SP 36-2 (1988): Compendium of Indian Standards on Soil Engineering: Part-2 Field Testing of Soils For Civil Engineering Purposes [CED 43: Soil and Foundation Engineering]

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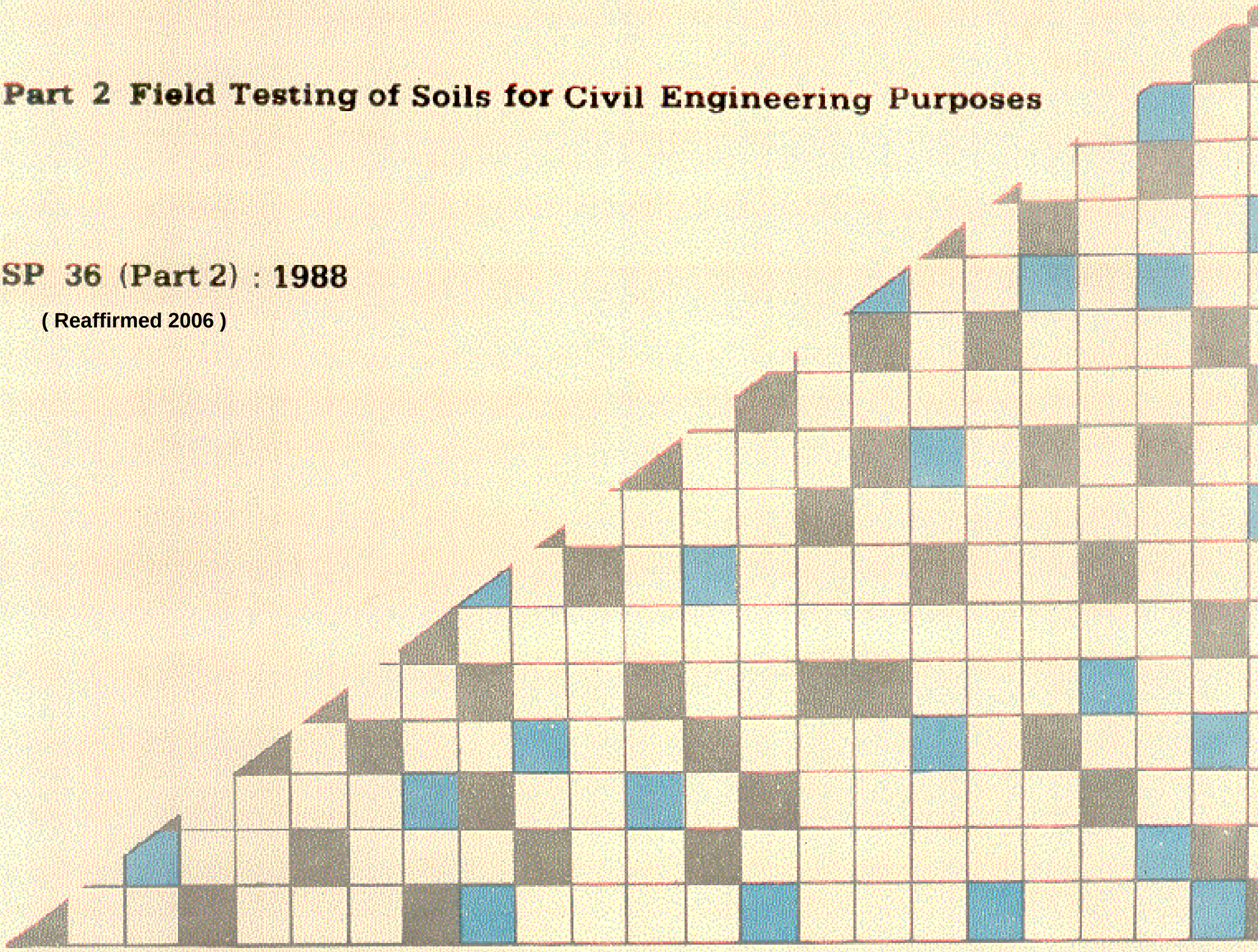
COMPENDIUM OF INDIAN STANDARDS ON SOIL ENGINEERING

Part 2 Field Testing of Soils for Civil Engineering Purposes



SP 36 (Part 2) : 1988

(Reaffirmed 2006)

BUREAU OF INDIAN STANDARDS

COMPENDIUM OF INDIAN STANDARDS ON SOIL ENGINEERING

PART 2

BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI 110002

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PREFACE

The Soil Engineering Sectional Committee of the Bureau of Indian Standards has so far formulated 84 Indian Standards which relate to classification, glossary, subsurface investigation, methods of laboratory testing, methods of field testing, soil testing equipment and soil based product. However, these are not serially numbered nor the Indian Standards belonging to specific area of soil engineering are grouped together. It may be difficult for the user to know about the full availability of Indian Standards. The Committee, therefore, recommended the publication of a compendium of all the Indian Standards formulated by it and this has resulted in this publication.

In order to establish uniform procedure for the determination of different characteristics of soils and also for facilitating comparative studies of the results, the Bureau of Indian Standards has brought out Indian Standards on methods of test for soils both for laboratory and field. It has also been recognized that reliable and comparable test results can be obtained only with standard testing equipment capable of giving the desired level of accuracy. The Bureau is, therefore, bringing out series of Indian Standards covering specification of equipments used for testing of soils so as to encourage the development and their manufacture in the country. All such Indian Standards published so far have also been included in this compendium.

For convenience of reference and use, this compendium is being brought out in two parts. The first part covers laboratory testing of soils for civil engineering purposes and this second part covers standards on field testing, sampling and products. The Indian Standards in each part have been arranged subjectwise details of which have been indicated in the contents. An index of Indian Standards arranged serialwise has also been provided for easy location of any Indian Standard covered in this compendium.

In reporting the result of test or analysis made in accordance with any of the Indian Standards, if the final value, observed or calculated, is to be rounded off, it should be done in accordance with IS 2 : 1960.

This publication covers Indian Standards issued up to 31 March 1988 and incorporates at the appropriate place all the amendments issued up to that time.

List of standards referred in this compendium is given in Annex A at the end.

CONTENTS

INDEX
(According to Serial Order of IS No.)

SECTION 1

Subsurface Investigation for Foundations

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Indian Standard
CODE OF PRACTICE FOR SUBSURFACE INVESTIGATION FOR FOUNDATIONS

(First Revision)

0. FOREWORD

0.1 A detailed investigation for site is essential before a design can be finalized. The object of subsurface and related site investigation is to provide the engineer or architect with as much information as possible about the existing conditions, for example, the exposed overburden, the course of a stream nearby, a rock outcrop or a hillock, vegetation, and other geological features of the area. It is equally important to know the subsoil conditions below a proposed structure.

0.1.1 The methods of subsurface investigation enable vertical sections of the strata to be drawn and samples to be tested, on the site or in a laboratory for determining shear strength parameters, bearing capacity of the soil, permeability, water table, type classification and other geophysical information in the field. This information, together with the normal topographical survey, provides the engineer with complete details of the site and enables him to prepare economical designs for the foundations.

0.1.2 Proper inspection and guidance in boring operations and investigations are essential to ensure that the required data are obtained.

0.2 Because of the complexity of natural deposits, no one method of exploration is best for all situations. The choice depends upon the nature of the material and on the purpose of the exploratory programme. This code is intended to summarize in a convenient form the information available so that the desirable information may be obtained. The code has been prepared in relation to conditions and practices existing in India. This standard was published in 1962. Based on further data collected in past 18 years, this revision has been prepared.

0.3 Though this code is mainly intended to cover subsurface investigation for foundations of multi-storeyed buildings, most of the provisions are generally applicable to other civil engineering works, such as roads, air fields, bridges and marine works.

1. SCOPE

1.1 This code deals mainly with subsurface investigations for foundations of multi-storeyed buildings to determine:

a) sequence and extent of each soil and rock stratum in the region likely to be affected by the proposed work;

b) nature of each stratum and engineering properties of soil and rock which may affect design, and mode of construction of proposed structures and their foundations; and

c) location of ground water, and possible corrosive effects of soil and water on foundation materials.

1.1.1 Aspects relating to procuring representative samples of the soils and rocks, obtaining general information on geology, seismicity of the area, surface drainage, etc, and subsurface investigations for availability of construction materials are also mentioned briefly.

1.1.2 Most of the provisions of this code are also applicable to subsurface investigation of underground and overhead water tanks, swimming pools and (abutments of) bridges, roads and air fields.

2. GENERAL

2.1 In areas which have already been developed advantage should be taken of existing local knowledge, records of trial pits, bore holes, etc., in the vicinity, and the behaviour of existing structures, particularly those of a nature similar to that of the proposed structure. In such cases, exploration may be limited to checking that the expected soil conditions are those as in the neighbourhood.

2.2 If the existing information is not sufficient or is inconclusive, the site should be explored in detail so as to obtain a knowledge of the type, uniformity, consistence, thickness, sequence and dip of the strata, and of the ground water conditions.

2.2.1 *Site Reconnaissance* — Site reconnaissance would help in deciding future programme of field investigations, that is, to assess the need for preliminary or detailed investigations. This would also help in determining scope of work, methods of exploration to be adopted, field tests to be carried out and administrative arrangements required for the investigation. Where detailed published information on the geotechnical conditions is not available, an inspection of site and study of topographical features are helpful in getting information about soil, rock and ground-water conditions. Site reconnaissance includes a study of local topography, excavations, ravines, quarries, escarpments; evidence of erosion or landslides, behaviour of existing structures at or near the site; water level in streams, water courses and wells; flood marks; nature of vegetation; drainage pattern, location of seeps, springs and swamps. Information on some of these may be obtained from topographical maps, geological maps, pedological and soil survey maps, and aerial photographs.

2.2.1.1 Data regarding removal of overburden by excavation, erosion or landslides should be obtained. This gives an idea of the amount of pre-consolidation the soil strata has undergone. Similarly, data regarding recent fills is also important to study the consolidation characteristics of the fill as well as the original strata.

2.2.1.2 The type of flora affords at times some indication of the nature of the soil. The extent of swamp, and superficial deposits and peats will usually be obvious. In general, such indications, while worth noting, require to be confirmed by actual exploration.

2.2.1.3 *Ground-water conditions* — The ground-water level fluctuates, and will depend upon the permeability of the strata and the head causing the water to flow. The water level in streams and water courses, if any in the neighbourhood, should be noted, but it may be misleading to take this as an indication of the depth of the water table in the ground. Wells at the site or in the vicinity give useful indications of the ground-water conditions. Flood marks of rivers may indicate former highest water levels. Tidal fluctuations may be of importance. There is also a possibility of several water tables at different levels, separated by impermeable strata, and some of this water may be subject to artesian head.

2.2.2 *Enquiries Regarding Earlier Use of the Site*—In certain cases, the earlier uses of the site may have a very important bearing on proposed new works. This is particularly so in areas where there have been underground workings, such as worked-out ballast pits, quarries, old brick fields, coal mines and mineral workings. Enquiries should be made regarding the location of shafts and workings, particularly shallow ones, where there may be danger of collapse, if heavy new structures are superimposed.

2.2.2.1 The possibility of damage to sewers, conduits and drainage systems by subsidence should also be investigated.

2.2.3 Geophysical investigations of the site may be conducted at the reconnaissance stage since it provides a simple and quick means of getting useful information about stratifications. Depending on these information, detailed subsoil exploration should be planned. Important geophysical methods available for subsoil exploration are:

a) Electrical resistivity method, and

b) Seismic method.

2.2.3.1 *Electrical resistivity method* — The electrical resistivity method, in which the resistance to the flow of an electric current through the subsurface materials is measured at intervals of the ground surface, may be useful for the study of foundation problems and particularly for finding rock strata under deep soil cover.

2.2.3.2 *Seismic method* — The seismic method makes use of the variation of elastic properties of the strata which affect the velocity of shock waves travelling through them, thus providing a usable tool for dynamic elastic moduli determinations in addition to the mapping of the subsurface horizons. The required shock waves can be generated by hammer blows on the ground or by detonating a small charge of explosives. This method is quite useful in delineating the bedrock configuration and the geological structures in the subsurface.

2.3 Outline of Procedure

2.3.1 *Number and Disposition of Trial Pits and Borings* — The disposition and spacing of the trial pits and borings should be such as to reveal any major changes in thickness, depth or properties of the strata over the base area of the structure and its immediate surroundings. The number and spacing of bore holes or trial pits will depend upon the extent of the site and the nature of structures coming on it. For a compact building site covering an area of about 0.4 hectare, one bore hole or trial pit in each corner and one in the centre should be adequate. For smaller and less important buildings even one bore hole or trial pit in the centre will suffice. For very large areas covering industrial and residential colonies, the geological nature of the terrain will help in deciding the number of bore holes or trials pits. Cone penetration tests may be performed at every 50 m by dividing the area in a grid pattern and number of bore holes or trial pits decided by examining the variation in the penetration curves. The cone penetration tests may not be possible at sites having gravelly or boulderous strata. In such cases, geophysical methods may be useful.

2.3.2 *Depth of Exploration* — The depth of exploration required depends on the type of proposed structure, its total weight, the size, shape and disposition of the loaded areas, soil profile, and the physical properties of the soil that constitutes each individual stratum. Normally, it should be one and a half times the width of the footing below foundation level. In certain cases, it may be necessary to take at least one bore hole or cone test or both to twice the width of the foundation. If a number of loaded areas are in close proximity, the effect of each is additive. In such cases, the whole of the area may be considered as loaded and exploration should be carried out up to one and a half times the lower dimension. In weak soils, the exploration should be continued to a depth at which the loads can be carried by the stratum in question without undesirable settlement and shear failure. In any case, the depth to which seasonal variations affect the soil should be regarded as the minimum depth for the exploration of sites. But where industrial processes affect the soil characteristics, this depth may be more. The presence of fast growing and water seeking trees also contributes to the weathering processes.

NOTE — Examples of fast growing and water seeking trees are Banyan (*Ficus bengalensist*), Pipal (*Ficus religiosa*) and Neem (*Azadirachta indica*).

2.3.2.1 An estimate of the variation with depth of the vertical normal stress in the soil arising from foundation loads may be made on the basis of elastic theory. The net loading intensity at any level below a foundation may be obtained approximately by assuming a spread of load of two vertical to one horizontal from all sides of the foundations, due allowance being made for the overlapping effects of load from closely spaced footings. The depth of exploration at the start of the work may be decided as given in Table 1, which may be modified as exploration proceeds, if required.

TABLE 1 DEPTH OF EXPLORATION
(*Clause* 2.3.2.1)

SL No.	TYPE OF FOUNDATION	DEPTH OF EXPLORATION (*D*)
i)	Isolated spread footing or raft	One and a half times the width (*B*) (*see* Fig. 1)
ii)	Adjacent footing with clear spacing less than twice the width	One and a half times the length (*L*) of the footing (*see* Fig 1)
iii)	Adjacent rows of footings	*See* Fig. 1
iv)	Pile and well foundations	To a depth of one and a half times the width of structure from the bearing level (toe of pile or bottom of well)
v)	1. Road cuts	Equal to the bottom width of the cut
	2. Fill	Two metres below ground level or equal to the height of the fill whichever is greater

2.4 Importance of Ground-Water Tables

2.4.1 For most types of construction, water-logged ground is undesirable because of its low bearing capacity. On sites liable to be water-logged in wet weather, it is desirable to determine the fluctuation of the water table in order to ascertain the directions of the natural drainage, and to obtain a clue to the design of intercepting drains to prevent the influx of ground water on to the site from higher ground. The seasonal variation in the level of water table should also be noted.

2.4.2 If in the eariler stages of investigations, dewatering problems are anticipated a detailed study should be carried out to ascertain the rate of flow and seepage.

2.4.3 For deep excavation, the location of water-bearing strata should be determined and the water pressure observed in each, so that necessary precautions may be taken during excavation, for example, artesian water in deep strata may give rise to considerable difficulties unless precautions are taken. An idea of the steady level of water should be obtained. Bore holes, which have been driven, may be used for this purpose, but since water levels in bore holes may not reach equilibrium for some time after boring, these should be measured 12 to 24 hours after boring and compared with water levels in wells that may be available in the area. It is seldom necessary to make detailed ground-water observations in each one of a group of closely spaced bore holes but sufficient observations should be made to establish the general shape of the ground-water table; however, observations should always be made in the first boring of the group. The minimum and maximum ground-water levels should be obtained from local sources and wells in the area would also give useful information in this regard

NOTE — For methods of determination of water level in a bore hole, IS 6935 : 1973 and for methods of determination of permeability of overburden, IS 5529 (Part 1) : 1969 may be referred.

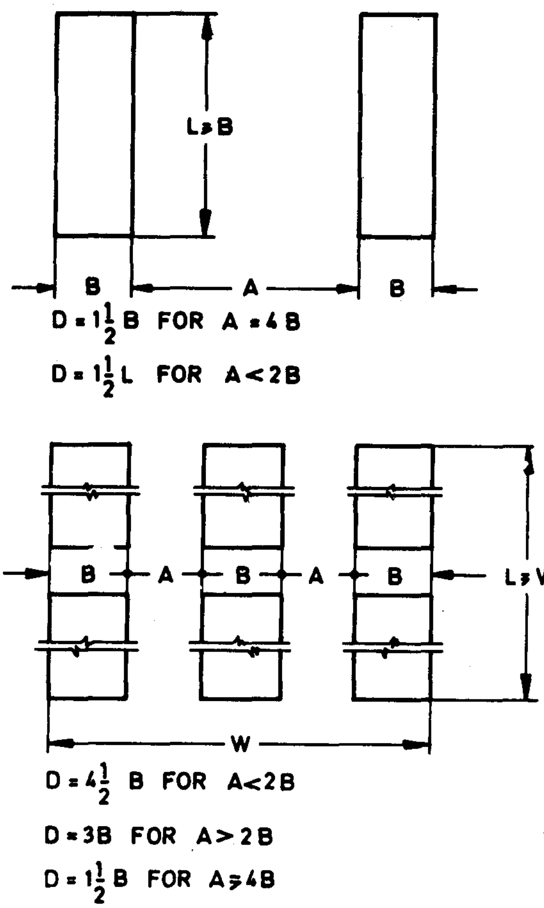


FIG. 1 DEPTH OF EXPLORATION

2.4.4 In making ground-water observations, it should be remembered that in some localities there may be one or more isolated bodies of water or perched ground-water tables above the main ground-water table. The formation of perched ground-water tables is caused by impervious strata which prevent the water from seeping down to the main body of ground water. The difference in water levels in bore holes spaced reasonably close to one another would indicate a perched water table.

2.5 Corrosive Soils, Waters and Effluents — In certain localities, ground water and soil may contain constituents in amounts sufficient to cause damage to foundations of structures. Cement concrete is liable to be attacked by water containing sulphates. Some soils have a corrosive action on metals, particularly on cast iron, due to either chemical or bacterial agency, and enquiry should be made in the locality to find if such corrosion has previously occured. In such cases, a chemical analysis of the soil should be made to assess the necessity of special precautions. Ground water may be tested for its salt content, alkalinity or acidity, etc, for its effect on foundations. For the purpose of chemical analysis about 5 litres of water should be collected from as near the bottom of the bore hole or trial pit as possible and not from the top. Water in the bore hole should be pumped out completely and after 24 hours the water which is collected in the bore hole should again be pumped out and sample of water obtained as near to the bottom as possible. This will prevent erroneous collection of water utilised in wash boring which is not the natural ground water. The water samples should be collected in plastic jerry cans filled up to the brim and should be air tight for the purpose of chemical analysis in laboratory.

2.5.1 In industrial areas, corrosive action may arise from industrial waste products that have been dumped on the site. Samples of such individual waste products should be collected and chemically analyzed in the laboratory.

2.6 Soil and Rocks — As the engineering classification deals mainly with soil as a material of construction, soil may be defined as that which comprises accumulations of solid particles, loose or cohesive deposits, such as gravel, sand, silt, clay or any combination thereof which is loose enough to be removed with a spade or shovel in a dry or saturated state. Their depths may vary from deep lying geological deposits to agricultural surface soils. Correspondingly, the term 'rock' may be applied to materials other than the above, that is, natural beds or large hard fragments or original igneous, sedimentary or metamorphic formations.

2.6.1 *Classification and Identification of Soils* — For this purpose, IS 1498 : 1970 should be referred.

3. METHODS OF SITE EXPLORATION

3.1 General — Subsurface explorations should generally be carried out in two stages, that is, preliminary and detailed.

3.1.1 *Preliminary Exploration* — The scope of preliminary exploration is restricted to the determination of depths, thickness, extent and composition of each soil stratum, location of rock and ground water, and also to obtain approximate information regarding strength in compressibility of the various strata. When reconnaissance is not possible, it is essential to carry out preliminary investigation to decide the method of approach of investigation. During preliminary investigation, geophysical methods and tests with cone penetrometres and sounding rods are useful guides.

3.1.2 *Detailed Exploration* — Detailed investigation follow preliminary investigation, and should be planned on the basis of data obtained during reconnaissance and preliminary investigations. This plan may require review as the investigations progress. The scope of detailed exploration is ordinarily restricted to the determination of engineering properties of strata which are shown by preliminary exploration to be critical. The object of detailed exploration is to determine shear strength and compressibility of all types of soils, density, density index, natural moisture content, and permeability. It may also be necessary to determine the preconsolidation pressure of the strata from oedometer tests and to determine the consolidation characteristics beyond preconsolidation pressure. Appropriate shear tests should also be conducted on samples subjected to ambient pressures beyond the preconsolidation range also. The detailed investigation includes boring programme and detailed sampling to determine these properties. Field tests which may be performed are *in-situ* vane shear tests and plate load tests. The field permeability test and the test for the determination of dynamic properties of soils may also be conducted, where necessary. More advanced methods of logging of bore holes by radioactive methods fall under the category of detailed investigations. All *in-situ* tests are to be supplemented by laboratory investigations. The various phases of currently used methods of exploration and their mode of application are indicated in Appendix A.

3.2 Geophysical Investigations — Geophysical surveys make use of differences in the physical properties like electrical conductivity and elastic moduli, density and magnetic susceptibility of geological formations in the area to investigate the subsurface. These methods may be employed to get preliminary information on stratigraphy or complement a reduced boring programme by correlation of stratigraphy between widely spaced bore holes. Of the four methods of geophysical surveys, namely, seismic, electrical, magnetic and gravity surveys; only seismic refraction and electrical resistivity surveys are widely used. Magnetic methods are occasionally used for detecting buried channels, dykes, ridges and intrusions in the subsurface rocks. A brief outline of the seismic method and electrical resistivity method is given in Appendix B.

3.3 Open Trial Pits — The method of exploring by open trial pits consists of excavating trial pits at the site and thereby exposing the subsoil surface thoroughly, enabling undisturbed samples to be taken. The undisturbed samples may be obtained by driving sharp-edged thin-wall tubes into the ground by gently hammering or pressure. Alternately, hand-cut samples known as chunk samples can be obtained. In sufficiently cohesive soil, undisturbed sample can be cut out and trimmed to regular shapes, say a cube or a cylinder. Wrap the chunk samples with plastic cloth, plastic paper (polythene film) or a waxed paper painted on the outside with molten wax. Two additional layers of cloth and wax are required to perfectly seal the sample. If a soil is easily disturbed a firmly constructed wooden box, with lid and bottom removed, is kept around the protruding sample block so as to leave a space of about 25 mm between the sample and sides of the box. The space between the sample and the sides of the box is packed with moist saw dust or similar packing material. The top of the sample is cut to size, cloth or paper and wax are applied, and then the lids are fitted. This method should generally be used for small depths (up to 3 m), but for greater depths (over 6 m) and below ground-water table, the method becomes expensive due to the expense of sheet piling or caissons which are required in such cases. Deep trial pits may be used to investigate open fissures, or to explore zones of weak rocks which would break up in the core barrel and are incapable of being recovered intact.

3.4 Subsurface Soundings — The geophysical methods of reconnaissance are sometimes supplemented by penetrometer tests by cones. The cone penetrometer, apart from its use for delineation of rock strata, may also be utilized for correlation with more detailed borings of soil characteristics like density, bearing capacity, etc. Sounding methods consist of measuring the variation in the resistance of the soil with depth by means of a penetrometer and may be conducted either by the static or the dynamic methods [*see* IS 4968 (Part 1) : 1976, IS 4968 (Part 2) : 1976 and IS 4968 (Part 3) : 1976]. The soundings by dynamic method may also be carried out in bore holes using a standard sampler. The sampler used and the procedure adopted shall be as specified in IS 2131 : 1963. The static cone penetration methods are not suitable for exploration of boulderous or gravelly strata and in very stiff cohesive soils. However, dynamic cone penetration methods may be conducted in such area to give an idea about the compactness of strata.

3.5 Exploratory Drilling — Preliminary borings by augers, either power or hand driven, are quick and economical up to a depth of about 6 m in alluvial deposits. They are difficult to operate below the water table. When detailed information is not required, wash boring with chopping and jetting may be utilized in cohesive and non-cohesive soils up to great depths. In the absence of casing, the sides of the holes, where required, should be stabilized with drilling fluid consisting of drilling mud (*see* **3.6.5.1**). In sandy soils, bentonite slurry in the bore hole should be maintained at a level of 1 to 1.5 m above the level of water table. In the wash boring method, changes in stratification can be ascertained only by the rate of progress of the drill or change in the colour of wash water or both. As the formation hardens, rotary drills, using churning bits may be utilized. In gravelly materials, percussion drilling with simultaneous advance of casing is the only easy method of advance. In hard and cemented formation like rock, the hole is advanced with cutting edges using steel shots, hardened metal bits, tungsten carbide or diamond bits.

3.6 Borings

3.6.1 *Auger Boring* — An auger may be used for boring holes to a depth of about 6 m in soft soil which can stand unsupported but it may also be used with lining tubes, if required. Mechanically operated augers are suitable for gravelly soils or where a large number of holes are to be made.

3.6.2 *Shell and Auger Boring* — A hand rig may be used for vertical boring up to 200 mm in dia and 25 m in depth. In alluvial deposits, the depth of the bore hole may be extended up to 50 m with a mechanized rig. The tool consists of augers for soft to stiff clay, shells for very stiff and hard clay and shells or sand pumps for sandy strata, attached to sectional boring rods. Small boulders and thin strata of rock may be broken up by a chisel bit attached to the boring rods. The boring rods are raised or lowered by means of shear legs and a winch, and are turned by hand. The casing is advanced by driving by means of a 'monkey' suspended from a winch.

3.6.2.1 No water shall be added while boring through soft cohesive soils and cohesionless soils above water table. Bailers should be used to remove soil cuttings. In stiff cohesive soils, it may be necessary to soak the bore hole before any progress can be made. While boring through cohesionless deposits below water table, water in the casing shall always be maintained at or above the water table. It is essential that the casing is kept full of water or with 5 percent bentonite slurry up to the top level. Special care should be taken while withdrawing the shell to avoid sand boiling. Start pouring the slurry into the casing pipe so that the sand boiling is avoided in the vacuum created by withdrawing the shell. While boring, care shall be taken to minimize disturbance to the deposits below the bottom of the bore hole. Disturbed soil of the deposits with all the constituent parts should be recovered at regular intervals or whenever there is a change of strata. These samples are suitable for conducting various identification tests in the laboratory.

3.6.3 *Percussion Boring* (*see* Fig. 2) — This method consists of breaking up of the formation by repeated blows from a bit or a chisel. Water should be added to the hole at the time of boring and the debris baled out at intervals. The bit may be suspended by a cable or rods from a walking beam or spudding device.

Where the boring is in soil or into soft rocks and provided that a sampler can be driven into them, cores may be obtained at intervals using suitable tools; but in soils, the material tends to become disturbed by the action of this method of boring and for this reason, the sample may not be as reliable as by the shell and auger method. As these machines are devised for rapid drilling by pulverizing the material, they are not suitable for careful investigation. However, this is the only method suitable for drilling bore holes in boulderous and gravelly strata.

3.6.4 *Wash Boring* (*see* Fig. 3) — In this method, water is forced under pressure through an inner tube which may be rotated or moved up and down inside a casing pipe. The lower end of the tube, fixed with sharp edge or a tool, cuts the soil which will be floated up through the casing pipe around the tube. The slurry flowing out gives an indication of the soil type. In this method, heavier particles of different soil layers remain under suspension in the casing pipe and get mixed up, and hence this method is not suitable for obtaining samples for classification. Whenever a change in strata is indicated by the slurry flowing out, washing should be stopped and a tube sampler should be attached to the end of the drill rod or the inner tube. Samples of the soil should be obtained by driving the sampler into the soil by hammering or jacking. Jacking or pulley method should be used when undisturbed samples are required. Initially fish-tail bit or pistol bits are used for drilling bore hole up to weathered material. These bits should be replaced by tungsten carbide or diamond bits. Double tube-core barrels are recommended for drilling in weathered rock

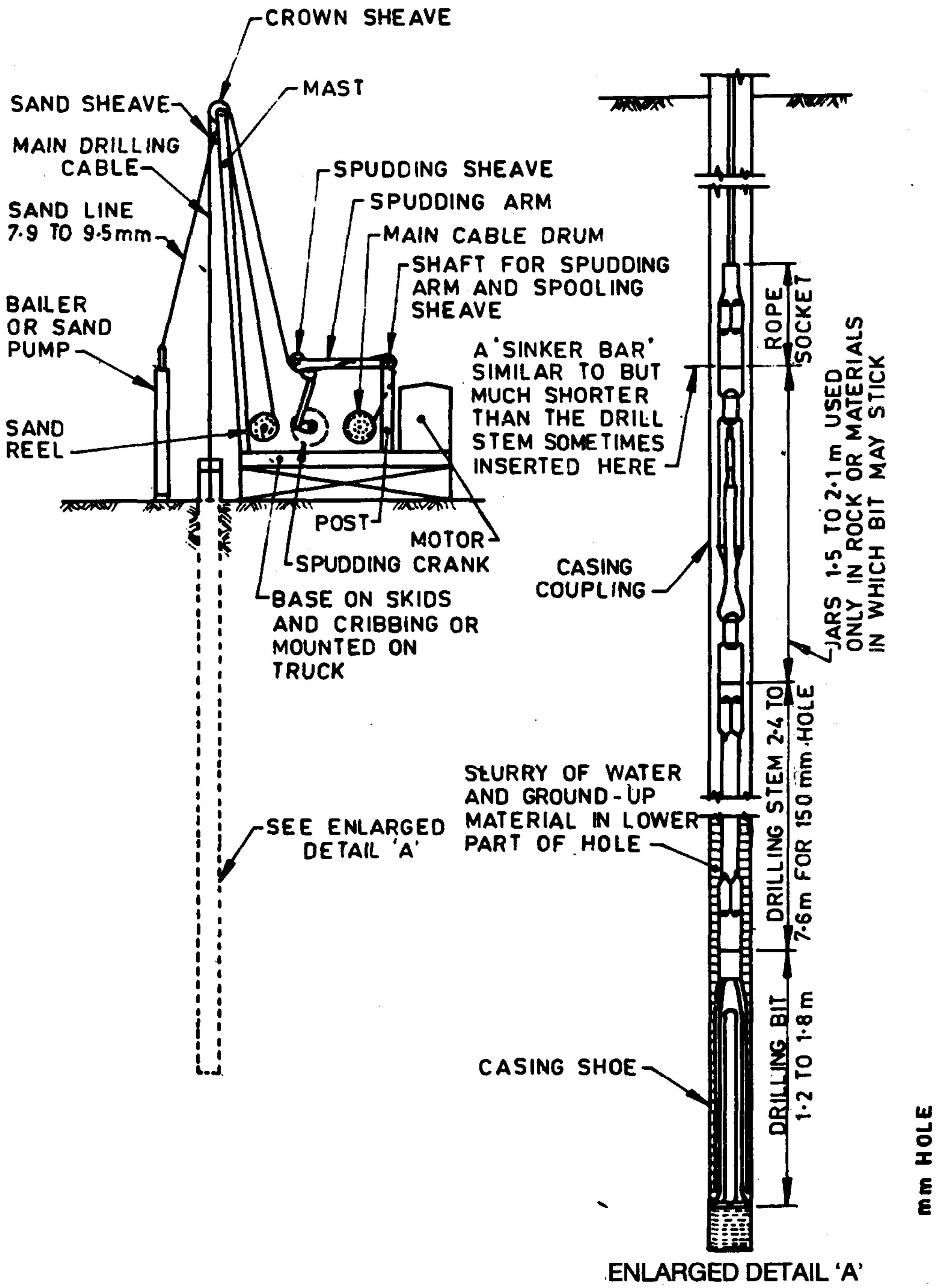


FIG. 2 TYPICAL ARRANGEMENT OF PERCUSSION DRILLING

stratum with seaming shells and core catcher as required.

3.6.5 *Rotary Boring*

3.6.5.1 *Mud-rotary drilling* (*see* Fig. 4) — In this system, boring is effected by the cutting action of a rotating bit which should be kept in firm contact with the bottom of the hole. The bit is carried at the end of hollow, jointed drill rods which are rotated by a suitable chuck. A mud-laden fluid or grout is pumped continuously down the hollow drill rods and the fluid returns to the surface in the annular space between the rods and the side of the hole, and so the protective casing may not be generally necessary. In this method, cores may be obtained by the use of coring tools.

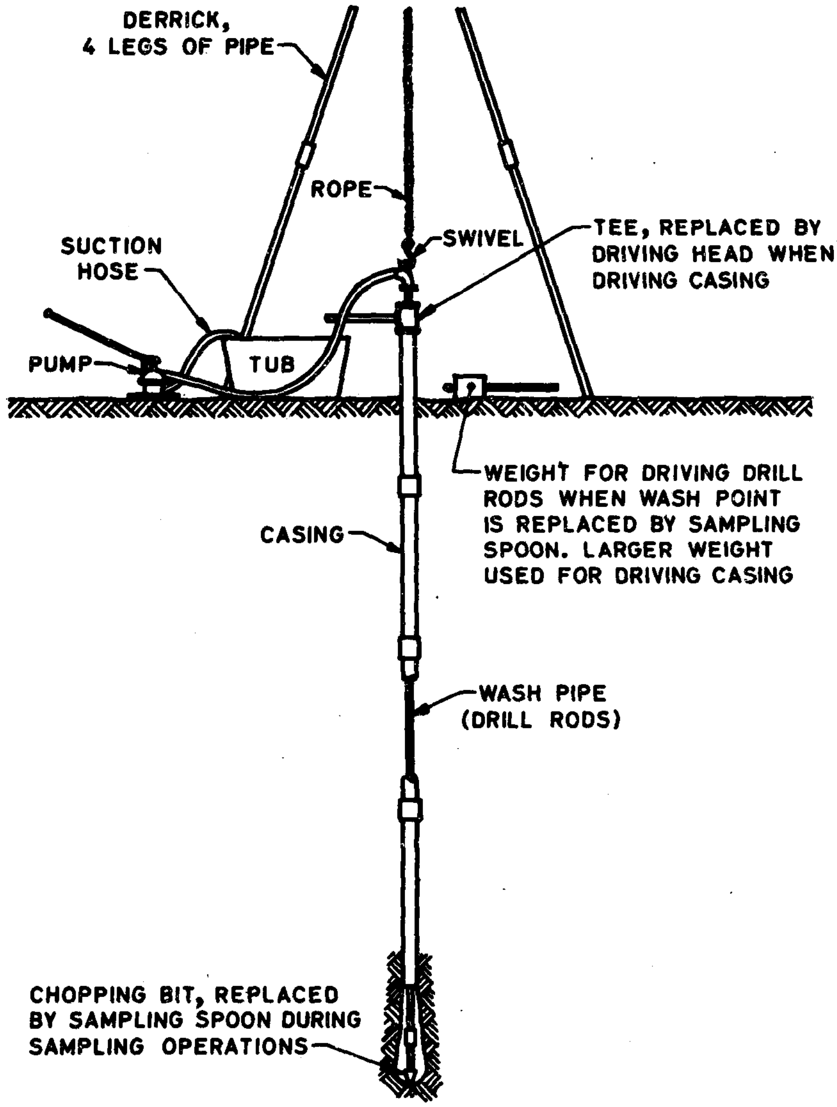


FIG. 3 TYPICAL ARRANGEMENT FOR WASH BORING

3.6.5.2 *Simplified mud-boring method* — In this method, the boring is advanced by a cutter fixed to drill rods which are rotating by means of pipe wrench. Bentonite is pushed simultaneously by a double piston pump. The slurry flowing out of cutter bottom, mixes up with the cut soil and flows to the bore hole surface, settling tank and back to the slurry tank. The process is continuous and the same slurry can be used several times. The drilling tool is lowered slowly with the help of manually operated winch fixed on a tripod. After the boring is advanced up to the desired depth, pumping of the slurry should be continued for 10 to 15 minutes.

In case gravel and *kankar* are encountered, a gravel trap fitted with stays around the drill rod, a little above the cutter, may be used. The trap consists of 80 to 100 cm long hollow cylinder having a conical shape at bottom. Holes of 3 mm diameter are also drilled in the drill rod within the trap as well as in the conical portion of the trap. During boring, gravel and *kankar* rise a little and then settle into the trap. With the provision of holes, no finer materials settle in the trap.

The small silt-sand stone or hard beds may be broken using conical or chisel-ended bits connected with drill rod. The broken pieces can subsequently be removed by means of the gravel trap.

3.6.5.3 *Core drilling* — Core drills shall be so designed that in sound rock, continuous

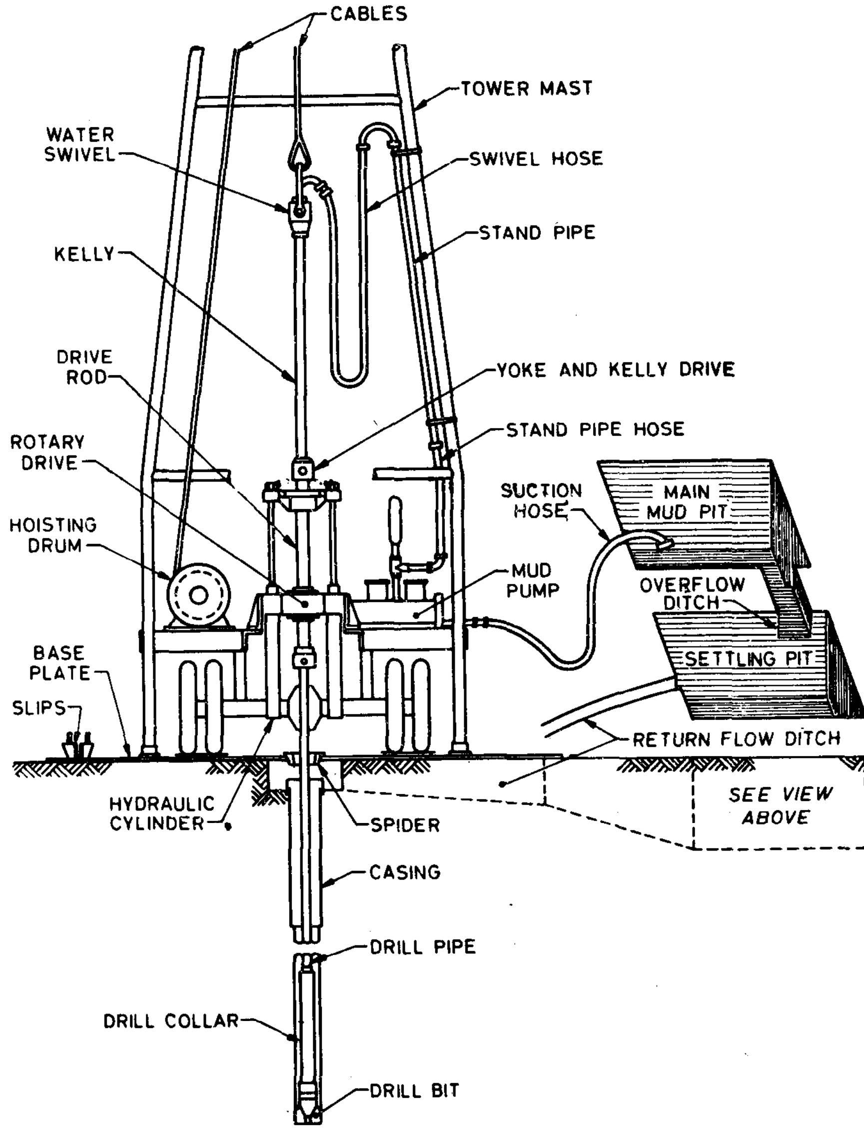


FIG. 4 TYPICAL ARRANGEMENT FOR ROTARY DRILLING

recovery of core is achieved. Water is circulated down the hollow rods, which returns outside them, carrying the rock cuttings to the surface as sludge. These shall be retained as samples in traversing friable rock where cores cannot be recovered. It is important to ensure that boulders or layers of cemented soils are not mistaken for bed rock. This necessitates core drilling to a depth of at least 3 m in bed rock in areas where boulders are known to occur. For shear strength determination, a core with diameter to height ratio of 1:1 is required. Rock pieces may be used for determination of specific gravity and classification.

3.6.5.4 *Shot drilling* — The system is used in large diameter holes that is over 150 mm. Due to the necessity of maintaining the shots in adequate contact with the cutting bit and the formation, holes inclined over 5° or 6° cannot be drilled satisfactorily. This system is different from other types of core drilling because the coarser cuttings do not return to the surface but are accumulated in a chip cup immediately above the bit and here the chilled shot is used as an abrasive in place of the drilling head.

3.7 Pressure Meter — A pressure meter (*see* Fig. 5) applies a uniform radial stress to the bore hole at any desired depth and measures consequent deformation. The test involves lowering of an inflatable cyclindrical probe to the test depth in a bore hole. The probe is inflated by applying water pressure from a reservior. Under

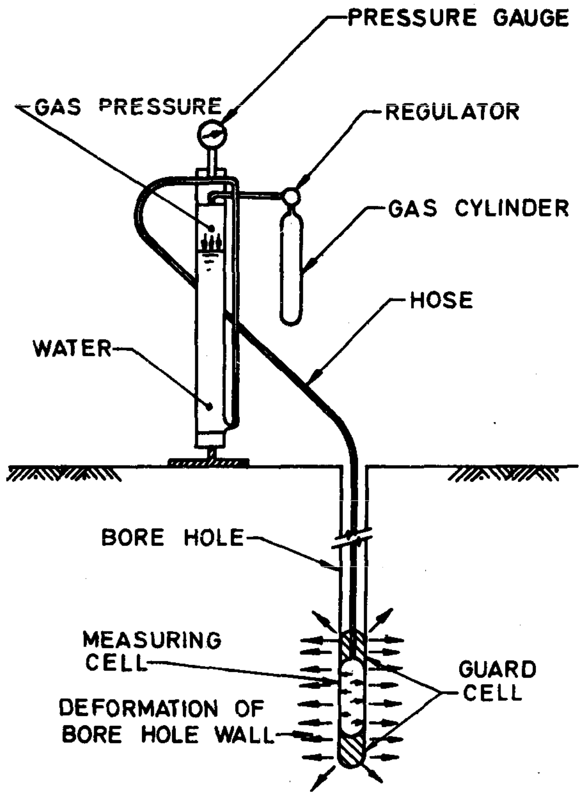
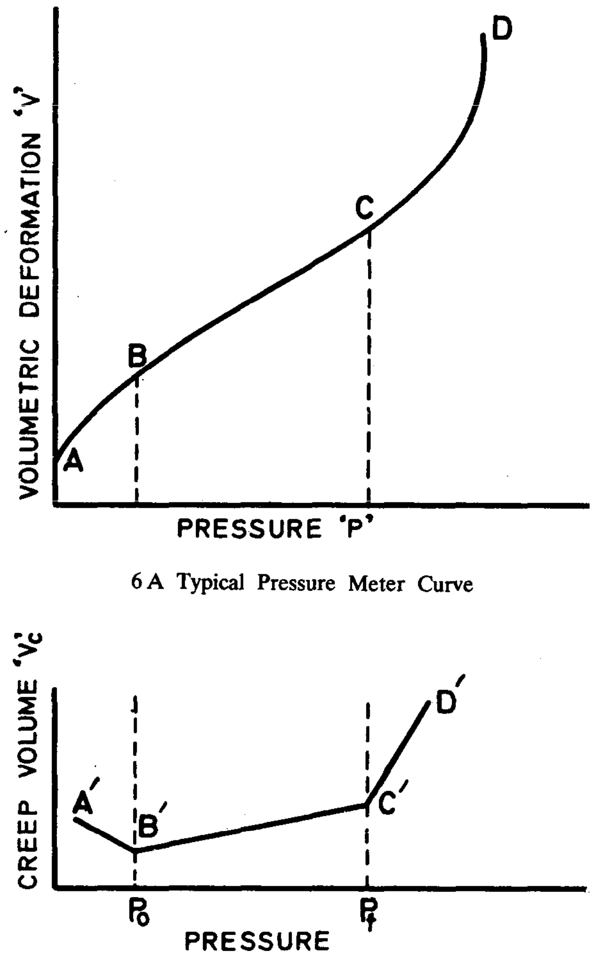


FIG. 5 PRINCIPLE OF PRESSURE METER



6 A Typical Pressure Meter Curve

6 B Corresponding Creep Diagram

FIG. 6 PRESSURE AND CREEP CURVES

pressure, it presses against the unlined wall of the bore hole and causes volumetric deformation. The stress on the bore hole wall is the pressure of water applied. The deformation of the bore hole is read in terms of volume corresponding to fall in water level of the reservior. The readings are plotted as shown in Fig. 6.

3.8 Field Tests — Certain tests, if required, are to be carried out on materials without actual removal of the material from its existing positon. Dilatancy, consistency, density, structure, colour and field identification should be carried out during reconnaissance at preliminary investigation stage.

3.8.1 *Tests Which Measure Properties of the Soil* — These tests are :

a) Vertical loading tests (*see* IS 1888 : 1971)
b) Deep penetration test — Standard penetration test (*see* IS 2131 : 1963) and Cone penetration tests [*see* IS 4968 (Part 1) : 1976, IS 4968 (Part 2) : 1976 and IS 4968 (Part 3) : 1976]
c) Vane shear tests (*see* IS 4434:1978)
d) Measurement of density of the soil [*see* IS 2720 (Part 38) : 1974 and IS 2720 (Part 39) : 1975]
e) Pressure meter test

A brief note on the tests (a), (b) and (c) is given in Appendix C.

3.9 Choice of Method — The choice of the method depends on the following factors:

a) Nature of Ground

1) *Soils* — In clayey soils borings are suitable for deep exploration and pits for shallow exploration. It is possible to take representative undisturbed samples in both cases.

In sandy soils, boring is easy but special equipment such as, Bishop or Osterberg piston samplers, should be used for taking undisturbed samples below the water table. Such samples can however be readily taken in trial pits provided that, where necessary, some form of ground water lowering is used.

2) *Rocks* — Borings are suitable in hard rocks and pits in soft rocks. Core borings are suitable for the identification of types of rocks but they provide braited data on joints and fissures. NX bore hole camera is

useful to photograph the stratification in drilled bore holes.

b) *Topography* — In hilly country the choice between vertical openings (for example, borings and trial pits) and horizontal openings [for example, exploratory drifts (*see* IS 4453 : 1980)] may depend on the topography and the geological structure. Steeply inclined strata and slopes are most effectively explored by drifts or inclined borings and low dipping strata or gentle slopes by trial pits or vertical borings.

   Swamps and areas overlain by water are best explored by borings which may have to be put down from floating craft.

c) *Cost* — For deep exploration, borings are usual as deep shafts are costly. For shallow exploration in soil, the choice between pits and borings will depend on the nature of the ground and the information required for shallow exploration in rock; the cost of bringing a core drill to the site will only be justified, if several holes are required; otherwise trial pits will be more economical.

3.9.1 *Trial Pits and Shallow Bore Holes* — Trial pits are preferable to shallow bore holes, since they enable sand and single strata to be seen in their undisturbed state and give a more accurate idea of timbering and pumping that may be required. Trial pits in stiff fissured clays also give fairly accurate idea of the depth of open excavations or vertical cuts that can be carried out without shoring. They also give a better picture of the patchy ground where the soil lies in pockets. In case of gravels and sandy soils fines tend to be washed out and the various layers are apt to become mixed as a result of 'piping' Hence it is difficult in such cases to obtain representative samples and, unless proper precautions are taken, a misleading impression may be obtained. The best procedure is to obtain samples from trial pits dug after the ground water has been lowered by means of wells or sumps with suitable filter linings.

4. SAMPLING TOOLS

4.0 General — To take undisturbed samples from bore holes properly designed sampling tools are required. These differ for cohesive and non-cohesive soils and for rocks.

4.1 The fundamental requirement of a sampling tool is that on being forced into the ground, it should cause as little displacement, remoulding and disturbance as possible. The degree of disturbance is controlled by the following three features of its design:

a) Cutting edge,
b) Inside wall friction, and
c) Non-return value.

4.1.1 *Cutting Edge* — A typical cutting edge is shown in Fig. 7. It should embody the following features:

a) *Inside clearance* (C_I) — The internal diameter (D_c) of the cutting edge should be slightly less than that of the sample tube (D_s) to give inside clearance. The inside clearance, calculated as follows, should be between 1 and 3 percent of the internal diameter of the sample tube. This allows for elastic expansion of the soil as it enters the tube, reduces frictional drag on the sample from the wall of the tube and helps to retain the core:

$$C_I = \frac{D_s - D_c}{D_c}$$

b) *Outside clearance* (C_o) — The outside diameter (D_w) of the cutting edge should be slightly larger than the outside diameter (D_T) of the tube to give outside clearance. The outside clearance should not be much greater than the inside clearance. This facilitates the withdrawal of the sampler from the ground. The outside clearance should be calculated as follows:

$$C_o = \frac{D_w - D_T}{D_T}$$

c) *Area ratio* (A_r) — The area ratio, calculated as follows, should be kept as low as possible consistent with the strength requirements of the sample tube. Its value should not be greater than about 20 percent for stiff formations; for soft sensitive clays an area ratio of 10 percent or less should be preferred. Where it is not possible to provide sufficient inside clearance, pistion sampler should preferably be used:

$$A_r = \frac{D_w^2 - D_c^2}{D_c^2} \times 100 \text{ percent}$$

4.1.2 *Wall Friction*— This can be reduced by:

a) suitable inside clearance,
b) a smooth finish to the sample tube, and
c) oiling the tube properly.

4.1.3 *Non-return Valve*— The valve should have a large orifice to allow the air and water to escape quickly and easily when driving the sampler.

4.1.4 *Recovery Ratio* — For a satisfactory undisturbed sample, taking into consideration the influence of the inside clearance [*see* **4.1.1** (a)]

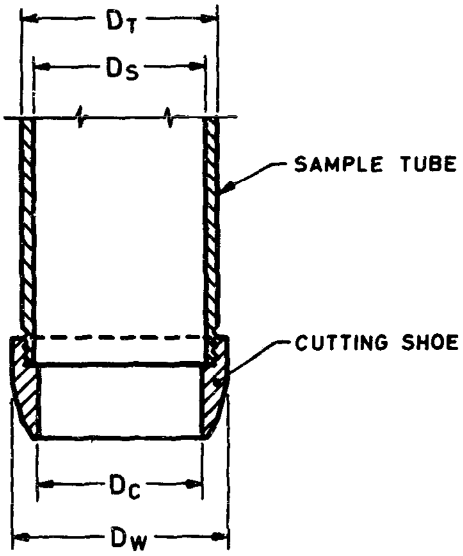


FIG. 7 DETAILS OF CUTTING EDGE

when excess soil is prevented from entering the tube, the recovery ratio calculated as follows should be between 98 and 96 percent.

$$R_r = \frac{L}{H}$$

where

R_r = recovery ratio,

L = length of the sample within the tube, and

H = the depth of penetration of the sampling tube.

4.2 Types of Samplers

4.2.1 *Cohesive Materials*

4.2.1.1 *Open tube sampler* — The open tube sampler is an ordinary seamless steel tube with its lower edge chamfered to make penetration easy. Subject to the requirements of undisturbed sampling, an open tube sampler with a separate cutting shoe may also be used. The sampler head which connects the tube to the boring rod is provided with vents to permit water to escape when sampling under water and check valve to help to retain the sample while withdrawing the sampler (*see* IS 2132 : 1972)

4.2.1.2 *Split spoon sampler* — Split spoon sampler is a modified form of the open tube sampler, in which the sampling tube is split into two halves held together by the cutting edge and the sampler head. This sampler makes the removal of the sample easier.

The split spoon sampler driven as specified in IS 2131 : 1963, may be used in foundation investigations to collect samples for visual identification and preliminary laboratory tests. This penetration results may be utilized to correlate *in-situ* properties like density, sheai strength and bearing capacity.

4.2.1.3 *Piston sampler* — A piston sampler consists of two separate parts: (a) the sample cylinder, and (b) the piston system; the latter which is actuated separately fits tightly in the sampler cylinder.

The single important control in the operation of the piston sampler is the separate actuation of the piston system. It may be done by separate drilling rods, or by a liquid pressure device or by a special lock and wire-rope system.

During the driving and till the start of the sampling operation, the bottom of the piston should be flush with the cutting edge of the sampler. At the desired sampling elevation, the piston should be fixed in relation to the ground and the sampler cylinder forced independently into the ground, thus punching a sample out of the soil.

The piston prevents water and dirt from entering the tube during the lowering operation. It also serves to keep the recovery ratio constant during the punch. As the sampler tube slides past the tight fitting piston during the sampling operation, a negative pressure is developed above the sample, which holds back the sample during withdrawal.

Both the sample cylinder and piston system should be finally withdrawn with the sample remaining in the sample cylinder.

4.2.2 *Cohesionless Materials*

4.2.2.1 In the case of either cohesionless materials such as silts and sands or soft soil strata, sampling operations are confronted with the possibility of the sample falling out of the sampler due to lack of cohesion; this is specially so with increasing diameter of sampling tubes. Hence some form of positive control should be incorporated at the top and/or bottom of the sampling device. This is affected in the following manner:

a) *Control at the top of the sampler* — A reduction of pressure on the sample is brought about by providing a ball valve in the sampler head or a properly packed free or stationary.piston in the sampling tube.

b) *Control at the bottom of the sampler* — This control is achieved by:

1) incorporating core retainers in the form of concealed springs, multiple flap valves, claw-shell valves in the sample shoe or introducing core retainers attached to an

auxiliary barrel pushed down the sampler after the drive; this will, however, disturb the samples to certain extent; and

2) maintenance of slight pressure below the sampler by the injection of compressed air into the space below the sampler formed by the introduction of an auxiliary core barrel; Bishop sampler may be used.

c) *Solidification by the introduction of chemicals or emulsions* — The solidification may be done at the bottom of the sampling tube after driving or a sufficient volume of the strata to be sampled may be solidified before the sampling operation starts.

4.2.3 *Rocks* — Cores of rock should be taken by means of rotary drills with a coring bit. Other types of drills, such as shot drills may also be used All types of rotary drills should be fitted with core barrels and core catchers which break off the core and retain it when the rods are withdrawn. Double tube core barrels should be used for ensuring better core recovery and picking up soft seams or layers in bed rocks.

5. METHODS OF SAMPLING

5.1 Samples are of two types:

a) *Disturbed Samples* — These are taken by methods which modify or destroy the natural structure of the material though, with suitable precautions, the natural moisture content can be preserved.

b) *Undisturbed Samples* — These are taken by methods which preserve the structure and properties of the material. Such samples are easily obtained from most rocks, but undisturbed samples of soil can only be obtained by special methods.

The following table indicates the methods that are usually employed :

Nature of Ground	*Type of Sample*	*Method of Sampling*
Soil	Disturbed	Hand samples Auger samples (for example, in clays) Shell samples (for example, in sand)
	Undisturbed	Chunk samples Tube samples
Rock	Disturbed	Wash samples from percussion or rotary drilling
	Undisturbed	Cores

In cohesive soils of all types, it is possible with most strata to procure undisturbed samples which are very satisfactory for examination and testing purposes. Undisturbed sampling of sand below the water table is not always an easy matter but special methods have recently been developed for this purpose and used satisfactorily.

6. PROCEDURE FOR TAKING SAMPLES

6.1 Disturbed Soil Samples — Disturbed samples of soils may be obtained in the course of excavation and boring. The taking of disturbed samples of clay may result in the remoulding of the material and may render it unsuitable for shear strength measurements unless it is required for fill. Such samples are suitable for mechanical analysis and tests for index properties. These samples may not be truly representative, specially when taken from below the ground-water level. This is more so in the case of gravels containing a portion of fine sand, since the finer fractions tend to be washed off the sampler by the water. For procuring true samples, where possible, the ground-water level may be lowered by means of pumping from filter wells before procuring samples, or special type of samplers used (*see* **4.2.2**). The quantity of sample generally required for testing purposes is given in Table 2 [*see also* IS 2720 (Part 1) : 1972].

6.2 Undisturbed Soil Samples— Samples shall be obtained in such a manner that moisture content and structure do not get altered. These may be attained by careful protection and packing, and by the use of a correctly designed sampler.

6.2.1 *Clay* — If strength of soft clay is to be known, the sampling procedure may be supplemented by *in-situ* tests like vane shear test (*see* IS 4434 : 1978) which gives a measure of the shear strength of the soil.

TABLE 2 QUANTITY OF SOIL SAMPLE REQUIRED
(*Clause* 6.1)

SL NO.	PURPOSE OF SAMPLE	SOIL TYPE	WEIGHTS OF SAMPLE REQUIRED kg
i)	Soil identification, natural moisture content tests, mechanical analysis and index properties Chemical tests	Cohesive soils	1
		Sand and gravels	3
ii)	Compaction tests	Cohesive soils and sands	12.5
		Gravelly soils	25
iii)	Comprehensive examination materials including soil stabilization	Cohesive soils and sands	25 to 50
		Gravelly soils	50 to 100

6.2.1.1 *Chunk samples* — Chunk samples may be taken where clay is exposed in excavation. A block of clay should be carefully removed with a sharp knife taking care that no water is allowed to come into contact with the sample and that the sample is protected from exposure to direct sun and wind. The chunk sample should be coated with molten wax so that the layer of wax prevents escape of moisture from the sample. Chunk samples are not suitable if those are to be transported to long distances because in such cases the samples will get disturbed in transit. Undisturbed samples may also be obtained by means of a sampling tube of 10 cm internal diameter provided with a cutting edge. In this procedure the soil surrounding the outside of the tube should be carefully removed while the tube is being pushed in.

6.2.1.2 *Core samples* — The sampler should be lightly oiled or greased inside and outside to reduce friction. It should then be attached to the boring rods and lowered to the bottom of the bore hole or trial pit. The sampler should be pushed into the clay by hand or by jacking. Where this is not possible, the sampler may be driven into the clay by blows from a 'monkey'.

The distance to which the sampler is driven should be checked, because, if driven too far, the soil will be compressed in the sampler. A sampling head with an 'overdrive' space will allow the sample tube to be completely filled without damaging the sample. After driving the rods, the sampler should be rotated to break off the core and the sampler should be steadily withdrawn. In soft clays and silty clays where samples are required from below the water table, with water standing in the casing pipe, a piston sampler may be used with advantage.

6.2.1.3 For compression test samples, a core of 40 mm diameter and about 150 to 200 mm long may be sufficient; but for other laboratory tests, a core of 75 to 100 mm diameter and preferably 300 mm long is necessary. The upper few millimetres of both types of sample should be rejected as the soil at the bottom of the bore hole usually gets disturbed by the boring tools.

6.2.2 *Sand* — Comparatively undisturbed samples of moist sand above ground-water level may be taken from natural exposures, excavations or borings by gently forcing a sampling tube into the soil. Undisturbed samples of sand below ground-water table may be obtained by the use of a compressed air sampler, which enables the sample to be removed from the ground into an air chamber and then lifted to the surface without contact with water in the bore hole. This may be done by another method which involves the use of a thin-walled piston sampler and bentonite or other types of drilling mud. The use of bentonite or other drilling mud obviates the need for casing pipes with thin wall samplers. In all methods, it is essential to maintain the water or drilling mud in the boring tube at or slightly above ground-water level. This prevents any disturbance of the structure of the sand by the flow of water into the bore hole.

