic, half cyclic, reversed cyclic, and shake table have been employed to evaluate structural behaviour of the specimens in full-scale or half-scale tests (Arabnejad Khanouki *et al*. 2010, Daie *et al*. 2011, Jalali *et al*. 2012, Khorami *et al*. 2017a, Armaghani *et al*. 2020, Naghipour *et al*. 2020, Shariati *et al*. 2020b, d, e, f).

As previously mentioned, the Cold-formed steel section has been also employed for storage uses and industrial applications such as steel racking and upright-beam systems. On the one hand, time-consuming and costly issues of experimental studies have been a barrier to the novel and innovative approaches, on the other hand, there are appealing methods which could appropriately cover the aforementioned shortcomings. Furthermore, artificial intelligence techniques have been introduced to structural engineering problems and their efficiency has been proved in both the prediction and optimization of the test results. Moreover, combining different classic numerical methods and optimization techniques have successfully presented reliable results. These techniques have been developed throughout the years in order to apply in different structural issues not only for the handy process and even simple calculations compared to the FE method but also for the shorter time that needs to obtain the results (Arabnejad Khanouki *et al*. 2011, Sinaei *et al*. 2011, Mohammadhassani *et al*. 2014a, d, Shah *et al*. 2015, Toghroli 2015, Shariat *et al*. 2018, Shariati *et al*. 2019a, Taheri *et al*. 2019, Trung *et al*. 2019b, Safa *et al*. 2020).

Although there are several investigations have been conducted on buckling of thin-walled columns, very few researches have been performed on the methods of improvement of behaviour and strength of cold-formed steel open columns. This study explains the details of such

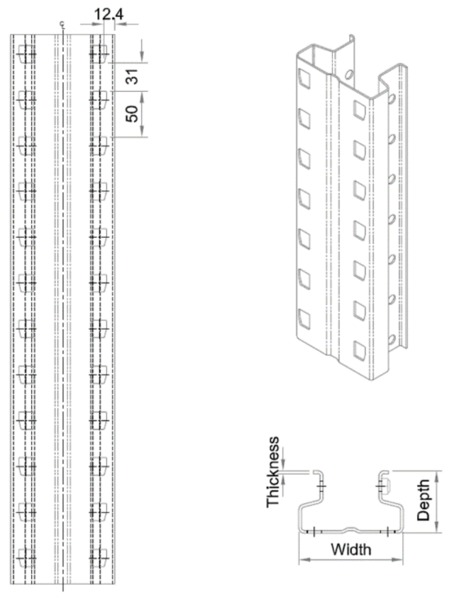


Fig. 4 Upright geometry

research. Several available techniques have been employed for data validation, where the best methods have been reported as extreme learning machine (Shariati *et al*. 2019b, Trung *et al*. 2019a), genetic programming, neural network and other natural basis functional networks (Mohammadhassani *et al*. 2013, 2014b, Schumacher and Shariati 2013, Toghroli *et al*. 2014b, Safa *et al*. 2016a, b, Shahabi *et al*. 2016c, Khorramian *et al*. 2017, Sadeghipour Chahnasir *et al*. 2018, Katebi *et al*. 2019, Milovancevic *et al*. 2019), also finite element and finite strip method have been proved to be as a reliable data authentication and prediction (Sinaei *et al*. 2012, Sharafi *et al*. 2018a, b, c, d, Kildashti *et al*. 2019, Shariati *et al*. 2019f, Taheri *et al*. 2019, Mortazavi *et al*. 2020). This study employs an ELM algorithm to verify the test data and predict the moment and rotation results.

2. Methodology

2.1 Monotonic test

According to the European Standard EN15512:2009, Clause A.2.4 - Bending tests on the beam end connector, the monotonic test was carried out (CEN 2009). Such a bending test on the beam end connection is an effective way to specify the rotational stiffness and the bending strength (failure moment) of the beam end connector relative to the connection. This test represents the global structure behaviour since a beam-to-upright connection is a critical assembly of the global racking structure. However, the monotonic test does not represent the behaviour under dynamic loading, i.e., seismic. This test is practicable only for the connection which has not been designed for seismic loading. Furthermore, the results of the tensile coupon test have tabulated in Table 1.

