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GUIDELINES FOR THE DESIGN OF HIGH EMBANKMENTS

(First Revision)

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GUIDELINES FOR THE DESIGN OF HIGH EMBANKMENTS

INTRODUCTION

IRC:75, “Guidelines for the Design of High Embankments” has been prepared to assist technical personnel in Highway engineering profession possessing a basic knowledge of Geotechnical Engineering to solve numerical problems of embankment design and to identify problems which will call for services of Geotechnical specialists. These will be of special interest and use to Engineers who have to build embankments in routine circumstances.

The guidelines deal with a wide spectrum of issues including general design considerations, sub surface and borrow area investigations, laboratory testing , stability analysis, settlement computation , quality control , construction alternatives, instrumentation etc. Detailed design procedures that are easily available in text books are not repeated. Earth Embankments are included but not embankments consisting of rock- fill.

IRC:75 was first published in 1979. Considering the new concepts developed in design of high embankments since the earlier publication it was decided to revise the design guidelines. A sub group was constituted by H4 committee with experts in the field of Road Embankment design .The convener of the subgroup was Mr. P.J. Rao, other members were, Mr. Sudhir Mathur , Mr. Guru Vittal, Ms.Minimol Korulla, Ms. Atasi Das , Mr. Saurabh Vyas , Mr. Atanu Adhikari and Ms. Anusha Nandavaram. After many deliberations in H-4 committee on the modifications incorporated in IRC:75 clauses, the draft document was submitted to HSS committee and the same was discussed during the meeting on 12th January 2015. The HSS committee recommended to place the document in council conducted in Bhubaneswar on 19th January 2015 with few suggestions for modifications. The subgroup modified the document as per the HSS recommendations and submitted the document to Council. On 19th January 2015, the document was approved by the council for printing.

The composition of the H-4 Committee is as given below:

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In brief, the major modifications incorporated in the present document compared to the earlier one are the following: (a) Design of high embankments for seismic conditions. (b) Liquefaction analysis and control measures. (c) Various types of ground improvement that are being widely followed are presented. (d) Instrumentation and monitoring is discussed

CHAPTER-1

GENERAL CONSIDERATIONS

1.1 Introduction

A structure of earth, gravel, light-weight material, etc. raised to prevent water from overflowing a level tract of land and to carry a roadway is called an embankment. For the purpose of guidelines, the terminology "high embankment" refers to a raised structure of height 6 m and above built for the purpose of road transportation. It consists of a series of compacted layers or lifts of suitable material placed on top of each other until the level of the subgrade surface is reached. The subgrade surface is the top of the embankment and the surface upon which the sub-base, base and pavement layers are placed.

Though the guidelines specifically indicates "high embankments" as those exceeding 6 m in height, embankments of any height less than 6 m founded on soft/compressible and/or loose strata are also considered as high embankments here. Design methodologies discussed herein apply for such cases also.

High embankments within the road networks are mostly joining to various types of structures such as bridges, vehicular/pedestrian underpasses, etc. apart from the requirement of geometric alignment due to right-of-way constraints, subsoil issues, etc.

1.2 General Considerations in Design

1.2.1 Most problems of embankment design and construction can be divided into three categories:

a) **Routine cases** such as embankments constructed over firm or reasonably favourable ground, using sand, gravel and other approved suitable fill materials.

b) **Special cases** where the difficult ground extends over a limited length; soft foundation layer/loose stratum exist for a shallow depth; or the embankment fill material is relatively un-favorable such as clays or organic material.

c) **Exceptional cases** in which embankments are routed over long distances on marine clays, tidal swamps, peats, creeks, floodplains, etc., where conditions could be critical in causing instability, and post-construction settlement might assume serious proportions and/or strata is expansive in nature exhibiting swelling characteristics/liquefaction potential due to the occurrence of loose soil at or up to deeper depths.

1.2.2 In all the above situations, failure of embankments generally takes place by one of the following modes:

i) Slip circle failure through the slope or through slope and base;

ii) Block sliding over weak soil strata in the foundation;

iii) Plastic squeezing;

iv) Liquefaction induced failure of embankment and foundation soil;

v) Excessive and uneven settlement of embankment and foundation soil;

vi) Erosion of embankment;

vii) Instability and scouring in vicinity to water bodies/rivers/ponds;

viii) Collapse due to inadequate drainage;

ix) Overtopping and subsequent washout by flood water.

A design cannot be considered as complete unless safety against failure by all the above modes is ensured. However, before actually embarking on design of the embankment, a designer must give due consideration to "engineering" and "economic" factors involved in design.

1.3 Engineering Consideration in Design

1.3.1 Each earth embankment is unique by itself since engineering considerations which determine the design of embankments are different for each situation. Some of the specific engineering considerations are:

i) *Foundation conditions*

Foundation conditions differ from site to site. The nature of the foundation material has a significant influence on the design of the embankment. For example, embankments resting on hard or favorable ground need to be analyzed essentially for slope failure. On the other hand, embankments resting on soft ground have to be analyzed not merely for slope stability, but also for base stability and anticipated settlement.

Also, if a thick layer of weak clay is sandwiched between stronger layers; a wedge failure across the weak layer will be more likely and design procedure must be tailored accordingly. An inclined hard stratum at shallow depth may indicate slippage along the stratum slope.

It is, therefore, essential that the soil profile below a proposed embankment should be investigated carefully and the physical and engineering properties of the subsoil be determined properly by in-situ and/or laboratory tests.

In case of embankments, bearing capacity problems assume significance if the embankment is resting on soft clay. The consideration for evaluating bearing capacity and settlement has been dealt with in the publication IRC:113 "Guidelines for the Design and Construction of Geosynthetic Reinforced Embankments on Soft Sub-soils".

ii) *Materials available at site*

A highway embankment, like all civil engineering structures is founded on earth. The only difference is that the structure also consists of earth fill. While building embankments, one has to consider, for economic reasons to use fill material available near the site. The variables that the designer can control are the water content of fill material during compaction, the amount of compaction, rate of loading and type of compaction. Choice of placement variables determines the density and structure of the compacted fill and in turn determines the engineering properties of the compacted fill.

iii) *Other considerations*

In addition to the above, several other parameters like climatic conditions, seismic effect, nature of loads, acceptable performance criterion such as global stability, settlements, bearing capacity and the time available for construction should be considered for design.

1.3.2 When designing a highway embankment over difficult foundation conditions, the designer may have to consider one or more of the following solutions:

- Ground Improvement by replacement of poor sub soil
- Provision of proper surface and subsurface drainage
- Provision of suitable erosion control measures
- Construction of embankment with berms
- Use of light weight material for subsoil/fill material for embankment construction
- Chemical stabilization
- Stage construction
- Preloading
- Sub soil stabilization by Prefabricated Vertical Drains (PVDs) or stone columns
- Basal reinforcement or Basal mattress
- Dynamic compaction
- Displacement of weak subsoil by surcharge weight or blasting
- Embankment fill supported on piles
- Pile Supported basal reinforced embankments
- Mitigation measures for expansive soil with swelling characteristics
- Mitigation measures for liquefiable soils

If one or more of the above solutions do not make the high embankment construction viable, then alternatives such as viaducts, relocation of alignment, etc. may be considered.

1.3.3 The different solutions should be evaluated objectively at the planning stage itself keeping in view the construction and maintenance costs, ecological and environmental effects, time available for construction, availability of fill soil and the right of way limitations. This can be possible only if the initial investigations recognize the various possibilities and consider all the pros and cons before a final decision has been taken about the final route alignment.

1.4 In the present guidelines, attempt has been made to address the designer's queries in a concise form within the chapters themselves.

Chapter 2: "Geotechnical Investigations" provides guidance on the selection of the field and laboratory investigations including geophysical investigations with advantages and limitations, which are necessary and appropriate for the design and construction of the high embankment.

Chapter 3: "Stability Analysis and Seismic Considerations" focuses on slope stability analyses taking into account earthquake forces and evaluation for liquefaction potential and corresponding design inputs and related considerations.

Chapter 4: "Settlement Analysis" addresses design inputs and major considerations for computation of settlement.

Chapter 5: "Ground Improvement" covers various methods in brief.

Chapter 6: "Instrumentation and Monitoring of Embankment" on soft soil deals with the instrumentation techniques available for the monitoring of embankment constructed on soft foundation strata.

1.5 Complete design and construction of highway embankment requires supplementary inputs as indicated below. For details of these requirements reference may be made to the documents mentioned therein.

- Selection and testing of fill materials, compaction and quality control-Ref. MORTH: Specifications for Roads and Bridge works Section 900
- Erosion control of slopes covered in IRC:56 "Recommended Practices for Treatment of Embankment and Roadside Slopes for Erosion Control" (First Revision). Also refer MORTH Section 300
- IRC:34 "Recommendations for Road Construction in Areas Affected by Water Logging, Flooding and/or Salt Infestation".
- IRC:SP:42 "Guidelines on Road Drainage".

1.6 Unconventional materials beyond the range specified by MORTH are increasingly coming into use, during the past few years, in the construction of highway embankments. Such materials include waste of mines of different ores and also fly ash from different thermal power plants etc., in all such cases it is desirable to take the advice of Geotechnical Engineer.

CHAPTER 2

GEOTECHNICAL INVESTIGATIONS

2.0 This chapter contains information related to soil exploration planning, methods and techniques including use of test pits, test borings, penetrometers and geophysical methods. Also presented is information on methods of sampling, measuring in situ properties of soil and rock, and field measurements. This chapter also covers laboratory test procedures, typical test properties, and the application of test results to design and construction of high embankments.

2.1 Planning For Field Investigations

The initial phase of field investigations shall consist of detailed review of project details, geological conditions at the site and in its general environs. This should include a desk study of available data including aerial photography, and field reconnaissance. The information obtained should encompass the following and should be used as a guide in planning the exploration.

Project details

- Plan and profile drawing of the project along with general cross-sections.
- The maximum height of the embankment.

Site specific details

- The nature, thickness and variation of soil strata along length of embankment.
- Procuring representative samples for assessing the physical properties of the soil strata encountered.
- The seasonal variations in ground water table and their possible effects on the soil parameters
- Properties including range of values of shear strength and compressibility of soil layers.

2.2 Exploration Phases

Project geotechnical exploration can generally have four phases:

i) Reconnaissance/feasibility exploration;

ii) Preliminary exploration;

iii) Detailed/final exploration; and

iv) Construction/post construction phases.

Additional exploration may be required during construction. Frequently, all preconstruction phases are combined into a single exploration effort.

Further, the geotechnical exploration includes embankment foundation or subsurface investigation, embankment fill materials or soils and materials investigation.

i) *Reconnaissance/feasibility*

Reconnaissance includes a review of available topographic and geologic information, aerial photographs, data from previous investigations, and site examination. Geophysical methods are applicable in special cases. Reconnaissance/feasibility frequently reveal difficulties which may be expected later in exploration phases and assists in determining the type, number and locations of borings.

ii) *Preliminary exploration*

This may include borings to recover samples for identification tests only.

iii) *Detailed exploration*

This phase normally includes borings, disturbed and undisturbed sampling for laboratory testing, standard penetration resistances, and other in situ measurements. At critical sites it may also include test pits, piezometer measurements, permeability test, other in-situ tests like pressure meter etc.

iv) *Construction/post construction phases*

Further evaluation of embankment foundation conditions may be required during the construction phase. Monitoring of the site or structure may be necessary throughout the construction and post construction phases.

2.3 Embankment Foundation or Subsurface Investigation

The objective of subsurface investigation is to determine the suitability of the soil or rock for the embankment foundation. Soil borings are the most common method of subsurface exploration in the field. Guidance may be taken from the following Codal provisions.

i) IS 1892 – Code of Practice for Subsurface Investigation for Foundations may be utilized for guidance regarding investigation and collection of data.

ii) Tests on soils shall be conducted in accordance with the relevant part of IS 2720 – Methods of Test for Soils. The tests on undisturbed samples shall be conducted as far as possible at simulated field conditions to get realistic values.

iii) IS 1498 – Classification and Identification of Soils for General Engineering Purposes. May be referred to the extent required for the purpose of embankment design. The data from subsoil investigation shall be plotted in a tabular form along with the subsoil profile.

A. *Boring methods.*

A common method of exploring sub surface conditions is by drilling exploratory borings along the proposed alignment. Shallow borings can be made with light weight hand operated augers while the deep borings can be made by adopting power driven rotary drilling or wash boring or percussion drilling depending on the sub soil conditions and requirements. The diameter of casing shall not be less than 150 mm diameter for boring up to the level of rock

B. *Boring depth.*

The boring depth is controlled to a great degree by the characteristics and sequence of the subsurface materials encountered.

C. *Boring layout.*

As a general guideline, for bridge approaches, where the extent of the high embankment is limited to 1 km or less, a minimum of three boreholes shall be drilled; where the extent is more than 1 km, one borehole per kilometer shall suffice in general conditions. For soft/ incompressible and/or loose subsurface (as noted in Tables 2.1 and 2.2), the intermediate spacing of boreholes within a kilometer shall be left to the judgment of the Engineer-in-charge.

General guidelines while deciding on the boring layout are given below:

1) Provide three borings in staggered way (BH1, BH2 and BH3 in **Fig. 2.1**) across the Critical section. Where detailed settlement, stability, or seepage analyses are required, include a minimum of two borings to obtain undisturbed samples of critical section for cohesive soil. Undisturbed samples may be collected as required for the tests

2) Boreholes to be so located to obtain geological profile in transverse direction.

For design of high embankment the following general layout shall be followed additionally where subsoil conditions differ, criteria for depth of boring are given in general termination criteria.

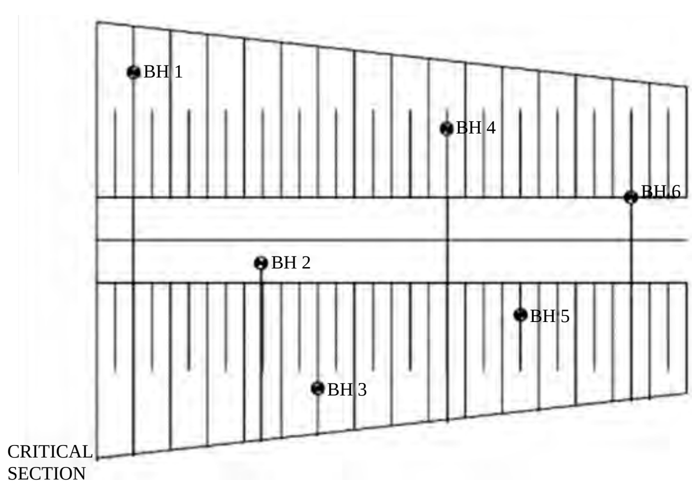


Fig. 2.1 General Boring Layout for High Embankments

Termination Criteria:

Depth of borehole shall normally be twice the height of embankment from the existing ground level. The boreholes shall be terminated at a lesser depth if rock is encountered or refusal (N-value > 50) is met for two consecutive values of SPT conducted at an interval of 1.5 m.

Further guidelines are provided below on site investigation while deciding on the termination depth of the boring.

i) The soil investigation shall start from 0.5 m depth from existing ground level. This will enable to collect soil samples from top and determine their properties.

ii) **Unsuitable Foundation Strata:** All borings shall extend through unsuitable foundation strata, such as unconsolidated fill; peat; highly organic materials; soft, fine-grained soils; and loose, coarse-grained soils to reach hard or compact materials of suitable bearing capacity, which may extend to a depth beyond twice the height of embankment

iii) **Fine-Grained Strata:** Extend borings in potentially compressible fine-grained strata of great thickness to a depth where stress from superposed load is low that consolidation of lower layers will not significantly influence surface settlement.

iv) **Compact Soils:** Where stiff or compact soils are encountered at shallow depths, extend boring(s) through this material to a depth where the presence of an underlying weaker stratum cannot affect stability or settlement.

Method of Sampling From Borings

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

The number and type of samples to be collected/recovered depend on the stratification and material encountered.

a) **Disturbed samples:** These are the hand, auger and wash samples and are primarily used for identification and soil classification tests. Take representative disturbed samples at vertical intervals of not less than 1.5 m and at every change in strata.

b) **Undisturbed samples:** These are collected in thin walled (also called Shelby tube) sampling tube and are taken primarily for laboratory strength and compressibility tests and in those cases where the in-place properties of the soil must be studied. Normally, sampling (disturbed and undisturbed) shall be carried out alternatively at an interval of 1.5 m or change of strata whichever is earlier.

Points to be noted for soil sampling:

- Undisturbed samples should comply with the following criteria: they should contain no visible distortion of strata, or cracks or softening of materials; specific recovery ratio (length of undisturbed sample recovered divided by length of sampling push) should exceed 95 percent; and they should be taken with a sampler with an area ratio (annular cross-sectional area of sampling tube divided by full area of outside diameter of sampler) less than 15 percent.
- Undisturbed samples in cohesive soil strata may be obtained such a way that there is at least one representative undisturbed sample in each boring for each 3 m depth.
- Relatively undisturbed cohesive and C-φ sample can be recovered by using Open tube thin walled sampler (IS: 2132). Detailed specifications of the sampler can be obtained from IS: 11594.

Additional cautions include the following:

1) **Caving:** Use casing or viscous drilling fluid to advance borehole, if there is danger of caving. If groundwater measurements are planned, drilling fluid should be of the revert type.
2) **Above Groundwater Table:** When sampling above groundwater table, borehole shall be maintained dry whenever possible.
3) **Below Groundwater Table:** When sampling below groundwater table, borehole shall be maintained full of water or drilling fluid during cleanout, during sampling and sample withdrawal, and while removing cleanout tools. Where continuous samples are required, casing should remain full for the entire drilling and sampling operation.
4) **Soft or Loose Soil:** Collection of sample in the same sampling tube shall be avoided, when the sample is collected from a soft or loose soil directly below a stiff or compact soil in the same tube should be avoided. Discontinue driving of sample tube when a sudden decrease in resistance occurs.

2.3.1 *Characterization of Sub-soil (Embankment Foundation) by In-situ Tests:*

Several in-situ tests define the sub-soil and obtain direct measurements of soil properties and geotechnical parameters. The common tests include: Standard Penetration (SPT), Static Cone Penetration Test (SCPT), Pressuremeter Test (PMT), and Vane Shear Test (VST). Each test applies different loading schemes to measure the corresponding soil response in an attempt to evaluate material characteristics, such as strength and/or stiffness. **Fig. 2.2** depicts these various devices. Brief descriptions of these tests are given in the subsequent sections and details are provided in relevant standards.

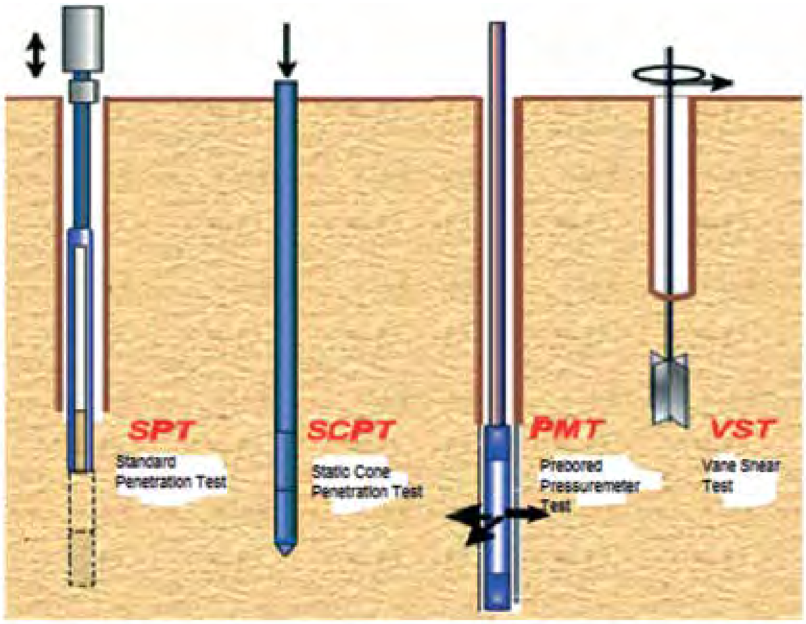


Fig. 2.2 Schematic Representation of various In-situ Tests

2.3.1.1 Penetration resistance tests

The most common test is the Standard Penetration Test (SPT) which measures resistance to the penetration of a standard sampler in borings. The method is rapid and when tests are properly conducted in the field, they yield useful data, although there are many factors which can affect the results. A more controlled test is the cone penetrometer test in which a cone shaped tip is jacked from the surface of the ground to provide a continuous resistance record.

i) Standard Penetration Test (S.P.T) (IS: 2131).

This test provides an indirect measure of shear strength and is especially suitable when undistributed samples cannot be extracted from cohesion less strata. Detailed test procedure and interpretation of SPT N–value can be referred from IS 2131.

ii) Static Cone Penetration Test (SCPT) (IS: 4968).

This field test enables continuous exploration throughout the stratum. SCPT is advantageous as compared to SPT especially in soils where in-situ soil density is likely to alter due to boring process. Detailed method is explained in IS: 4968.

Static Cone Penetration Test equipment, hydraulically operated is usually capable of conducting test up to 30 m depth as per site conditions.

The SPT N-value and SCPT-value obtained from the field exploration (SPT and SCPT tests respectively) provides a ready indication of the relative firmness of the strata and broad correlations are included in Tables 2.1 and 2.2.

Table 2.1 N vs. Compactness

SPT N-value	Compactness
0-4	Very loose
4-10	Loose
10-30	Medium
30-50	Dense
>50	Very Dense

Table 2.2 N and SCPT Value (kPa) vs. Compactness

Consistency	SPT N-value	Unconfined compressive strength (kPa)	SCPT value (kPa) (according to correlation given by Akca in 2003)
Very soft	0-2	<25	0 – 400
Soft	2-4	25-50	400 – 800
Medium	4-8	50-100	800 – 1600
Stiff	8-15	100-200	1600-3000
Very stiff	15-30	200-400	3000-6000
Hard	>30	>400	> 60000

Values given in Table 2.2 are indicative (refer IRC-113, Table:2)

2.3.1.2 In-Situ vane shear test (IS: 4434)

Vane shear apparatus is used for measuring the undrained shearing resistance and the sensitivity of the soft deposits of clay. Such tests are generally carried out in the bore hole after extracting an undisturbed sample. Equipment is also available with which vane shear test may be run on soft soils without making boring. If the soil contains thin layers or laminations of sand or dense silt, the results obtained by vane shear test may be misleading.

2.3.1.3 Pressuremeter test (ASTM D4719)

Pressure meter test is an in-situ test conducted in a size NX borehole (76 mm). It was developed to measure the strength and deformability of soil and rock. The test is conducted in a pre bored hole with a diameter that is between 1.03 and 1.2 times the nominal diameter of the probe.

2.3.1.4 Flat Dilatometer test (DMT) (ASTM D6635)

In addition to the listed methods, measurements can also be made in the field by using the Flat Dilatometer Test (DMT) wherein the dilatometer blade is advanced into the ground using a cone penetrometer or with a drill rig at a rate of 2cm/sec. The results obtained can be interpreted in terms of various soil parameters for application in various fields of civil engineering like computing settlement of shallow foundation, axially loaded pile, laterally loaded pile and pile groups, detection of slip surfaces in over consolidated clay slopes and determining liquefaction susceptibility at a particular site.

2.3.1.5 Geophysical exploration (IS: 1892)

Geophysical testing can be used as part of the initial site exploration to provide supplementary information to the data collected by other means (i.e., borings, test pits, geologic surveys, etc.). Geophysical testing can be used for establishing stratification of subsurface materials, the profile of the top of bedrock, the depth of groundwater, the boundaries of various types of soil deposits, the presence and depth of voids, buried pipes, and existing foundations. However, data from geophysical testing should always be correlated with information from the direct methods of exploration already discussed.

2.3.1.5.1 Types of geophysical tests

There are different types of geophysical in-situ tests that can be used to obtain stratigraphic information from which engineering properties can be estimated. Annexure 2.3 provides a summary of the various geophysical methods that are currently in practice and can be used to economic advantage. A general discussion regarding the major test methods listed in Annexure 2.3 is presented below, with particular emphasis on potential applications to highway engineering.

i) *Seismic methods*

These methods are becoming increasingly popular for geotechnical engineering practice because they have the potential to provide data regarding the compression and shear wave velocities of the subsurface materials. The shear wave velocity is directly related to small-strain material stiffness which in turn is often correlated to compressive strength and soil/rock type. These techniques are often used for assessing the vertical stiffness profile in a soil deposit and for assessing the location at depth of the interface between soil and rock. Seismic refraction method involves measurement of time of arrival of the initial ground motion generated by the energy source while the seismic reflection method involves measurement of the energy arrival after the initial ground motion.

ii) *Electrical Resistivity methods*

These methods are usually used to locate voids or locally distinct materials. Electrical resistivity methods provide qualitative information only and are usually part of a two- or three-phased exploration program.

iii) *Gravity and Magnetic methods*

These methods are similar to electrical methods, except that they rely on correlations between the potential gravitational and/or magnetic influence of voids and subsurface anomalies and measured differences in the earth's micro-gravitational and/or magnetic fields, rather than on changes in electrical fields. These methods provide measurements at specific points unlike seismic and electrical methods that provide measurements over large areas.

iv) Borehole methods

Downhole geophysical methods provide reliable indications of a wide range of soil properties. For example, downhole/crosshole methods provide reliable measures of shear wave velocity.

As indicated previously, shear wave velocity is directly related to small-strain stiffness and is correlated to strength and soil/rock type. Although downhole logging methods have seen little use in highway construction, they have been the mainstay for deep geologic characterization in oil exploration. The principal advantage of downhole logging is the ability to obtain several different geophysical tests/indicators by "stringing" these tools together in a deep borehole.

Near surface nuclear methods have been used for several years for compaction control of fills in the field. Through careful calibration, it is possible to assess the moisture content and density of compacted soils reliably. These methods have been widely adopted as reliable quantitative methods

2.3.1.5.2 Advantages and limitations of geophysical tests

As with the other methods of exploration, geophysical testing offers some advantages and some limitations that should be considered before these techniques are recommended for a specific application. These are summarized as follows:

2.3.1.5.2.1 Advantages of geophysical tests

i) Many geophysical tests are non-invasive. Therefore, such tests offer significant benefits in cases where conventional drilling, testing, and sampling are difficult (e.g., deposits of gravel, talus deposits, etc.) or where potentially contaminated soils may occur in the subsurface.

ii) In general, geophysical testing can cover a relatively large geographical area thereby providing the opportunity to characterize large areas with relatively few tests. Geophysical testing is particularly well-suited to projects that have large longitudinal extent such as new highway construction.

iii) Geophysical measurements are used to assess the properties of soil and rock at very small strains.

2.3.1.5.2.2 Limitations of geophysical tests

i) Most methods work best for situations in which there is a large difference in the property being measured between adjacent subsurface units. In seismic methods, it is difficult to develop good stratigraphic profiling if the general stratigraphy consists of hard material overlying soft material.

ii) Each geophysical method has limitations that may be associated with equipment, signal noise, unfavorable site and subsurface conditions, and processing constraints.

iii) Results can be non-unique and are generally interpreted qualitatively. Therefore useful results can be obtained only through analyses performed by a geotechnical specialist experienced with the particular testing method.

iv) Specialized and more electronically sophisticated equipment is required as compared to the more conventional subsurface exploration tools thus rendering it expensive in the Indian scenario.

2.3.1.5.2.3 Examples of uses of geophysical tests

The following are a few examples where geophysical testing could be used on highway projects to compliment conventional exploration.

i) Highly Variable Subsurface Conditions: In several geologic settings, the subsurface conditions along a transportation corridor may be expected to be variable. This variability could be from underlying karsts development above limestone; alluvial deposits, including buried terrace gravels, across a wide floodplain; buried boulders in a talus slope, etc. For these cases, conventional exploration techniques may be very difficult and if “refusal” is encountered at certain depth, there is a strong likelihood that different materials could underlie the hard strata region. Development of a preliminary subsurface characterization profile by using geophysical testing could prove advantageous in designing future focused explorations.

ii) Regional Studies: Along a transportation corridor it may be necessary to assess the depth to (and through) rippable rock or highly cemented caliche. Alternative alignments may or may not be possible, but the cost implications may be significant. Therefore, it is important to obtain a profile related to rock/soil stiffness. Geophysical testing is a logical consideration for this application as a precursor to invasive explorations.

iii) Settlement Sensitive Structures: The prior two examples related to cases where the geophysical testing served as the front-end of a multi-phase project. In the case where a settlement-sensitive structure is to be founded on deposits of sands, knowledge of the in-situ modulus of the sand deposit is critical. After the characteristics of the site are assessed, it may be helpful to quantify the deformation modulus by the use of geophysical testing at the specific foundation site.

2.3.2 *Characterization of Sub-soil by Laboratory Tests:*

Suitable selection of appropriate laboratory tests on the collected samples is of utmost importance. The following tests will normally be included in the test schedule.

i)	Sieve Analysis/Hydrometer	:	IS: 2720 (Part IV)
ii)	Natural Moisture Content/Bulk/ Dry Density	:	IS: 2720 (Part II)
iii)	Specific Gravity	:	IS: 2720 (Part III)
iv)	Liquid Limit/Plastic Limit/Plasticity Index	:	IS: 2720 (Part V)
v)	Direct Shear Test (for non-cohesive soils)	:	IS: 2720 (Part XIII)
vi)	Unconfined Compressive Strength Test (Cohesive Soils)	:	IS: 2720 (Part X)

vii)	Tri-axial Test (Cohesive Soils)*	:	IS: 2720 (Parts XI and XII)
viii)	Consolidation Tests (Cohesive soils below water table)	:	IS: 2720 (Part XV)
ix)	Chemical Analysis on Soil Samples	:	IS: 2720 & IS 3025

* The selection of the particular type of Tri-axial Test is discussed in Chapter 3.

2.4 Embankment Fill Material Investigation

For reaches involving new embankment construction, it is important to ensure that (a) each layer of embankment is constructed with select materials of approved borrow areas in layers of specified thickness and that necessary compaction is achieved before placing the subsequent layer; (b) suitable construction methodology is adopted on expansive clay stretches; and (c) all sources of materials (being used in embankments) have undergone full range of tests for compliance as per specifications.

2.4.1 *Field Investigation*

Detailed borrow material survey shall be conducted at closest lead distances, identify select ones at an interval of 5 km on both sides of the alignment, then dig test/trial pits for sample testing for suitability. Normally, test pits are dug up to 1 m to 2 m depth and disturbed samples are taken for testing.

2.4.2 *Laboratory Tests*

There are three basic requirements for a compacted embankment, namely:

a) Adequate shear strength.

b) Good drainability

c) Limited settlement within the body of the embankment.

The following tests will normally be included in the test schedule.

i)	Sieve Analysis	:	IS 2720 (Part-IV)
ii)	Atterberg's Limits	:	IS 2720 (Part-V)
iii)	Compaction Test (Modified Proctor Test)	:	IS 2720 (Part-VIII)
iv)	CBR at Single/Three energy level*	:	IS 2720 (Part-XVI)
v)	Free Swell Index (if LL>50%)	:	IS 2720 (Part-XL)
vi)	Shear Parameters (Direct Shear test/ Triaxial shear test)	:	IS 2720 (Part-XI, XII and XIII)
vii)	Permeability Test	:	IS 2720 (Part-XVII)

Details of shear tests required for stability analysis are presented in Chapter 3 and consolidations tests required for settlement calculations are presented in Chapter 4.

*CBR tests usually get conducted at 3 different energy levels corresponding to 10, 30 and 65 blows. During tests, normally 3 specimens of about 7 kg are compacted, so that their compacted densities may range from 95% to 100%.

2.5 Reporting and Presentation of Data

The results of reconnaissance, field and laboratory investigation should be consolidated in to a well-knit report. The record of findings and recommendations, if any, may be presented in the form of written text, graphs, Figures and tables, as appropriate for different types of data and findings.

Information and data to be contained in the report should include general location map, pertinent geological information on reconnaissance observations, sub-soil profile (**Fig. 2.3**) boring logs and summary of sub-soil properties (**Fig. 2.4**) graphs and tables related to laboratory investigations, results of borrow area investigations (**Fig. 2.5**) and recommendations, if any.

State----------------------

N. H. No. -----------------

Section-------------------

Location-----------------

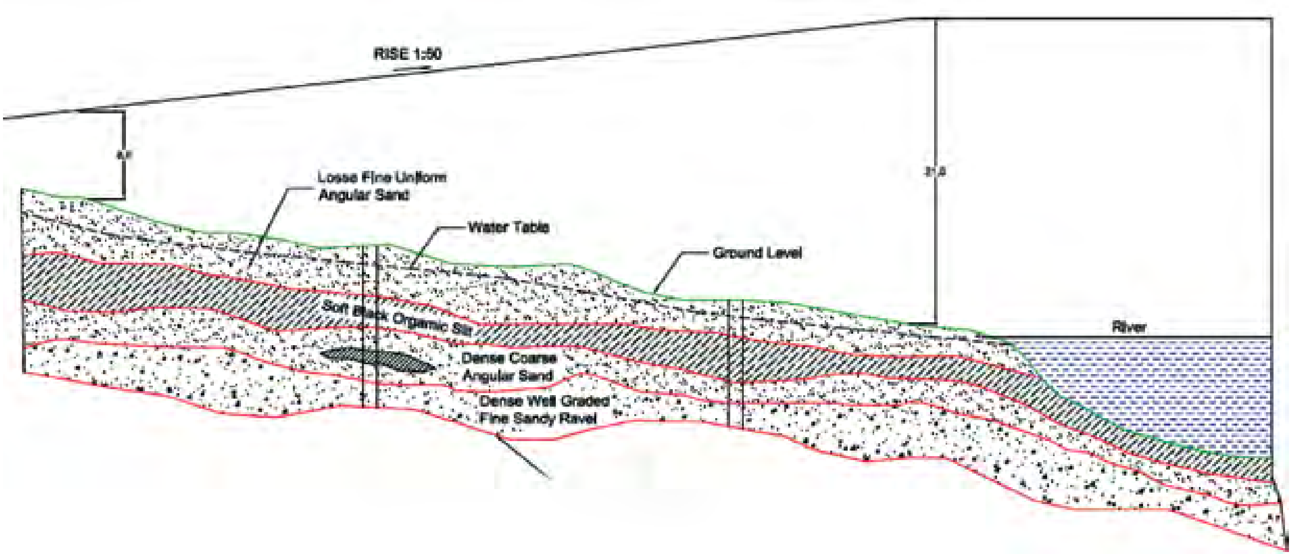


Fig. 2.3 Longitudinal Profile of the Subsoil

Name of Project………….. Section Location……………

State…………………………… Bore Hole No…………………

Name of Road…………….. Date of Boring………………

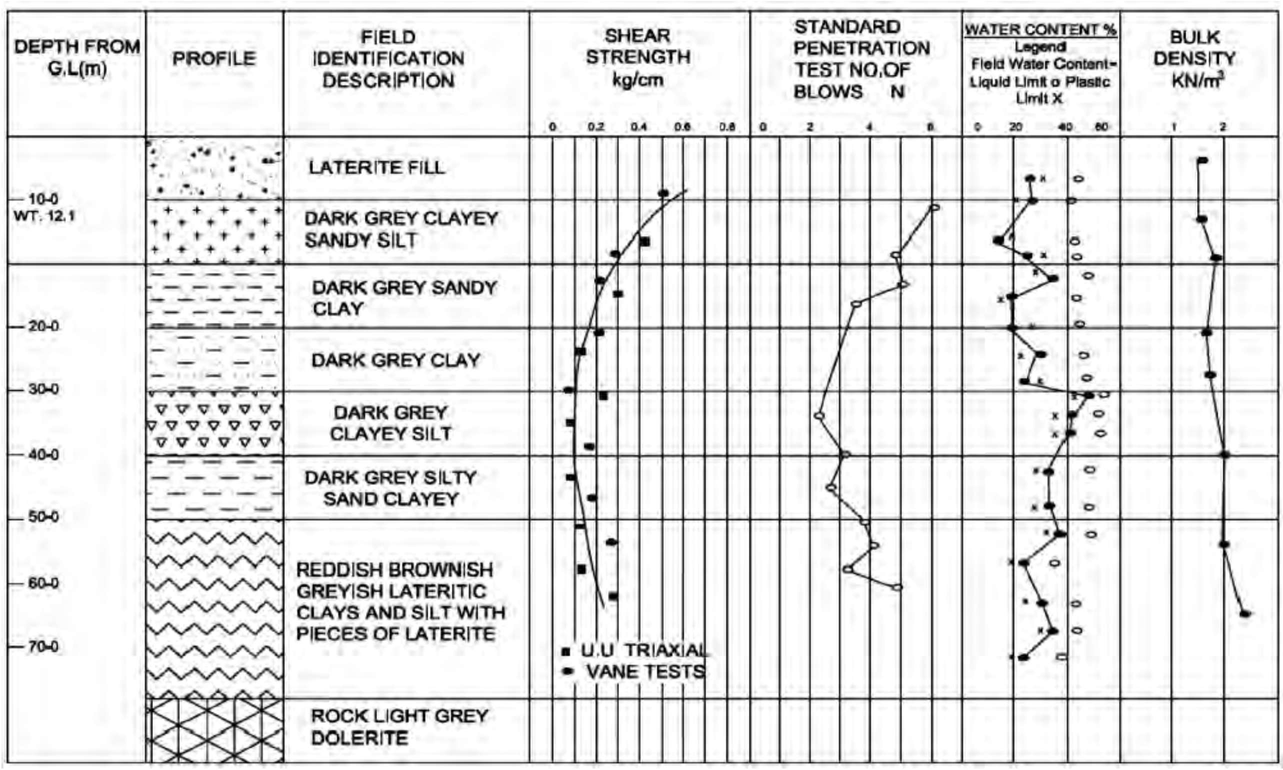


Fig. 2.4 Boring log and Summary of Sub-Soil Properties

Location of Borrow area
With reference to Index Map.......
Depth of Water table.......

Name of Road...........
Section...........
Location of Embankment.............