6.3 Rock Samples

6.3.1 *Disturbed Samples* — The sludge from percussion borings or from rotary borings which have failed to yield a core, may be taken as a disturbed sample. It may be recovered from circulating water by settlement in a trough. The rock type may be deduced by examining the material of which the sludge is composed.

6.3.2 *Undisturbed Samples*

6.3.2.1 *Block samples* — Such samples taken from the rock formation shall be dressed to a size convenient for packing to about 90 × 75 × 50 mm.

6.3.2.2 *Core samples* — Cores of rock shall be taken by means of rotary drills fitted with a coring bit with core retainer, if warranted. Good core recovery (*see* **4.1.4**) depends upon the correct operation and careful use of the equipment.

6.3.2.3 *Frequency of sampling* — In intermittent sampling, undisturbed soil samples are obtained at every change in stratum and at intervals not exceeding 1.5 m within a continuous stratum. On important investigations such as the foundations for an earth dam, continuous core sampling in any soft clay layers may be necessary.

6.4 Water Samples — If a trial pit has been excavated or a well exists near about the site of exploration, the collection of water samples does not present any difficulty. However, if it is to be collected from a bore hole made at the site, some difficulty is apprehended on account of the narrowness of the bore, caving-in of the sides, etc. In the latter case, therefore, it should suffice to collect the water sample from the bore hole with the help of a common suction pump having a hose pipe, rubber tubing, etc, which can be conveniently lowered down into the bore hole, connected at the suction end. The water may then be collected into a clean vessel, allowed to settle and the supernatant liquid poured out into a clean well-rinsed glass or polyethylene bottle. The water samples may then be sent to the laboratory for chemical analysis.

6.5 Records of Borings and Trial Pits

6.5.1 *Borings* — In recording exploratory work in connection with borings necessary information should be given, preferably on a record sheet of the type given in Appendix D. A site plan showing the disposition of the borings should be attached to the records. Where a deep boring has deviated from line, a plan and section should accompany the record.

6.5.2 *Trial Pits* — Plans and sections, drawn to the largest convenient scale, should be provided. The following information should also be given :

a) Agency;
b) Location with map and plan reference;
c) Pit number;
d) Reduced level (RL) of ground surface or other reference point;
e) Dates, started and completed;
f) Supervision;
g) Scales of plans and sections;
h) Dimensions, types of sheeting and other materials of stabilization, method of advancing the exploration, such as by hand tools, blasting, boring, etc;
j) General description of strata met with;
k) Position and attitude of contacts, faults, strong joints, slicken-sides, etc;
m) Inflow of water, methods of controlling the water, required capacity of pumps;
n) The level at which the subsoil water table is met with;
p) Dip and strike of bedding, and of cleavage; and
q) Any other information and remarks.

7. PROTECTION, HANDLING AND LABELLING OF SAMPLES

7.1 Care should be taken in protection and handling of samples and in their full labelling so that samples can be received in a fit state for examination and testing and can be correctly recognized as coming from a specified trial pit or boring. Suitable methods are given in Appendix E.

7.2 Extrusion of Samples — Undisturbed samples of soil retained in a liner or seamless tube sampler which arrive in the testing laboratory, sealed with wax at both ends, have to be taken out of the liners or tubes for actual testing. This should be done very carefully without causing any disturbance to the samples themselves. The wax may be chipped off by a penknife. This may also be done by slightly warming the sides of the tube or liner at the ends when the wax will easily come off. If the tubes or liners are oiled inside before use, it is quite possible for samples of certain moisture range to be pushed out by means of suitably designed piston extruders. If the extruder is horizontal, there should be a support for the sample as it comes out from the tube so that it may not break. For screw type extruders, the pushing head must be free from the screw shaft so that no troque is applied to the soil sample in contact with the pushing head. All extruding operations must be in one direction, that is, from cutting edge to the head of the sample tube. For soft clay samples pushing with an extruder piston may result in shortening or distortion of the sample. In such cases, the other alternative is to cut the tube by means of a high speed hacksaw in proper test lengths and fill the testing moulds, by placing the cut portions directly over the moulds and pushing the sample in with a suitable piston. After the sample is extruded, it should be kept either in humidity chamber or in a desiccator and taken out only when actual testing is carried out to avoid possible loss of moisture.

8. EXAMINATION AND TESTING OF SAMPLES

8.1 The samples of soils and rocks are to be tested in the laboratory for determining their engineering properties. The various tests that are usually necessary for different phases of exploration are given in Table 3.

TABLE 3 TESTS FOR DIFFERENT PHASES OF EXPLORATION
(*Clause* 8.1)

Phase of Exploration	Tests Necessary on a Sample		
	Type of Test	Detailed Tests	
i) Reconnaissance exploration		Visual classification (*see* IS 1498:1970[1])	
ii) Detailed exploration	Physical tests	Liquid and plastic limits [*see* IS 2720 (Part 5) : 1970[2]]	
		Grain size analysis [*see* IS 2720 (Part 4) : 1975[3]]	
		Specific gravity [*see* IS 2720 (Part 3) : 1980[4]]	
		Natural moisture content [*see* IS 2720 (Part 2) : 1973[5]]	
		Unit weight [*see* IS 2720 (Part 3) : 1980[4]]	
		Consolidation test (including preconsolidation pressure) [*see* IS 2720 (Part 15) : 1965[6]]	
		Shear strength :	
		Unconfined compression [*see* IS 2720 (Part 10) : 1973[7]]	
		Triaxial compression [*see* IS 2720 (Part 11) : 1971[8]]	
		Direct shear permeability test [*see* IS 2720 (Part 13) : 1972[9]]	
	Chemical tests	Soluble salt content :	
		Chlorides and sulphates [*see* IS 2720 (Part 27) : 1977[10]]	
		Calcium carbonate content (if warranted) [*see* IS 2720 (Part 23) : 1976[11]]	
		Organic matter content (if warranted) [*see* IS 2720 (Part 22) : 1972[12]]	
	Ground water	Chemical analysis including *p*H determination [*see* IS 2720 (Part 26) : 1973[13]]	
		Bacteriological analysis (if necessary)	
	Rock drilling	Visual examination	Petrographic analysis
		Unit weight	Compressive strength
		Water absorption	Shear strength
		Porosity	

[1]Classification and identification of soils for general engineering purposes *(first revision)*.
[2]Methods of test for soils : Part 5 Determination of liquid and plastic limits *(first revision)*.
[3]Methods of test for soils : Part 4 Grain size analysis *(first revision)*.
[4]Methods of test for soils : Part 3 Determination of specific gravity *(first revision)*.
[5]Methods of test for soils : Part 2 Determination of water content *(second revision)*.
[6]Methods of test for soils : Part 15 Determination of consolidation properties.
[7]Methods of test for soils : Part 10 Determination of unconfined compressive strength *(first revision)*.
[8]Methods of test for soils : Part 11 Determination of shear strength parametres of a specimen tested in unconsolidated undrained triaxial compression without the measurement of pore water pressure.
[9]Methods of test for soils : Part 13 Direct shear test *(first revision)*.
[10]Methods of test for soils : Part 27 Determination of total soluble sulphates *(first revision)*.
[11]Methods of test for soils : Part 23 Determination of calcium carbonate *(first revision)*.
[12]Methods of test for soils : Part 22 Determination of organic matter *(first revision)*.
[13]Methods of test for soils : Part 26 Determination of *p*H values *(first revision)*.

APPENDIX A

(*Clause* 3.1.2)

CURRENT METHODS OF SUBSOIL EXPLORATION

Sl No.	*Method*	*Mode of Operation*	*Type of Formation*
	A. RECONNAISSANCE METHODS		
		i) *Geophysical*	
1)	Electrical resistivity method ac or dc)	Measurement of variations in the apparent resistivity as measured on the ground	Alluvial deposits weathered and fissured rock, buried channels and ground water
2)	Seismic refraction method	Measurement of velocities of compressional waves from the travel time curves of seismic waves	do
		ii) *Sounding*	
3) a)	Standard penetration test (*see* IS 2131 : 1963)	Variations in the stratification is correlated with the number of blows required for unit penetration of standard penetrometer by a drive hammer	Non-cohesive soils without boulders
b)	Static cone penetrometer test [*see* IS 4968 (Part 3) : 1976]	The cone penetrometer is advanced by pushing and the static force required for unit penetration is correlated to the engineering properties like density, bearing capacity, settlement, stratification, etc.	Primarily used in cohesive soils
c)	Dynamic cone penetrometer test [*see* IS 4968 (Part 2) : 1976]	The cone is driven by a standard hammer and the rest is as in (b)	Primarily used in cohesive soils
	B. EXPLORATORY METHODS		
		i) *Drilling*	
4)	Shell and auger	Using auger for soft clays and shell for firm to stiff clays; in sand to be used with casing for lining and with bentonite; for boring at depths if more than 25 m power operator winches are used	All types of soils specially soils of mixed type
5)	Hand auger	The auger is power or hand operated with periodic removal of the cuttings	All soils except sands and gravels above water table
6)	Simplified mud boring	Manual rotation of cutter fixed with drill rods by means of pipe wrench; simultaneously pumping of bentonite slurry by manually operating a double piston pump. Chisel and gravel trap used for hard bed, gravel and *kankars*	Silts and sands or mixed soils specially below water table
7)	Wash boring	Light chopping, strong jetting and removal of cuttings by circulating water. Change of stratification could be guessed from the rate of progress and colour of the wash water	Soft to stiff cohesive soils and fine sand except gravel and boulders
8)	Percussion drilling	Power chopping, hammering and periodic removal of the slurry with bailers. The strata could be identified from the slurry	Rocks and soils with boulders, except clay or loose sand
9)	Rotary drilling	Power rotation of the coring bit which may vary from metal bits to tungsten carbide or diamond bits depending upon the hardness of formation (*see* IS 6926 : 1973 *and* IS 5313 : 1980)	Rocks, fissured rock and all soils except cobbles and boulders
		ii) *Exploratory Sampling*	
10)	Open tube sampler and split tube sampler	Driving standard sampler by a hammer weighing 65.0 kg through a drop of 750 mm (*see* IS 2131 : 1963)	Cohesive soils and silts
11)	Double tube core barrels	Used with a rotary machine; non-rotating inner barrel of swivel type slips over the sample and retains it as the outer bit advances (*see* IS 6926 : 1973)	Coarse sand and gravels; most suitable for soft rocks like shale and any weathered rock formation

Sl No.	Method	Mode of Operation	Type of Formation
		C. DETAILED INVESTIGATIONS	
		i) *Undisturbed Sampling*	
12)	Thin walled tubes 50 to 125 mm	The tubes are jacked into a cleaned hole under a static force (*see* IS 2132 : 1972)	Soils of medium strength
13)	Piston type sampler	The tubes are jacked into a cleaned hole under a static force	Clays and silts
14)	Samplers with special core retainers	do	do
15)	Sand sampler	The tubes are jacked into a cleaned hole under a static force (*see* IS 8763 : 1978)	Sand without boulder
16)	Solidification methods	Solidification at the bottom of the sampler after jacking the sampler into soils	do
17)	Open cuts and trenches	The sample is cut from the sides and bottom of a trench and sealed in a wooden box	All types of formations
		ii) *Bearing Capacity Tests*	
18)	Plate load test (soils)	Loading a steel plate at desired elevation and measuring the settlement under each load, until a desired settlement takes place or failure occurs (*see* IS 1888 : 1971)	Clay and sandy formations
19)	Load test (rocks)	Loading two discs placed diametrically opposite each other on two sides of a trench, by means of a jack and measuring the deflection near the sides	Rocks
20)	Vane shear test	Advancing a four-winged vane into a fresh soil at desired elevation and measuring the torque developed in rotating the vane (*see* IS 4434 : 1978)	Soft and sensitive clays
		iii) *Logging of Bore Holes by Geophysical Methods*	
21)	Electrical logging	Measuring the potential and resistances of formation by an electrode system at various elevations	—
22)	Neutron logging	Measuring the intensity of scattered radiation from a system at desired elevation	—
23)	Gamma ray logging	Measuring the intensity of scattered gamma radiation from a system at desired elevation	—

APPENDIX B

(*Clause* 3.2)

OUTLINE OF SEISMIC AND ELECTRICAL RESISTIVITY METHODS

B-1. SEISMIC METHOD

B-1.1 In this method shock waves are created into the soil, at ground level or at a certain depth below it, by striking a plate on the soil with a hammer or by exploding small charges in the soil. The radiating shock waves are picked up by the vibration detector (geophone) where the time of travel gets recorded. Either a number of geophones are arranged in a line or the shock producing device is moved away from the geophone to produce shock waves at given intervals. Some of the waves, known as direct or primary waves, travel directly from the shock point along the ground surface and are picked up first by the geophone. If the subsoil comprises two or more distinct layers, some of the primary waves travel downwards to the lower layer and get refracted at the surface. If the underlying layer is denser, the refracted waves travel much faster. As the distance from the shock point and the geophone increases, the refracted waves reach the geophone earlier than the direct waves. Figure 8 shows the diagrammatical travel of the primary and refracted waves. The results are plotted on a graph as shown in Fig. 9, between distance *versus* time of travel. The break in the curve represents the point of simultaneous arrival of primary and

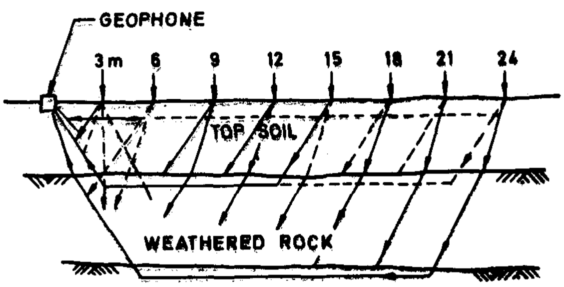


FIG. 8 WAVE REFRACTION PRINCIPLE

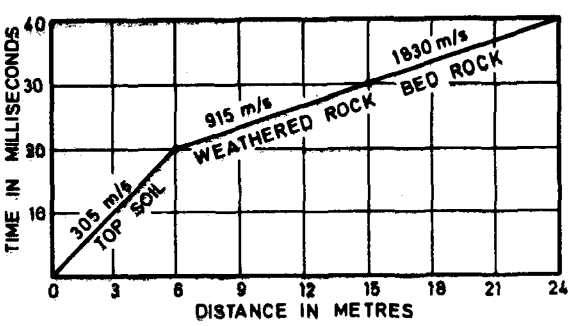


FIG. 9 TIME-DISTANCE GRAPH

retracted waves and its distance is known as critical distance which is a function of the depth and the velocity ratio of the strata. This method is effective when the velocities successively increase with depth.

The various velocities for different materials is given below as a guide:

Materials	*Velocity*, m/s
Sand and top soil	180 to 365
Sandy clay	365 to 580
Gravel	490 to 790
Glacial till	550 to 2 135
Rock talus	400 to 760
Water in loose materials	1 400 to 1 830
Shale	790 to 3 350
Sandstone	915 to 2 740
Granite	3 050 to 6 100
Limestone	1 830 to 6 100

B-2. ELECTRICAL RESISTIVITY METHOD

B-2.1 The electrical resistivity method is based on the measurement and recording of changes in the mean resistivity or apparent specific resistance of various soils. The resistivity (ρ ohm.cm) is usually defined as the resistance between opposite phases of a unit cube of the material. Each soil has its own resistivity depending upon water content, compaction and composition; for example, the resistivity is low for saturated silt and high for loose dry gravel or solid rock. The test is conducted by driving four metal spikes to serve as electrodes into the ground along a straight line at equal distances (*see* Fig. 10).

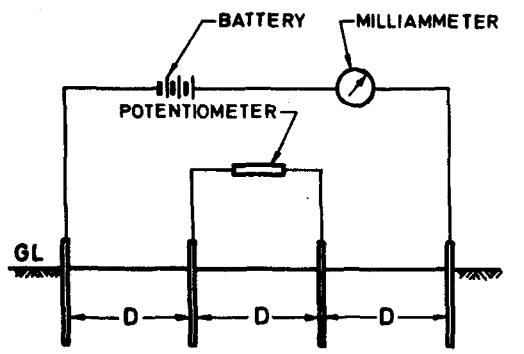


FIG. 10 POSITION OF ELECTRODES

B-2.1.1 A direct voltage is imposed between the two outer potentiometer electrodes and the potential drop is measured between the inner electrodes. The mean resistivity is given by the following formula:

$$\rho = 2\pi D \frac{E}{I}$$

where

ρ = mean resistivity (ohm. cm),
D = distance between electrodes (cm),
E = potential drop between inner electrodes (V), and
I = current flowing between outer electrodes (A).

B-2.1.2 To correctly interpret the resistivity data for knowing the nature and distribution of soil formations, it is necessary to make preliminary trial on known formations. Average values of resistivity (ρ) for various rocks and minerals are given below :

Material	*Mean Resistivity* ohm. m
Limestone (marble)	10^{12}
Quartz	10^{10}
Rock salt	10^{6}—10^{7}
Granite	5 000 —10^{6}
Sandstone	35 —4 000
Moraines	8 —4 000
Limestones	120 — 400
Clays	1 — 120

APPENDIX C

(*Clause* 3.8.1)

FIELD TESTS TO MEASURE PROPERTIES OF SOIL

C-1. VERTICAL LOADING TESTS

C-1.1 Loading tests may be used to determine whether the proposed loadings on foundations and subgrades are within safe limits, and subject to certain limitations, to assess the likely settlement of a structure. The greater the uniformity of the strata tested, the more reliance may be placed on the results obtained.

Table 4 gives guidance regarding the methods of estimating bearing capacity and settlement of structures for various types of soils.

TABLE 4 METHODS OF ESTIMATION OF BEARING CAPACITY AND SETTLEMENT

Sl No.	Type of Strata	Methods of Estimation	
		Ultimate Bearing Capacity	Settlement of Structures
1)	a) Hard rocks	L	L
	b) Soft rocks, such as shales, weak lime-stones and sand stones	FL	L
	c) Non-cohesive soils	FL	F
	d) Soft compressible soils	LF	LF
	e) Stiff, fissured clays	LF	LF
2)	Soft, compressible stratum overlying hard stratum	LF	L
3)	Hard stratum over-lying compressible stratum	LF*	L
4)	Very variable strata varying in type thick-ness and arrangement	Each case to be dealt with on its merits	

NOTE — Methods are given in order of preference:

F = Field load test

L = Laboratory tests : Compression and shear tests on undisturbed samples. Consolidation test on undisturbed samples. Elastic modulus tests on undisturbed samples.

* Tests should be made on each stratum.

C-1.2 The method of conducting load tests on soils is described in IS 1888:1971

C-2. DEEP PENETRATION TESTS

C-2.1 Penetration Tests in Bore Holes—These tests consist of measuring the resistance to penetration under static or dynamic loading of different shaped tools.

C-2.1.1 All the tests are empirical and their value lies in the amount of experience behind them.

C-2.1.2 Dynamic penetration tests in bore holes are the more usual and provide a very simple means of comparing the results of different bore holes on the same site and for obtaining an indication of the bearing value of non-cohesive soils which cannot easily be assessed in any other manner. The standard penetration test is the most widely used of these tests (*see* IS 2131:1963).

C-2.2 Sounding Tests — Deep sounding tests are carried out by means of apparatus consisting of an outer tube and an inner mandrel which can be driven by means of a hammer or caused to penetrate steadily by an increasing dead load or by jacking. Measurements are made of the resistance to penetration as the depth of penetration increases and the technique involves separate measurement of the direct toe resistance and the skin resistance [*see* IS 4968 (Part 1) : 1976, IS 4968 (Part 2) : 1976 and IS 4968 (Part 3) : 1976].

The method is used to determine the resistance to the driving of bearing piles and as a rapid means of preliminary site exploration and to supplement information obtained from borings.

C-3. VANE TESTS

C-3.1 The vane test has been shown to be a promising non-empirical method of measuring the shear strength of soft clay *in-situ* at all depths from the surface to at least 30 m. It is particularly useful in the measurement of strength in deep beds of soft sensitive clays.

C-3.2 The shear strength of soft clays can be measured *in-situ* by pushing into the clay a small four-bladed vane, attached to the end of a rod and then measuring the maximum torque necessary to cause rotation. To a close approximation this torque is equal to the moment developed by the shear strength of the clay acting over the surface of the cylinder with a radius and height equal to that of the vanes. The method of conducting vane test is given in IS 4434:1978.

APPENDIX D

(*Clause* 6.5.1)

RECORD OF BORING

Name of Boring Organization

Bored for: ....

Ground surface level: ....

Type of boring: Wash boring

Diameter of boring: ....

Inclination: Vertical

Boring ....

Location: Site

Boring No.: ....

Soil sampler used: ....

Date started: ....

Date completed: ....

Recorded: ....

Description of Strata	Soil Classification	Thickness of Stratum	Depth from Ground Surface	R.L of Lower Contact	Samples			Ground-Water Level	Remarks
					Type	No.	Depth and Thickness of Sample		
Fine to medium sands with practically no binder	SP		1 m 2 m 2·7 m		Undis-turbed	1	1 m 1·4 m 1·7 m 2 m		
Silty clays of medium plasticity. No coarse or medium sands	CI		3 m 4 m 5 m		Undis-turbed	2	3 m 4 m 4·3 m 5 m	Not struck up to 6m depth	

APPENDIX E

(*Clause* 7.1)

HANDLING AND LABELLING OF SAMPLES

E-1. HANDLING OF SAMPLES

E-1.1 Disturbed Samples of Soil — Where samples are required for testing, or where it is desirable to keep them in good condition over long periods, they should be treated as follows:

a) Immediately after being taken from the bore hole or trial pit, the sample should be placed in a cloth bag, tin or preferably, a glass jar of at least 0.5 kg capacity, and it should fill this container with a minimum of air space. The container should have an air-tight cover. In this way the natural water content of the sample can be maintained for one or two weeks without appreciable change.

b) The containers should be numbered and a label as described in **E-2** should be placed immediately under the cover in a container.

c) The containers should be carefully packed in a stout wooden box (preferably with separate partitions) with saw dust or other suitable material to prevent damage during transit.

d) Where necessary, the samples should be tested for natural water content immediately on arrival at the laboratory and an accurate description made of the sample. In such a case proper precautions should be taken to preserve the natural water content during sampling. During the interval, while the samples are awaiting transport, they should be stored, if possible, in a cool room.

E-1.2 Undisturbed Samples of Soil — The following conditions of handling and protection of undisturbed samples are to be regarded as a minimum requirement for samples taken by the usual methods; in special cases, it may be necessary to take more elaborate precautions:

a) Samples which are retained in a liner or which are retained in a seamless tube sampler should receive the following treatment:

Immediately after being taken from the boring or trial pit, the ends of the sample should be cut and removed to a depth of about 2.5 cm (or more in the top to cover any obviously disturbed soil). Several layers of molten wax should then be applied to each end to give a plug about 2.5 cm thick. If the sample is very porous, a layer of waxed paper should first be placed over the ends of the sample.

Any space left between the end of the liner or tube and the top of the wax should be tightly packed with saw dust or other suitable material, and a close-fitting lid or screwed cap be placed on each end of the tube or liner. The lids should, if necessary, be held in position by adhesive tape. If the longitudinal joint of the liner is not air-tight, this should be waxed and protected by adhesive tape in the same way as the lid.

b) Samples which are not retained in a tube should be wholly covered with several layers of molten paraffin wax immediately after being removed from the sampling tool, and then placed in a suitable metal container, being tightly packed in the container with saw dust or other suitable material. The lid of the container should be held in position by adhesive tape. If the sample is very porous, it may be necessary to cover it with waxed paper before applying the molten wax.

c) A label bearing the number of the sample, preferably of the type shown in **E-2**, should be placed inside the container just under the lid. It should be placed at the top of the sample. In addition, the number of the sample should be painted on the outside of the container and the top or bottom of the sample should be indicated.

d) The liner or containers should be placed in a stout wooden box, preferably with separate partitions, and packed with saw dust, paper, etc, to prevent damage during transit.

e) It is desirable to test the undisturbed samples within two weeks of taking them from the boring or trial pit, and during the interval while awaiting transport and test, they should be stored, if possible, in a cool room, preferably with a high humidity, say 90 percent.

E-1.3 Samples of Rock

E-1.3.1 *Block Specimens* — The reference number of the sample should be recorded on it either by painting directly on the surface of the specimen, or by attaching to the specimen a small piece of surgical tape on which the number is written in Indian ink or indelible pencil. Samples should then be wrapped in several thicknesses of paper and packed in a wooden box. It is advisable to include in the wrapping a label of the type described in **E-2.**

E-1.3.2 *Cores* — In the case of small diameter drill cores, it is usual to preserve the whole core. This is best done in core boxes which are usually 1.5 m long and divided longitudinally by light battens to hold 10 rows of cores. The box should be of such depth and the compartments of such width that there can be no movement of the cores when the box is closed. The lid of the box should be adequately secured (*see* IS 4078:1980).

E-1.3.2.1 Great care should be taken in removing the core from the core barrel and in placing it in the box to see that the core is not turned end for end, but lies in its correct position. Depths below the surface of the ground should be indicated at 1.5 m intervals by writing the depth in indelible pencil on a small block of wood which is inserted in its correct position in the box. The exact depth of any change of strata should be shown in the same way. Where there is a failure to recover core, this should be recorded in the same way.

E-1.3.2.2 Where specimens are required for examination or analysis, short lengths of core may be split longitudinally by means of a special tool known as core splitter. One-half of the specimen should remain preserved in the box. Large diameter drill cores are usually too heavy to be treated in this way. As a rule, they are laid out in natural sequences for examination on the ground. Specimens required for detailed work may be treated as block specimens (*see* **E-1.3.1**).

E-1.3.2.3 The properties of hard clays and soft rocks depend to some extent on their moisture content. Representative samples should therefore be preserved by coating them completely with a thick layer of wax after removing the softened skin.

E-2. LABELLING OF SAMPLES

E-2.1 All samples should be labelled immediately after being taken from the bore hole or trial pit.

Records should be kept on a sheet of the type shown below. These sheets are serially numbered and bound in book form in duplicate. Each sheet carries a portion which may be detached along a line of perforation and which is used as a label [*see* **E-1.1** (b) *and* **E-1.2** (c)]. On this portion, the serial number of the sheet is repeated three times so that the chance of its being defaced is diminished.

BOUND AT THIS EDGE — PERFORATED HERE — TEAR OFF SLIP

No. 1100	No. 1100
SAMPLE RECORD	
Location.......................... Date................	
Boring No.......... R.L. of ground surface........	
Position of sample, from.............................	
to............................. below ground surface	No. 1100
Container No....................... Type of sample	
Disturbed/Undisturbed	
Remarks :	
Signed :	No. 1100

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SECTION 2

Determination of Density

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Indian Standard

METHODS OF TEST FOR SOILS

PART 33 DETERMINATION OF THE DENSITY IN-PLACE BY THE RING AND WATER REPLACEMENT METHOD

(Incorporating Amendment No. 1)

0. FOREWORD

0.1 With a view to establishing uniform procedures for the determination of different characteristics of soils and also to faciliate comparative studies of results, the Indian Standards Institution is bringing out this standard on methods of test for soils (IS 2720) which will be published in parts. Thirty-one parts of this standard have been published so far.

0.2 This part (Part 33) deals with the determination of dry density of soil in-place by the water replacement method using a ring. The in-place density of natural soil is needed for the determination of bearing capacity of soils, for the purpose of stability analysis of natural slopes, for the determination of pressures on underlying strata for calculation of settlement, etc. In compacted soils, the in-place density is needed to check the amount of compaction that the soil has undergone for comparison with design data. The correct estimation of the in-place density of both natural and compacted soils is, therefore, of importance.

1. SCOPE

1.1 This standard (Part 33) covers the method for determining the in-place density of a coarse grained soil including gravels, cobbles, boulders and rock by the water replacement method using a circular ring on the surface and a plastics film to retain the water (*see* Notes 1 and 2).

NOTE 1 — If desired, successive density tests may be performed as the hole is progressively deepened to determine the variation of density with depth, for example, when placing and compacting material in relatively thick layers.

NOTE 2 — A field grading of the excavated soil, may, if desired, be done while the test hole is being excavated.

1.1.1 The field dry density is determined either for the total material or for the material smaller than a specified or given size.

2. APPARATUS

2.1 Density Ring and Steel Spikes (if required) — The ring diameter shall be at least 3 to 4 times the size of the largest particle. The diameter usually ranges between 0.5 and 2.5 m in multiples of 0.5 m. The rings may be made of 4 to 8 mm mild steel sheet, either in one unit (when smaller) or in segments (when bigger) with suitable provisions for assembling in the field. The rings may be of any construction provided the inner surface is that of a right cylinder approximately 10 to 20 cm high and horizontal flanges of 10 to 25 cm suitably varying with the diameter of the ring. Stiffening flanges shall be provided to resist distortion.

2.2 Straightedge

2.3 Plastics Film

a) 0.1 mm thick, 2 to 4 m square (for small diameter rings); and
b) 0.2 mm thick, 4 to 8 m square (for large diameter rings).

2.4 Pointer Gauge Assembly and Supports — Horizontal bar with supports resting on or outside the ring, fitted with an adjustable vertical pointer and lock nut.

2.5 Quick-Setting Plaster or Sand Filled Gunny Bags

2.6 Apparatus for Delivering into the Hole, Measuring and Removing the Volume of Water Required

2.6.1 For small test holes, and for test holes located in sites which are not readily accessible, the most convenient and economical method of delivering and measuring water may be by hand or by syphon hose from a small calibrated container (water can).

2.6.2 For larger test holes in readily accessible sites, it is usually advantageous to use one or more calibrated water tanks for measuring the water when filling the hole covered with plastics film, and to provide a portable, power driven, self priming pump for removing the water after each filling. Each calibrated tank should be provided with an outlet valve and an attached volume measuring gauge. Delivery holes from such tanks shall be fitted with a valve at the delivery end so that the flow of water into the hole covered with the plastics film can be suitably controlled when the water level approaches the level of the pointer gauge.

2.6.2.1 The area of each calibrated tank should be such that the graduations on its volume measuring gauge can be easily read. Tanks used for filling holes of large capacity should have volume gauges graduated at 10 and 1 litre intervals; gauges on tanks used for filling holes of smaller capacity should be graduated at 2 and 0.2 litre intervals.

2.6.2.2 The graduations on the volume measuring gauges of calibrated tanks shall be verified. To verify the graduations proceed as follows:

a) Fill each tank with water to the top graduation on the gauge and draw off successive volumes so that the water level drops to each graduation in turn.

b) Calculate the volumes drawn off from the weight of each withdrawal and compare with the volumes read on the gauge.

2.7 A Balance — capable of weighing up to 20 kg (class C type of IS 1433:1965), sensitiveness at no load and at full load 10 g, greatest error allowed when fully loaded 20 g (*see* Note under **2.9**).

2.8 Apparatus for Moisture Content Determination — shall be in accordance with IS 2720 (Part 2): 1973.

2.9 Platform Weighing Machine — capable of weighing up to 100 kg (dial type) in accordance with IS 1435:1960, sensitiveness when fully loaded 20 g, greatest error when fully loaded 40 g (*see* Note).

NOTE — Other types of weighing scales of similar accuracy may be used.

2.10 Containers

2.11 Suitable Hand Tools — for excavating and cleaning holes in coarse soils and rocky materials, such as pick, shovel, crowbar, broom and scoop.

2.12 IS Sieves — 100-mm, 80-mm, 40-mm, 25-mm, 20-mm, 10-mm and 4.75-mm, 30 cm in diameter, as required.

2.13 Syphon Can and Measuring Cylinders

3. PROCEDURE

3.1 Approximately level the ground at the site of the test. Place the ring on the levelled ground and secure it to the surface to prevent any movement during the performance of the test.

3.2 Fill the voids between the underside of the ring and the surface with quick-setting plaster. As the plaster is setting, clean the surplus from the inside of the ring. Remove all loose material and sharp projections from the test surface.

3.2.1 If required, sand-filled gunny bags may also be placed on the flange of the ring to prevent the movement of the ring.

3.3 Set up the pointer gauge assembly so that the pointer can be removed and returned to a fixed position below the top of the ring (*see* Note). Remove the pointer gauge bar to a safe position.

NOTE — For small rings the pointer is often mounted on a datum bar supported on legs driven into the ground outside the ring. The datum should be made so that it can be removed between readings, and replaced with the pointer in precisely the same position. For large rings, which are usually more stable, it is usual to lay a small datum bar on the rim of the ring if it is a flat flange, marking the position so that the bar can be returned accurately to the same position, or the bar may be suitably clamped to the ring.

3.4 After checking for punctures, spread the plastics film over the test surface and the ring taking care to remove all the wrinkles. Replace the pointer gauge bar.

3.5 Fill the plastics film-ring assembly with water to the precise level of the pointer (*see* Note 1). While filling, ensure the film makes full contact with the test surface and the inside surface of the ring. Check for leaks (*see* Note 2). The measured volume of water used is the initial reading V_i for the test.

NOTE 1 — The required accuracy of volume measurement depends on the volume of the test hole and the diameter of the ring used. For smaller test holes and rings, record the volume to the nearest 0.2 litre. For larger test holes and rings, a lesser accuracy may suffice. Since the test cannot be easily repeated, all observations and recordings should be independently checked.

NOTE 2 — Observe the water level at the pointer gauge tip for several minutes to determine whether water is leaking through the plastic film. If leakage is occurring, repeat the volume measurement with a sound film. Do not walk upon the plastics film or drag it across the ground or sharp projections.

3.6 Remove the pointer gauge bar to a safe position. Remove the water and the plastics film, checking the ground surface for indications of leakage.

3.7 Excavate, as nearly as practicable, a cylindrical cavity within the ring using the digging tools. When excavating very coarse materials, it may be necessary to employ a mechanical device, such as a tripod with either a block and tackle or a chain hoist, for lifting large rocks from the cavity. Make the wall of the cavity as near vertical as possible; but avoid under-cutting the ring and deformation of the cavity. The movement of heavy equipment in the immediate test area should not be permitted. Leave in place any large rocks near the cavity boundary. Keep the floor and wall of the cavity as even as possible and free from sharp protrusions which may puncture the plastics film. When the desired depth (*see* Note 1 under **1.1**) has been reached, clean all loose material from the cavity. Carefully collect all the excavated material in containers (*see* Note 1) and weigh each to the nearest 0.1 kg. Sum the individual weights of the material in the containers to obtain the total weight (W_w) of the excavated material (*see* Note 2).

NOTE 1 — Use containers with close fitting lids when testing soils and absorbent rocks holding significant amounts of water. To avoid undue loss of moisture, the cover shall be kept on the container at all times when the soil is not being placed in it. In hot and dry climate, shade for the test area and a damp cloth over the container shall be provided. When the material consists predominantly of hard, non-absorbent rock of negligible moisture content, open containers are satisfactory.

NOTE 2 — If practicable large rocks in excess of scale capacity may be broken into smaller pieces. Alternatively, their volume may be determined by water displacement and their weight computed using the specific gravity of the stone. If larger rocks are broken, it shall be ensured that all fragments from each rock are weighed.

3.7.1 The gradation of particles in the excavated material may be determined, if desired, by sieving it through sieves specified in **2.12**.

3.8 When the moisture content of all or part of the material will have a significant effect on the field dry density, determine the moisture content of the soil in accordance with IS 2720 (Part 2):1973.

3.8.1 The sample for moisture content shall be representative of the whole of the soil excavated except that, if only the density of the material smaller than a given size is required, any stone coarser than this size shall first be removed. The moisture sample should be as large as is practicable and convenient. It should be collected in an airtight moisture content container by incrementally sampling the excavated soil during the course of the digging operations and after the increments of W_w (*see* **3.7**) have been weighed. In taking moisture content sub-samples of soil containing coarse rock fragments, neglect rocks larger than 80 mm if these are predominantly non-absorbent and in surface dry condition.

3.9 After checking for punctures, and taking care to remove all wrinkles, spread the plastics film properly into the cavity thus formed. Replace the pointer gauge bar.

3.10 Fill the cavity covered with plastics film with water to the precise level of the pointer as set for the initial volume measurement (*see* Note 1 under **3.5**). When delivering water to larger test cavities from calibrated tanks, run an exact number of litres of water rapidly into the film-covered cavities, from a larger tank equipped with a delivery hose capable of supplying the bulk of the water in a few minutes. A smaller tank may then be used for slowly bringing the water level to the tip of the pointer gauge and for obtaining the required accuracy of the volume measurement. While filling, loosely support the sheet away from the wall of the cavity and allow the rising water to form the film to the shape of the cavity and the ring. Check for leaks (*see* Note 2 under **3.5**). The measured volume of water used is the final reading (V_f) for the test.

3.11 The steps given in **3.1** to **3.10** complete the work specifically required at the test site to determine the in-place density.

3.12 If a soil contains particles larger than a given size and only the density of the material smaller than this size is required, proceed as in **3.12.1** to **3.12.3**.

3.12.1 Sieve the material excavated from the cavity. Determine the weight W_r and volume V_r of stones retained on the sieve.

3.12.2 The volume V_r of the stones in the sample may be determined directly by displacement of water from a graduated flask or syphon can from which the overflow can be accurately measured or by weighing the stones, or by weighing the stones in air and water, calculating their specific gravity (*see* Note) and determining their volume by dividing their weight by their specific gravity.

NOTE — For construction control, the volume of stones need not be measured every time a test is made. From the experience gained after a number of successive tests, if it is found that the specific gravity of stones from particular source is constant, a suitable value for the specific gravity may be assumed and the volume computed by obtaining the weight of stones in a wet surface-dry condition and dividing the weight by the assumed specific gravity of the stone.

3.12.3 Calculate the dry density Y_d *of the soil from the formula :*

$$Y_d = \frac{W_w - W_r}{(V - V_r)\ (1 + \frac{w}{100})}$$

where

W_w = total weight of the material excavated to form the cavity,

W_r = total weight of the portion (stones) of the excavated material retained on a given sieve,

V = volume of the cavity,

V_r = volume of the stones in the excavated material retained on the given sieve, and

w' = moisture content of material finer than the given sieve determined in accordance with IS 2720 (Part 2) : 1973.

If there is a large proportion of stone in the sample, the calculated density value for the fraction passing the given sieve may lack physical significance.

4. CALCULATIONS

4.1 Calculate the volume of the cavity V from the formula :

$$V = V_f - V_i$$

where

V_f = final volume reading (*see* **3.10**), and

V_i = initial volume reading (*see* **3.5**).

4.2 Calculate the wet density of the soil Y from the formula :

$$Y = \frac{W_w}{V}$$

where

W_w = weight of the wet material from the cavity (*see* **3.7**), and

V = volume of the cavity (*see* **4.1**).

4.3 Calculate the dry density of the soil Y_d from the formula :

$$Y_d = \frac{Y \times 100}{(100+w)}$$

where

Y = wet density of the soil (*see* **4.2**), and

w = the moisture content in percent of the soil determined in accordance with IS 2720 (Part 2) : 1973 (*see also* **3.8.1** and **3.12**).

5. REPORTING OF RESULTS

5.1 The results of the test shall be suitably reported and the report shall specifically mention about the following. A recommended proforma for the record of test results is given in Appendix A :

a) The date of the test,
b) The test location,
c) The elevation of the test,
d) The soil description,
e) The method used,
f) The fraction of the soil for which the density has been determined, and
g) The dry density in kg/m³ to the nearest 10 kg/m³ or in g/cm³ to the second place of decimals.

APPENDIX A

(*Clause* 5.1)

DETERMINATION OF DENSITY OF SOIL IN-PLACE BY RING AND WATER REPLACEMENT METHOD

Project:
Date:
Test location :
Soil description :
Fraction of soil for which density is determined :

Test No. :
Elevation of test location :

DETERMINATION OF VOLUME OF CAVITY

Initial volume reading (with ring only) V_i			Final volume reading (with ring and cavity) V_f		
Initial reading	Final reading	Difference V_i	Initial reading	Final reading	Difference V_f

Volume of cavity $V = V_f - V_i$

DENSITY OF OVERALL MATERIAL

No. of container						
Weight of container + wet material						
Weight of container						
Weight of wet material						

Total weight of wet material $= W_w$

Volume of cavity $= V$

Wet density $\gamma = \frac{W_w}{V}$

Dry density $\gamma_d = \frac{100\,\gamma}{(100 + w)}$

Moisture content of portion for which density is determined, w, percent

GRADATION OF EXCAVATED MATERIAL

IS Sieve Size (mm)	100	80	40	20	10	4.75
Percent retained						

Gradation of stones larger than 100 mm	Size				
	Percent of total material				

Indian Standard

METHODS OF TEST FOR SOILS

PART 34 DETERMINATION OF DENSITY OF SOIL IN-PLACE BY RUBBER-BALLOON METHOD

(Incorporating Amendment No. 1)

0. FOREWORD

0.1 With a view to establishing uniform procedures for the determination of different characteristics of soils and also for facilitating comparative studies of the results, the Indian Standards Institution is bringing out this standard on methods of test for soils (IS 2720) which is being published in parts. This part deals with the procedure for the determination of the density in-place of compacted or firmaly bonded soil using a rubber-balloon apparatus. The in-place density of natural soil is needed for the determination of bearing capacity of soils, for the purpose of stability analysis of natural slopes, for the determination of pressures on underlying strata for calculation of settlement, etc. In compacted soils the in-place density is needed to check the amount of compaction that the soil has undergone for comparison with design data.

1. SCOPE

1.1 This standard (Part 34) covers the procedure for determining the density in-place of compacted or firmly bonded soil using a rubber-balloon apparatus. This method is not suitable for very soft soils which will deform under slight pressure or in which the volume of the hole cannot be maintained at a constant value.

2. APPARATUS

2.1 Calibrated Vessel — designed to contain a liquid with a relatively thin, flexible elastic membrane (rubber-balloon) for measuring the volume of the test hole under the conditons of this method (*see* Fig. 1). The calibrated equipment may also be a graduated glass cylinder provided with a suitable guard and guard base with provision for attachment of the elastic membrane without leakage. The graduations shall be such that the volumes can be read accurate to 5 ml. The apparatus shall be equipped so that an externally controlled pressure or partial vacuum can be applied to the contained liquid (*see* Note 1). Suitable provision shall also be made for the measurement of the pressure applied. It shall be of such weight and size that it will not cause distortion of the excavated test hole and adjacent test area during the performance of the test. Provision may be made for placing weights (surcharge) on the apparatus, if necessary, when the weight of the apparatus itself is not sufficient to hold it down during the test. The flexible membranes shall be of such sizes as to fill the test holes completely without wrinkles or folds when inflated within the test holes, and their strength shall be sufficient to withstand such pressures as are necessary to ensure complete filling of the test holes (*see* Note 2).

NOTE 1 — Any arrangement for providing pressure and partial vacuum which does not impair the portability of the apparatus may be used. A convenient method is to use a pressure actuator bulb similar to the one used in the blood-pressure measuring apparatus used by doctors. By providing suitable valves and other arrangements the same actuator can be used for creating the required vacuum.

NOTE 2 — The description and requirements given in **2.1** are intended to be non-restrictive. Any apparatus using a flexible membrane (rubber) and liquid that can be used to measure the volume of a test hole in soil under the conditions of this method to an accuracy within 1.0 percent is satisfactory.

2.2 Balances — A balance or scale of approximately 20 kg capacity accurate to 1 g and a balance of 2 kg capacity accurate to 0.2 g.

2.3 Apparatus for the Determination of Moisture Content — shall be in accordance with IS 2720 (Part 2) : 1973.

2.4 Miscellaneous Equipment — Small pick, chisels, spoons for digging test holes, plastic bags, buckets with lids, or other suitable metal containers that can be closed for retaining the soil taken from the test holes, thermometer for determining temperature of water, small paint brush.

3. CALIBRATION CHECK OF VOLUME INDICATOR

3.1 Verify the procedure to be used and the accuracy of the volume indicator by using the apparatus to measure containers or moulds of determinable volume that dimensionally simulate test holes that will be used in the field. The

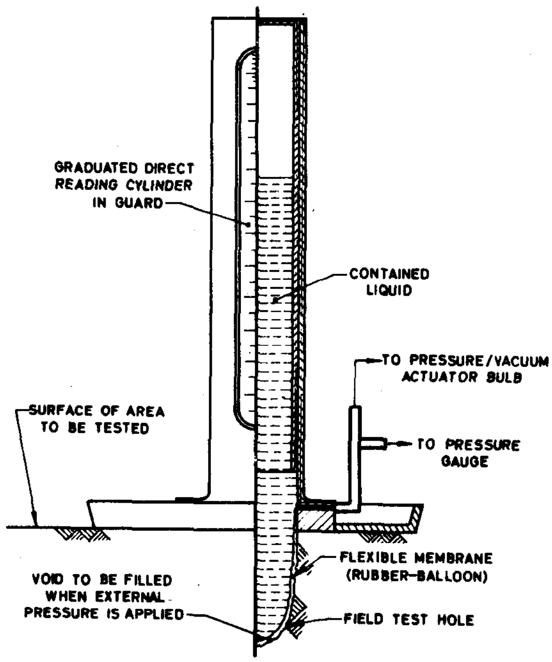


FIG. 1 SCHEMATIC DRAWING OF CALIBRATED VESSEL INDICATING PRINCIPLE

apparatus and procedure shall be such that these volumes shall be measured to within 1.0 percent (*see* Note). Containers of different volumes shall be used so that the calibration check of the volume indicator covers the range of anticipated test hole sizes.

NOTE — The 100-mm and 150-mm moulds described in IS 2720 (Part 7):1980 and IS 2720 (Part 8):1983 or other moulds prepared to simulate actual test holes may be used. Where several sets of apparatus are used, it may be desirable to cast duplicates of actual test holes. These sets should represent the range of sizes and irregularities in the walls of test holes that will be encountered. These fabricated holes may be used as standards for the calibration check of the volume indicator. This may be accomplished by forming plaster of Paris negatives in the test holes and using these as forms for Portland cement concrete castings. After removing the plaster of Paris negative from the concrete casting, the inside surface of the fabricated holes should be sealed watertight and their volume determined as indicated in **3.1**.

3.1.1 *Volumes of Containers* — Determine the weight of water, in grams, required to fill one of the containers. Slide a glass plate carefully over the top surface of the container in such a manner as to ensure that the container is filled completely with water. Determine the temperature of the water in the container. A thin film of cup grease smeared on the top surface of the container shall make a watertight joint between the glass plate and the top of the container. Calculate the volume of the container, in millilitres, by multiplying the weight of water, in grams, used to fill the container by the unit volume of water, in millilitres per gram, at the observed temperature, taken from Table 1. Repeat this procedure until three values are secured for the volume of the container having maximum range of variation of 3 ml. Repeat the procedure for each of the containers to be used in the calibration check.

3.1.2 *Calibration Check Tests* — Place the rubber-balloon apparatus filled with water to the required level (*see* Note 1) on a relatively smooth horizontal surface and take an initial reading on the volume indicator. Transfer the apparatus to one of the containers and take the reading on the volume indicator when the rubber-balloon completely fills the container (*see* Notes 2 and 3). Apply pressure to the liquid in the apparatus until there is no further change indicated on the volume indicator. Note and record the pressure. Where necessary, add weight (surcharge) to the apparatus to prevent it from rising (*see* Note 4). Note and record the total amount of weights added. The difference between the initial and final readings of the volume indicator is the indicator volume value for the container. The membrane may be withdrawn from the container by applying a partial vacuum to the liquid in the apparatus. Repeat the procedure for the other containers.

NOTE 1 — Water may be used as fill liquid and in freezing temperatures, anti-freeze fluids may be used in the calibrated vessel or cylinder.

NOTE 2 — If the calibration container or mould is airtight, it may be necessary to provide an air escape, since the rubber membrane can entrap air within the container and cause erroneous volume measurement. After the volume of the container has been determined with water and prior to the insertion of the rubber-balloon, small air escape-holes may be provided by placing lengths of small diameter string over the edge of the container and down the inside wall slightly beyond the bottom centre. This will permit air leakage during the filling of the container with the membrane. If such a procedure is necessary in the laboratory, it may be necessary to use a similar procedure on tightly-bounded soil in the field.

NOTE 3 — Before any measurements are made, it may be necessary to distend the rubber-balloon and remove air bubbles adhering to the inside of the membrane by kneading.

NOTE 4 — In field tests the additional weights (surcharge) will increase the stress in the unsupported soil surrounding the test hole and will tend to cause it to deform. The stress may be reduced by using a base plate.

4. PROCEDURE

4.1 Prepare the surface of the test hole site so that it is reasonably plane. Set the apparatus on the test hole site and take an initial reading on the volume indicator of the calibrated vessel using the same pressure on the liquid in the vessel and the same amount of surcharge weight as was used in the calibration check. After taking this initial reading

TABLE 1 VOLUME OF WATER PER GRAM BASED ON TEMPERATURE

(*Clause* 3.1.1)

TEMPERATURE °C	VOLUME OF WATER ml/g
12	1.000 48
14	1.000 73
16	1.001 03
18	1.001 38
20	1.001 77
22	1.002 21
24	1.002 68
26	1.003 20
28	1.003 75
30	1.004 35
32	1.004 97
34	1.005 63
36	1.006 33
38	1.007 06
40	1.007 86
42	1.008 57
44	1.009 39
46	1.010 31
48	1.011 12
50	1.012 04

on the volume indicator, scribe the outline of the apparatus on the test hole site. Record the pressure used, the amount of the surcharge, and the initial volume reading. If the apparatus was calibrated with a base plate, the base plate shall remain in-place throughout the field test.

4.2 Remove the apparatus from the test hole site and dig a hole centered within the outline scribed for the apparatus. Exercise care in digging the test hole so that soil around the top edge of the hole is not disturbed. Place all the soil removed from the test hole in an airtight container for weight and moisture content determinations. The test hole shall be of the minimum volume given in Table 2. Larger holes will provide improved accuracy and shall be used, where practicable. The dimensions of the test holes are related to the apparatus design and the pressure used. In general, the dimensions shall approximate those used in the calibration check procedure.

4.3 After the test hole has been dug, place the apparatus over the test hole in the same position used for the initial reading and inflate the flexible membrane in the hole, allowing air from the hole to escape without getting entrapped between the inner surface of the test hole and the flexible membrane (*see* Note 2 under **3.1.2**). Apply the same surcharge weight and pressure on the liquid in the vessel as used during the calibration check procedure. Take and record the reading on the volume indicator. The difference between this reading and the initial reading obtained in **4.1** is the volume of the test hole (*see* Note). Note the temperature of the water used and correct the volume for temperature, taking into consideration the temperature at which the apparatus was calibrated. After the test, pump the water and flexible membrane back into the cylinder by applying vacuum.

NOTE — Attention is called to instances in weak soils, where the pressure applied to the liquid in the vessel may deform the test hole to such an extent as to give an erroneous volume. In such instances, the apparatus shall be re-calibrated using less surcharge weight and less pressure on the liquid in the vessel, or it may be necessary to resort to another method such as that given in IS 2720 (Part 28) : 1974.

4.4 Determine the weight of all the moist soil removed from the test hole to the nearest 5 g. Mix this soil thoroughly, select a sample in accordance with Table 2 for the determination of moisture content and determine its weight to the nearest 0.1g. Dry the moisture content sample to a constant weight at a temperature 100 to 110° C and determine the dry weight to the nearest 0.1 g [*see also* IS 2720 (Part 2) : 1973].

TABLE 2 MINIMUM FIELD TEST HOLE VOLUMES AND MINIMUM MOISTURE CONTENT SAMPLES BASED ON MAXIMUM SIZE OF PARTICLE

(*Clauses* 4.2 *and* 4.4)

SL NO.	MAXIMUM PARTICLE SIZE	TEST HOLE VOLUME, *Min*	MOISTURE CONTENT SAMPLE, *Min*
(1)	(2)	(3)	(4)
	mm	cm^3	g
i)	4.75	700	200
ii)	10	1 400	300
iii)	20	2 100	500
iv)	40	2 800	1 000
v)	63	3 800	1 500

5. CALCULATIONS

5.1 Calculate the moisture content, w, of the soil as follows :

$$w = \frac{\text{weight of moisture}}{\text{weight of dry soil}} \times 100$$

5.2 Calculate the wet unit weight, Y_m, of the soil removed from the test hole, in g/cm^3, as follows :

$$Y_m = \frac{\text{weight of moist soil}}{\text{volume of test-hole}}$$

5.3 Calculate the dry unit weight Y_d, of the soil removed from the test hole, in g/cm^3, as follows :

$$Y_d = \left(\frac{Y_m}{w + 100}\right) \times 100$$

Indian Standard
METHODS OF TEST FOR SOILS
PART 29 DETERMINATION OF DRY DENSITY OF SOILS IN-PLACE BY THE CORE-CUTTER METHOD

(First Revision)

0. FOREWORD

0.1 With a view to establishing uniform procedure for the determination of different characteristics of soils and also for facilitating comparative studies of the results, the Indian Standards Institution has brought out this standard on methods of test for soils (IS 2720) which is published in parts. This part deals with the determination of dry density of soil in-place by using a core-cutter. The in-place density of soil is needed for stability analysis, for the determination of the degree of compaction of compacted soil, etc. The core-cutter method covered by this part is suitable for fine-grained soils free from aggregations. It is less accurate than the sand-replacement method and is not recommended, unless speed is essential or unless the soil is well compacted.Other parts relating to in-place determination of density of soils are :

Part 28 Determination of dry density of soils in-place by the sand replacement method

Part 33 Determination of the density in-place by the ring and water replacement method

Part 34 Determination of density of soil in-place by the rubber-balloon method

0.1.1 This standard was first published in 1966. In this revision, the test has been made applicable to soil 90 percent of which passes the 4.75-mm IS Sieve. The dimensions and requirements of the core-cutter have been modified. Detailed requirement for the steel rammer required for the test have been spelt out.

0.2 In the formulation of this standard, due weightage has been given to international co-ordination among the standards and practices prevailing in different countries, in addition to relating it to the practices in the field in this country. This has been met by basing the standard on the following publications :

BS 1377 : 1974 Methods of testing soils for civil engineering purposes. British Standards Institution.

INDIA MINISTRY OF IRRIGATION AND POWER CBIP Publication No. 42. Standards for testing soils, 1963. Central Board of Irrigation and Power, Delhi.

1. SCOPE

1.1 This standard (Part 29) covers the method for the determination of the in-place density of fine-grained natural or compacted soils free from aggregates using a core-cutter.

1.1.1 For the purpose of the tests described in this standard, a soil shall be termed as fine-grained soil if not less than 90 percent of it passes a 4.75-mm IS Sieve.

2. APPARATUS

2.1 Cylindrical Core-Cutter — of seamless steel tube, 130 mm long (*see* Note 1) and 10 cm internal diameter, with a wall thickness of 3 mm, bevelled at one end, of the type illustrated in Fig. 1. The cutter shall be kept properly greased or oiled.

NOTE 1 — *Length of Cutter* — If the average density over a smaller depth is required, then the appropriate length of cutter should be used.

NOTE 2 — Where situations permit, for quality control purposes smaller size cutters have also been used.

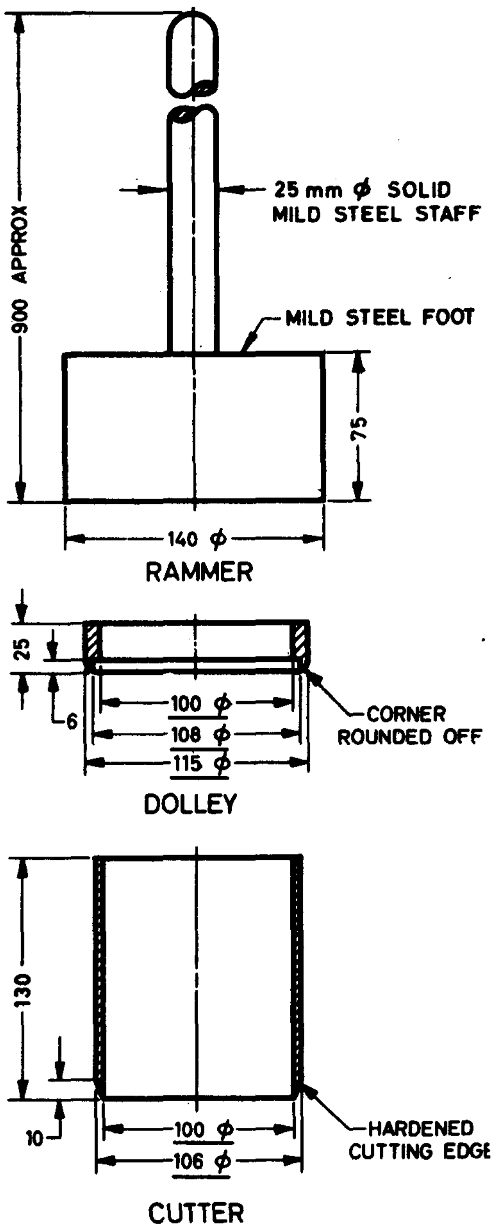
2.2 Steel Dolley — 2.5 cm high and 10 cm internal diameter with a wall thickness of 7.5 mm with a lip to enable it to be fitted on top of the core-cutter (*see* Fig. 1).

2.3 Steel Rammer — With solid mild steel foot 140 mm diameter and 75 mm height with a concentrically screwed 25 mm diameter solid mild steel staff. The overall length of the rammer including the foot as well as the staff should be approximately 900 mm. The rammer (foot and staff together) should weigh approximately 9 kg (*see* Fig. 1).

2.4 Balance — Accurate to 1 g.

2.5 Palette Knife — A convenient size is one having a blade approximately 20 cm long and 3 cm wide.

2.6 Steel Rule



NOTE 1 — These designs have been found satisfactory, but alternative designs may be employed provided that the essential requirements are fulfilled.

NOTE 2 — Essential dimensions are underlined. (Tolerance on all essential dimensions shall be ±0.25 mm).

All dimensions in millimetrès.

FIG. 1 CORE-CUTTER APPARATUS FOR SOIL DENSITY DETERMINATION

2.7 Grafting Tool or Spade or Pick Axe

2.8 Straight Edge — A steel strip about 30 cm long, 2.5 cm wide and 3 to 5 mm thick, with one bevelled edge will be suitable.

2.9 Apparatus for Extracting Samples from the Cutter — Optional.

2.10 Apparatus for Determination of Water Content — In accordance with IS 2720 (Part 2) : 1973.

3. PROCEDURE

3.1 The internal volume (V_c) of the core-cutter in cubic centimetres shall be calculated from its dimensions which shall be measured to the nearest 0.25 mm.

3.2 The cutter shall be weighed to the nearest gram (W_c).

3.3 A small area, approximately 30 cm square of the soil layer to be tested shall be exposed and levelled. The steel dolley shall be placed on top of the cutter and the latter shall be rammed down vertically into the soil layer until only about 15 mm of the dolley protrudes above the surface, care being taken not to rock the cutter (*see* Note). The cutter shall then be dug out of the surrounding soil, care being taken to allow some soil to project from the lower end of the cutter. The ends of the soil core shall then be trimmed flat to the ends of the cutter by means of the straight edge.

NOTE — The cutting edge should be kept sharp. The cutter should not be used in stony soils.

3.4 The cutter containing the soil core shall be weighed to the nearest gram (W_s).

3.5 The soil core shall be removed from the cutter and a representative sample shall be placed in an air-tight container and its water content (w) determined as in IS 2720 (Part 2) : 1973.

NOTE — It is necessary to make a number of repeat determinations (at least three) and to average results, since the dry density of the soil varies appreciably from point to point. The number of determinations should be such that an additional one would not alter the average significantly.

4. CALCULATIONS

4.1 The bulk density Y_b; that is, the weight of the wet soil per cubic centimetre shall be calculated from the following formula :

$$Y_b = \frac{W_s - W_c}{V_c}, \text{g/cm}^3$$

where

W_s = weight of soil and core-cutter in g,

W_c = weight of core-cutter in g, and

V_c = volume of core-cutter in cm^3.

4.2 The dry density Y_d, that is, the weight of the dry soil per cubic centimetre shall be calculated from the following formula:

$$Y_d = \frac{100\ Y_b}{100 + w},\ \text{g/cm}^3$$

where

Y_b = bulk density (*see* **4.1**), and

w = water content of the soil (percent) to two significant figures.

5. REPORTING OF RESULTS

5.1 The results of the test shall be recorded in a suitable form. A recommended proforma for the record of the results of this test is given in Appendix A.

