IITK-GSDMA GUIDELINES for SEISMIC DESIGN of EARTH DAMS AND EMBANKMENTS

Provisions with Commentary and Explanatory Examples

Indian Institute of Technology Kanpur

Gujarat State Disaster Management Authority

August 2005; Revised May 2007

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- IITK-GSDMA Guidelines for Seismic Design of Liquid Storage Tanks
- IITK-GSDMA Guidelines for Structural Use of Reinforced Masonry
- IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of Buildings

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Prepared by:
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Kanpur

With Funding by:
Gujarat State Disaster Management Authority
Gandhinagar

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Preparation of this document was supported by the Gujarat State Disaster Management Authority (GSDMA), Gandhinagar, through a project at Indian Institute of Technology Kanpur, using World Bank finances. The views and opinions expressed in this document are those of the authors and not necessarily of the GSDMA, the World Bank, or IIT Kanpur.

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FOREWORD

The earthquake of 26 January 2001 in Gujarat was unprecedented not only for the state of Gujarat but for the entire country in terms of the damages and the casualties. As the state came out of the shock, literally and otherwise, the public learnt for the first time that the scale of disaster could have been far lower had the constructions in the region complied with the codes of practice for earthquake prone regions. Naturally, as Gujarat began to rebuild the houses, infrastructure and the lives of the affected people, it gave due priority to the issues of code compliance for new constructions.

Seismic activity prone countries across the world rely on “codes of practice” to mandate that all constructions fulfill at least a minimum level of safety requirements against future earthquakes. As the subject of earthquake engineering has evolved over the years, the codes have continued to grow more sophisticated. It was soon realized in Gujarat that for proper understanding and implementation, the codes must be supported with commentaries and explanatory handbooks. This will help the practicing engineers understand the background of the codal provisions and ensure correct interpretation and implementation. Considering that such commentaries and handbooks were missing for the Indian codes, GSDMA decided to take this up as a priority item and awarded a project to the Indian Institute of Technology Kanpur for the same. The project also included work on codes for wind loads (including cyclones) and fires considering importance of these two hazards. Also, wherever necessary, substantial work was undertaken to develop drafts for revision of codes, and for development of entirely new draft codes. The entire project is described elsewhere in detail.

The Gujarat State Disaster Management Authority Gandhinagar and the Indian Institute of Technology Kanpur are happy to present the IITK-GSDMA Guidelines on Seismic Design of Earth Dams and Embankments to the professional engineering community in the country. It is hoped that the document will be useful to the professional engineers in developing a better understanding of the design methodologies for earthquake-resistant structures, and in improving our codes of practice.

PREFACE

During the earthquake of 26 January 2001, numerous earth dams and embankments in Gujarat suffered extensive damages. Fortunately, due to low water levels in the earth dams, secondary disaster due to breach of dams did not take place. However, the need for incorporating good earthquake engineering practices in the design and construction of earth dams and embankments was clearly reiterated by the earthquake experience.

IS1893-1984 provides some provisions on seismic design of earth dams and embankments. However, these are now obsolete and no longer considered adequate. Considering the rapid pace of infrastructure development in the country in several sectors including road and rail networks and the water resources, an urgent need exists for alternative design methodology. It was therefore decided to include development of the present guidelines within the scope of the project "Review of Building Codes and Preparation of Commentary and Handbooks" assigned by the Gujarat State Disaster Management Authority, Gandhinagar to the Indian Institute of Technology Kanpur in 2003.

The present document provides design guidelines in a format similar to that of the codes, a clause-by-clause commentary, and explanatory examples. Since soil liquefaction has a major influence on the seismic performance earth dams and embankments, an Appendix has been included outlining the procedure for liquefaction potential assessment. Inclusion of a few explanatory examples will facilitate implementation of the guidelines in real projects.

This document was developed by a team consisting of Professor Debasis Roy (Indian Institute of Technology Kharagpur), Professor Umesh Dayal (Consultant, Paul C. Rizzo Associates, Inc., USA; formerly Professor at IIT Kanpur), and Professor Sudhir K Jain (Indian Institute of Technology Kanpur). Some very thoughtful review comments on an earlier version of the document were provided by Professor V S Raju (formerly of Indian Institute of Technology Madras, and Indian Institute of Technology Delhi), Dr. Paul C. Rizzo (Paul C. Rizzo Associates, Inc., USA), Dr. Martin Wieland (Electrowatt-Ekono AG in Zurich, Switzerland and Chairman, Committee on Seismic Aspects of Dam Design, International Commission on Large Dams) and Dr. Peter Byrne (Professor Emeritus, Civil Engineering Department, University of British Columbia, Canada). Also, the document was circulated by the Central Board of Irrigation and Power (CBIP) New Delhi to a large number of organizations and individuals in the country for review and comments, and was also placed on the web site of National Information Centre of Earthquake Engineering (www.nicee.org). Review comments were received from CDO, NWRWS&K, Gandhinagar, Sardar Sarovar Narmada Nigam Limited, Vadodara, Himachal Pradesh State Electricity Board, Surendranagar, Indian Institute of Science Bangalore, Bangalore and Central Soil and Materials Research Station, Ministry of Water Resources, New Delhi which were duly considered in finalizing the present version. Special mention should be made of Sri M. G. Golwala of CDO, NWRWS&K Department, Gandhinagar for his enthusiastic participation in discussions on this document and for making several valuable suggestions.

It is hoped that the Guidelines will be useful for design and construction of earth dams and embankments in the seismic regions. Further, these may contribute to better understanding of aseismic construction aspects by the concerned professionals, and spur research and development in the country in this critical area.

All suggestions and comments are welcome and may be sent to Professor Sudhir K Jain, Department of Civil Engineering, Indian Institute of Technology Kanpur, Kanpur 208 016, skjain@iitk.ac.in

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IITK-GSDMA GUIDELINES
for SEISMIC DESIGN
of EARTH DAMS AND EMBANKMENTS

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PART 1: PROVISIONS AND COMMENTARY

PROVISIONS

1. General:

1.1 Scope:

Provisions of these guidelines are applicable to earth embankments and small to intermediate size earth and rockfill fill dams as classified in Indian code IS 11223-1985.

1.2 Failure Mechanisms:

Possible damaging effects of earthquakes on earth dams and embankments include:

- Slope failure because of inertial loading and/or softening of materials strength or liquefaction.
- Fault displacement under the foundation.
- Crest settlement of dam caused by settlement or by earthquake generated water waves in the reservoir.
- Permanent deformation of foundation soils or dam body.
- Sliding failure of an embankment composed of weak or liquefiable soils.
- Piping and erosion

COMMENTARY

C-1: General

C- 1.1: Scope

See Table C-1 for definition of small intermediate and large dams as per IS 11223-1985. While many of the provisions of this document will be applicable to large earth dams, the design requirements for such dams are, in general, more stringent than those included in this document.

C- 1.2: Failure Mechanisms

Major contributors to earth dam failure are overtopping, piping, and structural failure.

Overtopping:
Overtopping is defined as uncontrolled flow of water over the crest of the dam or embankment. Non-overflow (other than spillway) portions of a dam are not usually designed for erosional effect of flowing water, overtopping may lead to failure of the dam due to excessive erosion or saturation of the downstream slope. Adequate spillway capacity should be provided to prevent such damages.

Piping:
Due to the pervious nature of earth dams, the dam body acts as pathways for water seepage. If such seepage is uncontrolled in terms of volume and velocity, and material used in constructing the dam body are not carefully selected, particles of soil with which the dam body is constructed may be taken into suspension by seepage water and carried away. In order to prevent such an occurrence, (a) the seepage gradient is kept well below the critical gradient, (b) the particle size distribution of the filter material used in constructing the dam body is carefully chosen to meet the filter criteria, and (c) hydraulic stability of dam core and potentially dispersive fine-grained soils within foundation is demonstrated with pin-hole tests. Additionally, there should be appropriate seepage control measures at the contact between dam and foundation such as: (a) drainage blanket for soil foundation, (b) removal of rock mass affected by excessive cracking or jointing at dam-foundation interface before dam construction, and (c) ensuring absence of slopes steeper than 10 vertical to 1 horizontal at dam-foundation interface.

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Structural Failure:

Structural failure includes failure of upstream or downstream slopes of the dam, as well as cracking, deformation and settlement of the dam body that may lead to overtopping or a piping failure. Earthquake loading may trigger any one of the above failure modes or their combinations.

Defensive Design Measures:

Defensive design features should be incorporated in the foundation and embankment design of new dams regardless of the method of seismic analysis. These features include (USACE 2004):

1. Additional dam height to accommodate the loss of crest elevation due to deformation, slumping, and fault displacement.
2. Crest details that will minimize erosion in the event of overtopping.
3. Wider transition and filter sections as a defense against cracking.
4. Use of rounded or subrounded gravel and sand as filter material.
5. Adequate permeability of the filter layers.
6. Near vertical chimney drain in the center portion of the embankment.
7. Zoning of the embankment to minimize saturation of materials.
8. Wide impervious cores of plastic (non-brittle) cohesive fine-grained soils to accommodate deformation.
9. Well-graded core and uniformly graded filter materials to ensure self healing in the event cracking should occur.
10. Stabilization of reservoir rim slopes to provide safety against large slides into the reservoir.
11. Ground improvement or removal and replacement of foundation material to mitigate liquefaction potential
12. Stabilization of slope adjacent to operating facilities to prevent blockage from slide associated with the earthquake.
13. Flaring embankment sections at the abutment contacts.
14. Installation of suitable features to prevent piping through earthquake generated seepage cracks.

PROVISIONS

1.3 Earthquake Load:

Small and intermediate dams or embankments whose failure entails negligible risk may be designed for earthquake ground accelerations as specified in Clause 6.1 of this document. However, where site-specific seismic study has been carried out, the peak horizontal ground acceleration estimated therein may be used. Site-specific seismic assessment should be performed for all projects located in active fault zones.

1.4 Investigation:

The seismic evaluation and design of dams and embankments involves the participation of geologists, seismologists, and geotechnical engineers. The entire effort can be grouped into four main areas: field investigations, site characterization, analysis, and evaluation. The investigations and site characterization should be thoroughly evaluated to establish the nature, extent, and in-situ geotechnical properties of the materials in foundation, embankment, or dam being investigated.

COMMENTARY

C-1.3: Earthquake Loads

For the design of intermediate dams or embankments and dams and embankments whose failure entails unacceptable level of risk, a two-tier seismic design approach is usually adopted so that (a) the dam or embankment remains operational following an earthquake that has a reasonable probability of occurrence during the service life of the facility with distress of a minor nature, and (b) the dam or embankment does not collapse following an earthquake that has a small probability or occurrence during the life of the facility. The design earthquake for condition (a) usually corresponds to a probability of exceedance of 50% over the operational life of the facility and that for condition (b) is often specified as that having a probability of exceedance of 10% over the operational life of the facility.

Operational life of a dam or an embankment is usually 50 to 100 years. However, a longer operational life should be assumed in situations such as (a) a water-retaining dam that is not appropriately decommissioned at the end of operational life or where there is an unacceptable downstream risk in the event of a dam break, and (b) a tailings dam retaining radioactive or other wastes that may pollute groundwater or jeopardize downstream public health and safety in the event of dam break.

C- 1.4: Investigation

An assessment of site geologic and geotechnical conditions is one of the important aspects of the dam safety evaluation. Evaluation of safety of new and existing dams requires, among other things, that its foundation has been adequately examined, explored, and investigated. The investigation should include the following:

- **Seismological Investigations:** Studies should be made of the past occurrence of earthquakes in the general region of the site, and on this basis estimates are made of the probability of future earthquakes. In order for this approach to be valid, a sufficiently long seismic history must be available.
- **Geotechnical Investigations:** Investigations are made of geological formations, soil deposits and rock in and around the construction site for assessing their behavior during earthquake shaking, and how they might affect the ability of a structure to resist earthquake including evaluation of liquefaction potential, if appropriate. The investigation should focus on topics including:

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1. Topographic conditions.
2. Description of geology.
3. Composition and structure of foundation soils, soils from borrow area and bedrock.
4. Principal engineering properties of the rocks and soils including grain characteristics, plasticity, compaction characteristics, shear strength, dispersivity and hydraulic properties.
5. Geotechnical investigation would typically include drilling and sampling, in-situ testing (piezocone penetration test, Standard Penetration Test or Field Vane Shear Test, seismic velocity profiling) as appropriate.

2. Location:

2.1 Preference:

Earth dams and embankments should be ideally located away from any potentially active fault or an area underlain by liquefiable or sensitive soils or abutments prone to static or seismic instability.

C-2: Location

Earthquake damage to embankment can be due to actual ground rupture beneath the embankment and/or seismic shaking. Failure of a dam due to ground rupture is possible only when the dam is built over an active fault zone or across reactivated or newly activated landslide zone. The location of the dam over the fault zone should be reviewed at the time of site selection and appropriate measures should be taken in the design and construction of a dam over a fault. By far the more common problem in a dam design is to ensure that the dam will be stable under anticipated levels of seismic shaking.