Sample No.	Depth R.L. of Sample	Field Description	Particle Size Analysis				Atterberg Limits			Standard Proctor Test		Specific Gravity	Unconsolidated undrained triaxial test at 95%	
			Gravel above 2 mm%	Sand .06 to 2 mm%	Silt .002 to .06 mm%	Clay Below .002 mm%	L.L.%	P.L. %	P.I. %	Density kg/m³	OMC %		C_{uu} kPa kg/cm²	ϕ_{uu} Degrees

Fig. 2.5 Results of borrow area investigation

ANNEXURE 2.1

REQUIREMENTS FOR ADEQUATE SOIL DESCRIPTION

			Sand and gravel	Hardpan	Glacial Till	Inorganic silt	Loess	Modified Loess	Clay	Lake Marl	Marl	Organic silt	Organic Clay	Peat
General Information from field examination		Color	0	0	0	0	0	0	0	0	0	0	0	0
		Odour[2]										0	0	0
		Texture and structure[3]		0		0	0	0	0	0	0	0	0	0
		Dilatancy[4]				0	0	0		0	0	0		
		Grain Properties[5]	0	0	0									0
		Plasticity				0	0	0	0	0	0	0		
		Dry Strength[6]				0	0	0	0	0	0	0		
Results of classification tests	Intact Samples[1]	Natural water content, w		0	0	0	0	0	0	0	0	0	0	0
		Natural void ratio[7], e	0		0		0	0						
		Unconfined Compressive Strength, q_u		0	0	0	0	0	0	0	0	0	0	
		Sensitivity[8], S_t				0		0		0	0	0	0	
	Representative samples	Unit weight of solid constituents, γ_s		0	0	0	0	0	0	0	0	0	0	
		Maximum void ratio[9], e_{max}	0		0		0	0						
		Minimum void ratio[9], e_{min}	0		0		0	0						
		Liquid Limit, L_w				0	0	0	0	0	0	0	0	
		Plastic Limit[10], P_w				0	0	0	0	0	0	0	0	
		Shrinkage Limit, S_w						0	0	0	0	0	0	
		Mechanical Analysis[11]	0		0	0	0	0		0	0	0		
		Carbonate Content[12]			0	0	0	0	0	0	0	0	0	
		Organic Matter Content[13]								0	0	0	0	

1. If no undisturbed or tube samples are obtained, use the spoon samples.
2. If the odour is faint, heat the sample slightly. This intensifies the odour.
3. Describe appearance of fresh fracture of intact sample (granular, dull, smooth, and glossy). Then rub small quantity of soil between the fingers, and describe sensation (floury, smooth, gritty, and sharp). If large specimens break up readily into smaller fragments, describe appearance of walls of cracks (dull, slicken sided) and average spacing of cracks.

4. Perform shaking test. Describe results (conspicuous, week, none), depending on intensity of phenomenon observed.

5. Describe shape (angular, sub angular, sub rounded, rounded, well rounded) and mineralogical characteristics of macroscopic soil particles only. Mineralogical characteristics include type of rocks and minerals represented among the grains so far as they can be discerned by inspection under the hand lens. Describe rock fragments (fresh, slightly weathered, or thoroughly decomposed; hard or friable). If a sand contains mica flakes, indicate mica content (slightly, moderately, or very micaceous). In connection with peat, the term grain properties refer to the type and state of preservation of the predominant visible remnants of plants such as fibres, twigs or leaves.

6. Crush dry fragment between fingers, and indicate hardness (very low, low, medium, high, and very high).

7. If no undisturbed samples have been obtained, substitute results of standard penetration test or equivalent.

8. Applies only to clay and fine silt at a water content above the plastic limit.

9. e_{min} is the void ratio of the soil in its densest state, usually achieved by packing the soil into a container by means of a combination of static pressure and vibration.

10. In addition to numerical value of P_w state whether threads were tough, firm, medium, or weak.

11. Present results either in from of semi-logarithmic graph, or else by numerical values of D_{10} and $U= D_{60}/D_{10}$ accompanied by adjectives indicating the type of grain-size grading.

12. Calcium carbonate content can be detected by moistening the dry material with dilute HCI. Describe results of test (strong, weak, or no effervescence).

13. To determine presence of organic matter, determine, L_w first in fresh state and then after drying in over at 108°C. describe results of test (highly or slightly organic).

14. Add to data on texture a description of general appearance, structure, and degree of cohesiveness of chunks in fresh state and after soaking in water.

15. Add to data on texture a description of the macroscopic features of the loess, such as diameter and spacing of root holes.

Notes:

1. Table borrowed from “Foundation Engineering” by peck, Hanson and Thornburn.

2. The symbol “0” indicates the particular property that is relevant to the particular type of soil.

ANNEXURE 2.2

IMPORTANT INSTRUCTIONS FOR OBTAINING UNDISTURBED SAMPLES:

a) When sampling above ground water table, maintain bore hole dry, whenever possible. When sampling below ground water table, maintain bore hole full of water or drilling fluid during cleaning out, sampling, sample withdrawal and while removing clean out tools. If necessary, this should be accomplished by positive inflow at ground surface.

b) *Cleaning of bore hole*

   i) Use jet auger that deflects the flow of water or drilling fluid upward. Downward nor sideward jetting is not permitted when cleaning below casing. Cleaning with jet bits that direct the flow downward or sideward is permitted within the casing but should not be done within four inches of the intended top of samples. The last 100 mm are cleaned out with a jet auger that deflects water or drilling fluid upward.

   ii) When casing is extruded to sample depth, all soil must be cleaned out up to the casing trip at least and preferably 100 mm below the trip. Where continuous samples are taken, allow for 100 mm when determining final depth of casing before sampling. Coarse washed material must be removed from bore holes before sampling and the hole should be cleaned so that soil at the intended top of the sample is as nearly undisturbed as possible.

c) *Sample retrieval*

Take the sample as soon as possible after cleaning the hole. Cleaning of the hole should not be attempted if sampling is to be delayed.

d) *Sampling operation*

   i) Preparation: Sampler and tube must be properly cleaned with vents, valves, piston packing, etc. checked for proper placement and function.

   ii) Lowering Tube: Lower sampler slowly and carefully to bottom of hole without dropping. When encountering water table while lowering the sampler, precaution must be taken with samplers containing piston rod extensions to prevent an upward rise of the piston.

   iii) Securing Piston Rods: Provide piston extension rods with a positive locking device at ground surface, and securely lock piston rods before sampling.

   iv) Penetration: force the sample tube past the locked piston by uninterrupted hydraulic pushing. Do not rotate sample tube during downward movement.

v) Length of penetration: Length of sample penetration should never exceed length of sampler. For sampling tube 20 mm ID (with internal diameter), penetration should be exceeded 10 times ID for cohesionless soils and 15 times ID cohesive soils.

vi) Withdrawal: After penetration allow sampler to sit for at least 10 Min. before withdrawal. Then rotate sample tube two to three revolutions and withdraw slowly using moderate up ward pull on drill rod avoiding sudden acceleration, shock, or vibration.

vii) Tube Removal: After withdrawing the sampler from the hole, take care not to drop it on the ground. Remove the tube from the sampler head without disturbing the sample.

e) *Sample preservation:*

The procedure for sample preservation is as follows.

i) Handling: Handle sample tubes with extreme care at all times after removal from borehole.

ii) Sealing: Before sealing, remove any disturbed material from the tube and clean tube walls to provide good contact for sealer wax. After waxing the ends of the tube, place snugly fitting metal caps at each end tape them to the sample tube. Again, immerse the tube ends in wax. When there is an annular clearance between the sample and tube that cannot be completely sealed, remove the sample from the tube and wax the sample completely in a large container. If too great an inside tube clearance is suspected, obtain new tubes having a smaller clearance before further samples are taken.

iii) Identification: Mark sample tubes with boring number, sample number, depth, total drive, measured recovery of undisturbed soil before trimming and description of soil type at the upper end of the tube.

iv) Protection: Protect sample from extreme heat and freezing after withdrawal from hole and during transportation.

v) Packing: Pack sample tubes for shipment with sawdust in sturdy boxes.

vi) Sample Retention: Indefinite storage of samples is not warranted. They should normally be retained only until the construction contract is awarded.

Annexure 2.3 Geophysical Characterization for Design of High Embankments

Stage of Project	Objective	Suggested Method	Basic Field Procedure	Limitations	Relevant Codes
Preliminary Survey during Preliminary Engineering	Ground Characterization - depth to bedrock - depth to water table - soft/compressible and weak/loose strata - thickness and relative stiffness soil/rock layers - direct measurement of shear wave velocity (dynamic soil property)	1. Multichannel Analysis of Surface Wave (MASW) / Spectral Analysis of Surface Wave (SASW)	Impact load is applied to the ground surface. Surface waves propagate along ground surface and are recorded on the ground surface with two geophones positioned along a line.	Resolution decreases significantly with increasing depth; interpretation is difficult if a stiff layer overlies a soft layer and soft layer properties are desired. In absence of MASW test, suitable and justified field correlation between Standard Penetration Test (SPT) – N value with shear wave velocity may be recommended under the guidance/supervision of a subject expert.	(1) IS: 1892- Code of practice for subsurface investigation for foundation (2) IS: 15681- Geophysical exploration by geo physical method (eismic refraction) Code of Practice (3) IS: 15736- Geophysical exploration by geo physical method (Electrical resistivity) Code of Practice
		2. Electrical Imaging (2D)	DC current is applied to the ground by electrodes. Voltages are measured at different points on the ground surface with other electrodes positioned along a line.	Slow; must install electrodes directly in the ground; resolution decreases significantly with increasing depth; resolution is difficult in highly heterogeneous deposits	
		3. Seismic Refraction	Impact load is applied to the ground surface. Seismic energy refracts off soil/rock layer interfaces and the time of interval is recorded on the ground surface using several dozen geophones positioned along a line or performing repeated events using a single geophone.	Does not work if stiffness decreases with depth or if soft layer underlies stiff layer; works best when sharp stiffness discontinuity is present	
Detailed Survey during Detailed Engineering	• Ground Characterization • Design Parameters	1. All/part of above	- Do -	Noted above	

Stage of Project	Objective	Suggested Method	Basic Field Procedure	Limitations	Relevant Codes
Engineering	• Measurement of wave velocities for seismic site response analysis • Liquefaction Risk • Earthquake generated ground-surface movements	2. Crosshole/ Downhole Seismic test (CHST/DHST)	Energy sources and geophones are placed in boreholes and/ or on ground surface; interval travel times are converted into seismic wave velocity as a function of depth in the borehole.	Requires one or more borehole and significant support field equipment	(1) IS:13372 (Part 1): 1992, ***"Seismic Testing of Rock Mass- Code of Practice- Part 1: Within A Borehole",*** Bureau of Indian Standards, Delhi. (2) ASTM D7400-08, ***"Standard Test Methods for Downhole Seismic Testing,"*** American Society for Testing and Materials. (3) American Society for Testing and Materials, ***"Standard Test Methods for Cross-hole testing,"*** ASTM D4428-D4428M-00.
	Four independent measurements with depth: cone tip resistance, sleeve friction, penetration porewater pressure, and downhole shear wave velocity.	3. Seismic CPT	The addition of a geophone in the piezocone body enables the collection of seismic wave data and the calculation of shear and compression wave velocities during the cone penetration test.		

CHAPTER 3

STABILITY ANALYSIS

3.1 Introduction

Failures may occur slowly or suddenly, and stability analysis is meant to determine whether the proposed embankment slope will meet the safety requirements against failure arising from shear stress exceeding the tolerable limits. The analysis is generally made for the worst conditions which may occur during the service of the embankment. In this task, besides knowledge of the analytical method, experience and judgment are essential.

Stability of high embankment depends on various factors like foundation profile, fill material quality, extent of compaction, drainage arrangement both surface and sub-surface, and embankment geometry like height of embankment, slope angle, ground profile etc., external factors like traffic or earthquake load or presence of any water body by the side of the embankment or development of pore water pressure due to infiltration from heavy rain. All these parameters and conditions will make significant impact on overall stability of the embankment. Hence, it is very important to understand and evaluate these site specific conditions and interpretation of design parameters correctly before proceeding with design.

3.2 Types of Failure

Failure of highway embankments generally occurs in the following modes

a) Failure of an embankment: Embankments fail when a part of the soil mass moves in an outward and downward direction. This is often called as slope failure or a slide. The term "slide" is also associated with failure of natural slopes.

In the case of embankment failure the mass moving out may include soil in the fill only or such movement may include the fill and the natural ground or subsoil. Generally speaking, failures occur in rotational mode in soil slopes. Failure surface forms an arc of a circle, isolating the failed mass from the rest of the embankment. If the failure arc cuts the slope it is called as slope failure, if it meets the toe it is called toe failure. If the failure circle goes into the subsoil it is called as base failure. Presence of weaker layer may cause a failure surface to take a composite shape. **Fig. 3.1a & 3.1b** shows the failure surfaces discussed above. Generally, base failures do not occur if the foundation soil is firm and has an angle of internal friction greater than 30^0. i.e. if the soil is sandy or gravelly.

Long slopes of purely cohesionless soils may fail along planar surfaces.

b) Bearing capacity failure: Embankment may fail in bearing capacity if the soil on which it is founded does not have enough shear strength. In this mode embankment sinks into the ground causing large vertical settlements as well as lateral displacements in the soil adjacent to the toe.

Embankments also experience settlements and sometimes such settlements may occur over a long period of time and this process depends on many factors. Excessive settlements causes distress and are a cause of concern hence it is necessary that the magnitude and rate at which settlements progress is also evaluated.

In this chapter, methods of analysis of failure of embankments are discussed. Settlement analysis is dealt with in Chapter 4.

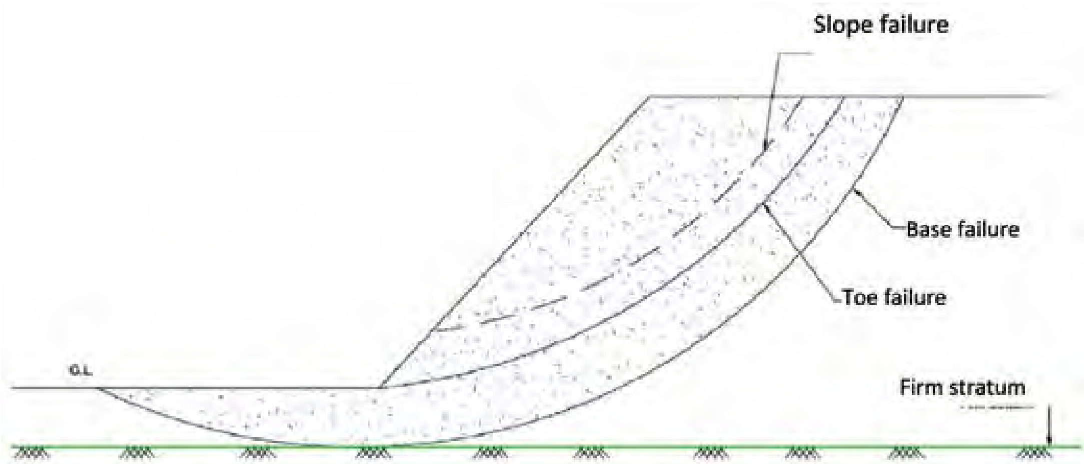


Fig. 3.1.a Rotational Failure Along Circular Surface

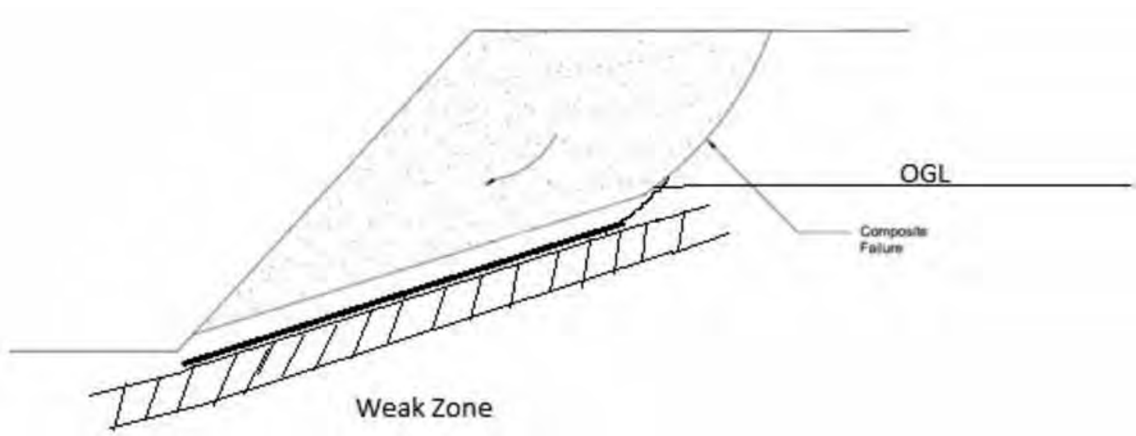


Fig. 3.1.b Composite Failures along non Circular Surface

Note: The failure surface is usually tangential to the weak zone

3.3 Basic Considerations in Design

There are some basic factors which influence analysis of slope stability problem. Principal among these are the choice of method of analysis (i.e. effective stress or total stress method), stage of construction for which the analysis is carried out (i.e. short term or long term condition) and the proposed factor of safety. Before going into the actual analysis, understanding of these factors is important.

3.3.1 *Total and Effective Stress Methods*

Analysis of stability can be done either in terms of total stress or effective stress depending upon the soil properties, loading conditions and the prevailing stage of construction.

a) *Total stress method*

Stability analysis may be carried out using either by total stress or effective stress method. Total stress method is applicable where an embankment is constructed on saturated clays of low permeability and no change in water content occurs in the subsoil prior to failure. Shear strength in this case may be given as follows

$$\tau=c_u+\sigma_n\tan\phi_u \qquad \text{...Eqn. 3.1}$$

Where c_u and ϕ_u are called undrained shear parameters. Saturated clays when tested to failure under undrained conditions yield shear strength parameter ϕ_u =0 and $c_u = (\sigma_1-\sigma_3)/2$. This analysis also called ϕ_u =0 analysis. This method is applicable to conditions where no dissipation of pore water pressures has yet occurred subsequent to loading. With the lapse of time the pore water pressure decreases. Under such conditions stability analysis may be carried out using total stress method by determining the new value of c_u by laboratory or filed tests. The above procedure may be repeated at as many time intervals as required in the project. The above statements are valid only for fully saturated clays.

b) *Effective stress method*

Effective stress method of analysis takes into account the pore water pressures for the stage at which stability is to be analysed. The relationship between shear strength and applied normal stress used in such analysis is given by the expression:

$$\tau=c'+(\sigma_n-u)\tan\phi' \qquad \text{...Eqn. 3.2}$$

c’ and ø’ are called as effective stress parameters. These parameters have to be determined from appropriate type of laboratory tests on soil samples.

Table 3.2 gives the type of laboratory tests to be carried out for undrained and drained shear strength parameters.

Using effective stress parameters, stability of the embankment can be determined any time during the life of embankment. Porewater pressures at the desired point of time shall be used.

3.3.2 *Porewater Pressure*

In making this analysis, the porewater pressure, ‘u’ at any point in the embankment is obtained from the expression:

$$u=u_0+\Delta u \qquad \text{...Eqn. 3.3}$$

Where, u_o represents the initial value of pore water pressure, and Δu denotes the incremental pore water pressure in the soil due to change in stress.

i) *Porewater pressure (Hydrostatic pressure)*

Arises from the presence of free water level within the vicinity of the embankment (u_o). The value of initial pore water pressure may be obtained from field piezometer measurements. If there is seepage through embankment the values may be obtained from flownets.

ii) *Hydrostatic excess pressure*

Arising from any additional load acting on the free body. The magnitude of such hydrostatic excess pressure is a function of load acting as well as properties of the material of the embankment. This component of pore water pressure is calculated using the following relationship (Skempton, 1954) where A and B are determined from laboratory tests.

$\Delta u = B\ [\Delta\sigma_3 + A\ (\Delta\sigma_1 - \Delta\sigma_3)$ …Eqn. 3.4

Generally for saturated clays B=1

Where $\Delta\sigma_1$ denotes the incremental total major principal stress

$\Delta\sigma_3$ denotes the incremental total minor principal stress

Use of Skempton's formula involves the use of following steps:

a. Determine Coefficients A and B from laboratory tests on undisturbed samples

b. Calculate $\Delta\sigma_1$ and $\Delta\sigma_3$ at different points in subsoil layer.

For this purpose it is necessary to calculate $\Delta\sigma_h$, $\Delta\sigma_v$ and shear stress at different points, using standard charts or formulae. From this stresses $\Delta\sigma_1$ and $\Delta\sigma_3$ can be calculated

Since the above procedure is complex, hence it is used in important cases. Generally $\Delta\sigma_u$ is equated to $\Delta\sigma_v$ and this is on the safe side.

iii) *Porewater pressure ratio*

Porewater pressure ratio at any depth is defined by the term:

$$r_u = \frac{u}{\gamma h}$$ …Eqn. 3.5

Where u is the pore water pressure, γh= total vertical stress at the same depth, (h is the depth of the point in the soil mass below the soil surface). r_u can be easily used for estimating slope stability from charts.

3.3.3 *Factor of Safety*

The results of the stability analysis are normally expressed in terms of a factor of safety with respect to shear strength. The factor of safety is defined as the factor by which the shear strength parameters (in terms of effective stress) c' and tanø' can be reduced before the slope is brought into the state of limiting equilibrium. The shear strength mobilized under these conditions is given by the expression

$$\tau = \frac{c'}{F} + (\sigma_n - u)\frac{\tan\emptyset'}{F}$$ …Eqn. 3.6

Where σ_n denotes the total stress normal to the potential failure surface and u denotes the pore water pressure. The definition is the same as that adopted by Taylor "the factor of safety with respect to shear strength", and is in accordance with that enunciated earlier by Fellinius (1927). It has the advantage of being applicable to circular and non circular slip surface alike without modifications and operates directly on the relevant strength parameters (Bishop & Morgenstern, 1960).

Loading Conditions

Live Load (External Traffic Load) 24 KN/m^2 is considered across the width of carriage way

Dead Load: Self weight of embankment and any structures resting on embankments

Static Case: Live Load + Dead Load

Seismic case: 50% Live load + Dead load + seismic load (As per IRC-6)

Table 3.1 Summary of Recommended Minimum Factors of safety (FOS) For Stability Analysis

Loading Condition	FOS under static loads	FOS under Seismic loads
Static Case	1.4 (at the end of construction)	1.1
	1.2 (*initial factor of safety)	
Sudden Drawdown	1.3	1.0
Steady Seepage	1.3	1.0

*Initial factor of safety 1.2 is applicable to situations where there is a gain in shear strength of subsoils due to ground improvement methods leading to increase in factor of safety with time. In such cases it is important that construction is continuously monitored for changes in pore water pressures, progress of settlements and occurrence of lateral deformations.

It should be remembered that maintenance of the design factor of safety in execution will invariably require a strict control over rate of construction in order to allow partial dissipation of pore water pressure at stages critical from the point of view of stability.

3.3.4 *Short Term and Long-Term Conditions*

Two types of slope stability problems occur in clayey soils; short term stability (end of construction case) and long term stability. When an embankment is constructed on a clayey soil, shearing resistance of the soil is drastically reduced by the development of excess pore water pressure during construction. Pore water pressure developed depends on the state of stress resulting from the weight of superimposed layer and the drainage conditions. With passage of time, porewater pressure is dissipated and soil improves its shearing strength. The rate of dissipation of pore water pressure is clearly related to the permeability of the soil. The critical period of shear failure in clayey soils is, therefore during construction and shortly after the completion of the embankment at which stage full embankment load is acting, but pore water pressure may not have completely dissipated. Eventually, the excess pore water pressure is dissipated and porewater acquires a state of equilibrium with ground water table, generating a steady state flow pattern. This stage is referred to as the long term stability or steady seepage case. In between there would be an intermediate stage when partial dissipation of pore water pressure has occurred and this is important for stage construction analysis. In embankments built of and resting on gravels, sands or cohesionless soils, the time required for dissipation of excess pore water pressure is very less and as such the stability is to be checked for long term condition only which is the same as short term condition.

The type of shear test /shear strength parameters depending on the stability analysis for the short term or long term stability is given in Table 3.2

Table 3.2 Strength Parameters for Stability Analysis

Sl. No.	Stage in the life of the embankment	Strength parameters	Shear test	Type of analysis
1.	(a) During construction or immediate post construction	c_{uu}, ϕ_{uu}	Unconsolidated undrained triaxial shear test on undisturbed samples and on as compacted embankment material IS 2720 Part XI	Total stress analysis assumes no drainage in field.
	(b) -do-	s_u	Unconfined compression strength (UCS – a special case of UU test where the confining pressure is zero) in laboratory or vane shear test. IS 2720 Part X	Total stress analysis for preliminary design
	Total strength parameters may be used in case of stage construction provided the undrained strength parameters are determined either in the field or in the laboratory for each stage of loading.			
	(c)-do-	c', ϕ'	Consolidated Un-Drained (CU) test with pore water pressure measurement on 'as compacted' soil samples of embankment material and on undisturbed samples IS 2720 Part XII	Effective stress analysis. Assumes effective stress in partially saturated soil is same as in saturated soil.(The above assumption neglects suction effects)
2	Long-term stability	c',ϕ'	-do-	Effective stress analysis. Used primarily for design of embankment constructed in stages provided pore water pressures at every stage are known.
Effective stress analysis may be used for stability analysis at the end of construction or after construction. It is assumed that the porewater pressure regime is stable or subjected to known changes				

3.3.5 *Bearing Capacity of Embankments*

Where subsoils are competent and have adequate bearing capacity to carry the load of the embankment , bearing capacity may be calculated according to the formulae given in IS: 6403 " Code of practice for determination of breaking capacity of shallow foundations"

$$Q_{nu} = cNc + q\,(Nq-1) + 0.5\gamma BN\gamma \qquad \text{...Eqn. 3.7}$$

Where Q_{nu} is net ultimate bearing capacity

c is cohesion, B= base width

Nc, Nq and Nγ are the bearing capacity factors which may be obtained from table 1 of IS: 6403

Effect of Water Table on Bearing Capacity

Modified bearing capacity formula considering water table effect is

$$Q_{nu} = cNc + q\ (Nq-1) + 0.5\gamma BN\gamma W' \qquad \text{...Eqn. 3.8}$$

a) If the water table is likely to permanently remain at or below a depth of (D_f + B) beneath the ground level surrounding the footing then W' = I.

b) If the water table is located at a depth D_f or likely to rise to the base of the footing or above then the value of W' shall be taken as 0.5

c) If the water table is likely to permanently got located at depth Df < D" < (D_f +B) t then the value of W' be obtained by linear interpolation.

Since embankments have large base width and subsoil may consist of many layers having varying values of c and ø in such a case weighted average of c and ø values over a depth of 1.5 H may be used.

Factor of Safety for Bearing Capacity of Embankments

Where subsoils are competent, minimum factor of safety of 1.5 shall be considered against bearing capacity failure.

Table 3.3 Recommended Factor of Safety for Bearing Capacity

	With Only Basal Reinforced Mattress	Ground Improvement	
		PVD's with stage construction (IS: 15284-part 2)	Stone columns (IS:15284-part 1)
Bearing capacity	1.5	1.25 (at the end of construction of a particular stage) 1.5 (at the end of waiting period specified for the stage)	2.0

Where sub-soils are soft and non frictional and ground improvement methods are used, bearing capacity may be calculated as per the formula given in FHWA NHI-95-038 (1998), Geosynthetic Design and Construction Guidelines. Participant Notebook for NHI Course No.1 3 213, and IRC:113

N_c may be calculated as given below

N_c = 5.14 for B/D <2

N_c= 4.14 + 0.5 B/D for B/D>2

Where B is the width of bottom of the embankment

D is the Depth of soft soil

3.4 Stability of Cohesionless Slopes

The stability of fill slopes built of cohesionless gravels, sands and silty sands, depends on; (a) the angle of internal friction of the fill material, ø', (b) the slope angle, (c) the unit weight of the fill, and (d) the pore water pressures. The critical failure mechanism is usually surface raveling or shallow sliding which can be analyzed using simple infinite slope analysis.

The values of ø' for stability analyses may be determined by drained triaxial or direct shear tests. Porewater pressure due to seepage through the fill reduces the stability of the slopes. But static water pressure with the same water level inside and outside the slopes has no effect on stability. The factor of safety of slopes formed by cohesionless materials resting on firm foundation can be determined as indicated below:

$$F = \frac{Tan\phi'}{Tan\beta} \qquad \text{...Eqn. 3.9}$$

Where ø' = angle of internal friction; and

β= angle of slope with horizontal.

The maximum stable slope angle of sandy embankment is related to the peak friction angle ø'. However, ø' is a function of void ratio, i.e. the density and the confining stress at which the sand exists. For dry loose sands, as in case of dumped sand or gravel, ø' is essentially equal to angle of repose. But slope steeper than angle of repose can be built in stable condition when the angle of friction is improved by compaction in thin layers. It is important to note that the angle of stable slope of cohesionless materials is independent of the height which may be indefinite. Sand dunes represent examples of natural slopes of varying height but constant slope. Furthermore, weight of the material does not affect the stability of slope, so that the safe angle for a submerged sand slope is the same as that for a slope composed of dry sand, with the exception of the special case of damp sand which has a high angle of repose due to capillary attraction. However limitation is imposed on height by other considerations like base failure and erosion.

Special conditions exist with partially submerged sand slopes affected by tidal conditions or seepage conditions (sudden draw down condition) or seepage condition which may cause the stability of fine sand slope to be considerably less than for dry or submerged sand. Factor of safety in such conditions is given by:

$$F = \frac{\gamma - \gamma_w}{\gamma} \frac{Tan\varphi'}{Tan\beta} \qquad \text{...Eqn. 3.10}$$

Since $\frac{\gamma - \gamma_w}{\gamma}$ ratio is typically about half for sands, the maximum stable slope is about half of that for dry or submerged condition.

Slopes in fine sands, silty sands, and silts are susceptible to failure by erosion due to surface runoff. Benches, paved ditches, and turfing on slopes can be used to reduce runoff velocities and retard erosion.

3.5 Slip Circle Analysis for Cohesive Soil Slopes

The first sign of imminent failure of slope is usually an outward or upward bulging near the toe and development of cracks usually along length wise direction and near the crest of the slope. Though in actual practice the failure plane may be a complex surface, in most stability analysis cases a circular cylindrical rupture surface is assumed to simplify the computations.

The analysis consists of drawing trial circles and calculating the factor of safety separately for each circle. For any given centre, several circles are drawn, passing through the toe, through the weakest sub-strata, and through other soil layers depending on the conditions obtaining, and the lowest factor of safety recorded. Analysis may be either by considering the stability of slope en-mass or by dividing the slip mass into many vertical slices and to consider the equilibrium of each slice, Methods that consider the slope en-mass include Culmann's method and Taylor's friction circle method. There are several versions of the method of slices available; the best known are Swedish Circle and Bishop's method. Each of these methods involves certain approximations. For most highway embankment problems it is sufficient to use approximate methods even though these may not fully satisfy the requirements of static equilibrium. The different methods available are however reviewed broadly in this section. Analysis in each case can be either by total stress analysis or effective stress analysis.

3.5.1 *Taylors' Method*

If the embankment and foundation are homogeneous and the slope is simple, Taylor's charts can be used directly for design of embankment. Even in other cases these charts can be made use of with advantage at the planning stage, especially when a number of alternatives are to be evaluated.

Taylor, after investigating a large number of trial circles in homogeneous soil by friction circle method, produced tables for locating critical circle and evolved design charts for determining safe slopes. Taylor's design curves are reproduced wide **Figs. 3.2 & 3.3. Fig. 3.2** shows, for various values of ø up to 25^0, the safe slope corresponding to stability number Ns which is equal to c/γH. For a particular value of stability number and ø, safe angle of slope can be determined by reading the chart. Alternatively for a particular value of angle of slope and ø, stability factor can be read from the chart and factor of safety computed as F=cNs/ γH. **Fig. 3.3** is applicable when, ø =0 and failure is over a shallow base.

3.5.2 *Swedish Slip Circle Method*

In this method (also referred to as the Fellenius method, the conventional method of slices or the USBR method) the soil mass is divided into a number of slices as shown in **Fig. 3.4.** The slices need not be of the same width. It is convenient to make the boundaries between slices to coincide with breaks in the surface or sudden transitions from one material to another, If the slope is stable, then each slice must be stable under its own weight and the forces on its boundaries. For each slice if the inter-slice forces are ignored .the reaction from below must be equal to and opposite in direction to its weight, as these are the only two forces. Resolving the reaction into normal force and shear force and taking into consideration the forces on a slice will be as shown in **Fig. 3.4.** Factor of safety is then obtained by summing over for all the slices. The expression for the factor of safety with reference to **Fig. 3.4** is given by:

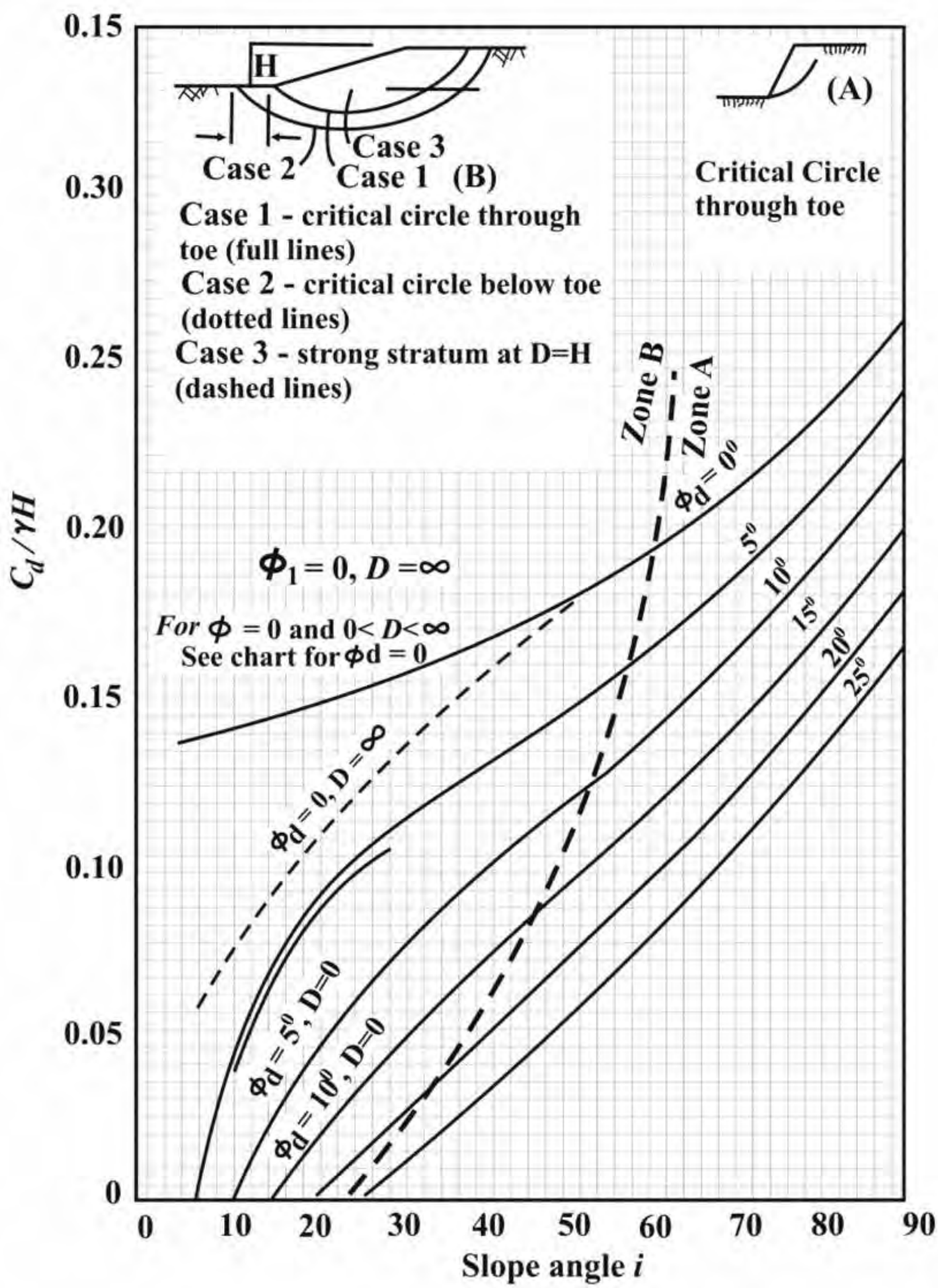


Fig. 3.2 Chart of Stability Numbers

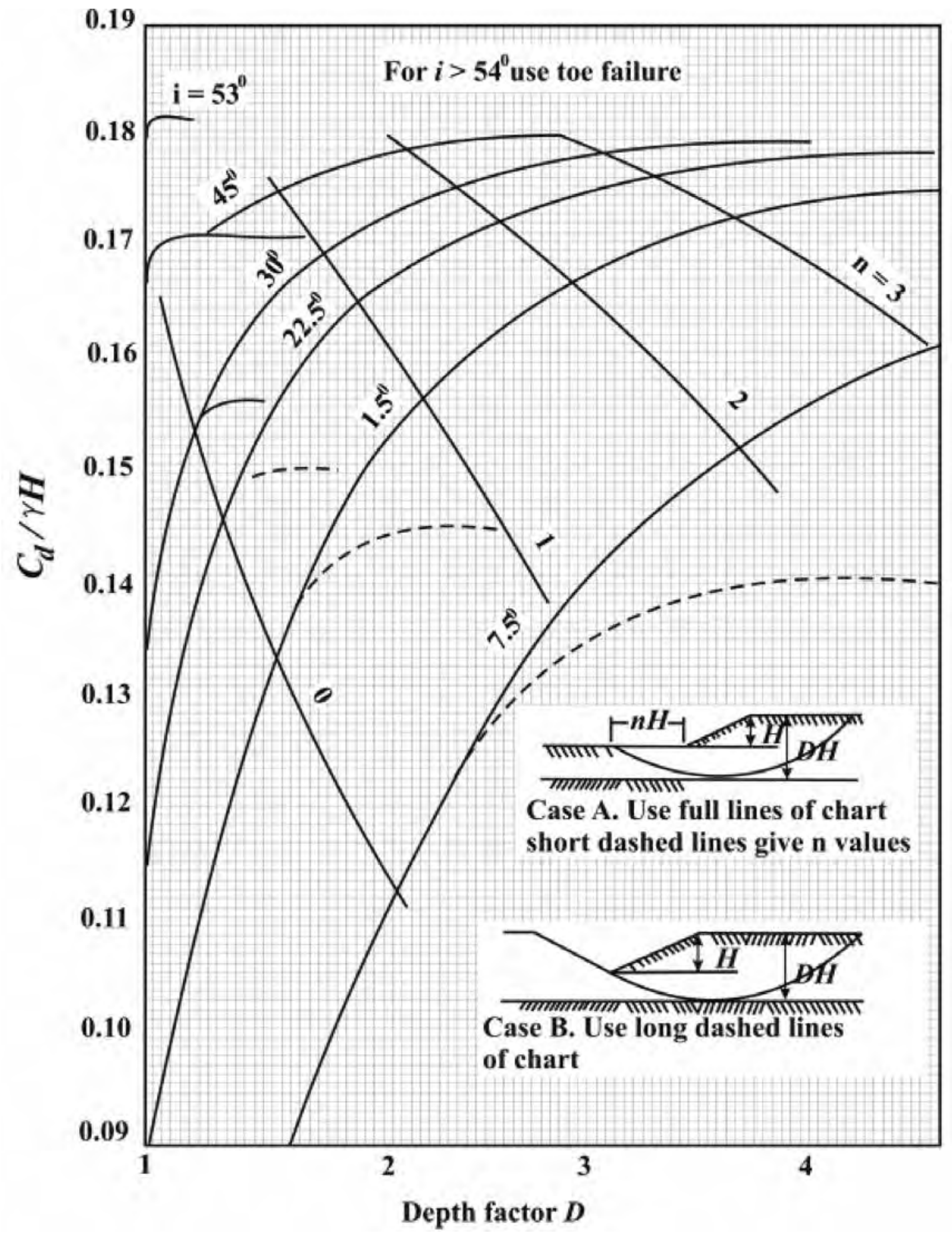


Fig. 3.3 Chart of Stability Numbers for the Case of Zero Friction Angle and Limited Depth

$$F = \frac{1}{\Sigma w \sin\alpha} \sum[c'l + (W\cos\alpha - ul)\tan\phi'] \quad \text{...Eqn. 3.11}$$

It is convenient in many cases to express the pore water pressure 'u' as function of the total weight of the column of soil above the point considered by using a ratio 'r_u' which is defined by the relation:

r_u** =u/γh ... Eqn. 3.12

Where h is the depth of the point in the soil mass below the soil surface, and γ is the bulk density of soil. The expression then becomes:

$$F = \frac{1}{\Sigma w \sin\alpha} \Sigma[c'l + (W\cos\alpha(1 - r_u \sec^2\alpha)\tan\phi'] \quad \text{... Eqn. 3.13}$$

The recommended method of recording the calculations is given in Table 3.4. A graphical approach to these calculations is also available (Murthy 1974).