5.2 The following values shall also be reported :

a) Dry density of the soil to second place of decimal in g/cm^3, and

b) Water content of the soil (percent) to two significant figures.

APPENDIX A

(*Clause* 5.1)

DETERMINATION OF DRY DENSITY OF SOIL IN-PLACE (CORE-CUTTER METHOD)

A-1. The test results shall be tabulated as follows:

Project : Tested by :

Location : Date :

1. Determination No.	1	2	3
2. Weight of core-cutter + wet soil (W_s), in g			
3. Weight of core-cutter (W_c), in g			
4. Weight of wet soil ($W_s - W_c$), in g			
5. Volume of core-cutter (V_c), in cm^3			
6. Bulk density ($Y_b = \frac{W_s - W_c}{V_c}$), in g/cm^3			
7. Water content container No.			
8. Weight of container with lid (W_1), in g			
9. Weight of container with lid and wet soil (W_2), in g			
10. Weight of container with lid and dry soil (W_3), in g			
11. Water content (w), in percent $w = \frac{W_2 - W_3}{W_3 - W_1} \times 100$			
12. Dry density ($Y_d = \frac{100 + Y_b}{100 + w}$), in g/cm^3			

Indian Standard
METHODS OF TEST FOR SOILS
PART 28 DETERMINATION OF DRY DENSITY OF SOILS, IN-PLACE, BY THE SAND REPLACEMENT METHOD

(First Revision)

0. FOREWORD

0.1 With a view to establishing uniform procedures for the determination of different characteristics of soils and also for facilitating comparative studies of the results, the Indian Standards Institution is bringing out this Standard on methods of test for soils (IS 2720) which will be published in parts. This part deals with the determination of dry density of soil, in-place, by the sand replacement method. The in-place density of natural soil is needed for the determination of bearing capacity of soils, for the purpose of stability analysis of natural slopes, for the determination of pressures on underlying strata for calculation of settlement, etc. In compacted soils the in-place density is needed to check the amount of compaction that the soil has undergone for comparison with design data. The correct estimation of the in-place density of both natural and compacted soils is therefore of importance.

0.1.1 This standard was originally published in 1966. In this revision, the sieve size for defining fine-grained soils has been changed to 2 mm. An appendix has been added for the determination of water content and dry density of medium- and coarse-grained soils containing appreciable gravel fraction.

0.2 This standard is divided into two sections. Section 1 prescribes the method suitable for fine- and medium-grained soils using the small sand pouring cylinder; Section 2 lays down the method which uses the large sand pouring cylinder and is suitable for fine- medium- and coarse-grained soils containing stones which make the test of Section 1 difficult to perform.

0.2.1 For the purpose of tests described in this standard soils shall be grouped as shown below :

Fine-grained soils	Soils containing not less than 90 percent passing a 2.0-mm IS Sieve (*see* IS 460 : 1985)
Medium-grained soils	Soils containing not less than 90 percent passing a 20-mm IS Sieve (*see* IS 460 : 1985)
Coarse-grained	Soils containing not less than 90 percent passing a 40-mm IS Sieve (*see* IS 460 : 1985)

0.3 In the formulation of this standard due weightage has been given to international co-ordination among the standards and practices prevailing in different countries in addition to relating it to the practices in the field in this country. This has been met by basing the standard on the following publications :

BS 1377 : 1961 Methods of testing soils for civil engineering purposes. British Standards Institution.

INDIA. MINISTRY OF IRRIGATION AND POWER. CBIP Publication No. 42. Standards for testing soils. 1963. Central Board of Irrigation and Power, New Delhi.

SECTION 1 METHOD SUITABLE FOR FINE- AND MEDIUM- GRAINED SOILS: SMALL POURING CYLINDER METHOD

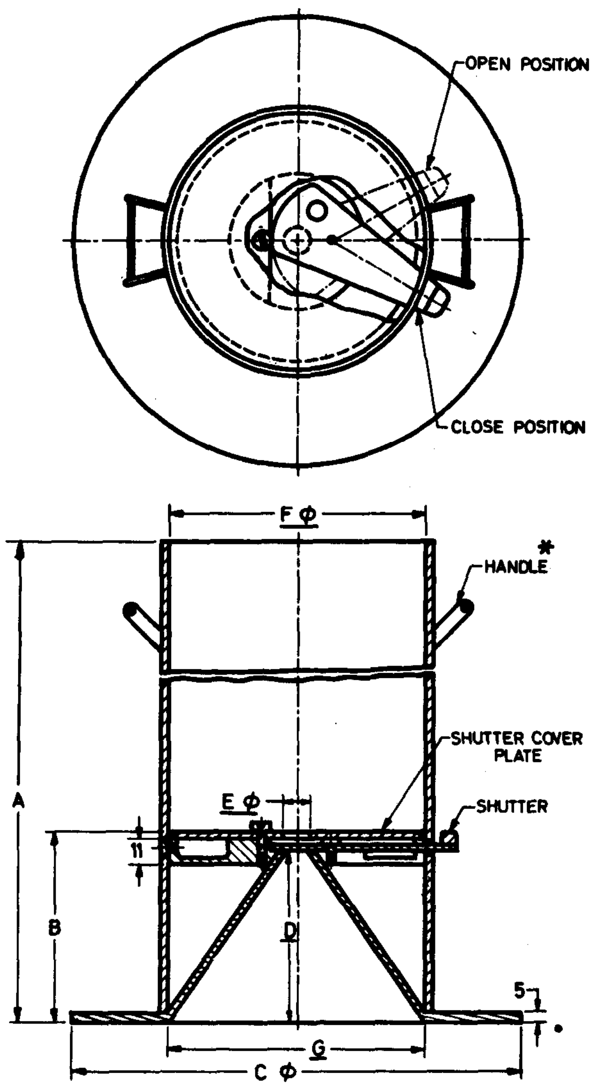
1. SCOPE

1.1 This method covers the determination, in-place, of the dry density (in g/cm^3 or kg/m^3) of natural or compacted fine-and medium-grained soils for which a small sand-pouring cylinder is used. The method is applicable to layers not exceeding 150 mm in thickness (*see* Note).

NOTE — With granular materials having little or no cohesion, particularly when they are wet, there is a danger of errors in the measurement of dry density by this method. These errors are caused by the slumping of the sides of the excavated density hole and always result in an over-estimation of the density.

2. APPARATUS

2.1 Small Sand-Pouring Cylinder — similar in essential details to that shown in Fig. 1.



*The handle may be required for large pouring cylinder only.

Pouring Cylinder Size	*A*	*B*	*C*	*D*	*E*	*F*	*G*	*Capacity in Litres*
Small (for fine- and medium-grained soils)	380	85	200	75	13 ±0.1	115	115	3
Large (for fine- medium- and coarse-grained soils)	610	175	350	160	25 ±0.1	215	215	16.5

NOTE 1 — This design has been found satisfactory, but alternative designs may be employed, provided that the essential requirements are fulfilled.

NOTE 2 — Essential dimensions are underlined.

NOTE 3 — Tolerance on essential dimensions ± 1 mm.

All dimensions in millimetres.

FIG. 1 SAND-POURING CYLINDER FOR THE DETERMINATION OF DENSITY

2.2 Tools for Excavating Holes — suitable tools, such as a scraper tool similar to that shown in Fig. 2 to make a level surface: bent spoon, dibber shown in Fig. 3.

2.3 Cylindrical Calibrating Container — with an internal diameter of 100 mm and an internal depth of 150 mm (*see* Note 1 under **4.2.2**) of the type illustrated in Fig. 4 fitted with a flange approximately 50 mm wide and about 5 mm thick surrounding the open end. The volume of the container should be given to an accuracy of 0.25 percent.

2.4 Balance — accurate to 1 g.

2.5 Plane Surface : Glass or Perspex Plate or Other Plane Surface — about 450 mm square and 9 mm thick or larger.

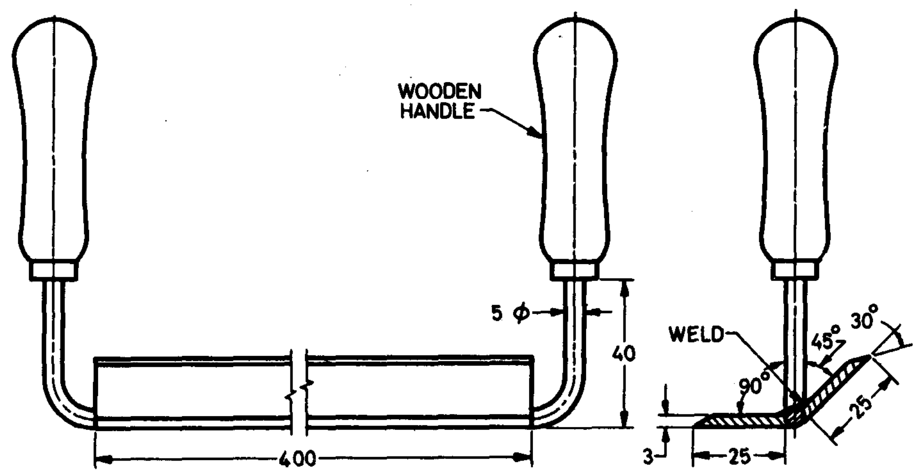
2.6 Metal Containers — to collect excavated soil. A convenient size is one about 150 mm diameter and 200 mm deep with a removable cover.

2.7 Cylindrical Steel Core-Cutter — of steel, 127.4 ± 0.1 mm long and 100 mm ± 0.1 mm internal diameter with a wall thickness of 3 mm bevelled at one end. One suitable type is illustrated in Fig. 5. The cutter shall be kept adequately greased.

2.8 Metal Tray with Hole — 300 mm square and 40 mm deep with a 100 mm hole in the centre.

3. MATERIAL (SAND)

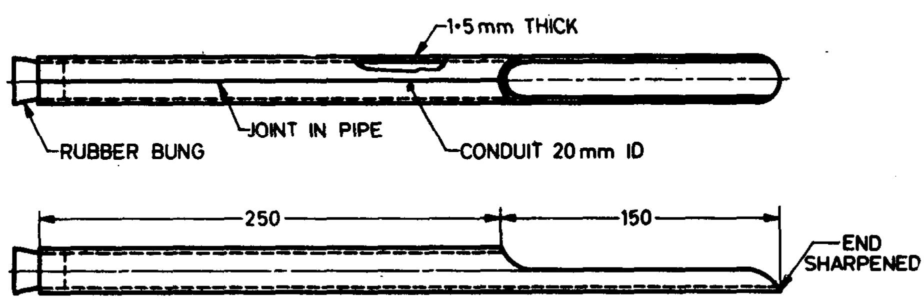
3.1 Clean, uniformly graded natural sand passing the 1.00-mm IS Sieve and retained on 600-micron IS Sieve shall be used. It shall be free from organic



NOTE — This design has been found satisfactory, but alternative designs may be employed.

All dimensions in millimetres.

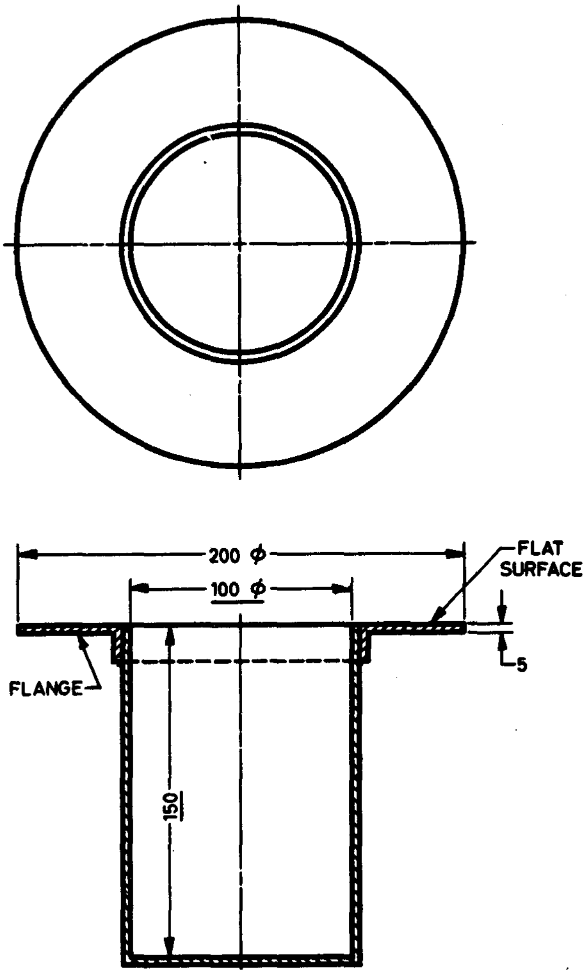
FIG. 2 SCRAPER FOR LEVELLING SURFACE OF SOIL



NOTE — This design has been found satisfactory, but alternative designs may be employed.

All dimensions in millimetres.

FIG. 3 DIBBER FOR DIGGING DENSITY HOLES



NOTE 1 — This design has been found satisfactory, but alternative designs may be employed, provided that the essential requirements are fulfilled.

NOTE 2 — Essential dimensions are underlined.

NOTE 3 — Tolerance on essential dimensions ±0.1 mm.

All dimensions in millimetres.

FIG. 4 CALIBRATING CONTAINER FOR USE WITH SMALL POURING CYLINDER

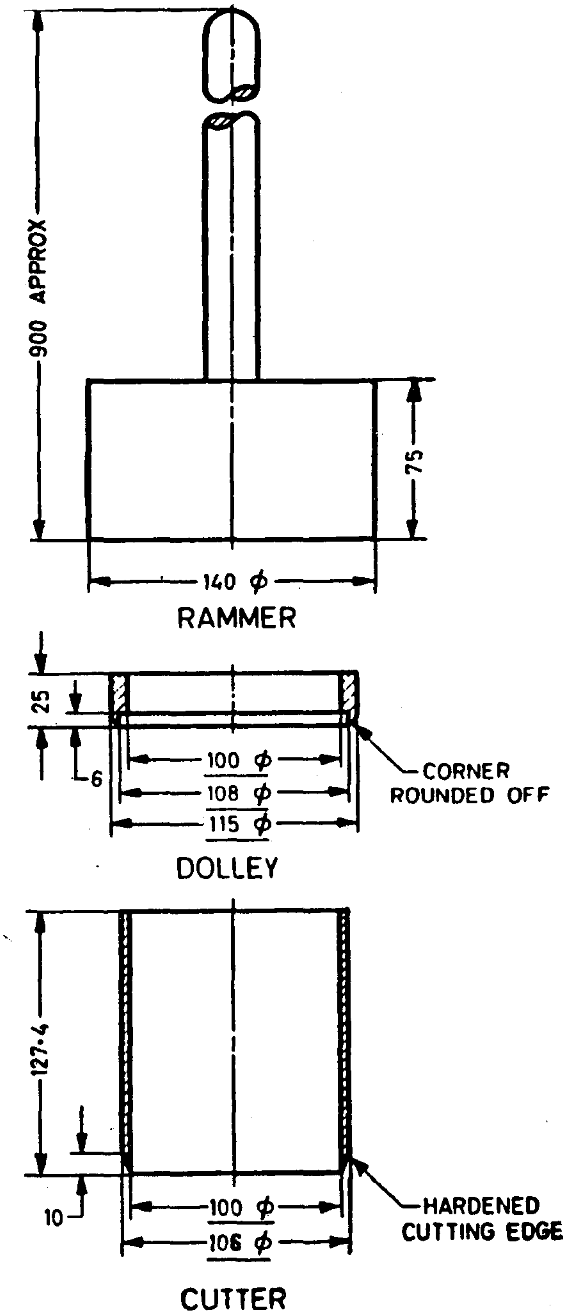
matter, and shall have been oven dried and stored for a suitable period to allow its water content to reach equilibrium with atmospheric humidity (*see* Note).

NOTE — Generally a storage period, after oven drying, of about 7 days is sufficient for the water content of the sand to reach equilibrium with the atmospheric humidity. The sand should not be stored in air-tight containers and should be thoroughly mixed before use. If sand is salvaged from holes in compacted soils after carrying out the test, it is advisable to sieve, dry and store this and again before it is used in further sand replacement tests.

4. PROCEDURE

4.1 Calibration of Apparatus

4.1.1 The method given in **4.1.1.1** to **4.1.1.4** shall be followed for the determination of the weight of sand in the cone of the pouring cylinder.



NOTE 1 — These designs have been found satisfactory, but alternative designs may be employed, provided that the essential requirements are fulfilled.

NOTE 2 — Essential dimensions are underlined.

NOTE 3 — Tolerance on essential dimensions ± 0.1 mm.

All dimensions in millimetres

FIG. 5 CORE-CUTTER APPARATUS FOR SOIL DENSITY DETERMINATION

4.1.1.1 The pouring cylinder shall be filled so that the level of the sand in the cylinder is within about 10 mm of the top. Its total initial weight (W_1) shall be found and shall be maintained constant throughout the tests for which the calibration is used. A volume of sand equivalent to that of the excavated hole in the soil (or equal to that of the calibrating container) (*see* Note 1 under **4.2.2**) shall be allowed to run out of the cylinder under gravity. The shutter on the pouring cylinder shall then be closed and the cylinder placed on a plane surface, such as a glass plate.

4.1.1.2 The shutter on the pouring cylinder shall be opened and sand allowed to run out. When no further movement of sand takes place in the cylinder, the shutter shall be closed and the cylinder removed carefully.

4.1.1.3 The sand that has filled the cone of the pouring cylinder (that is, the sand that is left on the plane surface) shall be collected and weighed to the nearest gram.

4.1.1.4 These measurements shall be repeated at least three times and the mean weight (W_2) taken.

4.1.2 The method described in **4.1.2.1** to **4.1.2.3** shall be followed for the determination of the bulk density of the sand (Y_s).

4.1.2.1 The internal volume (V) in ml of the calibrating container shall be determined from the weight of water contained in the container when filled to the brim (*see* Note 1 under **4.2.2**). The volume may also be calculated from the measured internal dimensions of the container.

4.1.2.2 The pouring cylinder shall be placed concentrically on the top of the calibrating container after being filled to the constant weight (W_1) as in **4.1.1.1**. The shutter on the pouring cylinder shall be closed during this operation. The shutter shall be opened and sand allowed to run out. When no further movement of sand takes place in the cylinder, the shutter shall be closed. The pouring cylinder shall be removed ~~and~~ weighed to the nearest gram.

4.1.2.3 These measurements shall be repeated at least three times and the mean weight (W_3) taken (*see* Note).

NOTE — Since variations in atmospheric humidity affect the water content of the sand, and hence its bulk density, the calibration should be made (or at least checked) during each day's work. To overcome the effects of slight variations in grading and particle shape between batches of sand, each batch should be sampled and calibrated.

4.2 Measurement of Soil Density — The following method shall be followed for the measurement of soil density.

4.2.1 A flat area, approximately 450 mm square, of the soil to be tested shall be exposed and trimmed down to a level surface preferably with the aid of the scraper tool.

4.2.2 The metal tray with a central hole shall be laid on the prepared surface of the soil with the hole over the portion of the soil to be tested. The hole in the soil shall then be excavated using the hole in the tray as a pattern, to the depth of the layer to be tested up to a maximum of 150 mm (*see* Note 1). The excavated soil shall be carefully collected, leaving no loose material in the hole and weighed to the nearest gram (W_w). The metal tray shall be removed before the pouring cylinder is placed in position over the excavated hole.

The following alternative method shall be used for fine-grained cohesionless soils :

> The steel core cutter shall be pressed evenly and carefully into the soil until its top edge is flush with the levelled surface. Soil to a depth of 100 mm (*see* Note 1) within the core cutter shall then be excavated by means of suitable tools. The excavated soil shall be carefully collected and weighed to the nearest gram (W_w). The core cutter shall remain in position during the remainder of the testing procedure.

NOTE 1 — If for any reason it is necessary to excavate the holes to depths other than 150 mm, the calibrating container should be replaced by one, the depth of which is the same as the hole excavated or its effective depth should be reduced to that of the hole excavated.

NOTE 2 — Care shall be taken in excavating the hole to see that the hole is not enlarged by levering the dibber against the side of the hole, as this will result in lower densities being recorded.

4.2.3 The water content (W) of the excavated soil shall be determined by the method specified in IS 2720 (Part 2) : 1973. Alternatively the whole of the excavated soil may be dried and weighed (W_d).

4.2.4 The pouring cylinder filled to the constant weight (W_1) as in **4.1.1** shall be so placed that the base of the cylinder covers the hole concentrically. The shutter on the pouring cylinder shall be closed during this operation. The shutter shall then be opened and sand allowed to run out into the hole. The pouring cylinder and the surrounding area shall not be vibrated during this period. When no further movement of sand takes place the shutter shall be closed. The cylinder shall be removed and weighed to the nearest gram (W_4) (*see* Note).

NOTE — It is necessary to make a number of repeated determinations (at least three) and to average the results, hence the dry density of the soil varies appreciably from point to point. The number of determinations should be such that an additional one would make no significant difference to the average.

5. CALCULATIONS

5.1 The weight of sand (W_a) in g, required to fill the calibrating container shall be calculated from the following formula :

$$W_a = W_1 - W_3 - W_2$$

where

W_1 = weight of pouring cylinder and sand before pouring into calibrating container in g,

W_3 = mean weight of cylinder with residual sand after pouring into calibrating container and cone in g, and

W_2 = mean weight of sand in cone in g.

5.2 The bulk density of the sand (Υ_s) in kg/m³ shall be calculated from the following formula :

$$\Upsilon_s = \frac{W_a}{V} \times 1\,000$$

where

V = volume of calibrating container in ml.

5.3 The weight of sand (W_b) in g, required to fill the excavated hole shall be calculated from the following formula :

$$W_b = W_1 - W_4 - W_2$$

where

W_1 = weight of cylinder and sand before pouring into hole in g,

W_4 = weight of cylinder and sand after pouring into hole and cone in g, and

W_2 = mean weight of sand in cone in g.

5.4 The bulk density Υ_b, that is, the weight of the wet soil per cubic metre shall be calculated from the following formula :

$$\Upsilon_b = \frac{W_w}{W_b} \times \Upsilon_s \text{ kg/m}^3$$

where

W_w = weight of soil excavated in g,

W_b = weight of sand required to fill the hole in g, and

Υ_s = bulk density of sand in kg/m³.

5.5 The dry density Υ_d, that is, the weight of the dry soil shall be calculated from the following formula :

$$\Upsilon_d = \frac{100\,\Upsilon_b}{100 + w} \text{ kg/m}^3$$

or

$$\Upsilon_d = \frac{W_d}{W_b} \times \Upsilon_s \text{ kg/m}^3$$

where

w = water content of the soil in percent,

W_d = weight of dry soil from the hole in g, and

W_b = weight of sand required to fill the hole in g.

6. REPORTING OF RESULTS

6.1 The following values shall be reported :

a) Dry density of soil in kg/m³ to the nearest whole number. The dry density may also be calculated and reported in g/cm³ correct to the second place of decimal.

b) Water content of the soil in percent reported to two significant figures.

6.2 The method used for obtaining the test results shall be stated as the small pouring cylinder method. The use of steel core cutter, if made, shall also be mentioned.

6.3 The results of the test shall be recorded suitably. A recommended proforma for the record of the test results is given in Appendix A.

SECTION 2 METHOD SUITABLE FOR FINE-, MEDIUM- AND COARSE-GRAINED SOILS : LARGE POURING CYLINDER METHOD

7. SCOPE

7.1 This method covers the determination, in-place, of the dry density (in g/cm³ or kg/m³) of natural or compacted soil containing stones which make the test of Section 1 difficult to perform. This is an alternative method of test to Section 1 for fine- and medium-grained soils and should be used instead of that test for layers exceeding 150 mm but not exceeding 250 mm in thickness (*see* Note under **1.1**).

8. APPARATUS

8.1 Large Sand-Pouring Cylinder — similar in the essential details to that shown in Fig. 1.

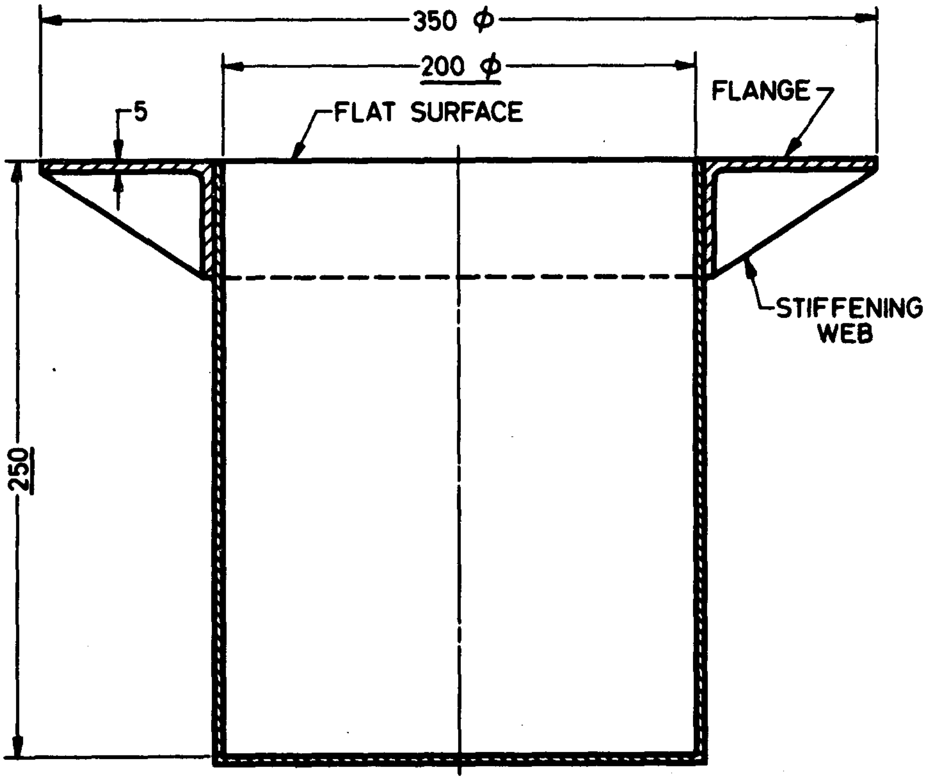
8.2 Tools for Excavating Holes — suitable tools, such as bent spoon, dibber (*see* Fig. 3), large screw driver, pointed steel rod about 300 mm long and 5 to 10 mm dia with a wooden handle.

8.3 Cylindrical Calibrating Container — with internal diameter of 200 mm and an internal depth of 250 mm (*see* Note 1 under **4.2.2**) of the type illustrated in Fig. 6 fitted with a flange 75 mm wide and about 5 mm thick surrounding the open end. The volume of the container should be given to an accuracy of 0.15 percent.

8.4 Balance — accurate to 1 g.

8.5 Plane Surface — a glass plate or other plane surface about 600 mm square and 10 mm thick or larger.

8.6 Metal Containers — to collect the excavated soil and to take the supply of sand to fill the pouring cylinder. This may be provided with a suitable cover.



NOTE 1 — This design has been found satisfactory, but alternative designs may be employed, provided that the essential requirements are fulfilled.

NOTE 2 — Essential dimensions are underlined.

NOTE 3 — Tolerance on essential dimensions ± 0.1 mm.

All dimensions in millimetres.

FIG. 6 CALIBRATING CONTAINER FOR USE WITH LARGE POURING CYLINDER AND IN THE HAND SCOOP METHOD

8.7 Metal Tray with Central Hole — a metal tray 450 mm square and 50 mm deep with a 200 mm dia hole in the centre.

9. MATERIAL (SAND)

9.1 Clean, uniformly graded natural sand passing the 1.00-mm IS Sieve and retained on the 600-micron IS Sieve shall be used. It shall be free from organic matter, and shall have been oven dried and stored for a suitable period to allow its water content to reach equilibrium with atmospheric humidity (*see* Note under **3.1**).

10. PROCEDURE

10.1 Calibration of Apparatus

10.1.1 The method described in **10.1.1.1** to **10.1.1.4** shall be followed for the determination of the weight of sand in the cone of the pouring cylinder.

10.1.1.1 The pouring cylinder shall be filled with a given initial weight of sand (W_1). This weight shall be maintained constant throughout the tests for which the calibration is used (*see* Note). A volume of sand equivalent to that of the excavated hole in the soil (or equal to that of the calibrating container) shall be allowed to run out of the pouring cylinder under gravity. The shutter on the pouring cylinder shall then be closed and the cylinder placed on the plane surface.

NOTE — The total weight of the pouring cylinder and sand is large, so that the method of filling and weighing is to weigh the sand in two or three containers and tip it into the pouring cylinder before using. Care shall be taken to see that the same constant initial weight as is used in calibrating the apparatus is used for each density measurement. Sufficient sand should be used to leave about 4 to 5 kg of sand in the pouring cylinder after the test is completed.

10.1.1.2 The shutter on the pouring cylinder shall be opened and sand allowed to run out. When no further movement of sand takes place in the cylinder the shutter shall be closed and the cylinder removed carefully.

10.1.1.3 The sand that has filled the cone of the pouring cylinder shall be collected and weighed to the nearest 0.1 percent of its total weight.

10.1.1.4 These measurements shall be repeated at least three times and mean weight (W_2) taken.

10.1.2 The method described in **10.1.2.1** to **10.1.2.3** shall be followed for the determination of the bulk density of the sand (Υ_s).

10.1.2.1 The internal volume (V) in ml of the calibrating container shall be determined by the weight of water contained in the container when filled to the brim (*see* Note under **10.2.2**).

10.1.2.2 The calibrating container should stand on a large tray during the procedure to collect the sand overflowing from the cone when the cylinder is removed. The pouring cylinder shall be placed concentrically on the top of the calibrating container and filled with the constant weight of sand (W_1) as in **10.1.1.1**. The shutter on the pouring cylinder shall be closed during this operation. The shutter shall be opened and the sand allowed to run out. When no further movement of the sand takes place in the cylinder the shutter shall be closed. The pouring cylinder shall be removed and the sand remaining in it weighed to the nearest 0.1 percent of its initial weight.

10.1.2.3 These measurements shall be repeated at least three times, and the mean weight (W_3) taken (*see* Note under **4.1.2.3**).

10.2 The method given in **10.2.1** to **10.2.4** shall be followed for the measurement of soil density.

10.2.1 A flat area, approximately 60 cm^2, at the place at which the soil is to be tested shall be exposed and trimmed down to a level surface.

10.2.2 The metal tray with a central hole shall be laid on the prepared surface of the soil with the hole over the portion of the soil to be tested. The hole in the soil shall then be excavated using the hole in the tray as a pattern, to the depth of the layer to be tested up to a maximum of 250 mm (*see* Note). The excavated soil shall be carefully collected leaving no loose material in the hole, and weighed to the nearest gram (W_w). The metal tray shall be removed before the pouring cylinder is placed in position over the excavated hole.

NOTE — If for any reason it is necessary to excavate holes to depths other than 250 mm the calibrating container should be replaced by one, the depth of which is the same as the hole excavated or its effective depth reduced to that of the hole excavated.

10.2.3 A representative sample of the excavated soil shall be placed in an air-tight container and its water content (w) determined by the method specified in IS 2720 (Part 2) : 1973.

10.2.4 The pouring cylinder filled with the constant weight of sand (W_1) as in **10.1.1.1**, shall be placed so that the base of the cylinder covers the hole concentrically. The shutter on the pouring cylinder shall be closed during this operation. The shutter shall then be opened and sand allowed to run out. When no further movement of the sand takes place the shutter shall be closed. The cylinder shall be removed and the sand remaining in it weighed to the nearest 0.1 percent of its initial weight (W_4) (*see* Note under **4.2.4**).

11. CALCULATIONS

11.1 The calculations shall be done as laid down in **5**.

11.2 For medium- and coarse-grained soils containing appreciable gravel fraction (plus 4.75-mm IS Sieve) the water content and dry density shall be determined as given in Appendix B.

12. REPORTING OF RESULTS

12.1 The results shall be reported as specified in **6** except that the method used for obtaining the test results shall be stated as large pouring cylinder method.

12.2 The results of the test shall be recorded suitably. A recommended pro forma for the record of test results is given in Appendix A.

APPENDIX A

(*Clauses* 6.3 *and* 12.2)

DETERMINATION OF DRY DENSITY OF SOIL, IN-PLACE, BY SAND REPLACEMENT

(Small Pouring Cylinder/Large Pouring Cylinder)

A-1. The test results for the two methods, namely, small pouring cylinder and large pouring cylinder may be tabulated as given below using the appropriate symbols and words in each case :

Project :
Location :

Tested by :
Date :

State whether steel core-cutter was used.

Calibration

1. Mean weight of sand in cone (of pouring cylinder) (W_2), in g	
2. Volume of calibrating container (V), in ml	
3. Weight of sand (+ cylinder) before pouring (W_1), in g	
4. Mean weight of sand (+ cylinder) after pouring (W_3), in g	
5. Weight of sand to fill calibrating container ($W_a = W_1 - W_3 - W_2$), in g	
6. Bulk density of sand $Y_s = \frac{W_a}{V} \times 1\,000$ kg/m^3	

Measurement of Soil Density

1. Determination No.	
2. Weight of wet soil from hole (W_w), in g	
3. Weight of sand (+ cylinder) before pouring (W_1), in g	
4. Weight of sand (+ cylinder) after pouring (W_4), in g	
5. Weight of sand in hole ($W_b = W_1 - W_4 - W_2$), in g	
6. Bulk density $Y_b = \frac{W_w}{W_b} \times Y_s$ kg/m^3	
7. Water content container No.	
8. Weight of soil for water content determination, in g	

9 Weight of oven dried soil, in g	
10. Water content (w), percent	
11. Dry density $Y_d = \dfrac{100\,Y_b}{100+w}$ kg/m^3	

APPENDIX B

(*Clause* 11.2)

DETERMINATION OF WATER CONTENT AND DRY DENSITY OF MEDIUM- AND COARSE-GRAINED SOILS CONTAINING APPRECIABLE GRAVEL FRACTION (PLUS 4.75-mm IS SIEVE)

B-1. IN-PLACE BULK DENSITY

B-1.1 The in-place bulk density (Y_b) of the soil shall be determined as described in Section 2.

B-2. PROCEDURE FOR DETERMINATION OF VOLUME AND WATER CONTENT

B-2.1 After obtaining the wet weight (W_w) of the total material removed from the hole, the soil shall be separated into plus 4.75-mm fraction (gravel) and minus 4.75 mm fraction by the 4.75-mm IS Sieve. This should be done rapidly to avoid loss of water.

NOTE — If this test is for construction control, the fraction passing the 4.75-mm IS Sieve should be placed in an air-tight container for further tests.

B-2.2 The fraction retained on the 4.75-mm IS Sieve (gravel) shall be washed on the sieve using a minimum of water, blotted dry with a towel to a wet surface-dry condition and weighed (W_g).

B-2.3 The volume of the gravel (V_g) in a wet surface-dry condition, shall then be determined by displacement of water from a siphon-container from which the overflow can be measured, or by weighing in air and in water. The specific gravity (G_g) of the gravel particles should then be computed.

NOTE — For construction control, the volume of gravel need not be measured every time a test is made. After several tests have shown that the specific gravity of the gravel from a particular source is virtually constant, the specific gravity may be assumed and the volume computed.

B-2.4 The wet gravel (W'_g) shall be placed in an oven and the oven-dry weight and water content (w_g) shall be determined.

B-2.5 The water content (w_s) in percent of the soil fraction passing the 4.75-mm IS Sieve shall also be determined by oven-drying a representative sample.

B-3. CALCULATIONS

B-3.1 Further calculations should be carried out as follows :

a) In-place bulk density $Y_b = \dfrac{W_w}{\text{Volume of hole}}$

b) Wet weight of minus 4.75-mm soil $= W_w - W_g$

c) Volume of minus 4.75-mm soil $=$ Volume of hole $- V_g$

d) Wet density of minus 4.75-mm soil $= \dfrac{(a)}{(b)}$

e) Dry weight of minus 4.75-mm soil $= \dfrac{(a)}{1 + W_s/100}$

f) Dry density of minus 4.75-mm soil $= \dfrac{(c)}{1 + W_s/100}$

g) Dry weight of total material (soil + gravel) $= W'_g + (d)$

h) Water content (w_T) of total material, percent $= \dfrac{W_w - (f)}{(f)} \times 100$

j) Percentage of gravel in the material on a dry weight basis $= \dfrac{W_g}{(f)} \times 100$

k) Dry density of the total material $= \dfrac{Y_b}{1 + w_T/100}$

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SECTION 3

Method of Load Test on Soils

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Indian Standard
METHOD OF LOAD TEST ON SOILS

(Second Revision)

0. FOREWORD

0.1 Visual examination of the soil exposed in suitably located trial pits at the site, combined with the already established data for different types of soils is commonly used for deciding on the safe bearing capacity. While this procedure may be adequate for light or less important structures under normal conditions, relevant laboratory tests or field tests are essential in the case of unusual soil types and for all heavy and important structures. This standard covers plate load test method for determination of ultimate bearing capacity of soil in place which assumes that soil strata is reasonably uniform. The load test included in the standard is also used to find modulus subgrade reaction useful in the design of raft foundation and in the design of pavements.

0.2 Plate load test, though useful in obtaining the necessary information about the soil with particular reference to design of foundation has some limitations. The test results reflect only the character of the soil located within a depth of less than twice the width of the bearing plate. Since the foundations are generally larger than the test plates, the settlement and shear resistance will depend on the properties of a much thicker stratum. Moreover this method does not give the ultimate settlements particularly in case of cohesive soils. Thus the results of the test are likely to be misleading, if the character of the soil changes at shallow depths, which is not uncommon. A satisfactory load test should, therefore, include adequate soil exploration (*see* IS 1892 : 1979) with due attention being paid to any weaker stratum below the level of the footing.

0.3 Another limitations is the concerning of the effect of size of foundation. For clayey soils, the bearing capacity (from shear consideration) for a larger foundation is almost the same as that for the smaller test plate. But in dense sandy soils the bearing capacity increases with the size of the foundation. Thus tests with smaller size plate tend to give conservative values in dense sandy soils. It may, therefore, be necessary to test with plates of at least three sizes and the bearing capacity results extrapolated for the size of the actual foundation (minimum dimensions in the case of rectangular footings).

0.4 This standard was first published in 1962 and subsequently revised in 1971. In the present revision, the use of apparatus has been generalized and also specific sizes of plates have been mentioned for the different types of soils, besides incorporating zero correction which was present in 1971 version and prescribing log log scale for cohesionless and partially cohesive soils.

1. SCOPE

1.1 This standard lays down the method for conducting load test for estimation of bearing capacity of soils and its settlement.

2. TERMINOLOGY

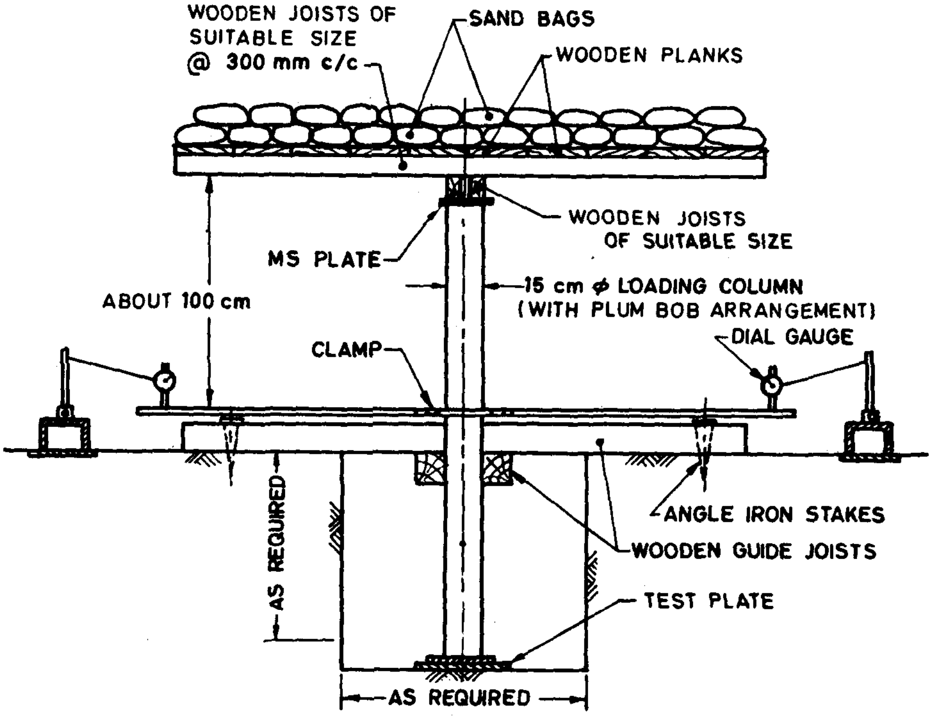
2.1 For the purpose of this standard, the definitions given in IS 2809:1972 and IS 6403:1981 shall apply.

3. APPARATUS

3.1 Loading platform truss of sufficient size and properly designed members so as to estimate load reaction for conducting the test shall be used. The typical set up used for gravity loading is given in Fig. 1, for reaction loading in Fig. 2 and for loading truss in Fig. 3.

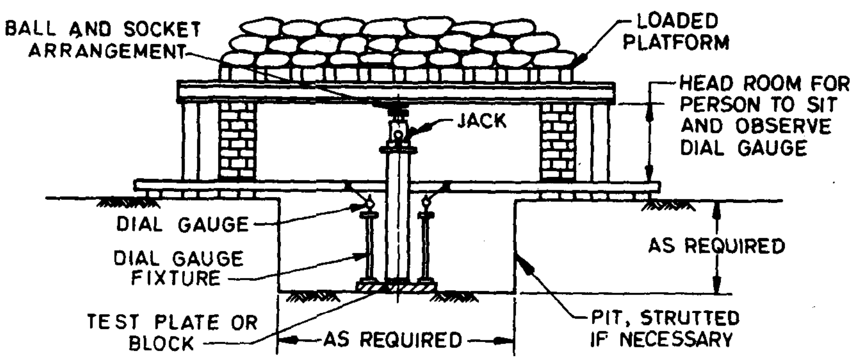
3.2 Hydraulic jack of required capacity with properly calibrated load measuring device, such as pressure gauge, electronic laod cell, or proving ring shall be used.

3.3 Bearing Plates — Circular or square bearing plates of mild steel, not less than 25 mm in thickness and varying in size from 300 to 750 mm with chequered or grooved bottom (*see* Fig. 4), provided with handles for convenient setting and centre marked. As an alternative, cast *in-situ* or precast concrete blocks may be used with depths not less than two-thirds the width.



NOTE — Clamp could also be at lower level.

FIG. 1 TYPICAL SET UP FOR GRAVITY LOADING PLATFORM



NOTE — Dial gauge fixture may be on the form clamp also.

FIG. 2 TYPICAL SET UP FOR REACTION LOADING PLATFORM

3.4 Settlement Recording Device — Dial gauges with 25 mm travel, capable of measuring settlement to an accuracy of 0.01 mm.

3.5 Datum Beam or Rod — Beam or rod of sufficient strength capable of maintaining straightness when fitted on two independent supports fitted with arms or magnetic bases for holding dial gauges.

3.6 Miscellaneous Apparatus — A ball and socket arrangement, loading columns, steel shims, wooden blocks, collar, reaction girder with cradles for independent fitting to the reaction platform as necessary to the particular set up.

4. PROCEDURE

4.1 Selection of Location — The locations for load test shall be based on exploratory borings, and unless otherwise desired, shall be conducted at an elevation of the proposed foundation level under

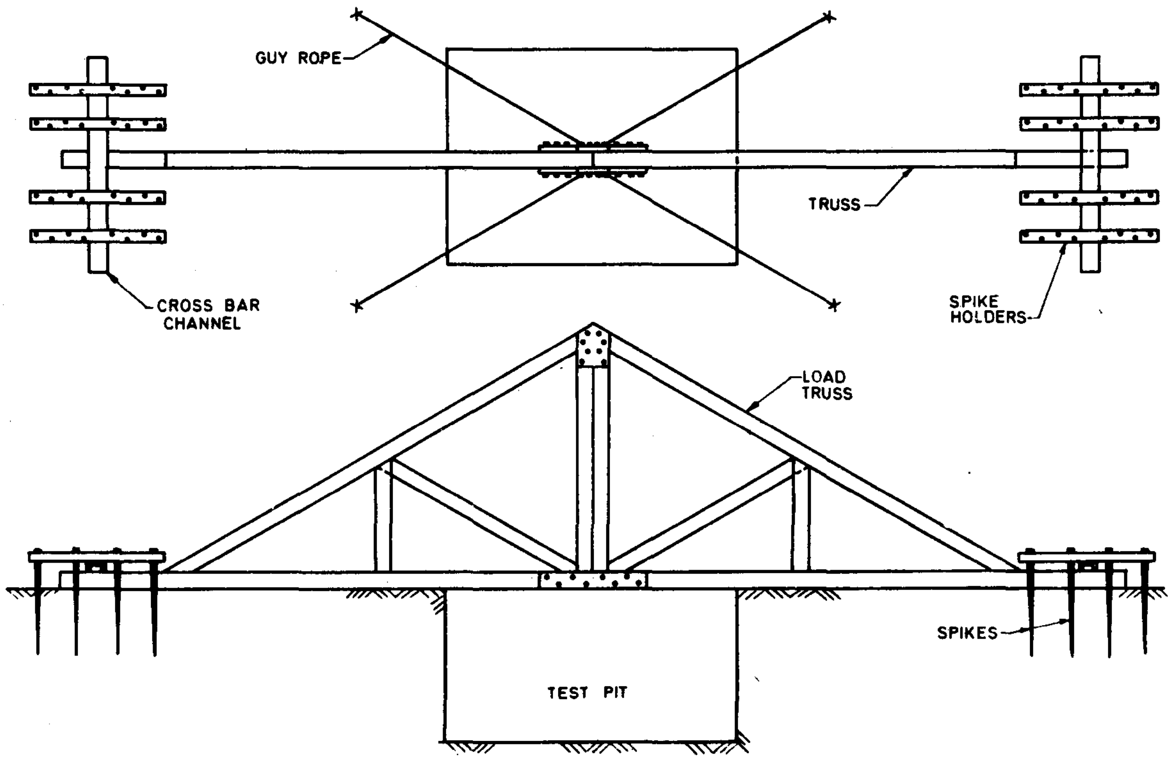
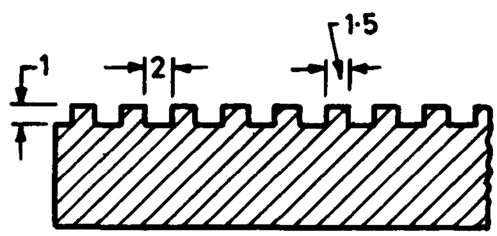


FIG. 3 TYPICAL SET UP FOR LOADING TRUSS



All dimensions in millimetres.

FIG. 4 DETAILS OF CHEQUERS OR GROOVES

the worst estimated conditions. In case the water table is within the depth equal to the width of the test plate, the test shall be conducted at water table level. In case water table is higher than the test level, it shall be lowered to the test level and maintained by pumping through a sump, away from the test plate, however, for the soils like cohesionless silt and fine sand which cannot be drained by pumping from the sump, the test level shall also be water table level.

4.2 Test Pit — The pits, usually at the foundation level, having in general normally of width equal to five times the test plate or block, shall have a carefully levelled and cleaned bottom at the foundation level; protected against disturbance or changes in natural formation.

4.3 Dead Load — The dead load ot all equipment used, such as ball and socket, steel plate, loading column, jack, etc, shall be recorded prior to application of load increment.

4.4 Size and Shape of Plate — Except in case of road problems and circular footings, square plates may be adopted. For clayey and silty soils and for loose to medium dense sandy soils with $N < 15$, a 450 mm square plate or concrete blocks shall be used. In the case of dense sandy or gravelly soils ($15 < N < 30$) three plates of sizes 300 mm to 750 mm shall be used depending upon practical considerations of reaction loading and maximum grain size. The side of the plate shall be at least four times the maximum size of the soil particles present at the test location.

NOTE — N is the standard penetration resistance value determined in accordance with IS 2131 : 1981.

4.5 Test Arrangement

4.5.1 The loading platform shall be supported by suitable means at least 2.5 m from the test area with a height of 1 m or more above the bottom of the pit to provide sufficient working space. No support of loading platform should be located within a distance of 3.5 times size of test plate from its centre.

4.5.2 The test plate shall be placed over a fine sand layer of maximum thickness 5 mm, so that the centre of plate coincides with the centre of reaction girder/beam, with the help of a plumb and bob and horizontally levelled by a spirit level to avoid eccentric loading. The hydraulic jack should be centrally placed over the plate with the loading column in between the jack and reaction beam so as to transfer load to the plate. A ball and socket arrangement shall be inserted to keep the direction of the load vertical throughout the test. A minimum seating pressure of 70 g/cm^2 shall be applied and removed before starting the load test.

4.5.3 The two supports of the reference beam or datum rod shall be placed over firm ground, fixed with minimum two dial gauges resting at diametrically opposite ends of the plates. The dial gauges shall be so arranged that settlement is measured continuously without any resetting in between.

4.6 Load Increments — Apply the load to soil in cumulative equal increments up to 1 kg/cm^2 or one-fifth of the estimated ultimate bearing capacity, whichever is less. The load is applied without impact, fluctuation or eccentricity and in case of hydraulic jack load is measured over the pressure gauge, attached to the pumping unit kept over the pit, away from the testing plate through extending pressure pipes.

4.7 Settlement and Observation — Settlements should be observed for each increment of load after an interval of 1, 2.25, 4, 6.25, 9, 16 and 25 minutes and thereafter at hourly intervals to the nearest 0.02 mm. In case of clayey soils, the 'time settlement' curve shall be plotted at each load stage and load shall be increased to the next stage either when the curve indicates that the settlement has exceeded 70 to 80 percent of the probable ultimate settlement at that stage or at the end of 24 hour period. For soils other than clayey soils, each load increment shall be kept for not less than one hour or up to a time when the rate of settlement gets appreciably reduced to a value of 0.02 mm/min. The next increment of load shall then be applied and the observations repeated. The test shall be continued till a settlement of 25 mm under normal circumstances or 50 mm in special cases, such as dense gravel, gravel and sand mixture is obtained or till failure occurs, whichever is earlier. Alternatively, where settlement does not reach 25 mm, the test should be continued to at least two times the estimated design pressure. If needed, rebound observations may be taken while releasing the load.

5. DETERMINATION OF ULTIMATE BEARING CAPACITY/SAFE BEARING PRESSURE/SETTLEMENT

5.1 Shape of the Load/Settlement Curve — A load settlement curve shall be plotted out to arithmetic scale. From this load settlement curve, the zero correction which is given by the intersection of the early straight lines or the nearly straight line part of the curves with zero deadline shall be determined and subtracted from the settlement readings to allow for the perfect seating of the bearing plate and other causes.

5.1.1 Four typical curves are shown in Fig. 5. Curve *A* is typical for loose to medium cohesionless soil; it is straight line in the earlier stages but flattens out after some time, but there is no clear point of failure. Curve *B* is for cohesive soil, it may not be quite straight in the early part and leans towards settlement axis as the settlement increases. For partially cohesive soils, curve *C* possessing the characteristics of both the curves *A* and *B* is obtained while curve *D* is purely for dense cohesionless soils.

5.2 From the corrected load settlement curves, no difficulty should be experienced in arriving at the ultimate bearing capacity in case of dense cohesionless soils or cohesive soils (*see* Fig. 5, curves *D* and *B*) as the failure is well defined. But in the case of curves *A* and *C*, where yield point is not well defined, settlements shall be plotted as abcissa against corresponding load intensities as ordinate, both to logarithmic scales (*see* Fig. 6), which give two straight lines, the intersection of which shall be considered as yield value of soil.

5.3 From Fig. 5, the safe bearing pressure for medium and dense sands could be read,

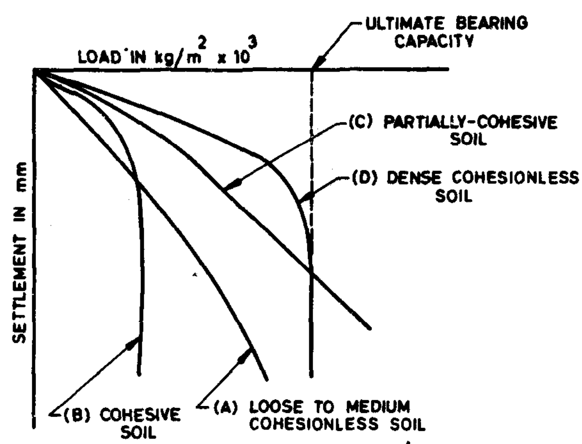


FIG. 5 LOAD SETTLEMENT CURVES

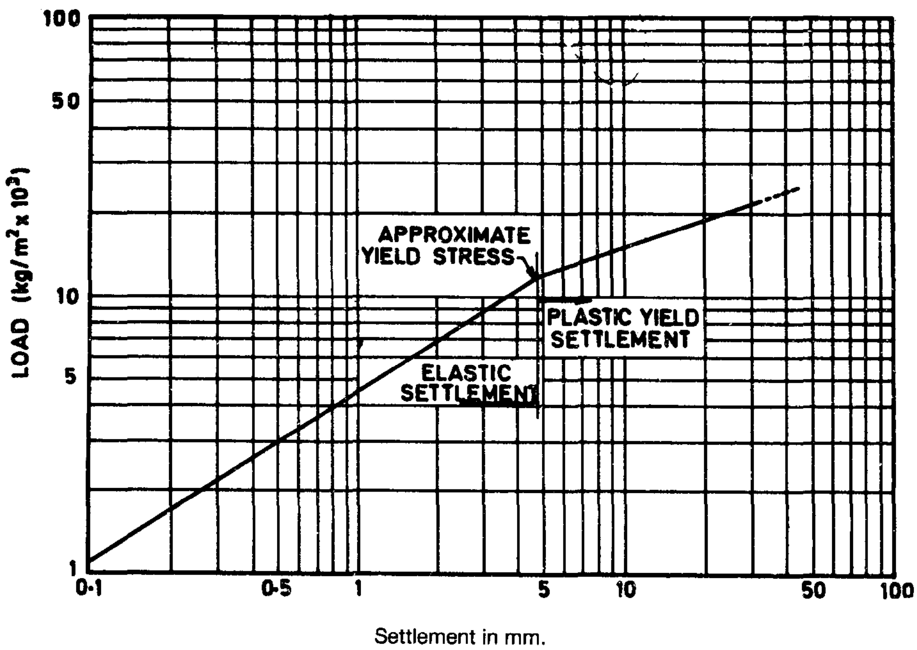


FIG. 6 LOAD SETTLEMENT CURVES (LOG-LOG SCALE)

corresponding to a settlement (S_p), which shall be calculated as under (S_f) taken as permissible settlement of footing (*see* IS 1904 : 1978) :

$$S_f = S_p \left[\frac{B(B_p + 0.3)}{B_p(B + 0.3)} \right]^2$$

where

B = the size of footing in m,
B_p = size of test plate in m,
S_p = settlement of test plate in m, and
S_f = settlement of footing in m.

From this formula, total settlement of footing (S_f) is calculated taking S_p as observed total settlement of plate.

6. REPORT

6.1 The continuous listing of all time, load and settlement data, for each test shall be recorded with details of test elevation, natural water table, profile of test pit, size of bearing plate and irregularity, if any, in routine procedure.

6.2 It is necessary to excavate soil below the test plate to a depth equal to twice the dimension of the plate so as to examine and record the subsoil profile.

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SECTION 4

Determination of Modulus of Subgrade Reaction

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Indian Standard
METHOD OF DETERMINATION OF MODULUS OF SUBGRADE REACTION (*K*-VALUE) OF SOILS IN FIELD

0. FOREWORD

0.1 The 'modulus of subgrade reaction test', usually known as *K*-value test, is essentially a plate bearing test. This test is used for the design of pavement structures and raft foundations.

0.2 Like all other soil strength tests, *K*-value test will not provide a representative measurement of subgrade strength unless performed under the conditions that would be expected after equilibrium of the subgrade with the environmental influences of moisture, density, frost, drainage and traffic. Equilibrium is not attained until the structure has been superimposed upon it for some time.

0.3 In the formulation of this standard, due weightage has been given to international co-ordination among the standards and practices prevailing in different countries in addition to relating it to the practices in the field in this country.

1. SCOPE

1.1 This standard deals with the method for the determination of modulus of subgrade reaction of the soil in-place (generally known as *K*-value of the subgrade) for evaluation of strength of subgrade for roads, runway pavements and raft foundations.

2. TERMINOLOGY

2.0 For the purpose of this standard, the definitions given in IS 2809 : 1972 and the following shall apply.

2.1 Modulus of Subgrade Reaction — Ratio of load per unit area (applied through a centrally loaded rigid body) of horizontal surface of a mass of soil to corresponding settlement of the surface. It is determined as the slope of the secant drawn between the point corresponding to zero settlement and the point of 1.25 mm settlement, of a load-settlement curve obtained from a plate load test on a soil using a 75 cm diameter or smaller loading plate with corrections for size of plate used.

2.2 Deflection — The amount of downward vertical movement of a horizontal surface due to the application of a load to the surface.

2.3 *K*-value — If the assumption that the reaction of the subgrade is proportional to the deflection is entirely correct, the curve in Fig. 1 should be straight line and the slope of this line should give the modulus of subgrade reaction measured in MPa/cm ($kgf/cm^2/cm$). The results, however, usually give a curve which is convex upwards and which has no straight portion even initially, *K*- value is, therefore, taken as the slope of the line passing through the origin and the point on the curve corresponding to 1.25 mm settlement (*see* Fig. 1) :

$$K = \frac{p}{0.125} \text{ MPa/cm } (\text{kgf/cm}^2/\text{cm})$$

where

p = load intensity corresponding to settlment of plate of 1.25 mm.

Alternatively, the *K*-value may be defined as a pressure of 0.07 MPa (0.70 kgf/cm^2) divided by the corresponding settlement. That is, when a standard 75 cm diameter steel bearing plate is subjected to a load of 3 100 kgf, say

$$K = \frac{0.07}{d} \text{MPa/cm}$$

$$\left[K = \frac{0.70}{a} \text{ kgf/cm}^2\text{/cm}\right]$$

where

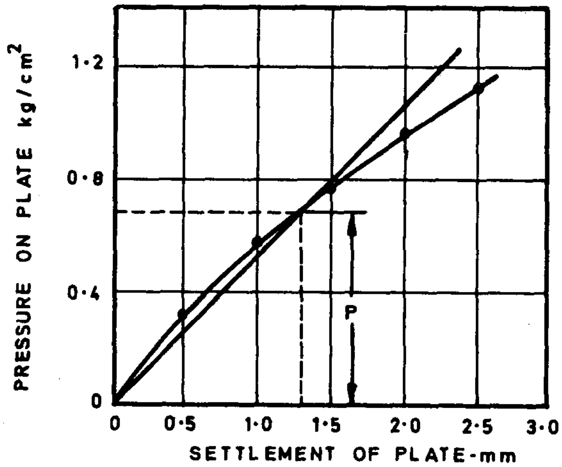
d = settlement in cm.

2.4 Stiffening Plates — Nest of plates stacked on the bearing test plate for stiffening it.

3. APPARATUS — *see* Fig. 2.

3.1 Bearing Plates — It is a circular mild steel plate of 75 cm diameter and 25 mm thickness. Smaller bearing plates of 45, 40 or 30 cm may also be used.

