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**STANDARD SPECIFICATIONS
AND
CODE OF PRACTICE
FOR
ROAD BRIDGES**

SECTION : VII

FOUNDATIONS AND SUBSTRUCTURE,

(Second Revision)



**THE INDIAN ROADS CONGRESS
2000**

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AND
CODE OF PRACTICE
FOR
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FOUNDATIONS AND SUBSTRUCTURE
(*Second Revision*)**



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(As on 19.8.2000)

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FOUNDATIONS AND SUBSTRUCTURE

BACKGROUND

The "Standard Specifications and Code of Practice for Road Bridges" Section : VII-Foundations and Substructure was first published in July 1980 as Part I - General Features of Design. Later first revision was published in December 1983 incorporating Part II and amendments 1, 2 and 3 to Part I as a Unified Code. The second revision of this code was undertaken by the Foundation and Substructure Committee (B-4) and the initial draft was finalised by the Committee under the Convenorship of Shri R.H. Sarma. Subsequently, the draft was reconsidered and discussed in various meetings by the reconstituted Foundation, Substructure and Protective Works Committee (B-4) (personnel given below) and the draft was finalised during its meeting held on 1st February, 1999 :

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The draft as finalised by (B-4) Committee was discussed by the Bridges Specifications & Standards (BSS) Committee in its meeting held on 7.12.99 and it was decided to modify the draft by the Convenor of (B-4) Committee in light of comments offered during the meeting. The modified draft was again discussed by BSS Committee in its meeting held on 19.8.2000 and was approved subject to certain modifications and authorised its Convenor to approve the document after incorporating the modifications. The final draft as approved by Convenor, BSS Committee was subsequently approved by the Executive Committee in its meeting held on 30.8.2000. It was later approved by the Council in its 160th meeting held at Calcutta on 4.11.2000 for publishing the revised IRC Bridge Code Section VII : IRC:78.

700. SCOPE

This code deals with the design and construction of foundations and substructure for road bridges. The provisions of this code are meant to serve as a guide to both the design and construction engineers, but mere compliance with the provisions stipulated herein will not relieve them in any way of their responsibility for the stability and soundness of the structure designed and erected.

701. TERMINOLOGY

The following definitions shall be applicable for the purpose of this code.

701.1. Abutment

The end supports of the deck (superstructure) of a bridge, which also retains earth, fill of approaches behind fully or partly.

701.1.1. **Box type abutment and return wall** : When the return walls on two sides are integrated with abutment and a back wall parallel to abutment is provided at the end of returns with or without additional internal wall along or across length, this structure is called box type abutment and return wall, or end block.

701.1.2. **Non-load bearing abutment** : Abutment, which supports the end span of less than 5 m.

701.1.3. **Non-spill through abutment** : An abutment structure where the soil is not allowed to spill through.

701.1.4. **Spill through abutment** : An abutment where soil is allowed to spill through gaps along the length of abutment, such as, column structure where columns are placed below deck beams and gap in between is free to spill earth. (Spilling of earth should not be permitted above a level of 500 mm below the bottom of bearings).

701.2. **Afflux**

The rise in the flood level of the river immediately on the upstream of a bridge as a result of obstruction to natural flow caused by the construction of the bridge and its approaches.

701.3. **Balancer**

A bridge/culvert like structure provided on embankment to allow flow of water from one side of the embankment to otherside, for purpose of avoiding heading up of water on one side or for avoiding blocking the entry to the other side.

701.4. **Bearing Capacity**

The supporting power of a soil/rock expressed as bearing stress is referred to as its bearing capacity.

701.4.1. **Allowable bearing pressure** : It is the maximum gross pressure intensity at which neither the soil fails in shear, (after accounting for appropriate factor of safety) nor there is excessive settlement beyond permissible limits, which is expected to be detrimental to the structure.

701.4.2. **Net safe bearing capacity** : It is the net ultimate bearing capacity divided by a factor of safety as per Clause 706.3.1.1.1.

701.4.3. **Net ultimate bearing capacity** : It is the minimum net pressure intensity causing shear failure of the soil.

701.4.4. **Safe bearing capacity** : The maximum pressure, which the soil can carry safely without risk of shear failure and it is equal to the net safe bearing capacity plus original overburden pressure.

701.4.5. **Ultimate bearing capacity** : It is the minimum gross pressure intensity at the base of the foundation at which the soil fails in shear.

701.5. **Bearing Stress**

701.5.1. **Gross pressure intensity** : It is the total pressure at the base of the foundation on soil due to the possible combinations of load and the weight of the earth fill.

701.5.2. **Net pressure intensity** : It is the difference in intensities of the gross pressure and the original overburden pressure.

701.6. **Cofferdam**

A structure temporary built for the purpose of excluding water or soil sufficiently to permit construction or proceed without excessive pumping and to support the surrounding ground.

701.7. **Foundation**

The part of a bridge in direct contact with and transmitting load to the founding strata.

701.8. **Pier**

Intermediate supports of the deck (superstructure) of a bridge.

701.8.1. **Abutment pier** : Generally used in arch bridges. Abutment pier is designed for a condition that even if one side arch span collapses it would be safe. These are provided after three or five spans.

701.9. **Piles**

701.9.1. **Bearing/friction piles** : A pile driven or cast-in-situ for transmitting the weight of a structure to the founding strata by the resistance developed at the pile base and by friction along its surface. If it supports the load mainly by the resistance developed at its base, it is referred to as an end-bearing pile, and if mainly by friction along its surface, as a friction pile.

701.9.2. **Bored cast-in-place pile** : A pile formed with or without a casing by boring a hole in the ground and subsequently filling it with plain or reinforced concrete.

701.9.3. **Driven cast-in-place pile** : A pile formed in the ground by driving a permanent or temporary casing, and filling it with plain or reinforced concrete.

701.9.4. **Driven pile** : A pile driven in to the ground by the blows of a hammer by a vibrator.

701.9.5. **Precast pile** : A reinforced or prestressed concrete pile cast before driving, or installing in bore and grouted.

701.9.6. **Raker or batter pile** : A pile installed at an inclination to the vertical.

701.9.7. **Sheet pile** : One or a row of piles driven or formed in the ground adjacent to one another in a continuous wall, each generally provided with a connecting joint or interlock, designed to resist mainly lateral forces and to reduce seepage; it may be vertical or at an inclination.

701.9.8. **Tension pile** : A pile subjected to tension/uplift is called tension pile.

701.9.9. **Test pile** : A pile to which a load is applied to determine and/or confirm the load characteristics (ultimate load/working load) of the pile and the surrounding ground.

701.9.10. **Working pile** : One of the piles forming the foundation of the structure.

701.10. **Retaining Wall**

A wall designed to resist the pressure of earth filling behind.

701.10.1. **Return wall** : A wall adjacent to abutment generally parallel to road or flared up to increase width and raised upto the top of road.

701.10.2. **Toe wall** : A wall built at the end of the slope of earthen embankment to prevent slipping of earth and/or pitching on embankment.

701.10.3. **Wing wall** : A wall adjacent to abutment with its top upto road top level near abutment and sloping down upto ground level or a little above at the other end. This is generally at 45° to the alignment of road or parallel to the river and follows profile of earthen banks.

701.11. Substructure

The bridge structure, such as, pier and abutment above the foundation and supporting the superstructure. It shall include returns and wing walls but exclude bearings.

701.12. Well Foundation

A type of foundation where a part of the structure is hollow, which is generally built in parts and sunk through ground or water to the prescribed depth by removing earth through dredge hole.

701.12.1. **Tilt of a well** : The inclination of the axis of the well from the vertical expressed as the tangent of the angle between the axis of the well and the vertical.

701.12.2. **Shift of a well** : The horizontal displacement of the centre of the well at its base in its final position from its designed position.

702. NOTATIONS

For the purpose of this code, the following notations have been adopted:

A_1	Dispersed concentric area
A_2	Loaded area
B	Width between outer faces of pile group in plan parallel to the direction of movement
C	The allowable bearing pressure with near uniform distribution on the founding strata
c	Cohesion
C_o	The permissible direct compressive stress in concrete at the bearing area of the base
D	Diameter of pile
D_b	Discharge in cubic metre/sec (cumecs) per metre width
d	External diameter of circular well in metre

d_m	Weighted mean diameter in mm of bed material
d_m	Mean depth of scour in metre below flood level
F_b	Longitudinal force due to braking
F^{cf}	Centrifugal force
F_d	Deformation effects
F_h	Horizontal force
F^{ep}	Earth pressure
F^{eq}	Seismic force
F^{er}	Erection effects
F_f	Frictional force at bearings
F_{im}	Impact due to floating bodies
F_s	Secondary effects
F_{te}	Temperature effects [See Note (i)]
F_{wc}	Water current
F_{wp}	Wave pressure [See Note (ii)]
G	Dead load
G_b	Buoyancy
G_s	Snow load
h	Minimum thickness of steining in metre
K_a	Co-efficient of active earth pressure
K_p	Co-efficient of passive earth pressure
K_{sf}	Silt factor
L	Length between outer faces of pile group in plan parallel to the direction of movement
l_{te}	Movement of deck over bearings, other than due to applied force
l	Depth of well in metre below top of well cap
N	Standard penetration test value
P_a	Total active pressure
P_p	Total passive pressure
Q	Live load
R_g	Dead load reaction
R_l	Live load reaction
V_r	Shear rating of elastomeric bearing
W	Wind load
α	Horizontal seismic coefficient
β	Change the ratio of long side to the short side of the footing
μ	Co-efficient of friction
ϕ	Angle of internal friction

- δ Settlement of pile
 δ_g Settlement of pile group

- Notes :* (i) Temperature effects (F_{te}) in this context is not the frictional force due to the movement of bearing but that which is caused by rib shortening, etc.
- (ii) The wave forces shall be determined by suitable analysis considering drawing and inertia forces, etc., on single structural members based on rational methods or model studies. In case of group of piles, piers, etc., proximity effects shall also be considered.

703. DISCHARGE AND DEPTH OF SCOUR FOR FOUNDATION DESIGN

703.1. Design Discharge of Foundation

703.1.1. To provide for an adequate margin of safety, the scour for foundation shall be designed for a larger discharge over the design discharge determined as per IRC:5 as given below :

Catchment area in km ²	Increase over design discharge in per cent
0 - 3000	30
3000 - 10000	30 - 20
10000 - 40000	20 - 10
Above 40000	10

- Notes :* (i) For intermediate values of catchment area, linear interpolation may be adopted.
- (ii) The minimum vertical clearance above the HFL already determined as per IRC:5 need not be increased due to larger discharge calculated above.

703.2. Mean Depth of Scour

The mean scour depth below highest flood level (HFL) for natural channels flowing over scourable bed can be calculated theoretically from the following equation :

$$d_{sm} = 1.34 \left(\frac{D_b^2}{K_{sf}} \right)^{\frac{1}{3}}$$

where D_b = The design discharge for foundation per metre with at effective linear waterway.

K_{sf} = Silt factor for a representative sample of bed material obtained upto the level of anticipated deepest scour.

703.2.1. The value of D_b may be determined by dividing the design discharge for foundation by lower of theoretical and actual effective linear waterway as given in IRC:5.

703.2.2. ' K_{sf} ' is given by the expression $1.76(d_m)^{1/2}$, d_m being the weighted mean diameter in millimetre.

703.2.2.1. The value of K_{sf} for various grades of sandy bed are given below for ready reference and adoption:

Type of bed material	d_m	K_{sf}
Coarse silt	0.04	0.35
Silt/fine sand	0.081 to 0.158	0.5 to 0.6
Medium sand	0.233 to 0.505	0.8 to 1.25
Coarse sand	0.725	1.5
Fine bajri and sand	0.988	1.75
Heavy sand	1.29 to 2.00	2.0 to 2.42

703.2.2.2. No rational formula or data for determining scour depth for bed material consisting of gravels and boulders (normally having weighted diameter more than 2.00 mm) and clayey bed is available. In absence of any data on scour for such

material, the mean scour depth may be calculated following the guidelines given in *Appendix-1*.

703.2.3. If there is any predominant concentration of flow in any part of waterway due to bend of the stream in immediate upstream or downstream or for any other reason, like, wide variation of type of bed material across the width of channel, then mean scour depth may be calculated dividing the waterway into compartments as per the concentration of flow.

703.2.4. In case of bridge mainly adopted as balancer, the mean scour depth ' d_{sm} ' may be taken as (Highest Flood Level-Lowest Bed Level) divided by 1.27.

703.2.5. Scour depth may be determined by actual observations wherever possible. This is particularly required for clayey and boundary strata. Soundings, wherever possible, shall be taken in the vicinity of the site of the proposed bridge and for any structures nearby. Such soundings are best during or immediately after a flood before the scour holes have had time to be silted up. The mean scour depth may be fixed based on such observations and theoretical calculation.

703.3. Maximum Depth of Scour for Design of Foundation

703.3.1. The maximum depth of scour below the Highest Flood Level (HFL) for the design of piers and abutments having individual foundations without any floor protection may be considered as follows.

703.3.1.1. Flood without seismic combination:

- (i) For piers - $2.0 d_{sm}$
- (ii) For abutments - (a) $1.27 d_{sm}$ with approach retained or lowest bed level whichever is deeper.
(b) $2.00 d_{sm}$ with scour all around.

703.3.1.2. Flood with seismic combination : For considering load combination of flood and seismic loads (together with other appropriate combinations given elsewhere) the maximum depth of scour given in Clause 703.3.1.1 may be reduced by multiplying factor of 0.9.

703.3.1.3. For low water level (without flood conditions) combined with seismic combination maximum level of scour below high flood level can be assumed as 0.8 times scour given in Clause 703.3.1.

Note: In respect of viaducts/ROBs having no possibility of scour, passive resistance of soil may be considered below a depth of "excavation +2m".

703.3.2. For the design of floor protection works for raft or open foundations, the following values of maximum scour depth may be adopted:

- | | | |
|------|---------------------|---|
| (i) | In a straight reach | $1.27 d_{sm}$ |
| (ii) | In a bend | $1.50 d_{sm}$ or on the basis of concentration of flow. |

The length of apron in upstream may be 0.7 times of the same in downstream.

703.4. Special studies should be undertaken for determining the maximum scour depth for the design of foundations in all situations where abnormal conditions, such as, the following are encountered :

- (i) a bridge being located in a bend of the river involving a curvilinear flow, or excessive shoal formation, or
- (ii) a bridge being located at a site where the deep channel in the river hugs to one side, or
- (iii) a bridge having very thick piers inducing heavy local scours, or

- (iv) where the obliquity of flow in the river is considerable, or
- (v) where a bridge is required to be constructed across a canal, or across a river downstream of storage works, with the possibility of the relatively clear water inducing greater scours, or
- (vi) a bridge in the vicinity of a dam, weir, barrage or other irrigation structures where concentration of flow, aggradation/degradation of bed, etc. are likely to affect the behaviour of the structures.
- (vii) an additional two-lane bridge when located near to the existing bridge, on major rivers.

Note: These studies shall be conducted for the increased discharge calculated vide Clause 703.1.1.

703.5. If a river is of a flashy nature and bed does not lend itself readily to the scouring effect of floods, the theoretical formula for d_{sm} and maximum depth of scour as recommended shall not apply. In such cases, the maximum depth shall be assessed from actual observations.

704. SUB-SURFACE EXPLORATION

704.1. Objectives

The objectives of the sub-surface exploration are :

(i) During Preliminary Investigation Stage

As a part of site selection process to study existing geological maps and other information, previously prepared and available site investigation reports, known data of nearby structures, if any, surface examination about river bed and banks, etc., which will help in narrowing down of sites under consideration for further studies for project preparation stage.

(ii) Detailed Investigation Stage

To determine the characteristics of the existing geo-materials, like, soil, rock, bed material in water courses, etc. in the zone

of influence of the proposed bridge sites in such a way as to establish the design parameters which influence the choice and design details of the various structural elements, especially the foundation type.

