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Design of Well Foundation

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Abstract*:* Due to exposure to potential scouring action of the river water, the foundation of transmission line tower at location no. 456 of Unnao-Bareilly transmission line is proposed to be provided in the form of a well. Detailed design procedure has been carried out also SAP modeling has been made to check the hoop stresses on the steining,

Key words: Well Foundation, Well Steining, SAP Modeling.

INTRODUCTION

Due to exposure to potential scouring action of the river water the foundation of transmission line tower at location no. 456 of Unnao-Bareilly transmission line is proposed to be provided in the form of a well.

DETAILS OF TOWER

A schematic cross section of the stream together with the location is available. The details of the proposed tower and the hydraulic and geotechnical data at the site of the proposed tower are given below.

Tower type	Base width at top of pedestal (mm)	Max. compression		Max. uplift		Lateral load	
		N.C. (kN)	B.W.C (kN)	N.C. (kN)	B.W.C (kN)	N.C. (kN)	B.W.C (kN)
A5+25	23000	776	1066	529	1639	36.5	89.85

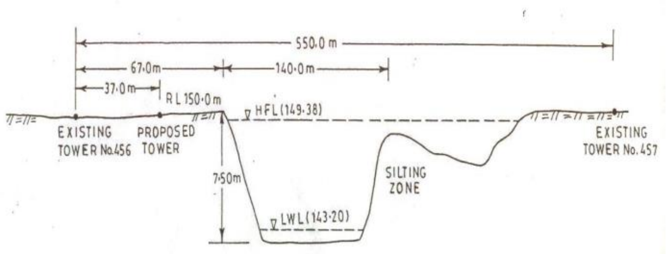

Fig. 1: Schematic cross- section of stream at location of tower No.456

HYDRAULIC DATA

Maximum flood discharge	=	1680 cumecs
Maximum stream velocity	=	1.94 m/s
R.L. of H.F.L.	=	149.4 m
R.L. of river bank	=	150.0 m
R.L. of river bed	=	142.5 m
Silt factor	=	0.83

SOIL CHARRACTERISTICS

- Poorly graded fine to medium sand (SP) upto 32m from G.L.
- Average corrected SPT 'N' value = 10
- Angle of friction = 32°

The legs of the tower are placed on the pedestal having height 2.5 m having cross- section decreasing from 600x600 at bottom to 400x600 at top.

WELL CONFIGURATION

Single well with outrigger arms supporting the tower legs.

PROPORTIONING OF FOUNDATION

The foundation shall be taken adequately below the minimum scour depth. The normal depth of scour is estimated using Lacey's formula as:

$$d = 0.473 \left(\frac{Q}{f}\right)^{\frac{1}{3}}$$

$$= 6.79 \text{ m}$$

where,

d = normal depth of scour

Q = design discharge in cumecs, and

f = 0.68 Lacey's silt factor.

IRC: 78- 2000 recommends that scour depth calculations for foundations may be made for a discharge larger than the design discharge. Accordingly, 20 % increase in design discharge has been assumed in scour depth calculations.

Since tower is to be located in straight reach of river, the maximum scour depth d_{max} is given by,

$$d_{max} = 1.27d = 8.62 \text{ m}$$

DIMENSIONING OF THE WELL FOUNDATION

From the scour considerations minimum grip length for the well foundation = 0.33 x max. scour depth

= 2.84 m $\approx$ 3 m

However provide a grip length of 9 m as a conservative measure.

R.L. of base of well w.r.t. HFL = 149.4 -9 -8.62 m

= 131.78 m

Height of well w.r.t. NGL = 150 – 131.78 m

= 18.22 m

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Thickness of well Steining

As per IRC 78- 2000 (Cl. 708.2.3)

$t = Kd\sqrt{l} = 1.152$ m

t	=	Thickness of well steining		
K	=	Constant	=	0.03 m
d	=	External diameter of well	=	9 m
l	=	Depth of well w.r.t. N.G.L.	=	18.22 m

Provide t = 1.5 m for sufficient sinking effect.

Internal diameter of well = 9 – 1.5 – 1.5

= 6 m > 2 m

as per IRC 78 : 2000 hence OK

The thickness of well cap is taken as = 1.5 m

Thickness of top plug = 0.6 m

(Because we are using well cap)

Height of well Curb = 0.5 x internal diameter of well

= 0.5 x 6

= 3.0 m

As per IRC 78:2000 projection ≥ 75 mm.

