CONNECTION DESIGN CALCULATION FOR STEEL BEAMS TO CONCRETE

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Abstract - *The most structural failures have been due to some form of connection failure. Steel connection have a direct influence on the cost of framing system. Under General it's classified as bolted & welded also according to load distribution and geometry its moment & shear connection. This project "design of embed connection & fin plate connection "and manual design is done as per SS EN 1993-1-1:2005 & SS EN 1993-1-8:2005.In this paper, a review of other literatures is done on the basis of connection .One of the most important aspects that one needs to take into count when designing a steel structure is the dissipative mechanism of the structure as well as the structural properties of the connection. A manual design can only be done for connection in steel structures for better understanding.*

Key Words: Embed plate connection, Fin plate connection

1.INTRODUCTION

The design process of structural Design requires imagination and conceptual thinking, and sound knowledge of structural engineering science besides the knowledge of practical aspects, such as recent design codes, goodbye laws, backed up by ample experience, intuition, and judgment. The purpose of standards is to ensure and enhance safety, keeping a careful balance between economy and safety. The process of Design commences with the planning of the structure, primarily to meet its functional requirements. Initially, the requirements proposed by the client are taken into consideration. They may be vague, ambiguous, or even unacceptable from an engineering point of view.

1.1 LITERATURE REVIEW:

1. Mr. Patil Rohan Shantinath, Mr. Naveen Kumar H.S, Mr. Akshayakumar V.H- comparative study of design, scheduling and embodied energy of RC and Steel structure is included.

2. Rohan S. Mutnal - The conception of design analysis as well as modeling of steel structures All the structural components were designed manually.

3. Pawooskar Rohit Satish, Vaijanath A Chougule- Design and Finite Element Analysis is then compared between the American Steel sections and the Indian Steel Sections to understand whether the Indian Steel Sections conform to the American Codal Provisions.

1.2 Flexible concept:

In flexible concept the shear capacity of bolts are checked for axial and shear force and for moment due to vertical shear. The eccentricity for moment is taken from thebolt C.G to face of embed.

- Fin plate is verified for bearing, block shear, compression, tension and flexural capacities.
- Beam web is verified for bearing, block shear, compression, tension and flexural capacities.
- Weld for fin plate is verified to transfer shear, axial force and moment due to localeccentricity of vertical shear

1.3 Embed plate and rebar / stud connection

- Axial force, shear force, major and minor axis bending and induced moment due to vertical and horizontal eccentricities are considered for embeds.
- Embed plate is checked for vertical shear, horizontal shear and major and minor axis bending and induced moment due to vertical and horizontal eccentricities.
- According to ACI 349-2R.07 embed plate major and minor axis bending due to shear stud tension is checked.
- Rebar / Shear studs are checked for forces transferred through embed plate.
- The concrete is checked for shear & tension forces.

1.3 Beam –Beam Web connection

Force transfer assumption

- Axial Force & vertical shear force are transferred to web.

The following checks are verified in the connection design as per codal provision,

- Bolts in the web are checked for shear due to axial force, vertical shear force & moment due to local eccentricity of vertical shear

- Beam web & Fin plates are verified for bearing, block shear, compression, tensionand flexural capacities.
- Weld for fin plate is verified for transfer shear, axial force and moment induced

2. SHEAR CONNECTION USING FIN PLATE - BEAM TO EMBED PLATE

All structural steelwork connections plate for connections as indicated in the connection details, with S355JR in accordance with BS EN 10025

- Structural steel grade (For all Steel members & connection plates): S355 JR
- Poisson's Ratio, η 0.30
- Young's Modulus, E 2.05X105 MPa
- Coefficient of Thermal expansion, α $12x10^{-6}$ /°C
- Concrete grade : C40 / 50 – Column & Core wall C32 / 40 – RC Beams
- Poisson's Ratio, η 0.20
- Young's Modulus, E 5000 √ fck.

  Coefficient of Thermal expansion, α 10x10-6 /°C
- Electrode class for all welding: E42(as per BS) / E70xx(as per AWS D1.1)

Fasteners:

Non-Preloaded bolts (at bearing connections); Gr. 8.8 Bolt and Gr 8. 8 Nut conforming to BS 4190, washer conforming to BS 4320. Or approved equivalent.

