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Designing the Column Base Plate of a Steel Industrial Building according to AISC-LRFD Method

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Abstract

Base plates used in steel column-concrete block connections as one of the most important elements in steel structures can influence the total behaviour of structures. Behaviour of base plates as one of the connections that are used in buildings, has its own complexity. The existence of different materials such as steel and concrete, interaction between materials, existence of axial force, shear and moment are the most important problems in analysing these connections. In this study, column-base plate connection design procedure of a steel industrial building according to the AISC Base Plate and Anchor Rod Design Guide 1-LRFD method was studied, the column base plate design procedure was explained for different five load cases. In this study, two dimensional analysis for an industrial building that supports was act as fixed was done in SAP2000 program, then the support reactions that taken from analysis results were used to evaluate column base plate dimensions according to AISC-LRFD procedure and details of column base plate connection was checked in ASDIP-steel program according to AISC-LRFD. As results, the chosen base plate and footing dimensions and anchor bolts were adequate for the design criteria.

Keyword: Steel, Industrial building, Column base plate, LRFD.

INTRODUCTION

Column base connections are critical components in steel structures because they must transfer column forces and bending moments safely to the foundation. Column base plates that used in steel structures may generally be classified into two groups, "exposed column base plates" and "embedded column base plates" [1]. When laterally loaded, exposed base plate used for steel columns bases deform under bending moments and (associated) shear forces mainly by rotations. Behaviour of column base plate connections is of major importance in the overall structural behavior under lateral loading conditions [2]. A lot of reports about exposed column base plates conclude that the exposed base plates must be modeled as a semi-rigid connection in order to more accurately represent the behaviour of frames subjected to important lateral forces [1]. Previous research studies, showed that connections between columns and foundation elements behave in a semi-rigid manner and, in most of the cases, heavily influence the overall structural system [1,3,4]. Typical steel column base with exposed base plate and concrete block connections are shown in Figure 1.

Base plates as one of the most important connection elements in steel structures can be influenced the total behaviour of structures. The existence of different materials such as steel and concrete, interaction between materials, existence of axial force, shear and moment are the most important problems in analysing these connections. The technical analysis of these connections has always had its special complexities, because the large number of parameters involve in the behaviour of column base plates [5]. In this study, a steel industrial building column base plate design was done according to LRFD design procedure that shown in AISC Design Guide 1: Base Plate and Anchor Rod Design [6]. This Guide is based on the 2005 AISC specification for structural steel buildings [7] and includes design guidance in accordance with both "Load and Resistance Factor Design (LRFD)" and "Allowable Stress Design (ASD)". In this study, an industrial building that column bases acts as fixed affected by horizontal and vertical loads and moments, base plate design was done according to AISC Design Guide 1-LRFD method (AISC-LRFD method) based on forces and moments that occurs do to applied loads. Column base plate connections controls were done in ASDIP-steel program according to AISC-LRFD method.

Figure 1. Typical steel column base with base plate and concrete block connection

Design of Steel Column Base Plate According To AISC-LRFD Method

AISC Design Guide 1 provides the design requirements for typical column base plate connections for five different design load cases; "concentric compressive axial loads", "tensile axial loads"," base plates with small moments", "base plates with large moments" and "design for shear". In this study, selected building column base was fixed so base plate under moment effect and tensile axial load design procedure was explained in detail and evaluate base plate dimensions.

Concentric Compressive Axial Loads

When a column base resists only compressive axial loads, the base plate must be large enough to resist the bearing forces transferred from the base plate and the base plate must be of sufficient thickness [6]. The design of column base plate subjected to axial compressive loads only is done according to three cases that are; $A2 = A1$, $A2 \geq 4A1$ and $A1 < A2 < 4A1$,where $A1$ is area of the base plate and $A2$ is maximum area of the portion of the supporting (footing area).

Base Plate With Moment (Small and Large Moments)

In AISC-LRFD method, when base plate affected by moment, the column base design is performed according to small or large eccentricities [6]. In Table 1, general design procedure in AISC Design Guide 1-LRFD method for a base plate under moment effect is shown.

Tensile Axial Loads

In AISC-LRFD method, the design of anchor rods for tension consists of four steps [6]. These are; "determine the maximum net uplift for the column", "select the anchor rod material and the number and size of anchor rods required to resist uplift", "determine the appropriate base plate size, thickness, and welding to transfer the uplift forces" and "determine the method for developing the strength of the anchor rod in the concrete (i.e., transferring the tension force from the anchor rod to the concrete foundation)".

Anchor Rod Tension

The tensile strength of an anchor rod is equal to the strength of the concrete anchorage of the anchor rod group (or those anchor rods participating in tension in the case of tension due to moment) or the sum of the steel tensile strengths of the contributing anchor rods according to the AISC design guide 1.

The limiting tension on an anchor rod is based on the minimum area along the maximum stressed length of that rod. For an anchor rod, this is typically within the threaded portion (except upset rods). ANSI / ASME B1.1 defines the rod threaded area as [6]:

$$A_{ts} = 0.785\left(D - \frac{0.974}{n}\right)^2 \qquad (1)$$

where, n is number of threads per inch, D is major diameter. The nominal tensile strength of an anchor rod according to the AISC Specification stipulates as:

$$R_n = 0.75 F_u A_b \qquad (2)$$

ϕ=0.75 value must be used to obtain the design tensile strength for LRFD;

$$\phi R_n = (0.75)(0.75) F_u A_b = 0.563 F_u A_b \qquad (3)$$

ACI 318-08, Appendix D provides the design tensile strength of an anchor by Eq. (4),

$$\phi R_n = \phi F_u A_{ts} = 0.75 F_{uta} A_{ts} \qquad (4)$$

Where, ϕ = 0.75, A_b = nominal bolt area, in^2, A_{ts} = tensile stress area, in^2 and F_{uta} is lesser of F_u, 1.9F_y and 125 ksi (861.84 MPa).

