Adaptability of Indian Standard Sections as Deep Wide Flange Steel Columns in Special Moment Frames

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Abstract— Steel hot rolled I-sections have been in use in different constructions since long in India. Deep wide flange steel columns which having column depth, $d > 16$ inches are typically used in steel moment-resisting frames (MRFs) mostly present in North America. Some Indian sections are selected according to their section details and make subjected to cyclic lateral loading combined with different compressive axial load ratios which actually representing the loading conditions of interior steel columns in SMFs. Selection of Indian sections can be made by considering their section properties. Sections having lower web and flange slenderness ratios experience rapid cyclic deterioration in flexural strength under high axial load ratios. Hysteresis curve is used to quantify the cyclic hardening and the cyclic deterioration in flexural strength of a steel beam-column once local buckling occurs. Column plastic flexural strength, Mp should be noted. As by this result and also by the lower section values the Indian hot-rolled I-sections (tapered and parallel flanges) are found inadequate for use in tall structures in high seismic regions. And also parallel flanged column sections are much better than tapered sections. We need to develop more advanced sections for the high range response of columns in special moment frames.

Key words: Deep Wide Flange Steel Column, Hysteretic Curve, ANSYS, Cyclic Deterioration, SMF-Special Moment Frame

I. INTRODUCTION

Moment-resisting frames are used as lateral load resisting systems in many steel building structures. In recent years, it has been recognized that there is a strong economic incentive for the design engineer to use deep columns to satisfy increasingly more stringent drift limitations. Structural engineers have used deeper columns for some steel building projects when they had resources to carry out the physical tests of project-based connections. Steel frame buildings with perimeter special moment resisting frames (SMFs) designed in highly seismic urban regions in North America, should comply with regional seismic design-code requirements (ASCE 2010; NBCC 2010). Normally, deep beams are used in SMFs in order to control the lateral drift demands. Therefore, designers tend to use deep wide-flange steel sections as columns to satisfy the strong-column-weak-beam criterion. These sections are able to respect the economy due to their low weight-to-inertia ratio. The term “deep” refers to wide-flange sections with depth larger than 400 mm (i.e., 16 inches). Special moment frames (SMFs) are developed to withstand significant inelastic deformation during a design earthquake. Hence it needs of special proportioning and detailing requirements in order to resist strong earthquake shaking. The Northridge earthquake that happened helps to expand the knowledge regarding the seismic response of steel moment frames, the design of SMFs and their connections. Beam and column sections shall be plastic or compact in SMFs.At potential plastic hinge locations, they shall be necessarily plastic. The objective of this paper can be summarized as follows:

- To analyze the Indian sections subjected to cyclic lateral loading combined with different compressive axial load ratios.
- To find the adaptability of Indian sections as deep wide flange steel columns for SMFs.

II. PREVIOUS STUDIES

For the past fifty years, several experimental studies have investigated the monotonic and/or cyclic behavior of wide-flange/I-shaped steel columns (Popov et al. 1975; MacRae et al. 1990; Nakashima et al. 1990; Newell and Uang 2006; Chen et al. 2014; Suzuki and Lignos 2015). These studies mainly investigated columns with shallow cross-sections (i.e., depth less than 400mm). Nonetheless, deep wide-flange cross-sections are commonly used in columns of steel moment resisting frames (MRFs), designed in highly seismic regions in North America. Those deep cross-sections provide sufficient strength and stiffness to satisfy capacity design requirements according to the current seismic provisions (AISC 2010a). It is for steel wide-flange beam columns and are based on experimental studies that were conducted on either small-scale (i.e., depth less than 400 mm) or full-scale but stocky wide-flange sections (i.e., $h/tw < 30$ and $b\ f\ /2t\ f < 8$). More specifically, deep and compact cross-sections with relatively large local slenderness ratios are employed in design in order to save steel weight. Consequently, their cyclic behaviour under lateral drifts, combined with compressive axial load becomes questionable. Notably, NIST (2010) established a research plan that highlighted the need for further research related to the hysteretic behaviour of deep wide flange beam-columns as part of steel MRFs under earthquake loading.

