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Exposed Column-Base Plate Connections Bending About Weak Axis: I. Numerical Parametric Study

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Abstract

An extensive numerical parametric study was conducted to evaluate the Drake and Elkin's method for the design of exposed-type column-base plate connections bending about weak axis and to investigate effects of the relative strength ratio among the connection elements (i.e., column, base plate, and anchor bolts) on the connection behavior under large column lateral displacements. For this numerical study, a Finite Element Analysis (FEA) model which could effectively simulate force transfer at major contact interfaces in the connection was developed and a total of 43 three-dimensional FEA meshes that have different base plate thicknesses, anchor bolt sizes (stiffnesses), and grout compressive strengths were configured and analyzed. The study revealed several possible limitations of the D&E method in its basic assumptions and subsequent design calculations and concurrently, pointed to several important design considerations needed to develop rational and reliable design methods. Major findings from the numerical parametric study are mainly discussed in this paper.

Keywords: Connection, Base Plate, Anchor Bolt, Grout, Weak Axis, Relative Strength Ratio

1. Introduction

Exposed-type column-base plate connection is one of the most important structural elements in a steel structure, especially in a steel Moment Resisting Frame (MRF). Base plate in a MRF connects the column to the concrete foundation directly so that lateral forces due to wind or seismic effects can be transferred through the base plate and anchor bolts to the grout and concrete foundation. Despite the significant role of the column-base plate connection in seismic performance of the steel MRFs, there has been no unified seismic design provisions for this connection in the U.S. Recently, the 2005 AISC Seismic Provisions (AISC, 2005) briefly addressed seismic design criteria for the column bases in Section 8.5 and its Commentary (Section C8.5). Several recommendations for the column-base plate connection design for axial column tension (or uplift), shear, or moment are also newly added in Part 14 of the 2005 AISC Manual of Steel Construction (AISC, 2005). However, these generalized provisions and recommendations have not strongly impacted the current design practices for the column bases especially in high seismic region due to lack of detailed practical information.

In the U.S., most exposed-type column-base plate connections resisting large column moments have been designed by referring to the earlier publications, such as Gaylord and Gaylord (1957 and 1972), Salmon *et al.* (1957), Blodgett (1966), Soifer (1966), McGuire (1968), Maitra (1978), DeWolf and Sarisley (1980), DeWolf (1982), Ballio and Mazzolani (1983), and Thambiratnam and Paramasivam (1986), or by using the AISC Design Guide No. 1, *Column Base Plates* (DeWolf and Ricker, 1990). Unfortunately, however, many column-base plate connections designed following the above references did not perform satisfactorily during the Northridge earthquake of January 17, 1994. Extensive damage to the column-base plate connection has been reported since this earthquake (Technical Council on Lifeline Earthquake Engineering, 1995, and Northridge Reconnaissance Team, 1996). The damage in the connection mostly consisted of brittle base plate fracture, excessive anchor bolt elongation, unexpected early anchor bolt failure, and concrete crushing (including grout crushing). Similar damage types in the column-base plate connections have also been reported in Japan since the 1995 Hyogo-ken Nanbu (Kobe) earthquake. Based on statistical investigation of the damage in steel structures compiled after the Kobe earthquake, Midorikawa *et al.* (1997) noted relatively high incidences of damage in the column-base plate connections. The lessons from the above observations have led to the need for improved understanding of the column-base plate connection behavior under large column lateral displacements and the need for development of more reliable design methods

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and more ductile connection details.

In order to understand complex force flow and stress distribution in the column-base plate connection subjected to large column moments, only a few analytical and experimental studies have been conducted in the U.S., including Salmon *et al.* (1957), Maitra (1978), DeWolf and Sarisley (1980), DeWolf (1982), Astaneh *et al.* (1992), Astaneh and Bergsma (1993), and Burda and Itani (1999). Recently, Drake and Elkin (1999) suggested a design procedure based on an LRFD method. They adopted an equivalent rectangular bearing stress block under the base plate, instead of the triangular shape that is assumed in the AISC Design Guide No. 1 (DeWolf and Ricker, 1990). Four different loading conditions and corresponding bearing stress distributions were considered in their design method, and a design procedure was provided for each case. This design method, however, has not been verified enough analytically or experimentally for either the strong or the weak axis bending case.

An analytical and experimental study was thus conducted at the University of Michigan to evaluate the Drake and Elkin's design method (referred to hereafter as the D&E method) and, if available, to develop more rational and reliable design methods. As part of this research, Fahmy (1999) studied the case of strong axis bending. For the weak axis bending case, analytical and experimental studies were undertaken by the authors (2001, 2002, and 2008). Two typical column-base plate connections in steel MRFs (i.e., exposed-type unstiffened 6-bolt and 4-bolt connections) were chosen for these studies. It should be noted that the new regulations of the Occupational Safety and Health Administration (OSHA) - Safety Standards for Steel Erection (OSHA, 2001), effective on January 18, 2002, require a minimum of 4 anchor bolts in the column-base plate connections.

An extensive numerical parametric study was primarily conducted to evaluate the D&E method numerically and to investigate effects of the relative strength ratio among the connection elements (i.e., column, base plate, and anchor bolt) on the connection behavior under large column lateral displacements. For this numerical study, a Finite Element Analysis (FEA) model which could effectively simulate force transfer at major contact interfaces in the connection was developed and a total of 43 three-dimensional FEA meshes that have different base plate thicknesses, anchor bolt sizes (stiffnesses), and grout compressive strengths were configured and analyzed. An experimental study was also conducted to evaluate the D&E method and to verify major findings from the numerical study. Four exposed-type column-base plate connection sub-assemblages (two for 6-bolt connection and two for 4-bolt connection) were fabricated using notch-tough filler metals and tested under the SAC Phase II loading history (SAC, 1997) in the direction of weak axis.

In this paper, major findings from the numerical parametric study are mainly discussed. Effects of base plate thickness and anchor bolt stiffness on the column-base plate connection behavior under large column moments are investigated in the 1st and 2nd parametric studies, respectively. Effects of grout compressive strength on change of the resultant bearing force location are examined in the 3rd parametric study. Results of this numerical parametric study reveal several possible weaknesses of the D&E method in its basic assumptions and subsequent design calculations and concurrently, point to several important design considerations needed to develop more rational and reliable design methods. Study results of cyclic performance of the four test specimens are summarized in the companion paper (Lee *et al*., 2007), which is published together.

2. Design of Two Column-Base Plate Connections

Using selected connection elements and their planar dimensions, two exposed-type column-base plate connections consisting of different number of anchor bolts (i.e., 6-bolts and 4-bolts) are designed based on the D&E method. The selected column base planar dimensions are shown in Fig. 1. Also presented in this figure are two assumed bending lines in the base plate, which are used for calculation of the required base plate strengths in the D&E method. In Fig. 1, b_f means column flange width. Column is placed at the center of the base plate so that geometry of the connection is axi-symmetric. Effects of column axial loads on the connection behavior are not studied in this research program. Thus, only shear and moment loads induced from lateral forces in a steel Moment Resisting Frame (MRF) are used for the design of base plate thicknesses and anchor bolt sizes. In the following, the selected connection elements are detailed and major design considerations are described.

2.1. Column and base plate

An assembly of 80 *in*. long W12×96 A572 Grade 50 column and 20 *in.*×20 *in*. A36 base plate is selected for both the numerical and experimental studies. It is assumed in this numerical study that the boundary of these two connection elements are fully restrained, simulating groove welds between these two. It is also assumed that top end of the 80 *in.* column represents an inflection point at the mid-height of the column in a steel MRF. Thus, the clear length of the column between base plate upper surface and lower surface of the steel girder framing into the column is 160 *in.*

In order to calculate the connection design moment (M_u) shown in Fig. 2, a probable yield stress of 58 *ksi* is used for the Grade 50 column member instead of the specified minimum yield stress of 50 *ksi*. This increase of the column strength is accounting for strain hardening and potential overstrength effects in the connection. This approach is basically similar to the probable beam moment approach used for the column-beam moment

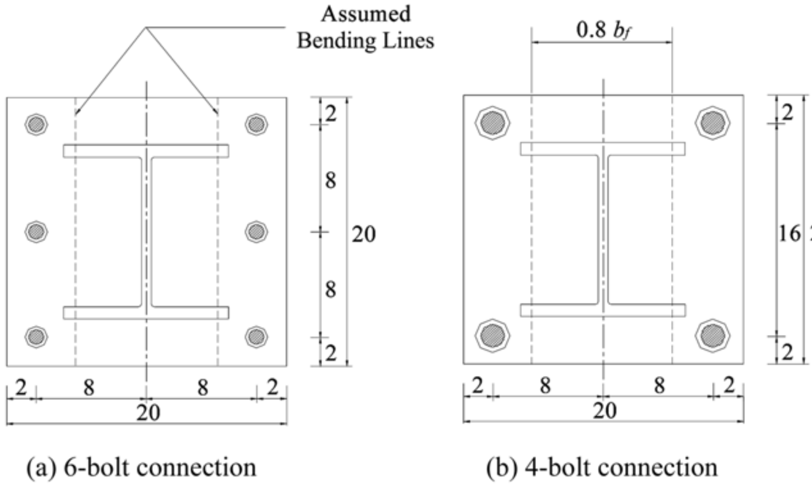


Figure 1. Base plate planar dimensions and assumed bending lines (units are in inches).

