grout after the tube is sunk, and forcing it by air pressure into the adjacent soil.

These piles are formed in three sizes: 340 mm dia (by using 300 mm dia. tubes), 440 mm dia. (400 mm tubes) and 500 mm dia. (460 mm tubes), upto a length of 25 m.

PROBLEMS

1. A single acting steam hammer weighing 2000 N and falling through a height of 80 cm drives a pile to an average penetration of 1 cm per blow under the last few blows. Determine the allowable load for the pile, using Enginering News Formula. **[Ans.** 213 kN]

2. A concrete pile 40 cm × 40 cm section and 20 m long is driven by a drop hammer having a weight of 40 kN and falling through a height of 1 m. The average penetration under last 10 blows is equal to 6 mm per blow. The efficiency of hammer = 100%, the coefficient of restitution is 0.4 and the total elastic compression 25 mm. Using Hilley's formula, determine (*i*) Q_f, (*ii*) the set to which the pile should be driven under the above conditions if Q_f desired is 1600 kN and (*iii*) $(Q_f)_{max}$.

[Ans. (*i*) 968 kN (*ii*) $S = -0.13$ cm (i.e. pile will be compressed and it will not be capable of developing $Q_f = 1600$ kN, (*iii*) $(Q_f)_{max} = 1432$ kN (at $S = 0$)].

3. A square pile of section 50 × 50 cm and 10 m long penetrates a deposit of clay with $c = 40\ kN/m^2$. Taking $m = 0.7$, determine the load carried by the pile by skin friction. **[Ans.** 560 kN]

4.* A-25 pile square group has to be proportioned in a uniform pattern in clay with equal spacing in all directions. Taking $c=0.7$, determine the optimum value of spacing. Neglect end bearing effect of the group. Each pile is square in cross-section, with sides of length a.

[Ans. s = 4.125 a]

5.* Determine the settlement in Example 26.6 if the length of piles is increased by 3 m. Other data remain unchanged. **[Ans** 15.1 cm]

6.* A 45 cm diameter concrete pile 7 m long is subjected to a horizontal load of 20 kN at a height of 2 m above the ground. Taking $\eta_h = 20\ N/cm^3$ and E for R.C.C. pile $= 3 \times 10\ N/cm^2$, calculate the maximum deflection and maximum moment induced in the pile.

[Ans (*i*) $y_{max} = 0.323$ cm (*ii*) $M_{max} = 52.7$ kN-m at a depth of 1 m below ground surface]

7.* A single pile with free head is subjected to lateral load at the ground surface. The deflection under a load of 20 kN is 2 cm. Compute the deflection if the pile were fixed at the head. **[Ans.** 0.76 cm.]

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27

Well Foundations

27.1. INTRODUCTION : CAISSONS

The term 'Caisson' is derived from the French word, *caisse*, meaning a chest or box. Caisson has come to mean a boxlike structure, round or rectangular, which is sunk from the surface of either land or water to some desired depth. The caissons are of three types:

(*i*) Box caissons
(*ii*) Open caissons (wells)
(*iii*) Peneumatic caissons.

A *box caisson* is open at top and closed at the bottom and is made of timber, reinforced concrete or steel. This caisson is built on land, then launched and floated to pier site where it is sunk in position. Such a type of caisson is used where bearing stratum is available at shallow depth, and where loads are not very heavy. Closed box caissons are also used for break waters and sea walls. An *open caisson* is a box of timber, metal, reinforced concrete or masonry which is open both at the top and at the bottom, and is used for building and bridge foundations. Open caissons are called *wells. Well foundations* form the most common type of deep foundations for bridges in India. A *pneumatic caisson* has its lower end designed as a working chamber in which compressed air is forced to prevent the entry of water and thus permit excavation in dry.

Whenever considerations for scour or bearing capacity require foundations being taken to a depth of more than 5 to 7 metres, open excavations become costly and uneconomic as heavy timbering has to be provided. Also because of the greater earth work involved due to side slopes, the progress of the work in open excavation will be very slow. Another disadvantage in adopting the ordinary type of footing is that excavated material refilled around the structure is loose and hence easily scourable as compared to natural ground.