Table 1 Summary of material specificatony

Component	Thickness, t [mm]	Average yield stress, f_y [MPa]	Average ultimate stress, f_u [MPa]
Column	2.0	483.0	562.3
	2.6	493.0	577.7
Beam	1.5	325.0	442.0
Connector	4.0	315.0	385.0

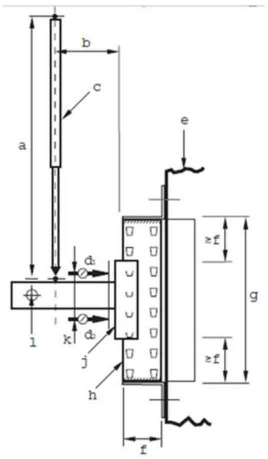


Fig. 5 Upright-beam test setup (CEN 2009)

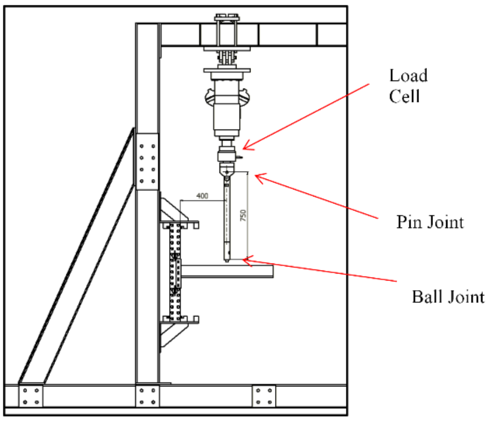


Fig. 6 Layout of the test rig

2.2 Monotonic test rig and setup

In order to verify that the experiment is appropriately performed, several important studies have been carried out because of its high sensitivity to frictional force. To provide frictionless lateral restraint of the beam element, very smooth sideways guidance must be considered at the end of the beam profile, which prevents the torsional twist of the beam profile. In order to ensure that the lugged connection is completely "locked" in the cache of the upright component, the pre-load impact on the connection is important prior to running the test. If the lugged connector has not fully entered the cache of the upright component, the looseness of the connection will hugely decrease the stiffness of the beam. Figs. 5 and 6 show the test setup and layout of the monotonic test rig, respectively.

where,

a $\geq 750\ mm$
b $400\ mm$
c *Loading Stroke*
d *Displacement Sensor*
e *Testing Structure Frame*
f *Upright Face Width*
g *Length of Test Specimen*
h *Upright*
j *Beam End Connector*
k *Lateral Restrain of the Beam*

2.3 Monotonic test results analysis

According to the standard EN 15512:2009(CEN 2009), the unadjusted moment (M_{ti}) and rotation (θ_{ti}) for the individual test sample *i* were computed as follows

$$M_{ti} = a.F_{max} \tag{1}$$

$$\theta_{ti} = \frac{d_2 - d_1}{k} \tag{2}$$

The unadjusted M-θ curves for each individual test can be found in Appendix B,

Where

M_{ti} = Measured failure moment
F_{max} = Maximum measured force
d = Distance measured by the displacement transducer.

The correction factor (C_m) for the upright component, beam and beam end connector was determined using the following equation

$$C_m = \left[\left(\frac{f_y}{f_t}\right)^{\alpha}\left(\frac{t}{t_t}\right)\right] \ but\ C_m \leq 1.0 \tag{3}$$

where

f_t = Measured yield stress for the relevant component;
f_y = Nominal yield stress for the relevant component;
t_t = t = Measured thickness for the relevant component;
t = Nominal thickness for the relevant component;
$\alpha = 0$ when $f_y \geq f_t$
$\alpha = 1.0$ when $f_y < f_t$

