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Analytical Study of End Plate Connections and Its Design

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Abstract:

The unexpected failure of welded connections observed during the 1994 Northbridge earthquake and the 1995 Kobe earthquake lead to the initiation of research for finding an alternative connection for use in seismic resisting steel moment frames. The Extended End plate connection is considered as a suitable and efficient alternative to welding in seismic regions. The study of end-plates in moment resisting connections for beam to beam splices and beam to column connections dates to the early 1960's. These connections were primarily used in pre-engineered metal buildings. This study deals with the design and analysis of Extended End Plate Connections. In this current study, 3 configurations of Extended End plate connections are designed using AISC design parameters, for American Steel Sections and Indian Steel Sections. These connections are then modelled and analyzed by Finite Element Analysis using the ANSYS Workbench software. 6 models are prepared in total and analyzed. Using the ANSYS Workbench Software parameters such as Total Deformation, Maximum Principal Stress, Maximum Principal Strain and Hysteresis Curves are evaluated. The results obtained from the Design and Finite Element Analysis is then compared between the American Steel sections and the Indian Steel Sections to understand whether the Indian Steel Sections conform to the American Codal Provisions.

Keywords — **Extended End Plate Connections, Stiffened Connection, Unstiffened Connection, Finite Element Analysis, ANSYS Workbench, Total Deformation, Maximum Principal Stress, Maximum Principal Strain, Hysteresis Curves.**

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1. INTRODUCTION

The study on the behavior and design of steel seismic load resisting moment frames has been going on for the past several years. In all these studies it was found that the bolted and riveted moment connections offered better performance as compared to the flange-welded moment connections. A good amount of research was funded by the FEMA (Federal Emergency Management Act) through the SAC joint venture. This research was also referred to as the SAC steel project and was divided into 2 phases. The first phase focused on determining the cause of failure of the welded connections while the second phase focused on finding an alternative to the seismic moment resisting connections. But because of limited cyclical testing of moment end plate connections, extensive research had to be sponsored by the FEMA. However, the research sponsored by FEMA does not include the investigation of built up sections. Therefore, the Metal Building Manufacturer's Association (MBMA), a group of private companies involved in the design, fabrication, and erection of pre-engineered metal buildings, along with thE American Institute of Steel Construction (AISC), had independently sponsored the testing of practical flush and extended moment end-plates under cyclic loading.

The Extended End plate connection is considered as a suitable and efficient alternative to welding in seismic regions. The study of end-plates in moment resisting connections for beam to beam splices and beam to column connections dates to the early 1960's. These connections were primarily used in pre-engineered metal buildings until the mid-1980 in the US. The Metal building industry, which primarily uses built up steel sections found these connections to be very economical for several reasons. The shop welding required for the endplates was feasible because of the welding already required for the built-up members. Also, the plate material used for the end plates could be cut from the same plate from which the flanges of the built-up sections were cut. Further, erection of the metal buildings was relatively simple due to all bolted connections, and therefore advantageous. No field welding was required, allowing construction to be done in cold conditions, and construction time to be reduced. The Extended End Plate Configurations designed and analyzed in this paper are shown below.

