GUIDELINES FOR THE GEOTECHNICAL DESIGN, UPGRADING AND REHABILITATION OF RIVER EMBANKMENTS

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A chi mi ha insegnato tutto e tutto mi ha dedicato: mia madre.

A mio padre, mio nonno Giuseppe e mia nonna Antonia.

Ai miei amici.

A noi e a me stessa.
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ABSTRACT

The greatest cities in human history have risen up on the banks of rivers and by the seaside. In fact, living by the water provides with a number of advantages and assets such as fertile farm land, transportation, trade and hydroelectric power. On the other hand, in doing so, people chose to live in close confines with an unruly force and, unfortunately, often they forget it.

A levee is a man-made embankment built to contain and control the flow of water (of a river) so as to provide protection against floods or to prevent ocean waves from washing into undesired areas. In parts of the Netherlands, for example, levees stop ocean waters from reclaiming thousands of miles of land (much of which is either at or below sea level). Often levees are little more than trapezoidal mounds of less permeable soil (clay) which run in a long strip for many kilometers along a river (or a lake or ocean).

Often levees are in urgent need of repair. The problems are several: poor maintenance; design and construction flaws, for example sometimes levees have inadequate freeboard, building encroachments. Moreover, the presence of burrowing animal holes or decayed pipes and pumping stations can put the levee structure in critical condition.

Local governments are responsible for upgrading unacceptable levees. Local governments which oversee the design and construction of levees have to update levees on compliance with levee construction, operation and maintenance standards, often with insufficient funds.

Moreover, climate changes have made severe storms more common in the latest years. Therefore, protection against flood is going to be more and more a national problem.

The main aim of the carried out research is the development of a set of technical guidelines for the design, upgrading and rehabilitation of river embankments. Therefore, this thesis deals with the main geotechnical problems characterizing river embankments.

It is divided into two sections: while the first one deals with river embankment design and construction, the second section deals with inspection, maintenance, monitoring and remediation.
Obviously, the first aspect to deal with is the design of the geotechnical campaign (Chapter 1). In fact, for river embankments, site characterization requires considerable expertise and flexible budgets since they run for many kilometers sometimes hundreds of kilometers. The second chapter deals with the most common levee failure mechanisms. Defining and understanding the levee failure mechanisms is essential to identify design issues. In the third chapter Technical Codes provision are summed up (Italian and USA codes).

As far as the second section is concerned, a monitoring system to realistically evaluate hydraulic and saturation conditions of an embankment is proposed. Moreover, since during the construction stage is essential to check the degree of compaction, an innovative method to evaluate the degree of compaction of both existing and new river embankments (fine grained soils) after their compaction, by using laboratory and in situ testing, is also proposed. Finally, a chapter deals with the countermeasures to prevent levee failure.

**Key words:**

Levee; embankment; geotechnical design; failure causes; internal erosion; overtopping; stability analysis; degree of compaction; monitoring system, soil moisture content.
List of Papers


INTRODUCTION

Climate changes have made severe storms more common in the latest years. Over time, higher and higher levels of flood protection should be sought to minimize flood damage that could occur during events that exceed design levels so embankment improvement alternatives that can be more easily expanded should be promoted. Therefore, protection against flood is going to be more and more a national problem. Conversely, flood protection is often provided by river embankment systems that were completed before the use of modern construction methods. Moreover, over time, most river embankment systems have lost integrity because of flood–related distress, settlements or erosion. Some river embankments contain penetrations. Therefore, our ability to verify the provided level of flood protection is put to the test. Local governments which oversee the design and construction of river embankment have to update levees on compliance with river embankment construction, operation and maintenance standards, often with insufficient funds.

In order to decide what is needed, risk based evaluation of potential failure modes is necessary. Some consistent quantitative measure by which to judge priorities are needed. Since a balance between public safety and costs is necessary, risk analysis allows the greatest risk reduction with the available funding to be reached.

Since each levee is unique, a detailed investigation and the evaluation of the potential failure modes and their consequences can allow a decision based on potential for failure and risk to be made.

River embankment failure, although it is of limited extension, causes the achievement of the ultimate limit state of the entire embankment system. River embankment breaches caused by slope instability and seepage or underseepage tend to occur rapidly and with little or no warning, leaving little opportunity for evacuation before flooding. Conversely, breaches caused by embankment overtopping tend to progress more slowly and are foreseeable so in some cases can be prevented, or, at least, them provide better opportunity to evacuate the threaten area and to minimize damages.

Criteria for operation, maintenance, inspection, monitoring and remediation of poor performance are needed to provide reasonable assurance that river embankment systems are being properly maintained and performing as intended (Department of Water Resources, 2012).
The main aim of this research is the development of a set of technical guidelines for the design, upgrading and rehabilitation of river embankments.
RIVER EMBANKMENTS DESIGN AND CONSTRUCTION
Levees are embankments whose primary purpose is to furnish flood protection from seasonal high water so they are subject to water loading for only few days or weeks during the year.

Even if they are similar to small earth dams they differ from earth dams because:
- they may become saturated for only a short period of time;
- their alignment is dictated by flood protection requirements;
- borrow is generally obtained from channels excavated adjacent to the levee.

Therefore, foundation soils are often poor while fill material is often heterogeneous and far from ideal and so levee sections are selected depending on the properties of the poorest material used.

Conversely, embankments that are subject to water loading for prolonged should be designed in accordance with earth dam criteria.

Herein only embankments which provide protection from flooding in communities, including their industrial, commercial and residential facilities, are dealt with.

Factors that must be considered in levee design are several and they may vary from project to project so it is not possible to establish a general project procedure but it is possible to follow logical steps based on successful past projects as shown by Table 1–1 from US Army Corps of Engineers' (EM 1110-2-1913) (Figure 1).

<table>
<thead>
<tr>
<th>Table 1-1 Major and Minimum Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Step</td>
</tr>
<tr>
<td>------</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
</tbody>
</table>
| 3    | Initiate final exploration to provide:
|      | a. Additional information on soil profiles. |
|      | b. Undisturbed strengths of foundation materials. |
|      | c. More detailed information on borrow areas and other required excavations. |
| 4    | Using the information obtained in Step 3:
|      | a. Determine both embankment and foundation soil parameters and refine preliminary sections where needed, noting all possible problem areas. |
|      | b. Compute rough quantities of suitable material and refine borrow area locations. |
| 5    | Divide the entire levee into reaches of similar foundation conditions, embankment height, and fill material and assign a typical trial section to each reach. |
| 6    | Analyze each trial section as needed for:
|      | a. Underseepage and through seepage. |
|      | b. Slope stability. |
|      | c. Settlement. |
|      | d. Trafficability of the levee surface. |
| 7    | Design special treatment to preclude any problems as determined from Step 6. Determine surfacing requirements for the levee based on its expected future use. |
| 8    | Based on the results of Step 7, establish final sections for each reach. |
| 9    | Compute final quantities needed; determine final borrow area locations. |
| 10   | Design embankment slope protection. |
“The minimum levee section shall have a crown width of at least 3.05 m (10 ft) and a side slope flatter than or equal to 1V on 2H, regardless of the levee height or the possibly less requirements indicated in the results of stability and seepage analyses. The required dimensions of the minimum levee section is to provide an access road for flood-fighting, maintenance, inspection and for general safety conditions.” (EM 1110-2-1913)

A 1V on 2H slope is generally accepted as the steepest slope that can easily be constructed and ensure stability of any riprap layers while a 1V on 3H slope is the steepest slope that can be conveniently traversed with conventional mowing equipment and walked on during inspections.

For sand levees, a 1V on 5H landside slope is considered flat enough to prevent damage from seepage exiting on the landside slope.

If foundations have adequate strength, steepness of levee slopes from the stability aspect is governed by the types of compaction, water content control and fill materials.

The width of the levee crown depends primarily on roadway requirements and future emergency needs. To provide access for normal maintenance operations and floodfighting operations, minimum widths of three meters are commonly used (with wider turnaround areas at specified intervals) which are the minimum feasible for construction using modern heavy earthmoving equipment.

Deterministic analysis using physical properties of the foundation and embankment materials should be used to set the final levee grade to account for settlement, shrinkage, cracking, geologic subsidence and construction tolerances.

The principal methods used to analyse levee embankments for stability against shear failure assume either a sliding surface having the shape of a circular arc within the foundation and/or the embankment or a composite failure surface composed of a long horizontal plane in a relatively weak foundation or thin foundation stratum connecting with diagonal plane surfaces up through the foundation and embankment to the ground surface.

Various methods of analysis are described in § 2.3.

Nowadays, the effort of making such analyses is greatly reduced because computer programs are available hence primary attention can be devoted to the definition of the shear strengths, unit weights, permeability and limits of possible sliding surfaces.

The various loading conditions to which a levee and its foundation may be subjected are:
- End of construction. This case represents undrained conditions for impervious embankment and foundation soils;

- Sudden drawdown from full flood stage. At least the major part of the upstream embankment portion is saturated by a prolonged flood stage that falls faster than the soil can drain. The development of excess pore water pressure may result in the upstream slope instability.

- Steady seepage from full flood stage, fully developed phreatic surface. It occurs when the water remains at or near full flood stage long enough so that the embankment becomes fully saturated and a condition of steady seepage occurs. This condition may be critical for landside slope stability.

- Earthquake loadings. Levees constructed of loose cohesionless materials or founded on loose cohesionless materials are particularly susceptible to failure due to liquefaction during earthquakes. However, earthquake loadings are not normally considered in analysing the stability of levees because of the low probability of earthquake coinciding with periods of high water (depending on the severity of the expected earthquake and the importance of the levee, seismic analyses to determine liquefaction susceptibility may be required).

Generally, evaluation and mitigation for seismic performance of river embankment systems has had low priority in the past. More currently, it is thought that intermittently loaded levees\(^1\) should be evaluated for seismic performance.

\(^1\) An **intermittently loaded levee** is a levee that does not experience a water surface elevation \(\geq 1\) foot above the elevation of its toe at least once a day for more than 36 days per year on average. (Department of Water Resources, 2012)
### Table 6.1a
Summary of Design Conditions

<table>
<thead>
<tr>
<th>Analysis Condition</th>
<th>Shear Strength</th>
<th>Pore Water Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>During and End-of-Construction</strong></td>
<td>Free draining soils - use effective stresses</td>
<td>Free draining soils - Pore water pressures can be estimated using analytical techniques such as hydrostatic pressure computations for no flow or steady seepage analysis techniques (flow nets, finite element analyses or finite difference analyses).</td>
</tr>
<tr>
<td></td>
<td>Low permeability soils - use undrained strengths and total stresses*</td>
<td>Low permeability soils - Total stresses are used; pore water pressures are set to zero in the slope stability computations.</td>
</tr>
<tr>
<td><strong>Steady State Seepage Conditions</strong></td>
<td>Use effective stresses. Residual strengths should be used where previous shear deformation or sliding has occurred.</td>
<td>Estimated from field measurements of pore water pressures, hydrostatic pressure computations for no flow conditions, or steady seepage analysis techniques (flow nets, finite element analyses or finite difference analyses).</td>
</tr>
<tr>
<td><strong>Sudden Drawdown Conditions</strong></td>
<td>Free draining soils - use effective stresses</td>
<td>Free draining soils - First stage computations (before drawdown) - steady-state seepage pore pressures as described for steady state seepage condition. Second and third stage computations (after drawdown) - pore water pressures estimated using same techniques as for steady seepage, except with lowered water levels.</td>
</tr>
<tr>
<td></td>
<td>Low permeability soils - Three stage computations: First stage use effective stresses; second stage use undrained shear strengths and total stresses; third stage use drained strengths (effective stresses) or undrained strengths (total stresses) depending on which strength is lower - this will vary along the assumed shear surface.</td>
<td>Low permeability soils - First stage computations - steady-state seepage pore pressures as described for steady state seepage condition. Second stage computations - Total stresses are used pore water pressures are set to zero. Third stage computations - Use same pore pressures as free draining soils if drained strengths are being used; where undrained strengths are used pore water pressures are set to zero.</td>
</tr>
</tbody>
</table>

* Effective stress parameters can be obtained from consolidated-drained (CD, S) tests (either direct shear or triaxial) or consolidated-undrained (CU, R) triaxial tests on saturated specimens with pore water pressure measurements. Direct shear or Bromhead nong shear tests should be used to measure residual strengths. Undrained strengths can be obtained from unconsolidated-undrained (UU, Q) tests. Undrained shear strengths can also be estimated using consolidated-undrained (CU, R) tests on specimens consolidated to appropriate stress conditions representative of field conditions; however, the “R” or “total stress” envelope and associated c and φ, from CU, R tests should not be used.

* For saturated soils use σ = 0; total stress envelope with σ = 0 is only applicable to partially saturated soils.

**Figure 2. Shear strength parameters suggested by (EM 1110-2-1913).**
1. Geotechnical Investigations

The first step in a levee design procedure consists in the design of the geotechnical campaign. Investigations are usually conducted in two stages: a preliminary stage and a design stage.

In the preliminary stage, they are not extensive since their purpose is to provide general information for project feasibility studies so they usually consist of a general geological reconnaissance with limited subsurface exploration and simple soil tests. Conversely, in the design stage, more comprehensive exploration is necessary and the extent of the field investigation depends on several factors summed up by Table 2–1 from (EM 1110-2-1913) (Figure 3).

<table>
<thead>
<tr>
<th>Factor</th>
<th>Field Investigations and Design Studies Should be more Extensive Where:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Previous experience</td>
<td>There is little or no previous experience in the area particularly with respect to levee performance</td>
</tr>
<tr>
<td>Consequences of failure</td>
<td>Consequences of failure involving life and property are great (urban areas for instance)</td>
</tr>
<tr>
<td>Levee height</td>
<td>Levee heights exceed 3 m (10 ft)</td>
</tr>
<tr>
<td>Foundation conditions</td>
<td>Foundation soils are weak and compressible</td>
</tr>
<tr>
<td></td>
<td>Foundation soils are highly variable along the alignment</td>
</tr>
<tr>
<td></td>
<td>Potential underseepage problems are severe</td>
</tr>
<tr>
<td></td>
<td>Foundation sands may be liquefaction susceptible</td>
</tr>
<tr>
<td>Duration of high water</td>
<td>High water levels against the levee exist over relatively long periods</td>
</tr>
<tr>
<td>Borrow materials</td>
<td>Available borrow is of low quality, water contents are high, or borrow materials are variable along the alignment</td>
</tr>
<tr>
<td>Structure in levees</td>
<td>Reaches of levees are adjacent to concrete structures</td>
</tr>
</tbody>
</table>

Figure 3. Table 2-1 from (EM 1110-2-1913).

The geological study usually consists of an office review of all available geological information on the area of interest and a field survey. Most levees are located in alluvial floodplains. Floodplain deposit characteristics may vary widely and are directly related to changes in the depositional environment of the river and its tributaries so the nature and distribution of sediments can be determined through a study of the pattern of local river changes.
The office study consists of a search of available information such as topographic, soil and geological maps; aerial photographs and information on existing construction in the area; available boring logs. Only after becoming familiar with the area through this study the field survey starts. Generally, the subsurface exploration for the design stage is accomplished in two phases. The first one has the main purpose of better defining the geology of the area, the soil types present and to develop general ideas of soil strengths and permeability; while, the second one, provides additional information on soil types and includes the taking of undisturbed samples for testing purposes. The first phase consists almost entirely of disturbed sample borings and test pits and may include geophysical surveys while the second phase exploration consists of both disturbed and undisturbed sample borings and also may include geophysical methods. The spacing of borings and test pits in the first phase is based both on available geological information determined in the preliminary stage, experience in the area and by the nature of the project. Obviously, the spacing of borings should not be arbitrarily uniform but rather should be closer spaced in expected problem areas and at least one boring should be located at every major structure. In the second phase, additional general sample borings are located based on the first phase results and undisturbed sample borings are located where data on soil shear strength are most needed. The (EM 1110-2-1913) recommended procedure is to group the foundation profiles into reaches of similar conditions and locate undisturbed sample borings so as to define soil properties in that critical reaches.

“Depth of borings along the alignment should be at least equal to the height of proposed levee at its highest point but not less than 3 m (nominally 10 ft). Boring depths should always be deep enough to provide data for stability analyses of the levee and foundation. (...) In borrow areas, the depth of exploration should extend several feet below the practicable or allowable borrow depth or to the groundwater table. If borrow is to be obtained from below the groundwater table by dredging or other means, borings should be at least 3 m (nominally 10 ft) below the bottom of the proposed excavation.” (EM 1110-2-1913).
Figure 4. Table 2-2 from (EM 1110-2-1913).

<table>
<thead>
<tr>
<th>Table 2-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stages of Field Investigations</td>
</tr>
<tr>
<td>1. Investigation or analysis produced by field reconnaissance and discussion with knowledgeable people is adequate for design where:</td>
</tr>
<tr>
<td>a. Levees are 3 m (10 ft) or less in height.</td>
</tr>
<tr>
<td>b. Experience has shown foundations to be stable and presenting no underseepage problems.</td>
</tr>
<tr>
<td>Use standard levee section developed through experience.</td>
</tr>
<tr>
<td>2. Preliminary geological investigation:</td>
</tr>
<tr>
<td>a. Office study: Collection and study of</td>
</tr>
<tr>
<td>(1) Topographic, soil, and geological maps.</td>
</tr>
<tr>
<td>(2) Aerial photographs.</td>
</tr>
<tr>
<td>(3) Boring logs and well data.</td>
</tr>
<tr>
<td>(4) Information on existing engineering projects.</td>
</tr>
<tr>
<td>b. Field survey: Observations and geology of area, documented by written notes and photographs, including such features as:</td>
</tr>
<tr>
<td>(1) Riverbank slopes, rock outcrops, earth and rock cuts or fills.</td>
</tr>
<tr>
<td>(2) Surface materials.</td>
</tr>
<tr>
<td>(3) Poorly drained areas.</td>
</tr>
<tr>
<td>(4) Evidence of instability of foundations and slopes.</td>
</tr>
<tr>
<td>(5) Emerging seepage.</td>
</tr>
<tr>
<td>(6) Natural and man-made physiographic features.</td>
</tr>
<tr>
<td>3. Subsurface exploration and field testing and more detailed geologic study: Required for all cases except those in 1 above. Use to decide the need for and scope of subsurface exploration and field testing:</td>
</tr>
<tr>
<td>a. Preliminary phase:</td>
</tr>
<tr>
<td>(1) Widely but not necessarily uniformly spaced disturbed sample borings (may include split-spoon penetration tests).</td>
</tr>
<tr>
<td>(2) Test pits excavated by backhoes, dozers, or farm tractors.</td>
</tr>
<tr>
<td>(3) Geophysical surveys (e.g., seismic or electrical resistivity) or cone penetrometer test to interpolate between widely spaced borings.</td>
</tr>
<tr>
<td>(4) Borehole geophysical tests.</td>
</tr>
<tr>
<td>b. Final phase:</td>
</tr>
<tr>
<td>(1) Additional disturbed sample borings.</td>
</tr>
<tr>
<td>(2) Undisturbed sample borings.</td>
</tr>
<tr>
<td>(3) Field vane shear tests for special purposes.</td>
</tr>
<tr>
<td>(4) Field pumping tests (primarily in vicinity of structures).</td>
</tr>
<tr>
<td>(5) Water table observations (using piezometers) in foundations and borrow areas.</td>
</tr>
</tbody>
</table>

Figure 5. Table 2-4 from (EM 1110-2-1913).

<table>
<thead>
<tr>
<th>Table 2-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase I Boring and Sampling Techniques</td>
</tr>
<tr>
<td>Technique</td>
</tr>
<tr>
<td>1. Disturbed sample borings</td>
</tr>
<tr>
<td>a. Split-spoon or standard penetration test</td>
</tr>
<tr>
<td>b. Auger borings</td>
</tr>
<tr>
<td>2. Test pits</td>
</tr>
<tr>
<td>3. Trenches</td>
</tr>
</tbody>
</table>

Geophysical Exploration

“Geophysical explorations are a fairly inexpensive means of exploration and are very useful for correlating information between
borings which, for reasons of economy, are spaced at fairly wide intervals. Geophysical data must be interpreted in conjunction with borings and by qualified, experienced personnel." (EM 1110-2-1913)

Geophysical methods can be subdivided into two classes: those accomplished from the ground surface and those which are accomplished from subsurface borings. Geophysical ground surface exploration methods include:

- seismic methods. They locate interfaces between zones of different velocities. In order to be effective they require contrast in wave transmission velocities and that any underlying stratum transmits waves at a higher velocity than the overlying stratum.
- electrical resistivity surveys. They are used to locate and define zones of different electrical properties such as pervious and impervious zones or zones of low resistivity (clayey strata). In order to be effective they require a resistivity contrast between materials.
- natural potential (SP) methods. They are based on change of potential of ground by human action or alteration of original condition and are particularly suitable for detection of anomalous seepage.
- electromagnetic (EM) induction methods. They are particularly suitable for horizontal profiling and for detecting anomalous conditions along the centerline of levees. Their interpretation is relatively simple.
- ground penetrating radar.

Downhole geophysical logging are useful to correlate subsurface soil and rock stratification and to provide quantitative engineering parameters (porosity, density, water content, moduli) and valuable data for interpreting surface geophysical data.
GROUNDWATER FLUCTUATIONS

Piezometers to observe groundwater fluctuations should be installed in areas of potential underseepage problems and permeability tests should always be made after installation of the piezometers to provide information on foundation permeability and to check piezometers operation.

Even if pervious foundation materials permeability can often be estimated with sufficient accuracy by using existing correlations with grain-size determination (see TM 5-818-5), field pumping tests are the most accurate means of determining permeability of stratified in situ deposits (EM 1110-2-1913). Unfortunately, field pumping tests are expensive so their use must be justified.

To estimate the permeability of pervious foundation deposits, field pumping tests or correlations between a grain-size parameter and the coefficient of permeability are generally utilized. Conversely, laboratory permeability tests are rarely performed on foundation sands because of difficulty and expense in obtaining undisturbed samples.

LABORATORY TESTING

Laboratory testing programs vary depending on the nature and importance of the project and on the foundation conditions. They generally consist of water content and
identification tests on most samples while shear, consolidation and compaction tests only on representative samples since they are expensive and time-consuming.

After visual classification and water content determinations, samples of fine-grained soils are selected for Atterberg limits tests, and samples of coarse-grained soils for gradation tests. Laboratory classifications are used in preparing graphic representations of boring logs.

Comparisons of Atterberg limits values with natural water contents of foundation soils and use of the plasticity chart itself together with split-spoon driving resistance, geological studies and previous experience often will indicate potentially weak and compressible fine-grained foundation strata and thus the need for shear and perhaps consolidation tests. Correlations between Atterberg limits values and consolidation or shear strength characteristics based on local soil types and which distinguish between normally and overconsolidated conditions are preferable to evaluate these characteristics. Such correlations may also be used to reduce the number of tests required for design of higher levees. As optimum water content may in some cases be correlated with Atterberg limits, comparisons of Atterberg limits and natural water contents of borrow soils can indicate whether the borrow materials are suitable for obtaining adequate compaction (EM 1110-2-1913).

When coarse-grained soils contain few fines, the consolidated drained shear strength is appropriate for use in all types of analyses. In most cases, conservative values of the angle of internal friction (φ) can be assumed from correlations and no shear tests will be needed.

Maximum density tests on available pervious borrow materials should be performed in accordance with (ASTM D 4253–00) so that relative compaction requirements for pervious fills may be checked in the field when required by the specification. The other methods used include comparison of in-place density to either the maximum Proctor density or the maximum density obtained by (ASTM D 4253–00) (if vibratory table is available).
### Table 3-1

**Laboratory Testing of Fine-Grained Cohesive Soils**

<table>
<thead>
<tr>
<th>Test</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Visual classification and water content determinations</td>
<td>On all samples</td>
</tr>
<tr>
<td>Atterberg limits</td>
<td>On representative samples of foundation deposits for correlation with shear or consolidation parameters, and borrow soils for comparison with natural water contents, or correlations with optimum water content and maximum densities</td>
</tr>
<tr>
<td>Permeability</td>
<td>Not required; soils can be assumed to be essentially impervious in seepage analyses</td>
</tr>
<tr>
<td>Consolidation</td>
<td>Generally performed on undisturbed foundation samples only where:</td>
</tr>
<tr>
<td></td>
<td>a. Foundation clays are highly compressible</td>
</tr>
<tr>
<td></td>
<td>b. Foundations under high levees are somewhat compressible</td>
</tr>
<tr>
<td></td>
<td>c. Settlement of structures within levee systems must be accurately estimated</td>
</tr>
<tr>
<td>Compaction</td>
<td>Not generally performed on levee fill; instead use allowances for settlement within levees based on type of compaction. Sometimes satisfactory correlations of Atterberg limits with coefficient of consolidation can be used. Compression index can usually be estimated from water content.</td>
</tr>
<tr>
<td>Shear strength</td>
<td>a. Required only for compacted or semi-compact ed levees</td>
</tr>
<tr>
<td></td>
<td>b. Where embankment is to be fully compacted, perform standard 25-blow compaction tests</td>
</tr>
<tr>
<td></td>
<td>c. Where embankment is to be semi-compact, perform 15-blow compaction tests</td>
</tr>
<tr>
<td></td>
<td>a. Unconfined compression tests on saturated foundation clays without joints or seepages</td>
</tr>
<tr>
<td></td>
<td>b. Q triaxial tests appropriate for foundation clays, as undrained strength generally governs stability</td>
</tr>
<tr>
<td></td>
<td>c. R triaxial and S direct shear. Generally required only when levees are high and/or foundations are weak, or at locations where structures exist in levees</td>
</tr>
<tr>
<td></td>
<td>d. Q, R, and S tests on fill materials compacted at appropriate water contents to determine from the expected field compaction effort</td>
</tr>
</tbody>
</table>

**Figure 7. Table 3-1 from (EM 1110-2-1913).**

### Table 3-2

**Laboratory Testing of Pervious Materials**

<table>
<thead>
<tr>
<th>Test</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Visual classification</td>
<td>Of all jar samples</td>
</tr>
<tr>
<td>In situ density determinations</td>
<td>Of Shelby-tube samples of foundation sands where liquefaction susceptibility must be evaluated</td>
</tr>
<tr>
<td>Relative density</td>
<td>Maximum and minimum density tests should be performed in seismically active areas to determine in situ relative densities of foundation sands and to establish density control of sand fills</td>
</tr>
<tr>
<td>Gradation</td>
<td>On representative foundation sands:</td>
</tr>
<tr>
<td></td>
<td>a. For correlating grain-size parameters with permeability or shear strength</td>
</tr>
<tr>
<td></td>
<td>b. For size and distribution classifications pertinent to liquefaction potential</td>
</tr>
<tr>
<td>Permeability</td>
<td>Not usually performed. Correlations of grain-size parameters with permeability or shear strength used. Where seepage problems are serious, best guidance obtained by field pumping tests</td>
</tr>
<tr>
<td>Consolidation</td>
<td>Not usually necessary as consolidation under load is insignificant and occurs rapidly</td>
</tr>
<tr>
<td>Shear strength</td>
<td>For loading conditions other than dynamic, drained shear strength is appropriate. Conservative values of $\phi$ can be assumed based on S tests on similar soils. In seismically active areas, cyclic triaxial tests may be performed</td>
</tr>
</tbody>
</table>

**Figure 8. Table 3-2 from (EM 1110-2-1913).**
1.1. **Geotechnical Campaign of the Serchio River Embankments**

The flood plain embankments of the Serchio River (Northern Tuscany, Italy) have been constructed since the end of the XVIII century and only some refurbishments have been applied after flood events hence construction details are not known. These embankments have experienced failures several times during their life, the last time in December 2009. After the last event, a detailed geotechnical investigation was carried out. Serchio River embankments have generally a height of less than 4 m and a width between 1.2 and 3 m. This has restricted the investigation tools that could be used in this peculiar case.

The necessary criteria for a cost–effective campaign, considering the total length of the embankments (30 km) and the requested level of detail are discussed herein. The campaign, in addition to laboratory tests, included boreholes, CPTu, permeability tests, 2D geo-electric tomography and 15 boreholes performed by the use of a continuous core drilling system (Principe et al. (1997)). This last tool, as confirmed by CPTu results, has proved to be very useful to obtain the more accurate evaluation of the in situ soil density. The capability of indirect methods (CPTu, 2D geo–electric tomography) to infer the soil stratigraphy has been analysed. CPTu test, economical and expeditious, has proved to be an indispensable tool to delineate soil stratigraphy, if the results are correctly calibrated against borehole logs. In fact, their results combined with the borehole logs and the laboratory tests, provide extensive information. Otherwise, geo-electric investigations can be very useful to highlight anomalies and heterogeneities in the cross section.

1.1.1. **Geographic Overview**

The Serchio River originates in Garfagnana area, in northwest Tuscany, on the slopes of the Monte Sillano from two main confluents: the Serchio di Soraggio (which has an Apennine origin) and the Serchio di Gramolazzo (which originates from the Apuan Alps). Before reaching Lucca plain, it joins with its most important tributary: the Lima creek. In the Lucca plain, it joins with the Freddana creek and then it flows in the Pisa area and reaches the Parco di San Rossore area (a few kilometers to the north of Pisa), where it flows into the Tyrrhenian Sea.
Serchio basin occupies an area of about 1565 km², includes 36 Municipalities and a population of about 27000 people. The geographic position of the Serchio basin: the presence of two mountain ranges (the Apennines and the Apuan Alps) extending in parallel along the coast, hence directly exposed to the Atlantic weather front, causes the ideal conditions for plentiful rainfalls.

In fact, the Serchio basin has an annual rainfall of 1946 mm. Therefore, even if the Serchio river is the third longest river (126 m) of Tuscany (after Arno and Ombrone), by annual average discharge (46 m³/s) it is the main river of Tuscany (Autorità di Bacino del Fiume Serchio, 2010).

1.1.2. BRIEF HISTORY OF SERCHIO RIVER EMBANKMENTS

The first examples of embankment system for the Serchio River date back to Roman age. After the fall of the Roman Empire, the embankments were completely neglected and for ages the river flowed freely in the Lucca plain. Between the XI century and the XVIII century some river defense works were realized.

Eventually, in 1761, Giovanni Attilio Arnolfini was charged with dealing the river flood protection. After his death, in 1818 Lorenzo Nottolini continued Arnolfini’s work and the present levee system was realized (Autorità di Bacino del Fiume Serchio, 2010).

In fact, the levee system has not experienced important changes, only some refurbishments have been applied after the main flood events.

The embankment sections are characterized by maximum height of about 6 meters and high face slope, about 1/1.5 on average.

The main flood events in the more recent past occurred:

- On November 17th, 1940 (there was a breach near Nodica (Pisa)).
- Between November 9th and November 12th, 1982;
- On June 9th, 1992 (there was overtopping);
- Between November 6th and November 7th, 2000 (there was a breach near Nozzano (Lucca));
1.1.3. Weather events in December 2009

On December 18th and 19th 2009, the Serchio basin was affected by widespread snowfall and frigid temperatures. The day after, the temperatures suddenly rose (from -7/-12°C to +5/+10°C) and the snow melted away. On December 21st and 23rd, as wet milder southern air established across the basin and battled with the existing cold air, a period of plentiful rainfall affected the basin. From December 22nd until 23rd, strong winds affected the coast so the sea level rose causing the water to pile up higher than the ordinary sea level (until +0.8 meters above average sea level). That weather events combined to bring about a flood event characterized by two flood waves, one after the other, both with a high flow rate. The two flood waves registered their peaks on December 23rd and 25th. So the second one happened when the first one was going down but was still present and caused the failures occurred on the night of December 25th.

Therefore, the embankment breaches happened in the presence of wet soil, maybe saturated and in a condition of river water flow to sea hindered (Autorità di Bacino del fiume Serchio, 2010(a)).

<table>
<thead>
<tr>
<th>FLOOD EVENT</th>
<th>MAXIMUM RIVER DISCHARGE m³/s</th>
<th>EMBANKMENT FAILURE</th>
</tr>
</thead>
<tbody>
<tr>
<td>November 9th 1982</td>
<td>2000</td>
<td>NOT</td>
</tr>
<tr>
<td>November 1st 2000</td>
<td>1580</td>
<td>NOT</td>
</tr>
<tr>
<td>November 9th 2000</td>
<td>1580</td>
<td>YES (*)</td>
</tr>
<tr>
<td>December 5th 2008</td>
<td>1025</td>
<td>NOT</td>
</tr>
<tr>
<td>December 10th 2009</td>
<td>1200</td>
<td>NOT</td>
</tr>
<tr>
<td>December 25th 2009</td>
<td>1900</td>
<td>YES (*)</td>
</tr>
</tbody>
</table>

(*) Repeated floods in 10–15 days during 2000 and 2009 events produced embankment failures in the same areas.

Table 1. Maximum discharge of the Serchio River for various past flood events.
FIGURE 9. AVERAGE HOURLY RAINFALL HEIGHT DATA FROM DECEMBER 21 TO DECEMBER 31, 2009 (MODIFIED FROM (AUTORITÀ DI BACINO DEL Fiume Serchio, 2010(a))): IT IS POSSIBLE TO DISTINGUISH THE TWO EVENTS. THE AVAILABLE DATA SHOW THAT IN THE SERCHIO RIVER BASIN WERE REGISTERED RAINFALLS GREATER THAN A 30-YEAR STORM (FOR 12 HOUR DURATIONS).

FIGURE 10. RIVER HEIGHT DATA BETWEEN DECEMBER 21 AND DECEMBER 26, 2009 (AUTORITÀ DI BACINO DEL Fiume Serchio, 2010(a)).
Figure 11. Annual maximum discharge of the Serchio River embankment: time series since 1923 (Autorità di Bacino del fiume Serchio, 2010(a)).

Figure 12. Serchio hydrogram (from December 21, 2009 to January 7, 2010 - section: Monte San Quirico (Lucca)). It is possible to see the two flood waves (Autorità di Bacino del fiume Serchio, 2010(a)).
1.1.4. Flood event in December 2009

Serchio River embankments have experienced failures several times during their life, the last time in December 2009. In fact, during the night of December 25\textsuperscript{th}, after the concurrence of various adverse factors such as the melting of the snow because of a sudden temperature increase and the contemporary long raining period, three failures occurred. Two failures occurred in the District of Lucca near the town of Santa Maria al Colle and had a total length of 100 m. The two failures, occurred two hours after the flood peak transit (in the recession part of the flood), caused the leakage of about 1000000 m\textsuperscript{3} of water (Autorità di Bacino del fiume Serchio, 2010(a)). There was not overtopping. The third failure occurred in the District of Pisa near the urban centres of Nodica and Migliarino and had a length of about 160 m. That failure occurred in an embankment section already involved in the breaches of 1940 and 1952. Before the breach occurring, it was observed a piping phenomenon through the embankment, landside over the river bank (Figure 17).

As a consequence of these failures, large urbanized areas were flooded with a water plus mud level as high as two meters with damages to the constructions and the infrastructures. In fact, the motorway connecting the cities of Genoa and Rome was closed at Pisa for a couple of weeks because of the overtopping and the large settlement of an embankment portion (there was the risk of further instabilities). Also the State Road SS1 connecting Genoa to Rome was closed between Migliarino and Pisa for several months because of the overtopping and the settlements occurred in a large portion of the embankments.

Lucca and Pisa Districts (Service for the Defense of the Territory) and Italian Civil Service decided the immediate repair of the failures and the consolidation of 3 km of embankments close to the failure zones. Moreover, the Geotechnical Laboratory of the University of Pisa was asked both to define and control a geotechnical investigation for the characterization of the three km of embankments close to the failure zones and of the remaining 24 km of river embankments to the mouth of the Serchio River.

In addition it was asked to define a stratigraphic and geotechnical model and to carry out a number of analyses to individuate the possible failure causes, to define consolidation measures and to individuate the risk areas.
Figure 13. Failures (red lines) and flooded areas in the District of Lucca.
FIGURE 14. THE TWO FAILURES IN THE DISTRICT OF LUCCA.
FIGURE 15. Flooded area in the District of Lucca.

FIGURE 16. Failures (red lines) and flooded areas in the District of Pisa. The green points represent draining pumps.
1.1.5. GEOTECHNICAL CAMPAIGN

After the flood event in December 2009, both in the District of Lucca and in the District of Pisa, two consecutive geotechnical campaigns were carried out. The first one was carried out to identify the failure causes and to increase the safety of the embankment conditions near the failures (about three km) while, the second one, more extensive, was carried out to get a geotechnical model of the levee system and to study the levee safety conditions by using stability analyses.

Because of the long extension of the area to be investigated and the detailed information needed, the following geotechnical campaign was decided:

1. One borehole (20 m deep) every 1000 m. In the District of Lucca all the boreholes were carried out from the embankment bank because of the limited width of the crest, while in the District of Pisa the tests were carried out from the crest of the embankment.

- 4 Osterberg samples retrieved from each borehole for laboratory testing
  - Classification;
  - Triaxial CIU tests;
• 4 Lefranc tests for each borehole;
• 2 Casagrande piezometers for each borehole.

2. CPTu (20 m deep) every 200 m. All the tests were carried out from the embankment crest both in the District of Lucca and in the District of Pisa.

3. 2D Electric Resistivity Tomography (ERT) every about 200 m or less. ERTs were mainly carried out along cross sections of the embankment; only two ERTs were carried out using an electrode – alignment parallel to the river embankment.

4. 15 Continuous sampling (4 m deep) carried out every 200 m using a specially devised micro–stratigraphic sampler (Principe et al., (1997), only for the three km of embankments subjected to consolidation works.

It was decided to have one CPTu and ERT located very close to each borehole and continuous sampling were carried out very close to already performed CPTu.

Continuous sampling was carried out using a specially devised micro stratigraphic sampler: the AF shallow core system (Principe et al. (1997)), with an inner diameter of 38 mm. The tests were carried out, from the embankment crest, down to a depth of 4 m (i.e. the average embankment height), measuring the sample compaction each 50 cm.

These continuous samples have been used to get a detailed grain size distribution of the soil and to evaluate the in situ soil density. This sampling was carried out only in an embankment section near the 2009 failures.

The values of natural volume weight obtained for the main soil textures existing in the body embankment were:
- Sandy silt to silty sand → from 12.3 – 12.8 kN/m$^3$;
- Coarse sand →17.7 kN/m$^3$.

The above reported values are very low but consistent with the results of CPTu indicating relative densities of about 10% for the silty sands and sandy silts.

The values of the natural volume weight that have been obtained from few Shelby samples (retrieved in the same area) were much higher than those inferred from continuous sampling. This confirms that, for these very loose (mainly granular) soils, the only possibility of avoiding soil compaction is the use of Osterberg sampler.

CPTu were interpreted using the CPeT-IT software (Geologismiki, 2009). CPeT-IT is a software package for the interpretation of Cone Penetration Test data. CPeT-IT takes CPT raw data and performs basic interpretation in terms of Soil Behaviour Type (SBT) and various other geotechnical parameters using the current
published correlations based on the review by (Lunne, Robertson, & Powell, 1997) and (Robertson, 2008).

Figure 18 shows a typical result based on the Robertson et al. (1986) SBT classification.

CPTu results have been also used to obtain the undrained shear strength in fine grained layers and the angle of shear resistance in granular layers.

The undrained shear strength has been computed assuming a bearing capacity factor $N_{ku}=14$. The angle of shear resistance has been computed using the (Schmertmann, 1978) equations after the assessment of the relative density from the tip resistance. The relative density was determined according to the empirical approach proposed by Jamiolkowski et al. (1985).

The same values of the angle of shear resistance were obtained from triaxial laboratory testing on specimens from undisturbed Osterberg samples. Obviously, the comparison was possible only for those layers where undisturbed sampling was possible.

ERTs were carried out using 96 electrodes and a 2 Ampere current. The inter–electrodes distance was 0.5 m. A Syscal Pro at 96 channels was used as data acquisition system (So.Ge.T s.n.c., 2011). The above indicated instrumentation gave the possibility of carrying out expeditious, high precision measurements and to investigate the subsoil down to 15 m.

As for the measurements, two different schemes were used: a) Wenner scheme (four – poles) and b) pole – dipole scheme (three poles).

Data interpretation has been carried out by using the software TomoLAB® (TomoLAB, 2009) based on a FEM mesh. Test results are shown as 2D tomography in terms of resistivity (Ohm*m) using appropriate chromatic scales (Figure 19).
1.1.6. CRITERIA FOR DEFINING A STRATIGRAPHIC MODEL

Because of the type of problem to analyse, the most important parameter for soil classification is the permeability and hence the grain size. Therefore, in order to achieve a stratigraphic model the soil was classified into four groups based on laboratory grain size distributions shown in Figure 20, Figure 21, Figure 22 and Figure 23.

In particular, the percentage finer than the No. 200 sieve was used (Table 2).

The four groups identified are:
- Sand;
- Silty sand;
- Sand with clayey silt;
- Clayey sandy silt.

<table>
<thead>
<tr>
<th>SOIL GROUP</th>
<th>PERCENTAGE FINER THAN THE NO. 200 SIEVE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1: Clayey sandy silt</td>
<td>&gt; 60% (clay &gt; 10%)</td>
</tr>
<tr>
<td>2: Sand with clayey silt</td>
<td>35% ÷ 60% (clay &lt; 10%)</td>
</tr>
<tr>
<td>3: Silty sand</td>
<td>&lt; 35%</td>
</tr>
<tr>
<td>4: Sand</td>
<td>&lt; 10%</td>
</tr>
</tbody>
</table>

**Table 2. Laboratory grain size distributions: soil groups identified.**
The soil stratigraphy has been found out by combining the information from boreholes, CPTu tests and laboratory tests.

The soil stratigraphy, which has been indirectly inferred from CPTu, has been compared to that directly obtained from the corresponding borehole.

The software CPeT-IT provides the thickness of the soil layers and the soil type according to the interpretation of Robertson et al. (1986) and (Robertson, 1990).

A correspondence between SBT lithology, soil groups defined by laboratory grain size distributions and borehole–logs was established. A fifth group, consisting of gravel and coarse sand, was defined by analyzing borehole–log (Table 3). Obviously, this soil class was not found by laboratory tests because for those layers undisturbed sampling was not possible. Piezocono penetration into this layer was also very limited.
A SBT class and a soil description, as from the stratigraphic log, was associated to each group (Table 3) obtaining the stratigraphic model shown in (Figure 24, Figure 25, Figure 26 and Figure 27).

Thanks to the high number of CPTu tests carried out, it was possible to define a detailed stratigraphic model.

**FIGURE 21. GRAIN SIZE DISTRIBUTION CURVES: SILTY SAND. DISTRICT OF LUCCA (LEFT) AND DISTRICT OF PISA (RIGHT).**
Figure 22. Grain size distribution curves: sand with clayey silt. District of Lucca (left) and District of Pisa (right).

<table>
<thead>
<tr>
<th>SOIL GROUPS</th>
<th>GRAIN SIZE DISTRIBUTION CURVES</th>
<th>BOREHOLE–LOGS</th>
<th>SBT LITHOLOGY</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Clayey sandy silt</td>
<td>Clay</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Sand with clayey silt</td>
<td>Clayey silt</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Clay and silty clay</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Silty sand</td>
<td>Sandy silt</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Sand</td>
<td>Sand and silty sand</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Gravel and coarse sand</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

Table 3. Correspondence between soil groups defined by laboratory grain size distribution curves (Table 2), borehole–logs and SBT lithology. The fifth group was defined by analyzing borehole–log because for those layers undisturbed sampling was not possible.
The stratigraphic model directly inferred from boreholes shows the extensive presence of granular soils, sand and silt, even in the first layers, while, the stratigraphic model inferred from CPTu test interpretation shows in the same layers the presence of finer soils.

The percentage of success of CPTu to give the same classification as from borehole–logs has been computed and the comparison is summarized by Table 4 (SBT classes according to (Robertson, 1990)). The percentage of success has been defined as length correctly estimated out of total length.

It is possible to conclude that CPTu systematically underestimate the grain size and in most cases the soil is classified in the lower class (i.e. 3 instead of 4). However, since the error is quite systematic, it is possible to use CPTu after a correct calibration to extend the information obtained from the borehole–logs to a larger portion of investigated soil.
This circumstance gives the opportunity to have a detailed SBT description with acceptable costs.

<table>
<thead>
<tr>
<th>SBT CLASSES FROM BOREHOLE - LOGS</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>SBT CLASSES FROM CPTU</td>
<td>3</td>
<td>0%</td>
<td>36%</td>
<td>46%</td>
<td>16%</td>
<td>18%</td>
</tr>
<tr>
<td>4</td>
<td>0%</td>
<td>14%</td>
<td>30%</td>
<td>15%</td>
<td>7%</td>
<td>18%</td>
</tr>
<tr>
<td>5</td>
<td>0%</td>
<td>43%</td>
<td>19%</td>
<td>22%</td>
<td>12%</td>
<td>34%</td>
</tr>
<tr>
<td>6</td>
<td>0%</td>
<td>7%</td>
<td>4%</td>
<td>46%</td>
<td>62%</td>
<td>43%</td>
</tr>
<tr>
<td>7</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
<td>1%</td>
<td>1%</td>
<td>0%</td>
</tr>
<tr>
<td>Other</td>
<td>0%</td>
<td>0%</td>
<td>1%</td>
<td>1%</td>
<td>0%</td>
<td>0%</td>
</tr>
</tbody>
</table>

Table 4. Percentage of success of CPTu.

ERTs were not used in this process because of the intrinsic limitations of this testing method which is very sensitive to the presence of water. ERTs were mainly used to get information on the homogeneity of the cross sections.

The soil stratigraphy, that have been indirectly inferred from ERTs, has been compared to that directly obtained from the corresponding borehole in a similar way as for CPTu. In order to carry out such a comparison, the soil description (soil texture) from boreholes has been uniformed and simplified referring to the Soil Behavior Type (SBT) classes proposed by (Robertson, 1990). The assumed correspondences between SBT classes, resistivity and soil texture are given in Table 5. The percentage of success of ERTs to give the same classification as from borehole–logs has been computed according to the correspondences of Table 5. The percentage of success is computed for each SBT class as the ratio between the length of correctly identified soil layers and the total length of layers belonging to that class (Table 6 and Table 7). The columns indicate the SBT classes as from the borehole–logs, while the rows indicate the SBT classes from ERTs, so the diagonal indicates the percentage of success. The sum of the percentages along a column is 100%. The column 3 is empty because this SBT class is not found in the borehole logs.

It is possible to conclude that ERTs have a very low percentage of success for partially saturated soils and in this case the “incorrect” soil identification is quite casual. On the other hand, for saturated conditions the percentage of success
greatly increases especially for fine soils and the error becomes mainly systematic. Therefore, ERTs systematically underestimate the soil grain size.

The results obtained from the two ERTs parallel to the river embankment are not in agreement with that inferred from ERTs carried out along cross-sections. This confirms that the embankment geometry is not suitable to carry out ERTs along longitudinal sections.

