

Seismic Analysis of Column Base Connections for Hollow Steel Sections

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ABSTRACT: Column base connections are the most important type of structural connections, transferring forces from the entire structure into the foundation. An analytical study was conducted to investigate the seismic response of exposed hollow steel section columns to base plate connections. A total of fifty-six FEA models were developed using ANSYS workbench and analysed. Parametric study includes different base plate thicknesses and anchor bolt sizes. Effects of axial force on connection behaviour were also included. All the specimens were subjected to lateral loading of cyclic nature. Current design approach prevalent in Steel Design Guide -1 does not address the effect of the middle row of rods; as a result, it cannot be used to design them. This study presents a refinement in a method proposed by Kanvinde [1] that explicitly incorporates the middle row of anchor rods, action of which is found to be very important under large lateral moments.

KEYWORDS: Column base plates, Seismic design, Steel connections; Anchor bolt tension, Finite element analysis.

I. INTRODUCTION

In steel structures, column base connections transfer forces from the steel column to the concrete footing. These connections include various components (i.e., column, base plate, anchor rods, and concrete foundation) schematically illustrated in fig.1[1] that interact under a variety of loading conditions such as axial tension/compression, flexure, and shear.

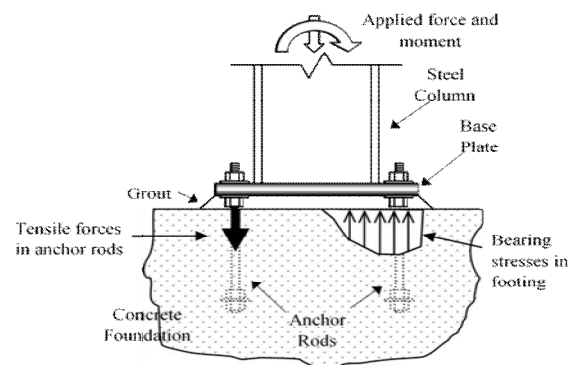


Fig 1: Schematic Illustration of Typical Exposed Base Plate Connection [1]

Column base connections are critical components in structures designed for seismic loading. In earthquake resistant structures, the column base also plays a very significant function from the standpoint of structural mechanics. This is because the earthquake ground motion is induced as an inertia force in the super structure through the column base, and the motion energy of super structure is simultaneously dispersed to soil through the column base. In this process, the column base is the passing point of forces; therefore, its failure will result in the collapse of structural framing. To

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prevent the disastrous collapse, the column base should be a moment carrying connection just like the beam to column connection. However, this research is still in the frontier stage. [3]

This study examines the response of hollow steel section (HSS) or box column base plate connections. Most of the previous studies are applicable to W-section columns and a connection configuration where four anchor rods are used; alternate configurations may be used with eight anchor rods. Only limited studies are available on alternate configurations. The strength characterization methods presented in prior research are applicable to a connection configuration where four anchor rods are used. This study is to examine and refine existing strength characterization methods for HSS column bases and those with alternate anchor rod layouts with the help of finite element analysis using ANSYS. A parametric study is also included.

II. LITERATURE REVIEW

A number of studies have been performed on the flexural response of exposed steel column base plate connections both experimental and analytical DeWolf and Sarisley [5] performed an experimental investigation to review the design practice for steel column base plates subjected to axial loads and moments. Two major conclusions were obtained from the study : (1) When the anchor rod strength is relatively large, the distance between the anchor rod on the tension side and the bearing zone on the compression side is small. (2) Increasing the base plate thickness can lead to decrease of the connection capacity due to large bearing stresses under the base plate and consequent premature concrete (grout) crushing. Drake and Elkin [11] suggested a design procedure that adopted an equivalent rectangular bearing stress block, instead of a triangular shape, for the application of the LRFD approach. Grauvillard et al. [10] A parametric study was conducted using the Drake and Elkin method for the design of exposed-type column-base plate connections bending about the weak axis, to investigate the effects of the relative strength ratio among the connection elements (i.e., column, base plate, and anchor rods) on the connection behaviour under large column lateral displacements. The Steel Design Guide One [2], published by AISC, provides guidance for the design of these connections. The method presented in Steel Design Guide One represents one of the most popular approaches to characterizing the strength of base connections worldwide. The axial-flexural strength design of base connections is based on the ultimate method proposed by Drake and Elkin. Kanvinde [1] Experiments suggests that the design approach in steel design guide I is reasonably conservative but does not address the effect of the third (i.e., central) row of anchor rods; as a result, it cannot be used to design them. A refinement is proposed to the current approach that explicitly incorporates the contribution of the inner row of anchor rods. Kanvinde et al. [7] presents a comprehensive FE simulation study which examines the physical response of exposed type connections, providing information about internal stress distributions. Several experimental and analytical studies have been carried out but, vast majority of experiments feature W-section columns subjected to major axis bending attached to base plates, and only limited one examines the response of hollow steel section (HSS) or box column base plate connections. The strength characterization methods presented in prior research (and most of the previous tests) are applicable to a connection configuration where four anchor rods are used. Often, alternate configurations may be used with eight anchor rods only limited test studies it. So it is important to study these areas in detail.

III. MODELLING OF SPECIMENS AND FINITE ELEMENT ANALYSIS USING ANSYS

There are two basic models .AutoCAD drawings of the two basic models are shown in Fig 2 and Fig 3, it shows all the geometrical dimensions adapted. The main parameters varying in the study are thickness of the base plate and the diameter of the anchor rod which are indicated as 't' and 'd' respectively in the Fig. 5.1 and Fig. 5.2.

