



IIT ROORKEE



NPTEL ONLINE
CERTIFICATION COURSE

EARTHQUAKE RESISTANT DESIGN OF FOUNDATIONS

Lecture 28: Lateral Stability of Well Foundations

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ERD of Foundations - Modules

1. Introduction
2. Shallow Foundations
3. Pile Foundations
4. Well Foundations
5. SSI for Deep Foundations



Module - 4

Well Foundations



Module -4: Chapters

1. Introduction
2. Lateral Stability of Well Foundations
3. Construction and Sinking of a Well
4. Numerical Problems



Force Acting on Wells

In addition to the dead load and live load well is also subjected to-

- a) Wind force/Seismic force
- b) Force due to water current
- c) Force due to movement of vehicle
- d) Temperature
- e) Earth pressure
- f) Buoyancy
- g) Centrifugal

Combination (I)

N-case where all forces are considered except seismic (S) and temperature (T) force

1. For N+T permissible stress increase by 15%
2. For N+S permissible stress increase by 25%
3. For N+T+S permissible stress increase by 50%

Force Acting on Wells

The three equations satisfying the conditions of equilibrium:

a) $\sum V = 0$

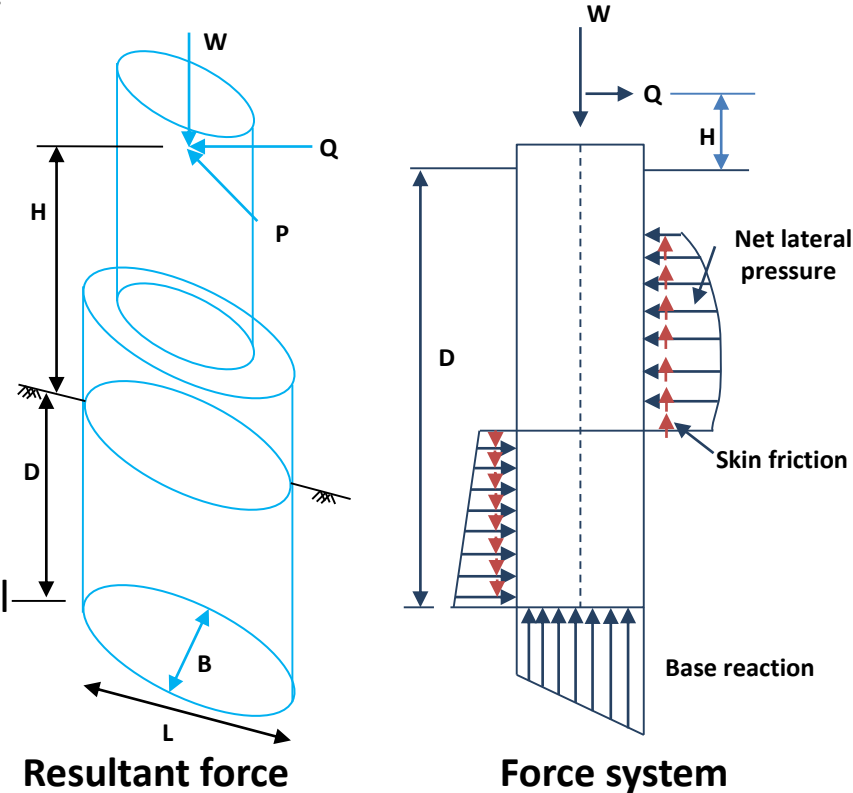
The total downward force (W) = Base reaction + friction on the side

b) $\sum H = 0$

Lateral force (Q) = Net lateral earth pressure including frictional at the base

c) $\sum M = 0$

$Q(H+D) =$ Sum of the moments due to net lateral earth pressure, friction on both side and base reaction



Chapters 2 Lateral Stability of Wells

Several methods are available:

Terzaghi (1943): Assumed a free, rigid bulkhead for lateral stability analysis of wells.

Pender (1947): The soil around the pier was assumed to behave like linear spring whereas stiffness increase with depth.

Banerjee & Gangopadhyay (1960): Utilized Pender's plastic flow concept with Terzaghi's rigid bulk stress distribution and subgrade modulus concept.

Indian Road Congress (IRC: 45-1972): Formulated recommendation for well in a cohesionless soil to estimate soil resistance bellow maximum scour level.