Concrete Anchorage for Tensile Forces

Base plate design under tensile force effect includes "concrete pullout strength", "concrete capacity design method (breakout strength)" and "development by lapping with concrete reinforcement". ACI concrete pullout strength is based on the ACI 318-08, Appendix D provisions (Section D5.3), [8]. Concrete pullout strength can be determined by Equation (5).

$$\phi N_p = \phi \psi_4 A_{brg} 8 f_c^{'} \qquad (5)$$

In this equation, ϕ = 0.70 and ψ_4 = 1.4 if the anchor is located in a region of a concrete member where analysis indicates no cracking at service levels, otherwise ψ_4 = 1.0. $f_c^{'}$ is specified compressive strength of concrete,psi and A_{brg} is net bearing area of the anchor rod head,in^2.

In the concrete capacity design (CCD) method, the concrete cone is considered to be formed at an angle of approximately 34° (1 to 1.5 slope). The cone is considered to be square rather than round in plan (Figure 2)[6]. According to ACI 318-08 Appendix D, the CCD method is valid for anchors with diameters not exceeding 2 in. (50.8 mm) and tensile embedment length not exceeding 25 in. (635 mm) in depth. The concrete breakout strength for a group of cast-in anchors in normal weight concrete is [8]:

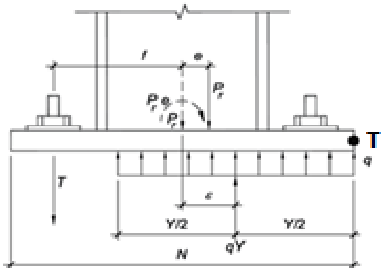
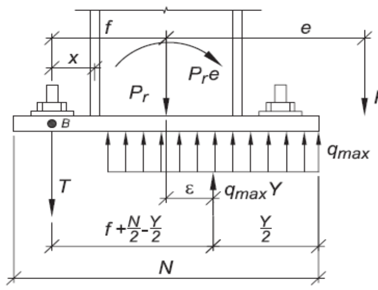
$$\phi N_{cbg} = \phi \psi_3 24 \sqrt{f_c^{'}} h_{ef}^{1.5} \frac{A_N}{A_{No}} \; for \; h_{ef} < 11 \; in (279 \; mm) \qquad (6.a)$$

$$\phi N_{cbg} = \phi \psi_3 \sqrt{f_c^{'}} h_{ef}^{5/3} \frac{A_N}{A_{No}} \; for \qquad (6.b)$$

$$25 \; in \; (635 \; mm) \geq h_{ef} \geq 11 \; in \; (279 \; mm)$$

Where, ϕ = 0.70, ψ_3=1.25 considering the concrete to be uncracked at service loads, otherwise ψ_3=1.0. h_{ef} is depth of embedment,in., A_N is concrete breakout cone area for group, in^2, A_{No} is concrete breakout cone area for single anchor, in^2. In development by lapping with concrete reinforcement the extent of the stress cone is a function of the embedment depth, the thickness of the concrete, the spacing between adjacent anchors, and the location of adjacent free edges in the concrete. The shapes of these stress cones for different situations are shown in Figures 2, 3 and 4. The anchor rod embedment lengths can be defined by the required development length of the spliced reinforcement. Hooks or bends can be added to the reinforcing steel (Figure 5) according to ACI 318-08, Appendix D [6].

Table 1. General design procedure for base plate with moment according to AISC Design Guide 1-LRFD method

Base plate with small moment	**Base plate with large moment**
	
1. Determine the axial load and moment.	1. Determine the axial load and moment.
2. Determine a trial base plate size, N× B.	2. Determine a trial base plate size, N× B.
3. Determine the equivalent and critical eccentricities, Equivalent eccentricity : e = M_r/P_r Critical eccentricity : $e_{crit} = \frac{N}{2} - \frac{P_r}{2\,q_{max}}$ If $e \le e_{crit}$, go to next step (design of the base plate with small moment); otherwise, refer to design of the base plate with large moment.	3. Determine the equivalent and critical eccentricities, Equivalent eccentricity : e = M_r/P_r Critical eccentricity : $e_{crit} = \frac{N}{2} - \frac{P_r}{2\,q_{max}}$ If $e > ecrit$, go to next step (design of the base plate with large moment); otherwise, refer to design of the base plate with small moment. Check the inequality of Equation below; $(f + \frac{N}{2})^2 \ge \frac{2\,P_r(e+f)}{q_{max}}$ If it is not satisfied, choose larger plate dimensions.
4. Determine the bearing length, Y. $Y = N - (2)(e)$	4. Determine the equivalent bearing length, Y and tensile force in the anchor rod, T_u . $Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_r(e+f)}{q_{max}}}$
5. Determine the required minimum base plate thickness $t_{p(req)}$. If $Y \ge m$; $t_{p(req)} = \sqrt{\frac{4\{f_p(\frac{m^2}{2})\}}{0.90\,F_y}} = 1.49m\sqrt{\frac{f_p}{F_y}}$ If $Y < m$; $t_{p(req)} = 2.11\sqrt{\frac{f_p\,Y(m-\frac{Y}{2})}{F_y}}$	5. Determine the required minimum base plate thickness $t_{p(req)}$ at bearing and tension interfaces. Choose the larger value. If $Y \ge m$; $t_{p(req)} = \sqrt{\frac{4\{f_p(\frac{m^2}{2})\}}{0.90\,F_y}} = 1.49m\sqrt{\frac{f_p}{F_y}}$ If $Y < m$; $t_{p(req)} = 2.11\sqrt{\frac{f_p\,Y(m-\frac{Y}{2})}{F_y}}$ Tension interface ; $tp_{(gerekli)} = 2.11\sqrt{\frac{T_u\,x}{B\,Fy}}$
6. Determine the anchor rod size.	6. Determine the anchor rod size.