Take 100 mm projection.

Size of ISA cutting edge = 150 x 150 x 18 mm.

= 6 m > 2 m

as per IRC 78 : 2000 hence OK

After using AutoCAD for drawing well the length of outrigger for supporting the pedestal is coming out to be = 12073.47 mm.

Taking,

$$\frac{l}{d} = 5$$

$$\frac{12073.47}{d} = 5$$

d= 2414.69 mm Take D = 2700 mm.

The outriggers is being tapered from 1 m to 2.7 m.

Thus, the well details are:

Total height of well	=	18.22 m
Grip length	=	09.00 m
External diameter of well	=	09.00 m
Internal diameter of well	=	06.00 m
Thickness of well steining	=	01.50 m
Length of outrigger	=	12.08 m
Max. depth of outrigger	=	02.70 m
Width of outrigger	=	00.60 m

The proportioned well dimensions are shown in fig. 2 and 3

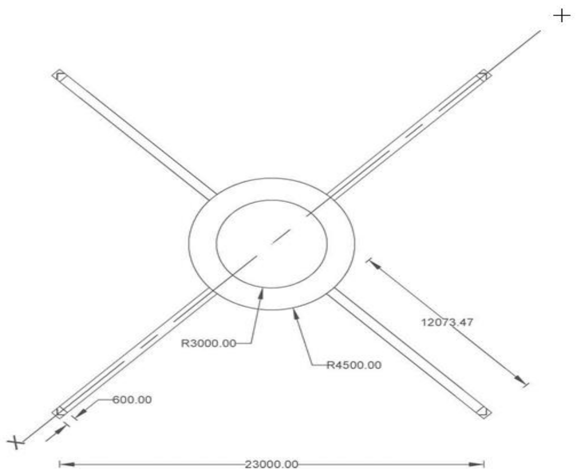


Fig. 2: Plan of well foundation (all dimensions are in mm)

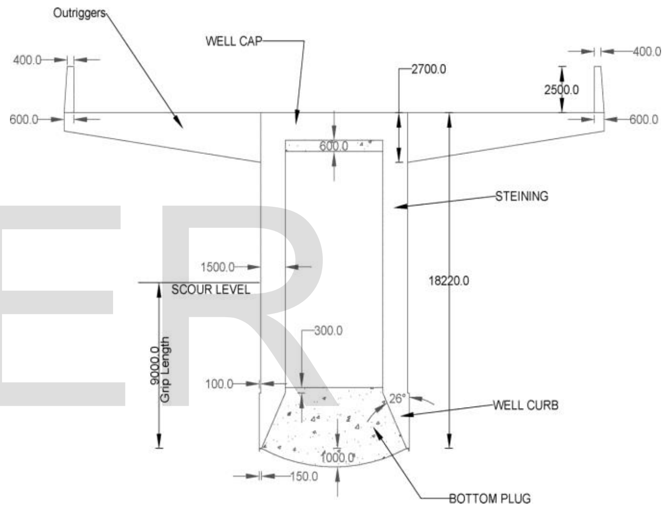


Fig. 3: Sectional elevation of well foundation at section X-X (all dimensions are in mm)

ESTIMATION OF WELL CAPACITY

1. **Uplift capacity**

The safe uplift capacity, Q_{safe}, may be calculated as the submerged weight of the well, conservatively ignoring the effect of side friction. Thus,

$$Q_{safe} = \frac{\pi(9^2-6^2)\text{x}18.35\text{x}(25-10)}{4} + (4 \text{ x } .5 \text{ x } (1+2.7) \text{ x } 12.08 \text{ x } 0.6 \text{ x } (25-10))$$

= 9728.138 kN > 1639 kN, O.K.

Hence the well is safe in uplift.

2. **Axial compression load capacity**

The effect of skin friction is conservatively ignored and the axial load capacity is taken as the base resistance with a factor of safety of 3. The base resistance, Q_{safe}, is calculated as

$$Q_{bsafe} = \frac{\bar{\sigma}_v \text{ at tip x } (N_q - 1) \text{ x } A_p}{F.O.S.}$$

Where,

$\bar{\sigma}_v$ = Effective overburden pressure at base

N_q = Bearing capacity factor

A_p = Area of base of well

F.O.S. = Factor of safety

For soil at base of well, φ = 32°, hence N_g = 24.36

$\therefore Q_{bsafe} = (9 \times 10) \times (24.36 - 1) \times (\frac{\pi \times 9^2}{4 \times 3})$

= 44582.9696 kN > 4264 KN

Hence OK.

