

draft

Commision of the European Communities
Guyana Sea Defence Lome IV Project
Essequibo/West Demerara Locations

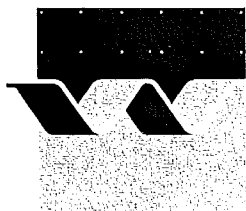
Investigation of slip failures at
Henrietta and Anna Regina

Delft Hydraulics
Delft Geotechnics

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DELFT GEOTECHNICS



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Executive summary

More than six kilometers of rehabilitation works on sea defences at the Essequibo and West Demerara coast of Guyana has been constructed within one year. This shows the fast construction of the riprap revetment design. The performance of the structure was good until the last month of construction when two slip failures occurred of about 100 m length each. The slip failures occurred at Henrietta and Anna Regina.

It is a well known fact that the coast of Guyana consists of (very) weak clay layers and various slip failures have happened in the past, and will probably happen again. That two slip failures occurred does not mean that the riprap design is not a good design. It shows that (also) with this design indeed slip failures are possible and that one should be aware of this possibility and take measures if required. An advantage of the riprap design is that a slip failure is fairly easy to repair, although not according to the original design slope.

The slip failures were reason for the Commission of the European Communities to invite two experts to investigate these slip failures. Dr J.W. van der Meer of Delft Hydraulics and Mr. J.M. de Wit of Delft Geotechnics, The Netherlands, visited Guyana from 23 September till 5 October 1996. The main conclusions are given in this Executive Summary. More detailed conclusions are given in various sections of the report. The final Chapter 5 gives recommendations for future riprap works in Guyana, which are not repeated here.

The main conclusions are as follows:

1. The slip failures of the slopes at Henrietta and Anna Regina are due to the generation of excess pore pressures in the underlying clay, which led to an inadmissible decrease of the shear resistance of the clay. There has been a number of factors that had an adverse effect on the actual shear resistance of the underlying soils in the area of the sliding plane.
2. The capacity of the existing subsoil to bear the loads imposed by raising the embankment depends on the speed or rate of construction in height. If raising embankments on clayey soils one should consider the risk for instabilities due to the existence of excess pore pressures, especially in soft clays. In this case the raising of the total embankment up to 4.9 m high at once, should be considered high. With tides of around 3 m twice per day, however, there is no other way than to construct directly to the full height. It should be noted that the construction rate *in height* is important, not the construction rate in the longitudinal direction.
3. It is the *combination* of circumstances that has caused the slip failures. These circumstances are:
 - the heavy rainfall during the whole construction period
 - the areas with possibly weaker soil, indicated by the flooding and deformation of the cofferdams
 - the removal of the sheet piles by means of vibration
 - the presence of critical cross-sections with respect to loading (amount of fill and difference between crest level and foreshore level).All these circumstances are described in more detail in Chapter 3.
4. The stability reports GSEC (1995-1 and 1995-2) are incomplete and unsuitable to draw conclusions from with respect to the degree of safety against sliding. These reports were accepted by the Supervisor. Based on the data in these reports Delft Geotechnics made

stability and settlement analyses. From these calculation results it can be concluded, that the degree of stability of the critical cross-sections in Area B during construction was close to 1.0. A calculation with the low values obtained from the soil investigations after the Henrietta slip failure (GSEC, 1996) showed clearly that weaker soil can lead to slip failure. The probability of occurrence of such a slip failure was increased by the removal of the sheet piles by vibration.

