STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR ROAD BRIDGES

SECTION : VII

FOUNDATIONS AND SUBSTRUCTURE

(Revised Edition)

(Incorporating all Amendments and Errata Published upto December, 2013)

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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 Unified Code. The second revision of this code was undertaken by the Foundation and Structure Committee (B-4) and the initial draft was finalized 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) under Convenorship of Shri S.A Reddi. 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 Kolkata on 4.11.2000 for publishing the revised IRC Code Section VII: IRC: 78:2000. Since then numerous amendments and errata were published to this Code based on development in design and construction technology.

The current Revised Edition of IRC:78 includes all the amendments and errata published from time to time upto December, 2013.

The Revised Edition of IRC:78 "Standard Specifications and Code of Practice for Road Bridges" Section VII – Foundation incorporating all amendments and Errata published till date was approved by Foundations and Substructure Foundation Substructure Protective Works and Masonry Structures Committee (B-3) in its meeting held on 16.10.2013. The Revised Edition of IRC:78 was approved by the Bridges Specifications and Standards Committee in its meeting held on 06.01.2014 and Executive Committee held on 09.01.2014 for publishing.

The composition of the B-3 Committee is given below:

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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 support 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 other side, 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 soil/rock expressed as bearing stress is referred to as it 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 gross 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 use in multiple span 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-situ 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-situ 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 into 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 increased 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 45^0 to the alignment of road or parallel to the river and follows profile or 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.

710.12.1 *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_{sm}	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_{wc}	Water Current
F_{te}	Temperature effects [See Note (i)]
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
l_s	Depth of well in metre, up to MSL.
N	Standard penetration test value
P_a	Total active pressure
P_p	Total passive pressure
Q	Live load

R_g	Dead load reaction
R_q	Live load reaction
V_r	Shear rating of elastomeric bearing
W	Wind load
α	Reduction factor.
β	Ratio of long side to the short side of the footing
S_u	Undrained shear strength
q_u	Undrained chohesion
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 and 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 km2	Increase over design Discharge in percent
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 width of effective 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 $1.76\sqrt{d_m}$ 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 forready 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 0.7
Medium sand	0.223 to 0.505	0.85 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 by 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 bouldery strata. Soundings, wherever possible, shall be taken in the vicinity of the site for 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 viaduct/ROB having no possibility of scour, resistance of soil may be considered below depth of excavation for services construction, or 2.0 m below ground level whichever is greater.

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 on upstream may be 0.7 times of the same on 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 on 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, aggradations/degradation of bed, etc. are likely to affect the behavior of the structures.

vii) An additional tow-lane bridge when located near to the existing bridge, on major rivers.

***NOTE*:** These studies shall be conducted for the increase 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 dsm 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 Clause **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 specialized 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 foundations 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 'dsm' 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 Foundation

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 credible 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.

	Type of Rock	Embedment depth
a)	For rocks of moderately strong and above in table 2 of classification of rock (under clause 8.2 of appendix 2) having UCS of more than 12.5 MPA or where it is not possible to take core to get the UCS but extrapolated SPT N value is more than 500	0.6 m
b)	For rock of moderately weak and below in table 2 of classification of rock (under clause 8.2 of appendix 2) having UCS < 12.5 MPA but ≥ 2.5 MPA or where it is not possible to take core to get the UCS but extrapolated SPT N value is more than 100 but less than 500	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 chiseling/blasting. Diameter of sump may be 1.5 to 2 m less than inner dredge-hole subject to a minimum size of 1.5m . 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.

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) $+ F_{eg} + F_{wp}$

or

ii) $+F_{im} + F_{wp}$

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 $33^1/_3$ percent for the combination of wind *(W)* and 50 percent for the combination with seismic (F_{eq}) or (F_{im}). The permissible increase in allowable base pressure should be 25 percent for all combinations except combination 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 percent.

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
Non-Seismic Combinations		
Greater of the two values given below :		
i)	$F_h - \mu(R_g + R_q)$	$\mu(R_g + R_q)$
ii)	$\dfrac{F_h}{2} + \mu\,(R_g + R_q)$	$\mu(R_g + R_q)$
Seismic Combinations		
	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) Telflon on stainless steel : 0.03 and 0.05 (whichever is governing)

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

$$\frac{F_h}{2} \text{ or } \mu(Rg) \text{ whichever is greater}$$

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

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

V_r = Shear rating of the electrometric 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 For open foundations and well foundation resting on soil, the allowable bearing pressure on ultimate bearing capacity may be taken as 2.5 for soil.

706.3.1.1.2 For open foundations and well foundation resting on rock, 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.** For Factor of safety in case of pile foundation the clause 709.3.2 shall be referred

The intermediate geo-material like 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 percent of the total area for load combination including seismic, or impact of barge, and 80 percent 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-efficient between concrete and soil/rock will be Tan Ø, Ø 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, 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 footing, 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 column/ 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 from 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 1/3rd 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 resisiting section as below:

a) In one-way reinforced footing, the reinforcement shall be same as calculated for critical unit width as mentioned in Clause **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:

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

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 as per IRC: 112, Clause number 16.5.1.1

707.2.8 All faces of the footing shall be provided with a minimum steel of 250 mm²/ 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 45^0 to vertical.

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 dispersions 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 minimize 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 connot be laid dry or the situation is such 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 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 Foundations supporting the superstructure located in deep water cannels shall comprise of properly dimensioned caissons preferably having a single dredge hole. While selecting the shape, size and type of well, the size of abutment and 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 data to be penetrated should be kept in view. The minimum dimensions of dredge-hole shall not be less than 3 m. In case there is deep standing water, properly designed floating caissons may be used as per Clause **708.12.**

However, in case of larger bridges across rivers in wide flood plains prone to scour, delta/tidal rivers, channels with inland waterway traffic and bridges in coastal/marine locations, the number of intermediate foundations shall be reduced as far as practicable.

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) Stress 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.

708.1.4 Bottom plug of well should be suitably designed to resist maximum upward force acting on it during construction following plugging as well as during life span of the structure.

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:

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

h = minimum thickness of steining in m

d = external diameter of circular well of 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 percent.

c) **Bouldery 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 m, 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 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.

708.3.2 For brick masonry wells, brick not less than Grade-A having strength not less than 7MPa- conforming to IS 1077 shall be used in cement mortar not leaner than 1:3.

708.3.3 For plain concrete wells, vertical reinforcements (whether mild steel or deformed bars) in the steining shall not be less than 0.12 percent 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 percent of the volume per unit length of the steining, as shown in the ***Appendix-3 (Fig. 2).***

708.3.4 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 percent (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 percent (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 percent 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.**

708.3.5 The vertical bond rods in brick masonry steining shall not be less than 0.1 percent 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 percent 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).***

708.3.6 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/m 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 lover 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 'as' 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 geotechnical 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/cum 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 concerting 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 termie to place 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 by tremie under still water condition and the cement content of mix be increased by 10 percent.

708.8.3 In case grouted colcrete, 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 interstices 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 be laid as low as possible but above the LWL in the active channel. Where the bed level is higher than LWL the bottom of well cap may be suitably raised.

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 at least 48 hours. A complete record of sinking operating 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 case if concrete steining is provided, it shall be rendered air tight by restricting the tension in concrete which will not exceed 3/8th 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 is proposed should not exceed 30 .

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 sub-surface 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 6 mm

For bridges located in land, steel liners of minimum thickness of 6 mm shall be provided in cases given below. The liner shall be provided up to depth up to which following situations prevail.

a) While constructing the pile foundation through very soft clay ($N \leq 3$) ,very loose sandy strata ($N \leq 8$), bouldery formation and artesian conditions, wherein the walls of boreholes cannot be stabilized by bentonite circulation.

b) Where sewage leakage is common phenomenon as well as sites with aggressive soil/water environment

709.1.5 *Spacing of piles and tolerances*

709.1.5.1 Spacing of piles

a) Where pier is supported on multiple piles, connected by frame structure or by solid pile cap, the spacing of piles should be considered in relation to the nature of the ground, their behavior in groups and the execution convenience. 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.

b) For land bridges, pier may be supported on single pile having diameter sufficiently large to accommodate construction tolerances of pile installation with reference to location of piers as well as having strength as required by the design. The pile should be designed to cater for the maximum eccentricity of vertical load in such case. Alternatively, pile shaft can be continued to act as a pier and get connected to pier cap which is designed to accommodate the eccentricities due to construction tolerances.

The size 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 type, size and spacing of piles.

The spacing of piles will be determined by many aspects mentioned above. The working rules which are generally, though not always, suitable, are as follows:

For friction piles, the spacing centre should not be less than the perimeter of the pile or for circular pile, 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 pile 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 shall be 1.0 m for river/marine bridges. For bridges beyond the water zone and for bridges on land, the diameter may be reduced upto 750 mm.

709.1.8 *Settlement, differential settlement and pile capacity*

The Differential settlement between two successive foundations taken at pile cap level, may be estimated from the maximum settlement expected at two foundations for the dead load, superimposed loads, live load and scour effect. The increase in settlement with time in clayey soils shall be accounted for. In absence of detailed calculations, for the purpose of preliminary design, it can be taken as not more than the maximum settlement of any of the two foundations.

The differential settlement shall be limited depending upon the following functional and structural considerations:

a) Functionally, acceptable differential settlement between two neighboring piers shall not be greater than 1 in 400 of the span to ensure riding comfort, as specified in Clause **706.3.2.1.**

b) The allowable settlement of a single pile considered for estimating the pile capacity shall be arrived from correlation of the settlement of pile group to that of single pile, as per Clause **709.3.4.**

c) It is further provided that the working load capacity of pile based on the sub-clause b) shall not exceed 40 percent of the load corresponding to the settlement of 10 percent of pile diameter (i.e. safety factor of 2.5 on ultimate load capacity is ensured).