<table>
<thead>
<tr>
<th>SBT CLASS (ROBERTSON 1990)</th>
<th>SOIL TEXTURE</th>
<th>RESISTIVITY [Ohm<em>m] (</em>)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Clay and silty clay</td>
<td>0 – 20</td>
</tr>
<tr>
<td>4</td>
<td>Silty clay to clayey silt</td>
<td>20 – 50</td>
</tr>
<tr>
<td>5</td>
<td>Sandy silt to silty sand</td>
<td>50 – 130</td>
</tr>
<tr>
<td>6</td>
<td>Sand</td>
<td>130 – 500</td>
</tr>
<tr>
<td>7</td>
<td>Gravel and coarse sand</td>
<td>≥ 500</td>
</tr>
</tbody>
</table>

(*) For a given class, the lower limit of the resistivity refers to partially saturated conditions.

Table 5. Correspondences between SBT classes, resistivity and soil texture.

<table>
<thead>
<tr>
<th>SBT CLASSES FROM ERTS</th>
<th>SBT CLASSES FROM BOREHOLE - LOGS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3</td>
</tr>
<tr>
<td>3</td>
<td>0%</td>
</tr>
<tr>
<td>4</td>
<td>0%</td>
</tr>
<tr>
<td>5</td>
<td>0%</td>
</tr>
<tr>
<td>6</td>
<td>0%</td>
</tr>
<tr>
<td>7</td>
<td>0%</td>
</tr>
</tbody>
</table>

Table 6. Percentage of success of ERTs for the layers above the water table.

<table>
<thead>
<tr>
<th>SBT CLASSES FROM ERTS</th>
<th>SBT CLASSES FROM BOREHOLE - LOGS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3</td>
</tr>
<tr>
<td>3</td>
<td>0%</td>
</tr>
<tr>
<td>4</td>
<td>0%</td>
</tr>
<tr>
<td>5</td>
<td>0%</td>
</tr>
<tr>
<td>6</td>
<td>0%</td>
</tr>
<tr>
<td>7</td>
<td>0%</td>
</tr>
</tbody>
</table>

Table 7. Percentage of success of ERTs for the layers below the water table.
Figure 24. Stratigraphic model: District of Lucca, right bank.
FIGURE 25. Stratigraphic model: District of Pisa, right bank (Label: blue = clayey sandy silt; yellow = sand; light green = silty sand; dark green = sand with clayey silt; brown = gravel and coarse sand.)
Figure 26. Stratigraphic model: District of Lucca, left bank (Label: blue = clayey sandy silt; yellow = sand; light green = silty sand; dark green = sand with clayey silt; brown = gravel and coarse sand.)
Figure 27. Stratigraphic model: District of Pisa, left bank (Label: blue = clayey sandy silt; yellow = sand; light green = silty sand; dark green = sand with clayey silt; brown = gravel and coarse sand.)
1.1.7. **The Geotechnical Model**

The mechanical parameters were inferred by using both the CPTu test interpretation and the triaxial laboratory testing on specimens from undisturbed Osterberg samples.

CPTu results have been used both to obtain the angle of shear resistance in granular layers and the undrained shear strength in fine grained layers. The angle of shear resistance has been computed using the (Schmertmann, 1978) equations after the assessment from the tip resistance of the relative density \( D_R \) according to the empirical approach proposed by Jamiolkowski et al. (1985).

The software CPeT-IT gives also directly the valuation of both the \( D_R \) and the angle of shear resistance.

CPeT-IT calculates the relative density by applying the following equation:

\[
D_R^2 = \frac{q_{tn}}{C_{D_R}} \tag{1}
\]

Where:

\( q_{tn} = \text{Normalized cone resistance} \)

\[
q_{tn} = \left( \frac{q_t - \sigma_{v0}}{p_a} \right) \left( \frac{p_a}{\sigma_{v0}} \right)^n \tag{2}
\]

\[
n = 0.381 \cdot I_c + 0.05 \cdot \left( \frac{\sigma_{v0}}{p_a} \right) - 0.15 \tag{3}
\]

Where:

\[
I_c = \left( (3.47 - \log Q_{t1})^2 + (\log F_r + 1.22)^2 \right)^{0.5} \tag{4}
\]

Where:

\( I_c = \text{SBT}_n \text{ Index. (SBT}_n = \text{Soil Behaviour Type normalized using q}_{tn} \text{ (Robertson, 1990))} \)

CPeT-IT calculates the \( D_R \) only for \( \text{SBT}_n \) 5, 6, 7 and 8, while, it rates \( D_R = 0 \) for \( \text{SBT}_n \) 1, 2, 3, 4 and 9.

CPeT-IT calculates the friction angle by applying the following equation:

\[
\varphi' = 17.60 + 11 \cdot \log q_t \tag{5}
\]

As per \( D_R \), the software calculates \( \varphi' \) only for \( \text{SBT}_n \) 5, 6, 7 and 8, while, it rates \( \varphi' = 0 \) for \( \text{SBT}_n \) 1, 2, 3, 4 and 9.

The values of \( D_R \) and \( \varphi' \) obtained by using the empirical approach proposed by Jamiolkowski et al. (1985) and (Schmertmann, 1978) equations and those calculated
directly by the software CPeT-IT have been compared (Figure 28). The comparison shows that the results are comparable.

The same values of the angle of shear resistance were obtained from triaxial laboratory testing on specimens from undisturbed Osterberg samples. The strength envelopes were obtained for various soil types from CIU Triaxial Compression Tests. Obviously, the comparison was possible only for those layers where undisturbed sampling was possible.

CPTu results have been also used to obtain the undrained shear strength in fine grained layers.

The undrained shear strength, $s_{uu}$, has been computed assuming a bearing capacity factor $N_{k} = 14$ by using the software CPeT-IT.
### Table 8. Average values obtained for the undrained shear strength by using the software CPET-IT.

<table>
<thead>
<tr>
<th></th>
<th>$S_u$ (kPa)</th>
<th>SD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Left levee</td>
<td>96.35</td>
<td>20.64</td>
</tr>
<tr>
<td>Right levee</td>
<td>102.09</td>
<td>26.00</td>
</tr>
<tr>
<td>Both left and right levee</td>
<td>98.12</td>
<td>23.73</td>
</tr>
</tbody>
</table>

As far as the volume weight is concerned, in the District of Lucca near the failure areas, very different values have been measured considering the Shelby samples and the continuous samples. The Shelby samples gave values of the volume weight between 19.1–19.6 kN/m$^3$ while, for the continuous samples, the volume weight ranged between 12.3–12.8 kN/m$^3$. The relative density inferred from CPT tests, carried out in the same areas, according to the Jamiolkowski et al. (1985) method, was as low as 10%. Moreover, the optimum dry volume weight of the soil under consideration as obtained from Standard Proctor test was compared against those inferred from continuous samples. The comparison shows that the low densities obtained from continuous samples and confirmed by CPT tests are coherent, on the contrary the dry volume weight inferred from Shelby samples are quite close to the optimum Proctor value. In addition triaxial compression tests (CIU) carried out on specimens from Shelby samples exhibited a clear dilatant behaviour. Therefore, the volume weight inferred from continuous sampling was considered more realistic.

Shelby samples gave very high values of the volume weight probably as a consequence of the compression of very loose cohesionless soil inside the tube sample during pushing. The areas close to the failures were firstly investigated. After that, Shelby samples were no more used being replaced with Osterberg samples.
The values of permeability measured in situ by means of Lefranc tests carried out inside boreholes were averaged for homogeneous soil layers.

The characteristic parameters for the soil groups, as obtained from laboratory and in situ testing, are summarized in Table 9. It is possible to observe that, in spite of the differences in terms of grain size distributions, both strength parameters and permeability are very similar. In fact, the strength parameters are quite low and the permeability is mainly in the range $10^{-5}$–$10^{-6}$ m/s (i.e. rather permeable soils). In addition, very low densities have been found as far as the embankment in the proximity of the December 2009 failures is considered (about 3 km). Therefore, it is possible to conclude that, both the embankment and the subsoil, have poor to very poor characteristics.

The stratigraphic and the geotechnical models were used for the stability analyses.
1.1.8. Final remarks

The described experience led to the following general considerations:

- Sampling in very loose material is a delicate operation which can lead to wrong estimation of mechanical and physical parameters;
- Use of AF sampler can be decisive to determinate physical properties of soil;
- Use of CPT test is suitable for cost–effectiveness purposes but calibration of SBT is necessary;
- Use of ERT is strongly influenced by water content. As a consequence there is a great uncertainty about soil type.

In addition, it is worthwhile to point out the costs of the carried out investigation campaigns:

- 60000 euros for the 3 km of embankments in proximity of the December 2009 failures;
- 390000 euros for another 24 km.

The above costs include those for test interpretation and geotechnical consultancy.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Lucca District</th>
<th>Lucca District - Failure areas</th>
<th>Pisa District</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\gamma_m$</td>
<td>$\phi'$ (°)</td>
<td>$c'$ (kPa)</td>
</tr>
<tr>
<td>Clayey Sandy Silt</td>
<td>18.8</td>
<td>33</td>
<td>0</td>
</tr>
<tr>
<td>Silty Clayey Sand</td>
<td>18.2</td>
<td>33</td>
<td>0</td>
</tr>
<tr>
<td>Silty Sand</td>
<td>17.7</td>
<td>33</td>
<td>0</td>
</tr>
<tr>
<td>Sand</td>
<td>19.6</td>
<td>35</td>
<td>0</td>
</tr>
<tr>
<td>Coarse Sand/Gravel</td>
<td>19.2</td>
<td>35</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 9. Soil parameters as deduced by in situ and laboratory tests.
2. River Embankments Failure Causes

Floods are the most common natural disasters. In fact, several floods occur every year in the world, in both developing and developed countries, mostly resulting from levee or dam breach, some with catastrophic outcomes, involving thousands of deaths and enormous economic losses. These disasters are growing in frequency worldwide due to a variety of human and environmental factors such as land use (urbanization and deforestation), flood plain habitation and climate changes. Often catastrophic failures are the result of the failure to devote inadequate resources to the defense of critical areas. In fact, it is not economically feasible to provide a high level of protection in such exposed locations. Therefore, a suitable land use planning is necessary.

Levees may develop a false sense of security. Conversely, even frequent inspections of levee systems cannot guarantee their safety. In fact, often levee damage is not apparent.

This work wants to highlight the importance of considering all potential failure modes and the need for realistic assessment of risk exposure as an element of flood protection policy.

According to USACE (EM 1110-2-1913), the main causes of levee failure are:

- Internal erosion;
- Overtopping. Levees are designed to provide a certain level of protection so, when larger flood events happen, a levee will overtop. If water flows over the top of a soil embankment, for even a relatively short period of time, the shear stress exerted by the flowing water can exceed the critical stress of the soil and the soil particles will begin to be removed.
- Surface erosion. Wave action caused by wind or boats can impact the riverside slope of the levee and it can fail from gradual wearing down of.
- Slides within the levee embankment or the foundation soils. This is a mechanical failure type that can be due to excess of driving stresses in comparison to the available strength. Moreover, differential settlements caused by unequal consolidation of the foundation soil can jeopardize the embankment stability.

Moreover, earth river embankments can experience ultimate limit state because of:
- Heave or hydraulic failure. Failure by heave is caused by hydraulic conditions that can induce zero effective geostatic stress in soils having a medium – high permeability;
- Accidental actions due to the clash against the embankment of boats or other floating bodies.

This chapter deals with the main failure causes for river embankments: in the first paragraph the internal erosion is dealt with, while, in the second and in the third paragraph overtopping and surface erosion and mechanical failure are respectively dealt with.

Figure 30. Levee failure causes (credit: Zina Deretsky, National Science Foundation).
2.1. **Internal Erosion**

A review of dam incidents up to 1986 by Foster et al. (1998) revealed that 48% of earth and rockfill dam failures were caused by internal erosion; unfortunately there are not available statistics for river embankments.

The term “internal erosion” is used as a generic term to describe erosion of particles by water passing through a body of soil.

The term “piping” is often used generically in the literature, but actually, it refers to a specific internal erosion mechanism.

Internal erosion occurs when a soil particle is detached from its parent material and is transported through sufficiently large voids and constrictions. Detachment and transport of soil particles require both lifting forces and drag forces strong enough to overcome the inter-granular forces and the weight of the grains. For internal erosion to continue, it calls for large enough soil constrictions to permit further movement of the particles.

(Garner & Fannin, 2010) suggested that internal erosion mechanism requires a combination of material susceptibility (internal instability), critical hydraulic load (seepage velocities and hydraulic gradients) and critical stress (Figure 31).

Internal erosion can be caused by design flaws such as internally unstable granulometry, no-compliance of filter conditions and too intense local flow; pipes through the structure; animal burrows; roots; desiccation cracks or tensile stress cracks.

Internal erosion can be very slow (several years) or extremely fast (even less than an hour).
Typical internal erosion mechanisms are:

1. **Backward erosion**. It occurs when there is detachment of the fine-grained particles and seepage exits to a free unfiltered surface. Seepage carries the particles downstream and erosion progresses gradually upstream towards the source of water.

2. **Concentrated leak erosion** occurs when there is erosion along a concentrated leak path. It may occur:
   - in a crack in the embankment exits foundation caused by differential settlement, desiccation or hydraulic fracture;
   - in a continuous permeable zone containing coarse and/or poorly compacted materials (Fell & Wan, 2005).

Concentrated leaks may be also caused by ice lenses (freeze – thaw action). The mechanism of a concentrated leak in a dam core that exits into a filter is described by Sherard (1984) (Figure 32):
   - concentrated leak develops and carries eroded particles of all sizes toward the filter;
- the fines seals the filter at the end of the leak and the sand particles get caught in the filter voids. Sealing causes the water pressure in the channel to rise and approach the reservoir level;
- high gradients are created over a short distance through the core to an unsealed filter face area nearby. The leak breaks through again due to the high pressure and by-passes the initial leak, so a new leak repeats the scenario.
This repetitive action may continue until the core dam is heavily damaged and the filter is diffusely sealed.

3. **Suffusion or internal instability.** A granular cohesionless soil can be seen as a matrix of coarse particles which are responsible of the soil resistance and fine particles that can easily move and that contribute marginally at the soil resistance.

Suffusion is the process of selective erosion of fine particles from a soil matrix composed of coarser particles under the action of a water flow. It occurs when the finer particles of a soil are eroded through the coarser fraction of that soil, leaving behind an intact soil skeleton that is formed by coarser particles and so it is more permeable (Fell & Fry, 2007).

Soils which are susceptible to suffusion are internally unstable. For suffusion to occur, high enough seepage gradient and an internally unstable grading are required (Fell & Fry, 2007). Coarse widely graded or gap graded soils are susceptible to suffusion. In soils which self filter, the coarse particles prevent the internal erosion of the fine particles. Soils which have experienced a lack of self filtering are susceptible to suffusion.

“Gap or widely graded soils with particles from silt or clay to gravel size, whose particle size distribution (PSD) curves are concave upward, are prone to suffusion. Its occurring within a dike or its foundation will result in a coarser soil structure, leading to higher permeability and seepage and probable settlement of the dike. This process leads to a higher likelihood of downstream slope instability which may result in failure of the whole construction”. (Salehi Sadaghiani & Witt, 2012)

Suffusion causes:
- the soil modification (situation not foreseen by the designer);
- the increase of porosity (that can cause locally concentrated flow and decrease in mechanical strength of the soil);
- the clogging of filters and drains (that can cause the local elevation of the pore pressure and, at a later stage, the instability).

If there is not enough fine content to fill the pores of the soil skeleton then there is a higher likelihood of internal instability. If the amount of cohesive fines is higher than the volume of skeleton pores then the soil has a fine matrix and in this case fine particles are under stress and they can be classified as internally stable soils (Salehi Sadaghiani & Witt, 2012).

4. **Contact erosion or “external suffusion”**. Contact erosion can occur at the interface between two layers of different grading submitted to a groundwater flow. It is frequent:
   - In alluvial foundations;
   - At the interface embankment – foundation;
   - At the contact between two successive layers of silt and gravel.

---

**Figure 32. Concentrated leak in a dam core that exits into a filter** (Sherard, Dunnigan, & Talbot, 1984)

**Piping.** Internal erosion by piping develops to a continuous pipe through the embankment by way of backward erosion or concentrated leak erosion. (Ronnqvist, 2010). Piping occurs if seepage continues beyond the onset of backward erosion (Garner & Fannin, 2010). It occurs when soil erosion begins at a seepage exit point and erodes backwards, supporting a pipe along the way. Development of piping needs:
   - A concentrated source of water of sufficient quantity and velocity to cause detachment;
- An unprotected seepage exit point;
- Erodible material in the flow path;
- Material capable of supporting a pipe or a roof (Von Thun, 1996).

**Progressive erosion** occurs when the soil is not capable of sustaining a pipe or a roof. While soil particles are eroded, a temporary void grows until a pipe can no longer be supported and the void collapses and progressively the downstream slope is over-steepled to the point of instability.

Sometimes, wrongly, others mechanisms are included between initiation mode of internal erosion. For example:

**Heave or “blowout” or “liquefaction”**. It can occur where an impervious layer overlies more pervious material near the downstream toe of an embankment. A buildup of pressure beneath the impervious layer can lead to high uplift forces which can move material from and breach of the impervious layer.

Heave occurs in cohesionless soils when pore pressure equals total stress. It may often be followed by backward erosion if the seepage gradient remains high at the surface. In cohesive soils this condition is known as hydraulic fracture (Fell & Wan, 2005).

If heave occurs, it is highly likely backward erosion will initiate for soils with low uniformity coefficients. In fact, to occur, backward erosion requires a seepage exit gradient that is high enough to mobilize soil particles, which normally is less than the critical gradient to cause heave.

**Scour**. It occurs when tractive seepage forces along a surface are able to erode soil particles. Scour is a process where water flowing in open joints or coarse soils in contact with the embankment soil may erode the soil (Fell & Wan, 2005).

Five basic failure paths related to piping and other types of seepage–related erosion could be considered:
- through the foundation, which includes considerations for heave;
- through the embankment;
- from the embankment into the foundation;
- into or along conduits through the embankment or soil foundation;
- into drains.

The generally accepted phases of internal erosion are:

I. Initiation of erosion;
II. Continuation of erosion;
III. Progression to form a pipe;
IV. Formation of a breach.

I. Initiation of internal erosion
Backward erosion occurs whether the seepage exit gradients is high enough to mobilize soil particles. This gradient is often lesser than the critical gradient required to cause heave so, if heave occurs, it is highly likely, especially for soils with low uniformity coefficients, that backward erosion will initiate.

Suffusion occurs whether the soil is internally unstable and the seepage gradient is high enough to move the fine particles within the coarse soil matrix. The hydraulic gradient is usually less than both those required to cause backward erosion and heave.

Erosion in a concentrate leak occurs whether the hydraulic shear stresses along the walls of the concentrated leak exceed shear stress of the soil. Most soils are erodible under the hydraulic shear stresses likely to exist in a crack through an embankment (Fell & Wan, 2005).

II. Continuation of internal erosion
It depends on whether the exit point of the seepage is free and unfiltered or it is filtered (Figure 33). In fact, if the filters satisfy modern filter criteria the internal erosion process will almost certainly not continue, while, if the filter particle size distribution is coarser than required for the “no – erosion” filters design, erosion may continue (Fell & Wan, 2005).

III. Progression to form a pipe
Backward erosion progresses forming a pipe whether:
- downstream exit point continue to exceed the critical required to move particles;
- flow velocity in the pipe which is forming is high enough to enlarge the pipe;
- pipe which is developing holds a roof. Cohesive soils and silty sands / sandy silts with > 15% fines passing 0.0075 mm are likely to hold a roof (Foster M. A., 1999).

In a soil which has been subject to suffusion, for a pipe to develop, the seepage flow gradient would have to exceed the critical gradient for backward erosion of the matrix of coarser particles. In that case, the process develops as a backward erosion process.

Whether internal erosion initiated by concentrated leak progresses forming a pipe depends on:
- the hydraulic gradient;
- the geometry of the eroding hole;
- the erodibility of the soil as measured by erosion rate index (Wan & Fell, 2002).

The erosion rate index is best determined from hole erosion tests but, when erosion test results are not available, it can be related to other soil properties (Fell & Wan, 2005).

IV. Formation of a breach

Internal erosion may form a breach by gross enlarge of a pipe, by local collapse of the pipe leading to the formation of a sinkhole up to the crest of the embankment (loss of freeboard), by collapse of the pipe leading to crest settlement and overtopping of the embankment or wetting–up of the downstream slope (slope instability) or, if a pipe exit develops at the downstream toe of the embankment, by unraveling of the downstream slope.

<table>
<thead>
<tr>
<th>INTERNAL EROSION THROUGH THE FOUNDATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cause</td>
</tr>
<tr>
<td>-------</td>
</tr>
<tr>
<td>BACKWARD EROSION</td>
</tr>
<tr>
<td>SUFFUSION</td>
</tr>
<tr>
<td>CONCENTRATED LEAK</td>
</tr>
</tbody>
</table>

Table 10. Initiation of internal erosion. Erosion through the foundation.

<table>
<thead>
<tr>
<th>INTERNAL EROSION FROM EMBANKMENT TO FOUNDATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cause</td>
</tr>
<tr>
<td>-------</td>
</tr>
<tr>
<td>BACKWARD EROSION</td>
</tr>
<tr>
<td>SUFFUSION</td>
</tr>
<tr>
<td>CONCENTRATED LEAK</td>
</tr>
</tbody>
</table>

Table 11. Initiation of internal erosion. Erosion from embankment to foundation.
GUIDELINES FOR THE GEOTECHNICAL DESIGN, UPGRADING AND REHABILITATION OF RIVER EMBANKMENTS

BARBARA COSANTI

Figure 33. Models for the development of failure by piping in the foundation and from embankment to foundation (Foster & Fell, 1999).

Figure 34. Examples of filtered and free exit points for internal through the foundation (Foster & Fell, 1999).

Figure 35. Suffusion (Fell & Fry, 2007)
2.1.1. Assessment of the Probability of Initiation of Internal Erosion

### TABLE 12. ASSESSMENT OF THE PROBABILITY OF INITIATION OF INTERNAL EROSION: DATA GATHERING

<table>
<thead>
<tr>
<th>Analysis for Assessment of the Probability of Internal Erosion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plans and reports describing the foundation geology.</td>
</tr>
<tr>
<td>Details of foundation preparation and embankment zoning.</td>
</tr>
<tr>
<td>Particle size distributions, Atterberg limits, dispersion tests (both for the embankment and the foundation).</td>
</tr>
<tr>
<td>Achieved density ratio, placement water content, in-situ relative density for cohesionless foundation soils.</td>
</tr>
<tr>
<td>Analysis of filter and transition zones for internal erosion ⇒ comparison with conventional no-erosion filter criteria.</td>
</tr>
<tr>
<td>Collection of photographs taken during construction (to make sure the constructions methods and quality control).</td>
</tr>
<tr>
<td>Collection of pore pressure and settlement data, analysis of seepage (to check any abnormalities).</td>
</tr>
</tbody>
</table>

2.1.1.1. Initiation by Backward Erosion

If heave occurs, backward erosion is likely to initiate. In fact, backward erosion may initiate with average vertical gradients less than the critical gradient e.g. 0.30–0.61 (Schmertmann, 2000).

Therefore, it is useful to gain an appreciate of the conditions where heave may occur. This was investigated by (Maniam, 2004). Factors which influence the critical average gradient include the particle size and the foundation strata.

Laboratory test data are based on defining critical gradients which lead to initiation of backward erosion and progression to form a complete pipe.

Many authors have proposed a relation for the critical head. Below the critical head, if the hydraulic head increases, some erosion is observed but a new equilibrium is reached and then erosion stops, conversely, beyond the critical head no equilibrium can be reached and progressive erosion occurs.

Two old empirical rules are the Bligh rule (Bligh, 1910) and the Lane rule (Lane, 1935).
HEAVE

<table>
<thead>
<tr>
<th>VARIABLE</th>
<th>EFFECT / INFLUENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>ANISOTROPY OF PERMEABILITY (KH/KV)</td>
<td>VERY IMPORTANT. THE FOUNDATION IS CRITICAL.</td>
</tr>
<tr>
<td>DEPTH OF THE FOUNDATION SOIL</td>
<td>SOME EFFECT. DEEPER SOILS GIVE WORSE CONDITIONS.</td>
</tr>
<tr>
<td>EMBANKMENT SLOPES</td>
<td>NOT VERY IMPORTANT. (THE FACTOR OF SAFETY ARE ONLY ABOUT 10% LOWER FOR THE EMBANKMENT WITH THE STEEPER SIDE SLOPES, EVEN THOUGH THE OVERALL AVERAGE SEEPAGE GRADIENT IS INCREASED BY 50% (Maniam, 2004).</td>
</tr>
<tr>
<td>PRESENCE OF A CLAY CONFining LAYER</td>
<td>IT GREATLY INCREASES THE LIKELIHOOD OF HEAVE BECAUSE THE SEEPAGE FLOW IS RESTRICTED FROM FLOWING TO THE SURFACE BY THE CLAY LAYER</td>
</tr>
<tr>
<td>CRACKING OF THE CLAY LAYER AT THE TOE OF THE EMBANKMENT</td>
<td>ONCE THE SOIL IS CRACKED, THE FACTOR OF SAFETY AGAINST HEAVE INCREASES VERY SIGNIFICANTLY.</td>
</tr>
</tbody>
</table>

Table 13. Factors which influence heave.

Bligh’s study (1910) is considered to be pioneering for stimulating interest in the phenomenon. He proposed an empirical rule for preventing piping, based on a large number of failures due to piping. Bligh’s model assumes a linear relationship between critical head and seepage length, characterized by a coefficient:

\[
\frac{L}{H_{crit}} = c_B
\]  

(6)

Where:

- \(H_{crit}\) = critical head
- \(c_B\) = coefficient that depends on the type of material and is given for different types of materials in Table 14.
- \(L = \) horizontal seepage length (base length of the embankment) [m]

(Lane, 1935) developed the following empirical rule to safeguard structures against piping:

\[
\frac{\left(\frac{L_H}{3} + L_V\right)}{H_{crit}} = c_L
\]  

(7)
Where:
\( c_L = \) coefficient
\( L_H = \) horizontal seepage length (base length of the embankment) [m]
\( L_V = \) vertical seepage length

<table>
<thead>
<tr>
<th>TYPE OF SOIL</th>
<th>( c_B )</th>
<th>( c_L )</th>
</tr>
</thead>
<tbody>
<tr>
<td>VERY FINE SAND OR SILT</td>
<td>18</td>
<td>8.5</td>
</tr>
<tr>
<td>FINE SAND</td>
<td>15</td>
<td>7</td>
</tr>
<tr>
<td>COARSE SAND</td>
<td>12</td>
<td>5</td>
</tr>
<tr>
<td>FINE GRAVEL</td>
<td>9</td>
<td>4</td>
</tr>
<tr>
<td>COARSE GRAVEL</td>
<td>5</td>
<td>3</td>
</tr>
</tbody>
</table>

Table 14. Bligh rule and Lane rule: coefficients.

Using a capillary model, Khilar et al. (1985) suggested the following equation for evaluating the critical gradient \( i_c \):

\[
i_c = \frac{H_{\text{crit}}}{L} = \frac{\tau_c}{2.828} \cdot \left( \frac{n}{K} \right)^{0.5}
\]

(8)

Where:
\( \tau_c = \) critical shear stress [dynes/cm]
\( n = \) porosity
\( K = \) hydraulic conductivity [cm/s]

Khilar et al. (1985) in the case of clays recommended a maximum value of the critical gradient \( i_c \) equal to 40 (This value of the gradient is also the maximum value which is used to identify dispersive soils in the pinhole test Sherard et al. (1976)).

Sellmeijer’s model is a semi-theoretical model which considers the equilibrium of grains at the bottom of the pipe. The critical head is calculated as the head drop at which the grains are in equilibrium.

(Sellmeijer, 1988) concluded that critical conditions occur when the slit length approaches half of the base length and the critical head, \( H_{\text{crit}} \), is given by:

\[
H_{\text{crit}} = c_1 \cdot \frac{\gamma_p}{\gamma_w} \cdot \tan \theta \cdot (1 - 0.65 \cdot c_1^{0.42}) \cdot L
\]

(9)

Where:
\( c_1 = 0.25 \pi \eta \left( \frac{2d^3}{KL} \right)^{1/3} \)
\( \theta = \) bedding angles [°]
\( \gamma_p = \) unit submerged weight of particles
\( \eta = \) drag coefficient
L = seepage length
k = intrinsic permeability

Later, (Weijers & Sellmeijer, 1993) modified the equation for modeling initiation and progression of backward erosion:

\[ H_{\text{crit}} = \alpha \cdot c \cdot \gamma_p \tan \theta (0.68 - 0.1 \ln c)L \]  

(10)

Where:

\[ \alpha = \left( \frac{D}{L} \right)^{0.28} \frac{D}{L}^{2.8} - 1 \]  

(11)

\[ c = \eta \left( \frac{d_{70}^2}{K} \cdot \frac{d_{70}}{L} \right)^{0.33} \]  

(12)

\[ K = \frac{v}{g} \cdot k \]  

(13)

\( \gamma_w \) = unit weight of water [kN/m³]
\( \gamma_p \) = submerged unit weight of soil particles [kN/m³]
\( \theta \) = angle of repose of soil particles
\( \eta \) = whites drag coefficient
\( d_{70} \) = sieve size for which 70% by weight of the soil is finer [m]

D = thickness of sand layer under the embankment [m]

L = seepage length (base length of the embankment) [m]

K = intrinsic permeability [m²]

v = kinematic viscosity

g = gravity [m/s²]

k = hydraulic permeability [m/s]

The model was calibrated and adapted by large-scale and small-scale experiments (Sellmeijer et al., (2011)).

Two types of hydraulic conductivity relationships are considered: in the first type the hydraulic conductivity depends on porosity (Kozeny-Carman equation), while in the second type (Hazen equation), the hydraulic conductivity is not related to porosity. Ojha et al. (2001) found that permeability is a key component of piping models in defining the critical head limits and that the permeability relations which depend only on the grain size are of limited value.

The presence of sand boils does not directly result in a critical situation. In fact, sand boils can occur at a level lower than the critical head. Therefore, real breach cases are very useful for model verification.
As discussed, several prediction models are available to calculate the critical head, at which breach will occur, such as the empirical model of (Bligh, 1910) and the Sellmeijer model ((Sellmeijer, 1988); (Weijers & Sellmeijer, 1993); Sellmeijer et al. (2011)). With these models a critical head drop can be calculated to be compared with the actual head drop across the levee.

In Dutch practice, it is common to correct the actual head drop for the presence of a top soil layer at the seepage exit point. The head loss as a result of the vertical seepage path through the top soil layer allows for a reduction of actual head drop equal to 1/3 of the total vertical seepage path (TAW, 1999), resulting in:

\[
H - 0.3d \leq H_{\text{crit}} \tag{14}
\]

Where:
- \(H\) = actual head drop;
- \(H_{\text{crit}}\) = critical head drop;
- \(d\) = thickness of the soft soil layer.

Van Beek et al. (2013) compared the outcome of two prediction models: Bligh’s model and Sellmeijer’s model with using data from cases in China and in the Netherlands. He found that Bligh’s model is easier to apply than Sellmeijer’s model, due to a small number of input parameters. In fact, it has been used as a first step in safety assessment for many years in Dutch practice, but, its use is limited due to the fact that it can give lower critical head predictions than the Sellmeijer’s model. On one hand, Sellmeijer’s model is more accurate than Bligh’s model because it takes into account influence of scale and sand characteristics. On the other hand, the use of that model can result in larger uncertainties because it is sensitive to input parameters like permeability and grain size. If there are uncertainties in the parameters, this sensitively results in a wide range for the critical head.

“To use conservative parameters may lead to unrealistic high failure probabilities so a probabilistic approach for parameter estimation combined with more detailed soil investigation where necessary would be a step forward in piping prediction”. (Van Beek, Yao, & Van, 2013).

\(H_{\text{crit}}\) is the critical gradient which cause piping to initiate and progress to form a complete pipe under the embankment but backward erosion initiates at gradients 40 – 60% of the critical gradients to cause backward erosion to initiate and progress (Weijers & Sellmeijer, 1993).
Dutch guidelines for dike design require that each parameter in the equation is assigned a characteristic value (= mean ± standard deviation) and a factor of safety of 1.2 is applied to the seepage path length calculated using the characteristic values for the required design head differential.

(Schmertmann, 2000) proposed a design method based around results of flume tests. The tests were carried out on a range of fairly uniform soils from fine to medium sands up to coarse sand/fine gravel mixes. The head differential was increased progressively until the critical head, forming a pipe to the upstream source, was reached.

The method calculates a factor of safety against piping initiating and progressing to form a complete pipe. The point factor of safety at point x along the flow path (along which piping may progress) is:

$$F_{px} = \frac{\left( C_D \cdot C_L \cdot C_S \cdot C_K \cdot C_Z \cdot C_Y \right) \cdot i_{pmf} \cdot C_a}{C_R \cdot i_{fx}} \tag{15}$$

Where:

- $D$ = depth of piping sand layer in direction perpendicular to inclination of the piping tunnel
- $L$ = direct length between ends of a completed pipe path (measured along the pipe path from downstream to upstream exit) [m]
- $C_D$ = correction factor for $(D/L)$ → Figure 36
- $C_L = \left( \frac{L_x}{L_f} \right)^{0.2} = \left( \frac{L/\sqrt{k_h/k_v}}{L_f} \right)^{0.2}$ = length factor
- $L_f$ = flume model length
- $C_S = \left( \frac{d_{10}}{0.20} \right)^{0.2}$ = grain size factor
- $D_{10f}$ = size of the 10% finer than by weight size in the piping soil at point x [mm]
- $C_K = \left( \frac{1.5}{R_{kf}} \right)^{0.5}$ = anisotropic permeability factor
- $R_{kf} = \frac{k_h}{k_v}$ (of the piping soil)
- $C_Z$ = correction factor for high–permeability underlayer → Figure 37
- $C_Y = 1 + 0.4 \left( \frac{D_{1f}}{100} - 0.6 \right)$ = Density factor
- $D_{1f}$ = relative density of the piping layer [%]
- $C_a$ = Adjustment for pipe inclination → Figure 38
- $i_{pmf}$ = maximum point seepage gradient needed for complete piping in
the flume test (strongly related to the soil coefficient of uniformity) → Figure 39

\[ C_R = \frac{R_1 + R_2}{2R} \]

→ correction factor for dam axis curvature this factor is applied if the embankment is curved in plan

R = radius to point on the pipe path in a dam with curved axis [m]

\( R_1; R_2 = \) shortest and longest radiuses to an end of completed pipe path [m]

\( i_{fx} = \) hydraulic seepage gradient at point x parallel to the flow path

**Figure 36.** CD (Depth/Length factor) versus Depth/Length (Schmertmann, 2000).

**Figure 37.** Factor C₂ versus layer depth (D)/pipe radius (r) (Schmertmann, 2000).
FIGURE 38. GRAPH TO OBTAIN $I_{po}$ FROM $I_{po}$ AND $\alpha$ (SCHMERTMANN, 2000).
Some observations were done from the tests by Schmertmann:

- There were no significant time effects in the experiments carried out;
- The process seemed to be independent of the effective confining stresses on the soil;
- Vibrations tended to reduce the critical gradient;
- Detachment of the particles occurred at very much lower gradients (40 to 90 times) than required to scour the surface for the soil tested;
- The pipe path formed a system of channels.

Two procedures are recommended by Schmertmann:

1) The Point Method.
Choose a representative section of the embankment and construct a flow net. Select a pipe through the section (the pipe path should follow a flow path) and obtain $i_{pm}$ for each of the soils in the trial pipe path by using piping flume tests or Figure 39. Apply the correction factors to the $i_{pm}$ to obtain the gradient needed to move the pipe path through each point $x$ and calculate $F_{px}$. Determine enough $F_{px}$ points to get a profile along the trial pipe path.

2) The Average Method.

Calculate an average $F_p$ using estimated average values for the correction factors and the gradient $H/L$.

In many case, important detail will be lost if only the average method is used (Fell & Wan, 2005).

### 2.1.1.2. Initiation by Suffusion

The Table 15 sums up the criteria which have to be satisfied for internal erosion initiation by suffusion occurs.

<table>
<thead>
<tr>
<th>Geometrical Criteria</th>
<th>Criterion 1</th>
<th>The size of the soil particles must be smaller than the size of the constriction between the coarser particles which form the basic skeleton of the soil.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Criterion 2</td>
<td>The amount of fine soil particles must be less than enough to fill the voids of the basic skeleton.</td>
<td></td>
</tr>
<tr>
<td>Hydraulic Criterion</td>
<td>Criterion 3</td>
<td>The seepage flow through the soil matrix must be high enough to move the loose fine soil particles through the constrictions between the larger soil particles.</td>
</tr>
</tbody>
</table>

**Table 15. Criteria have to be satisfied for internal erosion to initiate by suffusion.**

Soils can be internally unstable if they are gap graded or consist of coarse soils with a limited percentage of finer soil.

Soils which satisfy criterion 1 but have too much fine soil particles (so they don’t satisfy criterion 2) aren’t subjected to suffusion but may be subject to backward erosion. In fact, if there are more than enough fine soil particles for void filling, the coarser particles will be floating in the matrix of fine soil particles and suffusion will not occur. (Wan & Fell, 2004) suggested that suffusion doesn’t occur if the finer fraction is greater than about 40% of the total weight.
SOILS 10, 14A, 15 were internally unstable. Soil 13 did not show any erosion before heave occurred. Soil 11 had too much finer soil to be subject to suffusion, it was eroded by piping.

Figure 40. Grain size distribution curves of gap graded soils (Modified from (Wan & Fell, 2004)).

Figure 41. Grain size distribution curves for some broadly graded soils (Wan & Fell, 2004).
Figure 42. Contours of the probability of internal instability of silt-sand-gravel soils and clay-silt-sand-gravel soils with a plasticity index less than 12% and less than clay size fraction (% passing 0.002 mm) (Wan & Fell, 2004).

Figure 43. Contours of the probability of internal instability for sand-gravel soils with less than 10% non-plastic fines passing 0.075 mm (Wan & Fell, 2004).
Figure 42 and Figure 43 were developed by (Wan & Fell, 2004) to determine the probability of internal instability of silt–sand–gravel soils and clay–silt – sand–gravel soils.

Suffusion critical hydraulic gradient can be defined as (Skempton & Brogan, 1994)

\[ i_{cr} = \alpha \cdot i_{cr} \]

where the effective stress reduction factor \( \alpha \) is an unknown function of the soil physical parameters that is lesser than 1: \( \alpha \approx \frac{1}{3} \) to \( \frac{1}{5} \).

Vertical upflow suffusion tests carried out by Skempton and Brogan (1994) and by Wan and Fell (2004) show that the internally unstable soils all began to erode under vertical seepage gradient of 0.8 or less. Tests on the same soil carried out at different porosities resulted in different gradients at which erosion began: higher porosity soils beginning to erode internally at gradients less than 0.3. A lesser gradient is required to initiate internal erosion in a horizontal direction.

Therefore (Fell & Wan, 2005) suggest the following method for assessing the probability of initiation of suffusion:

- Check if the percentage of finer fraction is less than 40% to establish if the soil is potentially susceptible to suffusion. (For soils which have more than 40% of finer fraction assess if the soil is subject to backward erosion).
- Use Figure 42 or Figure 43 to assess the probability of the soil being subject to internal instability.
- If the soil is internally unstable, use Figure 44 to assess the probability that suffusion will occur taking account of the gradient and the porosity.

The probability of initiation of suffusion is the product of the probabilities that that soil is internally unstable and that the suffusion will occur.

For soils which are susceptible to suffusion, the fraction of the soil which may erode can be estimated by:
- Selecting the split between the fine and coarse fraction for gap–graded soils;
- Selecting the point where there is a marked change of slope for broadly graded soils;
- Using the approach suggested by (Wan & Fell, 2004) based on the Kenney and Lau method of plotting shape curves (for more details see § 2.1.3.2).

However, having determined the finer fraction through these methods, not all the finer fraction may erode.

### 2.1.1.3. Initiation by contact erosion

<table>
<thead>
<tr>
<th>INITIATION BY CONTACT EROSION</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Geometrical Criteria</strong></td>
</tr>
<tr>
<td><strong>Hydraulic Criterion</strong></td>
</tr>
</tbody>
</table>

Table 16. Criteria have to be satisfied for internal erosion to initiate by contact erosion.

Initiation by contact erosion occurs when a granular soil is in contact with another soil, cohesive or not, and the filter rules are not satisfied.

It occurs for critical hydraulic gradient between 0.08 and 1 and for Darcy velocity between 1 and 10 cm/s (Beguin et al. (2012)).

### 2.1.1.4. Initiation of erosion in a concentrated leak

It requires the presence of a crack or a high permeability zone in the embankment foundation by–passing any cutoff provided for the embankment. (Fell & Wan, 2005) defined this condition as:
In this context, a “high permeability zone” is high permeability within a cohesive soil e.g. holes of porous soils formed by laterisation effects. It is not internal erosion in a layer of sand or gravel which is covered in backward erosion or suffusion. Where a cohesive soil is adjacent a coarse high permeability layer in a foundation, the mechanism is one of backward erosion.

The most likely scenarios in which a crack or high permeability zone may be present is where these exist in the soil profile before the embankment was constructed, and were not removed by the foundation preparation for the cutoff and general foundation.

The presence of a crack or high permeability zone depends on the nature of the soils, the degree and depth of cracking in those soils due to desiccation, the presence of roots, voids or open fissuring. Besides these flaws need to be continuous beneath the embankment. Moreover, given there is a crack or high permeability zone, not always erosion will initiate because some soils are resistant to erosion.

The Slot Erosion Test (SET) and the Hole Erosion Test (HET) were developed by (Wan & Fell, 2002) to measure the erosion properties of the soils used in embankment dams (§ 2.1.5). Both the tests can be used to measure the erosion rate of a soil expressed in a form of an Erosion Rate Index, I, defined by:

\[ I = -\log(C_e) \]  
\[ \dot{\varepsilon}_t = C_e (\tau - \tau_c) \]

Where:
- \( \dot{\varepsilon}_t \) = erosion rate per unit area;
- \( C_e \) = coefficient of soil erosion;
- \( \tau \) = shear stress;
- \( \tau_c \) = critical shear stress for initiation of erosion.

Erosion rate index for the HET, \( I_{\text{HET}} \), is similar to that for the SET, \( I_{\text{SET}} \). Given that HET requires simpler equipment and is less costly, (Fell & Wan, 2005) adopted the HET as the measure of erosion test. They defined the representative erosion rate index \( \bar{I}_{\text{HET}} \) as the hole erosion index \( I_{\text{HET}} \) for soil compacted to a density ratio of 95% of standard maximum dry density at optimum moisture content. According to their representative \( \bar{I}_{\text{HET}} \), soils can be classified into 6 groups as it is indicated in Figure 45.

The issue is the determination of the initial shear stress for a particular soil (Fell & Wan, 2005):
“In practice it is difficult to determine the initial shear stress for a particular soil. To determine it from laboratory hole erosion tests at varying heads is costly, and even then the initial shear stress is not well defined. (…) Caution should be exercised in using the hole erosion tests and calculations using the tests as described above. It would be unwise to rely too heavily on these calculations given the uncertainty in estimating the initial shear stress and calculating the hydraulic shear stress in cracks or in high permeability.”

<table>
<thead>
<tr>
<th>Group No.</th>
<th>Erosion Rate Index</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>&lt;2</td>
<td>Extremely rapid</td>
</tr>
<tr>
<td>2</td>
<td>2 – 3</td>
<td>Very rapid</td>
</tr>
<tr>
<td>3</td>
<td>3 – 4</td>
<td>Moderately rapid</td>
</tr>
<tr>
<td>4</td>
<td>4 – 5</td>
<td>Moderately slow</td>
</tr>
<tr>
<td>5</td>
<td>5 – 6</td>
<td>Very slow</td>
</tr>
<tr>
<td>6</td>
<td>&gt;6</td>
<td>Extremely slow</td>
</tr>
</tbody>
</table>

Figure 45. Soils classification according their Representative Erosion Rate Index (From (Fell & Wan, 2005)).

The coefficient of erosion is an indicator of the failure time between the visual detection and the breach: the greater the erosion index, the greater the time to failure. There is not a real influence of the structure height on the failure time but a strong influence of the erosion index.

For levees with height < 10 m (Bonelli & Benahmed, 2011):
- If the erosion index is of the order of 2 the failure will take place within a few minutes;
- If the erosion index is of the order of 3 the failure will take place within several hours;
- If the erosion index is greater than 4 the failure will not occur until several days.
2.1.2. **Estimating the Likelihood of Continuation of Internal Erosion**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>≥ 85%</td>
<td>DF15 ≤ 9 DB85</td>
<td>6.4 - 13.5 DB85</td>
<td>DF15 ≤ 9 DB85</td>
</tr>
<tr>
<td>2A</td>
<td>35 - 85%</td>
<td>DF15 ≤ 0.7mm</td>
<td>0.7 - 1.7mm</td>
<td>DF15 ≤ 0.7mm</td>
</tr>
<tr>
<td>3</td>
<td>&lt; 15%</td>
<td>DF15 ≤ 4 DB85</td>
<td>6.8 - 10 DB85</td>
<td>DF15 ≤ 7 DB85</td>
</tr>
<tr>
<td>4A</td>
<td>15 - 35%</td>
<td>DF15 ≤ (40-pp%*0.075mm) x (4DB85-0.7)/25 + 0.7</td>
<td>1.6 - 2.5 DF15 of Sherard and Dunnigan design criteria</td>
<td>DF15 ≤ 1.6 DF15d, (2) where DF15d = (35-pp%*0.075mm)/(4DB85-0.7)/20 + 0.7</td>
</tr>
</tbody>
</table>

Notes:  
1. The subdivision for soil group 2 and 4 was modified from 40% passing 75mm, as recommended by Sherard and Dunnigan (1989), to 35% based on the analysis of the filter test data. The modified soil groups are termed group 2A and 4A. The fines content is the % finer than 0.075mm after the base soil is adjusted to a maximum particle size of 4.75mm.  
2. For highly dispersive soils (Punohole classification D1 or D2 or Emerson Class 1 or 2), it is recommended to use a lower DF15 for the no erosion boundary.  
   - For soil group 1 soils, suggest use the lower limit of the experimental boundary, i.e. DF15 ≤ 6.4 DB85.  
   - For soil group 2A soils, suggest use DF15 ≤ 0.5mm.  
   - The equation for soil group 4A would be modified accordingly.