(iii) **During Construction Stage**

To confirm the characteristics of geo-materials established in stage (ii) based on which the design choices are made and to re-confirm the same or modify to suit the conditions met at specific foundation locations.

704.2. **Zone of Influence**

Zone of influence mentioned in 704.1(ii) is defined as the full length of the bridge including portion of wing/return wall and part of approaches covering, (but not restricted to), the full flood zone for water courses, and upto depth below proposed foundation levels where influence of stresses due to foundation is likely to affect the behaviour of the structure, including settlement, subsidence under ground flow of water, etc. The width of the land strip on either side of the proposed structure should include zones in which the hydraulic characteristics of river water are likely to be changed affecting flow patterns, scour, etc.

704.3. **Methods of Exploration**

A large variety of investigative methods are available. A most suitable and appropriate combination of these shall be chosen. Guidelines for choice of types of investigations, properties of geo-materials that need be established, the in-situ testing, sampling, laboratory testing are given in *Appendix-2*. This may be further supplemented by specialised techniques depending on the need.

705. DEPTH OF FOUNDATION

705.1. General

The foundation shall be designed to withstand the worst combination of loads and forces evaluated in accordance with the provisions of Clause 706. The foundation shall be taken to such depth that they are safe against scour or protected from it. Apart from this, the depth should also be sufficient from consideration of bearing capacity, settlement, liquefaction potential, stability and suitability of strata at the founding level and sufficient depth below it. In case of bridges where the mean scour depth ' d_{sm} ' is calculated with Clause 703.2, the depth of foundation shall not be less than those of existing structures in the vicinity.

705.2. Open Foundations

705.2.1. **In soil :** The embedment of foundations in soil shall be based on correct assessment of anticipated scour considering the values given under Clause 703.

Foundation may be taken down to a comparatively shallow depth below the bed surface provided good bearing stratum is available, and the foundation is protected against scour.

The minimum depth of open foundations shall be upto stratum having safe bearing capacity but not less than 2.0 m below the scour level or the protected bed level.

705.2.2. **In rocks :** For open foundations resting on rock, the depth of rock, which in the opinion of the geological expert is weathered or fissured, shall be excluded in deciding the depth of embedment into the rock existing below. Where foundations are to rest on erodible rocks, caution shall be exercised to establish the foundation level at sufficient depth, so

as to ensure that they do not get undermined, keeping in view the continued erosion of the bed. After allowing for conditions stipulated above the minimum embedment of the foundations into the rock below shall be as follows, which in case of sloping rock profile can be provided by properly benching the foundations.

- (a) For hard rocks, with an ultimate crushing strength of 10 MPa or above arrived at after considering the overall characteristics of the rock, such as, fissures, bedding planes, etc. : 0.6 m
- (b) All other cases : 1.5 m

705.3. Well Foundations

705.3.1. **In soil :** Well foundations shall be taken down to a depth which will provide a minimum grip of 1/3rd the maximum depth of scour below the design scour level specified in Clause 703.3.

705.3.2. **In rocks :** As far as possible, the wells shall be taken by all the methods of sinking including pneumatic sinking (where considered necessary), dewatering, etc. to foundation level and shall be evenly seated all around the periphery on sound rock (i.e., devoid of fissures, cavities, weathered zone, likely extent of erosion, etc.) by providing adequate embedment. The extent of seating and embedment in each case shall be decided by the Engineer-in-charge keeping in view the factors mentioned above to ensure overall and long-term safety of the structure. It is advisable to make a sump (shear key) of 300 mm in hard rock or 600 mm in soft rock inside the well by chiselling/blasting. Diameter of sump may be 1.5 to 2 m less than inner dredge-hole subject to a minimum size of 1.5 m. Six dowel bars of 25 mm dia deformed bars may be anchored 1.5 m in rock and projected 1.5 m above. These may be anchored in

minimum 65 mm dia boreholes and grouted with 1:1½ cement mortar. The seating of well shall be such that 75 per cent perimeter is seated on rock.

705.4. Pile Foundations

705.4.1. In soil, the minimum depth of foundations below the point of fixity should be the minimum length required for developing full fixity as calculated by any rational formula.

705.4.2. In rocks, the pile should be taken down to rock strata devoid of any likely extension of erosion and properly socketed as required by the design.

706. LOADS, FORCES, STABILITY AND STRESSES

706.1. Loads, Forces and their Combinations

706.1.1. The loads and forces may be evaluated as per IRC:6 and their combinations for the purpose of this code will be as follows:

Combination (i): $G + (Q \text{ or } G_s) + F_{wc} + F_f + F_b + G_b + F_{cf} + F_{ep}$

Combination (ii): $(i) + W + F_{wp}$

or

$(i) + F_{eq} + F_{wp}$

or

$(i) + F_{im} + F_{wp}$

Combination (iii): $G + F_{wc} + G_b + F_{ep} + F_{er} + F_f + W \text{ or } F_{eq}$

706.1.2. The permissible increase in stresses in the various members will be $33\frac{1}{3}$ per cent for the combination of wind (W) and 50 per cent for the combination with seismic (F_{eq}) or impact (F_{im}). The permissible increase in allowable base

pressure should be 25 per cent for all combinations except (i). However, when temperature effects (f_{te}), secondary effects (F_s), deformation effects (F_d) are also to be considered for any members in combination with (i) then permissible increase in stresses in various members and allowable bearing pressure will be 15 per cent.

706.2. Horizontal Forces at Bearing Level

706.2.1. Simply supported spans

706.2.1.1. For simply supported span with fixed and free bearings (other than elastomeric type) on stiff supports, horizontal forces at the bearing level in the longitudinal direction shall be as given below :

	Fixed Bearing	Free Bearing
	<u>Non-Seismic Combinations</u>	
	Greater of the two values given below:	
(i)	$F_h - \mu (R_g + R_q)$	$\mu (R_g + R_q)$
(ii)	$F_h / 2 + \mu (R_g + R_q)$	$\mu (R_g + R_q)$
	<u>Seismic Combinations</u>	
	F_h	—

where

F_h = Applied horizontal force.

R_g = Reaction at the free end due to dead load

R_q = Reaction at the free end due to live load

μ = Co-efficient of friction at the movable bearing which shall be assumed to have the allowable values:

(i) For steel roller bearings : 0.03

(ii) For concrete roller bearings : 0.05

(iii) For sliding bearings :

(a) Steel on cast iron or steel on steel : 0.4

(b) Grey cast iron on grey cast iron
(Mechanites) : 0.3

(c) Concrete over concrete	: 0.5
(d) Teflon on stainless steel	: 0.03 and 0.05 (whichever is governing)

706.2.1.2. In case of simply supported small spans upto 10 metres and where no bearings are provided, horizontal force in the longitudinal direction at the bearing level shall be

$$\frac{F_h}{2} \text{ or } \mu R_g \text{ whichever is greater}$$

706.2.1.3. For a simply supported span sitting on identical elastomeric bearings at each end and resting on unyielding supports.

$$\text{Force at each end} = \frac{F_h}{2} + V_r l_{te}$$

V_r	=	Shear rating of the elastomeric bearings
l_{te}	=	Movement of deck above bearing, other than due to applied forces

706.2.2. **Simply supported and continuous span on flexible supports**

706.2.2.1. The distribution of applied longitudinal horizontal force (e.g., braking, seismic, wind, etc.) depends solely on shear rating of the supports and may be estimated in proportion to the ratio of individual shear rating of a support to the sum of the shear ratings of all the supports. Shear rating of a support is the horizontal force required to move the top of the support through a unit distance taking into account horizontal deformation of the bridge, flexing of the support and rotation of the foundation.

706.3. **Base Pressure**

706.3.1. The allowable bearing pressure and the settlement characteristics under different loads and stresses may be determined on the basis of sub-soil exploration and testing. Though the help of relevant Indian Standard Code of Practice may be taken, the allowable bearing pressure may be calculated as gross so that the gross pressure at the base without deducting the soil displaced can be computed.

706.3.1.1. **Factor of safety**

706.3.1.1.1. The factor of safety to calculate allowable bearing pressure on ultimate bearing capacity may be taken as 2.5 for soil.

706.3.1.1.2. The allowable bearing pressure on rock may be decided upon not only on the strength of parent rock but also on overall characteristics particularly deficiencies, like, joints, bedding planes, faults, weathered zones, etc. In absence of such details or analysis of overall characteristics, the value of factor of safety based on unconfined compressive strength of the parent rock may be taken as 6 to 8 unless otherwise indicated on the basis of local experience. The allowable bearing pressure, thus, obtained is to be further restricted to not over 3 MPa for load combination (i) given in Clause 706.1.1.

The disintegrated/weathered or very soft rock may be treated as soil.

706.3.2. **Allowable settlement/differential settlement**

706.3.2.1. The calculated differential settlement between the foundations of simply supported spans shall not exceed 1 in 400 of the distance between the two foundations from the consideration of tolerable riding quality unless provision has been made for rectification of this settlement.

706.3.2.2. In case of structures sensitive to differential settlement, the tolerable limit has to be fixed for each case separately.

706.3.3. Permissible tension at the base of foundation

706.3.3.1. No tension shall be permitted under any combination of loads on soils.

706.3.3.2. In case of rock if tension is found to be developed at the base of foundation, the base area should be reduced to a size where no tension will occur and base pressure is recalculated. The maximum pressure on such reduced area should not exceed allowable bearing pressure. Such reduced area shall not be less than 67 per cent of the total area for load combination including seismic, or impact of barge, and 80 per cent for other load combinations.

706.3.4. Factor of safety for stability

Factors of safety against overturning and sliding are given below. These are mainly relevant for open foundations :

	Without seismic case	With seismic case
(i) Against overturning	2	1.5
(ii) Against sliding	1.5	1.25
(iii) Against deep-seated failure	1.25	1.15

Frictional co-efficients between concrete and soil/rock will be $\tan \theta$, θ being angle of friction. Founding soil in foundation of bridge being generally properly consolidated, following values may be adopted:

- Friction co-efficient between soil and concrete = 0.5
- Friction co-efficient between rock and concrete = 0.8 for good rock and 0.7 for fissured rock

706.3.5. **Pile foundations** : The allowable load, the allowable settlement/differential settlement and the procedures to determine the same for pile foundations are given in Clause 709.

707. OPEN FOUNDATIONS

707.1. **General**

707.1.1. Provision of the Clause under 707 shall apply for design of isolated footings and, where applicable, to combined footings, strip footings and rafts.

707.1.2. Open foundations may be provided where the foundations can be laid in a stratum which is inerodible or where the extent of scour of the bed is reliably known. The foundations are to be reliably protected by means of suitably designed aprons, cut-off walls or/and launching aprons as may be necessary.

707.2. **Design**

707.2.1. The thickness of the footings shall not be less than 300 mm.

707.2.2. **Bending moments**

707.2.2.1. For solid wall type substructure with one-way reinforced footing, the bending moments can be determined as one-way slab for the unit width subjected to worst combination of loads and forces.

707.2.2.2. For two-way footings, bending moment at any section of the footing shall be determined by passing a vertical plane through the footing and computing the moment of the forces acting over the entire area of footings one side of the

vertical plane. The critical section of bending shall be at the face of the solid column.

707.2.2.3. In case of circular footings or polygonal footings, the bending moments in the footing may be determined in accordance with any rational method. Methods given by Timoshenko and Rowe for Plate Analysis are acceptable.

707.2.2.4. For combined footings supporting two or more columns, the critical sections for bending moments along the axis of the columns shall be at the face of the columns/walls. Further, for determination of critical sections for bending moments between the columns/walls, any rational method of analysis be adopted.

707.2.3. The shear strength of the footing may be checked at the critical section which is the vertical section at a distance 'd' from the face of the wall for one way action where 'd' is the effective depth of the section at the face of the wall.

707.2.3.1. For two-way action for slab or footing, the critical section should be perpendicular to plan of slab and so located that its perimeter is minimum, but need not approach closer than half the effective depth to the perimeter of concentrated load or reaction area.

707.2.4. To ensure proper load transfer, a limiting value of ratio of depth to length/width of footing equal to 1:3 is specified. Based on this, for sloped footings the depth effective at the critical section shall be the minimum depth at the end plus $1/3$ rd of the distance between the extreme edge of the footing to the critical section for design of the footing for all purposes.

707.2.5. The critical section for checking development length of reinforcement bars should be taken to be the same

section as given in Clause 707.2.3 and also all other vertical planes where abrupt changes in section occur.

707.2.6. Tensile reinforcement

707.2.6.1. The tensile reinforcement shall provide a moment of resistance at least equal to the bending moment on the section calculated in accordance with Clause 707.2.2.

707.2.6.2. The tensile reinforcement shall be distributed across the corresponding resisting section as below :

- (a) In one-way reinforced footing, the reinforcement shall be same as calculated for critical unit width as mentioned in para 707.2.2.1.
- (b) In two-way reinforced square footing, the reinforcement extending in each direction shall be distributed uniformly across the full section of the footing.
- (c) In two-way reinforced rectangular footing, the reinforcement in the long direction shall be distributed uniformly across the full width of the footing. For reinforcement in the short direction, a central band equal to the short side of the footing shall be marked along the length of the footing and portion of the reinforcement determined in accordance with the equation given below shall be uniformly distributed across the central band :

$$\frac{\text{Reinforcement in central band width}}{\text{Total reinforcement in short direction}} = \frac{2}{(\beta + 1)}$$

where β = the ratio of the long side to the short side of the footing

The remainder of the reinforcement shall be uniformly distributed in the outer portions of the footing.

- (d) In the case of a circular shaped footing, the reinforcement shall be provided on the basis of the critical values of radial and circumferential bending moments in the form of radial and circumferential steel. Alternatively, equivalent orthogonal grid can be provided.

707.2.7. The area of tension reinforcement should not be less than 0.15 per cent of the cross-sectional area when using S415 grade bars and 0.25 per cent of the cross-sectional area when using S240 grade bars.

707.2.8. All faces of the footing shall be provided with a minimum steel of $250 \text{ mm}^2/\text{metre}$ in each direction for all grades of reinforcement. Spacing of these bars shall not be more than 300 mm. This steel may be considered to be acting as tensile reinforcement on that face, if required from the design considerations.

707.2.9. In case of plain concrete, brick or stone masonry footings, the load from the pier or column shall be taken as dispersed through the footing at an angle not exceeding 45° .

707.3. **Open Foundations at Sloped Bed Profile**

707.3.1. Open foundations may rest on sloped bed profile provided the stability of the slope is ensured. The footings shall be located on a horizontal base.

707.3.2. For the foundations adjacent to each other, the pressure coming from the foundations laid on the higher level should be duly considered on the foundations at the lower level due to the dispersion of the pressure from the foundation at the higher level. The distance between the two foundations at different levels may be decided in such a way to minimise this effect taking into account the nature of soil.

707.4. **Construction**

707.4.1. The protective works shall be completed before the floods so that the foundation does not get undermined.

707.4.2. Excavation on open foundations shall be done after taking necessary safety precautions for which guidance may be taken from IS:3764.

707.4.3. Where blasting is required to be done for excavation in rock, and is likely to endanger adjoining foundations or other structures, necessary precautions, such as, controlled blasting, providing suitable mat cover to prevent flying of debris, etc. shall be taken to prevent any damage.

707.4.4. **Condition for laying of foundations**

707.4.4.1. Normally, the open foundations should be laid dry and every available method of dewatering by pumping or depression of water by well point, etc. may be resorted to. A levelling course of 100 mm thickness in M 10 (1:3:6) shall be provided below foundation.

707.4.4.2. If it is determined before-hand that the foundations cannot be laid dry or the situation is found that the percolation is too heavy for keeping the foundation dry, the foundation concrete may be laid under water only by tremie pipe. In case of flowing water or artesian springs, the flow shall be stopped or reduced as far as possible at the time of placing of concrete. No pumping of water shall be permitted from the time of placing of concrete upto 24 hours after placement.