3. Lateral load capacity

The lateral load acting on the well consists of two components :

Design lateral load corresponding to B.W.C. = 359.4 kN.

Lateral load due to water current force corresponding to H.F.L. acting on curved surface area of the well. Intensity of water current pressure at HFL = $0.52KV^2$

Where,

V = Velocity of the current at the point where the pressure intensity is being calculated, in meters per second.

K = a constant having a value of 0.66 for circular piers.

$P = 0.52 \times 0.66 \times (\sqrt{2} \times 1.94)^2$

$= 2.58\ kN/m^2$

Water current (lateral) force = 0.5 x 8.62 x 2.58 x 9

= 100.07 kN

Total lateral force = 359.4 + 100.07

= 459.47 kN

K_a = 0.31 and K_p = 3.25, for φ = 32°.

Let the total lateral is acting at a height 'h' above base of well.

459.47 x h = (100.07 x 14.74) + 359.4(18.22+2.5)

h = 19.42 m.

∴ Elevation of resultant lateral load = 19.42 m.

Safe lateral capacity, H_{safe}, may be computed as,

$$H_{safe} = \frac{0.5\gamma(K_p - K_a)(D-2D_1)d_e}{F.O.S.}$$

Where,

$$D_1 = \frac{3h \pm \sqrt{9h^2 - 6D\left(h - \frac{D}{3}\right)}}{2}$$

h	=	Height of resultant lateral load above base	= 19.42 m
D	=	Grip length	= 9 m
K_p	=	3.25	} For ϕ = 32°
K_a	=	0.31	
γ	=	Submerged unit weight of soil	= 10 kN/m³
F.O.S.	=	Factor of safety,	= 3.0
d_e	=	External dia. Of well	= 9 m

D_1 = 54.17 and 4.09 (we have to select lower value)

$$H_{safe} = \frac{0.5x10x(2.94)(9-8.18)x9}{2}$$

= 488.187 > 459.57 O.K.

4. Stability check of outriggers

Clear span of cantilever (l_o) = 12.07 m.

$D_{av.} = \frac{2.7+1}{2}$

= 1.85 m or 1850 mm

d_{av} = 1775 mm b = 600 mm

a) 25b = 15000 mm

b) $\frac{100\,b^2}{d_{av.}}$ = 20281.69 mm

Take smaller value between (a) and (b) = 15000 mm

l_o < 15000 mm O.K.

DESIGN OF WELL COMPONENTS

1. Design of outriggers

Due to the large moments coming on the critical section a no. of trials have to be done because of the change in the value of effective depth after the placement of the bars in the cross section. Final calculations has been shown here.

$L_{eff} = l_o + (1.5/2)$

= 12.07+ .75

= 12.82 m

Load (P) = 1085 kN

Taking

Overall depth (D) = 2700 mm

Effective depth (d) = 2500 mm

Effective cover (d') = 200 mm

Grade of concrete = M25

Grade of steel = Fe415

Self-weight of beam

Volume of RCC = 0.5 (2.7+1) x 12.82 x 0.6

= 14.2302 m³

Weight of beam = 25 x 14.2302 m³

= 355.755 kN

Factored Moment

M_u = 1.5 x (1085 x 12.48 + 355.755 x 6.41)

= 23732 kNm

= 23732 x 10^6 Nmm.

$$\frac{x_{u,max}}{d} = \frac{0.0035}{0.0055 + \frac{0.87 f_y}{E_s}}$$

= 0.4848

$M_{u,\,lim} = .362\,(\frac{x_{u,max}}{d})(1 - (0.416 \times \frac{x_{u,max}}{d})) \times f_{ck}bd^2$

= 13134.375 x 10^6 Nmm

Since $M_u > M_{u,}$ lim section is to be designed as doubly reinforced.