2.2 Design Measures:

If unavoidable, an earth dam or an embankment may be constructed over a fault or at a site underlain by potentially liquefiable or sensitive soils or between potentially unstable abutments only if (a) the dam is designed for the displacements and other dynamic effects of an earthquake that is likely to occur, and (b) potential failure is unlikely to lead to any loss of life and the risk associated with such a selection of site is acceptable.

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3. Freeboard:

It is recommended to provide a freeboard of at least 2% to 3% of dam height, but not less than 2m if there is a potential for occurrence of landslides near the dam abutment within the slopes of the reservoir margins or 1m if there is a negligible landslide potential near the dam abutments.

COMMENTARY

C-3: Freeboard:

The freeboard of all embankment dams should be based on most extreme conditions expected for which the dam is designed. The maximum reservoir elevation is determined for the design flood, wind speed, fetch and expected wave run-up conditions. In general, overtopping of the dam is not acceptable.

Ample freeboard should be provided to avoid the possibility of overtopping by (a) earthquake-generated water waves, (b) settlement and permanent deformation of crest due to liquefaction which may cause densification or loss of stiffness of the materials or fault rupture. In addition, it may be prudent to use riprap or other crest details that will resist erosion by a succession of overtopping waves.

PROVISIONS

4. Liquefaction:

4.1 Foundation:

Liquefaction of saturated sandy soils in a dam foundation may be caused due to pore water pressure build-up during earthquake shaking. Liquefaction leads to loss of strength and increase in settlement.

Appropriate methods, such as the one described in Annex A of this document, should be used to evaluate the liquefaction potential. If the material is found to be liquefaction susceptible, the deformation of the embankment should be estimated for the design ground motion as per Clause 7.

COMMENTARY

C- 4: Liquefaction

During an earthquake, the pore pressure within saturated soil often increases if the deposit is loose, sensitive or young and the earthquake is of moderate to large magnitude and intensity. The increase could be so large that the effective stress may approach zero. As a result, frictional soils may loose a substantial fraction of shear strength leaving the soil to behave like a viscous liquid. Such a phenomenon is conventionally referred to as "liquefaction." Although many deposits never attain the state of zero effective stress, they may deform substantially during earthquakes leading to the development of liquefaction-like failures. Liquefaction is therefore often functionally assumed to be the state in which a double amplitude shear strain of 5% develops. This definition will be adhered to in this commentary.

The pore pressure generated in the soil due to cyclic stresses depends mainly on the state of packing (*i.e.*, whether the soil is loose or dense), geologic age of the deposit for non-cohesive soils, and on plasticity and sensitivity for cohesive soils.

Saturated cohesive soils usually do not attain the state of zero effective stress during earthquakes. Nevertheless, many such deposits deform appreciably during earthquakes. As a result liquefaction-like features develop within such deposits. Among cohesive deposits, sensitive soils of moderate to low plasticity are especially vulnerable to liquefaction.

Among cohesionless soils, dense sands or well-compacted deposits that are of early to mid Pleistocene age (or older) are not susceptible to liquefaction. Holocene to late Pleistocene, loose, saturated sands with relative densities of up to 30-50% are more susceptible to liquefaction.

Dam and embankment failure under seismic loading may be caused by liquefaction of embankment and/or foundation materials. Dam and embankment constructed using loose, uncompacted material or those founded on liquefiable foundations are prone to catastrophic flow failure.

If an unacceptably large deformation of an earth dam or embankment is estimated because of liquefaction, considerations may be given for liquefaction potential mitigation measures such as:

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1. In situ ground improvement (*e.g.*, deep dynamic compaction, vibro-compaction, vibro-replacement, jet grouting, deep soil mixing, sand drains/wick drains, blast densification, sand and gravel piles);
2. Removal and replacement of liquefaction susceptible soil;
3. Surcharging, dewatering and reinforcement.

4.2 Compaction:

The material of all new embankments and dams should be compacted to a density that will cause them to dilate rather than liquefy during earthquake shaking. It is recommended that the compacted density of material should exceed 95% of Standard Proctor Maximum Dry Density (SPMDD) for rail or road embankments and 98% SPMDD for dam embankments. Cohesive materials used within the dam or embankment body should be placed with moisture content 2% to 4% higher than the optimum moisture content. For cohesionless soils, a relative density of 80% may be used as an alternative indicator of the minimum compaction requirement.

C- 4.2: Compaction

Rigorous protocol should be established for ensuring the compaction requirements during construction. These measures are essential in order to ensure ductile material behavior.

PROVISIONS

5. Seismic Slope Stability Assessment:

When the maximum cross section of the dam or embankment satisfies the limit equilibrium stability requirements specified in *Clause 6*, no further stability analysis is needed. Otherwise, the designer may proceed to the procedure described in *Clause 7*. If the stability requirements are still not satisfied, the cross section of the dam or embankment and/or foundation soils will require improvement.

COMMENTARY

C-5: Seismic Slope Stability Assessment

Seismic slope stability is influenced by the following two factors:

- Cyclic stresses induced by earthquake shaking, and
- The cyclic stress-stain behavior of the materials within the body of the dam or embankment and that of foundation soils.

Potential instability of an earth dam or an embankment during an earthquake may be due to the inertial effects or due to cyclic softening of soils.

Techniques ranging from very approximate to very elaborate are available for seismic stability analysis of dam and embankment. In the order of increasing complexity, these methods include:

- Equivalent-static Stability Analysis
- Sliding Block Method
- Dynamic Analysis (Simplified or Rigorous)

Seismic slope stability analysis often begins in a staged approach, which usually involves starting with a simpler analysis (equivalent-static) and progressing to more rigorous analyses (the sliding-block method and the simplified dynamic analysis) if appropriate.

An earth dam or an embankment is usually considered safe if it is found safe by equivalent-static or the sliding-block method. In such cases a more sophisticated analysis is not usually undertaken. On the other hand, if the sliding-block analysis indicates a potential for instabillity, then either a simplified dynamic analysis or a rigorous dynamic analysis could be undertaken to assess stability of the earth dam or embankment. The designer may also skip the equivalent-static or the sliding-block method and proceed directly to simplified dynamic analysis provided that high quality material- and site-specific input parameters are available for undertaking the dynamic analysis.

It should be noted that adoption of a more elaborate analytical procedure in the design should also require detailed and appropriate characterization of pre-failure undrained deformation behavior of the soils within the embankment and foundation as well as a suite of earthquake time histories which the earth structure may reasonably be expected to encounter during its design life.

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6. Equivalent–Static Slope Stability Assessment:

In equivalent-static analysis, the dynamic (random) earthquake shaking is replaced by a single constant unidirectional equivalent-static acceleration. Slope stability analysis is similar to that for static conditions except for the application of horizontal and vertical inertia forces over every portion of the potentially unstable soil mass.

This approach is based on seismic coefficients. Upon multiplication of the weight of the potential sliding mass with these coefficients an estimate of earthquake-related inertial forces are obtained. These forces are considered in addition to other (conventional) static forces in seismic slope stability assessment.

Usually the horizontal seismic coefficient used in this method is equal to the free-field peak ground acceleration corresponding to design level of earthquake shaking.

A limit equilibrium factor of safety of 1.0 is usually considered acceptable in the equivalent-static seismic slope stability assessment.

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C-6. Equivalent–Static Slope Stability Assessment

For many years the standard method of evaluating the safety of embankment dams against sliding during earthquakes has been the equivalent-static method of analysis. Equivalent-static method of analysis involves the computation of the minimum limit equilibrium factor of safety by including in the analysis static horizontal and vertical forces that represent the inertial effects of earthquake shaking. These equivalent-static forces are usually expressed as a product of horizontal or vertical seismic coefficients and the weight of the potential sliding mass. The horizontal equivalent-static force decreases the factor of safety by reducing the resisting force and increasing the driving force. The vertical equivalent-static force typically has less influence on the factor of safety. As a result, it is often ignored.

Although the equivalent-static approach to stability analysis is simple and straight forward producing an index of stability (factor of safety) which engineers are used to appreciating, it suffers from many limitations as it can not really simulate the complex dynamic effects of earthquake shaking through a constant unidirectional equivalent-static acceleration. These limitations are well recognized (Terzaghi 1950, Seed 1966 and Marcuson 1981). Of particular importance is the fact that in case of soils that build up large pore water pressures or have a degradation in strength of more than say 15% due to the earthquake shaking the analysis can be unreliable. As shown by Seed (1979) a number of dams such as the Upper and Lower San Fernando Dams, Sheffield Dam have in fact failed due to earthquakes although the calculated factors of safety were well above 1.0.

In the last couple of decades methods for the estimation of earthquake-related permanent slope deformations (as discussed in Clause 7) are finding increasing application. These methods are particularly suited to the case of earth dams where the magnitude of induced deformations is not only a measure of the stability of the embankment but also a measure of the effectiveness of the filter protection system.

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6.1 Equivalent-Static Forces:

In the absence of site-specific estimates of design peak ground, the design seismic inertia forces for equivalent-static slope stability assessment shall be taken as: $F_H = \frac{1}{3} \times Z \times I \times S \times W$, where F_H is the horizontal inertial force, Z is the Zone Factor given in IS:1893 - Part 1 (2002), I is the importance factor as per Table 1, S is an empirical coefficient to account for the amplification of ground motion between bedrock and the elevation of the toe of the dam or embankment (Table 2), and W is the weight of the sliding mass.

If the estimate of design peak ground horizontal acceleration (PHGA) at the elevation of the toe of the dam is available, the design seismic inertia forces for equivalent-static slope stability assessment shall be taken as: $F_H = \frac{1}{3} \times a_{\max} \times W$, where $a_{\max}$ is the design PHGA at the elevation of the toe of the dam.

The vertical inertial force during an earthquake may be neglected in the design.

6.2 Application of Forces:

Earthquake forces shall not be normally included in stability analyses for the construction stage or for the reservoir empty condition. However, where the construction or the operating schedule requires the reservoir empty condition to exist for prolonged periods, earthquake forces should be included.

6.3 Soil Properties:

Since earthquake loading is rapid, stability for an earth dam or an embankment is usually considered under undrained condition.

Soil properties used in analysis should reflect softening because of pore water pressure generation and strain development and cyclic strength degradation.

COMMENTARY

C-6.1: Equivalent Static Forces

The equivalent-static forces should approximate earthquake-related inertia of the potential failure mass reasonably. These forces therefore relate to (a) the peak ground acceleration for the design earthquake, (b) the amplification of ground motion through foundation soils.

C- 6.3 Soil Properties

Consolidated undrained strength parameters should ideally be obtained from simple shear testing of undisturbed samples carried out in the laboratory. Alternatively, the following empirical relationships (Olson and Stark, 2003) based on Standard Penetration Test (SPT) or Cone Penetration Test (CPT) may be used with due caution:

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$$s_u / \sigma'_v = 0.205 + 0.0143 q_{c1}$$ or

$$s_u / \sigma'_v = 0.205 + 0.0075 (N_1)_{60}$$

where q_{c1} and $(N_1)_{60}$ are stress-normalized values of the cone tip resistance (MPa) and SPT blow count, respectively. These relationships are valid for $q_{c1} \le 6.5$ MPa and $(N_1)_{60} \le 12$.

For larger values of penetration resistance, drained shear strength friction angle, ϕ', may be used in stability assessment with cohesion intercept, c', set to zero. The following correlations may be used for estimating ϕ' (Mayne, 1998; Kulhawy and Mayne, 1990):

$$\phi' = 20° + \sqrt{15.4 \times (N_1)_{60}}$$ or

$$\phi' = 17.6° + \log q_{c1}$$

If liquefaction is triggered during an earthquake, post-liquefaction shear strength of cohesionless soils are estimated from (Olson and Stark, 2003):

$$s_u / \sigma'_v = 0.03 + 0.0143 q_{c1}$$ or

$$s_u / \sigma'_v = 0.03 + 0.0075 (N_1)_{60}$$

The undrained shear strength for cohesive soils is estimated using field vane shear tests or other appropriate methods. If the dam foundation is underlain by sensitive soils, in undrained stability assessment residual undrained shear strength should be used. Compacted impervious and semi-pervious soils within the dam body are susceptible to strain softening. To account for such a behavior, a material shear strength of 80% of the corresponding peak value is used in stability analysis.

7. Sliding Block Method:

7.1 General:

This method involves evaluation of permanent deformation during an earthquake and comparing it with what is the acceptable deformation. This is usually carried out by the Newmark's sliding block analysis wherein the potential failure mass is treated as a rigid body on a rigid base with the contact in between as rigid plastic. The acceleration time history of the rigid body is assumed to correspond to the average acceleration time history of the failure mass. Deformations accumulate when the inertial load due to

C-7.1 General

Newmark (1965) introduced the concept that the effects of earthquakes on embankment stability could be assessed in terms of the deformations they produce rather than the minimum factor of safety. Here, the potential sliding mass is approximated as a rigid body resting on a rigid sloping base and the contact between the potential sliding mass and the underlying slope is assumed to be rigid-plastic.