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:

	Concrete Cast-in-situ by Tremie	**Precast Concrete**
Grade of concrete	M 35	M35
Min. cement contents	400 kg/m^3	400 kg/m^3
Max. W.C. ratio	0.4	0.4
Slump (mm)	150-200	50-75

NOTE : i) For improving resistance to penetration of harmful elements from soil use of mineral admixtures (flyash, silica fume, GGBS conforming to respective BIS/International standards) and as per IRC:112 is recommended.

ii) In marine conditions and areas exposed to action of harmful chemicals, protection of pile caps with suitable coating such as bituminous based, coal-tar epoxy based coating may be considered. High alumina cements, (i.e. quick setting cement) shall not be used in marine conditions. Also when both chlorides and sulphates are present, use of sulphate-resistant cement is not recommended.

709.2 Requirement and Steps for Design and Installation

709.2.1 The initial design of an individual pile, confirmation of its capacity by either initial load test or by re-confirmation of actual soil parameters, modification of design, if required, and final adoption should pass through following steps of investigations, design and load testing:

i) Comprehensive and detailed sub-surface investigation for piles to determine the design parameters of end bearing capacity, friction capacity and lateral capacity of soil surrounding the pile.

ii) Design of pile and pile group based on i) above for specified bearing strata.

iii) Initial load testing:

Initial load test on pile of same diameter as design pile for direct confirmation of design.

The initial load test is a part of the design process confirming the expected properties of bearing strata and the pile capacity.

iv) Steps ii) and iii) should be repeated for different types of strata met at site.

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

i) Sub-soil exploration to reconfirm soil parameters assumed in the design.

ii) Provide for the required design capacity of pile group based on tentative number and diameter of piles in a group.

iii) The allowable total/differential settlement of single pile should be based on the considerations as per Clauses **709.1.8** and **709.3.4.** Capacity of single pile is to be based on static formula considering ground characteristics. This step along with step ii) may be iterative.

iv) Structural design of piles.

v) Initial load test as mentioned in Clause **709.2.1 iii)** is for axial load capacity, including uplift capacity, if required, on trial piles of the same diameter as the design pile. The testing shall be done as per the procedure laid down in IS:2911, Part-IV. This load test shall be conducted for not less than $2^1/_2$ times the design load. The initial load test shall be cyclic load test for piles deriving strength from end bearing and side friction. The maintained load test can be performed for end bearing piles without relying on friction, and for the socketed piles in rock.

vi) If the initial load test gives a capacity greater than 25 percent of the capacity calculated by static formula, and if it is desired to take benefit of the higher capacity, 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.

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 load test

i) Routine load test should be done at locations of alternate foundations of bridges to reconfirm or modify the allowable loads. Vertical and horizontal load tests should be properly designed to cover particular pile group. The lateral load test may be conducted on two adjacent piles. However, results of routine load tests shall not be used for upward revision of design capacity of piles. The minimum number of tests to be conducted is as given below for confirming pile capacity.

Total number of Piles for the Bridge	Minimum No. of Test Piles
Upto 50	2
Upto 150	3
Beyond 150	2 percent of total piles (fractional number rounded to next higher integer number).

NOTE: The number of tests may be judiciously increased depending upon the variability of foundations strata.

ii) Permissible Over Load

While conducting routine test on one of the pile belonging to a pile group,if the pile is found to be deficient (based on the settlement criteria at 1.5 times the design load) an overload upto 10 percent of the reduced capacity may be allowed.

iii) For a quick assessment of pile capacity, high-strain dynamic tests may be conducted after establishing co-relations using the results of load tests. However, results of the strain dynamic tests shall not be used for upward revision of design capacity of pile. Detailed guidelines are at ***Appendix-7 Part 1.*** These methods can be followed.

iv) To have a fairly good idea about the quality of concrete and construction defects like voids, discontinuities, etc., pile integrity tests are extensively conducted. Detailed guidelines and references are at ***Appendix-7 Part 2.***

709.3 Geotechnical Capacity of Pile

709.3.1 *Resistance to vertical loads*

For calculating designed capacity of pile for load combination I of Clause **706.1.1**, recommendation of ***Appendix-5*** should be followed. For calculating capacity of pile group, refer Clauses **709.3.3** and **709.3.4** and the allowable settlement criteria as per Clause **709.1.8.** For purpose of these Clauses the following definitions will apply.

a) Cohesive soil (clay or plastic silt with $S_u \leq 0.25$ MPa);

b) Granular soil (sand, gravel or non-plastic silt with N (average within layer) ≤ 50 blows/0.3 m (50 blows/30 cms);

c) Intermediate Geomaterial

Cohesive: e.g. clay shales or mudstones with 0.25 MPa (2.5 tsf) < S_u < 2.5 MPa.

Cohesion less: e.g. granular tills, granular residual soils N>50 blows/0.3 m (50 blows/30 cm);

d) Rock: Cohesive, Cemented Geomaterial with $S_u \geq$ 2.5MPa (25 tsf) or $q_u \geq$ 5.0 MPa.

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 load combination I of Clause **706.1.1** for piles in soil. For piles in rock, factor of safety shall be 3 on end bearing component and 6 on socket side resistance component, for load combination I of clause **706.1.1** subject to further limits as stipulated in ***Appendix-5***.

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 2.0 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 lesser 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 in 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. Methods given in IS 8009 (Part-II) of any other rational method may be used. The settlement of group of piles in sands can be calculated by assuming that the load carried by the pile group is transferred to the soil through an equivalent raft located at one third of the pile length upwards from the pile toe for friction piles. For end bearing piles the settlement can be calculated by assuming the raft placed at the toe of the pile group.

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 (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 of pile-group must not exceed the sum of lateral resistance of individual piles. The safe lateral load resistance of individual pile depends on the modulus of horizontal sub-grade reaction of the foundation material as well as the structural rigidity of pile. Appropriate rational method of analysis using soil modulus as recommended by IS:2911 may be used to calculate the same. For safe lateral load resistance in load combination I of Clause **706.1.1**, the deflection at scour level shall not be greater than 1.0 percent of pile diameter. Checking of deflections in other load combinations is not required. For a group of vertical piles, confirmation of capacity of group by load testing is not required. For single pile the horizontal load test may be performed in accordance with IS:2911. Testing shall be for free head condition for piles having free standing shaft above scour level upto the pile cap. For conduction test at scour level, it will be necessary to drive a larger diameter casing upto scour level so that the test pile above is free to deflect. The deflection at scour level may be measured directly, or may be calculated from deflection measured at higher (ground) level assuming that the pile acts as structural cantilever from the point of fixity. The point of fixity can be taken from the analysis performed for the design or calculated by simplified method given in IS:2911.

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 0.70 on the same. In case of rock, the socket length shall be measured from 0.3 m depth to actual depth of socket. The weight of the pile shall also be taken as acting against uplift. Pull out test shall be conducted for verification of uplift capacity. Factor of (2.5/0.7) = 3.5 on the ultimate strength shall be used.

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 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*

When a soil stratum through which pile shaft has penetrated into an underlying hard stratum compresses due to its own weight, or remoulding, or surface load etc., additional vertical load is generated along the pile shaft in such stratum. 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 piles in the group times the individual downward drag forces, the same shall be cosidered in the design.

iv) This reduction in capacity of pile is in the ultimate capacity

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:112.

The test pile shall be separately designed to carry test load safely to the foundation.

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 loads the moments in pile shaft can be calculated as described in Clause **709.3.5.2.** For piles on land, if the pile group is provided with rigid cap, then the piles may be considered as having fixed head in appropriate direction for this purpose. Horizontal force may be distributed equally in all piles in a group with a rigid pile cap.

709.4.4 *Reinforcements for cast-in-situ piles*

The reinforcements in pile should be provided complying with the requirements of IRC:112, as per the design requirements. The area of longitudinal reinforcement shall not be less than 0.4 percent nor greater than 2.5 percent of the actual area of cross-section in all cast-in-situ concrete piles. The clear spacing between vertical bars shall not be less than 100 mm. Grouping of not more than two bars together can be made for achieving the same. Lateral reinforcement shall be provided in the form of spirals with minimum 8 mm diameter steel, spacing not more than 150 mm. For inner layer of reinforcement, separate links tying them to each other and to outer layers shall be provided.

709.4.5 For pre-cast driven piles, the reinforcement should comply with the provision of IRC:112, 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 percent;

b) For piles with a length 30 to 40 times the least width - 1.5 percent; and

c) For piles with a length greater than 40 times the least width -2 percent.

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 tolerance 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, 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 1.5 times the diameter of pile. Such a pile cap can be considered as rigid. The pile cap may be designed as thick slab or, by using 'strut & tie' method. All reinforcement in pile cap shall have full anchorage capacity beyond the point at which it is no longer required. It should be specially ascertained for pile cap designed by 'strut & tie' method. Where large diameter bars are used as main reinforcement, the corners of pile caps have large local cover due to large radius of bending of main bars. Such corners shall be protected by locally placing small diameter bars.

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 I dry conditions.

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 1/Section I).

709.6.1.2 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.

709.6.1.3 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.*1.4* 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 placing reinforcement.