3.2 Loading Attachment — Loads are applied by means of a hydraulic jack or a screw jack working against a reaction frame through bearing plates. The loading attachment should have a capacity of at least 150 kN (15 000 kgf) equipped with ball and socket joint between the test load and the jack to avoid eccentricity. The device should have an arrangement for attaching to a truck, trailer, truss or any other equipment load reaction.



$$K = \frac{P}{0.125} \text{ kgf/cm}^2\text{/cm}$$

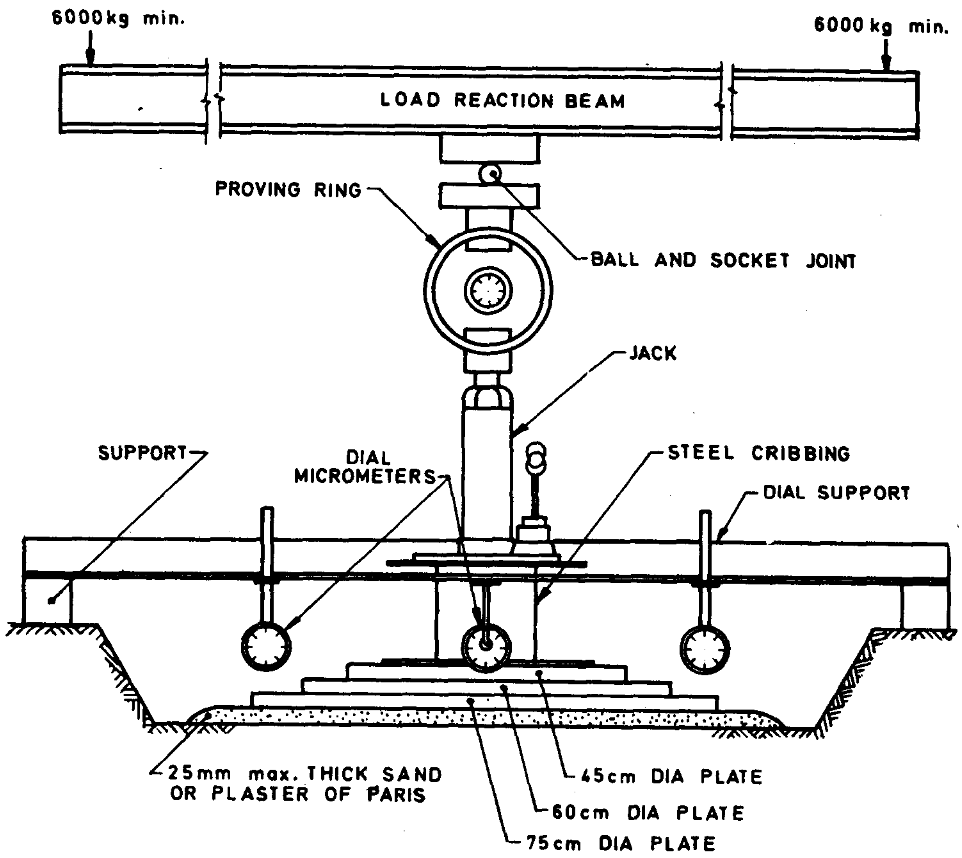
FIG. 1 RESULT OF PLATE BEARING TEST ON NATURAL SUBGRADE

3.3 Jacks — Hydraulic or screw jack of 150 kN (15 000 kgf) capacity.

3.4 Proving Ring — One calibrated proving ring of capacity 150 kN (15 000 kgf) with dial gauge to read to an accuracy of 0.002 mm. The proving ring should have an accuracy of one-half percent of the load measured.

3.5 Loading Reaction — The reaction for jacking can be provided by a truck, trailer or anchor frame such that its reaction shall be at least 2.5 m away from the centre of the bearing plates. When the test is to be conducted on granular subgrades, a reaction of at least 150 kN (15 000 kgf) will be required. For cohesive soils a 5 kN (5 000 kgf) reaction may be sufficient.

3.6 Measuring Deformation — The vertical movement resulting from applied loads will be measured by at least three dial gauges uniformly spaced 120° apart, preferably four uniformly placed at 90° apart, and placed at about 10 mm away from the rim of 75-cm plate. The gauges will be supported by an independent datum bar such that their positions are unaffected by the loading operations. These supports should be at least 2.4 m from the plates and the wheels. The settlement of the plate is taken as the average of the reading of the dial gauges used for the purpose. Gauges with an accuracy of 0.002 mm are desirable; however, gauges with an accuracy of 0.01 mm may be used if time-readings are made.



NOTE — Support for load reaction shall be 240 cm minimum from bearing plates.

FIG. 2 SCHEMATIC DIAGRAM OF PLATE BEARING TEST

3.7 Jack-Pads — Due to variation in the depth of test points, some distance pieces, for example, spacers, will be required between jack and proving ring. These can be solid cylindrical pieces of aluminium alloy or any other suitable material to withstand and help in transferring heavy loads on to the bearing plate. These spacers should be at least 15, 20 and 30 cm long. The exact requirement of these jack pads will vary from one test point to another according to depth of test point below ground surface.

3.8 Stiffening Plates — These are mild steel plates of 60, 45 and 30 cm diameter and 25 mm thickness.

3.9 Miscellaneous Apparatus — Datum bar of 5 m length with suitable dial gauge attachments, pick axes, showel, trowel, spatula, spirit level and plumb bob.

4. TEST PROCEDURE

4.1 Two alternative test procedures may be followed. More accurate tests are made with a 75-cm plate, and a load reaction arrangement, a loading jack, a proving ring to measure the load and three dial gauges placed diagonally apart about 10 mm from the rim to measure the vertical deflection.

4.2 Preparation of Test Area — *K*-value tests should be conducted on representative area. Most soils exhibit a marked reduction in the modulus of subgrade reaction with increase in moisture content, which cannot be generalized. Conditions of moisture content, density and type of material, all enter into the interpretation of test results to give a design value which will represent the condition of equilibrium that ultimately will exist in the subgrade. Generally, subgrade is composed of either natural ground or fill-material. Preparation of an area for testing will depend on composition of subgrade.

4.2.1 *Preparation of Test Area for Natural Ground* — Strip off an area equal to twice the area of the bearing plate to the proposed elevation of the subgrade surface. In any case, it is necessary to remove the top 25 cm of the soil before testing. The effect of surcharge can be eliminated by having supports of datum bar at least 1.25 m away from the nearest edge of bearing plates.

4.2.1.1 On fine-grained soils, the bearing plate, with its lower surface oiled, shall be placed and rotated. When the plate is removed, the irregularities in the surface, being marked with oil shall be trimmed off. This process is repeated until the plate is in contact with the soil over the whole area.

4.2.1.2 When the test is made on granular subgrade, extreme care shall be taken not to disturb the natural condition of the subgrade while the test site is being prepared. Prior to placement of 75 cm diameter plate, the area should be cleared of loose material and levelled.

4.2.1.3 On gravely soils, flat bearing surface can best be obtained with plaster of Paris, which should be levelled with the plate before the plaster has set. The *K*-value test should not be started until the plaster has sufficiently hardened.

4.2.2 *Preparation of Filled-Up Area* — In case the test is to be conducted on subgrade composed of fill materials, a test embankment of about 75 cm height should be constructed after necessary stripping. For design purposes, the conditions of the moisture content and dry density of the test area should be those which are likely to exist when the subgrade has reached a state of relative equilibrium subsequent to the construction of pavement. Generally, the subgrade will be compacted at optimum moisture content and specified density. If ordinary compaction equipment is not available, approximate compaction may be obtained by hand tamping in layers.

4.2.2.1 The test should be conducted keeping in view elimination of bearing pressure of reaction frame and datum bar as mentioned in **4.2.1**. The bearing plate with its lower surface oiled shall be placed on the prepared surface and rotated. When the plate is removed, all proud portions indicated by oil marks shall be as levelled as possible. If levelling is difficult due to presence of granular material, a layer of fine dry sand at no place thicker than 5 mm may be laid and the plate seated properly.

4.3 Loading Procedure — There are two methods for the determination of modulus of subgrade reaction as given in **4.3.1** and **4.3.2.**

4.3.1 *Method I* — The loading system and bearing plate should be seated by applying a load of 3.1 kN (310 kgf) (0.007 MPa for a standard 75 cm diameter plate), when the design thickness of pavement is less than 40 cm which is normally used for lightly loaded pavements. For heavy duty pavements, a seating loading of 6.2 kN (620 kgf) should be used. The seating load will be allowed to remain until practically complete deformation has taken place, at this time a reading should be taken on the dial gauges and adjusted to 'zero' reading. Cyclic loading under 3.1 kN (310 kgf) or 6.2 kN (620 kgf) seating load, as required, may be used to assure good seating of the bearing plate. Then without releasing the seating load an additional 31 kN (3 100 kgf) [that is, a total 34.1 kN (3 410 kgf) or 37.2 kN (3 720 kgf) load depending on the type of the pavement] should be applied to the plates and held until practically complete settlement has taken place.

For recording observations, proforma given in Appendix A should be used. Prior to releasing the 31 kN (3 100 kgf) load, a value of K_u will be computed for the average deflection at the plate rim by the formula :

$$K_u = \frac{0.07}{d} \text{ MPa/cm, or}$$

$$K_u = \frac{0.70}{d} \text{ kgf/cm}^2\text{/ cm}$$

where

d = deflection in cm.

One of the procedures given in **4.3.1.1** and **4.3.1.2** should be then followed depending upon the type of subgrade and the value of uncorrected modulus of subgrade K_u.

4.3.1.1 For cohesive subgrades with K_u equal to 0.555 MPa/cm (5.55 kgf/cm^2/cm) or less, the load will be released. For some clayey soils, it may be necessary to plot a time-settlement curve to aid determination of practically complete settlement. In general, the load will be held until the rate of deflection of the bearing plate is less than 0.000 5 cm/min (that is, 0.005 cm in 10 minutes). This rate of settlement indicates that the major portion of the settlement has occurred.

4.3.1.2 For granular subgrades or cohesive subgrade with K_u equal to 0.555 MPa/cm (5.55 kgf/cm^2/cm) or more, a load-deflection curve should be obtained by measuring the successive deformations caused by increasing the load in increments of 15.5 kN (1 550 kgf) to a maximum of 93 kN (9 300 kgf). The load should not be released between the increments of loading. Each increment of load shall be held for at least 15 minutes. The final load of 93 kN (9 300 kgf) shall be held until practically complete settlement is reached, but in not less than 15 minutes. In general, the 0.21 MPa (2.1 kgf/cm^2) load should be held until the rate of settlement of the plate is less than 0.000 2 cm/min (that is, 0.002 cm in 10 minutes). This rate indicates that the major portion of the settlement has taken place.

4.3.2 *Method II* — The plate shall first be seated by applying a load equivalent to a pressure of 0.007 MPa (0.07 kgf/cm^2) and releasing it after a few seconds. A load sufficient to cause approximately 0.85-mm settlement should be applied and when there is no perceptible increase in settlement or in the case of clayey soils, when the rate of increase in settlement is less than 0.025 mm/min, the average of the readings of the deflection dial gauges should be noted. The load, as measured by the pressure gauge attached to the jack or by the proving ring, should be noted, both immediately before and after the deflection readings. The load should be increased until there is an additional settlement of approximately 0.25 mm and the load and deflection again noted when there is no perceptible increase in settlement. This procedure should be repeated until a total settlement of not less than 1.75 mm has been produced. For recording observations proforma is given at Appendix B.

NOTE 1 — Rapid and less accurate tests may be made with a 45-cm or even a 30-cm diameter plate with a 5-tonne loaded lorry to provide the load reactions, a pressure gauge to measure the load and at least 3 dial gauges to measure the vertical deflection of the bearing plate. In such cases, the K_p value applied should be corrected for the standard 75-cm plate as explained in **5**.

NOTE 2 — The test may be conducted with a plate of diameter smaler than 75 cm when adequate reaction is not available. When a smaller plate is used, the K_p value should be corrected to get the K_r value corresponding to a standard 75-cm diameter plate as explained in **5**.

5. EVALUATION OF SUBGRADE TEST RESULTS

5.1 The corrections mentioned in **5.1.1** to **5.1.4** should be applied before a final value of subgrade reaction K is evaluated.

5.1.1 *Correction when Using Plates Smaller than 75 cm Diameter* — Theoretical relationship may be established between the modulus of subgrade reaction and the plate diameter. This value for a plate of particular diameter can be expressed as a percentage of the equivalent modulus of subgrade value of a 75-cm diameter from Fig. 3 and thus equivalent value of K_p for a 75-cm diameter plate can be evaluated.

5.1.2 *Correction of Load-Deflection Curve* — The correction should be necessary if the value of K_u is 0.555 MPa/cm (5.55 kgf/cm^2/cm) or more. In such a case, unit loads up to 93 kN (9 300 kgf) in 15.5 kN (1 550 kgf) increments should be applied and a load-deflection curve is plotted. In these cases, the load-deflection curve shall not be a straight line and hence a correction should be made. Generally, the load-deflection curve shall approximate a straight line between unit loads of 31 kN (3 100 kgf) and 93 kN (9 300 kgf) [0.07 to 0.21 MPa (0.7 to 2.1 kgf/cm^2)]. The correction should then consist of drawing a straight line parallel to the straight-line portion of the load-deflection curve through the origin. The deflection for computing the K_d value shall then be determined at a unit load of 31 kN (3 100 kgf) (0.07 MPa/0.7 kgf/cm^2) and the K_d computed by the formula given in **4.3.1**. In case no straight portion on the curve is observed, then at least three points in the region having the least curvature should be selected (*see* Fig. 4).

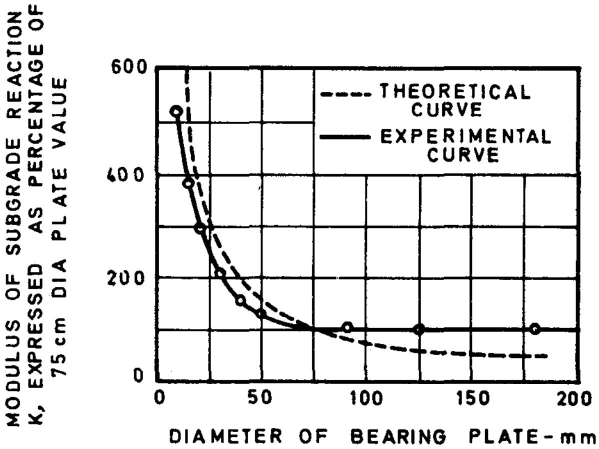


FIG. 3 RELATIONSHIP BETWEEN MODULUS OF SUBGRADE REACTION AND DIAMETER OF BEARING PLATE

5.1.3 *Correction for Bending of the Plates* — The bending of the bearing plate is greater at the centre than at the rims, even when nests of plates are used. A method for the correction of modulus of subgrade for plate bending is shown in Fig. 5.

The value of uncorrected subgrade reaction should be used as the ordinate to enter the curve in Fig. 5. The value obtained on the abscissa in the curve of Fig. 5 should be the K_b value corrected for bending of plate.

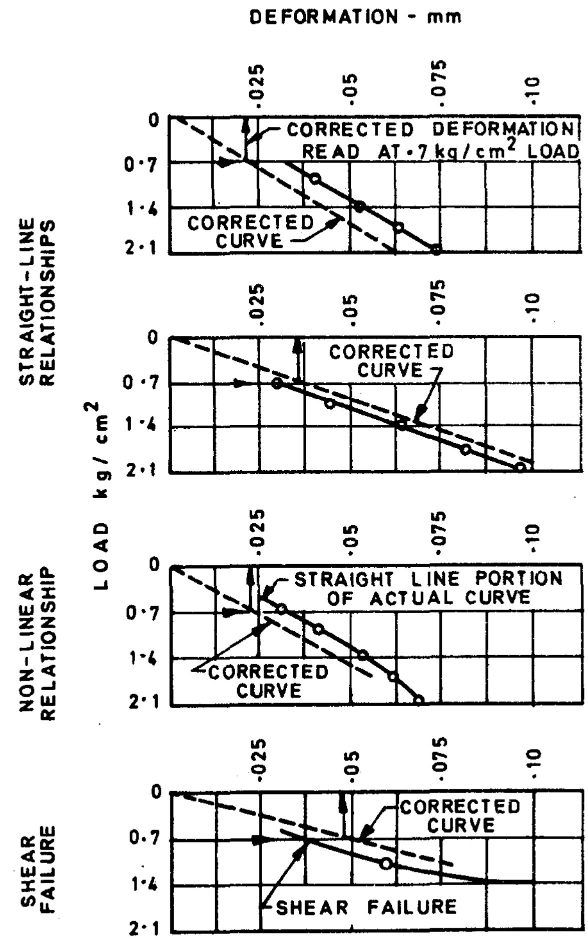
NOTE — When uncorrected value is less than 0.275 MPa/cm, the correction for plate bending is negligible and may be ignored.

5.1.4 *Correction for Saturation* — The moisture content of the subgrade at the time of the plate bearing test may increase after the pavement has been constructed and the worst condition may be covered by converting the value of modulus of subgrade obtained from bearing test to a value for the subgrade when soaked. It is impracticable to do this directly by artificially wetting the test area. Hence correction should be applied on the basis of consolidation tests on the subgrade material.

5.1.4.1 The correction of subgrade saturation shall consist of loading two samples of the undisturbed subgrade materials in a consolidometer as follows :

a) One at the *in-situ* moisture content, that is, at natural moisture content; and

b) The other at a saturated condition.



NOTE — Corrected curve may lie above or below actual curve.

FIG. 4 CORRECTION OF LOAD-DEFLECTION CURVE

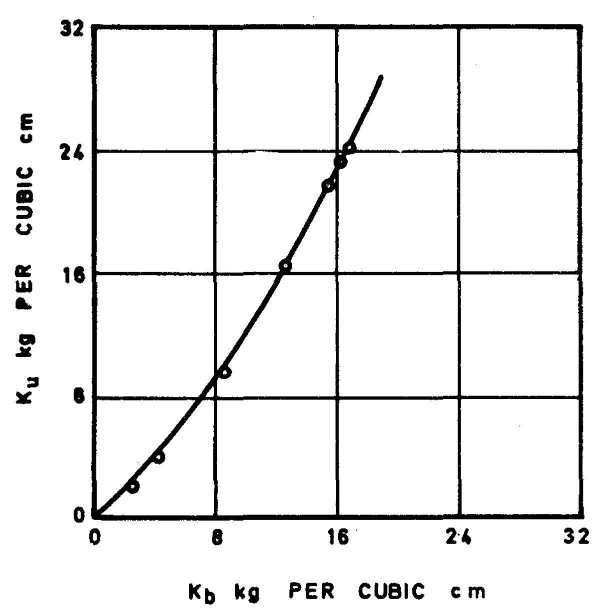


FIG. 5 CHART FOR CORRECTION FOR K_u FOR BENDING OF THE PLATE

5.1.4.2 The two specimens in the consolidometer shall be applied the same seating load that was used in the field plate bearing tests (*see* **4.3.1**). The seating load shall be allowed to remain on the specimen having *in-situ* water content until the vertical movement becomes stable, at this time a 'zero' reading should be taken. Now apply an additional 31 kN (3 100 kgf) load. The total load should then be allowed to remain until the vertical dial gauge reading becomes stable.

5.1.4.3 The second specimen should be allowed to saturate under the seating load. After the vertical dial gauge does not show any movement, a 'zero' reading should be taken and an additional 31 kN (3 100 kgf) load should now be applied. Take down the readings when vertical dial shows stable reading.

5.1.4.4 The corrections for saturation should be applied in proportion to the deformations of the two specimens under a unit load of 31 kN (3 100 kgf) in addition to the seating load as follows :

$$K_s = \frac{d}{d_s} \times \text{uncorrected value of modulus subgrade}$$

where

K_s = corrected value of subgrade for saturation,

d = deformation of a specimen with natural moisture content under unit load of 31 kN (3 100 kgf), and

d_s = deformation of a specimen when saturated under unit load of 31 kN (3 100 kgf).

If $\frac{d}{d_s} = 1$, this correction is inapplicable.

5.2 The corrections and order in which these should be applied are explained on a flow chart given in Appendix C.

5.2.1 For easy understanding of all the necessary corrections required to be applied in determination of K-value, a sample observations and calculations are given in Appendix D.

6. REPORT

6.1 The value shall be reported corrected to the second decimal place.

APPENDIX A

(*Clause* 4.3.1)

PROFORMA FOR IN-PLACE *K*-VALUE — TEST METHOD I

Location.. Tested by....................................
Material of test point.. Date...
Depth of test...
Condition of test surface : Soaked..................
Unsoaked
Period of soaking, if any...............................
Moisture content.......................................
Density..
Method used for determination of density.........

Dia of plate used:

Proving Ring Dial Gauge Readings	Load, kN (kgf)	Deflection Dial Gauge Readings				Average Deflection, mm	Cumulative Deflection, mm	*K*-value, kgf/cm²/cm MPa/cm					
		DG 1	DG 2	DG 3	DG 4			Uncorrected Value, K_u	Corrected for Load of Deflection, K_d	Corrected for Bending, K_b	Corrected for Saturation, K_s	Corrected for Size of Plate, K_p	Corrected *K*-value
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)

Rejected test with reason.........................
Result of repeat test if conducted..........

APPENDIX B

(*Clause* 4.3.2)

PROFORMA FOR IN-PLACE K-VALUE (MODULUS OF SUBGRADE REACTION) — TEST METHOD II

Location...
Material at test point..................................
Depth of test...
Condition of test surface : Soaked.................
Unsoaked
Period of soaking, if any..............................
Moisture content..
Density..
Method used for determination of density.........

Tested by.............................
Date...................................

Dia of plate used :

PROVING DIAL GAUGE READINGS	LOAD, kN (kgf)	DEFLECTION DIAL GAUGE READINGS				AVERAGE DEFLECTION, mm	CUMULATIVE DEFLECTION, mm	K_u MPa/cm (kgf/cm²/cm)	$K_b/K_p/K_s$ MPa/cm (kgf/cm²/cm)	FINAL CORRECTED K-VALUE MPa/cm (kgf/cm²/cm)
		DG 1	DG 2	DG 3	DG 4					
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)

NOTE — Load increments to be loaded till the total deflection is 1.75 mm or more.
K-value to be computed from load/deflection plot.

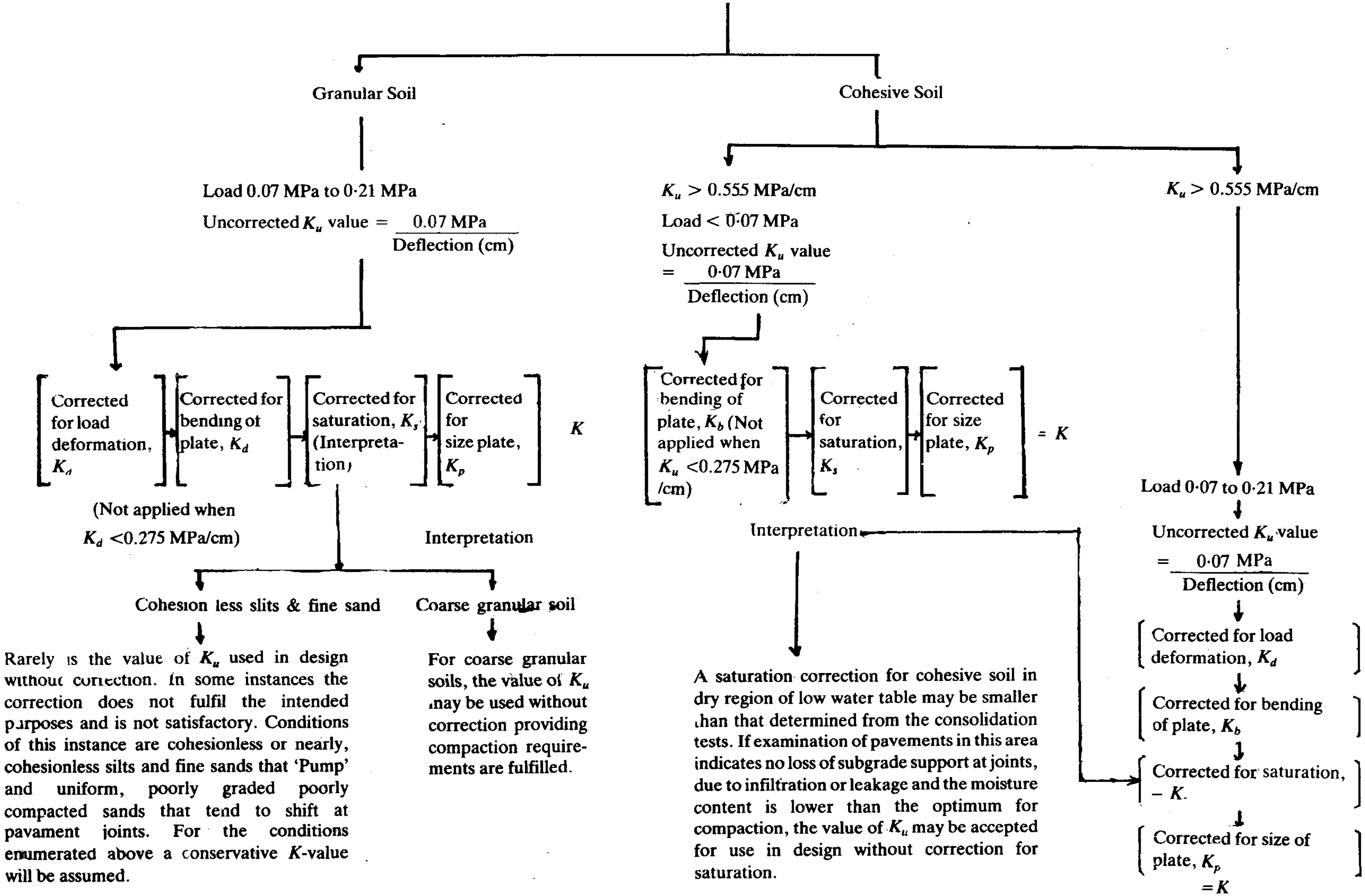
Rejected test with reasons ....

Result of repeat test if conducted ....

APPENDIX C

(*Clause* 5.2)

DETERMINATION OF *K*-VALUE 'FLOW CHART'



APPENDIX D

(*Clause* 5.2.1)

SAMPLE OBSERTVATION AND CALCULATIONS FOR DETERMINATION OF *K*-VALUE

Location	:	Tested by :
Material of test point 0	:	
Depth of test	: 0.50 cm	Date : Time :
Condition ot test subgraqe	: Partly saturated	
Period of soaking, if any	:	
Moisture content	: 18 percent	
Density	: 1.8 g/cm^3	
Dia of plate used	: 75 cm	

PROVING RING DIAL GAUGE READINGS	LOAD IN kN (kgf)	DEFLECTION (mm) DIALGAUGE READINGS DG 1	DG 2	DG 3	AVERAGE DEFLECTION, mm	K_u kg/cm^2/cm MPa/cm
10.04	3.1 kN (310 kgf) (0.007 MPa)	26.00	15.10	11.26	17.45	
10.72	34.1 kN (3 410 kgf) (0.07 MPa)	25.72	14.62	10.88	17.07	18.5
11.06	49.6 kN (4 960 kgf) (0.112 MPa)	25.37	14.52	11.73	16.87	
11.40	65.1 kN (6 510 kgf) (0.150 MPa)	25.00	14.00	10.36	16.45	
11.74	80.6 kN (8 060 kgf) (0.182 MPa)	24.70	13.90	10.16	16.25	
12.08	94.1 kN (9 410 kgf) (0.217 MPa)	24.77	13.85	9.99	16.20	

Calculations $K_u = \dfrac{0.07}{\text{[Total deflection in cm at 34.1 kN (3 410 kgf) Total deflection at 3.1 kN (310 kgf) load]}} = \dfrac{0.07}{0.038} = 1.85$ MPa/cm

or $\dfrac{0.7}{0.038} = 18.5$ kgf/cm^2/cm.

CORRECTION FOR K-VALUE

SL NO.	MODULUS OF SUBGRADE MPa/cm (kgf/cm²/cm)			
	Uncorrected value K_u for 75 cm ∅	* Corrected for load deformation curve for 75 cm ∅ K_d	† Corrected for bending of plate of 75 cm ∅ K_b	‡ Corrected for saturation 75 cm ∅ plate (to be done at the end) $K_s = \frac{d}{d_s} \times K_b$
1	1.85 (18.5)	1.555 (15.55)	1.21 (12.1)	0.97 (9.7)

* For applying correction to the K_u value obtained in the field using 75 cm dia plate for load deflection, plot load-deflection curves as explained in **5.2.2** and demonstrated in Fig. 4. Assuming that the curve obtained is for shear failure having concave shape upwards, draw a dotted correction curve passing through origin, parallel to the shear failure curve. Enter at the loading intensity of 0.07 MPa (0.7 kg/cm²) and determine corrected deflection. From Fig. 4, we obtain 0.045. Corrected value for load deflection is $K_d = \frac{0.07}{0.045} \left(\frac{0.7}{0.045}\right) = 1.555$ MPa/cm (15.55 kgf/cm²/cm).

† Correction for bending of plate of 75 cm dia shall be applied as shown in Fig. 5 (*see* **5.1.3**). Enter at $K = 1.555$ MPa/cm (15.55 kg/cm²/cm), and read the corrected values for bending of plate, that is $K_b = 1.21$ MPa/cm (12.1 kgf/cm²/cm).

‡ Assuming $\frac{d}{d_s} = 0.8$ which is obtained from consolidation test result (refer **5.1.4**).

Substituting value of $\frac{d}{d_s}$ in equation $K_s = \frac{d}{d_s} \times K_u = 0.8 \times 1.21$ $(0.8 \times 12.1) = 0.97$ MPa/cm (9.7 kgf/cm²/cm)

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SECTION 5

Standard Penetration Test

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Indian Standard

SPECIFICATION FOR SPLIT SPOON SAMPLER

(Incorporating Amendment No. 1)

0. FOREWORD

0.1 The equipment covered in this standa-d is used for conducting the *in-situ* standard penetration test in soils covered in IS 2131:1981.

1. SCOPE

1.1 This standard covers the requirements of split spoon sampler used for conducting *in-situ* standard penetration test in soils.

2. DIMENSIONS

2.1 Dimensions with tolerances of different component parts of this apparatus are given in Fig. 1 to 6. Except where tolerances are specifically mentioned against the dimensions, all dimensions shall be taken as nominal dimensions and tolerance shall be as given in IS 2102 (Part 1): 1980

3. MATERIALS

3.1 The materials of construction of the different component parts shall be as given in Table 1.

4. CONSTRUCTION

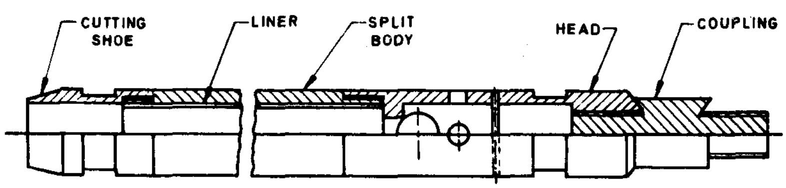
4.1 The split spoon sampler shall be constructed as per details given in Fig. 1 to 4 and 6. The split spoon sampler may also be provided with a liner in the body as per details given in Fig. 5 in which case it is called as composite sampler.

NOTE — The composite sampler is not used for the determination of *N* values.

5. MARKING

5.1 The following information shall be clearly and indelibly marked on each component of the equipment :

a) The name of the manufacturer or his registered trade-mark or both;
b) Date of manufacture; and
c) Type (*see* **4.1**).



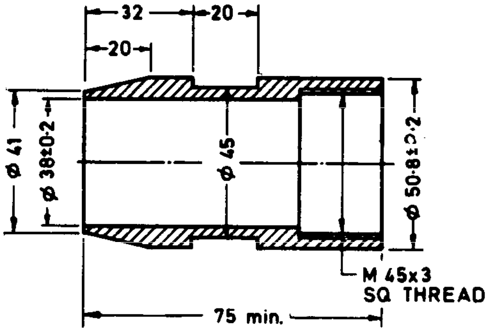
All dimensions in millimetres.

FIG. 1 ASSEMBLY OF SPLIT SPOON SAMPLER

TABLE 1 MATERIALS OF CONSTRUCTION OF DIFFERENT COMPONENT PARTS OF THE SPLIT SPOON SAMPLER

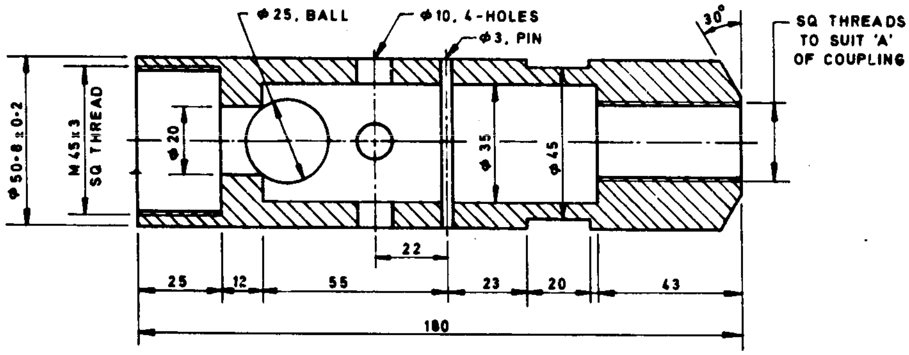
(*Clause* 3.1)

PART	MATERIALS	SPECIAL REQUIREMENTS	CONFORMING TO
Cutting shoe (*see* Fig. 2)	Mild steel, case-hardened	Cutting edge case-hardened to 45 HRC, *Min*	IS 4432 : 1967
Head (*see* Fig. 3)	Mild steel, case-hardened	Smooth surface	IS 4432 : 1967
Body (*see* Fig. 4)	Mild steel	Smooth surface	IS 513 : 1973
Liner (*see* Fig. 5)	Brass pipe	Smooth surface	IS 407 : 1966
Coupling (*see* Fig. 6)	Mild steel	To suit A-Type drill rods	IS 1239 (Part 1) : 1973



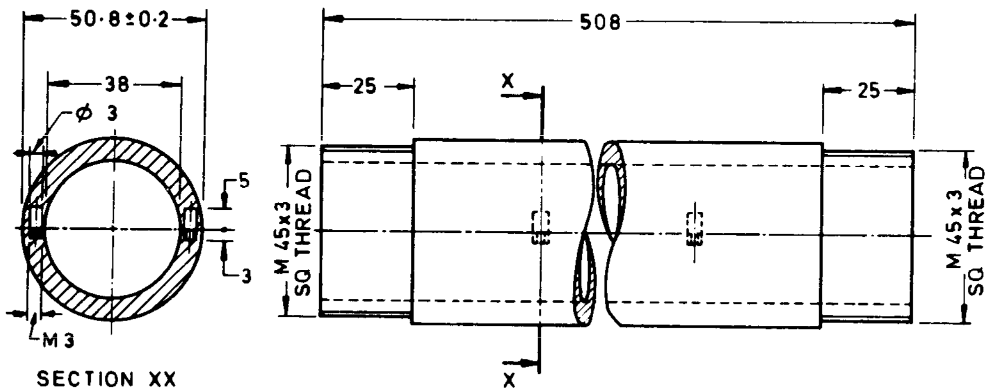
All dimensions in millimetres.

FIG. 2. CUTTING SHOE



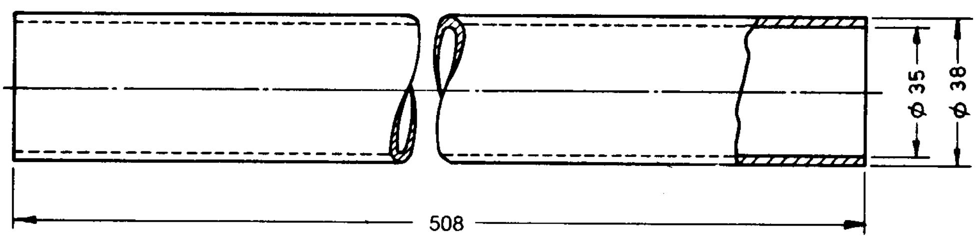
All dimensions in millimetres.

FIG. 3 HEAD



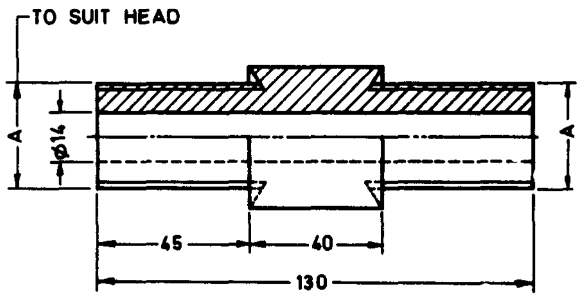
All dimensions in millimetres.

FIG. 4 SPLIT BODY (FOR USE WITH LINER)



All dimensions in millimetres.

FIG. 5 LINER



All dimensions in millimetres.

FIG. 6 COUPLING

5.1.1 The equipment may also be marked with the ISI Certification Mark.

NOTE — The use of the ISI Certification Mark is governed by the provisions of the Indian Standards Institution (Certification Marks) Act and the Rules and Regulations made thereunder. The ISI Mark on products covered by an Indian Standard conveys the assurance that they have been produced to comply with the requirements of that standard under a well-defined system of inspection, testing and quality control which is devised and supervised by ISI and operated by the producer. ISI marked products are also continuously checked by ISI for conformity to that standard as a further safeguard. Details of conditions under which a licence for the use of the ISI Certification Mark may be granted to manufacturers or processors, may be obtained from the Indian Standards Institution.

Indian Standard

METHOD FOR STANDARD PENETRATION TEST FOR SOILS

(First Rivision)

0. FOREWORD

0.1 Standard penetration test conducted by means of the split spoon, specified in this standard, furnishes data about resistance of the soils to penetration which can be used to evaluate standard strength data, such as *N* values (number of blows per 30 cm of penetration using standard split spoon) of the soil. Methods of calculation of bearing capacity of soils based on *N* values are covered in IS 6403 : 1981. For obtaining dependable and reproducable samples, a standard procedure is necessary and this code is intended to furnish necessary guidance to the soil explorer in this regard.

0.2 This standard was first published in 1963 and this revision has been done so as to include a standardized split spoon sampler for which a detailed specification has been formulated separately, besides including the details of the correction factors which are necessary in calculation of bearing capacity.

1. SCOPE

1.1 This standard specifies a standard procedure for conducting the standard penetration test for soils.

2. EQUIPMENT

2.1 Drilling Equipment

2.1.1 The equipment used shall provide a clean borehole, 100 to 150 mm in diameter, for insertion of the sampler to ensure that the penetration test is performed on undisturbed soil and shall permit driving of the split spoon sampler to obtain penetration record and the sample in accordance with the procedure specified in **3**.

NOTE — The stiffness of the drill rod used for testing influences the *N* value obtained by means of the test. A light rod 'whips' under the blows of the hammer. The drill rod shall preferably have a stiffness equal to A-rod (41.3 mm outer diameter). For depths of exploration more than 10 m, special precautions shall be taken to keep the rod vertical by using centering spacers and/or by using stiffer rods to minimize the whipping effect. Spacers may be provided at every 10 m or more frequently, if necessary.

2.1.2 *Casing or Drilling Mud* — It shall be used when drilling in sand, soft clay or other soils in which the sides of borehole are likely to cave in. In sandy and other non-cohesive soils below water table, it is often preferable to use drilling mud rather than a casing. If drilling mud alone is not successful, casing may be used along with the drilling mud.

2.2 Split Spoon Sampler — The split spoon sampler shall conform to IS 9640:1980.

2.3 Drive Weight Assembly

2.3.1 The drive weight assembly shall consist of a driving head and a 63.5 kg weight with 75 cm free fall. It shall be ensured that the energy of the falling weight is not reduced by friction between the drive weight and the guides or between rope and winch drum.

2.3.2 The rods to which the sampler is attached for driving should be straight, tightly coupled and straight in alignment. For driving the casing, a hammer heavier than 63.5 kg may be used.

2.4 Lifting Bail, Tongs, Rope, Screw Jack, etc

3. PROCEDURE

3.1 Driving the Casing — Where casing is used, it shall not be driven below the level at which the test is made or soil sample is taken. In the case of cohesionless soils which cannot stand without casing, the advancement of the casing pipe should be such that it does not disturb the soil to be tested or sampled; the casing shall preferably be advanced by slowly turning the casing rather than by driving, as the vibration caused by driving may alter the density of such deposits immediately below the bottom of the borehole.

3.2 Cleaning the Borehole

3.2.1 In case wash boring is adopted for cleaning the borehole, side-discharge bits are permissible, but in no case shall a bottom-discharge bit be permitted. The process of jotting through an open tube sampler, and then testing and sampling, when the desired depth is reached shall not be permitted.

3.2.2 While boring through soils such as sands that may be disturbed by the flow of water into the drill hole, no water shall be added to the borehole while boring above the water table. While boring below water table, the water in the borehole shall be maintained at least 1.5 m above the level of the water table. Bentonite slurry of appropriate consistency may be required to help the water level to be maintained above the water table. The raised level of the water in the borehole should be maintained even if casing is used to stabilize the borehole.

3.2.2.1 While boring through sand using casing to stabilize the sides of the borehole, the outer diameter of the shell shall be at least 25 mm smaller than the inner diameter of the casing. The distance between the end of the casing and the bottom of the borehole should be as close as possible and in any case not exceed 150 mm, if only water is used to stabilize the borehole; in case bentonite is used, this distance may be up to 300 mm.

3.2.3 The borehole shall be cleaned up to testing or sampling elevation, using suitable tools such as augers, that will ensure that there is minimum mixing up of the soil from the bottom of the borehole. In cohesive soils, the borehole may be cleaned with bailer with a flap valve. This should not be used in sands.

3.3 Obtaining the Samples

3.3.1 Tests shall be made at every change in stratum or at intervals of not more than 1.5 m whichever is less. Tests may be made at lesser intervals, if specified or considered necessary. The intervals be increased to 3 m if in between vane shear test is performed.

3.3.2 The sampler shall be lowered to the bottom of the borehole. The following information shall be noted and recorded:

a) Depth of bottom of borehole below ground level,
b) Penetration of the sampler into the soil under the combined weight of sampler and rods (to be noted from readings of the scale over the drill rod at the top),
c) Water level in the borehole or casing, and
d) Depth of bottom of casing below ground level.

3.3.3 The split spoon sampler resting on the bottom of borehole should be allowed to sink under its own weight; then the split spoon sampler shall be seated 15 cm with the blows of the hammer falling through 75 cm. Thereafter, the split spoon sampler shall be further driven by 30 cm or 50 blows (except that driving shall cease before the split spoon sampler is full). The number of blows required to effect each 15 cm of penetration shall be recorded. The first 15 cm of drive may be considered to be seating drive. The total blows required for the second and third 15 cm of penetration shall be termed the penetration resistance N; if the split spoon sampler is driven less than 45 cm (total), then the penetration resistance shall be for the last 30 cm of penetration (if less than 30 cm is penetrated, the logs should state the number of blows and the depth penetrated).

3.3.3.1 The entire sampler may sometimes sink under its own weight when very soft sub-soil stratum is encountered. Under such conditions, it may not be necessary to give any blow to the split spoon sampler and SPT value should be indicated as zero.

3.3.4 If, on lowering the sampler by means of a string of rods, it is found to rest at a level above the bottom of the casing, the penetration test and sampling should not be carried out at that stratum.

3.4 Removal of Sampler and Labelling

3.4.1 The sampler shall be raised to the surface and opened. A typical sample or samples of soil from the opened split spoon shall be put into jars without ramming. The jars shall have a self-sealing top, or shall be sealed with wax to prevent evaporation of the soil moisture. Jars shall be of such a size that they can be filled without deforming the sample. Typical samples shall be cut to such a size as to fill the jars and thereby reduce the water loss to the air in the jars. If packing, as specified, is not available, liner may be used in the sampling spoon. In such a case, the internal diameter of the sampling spoon should be so adjusted that the total internal diameter after incorporating the liner is 35 mm. The sample in the liner shall be waxed properly at both the ends to keep up the natural moisture content during transit.

3.4.2 Labels shall be fixed to the jar or notations shall be written on the covers (or both) with the following information:

a) Origin of sample,
b) Job designation,
c) Boring number,
d) Sample number,
e) Depth of sampling,
f) Penetration record,
g) Length of recovery, and
h) Date of sampling.

3.4.3 The jars containing samples shall be stored in suitable containers for shipment. Samples shall not be placed in the sun.

3.5 Field Observations

3.5.1 Information with regard to water table, elevations at which the drilling water was lost or elevations at which water under excess pressure was encountered shall be recorded on the field logs. Water levels before and after putting the casing, where used, shall be measured. In sands, the level shall be determined as the casing is pulled and then measured at least 30 minutes after the casing is pulled; in silts, at least 24 hours after the casing is pulled; in clays, no accurate water level determination is possible unless pervious seams are present. However, the 24 hours level shall be recorded for clays. When drilling mud is used and the water level is desired, casing perforated at the lower end shall be lowered into the borehole and the borehole bailed down. Ground water levels shall be determined after bailing at time intervals of 30 minutes and 24 hours until all traces of drilling mud are removed from inside the casing.

3.6 Corrections

3.6.1 *Due to Overburden* — The N value for cohesionless soil shall be corrected for overburden according to Fig. 1 (N')

3.6.2 *Due to Dilatancy* — The value obtained in **3.6.1** shall be corrected for dilatancy if the stratum consists of fine sand and silt below water table for values of N' greater than 15 as under (N'') :

$$N'' = 15 + \frac{1}{2} (N' - 15)$$

4. REPORT

4.1 Data obtained in borings shall be recorded in the field and shall include the following :

a) Date of boring,
b) Reference datum,
c) Job identification,

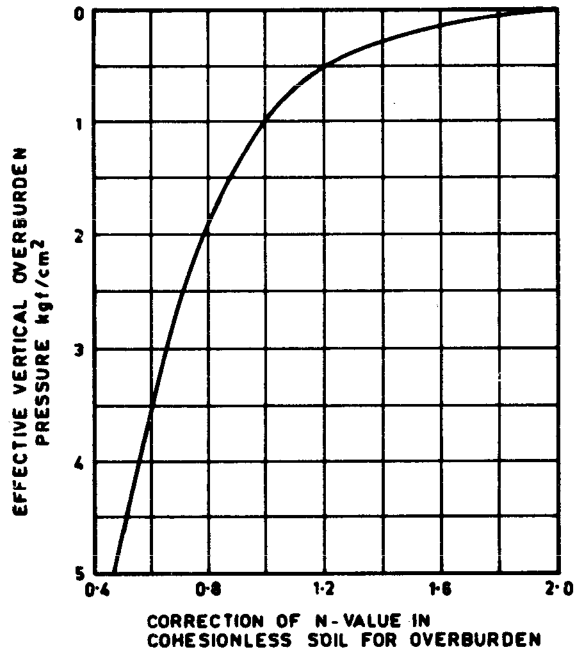


FIG. 1 CORRECTION DUE TO OVERRBURDEN

d) Boring number,
e) Sample number,
f) Type of sampler,
g) Drilling method,
h) Sample elevation and recovery ratio,
j) Limits of stratum,
k) Water table information (*see* 3 5),
m) Soil identification, including condition of samples.
n) Penetration records,
p) Casing used, and
q) Weather data.

4.2 The data obtained shall be prepared in a fianl form as a soil profile to show the nature and extent of the soil strata over the area under consideration.

SECTION 6

Subsurface Sounding of Soils

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Indian Standard

SPECIFICATION FOR EQUIPMENT FOR SUBSURFACE SOUNDING OF SOILS

0. FOREWORD

0.1 The equipment covered in this standard is used for determination of the resistance of soil strata to dynamic penetration as covered in IS 4968 (Part 1) : 1976 and IS 4968 (Part 2) : 1976.

1. SCOPE

1.1 This standard covers the specification of the equipment used for determining the subsurface sounding property of soil using cone with the dynamic method.

2. TYPES, DIMENSIONS AND CONSTRUCTION

2.1 There shall be two types A and B of equipment, the dimensions and tolerances of their parts shall be as detailed in Fig. 1 to 6 as applicable. Except where tolerances are especially mentioned, all dimensions should be taken as nominal dimensions and tolerances shall be as given for medium class in IS 2102 (Part 1):1980.

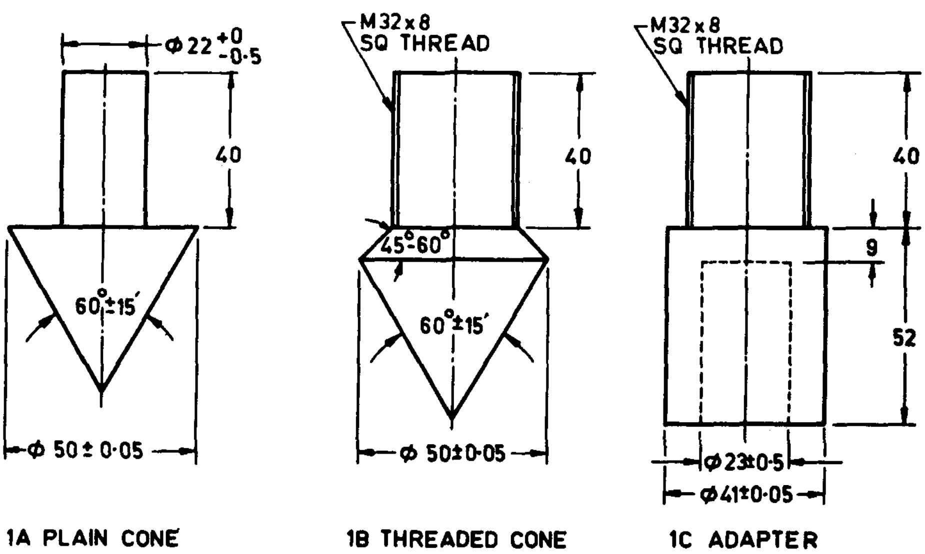
3. MATERIALS

3.1 Materials for construction of various parts of this equipment shall be as given in Table 1.

4. MARKING

4.1 The following information shall be clearly and indelibly marked on each equipment :

a) Name of the manufacturer or his registered trade-mark or both,
b) Date of manufacture, and
c) Type of equipment.



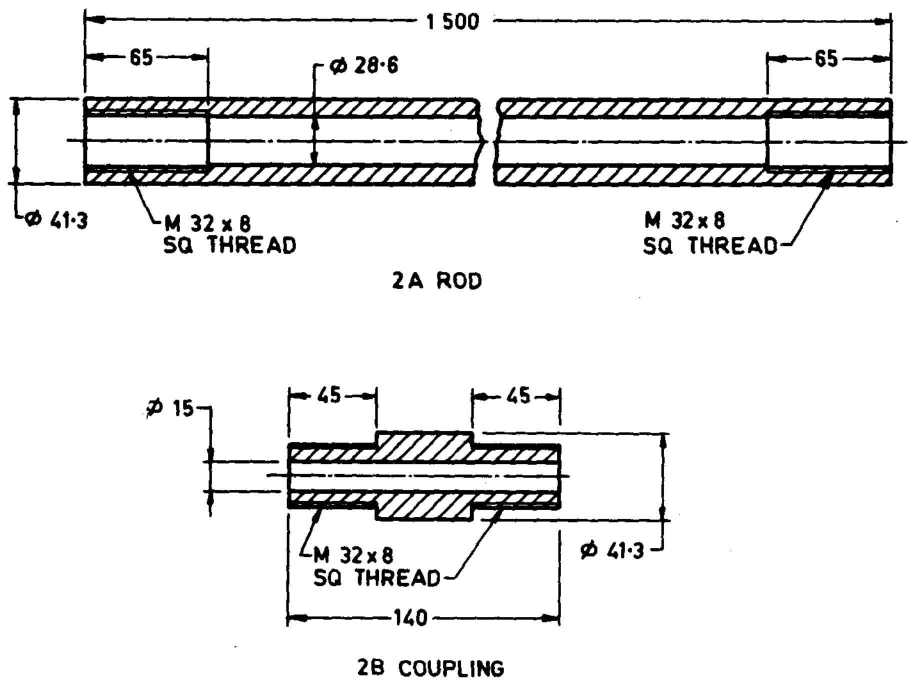
All dimensions in millimetres.

FIG. 1 CONE AND ADAPTER

TABLE 1 MATERIALS FOR CONSTRUCTION OF EQUIPMENT PART

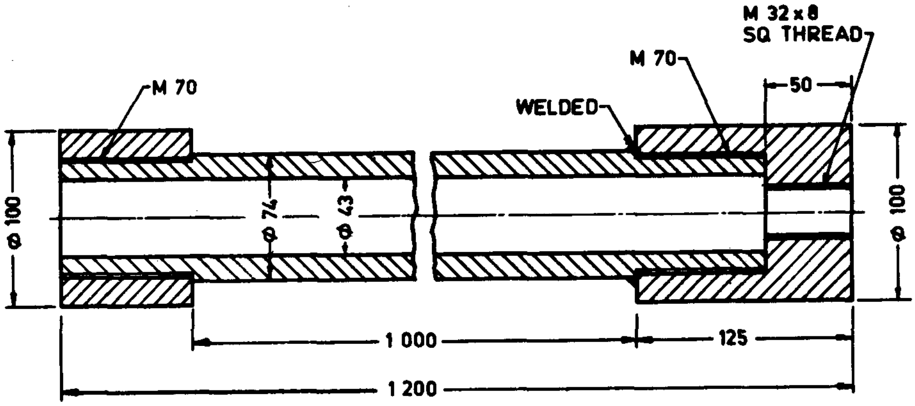
(*Clause* 3.1)

SL No.	EQUIPMENT PART	MATERIAL	SPECIAL REQUIREMENT	RELEVANT INDIAN STANDARD OR REFERENCE
i)	Threaded cone or plain cone with adapter	Steel	Hardened to 50-55 *HRC* and conical surface shall be machined smooth	IS 5517 : 1978
ii)	Driving rod, guide rod (for Type A), coupling and driving head (for Type B)	Steel	—	IS 5517 : 1978
	NOTE 1 — For Type B, a driving rod with driving head shall be used as guide rod.			
	NOTE 2 — The number of driving rod and coupling shall be as required.			
iii)	Hammer	Steel	The weight shall be 65 kg and tensile strength of wire rope shall be 1 400 kg/cm^2 minimum	IS 1875 : 1978
iv)	Hoisting equipment :			
	a) Tripod legs	Mild steel	—	IS 1239 (Part 1) : 1979 or IS 226 : 1975
	b) Pulley	Steel	—	IS 1875 : 1978
	c) Other parts like winch connecting pins, hook, axle, etc	Steel	—	IS 1875 : 1978

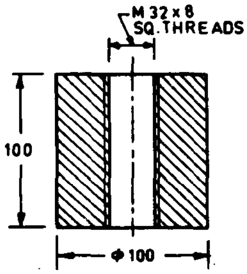


All dimensions in millimetres.

FIG. 2 DRIVING ROD



3 A Guide Rod for Type A Equipment

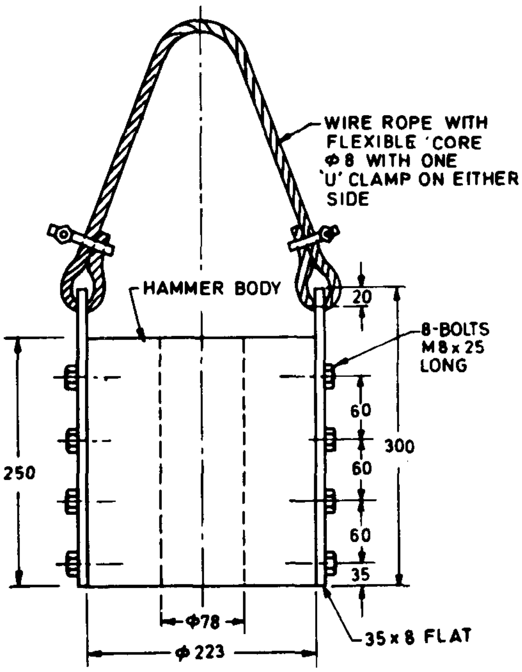


3 B Driving-Head for Type B Equipment

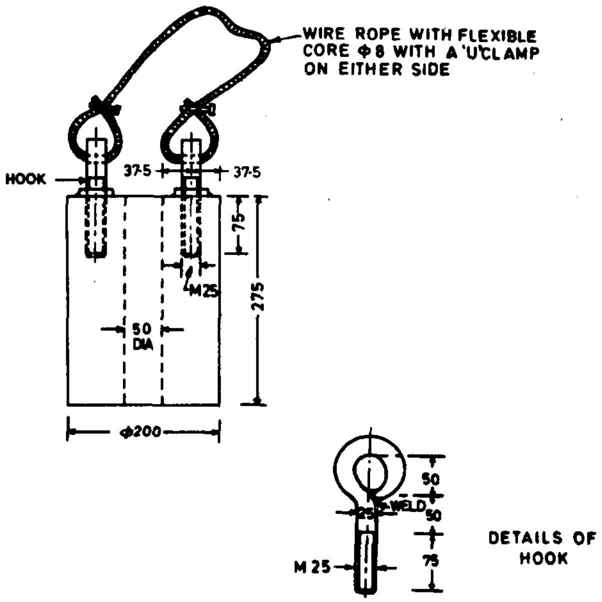
NOTE — The guide rod for Type B shall be ordinary driving rod given in Fig. 2 A fixed with this driving head

All dimensions in millimetres:

FIG. 3 GUIDE ROD



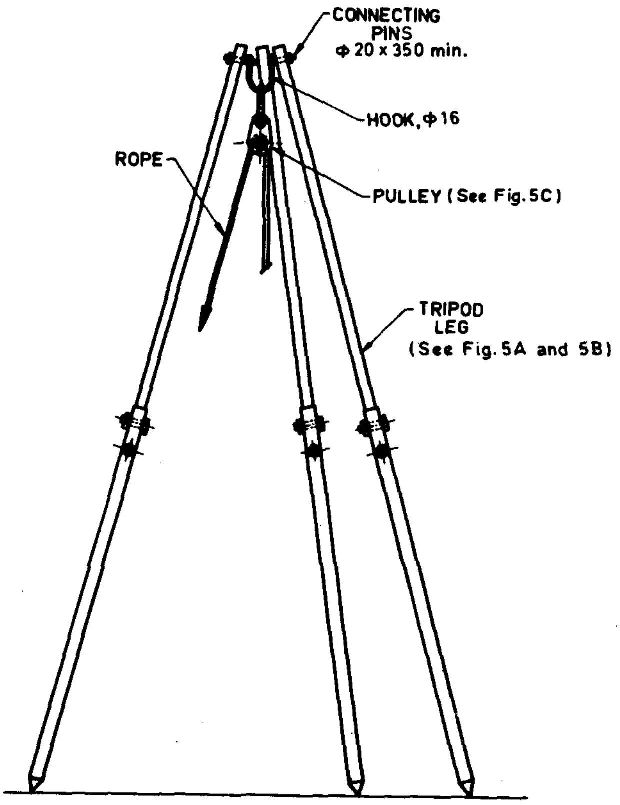
4 A For Type A Equipment



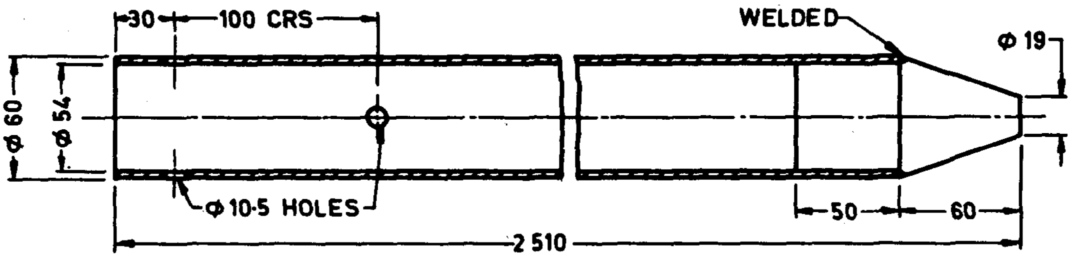
4 B For Type B Equipment

All dimensions in millimetres.