5. The high safety factors on stability as given in GSEC (1995-2) have given the impression that slip failures would not occur, that a slope of 1:3 was completely safe. This may have led to a lack of awareness that certain circumstances could decrease the stability in such a way that slip failures would occur. Awareness would have led to more attention for critical cross-sections, locally soft soil conditions, etc.
6. Normally, the analyses of soil investigations and the subsequent conclusions on settlement and stability are performed by a consultant or supervisor. Neither the tender for supervision services, nor the contract with the supervisor describes this work being part of the supervisor's duty. This omission may be caused by the following facts. DHV made only the tender documents for the contractor. Therefore, PEU had to make the tender documents for the supervision services. PEU has technical assistance from Halcrow, but no soil expertise is present within PEU. As the geotechnical analysis was not part of the tender documents for the supervisor, the supervisor did not offer these activities, nor were the analyses part of the contract for supervision. During the work the Contractor was ordered to perform the analyses, although it was not his task to take responsibility for the design.
7. Due to a contradiction in the Tender Dossier on the number of boreholes to be performed under the Contract it was discussed whether it was essential to carry out two boreholes per cross-section, see Section 2.2.4. The reason to perform boreholes at the inner side only has been given in personal communication with Mr. P.A.D. Allsopp, soil expert of SRKN'gineering. Mr. Allsopp has performed soil investigations between 1950 and 1970, at inner as well as at seaward locations. During these investigations he found that the subsoils present at the inner and the seaward side of the seawall were similar. Based on this experience he advised to perform soil investigations only at the inner side of the seawall. According to Mr. Allsopp the soils present at the shore side of the embankment have the same origin as the soils at the landside, as during time sea attack has resulted in withdrawal of the shore line. We have no reason to doubt these experiences, although it would have been better to sink at least at one or two locations a borehole at the seaward side of the seawall for verification.
8. Delft Geotechnics made settlement calculations based on the parameters of borehole 1B2, see GSEC (1995-1). A total settlement of 0.6-0.8 m is found and half of this settlement will occur within four to five months after construction. According to these calculations, the crest at the inner edge of the rip rap protection will have settled about 0.3-0.4 m and will, after mid-October 1996, still settle in the order of another 0.3-0.4 m. The total settlement accounted for during the construction was 0.2 m. The profile measurements after the slip failures were also done at sections where there was no slip failure. Based on an as built crest height of 19.1 m GD the average settlement of the crest, determined from the measurements, amounts to 0.30 m. Although the data for calculations and from measurements are scarce, it is likely that the settlement of the sea defence is larger than expected. It is, therefore, recommended to check as soon as possible the position of the crest as well as the top of the rip rap protection by levelling according to Georgetown Datum in order to establish whether the crest level is still in agreement with the design requirements.

9. The size of armour rock on the sea defence has been checked. It can be concluded that the armour rock is much too small, compared with the specifications. Instead of 100-600 kg rock with a mean weight of 300 kg, a 60-400 kg rock class is present with a mean weight of 150 kg. The difference in behaviour between 150 kg and 300 kg rock is well described by the DHV Tender Documents. In general more damage to the armour layer can be expected during high tides and waves, and probably more maintenance has to be done.
10. The slip failures occurred under certain unfavourable circumstances. Although further slip failures in the constructed work can never be completely excluded, they are not expected. The sea defence becomes geotechnically more stable with time. As long as flooding is prevented it is not necessary to take extra temporary measures at the slip failure areas. Only measures should be taken at the transition to the vertical wall near the sluice at Anna Regina.
11. Permanent measures are fairly easy to perform. It is proposed *not* to reconstruct the slipped areas according to the original design, but to leave the slope as it is and to extend slope up to a sufficient crest level by applying the present inclination. This crest level can be lower than the original crest level as a gentler slope has less wave runup. The proposed measures are described in Section 4.2.

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1 Terms of reference

The Commission of the European Communities represented by the Head of the Delegation in Guyana and on the agreement of the Government of the Co-operative Republic of Guyana, represented by the Minister of Agriculture, submitted a contract to Delft Hydraulics in order to investigate the slip failures at Henrietta and Anna Regina. These slip failures took place during sea defence construction works under the Lome IV Project along the Guyana coast.