**Figure 46. Results of testing to define the no erosion boundary and proposed criteria for the no erosion boundary of filter tests for the assessment of filters of existing dams ((Foster M. A., 1999), (Foster & Fell, 2001)). Figure from (Fell & Wan, 2005).**

The natural variability of the soil and its effect on the ability to act as a filter should be considered.  
Transition zones overlying foundations should be carefully considered because such zones usually are very variable in grading (with cobbles and little finer material) with segregation and large voids common.  
Even high fines content filters are critical. In fact, it should be considered, on one hand if the filter will hold a crack and, on the other hand, if the high fines content may reduce the permeability to such an extent that the high hydraulic gradient may fracture the filter and form a pipe.  
On holding a roof, the most important guide to performance is the fines content of the soil (% passing 0.075mm), and whether the soil is partially saturated or saturated. If a soil has ≥ 15% fines, it would hold a roof, regardless of whether the fines are non plastic or plastic (Foster M. A., 1999), (Foster & Fell, 1999).  
Poorly compacted soils, which has been placed without any rolling, will be less likely to hold a crack (Fell & Wan, 2005).
For internal erosion initiated by suffusion the mechanism is one of eroding the finer fraction from the coarser matrix, so progression to form a continuous zone of high permeability coarse material does not involve formation of an open pipe.

“We are not able to provide any guidance on how large a zone may be subject to suffusion. Some case studies which we believe may have been subject to suffusion have many sinkholes – even hundreds. (…) Backward erosion may reach an equilibrium condition without full development of a pipe, so it seems reasonable to assume the same can happen for suffusion.

Unfortunately we have no data to allow quantification of these factors, and given the uncertainty in assessing the critical gradients to initiate suffusion, it is suggested that it be assumed that if suffusion initiates, internal erosion will progress until a continuous path of soil with the finer fraction is eroded is formed. What this means overall is that if suffusion initiates, and the layer is continuous beneath the embankment, progression is highly likely.” (Fell & Wan, 2005).
RIVER EMBANKMENTS FAILURE CAUSES

<table>
<thead>
<tr>
<th>Fines Content % Passing 0.075mm</th>
<th>Fines Plasticity</th>
<th>Likelihood of holding a crack or eroding</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Well compacted</td>
</tr>
<tr>
<td>5% to 7%</td>
<td>Non plastic</td>
<td>Unlikely</td>
</tr>
<tr>
<td>7% to 15%</td>
<td>Non plastic</td>
<td>Likely</td>
</tr>
<tr>
<td>&gt; 15%</td>
<td>Non plastic</td>
<td>Very likely</td>
</tr>
<tr>
<td>5% to 7%</td>
<td>Plastic</td>
<td>Likely</td>
</tr>
<tr>
<td>7% to 15%</td>
<td>Plastic</td>
<td>Very likely</td>
</tr>
<tr>
<td>&gt; 15%</td>
<td>Plastic</td>
<td>Almost certain</td>
</tr>
</tbody>
</table>

**Figure 48. Likelihood of filters with excessive fines holding a crack (From (Fell & Wan, 2005)).**

<table>
<thead>
<tr>
<th>Erosion condition</th>
<th>Joint opening width, w</th>
</tr>
</thead>
<tbody>
<tr>
<td>No erosion</td>
<td>( w \leq D_{55} ) surrounding soil ( w \leq 0.5 D_{55} ) surrounding soil</td>
</tr>
<tr>
<td>Some erosion</td>
<td>( D_{44} &lt; w &lt; D_{55} ) surrounding soil ( 0.5 D_{44} &lt; w &lt; D_{55} ) surrounding soil</td>
</tr>
<tr>
<td>Excessive erosion</td>
<td>( D_{50} ) surrounding soil &lt; ( w &lt; D_{55} ) ( D_{55} ) surrounding soil &lt; ( w &lt; D_{35} )</td>
</tr>
<tr>
<td>Continuing erosion</td>
<td>( w \geq D_{55} ) surrounding soil ( w \geq D_{35} ) surrounding soil</td>
</tr>
</tbody>
</table>

**Figure 49. Joint openings for no, excessive and continuing erosion into pipes and conduits (From (Fell & Wan, 2005)).**

<table>
<thead>
<tr>
<th>Soil Classification</th>
<th>Percentage Fines</th>
<th>Plasticity of the Fines</th>
<th>Moisture Condition</th>
<th>Likelihood of Supporting a Roof</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clays, sandy clays (CL, CH, CL-CH)</td>
<td>&gt; 50%</td>
<td>Plastic</td>
<td>Moist or saturated</td>
<td>Certain</td>
</tr>
<tr>
<td>Sandy clays, Gravely clays, (SC, GC)</td>
<td>&lt; 50%</td>
<td>Plastic</td>
<td>Moist</td>
<td>Almost certain</td>
</tr>
<tr>
<td></td>
<td>&gt; 12%</td>
<td></td>
<td>Saturated</td>
<td>Highly likely</td>
</tr>
<tr>
<td>Silty sands, Silty gravels, Silty sandy gravel (SM, GM)</td>
<td>&gt; 15%</td>
<td>Non plastic</td>
<td>Moist</td>
<td>Highly likely</td>
</tr>
<tr>
<td>Granular soils with some cohesive fines (SC-SP, SC-SW, GC-GP, GC-GW)</td>
<td>5% to 15%</td>
<td>Plastic</td>
<td>Moist</td>
<td>Likely</td>
</tr>
<tr>
<td>Granular soils with some non plastic fines (SM-SP, SM-SW, GM-GP, GM-GW)</td>
<td>5% to 15%</td>
<td>Non plastic</td>
<td>Moist</td>
<td>Less likely</td>
</tr>
<tr>
<td>Granular soils, (SP, SW, GP, GW)</td>
<td>&lt; 5%</td>
<td>Non plastic</td>
<td>Moist and saturated</td>
<td>Very unlikely</td>
</tr>
</tbody>
</table>

**Figure 50. Likelihood of a well compacted soil being able to support a roof to an erosion pipe (Table 17 from (Fell & Wan, 2005)).**
Filter protection is very important in the design and evaluation of earth embankments.

Internal erosion may initiate for a number of reasons (internal instability, segregation, cracks, filter and core incompatibility, etc.) but the initiation may stop if there exists a filter function within the parent material.

In fact, filters act as barriers stopping continuation of erosion. The eroded soil is caught at the filter face and the filter cake is widening and it has a low permeability at some distance on each side of the crack. The pressure gradient is reduced in the crack and erosion stops. The remaining filter is open and it allows seepage flow.

If the filter fails, erosion progresses and may lead to breaching.

Filter rules are necessary in order to verify the interface between two soils regarding erosion risk and to design the transition soil (when it is necessary).

Sometimes, it is difficult to fulfill the filter rules between two different soils, then, it is necessary to interpose a transition soil of intermediate size, which must fulfill the filter rules with both the two soils.

Filters and transition layers must fulfill two functions:

- Retention function: they must prevent migration of particles from the base;
- Permeability function: they must be able to accept seepage without excessive pore-pressure build-up.

Moreover, filters and transition layers should not segregate; not degrade and change gradation; not have cohesion. They should be internally stable and able to seal concentrated leaks (ICOLD. International Commission on Large Dams, 1994).

Filter rules must be adopted to verify the interfaces between:

- the upstream fill and the foundation;
- the core and the downstream fill;
- the core and the foundation;
- the foundation and the drainage blanket.

Segregation causes non-homogeneities in a material.

Even if methods to quantify segregation are still unclear, there are guidelines in the literature that explain how to minimize segregation problems. Rönnqvist (2010) summarized some of them (Figure 51).
Numerous filter design criteria have been proposed from which a few are more accepted.

The first criteria for filter design, developed by (Terzaghi & Peck, 1967), and The US Bureau of Reclamation (1955), were mainly for filtering sandy soils as no studies had been made to test filters for silts and clays. Between 1980 and 1985, the Soil Conservation Service (SCS) conducted an extensive study of filters for dams. This study was made to test filter requirements for various soils, silt and clay soils as well as sandy soils where simulated cracks were formed in soil specimens and water under high pressure was passed through the cracks to simulate an earth core zone with cracks. In fact, generally, for some silt soils and most clays, it is not possible to create piping conditions if water passes only through soil pores without cracks or other opening or anomalies (their low permeability do not allow sufficient water velocity or volume to erode the soil particles) (Talbot, 2012). This kind of test was called the “No Erosion Filter Test” (§ 2.1.5.1). The filter study results were reported by various authors (Sherard, Dunnigan, & Talbot, 1984), (Sherard & Dunnigan, 1989), (U.S. Soil Conservation Service, 1994), (USACE, 2005), (Reclamation, 2011). Subsequently others tests were performed in Australia at the University of New South Wales ((Foster & Fell, 2001), (Wan & Fell, 2002)).

**Terzaghi Criteria**

Terzaghi advanced the following criteria:

\[
df_{15} \leq 4 \text{ to } 5 \ dB_{85} \tag{18}
\]

\[
NF_{15} \leq 4 \text{ to } 5 \ dB_{15} \tag{19}
\]

Where:

- \(df_{15}\) is the diameter for which 15% of the filter by weight is finer;
- \(dB_{15}\) is the diameter for which 15% of the base (core or foundation) by

<table>
<thead>
<tr>
<th>Source</th>
<th>Finer than 4.75 mm</th>
<th>D10 min</th>
<th>D_max</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sherard et al. (1984a)</td>
<td>At least 40 %</td>
<td>50 mm</td>
<td></td>
</tr>
<tr>
<td>Ripley (1986)</td>
<td>At least 60 %</td>
<td>18 mm</td>
<td></td>
</tr>
<tr>
<td>NEH (1994)</td>
<td>&lt; 0.5 mm</td>
<td>20 mm (Dmax)</td>
<td></td>
</tr>
<tr>
<td>NEH (1994)</td>
<td>0.5-1.0 mm</td>
<td>25 mm (Dmax)</td>
<td></td>
</tr>
</tbody>
</table>
weight is finer;
\[ d_{B85} \] is the diameter for which 85% of the base by weight is finer.

The first equation is intended to prevent significant erosion of the base materials into the filter, while, the second one is intended to make sure the permeability of the filter is significantly greater than that of the base.

A coefficient of four was established based on the diameter of a sphere that can fit into the voids created by the arrangement of spheres in a dense and loose state (Kezdi, 1979).

The criteria has a factor of safety of about 2 and is appropriate for general use (Sherard, Dunnigan, & Talbot, 1984).

2.1.3.1. Criteria for Assessing Self-Filtering and Filter Performance

The self-filtration mechanism starts when the fine base particles migrate into the voids of the filter while the coarser particles are left behind at the interface. They are gradually brought together and retain finer particles until further migration is impossible. It is important to highlight that the gradation of the material that has undergone self-filtering is different from the original base gradation.

**Kezdi Criterion**

Kezdi criterion (1979) consists of dividing the filter gradation curve into two parts for each point along the curve: the fine fraction works as the base while the coarse fraction works as the filter to the base. If both the two parts of the curve satisfy Terzaghi criterion: 
\[ \frac{df_{15}}{dB_{85}} \leq 4 \]
the soil is able to self-filter and so it is internally stable.

It is not necessary to split the original gradation curve into two parts for every checked point along the curve because \( df_{15} \) and \( dB_{85} \) can be obtained, once selected an arbitrary point \( S_0 \) on the original gradation curve, by determining the grain size at the passing weight \( F = 0.85 \cdot F_0 + 15 \) and \( F = 0.85 \cdot F_0 \) respectively.
RIVER EMBANKMENTS FAILURE CAUSES

**Figure 52. Kezdi criterion (Li M., 2008).** FF is the fine fraction of the split grading while FC is the coarse fraction.

**Sherard Criterion**

(Sherard, 1979) recommendation is also based on the Terzaghi criterion: a soil is self-filtering if $D_{15\text{coarse}} / d_{85\text{fine}} < 4$ to $5$ after splitting the gradation curve in two.

<table>
<thead>
<tr>
<th>SHERARD &amp; DUNNIGAN (1989)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>BASE SOIL (&lt; 4.75 mm)</strong></td>
<td><strong>FILTER</strong></td>
</tr>
<tr>
<td>c% &gt;85% Fine silt and clay</td>
<td>$D_{15}^{\text{filter}} &lt; 9d_{85}^{\text{soil}}$</td>
</tr>
<tr>
<td>40%&lt;c%&lt;85% Sand, silt, clay</td>
<td>$D_{15}^{\text{filter}} &lt; 0.7 \text{ mm}$</td>
</tr>
<tr>
<td>15%&lt;c%&lt;40% Silty and clayey sand and gravel</td>
<td>$D_{15}^{\text{filter}} (\text{mm}) &lt; \left[ 0.7 + \frac{(40 - c%)}{25} \right] \cdot \left( 4d_{85}^{\text{soil}} - 0.7 \right)$</td>
</tr>
<tr>
<td>c%&lt;15% Sand, gravel</td>
<td>$D_{15}^{\text{filter}} &lt; 4d_{85}^{\text{soil}}$</td>
</tr>
</tbody>
</table>

Where:

- $c\% = %$ passing $75\mu m$
- $D_{15}^{\text{filter}} = \text{diameter for which } 15\% \text{ of the filter by weight is finer}$
- $d_{85}^{\text{soil}} = \text{diameter for which } 85\% \text{ of the base soil by weight is finer}$

**Table 17. Sherard & Dunnigan criterion: summary.**
(Sherard, Dunnigan, & Talbot, 1984) found that for sandy soils there is a very narrow boundary between filter failure and success well defined by: $\frac{d_{f15}}{d_{B85}} = 9$ (or $d_{B85} \leq 0.11 d_{f15}$). A very slight increase in $d_{f15}$ can change the function of the filter from success to failure, so, filter design criteria should provide for a margin of safety. While for sand soils the boundary between success and failure is defined by a factor of 9, for silt and clay soils, the boundary is not a linear relationship. A margin of safety should be used to ensure that the boundary between success and failure is avoided. Tests could be made to determine more precise criteria for site specific design.

![Diagram of soil types which have experienced a lack of self filtering (Sherard, 1979).](image)

**Foster & Fell criteria**  
Foster (1999) and (Foster & Fell, 2001) developed the concept of no, excessive and continuing erosion for filters. The criteria for No Erosion Boundaries are based on logistic regression analyses of the results of more than 500 filter tests (Foster M. A., 1999), (Foster & Fell, 1999). The linear equations obtained by the logistic regression analyses can be conveniently used to estimate the probability that the erosion boundary is exceeded (Fell & Wan, 2005).  
(Foster & Fell, 2001) pointed out that the equivalent opening size between grains of a filter is given by $D_{filter}^9$. Based on this, the equivalent no, excessive and continuing opening size for an open joint are as shown in Figure 43.
A uniformly graded filter can be thought of as a sieve with an effective opening size given by $D_{15}^{\text{filter}}/9$. If the effective filter opening size is greater than the maximum particle size of the base material ($d_{100}^{\text{soil}}$), the filter will never be sealed no matter how much erosion of the base material occurs. This is verified by the results of slot tests, slurry tests and NEF tests (Fell & Wan, 2005).

<table>
<thead>
<tr>
<th>FOSTER &amp; FELL (2001)</th>
<th>FILTER</th>
</tr>
</thead>
<tbody>
<tr>
<td>BASE SOIL (&lt; 4.75 mm)</td>
<td>$D_{15}^{\text{filter}} &lt; 9d_{85}^{\text{soil}}$</td>
</tr>
<tr>
<td>1  c% &gt;85%</td>
<td>Fine silt and clay</td>
</tr>
<tr>
<td>2A 35%&lt;c%&lt;85%</td>
<td>Sand, silt, clay</td>
</tr>
<tr>
<td>4A 15%&lt;c%&lt;35%</td>
<td>Silty and clayey sand and gravel</td>
</tr>
<tr>
<td>3  c%&lt;15%</td>
<td>Sand, gravel</td>
</tr>
</tbody>
</table>

Where:

- $c\% = \%$ passing 75μm
- $D_{15}^{\text{filter}} = \text{diameter for which 15\% of the filter by weight is finer}$
- $d_{85}^{\text{soil}} = \text{diameter for which 85\% of the base soil by weight is finer}$

The bold type remarks the differences.

In many regions of the world fine grained low-plasticity soils are abundant and comprise the only affordable core materials for dam construction. However, from the viewpoint of internal erosion these soils are disadvantageous, as they have feeble erosion resistance, hence special attention should be devoted to selection of proper filters to prevent internal erosion.

Fine-grained low-plasticity silty and silty-sandy soils, such as CL, CL-ML, ML and SC, are considered as competent core materials given that they satisfy permeability requirements of central sealing elements but they require special attention in filtration related problems, as they have both small particle sizes and weak erosion resistance (Soroush, Shourijeh, & Mohammadinia, 2011).

“(…) Filter design criteria may not always ensure the safe filter action for protecting fine grained low plasticity soils, viz. CL, CL-ML and ML.

In the authors' experience, a filter criterion of $d_{15}/d_{85} \leq 6$ may be
required for safe filtration of low-plasticity group 1 base soils. (…) NEF testing is helpful in determining safe filters. For problematic soils and major projects conduction of NEF tests are vital for filter design. It should be noted however that the NEF test is a specialist test that requires proficient conduction and interpretation. (…) Conducting regular in-situ sand castle tests is recommended to control the quality of filter materials.” (Soroush, Shourijeh, & Mohammadinia, 2011).

<table>
<thead>
<tr>
<th>Investigator</th>
<th>Base soil designation</th>
<th>Group 1</th>
<th>Group 2</th>
<th>Group 3</th>
<th>Group 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sherard &amp; Dunnigan</td>
<td>%&lt;75μm</td>
<td>≥ 85</td>
<td>40-85</td>
<td>&lt; 15</td>
<td>15-40</td>
</tr>
<tr>
<td>Shourijeh &amp; Soroush</td>
<td>Criterion: D15 ≤ 0.7 mm</td>
<td>D15 ≤ 0.7 mm</td>
<td>D15 ≤ 0.7 mm</td>
<td>D15 ≤ 4D15</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Intermediate between value for group 2 and 3 based on %&lt;75μm</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes: *% finer than 0.075 mm in the gradation with maximum size of 4.75 mm; **D15 ≤ minimum of 0.7 mm and 6.4D15; ***D15 ≤ 0.5 mm for highly dispersive soils; ****D15 ≤ 6D15 for ML and CL-ML soils; *****D15 ≤ 7.5D15 for highly dispersive soils.

**Figure 54. Proposed filter criteria based on NEF testing. From (Soroush, Shourijeh, & Mohammadinia, 2011).**

**Figure 55. Conceptual filter erosion boundaries (Foster M. A., 1999). DF15 is the sieve size for which 15% of the filter is finer. Continuing erosion means that the filter is too coarse to allow the eroded base materials to seal the filter. The criteria for No Erosion Boundaries proposed are based on logistic regression analyses of the results of more than 500 filter tests.**
2.1.3.2. Criteria for assessing internal stability

**USACE (1953) criterion**
According to this criterion, for suffusion to occur in cohesionless filter material are required:

- turbulent flow conditions;
- hydraulic gradient \( \geq 5 \);
- coefficient of uniformity \( Cu >20 \).

Based on seepage tests of 20 soils, (Wan & Fell, 2004) obtained considerable scatter in the results when using the (USACE, 1953) criterion and they concluded that using \( Cu>20 \) as a predictor of internal instability was too conservative.

**Isotomina criterion**
Similar to the (USACE, 1953), this criterion is also based on the \( Cu \). (Isotomina, 1957) classification in terms of the potential for suffusion is as follows:

- \( Cu \leq 10 \) ⇒ no suffusion;
- \( 10 \leq Cu \leq 20 \) ⇒ transition;
- \( Cu \geq 10 \) ⇒ suffusion-liable.

\( Cu \) was found to be a less effective relationship to be used when assessing dams of broadly graded soils (Ronnqvist, 2010).

**Kenney and Lau method (1985)**
In an internally unstable material, certain constrictions will stay open and allow the migration of smaller-sized particles; this is most likely to occur in materials with gently inclined grading curves in the fine tail section (lower portion). According to this method, to evaluate the internal stability, the “\( H:F \) shape curve” should be established, which is the ratio of the mass fraction of particle sizes between \( d \) and \( 4d \) (\( H \)) and the passing weight at particle size \( d \) (\( F \)).

Given that the predominant filter constriction is approximately one-quarter the size of the small particles making up the filter, particles of size \( d \) can pass through constrictions of filters that have particles of size \( 4d \) and coarser. To avoid transportation of size \( d \) particles, the openings must be occupied by particles larger than \( d \); hence, the choice of the interval of four (Kenney & Lau, 1985).

The particles of size \( d \) or below will not be adequately protected against erosion if there is insufficient materials within the size range \( d \) to \( 4d \).

The boundary between stable and unstable is \( H=1.0F \) (Kenney & Lau, 1986) (initially it was \( H=1.3F \)): if \( H>F \) the soil is stable while if \( H<f \) the soil is unstable.
The soils investigated by most authors are sand-gravels free of cohesive fines while Burenkova (1993) investigated widely graded cohesive soils.

The method proposes to assess the potential suffusion exposure of a soil in terms of three representative soil grain sizes: \(D_{15}, D_{60}\) and \(D_{90}\).

A soil is internally stable if:

\[
0.76 \log h'' + 1 < h' < 1.86 \log h'' + 1
\]

(20)

Where:

\[
h'' = d_{90}/d_{15}
\]

\[
h' = d_{90}/d_{60}
\]

The larger the ratios of \(h'\) and \(h''\), the more widely graded the material becomes.


The influencing factor on the internal stability was found to be the presence of gap grading. An estimation of the internal stability of silt, sand and gravel mixtures is found out combining the criteria of Burenkova (1993) and (Kenney & Lau, 1985), (1986). Moreover, (Wan & Fell, 2004) determined that the maximum possible fraction
of erodible particles in internally unstable soils is not greater than 40%. Therefore, soils with a higher proportion of fine particles are not susceptible to internal instability. Anyway they may be susceptible to backward erosion or unable to self-filter.

Wan and Fell (2004) suggested the following approach for assessing the maximum fraction of eroded particles in internally unstable soils:

- Using the shape curve (H–F), identify a maximum F value, \( \hat{F}_{\text{max}} \), smaller than 0.4 at which the stability number, \( H/F \), is smaller than 1.3;
- \( \hat{F}_{\text{max}} \) is an estimate of the maximum fraction of erodible particles, and the particle size, \( d_{\text{max erod}} \), corresponding to \( \hat{F}_{\text{max}} \) is an estimate of the size of the largest erodible particles.

<table>
<thead>
<tr>
<th>Likelihood of Internal Instability</th>
<th>Kenney &amp; Lau (1985, 1986) method</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( H &lt; F )</td>
</tr>
<tr>
<td></td>
<td>( F \leq H &lt; 1.3F )</td>
</tr>
<tr>
<td></td>
<td>( H \geq 1.3F )</td>
</tr>
<tr>
<td>Burenkova (1993) method</td>
<td>Likely</td>
</tr>
<tr>
<td></td>
<td>Neutral - Likely</td>
</tr>
<tr>
<td></td>
<td>Very unlikely</td>
</tr>
<tr>
<td>( h' \leq 0.76 \log(h') + 1 )</td>
<td>Likely</td>
</tr>
<tr>
<td></td>
<td>Neutral - Likely</td>
</tr>
<tr>
<td></td>
<td>Very unlikely</td>
</tr>
<tr>
<td>( h' &gt; 0.76 \log(h') + 1 )</td>
<td>Unlikely</td>
</tr>
<tr>
<td></td>
<td>Very unlikely - Unlikely</td>
</tr>
<tr>
<td></td>
<td>Very unlikely</td>
</tr>
</tbody>
</table>

**Figure 57. Assessing the likelihood for internal instability of silt, sand and gravel mixtures (Wan & Fell, 2004).**

**Li and Fannin method**

(Li & Fannin, 2008) found out a method to estimate the internal stability of soils by combining the criteria of (Kezdi, 1979) and (Kenney & Lau, 1985), (1986) (Figure 58).
Cistin criterion

The criterion of Cistin (1967) was proposed by the Federal Waterways Engineering and Research Institute in Germany as a direct prediction method for the internal stability assessment of soils (at the moment there are some discussions about replacing of (Cistin, 1967) method with (Kenney & Lau, 1985)).

“The methods of [Kenney and Lau 1985, 1986] or [Sherard 1979] are the most popular methods to assess whether a soil in a dam/dike or its foundation is internally unstable, even if the soil is gap or widely graded with cohesive fines. This is because there are no other well accepted methods. Engineers using the mentioned methods do not know if their results are over or underestimated. [Wan and Fell 2008] proposed a modified approach of [Burenkova method 1993], which is not adequately assessed for the soils that have less than 15% finer fractions (fines fractions = fractions which are finer than a particle size of 75 μm”). (Salehi Sadaghiani & Witt, 2012).
(Salehi Sadaghiani & Witt, 2012) presented a new approach for analysis of internal stability for gap and widely graded soils based on cluster analysis of soil particle size distribution (PSD) using two normal distributions for identifying soil skeleton and its mobile particles. They analysed twenty PSDs with this method and compared the results with results of other methods. They concluded that:

“It is shown that the most broadly used methods to evaluate whether a soil is internally stable or not, are conservative. Probability density function clearly delivers more information about the soil structure and is beneficial for internal stability analysis. For important projects, it is recommended to carry out laboratory tests. It must be emphasized that the biggest problem of gap and widely graded soils is their prone to heterogeneity.” (Salehi Sadaghiani & Witt, 2012).

![Table 1: Summary of the results of different prediction methods](image)

Table 1: Summary of the results of different prediction methods

Figure 59. Comparison of different prediction methods with the method proposed by (Salehi Sadaghiani & Witt, 2012).
2.1.4. **Dispersive soils**

“Dispersive soils are those soils, which when immersed in relatively pure and still water will deflocculate causing the clay particles to go into suspension”. (Maharaj, 2011).

The use of dispersive soils in engineering structures such as hydraulic structures and roadway embankments can lead to failures if the problem is not appropriately identified and addressed.

When immersed in water, as opposed to flocculation, dispersive clay minerals tend to came apart and this behavior tends to make the particles smaller than what is measured in gradation tests. Therefore, different retention criteria should be used (Reclamation, 2011).

One of the main problems is to positively identify such soils. Many identification methods have been proposed: the crumb test (that is the simplest); the Pinhole Test; the SCS Double Hydrometer test and various chemical analyses.

“For base soils with more than 15% fines, adequate tests should be performed to establish whether the clay fines are dispersive in character”. (Reclamation, 2011).

2.1.5. **Test procedures**

In the following paragraph procedures for tests are presented.

**2.1.5.1. No Erosion Filter Test (NEF)**

The No Erosion Filter (NEF) test is widely recognized as the most appropriate test for filters in embankment dams. It is recognized as a competent filter test especially for fine grained soils.

The NEF test results define the boundary filter (by $D_{15b}$) that is the coarsest filter that prevents base soil erosion (Soroush, Shourijeh, & Mohammadinia, 2011).

Since the NEF test is not standardized by institutions such as ASTM, various testing techniques have been devised.

Experiences by Soroush et al. (2009); (2011), resulted in the described semi-standard procedures for NEF testing, described.
The main container of the apparatus is a plexiglas cylinder with internal diameter of 11 cm and height of 30 cm.
A drainage layer is placed at cylinder bottom. Filter materials are blended from fractions of washed sands, with uniform sizes, in four equal portions. Every portion is thoroughly mixed with 3% moisture content and carefully placed in the cylinder to prevent segregation. The amount of compaction for each layer is determined by trial and error, such that the final filter thickness produces the relative density required.

A plastic or rubber ring is forced to intrude the final filter layer (it replaces granular side materials). It is located flush with internal walls of the cylinder and thus is watertight.

Base soil materials are compacted in a special mold at 1–2% wet of optimum moisture content, and to $0.98 \gamma_{\text{d,max}}$. The compacted base soil specimen (3 cm thick) is detached from the mold and pushed into the apparatus cylinder to sit on top of filter materials.

A one mm hole is punched throughout the middle of the base specimen such that it extends one cm into filter materials.

A wire screen separator is placed on top of the base specimen. The space remained on top of the cylinder is filled with gravels while, the voids of gravel, are filled with water.

When the test starts the air vent is closed, the inlet valve is fully opened and the outlet valve is opened. During the test, the out-coming flow is collected in graduated cylinders. The test is usually continued for at least 20 minutes until flow rate and turbidity generally stabilize. The effluent flow rate and turbidity should be carefully monitored for information regarding the test behavior.

The test is successful if there is no visible erosion of the performed hole in the base specimen.

During NEF testing on fine grained low plasticity soils the upper regions of the hole soften can close the hole and filter functionality can not be tested (no flow emerges through the hole and the apparatus).

To solve this problem is recommended the application of a truncated cone (nipple) that intrudes the base specimen and supports the hole during testing (Soroush & Shourijeh, 2009).

(Foster & Fell, 2001) indicated it is uncommon for particles with a diameter $> 0.2$ mm to be transported.

(Hill & Beikae, 2007) proposed a simulation of filter performance to improve understanding of the phenomenon of the base-filter interaction and also to provide guidance in situations where existing filters do not meet filter criteria. That simulation of the NEF test randomly selects a particle from the base soil, assumes it gets transported to the face of the filter soil and then randomly selects three particles from
the filter and calculates the size of the opening formed by the three particles. If the particle passes through the opening, the new position within the filter layer is calculated, and the process is repeated, while if the particle becomes lodged in the filter it becomes added to a "filter cake" layer and the gradation of the filter is adjusted. The authors found that for uniform filters, the results of the simulation matched reasonably well with the results described for the laboratory NEF testing described by (Foster & Fell, 2001).

![NEF Test Apparatus](Figure 62)

**2.1.5.2. Continuing Erosion Filter Test (CEF)**

The Continuing Erosion Filter (CEF) Test was developed by Foster, Fell, and Spannagle (2000(a)), (2000(b)) to evaluate the potential for continuing erosion. The CEF test was developed as a modification to the NEF test (NEF test can still be performed in the CEF device).

The following modifications were made to the NEF test during the development of the CEF test ((Reclamation, 2011)):
- Water passing through the filter during the tests was collected, and the eroded materials were dried and weighed to determine the loss of base soil required to seal the filter.
- Progressively coarser filters were used until the filter was not sealed.
- Thicker base specimens were used to allow for greater erosion losses.

![Figure 63. CEF Test Apparatus (from Reclamation, 2011).](image)

### 2.1.5.3. Pinhole Test

The pinhole test device is an instrument for direct measurements of the dispersibility and erodibility of fine grained soil, using a water flow passing through a small hole in a soil specimen, under hydraulic heads.

The test was developed by Sherard et al. (1976) and in the past years has become a widely used physical test. According to Sherard et al. (1976), the test is highly reproducible and the results of each individual test can be categorized easily.
A 1.0 mm diameter hole is punched or drilled through a cylindrical soil specimen (length=25 mm; diameter=35 mm). Distilled water is percolated through the pinhole under heads of 50, 180, and 380 mm (hydraulic gradients of approximately 2, 7, and 15 and velocities ranging from approximately 30 to 160 cm/s), and the flow rate and effluent turbidity are recorded.

It is important that the test be made on soil at its natural water content because drying may affect test results for some soils. If the material contains coarse sand or gravel particles, these should be removed by working the sample through a 2 mm sieve (U.S. Standard No. 10).

Water flow through the pinhole simulates water flow through a crack or other concentrated leakage channel in the impervious core of a dam or other structure. Dispersibility is assessed by observing effluent color and flow discharge through the hole and by visual inspection of the hole after the completion of the test.

2.1.5.4. Emerson Crumb Test

The Emerson Crumb Test (Emerson, 1967) was developed as a simple procedure to identify dispersive soil behavior in the field, but is often used in the laboratory as well.

Emerson found the interaction of clay-sized particles in water to be a major determining factor in the stability of a soil in an agricultural context. Based on this deduction, Emerson developed the flow chart illustrates by Figure 64 which qualitatively divide soils into eight different classes.
The crumb test gives a good indication of the potential erodibility of clay soils, but, a dispersive soil may sometimes give a non-dispersive reaction in the crumb test. However, if the crumb test indicates dispersion, the soil is most likely dispersive. The mostly followed method, involves placing a crumb of soil in a beaker of solution and observing the reaction as the crumb begins to hydrate. The test is used as a visual assessment: after a certain time, usually 5-10 minutes, the soil crumb and the solution in the beaker are observed and the soil is classified according to the quantity of colloids in suspension. Four grades of dispersivity can be noted ranging from no reaction to strong reaction:

- **1 = No reaction.** The crumb may slake and run out on the bottom of the beaker in a flat pile but there is no sign of cloudy water caused by colloids in suspension.
- **2 = Slight reaction.**
- **3 = Moderate reaction.** From slight to moderate reaction: there are cloud of colloids in suspension that may be just at the surface of the crumb or spreading out in thin streaks on the bottom of the beaker.
- **4 = Strong reaction.** The colloid cloud covers nearly the entire bottom of the beaker, usually in a very thin layer and in extreme cases, all the water in the beaker becomes cloudy.
Emerson’s method has been often used incorrectly and it has been misinterpreted with regard to variables such as moisture content and dispersing medium (Maharaj, 2011). The available ASTM standard for the crumb test (ASTM D 6572-00) takes into account variables which have no effect on the dispersivity such as temperature and simple size. Conversely, no standard protocol regarding variables like immersion solution and condition of crumb is available. Tests are carried out using both dilute NaOH or distilled water; samples are either air dried, oven dried, remoulded or in situ and the time taken to run the test is variable. Variations that can occur due to the lack of a standard protocol for testing dispersivity of soils can lead to differences in the classification. All of these variables can have significant effects on the outcome of the test. (Maharaj, 2011) found that:

- The inconsistency of results associated with the crumb test is primarily due to the time of observation. The results of a crumb test after 10 minutes are not reliable as some of the fine particles, which are not necessarily dispersive, can still be in suspension. The colloidal suspension of a dispersive soil should not settle over time so it is recommended a minimum waiting period of one hour.
- Significant differences occur between the remoulded crumbs and those that are dried.
- Oven dried samples demonstrated the most inconsistent results because the physiochemical properties of the soil pore-water and adsorbed water may be changed when exposed to high temperatures. Remoulding the samples have the effect of enhancing the dispersive behavior of the soil and it also simulates the action of the working and compaction processes on the soil in the field, and is likely to give more realistic results.
- Different solutions have different effects on various soils and carrying out the test using both solutions should provide more useful results.

2.1.5.5. Sand castle test

Sand castle test, devised by (Vaughan & Soares, 1982), is an inexpensive method that is recommended to evaluate filter collapsibility and self-healing and to control the quality of filter materials. In fact, sand castle test was introduced to evaluate the self–healing properties of a filter zone in an embankment dam. (Vaughan & Soares, 1982) proposed this test to evaluate the cracking potential of filter material. If cracks form, for a filter to be effective, it is necessary for it to be
cohesionless. Otherwise, it may itself sustain an open flooded crack without collapse and so fail to protect a cracked core.

(Vaughan & Soares, 1982) described the test as:

“A simple test, suitable for use in a field laboratory, has been devised to examine filter cohesion. It consists of forming a cylindrical or conical sample of moist compacted filter, either in a compaction mould, or in a small bucket such as is used by a child on a beach; standing the sample in a shallow tray (if a bucket is used the operation is exactly as building a child’s sand castle) and carefully flooding the tray with water. If the sample then collapses to its true angle of repose as the water rises and destroys the capillary suctions in the filter, then the filter is noncohesive. Samples can be stored for varying periods to see if cohesive bonds form with time. This test is, in effect, a compression test performed at zero effective confining pressure and a very small shear stress, and it is a very sensitive detector of a small degree of cohesion.”

USACE adopted this test in (USACE, 1993).

In situ sand castle tests are also recommended for constructed filter layers in a dam, and are particularly applicable to filters over-compacted under construction equipment traffic, segregated filters, etc. (Milligan, 2003).

A detailed sand castle testing procedure was presented by Soroush et al (2008). Sand castle test results are qualitative so companion proficient engineering judgment is needed to verify its authenticity and reliability.

The lack of precision and the inability to express results quantitatively are faults of this test. Specimen preparation has also been identified as an issue by Reclamation that has undertaken a study to improve the test procedure (Bureau of Reclamation, 2011(a)). In the revised procedure proposed by Reclamation, the specimen is oven dried at 120 degrees Fahrenheit (°F) until its weight stabilizes. In fact, by observation, it has been noted that filter material placement in the field can be exposed to drying and warm summertime temperatures between placements, sometimes for several days and these conditions may contribute to forms of physiochemical bonding between soil grains. After curing, the samples are placed in water and the time to collapse is recorded. The curing step appears to be the critical element in making the Sand Castle Test sensitive to the conditions experienced by filter material in the field (Reclamation, 2011).
Figure 65. Vaughan Test (From (USACE, 1993)).

Figure 66. Relatively poor self-healing behavior: the sample does not collapse after 50% submersion (Figure from (Reclamation, 2011)).
2.1.6. **INTERNAL EROSION RISK FOR SERCHIO RIVER EMBANKMENTS**

In the previous paragraphs, several criteria for assessing internal stability, self-filtering and filter performance have been reported. A first kind of criteria assesses the suffusion risk and the self-filtering properties of a soil while it does not consider interface problems between two different soils. Conversely, filter rules are necessary in order to verify the interface between two soils regarding erosion risk and to design transition soils. Filter should fulfill both the filter rules and the criteria for assessing internal stability.

There are evidences that internal erosion could be one of the possible failure causes for Serchio River embankments in December 2009. In fact, Figure 17 shows the water flow through the upper part of the embankment at Nodica (Pisa) during the flood event.

Unfortunately, the embankments under consideration were mainly constructed before Terzaghi (since XVIII century) and there are not evidences of construction details. Moreover, in the District of Lucca, the embankment top is unreachable (width between 1.2–3 m) hence all the boreholes were carried out from the bank and there were not grain distribution curves of samples retrieved from the embankment body. Therefore, only the grain distribution curves of samples retrieved from 16 boreholes carried out in the District of Pisa were available.

Serchio River embankments do not have filters so in this study, it has been assessed their internal stability applying the criteria for assessing suffusion risk and their self-filtering properties.

Almost all existing methods for assessing internal stability and self-filtering were developed for granular material.

Serchio River embankments are made up by various kinds of soils: sands, silts and clays are present in different percentages as the stratigraphy illustrates (§ 1.1.6). However, silt and clay are always present in a certain amount so criteria suitable for cohesive soils should be applied.

Grain size analyses indicate that:

- for all the analysed samples $D_{15} < 0.4$ mm;
- the coefficient of uniformity, $C_U$, is generally between 6 and 50;
- extreme values of $C_U$ are 2.4 (one sample) and 70 to 90 (5% of analysed samples).
USACE criterion and Isotolina criterion were applied even if they are suitable for no cohesive soils. Most samples satisfy the USACE criterion while, according to Isotolina criterion, most samples are classified as “transition” soils in terms of the potential for suffusion because their coefficient of uniformity is included between the range 10–20.

**Figure 67. Results of USACE criterion application.**

**Figure 68. Results of Isotolina criterion application.**
GUIDELINES FOR THE GEOTECHNICAL DESIGN, UPGRADEING AND REHABILITATION OF RIVER EMBANKMENTS

BARBARA COSANTI

Figure 69. USACE and ISOTOMINA criteria application: left bank of the Serchio River embankments in the District of Pisa.

Figure 70. USACE and ISOTOMINA criteria application: right bank of the Serchio River embankments in the District of Pisa.

Burenkova criterion was developed for widely graded cohesive soils. Applying this criterion most samples, the 89%, are internally stable (Figure 71 and Figure 72).
According to Kenney & Lau criterion, all the grain distribution curves of the samples available have almost a part that does not satisfy the criterion that is usually the lower part of the curves.
Applying Kezdi criterion, almost all the samples turn out to be unable to self-filter and hence internally unstable.

The most critical parts of the gradation curve are the lower one and the upper one. Obviously, the lower part is the most dangerous. In fact, the seepage flow should be extremely high to detach and to transport the bigger and heavier soil particles. The most critical sample is the S1 C3. In fact, for that sample, the percentage of passing by weight of fine particles detached is around 29%, corresponding at a maximum particle size of 0.0207 mm.

Finally, a further kind of evaluation was done. The Terzaghi criteria were applied at the three samples retrieved from each borehole. Considering a descending seepage flow, the central sample works as a base for the lower one while it works as a filter for the upper one. All the samples analysed fulfilled the retention criterion while only one sample fulfilled the permeability criterion (sample S1 C3). This result suggests that the embankment soil is pretty homogeneous.
Figure 73. Example of application of the Kezdi criterion: sample S2 C1.
Figure 74. Example of application of the Kezdi criterion: Sample S1 C3.

Figure 75. Explanation of the application of the Terzaghi criteria.
Finally, the analysed soils do not seem to be particularly prone to piping phenomena but the tests performed in this study do not allow such a clear conclusion.

It should be remarked that investigations concern a very limited number of boreholes and it is not possible to exclude the presence of anomalies and heterogeneities within the embankment.

Moreover, the considered embankments do not have a filter. Internal erosion may initiate for a number of reasons but the initiation may stop if there exists a filter function within the parent material, conversely, if there is not a filter or the filter fails, erosion progresses and may lead to breaching.

Every incidence of initiation of internal erosion needs to be taken seriously. Unfortunately, internal erosion can initiate under low hydraulic gradients, it can go on for years and even with seepage monitoring it is often difficult to detect thereby problems can develop at any time, despite years of good performance.

Embankment lacking engineered cutoffs, filters, and drains should be considered safety deficient and regular inspection and seepage monitoring are essential requirements.

In order to decide what is needed, risk based evaluation of potential failure modes is necessary. Some consistent quantitative measure by which to judge priorities are needed.

Since a balance between public safety and costs is necessary, risk analysis allow the greatest risk reduction with the available funding to be reached and the use of event trees is helpful.

Since each levee is unique, a detailed investigation and the evaluation of the potential failure modes and their consequences can allow a decision based on potential for failure and risk to be made.

The decision to add a toe drain, filter or cutoff wall is so risk based and considers both the site specific case and overall situation in need to prioritize limited funds.

For Serchio River embankments, in order to prevent piping phenomena, the District of Lucca (Service for the Defense of the Territory) decided to install a metallic sheet pile diaphragm within the embankments (for more details see § 2.3.4.3). Moreover, the cross–section geometry of the embankments was modified by addicting a berm.
2.2. OVERTOPPING AND SURFACE EROSION

2.2.1. OVERVIEW

Overtopping is one of the main causes of failure for levees. Overtopping of earthen levees may occur during the periods of flood due to insufficient freeboard. It produces fast-flowing, turbulent water on the landside slope that can damage the protective grass covering and expose the soil to erosion.

Often levee are constructed using locally available materials dredged from the excavations of adjacent shipping channels thereby large portions of these levees are comprised of poorly compacted soils of variable character and consistency with poor resistance to erosion.

The potentially erodible nature of the materials dredged from adjacent navigational channels and placed at high water contents and without significant compaction is a known concern.

Employment of locally available soils from adjacent channel excavations results in initial cost-savings but at the cost of significantly increased risk of subsequent failure for the constructed levees.

Moreover, levee structures are one of the primary sources of coastal flood protection used worldwide. The recent increase in storm surge intensity and the increased incidence of hurricanes necessitate stronger and more resilient coastal protection structures.

The failure of the New Orleans (Louisiana) regional flood protection systems during Hurricane Katrina on August 29, 2005 is the most emblematic example.

Several levee failures in this region were due to overtopping and erosion. A number of studies addressing the levee failures and breaches have been carried out. (Seed, et al., 2008) found that some levees were comprised largely of clays (CH and CL), but contained numerous zones or lenses of fine sands and silty and clayey sands. These soils could not be compacted in their wet condition so they were emplaced in three–four stages. Each stage was placed with shallow initial side slopes and time was allowed to elapse for settlements and consolidation then, as the materials began to dry, the levees were reshaped and the process was repeated with another added stage. As a result of this lack of compaction, these soils had relatively low resistance to erosion and their performance was very poor.

Conversely, adjacent levee sections that were constructed with better compacted cohesive soils (sections constructed using clays that were placed and compacted at
significantly lower water contents representing more workable and compactable conditions) had good resistance to erosion (minimal to negligible damages).

Both field observations and analyses suggest that multiple modes of erosion were active in these failures.

More precisely, four erosion mechanisms are described by (Seed, et al., 2008):

- Sheet flow overtopping. As water passes over the top of the embankment and begins to descend landside slope, the velocity of flow down this back slope increases. This increase in velocity, in turn, increases the shear force between the water and the embankment soil. The erosive traction exerted by the water increases on the lower portion of the inboard side slope face thereby in this location erosion from simple overtopping is most critical. This type of overtopping erosion can be produced either by fully sustained overtopping or by cyclic overtopping by waves.

- Wave scalloping erosion. It is produced by wave attack waterside on the front faces of the embankment as wind-driven waves atop the rising surge break and runup against the levee.

- Notching (crenellation). As the storm surge rises higher, wind-driven waves can begin to scallop upwards with enough erosive force as to begin to erode notches in the front lip of the levee crest. As they progress, they can lead to notching completely through the levee crest (a phenomenon called “crenellation” after the crenellated walls that topped medieval castles (Seed, et al., 2008)).

- Through-flow seepage erosion due to flow passing through the levee embankment section and then exiting low on the inboard side slope face.

Herein only the erosion of levees overtopped by sheet flow is dealt with.

![Figure 76. Schematic illustration of four erosion mechanisms from (Seed, et al., 2008).](image-url)
2.2.2. Erodibility

Levee overtopping has been the subject of several research projects. Field tests have helped to identify the failure mechanism and have indicated that the erosion failure occurs first on the land side of the overtopped levee and progressively regresses.

Erodibility is sometimes defined as the relationship between the velocity of the water flowing over the soil and the corresponding erosion rate experienced by the soil. Since water velocity is a vector quantity which varies everywhere in the flow and is theoretically zero at the soil water interface, Briaud et al. (2008) preferred to quantify the action of the water on the soil by using the shear stress $\tau$ applied by the water on the soil at the water–soil interface and defined erodibility as the relationship between the erosion rate $\dot{z}$ and the hydraulic shear stress applied.

The erosion function $\dot{z}(\tau)$ can be obtained by using a laboratory device called the erosion function apparatus (EFA) (Briaud et al. (2001)). The EFA test consists of eroding a soil sample by pushing it out of a thin wall steel tube and recording the erosion rate for a given velocity of the water flowing over it. Several velocities are used and the erosion function is defined.

Briaud et al. (2001) proposed the erodibility classification chart of Figure 77 based on years of experience in testing soil samples in the EFA. The erosion chart classifies soils in erosion categories from category I (very high erodibility) to category VI (non–erosive)$^2$. Erosion categories associate a single number to the erosion. Highly erodible soil types (e.g. lightly compacted clean sands) tend to plot in the upper left hand zone of the erodibility classification chart while well–compacted, cohesive, clayey soils with intrinsically high resistance to erosion plot in the lower right corner. Therefore, it is clearly possible to select and emplace soils with intrinsically high resistance to erosion in order to achieve levee sections that can safely sustain some degree of overtopping.