$P_{t,lim} = 41.61\ (f_{ck}/f_y)(\frac{x_{u,max}}{d}$ = 1.2 %

$\Delta A_{st} = \frac{M_u - M_{u,lim}}{0.87 f_y (d-d')}$ = 12761.84 mm²

$A_{st,req.}$ = 12761.84 + $A_{st,lim}$

= 30567.44 mm²

Provide 28 nos of 36ϕ bars and 6 nos. of 22ϕ bars.

$A_{st,prov.}$ = 30781.32 > 30567.44 mm²

<u>Calculation of new d</u>

30781.32 d = 71735808.84 + 6 x (π/4) x 22² x 2337

d = 2504 mm ≈ 2500 mm . OK

d' = 196 mm

Calculation of A_{sc}

$\Delta A_{st,\,prov.}$ = 30781.32- 17805.6

= 12975.72 mm^2

$A_{sc,req.} = \frac{0.87 f_y \times \Delta A_{st}}{f_{sc} - 0.447\ f_{ck}}$

$f_{sc} = 0.0035 \frac{(x_{u,max} - d')}{x_{u,max}}$

=353.308 N/mm^2

$A_{sc,req.}$ = 13693. 165 mm^2

Provide 14 nos. 36ϕ bars

$A_{sc,prov.}$ = 14250.26428 mm^2 > 13693.165 mm^2 O.K.

P_c =14250.264/(600 x 2504) x 100

=.9485

P_t = 2.05

$P_c^* = \frac{0.87 f_y (p_t - p_{t,lim})}{f_{sc} - 0.447 f_{ck}}$

= 0.8969

P_c > P_c^* (hence beam is under reinforced) O.K.

2. Side face reinforcement

Side face reinforcement has to be provided because depth of the beam is more than 750 mm.

Minimum area = 0.001 x 600 x 2700

= 1620 mm ^{2}

At a spacing not exceeding 300mm.

Provide 8 nos. of 12ϕ bar at each face at equal spaces.

3. Design of shear reinforcement.

Shear at critical section

V = 0.5 x 3.7 x 12.07 x 25 x 0.6 +1085 kN

=1419.94 kN

V_u = 1.5V = 2129.94 kN

M_u = 1.5 ((334.94 x $\frac{12.07}{2}$) + (1085 x 11.73))

= 22122.61 kNm.

$\tan\beta = \frac{1.7}{12.07}$ = .141

(β = angle of sloping surface with the horizontal)

$\tau_v = \frac{V_u - \frac{M_u}{d}\tan\beta}{bd}$

= 0.588 N/mm^2

$100\frac{A_s}{bd} = \frac{30781.32}{600 \times 2504}$ x 100

= 2.05

From table 19 of IS 456:2000

τ_c = 0.826 N/mm^2

$\tau_c > \tau_v$, Hence section is safe in shear, and minimum shear reinforcement should be provided.

Minimum reinforcement should be provided as per the following formula.

$\frac{A_{sv}}{bs_v} \geq \frac{0.4}{0.87 f_y}$

Where,

A_{sv} = total cross sectional area od stirrups effective in shear.

s_v = stirrup spacing along the length of the member.

b = breadth of the beam (= 600 mm)

f_y = Characteristic strength of stirrup reinforcement in N/mm^2. (415 N/mm^2)

Provide 2-legged 10mm dia. Bars.

A_{sv} = 157.08 mm^2.

Putting the values in above formula we get

s_v = 236.31 mm.

Give shear reinforcement at 230 m c/c.

4. Development length

Grade of Concrete = M25

Grade of steel = Fe415

The development length L_d is given by

$L_d = \frac{\phi \sigma_s}{4\tau_{bd}}$

Where,

Φ = nominal diameter of the bar.

σ_s = Stress in bar at the section considered at design load = 0.87f$_y$), and

τ_{bd} = Design bond stress (= 1.4 for M25)

$L_d = \frac{0.87 \times 415\phi}{4 \times 1.4}$

L_d = 64.47 ϕ

For 36mm dia. bar, L_d = 2320.92 mm

For 22 mm dia. bar L_d = 1418.34 mm

5. Deflection

The total deflection shall be taken as the sum of short-term deflection and the long term deflection.

Short term deflection

We have

b = 600 mm D = 2700 mm

f_{ck} = 25MPa f_y = 415 MPa

W = 1085 kN l = 12830 mm.