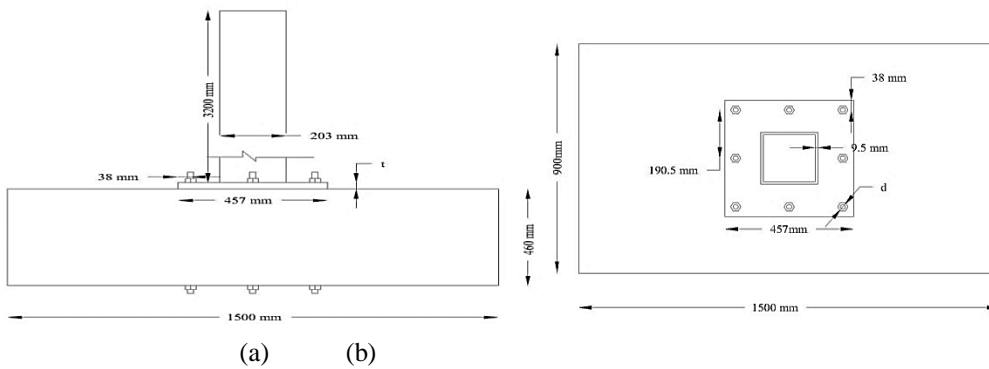


Fig 2: (a) Elevation of Base Model-1 (b) Plan of Base Model-1

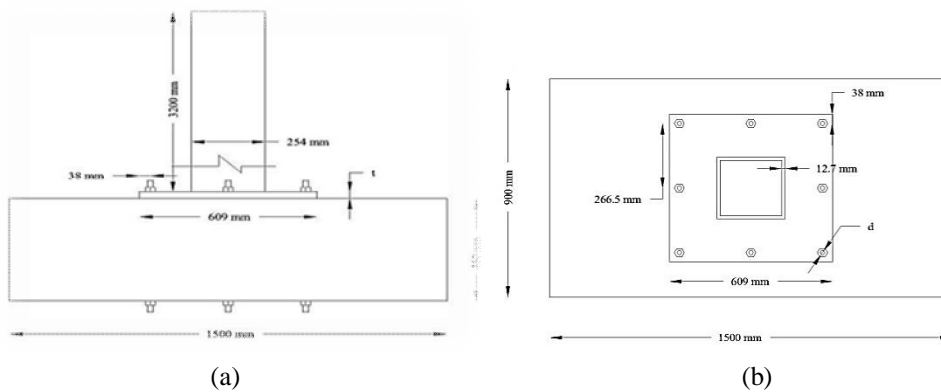


Fig 3: (a) Elevation of Base Model-2 (b) Plan of Base Model-2

Fig 4(a) shows the representative FE model constructed for the exposed base connection. In analysis, suitable numbers of elements were carefully chosen based on convergence studies in order to obtain accurate results without excessive use of computer time. The meshing is done using meshing tool and refined in the areas where strain or stress concentrations were anticipated Fig 4(b) shows the mesh adopted.

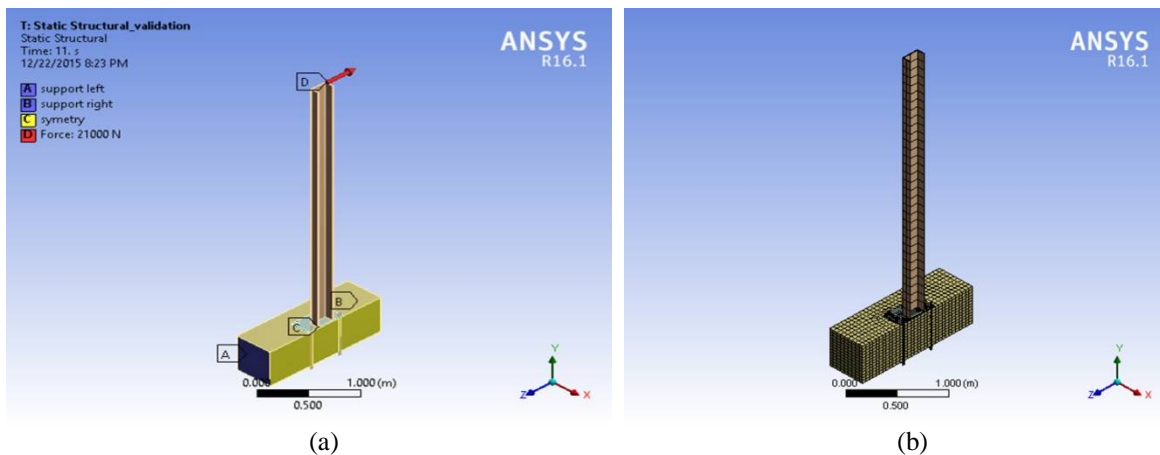


Fig 4: (a) Representative FE Model Constructed for the Exposed Base Connection (b) Model Meshing

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A. Component Interaction and Contact

Accurate modelling of component interaction is critical, since connection response is largely controlled by contact and gapping. The column and base plate were simulated as monolithic and the contact was given as bonded. The interaction between the rod and the nut was also given as bonded. The coefficient of friction used (i.e. 0.45) for the steel base plate and concrete interaction was obtained from previous friction testing by the lead author Gomez et al. [9]. The other contacts were given as no separation.

B. Model Specifications and Material Properties

Anchor rods used for models were F1554 Grade 105 ($f_y=861\text{N/mm}^2$) and poisson's ratio was taken as 0.3. The column was HSS203×203×9.5 of height 3200mm nominally A500 Grade B ($f_y=317\text{N/mm}^2$), whereas the plates were nominally A36 (i.e., $f_y= 248\text{N/mm}^2$) poisson's ratio for column and base plate were taken as 0.3 and 0.26 respectively (Kanvinde et al. [7]). The concrete footing was 1500×900×460mm 28th day cube strength as per Kanvinde [1] that is 35 N/mm² was used and poisson's ratio was taken as 0.2.

C. Application of Loads and Boundary Conditions

Three cycles of reverse cyclic loading was applied with maximum peak load of 21kN. The load corresponding to the maximum base moment as per Kanvinde [1] was calculated and was taken as the maximum peak load. Both the right and left ends of footing were provided with fixed boundary conditions. Top of column was left free.

D. Models Used in Study

A total of 56 models were used in the study. Plate thicknesses used in the study were P2-19mm, P3-25.4 mm, P4 - 31.75mm and P5- 38.1mm. Rod diameter used for parametric study include 19mm, 25.4mm, 31.75mm and 38.1mm. These parameters were studied on two basic models with HSS 203×203×9.5mm and base plate size 457×457 mm (HSS1) and The second base model with column HSS 254×254×12.7mm and base plate size 609×609mm (HSS2). For studying the relationship between the total tensile forces in the middle and outer row of rods some additional set of rod diameters (22.09mm, 29.9mm, 41.9mm ,33.02mm, 36.06mm ,50.8mm.) were used.

IV. RESULT AND DISCUSSIONS

A. Deformation of Column

Deformation of the column reduces with increase in plate thickness for both HSS1 and HSS2 models as shown in the Fig. 5. This is because column deformation mainly depends on the deformation of the base plate, which in turn depends on the stiffness of the plate. Lesser the thickness of the plate less will be its stiffness, and hence more will be the deformation of the plate and thus the column. The maximum deformation is observed for the model with least rod diameter and thin base plate. This implies that along with deformation of plate, rod deformation also has a vital role in deciding maximum deformation of column. So in order to control the maximum lateral displacement of the column it is important to design the base plate thickness and rod diameter with utmost care.