Fig-1 Extended End Plate Configurations

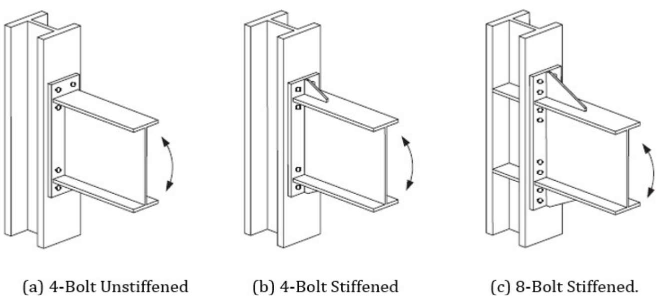

FIG.1 EXTENDED END PLATE CONNECTION

1.1 Literature Review

[1] Iman Faridmehr, Mamood Md. Tahir, Mohd Hanim Osman and **Mohammadamin Azimi.** They studied the cyclic behavior of connections wherein a test program was considered to clarify the cyclic characteristics of eight full- scale unstiffened extended endplates with variable parameters and one Side Plate moment connection. All specimens were subjected to 2010 AISC seismic provision loading protocol where flexural strengths were identified at each interstorey drift angle. The test specimen was casted with unstiffened extended end plate beam-column connection, wherein the column height adopted was 3m and the beam has a cantilever portion of 1.3m. A similar arrangement was casted for the Side Plate connection. The beam is restrained from lateral movement at mid-point. The sections used are Hot rolled I Shaped Perwaja Steel Sections (PSS) confirming to British Standard Institute BS EN (1993). The load is applied at the tip or edge of the beam i.e. Eccentric Loading. This loading is achieved using a hydraulic actuator. Loading sequence used for seismic evaluation was that prescribed by the AISC Specifications (2010). It is observed that the unstiffened extended end plate connection failed to meet the AISC seismic Specifications, the reason being that the strong column- weak beam principle recommended by the AISC was not considered in design. It was also observed that the unstiffened extended end plate connection also was subjected to a large shear load leading to unacceptable level of rotation. **[2] Anju Das K** and **Alice Johny**. The aim of their study is that to analyze the beam- column joint under seismic loading with bolts and endplates. The work is carried out at the joints having flush endplate & extended end plate with different thickness subjected to seismic loading. The sections used are CFST (Concrete Filled Steel Tube) column size: 300x300x3000x6mm., Beam size: H beam of 120x400x8x6mm, Beam length: 900mm, Endplate of size: 8,10, 12 mm., Bolt size: M22, 10.9 HSFG bolts (ISO 898- 1 Gr.10.9), Bolt type: M22, 10.9 HSFG bolts (IS1367(Part3): 20002, ISO 898-1 Gr. 10.9), Grade of concrete of steel pipe: M30. Loading is applied as axial force acting at joint: 300 KN. Seismic loading is applied in a cyclic manner with 10 load steps. Modelling and Analysis is done on ANSYS Workbench Software. **[3] B.T. Adey, G.Y. Grondin** and **J.J.R. Cheng**. In this paper an experimental investigation of 15 cyclically loaded extended end plate connections was undertaken to assess the significance of some design parameters. The parameters investigated were beam size, bolt layout, end plate thickness, and use of extension stiffeners, welding process and weld preparation. The study involved the testing of 15 full sized extended end plate connections under cyclic loading. The loading was applied using Applied Technology Council's (ATC) Guidelines for Cyclic Seismic Testing of Components of Steel Structures (ATC 1992). The loading was applied in two blocks of loading in the elastic range, with 3 loading cycles in each block. Eleven test specimens were designed to trigger failure in the end plate rather than in the beam. **[4] Shizhe Chen, Chao Zhou** and **Zhan Wang.** This study deals with the investigation of the rotational behavior of the extended end plate joints. The Numerical analysis is carried out using the ABAQUS software. Two specimens with different extended end plate joint configurations were employed for the tests. The experiments were mainly focused on initial stiffness, which reflects the mechanical behavior of panel zone. The loading is applied as an axial load placed on the column at a constant value of 325kN. The concentrated load was imposed simultaneously at each end of the steel beam by hydraulic jacks. The same was simulated using the finite element analysis software ABAQUS and a comparison between the experimental and numerical results is presented. The behaviors and the capacities of two extended end plate connections were investigated by using monotonic tests and FEM analysis. **[5] Mahyar Maali,** the paper presents the experimental and numerical results of 6 full scale beams to column connections with bolted end plates in two groups. The effects of vertical and horizontal stiffeners on the static behavior of the semi rigid beam to column bolted connections were investigated. The aim of this research was to analyze the influence of end plate connections that utilize the IPE profile with stiffeners welded on the behavior of steel connections, to provide the necessary data for improving Euro Code, efficient use of residue IPE profiles and back to the consumption cycle. The experimental models were developed to predict of failure modes the end plate connections with horizontal and vertical stiffeners under static loading in 2 groups. The FEMs of the semi-rigid vertical and horizontal stiffened bolted connections were made and their results were compared to the experimental results.

1.1 Objectives

i. To study the various moment connections in steel structure.
ii. To understand the design procedure for end plate moment connection in steel structure.
iii. To compare the behavior of American steel sections to the Indian Steel Sections.
iv. To analyze the American sections and Indian sections by Finite Element Analysis and compare the results.
v. To observe whether the Indian steel sections satisfy the AISC Codal Provisions.

