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MOMENT RESISTING CONNECTION WITH SIDEPLATE (GEOMETRIC ASPECTS)

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SUMMARY

Numerous steel moment-resisting connections failed by brittle fracture during the Northridge earthquake. Sideplate connection uses a pair of parallel full-depth side plates to connect the beam to the column, The beam never touches the column. The physical separation between beam and column eliminates the peaked triaxial stress concentration. Eleven 3D finite element models were developed to represent the nonlinear behavior of sideplate connections. The results indicate that this connection type has sufficient stiffness, strength and ductility to classify it as rigid, full-strength, ductile connection.

Keywords: Steel structure, moment connection, side plate, seismic resisting.

INTRODUCTION

The 1994 Northridge earthquake caused largely unexpected damages to steel moment-resisting frame connections. The majority of damage occurred at the widely used welded flange-bolted web (WFBW) "traditional" connection, what has become known as the "pre-Northridge" moment connection. This connection detail has been prescribed as ductile steel moment frame by nearly all major U.S. seismic codes.

The researches carried out just after the Northridge earthquake were intended primarily to find the causes and cures for the adverse behavior of the moment connections. Only after this phase was accomplished did they look for solutions. Although a variety of different types of fractures were observed in the moment frame joints, but the premature brittle fractures of groove weld and base metal due to triaxial/peaked stress concentration were the most common type of the damages in large number of steel buildings. Fractures near the interface of the beam flange groove weld and the column flange were also frequently observed. These included; 1) The through thickness divot pull-out of the column flange at the groove weld due to cyclic loading of flange or heat affected zone. 2) Fractures running across portions of the column flange (kinking of flange) and web due to reliance of column weak panel zone. In some cases, the fractures passed through the full depth of the column section. 3) Fracture initiating along the fillet transition between column flange and its web (K-line) of rolled shapes due to reduction of ductility and resistance of the web created during the mill process.

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Some other possible causes of the observed damage were as follows (Engelhardt,et al.[1]):

- Shortcoming of adequate welding inspection (ultrasonic testing techniques) and workmanship.
- Use of weld metal with low Charpy V-notch toughness.
- The notch effect created by left in place backup bars.
- Uneven distribution of stress across the width of the beam flange groove weld.
- High values of yield to tensile strength ratio (F_y/F_u). Excess yield strength of structural steel.
- Inadequate notch toughness of column material.
- Inadequate trough-thickness strength or ductility of column flanges.

It was seen that the poor performance of the “traditional” connections was a result of a number of complex and inter-related factors. Generally these causes are mostly intrinsic to moment-resisting connections that employ the use of T-joint complete-penetration groove weld to connect the beam flange directly to the column flange. The largest factor was that the geometry induced sever stress concentration in regions where material yielding was inhibited by triaxial restraint and where strength and toughness were highly sensitive to workmanship. The geometry also made it difficult to weld without large defects.

Different practical solutions for each type of damages were proposed. Actually the structural engineers have the choices between two large groups of connection when designing or retrofitting steel moment frame buildings in seismic zones.

- Use one of the nine beam-to-column pre-qualified connections design recommended by the SAC Steel Project.(FEMA 350 [2]). The selection of an appropriate connection for a specific application is based on an understanding of the strength and weakness of each type of connection regarding the seismic zones, and the type of framing system.

- Using one of the proprietary connection designs as discussed in FEMA350 [2], chapter 3.8.Following the Northridge earthquake, several innovative engineering firms designed and begin to market alternative moment connection details. One of these proprietary designs was SidePlate (SP) connection system, invented by David L. Houghton, S.E., CEO [3] and president of the SidePlate System, Inc.

SIDEPLATE MOMENT CONNECTION

Side plate connection technology uses a pair of parallel full-depth side plates to connect the beam to the column, as shown in figure 1. Thereby eliminating the traditional welded connection between the end of the beam and the face of the column flange. The beam never touches the column. The physical separation between beam and column eliminates the peaked triaxial stress concentration inherent in all other welded moment connection types (Houghton [3]). The increased stiffness of the side plates largely stiffens the global frame structure and eliminates reliance on panel zone deformation by providing three panel zones (the two side plates plus column’s own web). Top and bottom flange cover plates are used, when dimensionally necessary, to bridge the difference between flange widths of the beam and the column. The connection system uses all shop, fillet-welded fabrication done in flat or horizontal position using column-tree construction. Shop fabricated column-trees and link beams are erected and joined in the field. All connection fillet welds are loaded principally in shear along their length.