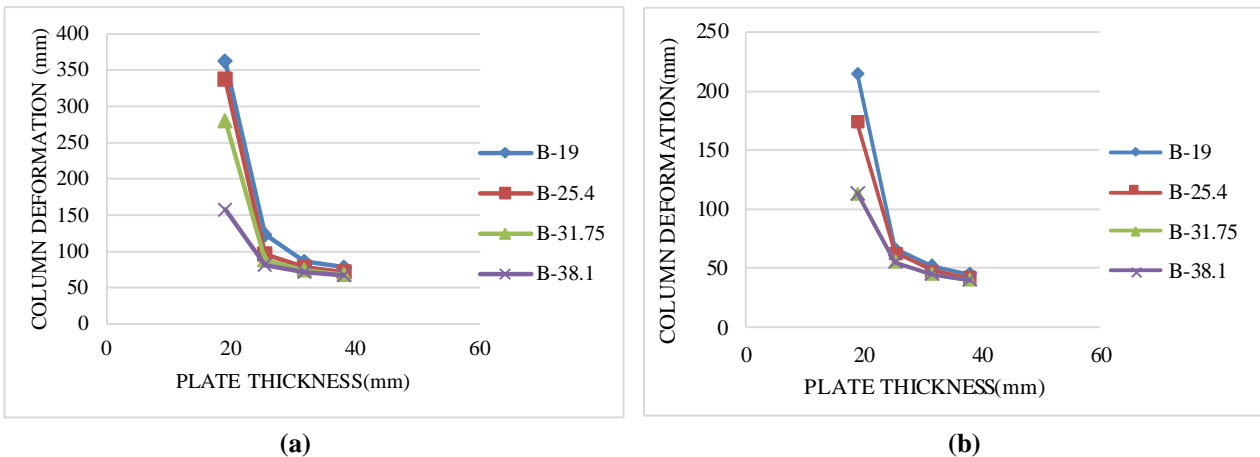


Fig5: Deformation of Column vs. Plate Thickness at Different Rod Diameter (a)HSS1 Models (b) HSS2 Models

B. Stress in Column

In all cases maximum column stresses are observed at the foot of the column. The maximum stress is found to decrease with the increase in plate thickness. This is because as the plate thickness increases deformation of plate decreases which in turn reducing stress at the bottom of the column. Variation of stress is almost similar for both HSS1 and HSS2 models. As expected HSS1 models have slightly more stress than HSS2 models as deformations are more for HSS1 models. Fig 6 shows the maximum stress in column vs. plate thickness at different rod diameter

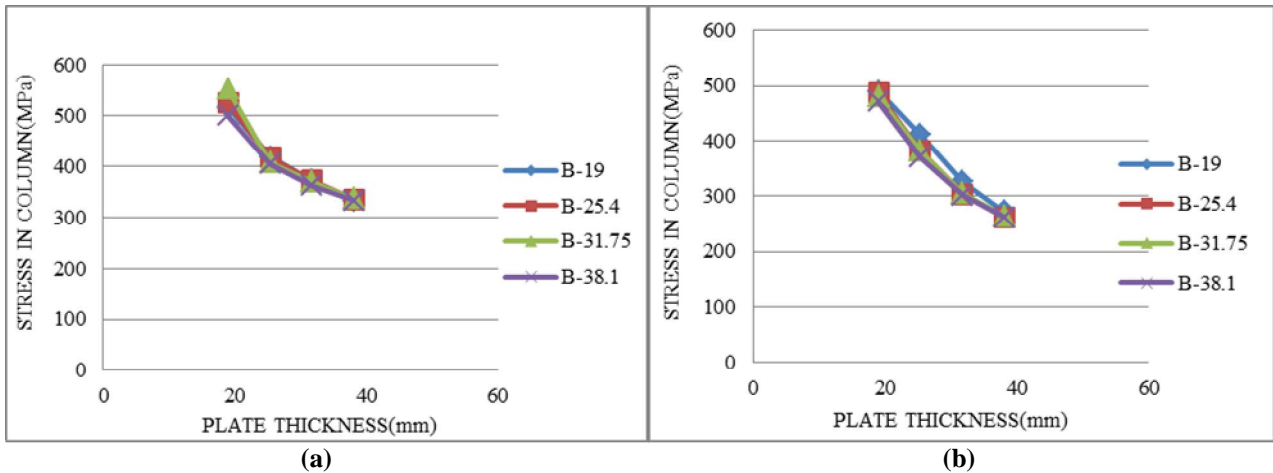


Fig 6: Maximum Stress in Column vs. Plate Thickness at Different Rod Diameter (a) HSS1 Models (b)HSS2 Models

C. Deformation of Plate

From the fig 7 it is clear that the maximum vertical displacement of the plate is for thin base plate with least rod diameter and for a particular rod diameter the deformation of plate decreases with increase in plate thickness.

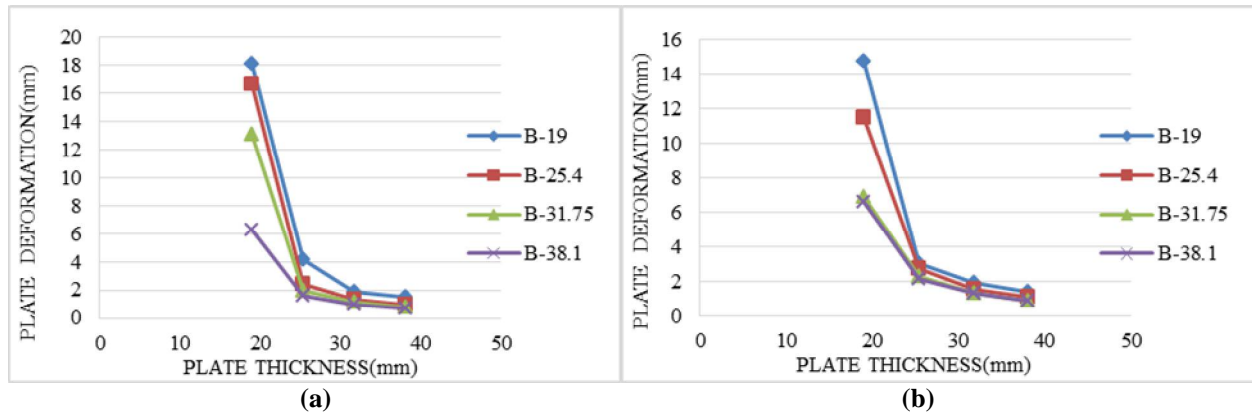


Fig 7: Deformation of Plate vs. Plate Thickness at Different Rod Diameter (a) HSS1 Models (b) HSS2 Models