3. SHEAR CONNECTION USING FIN PLATE - BEAM TO EMBED

Supported Beam **UB356X254X122**	
Depth df	363.2 mm
Breadth bf	256.5mm
Thickness of flange tf	21.7mm
Thickness of web tw	13mm
Root radius r	15.1mm
d'	289.6 mm
Area a	154.8 cm^2
Skewed Angle	90 degree

Beam End Forces :

Compression force C = 0 kN

Tension force T = 0kN

Connection details:

Vertical Shear force Fy = 155.28kN

Grade of bolt 8.8

Dia.of bolt(d) 20 mm

Dia hole of bolt(d0) 22 mm

Nr of bolt column (n2) 1

Nr of bolt Rows (n1) 4

Pitch(p) 65 mm

Edge distance(e) 40 mm

Thickness of fin plate(tp) 12 mm

Length of finplate(hp) 275 mm

Dist. between face of supporting mem. & supported beam

gh=25mm(End projection)

Nominal Yield Strength of Bolt (Table.3.1, BS EN 1993-1-8:2005) f_{yb}=640 M Pa

Ultimate Tensile Strength of Bolt (Table.3.1, BS EN 1993-1-8:2005) f_{ub}=800MPa

Minimum Yield Strength = Design strength Supported & Supporting beam: fy.b=345.0 M Pa

Fin Plate: fy.p =355 MPa

Check-1: Recommended detailing practice

[SCI Publications-P 358,C.L.5.5]

Length of fin plate hp = 40 + (3 x 65)+ 40 >= $0.6D_2$

= 275.0 mm > (217.9) **SAFE**

Spacing of Bolts [Table 3.3 of BS EN 1993-1-8:2005]

Minimum spacing

of bolts Spacing p1 = 2.2 d_0 < p

= 48.4 mm < 65 **SAFE**

Maximum spacing = Min(14 t , 200mm) > p

of bolts =168.0 mm > 65 **SAFE**

End & Edge Distances of Bolts:

Minimum end and edge distances of bolts = 1.2 d_0 < e

=26.4 mm < 40 mm

Maximum end and edge distances of bolts = 4 t + 40mm >e = 88.0 mm > 40mm

Distance between face of web/flange to first bolt ,

Z_p =65mm

Type of fin plate based on the length,

Short $t_p \geq 0.15\ Z_p$ Long $t_p < 0.15\ Z_p$

t_p =12 mm ≥ 9. 75 Short fin plate is provided

Recommended detailing practice is satisfied.

Basic Requirement (Bearing of bolts on fin plate and beam web)

$F_s < F_{b,Rd}$

Vertical Reaction V_{Ed} =155.3kN

Eccentricity of F_V about c.g. of bolt group ,

e= g_h+e+0.5(n2-1)p_2=65 mm

Moment due to eccentricity M = V_{Ed}xe =(155.3x65)/10^3

=10.09kN.m

Distance of outermost bolt from cg of bolt group

r = Sqrt(((n_2-1)*p_2/2)^2+((n_1-1)*p_1/2)^2)

=97.5 mm

Sum of square of 'r' for all bolts, Σr^2=21,125.0 mm^2

Vertical force on a bolt due to direct shear,

F_{sv} =V_{Ed}/n=38.82 kN

Horizontal force on a bolt due to Axial Force,

F_{ah} = Max (C , T)/n= max(0 , 0) /4=0kN

Maximum shear in bolt due to M ,

F_m = M_e*r/Σr^2=46.58kN

Vertical force on the outermost bolt due to moment M,

F_{smv} = F_m * cosθ =0kN

Horizontal force on the outermost bolt due to moment M,

F_{smh} = F_m * sinθ=46.58kN

Total vertical force on a bolt F_{vb} = F_{sv} + F_{smv}

=38.82+0

F_{vb} =38.82kN

Total horizontal force on a bolt F_{hb} = F_{ah} + F_{smh}

=0+46.58

F_{hb} =46.58kN

Resultant force on outermost bolt F_s = Sqrt ($F_{vb}^2 + F_{hb}^2$)