The Swedish method permits a quick and direct computation of the factor, of safety and is therefore advantageous where calculations are done by hand. A large number of slip circles is normally required to be analyzed in each case and this is facilitated in this method due to its simplicity.

3.5.3 *Bishop's Method*

It has been shown that the conventional method of slices (i.e, the Swedish slip circle) could be in error where the central angle α, and the pore water pressure factor r_u are large. The error increases with increasing values of α, and u. However, it is on the conservative side. This is the reason why engineers continue to prefer the conventional procedure. But in large scale work and high embankments, the conventional procedure results in overdesign and uneconomical sections. In such situations, Bishop's solution is preferable. This method also follows the method of slices but in addition recognizes the existence of side forces on each slice. There are two versions of the Bishop's method, one rigorous and the other simplified. Both-are reviewed below:

a) *Bishop's Rigorous method*

The-rigorous method yields the following expression for factor of safety, **Fig. 3.5.**

$$F = \frac{1}{\sum W sin\alpha} \sum [\{c'b + (W(1 - r_u) + (X_n - X_{n+1})]tan\emptyset'\} \frac{sec\alpha}{1 + \frac{tan\alpha\ tan\emptyset'}{F}}] \quad \text{... Eqn. 3.14}$$

For discussion of conditions to be satisfied to determine the internal forces X and E, the reader is referred to the earlier work by Bishop (1955). The determination of the interslice forces is necessary for a rigorous solution of the equation given above.

Sharma (1972) has suggested a simple graphical approach for circular arc analysis yielding values of factor of safety very close to those computed by most sophisticated methods requiring high speed digital computers.

**a) The r_u factor is linearly related to factor of safety F for range of r_u values from 0.0 to 0.7 usually encountered in engineering practices (Bishop 1952. 1955, Bishop and Morgenstern, 1960). The extra- ordinary advantage of this relationship is that it gives an immediate picture of the influence of pare pressure on the factor of safety.

b) Generally speaking, r_u is not constant all along the slip surface, but in most stability problems an average value can readily be calculated and used with little loss of accuracy.

$$F = \frac{1}{\sum W sin\alpha} \sum [c'l + (W cos\alpha - ul) tan\emptyset']$$

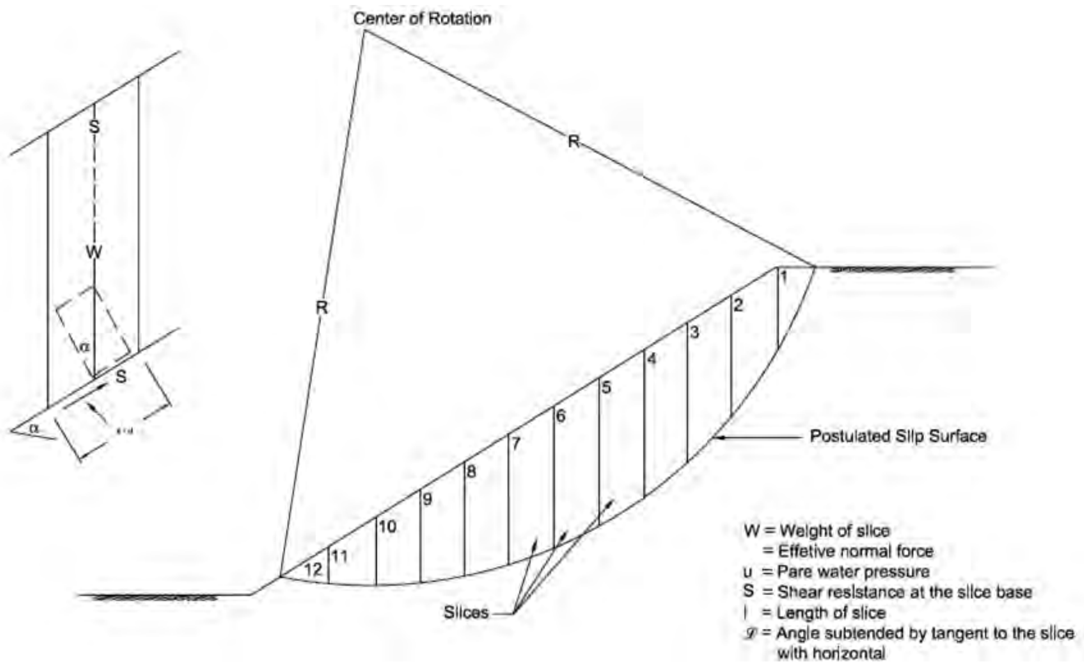


Fig. 3.4 Swedish Slip Circle Method

Table 3.4 Format of Table for Manual Calculations for Swedish Slip Circle Method

1	2	3	4	5	6	7		8		9	10	11	12	13	13	15
S L C E No.	α	Cos a	Sin a	l	β	Weight of the Slice		Pore Water Pressure		ul	C'l	W Cos a	(W sin a	(W Cos a-ul)	(W Cos a-ul) tan ϕ	Factor of safety Col. 10+Col. 14 / Col. 12
						Average Ht.	Weight W	Pore Water Ht.	Pore Pressure U							

b) *Bishop's routine method*

It is usually adequate for practical purposes, to neglect the term (X_n - X_n+1) in the equation above without any significant loss of accuracy.

The equation for the factor of safety, F, then becomes:

$$F = \frac{1}{\sum W sin\alpha} \sum [c'l + (W cos\alpha - ul) tan\emptyset']$$

... Eqn. 3.15

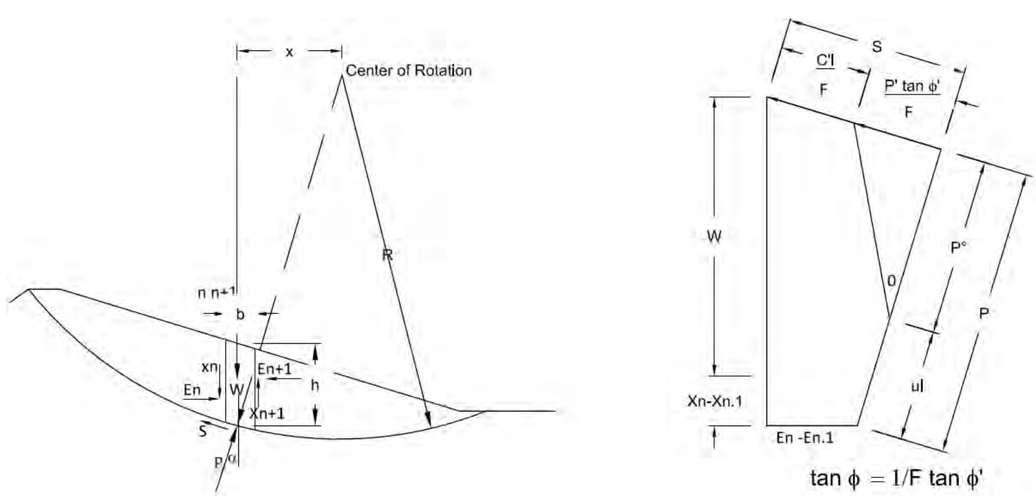
Where $\quad m\alpha = \cos\alpha\,(1 + \frac{\tan\alpha \tan\phi'}{F})$ ... Eqn. 3.16

The use of the above equation represents 'Bishop's Routine Method'; the recommended method of recording the calculations is given in Table 3.5, **Fig. 3.5.** In practical application of this method, as F appears on both sides of the equation, F has to be assumed in advance and mα is to be calculated for this F. For this purpose, a chart is given in **Fig. 3.5** which provides mα for known values of α and ø' and assumed F. The factor of safety is worked out as. Col. (14)/Col (7) of Table 3.5. This is compared with the assumed value of F. If they do not agree, a new value of F is assumed and the process repeated. The new trial requires only additional sub-columns under columns 13 and 14. Thus, after two or three trials, the correct factor of safety is evaluated for this assumed failure surface. To obtain the factor of safety for the slope, several failure surfaces have to be tried.

Table 3.5 Format of Manual Calculations for 'Bishop's Routine Method'

1	2	3	4		5	6	7	8	9	10	11	12	13	14
S L C E No.	a	h	Weight of the Slice		α	sin α	(W sin α	C'd	(W (1-ra) tan φ	S+9	Sin α	tan α	Sec α	10 × 13
			Mean Pr.	Weight W									$1 + \frac{\tan\phi' \ \tan\alpha}{F}$	

$$F = \frac{1}{\sum W sin\alpha} \sum [\{c^{'}b + (W(1 - r_u)tan\emptyset^{'}\} X \frac{sec\alpha}{1 + \frac{tan\alpha\, tan\emptyset^{'}}{F}}]$$



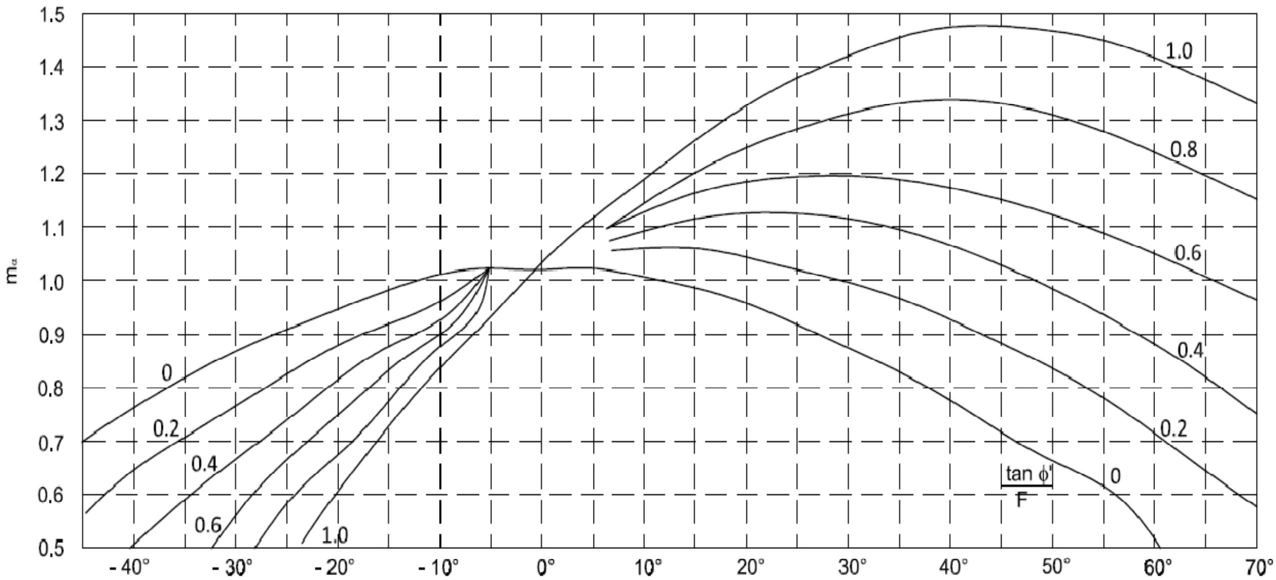


Fig. 3.5 Stability Analysis by Bishop's Routine Method (Chart for mα)

The use of Bishop's Routine Method can be extended to cover the cases of partially submerged embankment slopes which are common in engineering practice. **Fig. 3.6 (a)** shows a partially sub- merged embankment with circular arc failure surface and its centre of rotation. The various forces acting on one of the slices constituting the sliding mass are shown in **Fig. 3.6 (b)** and an equilibrium vector diagram is drawn in **Fig. 3.6 (c)**.

The expression for factor of safety can be written as

$$F = \frac{1}{\sum(w_1+w_2)sin\alpha}\sum[\{c'b + tan\emptyset'(w_1 + w_2 - u_s b\}X\frac{1}{m\alpha}]$$ …Eqn 3.17

Where mα = $cos\alpha(1 + \frac{tan\alpha\ tan\emptyset'}{F})$ …Eqn 3.18

the other notations are explained in **Fig. 3.6**.

The recommended method of recording the calculations is given in Table 3.3.

For total stress analysis, ø is taken as zero in the above mentioned equations and c' is replaced by c_u.

Thus, $F = \frac{\sum C_u l}{\sum WSin\alpha}$ …Eqn 3.19

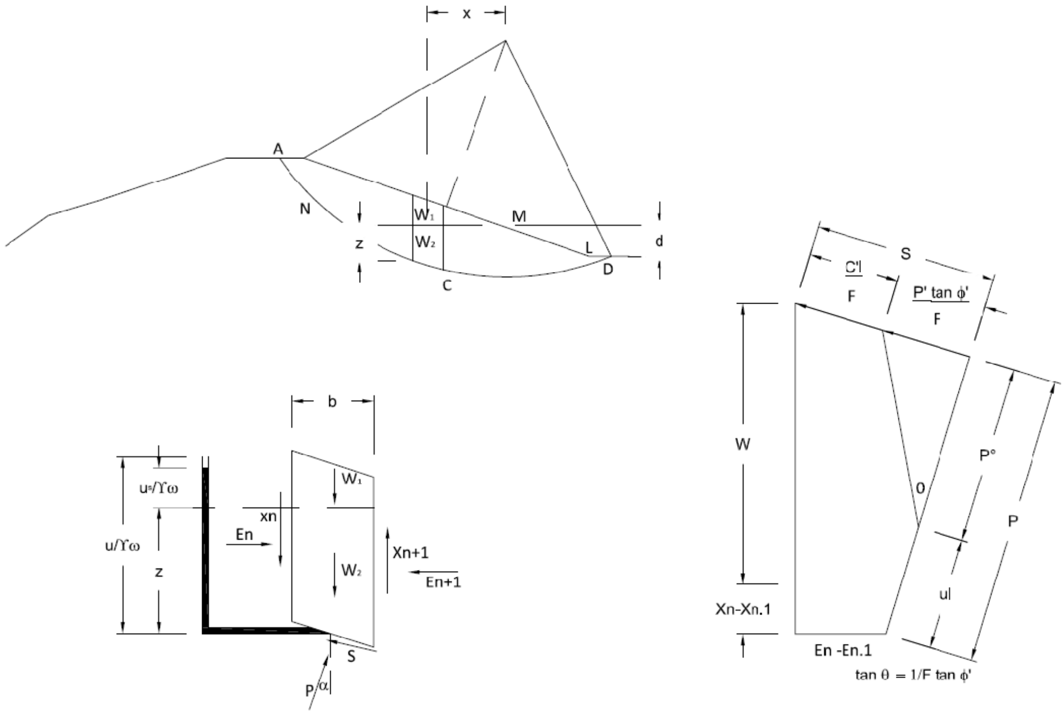


Fig. 3.6 Stability Analysis of Partially Submerged Slope by Bishop's Routine Method

Where En, En+1 denote the resultants of the total horizontal forces on the sections n and n+ 1 respectively,

X_n , X_{n+1} the vertical shear forces,

W denotes the total weight of the slice of soil.

P the total normal force acting on its base,

S the shear force acting on its base,

H the height of the slice,

b the breadth of the slice,

l the length BC,

α the angle between BC and the horizontal,

x the horizontal distance of the slice from the centre of rotation,

W_1 full weight of the soil in the slice above MN,

W_2 submerged weight of soil in the part of the slice below MN,

γ_w the density of water,

Z the depth of slice below MN

3.5.4 *Chart Solutions for Analysis*

A number of chart solutions for embankment stability problems have been developed to reduce the time involved in calculations. Main among these is the charts developed by Taylor, Bishop and Morgenstern (Geotechnique 10, 1960), Morgenstern, Spencer, Hunter, Hunter and Schuster and Huang. Of the chart solutions available, those of Taylor, Hunter and Hunter and Schuster are based on total stress analysis and are best suited for analyzing end-of-construction stability. These are ideal for use as regards road embankment design. Other charts are based on effective stress analysis and can be applied in all cases. Morgenstern's solution is particularly good for small dams and consequently might be applicable where highway embankment is used as an earth dam or where flooding might occur behind a highway fill. All these methods make similar assumptions regarding geometry etc., but differ in assumptions about variation of cohesion with depth, position of water table, base condition, drawdown condition and slope of failure surface. Review of the chart solutions is available in Highway Research Record No. 345 and Transport Research Record 548 (published by the Transportation Research Board, U.S.A.) In general, simpler solutions using total stress analysis are adequate for highway embankment design.

Table 3.6 Format of Manual Calculations for Bishop's Routine Method (for partially Submerged Slopes)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
Slice No.	b	w_1	w_2	$(W_1+W_2)b$	α	Sin α	(W_1+W_2)Sin a	h_1 (Excess Standpipc) Height	$u_s/\Upsilon\varpi\ h_1$	b. u_s	$(W_1+W_2\text{-b.}U_s)$	$(W_1+W_2\text{-b.}U_s)\tan\phi$	c' b	(13) + (14)	Sec α	tan α	$\frac{\text{Sin}\,\alpha}{1+\frac{\tan\phi'\,\tan\alpha}{F}}$	(15) + (18)

$$F = \frac{\Sigma(19)}{\Sigma(8)}$$

3.5.5 *Miscellaneous Hints about Slip Circle Analysis*

A few useful hints about design when conducting a slip circle analysis are brought out in the succeeding paragraphs for guidance.

3.5.5.1 Locating the centre of critical circle

In slip circle analysis, a trial and error approach for locating the circle having the smallest factor of safety is necessary. For this purpose, the normal procedure is to establish a grid of centers and calculate the factor of safety for each. The factor of safety is then entered on the grid and contours of equal safety factor drawn. The parameters which influence the position of the critical circle in a given case are: the slope of the embankment, depth of hard strata, the soil properties ø, c/ γh, and the pore-pressure. Fellenius charts or Taylor's Tables may be used for locating the approximate critical circle. However, it should be noted that for base failures, provision of a balancing berm shifts the critical centre towards the berm. Also there are usually two critical centers', one slightly above the embankment level and the other at a greater height. The latter is usually more critical.

For slope failures, contours showing the variation in factor of safely are roughly elliptical, with the major axis approximately at right angles to the surface of the slope and several times the minor axis. The centre of critical slip circle is usually located close to and slightly above the perpendicular bisector of the slope.

3.5.5.2 Tension cracks

One of the features of rotational slips in cohesive soils is the appearance of a vertical crack running parallel to the top of the slope and at some distance from it usually about 1.5 meter from the edge. The maximum depth of tension crack is given by the equation

$$Z = \tan\left(\frac{\pi}{4} + \frac{\emptyset}{2}\right) \cdot \frac{2C}{\gamma} \qquad \text{... Eqn.3.20}$$

To account for the effect of tension cracks in stability analysis, the effective length of the slip circle for the purpose of resisting moment is taken only up to the end of the tension crack. For calculation, the position of tension crack is so assumed that its lowest point touches the circle under consideration. The most dangerous condition in the case of tension cracks would occur during rainy season when these fill up with water and exert a hydrostatic pressure horizontally of the magnitude of

$$PH = 0.5\ \gamma\ Z^2 \qquad \text{... Eqn. 3.21}$$

In making the stability computations, this pressure is added to the total driving force.

3.6 Stability Analysis for other Modes of Failure

Planar and composite failure: Methods of analyzing stability for non-circular slip surfaces are numerous. For example see Kenney (1956), Janbu (1956), Nonveiller (1965), Morgenstern and Price (1965 and 1967). Of these, the method suggested by Janbu (1956) is recommended; since it permits easy hand calculations (refer **Fig. 3.7**). The expression for factor of safety, with reference to **Fig. 3.7** is given by:

$$F = f_o \sum[\{\frac{c'b+(W-ub)tan\emptyset'}{\sum Wtan\alpha}\}.\frac{1}{n\alpha} \qquad \text{... Eqn.3.22}$$

where $n_\alpha = cos^2\left(1+\frac{tan\alpha\ tan\emptyset'}{F}\right) = m_\alpha cos\alpha$ ... Eqn.3.23

and fo=Correction factor (see **Fig. 3.7b**) depending on the shear parameters and form of the slip surface. It takes account of the influence of the vertical shear forces between the slices on factor of safety.

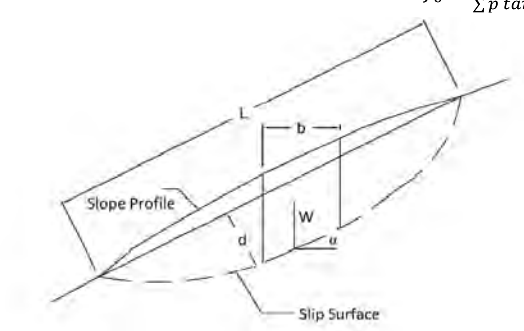
The above expression is for analysis according to effective stress method. In the case of total stress analysis, the equation for factor of safety could be written as:

$$F = f_0 \frac{\frac{\sum C_u \times l}{\cos\alpha}}{\sum W\ tan\alpha} \qquad \text{... Eqn.3.24}$$

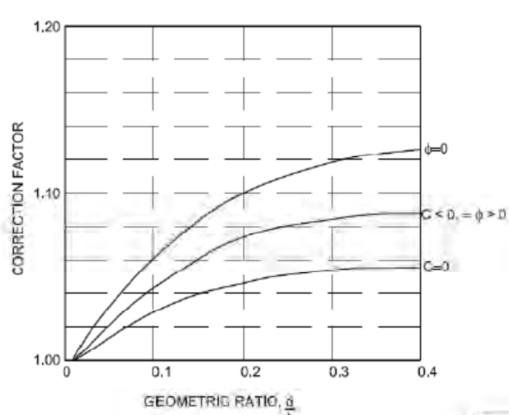
The suggested method of recording the calculations is given in Table 3.7. A chart enabling quick calculation of n is given in Table 3.7, **Fig. 3.7(c)**.

For more accurate treatment of stability on non-circular slip surfaces, the method developed by Morgenstern and Price (1967) can be used. However, it requires the use of a computer.

$$F = f_0 \frac{\frac{\sum c'+(p-u)tan\emptyset'}{n_\alpha}}{\sum p\ tan\emptyset'}$$



GEOMETRY OF SLIP SURFACE



(b) CORRECTIO N FACTOR

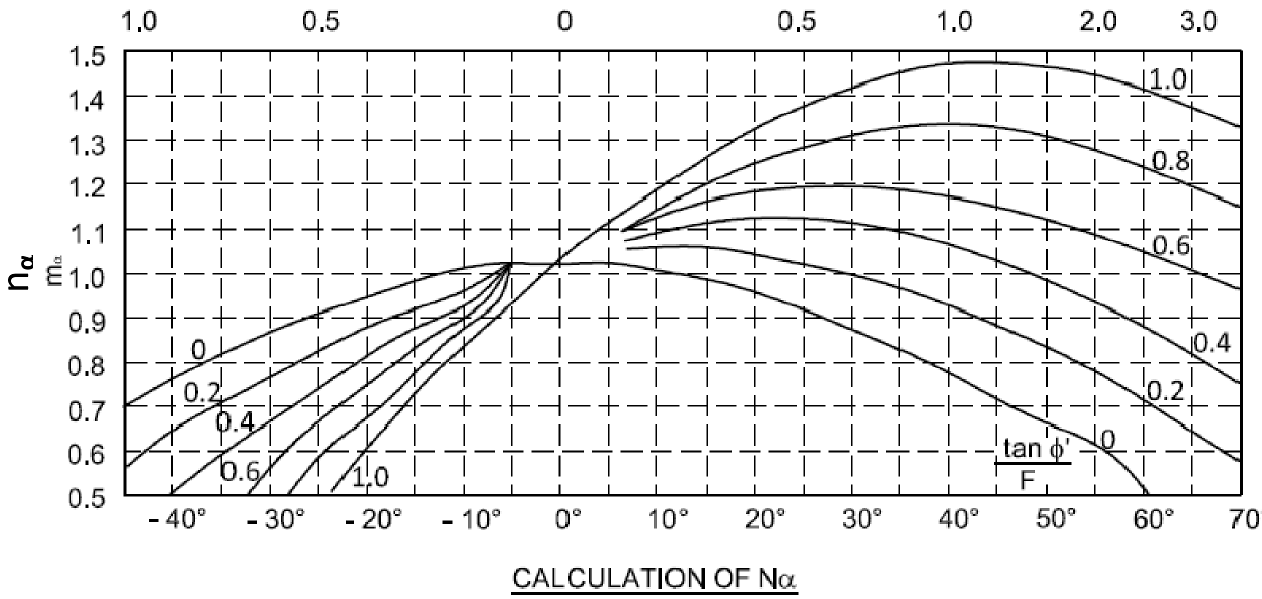


Fig. 3.7 Stability Analysis by Janbu's Method

Table 3.7 Format of manual calculations for Janbu's method

1	2	3	4	5	6	7	8	9	10	11
Slice No.	A	tan α	p	u	c	tanø'	p tan α	C+(p-u) tanø'	F=?	$\frac{C' + (p-u)\tan\phi'}{n\alpha}$

3.6.2 *Sliding Block Method*

The method is frequently applied in two circumstances:

(a) When a thin layer of soft soil (which may not necessarily be horizontal) is encountered at shallow depth in the foundation.

(b) When the embankment rests on a hard rock stratum which is unlikely to be involved in the failure.

The analysis is possible in terms of both total stress and effective stress depending upon the conditions of the project and availability of data on hand.

In this method, it is usual to divide the sliding mass into two or three large sections or wedges. The upper and the lower wedges are respectively called the active and the passive wedges. In a three wedge system, the middle wedge is generally referred to as the sliding block, **Fig 3.8**

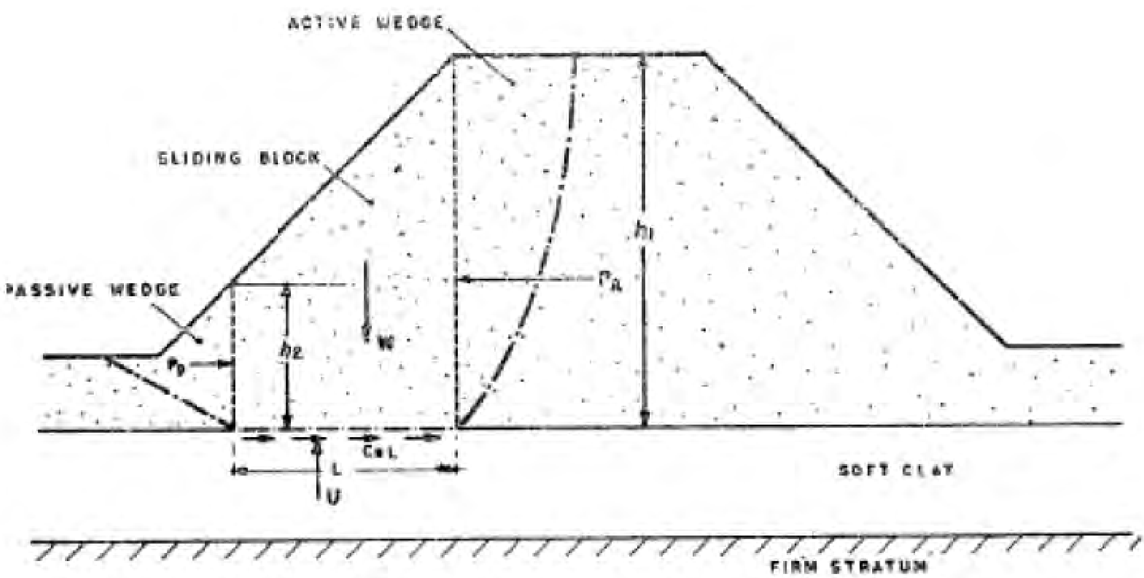


Fig. 3.8 Sliding Block Analysis

Assuming that sufficient deformations have occurred to generate active and passive failure wedges, and considering the stability of the sliding block for equilibrium in terms of total stress (**Fig 3.8**), factor of safety is given by expression:

$$F = \frac{C_u L}{P_A - P_P} \quad \text{... Eqn.3.25}$$

Similarly for equilibrium in terms of effective stress, factor of safety is given by the expression:

$$F = \frac{c'L + (W - U)\tan\phi'}{P_A - P_P} \quad \text{...Eqn.3.26}$$

Where $P_A = \frac{yh_1^2}{2n_\phi} - \frac{2c'h_1}{\sqrt{n_\phi}} + \frac{qh_1}{n_\phi}$ ... Eqn.3.27

$$P_p = \frac{\gamma h_2^2 N_\emptyset}{2} + 2C' N_\emptyset \,.\, h_2 + q N_\emptyset h_2 \quad \text{... Eqn.3.28}$$

$N\text{ø}=\tan^2 (45+\text{ø}'/2)$ ... Eqn.3.28

W= the total weight of the sliding block.

U=u.L

u= the pore water pressure acting on the sliding block

q= surcharge, if any. These formulae for working out active and passive pressure assume planar failure, but other sophisticated, methods are also available.

Limit equilibrium methods divide the slide mass in to a number of slices for the purposes of analysis and this process introduces more number of unknowns than knowns, making the problem statically indeterminate.

Assumptions are made in different equilibrium methods to make the problem statically determinate. The same are summarized in Table 3.8 below. This table lists the common methods of stability analysis and condition of static equilibrium that are satisfied in determining the factor of safety.

For further details of distribution of inter slice forces and related aspects please refer to Lee. W. Abramson et. al. chapter-6, Slope Stability concepts

Table 3.8 Static Equilibrium Conditions Satisfied by Limit Equilibrium Methods

Method	Force Equilibrium		moment Equilibrium
	x	y	
Ordinary method of slices (OMS)	No	No	Yes
Bishop's simplified	Yes	No	Yes
Janbu's simplified	Yes	Yes	No
Lowe and Karafiath's	Yes	Yes	No
Corps of Engineers	Yes	Yes	No
Spencer's	Yes	Yes	Yes
Bishop's rigorous	Yes	Yes	Yes
Janbu's generalized	Yes	Yes	No
Sarma's	Yes	Yes	Yes
Morgenstern-Price	Yes	Yes	Yes

3.7 Stability Analysis Using Software

Slip circle analysis is a method of checking the stability of any slope against its probability to fail in rotational mode; sometimes failure surface may not be circular. The factor of safety for a particular circle passing through the slope is calculated by taking into account force equilibrium and/or moment equilibrium. While some methods of analysis consider force equilibrium and moment equilibrium, some methods consider only force equilibrium (Abramson Lee. W. et. al.) “Slope stability and stabilization methods” (John Wiley”) the location (center and radius) of the most critical circle depends on all the factors described above. The critical circle is the one with lowest FOS which has the highest probability to fail in case the disturbing force is greater than or equal to the resisting force. Hence, the objective of the analysis is to find the most critical circle by an iterative method.

There are many types of software available for use. The modes in which the input parameters are provided in these softwares differ from each other. Similarly the output format also differs from one program to another. The user should be familiar with both these aspects.

In software, by defining the phreatic line, the unit weight above and below the phreatic lines are automatically considered by the software. The same condition can be modeled by defining two separate densities for soils above and below the phreatic line.

There are various ways and means by which the critical circle can be found and different software has different options and tools to find the circle which has the lowest FOS. The most common iterative method is to define a grid of centers and a defined point for the circle to pass. The user must check all possible circles by changing the location of grid and point of passing to find the critical circle.

The software automatically calculates the FOS for all possible circles passing through this point and varying the location of the centers within the defined boundary of the grid. There are few advanced softwares available which also automatically search the critical circle by increasing and decreasing the radius of the circle However, these features are provided just to help for a quick search. It is up to the user to ensure that all possibilities are checked by an iterative method before concluding for the most critical circle.

The critical circle may or may not pass through the toe of embankment; it depends on the properties of the foundation soil parameters. The weaker the foundation soil, higher is the probability of deep seated failure

The basic method to ensure that the critical circle has been derived is by drawing a contour map of all FOS The center of the critical circle shall lie within the defined grid of centers and not on the edge of the grid. It is strongly advised that at least in case of one circle preferably critical circle, the force evaluation and calculation of factor of safety may be carried out by hand calculations or have the data from the software imported into the format given in Tables 3.4 to 3.7 as relevant to the method of analysis adopted.

3.8 Seismic Slope Stability

The basic terminology used in this section is as follows

a_{max}/g= Peak Horizontal Ground Acceleration (PHGA)

K_H= Horizontal seismic coefficient expressed as a_{max}/g

K_h= Design horizontal seismic coefficient expressed as 0.5 a_{max}/g

(As per 1893 section 6.4.2)

Z= zone factor expressed as a_{max}/g

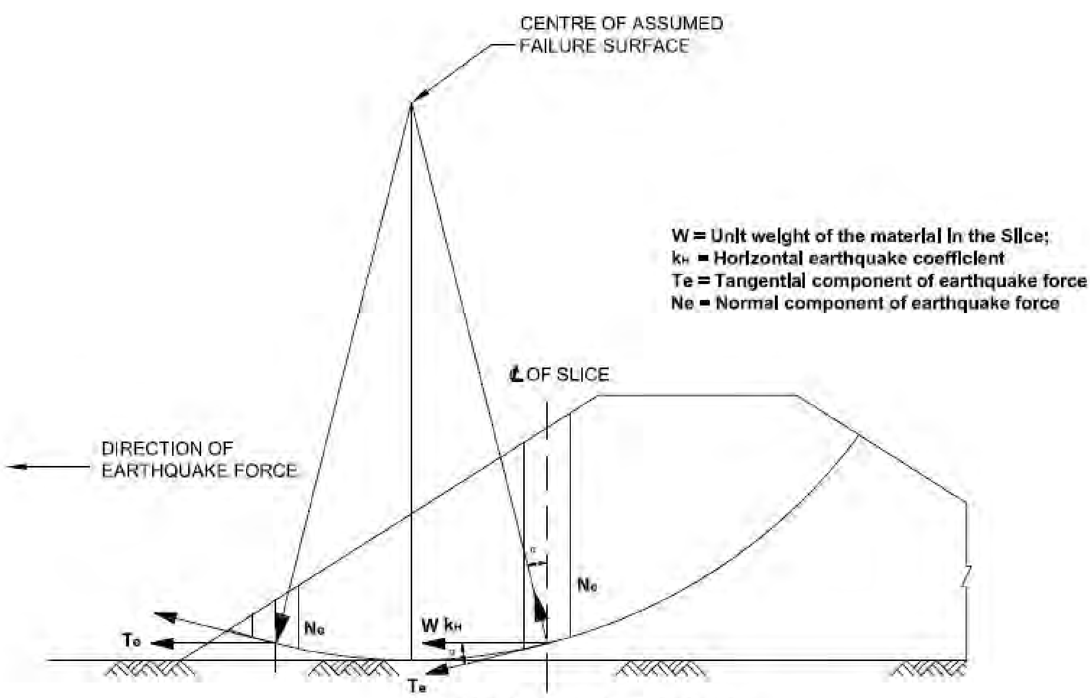
The ground accelerations associated with seismic events can induce significant inertia forces that may lead to instability of natural and man-made slopes and embankments.

In a pseudo-static limit equilibrium analysis, the earthquake inertia forces are represented by static loads applied at the centre of gravity of each “slice” through the potential failure mass. Numerous limit equilibrium methods and procedures are currently available to evaluate static slope stability. The search for the critical surface i.e. the surface with the lowest factor of safety or yield acceleration may have to be repeated because the critical surface from the static analysis is not necessarily the same as the critical surface for the dynamic analysis.

A wide variety of commercially available computer programs exist that can perform both static and pseudo-static limit equilibrium analyses. Most of these programs provide general solutions to slope stability problems with provisions for using the simplified Bishop, simplified Janbu, Bishop's Rigorous and Janbu's generalized method and/or Spencer's method of slices.

In principle, pseudo-static limit equilibrium analysis can be performed using either a total or an effective stress analysis. Problems of estimating pore water pressures induced by cyclic shearing are avoided by using a total stress analysis.

In the pseudo-static limit equilibrium analysis, a seismic coefficient is used to represent the effect of the inertia forces imposed by the earthquake upon the potential failure mass. The traditional pseudo-static limit equilibrium method of seismic stability analysis is illustrated in **Fig. 3.9.** Simplifications made in using the pseudo-static approach to evaluate seismic slope stability include replacing the cyclic earthquake motion with a constant horizontal acceleration equal to k_H x g, where k_H is the seismic coefficient, and g is acceleration of gravity, and assuming that this steady acceleration induces an inertia force $k_H W$ through the center of gravity of the potential failure mass, where W is the weight of the potential failure mass.



