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STABILITY ANALYSIS OF STEEL STORAGE RACK STRUCTURES

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ABSTRACT

Industrial racks are normally framed structures fabricated from cold-formed sections and relative to their self weight (Dead Load) carry very high Pallet Loads (live load) compared with conventional civil engineering structures. Lack of sufficient design rules and specifications provides an urgent need to better understand their performance under seismic loads as well as static load. Due to their slenderness controlling sway deformation is an important factor in the design of industrial racks and hence special attention must be given to factors such as 'beam to upright connections' and 'base plates connections'. This paper focuses on theoretical approaches to perform stability analyses of storage rack structures and considers the effects of incorporating the stiffness of baseplates and bracing elements in the critical buckling load. Also the effect of cyclic moment rotation deterioration on the global stability of the frame is highlighted. A stability limit has been defined for a maximum pallet load to be stored on a particular rack structure.

KEYWORDS

Rack structures, Stability analysis, Beam to upright connections, Upright-Base plate connection

INTRODUCTION

The process of moving the manufactured products from the producer to the end user needs highly engineered industrial storage rack facilities. The benefit of racking structures is that they allow comparatively efficient use of floor space combined with direct access to every item in the store. Compared with literature available for the analysis and design of conventional steel construction, relatively little has been published on different types of beam end connectors and base plate connections used in the racking systems and especially their performance under earthquake actions. Due to their peculiarities in comparison with conventional steel construction, special analysis and design rules are adopted in the racking industry (BS EN 2009, RMI 2008, AS 2012). In industrial racking, moment frame systems are typically used to resist excessive lateral displacement in the down aisle direction by relying primarily on the beam to upright connections stiffness. Spine bracing members may be also used to increase the down aisle stiffness, when applicable. However, due to access requirement, it is not feasible to put bracing members along the aisle. Usually dual systems are used in the middle frames along the aisle if there is enough space. Plane bracing may be used to

distribute the stiffness across the aisle and along the moment frames. This system is usually used in seismic design of high rise storage rack structures that have a relatively long fundamental period.
“The essential analytical requirement for the consideration of down-aisle stability is a suitable second-order elastic analysis for a large plane frame with semi-rigid joints” (Davies 1992). A simple 2D model of an un-braced down aisle frame is shown in Figure 1. Stark and Tilburgs (1979) proposed a simple model for down aisle stability analysis.

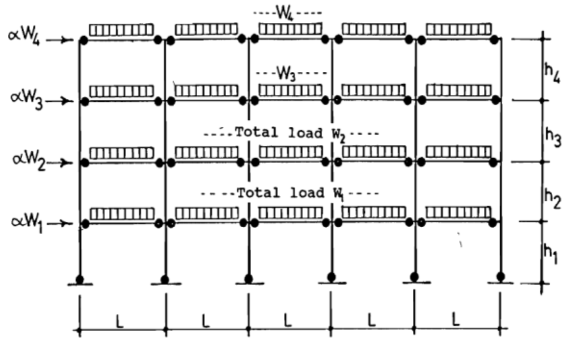

Figure 1. 2D analytical model of a down aisle frame

Their analysis method was based on simplified assumptions which are outlined below:

1. It is based on a single internal upright, regardless of the number of bays.
2. It only allows for column flexibility below the level of the first beam and the remainder of the column is treated as rigid.
3. Base plate behaviour is included by using “eccentricity method” in the model. For this reason the vertical reaction is offset by an amount “e”.
4. Second order effect is considered in the analysis

Later, Davies (1980) described a general exact procedure for determining the elastic critical load of frames with flexible joints. He mathematically modeled the entire un-braced frame by using matrix analysis.

Lewis (1991) produced a simple design approach to work out the critical pallet load of an unbraced down aisle frame using the energy method. The critical pallet load proposed by Lewis (1991) is given in Eq. 1.

$$p_{cr} = \frac{4\, f(\theta)}{h\,(C+1)\,\sin\varphi} \qquad (1)$$

Where, f(θ) is the moment rotation function of the beam to upright connections, “h” is the story height assumed to be constant and “C” is the number of story levels.