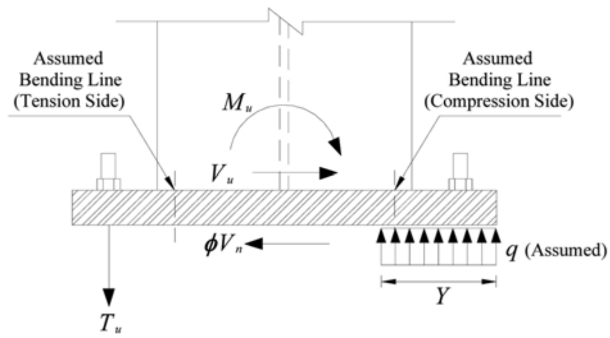


Figure 2. Design loads and unknown design parameters.

ratio requirement in Equation 9.3 of the 2005 AISC Seismic Provisions (AISC, 2005). The value of 58 *ksi* is larger than R_y*50 = 55 *ksi*, but less than 1.1*R_y*50 = 60.5 *ksi*. Using the above probable yield stress, the connection design moment (M_u) in presented Fig. 2 is calculated as:

$$M_u=(58\ ksi)\cdot Z_{y\_column}=(58\ ksi)\cdot(67.5\ in.^3)=3915\ k-in \tag{1}$$

The lateral force (V_u) that can develop a plastic hinge at the bottom of the 80 *in*. column element is thus:

$$V_u=\frac{M_u}{L_{column}}=\frac{3915K-in.}{80in.}=48.94\ kips \tag{2}$$

2.2. Compressive strength of grout, *f'c*

In order to design base plate thickness and anchor bolt size in a column-base plate connection, amount and location of the resultant bearing force (R_u) and the required total tensile strength of anchor bolts (T_u) should be calculated first. For these calculations, as shown in Fig. 2, the D&E method assumes rectangular shape of the bearing stress block under the base plate and the height of this stress block, q. Once the equivalent bearing length (Y) is obtained from the two equilibrium equations, i.e., vertical force and moment equilibrium equations, the amount and location of R_u and the amount of T_u can be easily calculated. Without axial loads, in this study, amount of T_u should be equal to amount of R_u (=qY). Hence, the larger base plate cantilever length measured from the assumed bending line is governing design of the base plate thicknesses.

The assumed height of the rectangular stress block, q, for the weak axis bending case can be expressed as:

$$q=\phi_c\cdot 0.85\cdot f_c\cdot N\cdot\sqrt{\frac{A_2}{A_1}}\quad\text{and}\quad\sqrt{\frac{A_2}{A_1}}\le 2 \tag{3}$$

where:

ϕ_c= resistance factor for bearing
f'_c= specified compressive strength of grout, *ksi*.
N= base plate width, *in*.
A_1= area of the base plate concentrically bearing on the grout (or concrete), *in.*^{2}
A_2= maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, *in.*^{2}

Equation 3 clearly shows that the amount of q is highly dependent on the specified compressive strength of grout (f'_c) for a given connection geometry. This indicates that the D&E design procedure may be very sensitive to the f'_c values selected for the connection design. A significant point which should also be mentioned herein is that the 2005 AISC LRFD Specifications (AISC, 2005) and the AISC Design Guide No. 1 (DeWolf and Ricker, 1990) recommend 0.6 for the ϕ_c in Eq. 3, whereas the ACI 318-02 (ACI, 2002) adopts 0.65 for the strength reduction factor in the case of bearing on concrete.

In order to show the sensitivity of the value of f'_c to the design of minimum required base plate thickness (tp) within the D&E method, Figure 3 is prepared. As shown in this figure, tension side governs the design procedure for the base plate thickness if the f'_c value less than 6 *ksi* is chosen. This is because a smaller f'_c value shortens the cantilever length measured from the assumed bending

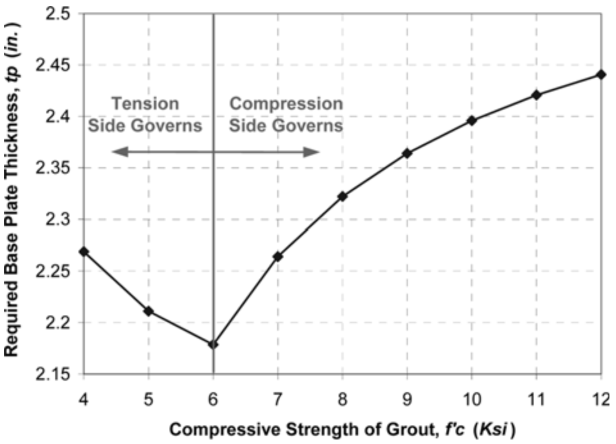


Figure 3. Effects of grout compressive strength (f'_c) on base plate thickness according to the D&E design method.

line on the compression side. On the contrary, compression side governs the design procedure if the f'_c value is larger than 6 *ksi*. Within the given range of f'_c values in Fig. 3, the maximum variation of the base plate thickness is larger than 1/4 *in.* It should be noted that this amount 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.

Based on previous experimental observations, it has been found that thinner base plates could be a good source of energy dissipation and provide for more ductile connection behavior during earthquake excitation (Astaneh *et al*., 1992). Higher ductility in the connection is always desirable in steel structures especially in high seismic zones. Thus, in this study, 6 *ksi* is selected for the f'_c value to maximize the possibility of meeting a flexible base plate. By choosing this grout compressive strength, more yielding in the base plate is expected when the connection develops its full moment capacity.

2.3. Base plate thickness

As mentioned, a 20 *in.*×20 *in.* A36 steel plate is chosen for the base plate planar size in this study. Assuming the area of A_2 is equal to A_1 and using 6 *ksi* for the f'_c value, 2.25 *in.* thick base plate is selected for both the 6-bolt and 4-bolt connections. This base plate thickness designed following the D&E method is denoted as "t_{po}" in this numerical parametric study.

2.4. Anchor bolt size

A354 Grade BD bolt (threaded rod) that has F_{u_bolt}= 150 *ksi* and F_{y_bolt}=130 *ksi* is selected for the anchor bolts to resist the calculated design tensile force (T_u) and shear force (V_u) in the connection. Mechanical properties of this anchor bolt are very similar to those of A490 high-strength bolt. Thus, in this study, the AISC LRFD Specifications (AISC, 2005) for the A490 bolts are adopted for the design of A354 Grade BD anchor bolts.

The designed minimum diameter of each anchor bolt for the 6-bolt connection is 1.107 *in.* and it is 1.356 *in.* for the 4-bolt connection. The total anchor bolt size on the tension side designed following the D&E method is denoted as "K_o". Over-sized bolt holes, as per the 2005 AISC LRFD Specifications (AISC, 2005), are prepared for both the 6-bolt and 4-bolt connections although specific sizes for the anchor bolt holes in base plates are recommended in Table 14-2 of the 2005 AISC Manual of Steel Construction (AISC, 2005). Distance between the base plate edge and the center of anchor bolt in the longitudinal direction, parallel to the column bending, is fixed at 2.0 *in.* so that the ratio between the anchor bolt distance from the edge (2.0 *in.*) and the total base plate length (20 *in.*) is equal to 0.1.

3. Finite Element Analysis Model

A total of 43 three-dimensional Finite Element Analysis (FEA) meshes that have different base plate thicknesses, anchor bolt sizes (stiffnesses), and grout compressive strengths are configured and analyzed in this numerical parametric study. Referring to the selected connection geometry and the designed base plate thickness and anchor bolt size, twenty FEA meshes that have five different base plate thicknesses and four different anchor bolt sizes are prepared for both the 6-bolt and 4-bolt connections. Effects of base plate thickness and anchor bolt stiffness on stress distribution in the connection and global connection response under large column lateral displacements are primarily investigated in this parametric study. Three more FEA meshes consisting of different grout compressive strengths are also prepared and analyzed only for the 6-bolt connection to examine effects of the grout compressive strength on variation of total resultant bearing force (R_u) due to the change of its location. These 43 FEA meshes are summarized in a matrix form in Table 1. As mentioned earlier, "t_{po}" and "K_o" in Table 1 denote, respectively, the designed base plate thickness and anchor bolt stiffness according to the D&E method.