Fs =60.64 kN

Check 2: Supported beam - Bolt Group

Bearing resistance per bolt [Table 3.4 of BS EN 1993-1-8:2005]

FinPlate :

F_b,R_d = Min. (F_b,Rd.f , Fb,Rd.b) *kbs

Vertical Bearing resistance of the plate per bolt,

Fb,ver,Rd.f = k_1*α_b*d*t_p*fup/γ_{M2}

k_1 = Min[(2.8e2/d0-1.7), 2.5]=2.50

α_b=Min[(min(e,e,1)/3d_0),(p_1/3d_0-1/4), ($f_{ub}/f_{u,p}$),1.0]=0.61

F_b,ver,Rd.f = 2.5*0.61*20*12*470/1.25=136.73 kN

Horizontal Bearing resistance of the plate per bolt,

F_b,hor,Rd.f = k_1*α_b*d*t_p*f_{up}/γ_{M2}

k_1 = Min[(2.8e/d_0-1.7), (1.4p1/d_0-1.7), 2.5]=2.44

α_b = Min[(e/3d_0), ($f_{ub}/f_{u,p}$),1.0]=0.61

F_b,hor,Rd.f = 2.44*0.61*20*12*470/1.25 =133.25 kN

Bearing resistance of the plate per bolt ,

F_b,Rd.f = Min[F_b,ver,Rd.f , F_b,hor,Rd.f] =133.25kN

Web:

Vertical Bearing resistance of the web per bolt ,

F_b,ver,$R_{d.b}$ = k_1*α_b*d*t_w*$f_{u,b}$/γ_{M2}

k_1 = Min[(2.8e,b/d_0-1.7), 2.5]=2.50

α_b = Min[(Min(e,b,he)/3d_0),(p/3d_0-1/4), ($f_{ub}/f_{u,p}$),1.0]=0.73

α_b = 2.5*0.73*20*12.954*470/1.25

Fb,ver,$R_{d.b}$ =178.96kN

Horizontal Bearing resistance of the web per bolt,

Fb,hor,Rd.b = k1*αb*d*tw*fub/γM2

k1 = Min[((min(2.8e,b,he)/d_0-1.7), (1.4p/d_0-1.7), 2.5]=2.44

αb = Min[(e,b/3d_0), (fub/fu,p),1.0] =0.61

αb = 2.44*0.61*20*12.954*470/1.25

Fb,hor,Rd.b =143.84kN

Bearing resistance of the web per bolt,

Fb,Rd.b = Min[Fb,ver,Rd.b , Fb,hor,Rd.b]

= Min[178.96kN , 143.84kN]=143.84kN

Bearing resistance per bolt ,

Fb,Rd = > Fs =133.25 kN > 60.64 kN **SAFE.**

Check 3: Supported beam -connecting elements-Fin plate

Shear and bending capacity of fin plate connected to supported beam

[SCI Publications-P 358-fin plates design procedure-check-3-pg-107]

(i) Shear:

Basic requirement, V_{Ed} < V_{Rd},min

Shear resistance of fin plate V_{Rd},min = Min.{gross section shear resistance (V_{Rd},g) ,

Net section shear resistance (V_{Rd},n) , Block shear resistance (V_{Rd},b)}

Gross section shear resistance of fin plate

V_{Rd},g = (h_p*t_p*$f_{y,p}$)/(1.27*√3*γ_{M0})

=(275*12*355)/(1.27*sqrt(3)*1**=532.57** kN

Net section shear resistance of fin plate:

Net section shear resistance of fin plate ,

V_{Rd},n = (Av.net*$f_{u,p}$)/(√3*γ_{M2})

Net Shear Area after deducting holes,

Av.net = t_p(h_p- n_1*d_0)=2244mm^2

VRd,n = (2244*470)/(sqrt(3)*1.1=553.56Kn

Block tearing resistance of fin plate

[C.L 3.10.2 of BS EN 1993-1-8:2005]

Block tearing resistance of fin plate,

0.5 fu,p Ant /fy,p Anv + fy,p Anv/(√3*γ_{M0})

