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An Analytical Model of Anchor Bolted Column-Baseplate Connections in Steel Moment Frame Buildings

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Abstract

Study on seismic response of steel moment frames is largely limited to beam-to-column connections while column-to-foundation connections receive little attention. This is mainly due to the complex behaviour of these connections, which makes determination of this load-deformation characteristics challenging. Hence, designers generally assume either fully fixed or ideally hinged boundary condition for design of columns and column bases. Besides, it is a general practise to assume a fixed column base, as it results in smaller design moment in columns. Further, for seismic applications, it is believed that fixed column bases will ensure columns to yield or develop an axial-flexural plastic hinge in the columns near the connection region facilitating ductile mechanism. However, the actual boundary condition is neither fixed nor hinged, and plastic hinges are rarely formed in columns as stated in many post-earthquake reconnaissance reports; damages were observed mainly in concrete pedestals, anchor bolts, base plates, or even in connecting welds. Hence, it is important to understand the actual boundary condition realized in such column bases to identify the sequence of damage and design the connection accordingly.

An analytical model is developed for unstiffened and stiffened anchor-bolted column baseplate connections to estimate anchor bolt and baseplate deformations, and obtain idealized load-deformation characteristics of such connections, under the action of combined gravity and lateral loads. The column base is idealised as a simple 3-member model to incorporate the flexibilities of baseplate and anchor bolts, especially the out-of-plane bending and subsequent uplift of the baseplate, and elongation of anchor bolts. These actions cause significant reduction in rotational stiffness of such anchor bolted baseplate connections compared to a fully fixed base scenario. Further, these deformations induce early yielding of baseplate and anchor bolts, which in turn, limits the connection strength; in most cases, the column remains elastic. The accuracy of the analytical model is substantiated using nonlinear finite element analyses. Thus, the analytical model helps to predict the sequence of damage in addition to providing reasonable estimates of its initial stiffness, yield strength and ultimate strength of anchor bolted baseplate connections. This facilitates more realistic modelling of column base boundary condition for structural analysis of moment frames compared to the conventional assumptions of fully fixed/ideally hinged column base.

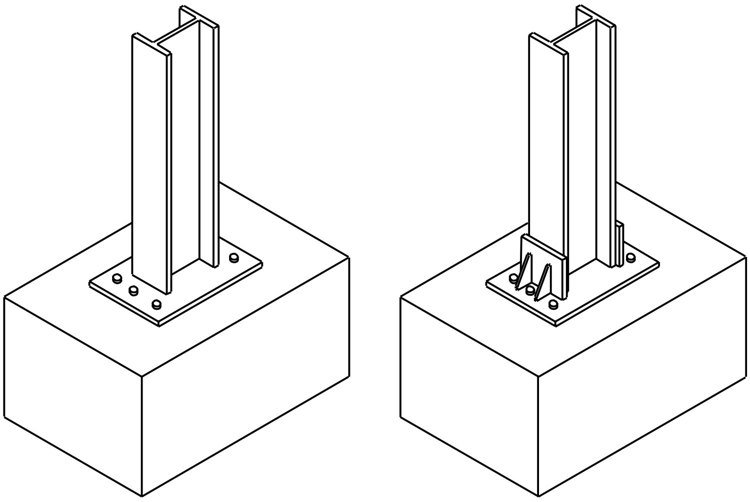
Keywords: seismic design, moment-rotation, connection stiffness, spring-model

1. Introduction

Steel as a material has high strength and high modulus of elasticity enabling it to undergo large inelastic deformations. However, steel moment resisting frames (MRFs) may not have adequate lateral stiffness, strength or ductility. This is because lateral stiffness, lateral strength and lateral ductility of steel structures are often limited by those of the *connections* in these MRFs. The vulnerability of steel MRFs to strong earthquake shaking was fully recognized only after the 1994 Northridge and 1995 Hyogo-ken Nanbu (Kobe) earthquakes. Before the 1994 Northridge earthquake, damages to steel structures were attributed to poor quality in construction, fracture of welded connections, or spalling of concrete in the foundation [1]. Hence, little attention was given to the behaviour of joints in steel frames till the early 90s. Severe damages to steel structures due to concentration of stresses at welded beam-column and column baseplate connections, resulting in the brittle fracture of welds were widely observed after the Northridge earthquake. Similar failures of steel structures were common in the 1995 Kobe Earthquake as well. Also, it was seen that the localised plastification of baseplate and anchor bolts lead to large and undesirable residual lateral displacement in MRFs [2, 3]. Further, during post-earthquake reconnaissances, varying degree of damage to connections was observed in steel structures with columns and beams remaining elastic. Experimental studies on behaviour of steel structures also demonstrated similar damages to connections. Significant amount of research over the next few decades led to the development of various ductile beam-to-column connection configurations and methods to obtain their load-deformation characteristics. But, similar quantity and quality of research on column-to-foundation connections is scarce, although the standard anchor-bolted baseplate connection on concrete foundation is used almost universally for MRFs.

1.1 Anchor-bolted Column Baseplate Connection

Anchor-bolted column-baseplate connections are either (a) unstiffened or (b) stiffened. A typical unstiffened anchor-bolted column-baseplate connection consists of a column (usually wide-flange section) welded to a baseplate, which in turn, is bolted to a reinforced concrete foundation (Fig. 1(a)). This configuration usually results in concentration of stresses at the joint under lateral load due to non-smooth flow of forces from the column to the baseplate. Stiffening this connection with cover plates and rib plates (*stiffeners*) avoids stress concentration at the joint and allows smoother flow of forces from the column to the baseplate (Fig. 1(b)). In addition, stiffeners also help minimise out-of-plane bending of the baseplate, and thereby the required thickness of the baseplate is reduced, resulting in relatively economical design.