The objectives of this investigation, as described by the Client, were:

- To ascertain the reasons that have caused the failures, including unforeseeable ground conditions, inappropriate construction methods, errors or omissions in the execution of the works, etc., etc.
- Review the slip investigation carried out by GSEC and comment on their postulated mode of failure and recommended remedial measures.
- Review all other existing data that could be of importance to clarify the circumstances under which the failures occurred.
- Comment on the significance of the omission from the testing programme of boreholes, as specified by DHV, on the foreshore at the toe of the old seawall.
- Carry out an investigation of the sea defence design's appropriateness for the shoreline existing in Guyana.
- Recommend of any additional testings that should be carried out in the slip areas before a final decision is taken on any remedial measures.
- Recommend in detail what alternative remedial measures can be taken to secure the foreshore in the slip areas, in the interim period until repairs are effected.
- Make recommendations on permanent repair works at the failure sites.
- Comment on the basis of existing data on the likelihood of other similar failures occurring in the constructed work.

The investigation was performed by means of a two weeks stay of two experts in Guyana, followed by one week of preparation of the investigations report in their head office. The experts who visited Guyana from 23 September till 5 October 1996 were Dr. J.W. van der Meer, hydraulic and structural expert of Delft Hydraulics, and Mr. J.M. de Wit, geotechnical expert of Delft Geotechnics. During the visit the following meetings were held and the following persons met:

24 September 1996

- 10.00 Commission of the European Communities, Delegation in Guyana. Mr. A. Baum and Mr. T. Strand.
- 11.00 PEU, Guyana Sea Defences, Project Execution Unit. Mr. N. Mohammed, Project Director; Mr. P. Foroudi, Coastal Engineer (Halcrow); Mr. M.A. Tordoff, Contracts Engineer (Halcrow).
- 14.00 SRKN'gineering. Mr. S.S. Naraine, Chairman; Dr. K. Naraine, Partner.

25 September 1996

- 6.00 Site visit to locations B, C, G and I. Mr. N. Mohammed, Mr. P. Foroudi, Dr. K. Naraine, Mr. F. Toffoli, Construction Company PAC/GELFI.

27 September 1996

- 10.00 PEU. Mr. N. Mohammed, Mr. P. Foroudi and Mr. F. Griffith, Assistant Resident Engineer (SRKN/DCSL).
- 11.00 PEU. Mr. P. Foroudi and Mr. C. Ceres (GSEC - Ground Structures Engineering Consultants).
- 15.00 SRKN'gineering. Mr. S.S. Naraine, Dr. K. Naraine, Mr. F. Griffith and Mr. P. Foroudi.

30 September 1996

- 11.00 GSEC. Mr. C. Ceres (Mr. J.M. de Wit)
- 13.30 SRKN'gineering. Mr. F. Griffith, Dr. K. Naraine (Dr. J.W. van der Meer).
- 15.30 Commission of the European Communities, Delegation in Guyana. Mr. A. Baum and Mr. T. Strand.
- 17.00 SRKN'gineering. Dr. K. Naraine, Mr. P. Foroudi.

1 October 1996

- 10.00 SRKN'gineering. Dr. K. Naraine and Mr. P.A.D. Allsopp, soil expert of SRKN'gineering.

2 October 1996

- 18.00 Convention Hall of Ocean View International hotel. Speeches on riprap design and soil investigations for about 30 people. Minister H. Nokta, Minister of Public Works and Communications. Mr. P. Sanasi, Chairman of the Sea Defence Board.

3 October 1996

- 10.00 Site visit other work under execution by local contractor.
- 14.00 Site visit area I; checking rock grading. Dr. K. Naraine, Mr. F. Griffith (Dr. J.W. van der Meer).

4 October 1996

- 9.00 Commission of the European Communities, Delegation in Guyana. Mr. A. Baum and Mr. T. Strand.

During the meetings a large number of documents, files and drawings were submitted for further study. All the information that has been studied is given under the Chapter References. Without any exception, all the persons met were fully cooperative in providing all the required information, by personal communication or by written material.

2 Slips at Henrietta and Anna Regina

2.1 Introduction

2.1.1 Description of the works and the design

The Government of Guyana is currently undertaking several coastal defence rehabilitation projects. One of these projects is the "Guyana Sea Defence Lome IV Project" - Essequibo/West Demerara Locations. The works are located at the coast of Guyana in Region 2 (Essequibo) and Region 3 (West Demerara), see DHV (May 1993, Annex 2).