Net area subjected to tension Ant=(tp(e*0.5*d_o))=348mm^2

Net area subjected to shear,

Anv = t_p(h_p-e-(n_1-0.5)*d_o)=1896mm^2

Block tearing resistance of fin plate VRd,b =462.95kN

Shear resistance of fin plate,

VRd,min **(462.95kN) >** VEd **(155.28kN)** **SAFE**

(ii) Bending [SCI Publications-P 358-fin plates design procedure-check-3-pg-108]

Basic requirement, VEd < VRd

Distance between face of web/flange to C.G of bolt z=65mm

h_p > 2.73z V_{Rd} **=** infinite

275>177.45 V_{Rd} is infinite V_{Ed}<V_{Rd}

Else, V_{Rd} = Wel,p*f_{yp}/z*γM0

Wel,p = t_p*h_p^2/6=151250mm^3

V_{Rd} =(151250x355/65x1)/1000=826.06kN

V_{Rd} (826.06kN) = **>** V_{Ed} **(155.28kN)** **SAFE**

Check 4: Supported beam - Beam web

Shear and bending resistance of the supported beam

Since the beam is Unnotched.

For Shear: [C.L 6.2.6 of BS EN 1993-1-1:2005]

Basic requirement, V_{Ed} < $V_{Rd,min}$

Shear resistance of the supported beam web VRd,min = Min [VRd,g , VRd,b]

Gross section shear resistance of beam VRd,g = fy,b * Av / √3*γM0

Shear Area Av = A_2- 2$B_2$$T_{f2}$ + (t_w+2r_{r2}) t_{f2} But ≤ h_{w2} tw

=5279.02 mm^2 > 4142.51 mm^2 >**Ok**

Depth between flanges h_{w2} = Df-2*T_f =319.786 mm

VRd,g =1051.51 kN

Net section shear resistance of beam

Net section shear resistance of beam ,

VRd,b = fu,b * Av,net / $\sqrt{3}$*γ_{M2}

Net Shear Area after deducting holes,

Av.net = Av - n_1*d_0*t_w =4139 07 mm^2

VRd,b = 1021.05 kN

Shear resistance of the supported beam web,

VRd,min = Min[1051.51, 1021.05]=1021.05 kN

Check 5: Weld between fin plate and web of supporting member

Basic requirement,

$\sqrt{\sigma\perp^2 + 3(\tau\perp^2 + \tau\|^2)} \le fu/(\beta w*\gamma_{M2})$ and $\sigma\perp \le 0.9fu/\gamma_{M2}$

[C.L 4.5.3.2 of BS EN 1993-1-8:2005]

Size of weld provided ,

sw =10mm

a = 0.707

sw=7.07 mm

Transverse force in weld ,F_T **= 0kN**

Angle between F_T & throat of weld, θ = 45deg

Longitudinal Shear force in weld, F_L **= 155.28kN**

Strength of weld fvw,d **=** 241.2 MPa

Length of weld l_w **=** (h_p - 2_s)*2**=510 mm**

Design value of weld force per unit length, FwL,Ed = F_L/l_w

logitudinal =0.30 kN/mm

Design value of weld force per unit length ,FwT,Ed = F_T/l_w

Transverse = 0 kN/mm

where, K = sqrt(3/(1+2cos^{2}θ))

K =1.225

Weld Interaction,

=1/a *Sqrt [FwT,Ed / K^2 + FwL,Ed^2] < fvw.d

(Publication P363 Chapter 11.2)

=1/7.07 *Sqrt [0.000 /1.550 + 0.093] < 241.2 M Pa

=43.07 M Pa < 241.2 M Pa **SAFE**

And ,

$\sigma\perp \le 0.9fu/\gamma_{M2}$ Normal stress,

$\sigma\perp$ = FwT,Ed sin θ/a 0.9fu/γ_{M2} = =(0.9*470/1.25)