The potential sliding mass can undergo downslope movement, if the downslope ground acceleration exceeds the threshold required to overcome cohesive-frictional resistance at the contact between the sliding mass and the rigid base.

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earthquake acceleration exceeds the plastic resistance at the interface between the sliding block and the underlying stable body. The acceleration level that triggers instability is referred to as "yield acceleration." Potential sliding mass is considered to mobilize down slope whenever earthquake acceleration in that direction exceeds yield acceleration. The movement continues after the earthquake acceleration becomes smaller than the yield acceleration until soil resistance brings the potential sliding block back to its stationary condition. The deformation is estimated by double integration of the acceleration time history equal to the difference between earthquake acceleration and the yield acceleration. Figure 1 illustrates the concept of deformation calculations.

Several alternative empirical approaches for estimation of permanent deformation are available incorporating the Newmark framework. One of these is described in Clause 7.3.

In these methods, the horizontal seismic coefficient for which the equivalent-static limit equilibrium factor of safety becomes unity is taken as an estimate of the yield acceleration.

In the absence of site-specific estimates of design peak ground acceleration, the following estimate shall be used:

$$a_{max} = Z \times I \times S$$

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Acceleration pulses in excess of this threshold are appropriately integrated against time to estimate the total downslope displacement of the potential sliding mass.

One of the limitations of the sliding block model is the assumption that the potential failure mass and the embankment are rigid. Although this assumption is reasonable for embankments composed of very stiff, hard or dense soils or slopes subject to low frequency motion, the assumption is not valid for soft or loose soils. In the latter case, lateral displacements throughout the potential failure mass may be out of phase, with inertial forces at different points in the potential failure mass acting in opposite directions. This effectively means that the resultant inertial force and the resulting permanent displacements calculated with the rigid sliding block method may be overestimated. This effect was studied by Chopra (1966) who employed dynamic stress-deformation finite element analyses to produce the time varying horizontal resultant force acting on the potential failure surface, by integrating the horizontal components of dynamic stresses on the potential failure surface. The average acceleration of the potential failure mass is then produced by dividing the horizontal resultant force by the potential failure mass.

A second limitation of this approach is that it assumes rigid-plastic material behavior. As a result, the deformation would be under-estimated for strain softening materials and over-estimated for strain hardening materials. For strain softening materials, the problem is handled by using residual shear strength and for strain hardening material strain dependent material property in stability assessment.

The sliding-block method is a relatively quick and inexpensive form of analysis, in which the potential sliding mass is treated as a rigid-plastic block on a rigid base subjected to an earthquake acceleration time history.

Sliding block deformation estimates are strictly applicable only to dams not subjected to liquefaction (stability) failure, although conservative estimates of deformation can be obtained using post-liquefaction shear strengths for liquefiable materials within dam body or foundation in the analysis.

PROVISIONS

7.2 Acceptable Deformation:

The deformation calculated along the failure plane by these methods should not generally exceed 1m. Larger deformation may be acceptable depending on available freeboard, and consequences of deformation.

7.3 Hynes-Griffin and Franklin Method:

Step 1: Estimation of Yield Acceleration: The yield acceleration is defined as the average acceleration producing a horizontal inertia force on a potential sliding mass so as to produce a factor of safety of unity and thus causes it to experience permanent displacement. The yield acceleration is essentially the horizontal seismic coefficient, which gives the limit equilibrium factor of safety of unity when used in a conventional slope stability analysis.

Step 2: Estimation of Permanent Deformation: Knowing the yield acceleration and the PHGA at the elevation of the toe of the embankment or dam, the permanent displacement of the embankment can be calculated using the upper-bound relationship of Fig 2 suggested by Hynes-Griffin and Franklin (1984).

Step 3: Adequacy of Design: The calculated permanent seismic deformation is checked, whether it is within acceptable limits of deformation or not. The magnitude of acceptable deformation may be taken as per section 7.2 or may be established by the design engineer on a case-by-case basis.

COMMENTARY

C-7.2: Acceptable Deformation

Experience indicates that permanent displacements are limited to less than 1m if the ratio of yield acceleration to peak acceleration (a_{max}) is at least 0.5

PROVISIONS

8. Dynamic Analysis

Dynamic analysis is recommended for important dams and embankments, failure of which may lead to high levels of risk, and dams located over active fault zone. Small to intermediate size dams found to be unsafe by simplified analyses described in Clauses 6 or 7 may also be subjected to dynamic analysis if adequate data and expertise are available for undertaking such an exercise.

COMMENTARY

C- 8: Dynamic Analysis

Dynamic analysis essentially involves estimation of the deformation behavior of an earth dam or an embankment using the finite element or finite difference method. A complete and detailed dynamic analysis is a major undertaking that requires extensive database and specialized skills.

The results of such analyses are sensitive to the input seismologic parameters and engineering properties. As a result, a pre-requisite for using these procedures is a thorough seismotectonic assessment and a detailed site and material characterization.

Dynamic analysis employing a non-linear stress-strain relationship provides a rational framework for estimation of deformation of an earth dam or an embankment. The biggest difficulty in employing these models is to obtain soil stress-strain models that are representative of the soil in-situ behavior. This approach requires an accurate characterization of the stress-strain behavior of the materials within the body of the earth dam or embankment and foundation. Dynamic analysis of earth dams and embankments also require a suitable earthquake time histories representing design earthquakes.

A wide range of approaches have been utilized to model deformation behavior of earth dams and embankments. They include, for instance, the two-dimensional, effective-stress model developed by Finn and his co-workers (Finn et al., 1991), the approximate two-dimensional decoupled approach developed by Beaty (2003), the two-dimensional elastic anisotropic plastic effective stress approach described by Byrne et al. (2000).

Development of such numerical models is usually expensive. Consequently, dynamic analysis is carried out only for major and critical earth dams and embankments.

Seed (1979) and Finn et al. (1986) summarize procedures for dynamic analyses of dams. These procedures usually involve the following steps:

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Step 1: Determine pre-earthquake static stress using a static numerical model of the embankment for initial effective normal stress and shear stress along the potential failure surface. It is a common practice to use a two dimensional numerical model of the maximum dam section in the analysis. The numerical models are usually based on finite element or finite difference approximations

Step 2: Evaluate the dynamic soil behaviour from in-situ and cyclic laboratory tests for input soil properties required in the dynamic analyses.

Step 3: For the numerical model developed in Step 1, determine the dynamic response of the dam or embankment and foundation using a basket of plausible base rock motions. The base rock motions should include appropriate accelerograms representing earthquakes of magnitude and peak acceleration similar to those of the design earthquake from earthquakes recorded in a similar geologic environment. The response of the embankment is determined by dynamic finite element or finite difference modeling, using either equivalent linear or nonlinear procedures.

Step 4: The stress-strain models used in the dynamic analysis should reasonably represent the following aspects of material behavior: (a) material non-linearity, (b) stress and strain dependence, (c) stress-path dependence, (d) inherent anisotropy, and (e) strain rate dependence. Calibration of the stress-strain model should ideally be based on testing of undisturbed samples.

Step 5: Evaluate embankment deformations on the basis of strain potential for the individual elements, which corresponds to the strain that would be experienced if the element were not constrained by surrounding soil.

Step 6: Calculate total embankment deformation on the basis of gravity loads and softened material properties to determine whether they are within the acceptable limits.

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9. Hydrodynamic Effect

To include the earthquake-related hydrodynamic effects in stability analysis of water-retaining dams, the hydrodynamic pressure, Δp_{hd}, at depth *y* below reservoir water level may be estimated from the following (Zangar 1952):

$$\Delta p_{hd} = C \times \left(a_{\max} / g \right) \times \gamma_w \times h$$

where *C* is hydrodynamic pressure coefficient obtained from Figure 3, γ_w is the unit weight of water, and h is the height of water surface above the base of the dam.

The hydrodynamic pressure may be directed inward or outward of the upstream dam face; the outward direction being critical for stability should be considered.

COMMENTARY

C 9.0 Hydrodynamic Pressure

The relationship developed by Zangar (1952) assumes that the dam is relatively rigid and water is incompressible. Limited experimental evidence that are available in the literature (Memos *et al.* 2001) indicates that these assumptions are reasonable for earth dams.

While hydrodynamic effects could be significant for near vertical dam-water interface, for earth dams and embankments its influence is usually limited.

PROVISIONS **COMMENTARY**

10. Instrumentation, Inspection and Monitoring:

10.1 Instrumentation

The predictive methods for assessing performance and integrity of dam have limitations because of (a) uncertainty in subsurface conditions that can reasonably be assessed from subsurface investigation, (b) uncertainty in assessing infrequent loads such as that due to an earthquake and (c) variations in materials of construction and compaction. To minimize or manage loss that may result from failure of a well-engineered earth dam or an embankment, dam design should include well-planned instrumentation and monitoring schemes. In addition, the instrumentation system provides an opportunity to observe the behavior of dam during and after reservoir filling and provides warning if something is not right.

Instruments should be installed within the body of all dams and embankments, where failure of the dam is likely to jeopardize life or pose serious financial or environmental risk. An appropriate level of redundancy in the instrumentation system should be provided to account for possible malfunction of some of the instruments. Precautions should be taken to ensure that the instruments are not damaged during routine operations. A list of typical instruments is presented in Table 3 for monitoring of dam safety.

The instruments used for construction monitoring may not be accessible after the completion of the dam or embankment and filling of the reservoir or impoundment. These instruments would not therefore be useful to monitor the dam or embankment over the long term.

10.2 Inspection

An embankment or a dam should be inspected once every year and/or after the occurrence of an earthquake of Magnitude 5 or above with an estimated free-field PHGA of 0.1g or more at the dam site. The inspection team should include a qualified geotechnical engineer and a surveyor with adequate experience in dam safety monitoring.

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10.3 Visual Inspection

The inspection team should look for signs of distress including appearance of cracks near dam crest or in other exposed surfaces, uneven settlement at the dam crest, undulations on the downstream slope face of the dam or in the vicinity of the toe of the dam, appearance of excessive or unusual seepage from the downstream slope.

10.4 Monitoring

Monitoring of instrumentation and surveying monuments should include the following:

- Inspection of instrument stations and instrument installation to assess their serviceability, *i.e.*, whether the instruments are in working order and in appropriate calibration.
- Monitoring of survey monuments and inclinometer casing installations for measurement of permanent deformations.
- Monitoring of continuous records of seepage quantities measured at flow measuring devices, *e.g.*, flume, to check whether there is any unusual fluctuation of seepage rate since previous monitoring.
- Examination of piezometer data since previous monitoring.

All instruments should be read on prescribed schedule and the data should be carefully evaluated by a qualified engineer.

10.5 Recording, Reporting and Communication

A record of all inspections, routine or otherwise, should be maintained in prescribed format. Reports from prior inspection and monitoring activities should be available to the inspection team at site during field work. Any unusual observation should be communicated to the dam authority and to the regulatory authorities by the inspection team when those observations become available.

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Table 1: Importance Factor, *I*

Type of Dam or Embankment	***I***
Ordinary embankments where failure is not critical	1.0
Important embankments where failure could cause disruption of vital services, major highways, and trunk railway routes	1.5
Small to Intermediate size Dams	2.0

Table 2: Site Amplification Factor, *S*

Stratigraphy	*S*			
	Zone II	*Zone III*	*Zone IV*	*Zone V*
Soil Type S1	1.0	1.0	1.0	1.0
Soil Type S2	2.0	1.5	1.2	1.0

Notes:

1. *Soil Type S1*: Hard rock, Soft rock, Hard soil

2. *Soil Type S2:*
   - Where the average $(N_1)_{60}$ value over a depth equal to embankment height is less than or equal to 15 in case of cohesionless soils.
   - Where the average s_u value over a depth equal to embankment height is less than or equal to 25 kPa in case of cohesive soils
   - When the soil strata contains both cohesive and non-cohesive soils over depth d_1 and d_2, respectively, within the depth equal to embankment height,

$$\frac{d_1}{d_1+d_2}\frac{(s_u)^{avg}}{25}+\frac{d_2}{d_1+d_2}\frac{(N_1)_{60}^{avg}}{15}\leq 1.0$$

   where $(s_u)^{avg}$ is the average undrained shear strength and $(N_1)_{60}^{avg}$ is the average $(N_1)_{60}$

3. To obtain average $(N_1)_{60}$ or average s_u, the following procedure shall be used. Soil Profiles shall subdivided into layers, each numbered from 1 to n with thickness of $d_1, \ldots, d_n$, respectively. Here, n is the total number of layers in a depth equal to height of the dam or embankment.

$$(N_1)_{60}^{avg}=\frac{\sum_{i=1}^{n}d_i}{\sum_{i=1}^{n}\frac{d_i}{(N_1)_{60i}}}$$

$$(s_u)^{avg}=\frac{\sum_{i=1}^{n}d_i}{\sum_{i=1}^{n}\frac{d_i}{(s_u)_i}}$$

Table 3. Typical Instruments for Dam Safety Monitoring

Instruments	Location	Purpose	Remarks
Survey monuments	Along cross sections starting from crest to downstream toe and near the abutments. One line of instruments on deepest section of the dam.	⇳ ⇔ ⬀ ⊠	All dams
Subsurface settlement point.	Same as above.	⇳ □	Should be installed in tailings dams or where foundation soils are compressible
Inclinometers casings: Vertical	Same as above.	⇳ ⇔ ( )	Should be installed in tailings dams or where foundation soils are highly compressible
Piezometeres: Stand-pipe, vibrating wire, resistive, pneumatic	On downstream slope: One line of instruments on maximum section.	▼	All dams
Digital Accelerographs, Peak-reading accelerometers	Crest, abutment and downstream of toe (at a distance of thrice the dam height from the toe) along deepest dam section.		Three or more digital accelerographs and one peak-reading accelerometer for dams in Zones III, IV and V.
Automated Seepage Weir	Downstream of toe at appropriate location.	≋	All dams
Other instruments: extensometers	For dams with high damage potential and dams that exhibit unusual behavior	⇳ ⇔ ⬀ □	

Notes. 1. Italics have been used to denote instruments that can be automated.