Popov et al. (1975) tested W8 sections under cyclic lateral displacement and constant axial load ranging from 30 to 80% Py, where Py is the axial yield strength of the corresponding cross section (i.e., $Py = Agross\ Fy$).They conducted cyclic tests on interior cruciform beam-column sub assemblages with axially loaded wide flange columns. Specimens were designed using a weak-column-strong-beam approach, and axial load ratios for different specimens were varied between approximately 30% and 80% of column yield. In columns subjected to larger axial loads, significant yield deformation and axial-shortening was noted and a C-shaped deformation pattern occurred after significant yielding (Figure 2-3). Plastic rotations developed unevenly in the top and bottom column segments, indicating the propensity for a story mechanism to form in systems with weak columns.

MacRae et al. (1990) tested a number of cantilever steel columns (250UC73) made of Grade 250 steel. These columns were designed as beam-column members in moment resisting frames. They were subjected to combined axial loads and reversed cyclic lateral deformations. Axial shortening exceeding 7% of the column length was reported in the columns. Axial shortening increased as local buckling became more significant during the loading history. This same study showed that axial shortening is independent of the plastic hinge length, Lp. A number of additional experimental and analytical studies related to column behavior under combined axial load and bending have been conducted in New Zealand (e.g., McRae et al., 2006; Peng et al., 2007, 2008; and NZSEE 2006). Newell and Uang (2006) tested a set of W14 sections ($6.9 < h/tw < 17.7$ and $3.1 < b f /2t f < 7.14$) under cyclic lateral displacement and cyclic axial load levels reaching 35–75% *Py* in compression. These beam-columns were able to sustain story drifts of 7–9%radians prior to 10%reduction in flexural strength due to cyclic deterioration. Interestingly, these tests showed that stocky sections experience minor strength deterioration and twisting even at larger drifts and high axial load levels. No web local buckling was observed. The relatively small amplitude of flange local buckling observed at 6% drift (more than three times the maximum expected drift) provided an indication that strength degradation due to flange local buckling is not expected to present a problem for the seismic design of the tested W14 column sections.

Lignos and Krawinkler (2009) evaluated the effects of lateral bracing on seismic performance of more than 300 beam-to-column connections tested worldwide. It was concluded that additional lateral bracing was not effective in improving the precapping plastic rotation of steel beams, but that lateral bracing was effective in reducing the rate of cyclic deterioration, particularly for beams with reduced beam sections (RBS). This can be seen in Figure 2-2, which shows the deduced moment rotation diagrams of two nominally identical RBS connections with W30×99 steel beams, in which the insertion of additional lateral bracing close to the RBS region resulted in a small, but measurable improvement in hysteretic behavior. Zhang and Ricles (2006) demonstrated experimentally that once plastic hinging occurs in reduced beam section (RBS) moment connections to a deep-wide-flange column, such columns are more susceptible to twisting, i.e., global instabilities may occur to the columns NIST (2011) established a research plan that highlighted the need for further research related to the behavior of deep wide-flange beam-columns as part of steel SMFs.

Elkady and Lignos (2012) showed that at high constant axial load ratios (e.g., 50% *Py)*, deep wide-flange beam-columns with web and flange slenderness ratios close to the seismic compactness requirements for highly ductile members can lose their flexural strength capacity before reaching chord rotations in the order of 3% radians.It confirm that the cyclic deterioration in strength and stiffness of such members is severe compared to that of stocky beam-columns.E.J.Kaufmann,B.Metrovich,A.W.Pense (2001)were carried out cyclic strain tests following ASTM E606 methodology on A 572 Gr.50 and A913 Gr.50 rolled sections. Strain aging effects were found to be small in A572 Gr. 50 rolled sections. The cyclic inelastic strain behaviour of both sections found similar.