The largest correction to the test values was used, irrespective of whether the end connector or the upright component had failed. Though the beam is not expected to be failed or $f_t \geq 1.25 \times f_y$, the correction relating to beam materials would be taken into account. A correction factor below 15% was neglected. The corrected failure moment thus becomes

$$M_{ni} = M_{ti}.x\ C \tag{4}$$

Where

$$C = 0.15 + C_m \quad \text{but} \quad C \leq 1.0 \tag{5}$$

The design value of the moment capacity, M_{rd}

$$M_{rd} = \eta \frac{M_{ti}}{\gamma_M} \tag{6}$$

Where

$\gamma_M =$ Partial safety factor of connection, 1.1
(Equivalent to the inverse of capacity reduction factor θ = 0.9 for Australian Standard)
$\eta =$ Variable moment reduction factor selected by the designer, 1.0
$M_k =$ Characteristic value of the failure moment (see below)

$$M_k = M_m - k_s . s \tag{7}$$

Where s is the standard deviation of the sample

$$s = \sqrt{\frac{1}{(n-1)} \sum_{i=1}^{n} (M_{ni} - M_m)^2} \tag{8}$$

$k_s =$ 3.37 (refer to Table 2 below)
$M_m =$ Mean value of the adjusted results (see below)

$$M_m = \frac{1}{n} \sum M_{n,max} \tag{9}$$

Where

$M_{ni} =$ individual test result, adjusted as shown in Table 2.

Once the adjusted curves for each individual test have been plotted, the bi-linear curve can be derived (refer to EN 15512:2009 Section A.2.4.5.2).

k_0 = initial slope of the unadjusted curve
θ_{slip} = initial slip observed (if any)
$\theta_{n,i}$ = rotation value at the design moment $M_{ni}$for test sample i

Table 2 ks coefficient based on 95% fractile at a confidence level of 75%

n	3	4	5	6	7	8
k_s	3.37	2.63	2.33	2.18	2.08	2.00

$$k_d = \frac{1}{n} \sum_{i=1}^{n} k_{ni} \tag{10}$$

Where

k_{ni} is the average slope of the line through the origin (refer to EN 15512:2009)

2.4 Monotonic test configuration

The total number of research work under monotonic test is listed in Table 3 below.

2.5 Data acquisition and instrumentation

The test cycle is indicated in Fig. 7. The control signal is generated by the computer to the actuator, which has a stroke length of 100 mm (± 50 mm) to press on the beam specimen. The load cell with 100 KN capacity measures the forces, while the two linear variable differential transformers (LVDTs) measure the rotation of the connection. The data were collected through a data logger, sent to the computer, and then processed with Microsoft Excel. The LVDT 1 and LVDT 2 measure the rotational values. At a frequency of 2 Hz, the data is measured which means every 0.5 s.

3. Results and discussions

Fig. 8 shows the moment rotation (MR) curve for the results of the monotonic test. First, the obtained results of X1 reveal a linear behavior which is demonstrated by the stiffness curve k_0 in the diagram. After the linear part k_0, the MR curve has a tendency to show non-linear behaviour before reaching the peak moment. In terms of rotation in the non-linear section, the connection rotation occurs more than twice of the linear section in the MR curve. The failure could be caused by the following factors which are observed through the failure mode of the specimen.-

- Looseness between upright and beam end connector lug connection
- Yielding at the upright front holes because of the localize stress concentration
- The end connector deformation.

Table 3 Test specimen geometry and details

Upright-beam specimen	Upright	Beam	Number of replicated test
X1	Length (cm) = 112.2 Width (cm) = 67.6 Thickness (mm) = 2.0	Length (cm) = 105 Width (cm) = 50 Thickness (mm) = 1.5	3
X2	Length (cm) = 113.1 Width (cm) = 68.3 Thickness (mm) = 2.6	Length (cm) = 105 Width (cm) = 50 Thickness (mm) = 1.5	3