The fig 8 shows the deformation of two plates, one with thickness 19mm and other with thickness 38.1mm both with same rod diameter 19mm and same applied lateral load, a visible difference in the deformation pattern of plates can be noticed. A convex shaped deformation is observed in between the rods on tension side and middle row of rods in case on 19mm thick plate whereas the deformation pattern is entirely different for 38.1mm rod. This is mostly due to the existence of constraints i.e., the anchor rods under large lateral forces the anchor rods on the tension side and the middle rods will resist the upward movement of the plate at middle and tension side but in between the thin plates as they are less stiff, yield early and deform in a convex manner. The 19 mm thick base plate deform outwardly on the tension side under large column lateral displacements, whereas the 38.1mm base plates deform inwardly. The outward deformation of the thin base plates can cause local anchor bolt deformation under the nut and thus result in high stress concentration in this region. This can be clearly seen from the deformed shape of the anchor rods and the stress contours presented in Fig 9. Hence, in order to prevent the possible anchor bolt failure due to such high stress concen-

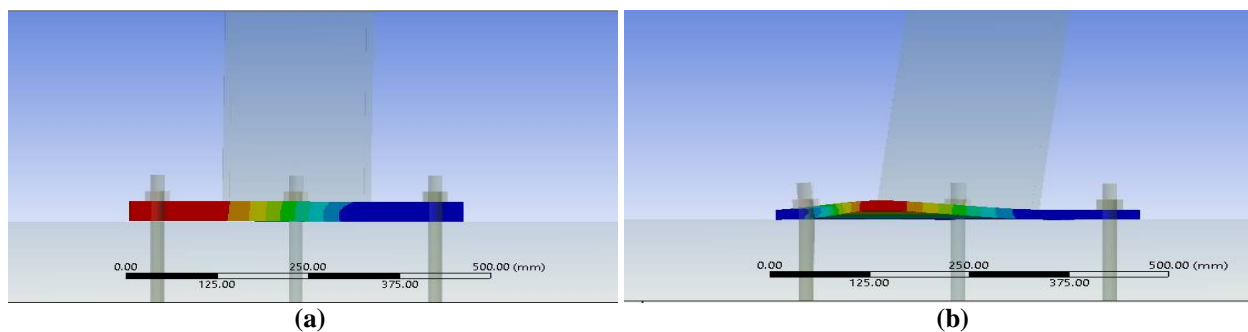


Fig 8: Deformation of Plate (a) HSS1_P2_19 (b) HSS1_P5_19

-tration, the minimum (19mm) thickness of the base plate should be provided when exposed-type column base plate connections are designed [4]. Thinner base plates could be a good source of energy dissipation and show more ductile connection behaviour under cyclic loading like severe earthquake excitation [8]. However, it must be noted that thinner base plates can cause high local stress concentration in the anchor rods.

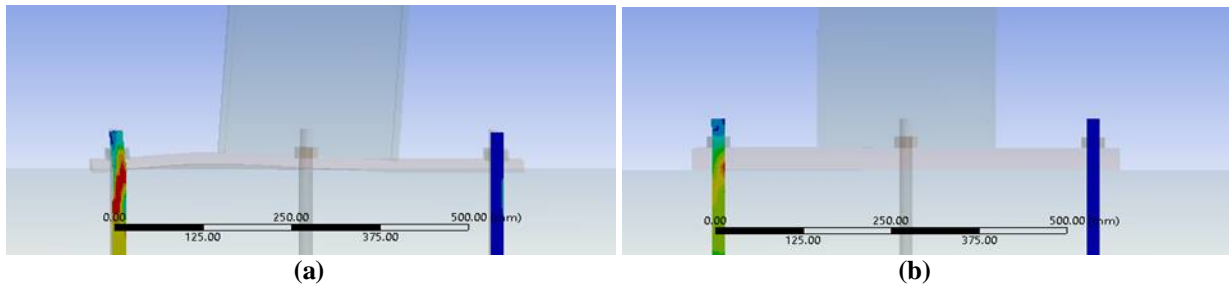


Fig 9: Maximum Stress on Tension Rod (a) HSS2_P2_19 (b) HSS2_P5_19

D. Stress Distribution in Base Plate

Fig 10 shows the stress distribution HSS1 and HSS2 models for P4_B2 combination. It can be noticed that maximum stresses are along the straight lines on both sides of the base plate (i.e., tension and compression sides) parallel to the HSS column edges as in [1]. This pattern is same for both HSS1 and HSS2 column models except for thin plates as thinner plate yield more for the same load. As there is constraints at the left middle and right sides of the plate this pattern is as expected. The Fig 10(c) shows the stress distribution on plate for P2 thickness. Even though stress is more on region between middle and the outer rods, stress is distributed over a large area.

Lower connection strength and rotational stiffness are commonly observed in case of the flexible base plate. These strength and stiffness reductions are due to early yielding in the base plate. The thicker base plate, on the other hand, do not show the early baseplate yielding. A stiff base plate usually forces major yields in other connection elements such as column and anchor bolts. Thinner base plates may result in strength and stiffness reductions in exposed-type column-base plate connections.

