CAPACITY BASED DESIGN OF COLD FORMED STORAGE RACK STRUCTURES UNDER SEISMIC LOAD FOR RIGID AND SEMI RIGID CONNECTIONS

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ABSTRACT :

Rack systems are very similar to the framed steelworks traditionally used for civil and commercial buildings, but great differences in member geometry and in connection systems. In the pallet rack system bracing systems are generally placed only in the cross aisle direction. Therefore design of pallet racks is quite complex. The capacity design based on deterministic allocation of strength and ductility in the structural elements for successful response and collapse prevention during a catastrophic earthquake by rationally choosing the successive regions of energy dissipation so that pre-decided energy dissipation mechanism would hold throughout the seismic action. The most accurate method of seismic demand prediction and performance evaluation of structures is nonlinear time history analysis. However, this technique requires the selection and employment of an appropriate set of ground motions and having a computational tool able to handle the analysis of the data and to produce ready-to–use results within the time constrains of design offices. A simple analysis method that has been gaining ground, as an alternative to time history analysis, is the nonlinear static pushover analysis. The purpose of the push over analysis is to assess the structural performance by estimating the strength and deformation capacities using static, nonlinear analysis and comparing these capacities with the demands at the corresponding performance levels. Model of conventional pallet racking systems were done using the finite element program Sap2000NL and were analyzed using non-linear static pushover analysis. Parameters selected for analysis and design are cross section of uprights, thickness of uprights, and stiffness of the connections (beam to upright and base). Nonlinear push over analysis found to be a useful analysis tool for the conventional pallet racking systems giving good estimates of the overall displacement demands, base shears and plastic hinge formation.

KEY WORD : Non linear analysis, pallet racks, cold formed steel, semi rigid joint, Sap 2000NL

1. INTRODUCTION

One of the most significant uses of cold-formed members is for steel storage racking structures, such as pallet, drive in, and drive through racking systems. In typical pallet rack structure, generally, beams (stringers) have boxed cross sections, while columns (uprights) are open thin walled perforated to accept the tabs of beam end connectors, which join beams and columns together without bolts or welds. Therefore design of pallet racks is quite complex. The behavior of the perforated columns, that are generally thin walled members, is affected by different buckling modes (local, distortional and global) as well as by their mutual interactions. The response of beam to column is typically nonlinear. Moreover, bracing systems are generally placed only in the cross aisle direction. The need for organizing pallet racks in such a way that the product is efficiently stored and sufficiently accessible hampers the presence of bracings in the down aisle direction. Lateral stability is, hence, provided by the sole degree of continuity associated with beam to column joints as well as by base plate connections. The analysis and design of thin walled cold formed steel pallet racking frames structure with perforated open upright section and semi rigid joints presents several challenges to the structural engineers. Presently, for the design of these frames no specific code of practice exists. Although in the United States and some other countries the specification published by the Rack Manufacturer's Institute (RMI) serves as a guideline. Therefore analysis and design of pallet racks is quite complex. The most accurate method of seismic demand prediction and performance evaluation of structures is nonlinear time history analysis. However, this technique requires the selection and employment of an appropriate set of ground motions and having a computational tool able to handle the analysis of the data and to produce ready to use results within the time constrains of design offices; clearly, a simpler analysis tool is desirable. One method that has been gaining ground, as an alternative to time history analysis, is the Nonlinear Static Pushover Analysis.

2. PUSH OVER ANALYSIS

Push-over analysis is a technique by which a computer model of the building is subjected to a lateral load of a certain distribution (i.e., inverted triangular or uniform).The purpose of the pushover analysis is to assess the structural performance by estimating the strength and deformation capacities using static, nonlinear analysis and comparing these capacities with the demands at the corresponding performance levels. The basic procedure of this method is to perform a sequence of elastic static analysis under monotonically increasing lateral loads in each of its principle directions to stimulate the loading history of the structure during collapse. In effect, the structure is pushed sideways well into the inelastic range till total failure or collapse occurs; hence, this method is called Pushover Analysis. The potential of the pushover analysis has been recognized in the last decade and it has found its way into seismic guidelines [ATC,1997; SEAOC, 1995; CEN 1995].It is expected to gain more popularity in the future and it is already included in some codes .The pushover is expected to provide information on many response characteristics that can't be obtained from an elastic static or dynamic analysis. The following are the examples of such response characteristics:

- Realistic force demands on potentially brittle elements, such as axial demands on columns, moment demands on beam to column connections or shear demands on short, shear dominated elements.
- Identification of the critical regions, where the inelastic deformations are expected to be high.
- Estimates of the deformation demands on elements that have to deform in elastically, in order to dissipate energy.
- Consequences of strength deterioration of particular elements on the overall structural stability.
- Estimates of inter-storey drifts, accounting for strength and stiffness discontinuities. In this way, damage on nonstructural elements can be controlled.
- Sequence of members yielding and failure and the progress of the overall capacity curve of the structure
- Identification of the strength irregularities in plan or elevation that causes changes in the dynamic characteristics in the inelastic range.
- Verification of the adequacy of the load path, considering all the elements of the system, both structural and nonstructural.

2.1 Assumptions Made in Pushover Analysis

The fundamental assumptions of pushover analysis limit the scope of the pushover analysis. The assumption are given as below,

- The capacity curve generally constructed in the pushover analysis based on the assumption that the fundamental mode of vibration is the predominant response of the structure. This is generally valid for the structure with fundamental period of vibration up to about one second.
- Higher mode effects are not considered in the nonlinear static pushover analysis. Hence, if, the participation of the higher modes is found to be significant then the effect of the same can be analyzed separately.

2.5 Past Study on push over analysis

Lawsonet al [1994], Krawinkler and Seneviratna [1998] has highlighted on the history of pushover analysis method. Initially the majority of work concentrated on discussing the range of applicability of the method and its advantages and disadvantages, compared to elastic or nonlinear dynamic procedures. Recently, Paret et al. [1996], Sasaki et al. [1998], Moghadam and Tso [2002], Chopra and Goel [2001, 2002] have taken efforts to extend pushover analysis to take into account higher mode. Bracci et al.[1997], Gupta and Kunnath [2000], Requena and Ayala [2000], Elnashai [2000], Antoniouet et al [2002], Aydinoglu [2003] also have done some attempts to derive fully adaptive procedures with update force distributions that take into account the strength and stiffness of the structure at each step. Mwafy and Elnashai [2000] and Lawsonet al.[1994] observed that pushover procedures are particularly poor in predicting the response of frame wall structures, probably due to significant period shift and change of inertia force distribution upon yielding of the wall base. Mwafy and Elnashai [2000], Gupta and Kunnath [2000] have found that, where as in the elastic range force distributions of a triangular or trapezoidal shape provide a better fit to dynamic analysis results, at large deformations the dynamic envelopes are closer to the uniformly distributed force solutions. Various codes and guidelines [CEN,1995; PCM,2003; ATC,1997] suggest that use of a “uniform” pattern, where the lateral forces are proportional to the local masses at each floor level, and a “modal” pattern, which is determined by a modal combination using a sufficient number of modes and an appropriate spectral shape. Alternatively, Fajfar and Fichinger [1988] had recommended the “triangular” pattern in which the accelerations are proportional to the storey heights, rather than the “modal” pattern, to the deflected shape of the structure, whereas Gupta and Kunnath [2000] and also Requena and Ayala [2000] suggested the derivation of the forces through modal combinations using the square root of the sum of squares (SRSS) method, taking into account a predefined number of modes of interest. Priestley [1993] considers the earthquake loading as a set of imposed energy input, ground displacements and deformations of the structural members, rather than a set of lateral forces, seems a much more rational approach. Antoniouand Pinho [2004] has propose that to apply displacements, rather than forces, requires adaptiveness meaning to update the displacement patterns, according to the structural properties of the analyzed model, such as the stiffness of the mass distribution. On one side, such procedure would be theoretically more rigorous and match the new trends for displacement based design and assessment, and, alternatively, it would expose the structural weakness that are concealed with fixed displacement patterns and yield accurate results both at the local and the global level.