709.6.2.3 Concreting of piles

Concreting shall be done by tremie method. In tremie method the following requirements are particularly applicable.

a) When concreting is carried out for a pile, a temporary casing should be installed to sufficient depth, so that fragments of ground cannot drop from the sides of the hole into the concrete as it is placed. The temporary casing is not required except near the top when concreting under drilling mud.

b) The hopper and tremie should be a leak proof system.

c) Tremie diameter of minimum 200 mm shall be used with 20 mm diameter down aggregate.

d) The first charge of concrete should be placed with a sliding plug pushed down the tube ahead of it or with a steel plate of adequate charge to prevent mixing of concrete and water. However, the plug should not be left in the concrete as a lump.

e) The tremie pipe should always penetrate well into the concrete with an adequate margin of safety against accidental withdrawal of the pipe. The tremie should be always full of concrete.

f) The pile should be concreted wholly by tremie and the method of deposition should not be changed part way up the pile, to prevent the laitance from being entrapped within the pile.

g) All tremie tubes should be scrupulously cleaned after use.

h) As tremie method of concreting is not under water concreting, these is no need to add 10 percent extra cement.

i) Normally concreting of the piles should be uninterrupted. In the exceptional case of interruption of concreting; but which can be resumed within 1 or 2 hours, the tremie shall not be taken out of the concrete. Instead it shall be raised and lowered slowly, from time to time to prevent the concrete form setting. Concreting should be resumed by introducing a little richer concrete with a slump of about 200 mm for easy displacement of the partly set concrete.

   If the concreting cannot be resumed before final set of concrete already placed, the pile so cast may be rejected or accepted with modifications.

j) In case of withdrawal of tremie out of the concrete, either accidentally or to remove a choke in the tremie, the tremie may be reintroduced in the following manner to prevent impregnation of laitance or scum lying on the top of the concrete already deposited in the bore.

k) The tremie shall be gently lowered on to the old concrete with very little penetration initially. A vermiculite plug/surface retarders should be introduced in the tremie. Fresh concrete of slump between 150 mm to 175 mm should be filled in the tremie which will push the plug forward and will emerge out of the tremie displacing the laitance/scum. The tremie will be pushed further in steps making fresh concrete sweep away laitance/scum in its way. When tremie is buried by about 60 to 100 cm, concreting may be resumed.

l) The 'L' bends in the reinforcements at the bottom of the piles should not be provided to avoid the formation of soft toe.

709.6.2.4 Removal of concrete above cut-off level

It is desirable that the concrete above cut-off level is removed before the concrete is set. The concrete may be removed manually or by specially made bailer or other device. Such removal of concrete helps in preventing the damages of the good concrete below the cut-off level which results from chipping by percussion method.

The removal of concrete can be within ± 25 mm from the specified cut-off level preferably on the (-) side. On removal of the such concrete, the concrete should be compacted with rammer with spikes or it shall be vibrated.

In case the concrete is not removed before setting, a groove shall be made on outer perimeter by rotary equipment before chipping by percussion method.

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 spelling by heat of welding. Location of mechanical couplers in neighboring 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 severe environment (as per Clause14.3.1 of 112 or as per clause 302.6 table 5 of IRC:21) 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 up to 90 percent 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 from 0.5 to zero

iii) For calculating the pressure on the portion of the substructure, at 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 a 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 pier consisting of two or more columns, the horizontal forces at the bearing be distributed on columns as required by appropriate analysis.

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 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. In absence of detailed calculations, a minimum difference of 1.5 m in water levels on two sides shall be assumed.

710.2.10 The lateral reinforcement of the walls of hollow circular RCC pier shall not be less than 0.3 percent of the sectional area of the walls of the pier. This lateral reinforcement shall be distributed 60 percent on outer face and 40 percent 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 percent 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 percent 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 running 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 percent where two columns have been provided and by 100 percent 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 earth fill. 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 of plain or reinforced concrete or of masonry. The abutment may be either solid type, buttressed type, counterfort type, box 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 submerged/ saturated unit weight of earth as appropriate 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.

710.4.11 In case of box type abutments, weep holes shall be provided similar to hollow piers as per Clause **710.2.9.**

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. It is designed for condition that even if arch on one side of it collapses, the pier and arches on other side will remain safe.

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 provide 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 200 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°. 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, over and above space for end block in cast in-situ cases.

v) The drainage arrangement for the water on the cap.

vi) Seismic arrestor, if provided

vii) To accommodate inspection ladders

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	**3 m**	**6 m**	**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.

710.8.7 Reinforcement in Pier and Abutment Caps where the bearing satisfied the square-root formula stated in Clause 307.1 of IRC:21, the pier caps shall be reinforced with a total minimum of 1 percent steel, assuming a cap thickness of 225 mm. The total steel shall be distributed equally and provided both at top and bottom in two directions. The reinforcement in the direction of the length of the pier shall extend from end to end of the pier cap while the reinforcement at right angles shall extend for the full width of the piers cap and be in the form of stirrups. In addition, 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 centres in both directions shall be provided directly under the bearings.

710.9 Cantilever Cap of Abutment and Pier

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 face 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 face 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 percent 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 The height of the pedestal should be between 150 mm and 500 mm. Where the depths of superstructures from two adjacent spans on a common pier differ and require use of pedestals of more height below one of the spans, the shape of pier cap or the diaphragm of superstructure shall be modified to restrict the height of pedestals to 500 mm. 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.

$$C = C_o \, x \sqrt{\frac{A_1}{A_2}} \ngtr C_o$$

where

C_o = 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.

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Appendix-1
(Clause **703.2.2.2)**

GUIDELINES FOR CALCULATING SILT FACTOR FOR BED MATERIAL CONSISTING OF CLAY

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 0.04mm offers more resistance 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 ø <15° and *c* (cohesion of soil) > 0.2 kg/cm² 'K_{sf}' calculated as follows:

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

where

F = 1.50 for Φ > 10° and < 15°

= 1.75 for Φ > 5° and < 10°

= 2.00 for Φ < 5°

ii) Soils having ø >15° will be treated as sandy soil even if c is more than 2N/mm² and silt factor will be as per provisions of Clause **703.2.2.**

Appendix-2
(Clause **704.3**)

GUIDELINES FOR SUB-SURFACE EXPLORATION

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/confirmatory 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 part 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 etc., 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 exploration 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 IS1892 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 into 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 of Exploration (Open Foundation)

These shall cover cases where the depth of exploration is not large 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 $^1/_2$ 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 for properties of soil are different for cohesive and cohesionless soils. These are indicated below. While selecting the tests and interpreting the results, limitations of applicability of chosen tests shall be taken into account. A most suitable and appropriate combination of these shall be chosen, depending on the properties needed for design and constructional aspects.

I) Cohesionless Soil

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, (as per IS 1888).

ii) Standard Penetration Tests (as per IS 2131)

iii) Dynamic Cone Pentration Test, (as per IS 4968 Part I or Part II).

iv) Static Cone Penetration Test, (as per IS 4968 Part III).

NOTE: Where dewatering is expected, permeability tests may be conducted (as per IS 2720 Part XVII).

II) Cohesive Soil

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, (as per IS:1888).

ii) Vane Shear Test, (as per IS:4434).

iii) Static Cone Penetration Test, (as per IS:4968 Part III).

iv) Standard Penetration Test, (as per IS:2131)

v) Dynamic Cone Penetration Test, (as per IS:4968 Part I or Part II).

NOTE: Where dewatering is expected, permeability tests may be conducted (as per 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.

Boring and sampling tends to cause remoulding of sensitive clays. Also for fissured or layered clays, the sample may not truly represent the in-situ properties, due to disturbance and stress changes caused by boring and sampling activity. In such cases, in-situ tests shall be performed.

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 $1\frac{1}{2}$ 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 $1\frac{1}{2}$ metre or at change of strata.

Table 1 Sub-Soil Data Required for Deep Foundations

Zones	Data/Characteristics Required	Sampling and Testing
Bed level to anticipated maximum scour depth	i) Soil Classification ii) Particles size distribution iii) Permeability, where dewatering is expected.	**Sampling -** Disturbed samples may be collected. **Laboratory Tests -** Classification Tests, including particle size distribution. **In-situ Tests:-** Permeability tests.
Maximum anticipated scour level to the foundation level	i) Soil Classification. ii) Particles size distribution iii) Moisture content, density, void ratio. iv) Shear strength. v) Compressibility. vi) Permeability where dewatering is expected. vii) Chemical analysis of soil and ground water (for aggressive elements).	**Sampling for Laboratory Tests -** Undisturbed samples shall be collected for these tests. As an exception, for (i) and (ii), disturbed samples may be permitted. **Laboratory Tests -** a) Classification Tests including particle size distribution. b) Moisture content, density, void ratio c) Shear strength - Triaxial tests to be done on undisturbed samples. Unconfined compression tests to be done on undisturbed and/or remoulded samples d) Consolidation tests. **In-situ Tests for Cohesionless soils-** a) Dynamic Cone Penetration Test. b) Standard Penetration Test. c) Down hole/Cross hole seismic surveys. d) Permeability tests. **In-situ Tests for Cohesive Soils -** a) Dynamic Cone Penetration Test. b) Static Cone Penetration Test - cone and skin resistance c) Field Vane Shear Test. d) Permeability tests. e) Down hole/Cross hole seismic surveys.
Foundation level to about 1.5 times of the width of foundation and below it.	i) Soil Classification ii) Shear Strength iii) Compressibility	Same as above

Notes:

1) Laboratory tests to be conducted according to the relevant parts of IS 2720.
2) Use of sophisticated equipment like, pressure meter may be made, if suitable co-relations for interpretation of data collected are available.
3) Down hole/Cross hole seismic surveys shall be as per ASTM 4428/D 4428 M
4) Seismic Methods and/or Electrical Resistivity Method can be used for soil/rock profiling. Down hole/Cross hole seismic surveys could be used for establishing elastic moduli and rock profiling at greater depths.
5) Down hole/Cross hole seismic surveys are useful for long bridges (i.e. of the order of 1 km and above) and for reducing the number of bores taken in the portions that are permanently under water. For these applications, geotechnical profiles obtained by seismic methods shall be calibrated/confirmed with actual profiles taken by bores at intermediate locations.