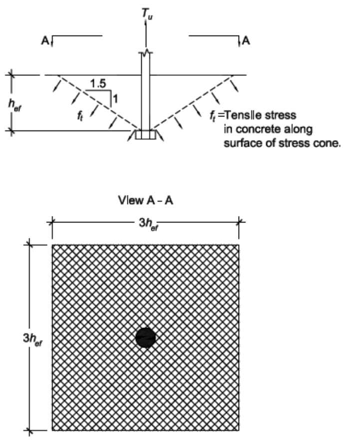


Figure 2. Full breakout cone do to tension force [6]

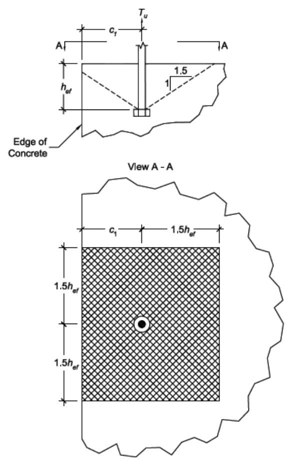


Figure 3. Breakout cone do to tension force near edge in concrete block [6]

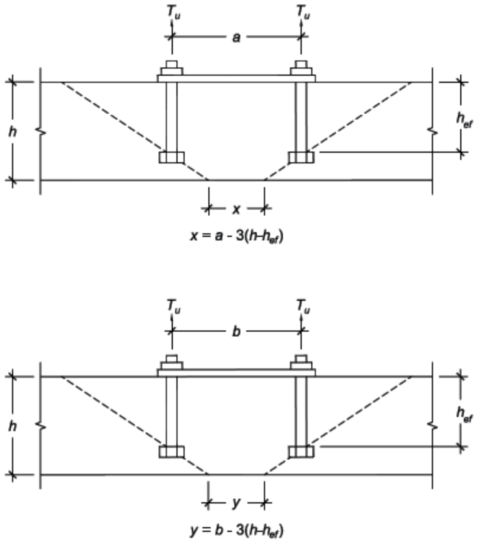


Figure 4. Breakout cone for group anchors [6]

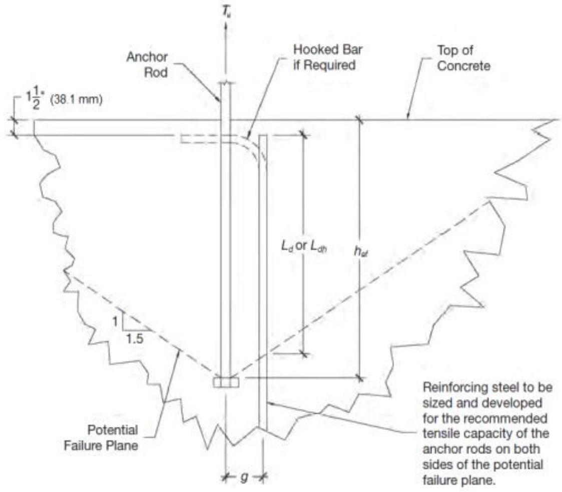


Figure 5. The use of steel reinforcement for in thin slab developing anchor rods [6]

Design for Shear

There are three principal ways for transferring shear effect from column base plates into concrete [6]. They are, "friction between the base plate and the grout or concrete surface", "bearing of the column and base plate, and/or shear lug, against a concrete surface" and "shear in the anchor rods".

Friction between the base plate and the grout or concrete surface

The shear strength can be calculated using Equation (7) in accordance with ACI 318-08 and ACI 349-06 Appendix D criteria,

$\phi V_n = \phi \mu P_u \leq (\phi 0.2\ f_c' A_c$ or $\phi 800 A_c$,whichever is smaller) (7)

For friction between steel base plates and concrete a μ value of 0.4 is given in ACI 349-06[9], Appendix D, $\phi = 0.75$

Bearing of the column and base plate, and/or shear lug, against a concrete surface

ACI 349-06, Section D.4.6.2 recommended the bearing limit as shown in equation below;

$$\phi P_{ubrg} = \phi 1.3 f_c' A_1 \ \textit{and} \qquad (8)$$
$$\textit{for}\ \phi = 0.65,\ \phi P_{ubrg} = 0.8 f_c' A_1$$

In here, A_l is embedded area of the shear lug. For bearing against an embedded base plate or column section where the bearing area is adjacent to the concrete surface, ACI 318-08 recommends that

$$\phi P_{ubrg} = 0.55 f_c' A_{brg} \qquad (9)$$

In here, A_{brg} is contact area between the base plate and/or column against the concrete.

Shear in the anchor rods

For the typical cast-in-place anchor group used in building construction, the shear strength determined by concrete breakout can be evaluated using Equations (10) and (11) according to the AISC Design Guide 1.

$$\phi V_{cbg} = \phi \frac{A_v}{A_{vo}} \psi_5 \psi_6 \psi_7 V_b \ , kips \qquad (10)$$

$$V_b = 7\left(\frac{l}{d_o}\right)^{0.2} \sqrt{d_o} \sqrt{f_c'}\, c_1^{1.5} \text{ for normal weight concrete} \qquad (11)$$

In these equations, ψ_5= 1.0 for all anchors at same load, , ψ_6= a modifier to reflect the capacity reduction when side cover limits the size of the breakout cone, ψ_7=1.4 for uncracked or with adequate supplementary reinforcement and $\phi = 0$. In here, c_1 is the edge distance (in.) in the direction of load

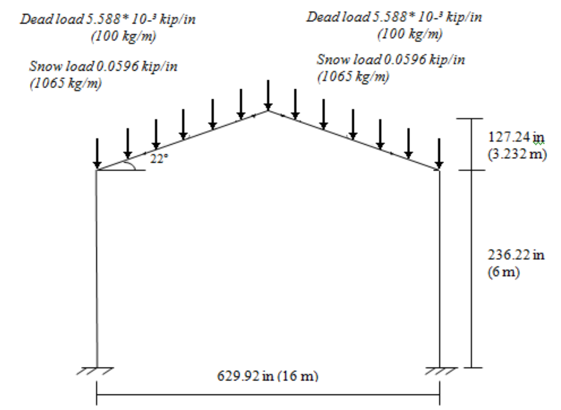
The pryout strength of a single anchor in shear is defined by Equation (12) according to ACI 318-08 [6].

$$\varphi V_{cp} = \varphi\ k_{cp} N_{cb} \qquad (12)$$

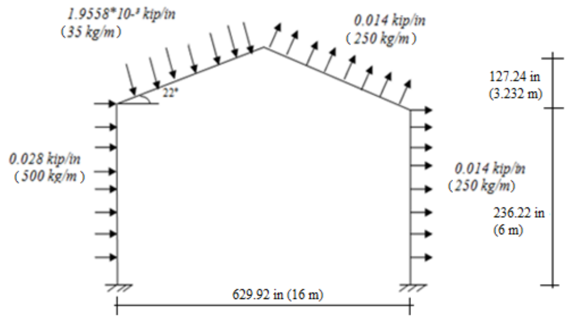
In the Eq. (12) ; φ = 0.70, k_{cp} = 1.0 for h_{ef} < 2.5 in. (63.5mm), k_{cp}=2.0 for $h_{ef} \geq$ 2.5 in. (63.5mm).