Moment transfer from the beam to side plates, and from the side plates to the column is achieved trough the fillet welds. The side plates are designed with adequate strength and stiffness to force all significant plastic behavior of the connection system into the beam, in the form of flange and web local buckling centered at a distance of approximately 1/3 the depth of the beam away from the side plates (FEMA 350 [2]). Sideplate connection has no limit on the size of the connection members or the type of column – it can be a wide flange shape or box column or a cruciform column for biaxial applications (Davis [4]).

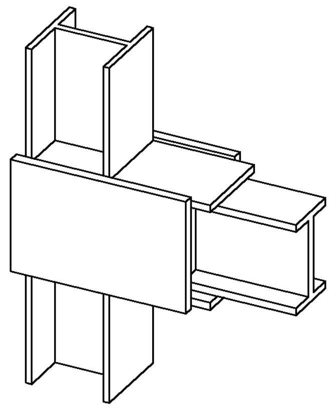

Figure 1. Sideplate System [3]

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The connection has been examined and pre-qualified for use with unlimited member size by three, American independent organizations: The ICBO Evaluation Service, Inc. [5], the City of Los Angeles Engineering Research Section(Department Of Building And Safety), and County of Los Angeles Technical Advisory Panel on Steel Moment Resisting Frame Connection Systems (FEMA 350 [2]).

SELECTED MODELS

To scrutinize the behavior of this innovative connection under the monotonic and cyclic loading and to compare the sideplate connection system with the other pre-qualified moment-resisting connections, a series of computational research were carried out. In the first part, we have studied the effect of geometric parameters on the behavior of the sideplate moment connections. Eight computer models were considered. We have studied the effect of:

T = Thickness of side plates.

s = Thickness of vertical shear plates.

c = Thickness of top and bottom beam flange cover plates.

The selected columns are IPB300 and IPB400 wide flange shapes. The beams are IPE shapes of 300, 400, 500, and 600 mm depth. The geometrical characteristics of the models are shown in table 1 and 2.

Table 1. Geometrical characteristics of general models

Model Name	Column Profile	Beam Profile	T (mm)	s (mm)	c (mm)
SPT1-1	IPB300	IPE300	15	10	10
SPT1-2	IPB300	IPE400	20	10	14
SPT1-3	IPB300	IPE500	25	10	15
SPT1-4	IPB300	IPE600	25	12	20
SPT1-5	IPB400	IPE300	15	10	10
SPT1-6	IPB400	IPE400	20	10	14
SPT1-7	IPB400	IPE500	25	10	15
SPT1-8	IPB400	IPE600	25	12	20

Table 2. Geometrical characteristics of models with different sideplate thickness

Model Name	Column Profile	Beam Profile	T (mm)	s (mm)	c (mm)
SPT1-1	IPB300	IPE300	15	10	10
SPT1-1a	IPB300	IPE300	10	10	10
SPT1-1b	IPB300	IPE300	20	10	10
SPT1-1c	IPB300	IPE300	25	10	10

FINITE ELEMENT MODELING

The finite element computation has been carried out with aid of ANSYS [6], which is a general purpose non-linear finite element program. The selected element was Solid 45. This element has 8 nodes with 3 degrees of translation freedom (in x, y and z direction) per nodes.

To simulate the size of the real structures the length of the connected beam and column were considered 200 cm and 300 cm respectively. The mesh sizes in different parts of the models were selected regarding the demanded precision. Since the gradient of the stress seems to be more sever near the connected zone, a very fine mesh was used in the vicinity of the connected areas (Figure 2).