The rehabilitation areas in Region 2, Essequibo Coast are:

- Rehabilitation Area B Approximately 1,400 m of sea defence in front of the estates "Richmond", "Henrietta", "Anna Regina" and "Cotton Field".
- Rehabilitation Area C Approximately 1,310 m of sea defence in front of the estates "Land of Plenty", "Three Friends" and "Aberdeen".

In Region 3, West Coast Demerara, these areas are:

- Rehabilitation Area F Approximately 1,325 m of sea defence in front of the estates "Le Destin", "Farm" and "Ruby".
- Rehabilitation Area G Approximately 1,271 m of sea defence in front of the estates "Barnwell", "Philadelphia" and "Vergenoegen".
- Rehabilitation Area I Approximately 1,062 m of sea defence in front of the estates "la Jalousie" and "Windsor Forest".

The condition of the existing sea walls has been described by DHV (October 1993, Volume 4). In the sections of the sea defence to be rehabilitated under the current programme there was a wide variety in the protection works on the existing sea walls. Three basic types were identified, namely:

- Concrete faced slopes, often with a concrete coping and sheet piles at the toe. In some areas the concrete slabs were fairly intact, in other areas they were broken down and reinforced with various types of patch works (concrete fill, sand-cement bags, sheet piles, gabions, rip rap, etc.).
- Sheet pile walls of various types and make. In some areas reinforced with rip rap and other types of patch works.
- Other types of protection like earth dams, rip rap slopes and patch works.

Additional information on the existing situation is given by the report of J.W. Hall. This report is included in DHV (October 1993, Volume 4) as appendix 4.C.

Cross-sections of the design of the rehabilitated sea defence are given in DHV (October 1993, Volume 3). Furthermore, PAC/GELFI (1996) gives plan views and cross-sections of all the areas. Cross-section 119 of Area B is reproduced in Figure 2.1.1. The design crest levels have been adapted for the allowance of subsoil settlements. For Area B this allowance was 0.2, which gave a design crest height of 19.1 m (Georgetown Datum).

The design dike profile is determined by a 3.5 m crest width, an outer slope 1:3 and an inner slope 1:2.5. The design slope protection has a median stone weight of 300 kg with a gradation of 100-600 kg. The layer thickness is 0.8 m, measured vertically. A filter cloth will prevent the washing out of subsoil via the pores in the rock layer. A layer of quarry run of 0.2 m, measured vertically, has been applied between the rock and the filter cloth.

The quarry stone is extended backwards on the crest of the dike over a width of 2 m at crest level. The slope protection is supported over its full height by a water tight earth embankment. Behind the quarry fill clay backfill is placed up to about 0.3 m below the top elevation of the rock. In sections where the foreshore level is substantially above the low water level, the protective layers are embedded or partly embedded in the foreshore soil. Only in sections where the foreshore level is near or below the low water level, the protective layer has been placed on top of the existing bottom. At Section 119 of Area B (Figure 2.1.1) for example, the foreshore level was fairly low (about 14.4 m) and only the filter layer was embedded.

The additional fill material, required to heighten the earth embankment, consisted of fine to medium coarse sand. The dike crest is protected against erosion by a 0.5 m thick clay layer to be rooted with grass. At sections where demolition of the existing protective layers generated debris, the debris has been broken into small pieces and spread near the toe of the existing slope in such a manner that the debris could be covered with an adequate layer of fill material.

2.1.2 Slip Failures at Henrietta and Anna Regina

In Area B slip failures occurred on 2 June and 6 July 1996. A plan view of Area B with the location of cross-sections is given in Figure 2.1.2. The distance between the cross-sections is 10 m.

Subsequent to the completion of the work in the Rehabilitation area B a 90 m long section at the estate "Henrietta" experienced a slip failure on the 2nd of June 1996. The exact location of the slip failure is from Section 81 to 90. The failure has been described in PEU (5 June 1996). The slip failure was evidenced by an about 1.6 m drop at the crest of the slope and outward movement of about 1.3 m. The toe of the sea defence embankment in the failed area had risen about 0.4 m. Directly following the failure a detailed soil investigation has been carried out as described by GSEC, commissioned by PEU. The final report is given in GSEC (1996).