This paper deals with the study of behavior of a cold-formed steel storage rack structure, with rigid and semi rigid connections, under gravity and seismic load and improvement in the base shear at the time of collapse. Pushover analysis is carried out on frames as it is capable to estimate several important characteristic of nonlinear structural behavior such as real strength, plastic mechanism and progressive deformation of the structure with increase in lateral load. In this study pattern of lateral load, which is obtained from seismic force distribution pattern as per IS 1893-2000 specification, is applied. Excel program is developed to calculate the lateral load acting on each pallet level. Modal analysis to find out the fundamental time period of all the frames is carried out before doing the push over analysis to overcome the limitation of pushover analysis. From modal analysis it is found that all the frames fundamental time period is less than 1 sec.

3. COLUMN SECTION USED IN THE STUDY

In this paper torsionally strengthen sections were used. Original open sections were strengthened by providing channel and hat stiffeners to avoid the local buckling of uprights before formation of plastic hinges. These sections are medium weight (MW) column section having three thicknesses 1.6 mm, 1.8 mm and 2.0 mm each with hat and channel stiffener and HW (Heavy Weight) column section having three thicknesses 2.0 mm, 2.25 mm and 2.5 mm each with hat and channel stiffener. Their cross sectional geometry is given in figure1to figure 3. Purpose of choosing three different thicknesses is to know the change in behavior when the sections are made locally stable by having higher thickness.

3.1 Calculation of Sectional Properties of the Columns Used in the Study

For the above sections, sectional properties are calculated based on weighted average section. A weighted average section is a section that uses an average thickness in the web portion to account for the absence of the material due to the holes along the length of the section and additional thickness for the additional material of channel and hat stiffener. Excel program is developed to calculate the sectional properties of sections used in this study.