(a) (b)

Fig. 1: (a) Unstiffened and (b) Stiffened Anchor-bolted baseplate connections for major axis bending of column

Boundary condition (rotational stiffness) assumed at the column bases is crucial in the design of frame elements. While a hinged column base is assumed to be ensured by providing bolts along the neutral axis of the baseplate, 2 or more bolts on either side of the neutral axis is assumed to ensure a fixed column base (Fig. 2). MRFs are often designed with the assumption of a fixed base as it results in economic design of column sections. Further, MRFs are expected to form ductile sway mechanism to maximise energy dissipation capacity, requiring *ductile* plastic hinges to form at column bases too, in addition to those at beam ends. Furthermore, it is assumed and preferred that the plastic hinges in the column base forms in the column, and not in the connection region. But, failures of steel structures in past earthquakes were concentrated in the column base connection region with the columns remaining elastic; yielding of the baseplate, brittle fracture of the baseplate and welds, and crushing of the concrete foundation were the common modes of failure of anchor-bolted base connections [2, 3, 4].

Early experimental studies on anchor-bolted column base connections developed methods to quantify the yield *strength* of such connections [5, 6]. And, the criticality of assuming a particular rotational *stiffness* of column bases in design got noticed, when column bases designed as fully fixed demonstrated partial fixity conditions even in the linear response range [7]. The semi-rigid behaviour of column base connections under lateral loading makes their design more complex [8, 9]. Thus, in addition to quantifying the strength of a column base connection, there is a need to quantify the semi-rigidity (rotational stiffness) of column base connections. For the purpose, a reasonably accurate analytical model to predict actual behaviour of column base connections incorporating the behaviour of each component of the connection, namely baseplate, anchor bolts and the concrete foundation, is required.

Further, most analytical models developed in the past to predict the load-deformation characteristics of column base connections often overestimated stiffness and strength of such connections. This is mainly attributed to assumptions made in their design like rigid baseplate and uniform or a linearly varying stress under the baseplate, which results in designing for less connection deformations. But, *prying* of the baseplate, *bending* and *elongation* of the anchor bolts lead to *uplift* of baseplate and concentration of stresses under the baseplate, especially under seismic actions, result in significant reduction in stiffness of the connection, compared to an ideally fixed column base boundary condition [8, 9, 10]. Hence, additional attention is required to address deformations induced due to these actions, to develop more realistic connection load-deformation characteristics.

This paper presents an analytical model to obtain the load-deformation characteristics of anchor-bolted column baseplate connections and help identify the sequence of possible yielding of the connection components. Further, using this information on the sequence of possible yielding in the connection, the developed model provides a simple way to achieve the desired mode of inelastic action at column bases, *i.e.*, formation of plastic hinges in the column, away from the connection region. The efficacy of the analytical model, in predicting the stiffness and strength of column base connections, is demonstrated using results of non-linear finite element analyses of typical column-base connection sub-assemblages.

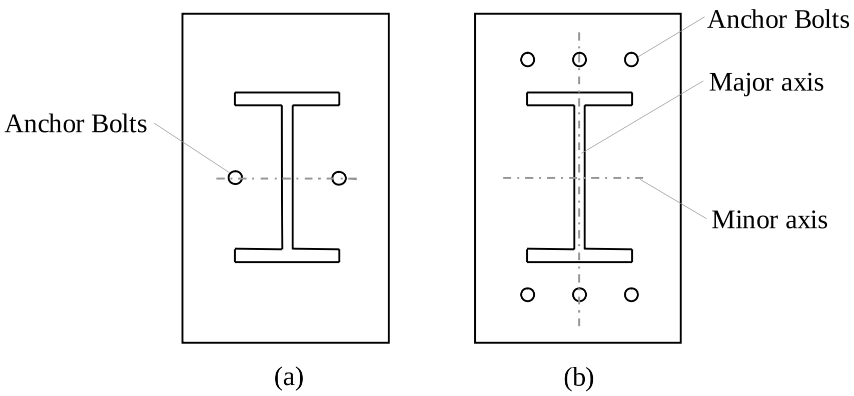


Fig. 2: (a) Hinged and (b) fixed column base connections using anchor-bolted baseplates

2. Beam-spring model for anchor-bolted column baseplate connection

Observations from experiments and post-earthquake reconnaissance of steel MRFs suggest that most column bases fail by prying and flexural yielding of baseplates or tensile yielding of anchor bolts. The analytical model proposed in this paper incorporates these realistic deformation modes of failure, to help estimate the stiffness and strength of the connection, and in turn, aims to predicting realistic behaviour of such connections, especially under lateral loading conditions as during earthquakes.

A column subjected to a lateral load F_{lat}, tends to bend (Fig. 3). Consequently, the baseplate ABCD (Points B and C represent the outer surfaces of the column flanges and points A and D represent the centre-lines of bolts on each side of the column flanges) is assumed to bend about point B, with a rotational stiffness, which depends on the remaining segment of the plate AB resting on concrete. Also, the lateral load is assumed to be transferred from the column to the connection as a tension-compression couple in the column flanges at the surface of the baseplate; the tension force F_t causing the uplift of the baseplate (Fig. 3). Thus, the baseplate is idealized as a beam and the bolts are modelled as linear springs (Fig. 4). The following assumptions are made in the modelling of baseplate:

(a) baseplate is under one-way bending due to the presence of bolts along the major axis of the column (this assumption is valid only for a connection with 3 or more bolts on either side of the column), and

(b) the segment AB of the baseplate is in full contact with the concrete foundation below.