On the 6th of July in the same Rehabilitation area B another 120 m long section failed at the estate "Anna Regina". The exact location is from Section 132 to 144. The distance between both failures was about 400 m. This slip failure has been described by PEU (11 July 1996, PEU/EEC/61/930). Initial investigation showed that the crest had settled some 1.6 m and moved 0.7 m seaward. The toe had risen some 0.2 to 0.3 m. Photo 1 gives both a stable section and the situation after the slip failure at Anna Regina.

2.2 Evaluation of works, excluding soil investigations

2.2.1 Evaluation of cross-sections in all Areas

The plan view of all Areas (B, C, F, G and I) and cross-sections, both before and after construction are given by PAC/GELFI (1996). One of these cross-sections is given in Figure 2.1.1. The rehabilitation is built over the old structure where the existing seawall was demolished. Sand fill was used in order to make the outer slope of 1:3. A slip failure in general will happen if the load exceeds the bearing capacity. Therefore, the larger the load on the existing structure and foreshore, the more critical the cross-section is for slip failure. All cross-sections have to bear the filter layer and armour layer, which are similar in weight for all cross-sections. The only varying measure was the amount of sand fill. This amount of sand fill can give an idea about which cross-sections had to bear the largest loads and were, therefore, most critical for slip failures.

The amount of sand fill for each cross-section is given on the drawings. In Area B cross-section 119 (Figure 2.1.1), for example, it is 27.4 m³/m (MQ = 27.4). All cross-sections of all Areas were analyzed.

Areas C and G gave only low amounts as the foreshore slopes were relatively high at about 15.3 m and 15.0 m, giving a maximum difference between crest height of armour and foreshore level of about 3.5 m and 3.9 m, respectively.

In Areas B, F and I sections were present with similar maximum amounts of sand fill. These sections and crest/foreshore level differences were as follows:

Area B:	Sections 78-84	sand fill: 27-29 m ³ /m	level difference: 4.9 m
	Sections 115-143	sand fill: 27-29 m ³ /m	level difference: 4.8 m
Area F:	Sections 67-82	sand fill: 26-28 m ³ /m	level difference: 4.7 m
Area I:	Sections 26-34	sand fill: 26-28 m ³ /m	level difference: 4.9 m
	Sections 86-92	sand fill: 26-28 m ³ /m	level difference: 4.9 m

The slip failures occurred in Area B, sections 81-90 and 132-144, both within above areas of maximum sand fill and, therefore, maximum loading. The failed area is in total 210 m, where the other part with maximum loading of 320 m did not fail. With respect to loading more critical sections were present than the failed areas. This means that also other sections could have failed, but as that did not happen other circumstances must have been present at the failed sections that caused these failures.

The middle of the Henrietta slip failure consisted of an old breach, reported by J.W. Hall, see DHV (October 1993, Volume 4), appendix 4.C. Such a breach, where the old wall fell down, may indicate a weak spot in the area. Hall mentions a breach area of 46 m. The long section 115-143 in Area B has very little variation in cross-section. Only sections 132-144 failed, where the other part remained stable.

2.2.2 Execution of work in Area B

In SRKN's engineering/DSCL (files, Weekly progress reports from April 1995 till August 1996) the progress of execution of the works has been described. From these files the execution in Area B can be retrieved. First of all it should be noted that the period April - June gave extensive rainfall. Some areas along the river were flooded and the government declared these floodings as a national disaster. From the SRKN-files the following description of the execution of work in Area B can be made. The location of the cofferdams constructed by the contractor are shown on Figure 2.2.1.

The temporary sheet piling commenced on March 26, 1996. In total 120 m was placed till weekend 2 April, from section 115 westwards. There was no rainfall in this week. Till weekend 7 April another 115 m was placed. The sheet piles were present from section 90-115. There was no rainfall in this week. Intermediate sections to the structure were made till weekend 14 April at sections 99 and 107, giving dry working areas from sections 99-115. Rainfall of 2 and 6 mm, respectively, were measured on 8 and 9 April. Till weekend 21 April 160 m of existing sea wall was demolished. On 20 April 13 mm rainfall was measured.