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 FOUNDATION 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 formation;
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 rock to the extent possible, where relevant

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 embedment 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 Practive for exploration by pits, trenches, drafts and shafts may be referred to. In case of 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 core shall be stored properly in accordance with IS:4078 - Code of Practice for Indexing and Storage of Drill Cores. Wherever triple tube core barrel in used, inner most transparent plastic tube shall be stored directly in core box with sample.

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

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. Particularly in weak and/or highly weathered rock, where triple tube core barrel is used for drilling, in-situ tests such as Standard Penetration Test and/or pressure meter test shall be conducted at every 1.5 m interval in the influence zone of footing or pile.

These in-situ tests are also useful in zones where water loss is recorded during drilling. However selections of these tests have to be done judiciously and planned along with drilling operation.

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 credibility 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, faults and weathered seams.

8.2 Classification of Rocks

Rocks may be classified based on their physical condition and Unconfined Compressive Strength as per Table-2.

9 Presentation of Data

The Presentation of Data collected shall be as done as illustrated in **Sheet No 1 and 2**

Table - 2 Classification of Rocks

Rock Type	Description	Unconfined Compressive Strength (UCS) in MPa
Extremely Strong	Cannot be scratched with knife or sharp pick. Breaking of specimen could be done by sledge hammer only.	>200
Very Strong	Cannot be scratched with knife or sharp pick. Breaking of specimens requires several hard blows of geologists' pick.	100 to 200
Strong	Can be scratched with knife or pick with difficulty. Hard blow of hammer required to detach hand specimen.	50 to 100
Moderately Strong	Can be scratched with knife or pick, 6 mm deep gouges or grooves can be made by hand blow of geologists' pick. Hand specimen can be detached by moderate blow.	12.5 to 50
Moderately Weak	Can be grooved or gouged 1.5 mm deep by firm pressure on knife or pick point. Can be broken into pieces or chips of about 2.5 mm maximum size by hard blows of the points of geologists' pick.	5 to 12.5

Weak	Can be grooved or gouged easily with knife or pick point. Can be break down in chips to pieces several cm's in size by moderate blows of pick point. Small thin pieces can be broken by finger pressure.	1.25 to 5
Very weak	Can be carved with knife. Can be broken easily with point of pick. Pieces 25 mm or more in thickness can be broken by finger pressure. Can be scratched easily by finger nail	< 1.25

Note:

1) The Unconfined Compressive Strength values are as in British Standard BS-5930 (Cl. 44.2.6).
2) Table-2 should not be used to infer the Unconfined Compressive Strength of rock. Actual laboratory test value of rock core should be used.

Appendix-3
(Clauses **708.2, 708.3** and **708.4**)

PROCEDURE FOR STABILITY CALCULATION

1 FORMULA 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 $2/3^{rd}$ of ø the angle of repose is subject to a limit of $22^1/_2$ 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 dully 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 variation of pressure) shall be separately evaluated and considered as load 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., $\frac{(P - P_a)}{FOS}$ 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.

