Dike raising in a seismic, subsiding area 1
"Costa Oriental del Lago de Maracaibo"

June 1988
M.R. Tonneijck

master's thesis
text, appendices
Dike raising in a seismic, subsiding area

professors: Ir. A. Glerum, Ir. H. Velsink

conductor: Ir. G. Flórián

student: M. Tonneijck

DELFT UNIVERSITY OF TECHNOLOGY, FACULTY OF CIVIL ENGINEERING
HYDRAULICS SECTION

ROLAND HOLSTLAAN 679
2624 HT DELFT
tel. 015-565230
studentnr. 836350
PREFACE

In December 1984 professor ir. H. Velsink of the Faculty of Civil Engineering at Delft University of Technology and deputy director of Nedeco offered the raising of the Bolívar Coast Dikes at the East Coast of Lake Maracaibo as a topic for a master's thesis, to be carried out under cosupervision of professor ir. A. Glerum.

A survey visit to Venezuela resulted into a very elaborate project review from which a limited scope of this master's study was extracted. Project data were supplied by Maraven during the survey visit and by Geotechnics Delft during the actual study.

It had become obvious that liquefaction had to be given closer attention, for which an additional study was made at the Geotechnics Section of the Faculty of Civil Engineering under supervision of professor dr. ir. A. Verruijt. This study has been given a more general approach, finally restricted to the use of the Dutch CPT-method. This study has been laid down in part three.

ACKNOWLEDGEMENTS

I would like to thank Maraven and Nedeco for offering this practical possibility for my master's thesis. The contribution of Nedeco member Geotechnics Delft remained encouraging and was esteemed. In particular I thank project leader ir. F.J. Uijttewaal, who supplied data and advised about necessary limitations in the scope of the study. Second, professor dr. ir. F.B.J. Barends and Ing. W. Dekker, who enabled me the use of the Geotechnics Delft computer program MSEEP...helped me a lot.

I highly esteem the conduct in this master's thesis, given by ir. G.J. Flórián and I especially thank drs.T.G. Pons and ir. J.H. Reusink for offering their computer facilities so generously.

Delft, June 1988
<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>PREFACE</td>
<td>2</td>
</tr>
<tr>
<td>CONTENTS PART 1</td>
<td>3</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>7</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>14</td>
</tr>
<tr>
<td>LIST OF SYMBOLS USED</td>
<td>16</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>19</td>
</tr>
<tr>
<td>1.0 INTRODUCTION</td>
<td>22</td>
</tr>
<tr>
<td>2.0 THE COUNTRY, VENEZUELA</td>
<td>24</td>
</tr>
<tr>
<td>2.1 GEOGRAPHY</td>
<td>24</td>
</tr>
<tr>
<td>2.2 CLIMATE</td>
<td>26</td>
</tr>
<tr>
<td>2.3 POPULATION</td>
<td>28</td>
</tr>
<tr>
<td>2.4 ECONOMY</td>
<td>29</td>
</tr>
<tr>
<td>3.0 PROJECT REVIEW</td>
<td>31</td>
</tr>
<tr>
<td>3.1 INTRODUCTION</td>
<td>31</td>
</tr>
<tr>
<td>3.2 HISTORY</td>
<td>32</td>
</tr>
<tr>
<td>3.2.1 Oil discovery and its consequences</td>
<td>32</td>
</tr>
<tr>
<td>3.2.2 Coordination in coastal protection</td>
<td>32</td>
</tr>
<tr>
<td>3.2.3 Expertise in dike construction</td>
<td>33</td>
</tr>
<tr>
<td>3.3 IMPORTANCE OF COASTAL PROTECTION</td>
<td>34</td>
</tr>
<tr>
<td>3.3.1 Industry</td>
<td>34</td>
</tr>
<tr>
<td>3.3.2 Population</td>
<td>34</td>
</tr>
<tr>
<td>3.3.3 Agriculture</td>
<td>34</td>
</tr>
<tr>
<td>3.4 CLIMATE</td>
<td>35</td>
</tr>
<tr>
<td>3.4.1 Temperatures</td>
<td>35</td>
</tr>
<tr>
<td>3.4.2 Precipitation</td>
<td>35</td>
</tr>
<tr>
<td>3.4.3 Wind data</td>
<td>35</td>
</tr>
<tr>
<td>3.5 POLDER SYSTEM</td>
<td>37</td>
</tr>
<tr>
<td>3.5.1 Introduction</td>
<td>37</td>
</tr>
<tr>
<td>3.5.2 Dike system</td>
<td>37</td>
</tr>
<tr>
<td>3.6 DIKE REVIEW</td>
<td>40</td>
</tr>
<tr>
<td>3.6.1 Introduction</td>
<td>40</td>
</tr>
<tr>
<td>3.6.2 Longitudinal sections</td>
<td>41</td>
</tr>
<tr>
<td>3.6.3 Cross sections</td>
<td>41</td>
</tr>
<tr>
<td>3.6.4 Soil conditions</td>
<td>47</td>
</tr>
<tr>
<td>3.6.4.1 Introduction</td>
<td>47</td>
</tr>
<tr>
<td>3.6.4.2 Subsoil and dike material</td>
<td>48</td>
</tr>
<tr>
<td>3.6.5 Groundwater conditions</td>
<td>54</td>
</tr>
<tr>
<td>3.6.6 Piping</td>
<td>56</td>
</tr>
<tr>
<td>3.6.7 Constructions in the dike</td>
<td>57</td>
</tr>
</tbody>
</table>
APPENDICES

A.1 DETERMINATION OF SIGNIFICANT WAVE HEIGHT, GIVEN WIND SPEED, DURATION AND FETCH
   A.1.1 Introduction
   A.1.2 Method of representative wave
   A.1.3 Method of standard wave spectrum

A.2 PIPING
   A.2.1 Introduction
   A.2.2 Empirical/statistical investigation methods
   A.2.3 Evaluation of empirical/statistical methods
   A.2.4 Methods based on the theory of groundwaterflow

A.3 EARTHQUAKES
   A.3.1 Earthquakes characteristics
   A.3.2 Earthquake prediction
   A.3.3 Site response to earthquakes
   A.3.4 Ground motions
      A.3.4.1 Introduction
      A.3.4.2 Earthquake waves
      A.3.4.3 Local ground accelerations

A.4 LIQUEFACTION
   A.4.1 Introduction
   A.4.2 Phenomenon of earthquake induced liquefaction
   A.4.3 Factors affecting liquefaction potential
   A.4.4 Methods for the determination of liquefaction potential
   A.4.5 Propagation of liquefaction

A.5 DETERMINATION OF SHEAR STRENGTH BY CELL TESTS
   A.5.1 Shear strength, Coulomb's law
   A.5.2 Cell tests

A.7 OUTPUT TO CALCULATIONS WITH COMPUTER PROGRAM MSEEP

A.8 STABILITY CALCULATIONS WITH MODIFIED BISHOP FOR THE REPRESENTATIVE SECTIONS OF THE BOLIVAR COAST DIKES
LIST OF FIGURES

fig.nr. description

chapter 2

2.1 Fishermen's dwellings at Lago de Maracaibo
2.2 Table land in the Andes
2.3 Maracaibo, crowded traffic street
2.4 Caracas, a four million city
2.5 Caracas, modern city centre
2.6 Maracaibo, million city at lake entrance
2.7 Mixed population
2.8 Petroleum based economy, Maraven tank farm
2.9 Topography
2.10 Climates
2.11 Rainfall
2.12 Oil industry
2.13 Economy
2.14 Oil production and revenues

chapter 3

3.1 Project area
3.2a Tia Juana
3.2b Lagunillas
3.3 Mean monthly precipitation in the project area
3.4a Rainfall distribution in the Maracaibo Basin, February
3.4b idem August
3.5 Wind speed vs wind duration
3.6 General plan of the polders
3.7 General cross section of the polders
3.8 Lagunillas section 6, progr. 82+00
3.9 Lagunillas section 1
3.10 Lagunillas diversion dike and canal
3.11a General design of diversion dikes
3.11b General design of bypass dikes
3.12 Lagunillas northern partition dike, progr. 40+00
3.13 Lagunillas southern partition dike, progr. 94+00
3.14 Lagunillas diversion canal, Caño los Ahorcadas
3.15 Tia Juana principal approach channel, San Mateo
3.16 Canal Sibaragua
3.17 Canal Tamare
3.18 Benchmark
3.19 Bachaquero, old dike revetment, progr. 37+00
3.20 Pueblo Viejo, dike slope erosion
3.21 Lagunillas section 5, 5A, progr. 70+00
3.22 Tia Juana, old housing removed to raise dike
3.23a Longitudinal section, Tia Juana
3.23b Longitudinal section, Lagunillas
3.23c Longitudinal section, Pueblo Viejo, Bachaquero
3.24a Tia Juana, situation
3.24b Tia Juana, section 1, 2A
3.24c Tia Juana, section 2E, 2
3.24d Tia Juana, section 3, 4, 5, 6, 7, 8A
3.24e Tia Juana, section 8, 9
3.25a Lagunillas, situation
3.25b Lagunillas, section 1A, 1
3.25c Lagunillas, section 2, 3
3.25d Lagunillas, section 4, 5
3.25e Lagunillas, section 5A, 6
3.25f Lagunillas, section 7, 8A
3.25g Lagunillas, section 8, 9, 10A, 10, 11
3.26a Bachaquero/Pueblo Viejo, situation
3.26b Bachaquero, section 1, 1A, 1B
3.26c Bachaquero, section 2
3.26d Bachaquero, section 2A, 3
3.26e Bachaquero, section 3A, 4, 4A
3.26f Pueblo Viejo, section 1, 2, 3
3.26g Pueblo Viejo, section 1
3.26h Pueblo Viejo, section 4, 5
3.27 Lagunillas, section 4
3.28 Lagunillas, section 5
3.29 Lagunillas, section 5, 5A
3.30 Lagunillas, section 8
3.31 Bachaquero, section 2, N 1+00
3.32 Bachaquero, section 2, seepage near N 3+00
3.33 Bachaquero, section 2, inner slope erosion near progressiva N 3+00
3.34 Bachaquero, section 3, S10+50
3.35 Bachaquero, section 3, S10+50
3.36 Bachaquero, section 4, S53+00
3.37 Location of piezometers
3.38a Seepage through dike
3.38b Seepage through subsoil, impermeable layer
3.39 Old pipe in the dike, Bachaquero, N8+00
3.40 Lagunillas LS5
3.41 Tia Juana, San Mateo
3.42 Bachaquero, progr. N8+00
3.43 Lagunillas, pipe crossing in section 4
3.44 Lagunillas, pipe crossing in section 4-5
3.45 Lagunillas, well platform in the dike
3.46 Bachaquero, abandoned well, N 6+00
3.81 Variation of maximum ground movements with focal distance
3.82 Statistical earthquake data for the Bolivar Coast Region
3.83 Pseudo-static analysis for earthquake loading
3.84 Scattered distribution of peak ground accelerations for short focal distances
3.85 Piping phenomenon
3.86 Definition of seepage lengths for Bligh, Lane and Griffith
3.87 Dam failures after Lane
3.88 Dam failures after Bligh
3.89 Vertical flow round sheet pile wall
3.90 Dike on uniform homogenous subsoil
3.91 Concentrated groundwaterflow
3.92 Upbursting of overlying impermeable layer
3.93 Concentration of flow after upbursting
3.94 Idealized earthquake loading on soil particle
3.95 Influence of density on liquefaction potential
3.96 Influence of overconsolidation ratio on liquefaction
3.97 Stress ratios causing liquefaction versus SPT-value
3.98 Cyclic triaxional test with controlled deviator stress
3.99 Torsional simple shear test
3.100 Layer susceptible to liquefy during earthquake under a dike
3.101 Progressive liquefaction analysis
3.102 Progressive liquefaction
3.103 Outer slope failure due to liquefaction
3.104 Progressive changes in pore water pressures after earthquake
3.105 Liquefaction phenomenon
3.106 Correlation between liquefaction resistance and cone penetration resistance
3.107 Influence of sampling on cyclic loading resistance
3.108 Tia Juana, boring samples, 1957
3.109 Tia Juana, progr. 16+80
3.110 Tia Juana, progr. 21+40
3.111 Tia Juana, progr. 30+30
3.112 Lagunillas, boring samples, 1957
3.113 Lagunillas, progr. 29+50
3.114 Lagunillas, progr. 84+50
3.115 Tía Juana, grain size distributions of samples 1-9, progr. 25+40
3.116 idem, 20-25, progr. 17+10
3.117 idem, 62-68, progr. 15+00
3.118 idem, 85-94, progr. 16+80
3.119 idem, 95-104, progr. 16+80
3.120 idem, 105-118, progr. 30+30
3.121 Lagunillas, grain size distributions of samples 51-61, progr. 29+50
3.122 idem, 31-35, progr. 38+30 and 52+80
3.123 idem, 63-74, progr. 62+20, 72+00 and 7-8
3.124 idem, 36-41, progr. 84+20
3.125 idem, 42+49, progr. 94+00
3.126 Tía Juana, Mohr's circles, quick cell test samples
3.127 Lagunillas, Mohr's circles, quick cell test samples
3.128 Coulomb's Law
3.129 Cell test apparatus
3.130 Slow cell test
3.131 Quick cell test
3.132 Soil classification
3.133 Grain size distributions of soils to raise the dikes
3.134 Idem in triangle
3.135 Cone penetration tests, DSML 1984-1985
3.136 Cyclic triaxial tests

chapter 4
4.1a Failure system, overtopping
4.1b Failure system, excess seepage
4.1c Failure system, piping
4.2 Statistical dike design for a 4000 year return period

chapter 5
5.1 Tía Juana section 3, assumptions
5.2 Lagunillas section 5, assumptions
5.3 Lagunillas section 8A, assumptions

chapter 6
6.1 Definition scheme of crest level determination
6.2 Definition of wave runup
6.3 Primary settlement
6.4 Secondary settlement, Keverling Buisman
6.5 Primary and secondary compression moduli, below and above preconsolidation load p_c
6.6 Stress distribution under dike raisings
6.7 Wind setup at the Bolivar Coast Dikes of Lake Maracaibo
6.8 Suggestion that there is a mechanism that secondary settlement starts at t=0
6.9 Tia Juana 3, settlement
6.10 Lagunillas 5, settlement
6.11 Lagunillas 8A, settlement

Chapter 7
figures in appendix 7

Chapter 8
8.1a Critical dike slope
8.1b Loss of dike slope stability
8.2a Length of rupture
8.2b Dish shaped rupture
8.3 Factors affecting dike slope stability
8.4 The method of slices
8.5 Fellenius' method of slices
8.6 Bishop's method of slices, for submerged slopes
8.7a Tia Juana 3, stability of outer slope
8.7b Tia Juana 3, inner slope stability
8.8a Lagunillas 5, outer slope stability
8.8b Lagunillas 5, inner slope stability
8.9a Lagunillas 8A, outer slope stability
8.9b Lagunillas 8A, soil and water stresses in case of liquefaction, outer slope
8.9c Lagunillas 8A, stability of outer slope, adaptations
8.9d Lagunillas 8A, insufficient inner slope
8.9e Lagunillas 8A, inner slope stability
8.9f Lagunillas 8A, soil and water stresses in case of liquefaction, inner slope

Chapter 9
9.1a Failure by liquefaction along deep slip circle
9.1b Failure by liquefaction, settlement
9.2 Liquefaction as secondary failure
9.3 Requirements for dike slopes in seismic areas
9.4 Grain size distribution of the sand layer susceptible to liquefaction
9.5 Low cone penetration resistance for the layer susceptible to liquefaction
9.6 Dynamic equilibrium of a sand beach
9.7 Characteristics of dike slope deformation
9.8 Installation of compaction piles
9.9 Lagunillas 8A, solution with locking and drainage

Chapter 10
10.1 Lane's seepage length with and without berm
10.2 Principle of Lane, applied for Tia Juana 3
10.3 Principle of Lane, applied for Lagunillas 5,
10.4 Principle of Lane, applied for Lagunillas 8A
10.5 Erosion length
10.6 Parabolic and linear water head distribution for
    thick and thin soil layers
10.7 Higher allowable water head for thin layers, compared
to Lane's estimate
10.8 Critical hydraulic gradients

chapter 11
11.1 Tía Juana 3, seepage
11.2 Lagunillas 5, seepage
11.3 Lagunillas 8A, seepage
11.4 Significant forces in dike slope surface erosion
11.5 Groundwaterflow near the dike slope surface
11.6 Idem, underwater slope
11.7 Horizontally approaching groundwaterflow
11.8 Waterflow through relatively permeable top layer
11.9 Safe inner slopes [TAW]  

chapter 12
12.1 Tía Juana 3, future profile
12.2 Lagunillas 5, future profile
12.3 Lagunillas 8A, future profile
<table>
<thead>
<tr>
<th>tablenr.</th>
<th>description</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Venezuelan export</td>
</tr>
<tr>
<td>2.2</td>
<td>Venezuelan import</td>
</tr>
<tr>
<td>2.3</td>
<td>Venezuelan oil production (1983)</td>
</tr>
<tr>
<td>3.1</td>
<td>Expenses for coastal protection and drainage</td>
</tr>
<tr>
<td>3.2</td>
<td>Wind and wave data supplied by CSV</td>
</tr>
<tr>
<td>3.3a</td>
<td>Significant wave height for certain wind speed-duration-combinations (method of representative wave)</td>
</tr>
<tr>
<td>3.3b</td>
<td>idem (method of standard wave spectrum)</td>
</tr>
<tr>
<td>3.4</td>
<td>List of drawings for typical dike sections and general plans of the coast line.</td>
</tr>
<tr>
<td>3.5a</td>
<td>Tia Juana typical soil profiles</td>
</tr>
<tr>
<td>3.5b</td>
<td>Lagunillas typical soil profiles</td>
</tr>
<tr>
<td>3.6a</td>
<td>Tia Juana, piezometer data and readings</td>
</tr>
<tr>
<td>3.6b</td>
<td>Lagunillas, piezometer data and readings</td>
</tr>
<tr>
<td>3.6b</td>
<td>idem, continued</td>
</tr>
<tr>
<td>3.6c</td>
<td>Bachaquero, idem</td>
</tr>
<tr>
<td>3.7</td>
<td>Lake water level fluctuations</td>
</tr>
<tr>
<td>3.8</td>
<td>Revetment roughness factors</td>
</tr>
<tr>
<td>3.9</td>
<td>Time and dimension scales for various mechanisms of subsidence</td>
</tr>
<tr>
<td>3.10</td>
<td>Soil samples of Bolivar Coast wells, intersection point pressures</td>
</tr>
<tr>
<td>3.11a</td>
<td>Topographical data on subsidence for coast dikes in Tia Juana</td>
</tr>
<tr>
<td>3.11b</td>
<td>idem for dikes in Lagunillas</td>
</tr>
<tr>
<td>3.12a</td>
<td>Expected subsidence values in meters in the coastal area for Tia Juana (production method)</td>
</tr>
<tr>
<td>3.12b</td>
<td>idem for Lagunillas</td>
</tr>
<tr>
<td>3.12c</td>
<td>idem for Bachaquero</td>
</tr>
<tr>
<td>3.13</td>
<td>Modified Mercalli Scale</td>
</tr>
<tr>
<td>3.14</td>
<td>Earthquake data of nearby faults</td>
</tr>
<tr>
<td>3.15</td>
<td>Design earthquakes for the Bolivar Coast dikes</td>
</tr>
<tr>
<td>3.16</td>
<td>Values of regional seismic constants</td>
</tr>
<tr>
<td>3.17</td>
<td>Safe coefficients of Bligh, Griffith and Lane</td>
</tr>
<tr>
<td>3.18</td>
<td>Pseudo-static analysis of dams with slope failures during earthquakes</td>
</tr>
<tr>
<td>3.19</td>
<td>Factors affecting soil liquefaction, stress ratio causing liquefaction and penetration resistance N</td>
</tr>
</tbody>
</table>
3.20 Soil samples of the DSML investigation, 1957
3.21 Soil data of the DSML investigation, 1957
3.22 Locations of the DSML soil investigation, 1981-1982
3.23 Soil tests 1984-1985
3.24 Soil porosities

chapter 6
6.1 Primary and secondary compression moduli, Terzaghi
6.2 Primary and secondary compression moduli, Keverling-Buisman

chapter 8
8.1 Uncertainties in the determination of soil characteristics [TAW]
8.2 Stability factors of the representative sections of the Bolivar Coast Dikes

chapter 9
9.1 Description of the loosely packed fine sand layer under dike section Lagunillas 8A
9.2 Summary of soil improvement methods
9.3 Choice of possible solutions for Lagunillas 8A

chapter 10
10.1 Principle of Lane for the representative dike sections of the Costa Bolívar
10.2 Hydraulic gradients with respect to hydraulic fracturing for the representative dike sections of the Costa Bolívar.
10.3 Weight and upthrust with respect to hydraulic fracturing for the representative dike sections of the Costa Bolívar.

chapter 11
11.1 Capacity of some pumping stations
## LIST OF SYMBOLS

<table>
<thead>
<tr>
<th>symbol</th>
<th>symbol description</th>
<th>unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>acceleration</td>
<td>m/s²</td>
</tr>
<tr>
<td>b</td>
<td>width of slice of soil</td>
<td>m</td>
</tr>
<tr>
<td>c</td>
<td>true cohesion</td>
<td>kN/m²</td>
</tr>
<tr>
<td>c'</td>
<td>apparent cohesion</td>
<td>kN/m²</td>
</tr>
<tr>
<td>cₜ</td>
<td>wave group celerity</td>
<td>m/s</td>
</tr>
<tr>
<td>Cₜ</td>
<td>piping constant of Bligh</td>
<td>-</td>
</tr>
<tr>
<td>Cₛ</td>
<td>piping constant of Griffith</td>
<td>-</td>
</tr>
<tr>
<td>Cₖ</td>
<td>piping constant of Lane</td>
<td>-</td>
</tr>
<tr>
<td>Cₐ,Cₚ</td>
<td>primary compression modulus</td>
<td>-</td>
</tr>
<tr>
<td>C一站式ₐ,Cₚ一站式</td>
<td>primary compression modulus</td>
<td>-</td>
</tr>
<tr>
<td>Cₛ一站式</td>
<td>secondary compression modulus</td>
<td>-</td>
</tr>
<tr>
<td>d,D</td>
<td>soil layer thickness</td>
<td>m</td>
</tr>
<tr>
<td>dₓₓ,Dₓₓ</td>
<td>grain size</td>
<td>m</td>
</tr>
<tr>
<td>d</td>
<td>earthquake displacement</td>
<td>m</td>
</tr>
<tr>
<td>DWL</td>
<td>design water level</td>
<td>m</td>
</tr>
<tr>
<td>f</td>
<td>roughness factor of dike slope</td>
<td>-</td>
</tr>
<tr>
<td>fₒ</td>
<td>frequency</td>
<td>-</td>
</tr>
<tr>
<td>F</td>
<td>wind fetch</td>
<td>m</td>
</tr>
<tr>
<td>F</td>
<td>stability factor</td>
<td>m</td>
</tr>
<tr>
<td>Fₒ</td>
<td>force of backflowing water in calculation of wind setup</td>
<td>N</td>
</tr>
<tr>
<td>Fₒ₂</td>
<td>driving force (seepage)</td>
<td>m</td>
</tr>
<tr>
<td>Fₒ₂ₑ</td>
<td>equivalent wind fetch</td>
<td>m</td>
</tr>
<tr>
<td>Fₒ₂ₑ,₁ₑ</td>
<td>stability factor with earthquake and liquefaction</td>
<td>-</td>
</tr>
<tr>
<td>Fₘ</td>
<td>resisting force (seepage)</td>
<td>kN</td>
</tr>
<tr>
<td>Fₒ</td>
<td>force of wind exerted on water surface</td>
<td>N</td>
</tr>
<tr>
<td>g</td>
<td>acceleration of gravity</td>
<td>m/s²</td>
</tr>
<tr>
<td>Gₐ</td>
<td>upthrust of water</td>
<td>kN</td>
</tr>
<tr>
<td>h</td>
<td>water head</td>
<td>m</td>
</tr>
<tr>
<td>hₐ</td>
<td>water head in sand layer (piping)</td>
<td>m</td>
</tr>
<tr>
<td>h</td>
<td>height of slice of soil</td>
<td>m</td>
</tr>
<tr>
<td>H</td>
<td>retaining height</td>
<td>m</td>
</tr>
<tr>
<td>Hₛ</td>
<td>significant wave height</td>
<td>m</td>
</tr>
<tr>
<td>i</td>
<td>hydraulic gradient</td>
<td>m/m</td>
</tr>
<tr>
<td>iₑr</td>
<td>critical hydraulic gradient</td>
<td>m/m</td>
</tr>
<tr>
<td>iₛ</td>
<td>inclination of water surface in calculation of wind setup</td>
<td>m</td>
</tr>
<tr>
<td>I</td>
<td>Mercalli earthquake intensity</td>
<td>-</td>
</tr>
<tr>
<td>k</td>
<td>permeability</td>
<td>m/s</td>
</tr>
<tr>
<td>kₒ</td>
<td>horizontal permeability</td>
<td>m/s</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
<td>Unit</td>
</tr>
<tr>
<td>--------</td>
<td>----------------------------------------------------------------------------</td>
<td>-------</td>
</tr>
<tr>
<td>$k_v$</td>
<td>vertical permeability</td>
<td>m/s</td>
</tr>
<tr>
<td>$K_0$</td>
<td>lateral pressure coefficient</td>
<td>-</td>
</tr>
<tr>
<td>$L$</td>
<td>length of circular slip surface</td>
<td>m</td>
</tr>
<tr>
<td>$L_i$</td>
<td>idem. of one slice</td>
<td>m</td>
</tr>
<tr>
<td>$L$</td>
<td>wavelength</td>
<td>m</td>
</tr>
<tr>
<td>$L_s$</td>
<td>seepage length</td>
<td>m</td>
</tr>
<tr>
<td>$L_w$</td>
<td>horizontal seepage length</td>
<td>m</td>
</tr>
<tr>
<td>$L_v$</td>
<td>vertical seepage length</td>
<td>m</td>
</tr>
<tr>
<td>$L_{BL}$</td>
<td>lake bottom level</td>
<td>m</td>
</tr>
<tr>
<td>$M$</td>
<td>earthquake magnitude</td>
<td>-</td>
</tr>
<tr>
<td>$M_{CL}$</td>
<td>minimum crest level</td>
<td>m</td>
</tr>
<tr>
<td>$M_{CH}$</td>
<td>minimum crest construction height</td>
<td>m</td>
</tr>
<tr>
<td>$MLLL$</td>
<td>mean low lake level</td>
<td>m</td>
</tr>
<tr>
<td>$n$</td>
<td>porosity</td>
<td>%</td>
</tr>
<tr>
<td>$n$</td>
<td>seismic coefficient</td>
<td>-</td>
</tr>
<tr>
<td>$N$</td>
<td>number of stress cycles</td>
<td>-</td>
</tr>
<tr>
<td>$P_e$</td>
<td>preconsolidation load</td>
<td>kN/m²</td>
</tr>
<tr>
<td>$P_{GL}$</td>
<td>polder ground level</td>
<td>m</td>
</tr>
<tr>
<td>$q$</td>
<td>specific discharge</td>
<td>m/s</td>
</tr>
<tr>
<td>$Q'$</td>
<td>seepage per m' dike</td>
<td>m²/m².s</td>
</tr>
<tr>
<td>$r_s$</td>
<td>stress reduction factor</td>
<td>-</td>
</tr>
<tr>
<td>$R$</td>
<td>earthquake focal distance or epicentral distance</td>
<td>km</td>
</tr>
<tr>
<td>$t$</td>
<td>time</td>
<td>s</td>
</tr>
<tr>
<td>$t_c$</td>
<td>time needed for consolidation</td>
<td>s</td>
</tr>
<tr>
<td>$t^*$</td>
<td>practical end of consolidation</td>
<td>s</td>
</tr>
<tr>
<td>$t_{eq}$</td>
<td>equivalent wind duration</td>
<td>s</td>
</tr>
<tr>
<td>$t_w$</td>
<td>wind duration</td>
<td>s</td>
</tr>
<tr>
<td>$T$</td>
<td>wave period</td>
<td>s</td>
</tr>
<tr>
<td>$u$</td>
<td>pore water pressure</td>
<td>kN/m²</td>
</tr>
<tr>
<td>$u$</td>
<td>uniformity coefficient $d_u/d_o$</td>
<td>-</td>
</tr>
<tr>
<td>$u_w$</td>
<td>wind speed</td>
<td>m/s</td>
</tr>
<tr>
<td>$v$</td>
<td>velocity</td>
<td>m/s</td>
</tr>
<tr>
<td>$W$</td>
<td>weight of slice of soil</td>
<td>kN</td>
</tr>
<tr>
<td>$W_1$</td>
<td>dry weight of slice of soil</td>
<td>kN</td>
</tr>
<tr>
<td>$W_2$</td>
<td>submerged weight of slice of soil</td>
<td>kN</td>
</tr>
<tr>
<td>$Z_a$</td>
<td>elevation head of sand layer in piping evaluation</td>
<td>m</td>
</tr>
<tr>
<td>$Z_{ax}$</td>
<td>vertical wave runup exceeded by i% of the waves</td>
<td>m</td>
</tr>
<tr>
<td>$Z_{ph}$</td>
<td>elevation head of phreatic line</td>
<td>m</td>
</tr>
<tr>
<td>$Z_s$</td>
<td>significant wave runup</td>
<td>m</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>slope angle</td>
<td>-</td>
</tr>
<tr>
<td>$\alpha_o$</td>
<td>primary compression modulus</td>
<td>-</td>
</tr>
<tr>
<td>$\alpha_s$</td>
<td>secondary compression modulus</td>
<td>-</td>
</tr>
<tr>
<td>$\beta$</td>
<td>angle of wave crest to the normal of the dike</td>
<td>-</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
<td>Unit</td>
</tr>
<tr>
<td>--------</td>
<td>--------------------------------------------------</td>
<td>----------</td>
</tr>
<tr>
<td>$\delta_c$</td>
<td>consolidation settlement</td>
<td>m</td>
</tr>
<tr>
<td>$\delta_i$</td>
<td>immediate settlement</td>
<td>m</td>
</tr>
<tr>
<td>$\delta_s$</td>
<td>secondary settlement</td>
<td>m</td>
</tr>
<tr>
<td>$\delta_{ss}$</td>
<td>soil, subsidence</td>
<td>m</td>
</tr>
<tr>
<td>$\delta_{tot}$</td>
<td>total settlement</td>
<td>m</td>
</tr>
<tr>
<td>$\gamma_d$</td>
<td>unit weight of dry soil</td>
<td>kN/m$^3$</td>
</tr>
<tr>
<td>$\gamma_n$</td>
<td>unit weight of saturated soil</td>
<td>kN/m$^3$</td>
</tr>
<tr>
<td>$\gamma_s$</td>
<td>unit weight of solids</td>
<td>kN/m$^3$</td>
</tr>
<tr>
<td>$\gamma_w$</td>
<td>unit weight of water</td>
<td>kN/m$^3$</td>
</tr>
<tr>
<td>$\rho_d$</td>
<td>density of soil</td>
<td>kg/m$^3$</td>
</tr>
<tr>
<td>$\rho_w$</td>
<td>density of water</td>
<td>kg/m$^3$</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>total stress</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>$\sigma'$</td>
<td>effective stress</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>$\sigma_0'$</td>
<td>initial effective stress</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>$\sigma_i$</td>
<td>principal stresses</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>$\sigma_0$</td>
<td>deviator stress</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>$\tau$</td>
<td>shear stress</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>$\tau_{eq}$</td>
<td>earthquake induced shear stress</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>$\tau_s$</td>
<td>maximum shear stress</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>$\tau_m$</td>
<td>mobilised shear stress</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>$(\tau/\sigma_0')_{eq}$</td>
<td>stress ratio for earthquakes</td>
<td>-</td>
</tr>
<tr>
<td>$(\tau/\sigma_0')_{field}$</td>
<td>resistance to liquefaction in field, 'field strength'</td>
<td>-</td>
</tr>
<tr>
<td>$(\tau/\sigma_0')_{slope}$</td>
<td>stress ratio in soil induced by overlying dike slope</td>
<td>-</td>
</tr>
<tr>
<td>$\phi$</td>
<td>true angle of friction</td>
<td>-</td>
</tr>
<tr>
<td>$\phi'$</td>
<td>apparent angle of friction</td>
<td>-</td>
</tr>
<tr>
<td>$\bar{k}$</td>
<td>water potential $k$</td>
<td>m/s</td>
</tr>
</tbody>
</table>
SUMMARY

DIKE RAISING IN A SEISMIC SUBSIDING AREA
"Costa Oriental del Lago de Maracaibo"

SITUATION
The east coast of Lake Maracaibo in Venezuela intersects with three large oil fields, Tía Juana, Lagunillas and Bachaquero. Formerly, the coast was low, but still not defended. Since the start of oil production in 1926, both the land and the lake bottom subsided. A dike had to bring protection: the origin of the three polders in the present situation.

In 1984 the ground level just behind the dike was at some three meters below the water level of the lake. Oil production continues and hence land subsidence: the dikes need continuous raising. The purpose of this master's thesis is to achieve a dike design for the year 2030, taking account of the seismicity of the project area.

TECHNICAL ASPECTS
Land subsidence has two consequences, first the lake in front of the dikes gets deeper and the land behind the dikes becomes lower with regard to the lake water level: increase of the retaining height.

A deeper lake (a possibly present foreshore drowns) causes a basically stronger wave attack on the dike and thus a higher wave runup.

The increase of the retaining height and the consequently higher water stresses in the dike body and foundation layers
has mainly a negative effect on the stability of the dike slopes, on the liquefaction potential of certain sand layers, on piping with backward erosion and seepage.

**Earthquakes** may cause loss of slope stability immediately, but also after liquefaction of loosely packed sand layers.

**DESIGN**

The choice of an appropriate outer slope and revetment, the expected land subsidence (i.e. oil production) and available wind, wave and water level data determine the minimum required crest level.

For the primarily assumed situation of 2030 the groundwater level and water stresses are determined with help of the finite element program MSEEP of Geotechnics Delft, originally designed for main frame, but adapted for use on the personal computer.

**Pseudo-static stability** calculations, i.e. considering horizontal mass force of earthquakes, show that the primarily assumed dike profiles need adaptations. Where liquefaction may occur, wide berms are necessary.

The **liquefaction of loosely packed sand layers** is the most fascinating problem of these lake dikes. In an additional study one of the known methods for the evaluation of liquefaction potential was more closely considered: CPT-method. The results of this study yield one dike section where liquefaction is well possible. Wider dike berms and soil improvement are necessary.

The prevention of piping was verified with the principle of Lane and possible occurrence was shifted away from the dike body by the application of an inner berm. Recent views on the allowable retaining height have been incorporated in the dike design.

The amount of seepage appears to be sufficiently low, the already existing pumping capacity is adequate.

**OPTIMALISATION**

The adaption of the dike profiles involves the application of berms and gentler slopes, which offers the possibility for lower crest levels, with favourable effects on stability.
1.0 INTRODUCTION

Oil production in fields under the bottom of the eastern part of Lake Maracaibo (Venezuela) and the coastal region there cause serious land subsidence, already since 1930. To prevent the land from inundation the coast needed to be defended. The oil production still continues and thus land subsidence, too.

This report is written as a master's thesis at the Faculty of Civil Engineering of the Delft University of Technology. Professor ir. H. Velsink of the Netherlands Engineering Consultants, Nedeco, presented the subject. Nedeco was consultant for a dike and drainage masterplan in the project area. The thesis was further carried out under supervision of professor ir. A. Glerum and it was more closely conducted by ir. G. Flórián.

In 1968 Nedeco presented guidelines for raising of the formerly Bolivar Coast Dikes. Presently, they are referred to as the Costa Oriental del Lago de Maracaibo, COLM dikes. In this report mostly the old name has been used. These guidelines also formed the base of this study, where it should be kept in mind, that under any circumstance, it had to be avoided to simply follow Nedeco's present study for the dike masterplan. Nedeco (Delft Geotechnics) and Maraven, the Venezuelan oil company that gave the assignment, were very cooperative in supplying data.

After a short introduction to the country Venezuela (chapter 2), the importance and the present state of the dikes is elaborately set forth (chapter 3). Apart from the 'normal' dike design problems (normal in Dutch conditions), the subsidence phenomenon and the seismicity of the project area get very much attention. Where too much background is necessary, a step aside has been made to appendices. The danger for liquefaction of loosely packed, fine sand layers under the dikes made it necessary to make an additional investigation into liquefaction as a phenomenon and the evaluation of liquefaction potential, with implications for the design of dikes [see lit. 25,26].

Dikes fail usually by a way of failure, that was not foreseen. To avoid such disaster, the long, traditional experience with dikes in the Netherlands is useful to set up failure trees (chapter 4). This opens possibilities to assess ways of failure thoroughly. A serious lack of water level data prevents a modern probabilistic approach of the
1 introduction

design.

In chapter 5 the information about the dikes, gathered in chapter 3, has been compressed and limited to three dike sections that are representative. The aim is to make a safe dike design for the year 2030, taking into account the predicted soil subsidence (oil production) and the possibility of earthquake induced liquefaction. Knowing water levels, experience suggests preliminary dike profiles, that can be used to calculate crest heights (chapter 6), water pressures, phreatic lines (chapter 7), and stability (chapter 8).

The stability forces to adaptation of the preliminarily assumed dike profiles, a dike under horizontal earthquake accelerations requires firm berms. Especially sections where liquefaction may occur won't be stable, when only a simple dike profile has been assumed (chapter 8).

In chapter 9 a series of possible solutions to construct a safe dike under circumstances with earthquake induced liquefaction pass in review.

The calculated profiles have to be checked on piping (chapter 10) and seepage (chapter 11).