=0 M Pa

=338.4 M Pa

4. DESIGN OF EMBED CONNECTIONS USING ANCHORS:

Type of Anchors: **Rebar**

Skewed Angle=90deg

Grade of anchors= Fy 460

Yield strength of anchor fya = 460N/mm^2

Yield strength of additional reinforcement, fya=500N/mm^2

Tensile strength of anchor,

(Min{1.08*fya , Min (1.9*f_{ya} , 860)} f_{ut} = 497N/mm^2

Ref: ACI-318M -14 - CL.17.4.1.2 & BS 4449:1997 , Table 7

Diameter of anchor,d_a = 25mm

Area of anchor ,A_{se} = 490.87mm^2

Length of anchor,h_{eff} = D_{er} = 240 mm

h_{eff}=D_{er}+ t_{ep}= 240+30 = 270.00mm

Steel strength reduction factor (Tension), $Ø_T$ = 0.75

Steel strength reduction factor (Shear),$Ø_S$ = 0.65

Reduction factor for anchors,$Ø_a$ =0.75

No. of anchors in column,N_c =2

No. of anchors in row,N_r = 3

Total no. of anchors, n = 6

Pitch and gauge:

Pitch distance,(Spacing between top two anchors) ,

p_1 =200mm

Pitch distance,(Spacing between other anchors) ,

p_2 =200mm

Total pitch distance (Y=p_1+p_2)

Y = 400 mm

Gauge distance , g_1 = 100 mm

g_2 = 0 mm

Total gauge distance ($X=g_1+g_2$) X = 100 mm

Embedment plate

Grade of material for rolled sections – Plates, =S355

Yield strength of plate ,p_y = 345 N/mm^2

Thickness of embed plate,t_{ep} = 30mm

Breadth of Embed plate,B_{ep} = 200mm

Depth of Embed plate,D_{ep} = 500mm

Horizontal &Vertical - Edge & end distance of Embed Plate (Top& Bottom),

e =50mm

Weld Details for anchors

Strength of weld,pw = 245N/mm^2

Leg size of weld provided between anchor to Embed plate,

s_{w1} = 16mm (Designed for 100 % anchor axial capacity)

Fin plate (From Fin Plate Design)

Thickness of fin plate,t_{fp} =16mm

Depth of fin Plate,d_{fp} = 285 mm

Fin plate weld,S_f = 12mm (Fillet weld)

Total anchor Spacing = Top anchor to Bottom anchor distance

= 400mm

Distance between extreme anchor to tip of fin plate per side,

d_t =30mm

Concrete Parameters:

Characteristic strength of concrete (Cube Strength),

f_c = 40 N/mm2

(Cylinder Strength) f 'c = 0.8*40= 32 N/mm^2

Depth of concrete column / Wall (Minimum) h_a = 400 mm

Normal weight concrete factor λ = 1.00

Ref: ACI 318M -14 - Table.19.2.4.2

Concrete edge detail with respect to anchors :

(Refer below Sketch)

Edge distance between wall corner to first column of anchor (at side) $d_{e1}=c_{a1}$ = 1000 mm > 405.00 mm

Edge distance between free wall surface to last column of anchor (at side) $d_{e2}=c_{a3}$ = 1000 mm > 405.00 mm

Edge distance between wall surface to last row of anchor (at bottom) $d_{e3}=c_{a2}$ = 1000 mm > 405.00 mm

Edge distance between wall surface to first row of anchor (at top) $d_{e4}=c_{a4}$ = 1000 mm > 405.00 mm

demin = 1000 mm > 405.00 mm

Eccentricities:

Eccentricity from fin plate bolt group center + imperfection - Z direction ,e_1 = 98 mm

Eccentricity due to imperfection in X direction e_2 = 25 mm

Eccentricity due to imperfection in Y direction e_3 = 25 mm

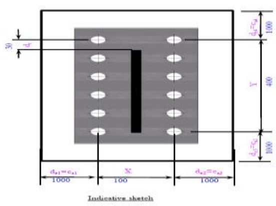

Fig -1: **Concrete edge detail with respect to anchors**

Force transfer to anchors:

In plane moment due to eccentricity (Vertical Shear) M_{z1} = Fy*e1 = 15.22 kN.m

In plane moment due to eccentricity (Axial Tension) M_{z2} = T*e_3 = 0.00 kN.m

Total In plane Moment due to eccentricity $M_{zi} = M_{Z1} + M_{Z2}$ = M_{zi} = 15.22 kN.m