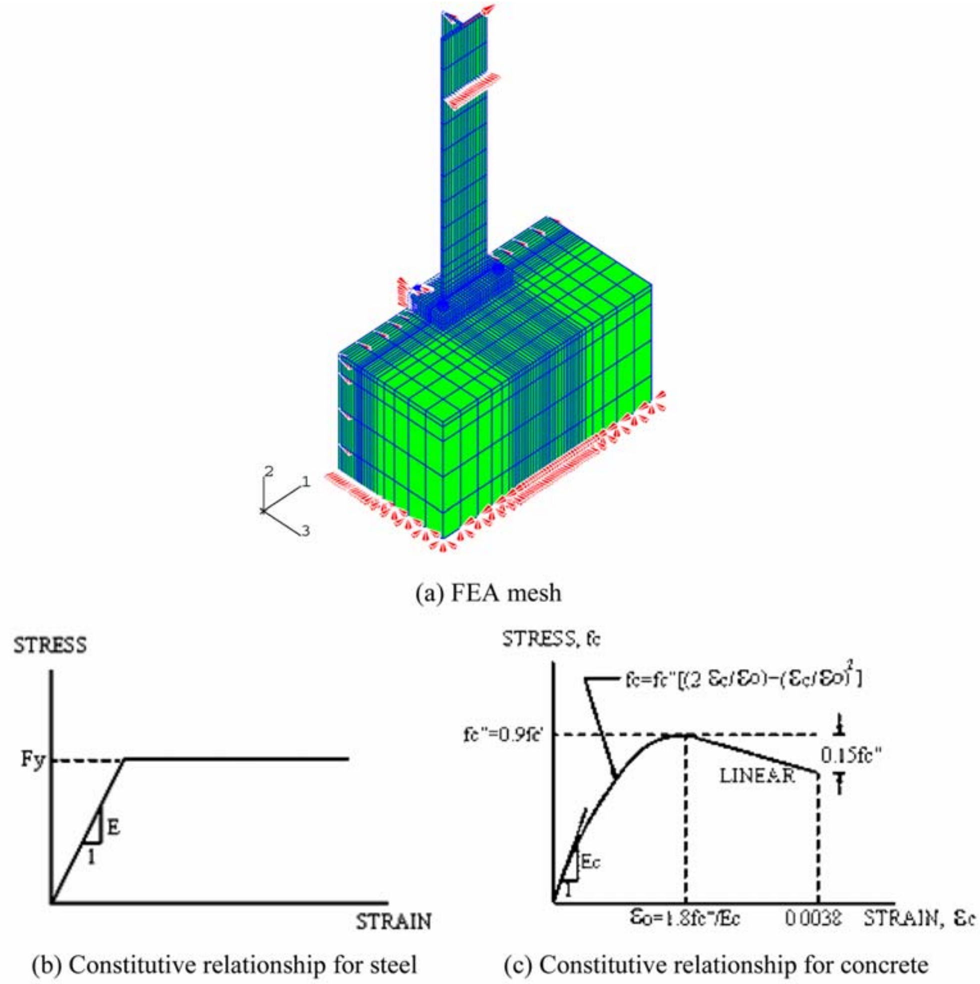
The ABAQUS program version 5.8 (HKS, 1998) is used to study nonlinear behavior and complicated stress distribution in the column-base plate connection. C3D8 (8 nodes linear brick element) finite element is selected to configure the FEA meshes. Because the connection is axi-symmetric, only one half side is modeled as shown in Fig. 4. The concrete foundation (74 *in.*×74 *in.*×36 *in.*) is assumed to be fixed to the ground at the bottom only. Thus, all degrees of freedom at the bottom surface of the foundation are constrained, whereas all degrees of freedom on lateral sides of the foundation are released. Each anchor bolt, which has an effective length of 32 *in.*, is also included in the FEA mesh with one end fixed directly to the inside of the concrete foundation. Figure 5 shows deformed shape of the FEA meshes under large lateral loads applied on the top end of the column.

One of the most difficult aspects in numerical analysis for exposed-type column-base plate connections is to

Table 1. Matrix of finite element analysis model

Base Plate Thickness	Anchor Bolt Stiffness							
	6-Bolt Connection				4-Bolt Connection			
	1.0 K_o	1.3 K_o	1.7 K_o	2.2 K_o	1.0 K_o	1.3 K_o	1.7 K_o	2.2 K_o
0.8 t_{po}		A				A		
0.9 t_{po}								
1.0 t_{po}	B	ABC	B	B	B	AB	B	B
1.1 t_{po}								
1.2 t_{po}		A				A		

A: selected for the study of base plate thickness effects (f'_c=6 *ksi*)
B: selected for the study of anchor bolt stiffness effects (f'_c=6 *ksi*)
C: selected for the study of grout compressive strength effects (f'_c=6, 8, 10, and 12 *ksi*)



(a) FEA mesh

(b) Constitutive relationship for steel

(c) Constitutive relationship for concrete

Figure 4. Three-dimensional FEA mesh and constitutive relationships.

model the three major contact interfaces, including the interfaces between (1) the nut and base plate upper surface; (2) the base plate lower surface and the grout; and (3) the anchor bolt and concrete foundation. Because of the difficulties in this modeling, 2D (or 3D) linear and nonlinear continuous spring models (Jaspart and Vandegans, 1998, and Fahmy, 1999) have been used for column base researches. However, these prior researches have not been able to model the interface between the nut and base plate upper surface, anchor bolt local bending below the nut (especially in the case of a thinner base plate), and the resisting mechanism in the connection against horizontal force (shear) components, including the anchor bolt and grout interaction. These phenomena are modeled more appropriately in the FEA meshes developed in this study (Lee and Goel, 2001).

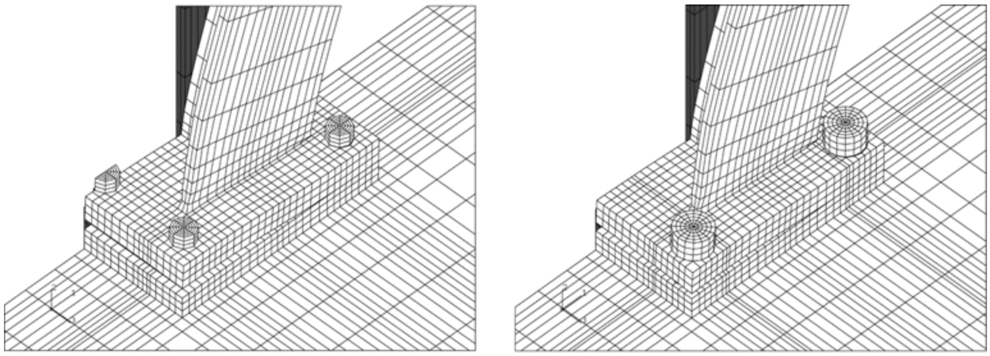


Figure 5. Magnified connection deformation.

In order to model force transfer in the above three major contact interfaces, the small-sliding formulation option of the ABAQUS program is used. For the friction coefficient, 0.5 is assumed for the first and second interfaces. However, no friction is assumed for the interface between the anchor rod and concrete foundation mostly based on the following observation: The bond between anchor rod and concrete foundation could be easily lost especially under large lateral deformation of the connection or under earthquake excitation (Sato and Kamagata, 1988, and Jaspart and Vandegans, 1998).

In this numerical study, constitutive relationships of the steel members are simplified by elastic-perfectly plastic bilinear lines as shown in Fig. 4. For the concrete members, the modified Hognestad stress-strain curve (Hognestad, 1951) is used to model the material non-linearity. The curvilinear portion in the modified Hognestad model is reformulated by ten linear segments for the application of the ABAQUS program. Main mechanical properties of each connection element used for the FEA modeling are summarized in Table 2. As explained earlier, considering the potential overstrength and strain hardening effects, 58 *ksi* is assumed for the strength of the Grade 50 column member.

4. First Parametric Study: Effects of Base Plate Thickness

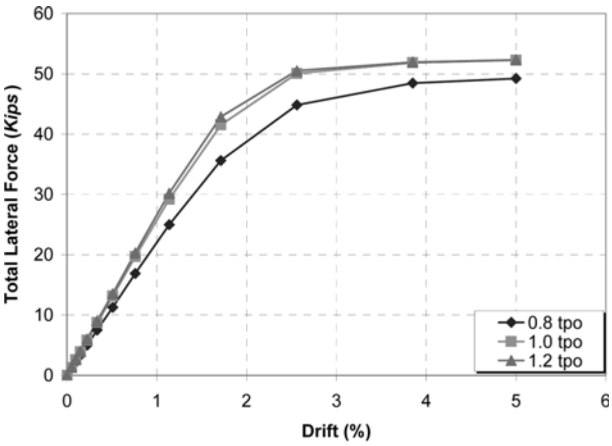
Finite Element Analysis (FEA) meshes that have five different base plate thicknesses are configured and analyzed for both the 6-bolt and 4-bolt connections to investigate role of the base plate thickness on connection behavior under large column lateral displacements. For convenience, however, analysis results of only three cases (i.e., 0.8 t_{po}, 1.0 t_{po}, and 1.2 t_{po}) are compared each other in this paper. The selected base plate thickness and anchor bolt stiffness combinations are marked as “A” on Table 1. In this table, 0.8 t_{po} represents a flexible base plate, while 1.2 t_{po} indicates a stiff base plate that usually forces major yields in other connection elements such as column and anchor bolts. For the anchor bolt size, 1.3 K_o, which is somewhat stiffer but still practically acceptable, is selected to avoid a significant yielding in the anchor bolts before the connection develops its full moment capacity. Based on comparisons of the analysis results, effects of the base plate thickness are investigated within the following six categories; (1) total lateral force, (2) total tensile bolt force, (3) base plate deformation, (4) base plate yielding pattern, (5) bearing stress distribution, and (6) variation of resultant bearing force location.