The above disadvantages are avoided in a well foundation which is a shell sunk by dredging inside of it and which finally becomes a part of the permanent structure.

27.2. SHAPES OF WELLS AND COMPONENT PARTS

The common types of well shapes are as follows (Fig. 27.1) :

(1) Single circular
(2) Twin circular
(3) Dumb-well
(4) Double-D
(5) Twin-hexagonal
(6) Twin-octagonal
(7) Rectangular.

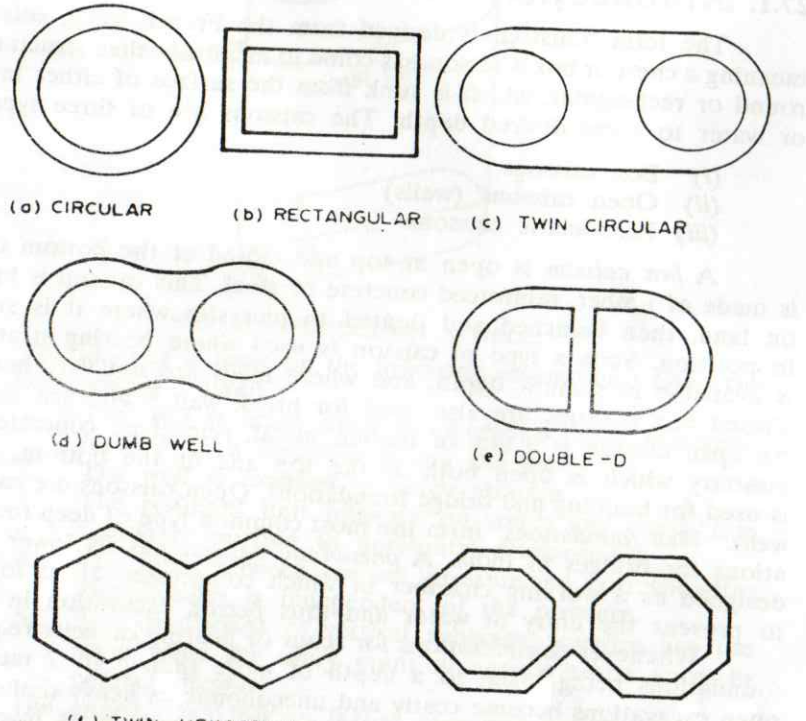

FIG. 27.1. SHAPES OF WELLS.

The choice of a particular shape depends upon the dimensions of the base of the pier or abutment, the care and cost of sinking, the considerations of tilt and shift during sinking and the vertical

and horizontal forces to which the well is subjected. A circular well has the minimum perimeter for a given dredge area and hence the ratio of sinking effort to skin friction is maximum. Also, since the perimeter is equidistant at all points from the centre of dredge hole, the sinking is more uniform than for other shapes. However, the disadvantage of a circular well is that in the direction parallel to the span of the bridge, the diameter of the well is much more than the minimum size required to accommodate the bridge pier and hence the circular well causes more obstruction to waterway than the bridge pier does. The disadvantage is avoided in the case of a double–D shape which conforms to the shape of the bridge pier in plan. The dredge area is also smaller for a double-D. Hence

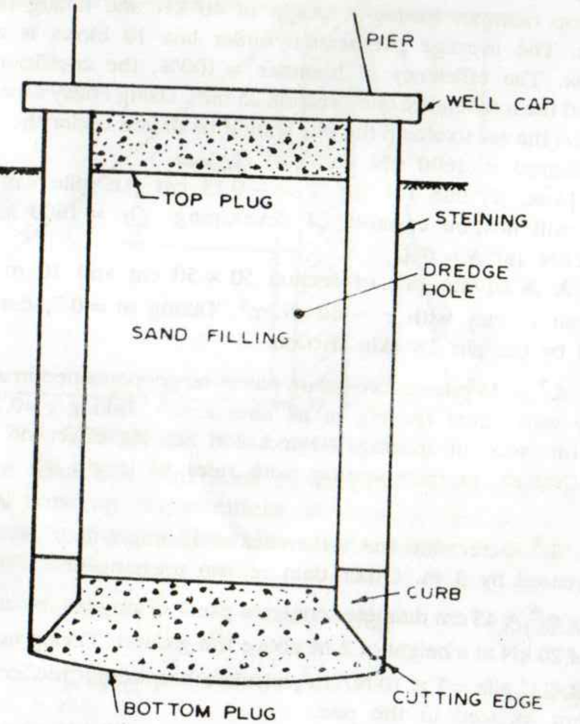

FIG. 27.2. SECTION OF A WELL FOUNDATION.