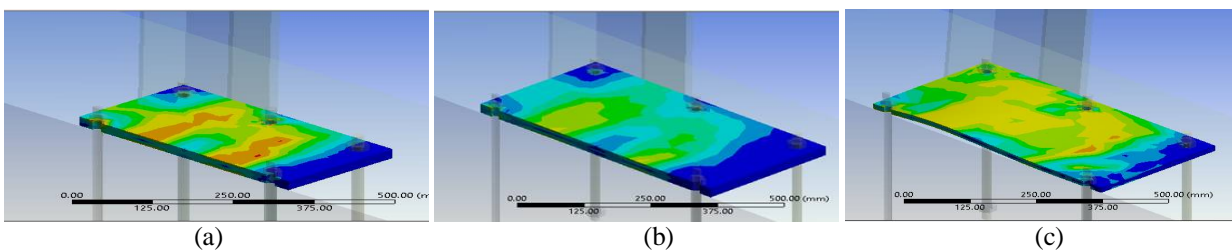


Fig 10: Stress Distribution of Plate for (a) HSS1_P4_19 (b) HSS2_P4_19 (c) HSS2_P2_19

E. Deformation of Rods

Deformation of rod decreases with increase in the plate thickness and the maximum deformation is noticed for least diameter rod. This is expected as thinner rods are more prone to large deformations as it is less stiff. Fig.11 shows the maximum rod deformation vs. plate thickness at different rod diameter

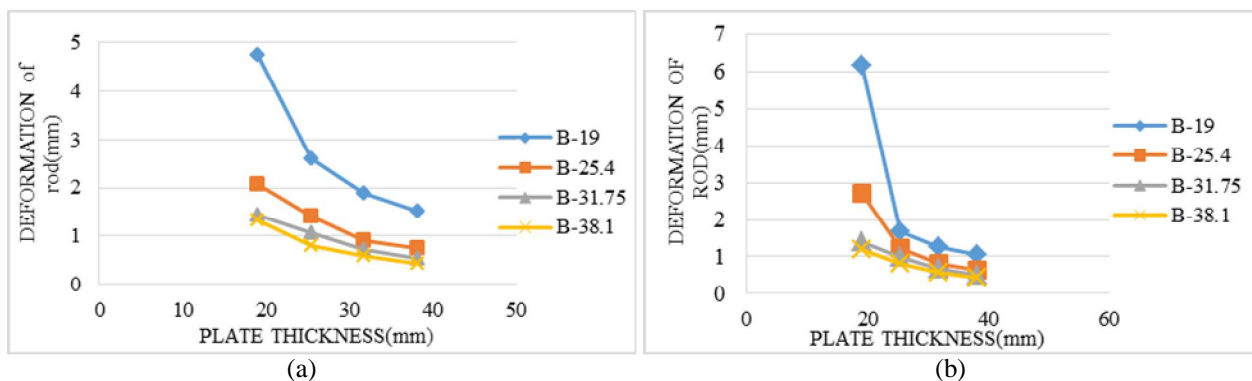


Fig.11: Maximum Rod Deformation vs. Plate Thickness At Different Rod Diameter (a) HSSI Models. (b)HSS2 Models.

F. Tensile Force on Rods

Fig. 12 shows the total tensile force distribution on right (R), middle (M) and left (L) row of rods with reverse cyclic loading. The maximum variation of the base plate thickness is larger than 0.635cm. It should be noted that this amount

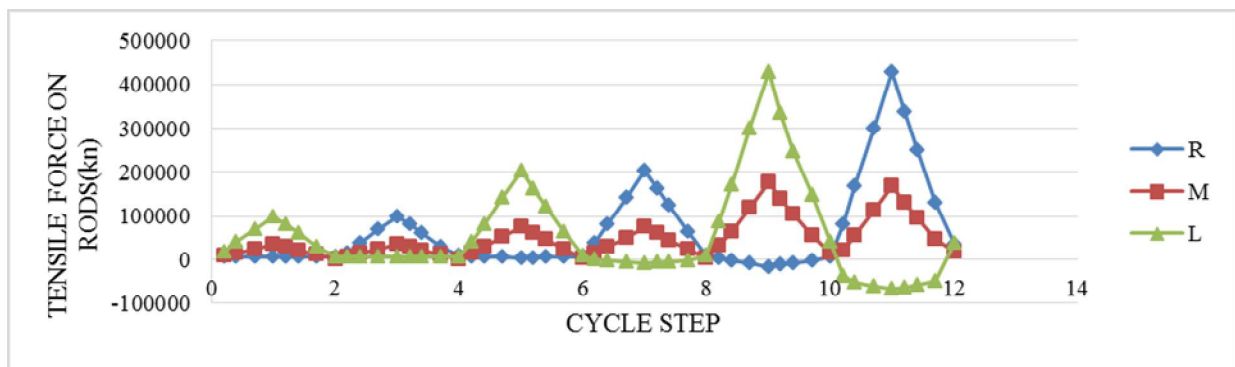


Fig. 12: Distribution of Tensile Force on Rods HSS2_P5_19

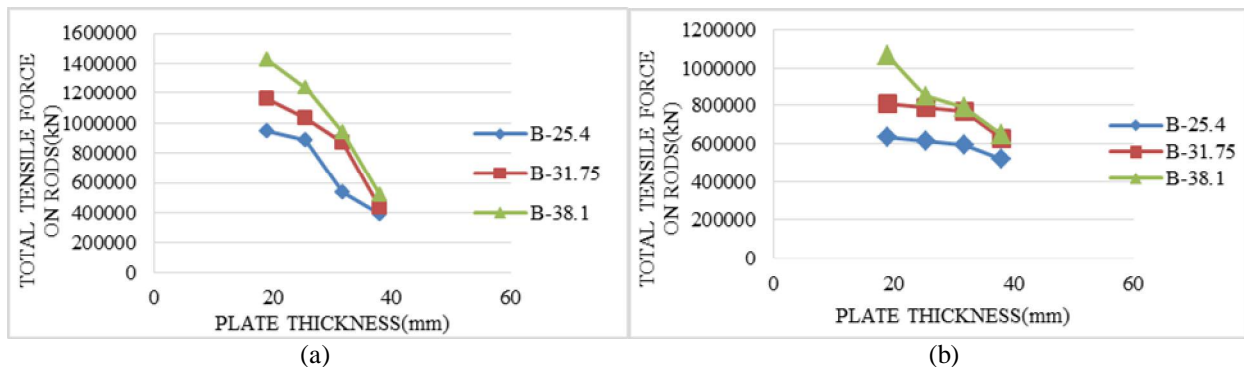


Fig. 13: Total Tensile Force on Rods vs. Plate Thickness for Different Rod Diameter (a) HSS1 (b) HSS2

of the variation in base plate thickness and the variation in $T_u (=R_u = qY$ in this study) may be sufficient to change seismic performance of the connection. Fig 13 shows Total Tensile Force on Rods vs. Plate Thickness for Different Rod Diameter. The Amount of the total tensile bolt force (T_u) can be affected by flexibility of the base plate for a given connection geometry. This is because location of the resultant bearing force (R_u) is highly dependent on deformed shape of the base plate. In case of the thick base plate, for instance, the decrease in T_u is due to lengthening of the moment arm between T_u and R_u as schematically presented in Fig. 14 In order to resist the same design moment (M_u) transferred from the column to the connection, the amount of T_u (and R_u) must decrease with longer moment arm.