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$^2$ Velocities are used instead of shear stresses for simplicity and as a first step in establishing the categories. Briaud et al. (2001).
Briaud et al. (2001) found that an increase in compaction effort increases the resistance to erosion and that the increase is more pronounced for soils with higher fine content. Moreover, Briaud et al. (2001) presented a recommended chart which can be used to select soils for overtopping resistance, on the basis of the erosion test results and of the observed behavior of the levees in New Orleans during the Katrina Hurricane (Figure 78).

This chart represents a levee guideline for erosion resistance during overtopping. Briaud et al. (2001) suggested that EFA erosion tests should be used in the future to predict levee behavior and ensure erosion resistance to overtopping.

In a complete model of the erosion process, the three resultant stresses, responsible for the erosion process, should be taken into account: the net shear stress in the horizontal direction ($\tau - \tau_c$), the fluctuation of that shear stress ($\Delta \tau$) and the fluctuation of the net uplift normal stress ($\Delta \sigma$) (Briaud et al. (2001)):
\[
\dot{Z} = \frac{\alpha \cdot (\frac{\tau - \tau_c}{\rho \cdot u^2})^m + \beta \cdot (\frac{\Delta \tau}{\rho \cdot u^2})^n + \delta \cdot (\frac{\Delta \sigma}{\rho \cdot u^2})^p}{u}
\]

(21)

Where:

\begin{itemize}
  \item \(\dot{Z}\) = erosion rate [m/s];
  \item \(u\) = water velocity [m/s];
  \item \(\rho\) = mass density of water [kg/m^3];
  \item \(\alpha; m; \beta; n; \delta; p\) = parameters characterizing the soil being eroded.
\end{itemize}

In common practice the following simpler model is well accepted:

\[
\dot{Z} = \frac{\alpha \cdot (\frac{\tau - \tau_c}{\rho \cdot u^2})^m}{u}
\]

(22)

This model is less thorough but is more practical because only two parameters entering into the model on a site specific basis are required. These two parameters are obtained by curve fitting the equation of the model (22) to the erosion function measured in the EFA.

### Soil Erosion and Levees: Netherlands’ Practice (Crow, 2000); (Taw, 1998)

**Flood Protection Systems Should:**

- Be able to resist a storm surge with an annual probability of occurrence of 1/10,000 for the Province of Holland;
- Be able to resist a storm surge with an annual probability of occurrence of 1/4,000 for less populated coastal areas;
- Be reviewed and evaluated every 5 years with associated recommendations to be constructed in the following 5 years.

**The Permitted Slope of the Levees Which Do Not Have Any Protection Is Related to the Erodibility of the Soils Used to Build the Levee:**

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>LL(%)</th>
<th>PI(%)</th>
<th>Percent sand(%)</th>
<th>Percent lutum(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>&gt;45</td>
<td>&gt;18</td>
<td>&lt;40</td>
<td>—</td>
</tr>
<tr>
<td>Clay</td>
<td>&lt;45</td>
<td>&gt;18</td>
<td>&lt;40</td>
<td>—</td>
</tr>
<tr>
<td>Clay</td>
<td>&lt;45</td>
<td>&lt;18</td>
<td>&gt;40</td>
<td>—</td>
</tr>
<tr>
<td>Sand</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

(From Briaud et al. (2001))

**Table 20. Soil Erosion and Levees: Netherlands’ Practice.**

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2.2.3. A FULL-SCALE TEST

Crest and landside slopes of levees should be protected with some type of strengthening method such as turf reinforcement, soil strengthening or hard armoring. The levee strengthening systems should resist the forces of fast-flowing, turbulent water that has overtopped the levee crest.

The most problematic cases involve the levee being overtopped by both surge and waves when the surge level exceeds the levee crest elevation with accompanying wave overtopping. In fact, during Hurricane Katrina, the most common cause of earthen levee or dike failure in New Orleans District was attributed to severe wave overtopping and erosion of the landward side slope.

Therefore, Colorado State University was commissioned by the New Orleans District Corps of Engineers to develop a testing facility, the Wave Overtopping Simulator, that would be capable of simulating full-scale wave overtopping to avoid significant scale effects related to grass strength and soil erosion. The Wave Overtopping Test Facility at Colorado State University is a permanent, fixed-in-place installation that can simulate large wave overtopping events and large steady overflow.

Thornton et al. (2012) conducted full-scale tests of several levee slope surfaces subjected to realistic wave overtopping using the Wave Overtopping Simulator with the goal of establishing new design criteria. They found that bare clay slope failed rapidly as expected and bare lime-reinforced clay failed sooner than expected. Well-maintained and healthy Bermuda grass did not fail under extraordinary levels of wave overtopping and this superior resiliency was attributed to a dense root system and ample thatching of the grass plants. Dormant Bermuda grass sustained significant damage at reduced overtopping loads. Finally, a typical articulated concrete block system proved effective at preventing erosion of the underlying bare clay during large wave overtopping.

Since in Italy levees do not undergo wave overtopping hydrodynamic loads, in this study, assessment of levee slope resiliency under hydraulic flows was focused on steady overflow.

Assessment of levee resiliency during overtopping using numerical analyses is complex hence it can be useful to use controlled experiments conducted at full scale to avoid significant scale effects.

Therefore, in this study, an experimental full scale embankment was constructed in the District of Lucca (by Del Debbio S.p.a.) and overtopping tests were conducted for assessing the levee resiliency during overtopping. The fine soil being used to
construct the experimental levee was quarry waste from a limestone quarry (Cave Pedogna S.p.a.).

The levee geometry tested consisted of a 2.6–m–high section on a 1–on–2/2.5 slope. This geometry was chosen because it is similar to those of levee section in the District of Lucca.

The embankment was constructed for a section, in the following referred to as T1, with the material from the quarry whereas, for a second section, in the following referred to as T2, with the same material but added with the 2% of lime, in order to compare the resistances. The plan shape of the embankment was rectangular hence the embankment enclosed a reservoir as shown by Figure 79, Figure 80 and Figure 81.
The landward-side levee slope was prepared for the experiments in the same manner as actual levee slopes.

T1 section was constructed 30 days before T2. The soil from the quarry was trucked to the experimental site and placed in 30/40–cm–thick layers with each layer being compacted by a Bomag roller compactor (operating weight of 18 t). The embankment was compacted by rolling along the longitudinal direction. At the moment of the compaction it was noticed that the soil was pretty wet.

In order to realize T2 section, the soil from the quarry was placed in 20/25–cm–thick layers and it was added with lime and mixed using a milling machine. After 24 hours of curing, the stabilized soil was trucked to the experimental site and placed in 30/35–cm–thick layers with each layer being compacted using a vibratory roller (operating weight of 18 t). The embankment was compacted by rolling along the cross-section direction.
On the two different sections, T1 and T2, two spillways with two ramps were constructed, in the following referred to as ST1 and ST2 respectively.

The quarry fines used for the experiment was tested with several laboratory analyses such as:

- Classification tests: determination of grain size distribution curve and Atterberg limits. The material was classified as CL according to USCS classification (ASTM D2487-00), as silt and sand with clay according to AGI classification (AGI, 1994) and as A6 according AASHTO classification (UNI 10006, 2002).
- Chemical analysis;
- Petrographic analysis;
- Gamma-ray spectroscopy;
- Variable head permeability tests on laboratory reconstituted samples. It was measured a permeability of about $1.6 \times 10^{-7}$ cm/s.

<table>
<thead>
<tr>
<th>SIEVE SIZE [mm]</th>
<th>SAMPLE 1</th>
<th>SAMPLE 2</th>
<th>SAMPLE 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>0.125</td>
<td>88.99</td>
<td>97.57</td>
<td>95.91</td>
</tr>
<tr>
<td>0.063</td>
<td>84.14</td>
<td>94.98</td>
<td>87.74</td>
</tr>
<tr>
<td>0</td>
<td>81.14</td>
<td>92.67</td>
<td>72.1</td>
</tr>
</tbody>
</table>

Figure 82. Grain size distribution of the quarry fines (Cave Pedogna S.p.A., 2010).
On March 2<sup>nd</sup> 2012, the basin was filled up with water and on March 14<sup>th</sup> 2012 the first overtopping test was carried out on the embankment section constructed with stabilized soil by releasing the spillway ST2. The test was carried out by dumping water into the basin without interruption for 6 hours, a time greater than the duration of a maximum Serchio flood event, using two pumps with a flow rate equal to 1000 l/min and 1300 l/min respectively. Prior to the start of each test, the reservoir was filled up and a constant flow rate was established corresponding to a specific average overtopping rate.
Overtopping loading was applied at constant discharge rate. As the flow was being adjusted, the spillway was released and the water was allowed to flow down the slope.

During overtopping, water overtops the crest and run down the landward-side slope. Overtopping tests were characterized by the flow discharge rate and flow thickness at the leading edge.

The water head on the spillway was about 15 cm.

The levee surface was tested for a total of 6 hours. After this time, at the end of the test, it was carried out an accurate topographic survey.

Figure 84. Section T1 before the overtopping test.

Figure 85. Section T1 before the overtopping test: spillway ST1.
The surface of the tested levee was surveyed: elevations were recorded at locations previously marked on the levee surface. In fact, in order to assess the resistance of the embankment to the overtopping, a topographic survey carried out after the overtopping test was compared with a topographic survey carried out before the overtopping test. Two different survey lines were spaced down the levee slope and elevations were recorded. Sections were constructed from the elevation data, and erosional differences between surveys were computed and plotted as illustrated in Figure 87.
At the end of the test insignificant soil loss had occurred as shown in the right-hand photo of Figure 88.

In the same vein, on March 15\textsuperscript{th} 2012, it was carried out the second overtopping test on the embankment section T2 and the spillway ST2.

Visual inspection and comparison surveys indicated little to no damage to the levee slop after the six–hour–test.
Figure 90. Section T2 and spillway ST2 before the overtopping test.

Figure 91. Section T2 during the second overtopping test.

Figure 92. Section T2 before (left) and after (right) the overtopping test.
Figure 87 and Figure 93 show the comparison between the different topographic surveys, before and after the overtopping tests, for the two sections, T1 and T2. The comparison shows the occurred erosion. It is possible to observe that both the lime-stabilized soil and the not stabilized soil have a high resistance to erosion due to overtopping, even without any reinforcement. Even at this extreme hydraulic loading, the levee slopes were nearly unscathed by the water as shown in the photographs hence bare soil exhibits good resiliency to overtopping.

The key finding from these tests was that the study material can withstand substantial overtopping without damage.

On the two sections T1 and T2, on March 19th 2012, two CPTu tests were carried out with a Pagani TG 63–150 penetrometer with a Begemann tip.

In the embankment section T1, where it was placed the spillway ST1, it was carried out a CPT test in the following referred to as CPT1, down to 4.8 m whereas in the embankment section T2, constructed with the stabilized soil, where it was placed the spillway ST2, it was carried out a CPT test in the following referred to as CPT2, down to 3.2 m.

For each section, two continuous samples were retrieved very close to the already performed CPTu for laboratory testing:

- Natural water content measurements. Each continuous sample was divided into 5 parts of equal length and for each part it was determined the water content and the degree of saturation by supposing a specific gravity of soil grains, $G_s$, equal to 2.7 (Table 23).
In situ soil density assessment. In fact, for every continuous sample, the value of the natural volume weight and the sample compaction were measured.

Figure 94. Profile of moisture content with depth from continuous samples S1C1 and S1C2 retrieved very close to CPT1 test in the embankment section T1.

Figure 95. Profile of moisture content with depth from continuous samples S2C1 and S2C2 retrieved very close to CPT2 test in the embankment section T2 (stabilized soil).
TABLE 23. TEST RESULTS.

<table>
<thead>
<tr>
<th>Embankment section</th>
<th>CPT test</th>
<th>Sample</th>
<th>Register number</th>
<th>Compaction [m]</th>
<th>$\gamma$ [kg/m$^3$]</th>
<th>$w$ [%]</th>
<th>$\gamma_d$ [kg/m$^3$]</th>
<th>e [%]</th>
<th>S [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>CPT1</td>
<td>S1 – C1</td>
<td>632</td>
<td>0.295</td>
<td>1500</td>
<td>18.75</td>
<td>1263</td>
<td>1.14</td>
<td>44</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S1 – C2</td>
<td>633</td>
<td>0.14</td>
<td>1719</td>
<td>19.78</td>
<td>1435</td>
<td>0.88</td>
<td>60</td>
</tr>
<tr>
<td>T2</td>
<td>CPT2</td>
<td>S2 – C1</td>
<td>634</td>
<td>0</td>
<td>1863</td>
<td>17.98</td>
<td>1579</td>
<td>0.71</td>
<td>68</td>
</tr>
<tr>
<td>(stabilized soil)</td>
<td></td>
<td>S2 – C2</td>
<td>635</td>
<td>0.035</td>
<td>2005</td>
<td>20.29</td>
<td>1667</td>
<td>0.62</td>
<td>88</td>
</tr>
</tbody>
</table>
The CPT profiles show that the stabilized soil had a tip resistance of almost 4 MPa whereas the untreated soil had a tip resistance of about 2 MPa. Probably, this difference was due to the degree of compaction that was lower for the untreated soil than for the stabilized soil, as well as to the lime presence. Moreover, the different techniques of compaction used for the two sections (T1 was rolled along the longitudinal direction while T2 was rolled along the cross-section direction) had a great influence on the achieved degree of compaction.

The valuation of the degree of saturation, even if it is approximate, shows that the seepage had not reached the steady state hence the material is a good barrier against water flow.
2.2.4. Remarks

This work points to the need to evaluate river embankments for erodible soils. The used soil has a high intrinsic resistance to erosion, as confirmed by the overtopping testing.

In fact, despite having been significantly overtopped for several hours (6 hours; for approximately 15 cm or more of sheet-flow overtopping), the experimental levee frontage suffered little damage; clearly demonstrating the value of spending a bit more to acquire, process, place, and compact suitable materials with high intrinsic resistance to erosion. Conversely, the employment of locally available soils with poor resistance to erosion, sometimes protected after placement, results in initial cost-savings but at the cost of significantly increased risk of subsequent failure for the constructed levees.
2.3. Mechanical Failure

The fundamental requirement for slope stability is that the soil shear strength must be greater than the shear stress required for equilibrium. Therefore, instability is due to the condition that soil shear strength is less than the shear strength required for equilibrium because the soil shear strength is decreased or the shear stress required for equilibrium is increased.

As stability analyses concern, most important lessons are learned from experience involving failures hence the state of the art is advanced through failure experiences and the currently used methods depend on experience. The current state of knowledge is not complete but the currently used methods can not be improved by altering only one part of the process.

It is important to understand the causes of slope failure both for designing and constructing new slopes and for purposes of repairing failed slopes to avoid additional failures.

Moreover, if the conditions at the time of sliding are well defined it is possible to determine the soil shearing resistance by back analyses. The strength is estimated and slope stability analyses are repeated until the calculated factor of safety is one.

Prolonged wet conditions play a significant role in the occurrence of the landslide. In fact, a relationship between rainfall and landslides has been observed by (Duncan & Wright, 2005):

“(…) Long periods of higher–than–average rainfall cause deep–seated, slow–moving slides, with shear surfaces that can extend tens of feet below the ground surface. One or two days of very intense rainfall, in contrast, tend to cause shallow slides involving only a few feet of soil, which move with high velocity once they are in motion”.

Several processes can lead to reduction in the shear strength of soil (Duncan & Wright, 2005):

- Increased pore pressure and associated decrease in effective stresses within slopes during periods of heavy rainfall;
- Cracks develop as a result of tension that exceeds the tensile strength of the soil near the crest of the slope;
- Highly plastic and heavily over-consolidated clays are affected by swelling when in contact with water;
- Development of slickensided surfaces especially in highly plastic clays as a result of shear on distinct planes of slip;
- Decomposition of clayey rock fill as it is wetted by infiltration or by groundwater seepage;
- Creep under sustained loads. Highly plastic clays may fail under sustained loads even at shear stresses significantly smaller than the short-term strength.
- Leaching that involves changes in the chemical composition of pore water;
- Brittle soils may be affected by strain softening;
- Rocks and strong soils may be subject to strength loss due to weathering;
- Loose soils and soils with particles that are weakly bonded into loose structures may be subject to loss of strength as a result of cyclic loading. In fact, cyclic loads may break bonds between soil particles and pore pressures may increase.

According to (Duncan & Wright, 2005), several mechanisms can lead to increase in the shear stresses of soil:
- Loads of the ground at the top of a slope;
- Water pressure in cracks at the top of the slope;
- Increase in soil weight due to increased water content;
- Excavation at the bottom of the slope;
- Drop in water level at the base of a slope;
- Earthquake shaking.

It is worthwhile to stress that water and clay mineral contents are the main factors involved in slope failures.

“(…) virtually every slope failure involves the destabilizing effects of water in some way, and often in more than one way. (…) The larger the content of clay minerals, and more active the clay mineral, the greater is its potential for swelling, creep, strain softening, and changes in behavior due to physicochemical effects. (…) It is safe to say that except for the effects of water and clayey soils, slope failures would be extremely rare” (Duncan & Wright, 2005).

Measures to stabilize include:
- improvement of surface drainage by installing horizontal drains for subsurface drainage;
- excavation to flatten and unload slopes;
- program of measurements of surface movements on slopes;
- systematic inspections of slopes for indications of instability.
A key step in slope stability analyses is estimating soil strengths. Slope stability during and at the end of construction is analysed using either drained or undrained strengths depending on the permeability of the soil. Moreover, undrained strengths for some soils and drained strengths for others soils can be used in the same analysis.

### 2.3.1. Mechanics of Limit Equilibrium Procedures

Once slope geometry, soil properties (shear strength etc) and pore water pressures are defined, slope stability analyses consist of computing a factor of safety.

The factor of safety is defined as:

\[
F = \frac{s}{\tau}
\]  

Where:

\( s \) = available shear strength;

\( \tau \) = equilibrium shear stress: shear stress required to maintain a just-stable slope.

In other words, the factor of safety \( F \) represents the factor by which the available shear strength \( s \) must be reduced so that it is just in equilibrium with the shear stress:

\[
\tau = \frac{s}{F}
\]  

If the shear strength is expressed by the Mohr – Coulomb equation and in terms of total stresses, it is possible to write:

\[
\tau = \frac{c + \sigma \tan \varphi}{F}
\]  

On the other hand, if the shear strength is expressed in terms of effective stresses:

\[
\tau = \frac{c' + (\sigma - u) \tan \varphi'}{F}
\]

There are several procedures of analysis but all of these employ the same definition of the factor of safety and compute the factor of safety using the equations of static equilibrium.

The term slip surface refers to an assumed surface along which sliding might occur. To calculate \( F \), a slip surface is assumed and equations of static equilibrium are used to calculate the stresses and factor of safety for each surface assumed. The factor of safety value is assumed to be the same at all points along the slip surface, it represents an average value for the assumed slip surface.
A number of slip surfaces must be assumed because it is necessary to find the slip surface that produces the minimum factor of safety that is termed critical slip surface. It represents the most likely sliding surface and the corresponding minimum factor of safety is unique for a given problem. The limit equilibrium procedures use static equilibrium conditions to compute the factor of safety:
- Equilibrium of forces in the vertical direction;
- Equilibrium of forces in the horizontal direction;
- Equilibrium of moments about any point.

Limit equilibrium analysis procedures use two different approaches:
- some procedures consider equilibrium for the entire mass of soil bounded by an assumed slip surface hence equilibrium equations are written and solved for a single free body;
- in other procedures, termed procedures of slices, the soil mass is divided into a number of vertical slices and equilibrium equations are written and solved for each slice.

Some procedures satisfy all of the equilibrium equations, others use and satisfy only some. In general, the procedures that satisfy complete static equilibrium are the most accurate.
Anyway, there are more unknowns than the number of equilibrium equations hence the problem is statically indeterminate and to resolve the problem assumptions must be made: different procedures make different assumptions to obtain a statically determinate solution.

2.3.2. PROCEDURES OF SLICES

Some procedures of slices assume a circular slip surface and consider equilibrium of moments about the center of the circle for the entire body while others procedures assume an arbitrary shape for the slip surface and consider equilibrium in terms of the individual slice.

2.3.2.1. CIRCULAR SLIP SURFACES

The driving moment can be expressed as:

$$M_d = \sum W_i \cdot a_i$$  \hspace{1cm} (27)

Where:
$W_i =$ weight of the $i^{th}$ slice;

$a_i =$ horizontal distance between the center of the circle and the center of the slice.

![Figure 100. Circular slip surface with overlying soil mass subdivided into vertical slices (Figure 6.7 from (Duncan & Wright, 2005)).](image)

Although the base of the slice is curved, with negligible loss in accuracy, it can be assumed to be a straight line hence $a_i$ can be expressed in terms of the radius of the circle and the inclination of the bottom of the slice:

$$a_i = r \cdot \sin \alpha_i$$  \hspace{1cm} (28)

Where:

$\alpha_i =$ angle measured between the base of the slice and the horizontal.

The driving moment becomes:

$$M_d = r \cdot \sum W_i \cdot \sin \alpha_i$$  \hspace{1cm} (29)

Where:

$W_i =$ weight of the $i^{th}$ slice;

$a_i =$ horizontal distance between the center of the circle and the center of the slice.

While the resisting moment can be expressed as:

$$M_r = r \cdot \sum S_i = r \cdot \sum \tau_i \cdot \Delta l_i \cdot 1$$  \hspace{1cm} (30)

Where:

$S_i =$ shear force on the base of the $i^{th}$ slice;

$\tau_i =$ shear stress;
Equating the resisting moment and the driving moment and replacing the expression of the factor of safety (23), it is possible to write (dropping the subscript i):

$$ F = \frac{\sum s \cdot \Delta l}{\sum W \cdot \sin \alpha} = \frac{\sum(c + \sigma \cdot \tan \varphi) \cdot \Delta l}{\sum W \cdot \sin \alpha}$$

This equation represents the static equilibrium equation for moments about the center of a circle for total stresses.

It is important to note that even if the radius has been canceled, the equation is valid only for circular slip surface.

If the friction angle is not equal to zero, the equation requires that the normal stress on the base of each slice be known hence the problem is statically indeterminate and requires additional assumptions. Different methods make different assumptions.

**Ordinary Method of Slices**

This procedure, that has also been referred to as the Swedish Method of Slices and the Fellenius Method, neglects the forces on the sides of the slice.

The normal force N is expressed by:

$$ N = W \cdot \cos \alpha$$

Therefore, the normal stress on the base of a slice is expressed by:

$$ \sigma = \frac{W \cdot \cos \alpha}{\Delta l \cdot 1}$$

$$ \sigma' = \frac{W \cdot \cos \alpha}{\Delta l \cdot 1} - u$$

Substituting this expression into equation (31) to give:

$$ F = \frac{\sum(c \cdot \Delta l + W \cdot \cos \alpha \cdot \tan \varphi)}{\sum W \cdot \sin \alpha}$$

The equation (35) expresses the factor of safety in terms of total stresses while when the shear stress is expressed in terms of effective stresses the equation becomes:

$$ F = \frac{\sum(c' + \sigma' \cdot \tan \varphi') \cdot \Delta l}{\sum W \cdot \sin \alpha}$$

$$ = \frac{\sum(c' \cdot \Delta l + (W \cdot \cos \alpha - u \cdot \Delta l) \cdot \tan \varphi')}{\sum W \cdot \sin \alpha}$$

Because of the assumption involved in the equation (33), this expression can lead to unrealistically low or negative values for the effective stresses on the slip surface then (Duncan & Wright, 2005) suggested an alternative expression for the factor of safety that improve the accuracy of the Ordinary Method of Slices for effective stress analyses.
RIVER EMBANKMENTS FAILURE CAUSES

SIMPLIFIED BISHOP PROCEDURE

According to this procedure the forces on the sides of the slice are assumed to be horizontal.

Referring to Figure 101 and resolving forces in the vertical direction, it is possible to write:

\[ N \cdot \cos \alpha + S \cdot \sin \alpha - W = 0 \]  

(38)

If the shear force \( S \) is expressed in terms of effective stresses with the Mohr–Coulomb equation, it results:

\[ S = \tau \cdot \Delta l = \frac{s \cdot \Delta l}{F} = \frac{1}{F} \left[ c' \cdot \Delta l + (N - u \cdot \Delta l) \cdot \tan \varphi' \right] \]  

(39)

Substituting this expression into equation (38) to give:

\[ N = \frac{W - 1/F \cdot (c' \cdot \Delta l - u \cdot \Delta l \cdot \tan \varphi')}{\cos \alpha + (\sin \alpha \cdot \tan \varphi')/F} \]  

(40)

The normal stress on the base of a slice is expressed by:

\[ \sigma' = \frac{N}{\Delta l \cdot 1 - u} \]  

(41)

By introducing equations (40) and (41) into the equation (36) we obtain the equation for the factor of safety for the simplified Bishop procedure:

\[ F = \sum \left[ \frac{c' \cdot \Delta l \cdot \cos \alpha + (W - u \cdot \Delta l \cdot \cos \alpha) \cdot \tan \varphi'}{\cos \alpha + (\sin \alpha \cdot \tan \varphi')/F} \right] \]  

\[ \sum W \cdot \sin \alpha \]  

(42)

Even if the Simplified Bishop procedure does not satisfy complete static equilibrium, it gives improved results over the Ordinary Method of Slices (especially when effective stresses are used and pore water pressure are high) and relatively accurate values for the factor of safety. On the other hand, the main practical limitation of the procedure is that it is restricted to circular slip surfaces.

(Bishop, 1955) presented also another procedure that considers all of the unknown forces acting on a slice and make assumptions to fully satisfy static equilibrium: the Complete Bishop procedure. Unfortunately, this second procedure has not been described completely (the specific assumptions were not stated).
Slopes can be subject to additional surcharge loads due to water or traffic, stockpiled materials or seismic loading. On the other hand, stability computations can include additional forces to represent reinforcements for reinforced slopes. However, the additional forces are known hence they can be included in the equilibrium equations without requiring any additional assumptions.

2.3.2.2. Noncircular Slip Surfaces

Many times slip surfaces have complex shapes because they follow weak interfaces between different soils or between soil and other material layers. Several procedures have been developed for analyses of these complex noncircular surfaces. All these procedures are procedures of slices. Some of them are based on satisfying only requirements of force equilibrium – force equilibrium procedures – while the others procedures consider all of the requirements for static equilibrium so these are known as complete equilibrium procedures.

2.3.2.2.1. Force Equilibrium Procedures

These procedures ignore moment equilibrium and use only the equations for equilibrium of forces in two mutually perpendicular directions. In order to obtain a statically determinate solution and compute the factor of safety, the forces on the base of each slice and the resultant interslice forces, the inclinations of the forces between each slice are assumed.
The interslice forces represent all the forces transmitted across a slice boundary: the effective stresses in the soil, the pore water pressures and, for reinforced slopes, the forces transmitted across the interslice boundaries through the reinforcing elements (such as geogrids, geotextiles, piles, soil nails, tieback anchors, etc).

**The Simplified Jambu Procedure** ([Janbu, Bjerrum, & Kjaersli, 1956]; [Jambu, 1973])

The Simplified Jambu procedure is based on the assumption that the interslices forces are horizontal hence there is no shear stress between slices. This assumption almost always produces factors of safety that are smaller than those obtained by using complete equilibrium procedures. Therefore, (Janbu, Bjerrum, & Kjaersli, 1956) proposed correction factors to increase the factor of safety to more reasonable values. These correction factors are based on 30 to 40 cases of slope stability computation analyses.

**U.S. Army Corps of Engineers Method** (USACE, 1970)

This procedure is based on the assumption that the interslice forces are parallel to average embankment slope.

The interslice force inclinations may be interpreted either as being the same for every slice or different from slice to slice but the standard practice is to assume that the interslice forces all have the same inclination.

Regardless of the interpretation, the procedure is unconservative: it can lead to factor of safety that are higher than those obtained by complete equilibrium procedures.

**Analytical solutions**

For an individual slice, considering positive the forces that act upward, summation of forces in the vertical direction gives:

\[
F_v + Z_i \cdot \sin \theta_i - Z_{i+1} \cdot \sin \theta_{i+1} - N \cdot \cos \alpha + S \cdot \sin \alpha = 0
\]  

(43)

Where:

- \(Z_i; Z_{i+1}\) = magnitudes of the interslice force respectively at the left and at the right of the slice \(i\);
- \(\theta_i; \theta_{i+1}\) = inclinations of the interslice force respectively at the left and at the right of the slice \(i\);
- \(F_v\) = sum of all known forces acting on the slice in the vertical direction. In the absence of any surface loads and reinforcement forces the magnitude of \(F_v\) is
equal to the weight of the slice \( W \).

Summation of forces in the horizontal direction gives:

\[
F_h + Z_i \cdot \cos \theta_i - Z_{i+1} \cdot \cos \theta_{i+1} - N \cdot \sin \alpha + S \cdot \cos \alpha = 0
\]  

(44)

Where:

- \( F_h \) = sum of all known forces in the horizontal direction. In the absence of any surface loads, reinforcement forces and seismic forces the force \( F_h \) is equal to zero.

By combining equations (43) and (44) with the Mohr–Coulomb equation for the shear force, it is possible to obtain:

\[
Z_{i+1} = F_o \cdot \sin \alpha + F_h \cdot \cos \alpha + Z_i \cdot \cos(\alpha - \theta) - [F_o \cdot \sin \alpha - F_h \cdot \sin \alpha + u \cdot \Delta \ell + Z_i \cdot \sin(\alpha - \theta)] \cdot (\tan \varphi' / F) + c' \cdot \Delta \ell / F
\]

(45)

Therefore, by assuming a trial value for \( F \), it is possible to calculate the interslice force \( Z_{i+1} \) on the right of the first slice where \( Z_i = 0 \). Proceeding on the next slice \( Z_i \) is equal to the value of \( Z_{i+1} \) calculated for the previous slice hence it is possible to calculate the interslice force on the right of the second slice and so on.

If the force \( Z_{i+1} \) on the right of the last slide is essentially zero than the value assumed for \( F \) is correct (it means that the slice is triangular) while if the force is not zero then a new trial value for \( F \) is assumed and the process is repeated.

2.3.2.2.2. Complete equilibrium procedures

There exist several procedures that make different assumptions to achieve a statically determinate solution.

**Spencer’s procedure (1967)**

Spencer’s procedure assumes that the normal force \( N \) acts at the center of the base of each slide and that the interslice forces are parallel but the inclination of these forces is unknown and is computed as one of the unknowns of the problem.

Two equilibrium equations that represent overall force and moment equilibrium for the entire soil are solved for the unknown factor of safety \( F \) and interslice force inclination \( \theta \):

\[
\sum Q_i = 0
\]

(46)

\[
Q_i = Z_i - Z_{i+1}
\]

(47)

Where:
\[ Q_i = \text{resultant of the interslice forces on the left, } Z_i \text{ and on the right of the slice } Z_{i+1} \text{ because of the assumption } Q_i, Z_i \text{ and } Z_{i+1} \text{ have the same direction.} \]

**Figure 102. Interslice forces and resultant when interslice forces are parallel (Figure 6.21 from (Duncan & Wright, 2005)).**

For moment equilibrium about the origin of a Cartesian coordinate system:

\[
\sum Q \cdot \left( x_B \cdot \sin \theta - y_Q \cdot \cos \theta \right) = 0 \quad (48)
\]

\[
y_Q = y_B + \frac{M_0}{Q \cdot \cos \theta} \quad (49)
\]

Where:
- \( x_B \) = horizontal coordinate of the center of the base of the slice;
- \( y_B \) = vertical coordinate of the center of the base of the slice;
- \( y_Q \) = vertical coordinate of the point on the line of action of the force \( Q \);
- \( M_0 \) = moment produced by any known force about the center of the slice. \( M_0 \) is zero and \( y_Q = y_B \) in the absence of seismic loads, surcharge loads and reinforcement forces.

For force equilibrium:

\[
N + F_v \cdot \cos \alpha - F_h \cdot \sin \alpha - Q \cdot \sin(\alpha - \theta) = 0 \quad (50)
\]

\[
S + F_v \cdot \sin \alpha + F_h \cdot \cos \alpha + Q \cdot \cos(\alpha - \theta) = 0 \quad (51)
\]

Where:
- \( F_h \) and \( F_v \) = respectively horizontal and vertical all known forces on the slice: weight, seismic loads, surcharge loads and reinforcement forces.

Combining these equations with the Mohr Coulomb equation for the shear force gives:

\[
Q = \frac{-F_v \cdot \sin \alpha - F_h \cdot \cos \alpha - (c' \Delta l/F) + (F_v \cdot \cos \alpha - F_h \cdot \sin \alpha + u\Delta l) \cdot (\tan \varphi')/F}{\cos(\alpha - \theta) + \frac{\sin(\alpha - \theta) \cdot \tan \varphi}{F}}
\]

(52)
By substituting equations (49) and (52) into the equations (46) and (48), we have two equations with two unknowns, \( F \) and \( \theta \), that can be solved by using trial–and–error procedures.

**Figure 103. Coordinates for noncircular slip surface (left) and slice with all known and unknown forces (right) for Spencer’s procedure (Figures from (Duncan & Wright, 2005)).**

**Morgenstern and Price Procedure (1965)**

It assumes that the shear forces between slices are related to the normal forces as:

\[
X = \lambda \cdot f(x) \cdot E
\]

Where:
- \( X \) = vertical forces between slices;
- \( E \) = horizontal forces between slices;
- \( \lambda \) = unknown scaling factor;
- \( f(x) \) = assumed function.

The procedure assumes the location of the normal force on the base of the slice. Typically it is assumed at the midpoint of the base of the slice or at a point on the base of the slice that is directly below the center of gravity.

The main difference between Spencer’s and Morgenstern and Price’s procedures is that the first one provides additional flexibility for the interslice force inclination assumptions.

However, there is little practical difference among Spencer’s and Morgenstern and Price’s procedure.

Moreover, if the function \( f(x) \) is assumed to be constant, Morgenstern and Price’s procedure produces results identical to those produced by Spencer’s procedure.
**Chen and Morgenstern Procedure**

It is a refinement of the Morgenstern and Price’s procedure. In fact, it attempts to account better for the stresses at the ends of the slip surface by assuming that at the ends of the slip surface the interslice forces become parallel to the slope. The procedure restricts the range of possible solution.

The equation (53) becomes:

\[ X = [\lambda \cdot f(x) + f_0(x)] \cdot E \]  

Where:

- \( f(x) \) = assumed function that defines the distribution of the interslice force inclination that is zero at each end of the slip surface;
- \( f_0(x) \) = assumed function that defines the distribution of the interslice force inclination that is equal to the tangent of the slope inclination at each end of the slip surface.

**Sarma’s Procedure (Sarma, 1973)**

This method was developed for assessing seismic stability. It considers the seismic coefficient \((k)\) to be unknown while the factor of safety \(F\) is considered to be known. Therefore, \(F\) is assumed to be one and then the seismic coefficient that is calculated represents the seismic coefficient required to cause sliding.

The shear force between slices is related to shear strength by:

\[ X = \lambda \cdot f(x) \cdot S_v \]  

Where:

- \( \lambda \) = unknown scaling parameter;
- \( f(x) \) = assumed function with prescribed values at each vertical slice boundary;
- \( S_v \) = available shear force on the slice boundary.

All of the complete equilibrium procedures give very similar value for the factor of safety.

Morgenstern and Price’s and Chen and Morgenstern’s procedures are the most rigorous and flexible of the complete equilibrium procedures. They are particularly useful for cases where interslice forces can have a significant effect on stability such as when the slip surface is forced to change abruptly direction because of the geometry and properties of the slope (Duncan & Wright, 2005).
2.3.3. Pore Water Pressure Representation

In the slope stability analyses it is necessary to determine and represent the pore water pressures.

An approximate representation method consists of defining the pore water pressures using a line that represents a phreatic surface that corresponds to the line of seepage from a flow net hence it is both a flow line and a line of zero pressure. The pore water pressure is equal to:

\[ u = h_p \cdot \gamma_w \]  

Where:
- \( h_p \) = pressure head;
- \( \gamma_w \) = unit weight of water.

If the phreatic surface line and the equipotential lines are straight lines, the pressure head is equal to:

\[ h_p = z_p \cdot (\cos i)^2 \]  

Where:
- \( z_p \) = vertical depth below the phreatic surface;
- \( i \) = slope of the phreatic surface.

While, if the phreatic surface and the equipotential lines are curved, we can write:

\[ z_p \cdot \gamma_w \cdot (\cos \alpha')^2 < u < z_p \cdot \gamma_w \cdot (\cos \alpha'')^2 \]  

Where:
- \( \alpha' \) = slope of the phreatic surface at the point where the equipotential line intersects the phreatic surface;
- \( \alpha'' \) = slope of the phreatic surface above the point of interest.

If \( \alpha'' < \alpha' \), it is conservative to consider:

\[ u = z_p \cdot \gamma_w \cdot (\cos \alpha'')^2 \]  

A more simplified method consists of considering a piezometric line. Therefore, the pore water pressures are computed by:

\[ u = z_p \cdot \gamma_w \]  

Where:
- \( z_p \) = vertical depth below the piezometric line.

Differences between the two representations, by phreatic surface and by piezometric line, are typically small because generally \( \alpha \) is small.

(Duncan & Wright, 2005) found that when flow is predominately horizontal, both a phreatic surface and a piezometric line can be used to approximate the pore water pressures with an error of only a few percent. On the other hand, as the component
of flow and head loss in the vertical direction increases, neither a phreatic surface nor a single piezometric line represent pore water pressures well and both may result in errors on the unsafe side.

Most analyses of seepage and groundwater flow are conducted using the finite element method. Their results consist of pore water pressure values at each nodal point in the finite element mesh. These values are used with a suitable interpolation scheme to calculate the pore water pressures at the center of the base of individual slices along the slip surface.

While most earliest finite element modeling schemes modeled only the region of saturated flow below the line of seepage (they assumed no flow above the line of seepage), current schemes model the entire section including the region above the seepage line where the pore pressures are negative and the soil may be unsaturated. However, even if both positive and negative pore water pressure values are calculated, for slope stability analyses, in the region where negative pressures have been calculated, the pore water pressures are usually assumed equal to zero.

To interpolate pore water pressures from nodal point values to points along a slip surface, interpolation schemes are available such as three and four point interpolation, spline interpolation finite element shape functions and triangular irregular network scheme (Duncan & Wright, 2005).

2.3.4. Stability analyses of Serchio River Embankments

The results of the geotechnical characterization described in § 1.1 were used to define a stratigraphic and geotechnical model in order to carry out some stability analyses.

In fact, following the geotechnical characterization of 30 km long floodplain embankments of the Serchio River (Tuscany, Italy), a number of analyses were carried out for various purposes: to clarify the causes of the December 2009 failures; to design appropriate repair of the failures and retrofit of the embankments in proximity of the failures and to identify the riskiest areas considering the whole extension of the embankments.

Assessment of the December 2009 failure causes was necessary in order to plan the repair works and the measures to be taken to improve the safety conditions near those failures: about three km of embankments in the Lucca District.
The investigations in the remaining part of the embankments, about 30 km, were aimed at individuating the riskiest areas in order to plan the economic resources for later improvements.

The case study refers to floodplain embankments that did not experience any overtopping hence the possible failure causes could be restricted to mechanical instability, heave failure or internal erosion.

The stability analyses were carried out considering both stationary flow and limit equilibrium method and non-stationary flow and Finite Element Method.

More specifically the following ultimate limit states were considered:

- Slope failure and hydraulic heave (under steady-state flow conditions) by the Limit Equilibrium Method. The stability analyses were carried out using the Bishop simplified method with circular sliding surfaces and using three different commercial codes. Safety factor against hydraulic heave was computed according to (NTC, 2008) (§ 3.2.1).

- Slope failure under transient flow conditions by the FEM analysis (Plaxis Flow, 2011) in order to assess the necessary time to approach the steady-state flow conditions. For the case under consideration it was estimated that 10 days are necessary to approach the steady-state flow conditions. This time is apparently much longer than the duration of the longest flood event (few hours). However, for the failures under consideration, that occurred with the concurrence of various adverse factors such as the melting of the snow because of a sudden temperature increase and the contemporary long raining period (two consecutive floods), it is reasonable to assume that the permanent flow conditions were probably reached.

2.3.4.1. LIMIT EQUILIBRIUM METHOD ANALYSIS

The limit equilibrium method was used to assess the stability of the embankments under steady-state flow conditions for the areas close to the failures.

A typical cross section of the embankment in proximity of the December 2009 failures and the stratigraphic/geotechnical model that have been adopted for the limit equilibrium analyses are shown by Figure 104.
FIGURE 104. TYPICAL CROSS SECTION OF THE SERCHIO RIVER EMBANKMENT IN PROXIMITY OF THE DECEMBER 2009 FAILURES (SLIDE VERSION 6.0., 2010). THE SITUATION DEPICTED IN THE FIGURE IS REPRESENTATIVE OF ALL ANALYSED.

FIGURE 105. SEEPAGE STUDY FOR THE SECTION DEPICTED BY FIGURE 104 (SLIDE VERSION 6.0., 2010).

The stability analyses were carried out using the Bishop simplified method with circular sliding surfaces and considering stationary flow with the water level coincident with the embankment crown on the riverside and with the groundwater...
line on the country side. Under these conditions safety factors of less than one were obtained.

![Stability Analysis](image)

**Figure 106. Stability analysis for the section depicted by Figure 104 (SLIDE version 6.0., 2010).**

The stability of soil against heave was also checked. Failure by heave occurs when upwards seepage forces act against the weight of the soil, reducing the vertical effective stress to zero. The safety conditions against potential heave failures do not respect the prescriptions of the Italian code (NTC, 2008). In fact, the water pressure on the country side may be higher than the total vertical geostatic stress not respecting the equation (NTC, 2008) (§ 3.2.1):

\[ 1.3 \cdot u \leq 0.9 \cdot \sigma_{v0} \]

(61)

Where:

\( u \) = pore pressure;

\( \sigma_{v0} \) = total vertical stress.

The above condition was checked at the nodes of the mesh used for seepage calculation and it was not verified at the country side embankment toe. It can be concluded, that the stability, both mechanical and hydraulic, of these embankments becomes critical under the circumstance of stationary flow which implies complete soil saturation.
In the case of steady–state flow, safety factor drastically reduces becoming lower than one. This result apparently contradicts the fact that the considered embankments are standing up since centuries and failures occurred only in the occasion of some floods. Therefore, the validity of stationary flow condition was analysed in the light of the results obtained with FEM analyses (PLAXIS, 2010) in order to clarify how much this hypothesis was realistic.

2.3.4.2. FEM ANALYSIS

In order to investigate the influence of the advance of the saturation front within the embankment body, some numerical analyses using (Plaxis Flow, 2011) module of (PLAXIS, 2010) were carried out.

To achieve the described goal, a Mohr–Coulomb constitutive model and the strength parameters used for the LEM stability analyses described above were used. The material stiffness was chosen very high in order to stress the influence of partial saturation in slope stability and compare the results with LEM analyses.

Since a characterization of partially saturated soil was not available, the default parameters proposed by the software library were used. The default parameters were grain size distribution depending.
In spite of the software capability, the transient flow was analysed with reference to the time history of the flood plain level. The flood plain level was considered at its maximum level for duration of 10 days. This time is longer than the real flood duration for the Serchio River, which is few hours long. This duration was fixed in agreement with Lucca District Officers in order to apply a sort of “stress test” for the embankments. In fact, as reported above, all the sections analysed in steady-state condition, indicate a global safety factor lower than one so it was not possible to define a list of priority for the interventions.

The situation depicted in Figure 108 is representative of all analysed sections. The blue line indicates the upper limit of the saturated zone, whereas the color map indicates the mobilized shear strength (dark red, full mobilized). The body of embankment remains unsaturated and the color map indicates the absence of collapse mechanism. The $\phi$–$c$ reduction procedure indicates a safety factor greater than one, even if this information has only a relative meaning.

**Figure 108. Results of transient flow analysis for section 9 RB L (PLAXIS, 2010).**
It is worthwhile to remark that the embankments under consideration survived to most of the past events but not to the 2000 and 2009 floods (Table 1). As described in §1.1.4, the flood in December 2009 was quite usual in term of water discharge. In fact, during the event the maximum water level never reached the crest of the embankment but the peculiar characteristic of the event was the occurrence of two closer floods (two floods in two weeks) and a rare snowfall. In the light of the analyses that have been reported above, the concurrence of all these adverse factors caused the failures. By the way, the occurrence of two floods in ten days in November 2000 caused the embankment failures in the same areas as in December 2009. It seems that the embankment saturation degree is more relevant than the river discharge.

The FEM analysis shows that the safety margins of the considered sections, in absence of filtration, are assigned to the partial saturation of the embankment. Unfortunately, an appropriate characterization of the material under conditions of partial saturation was not available.

For the case under consideration it was estimated that 10 days are necessary to approach the steady state flow conditions. This time is apparently much longer than the duration of the longest flood event (few hours). Therefore, it is possible to conclude that the hypothesis of permanent flow is generally too cautious. However,
for the failures under consideration, that occurred with the concurrence of various adverse factors, like the melting of the snow because of a sudden temperature increase and the contemporary long raining period, it is reasonable to assume that the permanent flow conditions were probably reached.

2.3.4.3. Countermeasures

In order to have a rapid increment of safety degree against floods, the Lucca District Officers proposed to install a metallic sheet pile diaphragm within the body of the embankment.

The sheet pile barrier installation was decided to prevent piping phenomena and to reduce the risk of mechanical and hydraulic embankment failures. In addition, since the bank was eroded after the failures, the sheet pile should guarantee the provisional territory protection.

The embankment extension to be consolidated (about 3 km in the Lucca District) was decided on the basis of vulnerability criteria, whose aim was to protect the most vulnerable part of the territory.

Therefore, FEM analyses were carried out in order to optimize the diaphragm height. As shown by Figure 110, the effect of the diaphragm was a reduction of saturation zone by about one meter. This effect was practically the same even changing the height of the diaphragm because of the presence of a high permeability stratum at shallow depth. Since the diaphragm base was inserted in this stratum, any increment of diaphragm height was ineffective. Therefore, it was decided to install a 10 m deep sheet pile barrier along the embankment, located at the centre of the embankment cross section. Moreover, the cross-section geometry was modified. In Figure 111 the situation for the refurbished section 25 is reported, in which is evident the addiction of a berm. The color map indicates the degree of saturation.

The embankments have been designed to protect the territory against a flood with a 30 years return period hence overtopping should only occur for floods with higher return periods.
2.3.4.4. Mapping riskiest areas

In order to individuate the riskiest areas, for the embankment portion not yet considered, expeditious criteria were used. In particular, the critical cross sections to be analysed were selected both on the basis of geometric criteria and considering the soil strength as inferred from CPTu.

The geometric criterion can be summed up by the ratio between the embankment height and its base length: the higher the ratio, the higher the failure risk.

As far as the soil strength is concerned, Figure 112 shows the minimum and maximum envelope of CPTu tip resistance that was carried out in the 3 km of embankments close to the December 2009 failures. Since this part of embankment was recognized poorly constructed, it was assumed as reference. The criterion
adopted to define the potential risk of a given portion of embankments on the basis of soil strength is shown by Figure 113.