for large piers, a double–D is more economical than a single circular well. Twin circular well aim at combining the advantage of a circular well and of a double–D, but the only snag is that the two wells sunk close to each other have a tendency to close in or move apart. However, in abutments and wing walls where the tilt and shift in position are not important, a battery of small diameter wells are adopted with advantage. The double hexagon and double octagon types, through provide efficient grabbing to all parts of the curb, suffer from the disadvantage that owing to sharp corners they can dig and are, therefore, more likely to tilt. Also, the sharp corners produce greater scour.

Fig. 27.2 shows a typical section of a well foundation with its component parts. The following components of a well have to be considered in the design of a well foundation :

(*i*) Well curb and cutting edge.

(*ii*) Steining.

(*iii*) Bottom plug.

(*iv*) Well cap.

27.3. DEPTH OF WELL FOUNDATION AND BEARING CAPACITY

The selection of the depth of a well is based on the following two criteria :

1. There should be adequate embedded length of well, called the *grip length* below the lowest scour level. In addition to minimum Rankine depth consideration, this is required for developing sufficient passive resistance to counteract the overturning moment due to horizontal forces acting on the bridge deck, as well as those due to wind and water.

2. The well should be taken deep enough to rest on strata of adequate bearing capacity in relation to the loads transmitted.

For alluvial soils, mostly met with in North Indian rivers, the *normal scour depth* can be calculated by Lacey's Formula :

$$R_L = 1.35\left(\frac{q^2}{f}\right)^{1/3} \quad ...(27.1)$$

where q = discharge in cumecs per linear metre of water way

f = Lacey's silt factor

$$= 1.76\sqrt{m_d} \quad ...(27.2)$$

m_d = mean weighted diameter in mm.

The maximum depth of scour, at the nose of pier, is found to be twice the Lacey's value of normal scour depth :

$$R = 2R_L \quad ...(27.3)$$

where R is measured below the high flood level (HFL).

$$\therefore \quad \text{Scour level} = \text{H.F.L.} - R$$
$$= \text{H.F.L.} - 2R_L \quad ...(27.4)$$

The grip length is taken as $\frac{1}{3}R$ below the scour level according to the code of practice of the Indian Roads Congress and as $\frac{1}{2}R$ in Railway practice. This means that the depth of foundation should be at least $1\frac{1}{3}R$ below HFL according to IRC code, and $1\frac{1}{2}R$ below HFL according to Railway practice . It is further recommended that the minimum depth of embedment below the scour level should not be less than 2.0 m for piers and abutment with arches and 1.2 m for piers and abutments supporting other types of superstructure.

According to Terzaghi and Peck, the ultimate bearing capcaity can be determined from the following expression:

$$Q_f = Q_p + 2\pi R f_s D_f \quad ...(27.5)$$

$$Q_P = \pi R^2 (1.2\, cN_c + \gamma D_f N_q + 0.6\gamma R N_\gamma) \quad ...(27.6)$$

where N_c, N_q, N_γ = Terzaghi's bearing capacity factors

R = radius of well

D_f = depth of well (depth of foundation)

f_s = average skin friction

27.4. FORCES ACTING ON A WELL FOUNDATION

In addition to the self-weight and buoyancy, a well carries the dead load of the super-structure, bearings pier and is liable to the following horizontal forces :

(*i*) bracking and tractive effort of the moving vehicles.

(*ii*) force on account of resistance of the bearings against movement due to variation of temperature,

(*iii*) force on account of water current,

(*iv*) wind focres,

(*v*) seismic forces,

(*vi*) earth pressure,

(*vii*) centrifugal forces.

The magnitude, direction and point of application of all the above forces can be found under the worst possible combinations and they can be replaced by two horizontal forces, P and Q and a single vertical force W as shown in Fig. 27.3.

P = Resultant of all horizontal forces in the direction across the pier.