Determination of Column Base Plate Dimensions That Used in Steel Industrial Building

In this study, the design of column base plate for a steel industrial building column shown in Figure (6) that had fixed support was evaluated, then checked according to AISC-LRFD method in ASDIP-steel program[10]. The considered loads on industrial building and schematic view of base plate dimensions are shown in Figure (6) and (7), respectively [11]. The column and beam sections for industrial building are, W18 x 119, d=18.97 in (482 mm), b_f=11.265 in (286 mm), steel material properties was taken as Grade 36, yield strength f_y=36 ksi (244.8 MPa) and concrete compressive strength f_c'= 4 ksi (27.2 MPa). In SAP 2000 program [12], the two dimensional analysis for industrial building according to load combinations that used in AISC-LRFD method was done and results for support for the most critical load combination (*1.2Dead+1.6Snow+0.8Wind)* was taken. The support reactions that obtained from analysis results were, moment M_u=*-2350.279 kip.in (-265.55 kN.m)*, vertical reaction P_u= *39.076 kip (173.82 kN)*, horizontal reaction V_u= *- 22.136 kip (- 98.466 kN)*.



(a)



(b)

Figure 6. (a) Dead and snow loads distribution, (b) wind load distribution on two dimensional industrial building

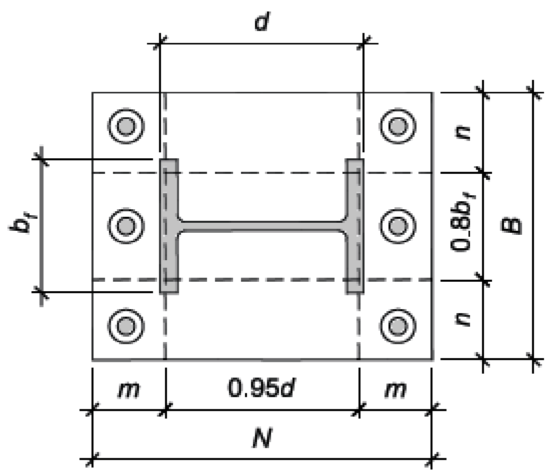


Figure 7. Schematic view of steel column base plate assumed bending lines

Base Plate Dimensions and Thickness Calculation

The support reactions that obtained from analysis results used to calculate the base plate dimensions according to AISC design guide 1-LRFD method, as below

*N > d+ 2 (3 in) ; N> 18.97+ 2*3 = 24.97 in (634.2 mm)*
*B > bf + 2 (3 in); B> 11.265 + 2*3 = 17.267 in (438.5 mm)*

Trial base plate size *N x B = 30 x 25 in (762 x 635 mm).*
Eccentricity:

$$e=\frac{M_u}{P_u}=\frac{2350.279}{39.076}=60.15\ in.\ (1527.8\ mm)$$

$$f_{p(\max)}=\phi_c(0.85f_c^{'})\sqrt{\frac{A_1}{A_2}}=0.65*0.85*4*1=2.21\ ksi$$
$$(15.028\ MPa)$$

$$q_{(\max)}=f_{p(\max)}.B=2.21*25=55.25\ kip/in\ (9.675\ kN/mm)$$

$$e_{crit}=\frac{N}{2}-\frac{P_u}{2q_{\max}}=\frac{30}{2}-\frac{39.076}{2*55.25}=14.65\ in\ (372.11\ mm)$$

$e > e_{crit}$, so base plate is at large moment effect. The anchor rod edge distance was assumed *2.5 in. (63.5 mm).*

According to AISC design guide 1-LRFD design procedure, the calculation controls were done and the chosen dimensions were found sufficient as shown below.

$$f=\frac{N}{2}-2.5=\frac{30}{2}-2.5=12.5\ in\ (317.5\ mm)$$

$$\left(f+\frac{N}{2}\right)^2=\left(12.5+\frac{30}{2}\right)^2=756.25\ in^2(487902.25\ mm^2)\geq$$
$$\frac{2P_u(e+f)}{q_{\max}}=\frac{2*39.076*(60.15+12.5)}{55.25}=102.76\ in^2$$
$$(66296.64\ mm^2)$$

At large moment effect according to AISC Steel Design Guide 1 the tensile force T_u that must transfer by anchor bolt was calculated as shown below.

$$Y=f+\frac{N}{2}\pm\sqrt{\left(f+\frac{N}{2}\right)^2-\frac{2P_u(e+f)}{q_{\max}}}=$$
$$\left(12.5+\frac{30}{2}\right)\pm\sqrt{\left(12.5+\frac{30}{2}\right)^2-\frac{2*39.076(60.15+12.5)}{55.25}}=$$
$$27.5\pm25.56=53.06\ in\ (1347.7\ mm)$$
$$=1.94\ in\ (49.3\ mm)$$

$$T_u=q_{\max}Y-P_u=(55.25*1.94)-39.076=68.109\ kip$$
$$(302.96\ kN)$$

Base plate thickness was calculated according to bearing and tension interfaces and chosen the largest one.