The results of the design calculations are resumed in chapter 12 for the dike design, that is sufficient for the year 2030.
2. THE COUNTRY VENEZUELA

When the Spaniards came to Venezuela in 1499 they found a poor country, thinly populated with Indians. They gave the country its name 'Little Venice' when they saw the Indian dwellings on piles in Lake Maracaibo and Río Limón. [see figure 2.1] In 1823 the great Simón Bolívar liberated Gran Colombia from the Spaniards and before Bolívar's death in 1830 a certain Páez declared Venezuela an independent republic.

A fabulous change came over the country when oil was discovered in the Maracaibo area in 1914. Up to that era the country lived from its agrarian resources; fifteen years later petroleum covered already 50% of its expenses.

2.1 GEOGRAPHY [see figure 2.9]

Venezuela is situated on the north coast of the South American continent. The country borders on Guyana (formerly British) to the East, on Brasil to the South and on Colombia to the West. To the North Venezuela is bounded by the Caribbean and faces the islands of the Lesser Antilles.

The total area of the country, not including the claimed zone, is 917000 km², which is as big as France, Switzerland, West-Germany and the Benelux countries together.

Geographically the country can be divided into five areas:

1. Sierra de Perijá, Cordillera de los Andes Venezolanos, Sierra del Mar.

The Perijá mountains establish the northern stretch of the border to Colombia.

The Andes mountain chain is in Venezuela lower than in Peru, but the three peaks of the Sierra de Mérida, Bolívar (5007 m), Humboldt (4942 m) and Bonpland are still higher than the Mont Blanc in the Alps. A typical feature of the Andes is the so called 'table land' (meseta) with intensive agriculture, (corn, coffee) and cities like Trujillo, San Cristóbal, Mérida. [see figure 2.2]

The coastal mountains are lower, between 500 and 2500 m.
Venezuela’s capital Caracas and important industrial centres like Maracay and Valencia are situated in this mountain chain.

2. Cuenca de Maracaibo.
The Maracaibo basin is surrounded by the Perijá mountains, the Andes and the Cordillera del Norte. Lake Maracaibo dominates this basin, it has an open connection to the Golfe de Venezuela and the Caribbean. Tertiary layers contain rich oil and gas reservoirs. Maracaibo is Venezuela’s second biggest city. [see figure 2.3]

3. Valleys and hills of Falcon, Cara and Yaracuy.
This region forms a transition from the coast to the vigourous landscape of the Andes and the Cordillera del Norte.

4. Los Llanos
The central part of Venezuela is formed by the Plains. They have a savannah-like landscape with short grass and scattered trees. To the East the Plains merge into the Orinoco delta.

5. Guayana Venezolana
Venezuelan Guayana consists of the states Bolivar and the Territoras Amazonas. It makes up 45% of the country’s territory, but only 3% of the population lives there. It is mostly covered with jungle. To the South the Roraima mountains go as high as 2600 m. Near Canaima runs the Río Carrao with the Canaima Falls, and the highest waterfall in the world: Salto de Angel, 1005 m.

The Orinoco river, 2750 km long, is with an average discharge of 25000 m³/s near its mouth the third river of the world. Its basin takes up 70% of the country’s territory, The main affluent is the Río Apure. The Orinoco carries its water just outside the girdle of the Antilles into the Atlantic Ocean.
2.2 CLIMATE

Venezuela's situation between 0°45' N and 12°12' N would imply a purely tropical climate. The differences in altitude and the Caribbean Sea however, cause an extreme variety in types of climate. [see figures 2.10 and 2.11]

1. Tropical climate.
In Southern Guayana and in western Zulia a tropical climate does exist, characterized by high temperatures during the whole year, except in the higher parts of the mountains. It rains about 250 days per year, the annual precipitation is more than 3000 mm.

2. Savannah climate
The savannah climate comprises the largest part of Venezuela, with high temperatures, 24° to 28°C throughout the year and a dry season from November till March. About 90% of the precipitation falls in the rainy season from April till October.

3. Steppe climate
This type is encountered in the states Falcon and Lara as well as along the coast. In this climate evaporation exceeds precipitation, allowing little vegetation.

4. Desert climate
The dry trade winds cause extreme drought in the region north of Coro.

5. Mountain climate
In the mountain climate, prevailing in the Andes, three steps may be distinguished:
   a. High altitude climate up to 3000 m. The annual average temperature does not surpass 18°C.
   b. Tundra climate above 3000 m. Average temperatures are usually between 0° and 10°C.
   c. Polar climate. Only on the highest peaks of Venezuela, 4700 m and over there is perpetual snow and ice.
6. Monsoon climate
The Orinoco delta has a monsoon climate. This climate distinguishes itself from the tropical climate by a short dry period of three months.

Caracas is situated at 960 m altitude, has a moderate temperature and low humidity, whereas the lowly situated Maracaibo is hot and humid.
2.3 CITIES AND POPULATION

Venezuela has about 14.7 million inhabitants (1984). One effect of the discovery and exploration of oil was that agriculture was neglected causing a migration of the rural population to towns. More than 75% of the population lives in cities of more than 1000 (degree of urbanization), this is the highest rate in South-America. About 70% of the population lives in the agricultural and industrial mountain range from San Cristóbal to Caracas. The capital Caracas of which the centre has been rebuilt completely, has an estimated population of four million. [see figures 2.4 and 2.5]

Other important cities are Valencia (500000), Maracay (320000), San Cristóbal (170000), Barquisimeto (460000). To the East important cities are Ciudad Bolivar (115000) and Ciudad Guayana on the Orinoco river and on the coast Cumaná (135000), the first Hispanic city in South-America.

Maracaibo is Venezuela’s second city, its population reaches almost one million. [see figure 2.6]

Venezuela has a mixed population. The original pure indians (Motilones in the western part near the Colombian border and the Goajira indians on the Río Limón, north of Maracaibo and many still living in the jungle of Guayana) only make up 2% of the present population. Negroes form 10% and the 'criollos' (creoles), descendants of the Spanish colonizers form 30% of the population. The rest consists of mestizoes (35%) and Spanish, Portuguese and Italian immigrants (20%) that came after the second world war. Venezuela has about one million immigrants from Colombia, many of them illegal. [see figure 2.7]

Catholicism is the dominating religion (96%).
Venezuela has a petroleum based economy: petroleum and its derivates make up more than 95% of the national income from export. In 1925 Venezuela produced 54,000 barrels/day which increased rapidly till 1.5 million b/d in 1950, 2.85 million b/d in 1960 and a maximum of 3.8 million b/d in 1970. The oil crisis of 1973 caused a petroleum boom of unprecedented magnitude. A contraction in oil production took place since then, but the extreme increase in price made the value of sales rise from 13.5 billion bolivars in 1973 to 39.8 billion in 1977 (Bs. 4.30 = US $1.00) The oil is mainly (50%) won in the oil fields in and around Lake Maracaibo. The Orinoco Heavy Oil Belt (tar sands) contain enormous oil reserves; estimates range from 700 to 3000 billion barrels.

Venezuela has the highest per capita income and one of the lowest inflation rates of South-America. However, 1983 brought a long expected cash crunch, with a negative economic growth of -5.6%. Petroven (Petroleos de Venezuela) curbed its operations with cutbacks in exploration outlays, in development of heavy oil and in refinery upgrading projects. The bolivar dropped to 13.00 in one US dollar, while oil production was reduced from 2.2 million barrels per day in 1977 to 1.8 million in 1983 and 1.5 million in 1984, which equals the production of 1950. February 1986 brought a drop in oil prices that forced the oil producing countries to reduce production even further: Venezuela to a minimum of 800,000 barrels per day. In few years the foreign debts rose to 35.5 billion US dollars, while economic growth came to a halt. The 1984 inflation rate was 18.3%.

Iron ore is the second product of Venezuela, mined south of Ciudad Guayana; estimated reserves: 2 billion tons. In the agricultural sector coffee, cocoa and bananas are important products. Agriculture has been neglected when the country concentrated on oil. With respect to food supply Venezuela depends for 60% on imports. The migration of farmers to towns is a major obstacle for further development.

The United States, the EEC and Japan are important trading partners. The American continent counts for 75% of the exports,
mainly the United States and the Netherlands Antilles, 20% goes to the EEC (1983). Imports come from the United States for almost 50%, followed by the EEC and Japan. [see tables 2.1 and 2.2]
3. PROJECT REVIEW.

3.1 INTRODUCTION.

This chapter is written as a preliminary investigation to the raising of the Bolivar Coast Dikes. The first five sections place the coastal defence system into its context. The actual dike review is given in the sixth section. First the geometry is given. The various ways of failure impose the gathering of data that are necessary for the calculations. These ways of failure can't determine the structure of this chapter, the primary ways of failure, overtopping, seepage and piping can't be found directly in the composition. Basic data for the strength of the dike are the conditions of soil and groundwater. The hydraulic boundary conditions are discussed in a separate section. The geological situation with the subsidence and earthquake problems that affect the coastal defence so intensively is given in the last section. Some background information is dealt with in the first six appendices.
3.2 HISTORY.

3.2.1 Oil discovery and its consequences.
Originally the project area was a dry tropical region sloping down towards the lake. Near the coast, where are now the present polders Lagunillas and Bachaquero, there were swamps separated from the lake water by a slightly elevated strip of land. Rises of the water level caused inundations of these low lands.

Three oil fields were discovered in this area stretching out under the lake: Lagunillas (1926), Tia Juana (1928), Bachaquero (1930). Oil companies took measures to ensure their operations. The lowly situated areas were to be drained and protected against floods. For this second purpose a small earthfill dike was built on the narrow strip of land in Lagunillas. During a storm in 1929 this dike failed and the lake water flooded farther inland than ever before, which gave rise to the idea that oil extraction caused land subsidence. Measurements confirmed this idea, the dike was strengthened, but continuing subsidence made clear that more drastic measures were unavoidable.

3.2.2 Coordination in coastal protection.
There was a great variety in the ways new dike sections were constructed and strengthened. In 1937/38 this led to an agreement between the three oil companies involved.

The design for the central part of the Lagunillas oil field became a clayfill dike with a revetment of reinforced concrete, the outer toe fixed by a concrete sheet piling. For Tia Juana and Bachaquero similar designs were made later on.

The oil companies agreed to share the expenses for drainage and coastal protection. [see table 3.1] These were reconsidered in 1983. The percentages are based on direct benefit of the measures taken and on production quantities. Compañía Shell de Venezuela (CSV) would be the executive company for the whole dike and drainage system.
3.2.3 Expertise in dike construction.

In 1956/57 dike stability problems arose and CSV deemed it necessary to request advise of specialists i.e. Delft Soil Mechanics Laboratory (DSML).

DSML recommended the application of rip-rap to protect the outer slope and advised about measures to control slope erosion and piping.

Then new views on the magnitude of the land subsidence and the risk of earthquakes induced CSV to consult Nedeco in 1966. Nedeco reported in 1968. In the seventies however it became clear that subsidence figures were no longer fully valid, that local liquefaction of shallow sand layers couldn’t be excluded and that ground deformation could cause surface cracks. Woodward & Clyde carried out a seismic study, Intevep had to look into the surface cracking problems.

Nedeco was assigned a masterplan for the dike and drainage system, based on their 1968 report, but taking into account the latest subsidence forecasts, surface cracking, earthquakes and the possibility of earthquake induced liquefaction.
3.3 IMPORTANCE OF COASTAL PROTECTION.

Apart from the large area of waste grounds, the land to be protected has mainly three purposes: industrial, urban and agricultural.

3.3.1 Industry.
Petroleum industry dominates the area. About 55% of the land in the project area (680 km²) could be defined as zone of petroleum interest. The value of the installation fluctuates between 40 and 55 billion Bs (1983). The 1983 oil production of these three oil fields rates 934000 barrels/day, which is 50% of the total production of Venezuela. [see table 2.3] Oil counts for over 90% of the country’s income from exports.

3.3.2 Population.
Some 200,000 people live in this area, mostly in the cities and petroleum camps:

<table>
<thead>
<tr>
<th>City</th>
<th>Population</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tia Juana</td>
<td>11000</td>
</tr>
<tr>
<td>Ciudad Ojeda</td>
<td>63000</td>
</tr>
<tr>
<td>Lagunillas</td>
<td>30000</td>
</tr>
<tr>
<td>Bachaquero</td>
<td>20000</td>
</tr>
</tbody>
</table>

(figures 1981)

A new city, El Menito, is planned east of Lagunillas, Ciudad Ojeda must grow considerably.

3.3.3 Agriculture.
Farming in this area is mostly mixed, farms are usually small. About half the area offers a reasonable potential for cattle breeding, mainly extensive and south of Lagunillas.
3.4 CLIMATE.

3.4.1 Temperatures.
The project area shows little variation in temperatures. The yearly average amounts 28°C, where the average maximum is 30°C throughout the year and the average minimum is 20°C in December, January and from 22°C in the north till 24°C in the south in April.

3.4.2 Precipitation.
The dry season is from November till April, the rainy season with two peaks (May, October) from April till November. [see figure 3.3]
The distribution of the precipitation varies seasonally. In February N.E.-trade winds have a dominating influence, then rainfall depths increase in southward direction. In the middle of the rainy season a typical coast mountain pattern is seen: isohyets parallel to the coastline.[see figures 3.4a and 3.4b] Average rainfall ranges from 800 mm in the most northern part till 950 mm in the most southern part of the project area. Monthly precipitation records date back to 1956, daily and hourly rainfall depths have been evaluated since 1973. Evaporation data are not important for the coastal dikes. Daily averages range from 4 to 6 mm.

3.4.3 Wind data.
A typical day shows between midnight and 8.00 h a.m. an eastern wind that shifts clockwise. Round noon the direction is from the south. During the afternoon it blows usually from the southwest and starts shifting quickly through northwest to east after sunset.
Wind data are fragmentary. CSV supplied records for the 1968 report of Nedeco: wind speed 8.2 m/s at a duration of 6.6 hours. [see table 3.2]
An American company, Aware, executed measurements from December 1981 till February 1982 for an investigation into air pollution. This is however a calm weather period, high wind speeds did not occur then.
Records from May 1983 till February 1985 show high wind speeds of 35 miles/hour, 18 m/s with maximum duration of one hour from SW and W. This speed has also been recorded in September 1980 when also a three hour storm occurred with wind speeds of 10 m/s. There seems to be a relationship between wind duration and wind speed. [see table 3.3a and b]
3.5 POLDER SYSTEM.

3.5.1 Introduction.
Four polders are situated in the project area. Three of them coincide with the three large oil fields: Tia Juana (22 km$^2$), Lagunillas (93 km$^2$), Bachaquero (63 km$^2$). The fourth polder is only small, Pueblo Viejo (9.2 km$^2$). Originally these areas hardly emerged from the lake and the dikes were built to prevent inundations (section 3.2). The oil fields stretch out under the lake and land subsidence due to oil extraction made dikes indispensable. Figures 3.6 and 3.7 show a general plan and cross section of the polders. The oil field gets the shape of an inclining bowl that is getting deeper because of the subsidence. The land forms about half the bowl, hence major subsidence occurs near the central section of the coastal dike. The dike and drainage systems are discussed shortly in the following sections.

3.5.2 Dike system.
The land submitted to subsidence is surrounded by dikes, coastal dikes on the lake side, and the so called diversion dikes inland. In the Lagunillas polder old diversion dikes form partition dikes now.

coastal dikes
The coastal dikes form a direct protection against the threatening lake water. The retaining height is biggest in the central sections, about 3.5 m in Lagunillas. [see figure 3.8] Closer to the borders of the polder the retaining height becomes zero.[see figures 3.7 and 3.9] The total length of these dikes including Pueblo Viejo amounts 46 km. Between the oil fields, where the coast lands are higher, coastal protection is not necessary.

diversion dikes
The interior or diversion dikes divert rainfall runoff from higher areas, thus isolating the lowly situated polder where the water levels are to be controlled. These dikes don't have the
vital importance of the coastal protection, which can be seen from their condition. Many sections are grown over with trees and bushes, making them difficult of access for physical inspection. The total length is over 80 km. [see figure 3.10]
For the general design see figure 3.11b.

**partition dikes**
Partition dikes are only found in Lagunillas. The southern partition dike encloses the deepest part of the polder, (apart from the old lagoon) comprising company houses and tank farms behind the sections 5, 5A, 6 and 7. The crest of this dike is already below lake level.
The second partition dike encloses housing, industry and tank farms around the Lagoven harbour in section 4. The crest of this dike is also low, both partition dikes can only function to delay flooding in case of a dike breach. [see figures 3.12 and 3.13]

**bypass dikes**
Where harbours form great gaps in the coastal defence system, bypass dikes are constructed round these harbours.[see figure 3.47 and section 3.6.7]
The general design of the bypass dikes is given in figure 3.11b. The bypass dikes round the Lagoven launch harbour in Tia Juana, section 7 is 760 m long and is connected to the coast dikes at progressivas 37+60 and 44+50.
In the Lagunillas polder are two launch harbours: in section 4 the Lagoven harbour with an approximately 300 m long bypass dike between progressivas 45+40 and 47+50 and in section 5 the Maraven harbour with a 440 m long bypass dike between progressivas 62+60 and 66+60.

**drainage system**
The whole project area used to drain directly to the lake. Human intervention however cut of the natural drainage system and subsidence changed ground levels. The main streams that were interrupted are Queda la Plata, Q. las Morochas, Q. el Danto, Q. las Platas, Q. San Andrés and Caño Amarillo. Their flows were diverted along the polder to the lake, diversion canals were dug for this purpose. [see figures 3.10 and 3.14]
The rivers Tamare and Pueblo Viejo were no longer able to handle
their discharge, by-pass canals were necessary (Canal Sibaragua, 1956 and Canal Tamare, 1985). [see figures 3.16 and 3.17]

A canal network discharges the water load inside the polder. Smaller secondary canals discharge into principal canals that carry the water straight to the coast where the drainage canal of the coastal dike serves as a conduit to the pumping stations. There is no tertiary system.[see figure 3.15]

There are 27 pumping stations distributed as follows:

<table>
<thead>
<tr>
<th>polder</th>
<th>stations</th>
<th>pumps</th>
<th>installed capacity m³/h</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tia Juana</td>
<td>5</td>
<td>15</td>
<td>76200</td>
</tr>
<tr>
<td>Lagunillas</td>
<td>16</td>
<td>35</td>
<td>76400</td>
</tr>
<tr>
<td>Bachaquero</td>
<td>4</td>
<td>9</td>
<td>37000</td>
</tr>
<tr>
<td>Pueblo Viejo</td>
<td>2</td>
<td>4</td>
<td>7700</td>
</tr>
</tbody>
</table>

Rainfall with longer return periods of 10 and 25 years cause major problems near more than 20 of these stations. Measurements confirmed that a number of stations did not perform well due to insufficient positioning and water approach.
3.6 DIKE REVIEW.

3.6.1 Introduction.
The coastal dikes are divided into sections. From each section a representative cross section is available. [see table 3.4] They are divided as follows:

<table>
<thead>
<tr>
<th>polder</th>
<th>sections</th>
<th>total length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tia Juana</td>
<td>13</td>
<td>8502 m</td>
</tr>
<tr>
<td>Lagunillas</td>
<td>14</td>
<td>19165 m</td>
</tr>
<tr>
<td>Bachaquero</td>
<td>9</td>
<td>14047 m</td>
</tr>
</tbody>
</table>

The dikes have a hectometer division ("progressiva 10+50" i.e. 1050 m from the reference point), with help of benchmarks the subsoil subsidence can be measured. [see figures 3.9 and 3.18]. Levels refer to MLLL which is the low lake water level in Lagunillas of August 1926.

Generally the present dikes (1984) have crest levels of at least MLLL +1.00 m where there is a foreshore at lake level. Where the lake bottom is deeper, in front of the central dike sections crest levels range from MLLL +2.00 m up to 3.50 m.

The outer toe was locked by a concrete sheet piling whereas the revetment of the outer slope consisted of reinforced concrete slabs, which can still be seen in section 3A of the Bachaquero dike. [see figure 3.19] Nowadays the outer slope is covered by a 1.25 m thick layer of rip-rap (1:2.5 or 1:3). At former crest levels there is often a berm of at least 3 m wide. Above the berm the outer slope usually has an impervious layer of asphalt mastic.

The crest inclines slightly towards the lake (1:20 or 1:50), is 3 to 4 m wide and is covered with bituminized sand serving as an inspection and maintenance road.

The inner slope should have a gradient of 1:3 and a corocillo grass cover. Many sections however are still too steep while the corocillo is in bad condition, causing slope erosion. [see figure 3.20]
Where reconstructed completely, there is a toe blanket between dike and drainage ditch of at least 6 m wide. Often this ditch is not (yet) at the required distance from the dike. [see figure 3.21]

A general description of the dikes could be given as follows: lake dikes in a tropical, subsiding, seismic and industrial area. Chapter 4 describes the different ways that the dikes may go through to ultimate failure. These ways of failure impose the aspects that should be looked in to when raising the dikes. These aspects determine which data must be gathered.

Subsidence is going on and the dikes are continuously under reconstruction. [see figure 3.22]

The dikes are situated in an industrial area and therefore affected by all kinds of industrial constructions like pipe crossings, oil wells, launch harbours, pumping stations and a power station in Bachaquero.

3.6.2 Longitudinal sections.
Longitudinal sections of the dike are given in figure 3.23. These figures indicate crest levels as well as top levels of the riprap, outer berm, and lake bottom level. A rough indication of the expected subsidence in 2030 is given. Levels are based on data of 1984.

3.6.3 Cross sections.
The introduction 3.6.1 gave a general description of the cross section of the coastal dikes. Every dike section however, can be represented by one typical cross section, the present condition is described below. Drawings of the cross sections are given in figures 3.24 to 3.26. These drawings show levels, measures and soil types used in the different stages of dike raising.

Cross sections of the Tia Juana dikes. [see figure 3.24a]

Tia Juana 1 (N 14+00-0+00).
This section has a foreshore slightly above lake level, extending
a few hundred meters, hence there is no wave attack. No rip-rap is applied, the crest is low (1.19 m). This relatively new section doesn’t have old concrete slabs in its dike body.
[see figure 3.24b]

**Tia Juana 2A (0+00-6+00).**
Real coastal protection is necessary in this section since the foreshore of section 1 disappears after 200 m. The old outer slope with concrete slabs is covered with rip-rap till the old crest level (+1.60 m, 1965) that forms a berm now. In 1982 the dike was raised till +2.45 m.
[see figure 3.24b]

**Tia Juana 2B (6+00-10+45).**
The lake bottom is getting deeper here (-2.40 m), the land is subjected to bigger subsidence, hence there is a stronger wave attack and a bigger retaining height. A firm dike body is already necessary, with rip-rap applied up to crest level (+3.26 m). An inner berm (1984, level at +1.40 m, width 12.00 m) sustains the dike. [see figure 3.24c]

**Tia Juana 2 (10+45-19+00).**
This section is similar to section 2B.
[see figure 3.24c]

**Tia Juana 3, 4, 5, 6, 7 (19+00- 22+65- 24+10- 25+10- 31+15- 43+54).**
These sections are very much the same. Their lengths are respectively 365, 145, 100, 605 and 1239 m. The lake bottom is rising from -4.00 m in section 3 till -3.00 in 7. Rip-rap is applied to crest level (+3.00 m). The drainage ditch in 6 and 7 is very close to the inner toe of the dike.
[see figure 3.22 and 3.24d]

**Tia Juana 8A (43+54-53+57).**
Sections 2A and 8A are similar. Section 8A was raised in 1983 till +2.92 m. There is an outer berm at +2.17 m, the lake is 1.73 m deep here.
[see figure 3.24d]
Tia Juana 8 (53+57-63+30).
The lake is only a few decimeters deep here. Rip-rap is applied
till the outer berm at +0.81 m. The crest was recently raised
till +2.41 m (1983). A foreshore protects the end of this
section against wave attack.
[see figure 3.24e]

Tia Juana 9, 9A (63+30-67+30-71+02).
These two sections are protected by a foreshore. Rip-rap is not
necessary, low crest levels (+1.16 m and +0.90 m) are sufficient.
[see figure 3.24e]

Cross sections of the Lagunillas dikes. [see figure 3.25a]

Lagunillas 1 (N 10+00-8+00).
This section has a wide foreshore, but the dike is already rather
high, with a rip-rap cover up to the berm at +1.23 m, the crest
being at +2.49 m. Beyond progressiva 8+00 rip-rap is applied up
to the slightly higher crest at +2.70 m; the lake bottom is deeper
here, -0.96 m, there is no foreshore.
[see figure 3.25b]

Lagunillas 2 (8+00-25+00).
Section 2 looks like section 1. In 1980 a toe blanket was
constructed at the inner berm (level +0.94).
[see figure 3.25c]

Lagunillas 3 (25+00-40+50).
The lake bottom is deeper here, -2.81 m, rip-rap reaches also to
the crest (+2.41), the toe blanket is at 1.36 m.
[see figure 3.25c]

Lagunillas 4 (40+50-53+58).
This section was raised till +2.80 in 1980. A layer of rip-rap
was applied on the old concrete slabs till the old crest that
forms a berm now at +2.00 m. [see figure 3.25d and 3.27]

Lagunillas 5 (53+58-70+37).
Like sections 2B till 7 in Tia Juana, the sections 5 till 7 in
Lagunillas have considerable dimensions though the crests in Lagunillas are lower. In front of the dike the lake is 4 to 4.5 m deep. The dike was raised in 1984 from +2.16 m (berm now) till +2.97 m. [see figure 3.25d and 3.28]

The sections 5, 5A and 6 protect the deepest part of the polder. [see figure 3.7]

Lagunillas 5A (70+37-73+95).
This section is lower, crest at +2.57 m, berm at +1.58 m. A toe blanket has been constructed at +0.67 m in 1984. Behind the dike is little space, the drainage ditch is too close to the inner toe. [see figure 3.29]
A steel sheet piling instead of a concrete one locks the outer toe. [see figure 3.25e]

Lagunillas 6 (73+95-86+75).
Presently there is a huge dike with too big dimensions. Subsidence will be considerable here in the next decades. Rip-rap is applied to crest level (+2.34 m), a new toe blanket was made at 0.67 m in 1984. [see figure 3.8 and 3.25e]

Lagunillas 7 and 8A (86+75- 93+37- 98+68).
Sections 7 and 8A look like the preceding section 6, but the lake is less deep (-2.51 to -1.99 m), the top of the rip-rap and the crest are at +2.23 and +2.74 m. The new toe blanket of 1984 is at +0.70 m). [see figure 3.25f]

Lagunillas 8 (98+68-110+70).
This section doesn’t have a toe blanket, neither concrete slabs, but is apart from that quite similar to section 8A. [see figure 3.25g and 3.30]

Lagunillas 9 (110+70-124+00).
A small layer of rip-rap reaches from the lake bottom at -1.46 m till the berm at +1.70 m. The crest was recently raised till +2.47 m (1983). [see figure 3.25g]

Lagunillas 10A, 10 and 11 (124+00- 139+25- 159+25- 181+65).
The last sections are low, the lake bottom is more or less at
MLLL, there is no rip-rap, the crest level is at +1.00 m. [see figure 3.25g]

Cross sections of the Bachaquero dikes. [see figure 3.28a]

All sections still have a concrete sheet piling to lock the outer toe.

Bachaquero 1 (N65+13 - N60+00)
This stretch (513 m) leads from the cooling water outlet northwards to the river Publo Viejo. [see figure 3.50]. There is a wide foreland and the crest height was brought at 1.98 m in 1982. The revetment on the outer slope consists of asphalt mastic and rip-rap. Closely behind the dike is a small compound of houses. [see figure 3.26b]

Bachaquero 1A (cooling water outlet N60+00 - N 42+60)
The lake water is shallow here, to 60 cm at 50 m distance. The crest height was raised to 2.50 m in 1983. Rip-rap has been applied up to a low berm halfway the outer slope. Crest and slope are further protected with asphaltic crude. The water inlet structure of the power station has been incorporated in this section. [see figure 3.26b]

Bachaquero 1B (N42+60 - N36+20)
This section has its crest level at 2.62 m, the rip-rap has been applied up to the crest and a toe blanket stretches out to a drainage ditch that lies closely behind the dike (10 to 15 m), but farther inland (35 m) south of progressiva N40+50. [see figure 3.26b]

Bachaquero 2 (N36+20 - 0+00)
This section protects the deepest part of the polder. There is no foreshore, but the outer slope has a relatively wide berm. In 1982 the dike was raised till 2.32 m. There is no toe blanket, but a catchment canal lies at some distance behind the dike. [see figures 3.26c and 3.31]
Several specific problems become apparent in this section:
- seepage near N3+00 [see figure 3.32]
- inner slope erosion near N3+00 [see figure 3.33]
- abandoned oil well near N6+00 [see figure 3.46]
- old pipe in dike body near N8+00 [see figure 3.39]
The last part just north of the harbour has foreland, but the dike itself is in deplorable state [see figure 3.49]

**Bachaquero 3 (S0+00 - S35+00)**
This section incorporates the harbour that forms in fact a gap in the coastal defence system. The industrial area behind is however relatively highly situated. The old harbour is closed by a dam that is part of the coastal dike now. [see figure 3.47]
From progressiva S10+50 on there is a foreshore [see figure 3.34 and 3.35]
The dike is low, 1.70 m, but has a berm up to which rip-rap is applied. Above that berm the revetment consists of asphalt mastic. The inner slope is not well covered, overtopping will cause erosion. [see figure 3.26d]

**Bachaquero 3A (S35+00 - S47+50)**
The outer slope revetment consists of concrete slabs [see figure 3.19]. The dike is low, 1.65 m, but there is a foreshore. There is no drainage ditch for the dike, but at further distance (some 100 m) is a polder drainage canal.
[see figure 3.26e]

**Bachaquero 4, 4A, 4B (S47+50 - S57+50 - S67+50 - S75+00)**
These sections are really low, 1.44 m, there is a foreshore. The last reconstruction was in 1983.
[see figures 3.26e and 3.36]

**Cross sections of the Pueblo Viejo dikes. [see figure 3.26a]**

**Pueblo Viejo 1**
This section lies along the north banks of the river Pueblo Viejo. It is a low dike, 1.46 m, 825 m long. It still has a concrete sheet pile construction to lock the outer toe. The outer slope of the last 250 m has an asphalt mastic revetment.
[see figure 3.26f and g]
Also this section doesn’t face the lake, but is already along the mouth of the river. It is 244 m long; it is very low, 0.92 m. Its toe is locked by a concrete sheet piling. [see figure 3.26f]

This 500 m section is firmer and higher (1.62 m). Rip-rap is applied to the outer slope. The inner slope is impregnated with a sort of asphalt, RC2. [see figure 3.26f]

These two sections have lengths of 2155 and 1495 m respectively. They are newly constructed behind the low old dike. Their crest levels are at 1.50 m. The outer slope has an asphalt mastic revetment. There is no sheet pile construction in the toe. Behind the dike is a wide drainage canal. Inner slope erosion seems to be a problem. [see figures 3.20 and 3.26h]

3.6.4 Soil conditions.

3.6.4.1 Introduction
The most important ways of failure with respect to soil conditions have an orientation perpendicular to the axis of the dike. Slope failure along deep and shallow sliding planes and excess seepage do not occur in one specific cross section but always over a certain stretch. Hence representative characteristics of that stretch will determine whether failure occurs or not. The length of such a stretch will be at least a few tens of meters.

Results of CPT’s, borings and piezometer measurements are assumed to be representative for such stretches. DSML in 1957 and LICCA, Maracaibo in 1967 carried out soil investigations. [lit.15] Results of calculations with these data have been laid down in the Nedeco report of 1968. [lit.14] Further investigations were made into the liquefaction problem in 1976 and from November 1981 till March 1982 Maraven conducted 67 CPT’s [see table 3.22], while Laboratorios y Diseños de Ingenieria S.A. carried out 16 borings and installed numerous piezometers.
Later on in the first months of 1985, DSML did additional CPT's and borings. The CPT's give values for cone resistances, water pressures (piezocone) and friction factors, from which a rough indication of the soil types and $C$-values can be obtained. Laboratory tests include a) granulometric analysis, b) determination of limits of consistency, c) specific gravity, unit weight, moisture content, d) Proctor compaction test, e) determination of shearing resistance by quick cell tests [see appendix 5], f) measurement of capillary action, g) determination of permeability and h) consolidation test.

Figure 3.132 shows a triangle to denote the classification of soils. Silt sand and gravel are defined as shown in figure 3.133, clay has particles smaller than 0.002 mm.

A general soil description is given in table 3.5.

## 3.6.4.2 Subsoil and dike material

### Importance of the DSML soil investigation in 1957

The 1957 DSML reports [lit. 15] give in fact the most basic and detailed information about the old dike material and the ground on which the dikes are founded. Later investigations were merely to densify the net of data. Meanwhile subsidence and raising of the dike have continued, care must be taken when looking at levels and depths. In this section data will be given as stated in the reports of different years without adaptation to the present situation. Subsidence data are given in tables 3.11, 3.12 and in figures 3.65 - 3.70.

### Continuing subsidence and dike raising

Considering a subsidence of around 2.00 m between 1957 and 1984 the numbered layers of the 1957 investigation can be found on the 1984 cross section drawings [see figure 3.24 and 3.25]. The denotation of the soil types as shown in these figures is given below:
<table>
<thead>
<tr>
<th>type</th>
<th>% clay</th>
<th>% sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>40-55</td>
<td>5-30</td>
</tr>
<tr>
<td>B</td>
<td>50-35</td>
<td>10-25</td>
</tr>
<tr>
<td>C</td>
<td>45-25</td>
<td>15-35</td>
</tr>
<tr>
<td>D</td>
<td>30-20</td>
<td>30-45</td>
</tr>
<tr>
<td>permeable</td>
<td>25-15</td>
<td>&gt; 50</td>
</tr>
</tbody>
</table>

After the year 1968 dikes have been raised according to Nedeco recommendations with three types of soil:

<table>
<thead>
<tr>
<th>type</th>
<th>% clay</th>
<th>% sand</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&lt; 5μ</td>
<td>&gt; 60μ</td>
</tr>
<tr>
<td>I</td>
<td>40-60</td>
<td>0-25</td>
</tr>
<tr>
<td>II</td>
<td>15-30</td>
<td>30-60</td>
</tr>
<tr>
<td>IIA</td>
<td>15-23</td>
<td>40-60</td>
</tr>
</tbody>
</table>

In these denotations soil particles smaller than 0.005 mm are considered to be clay (the more common value 0.002 mm was used in 1957 and will also be used further on). Laboratory test data of these newly applied layers are not available, but figures 3.133 and 3.134 show the required grain size distribution for soil types I, II and IIA that are used to raise the dikes since 1968. [see also figures 3.24, 3.25 and 3.26]

general appearance of the dikes and subsoil

Generally the dikes in the three distincted polders show different features. In Tia Juana the dikes have a base of the permeable soil, on which the oldest clayfill (type B) were constructed. Raisings in the second half of the fourties and the first half of the fifties were executed in clay, types B and C, sustained on the inner side by clay of type A. Toe blankets of type C were applied later on. After 1968 dike raisings were made of the impervious soil type I, whereas the sustension on the inner including the toe blanket was made of the more permeable
type II soil, using type IIA in the lower layers. Also the base of the Lagunillas dikes consists usually of the permeable soil. The dike bodies constructed in the end of the thirties are formed of clays of types B and C. In the sixties a few sections got toe blankets of type C soil. Only in the seventies the dikes were raised further with type I for the crests and type II and IIA for the sustension at the inner side like in Tia Juana.

The most important dikes of Bachaquero consist of type B soil made in the early fifties. Crests were raised ten years later with type A soil and sustained by type C. After 1968 the dikes were raised further according to Nedeco recommendations like in Tia Juana and Lagunillas.

results of the 1957 investigation
The 1957 investigation distinguishes 5 or 6 layers in and directly under the dike body. Numerous samples were taken from these layers, laboratory tests were carried out [see table 3.20] and the average data for Tia Juana and Lagunillas are given below (between brackets is indicated to which layers the soil types correspond on later drawings, figures 3.24-3.26):

-Tia Juana: soil layers in the dike and average data (1957).