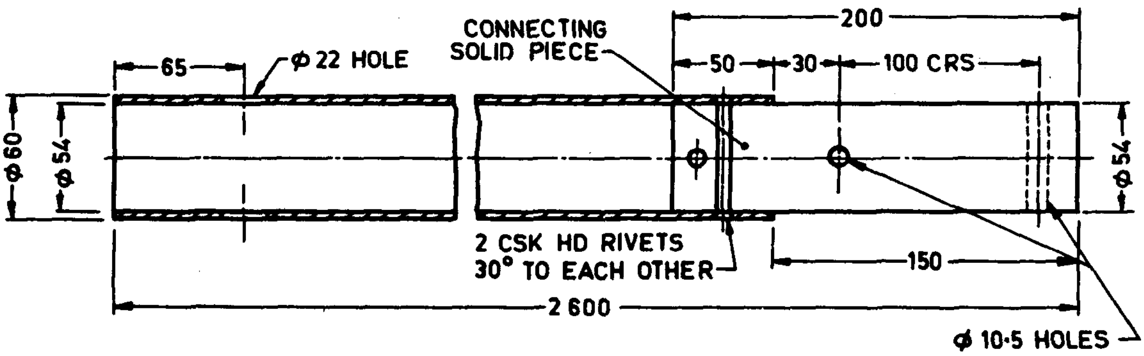
FIG. 4 HAMMER



General Assembly of Hoisting Equipment for Type A Equipment



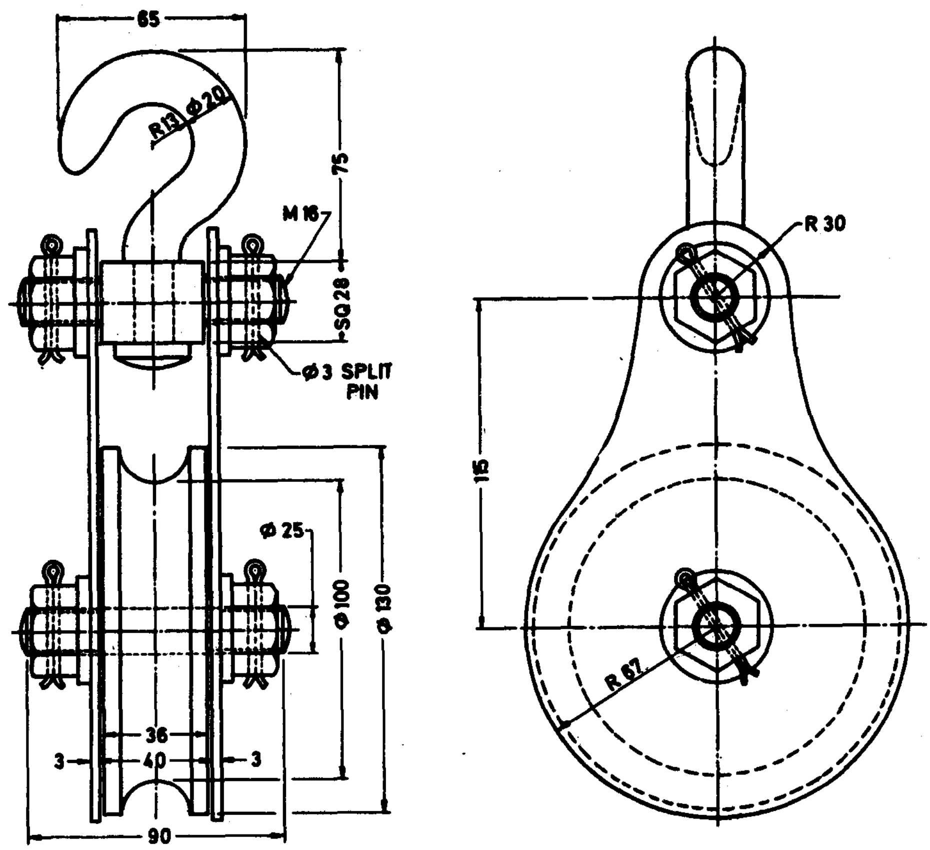
5 A Tripod Leg (End Piece) of Type A Equipment



5 B Tripod Leg of Type A Equipment

All dimensions in millimetres.

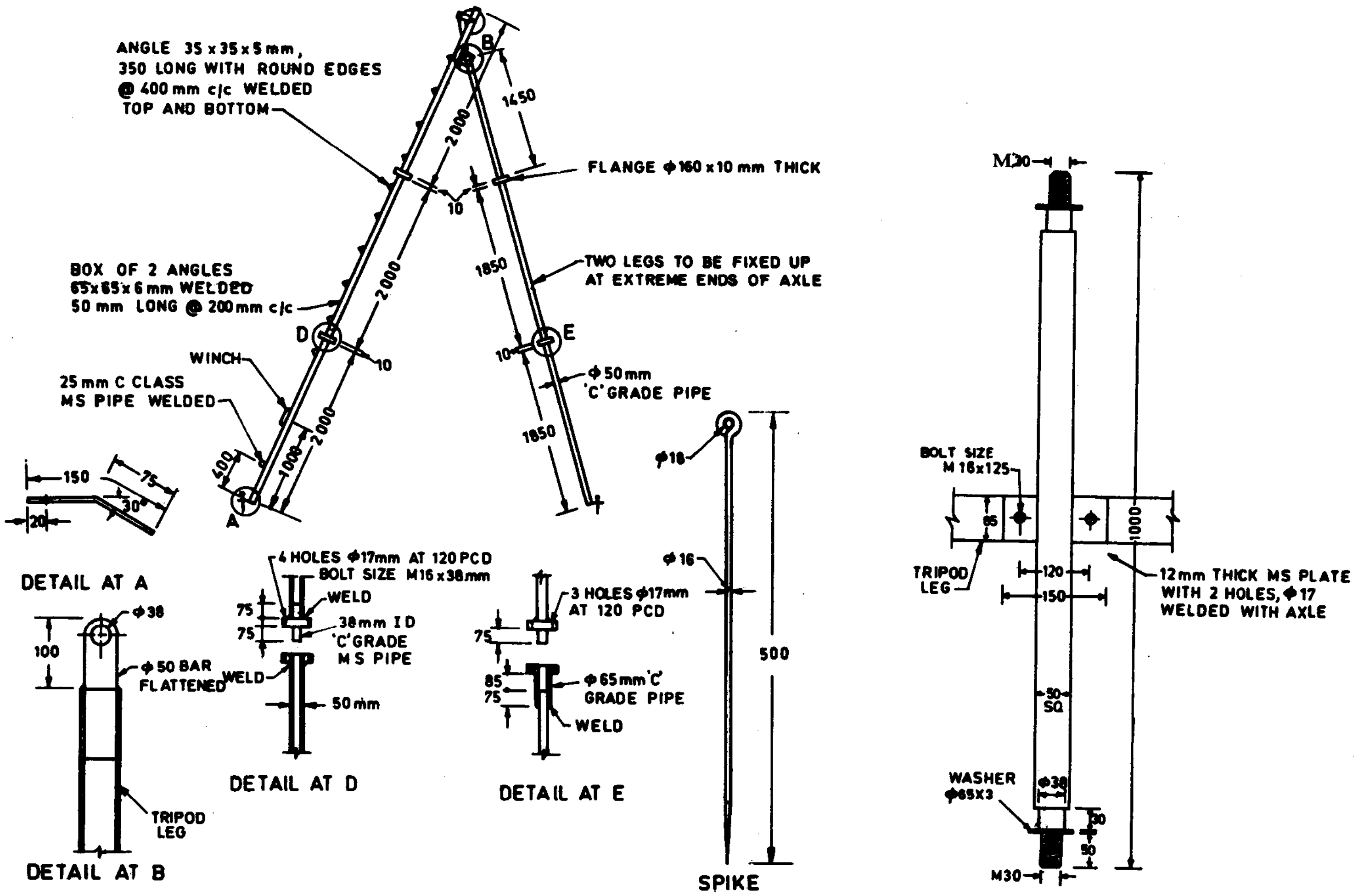
FIG. 5 DETAILS OF HOISTING EQUIPMENT TYPE A—*Contd*



5 C Pulley for Type A Equipment

All dimensions in millimetres.

FIG. 5 DETAILS OF HOISTING EQUIPMENT TYPE A

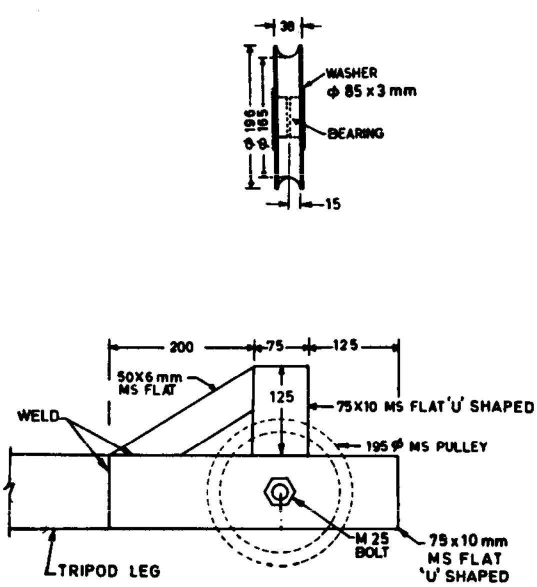


6 A Details of Tripod

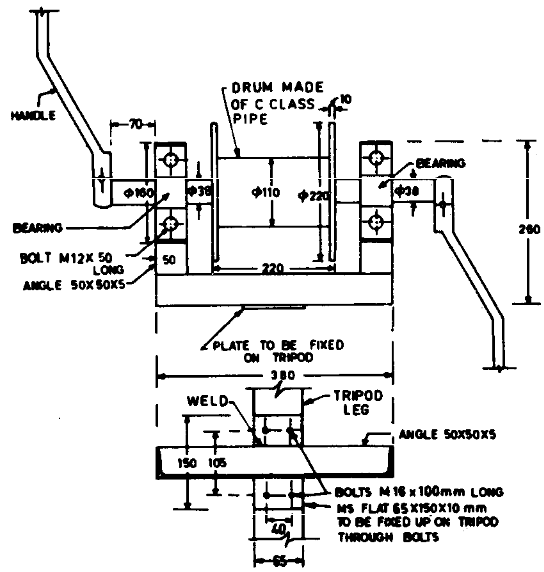
6 B Details of Axle and Fixing Arrangement for Type B Equipment

All dimensions in millimetres.

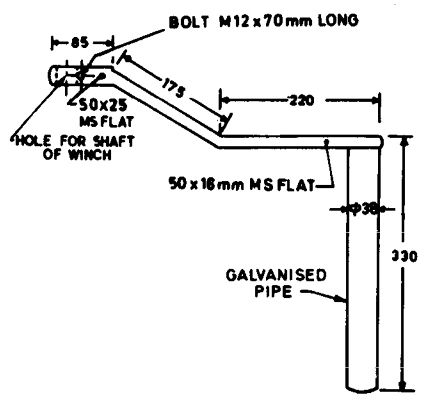
FIG. 6 DETAILS OF HOISTING EQUIPMENTS TYPE B—*Contd*



6 C Details of Pulley Fixture at Tripod for Type B Equipment



6 D Details of Winch for Type B Equipment



6 E Handle for Winch for Type B Equipment

All dimensions in millimetres.

FIG. 6 DETAILS OF HOISTING EQUIPMENTS TYPE B

4.1.1 The equipment may also be marked with the ISI Certification Mark.

NOTE — The use of the ISI Certification Mark is governed by the provisions of the Indian Standards Institution (Certification Marks) Act and the Rules and Regulations made thereunder. The ISI Mark on products covered by an Indian Standard conveys the assurance that they have been produced to comply with the requirements of that standard under a well-defined system of inspection, testing and quality control which is devised and supervised by ISI and operated by the producer. ISI marked products are also continuously checked by ISI for conformity to that standard as a further safeguard. Details of conditions under which a licence for the use of the ISI Certification Mark may be granted to manufacturers or processors, may be obtained from the Indian Standards Institution.

Indian Standard
METHOD FOR SUBSURFACE SOUNDING FOR SOILS
PART 1 DYNAMIC METHOD USING 50 mm CONE WITHOUT BENTONITE SLURRY

(First Rivision)

(Incorporating Amendment No. 1)

0. FOREWORD

0.1 The resistence, N_{cd} (*see* Note) to penetration of the cone in terms of number of blows per 300 mm of penetration may be correlated with the bearing capacity of cohesionless soils and also possibly with the load carrying capacity of piles. The correlations are qualitative rather than quantitative in nature, and are influenced by the character of the soils, such as grain-size distribution, surcharge pressure, permeability and degree of saturation. The extra work required to determine the penetration resistance is small compared to the value of the data obtained, but these data only provide a rough indication of the consistency or relative density of the soil.

NOTE — The resistance to penetration in the standard penetration test (IS 2131 : 1981) shall be designated as N, that to a 50 mm cone as N_{cd} and that to a 62.5 mm cone using bentonite slurry as N_{cbr} [IS 4968 (Part 2) : 1976].

0.1.1 Correlation between cone penetration values (N_{cd}) and penetration values obtained by other methods may be developed for a given site by conducting the latter tests adjacent (about 3 to 5 m) to the location of the cone test (*see* Note).

NOTE — However, for the 62.5 mm cone driven dry up to a depth of 9 m (without bentonite slurry) [*see* IS 4968 (Part 2) : 1976] for medium to fine sands, the following relationships have been developed by the Central Building Research Institute, Roorkee. These relationships, when utilized, shall be used with caution.

N_{cbr} = 1.5 N up to a depth of 4 m

N_{cbr} = 1.75 N for depths of 4 to 9 m.

where

N_{cbr} = cone resistance obtained with a 62.5 mm cone driven dry (number of blows for 300 mm penetration); and

N = resistance to penetration in the standard penetration test (in accordance with IS : 2131 : 1981), (number of blows for 300 mm penetration).

0.2 This standard was first published in 1968. In this revision, several changes have been made taking into consideration the experience gained in conducting the test and in the manufacture of the equipment. The major changes made relate to the material of the cone and the hammer criteria for stopping of driving of the cone and the limitations. Reference has also been made to the automatic arrangement for controlling the drop of the hammer.

0.3 In the formulation of this standard, due weightage has been given to international co-ordination among the standards and practices prevailing in different countries in addition to relating it to the practice in the field in this country.

1. SCOPE

1.1 This standard covers the procedure for determining the resistance of different soil strata to dynamic penetration of a 50-mm cone and thereby obtaining an indication regarding their relative strengths or density or both. The method helps reconaissance survey of wide areas in a shorter time which will enable selective *in-situ* testing or sampling for typical profile. It can provide useful data for local conditions where reliable correlations have been established.

2. EQUIPMENT

2.1 The cone driving rods, dividing head, hoisting equipment shall conform to IS 10589 : 1983.

3. PROCEDURE

3.1 The 50 mm diameter 60° cone shall be fitted loosely to the driving rod through a cone adopter or the threaded cone shall be screwed to the driving rod. The hammer head shall be joined to the other end of the driving rod with rod coupling. A guide rod 150 cm long shall be connected to the hammer head. This assembly shall be kept vertical with the cone resting on the ground to be tested. The cone shall then be driven into the soil by allowing the 65 kg hammer to fall freely through a height of 750 mm each time. The number of blows for every 100 mm penetration of the cone shall be recorded. The process shall be repeated till the cone is driven to the required depth (*see* Note and **4.1**).

NOTE — To save the equipment from damage, driving may be stopped when the number of blows exceeds 35 for 100 mm penetration.

4. LIMITATIONS

4.1 The maximum depth to which the cone should be driven will depend upon the type of soil, the position of the water table and the purpose of the test. If correlations of cone penetration values obtained by other methods is desired in interpretation, in cohesionless soils the depth may be limited to 5 m; in mixed soil with some binding material, the depth may be 10 m. If the test is used for obtaining a general qualitative idea of the strata, the cone may be driven to any convenient depth.

5. REPORT

5.1 The number of blows (N_{cd}) as a continuous record for every 300 mm of penetration shall be shown in a tabular statement or shown as a graph between N_{cd} and depth. Records of the test shall also include the following :

a) Date of probing;
b) Location;
c) Elevation of ground surface;
d) Depth of water table and its likely variation, from available information;
e) Total resistance at the required levels;
f) Any interruption in probing, with reasons;
g) Any other information available, for example, type of soil; and
h) Diameter of the cone used in the test.

Indian Standard

METHOD FOR SUBSURFACE SOUNDING FOR SOILS

PART 2 DYNAMIC METHOD USING CONE AND BENTONITE SLURRY

(First Rivision)

(Incorporating Amendment No. 1)

0. FOREWORD

0.1 Dynamic cone penetration test is a simple device for probing the soil strata and it has an advantage over the standard penetration test that making of a bore hole is avoided. Moreover, the data obtained by cone test provides a continuous record of soil resistance. The resistance N_{cbr} (*see* Note) to penetration in terms of blows per 30 cm of penetration of the cone specified in this standard and developed by the Central Building Research Institute, Roorkee, has been co-related quantitatively to the standard penetration value N obtained in accordance with IS 2131 : 1981. Studies with a view to establish a definite co-relation between N_{cbr} and N values for different regions of the country are in progress. The Sectional Committee responsible for the preparation of this standard decided to publish this standard in the meantime so that it could serve as a basis of test to various investigators and others engaged in subsurface exploration for foundations and thus make the results of investigations comparable.

NOTE — The resistance to penetration in the standard penetration test (IS 2131 : 1981) shall be designated as N, that to a 50 mm cone [*see* IS 4968 (Part 1) : 1976] as N_{cd} and that to a 62.5 mm cone using bentonite slurry as N_{cbr}.

0.2 This standard was first published in 1968. In this revision, several changes have been made taking into consideration the experience gained in conducting the test and in the manufacture of the equipment. The major changes made relate to the material of the cone and the hammer, and the criteria for stopping of driving of the cone. The diameter of the cone has been changed to 62.5 mm and the provision permitting the use of cones of other diameters has been withdrawn. Additional information has been given on the bentonite slurry used in the test. Correlations between N_{cbr} and N values have also been included.

0.3 Correlation between cone penetration values obtained using 62.5 mm cone (N_{cbr}), and penetration values obtained by other methods may be developed for a given site by conducting the latter tests adjacent (about 3 to 5 m) to the location of the cone test. However, for medium to fine sands the following relationships between the standard penetration value (N) obtained in accordance with IS 2131 : 1981 and the cone penetration value (N_{cbr}) in accordance with method specified in this standard have been developed by the Central Building Research Institute, Roorkee. These relationships, when utilized shall be used with caution.

a) When the 62.5 mm cone is driven dry up to 9 m (without bentonite slurry):

$N_{cbr} = 1.5\,N$ .... up to a depth of 4 m
$N_{cbr} = 1.75\,N$........ for depths of 4 to 9 m

b) When the 62.5 mm cone is penetrated by circulating slurry :

$N_{cbr} = N$

0.4 In the formulation of this standard due weightage has been given to international co-ordination among the standards and practices prevailing in different countries in addition to relating it to the practice in the field in this country.

1. SCOPE

1.1 This standard (Part 2) covers the procedure of dynamic driving of a 62.5 mm cone and thereby obtaining a record of resistance of the soil. The cone is directly driven into the ground and for eliminating the friction on the driving rods, bentonite slurry is used. The use of bentonite slurry may not be necessary when the investigation required is up to a depth of 6 m only.

2. EQUIPMENT

2.1 Cone — The cone shall be of suitable steel with the tip hardened. The dimensions and the shape of the cone shall be as given in Fig. 1. The cone should be suitably threaded to enable it to be attached to *A* rods used for driving.

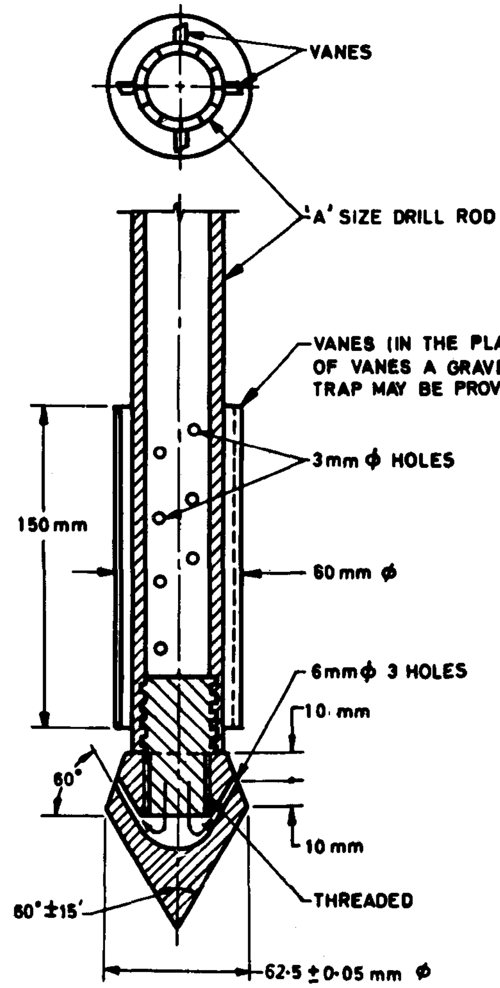


FIG. 1 CONE ASSEMBLY

2.2 Driving Rods — The rods used for the test should be *A* rods of suitable lengths with threads for joining *A* rod coupling at either end. The rods should be marked at every 100 mm.

NOTE — The outer and internal diameter of *A* rods are 41.27 and 28.57 mm respectively.

2.2.1 Four mild steel vanes as shown in Fig. 1 (*see also* **2.6**) shall be welded to the driving rod immediately above the cone. As an alternative, a gravel trap about 150 mm high of wire guage of 5 mm mesh may be provided on the rod immediately above the cone.

2.3 Driving Head — The driving head shall be of mild steel with threads at either end for *A* rods coupling (*see* Note under **2.2**). It shall have a diameter of 100 mm and a length of 100 to 150 mm.

2.4 Hoisting Equipment — Any suitable hoisting equipment, like a tripod may be used. The equipment shall be designed to be stable under conditions of impact of the hammer over the driving head when the cone is driven during the test. Provision shall be made to enable the operator to climb up the equipment for fixing the pulley, ropes, etc. A typical set up using a tripod is shown in Fig. 2. Suitable guides shall be provided for keeping the driving rods vertical and in position.

2.5 Hammer — The hammer used for driving the cone shall be of mild steel or cast iron with a base of mild steel. It shall be 250 mm high and of suitable diameter. The weight of the hammer together with the chain shall be 65 kg. It shall have hole at the centre running throughout its length and of suitable diameter for the *A* rod (*see* Note under **2.2**) and/or guide to pass freely through it. The clearance between the rod and/or guide and the hole in the hammer shall be about 5 mm.

NOTE — An automatic arrangement for controlling the drop of the hammer may be preferred if available.

2.6 Pumping Unit for Bentonite Slurry — It consists of slurry pump of capacity 35 to 45 l/min at a pressure of 700 to 850 kN/m^2 (7 to 8.5 kgf/cm^2) with a suction hose assembly and a swivel assembly. For better circulation of slurry at greater depths a vane borer consisting of four vanes and a number of drill holes for the escape of slurry may be provided in between the driving rod and the cone (*see* Fig. 1 and Fig. 2).

3. PROCEDURE

3.1 The vane shall be connected to the driving rods, with the vane borer/gravel trap in position. The driving head with the guide rod shall be fixed on the driving rods. This assembly shall be kept in position with the cone resting vertically on the ground at the point to be tested. For the circulation of slurry the guide rod shall be connected to a water swivel preferably through a flexible tube connection and then through another flexible tube to the pumping unit for bentonite slurry. The swivel assembly shall be held in position by a rope passing over the pulley provided for that purpose. The slurry tank shall be filled with bentonite slurry of suitable consistency (*see* Note). The slurry should generally be prepared separately and stored in drums. The tank end of the inlet tube to the pump shall be provided with suitable protection against entry of debris and it shall be kept immersed in the slurry tank. The hammer, to which a rope has been attached for operation, shall be slid over the guide rod, to rest on the driving head. A typical assembly of the equipment for test using a tripod is shown in Fig. 2.

NOTE — In the case of medium to fine sand, 5 percent bentonite slurry has been found useful. In the case of coarse sand, slurry of thicker consistency subject to circulation

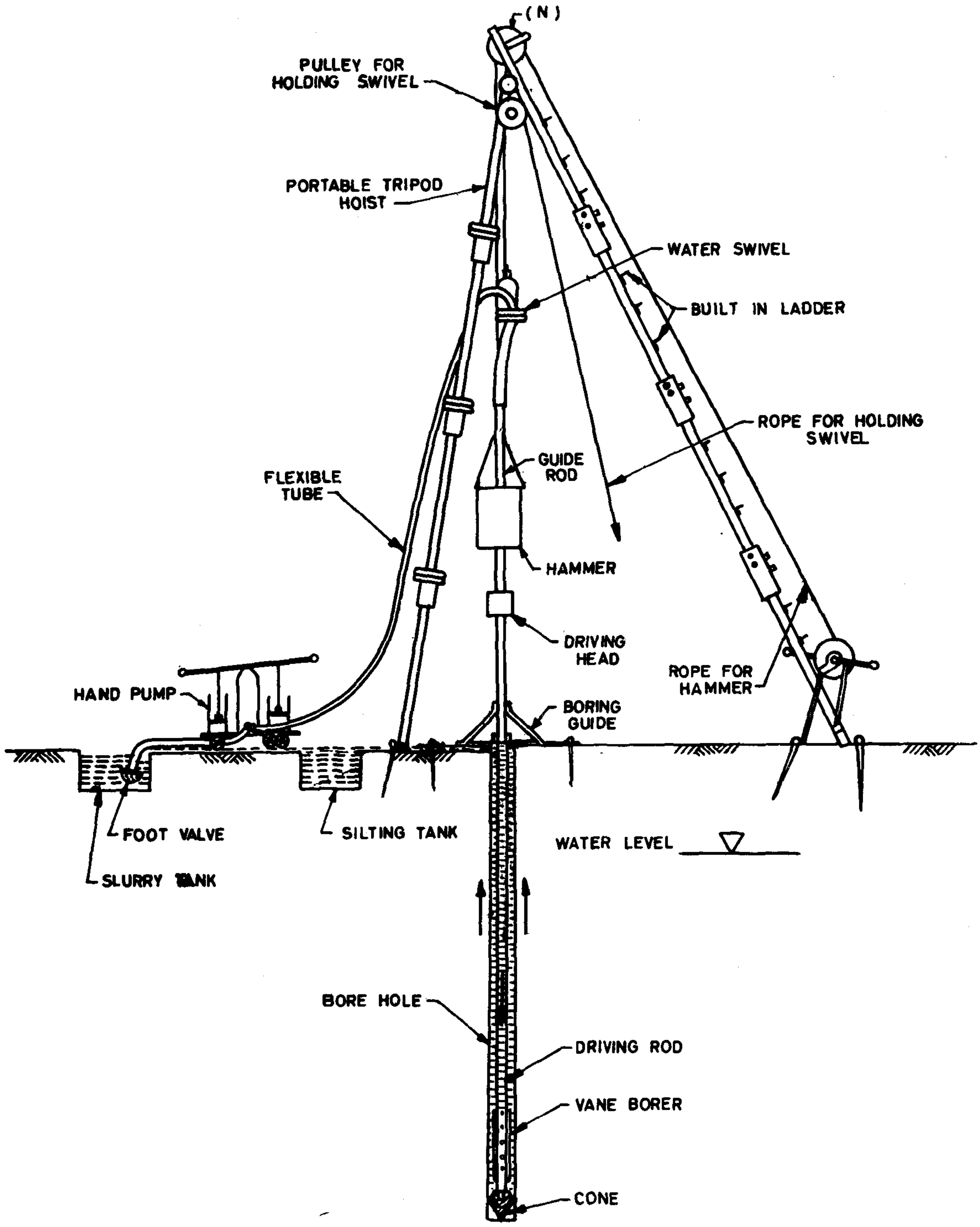


FIG. 2 A TYPICAL SET UP FOR DYNAMIC CONE PENETRATION TEST

requirements may be needed. In the case of hard water, addition of 1 percent soap solution has been found useful to get a better suspension of the bentonite.

3.2 The cone shall be driven by allowing the 65 kg hammer to drop freely through a height of 750 mm on the driving head. A drum type winch fixed to central leg of the tripod may be used for lifting the drop weight provided the free fall of the hammer is not affected. The driving of the cone and the pumping in of the slurry shall be started simultaneously. Driving shall not be done for more than 30 cm at a time after which it shall be stopped for a minute or two. Pumping shall, however, be continued. This helps in keeping the hole lined and also avoids the choking of the holes provided in the cone. The driving rods shall be given a few turns (about 4 or 5 turns) every now and then so that the hole above the cone is maintained. Efficient circulation of slurry is necessary for eliminating friction on the rods. The number of blows for every 100 mm penetration of the cone shall be recorded. The process shall be repeated till the cone is driven to the required depth (*see* Note).

NOTE — In order to avoid damage to the equipment, driving may be stopped when the number of blows exceeds 35 for 100 mm penetration when the cone is driven dry and 20 for 100 mm penetration when the cone is penetrated by cicrulating slurry.

4. REPORT

4.1 The number of blows (N_{cbr}) should be reported as a continuous record for every 300 mm penetration either in a tabular form or as a graph between N_{cbr} and depth. Records of the test shall also include the following :

a) Date of probing;
b) Location;
c) Elevation of ground surface;
d) Depth of water table and its likely variation, from available information;
e) Total resistance at the required levels;
f) Any interruptions in probing with reasons;
g) Any other information available, for example, type of soil; and
h) Diameter of the cone used in the test.

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SECTION 7

Static Cone Penetration Test

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Indian Standard

METHOD FOR SUBSURFACE SOUNDING FOR SOILS

PART 3 STATIC CONE PENETRATION TEST

(First Rivision)

0. FOREWORD

0.1 Among the field sounding tests the static cone test is a valuable method of recording variation in the *in-situ* penetration resistance of soils; in cases where the *in-situ* density is disturbed by boring operations, thus making the standard penetration test unreliable especially under water. The results of the test are also useful in determining the bearing capacity of the soil at various depths below the ground level. In addition to bearing capacity values it is also possible to determine by this test the skin friction values used for the determination of the required lengths of piles in a given situation. The static cone test is most successful in soft or loose soils like silty sands, loose sands, layered deposits of sands, silts and clays as well as in clayey deposits.

0.1.1 Experience indicates that a complete static cone penetration test up to depths of 15 to 20 m can be completed in a day with manual operations of the equipment, making it one of the inexpensive and fast methods of sounding available for investigation; in fact, in Europe it is invariably used for exploratory stage of investigations when both time and money are at a premium. In areas where some information regarding the foundation strata is already available, the use of test piles and loading tests thereof can be avoided by conducting static cone penetration tests.

0.2 This standard was first published in 1971. In this revision several changes have been made taking into consideration the experience gained in conducting the test and in the manufacture of the equipment. The essential requirements of the friction jacket have been added; tolerances have been indicated for the essential requirements; a rate of travel has been specified for the engine driven equipment. Opportunity has also been taken to give the requirements and example in SI units.

0.3 In the formulation of this standard due weightage has been given to international co-ordination among the standards and practices prevailing in different countries in addition to relating it to the practice in the field in this country.

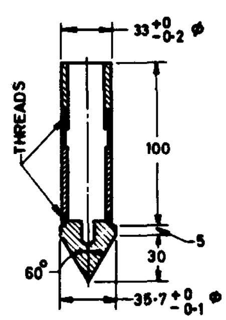
1. SCOPE

1.1 This standard (Part 3) covers the procedure for the determination of the cone resistance and friction resistance of soil at various depths below ground surface by the static cone method.

1.1.1 This standard gives the procedure for the test only and certain essential details of the equipment but does not include complete design of the equipment.

2. EQUIPMENT

2.1 Steel Cone — The cone shall be of suitable steel with its tip hardened. It shall have an apex angle of 60° ± 15 minutes and overall base diameter of $35.7^{+0}_{-0.1}$ mm giving a cross-sectional area of 10 cm^2 (*see* Fig. 1). The cone shall be so designed as to prevent the intrusion of soil particles into the moving parts of the cone assembly.



All dimensions in millimetres.

FIG. 1 CONE ASSEMBLY (WITHOUT FRICTION JACKET)

2.2 Friction Jacket — The friction jacket shall be of high carbon steel and of dimensions shown in Fig. 2.

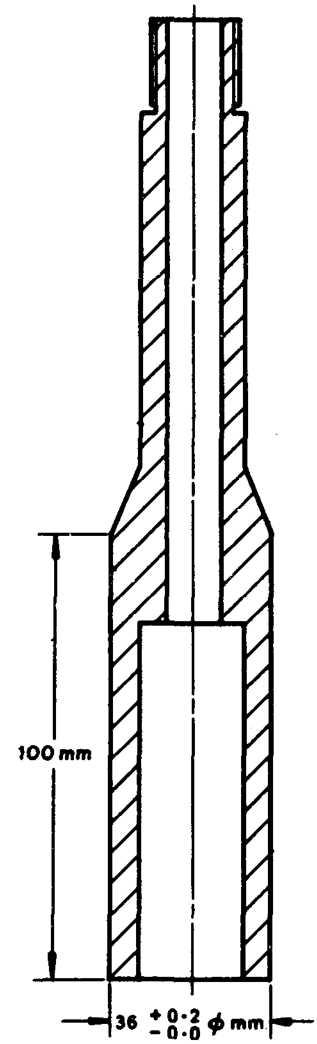
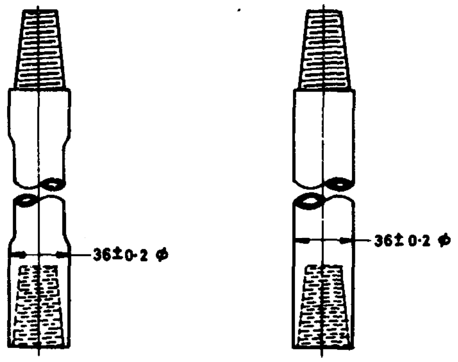


FIG. 2 FRICTION JACKET

2.3 Sounding Rod — Steel rod of 15 mm diameter which can be extended with additional rods of 1 m each in length. The sounding rod should be such that the base of the cone mentioned in **2.1** could be attached to it.

2.4 Mantle Tube — The mantle tube shall be of steel and is meant for guiding the sounding rod which goes through the mantle tube. The mantle tube should be in length of one metre with flush coupling. The diameter of the mantle tube may be non-uniform or uniform (*see* Fig. 3). In the manually operated equipment, for ease of operations, the non-uniform mantle tube is generally used.

2.5 Driving Mechanism — The driving mechanism should have a capacity of 2 to 3 tonnes for the manually operated equipment and 10 tonnes for the mechanically operated equipment. The driving mechanism essentially consists of a rack and pinion arrangement operated by a winch. The reaction for the thrust may be obtained by suitable devices capable of taking loads greater than the capacity of the equipment.



3 A Non-Uniform Dia 3 B Uniform Dia

FIG. 3 MANTLE TUBE

2.5.1 The hand-operated winch may be provided with handles on both sides of the driving frame to facilitate driving by four persons for loads greater than 2 000 kg. The winch should be equipped for two speeds controlled by 2 gears. The gear should be capable of being operated in slow and fast positions for penetration or withdrawal of the cone-friction jacket assembly.

2.5.2 For the engine driven equipment the rate of travel should be such that the penetration obtained in the soil during the test is between 1 to 1.5 cm/s.

2.6 Measuring Equipment — The sounding apparatus should be provided with hydraulically operated measuring device by which the pressure developed is indicated on the gauges. The cross-sectional area of the plunger of the measuring head may be either 10 cm^2 (same as the cross-sectional area of the cone) or 20 cm^2. Two pressure gauges should be connected to the driving head, one for high pressure and the other for low pressure, as follows for the plunger area of 20 cm^2 (*see* Note):

a) For the 2 to 3 t equipment:

1) 0 to 1 000 kN/m^2 (0 to 10 kgf/cm^2) with 25 kN/m^2 (0·25 kgf/cm^2) markings

or

0 to 5 000 kN/m^2 (0 to 50 kgf/cm^2) with 50 kN/m^2 (0·50 kgf/cm^2) markings

and

2) 0 to 15 000 kN/m^2 (0 to 150 kgf/cm^2) with 150 kN/m^2 (1·5 kgf/cm^2) markings.

As an alternative, a proving ring may also be used to record the penetration resistance of the cone fitted to a hand operated machine.

b) For the 10 t equipment:

1) 0 to 10 000 kN/m^2 (0 to 100 kgf/cm^2) with 100 kN/m^2 (1 kgf/cm^2) markings

and

2) 0 to 60 000 kN/m^2 (0 to 600 kgf/cm^2) with 500 kN/m^2 (5 kgf/cm^2) markings.

NOTE — If the plunger area is 10 cm^2, the capacity of the gauges and calculations should be adjusted appropriately.

2.6.1 In both the 2 to 3 t and 10 t equipment, the pressure gauges shall be so connected that the pressure gauge with the smaller capacity can be cut off both manually and automatically when the applied pressure exceeds its capacity.

2.7 Other Requirements of the Equipment — The equipment shall be so designed as to allow for pushing into the ground the cone alone, and the friction jacket fitted immediately above the cone and the cone together, alternately, through depths of a minimum of 35 mm each, each time. Provision shall also be made to enable the entire assembly to be advanced together continuously if skin friction readings are not required to be determined separately.

3. PROCEDURE

3.1 Basically the test procedure for determining the static cone and frictional resistances consists of pushing the cone alone through the soil strata to be tested, then the cone and the friction jacket, and finally the entire assembly in sequence and noting the respective resistance in the first two cases. The cone is pushed through a distance in accordance with the design of the equipment (*see* **2.7**) and the need for the substrata and the cone resistance noted. Thereafter, the cone and the friction jacket are pushed together for a distance depending upon the design of the cone and friction jacket assembly and the combined value of cone and friction resistance noted. This procedure is repeated at predetermined intervals. The set up for the test is illustrated in Fig. 4.

3.2 The equipment shall be securely anchored to the ground at the test point for obtaining the required reaction.

3.2.1 The rack of the driving mechanism shall be brought to the top most position. The cone-friction jacket assembly shall be connected to the first sounding rod and the mantle tube. This assembly shall be positioned over the test point through the mantle tube guide and held vertically. The plunger of the driving mechanism shall be brought down so as to rest against the protruding sounding rod.

3.2.2 For obtaining the cone resistance, the sounding rod only shall be pushed. Switching the gear clutch to the slow position, the drive handle shall be operated at a steady rate of 1 cm/s approximately (*see* Note) so as to advance the cone only to a depth which is possible with the cone assembly available (*see* **2.7**). During this pushing, the mean value of the resistance as indicated by the Bourdon gauges shall be noted ignoring erratic changes.

NOTE — In order to standardize the test procedure a rate of 1 cm/s has been specified. Tests conducted at slower rates (0·5 cm/s and 1/3 cm/s) have shown that in the case of both cohesive and non-cohesive soils the

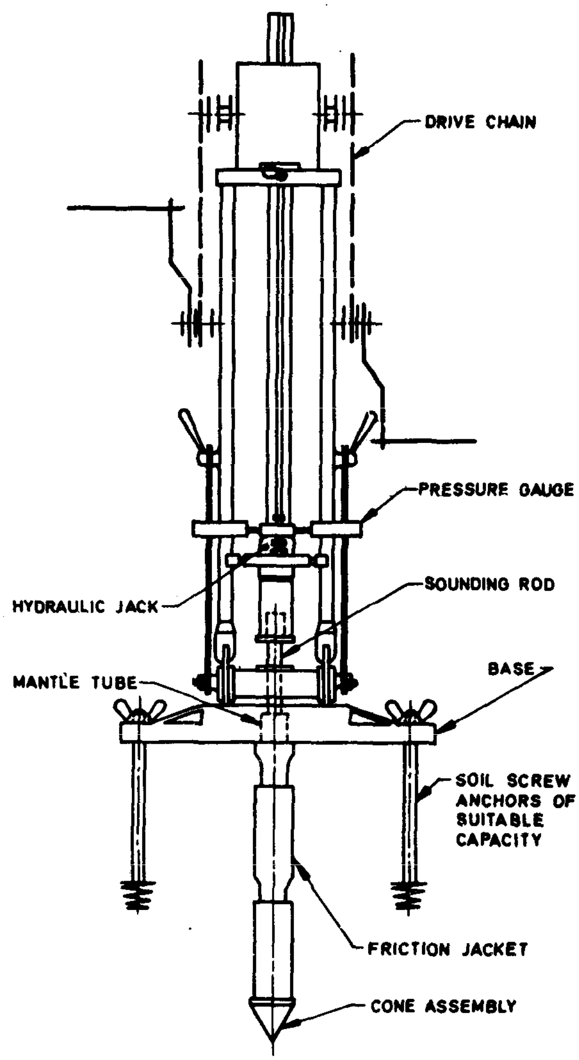


FIG. 4 TYPICAL SET UP FOR STATIC CONE PENETROMETER (HAND OPERATED)

effect of the time-rate of penetration on the cone resistance was not appreciable within the limits of these rates. Tests conducted at faster rates (2 cm/s and 3 cm/s) have shown the following effects:

a) For cohesive soils with cone resistance of above 1 000 kN/m^2 (10 kgf/cm^2) the effects of these rates were not significant;

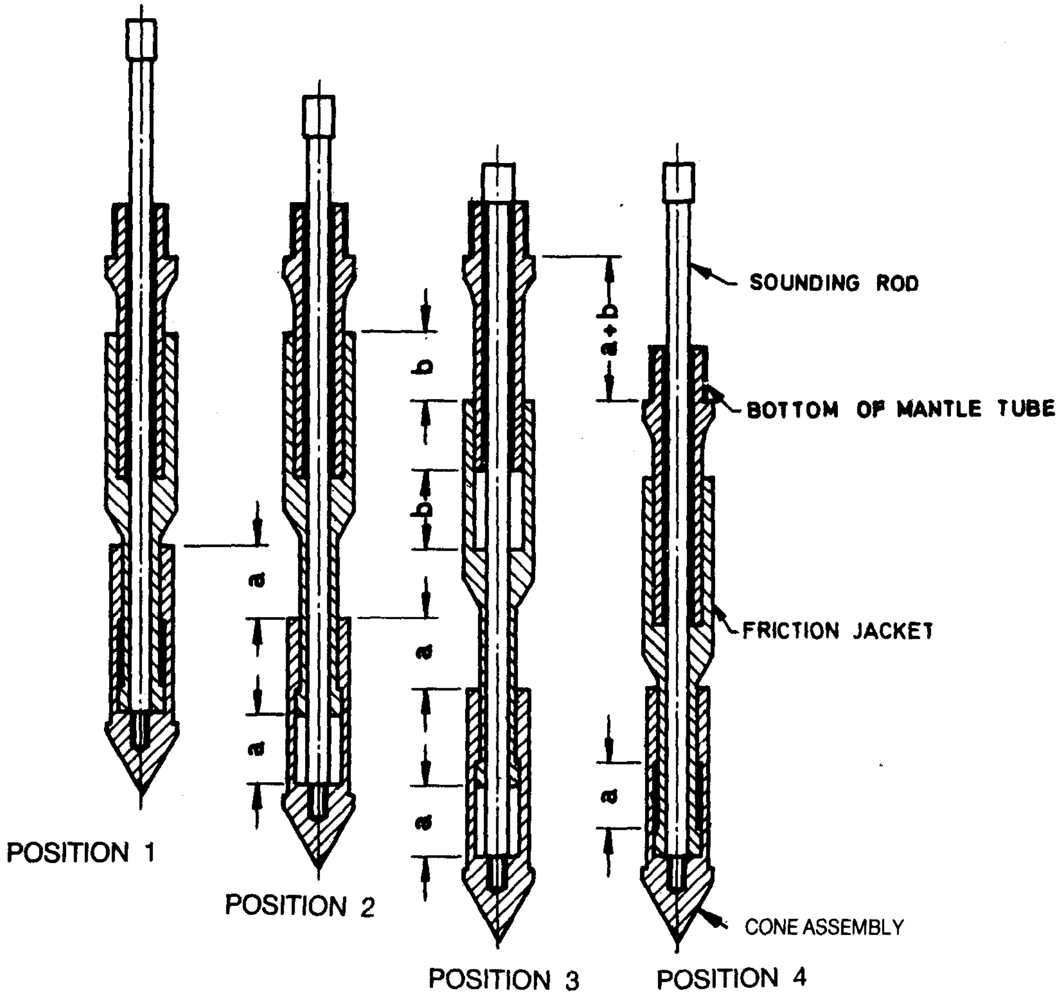
b) For cohesive soils with cone resistance of 400 kN/m^2 (4 kgf/cm^2) and lower, the values decreased appreciably with increase in the rate of penetration; and

c) For non-cohesive soils with cone resistance varying from 1 500 to 8 000 kN/m^2 (15 to 80 kgf/cm^2), the cone resistance increased by about 20 percent.

3.2.3 For finding the combined cone and friction resistance of the soil the sounding rod shall be pushed to the extent the cone has been pushed as in **3.2.2** at the rate of 1 cm/s (*see* Note under **3.2.2**) noting the mean resistance on the gauges, ignoring erratic changes. The sequence of operations is illustrated in Fig. 5.

3.3 The procedure given in **3.2.2** and **3.2.3** should be repeated after pushing the combined cone-friction jacket and mantle tube assembly to the next depth at which the cone and friction resistance values are required. Extension sounding rods and mantle tubes should be added after every one metre of pushing as the test proceeds. Alternatively, the resistances may be determined continuously, if so desired.

3.4 After reaching the deepest point of investigation the entire assembly should be extracted out of the soil by the special operations provided for in the equipment.



a = 35 mm, *Min*
b = 35 mm, *Min*

FIG. 5 FOUR POSITIONS OF THE SOUNDING APPARATUS WITH FRICTION JACKET

4. RECORDS AND CALCULATIONS

4.1 The results of the test shall be tabulated suitably. A recommended proforma for this purpose with an example is given in Appendix A. The results should also be presented graphically in two graphs, one showing the cone resistance in kN/m^2 (kgf/cm^2) with depth in metres and the other showing friction resistance in kN/m^2 (kgf/cm^2) with depth in metres together with a bore hole log.

4.2 The cone resistance shall be corrected for the dead weight of the cone and sounding rods in use. The combined cone and friction resistance shall be corrected for the dead weight of the cone, friction jacket and sounding rods. These values shall also be corrected for the ratio of ram area to the base area of the cone as illustrated in the example in Appendix A.

5. LIMITATIONS OF THE TEST

5.1 The test is unsuitable for gravelly soils and for soils with standard penetration value N (determined in accordance with IS 2131 : 1981) greater than 50. Also in dense sands the anchorage becomes too cumbersome and expensive and for such cases dynamic cone penetration tests [*see* IS 4968 (Part 1) : 1976 and IS 4968 (Part 2) : 1976] may be carried out. The test is also unsuitable for made-up or filled-up earth since erroneous values may be obtained due to the presence of loose stones, brick bats, etc. In such places either the made-up soil shall be completely removed to expose the virgin soil layer, or readings in the filled-up depth shall be ignored.

APPENDIX A

(*Clauses* 4.1 *and* 4.2)

PROFORMA FOR RECORD OF RESULTS OF STATIC CONE PENETRATION TEST

Projects:	Location of test point:
Site:	Ground elevation:
Bore hole reference:	Ground water level.

Correction:

Static cone resistance*

1) Mass of cone, m = 1·1 kg*
2) Mass of each sounding rod, m_1 = 1·5 kg*
3) Cone area at base, b = 10 cm^2
4) Plunger area (*see* Note)
5) Correction factor (to be added to the guage reading) = $(m+nm_1)$ 10 kN/m^2† $\left[\dfrac{m+nm_1}{10}\ \text{kgf/cm}^2\right]$

where

n = the number of rods in use.

NOTE — If plunger area is 20 cm^2 and base area of cone is 10 cm^2, the gauge readings should be multiplied by the ratio of the plunger area to the area of the base of the cone, that is 2.

*The figures given in the proform are by way of example only.

†1 kgf has been taken to be approximately equal to 10 Newtons. The exact value is 1 kgf = 9·806 65 N.

Depth Below Ground Level m	*Gauge Reading* kN/m² (kgf/cm²)	*Corrected Value of Cone Penetration Resistance* kN/m² (kgf/cm²)
(1)	(2)	(3)
0.20	2 150 (22.00)	2 176 (22.26)
0.40	900 (9.00)	926 (9.26)
0.60	800 (8.00)	826 (8.26)
0.80	1 000 (10.00)	1 026 (10.26)
1.00	400 (4.00)	426 (4.26)
1.20	500 (5.00)	541 (5.41)
1.40	550 (5.50)	591 (5.91)
1.60	800 (8.00)	841 (8.41)
1.80	450 (4.50)	491 (4.91)

Friction resistance measured at particular depths with the help of friction jacket attaced to the static cone*

Correction:

1) Mass of friction jacket $= mf$ kg
2) Area of surface of friction jacket, a $= \pi\, dh$ cm²
   where
   d = outer diameter of friction jacket, and
   h = length of friction jacket.
3) Cone area at base, b = 10 cm²
4) Correction factor (to be added) $= \dfrac{100\, mf}{a}$ kN/m² $\left[\dfrac{mf}{a}\text{ kfg/cm}^2\right]$
   = 1 kN/m² (0·01 kgf/cm²)

Depth Below Ground Level	*†Total Resistance,* kN/m² (kgf/cm²)	*Cone Resistance (Uncorrected)* kN/m² (kgf/cm²)	*Total Resistance Minus Cone Resistance* kN/m² (kgf/cm²)	*Frictional Resistance,* z *in* kN/m² (kgf/cm²) $\dfrac{(x-y)\,b}{a} = z$	*Corrected Frictional Resistance* kN/m² (kgf/cm²) $Z + \dfrac{100\,mf}{a}$ $\left(z + \dfrac{mf}{a}\right)$
m	x	y	$x-y$		
(1)	(2)	(3)	(4)	(5)	(6)
2·10	1 250 (13·0)	900 (9·0)	350 (4·0)	24·5 (0·28)	25·5 (0·29)
2·20	1 300 (13·5)	900 (9·0)	400 (4·5)	28·0 (0·32)	29·0 (0·33)
2·30	1 350 (14·0)	1 000 (10·0)	350 (4·0)	24·5 (0·28)	25·5 (0·29)
2·40	1 350 (14·0)	1 000 (10·0)	350 (4·0)	24·5 (0·28)	25·5 (0·29)
2·50	1 400 (14·5)	1 000 (10·5)	400 (4·0)	28·0 (0·28)	29·0 (0·29)
2·60	850 (8·5)	550 (5·5)	300 (3·0)	21·0 (0·21)	22·0 (0·22)
2·70	900 (9·0)	450 (4·5)	450 (4·5)	31·5 (0·32)	32·5 (0·33)
2·80	800 (8·0)	400 (4·0)	400 (4·0)	28·0 (0·28)	29·0 (0·29)
2·90	800 (8·0)	450 (4·5)	350 (3·5)	24·5 (0·25)	25·5 (0·26)

* The figures given in the proforma are by way of example only.

† Total resistance means resistance shown by the gauge due to penetration of cone and friction jacket.

SECTION 8

Vane Shear Test

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Indian Standard

CODE OF PRACTICE FOR *IN-SITU* VANE SHEAR TEST FOR SOILS

(First Rivision)

0. FOREWORD

0.1 The vane shear test is most appropriate for the determination of the shear strength of saturated clays, especially of the 'soft' to 'medium' consistency. The test is especially appropriate for determining the shear strength of sensitive soils which are highly susceptible to sampling disturbances.

The vane shear test consists of pushing a four-bladed vane in the soil and rotating it till a cylindrical surface in the soil fails by shear. The torque required to cause this failure is measured and this torque is converted to a unit shearing resistance of the cylindrical surface.

0.2 This standard was first published in 1967. In this revision several changes have been made taking into consideration the experience gained in conducting the test. The essential requirements of the torque applicator have been added. Maximum permissible area ratio of the vane has been related to the vane diameter. Torque applicators of two capacities have been specified; guidance has been given for the selection of the lesser capacity torque applicator in relation to the anticipated shear strength of the soil to be tested and the overall vane diameter. Opportunity has also been taken to give the requirements and examples in SI units.

0.3 In the formulation of this standard due weightage has been given to international co-ordination among the standards and practices prevailing in different countries in addition to relating it to the practices in this field in this country. This has been met by basing the standard on the following publications:

BS 1377 : 1975 Methods of test for soils for civil engineering purposes. British Standards Institution.

ASTM D 2573-67T Field vane shear test in cohesive soils. American Society for Testing and Materials.

E-20 Inplace vane shear test. Earth Manual, United States Bureau of Reclamation.

1. SCOPE

1.1 This standard covers the procedure of conducting *in-situ* vane shear test in saturated cohesive deposits for determining their inplace shearing resistance. Two methods of the test, namely, testing from bottom of a bore-hole and by direct penetration from ground surface, are covered.

2. APPARATUS

2.1 For Test from Bottom of Bore-Hole

2.1.1 *Vane* — It shall consist of four mutually perpendicular blades as illustrated in Fig. 1. The height of the vane shall be twice the overall diameter. It is recommended that the overall diameter of the vane should be 37·5, 50, 65, 75 or 100 mm. The design of the vane shall be such that it causes as little remoulding and disturbance as possible to the soil when inserted into the ground for a test. The blades shall be as thin as possible, consistent with the strength requirements. The vane should not deform under the maximum torque for which it is designed. The penetrating edge of the vane blades shall be sharpened having an included angle of 90°. The vane blades shall be welded together suitably either directly or to a central rod, the maximum diameter of which should preferably not exceed 12·5 mm. The area ratio of the vane shall be kept as low as possible and shall not exceed 18 per cent for the 37·5 mm vane and 12 per cent for the 50, 65, 75 and 100 mm diameter, vanes. The area ratio may be calculated using the following formula:

$$A_r = \frac{8t(D-d) + \pi d^2}{\pi D^2} \times 100\,(\%)$$

where

A_r = area ratio in percent,
t = thickness of vane blades in mm,
D = overall diameter of vane in mm, and
d = diameter of central vane rod including any enlargement due to welding in mm.

NOTE — The vane selected should be the largest size suitable for the general soil conditions at a site.

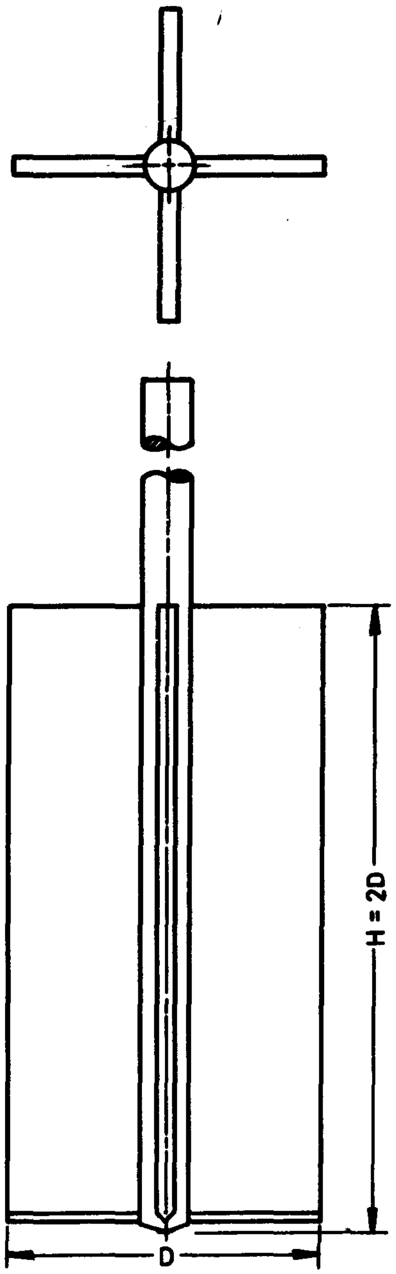

FIG. 1 GEOMETRY OF FIELD VANE

2.1.1.1 The vane rod (the rod to which the vane blades are fixed) may be enclosed in a suitably designed sleeve from just above the blades and throughout the length it penetrates the soil to exclude the soil particles and the effects of soil adhesion. This sleeve shall commence above the blades at a distance equivalent to about two diameters of the vane rod.

NOTE — The vanes shall be frequently checked for straightness.

2.1.2 *Torque Applicator* — The torque applicator shall have a clamping device to rigidly secure it to the anchor casing and shall have an attachment to securely hold the string of rods connecting the vane.

2.1.2.1 The instrument shall be capable of applying a torque to the vane through the string of rods and to measure the same. It should also have a device to read the angular rotation of the upper end of the extension rod. The torque applicator shall be provided with speed control so that the rate of rotation may be maintained at 0·1°/s. Friction exerted by the torque applicator should be of negligible magnitude and shall be checked periodically. Depending upon the estimated shear strength of the soil, Table 1 may be used as a guide for the selection of torque applicator of capacity 60 N.m (600 kgf.cm).

TABLE 1 SELECTION OF TORQUE APPLICATOR

ESTIMATED SHEAR STRENGTH kN/m² (kgf/cm²)	10 (0·1)	20 (0·2)	30 (0·3)	40 (0·4)	50 (0·5)	60 (0·6)	70 (0·7)
Vane size (dia) suitable for use with 600 kgf.cm torque applicator	All sizes	All sizes except 100 mm	All sizes except 100 mm	All sizes except 100 mm and 75 mm	All sizes except 100 mm and 75 mm	37·5 and 50 mm sizes	37·5 and 50 mm sizes

2.1.2.2 The capacity and accuracy of the instrument shall be one of the following as may be specified by the purchaser:

a) Measure torque up to 60 N.m (600 kgf.cm) to an accuracy of 1 N.m (10 kgf.cm), or

b) Measure torque up to 200 N.m (2 000 kgf.cm) to an accuracy of 2·5 N.m (25 kgf.cm).

2.1.3 *Rod System* — The string of torque rods connecting the vane to the torque applicator called the rod system may be of the quick coupling type or of the threaded type. The length of the rods shall preferably be 1 m with a few of smaller lengths. These rods shall have sufficient diameter such that their elastic limit is not exceeded when the vane is stressed to its capacity (*see* Note). The threaded rods shall be so coupled that the shoulders of the male and female ends shal meet to prevent any possibility of the coupling tightening when the torque is applied during the test. If a vane housing is used, the torque rods shall be equipped with well-lubricated bearings where they pass through the housing. These bearings shall be provided with seals to prevent soil from entering them. The torque rods shall be guided so as to prevent friction from developing between the torque rods and the walls of casing or boring.

NOTE — *If torque versus rotation curve is to be determined, it is essential that the torque rods be calibrated (prior to the use in the field). The amount of rod twist (if any) shall be* established in degree per metre per unit torque. This correction becomes progressively more important as the depth of test increases the calibration shall be made at least to the maximum depth of testing anticipated.

2.1.4 *Dummy Rod* — of dimensions equal to that of the vane rod of the vanes used.

2.1.5 *Guides for Rod* — of suitable type provided with ball bearing arrangement so as to enable the rod to rotate freely (*see* Note).

NOTE — During the test, it is essential that the rods and vane are placed centrally in the bore-hole. For this purpose guides shall be used at an interval in depth of not more than 5 m.

2.1.6 *Drilling Equipment* — The equipment used shall provide a clean hole of the required diameter for insertion of the vane to ensure that the vane test is performed on undisturbed soil.

2.1.7 *Jacking Arrangement* — for pushing the shoe and vane (where required).

NOTE — The apparatus shall be checked and calibrated as and when required.

2.2 For Tests by Direct Penetration from Ground Surface

2.2.1 *Vane* — as specified in **2.1.1**. In addition the vane shall be suitably protected by a shoe (*see* Fig. 2).

2.2.2 *Rod System* — as specified in **2.1.3** and of suitable type.

2.2.3 *Extension Pipes* — about one metre in length with coupling on the outer face to case the hole.

2.2.4 *Torque Applicator* — as specified in **2.1.2**.

NOTE — The apparatus shall be checked and calibrated as and when required.

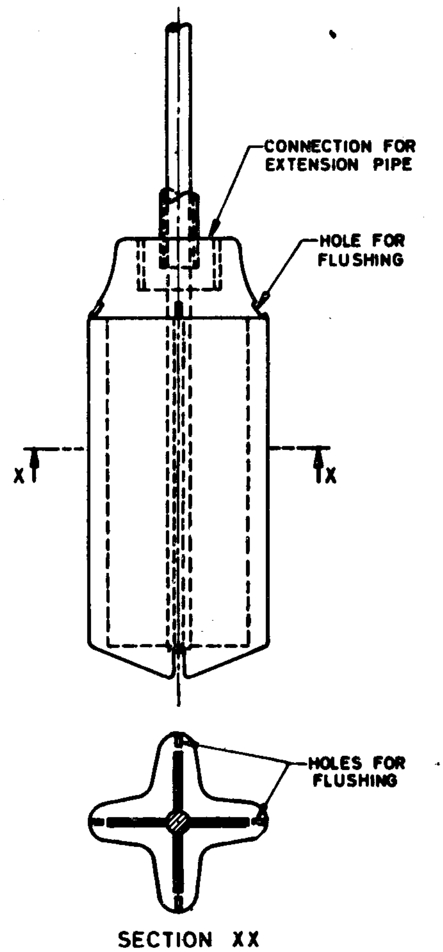

FIG. 2 TYPICAL VANE PROTECTING SHOE

3. PROCEDURE OF TESTING

3.1 Tests from Bottom of Bore-Hole

3.1.1 Sink the bore-hole up to the depth required and extend the casing up to the full depth. If the casing is loose, secure it so that it does not move during the tests. Fix the torque applicator anchor plate to the casing.