Krawinkler (1999)The objective of this work is to improve the knowledge base on the seismic behaviour of typical steel moment resisting frame structures, considering regions of different seismicity and sets of ground motions of various intensities and frequency characteristics. The behaviour and response of different height structures in losangeles, seattle, and boston were studied. Local(element) seismic demands are found to be very sensitive to a multitude of factors, which may result in a concentration of plastic deformation demands in either the beams or the panel zones, or in sharing of demands between these two elements and possibly also the columns.

III. METHODOLOGY

An American wide flange section is used to validate in ANSYS software by applying SAC symmetric cyclic loading protocol and a compressive axial load. Then we have to find out the Indian standard sections which can be used in special moment frames. This can be possible by choosing sections having larger web depth as well as lower slenderness ratios. Methodology can be summarized into:

- Selection of a deep wide flange steel section
- Validation using ANSYS
- Selection of Indian sections
- Modeling using ANSYS and apply same loading protocol
- Analyze deterioration curve
- Find out flexural strength capacity
- Changing axial load ratios
- Check the adaptability of Indian sections as deep wide flange steel columns used in special moment frames.
- Interpretation of results

Cycle Group	No. of Cycles	Drift Angle Amplitude (rad)
1	6	0.00375
2	6	0.005
3	6	0.0075
4	4	0.01
5	2	0.015
6	2	0.02
7	2	0.03
8 and greater	2	Increment of 0.01 until failure

Table 1: Sequence of Loading of SAC Standard Protocol

Section	Depth (d)mm	Flange Width (b_f)mm	Flange Thickness (t_f)mm	Web Thickness (t_w)mm
ISMB 600	600	210	20.3	12.0
ISWB 600	600	250	21.3	11.2
WPB 900	900	300	35.0	18.5

Table 2: Indian Section Details

Sections	P_y	0% P_y	20% P_y	50% P_y
ISMB600	7029.0kN	0	1405.89	3514.725

ISWB60	7667.1	0	1533.42	3833.55
WPB900	16708.5	0	3341.7	8354.25

Table 3: Applied Axial Loads (Py)

IV. FINITE ELEMENT ANALYSIS

Finite Element Analysis (FEA) is a computer based method of simulating or analysing the behaviour of engineering structures and components under a variety of conditions. It is an advanced engineering tool. The Finite Element Analysis is a numerical technique in which all the problems varying shape, boundary conditions and loads are maintained as they are but the solutions obtained are approximate. Solutions can be obtained for all problems by Finite Element Analysis.

V. FINITE ELEMENT MODELLING

ANSYS is general-purpose finite element software for numerically solving a wide variety of structural engineering problems. The ANSYS element library consists of more than 100 different types of elements. In the structures suite, ANSYS 18.1 has added new capabilities within its topology optimization technology to analyse complex materials and optimize designs making it easier for organizations to manufacture products.

Solid 186 is a higher order 3-D 20-node solid element that exhibits quadratic displacement behaviour. The element is defined by 20 nodes having three degrees of freedom per node: translations in the nodal x, y, and z directions.

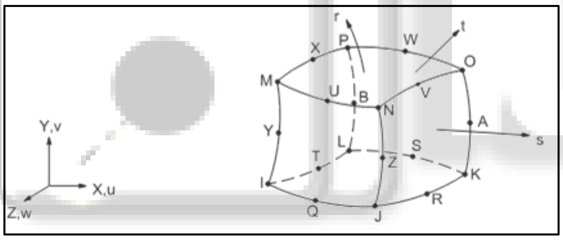

Fig. 1: SOLID 186 Element

The element supports plasticity, hyper elasticity, creep, stress stiffening, large deflection, and large strain capabilities. It also has mixed formulation capability for simulating deformations of nearly incompressible elasto plastic materials, and fully incompressible hyper plastic materials.

The meshing is the important step .This is the process of discretization of the structure to obtain accurate results. Finer the mesh size accuracy also increases. To obtain good results from the Solid186 element, a rectangular mesh is used. Therefore, the mesh is setup such that square or rectangular elements are created.