1.2 Methodology

i. The design procedure followed by the AISC and FEMA for the design of extended end plate connection is being thoroughly studied.
ii. Following the design procedure, design the connections for American as well as Indian steel sections.
iii. Three types of connections need to be designed i.e. 4-Bolt Extended Unstiffened End Plate Connection. 4-Bolt Extended Stiffened End Plate Connection. 8- Bolt Extended Stiffened End Plate Connection.
iv. Compare the design results of the American steel sections and the Indian steel sections.
v. Model and analyze the connections of the American steel sections and Indian steel sections in ANSYS Workbench Software. Compare the analysis results of the American steel sections and Indian steel sections.

2. DESIGN PROCEDURE

The Design procedure used for designing these connections is obtained from AISC 358-16 (Prequalified Connections for Special and Intermediate Steel Moment Frame for Seismic Applications), AISC Steel Design Guide 4 (Extended End Plate Moment Connections with Seismic and Wind Application) and FEMA 351(Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment Frame – Buildings).
The four primary design parameters for the design of extended end-plate moment connections subjected to cyclic loading are:

1. The required connection design moment
2. Connection bolt strength
3. End plate strength
4. Column flange bending strength

The procedures use yield-line theory for determination of the end plate strength and a simplified method to determine the bolt forces.

1.1 Section Properties

Table -1: Indian Section Properties

BEAM (ISMB500@ 86.9kg/m)	COLUMN (ISWB600@ 145kg/m)
Depth, D = 500 mm	Depth, D = 600 mm
Flange width, bf = 180 mm	Flange width, bf = 250 mm
Flange Thickness, tf = 17.2 mm	Flange Thickness, tf = 23.6 mm
Web thickness, tw = 10.2 mm	Web thickness, tw = 11.6 mm
Sectional Area, A = 111 cm^2	Sectional Area, A = 185 cm^2
Section Modulus, Zx = 1.8×10^6 mm^3	Section Modulus, Zx = 3.85×10^6 mm^3
Yield Strength, Fy = 250 N/mm^2	Yield Strength, Fy = 250 N/mm^2
Ultimate Strength, Fu= 410 N/mm^2	Ultimate Strength, Fu = 410 N/mm^2
HSFG 8.8 Grade Bolts with Ultimate Tensile Strength, Ft = 800 N/mm^2	

Table -2: American Section Properties

BEAM (W21 X 55)	COLUMN (W14 X 109)
Depth, D = 528 mm	Depth, D = 363 mm
Flange width, bf = 209 mm	Flange width, bf = 371 mm
Flange Thickness, tf = 13.3 mm	Flange Thickness, tf = 21.85mm
Web thickness, tw = 9.5 mm	Web thickness, tw = 13.34mm
Sectional Area, A = 104.51 cm^2	Sectional Area, A = 206.45 cm^2
Section Modulus, Zx = 2×10^6 mm^3	Section Modulus, Zx = 7.32×10^6 mm^3
Yield Strength, Fy = 345 N/mm^2	Yield Strength, Fy = 345 N/mm^2
Ultimate Strength, Fu= 448 N/mm^2	Ultimate Strength, Fu = 448 N/mm^2
ASTM A490 Bolts with Ultimate Tensile Strength, Ft = 780 N/mm^2	
ASTM A325 Bolts with Ultimate Tensile Strength, Ft = 621 N/mm^2	

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2.1 Design Results

The connections are designed both manually and by using Microsoft Excel software following the design procedure obtained from AISC 358-16 (Prequalified Connections for Special and Intermediate Steel Moment Frame for Seismic Applications), AISC Steel Design Guide 4 (Extended End Plate Moment Connections with Seismic and Wind Application). The design is carried out for a Shear Force = 178kN.
The following design results are obtained after designing the Connections for the 3 configurations.