2. ⇳, ⇔, ⬀ and ( ) denote vertical, horizontal, inclined and rotational deformation measurements, respectively. ⊠ and □ indicate surface and subsurface settlement measurements, respectively. ▼ denote pore water pressure measurement, denotes digital accelerometers and ≋ denotes automated seepage measurement.

Table C1: Size Classification of Dams (IS 1123-1985)

(Size classification is greater of that indicated by either of the two criteria given in this Table)

Class	Gross Storage (million cubic meter)	Hydraulic Head (meter)
Small	0.5 to 17.0	7.5 to 12.0
Intermediate	10.0 to 60.0	12.0 to 30.0
Large	> 60.0	> 30.0

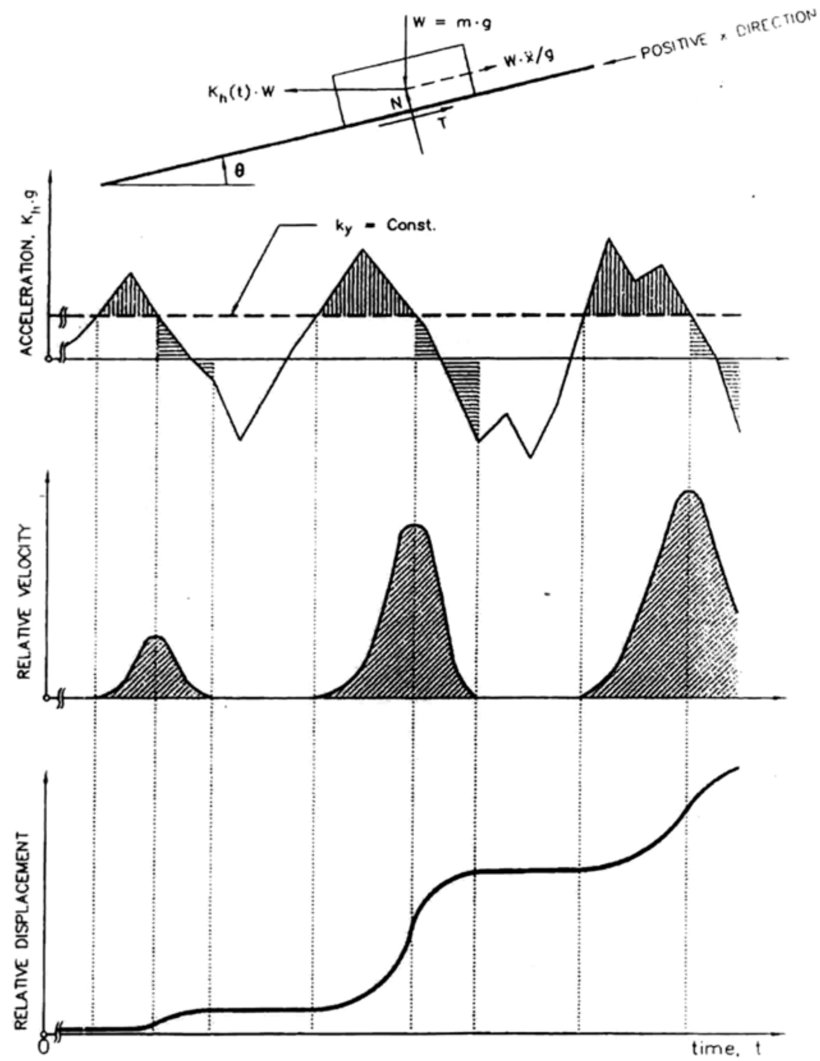

Figure 1: Basic Concept of Newmark's Sliding Block Model

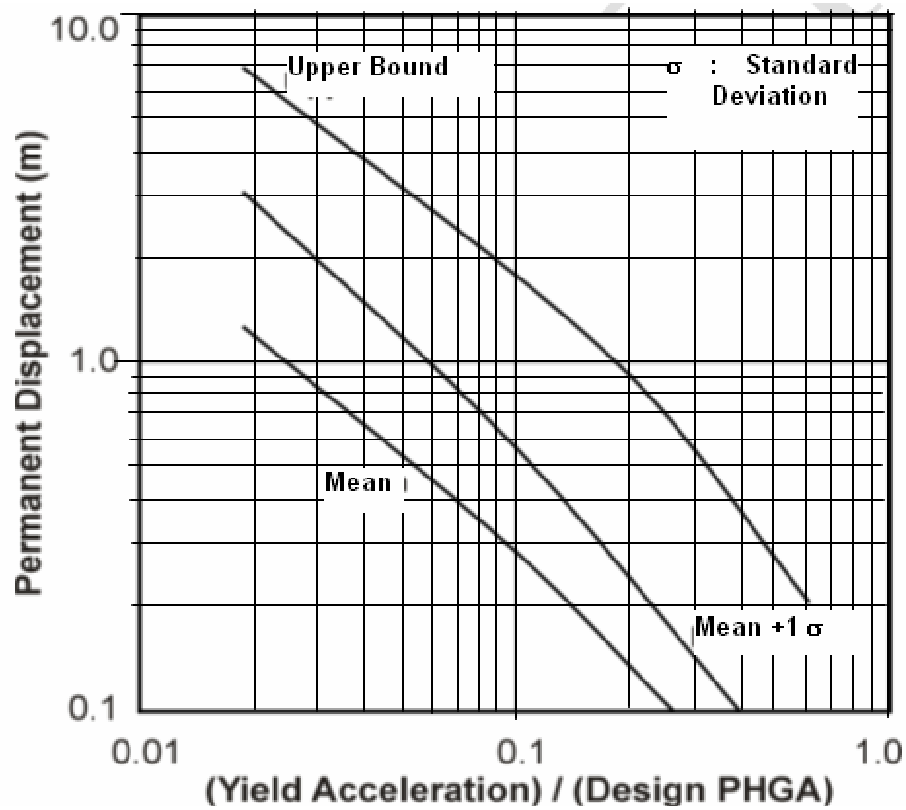

Figure 2: Relationship between Yield Acceleration and Permanent Deformation (Hynes-Griffin and Franklin, 1984)

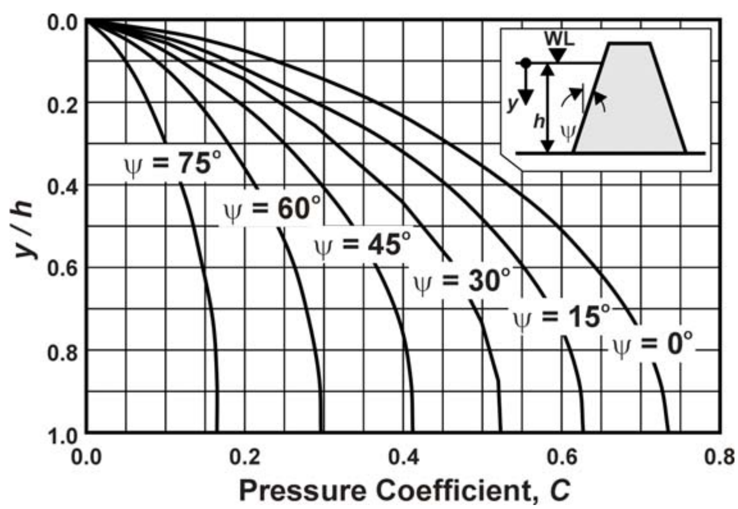

Figure 3: Hydrodynamic Pressure Coefficient (Zangar, 1952)

Annex A

Simplified Procedure for Evaluation of Liquefaction Potential

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1. Cohesionless Soils

Due to the difficulties in obtaining and laboratory testing of undisturbed representative samples from most potentially liquefiable sites, in-situ testing is often relied upon for assessing the liquefaction potential of cohesionless soils. Liquefaction potential assessment procedures involving both the SPT and CPT are widely used in practice. The most common procedure used in engineering practice for the assessment of liquefaction potential of sands and silts is the *Simplified Procedure*[1]. The procedure may be used with either SPT blow count, CPT tip resistance or shear wave velocity measured within the deposit as discussed below:

Step 1: The subsurface data used to assess liquefaction susceptibility should include the location of the water table, either SPT blow count (*N*), or tip resistance of a standard CPT cone (q_c) or the shear wave velocity, mean grain size (D_{50}), unit weight, and fines content of the soil (percent by weight passing the IS Standard Sieve No. 75 μ).

Step 2: Evaluate the total vertical stress (σ_v) and effective vertical stress (σ'_v) for all potentially liquefiable layers within the deposit.

Step 3: The following equation can be used to evaluate the *stress reduction factor* r_d:

$$r_d = 1 - 0.00765z \quad for\ z \le 9.15\ m\ and$$
$$r_d = 1.174 - 0.0267z \quad for\ 9.15 < z \le 23\ m$$

where *z* is the depth below the ground surface in meters.

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C-A1: Simplified Procedure for Cohesionless Soils

For estimating the effective and total stresses for submerged soils (Step 2 of Clause A1), consider the water table to be at the surface of the soil.

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Step 4: Calculate the *critical stress ratio* induced by the design earthquake, CSR, as;

$$CSR = 0.65(a_{max} / g) r_d (\sigma_v / \sigma'_v)$$

where σ_v and σ'_v are the total and effective vertical stresses, respectively, at depth *z*, $a_{\max}$ is the peak horizontal ground acceleration (PHGA), and *g* is the acceleration due to gravity. In the absence of site-specific estimates of $a_{\max}$, the PHGA may be estimated by $a_{\max} = ZIS$, where Z is the zone factor obtained from IS 1893 Part 1 (2002) as described earlier, I is the importance factor as per Table 1 and S is the site factor as per Table 2. For estimating the vertical total and effective stresses, the water table should be assumed at the highest piezometric elevation likely to be encountered during the operational life of the dam or the embankment except where there is a free standing water column. For assessing liquefaction potential of soil layers underneath free standing water column, the height of free standing water should be neglected and water table should be assumed at the soil surface.

For assessing liquefaction susceptibility using the SPT go to Step 5a, for the CPT go to Step 5b, and the shear wave velocity go to Step 5c, to compute cyclic resistance ratio *($CRR_{7.5}$)* for M_w 7.5 earthquakes. Cyclic resistance ratio, CRR for sites for earthquakes of other magnitudes or for sites underlain by non-horizontal soil layers or where vertical effective stress exceeds 1 atmospheric pressure is estimated by multiplying $CRR_{7.5}$ by three correction factors, K_m, K_α and K_σ respectively. Here correction factors for magnitude sloped stratigraphy and effective stress have been denoted with symbols K_m, K_α and K_σ, respectively. These correction factors are obtained from figures A-1, A-2 and A-3.

Step 5a:

Evaluate the *standardized SPT blow count* (N_{60}) which is the standard penetration test blow count for a hammer with an efficiency of 60 percent. Specifications of the "standardized" equipment corresponding to an efficiency of 60 percent are given in Table A-1 in the absence of test-specific energy measurement. The standardized SPT blow count is obtained from the equation:

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$$N_{60} = N.C_{60}$$

where C_{60} is the product of various correction factors. Correction factors recommended by various investigators for some common SPT configurations are provided in Table A-2.

Calculate the *normalized standardized SPT blow count*, $(N_1)_{60}$ using $(N_1)_{60} = C_N N_{60}$, where $(N_1)_{60}$ is the standardized blow count

normalized to an effective overburden pressure of 98 kPa in order to eliminate the influence of confining pressure. Stress normalization factor C_N is calculated from following expression:

$$C_N = \left(P_a / \sigma'_v\right)^{1/2}$$

Subjected to $C_N \le 2$, where P_a is the atmospheric pressure. However, the closed-form expression proposed by Liao and Whitman (1986) may also be used:

$$C_N = 9.79 \left(1/\sigma'_v\right)^{1/2}$$

The Critical Resistance Ratio (*CRR*) or the resistance of a soil layer against liquefaction is estimated from Figure A-5 for representative $(N_1)_{60}$ value of the deposit.