<u>a) Thickness calculation at bearing interface:</u>

$$m=\frac{N-0.95d}{2}=\frac{30-(0.95*18.97)}{2}=5.99\ in\ (152.1\ mm)$$
$$f_p=f_{p(\max)}=2.21\ ksi\ (15.028\ MPa)$$

$$for\ Y<m:$$

$$t_{p(req)}=2.11\sqrt{\frac{f_{p(\max)}.Y.(m-Y/2)}{F_y}}=$$
$$2.11\sqrt{\frac{2.21*1.94*(5.99-1.94/2)}{36}}=1.63\ in(41.4\ mm)$$

b) Thickness calculation at tension interface:

$$X = \frac{N}{2} - \frac{d}{2} + \frac{t_f}{2} - 2.5 = \frac{30}{2} - \frac{18.97}{2} + \frac{1.06}{2} - 2.5 = 3.545 \ in \quad (90.04 \ mm)$$

$$t_{p(req)} = 2.11\sqrt{\frac{T_u.x}{B.F_y}} = 2.11\sqrt{\frac{68.109*3.545}{25*36}} = 1.09 \ in \quad (27.69 \ mm)$$

The thickness was checked using the value of n.

$$n = \frac{B - 0.8b_f}{2} = \frac{25-(0.8*11.265)}{2} = 7.994 \ in \ (203 \ mm)$$

$$t_{p(req)} = 2.11\sqrt{\frac{f_{p(\max)}.Y.(n - Y/2)}{F_y}} =$$

$$= 2.11\sqrt{\frac{2.21*1.94*(7.994-1.94/2)}{36}} = 1.93 \ in \ (49 \ mm)$$

According to thickness calculation at bearing and tension interfaces results, the base plate thickness was taken as 2 in. (50.8 mm).

Determination of Anchor Bolt Size

According to base plate dimensions and the tension force that must be carried by anchor bolt, three anchor bolts were used on each face of the column. From above calculation tensile force that effect the base plate T_u= *68.109 kip (302.96 kN),* the force per rod = *22.703 kip (100.988 kN)*. From AISC Steel Design Guide 1, Table 3.1 for ASTM F1554 Grade 36 steel and *1 1/4 in (31.75 mm)* diameter anchor bolt available tensile strength= *40 kip (177.92 kN)*, from this table the hole size for *1 1/4 in (31.75 mm)* diameter anchor bolt was *2 1/16 in (52.4 mm)*. From this table, for $f_{c'}$ *=4,000 psi (27.2 MPa)* and *1 1/4 in (31.75 mm)* diameter anchor bolt, the anchor bolt concrete pullout strength was determined as *50.2 kip (223.29 kN)*. This shows that the chosen anchor bolt carried force *50.2 kip (223.29 kN)* was greater than the force that anchor rod must be transferred it *22.703 kip (100.988 kN)*, so the anchor rod size was sufficient.

Addition to base plate dimensions and anchor bolt sizes determination, anchor bolt embedment length= *20 in.(508 mm)*, concrete block thickness= *40 in (1016 mm)* and concrete block dimensions *(80 x 80)in. (2032 x 2034 mm)* were selected.

Design in ASDIP-STEEL Program

ASDIP is an engineering software program using for steel, concrete and footing design. In ASDIP Steel 3 version 3.5.1 program, AISC 360 (ASD (Allowable Stress Design), LRFD (Load and Resistance Factor Design)) and ACI 318 Appendix D codes are used for steel element design. In this study, the steel industrial building column base which initial dimensions was calculated above was checked according to AISC-LRFD in ASDIP-steel. The obtained results details are shown in Table 2 to 6. In Figure 8, load distribution under base plate, tension breakout area and shear breakout area determined by ASDIP are shown.

Table 2. Base plate design details for axially loaded plate

Axially loaded plates	
Bearing stress f_p = *P/(W*L)* = 39.1/(25*30) = 0.1 ksi (0.689 MPa)	
Bearing strength $F_p = 0.85*fc'\sqrt{\frac{A_1}{A_2}} = 0.85*4\sqrt{\frac{6000}{750}}$ = 6.8 ksi (46.88MPa)	ACI 10.14.1
Under –strength factor Φ = 0.65	ACI 9.3.2.4
$\textbf{Bearing strength ratio} = \frac{f_p}{\phi F_p} = \frac{0.1}{0.65*6.8} = \mathbf{0.01 < 1.0\ OK}$	
Critical section *m* = 0.5*(*L*-0.95**d*)=0.5*(30-0.95*19)=6 in (152.4 mm) Critical section *n* = 0.5*(*W*-0.80*b_f) = 0.5*(25-0.80*11.3) = 8 in (203.2 mm)	AISC-DG#13.1.2
$X = \left[\frac{4*d*b_f}{(d+b_f)^2}\right]*Bearing\ \ ratio = \left[\frac{4*19*11.3}{(19+11.3)^2}\right]*0.01 = 0.01$	AISC-DG#13.1.2
$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} = \frac{2\sqrt{0.01}}{1+\sqrt{1-0.01}} = 0.11$	
$n' = 0.25\sqrt{d.b_f} = 0.25*\sqrt{19*11.3} = 3.7$ in (93.98 mm)	
Controlling section k = Max (*m* , *n* , λ_n') = Max (6, 8, 0.11*3.7) = 8 in (203.2 mm)	
Plate moment $M = f_p * k^2/2$ = 0.1 * 8^2/2 = 1.7 k.in/in (19.21 KN.mm/mm)	
Plate thickness $t = k\sqrt{\frac{2.f_p}{P_{hi}.f_y}} = 8*\sqrt{\frac{2*0.1}{0.9*36}} = 0.45\ in\ \ (11.4\ mm)$	AISC-DG#13.1