ANALYSIS OF EARTHQUAKE FORCE

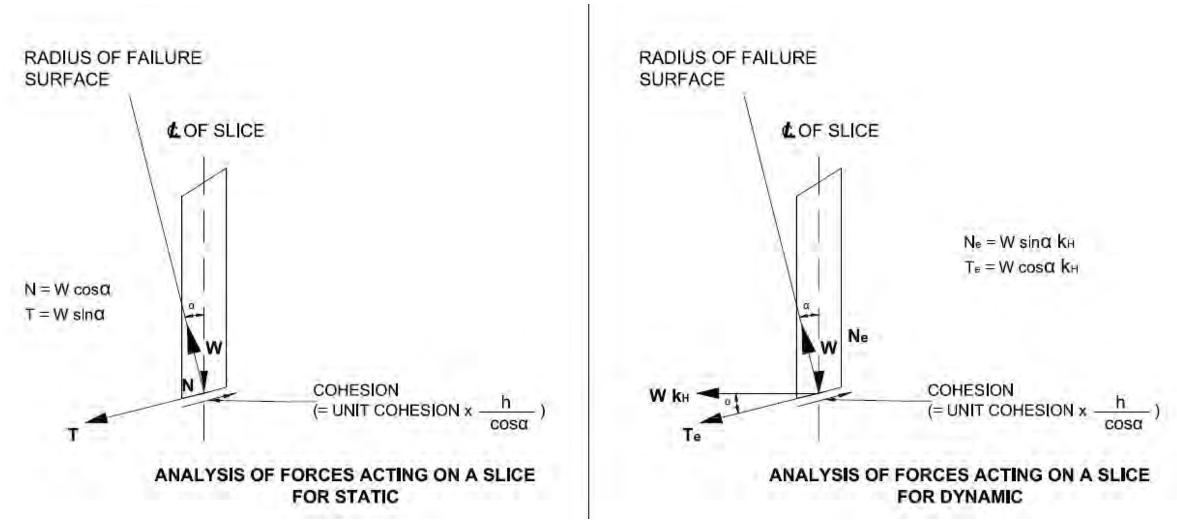


Fig. 3.9 Analyses of Earthquake Induced Forces (Circular Arc Method – Method of Slices)

$$FS = \frac{\sum[C + N\tan\Phi] - \sum(W\sin\alpha\tan\Phi\, k_H)}{\sum W\sin\alpha + \sum W\cos\alpha\, k_H}$$ (Total stresses are considered) …Eqn. 3.30

Where

FS = factor of safety;

C = cohesive resistance of the slice;

$C = c \text{ X } \frac{h}{\cos a} = \frac{ch}{\cos a}$

N = force normal to the arc of slice;

Φ = angle of internal friction;

W = weight of slice considered for driving force;

α = angle between the centre of the slice and radius of failure surface;

c = unit cohesion;

h = length of arc;

kH = horizontal seismic coefficient

Te = Wsin α . k_H and N_e = Wcos α . k_H

Design horizontal acceleration shall be 0.5 a^{max} as per the FHWA-SA-97-076 guidelines

3.9 Liquefaction

During an earthquake seismic waves travel vertically and rapid loading of soil occurs under undrained conditions since pore water has no time to move out. In saturated soils the seismic energy causes an increase in pore water pressure and subsequently the effective stress decrease. This results in loss of shear strength of soil and soil starts to behave as a fluid. This fluid is no longer able to sustain the load of structure and structure fails. This phenomenon is known as liquefaction. In other words the saturated soil which loses its strength and stiffness due to earthquake shaking is known as liquefiable soil.

The phenomenon of liquefaction can also be explained by considering the shear strength of soils. Soils fail under externally applied shear forces. The shear strength of soil is governed by effective or inter-granular stresses. Effective stress is equal to deference between total stress and porewater pressure i.e. $\sigma' = \sigma - u$

Shear strength τ of soil is given as: $\tau = c' + \sigma' \tan \varphi$ ... Eqn.3.31

It can be seen that cohesionless soil (where c = 0), such as sand, will not possess any shear strength when the effective stresses approaches zero and it will transform into liquid state. During liquefaction, soil strength and its ability to support foundation get reduced. Soils in liquefied state exert higher pressure on retaining walls, which can cause them to tilt or slide. This movement can cause settlement of the retained soil and distress of structures on the ground surface. Increased pore water pressure can also trigger slope failures in the form of slides and flows.

Unsaturated soils are not subject to liquefaction because vibratory forces from earthquakes do not cause any increase in pore water pressure in such soils.

Liquefaction generally takes place in loose fine grained sands (fines*<5%, 0.20 mm $<D_{60}$ <1.0 mm and C_u between 2 to 5) with N value less than 15. Seed (1971) concludes that in case of soil strata indicating corrected N>15, the liquefaction of soil will not possibly take place. Liquefaction potential needs to be assessed layer wise quantitatively by the procedure given below.

*In this context fines are defined as silt+clay content, particle size < 75μ as per IS 1498

3.10 Assessment of the Liquefaction Potential of Subsoil

The assessment of liquefaction potential is carried out mostly based on “Simplified Procedure” methodology developed by professors H. B. Seed & I. M. Idriss. FHWA-SA-97-077 chapter-5 adopts the same procedure. This procedure has become a Standard of Practice (SOP) all over the world. For details refer Youd. T. L. and others (2001).This is an exhaustive report and discusses the development of the various parameters involved in the calculations of liquefaction and Factor of safety.

Estimation of following two variables is required for knowing the susceptibility of soil for liquefaction. If induced Cyclic Shear Stress (CSR) is more than mobilized shear resistance (CRR) liquefaction will occur.

- Liquefaction potential or seismic resistance of the soil layers expressed in terms of CSR (Cyclic Stress Ratio) and
- Liquefaction capacity or the capacity of the soil to resist liquefaction expressed in terms of CRR (Cyclic Resistance Ratio).
- If the factor of safety as defined by CRR/CSR >1, the soil is considered as non liquefiable.

Following are the steps involved in the calculation:

Step 1: *Calculate Cyclic Stress Ratio (CSR).*

$CSR = \tau_{av}/\sigma'_{V0} = 0.65\ (a_{max}/g)\ (\sigma_{V0}/\sigma'_{V0})\ r_d$ ... Eqn.3.32

Ref. Youd. T. L. and others (2001)

Where,

a_{max} = peak horizontal acceleration at ground surface generated by earthquake;

g = acceleration due to gravity;

σ_{V0} & σ'_{V0} = total and effective vertical overburden stress respectively;

r_d = stress reduction coefficient (accounts for flexibility of soil)

$r_d = 1.0 – 0.00765z$ for $z < 9.15$ m

$r_d = 1.174 – 0.0267z$ for $9.15\ m < z < 23$ m

$r_d = 0.744 – 0.008z$ for $23\ m < z < 30$ m

$r_d = 0.50$ for $z > 30$ m

z is the depth below ground surface

Step 2: *Calculate Cyclic Resistance Ratio (CRR).*

The liquefaction resistance of soil (CRR) can be determined based on Field test results. Laboratory tests are difficult since in-situ state of stress generally cannot be re-established in the laboratory easily (except in case specialized sampling techniques such as ground freezing). Tests which are commonly used for assessment of liquefaction resistance of soil are: Standard Penetration Test (SPT); Cone Penetration Test (CPT); Shear Wave Velocity (SWVs); & Becker Penetration Test (BPT). Procedure for calculating CRR based on SPT is only discussed here, as it is the most common practice in vogue now a days:

Step 2A: *Calculation of $(N_1)_{60}$ SPT blow count normalized to an overburden Pressure of approximately 100 kPa*

$(N_1)_{60} = N_m C_N C_E C_B C_R C_S$ ... Eqn.3.33

Where, N_m is measured penetration resistance (five different corrections are applied for N_m and are listed below);

a) C_N is factor to normalize N_m to reference effective overburden pressure σ'_{V0} of approximately 100 kPa, $C_N = (Pa/\sigma'_{V0})^{0.5}$ & $C_N \leq 1.7$;

b) C_E is correction factor for hammer energy: $C_E = (ER/60)$

c) C_B correction factor for borehole diameter (1 for 65-115 mm, 1.05 for 150 mm & 1.15 for 200 mm dia.)

d) C_R correction factor for rod length =0.75 for 3.0 m to 4.0 m

=0.85 for 4.0 m to 6.0 m

=0.95 for 6.0 m to 10.0 m

=1.0 for 10.0m to 30.0 m

e) C_S is correction for sampler with or without liners (= 1 for sampler with liner & 1.1 to 1.3 for sampler without liner)

Step 2B: *Calculation of $(N_1)_{60cs}$ - SPT blow count normalized to an equivalent clean sand value.*

$$(N1)60cs = \alpha + \beta\,(N_1)_{60} \qquad \text{... Eqn.3.34}$$

Where,

$\alpha = 0$ & $\beta = 1.0$ for Fine Content (FC) $\leq 5\%$

$\alpha = \exp\,[1.76 - (190/FC^2)]$ & $\beta = [0.99 + FC^{1.5}/1000]$ for $5\% \leq FC < 35\%$

$\alpha = 5.0$ & $\beta = 1.2$ for $FC \geq 35\%$

Step 2C: *Calculation of cyclic resistance ratio CRR7.5 (for earthquake magnitude 7.5)*

$$CRR_{7.5} = 1/(34-(N_1)_{60}+((N_1)_{60}/135)+50/(10(N_1)_{60}+45)^2-(1/200) \qquad \text{... Eqn.3.35}$$

Ref. Youd. T. L. and others (2001)

The above equation is valid for $(N_1)60 < 30$. For $(N_1)60 \geq 30$, clean sand granular soils are too dense to liquefy and are classed as non-liquefiable.

Step 2D: *Deciding the magnitude scaling factor (MSF)*

The CRR evaluated in STEP 2C apply only to magnitude 7.5 earthquakes. For magnitudes of earthquake other than M = 7.5 EQ. “Magnitude Scaling Factor” have been introduced. The value MSF given by Idriss in 1995 for N= 5.5, 6, 6.5, 7, 7.5, 8, 8.5 are 2.2, 1.76, 1.44, 1.19, 1.00, 0.84 & 0.72 respectively. These MSF values are listed in Table 3.3 of Youd. T. L. and others (2001) which are reproduced in the table 3.9 below.

Step 2E: *Evaluation of CRR*

$$CRR = MSF \times CRR_{7.5} \qquad \text{... Eqn.3.36}$$

FOS against liquefaction = CRR/CSR. If FOS > 1; soil is non liquefiable.

Table 3.9 Magnitude Scaling Factor Values Defined by Various Investigators

Magnitude M	Seed and Idriss (1982)	Idriss
5.5	1.43	2.20
6.0	1.32	1.76
6.5	1.19	1.44
7.0	1.08	1.19
7.5	1.00	1.00
8.0	0.94	0.84
8.5	0.89	0.72

Table 3.10 Typical Computation of Liquefaction Potential by simplified Method

Depth below E.G.L., m	1.50	3.00	4.50	6.00	7.50	9.00	10.50	13.50	16.50	19.50
Type of strata	SM	SP-SM	SP-SM	SP	SP	SM	SP-SM	SP-SM	SP	SP-SM
Observed SPT value	12	8	6	4	7	4	22	100	32	31
Saturated density(t/m^3)	1.90	1.80	1.80	1.70	1.80	1.70	2.00	2.00	2.00	2.00
Submerged density(t/m^3)	0.90	0.80	0.80	0.70	0.80	0.70	1.00	1.00	1.00	1.00
Fine Content (%)	27.00	7.00	7.00	2.00	2.00	13.00	5.00	5.00	3.00	7.00
Stress reduction coefficient(rd)	0.99	0.98	0.97	0.95	0.94	0.93	0.89	0.81	0.73	0.65
Total overburden pressure(σ_{V0})t/m^2	2.85	5.55	8.25	10.80	13.50	16.05	19.05	25.05	31.05	37.05
Effective overburden pressure(σ'_{V0})	1.35	2.55	3.75	4.80	6.00	7.05	8.55	11.55	14.55	17.55
Cyclic stress ratio(CSR)	0.33	0.33	0.33	0.33	0.33	0.33	0.31	0.28	0.24	0.22
C_N	1.70	1.70	1.63	1.44	1.29	1.19	1.08	0.93	0.83	0.75
C_E	1.17	1.17	1.17	1.17	1.17	1.17	1.17	1.17	1.17	1.17
C_B	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
C_R	0.75	0.80	0.85	0.95	0.95	0.95	1	1	1	1
C_S	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SPT Corrected($N1)_{60}$	18.74	13.33	10.20	6.72	10.52	5.54	29.15	113.98	32.50	28.67
α	4.48	0.12	0.12	0.00	0.00	1.89	0.00	0.00	0.00	0.12
β	1.13	1.01	1.01	1.00	1.00	1.04	1.00	1.00	1.00	1.01
$(N1)_{60cs}$	25.66	13.56	10.41	6.72	10.52	7.64	29.15	113.98	32.50	29.03
$CRR_{M=7.5}$	0.31	0.15	0.12	0.09	0.12	0.09	0.42	NA	NA	0.41
CRR	0.36	0.17	0.14	0.10	0.14	0.11	0.50	NA	NA	0.49
FOS	1.12	0.52	0.42	0.30	0.42	0.33	1.60	>1	>1	2.28
Conclusion	NL	L	L	L	L	L	NL	NL	NL	NL

Note:

a) NL means NON-LIQUEFIABLE SOIL; L means LIQUEFIABLE SOIL

b) The project site falls in Zone-IV. A maximum earthquake intensity of 7.0 has been considered in the analysis.

c) The peak ground acceleration PGA considered as a_{max} /g=0.24(for Zone IV)

d) C_E= Correction for hammer energy ratio=ER/60. ER for Rope and pulley system=70%, Hence C_E=70/60=1.167

e) Borehole diameter=150mm, Hence C_B=1.05

f) C_s= Correction for standard sampler=1.0

g) Magnitude Scaling Factor(MSF)=1.19 has been taken in the analysis (Recommended revised MSF)

3.11 Distress Caused by Liquefaction and its Mitigation

In case of embankments, the ground beyond the toe may lose shear strength and suffer lateral flow. This may cause part of the embankment to settle. In case of large settlements, cracks may appear on the top crest of the embankment. To a certain extent these damage may be controlled by

i) Providing berms at the toe of the embankment or increasing the berm width wherever feasible.

ii) The subsoil may be densified by various methods of compaction including dynamic compaction, so that N value is higher than the liquefiable limits

iii) Adopting ground improvement methods such as stone columns or compacted granular columns basal reinforcement, pile supported basal reinforcement in the ground.

In general these methods may be costly and may only be adopted after detailed study. Experience concerning liquefaction and its control in case of embankments is scant. Since liquefaction of embankments is site-specific problem, generalized solution may not be feasible. Embankments are structures of low risk, and hence in certain cases, it may be economical to allow some distress/failures due to liquefaction and earthquakes and same can be repaired subsequently.

In case of structures, deep foundations resting on layers not susceptible to liquefaction are adopted to ensure their stability. This solution may not be economical for highway embankments.

Factor of safety value of 1.0 can be adopted when liquefaction analysis is carried out for design of embankments.

ANNEXURE 3.1

Table 3.11 Zone Factor

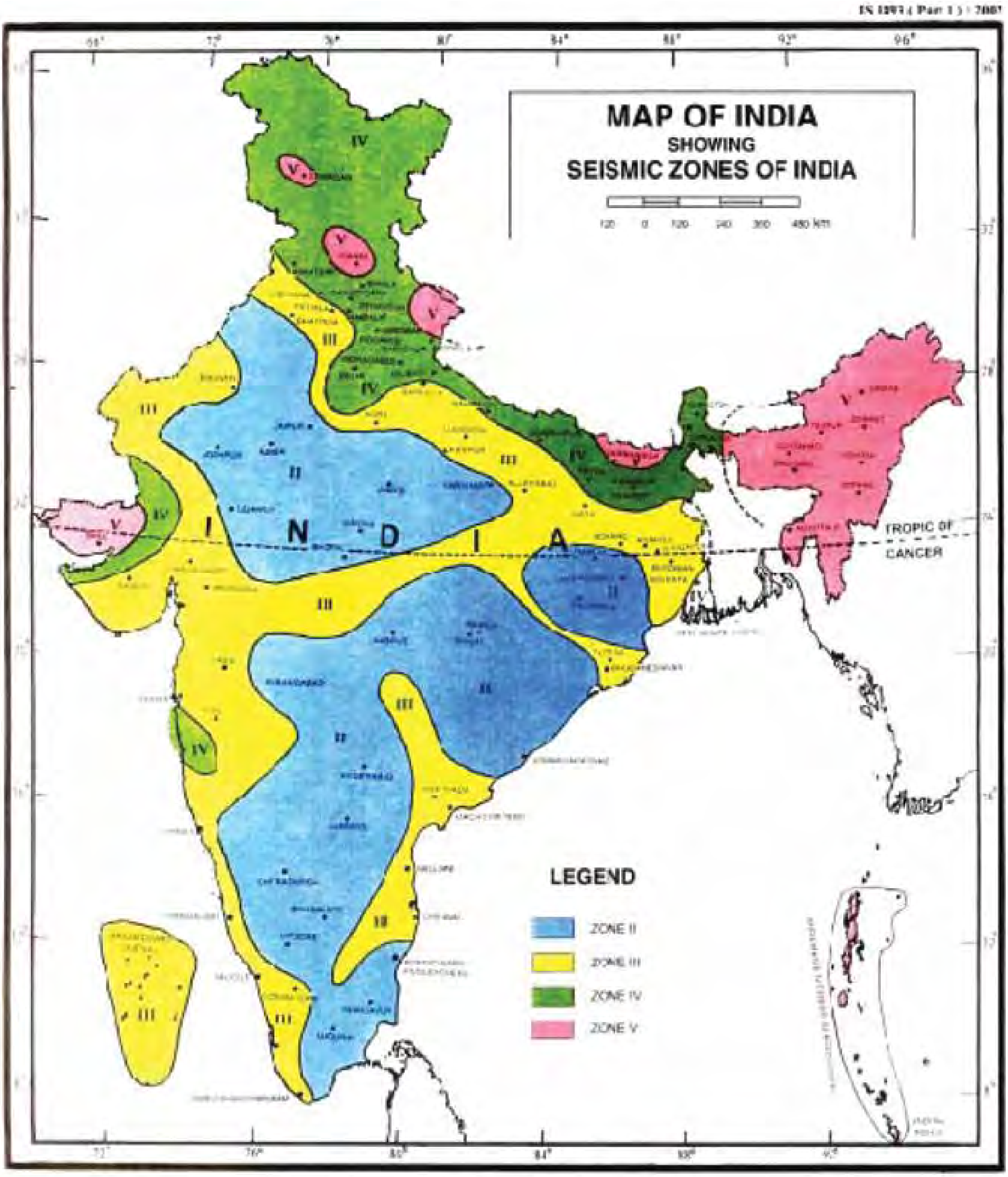
Seismic Zone	II	III	IV	V
Seismic	Low	Moderate	Severe	Very Severe
z	0.10	0.16	0.24	0.36

Table 3.12 Zone Factors for Some Important Towns

Town	Zone	Zone Factor, z	Town	Zone	Zone Factor, z
Agra	III	0.16	Chitradurga	II	0.10
Ahmedabad	III	0.16	Coimbatore	III	0.16
Ajmer	II	0.10	Cuddalore	III	0.16
Allahabad	II	0.10	Cuttack	III	0.16
Almora	IV	0.24	Dehra Dun	IV	0.24
Ambala	IV	0.24	Dharampuri	III	0.16
Amritsar	IV	0.24	Delhi	IV	0.24
Asansol	III	0.16	Durgapur	III	0.16
Aurangabad	II	0.10	Darbhanga	V	0.36
Bahraich	IV	0.24	Darjeeling	IV	0.24
Bangalore	II	0.10	Dharwad	III	0.16
Barauni	IV	0.24	Gangtok	IV	0.24
Bareilly	III	0.16	Guwahati	V	0.36
Belgaum	III	0.16	Goa	III	0.16
Bhatinda	III	0.16	Gulbarga	II	0.10
Bhilai	II	0.10	Gaya	III	0.16
Bhopal	II	0.10	Gorakhpur	IV	0.24
Bhubaneswar	III	0.16	Hyderabad	II	0.10
Bhuj	V	0.36	Imphal	V	0.36
Bijapur	III	0.16	Jabalpur	III	0.16
Bikaner	III	0.16	Jaipur	II	0.10
Bokaro	III	0.16	Jamshedpur	II	0.10
Bulandshahr	IV	0.24	Jhansi	II	0.10
Burdwan	III	0.16	Jodhpur	II	0.10
Cailcut	III	0.16	Jorhat	V	0.36
Chandigarh	IV	0.24	Kakrapara	III	0.16
Chennai	III	0.16	Kalapakkam	III	0.16

Town	Zone	Zone Factor, z	Town	Zone	Zone Factor, z
Kanchipuram	III	0.16	Pune	III	0.16
Kanpur	III	0.16	Raipur	II	0.10
Karwar	III	0.16	Rajkot	III	0.16
Kohima	V	0.36	Ranchi	II	0.10
Kolkata	III	0.16	Roorkee	IV	0.24
Kota	II	0.10	Rourkela	II	0.10
Kurnool	II	0.10	Sadiya	V	0.36
Lucknow	III	0.16	Salem	III	0.16
Ludhiana	IV	0.24	Simla	IV	0.24
Mumbai	III	0.16	Sironj	II	0.10
Mysore	II	0.10	Solapur	III	0.16
Madurai	II	0.10	Srinagar	V	0.36
Mandi	V	0.36	Surat	III	0.16
Mangalore	III	0.16	Tarapur	III	0.16
Monghyr	IV	0.24	Tazpur	V	0.36
Moradabad	IV	0.24	Thane	III	0.16
Nagpur	II	0.10	Thanjavur	II	0.10
Nagarjunasagar	II	0.10	Thiruvananthapuram	III	0.16
Nainital	IV	0.24	Tiruchirappali	II	0.10
Nasik	III	0.16	Tiruvennamalai	III	0.16
Nellore	III	0.16	Udaipur	II	0.10
Osmanabad	III	0.16	Vadodara	III	0.16
Panjim	III	0.16	Varanasi	III	0.16
Patiala	III	0.16	Vellore	III	0.16
Patan	IV	0.24	Vijayawada	III	0.16
Pilibhit	IV	0.24	Vishakhapatnam	II	0.10
Pondicherry	II	0.10			

Zone Factor is a factor to obtain the design spectrum depending on the perceived maximum seismic risk characterized by Maximum Considered Earthquake in the zone in which the structure is located. Reference can be made to IS 1893(part 1): Criteria for Earthquake Resistant Design of Structures.



Note: Towns falling at the boundary of zones demarcation line between two zones shall be considered in High Zone.

Fig. 3.10 Seismic Zoning Map of India

CHAPTER 4

SETTLEMENT ANALYSIS

4.1 General

4.1.1 Highway embankments constructed over soft and compressible soils undergo settlements. Such settlements cause unevenness of riding surface and eventually cracking of pavements. This leads to increase in maintenance cost of pavements. High quality asphaltic pavements currently in use are susceptible to failures even due to small settlements. Sudden depression at bridge approach also forms a major maintenance problem. It is therefore essential to pay attention to this aspect on high embankments so that the post construction settlement is contained within reasonable limits.

4.1.2 Settlement refers to the decrease in void ratio of the fill material constituting the body of the embankment and/or the subsoil constituting the foundation of the embankment. The process of reduction in the voids, accompanied by the expulsion of water under load, is familiarly known as "the process of consolidation". The settlements are calculated based on Terzaghi's theory of consolidation.

4.2 Consolidation of Sub-Soil

4.2.1. Most of the settlements of highway embankment are due to deformation of sub-soil under the embankment loads. This consolidation is traditionally considered to have three components: initial settlement; consolidation settlement; and secondary settlement. Immediately upon loading, a saturated cohesive soil deforms without movement of pore water; this is called initial settlement or settlement due to shear at constant volume since the volumetric compressibility of saturated clay is essentially zero. Subsequently, time dependent settlement occurs as pore water flows out of the soil and load is transferred to the soil skeleton. The part of the time dependant settlement, where rate is controlled by the rate of dissipation of the excess pore water pressure, is called consolidation settlement and it continues until the pore water pressure generated by the loading is in equilibrium with hydraulic boundary conditions. Finally, a time dependent settlement occurs that is not controlled by the rate of dissipation of excess pore water pressure. This component is referred to as secondary settlement or compression. The principal settlement relevant for highway embankments is consolidation settlement.

4.2.2 Initial settlement does not have much of a practical significance in case of highway embankments. However, if a designer is interested, this settlement can be calculated by use of charts prepared besides others by Giroud (1968).

4.2.3 Consolidation Settlement: Where undisturbed samples are obtainable, settlements can be predicted by conducting laboratory consolidation tests applying the appropriate loads. In the interpretation of laboratory consolidation tests it is common practice to include the immediate and secondary compressions of all previous increments in the calculations of the consolidated void ratio under the subsequent increment. This practice compensates (in a general way) for the immediate settlements that occur in the field although the time rate relationships are different.

4.2.4 For the many cases that arise in practice in which secondary compressions and creep at constant volume are not of great importance in which the compressible stratum is either deeply buried between layers of stiffer soil, or in thin layers compared to the size of the loaded area, the ultimate settlements can be calculated by one-dimensional theory. The error on this account in the prediction of consolidation settlements will seldom exceed ± 25 percent in the case of normally consolidated deposits. For over consolidated deposits, the percentage error may be much higher but the total settlements will always be much less.

4.2.5 in as much as volume changes are considered to be one dimensional, the apparent consolidation settlement Si of each segment of clay stratum can be computed using the equation.

$$S_i = \frac{C_c}{1+e_0} D \log_{10} \frac{P_o + \Delta P}{P_o} \qquad \text{...Eqn 4.1}$$

The apparent total consolidation S is

$$S=\sum_{i=1}^{i=n} S_i \qquad \text{...Eqn 4.2}$$

ΔP =Load increments

P_O= initial effective stress at mid depth of compressible clay

D= full depth of clay stratum

Cc=compression index evaluated over the range $P_O + \Delta P$

e_O= initial void ratio

The magnitude of total settlement of foundation strata is determined by summation of consolidation in the various strata forming the foundation. To allow for the variation of pressure with different depths, the substrata is generally divided into thin layers and settlements calculated for each layer separately before totaling. The first task is therefore to identify the number and the thickness of the layers to be considered. This can be done with the help of a borehole log.

4.2.6 *Prediction of Consolidation Settlements*

The purpose of performing consolidation tests is to determine the stress strain properties of the soil and thus they allow predicting consolidation settlements in the field. The computations are performed by projecting the laboratory test results (as contained in the parameters C_c, C_r ,e_o, σ_c') back to the field conditions .For simplicity, the discussions of consolidation settlement predictions in this chapter consider only the case of one dimensional consolidation and only the ultimate consolidation settlement will be computed. One dimension consolidation means only vertical strains occur in the soil. In this context, a compressible stratum refers to the strata that have a C_c or C_r large enough to contribute significantly to the settlement. The most common one dimensional consolidation problems are those that evaluate settlement due to the placement of a long and wide fill and due to the wide spread lowering of the ground water table.

The ultimate consolidation settlement is the value after all of the excess pore water pressures have dissipated, which may require many years or even decades. The ultimate consolidation settlement for normally consolidated and over-consolidated soil can be determined by the following formulas.

Case I normally consolidated soil

If $\sigma_{z0}' = \sigma_c'$ the soil by definition is normally consolidated. The initial and final conditions are shown in **Fig. 4.8** and the compressibility is defined by C_c, the slope of the virgin curve.

For normally consolidated soil the ultimate consolidation is calculated as per the following equation

$$\delta_{c,ult} = \sum \frac{C_c}{1+e_o} H\log\left(\frac{\sigma'_{zf}}{\sigma'_{zo}}\right) \qquad \text{...Eqn 4.3}$$

Where H= thickness of the soil layer

When using the above equation, compute σ_{z0}' and σ_{zf}' at the midpoint of each layer.

Case IIA Over-consolidated Soils ($\sigma_{z0}' < \sigma_{zf}' \leq \sigma'_c$)

If neither σ_{z0}' nor σ_{zf}' exceed σ'c, the entire consolidation process occurs on the recompression curve as shown in **Fig 4.1**. The analysis is thus identical to that for normally consolidated soils except we use the recompression index, C_r, instead of the compression index, C_c.

$$\delta_{c,ult} = \sum \frac{C_r}{1+e_0} H\log\left(\frac{\sigma'_{zf}}{\sigma'_{z0}}\right) \qquad \text{...Eqn 4.4}$$

Case IIB Over-consolidated Soils (σz0'< σ'c < σzf')

If the consolidation process begins on the recompression curve and ends on the virgin curve, as shown in **Fig 4.1**, then the analysis must consider both Cc and Cr:

$$\delta_{c,ult} = \sum \left[\frac{C_r}{1+e_0} H\left(\frac{\sigma'_c}{\sigma'_{z0}}\right) + \frac{C_c}{1+e_0} H\log\left(\frac{\sigma'_{zf}}{\sigma'_c}\right)\right] \qquad \text{...Eqn 4.5}$$

This condition is quite common, because many soils that might appear to be normally consolidated from a geologic analysis actually have a small amount of over consolidation (Mesri et al., 1994).

When using equation, σ_{z0}', σ'_c, σ_{zf}' must be computed at the midpoint of each layer.

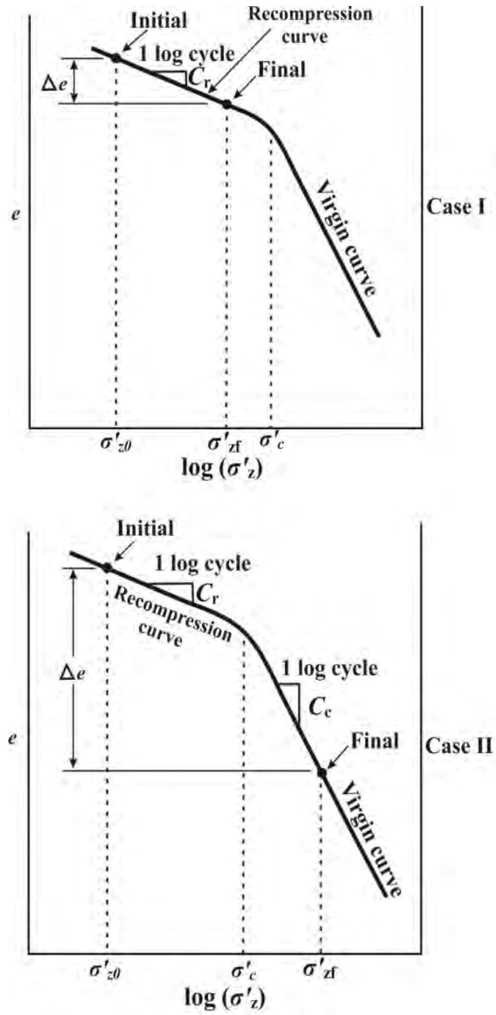


Fig. 4.1 Consolidation of Over-consolidated Soil

4.2.7 Coefficient of compressibility $a_{v,}$ coefficient of volume compressibility $m_{v,}$ coefficient of consolidation $C_{v,}$ and coefficient of permeability K, are related to one another by the relationship:

$$a_v = \frac{e_0 - e}{\Delta P} \qquad \text{...Eqn 4.6}$$

$$m_0 = \frac{a_0}{1+e_0} \qquad \text{...Eqn 4.7}$$

$$C_v = \frac{K(1+e}{a_v Y_w} \qquad \text{...Eqn 4.8}$$

$\Delta S = m_v.D.\Delta P$ ...Eqn 4.9

Y_w is the unit weight of water and e is the final void ratio.

4.2.8 For clays which are normally loaded and which are of ordinary sensitivity, experience has shown that the Compression Index C_c is related to liquid limit of the soil by statistical relationship C_c=0.009(L.L-10). It may therefore, be prudent to take advantage of this relationship to evaluate C_c in case of normally loaded clays without resorting to extensive consolidation tests. However, consolidation tests would be necessary where the embankment is built on deep deposit on soft clay and where both magnitude and rate of settlement have to be determined to formulate the method of foundation treatment and or the method of construction. Consolidation tests would also be necessary in the case of clays which are known to be over consolidated or preloaded and sometimes even to determine whether clay is normally loaded or preloaded. For over consolidated clays, the C_c value obtained from laboratory consolidation tests is apt to be higher than for normally consolidated clay. The prediction of settlements using the C_c value ignoring the fact that the clay had been pre-consolidated clay and treating it as though it had been normally consolidated would result in an error on the conservative side and the error may happen to be appreciable if the degree of over-consolidation is particularly high.

4.2.9 Rate of consolidation settlement: The time't' required to reach a certain percentage of consolidation in a stratum is given by the equation.

$$t = \frac{TH^2}{C_v} \qquad \text{...Eqn 4.10}$$

Where T= Time factor corresponding to the degree of consolidation

c_v = Coefficient of consolidation for range of stress applicable

H = the length of effective drainage path. For one-way drainage H=D and for two way drainage H=D/2

D = Depth of compressible strata

4.2.10 The value of time factor T for various degrees of consolidation and different drainage conditions is taken either from tables 4.1 and 4.2 or **Fig 4.2** which gives the relationship between the dimensionless time factor T and average percentage of consolidation 'U' for various typical boundary conditions. The choice of curve in **Fig. 4.2** or column in Tables 4.1 and 4.2 depends on the type of drainage conditions and porewater pressure distribution diagram. For typical embankment for clay layer in the subsoil is situated below a sand blanket on the top and gravely strata below, the drainage is two-way and curve 1 in **Fig. 4.2** or column 4 of table 4.1 should be used to evaluate T. In case there is no gravelly layer underlying the clay stratum, drainage will occur in only one direction towards the sand blanket and curve 2 in **Fig 4.2** or column 3 of Table 4.2 would be appropriate.

4.3 Consolidation Settlements Vis-a-Vis Loading

4.3.1 The loading period is generally proceeded by excavation and then the load is applied at a varying rate. Frequently, the loading is approximated by a uniform rate and the settlement at the end of the loading period is assumed to be the same as that which would have resulted in half the loading period had all the load been applied at once. In principle, the rate of primary consolidation can be calculated for any variation of loading to any degree of precision desired by splitting up the increase in load into small increments, calculating the rate of settlement for each increment independently, and adding the resulting values. This approach is too cumbersome for practical purposes.

4.3.2 The settlement analysis can give reasonably close forecast of the amount and rate of settlement provided care and judgment are exercised in the selection of samples and interpretation of test results. Bore-hole records covering the whole site should be carefully studied. If the soil strata over the site are similar, a representative soil profile can be drawn for the site and the average values of C_c marked there on for each stratum. The choice of a representative soil profile involves careful judgment. In the case of thick clay strata, compressibility must not be assumed as constant throughout. Normally loaded clays usually show progressively decreasing compressibility with increasing depth.

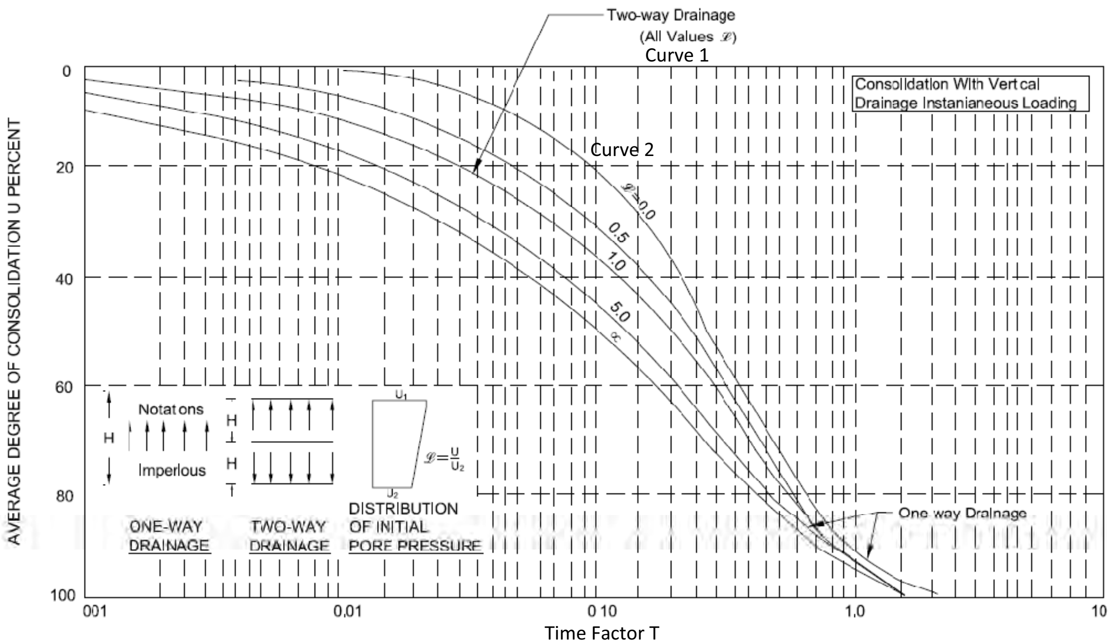


Fig. 4.2 Time Factors for Consolidation Analysis

4.3.3 Rate of settlement will be maximum when the embankment is saturated and the subsoil is buoyant. These parameters are liable to change with season. The embankment load will increase during rainy season when density may be close to saturated density and will reduce dry seasons. Thus settlement will be occurring at different rates during different seasons and the calculations of total time required for settlement will have to be modified to account for these variations.

In order to calculate the settlement, it is necessary to find out stress distribution with soil due to the embankment loading.

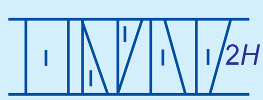
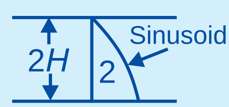
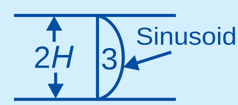
4.4 Settlement Analysis: Determination of Stresses within the Foundation

4.4.1 In equation 4.1 the value of ΔP depends on the contact pressure and the least dimension of the loaded area. Because of the considerable base width of most embankments, the stress beneath the centre of embankment usually decreases slowly with depth. The general approach for determining the stresses below the embankment is to integrate the Boussinesq solution for stresses due to single vertical load on a semi infinite homogenous isotropic mass. For common embankment problems, influence charts developed by Osterberg (1957) are useful and provide ready solution. These are reproduced here for ease of reference. The stress given by the chart is the vertical stress directly under the vertical face of an embankment of infinite extent. Vertical stresses for any point in the foundation can be found by super-imposition. For stresses under a corner, such as the vertical face of an embankment ending abruptly against the wall, stresses are one half of those given in the chart.

4.4.2 Several other solutions for determining the vertical stresses under an embankment are also available. For instance Middlebrooks (1936) Newmark (1941, 1942), Perloff et al (1967), W. Steinbrenner (1934) and R.E. Fadum etc have developed solutions for such cases.