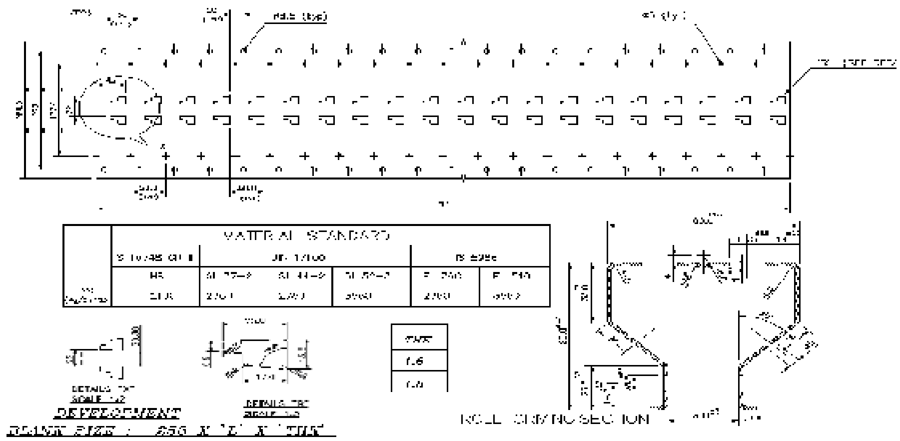

Figure 1 Medium Weight Sections 1.6, 1.8 and 2.0 mm

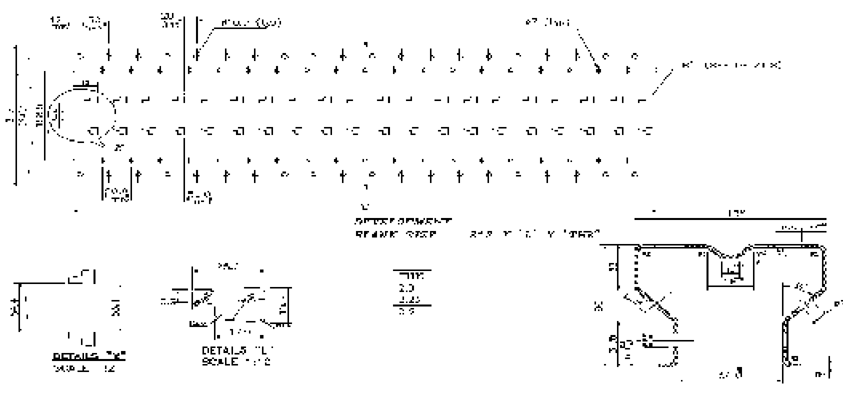

Figure 2 Heavy weight sections 2.0, 2.25 and 2.5 mm.

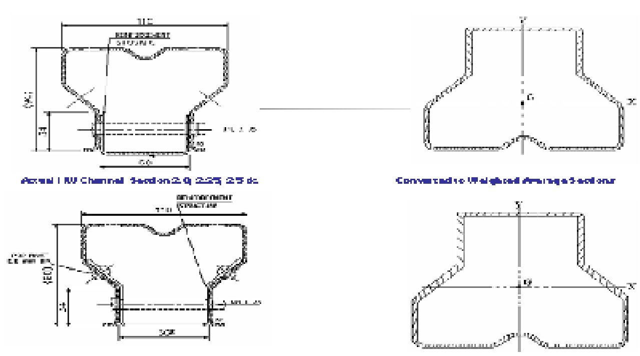

Figure 3 Tortionally strengthened MW and HW section with channel and hat stiffeners

4. STRUCTURAL DETAILS OF RACK STRUCTURES USED FOR ANALYSIS

Structural details of the rack structures are as fallows.

a) Upright sections i) Heavy Weight Hat Section of thickness 2.5 mm, 2.25 mm and 2.0 mm.
   ii) Heavy Weight Channel Section of thickness 2.5 mm, 2.25 mm and 2.0 mm.
   iii) Medium Weight Hat Section of thickness 2.0mm, 1.8mm and 1.6mm.
   iv) Medium Weight Channel Section of thickness 2.0mm, 1.8mm and 1.6mm.
b) Stringer beam section- rectangular hollow section 50 mm wide, 100 mm deep and 3mm thick.
c) Side bracing section, Floor bracing section and back bracing section- channel section 100mm x 40 x 3mm.
d) Width of bay= 2.4 m.
e) Depth of rack shelve =1m.
f) Total load on beam = 3.5 KN/m.
g) Height of the frame =7.6 m and 9.4 m.
h) Distance between two rows =150 mm.
i) Center to center distance between beam 0.9m

5. PARARAMETER CONSIDERED FOR STUDY

Parameters that influence the value of base shear and displacement at collapse of complete rack structure in the down aisle direction are summaries in following categories.

- The first parameter is the number of bays and stories, to account for this; five bays with eight and ten stories are studied.
- Second parameter is type of upright section, to account for this; here 12 types of upright sections as shown in figure 2 are selected.
- Third parameter is of beam column connections, to account for this; here 5 types of connection stiffness (rigid, 1000,500,100 and 75 KN-m) are considered.
- Fourth parameter is of upright frame configuration, i.e type of bracing system in cross aisle direction, to account for this; two type of upright frame configuration is considered (i.e horizontal with inclined bracing and only inclined bracing).
- Fifth parameter is of back side upright frame configuration, i.e type of bracing system, here 3 types of configurations are considered (i.e frame without back bracing, frame with single back bracing and frame with double back bracing).
- Sixth parameter is of floor configuration; here 3 floor configurations are considered (i.e frame without floor bracing, frame with single floor bracing and frame with double floor bracing).
- Seventh parameter is of material yield stress, here fy =250 N/mm^2, 340 N/mm^2, 415 N/mm^2 & fy=500 N/mm^2 are considered.