4.1. Total lateral force

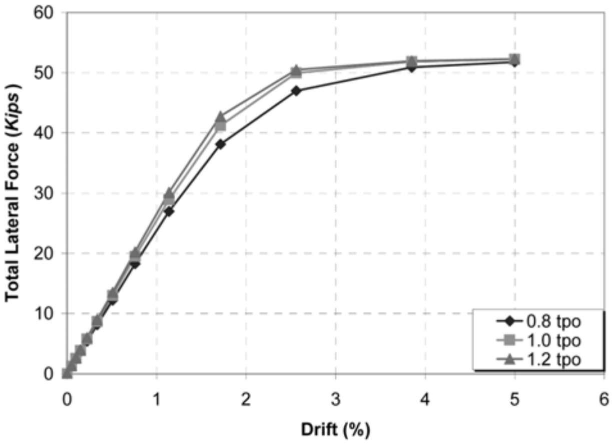
Total lateral forces needed to deform top end of the column are calculated up to 5.0% drift level, and the forces of the selected six connections (three for the 6-bolt connection and three for the 4-bolt connection) are compared in Fig. 6. In this figure, no significant differences in the global connection responses (such as initial rotational stiffness and maximum moment capacity of the connection) are observed between the 6-bolt and 4-bolt connections. For both connections, however, lower

Table 2. Mechanical properties of connection elements for the analysis model

Connection Elements		Yield Strength or Compressive Strength (*ksi*)	Young’s Modulus (*ksi*)	Poisson’s Ratio
Steel	Column	58	29000	0.3
	Base Plate	36	29000	0.3
	Anchor Bolt	130	29000	0.3
Concrete	Grout	6	4400	0.18
	Foundation	4	3600	0.18



(a) 6-bolt connection



(b) 4-bolt connection

Figure 6. Lateral forces versus drifts (for models with anchor bolt stiffness of 1.3 K_o).

connection strength and rotational stiffness are commonly observed in case of the flexible base plate (i.e., 0.8 t_{po}). These strength and stiffness reductions are due to early yielding in the base plate. The base plate designed by the D&E method (i.e., 1.0 t_{po}) and the thicker base plate (i.e., 1.2 t_{po}), on the other hand, do not show the early base plate yielding. In the latter two cases, most yields in the connection are concentrated in the column flanges. The above observations provide one conclusion: Thinner base plates may result in strength and stiffness reductions in exposed-type column-base plate connections such that a weak connection/strong column mechanism can be developed.

4.2. Total tensile bolt force

Amount of the total tensile bolt force (T_u) can be affected by flexibility of the base plate for a given connection geometry. This phenomenon can be clearly seen in Fig. 7. The T_u values in this figure are directly calculated from the strains on the surface of the anchor bolts at 3.85% drift level and their areas. As shown in Fig. 7, the amount of T_u varies significantly with change of the base plate thickness

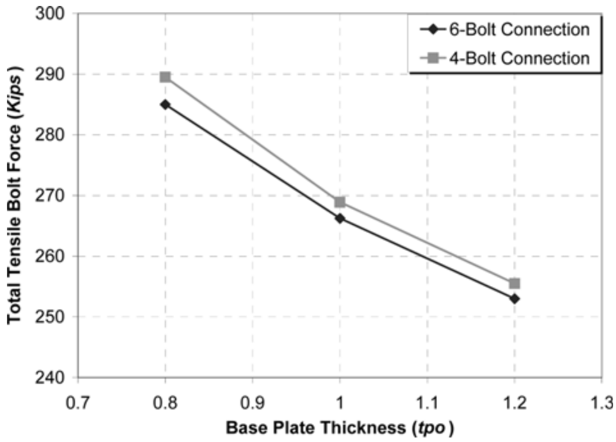


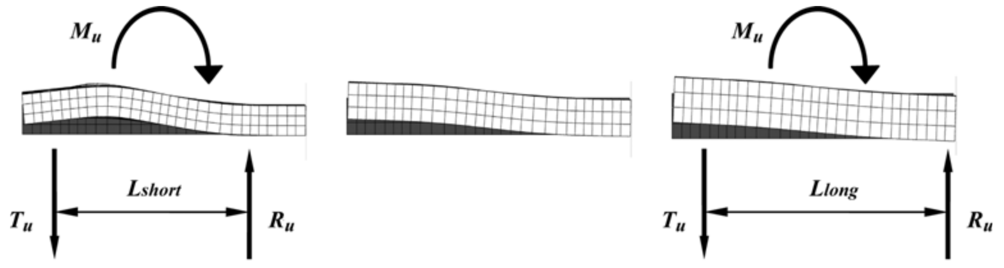
Figure 7. Total tensile bolt forces at 3.85% drift (for models with anchor bolt stiffness of 1.3 K_o).

in both the 6-bolt and 4-bolt connections. 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 (1.2 t_{po}), 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. 8(c). 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.

4.3. Base plate deformation

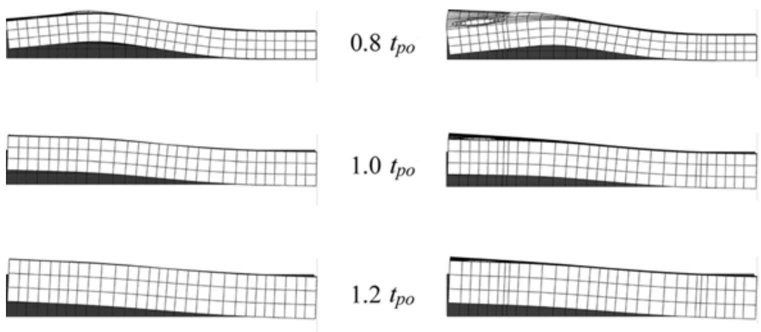
Base plate deformations of the selected six connections at the level of 3.85% drift are presented and compared in Fig. 9. In the cases of 1.0 t_{po} and 1.2 t_{po}, no significant differences in the deformed shape of the base plate are observed between the 6-bolt and 4-bolt connections, except for the deformed shape of the base plate in the transverse direction. Slightly convex-shaped base plate deformation is observed in the 4-bolt connections in this direction on the tension side. In the case of 0.8 t_{po}, however, significant out-of-plane base plate deformation is observed in the 4-bolt connection, whereas the 6-bolt connection shows a quite in-plane base plate deformation. The out-of-plane base plate deformation in the 4-bolt connection is mostly due to no existence of the constraint in the middle of the base plate. In order to ensure a consistent base plate deformation and seismic connection response, use of 6-bolt connections are thus recommended in this study as a minimum number of anchor bolts when designing flexible base plates are targeted although regulations of the Occupational Safety and Health Administration (OSHA) - Safety Standards for Steel Erection (OSHA, 2001), require a minimum of 4 anchor bolts in the column-base plate connections.

As mentioned earlier, it has been believed that thinner base plates could be a good source of energy dissipation and show more ductile connection behavior under severe earthquake excitation (Astaneh *et al.*, 1992). However, it must be noted that thinner base plates can cause high



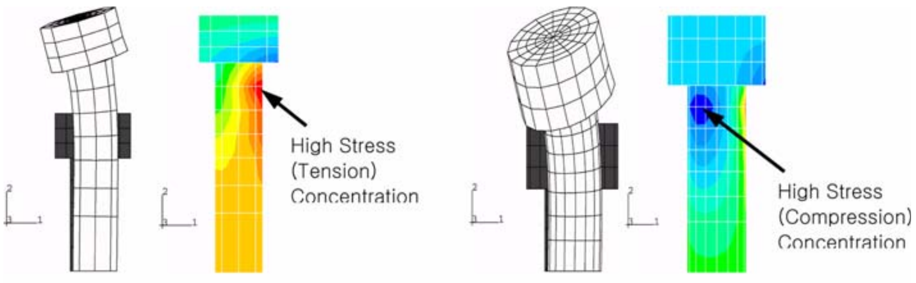
(a) 0.8 t_{po} (b) 1.0 t_{po} (c) 1.2 t_{po}

Figure 8. Deformed shape and location of the bearing resultant in 6-bolt connections at 3.85% drift (for models with 1.3 K_o anchor bolt stiffness).



(a) 6-bolt connection (b) 4-bolt connection

Figure 9. Base plate deformation at 3.85% drift (for models with anchor bolt stiffness of 1.3 K_o).



(a) 6-bolt connection (b) 4-bolt connection

Figure 10. Local anchor bolt deformation and stress contours at 3.85% drift (in the models with 1.3 K_o anchor bolts and 0.8 t_{po} base plate).

local stress concentration in the anchor bolts on the tension side. As shown in Fig. 9, the 0.8 t_{po} base plates deform outwardly on the tension side under large column lateral displacements, whereas the 1.0 t_{po} and 1.2 t_{po} 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 bolts and the stress contours presented in Fig. 10. Hence, in order to prevent the possible anchor bolt failure due to such high stress concentration, the minimum thickness of the base plate should be provided when exposed-type column base plate connections are designed.

4.4. Yielding pattern in base plate

In order to investigate effects of the base plate thickness on variation of base plate yielding pattern, equivalent Mises stress contours on the base plate top surface are compared in Fig. 11. Interestingly, the base plates designed by the D&E method (i.e., 1.0 t_{po}) do not fully yield across their whole widths in both the 6-bolt and 4-bolt connections. Instead, only small yielded parts are observed on the surface of the base plate. This indicates that the D&E method may result in stiffer base plates.