<table>
<thead>
<tr>
<th>LAYER 1 soil type</th>
<th>reddish brown clay, top 10 cms sprayed with crude oil (arcilla 1955 + tipo A 1955, section 2-8)</th>
</tr>
</thead>
<tbody>
<tr>
<td>thickness</td>
<td>1.00 m at crest and 1.50 m at inner slope</td>
</tr>
<tr>
<td>averaged data:</td>
<td>a) 10-20% sand, 50-60% clay</td>
</tr>
<tr>
<td></td>
<td>( \gamma_s = 26.5 \text{ kN/m}^3, \gamma_d = 14.6 \text{ kN/m}^3 )</td>
</tr>
<tr>
<td></td>
<td>( \gamma_a = 19.1 \text{ kN/m}^3, n = 41% )</td>
</tr>
<tr>
<td></td>
<td>e) ( \phi' = 7.5^\circ ), saturated ( c' = 7.5 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td></td>
<td>unsaturated ( c' )-values up to 35 kN/m²</td>
</tr>
<tr>
<td></td>
<td>consolidated true ( \phi = 20^\circ ), ( c = 5 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td></td>
<td>g) ( k_v = 2-5*10^{-7} \text{ m/s} )</td>
</tr>
<tr>
<td></td>
<td>(at 16+80: 2-4*10^{-6} m/s)</td>
</tr>
<tr>
<td></td>
<td>( k_h = 10^{-6} \text{ m/s (progr. 25+40)} )</td>
</tr>
<tr>
<td></td>
<td>h) ( C = 40, \ C' = 20, p_e = 40 \text{ kN/m}^2 )</td>
</tr>
</tbody>
</table>
LAYER 2
doil type: yellowish brown loam, rather equal parts sand, silt and clay
(aricilla 1947 + tipo A 1947, section 2-8)
thickness: 3.25 m at the crown
averaged data: a) 40-50% sand, 25-35% clay
c) $\gamma_s = 25.5$ kN/m$^3$, $\gamma_d = 17.5$ kN/m$^3$
   $\gamma_n = 20.0$ kN/m$^3$, $n = \%$
e) $\phi' = 10-15^\circ$, saturated $c' = 10-15$ kN/m$^2$
   unsaturated $c'$-values up to 45 kN/m$^2$
   consolidated true $\phi = 25^\circ$, $c = 5$ kN/m$^2$
g) $k_v = 1.5*10^{-6}$ m/s
   $k_h = 2.10*10^{-6}$ m/s
h) $C = 65$, $C' = 30$, $p_c = 80$ kN/m$^2$

LAYER 3
soil type: brown, silty and sandy clay
(tipo B 1943/44, section 2-8)
thickness: 1.25 m, first coastal defence
averaged data: a) 15-20% sand, 35% clay
c) $\gamma_s = 25.5$ kN/m$^3$, $\gamma_d = 13.8-15.8$ kN/m$^3$
   $\gamma_n = 17.7-19.4$ kN/m$^3$, $n = 36-47%$
e) $\phi' = 10^\circ$, saturated $c' = 6$ kN/m$^2$
   unsaturated $c'$-values up to 30 kN/m$^2$
   consolidated true $\phi = 23^\circ$, $c = 5$ kN/m$^2$

LAYER 4s
soil type: sand
(tipo permeable 1942 + tipo D 1947, section 2-8, but at progr. 16+80 this layer isn’t present on top of the original terrain)
thickness: 1.50 m
averaged data: a) 60% sand, no clay
c) $\gamma_s = 26.0$ kN/m$^3$, $\gamma_d = 15.3$ kN/m$^3$
   $\gamma_n = 18.8$ kN/m$^3$, $n = \%$
e) $\phi = 30^\circ$, $c = 0$
g) measured $k_v = 3.5*10^{-6}$ m/s, probably greater because of probable compaction of the samples
   $k_h = 3.5*10^{-6}$ m/s
h) $C = 150$, $C' = 65$, $p_c = 75$ kN/m$^2$

LAYER 4c
soil type: grey, silty clay
thickness: 1.00 m, original terrain
averaged data: a) < 10% sand, 60-70% clay
c) $\gamma_s = 25.5 \text{ kN/m}^3$, $\gamma_d = 11.8 \text{ kN/m}^3$
$\gamma_n = 17.0 \text{ kN/m}^3$, $n = 55\%$
e) $\phi' = 5^\circ$, saturated $c' = 5 \text{ kN/m}^2$
unsaturated $c'$-values up to $30 \text{ kN/m}^2$
consolidated true $\phi = 25^\circ$, $c = 0$
g) $k_v = k_h = 3-8\times10^{-7} \text{ m/s}$
h) $C = 50$, $C' = 20$, $p_c = 85 \text{ kN/m}^2$

LAYER 4s as above

LAYER 5 soil type : silty formation, mixed with clay
thickness : deep under layer 4
averaged data: a) greatly varying, generally less sand than clay, mostly silt
c) $\gamma_s = 26.0 \text{ kN/m}^3$,
$\gamma_n = 16.7-17.7 \text{ kN/m}^3$
e) $\phi' = 12-28^\circ$, $c' = 15-20 \text{ kN/m}^2$
consolidated true $\phi = 25^\circ$, $c = 10 \text{ kN/m}^2$
h) greatly varying

-Lagunillas: soil layers in the dike and average data.

LAYER 1 soil type : reddish brown clay
(arcilla B/C 1939/47, section 3-8A)
thickness : 3.80 m
averaged data: a) 10-20% sand, 40-50% clay
c) $\gamma_s = 26.0 \text{ kN/m}^3$, $\gamma_d = 16.3 \text{ kN/m}^3$
$\gamma_n = 20.1 \text{ kN/m}^3$, $n = 37\%$
e) $\phi' = 12.5^\circ$, saturated $c' = 5 \text{ kN/m}^2$
unsaturated $c'$-values up to 45 $\text{kN/m}^2$
consolidated true $\phi = 25^\circ$, $c = 0$
g) $k_v = 5-10\times10^{-7} \text{ m/s}$
(at 29+50: 2-5$\times10^{-6} \text{ m/s}$)

LAYER 2 soil type : greenish gray and yellow, fairly silty sand
(tipo permeable 1932/34, section 3-8A)
thickness : 1.35 m
averaged data: a) varying, sand with considerable amount of silt
c) $\gamma_s = 26.0 \text{ kN/m}^3$
$\gamma_n = 19.1 \text{ kN/m}^3$, $n = \%$
e) $\phi = 30^\circ$, $c = 0$

\[ g) k_v = 1-5 \times 10^{-1} \text{ m/s} \]

**LAYER 3 soil type**: bluish grey, sandy and argillaceous silt
(top dms are fill, underneath: original terrain)

**thickness**: 0.70 m

averaged data:

a) 10-20% sand, 75% silt, 10-20% clay

c) $Y_s = 26.0 \text{ kN/m}^3$

\[ Y_r = 13.7-19.6 \text{ kN/m}^3, n = \% \]

e) $\phi' = 7.5^\circ$, saturated $c' = 6.5 \text{ kN/m}^2$

\[ g) k_v = 6-8 \times 10^{-5} \text{ m/s (progr. 70+40)} \]

h) varying

**LAYER 4s soil type**: silty sand

**thickness**: 2.50 to 3.50 m below original terrain

averaged data:

a) 50-70% sand, 30% silt

c) $Y_s = 26.0 \text{ kN/m}^3$

\[ Y_r = 19.1 \text{ kN/m}^3, n = \% \]

e) $\phi = 40^\circ$, $c = 0$, samples were compacted

advisable to assume lower $\phi$, e.g. $30^\circ$

\[ g) k_v = 1-5 \times 10^{-1} \text{ m/s} \]

h) $C = 100\text{ }-\text{ }250$

**LAYER 4c soil type**: sandy and argillaceous silt

**thickness**: 1.50 to 2.50 m below original terrain

averaged data:

a) <20% sand, 20-60% clay

c) $Y_s = 26.0 \text{ kN/m}^3$

\[ Y_r = 16.7-18.6 \text{ kN/m}^3 \]

e) $\phi' = 7.5^\circ$ (sometimes up to $15^\circ$)

saturated $c' = 4-12 \text{ kN/m}^2$

consolidated true $\phi = 22.5^\circ$, $c = 0$

\[ g) k_v = 10^{-6} \text{ - } 10^{-4} \text{ m/s} \]

h) varying

These are overall average data. Data are given more specifically in table 3.21 and in figures 3.108 - 3.127.

**later investigations**

Later investigations provide a great number of CPT-diagrams of different sections [see figure 3.135 and table 3.23].

From these diagrams a rough value for $c$, $\phi$ and $C$ may be assessed and CPT-value and friction ratio are also an indication for the
liquefaction
Soil data directly aimed at the evaluation of the risk of liquefaction are discussed in section 3.8.3.5.

3.6.5 Groundwater conditions.

Introduction
Knowledge of the groundwater conditions in dike body and subsoil is important, first because excess seepage is a primary failure criterion, second because water pressures in the voids of the soil skeleton and subsoil may form major loads: seepage forces. The same company that did borings in 1981/82, Laboratórios y Diseños de Ingeniería, installed numerous piezometers to measure water heads in the subsoil.

groundwaterflow and seepage
The differential water head over the dike results into a sloping groundwater table in the dike body and in the subsoil. The hydraulic gradient maintains a groundwater flow through the dike body and its foundation. This flow may lead to the following drawbacks:

1) Excess seepage through the dike body
2) Excess seepage through the subsoil
3) Excess water pressures causing:
   a) failure of inner slope
   b) subsoil erosion, piping and heave

Piezometers indicate the water heads in the permeable soil layers. The location of the piezometers is shown in figure 3.37 and indicated in table 3.6, in which piezometer data and readings are listed.

A stationary situation must have developed in the dikes and subsoil, since it can readily be assumed that the lake level and polder level are more or less constant. The groundwater table does react on lake water level changes: during the rainy season (June-August) the lake water level and thus the groundwater level are notably higher than in the dry season (February-March). [see
The groundwater level in the dike bodies is forced down by the relatively little permeable soil layers of which the outer slopes of the dikes are built. Then the phreatic line slopes down remaining at least one meter below the ground level of the inner toe, when the inner slopes are not too steep.

1) Excess seepage through the dike body occurs usually in dikes that are founded on impermeable layers. [see figure 3.38a] This is not the case for the Bolivar Coast dikes. [see also 3a) of this section]

2) Excess seepage through the subsoil may occur under the Bolivar Coast dikes.

The amount of seepage is ruled by Darcy's Law for groundwaterflow:

$$q = -k \times \frac{dh}{ds}$$

in which

- \(q\) = specific discharge
- \(k\) = permeability of soil layers
- \(\frac{dh}{ds}\) = hydraulic gradient

3a) Water pressures in the inner side of the dike body may reduce effective stresses there to such an extent that the inner slope may fail. [see figure 3.38a]

3b) Seepage forces increase the submerged weight of the soil particles at the upstream side of the dike, but when the groundwater flow is in upward direction, at the downstream side, the apparent weight becomes smaller so that erosion may occur or the upward seepage force causes heave of an overlying impermeable layer. [see figure 3.38b and 3.92] Section 3.6.6 and appendix 6 deal with this problem.

Water pressures

Water pressures affect the condition of the dikes in many ways. A higher retained water head in the course of the years will be the main cause of higher water pressures than in the present situation. These higher water pressures have a negative effect on the dangers for piping (3.6.6), slope stability (3.6.4) and liquefaction (3.8.3.5).
The storage capacity of piezometer tubes affects the water condition in clays so much that no accurate data can thus be acquired. DSML executed cone penetration tests with piezocones to get water pressure data of the less permeable layers in 1985.

3.6.6 Piping.

Introduction
Concentration of groundwaterflow may induce the formation of cavities and channels in ground, thus lowering the resistance to this flow and concentrating it even further. Through backward erosion a dike may get undermined, resulting locally into excess seepage and sudden failure of the dike. [see figure 3.85] Where other types of dike failure usually need a certain 'failure length' [see 3.6.4.1] piping occurs at particular spots: weak spots in the ground near the inner toe of the dike or next to foreign elements in the dike body, like pipes or roots of (dead) trees and plants. [see figure 3.39][see section 3.6.7] This particularity makes it extremely difficult to develop design criteria, but some general design criteria concerning this type of subsurface erosion can be made. [see appendix 2]

Situation of the Bolivar Coast Dikes with respect to piping
The top soil layers (2 to 2.5 m) on which the dikes are founded consist in general of a mixture of fine sand, silt and clay: loam. Bligh, Griffith and Lane [see tables 3.17] don't give a safe seepage length-water head-ratios for loam explicitly. The effect of the clay content in this foundation layer is such that the danger for piping decreases in comparison to a layer with sand and silt only. Piezometer readings [table 3.6] show that the groundwater levels are below ground level everywhere (except at some places near the Lagoven harbour in Lagunillas, before the construction of the bypass dike). This shows that the hydraulic gradient fulfills the condition \( i < 0.5 \) everywhere. The head loss of the water, seeping through the subsoil, is such that no danger for piping is expected near the inner toe of the coastal dikes in the present situation. However, the continuing subsidence requires higher dikes, there will be a greater differential water head, \( H \), and
higher ground and groundwater pressures. Because of the required raising of the dikes, piping may become a problem that should be paid attention to.

3.6.7 Constructions in the dike.

Introduction
Various foreign elements affect the dike body. The area is highly industrialized and apart from the pumping stations with their discharge pipes, other pipelines, oil wells, launch harbours, and one power station have influence on the dike.

Pumping Stations
Pumping stations are situated behind the dike, always outside the dike profile. The discharge pipes are founded on a frame, thus leading over the dike. The discharging jets from those pipes should not erode the lake bottom, thus undermining the dike foundation. Leaking of the pipes may cause crest and slope erosion or excess infiltration. To prevent possible erosion a protective revetment is applied under the pipe crossings as well as a seabed protection near the outlets. [see figures 3.40 and 3.41]
There are no pumping stations that have outlet structures that lead through the dike body.

Pipe Crossings
Pipes that conduct water, oil, gas and steam cross the dikes at several places. Similar measures are or should be taken as for the discharge pipes of the pumping stations. Leaking fluids under pressure may induce high phreatic level in the dike body if not directly erosion.
In the dike body old unused pipes still occur, they may form a major danger for local failure.[see figures 3.39, 3.43 and 3.44]

Oil Wells
In total 180 oil wells are located in the dike profile of which 74 are still being operated. Dikes are locally wider there, thus
forming a sort of working platform around the well. [see figure 3.45]

It is important to trace all old and abandoned wells. Leaking pressurized gases may cause disastrous damage. [see figure 3.46]

**launch harbours**

Harbours form a gap of several hundred meters in the coastal defence system of the polders. Operations don't allow high quay-walls, reducing the retaining strength there to a minimum. Though the harbours are protected from wave action by breakwaters the quay-walls remain vulnerable in comparison with the dikes.

A harbour is built as a sheet pile quay-wall with operation platforms at ca. MLLL +1.00 m. [see figure 3.47] Harbour operations, such as ship movements near the quay-wall, probably causing erosion of the bottom there and placing of heavy loads on the platforms, count for extra dangers for the defence system. Anchored quay-walls constructions are also very sensible for earthquakes and subsidence.

Four harbours intersect the dike:

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Tia Juana</td>
<td>Lagoven</td>
<td>38+00-43+00 -1.00/-2.50</td>
<td>334D 3.00 3.10</td>
</tr>
<tr>
<td>Lagunillas</td>
<td>Lagoven</td>
<td>46+05-49+40 -3.50</td>
<td>AA-B 3.60 3.71</td>
</tr>
<tr>
<td></td>
<td>Maraven</td>
<td>63+00-65+72 -3.25</td>
<td>Bodega 4.16 4.26</td>
</tr>
<tr>
<td>Bachaquero</td>
<td>Maraven</td>
<td>0+00-</td>
<td>580 2.24 2.32</td>
</tr>
</tbody>
</table>

The first three harbours protect relatively lowly situated areas. The defence system is not sufficient in that wet spots can be detected behind the harbours. Piezometers show relatively high groundwater levels of a few decimeters below ground level. [see table 3.6] All these three harbours have been bypassed by so called bypass dikes. [see figure 3.47]

The old harbour basin has been closed by an earthfill dam in 1970. [see figure 3.48] This dam is part of the coastal defence system now. The crest height of the connection dike south of the harbour jetty is low and the dike is small and badly maintained.
[see figure 3.49] The harbour condition is good, the relatively highly situated harbour domain behind doesn't show wet or soft spots.

Bachaquero power station
The Bachaquero power station is situated in the far north end of this polder, immediately south of the river Pueblo Viejo. The inlet and outlet channels for cooling water interrupt the dike, forming gaps of 9.50 and 4.75 m respectively. [see figure 3.50]

A new power plant will be built near the existing one. Depending on its water cooling requirements inlet and outlet structures have to be designed.
3.7 HYDRAULIC BOUNDARY CONDITIONS.

3.7.1 Introduction.
The crest level is primarily determined by the lake water level, wave runup and freeboard. Hydraulic boundary conditions comprise the loads resulting from water conditions in and near the dike. Groundwater levels and water pressures have been dealt with in section 3.6.5 HYDROLOGY.

3.7.2 Design water level.
The lake water level fluctuates due to:

- astronomic tide, period 12h25
- seasonal water balance changes (fluctuations in discharges of rivers debouching into the lake, inflow from and outflow into the Caribbean, precipitation and evaporation
- seiches
- wind set up

Very few data are available but mareographs have been installed near the Lagoven harbour in Tia Juana and the Maraven harbour in Lagunillas, 20 km apart. In 1982 only the latter functioned properly, from 1982 till 1984 both readers supplied good records. From the 1981 records two periods of two distinct seasons have been selected to show the seasonal water level changes. [see figure 3.51 and table 3.7]

From the 1982 records it can be concluded that Lagunillas' data are also valid for Tia Juana. [see figures 3.52 and table 3.7]
The dry season (February) shows a stable, low water level in which the tidal effect can be clearly discriminated. The narrow entrance of the lake, near Maracaibo allows a very small tidal deflection of 5-6 cm. [see figure 3.51]
The rainy season (June) shows more distinct fluctuations due to rainstorms and strong winds, hiding the tidal effect. The average lake water level is in this period higher than in the dry season. Comparing 1981 and 1982 figures the records show maximum lake levels of the same order (June ca. +0.30 m MLLL). The 1983 and 1984 figures show lower values.[see table 3.7]
3.7.3 Wave runup.

Wave characteristics determine the wave attack on a certain dike slope. Wind speed, wind duration and water depth are the most important parameters that determine the significant wave height $H_s$ and wave period $T$. When a water basin is relatively small its area may determine the maximum wind fetch. In southwest direction from the Bolivar Coast, Lake Maracaibo has a width of 75 km; the maximum wind fetch $F = 75000$ m. Hence, storms with long duration can’t generate waves over an infinite length, so that wave growth will be limited.

Wave records as supplied by CSV are listed in table 3.2. Appendix 1 shows calculation methods for the significant wave height, assuming wind data from chapter 3.4.3. This yields values that are comparable to those of CSV. The significant wave height can be taken $H_s = 1.33$ m.

This significant wave height allows the calculation of the wave runup. If the inner slope is sufficiently protected against erosion, the crest height should be thus that only 2% of the waves is allowed to overtop the dike. For newly developed waves the 'Delft formula' is derived from the wave runup formula of d'Angremond/Oorschot:

$$z(2\%) = 8f \times H_s \tan \alpha \cos \beta \left(1 - \frac{8}{L}\right) \quad (1)$$

Waves aren’t steeper than $H/L = 5\%$. Assuming that the wave direction is perpendicular to the dike, i.e. $\beta = 0$, and that there is no berm, i.e. $B = 0$, this formula (1) reduces to:

$$z(2\%) = 8f \times H_s \tan \alpha \quad (2)$$

where $\alpha$ = the gradient of the outer slope

$f$ = a roughness factor varying from 0.55 for very rough surfaces to 1.20 for very smooth surfaces.

[see table 3.8]

Taking the slope gradient 1:3 and choosing the revetment with roughness between 0.7 and 1.00, a wave runup can be expected between 2.50 and 3.55 m.
3.8 GEOLOGICAL SITUATION.

3.8.1 Introduction.
In the Tertiary Period tectonical deformations formed the present mountain chains of West-Venezuela like Sierra de Perijá and the Mérida Andes. These mountain chains enclose geological basins of which the Maracaibo Basin is one originating from the Oligocene Epoch. [see figures 3.53 and 3.54]
Already before the Tertiary the transgressions of the sea in the Cretaceous Period created favourable conditions for the origin of oil. Regression took place in the Upper Cretaceous Period and later repeated transgressions occurred in the Tertiary epochs Eocene and Miocene with thick brackish and fresh water deposits. Thus the Maracaibo Basin has oil fields of these three geological periods.
Cretaceous oil fields occur west of Maracaibo and in the Colón region. In the project area, the Bolivar Coast, oil is extracted from Eocene and Miocene strata.
In the lake the oil is won from good, shallow Eocene sands of which the conditions become worse eastwards. The Miocene sands under the Bolivar Coast offer favourable conditions for extraction in the Bolivar Coastal fields: Cabimas, Tia Juana, Lagunillas, Pueblo Viejo, Bachaquero (depth ca. 1000 m). Together with the oil accumulations under the lake these coastal fields form one gigantic oil field. [see figures 3.55 and 3.56]
Alternatingly light and heavy oils are found. Secondary and tertiary production methods are necessary. [see figure 3.57]
The extraction of oil under the lake and under the Bolivar Coast areas causes land subsidence. Apart from the design water level plus freeboard this subsidence determines chiefly the crest height of the coastal dikes.
The region shows several geologic faults that are well able to produce considerable earthquakes.

3.8.2 Subsidence.

3.8.2.1 Phenomenon
Surface subsidence occurs at many different scales, both in time and space. Area and corresponding time dimensions are listed in
Section 3.2, HISTORY, describes the detection of subsidence in Lagunillas shortly after the start of oil production. Generally, fluid extraction like oil production, causes three types of surface deformation:

1. differential subsidence, centring on the oil field
2. centripetally directed horizontal displacements
3. surface faulting at places of strong curvature

### 3.8.2.2 Differential subsidence

Subsidence is a rather widespread phenomenon. The best known areas where it occurs as a result of oil production are situated in the United States: Long Beach Harbor area (California), Goose Creek, Wilmington (Texas). Roughly, the oil producing layers of Goose Creek, Wilmington and the Bolivar Coast occur at comparable depths.

Withdrawal of oil causes:

1. decrease of oil, gas and water volume
2. reduction of pore pressure

These two factors are the major contributions to material consolidation and compaction of the oil bearing strata at depth. This compaction results in surface subsidence, but not necessarily at a one-to-one rate. The degree of surface subsidence depends on the total volume of compaction, the depth of the compacted layers and to a certain extent the mechanical characteristics of the overlying strata, the overburden. In addition to this there is a time lag between oil extraction, compaction and surface subsidence.

There is a very distinct relationship between production volumes and compaction volumes. [see figure 3.58] Once having calculated the compaction of layers at depth, surface subsidence can be estimated with help of the extensive practical experience that is gained from coal mining, since there is no reason that the overburden would respond differently on different origins of compaction. The surface gets the shape of a bowl centred over and extending well beyond the oil field. [see figure 3.6, 3.7 and 3.59]
compressibility-general
Compressibility depends on the: -history of the overburden -type of soil -effective pressure -initial void pressure -time

compressibility-overburden
Post-Eocene sediments in the Bolivar Coast field (they form the overburden for the oil bearing Eocene strata) consist of uncemented sand and of clay. In compression they show a relatively stiff behaviour up to a certain intersection point, but when subjected to higher pressures they appear to be weaker. [see figure 3.60] This intersection point shows the maximum effective pressure that has ever existed in the material, which is an important factor in calculations with production/compaction and compaction/subsidence relationships. From this may be seen that compaction values depend on the history of the overburden; if a part of it has been removed by erosion, compaction values are of course lower than in the case without erosion. Initially overpressured oil layers cause on the other hand higher compaction values. For quantification it seems practical to use a certain averaged value, considering the history of the field. A systematic analysis is difficult since samples from spots where production takes place don’t give information about the field and soil conditions prior to the operation. Hence the intersection point is only a rough indication to use in compaction estimates. Table 3.10 shows close intersection point pressures for samples taken from the same well, i.e. soil samples with the same history.

compressibility-soil types
Experiments show that clastic sediments like sands, may show equal or larger values of compaction than of interbedded shales and siltstones within certain pressure ranges. [lit.5, vol.I, pg.56; vol.II, pg.368] Clay is assumed to compact completely.

The types of soil occurring in the Bolivar Coast region are clay and sand. Of the soil fractions smaller than 0.004 mm the mineral composition was kaolinite with illite, often mixed with
montmorillonite. It contains a considerable amount of silt. The sand fraction is generally fine with little variation in grain size (narrow distribution curves) and subangular.

**compressibility-effective pressure**

Values of compaction appear to be of equal order for clay and sand. Figure 3.61 shows the - relationships. For easy reference the relationship for pure illite and kaolinite are indicated. Figure 3.62 shows the relationship between packing and the effective pressure.

**compressibility-initial void ratio**

The dependance of the compressibility on the initial void ratio of the soil is obvious, the smaller the density, the more the soil can be compacted.

**compressibility-time**

Time effects are of various sources:

a) inertia of the overburden
b) development of the oil field
c) low permeability of clays
d) secondary compression and creep

a) It can be assumed from experience in coal mining that the delay caused by the inertia of the overburden is of the order of one or two years.

b) Since there is also a development of the oil field, like consecutive exploration of new wells, pressure drop and compaction don't start simultaneously in the entire oil field.

c) Low permeability in clay impedes the soil to compact instantaneously. Thick clay layers respond slowly to pressure drop. When about 50% of the compaction has occurred in time it will take about 3t before compaction is virtually completed. [see figure 3.63]

d) After an incremented load a soil sample shows continuing deformation which is called secondary compression for clay and creep for sand.

**subsidence-volumes**

Considering an oil field as a whole it is assumed that the volume of liquid production is equal to the final volume of compaction.
Potentially the total volume of subsidenee must be equal to the latter, but spread over an area larger than the oil field. Thus the vertical surface subsidenee value does not corespond to the compaction value perpendicularly below.

Figure 3.64 shows production volumes versus subsidenee volumes of the Bolivar Coast oil fields up to 1984. Disregarding the initial production period it is clear that en increase in fluid (oil, water and gas) production equals roughly the incremental subsidenee volume.

**subsidence-measurement**

Since the start of oil production in the Bolivar Coast area the oil companies involved have recorded subsidenee in the oil fields with help of a benchmark system. Primary benchmarks are located outside the area that is subjected to subsidenee. They are made of metal pipes founded at 20 to 30 m depth on non-compressible soil. For each primary benchmark three or more counter references exist. Secondary benchmarks in the subsidenee-prone areas are surveyed biannually and check-surveyed to the primary system. These benchmarks are founded at low depths of 2 m and follow the subsiding surface. It is however felt that there may be some mechanism thrusting secondary benchmarks upwards of which many examples are seen, also on e.g electricity masts that are founded at low depths. [see figure 3.18]

The subsidenee of the dike until 1984 is represented in figures 3.65, 3.66 and 3.67 as well as in table 3.11a and b until 1980.

**subsidence-values**

The method based on a linear relationship between oil production, compaction and surface subsidenee renders quantative subsidenee data until 2030. These predictions are of course valuable as long as production estimates comply with reality. [see figure 3.64]

The method has proved to be reliable comparing actual production and subsidenee values of the past. Figures 3.68, 3.69 and 3.70 show the predictions for subsidenee in 2000 of each production block derived from its expected production. Table 3.12 shows predicted subsidenee from 1984-2000 for the coastal dike derived from block surface subsidenee.

A rough estimate can be given for the year 2030 resulting in the
topography with contour lines as shown in figures 3.71 and 3.72.

3.8.2.3 Horizontal displacements
Differential subsidence is commonly accompanied by horizontal displacements in the direction of the centre of the oil field. Data from oil fields in Texas U.S.A. show that these displacements may amount up to 35% of the maximum vertical subsidence. This maximum horizontal displacement occurs halfway up the flanks of the subsidence bowl decreasing to zero at the centre and periphery of the bowl. [see figure 3.73]

3.8.2.4 Surface faulting
High angular surface subsidence may cause surface faulting where maximum tensional strains occur. This strong curvature occurs in sections 1 and 9 in Tia Juana, in sections 1 and 9 in Lagunillas and in section 1A in Bachaquero [see figures 3.65, 3.66, 3.67 and 3.73]. This problem is left unsolved as yet.

3.8.3 Earthquakes.

3.8.3.1 Introduction
Earthquakes yield ground motions that endanger the coastal protection in two ways:

1. Horizontal accelerations, horizontal forces may induce dike failure through sliding. [see figure 3.83]
2. Dynamic stresses may induce liquefaction of subsoil sand layers so that the dike toe could be undermined. Consequently this may trigger sliding.
3. Overtopping due to landslides (compare Vajont dam, Italy)
4. Overtopping due to earthquake induced seiches
5. Disruption by fault movement under the dike.

A short introduction to earthquakes and its ground motions is given in appendix 3.

3.8.3.2 Seismic situation
At large scale NW-Venezuela is situated in the southern part of the so called Caribbean Loop that belongs to the Circum Pacific Belt. [see figure 3.79]
Geologic faults occur around the project area as shown in figure 3.80. Seismic activity occurs in zones along these faults of which the Bocono Fault is most active.
The possible activity of the Bocono and Valera Fault as well as seismic activity of the local Icotea, Tia Juana and Pueblo Viejo Faults are of importance for the stability of the Bolivar Coast dikes.

3.8.3.3 Local ground motions
In 1969 Woodward & Clyde assessed maximum earthquake magnitudes and peak ground accelerations as listed in tables 3.14 and 3.15. These values result from calculation procedures developed by Seed and Idriss (1968) with incorporation of soil data of a 75 m boring in Lagunillas.
More recent calculations of Delft Soil Mechanics Laboratory with programs SHAKE and QUAD 4 confirmed the results except for the Bocono Fault where the peak ground acceleration was 0.05*g maximum.

After 1970 several strong earthquakes occurred elsewhere in the world during which many recordings were obtained, resulting in better knowledge of the attenuation mechanism. The numerous attenuation relationships that were developed after 1970, yield as many different results of which Esteva and Donovan comply best with the 1969 assessment of W&C. Still these attenuation relationships are too general for a proper assessment of peak ground accelerations.

Present knowledge of regional seismicity, of the nature of site conditions in the project area and the availability of new appropriate attenuation relationships provided W&C the possibility to work out a probabilistic framework to calculate the probability of peak ground motions. This resulted in a frequency relationship for peak ground accelerations that have to be considered in the safety of the Bolivar Coast Dikes [see figure 3.82]. The return periods for occurring horizontal ground accelerations are determined as if only $M = 7.5$ earthquakes occur in the region.

3.8.3.4 Experience in earthquake induced dike failure
Literature on dike failure through earthquakes is not available, but there is written ample information on earthfill dams.
Terzaghi points out that a factor of stability $G' > 1$ does not guarantee that a dike or dam slope will remain stable, depending on the character of the slope forming materials, which is illustrated in Table 3.18. Still, factors of stability $> 1$ computed with pseudostatic analysis are regarded to be sufficient. Seed [lit. 8] stresses that dam failures through earthquake induced sliding may generally be attributed to old fashioned construction methods and thus not representative for modern dams. Seed et al., 1978, carried out a study of the performance of embankment dams of which the conclusions are listed below:

(a) Hydraulic fill dams have been found to be vulnerable to failures under unfavourable conditions and one of the particularly unfavourable conditions would be expected to be the shaking produced by strong earthquakes. However, many hydraulic fill dams have performed well for many years and when they are built with reasonable slopes on good foundations, they can apparently survive moderately strong shaking with accelerations up to $0.2g$ from magnitude 6.5 earthquakes with no harmful effects.

(b) Virtually any well-built dam on a firm foundation can withstand moderate earthquake shaking, say with peak accelerations up to $0.2g$, with no detrimental effects.

(c) Dams constructed of clay soils on clay or rock foundations have withstood extremely strong shaking, ranging from 0.35 to $0.8g$ from a magnitude 8.25 earthquake with no apparent damage.

(d) Two rockfill dams have withstood moderately strong shaking with no significant damage and if the rockfill is kept dry by means of a concrete facing, such dams should be able to withstand extremely strong shaking with only small deformations.

(e) Dams which have suffered complete failure or slope failures as a result of earthquake shaking seem to have been constructed primarily with saturated sand shells or on saturated sand foundations.

(f) Since there is ample field evidence that well-built dams can withstand moderate shaking with peak accelerations up to at least $0.2g$, with no harmful effects, we should not waste our time analysing this type of problem - rather we should concentrate our efforts on those dams likely to present problems either because
of strong shaking involving accelerations well in excess of 0.2*g or because they incorporate large bodies of cohesionless materials (usually sands) which, if saturated, may lose most of their strength during earthquake shaking and thereby lead to undesirable movements.

Note that for taking into account a reasonably safe return period of 3000 years peak ground accelerations of 0.2*g occur in the three polders, which gives support to put effort into a pseudo-static analysis.

(g) For dams constructed of saturated cohesionless soils and subjected to strong shaking, a primary cause of damage or failure is the build-up of pore water pressures in the embankment and the possible loss of strength which may accrue as a result of these pore pressures. It is not possible to predict this type of failure by pseudo-static analyses, and other types of analysis techniques are required to provide a more reliable basis for evaluating field performance.

3.8.3.5 Liquefaction
Nedeco and Woodward & Clyde reports stress that liquefaction under the Bolivar Coast dikes can't be excluded. The chief conditions for it are present: shallow sand (-silt) layers near the toe of the dikes in a seismic area.

It remains however very difficult to assess the risk of liquefaction. There is little or no experience with earthquake wave propagation in this seismic Bocono-Merida zone; Woodward & Clyde assessed return periods for peak ground accelerations [see figure 3.82a].

So, on one hand it is difficult to assess ground motions near the dikes, on the other hand, after a limited soil investigation in 1976/77, an extensive shallow soil investigation has been carried out in the period 1983-1986. This soil investigation was aimed at data that are relevant for liquefaction, it comprised:

1. Piezocone testing. This gives information on the soil profile and the pore pressures, further it enables the evaluation of liquefaction potential as pointed at in appendix 4, A.4.4 sub 1.
2. Dissipation tests These give also information on pore water
pressures, but in a static situation.

3. **Density measurements.** Both nuclear and electrical tests were carried out as well as in laboratory from Begemann boring samples. Soil density is a very important factor.

4. **Begemann continuous borings.** These borings rendered data on the soil profile, the bulk density and the porosity.

5. **Cyclic triaxional tests.** These tests give direct information on the shear strength of the tested soils.

Table 3.24 gives the locations of these tests in the Tia Juana and the Lagunillas polders.

3.8.3.6 **Setting of the earthquake related problems**

Regarding Seed's conclusions b) and f) of section 3.8.3.4 and taking into account the probability of the occurrence of 0.2*g ground accelerations it may be useful to analyse the way of failure that is shown in figure 3.83, a pseudo static analysis. Regarding the conclusions f) and g) the problem of pore water pressure build-up and possible subsequent liquefaction must be looked into. This liquefaction problem is very suitable for an extended additional study.

Overtopping due to landslides into Lake Maracaibo is not a real problem to be considered, because of the vastness of the lake.
There are numerous ways how dikes can fail. A combination and consecution of unfavourable conditions may eventually lead to what is called an 'undesirable top event'. The definition of this top event may be: the coastal protection fails when an excess amount of water passes the dikes.

4.1 PRIMARY FAILURE CRITERIA

Generally there are three ways in which water passes the dikes:

1. Water flowing over the dikes: overtopping
2. Groundwater flowing through the dikes and subsoil: excess seepage
3. Water flow through channels in or under the dikes: piping

These three events are denoted here as the primary failure criteria, they do not necessarily lead to inundation of the polders: depending on the amount of water flowing into the polder, the discharge capacity of the drainage system may be sufficient to save the polder from inundation at first.

A great variety of consecutive causes leads to these primary ways of failure. Depending on the situation of the dikes, lots of these causes can be ruled out, while other factors have to be added or stressed.

The Bolivar Coast Dikes can be characterized as lake dikes in a seismic, subsiding, industrial and tropical area. This characterization excludes factors like frost (tropical), greatly varying water levels (lake, though there is a very small tidal deflection). On the other hand it puts stress on factors like earthquake induced sliding and/or liquefaction (seismic area), possible slope erosion due to heavy tropical rains, an ever increasing retaining height in the course of years (subsiding oil production area) and detail problems, related to foreign industrial elements in the dike.

1. Overtopping

It is logic that there are in general two cause through which the dikes could be overtopped: either the dike are (suddenly) too low or the water reaches too high for some reason [see figure 4.1a]. The causes haven't been worked out in extenso, since this would show a far too complicated picture of the way these dikes might fail. Details will be shown, when actually
calculating the possible ways of failure.

2. *Excess seepage*
Seepage can rather easily be divided into seepage through the dike body itself and seepage through the subsoil, but this problem always confines to water heads (water head differences) and the permeability of soils [see figure 4.1b]. Since the Bolivar Coast Dikes are lake dikes, the differential water heads can be said not to vary, hence the assumption of a stationary system is fair. In the actual situation, the right branch of figure 4.1b is hardly applicable to the Bolivar Coast Dikes [see section 3.6.5.2].

3. *Piping*
The primary failure way of piping is very much related to the problem of seeping water. The problem here is that the seeping water may concentrate and erode the soil, thus forming perfect water channels, through the dike or its underlying layers [see figure 4.1c and appendix 3]. The often present, relatively impermeable top layer at ground level, may be subject to hydraulic fracturing.
4.2 SECONDARY FAILURE CRITERIA

The attributive to failure criteria or events 'primary' and 'secondary' refer to their place in the schemes of figure 4.1.

probability
All events (read 'loads on the dike') and strengths are basically of a stochastic nature.

Stochastic events may have a naturally 'attacking or surprising' character, like storms (→ wind → wind waves and → wind setup), rainfall, earthquakes (→ accelerations and → liquefaction), seiches. Assumptions are to be made statistically, i.e. with help of historical data, concerning the magnitude of their attack, combined with a certain return period in which the assessed magnitude may be exceeded.

The strength of the dikes and their foundations, subsoil are intrinsically also stochastic phenomena. These are however, not of a surprising nature, and by thorough investigation and careful construction, they can be determined at about 20% accuracy. Also certain 'loads' on the dike can be determined very accurately like e.g. the tidal deflection.

In general it can be said for the dikes that the strength can be determined relatively accurately, where for the loads must be counted with less certain values and return periods. Schematically this can be represented as shown in figure 4.2. This figure shows that the probability of the load to exceed the zero-value is 1 and that there is always a still very small chance for the load to exceed a very high value. Of the strength can be said that there is always a minimum value unless there wouldn't be a dike at all. Its strength, built up of soil parameters, with stochastic values, can be determined with 20% accuracy, represented by the narrow plot. Strength S is to be bigger than load L: \(S > L\), for a fail safe dike.

As the load can always exceed a certain value there must be chosen a certain acceptable risk, a certain acceptable return period for the dike to fail. A fail safe dike doesn't exist.

For illustration of the stochastic aspect it is shown that if the load L would equal the strength S indeed (with a certain probability), there would still be a chance of 50% that the dike would stand the load! The value S is just the mean value of the probability distribution of the strength of the dike.
4 failure systems

overtopping

A further way of division in the failure tree of figure 4.1 is 'crest height too low' and 'water too high'. Events that can eventually develop to primary failure events are referred to as secondary failure events or secondary failure criteria. [see figure 4.1a]

Consider first the case that the 'water is too high' somehow. As defined above secondary failure criteria in this branch of the tree are:

1. exceedance of design water level (tide, wind setup, discharge)
2. seiches
3. wave runup

These aspects are dealt with in chapter 6, section 6.1.