707.4.5. All spaces excavated and not occupied by abutments, pier or other permanent works shall be refilled with earth upto the surface of the surrounding ground, with sufficient allowance for settlement. All backfill shall be thoroughly compacted and in general, its top surface shall be neatly graded.

707.4.6. In case of excavation in rock, the trenches around the footing shall be filled up with concrete of M 15 grade upto the top of the rock.

707.4.6.1. If the depth of fill required is more than 1.5 m in soft rock or 0.6 m in hard rock above the foundation level, then concrete may be filled upto this level by M 15 concrete and portion above may be filled by concrete or by boulders grouted with cement.

707.4.6.2. For design of foundation on rock in river bridges, the design loads and forces shall be considered upto the bottom of footing. The load of filling need not be considered in stability calculations.

708. WELL FOUNDATIONS

708.1. General

708.1.1. While selecting the shape, size and the type of wells for a bridge, the size of pier to be accommodated, need for effecting streamline flow, the possibility of the use of pneumatic sinking, the anticipated depth of foundation, and the nature of strata to be penetrated should be kept in view. Further, for the type of well selected, the dredge hole should be large enough to permit easy dredging, the minimum dimension being not less than 2 m. In case there is deep standing water, properly designed floating caissons may be used as per Clause 708.12.

708.1.2. If the external diameter of single circular well exceeds 12 m then Engineer-in-charge may take recourse to any of the following :

- (a) Stresses in steining shall be evaluated using 3-Dimensional Finite Element Method (3D FEM) or any other suitable analytical method.
- (b) Stiffening by compartments may be done for the single circular well. Design of such stiffened wells shall call for supplemental design and construction specifications.
- (c) Twin D-shaped well may be adopted.

708.1.3. The conditions arising out of sand blow, if anticipated, should be duly considered when circular well is analysed using 3D FEM/suitable analytical method or stiffened circular wells are used.

708.2. Well Steining

708.2.1. Thickness of the steining should be such so that it is possible to sink the well without excessive kentledge and without getting damaged during sinking or during rectifying the excessive tilts and shifts. The steining should also be able to resist differential earth pressure developed during sand blow or other conditions, like, sudden drop.

Stresses at various levels of the steining should be within permissible limits under all conditions for loads that may be transferred to the well.

708.2.2. Use of cellular steining with two or more shells or use of composite material in well steining shall not be permitted for wells upto 12 m diameter.

708.2.3. Steining thickness

708.2.3.1. The minimum thickness of the well steining shall not be less than 500 mm and satisfy the following relationship :

$$h = Kd \sqrt{l}$$

where h = minimum thickness of steining in m

d = external diameter of circular well or dumb bell shaped well or in case of twin D wells smaller dimension in plan in metres

l = depth of wells in metre below top of well cap or LWL whichever is more (for floating caisson ' l ' may be taken as depth of well in metres below bed level)

$K =$ a constant

Value of K shall be as follows :

- (i) Well in cement concrete $K = 0.03$
- (ii) Well in brick masonry $K = 0.05$
- (iii) Twin D wells $K = 0.039$

708.2.3.2. The minimum steining thickness may be varied from above in following conditions :

Strata	Variation from the minimum	Recommended variation upto
(a) Very soft clay strata	Reduced	10%
(b) Hard clay strata	Increased	10%
(c) Boulder strata or well resting on rock involving blasting	Increased	10%

708.2.3.3. However, following aspects may also be considered depending on the strata :

- (a) **Very soft clay strata** - Main criteria for reduction in steining thickness is to prevent the well penetrating by its own weight. When the thickness is so reduced, the steining shall be adequately reinforced to get sufficient strength.
- (b) **Hard clay strata** - Depending on the previous experience, the increase in steining thickness may be more than 10 per cent.
- (c) Boulder strata or well resting on rock involving blasting, higher grade of concrete, higher reinforcement, use of steel plates in the lower portions, etc., may be adopted.

708.2.3.4. The recommended values given in Clause 708.2.3.2 can be further varied based on local experience and in accordance with decision of Engineer-in-charge.

708.2.3.5. If specialised methods of sinking, such as, jack down method, are adopted then the steining thickness may be adjusted according to design and construction requirements.

708.2.3.6. Any variation from dimensions as proposed in Clause 708.2.3.1 should be decided before framing the proposal.

708.2.3.7. When the depth of well below well cap is equal to or more than 30 metres, the thickness of the steining of the well calculated as per Clause 708.2.3 may be reduced above scour level in a slope of 1 horizontal to 3 vertical such that the reduced thickness of the steining should not be less than required as per Clause 708.2.3 for the depth of well upto scour level with the reduced diameter.

The reduction in thickness shall be done in the outer surface of the well. The diameter of inner dredge hole shall be kept uniform.

The minimum steel and the concrete grade in the slope portion shall be same as for the steining below scour level.

Minimum development length of all the vertical steel bars shall be provided beyond the minimum section as shown in the *Appendix-3*, (Fig. 1).

The stress in the reduced section of steining shall also be checked.

708.3. Design Considerations

708.3.1. In case of plain concrete wells, the concrete mix for the steining shall not normally be leaner than M 15. In case of marine or other similar conditions of adverse exposure, the concrete in the steining shall not be less than leaner than M 20 with cement not less than 310 kg/m^3 of concrete and the water cement ratio not more than 0.45.

708.3.2. The external diameter of the brick masonry wells shall not exceed 6 m. Brick masonry wells for depth greater than 20 m shall not be permitted.

708.3.3. For brick masonry wells, brick not less than Grade-A having strength not less than 70 kg/cm^2 conforming to IS:1077 shall be used in cement mortar not leaner than 1:3.

708.3.4. For plain concrete wells, vertical reinforcements (whether mild steel or deformed bars) in the steining shall not be less than 0.12 per cent of gross sectional area of the actual thickness provided. This shall be equally distributed on both faces of the steining. The vertical reinforcements shall be tied up with hoop steel not less than 0.04 per cent of the volume per unit length of the steining, as shown in the *Appendix-3*, (Fig. 2).

708.3.5. In case where the well steining is designed as a reinforced concrete element, it shall be considered as a column section subjected to combined axial load and bending. However, the amount of vertical reinforcement provided in the steining shall not be less than 0.2 per cent. (for either mild steel or deformed bars) of the actual gross sectional area of the steining. On the inner face, a minimum of 0.06 per cent (of gross area) steel shall be provided. The transverse reinforcement in the steining shall be provided in accordance with the provisions for a column but in no case shall be less than 0.04 per cent of the volume per unit length of the steining.

The horizontal annular section of well steining shall also be checked for ovalisation moments by any rational method taking account of side earth pressures evaluated as per Clause 708.4.

708.3.6. The vertical bond rods in brick masonry steining shall not be less than 0.1 per cent of the cross-sectional area and

shall be encased into cement concrete of M 15 mix of size 150 mm x 150 mm. These rods shall be equally distributed along the circumference in the middle of the steining and shall be tied up with hoop steel not less than 0.04 per cent of the volume per unit length of the steining. The hoop steel shall be provided in a concrete band at spacing of 4 times of the thickness of the steining or 3 metres, whichever is less. The horizontal RCC bands shall not be less than 300 mm wide and 150 mm high, reinforced with bars of diameter not less than 10 mm placed at the corners and tied with 6 mm diameter stirrups at 300 mm centres, as shown in the *Appendix-3*, (Fig. 3).

708.3.7. The stresses in well steining shall be checked at such critical sections where tensile and compressive stresses are likely to be maximum and also where there is change in the area of reinforcement or in the concrete mix.

708.4. **Stability of Well Foundations**

708.4.1. The stability and design of well foundations shall be done under the most critical combination of loads and forces as per Clause 706. The pressure on foundations shall satisfy the provisions of Clause 706.

708.4.2. **Side earth resistance**

708.4.2.1. The side earth resistance may be calculated as per guidelines given in *Appendix-3*. The use of provisions IRC:45 may be used for pier well foundations in cohesionless soil.

708.4.2.2. The side earth resistance shall be ignored in case of well foundations resting on rock. If rock strata is such that the allowable bearing pressure is less than 1 MPa, then the side earth resistance may be taken into account.

708.4.3. Earth pressure on abutments

708.4.3.1. If the abutments are designed to retain earth and not spilling in front, the foundations of such abutments shall be designed to withstand the earth pressure and horizontal forces for the condition of scour depth in front of $1.27 d_{sm}$ with approach retained and $2 d_{sm}$ with scour all around. In case of scour all around, live load may not be considered.

708.4.3.2. However, where earth spilling from the approaches is reliably protected in front, relief due to the spilling earth in front may be considered from bottom of well cap downwards.

708.4.4. Construction stage

708.4.4.1. Stability of the well shall also be checked for the construction stage when there is no superstructure and the well is subjected to design scour, full pressure due to water current and/or full design earth pressure as in the case of abutment wells.

708.4.4.2. During the construction of wells when it has not reached the founding level or has not been plugged, the wells are likely to be subjected to full pressure due to water current upto full scour. This may result in tilting, sliding and shifting. As a part of the safety during construction, this should be considered and safety of well must be ensured by suitable methods, where required.

708.5. Tilts and Shifts

708.5.1. As far as possible, the wells shall be sunk plumb without any tilts and shifts. However, a tilt of 1 in 80 and a shift of 150 mm due to translation (both additive) in a direction which will cause most severe effect shall be considered in the design of well foundations.

708.5.2. If the actual tilts and shifts exceed the above limits, then the remedial measures have to be resorted to bring the well within that limit. If it is not possible then its effect on bearing pressure, steining stress and other structural elements shall be examined, and controlled if necessary and feasible by resorting to change in span length. The Engineer-in-charge may like to specify the maximum tilts and shifts upto which the well may be accepted subject to the bearing pressure and steining stress being within limits, by changing the span length if needed, and beyond which the well will be rejected irrespective of the result of any modification.

708.6. **Cutting Edge**

708.6.1. The mild steel cutting edge shall be strong enough and not less than 40 kg/m² to facilitate sinking of the well through the types of strata expected to be encountered without suffering any damage. It shall be properly anchored to the well curb. For sinking through rock cutting edge should be suitably designed.

708.6.2. When there are two or more compartments in a well, the lower end of the cutting edge of the middle stems of such wells shall be kept about 300 mm above that of the outer stems to prevent rocking, as shown in the *Appendix-3*, (Fig. 2).

708.7. **Well Curb**

708.7.1. The well curb should be such that it will offer the minimum resistance while the well is being sunk but should be strong enough to be able to transmit superimposed loads from the steining to the bottom plug.

708.7.2. The shape and the outline dimension of the curb as given in *Appendix-3*, (Fig. 2) may be taken for

guidance. The internal angle of the curb ' α ' as shown in *Appendix-3*, (Fig. 2) should be kept at about 30° to 37° and may be increased or decreased based on past experience and geo-technical data.

708.7.3. The well curb shall invariably be in reinforced concrete of mix not leaner than M 25 with minimum reinforcement of 72 kg/cu.m. excluding bond rods. The steel shall be suitably arranged to prevent spreading and splitting of the curb during sinking and in service.

708.7.4. In case blasting is anticipated, the inner faces of the well curb shall be protected with steel plates of thickness not less than 10 mm upto the top of the well curb. If it is desired to increase the steel lining above the well curb then the thickness can be reduced to 6 mm for that increased height. In any case, this extra height of the steel should not be more than 3 metres unless there is a specific requirement. The curb in such a case should be provided with additional hoop reinforcement of 10 mm dia mild steel or deformed bars at 150 mm centres which shall also extend upto a height of 3 m into the well steining above the curb. Additional reinforcement above this height upto two times the thickness of steining should be provided to avoid cracking arising out of sudden change in the effective section due to curtailment of plate.

708.8. **Bottom Plug**

708.8.1. The bottom plug shall be provided in all wells and the top shall be kept not lower than 300 mm in the centre above the top of the curb, as shown in the *Appendix-3*, (Fig. 2). A suitable sump shall be below the level of the cutting edge. Before concreting the bottom plug, it shall be ensured that its inside faces have been cleaned thoroughly.

708.8.2. The concrete mix used in bottom plug shall have a minimum cement content of 330 kg/m^3 and a slump of about 150 mm to permit easy flow of concrete through tremie to fill up all cavities. Concrete shall be laid in one continuous operation till dredge hole is filled to required height. For under water concreting, the concrete shall be placed gently by tremie boxes under still water condition and the cement contents of mix be increased by 10 per cent.

708.8.3. In case grouted concrete, e.g., concrete is used, the grout mix shall not be leaner than 1:2 and it shall be ensured by suitable means, such as, controlling the rate of pumping that the grout fills up all inter-stices upto the top of the plug.

708.8.4. If any dewatering is required it shall be carried out after 7 days have elapsed after bottom plugging.

708.9. **Filling the Well**

708.9.1. The filling of the well, if considered necessary, above the bottom plug shall be done with sand or excavated material free from organic matter.

708.10. **Plug over Filling**

708.10.1. A 300 mm thick plug of M 15 cement concrete shall be provided over the filling.

708.11. **Well Cap**

708.11.1. The bottom of well cap shall preferably be laid as low as possible taking into account the L.W.L.

708.11.2. As many longitudinal bars as possible coming from the well steining shall be anchored into the well cap.

708.11.3. The design of the well cap shall be based on any accepted rational method, considering the worst combination of loads and forces as per Clause 706.

708.12. Floating Caissons

708.12.1. Floating caissons may be of steel, reinforced concrete or any suitable material. They should have at least 1.5 metres free board above the water level and increased, if considered necessary, in case there is a possibility of caissons sinking suddenly owing to reasons, such as, scour likely to result from lowering of caissons, effect of waves, sinking in very soft strata, etc.

708.12.2. Well caissons should be checked for stability against overturning and capsizing while being towed, and during sinking, due to the action of water current, wave pressure, wind, etc.

708.12.3. The floating caisson shall not be considered as part of foundation unless proper shear transfer at the interface is ensured.

708.13. Sinking of Wells

708.13.1. The well shall as far as possible be sunk true and vertical. Sinking should not be started till the steining has been cured for atleast 48 hours. A complete record of sinking operations including tilt and shifts, kentledge, dewatering, blasting, etc. done during sinking shall be maintained.

For safe sinking of wells, necessary guidance may be taken from the precautions as given in *Appendix-4*.

708.14. Pneumatic Sinking of Wells

708.14.1. Where sub-surface data indicate the need for pneumatic sinking, it will be necessary to decide the method and location of pneumatic equipment and its supporting adapter.

708.14.2. In cases where concrete steining is provided, it

shall be rendered air tight by restricting the tension in concrete which will not exceed $\frac{3}{8}$ th of the modulus of rupture. For the circular wells, the tension in steining may be evaluated by assuming it to be a thick walled cylinder.

708.14.3. The steining shall be checked at different sections for any possible rupture against the uplift force and, if necessary, shall be adequately strengthened.

708.14.4. The design requirements of the pneumatic equipment, safety of personnel and the structure shall comply with the provisions of IS:4138 "Safety Code for Working in Compressed Air". It is desirable that the height of the working chamber in a pneumatic caissons should not be less than 3 metres to provide sufficient head room when the cutting edge is embedded a short distance below the excavated level and in particular to allow for blowing down. The limiting depth for pneumatic sinking should be such that the depth of water below normal water level to the proposed foundation level upto which pneumatic sinking should not exceed 30 m.

708.15. **Sinking of Wells by Resorting to Blasting**

Blasting may be employed with prior approval of competent authority to help sinking of well for breaking obstacles, such as, boulders or for levelling the rock layer for square seating of wells. Blasting may be resorted to only when other methods are found ineffective.

709. PILE FOUNDATION

709.1. **General**

709.1.1. Piles transmit the load of a structure to competent subsurface strata by the resistance developed from bearing at the toe or skin friction along the surface or both. The

piles may be required to carry uplift and lateral loads besides direct vertical load.