Step 5b:

Calculate normalized cone tip resistance, $(q_{c1N})_{cs}$, using

$$(q_{c1N})_{cs} = K_c \left(P_a/\sigma'_v\right)^n \left(q_c/P_a\right)$$

where q_c is the measured cone tip resistance corrected for thin layers, exponent n has a value of 0.5 for sand and 1 for clay, and K_c is the correction factor for grain characteristics estimated as follows.

$$K_c = 1.0 \qquad \text{for } I_c \le 1.64 \text{ and}$$

$$K_c = -0.403 I_c^{\,4} + 5.581 I_c^{\,3} - 21.63 I_c^{\,2} + 33.75 I_c - 17.88 \qquad \text{for } I_c > 1.64$$

The soil behavior type index, I_c, is given by

$$I_c = \sqrt{(3.47 - \log Q)^2 + (1.22 + \log F)^2}$$

Where $Q = \left[(q_c - \sigma_v)/P_a\right]\left(P_a/\sigma'_v\right)^n$, $F = \left[f/(q_c - \sigma_v)\right] \times 100$, *f* is the measured sleeve friction and *n* has the same values as described earlier. Assess susceptibility of a

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soil to liquefaction using Figure A-6.

The *CRR* for a soil layer is estimated from Figure A-6 using the $\left(q_{c1N}\right)_{cs}$ value representative of the layer.

Although soils with I_c >2.6 are deemed non-liquefiable, such deposits may soften and deform during earthquakes. General guidance is not available to deal with such possibilities.

Softening and deformability of deposits with I_c>2.6 should thus be treated on a material specific basis.

Step 5c:

Calculate normalized shear wave velocity, V_{s1}, for clean sands using: $V_{s1} = V_s \times \left(P_a / \sigma'_v\right)^{0.25}$ subjected to $V_{s1} \leq 1.3 \times V_s$.

The *CRR* for a soil layer is estimated from Figure A-7 using the V_{s1} value representative of the layer. Appropriate *CRR*-V_{s1} curve should be used in this assessment depending on the fines content of the layer.

Step 6: Correct $CRR_{7.5}$ for earthquake magnitude (M_w), stress level and for initial static shear using correction factors k_m, k_σ and k_α, respectively, according to:

$$CRR = CRR_{7.5}.k_M\, k_\sigma .k_\alpha$$

where, k_m, k_σ k_α are correction factors, respectively for magnitude correction (Figure A-1), effective overburden correction (Figure A-2) and aloping ground correction (Figure A-3), in combination with figure A-4. The Critical Stress ratio $CRR_{7.5}$ is estimated from Figure F-5 for SPT, Figure F-6 for CPT and Figure F-7 for shear wave velocity data.

Step 7: Calculate the factor of safety against initial liquefaction, FS , as:

$$FS = CRR / CSR$$

where *CSR* is as estimated in Step 4 and *CRR* is from Step 6a, 6b or 6c. When the design ground motion is conservative, earthquake-related permanent ground deformation is generally small if $FS \geq 1.1$.

2. Cohesive Soils

Cohesive soils are often deemed to be non-liquefiable if any one of the following conditions is not satisfied (Figure A-8a):

- Percent (by weight) finer than 5 μm ≤ 15 %

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- $w_l \leq 35$ %
- $w_n \leq 0.9 \times w_l$

where w_l is the Liquid Limit and and w_n is the Natural Moisture Content, respectively. These conditions are collectively referred to as the Chinese Criteria. Since the Chinese Criteria are not always conservative, Seed et

al. (2003)[2] recommend the following alternative (Figure A-8b):

- Cohesive soils should be considered liquefiable if $w_l \leq 37$ %, $I_p \leq 12$ % and $w_n \leq 0.85 \times w_l$, where I_p is the Plasticity Index
- Liquefaction susceptibility of soils should be considered marginal if $w_l \leq 47$ %, $I_p \leq 20$ % and $w_n \leq 0.85 \times w_l$, where I_p is the Plasticity Index and for such soils liquefaction susceptibility should be obtained from laboratory testing of undisturbed representative samples

Cohesive soils should be considered non-liquefiable if $w_l \geq 47$ % or $I_p \geq 20$ % or $w_n \geq 0.85 \times w_l$, where I_p is the Plasticity Index

[2] B. Seed, K. O. Cetin, R. E. S. Moss, A. M. Kammerer, J. Wu, J. M. Pestana, M. F. Riemer, R.B. Sancio, J.D. Bray, R. E. Kayen, and A. Faris 2003. Advances in Soil Liquefaction Engineering: A Unified and Consistent Frame Work, **Proceedings of 26th Annual ASCE Los Angeles Geotechnical Spring Seminar**, Keynote Presentation, Long Beach, California.

Table A-1: Recommended "Standardized' SPT Equipment.

Element	Standard Specification
Sampler	Standard split-spoon sampler with: (a) Outside diameter = 51 mm, and Inside Diameter = 35 mm (constant – i.e., no room for liners in the barrel)
Drill Rods	A or AW-type for depths less than 15.2 m; N- or NW-type for greater depths
Hammer	Standard (safety) hammer: (a) drop hammer (b) weight = 65 kg; (c) drop = 750 mm (d) delivers 60% of the theoretical potential energy
Rope	Two wraps of rope around the pulley
Borehole	100 to 130mm diameter borehole
Drill Bit	Upward deflection of drilling mud (tricone or baffled drag bit)
Blow Count Rate	30 to 40 blows per minute
Penetration Resistant Count	Measured over range of 150 to 450 mm of penetration into the ground

Notes:

(1) If the equipment meets the above specifications, $N = N_{60}$ and only a correction for overburden are needed.

(2) This specification is essentially the same to the ASTM D 1586 standard.

Table A-2: Correction Factors for Non-Standard SPT Procedures and Equipment.

Correction for	Correction Factor
Nonstandard Hammer Type (*DH*= doughnut hammer; *ER* = energy ratio)	C_{HT} =0.75 for *DH* with rope and ulley C_{HT} =1.33 for *DH* with trip/auto and *ER* = 80
Nonstandard Hammer Weight or Height of fall (*H* = height of fall in mm; *W* = hammer weight in kg)	$C_{HW} = \frac{H \times W}{63.5 \times 762}$
Nonstandard Sampler Setup (standard samples with room for liners, but used without liners	C_{SS} =1.10 for loose sand C_{SS} =1.20 for dense sand
Nonstandard Sampler Setup (standard samples with room for liners, but liners are used)	C_{SS} =0.90 for loose sand C_{SS} =0.80 for dense sand
Short Rod Length	C_{RL} =0.75 for rod length 0-3 m
Nonstandard Borehole Diameter	C_{BD} =1.05 for 150 mm borehole diameter C_{BD} =1.15 for 200 mm borehole diameter

Notes : *N* = Uncorrected SPT blow count.

$C_{60} = C_{HT}\, C_{HW}\, C_{SS}\, C_{RL}\, C_{BD}$

$N_{60} = N\, C_{60}$

C_N = Correction factor for overburden pressure

$(N_1)_{60} = C_N\, N_{60} = C_N\, C_{60}\, N$

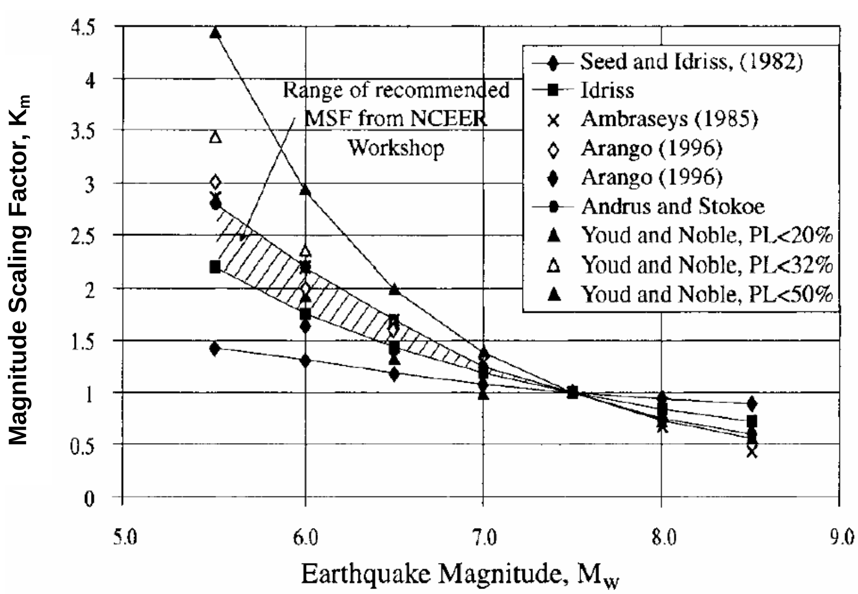


Figure A-1: Magnitude Correction factor

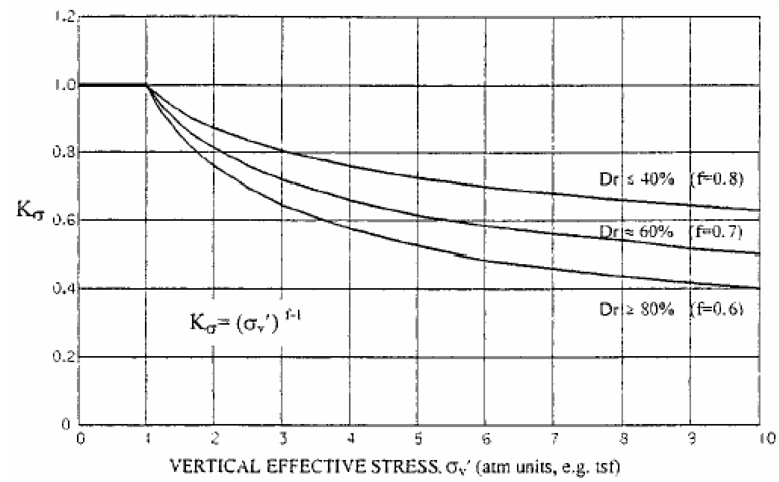


Figure A-2: Stress correction factor

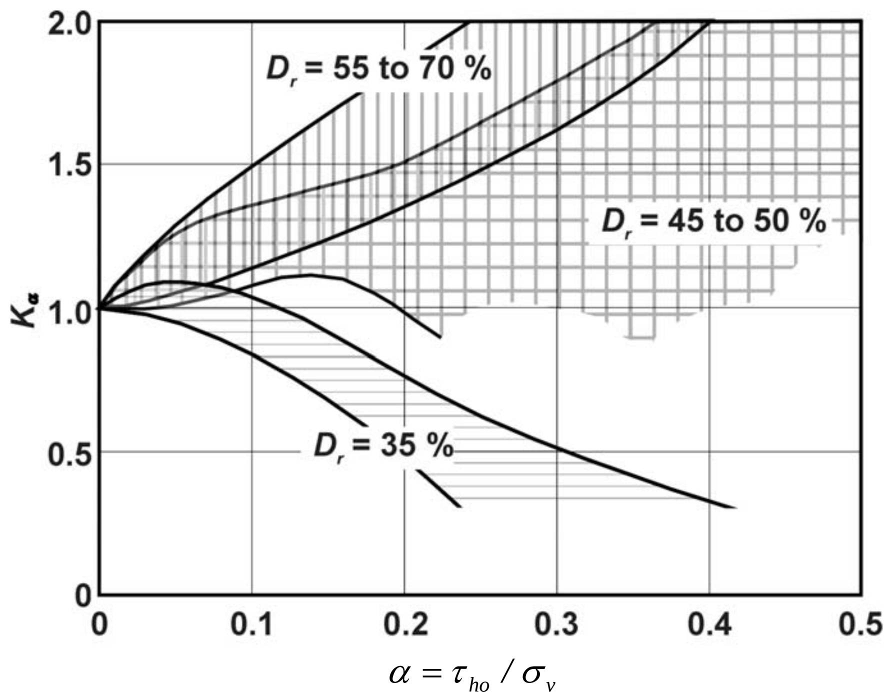
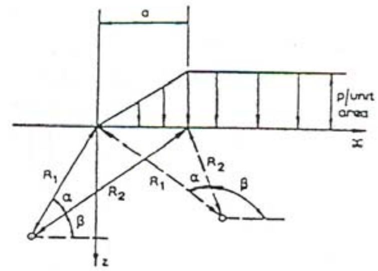


Figure A-3: Correction for initial static shear (Note: Initial static shear for an embankment may be estimated from Figure A-4)



$$\sigma_z = \frac{p}{\pi a}\left[a\beta + x\alpha\right]$$

$$\sigma_x = \frac{p}{\pi a}\left[a\beta + x\alpha + 2z \log_e \frac{R_2}{R_1}\right]$$

$$\tau_{xz} = -\frac{p}{\pi a} z \alpha$$

$$\left.\begin{matrix}\sigma_1\\ \sigma_3\end{matrix}\right\} = \frac{p}{\pi a}\left[\left(a\beta + x\alpha + z \log_e \frac{R_2}{R_1}\right) \pm z \left(\log_e^2 \frac{R_2}{R_1} + \alpha^2\right)^{1/2}\right]$$

$$\tau_{max} = \frac{pz}{\pi a}\left(\log_e^2 \frac{R_2}{R_1} + \alpha^2\right)^{1/2}$$