Table 3. Base plate design details for plate under moment

Base plate with moment	
Blodgett Method	
Eccentricity $e=M/P$=195.9*12/39.1=60.1 in (152.65cm) > (L-Rod offset)/3= (30-12.5)/3 = 5.8 in (147.3 mm)	
Factor $k1$ = 3*(e – $L/2$) = 3*(60.1 - 30/2) = 135.4	
Factor $k2$ = 6 * n * Tension rods * Area/ W * (Rod offset + e) = 6* 8*3*1.23/25*(12.5+60.1) = 515.9	
Factor $k3$ = -$k2$ * ($L/2$ + Rod offset) = -515.9 *(30/2 + 12.5) = -14188.0	
Solving the polynomial $Y^3 + k1 * Y^2 + k2 * Y + k3 = 0$	
Bearing length Y=8.3 in (210.8 mm)	
Tension $T = -P*[(L/2 – Y/3 - e) / (L/2 – Y/3$ + Rod offset)] T = -39.1*[(30/2-8.3/3-60.1) / (30/2 – 8.3/3 + 12.5)] = 75.7 kip (336.73 kN)	
Max bearing stress $f_p = \frac{2*(P+T)}{Y+W} = \frac{2*(39.1+75.7)}{8.3*25} = 1.1$ ksi (7.584 MPa)	
Bearing at critical section f_{p1}=(Y-m)*f_p/Y= (8.3-6)*1.1/8.3=0.3 ksi (2.068 MPa)	
Moment due to bearing M_b=0.5*[f_{p1}*m^2 + m^2*2/3*(f_p - f_p1) M_b=0.5*[0.3*6^2 + 6^2*2/3*(1.1 – 0.3)]= 15 k.in/in (169.48 kN.mm/mm)	
Moment due to tension M_t=T*[m-($L/2$ – Rod offset)] / [2*(m-($L/2$-Rod offset))] M_t=25.2*[6-(30/2-12.5)]/[2*(6-(30/2-12.5))]= 12.6 k.in/in (142.36 kN.mm/mm)	
Plate thickness $t = \sqrt{\frac{4.M_c}{P_{hi}.f_y}} = \sqrt{\frac{4*15}{0.9*36}} = 1.36$ in (34.5 mm)	AISC-DG#13.1.2

Table 4. Base plate design details for anchorage design

Anchorage Design
Rod material specification……A36
6 Rods , f_{ya} = 36 ksi (244.8 MPa), f_{uta} = 58 ksi (394.4 MPa)
Anchor rod siz 1-1/4'' diam. $\times$20 in emb. A_{se}= 0.97 in² (626 mm²), A_{brg} = 2.24 in² (1445 mm²)

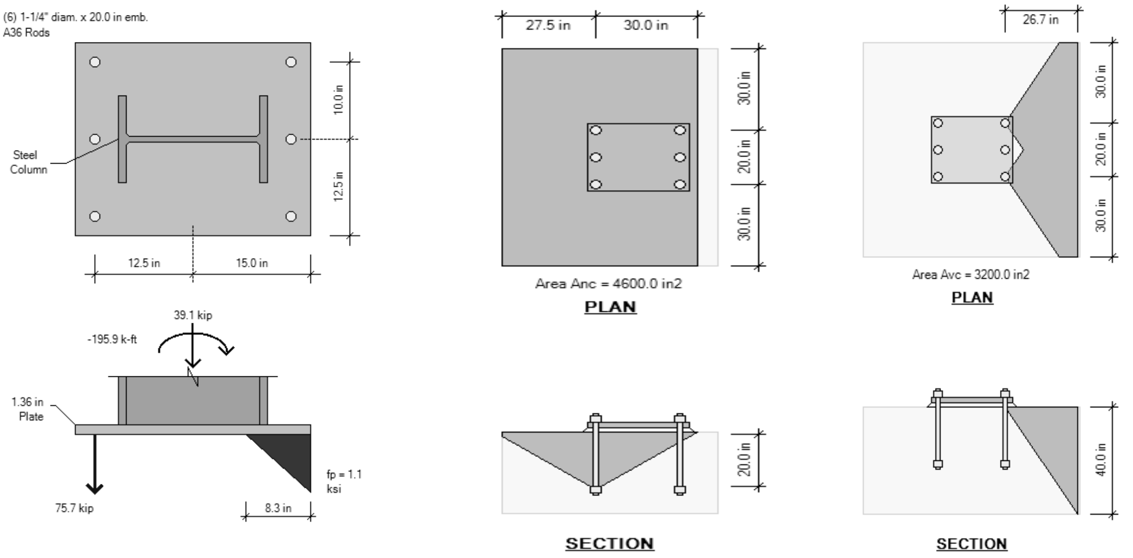
Table 5. Base plate design detail for anchorage design in tension force effect

Tension Analysis	ACI D.5
Total tension force N_u = 75.7 kip (336.73 kN). # of tension rods = 3 Tension force per rod N_{ui} = 25.2 kip (112.095 kN)	
-Steel strengt.h of anchors in tension	**ACI D.5.1**
steel strength $N_{sa} = A_{se} * f_{uta}$ = 0.969 * 58 = 56.2 kip (249.99 kN)	ACI Eq.(D-2)
Under-strength factor Φ = 0.75	ACI D.4.3
Steelstrength ratio $= \frac{N_{ui}}{\phi N_{sa}} = \frac{25.2}{0.75*56.2} = \mathbf{0.60 < 1.0}$ **OK**	**ACI D.4.1.1**
-Concrete breakout strength of anchors in tension	
No reinforcing bars provided	ACI D.5.2
Effective embedment h_{ef} = 20 in (50.8 cm)	ACI D.5.2.3
Anchor group area $A_{nc} = (C_a + C_{b1})*(C_a + S_a + Cb_1)$	
A_{nc}= (30 + 27.5) * (30 + 20 +27.5) = 4600 in^2 (29677.36cm²)	ACI D.5.2.1
Single anchor area $A_{nco}=9\ h_{ef}^2 = 9*(20)^2$ = 3600 in^2 (23225.76 cm²)	Eq.(D-5)
Single anchor strength N_b= $24\sqrt{f_c'}\ .\ h_{ef}^{1.5} = 24\sqrt{4000}\ *20^{1.5}$ =135.8 kip (604.068 kN)	Eq. (D-6)
Eccentricity factor Ψ_{ec}= 1.00 (No eccentric load)	ACI D.5.2.4
Edge effects factor $\psi_{ed} = 0.7 + 0.3*\frac{C_{a,\min}}{1.5.h_{ef}} = 0.7 + 0.3*\frac{27.5}{1.5*20} = 0.98$	ACI D.5.2.5
Cracking factor Ψ_{cn} = 1.25 (Uncracked concrete at service load level)	ACI D.5.2.6
Breakout strength $N_{cbg} = \frac{A_{nc}}{A_{nco}}.\psi_{ec}.\psi_{ed}\psi_{cn}.N_b$ $N_{cbg} = \frac{4600}{3600}*1.00*0.98*1.25*135.8 = 211.4$ kip (940.35 kN)	Eq.(D-4)
Under –strength factor Φ = 0.70	ACI D.4.3
Breakout strength ratio $= \frac{N_u}{\phi N_{cbg}} = \frac{75.7}{0.70*211.4} = \mathbf{0.51 < 1.0}$ **OK**	**ACI D.4.1.1**
Breakout strength ratio controls (0.51 < 1.51)	ACI D.5.2.9