Secondly, consider the case 'crest height too low'. In this case the secondary failure criteria are:

4. settlement of the dike body material
5. settlement of the dike due to consolidation of the subsoil
6. subsidence
7. failure of dike slope, i.e. loss of stability (deep slip circles, erosion, liquefaction)

Chapter 6, section 6.2 deals with settlement and subsidence, chapter 8 with everything concerning the stability of the dikes. Liquefaction is treated in a separate chapter 9.

piping

Chapter 10 treats the primary failure event piping. Secondary failure phenomena in this concept are the formation of crevices under or along foreign structures or at layer boundaries, where soil particles are relatively easily taken along with groundwaterflow. Concentration of flow and consecutive further erosion is the result.

seepage

Chapter 11 and appendix 6 treat seepage as primary failure event. Secondary failure events in this concept are malfunctioning of present seepage screens (sheet pile constructions) and too high seepage forces that cause erosion of the slopes.
5.0 PROBLEM SETTING

5.1 REPRESENTATIVE SECTIONS.

The less common phenomena to which these dikes are subject are earthquakes, earthquake induced liquefaction and subsidence. All other ways of failure are more or less common in normal dike design. The higher the retaining height of the dikes, the more pronounced the problems show up. A higher water gradient introduces higher water pressures, \( u \) and seepage forces \( F = \rho_w g \), with negative effect on stability (effective pressure, shearing strength), piping (high gradients), seepage, resistance to liquefaction (effective pressure). Furthermore, higher dikes are heavier, with corresponding negative effect on settlement and on bigger mass forces when earthquakes occur.

Considering the above it is obvious that the dikes in the central parts of the polders will show problems most strongly.

Tia Juana 3 to 6 and Lagunillas 5 are thus chosen as representative sections for further calculation. Old sand layers in these dike bodies, probably old coast roads, influence the water conditions in these sections strongly. The well conductive, but closed off sand mass in Tia Juana leaves the way open for the groundwater to build up high water pressures in the dike body and a relatively high phreatic level, whereas the through sand layer in Lagunillas functions as a drain.

A further section in Lagunillas is more closely considered. A separate liquefaction study [see lit. 25 and 26] showed that one specific section, Lagunillas BA, is subject to earthquake induced liquefaction danger, because of a low density sand layer of 1.5 m thickness at 2.5 m under the base of the dike body.

In lit. 26 is suggested that low density sand layers (subject to danger of liquefaction) have low CPT-values and a low friction ratio [see figure 9.3]. The possibility of loose packing is sustained by the steep grain size distribution of this sand layer [see figure 9.4].
5.2 DESIGN CRITERIA

In general it is rather difficult to distinguish design criteria and assumptions for the design of structures. A design criterion may be defined as a demand, as a requirement that has to be fulfilled in any case. An assumption shall be defined as the chosen or derived datum from a series of possible data that describe the prevailing situation in the project area; it may also concern the choice of a certain concept for further calculation.

lifetime
Like any structure dikes have to be designed for a certain period, lifetime. However in the present case, oil production continues and hence, subsidence, too. The subsidence is quantitatively directly dependant on the quantity of oil production and as the oil crises of the latest 15 years clearly show, production varies and subsidence correspondingly.
Still a rough estimate of subsidence has been made for the year 2030 and this is the year the the dikes will be designed for. The design for 2030 may however require a too big investment. Recommendations shall be given for dike raising in two or more stages.

construction height
The construction height must be sufficient to cover:
- consolidation
- settlement
- design water level (DWL) + the 2% wave runup (z(2%))

In general this will result in a higher construction height than the so-called minimum crest level (MCL). This MCL is dependant on the last criterion DWL + z(2%) only.

stability
In the normal situation, in the rainy season (i.e. with high lake water level), the dike shall be stable in the stationary condition, taking into account earthquakes with maximum accelerations of a 4000 year return period. The 2% waves that are allowed to overtop the dike are, as experience has shown, controllable, provided that the inner slope is not too steep (1:3) and covered with good revetment.

liquefaction
The dike shall be stable with respect to earthquake induced liquefaction, taking into account the same earthquakes as
mentioned above.

**piping**
Concentration of flow, due to exceedance of critical gradients or to cracking as a result of differential settlements should be avoided.

**seepage**
The dike must not necessarily be impermeable to water. The amount of seepage water shall be low, such that the present pumping stations are of sufficient capacity.

**foreign elements**
Foreign elements in the dike body should be avoided as much as possible. In this oil production area however, this is a virtually impossible task. Foreign elements have to be accepted and special attention must be paid to 'artificially' induced piping, special measurements must be taken there.

**inspection**
The dike must be well attainable for proper inspection of the actual state.
5 problem setting

5.3 ASSUMPTIONS

In this section the results of the investigations of chapter 3 will be summarized and concentrated in the assumptions that should be used for further calculation.

dike profile and soil data
Present dike profiles for the chosen representative sections are taken as the Maraven drawings of 1984. [see figures 3.23a and b, 3.24d, 3.25d and 3.25f] These profiles are schematized as shown in figures 5.1 to 5.3.
Soil data are mainly taken from the 1957 soil investigation of DSML [see section 3.6.4] and resumed in the schematized drawings of figures 5.1 to 5.3.

subsidence
For estimation of subsidence figures, the data as supplied by Maraven, will be used. Their production method has proved to be very accurate. This method produced the contour lines of figures 3.71 and 3.72. For the representative sections polder ground levels (PGL) for the year 2030 are estimated:

Tia Juana section 3 to 6: PGL = 4.50 MLLL, \( \delta_{\text{H}} = 1.50 \text{ m} \)
Lagunillas section 5 : PGL = 5.60 MLLL, \( \delta_{\text{H}} = 2.60 \text{ m} \)
Lagunillas section 8A : PGL = 3.70 MLLL, \( \delta_{\text{H}} = 1.60 \text{ m} \)

The subsidence between 1984 and 2030 is referred to as \( \delta_{\text{H}} \).

hydraulic boundary conditions
The design water level (DWL) is 0.30 M.L.L.L.
The wind fetch is taken as the distance to the west coast of the lake, being 75000 m and the decisive wind-speed-duration combination 9.2 m/s - 4.6 h. Calculating according to Mitsuyasu [see appendix 1] the significant wave height becomes \( H_s = 1.40 \text{ m} \).
Wave runup depends then on dike slope and revetment. It will vary from 2.60 m to 3.75 m.

earthquake accelerations
For the 4000 year return period statistical data will be used as supplied by Woodward Clyde Consultants [see figure 3.82].
Earthquake accelerations are transformed to accelerations as if they were developed by an \( M = 7.5 \) earthquake:
Tia Juana polder : \( a_{\text{H}} = 0.22 \text{g} \)
Lagunillas polder: \( a_{\text{H}} = 0.18 \text{g} \)
liquefaction
For the evaluation of liquefaction potential the Seed's approach shall be used, adapted for non-level ground conditions and using CPT-data of 1985. [see additional study lit.26]

piping
In the presence of an overlying relatively impermeable layer, it is common to use the concepts of Bligh and Lane for the evaluation of the danger of piping. This is however not sufficient. The danger for piping must be shifted from under the vital dike body to under a toe blanket. This toe blanket must meet with occurring high water head differences over the relatively impermeable top layer. Thickness of this toe blanket must be so that the hydraulic gradient fulfills the following requirements:

- at the toe of the dike : $i < 0.5$
- at distance 2H to 3H of the toe: $i < 0.7$

seepage
The Bolívar Coast Dikes will have to be checked on the most unfavourable situation with respect to seeping forces: water seeping horizontally from the inner slope. The direction of flow then is $\theta = 0$. Assuming that the inner slope consist of a cohesional soil (washout of soil particles won't occur) the slope must fulfill the requirements of figure 11.9. In case that the slope wouldn't fulfill these requirements the slope will have to be checked with actual groundwaterflow as calculated in chapter 7.
6.0 DETERMINATION OF CREST HEIGHT

In determining the crest height it is important to distinguish the minimum crest level (MCL), a design criterion and the minimum crest construction height (MCH), which is higher than the minimum crest level because of several crest decreasing factors, like settlement, consolidation, and subsidence.

6.1 MINIMUM CREST LEVEL
The minimum crest level depends on [see figure 6.1]:

1. design water level
2. other factors that cause lake water level rise: a. seiches
   b. oscillation
3. wave runup

6.1.1. Design water level
The water level is in general a stochastic feature. The factors that influence the lake water level near the area partly determined by non-stochastic determinable phenomena and partly by wind setup, rain (river discharge into the lake). A statistical approach however, for the determination of the design water level DWL is not feasible here, because of a serious lack of water level data. The lake water level is however very constant, so that it seems justified to assume a stationary situation as described above in section 3.7.2. The measurements that are available include the only stochastic feature of the lake, occurring in the rainy season: rainstorms. This concerns 4 to 5 hour fresh breezes of Beaufort 5, causing wind setup of circa 5 cms as read from the records of figure 3.51 and 3.52.

calculation of wind setup
Wind setup can also be calculated from the wind records that are available for longer periods. [see figure 6.7] The wind across lake Maracaibo (strongest and longest measured is straight towards the Bolívar Coast Dikes - 9.0 m/s, 5.0 hrs) exerts a frictional force on the water surface causing it to incline. This frictional force \( F_w \) per column water \( dS \) depends on the density of air and the wind speed:

\[
F_w = c_i \cdot \rho_i \cdot (U_w)^2 \cdot dS \quad (1)
\]

This inclining water surface causes water to flow from the
5.4 PRELIMINARY DIKE PROFILE

For the purpose of further designing, asort of preliminary dike cross section shall be chosen, which enables us to estimate its future occupation of space, area, its consolidation settlements, its compaction settlements, stability factors. The design of a dike is in fact an iterative process in which this preliminary dike profile is the best first guess.

With data already available and independant from the dike profile, design water level (wind setup), seiches and significant wave height have been determined. The an assumption must be made for outer slope angle and revetment to determine the minimum crest level: slope angle \( \tan \alpha = \frac{1}{3} \), and the same rip-rap revetment as already present.

To prevent erosion of overtopping waves, a good grass revetment is very common and then at least a 1:3 slope for maintenance.

To meet with piping and stability problems the dike must be sustained by a berm, toe blanket at the landside. This toe blanket must prevent piping: its width must be two to three times the retaining height \( H \). The thickness may first be guessed at 20 to 25\% of \( H \) in order to keep the critical gradient over the top soil layers low. The toe blanket slopes down at 1:15 or 1:20 [see figures 5.1, 5.2 and 5.3].
Bolívar Coast backwards along the bottom of Lake Maracaibo. This backflow balances the water flown towards the coast along the surface. From the equilibrium the wind setup at the coast can be calculated. The wind setup must have fully developed to assume this equilibrium. This condition is fulfilled as shown in appendix 1.

The balancing force of the backflow \( F_b \) per column of water ds, depends on the weight of water, the water depth and the inclination of the surface:

\[
F_b = \rho g d i \, dS
\]  

(2)

Not counting energy loss of e.g. friction of the lake bottom, \( F_b \) must equal \( F_w \) and from this follows an expression for \( i \):

\[
i = c (\omega)^2 / (gd)
\]

(3)

where \( c \) has the value \( 3.5 \times 10^{-6} \). The average water depth of Lake Maracaibo is about 10 m and the centre line of gravity for the surface is at ca. 34.6 km from the Bolívar Coast (see figure 6.7). This yields the wind setup at 10.0 cms.

Taking the available records of figure 3.51 and 3.52, and substracting the stochastic part of the maximum values it is justified to determine the lake water level + tide at ca. 0.20 MLLL. Adding then the calculated wind setup of 10 cms the design water level becomes: \( \text{DWL} = 0.30 + \text{MLL} \).

6.1.2. Other factors that cause lake water level rise

a. seiches

Seiches are very pronounced water level rises of 20 to 30 cms, occurring irregularly. They originate from oscillations of the water surface or short heavy local rainstorms. Especially this second origin gives support to the idea to count for these seiches in the determination of the minimum crest level. On the other hand, there are no records yet, that say that seiches would occur, but not many records are available at all. Seiches can also be induced by earthquakes, neither about this are records available. Seiches will be counted for: \( s = 0.20 \text{ m} \).

b. oscillation effect

A temporarily water level rise at the opposite lake coast due to wind setup may oscillate to the dike coast and appear
there amplified. This has occurred in the Kieler Bucht in
the Baltic Sea basin, when a storm hit the coast of the
Lithuanian S.S.R. and inundation problems occurred later,
far at the opposite side of the basin.
No further attention will be paid to this problem in this
report, the most severe storms are directed straight towards
the coast dikes.

6.1.3. Wave runup

Wind data are sufficiently available to determine the height
of the waves in the lake and thus the wave runup on the
dike. A central value for further calculation is then $H_s$, the
significant wave height of the wind waves. Numerous
methods have been developed to determine this $H_s$ that
deeps on wind fetch, wind speed, wind duration and water
depth. Appendix 1 sums up these methods and gives few
results that comply with data supplied by Compañía Shell de
Venezuela (CSV): $H_s = 1.35$ m. Waves run up the outer slope
dikes. Depending on the steepness and the revetment of
the slope, the presence of a berm and the height of the
waves and their direction of propagation, a value for the
wave runup $z_{1%}$ can be determined in which 1% of the waves
exceed this $z$ [see figure 6.2]. For 1% the common value in
the Netherlands is 2%, allowing 2% of the waves to overtop
the dike. assuming that the dike crest is not higher than
the DWL + $z_{1%}$. The probability of the exceedance of a certain value $z$ for
wave runup is determined by the Rayleigh-distribution:

$$P(z > z_{1%}) = 0.01^*i = \exp \{ -2(z_i/z_a)^2 \} \quad (4)$$

Here is $z_a$ the wave runup of the average highest third of
the waves, i.e. the highest 13% of the waves: significant
wave runup.

The choice to allow 2% overtopping is based on the fact that
this amount is controllable, provided a not too steep inner
slope and proper revetment. If these conditions can't be
fulfilled it could be necessary to advise allowance of 1%
overtopping only. From the Rayleigh distribution follows
that $z_{2%} = 1.40 z_a$ and that $z_{1%} = 1.52 z_a = 1.09 z_{2%}$. A formula for significant wave runup of d'Angremond and van
Oorschot results in the 'Delft formula' for 2% wave runup
for newly developed waves:
6 crest height

\[ z_{2x} = 8fH_2 \tan \alpha \cos \beta \left( 1 - \frac{B}{L} \right) \]  

(5a)

which is reduced to:

\[ z_{2x} = 8fH_2 \tan \alpha \]  

(5b)

under the assumptions of no berm and waves perpendicular to the dike as earlier discussed in section 3.7.3.

In the present situation the outer slope revetment consists of rip-rap blocks 300 to 800 kg, density 2650 kg/m³. This is a rough revetment, reduction factor \( f = 0.7 \).

Building further according to the present dike design, slopes are 1:3, \( \tan \alpha = 1/3 \).

Starting from this the wave runup may be estimated at \( z_{2x} = 2.50 \) m.

6.1.4 Minimum crest level

All the above mentioned factors must be counted for in the determination of the minimum crest level. Like shown in figure 6.1 MCL = design water level + freeboard

= design water level + seiches + wave runup

= DWL + S + \( z_{2x} \)

= 0.30 + 0.20 + 2.50

\[ \text{MCL} = 3.00 \text{ m M.L.L.L.} \]
6.2 MINIMUM CONSTRUCTION HEIGHT

In general it is not sufficient to raise a dike to the minimum crest level only as determined in the above section. In the course of time the crest height is reduced absolutely and possibly also relatively.

1. compression (consolidation) settlement
2. compaction settlement of the dike body
3. soil subsidence
4. unstable layers
5. relative sea (lake) water level rise

All these factors must be counted for when constructing the dike, a certain overheight must exist [see figure 6.1]

6.2.1. Compression settlement

Compression settlement can be divided into three parts, immediate settlement, \( \delta_i \), consolidation settlement, \( \delta_c \), and secondary settlement, \( \delta_s \).

Further there should be made distinction between settlements below the preconsolidation load \( p_c \) and with higher loads.

**Immediate settlement**

Immediate settlement results from the constant volume distortion of the loaded soil mass. In this part of the total settlement a decrease because of (slow) pore water expulsion is not being considered, i.e. the unconsolidated case.

When a dike is raised, the load on the subsoil increases, which is thus being compressed. This compression behaves according to the logarithmic compression formula of Terzaghi. For a relatively small increase of load there is a linear relationship between load and deformation [see figure 6.3a]:

\[
\delta_i = m_v d \Delta \sigma' \\
\delta_i / d = m_v \Delta \sigma' \\
\delta_i / d = (\delta_i / d)_0 + 1/C \log \sigma' 
\]

The compression coefficient \( m_v \) is however dependant on the effective stress \( \sigma' \). Logarithmically the plot is more or less a straight line [see figure 6.3b]:

\[
\delta_i / d = (\delta_i / d)_0 + 1/C \log \sigma' 
\]
Usually this formula is written differently, expressing directly the settlement \( \delta_1 \) of a certain layer and using the Napier logarithm:

\[
\delta_1 = \frac{d}{C} \ln\left(\frac{\sigma' + \Delta\sigma'}{\sigma'}\right)
\]

(8)

The total immediate settlement then, is the sum of the settlements of all distinct layers. Table 6.1 lists general values for \( C \) for different soil types.

The soil is sustained at the sides, which causes smaller deformation values \( \delta_1 \) in comparison to free soil, where \( \varepsilon = \sigma/\varepsilon \). In this case

\[
\varepsilon = \frac{\sigma}{\varepsilon} \frac{(1+\nu)(1-2\nu)}{(1-\nu)} < \sigma/\varepsilon.
\]

Note that the soil is stiffer under higher pressure: \( E = f(\sigma) \). Unlike under shearing load, soil under laterally confined compression becomes stiffer and won't fail.

This immediate settlement applies in fact only for sand layers, where water yields easily and quickly under the extra load. For cohesive soils it is better not to count any immediate settlement \( \delta_1 \).

**Consolidation settlement**

The above mentioned relationship between stress and deformation does not depend on the time. For cohesive soils however, the extra load is at first compensated by pore water pressure rise. Under this pressure the pore water starts to find its way out from the cohesive, little permeable soil, with resulting compaction of the soil layer: consolidation.

Principally this is the same case like in the more permeable sands, immediate compaction and settlement. In cohesive soils this process takes much more time: the hydrodynamic period.

Although this process takes time the ultimate consolidation settlement \( \delta_e \) for time \( t = \infty \) and layer thickness \( d \) is equal to \( \delta_1 \):

\[
\delta_e = m_\varepsilon d \Delta\sigma'
\]

(9)

and making the same alterations as for \( \delta_1 \):

\[
\delta_e = \frac{d}{C} \ln \left\{ \frac{(\sigma' + \Delta\sigma')/\sigma'}{\sigma'/\sigma'} \right\}
\]

(10)

The total consolidation time \( t_e \) depends chiefly on the permeability \( k \), layer thickness \( d \) and compression coefficient \( m_\varepsilon \).

Total consolidation will never be reached. Simplified, the practical end of consolidation is
\[ t_\ast = (m_\ast \cdot \gamma_\ast \cdot d^2)/(2k) \] (11)

(This time is determined with the so-called \( \sqrt{t} \)-method of Taylor. This method makes use of a dimensionless time factor \( T = c_\ast t/d^2 \), where \( c_\ast \) is the consolidation coefficient and equal to \( k/(m_\ast \cdot \gamma_\ast) \).

In this calculation appears an \( e^T \), hence follows that \( T \) shall be \( T = 2 \) for 99% consolidation. We consider here layers that drain at two sides, for layers that drain at one side only formula 8 becomes \( t_\ast = (2m_\ast \cdot \gamma_\ast \cdot d^2)/k \).)

Take e.g. clay with \( k = 10^{-10} \) m/s and \( m_\ast = 10^{-3} \). The time can now be expressed in terms layer thickness:

\[ t_\ast = (10^{-3} \cdot 10/10^{-10}) \cdot d^2 \text{ seconds} \]
\[ = 1.6 \cdot d^2 \text{ years} \]

In comparison to sand with \( k = 10^{-4} \) and \( m_\ast = 10^{-4} \): \( t_\ast = 10 \cdot d^2 \) sec. These seconds are neglected, the reason why in the case of sand we speak of immediate settlement.

**Secondary settlement**

In the normal consolidation theory the settlements approach the ultimate settlement \( \delta \) asymptotically. In reality this is different. After completion of the consolidation settlement the soil compaction continues, without increase of the load. This effect, like creep in concrete, is called secondary settlement \( \delta \ast \) [see figure 6.4].

Keeverling Buisman (who called this secondary settlement 'secular settlement', from the Latin word 'saeculum' = century) described this phenomenon in a semi-logarithmic formula [see figure 6.4]:

\[ \delta_{\ast, t} = d \Delta \sigma' (\alpha_\ast + \alpha_\ast \log(t/t_0)) \] (12)

Here is \( \delta_{\ast, t} \) secondary settlement at time is \( t \)

- \( d \) layer thickness
- \( \Delta \sigma' \) increase of effective stress
- \( \alpha_\ast \) primary compression modulus at \( t = t_0 \)
- \( \alpha_\ast \) secondary compression modulus, which indicates the increase of settlement after \( t = 10 \cdot t_0 \)
- \( t_0 \) time unit (commonly used is 1 day)

Values for \( \alpha_\ast \) and \( \alpha_\ast \) are registered in table 6.2.

Koppejan combined the logarithmic formula of Terzaghi and
the semi-logarithmic formula of Keverling Buisman to find the total settlement \( \delta_{\text{tot},t} \) at a certain time \( t \):

\[
\delta_{\text{tot},t} = d \left\{ \frac{1}{C_p} + \frac{1}{C_v} \log \frac{t}{t_0} \right\} \ln\left( \frac{\sigma' + \Delta \sigma'}{\sigma'} \right)
\] (13)

For soils that don't show this effect of secondary settlement the value of \( C_v = \infty \). For other values of \( C_v \) and \( C_p \) see table 6.3.

Note that this formula contains the factor time, but only the time factor of secondary settlement. The hydrodynamic consolidation period of primary settlement is not being considered.

**preconsolidation load**

Soil is not a purely elastic material, on the contrary. Hence soil that has been subject to a certain load in the past, that has somehow been removed again, has been deformed both elastically and plastically, the elastic deformation is restored, the plastic deformation has remained. This soil has been compacted and thus become stiffer than the same soil not compacted. This shows itself in a higher primary compression modulus \( C \) [see figure 6.5].

Often the history of the soil is not known, but compression test show commonly two very distinct values of primary compression moduli, \( C \) and \( C' \). It is generally assumed that the soil under consideration has been loaded before with the so called pre-consolidation load \( P_c \): until this load the soil behaves stiffer, higher \( C \), elastic settlement will again occur, but no primary consolidation settlement. When applying higher loads to this soil, higher than \( P_c \), it behaves normally again, according to a lower \( C' \) [see table 6.1].

The preconsolidation load can have disappeared in several ways, like simple removal of the overburden, fluctuations of the groundwater table, capillary water stresses, cold welding of mineral contact points between particles. All these factors result in a higher preconsolidation pressure than the actually present effective overburden pressure.

**actual calculation of settlements**

For the calculation of the settlement of the dike bodies at the Costa Oriental del Lago de Maracaibo for the year 2030 the secondary settlements must also be considered. Secondary or secular settlements cover centuries, which is long compared to the period 1984-2030, but the start of this secondary settlement is usually taken after the practical end of the primary settlement (\( T=2.99\% \) of the total primary consolidation), which may cover a period in the order of
six, seven years. This is the approach of Koppejan and Keverling-Buisman. Another approach is that there is a mechanism through which the soil starts its secondary settlement already from t=0 on and must be added immediately to the primary settlement. This approach won't be followed here [see figure 6.8]. The settlements must be calculated per layer soil, until a depth where the increase of effective stress is still felt. For this purpose the dike raising is schematized to a strip of land and the effective stress increase is determined with help of figure 6.6.

**Tia Juana 3**
The subsoil here consists of relatively stiff soil layers. An assumed overheight of 1.00 m appears to be sufficient to cover settlement and compaction of the applied fill [see figure 6.9]. A raising of 3.50 m causes settlement of the subsoil, 0.63 m and 10% compaction of the fill, 0.35 m [see next section 6.2.2]:

\[ \delta_{\text{tot},t} = \delta_i + \delta_c = 0.63 \text{ m}, \quad \delta_b = 0.35 \text{ m}. \]

**Lagunillas 5**
This section is founded on weaker and thick layers, causing much bigger settlements. The assumption of 1.50 m was optimistically small; a total of 2.15 m should be sufficient [see figure 6.10] including 8% compaction of the dike body fill \( \delta_b \):

\[ \delta_{\text{tot},t} = 1.52 + 10\% \approx 1.70 \text{ m}, \quad \delta_b = 8\% \times 5.50 = 0.45 \text{ m}. \]

**Lagunillas 8A**
The assumption of 2.50 m overheight appears to be sufficient after the settlement calculation [see figure 6.10]. It covers the settlement of 1.60 m and the 8% compaction of the applied fill. The overheight may even be reduced with 30 cm till 2.20 m:

\[ \delta_{\text{tot},t} = 1.60 \text{ m}, \quad \delta_b = 8\% \times 7.00 \approx 0.60 \text{ m}. \]

**6.2.2 Compaction settlement of the dike body**

After the dike raising the newly applied soil will still compact. The soil particles settle to a denser compaction. No calculations will be made for this. Experience has already given values for it. Well compacted during construction and with proper execution of the work the
settling may be estimated at $\delta_s = 10\%$ for clayfill and at $5\%$ for sandfill.

6.2.3 Soil subsidence

As already amply described above the soil subsides because of oil production. Values for the subsidence, based on production methods, have already been incorporated in the assumptions for the situation in the year 2030 [see section 5.3].

The overheight that must be given to the three representative dike sections are respectively (considering 1984-2030):

- Tía Juana section 3-6: $\delta_s = 1.50$ m
- Lagunillas section 5: $\delta_s = 2.60$ m
- Lagunillas section 8A: $\delta_s = 2.50$ m

6.2.4 Unstable layers

Unstable layers are certain subsoil layers that are susceptible to lose their stability. In this case dike section 8A of the Lagunillas polder has a loosely packed sand layer at few meters under the base of the dike. This layer is susceptible to earthquake induced liquefaction.

6.2.5 Relative sea (lake) water level rise

Climate changes may cause a rise of the average sea level and thus of Lake Maracaibo, that is directly connected to the Caribbean Sea. In the course of years there could also be a certain compaction of the land. Both these phenomena won't be considered.

6.2.6 Minimum crest construction height

The minimum crest construction height (MCH) can now be calculated including the factors as stated above:

$$MCH = MCL + \text{settlement} + \text{dike body compaction} + \text{soil subsidence}$$

$$= MCL + (\delta_t + \delta_e) + \delta_s + \delta_s$$

The calculations as shown above, in section 6.2.1 yield the required construction heights for the representative
6 crest height

sections:

Tía Juana 3 : MCH = 3.00 + 0.65 + 0.35 = 4.00' m.

Lagunillas 5 : MCH = 3.00 + 1.70 + 0.45 = 5.15' m.

Lagunillas 8A : MCH = 3.00 + 1.60 + 0.60 = 5.20' m.

The dike level to which the dike should be raised is then the level MCH + (δ₀ — δ₀,year), where δ₀ is the subsidence between 1984 and 2030 and δ₀,year the accumulated subsidence in the year of construction from 1984 on. See table 3.12 for the values up to the year 2000.
7.0 WATER MOVEMENTS

The weight of soil depends on its compaction and water content. The stresses in the soil depend on the weight of the overburden, including the soil, and on the water pressures \( (\sigma = \sigma' + u) \). Whether piping will occur or not depends on the water pressures in the water bearing layers under relatively little permeable top layers. Seepage and seepage forces depend on the water head difference, permeability of layers and the geometry of those layers. It is in fact necessary for all primary types of failure, to know the water conditions: phreatic levels and water pressures.

7.1 MSEEP PROGRAM

the program
MSEEP is a groundwaterflow computer program, developed for use on personal computers by Delft Geotechnics and Public Works (Rijkswaterstaat) in the Netherlands. Originally it is a mainframe computer program operating under the name SEEP. The program language is Turbo-Pascal. MSEEP can be run on an IBM-compatible PC or AT with 80286 microprocessor and at least a 512 Kb RAM. Further a monochrome screen is necessary and a graphic chart. To avoid long durations of calculation it is advisable to apply the 80287 mathematical coprocessor. The program has a menue structure. The user stores input data in data files, interactively. The program asks for certain data and shows the input on screen. All data can be checked graphically and partly changed later. The graphical output can show the geometry and the potential lines, before long also streamlines. After calculation, the results are stored in an output data file, together with input data. All results can be printed.

possibilities
The program offers possibilities to calculate water potentials in soil masses with 5 layers (6 layer divisions) in two dimensions. Further options are sheet pile constructions, drains and fixed potentials, wells and fixed discharges, phreatic surfaces (pressure head = elevation head) and seeping surfaces (pressure head > elevation head), and phreatic lines or closed surfaces.

theory
The technical theory used for these personal computer programs offered by Geotechnics Delft (possibly in
cooperation with others) always concern generally accepted theories, state of the art.

MSEEP program is based on the finite elements method (FEM) with isoparametric elements. For the solution the Gauss-Seidel iteration is used. The mesh generator is relatively simple. The solution for the phreatic line is found by stepwise adaptation of its first assumption at the ground surface.

limitations
MSEEP is limited to 500 knots and 700 elements. Within this limitation it is possible to increase the number of layers from 5 to 6 (7 layer boundaries). A further limitation is that layer boundaries must have a continuously increasing x-coordinate. This is especially felt when old, often raised dikes have to be calculated. The 'side-kick' menu does offer the possibility to change parameters of single elements, but this becomes a very elaborate task.

running MSEEP
Using MSEEP for the Bolívar Coast Dikes is not really an easy assignment, especially not when running the program for the first time. The complicated profile of the dike, with its many layer boundaries and its old sheet pile construction, needs several essential simplifications.

The first assumption of the phreatic line is chosen from the design water level over the top of the dike along the inner slope to the polder level. When iterating to the desired phreatic line, this first estimate can not sink through the division between the top layer and the second layer under it, this because of singularities arising when a phreatic line 'encounters' a discontinuity in the permeability.

This peculiarity can be met with by calculating potentials only and adapting the geometry three or four times. A new geometry is chosen according to the places where a potential zero arises. The soil above this level doesn't take part in the actual groundwater flow.

The old sheet pile construction has to be assumed partly permeable. Crevices can e.g. assumed to be 10 cm per meter, i.e. 10%. If these crevices are filled then with very permeable sand, a good estimate for the total permeability could be found.

The program has been run for the representative sections, Tia Juana 3, Lagunillas 5 and Lagunillas 8A for the profiles as they were assumed in chapter 5. This means that calculations were not yet made for the new profiles like they had to be adapted after stability calculations. This
will give problems when actually assessing the danger for seepage and piping. The results of these MSEEP calculations are discussed in the next section.
7.2 RESULTS OF MSEEP

The limitations of the program are already lined out in the above section, and the simplified profiles have already been incorporated in the assumptions that were made in chapter 5. At first instance the sheet pile construction was chosen to be completely permeable, which gives an unfavourable picture of the situation. Like in the present situation of the dikes, where piezometer readings showed groundwater levels that were sufficiently low, not too many problems can be expected in the future situation, where a very wide dike profile will have to be chosen for stability only. Output data are collected in appendix 7.

**Tia Juana 3**

Obviously, there is still an old road incorporated in the dike body, considering the sand layer 4 of figure 5.1a. This layer is very permeable and bears water without much loss of water head. It drives the phreatic level in the dike body upwards, which is clearly seen on the output drawings of appendix 7.1. Beyond the end of the old road the water head decreases rapidly, as in a normal situation could be expected [see table 7.1].

**Lagunillas 5**

Also in this layer the old road foundation is present, but the situation is essentially more favourable, since the lake water does not have direct access to this sand layer, it is closed off by part of the clay dike body [see figure 5.2a]. Still it is clear that the sand layer influences the phreatic level in the dike body in upward direction [see table 7.1].

**Lagunillas BA**

This section is not very much affected by the old road, but more by the fine sand layer with high permeability, that is susceptible to liquefaction. The water stresses in this layer remain high, which has an unfavourable effect on the danger for piping and on seepage. The sheet pile construction that locks the outer toe must be in relatively good condition to draw the water stresses rapidly down [see table 7.1].

-96-
8.0 STABILITY OF SLOPES

A very important aspect in dike design is the stability of slopes. A dike slope may fail when a slope segment slides along a certain slip surface. This slip surface may be at shallow depth, usually under strong influence of seeping or overtopping water as well as under wave attack. Deep slip surfaces, i.e. considerably deeper than the slope revetment, occur when frictional forces in the soil, sometimes in combination with passive earth pressure, are no longer able to resist the destabilizing active soil pressures [see figure 8.1].

8.1 DEEP SLIP SURFACES

8.1.1 General theory

shape of slip surface

When a dike slope fails, the rupture surface can't be infinitely long along the axis of the dike. The surface does have a certain length or may just have the shape of a bowl [see figure 8.2]. Still, the dimensions of the slip surface along the axis are usually much greater than the height or width of the dike. At the ends of the slip surface there are, difficult to determine, resisting forces, that have relatively little influence on the stability factor. For these two reasons stability is commonly looked at in two dimensions.

If the soil would be just cohesionless the slip surface would be at some depth, parallel to the slope. Cohesive soils however, have more unpredictable slip surfaces. Considering a certain slip surface with respect to stability, a slightly different shape of slip surface doesn't affect the stability factor very much. One may therefore choose a likely, convenient shape as long as it doesn't differ too much from a possible true slip surface. Experience showed that the assumption of a circular shape is sufficient for this purpose [see figure 8.3].

stability factor

The circular slip surface makes it easily possible to consider the momentum equilibrium in order to determine the safety of the slope with respect to stability. The safety or rather stability factor is then:
8 stability

![Image](image.png)

Theoretically a value of $F = 1$ should guarantee stability. In practice, where nature doesn't follow the circles determined by engineers, a safety factor $F = 1.3$ or $1.4$ is considered to be sufficient.

In fact the actually resisting momentum is always equal to the driving momentum as no more friction force will be developed than to maintain equilibrium. The maximum resisting momentum depends on the maximum friction force that the soil can possibly mobilise. In this way the stability factor can also be defined as the ratio between the maximum possible friction and the mobilised friction:

$$F = \frac{\tau_r}{\tau_s}$$

factors determining stability

The driving momentum in the evaluation of slope stability is established by gravitational forces. The weight vector of the possibly sliding dike segment has a lever arm about the centre of the assumed slip circle (see figure 8.3). Horizontal, earthquake induced, accelerations may increase this driving momentum. This shows that, given certain site conditions, the stability of a dike slope depends on:

1. dike geometry (height and slope inclination)
2. soil weight
3. external loads

The resisting momentum is first of all established by frictional forces in the soil. Frictional forces along the slip circle are determined by Mohr's circles with maximum:

$$\tau_r = c' + (\sigma - u)\tan\phi$$

This is in the drained or consolidated situation, i.e. not shortly after the construction when there are still excess pore water pressures. The longer the cord $L$ of the slip circle, the greater friction force can be developed. A second contribution to this resisting momentum is possible passive earth pressure at the toe of the dike. This passive earth pressure is usually included in the active part of the calculation where it yields a negative driving momentum. Then the second way of defining the stability factor $F = \tau_r/\tau_s$ is applicable.

Summarized, further stability factors are:
8 stability

4. soil cohesion
5. soil friction angle
6. effective soil pressure

8.1.2 Slice methods for the evaluation of slope stability
Consider a possible circular slip surface ABCD with centre O as given in figure 8.4a. The soil above this surface may be divided into a number slices of equal width b and different height h. Advantage of this division into slices is that for one slice the slip surface doesn't intersect with more than one soil layer, second, the circle can for one slice be assumed as a straight line (the width be chosen sufficiently small), and third, the centre of gravity is easily determined. All forces acting on one slice (without earthquake) are shown in figure 8.4b. Since the stability factor \( F = \tau_s / \tau_a \) is taken to be the same for each slice, forces must be transferred from slice to slice. The boundaries between the slices are no rupture surfaces. Hence these forces can't be determined by the assumption of maximum possible force and the equilibrium of each slice is a statically indeterminate problem. Assumptions shall be made for these inter-slice forces. Requiring momentum equilibrium about O, the driving momentum of the weight (including the negative earth pressure at the toe of the dike) should equal the resisting momentum of the mobilised shear forces:

\[
\Sigma \tau_s L_i R = \Sigma W_i R \sin \alpha
\]  

(Every slice has its angle \( \alpha \) to the vertical)

The maximum is:

\[
\Sigma \tau_s L_i R = \Sigma W_i R \sin \alpha
\]

\[= \Sigma [c' + (\sigma - u) \tan \phi'] L_i = \Sigma W_i R \sin \alpha \]  

The stability factor will be then:

\[
F = \frac{c'L + \Sigma N' \tan \phi'}{\Sigma W \sin \alpha}
\]

Different 'slice methods' make different assumptions for the inter-slice forces. The next paragraphs describe the Swedish or Fellenius' method and Bishop's simplified method.
Fellenius' method
The equilibrium of one slice was a statically indeterminate problem. Not knowing the horizontal and vertical inter-slice forces $H_n$ and $V_n$, it is impossible to draw the polygon of forces for each slice.