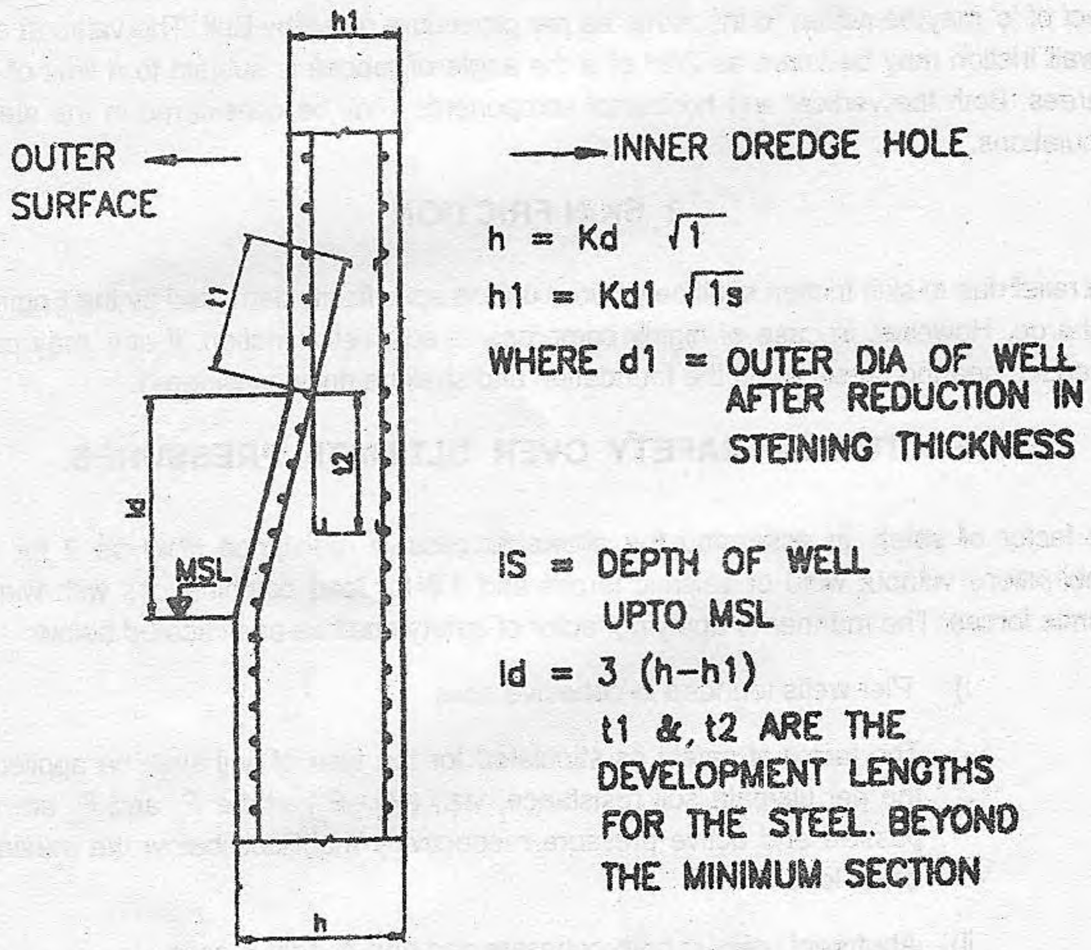


Fig. 1 Sketch for Reduction of Steining Thickness

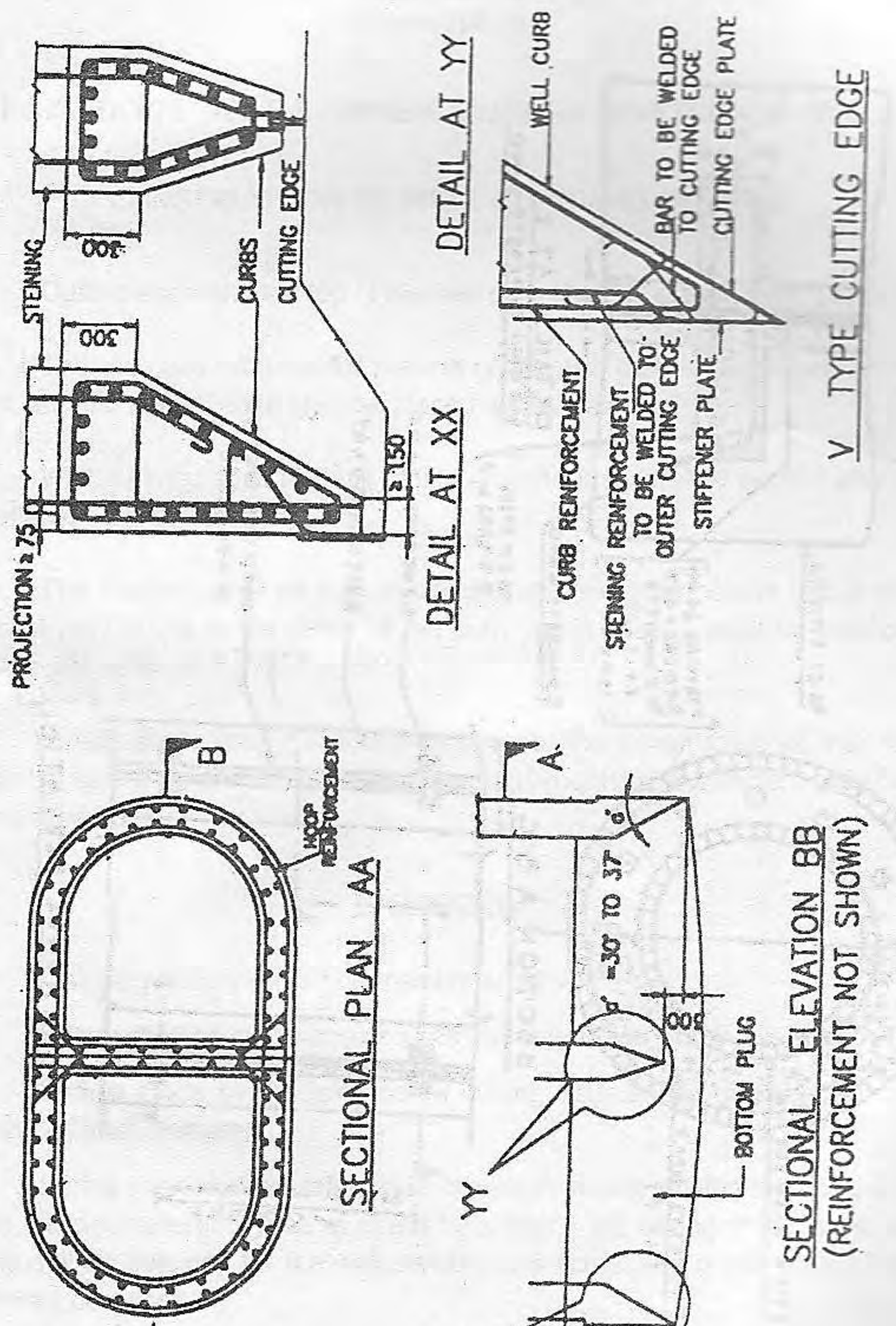


Fig. 2 Details of Well Curbs and Cutting Edge

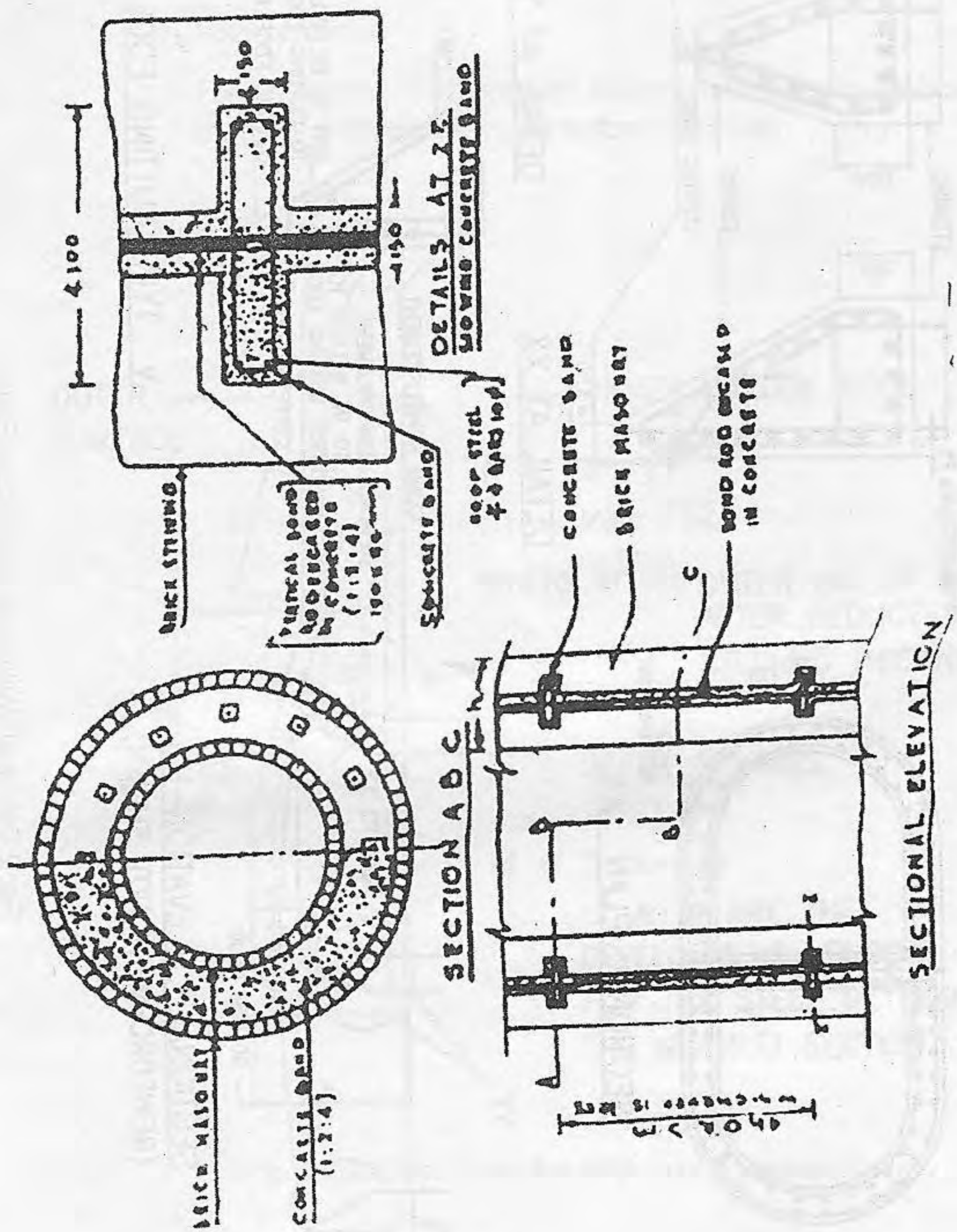


Fig. 3 Details of Well Curbs and Cutting Edge

Appendix-4
(Clause **708.13**)

PRECAUTIONS TO BE TAKEN DURING SINKING OF WELLS

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 to ensure that they do not fail 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 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 minimized 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 armored 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 divers conforms to the appropriate rate as laid down in the regulation.

5.5 All men employed for diving purpose 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 by 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 m and the spacing of holes may normally be kept 0.5 to 0.6 m.

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 oozing 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 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 heave 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 m 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 then three times the dimension in plan of the well or caisson.

Appendix-5

(Clause **709.3.1**)

CAPACITY OF PILE BASED ON PILE SOIL INTERACTION

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 = R_u, i.e., Ultimate base resistance may be calculating from the following:

$$R_u = A_p \left(\tfrac{1}{2} D Y N_y + P_d N_q\right) + A_p N_c C_p$$

where

A_p = Cross-sectional area of base of pile

D = Pile diameter in cm

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

N_q & N_y = 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 untrained 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 layer ith where *i* varies from *I* 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 ith layer, where *i* varies from *1* to *n*

A_s = Surface area of pile shaft in cm^2

α = Reduction factor

c = Average cohesion in kg/cm^2 throughout the embedded length of pile (from unconsolidated untrained 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 up to 1.8 in particular cases as specified in Clause **709.2.2 (v).**

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

Consistency	**N Value**	**Bored piles Cast-in-situ**	**Driven case-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

$$Q_u = q_b A_p + f_s . A_s$$

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:

Type of Soil	*Side Friction, f_s*
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 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 INTERMEDIATE GEO-MATERIAL AND ROCK

9.1 Axial Load Carrying Capacity :

Piles in rocks and weathered rocks of varying degree of weathering derive their capacity by end bearing and socket side resistance. The ultimate load carrying capacity may be calculated from one of the two approaches given below:

Where cores of the rock can be taken and unconfined compressive strength directly established using standard method of testing, the approach described in method 1 shall be used. In situations where strata is highly fragmented, where RQD is nil or (CR+RQD)/2 is less than 30 percent, or where strata is not classified as a granular or clayey soil, or when the crushing strength is less than 10 MPa, the approach described in method 2 shall be used. Also, for weak rock like chalk, mud stone, clay stone, shale and other intermediate rocks, method 2 is applicable.

MEHTOD 1:

$$Q_u = R_e + R_{af} = K_{sp}.q_c\ d_f.\ A_b + A_s\ C_{us}$$

$$Q\ allow = (R_e/3) + (R_{af}/6)$$

Where,

Q_u = Ultimate capacity of pile socketed into rock in Newtons

Q allow = Allowable capacity of Pile

R_e = Ultimate end bearing

R_{af} = Ultimate side socket shear

K_sp = An empiricalco-efficient whose value ranges from 0.3 to 1.2 as per the table below for the rocks where core recovery is reported, and cores tested for uniaxial compressive strength.

(CR + RQD 2	K_{sp}
30%	0.3
100%	1.2

CR = Core Recovery in percent

RQD = Rock Quality Designation in percent

For Intermediate values, K_sp shall be linearly interpolated

q_c = Average unconfined compressive strength of rock core below base of pile for the depth twice the diameter/least lateral dimension of pile in MPa

A_b	=	Cross-sectional area of base of pile
d_f	=	Depth factor = $1 + 0.4 \times \frac{\text{Length of Socket}}{\text{Diameter of Socket}}$
		However, value of d_f should not be taken more than 1.2.
A_s	=	Surface area of socket
c_{us}	=	Ultimate shear strength of rock along socket length,
	=	$0.225 \sqrt{q_c}$, but restricted to shear capacity of concrete of the pile, to be taken as 3.0 MPa for M 35 concrete in confined coudition, which for other strength of concrete can be modified by a factor $\sqrt{(fck/35)}$

METHOD 2:

This method is applicable when cores and/or core testing result are not available, or when geo-material is highly fragmented. The shear strength of geo-material is obtained from its correlation with extrapolated SPT values for 300 mm of penetration as given in table below:

Shear Strength/Consistency	**Moderately Weak**	**Weak**	**Very Weak**
Approx. N Value	300-200	200-100	100-60
Shear Strength/Cohesion in MPa	3.3-1.9	1.9-0.7	0.7-0.4

$$Q_u = R_e + R_{af} = C_{u.b} N_c A_b + C_{u.s} A_s$$

$$Q_{allow} = (Re/3) + (R_{af}/6)$$

where

$C_{u.b}$ = Average shear strength below base of pile, for the depth equal to twice the diameter/least lateral dimension of pile, *based on average 'N' value of this region*

C_{us} = Ultimate shear strength along socket length, to be obtained from table, based on average 'N' value of socket portion. This shall be restricted to shear capacity of concrete of the pile, to be taken as 3.0 MPa for M 35 concrete in confined condition, which for other strengths of concrete can be modified by a factor $\sqrt{(fck/35)}$ Intermediate values c_{ub} and c_{us} can be interpolated linearly.

L = Length of socket.

N_c = 9.

Q_{allow} = Allowable capacity of pile.

The extrapolated values of 'N' greater than 300 shall be limited to 300 while using this method.

General notes common to Method 1 and Method 2:

1) For the hinged piles resting on rock proper seating has to be ensured. The minimum socket length should be 300 mm in hard rock, and 0.5 times the diameter of the pile

in weathered rock.

2) The allowable end bearing component after dividing by factor of safety shall be restricted to 5 MPa.