709.1.2. The construction of pile foundation requires a careful choice of piling system depending upon subsoil conditions and load characteristics of structures. The permissible limits of total and differential settlement, unsupported length of pile under scour and any other special requirements of project are also equally important criteria for adoption.

709.1.3. **Design and construction** : For design and construction of piles guidance may be taken from IS:2911 subject to limitations/stipulations given in this code. *Appendix-5* gives the design formulae and their applicability.

709.1.4. For piles in streams, rivers, creeks, etc., the following criteria may be followed:

- (i) Scour conditions are properly established.
- (ii) Permanent steel liner should be provided at least upto maximum scour level. In case of marine clay or soft soil or soil having aggressive material, permanent steel liner of sufficient strength shall be used for the full depth of such strata. The minimum thickness of liner should be 5 mm.

709.1.5. **Spacing of piles and tolerances**

709.1.5.1. **Spacing of piles** : The spacing of piles should be considered in relation to the nature of the ground, their behaviour in groups and the overall cost of the foundation. The spacing should be chosen with regard to the resulting heave or compaction and should be wide enough to enable the desired number of piles to be installed to the correct penetration without damage to any adjacent construction or to the piles themselves.

The cost of a cap carrying the load from the structure to

the pile heads, or the size and effective length of a ground beam, may influence the spacing, type and size of piles.

The spacing of piles will be determined by:

- (a) the method of installation, e.g., driven or bored;
- (b) the bearing capacity of the group.

Working rules which are generally, though not always, suitable, are as follows:

For friction piles, the spacing centre should be not less than the perimeter of the pile or, for circular piles, three times the diameter. The spacing of piles deriving their resistance mainly from end bearing may be reduced but the distance between the surfaces of the shafts of adjacent piles should be not less than the least width of the piles.

709.1.5.2. Permissible tolerances for piles shall be as under

- (i) For vertical piles 75 mm at piling platform level and tilt not exceeding 1 in 150;
- (ii) For raker piles tolerance of 1 in 25.

709.1.6. The maximum rake to be permitted in piles shall not exceed the following :

- (i) 1 in 6 for all bored piles;
- (ii) 1 in 6 for driven cast-in-situ piles; and
- (iii) 1 in 4 for precast driven piles.

709.1.7. The minimum diameter of piles shall be as follows:

	Bridges on Land	River Bridges
Driven cast-in-situ piles	0.5 m	1.2 m
Precast piles	0.35 m	1.0 m
Bored piles	1.0 m	1.2 m

709.1.8. The settlement, differential settlement, lateral deflection at cap level may be limited for any structure as per the requirement.

709.1.9. For both precast and cast-in-situ piles, the values regarding grade of concrete, water cement ratio, slump shall be as follows:

	Tremie Concrete Cast-in-situ	Driven Cast- in-situ	Precast Concrete
Grade of concrete	M 35	M 35	M 35
Min. cement contents	400 Kg/m ³	400 Kg/m ³	400 Kg/m ³
Max. W.C. ratio	0.4	0.4	0.4
Slump (mm)	150-200	100-130	50

709.2. Requirement and Steps for Design and Installation

709.2.1. The initial design of an individual pile, group of piles and final adoption should pass through two types of major investigation and tests as follows:

- (i) Comprehensive and detailed sub-surface investigation for piles to determine the design parameter of end bearing capacity, friction capacity and lateral capacity of soil surrounding the pile.
- (ii) Initial load test on trial piles for confirmation/modification of design and layout and routine load test on working piles for acceptance of the same.

709.2.2. The steps for design and confirmation by tests are given below :

- (i) Subsoil exploration to establish design soil parameters.
- (ii) Required capacity of pile group based on tentative number and diameter of piles in a group.

- (iii) Capacity of pile based on static formula considering ground characteristics. The allowable total/differential settlement should be duly considered. This step alongwith step (ii) may be iterative.
- (iv) Structural design of piles.
- (v) Initial load test for axial capacity, lateral load capacity and uplift load capacity on trial piles to verify/confirm or modify the design consideration of piles done by steps (ii), (iii) and (iv). The load tests may be conducted for the ultimate capacities. Initial load test shall be cyclic load test. If the initial load test gives a capacity greater than 25 per cent of the capacity calculated by static formula, another two load tests shall be carried out to confirm the earlier value and minimum of the three shall be considered as initial load test value. The number of initial tests shall be determined by the Engineer-in-charge taking into consideration the bore log and soil profile.

For load testing of piles, reference is made to IS:2911 (Part - IV).

- (vi) Routine load tests may be conducted again to reconfirm or modify the allowable load. Tests should be properly designed to cover particular group for single pile test and double pile test. The lateral load test may be conducted on two adjacent piles.

709.2.3. For abutment, it is important to consider overall stability of the structure and abutment. The piles should also be designed to sustain surcharge effect of embankment.

709.2.4. **Routine tests** : Routine load tests should be done on one pile for alternate foundation for bridges. The number may be suitably increased/reduced taking into consideration the borelog and soil profile.

709.3. **Capacity of Pile**

709.3.1. For calculating designed capacity of pile/pile group methods/recommendation of IS:2911 should be followed.

Appendix-5 gives formulae for estimating pile capacity based on soil/rock interaction with pile.

709.3.2. Factor of safety : The minimum factor of safety on ultimate axial capacity computed on the basis of static formula shall be 2.5 for piles in soil. For piles in rock, factor of safety shall be 5 on the bearing component and 10 on socket side resistance component.

709.3.3. Capacity of piles/group action : The axial capacity of a group of piles should be determined by a factor to be applied to the capacity of individual piles multiplied by the number of piles of the group.

- (i) Factor may be taken as 1 in case of purely end bearing piles having minimum spacing of 2.5 times the diameter of pile and for frictional piles having spacing of minimum 3 times diameter of pile.
- (ii) For pile groups in clays, the group capacity shall be minimum of the following :
 - (a) Sum of the capacities of the individual piles in the group.
 - (b) The capacity of the group based on block failure concept, where the ultimate load carrying capacity of the block enclosing the piles is estimated.

709.3.4. Settlement of pile group

709.3.4.1. The capacity of a pile group is also governed by settlement criterion. Settlement of a pile group may be computed on the basis of following recommendations or by any other rational method.

709.3.4.2. Settlement of pile group in sands : The settlement of a pile group is affected by the shape and size of group, length, spacing and method of installation of piles. There is no rational method available to predict the settlement of group

of piles in sands. It is recommended to use empirical relationship proposed by Vesic for obtaining the settlement of pile group. In this method, the settlement of the group is predicted based on settlement of a single pile obtained from load test. The following table indicates the relationship :

Width of Group/Pile dia	Settlement Ratio δ_g/δ
5	2.5
25	5
50	7.5
60	8

where δ_g = settlement of pile group

δ = settlement of single pile

709.3.4.3. Settlement of pile group in clays : The settlement of pile group in homogeneous clays shall be evaluated using Terzaghi and Peck Approach which assumes that the load carried by the pile group is transferred to the soil through an equivalent footing located at one third of the pile length upwards from the pile toe. The load under the equivalent footing is assumed to spread into soil at a slope of 2 slope of 2 (vertical): 1 (horizontal).

The settlement for equivalent footing shall be evaluated in accordance with IS:8009 (Part II).

709.3.4.4. Settlement of pile group in rock : Settlement of piles founded in rock may be computed as per IS:8009 (Part II) considering the value of in-situ modulus of rock mass.

709.3.5. Resistance to lateral loads

709.3.5.1. The ultimate lateral resistance of a group of vertical piles may be taken as the passive pressure acting on the

enclosed area of the piles. Such passive pressure may be calculated over an equivalent wall of depth equal to $6D$ and width equal to $L + 2B$.

where D = Diameter of pile

L = Length between outer faces of pile group in plan perpendicular to direction of movement

B = Width between outer faces of pile group in plan parallel to the direction of movement

The minimum factor of safety on ultimate lateral resistance shall be 2.5.

709.3.5.2. The safe lateral resistance must not exceed the sum of lateral resistance of the individual piles. The safe lateral resistance of individual pile shall be corresponding to a 5 mm deflection at ground level in accordance with IS:2911 with full 'E' value and for appropriate pile-head condition in Load Combination, 1 of Clause 706.1.1. For river bridges with scourable bed, the 5 mm deflection may be taken as the deflection at scour level.

709.3.6. Uplift load carrying capacity

709.3.6.1. Piles may be required to resist uplift forces of permanent or temporary nature when used in foundations subjected to large overturning moments or as anchorages in structures, like, underpasses subjected to hydrostatic uplift pressure.

709.3.6.2. The ultimate uplift capacity may be calculated with the expression of shaft resistance/skin friction only, of the static formulae for compression loads and applying a reduction factor of 0.50 on the same. However, in the case of rock, the length of socket need not be restricted to $0.5 \times$ dia of socket. The

weight of pile shall also act against uplift. Pull out tests may be conducted for verification of uplift capacity.

709.3.6.3. The uplift capacity of pile group is lesser of the two following values:

- the sum of the uplift resistance of the individual piles in the group, and
- the sum of the shear resistance mobilised on the surface perimeter of the group plus the effective weight of the soil and the piles enclosed in this surface perimeter.

709.3.6.4. Piles should be checked for structural adequacy against uplift forces together with other co-existent forces, if any.

709.3.6.5. The minimum factor of safety on ultimate uplift load calculated on the aforesaid basis shall be 2.5.

709.3.7. **Piles subjected to downward drag** : A pile may be subjected to additional load on account of downward drag resulting from consolidation of a soft compressible clay, layer due to its own weight, remoulding or surface load. Such additional load coming on pile may be assessed on the following basis:

- (i) In the case of pile deriving its capacity mainly from friction, the value of downward drag force may be taken as 0.2 to 0.3 times undrained shear strength multiplied by the surface area of pile shaft embedded in compressible soil.
- (ii) In case of pile deriving its capacity mainly from end bearing, the value of downward drag force may be considered as 0.5 times undrained shear strength multiplied by the surface area of pile shaft embedded in compressible soil.
- (iii) For a group of piles, the drag forces shall also be calculated considering the surface area of the block (i.e., perimeter of the group times depth) embedded in compressible soil. In the event

of this value being higher than the number of pile in the group times the individual downward drag forces, the same shall be considered in the design.

709.4. Structural Design of Piles

709.4.1. A pile as a structural member shall have sufficient strength to transmit the load from structure to soil. The pile shall also be designed to withstand temporary stresses, if any, to which it may be subjected to, such as, handling and driving stresses. The permissible stresses should be as per IRC:21.

709.4.2. The piles may be designed taking into consideration all the load effects and their structural capacity examined as a column. The self load of pile or lateral load due to earthquake, water current force, etc. on the portion of free pile upto scour level and upto potential liquefaction level, if applicable, should be duly accounted for.

709.4.3. For the horizontal load at the cap level, the moment in the pile stem can be determined by any rational theory. In the absence of any rational theory, the method given in IS:2911 (Part I /Sec 2) may be adopted. If the pile group is provided with rigid cap, then the piles should be considered as having fixed head for this purpose. Horizontal force may be distributed equally in all piles in a group with a rigid pile cap.

709.4.4. **Minimum reinforcement** : The reinforcements in pile should be provided for the full length of pile, as per the design requirements. However, the minimum area of longitudinal reinforcement shall be 0.4 per cent of the area of cross-section in all concrete piles. Lateral reinforcement shall be provided in the form of links or spirals with minimum 8 mm diameter steel, spacing not less than 150 mm. Cover to main reinforcements shall not be less than 75 mm.

709.4.5. The reinforcement should comply with the provision of IRC:21 for resisting stresses due to lifting, stacking and transport, any uplift or bending transmitted from the superstructure and bending due to any secondary effects. The area of longitudinal reinforcement shall not be less than the following percentages of the cross-sectional area of the piles:

- (a) For piles with a length less than 30 times the least width – 1.25 per cent;
- (b) For piles with a length 30 to 40 times the least width – 1.5 per cent; and
- (c) For piles with a length greater than 40 times the least width – 2 per cent.

709.5. **Design of Pile Cap**

709.5.1. The pile caps shall be of reinforced concrete of size fixed taking into consideration the allowable tolerances as in Clause 709.1.5.2. A minimum offset of 150 mm shall be provided beyond the outer faces of the outer-most piles in the group. If the pile cap is in contact with earth at the bottom, a levelling course of minimum 80 mm thick plain cement concrete shall be provided.

709.5.2. The top of the pile shall project 50 mm into the pile cap and reinforcements of pile shall be fully anchored in pile cap.

709.5.3. In marine conditions or in areas exposed to the action of harmful chemicals, etc., use of dense compacted concrete shall be made. In addition, the pile cap shall be protected with a suitable anti-corrosive paint. High alumina cement, i.e., quick setting cement shall not be used in marine constructions.

709.5.4. The minimum thickness of pile cap should be at least 0.6 m or 1.5 times the diameter of pile whichever is more.

709.5.5. Casting of pile cap should be at level higher than water level unless functionally it is required to be below water level at which time sufficient precautions should be taken to dewater. The forms to allow concreting in dry condition.

709.6. **Important Consideration, Inspection/ Precautions for Different Types of Piles**

709.6.1. **Driven cast-in-situ piles**

709.6.1.1. Except otherwise stated in this code, guidance is to be obtained from IS:2911(Part I/Section I).

709.6.1.2. While concreting the uncased piles, voids in concrete may be avoided and sufficient head of concrete is to be maintained to prevent inflow of soil or water into the concrete. It is also necessary to take precaution during concreting to minimise the softening of the soil by excess water. Uncased cast-in-situ piles shall not be allowed where mudflow conditions exist.

709.6.1.3. The pile shoes which may be either of cast iron conical type or of mild steel flat type should have double reams for proper seating of the removable casing tube inside the space between the reams.

709.6.1.4. Before commencement of pouring of concrete, it should be ensured that there is no ingress of water in the casing tube from the bottom. Further adequate control during withdrawal of the casing tube is essential so as to maintain sufficient head of concrete inside the casing tube at all stages of withdrawal.

709.6.1.5. Concrete in piles shall be cast upto a minimum height of 600 mm above the designed top level of pile, which shall be stripped off to obtain sound concrete either before final set or after 3 days.

709.6.2. **Bored cast-in-situ piles**

709.6.2.1. The drilling mud, such as, bentonite suspension shall be maintained at a level sufficiently above the surrounding ground water level to ensure the stability of the strata which is being penetrated throughout the boring process until the pile has been concreted.

709.6.2.2. The bores must be washed by fresh bentonite solution flushing to ensure clean bottom at two stages prior to concreting and after putting reinforcement.

709.6.2.3. In case of bored cast-in-situ piles tremies of 200 mm diameter shall be used for concreting. The tremie should have uniform and smooth cross-section inside, and shall be withdrawn slowly ensuring adequate height of concrete outside the tremie pipe at all stages of withdrawal. Other recommendations for tremie concreting are:

- (i) The sides of the borehole have to be stable throughout;
- (ii) The tremie shall be watertight throughout its length and have a hopper attached at its head by a watertight connection;
- (iii) The tremie pipe should be lowered to the bottom of borehole, allowing ground water or drilling mud to rise inside it before pouring concrete;
- (iv) The tremie pipe should always be kept full of concrete and should penetrate well into the concrete in the borehole with adequate margin of safety against accidental withdrawal if the pipe is surged to discharge the concrete.

709.6.3. **Driven precast concrete piles**

709.6.3.1. Except otherwise stated in this code, guidance is to be obtained from IS:2911(Part I/Section 3).

709.6.3.2. This type of piles for bridges may be adopted when length of pile as per design requirement is known with reasonable degree of accuracy. Extra length of pile may be cast to avoid lengthening of piles as far as possible. When unavoidable, the splicing for lengthening of steel may be used only after the method of splicing is tested and approved earlier.

The longitudinal reinforcement shall be joined by welding or by mechanical couplers. The concrete at top of original pile shall be cut down to sufficient length to avoid spalling by heat of welding. Location of mechanical couplers in neighbouring reinforcement shall be such as to permit concreting between the bars.