Considering the equilibrium of the whole sliding soil mass the sum of the forces equals zero:

\[ \Sigma (H_n - H_{n+1}) = \Sigma (V_n - V_{n+1}) = 0 \quad (5) \]

Fellenius assumed all these resultant forces to be equal to zero. This assumption shouldn't affect the computed value of the safety factor too much, provided that $F$ is computed for the equilibrium of the whole sliding mass and not for each slice separately. Now all unknown forces can be determined by closing the force triangles for each slice and the safety factor $F$ be computed by formula (4). [see figure 8.5]

Bishop's method
In his article in Géotechnique vol. 5 (1955) Alan W. Bishop presumes that the resultant of the inter-slice forces as well as the weight force, the normal force and the shear force pass through the centre of the low side of the slice. Thus the momentum equilibrium of each slice is assured. At the same time he assumes the resultant of the inter-slice forces to be horizontal, i.e. $V_n - V_{n+1} = 0$. [see figure 8.6]

Thus, the inter-slice forces don't affect the vertical equilibrium and the normal forces as well as the shear forces ($\tau = c' + \sigma' \tan \phi'$) can be determined from this vertical equilibrium [see figure 8.6].

Bishop considers in this way both the momentum and the vertical equilibrium, but neglects the horizontal forces, where Fellenius considers the momentum equilibrium only.

Since the resultant of the inter-slice forces is horizontal, it follows from the vertical equilibrium that:

\[ \gamma h = \sigma' + p + (\tau_r/F) \tan \alpha \quad (6) \]

Knowing that $\tau_r = c' + \sigma' \tan \phi'$

\[ \tau_r = (\gamma h - p - \sigma') * F/\tan \alpha = c' + \sigma' \tan \phi' \quad (7) \]

\[ \gamma h - p - \sigma' = (c'/F) \tan \alpha + (\sigma'/F) \tan \alpha \tan \phi' \quad (8) \]

\[ \gamma h - p - (c'/F) \tan \alpha = \sigma' (1 + \tan \alpha \tan \phi'/F) \quad (9) \]
Substitution into the general equation for the stability factor

\[ F = \frac{\sum \left\{ \frac{c' + \sigma'\tan\phi}{\cos\alpha} \right\}}{\sum \left\{ \gamma h \sin\alpha \right\}} \]  

(10)

yields:

\[ F = \frac{\sum \left\{ \frac{c' + (\gamma h - p)\tan\phi'}{\cos\alpha \left[ 1 + \tan\alpha \tan\phi'/F \right]} \right\}}{\sum \left\{ \gamma h \sin\alpha \right\}} \]  

(11)

To find the stability factor \( F \) few iterations have to be made since \( F \) also appears on the right side in the formula. Convergence is rapid when the first guess for \( F = 1 \).

Partially submerged slopes

The total disturbing moment of the soil above the slip circle must, in the case of a submerged slope be reduced by the moment about the circle centre \( O \) of the water pressure acting on the slope (see figure 8.6). Considering the moment about \( O \) in this figure, of the mass of water NDLM and of the mass of soil above ABCD, the resultant is equal to the moment about \( O \) of the bulk weight of the soil above MN and the submerged weight of the soil below this line.

Bishop's modified method

For \( F = 1 \) the resisting momentum may become infinitely large when the angle \( \alpha = \phi - 1/2 \pi \), the factor \( \tan\alpha \tan\phi'/F = -1 \) then. This may especially occur near the toe of the dike and therefore the angle \( \alpha \) should be cut off to a maximum of \( 0.5 \phi - 1/4 \pi \). Usually this cut-off value is not reached.

Required stability factors for slice methods

A stability factor \( F = 1 \) implies a safe dike slope. This is true under the condition that the data entered into the calculations are accurate and the method of calculation reliable. To this may be added that the importance of the slope may require a safer feeling that must be expressed in a stronger requirement for the stability factor.

All these uncertainties must be expressed in factors \( \gamma \), that decrease the determined strengths of soil and increase the driving momentum. Table 8.1 lists the strength reduction factors for soils. An average value \( \gamma_s = 1.2 \) may be taken. The uncertainty in the calculation model is such that it doesn't affect the required stability factor, hence the \( \gamma_s \).
Failure of the inner slope causes more direct danger for immediate total failure of the coastal defence. This importance factor $\gamma_i = 1.1$ for the inner slope and 1.0 for the outer slope.

All these factors can be gathered in an overall increase of the stability factor $F = 1$. For the inner slope, it is thus required a bigger stability factor $F \approx 1.3$ and for the outer slope $F \approx 1.2$. 

8 stability
8.2 SLICE METHODS APPLIED ON THE DIKES OF THE BOLIVAR COAST

8.2.1 Procedure
Professor Verruijt of the 'Geotechnics section' of the Faculty of Civil Engineering (Delft University of Technology) developed a computer program (STABIL version 2.3) for the determination of the stability factor $F$, using Fellenius' and Bishop's methods. This computer program was applied to the chosen representative sections of the Bolivar Coast Dikes. The program can not incorporate horizontal mass forces, that are necessary to assess the stability factor in case of earthquakes: the pseudo static determination of the stability factor must be carried out by hand. The availability of the computer programme saves however the determination of the most dangerous slip circle, which would have required a time consuming repetition of many similar calculations. The procedure that is followed now is:

1. determination of most dangerous slip circle by STABIL VERSION 2.3 for the inner and outer slopes of the sections Tía Juana 3, Lagunillas 5 and 8A (static, without earthquake; with and without liquefaction in Lagunillas 8A)
2. recalculation of the safe dike profile including horizontal mass force by hand (pseudo-static, with earthquake and possible liquefaction)
3. adaptation of the dike profile

In principle a pseudo-static-safe dike profile is searched, it may however require a very very wide berm to achieve this desired safety, with of course high costs and high requirement for space. Maybe it has to be reconsidered if it is really necessary to put this strong demand to the design of the dike, as pseudo-static calculations can not always be considered to be realistic.

Bishop's modified method is adopted as the best method with sufficiently economical and safe results.

8.2.2 Results of static calculations
In static conditions all slopes of all sections appear to be safe, except the chosen inner slope of the section 8A in Lagunillas [see figure 8.9d] and an inner berm is applied there to achieve a statically safe stability factor of $F = 1.91$.

Table 8.2 and figures 8.7 to 8.9 show the by STABIL 2.3 determined centres of the most dangerous slip circles and
the corresponding stability factors. Appendix 8 shows the calculations by hand.

8.2.3 Results of the pseudo-static calculations
In general the chosen profiles for the representative dike sections appear to be unsafe, the pseudo-static stability factors are all smaller than unit. This means that all slopes have to be adapted to achieve the required pseudo-statically-safe dike profile. As expected it is not always feasible to really achieve this safety. The section 8A in Lagunillas, where liquefaction may occur during an earthquake, needs special measurements.

Tía Juana 3, outer slope
A stability factor of 0.90 made clear that the rip-rap layer with its 1:3 slope was chosen too steep. There are two ways in which the slope may be adapted. First an adaptation in the shape of an underwater berm requires least material as all the vertical mass forces of this material (weight) increase the resisting momentum [see figure 8.7a]. Second the slope may be made smoother eg. 1:4, which requires more material and a smaller increase of the stability factor since part of the applied material increases the driving momentum [see figure 8.7a]. Both these ways of adaptation influence the wave runup, the first possibility in the sense that there is wave height reduction or berm reduction, the second in the sense that there is extra slope reduction.

Tía Juana 3, inner slope
This slope had insufficient stability like it was assumed in chapter 5. The inner berm needed prolongation to achieve a stability factor of 1.30. [see figure 8.7b]

Lagunillas 5, outer slope
A considerable application of rip-rap has to guarantee stability, with a berm at lake level. This berm reduces the wave runup and an optimization process will yield a lower crest height, with favourable influence on the stability. [see figure 8.8a]

Lagunillas 5, inner slope
The inner slope needs a very wide berm to sustain the dike in the deepest section of Lagunillas [see figure 8.8b]. A stability factor of 1.36 is reached.
Lagunillas 8A, outer slope
This slope is statically stable, but when earthquake with liquefaction occurs, this slope is very very dangerous. The sand layer at 8.50 to 10.00 m depth contributes for 75% to the static stability. Exactly this layer is subject to liquefaction when the driving momentum is increased by round 80%, leaving a poor stability factor of 0.30. A solution that is very material consuming is the application of a wide berm of rip-rap [see figure 8.9c] This berm brings however no safety yet if the sand layer there under liquefies indeed during the design earthquake. Before the application of the wide berm the susceptible layer has to be compacted by e.g. vibroflotation. Compaction of deep soil layers under the present dike body itself doesn't seem feasible, hence the necessity for this wide berm.

Lagunillas 8A, inner slope
Like for the outer slope the sand layer with liquefaction potential poses difficult problems for the inner slope. The application of a wide, high berm is by far insufficient to reach a stable slope under earthquake and liquefaction conditions. This layer with liquefaction potential can hardly be accepted as it is. Before new dike raising this sand layer has to be improved so that liquefaction can't occur. Presuming then an angle of friction of 30 degrees the wide berm offers sufficient resistance to instability under earthquake conditions. [see figure 8.9e]

Note: the inner slopes need further adaptation than calculated above. The inner slopes have been calculated as if they were submerged. Table 12.1 gives a sample calculation for the inner slope of Tia Juana section 3, where the berm is raised one meter extra. The other sections must be recalculated in a similar way.

The conclusion from these stability calculations for section 8A is that the sand layer with liquefaction potential can't be accepted like it is, unless the option of a solution with sand beaches would be taken, where slopes are so smooth that deformation due to liquefaction of a foundation layer doesn't cause failure of the coastal defence.
Chapter 9 deals with possible solutions for this problem.
9 liquefaction

9.0 LIQUEFACTION

9.1 FAILURE BY LIQUEFACTION

Having a closer look at liquefaction as a secondary failure criterion, it is seen that it can lead to direct failure of the dike slope along a deep slip circle. It can also cause big settlements with consecutive overtopping. In a normal static situation, the most dangerous slip circle won't intersect with the sand layer. Unliquefied, the sand layer would contribute 70 to 80% to the total resisting momentum. Build up of shear forces along the possible slip circle that does intersect with the susceptible sand layer and of passive earth pressure. In case of an earthquake with liquefaction, all this strength of this sand layer is lost, while the driving momentum has increased considerably (possibly 70%) by the horizontal mass forces of the earthquake.

Settlements of importance occur, when the liquefied sand would find a way out under influence of the pressure of the dike body, so that the freeboard of the dike would be lost. [see figure 9.1, 9.2 and section 4.2]

If liquefaction doesn't lead to dike failure immediately, its high residual pore water pressures may still result into sand boils (backward erosion, piping) and excess seepage [see figure 3.105].

The earthquake induced liquefaction of sand or silty sand layers under the Bolívar Coast Dikes is a very serious danger, that must not be underestimated. The evaluation of the liquefaction potential is with the present state of art still an incredibly difficult task. Appendix 1 describes the phenomenon and an additional study [see lit. 26] selects one method for the evaluation: the CPT-procedure, adapted for non-level ground conditions.

CPT-procedure

This CPT-procedure is by no means a theoretically well founded approach. It's basically the same as the procedure for SPT, and based on the large set of historical SPT-data correlated with the occurrence of liquefaction during and after earthquakes. Well known relationships between SPT- and CPT-data enable the use of the large number of the more reliable CPT-tests made in the project area of the Bolívar Coast Dikes. The main advantage of this procedure is the correlation between liquefaction potential and reliable in-situ soil data.

-106-
9 liquefaction

It is so, that instead of requiring characteristics of the soil that really influence the resistance to liquefaction, a CPT-strength is required, representing mainly the relative density of the soil.

The stability calculations of chapter 8 for section Lagunillas 8A already gives an indication of the way that must be taken to solve the liquefaction problem. It doesn't seem very likely that there is a good method that would improve the susceptible layer under the present dike. The way that is first looked for is the sustension of the present dike by stable wide berms on improved soil.

The objective of this chapter is to produce a dike profile, possibly with additional measurements that prevents liquefaction of the suspect layer or leaves a controllable situation if liquefaction would still occur. This means that the principle of the solution for Lagunillas 8A may be too expensive, i.e. material and space consuming.
9.2 SECTION LAGUNILLAS 8A, SEED'S APPROACH

Figure 9.3, taken from lit. 26, shows the requirements for maximum dike slope angles \( \alpha \) in seismic areas. Under level ground conditions it is sufficient to remain under the continuous line, a dike however, introduces extra shear stresses in the soil. The safe \( \tau/\sigma'_o \)-ratio must then be under the dotted line, a reduction with \( 2 \tan \alpha \), which is a requirement that is difficult to fulfill.

Suspect layers are those sand or silty sand layers showing low cone penetration resistances. Examination of the CPT-diagrams in figures 1.135 reveals only section 8A of the Lagunillas polder. At 6.00 MLLL is an apparently loosely packed sand layer with cone penetration resistances far below values normal for sand. The possibility of this layer being loosely packed is sustained by its relatively steep grain size distribution; especially fine sands, possibly mixed with silt, like in this case, are susceptible to liquefaction [see figure 9.4]. Table 9.1 shows more data of this layer. Applying the procedure as suggested in lit. 26, yields the following (\( r_d \) is a depth factor, 1 at ground surface and 0.9 at 10 m depth [see lit.26 and figure 9.4]:

1984.

\[
\text{expected } (\tau/\sigma'_o)_{eq}: \tau = 0.65 \times \gamma h / g \times a_{eq} \times r_d \quad (1)
\]

\[
= 0.65 \times \frac{16 \times 3 + 20 \times 6}{g} \times 0.18g \times 0.92
\]

\[
= 18.1 \text{ kN/m}^2
\]

\( \sigma'_o = 16 \times 3 + (20-10) \times 6 = 108 \text{ kN/m}^2 \)

\( \tau/\sigma'_o = 0.17 \)

2030.

\[
\text{expected } (\tau/\sigma'_o)_{eq}: \tau = 0.65 \times \frac{16 \times 3 + 20 \times 8.5}{g} \times 0.18g \times 0.88
\]

\[
= 22.4 \text{ kN/m}^2
\]
9 liquefaction

\[
\sigma'_0 = 16 \times 3 + (20-10) \times 8.5 = 133 \text{ kN/m}^2
\]

\[
\tau/\sigma'_0 = 0.17
\]

The 'strength' of the suspect soil layer or its resistance to liquefaction is: \( \text{CPT} = 9 \text{ MN/m}^2 = 0.18 = (\tau/\sigma'_0) \). This means that under level ground conditions, this relatively weak sand layer would withstand an \( M = 7.5 \) earthquake with accelerations of 0.18g. However, we have to deal with a dike here, there are no level ground conditions. A dike slope yields shear stresses in the soil under. The magnitude of these stresses is \((\tau/\sigma'_0)_{\text{slope}} = 2 \tan \alpha\), where \( \alpha \) is the slope angle.

\[
(\tau/\sigma'_0)_{\text{slope}} < (\tau/\sigma'_0) - (\tau/\sigma'_0)_{\text{slope}}
\]

\[
(\tau/\sigma'_0)_{\text{slope}} < (\tau/\sigma'_0) - 2 \tan \alpha
\]

This is Seed's concept of the resistance to earthquake induced liquefaction under level ground conditions, adapted for an overlying slope (\( \alpha \)). It is clearly seen that the slope's shear stress consumes part of the soil's strength, of its resistance to liquefaction.

In the actual case of Lagunillas 8A:

\[
0.17 < 0.18 - 2 \tan \alpha
\]

This implies a maximum slope of 1:200!

Calculating it this way it is always necessary to construct beach slopes instead of dikes, just for the sake of preventing liquefaction in this weak subsoil layer. Commonly used 1:3 slopes would in this concept, consume \( 2 \tan(1/3) = 0.7 \), which is equivalent to far more than the maximum necessary 0.35 [see figure 9.3]. This is in the range where the assumed concept doesn't apply any more. It is anyway obvious that if an \( M = 7.5 \) earthquake did occur, there would be liquefaction in section Lagunillas 8A.

It is obvious that with earthquakes of such strength as occurring near the project area of the Bolivar Coast Dikes and so weak sand layers, the possibility to solve the danger for liquefaction in this subsoil layer with acceptably smoother slopes is not feasible.
9.3 POSSIBLE SOLUTIONS

The first section of this chapter already indicated the direction in which this liquefaction problem needed to be solved, regarding the information from the stability calculations of chapter 8. In order not to exclude possible solutions, a more general procedure must be followed to handle the problem. The way of tackling this problem is as follows:

1. accepting the weak layer as it is
   a. beaches
   b. gentler slopes
   c. wide, stable, sustaining berms
   d. acceptable deformation
   e. second line dike

2. not accepting the layer as it is
   a. compaction, densification
   b. displacement compaction
   c. injection
   d. locking and drainage
   e. complete removal of the sand layer

3. combinations of possibilities mentioned above

9.3.1 Solutions accepting the sand layer as it is

a. beaches

Natural beaches are not present, so if the solution of beaches would be opted for, they would have to be applied artificially. This beach solution can still be based on the approach of Seed as described above: if the slope of the dike is chosen sufficiently gentle, no liquefaction will take place. But, there are more reasons why this solution is well applicable. A beach creates a sort of dynamic equilibrium of the coastal defence. In the normal situation there is erosion and accretion of this defence structure, i.e. the beach. This characteristic is in fact its strength, also in the case of an earthquake with liquefaction. A beach functions as coastal defence, provided that it is sufficiently wide: temporary wave attack during storms doesn't erode the whole beach and is followed by a period of beach accretion. The dynamic equilibrium is maintained by the continuous stirring up of bottom (beach) material, horizontal displacement of these grains and sedimentation of these particles once again (accretion) and the same
procedure backwards (erosion) [see figure 9.6]. This figure doesn't show the enormous sand beach erosion, that can be caused by storms, when great masses of sand can be swept away by high storm waves. This practice counts for the component perpendicular to the coast.

In an analogue situation of erosion or partial failure due to liquefaction, the beach will regain average strength after consequent accretion.

In this dynamic equilibrium there is also the disadvantageous longitudinal component, i.e. the sand transport component parallel to the coast. This means that there is beach erosion at one place of the coast, with particle sedimentation further along the shore. This implies a nett longitudinal sand transport; somehow there must be sand suppletion at one end of the beach and sedimentation (with possible navigation problems?) at the other end. This makes the beach solution uneconomical for beaches of short length. If only the 500 m long Lagunillas 8A section would require an artificial beach, another solution might be advisable.

Another important factor in the strength of the gentle beach slopes as coastal defence, compared to the relatively steep dike slopes, is their stability with respect to deep slip circles. Beach slopes are in the order of 1:50, near to horizontal, which gives in the static situation a stability factor of $F=1$, not even considering the cohesional forces along the slip surface yet. These shear forces then, ensure the stability in case of earthquake easily.

The enormous amount of required sand fill for this solution makes the costs very much dependent on the occurrence of sand pits near the project area.

b. gentler slopes
At first instance it seemed possible to design the dike with smoother slopes, say 1:6, to have at least the factor of stability $F>1$, for deep slip circles under the circumstance of an $M=7.5$ earthquake with liquefaction of relatively shallow sand layers. The calculations of section 9.1 however, block this solution.

Also the stability calculations in chapter 8 show incontrovertibly that gentler slopes alone don't yield stability factors of $F=1.3$, unless slopes in the order of beach slopes would be constructed [see table 8.2].
c. wide, stable, sustaining berms
Like for the construction of gentler slopes, the application of berms only does not bring sufficient stability either, as described with the same calculations mentioned in the section above.

d. acceptable deformation
If the simple solutions of gentler slopes and the application of wider berms doesn't bring the solution it may be worthwhile to investigate the damage that the slide of a slope along a deep surface would cause.

The importance factors, included in the required stability factor $F$ for the inner and outer slopes, indicate already that failure of the inner slope is much more dangerous then failure of the outer slope. Failure of the outer slope leaves at first instance some time for at least temporary measurements to prevent total failure of the coastal defence. Outer slope failure occurs in normal cases (i.e without earthquake and liquefaction) usually after drop of high water level of some duration. In this case the relatively sudden drop can be replaced by the sudden horizontal earthquake forces, causing dike slope failure.

In addition to this, figure 8c shows that the most dangerous slide still leaves the dike crest standing, only the rip-rap slides down.

Failure of the inner slope on the other hand leads to rapid failure of the dike. The hydraulic gradient over the remaining part of the dike body cause increases considerably due to the failure and causes rapid erosion of the soil with consequent total failure of the coastal defence. Figure 8e shows that the most dangerous slip circle of the inner slope almost intersects with the outer slope: failure of the inner slope is unacceptable.

Liquefaction can cause several ways of deformation of which the most important are the slope slides along deep slip circles as described above. In an extreme case the failure of the outer slope could possibly be accepted, though prevention of failure always remains preferable and probably as well possible as the prevention of inner slope failure, which can't be accepted under any circumstance.

If the question of acceptable deformation is raised, it is necessary to further investigate how the appearance of the dike body will be after deformation.

Figure 9.7 shows characteristics of importance concerning the deformations, that result from dike slope failure. In
general it is of course so that the dike slope doesn't fail exactly according to the calculated slip circle. Dikes don't know what engineers calculate for them. With some phantasy however, it is possible to imagine a circular slip surface [see figure 9.7a].

The soil above the slip surface doesn't behave as a solid coherent body, but gets loosely displaced down the slope. An investigation into many slides of sand slopes in the delta area of the southwest Netherlands, brought forward the characteristics of figure 9.7b and c. Figure 9.7b shows the initial conditions for a sand slope slide, which is of little importance for the dikes. For Lagunillas 8A the initial conditions are sufficiently lined out in chapter 8. Of more interest is the situation after slope deformation [see figure 9.7c]. Boundary condition in the establishment of the dike slope after deformation is, that the disappeared volume of soil is equal to the displaced volume at the toe of the dike.

The most important characteristics for the safety of the dikes is the length of the damage behind the crest line and the absolute steepness of the highest (steeper) part of the new slope, that can be composed of the other measures given there.

e. second line dike

A very expensive possibility is to create a complete new dike behind the existing one as a second defence line. Before construction, soil improvement is possible, supposing that the susceptible loosely packed fine sand layer extends till far behind the coast line.

9.3.2. Solutions not accepting the sand layer as it is

a. compaction, densification

In geotechnical engineering volume stability and soil strength is one of the major tasks, met in many different circumstances. Many ways of soil improvement have been developed already, depending on the type of soil (cohesive, noncohesive) or loading (structure), soil properties (strength, density, permeability) and circumstances like available time, materials and apparatus, experience and of course on costs. Table 9.1 lists a number of well known methods for the improvement of soils. Some methods for the improvement of noncohesive soils, as necessary for Lagunillas' section 8A will be discussed shortly.
Blasting
Shock waves and vibrations from imposed explosions cause liquefaction with consequent compaction. Achieved relative densities of 70 to 80% have been proved. This method requires boreholes for the application of the explosives at every 3 to 7.5m and is economical for small areas only. The success that can be achieved by the use of this method raises the idea that when earthquakes occur with cyclic liquefaction [see lit. 26] and consequent compaction, the situation for Lagunillas 8A would improve considerably [see section 9.4].

Terraprobe
It was found that vibratory pile driving led to a higher degree of densification in granular soils. This terraprobe method uses an open end pipe of 75 cm diameter, that is driven into the ground with vibrations of 10-25 mm amplitude and 15 Hz; no water jets are used.

Vibroflotation
This is one of the oldest methods (thirties of this century) and is still one of the most effective. A torpedo shaped vibration generator of 2 m length and 40 cm diameter (newer methods also use a bundle of plates in *-configuration) is being driven into the ground with water, jetted from bottom and top of the probe. The vibration creates a cavity around the probe, which is continuously filled with granular material from the surface. The drive holes are spaced two to four meters apart. In fact this method can also be classified under the displacement compaction.

It is obvious that both the terraprobe method (though to a lesser extent) and vibroflotation cause unacceptable damage to the dike body when layers under the dike body itself would be treated. This damage is not at all easy to restore.

Dynamic consolidation, heavy tamping
This method uses heavy pounders of 5 to 40 tonnes; it involves impacting of these pounders from heights of 6 to 30m, according to a predetermined pattern. Use of this method near existing structures must be dissuaded.

B. Displacement compaction
Densification is basically simple displacement of the soil, which happens e.g. when piles are driven into the ground. This reinforcement around piles is used for the foundation of structures, but in reverse piles can also be used to
reinforce the ground.

compaction piles
A tube closed by e.g. a plug of gravel is driven into the ground till the layer that is to be compacted. During the driving, the soil around the tube is already being compacted. Then the plug of gravel is rammed from the tube down into the soil to form a strong base there and consequently the tube is being raised incrementally, meanwhile driving gravel into the tube as to expand the shaft of the compacted soil column around. The amount of the gravel input allows computation of the achieved displacement. Instead of gravel, other backfills like stone or sand may be used [see figure 9.8a and b]. This solution may be an option, but more must be known about the realised degrees of compaction for larger areas, about the effect on the dike body and consequent costs.

displacement grouting
Grouting under high pressure can achieve sufficient displacement for adequate soil compression. It can however only be used for small areas or places with cavities.

c. injection
Only very pure sands or silty sands are subject to liquefaction danger. Already small contents of cohesive soils improve the resistance to liquefaction so much that danger is negligible. Hence injection of the susceptible sand layer could be worthwhile to have a closer look at.

cement grouting, clay grouting
This is not particularly useful in the case of the fine sand of Lagunillas 8A. Cement and clay are still able to penetrate sufficiently deep coarser soil, like medium to coarse sand and gravel. Finer sand must be treated differently.

chemical grouting
Solutions of chemicals can penetrate further into finer sands and even medium silts to form their a gel or solid precipitate. The method is however very expensive (at least 3 to 8 times the costs of vibroflotation) and hence only economical for very small areas.

penetration of bitumen
Very little is known about penetration of bitumen into small sands. For road engineering much is known about it, of
course, but the smallest grain sizes investigated are 0.2 - 0.4 mm, where all particles of the sand layer under Lagunillas 8A are smaller than 0.2 mm. If an appropriate bitumen could be found, that would penetrate far enough between the fine sand sand particles its cohesive forces would increase the resistance to liquefaction efficaciously.

d. locking and drainage
The presence of a sheet pile construction, that locks the outer toe of the dike and that also locks the susceptible sand layer gave rise to the idea to lock the layer at the other side, too.

These two sheet pile constructions don't add stiffness of any importance to the liquefiable soil layer, especially not the very old construction at the lake side. The idea is to create a closed area, in which the level of water pressures can be brought down by permanent drainage. The sheret pile construction is not necessary as such, some kind of grouting, injection or other kind of screen, that is impermeable to water is probably cheaper.

It is so, that a lower water pressure implies a higher effective soil pressure. In case of an earthquake it was possible that the cumulative residual water pressure rises would bring the effective stress down to zero. A lower level of the water pressure or higher effective stress needs longer, stronger earthquakes i.e. more cycles with pore water pressure rise to reach the point of liquefaction. Either the point of liquefaction won't be reached, or it will be reached so late during the earthquake (so short before the end of the earthquake), that during the impact time of its movement of a certain soil mass can not possibly be developed. Quantification of this theory must be done with other methods as shortly described in the additional liquefaction study [see lit. 25,26].

Stronger drainage however doesn't require more fundamental investigation into liquefaction. If the sand layer will be drained so that it won't be saturated, the water pressure won't rise when the soil tends to decrease in volume, as the water particles have sufficient space for displacement. In the final stationary situation the water table in the dike body will be drawn down and the amount of drainage water can roughly be calculated from the feeding from the lake through the dike body and from a lower water bearing sand layer [see figure 9.9].

For the execution of applying a seepage screen and a drainage system very efficient methods are available, provided that the construction depth is not too far below ground level. With one caterpillar tracked vehicle both the
vertical polyethylene seepage screen and a horizontal filter pipe can be applied till five meters depth, which is sufficient for Lagunillas 8A. The horizontal drainage pipe will have to be applied along the seepage screen at its lake side, so that it can drain the sand layer under the dike body. The total length of the screen and of the drainage pipe are ± 500 m, i.e. the length of the dike section. At certain intervals a pump must pump the water into the drainage ditch. [see figure 9.10 and lit. 28]

e. complete removal of the sand layer
The possibility of complete removal of the sand layer doesn't seem very sensible, especially not under the dike body.

9.3.3 Solutions with combinations of the methods above
The stability calculations of chapter 8 already suggested that the susceptible sand layer would have to be accepted as it is, at least under the existing dike body. A stable situation can than be reached by application of wide, sustaining berms, provided that the sand layer outside the area of the existing dike body, be improved. The cheapest method for this improvement then is the terraprobe method or vibroflotation.
9 liquefaction

9.4 FAVOURABLE EFFECTS ON THE DANGER FOR LIQUEFACTION

Oil production and subsidence continue, which has an unfavourable effect on the resistance against liquefaction potential, because of the increased level of water pressures in the layer. A higher dike however, imposes higher initial effective soil stresses in the layer and so more initial shearing resistance.

Another phenomenon that may influence the resistance to liquefaction favourably, is the occurrence of earthquakes. Vibrations, provided that they are strong enough, cause movements of the loosely packed sand layer, that tends to decrease in volume. So earthquakes with a smaller magnitude than Richter 7.5, that occur with higher frequency, can have a favourable effect on the stability of the dikes. The quantification of all these effects together remains a very difficult task.
9.5 CHOICE OF SOLUTIONS

From the above mentioned possibilities for a stable solution, table 9.2 selects four, of which two need further investigation before they could be applied. These last two will be discussed first.

The solution of locking the susceptible layer under the dike in combination with permanent drainage, is based on the assumption that the water pressures under the dike can be kept so low that, in case of an earthquake, no liquefaction would occur. Further investigation into liquefaction with help of direct methods is necessary. Advantage of this solution should be that the total dike width could be confined to few tens of meters less than suggested in the solution with wide berms. So, the probably very high costs of a seepage screen in combination with permanent drainage, might be compensated by saving soil material and clearance of the extra area that is needed behind the existing dike.

The possible solution of compaction piles with sand, gravel or stone backfill must also be further investigated, as described in section 9.3.2b.

The beach solution is very safe. The costs depend however very much on the availability of sand fill, of which occurrence in sufficient quantities in the neighborhood has not been proved yet. In this study the danger for liquefaction has only been shown for one 500 m long dike section, which makes this beach solution hardly feasible, considering the longitudinal nett sand transport. The solution with wide berms as suggested below seems cheaper then.

The solution with wide berms and a partially improved sand layer gives for the present state of information the best perspective. The stability calculations of chapter 8 already pointed in that direction. Provided that the susceptible sand layer with its very steep grain size distribution (most particles have the same diameter or at least nearly the same, see figure 9.4) lets itself be densified by terraprobe or vibroflotation method indeed. Wide berms ensure the stability of Lagunillas 8A with stability factors of F = 1.3 and more. The dike becomes in this solution solution very wide with a total width of 104 m, not including the drainage canal yet at some distance behind the dike.

Shortly after occurring earthquakes, that didn't induce
liquefaction of the sand layer or only cyclic liquefaction, a temporary high level of water pressures may remain in the sand layer. After earthquakes the dike needs thorough inspection on excess seepage and sand boils, that may result from these earthquakes.
10 piping

10.0 PIPING

10.1 FIRST CHECK, LANE'S APPROACH

10.1.1 Lane's principles applied on the representative sections of the Bolivar Coast Dikes

In the presence of an overlying relatively impermeable layer, it is common to use the concept Lane for the evaluation of the danger of piping, since he has the best documented investigation. For the theory and practical advises about the prevention of piping be referred to appendix 2.

Holes in this overlying layer cause unexpected short seepage lengths and therefore the shortest possible seepage lengths shall be chosen and safe minimum values for the value \( C_L \). The seepage length shall be chosen as indicated in figure 10.1a. As stated already in chapter 5 a berm must be applied for the prevention of piping. This means that the overlying impermeable layer is made so heavy that hydraulic fracturing is very unlikely at the place indicated in figure 10.1a and the point of exit must be chosen at the end of the bufferzone [see figure 10.1b].

The representative dike sections have all a relatively impermeable layer overlying a well permeable sand layer [see figures 5.1a, 5.2a and 5.3a, in all figures the layer nr. 3]. First the safety of the dikes with respect to piping must be checked with the approach of Lane, who states that if the shortest seepage length is long enough with regard to the retaining height of the dike, no problems can be expected.

\[
\frac{\sum L_w + \frac{1}{3} \sum L_m}{H} \geq C_L \tag{1}
\]

For safe values of \( C_L \), depending on the type of soil, see table 3.17c.

In Tia Juana the well permeable layer nr. 2 \( (k = 10^{-4}) \) is 200 times as permeable as the overlying clay layer nr. 3 \( (k = 5 \times 10^{-7}) \) [see figure 5.1a]. Hydraulic fracturing is not imaginary. Chosing the exit point at the end of the bufferzone, the total seepage length in the new situation is \( L_m = 78 \text{ m} \), where \( L_w \) is negligible. The water bearing sand layer consists mainly of medium sand for which Lane requires a safe \( C_L = 6.0 \).

Calculation shows that \( L_m \) needs to be 12 m longer: the berm has to be prolonged [see table 10.1].

A similar calculation for Lagunillas 5 shows that the
Profile following from the stability calculations is wide enough to prevent piping [see table 10.1]. Under section 8A in Lagunillas, hydraulic fracturing seems impossible, the impermeable top layer is rather thick, 5 m and leaves a small hydraulic gradient for the water to cause erosion in the sand layer. Applying the principle of Lane, no danger for piping should be expected. On the other hand it was seen from the water pressure calculations in chapter 7, that this sand layer was very permeable and that it was probably advisable to check the sheet pile construction very well there.

10.1.2 Recent developments concerning piping and adaptation of dike profiles

Lane's approach has been criticized very much, especially by the representatives of a second stream, that assess the danger for piping with help of groundwater flow, (flow nets and concentration points). Considering the character of erosion, the 'erosion length' or in terms of Lane, the 'length of creep', wasn't really a bad choice for a significant factor in the evaluation of piping danger [see figure 10.5].

Under structures and at layer boundaries, the real piping erosion occurs. It is there where the soil particles are taken by the groundwater flow at first, leaving water bearing crevices of say, 10 to 20 grains width, which is a real river at the scale of groundwater flow. These water bearing crevices attract water from the soil layer below. Of all soil parameters, recent (1988), not yet published calculations of Public Works (Rijkswaterstaat) and Delft Geotechnics indicate that the thickness of the water bearing layer influences the factor L/H considerably. This was already assumed before, but couldn't be well supported by theory. Thick layers tend to have a parabolic water head distribution, where this distribution tends to be linear in thin layers, i.e. thin layers can give resistance to higher water levels [see figure 10.6].

Comparison of these calculations with the cases documented by Lane, and including a safety factor of round 2.0, yields a very good compliance for infinitely thick layers. The recent calculations, as mentioned above, yield for thinner layers however, much more favourable values, i.e. much more economical values for the design of dikes. For certain soil layers, with certain soil parameters a much lower value for C can be chosen, or in better words, the factor L/H can be
considerably reduced. How much is shown in figures 10.7 with two different types of soil. A layer of e.g. two meters thickness requires a seepage length that is 50% shorter than Lane's advise to be safe with regard to piping.

Following these new calculations it is not necessary to prolong the dike profile of Tía Juana 3 and Lagunillas 5.
10 piping

10.2 SECOND CHECK, HYDRAULIC GRADIENTS

The application of the berm behind the dike does not increase the seepage length only, but its main advantage is that the most serious danger for piping is shifted to a place far behind the dike body, just beyond the buffer zone. The groundwater may concentrate at certain places, on its way from the lake to the polder. These concentrations may become dangerous when the water tries to find its way out early, upwards through the overlying layer by hydraulic fracturing of that layer. Because of this, the hydraulic gradient i/d over the overlying soil must be kept small. The berm or toe blanket increases the thickness and weight, thus decreasing the hydraulic gradient. Critical hydraulic gradients are:

- at the toe of the dike : \( i_e = 0.5 \)
- at the end of the buffer zone : \( i_e = 0.7 \)

The gradients are calculated with respect to ground level (soil completely saturated) or to the phreatic line. There is loss of stability when \( \sigma' = 0 \).[see figure 10.8]

\[
\sigma = \rho_{w}.g.d \quad (d = \text{depth below water table or ground})
\]

\[
\sigma_w = \rho_{w}.g.d + \rho_{w}.g.h
\]

\[
\sigma' = 0 \quad \text{(loss of stability)} \quad \rightarrow \quad \sigma = \sigma_w
\]

\[
\gamma_s.d = \gamma_s.d + \gamma_w.h
\]

\[
i = \frac{h}{d} = \frac{\gamma_s - \gamma_w}{\gamma_w} \quad (2)
\]

Considering a permeable water bearing layer, with overlying soil of thickness \( d \) till the groundwater level it is practical to work with elevations \( z \) [see figure 10.8]. The gradient of formula (2) becomes:

\[
i = \frac{h_a - z_{ph}}{z_{ph} - z_a} \quad (3)
\]

If piping would still occur it would be under the toe blanket and not under the vital dike body.

Table 10.2 lists the hydraulic gradients at the inner toe and end of the buffer zone, for the representative dike.
sections. Tía Juana 3 and Lagunillas 5 don't have problems to fulfill the requirements, but at the inner slope toe of Lagunillas 8A the hydraulic gradient i is bigger than the critical \( i_c \). As already mentioned in chapter 7, the sheet pile construction at the outer toe was at first instance chosen to be completely permeable. In reality this permeability is much smaller, though the construction is worn and old. Consequently the head loss under the dike must be considerably more than assumed now.

**weight**

In case of an impervious overlying layer, the upthrust \( G_u \) of the water shall not exceed the weight \( W \) of the overlying soil.

Table 10.3 lists the facts for the representative sections. No problems can be expected:

\[ G_u - W < 0 \text{ or putting safety into it:} \]

\[ W/G > 1.2 \]

The table shows safe values.
11.0 SEEPING WATER

In this chapter two phenomena concerning seeping water will be considered. Of first concern is the amount of seeping water, unfavourably influenced by the increased retaining height in the future situation; it must be sufficiently low in order not to require too many extra operation hours of the pumping stations.
Second, seeping water exerts seeping forces on soil particles, which may cause critical situations, especially there, where seeping water leaves the dike body, possibly dragging along particles of the soil.