**Figure 112. Minimum and maximum envelope for QT.**

**Figure 113. Exemplification of second criterion application. Risky section (left), Not risky section (right).**

The mapping of the risk areas based on the above criteria is shown by Figure 114.
The same analysis procedures above described were applied to evaluate refurbishment measures for the embankments. In order to obtain a list of priority a stress test was applied to various sections in the risk areas. In particular, the flow conditions were considered both as stationary and transient.

Limit equilibrium analyses were repeated for the risk areas considering some critical cross sections, Bishop simplified method with circular sliding surfaces, steady–state flow condition and the water table elevation coincident with the embankment crest. For these steady–state analyses three different types of commercial software were used: SLIDE (SLIDE version 6.0., 2010), both the codes SLOPE/W & SEEP/W (GeoStudio, 2007) and PC STABL 5M (Achilleos, 1988). The analysis results show that, for the selected cross sections, the safety factors are lower than one. The different codes indicated very similar failure surfaces corresponding to the minimum (meaningful) values of the global safety factor. Some differences on the values of the global safety factor were observed by comparing the results obtained from the three codes. More specifically, the analysis results show that, for the selected cross sections, the safety factors are rather small and approaching to unity, if the seepage forces are not considered (the old version of PC STABL 5M (Achilleos, 1988) does not take into account the seepage).
Table 24. Steady state analyses: values of safety factor by different software.

<table>
<thead>
<tr>
<th>SECTION</th>
<th>SLIDE</th>
<th>PC - STABL</th>
<th>SLOPE/W</th>
</tr>
</thead>
<tbody>
<tr>
<td>9 rb L</td>
<td>0.621</td>
<td>0.76</td>
<td>0.729</td>
</tr>
<tr>
<td>13 rb L</td>
<td>0.677</td>
<td>0.87</td>
<td>0.655</td>
</tr>
<tr>
<td>25 rb L</td>
<td>0.560</td>
<td>0.62</td>
<td>0.640</td>
</tr>
<tr>
<td>39 lb L</td>
<td>0.819</td>
<td>0.89</td>
<td>0.898</td>
</tr>
<tr>
<td>51 lb L</td>
<td>0.676</td>
<td>0.71</td>
<td>0.710</td>
</tr>
<tr>
<td>57 lb L</td>
<td>0.746</td>
<td>0.78</td>
<td>0.798</td>
</tr>
<tr>
<td>4 lb P</td>
<td>1.264</td>
<td>1.40</td>
<td>1.318</td>
</tr>
<tr>
<td>27 lb P</td>
<td>1.047</td>
<td>1.19</td>
<td>1.132</td>
</tr>
<tr>
<td>32 lb P</td>
<td>0.728</td>
<td>0.99</td>
<td>0.760</td>
</tr>
<tr>
<td>48 lb P</td>
<td>0.794</td>
<td>1.02</td>
<td>0.828</td>
</tr>
<tr>
<td>51 lb P</td>
<td>0.761</td>
<td>1.09</td>
<td>0.763</td>
</tr>
<tr>
<td>57 lb P</td>
<td>0.905</td>
<td>0.93</td>
<td>0.906</td>
</tr>
</tbody>
</table>

For the examined cases the two different hypotheses, steady–state and transient flow, change dramatically the results. In the first case none of examined sections can sustain the flow. This result is not consistent with experimental evidence. In fact, even if the XVIII century embankments are quite weak, they have been often damaged by flood and often they have endured, with some few exceptions. As a consequence a steady–state can be considered too much conservative or, more appropriately, inapplicable to this case.

A more plausible result was obtained by means of transient flow analyses. Even if they were carried out in lack of information about partially saturation behavior of soil, the analyses show that the embankments owe their resistance to partial saturation.

NOTES: L = Lucca District; P = Pisa District; LB = Left Bank; RB = Right Bank.
Figure 115. Section 9rb L: Results of the stability analyses carried out using PC STABL (ACHILLEOS, 1988).

Figure 116. Seepage study carried out using Slide (SLIDE version 6.0., 2010).
FIGURE 117. SECTION 9 RB L: RESULTS OF THE STABILITY ANALYSES CARRIED OUT USING SLIDE (SLIDE VERSION 6.0., 2010).

FIGURE 118. SECTION 9 RB L: RESULTS OF THE STABILITY ANALYSES CARRIED OUT USING BOTH SEEP/W AND SLOPE/W.
3. TECHNICAL CODES

3.1. EUROCODE 7 (EN 1997-1, 2004)

The Eurocode 7 deals with levee in Section 12: "Embankments".

The provisions of this section shall apply to embankments for small dams and for infrastructure. For placement and compaction of fill the provisions in Section 5 "Fill, dewatering, ground improvement and reinforcement" should be applied.

A list shall be compiled of limit states to be checked in the design of the embankment. The limit states and the actions summed up by Table 25 should be taken into account.

<table>
<thead>
<tr>
<th>DESIGN OF EMBANKMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>LIMIT STATES</strong></td>
</tr>
<tr>
<td>✓ LOSS OF OVERALL SITE STABILITY</td>
</tr>
<tr>
<td>✓ FAILURE IN THE EMBANKMENT SLOPE OR CREST</td>
</tr>
<tr>
<td>✓ FAILURE CAUSED BY INTERNAL EROSION</td>
</tr>
<tr>
<td>✓ FAILURE CAUSED BY SURFACE EROSION OR SCOUR</td>
</tr>
<tr>
<td>✓ SETTLEMENTS AND CREEP DISPLACEMENTS LEADING TO DAMAGES OR LOSS OF SERVICEABILITY IN NEARBY STRUCTURES OR UTILITIES</td>
</tr>
<tr>
<td>✓ EXCESSIVE DEFORMATIONS</td>
</tr>
<tr>
<td>✓ CREEP IN SLOPES DURING THE FREEZING AND THAWING PERIOD</td>
</tr>
<tr>
<td>✓ DEFORMATIONS CAUSED BY HYDRAULIC ACTIONS</td>
</tr>
<tr>
<td>✓ CHANGES OF ENVIRONMENTAL CONDITIONS</td>
</tr>
<tr>
<td>✓ IMPOSED PRE-STRESS IN GROUND ANCHORS OR STRUTS</td>
</tr>
</tbody>
</table>
GUIDELINES FOR THE GEOTECHNICAL DESIGN, UPGRADING AND REHABILITATION OF RIVER EMBANKMENTS

BARBARA COSANTI

- Temperature effects, including frost action and ice loading
- Movements due to creeping or sliding or settling ground masses
- Movements due to degradation, dispersion, decomposition, self-compaction and solution
- Movements caused by mining or other caving or tunnelling activities
- Movements and accelerations caused by earthquakes, explosions, vibrations and dynamic loads
- Mooring forces
- Traffic loads
- Downdrag

Table 25. Design of embankments: limit states and action to take into account.

The most unfavourable hydraulic conditions should be considered (normally steady seepage for the highest possible ground-water level and rapid draw-down of the free water level).

"Design of embankments should ensure that:
- the bearing capacity of the subsoil is satisfactory;
- the drainage of the various fill layers is satisfactory;
- the permeability of the fill material in dams is as low as required;
- filters or geosynthetics are specified where necessary to fulfil filter criteria;
- the fill material is specified according to the criteria in 5.3.2."

Paragraph 5.3.2. states:

"The criteria for specifying material as suitable for use as fill shall be based on achieving adequate strength, stiffness, durability and permeability after compaction. These criteria shall take account of the purpose of the fill and the requirements of any structure to be placed on it."

Embankment slope surfaces exposed to erosion shall be protected. If berms are designed, a drainage facility shall be specified for the berm.
In analysing the stability, all possible failure modes shall be considered, as stated in Section 11 "Overall stability". The limit states, actions and design situations summed up by Table 26 should be taken into account.

<table>
<thead>
<tr>
<th>OVERALL STABILITY</th>
<th>LIMIT STATES</th>
<th>ACTIONS</th>
<th>DESIGN SITUATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>LOSS OF OVERALL STABILITY</td>
<td>of the ground and associated structures</td>
<td>See Table 25</td>
<td>NEW SLOPES OR STRUCTURES ON OR NEAR THE PARTICULAR SITE</td>
</tr>
<tr>
<td>EXCESSIVE MOVEMENTS IN THE</td>
<td>GROUND DUE TO SHEAR DEFORMATIONS, SETTLEMENT, VIBRATION OR HEAVE</td>
<td></td>
<td>PREVIOUS OR CONTINUING GROUND MOVEMENTS FROM DIFFERENT SOURCES</td>
</tr>
<tr>
<td>DAMAGE OR LOSS OF Serviceability in neighbouring structures, roads or services due to movements in the ground</td>
<td></td>
<td></td>
<td>CLIMATIC VARIATIONS: TEMPERATURE CHANGE (FREEZING AND THAWING), DROUGHT AND HEAVY RAIN</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The overall stability of slopes including existing, affected or planned structures shall be verified in ultimate limit states GEO and STR\(^3\) with design values of actions, resistances and strengths. The values of the partial factors may be set by the National annex; the recommended values for persistent and transient situations are given in Tables A.3, A.4 and A.14 (Figure 119).

\(^3\) GEO: failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance; STR: internal failure or excessive deformation of the structure or structural elements, in which the strength of structural materials is significant in providing resistance (EN 1997-1, 2004).
Table A.3 - Partial factors on actions ($
\gamma$) or the effects of actions ($\gamma_i$)

<table>
<thead>
<tr>
<th>Action</th>
<th>Symbol</th>
<th>Set</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unfavourable</td>
<td>$\gamma$</td>
<td>1,35</td>
</tr>
<tr>
<td>Favourable</td>
<td></td>
<td>1,0</td>
</tr>
<tr>
<td>Variable</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unfavourable</td>
<td>$\gamma_v$</td>
<td>1,3</td>
</tr>
<tr>
<td>Favourable</td>
<td></td>
<td>0</td>
</tr>
</tbody>
</table>

Table A.4 - Partial factors for soil parameters ($\gamma_s$)

<table>
<thead>
<tr>
<th>Soil parameter</th>
<th>Symbol</th>
<th>Set</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle of shearing resistance</td>
<td>$\gamma_c$</td>
<td>1,0</td>
</tr>
<tr>
<td>Effective cohesion</td>
<td>$\gamma_c$</td>
<td>1,0</td>
</tr>
<tr>
<td>Undrained shear strength</td>
<td>$\gamma_u$</td>
<td>1,0</td>
</tr>
<tr>
<td>Unconfined strength</td>
<td>$\gamma_u$</td>
<td>1,0</td>
</tr>
<tr>
<td>Weight density</td>
<td>$\gamma_i$</td>
<td>1,0</td>
</tr>
</tbody>
</table>

* This factor is applied to $\tan \phi$

Table A.14 - Partial resistance factors ($\gamma_R$) for slopes and overall stability

<table>
<thead>
<tr>
<th>Resistance</th>
<th>Symbol</th>
<th>Set</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earth resistance</td>
<td>$\gamma_{re}$</td>
<td>1,0</td>
</tr>
</tbody>
</table>

To avoid ultimate limit states caused by surface erosion, internal erosion or hydraulic pressure, the provisions in Sections 10 and 11 shall be fulfilled.

The provisions in Section 10 "Hydraulic failure" apply to failure by uplift (buoyancy); by heave; by internal erosion and by piping. In situations where the pore-water pressure is hydrostatic (negligible hydraulic gradient) it is only required to check failure by uplift. The determination of hydraulic gradients, pore-water pressures or seepage forces shall take account of the variations of soil permeability; variations in water levels and pore-water pressure in time and any modification of the boundary conditions.
Total stress analysis is applied to failure by uplift; both total and effective stresses are applied for failure by heave whereas conditions are put on hydraulic gradients in order to control internal erosion and piping.

When hydraulic heave, piping or internal erosion are significant dangers to the integrity of the structure, measures shall be taken to decrease the hydraulic gradient (§ 6).

**Failure by Uplift (UPL)**

"The stability of a structure or of a low permeability ground layer against uplift shall be checked by comparing the permanent stabilising actions (for example, weight and side friction) to the permanent and variable destabilising actions from water and, possibly, other sources."

Verification for uplift shall be carried out by checking that the design value of the combination of destabilising permanent and variable vertical actions \( V_{dst,d} \) is less than or equal to the sum of the design value of the stabilising permanent vertical actions \( G_{stb,d} \) and of the design value of any additional resistance to uplift \( R_d \):

\[
V_{dst,d} \leq G_{stb,d} + R_d \quad (62)
\]

Where:

\[
V_{dst,d} = G_{dst,d} + Q_{dst,d} \quad (63)
\]

- \( G_{dst,d} \) = design value of the destabilising permanent actions for uplift verification;
- \( Q_{dst,d} \) = design value of the destabilising variable vertical actions for uplift verification.

The values of the partial factors may be set by the National annex; tables A.15 and A.16 give the recommended values (Figure 120).

<table>
<thead>
<tr>
<th>Action</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent</td>
<td>( \gamma_{st} )</td>
<td>1.0</td>
</tr>
<tr>
<td>Unfavourable(^a)</td>
<td>( \gamma_{st} )</td>
<td>0.9</td>
</tr>
<tr>
<td>Favoured(^b)</td>
<td>( \gamma_{st} )</td>
<td>1.5</td>
</tr>
</tbody>
</table>

\(^a\) Destabilising; \(^b\) Stabilising

**Table A.15 - Partial factors on actions (\( \gamma \))**

**Table A.16 - Partial factors for soil parameters and resistances**

<table>
<thead>
<tr>
<th>Soil parameter</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle of shearing resistance(^b)</td>
<td>( \gamma )</td>
<td>1.25</td>
</tr>
<tr>
<td>Effective cohesion</td>
<td>( \gamma )</td>
<td>1.25</td>
</tr>
<tr>
<td>Undrained shear strength</td>
<td>( \gamma_0 )</td>
<td>1.40</td>
</tr>
<tr>
<td>Tensile pie resistance</td>
<td>( \gamma_t )</td>
<td>1.40</td>
</tr>
<tr>
<td>Anchorage resistance</td>
<td>( \gamma_a )</td>
<td>1.40</td>
</tr>
</tbody>
</table>

\(^b\) This factor is applied to \( \tan \varphi \)

**Figure 120. Uplift: recommended values for the partial factors (EN 1997-1, 2004).**
**Failure by heave (HYD)**

The stability of soil against heave shall be checked by verifying, for every relevant soil column, that the design value of the destabilising total pore water pressure \(U_{dst,d}\) at the bottom of the column, or the design value of the seepage force \(S_{dst,d}\) in the column is less than or equal to the stabilising total vertical stress \(\sigma_{stb,d}\) at the bottom of the column, or the submerged weight \(G'_{stb,d}\) of the same column:

\[
\begin{align*}
    U_{dst,d} & \leq \sigma_{stb,d} \\
    S_{dst,d} & \leq G'_{stb,d}
\end{align*}
\]

Equation (64) expresses the condition for stability in terms of pore-water pressures and total stresses while equation (65) expresses the same condition in terms of seepage forces and submerged weights.

The values of the partial factors may be set by the National annex; table A.17 gives the recommended values (Figure 121).

The determination of the characteristic value of the pore-water pressure shall take into account all possible unfavourable conditions (such as thin layers of soil of low permeability).

"Where the soil has a significant cohesive shear resistance, the mode of failure changes from failure by heave to failure by uplift. (...) Stability against heave will not necessarily prevent internal erosion, which should be checked independently, when relevant."

<table>
<thead>
<tr>
<th>Action</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unfavourable a</td>
<td>(\gamma_{U_{dst}})</td>
<td>1.35</td>
</tr>
<tr>
<td>Favourable b</td>
<td>(\gamma_{U_{stb}})</td>
<td>0.90</td>
</tr>
<tr>
<td>Variable</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unfavourable a</td>
<td>(\gamma_{S_{dst}})</td>
<td>1.50</td>
</tr>
<tr>
<td>a Destabilising</td>
<td></td>
<td></td>
</tr>
<tr>
<td>b Stabilising</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table A.17 - Partial factors on actions \(\gamma\)**

*Figure 121. Heave: recommended values for the partial factors (EN 1997-1, 2004).*

**Failure by internal erosion**

Filter criteria shall be used to limit the danger of material transport by internal erosion.
Filter protection should generally be provided by use of natural non-cohesive soil that fulfils adequate design criteria for filter materials. (...) Alternatively, artificial filter sheets such as geotextiles may be used provided it can be established that they sufficiently prevent transport of fines.

If the filter criteria are not satisfied, it shall be verified that the design value of the hydraulic gradient is well below the critical hydraulic gradient at which soil particles begin to move. The critical hydraulic gradient shall be established taking into account at least the direction of flow; grain size distribution and shape of grains and stratification of the soil.

**Failure by piping**

Prescriptive measures shall be taken to prevent the onset of the piping process, either by the application of filters or by taking structural measures, such as berms or impermeable screens below the base of the embankment, to control or to block the ground-water flow. Failure by piping shall be prevented by providing sufficient resistance against internal soil erosion in the areas where water outflow may occur. During periods of extremely unfavourable hydraulic conditions such as floods, areas susceptible to piping shall be inspected regularly.

**Serviceability limit state**

As far as the serviceability limit state is concerned the design shall show that the deformation of the embankment will not cause a serviceability limit state in the embankment or in other structures. The possibility of deformations due to changes in the ground-water conditions should be taken into account and in cases where the deformations are difficult to predict, the methods of pre-loading or trial embankments should be considered.

**Monitoring**

Monitoring of embankments should be applied where the stability of an embankment acting as a dam to a large degree depends on the pore-water pressure distribution in and beneath the embankment and where surface erosion is a considerable risk.

"The construction of embankments on soft soil with low permeability should be monitored and controlled by means of pore-water pressure measurements in the soft layers and settlement measurements of the fill."
A monitoring programme for an embankment should contain the records that are summed up by Table 27.

<table>
<thead>
<tr>
<th>MONITORING PROGRAMME FOR EMBANKMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>✓ PORE-WATER PRESSURE MEASUREMENTS (IN AND BENEATH THE EMBANKMENT)</td>
</tr>
<tr>
<td>✓ SETTLEMENT MEASUREMENTS FOR THE WHOLE OR PARTS OF THE EMBANKMENT AND INFLUENCED STRUCTURES</td>
</tr>
<tr>
<td>✓ HORIZONTAL DISPLACEMENT MEASUREMENTS</td>
</tr>
<tr>
<td>✓ CHECKS ON STRENGTH PARAMETERS OF FILL MATERIAL DURING CONSTRUCTION</td>
</tr>
<tr>
<td>✓ CHECKS ON PERMEABILITY OF FILL MATERIAL AND OF FOUNDATION SOIL DURING CONSTRUCTION</td>
</tr>
<tr>
<td>✓ OBSERVATIONS OF EROSION PROTECTION</td>
</tr>
<tr>
<td>✓ DEPTH OF FROST PENETRATION IN THE CREST OF EMBANKMENTS</td>
</tr>
<tr>
<td>✓ CHEMICAL ANALYSES BEFORE, DURING AND AFTER CONSTRUCTION, IF POLLUTION CONTROL IS REQUIRED</td>
</tr>
</tbody>
</table>

Table 27. Monitoring programme for embankments.
3.2. **ITALIAN DESIGN CRITERIA**

Table 28 sums up the Italian regulations dealing with river embankment design and construction that have been issued over time. It is possible to observe that many of them were issued after catastrophic flood events. Unfortunately, most important lessons are learned from catastrophic experiences.

Table 29 sums up the main provisions introduced by the listed regulations. The Italian code in force, NTC2008, is dealt with in the following paragraph.

<table>
<thead>
<tr>
<th>YEAR</th>
<th>ITALIAN TECHNICAL REGULATIONS</th>
</tr>
</thead>
</table>
| 1873 | CIRCOLARE DEL 12 FEBBRAIO 1873 N°3651/2200 EMANATA DAL MINISTERO LL.PP.  
      *(AFTER FLOOD EVENT IN OCTOBER 1872).* |
| 1904 | T.U. SULLE OPERE IDRAULICHE (RD 25/7/1904 N.593, ART. 96, LETTERA F). |
| 1929 | ISTRUZIONI GENERALI DEL CIRCOLO SUPERIORE DI ISPEZIONE PER IL PO DELL’APRILE 1929.  
      *(AFTER FLOOD EVENT IN MAY-JUNE, 1926).* |
| 1952 | CIRCOLARE DEL 25 LUGLIO 1952 EMANATA DAL CIRCOLO SUPERIORE DI ISPEZIONE PER IL PO.  
      *(AFTER FLOOD EVENT IN OCTOBER 1951).* |
| 1982 | D.M. 24/03/1982, PUNTO H: “DIGHE IN MATERIALI SCIOLTÌ”; |
| 1982 | PIANO S.I.M.P.O.1982 DELL’AUTORITÀ DI BACINO DEL PO. |
| 1998 | LINEE GUIDA PER L’ESECUZIONE DEGLI INTERVENTI DIADEGUAMENTO DELLE ARGINATURA DI PO IN CORSO DI ESECUZIONE E DI PROGETTAZIONE ”DEL MAGISTRATO PER IL PO DEL 22 OTTOBRE 1998. |
| 1999 | INTEGRAZIONE DELLE LINEE GUIDA IN SEGUITO ALLA CONVENZION EN°5984 DEL 5 LUGLIO 1999 DEL MAGISTRATO DEL PO IN COLLABORAZIONE CON LE UNIVERSITÀ DEGLI STUDI DI PARMA, DI BRESCIA, DI ROMA “LA SAPIENZA” E DI NAPOLI “FEDERICO II”. |

*Table 28. Italian technical regulations dealing with river embankment design and construction.*
<table>
<thead>
<tr>
<th>GUIDELINES FOR THE GEOTECHNICAL DESIGN, UPGRADING AND REHABILITATION OF RIVER EMBANKMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CREST LEVEL → FLOOD AGAINST WHICH THE EMBANKMENT PROVIDES PROTECTION</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td><strong>FREEBOARD</strong></td>
</tr>
<tr>
<td><strong>CREST WIDTH</strong></td>
</tr>
</tbody>
</table>
| **EMBANKMENT SLOPES** | **LANDWARD**: 2:1 (H:V)  
**RIVERWARD**: 3:2 out of the water and 2:1 inside water (H:V) |
| **BERMS** | 6–10 m width;  
Distance from one to another: 3 m |
| **SURFACE PROTECTION** | Riprap |
| **MATERIALS** | Any provision |

**ISTRUZIONI GENERALI DEL CIRCOLO SUPERIORE DI ISPEZIONE PER IL PO DELL’APRILE 1929.**

**LANDWARD SLOPE** | It must cover a hypothetical saturation line with slope 1:4–1:6 (V:H) |

**CIRCOLOARE DEL 25 LUGLIO 1952 EMANATA DAL CIRCOLO SUPERIORE DI ISPEZIONE PER IL Po.**

**EMBANKMENT SECTION** | It must cover a hypothetical saturation line with slope 1V:6H. |
| **FREEBOARD** | Increased of 20 cm in the nearby of urban centers. |
| **BUILDING DISTANCE (FROM EMBANKMENT TOE)** | ≥50 m for new buildings;  
Existing buildings at distance <30 m must be demolished. |
| **PS45 DEL COMITATO ISTITUZIONALE DELL’AUTORITÀ DI BACINO DEL PO, DELIBERA N°9 DEL 10 MAGGIO 1995.** |
| **DESIGN FLOOD LEVEL** | 200 year flood event |

**INTEGRAZIONE DELLE LINEE GUIDA IN SEGUITO ALLA CONVENZIONE EN°5984 DEL 5 LUGLIO 1999 DEL MAGISTRATO DEL PO IN COLLABORAZIONE CON LE UNIVERSITÀ DEGLI STUDI DI PARMA, DI BRESCIA, DI ROMA “LA SAPIENZA” E DI NAPOLI “FEDERICO II”.**

**ENLARGED EMBANKMENT** | **RIVERSIDE**: soil A6–A7 (UNI 10006, 2002) with sand content ≥15%;  
**LANDSIDE**: soil A6–A4 (UNI 10006, 2002) with sand content ≤50% + toe trench. |
| **EMBANKMENT MATERIAL** | If the available material is not suitable to use clay core and diaphragm to avoid heave risk. |
| **DEGREE OF COMPACTION** | With reference to Standard Proctor |

**Table 29. Main provisions introduced by the listed regulations.**
3.2.1. NTC 2008

The NTC2008 deals with loose material constructions such as embankments and river embankments at § 6.8. The code refers also to constructions of loose materials with filter or drainage function whereas earth dams are dealt with specific codes. The code establishes that the project must consider the performance requirements and it must include the construction material choice and the construction criteria. In particular, the project must specify the characteristic of the materials and the required degree of compaction and layer deformability.

During the construction stage, both the degree of compaction, the soil moisture and the layer deformability must be checked.

It must result:
\[ E_d \leq R_d \] (66)

Where:
\[ E_d = \text{design value of effect of actions} \]
\[ E_d = E \cdot \left[ \gamma_F \cdot F_k; \frac{X_k}{\gamma_M}; a_d \right] \] (67)

\[ R_d = \text{design value of the resistance} \]
\[ R_d = \frac{1}{\gamma_R} \cdot R \cdot \left[ \gamma_F \cdot F_k; \frac{X_k}{\gamma_M}; a_d \right] \] (68)

\[ E = \text{effect of actions;} \]
\[ R = \text{resistance;} \]
\[ F_k = \text{characteristic value of an action;} \]
\[ X_k = \text{characteristic value of a material property;} \]
\[ a_d = \text{design values of geometrical data;} \]
\[ \gamma_F = \text{partial factor for actions, which takes account of the possibility of unfavorable deviations of the action values from the representative values also accounting for model uncertainties and dimensional variations;} \]
\[ \gamma_R = \text{partial factor associated with the uncertainty of the resistance model;} \]
\[ \gamma_M = \text{partial factor for a soil parameter (material property), also accounting for model uncertainties and dimensional variations.} \]
The inequality 66 must be satisfied by using the coefficients reported in the following tables.

In particular, the coefficients $A_2$, $M_2$ and $R_2$ must be used (approach 1, combination 2).

The stability of the embankment–soil combination must be studied in relationship with the different stages of construction: at the end of the construction and during the life of the embankment.

If the embankment is located on a slope then it is necessary to study the effect of the embankment on the slope stability even in relationship with hydraulic condition changes.

**Figure 122. Table 6.2.I from (NTC, 2008): Partial factors for an action ($\gamma_F$) or for the effect of an action ($\gamma_E$). $\gamma_{G1}$: Partial factor for a permanent action; $\gamma_{G2}$ partial factor for a permanent but not structural action; $\gamma_{Q1}$: Partial factor for a variable action.**

**Figure 123. Table 6.2.II from (NTC, 2008): Partial factors for the soil parameters ($\gamma_M$) also accounting for model uncertainties. $\gamma_{\phi'}$: Partial factor for the angle of shearing resistance ($\tan \phi'$); $\gamma_C$: Partial factor for the effective cohesion; $\gamma_{CU}$: Partial factor for the undrained shear strength; $\gamma_g$: Partial factor for weight density.**
It is particularly important to pay attention to both heave and erosion risk.

To avoid the instability in the case of uplift:

\[ V_{\text{inst},d} \leq G_{\text{stb},d} + R_d \]  \hspace{1cm} (69)

Where:

\[ V_{\text{inst},d} = G_{\text{inst},d} + Q_{\text{inst},d} \]  \hspace{1cm} (70)

\( V_{\text{inst},d} \) = design value of the destabilising action;

\( G_{\text{inst},d} \) = design value of the permanent actions that contribute to instability;

\( Q_{\text{inst},d} \) = design value of the variable actions that contribute to instability;

\( G_{\text{stb},d} \) = design value of the actions that contribute to stability;

\( R_d \) = design value of the resistance.

The partial coefficients to apply to the actions are summed up by Table 6.2.III while the partial coefficients to apply to the geotechnical parameters are those referred to as M2 that are summed up by Table 6.2.II (Figure 123 and Figure 125).

To avoid the risk of heave it must result:

\[ u_{\text{inst},d} \leq \sigma_{\text{stb},d} \]  \hspace{1cm} (71)

Where:

\( u_{\text{inst},d} \) = design value of destabilising total pore-water pressure;

\( \sigma_{\text{stb},d} \) = design value of the total stress that contribute to stability.

The partial coefficients summed up by table 6.2.IV must be applied (Figure 126).
It is also necessary to verify that the settlements are compatible with the embankment functionality.

Displacements and pore pressures should be controlled using monitoring systems to make sure that the values are compatible with the safety and functionality requirements.
3.3. USA DESIGN CRITERIA

In the USA, there are two commonly used approaches to design and certify levees: the FEMA (Department of Homeland Security’s Federal Emergency Management Agency) approach and the USACE approach.

The first one is a deterministic design approach based on the median 100–year water surface elevation. Levees are analysed for erosion, stability, seepage and settlement based on this water surface and a minimum amount of freeboard above it (typically 3 feet) is required. It is commonly used by engineers and provides for a reasonably conservative levee height but it does not consider the consequences of a levee breach.

The second one is a combined and probabilistic approach both developed and used by USACE. It considers uncertainty in design water surface elevation, DWSE, combined with a deterministic geotechnical levee evaluation. In fact, the DWSE calculated using probabilistic methods, is used to perform a deterministic geotechnical evaluation of the levee. The USACE procedure for certification, NFIP levee system evaluation, uses deterministic seepage and slope stability analyses and conventional factors of safety for the 90% assurance 100–year water surface elevation and it also requires at least 3 feet of freeboard. The hydraulic modeling assumes that other levees in the region, even when overtopped, not breach.

Levees protecting urban and urbanizing area should be designed for a landside slope stability and seepage (or underseepage) factors of safety greater than one, for flood stages at the top of the levee so that erosion from overtopping would be the cause of levee failure for extreme flood events. They should be designed as a system. It should be assumed that other levees in the regional system upstream and downstream from the area:

- Are no lower than their authorized design elevations;
- do not breach even when overtopped;
- have at least 3 feet of freeboard with respect to the 200–year water surface.

**DWSE**

According to the FEMA approach the median 200–year water surface elevation is the unadjusted DWSE, while, according to the USACE approach, the unadjusted DWSE is the 90% assurance 200–year water surface elevation.
Adjustments to the DWSE can be made if there is a bend in the channel that could cause superelevation along the outside of the bend or to account for the potential increases in water surfaces caused by climate changes, update hydraulic models and sea level rise.

**MTOL**

Minimum Top of Levee, MTOL, is the required minimum elevation for the physical top of the levee to provide an adequate factor of safety that the levee will contain the DWSE, including adjustments above mentioned, without being overtopped.

While the FEMA approach uses the freeboard to provide this factor of security, under the USACE approach the needed factor of safety is provided by a combination of freeboard and the use of a DWSE with a high degree of assurance.

According to the FEMA approach, the MTOL is the higher of:

- the DWSE plus 3 feet;
- the DWSE plus the computed wind setup and wave runup\(^4\).

According to the USACE approach, the MTOL is the higher of:

- the DWSE;
- the median 200–year water surface elevation plus adjustments and 3 feet;
- the median 200–year water surface elevation plus adjustments and computed wind setup and wave runup.

**HTOL**

Hydraulic top of levee, HTOL, is a water surface elevation at or between the DWSE and the MTOL that is used to provide reasonable assurance that the levee will be stable for extreme loading conditions.

The HTOL is the higher of either A or B, where A is the lower of:

- the median 200-year water surface elevation plus 3 feet;
- the median 500-year water surface elevation;
- the MTOL.

---

\(^4\) Potential wave runup is the height above the still water level that a wave breaking on a structure slope will reach as it travels up the slope, assuming the slope extends above the runup height. It is an approximate indicator of water velocity on the structure slope, and thus the potential for erosion of the waterside levee slope. The actual wave runup height depends on the water level and structure crest elevation, which may limit sunup height (USACE, 2006).
B is the DWSE (Department of Water Resources, 2012).

**Figure 127. HTOL for a typical levee (Figure 3-1 from (Department of Water Resources, 2012)).**

It is necessary to distinguish between intermittently loaded and frequently loaded levees.

An intermittently loaded levee is defined as:

“…a levee that does not experience a water surface elevation of one foot or higher above the elevation of the levee toe at least once a day for more than 36 days per year on average.”

Conversely a frequently loaded levee is defined as:

“…a levee that experiences a water surface elevation of one foot or higher above the elevation of the landside levee toe at least once a day for more than 36 days per year on average (10 percent of the number of days in a year).”

USACE’s (EM 1110-2-1913) states the following:

“… Embankments that are subject to water loading for prolonged periods (longer than normal flood protection requirements) or permanently should be designed in accordance with earth dam criteria rather than the levee criteria given herein.”

Therefore, only intermittently loaded levees will be dealt with herein.
3.3.1. Slope Stability Analyses for Intermittently Loaded Levees

3.3.1.1. Landside Slope Stability Analyses

Landside slope stability analyses are to use phreatic surfaces based on the DWSE and HTOL if it is more than 0.5 foot (≈ 0.15 m) above the DWSE (only in that latter case a separate slope stability analysis with the HTOL is required.). For failure surfaces based on the DWSE, that intersect the levee crown and are greater than a few feet deep in the levee slope, a minimum factor of safety of 1.4 is required. Differently, for failure surfaces based on the HTOL, that intersect the levee crown and are greater than a few feet deep in the levee slope, a minimum factor of safety of 1.2 is required.

The potential threat from shallow slip surfaces, which cut only a small portion of the levee slope and do not penetrate more than a few feet into the levee section, varies greatly. It depends on levee geometry, fill material, levee and foundation soils, soil strength characteristics, seepage conditions, potential for erosion.

If the shallow slip surface is above the phreatic surface and within a non–erosive cohesive material, it is not considered as a threat to the levee integrity. Conversely, a shallow slide in the lower portion of a no cohesive material levee can be very dangerous because it can lead to a progressive through–levee seepage/stability failure (Department of Water Resources, 2012). So it is necessary to use guidance from (USACE, 2003) and sound engineering judgment to decide if a shallow sliding surface could be a threat to levee integrity.

The steady–state phreatic surface is generally considered to be appropriate. Deviations from use of a steady–state phreatic surface (achieved through transient seepage analyses) must be substantiated.

A lower phreatic surface is only justified for levee and foundation soils and construction methods that are known and well documented (except for levee with a cutoff or internal drainage features). For a homogeneous levee, the lowest phreatic surface that can be justified is along a straight line extending from the landside levee toe to the point where the DWSE, or the HTOL, intersects the waterside levee slope. Steady state pore pressures within confined aquifers should generally be assumed for underseepage analyses.
3.3.1.2. Waterside slope stability analyses

Depending on the extent to which the DWSE may have saturated the waterside levee slope, a stability factor of safety of 1.0 to 1.2 is required (USACE guidance). Shallow sliding surfaces should be examined for their potential threat to levee safety. Slopes steeper than 3H:1V should be closely reviewed for stability.

3.3.1.3. Levee underseepage criteria for intermittently loaded levees

The factor of safety for underseepage is calculated by applying the following equations:

\[ FS = \frac{i_c}{i_e} \]  

Where:

\[ i_c = \frac{(\gamma_s - \gamma_w)}{\gamma_w} \]  

FS = factor of safety;

\( i_c \) = critical gradient;

\( i_e \) = calculated exit gradient;

\( \gamma_s \) = saturated unit weight of soil;

\( \gamma_w \) = unit weight of water.

Using a steady–state seepage analysis for a water surface set at the DWSE:

1. If the saturated unit weight of the blanket layer is \( \geq 112 \text{pcf} \) (\( \approx 17.6 \text{kN/m}^3 \)): 

![Diagram of steady-state condition stability analyses: potential slip surfaces.](image)

Figure 128. Steady-state condition stability analyses: potential slip surfaces. (Modified from (Department of Water Resources, 2012)).
- the underseepage exit gradient at the landside levee toe, is required to be $\leq 0.5$;
- the underseepage exit gradient at the toe of a seepage berm less than 300 feet wide, is required to be $\leq 0.8$;

2. If the saturated unit weight of the blanket layer is $< 112$ pcf ($\approx 17.6$ kN/m$^3$):
   - a minimum factor of safety for underseepage of 1.6 is required at the landside levee toe;
   - a minimum factor of safety for underseepage of 1.0 is required at the toe of the seepage berm.

If a seepage berm is needed, the required berm width is four times the levee height.

For a water surface set at the DWSE, the permitted underseepage exit gradient through the combined seepage berm/blanket layer, between the levee toe and the seepage berm toe, is determined by interpolation using 0.5 at the levee toe and 0.8 at the seepage berm toe.

If the saturated unit weight of either the blanket layer or seepage berm is $< 112$ pcf, the minimum factor of safety for underseepage through the combined seepage berm/blanket layer, between the levee toe and the seepage berm toe, is determined by interpolation using 1.6 at the levee toe and 1 at the seepage berm toe.

A separate seepage analysis with the HTOL is only required if the HTOL is more than 0.5 foot above the DWSE.

In that case, using a steady–state seepage analysis for a water surface set at the HTOL:

1. If the saturated unit weight of either the blanket layer or seepage berm material is $\geq 112$ pcf ($\approx 17.6$ kN/m$^3$), the permitted underseepage exit gradient through the combined seepage berm/blanket layer, at the landside levee toe is required to be $\leq 0.6$.

2. If the saturated unit weight of either the blanket layer or seepage berm material is $< 112$ pcf, a minimum factor of safety for underseepage through the combined seepage berm/blanket layer of 1.3 is required at the landside levee toe.

“...Seepage berms should be able to experience some repairable foundation damage from boils for a limited period during an extreme event without seriously compromising the integrity of the levee. This would be expected to be particularly true for berms wider than 100 feet or so. ...”. (Department of Water Resources, 2012).
For seepage berms less than 100 feet wide, the allowable underseepage exit gradient at the berm toe, using a steady-state seepage analysis for a water surface set at the HTOL, may increase by up to:

- 20% as compared to the DWSE, if the saturated unit weight of the blanket layer is ≥ 112 pcf;
- 10% as compared to the DWSE, if the saturated unit weight of the blanket layer is < 112 pcf.

### 3.3.1.4. Erosion

Potential levee safety risks due to erosion must be evaluated. Performance based analyses should be considered as well as predictive models. (See § 2.1) Erosion damage is usually due to high velocity flows coupled with erosive levee materials and/or poor hydraulic conditions; large waves and boat wakes.

It is necessary to evaluate if dispersive soils are in the vicinity and thus may have been incorporated into the levee embankments.

![Figure 129. How to project the waterside levee slope to determine acceptable bank erosion (From Department of Water Resources, 2012).](image)

### 3.3.1.5. Levee Geometry

Minimum geometry requirements (associated with generally uniform, levee materials and homogeneous embankments) for new levees or levees with extensive reconstruction, situated along major waterways, provide for a minimum 20-foot-wide
crown width and 3H:1V waterside and landside slopes (4H:1V waterside slope for bypass levees).

Exceptions (steeper slope or narrower crown) may be allowed for reconstruction of existing levees in certain circumstances where levees are demonstrated to meet minimum seepage and stability criteria and slope protection is provided. Levees with steeper slopes, for example, may be acceptable with elements that substantially decrease seepage hazards and increase slope stability such as central clay cores, seepage cutoff walls, landside filters or drains, or soil reinforcement. Moreover, levees that are wider than the minimum requirement may have steeper slopes if the minimum required dimensions would fit entirely within the actual levee, and if seepage and slope stability criteria are met.

Patrol roads both along the crown of the levee and near the toe of wider seepage berm are required for inspection. Access ramps to the levee should be provided at reasonably close intervals.

Interfaces and transitions between different types of levee sections and features along a levee system, such as at the ends of seepage berms, seepage cutoff walls, revetments, and floodwalls, must be carefully evaluated and designed.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Waterside Levee Slope</td>
<td>2h:1v</td>
<td>3h:1v (major stream levees) 12 (minor stream levees)</td>
<td>3h:1v</td>
</tr>
<tr>
<td>Minimum Landside Levee Slope</td>
<td>2h:1v</td>
<td>3h:1v (bypass levees) 2h:1v (existing levees)</td>
<td>3h:1v (new levees) 2h:1v (existing levees with good performance)</td>
</tr>
</tbody>
</table>

**Figure 130. American existing levee geometry guidance (Summary by (Department of Water Resources, 2012)).**

### 3.3.1.6. Seismic Vulnerability

Generally, earthquake damages levees and their foundation causing lateral spreading and cracking associated with earthquake shaking together with potential strength losses (e.g., liquefaction).
To meet the urban level of flood protection, an analysis of seismic vulnerability of the levee system for 200-year return period ground motions is required (Department of Water Resources, 2012).

If seismic damage from 200-year-return-period ground motions is expected after the urban level of flood protection is achieved, a post-earthquake remediation plan is required as part of a flood safety plan. It estimates the damages that might be sustained following an earthquake, and the general magnitude of earth and other materials that would be required to restore a modest level of flood protection within 8 weeks. The post-earthquake remediation plan also includes a general set of repair procedures for the interim remediation of cracked and slumped levee sections. Levees that are vulnerable to seismic damage should be repaired and improved with alternatives that are more resistant to seismic damage and easily and economically repaired following an earthquake over other cost-comparable alternatives: for example, a berm is usually preferable to a seepage cutoff wall.

3.3.1.7. OTHER CONSIDERATIONS

The potential for burrowing animal damage and associated remediation should be considered during design. In fact, burrowing animals can present a significant threat to levee integrity. Proactive animal control and damage repair are also required for levee maintenance.

“... DWR is committed to developing flood risk reduction solutions that also integrate environmental stewardship. Guidance for levee vegetation management is focused on improving public safety by providing for levee integrity, visibility, and accessibility for inspections, maintenance, and flood fight operations, while at the same time protecting important and critical environmental resources, including the remaining shaded riverine aquatic habitat along many levees”. (Department of Water Resources, 2012)."
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>DWSE (Option 1)</td>
<td>Median 200-year WSE</td>
</tr>
<tr>
<td>DWSE (Option 2)</td>
<td>90% assurance 200-year WSE</td>
</tr>
<tr>
<td>MTOL (Option 1)</td>
<td>Median 200-year WSE + higher of (1) 3 feet, or (2) height for wind setup and wave runup</td>
</tr>
<tr>
<td>MTOL (Option 2)</td>
<td>Lower of A or B, where:</td>
</tr>
<tr>
<td></td>
<td>• A is the higher of (1) 90% assurance 200-year WSE, (2) median 200-year WSE plus 3 feet, or (3) median 200-year WSE plus height for wind setup and wave runup</td>
</tr>
<tr>
<td></td>
<td>• B is the higher of (1) 95% assurance 200-year WSE, (2) median 200-year WSE plus 2 feet, or (3) median 200-year WSE plus height for wind setup and wave runup</td>
</tr>
<tr>
<td>HTOL (Option 1)</td>
<td>Lower of (1) median 200-year WSE plus 3 feet, or (2) median 500-year WSE</td>
</tr>
<tr>
<td>HTOL (Option 2)</td>
<td>Higher of A or B, where:</td>
</tr>
<tr>
<td></td>
<td>• A is the lower of (1) median 200-year WSE plus 3 feet, (2) median 500-year WSE, or (3) MTOL (Option 2)</td>
</tr>
<tr>
<td></td>
<td>• B is the DWSE</td>
</tr>
<tr>
<td>Seepage-Exit Gradient at Levee Toe</td>
<td>For DWSE</td>
</tr>
<tr>
<td></td>
<td>$\gamma \geq 112$ pcf</td>
</tr>
<tr>
<td></td>
<td>$i \leq 0.5$</td>
</tr>
<tr>
<td>Seepage-Exit Gradient at Seepage Berm Toe</td>
<td>$i \leq 0.8$</td>
</tr>
<tr>
<td>Steady-State Slope Stability (Landside)</td>
<td>$FS \geq 1.4$</td>
</tr>
<tr>
<td>Rapid Drawdown Slope Stability (Waterside)</td>
<td>$FS \geq 1.2$ (prolonged high stage)</td>
</tr>
<tr>
<td>Seismic Vulnerability</td>
<td>Restore grade and dimensions for at least 10-year WSE plus 3 feet of freeboard or higher for wind setup and wave runup within 8 weeks</td>
</tr>
<tr>
<td>Levee Geometry</td>
<td>For new or extensive reconstruction on a major stream, minimum 20-foot-wide crown, 3h:1v waterside and landside slopes for all levees except bypass levees (4h:1v waterside slope)</td>
</tr>
</tbody>
</table>

Notes:
- This table includes only criteria that are easily quantified.
- The median 200-year WSE, the 90 percent assurance 200-year WSE, and the 95 percent assurance 200-year WSE in this table are assumed to have been increased appropriately as discussed in Section 7.1.3.
- Whichever option is selected, that same option is to be used for the DWSE, MTOL, and HTOL.

Key:
- Option 1 = FEMA Approach
- Option 2 = USACE Approach
- DWSE = design water surface elevation
- FS = factor of safety
- HTOL = hydraulic top of levee
- i = exit gradient
- pcf = pounds per cubic foot
- MTOL = minimum top of levee
- WSE = water surface elevation
- $\gamma$ = saturated unit weight of soil (blanket layer)

Figure 131. Urban levee design criteria summary for intermittently loaded levees. (Table 7-2 from Department of Water Resources, 2012).
3.4. Remarks

Observing levee safety assessments in Italy and in the USA, some interesting differences have surfaced.

The Italian code in force does not address levee geometry requirements.

The level of protection offered by a levee is typically described in terms of the flood size, or floodwater level, that the levee is capable of containing. Levees are designed to have a particular size and shape to enable them to withstand the corresponding floodwater level. By comparing Italian design criteria and USA design criteria, it is possible to observe that the Italian code does not give any indication on the matter.

In Italy, the safety levels of levees are assessed with reference to a design event having a certain return period, expressed as the inverse value of the return period but the evaluation water surface event is not defined. Moreover, NTC2008 does not delegate this choice to other norms or institutions. Conversely, levee evaluations in the USA change due to changing situations and insights that can result in levee evaluations for multiple water surface elevations and in a better understanding of the levee resilience.
INSPECTION,
MAINTENANCE,
MONITORING AND
REMDIATION
The Department of Civil Engineering of the University of Pisa, nowadays Department of Energy, System, Territory and Construction Engineering (DESTeC), has started since 2009, in cooperation with District of Lucca, Service for the Defense of the Territory (Provincia di Lucca, Servizio Difesa del suolo), a research activity aimed at the reduction of failures of floodplain embankments.

Among the undertaken activities, an innovative method to evaluate the degree of compaction of both existing and new river embankments (fine grained soils) after their completion has been successfully evaluated and improved (§ 4).

Moreover, a monitoring system was installed within a real scale embankment (Bottacci site, Province of Lucca, Northern Tuscany, Italy) to realistically evaluate hydraulic and saturation conditions during different periods of the embankment life (dry season and flood events) (§ 5).