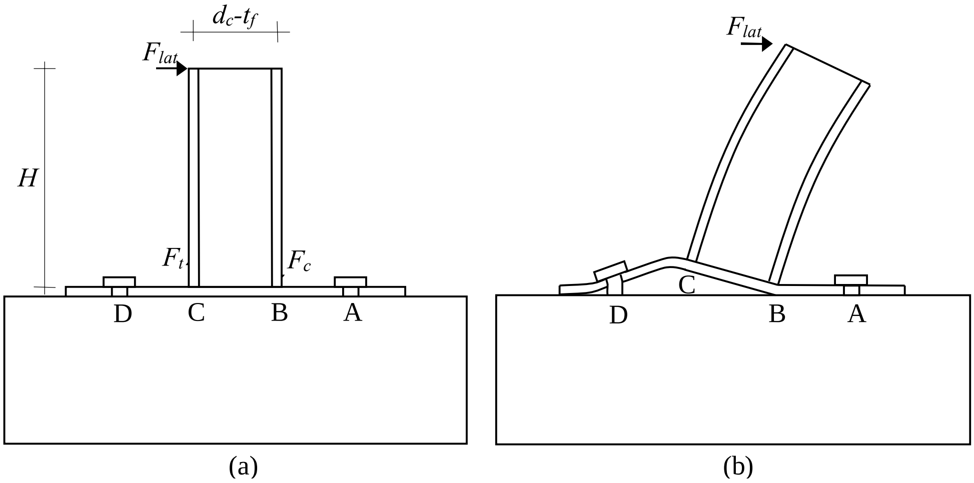


Fig. 3: (a) Transfer of load from column to baseplate, (b) deflected shape of a column base connection under the action of the lateral load.

Therefore, the segment BCD of the baseplate is idealized as a beam hinged at B, with a rotational stiffness which depends on the remaining segment of the plate AB resting on concrete. In unstiffened connections, segment CD has stiffness equal to that of the baseplate. In stiffened connections, additional stiffness due to the presence of cover plates and rib plates is also considered while computing the stiffness of the segment CD (Fig. 5). Segment BC is relatively stiffer than segment CD due to the stiffness offered by the column web. The segment BA of the baseplate beyond the compression flange is assumed to be resting on linear elastic springs, similar to elastic foundations (Fig. 6). This is based on the assumption that reaction at a point of a beam on an elastic foundation will be directly proportional to the deflection at that point with a constant of proportionality k known as *Winkler's constant* [11].

Further, the three bolts near the tension flange of the column are collectively modelled as an axial spring with stiffness equal to three times the axial stiffness of a single bolt (Fig. 4). The additional rotation at B, due to the uplift and bending of the baseplate, and elongation of anchor bolts is incorporated in the model to obtain a realistic estimate of stiffness and strength of the connection (Fig. 5).

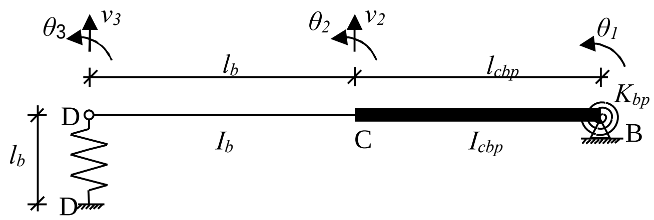


Fig. 4: *Beam-Spring Model* for anchor-bolted column baseplate connection.

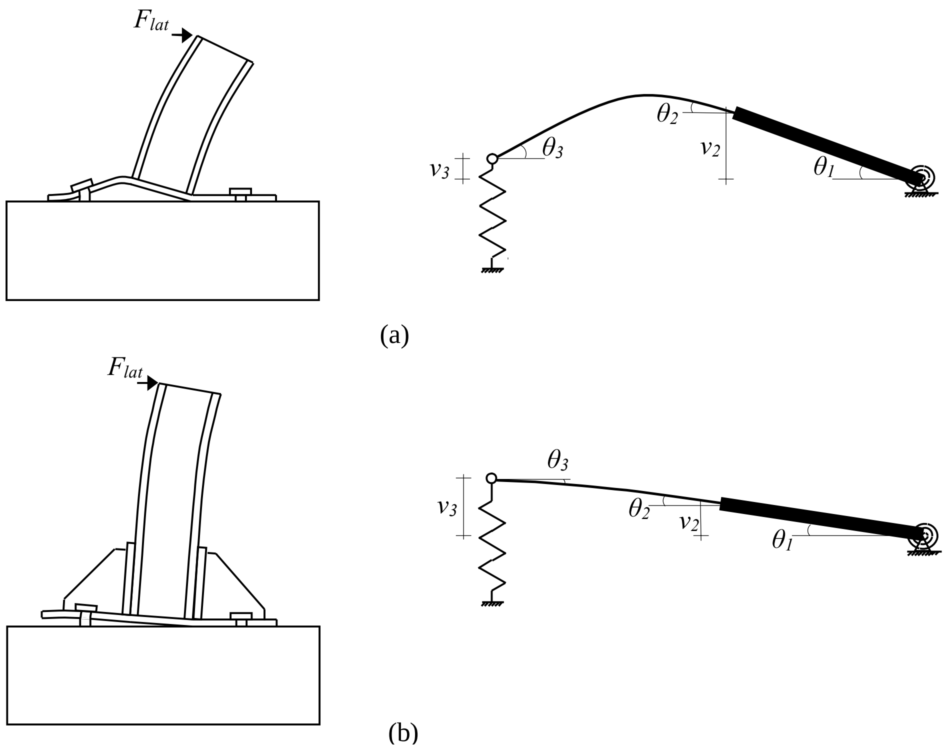


Fig. 5: Connection deformation under the action of a lateral load:
(a) unstiffened baseplate, and (b) stiffened baseplate.