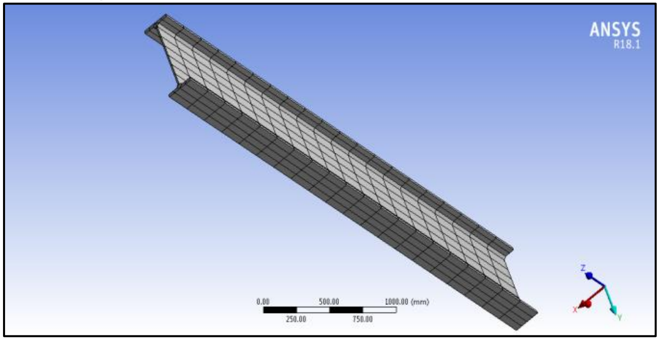

Fig. 2: Finite Element Mesh

The support conditions are provided below and above the steel column. Fully fixed boundary conditions are applied at the base of the column (i.e., fixed support) while partially fixed boundary conditions with flexible rotational stiffness are applied at the top of the column (i.e., flexible support) in order to consider the flexibility of the beam-to-column connection at the same location. These boundary conditions reflect more realistically first-story steel columns in SMFs because they can capture the moment gradient changes along a first story column once plastic hinging occurs at its base. For the fixed support, all six degrees of freedom is restrained.

VI. RESULTS & ANALYSIS

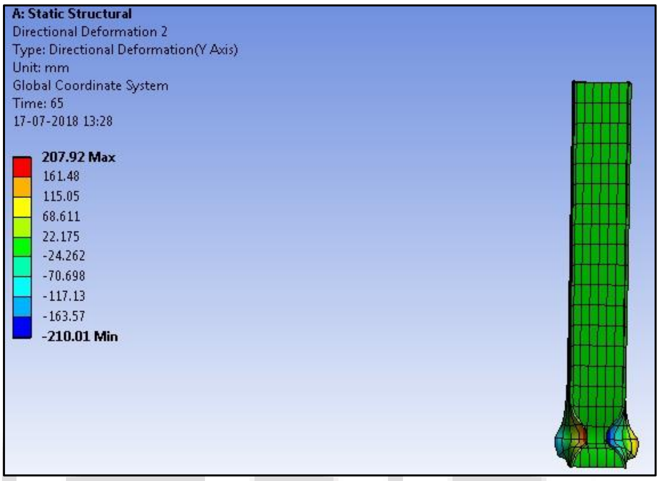
A: Static Structural
Directional Deformation 2
Type: Directional Deformation(Y Axis)
Unit: mm
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Fig. 3: Directional Deformation (Y Axis)

Fig.3 shows the total deformation of base of steel column. It clearly shows the local buckling of flanges and web. FE model is able to capture the local instabilities adequately. At a given drift level, the column flange local buckling deformation pattern is successfully captured. It shows maximum deformation occurred in y axis or along the y direction, it was because of the lateral loading.