M = 15821330000 Nmm.

$I_{gr} = \frac{bD^3}{12}$ = 9.84155 x 10^{11} $f_{cr} = 0.7\sqrt{f_{ck}}$ = 3.5 N/mm^2

y_t = D/2 = 1350 mm $M_{cr} = \frac{f_{cr} I_{gr}}{y_t}$ = 2.6 x 10^9 Nmm

$E_c = 5000\sqrt{f_{ck}}$ = 25000 N/mm^2

E_s = 2 x 10^5 N/ mm^2

m = 8

Let x be the depth of neutral axis, then taking moment of transformed section about N.A.

We get,

x = 980.48 mm

$I_{cr} = \frac{bx^3}{3} + (m-1)A_{sc}(x - d')^2 + mA_{st}(d - x)^2$

I_{cr} = 8.355 x 10^{11} mm^4

$I_{eff.} = \frac{I_r}{1.2 - \frac{M_r}{M}\frac{z}{d}\left(1 - \frac{x}{d}\right)\frac{b_w}{b}}$

I_{eff} = 7.49 x 10^{11}

Since, $I_{eff} < I_{cr}$, hence I_{eff} = 8.355 x 10^{11} mm^4

$\Delta_{short\ term} = \frac{Wl^3}{3EI_{eff.}}$ = 36.56 mm

6. Deflection due to shrinkage

$\alpha_{cs} = k_3 \Psi_{cs} l^2$

here, k_3 = 0.5

$\Psi_{cs} = k_4 \frac{\epsilon_{cs}}{D}$, $\epsilon_{cs} = 0.0003$

$k_4 = 0.72 \times \frac{P_t - P_c}{\sqrt{P_t}}$

where

P_t = 2.05, P_c = 0.95

Putting the values we get

α_{cs} = 5.06 mm

Deflection due to creep
Calculation of deflection due to creep is same as that of short term deflection but with modified E given by

$E_{ce} = \frac{E_C}{1+\theta}$; θ being the creep coefficient.

E_{ce} = 9615.38

Δ_{creep} = 57.73 mm

Δ_{total} = 36.56+5.06+57.73 = 99.355 mm

7. Design of well cap

Since no direct load is coming on well cap, minimum should be provided.

i.e. 0.12% of gross sectional area

$= \frac{.12}{100} \times 1500 \times 1000$

$= 1800\ mm^2$

Provide 18 mm. diameter bar at 300 c/c on top and bottom faces of the well cap at a clear cover of 75 mm.

A_{st} provided = 2035.75 > 1800 mm^2 O.K.

8. Design of well steining

Lateral load acting on well = 359.4+ 100.07 kN
= 459.47 kN

Distance of lateral load from base of well = 19.42 m.

The resultant earth pressure force at depth 'y' below M.S.L. is given by :

$= 0.5 \times \gamma_{sub} \times (K_p - K_a) \times y^2 \times D_e$

Equating the lateral loads at depth y gives the location of zero shear (and max. moment section).

$459.47 = 0.5 \times 10 \times (3.25 - .31) \times y^2 \times 9$

y = 1.86 m

weight of well steining of 1.86 m height

$= \frac{\pi}{4} \times (9^2 - 6^2) \times 1.86 \times 25$

= 1643.45 kN

Moment of lateral forces about section of zero shear

M = 459.47 x (18.22-9+1.86)

= 5090.93 kNm

Total axial load at section of zero shear

= Load from tower + Weight of pedestal + Weight of outriggers + Weight of well cap + Weight of steining

$= (4 \times 1066) + (4 \times 18.75) + (4 \times 355.76) + (\frac{\pi}{4} \times 9^2 \times 25 \times 1.5) + 1643.45$

= 9791.14 kN = P

Area of cross section of steining $= \frac{\pi}{4} \times (9^2 - 6^2) = 35.34\ m^2$

$I_{xx} = I_{yy} = \frac{\pi}{64}(9^4 - 6^4) = 258.44\ m^4$

$y = \frac{D_E}{2} = 4.5$ m

The stresses in the steining

$f_{1,2} = \frac{P}{A} \pm \frac{My}{I}$

putting the values we get.

$f_{1,2} = 277.06 \pm 88.64$

f_1 = .365 MPa < 8MPa

f_2 = .188 MPa > 0

Both the stresses f_1 and f_2 are compressive and significantly smaller than the allowable stresses for M-25 grade concrete. Hence, the steining section is safe.