3) For calculation of socket friction capacity, the top rock 300 mm depth of rock shall be neglected. The friction capacity shall be further limited to depth of six times diameter of pile.

4) For the termination of working piles in the rocky strata methodology given in sub-clause 10 can be used as a quality control tool.

9.2 Moment Carrying Capacity of Socketed Piles :

For the socketed pile, the socket length in the rock may be calculated from following equation:

$$L_s = \frac{2H}{\sigma_1 D} + \sqrt{\frac{4H^2}{\sigma_1^2 D^2} + \frac{6M}{\sigma_1 D}}$$

where

L_s = Socket length

H = Horizontal force at top of the socket

M = Moment at the top of the socket

D = Diameter of the pile.

σ_1 = Permissible compressive strength in rock which is leaser of 30 kg/sq cm or $0.33q_c$.

In case of socketed piles, for the satisfactory performance of the socket as fixed tip, the rotation at the top of the socket for the fixed condition (θ) should be less than or equal to 5 percent of the rotation for the pinned condition at the top of the socket (θ)

10 PILE TERMINATION CRITERIA AS A QUALITY CONTROL TOOL IN ROCKS

For establishing the similarity of soil strata actually met while advancing the pile-bore with the strata selected for terminating the pile on the basis of N values equivalent energy method can be used.

The concept of Pile Penetration Ratio (PPR) is used in this method.

The pile penetration ratio (PPR) reflects the energy in ton-meter required to advance the pile bore of one sq. meter cross-sectional area by 1 cm.

1) In case of SPT test its PPR can be worked out as follows:

Energy E spent for N blows = 63.5 kg x 75 cm x N blows (in kg - cm units) = E x 10^{-5} ton meter. Area of samples is 0.758 x $(5.2)^2$ sq. cm = 21.24 sq.cm, penetrating 30 cm.

Hence PPR = 63.5 x 75 x N x 10^{-5}/ (21.24x10^{-4}x30) =0.747 N

PPR for N = 50 = 37.35 tm/m^2/cm

and for N = 200 =149.4 tm/m^2/cm

where

tm = energy

m^2 = area

cm = penetration

2) PPR (P), (for percussion piles) = $\dfrac{W\,h\,n}{A\,P}$

W = Weight of chisel in MT

h = Fall of chisel in 'm'

n = Number or blows of hammer

A = Area of pile in 'm^2'

p = Penetration in 'cm'

3) PPR (R), (for rotary piles) = $\dfrac{2\pi\,nTt}{AP}$

where

N = Revolution per minute

T = Torque in 'tm' for corresponding 'n'

t = time in minutes

A = Area of piles in 'm^2'

P = Penetration in 'cm'

Appendix-6
(Clause **710.1.4**)

FILLING BEHIND ABUTMENTS, WING AND RETURN WALLS

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 percent**	**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.

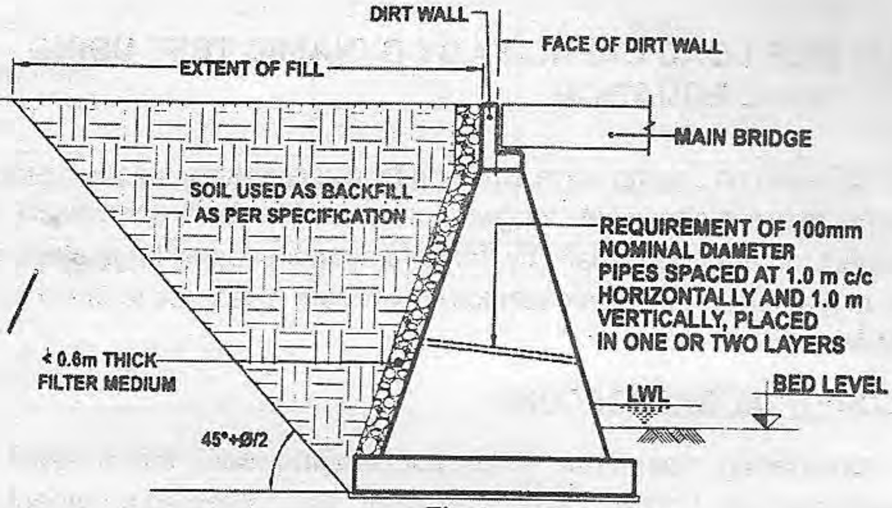


Fig. 1

Notes

1. Active wedge of soil mobilized in developing active pressure has to be filled by selected earth.
2. In case, projection of footing towards earth-fill is less than 600 mm, the filter shall be supported near the top of the footing.

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 along with 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 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.

Appendix-7
(Clause - **709.2.4**)

PART-1

METHOD-1: PILE LOAD CAPACITY BY DYNAMIC TEST USING WAVE EQUATION

This method is based on solving wave equation by using idealized model using strata wise soil parameters to arrive at pile 'set' for given pile load. The force and velocity response of pile to an impact force applied axially by drop of hammer causing large strain at top of pile (of the order of magnitude of ultimate capacity of pile) are measured to arrive at the ultimate capacity of pile.

1 THEORETICAL BACKGROUND

For piles considering resistance from surrounding soil, the internal force and displacements produced on segment of prismatic bar subjected to impact at one end, the wave equation can be derived as

$$\frac{\partial^2 D}{\partial t^2} = \left(\frac{E}{P}\right)\left(\frac{\partial^2 D}{\partial \chi^2}\right) \pm R$$

where

D = longitudinal displacement of a point of the bar from its original position

E = modulus of elasticity of bar p = density of bar material + t = time

x = direction of longitudinal axis & R = soil-resistance term

The above equation may be solved for appropriate boundary conditions and the relationship among displacement, time, position in the pile and stress are determined usually by numerical methods. Solution requires idealization of model considering load deformation diagram for each segment, the 'quake' i.e. maximum deformation that can occur elastically, 'Ru' the ultimate soil resistance and the damping factor. The empirical value of 'quake', damping factor and percent side adhesion as reported by Forehand and Reese are reproduced for reference in **Table 1.**

Table 1 Empirical Values of Q,J, and Percent Side Adhesion

Soil	Q (in.)	J(P) (Sec/ft.)	Side Adhesion (% of R_u)
Coarse sand	0.10	0.15	35
Sand gravel mixed	0.10	0.15	75-100
Fine Sand	0.15	0.15	100
Sand and clay or loam, at least 50 percent of pile in sand	0.20	0.20	25
Silt and fine sand underlain by hard strata	0.20	0.20	40
Sand and gravel underlain by hard strata	0.15	0.15	25

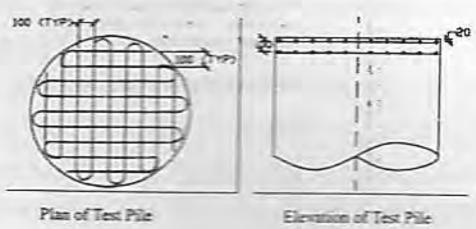
2 PILE AND TEST PREPARATION

1) The testing should be conducted by fixing instrumentation that should include strain sensors and accelerometers to the sides of the pile at a depth of 1.5 x pile diameters from top of pile and then connecting them to the measuring equipment.

2) For this it is desirable that the pile is extended to suitable length after chipping top loose concrete. This can be done either using formwork or permanent casing. Alternatively if it is a liner pile, two openings/windows approximately 300 mm x 300 mm and diametrically opposite to each other shall be made into the liner at 1.5 x pile diameter from top.

3) In case pile head is extended, it shall be axial, flat and have same strength as pile concrete. The pile head may even be one grade higher so as to attain early strength. The rebars and helical reinforcement shall also be extended to avoid cracking of concrete under hammer impact.

Refer to Fig.1 for a sketch of reinforcement in the extended pile and the diameter of bars shall generally be the same as pile reinforcement. Further, the concrete at the sensor level shall be smooth hard and uniform.



1. All dimensions are in mm
2. Clear cover to mesh reinforcement is 20 mm.
3. Diameter of mesh reinforcement bars is 8 mm.
4. Spacing of mesh reinforcement bars is 100 mm.

Fig. 1: Details of Rebar cage for extended portion of pile for dynamic test

4) A pile top cushion consisting of sheets of plywood with total thickness between 25 mm to 100 mm or as determined by the Test Engineer shall be placed on the top of the pile before testing.

5) Steel helmet 25 mm-50 mm thick or as determined by the Test Engineer shall be kept ready at the time of testing.

6) A hammer of suitable weight (1-2 percent of test load or 5-7 percent of the dead weight of the pile whichever is higher) shall be used for testing the pile unless specified otherwise by the Test Engineer. The fall height generally varies from 0.5 m to 3 m.

7) Wherever essential, a suitable guide shall be provided to ensure a concentric fall.

8) A suitable crane or equivalent mechanism capable of freely falling the required hammer shall be arranged on site in consultation with the test engineer. Refer to Fig. 2 showing the setup arrangements.