The aims of the proposed monitoring system are to calibrate stability analyses under unsteady flow conditions and, at the same time, to assess the effectiveness of possible countermeasure, and in particular, of a plastic diaphragm that was realized using a quite innovative technique (§ 6).

This section illustrates the available results and eventually describes the principal control measures for the main failure causes as depicted at § 2.
4. ASSESSMENT OF THE DEGREE OF COMPACTION OF RIVER EMBANKMENTS

4.1. INTRODUCTION

The Italian Building Code (NTC, 2008) establishes that during the construction stage both the degree of compaction, the soil moisture and the layer deformability must be checked.

In the construction of embankments, earth dams and many other engineering structures, loose soils must be compacted to increase their unit weights. Compaction increases the strength characteristics of soils and the stability of slopes of embankments and decreases the amount of settlements of the structures. Conventional tests to check the degree of compaction of soils (e.g. in situ density tests, plate load tests, etc.) are time consuming. In addition, these tests give information only about the shallower layer and hence are not suitable in the case of existing embankments. Therefore, this research project proposes an innovative method to evaluate the degree of compaction of both existing and new river embankments (fine grained soils) after their compaction, by using laboratory and in situ testing.

More specifically, a cone penetration test (CPT) tip resistance target profile is inferred from laboratory tests in a mini calibration chamber (CC) using a mini CPT (8 mm in diameter). The “laboratory” tip resistance, $q_{c,LAB}$, is expressed as a function of the expected density and of the vertical and horizontal stress components when these latter are relevant. In fact, the experience has shown that, unlike the sands, for fine grained soils the tip resistance is essentially a function of the compaction energy.

Such a dependence of $q_{c,LAB}$ is obtained carrying out a number of repeated tests in the CC at given density and different consolidation stresses. In situ stresses are inferred by combining Flat Plate Dilatometer (DMT) results and an estimate of the vertical stress component. The DMT can give an indication about the coefficient of earth pressure at rest ($K_o$) as a function of the lateral stress index ($K_d$). A comparison between the $q_{c,LAB}$ profile, from CC testing, and the $q_c$, as inferred from in situ CPT,
guides the possibility of assessing the density of existing embankments. As for new embankments, the method defines the expected in situ $q_c$ for a given target density. The proposed method seems applicable to any earthwork using fine soils. Obviously, the method reliability is based on a correct estimate of both in situ and laboratory stresses within the embankment and the mini calibration chamber. In the case of partially saturated soils, a correct estimate of effective stresses in situ and laboratory requires a knowledge of soil suction. This information can be inferred both indirectly from moisture content and grain size characteristics or directly measured in situ or in the laboratory by means of appropriate measuring methods.

The proposed methodology has been studied for some time at the University of Pisa. The methodology was positively evaluated at a river embankment constructed using compacted fine-grained soils. The practical application of the method gave a verification of the correctness of the hypotheses. In fact, undisturbed samples, retrieved with different methods at the river embankment, were used to have a direct evaluation of the soil density in situ (Squeglia & Lo Presti, 2010). Over time, the equipment has been modified because the investigations have outlined the boundary conditions importance on the measured tip resistance.

This work presents the state of progress. It focuses on the influence of the boundary stresses on the tip resistance and on the relationship between tip resistance and compaction energy for fine grained soils. Therefore, mini CC data on clean sand and fine grained soil (low IP clayey, sandy silt) are shown. Finally, in order to control the degree of compaction of an embankment during construction, a correlation between $q_c$ and the elastic modulus ($E_u$) from Light Falling Weight Deflectometer (LFWD) is also proposed. LFWD indeed is an economical and expeditious test that can be used for quick quality control during various construction stages.

### 4.2. Methodology

The tip resistance target profile is inferred from laboratory tests in a mini calibration chamber (CC) using a mini CPT. The CC used for this study has been developed by the Geotechnical Laboratory of the University of Pisa in partnership with Pagani Geotechnical Equipment. The CC has a diameter of 320 mm and a height of 210 mm. The top boundary of the CC is rigid while both the lateral and the bottom boundaries of the CC are flexible. The lateral wall and the bottom of the chamber are provided with latex membranes. These membranes allow the independent
application of horizontal and vertical stresses through a compressed air system. All the possible chamber boundary conditions, BC1, BC2, BC3 and BC4, can be applied.

The four types of boundary conditions distinguish themselves for stress or displacement boundary condition imposition on the top, bottom and circumferential surfaces of the sample:

- **BC1** corresponds to a constant stress during cone penetration both on the lateral and on the top and bottom boundary;
- **BC2** corresponds to impeded lateral displacement and impeded displacement on the top and the bottom of the sample during cone penetration;
- **BC3** corresponds to impeded lateral displacement and constant stress on the top and the bottom of the sample during cone penetration;
- **BC4** corresponds to a constant stress on the lateral boundary and impeded displacement on the top and the bottom of the sample during cone penetration.

The mini CPT is similar to the standard cone penetrometer that has a 35.7 mm diameter (60° conical tip) but it has a cone diameter of 8 mm. CPT tests in the CC are conducted at the standard rate of 20 mm/s. A load cell external to the cone is used to register the axial force at the front of the penetrometer. The tests are performed along the axis of the sample.

The ratio $D$ of chamber diameter ($D_C$) to cone diameter ($d_c$) has a value of 40. All the CC tests have been done using boundary conditions BC1. After penetration is concluded the tip resistance target profile, for the given degree of compaction, is directly inferred.

Therefore, the “laboratory” tip resistance ($q_{c,LAB}$) is expressed as a function of the expected density and of the vertical–horizontal stress components. Such a dependence of $q_{c,LAB}$ is obtained carrying out a number of repeated tests in the CC at given density and different consolidation stresses.

The in situ stress state in the field is evaluated by combining DMT results with an estimate of the vertical stress. DMT can give indication about the coefficient of earth pressure at rest ($K_0$) as a function of the lateral stress index ($K_D$) Jamilolkowski et al. (1988). In particular, $K_0$ is determined by using $K_D$ obtained from DMT (Marchetti & Crapps, 1981) and the tip resistance measured from CPT carried out in the close vicinity.
In fact, $K_0$ is given by:

$$K_0 = \frac{p_0 - u_0}{\sigma_{v0}}$$

(74)

Where:

$p_0$ = DMT lift-off pressure;

$u_0$ = in situ pore water pressure

Then, in the investigated real case, for $K_0$ the following equation for Ticino sand was used (Jamiolkowski et al. (1988)):

$$K_0 = 0.376 + 0.095 \cdot K_D - 0.0046 \frac{q_c}{\sigma_{v0}}$$

(75)
In the case of partially saturated soils, a correct estimate of effective stresses in situ and laboratory requires the knowledge of soil suction. At the moment this aspect is subject to further research.

A comparison between the $q_{\text{c,LAB}}$ profile, from CC testing, and the $q_c$, as inferred from in situ CPT, gives the possibility of assessing the density of existing embankments, while, for new embankments, the method defines the expected in situ $q_c$ for a given target density.

4.3. PREVIOUS STUDIES OVERVIEW

The proposed innovative procedure for assessing the degree of compaction of earth works of fine grained soils has been developed and evaluated at the University of Pisa. The assumptions at the base of the proposed method have been experimentally verified and the method has been applied in a real case. The practical application of the method gave a further verification of the correctness of the hypotheses. The method was evaluated at a river embankment constructed using compacted fine-grained soils.

A similar procedure is described by (XP P 94-063, 1997) and (XP P 94-105, 2000) for coarse grained soils and requires the construction of a trial embankment and the performance of dynamic penetration tests. This procedure is applied to the control of the degree of compaction for trenches (Setra–Lcpc, 1994), (Setra–Lcpc, 2007).

Since there are few specific studies concerning the performance criteria of river embankments, in most cases the same design prescriptions, which control type of material and degree of compaction, are adopted as for road embankments or for earth dams.

The proposed methodology is based on the hypothesis that the tip resistance measured in situ by using a standard cone penetrometer is comparable with the tip resistance measured in laboratory in a CC by using a mini–penetrometer. If the tip resistance is not affected by the tip diameter then, in the same soil under the same conditions, the same tip resistance is expected to be measured using a standard cone having a diameter of 35.7 mm and a mini–penetrometer with a diameter of 8 mm.

This assumption was verified in situ (Calendasco site, Piacenza, Italy) by pushing into the soil (dry sandy silts) both the standard cone penetrometer tip and the mini–penetrometer tip, very close to each other, to compare the results (Squeglia & Lo Presti, 2011). Four tests with the standard cone and four tests with the mini–cone...
were run. The upper and lower envelopes obtained with the two different cones are shown by Figure 133.

It was found that the two tip resistance profiles were very similar and that there were not systematic differences due to the tip size.

It is known that penetration resistance in clay increases with reduced penetrometer velocity, \( v \). This trend is attributed to the increased degree of consolidation occurring at the lower velocities, as more time for consolidation around the penetrometer is possible. Increases in resistance with a rise in velocity can also occur when viscous effects dominate (Lunne, Robertson, & Powell, 1997).

For varved and silty clay \( q_c \) is at a minimum value when the penetrometer velocity is about 2 mm/s, but increases above this minimum with a reduction in velocity (Bemben & Myers, 1974), Roy et al. (1982).

Dimensional analysis showed that the degree of partial consolidation during continuous penetration is controlled by the normalised velocity, \( V \), defined as (Finnie & Randolph, 1994):

\[
V = \frac{v \cdot d}{c_v} \quad (76)
\]

Where:
- \( d \) is the diameter of the penetrometer;
- \( c_v \) is the coefficient of consolidation of the soil.

\( V \) describes the relative importance of \( v \), \( d \) and \( c_v \) when consolidation is dominant, but it cannot be applied over the full range of penetrometer velocities. In fact, at high velocities the penetration resistance under fully undrained conditions might be expected to vary as a function of the strain rate, which is related to the ratio of the
penetrometer velocity to the diameter, \( v/d \), while, at low velocities in the fully drained range the penetration resistance is likely either to be independent of \( v \) or to vary with \( v/d \), if viscous effects are present (Lehane et al. 2009).

However, for the soils under consideration and for this scope of application, it is possible to leave out the effect due to the use of a small tip.

In a second step, the hypothesis that the effects of the mini–chamber sizes can be considered negligible was verified. For this purpose, calibration tests were carried out in laboratory in a CC using the Ticino sand (Squeglia & Lo Presti, 2011). The behavior of this material has been widely studied in literature and in particular there is a huge literature concerning the cone penetration tests run in CC on Ticino sand samples to which refer the results (Baldi et al. 1986, Jamiolkowski et al. 2000, 2001).

Penetration tests, using the mini–cone, were run on Ticino sand samples reconstituted to a given relative density (about 50%) in the mini–CC and consolidated under BC3 conditions (no lateral strain) under a given vertical pressure (100÷400 kPa). More specifically, the so–called TS4 (\( \gamma_d^\text{min}=13.91 \text{kN/m}^3; \gamma_d^\text{max}=17.00 \text{kN/m}^3 \)) was used for the tests. For these tests the ratio \( D \) of chamber diameter to cone diameter (\( D_c/d_c \)) had a value of 19.5. In fact, in this first stage of the study, the mini calibration chamber equipment consisted of two end platens connected by three tie rods, an air piston fixed onto the lower end platen and a Proctor mold, which represented the mini–CC. The mold, that contained the test soil compacted to the desired density, was located between the air piston and the upper end platen. The air piston was able to apply a vertical pressure to the soil in the Proctor mold through a rigid platen. The contrast was given by the upper end platen.

The results were compared to those obtained in a large CC (\( D=33.6 \)) using the standard cone under BC1 conditions (constant vertical and horizontal stresses). According to a well–established practice, the tip resistance at mid–height of CC was selected as reference value (Garizio, 1997). The correlations given for the Ticino sand by Baldi et al. (1986) and Jamiolkowski et al. (1988) were used for comparison.

The tip resistance in the large CC with a standard cone was determined according to the following equation:

\[
q_c = C_0 \cdot \sigma'_{vo}^{C_1} \cdot e^{D_r \cdot C_2}
\]  
(77)

Where:

\( C_0, \ C_1, \ C_2 \) are empirical coefficients respectively equal to 172, 0.51 e 2.73;

201
\[ \sigma'_{v_0} = \text{applied vertical pressure}; \]
\[ D_R = \text{relative density of the sample in the mini-CC}. \]

The comparison is shown by Figure 134.

![Figure 134. Comparison of results obtained with mini cone in mini–CC with equivalent standard cone in large CC using (77) (Squeglio & Lo Presti, 2010).](image)

There were several reasons to suppose that \( q_c \) in the mini–CC, \( q_{c(\text{mini})} \), must be different than that in the large CC, \( q_{c(CC)} \). In fact, the \( D_v/d_c \) ratio was different and tests performed with the mini–penetrometer in the mini–CC were run under BC3 condition while the tests in the large CC were run under BC1 condition which is more representative of the conditions in an embankment. For this two reasons the \( q_{c(\text{mini})} \) was expected lower than \( q_{c(CC)} \). Moreover, while the large CC had flexible boundaries (i.e. about nil friction), on the contrary the mini–CC had rigid boundaries and therefore very high friction. For this reason, \( q_{c(\text{mini})} \) was expected higher than \( q_{c(CC)} \). In the light of the above considerations, it is possible to assume that for the selected relative density there is a sort of effect compensation of the recalled phenomena within the pressure interval 100–300 kPa, so that \( q_{c(CC)}/q_{c(\text{mini})} \) is about equal to one. The indicated interval contained the stress level involved in the investigated real case.

The proposed methodology was also verified in a real case. In fact, it was used to control the degree of compaction of a detention basin embankment realized near the Serchio River in the District of Lucca. The construction material was classified as A4 to A6 according to (AASHTO M 145 , 1991). The main prescriptions for the contractor required that the minimum degree of compaction corresponded to a dry volume weight was not less than 90% of the optimum density, without any
specification of which optimum should be considered (Standard Proctor or Modified Proctor).

In the laboratory, CPTs were carried out in the CC with the mini–penetrometer using soil samples from the embankment. Soil samples were reconstituted at a unit weight equal to the 80% and the 90% of the optimum unit weight (Modified Proctor Compaction Test). For each density several specimens were reconsolidated at different vertical pressures.

CPTs were carried out in situ at three different locations in the embankment by using a TG63-200 static/dynamic penetrometer by Pagani Geotechnical Equipment (Pagani, 2009) and both undisturbed and partially disturbed samples were retrieved using different techniques at the river embankment, giving a direct evaluation of the soil density in situ.

More specifically three very shallow block samples were retrieved. In addition, at two locations a specially devised sampler (AF shallow coring system) was used (Principe et al. (1997); (2007)). The first sample extended down to a depth of 340 cm while the second only reached a depth of 90 cm, because of a failure of the equipment.

Undisturbed sampling was used to check the in situ dry density.

Figure 135 shows: the in situ CPT profile at a given location, interpolation curves of the laboratory mini cone tip resistance, dry unit weight from “undisturbed” samples as a percentage of the optimum density (Modified Proctor) and end of the embankment. It is possible to observe that for the study embankment the degree of compaction is lower than the minimum required value.

Moreover, it is possible to observe that the measured laboratory and in situ values of tip resistances are consistent each other considering the in situ determined dry density.

Therefore, the proposed method was successfully applied as a quick control tool of the density of a river embankment.
Since the indicated assumptions were verified, it is possible to measure the tip resistance in the laboratory ($q_{c,LAB}$) using specimens reconstituted at the prescribed density and consolidated at different pressures. It is expected that the tip resistance measured in situ ($q_c$) is the same as $q_{c,LAB}$ for the same soil with the same density and effective stresses. Therefore, it is possible, a-priori, to establish which is the expected $q_c$ corresponding to a prescribed density.

The proposed method seems applicable to any earthwork using fine soils.

The experimental results outlined the boundary conditions importance on the measured tip resistance and the necessity of improving the CC apparatus. Therefore, the CC has been modified as described in the previous paragraph.

4.4. **Tip Resistance Stress Dependence**

This research work focuses on the influence of the boundary stresses on the tip resistance.

For this study two different materials have been tested in the CC: a fine grained soil that has been used as building material for river embankments, and a crushed sand that has been used as calibration material. The fine grained soil is a clayey sandy silt with low Plasticity Index (IP<10%), as shown by Table 30 and Figure 136. For the
tests in the CC, only the particle size fraction of the fine grained soil which passed the 2 mm sieve was used.

In the CC, fine grained soil samples were reconstituted in layers at a known unit weight (γ) using static compaction (hydraulic ram). The compacted soil has an optimum moisture content (Modified Proctor Compaction Test) of about 13%. Given the influence of compaction energy on soil engineering properties, the compaction effort, required to consolidate the sample, is registered.

Dry sand samples were reconstituted inside the calibration chamber to a given relative density (D_r) by pluvial deposit (Lo Presti, Pedroni, & Crippa, 1992). After completion, each CC test provides one value of q_c for a given value of DR or γ (respectively for sand and fine grained samples) and stress state. A number of tests, covering the range of stresses of interest have been carried out (Table 31). The inferred data provide the basis for establishing the empirical relationship between q_c and stress state by doing regression analysis.

The following equation has been used:

\[ q_c = A \cdot (\sigma'_v)^{n_v} \cdot (\sigma'_h)^{n_h} \]  

(78)

The results are summed up by Table 32 and Table 33.

Since the fine grained soil (Freddana) is partially saturated, the soil suction has been evaluated according to Aubertin et al. (2003). Aubertin et al. (2003) presented a general set of equations developed for predicting the relationship between volumetric water content, θ, and suction, ψ and gave complementary relationships developed for specific applications on granular materials and on fine grained soils. The set of equations developed to predict the water retention curve (WRC) are derived from the Kovács model (1981). The proposed model assumes that water retention results from the combined effect of capillarity and adhesion forces and provides a simple means to estimate the WRC from basic geotechnical properties.

In particular, the suction, ψ, has been estimated by using simple geotechnical parameters such as the grain size, void ratio, liquid limit, solid grain density and the gravimetric water content. A suction value of 638 cm (given in cm of water) was determined. Therefore, for the fine grained samples the effective stresses have been considered equal to the nominal stresses with an increase of 63.8 kPa.

It is possible to observe that for the dry sand the parameters of the equation (78) are clearly defined. In particular it has been systematically found an exponent value n_h greater than n_v.

For the partially saturated fine grained samples the definition of n_h and n_v was not as certain as for the sand. The results seemed to show that suction and pre-
consolidation stress were more influential than nominal stress. Therefore, further tests have been carried out in order to highlight the relationship between tip resistance, stress state and compaction energy.

**TABLE 30. MATERIALS TESTED IN THE CC.**

<table>
<thead>
<tr>
<th>Main Geotechnical Properties</th>
<th>Fine Grained Soil “Freddana”</th>
<th>Crushed Sand “Sand”</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid Limit, LL = 28.2 %</td>
<td>d₅₀ = 0.5 mm</td>
<td></td>
</tr>
<tr>
<td>Plastic Limit, LP = 19.9 %</td>
<td>Cₜₜ = 2</td>
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<tr>
<td>Plasticity Index, IP = 8.2 %</td>
<td></td>
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</tbody>
</table>

**SAMPLE COMPACTION DEGREE**

| γ = 0.8; γₒₜₜ = 1.81 kg/dm³ |

**SAMPLE MOISTURE CONTENT**

| w = 13% (≈ wₒₜₜ)           |

**Figure 136. Grain size distribution curves of the fine grained soil.**
**FIGURE 137. Grain size distribution curve of the sand.**

**LABORATORY TESTS**

<table>
<thead>
<tr>
<th>FREDDANA (fine grained soil)</th>
<th>SAND</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma = 0.8\gamma_{opt}$</td>
<td>$\gamma = 0.8\gamma_{opt}$</td>
</tr>
<tr>
<td>$w = 13%$</td>
<td>$w = 13%$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TEST SERIES</th>
<th>$\sigma'h$ [kPa]</th>
<th>$\sigma'v$ [kPa]</th>
<th>TEST SERIES</th>
<th>$\sigma'h$ [kPa]</th>
<th>$\sigma'v$ [kPa]</th>
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</thead>
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<td>30</td>
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</tbody>
</table>

**Table 31. Tests carried out in the CC.**
**GUIDELINES FOR THE GEOTECHNICAL DESIGN, UPGRADE AND REHABILITATION OF RIVER EMBANKMENTS**

BARBARA COSANTI

**FREDDANA (fine grained soil)**

\( \gamma = 0.8 \gamma_{\text{opt}} \)

<table>
<thead>
<tr>
<th>SUCTION OMITTED</th>
<th>SUCTION EVALUATED</th>
<th>INCREASE</th>
</tr>
</thead>
<tbody>
<tr>
<td>( q_{c, \text{LAB}} ) [kPa]</td>
<td>( \sigma'_h ) [kPa]</td>
<td>( \sigma'_{v} ) [kPa]</td>
</tr>
<tr>
<td>4777</td>
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<td>2377</td>
<td>60</td>
<td>120</td>
</tr>
<tr>
<td>5616</td>
<td>15</td>
<td>30</td>
</tr>
<tr>
<td>6047</td>
<td>60</td>
<td>123.8</td>
</tr>
</tbody>
</table>

**Table 32. Results of the tests carried out in the CC for the fine grained soil.**

**SAND**

\( D_R = 60\% \)

<table>
<thead>
<tr>
<th>( q_{c, \text{LAB}} ) [kPa]</th>
<th>( \sigma'_h ) [kPa]</th>
<th>( \sigma'_{v} ) [kPa]</th>
<th>( n_v )</th>
<th>( n_h )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1293</td>
<td>30</td>
<td>15</td>
<td>0.172</td>
<td>0</td>
</tr>
<tr>
<td>1490</td>
<td>30</td>
<td></td>
<td>0.172</td>
<td>0</td>
</tr>
<tr>
<td>1641</td>
<td>60</td>
<td></td>
<td>0.172</td>
<td>0</td>
</tr>
<tr>
<td>1984</td>
<td>60</td>
<td>30</td>
<td>0.108</td>
<td>0</td>
</tr>
<tr>
<td>1905</td>
<td>60</td>
<td></td>
<td>0.108</td>
<td>0</td>
</tr>
<tr>
<td>2305</td>
<td>120</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>949</td>
<td>15</td>
<td>30</td>
<td>0</td>
<td>0.510</td>
</tr>
<tr>
<td>1925</td>
<td>30</td>
<td></td>
<td>0.510</td>
<td></td>
</tr>
<tr>
<td>1168</td>
<td>60</td>
<td></td>
<td>0.510</td>
<td></td>
</tr>
<tr>
<td>1041</td>
<td>30</td>
<td>60</td>
<td>0</td>
<td>0.499</td>
</tr>
<tr>
<td>1290</td>
<td>60</td>
<td></td>
<td>0.499</td>
<td></td>
</tr>
<tr>
<td>2080</td>
<td>120</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table 33. Results of the tests carried out in the CC for the sand.**

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Figure 138. Tip resistance profile from laboratory tests in the CC using sand samples. Tests at constant horizontal stress equal to 30 kPa.

Figure 139. Tip resistance profile from laboratory tests in the CC using sand samples. Tests at constant horizontal stress equal to 60 kPa.
Figure 140. Tip resistance profile from laboratory tests in the CC using sand samples. Tests at constant vertical stress equal to 30 kPa.

Figure 141. Tip resistance profile from laboratory tests in the CC using sand samples. Tests at constant vertical stress equal to 60 kPa.
**Figure 142.** Tip resistance profile from laboratory tests in the CC using fine grained soil samples. Tests at constant horizontal stress equal to 30 kPa.

**Figure 143.** Tip resistance profile from laboratory tests in the CC using fine grained soil samples. Tests at constant horizontal stress equal to 60 kPa.
4.5. **Tip Resistance Compaction Energy and Water Content Dependence**

The test series that are summed up by Table 34 were carried out in the CC using fine grained soil samples, in order to assess the influence of both the compaction energy and the water content on the results of the CC tests and to understand if, as it was supposed, confinement pressures have little effect on the tip resistance for fine grained soils.

More specifically, a first group of tests was carried out on samples that were reconstituted in five layers at different unit weights (92.5%\(\gamma_{\text{opt}}\), 90%\(\gamma_{\text{opt}}\), 80%\(\gamma_{\text{opt}}\)) with a water content equal to the optimum moisture content.

A second group of tests was carried out using samples that were reconstituted in five layers at the constant unit weight \(\gamma=80\%\gamma_{\text{opt}}\) with different water contents: 0% (dry samples), 4% and 8%.

All the tests were carried out using boundary conditions BC1 in isotropic effective stress conditions: \(\sigma_{\text{h}}^*=\sigma_{\text{v}}^*=30\text{ kPa}\).

Each test run was repeated using three samples in order to compare the results.

<table>
<thead>
<tr>
<th>FINE GRAINED SOIL (&quot;FREDDANA&quot;)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>GROUP 1:</strong> (\gamma=f(\gamma_{\text{opt}})) (w=w_{\text{opt}}=12%)</td>
</tr>
<tr>
<td><strong>GROUP 2:</strong> (\gamma=80%\gamma_{\text{opt}}) (w=f(w_{\text{opt}}))</td>
</tr>
<tr>
<td><strong>SERIES 1</strong> (\gamma=92.5%\gamma_{\text{opt}}) (w=0%)</td>
</tr>
<tr>
<td>Sample 1</td>
</tr>
<tr>
<td><strong>SERIES 2</strong> (\gamma=90%\gamma_{\text{opt}}) (w=4%)</td>
</tr>
<tr>
<td>Sample 1</td>
</tr>
<tr>
<td><strong>SERIES 3</strong> (\gamma=80%\gamma_{\text{opt}}) (w=8%)</td>
</tr>
<tr>
<td>Sample 1</td>
</tr>
</tbody>
</table>

**TABLE 34. TESTS SERIES CARRIED OUT.** *Note: The test on the dry sample was carried out using sample compacted at a lower unit weight.*

Figure 144, Figure 145 and Figure 146 show the tip resistance profile inferred from the first group of tests (variable \(\gamma\)). It is clear that as the unit weight of the sample increases, \(q_c\) increases.
ASSESSMENT OF THE DEGREE OF COMPACTION OF RIVER EMBANKMENTS

**Figure 144. Test series 1, group 1.**

**Figure 145. Test series 2, group 1.**
Figure 146. Test series 3, group 1.

Figure 147, Figure 148 and Figure 149 show the tip resistance profile inferred from the second group of tests (variable w). It is important to remark that the test on the dry sample was carried out using sample compacted at a lower unit weight ($\gamma=76\%\gamma_{\text{opt}}$) because of the difficulties in the compaction.
Figure 148. Test series 2, group 2.

Figure 149. Test series 3, group 2.

Figure 150 and Figure 151 sum up the results. Figure 152 shows that it is necessary to spend more compaction energy to reach a greater unit weight and then a greater degree of compaction.
**Figure 150.** Tip resistance profiles as inferred by the different test series (considered $q_c =$ mean value of the three samples).

**Figure 151.** Tip resistance profiles as inferred by the different test series between 6 and 12 cm deep (central part of the samples).
Figure 152. Required compaction energy for different unit weights (for each compacted layer).

Figure 153 and Figure 154 show that there is a clear relationship between compaction energy and tip resistance: it is necessary to spend more energy to reach a greater unit weight and at a greater compaction energy corresponds a greater tip resistance.

Figure 153. Tip resistance vs. required compaction energy for each compacted layer.
Figure 154. Tip resistance versus required compaction energy (medium value).

Figure 155. Degree of compaction versus required compaction energy.
Therefore, these tests have shown that there is a clear relationship between tip resistance – degree of compaction – compaction energy.
4.6. CPT AND LFWD

The Light Falling Weight Deflectometer (LFWD) is a portable FWD that has been developed as an alternative in situ device to the plate load test with the ability to overcome accessibility problems on roads under construction.

In order to control the degree of compaction of embankments under construction, a correlation between $q_c$ and the elastic modulus, $E_d$, from LFWD is proposed. If it was possible to establish a correlation between $E_d$ and $q_c$ then it would be possible to have a simple prevision method for quality controls. In fact, it would be possible to predict the $q_c$ value for a given measured $E_d$.

A recently retrofitted levee section of the Serchio River in the District of Lucca has been used as an experimental field to carry out a geotechnical campaign to verify the existence of the above described correlation.

For the construction of the levee, the same material used for the construction of the experimental embankment described at § 2.2 was used (from Cave Pedogna S.p.a). The aim of the campaign was to control the degree of compaction and the mechanical resistance of the embankment. The campaign consisted of CPTu, continuous sampling, Dynamic Cone Penetrometer tests (DCP) and LFWD tests. Continuous samples were retrieved very close to already performed CPTu for subsequent laboratory testing:

- Classification: Atterberg limits and grain size distribution curves;
- Natural water content measurements. Each continuous sample was divided into 5 parts of equal length and for each part it was determined the water content and the degree of saturation by supposing a specific gravity of soil grains, $G_s$, equal to 2.7.
- In situ soil density assessment. For every continuous sample, the value of the natural volume weight and the sample compaction were measured.

It was also decided to have every LFWD located very close to each CPTu to compare the results.

The LFWD tests were carried out with two different devices: the Dynatest 3031 and the ZFG 2000 Light Drop Weight Tester. The CPTu tests were carried out with the Pagani TG 73 200 kN penetrometer (Pagani, 2009).
4.6.1. **DCP (ASTM D 6951-03)**

The test is used to assess in situ strength of undisturbed soil and compacted materials, typically for pavement applications. It can be used to estimate the strength characteristics of fine and coarse grained soils, granular constructions materials and weak stabilized or modified materials while it cannot be used in highly stabilized or cemented materials and for granular materials containing a large percentage of aggregates greater than 50 mm.

The apparatus consists of a 15.8 mm diameter steel drive rod with a replaceable point or disposable cone tip, an 8 kg hammer which is dropped a fixed height of 575 mm, a coupler assembly and a handle. The tip has an included angle of 60° and a diameter at the base of 20 mm.

The operator holds the device by the handle in a vertical or plumb position and lifts and releases the sliding hammer from the standard drop height. The total penetration for a given number of blows is recorded in mm/blow.

The depth of penetration varies with application (a penetration less than 900 mm is generally adequate for typical highway applications). If after 5 blows the device has not advanced more than 2 mm or the handle has deflected more than 75 mm from the vertical position the test shall be stopped (refusal).

The instrument is used to assess material properties down to a depth of 1000 mm below the surface.
The penetration rate can be used to estimate in-situ CBR (California Bearing Ratio) (it will not normally correlate with the laboratory or soaked CBR of the same material), to identify strata thickness, shear strength of strata and other material characteristics. Soil density of fairly uniform material may be estimated if the soil type and moisture content are known by relating density to penetration rate on the same material.

The estimated in situ CBR is computed using the DCP index and the appropriate correlation. The correlation of penetration per blow in Figure 159 is derived from the equation:

$$ CBR = \frac{292}{DCP^{1.12}} $$

(79)

that is recommended by the US Army Corps of Engineers for all soils except for CL soils below CBR 10 and CH soils. For these soils, the US Army Corps of Engineers recommends:

$$ CBR = \frac{1}{(0.017919 \cdot DCP)^2} \quad \text{for CL soils } CBR < 10 $$

(80)

$$ CBR = \frac{1}{0.002871 \cdot DCP} \quad \text{CH soil} $$

(81)

“If a distinct layering exists within the material tested, a change of slope on a graph of cumulative penetration blows versus depth will be observed for each layer. The exact interface is difficult to define because, in general, a transition zone exists between layers. The layer thickness can be defined by the intersection of the lines representing the average slope of adjacent layers. Once the layer thickness have been defined, the average penetration rate per layer is calculated” (ASTM D 6951-03).
4.6.2. LFWD

The Light Falling Weight Deflectometer (LFWD) is a portable FWD that has been developed as an alternative in situ device to the plate load test with the ability to overcome roads under construction accessibility problems.

It is a nondestructive testing technique (NDT) widely used for in situ assessment of elastic properties of soils, subgrade and pavement foundations.

The LFWD consists of a loading plate that produces a defined load pulse and a set of geophone sensors to measure the deflection. The LFWD elastic modulus is calculated from the applied load pulse and the recorded deflection.

Several types of LFWD, with differences in design and mode of operations, have been introduced in the market which have led to some variations in the measured results.

Several studies have been conducted to assess LFWD measurements and to evaluate the influence of some relevant parameters such as grading, moisture content, temperature, and compaction and so on. The results of these studies indicate that:

- LFWD measurements are affected by the grain size of the tested material;
- As the water content decreases, the soil moduli increases hence it is important to include the effect of water content when interpreting LFWD tests.
- The temperature becomes a critical parameter if the value increases beyond 30°C (asphalt pavements);
- About the spatial domain of influence, the influence depth for the LFWD device is approximately 1.5 times the diameter of the loading plate. (This depth is less than the influence depth for the plate loading test, which is approximately two times the diameter of the plate).
- The thickness of the thin layer of fine sand, placed over the test point, that makes the surface uniform for positioning the loading plate, is one of the most critical aspects. The thickness must not be over 20 mm. In fact, the measured deflections changed more than 10% when the thickness of the sand layer increased over 20 mm (Benedetto, Tosti, & Di Domenico, 2012).

Considering the limited dimension of the soil volume investigated by the LFWD, it results that the distribution of the deflections and the dynamic response have a reasonable degree of radial symmetry around the center of the plate if there are no particular singularities (e.g. multilayer, cracks, etc.).

The half-space theory, where the soil is assumed to be homogenous, isotopic and linear elastic half-space, is often used in back calculation procedure. In fact, although the test is dynamic, the elastostatic model, based on the Boussinesq's theory, is used in back-calculation procedure of homogenous elastic modulus. For a distributed load on a circular area of the free surface of a homogenous, isotropic and linear elastic half-space, the elastic modulus can be obtained by:

\[ E = \frac{(1 - \nu^2)}{\beta \cdot a} \cdot k \]  

(82)

\( \nu \) = Poisson’s ratio
\( \beta \) = shape factor that depends on the stress distribution under the loading plate. Commonly, for portable falling weight deflectometer data interpretation, as the loading plate is stiffness compared to the soil, an inverse parabolic stress distribution is used and then \( \beta = 2 \).

(Uniform stress distribution \( \Rightarrow \beta = \pi/2 \);
Parabolic stress distribution \( \Rightarrow \beta = 3\pi/2 \))

\( a \) = radius of the loading plate;
\( k \) = elastic stiffness of the loading plate / half-space system.

The main problem in the back-calculation procedure is how to extract the elastic stiffness from the time dependent data.
Commonly the peak value method is applied hence the static stiffness, $k$, is obtained by considering only a simple reading of the maximum value of the load and deflection from the temporal records, then, the elastic modulus is calculated using equation (82).

Recently questions have been arose about the reliability and accuracy of the peak value method commonly used to extract the static stiffness of soils and subgrade from the dynamic transient data. In fact, many authors have shown that this method leads to inaccurate estimation of the static stiffness and significant systematic errors. Asli et al. (2012) proposed an alternative method to the peak value method, based on a single degree of freedom modeling of the loading plate system and the linear visco-elastic behavior of soil and subgrade assumption, that makes use of a minimization technique combined with the least square method for identification of the soil and subgrade static stiffness.

The assumption of isotropy can be reasonably accepted, but, it is more valid on regular paved surfaces than on unpaved road (natural compacted soil). In fact, it remains reasonably isotropic for compacted soil, but, as the heterogeneity of the soil increases even the directional variation in observed deflections increases (Benedetto, Tosti, & Di Domenico, 2012).

(Benedetto, Tosti, & Di Domenico, 2012) presented an elliptic model for prediction of deflections induced by a Light Falling Weight Deflectometer, based on the Boussinesq theory. The model has been calibrated using the outcomes of experimental tests carried out on both plastic and elastic soil by using the Prisma 100 LFWD. It makes it possible to evaluate the iso–deflection curves resulting from LFWD loadings.

![Figure 160. LFWD Dynatest 3031 used for the tests.](image)
### 4.6.3. Campaign Results

Table 35 sums up the tests carried during the geotechnical campaign. Over time the tests were repeated in the same locations in order to compare the results and to evaluate changes in the soil resistance. More specifically, the investigation campaign was carried out in four phases from June, 2012 to January, 2013.

<table>
<thead>
<tr>
<th>Date</th>
<th>Location</th>
<th>CPT</th>
<th>Depth [m]</th>
<th>Samples</th>
<th>Date of Completion of the Embankment Section</th>
<th>LFWD Tests</th>
<th>DCP Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>June 16th, 2012</td>
<td>A</td>
<td>CPT1</td>
<td>6.5</td>
<td>642; 643</td>
<td>May 21st, 2012</td>
<td>4 (Table 38) + 35 tests (Table 39) carried out by using Light Drop Weight Tester ZORN</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>CPT2</td>
<td>6.5</td>
<td>644; 645</td>
<td>June 4th, 2012</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>CPT3</td>
<td>6.5</td>
<td></td>
<td>June 20th, 2012 (NC)</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>July 20th, 2012</td>
<td>A</td>
<td>CPT1bis</td>
<td>6.5</td>
<td></td>
<td>May 21st, 2012</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>CPT2bis</td>
<td>6.5</td>
<td></td>
<td>June 4th, 2012</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>CPT3bis</td>
<td>6.5</td>
<td></td>
<td>June 20th, 2012</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>CPT4bis</td>
<td>6.5</td>
<td></td>
<td>June 30th, 2012</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>October 30th, 2012</td>
<td>E</td>
<td>CPTU2</td>
<td>6.5</td>
<td>(7 shallow samples)</td>
<td>July, 2012</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>F</td>
<td>CPTU3</td>
<td>6.5</td>
<td></td>
<td></td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>G</td>
<td>CPTU4</td>
<td>6.5</td>
<td></td>
<td></td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>CPTU5</td>
<td>6.5</td>
<td></td>
<td></td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>January 9th, 2013</td>
<td>E</td>
<td>CPTU22</td>
<td>6.5</td>
<td>667/a 667/b</td>
<td>=</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>F</td>
<td>CPTU33</td>
<td>6.5</td>
<td>668/a 668/b</td>
<td>=</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>G</td>
<td>CPTU44</td>
<td>6.5</td>
<td>669/a 669/b</td>
<td>=</td>
<td>2</td>
<td>3</td>
</tr>
</tbody>
</table>

*Table 35. Investigation campaign.*
The embankment material was classified as A4/A6 according to (UNI 10006) classification.
**GUIDELINES FOR THE GEOТЕХNICAL DESIGN, UPGRADING AND REHABILITATION OF RIVER EMBANKMENTS**

**BARBARA COSANTI**

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**Figure 163. Grain size distribution curves of the samples retrieved in January, 2013.**

![Grain size distribution curves](image)

---

### Table 36. Continuous samples retrieved in June, 2012.

<table>
<thead>
<tr>
<th>Register Number</th>
<th>Sample</th>
<th>Liquid Limit (LL)</th>
<th>Plastic Limit (LP)</th>
<th>Plasticity Index (IP)</th>
<th>Water content w [%]</th>
<th>Classification (UNI 10006)</th>
</tr>
</thead>
<tbody>
<tr>
<td>642</td>
<td>S1-C1</td>
<td>32</td>
<td>20</td>
<td>10</td>
<td>20.4</td>
<td>A4</td>
</tr>
<tr>
<td>643</td>
<td>S1-C2</td>
<td>32</td>
<td>23</td>
<td>9</td>
<td>19.1</td>
<td>A4</td>
</tr>
<tr>
<td>644</td>
<td>S2-C1</td>
<td>29</td>
<td>22</td>
<td>7</td>
<td>17.9</td>
<td>A4</td>
</tr>
<tr>
<td>645</td>
<td>S2-C2</td>
<td>32</td>
<td>22</td>
<td>10</td>
<td>17.1</td>
<td>A4</td>
</tr>
</tbody>
</table>

---

### Table 37. Continuous samples retrieved in January, 2013.

<table>
<thead>
<tr>
<th>Register Number</th>
<th>Sample</th>
<th>Liquid Limit (LL)</th>
<th>Plastic Limit (LP)</th>
<th>Plasticity Index (IP)</th>
<th>Water content w [%]</th>
<th>Classification (UNI 10006)</th>
</tr>
</thead>
<tbody>
<tr>
<td>667</td>
<td>667 A/1</td>
<td>36</td>
<td>23</td>
<td>13</td>
<td>21.3</td>
<td>A6</td>
</tr>
<tr>
<td>667</td>
<td>667 A/5</td>
<td>37</td>
<td>28</td>
<td>9</td>
<td>20.7</td>
<td>A6</td>
</tr>
<tr>
<td>667</td>
<td>667 B/3</td>
<td>39</td>
<td>33</td>
<td>6</td>
<td>23.0</td>
<td>A6</td>
</tr>
<tr>
<td>668</td>
<td>668 A/1</td>
<td>35</td>
<td>24</td>
<td>11</td>
<td>25.5</td>
<td>A6</td>
</tr>
<tr>
<td>668</td>
<td>668 A/5</td>
<td>33</td>
<td>17</td>
<td>16</td>
<td>20.8</td>
<td>A6</td>
</tr>
<tr>
<td>668</td>
<td>668 B/3</td>
<td>35</td>
<td>23</td>
<td>12</td>
<td>22.0</td>
<td>A6</td>
</tr>
<tr>
<td>669</td>
<td>669 A/1</td>
<td>32</td>
<td>28</td>
<td>4</td>
<td>17.6</td>
<td>A4</td>
</tr>
<tr>
<td>669</td>
<td>669 A/5</td>
<td>29</td>
<td>20</td>
<td>9</td>
<td>9.6</td>
<td>A4</td>
</tr>
<tr>
<td>669</td>
<td>669 B/3</td>
<td>29</td>
<td>22</td>
<td>7</td>
<td>13.3</td>
<td>A4</td>
</tr>
</tbody>
</table>

---

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In the following, the $q_c$ and $f_s$ profiles as inferred from the CPTu tests carried out in the four stages of the investigation campaign are reported. The figures allow to compare the results over time and to identify resistance changes of the embankment soil.

**Figure 164.** $q_c$ and $f_s$ as inferred from CPTs carried out in June 2012.

**Figure 165.** $q_c$ and $f_s$ as inferred from CPTs carried out in July 2012. The tests were carried out in the same locations of the tests carried out in June 2012.
FIGURE 166. $q_c$ and $f_s$ as inferred from CPTs carried out in October 2012.

FIGURE 167. $q_c$ and $f_s$ as inferred from CPTs carried out in January 2013. The tests were carried out in the same locations of the tests carried out in October 2012.

By comparing Figure 164 with Figure 165, it is possible to observe an increase of the tip resistance over time.
Figure 168. Tip resistance profile and water content profile with depth for the two sections A and B (where CPT 1 and CPT 2 were carried out in June 2012).

Figure 169. Tip resistance profile and water content profile with depth for the two sections E and F (where CPTU 22 and CPTU 33 were carried out in January 2013).
For every continuous sample the natural water content was measured (as explained for the embankment constructed with the same material subject to overtopping tests at § 2.2). It was observed that the measured natural water contents were always higher than the optimum value (12%).

Moreover, the profiles of moisture content with depth were inferred and were compared with the tip resistance profiles. Tip resistance profiles with depth as measured in situ show that $q_c$ increases for lower moisture contents. In particular, $q_c$ registers the highest values when the moisture content reaches the optimum value.

<table>
<thead>
<tr>
<th>TEST LOCATION</th>
<th>CPT IN THE NEARBY</th>
<th>ED [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>CPT1</td>
<td>20.3</td>
</tr>
<tr>
<td>B</td>
<td>CPT2</td>
<td>26.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>28</td>
</tr>
<tr>
<td>C</td>
<td>CPT3</td>
<td>4</td>
</tr>
</tbody>
</table>

Table 38. LFWD tests carried out by using Light Drop Weight Tester ZORN in June 2012 in the nearby of the CPT tests.
Table 39. Results of the LFWD tests carried out by using the Light Drop Weight Tester ZORN in June 2012 along the study area.

<table>
<thead>
<tr>
<th>Test N°</th>
<th>Date</th>
<th>Ed [Mpa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>06/06/2012</td>
<td>4.7</td>
</tr>
<tr>
<td>2</td>
<td>06/06/2012</td>
<td>3.2</td>
</tr>
<tr>
<td>3</td>
<td>07/06/2012</td>
<td>4.4</td>
</tr>
<tr>
<td>4</td>
<td>07/06/2012</td>
<td>4.3</td>
</tr>
<tr>
<td>5</td>
<td>07/06/2012</td>
<td>11.4</td>
</tr>
<tr>
<td>6</td>
<td>07/06/2012</td>
<td>3.3</td>
</tr>
<tr>
<td>7</td>
<td>08/06/2012</td>
<td>8.4</td>
</tr>
<tr>
<td>8</td>
<td>08/06/2012</td>
<td>20</td>
</tr>
<tr>
<td>9</td>
<td>08/06/2012</td>
<td>4.7</td>
</tr>
<tr>
<td>10</td>
<td>08/06/2012</td>
<td>3.4</td>
</tr>
<tr>
<td>11</td>
<td>19/06/2012</td>
<td>11.7</td>
</tr>
<tr>
<td>12</td>
<td>19/06/2012</td>
<td>9</td>
</tr>
<tr>
<td>13</td>
<td>19/06/2012</td>
<td>36.7</td>
</tr>
<tr>
<td>14</td>
<td>19/06/2012</td>
<td>41.7</td>
</tr>
<tr>
<td>15</td>
<td>25/06/2012</td>
<td>18.2</td>
</tr>
<tr>
<td>16</td>
<td>25/06/2012</td>
<td>12.1</td>
</tr>
<tr>
<td>17</td>
<td>25/06/2012</td>
<td>7.2</td>
</tr>
<tr>
<td>18</td>
<td>25/06/2012</td>
<td>18.7</td>
</tr>
</tbody>
</table>

Figure 171. Plan locations of the LFWD tests carried out using the ZFG 2000 Light Drop Weight Tester (Table 39). Some tests were carried out in the same locations but on different layers because the embankment section was under construction (for example tests 1, 5, 7 and 8).
The same behavior is shown by the elastic modulus, $E_d$, inferred from LFWD: as the water content decreases, $E_d$ value increases.

Figure 172 shows $E_d$ relative frequency distribution and compares it with the moisture content. It is clear that $E_d$ values that fall into the higher classes correspond to lower moisture contents.

The relationship between $E_d$ and the water content is important because of the importance of the water content role in the soil compaction.

The main goal of this investigation was looking for a possible correlation between the elastic modulus $E_d$ and the tip resistance $q_c$. If it was possible to establish a correlation between $E_d$ and $q_c$ then it would be possible to have a simple prevision method for quality control. In fact, the described method would allow to predict the $q_c$ value for a given measured $E_d$.

The supposed correlation has been assessed by comparing the elastic modulus $E_d$ with the average $q_c$ measured within the influence depth for the LFWD device that is approximately 1.5 times the diameter of the loading plate (45 cm).
**Figure 172.** $E_d$ relative frequency distribution and moisture content.

**Figure 173.** Elastic modulus $E_d$ versus moisture content.
At the moment the available measurements are limited, but the first results show a good correlation between $q_c$ and $E_d$ (Figure 175).