Terzaghi (1943)

When a rigid well sand deposit moves parallel to its original position, the sand on the front face of the well is transformed into a passive state whereas on the rare face, it is transformed into an active state. Thus when the passive and active pressure are fully developed, the net resultant pressure p_z at any depth z is given by

$$p_z = \gamma z (K_p - K_A) \quad (1)$$

Where, γ =Unit weight of soil

K_p = Coefficient of passive earth pressure

K_A =Coefficient of active earth pressure

Terzaghi (1943) Cont....

Considering a free, rigid bulkhead in a sand deposit. The bulkhead is considered free if it owes its stability solely to the lateral resistance of the earth adjoining its buried part. The bulkhead with an embedment depth D is subjected to a horizontal force q . As the magnitude of the horizontal force increases gradually, the bulkhead tends to rotate about certain point O above the lower edge of the bulk head. At failure, the soil is in a state of plastic equilibrium.

Terzaghi (1943) Cont....

The resistance offered by the soil can be approximated by the pressure diagram

Resultant pressure per unit length = $q_{\max}$

$q_{\max}$ = Area of pressure diagram (EHB-FHG)

$$= \frac{1}{2} \gamma D^2 (K_P - K_A) - \frac{1}{2} 2\gamma D (K_P - K_A) D_1$$

$$= \frac{1}{2} \gamma D (K_P - K_A) (D - 2D_1)$$

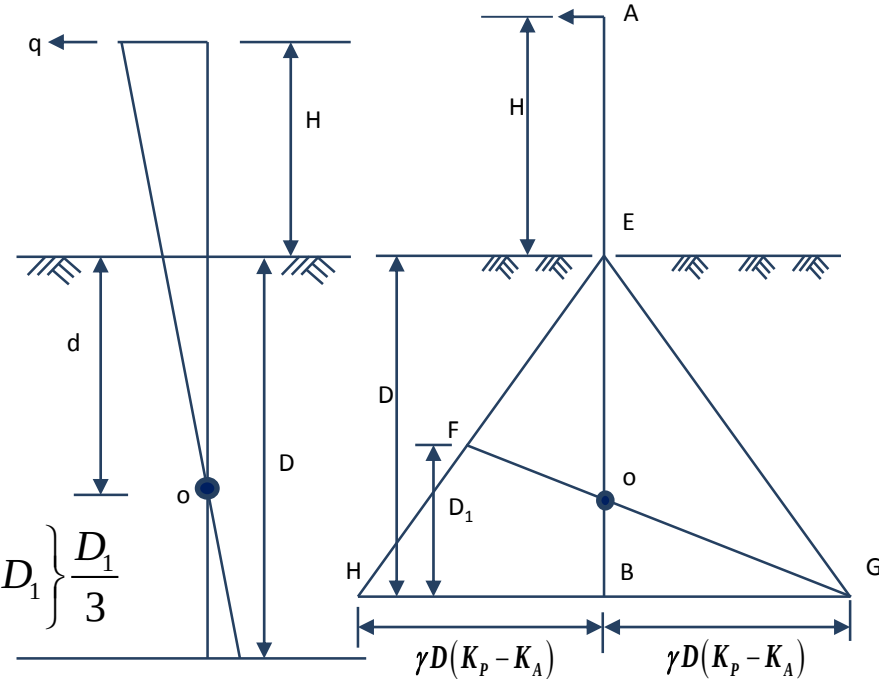
Equating the moments about the base,

$$q_{\max} (H + D) = \left\{ \frac{1}{2} \gamma D^2 (K_P - K_A) \right\} \frac{D}{3} - \left\{ \frac{1}{2} 2\gamma D (K_P - K_A) D_1 \right\} \frac{D_1}{3}$$

$$= \frac{1}{2} \gamma D (K_P - K_A) \left\{ \frac{D^2}{3} - \frac{2}{3} D_1^2 \right\}$$

(2)

(3)



Force on a bulkhead

Terzaghi (1943) Cont....

Substituting the values of $q_{\max}$ in Eq 3. from Eq. 2 and take $H+D=H_1$

$$\frac{1}{2}\gamma D(K_P - K_A)(D - 2D_1)(H + D) = \frac{1}{2}\gamma D(K_P - K_A)\left\{\frac{D^2}{3} - \frac{2}{3}D_1^2\right\}$$

$$(D - 2D_1)H_1 = \frac{1}{3}(D^2 - 2D_1^2)$$

$$3H_1(D - 2D_1) = (D^2 - 2D_1^2)$$

$$2D_1^2 - 6H_1D_1 + 3DH_1 - D^2 = 0$$

$$D_1^2 - 3H_1D_1 + (1.5DH_1 - 0.5D^2) = 0$$

Solve for D_1

$$D_1 = \frac{3H_1 \pm \sqrt{9H_1^2 - 4(1.5DH_1 - 0.5D^2)}}{2}$$

$$2D_1 = 3H_1 \pm \sqrt{9H_1^2 - 2D(3H_1 - D)}$$

Thus D_1 can be found from the Eq. 4 and substitutes the value of D_1 in Eq. 2, $q_{\max}$ can be found

(4)

Terzaghi (1943) Cont....