709.6.3.3. During installation of piles, the final set or penetration of piles per blow of hammer should be checked taking an average of last 100 blows.

710. SUBSTRUCTURE

710.1. **General**

710.1.1. In case of plain concrete substructure, surface reinforcement at the rate of 2.5 kg/m^2 shall be provided in each direction, i.e., both horizontally and vertically. Spacing of such bars shall not exceed 200 mm. In case of substructure in highly corrosive atmosphere, the surface reinforcement can be dispensed with if specifically allowed but the dimension of the substructure should be so proportioned to keep the stresses only upto 90 per cent of the allowable stress.

710.1.2. For the design of substructure below the level of the top of bed block, the live load impact shall be modified by the factors given below :

- | | | |
|-------|--|-------------------------------------|
| (i) | For calculating the pressure at the bottom surface of the pier/abutment cap | 0.5 |
| (ii) | For calculating pressure on the top 3 M of substructure below pier/abutment cap | Decreasing uniformly
0.5 to zero |
| (iii) | For calculating the pressure on the portion of the substructure more than 3 M below the pier/abutment cap. | zero |

710.1.3. Structures designed to retain earthfill shall be proportioned to withstand pressure calculated in accordance with any rational theory. No structure shall, however, be designed to withstand to horizontal pressure less than that exerted by a fluid weighing 480 kg/m^3 , in addition to the live load surcharge if any.

710.1.4. The backfill behind the wing and return walls shall conform to the specifications in *Appendix-6* with provision for proper drainage.

710.2. Piers

710.2.1. Piers in stream and channel should be located to meet navigational clearance requirements and give a minimum interference to flood flow. In general, piers should be placed parallel with the direction of stream current at flood stage. Piers in other locations, like, viaducts or land spans should be according to the requirement of the obstacles to cross over.

710.2.2. Where necessary, piers shall be provided at both ends with suitably shaped cut waters as given in IRC:6. However, cut and ease water where provided shall extend upto

affluxed H.F.L. or higher, if necessary, from consideration of local conditions, like, waves, etc.

710.2.3. Pier may be in PSC, RCC, PCC or masonry. Only solid section should be adopted for masonry piers. The design of masonry piers should be based on permissible stresses as provided in IRC:40.

710.2.4. The thickness of the walls of hollow concrete piers should not be less than 300 mm.

710.2.5. The multi-column piers of bridges across rivers carrying floating debris, trees or timber should be braced throughout the height of the piers by diaphragm wall of minimum 200 mm thickness. Unbraced multiple column piers may be allowed depending upon the performance of similar structures in similar conditions of river. However, type and spacing of such bracing, when adopted, shall be predetermined.

710.2.6. Piers shall be designed to withstand the load and forces transferred from the superstructure and the load and forces on the pier itself apart from the effect of its self-weight. In general, pier may be solid, hollow or framed structures.

710.2.7. In case of piers consisting two or more columns, the horizontal forces at the bearing can be distributed on all the columns in proportion to their relative rigidities, if the thickness of the pier cap is at least one and a half times the thickness of the column.

710.2.8. If the piers consist of either multiple piles or trestle columns spaced closer than three times the width of piles/columns across the direction of flow, the group shall be treated as a solid pier of the same overall width and the value of K taken as 1.25 for working out pressure due to water current

according to relevant Clause 213.7 of IRC:6. If such piles/columns are braced then the group should be considered a solid pier irrespective of the spacing of the columns.

710.2.9. Hollow piers shall be provided with suitably located weep-holes of 75 to 100 mm diameter for enabling free flow of water to equalise the water levels on inside and outside; considering rate of rise/fall of flood/tide water. The pier walls should be checked for expected differential water-head/wave pressure and silt pressure.

710.2.10. The lateral reinforcement of the walls of hollow circular RCC pier shall not be less than 0.3 per cent of the sectional area of the walls of the pier. This lateral reinforcement shall be distributed 60 per cent on outer face and 40 per cent on inner face.

710.3. **Wall Piers**

710.3.1. When the length of solid pier is more than four times its thickness, it shall also be checked as a wall.

710.3.2. The reinforced wall should have minimum vertical reinforcement equal to 0.3 per cent of sectional area.

*710.3.3. For eccentric axial load, the wall should be designed for axial load with moment. The moments and the horizontal forces should be distributed taking into account the dispersal by any rational method.

710.3.4. The vertical reinforcement need not be enclosed by closed stirrups, where vertical reinforcement is not required for compression. However, horizontal reinforcement should not be less than 0.25 per cent of the gross area and open links (or S-loops) with hook placed around the vertical bar should be placed at the rate of 4 links in one square metre.

710.3.5. When walls are fixed with superstructure, the design moment and axial load should be worked out by elastic analysis of the whole structure.

710.4. **Abutments**

710.4.1. The abutments will carry superstructure from one side. It should be designed/dimensioned to retain earth from the approach embankment.

710.4.2. The abutments should be designed to withstand earth pressure in normal condition in addition to load and forces transferred from superstructure. In addition, any load acting on the abutment itself, including self-weight, is to be considered.

710.4.3. In case of spill through type abutment, the active pressure calculated on the width of the column shall be increased by 50 per cent where two columns have been provided and by 100 per cent where more than two columns have been provided.

710.4.4. All abutments and abutment columns shall be designed for a live load surcharge equivalent to 1.2 m height of earthfill. The effective width of the columns need not be increased as in Clause 710.4.3 for surcharge effect when spill through abutment is adopted.

710.4.5. Abutment should also be designed for water current forces during 'scour all round' condition.

710.4.6. The abutment may be plain or reinforced concrete or of masonry. The abutment may be either solid type, buttressed type, counterfort type or spill through type. For spill through abutment, column type or wall type analysis may be carried out as for piers. Counterfort type abutment may be treated as T or L type as the case may be and the slab may be designed as continuous over counterforts.

710.4.7. Fully earth retaining abutments should be designed considering saturated unit weight of earth during H.F.L. or L.W.L. condition. In case of footings, the submerged unit weight of soil where considered shall not be less than 1000 kg/m^3 .

710.4.8. The weight of earth filling material on heel may be considered. In case of toe, the weight may be considered if the bed is protected.

710.4.9. In case of spill through type abutment, it should be ensured that the slope in front of the abutment is well protected by means of suitably designed stone pitching and launching aprons.

710.4.10. In case of abutments having counterfort, the minimum thickness of the front wall should not be less than 200 mm and the thickness of the counterfort should not be less than 250 mm.

710.5. **Abutment Pier**

710.5.1. Abutment piers may have to be provided at locations where there may be a need of increasing waterway subsequently. The design of such abutment piers shall be such that it should be possible to convert them to the similar shape as piers in the active channel.

710.5.2. For multiple span arch bridges, abutment piers shall be provided after every fifth span or closer.

710.6. **Dirt Walls, Wing Walls and Return Walls**

710.6.1. Wing walls shall be of sufficient length to retain the roadway to the required extent and to furnish protection against erosion.

710.6.2. A dirt wall shall be provided to prevent the earth from approaches spilling on the bearings. A screen wall of sufficient depth (extended for at least 500 mm depth into the fill) to prevent slipping of the backfill in case the abutment is of the spill through type, shall be provided.

710.6.3. The wing walls may be of solid type. The return walls may be of solid or counterfort type. The material used may be plain or reinforced concrete or masonry.

710.6.4. Dirt wall/ballast wall and screen wall shall be provided with minimum thickness of 300 mm.

710.6.5. The wing walls should be designed primarily to withstand the earth pressure in addition to self-weight.

710.6.6. The top of the wing/return walls shall be carried above the top of embankment by at least 100 mm to prevent any soil from being blown or washed away by rain over its top. A drainage arrangement for return wall/wing wall may be provided similar to that for the abutment specified in *Appendix-6*.

710.6.7. The cantilever returns where adopted should not be more than 4 metres long.

710.6.8. In case of open foundations, wing and return walls should be provided with separate foundations with a joint at their junction with the abutment.

710.6.9. Wing walls may be laid at any suitable angle to the abutment. In case of river bridges, these are normally splayed in plan at 45 degrees. The return walls may be provided at right angles to the abutment. Return walls shall be designed to withstand a live-load surcharge equivalent to 1.2 m height of earthfill.

710.6.10. The box type return wall at right angles at both ends of the abutments connected by wall type diaphragm may be adopted where found suitable. However, in such cases, no reduction in the earth pressure for the design of the abutment should be considered. The top of diaphragm should slope inwards to the centre of carriageway for facilitating proper rolling of the embankment behind the abutment.

710.6.11. Solid type of wing/return walls on independent foundations can be suitably stepped up towards the approaches depending upon the pattern of scour, local ground conditions and its profile, safe bearing capacity, etc.

710.6.12. In case of wing walls or return walls, the foundation shall be taken adequately into the firm soil.

710.7. **Retaining Walls**

710.7.1. The minimum thickness of reinforced concrete retaining wall shall be 200 mm.

710.7.2. The retaining walls shall be designed to withstand earth pressure including any live load surcharge and other loads acting on it including self-weight in accordance with the general principles specified for abutments. Stone masonry and plain concrete walls shall be of solid type. Reinforced concrete walls may be of solid, counterfort, buttressed or cellular type.

710.7.3. The vertical stems of cantilever walls shall be designed as cantilevers fixed at the base. The vertical or face walls of counterfort type and buttressed type shall be designed as continuous slabs supported by counterforts or buttresses. The face walls shall be securely anchored to the supporting counterforts or buttresses by means of adequate reinforcements.

710.7.4. Counterforts shall be designed as T-beams or L-beams. Buttresses shall be designed as rectangular beams. In connection with the main tension reinforcement of counterforts, there shall be a system of horizontal and vertical bars or stirrups to anchor the face walls and base slab to the counterfort. These stirrups shall be anchored as near to the outside faces of the face walls and as near to the bottom of the base slab as practicable.

710.8. Pier and Abutment Caps

710.8.1. The width of the abutment and pier caps shall be sufficient to accommodate :

- (i) the bearings leaving an offset of 150 mm beyond them.
- (ii) the ballast wall.
- (iii) the space for jacks to lift the superstructure for repair/replacement of bearings, etc.
- (iv) the equipment for prestressing operations where necessary.
- (v) the drainage arrangement for the water on the cap.

710.8.2. The thickness of cap over the hollow pier or column type of abutment should not be less than 250 mm but in case of solid plain or reinforced concrete pier and abutment, the thickness can be reduced to 200 mm.

710.8.3. Pier/Abutment caps should be suitably designed and reinforced to take care of concentrated point loads dispersing in pier/abutment. Caps cantilevering out from the supports or resting on two or more columns shall be designed to cater for the lifting of superstructure on jacks for repair/replacement of bearings. The locations of jacks shall be predetermined and permanently marked on the caps.

710.8.4. In case bearings are placed centrally over the columns and the width of bearings/pedestals is located within

half the depth of cap from any external face of the columns, the load from bearings will be considered to have been directly transferred to columns and the cap beam need not be designed for flexure.

710.8.5. The thickness of the cap over masonry piers or abutment shall not be less than 500 mm. The minimum width at the top of such piers and abutments of slab and girder bridges just below the caps shall be as given below:

Span in metres	3m	6m	12m	24m
Top width of pier carrying simply supported spans in m	0.50	1.0	1.2	1.6
Top width of abutment and of piers carrying continuous spans in m	0.40	0.75	1.0	1.3

710.8.6. Except the portion under bearings, the top surface of caps should have suitable slope in order to allow drainage of water.

710.9. Cantilever Pier and Abutment Cap

710.9.1. When the distance between the load/centre line of bearing from the face of the support is equal to or less than the depth of the cap (measured at the support) the cap shall be designed as a corbel.

710.9.2. The equivalent square area may be worked out for circular pier to determine the force of support for calculating bending moments.

710.9.3. In case of wall pier and the pier cap cantilevering out all around the measurement of distance for purpose of the design as bracket and the direction of provision of reinforcement should be parallel to the line joining the centre of load/bearing with the nearest supporting space of pier.

710.9.4. Where a part of the bearing lies directly over the pier, calculation of such reinforcement should be restricted only for the portion which is outside the face of the pier. Moreover, in such cases the area of closed horizontal stirrups may be limited to 25 per cent of the area of primary reinforcement.

710.10. Pedestals below Bearing

710.10.1. The pedestals should be so proportioned that a clear offset of 150 mm beyond the edges of bearings is available.

710.10.2. For pedestals whose height is less than its width the requirement of the longitudinal reinforcement as specified for short column need not be insisted upon.

710.10.3. The allowable bearing pressure with near uniform distribution on the loaded area of a footing or base under a bearing or column shall be given by the following equation:

$$C = C_0 \times \sqrt{\frac{A_1}{A_2}}$$

where C_0 = the permissible direct compressive stress in concrete at the bearing area of the base

A_1 = dispersed concentric area which is geometrically similar to the loaded area A_2 and also the largest area that can be contained in the plane of A_1 (maximum width of dispersion beyond the loaded area face shall be limited to twice the height)

A_2 = loaded area and the projection of the bases or footing beyond the face of the bearing or column supported on it shall not be less than 150 mm in any direction.

710.10.4. The two layers of mesh reinforcement - one at 20 mm from top and the other at 100 mm from top of pedestal or pier cap each consisting of 8 mm bars at 100 mm in both directions, shall be provided directly under the bearings.

GUIDELINES FOR CALCULATING SILT FACTOR FOR BED MATERIAL CONSISTING OF GRAVELS AND BOULDERS

(Ref. Clause 703.2.2.2)

In absence of any formula ' K_{sf} ' may be determined as per Clause 703.2.2 and may be adopted based on site information and behaviour history of any existing structure. The clayey bed having weighted diameter normally less than 0.04 offers more resistance to scour than sand though mean depth of scour as per the formula given in Clause 703.2 indicates more scour. In absence of any accepted rational formula or any data of scour at the site of the proposed bridge; the following theoretical calculation may be adopted:

- (i) In case of soil having $\phi < 15^\circ$ and c (cohesion of soil) $> 0.2 \text{ kg/cm}^2$, ' K_{sf} ' calculated as follows:

$$K_{sf} = F(1 + \sqrt{c}) \text{ where } c \text{ is in } \text{kg/cm}^2$$

$$\text{where } F = 1.50 \text{ for } \phi > 10^\circ \text{ and } < 15^\circ$$

$$= 1.75 \text{ for } \phi > 5^\circ \text{ and } < 10^\circ$$

$$= 2.00 \text{ for } \phi < 5^\circ$$

- (ii) Soils having $\phi > 15^\circ$ will be treated as sandy soil even if c is more than 0.20 kg/cm^2 and silt factor will be as per provisions of Clause 703.2.2.

GUIDELINES FOR SUB-SURFACE EXPLORATION

(Ref. Clause 704.3)

1. GENERAL

The objective of sub-surface exploration is to determine the suitability of the soil or rock, for the foundation of bridges. The sub-surface exploration for bridges is carried out in two stages, namely, preliminary and detailed. It may require additional/conformatory exploration during construction stage.

Guidance may be taken from the following:

- (i) IS:1892 – Code of Practice for Site Investigation for Foundations may be utilised for guidance regarding investigation and collection of data.
- (ii) Test on soils shall be conducted in accordance with relevant parts of IS:2720 – Methods of Test for Soils. The tests on undisturbed samples be conducted as far as possible at simulated field conditions to get realistic values.
- (iii) IS:1498 – Classification and Identification of Soils for general engineering purposes.

For preliminary and detailed sub-surface investigation, only rotary drills shall be used. The casing shall also be, invariably provided with diameters not less than 150 mm upto the level of rock, if any. However, use of percussion or wash boring equipment shall be permitted only to penetrate through bouldery or gravelly strata for progressing the boring but not for collection of samples, while conducting detailed borings, the resistance to the speed of drilling, i.e., rate of penetration, core loss, etc. shall be carefully recorded and presented in "Borelog

chart and data sheet” to evaluate the different types of strata and distinguish specially sand from sandstone, clay from shale, etc.