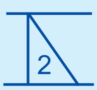
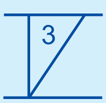
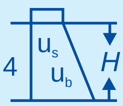
4.4.3 The design load used to evaluate the settlement and stability is the weight of overlying embankment and pavement materials. Except for the upper one meter or so, embankments are not seriously affected by traffic loads and as such traffic loads are generally neglected. When designing approaches, if the abutment for floating span rests on the embankment, load due to this however has to be considered. Another requirement in the case of floating abutment is to determine the bearing capacity near the slope according to procedure recommended by Meyerhof (1957)

Table 4.1 Time Factor- Degree of Consolidation Values for Two-way Drainge

Time Factor	Degree of Consolidation, μ		
			
1	2	3	4
0.000	0.0000	0.0000	0.0000
0.004	0.0795	0.0649	0.0098
0.008	0.1038	0.0862	0.0195
0.012	0.1248	0.1049	0.0292
0.020	0.1598	0.1367	0.0481
0.028	0.1889	0.1638	0.0667
0.036	0.2141	0.1876	0.0850
0.048	0.2464	0.2196	0.1117
0.060	0.2764	0.2481	0.1376
0.072	0.3028	0.2743	0.1628
0.083	0.3233	0.2967	0.1852
0.100	0.3562	0.3288	0.2187
0.125	0.3989	0.3719	o.2654

0.150	0.4370	0.4112	0.3093
0.167	0.4610	0.4361	0.3377
0.175	0.4718	0.4473	0.3507
0.200	0.5041	0.4809	0.3895
0.250	0.5622	0.5417	0.4603
0.300	0.6132	0.5950	0.5230
0.350	0.6582	0.6421	0.5783
0.40	0.6973	0.6836	0.6273
0.50	0.7640	0.7528	0.7088
0.60	0.8156	0.8069	0.7725
0.70	0.8559	0.8491	0.8222
0.80	0.8874	0.8821	0.8611
0.90	0.9119	0.9079	0.8915
1.00	0.9313	0.9280	0.9152
2.00	0.9942	-	-
∞	1.0000	1.0000	1.0000

Table 4.2 Time Factor- Degree of Consolidation Values for Two-way Drainge

	Degree of Consolidation, μ			
Time Factor				
1	2	3	4	5
0.000	0.0000	0.0000	0.0000	$\mu_T = \frac{2\mu_R + \mu_{D}(\eta - 1)}{\eta + 1}; \quad \eta = \frac{u_b}{u_s}$
0.004	0.0795	0.0085	0.1505	
0.008	0.1038	0.162	0.1914	
0.012	0.1248	0.0241	0.2255	
0.020	0.1598	0.0400	0.2796	
0.028	0.1889	0.0560	0.3218	
0.036	0.2141	0.0720	0.3562	
0.048	0.2464	0.0950	0.3978	
0.060	0.2764	0.1198	0.4330	
0.072	0.3028	0.1436	0.4620	
0.083	0.3233	0.1646	0.4820	
0.100	0.3562	0.1976	0.5148	
0.125	0.3989	0.2442	0.5536	
0.150	0.4370	0.2886	0.5854	
0.167	0.4610	0.3174	0.6046	
0.175	0.4718	0.3306	0.6130	
0.200	0.5041	0.3704	0.6378	

0.250	0.5622	0.4432	0.6812	
0.300	0.6132	0.5078	0.7186	
0.350	0.6582	0.5649	0.7515	
0.40	0.6973	0.6154	0.7792	
0.50	0.7640	0.6994	0.8286	
0.60	0.8156	0.7652	0.8660	
0.70	0.8559	0.8165	0.8953	
0.80	0.8874	0.8566	0.9182	
0.90	0.9119	0.8880	0.9358	
1.00	0.9313	0.9125	0.9501	
2.00	0.9942	0.9930	0.9960	
∞	1.0000	1.0000	1.0000	

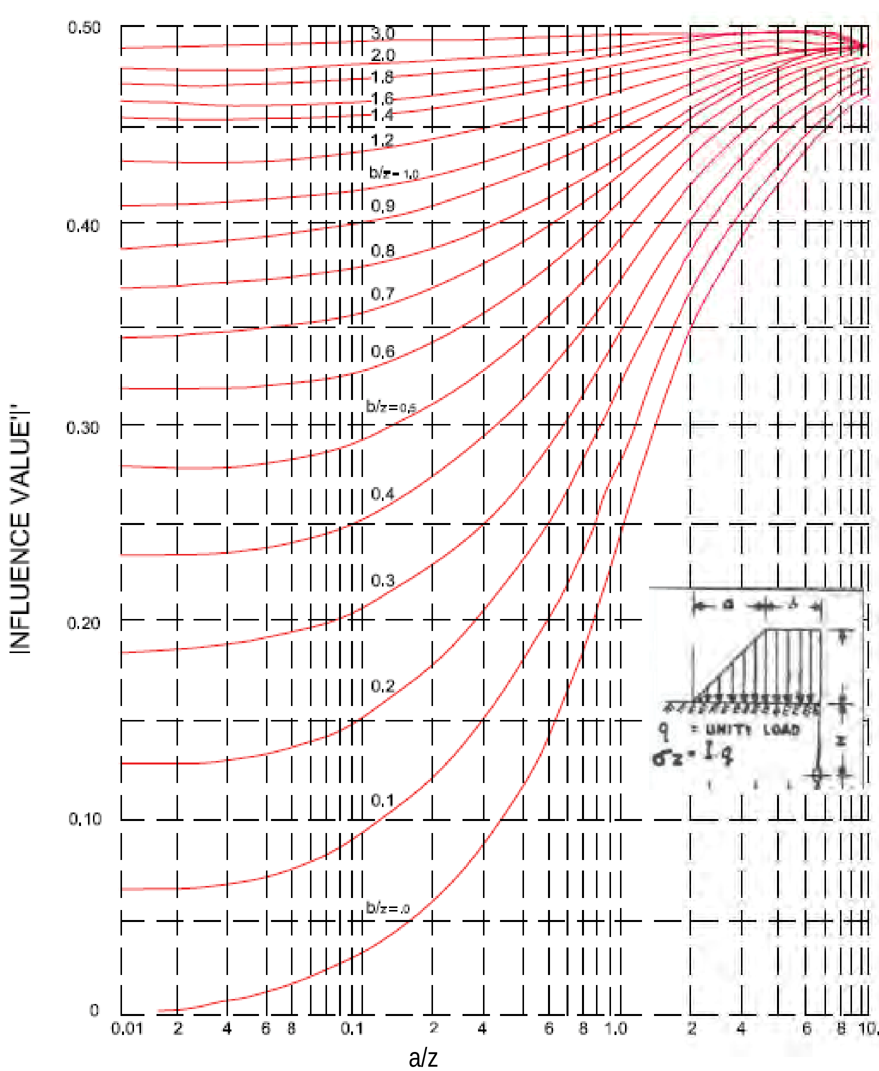


Fig. 4.3 Influence Chart for Vertical Stress Embankment Loading Infinite Extent Boussinesq Case

Source: Proceedings of Fourth International Conference on Soil Mechanics and Foundation Engineering Vol.1

Extracts from Influence values for vertical stresses in a semi-infinite mass due to embankment loading. (Page 393) by Br. J.O Osterberg (Illinois, U.S.A)

Illustrations to use Osterberg's chart for calculation of stresses.

1) Find the vertical stress beneath an embankment at the location shown in figure 'a'

For the left side, $\frac{a}{z} = 1, \frac{b}{z} = 0.5$,

And from chart I = 0.397,

Similarly for Right Side, I = 0.478

And the total I = 0.397+0.478 =0.875

The vertical stress is then $\sigma_z = 0.875q$.

2) Find the vertical stress beneath an embankment at the location shown in figure 'b'

For dashed and solid portion,

$\frac{a}{z} = 1, \frac{b}{z} = 4$, and I = 0.499,

Subtract the influence value for dashed portion,

$\frac{a}{z} = 1, \frac{b}{z} = 1$, and I = 0.455,

The stress then is $\sigma_z = 0.044q$

3) Find the vertical stress beneath the embankment at the location shown in figure 'c'

Stress due to abc, is the same as due to cde, since one is plus and the other is minus, the stress is same as the embankment was vertical at b

$\frac{a}{z} = 1, \frac{b}{z} = 2.5$, and from chart I = 0.492

The stress then is $\sigma_z = 0.4q$

Notes:

1) The stress given by the chart is the vertical stress directly under the vertical face of an embankment of infinite extent

2) For stress under a corner such as under the vertical face of embankment ending abruptly against a wall, the stresses are 0.5 of those given in the chart.

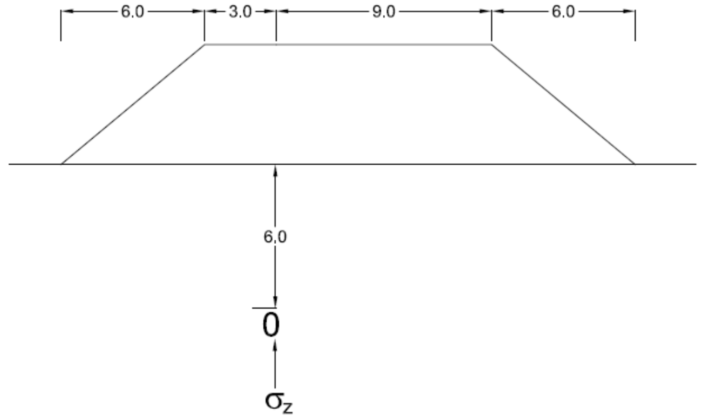


Fig (a)

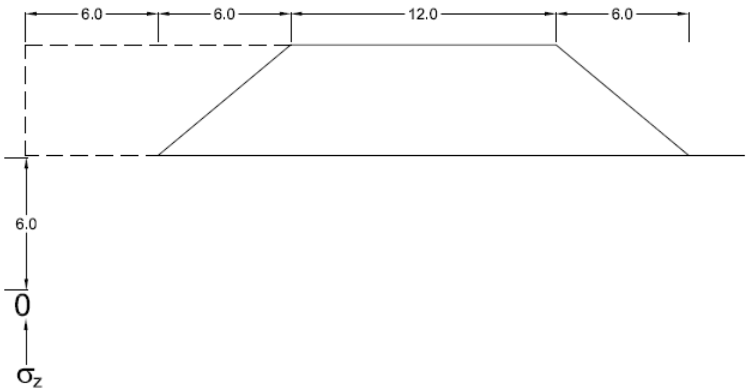


Fig (b)

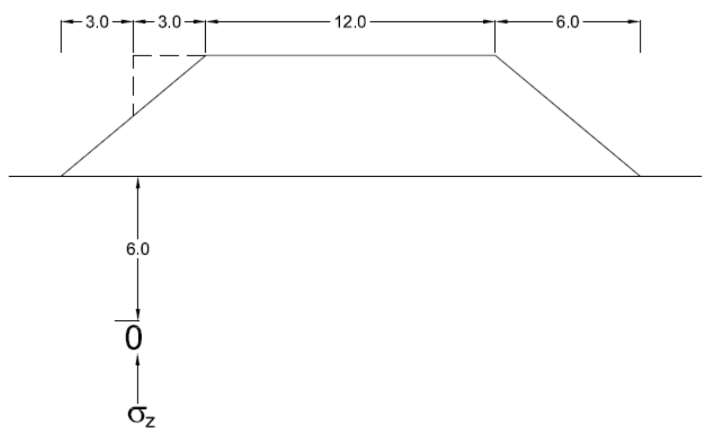


Fig (c)

4.5 Tolerable Settlements

Settlements of embankments have the following components:

I. Settlement due to self-weight of fill.

II. Settlement of the subsoil.

Settlement of embankments and more importantly, rate of settlement consideration are of relevance due to the effect such settlements will have on the pavement performance, especially in terms of developing uneven and rough riding surface.

4.5.1 *Settlement due to Self-Weight of Fill*

Settlement within the fill due to self-weight depends on the type of fill material, degree of compaction and fills height. Materials which shall not be used for embankment fill are listed in Clause.305.2.1.1 of the MORTH "Specifications for Road and Bridge works. When acceptable fill materials, are used and the fill is well compacted to the minimum requirements specified in Cl.305.2.1.5 of the publication referred to above, settlements of the embankments due to self-weight are generally not of serious concern.

Settlements progress as the embankment is built and post construction or residual settlements are not likely to occur.

If the fill material is not well compacted during the course of construction, it has been the common experience that such fills will continue to settle for a long time and there is no easy and economical solution to control such settlements of the fill, in the post construction phase. Hence the compaction of the fill ab-initio is very important and shall be adhered to in practice and all quality control steps shall be taken as per Clause. 903.2, of MORTH "Specifications for Road and Bridge Works".

Reference may also be made to Table 7, I.S.- 1498- "Classification and Identification of soil for general Engineering Purposes" where in Col.4 the compressibility of different soil types when compacted and saturated is given. The Table is useful to understand the behavior of fill material proposed to be used, as well as in the selection of fill materials.

4.5.2 *Settlement of Subsoil*

Subsoil layers also experience settlement due to embankment loads the magnitude and time rate at which these settlements progress depends on the nature of the subsoil.

i) Where subsoil layers are essentially low plastic or non-plastic soils, with adequate bearing capacity, settlements in the subsoil progress as the embankment is built up. Thus there would be none or very small residual settlements at the end of construction. Hence, such settlements are not of concern.

ii) Where the subsoil consists of soft compressible clay layers in saturated condition, large settlements world occur. These settlements follow the "Terzaghi's theory of consolidation" and require long time period for completion. It is essential that such subsoil conditions are identified at or prior to design stage and suitable ground improvement technique is adopted if required (Suitable ground improvement techniques have been discussed in chapter 5). Such techniques accelerate the settlement rate based on the design adopted. However even at the end of the design waiting period after ground improvement method adopted, some settlements may continue to occur. These may be termed "residual settlements".

The allowable limit for such residual settlements may be considered as 300mm. In general, these settlements progress at very slow rate. Hence it would be economical to allow such settlements to run their course than aim a design which has "negligible" residual settlements. This observation is particularly relevant where PVDs or stone columns are adopted for ground improvement.

The designer may indicate the amount of residual settlement expected and time period for the same while designing embankments over soft subsoil deposits.

In case of embankments on soft clays, where residual settlements as mentioned above are difficult to avoid, rigid pavements may not be suitable. Appropriate type of flexible pavement may be provided avoiding costly BC layer, till the rate of progress of residual settlement reduces to less than 25-30 mm/year.

CHAPTER-5

GROUND IMPROVEMENT

5.1 Introduction

Ground improvement technologies are geotechnical construction methods used to alter or improve poor ground conditions in order that construction of embankment can meet project performance requirements in terms of (a) Improved stability of slopes, (b) reduced settlements, and (c) Improved bearing capacity of the subsoil. Where the soil is susceptible to liquefaction, its resistance to liquefaction can also be improved by some of the ground improvement methods listed below. Ground improvement is called for where construction on untreated ground may lead to either excessive settlement or failure in rotational mode/bearing capacity.

Ground improvement has one or more of the following main functions:

- To increase bearing capacity, shear or frictional strength,
- To increase density,
- To control deformations,
- To accelerate consolidation,
- To decrease imposed loads,
- To provide lateral stability,
- To form seepage cut-offs or fill voids,
- To increase resistance to liquefaction and,
- To transfer embankment loads to more competent layers.

5.2 Ground Improvement Techniques

Adopting suitable ground improvement technique depends on the type of soil, construction feasibility, cost benefits and performance requirements in relation to slope stability, bearing capacity, settlement and liquefaction.

State of art reports IRC-HRB: SR-13, SR-14 and IRC:113 deal with ground improvement in the context of embankment design and construction in an extensive manner. Contents of these publications are briefly mentioned below,

- SR-13- describes the fundamental concepts, properties of soft clays, building of embankments using stage construction.
- SR-14- deals with various ground improvement methods including vertical drains (PVDs), lime columns, stone columns, geosynthetics and dynamic consolidation.

- IRC:113 presents design and construction guidelines for construction of high embankments on soft soils and concerns mainly with geosynthetic basal reinforcement.

Reference shall be made to the publications listed above for design and construction purposes.

In this chapter the above techniques are mentioned briefly. Further, ground improvement techniques not mentioned in the above publications are also presented briefly.

Ground Improvement methods are presented in subsequent sessions as listed below.

- Partial or total removal of undesirable material (section 5.2.1)
- Use of light weight material as embankment fills with poor base soil replacement (section 5.2.2.)
- Stage wise construction for embankment with poor base soil replacement (section 5.2.3.)
- Soil Stabilisation by lime, lime Pozzolana, cement and other chemicals (section 5.2.4.)
- Preloading (section 5.2.5.)
- Methods based on consolidation of subsoil like use of PVDs (section 5.2.6.)
- Stone Columns, reinforced, grouted and encased stone columns with specific reference to vibro replacement and vibro displacement (section 5.2.7.)
- Densification methods referring dynamic compaction and vibro compaction (section 5.2.8.)
- Compaction grouting. (Section 5.2.9.)
- Dynamic deep replacement (section 5.2.10.)
- Basal reinforced embankments on soft ground (section 5.2.11.)
- Pile supported basal reinforced embankments (section 5.2.12.)

Choice of a particular ground improvement method depends on

i) Nature and extent of the problem in terms of initial strength, depth of soft clay, and height of embankment to be built.

ii) Time available for ground improvement and the cost involved also play an important role.

Most of the ground improvement methods suggested above involve detailed construction and design methodologies. Hence a careful technical study shall be made for the selection of suitable ground improvement method.

5.2.1 *Partial or Total Removal of Undesirable Material*

When unsatisfactory material not confirming to MORTH section 300 is encountered at or near the surface, it may be economical and prudent to remove part or all of it and replace it with acceptable materials than to deal with problem of countering subsidence over years.

This treatment is often used in swampy areas of peat and muck deposits. Generally on high embankment projects, removal of 1.5 – 3m of unsuitable material from original ground level may be possible over the full width of the fill. However, on the whole, this will be governed by the costs involved relative to other methods. Excavation of soft foundation soils and their replacement are often considered relatively simple operations. Practically, however, this is not the case and stringent inspection and control may usually be necessary to assure satisfactory and economical results. At times, more sophisticated techniques like controlled blasting may have to be adopted to achieve displacement of the soft material, followed by controlled placement of the foundation and embankment fill.

5.2.2 *Use of Light Weight Fill Materials as Embankment Fills with or Without Replacement of Poor Base Soil*

Settlement and stability problems can be decreased if the weight of the embankment is reduced. Light weight materials such as flyash, expanded shale, cinder, slag, saw dust etc; have been used with good degree of success in several cases for embankment construction to lessen the load on the foundation materials. However, availability of these materials and the relative costs are factors that may affect their use. IRC:SP-58 provide guidelines for use of flyash in road embankments

5.2.3 *Stage Wise Construction of Embankment with or Without Replacement of Poor Base Soil-*

For detailed description, please refer Special Report 14, "State of Art, High Embankments on Soft Ground Part A – Stage Construction".

5.2.4 *Soil Stabilization using Lime, Cement or other Chemicals*

5.2.4.1 Lime and lime- pozzolana stabilization

Lime is an excellent choice for short-term modification of soil properties, especially where the subsoil is of expansive nature, for example black cotton soil. Lime can modify almost all fine-grained soils, but the most dramatic improvement occurs in clayey soils of moderate to high plasticity. Modification occurs because calcium cations supplied by the hydrated lime replace the cations normally present on the surface of the clay mineral, promoted by the high pH environment of the lime-water system. Thus, the clay surface mineralogy is altered, producing benefits of

- Plasticity reduction
- Reduction in moisture-holding capacity (drying)
- Swell reduction
- Improved stability
- The ability to construct a solid working platform

Stabilisation by lime slurry injection as well as by adopting lime column technique is discussed in SR-14, IRC-HRB. For details regarding chemical reactions involved (Pozzolanic reactions) IRC SP 89 can be referred.

5.2.4.2 Cement stabilization

Where subsoil is of expansive nature then mitigation measure for the unsuitable stratum normally involves removal of poor strata and replacement with suitable soil. However, when depth and extent of poor soil is large, it is not feasible for this technique and cement stabilization could be another possible mitigation technique. Details of cement stabilization are covered in IRC:SP 89 “Guidelines for Soil and Granular Material Using Cement, Lime and Fly Ash and are not repeated here.

5.2.5 *Preloading*

This section refers to use of preload without resorting to any process to accelerate the consolidation of soft subsoil. Reference can be made to SR-13 IRC HRB Chapter-5. The term “preloading” implies that a greater height of fill that is placed initially than required for the final level of the embankment. Alternatively the same can be accomplished by stage construction of embankment.

Preloading is a simple and an economical method for accelerating consolidation as compared with other methods of improving ground support. However, adequate instrumentations for monitoring the settlements and the development and dissipation of pore water pressures are essential for the success of this technique. Preloading is especially attractive when fill material is subsequently used on the same project for site preparation. By measuring the ground settlements and porewater pressure, it is possible to assess quantitatively the extent of ground improvement in terms of increase in the shear strength and predict its future behavior. The duration of preloading from the beginning of embankment placement to the end of removal of load depends on the ground response. The pre-loading techniques is likely to be inefficient when used alone because of very long periods of time required for obtaining significant consolidation settlements and subsequent appreciable strength gain of the soft clay to support the embankment loads. The preload time can be drastically reduced by the installation of vertical drains as they shorten the drainage path under which the clay will consolidate.

5.2.6 *Ground Improvement Based on Consolidation of Subsoil*

5.2.6.1 Prefabricated Vertical Drains (PVDs)

Prefabricated vertical drains are used in construction of embankments on soft clays to accelerate consolidation process. The construction of a new embankment or structure on soft ground causes the following

a) Increase in the pore water pressure in the subsoil , these pore water pressures are termed as excess hydrostatic pressures

b) With lapse of time pore water pressure decreases which in turn results in an increase in the effective stress in the soil. This increase in effective stresses lead to the increase in shear strength of the subsoil.

c) At the same time the entire process is accompanied by decrease in the volume of the soil leading to settlements.

PVDs are band shaped (rectangular cross-section) products consisting of a geotextile filter layer surrounding a plastic core. PVDs are also referred to as band drains. The width of band drains is normally 100 mm. For detailed specifications of PVDs MORTH Specifications for Roads and Bridge works clause 704.2.2 can be referred. Porewater flows through the permeable outer geotextile filter layer and is transmitted along the annular space between the core and the geotextile. Installation of prefabricated vertical drains accelerates the rate of dissipation of pore water pressure and leads to a corresponding increase in effective granular stress in the subsoil. This in turn leads to increase in shear strength of subsoil. Similarly time rate of settlement is accelerated. Detailed design of ground improvement using PVDs is discussed in IRC-HRB: SR-14 and various factors involved such as relationship between increase in shear strength and inter-granular stress, time vs. consolidation are presented. The embankment is constructed in stages, with successive stages being built taking advantage of strength gained from the previous stage of loading. PVDs are installed in equilateral triangular pattern and the effective spacing between the band drains shall preferably be not more than 1200 mm. Reference shall also be made to IS 15284-Part-2: Design and Construction guidelines for pre-consolidation using vertical drains. Soft clays are prevalent in the coastal areas. Highway embankments constructed in these areas have used PVDS to enable rapid completion of the highway projects. Reference may be made to papers published viz., Mandal. A. K. et al. (IGC-2006, Guntur), Rao P.J. et al. (IGC-2011, Kochi), wherein two case histories dealing with use of PVDs including design, construction and monitoring are discussed. A typical cross-section of a PVD installation for embankments is shown in **Fig. 5.1** (Typical example for design of PVD is included in the Annexure 5.1)

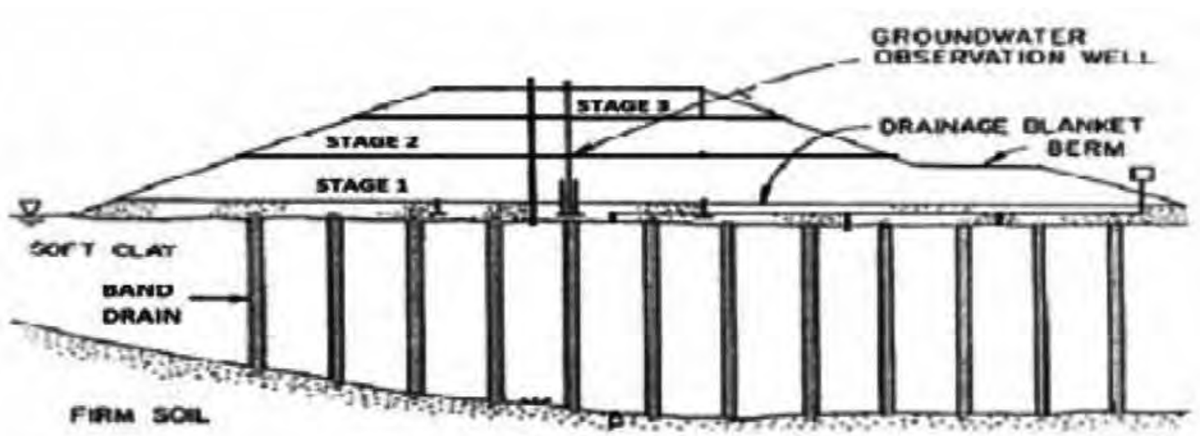


Fig. 5.1 Typical Cross Section of PVD Installation for Embankments

Fig. 5.2 Installation of Prefabricated Vertical Drains (PVDs)

5.2.7 *Stone Columns*

Use of stone columns is a highly effective technique to improve the strength and compressibility characteristics of soft clays. Stone columns are formed by making a borehole and backfilling with compacted stone. In the earlier phase of utilisation of this technique stone backfill was placed in layers and compacted by dropping a rammer. Stone columns so formed are termed as “rammed columns”. Rao P. J. et al. (1991) presents the case history of ground improvement using “Rammed column technique” in 12 m deep clay at Visakhapatnam on the east coast of India.

Rammed column technique is a slow process and hence it is normally used where length of the stretch to be improved is not long. In large projects or where deep layer of soft clay is present stone columns formed by vibratory methods are adopted. Vibratory stone columns are formed by vibro-replacement (wet, top feed) and vibro-displacement (dry, top or bottom feed).

Embankments on soft ground have also been built using, reinforced stone columns and grouted stone columns. Experimental work on reinforced stone columns is reported in IRC: HRB- SR-14, Rao. P. Jagannatha and Kumar, Satish (IGC 1989) and the reinforced stone columns were found to develop much higher bearing capacity than unreinforced ground. Recently Niroumand and others (2011), Tandel and others (2012) Heitz and others (2005) have published papers regarding reinforced stone columns. Encased stone columns shall be used when the soil around need to be separated from the stones. A geosynthetic filter shall be selected as per the gradation and properties of stones of stone column and soil around.

Typical example for design of stone columns (rammed/vibratory) is included in Annexure 5.1. If necessary, rotational stability of the embankment may be checked after determining the in-situ shear strength of the subsoil by suitable method.

5.2.7.1 Vibro-replacement- wet method:

It refers to the wet, top feed process in which jetting water is used to aid the penetration of the ground by the vibrator. Due to the jetting action, part of the in-situ soil is washed to the surface. The oscillating vibrator sinks under its own weight. When the designed depth is reached, the borehole is cleaned of any loose muck and aggregates are poured down the borehole. The vibrator is slowly withdrawn in steps of 0.7 to 1.0 m and stone falls to the tip of vibrator. The vibrator is then lowered back into the borehole between 0.70 to 0.80 m with constant water jet at the tip of vibrator thereby creating a 0.2 to 0.3 m length of stone column. The action of vibrator presses the stone radially into the surrounding soil and compacts the stone in the vertical direction as well. This procedure is repeated till the desired length of stone column is formed.

5.2.7.2 Vibro displacement: Refers to the dry, top or bottom feed process; almost no in-situ soil appears at the surface, but is displaced by the backfill material. However, the wet replacement method is more commonly used.

Formation of stone columns by above method is monitored by measuring consumption of power by the vibrator during the penetration for drilling hole and also during the compaction of backfill stone. The quantity of stone consumed per meter of the column is also continuously monitored. Reference may be made to Rao. P. J., Biswas. P. et. al. (2011).

5.2.8 *Densification*

Densification process is applicable for non-cohesive soils. There are numerous natural and man-made deposits where densification, can be adopted including granular hydraulic fills, coastal plain sediments, alluvial soils, and miscellaneous granular fills and/or deposits. Densification causes (a) increase in shear strength (b) reduction in settlements thereby enabling the construction of shallow foundations. Also, liquefaction potential can be reduced by densification of loose granular soil to a density beyond the threshold density triggering liquefaction. In earth retaining problems, the process can be performed prior to wall construction to decrease active earth pressure and increase passive resistance as the density is improved.

Some of the densification methods adopted are:

i) Dynamic compaction

ii) Vibro-compaction,

iii) Compaction grouting.

5.2.8.1 Dynamic Compaction

Dynamic compaction is a method of ground improvement that results from the application of high levels of energy at the ground surface. The energy is applied by repeatedly raising and dropping a tamper with a mass ranging from 15 to 35 tonnes at heights ranging from 9 to 30 m. The tamper is lifted and dropped by a conventional crane with a single cable plus a winch that has a free spool attachment that allows the single cable to unwind with minimum friction. The tamper's energy of impact at the ground surface results in densification of the deposit to depths that are proportional to the energy applied. The depth of improvement generally ranges from about 10 to15 m for light to heavy energy applications, respectively.

If ground improvement is needed to provide a suitable bearing stratum for an embankment or structure, dynamic compaction may be a viable solution. Schematic arrangement of dynamic compaction is illustrated in **Fig. 5.3.**

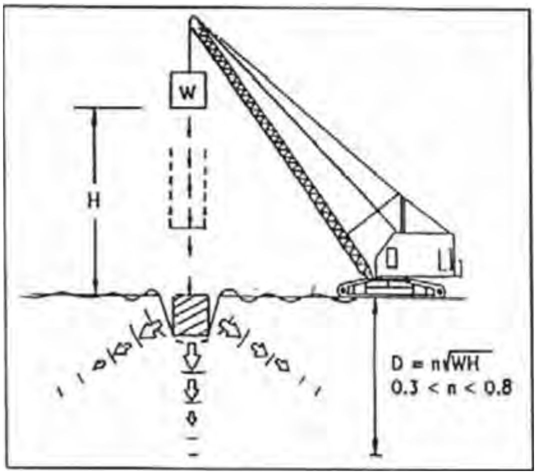


Fig. 5.3 Systematic Arrangement of Dynamic Compaction

Dynamic compaction can be used at sites with a very heterogeneous mixture of deposits and at sites with gradation ranges from large boulders and broken concrete to silty soil particles.

Densification can be achieved below the water table in pervious and semi-pervious deposits, which eliminates costly dewatering and/or lateral bracing systems required for conventional excavation and replacement techniques. Dynamic compaction produces ground vibrations that can travel significant distances from the point of impact. In urban areas, this may require the use of light weight tamper sand low drop heights, as well as limiting dynamic compaction to areas well within the property lines. At some sites, shallow isolation trenches have been cut through the upper portion of the soil mass to reduce the transmission of energy off site.

Dynamic compaction densifies the soil mass and this in turn, improves its shear strength and reduces compressibility. An estimate of improvement in the soil properties shall be made before the compaction work is taken up. Soil parameters may be evaluated before and after treatment by carrying out Standard Penetration Test (SPT) or Cone Penetration Test (CPT) or Pressure Meter Test (PMT) at the requisite number of locations. On this basis it can be determined whether dynamic compaction is capable of producing the desired effect.SPT or CPT values are used to define the susceptibility of deposit to liquefaction. If the dynamic compaction increases the SPT or CPT values of a loose deposit to the required value, then the method is deemed to be successful making the soil resistant to liquefaction

5.2.8.2 Vibro compaction

Vibro compaction technique enables the improvement of loose granular deposits up to depths of 20m. In this process a metal tube or probe to which an electric motor is attached at the end is inserted into the ground. The electric motor drives an eccentric weight generating vibrations which causes the ground to densify. To assist the penetration of vibrator water is jetted through the tip of the probe.

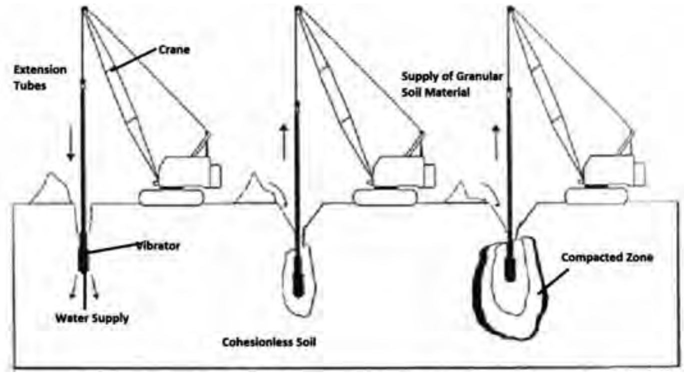


Fig. 5.4 Vibro Compaction Process

Vibro-compaction is effective only in granular, cohesion-less soils. The realignment of the sand grains and, therefore, proper densification generally cannot be achieved when the granular soil contains more than 10% silt or more than 2% clay.

A more detailed soil analysis may be required for vibro-compaction than for a deep foundation project. This is because the vibro-compaction process utilizes the native soil to the full depth of treatment to achieve the end result. A comprehensive understanding of the total soil profile is therefore necessary. A vibro-compaction investigation will require continuous standard penetration tests (SPT), and/or cone penetrometer (CPT), Pressure Meter (PMT) tests, as well as gradation tests to verify that the soils are suitable for vibro-compaction.

5.2.9 Compaction Grouting

The process of grouting consists of filling pores or cavities in soil or rock with a liquid form material to decrease the permeability and improve shear strength by increasing the cohesion when it is set. The injected grout pushes the soils to the side as it forms a grout column or bulb. The soil becomes increasingly dense as water and/or air are forced out and soil particles are rearranged by the incoming grout. Grout injections can be continued until grout forces overcome overburden or containment pressures and lift occurs. Compaction grouting requires close coordination between the soil properties, grout injection rates, grout mix designs, in-situ soil conditions, and equipment capabilities. When compaction grout is injected into loose soils, homogenous grout bulbs are formed that displace, densify and thus strengthen the surrounding soil. Cement base grout mixes are commonly used for gravely soils or fissure rocks.

5.2.10 *Dynamic Deep Replacement*

Dynamic Replacement is an extension of Dynamic Compaction to highly compressible and weak soils. In this application, the tamping energy drives granular material down into the compressible soils to form large diameter soil reinforcement columns (with diameter around 2 to 3.5 m). Additional improvement can be obtained in the underlying layers through the transmission of the energy of the weight at depth. This method thus combines advantages from both Dynamic Consolidation and Stone Columns by creating large-sized Dynamic Replacement Inclusions with high internal shear resistance. Dynamic Replacement is well adapted to substantial loading conditions (up to 150 tons per column) as well as under embankments to improve the factor of safety against slope failure. With this technique, replacement ratios of up to 20-25% can be achieved. While Vibro Stone Columns have a limited range of application is organic soft soils (peat and organic clays), Dynamic Replacement Columns can be used in peat or in soils with high organic content without the risk of bulging due to their relatively low slenderness (ratio of height over diameter). Construction sequence of dynamic replacement is illustrated in **Fig. 5.5.**

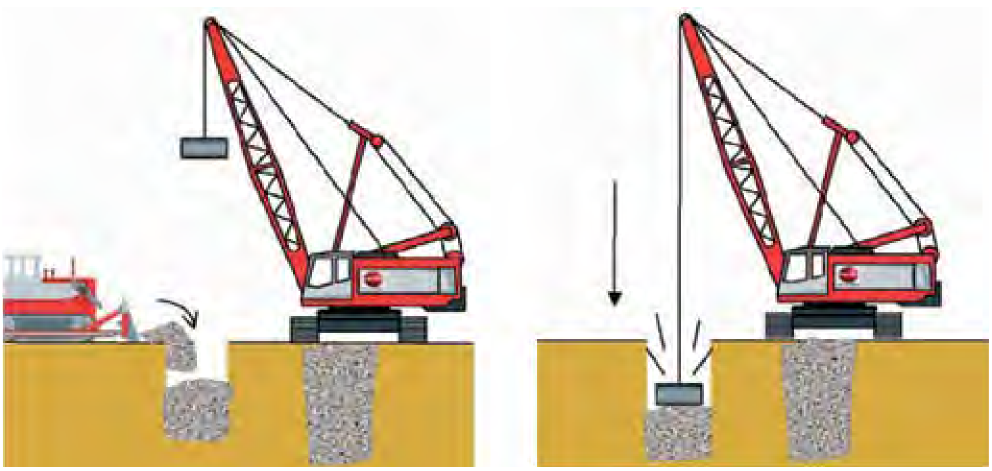


Fig 5.5 Construction Sequence Dynamic Replacement

Dynamic Replacement Columns can increase the time rate of consolidation due to their draining potential.

5.2.11 *Embankments on Soft Ground Using Basal Reinforcement*

Embankments on soft ground may be built using stiff layers of geosynthetic reinforcement of high strength placed on the ground. To ensure the safety of the embankment in various modes of failure, the geosynthetic reinforcement shall have enough tensile strength. This technique is more adopted in situations where the clay layer is relatively of shallow depth as compared to the base width. The ratio B/D (where B= width of the embankments and D= depth of soft clay layer) shall be greater than 4 for any increase in bearing capacity to be available. Reference may be made to IRC:113-Guidelines for the Design and Construction of geosynthetic reinforced embankments on soft subsoils. The publication also includes number of case histories. A representative sketch of Basal reinforced embankment is shown in **Fig. 5.6.** Anchorage Blocks may be provided as shown in **Fig 5.6.** Alternatively the reinforcing layer may be provided with extra length which can be wrapped around the next layer during the process of laying and compaction. Reinforced earth walls upto 15 m height were built on NH-5 (Now NH-16) in the Godavari Delta stretch, using basal reinforcement, which consisted of high strength geo-grid of 200 KN/m with gravel fill between the layers. The project was completed in 2002 and the performance of the embankment is excellent. For details Ref. P. Jagannatha Rao (2004)

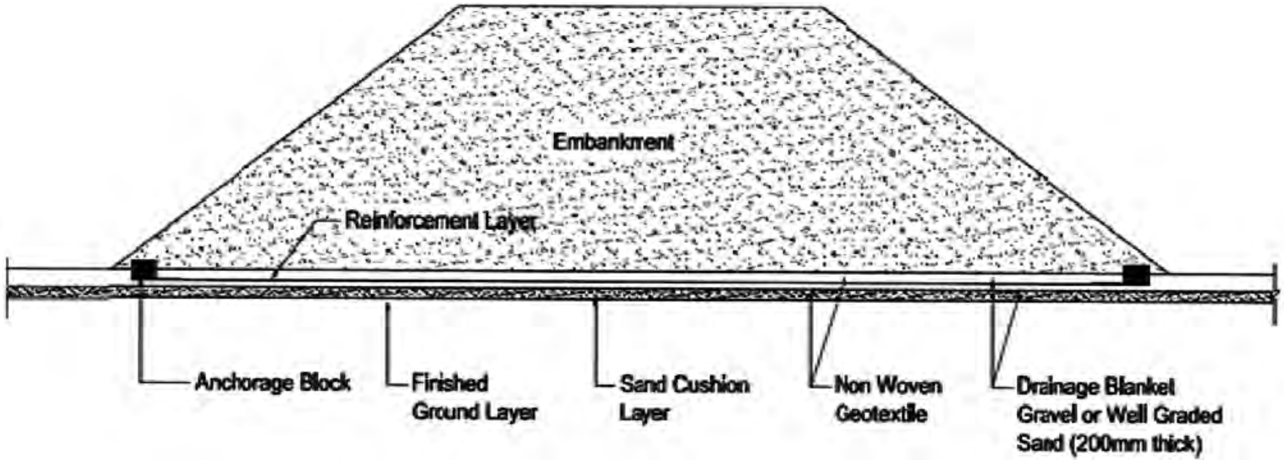


Fig. 5.6 Basal Reinforced Embankments (for more details, refer IRC:113)

5.2.12 *Pile Supported Basal Reinforced Embankments on Soft Ground*

Embankments on very soft ground may be built ensuring stability as well as minimising settlements to very low values by using the system of pile supported embankments. Piles are built in the soft clay and geogrid layer is placed on the pile caps. Piles along with geo grid layer serve as a support system for the embankment fill. For design details refer to BS: 8006. **Fig 5.7** illustrates the scheme of pile supported basal reinforced embankments.