Terzaghi's theory yield conservative results as it neglect friction at the base and the sides. If surrounding soil is submerged, the submerged unit weight of soil should be used for $q_{\max}$. A heavy well under a lateral load will rotate about the base. The value of $q_{\max}$ can be computed by taking moments about base.

$$q_{\max} = \frac{1}{6} \gamma (K_P - K_A) \frac{D^3}{H + D} \quad (5)$$

If there is unscoured soil upto the bed level, its influence can also be taken into account by considering earth pressure due to the surcharge. If Z is equivalent surcharge height, the value of $q_{\max}$ is given by

$$q_{\max} = \frac{1}{6} \gamma (K_P - K_A) \frac{D^2 (D + Z)}{H + D} \quad (6)$$

For a well with a length or diameter equal to L , the total resistance force $Q_{\max}$ is given by

$$Q_{\max} = q_{\max} \times L \quad (7)$$

Terzaghi (1943) Cont....

Adopting a factor of safety F against the passive resistance of the soil, the allowable horizontal force Q_a is given by

$$Q_a = \frac{q_{\max} \times L}{F} \quad \text{usually } F \geq 2 \quad (8)$$

If the applied horizontal force on the well is Q and the allowable equivalent force is Q_a , there is an unbalanced horizontal force, $(Q - Q_a)$ acting at a height H above the scour level which results in an overturning moment M_B at the base.

$$M_B = (Q - Q_a)(H + D) \quad (9)$$

Thus the maximum and minimum foundation pressure at the base, $f_{\max}$ and $f_{\min}$ are given by

$$f_{\max}, f_{\min} = \frac{W}{A} \pm \frac{M_B}{Z_B} \quad (10)$$

Where, W = Net direct vertical load on the base of the well after taking into account the buoyancy and skin friction, A = Area of the base of the well and Z_B = Section modulus of the wall base

Terzaghi (1943) Cont....

The maximum foundation pressure should be less than the allowable bearing pressure. The well steining is subjected to a bending moment due to the horizontal load Q . The maximum bending moment in the steining occurs where the resultant shear force is zero.

The point of zero shear which lies at a depth x below scour level is given by

$$x = \left[\frac{2FQ}{\gamma(K_P - K_A)L} \right]^{\frac{1}{2}} \quad (11)$$

Taking moments about the point of zero shear,

$$\begin{aligned} M_{\max} &= Q(H + x) - \frac{Qx}{3} \\ &= QH + \frac{2}{3}Qx \end{aligned} \quad (12)$$

If the applied moment at scour level is M , then

$$M_{\max} = M + \frac{2}{3}Qx \quad (13)$$

Pender's Theory

Pender proposed an analysis for lateral stability of wells. Two different types of wells, namely, light and heavy wells were considered. The **sandy soil** around the well was assumed to behave as **linear spring with stiffness increasing with depth**.

In a heavy well, friction was assumed to develop at the base, whereas in the case of light well, no friction was assumed to develop at the base. In heavy well, the well was assumed to rotate about the base; whereas in the light well, the rotation was about a point above the base. Equations satisfying the conditions of static equilibrium were solved with the soil modulus obtained in terms of Rankine's earth pressure coefficients, using the concept of imminent plastic condition at the surface.

Banerjee & Gangopadhyay (1960)

Using Pander's (1947) plastic flow concept and Terzaghi's rigid bulkhead stress distribution with subgrade modulus concept, Banerjee & Gangopadhyay (1960) proposed an analysis for the lateral stability of well. The analysis was developed for a well in sandy soil. Two cases were considered: a general case with **no plastic flow** and a second case with **partial plastic flow**. The equation for the various forces acting on the well were worked out and conditions of equilibrium satisfied. Modification for heavy wells with rotation about the base have also been suggested.

Indian Road Congress (IRC) method

Lateral stability of the well is provided by the passive resistance of the soil on its side and base. The resistance developed at working loads is different from that at ultimate load.