Davies (1992) improved the down aisle frame model based on the work of Horne (1975). He described simplified model for determination of sway behaviour of slender pallet racks with semi-rigid connections. An estimation of the elastic critical load and bending moment distribution were also described in his paper. Feng et al (1993, 1994) developed a single column model incorporating the semi rigidity of the connections and flexural deformation of the upright which can be used for determination of frame’s buckling load.

Godley et al (2000) developed a theoretical method for the analysis and design of unbraced down aisle pallet rack structures. The proposed method accounts for multi bay frames with variable number of stories, semi rigid base plates and beam to upright connection. The proposed method was later improved to consider the effect of splices in spliced down aisle frames (Beale et al 2004).
The American specification, RMI (2008) states in clause 6.3.1:

“For the portion of the column between the bottom beam and the floor as well as between the beam levels, the effective length factor K shall be taken as 1.7 or as otherwise determined by an analysis properly accounting for the member stiffness, the semi-rigid nature of the beam to column connections and the partial fixity of the base, allowing for average load reduction, as applicable. The effective length factor for pallet racks, stacker racks, and movable-shelf racks is “K = 1” provided that all such racks have diagonal bracing in the vertical plane and that such racks have either a rigid and fixed top shelf, or diagonal bracing in the horizontal plane of the top fixed shelf.” (RMI 2007) This approach can be potentially unsafe however exact analyses show that values of “K” well in excess of 1.7 are not uncommon. “It is salutary to note that, as the stiffness of the semi-rigid joints is reduced, the effective length of the uprights in the down-aisle direction of a rack structure tends to infinity.” (Davis 1992). In addition, Teh et al (2004) studied the three dimensional frame buckling behaviour of high rise double deep selective racks by developing a 2D model which can give a reliable estimation of elastic flexural – torsional buckling stresses of the upright columns. They acknowledged that the effective length factors of 1.0 for a braced frame and 1.7 for an un-braced frame underestimate the flexural-torsional buckling stress. (Teh et al. 2004)

STABILITY ANALYSIS

In this paper the stability analysis method of Lewis (1991) which accounts for the nonlinearity of the beam to upright connections will be improved using the same assumptions by incorporating the stiffness effects of spine bracing and base plate connections. Also in order to use the stability equations for the systems under seismic actions, an approximate model of moment rotation curve of the beam to upright connections will be defined to best model the behaviour of the connections under cyclic loads patterns. In this approach it is assumed that the same loads are applied at each level and their lines of action remain vertical during the displacement of the frame. Simplified models of braced and un-braced frames are shown in Figure 2. The bending distortion of the upright is small when compared with the lateral displacement of the upright. The upright rotation as well as beam to upright connections is denoted by “θ”. Nonlinear behavior of base plate and beam to upright connections are considered. An initial looseness (out of plumbness) is denoted by “α” which is not shown in Figure 2.

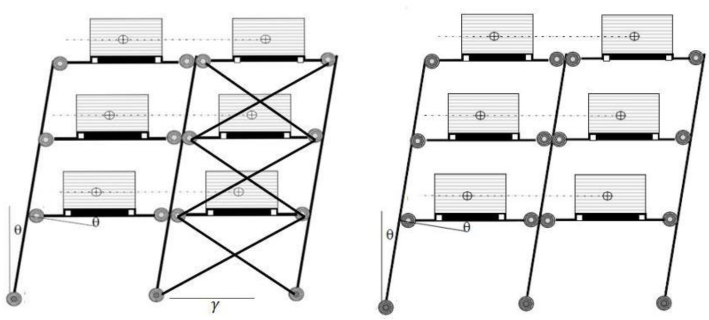

Figure 2. Braced and un-braced frames

The total potential energy of the system can be written as:

$$W = U + P_E \tag{2}$$

Where: U = Internal work
P_E = External work

A significant component of internal work includes the work of semi-rigid connections including base plates and beam to upright connections. In comparison, work due to bending and axial deformation of beams and uprights is assumed to be relatively small and therefore will not be considered in the expression derived below. The work done by semi rigid connections can be expressed as:

$$U = \int N_c f_c(\theta) d\theta + \int N_b f_b(\theta) d\theta \tag{3}$$

$N_b = Number\ of\ base\ plate\ connections$
$N_c = Number\ of\ beam\ to\ upright\ connections$
$f_c(\theta) = Moment\ rotation\ function\ of\ Beam\ to\ upright\ connections$
$f_b(\theta) = Moment\ rotation\ function\ of\ Base\ plate\ connections$