The permanent works started on 22 April. Till weekend 28 April 153 m was completed, being sections 99-115. From Monday till Sunday 94 mm of rainfall was measured, respectively 0, 20, 17, 18, 7, 22 and 10 mm. Till weekend 5 May another 121 m of rehabilitation works was completed, being sections 90-99. Sheet piles were also placed at section 81. The toe from 81-90 was finished. From Monday till Sunday 129 mm of rainfall was measured, respectively 3, 0, 24, 40, 31, 23 and 48 mm. Poor progress during the latter half of the week was due to deteriorated condition of the site access road after the rainfall on 2 May. Further, high spring tides overtopped and flooded the cofferdam twice each day, from 2 to 4 May, causing a considerable halt to the permanent work. The flooded area was 81-90. The spring tides were between 3.0 m and 3.1 m GD.

Till weekend 12 May 66 m rehabilitation works was done in sections 81-90, of which 57 m in the first two days and 9 m the rest of the week. The poor progress from 8 May was due to the lack of preparation of a complete cell for the execution of the permanent works. The rainfall measured from Monday till Sunday was 115 mm, respectively 25, 39, 16, 9, 0, 5, and 21 mm.

Till weekend 19 May another 81 m of rehabilitation works was made, being sections 72-90. The work was done tidally, as no cofferdam could be made, due to a broken down piling pontoon. The rainfall measured from Monday till Sunday was 370 mm, respectively 71, 56, 69, 40, 21, 53 and 60 mm. Heavy rain and the resulting deterioration of the site access and haul roads further contributed to the poor progress. Till weekend 26 May another 120 m of the works was finished, working tidally. This was between sections 61-72. More than 300 mm of rainfall fell this week, giving a total of 940 mm in May.

On weekend 2 June sections 42-61 were made tidally, being 188 m in one week. The contractor was, with an improvement of weather conditions, able to keep the site well supplied with quarry materials. This coupled with the fact that there had been a large supply of boulders from existing sea defences, had allowed the works to proceed with a minimum of disruption. The contractor continued to construct the sea defences by working tidally.

During periods of high water, when little or no work areas were available, the contractor concentrated on building haul roads in advance of the permanent works. Although no cofferdams were made, the sheet pile in front of the works was present. An earth dam was constructed between the works and the sheet piles, formed from excavated material. This earth dam was protected from waves by the sheet piles. Sheet piles were removed from section 81-90 on 29 May. Rainfall this week from Monday till Sunday was 48 mm, respectively 23, 5, 16, 4, 0, 0 and 0 mm.

On Sunday 2 June sections 81-90 slipped. Due to prevailing spring tides the contractor found it necessary to construct a small earth berm to some 50 metres of this area to minimise the effect of an overtopping. The spring tide on 2 June was 3.15 m GD.

Till weekend 9 June 158 m of rehabilitation works were made (sections 29-42). The contractor completed the northern joint and transition on Saturday, 8 June. Operations were then transferred to the south end of the works and construction was commenced in the cofferdam between sections 115 and 123. Some 206 m² of sea defences were made ready. In total 53 mm of rainfall was measured this week, respectively 9, 13, 1, 0, 5, 9 and 16 mm from Monday till Sunday.

Till weekend 16 June 141 m sea defence was constructed (sections 115-130). The general progress of the work was affected by shortages of materials at the work site (quarry materials and sand). In advance of the work 80 m of sheet piles were made (sections 130-138). In addition a sheet pile wall has been driven in line with the southern joint (section 144). A section of the main sheet pile wall to the cofferdam currently in use has started to deform and the sheet piles have settled. This has led to the flooding of the work area at periods of high water. This was expected to lead to some delays next week. The contractor had driven a secondary outer sheet pile wall to protect this area from wave action. The contractor commenced the finishing works to the north of the site on June 15. The rainfall this week was 110 mm, respectively 29, 11, 28, 16, 7, 5 and 14 mm.