*Remarks:- "M" represent for monotonic test

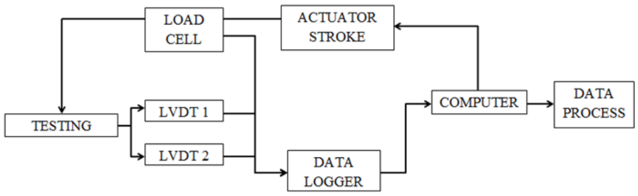


Fig. 7 Data acquisition and instrumentations

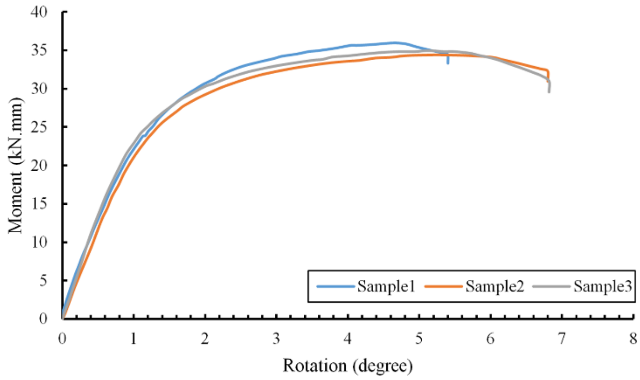


Fig. 8 Test data for X1 specimens

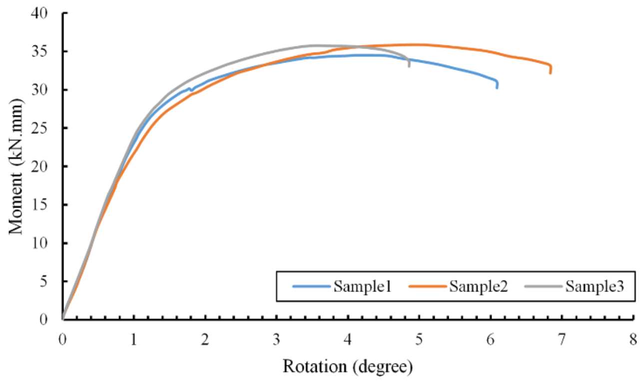


Fig. 9 Test data for X2 specimens

Table 4 Comparison chart for monotonic test

Monotonic			
Characteristic	X1	X2	Differences
Characteristic failure moment, M_k [kNcm]	32.57	32.76	0.58%
Design failure moment, M_{rd} [kNcm]	29.61	29.79	0.58%
Design stiffness, k_d [kNcm/rad]	10412.94	11494.86	10.39%

For Monotonic Test Results for X2, Fig. 9, as shown by the stiffness curve K_0, a linear behavior, in the beginning, can be seen in the MR curve. Also, after the linear section k_0, the MR curve has a non-linear performance before reaching the peak moment. In terms of rotation in the non-linear section, the connection rotation occurs more than twice of the linear section in the MR curve. The factors of the failure mode can be achieved by observing the failure mode of the specimen:-

- Looseness between upright and beam end connector lug connection
- Yielding at the connection tabs due to localize stress concentration

The end connector deformation.

From Table 4, the monotonic test for X1 and X2 produced very similar values of particular failure moment, M_k. Though, the stiffness of both experiments shows a difference of about 10.39%, which shows that at 2.6 mm, the connection of beam to the upright is actually "tighter" compared to 2.0 mm upright. However, the similarity of failure moments for both tests is very close to each other. X1 failure mode occurred at the tearing of the upright whereas X2 failure mode occurred at the tearing at the end connector lug. The differences of 0.58% and the difference in failure mode suggest that connectors with 4mm thickness have more resistance in failure compared to the upright.