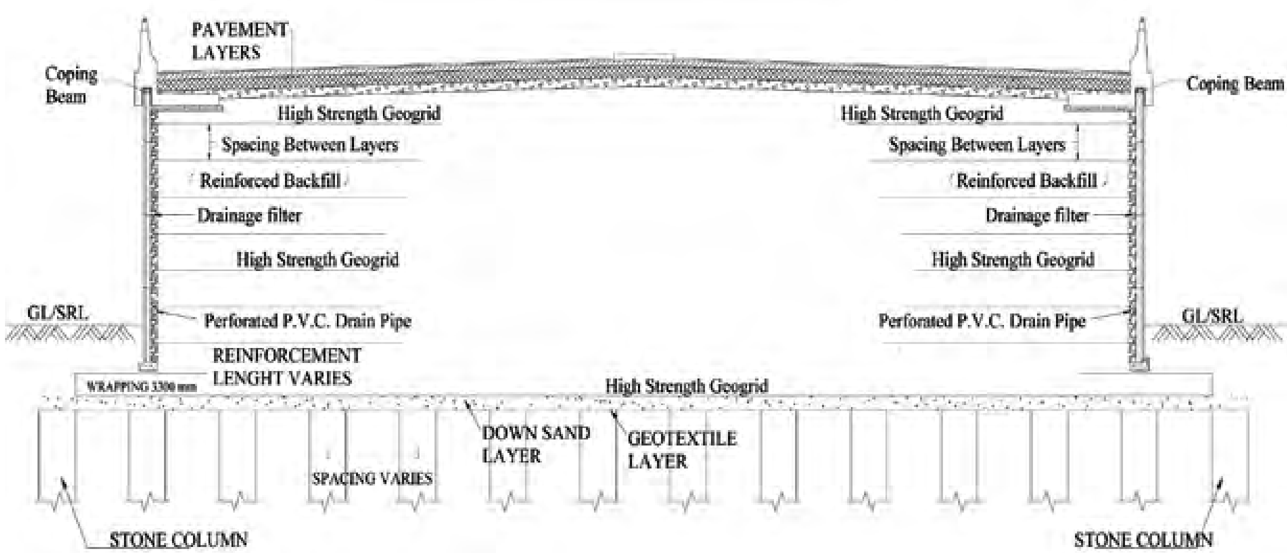


Fig. 5.7 Pile supported Basal Reinforced Embankment

5.3 Widening of Embankments

5.3.1 *Widening of Embankment by Cutting Benches and Filling*

The need for widening existing highway embankments by a lane or more is being increasingly felt. **Figs. 8 (a) & 8 (b)** provides a brief description of methodology that may be adopted for this purpose. Referring to **Fig 8(a)** benches may be cut in the slope face. The depth to which these benches may be cut may range from 0.75 m to 1 m and width of the bench as per existing slope in relation to the depth of the bench. Benches are cut starting from top to bottom. The new fill is gradually filled up in layers in the standard manner. Where stability of the fill is in doubt, the same may be ensured by providing a soil reinforcement system as shown in **Fig 8 (a).** Geosynthetic having adequate frictional properties as per MORTH section 3100 may be used for this purpose. The number of benches to be reinforced may be worked out using standard procedures of slope stability. In particular, planar sliding between the earlier slope face and the added fill may be checked.

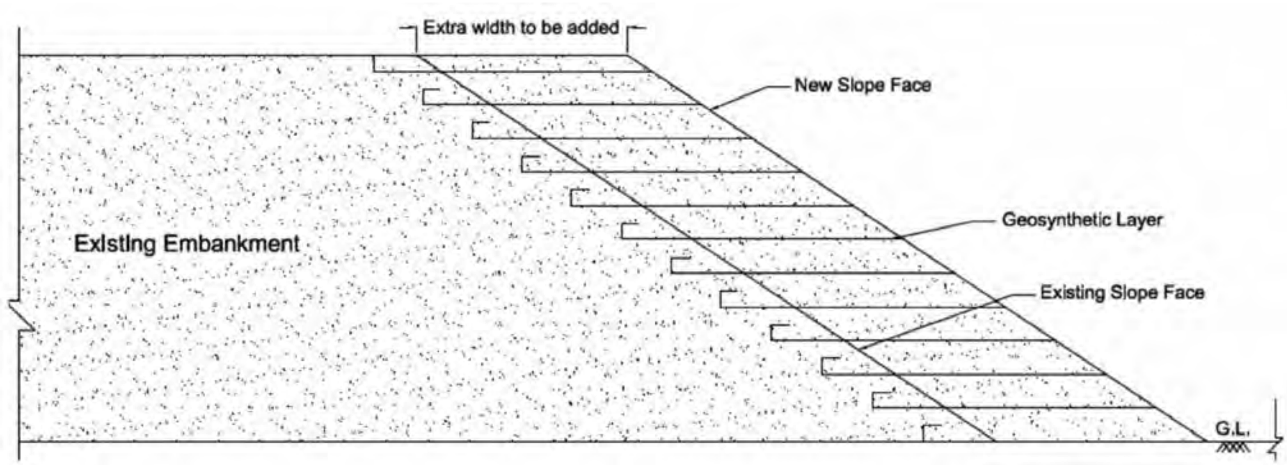


Fig. 8 (a) Widening of Embankment by cutting Benches and Filling

5.3.2 *Widening of Embankment by using Soil Nailing*

In situations where the embankment to be widened exceeds 10 m in height for considerable length, the increased width along with the original embankment may be kept in a stable condition by adopting soil nailing technique. Details of the technique are given in MORTH section 3200. Schematic sketch of the same is given below.

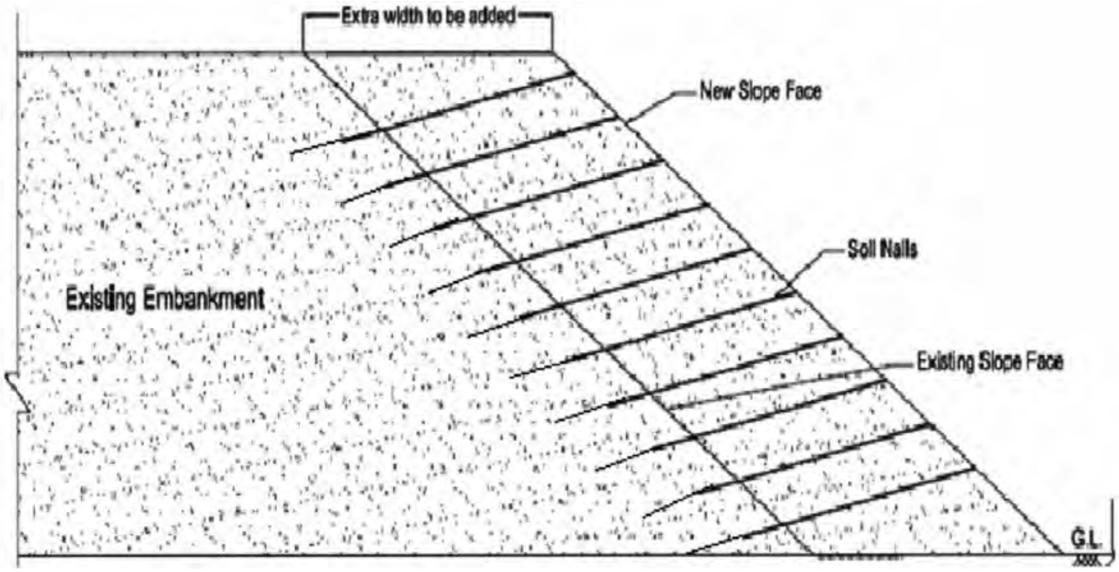


Fig. 8 (b) Widening of Embankment using Soil Nailing

Surface of the existing embankment may be cut by a few centimeters or scarified so as to make it rough. Lying of the fill may start from ground. The thickness of the first layer as well as other layers would depend on the design of soil nail system. At the calculated height first nail is driven. Then next layer of fill is placed again at the calculated height and so on. This technique may be adopted after taking into consideration of economies.

5.3.3 *Restoration of Failed Embankments*

Failure of embankments occur due to number of reasons and require correct diagnosis for choice of remedial/restoration methods. Necessary subsoil investigation and testing needs to be carried out and design on the basis of which embankments has been made needs to be checked.

Embankment safety can be achieved by:

i) Removing some of the weight tending to cause failure or adding fresh material to flatten the slope

ii) By increasing the strength of the soil in the portions where the slip failure occurred; and

iii) By proper drainage and prevention of water percolation in the embankment.

iv) By providing external support. This can be done by means of retaining wall of required height at the toe or providing balancing berms of necessary width.

The two methods described in section 5.3.1 and 5.3.2. may be used to restore failed slopes as well.

In all cases of restoration of failed embankments, slope stability of restored embankment shall be checked for the revised configuration.

ANNEXURE 5.1

1. Additional Ground Improvement Methods

Ground improvement methods discussed in the previous sections improves the bearing capacity to a limited extent. Where much higher degree of ground improvement is needed, techniques which reinforce the ground have come into practice over the last few decades. Ground improvement methods based on reinforcing technique is suited for improving clays, silts, and loose silty sands. Rigid inclusion using controlled modular methods is a proven technology since last 30 years as a cost effective alternative to pile foundation system. It is used to improve the soil characteristics of a compressible soil layer on a global scale and to reduce its compressibility by use of rigid soil reinforcement columns. Unlike a piling solution which is designed to support the entire load of the structures on the piles, the objective of a rigid inclusion solution is to increase the stiffness of the soil mass to globally reduce both total and differential settlements by sharing the load of the structure between the soils and the inclusions. Rigid inclusion columns range between 250 to 450 mm dia.

Irrespective of stone columns, rigid inclusions can be used for any types of compressible soils, including soils with significant organic contents (Peat, organic clays etc.).The technique is more advisable for the projects where construction time plays an important role and can be used to the close vicinity of the adjoining structures.

Preliminary design guidelines of CMCs can be done by using BS 8006 -1& ASIRI (French Code). Detail design using FEM Modeling as per the applications need to be done by specialized contractors who are having prior experience in taking the project on design and build basis. Post construction load test can be performed ASTM D 1143 – Standard Test Method for Deep Foundations under Static Axial Compressive Load.

2. Vacuum Consolidation

Consolidation of compressible soils by vacuum was conceptually introduced in the 1950s and has recently evolved as a reasonably reliable technology. The basic premise for vacuum consolidation consists of removing atmospheric pressure from a confined sealed soil to be consolidated and maintaining the vacuum during a predetermined period of time. The soil is, therefore, loaded uniformly throughout its depth by the equivalent 70% of 1 atmosphere as shown in **Fig.5.9**

Vacuum Consolidation method is an atmospheric consolidation system used for preloading soft saturated fine-grained soils (clay, silt, peat). The procedure consists of installing a vertical and horizontal drain system and vacuum pumping system under an airtight impervious membrane. The area is sealed by embedding the membrane into peripheral trenches. These trenches are continuously recharged and filled with water to maintain full saturation of the soils and to avoid a general lowering of the ground water table within the treatment area. Vacuum Consolidation method has been successfully applied since the late 80’s in various types of structures and applications (power plants, Sewage treatment plants, highway embankments, airport runways). The procedure requires installation of air and water pumping system to create vacuum in the soils below the impervious membrane. Vacuum produced is equivalent to a depression of between 60 and 80 kPa, depending on the global efficiency of the system – (this pressure is similar to the stress observed under a 3 to 4 m high embankment).

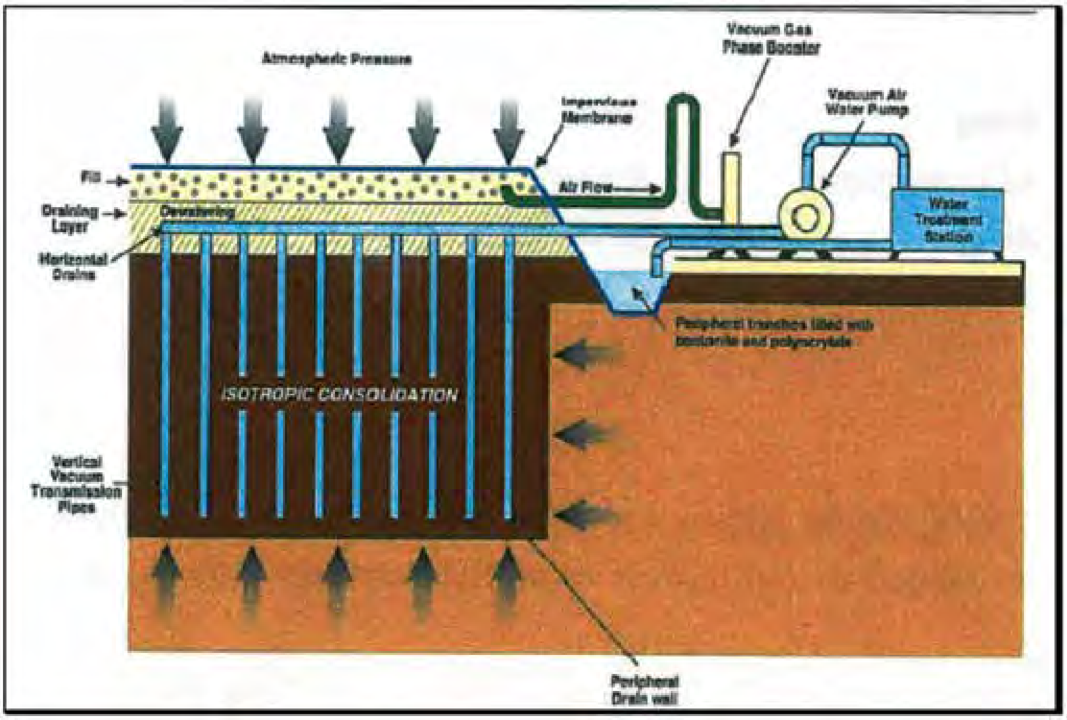


Fig. 5.9 Vacuum Consolidation

Main advantage of Vacuum Consolidation is time savings over other classical consolidation methods with surcharge and band drains. Under vacuum consolidation, consolidation period usually ranges between 4 and 6 months

In practice, PVDs/Vacuum Consolidation are most commonly used where the soil to be treated is moderately to highly compressible with low permeability and fully saturated in its natural state and in situations where embankments are routed over long distances on marine clays, tidal swamps, peats, creeks etc., where drainage conditions is critical in causing instability, and post-construction settlements assume serious proportions. Such soils are typically described as silts, clays, organic silts, organic clays, muck, peat, swamps and sludge.

In general, PVDs/Vacuum Consolidation techniques are best suited for soft saturated and normally to slightly over-consolidated soils, prior to loading. The loading should exceed the maximum past consolidation pressure to be totally effective.

Field instrumentation, such as piezometers, settlement platforms and inclinometers, are used to monitor performance and possibly control the rate of construction of embankment and/or surcharge. Settlements measuring devices, deep settlement points are used to measure only the rate and total amount of consolidation. For details refer Chapter-6

CHAPTER 6

INSTRUMENTATION AND MONITORING OF EMBANKMENT ON SOFT SOILS

6.1 Field Observations/Monitoring

Experience has shown that safety during construction of embankments over soft sub-soils cannot be ensured by only geotechnical investigations and design, since many unforeseen factors may arise while the work is in progress which could have profound influence on stability. Further, during design of embankments on soft sub-soils, low initial factor of safety is adopted for reasons of economy, as the long term factor of safety is anticipated to be higher than the initial factor of safety. The increase in the factor of safety over time, takes place due to strength gain in the soft clay on the account of consolidation due to loading. Hence progress of consolidation of the soil layer beneath the embankment must be monitored. For this purpose the following parameters are subjected to close watch:

i) Build up and dissipation of pore water pressure

ii) Rate and magnitude of the vertical settlement of the sub-soil under the applied embankment loads

iii) Horizontal spreading of the sub-soil under the applied loads

iv) In-situ shear strength

All the parameters need to be measured periodically at different sections of the embankment. The periodicity of measurements and location of the instruments has to be specified in the project report. Each of the above parameters when monitored and evaluated gives the engineer a clear indication of the state of stability of the embankment and of any variations that may be taking place in the same. These parameters also allow an evaluation of the effectiveness of ground improvement techniques adopted.

Impending instability can be detected well in advance and the engineer will have forewarning to take steps to arrest the damage. The data can be used to verify whether the embankment is in stable state or any impending failure exists. This exercise is based on experience and available theoretical considerations. For example, if there is a sudden increase in pore water pressure, it is easy to calculate the decrease in the factor of safety and whether such decrease likely causes a failure. On the other hand the increase in the magnitude and rate of settlements and/or in the rate of lateral movements is indicative of the distress experienced in the embankment, it is not possible at present to specify the limits on either the magnitude or the rate of change of movement so as to form a generalized method of control.

Additionally during construction, periodical visual observations should be carried out to detect surface tension cracks in the embankment. Usually such cracks which are precursor to failure **(Fig. 6.1)** appear as longitudinal cracks running parallel to lengthwise direction of embankment.

In all road projects wherein embankments of height more than one meter are constructed over soft clayey unconsolidated soils, having water table at shallow depths, provision should be made for installing instrumentation right at the inception of the project i.e. during preparation of the Feasibility Report or Detailed Project Report. Adequate resources in the form of money and manpower shall be provided to meet the objectives of instrumentation program. It would also be advisable to appoint qualified geotechnical engineer assigned by the contractor as well as by the supervising agency to ensure that instrumentation program is implemented correctly and monitoring/ interpretation of data is done properly.


Fig. 6.1 Typical Failure of Embankment on Soft Sub-soil

The extent of instrumentation for field observations will depend on prevailing site conditions and height of embankment proposed in a road project. If a project is large and the construction time required is expected to be long, it may be worthwhile going in for as much instrumentation as possible in the early phase of construction because, that may permit reduction in size of the berms or other features which may reduce the construction costs. Table 6.1 provides details about different instruments used during construction of embankment over soft sub soil.

Table 6.1 Different Instruments for Construction Monitoring and Control

Parameter	Types and Location of instrument/point of measurement
Porewater Pressures	Piezometers of suitable type may be installed at different depths and locations in the subsoil beneath the embankment.
In-situ shear strength	In-situ shear strength may be measured by vane shear test in the bore hole. Alternatively, undisturbed samples may be recovered from boreholes made at a given stage of construction and shear strength of sample determined in the laboratory
Vertical Settlement	Settlement gauges on original ground surface or base of excavation. Settlement markers on surface of fill or ground outside the embankment. Full-profile settlement gauges under the embankment.
Horizontal Movement	Inclinometers in the subsoil at toe of embankment. Displacement markers at the top and toe of embankment.
Heave	Heave stakes may be installed near the toe of the embankment

6.2 Piezometer

A piezometer is an instrument for measuring pore water pressure. Piezometers are installed in the ground to measure the pressure head at different locations in the sub-soil. An ideal piezometer is one, which is reliable, sensitive, robust and easy to operate. Many types of piezometers are commercially available.

6.2.1 Casagrande piezometer (Open tip type - **Fig 6.2**) is easy to install, simple to operate and rugged but has a long response time. Casagrande open standpipe piezometer consist of a ceramic porous tip connected to an open standpipe. The ceramic tip is generally of low air entry valve, which exhibits very high water permeability. Depending upon the pore water pressure existing at the porous tip, water would rise in the standpipe until the hydrostatic head of the column of water in the standpipe is equal to the pore water pressure.

The ceramic tip of Casagrande piezometer is attached to the bottom of un-perforated plastic tube and placed at a pre-determined depth. Ceramic tip can be wrapped in geotextile to prevent clogging. Monitoring the pore water pressure at designated depth is achieved by providing a short response zone of sand backfill around the piezometer tip with a bentonite or bentonite-cement mixture seal above. The seal may be in the form of granules, or pellets, balls or pumpable grout. It is preferable to backfill the whole of the borehole above and below the sand response zone with grout. Installation of Casagrande piezometer is shown in **Fig. 6.3**.

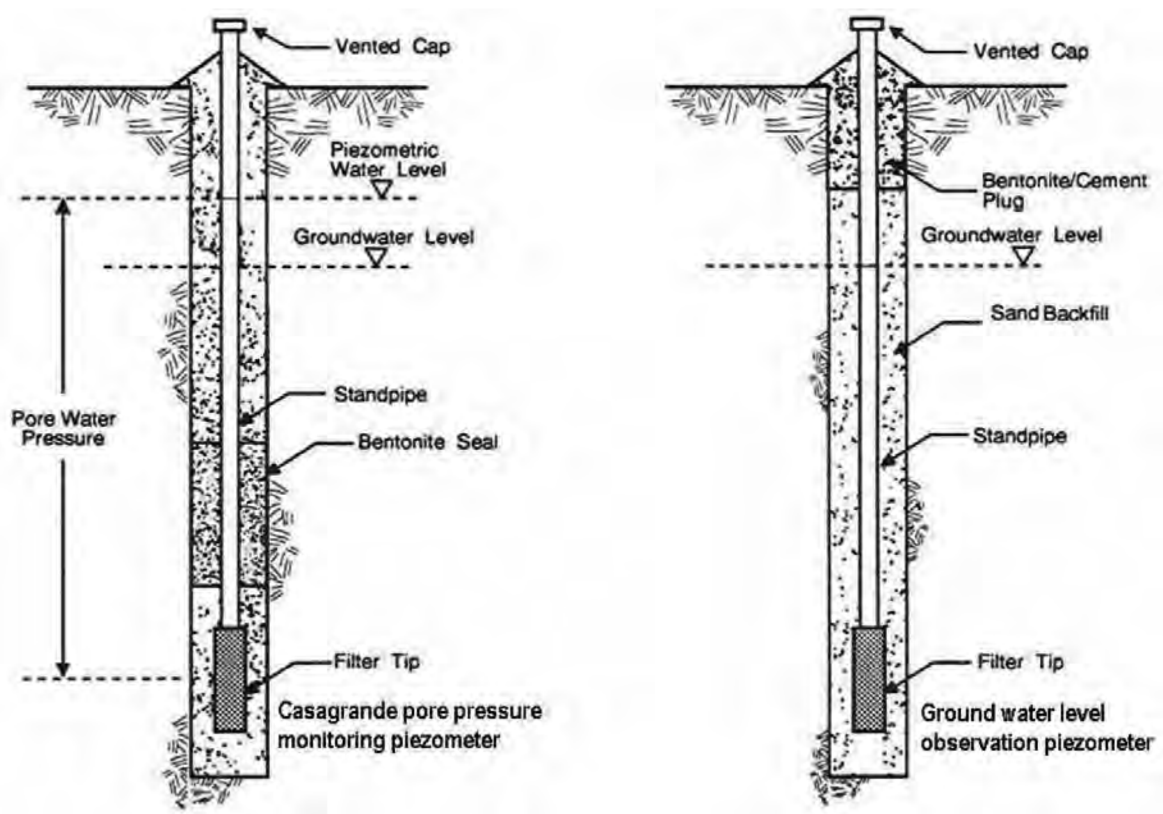


Fig. 6.2 Casagrande Piezometer

Ceramic Tip | Covering Tip | Bentonite Pellets | Monitoring Pore water

Fig. 6.3 Casagrande Piezometer Installation and Monitoring

6.2.2 The vibrating wire piezometer **(Fig. 6.4)** contains a tensioned stainless steel wire attached to a diaphragm. One side of the diaphragm is in contact with the groundwater pressure inside through a porous ceramic tip. The other side of the diaphragm is connected to atmospheric pressure by an air line. The pore water pressure causes the diaphragm to deflect which in turn changes the tension in the steel tension wire, hence its frequency of vibration changes. This frequency is calibrated to provide pressure readings. These piezometers have a shorter response time as compared to Casagrande piezometer and do not interfere with the compaction process.

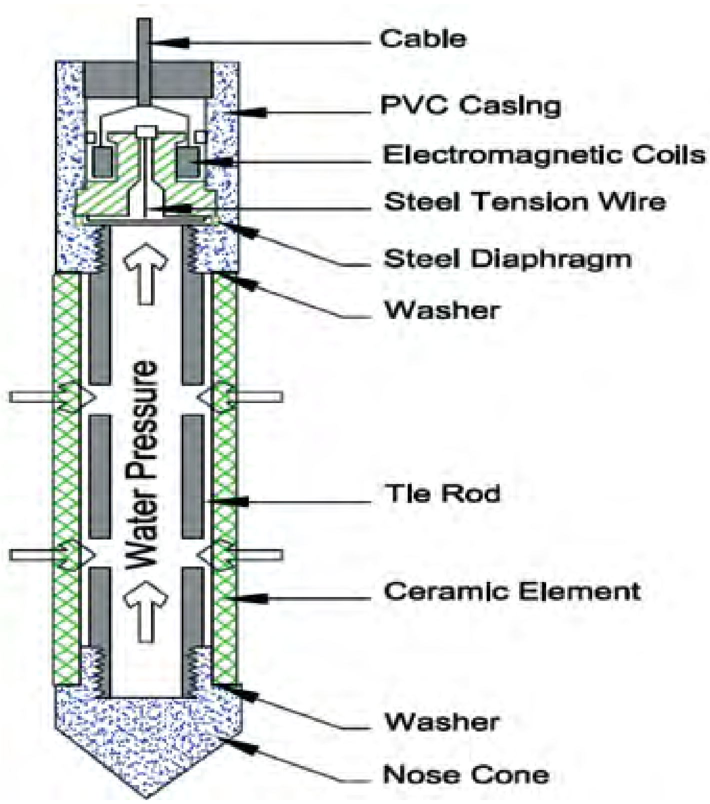


Fig. 6.4 Vibrating Wire Piezometer

6.3 In-situ Shear Strength

In all constructions on soft ground, shear strength is the parameter that directly controls the stability. Hence it is important that shear strength is measured at different stages of construction of embankment. As the embankment height increases, increase in shear strength of the subsoil occurs as the pore water pressure decreases and effective stress increases as per theory of consolidation. At the end of each stage of construction, stability can be evaluated using new shear strength values. Formulae giving the relationship between increase in shear strength of the subsoil and the increase in the effective stress are given in IRC-HRB SR:13, Clause 4.2.1.

In practise, the simplest way is to measure in-situ shear strength in the boreholes made for this purpose using field vane shear test apparatus.

Combined with the measurement of pore water pressures, in-situ shear strength values provide the designers a powerful and reliable tool to monitor the stability of the embankment.

6.4 Surface Settlement Markers/Settlement Platforms

In the simplest form, the surface settlement markers (settlement platforms) may consist of a square steel plate or concrete pad supporting a flange to which a section of pipe, usually about 1.5 meter long, is attached **(Fig. 6.5).** As the fill is built up, additional sections are coupled to the pipe. The size of the plate will depend on the material underlying the fill and may be from 0.6 to 1.2 m square. The pipes are protected from the surrounding soil by an outer PVC or metal pipe casing of appropriate diameter.

Settlement platforms are generally installed on the original ground in accordance with the designer’s stipulations. To protect the settlement platform during compaction, temporary barrier may be built around it and soil near the platform should be compacted using plate compactors to prevent rollers from disturbing the installed position of settlement platform.

Fig. 6.5 Installation of Surface Settlement Marker (Settlement Platform)

6.5 Magnetic Settlement Gauge

While surface settlement markers measure total settlement of original ground below the embankment, magnetic settlement gauges provide data on total surface settlement below the embankment and also settlement at various depths of sub-soil layers below the embankment. Hence magnetic settlement gauges are referred to as full profile settlement gauges also. Magnetic settlement gauge works on the principle that a sensor gets activated when it enters a magnetic field axially and can be made to emit a signal (buzzing sound) at the ground level. The magnetic ring consists of four arc shaped magnets fixed to four sides of a Perspex ring. Each magnetic ring (also called 'spider') has four upward leaf springs mounted at intervals of 900 around the ring **(Figs. 6.6 & 6.7).** These magnetic spiders are inserted at pre-decided depths around a PVC pipe which also acts as access tube for the probe. The probe is a brass rod with a reed switch inside it. The leads of the reed switch are connected with the help of a graduated wire to a control box housing an electronic circuit. When the circuit is completed a sound signal is generated. The measurements are to be made by inserting the probe, which detects the magnet. The exact depth of each magnetic ring can thus be ascertained. The net settlement of each magnetic ring can be determined by comparing with the initial levels of those magnets similarly recorded earlier.

For installing these gauges, bore holes are made up to hard stratums below the soft soil. One end of the PVC access tube is sealed and it is then rested on the hard stratum. Magnetic rings (spiders) are installed around the access tube and anchored in the soil around it, to enable its displacement along with the surrounding soil. As the sub-soil consolidates and settles downwards, spiders also move down. For measuring settlement of original ground, magnetic Perspex ring installed in a plastic plate is placed at original ground level below the embankment. Downward movement of these spiders and the magnetic plate (placed at original ground level) are monitored regularly to obtain settlement data.

Fig. 6.6 Magnetic Spider, PVC Access Tubes and Installation of Magnetic Settlement Gauge

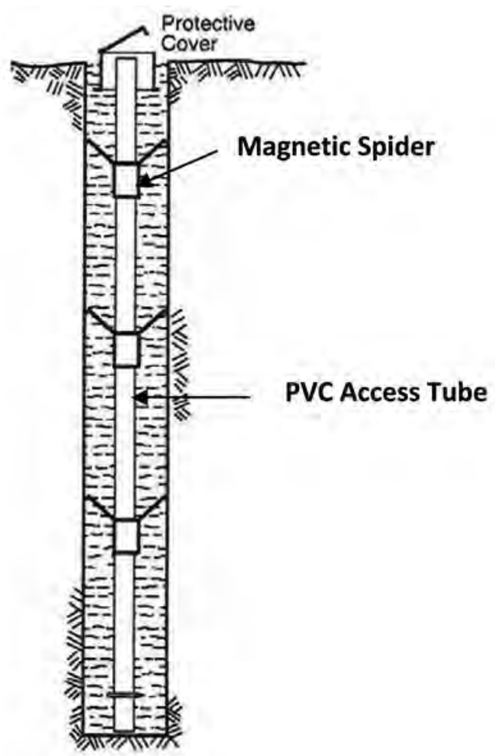


Fig. 6.7 Parts of Magnetic Settlement Gauge

6.6 Inclinometers

Inclinometers are the devices for monitoring deformation (deflection) normal to the axis of a pipe by means of a probe passing along the pipe. Inclinometer system is installed to measure extent of horizontal movements/plastic flow inside the sub-soil at various depths. The probe contains a gravity sensing transducer (two servo accelerometer) to measure inclination with respect to vertical **(Fig 6.8).** The inclinometer system has four components:

1. A permanently installed guide casing (inclinometer pipe) with grooves in two perpendicular directions, made of plastic or aluminum alloy. The guide casing usually has tracking grooves for controlling orientation of the probe.
2. A portable probe with retractable wheels, containing a gravity sensing transducer (Two servo accelerometers fitted inside hollow steel pipe at a distance of 0.5 m and then sealed)
3. A portable readout unit with power supply for indication of probe inclination
4. A graduated electrical cable linking the probe to a read out unit

The guide casing is installed in the borehole in a near vertical alignment so that the inclinometer provides data for measuring lateral deformations of sub-soil. After installation of casing, the borehole is backfilled. The grooves of the tube also serve to indicate the direction of the readings in relation to the embankment construction. For embankments on soft soils, the guide casing should be installed in such a way that one set of grooves are perpendicular to the foot of the embankment (lengthwise direction of the embankment). Horizontal movement of sub-soil is always measured along this set of the grooves. The probe is lowered to the bottom of the borehole and a reading of the tilt is made. Additional readings are made as the probe is raised incrementally to the top of the casing, providing data for determination of the initial alignment. The difference between initial alignment of the casing and subsequent alignment indicates the amount of lateral movement (deflection) of the sub-soil. Since horizontal displacement can be sometimes high, it is recommended that the integrity of the guide casing be first verified before each monitoring. This is done by lowering a dummy probe (without gravity sensing transducers) and pulling it back to avoid losing the actual probe. Readings are taken at constant intervals in the ascending direction. **Fig. 6.9** shows installation of inclinometer. Output of typical inclinometer at the toe of embankment is shown in **Fig 6.10.**

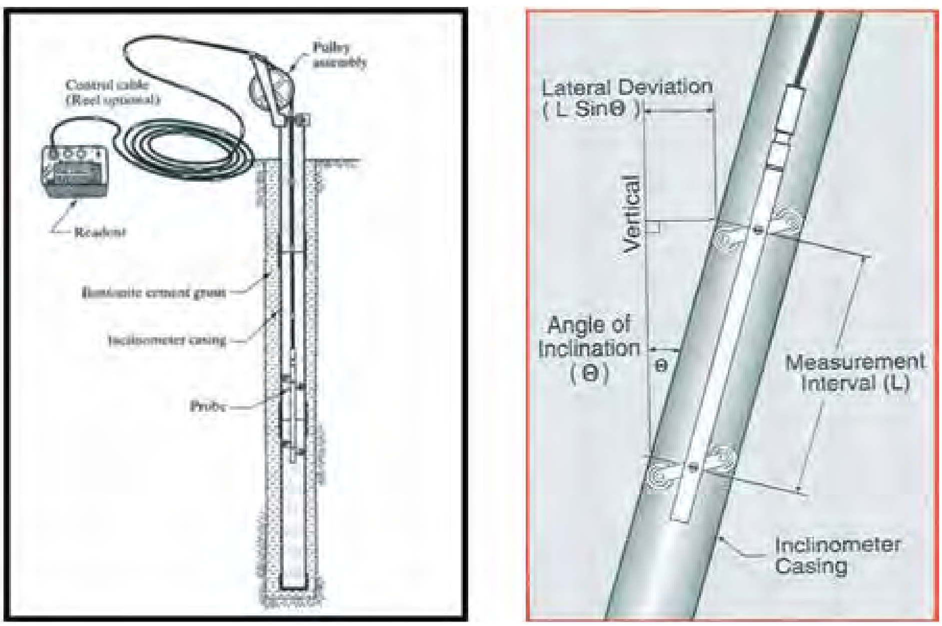


Fig. 6.8 Inclinometer

Inclinometer Guide Casing

Coupling for Required Length

Installing Inclinometer in Bore Hole

Fig. 6.9 Inclinometer Installation

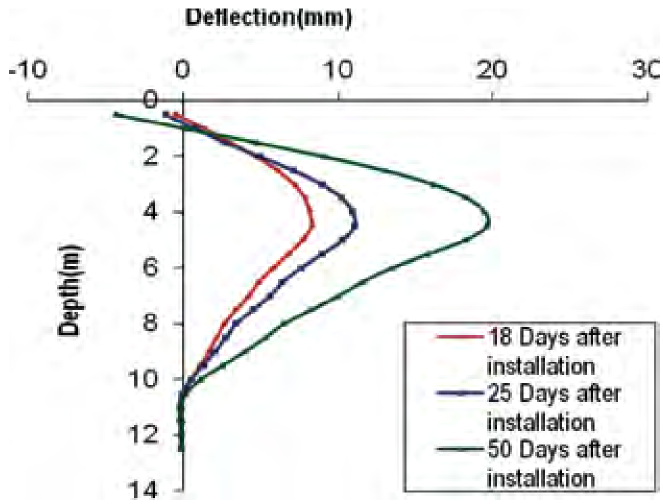


Fig. 6.10 Typical Inclinometer Output

6.7 Heave Stake/Pegs

One of the common methods of detecting lateral movement of embankment side slopes is provision of heave stakes or pegs (also called displacement markers). These are generally installed in a straight line near or beyond toe and top/ side slopes of the embankment in order to detect any heave in natural ground or lateral movement of embankment side slope which usually precede a shear failure. The most suitable position for these, with reference to the slope, would depend upon the type of failure which is anticipated.

Heave stakes may consist of vertical wooden pegs 50 mm x 100 mm in cross section or a pipe section of appropriate length driven into ground with a horizontal cross piece on top. Reference marks are put on the vertical and horizontal surface of the stake to aid observation about the vertical and horizontal movement. It can have many different designs. Alternatively, 50 mm x 50 mm x 75 mm timber pegs driven into ground and concrete base if required may be provided.

The stakes are normally installed with spacing of 15 to 20 meters on straight reaches. On curved section, the spacing may be reduced to 10 meters or less. In straight sections of embankment, it is useful to align the stakes in a straight line so that if there is a movement, it can be detected even without a surveying instrument by sighting along the line of stakes. Normally the movement of stakes is monitored from a survey bench mark located outside the zone of disturbance. In most cases, the zone of disturbance will not extend beyond 30m from the embankment. Observations are to be made regularly as the construction of the embankment progresses.

6.8 Construction Monitoring and Control

Embankments on soft grounds are monitored for assessment of the design assumptions; planning of the field work, especially in terms of loading and unloading stages; and to ensure the structural integrity of the embankment for preventing failures. As already stated, embankments on soft ground are usually designed by considering a lower factor of safety, since incorporating changes in designed cross section is comparatively easy in earth work. Hence safety of the embankment should be controlled by using information obtained from installed instruments. In order to achieve the above objectives, some of the important following criteria must be met:

- The magnitude of each type of measurement as well as the range of expected variations must be known in advance;
- The analyses should be performed immediately after taking the readings, to provide adequate time for incorporating changes if any in the field works;
- The plan of instrumentation should inform how and where the instruments are to be installed and the recommended frequencies for monitoring.