3.1.2 Connect the vane of suitable size (*see* **2.1.1**, Note) to the rods and lower it to the bottom of the bore-hole, putting guides at suitable intervals but not more than about 5 m as the rods are extended. Push the vane with a moderate steady force up to a depth of five times the diameter of the bore-hole below the bottom of the bore-hole or shoe. Take precautions to make sure that no torque is applied to the torque rods during the thrust. No hammering shall be permitted. Fix the torque applicator with frame to the anchor plate and connect the rods to it. Tighten the torque applicator to the frame properly. A diagrammatic arrangement for vane test from bottom of bore-hole is shown in Fig. 3.

3.1.3 Allow a minimum period of five minutes after insertion of the vane. Turn the gear handle so that the vane is rotated at the rate of 0·1°/s. Note the maximum torque reading attained. If necessary, note the torque indicator dial gauge readings at half-minute intervals and continue rotating the vane until the reading drops appreciably from the maximum.

3.1.4 Just after the determination of the maximum torque, rotate the vane rapidly through a minimum of ten revolutions. The remoulded strength should then be determined (*see* **3.1.3**) within one minute after completion of the revolutions.

3.1.5 Remove the vane testing assembly, continue boring, and collect soil sample from the level of the vane testing for laboratory analysis to

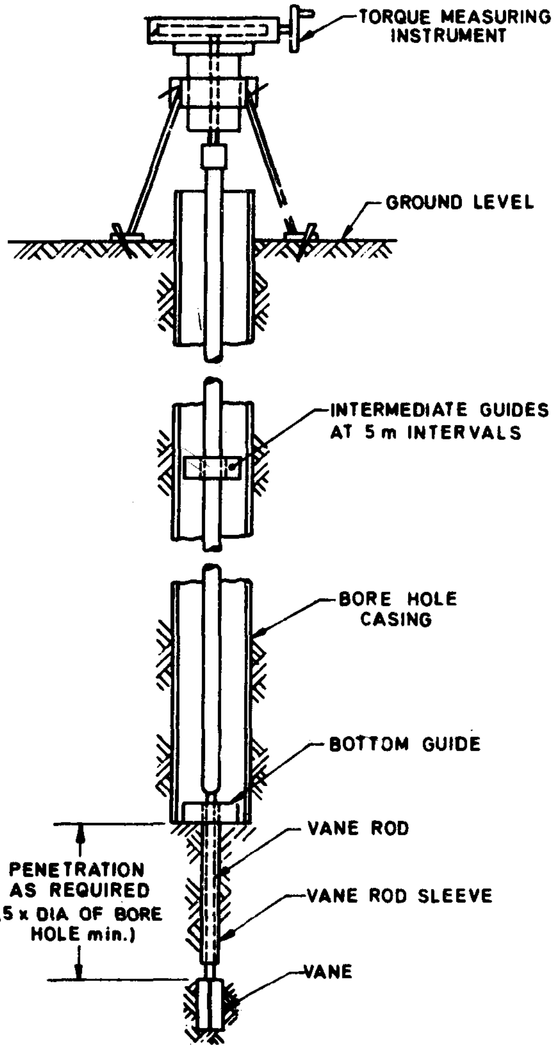

FIG. 3 DIAGRAMMATIC VANE TEST ARRANGEMENT (FOR TEST FROM BOTTOM OF BORE-HOLE)

ascertain whether the deposit will behave as a purely cohesive soil.

3.1.6 In case where a sleeve is not provided for the vane rod and the soil is in contact with the rod, determine the friction between the soil and the vane rod by conducting tests at appropriate depths using the dummy rod corresponding to that of the vane used in the test. The test should be conducted as with the vane except that the vane is replaced by dummy rod. The test should be conducted in an adjacent bore-hole at the same depth at which the vane tests were conducted. The dummy rod should be pushed into the ground to the same distance as the vane rod at that depth.

3.2 Test by Direct Penetration from Ground Surface

3.2.1 Lock the vane in-place inside the protecting shoe and jack or drive it to the required depth. Care shall be taken to see that the rods remain tight while the vane is lowered. Place guides about every 3 m to centralize and reduce friction between the rods and extension pipes.

3.2.2 When the vane and protecting shoe have penetrated to the required depth, push the vane steadily, without twisting, a distance of 5 times the diameter of the shoe, into the undisturbed soil below the protecting shoe. Rotate the vane till the soil fails as in **3.1**. A diagrammatic vane test arrangement for testing by direct penetration from ground surface is given in Fig. 4.

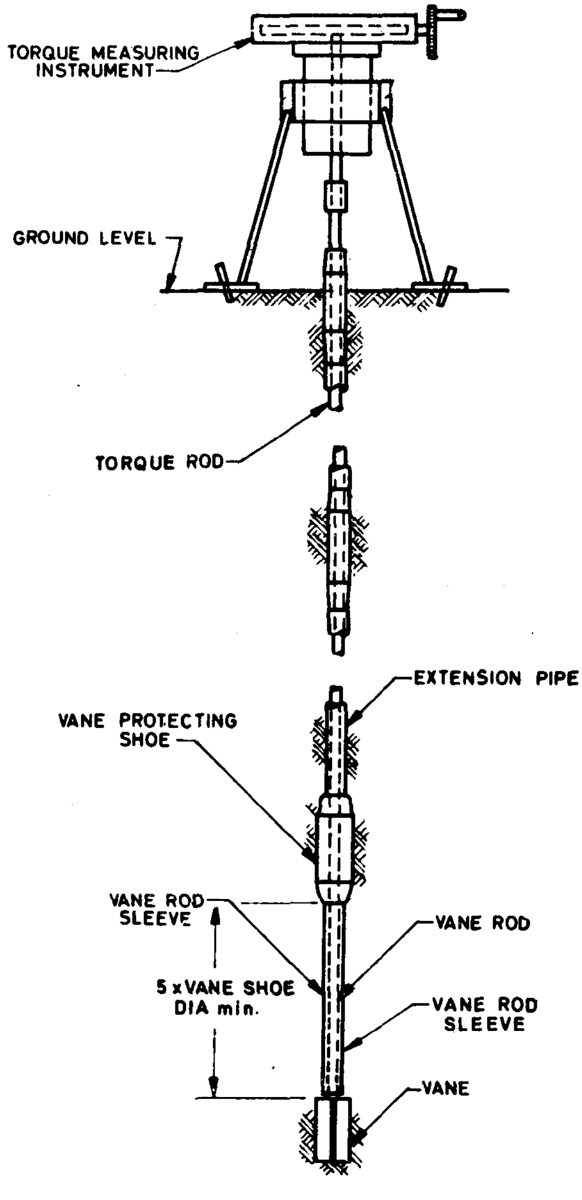

FIG. 4 DIAGRAMMATIC VANE TEST ARRANGEMENT (TEST BY DIRECT PENETRATION FROM GROUND SURFACE)

3.2.3 Remove the torque measuring instrument and pull back the vane fully into its protecting shoe before advancing for another test or before being removed from the ground taking precautions that the vane is not damaged by the shoe.

3.2.4 In the case where soil is in contact with the torque rods, determine the friction between the soil and the rod by means of torque tests conducted on similar rods at similar depths with no vane attached. Conduct the rod friction test at least once on each site; this shall consist of a series of torque tests at varying depths. A dummy should be used instead of the vane if the vane rod is not provided with a sleeve.

4. RECORDS

4.1 Records of vane shear test shall be maintained in a suitable form including details given in Appendix A, which gives a recommended proforma for the record of results.

5. COMPUTATIONS

5.1 For vane testing instruments that do not read the torque directly a calibration curve to convert the readings to newton metre (kilogram force centimetre) of torque shall be provided. These calibration curves shall be checked periodically.

5.2 For a rectangular vane, calculate the shear strength of the soil using the following formula (*see* Note 1):

$$S = \frac{M}{\pi\left(\frac{D^2H}{2} + \frac{D^3}{6}\right)} \times 10^6 \quad \left[S = \frac{M}{\pi\left(\frac{D^2H}{2} + \frac{D^3}{6}\right)} \times 10^3 \right]$$

where

S = shear strength in kN/m² (kgf.cm²);

M = torque, to shear the soil in N.m (kgf.cm) (corrected for vane rod and torque rod resistance, if any);

D = overall diameter of vane in mm (*see* Note 2); and

H = height of vane in mm (*see* Note 2).

NOTE 1 — This formula is based on the assumptions that (a) Shearing strength in the horizontal and vertical directions are the same. (b) At the peak value, shear strength is equally mobilized at the end surface as well as at the centre and that both the top and the bottom ends of the vane take part in shearing the soil. (c) It is assumed that the shear surface is cylindrical and has a diameter equal to the diameter of the vane.

NOTE 2 — It is important that the dimensions of the vane are checked periodically to ensure that the vane is not distorted or worn. Actual values should be used in the calculation.

5.2.1 If $H = 2\,D$, then the formula given in **5.2** reduces to

$$S = \frac{3M}{11\,D^3} \times 10^6 \text{ (S in kN/m}^2\text{)}$$

$$S = \frac{3M}{11\,D^3} \times 10^3 \text{ (S in kgf/m}^2\text{)}$$ (*see* Notes 1 and 2 of **5.2**)

APPENDIX A

(*Clause* 4.1)

PROFORMA FOR FIELD VANE SHEAR TEST

GENERAL

Project:
Bore-hole No.:
(if any)
Date of test:

Drilling or testing foreman:
Supervising engineer:

DETAILS OF BORING (IF ANY)

Location:
Reference elevation:
or
Ground elevation:
Method of making the hole:
Cased/uncased:
Level of water in the bore-hole/ level of ground water at the *time* of test:
Notes on driving resistance:

Log of soil conditions:

DETAILS OF VANE TEST

Test from bottom of bore-hole:

Test by direct penetration from ground surface:

Vane test apparatus No.:
Vane Size:
Diameter of dummy rod (if used):
Conversion factor for torque measuring equipment:

Vane constant:

Depth

	D	U	R	D	U	R	D	U	R	D	U	R	D	U	R	D	U	R	D	U	R
Depth of vane tip below bottom of bore hole or vane shoe																					
Time to failure																					
Maximum reading on torque measuring equipment																					
Maximum torque																					
Number of revolutions for remoulding																					
Shear strength of undisturbed soil, kN/m^2 (kgf/cm^2)																					
Shear strength of remoulded soil, kN/m^2 (kgf/cm^2)																					
Sensitivity																					

D = test with dummy if used or any other test for the determination of friction of vane rod and/or torque rods.
U = test with vane in undisturbed soil.
R = test with vane in remoulded soil.
Record of deviation from standard procedure, if any, with reasons.

Time — Rotation Readings

Time in Minutes	Depth																				
	D	U	R	D	U	R	D	U	R	D	U	R	D	U	R	D	U	R	D	U	R
0																					
$\frac{1}{2}$																					
$1\frac{1}{2}$																					
2																					
$2\frac{1}{2}$																					
3																					
$3\frac{1}{2}$																					
4																					
$4\frac{1}{2}$																					
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7																					
$7\frac{1}{2}$																					
8																					
$8\frac{1}{2}$																					
9																					
$9\frac{1}{2}$																					
10																					

D = test with dummy if used or any other test for the determination of friction of vane rod and/or torque rods.

U = test with vane in undisturbed soil.

R = test with vane in remoulded soil.

Record of deviations from standard procedure, if any, with reasons.

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SECTION 9

Direct Shear Test

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Indian Standard

CODE OF PRACTICE FOR SITE INVESTIGATIONS FOR FOUNDATION IN GRAVEL-BOULDER DEPOSIT

0. FOREWORD

0.1 The advent of the large river valley projects in India necessitated the erection of heavy industrial complexes in river or old river courses. It will, therefore, be necessary to determine foundation conditions in respect of the following before a design can be finalized:

a) Sequence and extent of overburden soil undelain by boulder-gravel soil deposit, to be affected by the proposed work;
b) Nature of matrix of boulders;
c) The amount and state of packing of the boulders, their nature (rounded or otherwise) and the size of the boulders present;
d) Whether the boulder-gravel is laying in the matrix of material or otherwise;
e) Nature of each stratum if there is any change, ground water table and its possible effects on foundation material; and
f) General information on geology and surface drainage, etc.

This standard has, therefore, been formulated to cover these aspects, as methods adopted for soils (*see* IS 1892 : 1979) will not be applicable in such cases.

0.2 In the formulation of this standard, considerable assistance has been given by Central Building Research Institute, Roorkee.

1. SCOPE

1.1 This code deals with the subsurface investigation in relation to design of foundations for single and multistoreyed buildings, overhead water tanks, piers and abutments of bridges in boulder-gravel deposits.

2. TERMINOLOGY

2.0 For the purpose of this standard, the following definitions in addition to those given in IS 2809 : 1972 shall apply.

2.1 Boulder-Boulder Test (BBT) — A block cut out of the boulder soil is sheared under a normal load.

2.2 Concrete-Boulder Test (CBT) — A cast *in-situ* concrete block on boulder soil and pushing the block laterally under a normal load.

2.3 Depth on Foundation — The minimum vertical distance between the soil surface and base on the foundation.

2.4 Dynamic Cone Penetration Test (DCPT) — A subsurface sounding used to ascertain the soil strata.

2.5 Residual Shear Strength — Minimum shear strength exhibited by the soil.

3. SYMBOLS

3.1 For the purpose of this standard and unless defined in the text, the following letter symbols shall have the meaning indicated against each :

B_f = width of the strip foundation, side of square foundation, diameter of circular foundation expressed in metres,

q_a = allowable soil pressure corresponding to 12 mm or 25 mm settlement in tonnes/m^2,

τ_o = residual soil strength in t/m^2,

S_a = allowable settlement for a structure in cm,

N'' = cummulative number of blows corresponding to a depth D_c in m,

D_c = depth of penetration in cm,

B_c = diameter of cone in cm, and

γ_c = natural unit weight of the soils in t/m^2.

4. GENERAL

4.1 The type of material found in such locations as river beds or old river courses falls neither in the class of soils nor rocks but in the form of gravel-boulders.

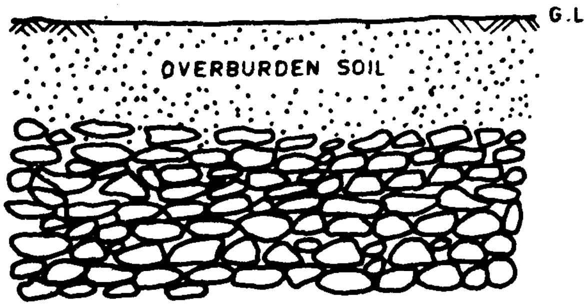
The average size of the boulder is larger than 300 mm and it is generally mixed with fine (4·75 – 20 mm) to coarse (20 – 80 mm) gravels. Soils with a large quantity of gravel-boulders deposit pose several problems in investigation. The presence of large sized particles precludes the sampling by the usual methods of soil sampling in the true sense. Tests on disturbed samples are likely to yield unreliable results, as the natural arrangements of the grains and matrix material are never achieved by recompaction. The best results are obtained by properly chosen field tests.

4.2 Nature of the Deposit — The deposits commonly called boulder deposits may be of fluvial or glacial origin. Generally, there exists some filler material which may be sand, silt or clay mixed with fine to coarse gravels.

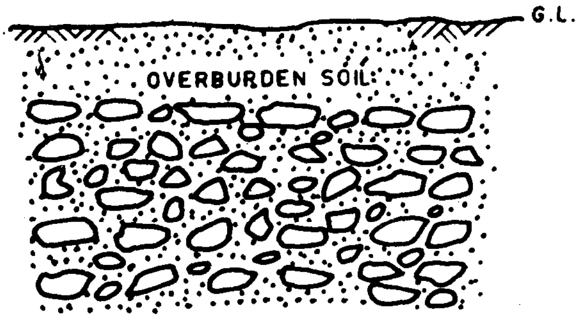
4.3 Behaviour Under Load — The performance of such deposits under load, a matter of intelligent guess, is generally made on conservative side leading to high cost of foundations. The behaviour of boulder deposits under high loads also depends upon the size and quantity of gravel-boulder and also the nature and amount of the filler material. If the filler material exists only in the interstices of the boulder (*see* Fig. 1A), the behaviour depends upon the state of packing of the boulders, nature (rounded or otherwise) and the size of the boulder; on the other hand if the boulder exists in the matrix of the filler material (*see* Fig. 1B), the behaviour will be governed by the size, quantity and distribution of the boulder in the filler. When the filler material is absent, the load carrying capacity is high and the compressibility is low. When there exists a filler material, there is an initial compression stage followed by low compression stage when the load carrying capacity is high. If the gravel lies in the matrix of the filler material, the behaviour is governed by the nature of the filler material and it is likely ro reduce the compressibility. The boulder soil unlike ordinary soil shows certain peculiar characteristics when the boulder proportion is large (> 30 percent); the deposit shows an initial rapid compression followed by a stage where the compression decreases considerably as the boulders take over the load carrying function (*see* Fig. 2). In such cases, it is of advantage to have the allowable load well in excess of the load at which initial compression occurs, thereby reducing deformations at design loads.

However, in the other situations when the boulder-gravel quantity is small (< 30 percent), normal methods of interpretation (IS 1888 : 1982) will be used (*see* Fig. 3, curves A, E and G).

4.4 Influence of Filler Material — If the cobbles/boulders exist in the matrix of the filler material, properties of the matrix govern the overall behaviour, though the presence of gravel/boulder shall reduce the compressibility of

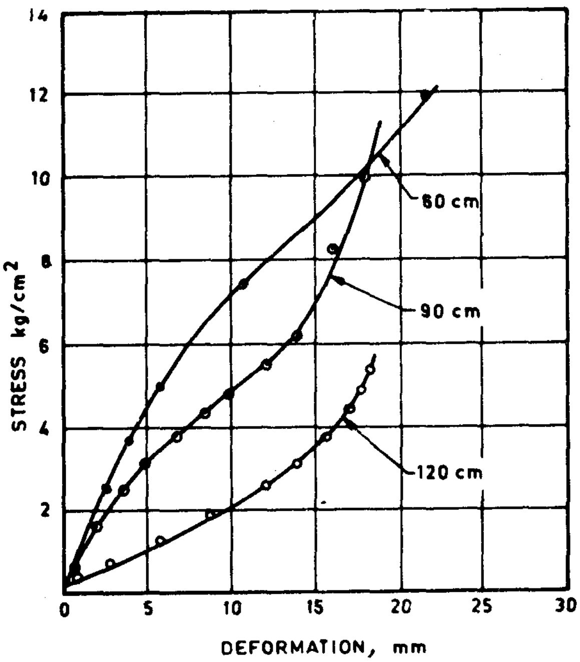


1A Filler Material Existing in the Interstices of Boulder/Gravel



1B Boulder Gravel Exising in the Matrix of the Interstitial Material

FIG. 1 GRAVEL-BOULDER SOIL STRATUM



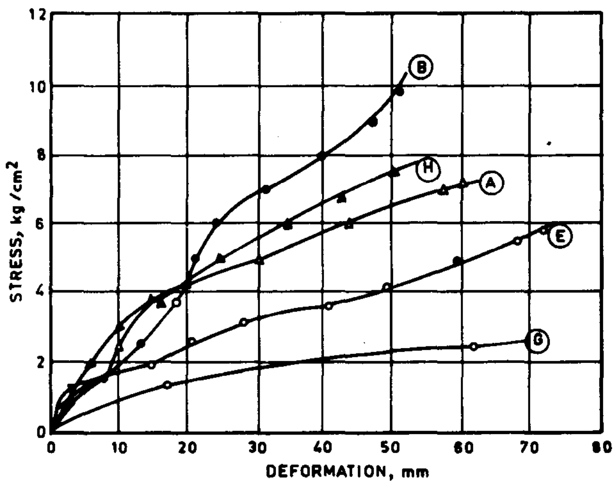
Natural density = 2.21 gm/cc

Grain ≥ 5 cm = 57%

Max grain size = 350 mm

Grain > 12 mm = 32%

FIG. 2 LOAD SETTLEMENT CURVES MATERIAL LIES IN THE MATRIX OF BOULDERS the matrix material to some extent (*see* Fig. 3 and 4). However, if this material exists only in the interstices of the boulders, the boulders being in contact (*see* Fig. 1A), the behaviour is essentially governed by the boulder.



TEST No.	INTERSTITIAL MATERIAL < 0.25mm²	REMARKS
B	27	All footing 60cm
H	52	Square
A	68	
E	27	Reason not
G	100	known

FIG. 3 LOAD SETTLEMENT CURVES (BOULDER LIES IN THE MATRIX OF MATERIAL)

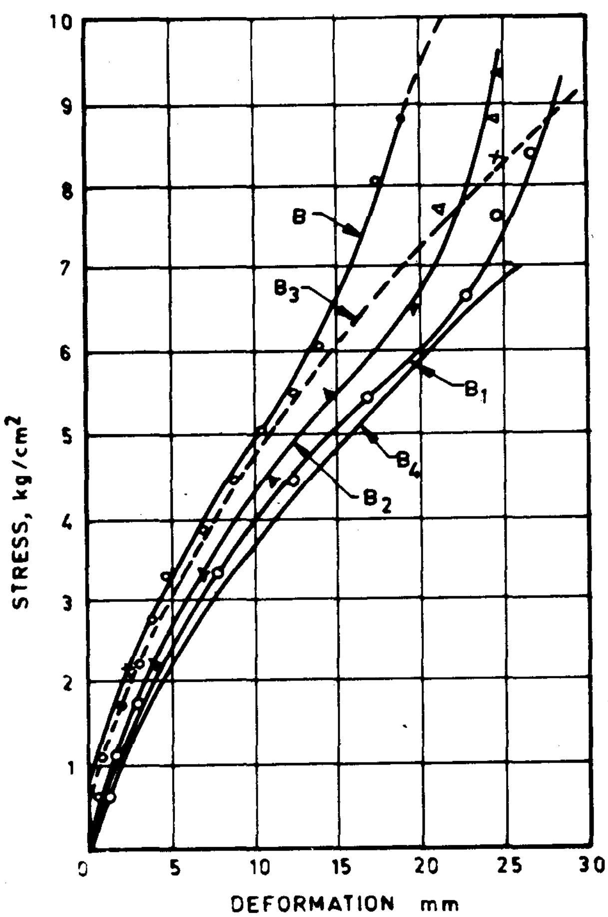
5. METHODS OF INVESTIGATIONS

5.1 Field Test — The method of exploring by open trial pits consists in excavating trial pits at the site and thereby exposing the sub-soil strata thoroughly, enabling identification and classification of the soil. The pit size shall be sufficiently large (5 × 5 m) but not less than 2 × 2 m. The depth of excavation is 4 to 6 m. The sides of the pit shall be sloped or the shored excavation shall be employed to check against any possible slides in case of deep excavation when the depth of the boulder stratum is large (10 to 15 m), with large size of the pit (15 × 15 m). Large pits are necessary when heavy structures are founded on bouldery soils.

5.2 Soil Classification and Sampling — Due to the native nature of the material, the undisturbed sampling is not possible for identification and testing purposes. During the course of excavation of the technical pit, disturbed samples shall be collected at about 1 m depth or change of strata for identification and grain size analysis.

NOTE — These samples shall also be used to determine the moisture content of the material and the overall proportion of material in the boulder soil strata.

5.3 *In-Situ* Density — The measurement of natural density shall be made by water replacement method/sand replacement method. For this purpose, a pit of known dimensions is excavated (1 m × 1 m × 50 cm).



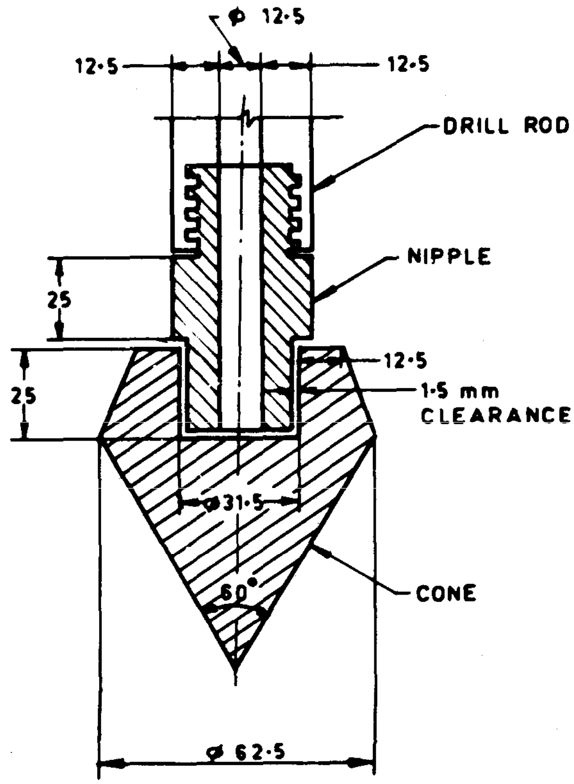
TEST No.	INTERSTITIAL MATERIAL < 5 cm%	REMARKS
B	47	All footings 90 cm²
B_1	40	
B_2	60	
B_3	57	
B_4	58	

FIG. 4 INFLUENCE OF INTERSTITIAL MATERIAL ON THE LOAD CARRYING CAPACITY OF BOULDER SOILS

The entire excavated soil is carefully collected weighed with the help of a spring balance (100 kg capacity). The pit is then be covered with a polyethylene sheet large enough to touch the sides and the base of the pit. The pit is filled with known volume of water up to the surface of the pit [IS 2720 (Part 33) : 1971]. The density is calculated from the known mass and volume of the soil. Sometimes dry clean sand may also be well adopted to measure the volume of the excavated soil. The collected soil may also be used for grain size analysis and soil classification.

5.4 Dynamic Cone Penetration Test (DCPT) — This test [IS 4968 (Part 1) : 1976] shall be performed, if the aggregate size is not larger than 100 to 120 mm. The test set-up is similar to that in the standard penetration test, the standard SPT Sampler being replaced by a push fit cast iron cone with 60° apex angle and base diameter 62.5 mm. Use of cone of this size ensure sufficient clearance (11.36 mm) with the standard *A* drill rod and helps in reducing friction on the rod (*see* Fig. 5). The cone is driven from the level of the footing with a 65 kg hammer dropping from a height of 750 mm The number of blows for every 15 cm shall be counted and a plot between cumulative number of blows N″ and D/B_c is prepared. A typical plot is shown in Fig. 6.

NOTE — In order to avoid damage to the equipment, driving may be stopped when the number of blows for the last 30 cm is 100. Assembly of the equipment shall be in accordance with Fig. 2 of IS 4968 (Part 2) : 1976.



All dimensions in millimetres.

FIG. 5 DIAGRAMMATIC SKETCH OF 62·5 mm DIA CONE

5.5 Load Test on Cast *in-situ* Footing — Considering the large size of aggregates involved, large sized footing need to be used in the test. Steel plates are not of much use because of seating difficulties. Best results are obtained with cast *in-situ* concrete blocks or with precast blocks, set with fresh mortar, so that there is perfect bond between the soil and the block.

The size of the footing shall be such that it will span over several boulders so as to mobilize group action under load. To satisfy this criterion, the minimum size of the footing shall not be less than 10 times the average grain size to a minimum of 150 cm.

NOTE — The maximum limit may be increased depending upon feasibility.

The footing shall be loaded using dead load Kentledges platform through a hydraulic jack (100 tonnes capacity). The deformation shall be measured with 4 dial gauges (0.001 mm) in accordance with IS 1888 : 1982. For each increment of load, the deformation shall be noted after intervals of 1, 4, 10, 20, 40 and 60 min and thereafter at hourly intervals. Each load increment shall be kept for not less than 1h or up to a time when the rate of deformation gets reduced appreciably (to a value of 0·02 mm/min). The deformation shall be taken as the average of 4 dial gauges readings. The next increment of load shall then be applied and the observation repeated. The test should be continued till a deformation of 50 mm is reached or till failure occurs, whichever is earlier. A minimum of eight load increments shall be applied to obtain a well defined load settlement curve. Typical graphs are shown in Fig. 2. A typical set is shown in Fig. 7.

If needed, rebound observation may be taken while releasing the load in a similar manner.

5.6 *In-situ* Shear Test — The test best suited for such type of deposit to determine strength parameter is *in-situ* shear test. It consists in shearing a block of sample under a given normal load. The *in-situ* test is a simple form of laboratory shear box test. The interpretation of test results causes difficulties, as identical samples are not likely to be available for many tests under different loads. Two types of tests have been found suitable for such deposits.

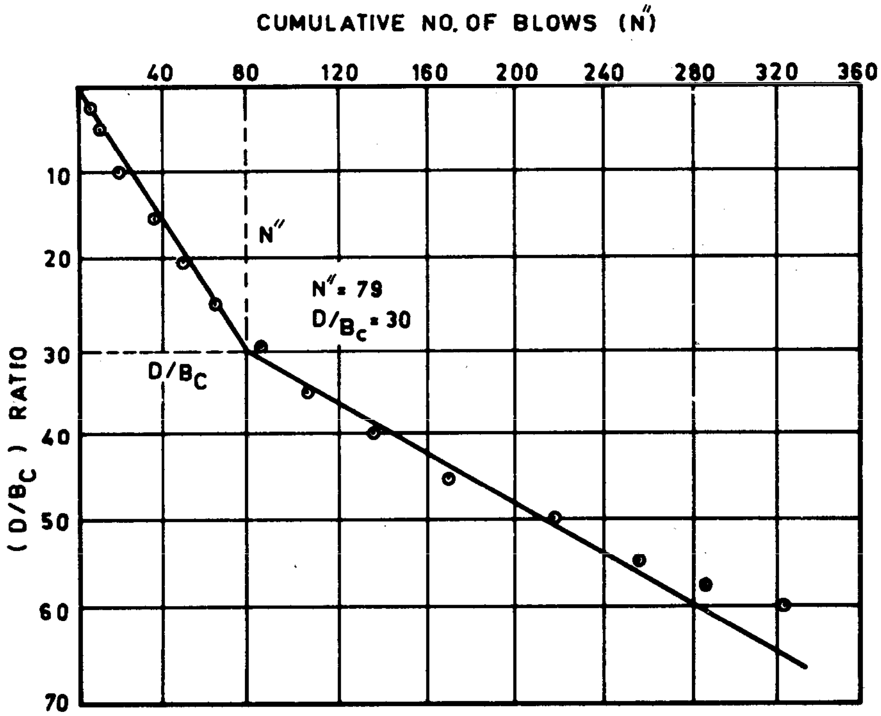


FIG. 6 TYPICAL PLOT OF DEPTH/WIDTH (D/B_c) RATIO AND CUMULATIVE NO OF BLOWS (N″)

5.7 Boulder-Boulder Test (BBT)* — It consists in cutting a block of boulder soil and slowly confining it into a rigid well-designed steel former [IS 7746 : 1975 and IS 2720 (Part 39/Sec 1) : 1977]. This soil block is sheared under a normal load. Then the shear (kg/cm^2) is plotted against displacement (mm).

5.8 Concrete-Boulder Test (CBT) * — This test, similar to that recommended in IS 7746 : 1975 consists in casting a reinforced block and pushing it laterally under a given normal load.

For a shear stress, the corresponding displacement is noted and the shear stress (kgf/cm^2) — displacement (mm) curve is plotted. This test has the advantage that it eliminates the need for a steel former and an undisturbed soil block. The concrete blocks used for load test, can also be used for this purpose. Precast concrete blocks may also be used after they are seated with mortar on the foundation bed.

The residual shear stress (τ_0) values obtained from both types of tests (BBT and CBT) are the same. The test set-up for both types of tests and sample results are shown in Fig. 8, 9 and 10.

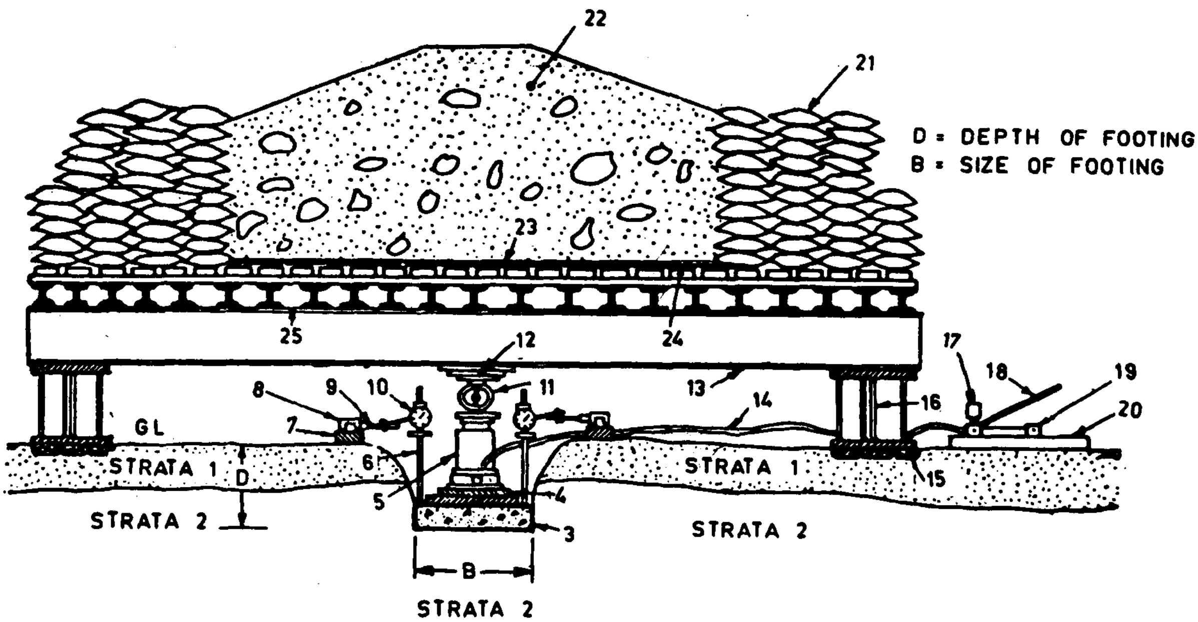
5.9 Sample Size — In the case of Boulder-Boulder Test (BBT), the minimum size of the sample shall not be less than 10 times the average size of boulders or 120 × 120 cm, whichever is less. The height of the sample shall however, be kept at 30 to 45 cm, depending upon the average aggregate size. The minimum size of the concrete block in CBT shall also be 120 × 120 cm in plan and the thickness may be kept as 30 cm.

NOTE — The size may be increased, depending upon feasibility IS 2720 (Part 39/Sec 2) : 1979

5.10 Application of Normal Load — The sample in BBT or the concrete block in CBT shall be sujected to a normal load equal to the overburden pressure or the anticipated normal load. The load on the sample may be placed in the form of 20 kg concrete cubes. Alternatively, sand bags of known weights may also be used. However, the intensity of the normal load shall not exceed two times the shear strength of the soil, to avoid a bearing capacity type of failure.

NOTE — For the sample likely to be sheared under high normal loads for the angle of shearing resistance, if considered, the procedure outlined in IS 7746 : 1975 and IS 2720 (Part 39/Sec 2)-1979 may be followed.

* The minimum number of tests depends on the area over which the structure has to come and uniformity of strata. However, in the normal case a minimum of two tests are required.



1.	Overburden soil	14.	Flexible pipe
2.	Boulder deposit	15.	M.S. channel/wooden sleepers
3.	Cast *in-situ* concrete footing	16.	Column
4.	M.S. plates	17.	Pressure indicator
5.	Hydraulic jack	18.	Jack handle
6.	Dial guage stand	19.	Pump for the jack
7.	Datum bar	20.	M.S. plates/wooden sleepers
8.	Magnetic base	21.	Loading bags
9.	Dial gauge rod	22.	Loading material (excavated soil)
10.	Dial gauge	23.	M.S. sheets
11.	Proving ring	24.	Sleepers
12.	Spacer plates	25.	Girder
13.	Main girder		

FIG. 7 *In-situ* FULL SIZE FOOTING TEST

6. ALLOWABLE SOIL PRESSURE

6.1 Load Test — The allowable soil pressure from one or two load tests shall be obtained as outlined in **4.3** and **5.5**. These load tests shall be used to confirm results from other large number of less expensive tests such as dynamic cone penetration tests (DCPT) and *in-situ* shear tests (BBT or CBT), particularly when the site under investigation is large.

6.2 Dynamic Cone Penetration Test — The results of dynamic cone penetration test [IS 4968 (Part 1) : 1976] shall be used to compute allowable pressure (q_a) from the following equation when the maximum grain size is not exceeding 100 mm:

$$q_a = \frac{1}{2{\cdot}54}\left[\frac{N''S_a}{D\,B_f}\right] \quad \ldots\ldots\ldots (1)$$

In Equation (1), the value of allowable deformation S_a (cm) and the footing width B_f (m) shall be assumed in accordance with the type of structure to be constructed. The values of N_c and D shall be taken from Fig. 6.

6.3 *In-situ* Shear Tests — The allowable pressure (q_a) shall be computed from the residual shear stress values (ζ) obtained from *in-situ* shear tests (BBT or CBT) using the following equation:

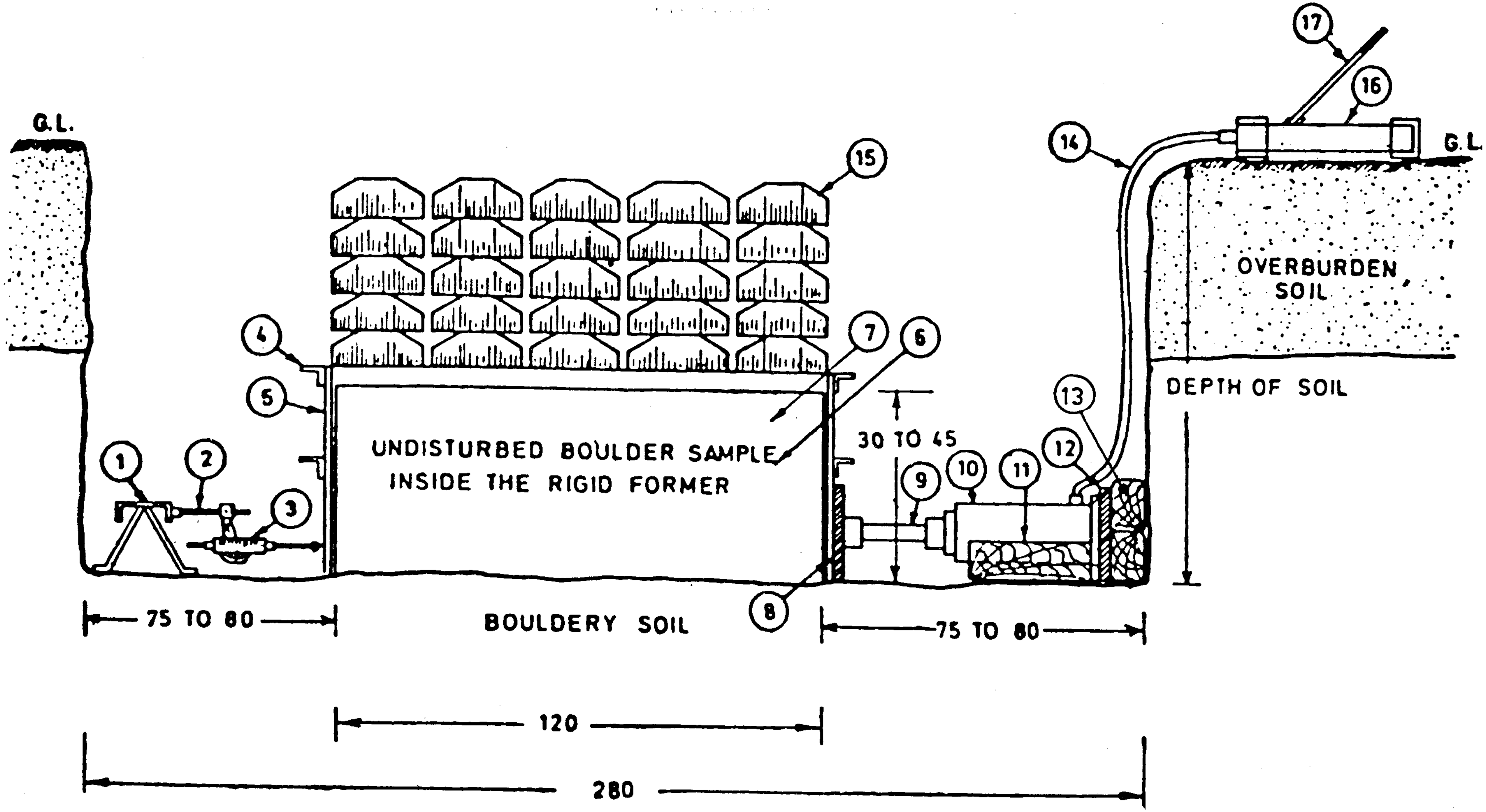
$$q_a = (X)\,\tau_o\left[\frac{B + 0{\cdot}3}{B}\right]^2 \gamma \quad \ldots\ldots (2)$$

where

X = constant, 6·25 for a deformation of 12 mm and 8 for a deformation of 25 mm. The value of the width of the footing (B) shall be assumed depending upon the situation and type of structure.

7. LABORATORY TEST

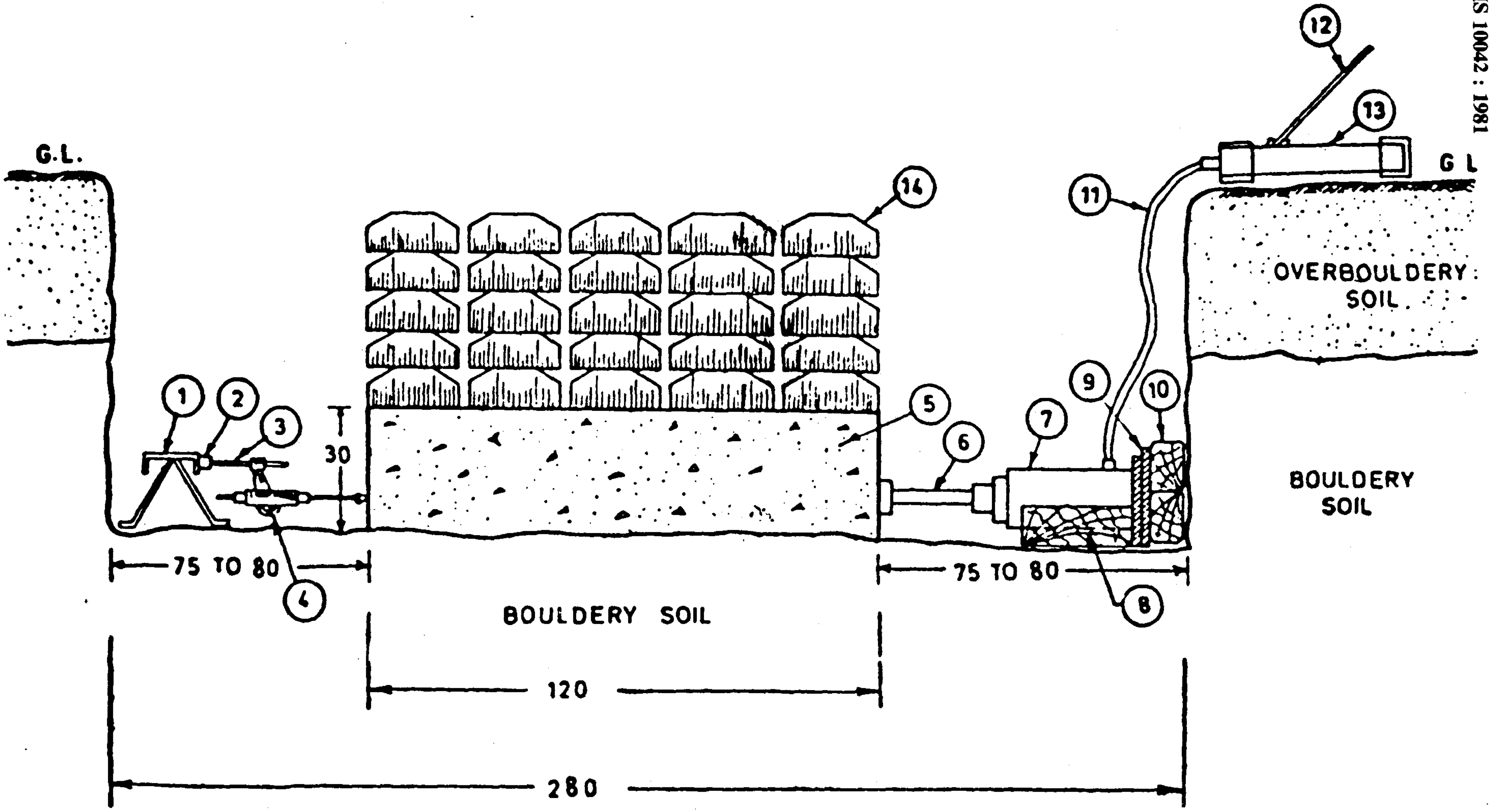
7.1 Soil Sampling — Owing to the nature of the



1. Datum bar
2. Magnetic holder
3. Dial guage
4. Angle iron stiffeners
5. Steel shear box
6. Boulder gravel sample
7. Filler material
8. Steel plates
9. 20 T capacity proving ring
10. Hydraulic jack 20 T capacity
11. Wooden sleeper support for jack
12. Steel plates
13. Wooden sleepers
14. Flexible pipe
15. Steel weights/concrete blocks/sand bags
16. Pump for the jack
17. Handle for the jack

All dimensions in centimetres

FIG. 8 TEST SET-UP SHOWING DETAILS OF BOULDER-BOULDER TEST



1. Datum bar
2. Magnetic base
3. Stand for dial gauge
4. Dial guage
5. Concrete sample
6. Proving ring
7. Hydraulic jack 20 T capacity
8. Wooden sleeper support for jack
9. Steel plates
10. Wood sleeper
11. Flexible pipe
12. Handle for jack
13. Pump for jack
14. Steel weights/concrete blocks/sand bags

All dimensions in centimetres.

FIG. 9 TEST SET-UP SHOWING DETAILS OF CONCRETE- BOULDER TEST

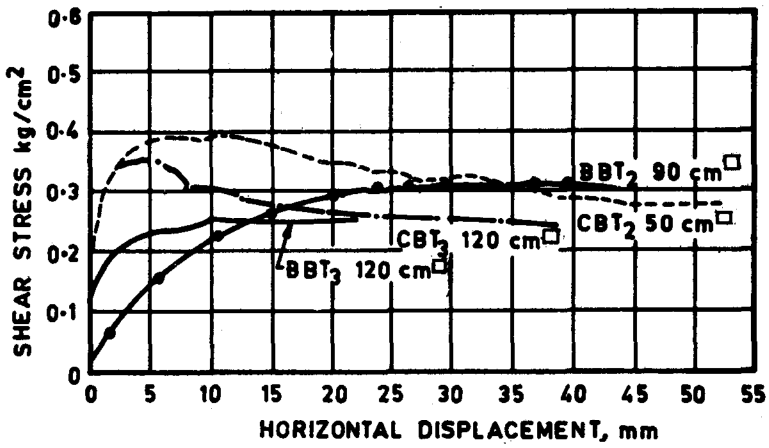


FIG. 10 COMPARISION OF BBT AND CBT SHEAR STRESS

bouldery soil, undisturbed sampling is not possible and hence only disturbed samples form the technical pit shall be collected from different elevations. The quantity of samples from each depth shall be large enough (1 000 kg) to reflect the actual soil proportions. The material above particle size 80 mm shall be separated and its grading established in the field itself. However, the material below particle size 80 mm shall be collected in suitable containers (gunny bags) and properly labelled immediately, as given in Appendix E of IS 1892 : 1979.

7.2 Grain Size Analysis — The only laboratory test conducted on disturbed samples of bouldery soils is the grain size distribution using appropriate IS sieves to separate boulder/cobbles/gravel (> 4·75 mm) and matrix (< 4·75 mm).

7.2.1 From the known quantity of the samples collected, and the grain size analysis, the overall proportion of the boulder/gravel shall be collected. This shall be used in deciding if the boulder/gravel lies in the matrix of material or vice versa.

Indian Standard

SPECIFICATION FOR SHEAR BOX (LARGE) FOR TESTING OF SOILS

0. FOREWORD

0.1 The Indian Standards Institution has already published a series of standards on methods of testing soils. It has been recognized that reliable and intercomparable test results can be obtained only with standard testing equipment capable of giving the desired level of accuracy. Series of Indian Standards covering the specifications of equipments used for testing soils are therefore being formulated to encourage their development and manufacture in the country.

0.2 The equipment covered in this standard is used as a part of the assembly for the equipment used for the laboratory determination of shear strength of the soil [*see* IS 2720 (Part 39/Sec 1) : 1977].

1. SCOPE

1.1 The equipment covered in this standard is used as a part of the assembly for the equipments used for laboratory determination of direct shear strength of the soil material with particle size up to 25 mm, that is, soils containing *moorums*, sands, gravels and other aggregates.

2. GENERAL REQUIREMENTS

2.1 The shear box shall consist of the following (*see* Fig. 1) :

a) Upper and lower parts of the shear box coupled together with two pins,
b) Grid plates — 2 pairs,
c) Spacer plates,
d) Base plate,
e) Loading pad, and
f) Water jacket.

3. MATERIALS

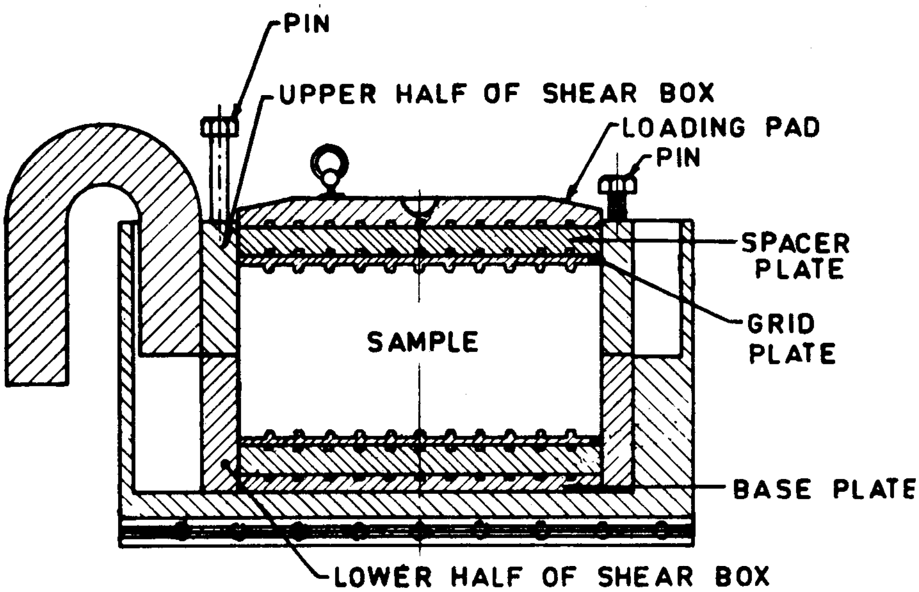
3.1 The material used for the construction of the different component of shear box shall be as given in Table 1.

4. SHAPE AND DIMENSIONS

4.1 The shape and dimensions of the various components of the shear box shall be as given in Fig. 2 to 7. The tolerance to the dimensions shall be as given in IS 2102 (Part 1) : 1980 and shall be of medium class.

5. MARKING

5.1 The following information shall be clearly and

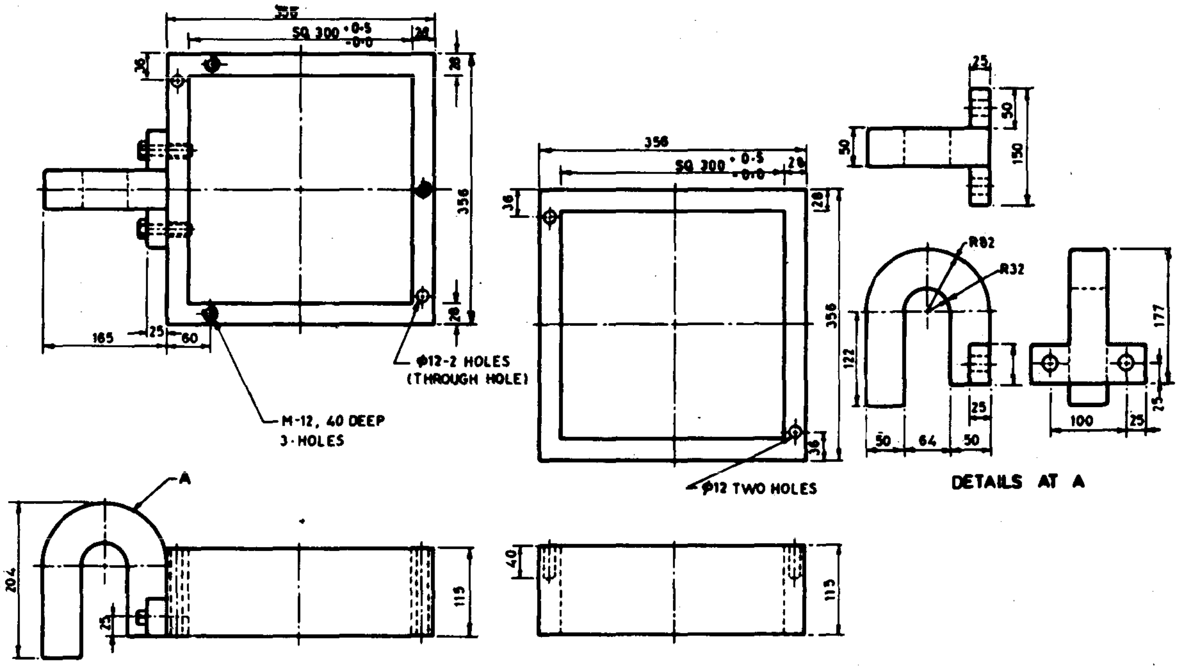


All dimensions in millimetres.

FIG. 1 SHEAR BOX (LARGE) ASSEMBLY

TABLE 1 MATERIALS OF CONSTRUCTION OF DIFFERENT COMPONENTS PARTS OF SHEAR BOX

Sl No.	Component	Material	Reference to Indian Standard
i)	Upper and lower parts of shear box	Mild steel	IS : 513-1973
ii)	Grid plates — 2 pairs	,,	,,
iii)	Spacer plates	,,	,,
iv)	Base plate	,,	,,
v)	Loading pad	,,	,,
vi)	Water jacket	,,	,,



2A Upper Half of Shear Box

2B Lower Half of Shear Box

All dimensions in millimetres.

FIG. 2 DETAILS OF UPPER AND LOWER HALVES OF SHEAR BOX

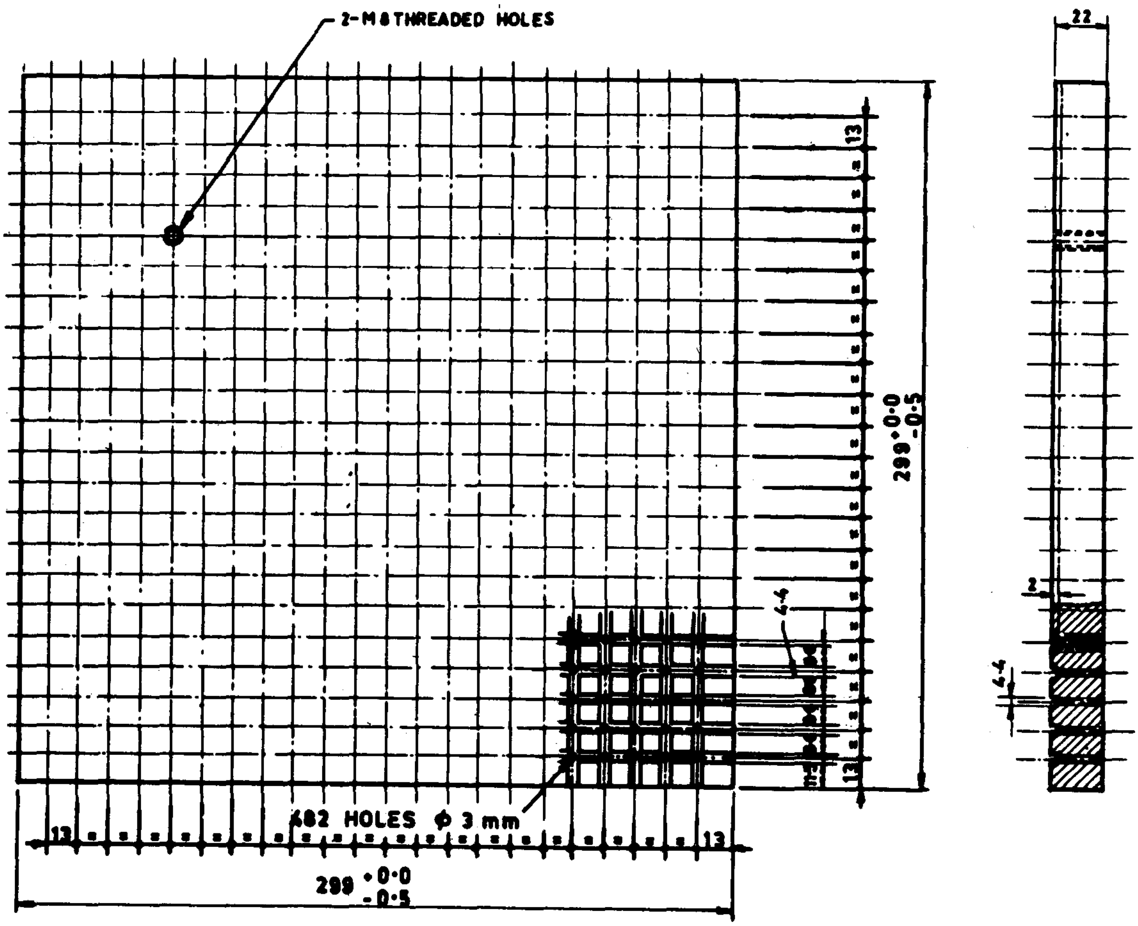
indelibily marked on each component of equipment :

a) Name of the manufacturer or his registered trade-mark; and

b) Date of manufacture.