Only 75 m of rehabilitation works was finished till weekend 23 June, being sections 130-138. The general progress of the works was affected by continuing poor weather. There were also delays to deliveries of materials. The contractor used reinforced concrete, from the demolished sea wall, to form a sub-base to the haul road. This was then topped of with a thin layer of quarry material. During periods of heavy rainfall the road became badly rutted exposing the old reinforcement. This has caused in excess of 10 punctures on some days, with both wagons being unavailable on some occasions. Further delays had also been caused due to the flooding of the cofferdam area at periods of high water. This has been due to the main sheet pile wall leaning inwards and settling. There was no spring tide and the high tides were around 2.8-2.9 m GD. The driving of sheet piles was completed on 19 June (sections 138-144). Extraction of sheet piles to the previous cofferdam (sections 123-130) were undertaken. There was no progress to the finishing works. The contractor attempted to carry out works to the crest area on Monday. This aspect of the works had to be abandoned as the clay was too soft to allow wagons to pass it. The poor weather conditions meant that it was not possible to place any clay. The rainfall this week was 133 mm from Monday till Sunday respectively, 3, 31, 45, 18, 5, 14 and 17 mm.

Till weekend 30 June the progress was poor. The contractor completed the permanent works on 27 June (sections 138-144). There was no progress in the finishing works. The rainfall this week was 120 mm, from Monday till Sunday respectively, 27, 42, 30, 12, 9, 0 and 0 mm.

The contractor recommenced the finishing works on 2 July and the works were undertaken to the southern end of the site. Till weekend 7 July the contractor had placed clay to 116 m of berm, though only 76 m was 100% complete. In addition 96 m of boulders were placed to 96 m of crest. The works had to be abandoned on Saturday 6 July, following a slip failure in this section of the works. A number of sheet piles to the affected area were extracted on the morning of Friday 5 July. These works were completed by mid-morning and consisted of the removal of the end wall, at section 144, plus some 30 m of main sheet pile wall. It should be noted that there was still some 57 m of sheet pile wall to the general area. These areas in two lengths of 43 and 13 m. On the afternoon of Friday 5 July at 14.30 hours cracks were observed running longitudinally along the top of the clay berm. These extended from the southern end of the works over a length of some 40 m and in places were some 20 mm wide. On the morning of Saturday 6 July, at 5.00 hours, the area had slipped. The rainfall measured from Monday till Sunday was only 12 mm, all on Saturday.

2.2.3 Evaluation of conditions

From the evaluation of critical cross-sections (section 2.2.1) and the description of the works (sections 2.2.2) a few conclusions can be drawn.

- The construction of whole Area B took place during heavy rainfall. This is a worse condition with respect to slip failures than construction during good weather. During construction of sections 61-81 the rainfall was extremely high. These sections, however, did not fail, mainly because the sections were not critical as they had a high foreshore level. The rainfall may have been a worse condition, it can not explain why slip failures occurred at sections 81-90 and 132-144.
- The slip failure areas were the only areas where the cofferdam flooded. The reason for flooding was mainly that the soils supporting the sheet piles were not stable enough. For sections 81-90 this was during spring tide. In sections 130-144 sheet piles deformed and settled under their own weight. Also a second sheet pile had to be placed in sections 130-144 to prevent construction from wave action. The deformation and flooding of the cofferdams gives an indication that at these areas the soil may have been weaker than in other areas.
- The flooding of the areas gives not necessarily worse working conditions. It is similar to working tidally. The flooding may only give the above conclusion that may be weaker soil was present.
- The slip failure areas coincide with the areas between the cofferdams. This leads to the conclusion that it is possible that the cofferdams had an effect on the occurrence of the slip failures. Specially if the first slip failure from sections 81-90 is considered. The section with a critical loading condition (see section 2.2.1) lies between 78 and 84, where the boundary of the slip failure is at section 81, exactly the location of the cofferdam.