4. Extreme Learning Machine (ELM)

4.1 General

Recent researches by Trung *et al*. 2019, Shariati *et al*. 2020h, i) proposed the extreme learning machine as an artificial intelligence (AI) tool for single-layer feed-forward neural network (SLFN) architecture. In the ELM algorithm, the weights of SLFN input are selected randomly, while the output weights are analytically determined. The most considerable advantage of the ELM algorithm over other intelligence methods is its breakneck speed in finding the weights of the network (Armaghani *et al*. 2019, Xu *et al*. 2019, Shariati *et al*. 2019b, d). Also, ELM systematically determines all the network factors and therefore prevents unnecessary interference of humans. This method offers a different approach from ANN (Shariati *et al*. 2011b, c, d, 2012b, c, e, 2013, 2015a, b, 2016, Mohammadhassani *et al*. 2013, Toghroli et al. 2014b, Shahabi *et al*. 2016a), ANFIS (Mohammadhassani *et al*. 2014c, Toghroli *et al*. 2014a, Hamdia *et al*. 2015, Safa *et al*. 2016a, Sedghi *et al*. 2018, Toghroli *et al*. 2018a), and SVM as it uses ELM algorithm for finding the weights of the SLFN. ELM is a newer tool in comparison with the intelligence methods as mentioned above. Many benefits of this approach have increased its popularity and usage so that the performance of this method has been evaluated in different fields of study.

A three-step procedure is involved in developing ELM model as follows: (i) a single layer feed-forward neural network (SLFN) is constructed; (ii) weights and biases of the network are randomly selected; (iii) by inverting the hidden layer output matrix, the output weights are obtained.

For a dataset containing d-dimensional vectors for $i = 1, 2, 3, \dots, N$ training sample, the SLFN with L hidden nodes is represented mathematically by the following equation.

$$f_L(x) = \sum_{i=1}^{L} \beta_i G(a_i, b_i, x), \qquad (11)$$
$$x \in R^n, \qquad a_i \in R^n$$

Where:

a_i and b_i = learning parameters of hidden nodes.
β_i = the output weight matrix between the hidden neurons and output neurons.
G (a_i, b_i, x) = the output value of the ith hidden node regarding with input x

$G(a_i, b_i, x)$, $g(x): R \to R$ is a non-linear piecewise continuous function that should meet the ELM approximation theorem. Different activation functions that are generally used in neural network-based modelling can be applied. The sigmoid equation was used herein to develop the ELM model as following

$$G(a_i, b_i, x) = \frac{1}{1 + exp(-a_i x + b_i)} \qquad a_i \ \& \ b_i \ \in R \tag{12}$$

According to Huang *et al*. (2006), the approximation error should be reduced to solve the weights connecting the hidden and output layer (β) by the use of least square fitting.

$$min\|H\beta - T\|^2 \qquad \beta \in R^{L\times m} \tag{13}$$

In this equation, the term $\|H\beta - T\|$ is the Frobenius norm, and H is the randomised hidden layer output matrix in the form of

$$H = \begin{bmatrix} G(a_1, b_1, x_1) & \cdots & G(a_L, b_L, x_1) \\ \vdots & \cdots & \vdots \\ G(a_1, b_1, x_N) & \cdots & G(a_L, b_L, x_N) \end{bmatrix}_{N\times L} \tag{14}$$

Moreover, the target matrix in the data training period is represented as

$$T = \begin{bmatrix} t_1^T \\ \vdots \\ t_L^T \end{bmatrix}_{N\times m} = \begin{bmatrix} t_{11} & \cdots & t_{1m} \\ \vdots & & \vdots \\ t_{N1} & \cdots & t_{Nm} \end{bmatrix}_{N\times m} \tag{15}$$

By solving the following equation, an optimal solution can be determined.

$$\beta = H^{\dagger}T \tag{16}$$

where $H^{\dagger}$ is the Moore-Penrose generalized inverse function and β is the output weights of the network as following

$$\beta = \begin{bmatrix} \beta_1^T \\ \vdots \\ \beta_L^T \end{bmatrix}_{L\times m} = \begin{bmatrix} \beta_{11} & \cdots & \beta_{1m} \\ \vdots & & \vdots \\ \beta_{L1} & \cdots & \beta_{Lm} \end{bmatrix}_{L\times m} \tag{17}$$

The output weight β can then be used to estimate the targets of the problem for any given input vector, x.

4.2 Models development

As stated in the introduction section, the secondary purpose of this paper is to challenge the test data and predict the moment and rotation. Thus, using six inputs, one can analyse the impact of inputs so that, by comparing the obtained results from their placement in artificial intelligence models, the quality of their impact and determination will be understood. A summary of this information can be found in Table 4.