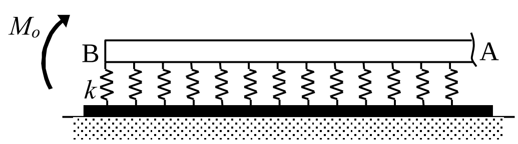


Fig. 6: Idealised *Winkler beam* model for baseplate segment AB resting on concrete

The magnitude of force transferred through the column flange is computed as,

$$F_t = F_{lat} \times \frac{H}{d_c - t_f} \tag{1}$$

where, F_{lat} is the lateral load acting on the top of the column, H the height of the column, d_c the centre-to-centre distance between the column flanges and t_f the thickness of the column flange. Then, the initial stiffness of the connection is calculated as the ratio of the moment generated at the surface of the baseplate to the rotation of the baseplate about point B (θ_1) (Fig. 5). The nodal deformations corresponding to force F_t are calculated using the following equilibrium equations:

$$[K]\times\begin{Bmatrix}\theta_1\\ v_2\\ \theta_2\\ v_3\\ \theta_3\end{Bmatrix}=\begin{Bmatrix}0\\ F_t\\ 0\\ 0\\ 0\end{Bmatrix}, \tag{2}$$

where the global stiffness matrix $[K]$ is,

$$[K]=\begin{bmatrix}\frac{4EI_{cbp}}{l_{cbp}}+K_{bp} & \frac{6EI_{cbp}}{l_{cbp}^{2}} & \frac{2EI_{cbp}}{l_{cbp}} & 0 & 0\\ \frac{6EI_{cbp}}{l_{cbp}^{2}} & 12E\left(\frac{I_{cbp}}{l_{cbp}^{3}}+\frac{I_{bp}}{l_{bp}^{3}}\right) & 6E\left(\frac{I_{cbp}}{l_{cbp}^{2}}-\frac{I_{bp}}{l_{bp}^{2}}\right) & -\frac{12EI_{bp}}{l_{bp}^{3}} & -\frac{6EI_{bp}}{l_{bp}^{2}}\\ \frac{2EI_{cbp}}{l_{cbp}} & 6E\left(\frac{I_{cbp}}{l_{cbp}^{2}}-\frac{I_{bp}}{l_{bp}^{2}}\right) & 4E\left(\frac{I_{cbp}}{l_{cbp}}+\frac{I_{bp}}{l_{bp}}\right) & \frac{6EI_{bp}}{l_{bp}^{2}} & \frac{2EI_{bp}}{l_{bp}}\\ 0 & -\frac{12EI_{bp}}{l_{bp}^{3}} & \frac{6EI_{bp}}{l_{bp}^{2}} & \frac{12EI_{bp}}{l_{bp}^{3}}+K_b & \frac{6EI_{bp}}{l_{bp}^{2}}\\ 0 & -\frac{6EI_{bp}}{l_{bp}^{2}} & \frac{2EI_{bp}}{l_{bp}} & \frac{6EI_{bp}}{l_{bp}^{2}} & \frac{4EI_{bp}}{l_{bp}}\end{bmatrix}. \tag{3}$$

Here, E is the modulus of elasticity of baseplate, I_{cbp} and I_{bp} the second moments of area of segments BC and CD respectively, l_{cbp} and l_{bp} the lengths of segments BC and CD respectively, K_b the axial stiffness of spring DD' and K_{bp} the rotation stiffness at the hinge B. Further, the uplift force F_t (Fig. 3(a)) acting on the baseplate is calculated using,

where, F_{lat} is the lateral load acting on the top of the column and d the depth of the column section.

The stiffness of the spring DD' (representing the bolts) is calculated as the effective axial stiffness of 3 bolts acting in parallel, given by:

$$K_b=\frac{3\times A_bE}{l_b} \tag{4}$$

where, A_b is the area of cross-section of the bolt and l_b the length of the bolt outside the pedestal.

Second moment of area of cross-section of segment CD and segment BC of *unstiffened* connection are estimated using Eq. (5) and Eq. (6), respectively.

$$I_{bp}=\frac{b_{eff}t_{bp}^{\ 3}}{12} \tag{5}$$

$$I_{cbp}=\frac{b_{eff}t_{bp}^{\ 3}}{12}+\frac{t_wH^3}{3}+b_{eff}t_{bp}\left(x_c-\frac{t_{bp}}{2}\right)^2+Ht_w\left(\frac{H}{2}+t_{bp}-x_c\right)^2 \tag{6}$$

where, t_{bp} is the thickness of the baseplate, and b_{eff} the effective width of the baseplate which is assumed to be the centre-to-centre distance between the outer bolts, t_w is the web thickness of the column, H the height of the column, and x_c the distance of the centroidal axis of the cross-section of the beam segment BC from the bottom of the baseplate. In stiffened connections, the rib plate is idealised as a prismatic beam with an effective height αh in order to compute the stiffness of the plate (Fig. 7); α is taken as 0.3 in the current study. Hence, the second moment of area of the segment CD of *stiffened* connections is estimated using:

$$I_{bp} = \frac{Bt_{bp}^{\ 3}}{12} + \frac{2t_{rp}(\alpha h)^3}{12} + Bt_{bp}\left(x_{rp} - \frac{t_{bp}}{2}\right)^2 + 2\alpha ht_{rp}\left(\frac{\alpha h}{2} + t_{bp} - x_{rp}\right)^2 \tag{7}$$

where, t_{bp} is the thickness of the base plate B is the total width of the base plate, t_{rp} is the thickness of the rib plate, αh is the height of the idealised rectangular rib plate and x_{rp} is the distance of the centroidal axis of the cross-section of the beam CD from the bottom surface of the base plate.