Out of plane moment due to eccentricity (Axial Tension) M_y = T*e_2 = 0.00 kN.m

Direct tension/anchor due to axial tension T_1 = 0.00 kN

Tension @ top row per anchor due to M_z /anchor (In Plane Moment) T_2 = 15.22 kN

Tension @ 1st column or anchor due to My /anchor (Out of Plane Moment) T_3 = 0.00 kN

Max. Tension in one anchor Ttot = $T_1+T_2+T_3$

T_{tot} = 15.22 kN

Total Tension in anchor group $T_{gr.tot}$ = 45.65 kN

Total tension in Top anchors = 15.22+15.22+0 = 30.44 kN

Check 1: Check for anchor

Tension capacity Ref: ACI 318M -14 - CL.17.4.1.2

Tension capacity of anchor group $Ø_T*N_{sa} = Ø_T*n*A_{se}*f_{ut}$

= 0.75*6*490.87*496.8

= 1097388.97 N= 1097.39 kN > 45.7 kN **SAFE**

Stress Ratio = 0.04

Tension capacity of single anchor $Ø_T*N_{sa} = Ø_T *A_{se} *f_{ut}$

$Ø_T*N_{sa}$ = 0.75*490.87*496.8= 182898.16 N

$Ø_T*N_{sa}$ = 182.90 kN > 15.22 kN **SAFE**

Stress Ratio = 0.08

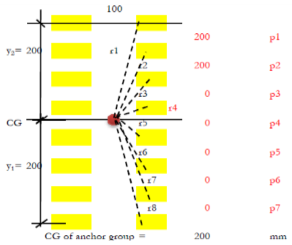

Fig -2: CG of anchor group

Check 2: Shear capacity

Vertical shear force per anchor $F_{v1} = F_y/n$ = 155.28/6 kN

= 25.88 kN

Horizontal shear force per anchor $F_{h1} = F_z/n$ = 0/6 = 0.00 kN

Eccentricity of Fy & Fz about c.g. of anchor group e_2 = 25 mm e_3 = 25 mm

$M = (F_y*e_2 + F_z*e_3)$ = (155.28*25)+(0*25)/1000 = 3.88kN.m

r_1^2	42500mm²
r_2^2	2500 mm²
r_3^2	42500mm²
r_4^2	42500mm²
r_5^2	42500mm²
r_6^2, r_7^2, r_8^2	0 mm²

Sum of square of 'r' for the anchor group Σr^2= 345000 mm²

Horizontal force on the outermost anchor due to moment M , $F_{sh} = (M* Max(y_1,y_2)/\Sigma r^2$=2.25 kN

Vertical force on the outermost anchor due to moment M

$Fsv = M*((n_c-1)*g/2)/\Sigma r^2$ =0.56 kN

Total Vertical force $F_v = F_{v1}+F_{sv}$ = 25.88+0.56 kN= 26.44 kN

Total Horizontal force $F_h = F_{h1}+F_{sh}$ = 0+2.25 kN

Sum of square of 'r' for the anchor group Σr^2= 345000 mm²

Horizontal force on the outermost anchor due to moment M , $F_{sh} = (M* Max(y_1,y_2)/\Sigma r^2$=2.25 kN

Vertical force on the outermost anchor due to moment M ,

$F_{sv} = M*((n_c-1)*g/2)/\Sigma r^2$ =0.56 kN

Total Vertical force $F_v = F_{v1}+F_{sv}$ = 25.88+0.56 kN= 26.44 kN

Total Horizontal force, $F_h = F_{h1}+F_{sh}$ = 0+2.25 kN

Resultant force on outermost anchor ,

R = Sqrt $(F_{v2} + F_{h2})$ 2 = 0 mm²

R = 26.54 kN

Total shear force on anchor group V_{tot} = n*R = 159.21 kN

Shear capacity of anchor group, $Ø_s*V_{sa} = Ø_s * n *A_{se} * f_{ut}$ (Ref: ACI 318M -14 - CL.17.5.1.2a)