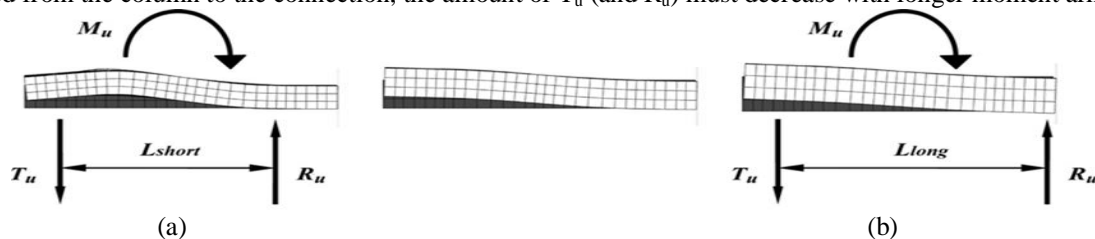


Fig. 14 Deformed Shape and Bearing Resultant in (a) P2 and (b) P5 Models [4]

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G. Effect of Axial Force

Fig15(a) shows the variation of column stresses vs. axial force, it is found that as the axial force increases the stress on the foot of column also increases. The stress distribution on column is as shown in the Fig 15(b). Significant stresses are developed mainly at the area of the foot of the column and there is a chance for development of plastic areas to create a plastic hinge at the lower part of the steel column. Thus, the collapse of the structure may occur due to the plastification of the steel column and the other parts of the connection are not critical.

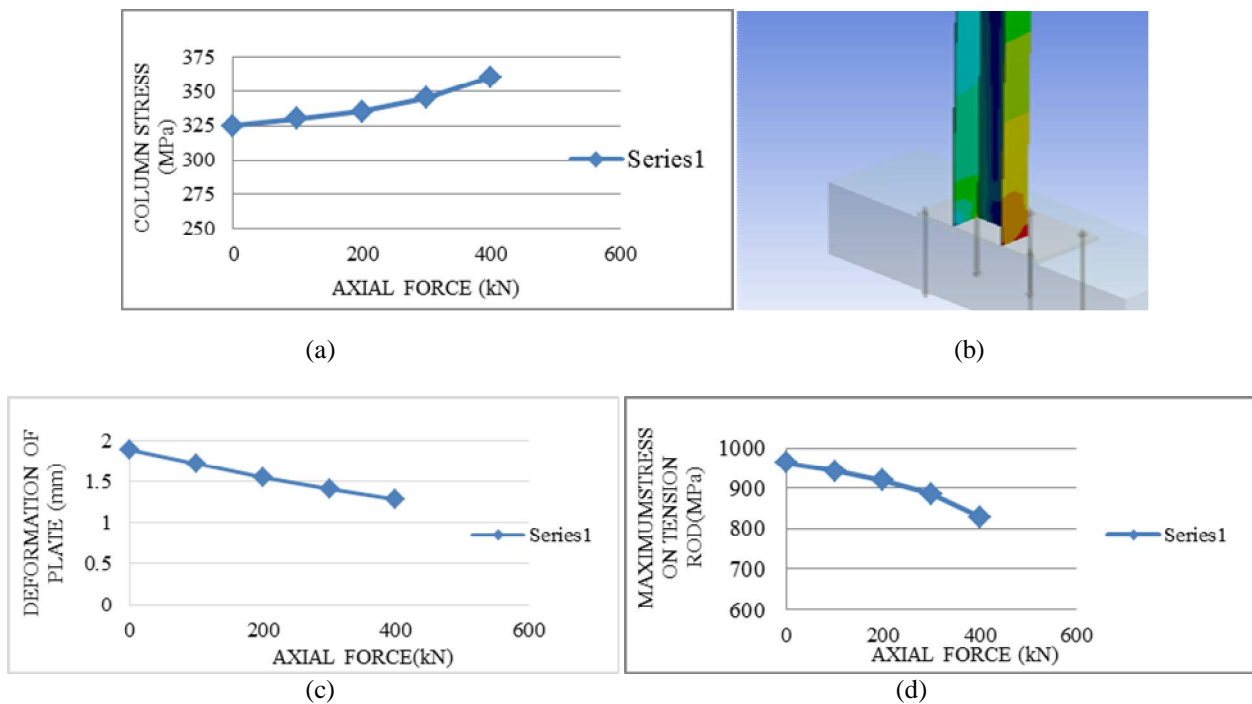


Fig15: (a) Maximum Column Stress at Foot of Column vs. Axial Force with 400 kN Axial Load for HSS2_P4_B2 (b) The Stress Distribution on Column for HSS2_P4_B2 (c) Maximum Deformation of Plate vs. Axial Force for HSS2_P4_B2 (d) Stress on Tension Rod vs. Axial Force for HSS2_P4_B2

Area of contact between the base plate and the concrete increases with increase in axial force and this action tend to reduce the deformation of the base plate as shown in Fig 15 (c). The Fig. 15 (d) shows the variation in the stress on the tension rods vs. axial force. The stress concentration on tension rods with the increase axial load is found to be decreasing and this can be attributed to decrease in deformation of plates with increase in axial force. The ratio of tensile force in the middle rod to the outer rod is found to decrease with the increase in axial load (Fig. 6.38) and this is expected as the deformation of plate decreases the action of the middle rod (or inner rod) will tend to reduce. This variation can be seen in the graph plotted (Fig 16)

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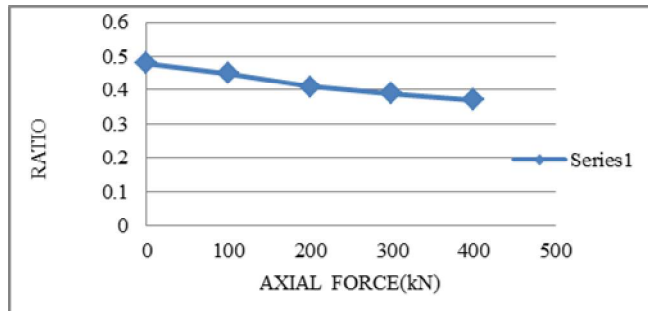