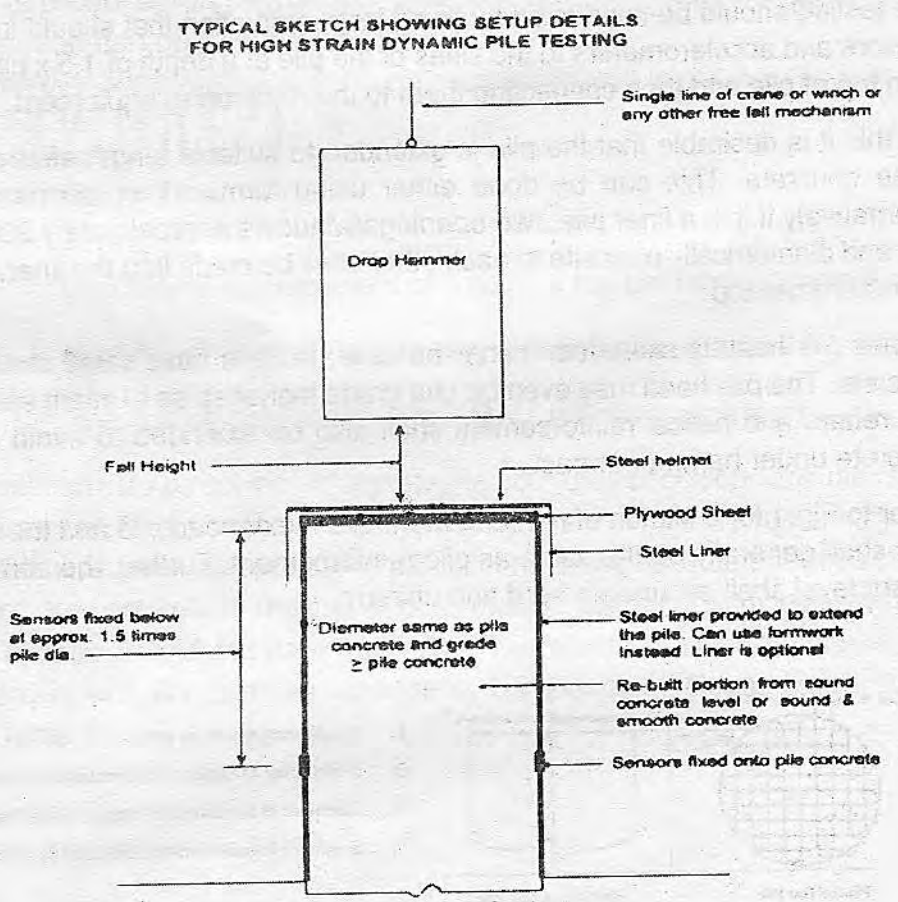


Fig. 2: Typical sketch showing setup details for high strain dynamic pile testing

9) A suitable power source supply shall be provided for fixing sensors and for the test equipment.

3) **PILE MONITORING**

1) The testing may be conducted atleast 15 days after the pile is installed and the concrete pile as well as extended portion if any has achieved the required strength.

2) Dynamic pile testing (based on wave equation) should be conducted by attaching strain transducers and accelerometers to the sides of the pile, approximately 1.5 times pile diameter below the pile top. A pair of transducers / needs to be fixed onto opposite sides of the pile, so as to detect bending in/ the pile if any, during testing.

3) These transducers should be then connected through the cable to measuring equipment to record strain and acceleration measurements and display them on an oscilloscope or screen.

4) The testing should be conducted by impacting the pile with blows of the hammer, generally starting with a smaller drop of 0.5 m. For each hammer blow, the strain transducers should measure strains whereas accelerations are measured by accelerometers connected on either sides of the pile. These signals are then converted to digital form by the equipment and then converted to force and velocity respectively by integration.

5) For each hammer blow, the test system should display immediate field results in the form of the mobilized capacity of the pile, pile top compression, integrity, stresses etc. The force and velocity curve shall be generally as defined in ASTM D4945.

6) Testing should be continued by increasing height fall of the hammer by approximately 0.5 m increment till the time either the pile set or the pile capacity reaches the required limiting values. A typical force velocity response is also described in Fig.3.

7) The pile capacity shall be generally considered to be fully mobilized if the energy levels due to hammer impact are sufficient so as to cause a measurable net displacement of atleast 3-4 mm per blow for a minimum three successive impacts. If the pile set is less than 3-4 mm per blow and the pile achieves required capacity, then it implies that not all the static pile resistance has been mobilized and that the pile still has some capacity that could not be measured or was not required to be measured at the time of testing.

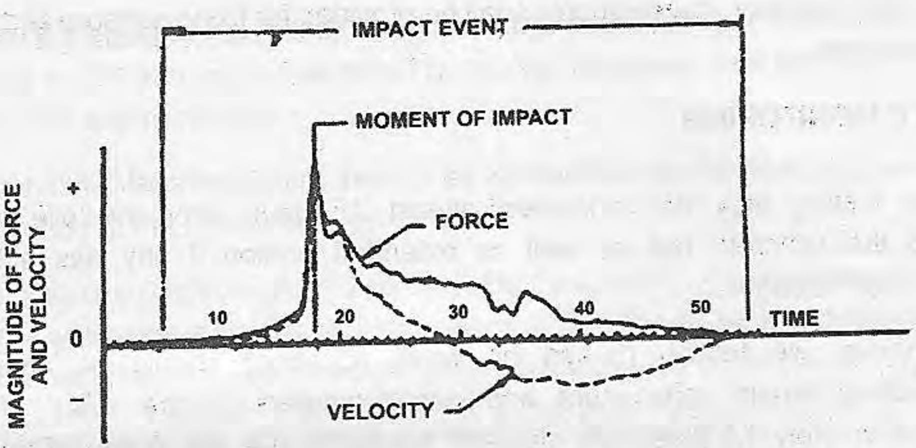


Fig. 3 : Typical force-velocity trace generated by measuring equipment

8) Analysis and Interpretation : Using strain and acceleration data in a suitable model, based on local parameters of pile and soil strata, the equivalent static load bearing capacity shall be calculated. The final report should specffy the parameters of soil and pile strata considered and the safe capacity arrived. Atypical blow is then selected for Signal Matching Analysis.

4) TEST LIMITATIONS

1) Evaluation of static soil resistance and it's distribution can be based on a variety of analytical methods and is the subject of individual engineering judgment. The input into the analytical methods may or may not result in the dynamic evaluation matching static load test data. It is necessary to calibrate the result of the dynamic analysis with those of a static pile load test carried out according to IS 2911.

2) Based on above, it can be said that it is difficult to predict rock socket friction and actual end bearing for rock socketed piles that do not show substantial net displacement under the impacts.

3) Unlike static testing, evaluation of dynamic pile test results requires an experienced engineer trained in interpretations of the results.

METHOD-2: PILE DYNAMIC TEST METHOD BASED ON HILEY'S FORMULAE

(BY LASER/INFRARED OPERATED EQUIPMENT)

1 INTRODUCTION

Since the early days of driven piles, the termination criteria based on "Sets observed", are followed. Various formulae are available. The IS Code 2911 Part - 1 covering driven piles provides one such formula. The principle followed is recording the penetration per blow of the hammer, and on that basis having obtained the desired set i.e. average penetration of standard numbers of blows of hammer, the ultimate capacity for the pile is worked out and then with suitable factor of safety the safe capacity is arrived at. The bored cast-in-situ piles after attaining strength (i.e. after curing) can be treated as precast pile to be advanced further in the founding strata (i.e. strata on which terminated) by dynamic impact energy. The load carrying capacity of bored cast-in-situ pile subjected to impact energy can then be estimated on measuring consequent displacement by sophisticated optoelectronic instruments on resorting to IS 2911 procedure. The procedure will help in ascertaining the quality of workmanship on a large number of piles without much of time wasting and avoiding delays in a construction activity with relatively less cost.

METHODOLOGY

The methodology of test is based on a large falling weight giving the dynamic impact to the elastic body. It equates the energy of hammer blow to work done in overcoming the resistance of the founding strata to the penetration of the ordinary cast-in-situ piles as well as grouted micro piles. Allowance is made for losses of energy due to the elastic compression of the pile, and subsoil as well as losses caused by the impact of the pile. The Modified Hiley's formulae given in the code IS 2911 Part -1, Section I are used in estimating the ultimate driving resistance in tonnes. Applying the factor of safety as outlined in the code, the safe load on pile can be worked out.

The instantaneous displacements including rebounds of the pile are precisely recorded in automatic data acquisition system. This is done for several cycles and then using formulae as accepted in IS Code 2911 the safe loading capacity is calculated. The opto-electronic instrument is used for position sensitive measurement by non contact continuous measurement, using instrument placed away from the vibrations due to impact load. The system is based on combined light emitting diode transmitters and a position sensitive detector The transmitter and receiver are installed so that the infrared light beam forms a reference line from transmitter, receiver to the prism group reflectors. The reflected light is received and recorded 100 times per second. Using the energy transmitted to the pile and accounting for temporary compression of pile, ground and dolly occurring during the impact loading, the ultimate driving resistance is calculated. Applying the factor of safety the safe load for the pile is calculated.

The modified Hiley formula is :

$$R = \frac{Wh\eta}{S+C/2}$$

where

R = ultimate driving resistance in tonnes. The safe load shall be worked out by dividing it with a factor of safety of 2.5.

W = mass of the ram in tonnes;

h = height of the free fall of the ram or hammer in cm, taken at its full value for trigger-operated drop hammers, 80 percent of the fall of normally proportioned winch operated drop hammers, and 90 percent of the stroke for single-acting hammers. When using the McKiernanTerry type of double acting hammers, 90 percent of the maker's rated energy in tonne-centimeter per blow should be substituted for the product (Wh) in the formula. The hammer should be operated at its maximum speed while the set is being taken;

η = efficiency of the blow, representing the ratio of energy after impact to the striking energy of ram;

S = final set or penetration per blow in cm;

C = sum of the temporary elastic compressions in cm of the pile, dolly, packings and ground

P = Mass of pile in tonnes

Where W is greater than Pe and the pile is driven into penetrable ground,

$$\eta = \frac{W + Pe^2}{W + P}$$

Where W is less than Pe and the pile is driven into penetrable ground.

$$\eta = \frac{W + Pe^2}{W + P} - \left(\frac{W - Pe}{W + P}\right)^2$$

The following are the values of η in relation to e and to the ration of P/W

Ratio of P/W	e = 0.5	e = 0.4	E = 0.32	e = 0.25	e = 0
1/2	0.75	0.72	0.70	0.69	0.67
1	0.63	0,58	0.55	0.53	0.50
$1\,^{1}/_{2}$	0.55	0.50	0.47	0.44	0.40
2	0.50	0.44	0.40	0.37	0.33
$2^{1}/_{2}$	0.45	0.40	0.36	0.33	0.28
3	0.42	0.36	0.33	0.30	0.25
$3^{1}/_{2}$	0.39	0.33	0.30	0.27	0.22
4	0.36	0.31	0.28	0.25	0.20
5	0.31	0.27	0.24	0.21	0.16
6	0.27	0.24	0.21	0.19	0.14
7	0.24	0.21	0.19	0.17	0.12
8	0.22	0.20	0.17	0.15	0.11

P is the weight of the pile, anvil, helmet and follower (if any) in tonnes.