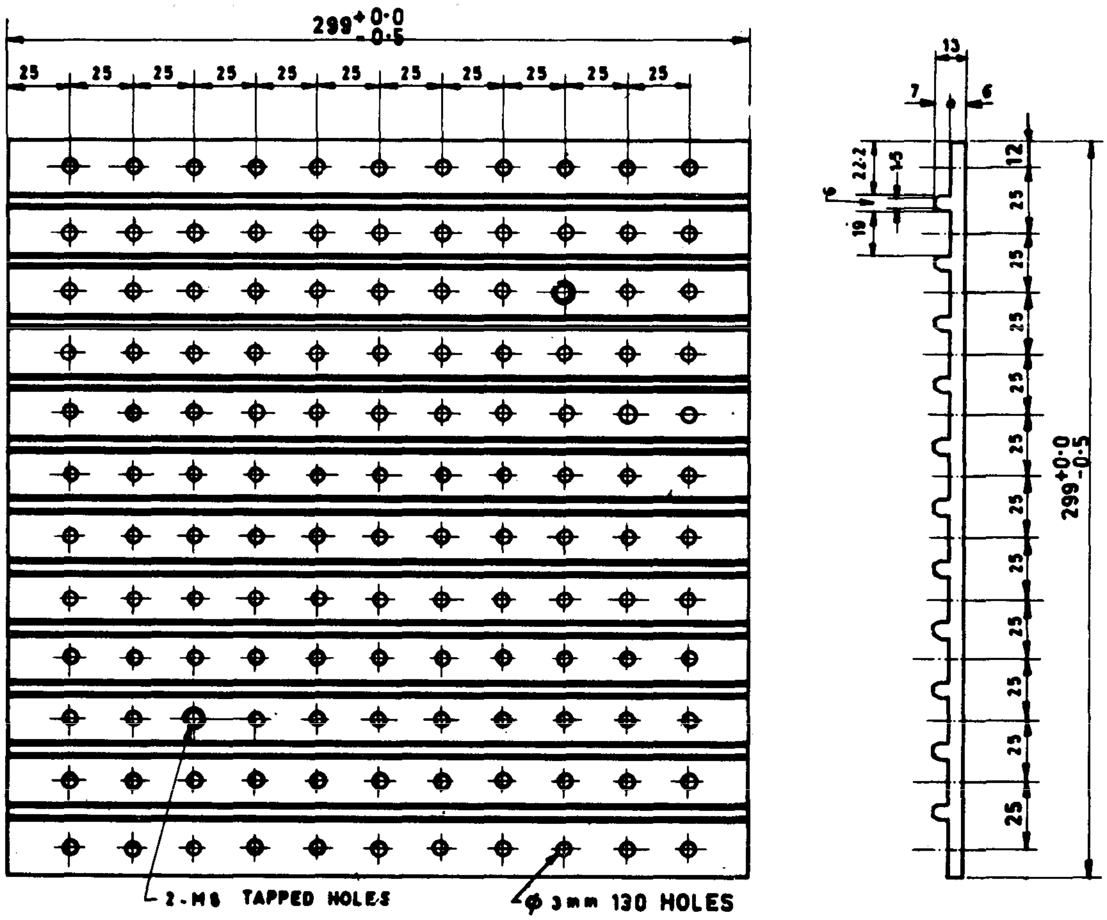
5.1.1 The equipment may also be marked with ISI Certification Mark.

NOTE — The use of the ISI Certification Mark is governed by the provisions of the Indian Standards Institution (Certification Marks) Act and the Rules and Regulations made thereunder. The ISI Mark on products covered by an Indian Standard conveys the assurance that they have been produced to comply with the requirements of that standard under a well-defined system of inspection, testing and quality control which is devised and supervised by ISI and operated by the producer. ISI marked products are also continuously checked by ISI for conformity to that standard as a further safeguard. Details of conditions under which a licence for the use of the ISI Certification Mark may be granted to manufacturers or processors may be obtained from the Indian Standards Institution.



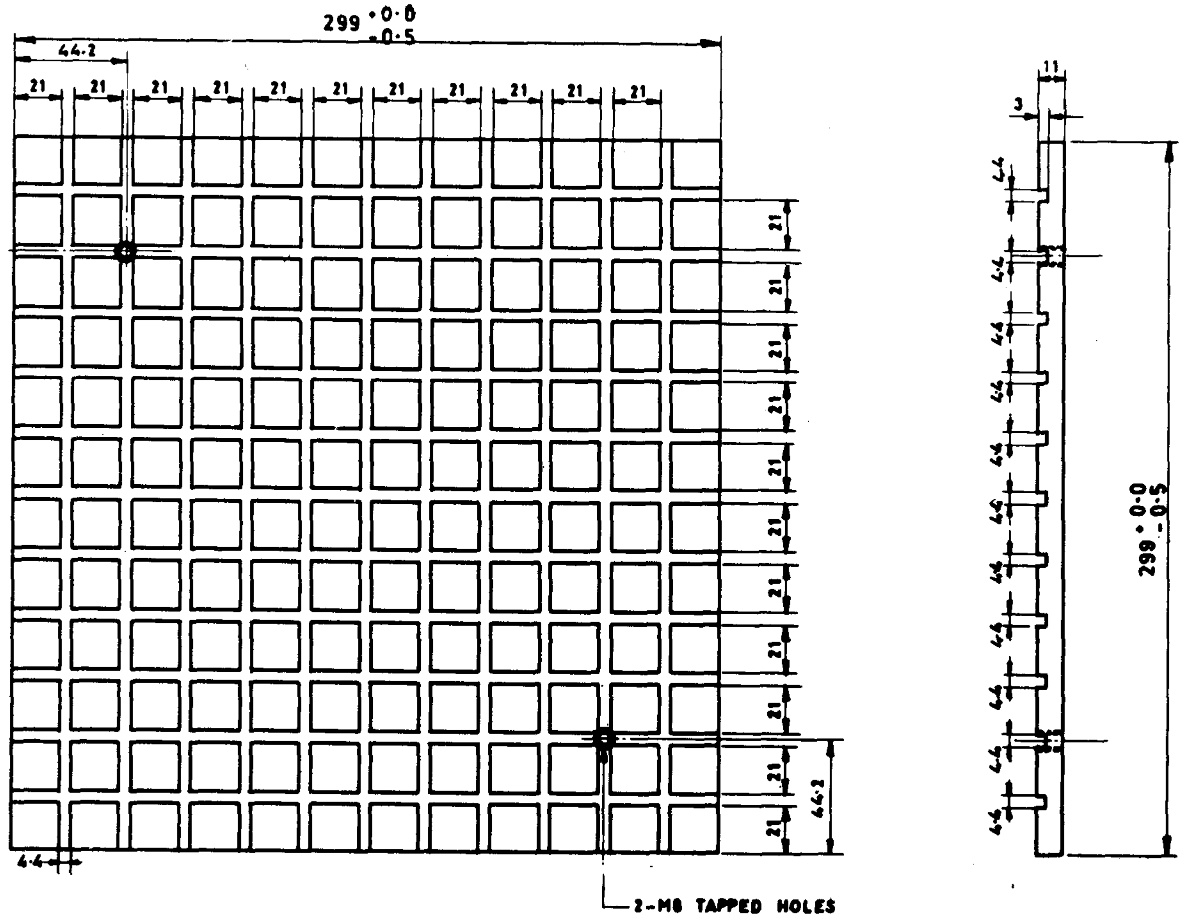
All dimensions in millimetres.

FIG. 3 SPACER PLATE



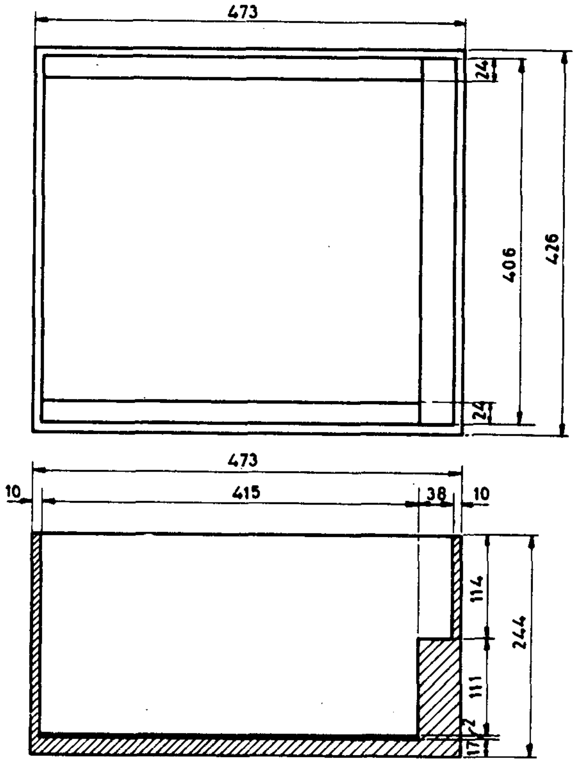
All dimensions in millimetres.

FIG. 4 GRID PLATE



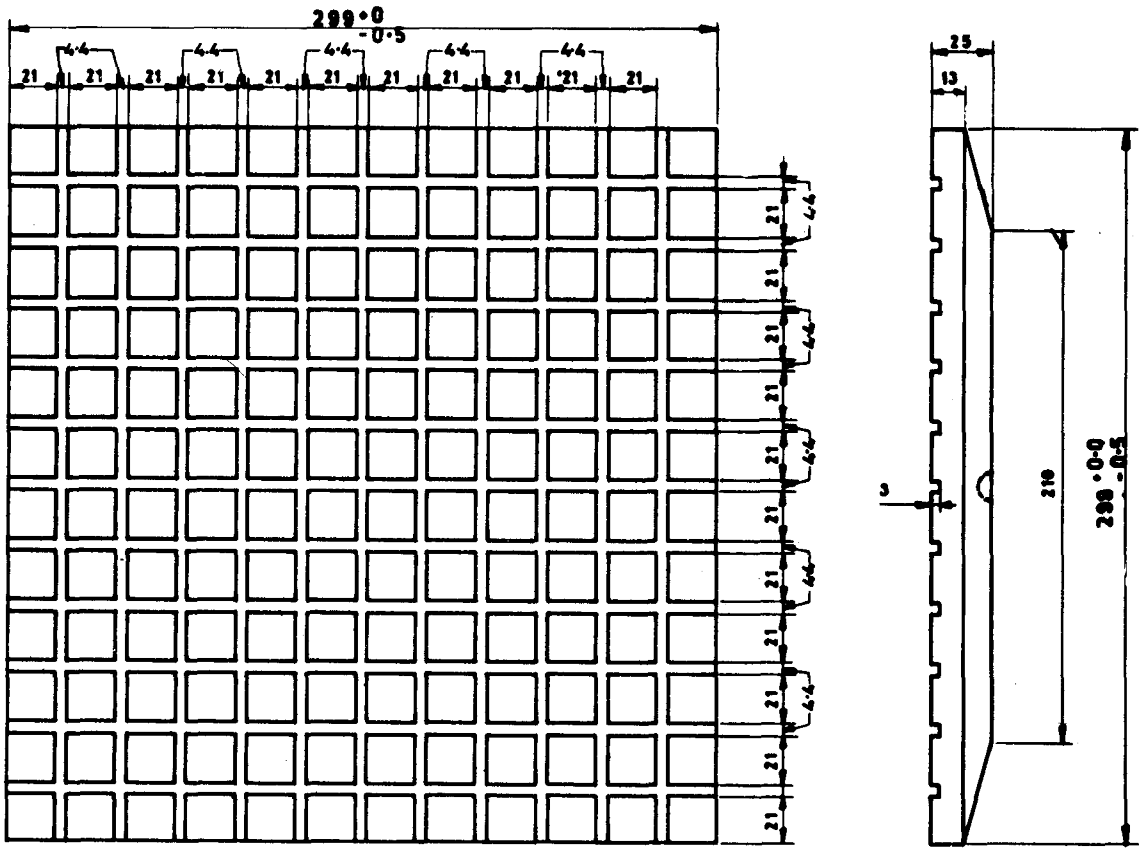
All dimensions in millimetres.

FIG. 5 BASE PLATE



All dimensions in millimetres.

FIG. 6 WATER JACKET



All dimensions in millimetres.

FIG. 7 LOADING PAD

Indian Standard
METHODS OF TEST FOR SOILS
PART 39 DIRECT SHEAR TEST FOR SOILS CONTAINING GRAVEL
Section 2 *In-Situ* Shear Test

0. FOREWORD

0.1 With a view to establishing uniform procedures for the determination of different characteristics of soils and also for facilitating a comparative study of the results, the Indian Standards Institution is bringing out this Indian Standard on methods of test for soils (IS 2720) which is being published in parts. Fortyone parts of this standard have been published so far. This part covers direct shear test. The test is of two kinds depending upon the state of samples, namely, laboratory test and *in-situ* test. The laboratory test is covered in Section 1 of this part. This part deals with *in-situ* determination by direct shear, the shear strength of soils containing gravel and cobblestone.

0.2 In the formulation of this standard due weightage has been given to international co-ordination among the standards and practices prevailing in different countries in addition to relating it to the practices in the field in this country.

1. SCOPE

1.1 This standard (Part 39/Sec 2) covers the method for the determination by direct shear, the *in-situ* shear strength of soils containing gravels and cobblestone.

2. APPARATUS

2.1 Shear Box — The side of the shear box shall be not less than 10 times the maximum expected particle size and the thickness of the samples not less than three times the maximum particle size. For convenience in handling the box could be of built-up sections from plates. The four sides of the box could be connected through bolts and nuts designed properly to form the box. Figure 1 shows the suggested size with 1 500 × 1 500 mm sample size.

2.2 Top Loading Plate (*see* Fig. 2) — A rigid steel plate fitting in the shear box suitably designed to distribute the load uniformly over the sample normal to the shear plane

2.3 Hydraulic Jack — Suitable remote control hydraulic jack of adequate capacity for applying shear force.

2.4 Rolled Steel Joist and Wooden Sleepers — Adequate number of rolled steel joist and wooden sleepers and sand bags for making a platform and providing adequate kentledge for applying normal load on the sample.

2.5 Rollers — Suitable size of rollers equal to the width of plates forming the shear box frame to be placed in between the side of the box and a bed plate on either side of the box.

2.6 Datum Bars — Suitable section of steel bars to be hinged for two pegs fixed at a distance equal to the side of the box driven to a depth of minimum 500 mm on either side of the box.

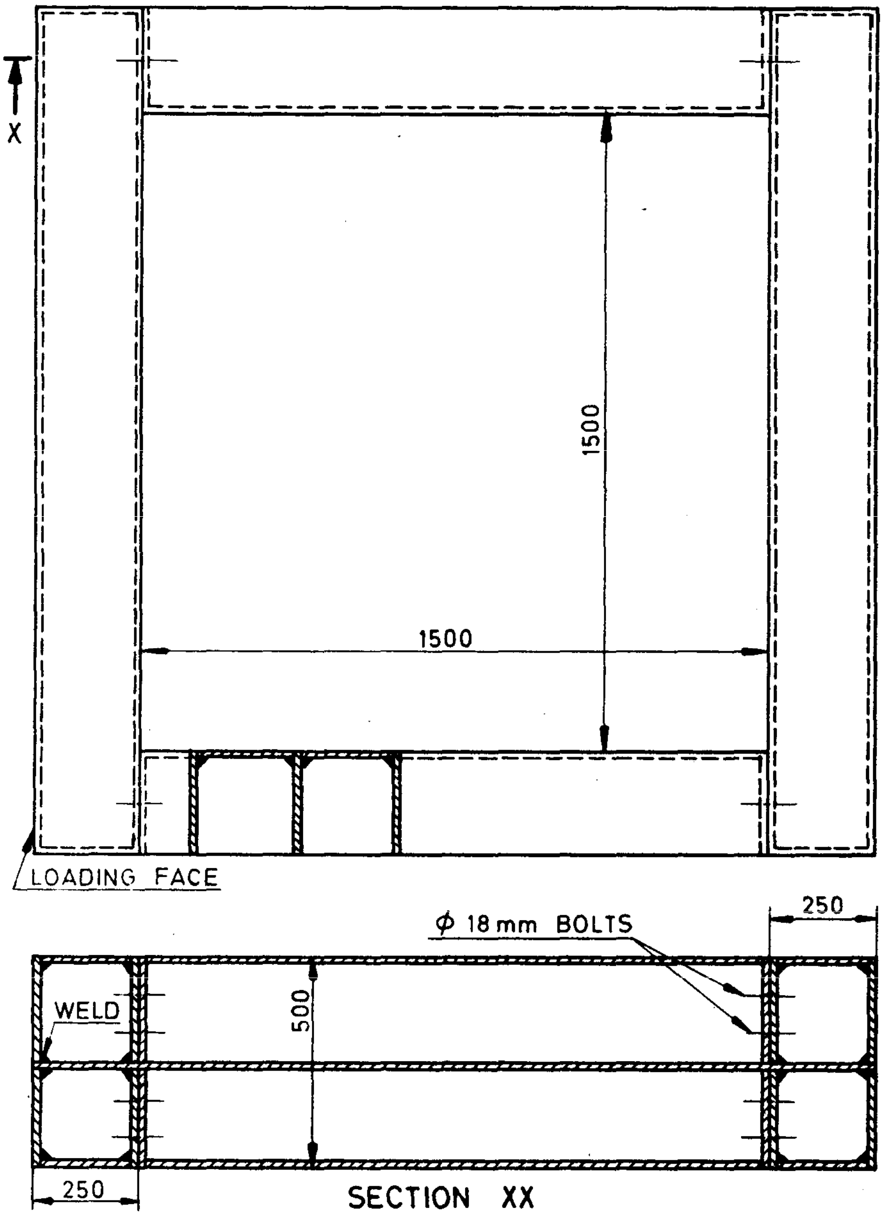
2.7 Spring Balance — Spring balance of 10 kg capacity of sensitivity 1.0 kg to weigh the sand bags.

3. PREPARATION OF SPECIMEN

3.1 A steel box made out of mild steel plates of adequate thickness, provided with a cutting edge with the required internal dimension may be used for trimming the sample. This hollow box be pressed into the deposit under a load applied by hydraulic jack (*see* Fig. 3). The soil around the box be excavated, simultaneously with the penetration of the box to facilitate its easy sinking. Care shall be taken to ensure that the box shall sink in vertical position.

3.1.1 Alternatively, during excavation two blocks, of the required size be left undisturbed at the desired position. After the excavation is completed the assembled boxes shall be put on the top of the block and soil below the plate shall be excavated gradually till the boxes reach the required position.

3.2 Two rolled steel joists from the bed plates. A train of rollers shall be put in between the sides of the box and the bed plates on either side of the box. Gravels projecting above box frames shall be removed and the gap shall be filled up with



NOTE — All members fabricated out of 10 mm thick plates.

All dimensions in millimetres.

FIG. 1 SHEAR BOX 1 500 × 1 500 mm SIZE

medium to fine sand to give level surface for better seating of the top plate.

3.3 The loading cap befitting the internal dimension of the box made of steel plates of adequate thickness be placed on the soil.

3.4 The test should be carried out at moisture content as close to field condition as possible.

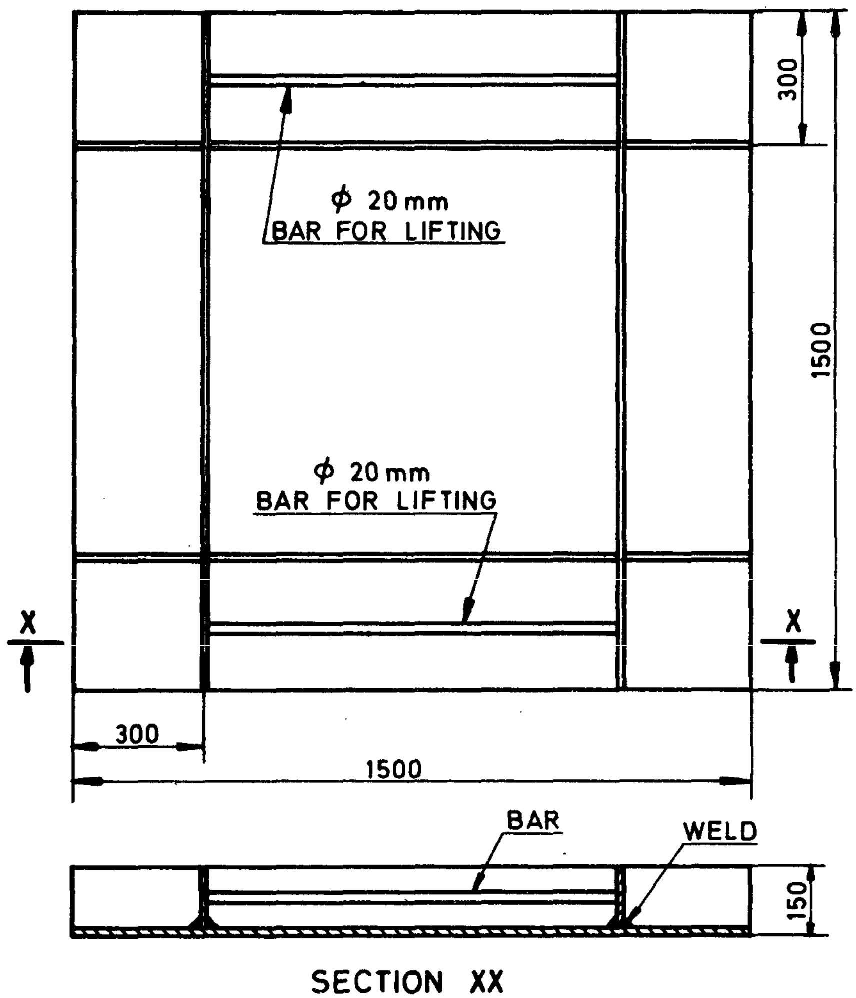
NOTE — In case of soils containing fines more than 12 precent, the test may be conducted in soaked sate to stimulate for worst field conditions; the soaking period may extend up to 4 days depending upon type of soils.

4. APPLICATION OF NORMAL LOAD

4.1 The normal load on the sample shall be applied with the help of a platform made of rolled steel joist and wooden sleepers and loaded with sand bags (*see* Fig. 4).

5. TEST PROCEDURE

5.1 The shear force shall be applied through a remote control hydraulic jack and proving ring



NOTE — All members fabricated out of 10 mm thick plates.
All dimensions in millimetres.

FIG. 2 TOP LOADING PLATE

arrangement taking reaction from the adjacent box (*see* Fig. 4).

5.2 Two tests at different normal pressures shall be carried out at one location. After the block with lesser normal pressure failed, the space between the failed block and the side of the pit shall be blocked by boulders and also by struting. The normal load on failed block shall be increased, and then the test on the other block shall be completed by taking reaction of shear force from the failed block.

5.3 The jack shall be so fitted so that the application of the lateral load occurs as far as possible near to the plane of shear.

5.4 The test shall be conducted by giving an equal increment of shear load under the normal load each increment of shear load shall be maintained constant till the equilibrium conditions are reached, the readings shall be recorded with the help of suitably mounted dial gauges. The next increment of shear load shall then be applied and the process continued till the failure of the specimen occurs. The normal loads applied shall be more than the existing over-burden and cover the anticipated loading range in the area. The range of normal load should represent the site loading conditions as far as possible. Two more tests at different normal loads shall be carried out at adjacent location, to make four tests, a minimum number required necessary for interpretation.

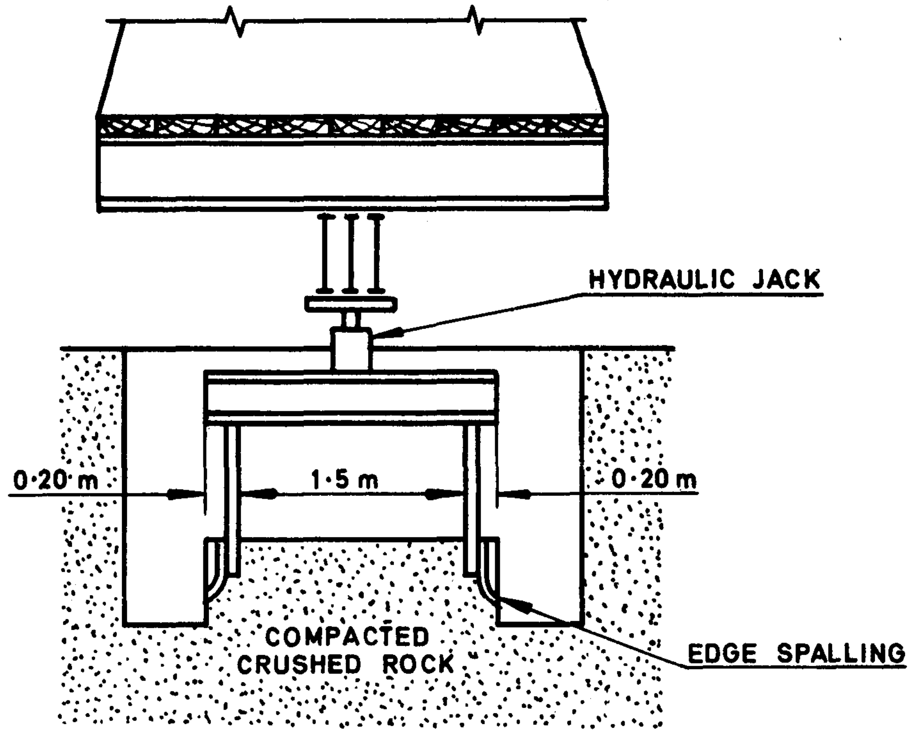


FIG. 3 ARRANGEMENT FOR OBTAINING SPECIMEN FOR *IN-SITU* SHEAR TEST

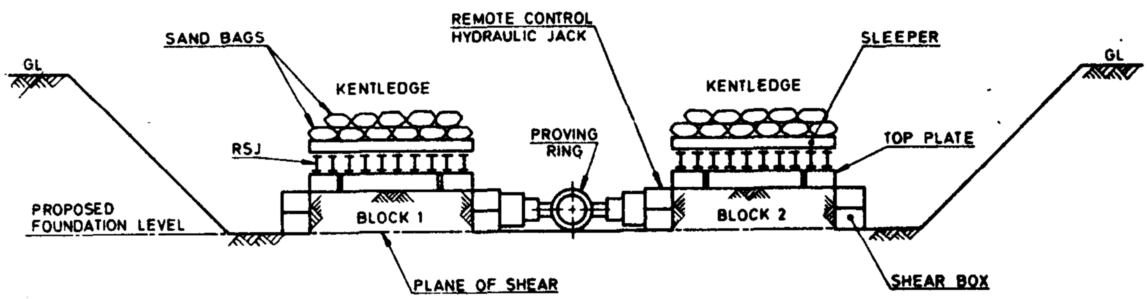


FIG. 4 TEST SET UP FOR LARGE SIZE *IN-SITU* SHEAR TEST

6. CALCULATION AND REPORT

6.1 Results of test shall be recorded suitably. A recommended proforma for recording the results is given in Appendix A.

6.2 The horizontal displacement at a particular load shall be recorded from shear displacement dial readings.

6.3 The maximum shear force shall be peak load from load displacement curve or where the tangent of flater portion of the later part of the curve leaves in case the curve does not give peak point.

6.4 The maximum shear stress and the corresponding horizontal displacement and applied normal stress shall be recorded for each test and the result be presented in the form of a graph in which the applied normal stress is plotted as abscissa and shear stress as ordinate. The angle which the resulting straight line makes with the horizontal axis and the intercept which the straight line makes with the vertical axis shall be reported as the angle of shearing resistance and cohesion respectively.

NOTE — The normal stress *versus* maximum shear stress relationship may not be straight line in all cases. In such cases the shear parameter shall be obtained by drawing tangent to the normal stress and maximum shear stress curve at the point of normal stress expected in the field.

APPENDIX A

(Clause 6.1)

I PROFORMA FOR RECORDING IN-SITU TEST RESULTS

Project..........................

Rate of load increment..........................

Location of sample..

Specimen No..

Depth of test..

Proving ring No...

Providing ring constant.....................................

Normal load applied..

SOIL SPECIMEN MEASUREMENT

Dimensions...........................

Initial water content................

Final water content................

Maximum size of particle.................................

Area of specimen.....................................

Height of specimen...

Volume of specimen.......................................

Unit weight of soil...

II PROFORMA FOR RECORDING SHEARING STAGE

i) Thickness of sample......................................mm

ii) Area of cross section of sample.........................cm^2

iii) Rate of shearing..mm/min

iv) Normal stress applied...................................kg/cm^2

Date & Time	Shear Dis-placement Dial Reading		Shear Dis-placement		Average Shear Displace-ment	Proving Ring Reading	Shear Force	Shear Stress	Vertical Reading		Vertical Displacement	
	D_{h1}	D_{h2}	D_{h1}	D_{h2}					D_{r1}	D_{r2}	D_{r1}	D_{r2}
Average Vertical Displace-ment												

Plot — Shear stress versus shear displacement and find :

a) maximum shear stress at the peak of curve, and

b) corresponding shear displacement

III PROFORMA FOR RECORDING SUMMARY OF RESULTS

Test No.	Normal Stress	Proving Ring Constant	Shear Stress at Failure	Shear Displacement at Failure	Initial Water Content	Final Water Content	Remarks

Plot — Shear stress-normal stress relationship to obtain :

a) cohesion intercept, and
b) angle of shearing resistance.

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SECTION 10

Determination of CBR

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Indian Standard

METHODS OF TEST FOR SOILS

PART 31 FIELD DETERMINATION OF CALIFORNIA BEARING RATIO

(Incorporating Amendment No. 1)

0. FOREWORD

0.1 The bearing ratio test (generally known as the California bearing ratio test) is an ad hoc penetration test used for the evaluation of strengths of sub-grade and bases for roads and runway pavements. The results obtained by these tests are used in conjunction with empirical curves, based on experience for the design of flexible pavements. The test gives empirical strength values which may not be directly related to fundamental properties governing the strength of soil.

0.1.1 The test is either performed in the laboratory on undisturbed or recompacted soil specimens or directly in the field. Where facilities for reaction loading are not available undisturbed samples may be obtained and tested in the laboratory. The laboratory procedure has been covered in IS 2720 (Part 16) : 1979.

1. SCOPE

1.1 This standard (Part 31) covers the method for the determination of the bearing ratio (generally known as the California bearing ratio) of soils in-place for the evaluation of strengths of sub-grade and bases for roads and runway pavements.

2. TERMINOLOGY

2.0 For the purpose of this standard, the definitions given in IS 2809 : 1972 and the following definitions shall apply.

2.1 Bearing Ratio — (Generally known as California bearing ratio or CBR.) The ratio of the force per unit area required to penetrate a soil mass with a standard circular piston at the rate of 1.25 mm/min to that required for corresponding penetration of a standard material.

2.2 Standard Load — Load which has been obtained from the test on crushed stone which is defined as having a bearing ratio of 100 percent (*see* **5.2**).

3. APPARATUS

3.1 Loading Device — A mechanical screw loading jack with swivel head for applying load to the penetration piston. The device should have an arrangement for attaching to truck, tractor, truss or any other equipment used to provide load reaction. The jack should be such that a uniform penetration rate of 1.25 mm/min can be achieved. The capacity of the jack should not be less than 5 000 kg.

3.2 Equipment for Providing Reaction for Loading — Truck, tractor, truss or any other suitable equipment. If truck or tractor is used they should be loaded suitably to give the necessary reaction. If truss is used it should be suitably anchored.

3.3 Jacks — Two track-type jacks of 5 to 12 tonnes capacity and having double acting combination trip and automatic lowering in cases where loaded truck or tractor is used for providing the necessary reaction.

3.4 Proving Ring — One calibrated proving ring of capacity 5 000 kg with a dial gauge to read to an accuracy of 0.002 mm and having a travel of 5 mm. The proving ring should have an accuracy of half percent of the load measured.

3.5 Metal Penetration Piston — 50 ± 0.1 mm in diameter and not less than 100 mm long.

3.6 Extensions — Internally threaded pipe or rod extensions not less than 200 cm long furnished in the following quantities and lengths :

Length of Extension (see *Note*) cm	*Number of Extensions*
5	2
10	2
30	1
50	1
100	1

NOTE — Other convenient lengths may also be used.

3.7 Connectors — For coupling the penetration piston and proving ring assembly either directly or through extension pieces.

3.8 Dial Gauge — Reading to 0.01 or 0.02 mm having a travel of 25 mm, for measuring the penetration of the piston.

3.9 Dial Gauge Support — Rigid end of steel angle welded construction or light alloy pipe construction about 2 m long, of overall height 30 cm and 45 cm wide at the feet with universal or ordinary dial gauge holder adjustable anywhere along the length of the support.

3.10 Surcharge Weight — One annular metal weight weighing 5 kg and of 250 mm diameter with a central hole 53 mm in diameter. Two circular slotted weights of weight 5 kg and of diameter 215 to 250 mm with a central hole and slot width of 53 mm. Two circular slotted weights of weight 10 kg and of diameter 215 to 250 mm with a central hole and slot width of 53 mm.

3.11 Miscellaneous Apparatus — Other general apparatus, such as spirit level, pick, spade, scoop and brush, apparatus for moisture determination [*see* IS 2720 (Part 2) : 1973] and density determination [*see* IS 2720 (Part 28) : 1974 and IS 2720 (Part 29) : 1975].

4. PROCEDURE

4.1 The general surface area to be tested should be exposed, cleaned of all loose and dried material and levelled. Extreme care shall be taken not to disturb the test surface. The spacing of the tests should be such that operations in one area do not disturb the soil in the other area. For testing operations this spacing may range from 15 to 20 cm in cohesive soils and 35 to 40 cm in cohesionless soils, for the penetration piston used in the test.

4.2 If actual service conditions in the field warrant, the surface to be tested may be soaked to the desired degree using surcharge weights, if necessary. The test surface should be drained of all free water and allowed to stand for at least 15 minutes before starting further operations.

4.3 The equipment used to provide load reaction (truck, tractor, truss etc), should be so located that the centre of the beam against which the loading jack will work is over the centre of the surface to be tested. If loaded truck or tractor is used for providing the necessary reaction, the rear wheels of the truck or tractor should be completely raised by means of the track type jacks placed below the frame of the body near the wheels in order to avoid the loss of loading effort which would otherwise be spent on the flexing of the axial springs of the vehicle at the time of testing. In order to avoid accidents due to the failure of jacks near the wheels and the lifting of the vehicle at higher loads, the rear side of the body of the vehicle should be placed over two rigid supports. The screw jack with swivel should be installed to the underside of the equipment providing reaction, at the correct position for the test. The proving ring should be connected to the bottom end of the jack and the piston connector to the bottom of the proving ring. The piston should then be connected using lengths of extension pipes or rods, if necessary. It should be ensured that the entire assembly is plumb and the loading jack should be clamped in position.

4.4 The surcharge annular weight of 5 kg should be kept in position on the surface to be tested so that when the piston is lowered it will pass through the hole in the annular weight. The penetration piston should be seated with the smallest possible load but in no case in excess of a total load of 4 kg or 0.002 kg/mm^2 so that full contact is established between the piston and the surface to be tested. For materials with irregular surface the piston may be seated on a thinnest practical layer of fine limestone screenings or plaster of paris spread over the surface.

4.5 While seating load is on the piston a 3 to 6 mm layer of clean sand should be spread over the surface to be covered by the surcharge annular weight. This helps in distributing the surcharge load over the surface uniformly.

4.6 Surcharge weights, sufficient to produce an intensity of loading equal to the weight of the base material and pavement, except that the minimum weight applied should be 15 kg including that of the annular weight [this weight gives an intensity of loading approximately equal to that in the laboratory bearing ratio test, *see* IS 2720 (Part 16): 1965] should be applied. The penetration indicating dial should be suitably fixed for reading the penetration and the dial set to zero. A diagramatic set up of the test is shown in Fig. 1.

4.7 Load shall be applied on the penetration piston so that the penetration is approximately 1.25 mm/min. The load readings shall be recorded at penetrations of 0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 7.5, 10.0 and 12.5 mm. The maximum load and penetration shall be recorded if it occurs for a penetration less than 12.5 mm. The set up may then be dismantled.

4.8 After completion of the test, a sample shall be obtained at the point of penetration, for moisture content determination. The moisture content shall be determined in accordance with IS 2720 (Part 2):1973. The in-place density shall be determined in accordance with IS 2720 (Part 28) : 1974 or IS 2720 (Part 29) : 1975 about 15 cm away from the point of penetration.

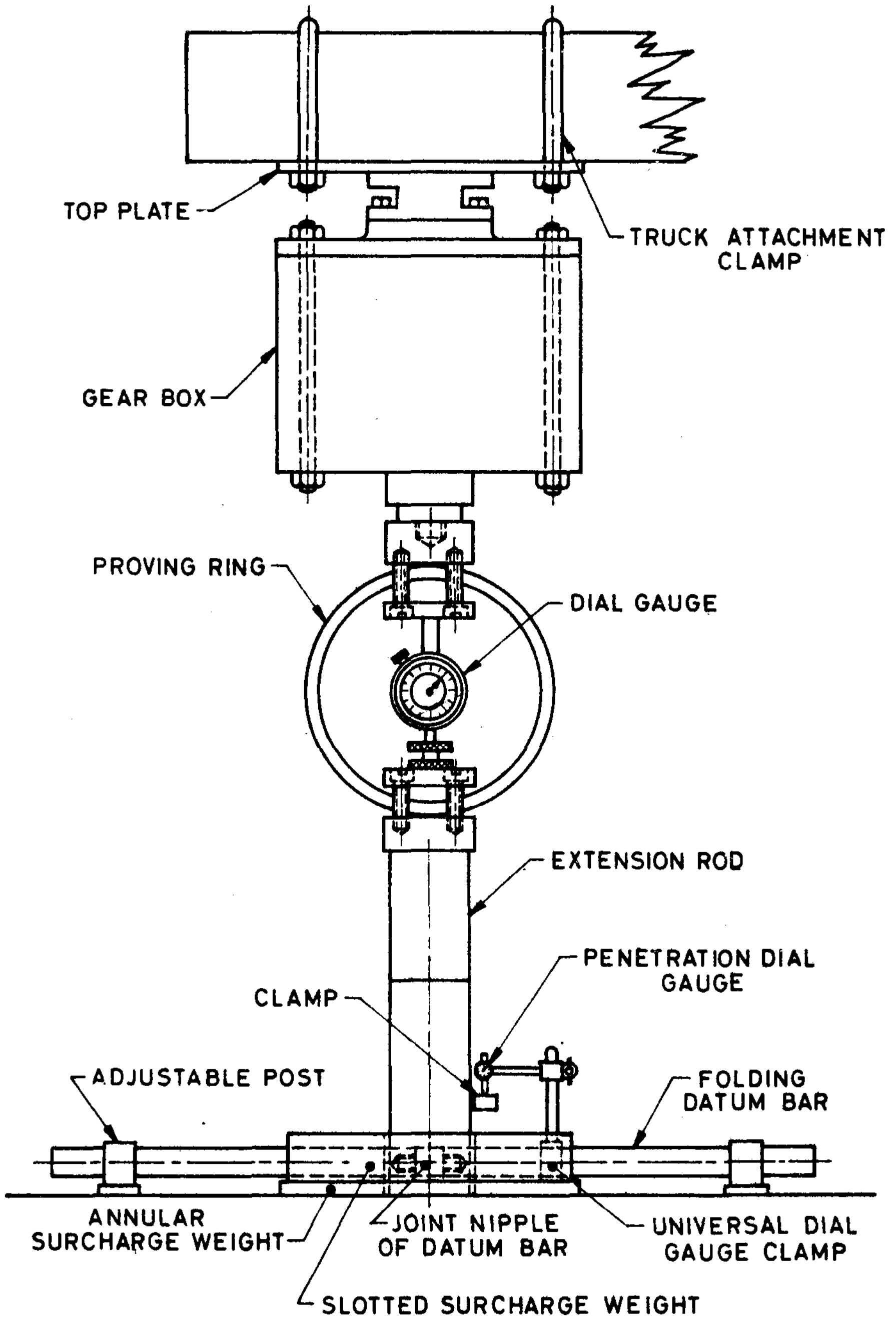

FIG. 1 FIELD CALIFORNIA BEARING RATIO APPARATUS

5. CALCULATIONS

5.1 Load Penetration Curve — The load penetration curve shall be plotted (*see* Fig. 2). This curve will be mainly convex upwards although the initial portion of the curve may be concave upwards due to surface irregularities. A correction shall then be applied by drawing a tangent to the curve at the point of greatest slope. The corrected curve shall be taken to be this tangent together with the convex portion of the original curve with the origin of strains shifted to the point where the tangent cuts the horizontal strain axis as illustrated in Fig. 2.

5.2 Bearing Ratio — Corresponding to the penetration value at which the bearing ratio is desired, corrected load values shall be taken from the load penetration curve and the bearing ratio

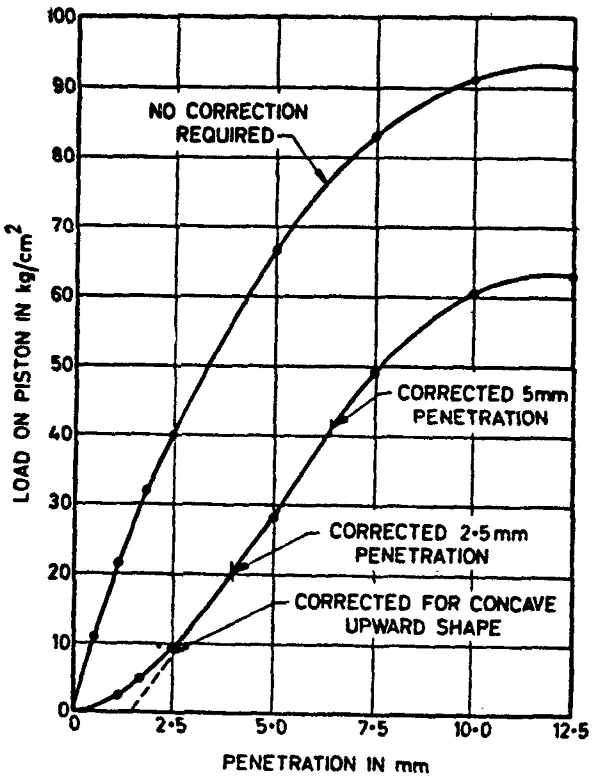

FIG. 2 CORRECTION OF LOAD PENETRATION CURVES

calculated as follows :

$$\text{Bearing ratio} = \frac{P_T}{P_S} \times 100$$

where

P_T = corrected unit (or total) test load corresponding to the chosen penetration value read from the load penetration curve, in kgf/cm^2; and

P_S = unit (or total) standard load for the same depth of penetration as per P_T taken from Table 1, in kgf/cm^2.

TABLE 1 STANDARD LOAD

(Clause 5.2)

PENETRATION DEPTH	UNIT STANDARD LOAD	TOTAL STANDARD LOAD
mm	kgf/cm^2	kgf
2.5	70	1 370
5.0	105	2 055
7.5	134	2 630
10.0	162	3 180
12.5	183	3 600

5.2.1 The bearing ratios are usually calculated for penetrations of 2.5 mm and 5 mm. Generally the bearing ratio at 2.5 mm penetrations will be greater than that at 5 mm penetration and in such a case the former shall be taken as the bearing ratio for design purposes. If the bearing ratio corresponding to a penetration of 5 mm exceeds that for 2.5 mm, the test shall be repeated. If identical results follow, the bearing ratio corresponding to 5 mm penetration shall be taken for design.

6. REPORT

6.1 The bearing ratio shall be reported correct to the first decimal place. The details in the recommended proforma for the record of test results given in Appendix A shall be given.

7. NUMBER OF FIELD TESTS

7.1 Three in-place bearing ratio tests shall be performed at each elevation to be tested. However, if the results of the three tests in any group do not show reasonable agreement, three additional tests shall be made at the same location and numerical average of the six tests shall be used as the bearing ratio at that location. A reasonable agreement between the minimum and maximum values of three tests where the bearing ratio is less than 10, permits a tolerance of 3; from 10 to 30, a tolerance of 5; from 30 to 60, a tolerance of 10; and greater than 60, a tolerance of 25. If it is known that a single value is erratic for any reason, the test value should be discarded and another test performed.

APPENDIX A

(Clause 6.1)

PROFORMA FOR IN-PLACE BEARING RATIO TEST

Location..Tested by..

Material at test point... Date...

Depth of test..

Condition of test surface : soaked / unsoaked

Period of soaking, if any..

Surcharge weight used during soaking..

Moisture content..

Density..

Method used for determination of density...

Penetration test

Surcharge weight used..

PENETRATION mm	PROVING RING DIAL GAUGE READINGS	LOAD kg	CORRECTED LOAD kg (*see* 5.1)
0.5			
1.0			
1.5			
2.0			
2.5			
3.0			
4.0			
5.0			
7.5			
10.0			
12.5			

Bearing ratio at 2.5 mm penetration $\left.\begin{matrix}1\\2\\3\end{matrix}\right\}$ Average

Bearing ratio at 5 mm penetration $\left.\begin{matrix}1\\2\\3\end{matrix}\right\}$ Average

Rejected test with reasons..

Result of repeat test, if conducted...

Indian Standard

CODE OF PRACTICE FOR FIELD CONTROL OF MOISTURE AND COMPACTION OF SOILS FOR EMBANKMENT AND SUBGRADE

0. FOREWORD

0.1 The earthwork involved in embankments and subgrades has to be controlled so that the average properties of the soil are equal to those adopted in design. A number of field control methods have been evolved. This standard covers such methods and also gives guidance for use in various situations. It is suggested that the tests mentioned in this standard are conducted at regular intervals so that the results are available for every 1 000 m^2 of earth file.

1. SCOPE

1.1 This standard covers various methods of field control of compaction and moisture contents of soil for embankment and subgrade.

2. METHODS APPLICABLE TO NON-GRAVELLY SOILS

2.1 Method 1 — In this method, compaction parameters, that is, optimum moisture content and maximum dry density are determined according to the procedure described in IS 2720 (Part 7) : 1980 and IS 2720 (Part 8) : 1974. The *in-situ* moisture content of compacted soil is determined by one of the procedures given in IS 2720 (Part 2) : 1973. The field dry density is determined by any one of the methods given in IS 2720 (Part 28) : 1974, IS 2720 (Part 29) : 1975 or IS 2720 (Part 34) : 1972. The test shall be performed after removing the top 5 cm layer of earth.

The compaction efficiency is then obtained by expression of field density-laboratory maximum dry density.

2.2 Method 2 — This method allows the determination of the relationship between the embankment moisture content, dry density and the laboratory optimum conditions without the necessity of measurement of water content and the results can be obtained in less than one hour. This method, as given in IS 2720 (Part 38) : 1976 can be used directly for both moisture and density controls or only density control.

2.3 Method 3 — In certain weathered soils, field moisture content and dry density differ from the laboratory compaction values. In such soils, a test embankment under nearly identical operating conditions for thickness of soil, watering, mixing and compacting is used to determine field moisture content and dry density attainable. The specified layer of soil should be spread on a test strip 3×10 m, watered and left for 5 to 30 minutes depending upon type before rolling. The water content is varied in layers within + 6 percent of laboratory values. Each strip is rolled by the roller and the density of soil is measured by either of the methods mentioned in **2.1** after every two passes. A graph of the number of passes against dry density is drawn for each water content. A graph of maximum dry density attained when plotted against water content gives field moisture content and attainable field dry density. The trial gives a minimum number of passes of compaction roller which at field moisture content will give maximum dry density.

Based on this test embankment, indirect control of number of passes with controlled water using any one of the methods for determining the moisture content [IS 2720 (Part 21) : 1973] can be used for the earthwork.

3. METHOD APPLICABLE TO SOIL CONTAINING GRAVELS AND ROCKFILLS

3.1 In addition to the methods given for non-gravelly soils, the following provisions shall be applicable.

3.2 The total density of soil increases and moisture content decreases with increasing percentage of gravel size fraction up to 60 to 75 percent and above this value density again decreases.

3.3 For the soils with gravels up to 30 percent, recommended method is to establish moisture density relationship [*see* IS 2720 (Part 7) : 1980 and IS 2720 (Part 8) : 1974] in the laboratory on soil fraction passing 40 mm IS Sieve. The embankment density may be compared with the laboratory density so obtained. The field density and the moisture content of the embankment may be determined by the method preferably given in IS 2720 (Part 33) : 1971 or alternatively as given in IS 2720 (Part 28) : 1974.

3.4 As shear strength of compacted gravel and rockfill does not vary much with small changes in the density and higher precise densities can be attained without precise control of water content as in the case of fine grained soil, controlled testing may not be necessary.

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SECTION 11

Sampling of Soils

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Indian Standard

SPECIFICATION FOR EARTH AUGERS (SPIRAL TYPE)

0. FOREWORD

0.1 The equipment covered in this standard is used for piling work, soil boring and sampling works.

1. SCOPE

1.1 This standard covers dimensional and general requirements for earth augers (spiral type), used in piling, soil boring and sampling works.

2. SIZES, DIMENSIONS AND TOLERANCES

2.1 These shall be as given in Table 1, read with Fig. 1. The nominal size refers to the diameter bored by augers.

3. MATERIALS

3.0 The material for construction of various parts shall be as under.

3.1 Pilot Bit — It shall conform to steel conforming to designation T 90 of Schedule VI of IS 1570 : 1961 with a maximum sulphur and phosphorus content of 0.05 percent each.

3.2 Blades — It shall conform to steel conforming to designation T 90 of Schedule VI of IS 1570 : 1961 with a maximum sulphur and phosphorus content of 0.05 percent each.

3.3 Base Plate — It shall be made of mild steel conforming to IS 513 : 1973.

3.4 Spirals — It shall be made of mild steel conforming to IS 513 : 1973.

3.5 Pipe Shaft — It shall be made of mild steel of heavy grade as specified in IS 1239 (Part 1) : 1979.

3.6 Couplers — It shall be made of mild steel conforming to IS 513 : 1973 hardened and tempered to produce a hardness reading within range 360 to 420 HV 10 (*see* IS 1501 : 1968).

3.7 Extension Rods — It shall be made of mild steel of heavy grade as specified in IS 1239 (Part 1) : 1979.

3.8 Handles and Extension to Handles — It shall be made of mild steel of heavy grade conforming to IS 1239 (Part 1) : 1979.

4. CONSTRUCTION

4.1 The blade shall be either plain or toothed (*see* Fig. 1). The edges of the blade shall be backed with non-erodable welding so as to have hardness 600 to 700 HV 10 (*see* IS 1501 : 1968). The pilot bit, blades, spirals and coupler shall be welded to the shaft. The plate coupler shall be of size 100 × 75 × 8 mm except for auger sizes 550 and 600, it shall be 125 × 100 × 8 mm.

5. PERFORMANCE TEST

5.1 The auger shall bore satisfactory a minimum depth of spiral length in a fairly consolidated soil. The auger shall not be withdrawn during the test but shall withdraw easily after it reaches the required depth. At the end of the test, the auger shall show no sign of damage, fracture or flaw.

6. WORKMANSHIP

6.1 The blades and pilot bits shall be free from cracks, seams, etc.

7. TREATMENT

7.1 The auger shall be coated with one coat of red oxide or anti-corrosive paint and finally painted with blue enamel paint.

8. MARKING

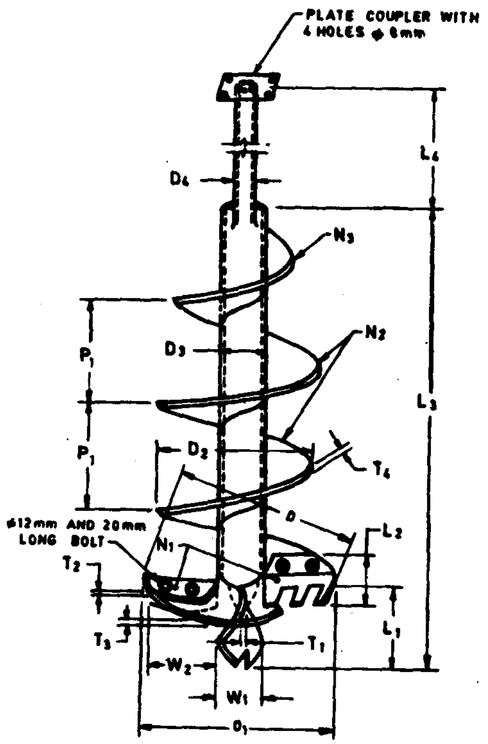
8.1 — The following information shall be clearly and indelibly marked on each auger :

a) Name of the manufacturer or his registered trade-mark or both,
b) Type (*see* **4.1**), and
c) Date of manufacture.

8.1.1 The auger may also be marked with the ISI Certification Mark.

NOTE — The use of the ISI Certification Mark is governed by the provisions of the Indian Standards Institution (Certification Marks) Act and the Rules and Regulations made thereunder. The ISI Mark on products covered by an Indian Standard conveys the assurance that they have been produced to comply with the requirements of that standard under a well-defined system of inspection, testing and quality control which is devised and supervised by ISI and operated by the producer. ISI marked products are also continuously checked by ISI for conformity to that standard as a further safegurad.

Details of conditions under which a licence for the use of the ISI Certification Mark may be granted to manufacturers or processors, may be obtained from the Indian Standards Institution.



NOTE — The angle of taper for blades shall be 30 ± 5°.

FIG. 1 EARTH AUGER (SPIRAL TYPE)

TABLE 1 SIZES, DIMENSIONS AND TOLERANCES OF AUGER

(*Clause* 2.1)

NOMINAL SIZE	PILOT BITS			BLADES				BASE PLATE		SPIRALS					SHAFT			
D	$L_{1\,-3}^{\,+5}$	$W_{1\,-3}^{\,+5}$	$T_{1\,-1}^{\,+1}$	$L_{2\,-3}^{\,+5}$	$W_{2\,-3}^{\,+5}$	$T_{2\,-1}^{\,+1}$	N_1	$D_1\,{}^{5}_{4}$	$T_{3\,-1}^{\,+1}$	$D_2\,{}^{5}_{4}$	$T_{4\,-0.5}^{\,+1}$	P_1	N_2	N_3	$L_3 \pm 5$	$D_{3\,-0.5}^{\,+0.5}$	$L_4 \pm 5$	$D_4 + 0.5$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
mm	mm	mm	mm	mm	mm	mm	Nos	mm	mm	mm	mm	mm	Nos	Nos	mm	mm	mm	mm
100	60	25	6	50	40.0	6	1	90	8	85	3	85±5	3	1	250	26.9	750	26.9
150	90	40	6	85	57.5	6	1	140	8	135	3	85±5	3	1	300	33.8	700	33.8
200	115	50	8	115	77.5	8	1	185	8	150	3	110±5	3	1	400	42.5	600	42.5
250	115	50	8	115	102.5	8	1	235	8	230	3	110±5	3	1	400	42.5	600	42.5
300	115	50	8	125	127.5	8	1	285	8	280	3	110±5	3	1	400	42.5	600	42.5
375	150	100	12	150	147.5	10	2	360	10	350	4	165±10	2	1	550	76	450	42.5
400	150	100	12	150	160.0	10	2	385	10	375	4	165±10	2	1	550	76	450	42.5
450	150	100	12	150	185.0	10	2	435	12	425	4	165±10	2	1	550	76	450	48.4
500	150	100	12	150	210.0	10	2	485	12	475	4	165±10	2	1	550	76	450	48.4
550	150	100	12	150	235	10	2	535	12	520	4	200±10	2	1	650	88.7	350	60.2
600	150	100	12	150	260	10	2	585	12	570	4	200±10	2	1	650	88.7	350	60.2

Indian Standard
CODE OF PRACTICE FOR SAMPLING OF SOILS BY THIN WALL SAMPLER WITH STATIONARY PISTON

0. FOREWORD

0.1 Undisturbed samples of soil are required for a number of soil tests, such as unconfined compression test, consolidation test, permeability test and triaxial compression test. It has been recognized that it is not practicable to obtain a truely undisturbed sample, but if certain procedures and precautions are observed, it is possible to get relatively undisturbed samples which may be considered sufficient keeping in view the nature of tests to be performed on these samples. This code deals with the method of obtaining such samples using thin wall sampler with stataionary piston, which are normally used for clay and silt formation.

1. SCOPE

1.1 This standard describes the method for obtaining undisturbed soil samples in fine grained soils for laboratory tests using thin wall sampler with stationary piston.

2. TERMINOLOGY

2.1 For the purpose of this standard, the definitions given in IS 2809 : 1972 and the following shall apply.

2.1.1 *Thin Wall Sampler with Stationary Piston* — A sampler having a piston inside the thin wall sampling tube, in which during sampling, the position of the piston remains stationary and the sampling tube penetrates into the soil.

2.1.2 *Undisturbed Sample* — The sample taken with the minimum disturbance, maintaining the structure and engineering characteristics of soil as close as possible to its conditions *in-situ*.

2.1.3 *Area Ratio* — The area ratio (A_r) represents the volume of soil displaced by the sampler in proportion to the volume of the soil sample and is calculated as follows :

$$A_r \text{ (precent)} = \frac{D_e^2 - D_i^2}{D_i^2} \times 100$$

where D_e and D_i are as shown in Fig. 1.

2.1.4 *Inside Clearance* — For reducing the friction between the soil sample and inside of the sampler, the inside diameter of the sampling tube is kept slightly bigger than the diameter at its cutting edge. The inside clearance (C_e) is defined as $C_e = \frac{D_i - D}{D}$, where D_i and D are as shown in Fig. 1.

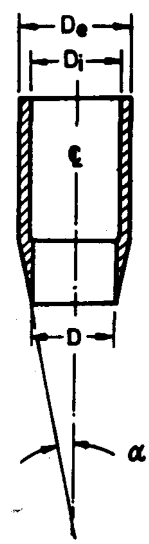


FIG. 1 DETAIL OF CUTTING EDGE

2.1.5 *Angle of the Cutting Edge* (α) — The angle made by the outer side of the cutting edge with the centre line of the sampling tube, as shown in Fig. 1.

2.1.6 *Gross Recovery Ratio* — The ratio of the gross length of the sample obtained in the sampling tube to the length of the sampler penetrating into the soil stratum being sampled.

2.1.7 *Effective Length of the Sampling Tube* — The length of the empty sampling tube, left after deducting from its complete length those portions which are used for fixing it with the sampler head and for accommodating the piston in its uppermost position.

3. EQUIPMENT

3.1 Boring Equipment — Any equipment capable of making a bore-hole of required depth and diameter, without disturbing the soil which is to be sampled.

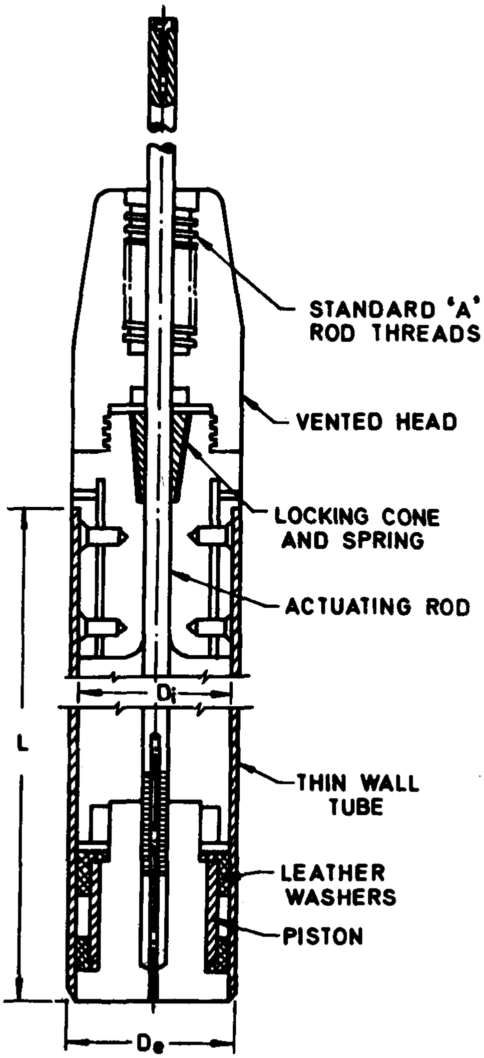


FIG. 2 STATIONARY PISTON SAMPLER

3.2 Sampler

3.2.1 The thin wall sampler with stationary piston consists of the sampling tube, sampler head and piston (*see* Fig. 2). The sampling tube must be connected with the sampler head tightly so as to work as a single unit. The piston should slide smoothly in the sampling tube maintaining vacuum.

3.2.2 *Sampling Tube* — The sampling tube shall be a cold drawn seamless pipe made of stainless steel, brass or mild steel chrome plated having the following dimensions (*see* Fig. 3).

Diameter at the cutting edge, D	74 ± 0.5 mm	49.5 ± 0.5 mm
Inside diameter, D_i	75 ± 0.5 mm	50 ± 0.5 mm
Thickness for steel	1.5 ± 0.1 mm	1.5 ± 0.1 mm
Thickness for brass	2.0 ± 0.1 mm	1.5 ± 0.1 mm
Angle of cutting edge (∝)	10 ± 1°	10 ± 1°
Thickness at the edge	0.2 ± 0.05 mm	0.2 ± 0.05 mm
Length, L	75 cm	60 cm

NOTE 1 — In the case of stiff clays or clays mixed with silt or fine sand, if necessary, the thickness of the sampling tube may be increased suitably with reference to Fig. 4, realizing that the increase in are ratio will increase the degree of disturbance of the soil sample.

NOTE 2 — The degree of distortion of the sampling tube should be checked by measuring the maximum and minimum values of the outside diameter with the help of the vernier calliper along the length of the tube. The difference between the maximum and minimum values of the diameter should not exceed 1.5 mm.