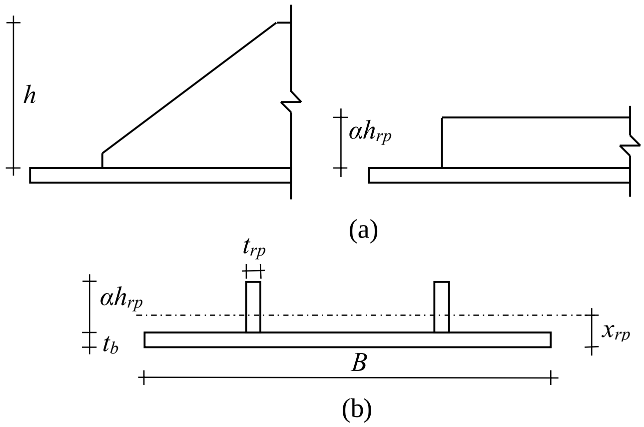


Fig. 7:(a) Rib plate idealised as a plate of uniform height, (b) cross-section of the idealised beam CD.

The rotational stiffness (K_{bp}) of the spring at B is calculated as the ratio of moment generated at point B to the rotation at the same point of the idealised semi-infinite *Winkler's beam* (Fig. 6) [11]. And, the rotation of the baseplate (θ_1) is obtained directly from the equilibrium equation (Eq. (2)). Alternatively, it is computed as the ratio of uplift under the tension flange to the length of the segment BC (l_{cbp}) as:

$$\theta_{bp} = \theta_1 \approx \frac{v_2}{l_{cbp}} \tag{8}$$

Thus, with the help of the analytical model, the global boundary condition at the column base can now be idealised as a hinge with a finite rotational stiffness (Fig. 8(a)). The load-deformation characteristics of the connection (rotational spring) under the action of a lateral load, is represented as a tri-linear relationship between the moment generated at the top surface of the baseplate and the rotation of the baseplate in case of unstiffened connections and between the moment generated at the end of the cover plate (connection reinforcement) and the rotation of the baseplate in case of stiffened connections (Fig. 8(b)).

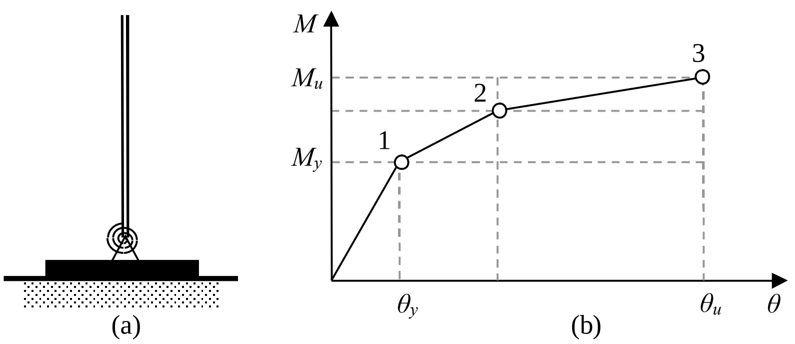


Fig. 8: (a) Idealised boundary condition at the column base and (b) *M*-*θ* relationship of the connection obtained from the analytical model

2.1 Failure Modes

The analytical beam-spring model proposed above helps to identify the failure modes and sequence of yielding in both unstiffened and stiffened column baseplate connections (Fig. 8 (b)). In unstiffened connections, yielding of the connection is characterised by flexural yielding of the baseplate under the tension flange (Point 1). This is followed by yielding of the baseplate under the compression flange, which gives the second point (Point 2) of the M-θ curve. However, in stiffened connections, connection yielding is characterised by yielding of anchor bolts in tension (Point 1), followed by flexural yielding of the baseplate under the compression flange (Point 2).

3. Numerical Study

Monotonic displacement-controlled nonlinear finite element analyses of typical column-to-foundation subassemblages are carried out using ABAQUS 6.12 [12] in order to substantiate the accuracy of the proposed analytical beam-spring model. Ten unstiffened and stiffened connections each, subjected to 3 load cases are analysed. The stiffened connections are designed using *capacity design* concept for a moment demand of 1.2 times the plastic moment capacity of the column (M_{pc}). The baseplate and anchor bolts are designed as per AISC Design Guide [13] and the concrete foundation is designed as per ACI318 [14]. The details of the connections are given in Table 1. Finally, the results obtained from the analytical model are compared with finite element analyses results.

3.1 Geometry

The finite element models of the connection sub-assemblage are created using linear 8-noded solid brick elements (C3D8R) and linear 6-noded solid wedge elements (C3D6). Due to geometric symmetry of the connection, only half the assembly is modelled to reduce computation time (Fig. 9). The concrete foundation block is assumed to be fixed to the ground at the bottom surface (realised by restraining all degrees of freedom of the nodes at the bottom surface). The bolts are assumed to be fixed to the concrete pedestal. This is achieved by merging the nodes along the bolt-concrete interface. A *hard contact*, that allows separation, is specified between the following surfaces:

1. Bottom surface of the baseplate and top surface of the pedestal
2. Bottom surface of the bolt head and top surface of the baseplate
3. Surface of the bolt shaft and inner surface of holes in the baseplate

This facilitates baseplate uplift and bolt elongation during analysis. Moreover, the hard contact ensures that the baseplate transfer the loads to the pedestal and the anchor bolts.