Where the pile finds refusal in rock, 0.5 *P* should be substituted for *P* in the above expression for η.

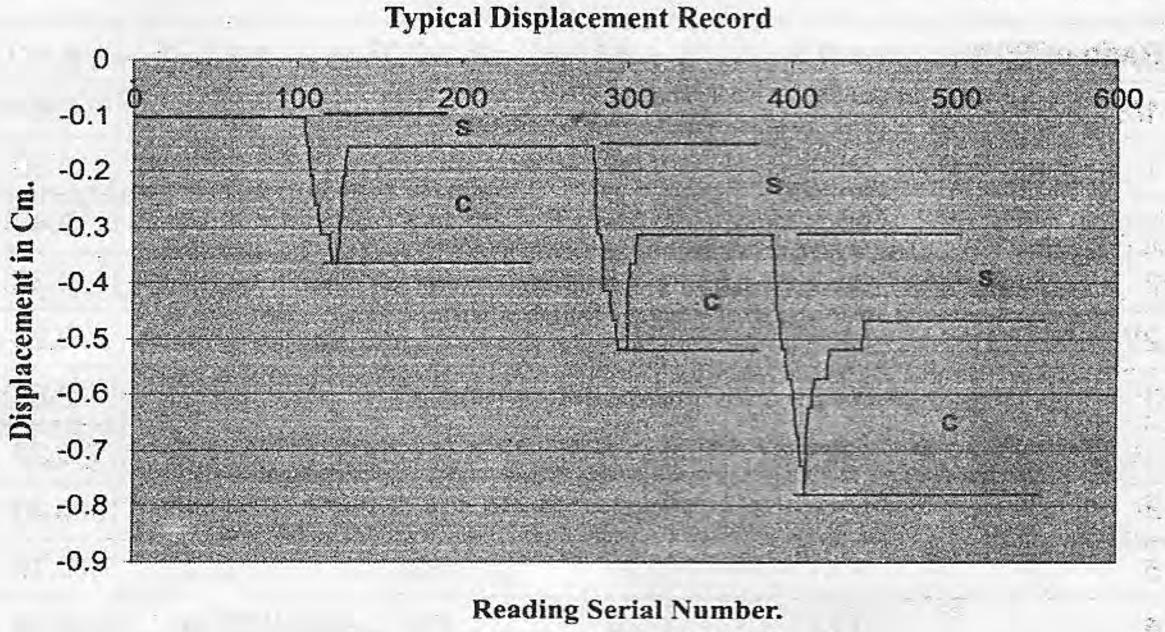
e is the coefficient of restitution of the materials under impact as tabulated below.

a) For steel ram of double-acting hammer striking on steel anvil and driving reinforced pile, *e* = 0.5

b) For cast-iron ram of single-acting or drop hammer striking on head of reinforced concrete pile, e = 0.4.

c) Single-acting or drop hammer striking a well-conditioned driving cap and helmet with hard wood dolly in driving reinforced concrete piles or directly on head of timber pile, e = 0.25.

Numbers of models of Laser/infrared operated instruments measuring accurately the deformation are available these days. The required sensitivity of the equipment shall be such as to read the angular deformation to the accuracy of 10-3 radians and the instrument should be capable of recording about 100 readings per second. From the angular deformation, on knowing the distance of the reflector from the instrument,



vertical movement of the shaft under the given impact energy, (both elastic and permanent) can be measured accurately. These measurements of the displacement can then be substituted in modified Hiley's formulae stated in IS 2911. The ultimate load carrying capacity of the pile can be worked out, resorting to the modified Hiley's formulae outlined in the code and from that the safe load carrying capacity of pile can be estimated.

Appendix-7

PART-2

STANDARD TEST METHOD FOR LOW STRAIN PILE INTEGRITY TESTING

1 SIGNIFICANCE AND USE

Pile Integrity Testing (PIT) is a Non-Destructive integrity test method for foundation piles. The method evaluates continuity of the pile shaft and provides information on any potential defects due to honeycombs, necking, cross-section reduction, potential bulbs, sudden changes in soil stratum, concrete quality in terms of wave speed etc. It is known as "Low Strain" Method since it requires the impact of only a small hand-held hammer and the resultant strains are of extremely low magnitude. The test procedure is standardized as per ASTM D5882 and also forms part of various specifications and code provisions worldwide as indicated in **Table-3.** The number of tests shall be decided by the engineer to the project.

Table 3 Major Standards or Codes for Integrity Testing

Sr no	Method	Country	Reference	Title
1)	LST, CSL	Australia	Australian Standard As 21 59-1 995	Pile Designing and installation
2)	LST, CSL	China	JGJ 94-94	Technical code for Building Piles foundations Chapter 9: Inspection and Acceptance of pile Foundation Engineering. 9.1 Quality Inspection of Pile installation.
3)	LST.CSL	China	JGJ 93-95	Specification for Low Strain Dynamic Testing of Piles.
4)	LST	France	Norms Francaiso	Soil investigation and testing ascnliation of buried work method by reflection impedance
5)	LST.CSL	Germany	DGGT	Empfohlung Integritataprufungen -
6)	LST,CSL	UK	Institution of Civil Engineers. (ICE)	Specification of Piling.
7)	LST	USA	ASTM D5882	Standard specification for Low Strain Imaginary Testing of Piles.

2 TYPICAL METHODS

Various methods used to evaluate the integrity of the pile are briefly described below. The evaluation of PIT records can be described as follows.

- Pulse-Echo Method (or Sonic Echo - a time domain analysis)
- Force Velocity Approach
- Transient Response Procedure (Frequency Domain Analysis)
- Cross Hole Sonic Logging

The Force Velocity approach is sometimes used to evaluate defects near the pile top that maybe difficult to evaluate in Pulse Echo Method. The Pulse Echo is the most commonly used method and is described below.

3 TEST EQUIPMENT

The test should be performed using digital data acquisition equipment like a Pile Integrity Tester or any equivalent that meets ASTM D5882 requirements. The equipment must also include a sensitive accelerometer, instrumented or non-instrumented hammer etc. The data must be displayed in the field for evaluations of preliminary data quality and interpretation and a field printout should be possible.

4 TEST PROCEDURE

The testing shall be conducted at least 7 days after pile concreting by an experienced engineer/technician. The concrete at the pile top surface must be relatively smooth with sufficient space for attachment of the motion sensing device and hammer impact area. The testing involves attachment of an accelerometer onto the pile top (not near its edge) with the help of bonding material like candle wax, vaseline etc. After attachment, the pile is impacted with a hand-held device (a hand-held hammer).

The test involves collection of several blows during the stage of testing. All such similar blows are averaged before display. For larger diameter piles of 600 mm and above testing may be conducted on at least 3 locations, whereas minimum one location is enough for smaller diameter piles. The typical data sets for good or damaged pile shall generally be as per **Fig.1** and is also defined in ASTM D5882.

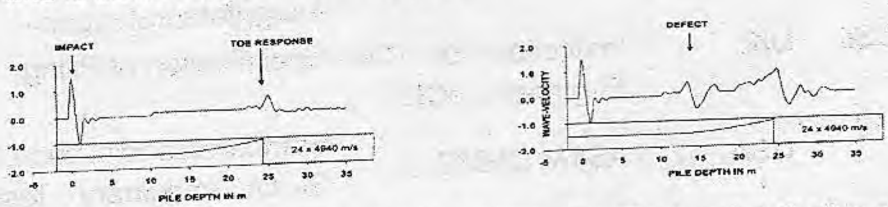


Fig. 1: PIT Velocity versus Depth plot for normal pile (Top) and defective pile (bottom)

5 REPORT SUBMISSION

The final report shall include the following:

1) Project Identification & Location
2) Test Pile Identification including Length, Nominal Cross-Sectional area and Concrete Mix as per installation record
3) Type of Pile and description of special installation procedures used if any.
4) Description of all the components of the apparatus for obtaining integrity measurements and recording/displaying data.
5) Graphical representation of Velocity measurements in time domain.
6) Comments on the quality of the Pile Concrete.
7) Comments on any potential defects/damage and its location.
8) Comments on Integrity of Pile based on above.

6 LIMITATIONS

Certain limitations are inherent in the low strain test method and hence it should be treated only as an indicator of quality of work and not as a conclusive test. The limitations mentioned below must be understood and taken into consideration in making the final integrity evaluation.

1) Integrity evaluation of a pile section below a crack that crosses the entire pile cross-sectional area or a manufactured mechanical joint is normally not possible since the impact wave likely will reflect completely at the discontinuity.
2) Piles with highly variable cross-sections or multiple discontinuities maybe difficult to evaluate.
3) The method is intended to detect major anomalies and minor defects may not be detected by this method.
4) The test is not applicable to jointed pre-cast piles or hollow steel pipes or H-sections.
5) The method cannot be used to derive the pile capacity.

(The Official amendments to this document would be published by the IRC in its periodical, 'Indian Highways' which shall be considered as effective and as part of the code/guidelines/manual, etc. from the date specified threin)