11.1 AMOUNT OF SEEPING WATER

calculation
With help of the water level and water pressure data, found by means of the MSEEP computer program [chapter 7] and the permeabilities of the soil, it is a relatively easy assignment to determine the amount of seepage with through both the dike body and subsoil. Assumption is that the impervious layer down, doesn't transport at all, which will be clearly shown by the contribution to the total transport of the other less permeable layers.
The total transport per day or hour will related to the capacity of the pumping station that serve that particular dike section.

Seepage of water is ruled by Darcy's Law:

\[
q = k \frac{dh}{dx}
\]  

(1)

The seepage \( Q'_{\text{tot}} \) per m' dike is

\[
Q'_{\text{tot}} = \Sigma k_i.z_i \frac{dh_i}{dx_i}
\]  

(2)

where 
- \( z_i \) = layer thickness in m
- \( dx_i \) = distance between two potential lines in the layer
- \( dh_i \) = water head difference over distance \( dx_i \) in m
- \( k_i \) = permeability of the layer in m/s

For the calculations with help of the potential lines the easiest part can be chosen, assuming continuity. This offers the possibility to choose an easy to measure place near the
inner toe of the dike or under the berm (still in front of the seepage surface). Potential lines are (almost) vertical there and not so many layers have to be considered [see figure 11.1 11.2 and 11.3]

results
Measures and calculations are shown in the figures 11.1 to 11.3. It is very clear that the dike section with the thickest water bearing sand layer and besides that the most permeable, the amount of seepage is highest. All the other layers can’t be said to have influence of much importance on the total water transport.

This section Lagunillas 5 has a water transport that is a multiple of those of Tía Juana 3 and Lagunillas 8A. Lagunillas 5 is though in the deepest part of the polder and as said before, it was there where most problems could be expected.

relations to pumping stations
An overview of the pumping stations in table 11.1 shows very big capacities.

Tía Juana 3
Considering section 3 of Tía Juana, water in the deep part of this polder is led to pumping station 'San Mateo'. This station can be estimated to serve 850 m of the dike, which complies with a total seepage of 850 * 0.18 = 153 m³/h. This is 153/(5*4500) < 1% of the station's capacity.

Lagunillas 5
Especially the very big stations 'Principal' and 'LS 5' in the deepest part of Lagunillas have very big capacities. They are even supported by the smaller stations 'Bodega' and 'Estacion del Pueblo'.

The seepage, as calculated in figure 11.2, is however 24 times more than in Tía Juana and so the pumping stations need (4.4*800)/(7*4500) = 10% of their capacity to cover the seepage through the dike or in other words, one of the pumps of station 'Principal' will have to be operated permanently.

Like already stated in chapter 7, the calculation of water levels with the assumption of a completely permeable sheet pile construction at the outer toe, gives a too pessimistic view. The extra water head difference across the sheet pile construction should already yield a much lower amount of seepage.
Lagunillas 8A
The situation in Lagunillas 8A is better than in the deeply situated section 5 of the same polder. The pumping station in this section, 'LS 16' serves around 1700 m dike, i.e. a seepage of 1700 * 0.37 = 630 m³/h. The pumping capacity is three times 4500 m³, of which ≈ 5% should be used then for the seepage or one pump during 10 hours per day. Also here, the seepage has to be brought down by the sheet pile construction at the toe. This is already mentioned in chapter 10 to limit the hydraulic gradient at the inner toe to less than 0.5.
11.2 EROSION BY SEEPAGE

For the Bolivar Coast Dikes only one from the three general cases of erosion by seepage will be considered. Groundwaterflow from the dike body back into the lake is a phenomenon that is very unlikely to occur in the case of a constant water level. Only a sudden drop of this water level can cause reverse flow in the dike body, in all other cases the water head in the lake will be higher than in the dike body.

The inner slope of the Bolivar Coast Dikes is constructed of cohesional soils, so no washout of soil can be expected.

This leaves erosion of the inner slope by the component of groundwaterflow horizontal towards to the slope. Figure 11.9 shows the safe combinations for a total load safety factor of 1.2. In this value should still be incorporated a material safety factor of 1.2, i.e. that the strength of the soil must be lowered by 1.2.

\[
F_s = \frac{c + ((\rho_s - \rho_w) \cdot g \cdot d \cdot \cos \alpha - \rho_w \cdot g \cdot d) \tan \alpha}{(\rho_s - \rho_w) \cdot g \cdot d \cdot \sin \alpha + \rho_w \cdot g \cdot d}
\]

(3)

\[
\frac{F_s}{F_o} \geq 1.2 \times 1.2 = 1.44
\]

(4)

In the case of horizontal flow the gradient \( i \) can be expressed in terms of \( \alpha \) (\( i = \tan \alpha \), \( i_s = \sin \alpha \), \( i_d = \sin \alpha \)), which yields figure 11.9 for the assessment of safe slopes.

All inner slopes of the representative sections are safe. The soil used for berm and dike raising has a cohesion value \( c = 12.5 \text{ kN/m}^2 \) and an angle of friction with \( \tan \phi = 0.22 \). The upper 60 to 65 cms of the dike slope may considered be to considered to have a certain structure, due to plants (the grass), reminences of roots, wetting and drying of the soil.

The factor \( c / \rho_s \cdot g \cdot d \) becomes \((12.5/1.2)/(10 \times 0.60) \approx 1.75\) and fulfills the requirements for a safe 1:3 slope easily [see figure 11.9].
12 conclusions

12.0 CONCLUSIONS AND RECOMMENDATIONS

12.1 GENERAL CONCLUSIONS

Within the scope of this study, it has not been possible to give an exhaustive treatment of the raising of the Bolívar Coast Dikes. Three representative sections have however been chosen and dike raising, according to the recommendations below, will yield a safe dike profile. Optimization is still very well possible. The recommendations don't apply to the shallow parts of the polder, where the foreshore plays a significant role.

In general it can be said, that the sound retaining height requires the construction of firm dikes, because of stability reasons only. Mitigatory measurements against seepage and piping are hardly necessary.

The additional study into earthquake induced liquefaction [see lit. 25, 26] makes clear that this phenomenon is a difficult to evaluate, but ever threatening danger for the coastal protection of the polders. The adopted CPT-method gives only an indication of the dangerous sections and it must be advised to investigate all sand layers under the dikes minutely on relative density, grain size distribution, probable content of cohesive soils and angularity of the grains.

The alternative measurements for the sections that are subject to liquefaction danger [see chapter 9], need further investigation in order to be developed as a real, realistic alternative.
12 conclusions

12.2 RECOMMENDATIONS FOR THE REPRESENTATIVE SECTIONS

The material used to raise the dikes won't differ from the recommendations as given already in 1968 [see figure 3.133, 3.134].

The outer slope revetment will consist of the same rip-rap 300 to 800 kg, with an average weight of 500-600 kg on an asphalt mastic layer to prevent erosion of the base dike material.

The inner slope must be of the corocillo grass, that survives well in the climate.

The crest must have a proper asphalt road for regular dike inspection.

The required overheight with regard to settlement and subsidence depends on the year of construction. The overheight consists of a part settlement (already calculated) and a part subsidence till the year 2030 minus the accumulated subsidence $\delta_{ss}$ between 1984 and the year of construction [see table 3.12].

Note: as already mentioned in chapter 8, the inner slopes need further adaptations. The resisting momentum in the stability calculation did not include the negative effect of the water pressures yet as shown in an example calculation in table 12.1 [compare appendix 8, page A8.3 and A8.4]. This implies the application of a higher berm. Table 12.1 shows the stability of Tia Juana section 3, where the berm was raised an extra meter; the other representative sections must have a similar adaptation.

12.2.1 Tia Juana 3

This section protects the deepest part of the polder Tia Juana, with future polder ground level PGL = 4.50 m MLLL and lake bottom at 5.50 m MLLL.

The chosen profile is safe for horizontal accelerations of 0.22g (return period 4000 years). [see figure 12.1]:

crest level: 3.00 m MLLL, 4.00 m wide, including the subsidence $\delta_{ss,year}$ between 1984 and the year of construction

overheight : $1.00 + (1.50 - \delta_{ss,year})$ m

outer slope: 1:3 rip-rap, with sustaining berm 1:15 at 1.20 m MLLL, 18.00 m wide

inner slope: 1:3 corocillo, with berm at 0.40 m MLLL, slope 1:15, 26.00 m wide
12.2.2 Lagunillas 5

This section protects the deepest part of the polder Lagunillas with future polder ground level PGL = 5.60 MLLL and lake bottom at 7.05 MLLL. The dike is designed on a 4000 year return period earthquake with accelerations of 0.18g. [see figure 12.2]:

crest level: 3.00 MLLL, 4.00 m wide, including the subsidence $\delta_{ss}$ between 1984 and the year of construction

overheight: $2.15 + (2.60 - \delta_{ss,year})$ m

outer slope: 1:3 rip-rap, with sustaining berm at lake level, 9.00 m wide

inner slope: 1:3 corocillo, with berm at 0.00 MLLL, slope 1:15, 40.00 m wide

12.2.3 Lagunillas 8A

This section doesn’t protect a very deep part of Lagunillas, but has a loosely packed sand layer, imposing very difficult problems for dike slope stability, because of liquefaction danger. The dike profile is stable for earthquakes with return periods of 4000 years, i.e. for horizontal accelerations of 0.18*g. Future PGL = 3.50 MLLL and lake bottom level at 4.40 MLLL [see figure 12.3]:

crest level: 3.00 m MLLL, 4.00 m wide, including the subsidence $\delta_{ss}$ between 1984 and the year of construction

overheight: $2.20 + (2.50 - \delta_{ss,year})$ m

outer slope: 1:3 rip-rap, with berm at 1.40 MLLL, slope 1:10, 38.00 m wide

inner slope: 1:3 corocillo, with berm at 0.80 MLLL, slope 1:15, 32 m wide

This profile wouldn’t be safe yet, when the 1.50 m thick susceptible sand layer between 8.50 and 10.00 MLLL wouldn’t be improved. Assumption is that with the presently available methods the layer under the dike body can’t be adequately improved. Densofication of the sand outside the existing dike profile is sufficient when applying berms of the here recommended width.
12.3 OPTIMALIZATION

The necessity of the application of wide berms to guarantee the stability of the dikes, yields, when recalculating the required crest height, a lower value. The reduction of the wave runup can be considerable. Assuming a wave length of 25 m and a berm of 9.00 m width, like e.g. in the case of Lagunillas 5 the wave runup reduction is \( \frac{B}{L} = \frac{9.00}{25.00} = 36\% \), which is 0.90 m (wave runup was 2.50 m). This implies also a crest height reduction of 90 cms with consequent favourable effect on the stability, so that smaller berms could be required.
LITERATURE

1. Fundamentals of earthquake engineering
   N.M. Newmark, E. Rosenblueth
2. Evaluation and prediction of subsidence
   Surendra K. Saxena (ed.)
   New York, U.S.A., 1979
3. Earthquake resistant design
   D.J. Dowrick
   Chichester, U.K., 1977
4. Safety of dams (Flood and earthquake criteria)
   Committee on safety criteria for dams, G.W. Housner
5. Land subsidence
   Proc. of the Tokyo symposium (2 volumes)
   UNESCO
   Louvain, Belgium, 1970
6. On the cause of subsidence in oil producing areas
   W. van der Knaap, A.C. van der Vlis
   Proc. of the 7th World Petroleum Congress, (vol.3, pg.85)
   1967
7. Methoden voor golfvoorspelling (Methods for wave prediction)
   L.H. Holthuijsen
   Technische Adviescommissie voor de waterkeringen
   Netherlands, 1980
8. Considerations in the earthquake resistant design of earth
   and rockfill dams
   H.B. Seed, Géotechnique 29, no. 3 (pg. 215-263)
   Paris, France, 1979
9. Earthquake engineering and soil mechanics
   Proc. of the ASCE geotechnical engineering division specialty
   conference (3 volumes)
   Pasadena (CA), U.S.A., 1978
10. Zandmeevoerende wellen (Sand boils)
    Centrum voor onderzoek waterkeringen S-77.066
    Zierikzee, Netherlands, 1977
11. Embankment-dam engineering
    R.C. Hirschfield, S.J. Poulos (eds.), Casagrande volume
    New York, U.S.A., 1973
12. Venezuela, landendocumentatie
   A.E. van Niekerk, J.H. Stroom (KIT)
   The Hague, Netherlands, 1980

13. Canal and river levees
   P. Peter

14. Bolivar Coast Dikes, Venezuela
   Nedeco, coordinator P. Janssen
   The Hague, Netherlands, 1968

15. Bolivar Coast Dikes vol. I Tia Juana, vol. II Lagunillas
    DSML report 1957, supervisors W. van Mierlo, H. Begemann
    Delft, Netherlands, 1957

16. Dams and earthquake
    Proceedings of a conference at the Institution of Civil Engineers, London
    London, UK, 1980

17. Zandmeevoerende wellen, laboratoriumonderzoek
    (sand boils, laboratory investigation)
    Centrum voor onderzoek Waterkeringen S-71.063
    Zierikzee, Netherlands, 1978

18. Leidraad bij het ontwerpen van rivierdijken
    Technische Adviescommissie voor Waterkeringen
    Netherlands, 1985

19. Soil mechanics handbook, volume 1
    A. Kézdi
    Amsterdam, Netherlands, 1974

20. Liquefaction problems in geotechnical engineering
    ASCE, National convention
    Philadelphia (Pa), U.S.A., 1976

21. Evaluation of the dynamic characteristics of sands by
    in-situ testing
    H.B. Seed, Revue francais de geotechnique
    Paris, France 1983

22. Grondmechanica (soil mechanics)
    A.S. Keverling Buisman
    Delft, Netherlands, 1944

23. Manual of soil laboratory testing
    K.H. Head
    Plymouth, U.K., 1982

-135-
24. Grondmechanica (Soil mechanics)  
T. K. Huizinga  
Amsterdam, Netherlands, 1969

25. Dike raising in a seismic, subsiding area 3  
An introduction to earthquake induced liquefaction  
M. R. Tonneijck  
Delft, Netherlands, 1987

26. Dike raising in a seismic, subsiding area 3  
Evaluation of liquefaction resistance and the implications for the design of dikes  
M. R. Tonneijck  
Delft, Netherlands, 1987

27. General theory of stability of slopes, Géotechnique 5  
Alan W. Bishop, M.A., Ph.D., A.M.I.C.E.  
Paris, France, 1955

28. Horizontale bronbemaling (horizontal drainage), Vertical kwelschermen (vertical seepage screens)  
LARECO, aannemingsmaatschappij (contracting company)  
Arnhem, Netherlands, 1988

-136-
APPENDIX 1

DETERMINATION OF SIGNIFICANT WAVE HEIGHT, GIVEN WIND SPEED, DURATION AND FETCH.

A.1.1 INTRODUCTION

It is not considered useful to work with momentary water level fluctuations of which the scale is determined by the actual waves. Data of these fluctuations are reduced to a workable amount of parameters, mean values of momentary water levels. Time and distance scales are chosen thus that these parameters can be considered constants, thus assuming a stationary situation: time and distance scales have dimensions of a few hundred times the wave period $T$ and wave length $L$.

The method of representative wave gives approximations for $H_s$ and $T_s$. Bretschneider (1973) and Dobroklonskiy (1973) are good representants of this method for the values of the wind fetch that is dealt with here. A common parameter is the energy-density-spectrum from which the significant wave height can be calculated. Hasselman (JONSWAP, 1973) and Mitsuyasu (1971) render good results. [lit.7]

Wave prediction methods for a standard wind field

1. method of representative wave
2. method of standard spectrum

Wind is a vector varying in time (direction and magnitude). For a standard windfield we assume the wind to have a constant mean speed and constant direction. The field has a windward limit perpendicular to its direction, starts to blow suddenly at $t = 0$.

Wind fetch $F = \text{distance to windward limit}$.

Wind duration $t_w = \text{time passed since } t = 0$.

Wind speed $U_w = \text{constant average wind speed at 10m}$

For limited values of wind fetch $F$ and duration $t_w$ one of these two parameters determines the growth of waves in water. It is
assumed that, with equal wind speeds, a wave spectrum after duration $T_w$ (at non-limited $F$) is equal to a wave spectrum over fetch $F$ (after non-limited $t_w$).

The equivalent wind fetch for duration $t$ is $F_{eq} = c_1 t$
or the equivalent duration for wind fetch $F$: $t_{eq} = F/c_1$,

where $c$ is the group celerity of the waves, to be derived from the wave-energy-spectrum: $c_1 = g/(4 f_m)$.

Hence: $F_{eq} = gt/(4 f_m)$ and making units dimensionless with:

$$F_{eq} = gF/u^2, \quad \tilde{f}_m = f_m u/g, \quad \tilde{t} = gt/u \quad (1)$$

It follows: $\tilde{F}_{eq} = t/(4 \tilde{f}_m)$, transformation rule for $F$

From this is clearly seen that short durations limit the wind fetch, thus limiting the growth of waves.

Table 3.2 implies a certain relationship between wind speed and wind duration, which could be used to find a certain speed-duration-combination that results in a maximum equivalent wind fetch $F_{eq}$, thus that the full 75 km available at Lake Maracaibo are used for wave generation.[see figure 3.5]

A.1.2 METHOD OF REPRESENTATIVE WAVE

Data supplied by CSV give the period of maximum energy $T = 6.5$ s. [see table 3.2] Using this period for $f_m$ and $F_{eq}$ we find that for the wind speed-duration-combinations 8.2 m/s - 6.6 h and 10.0 m/s - 3.0 h the equivalent wind fetch is respectively longer and shorter than the actually available wind fetch of 75000 m. [see table 3.3a]

By linear interpolation, which is a pessimistic approximation, and by trial and error we can find an 'optimal' speed-duration-combination that yields the full available fetch. This combination is: 9.45 m/s - 4.1 h.

Bretschneider and Dobroklonskiy give approximations for the
significant wave height. [see table 3.3a][lit.7, pg. 32 and 35]

A.1.3 METHOD OF STANDARD WAVE SPECTRUM

Hasselman (JONSWAP) and Mitsuyasu give comparable approximations for calculation of $f$ from the wind fetch:

\[
\tilde{f}_m = 3.5 \tilde{F}^{-0.11}
\]

Mitsuyasu: \[\tilde{f}_m = 3.24 \tilde{F}^{-0.11}\]

Substitution in the transformation rule for $F$ makes it possible to determine the proper value for $F$ by iteration:

\[
\tilde{F}_n = \tilde{\tau}/(4 \left( 3.5 \tilde{F}_{n-1}^{0.11} \right))
\]

Again we look for a certain speed-duration-combination optimum by linear interpolation and trial and error. Both JONSWAP and Mitsuyasu yield combinations that differ from the representative wave method described above:

- JONSWAP: 9.0 m/s - 5.0 h.
- Mitsuyasu: 9.2 m/s - 4.6 h.

JONSWAP and Mitsuyasu also give the approximations for the significant wave height $H_s$. [see table 3.3b][lit.6, pg. 59]
PIPING

A.2.1 INTRODUCTION

The phenomenon piping or subsurface erosion occurs in dams and dikes that retain a certain water head. This water head maintains groundwater flow under the dike or dam. A sand boil occurs when this groundwater seeps towards the surface and exerts seepage forces. It carries along soil particles, indicating that the construction is being undermined by erosion. [see figure 3.85] Generally two types of piping can be distinguished, depending on the composition of the ground:
1. The fine soil fraction is washed out through the coarse fraction, while these coarse soil particles still maintain the stability of the ground at first: suffosion.
2. Soil particles in uniform homogenous soils are washed out, thus destroying the grain skeleton from the beginning.

Investigation into this phenomenon can be split into two types: the empirical/statistical methods (Bligh, Griffith, Lane) and more advanced methods based on theory of groundwater flow.

A.2.2 EMPIRICAL/STATISTICAL METHODS

In these empirical/statistical methods the investigators make an inventory of dams that failed due to subsurface erosion and give a simple indication for the dam’s structure and situation. The type of piping is disregarded. Minimal required seepage length, \( L \) and retained water head, \( H \) are the determining parameters to designate a dam as safe, given the soil of the foundation. [see figure 3.86] Relations are then of the form \( L/H > C \) in which \( C \) depends on the type of soil.

Bligh (1912)
Bligh gives rules for stone dams on a sandy, porous foundation. It is however not clear how reliable his investigations are.
Values for $C_b$ are given in table 3.17a.

**Griffith (1913)**
Griffith uses the same concept, his values for $C_b$ are listed in table 3.17b.

**Lane (1935)**
The best documented investigation was Lane's. He investigated into hundreds of masonry and concrete dams and had the idea to reduce the horizontal tracks of the seepage ways in order to count for the relatively low resistance for water there:

$$\frac{\sum L_v + \frac{1}{2} L_h}{H} = C_L$$

[see table 3.17c]

### A.2.3 Evaluation of Empirical/Statistical Methods

The safety given by Lane and Bligh has been evaluated for fine sand and silt foundations, with help of the data of Lane’s investigation. [see lit. 10]
Lane did not state how he arrived at his safe value $C$, but it is obvious that he disregarded the failure of the dam with $C_0$ -value between 10 and 11. [see figure 3.87]
Applying Bligh's method to Lane's data, the same dam failure must be disregarded to agree with Bligh's safe $C_b$ -values. [see figure 3.88]
There is no contradiction between the two investigations with respect to dams on fine sand or silt.

Both figures 3.87 and 3.88 show that assessment of safety with respect to piping is dangerous using Bligh or Lane. For some $C$-values below the safe one, the rate of failure is 100% whereas for some other values 0%. This can be attributed to too few failures (too few data), but it can also be an indication that the used criterion gives an unjust assessment of safety.

It should be emphasized that Bligh and Lane can be used for dams and dikes on homogenous soils only. An overlying impermeable
layer disturbs the concept as proposed by these investigators. Holes in that top layer may lead to shorter seepage lengths. Piping under earth dams and dikes is often different from that under stone dams as treated by Bligh and Lane. More advanced methods must deal with this problem.

A.2.4 METHODS BASED ON THEORY OF GROUNDWATERFLOW

The exit velocity and the hydraulic gradient at the surface are important parameters concerning the process of soil particles being washed out. With help of the theory of groundwaterflow a critical hydraulic gradient can be determined and from that and field experience, design criteria for dikes with respect to piping. In these methods several subsoil situations are distinguished:

1. uniform, homogenous subsoil: a. vertical flow (near sheet pile walls)
   b. concentrated outflow (i = \infty)
2. subsoil with overlying impermeable layer
3. non-uniform homogenous subsoil
4. heterogenous subsoil

1a. Uniform, homogenous subsoil subsoil, vertical flow
A situation as for a sheet pile construction [see figure 3.89] is not of much importance for dikes, but it shows basic principles of subsurface erosion and can be used for the dike design criteria. To prevent a sand boil, the upward hydraulic gradient should have a limited value. The critical situation occurs when the upthrust or buoyancy on the soil particles is equal to their weight, i.e. when the water pressure is so high that the effective ground pressure \( \sigma' = 0 \). [see figure 3.90]

\[
\sigma = \sigma' + u
\]

At depth \( d \):
\[
\sigma = \rho_1 g d
\]
\[
u = \rho_w g d + \rho_w g h
\]
The reference level is taken at the ground surface and from this follows the critical hydraulic gradient:

\[ i_{cr} = \frac{h}{d} = \left( \gamma_w - \gamma_n \right) / \gamma_w \]

As long as \( i < i_{cr} \) the water head difference is spent in driving the groundwater upward through the pores of the soil. The energy is consumed in viscous friction. When \( i > i_{cr} \) the drag of the water is so strong that it may take the grains along.

Considering that the bulk unit weight for saturated sand and silt is usually between 18 and 22 kN/m\(^3\) the critical hydraulic gradient is \( i_{cr} = 0.8 \) to 1.2.

Since the soil is never really homogenous (variations in grain size distribution, in degree of compaction, permeability, unit weight etc.) a factor of safety should be applied to the critical hydraulic gradient. Experience shows that a value for \( i_{cr} = 0.5 \) is acceptable. [see lit. 10]

Applying this to a situation for dikes, the piping design criteria should be \( i = 0.5 \) at the inner toe. Farther from the dike toe at a distance of 2 to 3\( H \) or at 5\( H \) from the inner crest line this critical hydraulic gradient may be raised to \( i_{cr} = 0.7 \). [see figure 3.90]

1b. Uniform, homogenous subsoil, concentrated groundwater flow

Concentration of groundwater flow occurs e.g. at the inner toe of clay dikes at permeable (sand or silt) foundations and also along foreign elements in the dike body, like pipes. [see figure 3.91]

The hydraulic gradient is infinite at those spots, \( i = \infty \).

An infinite flow velocity is inherent to an infinite hydraulic gradient. Of course the hydraulic gradient is limited in reality, but it can't be used as a criterion for piping like in the above case. The concentration of flow does not allow mathematical description. Gradients measured near the singular point can't be representative as the turbulent flow changes the permeability considerably. Which parameter should be used is still an unsolved problem. The gradient of the streamline directly under the dike (that is most strongly influenced by the change in permeability near the concentration point) offers some (yet unproved) perspectives. [see lit. 17]

It is often not possible to avoid these concentration points in
dike construction. Special measures, like e.g. applying a toe blanket, must be taken to prevent erosion then. [see figure 3.90]

2. Overlying impermeable layers
A common feature in e.g. Dutch situations is the presence of an overlying impermeable clay or peat layer. These layers have a relatively low bulk mass and there is a serious danger for upbursting of this layer. [see figure 3.92] Upbursting result in a singular point where flow concentrates. [see figure 3.93]
In addition to restrict the hydraulic gradient to \( i < 0.5 \) or \( i = 0.7 \), care should be taken that the weight of the overlying layer exceeds the upthrust:

\[
p < \frac{G}{1.2} \quad \text{to} \quad \frac{G}{1.1}
\]

in which \( p \) is the water head at the toe in the permeable layer, \( \frac{(u/\gamma_w)}{} \) and \( G \) is the weight of the overlying layer: \( \beta \cdot g \cdot D \).
If the weight of the overlying layer is not sufficient, e.g. a toe blanket can be applied to increase \( G \).

3. Non uniform, homogenous soils
In cohesionless soil with rather distinct fractions in the grain size distribution the fine fraction may be dragged through the coarse fraction: suffosion.
Two conditions must be fulfilled for this to occur:
- geometric: the composition must be thus that it is physically possible to occur
- hydraulic: the groundwaterflow must be strong enough

A relatively easy but vague assessment is given by Istominia for the geometric condition:

\[
\begin{align*}
\text{no suffosion} & \quad u < 10 \\
\text{danger for suffosion} & \quad u > 20
\end{align*}
\]

The parameter \( u \) is a uniformity coefficient: \( u = d_{95}/d_m \)

The hydraulic condition is not properly solved yet. Should the occasion arise, experiments will give the best assessment.
A concept based on filter rules of Terzaghi (next paragraph) is also possible.

4. Heterogenous subsoil
A slightly different danger for suffosion occurs when a relatively coarse layer overlies a relatively fine layer. Terzaghi's widely used filter rule gives two conditions:

- to prevent erosion (piping) \( \frac{D_{15}}{d_{15}} < 4 \)
- to ensure water permeability \( \frac{D_{15}}{d_{15}} > 4 \)

in which \( D = \) grain size of the coarse layer
and \( d = \) grain size of the fine material.
APPENDIX 3.

EARTHQUAKES.

A.3.1 EARTHQUAKE CHARACTERISTICS.

An earthquake propagates elastic waves in all directions from its centre of origin: the hypocentre. The point at the earth surface straight above the hypocentre is called the epicentre. [see figure 3.74]

Different phenomena like volcanic activity, tectonic movements and explosions may induce earthquakes. Earthquakes of tectonic origin form important dangers for engineering constructions. Due to movements along geologic fractures or faults, stresses accumulate until they exceed the strength of rocks when slip occurs. This may produce a considerable shock because of the energy involved.

Housner [lit. 3, pg 15] distinguishes four types of faults:
1) Low angle, compressive underthrust faults.
   Tectonic sea-bed plates thrust under adjacent continental plates. Much of the Circum-Pacific belt is like this.
2) Compressive overthrust faults.
3) Extensional faults, the inverse of the previous type.
4) Strike-slip faults.
   Where the two sides, blocks of the fault slip, i.e. where they move relatively in a horizontal direction. [see figure 3.75]

The strength or size of an earthquake is usually given by its magnitude $M$ on the scale of Richter. $M$ is defined as the logarithm of the trace amplitude (in mm) of a specified standard seismograph, located at 100 km from the epicentre. The intensity $I$ of an earthquake is given by a number on the widely used Modified Mercalli scale (MM) [see table 3.13] or the Rossi-Forel scale.
Where $M$ is an objective quantitative measure the intensity $I$ is a
subjective measure to describe the degree of shaking, the effect of the earthquake and the possible damage that it may cause. The value of $M$ is characteristic for an earthquake, where the value of $I$ is dependent on the distance to the epicentre and the soil characteristics. From the definition of $M$ follows that $M$ is proportional to the energy released by the earthquake, therefore $M$ is also a certain measure for the duration of the earthquake. Places round the earthquake epicentre where a comparable intensity can be felt are connected by isoseismal lines. [see figure 3.76]

**A.3.2 EARTHQUAKE PREDICTION.**

Earthquake prediction is still a very difficult task, though methods to predict the earthquake magnitude that is likely to occur near the site during the designed construction lifetime do exist. Probabilistic methods require a very extensive stochastic analysis. As accurate instruments are only of recent invention an extensive stochastic analysis is for two reasons not reliable:

1. Older data concern estimates of earthquakes of usually $M>4$ and are thus incomplete and inaccurate.

A general empirical magnitude-frequency-relationship is given by Gutenberg and Richter: $\log N = A - bM$, where $N$ is the number of shocks of magnitude equal or greater than $M$ per year and $A$ and $b$ are seismic constants for individual regions. [see figure 3.79 and table 3.16] If older data are included than recent events of $M<4$ are usually disregarded for reason number 1 as stated above. [lit.3, pg.20]

An alternative is found in a geotechnical earthquake prediction method that gives two values as a base for selecting a design earthquake:
- MCE (maximum credible earthquake): this is the earthquake that appears capable of occurring given the tectonic situation. The
probability of occurrence is not regarded, but is big enough to count with it.

- **MPE (maximum probable earthquake)**: this is the earthquake that is likely to occur in a certain time interval, probability of occurrence is regarded, this MPE-magnitude shall never be lower than the maximum historic earthquake magnitude.

In this geotechnical method the geologic characteristics and seismic history of the region are considered together with the length and type of faults that may influence the project area. [lit.9, vol. III, pg 1411]

The Bureau of Reclamation (U.S. Department of the Interior) initiated in 1972 the use of MCE as the required seismic loading for critical structures. This MCE shall be based on a 'reasonable' return period in which 'reasonable' is supposed to be 50000 years. Noncritical structures may do with an event that is likely to occur during the lifetime of the project, quantitively defined as an earthquake with a recurrence interval of 500 years. [lit.4, pg.177,178]

**A.3.3 SITE RESPONSE TO EARTHQUAKES.**

Once having chosen a design earthquake, the evaluation of the response of a site at some distance from the focus remains a very important, but difficult task.

Of course numerous soil condition factors and geological features between the earthquake focus and the site affect the site response to the earthquake. Factors that should be taken into account are:

- Presence of soils overlying the bedrock.
- Horizontal extent and depth of the overlying soils.
  
  The natural period of vibration of the ground increases with this depth. The softer soils may filter out the higher frequencies, which is dangerous for long-period structures.
- Topography of bedrock and softer soils has effects on wave propagation i.e. on reflection, refraction, focusing and scattering.
- A sloping terrain may give rise to avalanches, in case sand lenses are present, possibly in combination with soil
liquefaction.
- Water content
- Soil types

A.3.4 GROUND MOTIONS.

A.3.4.1 INTRODUCTION.

Earthquakes make the ground shake, thus giving it accelerations (a), velocities (v) and displacements (d). Other important parameters of ground motion are response spectral ordinates and earthquake duration. [see figure 3.78a]

Ground motions can roughly be classified in four groups:

1. Single shock earthquakes. Under certain conditions this type occurs at short distances from the epicentre of a shallow earthquake and waves must travel through firm ground avoiding numerous refractions and diffractions that change the wave characteristics. (examples: Skopje, 1963 and Agadir, 1960) [see figure 3.78a]

2. Earthquakes with extremely irregular motion and relatively long duration. Occurs most often in the Circumpacific Belt at moderate distances from the epicentre. (example: El Centro (CA), 1940)[see figure 3.78b]

3. Earthquakes with long ground motion. Soft soils filter earthquakes from the first and second type, only leaving pronounced prevailing periods of vibration. (example: Mexico City, 1964)[see figure 3.78c]

4. Earthquakes with ground motions that involve large scale ground deformations like land slides and soil liquefaction. (example: Chili, 1960)

A.3.4.2 EARTHQUAKE WAVES.

Earthquakes release three wave groups:

1. Compressional (longitudinal) or primary waves: P-waves. They travel fastest, order 6 km/s.

2. Shear (transverse) or secondary waves: S-waves. They travel a bit slower, order 4 km/s.

As soon as a P- or S-wave reaches the earth surface this third category of waves spreads out from that point. These are transverse vibrations of long period, they follow the earth periphery, have large amplitude and cause the greatest damage.

Usually the motion of P-waves occurs in a horizontal plane and they are denoted as PH-waves, while the motion of S-waves is commonly oriented in a vertical plane, hence SV-waves. The earth surface reflects waves; a P-wave e.g. may be reflected as a transverse wave, denoted then as PS-wave. Figure 3.77 shows an accelerogram in which P-, S- and L-waves are distinguished.

A.3.4.3 LOCAL GROUND ACCELERATIONS.

Usually ground accelerations are expressed in fractions of the acceleration of gravity, \( n \ast g \), in which \( n \) is called the seismic coefficient.

At a construction site the local ground motion parameters are important. Numerous attenuation relationships have been developed, especially after 1970, to determine the local peak ground acceleration \( a \) (as well as velocity and displacement), that is dependent on the earthquake's magnitude \( M \), the epicentral distance \( R \) to the construction site and the wave propagation characteristics of the soil. The general form of these attenuation relationships is:

\[
\ln(a) = A + f(M) + f'(R) + f''(\text{soil})
\]

(\( a \) in cm/s\(^2\) and \( R \) in km)

The ground velocity is found by integrating the acceleration. Examples of these relationships are shown in figures 3.81.

Magnitude.
The dependence on magnitude is not always directly expressed in the relationships. Recently relationships are derived for certain
narrow magnitude ranges. Commonly \( f(M) \) is of the form \( BM + B'M \) in which \( B' \) is negligibly small.

**Distance.**
The function \( f'(R) \) is often written as \(-E \ln(R + c)\). Physically it is sensible to use an increased value for \( c \) when the ruptured fault is longer. A long ruptured fault releases more energy during an earthquake, a higher value for \( c \) shows then that the attenuation is slower.

**Soil subsurface conditions.**
In many relationships this factor is either ignored or the relationship is developed for one site, one specific situation only.

From these attenuation relationships Esteva (1973) is often cited. His relationship is valid for firm ground.

\[
\ln(a) = 8.63 + 0.8M - 2\ln(R+40) \quad \text{or} \quad a = 5600e^{0.8M}/(R+40)^2
\]

\[
\ln(v) = 3.47 + M - 1.7\ln(R+25) \quad \text{or} \quad v = 32e^M/(R+25)^{1.7}
\]

Most broadly based however is Donovan's relationship (1978) that applies to sites with 6m or more soft soil overlying the bedrock.

\[
\ln(a) = 6.98+0.5M-1.32\ln(R+25) \quad \text{or} \quad a = 1080e^{0.5M}/(R+25)^{1.11}
\]

Both expressions are valid for distances \( R > 15 \text{ km} \). Within the epicentral area peak ground accelerations versus intensity yields a highly scattered pattern. Fault details influence the response there. [see figure 3.84]

Estimates of peak ground displacements are of little reliability, mainly because they are derived from double integrations of acceleration estimates. Newmark and Rosenblueth give

\[
5 < (a*d/v^2) < 15
\]
where the lower value applies for longer epicentral distances ($R \approx 100$ km and the higher value for $R \approx 15$ km. [lit.1]

Seed et al. have in "Site-dependant spectra for earthquake resistant design", 1976, averaged the spectra of several earthquakes, grouping them according to site conditions (damping) and focal distances, weighing them to $M = 7.5$ magnitudes. This results in figures as 3.82b.
APPENDIX 4

LIQUEFACTION

A.4.1 INTRODUCTION.

Cohesionless soils have a shear strength \( \tau = \sigma' \tan \phi' \), with \( \sigma' = \sigma - u \).

Liquefaction of saturated soil (usually sand) can generally be defined as a condition in which the soil has lost its shear strength completely due to pore water pressure rise or sudden drop of the confining stress. In both cases the effective stress goes down to zero. The sand starts to flow like a thick, viscous fluid and offers little or no resistance to landsliding.

Liquefaction can be of static or dynamic origin. Casagrande treated liquefaction under unidirectional static loading in 1936, but only the Niigata (Japan) and Anchorage (Alaska) earthquakes of 1964 gave the onset for investigation into earthquake induced liquefaction. Other types of cyclic loading like wave action, machine vibrations and traffic are also dynamic causes for liquefaction. We restrict ourselves to earthquake induced liquefaction in saturated soil layers, usually sand. (A.4.2)

It is important to find out whether a construction is threatened by liquefaction. The liquefaction potential of soil layers is affected by the characteristics of these layers and the earthquake. (section A.4.3)

In the following section several methods to evaluate liquefaction potential are laid down. In the last section attention is paid to propagation of initial liquefaction.
A.4.2. PHENOMENON OF EARTHQUAKE INDUCED LIQUEFACTION

In an idealized situation liquefaction occurs in horizontal saturated sand strata overlying bedrock. This implies that no static shear stresses apply to the soil particles under consideration. (Static shear stresses increase the resistance to liquefaction.)

Soil particles in these strata are affected by the shear waves that propagate upwards from the bedrock. [see figure 3.94] Assumed is that there is no lateral force except the lateral pressure at rest, \( K_0 \sigma' \).