-Concrete pullout strength of anchors in tension	**ACI D.5.3**
Single anchor strength N_p= 8 A_{brg} f_c´= 8*2.24*4=71.6 kip(318.49 KN)	ACI Eq(D-14)
Cracking factor Ψ_{cp} =1.40 (Uncracked concrete at service load level)	ACI D.5.3.6
Pullout strength N_{pn}= Ψ_{cp} N_p = 1.40*71.6 = 100.2 kip (445.71 KN)	ACI Eq(D-13)
Under –strength factor Φ = 0.70	ACI D.4.3
$\textbf{Pullout strength ratio} = \frac{N_u}{\phi N_p} = \frac{25.2}{0.70*100.2} = 0.36 < \mathbf{1.0\ OK}$	**ACI D.4.1.1**
-Concrete side-face blowout strength of anchors in tension	**ACI D.5.4**
Side –face blowout N_{sbg} = N.A. (Embed < 2.5Ca .20<2.5*27.5=68.8)	ACI D.5.4.1
$\textbf{Tension design ratio} = \frac{N_u}{\phi N_n} = \mathbf{0.60 < 1.0\ OK}$	**ACI D.4.1.1**

Table 6. Base plate design details for anchorage design in shear force effect

Shear Analysis	ACI D.5
Shear resisted by anchor rods only (Anchor rods are not welded to the base plate) Total shear force V_u= 22.1kip (98.305 kN). Shear per rod V_i= 7.4kip (32.92 kN) (Only front rods are effective)	
-Steel strength of anchor rods in shear	
Steel strength V_{sa}=0.6*A_{se}*f_{uta}*groutfactor=0.6*0.97*58*0.8=27kip(120.10 kN)	ACI D.6.1.2
Under –strength factor Φ = 0.65	ACI D.4.3
$\textbf{Steel strength ratio} = \frac{V_i}{\phi V_{sa}} = \frac{7.4}{0.65*27} = \mathbf{0.42 < 1.0\ OK}$	**ACI D.4.1.1**
-Concrete breakout strength of anchors in shear	ACI D.5.2
No reinforcing bars provided	
Anchor group area A_{vc} = (1.5*Ca_1)*(Ca_2 + Sa + Cb)	
A_{vc} = (1.5*26.67)*(30 + 40 + 30) = 3200 in^2 (20645.12cm²)	ACI D.6.2.1
Single anchor area A_{vco}=4.5*Ca^2 =4.5*$(26.67)^2$ =3200in^2 (20645.12cm²)	Eq.(D-32)
Single anchor strength $V_b = 7.\left[\frac{f_e}{d_a}\right]^2.\sqrt{d_a}.\sqrt{f_c^{'}}.C_a^{1.5}$	
$V_b = 7*\left[\frac{10}{1.2}\right]^2 * \sqrt{1.2} * \sqrt{4000} * 26.7^{1.5} = 78.4$ kip (348.74 kN)	Eq.(D-33)
Eccentricity factor Ψ_{ec} = 1.00 (No eccentric load)	ACI D.6.2.5
Edge effects factor $\psi_{ed} = 0.7 + 0.3\frac{c_a}{1.5c_a} = 0.7 + 0.3*\frac{30}{1.5*26.7} = 0.93$	ACI D.6.2.6
Cracking factor Ψ_{cv} = 1.4 (Uncracked concrete at service load level)	ACI D.6.2.7
Thickness factor Ψ_{hv} = 1.0	ACI D.6.2.8
Breakout strength $V_{cbg} = \frac{A_{vc}}{A_{vco}}\psi_{ec}.\psi_{ed}.\psi_{cv}.\psi_{hv}.V_b$ $V_{cbg} = \frac{3200}{3200}*1.00*0.93*1.40*1.00*78.4 = 101.5$ kip (451.49 kN)	Eq.(D-31)
Under –strength factor Φ = 0.70	ACI D.4.3
$\textbf{Breakout strength ratio} = \frac{V_u}{\phi V_{cbg}} = \frac{22.1}{0.70*101.5} = \mathbf{0.31 < 1.0\ OK}$	**ACI D.4.1.1**
Breakout strength ratio controls (0.31 <0.68)	ACI D.6.2.9
-Concrete pryout strength of anchors in shear	
Pryout strength V_{cpg} = 2.0*V_{cbg} = 2.0 * 101.5 = 422.8 kip (1880.71 KN)	ACI D.6.3.1
Under-strength factor Φ = 0.65	ACI D.4.3
$\textbf{Pryout strength ratio} = \frac{V_u}{\phi V_{cpg}} = \frac{22.1}{0.65*422.8} = \mathbf{0.07 < 1.0\ OK}$	**ACI D.4.1.1**
$\textbf{Shear design ratio} = \frac{V_u}{\phi V_n} = \mathbf{0.42 < 1.0\ OK}$	**ACI D.4.1.1**
-Tension-Shear interaction	
Combined stress ratio = $[(\text{tension ratio})^{1.67} + (\text{shear ratio})^{1.67}]^{0.6}$	
Combined stress ratio =$(\mathbf{0.60^{1.67} + 0.42^{1.67}})^{\mathbf{0.6}}$= 0.78 < 1.0 OK	**ACI RD.7**
Anchorage Design is Ductile.	



(a) **(b)** **(c)**

Figure 8. ASDIP-steel results (a) load distribution under base plate in ASDIP program (b) tension breakout area, (c) shear breakout area

CONCLUSIONS

A steel column base consists of a column, a base plate, concrete block and an anchoring assembly. In steel structures, column bases are critical components that must transfer loads from building into foundation system. In general, they are designed with unstiffened base plates, but stiffened base plates may be used where the connection is required to transfer high bending moments. In this study, column base plate design procedure was explained based on AISC design guide 1-LRFD method and ACI-318 code. Loaded steel industrial building column base plate dimensions were calculated and the required plate thickness was found as 2 inch (50.8 mm). Then, the obtained base plate dimensions and anchorage members were checked in ASDIP-Steel program according to AISC-LRFD and ACI318 code. In this program; plate thickness, maximum bearing stress, bearing strength and design ratio were calculated. The design was obtained ductile, anchorage and base plate design were adequate the design criteria and reliable results were obtained.