As already shown in Fig. 9, different anchor bolt location results in different base plate deformation on the tension side. Such different deformation causes significantly different stress distribution in the base plate. The comparison

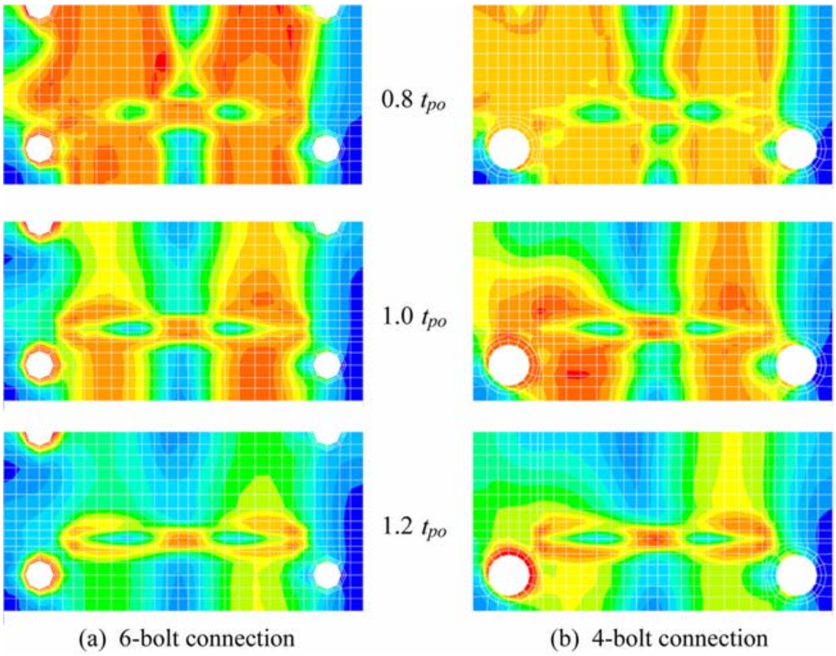


Figure 11. Equivalent von Mises stress contours on the base plate surface at 3.85% drift (in the models with 1.3 K_o anchor bolt stiffness).

shown in Fig. 11 indicates that it is very difficult to develop a simplified design model that covers both the 6-bolt and 4-bolt connections. In addition, the FEA results shown in Fig. 11(b) imply that the simplified two-dimensional design approach used for the current U.S. column base design practice is much ineffective for the 4-bolt connections.

In the 6-bolt connection, shown in Fig. 11(a), major base plate yielding grows along the straight bending lines on both sides of the base plate (i.e., tension and compression sides) which are assumed in the D&E method as well as in the AISC Design Guide No. 1 (DeWolf and Ricker, 1990). A significant point which should be mentioned is that the relative strength between the base plate and anchor bolts may cause change of the major base plate yielding location. For the same column and anchor bolt sizes, larger portion of the base plate yields can be seen on the compression side in the cases of 1.0 t_{po} and 1.2 t_{po} base plates, while 0.8 t_{po} base plate shows larger base plate yielding on the tension side.

4.5. Bearing stress distribution on grout

Bearing stress contours for the selected six connections are presented and compared in Fig. 12. In both the 6-bolt and 4-bolt connections, high stress concentrations are observed under the column flanges in the case of thin base plate (0.8 t_{po}), whereas the thick base plate case (1.2 t_{po}) shows high stress concentrations along the grout edge. Such significant variation of bearing stress distribution is a result of different thickness of the base plate. In case of the thin base plate, flexural base plate deformation causes high local compressive force concentration right below the column flanges. In case of the thick base plate, however, the base plate deforms as a rigid plate so that bearing stresses concentrate along the free edge of the grout. The FEA results, presented in Fig. 12, provide one important conclusion: Designing excessively thick base plates should be avoided because they may cause undesirable grout crushing along the free edge of the grout on the compression side. Based on the experimental observation, Thambiratnam and Paramasivam (1986) also noted that the high stress concentration on the grout edge could cause an unexpected grout crushing under small connection lateral deformation.

The analysis results presented in Fig. 12 also enable two more conclusions. First, due to the complicated bearing stress distribution in the transverse direction, assuming the simplified rectangular bearing stress block used in the D&E method may not be realistic even for the 6-bolt connections. Second, different number (arrangement) of anchor bolt can result in different bearing stress distribution on the compression side. Higher bearing stress concentration is observed at the corner of the grout in the 4-bolt connections. This higher stress concentration in the 4-bolt connection is mostly due to the convex-shaped base plate deformation transferred to the compression side and decreased bearing area caused by relatively large anchor bolt sizes. Hence, it can be concluded that the 4-bolt connection is more critical for the designing and detailing of the grout.

4.6. Variation of resultant bearing force location

The three-dimensional FEA model developed in this study makes it possible to more accurately estimate locations of the resultant bearing force (R_u) on the compression side. These locations can be calculated

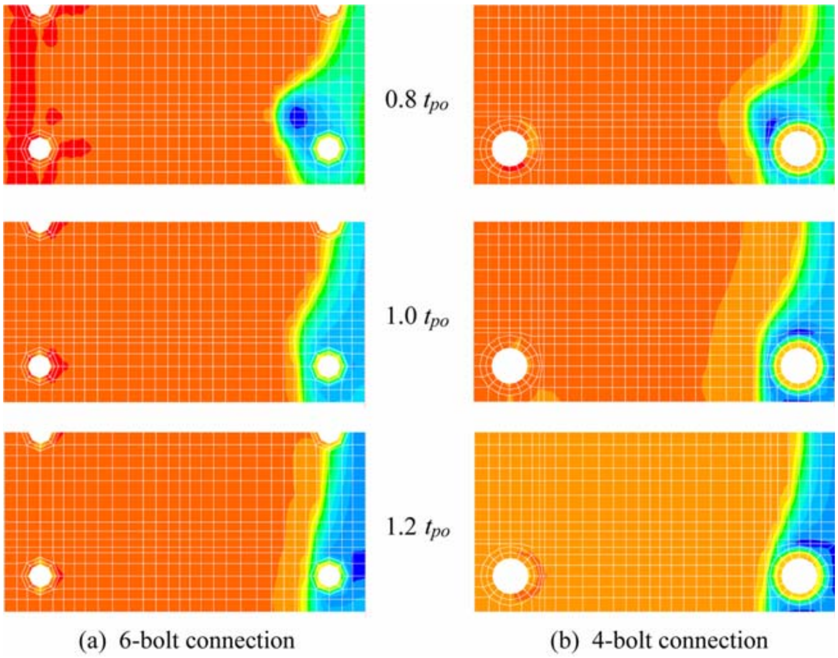


Figure 12. Bearing stress distribution on the grout surface at 3.85% drift (in the models with 1.3 K_o anchor bolt stiffness).

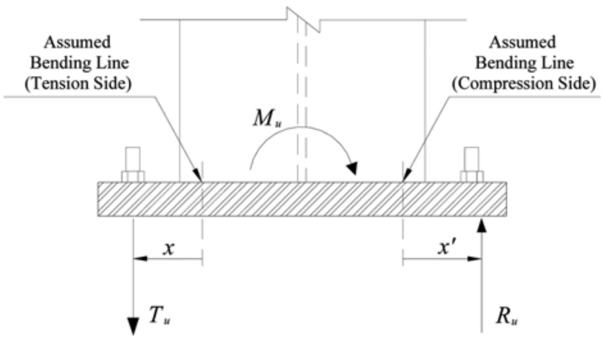


Figure 13. Two cantilever lengths, x and x'.

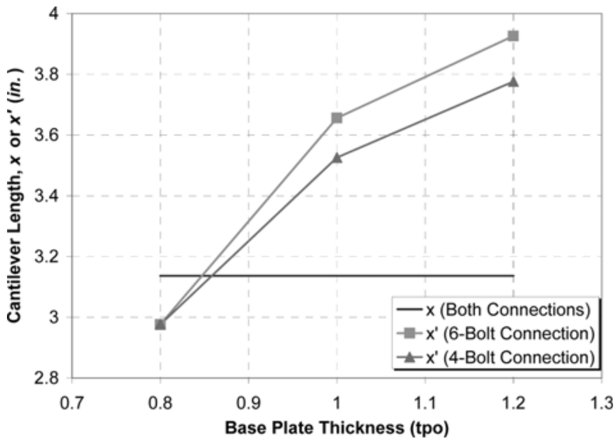


Figure 14. Effects of base plate thickness on resultant bearing location (in the models with 1.3 K_o anchor bolt stiffness at 3.85% drift).

directly from the bearing stresses presented in Fig. 12. In order to compare the total tensile bolt force (T_u) and R_u locations of the selected six connections, two moment arms are defined in Fig. 13. In this figure, x indicates the distance between T_u and the assumed bending line on the tension side whereas x' means the cantilever length measured from the assumed bending line on the compression side to the R_u location. For the selected connection dimensions, x is equal to 3.136 *in.* and the maximum length of x' (to the edge of the base plate) is 5.136 *in.*

Using the new variable, x', the R_u locations are plotted and compared in Fig. 14. This figure clearly shows that the location of R_u can be significantly affected by thickness of the base plate. For the selected range of the base plate thickness (i.e., 0.8 t_{po} to 1.2 t_{po}), maximum variation of x' is (3.926-2.977)/5.136=0.185 (18.5%) for the 6-bolt connection and this is (3.776-2.976)/5.136= 0.156 (15.6%) for the 4-bolt connection. The analysis results shown in Fig. 14 indicate that undesirable grout crushing on the compression side, due to high bearing stress concentration near the free edge of the grout, can be prevented by decreasing the base plate thickness. The location of R_u can also be affected by anchor bolt stiffness for a given base plate thickness. This will be further discussed in the following numerical parametric study.