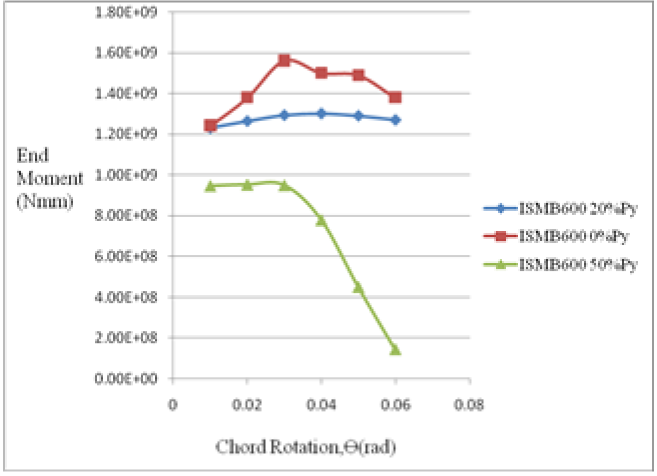

Fig. 4: End Moment-Chord Rotation Curve For ISMB600 0% P_y, 20% P_y and 50% P_y

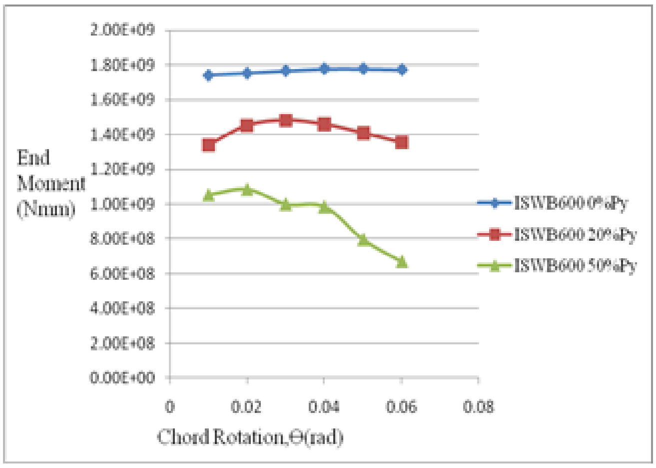

Fig. 5: End Moment-Chord Rotation Curve for ISWB600 0% P_y, 20% P_y and 50% P_y

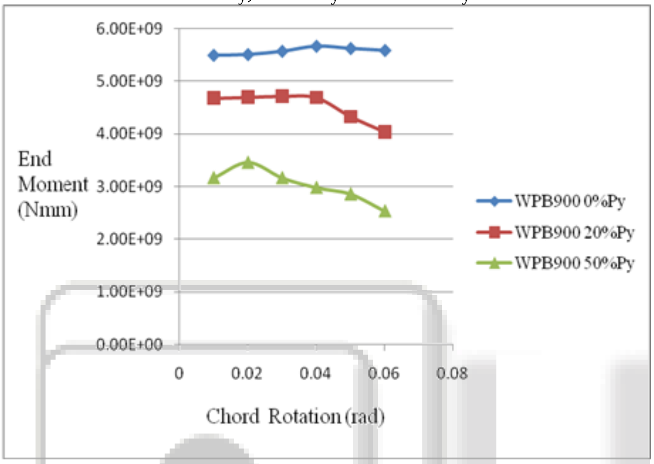

Fig. 6: End Moment-Chord Rotation Curve for WPB600 0% P_y, 20% P_y and 50% P_y

VII. CONCLUSIONS

The following conclusions can be drawn:

1) Higher the axial load ratio M_{max} gets decreasing also earlier the occurrence of local buckling.
2) Cyclic hardening decreases due to early occurring local buckling.
3) Lower the slenderness ratio gives more strength to sections by delaying local buckling. This is attributed to:
4) The stocky web and flanges of sections which delay the onset of local buckling and hence flexural strength deterioration.
5) A large number of small-drift inelastic cycles prior to the occurrence of local buckling based on the symmetric lateral loading protocol.
6) Is parallel flange sections are better than tapered flange sections. Because parallel flange sections have a more sectional area to carry much more load values.
7) Indian sections have less depth for webs compared to the sections of other advanced countries. Hence we need to develop sections especially Parallel flange sections, with higher plastic moment capacity for the better seismic adaptability.
8) By the available parallel flanged sections we can develop special moment frames but can be possible to moderate seismic areas rather than high seismic regions.

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REFERENCES

[1] Dafalias Y, Popov E (1975) “A model of nonlinearly hardening materials for complex loading*”. J Acta Mech 21(3):173–192.* doi:10.1007/BF01181053 .

[2] De Souza Neto EA, Peri´c D, Dutko M, Owen DRJ (1996*)* “Design of simple low order finite elements for large strain analysis of nearly incompressible solids”. *Int J Solids Struct 33(20–22):3277–3296.* doi:10.1016/ 0020-7683(95)00259-6.

[3] Elkady A, Lignos DG (2012*)* “Dynamic stability of deep slender steel columns as part of special MRFs designed in seismic regions: finite element modeling*”. In: Proc. first international conference on performance-based and life-cycle structural engineering (PLSE), Hong Kong, December 5–7, 2012* .