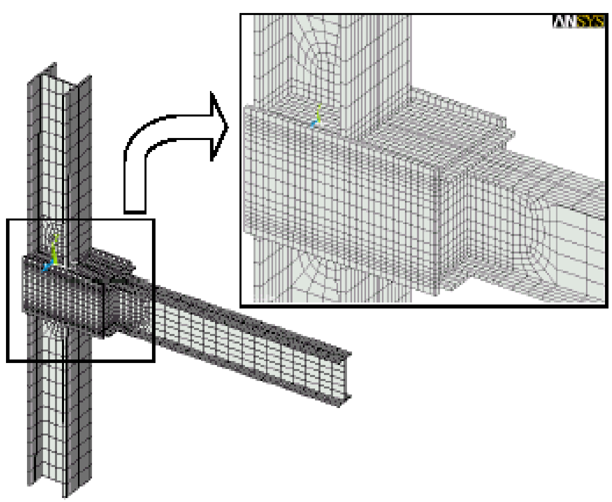


Figure 2. Finite Element Mesh

The boundary conditions for all models were the same. All degrees of freedom at the bottom of the columns were restricted. The nodes on the upper end of columns were also restricted in two directions perpendicular to the column, while the vertical displacements were permitted (See figure 3). The beams were loaded by vertical displacement at their free end. Two types of loading were applied: monotonic and cyclic load. The monotonic load was applied to all the computed models. To evaluate the cyclic behavior of the sideplate connection the SPT1-1 model was also tested under the cyclic load. (Not discussed in this paper). Monotonic loads were increased from zero to their maximum according to determined sub-steps.

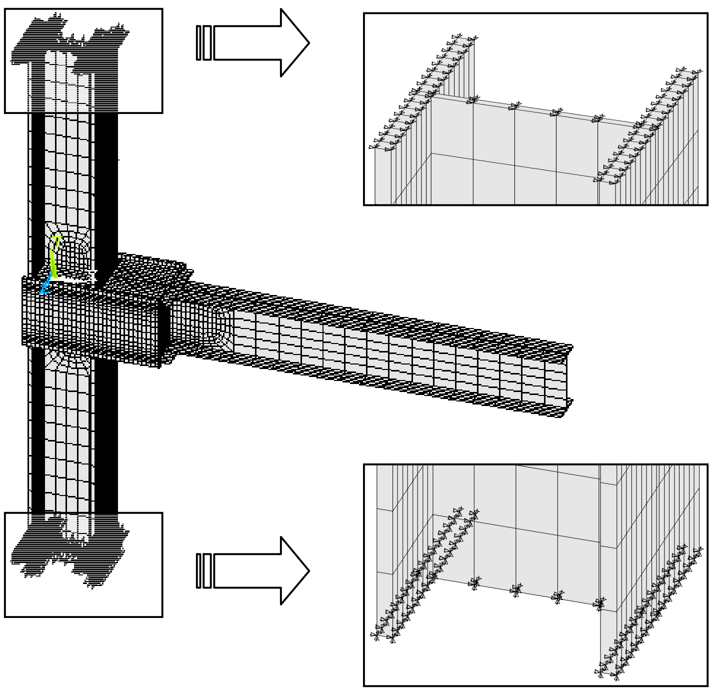


Figure 3. Boundary Conditions

MATERIAL PROPERTIES

A bilinear model was selected to represent the stress-strain curve of material (steel). A line having the slope equal to the elastic modulus of steel (200GPa) presented the elastic behavior of steel. The yield point of steel was considered at 240 MPa. A nearly horizontal line having a slope of 0.135 GPa represented the behavior of steel beyond the yield point (Figure 4). The material considered behaves as isotropic hardening.

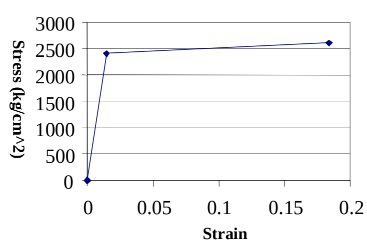


Figure 4. Bilinear model for steel

RESULTS

Effects of beams and columns sizes

The Von-Mises stress distribution for model SPT1-1 is shown in figure 5a. To have a better view of the stress distribution in the interior parts of the connection, in figure 5b we have displaced the side plate. It can be seen that the plastic hinge is shifted from the connected zone toward the beam. This means that, in sideplate connections, the brittle fracture of groove weld or formation of plastic hinge in the connecting zone is not the main cause of the damage. Another important point shown in figure 5, is that the stress in panel zone remains lower than the elastic limit. This is due to the existence of the two side plates. The reactions are divided into 3 plates (2 side plates and the web of the column).