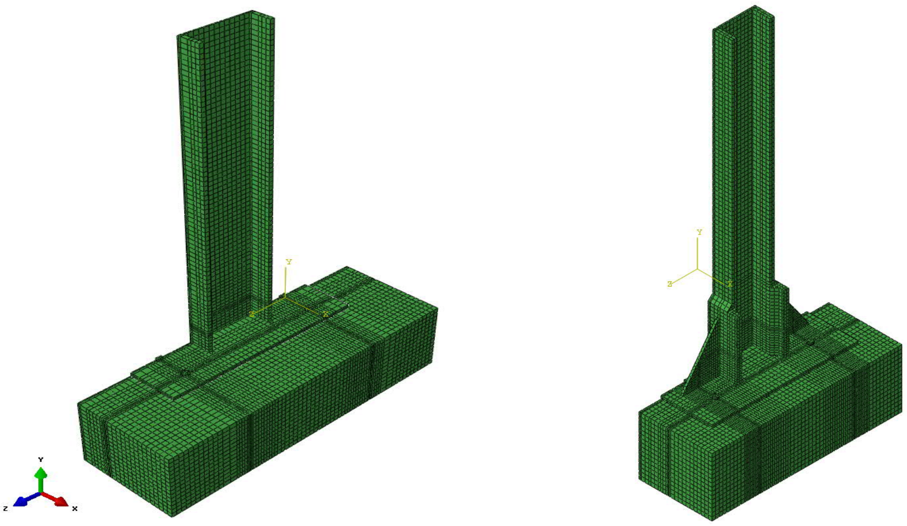


Fig. 9: Finite Element model of a typical unstiffened and stiffened column base connection.

Table 1: Dimensions of the connection components (all dimensions in mm)

Column Section	Unstiffened								Stiffened					
	Pedestal			Baseplate			Anchor Bolts		Cover Plate			Rib Plate		
	l_{ped}	h_{ped}	b_{ped}	l_{bp}	t_{bp}	b_{bp}	l_b	d_b	l_{cp}	t_{cp}	b_{cp}	l_{rp}	t_{rp}	b_{rp}
W10×112	1300	400	1000	1000	22	700	250	22	400	32	400	335	15	250
W12×210	2350	600	1800	1150	35	1000	400	32	420	48	520	390	25	240
W12×336	3050	900	2350	1400	45	1100	550	42	500	75	640	350	40	260
W14×176	1950	500	1400	1300	28	1000	350	28	500	34	540	425	20	330
W14×257	2100	600	1400	1500	32	1000	350	32	550	48	600	490	25	400
W14×455	2950	750	1900	1800	44	1200	500	42	600	82	750	500	40	450
W14×730	3300	750	2000	2200	58	1400	500	56	650	125	950	550	50	490
W16×100	1850	450	1250	1200	25	600	300	22	450	25	380	350	20	270
W21×147	2200	500	1400	1500	32	800	350	32	500	30	440	420	25	350
W27×178	2600	600	1600	1800	38	900	400	36	600	30	480	490	30	400

3.2 Material Properties

Mechanical properties of the connecting steel elements are listed in Table 2. M30 grade concrete (standard cube strength) is used for concrete foundation. Bi-linear stress-strain relationships are assumed for modelling concrete, and anchor bolts.

Table 2: Mechanical properties of steel components

Component	Material	Yield Strength (MPa)	Ultimate Strength (MPa)	Modulus of Elasticity (GPa)	Poisson's Ratio
Column	Steel	345	585	200	0.30
Baseplate		345	585		
Bolts (M20)		640	800		

3.3 Load Cases

The column in each case is subjected to axial compressive load on the top, followed by monotonic lateral displacement at its free end, in increments. Magnitude of axial load and lateral displacement vary depending on the load case; the subassemblages are subjected to 3 different load cases, which are:

(1) Pushover of 80mm (without axial load)
Apply monotonic horizontal displacement of 80mm (4% drift) at the free end of the column in increments.

(2) $0.2P_y$ Axial Load plus Pushover of 80mm
Step 1: Apply a compressive load equal to 0.2 P_y on the top face of the column
Step 2: Apply monotonic horizontal displacement of 80mm (4% drift) at the free end of the column in increments.

(3) $0.4P_y$ Axial Load plus Pushover of 80mm
Step 1: Apply a compressive load equal to 0.4 P_y on the top face of the column
Step 2: Apply monotonic horizontal displacement of 80mm (4% drift) at the free end of the column in increments.

3.4 Results

The following are the observations from finite element analysis of unstiffned and stiffened column-to-foundation anchor bolt connections:

(a) baseplate undergoes significant out-of-plane bending and separates from the underlying concrete at the tension side of the column.
(b) baseplate yields and contributes significantly to the overall lateral deformation of the column free end, while the *column remains elastic* (Fig. 10).
(c) the deformation of anchor bolts and out-of-plane bending of baseplate, leads to high localised stresses in the concrete foundation.

The results obtained from both analytical and numerical models for both stiffened and unstiffened connections have been tabulated in Tables 3 and 4 respectively. Although the average error in estimates of strength is about 16% and 28% for unstiffened and stiffened connections respectively, the estimates of initial stiffness is better at about 16% and 13%.