The potential energy lost by the vertical external loads can be written as:

$$PE = -N \sum_{i=1}^{c} p\ i\ h(1 - \cos\varphi) \quad (4)$$

$C = Number\ of\ story\ levels$
$N = Number\ of\ beams\ at\ every\ levels\ (N_c = 2\ C\ N)$
$\varphi = \theta + \alpha$
$p = Pallet\ load\ distributed\ at\ every\ beam$
$\alpha : Imperfection\ (out\ of\ plumb)$

The total potential energy of the system can be written as:

$$W = N_c \int f_c(\theta)d\theta + N_b \int f_b(\theta)d\theta - \frac{N_c ph(C+1)}{4}(1 - \cos\varphi) \quad (5)$$

$$\frac{\partial W}{\partial\theta} = N_c \frac{\partial}{\partial\theta}\int f_c(\theta)d\theta + N_b \frac{\partial}{\partial\theta}\int f_b(\theta)d\theta - \frac{N_c ph(C+1)}{4}(\sin\varphi) = 0 \quad (6)$$

Therefore:

$$p_{cr} = \frac{4\,[N_c f_c(\theta) + N_b f_b(\theta)]}{N_c h\,(C+1)\ \sin\varphi} \quad (7)$$

By applying bracing members the internal work done by the system will change as follows:

$$U = \int N_c f_c(\theta)d\theta + \int N_b f_b(\theta)d\theta + \frac{N_{brace} h^2 \sin^2\varphi \cos^2\gamma\ E_{brace} A_{brace}}{2 l_{brace}} \quad (8)$$

Where,
N_{brace} = Number of bracing members in tension
γ = the angle between brace member and the horizontal direction (Figure 2)
A_{brace} = cross section area of bracing members[1]

It is assumed that only those bracing members that are in tension are participating and pallet loads at every beam are of equal values. Using the same methodology and taking the first derivative of the energy function, the load bearing capacity of every beam level will be increased as expressed by Eq. 9.

$$p_{cr} = \frac{4\left([N_c f_c(\theta) + N_b f_b(\theta)] + \frac{N_{brace} h^2 \sin\varphi \cos\varphi \cos^2\gamma\ E_{brace} A_{brace}}{l_{brace}}\right)}{N_c h\,(C+1)\ \sin\varphi} \quad (9)$$

In order to determine the type of stability of the system, identifying the sign of the second derivatives of the total potential energy function "W" is necessary, which is evaluated as shown below:

$$\frac{d^2 W}{d^2\theta} = N_c \frac{\partial^2}{\partial^2\theta}\int f_c(\theta)d\theta + N_b \frac{\partial^2}{\partial^2\theta}\int f_b(\theta)d\theta +$$

$$\frac{\partial^2\left(\frac{N_{brace} h^2 \sin^2\varphi \cos^2\gamma\ E_{brace} A_{brace}}{2 l_{brace}}\right)}{\partial^2\theta} - \frac{N_c ph\ (C+1)(\cos\varphi)}{4} \quad (10)$$

Substituting (9) into (10) leads to:

$$\frac{d^2 W}{d^2\theta} = N_c f'_c(\theta) + N_b f'_b(\theta) - \frac{N_c f_c(\theta) + N_b f_b(\theta)}{\sin\varphi}(\cos\varphi) - \frac{N_{brace} h^2 \sin^2\varphi \cos^2\gamma\ E_{brace} A_{brace}}{l_{brace}} \quad (11)$$

[1] The cross sectional area of the bracing members can be reduced to account for the softness of the end connections

As shown by Lewis (1991) for positive values of α, the sign of Eq. 11 is negative and hence the whole frame will be unstable under any pallet loads greater than p_{cr}. Figure 3 shows a typical representation of the equilibrium states of a system with bi-linear connection characteristic. The red line indicates decreasing ultimate loads of imperfect systems by increasing $\boldsymbol{\alpha}$ values.