Fig 16: The Ratio of Tensile Force in The Middle Rod to The Outer Rod vs. Axial Force for HSS2_P4_B2

V. REFINED METHOD

As per the approach presented in the Steel Design Guide One, the base connection resists the applied axial compression and moment through the mechanism illustrated in Fig 1.2. The forces in the anchor rods T^{DG-1} and the bearing length Y^{DG-1} may be calculated using Equations (1.1) and (1.2) in what follows, also presented in Steel Design Guide One:

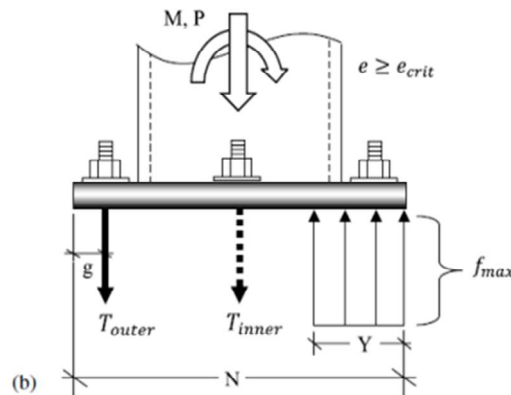


Fig 17: Assumed Stress Distributions for Base Connection Design [1]

$$Y^{DG-1} = (N - g)^2 - \sqrt{(N - g)^2 - \frac{2 \left[M + P \left(\frac{N}{2} - g \right) \right]}{f_{max}^{DG-1}}} \quad (1.1)$$

$$T^{DG-1} = f_{max}^{DG-1} \cdot B \left\{ (N - g) - \frac{2 \left[M + P \left(\frac{N}{2} - g \right) \right]}{f_{max}^{DG-1}} \right\} \quad (1.2)$$

$$f_{max}^{DG-1} = 0.85 \times f_c' \times \sqrt{\frac{A_2}{A_1}} \quad (1.3)$$

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Where, Y^{DG-1} - Bearing width per Steel Design Guide One, N -Length of base plate, g -Rod edge distance, M - Bending moment at base of column, P - Axial compression in column, f_{max}^{DG-1} - Maximum bearing stress per Steel Design Guide One, T^{DG-1} - Tension force in outer row of rods per Steel Design Guide One, f_c - Crushing strength of concrete, A_1 -Bearing area of plate, A_2 -Plan area of foundation.

The equations are derived by establishing vertical and rotational equilibrium on the connection. In the context of this study, it is useful to note two points. First, T^{DG-1} represents the total tension force in all anchor rods on the tension side; if only two are present, then the sharing of load between them may be assumed to be equal, not necessarily the case if alternate rod layouts are used. Second, the methods outlined in Steel Design Guide One do not address the effect of an additional set of anchor rods. As a result, the force T^{DG-1} is an estimate of T_{outer} (total tensile force on outer row of rods) shown in the fig. 17 Note that the addition of the inner rods introduces indeterminacy in the system such that the internal forces may no longer be inferred based only on the force and moment equilibrium of the connection. Consideration of additional rows of rods requires fundamental changes to the method to address the indeterminacy produced by the additional rods; even for connections similar to those implied in the design methods, i.e., those with two rows of rods, the distribution of forces among these rods is not addressed by the current methods; and the internal stress and force distributions are not based on independent measurements of rod force but rather the overall connection response. Referring to Fig. 17 as per Kanvinde [1] the applied force and moment may be resisted through the development of a bearing stress block on the compression side and forces in the anchor rods on the tension side of the connection. For large lateral loads the anchor rods are engaged. The following relationships may be established based on equilibrium of the entire connection:

$$P + T_{outer} + T_{middle} \text{ (or } T_{inner} \text{)} = B \times Y^{new} \times f_{max}^{DG-1} \quad (1.4)$$

$$T_{outer} \times \left(\frac{N}{2} - g \right) + B \times Y^{new} \times f_{max}^{DG-1} \times \left(\frac{N}{2} - \frac{Y^{new}}{2} \right) = M \quad (1.5)$$

Where, T_{outer} -Tension Forces in Outer Row of Rods, T_{middle} - Tension Forces in Middle or Inner Row of Rods, P - Axial Compression in Column, B - Width of Base Plate, Y^{new} - Bearing Width as Per New Method, N - Length of Base Plate f_{max}^{DG-1} - Maximum Bearing Stress as Per Steel Design Guide One, M - Bending moment at the base of the column.

Eq. (1.4) establishes the vertical force equilibrium, whereas Eq. (1.5) establishes the moment equilibrium. The two preceding equations contain three unknowns (i.e., T_{outer} , T_{middle} (or T_{inner}) and Y^{new}) and thus cannot be solved unless an additional condition is introduced. It is proposed that the tension in the middle rods should be considered as being directly proportional to the tension in the outer rods, i.e., T_{middle} (or T_{inner}) = $k \times T_{outer}$ [1]. This assumption is based on a general kinematic consideration by which the deformations in the two rows of rods are constrained by the plate. However, it is also recognized that the plate is flexible and that the plate or rods will likely yield as loading progresses, such that this direct proportion may no longer be valid. According to Kanvinde[1] $K=0.5$. This study points out to some refinement in values of k . For the case if it considered $P = 0$; substituting this into Eqs. (1.4) and (1.5) and solving simultaneously results in

$$T_{outer} = \frac{N}{N - \frac{g}{2} (1 + k)} \quad (1.6)$$

$$Y^{new} = \frac{T_{outer} \times (1 + k)}{B \times f_{max}^{DG-1}} \quad (1.7)$$

Once T_{outer} is determined, the force in the inner row of rods may be determined as T_{middle} (or T_{inner}) = $k \times T_{outer}$. Note that for this study, measurements of T_{outer} and T_{middle} (or T_{inner}) are available from analysis. Based on these

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measurements, a value of k is determined to achieve the best overall agreement with force measurements from all analysis. k values for all thirty-two models used for parametric study are given in Table 1. Now in order to generalize the result, results of some additional models are given in table. 2. By taking the average ratio for each plate thickness it can be seen that an average ratio of 0.65 can be taken for baseplate with thickness less than or equal to 19mm, for plates greater 19mm to 25.4mm it can be taken as 0.6. A ratio 0.55 can be taken for plate thickness in between 25.4mm and 31.75 mm and for 38.1 mm plate take 0.45 as the ratio. It is desirable keep the ratio equal to 0.4 for thickness greater than 38.1mm. This is valid for rod diameters ranging from 19mm to 50.8mm. As this ratio is found to reduce with the application of axial load, reduce the ratio obtained by .01 for every 100kN force for design.