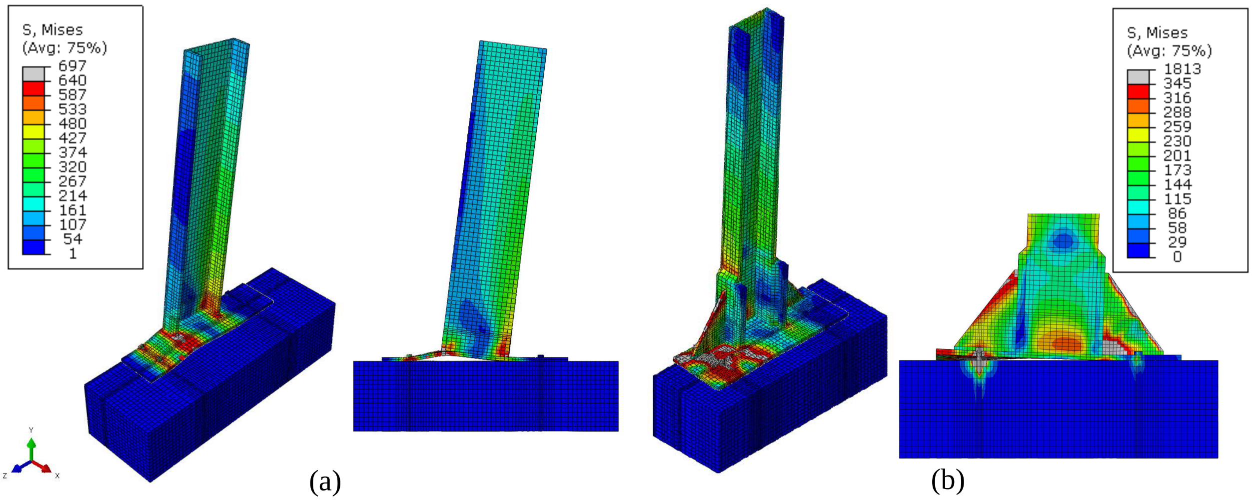


Fig. 10: Finite element analysis results of a typical column (W10×112) with
(a) unstiffened, and (b) stiffened anchor-bolted column baseplate connection.

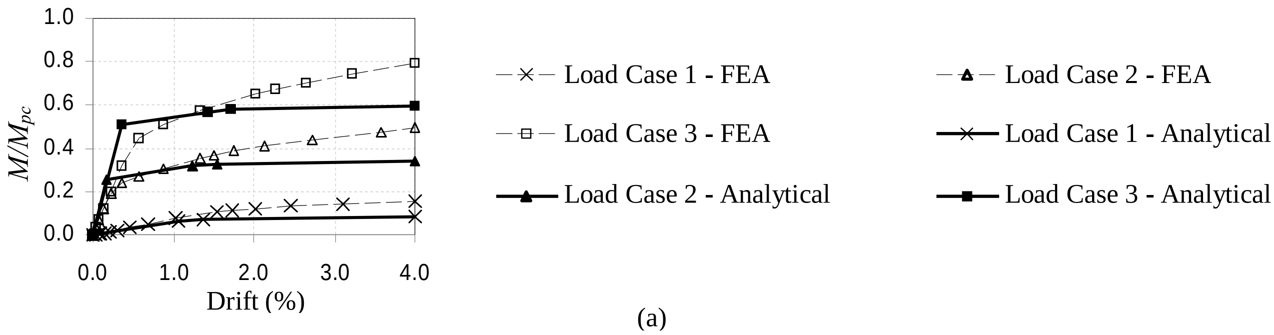
Table 3: Comparison of results obtained from numerical and analytical model: *Unstiffened* connection (Load Case 1 - Zero Axial Load)

Section	Stiffness			Yield Strength		
	Theoretical (kN/m)	**Numerical (kN/m)**	**% error**	**Theoretical (M_{pc})**	**Numerical (M_{pc})**	**% error**
W10×112	2,134	2,477	-13.84	0.07	0.08	-13.36
W12×210	7,786	7,963	-2.22	0.11	0.11	-6.01
W12×336	18,207	18,304	-0.53	0.11	0.10	9.13
W14×176	3,531	4,305	-17.98	0.07	0.08	-9.80
W14×257	4,003	4,961	-19.31	0.06	0.07	-16.64
W14×455	8,859	10,287	-13.88	0.07	0.07	4.27
W14×730	19,140	23,304	-17.87	0.08	0.12	-32.29
W16×100	3,457	3,864	-10.52	0.07	0.12	-44.32
W21×147	5,690	7,861	-27.62	0.08	0.08	-10.61
W27×178	8,369	14,423	-41.97	0.08	0.09	-15.01

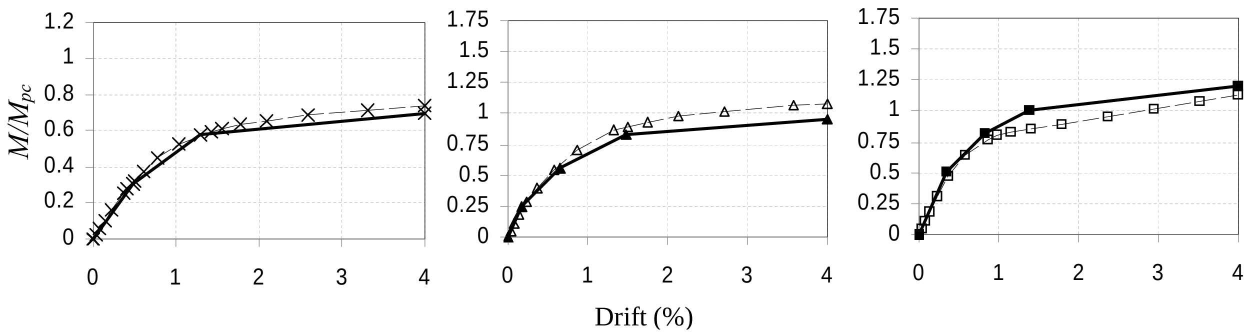
Table 4: Comparison of results obtained from numerical and analytical model: *Stiffened* connection (Load Case 1 - Zero Axial Load)