The beam length, width, and other beam properties have not been considered as input due to their constant value in each specimen. The database has been set for the upright-beam system variables, upright length, upright width, upright thickness, upright tensile strength, moment and rotation which also directly affected the flexural and torsional capacity of the upright-beam system. Moment and rotation were replaced by each other, in order of placement as input or output.

4.3 Performance evaluation

To evaluate the performance of all the developed models, root mean squared error (RMSE), Pearson correlation coefficient (r), and determination coefficient (R^2) were used. These statistical indicators can be characterized as follows

$$RMSE = \sqrt{\frac{\sum_{i=1}^{n}(P_i - O_i)^2}{n}} \tag{18}$$

$$r = \frac{n(\sum_{i=1}^{n} O_i \cdot P_i) - (\sum_{i=1}^{n} O_i) \cdot (\sum_{i=1}^{n} P_i)}{\sqrt{\left(n\sum_{i=1}^{n} O_i^2 - \left(\sum_{i=1}^{n} O_i\right)^2\right) \cdot \left(n\sum_{i=1}^{n} P_i{}^2 - \left(\sum_{i=1}^{n} P_i\right)^2\right)}} \tag{19}$$

$$R^2 = \frac{\left[\sum_{i=1}^{n}\left(O_i - \overline{O_i}\right) \cdot \left(P_i - \overline{P_i}\right)\right]^2}{\sum_{i=1}^{n}\left(O_i - \overline{O_i}\right) \cdot \sum_{i=1}^{n}\left(P_i - \overline{P_i}\right)} \tag{20}$$

Table 4 Inputs and outputs of database

Inputs and outputs	Variables	Minimum	Maximum	Mean value	Std**
Input 1	upright length (cm)	112.2	113.1	112.688764	0.449169209
Input 2	upright width (cm)	67.6	68.3	67.98157895	0.349229539
Input 3	upright thickness (cm)	2	2.6	2.324626866	0.299546866
Input 4	upright tensile strength (Mpa)	325	493	415.5576208	83.89973487
Input 5	Moment (kN.mm)	0	35.9495661	25.58054402	10.55335622
Input 6	Rotation (degree)	0	6.835730271	2.54019535	2.006430976

*Moment and rotation were employed both as input and output according to their order in the database

**Std = Standard Deviation

Table 5 Estimation evaluation criteria

	Test		Train	
Moment prediction	R^2	0.9948	R^2	0.9957
	r	0.997387023	r	0.997869694
	RMSE	0.792422099	RMSE	0.677403395
	Test		Train	
Rotation prediction	R^2	0.7748	R^2	0.7291
	r	0.880232598	r	0.85389782
	RMSE	0.94066448	RMSE	1.072279217

Where P_i and O_i are the predicted and observed variables and n is the total number of considered data. To compare the performance of the ELM, codes were developed in the MATLAB 2019 environment, and the available MATLAB functions were only used. This is also important to note that all the codes were run in the same computer system, and no external compiler or toolbox was employed in this procedure.