IRC (45-1972) recommends elastic theory method to estimate the soil pressure on the side and at the base under design load and ultimate soil resistance for actual factor of safety against shear failure. This is applicable for non-cohesive soil like sand and surrounded by some soil bellow maximum scour level. The recommendation will not apply, if the depth of embedment is less than 0.5 times the width of the foundation in the direction of lateral force.

Elastic Theory Method

The assumption made in the elastic theory method are:

- The soil surrounding the well and below the base is perfectly elastic, homogeneous and follows Hooke's law.
- Under design load, the lateral deflection are so small that the unit soil reaction, p increases linearly with increasing lateral deflection Z such that $p=K_H Z$ where K_H is the coefficient of horizontal subgrade reaction at base.
- The coefficient of horizontal subgrade reaction increases linearly with depth in cohesionless soil.
- The well behaves as a rigid body and is acted upon by a unidirectional horizontal force H and a moment M at scour level.

The various steps in calculations are:

Step 1. Having determined the minimum grip length, the applied loads and moments are calculated.

Let W = total downward load acting at the base of the well

H = horizontal force acting on the well at scour level

M = total applied moment about the base of the well, including those due to tilts and shifts

Step 2. Using the well dimensions, calculate the following geometrical properties:

I_B = moment of inertia of the base about an axis passing through the center of gravity and perpendicular to the horizontal resultant force.

$$= \frac{\pi}{64} B^4 \text{ for a circular well of diameter } B.$$

I_V = moment of inertia about the horizontal axis passing through the center of gravity of the projected area in elevation of the soil mass offering resistance.

$$= \frac{LD^3}{12} \text{ and } I = I_B + mI_V (1 + 2\mu'\alpha) \quad (1)$$

Where, B = diameter of circular well or the width of the base parallel to the direction of the horizontal force.

L= projected width of well in contact with soil offering passive resistance multiplied by appropriate shape factor. A value of 0.9 is recommended for shape factor for circular wells, whereas 1.0 is recommended for square and rectangular wells.

m = ratio of horizontal to vertical coefficient of subgrade reaction $= \frac{K_H}{K_V}$
 $\mu' =$ coefficient of friction between the sides of well and the soil $= \tan \delta$

$\alpha = \frac{B}{2D}$ for a rectangular well (D = depth of well below scour level)

Step 3. Check that the point of rotation of the well lies at the base by ensuring that the frictional force at the base is adequate to restrain the movement of well.

$$H > \frac{M}{r}(1 + \mu\mu') - \mu W \quad (2a)$$

$$H < \frac{M}{r}(1 - \mu\mu') + \mu W \quad (2b)$$

$$r = \frac{D}{2} \left(\frac{I}{mI_v} \right) \text{ and } \mu = \tan \phi \quad (2c)$$

Where, μ = is the coefficient of friction between the base and soil. The angle is taken equal to ϕ due to roughness of the concrete plug.

Step 4. Check that the soil on the sides remain elastic. This can be ensure by keeping the slope of the pressure parabola at the top below the passive line, that is,