This aspect will be analysed by means of further investigations.
**Figure 176. Section E: Comparison between CPT tests and LFWD tests.**

**Figure 177. Section F: Comparison between CPT tests and LFWD tests.**
Finally, Figure 179, Figure 180 and Figure 181 show the DCP test results carried out on January 9th, 2013.
ASSESSMENT OF THE DEGREE OF COMPACTION OF RIVER EMBANKMENTS

**FIGURE 180. SECTION F: DCP INDEX.**

**FIGURE 181. SECTION G: DCP INDEX.**
4.7. **Final remarks**

This chapter describes an innovative method to evaluate the degree of compaction of both existing and new river embankments after their completion. A tip resistance target-profile is inferred from laboratory tests in a mini calibration chamber (CC) using a mini CPT. The “laboratory” tip resistance, \( q_{c,\text{LAB}} \), is expressed as a function of the expected density and of the vertical and horizontal stress components when these latter are relevant. In fact, the experience has shown that, unlike the sands, for fine grained soils the tip resistance is essentially a function of the compaction energy. Such a dependence of \( q_{c,\text{LAB}} \) is obtained carrying out a number of repeated tests in the CC at given density and different consolidation stresses. In situ stresses are inferred by combining DMT results and an estimate of the vertical stress component. A comparison between the \( q_{c,\text{LAB}} \) profile, from CC testing, and the \( q_c \), as inferred from in situ CPT, gives the possibility of assessing the density of existing embankments, while, for new embankments, the method defines the expected in situ \( q_c \) for a given target density.

The methodology was positively evaluated at a river embankment constructed using compacted fine grained soils. The practical application of the method gave a verification of the correctness of the hypotheses.

This chapter, after the description of the method, presents the state of progress of the research and focuses on the influence of the boundary stresses on the tip resistance and on the relationship between tip resistance and compaction energy for fine grained soils.

For this purpose two different materials have been tested in the CC: a partially saturated fine grained soil and a dry sand. A number of tests, covering the range of stresses of interest have been carried out (Table 31).

After completion, each CC test provides one value of \( q_c \) for a given value of \( D_R \) or \( \gamma \), respectively for sand and fine grained samples, and stress state. The inferred data have provided the basis for establishing the empirical relationship between \( q_c \) and stress state by doing regression analysis.

It is shown in this chapter that for the dry sand, at a given \( D_R \) value, horizontal stress is the most influential parameter on cone tip resistance. This statement is in agreement with available literature results (Baldi et al. (1986); Ahmadi et al. (2005); (Houlsby & Hitchman, 1988); (Salgado, 1993); (Ahmadi & Karambakhsh, 2010)). Conversely, for the partially saturated fine grained samples the results seemed to show that suction and pre-consolidation stress were more influential than nominal...
stress. Therefore, further tests have been carried out in order to highlight the relationship between tip resistance, stress state and compaction energy. More specifically, a first group of tests was carried out on samples that were reconstituted at different unit weights with a water content equal to the optimum moisture content while a second group of tests was carried out using samples that were reconstituted at a constant unit weight with different water contents. All the tests were carried out using boundary conditions BC1 in isotropic effective stress conditions and constant stresses.

The results show that as the unit weight of the sample increases, $q_c$ increases. It is necessary to spend more energy to reach a greater unit weight and higher relative compaction energy results in higher cone tip resistance values. Therefore, the tests have shown that for fine grained soils there is a clear relationship between tip resistance, degree of compaction and compaction energy. In fact, the cone tip resistance is controlled primarily by the compaction energy and then it may be possible to relate cone tip resistance with compaction energy.

Finally, in order to control the degree of compaction of an embankment during construction, a correlation between $q_c$ and the elastic modulus, $E_d$, from Light Falling Weight Deflectometer (LFWD) is also proposed. LFWD is an economical and expeditious test that can be used for quick quality control during various construction stages. If it was possible to establish a correlation between $E_d$ and $q_c$ then it would be possible to have a simple prevision method for quality controls. In fact, it would be possible to predict the $q_c$ value for a given measured $E_d$. At the moment, the available measurements are limited but show a good correlation between $q_c$ and $E_d$. Moreover, as with $q_c$, lower soil moisture contents results in higher $E_d$ values. Obviously, the relationship between $E_d$ and the water content is important because of the importance of the water content role in the soil compaction. These aspects will be analysed by means of further investigations.
5. A MONITORING SYSTEM TO STUDY SEEPAGE THROUGH RIVER EMBANKMENTS

Since the results of stability analyses of river embankments under unsteady flow conditions are uncertain for the limited knowledge of the initial conditions and the lack of a detailed geotechnical characterization of both the embankment and the foundation soil, among the undertaken activities, a study embankment was selected in order to clarify some basic aspects of the hydro–mechanical resistance of a bank of fine grained soil, which usually is in a condition of partial saturation.

A monitoring system was installed within the real scale embankment to realistically evaluate hydraulic and saturation conditions during different periods of the embankment life: dry seasons and flood events.

The aims of the proposed monitoring system are to calibrate stability analyses under unsteady flow conditions (Plaxis Flow, 2011) and, at the same time, to assess the effectiveness of possible countermeasure, and in particular, of a plastic diaphragm that was realized using a quite innovative technique (Intersonda S.r.l., 2012).

The selected embankment is located in the Bottacci site (Province of Lucca, Northern Tuscany, Italy). It encloses an artificial basin. Figure 184, Figure 183 and Figure 185 show the extension in plan of the constructed diaphragm and of the monitoring system and the cross section of the embankment with a scheme of the monitoring system.
Figure 182. Selected embankment.

Figure 183. Extension in plan of the monitoring system.
In order to minimize the risk of internal erosion or hydraulic heave, the use of plastic diaphragms or sheet piles is recommended (§ 6).

The construction technology, used for the Bottacci embankment, consists of dry mechanical mixing (for more details see § 6.1).

The main advantages of the proposed construction method are the fact that the serviceability of the existing embankment is not compromised by the construction of the diaphragm; transportation of soil from or towards the construction site is not required and in addition no excavation to be supported with bentonite mud is required. Eventually, very low air and water pressures are used (3–6 bar).

For the study diaphragm, direct controls consisted of visual inspection of the wall itself after soil excavation for a limited length and coring of the mixed soil from few
columns. Moreover, mixed soil specimens (Figure 186) were subjected to uniaxial compression in the laboratory obtaining a strength of up to 3 MPa and a unit weight of 1750 kg/m³ on average (Intersonda S.r.l., 2012).

Figu r e 186. Example of a mixed soil specimen.

5.1. Geotechnical Characterization of the Embankment

For the geotechnical characterization of the Bottacci embankment and its foundation soil, two rotary core boreholes, in the following referred to as SA and SB, were carried out, down to 15 m from the crest of the bank. In addition, four static cone penetration tests (CPT) were carried out. Two CPTs, in the following referred to as CPT SA and CPT SB, were carried out by using a mechanical tip down to 10 m from the crest of the bank while another two tests, in the following referred to as CPTu P1 and CPTu P2, reached the depth of 11 m from the crest of the bank and were carried out using a piezocone. As for test CPT SA, after 2.2 m the mechanical tip was replaced with the piezocone (CPTu SA). Penetration tests were carried out by using a Pagani TG–63 penetrometer and piezocone (Pagani, 2009).

Four geo-electrical resistivity tests were carried out along four different cross sections of the embankment (P1, P2, P3 and P4) and a fifth test (P5) was carried out along the embankment. Figure 183 shows the plan-location of the tests.
A MONITORING SYSTEM TO STUDY SEEPAGE THROUGH RIVER EMBANKMENTS

Figure 187. P1.

Figure 188. P2.

Figure 189. P3.
Three Lefranc permeability (variable head) tests were carried out inside each borehole and totally three Shelby samples were retrieved (from borehole SA at depth of about 1.5 and 9 m, from borehole SB at depth of about 3 m). Table 41 summarizes the results of laboratory tests (LabGeo Pisa, 2012).

In situ tests indicate variable values of the undrained shear strength ($s_u$) ranging from 90 to 120 kPa within the embankment body (2.5 – 3 m below the bank crest). As for the foundation soil a lower $s_u$ has been observed (40 – 70 kPa) in agreement with an increase of the fines and of the plasticity index down to depths of about 5.5 – 7.8 m below the bank crest. Below this soft layer the $s_u$ again increases to values of about 90 – 120 kPa and at depths of about 10 – 11 m reaches values up to 150 – 200 kPa.

As for the permeability, CPTs indicate a value of about $10^{-7}$ m/s for the embankment and values ranging from $10^{-8}$ to $10^{-9}$ m/s for the soft foundation soil.

As confirmed by borehole logs, CPTs and geo-electrical resistivity tests, a certain homogeneity of both the bank and the foundation soils is observed, at least down to a depth of about 8 m below the crest.
**Figure 192. CPT test results.**

![CPT test results graph]

<table>
<thead>
<tr>
<th>DEPTH (M)</th>
<th>Ww (%)</th>
<th>IP (%)</th>
<th>LL (%)</th>
<th>Cs - Cc</th>
<th>Cv (M²/S)</th>
<th>K (M/S)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5 – 2.0 (d)</td>
<td>26.1</td>
<td>12</td>
<td>36</td>
<td>0.011 – 0.069</td>
<td>2 10-1</td>
<td>2 10-6</td>
</tr>
<tr>
<td>1.5 – 2.0 (w)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.035 – 0.092</td>
<td>3 10-3</td>
<td>8 10-8</td>
</tr>
<tr>
<td>9.0 – 9.6 (d)</td>
<td>32.5</td>
<td>14</td>
<td>36</td>
<td>0.013 – 0.157</td>
<td>2 10-1</td>
<td>2 10-6</td>
</tr>
<tr>
<td>9.0 – 9.6 (w)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.023 – 0.142</td>
<td>3 10-3</td>
<td>1 10-7</td>
</tr>
<tr>
<td>3.0 – 3.7 (d)</td>
<td>29.7</td>
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<td>50</td>
<td>0.012 – 0.138</td>
<td>2 10-1</td>
<td>3 10-6</td>
</tr>
<tr>
<td>3.0 – 3.7 (w)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.103 – 0.379</td>
<td>3 10-3</td>
<td>1 10-7</td>
</tr>
</tbody>
</table>

*Table 41. (d) = oedometer tests carried out on specimens at the natural water content; (w) = oedometer tests carried out following the standard procedure.*

**Classification**

<table>
<thead>
<tr>
<th>BOREHOLE</th>
<th>SAMPLE</th>
<th>DEPTH [M]</th>
<th>(AASHTO M 145)</th>
<th>USCS (ASTM D2487-00)</th>
<th>(AGI, 1994)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SA</td>
<td>627-1</td>
<td>0.50 – 0.60</td>
<td>A7 - A5</td>
<td>MH</td>
<td>SILT WITH WEAKLY SANDY CLAY</td>
</tr>
<tr>
<td></td>
<td>624*</td>
<td>1.50 – 2.00</td>
<td>A6</td>
<td>ML</td>
<td>SILTY AND WEAKLY SANDY CLAY</td>
</tr>
<tr>
<td></td>
<td>627-2</td>
<td>2.40 – 2.50</td>
<td>A6</td>
<td>CL</td>
<td>SILT WITH SANDY CLAY</td>
</tr>
<tr>
<td></td>
<td>637-3</td>
<td>3.40 – 3.50</td>
<td>A6</td>
<td>CL</td>
<td>PEBBLY SANDY CLAYEY SILT</td>
</tr>
<tr>
<td></td>
<td>627-4</td>
<td>6.30 – 6.40</td>
<td>A7 - A6</td>
<td>CL</td>
<td>CLAY AND WEAKLY SANDY SILT</td>
</tr>
<tr>
<td></td>
<td>625*</td>
<td>9.00 – 9.60</td>
<td>A7</td>
<td>CL</td>
<td>CLAY WITH SANDY SILT</td>
</tr>
<tr>
<td>SB</td>
<td>628-1</td>
<td>1.20 – 1.30</td>
<td>A7 - A6</td>
<td>ML</td>
<td>SILT WITH WEAKLY SANDY CLAY</td>
</tr>
<tr>
<td></td>
<td>628-2</td>
<td>1.70 – 1.80</td>
<td>A7 - A5</td>
<td>ML</td>
<td>SILT WITH WEAKLY SANDY CLAY</td>
</tr>
<tr>
<td></td>
<td>626*</td>
<td>3.00 – 3.70</td>
<td>A7</td>
<td>ML</td>
<td>CLAY WITH PEBBLY SILT</td>
</tr>
<tr>
<td></td>
<td>628-3</td>
<td>3.70 – 3.80</td>
<td>A4</td>
<td>ML</td>
<td>CLAYEY SANDY SILT</td>
</tr>
<tr>
<td></td>
<td>628-4</td>
<td>5.70 – 5.80</td>
<td>A4</td>
<td>CL</td>
<td>CLAYEY SANDY SILT</td>
</tr>
</tbody>
</table>

*Table 42. Results of the classification. * = undisturbed samples.*
Comprehensive ground-based soil moisture observational networks are still scarce, although the investigation of soil moisture-climate relationships has gained increasing attention in recent years. Through the increasing interest of soil moisture in different disciplines, some networks have been recently established. In 2010 the International Soil Moisture Network (ISMN) was established (http://www.ipf.tuwien.ac.at/insitu). Many of the newly available but also already existing long-term networks, as well as remote sensing observations have been collected as part of the ISMN.

Over the years, different techniques for in situ soil moisture measurements have been developed. Electromagnetic sensors are most commonly used to establish continuous in situ soil moisture networks. Sensors which measure the dielectric constant of bulk soil and use that measurement to infer the volumetric water content (VWC) of the soil are becoming increasingly popular.

In fact, the most common techniques make use of the dependency of the permittivity of the soil on volumetric water content. These are either based on time domain reflectometry (TDR), frequency domain reflectometry (FDR) or capacitance techniques.

TDR technique makes use of the travel time of an electromagnetic impulse propagating along the rods of a sensor. The signal is reflected at the end of the rods and the returned signal is sampled and the travel time of the impulse is related to the permittivity of the medium.

Capacitance technique makes use of the charging time of an electromagnetic field that is related to the capacitance of the soil, which is itself related to the permittivity of the medium.

TDR sensors operate at higher frequencies and have been shown to be of higher accuracy than the FDR and capacitance-based sensors. In fact, due to their accuracy, these sensors are been used as reference sensors in several studies.

On the other hand, the capacitance sensors are less accurate but are also of significantly lower cost and this allows the use of a higher number of instruments and thus much denser networks. Given the strong spatial and temporal variability of soil moisture at relatively small scales, for some applications, it could be an advantage to choose less accurate but cheaper sensors in order to decrease the sampling error due to spatial variability.
In addition to their cost, another dissuasive argument for the TDR sensors use may be their high power consumption (for example if a site has to be operated with stand-alone power supply).

Sensor accuracy may not only be related to the measurement technique, but also to the site characteristics such as soil moisture regime, soil type, soil homogeneity, presence of stones and roots and to the sensor design. More specifically, sensors with long rods provide a more representative soil moisture measurement due to the integration over a larger volume but their installation can be more difficult compared to that with smaller and more compact sensors.

The accuracy of the measured VWC, when using universal calibration functions provided by the manufacturers, needs to be carefully evaluated using laboratory and field measurements. Moreover, soil and site-specific calibration functions are recommended (Mittelbach et al. (2011), (2012)).

5.3. **10HS Decagon Sensor**

Monitoring of the soil moisture in the Bottacci site is based on the continuous measurement of the dielectric permittivity using 10HS Decagon capacitive sensors (Figure 193). Data are logged with EM50 data logger (Decagon Devices, 2010).
The 10HS Decagon sensor measures the volumetric water content of the soil using a capacitance technique. It measures the dielectric constant of the soil in order to find its volumetric water content. An electromagnetic field is generated by rapidly charging and discharging a positive and ground electrode (capacitor) in the soil. The electromagnetic field charge time ($t$) is related to the capacitance ($C$) of the soil by the following equation:

$$t = R \cdot C \cdot \ln \left( \frac{V - V_i}{V - V_f} \right)$$

Where:
- $R$ = series resistance;
- $V$ = voltage at time $t$;
- $V_i$ = starting voltage;
- $V_f$ = applied or supply voltage.

The capacitance is related to the dielectric permittivity ($\varepsilon$) of the medium between the capacitor electrodes by:

$$C = \varepsilon \cdot \varepsilon_0 \cdot F$$

Where:
- $\varepsilon_0$ = permittivity of free space;
- $F$ = geometrical factor of the capacitor.

The $\varepsilon$ of the soil can be determined by measuring the change time ($t$) of a sensor buried in the soil. Since the dielectric constant of water is much higher than that of air or soil minerals, the charge time $t$ in the soil of equation (83) can be correlated with soil volumetric water content.

The 10HS Decagon is the successor of the commonly used EC-5 Decagon soil moisture sensor. 10HS is superior to the forerunner EC-5 sensor because of its independency from the excitation voltage and the larger sampling volume, which results in more robust estimates of average spatial soil moisture conditions. The 10HS sensor consists of two parallel pronged plastic rods of 100 mm length and 9.8 mm width, and a spacing of 12.1 mm and operates between 0 and +50°C and at a frequency of 70 MHz, with a measurement range indicated by the manufacturer between 0 and 0.57 m$^3$/m$^3$.

The 10HS volume of sensitivity is encompassed by an envelope shown in Figure 194. The total volume of influence of the 10HS is approximately 1160 cm$^3$. Anyway, the electric field distribution inside the volume of sensitivity is strongly weighted toward the sensor surfaces, so this volume should be taken as a maximum possible measurement volume. In fact, the electromagnetic field produced by the probe
decreases with distance from the probe surface. Decagon recommends that the 10HS not be installed within 10 cm of the soil surface or any foreign object in the soil (Cobos D., 2008). Large metal objects near the sensor can attenuate the sensor’s electromagnetic field and adversely affect readings.

Soil-to-probe contact is critical for accurate results. Any air gaps or excessive soil compaction around the sensor and in between the sensor prongs can profoundly influence the readings because the soil adjacent to the sensor surface has the strongest influence on the sensor reading. Therefore, when installing the 10HS, the sensor should be inserted into undisturbed soil and the contact between the sensor and soil should be maximized.

The sensor can be oriented in any direction but orienting the flat side perpendicular to the surface of the soil will minimize effects on downward water movement. The sensor measures the average VWC along its length hence a vertical installation will integrate VWC over a 10 cm depth while a horizontal orientation will measure VWC at a more discrete depth.

The sensor requires an excitation voltage in the range of 3 to 15V and produces an output voltage that is related to the VWC of the soil and ranges between approximately 0.3 to 1.25V. The output of the 10HS is independent of the excitation voltage between 3 and 15V. Any datalogger which can produce a 3 to 15V excitation with approximately 10 ms duration and read a volt-level signal with 12-bit or better resolution should be compatible with the 10HS (Decagon Devices, 2010).

Characteristics of sensors as provided by the manufacturer are listed in Figure 199.
5.3.1. Accuracy

Since dielectric sensors sense the bulk dielectric permittivity of the soil, in determining accuracy are involved the accuracy with which the sensor is able to determine bulk dielectric constant and the accuracy of the relationship between bulk dielectric constant and soil water content.

The following equation was used by Campbell et al. (2009) to determine the sensitivity of predicted water content to uncertainties in the various parameters that determine water content:

\[
X_w = \frac{\varepsilon_b^{1/2} - 1 - \left(\varepsilon_m^{1/2} - 1\right) \cdot \rho_b}{\varepsilon_w^{1/2} - 1}
\]

Where:
- \(\varepsilon_b\) = bulk permittivity;
- \(\varepsilon_w\) = water permittivity;
- \(\varepsilon_m\) = mineral permittivity;
- \(\rho_b\) = bulk density;
- \(\rho_s\) = particle density.

Table 1 from Campbell et al. (2009), shown by Figure 195, gives the found sensitivities.

If there is no independent measurement of density the limits of accuracy for mineral, agricultural soils, considering only uncertainty in density, is ± 2.5% in water content while considering organic and compacted soils the error is much larger.

Campbell et al. (2009) tested the EC–5 Decagon sensor and found that:
- no significant sensor to sensor variation was observed between all the sensors tested;
- soil moisture sensor calibrations were not significantly affected by soil type or salinity in several mineral soils and potting soils tested hence the sensor do not require calibration when used in mineral soils.

"The data show that the same calibration equation can be used for any of the potting soils tested, regardless of potting soil mixture or electrical conductivity. The calibration for potting soil is different from mineral soils due to large difference in bulk density as noted above." (Campbell et al. (2009)).
- changing salinity conditions have little effect on sensor measurements;
- the manufacturer’s calibration provided accurate water content measurements in all soils tested in the laboratory.

![Table 1](image1.png)

**Figure 195. Table 1 from Campbell et al. (2009). Sensitivity of predicted water content to uncertainties in the various parameters that determine water content by using Equation (85).**

![Figure 1](image2.png)

**Figure 1. Calibration data for five water content sensors running at 70 MHz in four mineral soils over a range of electrical conductivities (shown in parenthesis).**

**Figure 196. Results from Campbell’s tests using EC5 Decagon sensors (Campbell et al. (2009)).**
Decagon has developed a standard calibration equation for mineral soils to be used with the 10HS with Decagon data acquisition equipment:
VWC \left[ \text{m}^3/\text{m}^3 \right] \quad (86)

\begin{align*}
&= 1.17 \cdot 10^{-9} \cdot \text{raw counts}^3 \\
&- 3.95 \cdot 10^{-6} \cdot \text{raw counts}^2 \\
&+ 4.90 \cdot 10^{-3} \cdot \text{raw counts} - 1.92
\end{align*}

Where:

Raw counts = value read directly from the transducer. It is the digital value read from the A-to-D converter. This value is then converted to Volts using the factory-set calibration values of the module. The volts register is the voltage feedback from an analog transducer. It is derived from the raw counts register by applying:

\[ \text{mV} \rightarrow \text{raw counts} \cdot \left( \frac{3000}{4095} \right) \]

According to (Decagon Devices, 2010), with this standard calibration equation and careful sensor installation, accuracy of better than ±3% VWC (0.03 m$^3$/m$^3$) is possible with most mineral soils. In these soils, it is generally not necessary to calibrate the 10HS for a particular soil type, and the standard mineral calibration below can be used.
5.4. The Monitoring System

In this study, measurements of soil moisture down to 3 m by using 10HS Decagon sensors, precipitations as well as data from the piezometers and from a gauging station of the time period September 2012 to September 2013 were considered. The temporal evolution of these variables is shown at § 5.4.1. Basic soil characteristics for the site are listed in Table 41 and Table 42.

The sensors of the monitoring system were located in two different cross sections (Figure 184 and Figure 183). Section 2 is placed in correspondence to the plastic diaphragm while section 1 is placed far from the diaphragm in order to evaluate its effectiveness.
The scheme of the sensors installed in each cross section is shown in Figure 185. For each cross section the system consists of:

- 10 sensors for the continuous monitoring of the soil moisture embedded into the soil at different depths (10HS Decagon), including the data acquisition system, 2 Dataloggers EM50 Decagon Devices. Each data acquisition system is protected by means of a concrete case.

- 2 open-pipe piezometers. A 2 inches piezometer (water–side) was installed inside the borehole. A 1 inch piezometer (country–side) was installed in the borehole originated by the mechanical CPT penetration. Continuous measurements of the piezometric head were carried out by means of pressure transducers located inside the piezometer pipes, below the permanent water table.

The sensors were installed inside boreholes originated by ad–hoc continuous dynamic super-heavy penetration tests (DPSH). DPSHs were carried out using a 73 kg hammer with a drop of 76 cm and the ISSMGE tip (90° tip angle). A DPSH test was carried out for the installation of each sensor. The distance in plan of contiguous tests (i.e. sensors) was of about 20 cm along the embankment. Each sensor was located at the prefixed depth (end of each DPSH) and after that the borehole was filled with mud (local soil mixed with the water at twice the liquid limit). When the
holes were dug for installation of the sensors, care was taken to preserve the original sequence of soil horizons. Soil moisture sensors were installed vertically into the undisturbed soil. After the installation of the sensors, the soil was compacted upon refilling taking care that the soil horizons were arranged in the original order and with the original density.

![Image of sensor installation](image)

**Figure 201. 10HS Decagon sensor installation.**

More specifically, the setup of the site was carried out in the following steps:
- the holes were dug taking into account the original sequence of the soil horizons;
- disturbed and undisturbed soil samples were taken from each soil horizon for subsequent soil analysis;
- sensors were installed vertically in the undisturbed soil to provide similar conditions;
- the holes were refilled, ensuring that the soil horizons were arranged in the original order and close to the original density using compaction.

For the Bottacci site, the 10HS Decagon sensors were installed side-by-side at five depths down to 3 m.
Table 43 shows the depth of each sensor. “A” group of sensors is located close to SA borehole; “B” group of sensors is in between SA borehole and CPT SA; “C” group is close to CPT SB and eventually “D” group is close to SB borehole.

The dynamic penetration results also confirm the relative homogeneity of the bank soil.

<table>
<thead>
<tr>
<th>SENSOR</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>LOCATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td>2.99</td>
<td>2.69</td>
<td>1.98</td>
<td>1.30</td>
<td>0.68</td>
<td>A</td>
</tr>
<tr>
<td>Depth (m)</td>
<td>3.17</td>
<td>2.63</td>
<td>1.96</td>
<td>1.34</td>
<td>0.71</td>
<td>B</td>
</tr>
<tr>
<td>Depth (m)</td>
<td>3.06</td>
<td>2.49</td>
<td>1.98</td>
<td>1.29</td>
<td>0.68</td>
<td>C</td>
</tr>
<tr>
<td>Depth (m)</td>
<td>3.09</td>
<td>2.61</td>
<td>1.96</td>
<td>1.26</td>
<td>0.68</td>
<td>D</td>
</tr>
</tbody>
</table>

Table 43. Depth of sensor from GT. The depth indicates the position of the sensor tip.

<table>
<thead>
<tr>
<th>DEPTH (M)</th>
<th>SENSOR 1</th>
<th>SENSOR 2</th>
<th>SENSOR 3</th>
<th>SENSOR 4</th>
<th>SENSOR 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 – 0.2</td>
<td>NA/22/4/4</td>
<td>6/18/4/4</td>
<td>5/15/3/3</td>
<td>3/14/3/3</td>
<td>4/14/4/3</td>
</tr>
<tr>
<td>0.2 – 0.4</td>
<td>NA/12/7/8</td>
<td>4/16/5/8</td>
<td>3/20/3/8</td>
<td>2/18/5/6</td>
<td>4/16/6/5</td>
</tr>
<tr>
<td>0.4 – 0.6</td>
<td>NA/5/7/7</td>
<td>4/11/5/6</td>
<td>4/12/7/7</td>
<td>2/11/4/6</td>
<td>2/10/3/5</td>
</tr>
<tr>
<td>0.6 – 0.8</td>
<td>NA/2/6/6</td>
<td>3/5/3/7</td>
<td>3/4/6/6</td>
<td>3/5/4/5</td>
<td>-</td>
</tr>
<tr>
<td>0.8 – 1.0</td>
<td>NA/2/6/5</td>
<td>2/2/1/5</td>
<td>3/2/8/3</td>
<td>2/3/3/3</td>
<td>-</td>
</tr>
<tr>
<td>1.0 – 1.2</td>
<td>NA/2/5/5</td>
<td>1/2/1/4</td>
<td>3/2/4/3</td>
<td>2/2/1/3</td>
<td>-</td>
</tr>
<tr>
<td>1.2 – 1.4</td>
<td>NA/4/7/4</td>
<td>5/1/5/3</td>
<td>5/3/2/2</td>
<td>NA/1/3/1</td>
<td>-</td>
</tr>
<tr>
<td>1.4 – 1.6</td>
<td>NA/2/10/3</td>
<td>4/3/16/3</td>
<td>5/3/2/2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.6 – 1.8</td>
<td>NA/1/11/4</td>
<td>3/1/7/3</td>
<td>3/2/3/3</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.8 – 2.0</td>
<td>NA/2/6/3</td>
<td>2/2/1/3</td>
<td>2/1/3/3</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2.0 – 2.2</td>
<td>NA/1/5/2</td>
<td>1/2/6/2</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2.2 – 2.4</td>
<td>1/1/8/3</td>
<td>1/1/4/2</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2.4 – 2.6</td>
<td>3/2/6/3</td>
<td>2/1/4/2</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2.6 – 2.8</td>
<td>1/1/4/2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2.8 – 3.0</td>
<td>1/1/4/3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3.0 – 3.2</td>
<td>1/1/4/3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 44. N20 values for each DPSH. Groups A/B/C/D – NA = Not available.
5.4.1. Results from the Monitoring System

The Bottacci embankments were subjected to repeated floods in the fall–winter of 2012–2013. The temporary closure of the Bottacci artificial basin involves an increase of the hydraulic level. The temporary closure of the basin was programmed and controlled by the Lucca District (Service for the Defense of the Territory) in agreement with DESTeC.

More specifically, the instrumented bank was subjected to flood in the occasion of the following events:

- Event 1: October 9th–11th, 2012;
- Event 2: October 25th, 2012;
- Event 3: October 30th, 2012;
- Event 4: November 9th, 2012;
- Event 5: November 11th–13th, 2012;
- Event 6: December 1st, 2012;
- Event 7: December 4th–6th, 2012;
- Event 8: December 14th–16th, 2012;
- Event 9: March 11th–12th, 2013;

The monitoring measurements for the time window between September 7th, 2012 and September 7th, 2013 are available and discussed herein.
The observation period has been divided into two parts: the "fall–winter" period from September 7th, 2012 to March 21st, 2013 and the "spring–summer" period from March 21st, 2013 to September 7th, 2013.

The available data allow to compare piezometric heads, rainfall measurements, water levels in the detention basin and soil moisture data.

In order to compare the available information, the used temporal scale is the same, with the zero coinciding with 12:00 a.m. on September 7th, 2012.

First investigations have been performed based on hourly-averaged values from September 7th, 2012 to September 7th, 2013 focusing on the VWC obtained by applying the calibration function by the manufacturer.

Figure 204 and Figure 205 show the total rainfalls in mm/day as measured within the considered time window by two different weather stations, Lucca and Pontetetto, located at distances of 5 km and 1.5 km respectively from the monitored embankment. The data have been provided by Servizio Idrologico Regionale, SIR (http:\\www.sir.toscana.it/).

**Figure 203. Location and information about the pluviometric stations. The red circle represents the Bottacci site.**
Figure 204. Daily rainfall in the fall-winter period.

Figure 205. Daily rainfall in the spring-summer period.
Figure 206 and Figure 207 show the water level height of the basin. The horizontal line represents the embankment height. The data have been provided by Autorità di Bacino Pilota del fiume Serchio.

The maximum value of the water level height was registered on November 13th, 2012 and it was equal to 3.93 m.

Figure 208 and Figure 209 show the water table depth (from ground level) as measured inside the four piezometers for the whole observation period.

It is possible to observe that water side, in absence of rainfall and flood events, the water table during the fall–winter period is about 3.5–meter deep while, in the spring–summer period, it is about 4.5–5–meter deep.

During the fall–winter period, in absence of rainfall and flood events, landside, the piezometer at location CPT SB registered the water table at a depth of about 5 meters.

Unfortunately, because of diseases, the data provided by the piezometer at location CPT SA are scarce and fragmentary.
FIGURE 207. WATER LEVEL IN THE DETENTION BASIN IN THE SPRING–SUMMER PERIOD.

FIGURE 208. WATER TABLE DEPTH FROM PIEZOMETERS IN THE FALL–WINTER PERIOD.
A MONITORING SYSTEM TO STUDY SEEPAGE THROUGH RIVER EMBANKMENTS

**Figure 209. Water table depth from piezometers in the spring-summer period.**

**Figure 210. Volumetric water content measured by sensor group A in the fall-winter period.**
FIGURE 211. Volumetric water content measured by sensor group A in the spring-summer period.

FIGURE 212. Volumetric water content measured by sensor group B in the fall-winter period.
A MONITORING SYSTEM TO STUDY SEEPAGE THROUGH RIVER EMBANKMENTS

**Figure 213.** Volumetric water content measured by sensor group C in the fall–winter period.

**Figure 214.** Volumetric water content measured by sensor group C in the spring–summer period.
FIGURE 215. VOLUMETRIC WATER CONTENT MEASURED BY SENSOR GROUP D IN THE FALL–WINTER PERIOD.

FIGURE 216. VOLUMETRIC WATER CONTENT MEASURED BY SENSOR GROUP D IN THE SPRING–SUMMER PERIOD.
The previous figures show the moisture content at different depths for the locations A, B, C and D. The lack of data (for the above mentioned figures) is due to temporary problems of the data acquisition system.

It is possible to state some comments on the available measurements. The initial values of the soil moisture are in between 0.25 and 0.35 increasing with depth. As for the C location (behind the diaphragm) the initial values of moisture content are the highest (0.32–0.40). Initial values have been measured during a very dry period.

The shallower moisture sensors (0.7, 1.3 m depth) are especially sensitive to the water infiltration after rainfall. In fact, over the measurement period, the registered peaks for the VWC match with rainfall events.

The deep sensors at the C location (sensors that are protected by the diaphragm) are almost stable for all the monitoring period. As for the C location, the October 10\textsuperscript{th} flood (event 1) caused a sudden increase of the moisture content even for the sensor located at a depth of about 2 m. The mentioned event reached a water level above the bank of about 40 cm (overtopping).

As for the other locations A, B and D, the sensors are affected by any meteoric event as well as flood. Moreover, the values registered by different sensors at the same depth are similar.

Piezometer measurements at different locations provide almost identical values, anyway, at C location, the water table is systematically lower (on average of about 1.5 m). Moreover, the water table at C location seems to increase especially because of rainfalls.

Therefore, both piezometer and moisture measurements seem to indicate a very high effectiveness of the constructed diaphragm.

\textbf{5.4.2. Calibration}

\textquotedblleft(...) Due to the complexity of soils, the accuracy of the VWC measurement can be poor despite an accurate measurement of dielectric permittivity. Some examples of this are highly compacted soils, very low bulk density soils, soils with abnormally high organic matter content, and soils with high-dielectric mineral composition (e.g. TiO\textsubscript{2} sands). Additionally, the accuracy of the 10HS may suffer in soils with very high electrical conductivity (> 10 dS/m solution EC). In these soils, it may be necessary to calibrate the 10HS to your specific...\textquotedblright

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soil type. With a soil-specific calibration, the accuracy of the VWC measurement will be improved to 1-2% in any soil or other porous medium.” (Decagon Devices, 2010).

Mittelbach et al. (2011) evaluated the performance of the low-cost 10HS capacitance–based soil moisture sensor (Decagon Devices, 2010) using laboratory and field measurements at two sites with different soil types in Switzerland. Measurements with 10HS Decagon sensors of volumetric water content (VWC, Vol. %), were compared with corresponding measurements from gravimetric samples and time domain reflectometry (TDR) sensors (TRIME-EZ and TRIME-IT sensors, IMKO GmbH, Germany). Measurements from gravimetric samples and TDR sensors were taken as reference. The main objectives of the study were to derive error estimates associated with these instruments and to evaluate both the possibility to transfer laboratory findings of the performance of the low-cost sensor to field conditions and to represent VWC and absolute soil moisture using the capacitance sensor.

Mittelbach et al. (2011) found that variations between different 10HS sensors are small. For low VWC (<30–40 Vol. %), this sensor type presents good agreement for laboratory and field measurements. Nevertheless, the measurement accuracy of the 10HS sensor reading considerably decreases with increasing VWC. The decreasing sensitivity is caused by the measurement principle of capacitance sensors, by which the capacitor charges slower at high VWC. The 10HS sensors tend to become insensitive to variations of VWC above 40 vol. %.

The standard calibration provided by the manufacturer does not accurately predict the absolute VWC over the whole measurement range, neither under laboratory conditions nor under field conditions. More specifically, it was found not to be appropriate above 30–40 Vol. % while good agreement was shown for low VWCs. This result implies a considerably lower performance than that of about 57 Vol. % indicated by the manufacturer (Decagon Devices, 2010).

The performance of the 10HS sensor was found to vary as a function of the soil conditions (the field measurements revealed a soil type dependency of the 10HS sensor performance). Due to this dependency, it was not possible to transfer findings of calibration functions established under laboratory conditions to field conditions.

With site-specific exponential calibration functions derived from parallel 10HS and TDR measurements, the error of the 10HS compared to the TDR measurements can be decreased and the measurement range of the 10HS sensor can be increased to up to 50 Vol. %.(The calibration was conducted not only using the local soil texture,
but also taking into account the actual climate conditions, rise and recession of soil moisture).

Mittelbach et al. (2011) concluded that the 10HS sensor is appropriate for setting up dense soil moisture networks when focusing on medium to low VWC and using an established site-specific calibration function. Conversely, care has to be taken when measuring at high VWC levels.

They suggested that, given the high variability of soil moisture and cost of TDR sensors (very high accuracy), the most appropriate setup for efficient soil moisture networks consists of parallel capacitance and TDR measurements, using the latter as reference for the calibration of the low-cost sensors.

For this study laboratory sensor calibration has been carried out. The sensors have been evaluated regarding their representation of the volumetric water content (VWC). The focus of the study is on the uncertainties in measured VWC and its anomalies, as well as on the soil type and degree of compaction dependency of the measurements when the manufacturers’ calibration function is applied without correction.

For the evaluation of the 10HS Decagon sensor, the VWCs (Vol. %) for the laboratory measurements have been taken into account. This criterion considers the transformation of the 10HS sensor reading (mV) to VWC (Vol. %) by using two different approaches:

- The third-order polynomial function provided by the manufacturer;
- A calibration function established by relating the sensor reading (mV) to the reference VWC (Vol. %) by a least square fit. For the laboratory measurements the gravimetric samples were used as reference.

A calibration procedure similar to that illustrated in the instruction guide for performing a soil specific calibration on ECH2O sensors provided by (Cobos & Chambers, 2009) has been used.

More specifically the following step–by–step calibration procedure has been followed:

- Soils have been packed into the calibration container at given densities. Starting with dry soil, the bulk density has been controlled by packing a known mass of soil into a known container volume.
- The sensors have been fully inserted into the soil, including the black plastic base of the sensor. The sensor should be surrounded by continuous soil for a
radius of at least 10 cm from the flat sensing portion of the sensor. Therefore, apposite containers have been realized (Figure 217).

- The soils have been added in four layers by packing each layer before adding the next. Before inserting the sensor, only a little over half of the soil into the container has been packed. The bulk density of the sample has been maintained throughout the calibration process by packing the same soil sample to the same level on the calibration container at each water content.

- The procedure has been repeated for different water contents until the soil has neared saturation. The soil has been mixed until the mixture was homogenous.

- The raw data have been collected from each sensor (no calibration applied) by using ProCheck Decagon (Decagon Devices Inc., 2014). It is a handheld readout device for use with all soil moisture sensors and environmental monitoring sensors made by Decagon Devices. Since by using ProCheck Decagon it is impossible to make continuous soil moisture measurements then, for each sensor, they were recorded 3 measures every 10 minutes for 90 minutes (180 measures in all for each calibration run). Since, if 10HS probe reads below 200 mV or above 1200 mV (or between 275 to 1700 ADC) there is a good chance that it is defective, assuming the probe is wired and operated correctly (Decagon Devices, 2009), then, data raw were filtered to exclude values < 200 mV or > 1200 mV.

- Without removing the sensor, a volumetric soil sample has been collected into the undisturbed soil near the sensor.

- Each soil sample has been placed into a drying container and the mass of the wet soil + container has been measured.

- The volumetric soil samples (already-weighed) have been dried into the 105°C oven for 24 hours.

- The dry soil + container have been weighted.
The volumetric water content (cm³/cm³) θ is given by:

$$\theta = \frac{V_w}{V_t}$$  \hspace{1cm} (87)

Where:

- $V_w$ = volume of water;
- $V_t$ = sample total volume.

$V_t$ is already known, while, to find $V_w$, the volume of the water that is lost from the soil sample during oven drying is determined by:

$$m_w = m_{wet} - m_{dry}$$  \hspace{1cm} (88)

Where:

- $m_w$ = mass of water;
- $m_{wet}$ = mass of moist soil;
- $m_{dry}$ = mass of the dry soil.

$$V_w = \frac{m_w}{\rho_w}$$  \hspace{1cm} (89)

Where:

- $\rho_w$ = density of water;

In addition to the volumetric water content, the density of dry soil, $\rho_b$, can also be calculated. It is given by:
During the tests, six different 10HS Decagon sensors were used and each sensor was coupled with a specific container (in the following referred to as P1, P2, P3, P4, P5 and P6).

The first test was carried out leaving the sensors hanging in the air (Figure 218) in order to measure the air VWC.

For each sensor, they were recorded 3 measures every 10 minutes for 90 minutes (180 measures in all) by using ProCheck Decagon.

The second test was carried out putting each sensor in the related container after that it had been completely filled with water (Figure 219).

These two tests were carried out to test the sensors under two extreme situations.

It is possible to observe that the recorded values approach the extreme values of the valid range of values reported by the manufacturer: between 200 to 1200 mV or between 275 to 1700 ADC (Decagon Devices, 2009).

\[
\rho_b = \frac{m_{\text{dry}}}{V_t} \tag{90}
\]

<table>
<thead>
<tr>
<th>SENSOR</th>
<th>MEAN VALUE [RAW COUNTS]</th>
<th>MEAN VALUE [MV]</th>
<th>MEAN VALUE [RAW COUNTS]</th>
<th>MEAN VALUE [MV]</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>484</td>
<td>355</td>
<td>1600</td>
<td>1172</td>
</tr>
<tr>
<td>P2</td>
<td>483</td>
<td>354</td>
<td>1635</td>
<td>1198</td>
</tr>
<tr>
<td>P3</td>
<td>492</td>
<td>360</td>
<td>1632</td>
<td>1196</td>
</tr>
<tr>
<td>P4</td>
<td>485</td>
<td>356</td>
<td>1625</td>
<td>1190</td>
</tr>
<tr>
<td>P5</td>
<td>495</td>
<td>362</td>
<td>1603</td>
<td>1174</td>
</tr>
<tr>
<td>P6</td>
<td>488</td>
<td>358</td>
<td>1619</td>
<td>1186</td>
</tr>
</tbody>
</table>

Table 45. Tests carried out into air and water.
Three different soils were used for the calibration procedure:
some soil samples from the Bottacci site in the following referred to as 728;
- a soil from a river embankment under construction in the following referred to as 724;
- the same sand used for the tests in the calibration chamber (§ 4).

The sand was used dry. The experiment included 3 calibration runs with increasing relative density ($D_R$): 25%, 50% and 75% in order to establish the existence of a relationship between response accuracy of the sensor and degree of compaction of the material. The desired degree of compaction was achieved by vibratory compaction using a rubber mallet (for increasing $D_R$ it was necessary a greater number of hits).

The results are depicted by Figure 220. It is possible to observe that as the $D_R$ increases the measured VWC decreases and the error increases. Therefore, the experiment confirms the sensitivity of sensor measurements to degree of compaction of the soil.

**Figure 220. Dry sand: VWC for different Relative Densities.**
As far as the two fine grained soils are concerned, the experiment included 8 calibration runs for the soil from Bottacci site (728) and 4 calibration runs for the other soil (724) respectively, with increasing VWC (from 0 to about 45%). For each calibration run three plastic containers with the same soil, the same degree of compaction and the same level of water content were prepared (hereafter referred to as P1, P2, P3). For each plastic container a 10HS Decagon sensor was used (the same sensor for each calibration run).

Since our aim was to test the sensors in conditions comparable with site conditions, where compaction specifications are based on achieving a certain value of dry unit weight, $\gamma_d$, typically from 90% – 95% as compared to the modified Proctor, the major difficult in preparing the samples was to insert the sensor in the container without damages.
Therefore, the grater achievable $\gamma_d$ were respectively 15 kN/m$^3$ for the soil 728 (Bottacci site) and 14 kN/m$^3$ for the soil 724. In order to achieve the indicated degree of compaction a manual Proctor compaction hammer was used.

The necessary mass of soil and water were calculated using the known volume of the plastic containers and the density of solid material.

The soil was mixed with the water by using a portable cement mixer (MINIBETA, IMERGROUP®).

The following tables sum up the results of the laboratory tests.

<table>
<thead>
<tr>
<th>Soil 728 (Bottacci site)</th>
<th>Calibration run 1: VWC=0%</th>
<th>Calibration run 2: VWC=5%</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>P1</td>
<td>P2</td>
</tr>
<tr>
<td>Raw counts</td>
<td>723.8</td>
<td>725</td>
</tr>
<tr>
<td>mV</td>
<td>530.3</td>
<td>531.1</td>
</tr>
<tr>
<td>$\theta$ [%]</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>n [%]</td>
<td>45.77</td>
<td>43.08</td>
</tr>
<tr>
<td>S [%]</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 46. Soil 728. Calibrations runs 1 and 2: VWC 0 and 5%.