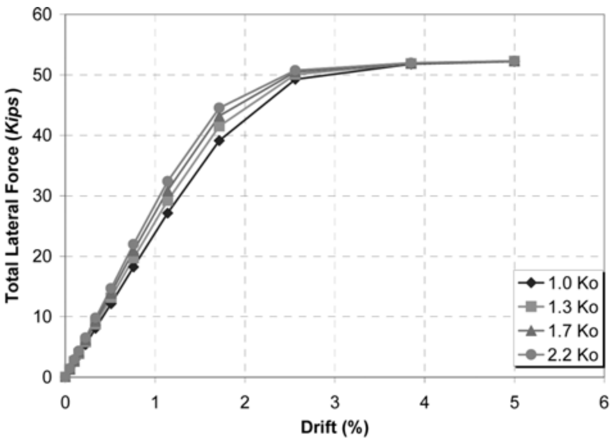
5. Second Parametric Study: Effects of Anchor Bolt Stiffness

In order to investigate effects of the anchor bolt size (stiffness) on connection behavior under large column lateral displacements, Finite Element Analysis (FEA) meshes consisting of four different anchor bolt stiffnesses (i.e., 1.0 K_o, 1.3 K_o, 1.7 K_o, and 2.2 K_o) are prepared and analyzed for both the 6-bolt and 4-bolt connections. Weak anchor bolts (<1.0 K_o) are excluded in this parametric study because those cases are undesirable especially for the column bases in high wind or seismic region, which often have to be designed for uplift column forces. Additionally, excessive anchor bolt elongation in a column-base plate connection makes it more difficult to be repaired once the connection is damaged. The selected anchor bolt stiffness and base plate thickness combinations are marked as "B" on Table 1. The numerical analyses within the six categories, used for the first parametric study, are repeated in the following.

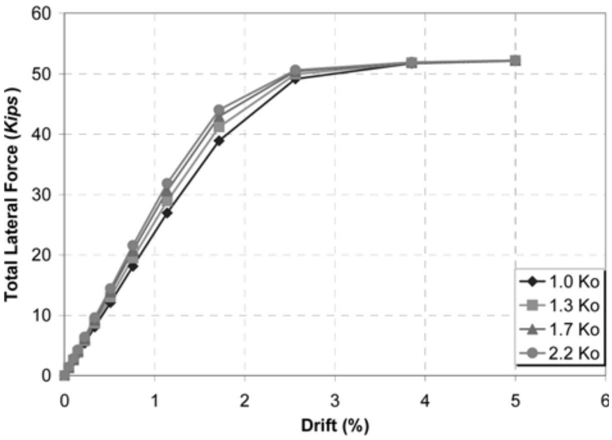
5.1. Total lateral force

Total lateral forces of the selected eight connections (i.e., four for the 6-bolt connection and four for the 4-bolt connection) are plotted and compared in Fig. 15. In both the 6-bolt and 4-bolt connections, initial stiffness of the connection varies with the change of the anchor bolt stiffness. Evidently, larger anchor bolts result in stiffer response in the connection. Flat portion of each graph after around 3.0% drift level implies that the connection reaches its full moment capacity. Through the first numerical parametric study, it has been showed that the base plate designed following the D&E method (i.e., 1.0 t_{po}) results in rather stiff connection responses. Thus, with stronger anchor bolts (>1.0 K_o) in this study, the flat portion of each graph is mostly due to widespread yielding in the column flanges.

The analysis results presented in Fig. 15 provide two conclusions: First, initial rotational stiffness of the connection is proportional to the anchor bolt stiffness for both the 6-bolt and 4-bolt connections. However, within the practical range of the anchor bolt stiffness, the variation is very small and may be neglected. Second, all of the selected base plate thickness/anchor bolt stiffness combinations for the second numerical parametric study are good enough to develop a strong connection/weak column mechanism.



(a) 6-bolt connection



(b) 4-bolt connection

Figure 15. Total lateral forces versus drifts (using 1.0 t_{po}).

5.2. Total tensile bolt force

The calculated total tensile bolt forces (T_u) are compared in Fig. 16. In both the 6-bolt and 4-bollt connections, a stiffer anchor bolt results in a higher value of the T_u. This is due to shortening of the overall moment arm between T_u and the resultant bearing force (R_u). In case of the stiff anchor bolt (2.2 K_o), due to constraint of the base plate rotation on the tension side, the overall moment arm between T_u and R_u is shortened as presented in Fig. 17(c). In order to resist the same amount of the design moment (M_u), transferred from the column, the value of T_u (and R_u) must increase with the shorter moment arm.

5.3. Base plate deformation

Base plate deformations of the selected four connections are presented in Fig. 18. Due to relatively large elongation of the anchor bolt, the largest base plate rotation is observed in the case of 1.0 K_o in both the 6-bolt and 4-bolt connections. As shown above, the anchor bolt elongation (base plate rotation) as well as the base plate flexibility is the key factor which controls location of the resultant bearing force (R_u) on the compression side.

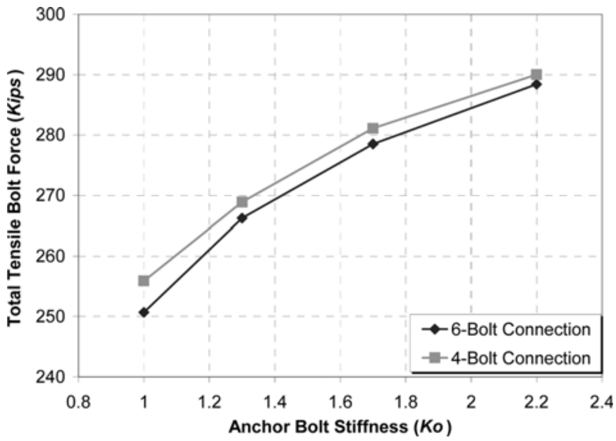


Figure 16. Total tensile bolt forces at 3.85% drift (using 1.0 t_{po}).

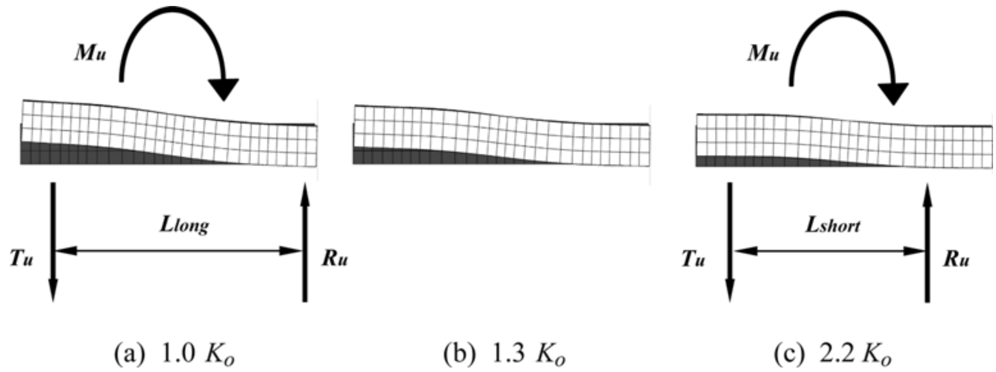


Figure 17. Variation of resultant bearing force location in 6-bolt connections (using 1.0 *tpo*, at 3.85% drift).

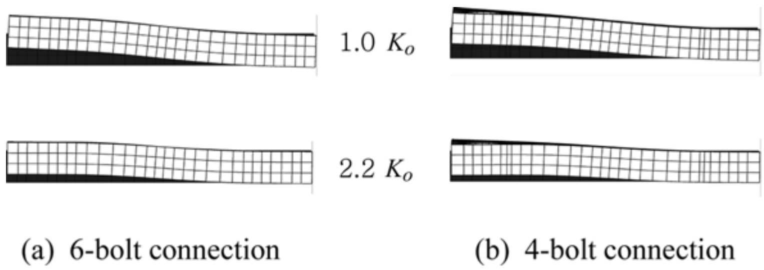


Figure 18. Base plate deformation at 3.85% drift (using 1.0 *tpo*).

Thus, for better-controlled performance of the exposed-type column-base plate connections, it may be necessary to limit permissible anchor bolt elongations.

In general, stiffer anchor bolts increase the potential for outward base plate deformation on the tension side even though the base plate is strong enough to develop full moment capacity of the column in the connection. As explained in Fig. 10, the outward base plate deformation causes high local stress concentration in the anchor bolts. The chance for this type of local anchor bolt failure can be maximized when stiffer anchor bolts are coupled with thinner base plates. Hence, it may be necessary to limit the relative strength ratio among the connection elements (i.e., column, base plate, and anchor bolts) so that undesirable early connection failure on the tension side can be avoided.