For preliminary and detailed sub-surface investigation, only double tube diamond drilling method shall be used. In soft and weak rocks such tuffs, soft shales triple tube diamond drilling shall be used.

2. PRELIMINARY INVESTIGATION

2.1. Preliminary investigation shall include the study of existing geological information, previous site reports, geological maps, etc., and surface geological examination. These will help to narrow down the number of sites under consideration and also to locate the most desirable location for detailed sub-surface investigation.

3. DETAILED INVESTIGATION

3.1. Based on data obtained after preliminary investigations, the bridge site, the type of structure with span arrangement and the location and type of foundations, the programme of detailed investigations, etc. shall be tentatively decided. Thereafter the scope of detailed investigation including the extent of exploration, number of bore holes, type of tests, number of tests, etc. shall be decided in close liaison with the design engineer and the exploration team, so that adequate data considered necessary for detailed design and execution are obtained.

3.2. The exploration shall cover the entire length of the bridge and also at either end a distance of zone of influence, i.e., about twice the depth below bed of the last main foundation to assess the effect of the approach embankment on the end foundations. Generally, the sub-surface investigations should

extend to a depth below the anticipated foundation level equal to about one and a half times the width of the foundation. However, where such investigations end in any unsuitable or questionable foundation material, the exploration shall be extended to a sufficient depth into firm and stable soils or to rock.

3.2.1. Additional drill holes : Where the data made available by detailed exploration indicate appreciable variation specially in case of foundations resting on rock, it will be necessary to resort to additional drill holes to establish a complete profile of the underlying strata. Location and depth of additional drill holes will have to be decided depending upon the extent of variation in local geology and in consultation with design engineer.

3.3. The scope of the detailed sub-surface exploration shall be fixed as mentioned in para 3.1 and 3.2. However, as a general guide it shall be comprehensive enough to enable the designer to estimate or determine the following:

- (i) engineering properties of the soil/rock;
- (ii) location and extent of weak layers and cavities, if any, below hard founding strata;
- (iii) the sub-surface geological condition, such as, type of rock, structure of rock, i.e., folds, faults, fissures, shears, fractures, joints, dykes and subsidence due to mining or presence of cavities;
- (iv) ground water level;
- (v) artesian conditions, if any;
- (vi) quality of water in contact with the foundation;
- (vii) depth and extent of scour;
- (viii) suitable foundation level;

- (ix) safe bearing capacity of foundation stratum;
- (x) probable settlement and probable differential settlement of the foundations;
- (xi) likely sinking or driving effort; and
- (xii) likely construction difficulties.

4. CONSTRUCTION STAGE EXPLORATION

Such explorations may become necessary to verify the actually met strata vis-a-vis detailed investigation stage or when a change in the sub-soil strata/rock profile is encountered during construction. In such situations, it may be essential to resort to further explorations to establish the correct data, for further decisions.

5. METHOD OF TAKING SOIL SAMPLES

The size of the bores shall be predetermined so that undisturbed samples as required for the various types of tests are obtained. The method of taking samples shall be as given in IS:1892 and IS:2132. The tests on soil samples shall be conducted as per relevant part of IS:2720.

6. DETAILS OF EXPLORATION FOR FOUNDATIONS RESTING ON SOIL (ERODIBLE STRATA)

6.1. The type and extent of exploration shall be divided in to the following groups keeping in view the different requirements of foundation design and the likely method of data collection:

- (i) Foundations requiring shallow depth of exploration;
- (ii) Foundations requiring large depth of exploration; and
- (iii) Fills behind abutments and protective works.

6.2. Foundations Requiring Shallow Depth Exploration (Open Foundation)

These shall cover cases where the depth of exploration is not deep and it is possible to take samples from shallow pits or conduct direct tests, like, plate load tests, etc. This will also cover generally the foundation soil for approach embankments, protective works, etc.

6.2.1. The primary requirements are stability and settlement, for which shearing strength characteristics, load settlement characteristics, etc. need determination.

6.2.2. Tests shall be conducted on undisturbed representative samples, which may be obtained from open pits. The use of plate load test (IS:1888-Method of Load Test on Soils) is considered desirable for ascertaining the safe bearing pressure and settlement characteristics. A few exploratory bore holes or soundings shall be made to safeguard against presence of weak strata underlying the foundation. This shall extend to a depth of about $1\frac{1}{2}$ times the proposed width of foundation.

Note: For better interpretation, it will be desirable to correlate the laboratory results with the in-situ tests, like, plate load tests, penetration test results.

6.2.3. The tests to be conducted at various locations for properties of soil, etc. are different for cohesive and cohesionless soils. These are indicated below and shall be carried out wherever required according to soil type:

I. Cohesionless Soils

(a) Laboratory Tests

- (i) Classification tests, index tests, density determination, etc.

- (ii) Shear strengths by triaxial/direct shear, etc.
- (b) Field Tests
 - (i) Plate Load Test
 - (ii) Standard Penetration Tests (as per IS:2131)

Use of Dynamic Cone Penetration Test as per IS:4968 (Part 1 or Part 2) may be conducted where considered appropriate).

II. Cohesive Soils

- (a) Laboratory Tests
 - (i) Classification tests, index tests, density determination, etc.
 - (ii) Shear strengths by triaxial/direct shear, etc.
 - (iii) Unconfined compression test (IS:2720 Part X)
 - (iv) Consolidation test (IS:2720 Part V)
- (b) Field Tests
 - (i) Plate Load Test
 - (ii) Vane Shear Test (IS:4434)
 - (iii) Static Cone Penetration Test (IS:4968 Part III)

Note: Where dewatering is expected, the samples may be tested for permeability (IS:2720 Part-XVII).

6.3. Foundations Requiring Large Depth of Exploration

6.3.1. In this group are covered cases of deep wells, pile foundations, etc. where the use of boring equipment, special techniques of sampling, in-situ testing, etc. become essential. In addition to the problems of soil and foundation interaction an important consideration can be the soil data required from construction considerations. Often in the case of cohesionless

soils, undisturbed samples cannot be taken and recourse has to be made to in-situ field tests.

6.3.2. The sub-surface exploration can be divided into three zones:

- (i) between bed level and upto anticipated maximum scour depth (below H.F.L.)
- (ii) from the maximum scour depth to the foundation level, and
- (iii) from foundation level to about 1½ times the width of foundation below it.

6.3.3. Sampling and testing (in-situ and laboratory) requirement will vary in each case and hence are required to be assessed and decided from case to case. The sub-soil water shall be tested for chemical properties to evaluate the hazard of deterioration to foundations. Where dewatering is expected to be required, permeability characteristics should be determined.

6.3.4. For the different zones categorised in para 6.3.2., the data required, method of sampling, testing, etc. are given in Table 1. Samples of soils in all cases shall be collected at every 1 to 1½ metre or at change of strata.

Table 1. Sub-soil data required for deep foundations

Zones	Data Required	Sampling and Testing	Remarks – Including limitations
Bed levels to anticipated maximum scour depth	(i) Soil classification (ii) Particles size distribution	Sampling For (i) and (ii) disturbed samples may be collected. For (iii) and (iv) undisturbed samples shall be collected. In-Situ Tests Cohesionless Soils – Dynamic Penetration	(i) Laboratory tests to be conducted according to the relevant parts of IS:2720. (ii) Undisturbed sampling cohesionless soils is a difficult and expensive process. In general, in such cases, in-situ tests may be adopted.

Tests as per details in para 704.4.2.3.

Cohesive Soils –

- (i) Static penetration Tests – cone and skin resistance to be obtained.
- (ii) Field vane shear test may be done.

Laboratory Tests

- (i) Classification Tests including particle size distribution.
- (ii) Shearing strength – Triaxial tests to be done on undisturbed samples. Unconfined compression tests to be done on undisturbed and remoulded samples.

- (iii) Boring and sampling tends to cause remoulding of sensitive clays. Disturbance and stress changes for fissured or layered clays may also make the sample not truly representative of the in-situ condition. In such cases use of in-situ tests may give results more representative of the actual soil characteristics.

Maximum anticipated scour level to the foundation level	(i) Soil classification (ii) Shearing strength characteristics (iii) Compressibility (iv) Permeability-where dewatering is expected. (v) Moisture content, density, void ratio	Same as above	Same as above
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Foundation level to about 1½ times of the width of foundation below it	(i) Soil classification (ii) Shearing strength (iii) Compressibility	Same as above and Consolidation test to be done on undisturbed samples	Same as above
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Note: Use of sophisticated equipment, like, the pressure meter may be made, if suitable co-relations for interpretation of data collected are available.

6.4. Fill Materials

Representative disturbed samples shall be collected from the borrowpit areas. Laboratory tests shall be conducted for determining the following:

- (i) classification and particle size
- (ii) moisture content
- (iii) density vs. moisture content relationship
- (iv) shearing strength
- (v) permeability

Note :The shearing strength shall be obtained for the density corresponding to the proposed density for the fill.

7. DETAILS OF EXPLORATION FOR FOUNDATIONS RESTING ON ROCK

7.1. Basic Information Required from Explorations

- (i) Geological system;
- (ii) Depth of rock and its variation over the site;
- (iii) Whether isolated boulder or massive rock information;
- (iv) Extent and character of weathered zone;
- (v) The structure of rock – including bedding planes, faults, etc.;
- (vi) Properties of rock material, like, strength, geological formation, etc.;
- (vii) Quality and quantity of returning drill water; and
- (viii) Erodibility of work to the extent possible.

7.2. Exploration Programme

If preliminary investigations have revealed presence of rock within levels where the foundation is to rest, it is essential to take up detailed investigation to collect necessary information mentioned in the preceding para.

7.2.1. The extent of exploration shall be adequate enough to give a complete picture of the rock profile both in depth and across the channel width for assessing the constructional difficulties in reaching the foundation levels. Keeping this in view it shall be possible to decide the type of foundations, the construction method to be adopted for a particular bridge, the extent of even seating and embodiment into rock of the foundations. It is desirable to take atleast one drill hole per pier and abutment and one on each side beyond abutments.

7.2.2. The depth of boring in rock depends primarily on local geology, erodibility of the rock, the extent of structural loads to be transferred to foundation, etc. Normally, it shall pass through the upper weathered or otherwise weak zone, well into the sound rock. The minimum depth of drilling shall be as per para 3.2 above.

7.3. Detailed Investigations for Rock at Surface or at Shallow Depths

In case of rock at shallow depths which can be conveniently reached, test pits or trenches are the most dependable and valuable methods, since they permit a direct examination of the surface, the weathered zone and presence of any discontinuities. For guidance, IS:4453 - Code of Practice for exploration by pits, trenches, drafts and shafts may be referred to. In case structurally disturbed rocks, in-situ tests may be made in accordance with IS:7292 - Code of Practice for in-situ determination of rock properties by flat jack, IS:7317 - Code of Practice for Uni-axial Jacking Test for Deformation Modulus and IS:7746 - Code of Practice for in-situ Shear Test on Rock.

7.4. Detailed Investigation for Rock at Large Depths

7.4.1. This covers cases where recourse is to be made to

sounding, boring and drilling. An adequate investigation programme has to be planned to cover the whole area for general characteristics and in particular the foundation location, for obtaining definite information regarding rock-depth and its variation over the foundation area. The detailed programme of exploration will depend on the type and depth of overburden, the size and importance of the structure, etc. To decide this, geophysical methods adopted at the preliminary investigation stage may be helpful.

7.4.2. The investigation of the overburden soil layers shall be done as per details given for the foundations resting in soil. However, in case of foundations resting on rock, tests on overburden shall be carried out only when necessary, e.g., foundation level lower than scour levels.

7.4.3. The cores shall be stored properly in accordance with IS:4078- Code of Practice for Indexing and Storage of Drill Cores.

7.4.4. The rock cores obtained shall be subjected to tests to get necessary data for design as follow:

- (i) Visual identification for
 - (a) Texture
 - (b) Structure
 - (c) Composition
 - (d) Colour
 - (e) Grain size
 - (f) Petrography
- (ii) Laboratory tests may be done for
 - (a) Specific gravity
 - (b) Porosity
 - (c) Water absorption
 - (d) Compressive strength

Note: Generally, shear strength tests will suffice for design purposes. Other tests may need to be done in special case. The shear strength tests can be done as unconfined compression, triaxial compression or direct shear test.

7.4.5. Use of in-situ tests for measuring strength and deformation characteristics may be made. Use of bore hole photography will be desirable to evaluate the presence of faults, fissures or cavities, etc.

7.5. Special Cases

7.5.1. **Investigation for conglomerate** : A drill hole shall be made same as for rock. The samples collected shall be subjected to suitable tests depending upon the material. Special care shall be taken to ascertain the erodibility of the matrix.

7.5.2. **Investigation for laterites** : The investigation shall generally be similar to that required for cohesive soils. In case of hard laterite, recourse may have to be made to core drilling as for soft rocks.

8. CLASSIFICATION AND CHARACTERISTICS OF ROCKS

8.1. Identification and classification of rock types for engineering purposes may, in general, be limited to broad, basic physical condition in accordance with accepted practice. Strength of parent rock alone is of limited value because overall characteristics depend considerably on character, spacing and distributions of discontinuities of the rock mass, such as, the joints, bedding planes, faults and weathered seams.

8.2. Classification of Rocks

Rocks may be classified or identified based on their physical condition as indicated below. For foundation design, these are to be classified in three groups as in Table 2. As a

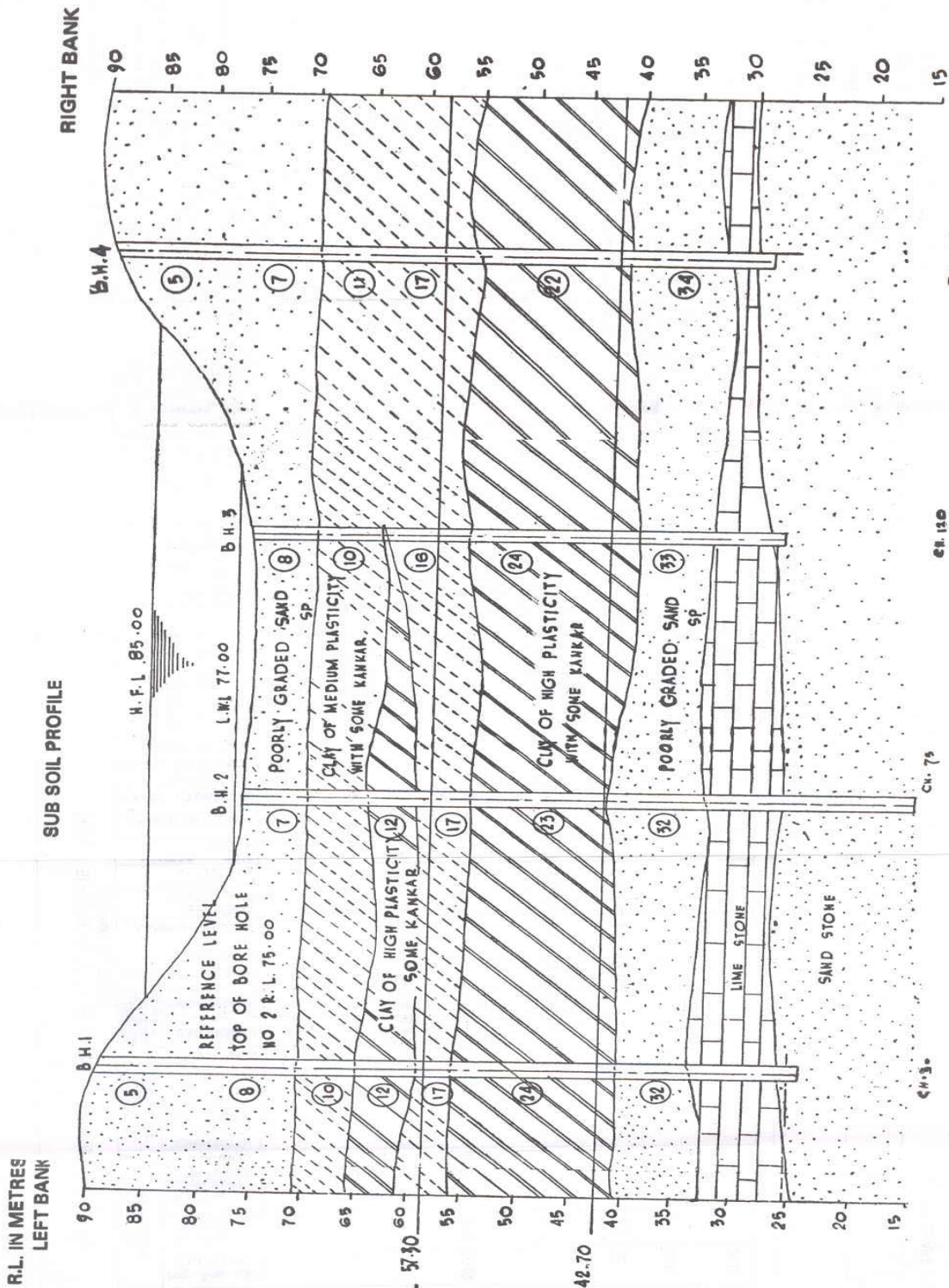
guide, the allowable bearing values of the rocks of different conditions may be taken from the values given in Table 2, duly modified after taking into account the various characteristics of rocks.