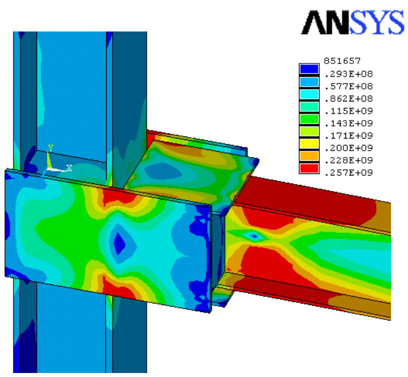


Figure 5a. Stress distribution

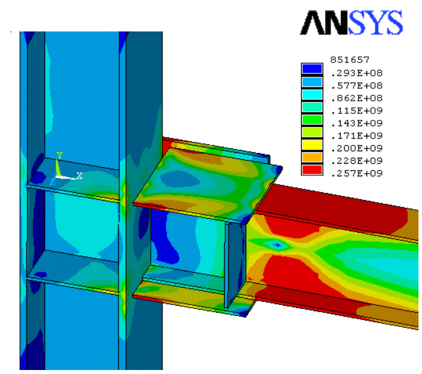


Figure 5b. Stress distribution (interior)

Figure 5. Stress distribution in model SPT1-1

The plastic strain in different parts of connection is shown in figure 6. We can see that all the plastic deformations take place in the beam, and the connection itself does not withstand the plastic deformation. It can be noticed that the connections with sideplate have remarkable ductility and energy absorption capacity.

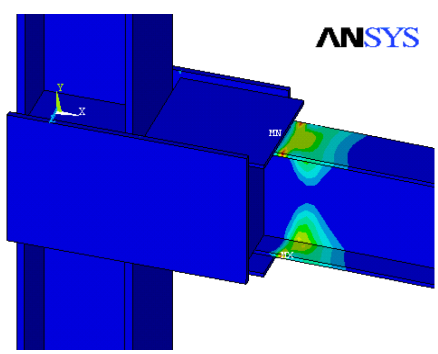


Figure 6. Plastic Strain in model SPT1-1

The load-deflection.(P-Δ) and moment-rotation (M-θ) diagrams for all models are demonstrated in figure 7 and 8 respectively. In figure 7, we can see that the models, which have IPB300 column are more ductile compared to the models with heavier columns (IPB400). Comparing the moment-rotation curves for different models, it can be seen that the size of columns has negligible effect on the plastic behavior of the models, This should be due to formation of plastic hinge in the beam section, which prevents transferring of deformation to the columns. It should be noted that the model SPT1-4 is an exception. In this model the beam section is much stronger than the column section. Hence, due to relative column weakness the plastic hinge takes place in the column web instead of the beam. For this reason the moment-rotation and load-deflection curves end up below the elastic limit. As we can see from figure 8, the nominal strength of the connections is greater than the strength of their connected beam. So we can conclude that all the studied connections (except model SPT1-4) can be considered as full strength.

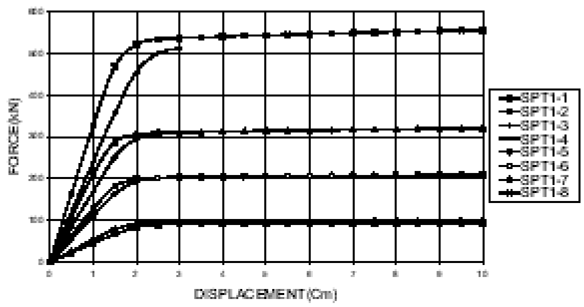


Figure 7. load-Deflection Diagrams

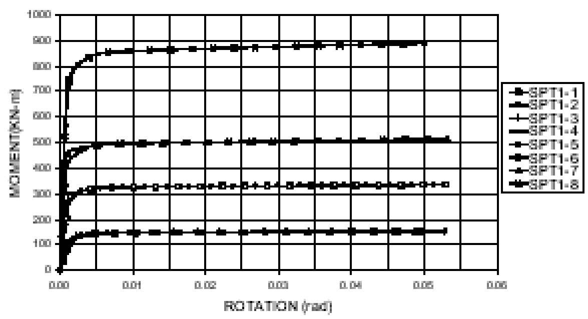
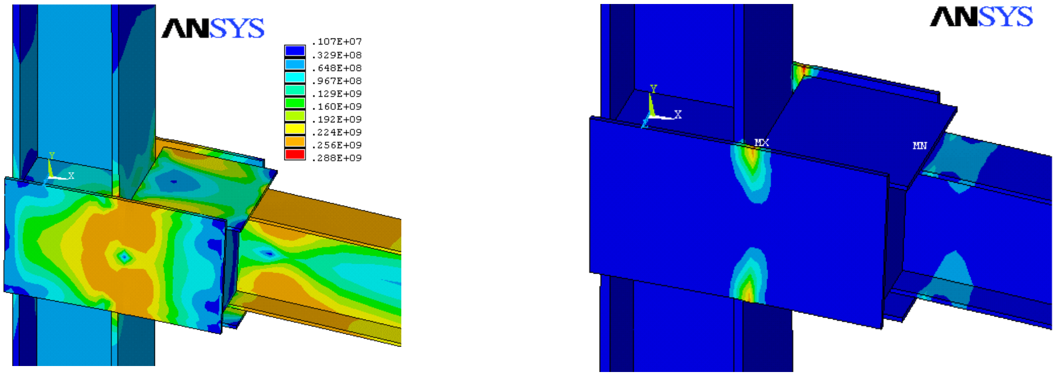