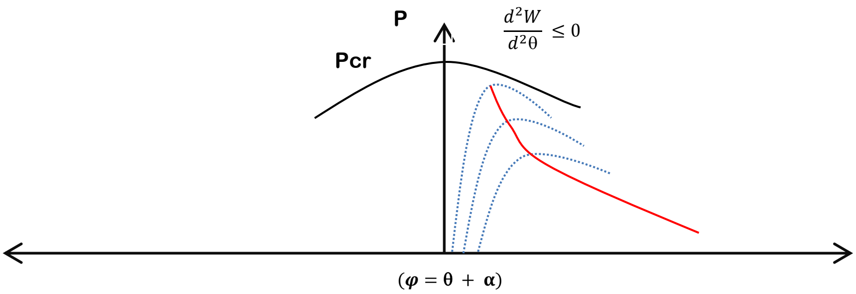

Figure 3. Equilibrium states of a system with Bi-Linear connection characteristic

Stability of the frames under seismic actions

The approach presented below introduces a stability limit for the seismic design of rack structures where the critical pallet load at the maximum drift of the structure "θ", which is calculated from conventional seismic analyses methods, will be checked against the stored pallet loads on the rack.

In Eqs. 7 and 9, the stability of the entire moment frame in down aisle direction for both braced and un-braced frames essentially relies on the moment-rotation behaviour of beam to upright connections as well as base plate connections. However, to define a stability limit for the maximum drift of the racking system in seismic areas, the monotonic moment-rotation function of the connections may not be reliable because the connections will progressively deteriorate while subjected to cyclic reversal load patterns. This phenomenon is explained by Aguirre (2004) and was also investigated by Reyes (2013). However, the stability analysis proposed by Lewis (1991) is only sensitive to strength deterioration while stiffness deterioration has no effect on the final result. By investigating a typical storage rack, Reyes (2013) observed that the dynamic behavior of the system and thereby the maximum seismic drift is significantly dependent on both strength and stiffness deteriorations. He proposed a method to define an equivalent cyclic moment-rotation back bone that considers the strength deterioration of the connections which can be used as a moment-rotation function "$f_c(\theta)$" in stability analysis of his down aisle frame. As shown in Figure 4, the moment function "$f_c(\theta)$" has higher values from monotonic tests compared with moment values obtained from cyclic hysteresis tests for the same rotation "θ". Hence in this case using the monotonic moment rotation curve gives an un-conservative stability limit.

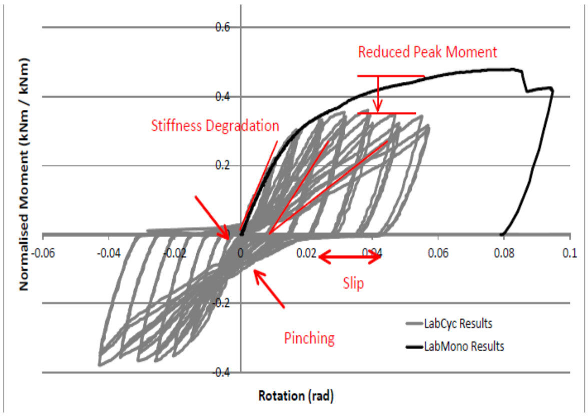

Figure 4. Typical Beam-Upright Connection Non-Linear Features (Reyes 2013)

It is therefore proposed that a more realistic stability limit can be obtained from Eqs. 7 & 9 by adopting a reduced moment according to an equivalent cyclic moment-rotation back bone. Further investigation is required to confirm the validity of this approach to storage racks with other connection types.

CONCLUSION

In this paper the stability analysis proposed by Lewis (1991) was improved in order to consider dual storage rack systems that incorporate spine bracing and base plates. It was shown that the cyclic moment rotation back bone should be used as equivalent moment rotation functions of $(f_c(\theta))$ and $(f_b(\theta))$ for both beam to upright connections and base plates when using the monotonic moment rotation curves is un-conservative.

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