Figure A-4: Initial static shear under an embankment

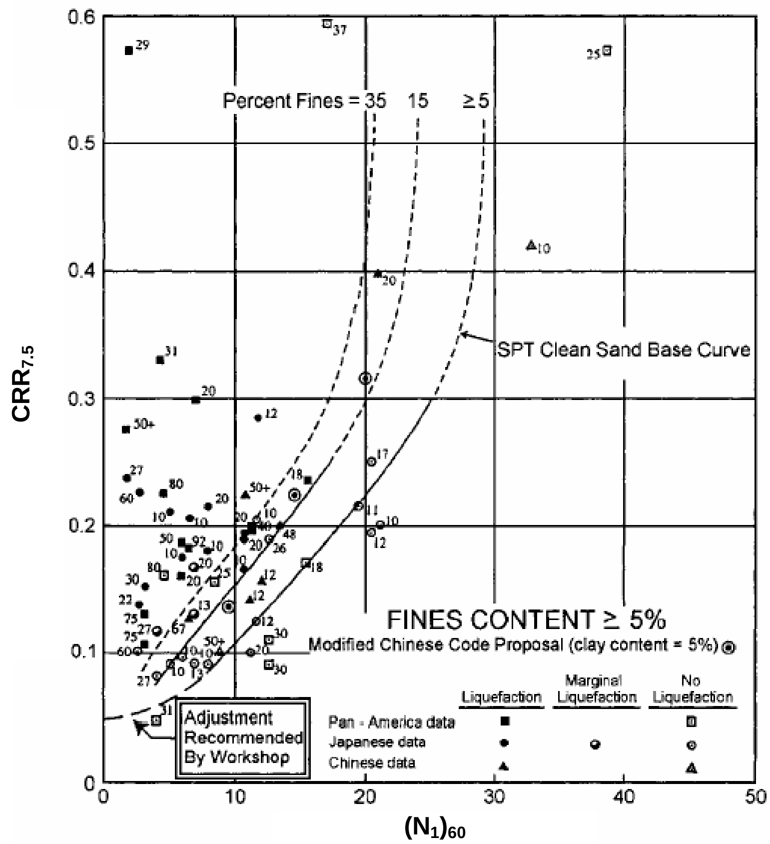


Figure A-5: Relationship between *CRR* and $(N_1)_{60}$ for sand for M_w *7.5* earthquakes

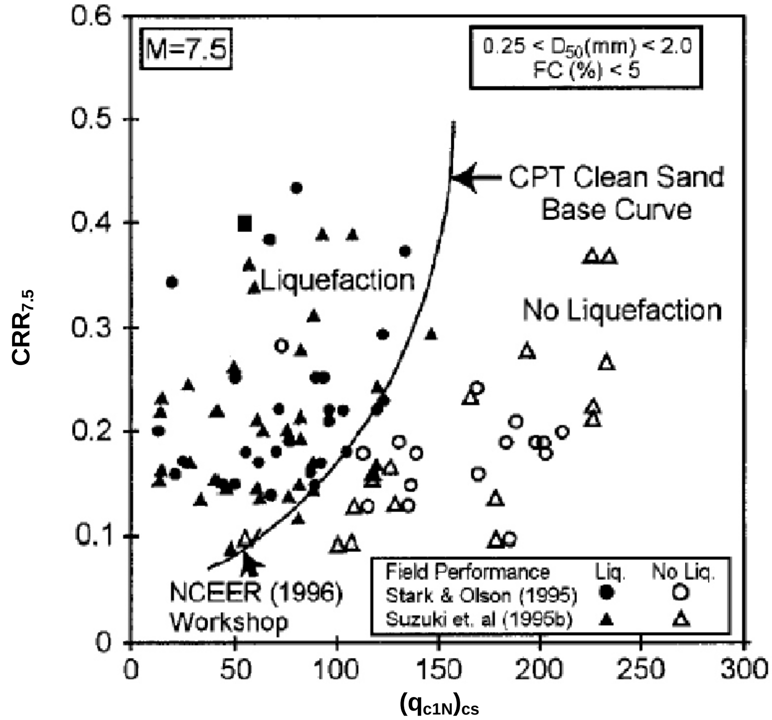


Figure A-6: Relationship between *CRR* and $(q_{c1N})_{cs}$ for M_w, *7.5* earthquakes

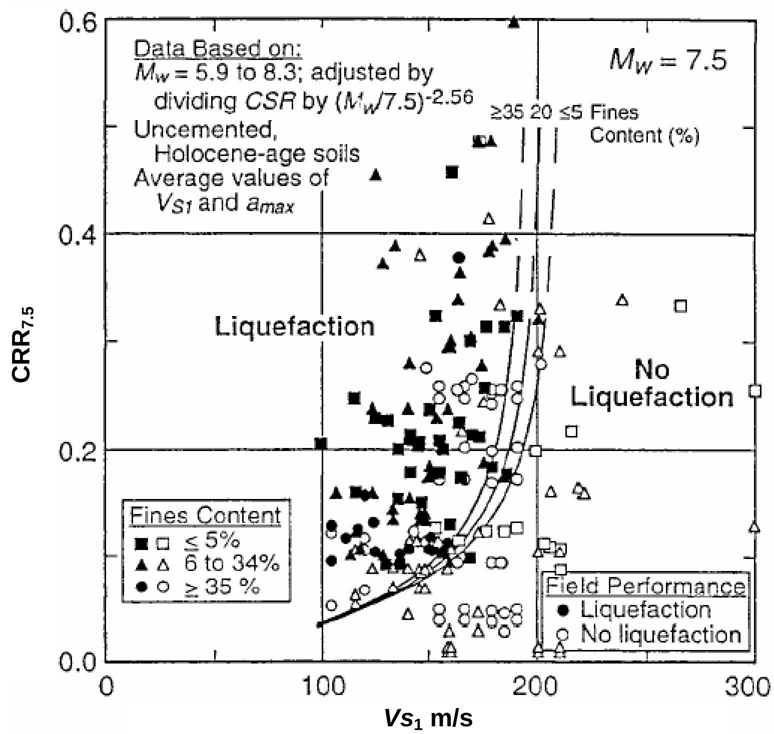


Figure A-7: Relationship between *CRR* and Vs_1 for M_w, *7.5* earthquakes

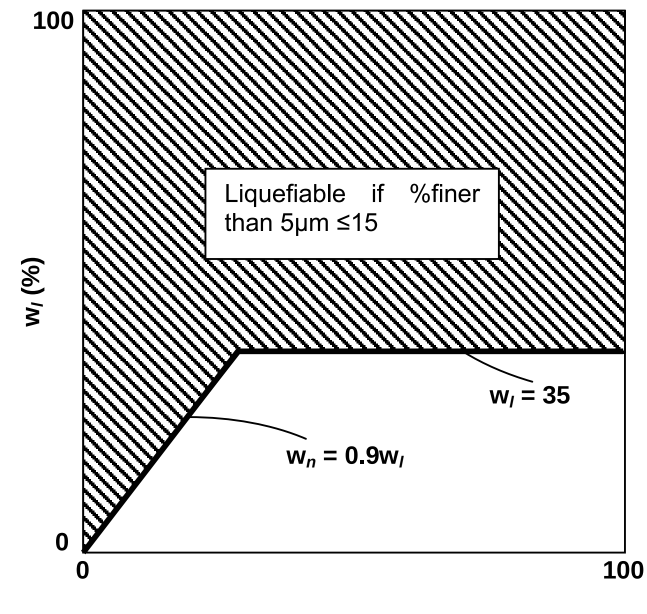


Figure A-8a: The Chinese Criteria (Seed et.al., 2003)

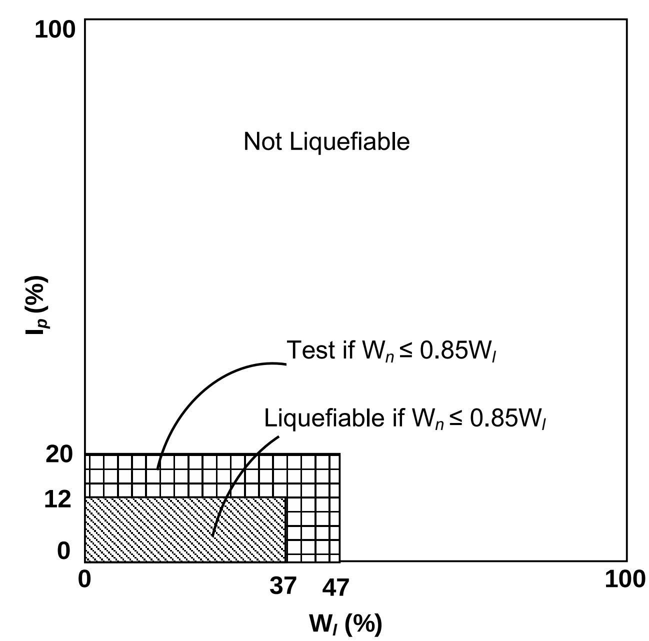


Figure A-8b: Proposal of Seed et al. (2003)

IITK-GSDMA GUIDELINES for SEISMIC DESIGN of EARTH DAMS AND EMBANKMENTS

Provisions with Commentary and Explanatory Examples

PART 2: EXPLANATORY EXAMPLES

Example: 1 Liquefaction Analysis using SPT data

Problem Statement:

The measured SPT resistance and results of sieve analysis for a site in Zone IV are given in Table 1.1. Determine the extent to which liquefaction is expected for a 7.5 magnitude earthquake. The site is level, the total unit weight of the soil layers is 18.5 kN/m3, the embankment height is 10 m and the water table is at the ground surface. Estimate the liquefaction potential immediately downstream of the toe of the embankment.

Table 1.1: Result of the Standard penetration Test and Sieve Analysis

Depth (m)	N_{60}	Soil Classification	Percentage fine
0.75	9	Poorly Graded Sand and Silty Sand (SP-SM)	11
3.75	17	Poorly Graded Sand and Silty Sand (SP-SM)	16
6.75	13	Poorly Graded Sand and Silty Sand (SP-SM)	12
9.75	18	Poorly Graded Sand and Silty Sand (SP-SM)	8
12.75	17	Poorly Graded Sand and Silty Sand (SP-SM)	8
15.75	15	Poorly Graded Sand and Silty Sand (SP-SM)	7
18.75	26	Poorly Graded Sand and Silty Sand (SP-SM)	6

Solution:

Site Characterization:

This site consists of loose to dense poorly graded sand to silty sand (SP-SM). The SPT values ranges from 9 to 26. The site is located in zone IV. The peak horizontal ground acceleration value for the site will be taken as 0.24g corresponding to zone factor Z = 0.24

Liquefaction Potential of Underlying Soil

Step by step calculation for the depth of 12.75m is given below. Detailed calculations for all the depths are given in Table 1.2. This table provides the factor of safety against liquefaction (FS), maximum depth of liquefaction below the ground surface.

$$a_{\max} = Z \times I \times S$$

$$a_{\max} = 0.24 \times 1 \times 1 = 0.24$$

$$M_w = 7.5\,,\ \gamma_{sat} = 18.5\ kN/m^3\,,$$

$$\gamma_w = 9.8\ kN/m^3$$

Considering water table at ground surface, sample calculations for 12.75m depth are as follows.

Initial stresses:

$$\sigma_v = 12.75 \times 18.5 = 235.9\ kPa$$

$$u_0 = (12.75 - 0.00) \times 9.8 = 124.95\ kPa$$

$$\sigma_v' = (\sigma_v - u_0) = 235.9 - 124.95$$

$$= 110.95\ kPa$$

Stress reduction factor:

$$r_d = 1.174 - 0.0267z = 1.174 - 0.0267 \times 12.75 = 0.83$$

Critical stress ratio induced by earthquake:

$$a_{max} = 0.24g\,,\ M_w = 7.5$$

$$CSR = 0.65 \times (a_{maz} / g) \times r_d \times (\sigma_v / \sigma_v')$$

$$CSR = 0.65 \times (0.24) \times 0.83 \times (235.9 / 110.95)$$

$$= 0.28$$

Correction for SPT (N) value for overburden pressure:

$$(N_1)_{60} = C_N \times N_{60}$$

$$C_N = 9.79\,(1/\sigma_v')^{1/2}$$

$$C_N = 9.79\,(1/110.95)^{1/2} = 0.93$$

$$(N_1)_{60} = 0.93 \times 17 = 16$$

Cyclic stress ratio resisting liquefaction:

For $(N_1)_{60} = 16$, fines content of 8%

$CRR_{7.5} = 0.22$ (Figure A-5)

Corrected Cyclic Stress Ratio Resisting Liquefaction:

$CRR = CRR_{7.5}\ k_m\ k_\alpha\ k_\sigma$

$K_m =$ Correction factor for earthquake magnitude other than 7.5 (Figure A-1)
$= 1.00$ for $M_w = 7.5$

$K_\alpha =$ Correction factor for initial driving static shear (Figure A-3)
$= 1.00$, since no initial static shear

$K_\sigma =$ Correction factor for stress level larger than 96 kPa (Figure A-2) $= 0.88$

$$CRR = 0.22 \times 1 \times 1 \times 0.88 = 0.19$$

Factor of safety against liquefaction:

$$FS \ = CRR / CSR = 0.19 / 0.28 = 0.70$$

It shows that the considered strata is liable to liquefy.