$Ø_s*V_{sa}$ = 0.65*6*490.87*496.8 = 951070.44 N

$Ø_s*V_{sa}$ = 951.07 kN > 159.21 kN **SAFE**

Stress Ratio = 0.17

Shear capacity of single anchor $Ø_s*V_{sa} = Ø_s *A_{se} * f_{ut}$ (Ref: ACI 318M -14 - CL.17.5.1.2a)

$Ø_s*V_{sa}$ = 0.65*490.87*496.8= 158511.74 N

$Ø_s*V_{sa}$ = 158.51 kN > 26.54 kN **SAFE**

Stress Ratio = 0.17

Interaction for shear and tension in anchor

$(F_s / P_s) + (F_t / P_{nom}) \le 1.2$

0.17+ 0.08 ≤ 1.2

0.25 ≤ 1.2 **SAFE**

Stress Ratio = 0.21

Check 3: Check for anchorage depth

Anchorage bond stress fb = Fs/πϕe l (Ref: BS 8110-1:1997 Cl. 3.12.8.3)

fb = β√fc (Ref: BS 8110-1:1997 Cl. 3.12.8.4)

Deformed bars β = 0.50 (Ref: BS 8110-1:1997 Table 3.26)

fb = 3.16

The required anchorage length based on bond stress l

= 15.22/π*25*3.16= 61.33 mm < 270 mm

Stress ratio = 0.23 **SAFE**

Check 4: Check for weld between anchor to Embed plate

Available length of weld per anchor $l_w = \pi * d_a$

$l_w = \pi*25$ = 78.54 mm

Size of weld s_w = 16 mm

Strength of weld p_w = 245 N/mm2

Longitudinal Shear capacity of weld per anchor,

$p_L = p_w*0.707*s_w*l_w$

p_L = 245*0.707*16*78.54/1000

p_L = 217.67 kN (As per BS 5950-1-2000, clause 6.8.7.3)

Longitudinal Shear capacity of weld for single anchor ,

$P_L = p_L$ K = 1.25

= 217.67 kN >= 26.54 kN (Actual shear force of anchor)

Transverse capacity of weld, $P_T = K*P_L$

= 272.09 kN >= 182.90 kN (Axial capacity of anchor)

Check 5: Check for interaction:

$(F_L/P_L)^2+(F_T/P_T)^2$=

= (26.54/217.67) ²+ (182.9/272.0875) ²

= 0.47 < 1 **SAFE**

Check 6:Check for embed plate

Shear capacity of Embed plate:

Basic requirement,

$F_v < P_v$ (Ref BS 5950:I:2000 Cl 4.2.3 & 6.2.3)

Shear capacity of embed plate ,$P_v = 0.6 p_y A_v$

Shear Area, $A_v = 0.9 \times d_e \times t_e$= 0.9*500*30

A_v = 13500 mm²

Shear capacity of embed plate ,P_v = 2794.5 kN

P_v = 2794.50 kN > 155.28 kN **SAFE**

Stress Ratio = 0.06

Check 7:Check for Embed plate bending

Case 1: Top yielding

Leff1 = Breadth of Embed plate

$M_t = T*(d_t+25)$

M_t = Tension *((Ecc.of extreme anchor to tip of fin plate per side+Construction Tolerence for fin Plate in Vertical)

M_t = 30.44*((30+25))/1000= 1.67 kN.m

$M_t = f_y * (b * t_2 / 6)$

t_{p1}=sqrt(6* Mt/(fy . L_{eff1}))= (Sqrt((6*1.67*10⁶)/(200*345)

t_{p1}= 12.05 mm < 30.00 mm **SAFE**

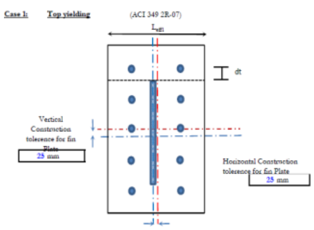

Fig -3: Top yielding

Case 2: Side yielding

Leff2=Verticalend distance (e_{v1})+Pitch distance(Y_{sr1})+(Pitch distance(Y_{sr2})/2)

Ms = Tension (T_1+T_2) * {[(gauge/2)-(t_k of Plate/2)-(leg of weld)]+Construction Tolerence for fin plate in Horizontal}