Section	Stiffness			Yield Strength		
	Theoretical (kN/m)	Numerical (kN/m)	% error	Theoretical (M_{pc})	Numerical (M_{pc})	% error
W10×112	16,902	18,325	-7.77	0.50	0.29	73.63
W12×210	46,697	48,181	-3.08	0.44	0.36	21.97
W12×336	79,617	101,404	-21.49	0.35	0.43	-18.93
W14×176	42,089	38,855	8.32	0.45	0.31	44.25
W14×257	61,146	67,673	-9.64	0.42	0.35	21.11
W14×455	106,470	153,858	-30.80	0.40	0.27	46.61
W14×730	178,338	314,422	-43.28	0.37	0.31	18.53
W16×100	32,372	31,385	3.15	0.55	0.42	28.95
W21×147	66,989	68,344	-1.98	0.51	0.46	10.50
W27×178	114,212	111,948	2.02	0.51	0.50	0.61

The normalised moment versus drift graphs are obtained from the finite element analyses results, corresponding to 3 load cases for all unstiffened and stiffened connections. These are shown together with those obtained from analytical model for a typical connection (Fig. 11). It is demonstrated that the analytical model predicts the stiffness and strength of the connection with reasonable accuracy. In addition, the sequence of failure of the connection components predicted by the analytical model is *consistent* with the finite element analyses results for both unstiffened and stiffened column bases.



(a)



(b)

Fig. 11: Moment versus drift graphs of a typical connection (W14×257) for all 3 load cases obtained from numerical and analytical models: (a) Unstiffened, and (b) Stiffened connections.

In most cases, the model underestimates the strength beyond the second yield point. This is due to bilinear stress-strain used for the model which does not account for strain hardening. Interestingly, the model overestimates the strength in one out of the 10 connections (W10×112). Closer observation of the finite element analysis results revealed that the connection failed by buckling of rib plates, which is not captured by the model.

4. Conclusions

1. Bending and subsequent uplift of baseplate, and elongation of anchor bolts causes significant reduction in stiffness of anchor bolted baseplate connections compared to a fully fixed base. The assumption of a rigid baseplate highly overestimates the strength and stiffness of column base connections.
2. Early yielding of baseplate and anchor bolts cause significant decrease in the connection strength; in most cases, the column remains elastic.
3. The proposed analytical model is capable of providing (a) reasonable estimates of initial stiffness, yield and ultimate strengths of anchor bolted columns baseplate connections, and (b) accurate prediction of the sequence of yielding in different components of such connections.

5. References

[1] FEMA-355E (2000): *State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes*," SAC Joint Venture, CA, USA.

[2] EQE (1995): *The January 17, 1995 Kobe Earthquake: An EQE Summary Report.* EQE International, San Fransisco, USA.

[3] AIJ (1995): *Reconnaissance Report on Damage to Steel Building Structures Observed from the 1995 Hyogoken-Nanbu (Hanshin/Awaji) Earthquake.* Architectural Institute of Japan (in Japanese with English summary).

[4] Hitaka T, Suita K, Kato M (2003): CFT Column Base Design and Practice in Japan, *Proceedings of the International Workshop on Steel and Concrete Composite Construction*, National Center for Research on Earthquake Engineering (NCREE), Taiwan, 281-290.

[5] Kanvinde AM, Grilli DA, Zareuan F (2012): Rotational Stiffness of Exposed Column Base Connections: Experiments and Analytical Models. *Journal of Structural Engineering*, 138(5), 549–560.

[6] Myers AT, Kanvinde AM, Deierlein GG, Fell BV (2009): Effect of weld details on the ductility of steel column baseplate connections. *Journal of Constructional Steel Research*, 65(6), 1366–1373.

[7] Lee KH, Goel SC, Stojadinovic B (2000): Boundary Effects in Steel Moment Connections. *Proceedings in 12th World Conference on Earthquake Engineering*, 12WCEE, Paper no. 1098, Auckland, New Zealand.

[8] Ermopoulos J, Stamatopoulos, G (1996): Mathematical Modelling of Column Baseplate Connections. *Journal of Constructional Steel Research*, 36(2), 79–100.

[9] Wald F, Sokol Z, Steenhuis M, Jaspart JP (2008): Component Method for Steel Column Bases. *Heron*, 53(1), 21-50.

[10] Latour M, Piluso V, Rizzano G (2014): Rotational behaviour of column baseplate connections: Experimental analysis and modeling. *Engineering Structures*, 68(1), 14-23.

[11] Teodoru IB (2009): Beams on Elastic Foundation - The Simplified Continuum Approach. *Bulletin of the Polytechnic Institute of Jassy-Construction Architecture Section*, 55(4), 37–45.

[12] SIMULIA (2013): *Abaqus Analysis User's Manual, Version 6.12.* Dassault Systémes Simulia Corp., Providence, RI.

[13] AISC, (2006): *Design Guide 1: Base Plate and Anchor Rod Design, Second Edition*, American Institute of Steel Construction, Chicago.

[14] ACI 318-14 (2014): *Building Code Requirements for Structural Concrete and Commentary*, American Concrete Institute, Farmington Hills, USA.