Table 6 Parameter characteristics used for ELM

Classifier	Regression	Hidden neurons	Activation function
1.0	0.0	200	Sine

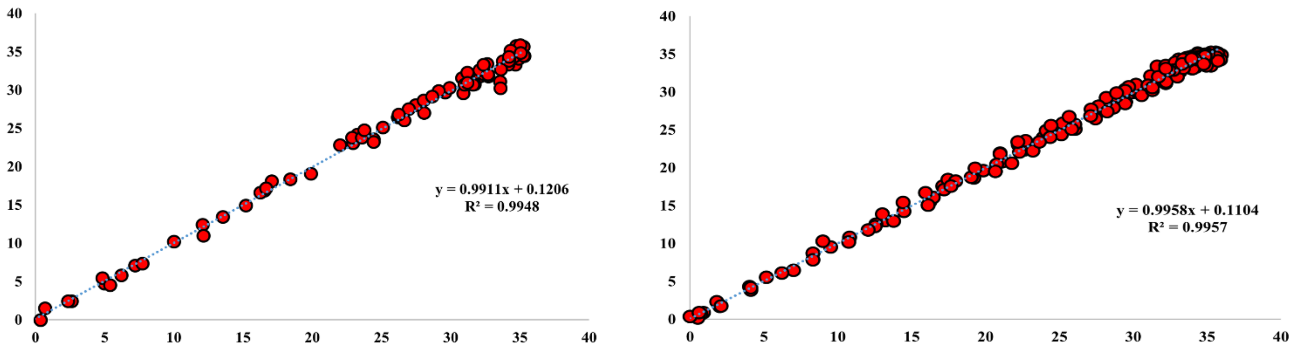
4.4 ELM results

Table 5 and Figs. 10 and 11 indicate the results of the ELM on the test data.

The ELM method was performed according to the settings of Table 6. The obtained results from this method are acceptable for both outputs. However, it was found that by comparing the Moment output evaluation criteria to the Rotation value ones, and by comparing the Test and Train results, the Moment outputs were very close to the real values while different for the Rotation output. It is obvious that experimental results perform excellent predictability, which can verify the test data. Moment data have shown the best predictability and quality; however, the prediction of the rotation was also acceptable.

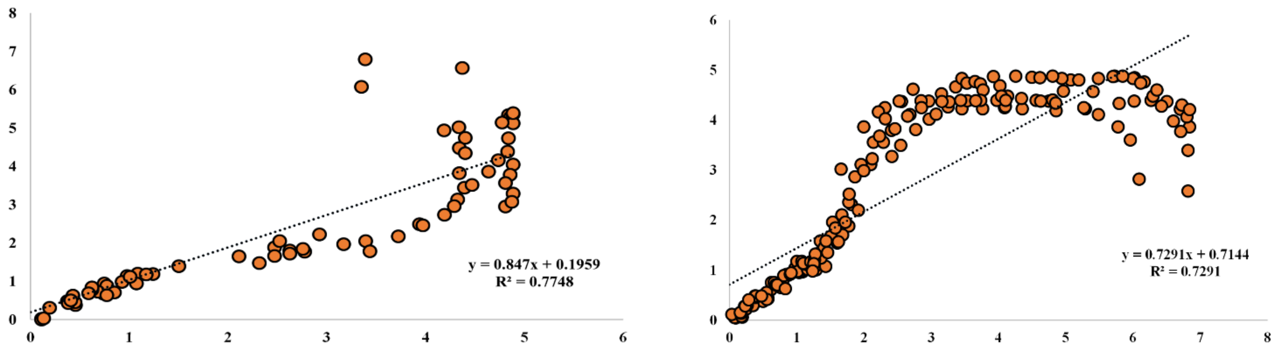
5. Conclusions

Under the monotonic test, the beam with original 5 lug connector connecting with upright X1 (2.0 mm thickness) provides the characteristic value of failure moment 32.57 kNmm, the design value of failure moment of 29.61 kNmm, initial stiffness of 14785.22 kNcm/rad and design value for connection stiffness of 10412.94 kNcm/rad. Meanwhile, for a beam with original 5 lug connector connecting with upright X2 (2.6 mm thickness), the characteristic value of failure moment 32.76 kNmm, design value of failure moment of 29.79 kNmm, initial stiffness of 14970.94



(a) (b)

Fig. 10 Moment prediction vs experimental results regression for: (a) test phase; (b) train phase



(a) (b)

Fig. 11 Rotation prediction vs experimental results regression for: (a) test phase; (b) train phase kNcm/rad and design value for connection stiffness of 11494.86 kNcm/rad were obtained. By comparing both results, the differences in the characteristic value are not so obvious, however, from the observation, both connections have a difference in failure mode which the connection X1 (2.0 mm upright) is upright web tearing and X2 (2.6 mm upright) is the shear-off from the connector lug. To provide better ductility without any add-on, the experiment was confined to the connector steel properties. The use of the material connector with high elongation is recommended which indicates a high ratio of tensile stress to yield stress material. By the use of new material with a high ratio, the more rotation is obtained by material connector before the failure which leads to rising plastic part in the diagram and finally, the ductility of the connection is improved.

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