M_s = ((15.22+7.61)*[(100*0.5)-(16*0.5)-12+25])/1000

M_s = 1.26 kN.m

$t_{p2,a}$ = sqrt(6* M/(f_y . L_{eff2}))

= (Sqrt(6*1.26*10^6/((345)*((50+200+(200/2))

= 7.91 mm < 30.00 mm **SAFE**

L_{eff2a} = Vertical end distance (e_{v1})+(Pitch distance(Y_{sr1})/2)

Ms = Tension (T_1) * {[(gauge/2)-(t_k of Plate/2)-(leg of weld)]+Construction Tolerence for fin plate in Horizontal}

= ((15.22)*[(100*0.5)-(16*0.5)-12+25)]/1000= 0.84 kN.m

$t_{p2,b}$ = sqrt(6* Ms/(fy * Leff2))

= (Sqrt(6*0.84*10^6/((345)*((50+(200/2))

= 9.87 mm < 30.00 mm **SAFE**

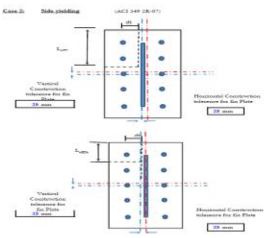

Fig -4: Side yielding

Case 3:

Minimum thickness of Plate as per ACI is 3/8 inch,

(ACI 349 2R-07) t_{p3} = 9.53 mm < 30.00 mm **SAFE**

Thickness of embed plate Required ,

Max (t_{p1}, t_{p2a}, t_{p2b}, t_{p3}) = Max (12.05 , 7.91 , 9.87, 9.53)

= 12.05 mm < 30.00 mm **SAFE**

Stress Ratio = 0.40

Check 8: Check for web weld

Supported **UB305X102X328**	Beam
Depth d_f	31.27 mm
Breadth b_f	102.4 mm
Thickness of flange t_f	10.8 mm
Thickness of web t_w	6.6 mm
d'	275.9 mm
Area a	41.80 cm2

Vertical Shear force ,F_y 22.04kN

Design strength of weld, p_w 241.2 MPa

Size of weld, S_w (Fillet) 6 mm

a 0.707

weld a =S_wx a =6 x0.707 =4.24

Length of weld l_w **=** (2x d') =2x 275.9=**551.8 mm**

Shear capacity of weld =p_w x l_w x a

= (241.2 x 551.8 x 4.24) /1000

=564.31 >22.06 **SAFE**

Stress Ratio = 0.03

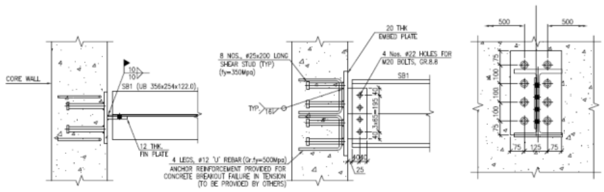

Fig -5: AutoCAD Detail for connection design

5. CONCLUSIONS

- Structural Engineers will calculate the design load required at the steel to concrete connection point and need to select a concrete embed plate that will safely carry the design loads. Factors that are taken into account are the type of concrete being specified, the steel material, the diameter and number of anchors from the plate into the concrete and the steel plate thickness. A concrete embed plate is designed manually in accordance with (Ref: ACI 318-14, Chapter 17).

- The steel to concrete connection can be a difficult one to engineer and design. When you are connecting steel beams to concrete, you are introducing a load that's concentrated in a small area, and this load may be in shear or may produce a moment that puts some part of the connection point in tension. These steel to concrete connection points must be carefully designed and reliably constructed in order for the structural system to meet applicable Codes and be safe and durable for the long run.

- To make a connection between steel and concrete, as in when you connect a structural steel beam to concrete you commonly use concrete embeds, such as a concrete embed anchor or a steel embed plate
- Embed plate with anchor- connection offers direct load transfer to each anchor bolt – no localized stresses in plate.
- More consistent connection – not relying on quality of weld.
- All the structural components were designed manually and detailed using AutoCAD.
- With the demand to capacity ratio less than 1.0, the design is satisfied for all the checks.

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BIOGRAPHY

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