Figure 8. Moment-Rotation Diagrams

Effects of sideplates thickness

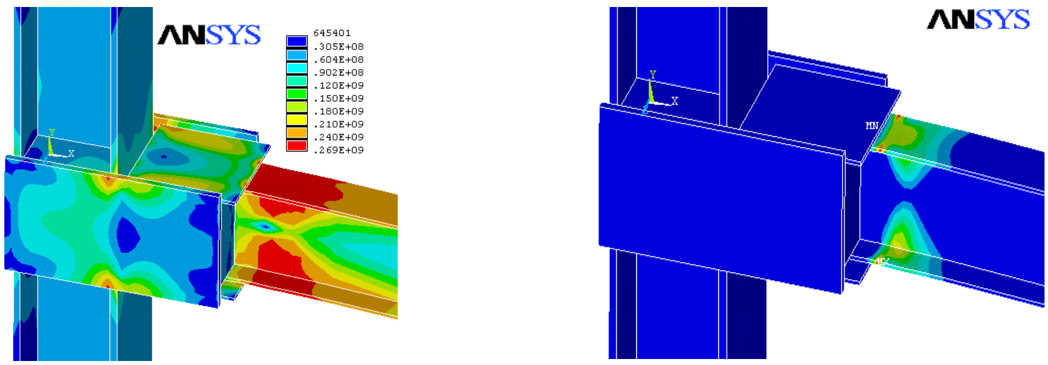
To evaluate the effect of sideplate thickness on the behavior of connection, we have studied new models with different sideplate thickness. As it is indicated in table 2, we have considered four models, SPT1-1a, SPT1-1, SPT1-1b, and SPT1-1c with plate thickness of 10, 15,20 and25 mm. respectively. The other characteristics of these models were exactly the same.

The distribution of stress and plastic strain for these models are shown in figures 5,6,9,10 and 11. It can be seen that in the case of connection using 10 mm. thick sideplates (figure 9) the plastic hinge has been produced in the side plates instead of the beam. The behavior of the models with thicker sideplates are nearly similar to each other, except that for the thicker plates the displacements shift toward the beam and the connections behave more stiffly. For the selected models it seems that the optimum thickness for the sideplates is 15 mm.



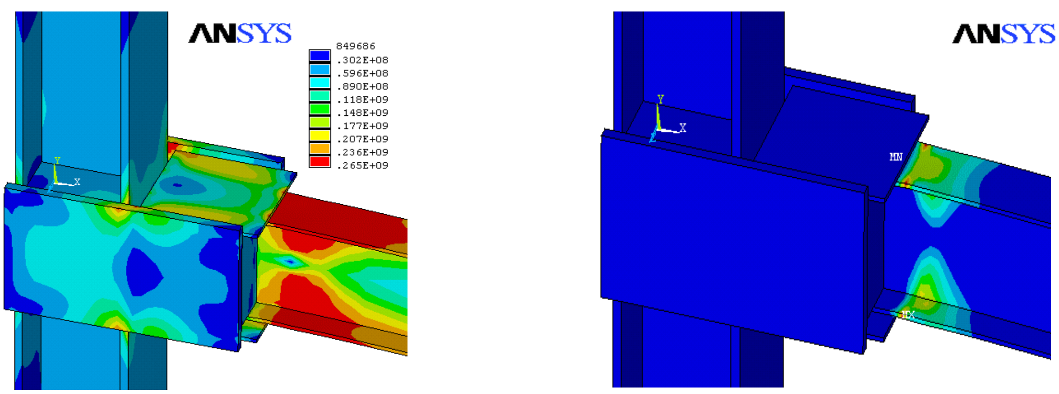
a. Stress distribution **b. Plastic deformation**