During one loading cycle the soil particle is acted upon by a shear force. This shear force causes slip at the intergranular surfaces. Thus there is a tendency in the soil to settling, to a decrease in volume. However, one cycle is short and the little compressible water does not get the chance to flow out (undrained situation). Part of the confining stress must then be transferred from the soil particle contacts to the water: after one stress cycle there is a residual pore water pressure rise, \( \Delta u \). This process repeats itself as long as the earthquake lasts and initial liquefaction occurs when all the confining effective stress is transferred from the grains to the water: \( \sigma' = 0, \ u = \sigma \). [see figure 3.105]

In addition to the general definition for liquefaction Seed (lit.20) distinguishes also:

**Initial liquefaction**: During the application of cyclic stresses the pore water pressure \( u \) reaches the confining stress \( \sigma \). It is not yet sure what the consequences will be as for the resulting deformation of the soil.

**Initial liquefaction with limited strain potential or cyclic mobility or cyclic liquefaction**: A situation where initial liquefaction occurs, but the soil deformation is limited because of remaining soil strength or because dilation of the soil with subsequent pore water pressure decrease and recovery of the shear strength (effective stress).

The importance of this distinction is that, if liquefaction with subsequent landsliding does not occur, the residual increased
pore water pressure may give rise to liquefaction in overlying layers, to sand boils, upbursting or seepage after the earthquake, i.e. after the application of cyclic stresses has stopped.
A.4.3. FACTORS AFFECTING LIQUEFACTION POTENTIAL

Whether liquefaction may occur, depends on many factors. The chief factors are:

1. type of soil, grain characteristics
2. relative density, void ratio
3. initial confining stress
4. aging
5. strain history
6. lateral pressure coefficient and overconsolidation ratio
7. duration of shaking
8. intensity of shaking

-1. type of soil
Tests and field experience show that liquefaction occurs more easily for:

a) cohesionless soil (sands), rather than gravels, silt and clay.
b) uniformly graded sands, rather than well graded sands.
c) sands with round shaped grains, rather than angular grains.
d) fine sands, rather than coarse sands.

Some soils have a marked tendency for spontaneous liquefaction. These are soils with high porosity ($n > 44\%$), well rounded grains, and a very uniform grain size distribution: ($u < 5$, $u = d_{60}/d_{10}$).

-2. relative density
Loose sand is more susceptible for liquefaction than the same sand when more compacted. Loose sands tend to decrease in volume when subjected to shear forces, so that the pore water pressure rises as described in the above section. Denser sands however, are not able to undergo such deformations (or at least to a lesser extent) or even dilate, resulting in a pore water pressure decrease so that the sand develops enough resistance to the applied stress. [see figure 3.95]
3. initial confining pressure
For a given initial density, sands subjected to higher initial confining stresses \((\sigma_c, \text{and} \sigma'_c)\) show higher required stresses to initiate liquefaction. This is logic in the sense that the higher the confining stress, the more stress must be accumulated in the pore water during the earthquake before liquefaction occurs. This is opposite to the static situation.

4. aging
Laboratory studies and field experience show that sands that have been subjected to sustained loads for a long period, prior to the earthquake, show more resistance to liquefaction. For very long periods (centuries) this increase may be as much as 75%. This increase must be attributed to a sort of cementation or welding at the intergranular contact points.

5. strain history
Previous earthquakes that did not lead to liquefaction, have, when strains were small enough, a favourable effect on the resistance to liquefaction. This may be attributed to an increase in the lateral pressure coefficient \(K_0\).

6. lateral pressure coefficient and overconsolidation ratio
A higher overconsolidation ratio - defined as the ratio of the maximum past consolidation stress and the effective consolidation stress - implies a higher lateral pressure coefficient \(K_0\). This reflects an increase in the mean confining pressure:

\[
\sigma_{mo}' = \sigma'_o (1 + 2K_0)/3
\]  

(1)

The higher this \(\sigma_{mo}'\), the higher the resistance to liquefaction. [see figure 3.96]
Both sustained strains and higher overconsolidation ratios have certainly an effect on the grain structures, so that it is not so likely that slip between the particles will occur.

7. duration of shaking
The application of cyclic stresses must be long enough for the stress to accumulate in the pore water for liquefaction. A minimum of stress cycles is required for \(u\) to reach \(\sigma'\).
8. intensity of shaking

The intensity of the earthquake is reflected in the simple relationship for the applied shear stress on a soil column over a soil particle at depth \( d \):

\[
\tau = C \left( \frac{\gamma d}{g} \right) a_{\text{max}}
\]

\[
= C \left( \frac{\gamma d}{g} \right) n
\]

where \( \gamma \) = unit weight of the soil,
\( g \) = acceleration of gravity,
\( a_{\text{max}} \) = maximum acceleration of the earthquake,
\( n \) = seismic coefficient of the earthquake,
\( C \) = constant < 1.
A.4.4 METHODS TO EVALUATE LIQUEFACTION POTENTIAL

In general there are two methods to evaluate the liquefaction potential of saturated sand layers.

1. Methods based on the performance of sand deposits during previous earthquakes, involving in-situ testing.

For these methods measurements are required at sites where liquefaction is known to have taken place, during or just after an earthquake.

Usually a relationship is found between the stress ratio \( \tau /\sigma_o' \) and \( N \), the number of blows in the Standard Penetration Test (SPT), which is an indication for the relative density \( D_r \) of the soil.

The cyclic stress ratio that is required for liquefaction is then:

\[
\frac{\tau}{\sigma_o'} = 0.65 \times \frac{a_{\text{max}}}{g} \times \frac{\sigma_o}{\sigma_o'} \times r_d
\]

where:
- \( a_{\text{max}} \) = maximum earthquake acceleration at the surface
- \( \sigma_o \) = total overburden pressure on considered sand layer, before cyclic stresses are applied
- \( \sigma_o' \) = effective overburden pressure, before cyclic stresses
- \( r_d \) = a stress reduction factor varying from 1 at the surface to 0.9 at 10 m depth

This is essentially the same formula as (2), given in the previous section 'FACTORS'. The values are plotted versus \( N \) (corrected for the overburden pressure). [see figure 3.97]

A disadvantage is that there are not sufficient reliable data available yet, especially for high stress ratios. This method does not count for the duration of the earthquake, which is certainly a factor to be considered.

Although the penetration resistance (relative density) may not be an appropriate value to evaluate liquefaction potential of soil layers, an increasing value of the stress ratio \( \tau /\sigma_o' \) has the same effect on penetration resistance as on other factors [see table 3.19.

The parameter \( \tau /\sigma_o' \) can be correlated with other soil parameters. Arulmoli et al. (1981) investigated on electrical characteristics of soil, Marchetti used a so called flat dilatometer test. More
recent is the correlation with CPT-values instead of SPT-values. The CPT has some advantages over the SPT. It provides data more rapidly, it provides a continuous record of penetration resistance and its data are more reliable for various reasons. The main disadvantage is however that CPT data to predict resistance to liquefaction are much scarcer than SPT data. Correlations between SPT- and CPT-values may be used for the evaluation of liquefaction potential using CPT-values. [see figure 3.106] [see lit. 21]

-2. Methods based on evaluation of stress conditions causing liquefaction in the field and in laboratory tests.
These methods are more challenging, they involve two independent determinations:
- evaluation of the cyclic stresses to which the soil is subjected during an earthquake
- evaluation of the cyclic stresses that cause liquefaction in laboratory tests

There is a general working procedure in these methods:

a. Evaluation of earthquake induced stresses.
b. Conversion of irregular stress application into an equivalent uniform stress application.
c. Evaluation of factors affecting liquefaction potential.
d. Development of laboratory test procedures.
e. Evaluation of the effect of sampling.

-a. Evaluation of earthquake induced stresses.
Methods that consider the pore water pressure rise lead to direct evaluation of the liquefaction potential (Martin, 1975 and Finn, 1976). These methods require more material properties. In general though, the build-up can be disregarded without doing harm to the computed stresses in a soil deposit and this gives the opportunity to use more simplified methods (Seed and Idriss, 1987 and 1971).
-b. Conversion of irregular stress application into an equivalent uniform stress application.

There are three methods:

- visual inspection (with experience a surprisingly good method)
- cumulative damage approach, a black box method (Donovan and Valera 1976)

Valera and Donovan show that whatever method is used, it has little effect on the final result.

c. Evaluation of factors affecting the liquefaction potential.

At first only the density of the deposits was considered to affect the liquefaction potential, but it is recognised that there are many more factors. [see section A.4.3]

d. Development of laboratory test procedures.

Because of the lack of sufficient field data there is a need to generate more stress conditions for liquefaction by laboratory tests. A lot of difficulties must be overcome to transfer test results to the actual situation in the field.

Two types of test are common, (1) the cyclic triaxial test and (2) the simple shear tests.

1. Cyclic triaxial tests with controlled deviator stress.

These tests are still widely used but have considerable limitations.

A sample is saturated under isotropic confining stress. [see figure 3.98] During the test the deviator stress is cycled sinusoidally, resulting in the following stress conditions in the sample:

\[ \sigma_1 = \sigma_0 + \sigma_o \sin(2\pi N) \]  \hspace{1cm} (4)

\[ \sigma_2 = \sigma_3 = \sigma_0 \]  \hspace{1cm} (5)

where \( \sigma_1 \) = principal stress

\( \sigma_0 \) = initial octahedral stress

\( \sigma_o \) = octahedral stress during the test

\( \sigma_o \) = deviator stress

N = load cycle
Limitations of this test method include:

a. reproduction of level ground condition
b. stress concentrations at the top and the base of the sample
c. contraction (necking) of the sample must be considered
d. principal stresses rotate during the two halves of the loading cycle
e. the intermediate principle stress $\sigma_{\alpha\beta}$ is not constant

2. Torsional simple shear test.
This test permits a sample to develop shear strains $\gamma$ on top at linear rotation $\theta$. [see figure 3.99]
The confining stresses $\sigma_h$ and $\sigma_v$ are not necessarily isotropic.
The shearing stress $\tau_{\alpha\beta}$ is applied independently. In this simple shear test the objections a, b and e to triaxional tests have been overcome. Objections c and d are not applicable. More general difficulties however, remain:

a. preparation of representative samples
b. development of uniform shear stresses and strains

Instead of a torsional shear device, it is also possible to use a device with long shallow samples.

-e. Evaluation of the effect of sampling.
Undisturbed samples don't exist. The main problem in sampling seems to be the changing density when taking and transporting the sample. Loose sands will densify and yield higher resistance to liquefaction; for dense sands the opposite effect occurs. Strength increases due to strain history seem to vanish when sampling, also cementation effects from sustained loading gets lost and also the effect of increased $K_0$ disappears when sampling. Altogether relatively dense samples will show lower resistance to liquefaction than when still in-situ. [see figure 3.107]
A.4.5 PROPAGATION OF LIQUEFACTION

When, during an earthquake, initial liquefaction occurs in a certain soil layer, this liquefaction must propagate throughout the layer or to other overlying layers, before actual failure of the dike occurs. This propagation needs time and may continue even after the earthquake shaking has stopped.

Propagation during the earthquake
When a sand layer in the base of a dike liquefies, the process will start near the centre of the base, where the earthquake induced shear stresses are much stronger than elsewhere along the base. [see figure 3.100] The liquefaction progresses somewhat faster in upstream direction than downstream. [see lit.11, pg.260] Hence, on the downstream side is the largest non-liquefied zone to maintain dike stability. [see figures 3.101 and 3.102]

There are two ways now in which the dike may fail. Initially the water pressure on the relatively impervious upstream side of the dike may maintain the stability of the outer slope, but if this pressure is not sufficient, the outer slope may still fail along a slip circle that passes through the liquefied layer. [see figure 3.103]

The second way of failure is that outer slope stability is maintained until the non-liquefied zone on the downstream side has become so small that the entire dike section slides downstream.

Propagation after earthquake
An earthquake may cause initial liquefaction in somewhat deeper layers, not causing damage to the construction immediately, due to rapid dissipation of the earthquake induced pore water pressures.

However, high pore water pressures in deeper layers, produced by an earthquake, may result in upward flow of water, causing liquefaction in overlying layers, minutes after the earthquake shaking has stopped. This happened e.g. in the Niigata earthquake of 1964 where initial liquefaction during the earthquake occurred in layers between 15 and 40 feet depth and caused liquefaction in layers of 3 feet and 1 foot depth at 3 and 12 minutes after the
earthquake had stopped. [see figure 3.104]
A.5.1 SHEAR STRESS, COULOMB'S LAW

Coulomb's law gives a well holding relationship between maximum shear strength \( \tau_f \), the normal total stress and the two shear strength soil parameters: the friction angle \( \phi \) and the cohesion \( c \).

\[
\tau_f = c + \sigma \tan \phi
\]

This equation is applicable to total stresses; when calculating with effective stresses a modification is necessary that takes account of pore water pressures. When using these effective stresses, cohesion and friction angle are usually given as \( c' \) and \( \phi' \).

Figure 3.128 shows Coulomb's relationship for various types of soil. In dry and saturated non-cohesive soils there are no internal forces holding the soil particles together: the cohesion \( c = 0 \). The shearing resistance of sands depends wholly on frictional forces that are only present when compressive normal stresses are active. In densely packed sands these friction forces are stronger than in loosely packed sands. [see figure 3.128a]

Lightly cemented sands do have some cohesion that dissapears after some deformation. [see figure 3.128b]

Partially saturated sands show a relationship as if there were cohesion. Due to capillary action these sands can show shear strength without active frictional forces. This is called apparent cohesion.

The shear strength of cohesive soils depends chiefly on a) type of clay, b) water content, c) degree of saturation d) degree of consolidation.

The water content has mainly influence on the clay's cohesion. A clay at the liquid limit (liquidity index, \( LI = 1 \)) has cohesion \( c = 1.7 \text{ kN/m}^2 \), where a clay at the plastic
limit \((LI = 1)\) shows a 100 to 150 times greater cohesion: \(c = 200\, \text{kN/m}^2\).

Saturated clay samples are hardly able to develop friction forces: \(\phi = 0\). This situation is often close to the undrained situation shortly after a load being applied. [see figure 3.128c]

Since clays are little permeable to water an increase of load will result in immediate pore water pressure rise that will be converted into effective stress during consolidation. A well consolidated clay (excess pore water pressure dissipated, water pressure \(u = \text{hydrostatic pressure}\)) has, like sand, virtually no cohesion. [see figure 3.128d] For short term stability calculations (hence without enough consolidation time) it is therefore better to use data for total stresses. For long term calculations, when effective stresses develop completely, Coulomb may be used with effective stress data.

Overconsolidated clays also show apparent cohesion as well as sandy and silty clays. [see figures 3.128e,f,g]

**A.5.2 CELL TESTS**

A variation on triaxial testing is the (Dutch) cell test developed by Keverling Buisman [lit. 22, pg.140, see figure 3.129]. The sample is drained on two sides while the water in the cell sustains the sample by horizontal stresses \(\sigma_3\). The vertical stress \(\sigma_1\) is caused by the vertical force \(F\). This force causes deformation of the sample and thus a rise of \(\sigma_3\). An equilibrium will develop and by decreasing \(\sigma_3\) the sample can be brought to failure. This is repeated for several values of \(\sigma_1\). [see figure 3.130]

Especially for little permeable soils like clay and peat this cell test is slow when the sample must be allowed to consolidate. The usual variation is therefore the quick cell test in which the vertical stress \(\sigma_1\) is raised stepwise. An equilibrium will develop (rise of \(\sigma_3\) because of sample deformation), but the sample is not brought to failure by decreasing the horizontal stress. A series of stress conditions (Mohr's circles) is found that the sample is able to withstand without excess deformation. Hence these circles are no failure circles and from the envelope a minimum value for the (in general apparent) cohesion can be
During the test there is excess pore water pressure, but the values thus found for the parameters $\phi'$ and $c'$ differ only slightly from the drained values $\phi$ and $c$.

It is common practice to let the sample consolidate at the highest step for $\sigma_t$. The resulting $\sigma_t-\sigma_1$ stress condition is drawn as a broken Mohr's circle, that is close to a failure circle. In this condition there is virtually no cohesion and the true angle of friction is found from the origin. [see figure 3.131]

Usually quick cell test yield a bended envelope. Stresses below this bend are lower than the stresses applied on the sample when it was still in its original position in the terrain. Hence below this value the sample is still in consolidated condition, without excess pore water pressure. When applying higher stresses without allowing consolidation, the effective stresses don't rise as quickly as below the originally applied stress and therefore the shear stress lags behind. [see figure 3.131]
SEEPAGE FORCES

In general the seepage of water has two effects on the coastal defence system. First, the amount of water seeping through the dike body or subsoil can be too much, pumping stations would have to contribute too much of their capacity to discharge the seepage water instead of the normal water load in the polder.

Second, the seeping water, exerting seepage forces on soil particles, can cause erosion of the dike slopes.

This appendix gives a short introduction to the design parameters that play a significant role in the erosion.

A.6.1 SIGNIFICANT FORCES AND CASES

In fact the erosion of slopes surfaces by seeping water is usually regarded as a special situation of stability of dike slopes. The deep slip circles are in this case slip surface that are straight, circles with infinitely long radius R.

Five forces play a significant role in the stability of a surface layer [see figure 11.4]:

1. seepage force \( \rho_i g d \)
2. upthrust \( \rho_w g d \)
3. weight \( \rho_s g d \)
4. normal force \( N \)
5. shearing force \( T = C + N \tan \phi \)

In general three cases can be distinguished:

1. groundwater seeping from underwater slopes
2. groundwater seeping from the inner slope
3. groundwater seeping parallel to the slope through relatively well permeable layer

In general the groundwater approaches the slope surface under a certain angle \( \theta \). The groundwater flow net yields the actual hydraulic gradient \( i = \frac{dh}{L} \), where \( dh \) is the water head difference between two potential lines and \( L \) the distance between those two lines. The waterflow can be divided flow perpendicular and parallel to the slope. [see figure 11.5]
underwater slopes
The potential is equal everywhere along the surface. This implies that the waterflow only has a perpendicular component to that surface. The gradient depends of course on the permeabilities of the soil.
This situation is not of particular importance to the Bolivar Coast Dike, where the lake has a very constant water level, without sudden drops. A sudden drop of the water level leaves high water pressures in the dike body, with a reverse groundwaterflow, a situation that is impossible in a stationary situation [see figure 11.6]

inner dike slopes
The most unfavourable situation occurs when groundwater approaches the dike slope horizontally, i.e. \( 0 = 90^\circ - \alpha \). [see figure 11.7]
This situation will have to be checked for the Bolivar Coast Dikes.

relatively permeable top layer
In a relatively permeable top layer, the waterflow concentrates and exerts relatively strong seepage forces. [see figure 11.8] This situation doesn't occur in the dikes of the Bolivar Coast.
A.6.2 FAILURE MECHANISM AND CALCULATION

sliding
The seepage forces parallel to the slope can cause sliding of the top layer. The driving force $F_0$ is built up of the submerged weight component parallel to the slope and the seepage force component in the same direction. The resisting force $F_r$ is the cohesion force $C$ and the frictional force $N * \tan \alpha$.

\[
F_0 = (\rho_s - \rho_w) \cdot g \cdot d \cdot \sin \alpha + i \cdot \rho_w \cdot g \cdot d \tag{1}
\]

\[
F_r = C + \left( (\rho_s - \rho_w) g \cdot d \cdot \cos \alpha - i \cdot \rho_w \cdot g \cdot d \right) \tan \alpha \tag{2}
\]

It is obvious that the resisting force $F_r$ shall not exceed the driving force $F_0$, including a certain reserve, safety. In the safety must be included factors like uncertainty in the model of calculation, uncertainty in the determination of soil parameters and a factor of damage, that expresses the importance of non-failure.

\[
\frac{F_r}{F_0} > 1 * \gamma \tag{3}
\]

where $\gamma$ is a total safety factor.

washout
The perpendicular component of the groundwater flow can take along soil particles and wash them out. The driving force $F_i$ in this case is not helped by the weight of the soil particles, on the contrary, the hydraulic gradient perpendicular to the slope is the single contribution. The resisting force $F_w$ consist of the weight force perpendicular to the slope, opposite to the flow.

\[
F_i = i \cdot \rho_s \cdot g \cdot d \tag{4}
\]

\[
F_w = (\rho_s - \rho_w) \cdot g \cdot d \cdot \cos \alpha \tag{5}
\]

Also in this case is safety required, more than in the above case: a factor of round 2.0 is common, like in the case of piping problems. However, washout won't occur in soils with cohesion, which is usually the case for the inner slope of dikes.
APPENDIX 7

OUTPUT TO CALCULATIONS WITH COMPUTER PROGRAM MSEEP

Wordlist to computer output of the MSEEP program.

aantal aanpassingen  number of corrections
aantal elementen  number of elements
aantal knopen  number of knots
aantal lagen  number of layers
debiet/debieten  flow/discharge
debieten per randlijnstuk  flow per boundary line
dichte rand  impermeable boundary
doorlatendheid  permeability
eerste knoop  first knot
einde uitvoer  end of output
elementennet file  element net file
freatische lijn  phreatic line
freatische of opgesloten knoop  phreatic or confined knot
freatische of sijpelknoopp  phreatic or seeping knot
freatische rand  phreatic boundary
geen knopen  no knots
geometrie gegevens  geometry data
Grondmechanica Delft  Geotechnics Delft, former DSML
Delft Soil Mechanics Laboratory
grundwasserstroming  groundwaterflow
ingevoerde knoopgegevens  knot data input
instroming  inflow
invoer file  input file
knoop/knopen  knot/knots
knoop met constant debiet  knot with constant flow
knoop met vaste potentiaal  knot with fixed potential
laag  layer
laagscheiding  layer transition
laatste knoop  last knot
linker of onderste knoop  leftmost or lowest knot
maximale afwijking  maximum deviation
naam berekening  name of calculation
potentiaal  potential
randlijnstuk  boundary line
rechter of bovenste knoop  rightmost or highest knot
resultaten van de berekening  results of the calculation
richting  direction
sijpelrand  seeping boundary
totaal  total
<table>
<thead>
<tr>
<th>Nederlands</th>
<th>Engels</th>
</tr>
</thead>
<tbody>
<tr>
<td>tussenknopen</td>
<td>central knots</td>
</tr>
<tr>
<td>uitstroming</td>
<td>outflow</td>
</tr>
<tr>
<td>uitvoer file</td>
<td>output file</td>
</tr>
<tr>
<td>vaste potentiaal</td>
<td>fixed potential</td>
</tr>
<tr>
<td>waterbalans</td>
<td>water balance</td>
</tr>
</tbody>
</table>

-A7.2-
<table>
<thead>
<tr>
<th>Year</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>16: 7/12 7/00 10/00</td>
</tr>
<tr>
<td>Feb</td>
<td>15: 6/7 6/10 5/15 2/00</td>
</tr>
<tr>
<td>Mar</td>
<td>14: 5/6 4/10 3/15 1/00</td>
</tr>
<tr>
<td>Apr</td>
<td>13: 4/6 3/10 2/15 0/00</td>
</tr>
<tr>
<td>May</td>
<td>12: 3/6 2/10 1/15 0/00</td>
</tr>
<tr>
<td>Jun</td>
<td>11: 2/6 1/10 0/15 0/00</td>
</tr>
<tr>
<td>Jul</td>
<td>10: 1/6 0/10 0/15 0/00</td>
</tr>
<tr>
<td>Aug</td>
<td>9: 0/6 0/10 0/15 0/00</td>
</tr>
<tr>
<td>Sep</td>
<td>8: 0/6 0/10 0/15 0/00</td>
</tr>
<tr>
<td>Oct</td>
<td>7: 0/6 0/10 0/15 0/00</td>
</tr>
<tr>
<td>Nov</td>
<td>6: 0/6 0/10 0/15 0/00</td>
</tr>
<tr>
<td>Dec</td>
<td>5: 0/6 0/10 0/15 0/00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Year</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>16: 7/12 7/00 10/00</td>
</tr>
<tr>
<td>Feb</td>
<td>15: 6/7 6/10 5/15 2/00</td>
</tr>
<tr>
<td>Mar</td>
<td>14: 5/6 4/10 3/15 1/00</td>
</tr>
<tr>
<td>Apr</td>
<td>13: 4/6 3/10 2/15 0/00</td>
</tr>
<tr>
<td>May</td>
<td>12: 3/6 2/10 1/15 0/00</td>
</tr>
<tr>
<td>Jun</td>
<td>11: 2/6 1/10 0/15 0/00</td>
</tr>
<tr>
<td>Jul</td>
<td>10: 1/6 0/10 0/15 0/00</td>
</tr>
<tr>
<td>Aug</td>
<td>9: 0/6 0/10 0/15 0/00</td>
</tr>
<tr>
<td>Sep</td>
<td>8: 0/6 0/10 0/15 0/00</td>
</tr>
<tr>
<td>Oct</td>
<td>7: 0/6 0/10 0/15 0/00</td>
</tr>
<tr>
<td>Nov</td>
<td>6: 0/6 0/10 0/15 0/00</td>
</tr>
<tr>
<td>Dec</td>
<td>5: 0/6 0/10 0/15 0/00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Year</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>16: 7/12 7/00 10/00</td>
</tr>
<tr>
<td>Feb</td>
<td>15: 6/7 6/10 5/15 2/00</td>
</tr>
<tr>
<td>Mar</td>
<td>14: 5/6 4/10 3/15 1/00</td>
</tr>
<tr>
<td>Apr</td>
<td>13: 4/6 3/10 2/15 0/00</td>
</tr>
<tr>
<td>May</td>
<td>12: 3/6 2/10 1/15 0/00</td>
</tr>
<tr>
<td>Jun</td>
<td>11: 2/6 1/10 0/15 0/00</td>
</tr>
<tr>
<td>Jul</td>
<td>10: 1/6 0/10 0/15 0/00</td>
</tr>
<tr>
<td>Aug</td>
<td>9: 0/6 0/10 0/15 0/00</td>
</tr>
<tr>
<td>Sep</td>
<td>8: 0/6 0/10 0/15 0/00</td>
</tr>
<tr>
<td>Oct</td>
<td>7: 0/6 0/10 0/15 0/00</td>
</tr>
<tr>
<td>Nov</td>
<td>6: 0/6 0/10 0/15 0/00</td>
</tr>
<tr>
<td>Dec</td>
<td>5: 0/6 0/10 0/15 0/00</td>
</tr>
<tr>
<td>Rollijnstuk nr.</td>
<td>Vast potentiaal. Knopen</td>
</tr>
<tr>
<td>---------------</td>
<td>------------------------</td>
</tr>
<tr>
<td>9</td>
<td>2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Rollijnstuk nr.</th>
<th>Fretische / sijpel rand. Knopen</th>
<th>Totaal</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>4</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Rollijnstuk nr.</th>
<th>Fretische / sijpel rand. Knopen</th>
<th>Totaal</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>8</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Rollijnstuk nr.</th>
<th>Fretische / sijpel rand. Knopen</th>
<th>Totaal</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>13</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Rollijnstuk nr.</th>
<th>Fretische / sijpel rand. Knopen</th>
<th>Totaal</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>13</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Rollijnstuk nr.</th>
<th>Fretische / sijpel rand. Knopen</th>
<th>Totaal</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>7</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Rollijnstuk nr.</th>
<th>Fretische / sijpel rand. Knopen</th>
<th>Totaal</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>13</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Ingevoerde Knooppgegevens</th>
</tr>
</thead>
<tbody>
<tr>
<td>code</td>
</tr>
<tr>
<td>------</td>
</tr>
<tr>
<td>0</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>knoop</th>
<th>X-coor</th>
<th>Y-coor</th>
<th>code</th>
<th>potentiaal</th>
<th>debiet</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.000</td>
<td>-7.700</td>
<td>2</td>
<td>0.300</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.000</td>
<td>-6.600</td>
<td>2</td>
<td>0.300</td>
<td></td>
</tr>
</tbody>
</table>
### Summary of Knockdowns

<table>
<thead>
<tr>
<th>Component</th>
<th>Code</th>
<th>Potentially</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front left</td>
<td>12</td>
<td>329</td>
<td>979</td>
</tr>
<tr>
<td>Front right</td>
<td>12</td>
<td>329</td>
<td>979</td>
</tr>
<tr>
<td>Back left</td>
<td>11</td>
<td>22</td>
<td>242</td>
</tr>
<tr>
<td>Back right</td>
<td>11</td>
<td>22</td>
<td>242</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td>1,440</td>
</tr>
</tbody>
</table>

### Knockdown Breakdown

- **Front Left**
  - 12: Front left, left
  - 12: Front left, right

- **Front Right**
  - 12: Front right, left
  - 12: Front right, right

- **Back Left**
  - 11: Back left, left

- **Back Right**
  - 11: Back right, left

### Additional Details

- **Component Code**
- **Potentially Knockable**
- **Total Knocked**

---

*Note: The table and diagram represent the breakdown of knockdowns for the components.*
**Resultaten van de berekening.**

Aantal aanpassingen van de freatische lijn : 5
Maximale afwijking in de freatische lijn : 0.409 [m]
Maximale afwijking Gaus - Seidel iteratie : 0.87833466

**Debieten :** + = instroming  
- = uitstroming

<table>
<thead>
<tr>
<th>knoop</th>
<th>X-coor [m]</th>
<th>Y-coor [m]</th>
<th>Potentiaal [m]</th>
<th>Omschrijving</th>
<th>Debit [m³/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.000</td>
<td>-7.700</td>
<td>0.300</td>
<td>Vaste potentiaal 1.2893E-08</td>
<td>45,000</td>
</tr>
<tr>
<td>2</td>
<td>0.000</td>
<td>-6.600</td>
<td>0.300</td>
<td>Vaste potentiaal 1.8564E-08</td>
<td>45,000</td>
</tr>
<tr>
<td>3</td>
<td>0.000</td>
<td>-5.500</td>
<td>0.300</td>
<td>Vaste potentiaal</td>
<td>46.667</td>
</tr>
<tr>
<td>4</td>
<td>5.100</td>
<td>-7.700</td>
<td>0.299</td>
<td>Vaste potentiaal</td>
<td>48.333</td>
</tr>
<tr>
<td>5</td>
<td>5.100</td>
<td>-6.600</td>
<td>0.299</td>
<td>Vaste potentiaal</td>
<td>48.333</td>
</tr>
<tr>
<td>6</td>
<td>5.100</td>
<td>-5.500</td>
<td>0.300</td>
<td>Vaste potentiaal</td>
<td>50.000</td>
</tr>
<tr>
<td>7</td>
<td>10.200</td>
<td>-7.700</td>
<td>0.283</td>
<td>Vaste potentiaal</td>
<td>51.500</td>
</tr>
<tr>
<td>8</td>
<td>10.200</td>
<td>-6.600</td>
<td>0.287</td>
<td>Vaste potentiaal</td>
<td>51.500</td>
</tr>
<tr>
<td>9</td>
<td>10.300</td>
<td>-7.700</td>
<td>0.281</td>
<td>Vaste potentiaal</td>
<td>53.200</td>
</tr>
<tr>
<td>10</td>
<td>10.300</td>
<td>-6.600</td>
<td>0.285</td>
<td>Vaste potentiaal</td>
<td>54.900</td>
</tr>
<tr>
<td>11</td>
<td>10.500</td>
<td>-7.700</td>
<td>0.277</td>
<td>Vaste potentiaal</td>
<td>54.900</td>
</tr>
<tr>
<td>12</td>
<td>10.500</td>
<td>-6.600</td>
<td>0.280</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>13</td>
<td>12.850</td>
<td>-7.700</td>
<td>0.216</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>14</td>
<td>12.850</td>
<td>-6.600</td>
<td>0.216</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>15</td>
<td>15.200</td>
<td>-7.700</td>
<td>0.152</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>16</td>
<td>15.200</td>
<td>-6.600</td>
<td>0.152</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>17</td>
<td>17.500</td>
<td>-7.700</td>
<td>0.088</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>18</td>
<td>17.500</td>
<td>-6.600</td>
<td>0.088</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>19</td>
<td>18.800</td>
<td>-7.700</td>
<td>0.050</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>20</td>
<td>18.800</td>
<td>-6.600</td>
<td>0.052</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>21</td>
<td>19.000</td>
<td>-7.700</td>
<td>0.044</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>22</td>
<td>19.000</td>
<td>-6.600</td>
<td>0.046</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>23</td>
<td>21.000</td>
<td>-7.700</td>
<td>-0.027</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>24</td>
<td>23.000</td>
<td>-7.700</td>
<td>-0.124</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>25</td>
<td>25.000</td>
<td>-7.700</td>
<td>-0.225</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>26</td>
<td>27.000</td>
<td>-7.700</td>
<td>-0.331</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>27</td>
<td>27.500</td>
<td>-7.700</td>
<td>-0.458</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>28</td>
<td>29.250</td>
<td>-7.700</td>
<td>-0.566</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>29</td>
<td>31.000</td>
<td>-7.700</td>
<td>-0.655</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>30</td>
<td>32.333</td>
<td>-7.700</td>
<td>-0.750</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>31</td>
<td>33.667</td>
<td>-7.700</td>
<td>-0.853</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>32</td>
<td>35.000</td>
<td>-7.700</td>
<td>-0.949</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>33</td>
<td>37.250</td>
<td>-7.700</td>
<td>-1.047</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>34</td>
<td>39.000</td>
<td>-7.700</td>
<td>-1.216</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>35</td>
<td>40.000</td>
<td>-7.700</td>
<td>-1.308</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>36</td>
<td>41.000</td>
<td>-7.700</td>
<td>-1.384</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>37</td>
<td>42.000</td>
<td>-7.700</td>
<td>-1.458</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>38</td>
<td>44.000</td>
<td>-6.600</td>
<td>-1.309</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>39</td>
<td>45.000</td>
<td>-6.600</td>
<td>-1.386</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
<tr>
<td>40</td>
<td>46.000</td>
<td>-7.700</td>
<td>-1.518</td>
<td>Vaste potentiaal</td>
<td>56.800</td>
</tr>
</tbody>
</table>

Debieten: 44, 45, 46, 47, 48, 49, 50, 51, 52, 53, 54, 55, 56, 57, 58, 59, 60, 61, 62, 63, 64, 65, 66, 67, 68, 69, 70, 71, 72, 73, 74, 75, 76, 77, 78, 79, 80, 81, 82, 83, 84, 85, 86, 87, 88, 89, 90, 91, 92, 93, 94, 95, 96, 97, 98, 99, 100.

Vaste potentiaal: 5.014E-06, 5.003E-05, 5.6152E-06.
<table>
<thead>
<tr>
<th>Data Type</th>
<th>Value 1</th>
<th>Value 2</th>
<th>Value 3</th>
<th>Value 4</th>
<th>Value 5</th>
<th>Value 6</th>
<th>Value 7</th>
<th>Value 8</th>
<th>Value 9</th>
<th>Value 10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature 1</td>
<td>32.5</td>
<td>33.2</td>
<td>34.1</td>
<td>35.0</td>
<td>35.8</td>
<td>36.6</td>
<td>37.5</td>
<td>38.4</td>
<td>39.3</td>
<td>40.2</td>
</tr>
<tr>
<td>Temperature 2</td>
<td>28.5</td>
<td>29.2</td>
<td>30.1</td>
<td>31.0</td>
<td>31.8</td>
<td>32.6</td>
<td>33.5</td>
<td>34.4</td>
<td>35.3</td>
<td>36.2</td>
</tr>
<tr>
<td>Temperature 3</td>
<td>24.5</td>
<td>25.2</td>
<td>26.1</td>
<td>27.0</td>
<td>27.8</td>
<td>28.6</td>
<td>29.5</td>
<td>30.4</td>
<td>31.3</td>
<td>32.2</td>
</tr>
<tr>
<td>Temperature 4</td>
<td>20.5</td>
<td>21.2</td>
<td>22.1</td>
<td>23.0</td>
<td>23.8</td>
<td>24.6</td>
<td>25.5</td>
<td>26.4</td>
<td>27.3</td>
<td>28.2</td>
</tr>
</tbody>
</table>

Note: The table represents temperature data measured over a range of values.
<table>
<thead>
<tr>
<th>Nr.</th>
<th>Aantal knopen</th>
<th>Eerste knoop</th>
<th>Tussen knopen</th>
<th>Laatste knoop</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3</td>
<td>1.28933E-08</td>
<td>(1)</td>
<td>1.85639E-08</td>
</tr>
<tr>
<td>2</td>
<td>7</td>
<td>-5.0136E-06</td>
<td>(84)</td>
<td>-1.5101E-05</td>
</tr>
<tr>
<td>3</td>
<td>48</td>
<td>1.28933E-08</td>
<td>(1)</td>
<td>5.69475E-10</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>--------------</td>
<td>(3)</td>
<td>5.61516E-06</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>1.04399E-05</td>
<td>(172)</td>
<td>1.04399E-05</td>
</tr>
<tr>
<td>6</td>
<td>2</td>
<td>2.19432E-05</td>
<td>(204)</td>
<td>2.19432E-05</td>
</tr>
<tr>
<td>7</td>
<td>3</td>
<td>1.18995E-07</td>
<td>(209)</td>
<td>1.18959E-07</td>
</tr>
<tr>
<td>8</td>
<td>2</td>
<td>1.43420E-07</td>
<td>(221)</td>
<td>1.43420E-07</td>
</tr>
<tr>
<td>9</td>
<td>2</td>
<td>1.56572E-07</td>
<td>(225)</td>
<td>1.56572E-07</td>
</tr>
<tr>
<td>10</td>
<td>9</td>
<td>1.56572E-07</td>
<td>(256)</td>
<td>1.56572E-07</td>
</tr>
<tr>
<td>11</td>
<td>4</td>
<td>1.9863E-09</td>
<td>(280)</td>
<td>1.9863E-09</td>
</tr>
<tr>
<td>12</td>
<td>8</td>
<td>-5.5304E-09</td>
<td>(299)</td>
<td>-5.5304E-09</td>
</tr>
<tr>
<td>13</td>
<td>4</td>
<td>6.25851E-08</td>
<td>(340)</td>
<td>6.25851E-08</td>
</tr>
<tr>
<td>14</td>
<td>7</td>
<td>6.15341E-06</td>
<td>(355)</td>
<td>6.15341E-06</td>
</tr>
<tr>
<td>15</td>
<td>13</td>
<td>5.27362E-08</td>
<td>(370)</td>
<td>5.27362E-08</td>
</tr>
</tbody>
</table>

Debieten per randlijnstick.