6. RESULTS

In the above article, the factors that affect the stability of rack structure under gravity and seismic load were highlighted and to account for the same, different parameters, their combinations and structural details were decided. Few results obtained from the analysis, with different combinations, are presented in this paper. Base shear at the time of collapse is improved step by step either by changing the bracing combination or changing the cross section of upright at points where plastic hinges are forming initially. Base shear at the time of collapse is improved till it is greater than the applied lateral load as per IS 1893-2000 . The results are presented in tabular form in table 1and table 2.

Table 1 Results of Base Shear Improvement For Semi Rigid Frames

Frame Configuration	Yield Stress (N/mm^2)	Applied Lateral Load (KN)	Base Shear (KN)	Displacement (mm)	Fundamental Time period (Sec)
SB1A+FB0BB0 HWCS 2.5 Number of Storey = 8 Height of frame = 7.6 m	250	943.490	252.94	127.5	0.18173
	500	943.490	642.37	320.7	0.18173
SB1B+FB0BB0HWCS 2.5 Number of Storey = 8 Height of frame = 7.6 m	250	943.490	257.94	137.5	0.18175
	500	943.490	647.97	298.7	0.18175
SB1B +FB0BB0HWCS 2.5 Number of Storey = 8 Height of frame = 7.6 m Bottom 4 storey upright section = HWHS2.5	500	943.490	691.86	270.7	0.1788
SB1A+FB1BB1HWCS 2.5 Number of Storey = 8 Height of frame = 7.6 m	250	959.756	598.30	17.3	0.09036
	340	959.756	976.74	26.9	0.09036
	500	959.756	1640.9	43.8	0.09036
SB1A+FB2BB2HWCS 2.5 Number of Storey = 8 Height of frame = 7.6 m	250	976.022	619.50	10.4	0.10274
	340	976.022	1018.9	15.8	0.10274
SB1B+FB1BB1HWCS 2.5 Number of Storey = 8 Height of frame = 7.6 m	250	959.756	602.81	17.8	0.08476
	340	959.756	989.51	27.8	0.08476
SB1B+FB2BB2HWCS 2.5 Number of Storey = 8 Height of frame = 7.6 m	250	976.022	618.85	10.4	0.09656
	340	976.022	1019.4	15.7	0.09656
SB1A+FB0BB0HWHS 2.5 Number of Storey = 8 Height of frame = 7.6 m	250	944.364	300.67	162.2	0.1798
	500	944.364	691.84	265.4	0.1798
SB1B+FB0BB0HWHS 2.5 Number of Storey = 8 Height of frame = 7.6 m	250	944.364	305.23	167.6	0.17985
	500	944.364	692.21	270.3	0.17985
SB1A+FB1BB1HWHS2.5 Number of Storey = 8 Height of frame = 7.6 m	250	960.630	730.29	19.7	0.08719
	340	960.630	1160.3	30.3	0.08719
SB1A+FB2BB2HWHS2.5 Number of Storey = 8 Height of frame = 7.6 m	250	976.898	764.88	10.9	0.10052
	340	976.898	1214.8	16.4	0.10052
SB1B+FB1BB1HWHS2.5 Number of Storey = 8 Height of frame = 7.6 m	250	960.630	734.9	19.8	0.08253
	340	960.630	1167.8	30.4	0.08253
SB1B+FB2BB2HWHS2.5 Number of Storey = 8 Height of frame = 7.6 m	250	976.898	765.16	10.9	0.09385
	340	976.898	1215.1	16.3	0.09385