6.8.1 *Typical Instrumentation Scheme*

In a road project, embankment constructed on soft sub-soil may extend over many kilometers. Hence it would be prudent to select 'Typical sections' of about 50 to 70 m length for installation of instruments. This would help for easy monitoring and also minimize inconvenience to construction activities. The selection of locations for instrumentation shall be governed by factors such as height of the embankment, sub-soil properties over the stretch, etc. Usually, piezometers and settlement gauges/markers are installed at centre line of the embankment while inclinometers are installed near toe of the embankment. Plan view and cross section of typical layout for instruments is given in **Figs. 6.11 & 6.12.** Details about instrumentation scheme (locations and number of instruments), monitoring frequency, etc shall be indicated in the design report or detailed project report.

It is important that the entire system of recording and monitoring of data from the instruments shall be maintained in a computerized form. The system shall provide connection of all the instruments through independent cables to a microprocessor based 'Data Logger' having flexibility of automatic recording of data from individual connection at fixed interval of time.

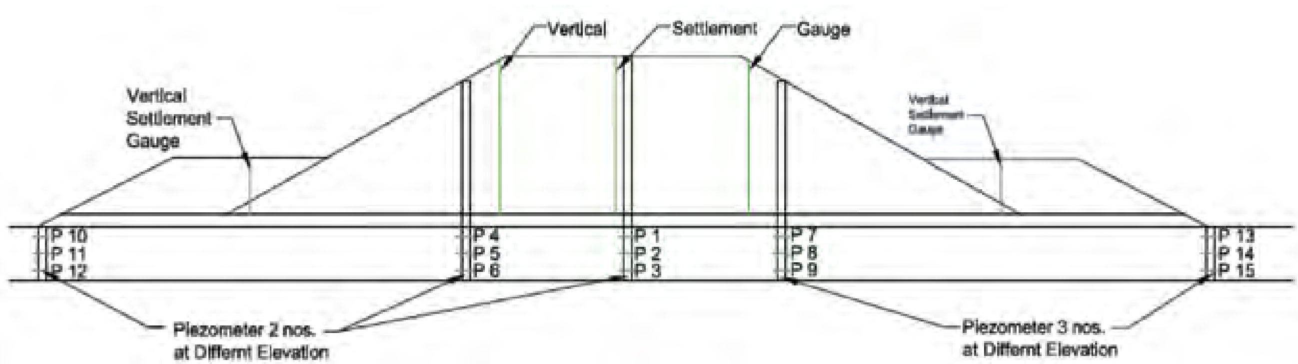


Fig. 6.11 Typical Cross Section of Earth Embankment with Instrumentation

Notes:

1. Piezometer and vertical settlement gauge should be staggered longitudinally to avoid interference with each other.
2. All instruments to be protected by chamber of approximate size 30 cm x 30 cm x 45 cm depth where necessary

Initial readings of all the instruments shall be recorded as reference values. Subsequently periodic data shall be collected and analyzed.

Such data recording and analysis shall be taken up by the geotechnical engineer assigned to the project and if necessary he should be in close contact to the designer. This will help in early identification if any signs of impending failure.

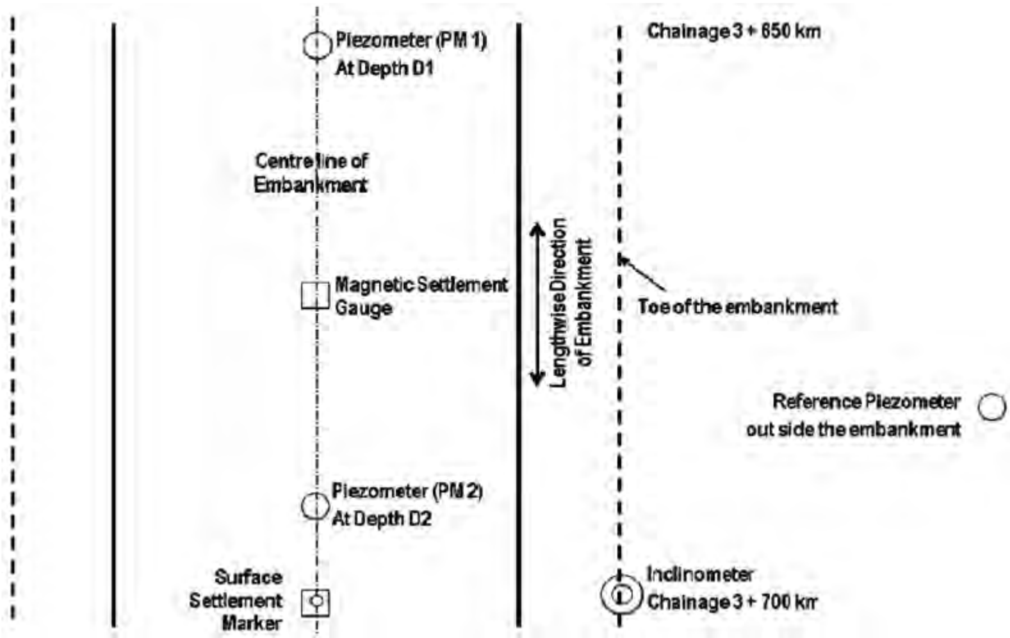


Fig. 6.12 Typical Plan for Instrument Installation in Embankment over Soft Soil

Spacing of Monitoring Instruments

Spacing of various monitoring instruments will depend on factors such as height of embankment, depth of soft clay layer, total length of the stretch etc. Hence to a certain extent, the designer has to use his engineering judgment. However, as a general guideline the following may be kept in mind.

a) In-situ shear strength and its change with time (due to progress of consolidation) shall be monitored every 100 m either by in-situ vane shear or taking undisturbed samples and testing them in lab for shear strength, water content and void ratio. In either case accuracy and care is essential and all precautions regarding recovery of samples from soft clay shall be ensured. Samples collected shall be tested without any delay.

   Emphasis is placed on changes in shear strength since this parameter is most vital in controlling the rate of construction.

b) Likewise, it is desirable to measure pore water pressure changes at every 100 m intervals.

c) Settlements, heave, lateral movements may be measured at every 200 m.

If the designer has any reason to believe that there may be problems of settlement/ heave/ lateral displacements spacing may be decreased to 100 m. These guidelines may be followed for embankments of length upto 1km, where the construction on weak ground extends for many kilometres, spacing needs to be adopted keeping in view the changes in height and changes in the sub soil profile

6.8.2 *Observational Procedure for Settlement Prediction and Degree of Consolidation*

An important aspect of instrumented monitoring of embankments over soft clay pertains to, estimation of percentage of consolidation achieved in the sub-soil. The degree of consolidation is directly proportional to settlement of sub-soil. Theoretically the settlement of the sub-soil can be computed using Terzaghi's one dimensional consolidation theory. However natural clay deposits consist of multilayered soils having different properties. Limited number of laboratory tests on sub-soil with respect to consolidation behavior can provide us with only local and microscopic information about the ground and the results may be unreliable due to sample disturbances. Hence, such variations in the sub-soil properties affect theoretical prediction of total settlement.

To overcome this problem observational procedure of settlement prediction as proposed by 'Akira Asaoka' (1978) can be adopted. Broadly, this procedure consists of extrapolating the settlement values observed in the field to estimate the final (ultimate) total settlement. For calculating ultimate total settlement, time vs. settlement curve for the settlement data available as on date is first drawn. The total time period in this time-settlement curve is divided into convenient number blocks of equal time interval and settlement value at each time interval is noted **(Fig 6.13).** To predict the ultimate total settlement, settlement at particular time interval is (St) is plotted along Y axis, while the corresponding settlement for the previous time interval (St-1) is taken as X coordinate of such points. These points are joined together to intersect a 450 line drawn from the origin. The point where this line intersects the 450 line indicates final settlement **(Fig 6.14)**. To determine the degree of consolidation achieved, total settlement of the ground below the embankment (recorded through surface settlement marker or magnetic settlement gauge) as on date is compared with predicted final settlement (as per Asaoka's method) and percentage of consolidation is calculated.

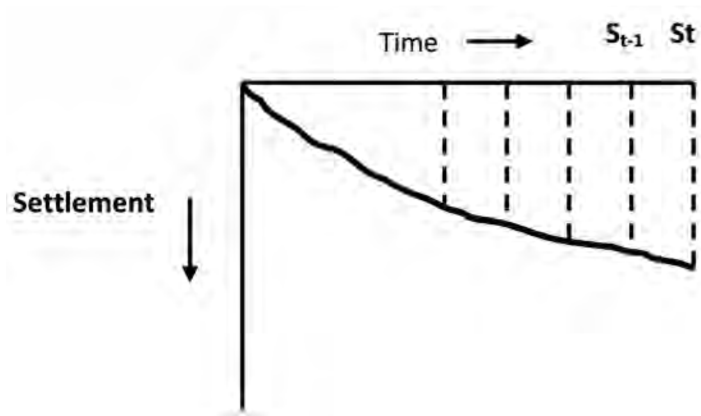


Fig. 6.13 Graphical Method of Asaoka

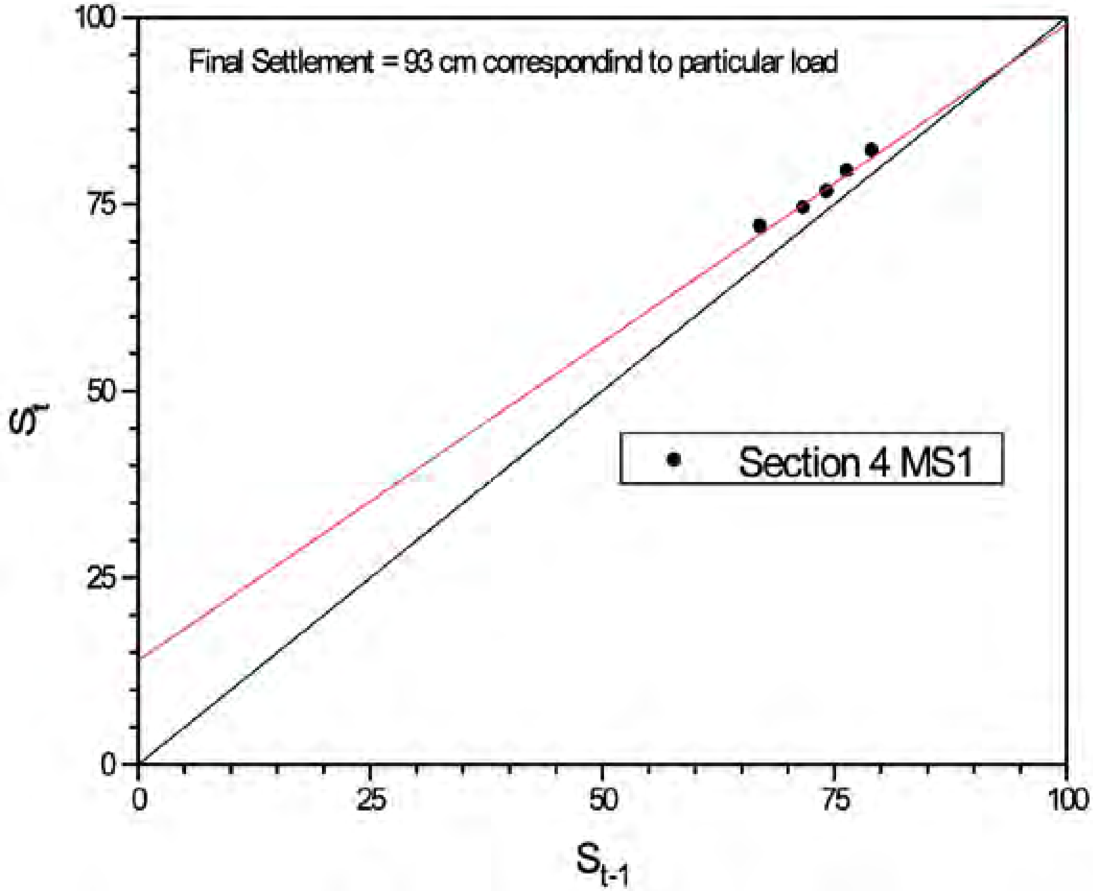


Fig. 3.14 Predicting Final Settlement Using Asaoka Method

6.8.3 *Control of Embankment Stability*

When soft sub-soil is subjected to loading, not only consolidation but also plastic horizontal flow (shear deformation) occurs. This makes it difficult to distinguish between the settlement occurring due to consolidation and settlement/ displacement due to plastic flow of sub-soil. Qualitatively, failure will occur when the progress of the shear deformation (plastic flow of sub-soil) is faster than that of consolidation settlement. Hence, if embankment construction is carried out at a rapid pace, plastic flow may exceed the rate of consolidation settlement, thereby endangering the stability of the embankment. This makes it especially important to monitor plastic flow of sub-soil and correlate it with the consolidation settlement.

While settlement (ρ_t) at the centre of the embankment can be monitored using settlement marker or magnetic settlement gauge, lateral displacement (plastic flow of subsoil) δ of the sub-soil can be obtained using inclinometer data. The progress of δ in relation to ρ_t can be used as an indicator of embankment stability and also for any impending failure. Using these two parameters, Minoru Matsuo et al (1977) developed an observational method for prediction of failure of embankment constructed on soft subsoil. They developed a failure prediction diagram based on the settlement data (ρ_t) and plastic flow of sub-soil (δ) obtained from monitoring data for many embankments constructed on soft grounds **(Fig 6.15).** For using this diagram to verify the stability of embankment being constructed on soft sub-soils, the settlement data (ρ_t) and plastic sub-soil movement (δ) are to be plotted in the diagram **(Fig 6.15)** for different time intervals. The position where these points lie shows approximate Factor of Safety (FOS) of the embankment as on date. In case the points lie very near to the FOS line of 1.0 and show a tendency to move further up (where in FOS would be less than 1), adequate precautions in the form of stopping further loading/ construction of embankment (may be even removing a part of loading) and increasing the frequency of recording field monitoring data should be resorted to till the points move further down from FOS equal to 1.0 line.

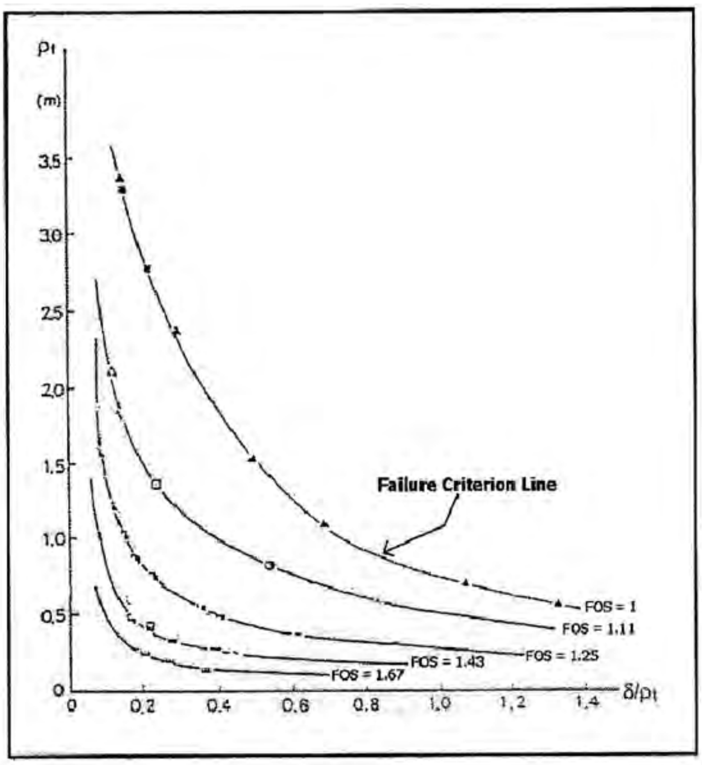


Fig. 6.15 Minoru Matsuo Embankment Stability Diagram

APPENDIX-A

Solved Examples

1. A simple embankment in approach to a bridge is of 12 meter height and has side slopes at 45 degrees. The soil is saturated and highly impervious. At present the embankment is completely submerged due to backwater flow from a river. Back waters generally recede in a short time and eventually the water table recedes to an average level somewhat below the toe of the slope.

Laboratory tests on specimen of soil used in embankment gave the following soil characteristics:

Bulk density=19.62KN/m^3

c'=29.43KN /m^3

Ø'=20°

Determine the Factor of Safety under different conditions using Taylor's Charts:

Solution:

a) *Submerged Case*

From **Fig. 3.3** for i=45 degrees and ø=20 degrees

Stability Number=0.062

$$0.062 = \frac{C}{F\ X\ \gamma\ buoyant\ X\ H} = \frac{29.43}{F\ (19.62 - 9.81)\ X\ 12}$$

$$F= \frac{29.43}{0.062\ X\ 9.81\ X\ 12} = 4.03 \text{ (safe)}$$

b) Sudden Drawdown Case

$$ø_w = \frac{19.62-9.81}{19.62} \times 20° = 10°$$

For ø= 10^0 and i=45° from **Fig. 3.3**

Stability Number=0.11

$$0.11= \frac{3}{FX\ 19.62\ X\ 12}$$

$$F= \frac{29.43}{0.11 \times 19.62 \times 12} =1.137\text{(Just Safe)}$$

c) *Normal Case with Embankment Saturated*

For ø=20° and i=45°,

Stability number=0.062

$$0.062= \frac{29.43}{FX\ 19.62\ X\ 12}$$

F=2.01 (Safe)

2. An embankment having uniform side slopes of 1.5 horizontal to 1 vertical was built at a very slow rate to a height of 6 m on rocky foundation with the provision of a toe filter as shown in **Fig. 3.10.** The average effective stress parameters of the embankment material (r=2T/m^3), were found to be c'=440 kg/m^2 and ø'=32°. Determine the stability of the sliding mass in terms of effective stress using (a) Swedish Slip Circle Method (b) Bishop Routine Method. Assume that the pore water pressures along the potential slip surface are governed by the steady state seepage flow net.

Solution: The sliding mass could be divided into any finite number of vertical slices. In the present case, the embankment slope being uniform and its composition being homogeneous, seven slices in all, **Fig. 1**, are considered adequate.

Stability analysis' in terms of effective stress, in general, requires two basic calculations viz., the calculation of the weights of the slices and porewater pressure acting on them. The weight vectors could be calculated by multiplying the average height of any slice with its average width and unit weight. The pore water pressure values are either available from piezometric observations or from steady state seepage flow net. In this case, seepage flow net takes the shape shown in **Fig.1.**

Based on equations presented in Chapter-3, the calculations using Swedish Slip Circle Method and Bishop Routine Method are presented in Table 1 and Table 2 respectively. The presentation is self-explanatory.

3. A compacted clay fill c' = 0; ø = 28° and γ = 1700 kg/m^3) is required to be placed above an old embankment, at a uniform slope of 30° with the horizontal in order to meet the renewed requirements of grade and top width. Examine the stability of the newly placed fill, in terms of effective stress, along the interface c' = 0; ø= 19°) using Janbu's Method. Compare the results with those obtained if, interfacial sliding is inhibited and, failure is considered possible only by sliding within the compacted Clay fill. Assume the piezometric head on the surface of sliding as shown in **Fig. 2.** What would be the corresponding results if the piezometric head is to drop down to zero all along the slip surface?

Solution

The problem involves effective stress analysis on a non-circular slip surface using the method suggested by Janbu. Porewater pressure conditions have been defined in **Fig. 2.**

The calculations corresponding to two cases, viz., (a) porewater pressures as defined by the piezometric head (b) zero pore water pressure, are furnished in two separate Tables in **Fig. 32.** These calculations are self explanatory.

It must be noted that, if interfacial sliding is prevented, the factor of safety improves. Towards this effect, it helps to provide suitably cut benches (discontinuous boundary between the old and the new fill) which improve the shearing resistance of the sliding mass along the interface.

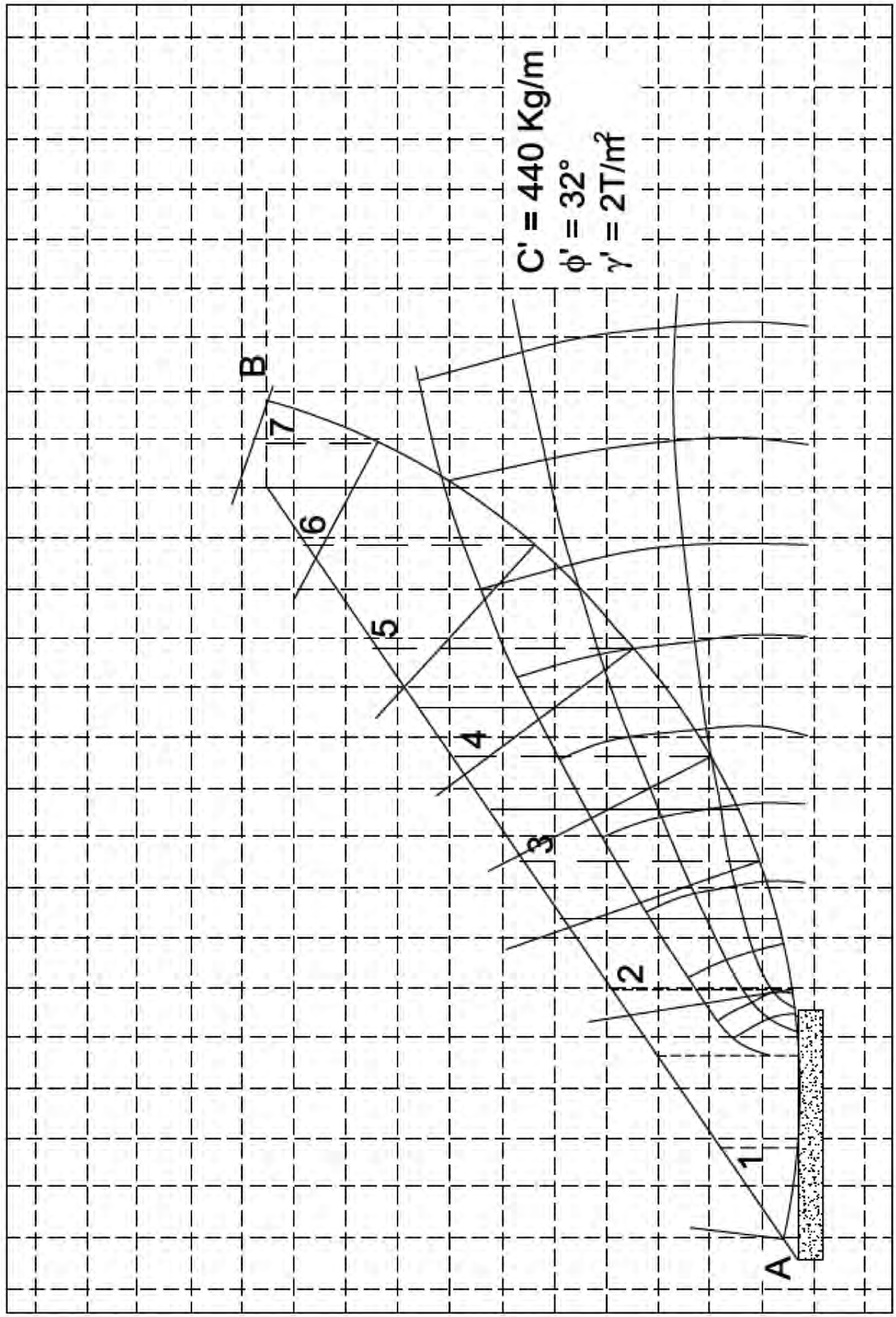


Fig .1 Stability Analysis in Static Conditions

$$F = \frac{\Sigma Cl + \Sigma (W \cos a - ul) \tan \phi'}{\Sigma W \sin a}$$

Table 1 Swedish Slip Circle Method

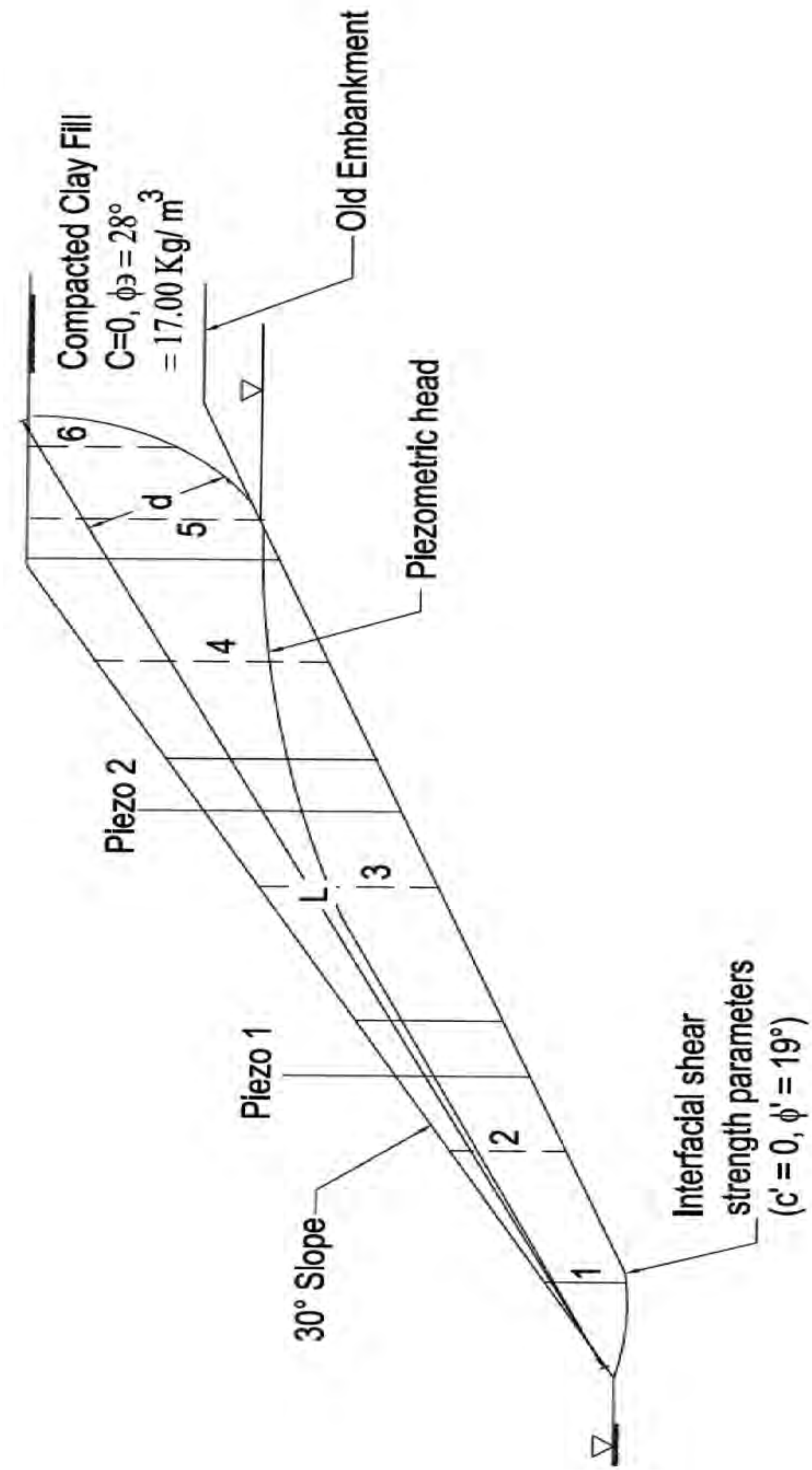
1	2	3	4	5	6	7		8		9	10	11	12	13	14
SLICE No.	α	Cos α	Sin α	l	b	Weight of the Slice Average Weight		Pore Water Pressure		ul $x10^3$	C'l	W Cos α $x10^3$	W Sin α $x10^3$	(W Cos α–ul) $x10^3$	Factor of safety $\frac{\Sigma Col.10+\Sigma Col.13}{\Sigma Col.12}$
						Ht. (m)	Weight W10³	Pore Water Ht.	Pore Pressure U						
1	0	1	0	2.16	2.16	0.85	3.66	0	0	0	950	3.66	0	3.66	1.16
2	12	.978	.208	1.84	1.8	1.92	6.80	84	.84	1.54	810	6.65	1.42	5.11	
3	22	.927	.208	1.29	1.2	2.52	6.05	1.32	1.2	1.41	565	5.61	2.27	4.20	
4	31	.854	.515	1.4	1.2	2.76	6.60	1.32	1.32	1.84	615	5.65	3.4	3.81	
5	40	.766	.643	1.56	1.2	2.70	6.47	1.02	1.02	1.59	685	4.97	4.16	3.88	
6	51	.629	.771	1.91	1.2	2.40	5.75	.48	.48	.92	840	3.62	4.44	2.70	
7	63.5	.446	.895	2.0	0.9	1.08	1.92	0	0	0	880	0.855	1.72	0.855	
											Σ5345		Σ17.41	Σ23.715	

$$F = \frac{1}{\Sigma W Sin a} \sum \left[\{cb + (W(1-r_u)\tan\phi\} x \frac{1}{ma} \right]$$

Where mα = cosα $\left(1+\frac{\tan\alpha \tan\phi'}{F}\right)$

Table 2 Bishop Simplified Method

SLICE No.	Width b (m)	Av. ht. h (m)	Weight W KGx10^3	α	Sin α	W Sin α x10^3	C'b	u x10^3	u'b x10^3	(W-ub) tanφ' * KGx10^3	Col.(8+1) x10^3	Sec α	tan α	$\frac{\sec\alpha}{1+\frac{\tan\phi+\tan\alpha}{F}}$		Col.12X15	
														F = 1.1	F = 1.3	F = 1.1x10^3	F = 1.3
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15		16	
1	2.16	0.85	3.66	0°	0	0	950	0	0	3.66x6248=2.28	3.23	1	0	1	1	3.23	3.23
2	1.8	1.92	6.80	12°	.208	1.92	790	.84	1.51	5.20x6248=3.3	4.09	1.02	.212	.91	.925	3.725	3.8
3	1.2	2.52	6.05	2.2°	.375	2.27	530	1.2	1.44	4.61x6248=2.87	3.4	1.078	.404	.875	0.9	2.97	3.05
4	1.2	2.76	6.60	31°	.515	3.4	530	1.32	1.58	5.02x6248=3.14	3.67	1.116	.601	.87	.905	3.2	3.3
5	1.2	2.7	6.47	40°	.643	4.16	530	1.02	1.22	5.25x6248=3.28	3.81	1.305	.839	.885	.93	3.36	3.54
6	1.2	2.4	5.75	51°	.771	4.44	530	0.48	.575	5.175x6248=3.24	3.77	1.589	1.235	.935	1.0	3.51	3.77
7	0.9	1.08	1.92	63.5°	.895	1.72	395	0	0	1.92x6248=1.2	1.595	2.241	2.00	1.05	1.14	1.68	1.82



(a)

Fig. 2: Stability Analysis by Janbu's method

b) *Janbu's Method*

SLICE No.	α	$\tan\alpha$	Pressure Due to Self weight p kg/m²	Pore water Height h m	Pore Water Pressure $u=\gamma_w h$ kg/m²	(p-u) kg/m²	b m	$\cos^2\alpha(1+\frac{\tan\alpha\tan\phi'}{F})$ F=1	F=0.8	$\frac{b}{n\alpha}$ (p-u) F=1	F=0	ptanα b	$F=\frac{f_o\Sigma\frac{(p-u)b\tan\phi'}{n}}{\Sigma pb\tan\alpha}$
1	0	0	1190	0.45	450	740	2	1.00	1.00	1480	1480	0	
2	21	0.3838	3060	1.00	1000	2060	5	0.985	1.01	10450	10080	5860	$F_1=1.03\ \frac{55850\tan19°+9020\tan28°}{32590}$
3	21	0.3838	4500	1.30	1300	3200	3	0.985	1.01	16200	15900	8630	=0.76
4	21	0.3838	5780	0.80	800	4980	3.8	0.985	1.01	19200	18600	8030	$F_f=1.03\ \frac{54230\tan19°+8300\tan28°}{32590}$
5	21	0.3838	5780	0	0	3780	1.7	0.985	1.01	10000	9650	3780	=0.74
6	58	1.6003	3910	0	0	3910	1.0	0.519	0.57	7540	6820	6250	

D=2.2 m L=20.6

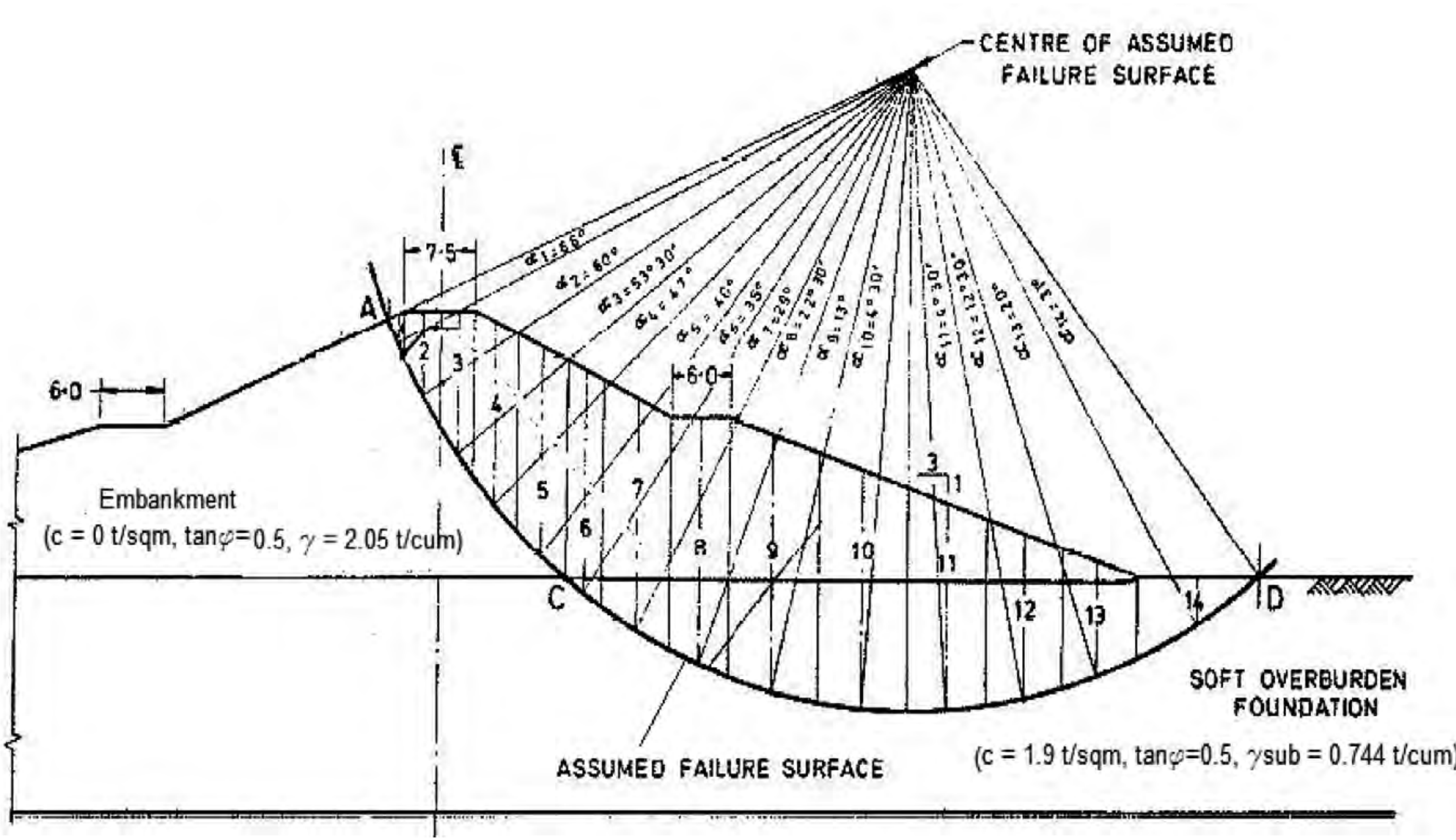
d/L=0.07 f_o=1.03

c) *Zero Pore Pressure Case*

SLICE No.	P kg/m²	nα F=1	$\frac{b}{n\alpha}$ (p)	$F=\frac{f_o\Sigma\frac{(p-u)b\tan\phi'}{n}}{\Sigma pb\tan\alpha}$
1	1190	1.00	2380	
2	3060	0.985	15500	
3	4500	0.985	22800	$F=1.03\ \frac{70500\tan19°+9920\tan28°}{32590}$
4	5780	0.985	22200	=0.94
5	5780	0.985	10000	
6	3910	0.519	7540	

Stability analysis for seismic case

Typical calculations for working out the factor of safety of high embankment for a condition of seismic case in Zone IV as given by arithmetical method are given in Table 3 and **Fig. 3.**



CIRCULAR ARC METHOD

All dimensions in metres,

Fig. 3: Typical Calculations for Seismic Slope Stability Analysis by Method of Slices

Table 3 Computation of Pseudo-Static Factor of Safety by Method of Slice

Slice No.	Horizontal Width (m)	Weight of the Slice (m)			Area m²	Weight of the Slice	Height of Slice (m)			Area m²	Weight of the Slice	Total Weight of Slice (W)	Angle by the radius with center of slice (degree)			Effective Normal Force (N)	Driving Force (Tonne)	N. tanø	Resisting Forces	
		Embankment			ton		Foundation				ton	ton	a	cosa	sina	Wcosa	Wsina	tanø = 0.5	Embankment	Foundation
		Left	Right	Average	y= 2.05 t/m³		Left	Right	Average		y_{sub} = 0.744 t/m³					ton			c = 0, L=29 m	c = 1.9 t/m³, L=71 m
1	1.55	0	4	2	3.10	6.36						6.36	66	0.407	0.913	2.588	5.804	1.294	0	134.9
2	3.75	4	8.75	6.375	23.91	49.01						49.01	60	0.500	0.866	24.526	42.429	12.263	0	
3	3.75	8.75	14	11.375	42.66	87.45						87.45	53.3	0.598	0.801	52.293	70.087	26.146	0	
4	4.50	14	16.25	15.125	68.06	139.53						139.53	47	0.682	0.731	95.200	102.005	47.600	0	
5	4.25	16.25	18.5	17.375	73.84	151.38						151.38	40	0.766	0.643	115.998	97.264	57.999	0	
6	3.25	18.5	16.5	17.5	56.88	116.59	0.00	2.00	1.00	3.25	2.418	119.01	35	0.819	0.573	97.510	68.232	48.755	0	
7	6.75	16.5	13.5	15	101.25	207.56	2.00	6.00	4.00	27.00	20.088	227.65	29	0.875	0.485	199.136	110.316	99.568	0	
8	6.00	13.5	13.5	13.5	81.00	166.05	6.00	8.00	7.00	42.00	31.248	197.30	22.3	0.925	0.379	182.557	74.830	91.278	0	
9	9.00	13.5	10.5	12	108.00	221.40	8.00	10.25	9.13	82.13	61.101	282.50	13	0.974	0.225	275.268	63.517	137.634	0	
10	8.00	10.5	8	9.25	74.00	151.70	10.25	11.00	10.63	85.00	63.24	214.94	4.3	0.997	0.075	214.336	16.108	107.168	0	
11	7.75	8	5.5	6.75	52.31	107.24	11.00	10.50	10.75	83.31	61.9845	169.23	4.3	0.997	0.075	168.749	-12.682	84.375	0	
12	7.00	5.5	3	4.25	29.75	60.99	10.50	9.00	9.75	68.25	50.778	111.77	12.3	0.977	0.213	109.203	-23.798	54.601	0	
13	7.00	3	0	1.5	10.50	21.53	9.00	6.50	7.75	54.25	40.362	61.89	20	0.940	0.342	58.159	-21.156	29.079	0	
14	11.00	-	-	-			6.50	0.00	3.25	35.75	26.598	26.60	31	0.857	0.515	22.803	-13.693	11.401	0	
																1618.325	579.263	809.1620.000		134.900

$k_H = 0.5 \times 0.24 = 0.12g$

Total Resisting Force = 809.162 + 134.9 - 579.263 x 0.5 x 0.12 = 909.307 t

Total Driving Force **= 579.263 + 1618.325 x 0.12** **= 773.462 t**

Factor of Safety = 909.307 / 773.462 = 1.176

Hence OK

Typical Calculations for a condition of seismic case in zone IV for the example 3.3 is given in the following table

Table 4 Computation of Pseudo-Static Factor of Safety by Method of Slices

Slice No.	Slice Details (m)			Weight of the Slice (W)	Angle by the radius with center of slice (degree)			Effective Normal Force (N)	Driving Force (T)	N. tanø	Resisting Forces
	Horizontal Width	Length	Height	ton	a	cosa	sina	Wcosa	Wsina	tanf = 0.625	Embankment
				g = 2 t/m³				ton			c = 440 kg/m²
1	2.16	2.16	0.85	3.67	0	1.000	0.000	3.672	0.000	2.295	950.4
2	1.80	1.84	1.92	6.91	12	0.978	0.208	6.761	1.436	4.226	809.6
3	1.20	1.29	2.52	6.05	22	0.927	0.374	5.608	2.265	3.505	567.6
4	1.20	1.4	2.76	6.62	31	0.857	0.515	5.679	3.410	3.549	616
5	1.20	1.56	2.70	6.48	40	0.766	0.643	4.965	4.164	3.103	686.4
6	1.20	1.91	2.40	5.76	51	0.630	0.777	3.627	4.475	2.267	840.4
7	0.90	2	1.08	1.94	63.5	0.447	0.895	0.868	1.739	0.543	880
								31.181	17.488	19.488	5350.400
											5.3504

Total Resisting Force = 5.35+19.488-17.488*0.625*0.12 = 36.575t

Total Driving Force = 33.279 t

Factor of Safety = 36.575 / 33.279 = 1.1

Hence o.k

Examples on Settlement Analysis

1. An embankment 8 m height is built over a clay layer of 4.27 m thickness which is underlain by impervious rock. The soils involved have the following characteristics:

Sub Strata

C_c= 0.2634 from consolidation test result

C_v= 0.947 X 10-^{4} cm^2/sec

S_p. gr. G= 2.67

d= 14.22 kN/m^3

n= 0.486

Embankment soil

Ysat = 22.54 kN/m^3

Y_d= 20.4 kN/m^3 gm/cc

Determine (i) the total settlement

(ii) Time for total settlement

(iii) Settlement at the end of 1 yrs., 2 yrs., 3 yrs., 4 yrs., and 5 yrs.