Table 2

Types of Rock/Condition	Suggested Allowable Bearing Values for Average Condition
Hard Rocks	2.0 to 3.0 MPa
Soft Rocks	1.0 to 2.0 MPa
Weathered Rocks, Conglomerates and Laterites	Not more than 1.0 MPa

9. PRESENTATION OF DATA

The presentation of data collected shall be done as illustrated in Sheets No.1 and 2.



Notes :

1. Cross section of River at 'AA' looking down stream
2. Numbers in the circles represent field S.P.T. n values
3. The detail shown are as a sample only

PROJECT -----
SUB SOIL PROFILE

PROCEDURE FOR STABILITY CALCULATION

(Ref. Clauses 708.2 & 708.3)

1. FORMULAE FOR ACTIVE OR PASSIVE PRESSURE IN SOIL

The active and passive pressure co-efficient (k_a & k_p respectively) shall be calculated according to Coulomb's formula taking into account the wall friction. For cohesive soils, the effect of 'c' may be added to the same as per procedure given by Bell. The value of angle of wall friction may be taken as 2/3rd of ϕ the angle of repose is subject to a limit of 22½ degrees. Both the vertical and horizontal components shall be considered in the stability calculations.

2. SKIN FRICTION

The relief due to skin friction shall be ignored unless specifically permitted by the Engineer-in-charge. However, in case of highly compressive soils, skin friction, if any, may cause increased bearing pressure on the foundation and shall be duly considered.

3. FACTOR OF SAFETY OVER ULTIMATE PRESSURES

The factor of safety in assessing the allowable passive resistance shall be 2 for load combinations without wind or seismic forces and 1.6 for load combinations with wind or seismic forces. The manner of applying factor of safety shall be as indicated below:

- (i) Pier wells founded in cohesive soils

The factor of safety as stipulated for the type of soil shall be applied for the net ultimate soil resistance, viz., $(P_p - P_a)$ where P_p and P_a are total passive and active pressure respectively mobilised below the maximum scour level.

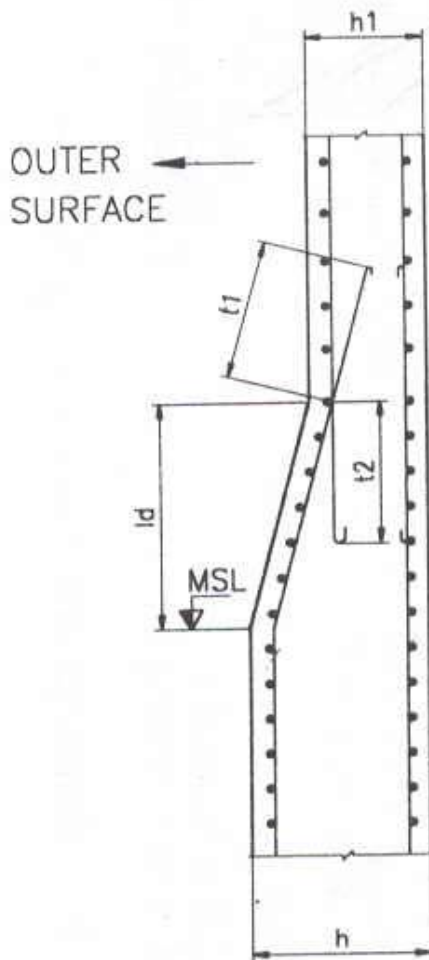
(ii) Abutment wells in both cohesive and non-cohesive soils

In the case of abutment wells, the active pressure on soil above the maximum scour level (Triangular verification of pressure) shall be separately evaluated and considered as loads combined with the other loads acting on the abutment and no factor of safety shall be taken for the above components of active pressure. Effects of surcharge due to live load should be restricted only upto the abutment portion.

(iii) However, the lateral resistance of soil below the scour level at ultimate value shall be divided by the appropriate factor of safety, viz., $(P_p - P_a)$ f.o.s. as stated in the case of pier wells.

(iv) Point of rotation

For the purpose of applying the above formulae, it may be assumed that the point of rotation lies at the bottom of the well.



→ INNER DREDGE HOLE

← OUTER SURFACE

$$h = Kd \sqrt{1}$$

$$h1 = Kd1 \sqrt{1s}$$

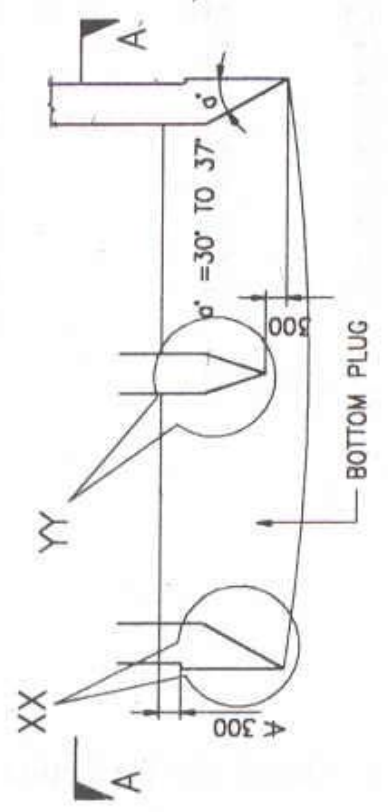
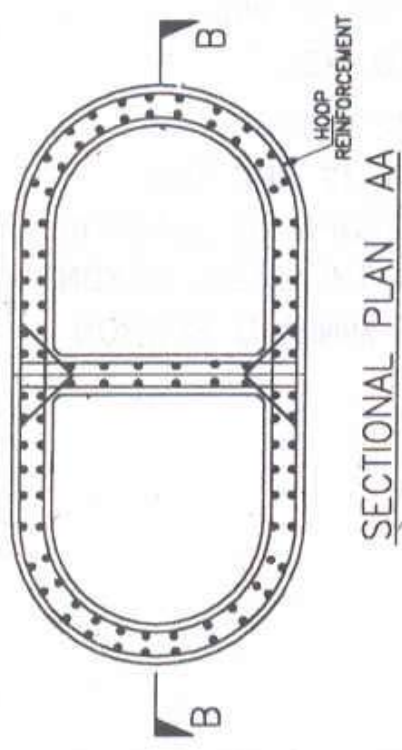
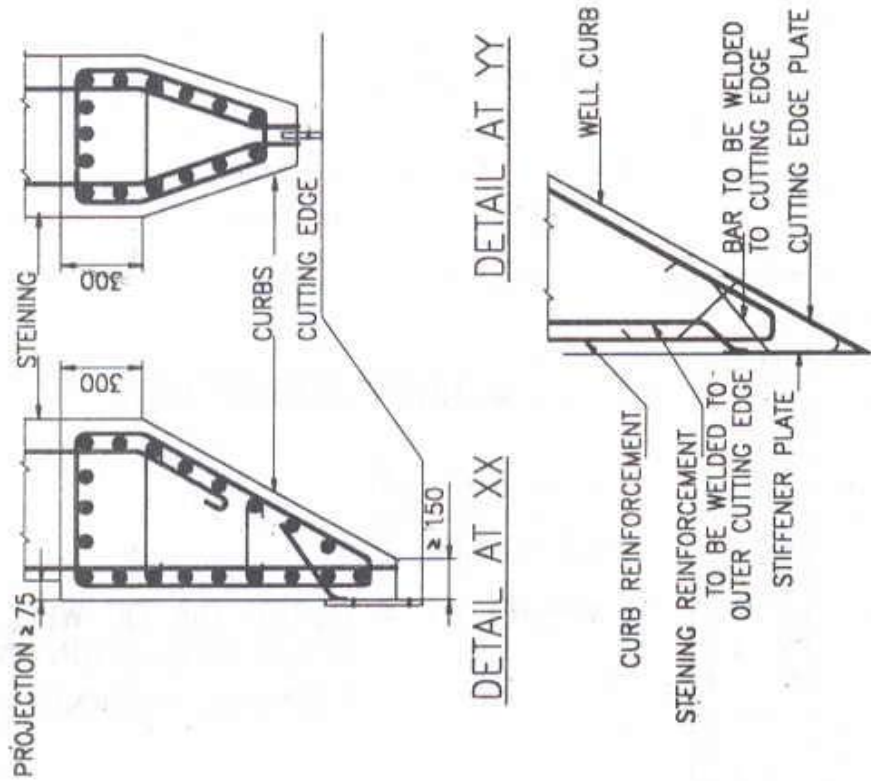
WHERE $d1$ = OUTER DIA OF WELL
AFTER REDUCTION IN
STEINING THICKNESS

IS = DEPTH OF WELL
UPTO MSL

$$ld = 3 (h - h1)$$

$t1$ & $t2$ ARE THE
DEVELOPMENT LENGTHS
FOR THE STEEL BEYOND
THE MINIMUM SECTION

Fig. 1. Sketch for Reduction of Steining Thickness



SECTIONAL ELEVATION BB
(REINFORCEMENT NOT SHOWN)

V TYPE CUTTING EDGE

Fig. 2 . Details of Well Curbs and Cutting Edge

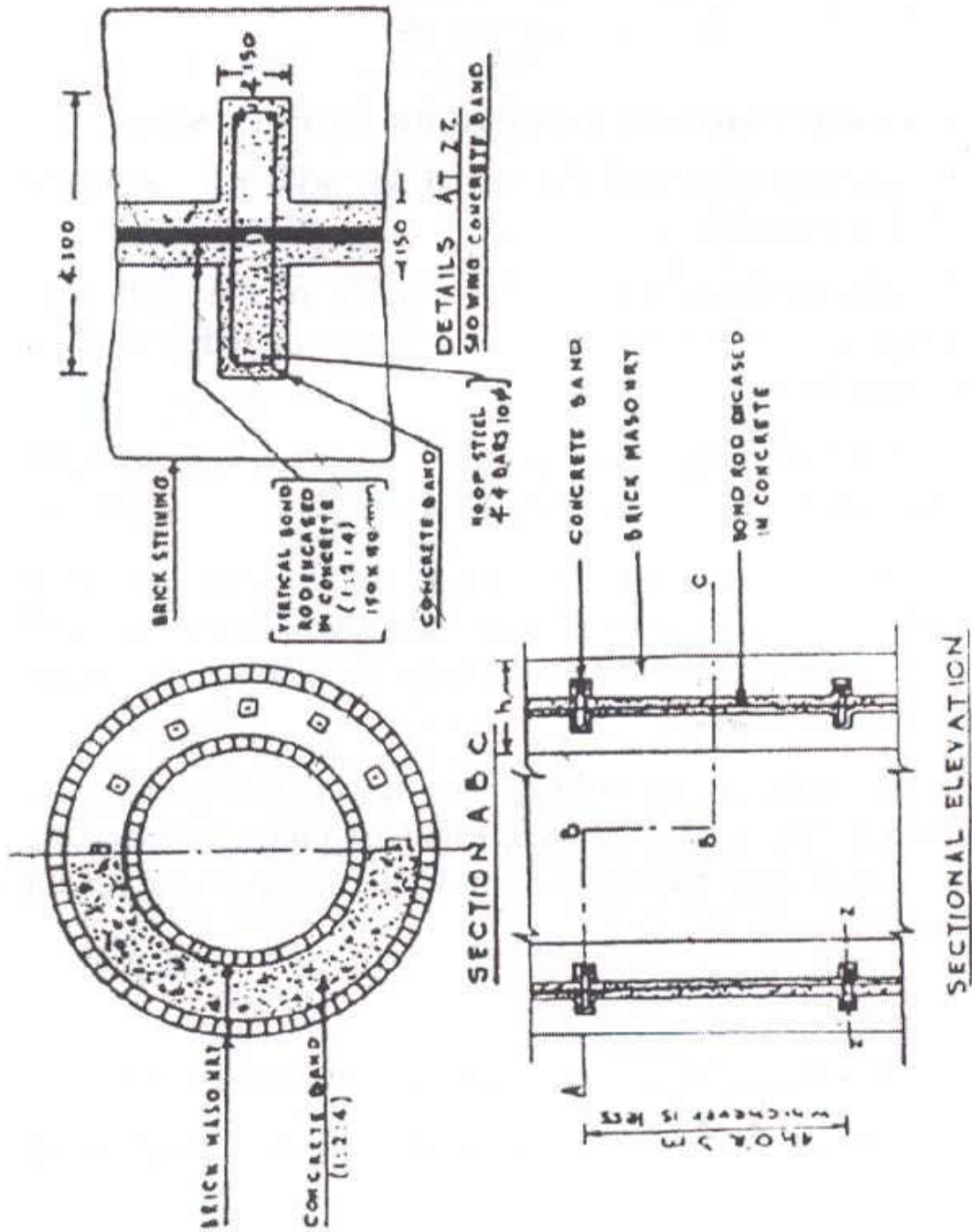


Fig. 3. Details of Well Curbs and Cutting Edge

PRECAUTIONS TO BE TAKEN DURING SINKING OF WELLS

(Ref. Clause 708.13)

1. CONSTRUCTION OF WELL CURB AND STEINING

1.1. Cutting edge and the top of the well curb shall be placed truly horizontal.

1.2. The methods adopted for placing of the well curb shall depend on the site conditions, and the cutting edge shall be placed on dry bed.

1.3. Well steining shall be built in lifts and the first lift shall be laid after sinking the curb atleast partially for stability.

1.4. The steining shall be built in one straight line from bottom to top and shall always be at right angle to the plane of the curb. In no case it shall be built plumb in intermediate stages when the well is tilted.

1.5. In soft strata prone to settlement/creep, the construction of the abutment wells shall be taken up after the approach embankment for a sufficient distance near the abutment has been completed.

2. SINKING

2.1. A sinking history record be maintained at site.

2.2. Efforts shall be made to sink wells true to position and in plumb.

2.3. Sumps made by dredging below cutting edge shall preferably not be more than half the internal diameter.

2.4. Boring chart shall be referred to constantly during sinking for taking adequate care while piercing different types of strata by keeping the boring chart at the site and plotting the soil as obtained for the well steining and comparing it with earlier bore data to take prompt decisions.

2.5. When the wells have to be sunk close to each other and the clear distance is less than the diameter of the wells, they shall normally be sunk in such a manner that the difference in the levels of the sump and the cutting edge in the two wells do not exceed half the clear gap between them.