5.4. Yielding pattern in base plate

Equivalent Mises stress contours on the base plate surface are compared in Fig. 19. In the 6-bolt connection, as shown in Fig. 19(a), relatively large base plate yielding is observed on the compression side in the case of 1.0 K_o while larger portion of the base plate yield is observed on the tension side in the case of 2.2 K_o. The main reason of this variation can be explained effectively from deformed shape of the base plate in the longitudinal direction. As already shown in Fig. 18(a), the smaller anchor bolt stiffness (1.0 K_o) results in the larger base plate rotation. Due to small base plate settlement on the grout, relatively large base plate curvature forms on the compression side in this case. The larger curvature, of course, directly causes larger base plate yielding on the compression side. Similarly, larger anchor bolt stiffness (2.2 K_o) which is forcing larger base plate curvature on the tension side results in relatively large base plate yielding on the same side.

5.5. Bearing stress distribution on grout

In order to investigate effects of the anchor bolt stiffness on bearing stress distribution in the grout, bearing stress contours of the selected four connections are compared in Fig. 20. In both the 6-bolt and 4-bolt connections, the contact (bearing) area between the base plate and the grout in the longitudinal direction varies with the change of the anchor bolt stiffness. In the case of 1.0 K_o, base plate deforms as like a rigid plate due to relatively large anchor bolt elongation on the tension side so that the bearing area diminishes. Such reduction of the bearing area results in an increase of the bearing stresses in the grout near the free edges. As explained earlier, due to the convex shape of the base plate deformation under the column weak axis bending, relatively high bearing stresses are observed at the corners of the grout in the 4-bolt connections.

5.6. Variation of resultant bearing force location

Locations of the resultant bearing force (R_u) for the eight connections, marked as “B” in Table 1, are plotted in Fig. 21 using the cantilever length, x', defined in Fig. 13. For the selected range of the anchor bolt stiffness (i.e., 1.0 K_o to 2.2 K_o), maximum variation of x' is (3.856-3.186)/5.136=0.130 (13.0%) for the 6-bolt connection and this is (3.716-3.126)/5.136=0.115 (11.5%) for the 4-bolt connection. In both the 6-bolt and 4-bolt connections,

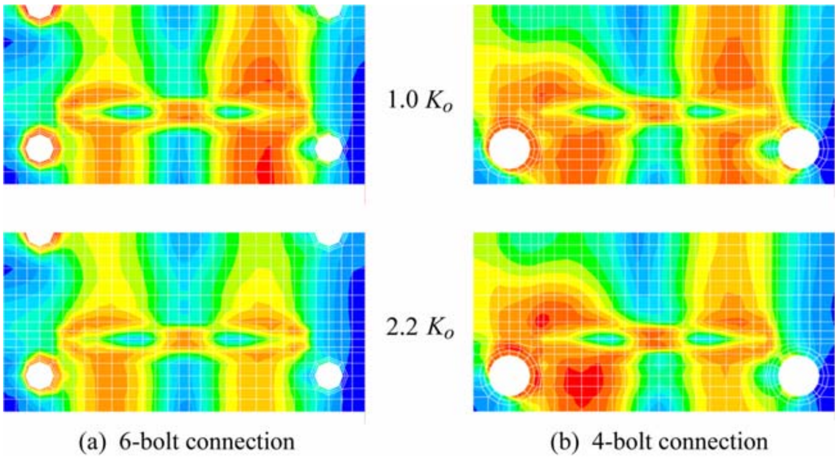


Figure 19. Equivalent Mises stress contours on base plate surface at 3.85% drift (using 1.0 t_{po}).

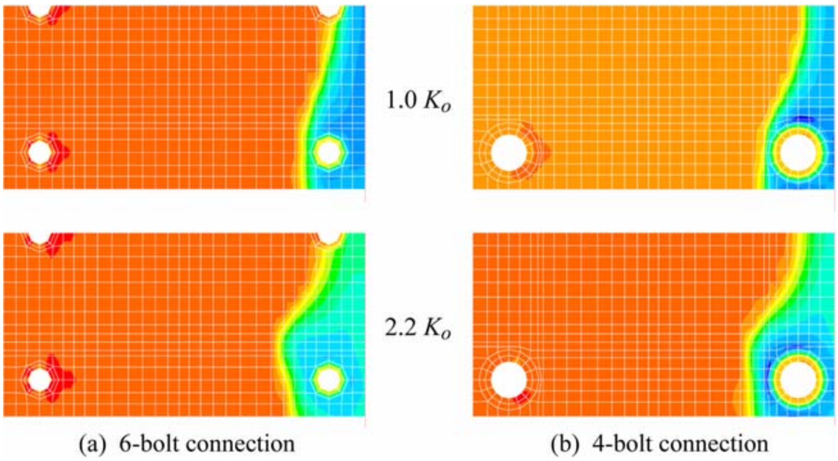


Figure 20. Bearing stress distribution on grout surface at 3.85% drift (using 1.0 t_{po}).

the increase of the anchor bolt stiffness results in a decrease of the cantilever length x' on the compression side. The analysis results presented in Fig. 21 leads to a very important conclusion; undesirable grout crushing due to high bearing stress concentration near the free edge can be prevented by increasing the anchor bolt size.

6. Third Parametric Study: Effects of Grout Compressive Strength

Three additional Finite Element Analysis (FEA) meshes consisting of different grout compressive strengths (i.e., 8 *ksi*, 10 *ksi*, and 12 *ksi*) are prepared and analyzed to numerically investigate effects of the grout compressive strength (f'_c) on change of the resultant bearing force (R_u) location on the compression side. This third numerical parametric study is conducted only for the 6-bolt connection. The selected base plate thickness and anchor bolt stiffness combination is marked as "C" on Table 1. For the two previous parametric studies, value of the grout compressive strength has been fixed as 6 *ksi*. Including the previous FEA results of the 1.0 t_{po}-1.3 K_o-6 *ksi* combination, a total of four different grout compressive strengths are considered for this third parametric study.

As shown in Fig. 3, the D&E method appears quite sensitive to variation of f'_c. This is because the D&E design procedure is highly dependent on the assumed concrete (or grout) bearing capacity per unit length (q) as well as the assumed shape of the bearing stress. As a result, as shown in Fig. 22, maximum variation of x' is (4.196-3.136)/5.136=0.206 (20.6%) for the selected range of the grout compressive strength (i.e., 6.0 f'_c to 12.0 f'_c). However, the FEA results plotted in the same figure show that the cantilever length x' does not change significantly with the change of f'_c. Instead, as shown through the two previous parametric studies, the R_u location is highly dependent on the base plate thickness and anchor bolt size (stiffness) for a given column member size, or the relative strength ratio among those three connection elements.

7. Summary and Discussion

Using the developed three-dimensional Finite Element Analysis (FEA) model, an extensive numerical parametric study has been conducted at the University of Michigan.

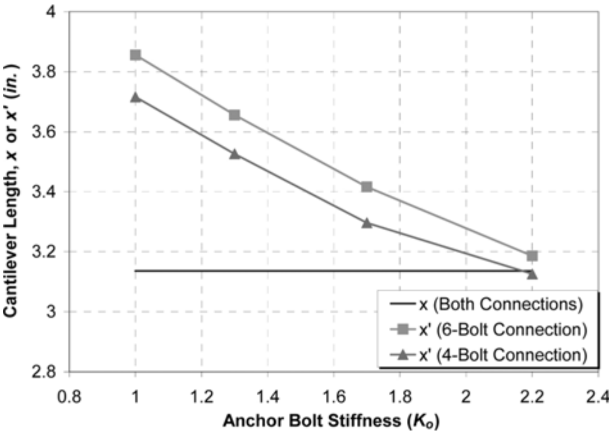


Figure 21. Effects of anchor bolt stiffness on cantilever length x' (using 1.0 t_{po}, at 3.85% drift).

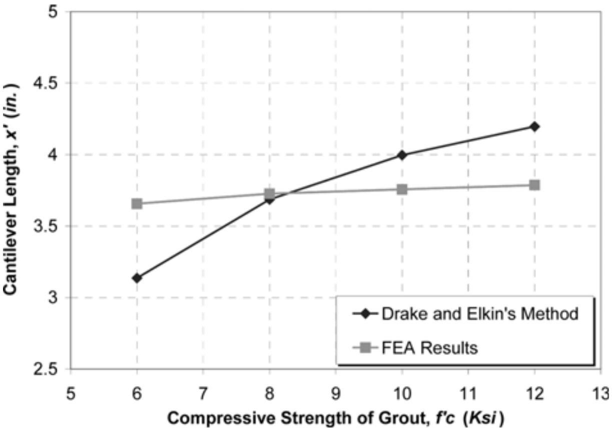


Figure 22. Variation of cantilever length x' on the compression side with respect to the change of f'_c (6-bolt connection, using 1.3 K_o and 1.0 t_{po}, at 3.85% drift).