<table>
<thead>
<tr>
<th>Soil 728 (Bottacci site)</th>
<th>Calibration run 3: VWC=10%</th>
<th>Calibration run 4: VWC=15%</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>P1</td>
<td>P2</td>
</tr>
<tr>
<td>Raw counts</td>
<td>964.8</td>
<td>964.9</td>
</tr>
<tr>
<td>mV</td>
<td>706.8</td>
<td>706.9</td>
</tr>
<tr>
<td>$\theta$ [%]</td>
<td>10.28</td>
<td>10.21</td>
</tr>
<tr>
<td>n [%]</td>
<td>44.45</td>
<td>44.43</td>
</tr>
<tr>
<td>S [%]</td>
<td>23.14</td>
<td>22.99</td>
</tr>
</tbody>
</table>

Table 47. Soil 728. Calibrations runs 3 and 4: VWC 10 and 15%.
### Soil 728 (Bottacci site)

#### Calibration Run 5: VWC≈20%

<table>
<thead>
<tr>
<th></th>
<th>P1</th>
<th>P2</th>
<th>P3</th>
<th>Mean value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Raw counts</td>
<td>1079.4</td>
<td>1075.6</td>
<td>1075.7</td>
<td>1076.9</td>
</tr>
<tr>
<td>mV</td>
<td>790.8</td>
<td>788.0</td>
<td>788.0</td>
<td>788.9</td>
</tr>
<tr>
<td>γ&lt;sub&gt;dry&lt;/sub&gt; [kN/m&lt;sup&gt;3&lt;/sup&gt;]</td>
<td>14.64</td>
<td>14.74</td>
<td>14.69</td>
<td>14.69</td>
</tr>
<tr>
<td>θ [%]</td>
<td>21.71</td>
<td>19.47</td>
<td>19.74</td>
<td>20.31</td>
</tr>
<tr>
<td>n [%]</td>
<td>44.18</td>
<td>43.78</td>
<td>43.98</td>
<td>43.98</td>
</tr>
<tr>
<td>S [%]</td>
<td>49.14</td>
<td>44.48</td>
<td>44.88</td>
<td>46.16</td>
</tr>
</tbody>
</table>

#### Calibration Run 6: VWC≈25%

<table>
<thead>
<tr>
<th></th>
<th>P1</th>
<th>P2</th>
<th>P3</th>
<th>Mean value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Raw counts</td>
<td>1221.3</td>
<td>1232.3</td>
<td>1166.7</td>
<td>1206.8</td>
</tr>
<tr>
<td>mV</td>
<td>894.7</td>
<td>902.8</td>
<td>854.7</td>
<td>884.1</td>
</tr>
<tr>
<td>γ&lt;sub&gt;dry&lt;/sub&gt; [kN/m&lt;sup&gt;3&lt;/sup&gt;]</td>
<td>14.70</td>
<td>14.69</td>
<td>14.64</td>
<td>14.68</td>
</tr>
<tr>
<td>θ [%]</td>
<td>25.79</td>
<td>27.82</td>
<td>25.39</td>
<td>26.33</td>
</tr>
<tr>
<td>n [%]</td>
<td>43.93</td>
<td>43.98</td>
<td>43.98</td>
<td>43.97</td>
</tr>
<tr>
<td>S [%]</td>
<td>58.70</td>
<td>63.24</td>
<td>57.48</td>
<td>59.81</td>
</tr>
</tbody>
</table>

#### Calibration Run 7: VWC≈35%

<table>
<thead>
<tr>
<th></th>
<th>P1</th>
<th>P2</th>
<th>P3</th>
<th>Mean value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Raw counts</td>
<td>1309.9</td>
<td>1298.7</td>
<td>1264.4</td>
<td>1291</td>
</tr>
<tr>
<td>mV</td>
<td>959.6</td>
<td>951.4</td>
<td>926.3</td>
<td>945.8</td>
</tr>
<tr>
<td>γ&lt;sub&gt;dry&lt;/sub&gt; [kN/m&lt;sup&gt;3&lt;/sup&gt;]</td>
<td>14.62</td>
<td>14.69</td>
<td>14.68</td>
<td>14.66</td>
</tr>
<tr>
<td>θ [%]</td>
<td>36.50</td>
<td>35.46</td>
<td>33.50</td>
<td>35.15</td>
</tr>
<tr>
<td>n [%]</td>
<td>44.20</td>
<td>43.80</td>
<td>44.00</td>
<td>44.00</td>
</tr>
<tr>
<td>S [%]</td>
<td>82.57</td>
<td>80.95</td>
<td>76.13</td>
<td>79.88</td>
</tr>
</tbody>
</table>

#### Calibration Run 8: VWC≈45%

<table>
<thead>
<tr>
<th></th>
<th>P1</th>
<th>P2</th>
<th>P3</th>
<th>Mean value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Raw counts</td>
<td>1398.8</td>
<td>1388.9</td>
<td>-</td>
<td>1393.8</td>
</tr>
<tr>
<td>mV</td>
<td>1024.7</td>
<td>1017.5</td>
<td>-</td>
<td>1021.1</td>
</tr>
<tr>
<td>γ&lt;sub&gt;dry&lt;/sub&gt; [kN/m&lt;sup&gt;3&lt;/sup&gt;]</td>
<td>12.97</td>
<td>13.45</td>
<td>-</td>
<td>13.21</td>
</tr>
<tr>
<td>θ [%]</td>
<td>45.92</td>
<td>45.39</td>
<td>-</td>
<td>45.66</td>
</tr>
<tr>
<td>n [%]</td>
<td>50.53</td>
<td>48.71</td>
<td>-</td>
<td>49.62</td>
</tr>
<tr>
<td>S [%]</td>
<td>90.88</td>
<td>93.20</td>
<td>-</td>
<td>92.04</td>
</tr>
</tbody>
</table>

Table 48. Soil 728. Calibrations runs 5 and 6: VWC 20 and 25%.

Table 49. Soil 728. Calibrations runs 7 and 8: VWC 35 and 45%.
**SOIL 724**

<table>
<thead>
<tr>
<th>CALIBRATION RUN 1: VWC=0%</th>
<th>CALIBRATION RUN 2: VWC=15%</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>P2</td>
</tr>
<tr>
<td>Raw counts</td>
<td>733</td>
</tr>
<tr>
<td>mV</td>
<td>537.0</td>
</tr>
<tr>
<td>$\gamma_{\text{dry}}$ [kN/m³]</td>
<td>14.05</td>
</tr>
<tr>
<td>$\theta$ [%]</td>
<td>0.00</td>
</tr>
<tr>
<td>n [%]</td>
<td>46.48</td>
</tr>
<tr>
<td>S [%]</td>
<td>0.00</td>
</tr>
</tbody>
</table>

*Table 50. Soil 724. Calibrations runs 1 and 2: VWC 0 and 15%.*

<table>
<thead>
<tr>
<th>CALIBRATION RUN 3: VWC=25%</th>
<th>CALIBRATION RUN 4: VWC=45%</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>P2</td>
</tr>
<tr>
<td>Raw counts</td>
<td>1164.2</td>
</tr>
<tr>
<td>mV</td>
<td>852.9</td>
</tr>
<tr>
<td>$\gamma_{\text{dry}}$ [kN/m³]</td>
<td>14.03</td>
</tr>
<tr>
<td>$\theta$ [%]</td>
<td>25.13</td>
</tr>
<tr>
<td>n [%]</td>
<td>46.53</td>
</tr>
<tr>
<td>S [%]</td>
<td>54.01</td>
</tr>
</tbody>
</table>

*Table 51. Soil 724. Calibrations runs 3 and 4: VWC 25 and 45%.*

It is possible to observe that variations between different 10HS sensors are small.
The tests carried out using dry samples have shown that for sand samples the accuracy decreases even if the error is contained in the limits of accuracy declared by Decagon (±3% VWC).
Figure 223 depicts results from laboratory tests on soil 728 (Bottacci site) and compares them with the standard calibration by Decagon. It is possible to observe that the standard calibration provided by the manufacturer does not accurately predict the absolute VWC over the whole measurement range. More specifically, it has been found not to be appropriate below 20 Vol. %.
Figure 224 depicts the linear calibration functions established by relating the sensor reading (raw counts) to the reference VWC (Vol. %) for the laboratory measurements for the soil 728. It is possible to achieve a greater value for $R^2$ (0.9916) by excluding the extreme VWC data (dry soil and soil near to saturation). Therefore, the best fitting function is a polynomial.

The equation for the quadratic function best fitting the data set is (Figure 225):

$$VWC [\text{m}^3/\text{m}^3] = 5 \cdot 10^{-7} \cdot \text{raw counts}^2 - 0.004 \cdot \text{raw counts} + 0.0347$$

For the degree of saturation it is possible to write:

$$S [%] = 7 \cdot 10^{-7} \cdot \text{raw counts}^2 - 6 \cdot 10^{-5} \cdot \text{raw counts} + 0.3614$$

Therefore, by applying the above equation it is possible to establish that soil 728, from Bottacci site, reaches saturation when the measured raw counts is equal to 1440.

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Figure 224. Soil from the Bottacci site: VWC vs. raw counts. Linear calibrations.

Figure 225. Parabolic calibration established for soil 728.
As far as soil 724 is concerned, the results of the laboratory tests are less scattered and they show better agreement with the standard calibration provided by the manufacturer, as depicted by Figure 229.
A MONITORING SYSTEM TO STUDY SEEPAGE THROUGH RIVER EMBANKMENTS

**FIGURE 228.** LABORATORY CALIBRATIONS ESTABLISHED FOR THE TWO FINE GRAINED SOILS.

**FIGURE 229.** CALIBRATION RESULTS: LABORATORY RESULTS VS STANDARD CALIBRATION BY DECAGON.

For data set of soil 724, the best fitting function is a line (using the least-squares method).

The equation for the line of best fit is:

\[ y = 0.0007x - 0.4975 \]

\[ R^2 = 0.9972 \]
The soil moisture contents as recorded by the 10HS Decagon sensors for the four location (A, B, C and D) in the fall–winter period, have been re-elaborated using the established laboratory calibration (soil 728, Bottacci site).

\[
\text{VWC \left[ m^3/m^3 \right]} = 7 \cdot 10^{-4} \cdot \text{raw counts} - 0.4975 \tag{93}
\]
A MONITORING SYSTEM TO STUDY SEEPAGE THROUGH RIVER EMBANKMENTS

Figure 231. Sensor Group B: soil moisture contents as recorded by the 10HS Decagon sensors and re-elaborated using the established laboratory calibration.

Figure 232. Sensor Group C: soil moisture contents as recorded by the 10HS Decagon sensors and re-elaborated using the established laboratory calibration.
5.4.3. **Hydraulic Characterization of the Embankment**

Groundwater levels, as measured by the two waterside piezometers, SA and SB, over the observation time, have been used to estimate the permeability of the embankment.

In fact, in order to determine the hydraulic conductivity of the embankment, groundwater level data after some "flood" events have been interpreted according to the theory of time lag. Events during which water level had overtopped the bank were excluded.

Time lag of a piezometer represents the time that it takes for the instrument to reach equilibrium when there is a pore water pressure change in the soil. The magnitude of the time lag depends on the type and dimensions of the pressure measuring installation and it is inversely proportional to the permeability of the soil. Therefore, for the selection of the proper type of installation for given conditions, it is necessary to estimate the time lag.
Observation of the basic time lag for borings and piezometers provides a very simple method for determination of the permeability of soil in situ.

For variable head but constant groundwater level or pressure, it is possible to determine the coefficients of permeability of the soil, \( k \), in situ by field observations by means of the following equation:

\[
\frac{k}{F} = \frac{A}{(t_2 - t_1)} \cdot \ln \frac{h_1}{h_2} \tag{94}
\]

Where:

- \( A \) = cross sectional area of the pipe;
- \( t_2 - t_1 \) = period of time between successive water level measurements (time for water level change from \( h_1 \) to \( h_2 \));
- \( h_1, h_2 \) = distance from static water level at times \( t_1 \) and \( t_2 \);
- \( F \) = shape factor.

That is also the formula commonly used for determination of coefficients of permeability in the laboratory by means of a variable head permeameter.

Derivation of the basic differential equation for determination of the time lag is similar to that of the equations for a falling-head permeameter and is based on the assumption that Darcy’s law is valid and that water and soil are incompressible.

At the time \( t \), with reference to Figure 234, the active head, \( H \), is:

\[
H = z - y \tag{95}
\]

(where \( z \) may be a constant or a function of \( t \).)

The corresponding flow, \( q \), may be expressed by the following equation:

\[
q = F \cdot k \cdot H = F \cdot k \cdot (z - y) \tag{96}
\]

Where:

- \( F \) = shape factor.

The friction losses in the pipe are assumed negligible for the small rates of flow occurring during pressure observations.

Considering the volume of flow during the time \( dt \), the following equation is obtained:

\[
q \cdot dt = A \cdot dy \tag{97}
\]

Where:

- \( A \) = cross-sectional area of the standpipe.

By introducing \( q \) from equation (96):

\[
\frac{dy}{z - y} = \frac{F \cdot k}{A} \cdot dt \tag{98}
\]

The total volume of flow required for equalization of the pressure difference, \( H \), is:

\[
V = A \cdot H \tag{99}
\]
The basic time lag, \( T \), is then defined as the time required for equalization of this pressure difference when the original rate of flow, \( q \), is maintained:

\[
T = \frac{V}{q} = \frac{A}{F \cdot k} \quad (100)
\]

The basic differential equation for determination of the hydrostatic time lag and its influence is then:

\[
\frac{dy}{z - y} = \frac{dt}{T} \quad (101)
\]

The simplest expression for the coefficient of permeability is obtained by determination of the basic time lag \( T \):

\[
k = \frac{A}{F \cdot T} \quad (102)
\]

**Figure 234. Theory of time lag: basic definitions and equations (Waterways Experiment Station, 1951).**
For the geometry under consideration, the appropriate shape factor equation according (Hvorslev, 1951) and (Wilkinson, 1968) is:

\[
F = \frac{3\pi L}{\ln \left( 1.5 \frac{L}{D} + \sqrt{1 + \left( 1.5 \frac{L}{D} \right)^2} \right)}
\]

(103)

Where:
- \( L \) = length intake sample;
- \( D \) = inside diameter of piezometer = 2 inches for piezometers SA and SB.

After each considered "flood" event in the detention basin, for each piezometer, the basic time lag was easily determined by means of the diagram of time versus head. In fact, the basic time lag represents the time that it takes for the piezometer to reach the hydraulic equilibrium with the detention basin after the flood event when the water-table declines. The time-lag represents the time necessary for the pore water to flow from the piezometer to reach the hydraulic equilibrium.

It has been estimate a basic time lag ranging between 35–45 hours and then a permeability of about \( 10^{-6} \) m/s.

The above mentioned value represents an approximate evaluation of the in situ permeability. Many sources of error are encountered in the practical application of the method (e.g. the shape factor of the installation is computed with empirical or only approximately correct formulas which are all based on the assumption of infinite thickness of the soil layer in which the piezometer is installed).

However, the estimated value seems more realistic and reliable than the value gathered from the field permeability tests carried out during the stage of geotechnical characterization of the embankment (§ 5.1). In fact, the latter shows a high variability (scattered results).

Conversely, the estimated value that has been obtained by using the time lag seems more consistent because the test involves a greater volume of soil and it is carried out when the soil is certainly saturated.

Finally, the groundwater levels and the water levels in the detention basin have been used to get an estimate of the location of the saturation line within the embankment, during some events (time = 1946 h; 1949 h; 1954 h; 2044 h; 2133 h; 2397 h; 2672 h). The results are shown by the following figures. The soil moisture contents as recorded by the 10HS Decagon sensors and re-elaborated using the established laboratory calibration, is also reported in the figures in order to compare the results.
**GUIDELINES FOR THE GEOTECHNICAL DESIGN, UPGRADING AND REHABILITATION OF RIVER EMBANKMENTS**

BARBARA COSANTI

**Figure 235. Location of the estimated saturation line. Time = 1946 h.**

**Figure 236. Location of the estimated saturation line. Time = 1949 h.**
Figure 237. Location of the estimated saturation line. Time = 1954 h.

<table>
<thead>
<tr>
<th>Depth [m]</th>
<th>Location C</th>
<th>Location D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Port 1: 0.70m</td>
<td>80</td>
<td>86</td>
</tr>
<tr>
<td>Port 2: 1.30m</td>
<td>77</td>
<td>85</td>
</tr>
<tr>
<td>Port 3: 2.00m</td>
<td>92</td>
<td>100</td>
</tr>
<tr>
<td>Port 4: 2.60m</td>
<td>92</td>
<td>83</td>
</tr>
<tr>
<td>Port 5: 3.10m</td>
<td>84</td>
<td>76</td>
</tr>
</tbody>
</table>

Figure 238. Location of the estimated saturation line. Time = 2044 h.

<table>
<thead>
<tr>
<th>Depth [m]</th>
<th>Location C</th>
<th>Location D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Port 1: 0.70m</td>
<td>100</td>
<td>88</td>
</tr>
<tr>
<td>Port 2: 1.30m</td>
<td>90</td>
<td>95</td>
</tr>
<tr>
<td>Port 3: 2.00m</td>
<td>93</td>
<td>100</td>
</tr>
<tr>
<td>Port 4: 2.60m</td>
<td>93</td>
<td>96</td>
</tr>
<tr>
<td>Port 5: 3.10m</td>
<td>83</td>
<td>77</td>
</tr>
</tbody>
</table>
Figure 239. Location of the estimated saturation line. Time = 2133 h.

Figure 240. Location of the estimated saturation line. Time = 2397 h.
The results seem to indicate a good effectiveness of the constructed diaphragm. In particular, with reference to Figure 239, it is possible to observe that the estimated saturation line is below a hypothetical saturation line with slope 1V:6H as stated by old Italian design criteria (see Table 29, § 3.2).
5.5. Remarks

The installed monitoring system allows groundwater levels, water levels in the detention basin, daily rainfall data and soil moisture contents, as measured by 10HS Decagon sensors, to be compared.

Unfortunately, during the observation time, due to temporary problems of the data acquisition systems some of the data are missing or fragmentary. In fact, this experience has shown that the installed system is vulnerable to atmospheric agents and can correctly work only with constant maintenance. In this case the site was often unreachable and thereby the maintenance was sometimes impossible.

However, both piezometer and moisture available measurements seem to indicate a high effectiveness of the constructed diaphragm. In fact, the 10HS Decagon sensors that are protected by the diaphragm have recorded stable values of the VWC over all the monitoring period and the groundwater levels measured by the piezometer that is protected by the diaphragm are systematically lower.

The laboratory calibration has shown that variations between different 10HS sensors measurements are small.

Laboratory experiments have confirmed the sensitivity of sensor measurements to degree of compaction of the soil. In fact, it has been observed that as the $D_r$ increases, the error increases. Due to this dependency, soil sample used for laboratory calibration have been compacted as much as possible in order to approach the degree of compaction in field condition.

The standard calibration provided by the manufacturer does not accurately predict the absolute VWC over the whole measurement range. In particular, it has been found not to be appropriate above 20 Vol. %.

A site-specific parabolic calibration function derived from parallel 10HS Decagon and laboratory measurements has been proposed.

Calibration function established under laboratory conditions has been transferred to field conditions and the groundwater levels and the water levels in the detention basin have been used to gain an idea about the location of the saturation line within the embankment during some events. It is possible to observe that the estimated saturation line is below a hypothetical saturation line with slope 1V:6H, as it is stated by some Italian design criteria. This circumstance seems to confirm a good effectiveness of the constructed diaphragm.
At the moment this aspects are subject to further research.
The installed monitoring system should be improved. The low cost capacitance sensors should be supported by more accurate sensors (e.g. TDR sensors) that could be used as reference sensors in field conditions.
The field monitoring data indicate that the shallower moisture sensors are especially sensitive to the water infiltration after rainfall. Therefore, a hydrological model of the embankment should take into account alteration by rainfall infiltration.
Future research on this site will continue to provide a hydrological model of the embankment which reproduces also the response of the slope to the rainfall infiltration.
6. COUNTERMEASURES

6.1. EMBANKMENT STABILITY

For improving embankment stability (EM 1110-2-1913) suggests several methods:

- Flattening embankment slopes reduces gravity forces tending to cause failure and increases the length of potential failure surfaces (and hence increases resistance to sliding);

- Berms act by providing the same effect as flattening embankment slopes but are generally more effective because of concentrating additional weight where it is needed most and by forcing an increase in the failure path. They are an effective means of stabilization both for shallow foundation and for more deep-seated foundation failures. Berm thickness and width should be determined from stability analyses and the length should be great enough to encompass the entire problem area.

After the flood event in December 2009, in order to have a rapid increment of safety degree against floods, the District of Lucca, Service for the Defense of the Territory (Provincia di Lucca, Servizio Difesa del Suolo), proposed to install a metallic sheet pile diaphragm within the body of the embankment.

The sheet pile barrier installation was decided to reduce the risk of mechanical and hydraulic embankment failures and to prevent piping phenomena. In addition, since the bank was eroded after the failures, the sheet pile should guarantee the provisional territory protection.

Therefore, FEM analyses were carried out in order to optimize the diaphragm height. The results are presented § 2.3.4. Moreover, the cross-section geometry was modified through the addiction of a berm.

The main advantage of sheet piles is the structural resistance of this type of barrier which contributes to the mechanical resistance of the embankment and provides a hydraulic barrier even in the case that overtopping of the embankment causes the erosion of the bank itself on the country side. Moreover, they can be installed quickly without compromising the serviceability of the embankment.

On the other hand, sheet piles are very expensive and subject to possible corrosion or thickness reduction because of aggressive water and parasite currents into the soil. By the way, on the occasion of the installation of the sheet pile within the
Serchio River embankments, a sheet pile prototype was installed in order to control the degree of corrosion over the time. Unfortunately, since the extraction of the prototype is programmed for July 2014, at the moment the data are not available. Conversely, plastic diaphragms are less expensive than sheet piles. The construction technology used for the Bottacci embankment (§ 5), consists of dry mechanical mixing. Initially, the soil is completely remolded by means of augers. It is possible to use from one to three augers at a given position. During this stage it is possible to inject water in order to speed up the remolding process. After reaching the design depth, the augers stop penetration and start to move upward. During this stage the soil is mechanically mixed with dry cement and additives. The cement is injected by air pressure (3 to 6 bar). Appropriate dry cement mix has to be used, depending on soil characteristics.

The main advantages of the described construction method can be summarized as follows:

- the serviceability of the existing embankment is not compromised by the construction of the diaphragm;
- the material of construction (except the cement mix) is already in place and transportation of soil from or towards the construction site is not required. This aspect is especially relevant in the case of contaminated soil.
- No excavation to be supported with bentonite mud is required;
- very low air and water pressures are used (3–6 bar).

First results from the installed monitoring system (both piezometer and moisture measurements) seem to indicate a good effectiveness of the constructed diaphragm.

![Figure 242. Execution of two columns of treated soil (left). Sequence of construction (right).](image)
FIGURE 243. OPERATING MACHINE.

FIGURE 244. CONSTRUCTION TECHNOLOGY.
6.2. **Settlements**

Evaluation of the settlements that can occur from consolidation of both embankment and foundation may be important if the settlements would result in loss of freeboard of the embankments. It is possible to overbuild a levee by a given percent of its height to take into account anticipated settlement.

Levees located on weak foundation soils that cannot support the embankment because of inadequate shear strength such as very soft clays, sensitive clays, loose sands and natural organic deposits require some type of foundation treatment.

| Table 7-2
<table>
<thead>
<tr>
<th>Embankment Construction Deficiencies</th>
<th>Possible Consequences</th>
</tr>
</thead>
<tbody>
<tr>
<td>Organic material not stripped from foundation</td>
<td>Differential settlements; shear failure; internal erosion</td>
</tr>
<tr>
<td>Highly organic or excessively wet or dry fill</td>
<td>caused by through seepage</td>
</tr>
<tr>
<td>Placement of pervious layers extending completely through</td>
<td>Excessive settlements; inadequate strength</td>
</tr>
<tr>
<td>the embankment</td>
<td></td>
</tr>
<tr>
<td>Inadequate compaction of embankment (lifts too thick,</td>
<td>Excessive settlements; inadequate strength; through</td>
</tr>
<tr>
<td>haphazard coverage by compacting equipment, etc.)</td>
<td>seepage</td>
</tr>
<tr>
<td>Inadequate compaction of backfill around structures in</td>
<td>Excessive settlements; inadequate strength; provides</td>
</tr>
<tr>
<td>embankment</td>
<td>seepage path between structure and material which may</td>
</tr>
<tr>
<td></td>
<td>lead to internal erosion and failure</td>
</tr>
</tbody>
</table>

*Figure 245. Embankment construction deficiencies from (EM 1110-2-1913, 2000).*

6.3. **Underseepage**

Underseepage in pervious foundations beneath levees may result in excessive hydrostatic pressures beneath an impervious top stratum on the landside, sand boils and piping.

If a pervious substratum underlies a levee and extends both landward and riverward of the levee and where a relatively thin top stratum exists on the landside of the levee, problems are most severe.

As described above (§ 6.1), in order to minimize the risk of internal erosion or hydraulic heave, the use of plastic diaphragms or sheet piles is recommended. According to (EM 1110-2-1913) principal seepage control measures for foundation underseepage are:

- cutoff trenches;
- riverside impervious blankets;
- landside seepage berms;
- pervious toe trenches;
- pressure relief wells.

**Cutoffs**

Cutoffs are the most positive means of eliminating seepage problems (EM 1110-2-1913).

Cutoffs may consist of excavated trenches backfilled with compacted earth or slurry trenches usually located near the riverside toe. Cutoffs made by the slurry trench method (EM 1110-2-1913) can be made without a dewatering system, and their cost is favorable in many cases in comparison with cost of compacted earth cutoffs. Since a cutoff must penetrate approximately 95 percent or more of the thickness of pervious strata to be effective, it is not economically feasible to construct cutoffs where pervious strata are of considerable thickness. For this reason cutoffs will rarely be economical where they must penetrate more than about 12 m (EM 1110-2-1913).

Steel sheet piling is not entirely watertight due to leakage at the interlocks but can significantly reduce the possibility of piping of sand strata in the foundation. Open trench excavations can be readily made above the water table, but if they must be made below the water table, well point systems will be required.

**Riverside Blankets**

Where surface strata constitute impervious or semipervious blankets and they are continuous and extend riverward for a considerable distance, they can effectively reduce seepage flow and seepage pressures landside of the levee. The effectiveness of the blanket depends on its thickness, length, distance to the levee riverside toe, and permeability and can be evaluated by flow-net or approximate mathematical solutions. Riverside blanket should be protected against erosion.

**Landside Seepage Berms**

Landside berms may reinforce an existing impervious or semimpervious top stratum. Berms afford protection against heaving and sand boils that can occur when uplift pressures in pervious deposits underlying an impervious top stratum landward of a levee become greater than the effective weight of the top stratum. In fact, they provide an additional weight that counteracts the upward seepage forces and an additional length that reduce uplift pressures at the toe of the berm.
Berms are simple to construct and require very little maintenance but they require additional fill material and space.

There are different types of berms:

- impervious berms;
- semipervious berms that are constructed using material with permeability equal to or greater than the of the top stratum. In this case some seepage will pass through the berm and emerge but since the presence of the berm creates additional resistance to flow, subsurface pressures at the levee toe will be increased.
- Sand berms that are constructed with material with a minimum permeability of 0.01 cm per s. Sand berms require less material and occupy less space providing the same degree of protection. Sand berms offer less resistance to flow than semipervious berms but if they do not have the capacity to conduct seepage flow landward without excessive internal head losses they may also cause an increase in substratum pressures at the levee toe.
- Free–draining berms are composed of random fill overlying horizontal sand and gravel drainage layers with a terminal perforated collector pipe system, designed by the same methods used for drainage layers in dams. They can afford protection against underseepage pressures with less length and thickness than the other types of seepage berms but their cost is generally much greater.

Figure 246. Example of incorrect and correct berm length according to existing foundation conditions (EM 1110-2-1913, 2000).
Pervious toe trench

The main use of a pervious toe trench is to control shallow underseepage and protect the area in the vicinity of the levee toe.

A partially penetrating toe trench can improve seepage conditions at or near the levee toe where a levee is situated on deposits of pervious material overlain by little or no impervious material.

Pervious toe trenches may be used in conjunction with relief well systems: the wells collect the deeper seepage and the trench collects the shallow seepage (EM 1110-2-1913).

The sand backfill for trenches must be designed as a filter material (§ 2.1.3).

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**Figure 247. Typical partially penetrating pervious toe trench: Figure 5-2 from (EM 1110-2-1913, 2000).**

**Figure 248. Pervious toe trench located beneath landward slope: Figure 5-4 from (EM 1110-2-1913, 2000).**
Pressure relief wells

They may be installed along the landside toe of levees to reduce uplift pressure and are used where there is not enough space for using landside berms and where pervious strata underlying a levee are too deep or too thick to be penetrated by cutoffs or toe drains (EM 1110-2-1913).

Wells should adequately penetrate pervious strata and be spaced sufficiently close to intercept enough seepage. They need periodic maintenance (corrosion and bacterial clogging).
6.4. SEEPAGE THROUGH EMBANKMENTS

Seepage exiting on the landside slope in an embankment could result in high seepage forces decreasing the stability, sloughing of the slope or could lead to piping.

"In many cases, high water stages do not act against the levee long enough for this (seepage emergence on the landside slope) to happen, but the possibility of a combination of high water and a period of heavy precipitation may bring this about" (EM 1110-2-1913).

Landside berms or berms to control underseepage are useful to prevent seepage emergence on the landside slope.

Otherwise, to prevent seepage from emerging on the landside slope, horizontal or inclined drainage layers or toe drains should be incorporated in the levee section. These add an appreciable cost to the levee construction because of the request of
selected pervious granular material unless large quantities of pervious materials are available in the borrow areas.

**Figure 251. Seepage through embankment. Figure 5-8 from (EM 1110-2-1913, 2000).**

**Pervious Toe Drain**
A pervious toe provides a ready exit for seepage through the embankment and can lower the phreatic surface sufficiently so that no seepage will emerge on the landside slope. It can also be combined with partially penetrating toe trenches as a method for controlling shallow underseepage.

**Horizontal Drainage Layers**
Horizontal drainage layers provide the same purpose as a pervious toe but are advantageous because they can also serve to protect the base of the embankment against high uplift pressures where shallow foundation underseepage is occurring. In
fact, they can be extended further under the embankment requiring a relatively small amount of additional material.

Sometimes horizontal drainage layers serve also to carry off seepage from shallow foundation drainage trenches some distance under the embankment. According to (EM 1110-2-1913) horizontal drainage layers should have a minimum thickness of 18 in. (about 457 mm) for construction purposes.

**INCLINED DRAINAGE LAYERS**

Inclined drainage layers are one of the more positive means of controlling internal seepage. They completely intercept embankment seepage regardless of the degree of stratification in the embankment or the material type riverward or landward of the drain. Therefore the use of this type of drain allows the landside portion of a levee to be built of any material of adequate strength regardless of permeability. They are used extensively in earth dams while are rarely used in levee construction because of the cost.

Inclined drains must be tied into horizontal drainage layers to provide an exit for the collected seepage.

The design of drainage layers must satisfy the criteria for filter design (§ 2.1.3). The design of pervious toe drains and horizontal and inclined drainage layers must ensure that such drains have adequate thickness and permeability to transmit seepage without any appreciable head loss while at the same time preventing migration of finer soil particles. Placement and compaction of drainage layers must ensure that adequate density is attained but should not allow segregation and contamination to occur.
6.5. OVERTOPPING AND SURFACE EROSION

There exist soils with intrinsically high resistance to erosion. Therefore, it is possible to select and emplace soils with high resistance to erosion in order to achieve levee sections that can safely sustain some degree of overtopping. Briaud et al. (2001) presented a recommended chart which can be used to select soils for overtopping resistance. This chart represents a levee guideline for erosion resistance during overtopping (§ Figure 78). Briaud et al. (2001) suggested that EFA erosion tests should be used to predict levee behavior and ensure erosion resistance to overtopping.

Moreover, Briaud et al. (2001) found that an increase in compaction effort increases the resistance to erosion and that the increase is more pronounced for soils with higher fine content.

Crest and landside slopes of levees should be protected with some type of strengthening method such as turf reinforcement, soil strengthening or hard
armoring. The levee strengthening systems should resist the forces of fast–flowing, turbulent water that has overtopped the levee crest.

High performance turf reinforcement mat (HPTRM) is one of the strengthening systems that can be used on the crest and landward–side of earthen levee. The HPTRMs are three–dimensional TRMs joined at the intersections of randomly oriented nylon filaments with high tenacity polyester geogrid reinforcement at low strains. The open space of HPTRM (nearly 95% of space is open in this material) allows roots grow through and entwined with the HPTRM to reinforce the plant roots. In fact, as the grass roots grow through the open space of HPTRM, roots become entwined within the turf reinforced mat and the interlocking between roots and HPTRM can enhance the roots resistance against shear forces created by high water flow. A specific gravity of nylon in the HPTRM more than 1.0 ensures that the HPTRM will not float under any hydraulic condition.

(Li, Amini, & Pan, 2013) presented the result of a full–scale experiment of combined wave and surge overtopping of a trapezoidal levee cross section strengthened with HPTRM system that was conducted to study the hydraulic overtopping parameters and design guidelines.

For this research study, an experimental full scale embankment was constructed and overtopping tests were conducted for assessing the levee resiliency during overtopping (§ 2.2). The fine soil being used to construct the experimental levee was quarry waste from a limestone quarry.

The embankment was constructed for a section, T1, with the material from the quarry whereas, for a second section, T2, with the same material but added with the 2% of lime, in order to compare the resistances.

The comparison between the different topographic surveys, before and after the overtopping tests, for the two sections, T1 and T2, shows the occurred erosion. It is possible to observe that both the lime–stabilized soil and the not stabilized soil have a high resistance to erosion due to overtopping even without any reinforcement. Even at this extreme hydraulic loading, the levee slope were nearly unscathed by the water hence bare soil exhibits good resiliency to overtopping.

Therefore, the key finding from these tests was that the study material can withstand substantial overtopping without damage.
7. **Conclusions**

The main aim of the carried out research is the development of a set of technical guidelines for the design, upgrading and rehabilitation of river embankments. Therefore, this thesis deals with the main geotechnical problems characterizing river embankments. It is divided into two sections: while the first one deals with river embankment design and construction, the second section deals with inspection, maintenance, monitoring and remediation.

The first aspect dealt with is the design of the geotechnical campaign. In fact, for river embankments, site characterization requires considerable expertise and flexible budgets since they run for many kilometers sometimes hundreds of kilometers. Therefore, in the first chapter the needed criteria for a cost–effective campaign, considering the length of river embankments and the requested level of detail are discussed. This work describes the investigations carried out to explain three river embankment failures occurred in 2009 in the Serchio River embankments (Northern Tuscany, Italy). The described experience led to some general considerations.

The conducted geotechnical campaign (30 km), in addition to laboratory tests, included boreholes, CPTu, permeability tests, 2D geo–electric tomography and boreholes performed by the use of a continuous core drilling system. This last tool, as confirmed by CPTu results, has proved to be very useful to obtain the more accurate evaluation of the in situ soil density. In fact, sampling in very loose material is a delicate operation which can lead to wrong estimation of mechanical and physical parameters and the use of AF sampler was decisive to determinate physical properties of soil in the case under consideration. The use of Osterberg sampler was also suitable.

The capability of indirect methods (CPTu, 2D geo–electric tomography) to infer the soil stratigraphy has been analysed. CPTu test, economical and expeditious, has proved to be an indispensable tool to delineate soil stratigraphy, if the results are correctly calibrated against borehole logs. In fact, use of CPT test is suitable for cost–effectiveness purposes but calibration of SBT is necessary. CPT results combined with the borehole logs and the laboratory tests, provide extensive information.
Use of ERT is strongly influenced by soil water content thereby there is a great uncertainty about soil type. However, geo–electric investigations can be very useful to highlight anomalies and heterogeneities in the cross section.

In the second chapter, the main failure causes for river embankments are dealt with: internal erosion, overtopping and external erosion and mechanical failure. Defining and understanding the levee failure mechanisms is essential to identify design issues. In fact, not addressing geotechnical failure modes underestimates levee failure risks. All possible failure causes should be taken into account to design, in an appropriate way, both new levees and retrofitting of the existing embankments. Since each levee is unique, a detailed investigation and the evaluation of the potential failure modes and their consequences can allow a decision based on potential for failure and risk to be made.

As far as internal erosion is concerned, firstly, the phenomenon and the related terminology have been explained and several available criteria for assessing internal stability, self–filtering properties and filter performance have been illustrated.

Since there are evidences that internal erosion could be one of the possible failure causes for Serchio River embankments in December 2009, the internal stability of these embankments have been assessed applying the criteria for assessing suffusion risk and self–filtering properties. Unfortunately, there are not evidences of the construction details for these embankments. Moreover, in the Province of Lucca, the embankment top is unreachable (width between 1.2–3 m) hence all the boreholes were carried out from the bank and there were not grain distribution curves of samples retrieved from the embankment body. Therefore, the available data are limited. Even if the analysed soils do not seem to be particularly prone to internal erosion phenomena, the tests performed in this study do not allow such a clear conclusion. It should be remarked that investigations concern a very limited number of boreholes and it is not possible to exclude the presence of anomalies and heterogeneities within the embankments. Moreover, the considered embankments do not have filters.

Unfortunately, internal erosion can initiate under low hydraulic gradients, it can go on for years and even with seepage monitoring it is often difficult to detect thereby problems can develop at any time despite years of good performance. Therefore, every incidence of initiation of internal erosion needs to be taken seriously and embankment lacking cutoff walls, filters and toe drains should be considered safety deficient and regular inspection and seepage monitoring are essential requirements.
In order to decide what is needed, risk based evaluation of potential failure modes is necessary. Some consistent quantitative measure by which to judge priorities are needed. Since a balance between public safety and costs is necessary, risk analysis allow the greatest risk reduction with the available funding to be reached and the use of event trees is helpful. The decision to add a toe drain, filter or cutoff wall is so risk based and considers both the site specific case and overall situation in need to prioritize limited funds.

In the case under consideration it was decide to install a metallic sheet pile diaphragm within the body of the embankment both to prevent internal erosion phenomena and to reduce the risk of mechanical and hydraulic embankment failures. Moreover, the cross–section geometry of the embankment was modified by addicting a berm.

As far as overtopping is concerned, a full–scale test was carried out. An experimental full scale embankment was constructed and overtopping tests were conducted for assessing the levee resiliency during overtopping. For the construction of the experimental embankment a soil with a high intrinsic resistance to erosion was used, as confirmed by the overtopping testing. The fine soil being used to construct the experimental levee was quarry waste from a limestone quarry. The embankment was constructed for a section with the material from the quarry whereas, for a second section with the same material but added with the 2% of lime, in order to compare the resistances. The comparison between the different topographic surveys, before and after the overtopping tests, for the two sections shows the occurred erosion. It is possible to observe that both the lime–stabilized soil and the not stabilized soil have a high resistance to erosion due to overtopping even without any reinforcement. In fact, despite having been significantly overtopped for several hours (6 hours; for approximately 15 cm or more of sheet-flow overtopping), the experimental levee frontage suffered little damage. Even at this extreme hydraulic loading, the levee slopes were nearly unscathed by the water hence bare soil exhibits good resiliency to overtopping.

This work points to the need to evaluate river embankments for erodible soils and to the advantages of using soil with a high intrinsic resistance to erosion; clearly demonstrating the value of spending a bit more to acquire, place, and compact suitable materials. Conversely, the employment of locally available soils with poor resistance to erosion, sometimes protected after placement, results in initial cost-savings but at the cost of significantly increased risk of subsequent failure for the constructed levees.
As far as mechanical failure is concerned, following the geotechnical characterization of the Serchio River embankments, a number of numerical analyses have been carried out for various purposes: to clarify the causes of the December 2009 failures; to design appropriate repair of the failures and retrofit of the embankments in proximity of the failures and to identify the flood risk areas considering the whole extension of the embankments. The analyses were carried out by means of different computational tools and under different hypotheses. In particular, the flow conditions were considered both as stationary and transient. For the examined cases the different hypotheses change dramatically the results. The limit equilibrium method was used to assess the stability of the embankments under steady state flow conditions (areas close to the failures). For these analyses three different types of commercial software were used. The stability analyses were carried out using the Bishop simplified method with circular sliding surfaces. The different codes indicated very similar failure surfaces corresponding to the minimum (meaningful) values of the global safety factor. Some differences on the values of the global safety factor were observed by comparing the results obtained from the three codes. The analysis results show that, for the selected cross sections, the safety factors are rather small and approaching to unity, if the seepage forces are not considered. In the case of steady–state flow, safety factor drastically reduces becoming lower than one. Therefore, none of examined sections can sustain the steady flow. This result is not consistent with experimental evidence: the XVIII century embankments of the Serchio River, although they are quite weak, are standing up since centuries and failures occurred only during some flood events. Therefore, it is possible to conclude that the hypothesis of permanent flow is generally too cautious. The FEM analyses show that the safety margins of the considered sections, in absence of filtration, are assigned to the partial saturation of the embankments. Unfortunately, an appropriate characterization of the material under conditions of partial saturation was not available. Therefore, the FEM analyses were also aimed at determining the necessary time to approach the steady–state flow conditions. For the case under consideration it was estimated that 10 days are necessary to approach the steady–state flow conditions. This time is apparently much longer than the duration of the longest flood event (few hours). Therefore, it is possible to conclude that even if the hypothesis of permanent flow is generally too cautious, for the failures in December 2009, that occurred with the concurrence of various adverse factors like the melting of the snow because of a sudden temperature increase and the contemporary long raining period (two consecutive floods), it is reasonable to assume that the permanent flow conditions were probably reached.
Moreover, assessment of the risk areas, considering the whole extension of the embankments has been carried out following expeditious criteria which were based on the embankment geometry and the mechanical soil characteristics. More precisely, the geometric criterion can be summed up by the ratio between the embankment height and its base length: the higher the ratio, the higher the failure risk. As far as the soil strength is concerned, it was assumed as reference the minimum and maximum envelope of CPTu tip resistance that was carried out in the 3 km of embankments close to the December 2009 failures, since this area was recognized poorly constructed.

As far as the second section is concerned, an innovative method to evaluate the degree of compaction of both existing and new river embankments (fine grained soils) after their completion, by using laboratory and in situ testing, is described. The “laboratory” tip resistance, $q_{\text{cLAB}}$, is expressed as a function of the expected density and of the vertical and horizontal stress components when these latter are relevant. In fact, the experience has shown that, unlike the sands, for fine grained soils the tip resistance is essentially a function of the compaction energy. Such a dependence of $q_{\text{cLAB}}$ is obtained carrying out a number of repeated tests in the CC at given density and different consolidation stresses. In situ stresses are inferred by combining DMT results and an estimate of the vertical stress component. A comparison between the $q_{\text{cLAB}}$ profile, from CC testing, and the $q_{\text{c}}$, as inferred from in situ CPT, gives the possibility of assessing the density of existing embankments, while, for new embankments, the method defines the expected in situ $q_{\text{c}}$ for a given target density.

The methodology was positively evaluated at a river embankment constructed using compacted fine grained soils. The practical application of the method gave a verification of the correctness of the hypotheses. This work focuses on the influence of the boundary stresses on the tip resistance and on the relationship between tip resistance and compaction energy for fine grained soils. One of the most critical aspects of the method is the assessment of the tip resistance stress dependence. This work outlines the influence of the boundary stresses on the tip resistance.

For this purpose two different materials have been tested in the CC: a partially saturated fine grained soil and a dry sand. A number of tests, covering the range of stresses of interest have been carried out. The inferred data have provided the basis for establishing the empirical relationship between $q_{\text{c}}$ and stress state by doing regression analysis.
It was found that for the dry sand, at a given relative density value, horizontal stress is the most influential parameter on cone tip resistance. This statement is in agreement with available literature results.

Conversely, for the partially saturated fine grained samples the results seemed to show that suction and pre-consolidation stress were more influential than nominal stress. Therefore, further tests have been carried out in order to highlight the relationship between tip resistance, stress state and compaction energy. A first group of tests was carried out on samples that were reconstituted at different unit weights with a water content equal to the optimum moisture content while a second group of tests was carried out using samples that were reconstituted at a constant unit weight with different water contents. All the tests were carried out using boundary conditions BC1 in isotropic effective stress conditions.

The results show that as the unit weight of the sample increases, $q_c$ increases. It is necessary to spend more energy to reach a greater unit weight and higher relative compaction energy results in higher cone tip resistance values. Therefore, the tests have shown that for fine grained soil there is a clear relationship between tip resistance, degree of compaction and compaction energy. Confinement pressures have been found to have little effect on the measured tip resistance. In fact, the cone tip resistance is controlled primarily by the compaction energy and then it may be possible to relate cone tip resistance with compaction energy.

Finally, in order to control the degree of compaction of an embankment during construction, a correlation between $q_c$ and the elastic modulus, $E_d$, from Light Falling Weight Deflectometer (LFWD) is also proposed. Since LFWD is an economical and expeditious test that can be used for quick quality control during various construction stages, if it was possible to establish a correlation between $E_d$ and $q_c$ then it would be possible to have a simple prevision method for quality controls. In fact, it would be possible to predict the $q_c$ value for a given measured $E_d$. At the moment the available measurements are limited but show a good correlation between $q_c$ and $E_d$. Moreover, the available results show that, as with $q_c$, lower soil moisture contents result in higher $E_d$ values.

Since the results of stability analyses of river embankments under unsteady flow conditions are uncertain for the limited knowledge of the initial conditions and the lack of a detailed geotechnical characterization of both the embankment and the foundation soil, among the undertaken activities, an embankment of a detention basin was selected in order to clarify some basic aspects of the hydro–mechanical resistance of a bank of fine grained soil, which usually is in a condition of partial
saturation. A monitoring system was installed within the real scale embankment to realistically evaluate hydraulic and saturation conditions during different periods of the embankment life: dry seasons and flood events. The aims of the proposed monitoring system are to calibrate stability analyses under unsteady flow conditions and, at the same time, to assess the effectiveness of possible countermeasure, and in particular, of a plastic diaphragm that was realized using a quite innovative technique (Intersonda S.r.l., 2012).

The monitoring system and the geotechnical characterization of the study embankment are illustrated.

Monitoring of the soil moisture is based on the continuous measurement of the dielectric permittivity using the capacitive sensors 10HS Decagon Devices. The 10 HS Decagon sensors have been located in two different cross sections. For each cross section the monitoring system consists of: 10 sensors for the continuous monitoring of the soil moisture embedded into the soil at different depths (10HS Decagon Devices), including the data acquisition system (2 Dataloggers EM50 Decagon Devices) and the concrete protection cases and 2 open pipe piezometers. Continuous measurements of the piezometric head are carried out by means of pressure transducers located inside the piezometer pipes, below the permanent water table.

The installed monitoring system allows groundwater levels, water levels in the detention basin, daily rainfall data and soil moisture contents, as measured by 10HS Decagon sensors, to be compared.

It is possible to observe that the shallower moisture sensors are especially sensitive to the water infiltration after rainfall.

Both piezometer and moisture available measurements seem to indicate a high effectiveness of the constructed diaphragm. In fact, the 10HS Decagon sensors that are protected by the diaphragm have recorded stable values of the VWC over all the monitoring period and the groundwater levels measured by the piezometer that is protected by the diaphragm are systematically lower.

The laboratory calibration has shown that variations between different 10HS Decagon sensors measurements are small. Laboratory experiments have confirmed the sensitivity of sensor measurements to degree of compaction of the soil. In fact, it has been observed that as the relative density increases the error increases. Due to this dependency, soil sample used for laboratory calibration have been compacted as much as possible in order to approach the degree of compaction in field condition.

The standard calibration provided by the manufacturer does not accurately predict the absolute VWC over the whole measurement range. In particular, it has been
found not to be appropriate above 20 Vol. %. Therefore, a site-specific parabolic calibration function derived from parallel 10HS Decagon and laboratory measurements has been proposed.

Calibration function established under laboratory conditions has been transferred to field conditions and the groundwater levels and the water levels in the detention basin have been used to gain an idea about the location of the saturation line within the embankment during some events. It is possible to observe that the estimated saturation line is below a hypothetical saturation line with slope 1V:6H, as it was stated by some old Italian design criteria. This circumstance seems to confirm a good effectiveness of the constructed diaphragm.

These aspects will be subject to further research.

This experience has shown that the installed monitoring system needs to be improved. The low cost capacitance sensors should be supported by more accurate sensors (e.g. TDR sensors) that could be used as reference sensors in field conditions. The field monitoring data indicate that the shallower moisture sensors are especially sensitive to the water infiltration after rainfall. Therefore, a hydrological model of the embankment should take into account alteration by rainfall infiltration.

Future research will continue to provide a hydrological model of the embankment which reproduces also the response of the embankment to the rainfall infiltration.
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