NOTATIONS

$A1$: Area of the base plate
$A1$: Embedded area of the shear lug (design for shear)
$A2$: Maximum area of the portion of the supporting
A_{brg}: Contact area between the base plate and/or column against the concrete
A_b : Nominal bolt area
A_{ts} : Tensile stress area
A_{No}, A_{nco}: Concrete breakout cone area for single anchor
A_N, A_{nc} : Concrete breakout cone area for group
A_v, A_{vc} :The total breakout shear area for a single anchor,or a group of anchors
A_{vo},, A_{vco}: The area of the full shear cone for a single anchor or a group of anchors
A_c : Area of concrete section resisting shear transfer
A_{se} : Effective cross-sectional area of anchor in shear or tension
B : Base plate width
b_f : Column flange width
c_1 : The edge distance in the direction of load
C_a : Distance from the center of an anchor shaft to the edge of concrete in one direction
C_{b1} : Smaller of: the distance from center of a bar or wire to nearest concrete surface, and one-half the center-to-center spacing of bars or wires being developed
d : Overall column depth
d_o : The rod diameter
d_a : Outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt
e : The eccentricity
e_{cri} : The critical eccentricity
F_u : Factored force acting in a strut, tie, bearing area, or nodal zone in a strut-and-tie model
f_p : Bearing stress between the plate and concrete
f_{p1} : Bearing stress at critical section
F_p : Bearing strength
$f_{c'}$: Specified compressive strength of concrete
F_y : Specified yield stress of base plate
f_{uta} : Specified tensile strength of anchor steel
h_{ef} : Depth of embedment
ℓ : Embedment depth
L_d : Development length in tension of deformed bar, deformed wire, plain and deformed welded wire reinforcement, or pretensioned strand
L_{dh} : Development length in tension of deformed bar or deformed wire with a standard hook, measured from critical section to outside end of hook
M_r : Bending moment M_u according to Drake and Elkin assumption
M_u : The factored bending moment
M_b : Moment due to bearing
M_t : Moment due to tension
N : Base plate length
N_{cbg}: Concrete breakout strength for a group of anchors
N_n : Nominal tension force
N_{sa} : Steel strength
N_{rg} : Rebars strength
N_{sbg} : Side-face blowout strength of a group of anchors
N_u : Total tension force
N_{ui} : Tension force per rod
N_{cb} : Nominal concrete breakout strength in tension of a single anchor

N_b : Basic concrete breakout strength in tension of a single anchor in cracked concrete
N_p : Pullout strength in tension of a single anchor in cracked concrete
N_{pn} : Nominal pullout strength in tension of a single anchor
n' : Yield-line theory cantilever distance from column web or column flange
P_r : Axial force P_u according to Drake and Elkin assumption
P_u : The factored axial compressive force
q_{max}: Maximum bearing force
q_Y : Resultant bearing force
S_a : Center-to-center spacing of bolts
T_u : Tensile force in the anchor rod
$t_{p(req)}$: Minimum plate thickness
V_u : The factored shear force
V_n : Nominal shear strength
V : Total shear force or shear force per rod
V_{sa} : Steel strength
V_{rg} : Rebars strength
V_b : Basic concrete breakout strength in shear of a single anchor in cracked concrete
V_{cbg}: Nominal concrete breakout strength in shear of a group of anchors
V_{cpg}: Nominal concrete pryout strength of a group of anchors
Y : Bearing length
P_{ubrg}: The bearing limit (design for shear)
ϕ : Strength reduction factor
μ : The friction coefficient

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REFERENCES

[1] Grauvilardell J. E., Lee D., Hajjar J. F. and Dexter R.J., 2005. Synthesis of Design, Testing and Analysis Research on Steel Column Base Plate Connections in High-Seismic Zones, Struct. Engng. Report No. ST-04-02, Dept. of Civil Engng., Univ. of Minnesota, USA, October.

[2] Melenciuc, S.C., Ştefancu, A-I, Budescu, M., 2011. Experimental Studies on the Behavior of Steel Column Base Connections, Bulletin of the Polytechnic Institute of Jassy, Construction, Architecture Section, LVII (LXI)-(3): 149-156.

[3] Hamizi M. and Hannachi N.E., 2007. Evaluation by a Finite Element Method of the Flexibility Factor and Fixity Degree for the Base Plate Connections Commonly Used, Strength of Materials, 39 (6): 588-599.

[4] Stamatopoulos G.N., Ermopoulos J.Ch., 2011. Experimental and Analytical Investigation of Steel Column Bases, Journal of Construct. Steel Research, 67 (9): 1341-1357.

[5] Shafieifar, M.R., Khonsari, S.V., 2012. Studying the Behaviour of Base Plates with High Degree of Rigidity, 15th WCEE, Lisbon, Portugal, 24-28.09.2012.

[6] Fisher, J.M. and Kloiber, L.A., 2010. AISC Steel Design Guide 1:,Base plate and Anchor Rod Design (Second Edition, second printing).

[7] ANSI/AISC 360-05, 2005. Specification for Structural Steel Buildings, American Institute of Steel Construction, Chicago, Illinois, March.

[8] ACI318-08, 2008. Building Code Requirements for Structural Concrete and Commentary, ACI318-08/ACI 318R-08, American Concrete Institute, Farmington Hills, MI.

[9] ACI 349-06, 2006. Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary, ACI 349-06, American Concrete Institute, Farmington Hills, MI.

[10] ASDIP STEEL 3, V 3.5.1 program, http://www.asdipsoft.com/products/steel/.

[11] Hussein, P.S.H., 2015. Investigation of Steel Column Base Plate According to LRFD and EuroCode 3 Codes, graduate seminar, Selcuk University, Konya, Turkey (in Turkish).

[12] CSI. SAP2000.V-15. Integrated finite element analysis and design of structures basic analysis reference manual. Berkeley (CA. USA): Computers and Structures Inc.