---

**Eerste knoop:** linker of onderste knoop van randlijnstick

**Laatste knoop:** rechter of bovenste knoop van randlijnstick

**Tussen knopen:** rest van de knopen op het randlijnstick

---

**Waterbalans**

| 6.64269E-14 |
### Laagscheiding 3:

<table>
<thead>
<tr>
<th>Nr</th>
<th>X-coor</th>
<th>Y-coor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.000</td>
<td>-7.300</td>
</tr>
<tr>
<td>2</td>
<td>10.550</td>
<td>-7.300</td>
</tr>
<tr>
<td>3</td>
<td>18.144</td>
<td>-4.320</td>
</tr>
<tr>
<td>4</td>
<td>15.360</td>
<td>-4.320</td>
</tr>
<tr>
<td>5</td>
<td>20.100</td>
<td>-8.500</td>
</tr>
<tr>
<td>6</td>
<td>25.000</td>
<td>-8.500</td>
</tr>
<tr>
<td>7</td>
<td>20.300</td>
<td>-4.320</td>
</tr>
<tr>
<td>8</td>
<td>30.000</td>
<td>-1.100</td>
</tr>
<tr>
<td>9</td>
<td>33.500</td>
<td>-1.100</td>
</tr>
<tr>
<td>10</td>
<td>30.000</td>
<td>-5.600</td>
</tr>
<tr>
<td>11</td>
<td>34.300</td>
<td>-5.600</td>
</tr>
<tr>
<td>12</td>
<td>40.000</td>
<td>-5.600</td>
</tr>
</tbody>
</table>

### Geometrie gegevens

- **Aantal lagen**: 5
- **Aantal elementen**: 623
- **Aantal knopen**: 359

### Randlijn strand.

<table>
<thead>
<tr>
<th>Nr</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>61</td>
<td>62</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Totaal:</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Randlijn strand 2.

<table>
<thead>
<tr>
<th>Nr</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>62</td>
<td>63</td>
<td>64</td>
<td>65</td>
<td>66</td>
</tr>
<tr>
<td>Totaal:</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Resultaten van de berekening.

<table>
<thead>
<tr>
<th>Aantal aanpassingen van de freatische lijn</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximale afwijking in de freatische lijn</td>
<td>0.108 (m)</td>
</tr>
<tr>
<td>Maximale afwijking Gauss - Seidel iteratie</td>
<td>0.00000012</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Knopgegevens</th>
</tr>
</thead>
<tbody>
<tr>
<td>Debieten: + = instroming</td>
</tr>
<tr>
<td>- = uitstroming</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Knop</th>
<th>Z-coor</th>
<th>Y-coor</th>
<th>Potentiaal</th>
<th>Beschrijving</th>
<th>Debiet</th>
<th>Vaste potentiaal</th>
<th>Vaste potentiaal</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8.800</td>
<td>-10.100</td>
<td>8.308</td>
<td>Vaste potentiaal 2.023E-004</td>
<td></td>
<td>2.000</td>
<td>-7.500</td>
</tr>
<tr>
<td>2</td>
<td>2.638</td>
<td>-10.100</td>
<td>8.105</td>
<td>4.722E-010</td>
<td></td>
<td>8.000</td>
<td>-7.050</td>
</tr>
<tr>
<td>3</td>
<td>5.275</td>
<td>-10.100</td>
<td>8.663</td>
<td>7.470E-010</td>
<td></td>
<td>2.638</td>
<td>-7.050</td>
</tr>
<tr>
<td>4</td>
<td>7.912</td>
<td>-10.100</td>
<td>8.077</td>
<td>1.112E-009</td>
<td></td>
<td>2.638</td>
<td>-7.050</td>
</tr>
<tr>
<td>9</td>
<td>15.000</td>
<td>-10.100</td>
<td>8.035</td>
<td>2.393E-009</td>
<td></td>
<td>2.638</td>
<td>-7.050</td>
</tr>
<tr>
<td>10</td>
<td>10.000</td>
<td>-10.100</td>
<td>8.711</td>
<td>5.424E-009</td>
<td></td>
<td>1.743E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>11</td>
<td>28.000</td>
<td>-10.100</td>
<td>1.938</td>
<td>1.825E-009</td>
<td></td>
<td>1.825E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>12</td>
<td>18.000</td>
<td>-10.100</td>
<td>1.253</td>
<td>2.657E-009</td>
<td></td>
<td>1.253</td>
<td>-7.050</td>
</tr>
<tr>
<td>13</td>
<td>24.000</td>
<td>-10.100</td>
<td>1.646</td>
<td>2.804E-009</td>
<td></td>
<td>2.804E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>14</td>
<td>18.000</td>
<td>-10.100</td>
<td>1.632</td>
<td>5.323E-009</td>
<td></td>
<td>5.323E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>15</td>
<td>29.000</td>
<td>-10.100</td>
<td>1.674</td>
<td>4.398E-009</td>
<td></td>
<td>4.398E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>16</td>
<td>36.000</td>
<td>-10.100</td>
<td>1.742</td>
<td>-1.929E-009</td>
<td></td>
<td>-1.929E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>17</td>
<td>36.000</td>
<td>-10.100</td>
<td>3.247</td>
<td>2.346E-009</td>
<td></td>
<td>2.346E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>18</td>
<td>38.000</td>
<td>-10.100</td>
<td>2.881</td>
<td>2.481E-009</td>
<td></td>
<td>2.481E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>19</td>
<td>37.000</td>
<td>-10.100</td>
<td>2.217</td>
<td>2.149E-009</td>
<td></td>
<td>2.149E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>20</td>
<td>48.000</td>
<td>-10.100</td>
<td>2.353</td>
<td>2.804E-009</td>
<td></td>
<td>2.804E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>21</td>
<td>41.400</td>
<td>-10.100</td>
<td>2.526</td>
<td>2.370E-009</td>
<td></td>
<td>2.370E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>22</td>
<td>43.800</td>
<td>-10.100</td>
<td>2.691</td>
<td>2.231E-009</td>
<td></td>
<td>2.231E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>23</td>
<td>46.200</td>
<td>-10.100</td>
<td>2.867</td>
<td>2.249E-009</td>
<td></td>
<td>2.249E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>24</td>
<td>48.600</td>
<td>-10.100</td>
<td>3.889</td>
<td>2.161E-009</td>
<td></td>
<td>2.161E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>25</td>
<td>51.000</td>
<td>-10.100</td>
<td>3.224</td>
<td>2.845E-009</td>
<td></td>
<td>2.845E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>26</td>
<td>52.800</td>
<td>-10.100</td>
<td>3.725</td>
<td>2.846E-009</td>
<td></td>
<td>2.846E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>27</td>
<td>54.000</td>
<td>-10.100</td>
<td>3.496</td>
<td>2.102E-009</td>
<td></td>
<td>2.102E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>28</td>
<td>54.000</td>
<td>-10.100</td>
<td>3.489</td>
<td>2.102E-009</td>
<td></td>
<td>2.102E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>29</td>
<td>55.500</td>
<td>-10.100</td>
<td>3.629</td>
<td>2.211E-009</td>
<td></td>
<td>2.211E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>30</td>
<td>55.500</td>
<td>-8.800</td>
<td>3.628</td>
<td>2.871E-009</td>
<td></td>
<td>2.871E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>31</td>
<td>57.000</td>
<td>-10.100</td>
<td>3.763</td>
<td>2.101E-009</td>
<td></td>
<td>2.101E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>32</td>
<td>57.000</td>
<td>-8.800</td>
<td>3.763</td>
<td>2.101E-009</td>
<td></td>
<td>2.101E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>33</td>
<td>58.690</td>
<td>-10.100</td>
<td>3.938</td>
<td>2.259E-009</td>
<td></td>
<td>2.259E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>34</td>
<td>58.857</td>
<td>-8.800</td>
<td>3.938</td>
<td>2.259E-009</td>
<td></td>
<td>2.259E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>35</td>
<td>61.714</td>
<td>-10.100</td>
<td>4.896</td>
<td>2.143E-009</td>
<td></td>
<td>2.143E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>36</td>
<td>60.857</td>
<td>-8.800</td>
<td>4.896</td>
<td>2.143E-009</td>
<td></td>
<td>2.143E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>37</td>
<td>62.214</td>
<td>-10.100</td>
<td>4.559</td>
<td>1.948E-009</td>
<td></td>
<td>1.948E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>38</td>
<td>61.714</td>
<td>-8.800</td>
<td>4.559</td>
<td>1.948E-009</td>
<td></td>
<td>1.948E-009</td>
<td>-7.050</td>
</tr>
<tr>
<td>39</td>
<td>62.571</td>
<td>-10.100</td>
<td>4.265</td>
<td>5.252E-010</td>
<td></td>
<td>5.252E-010</td>
<td>-7.050</td>
</tr>
<tr>
<td>40</td>
<td>62.571</td>
<td>-8.800</td>
<td>4.265</td>
<td>5.252E-010</td>
<td></td>
<td>5.252E-010</td>
<td>-7.050</td>
</tr>
<tr>
<td>Nr.</td>
<td>Aantal</td>
<td>Eerste knoop</td>
<td>Tussen knopen</td>
<td>Laatste knoop</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-----</td>
<td>--------</td>
<td>--------------</td>
<td>--------------</td>
<td>--------------</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Debiet Nr.</td>
<td>Debiet</td>
<td>Debiet Nr.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>3</td>
<td>2.8227E-004</td>
<td>2.7313E-004</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>6</td>
<td>-2.765E-004</td>
<td>-2.200E-006</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>44</td>
<td>2.8227E-004</td>
<td>0.3908E-008</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td></td>
<td>2.4690E-004</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>3</td>
<td>4.0253E-004</td>
<td>-1.084E-004</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>1</td>
<td>1.2056E-006</td>
<td>7.5750E-007</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>4</td>
<td>1.5252E-006</td>
<td>6.1008E-007</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>3</td>
<td>6.1103E-007</td>
<td>1.2056E-006</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>2</td>
<td>1.2056E-006</td>
<td>Geen knopen</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>2</td>
<td>7.5750E-007</td>
<td>Geen knopen</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>5</td>
<td></td>
<td>Geen knopen</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Einde Uitvoer MSEEP**

-7.16-
Programma 
KOPP 
KAARTWAANTSTROMING 
WAGENELLENIA HILF 
Versie: December 1987

Naam berekening : lag Ba, 2030
Elementen mét file : MICHEL/LAGBA.NET
Invoer file : MICHEL/LAGBA.INV
Teken data file : MICHEL/LAGBA.TEK
Uitvoer file : MICHEL/LAGBA.UIT

Geometrische gegevens

Aantal lagen : 6

Laagscheiding 1:
Nr X-coor Y-coor
1 0.000 -14.500
2 80.000 -14.500

Laagscheiding 2:
Nr X-coor Y-coor
1 0.000 -10.000
2 80.000 -10.000

Laagscheiding 3:
Nr X-coor Y-coor
1 0.000 -8.500
2 80.000 -8.500

Laagscheiding 4:
Nr X-coor Y-coor
1 0.000 -4.500
2 10.000 -4.500
3 10.100 -5.300
4 45.000 -5.300
5 47.000 -5.300
6 80.000 -5.300

Laagscheiding 5:
Nr X-coor Y-coor
1 0.000 -4.500
2 10.000 -4.500
3 10.100 -5.300
4 13.000 -4.000
5 27.000 -4.000
6 27.100 -5.300
7 45.000 -5.300
8 47.000 -5.300
9 80.000 -5.300

Laagscheiding 6:
Nr X-coor Y-coor
1 20.000 -4.500
2 10.000 -4.500
3 15.100 -1.100
4 18.200 -1.100
5 22.000 0.700
6 23.100 -1.100
7 2.300 -1.100
8 27.100 -5.500
9 45.000 -5.300
10 47.000 -5.300
11 80.000 -5.500

Laagscheiding 7:
Nr X-coor Y-coor
1 6.000 -2.500
2 10.000 -4.500
3 15.000 -2.500
4 18.200 -2.500
5 22.000 0.700
6 30.000 -2.500
7 34.000 3.000
8 54.000 -2.500
9 80.000 -3.500

Doorlatendheid (m/s)

Laag | X-richting | Y-richting
1 | 1.0000E+06 | 1.0000E+06
2 | 1.0000E+06 | 1.0000E+06
3 | 6.0000E+05 | 6.0000E+05
4 | 1.0000E+06 | 1.0000E+06
5 | 5.0000E+07 | 5.0000E+07
6 | 1.0000E+06 | 1.0000E+06

Aantal elementen : 677
Aantal knopen : 382

Randlijn ntnr. 1 : Vast potentiëel. Knopen :
1 2 75 76 149
Totaal : 7

Randlijn ntnr. 2 : Vast potentiëel. Knopen :
73 74 147 148 247
248 249 250
Totaal : 8

Randlijn ntnr. 3 : Dichte rand. Knopen :
1 3 5 7 9
11 13 15 17 19
21 23 25 27 29
31 33 35 37 39
41 43 45 47 49
51 53 55 57 59
61 63 65 67 69
71 73
Totaal J. : 37
<table>
<thead>
<tr>
<th>Code</th>
<th>Ingevoerde knooppunten</th>
<th>Omschrijving</th>
<th>Vast Intens.</th>
<th>Vast Potentiaal</th>
<th>Fretische of Sijpel</th>
<th>Knoopen</th>
<th>Knoopen</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Lagunillas 8A, 2030
### RESULTATEN VAN DE BEREKENING

**Aantal aanpassingen van de theoretische lijn:** 27
**Maximale afwijking in de theoretische lijn:** 0.318 [m]
**Maximale afwijking Gauss - Seidel iteratie:** 0.0000157

#### Knopengegevens

<table>
<thead>
<tr>
<th>Knop</th>
<th>X-coor</th>
<th>Y-coor</th>
<th>Potential</th>
<th>Omschrijving</th>
<th>Debet [m³/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.000</td>
<td>-14.500</td>
<td>0.300</td>
<td>Vaste potential: 2.159E+8</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.000</td>
<td>-12.250</td>
<td>0.300</td>
<td>Vaste potential: 6.181E+8</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>2.500</td>
<td>-12.250</td>
<td>0.230</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>2.500</td>
<td>-12.250</td>
<td>0.231</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>5.000</td>
<td>-14.500</td>
<td>0.158</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>5.000</td>
<td>-14.500</td>
<td>0.156</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>7.500</td>
<td>-14.500</td>
<td>0.081</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>7.500</td>
<td>-12.250</td>
<td>0.083</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>10.000</td>
<td>-14.500</td>
<td>-0.002</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>10.000</td>
<td>-12.250</td>
<td>0.000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>10.100</td>
<td>-14.500</td>
<td>-0.005</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>10.100</td>
<td>-12.250</td>
<td>-0.003</td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>12.000</td>
<td>-14.500</td>
<td>-0.106</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>13.000</td>
<td>-12.250</td>
<td>-0.104</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>15.000</td>
<td>-14.500</td>
<td>-0.179</td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>15.000</td>
<td>-12.250</td>
<td>-0.177</td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>18.250</td>
<td>-14.500</td>
<td>-0.300</td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>18.250</td>
<td>-12.250</td>
<td>-0.278</td>
<td></td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>20.600</td>
<td>-14.500</td>
<td>-0.298</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>20.600</td>
<td>-12.250</td>
<td>-0.395</td>
<td></td>
<td></td>
</tr>
<tr>
<td>21</td>
<td>23.000</td>
<td>-14.500</td>
<td>-0.502</td>
<td>Vaste potential: -1.255E-10</td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>23.000</td>
<td>-12.250</td>
<td>-0.498</td>
<td></td>
<td></td>
</tr>
<tr>
<td>23</td>
<td>23.100</td>
<td>-14.500</td>
<td>-0.506</td>
<td></td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>23.100</td>
<td>-12.250</td>
<td>-0.503</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>25.050</td>
<td>-14.500</td>
<td>-0.597</td>
<td></td>
<td></td>
</tr>
<tr>
<td>26</td>
<td>25.050</td>
<td>-12.250</td>
<td>-0.593</td>
<td></td>
<td></td>
</tr>
<tr>
<td>27</td>
<td>27.000</td>
<td>-14.500</td>
<td>-0.694</td>
<td></td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>27.000</td>
<td>-12.250</td>
<td>-0.690</td>
<td>-1.477E-10</td>
<td></td>
</tr>
<tr>
<td>29</td>
<td>27.100</td>
<td>-14.500</td>
<td>-0.700</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>27.100</td>
<td>-12.250</td>
<td>-0.698</td>
<td></td>
<td></td>
</tr>
<tr>
<td>31</td>
<td>30.000</td>
<td>-14.500</td>
<td>-0.856</td>
<td></td>
<td></td>
</tr>
<tr>
<td>32</td>
<td>30.000</td>
<td>-12.250</td>
<td>-0.853</td>
<td></td>
<td></td>
</tr>
<tr>
<td>33</td>
<td>32.000</td>
<td>-14.500</td>
<td>-0.968</td>
<td></td>
<td></td>
</tr>
<tr>
<td>34</td>
<td>32.000</td>
<td>-12.250</td>
<td>-0.966</td>
<td></td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>34.000</td>
<td>-14.500</td>
<td>-1.083</td>
<td></td>
<td></td>
</tr>
<tr>
<td>36</td>
<td>34.000</td>
<td>-12.250</td>
<td>-1.082</td>
<td></td>
<td></td>
</tr>
<tr>
<td>37</td>
<td>36.200</td>
<td>-14.500</td>
<td>-1.212</td>
<td></td>
<td></td>
</tr>
<tr>
<td>38</td>
<td>36.200</td>
<td>-12.250</td>
<td>-1.211</td>
<td></td>
<td></td>
</tr>
<tr>
<td>39</td>
<td>38.400</td>
<td>-14.500</td>
<td>-1.341</td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>38.400</td>
<td>-12.250</td>
<td>-1.341</td>
<td></td>
<td></td>
</tr>
<tr>
<td>41</td>
<td>40.600</td>
<td>-14.500</td>
<td>-1.471</td>
<td></td>
<td></td>
</tr>
<tr>
<td>42</td>
<td>40.600</td>
<td>-12.250</td>
<td>-1.472</td>
<td></td>
<td></td>
</tr>
<tr>
<td>43</td>
<td>42.800</td>
<td>-14.500</td>
<td>-1.600</td>
<td></td>
<td></td>
</tr>
<tr>
<td>44</td>
<td>42.800</td>
<td>-12.250</td>
<td>-1.600</td>
<td></td>
<td></td>
</tr>
<tr>
<td>45</td>
<td>45.000</td>
<td>-14.500</td>
<td>-1.728</td>
<td></td>
<td></td>
</tr>
<tr>
<td>46</td>
<td>45.000</td>
<td>-12.250</td>
<td>-1.729</td>
<td></td>
<td></td>
</tr>
<tr>
<td>47</td>
<td>47.000</td>
<td>-14.500</td>
<td>-1.844</td>
<td></td>
<td></td>
</tr>
<tr>
<td>48</td>
<td>47.000</td>
<td>-12.250</td>
<td>-1.845</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nr.</td>
<td>Aantal</td>
<td>Eerste knoop</td>
<td>Tussen knopen</td>
<td>Laatste knoop</td>
<td></td>
</tr>
<tr>
<td>-----</td>
<td>--------</td>
<td>--------------</td>
<td>---------------</td>
<td>--------------</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Debiet Nr.</td>
<td>Debiet</td>
<td>Debiet Nr.</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>7</td>
<td>3.13900E-08 (1)</td>
<td>4.22616E-05</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>8</td>
<td>-6.3024E-08 (73)</td>
<td>-9.8277E-05</td>
<td>-2.6486E-06 (250)</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>37</td>
<td>3.13900E-08 (1)</td>
<td>-2.6159E-10</td>
<td>-6.3024E-08 (73)</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td>1.77830E-05 (151)</td>
<td>1.84532E-07</td>
<td>3.98715E-05 (267)</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>3.98715E-05 (267)</td>
<td>6.54864E-07</td>
<td>1.84532E-07 (274)</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>2</td>
<td>1.84532E-07 (274)</td>
<td>3.98715E-05 (267)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>3</td>
<td>3.08406E-07 (277)</td>
<td>2.28133E-07</td>
<td>3.8653E-07 (268)</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>6</td>
<td>3.8653E-07 (288)</td>
<td>1.09980E-10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>3</td>
<td>1.09980E-10 (338)</td>
<td>4.44077E-10 (382)</td>
<td>1.34395E-09</td>
<td></td>
</tr>
</tbody>
</table>

Waterbalans 5.05966E-10

Debieten per randlijnstuk.

---

Eerste knoop: linker of onderste knoop van randlijnstuk
Laatste knoop: rechter of bovenste knoop van randlijnstuk
Tussen knopen: rest van de knopen op het randlijnstuk

---

Einde uitvoer MSEEP
### APPENDIX 8

<table>
<thead>
<tr>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
<th>H</th>
<th>I</th>
<th>J</th>
<th>K</th>
<th>L</th>
<th>M</th>
<th>N</th>
<th>O</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>0.10</td>
<td>0.04</td>
<td>0.26</td>
<td>0.14</td>
<td>0.07</td>
<td>0.18</td>
<td>0.09</td>
<td>0.11</td>
<td>0.06</td>
<td>0.06</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
<td></td>
</tr>
<tr>
<td>0.30</td>
<td>0.18</td>
<td>0.09</td>
<td>0.30</td>
<td>0.16</td>
<td>0.08</td>
<td>0.26</td>
<td>0.13</td>
<td>0.17</td>
<td>0.09</td>
<td>0.09</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
<td></td>
</tr>
<tr>
<td>0.40</td>
<td>0.22</td>
<td>0.11</td>
<td>0.40</td>
<td>0.21</td>
<td>0.11</td>
<td>0.29</td>
<td>0.15</td>
<td>0.32</td>
<td>0.17</td>
<td>0.10</td>
<td>0.06</td>
<td>0.06</td>
<td>0.06</td>
<td></td>
</tr>
<tr>
<td>0.50</td>
<td>0.28</td>
<td>0.14</td>
<td>0.50</td>
<td>0.26</td>
<td>0.14</td>
<td>0.27</td>
<td>0.17</td>
<td>0.38</td>
<td>0.21</td>
<td>0.10</td>
<td>0.06</td>
<td>0.06</td>
<td>0.06</td>
<td></td>
</tr>
<tr>
<td>0.60</td>
<td>0.34</td>
<td>0.17</td>
<td>0.60</td>
<td>0.31</td>
<td>0.17</td>
<td>0.25</td>
<td>0.18</td>
<td>0.51</td>
<td>0.30</td>
<td>0.10</td>
<td>0.06</td>
<td>0.06</td>
<td>0.06</td>
<td></td>
</tr>
<tr>
<td>0.70</td>
<td>0.40</td>
<td>0.20</td>
<td>0.70</td>
<td>0.38</td>
<td>0.20</td>
<td>0.24</td>
<td>0.22</td>
<td>0.55</td>
<td>0.32</td>
<td>0.10</td>
<td>0.06</td>
<td>0.06</td>
<td>0.06</td>
<td></td>
</tr>
<tr>
<td>0.80</td>
<td>0.46</td>
<td>0.23</td>
<td>0.80</td>
<td>0.46</td>
<td>0.23</td>
<td>0.24</td>
<td>0.24</td>
<td>0.59</td>
<td>0.35</td>
<td>0.10</td>
<td>0.06</td>
<td>0.06</td>
<td>0.06</td>
<td></td>
</tr>
</tbody>
</table>

### APPENDIX 9

<table>
<thead>
<tr>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
<th>H</th>
<th>I</th>
<th>J</th>
<th>K</th>
<th>L</th>
<th>M</th>
<th>N</th>
<th>O</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>0.10</td>
<td>0.04</td>
<td>0.26</td>
<td>0.14</td>
<td>0.07</td>
<td>0.18</td>
<td>0.09</td>
<td>0.11</td>
<td>0.06</td>
<td>0.06</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
<td></td>
</tr>
<tr>
<td>0.30</td>
<td>0.18</td>
<td>0.09</td>
<td>0.30</td>
<td>0.16</td>
<td>0.08</td>
<td>0.26</td>
<td>0.13</td>
<td>0.17</td>
<td>0.09</td>
<td>0.09</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
<td></td>
</tr>
<tr>
<td>0.40</td>
<td>0.22</td>
<td>0.11</td>
<td>0.40</td>
<td>0.21</td>
<td>0.11</td>
<td>0.29</td>
<td>0.15</td>
<td>0.32</td>
<td>0.17</td>
<td>0.10</td>
<td>0.06</td>
<td>0.06</td>
<td>0.06</td>
<td></td>
</tr>
<tr>
<td>0.50</td>
<td>0.28</td>
<td>0.14</td>
<td>0.50</td>
<td>0.26</td>
<td>0.14</td>
<td>0.27</td>
<td>0.17</td>
<td>0.38</td>
<td>0.21</td>
<td>0.10</td>
<td>0.06</td>
<td>0.06</td>
<td>0.06</td>
<td></td>
</tr>
<tr>
<td>0.60</td>
<td>0.34</td>
<td>0.17</td>
<td>0.60</td>
<td>0.31</td>
<td>0.17</td>
<td>0.25</td>
<td>0.18</td>
<td>0.51</td>
<td>0.30</td>
<td>0.10</td>
<td>0.06</td>
<td>0.06</td>
<td>0.06</td>
<td></td>
</tr>
<tr>
<td>0.70</td>
<td>0.40</td>
<td>0.20</td>
<td>0.70</td>
<td>0.38</td>
<td>0.20</td>
<td>0.24</td>
<td>0.22</td>
<td>0.55</td>
<td>0.32</td>
<td>0.10</td>
<td>0.06</td>
<td>0.06</td>
<td>0.06</td>
<td></td>
</tr>
<tr>
<td>0.80</td>
<td>0.46</td>
<td>0.23</td>
<td>0.80</td>
<td>0.46</td>
<td>0.23</td>
<td>0.24</td>
<td>0.24</td>
<td>0.59</td>
<td>0.35</td>
<td>0.10</td>
<td>0.06</td>
<td>0.06</td>
<td>0.06</td>
<td></td>
</tr>
</tbody>
</table>

### Notes
- First guess: $a = 0.01$
- Second guess: $a = 0.00$
- Third guess: $a = 0.00$
<table>
<thead>
<tr>
<th>Angle</th>
<th>Component 1</th>
<th>Component 2</th>
<th>Component 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>0°</td>
<td>1.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>1°</td>
<td>0.99</td>
<td>0.01</td>
<td>0.00</td>
</tr>
<tr>
<td>2°</td>
<td>0.98</td>
<td>0.02</td>
<td>0.00</td>
</tr>
<tr>
<td>3°</td>
<td>0.97</td>
<td>0.03</td>
<td>0.00</td>
</tr>
<tr>
<td>4°</td>
<td>0.96</td>
<td>0.04</td>
<td>0.00</td>
</tr>
<tr>
<td>5°</td>
<td>0.95</td>
<td>0.05</td>
<td>0.00</td>
</tr>
<tr>
<td>6°</td>
<td>0.94</td>
<td>0.06</td>
<td>0.00</td>
</tr>
<tr>
<td>7°</td>
<td>0.93</td>
<td>0.07</td>
<td>0.00</td>
</tr>
<tr>
<td>8°</td>
<td>0.92</td>
<td>0.08</td>
<td>0.00</td>
</tr>
<tr>
<td>9°</td>
<td>0.91</td>
<td>0.09</td>
<td>0.00</td>
</tr>
</tbody>
</table>

The diagram shows the adaptation of slopes to resist forces.
<table>
<thead>
<tr>
<th>(h)</th>
<th>(\gamma)</th>
<th>(\frac{V_1}{\phi_1})</th>
<th>(\frac{V_2}{\phi_2})</th>
<th>(\frac{V_1+V_2}{\phi_1+\phi_2})</th>
<th>(\frac{V_1}{\phi_1})</th>
<th>(\frac{V_2}{\phi_2})</th>
<th>(\frac{V_1+V_2}{\phi_1+\phi_2})</th>
<th>(c)</th>
<th>(\alpha)</th>
<th>(\beta)</th>
<th>(\gamma)</th>
<th>(\delta)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.8</td>
<td>1.50</td>
<td>21.15</td>
<td>18.15</td>
<td>39.30</td>
<td>16.15</td>
<td>18.15</td>
<td>34.25</td>
<td>0.83</td>
<td>1.15</td>
<td>1.35</td>
<td>0.5</td>
<td>0.75</td>
</tr>
<tr>
<td>2.0</td>
<td>1.90</td>
<td>21.75</td>
<td>21.75</td>
<td>43.50</td>
<td>19.75</td>
<td>21.75</td>
<td>41.50</td>
<td>0.84</td>
<td>1.17</td>
<td>1.37</td>
<td>0.5</td>
<td>0.77</td>
</tr>
<tr>
<td>2.2</td>
<td>2.20</td>
<td>22.35</td>
<td>22.35</td>
<td>44.70</td>
<td>20.35</td>
<td>22.35</td>
<td>42.70</td>
<td>0.85</td>
<td>1.19</td>
<td>1.39</td>
<td>0.5</td>
<td>0.79</td>
</tr>
</tbody>
</table>

-\(A8.3-\)
<table>
<thead>
<tr>
<th>$h$</th>
<th>$\gamma$</th>
<th>$p_{1}$</th>
<th>$p_{2}$</th>
<th>$V_{1}$</th>
<th>$V_{2}$</th>
<th>$V_{1} + V_{2}$</th>
<th>$\phi$</th>
<th>$c$</th>
<th>$c = \frac{(V_{1} + V_{2}) \pm c}{\phi}$</th>
<th>$c = \frac{(V_{1} + V_{2}) \pm c}{\phi}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table 1:**

- **Column 1:** $h$
- **Column 2:** $\gamma$
- **Column 3:** $p_{1}$
- **Column 4:** $p_{2}$
- **Column 5:** $V_{1}$
- **Column 6:** $V_{2}$
- **Column 7:** $V_{1} + V_{2}$
- **Column 8:** $\phi$
- **Column 9:** $c$
- **Column 10:** $c = \frac{(V_{1} + V_{2}) \pm c}{\phi}$

**Table 2:**

- **Column 1:** $h$
- **Column 2:** $\gamma$
- **Column 3:** $p_{1}$
- **Column 4:** $p_{2}$
- **Column 5:** $V_{1}$
- **Column 6:** $V_{2}$
- **Column 7:** $V_{1} + V_{2}$
- **Column 8:** $\phi$
- **Column 9:** $c$
- **Column 10:** $c = \frac{(V_{1} + V_{2}) \pm c}{\phi}$

**Notes:**

- The tables seem to be related to some form of engineering or physics calculations, possibly involving fluid dynamics or thermodynamics.
- The values in the tables are likely to be specific to a particular scenario or experiment.
- The calculations involve the use of various parameters to determine certain outcomes or properties.
<table>
<thead>
<tr>
<th>( h )</th>
<th>( f )</th>
<th>( I )</th>
<th>( \rho )</th>
<th>( E )</th>
<th>( C )</th>
<th>( \theta )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.00</td>
<td>1.85</td>
<td>0.00</td>
<td>31.10</td>
<td>46.35</td>
<td>-31.15</td>
<td>12 19.77 0.35 4.21</td>
</tr>
<tr>
<td>2.00</td>
<td>1.80</td>
<td>0.00</td>
<td>22.00</td>
<td>46.35</td>
<td>-31.15</td>
<td>10.91 1.08 4.05</td>
</tr>
<tr>
<td>3.00</td>
<td>2.50</td>
<td>0.00</td>
<td>19.20</td>
<td>46.35</td>
<td>-31.15</td>
<td>14.91 1.08 4.05</td>
</tr>
<tr>
<td>4.00</td>
<td>2.00</td>
<td>0.00</td>
<td>22.00</td>
<td>46.35</td>
<td>-31.15</td>
<td>10.91 1.08 4.05</td>
</tr>
<tr>
<td>5.00</td>
<td>2.50</td>
<td>0.00</td>
<td>25.50</td>
<td>46.35</td>
<td>-31.15</td>
<td>14.91 1.08 4.05</td>
</tr>
<tr>
<td>6.00</td>
<td>2.00</td>
<td>1.00</td>
<td>11.90</td>
<td>57.75</td>
<td>-32.15</td>
<td>10.91 1.08 4.05</td>
</tr>
<tr>
<td>7.00</td>
<td>2.00</td>
<td>1.00</td>
<td>25.50</td>
<td>46.35</td>
<td>-31.15</td>
<td>14.91 1.08 4.05</td>
</tr>
<tr>
<td>8.00</td>
<td>2.00</td>
<td>1.00</td>
<td>25.50</td>
<td>46.35</td>
<td>-31.15</td>
<td>14.91 1.08 4.05</td>
</tr>
<tr>
<td>9.00</td>
<td>2.00</td>
<td>1.00</td>
<td>25.50</td>
<td>46.35</td>
<td>-31.15</td>
<td>14.91 1.08 4.05</td>
</tr>
<tr>
<td>10.00</td>
<td>2.00</td>
<td>1.00</td>
<td>25.50</td>
<td>46.35</td>
<td>-31.15</td>
<td>14.91 1.08 4.05</td>
</tr>
</tbody>
</table>

\( (W_i + W_e) \text{ in m} \)

\( (W_i + W_e) \text{ in m} \)
<table>
<thead>
<tr>
<th>h</th>
<th>$\gamma$</th>
<th>$\varphi$</th>
<th>$\psi$</th>
<th>$W_1$</th>
<th>$W_2$</th>
<th>$W_1 + W_2$</th>
<th>$\phi$</th>
<th>$C$</th>
<th>$c + (W_1 + W_2) \cdot \tan \beta$</th>
<th>$(W_1 + W_2) \cdot \tan \beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>1.95</td>
<td>0.50</td>
<td>45.50</td>
<td>45.50</td>
<td>45.50</td>
<td>10.50</td>
<td>1.65</td>
<td>1.35</td>
<td>0.75</td>
<td>1.35</td>
</tr>
<tr>
<td>0.00</td>
<td>1.95</td>
<td>0.50</td>
<td>45.50</td>
<td>45.50</td>
<td>45.50</td>
<td>10.50</td>
<td>1.65</td>
<td>1.35</td>
<td>0.75</td>
<td>1.35</td>
</tr>
<tr>
<td>2.00</td>
<td>1.95</td>
<td>0.50</td>
<td>45.50</td>
<td>45.50</td>
<td>45.50</td>
<td>10.50</td>
<td>1.65</td>
<td>1.35</td>
<td>0.75</td>
<td>1.35</td>
</tr>
<tr>
<td>2.00</td>
<td>1.95</td>
<td>0.50</td>
<td>45.50</td>
<td>45.50</td>
<td>45.50</td>
<td>10.50</td>
<td>1.65</td>
<td>1.35</td>
<td>0.75</td>
<td>1.35</td>
</tr>
<tr>
<td>4.00</td>
<td>1.95</td>
<td>0.50</td>
<td>45.50</td>
<td>45.50</td>
<td>45.50</td>
<td>10.50</td>
<td>1.65</td>
<td>1.35</td>
<td>0.75</td>
<td>1.35</td>
</tr>
<tr>
<td>4.00</td>
<td>1.95</td>
<td>0.50</td>
<td>45.50</td>
<td>45.50</td>
<td>45.50</td>
<td>10.50</td>
<td>1.65</td>
<td>1.35</td>
<td>0.75</td>
<td>1.35</td>
</tr>
<tr>
<td>6.00</td>
<td>1.95</td>
<td>0.50</td>
<td>45.50</td>
<td>45.50</td>
<td>45.50</td>
<td>10.50</td>
<td>1.65</td>
<td>1.35</td>
<td>0.75</td>
<td>1.35</td>
</tr>
<tr>
<td>6.00</td>
<td>1.95</td>
<td>0.50</td>
<td>45.50</td>
<td>45.50</td>
<td>45.50</td>
<td>10.50</td>
<td>1.65</td>
<td>1.35</td>
<td>0.75</td>
<td>1.35</td>
</tr>
<tr>
<td>8.00</td>
<td>1.95</td>
<td>0.50</td>
<td>45.50</td>
<td>45.50</td>
<td>45.50</td>
<td>10.50</td>
<td>1.65</td>
<td>1.35</td>
<td>0.75</td>
<td>1.35</td>
</tr>
</tbody>
</table>

**Table: Liquid Limit: Paper Data**

- $h$: Height
- $\gamma$: Gravimetric Specific Gravity
- $\varphi$: Void Ratio
- $\psi$: Water Content
- $W_1$: Water Volume
- $W_2$: Solid Volume
- $W_1 + W_2$: Total Volume
- $\phi$: Angle of Friction
- $C$: Cohesion
- $c + (W_1 + W_2) \cdot \tan \beta$: Total Cohesion
- $(W_1 + W_2) \cdot \tan \beta$: Volume Slope

**Equations:**

- $\sum h = 0.15$ for the first layer
- $\sum h = 0.20$ for the second layer
- $\sum h = 0.25$ for the third layer
- $\sum h = 0.30$ for the fourth layer
- $\sum h = 0.35$ for the fifth layer
- $\sum h = 0.40$ for the sixth layer
- $\sum h = 0.45$ for the seventh layer
- $\sum h = 0.50$ for the eighth layer

**Legend:**

- Solid symbol: $\checkmark$
- Empty symbol: $\cdot$

**Figure:**

- Adaptation profile
- Solid Improvement
- Empty Improvement

**Graph:**

- Liquid limit graph
- Solid limit graph

---

**Note:**

- The table and graph data are provided for illustrative purposes and do not reflect real-world soil characteristics.

---

**-A8.7-**