Table -3: Design Results of 4 – Bolt Extended Unstiffened End Plate Connection

American **4-Bolt Unstiffened**	**Indian** **4-Bolt Unstiffened**
Moment at the face of the column, M_{uc} = 908 kN-m	Moment at the face of the column, M_{uc} = 592 kN-m
Bolt Diameter d_b = 32 mm	Bolt Diameter d_b = 30 mm
No prying bolt moment $ØM_{np}$ = 953.08 kN-m	No prying bolt moment $ØM_{np}$ = 917 kN-m
End Plate Thickness, t_p = 32 mm	End Plate Thickness, t_p = 32 mm
Factored beam Flange Force, F_{fu} = 1762 kN	Factored beam Flange Force, F_{fu} = 1227 kN

Table - 4: Design Results of 4 – Bolt Extended Stiffened End Plate Connection

American **4-Bolt Stiffened**	**Indian** **4-Bolt Stiffened**
Moment at the face of the column, M_{uc} = 908 kN-m	Moment at the face of the column, M_{uc} = 592 kN-m
Bolt Diameter d_b = 32 mm	Bolt Diameter d_b = 30 mm
No prying bolt moment $ØM_{np}$ = 953.08 kN-m	No prying bolt moment $ØM_{np}$ = 917 kN-m
End Plate Thickness, t_p = 29 mm	End Plate Thickness, t_p = 29 mm
Factored beam Flange Force, F_{fu} = 1762 kN	Factored beam Flange Force, F_{fu} = 1227 kN
Minimum Design Strength Min $ØR_n$ = 1065 kN	Minimum Design Strength Min $ØR_n$ = 596 kN
Stiffener Design Force F_{cu} = 700 kN	Stiffener Design Force F_{cu} = 631 kN
End Plate Stiffener used 9.5 mm x 92 mm x 165 mm	End Plate Stiffener used 7.6 mm x 66 mm x 152 mm
Column Flange Stiffener thickness, t = 13 mm	Column Flange Stiffener thickness, t= 30 mm

Table -5: Design Results of 8 – Bolt Extended Stiffened End Plate Connection

American **8-Bolt Stiffened**	**Indian** **8-Bolt Stiffened**
Moment at the face of the column, M_{uc} = 913 kN-m	Moment at the face of the column, M_{uc} = 600 kN-m
Bolt Diameter d_b = 25 mm	Bolt Diameter d_b = 20 mm
No prying bolt moment $ØM_{np}$ = 972 kN-m	No prying bolt moment $ØM_{np}$ = 753 kN-m
End Plate Thickness, t_p = 23 mm	End Plate Thickness, t_p = 25 mm
Factored beam Flange Force, F_{fu} = 1774 kN	Factored beam Flange Force, F_{fu} = 1242 kN
Minimum Design Strength Min $ØR_n$ = 1255 kN	Minimum Design Strength Min $ØR_n$ = 774 kN
Stiffener Design Force Fcu = 519kN	Stiffener Design Force Fcu = 467 kN
End Plate Stiffener used 13mm x 152mm x 267mm	End Plate Stiffener used 13mm x 120mm x 244mm
Column Flange Stiffener thickness, t = 13 mm	Column Flange Stiffener thickness, t = 25 mm

3. CONCLUSIONS

This Study deals with design and analysis of Extended End Plate Connections in Steel Structures designed using AISC Codal Provisions. The design is carried out on 3 configurations of the Extended End Plate Connections for both American and Indian Steel sections. FEM analysis is carried out for these sections using the ANSYS Workbench Software. The following conclusions are obtained from this study

1. The Design results show us that the moment capacity and design strength of the Extended End Plate Connections of American Steel Sections is higher than the Extended End Plate Connections of Indian Steel Sections.
2. From the design results it can also be observed that the thickness of the end plate impacts the stiffness of the connection i.e. increasing the thickness of the end plate increases the stiffness of the connection.
3. It can also be seen from the design results

that the Beam Flange Force of Extended End Plate connection of the American Steel Sections is higher than that of the Extended End Plate connection of the Indian Steel Sections.

4. From the design results it can also be observed that the No Prying Bolt Moment of Extended End Plate connection of the American Steel Sections is higher than that of the Extended End Plate connection of the Indian Steel Sections.
5. From the analysis results it can be observed that the Extended End Plate connection of Indian Steel Sections undergoes nearly 30% less Total Deformation as compared to Extended End Plate connection of the American Steel Sections.
6. It can also be observed that the Extended End Plate connection of Indian Steel Sections are subjected to nearly 20% less Maximum Principal Stress as compared to Extended End Plate connection of the American Steel Sections.
7. From the analysis results it can also be observed that the Extended End Plate connection of Indian Steel Sections undergoes nearly 20% less Maximum Principal Strain as compared to Extended End Plate connection of the American Steel Sections.

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