[4] Elkady A, Lignos DG (2013) “Modeling of the composite action in fully restrained beam-to-column connections: implications in the seismic design and collapse capacity of steel special moment frames*”. J Earthq Eng Struct Dyn.* doi:10.1002/eqe.2430 .

[5] Elkady *et al.* (2014).“Analytical Investigations of the Cyclic Behaviour and Plastic Hinge Formations in Deep Wide-Flange Steel Beam-Columns*” (ASCE).*

[6] FEMA (2000) “State of the art report on connection performance”, Report FEMA-355D. Federal Emergency Management Agency, Washington .

[7] Gupta A, Krawinkler H (1999) “Seismic demands for the performance evaluation of steel moment resisting frame structures”, *Report No. 132. The John A. Blume Earthquake Engineering Center, Stanford University, Stanford.*

[8] Kaufmann EJ, Metrovich BR, Pense AW (2001) “Characterization of cyclic inelastic strain behavior on properties of A572 Gr. 50 and A913 Gr. 50 rolled sections*”, ATLSS Report 01–13. National Center for Engineering Research on Advanced Technology for Large Structural Systems, Bethlehem.*

[9] Krawinkler H, Zohrei M, Lashkari-Irvani B, Cofie NG, Hadidi-Tamjed H (1983) “Recommendations for experimental studies on the seismic behavior of steel components and materials*”, Report No. 61. The John A. Blume Earthquake Engineering Center, Stanford University, Stanford* .

[10] Lignos DG, Krawinkler H (2011) “Deterioration modeling of steel components in support of collapse prediction of steel moment frames under earthquake loading.”*J Struct Eng 137(11):1291–1302.* doi:10.1061/ *(ASCE)*ST.1943-541X.0000376 .

[11] Lignos DG, Hikino T, Matsuoka Y, Nakashima M (2013) "Collapse assessment of steel moment frames based on E-defense full-scale shake table collapse tests". *J Struct Eng 139(1):120–132.* doi:10.1061/(*ASCE)*ST. 1943-541X.0000608.
[12] Lignos DG, Krawinkler H (2013) "Development and utilization of structural component databases for performance-based earthquake engineering*". J Struct Eng 139(8):1382–1394.* doi:10.1061/(*ASCE)*ST. 1943-541X.0000646.
[13] MacRae GA, Carr AJ, Walpole WR (1990) "The seismic response of steel frames", Report No. 90–6. Department of Civil Engineering, University of Canterbury, Christchurch .
[14] MacRae GA, Hyde K, Walpole WR, Moss PJ, Hyland C, Clifton GC, Mago N (2006) "Column axial shortening effects in steel frames*". In: Proceeidngs of New Zealand society for earthquake eng. conference, Napier, New Zealand, March 10–12, 2006* .
[15] MacRae GA, Urmson CR, Walpole WR, Moss P, Hyde K, Clifton C (2009) "Axial shortening of steel columns in buildings subjected to earthquakes*". Bull N Z Soc Earthq Eng 42(4):275–287* .
[16] Newell JD, Uang CM (2006) "Cyclic behavior of steel columns with combined high axial load and drift demand", *Report No. SSRP-06/22. Department of Structural Engineering, University of California, San Diego* .
[17] NIST (2011) "Research plan for the study of seismic behavior and design of deep, slender wide flange structural steel beam-column members*", NIST GCR 11–917-13. NEHRP consultants Joint Venture* .
[18] PEER/ATC (2010) "Modeling and acceptance criteria for seismic design and analysis of tall buildings", PEER/ATC 72–1. Prepared for Pacific Earthquake Engineering Research Center (PEER) by Applied Technology Council (ATC), Redwood City, San Diego.
[19] Popov EP, Bertero VV, Chandramouli S (1975) "Hysteretic behavior of steel columns". *Earthquake Engineering Research Center, University of California, Oakland* .
[20] Zareian F, Lignos DG, Krawinkler H (2010) "Evaluation of seismic collapse performance of steel special moment resisting frames using FEMA P695 (ATC-63) methodology". *Proc. ASCE Structures Congress, Orlando, May 12–15,* pp 1275–1286. doi:10.1061/41130(369)116