Figure 9. Model SPT 1-1a (sideplate thickness= 10 mm)



a. Stress distribution **b. Plastic deformation**

Figure 10. Model SPT 1-1b (sideplate thickness= 20 mm)



a. Stress distribution **b. Plastic deformation**

Figure 11. Model SPT 1-1c (sideplate thickness= 25 mm)

Figure 12 and 13 represent the diagrams of load -deflection (P-Δ) and moment - rotation (M-θ) for the models SPT1-1, SPT1-1a, SPT1-1b, and SPT1-1c. The only difference between these models is their sideplates thickness. It is interesting to note that the connection with 10 mm. sideplate thickness has less strength and stiffness compared to the other connections with thicker sideplates. But regarding plastic rotation, connection with 10 mm. thick sideplates behaves nearly the same as the other models except that it demonstrates a rotation approximately one half of the others. The connection with side plate of 15, 20, and 25 mm.display nearly the same strength and stiffness.

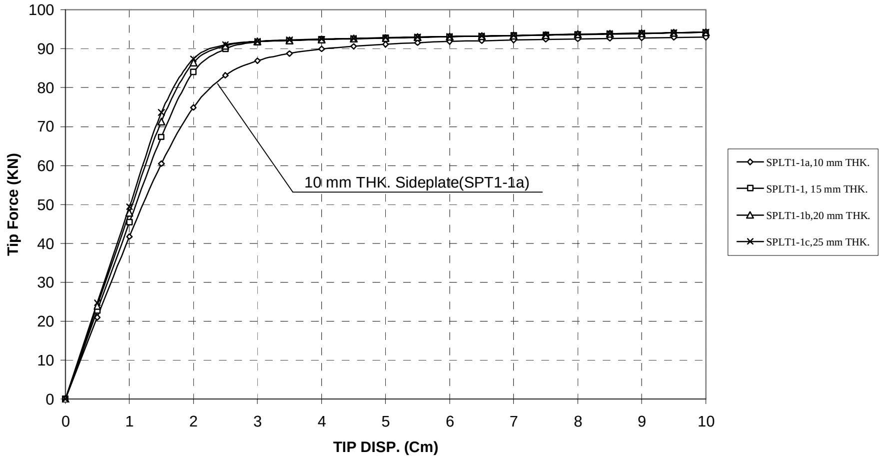


Figure 12. Load-Deflection Diagrams

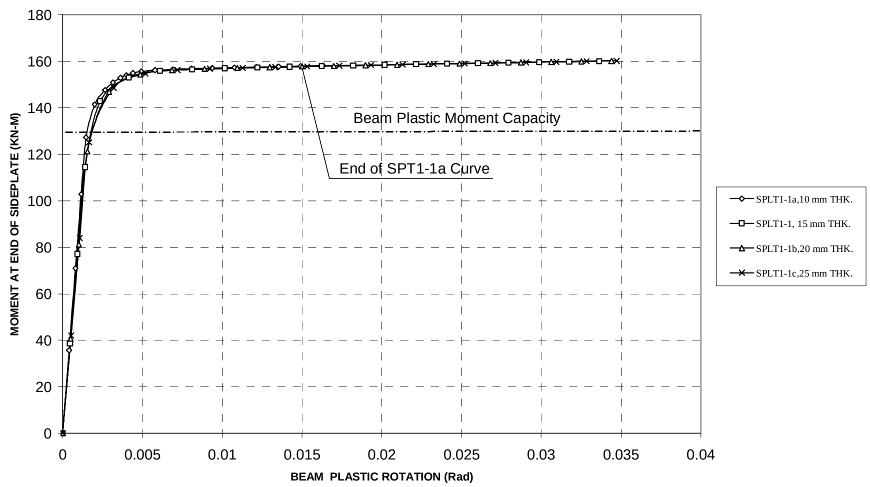


Figure 13. Moment-Rotation Diagrams

CONCLUSIONS

The important results obtained from this non-linear finite element analysis are summarized below:

- Plastic deformations take place in the beam instead of connection.
- Sideplate Connections have remarkable ductility and energy absorption capacity.

- Size of columns has negligible effect on the plastic behavior of connection.
- Sideplate thickness has negligible effect on the behavior of the connection.

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