Q = Resultant of all horizontal forces in the direction along the pier.

W = Resultant of all vertical forces.

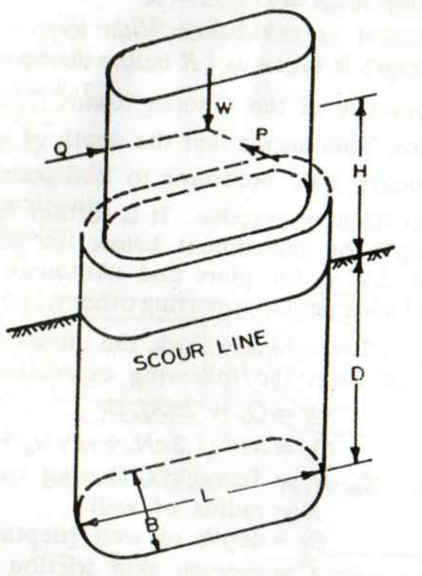

FIG. 27.3. FORCES ON A WELL.

The analysis is done on the following assumptions (Banerjee and Gangopadhyay, 1960) :

1. The well is acted upon by an unidirectional horizontal force P in a direction across the pier.
2. The well is founded in sandy stratum.
3. The resultant unit pressure on soil at any depth is in simple proportion to horizontal displacement.
4. The ratio between contact pressure and corresponding displacement is independent of the pressure.
5. The co-efficient of vertical subgrade reaction has the same value for every point of surface acted upon by contact pressure.

The analysis that follows is that suggested by Banerjee and Gangopadhyay (1960).

27.5. ANALYSIS OF WELL FOUNDATION

1. Horizontal soil reactions. When a rigid well, embedded in sand, starts moving parallel to its original position, under the action of a horizontal force P, it transforms the soil on one side to passive state of plastic equilibrium and the other side into active state. Assuming that the well movement ρ_1 is sufficient to mobilise fully the active and passive earth pressure, the resultant unit pressure at a depth z below the surface is given by

$$p_1 = \gamma . z\,(K_p - K_a) \quad ...(27.7)$$

where γ = unit weight of soil

K_p, K_a = co-efficient of passive and active earth pressure, and depend upon the angle of internal friction ϕ, and angle of wall friction δ.

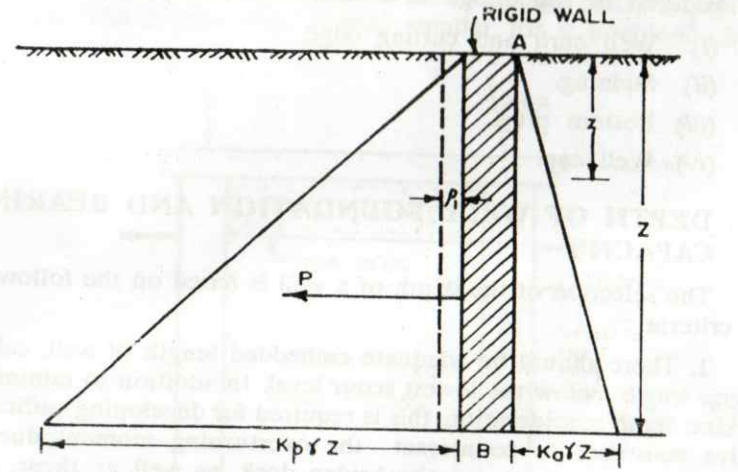

FIG. 27.4. EFFECT OF WALL MOVEMENT.

Let p be the load per unit area of vertical surface of sand and ρ be the corresponding displacement. Assuming that ρ_1 is the displacement required to increase the value of resultant unit pressure form zero to p_1, we have.

$$p = p_1 \frac{\rho}{\rho_1} = \frac{\rho}{\rho_1} \gamma z\,(K_p - K_a) \quad ...(27.8)$$

or

$$\frac{p}{\rho} = m . z \quad ...(27.9)$$

where

$$m = \frac{\gamma\,(K_p - K_a)}{\rho_1} \quad ...(27.10)$$

Factor m is called the *coefficient of horizontal soil reaction* which depends not only on the nature of soil but also on the size and shape of the area which carries the load.