$$\frac{mM}{I} \nless \gamma (K_p - K_A) \quad (3)$$

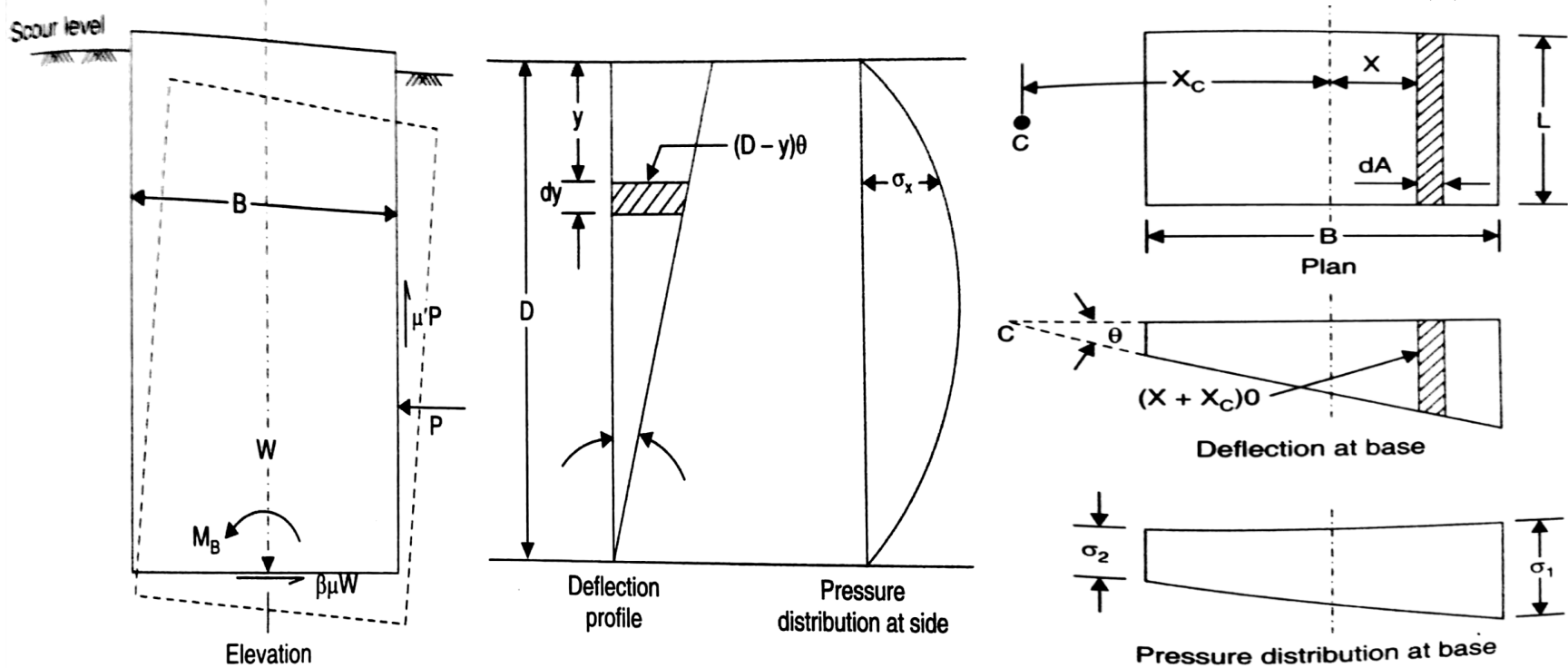
Step 5. Check the base pressure, ensuring that the maximum compressive pressure is less than the allowable bearing pressure and that the minimum pressure is not tensile.

$$\sigma_1, \sigma_2 = \frac{W - \mu'P}{A} \pm \frac{MB}{2I} \quad (4)$$

Where, A = base area of well, P= total horizontal reaction from the side = M/r and B= width of the base of the well in the plan of bending.

If the conditions in steps 3,4 and 5 above are not satisfied, the well shall be redesigned.
The equations used in elastic theory are derived as follows:

Total deflection at depth y below scour level $(D-y)\theta$



Horizontal soil reaction,

$$\begin{aligned}\sigma_x &= K_H \frac{y}{D} (D - y) \theta \\ &= m \frac{K_V \theta}{D} y (D - y)\end{aligned}\quad (6)$$

Conditions of stability:

- I. If the soil remains in an elastic state, the minimum soil reaction from the sides cannot exceed the minimum passive pressure at any depth. Thus, at any depth y

$$\sigma_x \nless (K_P - K_A)\gamma y \quad (7)$$

Thus from Eq. (6)

$$\begin{aligned} \frac{mK_V\theta}{D} y(D-y) &\nless (K_P - K_A)\gamma y \\ \text{or} \\ \frac{mK_V\theta}{D} (D-y) &\nless (K_P - K_A)\gamma \end{aligned} \quad (8)$$

The term on left side of Eq. (8) is max at $y = 0$

Hence,

$$mK_V\theta \nless \gamma(K_P - K_A) \quad (9a)$$

or

$$m\frac{M}{I} \nless \gamma(K_P - K_A) \quad (9b)$$

Conditions of stability:

II. The maximum soil pressure at base σ_1 should be less than the allowable bearing pressure ($\sigma_1 < q_a$) and the minimum soil pressure at base σ_2 should be less than zero i.e. no tension should develop at the base ($\sigma_2 > 0$)

References

1. Gopal Ranjan and A S R Rao (2016) “Basic and Applied Soil Mechanics” New Age International, New Delhi, India. (Third Edition)
2. IS 3955-1967 “Code of Practice for Design and Construction of Well Foundations” Bureau of Indian Standards, New Delhi, India.
3. IRC: 45 (1972)- “Recommendations for Estimating the Resistance of Soil Below the Maximum Scour Level in the Design of Well Foundations of Bridges”, Indian Road Congress, New Delhi.