The numerical study revealed several possible limitations of the D&E method in its basic assumptions and subsequent design calculations and concurrently, pointed to several important design considerations needed to develop more rational and reliable design methods. Also revealed in this study were significant contributions of the relative strength ratio among the connection elements on the exposed-type column-base plate connection behavior under large column lateral displacements. Major findings from this numerical study are further discussed below.

7.1. Unrealistic application of concrete beam design methodology

In order to calculate the unknown design parameters, q and Y presented in Fig. 2, the D&E method adopts an equivalent rectangular bearing stress block which is generally used for the ultimate strength design of concrete beams. However, this concrete beam design methodology may not be directly applicable to the exposed-type column-base plate connection design because of the following three reasons. First, the base plate and grout is not monolithic so that the traditional "plane sections remain plane" assumption used for the development of beam theory does not hold. Second, due to small or no bond stresses between the anchor rods and concrete (and grout) especially under earthquake excitation, the "the strain in the reinforcement is equal to the strain in the concrete at the same level" assumption may not be realistic. Lastly, inconsistent base plate deformations on both the tension and compression sides due to relative strengths among the connection elements make the above two simplifications less trustful.

The FEA results, presented in Figs. 12 and 20, imply difficulties in the development of a simplified bearing stress distribution in the longitudinal direction for both the 6-bolt and 4-bolt connections. The bearing stress distributions in this direction are not even close to the rectangular shape that is assumed in the D&E method. Even for the 1.0 t_{po}-1.0 K_o combinations, shown in the first row of Fig. 20, the bearing stress distributions look more like a triangular or trapezoidal shape instead of the rectangle. Bearing stress distribution in the transverse direction requires a more complicated bearing stress model especially for the 4-bolt connections. Conclusively, further research should be conducted to apply the concrete beam design methodology directly to the exposed-type column-base plate connection design. The upgraded design methodology should provide a rational and reliable bearing stress model that accounts for the relative strength effects and the non-uniform bearing stress distribution in the transverse direction. Alternatively, another design approach that is independent of or less dependent on the bearing stress distributions in both the longitudinal and transverse directions may be developed.

7.2. Over-sensitive design procedure to q (or f'_c)

The concrete bearing capacity per unit length (q), shown in Fig. 2, is one of the most important design parameters in the D&E method. With the assumed expression of the q (in terms of f'_c), presented in Eq. 3, the D&E method calculates the required total tensile strength of the anchor bolt (T_u) and the resultant bearing force (R_u) by using the two equilibrium equations, i.e., vertical force and moment equilibriums. Because the required base plate flexural strength on each side is calculated directly from those two force resultants (i.e., T_u and R_u), designing base plate thickness is highly dependent on the q (or f'_c) values. Within the given range of the f'_c, shown in Fig. 3, the maximum variation of the base plate thickness is slightly over 1/4 *in*. As mentioned, this variation is large enough to change the connection response under earthquake excitation.

As shown in Fig. 22, the value of q (or f'_c) can significantly change the location of R_u in the D&E method. For the given range of f'_c, the maximum change of the cantilever length x' is more than 1.0 *in.* This amount is almost 20% of the maximum x' variation, i.e., 5.136 *in*., which is measured from the assumed bending line on the compression side to the edge of the base plate. In contrast, the numerical parametric study shows very different results. The FEA results shown in Fig. 22 indicate that x' does not varies with the change of the f'_c value. Instead, it has been shown in this numerical study that the location of R_u is highly dependent on the relative strength ratio among the connection elements. For instance, base plate inward or outward deformations and amount of anchor bolt elongation can change the location of R_u significantly. Hence, the design procedures that are highly dependent on the assumed shape of the bearing stress distribution and its assumed height may need to be cautiously reassessed especially for the application to the exposed-type column base plate connections in high seismic zones.

7.3. No Explanation of Anchor Bolt Location Effect

Despite its significance, the D&E method does not recognize effects of the different anchor bolt location (or arrangement) on the column-base plate connection behavior under large column moments. Two major effects are summarized in the following. First, different number of anchor bolt leads a different base plate deformation on the tension side, which results in significantly different stress distribution in the base plate. As shown in Figs. 9 and 18, a three-dimensional out-of-plane base plate deformation can be developed in the 4-bolt connections while the base plates in the 6-bolt connections remain uniform in the transverse direction. The out-of-plane base plate deformation in the 4-bolt connection can be more serious when the base plate is thinner, as shown in the first row of Fig 9(b). As a result of the different base plate deformation, the 4-bolt connections develop a circular yielding pattern in the base plate on the tension side, whereas the 6-bolt connections show quite linear base plate yields as shown in Figs. 11 and 19. Second, different anchor bolt location can also significantly change the bearing stress distribution on the compression side. As shown in Figs. 12 and 20, due to the convex-shaped base plate deformation transferred to the compression side, relatively high bearing stress concentrations are observed at the corner of the grout in the 4-bolt connections.

The regulations of the Occupational Safety and Health Administration (OSHA) - Safety Standards for Steel Erection (OSHA, 2001) require a minimum of 4 anchor bolts in column-base plate connections. However, because of much complicated base plate yielding pattern on the tension side and undesirable high bearing stress concentration at the corner of the grout in the 4-bolt connections, use of 6 anchor bolts is highly recommended in this study as a minimum number of anchor bolts especially for the exposed-type column-base plate connections bending about weak axis.

7.4. No identification of relative strength ratio effect

Using the calculated total tensile bolt force (T_u) and the design shear force (V_u), presented in Fig. 2, the D&E method designs minimum required anchor bolt sizes based on the 2005 AISC LRFD Specifications (AISC, 2005). Structural design engineers in high wind or seismic region, however, may frequently want stronger anchor bolts than what the D&E method provides. This is because, in those zones, excessive anchor bolt yield and elongation are not desirable for the exposed-type column bases which often have to resist high uplift column forces. Unfortunately, the D&E method seems not to be directly used for the design of column-base plate connections that have stronger anchor bolts because of the following reason: This design method is developed based on the “the anchor bolts on the tension side yield at the ultimate state of the connection” assumption.

When the design problem stated above is met, the increase of the anchor bolt size and base plate thickness should be carefully decided because of the relative strength ratio effects which have been shown through the first two numerical parametric studies. For instance, the anchor bolt size (stiffness) can change location of the resultant bearing force (R_u) as well as its amount due to the change of the overall moment arm between T_u and R_u. More seriously, as shown in Figs. 9 and 10, relative strength between the anchor bolt and the base plate for a given column size can result in the outward base plate deformation which may cause high local stress concentration in the anchor bolts. Conclusively, the D&E method should be modified or further developed for the design of column-base plate connections that have stronger anchor bolts (i.e., anchor bolts larger than 1.0 K_o). Alternatively, another design approach that can complement the weaknesses of the current design practice may be developed. In the latter case, relative strength ratio among the connection elements (i.e., column, base plate, and anchor bolts) can be used as a main parameter to estimate the connection response at its ultimate state.

7.5. Design of stiff base plate

By choosing 6 *ksi* for the grout compressive strength in Fig. 3, it has been intended in this study to design the minimum thickness of the base plate for the given connection geometry within the D&E method. As explained earlier, however, base plates designed by the D&E method did not fully yield across the whole width in both the 6-bolt and 4-bolt connections while these connections developed their full moment capacities. Only limited yield region was observed on the surface of these base plates, resulting in stiffer base plates. This indicates that connections designed by the D&E method may not behave as intended.

8. Observations and Conclusions

The Drake and Elkin’s design method has been evaluated based on an extensive numerical parametric study. Also investigated are effects of the relative strength ratio among the connection elements on the connection behavior under large column lateral displacements in the direction of weak axis. Several conclusions are drawn from this numerical study:

The Finite Element Analysis (FEA) results showed that the D&E method results in stiffer base plates. This indicates that connections designed by this method may not behave as intended especially in ultimate state of the connection.

In the D&E method, location of the resultant bearing force (R_u) is highly dependent on the assumed value of q (or f'_c of the grout). However, results of the numerical analysis plotted in Fig. 22 showed the opposite.

The numerical parametric study revealed that location of the resultant bearing force (R_u) as well as its amount varies with the change of the base plate thickness and anchor bolt stiffness. Thinner base plates and stiffer anchor bolts increase the amount of R_u due to the shortened overall moment arm between the total tensile bolt force (T_u) and R_u.

In order to prevent undesirable early grout crushing under the base plate edge on the compression side, designing an excessively thick base plate coupled with weaker anchor bolts should be avoided.

A thinner base plate can cause high local stress concentration in the anchor bolts on the tension side due to outward base plate deformation. Hence, for a selected anchor bolt size, minimum thickness of the base plate should be provided to prevent such condition in the anchor bolts before the connection reaches its ultimate state.

The 4-bolt connections may cause a convex-shaped out-of-plane base plate deformation, which is resulting in very complicated base plate yielding pattern on the tension side and undesirable bearing stress concentration at the corner of the grout on the compression side. For these reasons, in this study, use of 6 anchor bolts are recommended as a minimum number of anchor bolts for the exposed-type column-base plate connections especially in high seismic zones.

It has been shown through the numerical parametric study that the relative strength ratio among the connection elements (i.e., column, base plate, and anchor bolts) could be used to control the connection responses under large column moments. This concept may also be very useful for the upgrade of the existing design practices or for the development of more rational and reliable design methods.

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