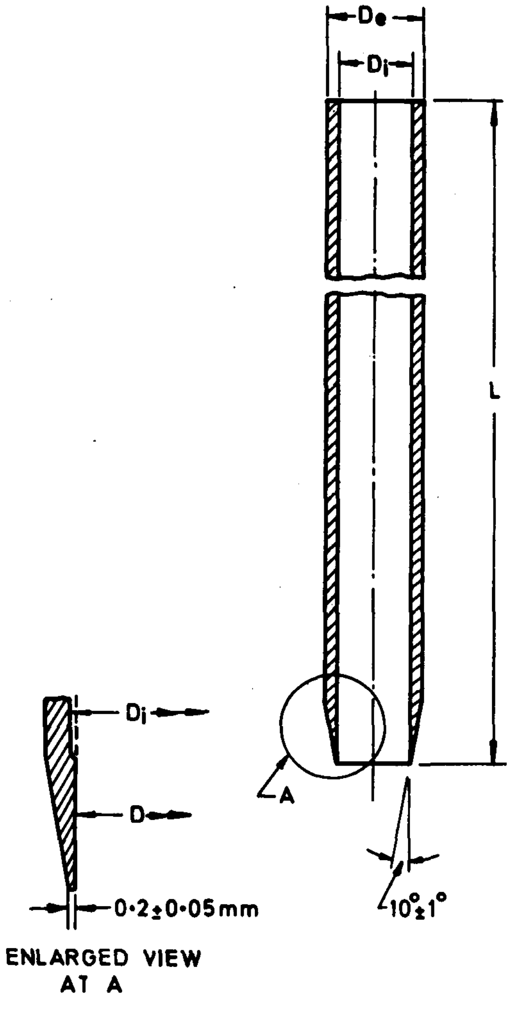


FIG. 3 DIMENSIONS OF SAMPLING TUBE

3.2.3 *Sampler Head* — The sampler head is connected tightly with a drill rod at its top and with a sampling tube at its lower end. It is installed with a locking device to allow movement of the piston rod in one direction only and a drain hole through which water is pushed away by the piston.

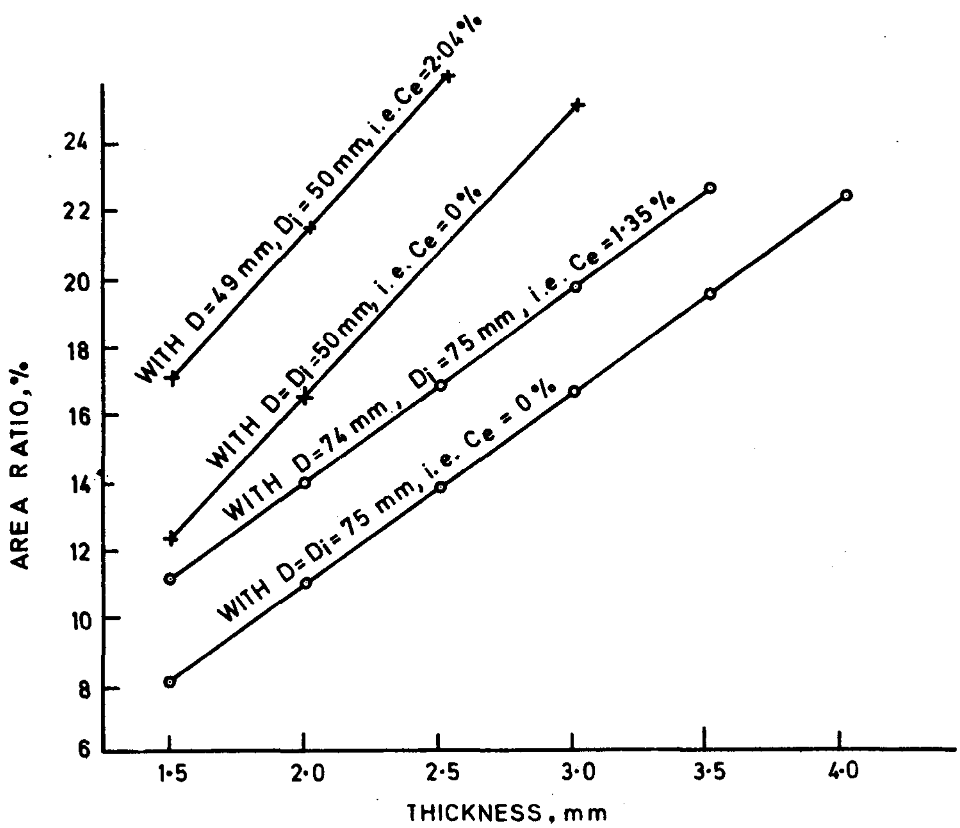


FIG. 4 VARIATION OF AREA RATIO WITH INSIDE CLEARANCE AND THICKNESS OF THE TUBE FOR SAMPLING TUBE OF INTERNAL DIAMETER OF 50 mm AND 75 mm

3.2.4 *Piston* — The piston, consisting of the piston base, leather packing and piston rod, is connected with piston extension rod to its upper end. The piston should be equipped with a ventilation arrangement to avoid build-up of negative pressure while the sampler is disconnected after sampling.

3.3 Rod

3.3.1 *Drill Rod* — The rod to transmit force to push down the sampler must be of any standard size having diameter not less than 40 mm.

3.3.2 *Piston Extension Rod* — In order to resist downward force applied to a piston while the sampling tube is being pushed into the ground, the piston rod, at its end outside the sampler, is connected to a steel member, known as piston extension (PE) rod, which has the same diameter as that of the piston rod. This rod is generally of 12 mm diameter and it operates inside the hollow drill rod. Joints in the piston extension rod are displaced about 15 cm from joints in the drill rods.

3.4 Locking of Piston Extension Rod — The mechanism shown in Fig. 5 or any other alternative may be used to provide a fixed support to the piston extension rod at the ground surface in order that the piston remains stationary when the sampling tube penetrates into the ground.

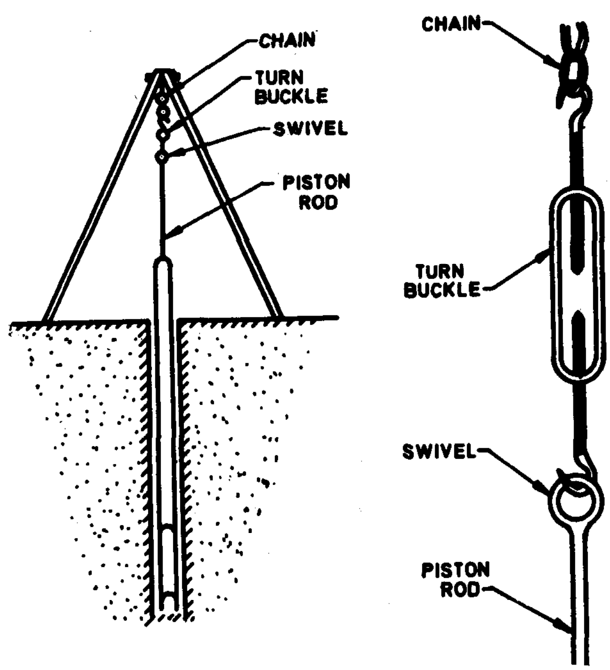


FIG. 5 SUPPORT OF THE PISTON EXTENSION ROD

3.5 Apparatus to Push a Sampling Tube — An apparatus having a hydraulic jack or working with compressed air or a mechanical jacking is required to provide the necessary force to push a sampling tube, quickly and avoiding shocks, into the soil which is to be sampled.

4. PROCEDURES

4.1 Boring and Cleaning of a Borehole — The borehole shall be made to a desired depth using a suitable method and ensuring that the soil at the bottom of the hole remains undisturbed. Casing pipes and/or bentonite mud may be used to avoid collapse of borehole walls. The cuttings of soil from the borehole shall be removed before sampling.

4.2 Sampling

4.2.1 *Inspection and Maintenance of Sampler* — The sampler shall be thoroughly inspected before use with particular reference to loosening of components, functioning of piston rod lock device and distortion of sampling tubes. The damaged parts shall be repaired or replaced before using the sampler. The outside diameter of the sampling tube shall be measured at cross-sections at distances of 30, 40 and 80 cm from the edge of the tube. The maximum and minimum inside diameters of the tube shall also be checked.

4.2.2 *Assembling of Sampler* — In assembling the sampler, close the ventilation arrangement of the piston, and check if the backward and forward movements of the piston inside the sampling tube are without obstruction. Connect it to the sampler head tightly using screws. The assembled sampler shall be stored properly so as to protect the edge of the sampling tube against damage.

4.2.3 The depth of the bottom of the casing, if used below ground level, and water level in the borehole shall be noted.

4.2.4 Sampling shall be done as soon as possible after the clean-out operation and shall not be done after an interval, for example, where a borehole has been cleaned out and left overnight.

4.2.5 *Lowering of the Sampler* — While lowering the sampler into the borehole, the piston is kept at its lowest point thus closing the lower end of the sampler and preventing the entry of any foreign matter into the sampler. The conical ball bearing catch, termed as piston rod lock in Fig. 2, prevents the piston rod from slipping downward with respect to the head of the sampler. To prevent upward movement of the piston as the sampler is lowered into the borehole, the piston rod has a short section of left-handed threads which engages a matching section of threads in the sampler head. By rotating the piston extension rod counter-clockwise, the rod is threaded into the sampler head and the piston is locked at the bottom of the sampler. The principle of this operation is explained by a simplified diagram in Fig. 6 A. When the sampler reaches the bottom of the borehole, hold the drill rod by a rod holder to prevent sinking of the sampler.

4.2.6 *Penetration of Sampling Tube* — After lowering the sampler up to the desired depth in the borehole, give several clockwise turns to the piston extension rod, so that the piston gets released from the sampler. Now fix the piston extension rod with the stationary tower, as shown in Fig. 5, so that the piston remains stationary at the level of the bottom of the borehole. Ensure that the tower which supports the piston extension rods is rigid, as any downward movement of the piston at the time of penetration of the sampling tube will cause over-compression of the soil sample. Next, by an apparatus mentioned in **3.5**, push the sampling tube into the soil for a length which is at least 90 percent of the effective sampling length of the tube, as explained in **2.1.7**. The principle of this operation is explained by a simplified diagram in Fig. 6B. The sampler should be made to penetrate quickly by a continuous action without giving shock to it. The rate of penetration should be preferably 10 to 15 cm per second. In case the penetration has to be stopped midway, record its depth. In case the soil becomes stiffer midway of penetration and the sampler cannot be pushed any more, do not push it by force but terminate sampling at that depth and record the same.

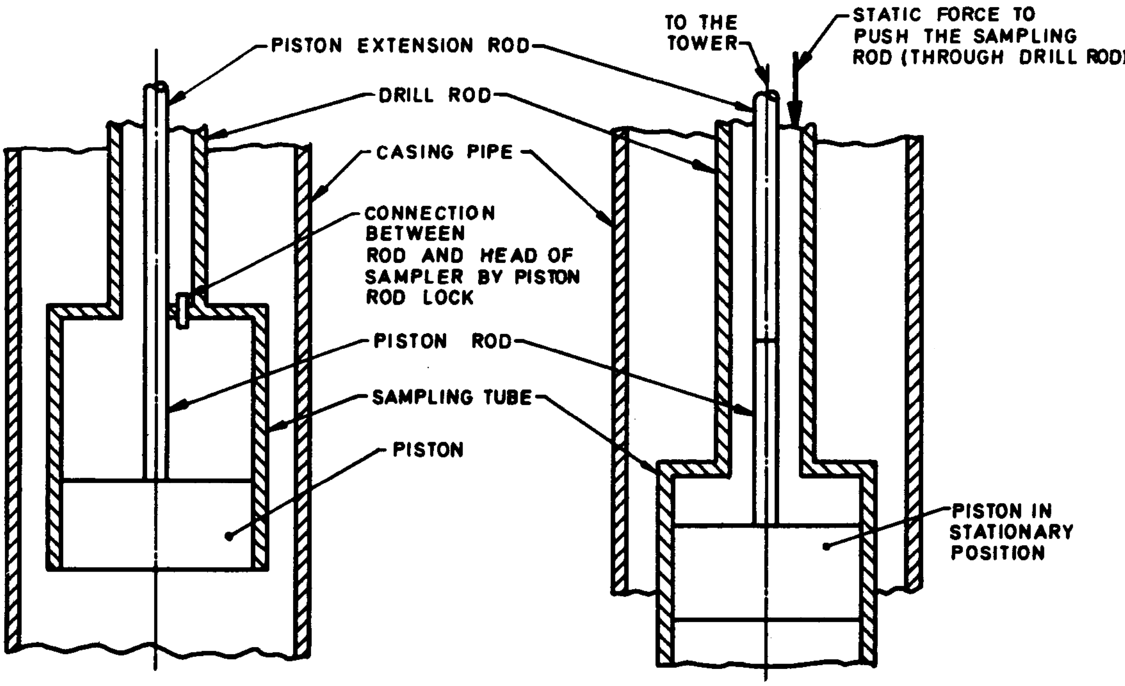
Measure the sampling length which is equal to the extent of displacement of the drill rod with respect to the bench mark on the drill rig.

4.2.6.1 The following precautions during penetration of the sampling tubes may also be taken :

a) There must not be any rotation of the sampling tube during downward movement and penetration.

b) The total penetration should not exceed the net length of the sampler.

4.2.7 *Lifting the Sampler* — The sampler should be torn at its bottom by giving rotation before lifting it out, taking sufficient care not to give any shock to the sampler. After completion of the driving it is advisable to wait for 10 to 20 minutes before starting the actual separation and withdrawal operation in order to allow full development of adhesion and friction between the sample and the sampling tube.



6 A During Towering of Sampler

6 B During Penetrating the Tube in soil

FIG. 6 SIMPLIFIED DIAGRAM EXPLAINING PRINCIPLE OF OPERATION OF PISTON SAMPLER WITH STATIONARY PISTON

4.2.8 *Disembarkment of the Sampler* — The sampler shall be disconnected after confirming whether the soil sample is secured or partly dropped out. Before extracting the piston from the sampling tube, loosen the ventilation arrangement in the piston, and be careful not to deform the tube or to give shock to the sample.

NOTE 1 — In very loose sand and silty soil below water table, provision of core catcher made of spring leaves at the cutting edge of the sampler, may be necessary to avoid loss of sample while lifting it (*see* Fig. 7).

NOTE 2 — For minimising the disturbances further, the thin wall piston sampler should be operated hydraulically, for which the kit may be modified to suit the principle of operation explained in Fig. 8. It confers two advantages, namely, (a) needs only one set of rods, that is, ordinary drill rods, and (b) at full stroke, a hole in the position rod releases the soil pressure and avoids overdriving.

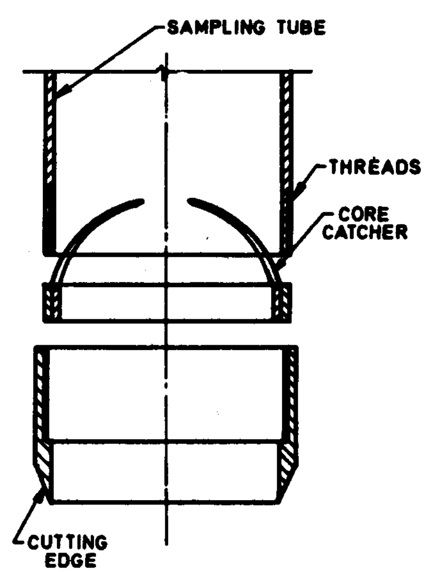


FIG. 7 FIXING CORE CATCHER ON THE INSIDE OF THE CUTTING EDGE OF THE SAMPLER

4.2.9 Samples shall be taken by repeating the sampling procedures at every change in stratum or at intervals not more than 1.5 m, whichever is less. Samples may be taken at lesser intevals if specified or found necessary; when in between vane shear test is conducted the interval be increased to 3 m.

4.3 Field Observations — Water table information, including ground water level, elevations at which the drilling water was lost, or elevations at which water under excess pressure was encountered, should be recorded on the field logs. Particular mention should be made if these occurred at the time of sampling. Water levels before and after insertion of the casing, where used, should be measured. In sandy soils, the level should be determined as the casing is pulled and then measured at least 30 minutes after the casing

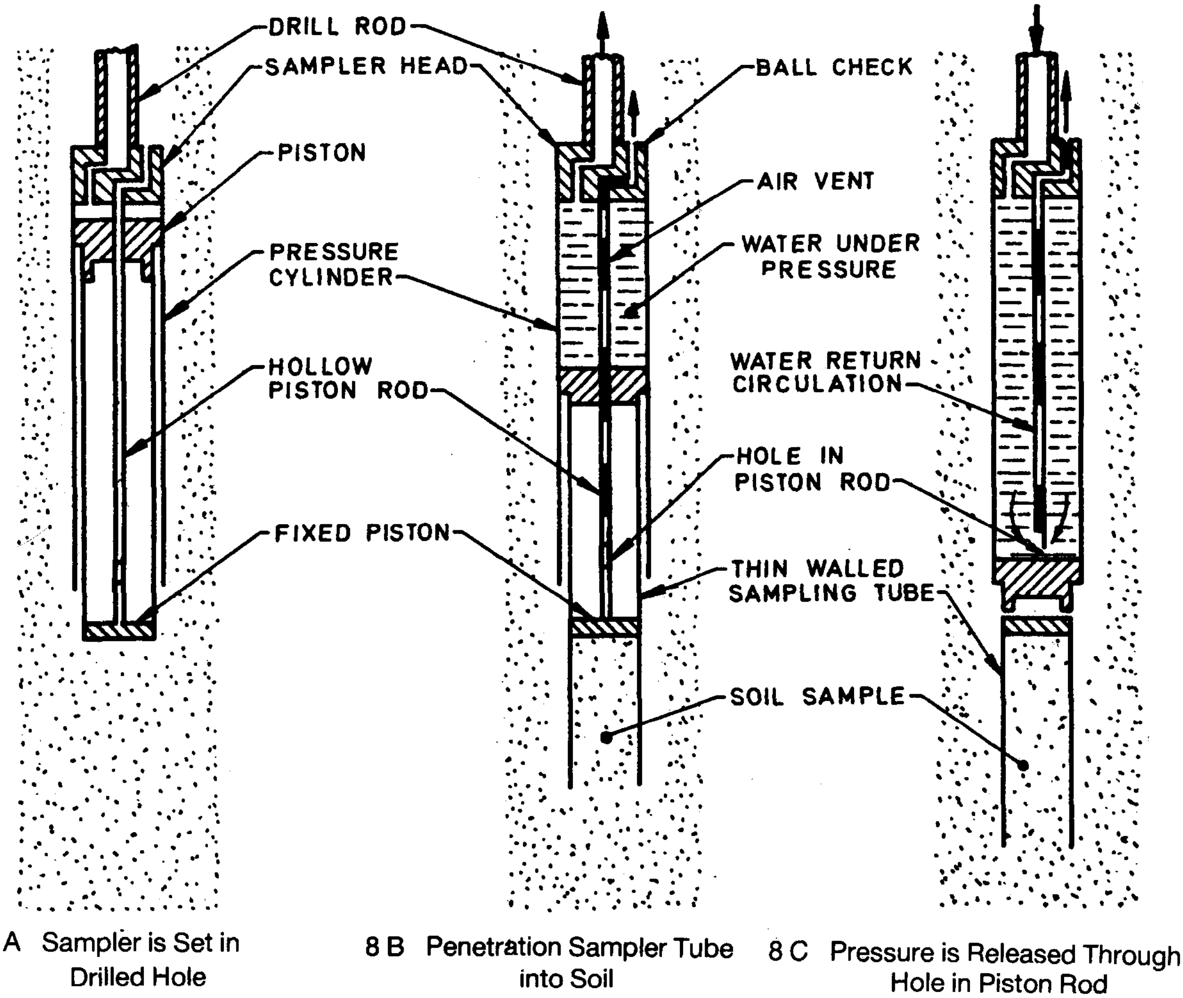


FIG. 8 DIAGRAMATIC SKETCH OF HYDRAULICALLY OPERATED PISTON ROD

is pulled; in silty soils at least 24 hours after the casing is pulled; in clays no accurate water level determination is possible unless pervious seams are present. However, the 24 hours level should also be recorded for clays. When drilling mud is used and the water level is desired, casing perforated at the lower end shall be lowered into the hole and the hole bailed down until all traces of drilling mud are removed from inside the casing. Ground water levels shall be determined after bailing at time intervals of 30 minutes and 24 hours.

4.4 Preparation for Shipment

4.4.1 Upon removal of the sampling tube, measure the length of the sample obtained in the sampling tube and from the knowledge of the depth of penetration of the sampler, calculate and record the gross recovery ratio as given in **2.1.6**. For a sample acceptable as undisturbed, the gross recovery ratio shall not be less than 95 percent.

4.4.2 Observe both ends of the sampler. If there are some soil fragments sedimented on the top of the sample, remove them and record it.

4.4.3 After reaming the soil at both ends of the tube up to the required extent, seal the ends of the sample with paraffin wax, etc, in order to prevent expansion or displacement of the sample or evaporation of moisture. Any wax that does not have appreciable shrinkage or does not permit evaporation of water from the sample shall be used. Micro-crystalline wax, if available, may be used in preference to paraffin wax. A mixture of paraffin wax and bees wax in the proportion 4:1 has also been found to be suitable. Thin discs of steel or brass that are slightly smaller than inside diameter of the tube are desirable for plugging both ends before sealing with wax. Suitable expanding packers may also be used.

The thickness of sealing shall not be less than 1 cm at the lower end of the sampler and not less than 3 cm at its top end.

4.4.4 Record the following on the outside of the sampling tube:

a) Name of the project,

b) Number of boring and that of sample,
c) Depth of sampling,
d) Date of sampling,
e) Top and/or bottom end of the sample.

These particulars may preferably be given on a table indicated in IS 1892 : 1980.

4.4.5 When samples are temporarily stored at the work site, be careful not to subject them to serious change of temperature, as by direct exposure to sun.

4.5 Transportation

4.5.1 Sufficient care should be taken not to give impact or serious change of temperature to the samples during transportation.

4.5.2 When the samples are being stored in the laboratory, confirm sufficient sealing on both ends of the samples and then place them in appropriate lots confirming the particulars recorded on the sampling tube. Store the samples in a dark and humid room.

4.6 Extraction of Sample

4.6.1 The sample should be extracted in a humid room shaded from the sunshine. Remove the seal at both the ends and extrude the sample by a suitable extruder continuously, so that there is minimum disturbance to the sample. Also, avoid any cause of bending or breakage of the sample by its own weight.

4.6.2 Examine the extruded sample very closely and locate the relatively disturbed and undisturbed portions of the sample so as to select an appropriate part of the sample which will suit the permissible degree of disturbance of sample for the desired test.

5. REPORT

5.1 All data obtained during the boring and sampling operations shall be recorded in the field and shall include the following :

a) Job identification;
b) Date of boring — start, finish;
c) Boring number and co-ordinates, if available;
d) Surface elevation, if available;
e) Drilling method;
f) Sample number and depth;
g) Method of advancing sampler, penetration and recovery ratio, and pressure required for pushing the sampler, if available;
h) Type and size of sampler;
j) Depth to water surface, to loss of water, to artesian head, and times at which readings were made;
k) Size of casing, depth of cased hole;
m) Description of soil based examination of soil removed from the ends of tubes;
n) Thickness of layer;
p) Weather conditions; and
q) Other observations and remarks.

These observations shall be recorded in a suitable proforma. A recommended proforma is given in Appendix A of IS 2132 : 1972.

Indian Standard

GUIDE FOR UNDISTURBED SAMPLING OF SANDS

0. FOREWORD

0.1 Undisturbed sampling of soil is a common feature in the field of soil mechanics and foundation engineering for finding *in-situ* characteristics of soils. In nature soils are found in variety and in different states of compactness and samplers have been designed to collect soils with least disturbance within practical limitations. Cohesionless soils are still problematic as far as undisturbed samling is concerned, and hence *in situ* testing is more common for these soils. Samples of coarse or loose sand readily fall out when ordinary sampling equipment with an open end is used. This guide has, therefore, been prepared to provide guidance in obtaining undisturbed samples in sand and covers two important techniques of undisturbed sampling in uncemented sands, namely, stationary piston sampling with drilling fluid circulation technique and compressed air technique. However, even with these methods the sample obtained may be considered to be only relatively undisturbed. These samples are generally used for the determination of *in-situ* density. It also briefly mentions the technique of rotary core drilling in cemented sands.

0.2 Special techniques in sampling of sand have not been covered in this guide as these techniques are costly and are employed on a limited scale in very special cases. Some of such techniques in use are mentioned in **0.2.1**.

0.2.1 Freezing or impregnation form special techniques beneficially used in sampling sands under favourable conditions. Freezing ensures solidification of the lower part of the sample to retain it in the sampler tube Solidification can also be achieved in some cases by impregnating a chemical such as kerosene at subzero temperature in place of drilling fluid, mixing alchohol with dry ice, emulsified asphalt and grout, etc.

0.3 In the formulation of this standard due weightage has been given to international co-ordination among the standards and practices prevailing in different countries in addition to relating it to the practices in this field in this country.

1. SCOPE

1.1 This standard covers the following two techniques of undisturbed sampling in uncemented sands :

a) Stationary piston sampling with drilling fluid circulation technique, and
b) Compressed air technique.

2. STATIONARY PISTON SAMPLING WITH DRILLING FLUID CIRCULATION TECHNIQUE

2.1 Equipment — Figure 1 illustrates the various components of equipment apart from drilling quipment and pump. There are as follows :

a) A thin walled sampler conforming to IS 2132 : 1972.
b) An airtight piston with a vacuum breaking arrangement.
c) The sampler head comprising the following:
   1) Suitable set screws to join the sampling tube to the head of the sampler,
   2) A vent hole, and
   3) A clamping arrangement to prevent the piston rod from falling down during lowering or withdrawal.
d) Sturdy and straight piston rods with 1 m joint to joint spacing.
e) A storage tank wherein drilling fluid (generally bentonite slurry) of required consistency is kept constantly agitated by paddle or any other suitable arrangement.
f) A tank to receive the efficient (drilling fluid) to separate the sand particles from the drilling fluid to allow for recirculation of the latter.

2.2 Description of Technique and Procedure of Sampling

2.2.1 *General* — In this method, partial vacuum is created above the sample while withdrawing the stationary piston sampler. The coating of drilling fluid at the shoe keeps the sand sample intact during withdrawal. Since the piston will be at the shoe of the sampler at the beginning of the sampling operation, no shavings can enter

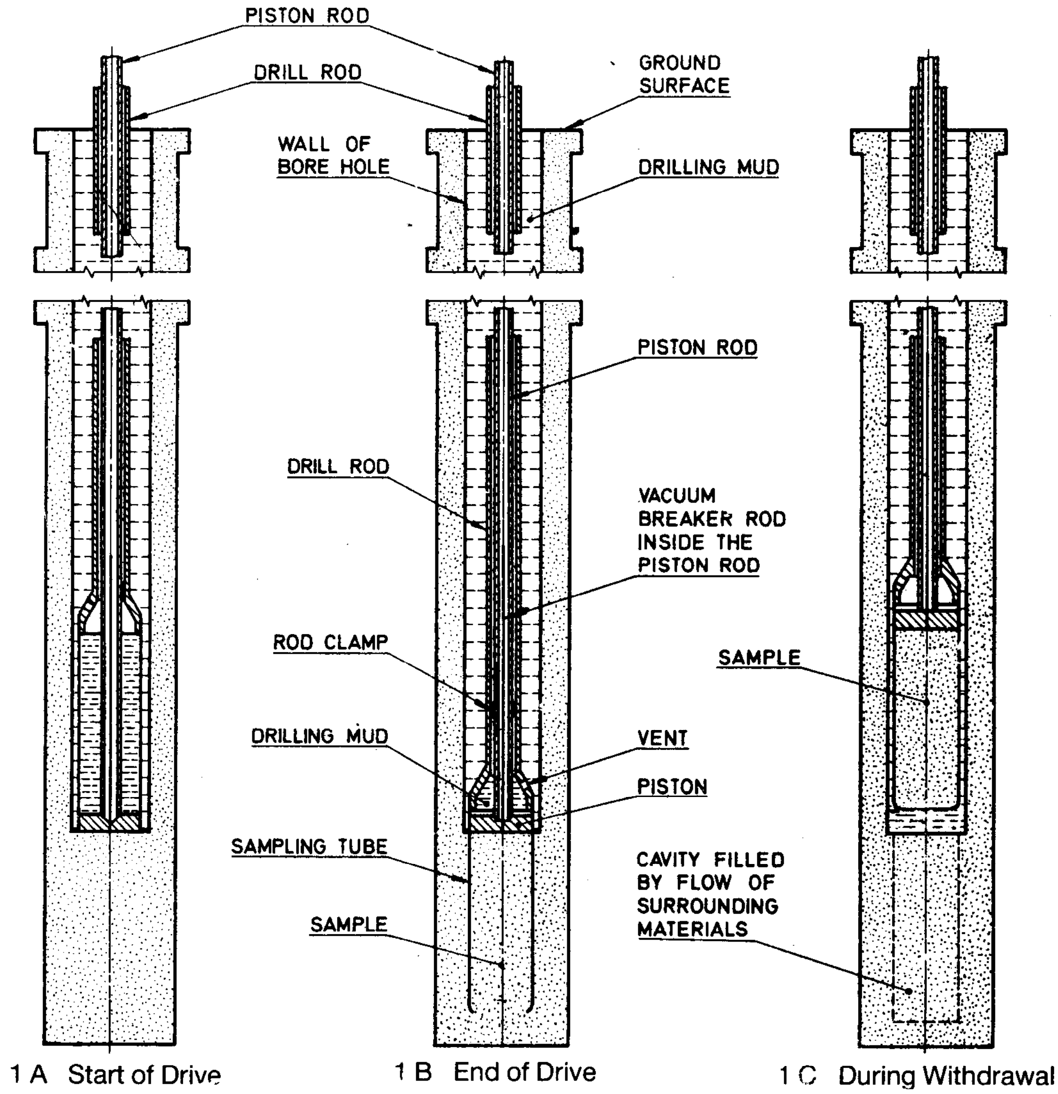


FIG. 1 CROSS SECTION (DIAGRAMATIC) THROUGH BORING DURING SAMPLER DRIVE AND WITHDRAWAL

the tube during sampling. The consistency of the drilling fluid shall depend on the grain size of sands, the relative density, and the position and condition of water table. For fine sand, a drilling fluid with a specific gravity of 1.05 will be satisfactory.

2.2.2 *Procedure of Sampling*

2.2.2.1 The bore hole shall be advanced with any suitable technique. It is preferable to use rotary drilling in combination with drilling fluid for advancement of bore hole; particularly, for deeper depths which limits the lengths of casing to the upper depths.

2.2.2.2 In case of rotary drilling using drilling fluid, the drilling fluid of required consistency shall be kept continuously agitated in a tank by paddle or any other arrangement. This fluid shall be circulated through a drill rod during drilling operation. It is advantageous to use fish tail bit for such drilling.

2.2.2.3 The outcoming fluid shall be collected in a separate tank and the sand particles allowed to settle down. The supernatant fluid shall then be used for recirculation.

2.2.3 *Sampling Technique*

2.2.3.1 Having advanced the drill hole, the sampler with the rod in extended position, shall be lowered. The drill rods and the piston rods help reaching down to the surface of contact where sampling is to be done. The piston rod shall be clamped to the drilling machine or tripod and the sampling tube shall be pushed continuously into the virgin soil.

2.2.3.2 Before withdrawal of the sample, it shall be given a rotary motion to shear the sample at the bottom of the tube. The piston shall be locked so that it does not move downwards while the sample is being cut. Both the drill rods and the piston rods shall be removed in stages.

2.2.3.3 Necessary precaution shall be taken to prevent the piston or piston rod from falling down. This shall be ensured by a suitable piston rod locking device such as a conical catch, which shall be checked to be in satisfactory working conditon prior to use.

3. COMPRESSED AIR TECHNIQUE

3.1 Equipment — The equipment is shown in Fig. 2. It shall have the following components apart from the drilling equipment and casing pipes (152 mm diameter)

a) Compressed air bell to house the sampler tube connected through a hose to a foot pump at the ground surface.
b) Sampler tube (63 mm diameter and 1.7 mm wall thickness).
c) A special head comprising the following:
   1) Set screws to connect the sampler head to sampling tube,
   2) Rubber sealing rings,
   3) Water exit ports,
   4) A rubber diaphragm valve,
   5) A relief valve,
   6) A bronze bushing,
   7) Special sealing ring (Angus type), and
   8) A steel head for the bell.
d) A removable spacer block.
e) Guide rod.
f) A socket block encasing guide rod along with a shackle to push the sampler tube.
g) Lifting cable.

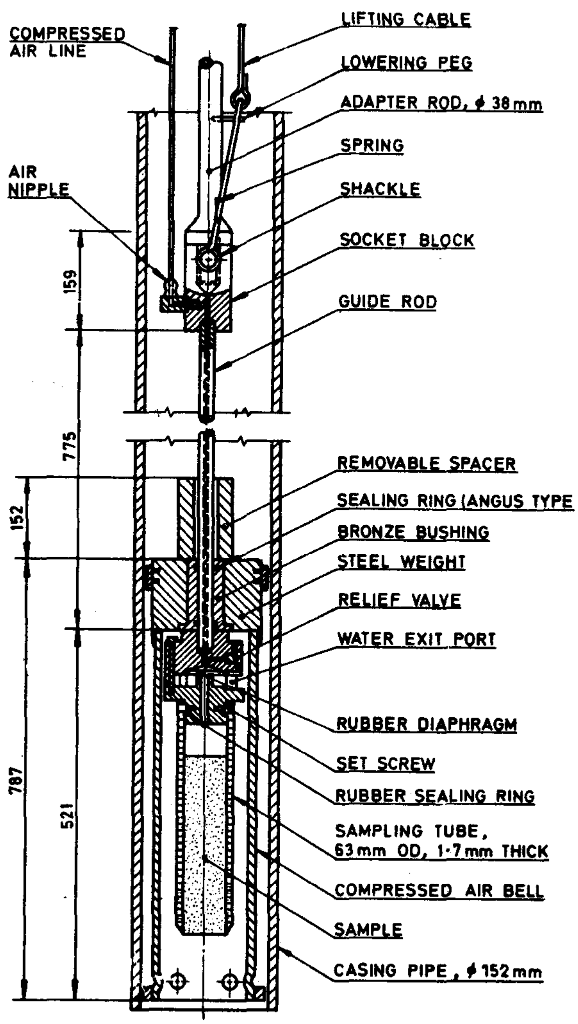


FIG. 2 GENERAL LAYOUT OF SAND SAMPLER WITH AUXILIARY BELL FOR COMPRESSED AIR

3.2 Description of Technique and Procedure of Sampling

3.2.1 *General* — In this technique compressed air is used to keep the ground water separated from the sample in order to avoid dispersion of sampled sand. This is done by withdrawing after sampling, the sampler tube into a bell where the ground water has been displaced by compressed air through a continuous pumping process. The depth of water in the drill holes govern the pressure of compressed air. The method is suitable for sampling sand under water table.

3.2.2 *Procedure of Sampling*

3.2.2.1 The drill hole shall be advanced with a suitable boring method using 152 mm diameter casing down to the depth of sampling.

3.2.2.2 Having reached the required depth, the sampler shall be pushed into soil by means of a drill rod and the spacer block and shackle arrangement. The spacer block above the bell limits the length of sampling stroke, thus, avoiding overdriving.

3.2.2.3 The drill rod shall then be withdrawn. Compressed air shall be forced into the bell by means of a foot pump. The air in turn pushes the diaphragm of the relief valve so as to maintain an excess pressure of 140 kN/m^2 (1.4 kgf/cm^2) thus closing the diaphragm check valve.

3.2.2.4 Having expelled the water in the bell, as indicated by the rising air bubbles, the sampler shall be withdrawn into the bell and the entire assembly raised to the surface by means of a cable. During raising of the assembly to the surface, water should be poured continuously to keep the drill hole full. The foot pump shall be continuously operated during withdrawal.

3.2.2.5 The spacer block above the bell shall then be removed so that the sampler is pushed out of the bell and sampling tube disconnected. A filter plug shall be placed in the lower end The suction created by check valve shall then be released and undisturbed sample obtained.

4. SAMPLING IN CEMENTED SANDS

4.1 Slow rotary technique using core barrels may be used to obtain undisturbed cores in cemented sand. If necessary, drilling fluid may be used during advancement for stabilization of the hole. In certain cases, where drilling is susceptible to cave in double tube core barrels may be used. If such cemented sands exist at shallow depths, preferably block samples may be obtained, by isolating a 200-300 mm square column of soil followed by covering it by a slightly larger hollow box open at top and bottom. The annular space between the rock and box shall then be filled by paraffin. The sample shall be trimmed by a spade and then covered at top and bottom also by paraffin so as to preserve its moisture.

Indian Standard

SPECIFICATION FOR MILD STEEL THIN WALLED SAMPLING TUBES AND SAMPLER HEADS

0. FOREWORD

0.1 The Indian Standards Institution has already published a series of standards on methods of testing soils. It has been recognised that reliable and comparable tests results can be obtained only with standard testing equipment capable of giving the desired level of accuracy. A series of Indian Standards covering specification of equipment used for testing soils are being formulated so as to encourage its development and manufacture in the country.

0.2 The equipment covered in this standard is used for carrying out undisturbed sampling of soils covered in IS 2132 : 1985.

0.3 For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 1960. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

1. SCOPE

1.1 This standard covers requirements of thin wall sampling tubes and sampler heads for *in-situ* sampling of soils, as required for open drive tube samplers.

2. TERMINOLOGY

2.1 For the purpose of this standard, definitions given in IS 2809 : 1972 shall apply.

3. MATERIALS

3.1 Material for the construction of sampling tubes shall be as given in Table 1.

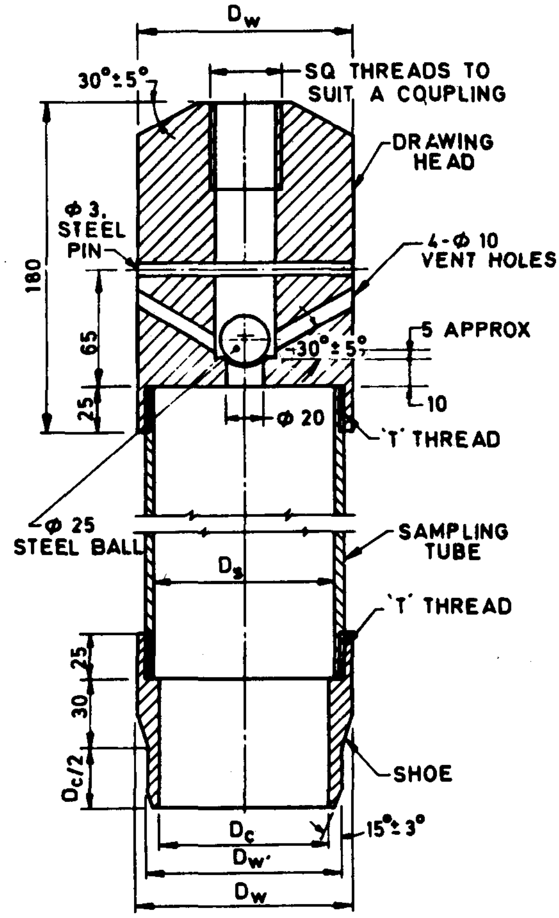
4. DIMENSIONS

4.1 There shall be 4 sizes, 40, 65, 80 and 100 mm based on internal diameter of the tube. The tolerance on all dimensions shall be ± 0.5 mm.

5. CONSTRUCTION

5.1 The sampling tubes, cutting shoes and sampling heads shall be made according to details given in Fig. 1. The length shall be as desired.

NOTE — The cutting shoes have been so designed that these give area ratio within 10 percent, and inside clearance 1 to 3 percent.



Size	40	65	80	100
Thread size *(T)*	M45 × 3	M75 × 3	M85 × 3	M110 × 3
Outermost dia of the shoes and sampler head (D_w)	50	77	94	115
Nominal dia of tube (D_s)	40	65	80	100
Outer dia of shoes $(D_{w'})$	41	66	82	103
Internal dia of shoes (D_c)	39	64	78	98

All dimensions in millimetres.

FIG. 1 DETAILS OF SAMPLING TUBE WITH DRIVING HEAD AND SHOE

TABLE 1 MATERIALS OF CONSTRUCTION FOR DIFFERENT PARTS OF THE SAMPLING TUBES

(Clause 3.1)

PART	MATERIAL	SPECIAL REQUIREMENT	CONFORMING TO INDIAN STANDARD
Tube	Mild steel	Smooth surface	Grade light of IS 1239 (Part 1) : 1979
Cutting shoes	Mild steel case hardened	45-50 *HRC* with smooth surface	IS 4432 : 1967
Sampling head	Mild steel		IS 226 : 1975

6. MARKING

6.1 The following information shall be clearly and indelibly marked on each component of the equipment :

a) The name of the manufacturer or his registered trade-mark or both, and

b) Size and length.

6.1.1 The equipment (each part) may also be marked with the ISI Certification Mark.

NOTE — The use of the ISI Certification Mark is governed by the provisions of the Indian Standards Institution (Certification Marks) Act, and the Rules and Regulations made thereunder. The ISI Mark on products covered by an Indian Standard conveys the assurance that they have been produced to comply with the requirements of that standard under a well-defined system of inspection, testing and quality control which is devised and supervised by ISI and operated by the producer. ISI marked products are also continuously checked by ISI for conformity to that standard as a further safeguard. Details of conditions, under which a licence for the use of the ISI Certification Mark may be granted to manufacturer or processors, may be obtained from the Indian Standards Institution.

Indian Standard

CODE OF PRACTICE FOR THIN-WALLED TUBE SAMPLING OF SOILS

(Second Revision)

0. FOREWORD

0.1 Undisturbed samples of soil are required for a number of soil test such as unconfined compression test, consolidation test, permeability test and triaxial compression test. It has been recognized that it is not practicable to obtain a truly undistrubed sample but if certain procedures and precautions are observed it is possible to get relatively undisturbed samples which may be considered sufficient keeping in view the nature of tests to be performed on these samples. This code deals with the method of obtaining such samples using thin walled tube samplers with sampler heads (with and without check valves).

0.1.1 This standard was first published in 1963 and revised in 1972. In this revision, requirements regarding specifications for sampling tubes have been reviewed based on indigenous availability taking into consideration the general practice in the country, the sampling tubes have been restricted to four sizes only. The detail specification of sampling tubes and sampler head are covered in separate Indian Standard.

0.2 In very loose saturated sandy and silty soils and clays the use of a piston sampler may often be necessary to secure a suitable undisturbed sample, the details of which are covered in IS 10108 : 1982.

1. SCOPE

1.1 This standard describes the method for obtaining relatively undisturbed cohesive and C — Ø soil samples suitable for laboratory tests, using a thinwalled metal tube.

2. TERMINOLOGY

2.1 For the purpose of this standard, the definitions given in IS 2809 1972 shall apply.

3. EQUIPMENT

3.1 Drilling Equipment — The equipment used shall provide a reasonably clean hole before insertion of the thin-walled tube, shall not disturb the soil to be sampled, and shall effect a rapid penetration of the tube into the soil to be sampled.

NOTE — Where casing is used, the equipment shall be capable of driving and removing the casing and shall include a pressure pump for clean-out operations. Where drilling fluid is used, a suitable mud pump is required. Where augers are used for clean-out purposes no special equipment other than that for sampling is generally requried. The hole may be cleaned with a bailer with a flap valve but this should not be used in sandy soils.

3.2 Sampler Head — It shall conform to IS 11594 : 1985.

3.3 Thin-Walled Tubes — It shall conform to IS 11594 : 1985. These may be of stainless steel and copper.

3.4 Sealing Material — Any wax that does not have appreciable shrinkage or does not permit evaporation of water from the sample shall be permitted. A mixture of paraffin wax and bees wax in the proportion 4 : 1 has also been found to be suitable.

3.5 Miscllaneous Items — Labels, data sheets, shipping containers, etc.

4. PROCEDURE

4.1 Driving the Casing — Where casing is used it shall not be driven below the sampling level, and casing pipe should be in such a way that it does not disturb the soil to be sampled.

4.2 Cleaning the Hole

4.2.1 The hole shall be cleaned to sampling elevation using whatever method is preferred that will ensure that the soil to be sampled is not disturbed. In saturated sandy and silty soils the drilling equipment should be withdrawn slowly to prevent loosening of the soil around the hole.

4.2.2 Where casing is used, the hole shall be cleaned out to the bottom or just below the casing. A clean-out auger should be used to clean the bottom of the hole, when necessary.

4.2.3 Bottom discharge bits shall not be permitted for clean-out purposes; side or upward discharge bits may be permitted.

4.2.4 The water level in the hole should be maintained at or above the ground water level, especially in soils that might be disturbed by the flow of ground water into the drill hole such as sandy and silty soils.

4.3 Obtaining Soil Sample

4.3.1 The depth of bottom of the casing, if used, below ground level and the water level in the bore hole should be noted.

4.3.2 Sampling shall be done as soon as possible after the clean-out operation and shall not be done after an interval, for example, where a hole has been cleaned-out and left overnight.

4.3.3 The assembled sampling tube should be lowered to the bottom of the hole, and the following information should be noted:

a) Depth of bottom of bore hole below ground level;
b) Amount of penetration of the sampling tube into the soil, under the combined weight of the tube and the rods; and
c) Water level in the bore hole.

4.3.4 The sampling tube shall then be pushed into the soil by a continuous and rapid motion. In no case the tube shall be pushed farther than the length provided for the sample. About 50 mm shall be allowed for cuttings and sludge. A clearance of 10 to 20 mm shall be allowed below the sampled head in the tube. The depth of penetration of the tube shall also be noted. Before pulling out the tube, at least 5 minutes shall be allowed to elapse after pushing the tube after which the tube shall be turned at least for two revolutions to shear the sample off at the bottom.

NOTE — In case the equipment used for SPT is also used for driving the sampling tube, then the length of penetration shall be limited to 50 blows.

4.3.5 Samples shall be taken, by repeating the sampling procedures, at every change in stratum or at intervals not more than 1.5 m, whichever is less. Samples may be taken at lesser intervals if specified or found necessary. The intervals be increased to 3 m if in between vane shear test or SPT is performed.

4.3.6 Field Observations — Water-table information including ground water level, elevations at which the drilling water was lost, or deviation at which water under excess pressure was encountered should be recorded on the field logs.

4.4 Preparation for Shipment

4.4.1 Upon removal of the sampling tube, the length of the sample in the tube and the length between the top of the tube and the top of the sample in the tube shall be measured and recorded.

4.4.2 The disturbed material in the upper end of the tube shall be completely removed before applying wax for sealing. The length and type of the sample so removed should be recorded.

4.4.3 The soil at the lower end of the tube shall be reamed to a distance of about 20 mm. After cleaning both ends shall be sealed with wax applied in a way that will prevent wax from entering the sample. Wax used for sealing should not be heated to more than a few degrees above its melting temperature. The empty space in the samplers, if any, should be filled with moist soil, saw dust etc, and the ends covered with tight fitting caps.

4.4.4 If it becomes necessary to keep the samples at the site for some time, they shall be kept in the shade. They should be kept over a bed of sand, jute bags, saw dust, etc, and covered over on top with similar material (sand, jute bags, saw dust, etc). The bed and top cover should be kept moist. Such bedding and top cover may also be provided at the time of shipment of the samplers with samples (*see* **4.4.3**).

4.5 Labelling and Shipping

4.5.1 Labels giving the following information should be affixed to the tubes :

a) Tube number,
b) Job designation,
c) Sample location,
d) Boring number,
e) Sample number,
f) Depth,
g) Penetration, and
h) Gross recovery ratio.

4.5.1.1 The tube and boring numbers should be marked in duplicate.

4.5.2 Duplicate markings of the boring number and sample number on a sheet which will not be affected by moisture should be enclosed inside the tube.

5. REPORT

5.1 All data obtained during the boring and sampling operations shall be recorded in the field as per details given in Appendix A.

APPENDIX A

(*Clause* 5.1)

PROFORMA FOR RECORD OF OBSERVATIONS DURING UNDISTURBED SAMPLING OF SOILS USING THIN-WALLED TUBE SAMPLES (*see* Note)

Name of Project :

a) **Drilling Details**	*OR*	b) **Trial Pit**
Bore hole No. and coordinates: Drilling method: Surface elevation at bore hole top:		Location: Dimensions: Elevation at top dimensions

Date of boring : Start __________ Finish __________

Details of casing, if used :

Name of driller :

c) **Observations of Water Levels in the Bore Holes**

1) Ground water level
2) Elevations at which drilling water was lost with the related to sampling
3) Elevations at which water under excess pressure was encountered with time related to sampling
4) Water level before insertion of casing if used
5) Water level after insertion of casing if used
6) Water level after pulling out of casing if used and possible
7) Whether drilling mud was used

d) **Sampling Operations**

1) Sampling tube No. 1 2 3 4 5 6 7 8
2) Sample No.
3) Method used for cleaning bottom of hole with date and time
4) Depth to bottom of casing below ground level
5) Date and time of sampling
6) Sampling details :
   i) Total lengths, L_s
   ii) Size
7) Level of water maintained in the bore hole
8) Depth to bottom of cleaned bore hole below GL
9) Level of water in the hole at the time of sampling
10) Amount of penetration of the tube under its weight and weight of rods
11) Method used for pushing the tube
12) Depth of penetration of the tube
13) Distance between top of tube and top of sample (measured after withdrawal), L_e
14) Whether soil sample in the tube was up to the cutting edge of the tube after withdrawal, if not how much within
15) Any evidence of slipping of the soil sample in the tube at the time of withdrawal
16) Thickness of soil sample removed from the cutting edge (bottom) end of the tube
17) Any disturbed material removed from the top end of the tube
18) Length of soil sample left in the tube
19) Weight of tube with soil sample left in the tube
20) Field description of soil, from soil removed from the ends of the tube (composition, condition, colour, structure, consistency, etc)
21) Remarks and special observations, if any

NOTE — This proforma has been made comprehensive to include all observations indicated in the code. The proforma may be modified to suit individual job conditons. Some of the items indicated in the proforma may not be needed when sampling from a open trial pit. In such a case the direction of sampling, horizontal or vertical should also be indicated.

SECTION 12

Soil Product

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Indian Standard

SPECIFICATION FOR SOIL BASED BLOCKS USED GENERAL BUILDING CONSTRUCTION

(First Revision)

0. FOREWORD

0.1 Development during the last two decades in the use of soil based blocks in different parts of the world and the experience which has been gained for nearly a decade in the field of construction in India hold out a great promise for the use of soil based blocks in general building construction, particularly in low-cost structures. Experience shows that most soils can be satisfactorily stabilized with cement-lime. It is, however, necessary to conduct comprehensive tests on soils in a laboratory in order to determine the optimum requirements to give the specified properties. While in general building construction soil based blocks may be used as a substitute for bricks, their use should be avoided in the case of isolated load bearing columns, piers and such other heavily loaded structures.

0.1.1 This standard was first published in 1960. Based on further studies conducted, this revision has been prepared. The principal modification is in respect of weathering test, which has been prescribed as per studies conducted at Indian Institute of Science, Bangalore. The revision now covers all types of soil based blocks.

1. SCOPE

1.1 This standard covers the requirements and test for soil based blocks for use in general building construction.

2. GENERAL

2.1 Soil based blocks shall be manufactured from a mixture of suitable soil and ordinarily Portland cement or lime Pozzolana mixture thoroughly mixed together, preferably in a mechanical mixer. The mixture is moulded and cast into blocks.

3. CLASSIFICATION

3.1 The blocks shall be of two classes, Class 20 and Class 30 (*see* **5.1**).

4. SIZES

4.1 There shall be three sizes of soil-cement blocks, the dimensions of which shall be as follows :

Length cm	*Breadth* cm	*Height* cm
19	9	9
19	9	4
29	19	9

4.2 The dimensions shall be tested in accordance with the procedure given in **4.2.1** and shall be within following limits per twenty blocks.

Block Size cm	*Length* cm	*Breadth* cm	*Height* cm
19 × 9× 9	372 to 388	174 to 186	174 to 186
19 × 9× 4	372 to 388	174 to 186	74 to 86
29 ×19×9	570 to 590	372 to 388	174 to 186

4.2.1 Twenty (more according to the size of stack) whole blocks shall be selected at random from the sample selected under **6**. All blisters, loose particles of clay and small projections shall be removed. They shall then be arranged upon a level surface successively in contact with each other and in a straight line. The overall length of the assembled blocks shall be measured with a steel tape or other suitable inextensible measures sufficiently long to measure the whole row at one stretch. Measurement by repeated application of short rule or measure shall not be permitted. If, for any reason, it is found impracticable to measure blocks in one row, the sample may be divided into rows of 10 blocks each, which shall be measured separately to the nearest millimetre. All these dimensions shall be added together.

4.3 Each block shall also have a frog one centimetre deep and 10 × 4 cm on one of its flat sides.

5. PHYSICAL REQUIREMENTS

5.1 Compressive Strength — The blocks when tested in accordance with the procedure laid down in IS 3495 (Part 1) : 1976 shall have a minimum average compressive strength of not less than 20 kgf/cm^2 for Class 20 and 30 kgf/cm^2 for Class 30.

5.1.1 The compressive strength of any individual block shall not fall below the minimum average compressive strength by more than 20 percent.

5.2 Water Absorption — The block when tested in accordance with the procedure laid down in IS 3495 (Part 2) : 1976, after immersion in cold water for 24 hours, an average water absorption shall not be more than 15 percent by weight.

5.3 Weathering — When tested in accordance with Appendix A, the maximum loss of weight shall not exceed 5 percent.

6. SAMPLING AND CRITERIA FOR CONFORMITY

6.1 Sampling and criteria for conformity of the blocks shall be done in accordance with the procedure laid down in IS 5454 : 1978.

7. MARKING

7.1 Each block shall be marked in the frog with the manufacturer's identification mark or initials.

7.1.1 The manufacturers may also use the ISI Certification Mark.

NOTE — The use of the ISI Certification Mark is governed by the provisions of the Indian Standards Institution (Certification Marks) Act and the Rules and Regulations made thereunder. The ISI Mark on products covered by an Indian Standard conveys the assurance that they have been produced to comply with the requirements of that standard under a well-defined system of inspection, testing and quality control which is devised and supervised by ISI and operated by the producer. ISI marked products are also continuously checked by ISI for conformity to that standard as a further safeguard. Details of conditions under which a licence for the use of the ISI Certification Mark may be granted to manufacturers or processors, may be obtained from the Indian Standards Institution.

APPENDIX A

(*Clause* 5.3)

WEATHERING TEST

A-1. PRINCIPLE

A-1.1 The parameters that need be simulated in the weathering test are the (i) rain drop diameter at impact (range in 2 mm for medium intensity and 4 mm for high intensity), (ii) maximum terminal velocity of 6.5 m/s at impact, and (iii) maximum intensity of rainfall, 15-30 mm/h.

A-2. TEST SPECIMENS

A-2.1 Three whole blocks shall be selected from the sample of blocks produced after carrying out the test for dimensional conformity. These blocks shall be designated as specimen A, B and C respectively.

A-3. SPRAY TEST

A-3.1 A set of spray non-rustable showers that can produce a hard spray all over the block should be used. The diameter of each shower is 10 cm with 36 holes of 2 mm diameter. A facility for providing a device pump to create a constant pressure of 1.5 ± 0.2 kgf/cm^2 should be available for this test.

A-4. PROCEDURE

A-4.1 The block to be tested is to be mounted on a test rig, such that only one face is exposed to shower and discharged water should find an exit without wetting the other faces or getting collected such that blocks get immersed. These showers are placed at a distance of 18 cm from the block and are arranged by the side, such that the complete face gets exposed. The period of exposure is limited to 2 hours and then the exposed surfaces are examined for possible pitting. The tests are carried out on at least 3 blocks. The limiting diameter of the pit formed is to be within 1 cm for passing this weathering test.

ANNEX A

LIST OF REFERRED INDIAN STANDARDS

IS 2 : 1960	Rules for rounding off numerical values *(revised)*
IS 226 : 1975	Structural steel (standard quality) *(fifth revision)*
IS 407 : 1981	Brass tubes for general purposes *(third revision)*
IS 460	Test sieves
(Part 1) : 1985	Wire cloth test sieves *(third revision)*
(Part 2) : 1985	Perforated plate test sieves *(third revision)*
IS 513 : 1986	Cold rolled low carbon steel sheets and strips *(third revision)*
IS 1239 (Part 1) : 1979	Mild steel tubes, tubulars and other wrought steel fittings: Part 1 Mild steel tubes *(fourth revision)*
IS 1433 : 1965	Beam scales *(revised)*
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IS 1501 (Part 1) : 1984	Method for Vickers hardness test for metallic materials: Part 1 HV 5 to HV 100 *(second revision)*
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IS 1875 : 1978	Carbon steel billets, blooms, slabs and bars for forgings *(fourth revision)*
IS 1888 : 1982	Method of load test on soils *(second revision)*
IS 1892 : 1979	Code of practice for subsurface investigation for foundations *(first revision)*
IS 1904 : 1986	Code of practice for design and construction of foundations in soils : General requirements *(third revision)*
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IS 2131 : 1981	Method for standard penetration test for soils *(first revision)*
IS 2132 : 1986	Code of practice for thin-walled tube sampling of soils *(second revision)*
IS 2720	Methods of tests for soils
(Part 1) : 1983	Preparation of dry soil samples for various tests *(second revision)*
(Part 2) : 1973	Determination of water content *(second revision)*
(Part 3) :	Determination of specific gravity
(Part 3/Sec 1) : 1980	Fine-grained soils *(first revision)*
(Part 3/Sec 2) : 1980	Fine, medium and coarse grained soils *(first revision)*
(Part 4) : 1985	Grain size analysis *(second revision)*
(Part 5) : 1985	Determination of liquid and plastic limits *(second revision)*
(Part 7) : 1980	Determination of water content — dry density relation using light compaction *(second revision)*
(Part 8) : 1983	Determination of water content — dry density relation using heavy compaction *(second revision)*
(Part 10) : 1973	Determination of unconfined compressive strength *(first revision)*
(Part 11) : 1971	Determination of the shear strength parameters of a specimen tested in unconsolidated undrained triaxial compression without the measurement of pore water pressure
(Part 13) : 1986	Direct shear test *(second revision)*
(Part 15) : 1986	Determination of consolidation properties *(first revision)*
(Part 16) : 1987	Laboratory determination of CBR *(second revision)*
(Part 17) : 1986	Laboratory determination of permeability *(first revision)*
(Part 22) : 1972	Determination of organic matter *(first revision)*
(Part 23) : 1976	Determination of calcium carbonate *(first revision)*
(Part 26) : 1987	Determination of *p*H values *(second revision)*
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(Part 29) : 1975	Determination of dry density of soils in-place, by the core cutter method *(first revision)*

(Part 33) : 1971	Determination of the density in-place by the ring and water replacement method
(Part 34) : 1972	Determination of density of soil in-place by rubber-balloon method
(Part 38) : 1976	Compaction control test (Hilf method)
(Part 39/Sec 1) : 1977	Direct shear test for soils containing gravel, Section 1 Laboratory test
(Part 39/Sec 2) : 1979	Direct shear test for soils containing gravel, Section 2 *In-situ* shear test
IS 2809 : 1972	Glossary of terms and symbols relating to soil engineering *(first revision)*
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IS 4432 : 1967	Case hardening steels
IS 4453 : 1980	Code of practice for subsurface exploration by pits, trenches, drifts and shafts *(first revision)*
IS 4968 (Part 1) : 1976	Method for subsurface sounding for soils : Part 1 Dynamic method using 50 mm cone without bentonite slurry *(first revision)*
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IS 5313 : 1980	Guide for core drilling observations *(first revision)*
IS 5454 : 1978	Methods for sampling of clay building bricks *(first revision)*
IS 5517 : 1978	Steels for hardening and tempering *(first revision)*
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IS 6403 : 1981	Code of practice for determination of bearing capacity of shallow foundations *(first revision)*
IS 6926 : 1973	Code of practice for diamond core drilling for site investigation for river valley projects
IS 6935 : 1973	Method of determination of water level in bore holes
IS 7746 : 1975	Code of practice for *in-situ* shear test on rock