Table 1: Ratio of Tensile Force Middle Rod to Outer-1

Model	k-value	Model	k-value
HSS1_P2_19	0.64	HSS2_P2_19	0.72
HSS1_P3_19	0.58	HSS2_P3_19	0.63
HSS1_P4_19	0.5	HSS2_P4_19	0.51
HSS1_P5_19	0.4	HSS2_P5_19	0.43
HSS1_P2_25.4	0.67	HSS2_P2_25.4	0.7
HSS1_P3_25.4	0.54	HSS2_P3_25.4	0.58
HSS1_P4_25.4	0.49	HSS2_P4_25.4	0.48
HSS1_P5_25.4	0.4	HSS2_P5_25.4	0.41
HSS1_P2_31.75	0.65	HSS2_P2_31.75	0.72
HSS1_P3_31.75	0.55	HSS2_P3_31.75	0.63
HSS1_P4_31.75	0.52	HSS2_P4_31.75	0.49
HSS1_P5_31.75	0.44	HSS2_P5_31.75	0.42
HSS1_P2_38.1	0.6	HSS2_P2_38.1	0.73
HSS1_P3_38.1	0.55	HSS2_P3_38.1	0.67
HSS1_P4_38.1	0.56	HSS2_P4_38.1	0.6
HSS1_P5_38.1	0.4	HSS2_P5_38.1	0.44

Table 2: Ratio of Tensile Force Middle Rod to Outer-2

Model	k-value	Model	k-value
HSS2_P2_22.09	0.65	HSS2_P2_33.02	0.69
HSS2_P3_22.09	0.56	HSS2_P3_33.02	0.59
HSS2_P4_22.09	0.48	HSS2_P4_33.02	0.52
HSS2_P5_22.09	0.4	HSS2_P5_33.02	0.41
HSS2_P2_29.9	0.7	HSS2_P2_36.06	0.75
HSS2_P3_29.9	0.63	HSS2_P3_36.06	0.67
HSS2_P4_29.9	0.48	HSS2_P4_36.06	0.62
HSS2_P5_29.9	0.44	HSS2_P5_36.06	0.53
HSS2_P2_41.9	0.78	HSS2_P2_50.8	0.7
HSS2_P3_41.9	0.67	HSS2_P3_50.8	0.61
HSS2_P4_41.9	0.59	HSS2_P4_50.8	0.59
HSS2_P5_41.9	0.52	HSS2_P5_50.8	0.51

Table 3: Average k-value

Plate Thickness	k -Value
$t \leq 19\text{mm}$	0.65
$19\text{mm} < t \leq 25.4\text{mm}$	0.6
$25.4\text{mm} < t \leq 31.75\text{mm}$	0.55
$31.75\text{mm} < t \leq 38.1\text{mm}$	0.45
$t > 38.1\text{mm}$	0.4

VI. CONCLUSION

Based on the results of the study the following conclusions are made.

- The deformation of column, base plate and anchor rod reduces with an increase in plate thickness.
- Thinner base plates and stiffer anchor bolts increase the amount of resultant bearing force R_u due to the shortened overall moment arm between the total tensile bolt force (T_u) and R_u .

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- A thinner base plate can cause high local stress concentration in the anchor bolts on the tension side due to outward base plate deformation.
- As the axial force increases the stress on the foot of column also increases. Significant stresses are developed mainly at the area of the foot of the column and there is a chance for development of plastic areas to create a plastic hinge at the lower part of the steel column.
- The ratio of tensile force in the middle row of rods to the outer row rods is found to decrease with the increase in axial load and this is expected as the deformation of plate decreases the action of the middle rod will tend to reduce.
- The ratio of maximum tensile force on middle row of rods to outer row of rods for thinner baseplate is found to be more when compared to that of thicker base plate. This can be attributed to the deformed shape of the plate
- As this ratio is found to decrease with increase in axial load, reduce the ratio obtained by 0.01 for every 100kN force for design.

REFERENCES

- [1] Kanvinde, A. M., Higgins, P., Cooke, R. J., Perezand, J., Higgins, J., "Column Base Connections for Hollow Steel Sections: Seismic Performance and Strength Models" J. Struct Eng, ASCE, ISSN 0733-9445/04014171,2014.
- [2] Fisher, J. M. and Kloiber L. A., "Steel design guide 1—Base plate and anchor rod design." Steel Design Guide Series, 2nd Ed., AISC 801-06, Chicago,2006.
- [3] Sato K., "A Research on the Aseismic Behaviour of Steel Column Base for Evaluating Its Strength Capacity and Fixity," Report No. 69, Kajima Institute of Construction Technology, Tokyo, Japan 1987.
- [4] Dae-Yong Lee, Subhash C. Goel and Bozidar Stojadinovic., "Exposed column-base plate connections bending about weak axis: Numerical parametric study "Steel Structures 8 11-27, 2008.
- [5] DeWolf, J. T., and Sarisley, E. F., "Column base plates with axial loads and moments." J. Struct. Div., 106(11), 2167–2184, 1980.
- [6] Thambiratnam, D. P., and Paramasivam, P., "Base plates under axial loads and moments." J. Struct. Eng., 10.1061/ (ASCE) 0733-9445,112:5(1166), 1166–1181,1986.
- [7] Kanvinde, A. M., Jordan, S. J., and Cooke R. J., "Exposed column baseplate connections in moment frames—Simulations and behavioural insights." J. Constr. Steel Res., 84, 82–93,2013.
- [8] Wald, F., Sokol Z., and Baniotopoulos C., "Column Base Connections" Journal of Constructional Steel Research.,1999.
- [9] Gomez, I., Kanvinde, A. M., and Deierlein G., "Exposed column base connections subjected to axial compression and flexure." Final Rep., American Institute of Steel Construction, 2010.
- [10] Grauvilardell, J. E., Lee, D., Hajjar, J. F., and Dexter R. J., "Synthesis of design, testing and analysis research on steel column base plate connections in high seismic zones." Structural Engineering Rep. No. ST-04-02, Dept. of Civil Engineering, Univ. of Minnesota, Minneapolis, 2005.
- [11] Drake, R. M., and Elkin S. J., "Beam-column base plate design—LRFD method." Eng. J. AISC, 36(1), 29–38, 1999..

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