Table 2 Results of Base Shear Improvement For Semi Rigid Frames

<table>
<tr><th colspan="2">Frame Configuration</th><th>Yield Stress (N/mm^2)</th><th>Applied Lateral Load (KN)</th><th>Base Shear (KN)</th><th>Displacement (mm)</th><th>Fundamental Time period (Sec)</th></tr>
<tr><td colspan="2" rowspan="3">SB1A+FB0BB1HWCS 2.00 S75
Number of Storey = 8
Height of frame = 7.6 m
Bottom 4 storey upright section = HWHS2.5
Stiffness of Connection for fy 250 N/mm^2 is 75 KN-m and for fy 415 and 500 N/mm^2 =1000 KN-m</td><td>250</td><td>945.64</td><td>278.39</td><td>18.9</td><td>0.08956</td></tr>
<tr><td>415</td><td>945.64</td><td>851.39</td><td>49.6</td><td>0.08412</td></tr>
<tr><td>500</td><td>945.64</td><td>1078.36</td><td>62.5</td><td>0.08412</td></tr>
<tr><td colspan="2" rowspan="2">SB1A+FB1BB1HWCS2.00 S75
Number of Storey = 8
Height of frame = 7.6 m</td><td>250</td><td>945.64</td><td>421.73</td><td>14.1</td><td>0.09394</td></tr>
<tr><td>340</td><td>945.64</td><td>736.35</td><td>22.7</td><td>0.09394</td></tr>
<tr><td colspan="2" rowspan="2">SB1A+FB1BB1HWCS2.00 S1000
Number of Storey = 8
Height of frame 7.6 m
Bottom 4 storey upright section =HWCS2.5</td><td>250</td><td>945.64</td><td>607.39</td><td>17.7</td><td>0.08925</td></tr>
<tr><td>340</td><td>945.64</td><td>993.68</td><td>27.6</td><td>0.08925</td></tr>
<tr><td colspan="2">SB1A+FB0BB1HWCS200 S75
Number of Storey = 10
Height of frame = 9.6 m</td><td>250</td><td>1156</td><td>205.82</td><td>15.3</td><td>0.1235</td></tr>
<tr><td rowspan="4">SB1A+FB0BB1HWCS2.00
Number of Storey = 10
Height of frame = 7.6 m</td><td>Bottom 4 storey upright section HWCS2.5
S = 1000 KN-m</td><td>500</td><td>1156</td><td>1002.25</td><td>59.7</td><td>0.11539</td></tr>
<tr><td>Bottom 4 storey upright section HWHS2.0
S =1000 KN-m</td><td>415</td><td>1156</td><td>731.84</td><td>43.7</td><td>0.1174</td></tr>
<tr><td>Bottom 4 storey upright section HWHS2.5 S1000 KN-m</td><td>415</td><td>1156</td><td>892.49</td><td>51.1</td><td>0.1117</td></tr>
<tr><td>Bottom 4 storey upright section HWHS250
S =1000 KN-m</td><td>500</td><td>1156</td><td>1139.25</td><td>64.9</td><td>0.11175</td></tr>
</table>

SB1A+FB0BB1HWCS2.00
SB - Side Bracing (Bracing in cross aisle direction) 1 A- Horizontal with inclined bracing
1 B- Only inclined bracing.
FB – Floor Bracing , BB – Back Bracing a) 0 – without floor bracing b) 1- Single Bracing c) 2 Double bracing.
HWCS – Heavy weight section with channel stiffener. 2.0 Thickness of the section.
S- Stiffness of connection.

7. CONCLUSIONS

The objective of this study was to determine the base shear at the time of collapse and maximum displacement, to study the formation of plastic hinges, to study the collapse mechanism and to improve the base shear at the time of collapse of cold formed rack structure. Storage rack structures made from cold formed section were analyzed using nonlinear static pushover analysis method under the varying parameters like cross section of members, thickness of members, stiffness of beam column connections, yield stress and type of frame structure. Modal analysis, to find out the fundamental time period of all the frames is carried out before doing the push over analysis to overcome the limitation of pushover analysis. Non linear pushover analyses were found to be a useful analysis tool for the storage rack structure giving good estimates of the base shear, displacement and formation of plastic hinge at every specified load increment. Some important point is highlighted below

- To minimize the twisting effect it is advantages to provide two rows of rack because improvement in base shear at the time of collapse is 2.5 times that of single row.
- It is advantageous to provide different type of upright section at different level. Up to first 3-4 story it is advantageous to provide higher thickness section because this is the weakest portion of rack structure and formation of plastic hinges start at weakest portion ,specially for brace type rack structure.

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