Solution

Assuming the worst case i.e. embankment to be saturated and substrata as submerged.

For subsoil $= (1-n)G\,\gamma_w + n\,Y_w$

$=18.23$ kN/m^3

Initial void ratio e_o $= \frac{G y_w - y_d}{yd}$

$= \frac{26.17-14.22}{14.22}$

$=0.841$

Pa= Y_b X depth

$=8.420\text{X} \frac{4.27}{2} = 17.98$kN/m^2

$\Delta P = Y_{sat}$ X h = 22.56 X 4 =90.25kN/m^2

Total settlement $= \frac{Cc}{1+e_o} H \log_{10} \frac{P_o+\Delta P}{P_o}$

$= 4.27\text{X} \frac{0.2634}{1+0.841} H \log_{10} \frac{17.98 + 90.25}{19417.98}$

= 0.476 mm = 47.6cm

C_v = 0.947 X 10-^{4} cm^2/sec

= 0.947 X 10-^{4} X 365 X 24 X 60 X 60 cm^2/yr

= 2985 cm^2/yr

Time factor T = $\frac{C_v t}{H_2}$

For t = 1 yr, Ti = $\frac{2985 \times 1}{427^2}$ = 0.001636

From U-T Table for one way drainage and pressure distribution in columns 2 of Table 4.2

For T =0.01636, U=0.1432

Settlements in 1 yr; S1 =47.6 X 0.143= 6.81 cm

Similarily,

t=2 yrs,	T_2=0.03272,	U_2=0.2040,	S_2=9.71cm
t=3 yrs,	T_2=0.04908,	U_2=0.2489,	S_2=11.85cm
t=4 yrs,	T_2=0.06544,	U_2=0.2785,	S_2=13.26cm
t=5 yrs,	T_2=0.08180,	U_2=0.3204,	S_2= 15.25cm

Settlement per year is only 3 to 4 cm

For U= 0.9942 T=2

Period for total settlement = t = $\frac{2 \times 427^2}{2985}$ =122 yrs

2. A clay layer, 9m thick, is underlain by impervious rock and is covered with free drainage sand. Laboratory consolidation test on a 2.5 cm thick sample, obtained from the clay layer, requires 500 seconds for 50 percent consolidation. The laboratory sample was drained both at top and at bottom. Calculate the time required for 50 percent consolidation in the field.

Solution: The value of dimensionless time factor, T, for 50 percent consolidation both in the field and the laboratory is the same.

$$t = \frac{TH^2}{C_v}$$

Assuming that coefficient of consolidation C_v remains constant for all ranges of pressure, with subscripts f and l referring to field and laboratory samples respectively,

$$\frac{C}{T} = \frac{H_l^2}{t_l} = \frac{H_f^2}{t_f}$$

Since the laboratory sample has two-way drainage.

H_l= $\frac{2.5}{2}$ =1.25 cm

$$\frac{1.25^2}{500} = \frac{(9x100)^2}{t_f}$$

i.e. t_f= $(900)^2$X $\frac{500}{1.25 \times 1.25}$ seconds

= 3000 days.

Example of Ground Improvement using PVD and Stone Column

Table 5 Description of Subsoil Properties

Depth Below GL (m)		Description of Strata	Sand%	Silt%	Clay%	N-avg	Layer thickness (m)	ϕ^0	C (kN/sq.m)
From	To	Silty Clay	4.00	53	43	4	7.00	0	14
0.00	7.00	Completely weathered rock							
8.00	12.00	Highly to moderately weathered rock							

Atterberg properties of clay strata, LL= 70%, PL=39%, PI=31%. Natural Moisture Content = 68%, Bulk density = 15.12 kN/m^3, dry density=9kN/m^3, Void ratio= 2.226, compression index C_c=0.484

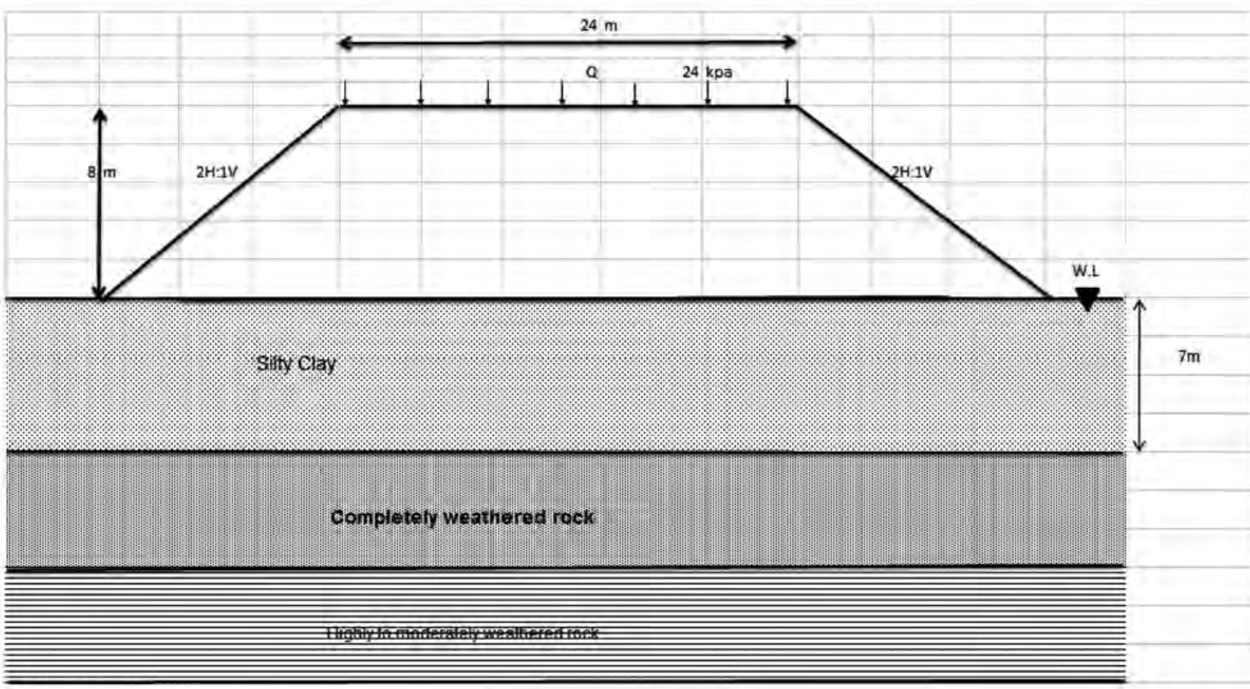


Fig. 4: Use of PVDs for Ground Improvement

The design of embankments on soft ground using PVDs, stage construction accompanying strength improvement are discussed in this section

PVDs installed in triangular pattern at a spacing of 1m are adopted in this example and typical parameters of PVDs are as follows

Width of band drain	b	=	100	mm
Thickness of band drain	t	=	4	mm
Spacing of band drain	s	=	1	m
Pattern of installation		=	Triangular	

Coefficient of vertical consolidation	C_v	=	3.00E-04	cm^2/sec
			0.94608	m^2/year
Ch/Cv ratio of the soil		=	1.5	
Coefficient of horizontal consolidation	C_h	=	4.50E-04	cm^2/sec
		=	1.41912	m^2/year
Drainage (1 = single, 2 = double)		=	1	
Drainage path		=	7 m	
Area treated by single band drain	A	=	0.866 m^2	
For Rectangular Grid - S2				
For Triangular Grid - 0.866 x S2				
Equivalent diameter of cylindrical column	D	=	2 * sqrt (A/p)	
		=	1.0500597	m
Equivalent diameter of band drain	d	=	2 * (b + t)/p	
		=	66.208456	mm
		=	0.0662085	m

Time required for Degree of radial consolidation = 90%

Hansbo's Equation, (Hansbo, S. (1981), Consolidation of fine-grained soils by prefabricated drains, Proc. 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, Vol. 3, pp. 12-22.)

$$t = \frac{D^2}{8 \times C_h} \times \left[\frac{1}{1-\left(d/D\right)^2} \times Ln\left(\frac{D}{d}\right) - \frac{3}{4} + \frac{1}{4}\left(\frac{d}{D}\right)^2 \right] \times Ln\left(\frac{1}{1-U}\right) \quad \text{...Eqn 1}$$

As per Hansbo's Equation (based on the above parameters), following table has been prepared for various percentage of consolidation, for the design parameters adopted.

Since, the permeability of clayey soil are very low, it requires much more time for the consolidation process.

Time required, $t = \frac{T_v x d^2}{C_v}$ (from eqn 4.7)

where,

t is time required for specified degree of consolidation

Table 6 Time vs. Consolidation, with PVDs for the Spacing Adopted

Ur %	t (days)
0	0.00
5	3.68
10	7.57
15	11.67
20	16.02
25	20.66
30	25.61
35	30.94
40	36.68
45	42.93
50	49.78
55	57.34
60	65.80
65	75.39
70	86.46
75	99.56
80	115.58
85	136.24
90	165.36

Time required for 90% consolidation (without PVDs)

Time factor for the drainage system (Single face Drainage, 90% verticalconsolidation)

T_V = 0.848

Length of drainage path, d = 7 m (Single face Drainage)

Coefficient of vertical consolidation, C_v = 0.94608 m^2/year

Hence, time required for consolidation = 43.9 Yrs

= 16031 days

Since the time required is so large that PVDs need to be used to reduce the time for consolidation.

<u>Time required for 90% consolidation using PVDs</u>

Time required for consolidation using PVDs using Hansbo's equation (see equation 1 and Table 6)

Hence, time period for consolidation, t = 0.45 Yrs

= 5.44 Yrs

= 165.36 Days

Hence, from the above calculation it is seen that the process of consolidation is accelerated using PVDs, which saves time and is a practically feasible solution.

Bearing Capacity Considerations

If the bearing capacity is not adequate to carry the required height of embankment, stage construction in conjunction with PVDs may be adopted.

The design will be based on gain in shear strength in each stage being used to build the next stage. Execution time for construction of embankment has to be worked out taking in to the consideration time taken to achieve required shear strength which is a function of degree of consolidation.

Following equation given by Skempton's formula, which is based on plasticity index of soil, may be adopted to find the enhanced cohesion value.

Gain in strength, cohesion component, $\Delta c = k \times U \times \Delta\sigma$ … Eqn 2

Enhanced cohesion $c_f = c_i + \Delta c$ … Eqn 3

ci = Initial cohesion value

Where, $k = 0.11 + 0.0037 \times PI$ … Eqn 4

U= Degree of consolidation

$\Delta\sigma$ = increased overburden pressure (Due to embankment construction)

In the example given below an embankment of 8m height is designed to be built on a soft clay layer of 7 m thick using PVD's with stage construction.

Input Parameters for Embankment

Top width of embankment=24 m

Height of embankment = 8 m

Side slopes= 1V:2H

Unit weight of fill material= 18kN/m^3

Thickness of soft clay layer= 7 m, this layer is followed by hard stratum

Properties of soft clay

c=14kN/m^2

Liquid limit=70%

Plastic limit= 39%

Calculation of Bearing Capacity

Base width of embankment (B) = 56 m

Depth of soft clay (D) = 7 m

B/D=8

Nc= 4.14 +0.5 x B/D = 8.14 (As per IRC-75) ….Eqn 5.

Bearing Capacity of clay layer= 114 kN/m^2

Load due to embankment= 8 x 18= 144 kN/m^2

Hence soil will not be able to take the entire load of 8m embankment in single stage; hence two stage construction will be adopted with use of PVD's to reduce the waiting period.

First Stage Construction (Assume 4m will be built)

For 4 m height of embankment FOS in bearing capacity = (8.14 x 14)/ (4 x18) = 1.58

Hence 4 m high embankment can be built in the first stage.

Assume PVD's have been installed before the initial stage of construction.

Required bearing capacity for 8m high embankment= 144 x 1.25= 180 kN/m^2

Provide 6 months waiting period after the first stage of embankment is built which is time required for achieving 90% consolidation, as per Table 5.1 given by Hansbo

Bearing Capacity for Second Stage

In order to check the bearing capacity for second stage of embankment gain in strength due to first stage surcharge load is calculated using Skempton's formula ref IRC: HRB -13

Gain in strength due to consolidation, cohesion component, $\Delta c = k \times U \times \Delta\sigma$

Enhanced cohesion $c_f = c_I + \Delta c$

c_I = Initial cohesion value

Where, $k = 0.11 + 0.0037 \times PI$

U = Degree of consolidation (90% in this case)

$\Delta\sigma$ = increased overburden pressure (Due to embankment construction)

In this case K= 0.11 + 0.0037 x 39= 0.254

Hence Δc= 0.254 x 0.9 x 72 = 16.45 kN/m^2

Hence total cohesion available at the end of first stage waiting period = 14+ 16.45 =30.45 kN/m^2

Bearing capacity = 8.14 x 30.45= 247. 90kN/m^2 (As per eqn 5.)

Load acting on subsoil for 8m high embankment is 144 kN/m^2

Hence FOS=247. 90/144 = 1.7 is more than minimum required. Hence the design is satisfactory.

Increase in strength due to 8m embankment will be (after 2nd stage waiting period) due to additional 4 m embankment

Δc= 16.45 kN/m^2

Hence, Total cohesion available = 14 + 16.45 + 16.45 = 46.9 kN/m^2

Settlement Calculations (with no ground improvement adopted)

Top Formation Width= 24 m

Width of Embankment=56 m

Height of Embankment (H) = 8 m

Slope = 2H: 1V

Density of Embankment fill= 18 kN/m^3

Total Compressible Layer = 7 m

Foundation Soil

Dry density of soil= 9

Specific gravity of soil = 2.74

Initial void ratio = 2.226

Compression index Cc = 0.656

Submerged density=(G-1) x d/G

5.7 kN/m^3

Effective Overburden Pressure at Mid depth of soft clay layer Po = 5.7 x 3.5

= 20.00 kN/m^3

Stress due to 8m high embankment ΔP = 18 x 8 = 144 kN/m^3

Stress due to 4m high embankment ΔP = 18 x 4 = 72 kN/m^3

Settlement due to 8m high embankment =S = $\frac{Cc}{1+e_o} H \log \frac{P_o + \Delta P}{P_o}$ (from eqn 4.1)

$$\frac{0.656}{1+2.226} 7 \log \frac{20+144}{20}$$

= 1300 mm

Settlement due to 4m high embankment= S = $\frac{Cc}{1+e_o} H \log \frac{P_o + \Delta P}{P_o}$

$$\frac{0.656}{1+2.226} 7 \log \frac{20+72}{20}$$

= 943 mm

Settlements with Ground Improvement

Combined degree of consolidation (after 2^{nd} stage waiting period)

U = 1- ((1-Uz) x (1-Ur))

= 1-((1-0.15) x (1-0.9))

= 0.915

Hence combined degree of consolidation is 91.5%

Settlement remaining at the end of the second stage waiting period= (100-91.5) x total settlements

= (8.5 %) x total settlements

= 943 x 0.085

= 80.15 mm

Residual settlement of 80mm will decrease very slowly, since time Vs consolidation relationship becomes asymptotic at this point. This settlement is within the tolerable limits of 300 mm and hence allowable.

Factor of safety in Rotational Stability

Design cohesion values at the end of each stage waiting period are given below. These values are given in the calculation of rotational stability at the appropriate stage.

Without ground improvement: c= 14 kN/m^2

At the end of first stage waiting period with surcharge of 4 m: c= 30.45 kN/ m^2

At the end of second stage waiting period with additional surcharge of 4 m: c= 46.9 kN/ m^2

Table 7 Summary of Factor of Safety in Rotational Stability for Various Times and Loading Conditions

Height of Embankment	F.O.S	
4 m	1.129	Without considering ground improvement Note: The F.O.S increases steadily due to ground improvement
4 m	2.302	With ground improvement and end of 1st Stage waiting period.
8 m	0.545	Without ground improvement This is unsafe and hence ground improvements and stage construction is needed.
8 m	1.225	F.O.S with cohesion value corresponding to end of 1st stage waiting period.
8 m	1.50	F.O.S with cohesion value corresponding to end of 2nd stage waiting period and live load surcharge of 24 kN/m

Design Cohesion values

Untreated soil C = 14 kN/m^2

End of 1st stage waiting period (with 4 m embankment load) → C = 30.45 kN/m^2

End of 2nd stage waiting period (with 8 m embankment load) → C = 46.9 kN/m^2

Note:

1. In all the calculations the waiting period has been rounded off to 6 months, instead of the calculated value of 5.45 months (see Table 6)

2. It is assumed that the fill in each stage is placed in a relatively short time. Hence waiting period starts after the design fill height has been placed. i.e. some strength increase that will occur during the period required to place the fill is not taken into consideration. This is on the conservative side. However, it must be ensured at no stage, the F.O.S is below safe value.

3. It is essential that increase in shear strength and progress of settlement are monitored (see chapter. 6).

Good Construction Practices that may be Adopted

1. Granular blanket of 500 mm thickness may be placed on the soft ground. The granular blanket shall extend 500 mm beyond full width of embankment on either side including the additional PVDs provided at toe.

2. A layer of separator geotextile shall be placed above the soft clay, before the gravel layer is placed. Likewise a layer of separator geotextile may be placed between the bottom of the embankment fill and the gravel layer. The steps will ensure free drainage of gravel blanket and keep it free from contamination.

3. One layer of biaxial geogrid of 100kN x 100kN (minimum) ultimate tensile strength may be placed at the middle of the gravel layer. This will provide some rigidity to the gravel layer and uniformity of contact.

4. Minimum 3 rows of PVDs shall be provided beyond either toe. This will help provide lateral support at the toe.

Use of Stone Column as Ground Improvement

Design of stone column using soil parameters as shown below for 6 m high

Embankment	Soil parameters
C=	25
φ=	0
φ column =	40
γ=	15.6
γ'=	5.79

1st Approximation of Design:

Let's consider triangular design pattern with following governing design parameters.

Diameter of stone column, D = 1 **Assume**

Spacing, = 2.5 **Assume**

Column Pattern = Triangular

Area of stone column, As = 0.785 m^2

Equivalent diameter of unit cell, De = 1.05 x Spacing

2.625 m

Area of unit cell, A = 5.412 m^2

Calculation is performed on 3 basis (IS 15284 Part 1, Clause 9.3.2)

A. Capacity based on bulging of column

B. Capacity based on surcharge effect

C. Capacity based on bearing support by intervening soil

Calculation:

A. Capacity based on bulging of column

Limiting axial stress in column $\sigma_v = \sigma_{rL} * k_{pcol}$

$\sigma_{rL} = (\sigma ro + 4c_u)*k_{pcol}$

Where, $\sigma_{ro} = k_o \sigma_{vo}$ (Initial effective stress)

For, this problem,

kpcol = $\tan^2(45^o + \varphi^{column}/2)$

kpcol = 4.60

ko = 0.6 (For clay as per IS 15284 part 1)

$\sigma^{vo} = 2\gamma D = 11.58$

Now,

Limiting radial stress, $\sigma_{rL} = (\sigma_{ro} + 4c_u)*k_{pcol}$

513.15 kN/m^2

Safe load on column alone, Q1 = $(\sigma_v * A_s)$/FOS FOS = 2

= 201.5 kN

B. Capacity based on surcharge effect

q_{safe} = C*Nc/2.5 Where, Nc = 5.14

= 51.40 kN/m^2

Increase in radial stress ro = $q_{safe}*(1 + 2*ko)/3$

= 37.69 kN/m^2

Increase in ultimate cavity expansion stress radial stress = $r_o * Fq' * k_{pcol}$

= 173.35 kN/m^2

Where, Fq' = 1, for $\varphi = 0$

Safe load on column alone, Q2 = $r_o * F_q * k_{pcol} * A_s/2$ FOS = 2

= 68.07 kN

C. Capacity based on bearing support by intervening soil

Area of intervening soil for each column, Ag = A - A_s

= 4.626 m^2

Safe load taken by intervening soil, Q3 = q_{safe}*A_g

= 237.80 kN

Hence total safe load = A + B + C Q = 507.39 kN

Now, Meyerhoff's stress for 6 m high Embankment = 108 kN/m^2

(6m height, 18 kN/m3 as unit weight, no

Area of ground improvement A_{GI} = L x B ecc.)

= 10000 m2 Say 100 m x 100 m

Total load on the ground, Load T = σ x A_{GI}

= 1080000 kN

Hence, for this load on ground and available Q safe of ground let's do back calculation to check if spacing is sufficient

Number of stone columns N= Load T/Q

2128.552 No.

Therefore area per column Asc= AGI/N

= 4.698 m^2

Now, using the (De =1.05 x S) for this area

S=sqrt(4x Ac/(3.14 x 1.05 2)) = 2.329 m c/c

Since, the assumed spacing is greater than final spacing, Hence, we need to decrease the spacing of stone column and redo the calculation.

2nd Approximation of Design:

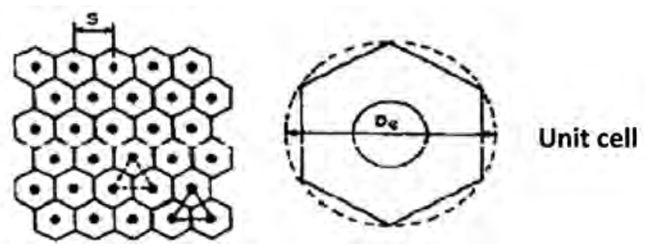
Let's consider triangular design pattern with following governing design parameters.

Diameter of stone column, D = 1 **Assume**

Spacing, S = 2.15 **Assume**

Column Pattern = Triangular

Area of stone column, As = 0.785 m^2



Equivalent diameter of unit cell, De= 1.05 x Spacing

2.258 m

Area of unit cell, A= 4.003 m^2

Calculation is performed on 3 basis (IS 15284 Part 1, Clause 9.3.2)

A. Capacity based on bulging of column

B. Capacity based on surcharge effect

C. Capacity based on bearing support by intervening soil

Calculation:

A. Capacity based on bulging of column

Limiting axial stress in column $\sigma_v = \sigma rL * k_{pcol}$

Where, σ_{rL} = Limiting radial stress

$\sigma_{rL} = (\sigma ro + 4c_u) * k_{pcol}$

Where, $\sigma_{ro} = k_o \sigma_{vo}$ (Initial effective stress)

For, this problem,

kpcol = $\tan^2(45^o + \varphi_{column}/2)$

kpcol = 4.60

ko = 0.60 (For clay as per IS 15284 part 1)

$\sigma_{vo} = 2\gamma D = 11.58$

Now,

Limiting radial stress, $\sigma_{rL} = (\sigma_{ro} + 4c_u) * k_{pcol}$

513.15 kN/m^2

Safe load on column alone, Q1 = $(\sigma_v * As)$/FOS FOS = 2

= 201.51 kN

B. Capacity based on surcharge effect

q_{safe} = C*Nc/2.5 Where, Nc = 5.14

= 51.40 kN/m^2

Increase in radial stress ro = q_{safe}*(1 + 2*ko)/3

= 37.69 kN/m^2

Increase in ultimate cavity expansion stress radial stress = r_o* Fq'*k_{pcol}

= 173.35 kN/m^2

Where, Fq' = 1, for φ = 0

Safe load on column alone, Q2 = r_o* Fq*k_{pcol}*A_s/2 FOS = 2

= 68.07 kN

C. Capacity based on bearing support by intervening soil

Area of intervening soil for each column, Ag = A - A_s

= 3.217 m^2

Safe load taken by intervening soil, Q3 = q_{safe}*A_g

= 165.37 kN

Hence total safe load = A + B + C

Q = 434.95 kN

Now, Meyerhoff's stress for 6 m high Embankment = 108 kN/m2 (6m height, 18 kN/m3 as unit weight

Area of ground improvement AGI = L x B)

= 10000 m^2 Say 100 m x 100 m

Total load on the ground, Load T = σ x AGI 1080000 kN

Hence, for this load on ground and available Q safe of ground let's do back calculation to check if spacing is sufficient

Number of stone columns N= Load T/Q

2483.035 No.

Therefore area per column Asc = AGI/N

= 4.027 m^2

Now, using the (De =1.05 x S)for this area

S=sqrt(4x Ac/(3.14 x 1.05^2)) = 2.157 m c/c

Since, the assumed spacing is matching with the final calculation. Hence, this design represents the optimum design of stone column.

Design summary

Diameter of stone column, D = 1.00

Spacing, S = 2.15

Column Pattern = Triangular

Area replacement ratio = 0.907*(D/S)^2 As per IS 15284:2003 Part 1, Clause 7.5.2

= 0.196

Settlement (without ground improvement) due to 4m high embankment= S = $\frac{Cc}{1+e_o} H \log \frac{P_o+\Delta P}{P_o}$

$\frac{0.656}{1+2.226} 7 \log \frac{20+108}{20}$

= 1147mm

Settlement reduction factor β = $\frac{1}{1+(n-1)A_s}$ Assume n=5 I.S. 15284 part 2 clause 9.3.2

= 0.24

Net settlements after ground improvement = 0.24 x 1147

= 275 mm < 300 mm

Since stone columns also acts as drains these settlements will decrease further rapidly as they are closely spaced. Stone columns must be tested for load bearing capacity as stated in I.S 15284 clause 9.3.2

All the good construction practices mentioned under PVDs shall be adopted for stone columns.

References

National Codes and Guidelines

1. IRC:36 Recommended Practices for Construction of Earth Embankments and Subgrades for Roadworks.
2. IRC:56 Recommended Practice for treatment of Embankment and Roadside Slopes for Erosion Control.
3. IRC-113- Guidelines for the Design and Construction of Geosynthetic Reinforced Embankments on Soft Subsoils
4. IRC:SP:58 Guidelines for Use of Flyash in Road Embankments
5. IRC:SP:11–Handbook of Quality Control for Construction of Roads and Runways.
6. IRC:78 Standard specifications and code of practice for road bridges-Section VII-Foundations and substructure.
7. HRB SR No 15 (1995), 'State of the Art:Landslide Correction Techniques', Published by Indian Roads Congress, New Delhi
8. HRB SR No 13 (1994), 'State of the Art:High Embankments on Soft Ground, Part A–Stage Construction', Published by Indian Roads Congress, New Delhi
9. HRB SR No 14 (1994), 'State of the Art:High Embankments on Soft Ground, Part B–Ground Improvement', Published by Indian Roads Congress, New Delhi
10. IS: 15284- part-2:Design and Construction for Ground Improvement—Guidelines for Pre-consolidation Using Vertical Drains.
11. IS: 15284-part-1 Design and Construction for Ground Improvement-Guidelines: Stone columns
12. IS: 7894: "Code of Practice for Stability Analysis of Earth Dams"
13. IS: 1498:"Classification and Identification of soil for general Engineering Purposes"
14. IS: 2720:Methods of tests for soils
15. IS: 11594:Mild steel thin walled sampling tubes and sampler heads
16. IS: 2132:Code of practice for thin walled tube
17. IS: 2131:Method for Standard penetration test for soils
18. IS: 4968:Method for subsurface sounding for soils
19. IS: 4434:Code of practice for In-situ vane shear test for soils
20. IS: 6403:Code of practice for determination of breaking capacity of shallow foundations
21. IS: 1892:Code of practice for subsurface investigations for foundations
22. IS: 1893: Criteria for Earthquake resistant: Design for structures.

23. IS: 789: Code of Practice for Stability Analysis of Earth Dams

24. IITK GSDMA-Guidelines for Seismic Design of Earth Dams and Embankments

25. MORTH-Specifications for Road and Bridge Works-fifth revision

International Codes and Guidelines

1. BS:8006 “Code of Practice for strengthened/Reinforcesd Soils and other Fills”.

2. ASTM D4719:Pressure meter test analysis report

3. ASTM D6635:Standard Test Method for Performing the Flat Plate Dilatometer

4. ASTM D 1143-Standard Test Method for Piles Under Static Axial Compressive Load.

5. FHWA-SA-97-077: Geotechnical Engineering Circular No.3: Design Guidance – Geotechnical Earthquake Engineering for Highways.

6. FHWA NHI-95-038 (1998), Geosynthetic Design and Construction Guidelines. Participant Notebook for NHI Course No.1 3 213

Other References

1. http:/commons.wikimedia.org/wiki/File:Vibrating_wire_piezometer_from_USBR_6515-09_Fig._1.png.

2. Abramson Lee. W., et. al (2002) “Slope Stability and Stabilization Methods”. Second Edition, Published by John Wiley and Sons Ltd.

3. Akca, N. 2003. Correlation of SPT-CPT data from the United Arab Emirates. Engineering Geology, V.67, 219-231.

4. Arora K. R.(2005) Soil Mechanics and Foundation Engineering, Standard Publishers, Delhi

5. Asaoka Akira (1978), ‘Observational Procedure of Settlement Prediction’, Journal of Soils and Foundations, Published by Japanese Society of Soil Mechanics and Foundation Engineering, Vol 18, No 4, pp 87-101.

6. Bhowmick Alok and Harpreet Singh, “Assessment of liquefaction potential for bridge design”.

   Indian Highways, September 2012.

7. Bishop, A.W. (1955)- The use of Slip circle in the stability of earth slopes-geotechnique Vol.5 (pp. 7-17)

8. Bishop, A.W. & Morgenstern, N.R. (1960) –stability coefficients for earth slopes. Geotechnique 10 (p. 129-150)

9. Bonaparte, R. and Christopher, B.R., Design and Construction of Reinforced Embankments Over Weak Foundations, Proceedings of the symposium on Reinforced Layered Systems, Transportation Research Record 1153, Transportation Research Board, Washington, D.C., 1987, pp.26-39

10. Fadum, R.E. (1948)-Influence values for Estimating Stresses in Elastic Foundations- Proc. 2nd International Congress SMFGV Vol.3.

11. Felliinius, W. (1927) Erdstatische Berechnungen, Berlin (Revised Edn. 1939).

12. Giroud, J.P., (1968)- Settlement of an embankment resting on Semi-infinite Elastic Soil, Highway research Record No. 223, highway Research Board.

13. Guru Vittal U.K. and Sudhir Mathur (2006), 'Case Studies on Prefabricated Geodrains at Visakhapatnam', Published in the National Seminar on Developments in Geosynthetics and Reinforced Earth in Civil Engineering Practice', organised by Indian Geotechnical Society, Baroda Chapter and Institution of Engineers (Vadodara Local Centre) at Vadodara

14. Guru Vittal U.K., P.S.Prasad, and Sudhir Mathur (2006), 'Geotechnical Properties of Visakhapatnam Marine Clay', published in the Indian Geotechnical Conference – IGC 2006 at Chennai

15. Janbu, N, Bjerrum, L. & Kjaernsli, B. (1956)–Veiledning Ved Losning av Fundameteringsoppgaver-Publ. Norwegian Geotech. Inst. No. 16

16. Kenney, T.C. (1956) - An examination of the Methods of calculating the stability of slopes. M.Sc. Thesis, University of London

17. Mandal. A. K. et al. "Design, Construction and Performance of High Embankment on Soft Clay Deposits" Proceedings of Indian Geotechnical Conference: IGC-2006, Guntur

18. Matsuo Minoru and Kunio Kawamura (1977), 'Diagram for Construction Control of Embankment on Soft Ground', Journal of Soils and Foundations, Published by Japanese Society of Soil Mechanics and Foundation Engineering, Vol 17, No 3, pp 37-52.

19. Meyerhof G.G (1957)-Ultimate Bearing Capacity of Foundation on Slopes. Proc. 4th Int. Conf. on Soil Mechanics and Foundation Engineering.

20. Middlebrooks, T. A. (1936) Foundation Investigation of Fort Peck Dam Closure Section, Proc. 1st Int. Conf. on Soil Mechanics and Foundation Engineering

21. Morgenstern, N.R. & Price, V.E. (1967) - A numerical method for solving the equations of stability of general slip surfaces- Computer Jour. 2 (p. 388-93).

22. Morgenstern, N.R. & Price, V.E. (1965) - The Analysis of the Stability of General Slip Surfaces, Geotechnique 15 (p. 79-93).

23. Newmark, N. M. (1941, 1942) - Influence charts for computation of elastic stresses in Foundations- University of Illinois Eng. Exp. Sta. Bull. No. 338-1942.

24. Niroumand. H. Et al. "Soil Improvement by Reinforced Stone Columns Based on Experimental Work" European journal of Geotechinical Engineering Vol. 16 [2011].

25. Nonveiler, E. (1965) - The stability analysis of slopes with a slip surface of general shape. Proc. 6th Int. Conf. Soil Mech. (Montreal) 2 (p. 522-25).

26. Osterberg, J.O. (1957)- Influence Value for Vertical stresses in a Semi-Infinite Mass due to an Embankment Loading, Proc. 4th Int. Conf. on SMFE, Vol.1

27. Perloff, W.H., Baladi, G.Y. and Harr, M. E. (1967)- Stress Distribution within and under Long Elastic Embankments, Highway Research Record No. 181, Highway Research Board.

28. Rao. P. Jagannatha and Kumar, Satish(1989), “Experimental Studies on Load Bearing Capacity of Different Types of Stone Columns”, Proc. Indian Geotechnical Conference, Vishakhapatnam, 1989.

29. Rao. P. J. et al. (1991) “Improvement of Bearing Capacity of Soft Marine Clay Using Stone Columns” Geo-Coast, September 1991, Yokohama Japan.

30. P.Jagannatha Rao & Rao, K. Srinivasa (2004)" Design and Construction of Reinforced Soil Walls on Clay Subsoils "I GC- 2004 Warangal, p. 406.

31. Rao. P.J. et al. Ground Improvement for Embankments on Deep Soft Clay Deposits Along Highway to ICTT, Vallarpadam Port in Cochin, Proceedings of Indian geotechnical conference: December 15-17, 2011, Kochi (Paper No. G 110).

32. Skempton, A. W. (1954) The pore water pressure coefficients A & B. Geotechnique 4 (p. 143-47).

33. Steinbrenner, W. (1934)- Tafeln zu Setzungs- berechung- Die Stresse- Vol. 1.

34. Tandel Y.K. et al.“Reinforced granular column for deep soil stabilization” International Journal of Civil and Structural Engineering Volume 2, No 3, 2012.

35. Transport Research Board (1975) : NCHRP Synthesis of Highway Practice No. 29- Treatment of Soft Foundations for Highway Embankments.

36. Youd. T. L. and others (2001) “Liquefaction Resistance of Soils: Summary Report From the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils”. Journal of Geotechnical and Geoenvironmental Engineering(ASCE, 2001).

Bibliography

1. Arora K.R. Soil Mechanics and Foundation Engineering. Seventh Edition, Standard Publishers, New Delhi.

2. Bowles Joseph. E. (1997) Foundation Analysis and Design. Tata McGraw-Hill Publishing Company Limited, New Delhi.

3. Gulhati, S.K. and Datta Manoj (2005). Geotechnical Engineering. Tata McGraw-Hill Publishing Company Limited, New Delhi.

4. Hausmann M.R. (1990). Engineering Principles of Ground Modification. International Edition. McGraw-Hill Book Company, Singapore.

5. Leonards, G. (1962)- Foundation Engineering .McGraw Hill Book Company, Inc.
6. Murthy, V.N.S. Principles and Practices of Soil Mechanics and Foundation Engineering
7. Nayak N. V. Foundation Design Manual.
8. Ranjan, G. and Rao, A.S.R. (2004). Basic and Applied Soil Mechanics. Second Edition, New Age International (P) Ltd., New Delhi.
9. Taylor D. W. (1948), Fundamentals of Soil mechanics.
10. Venkatramaiah C. (2006) Geotechnical Engineering

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