2.6. When group of wells are near each other, special care is needed that they do not foul in the course of sinking and also do not cause disturbance to wells already sunk. The minimum clearance between the wells shall be half the external diameter. Simultaneous and level dredging shall be carried out in the dredging holes of all the wells in the group and plugging of all the wells be done together.

2.7. During construction partially sunk wells shall be taken to a safe depth below the anticipated scour levels to ensure their safety during ensuing floods.

2.8. Dredged material shall not be deposited unevenly around the well.

3. USE OF KENTLEDGE

3.1. Where a well is loaded with kentledge to provide additional sinking effort, such load shall be placed evenly on the loading platform, leaving sufficient space in the middle to remove excavated material.

3.2. Where tilts are present or there is a danger of well developing a tilt, the position of the load shall be regulated in

such a manner as to provide greater sinking effort on the higher side of the well.

4. SAND BLOWS IN WELLS

4.1. Dewatering shall be avoided if sand blows are expected. Any equipment and men working inside the well shall be brought out of the well as soon as there are any indications of a sand-blow.

4.2. Sand blowing in wells can often be minimised by keeping the level of water inside the well higher than the water table and also by adding heavy kentledge.

5. SINKING OF WELLS WITH USE OF DIVERS

5.1. Use of divers may be made in well sinking both for sinking purposes, like, removal of obstructions, rock blasting, etc. as also for inspection. All safety precautions shall be taken as per any acceptable safety code for sinking with divers or any statutory regulations in force.

5.2. Only persons trained for the diving operation shall be employed. They shall work under expert supervision. The diving and other equipments shall be of an acceptable standard. It shall be well maintained for safe use.

5.3. Arrangement for ample supply of low pressure clean cool air shall be ensured through an armoured flexible hose pipe. Standby compressor plant will have to be provided in case of breakdown.

5.4. Separate high pressure connection for use of pneumatic tools shall be made. Electric lights, where provided, shall be at 50 volts (maximum). The raising of the diver from the bottom of wells shall be controlled so that the decompression

rate for rivers conforms to the appropriate rate as laid down in the regulation.

5.5. All men employed for diving purposes shall be certified to be fit for diving by an approved doctor.

6. BLASTING

6.1. Only light charges shall be used under ordinary circumstances and should be fired under water well below the cutting edge so that there is no chance of the curb being damaged.

6.2. There shall be no equipment inside the well nor shall there be any labour in the close vicinity of the well at the time of exploding the charges.

6.3. All safety precautions shall be taken as per IS:4081 "Safety Code for Blasting and Related Drilling Operations", to the extent applicable, whenever blasting is resorted to. Use of large charges, 0.7 kg. or above, may not be allowed except under expert direction and with permission from Engineer-in-charge. Suitable pattern of charges may be arranged with delay detonators to reduce the number of charges fired at a time. The burden of the charge may be limited to 1 metre and the spacing of holes may normally be kept at 0.5 to 0.6 metre.

6.4. If rock blasting is to be done for seating of the well, the damage caused by the flying debris should be minimised by provisions of rubber mats covered over the blasting holes before blasting.

6.5. After blasting, the steining shall be examined for any cracks and corrective measures shall be taken immediately.

7. PNEUMATIC SINKING

7.1. The pneumatic sinking plant and other allied machinery shall not only be of proper design and make, but also shall be worked by competent and well trained personnel. Every part of the machinery and its fixtures shall be minutely examined before installation and use. Appropriate spares, standbys, safety of personnel as recommended in the IS:4188 for working in compressed air must be kept at site. Safety code for working in and other labour laws and practices prevalent in the country, as specified to provide safe, efficient and expeditious sinking shall be followed.

7.2. Inflammable materials shall not be taken into air locks and smoking shall be prohibited.

7.3. Whenever gases are suspected to be using out of dredge hole, the same shall be analysed by trained personnel and necessary precautions adopted to avoid hazard to life and equipment.

7.4. Where blasting is resorted to, it shall be carefully controlled and all precautions regarding blasting shall be observed. Workers shall be allowed inside after blasting only when a competent and qualified person has examined the chamber and steining thoroughly.

7.5. The weight of pneumatic platform and that of steining and kentledge, if any, shall be sufficient to resist the uplift from air inside, skin friction being neglected in this case.

7.6. If at any section the total weight acting downwards is less than the uplift pressure of air inside, additional kentledge shall be placed on the well.

7.7. If it is not possible to make the well heavy enough

during excavation. "Blowing Down" may be used. The men should be withdrawn and the air pressure reduced. The well should then begin to move with a small reduction in air pressure. "Blowing Down" should only be used where the ground is such that it will not have up inside the chamber when the pressure is reduced. When the well does not move with a reduction in air pressure, kentledge should be added. Blowing down should be in short stages and the drop should not exceed, 0.5 metre of any stage. To control sinking during blowing down, use of packs or packagings may be made.

8. TILTS AND SHIFTS OF WELLS

8.1. Tilts and shifts shall be carefully checked and recorded regularly during sinking operations. For the purpose of measuring the tilts along and perpendicular to the axis of the bridge, level marks at regular intervals shall be painted on the surface of the steining of the well.

8.2. Whenever any tilt is noticed, adequate preventive measures, like, putting eccentric kentledge, pulling, strutting, anchoring or dredging unevenly and depositing dredge material unequally, putting obstacles below cutting edge, after jetting etc., shall be adopted before any further sinking. After correction, the dredged material placed unevenly shall be spread evenly.

8.3. A pair of wells close to each other have a tendency to come closer while sinking. Timber struts may be introduced in between the steining of these wells to prevent tilting.

8.4. Tilts occurring in a well during sinking in dipping rocky strata can be safeguarded by suitably supporting the kerb.

9. SAND ISLAND

9.1. Sand island where provided shall be protected against scour and the top level shall be sufficiently above the prevailing water level so that it is safe against wave action.

9.2. The dimension of the sand island shall not be less than three times the dimension in plan of the well or caisson.



CAPACITY OF PILE BASED ON PILE SOIL INTERACTION

(Ref. Clause 709.3.1)

1. AXIAL CAPACITY OF PILES IN SOIL

Axial load carrying capacity of the pile is initially determined by calculating resistance from end bearing at toe/tip or wall friction/skin friction along pile surface or both. Based on the soil data, the ultimate load carrying capacity (Q_u) is given by:

$$Q_u = R_u + R_f$$

where, R_u = Ultimate base resistance

R_f = Ultimate shaft resistance

1.1. R_u' , i.e., Ultimate base resistance may be calculated from the following:

$$R_u = A_p(\frac{1}{2}D \cdot \gamma N_\gamma + P_d N_q) + A_p N_c C_p$$

where A_p = Cross sectional area of base of pile

D = Pile diameter in cm

γ = Effective unit weight of soil at pile tip in kg/cm^3

N_q & N_γ = Bearing capacity factors based on angle of internal friction at pile tip

N_c = Bearing capacity factor usually taken as 9

C_p = Average cohesion at pile tip (from unconsolidated undrained test)

P_d = Effective overburden pressure at pile tip limited to 20 times diameter of pile for piles having length equal to more than 20 times diameter

2. R_f i.e., Ultimate side resistance may be calculated from the following:

$$R_f = \sum_{i=1}^n K P_{di} \tan \delta A_{si} + \alpha \bar{C} A_s$$

where K = Coefficient of earth pressure

P_{di} = Effective overburden pressure in kg/cm^2 along the embedment of pile for the i th layer where i varies from 1 to n

δ = Angle of wall friction between pile and soil in degrees. It may be taken equal to angle of internal friction of soil

A_{si} = Surface area of pile shaft in cm^2 in the i th layer, where i varies from 1 to n

A_s = Surface area of pile shaft in cm^2

α = Reduction factor

\bar{C} = Average cohesion in kg/cm^2 throughout the embedded length of pile (from unconsolidated undrained test)

3. While evaluating effective overburden pressure, total and submerged weight of soil shall be considered above and below water table respectively.

4. The initial value of K may be taken as 1.5 which can be further increased upto 1.8 in particular cases as specified in Clause 709.2.2 (v).

5. The following value of α may be adopted depending upon consistency of soil:

Consistency	N Value	Bored piles cast-in-situ	Driven cast-in- situ piles
Soft to very soft clay	<4	0.7	1
Medium	4-8	0.5	0.7
Stiff	8-15	0.4	0.4
Very stiff	>15	0.30	0.3

6. For piles in over consolidated soils, the drained capacity may be evaluated.

7. When full static penetration data is available for the entire depth, then

$$Q_u = q_b \cdot A_p + f_s \cdot A_s$$

where q_b = Point resistance at base to be taken as average of the value over a depth equal to 3 times the diameter of pile above and one time the diameter of pile below the tip.

A_p = Cross-sectional area of base of pile

f_s = Average side friction and following co-relation may be used as a guide:

<u>Type of soil</u>	<u>Side Friction, f_s</u>
---------------------	--

Clay

Soft

$$q_c/25$$

Stiff

$$q_c/15$$

Mixture of silts and sand with traces of clay

Loose

$$q_c/50$$

Dense

$$q_c/100$$

q_c = Static point resistance

8. Where soft compressible clay layer is encountered, any contribution towards capacity of pile from such soil shall be ignored and additional load on pile on account of downward drag on pile due to consolidation of soft soil shall be considered.

Note: For factors of safety of piles in soil, refer Clause 709.3.2.

9. CAPACITY OF PILES IN ROCK

A pile socketed into rock derives its capacity from end bearing and socket side resistance. The ultimate load carrying capacity may be calculated from

$$Q_u = R_e + R_{af} = K_{sp} \cdot q_c \cdot d_f \cdot A_b + A_s q_s$$

where

- Q_u = Ultimate capacity of pile socketed into rock
- R_e = Ultimate end bearing
- R_{af} = Ultimate side socket shear
- K_{sp} = An empirical co-efficient whose value ranges from 0.1 to 0.4
- q_c = Average uniaxial compressive strength of rock at tip level
- A_b = Cross sectional area of base of pile
- d_f = Depth factor = $1 + 0.4 \times \frac{\text{Length of socket}}{\text{Dia. of socket}}$
Length of socket may be limited to 0.5 x dia. of socket
- A_s = Surface area of socket
- q_s = Ultimate shear along the socket value of q_s may be taken as 50 kg/cm² for normal rock which may be reduced to 20 kg/cm² for weathered rocks.

- Note : 1. For factors of safety on R_e & R_{af} refer Clause 709.3.2.
2. The maximum allowable end bearing pressure should be limited to 30 kg/cm² after applying factor of safety.

FILLING BEHIND ABUTMENTS, WING AND RETURN WALLS

(Ref. Clause 710.1.4)

1. FILLING MATERIALS

The type of materials to be used for filling behind abutments and other earth retaining structures, should be selected with care. A general guide to the selection of soils is given in Table 1.

**Table 1. General Guide to the Selection of Soils on Basis of
Anticipated Embankment Performance**

Soil group according to IS:1498-1970		Visual description	Max. dry density range kg/m ³	Optimum moisture content range per cent	Anticipated embankment performance
Most probable	Possible				
GW, GP, GM, SW, HP		Granular materials	1850-2280	7 - 15	Good to Excellent
SB, SM, GM, GC, SM, SC		Granular materials with soil	1760-2160	9 - 18	Fair to Excellent
SP		Sand	1760-1850	19 - 25	Fair to Good
ML, MH, DL	CL, SM, SB, SC	Sandy Silts & Silts	1760-2080	10 - 20	Fair to Good

2. LAYING AND COMPACTION

2.1. Laying of Filter Media for Drainage

The filter material shall be well packed to a thickness of not less than 600 mm with smaller size towards the soil and bigger

size towards the wall and provided over the entire surface behind abutment, wings or return walls to the full height.

Filter materials need not be provided in case the abutment is of spill through type.

2.2. Density of Compaction

Densities to be aimed at in compaction shall be chosen with due regard to factors, such as, the soil type, height of embankment, drainage conditions, position of the individual layers and type of plant available for compaction.

Each compacted layer shall be tested in the field for density and accepted before the operations for next layer are begun.

3. EXTENT OF BACKFILL

The extent of backfill to be provided behind the abutment should be as illustrated in Fig. 1.

4. PRECAUTIONS TO BE TAKEN DURING CONSTRUCTION

4.1. The sequence of filling behind abutments, wing walls and return walls shall be so controlled that the assumptions made in the design are fulfilled and they should clearly be indicated in the relevant drawings. For example, if the earth pressure in front of the abutment is assumed in the design, the front filling shall also be done simultaneously alongwith the filling behind abutment, layer by, and in case the filling behind abutment before placing the superstructure is considered not desirable, the filling behind abutment should also be deferred to a later date. In case of tie beams and friction slabs, special care shall be taken in compacting the layer underneath and above

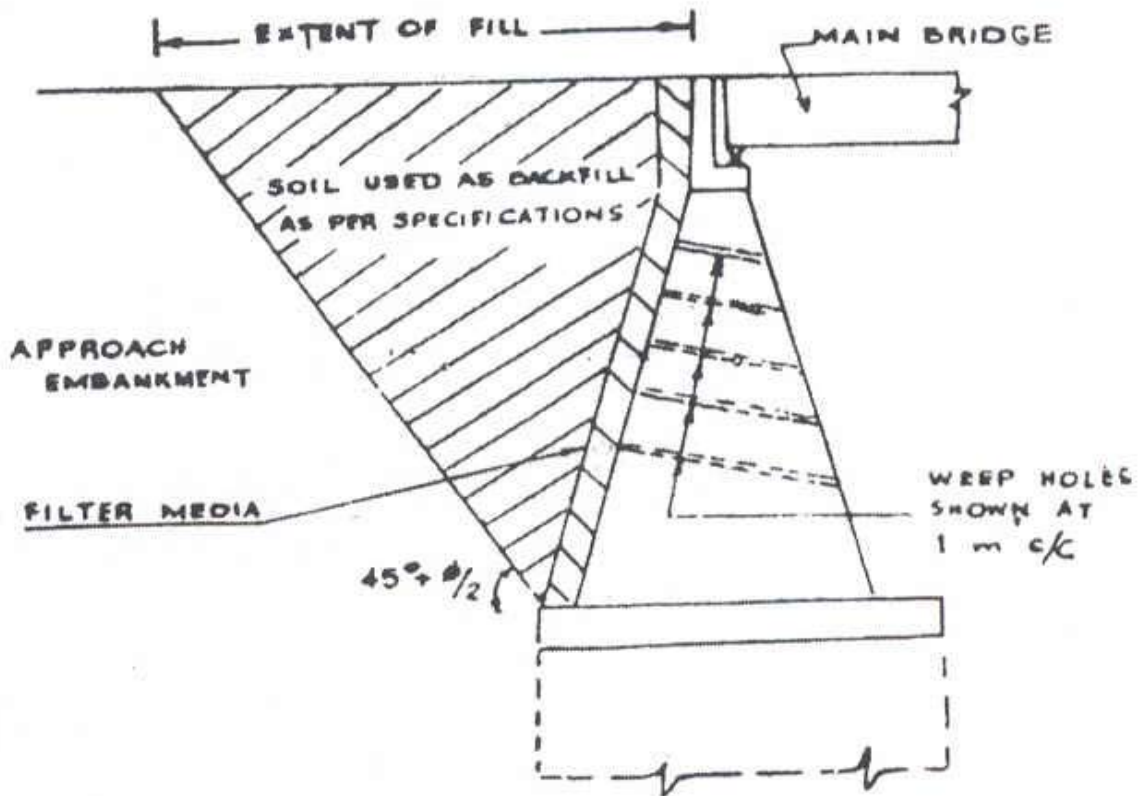


Fig. 1.

them so that no damage is done to them by mechanical equipment.

4.2. Special precautions should be taken to prevent any wedging action against structures, and the slopes bounding the excavation for the structure shall be stepped or strutted to prevent such wedging action.

4.3. Adequate number of weep holes not exceeding one metre spacing in both directions should be provided to prevent any accumulation of water and building up of hydrostatic pressure behind the walls. The weep holes should be provided above the low water level.