Summary:

The extent of liquefaction for the strata of considered site on be read from Table 1.2, where F. S. < 1.0 indicates the possibility of liquefaction.

Table 1.2: Liquefaction Analysis: Water Level at GL

Depth	%Fine	σ_v (kPa)	σ_v' (kPa)	N_{60}	C_N	$(N)_{60}$	r_d	CSR	$CRR_{7.5}$	CRR	FS
0.75	11.00	13.9	6.5	9.00	2.00	18	0.99	0.33	0.24	0.27	0.82
3.75	16.00	69.4	32.6	17.00	1.71	29	0.97	0.32	0.32	0.34	1.04
6.75	12.00	124.9	58.7	13.00	1.28	17	0.95	0.31	0.21	0.20	0.65
9.75	8.00	180.4	84.8	18.00	1.06	19	0.91	0.30	0.23	0.21	0.69
12.75	8.00	235.9	110.9	17.00	0.93	16	0.83	0.28	0.22	0.19	0.70
15.75	7.00	291.4	137.0	15.00	0.84	13	0.75	0.25	0.16	0.13	0.53
18.75	6.00	346.9	163.1	26.00	0.77	20	0.67	0.22	0.22	0.18	0.80

Example: 2 Liquefaction Analysis using CPT data

Problem Statement:

Prepare a plot of factors of safety against liquefaction versus depth. The results of the cone penetration test (CPT) of 15m thick layer in Zone V are provided in the first three columns of Table 2.1. Assume the water table to be at a depth of 2.35 m, the unit weight of the soil to be 18 kN/m^3 and the magnitude of 7.5 and the peak horizontal ground acceleration as 0.15g.

Table 2.1: Result of the Cone penetration Test

Depth (m)	$\overline{q_c}$	$\overline{f_s}$	Depth (m)	$\overline{q_c}$	$\overline{f_s}$	Depth (m)	$\overline{q_c}$	$\overline{f_s}$
0.50	64.56	0.652	**5.50**	49.70	0.235	**10.50**	116.1	0.248
1.00	95.49	0.602	**6.00**	51.43	0.233	**11.00**	97.88	0.159
1.50	39.28	0.281	**6.50**	64.94	0.291	**11.50**	127.5	0.218
2.00	20.62	0.219	**7.00**	57.24	0.181	**12.00**	107.86	0.193
2.50	150.93	1.027	**7.50**	45.46	0.132	**12.50**	107.2	0.231
3.00	55.50	0.595	**8.00**	39.39	0.135	**13.00**	124.78	0.275
3.50	10.74	0.359	**8.50**	36.68	0.099	**13.50**	145.18	0.208
4.00	9.11	0.144	**9.00**	45.30	0.129	**14.00**	138.53	0.173
4.50	33.69	0.297	**9.50**	102.41	0.185	**14.50**	123.95	0.161
5.00	70.69	0.357	**10.00**	92.78	0.193	**15.00**	124.41	0.155

Solution:

Liquefaction Potential of Underlying Soil:

The result of assessment of liquefaction potential provided in the last column of Table 2.1, where FS denotes the factor of safety against liquefaction (= $CRR_{7.5}$/CSR). Step by step calculation for the soil at depth of 4.5m is given below for illustration. Detailed calculations are given in Table 2.2, which provides the factor of safety against liquefaction (FS_{liq}).

a_{max}/g = 0.15, M_w=7.5,

$$\gamma_{sat} = 18\ kN/m^3,\ \gamma_w = 9.8\ kN/m^3$$

Depth of water level below G.L. = 2.35m

Depth at which liquefaction potential is to be evaluated = 4.5m

Initial stresses:

$$\sigma_v = 4.5 \times 18 = 81.00\ kPa$$

$$u_0 = (4.5 - 2.35) \times 9.8 = 21.07\ kPa$$

$$\sigma_v' = (\sigma_v - u_0) = 81 - 21.07 = 59.93\ kPa$$

Stress reduction factor:

$$r_d = 1 - 0.00765\ z$$
$$= 1 - 0.00765 \times 4.5 = 0.965$$

Critical stress ratio induced by earthquake:

$$CSR = 0.65 \times (a_{max}/g) \times r_d \times (\sigma_v / \sigma_v')$$

$$CSR = 0.65 \times (0.15) \times 0.965 \times (81/59.93)$$
$$= 0.13$$

Correction factor for grain characteristics:

$$K_c = 1.0 \qquad for\ I_c \le 1.64\ and$$
$$K_c = -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75 I_c - 17.88$$
$$for\ I_c > 1.64$$

where the soil behavior type index, I_c, is given by

$$I_c = \sqrt{(3.47 - \log Q)^2 + (1.22 + \log F)^2}$$

$$I_c = \sqrt{(3.47 - \log 42.19)^2 + (1.22 + \log 0.903)^2}$$
$$= 2.19$$

Where,

$$F = [f/(q_c - \sigma_v)] \times 100$$

$F = [29.7/(3369 - 81)] \times 100 = 0.903$ and

$$Q = [(q_c - \sigma_v)/P_a](P_a/\sigma'_v)^n$$

$$Q = [(3369 - 81)/101.35] \times (101.35/59.93)^{0.5}$$
$$= 42.19$$

$$K_c = -0.403(2.19)^4 + 5.581(2.19)^3$$
$$- 21.63(2.19)^2 + 33.75(2.19) - 17.88 = 1.64$$

Normalized Cone Tip Resistance:

$$(q_{c1N})_{cs} = K_c (P_a/\sigma'_v)^n (q_c/P_a)$$

$$(q_{c1N})_{cs} = 1.64(101.35/59.93)^{0.5}(3369/101.35)$$
$$= 70.77$$

Factor of safety against liquefaction:

For $(q_{c1N})_{cs} = 70.77$,

$CRR_{7.5} = 0.11$ (Figure A-6)

Corrected Critical Stress Ratio Resisting Liquefaction:

$$CRR = CRR_{7.5}\ k_m\ k_\alpha\ k_\sigma$$

$K_m =$ Correction factor for earthquake magnitude other than 7.5 (Figure A-4)
$= 1.00$ for $M_w = 7.5$

$K_\alpha =$ Correction factor for initial driving static shear (Figure A-6)
$= 1.00$, since no initial static shear

$K_\sigma =$ Correction factor for stress level larger than 100 kPa (Figure A-5)
$= 1.00$

$$CRR = 0.11 \times 1 \times 1 \times 1 = 0.11$$

$$FS = CRR / CSR$$

$$FS = 0.11 / 0.13 = 0.86$$

Summary:

The analysis shows that the strata between depths 4-9m are liable to liquefy under earthquake shaking corresponding to peak ground acceleration of 0.15g. The plot for depth verses factor of safety is shown in Figure 2.1.

Table 2.2: Liquefaction Analysis: Water Level 2.35 m below GL (Units: kN and Meters)

Depth	σ_v	σ_v'	r_d	qc (kPa)	fs (kPa)	CSR	F	Q	Ic	Kc	(qc1N)cs	$CRR_{7.5}$	CRR	FS
0.50	9.00	9.00	1.00	6456	65.20	0.10	0.45	241.91	1.40	1.00	242.06	0.20	0.20	2.10
1.00	18.00	18.00	0.99	9549	60.20	0.10	0.63	159.87	1.63	1.00	160.17	100.00	100.00	1033.55
1.50	27.00	27.00	0.99	3928	28.10	0.10	0.72	65.43	1.97	1.27	83.53	0.13	0.13	1.39
2.00	36.00	36.00	0.98	2062	21.90	0.10	1.08	33.54	2.31	1.99	68.04	0.11	0.11	1.14
2.50	45.00	43.53	0.98	15093	102.70	0.10	0.68	226.55	1.53	1.00	227.23	100.00	100.00	1011.48
3.00	54.00	47.63	0.98	5550	59.50	0.11	1.08	79.10	2.01	1.31	105.02	0.19	0.19	1.74
3.50	63.00	51.73	0.97	1074	35.90	0.12	3.55	13.96	2.92	5.92	87.81	0.14	0.14	1.24
4.00	72.00	55.83	0.97	911	14.40	0.12	1.72	11.15	2.83	5.01	60.64	0.10	0.10	0.83
4.50	81.00	59.93	0.97	3369	29.70	0.13	0.90	42.19	2.19	1.64	70.77	0.11	0.11	0.89
5.00	90.00	64.03	0.96	7069	35.70	0.13	0.51	86.63	1.79	1.10	96.60	0.16	0.16	1.24
5.50	99.00	68.13	0.96	4970	23.50	0.14	0.48	58.62	1.93	1.22	72.68	0.12	0.12	0.85
6.00	108.00	72.23	0.95	5143	23.30	0.14	0.46	58.85	1.92	1.21	72.45	0.12	0.12	0.83
6.50	117.00	76.33	0.95	6494	29.10	0.14	0.46	72.50	1.83	1.13	83.61	0.13	0.13	0.95
7.00	126.00	80.43	0.95	5724	18.10	0.14	0.32	62.00	1.83	1.13	71.56	0.11	0.11	0.79
7.50	135.00	84.53	0.94	4546	13.20	0.15	0.30	47.66	1.92	1.21	59.46	0.10	0.10	0.68
8.00	144.00	88.63	0.94	3939	13.50	0.15	0.36	40.04	2.02	1.33	55.18	0.10	0.10	0.64
8.50	153.00	92.73	0.93	3668	9.90	0.15	0.28	36.26	2.02	1.33	50.45	0.09	0.09	0.61
9.00	162.00	96.83	0.93	4530	12.90	0.15	0.30	44.09	1.95	1.24	56.79	0.10	0.10	0.64
9.50	171.00	100.93	0.92	10210	18.50	0.15	0.37	48.78	1.95	1.24	62.62	0.18	0.18	1.16
10.00	180.00	105.03	0.91	9278	19.30	0.15	0.43	43.22	2.02	1.33	59.94	0.15	0.15	0.97
10.50	189.00	109.13	0.89	11610	24.80	0.15	0.44	53.40	1.95	1.23	68.16	0.21	0.21	1.36
11.00	198.00	113.23	0.88	9788	15.90	0.15	0.34	43.84	1.98	1.27	58.01	0.15	0.15	1.01
11.50	207.00	117.33	0.87	12750	21.80	0.15	0.35	56.56	1.88	1.17	68.51	0.23	0.23	1.53
12.00	216.00	121.43	0.85	10786	19.30	0.15	0.37	46.67	1.97	1.26	61.23	0.17	0.17	1.12
12.50	225.00	125.53	0.84	10720	23.10	0.15	0.45	45.53	2.01	1.31	62.48	0.16	0.16	1.09
13.00	234.00	129.63	0.83	12478	27.50	0.15	0.46	52.39	1.96	1.25	68.09	0.20	0.20	1.37
13.50	243.00	133.73	0.81	14518	20.80	0.14	0.40	44.79	2.00	1.29	60.67	0.26	0.26	1.81
14.00	252.00	137.83	0.80	13853	17.30	0.14	0.35	41.93	2.00	1.30	57.21	0.23	0.23	1.61
14.50	261.00	141.93	0.79	12396	16.10	0.14	0.37	36.68	2.06	1.39	53.90	0.18	0.18	1.29
15.00	270.00	146.03	0.77	12441	15.50	0.14	0.35	36.23	2.06	1.38	53.24	0.18	0.18	1.29