<u>**Reinforcement in well steining**</u>

Provide vertical steel = 0.12 % of gross sectional area = 11907 mm^2

Provide vertical steel equally on both faces of steining.

$\therefore$ Area of vertical steel on each face = 21206 mm^2

Provide 48 Nos. of equally spaced 25ϕ bars on the inner and outer faces of the steining. Keep the vertical bars equally spaced.

Area of vertical steel provided = 23561.94 > 21206 mm^2, Hence O.K.

Provide hoop steel at 0.04 % of the volume per unit height of steining.

$\therefore$ Volume of hoop steel per 'm' height of steining.

$= \frac{0.04}{100} \times \frac{\pi}{4} \times (9^2 - 6^2) \times 1.0$

$= .014137107\ m^3$

$= 141.37107\ mm^3$

Volume of hoop steel required on each face

$= 7068583.5\ mm^3$

Total cross-sectional area of hoop bars required on each face per meter height of steining.

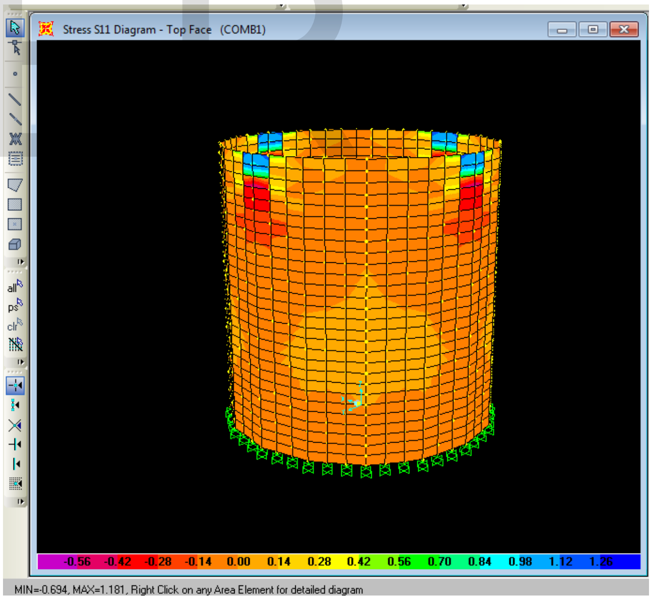
$= \frac{7068583.5}{[\pi \times (9000 - 75 - 5)]} = 252.24\ mm^2$

Provide 12ϕ hoops in the form of closed rings on both the inner as well as the outer face of the well steining @ 250 mm c/c.

Area provided = 452.39 mm^2 > 252.24 mm^2 O.K.

9. SAP Model of Steining

To check the hoop stresses on the steining, a modelling on SAP has been done



As can be seen from the stress contours all the stresses are within the permissible limits for M25. Hence steining is safe.

10. Design of well curb

Provide normal steel at 72 kg/m^3 in the well curb.

Vol. of concrete in well curb

$= \frac{1.6+.15}{2} \times 3 \times \square \times \left[9 - \left(\frac{1.6-.15}{2}\right) x2\right]$

$= 62.26\ m^3$

Total weight of steel in well curb = 72 x 62.26
=4482.72 kg

Consider the following arrangement of steel in well curb.

(i) 40 nos. of 25ɸ hoops of average dia. = 7.25 m.
$\therefore$ Weight provided = 3510.6491 kg.
(ii) 20 mm ɸ triangular at 280mm c/c.
$\therefore$ Total no. of rings = 80
Average Length of one ring
= (1.6 – (2 x 0.075)) + (3 - (2 x 0.075)) + (3.332 - 2.075)
= 5.557 m
$\therefore$ weight of 80 stirrups.

$$= 80 \times \frac{\pi \times 0.020^2}{4} \times 5.557 \times 7850 = 1096.35 \text{ kg.}$$

Total weight of steel provided in well curb
= 3510.65 + 1096.35 kg
= 4607 > 4482.72 kg

Hence O.K.

CONCLUSION

SAP modeling has been carried out to check the stress contours for all the stresses are within the permissible limits for M25 or not And it is found that all the stresses are in permissible limit. Hence steining is safe.

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