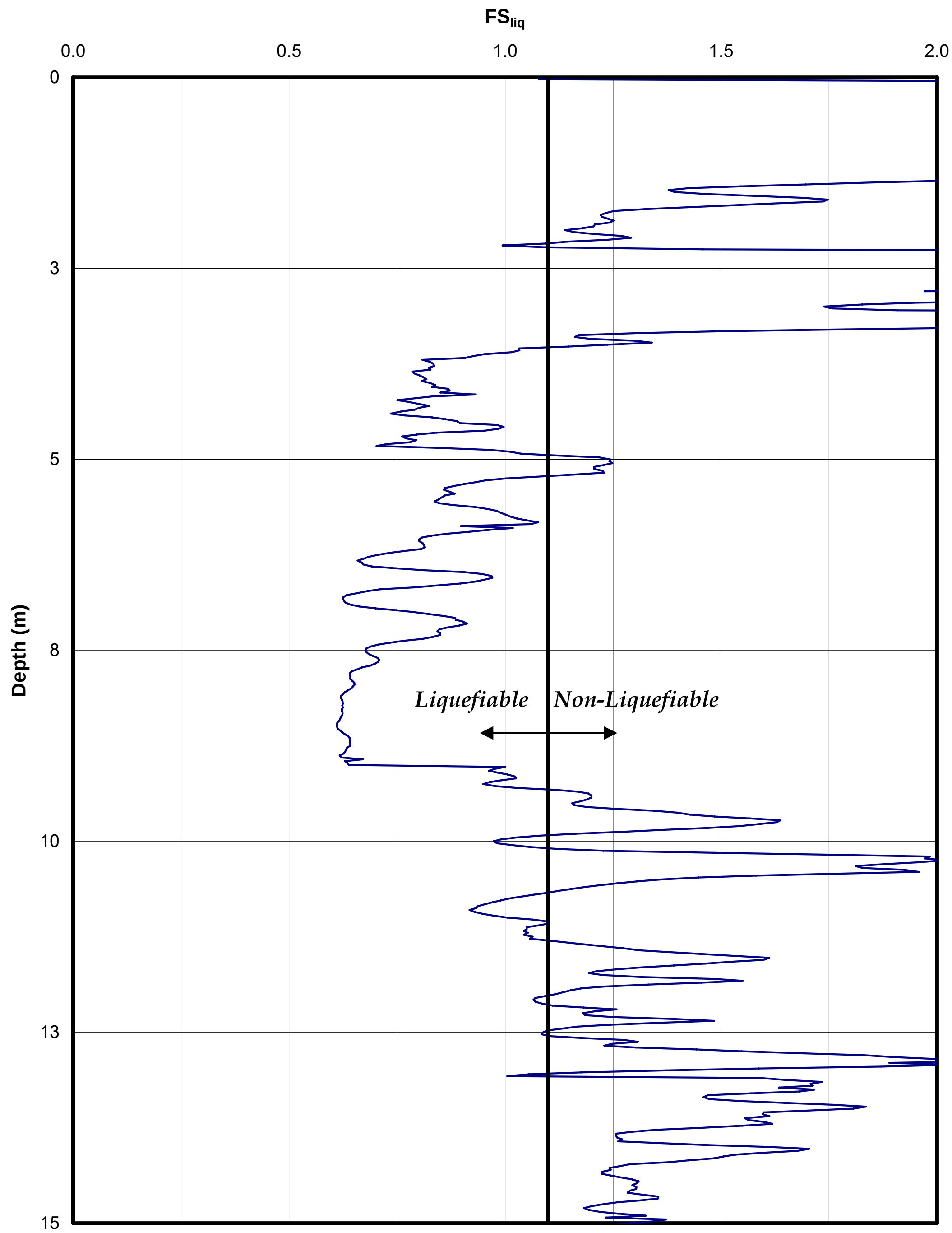


Figure 2.1: Factor of Safety against Liquefaction

Example: 3 Deformation Estimation for an Embankment

Problem Statement:

A 10m high embankment (Figure 3.1) is located in seismic zone IV. Take importance factor I as 1.0. The foundation conditions include a non-liquefiable crust of clayey silt extending to 1.5 m below ground surface underlain by a 1.5-m thick liquefiable layer and a non-liquefiable soils. The embankment is constructed with well compacted granular soils not expected to liquefy during the design earthquake. Evaluate the slope stability. Assume the magnitude of earthquake as 6.5 if required.

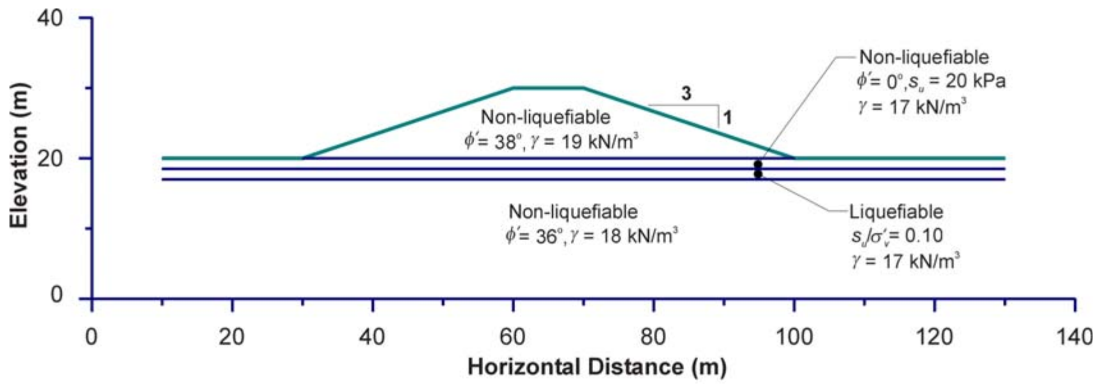


Figure 3.1: Typical Cross Section of the Embankment

Solution:

Figure 3.2 presents the results of a equivalent-static stability analysis carried out to estimate the "yield acceleration." The yield acceleration is defined as the horizontal seismic coefficient for which the earth structure remains marginally stable with a limiting equilibrium factor of safety of 1.0. which represents the potential sliding mass (the soil mass above the circular failure surface in Figure 3.2) is on the verge of being mobilized downslope. The Modified Bishop Method was used in the equivalent-static analysis. The results presented in Figure 3.2 indicate the yield acceleration of 0.056g.

For seismic zone IV, $Z = 0.24$

Importance factor, $I = 1.0$ (given)

Horizontal peak ground acceleration = $Z \times I$

$= 0.24g \times 1.0$

$= 0.24g$

$$\frac{Yield\ Acceleration}{Design\ PHGA} = \frac{0.056\,g}{0.24\,g} = 0.23$$

From the relationship of Hynes-Griffin and Franklin, 1984 (Figure 2)

Upper bound permanent displacement = 0.8 m

Mean plus sigma displacement = 0.2 2m

Mean displacement = 0.13 m

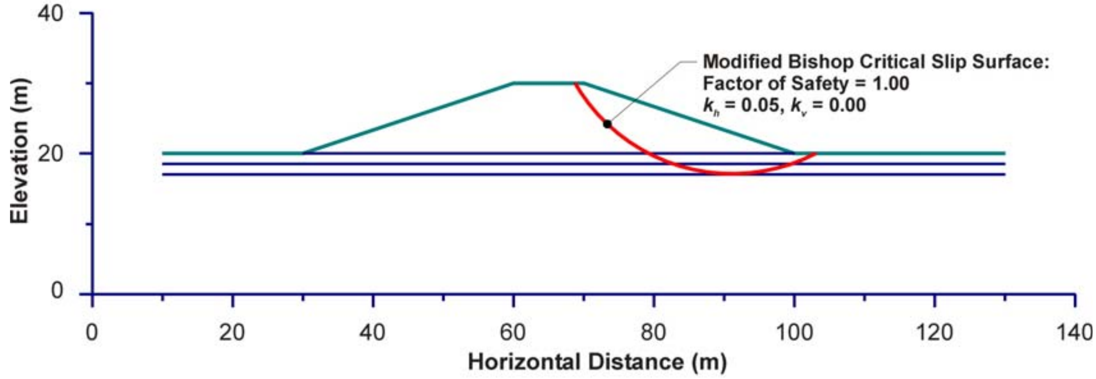


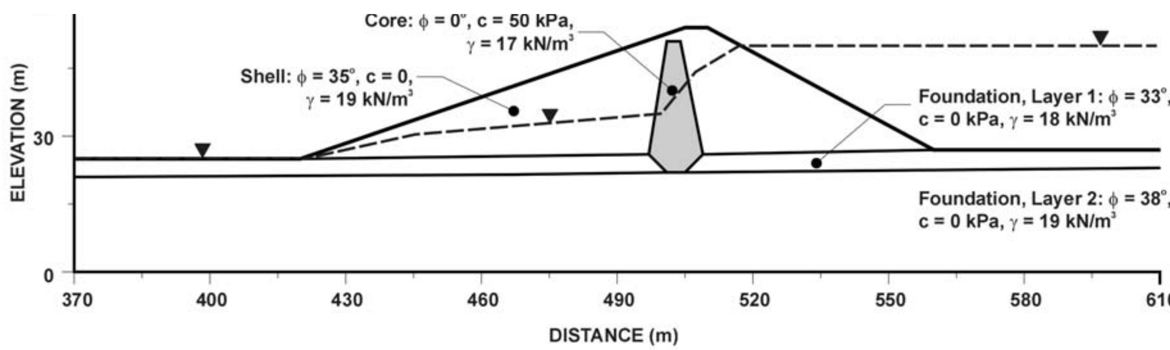
Figure 3.2: Results for Stability Analysis for Yield Acceleration

Example: 4 Deformation Estimation for an Earthen Dam

Problem Statement:

To evaluate a proposal for changes in land use downstream of an existing dam, the stability of the dam during an extreme earthquake event needs to be checked to preclude dam break during the extreme event. The dam is located at a site underlain by deep alluvium comprised of medium dense to dense sand near surface, which is in turn underlain by stiff clay. The estimates of the magnitude and the free-field peak ground acceleration at a hard-soil/soft-rock site are 7.7 and 0.5*g*, respectively, for the extreme earthquake. Materials used in the construction of dam and those within the underlying foundation layers are dilatant and are therefore not expected to liquefy during the extreme earthquake. The maximum dam section is shown on Figure 4.1a along with material properties and the wet-season piezometric surface inferred from piezometers installed across the maximum dam section. The strength parameters for shell of the dam and foundation layers are based on SPT, while the strength parameters for the dam core are from field vane testing. The in-situ tests were conducted specifically for the present dam safety reassessment. Assess the stability of the dam during the extreme earthquake.

a. Profile, soil properties and piezometric surface



b. Result of Slope Stability Analysis

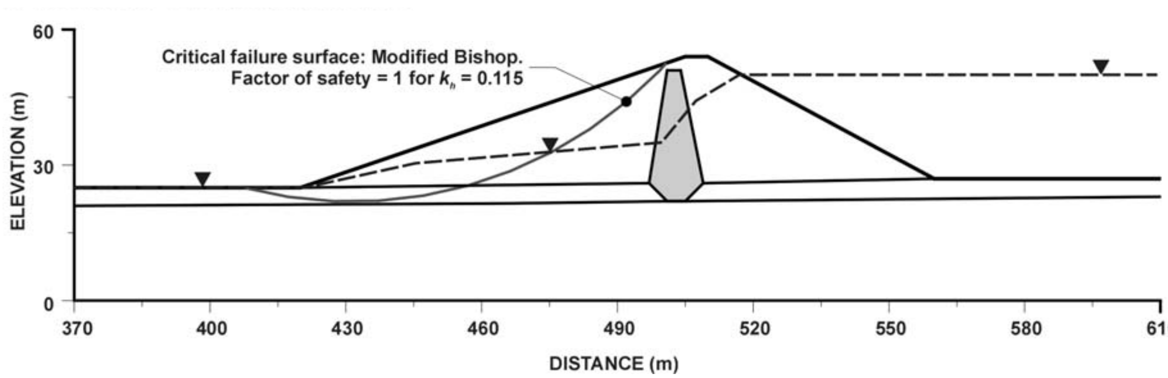


Figure 4.1 Maximum dam section and equivalent-static slope stability results

Solution

Although earthquake-related slope stability analysis is essentially an undrained problem, since the soils within the dam body and the underlying foundation are dilatant, the drained shear strength parameters can conservatively be used in equivalent-static slope stability calculations. To account for the possibility of strain softening of the soils within the dam body and those within the underlying foundation factored strength are used in the equivalent- static slope stability analysis. Using a resistance factor of 0.8 for all soil types, the following shear strength parameters are obtained:

1. Core: $c^* = 0.8 \times c = 0.8 \times 50 = 40\ kPa$, $\phi^* = 0$
2. Shell: $c^* = 0$, $\phi^* = tan^{-1}(0.8 \times tan\,\phi) = tan^{-1}(0.8 \times tan 35°) = 29.3°$
3. Foundation layer 1: $c^* = 0$, $\phi^* = tan^{-1}(0.8 \times tan\,\phi) = tan^{-1}(0.8 \times tan 33°) = 27.5°$ and
4. Foundation layer 2: $c^* = 0$, $\phi^* = tan^{-1}(0.8 \times tan\,\phi) = tan^{-1}(0.8 \times tan 38°) = 32°$

where symbols with asterisks represent factored shear strength.

Equivalent-static stability assessment using the modified Bishop method and factored soil shear strength parameters indicate that the yield acceleration is 0.115*g* (Figure 4.1b). Since the site is underlain primarily by dense sand and stiff clay, the estimated peak horizontal ground acceleration for hard soil site can be used in the sliding block analysis directly. Consequently, the ratio of yield acceleration to peak horizontal ground acceleration becomes 0.115/0.5 = 0.23, for which the upper-bound relationship of Figure 2 indicates that a displacement of about 0.25 m may develop during the extreme earthquake. Since the estimated displacement is smaller than 1 m, the dam performance during the extreme event may be considered as acceptable.

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August 2005; Revised May 2007