- Sheet piles were placed and pulled by a vibrating hammer. The sheet piles in sections 81-90 were removed on 29 May. The slip failure occurred on 2 June after a spring tide of 3.15 m GD. Half of the sheet piles at sections 130-144 were removed when the first cracks were observed immediately after that. The slip failure occurred one day later. Pulling by vibration, just in front of the toe (and of the slip circle) has decreased the resistance of the soil. The pulling of the sheet piles is one of the causes for the slip failures. It is not the only one, as slip failures happened only in two areas.
- The *combination* of circumstances has caused the slip failures. These circumstances are:
 - the heavy rainfall during the whole construction period
 - the areas with possibly weaker soil, indicated by the flooding and deformation of the cofferdams
 - the pulling out of the sheet piles by vibration
 - the presence of critical cross-sections with respect to loading (amount of sand fill and difference between crest level and foreshore level).

2.2.4 Contractual matters

The tender documents were made by DHV. DHV was not involved in the construction. Tendering was performed by the PEU, both for the contractor and for the supervision. In the contract of the contractor it is described that he should perform soil investigations. These soil investigations have indeed been performed: bore holes were sunk and laboratory tests were performed. The contract states also that during construction the contractor should take measures to prevent slip failures. The contract does not specify that the contractor should *analyze the soil investigations* in order to judge the expected settlement and slope stability, during the execution and during the life time of the structure.

Normally, this analysis of soil investigations and the subsequent conclusions on settlement and stability are performed by a consultant or supervisor. Neither the tender for supervision services, nor the contract with the supervisor describes this work being part of the supervisor's duty. This omission may be caused by the following facts. DHV was not further involved and DHV made only the tender documents for the contractor. Therefore, PEU had to make the tender documents for the supervision services. PEU has technical assistance from Halcrow, but no soil expertise is present within PEU. As the soil analysis was not part of the tender documents for the supervisor, the supervisor did not offer these activities, nor were the analyses part of the contract for supervision.

The supervisor has the opinion that the analysis was the task and the responsibility of the contractor and therefore, the contractor was ordered to perform this analysis. Another way, which was not followed in this case, would have been that the supervisor had discussed this omission in work with PEU and had offered to do the analysis as extra work. Now both the contractor (or his representative GSEC) and the supervisor felt not responsible for the analysis of the soil investigation.

Another contractual point is that there has been confusion about the number and locations of boreholes. The technical specifications give two boreholes per cross-section, one on the inner and one on the outer toe of the seawall, both 20 m deep. The bill of quantities, however, suggests one borehole of 30 m. On advice of the supervisor the PEU decided to reduce the number of boreholes and to sink boreholes only at the inner toe of the seawall. This means that no data have become available of subsoil at the seaward side of the seawall. The effect of this decision is described in Section 2.3.5.

2.3 Evaluation of soil investigations and analyses

2.3.1 Existing soil investigation data

During the briefing on the geotechnical aspects of the sea defence design a number of documents were made available for evaluation, as discussed below.

Nedeco (1972) describes the results of a soil investigation along the complete Guyana coast. The results indicate that the upper soil layers at the location of the shore line consist of predominantly very soft to soft silty to sandy clay. The thickness of the clay can vary between approximately 6 m and 20 m at that time. The estimated average distance (6 km) between the survey locations from Anna Regina to Skeldon (excluding the Georgetown area), is such that insufficient information can be obtained for detailed construction in the relevant Rehabilitation Areas along the Essequibo and Demerara Coasts.

The CPT resistance (Cone Penetration Test), measured in the soft clays of the upper layers in the total coastal area (excluding the Georgetown area) in Nedeco (1972) predominantly lay between approximately 0.2 MPa and 0.5 MPa. This is a fairly narrow range and, therefore, one may not expect great variations. The results of the settlement and stability calculations in this study, made on the basis of the parameters derived from a laboratory testing programme, are a strong indication for the necessity to verify the stability of the 1:3 slope of the present design.

2.3.2 Geotechnical investigations of rehabilitation areas

Before starting the rehabilitation works a soil investigation programme had to be carried out by the contractor with the following purposes:

- to specify the settlement allowances
- to verify the slope inclinations of the protective layer and the clay layer.

According to the Tender Dossier (DHV, October 1993), Volume 2, part 2.2 - Technical Specifications, two borings were to be made at prescribed locations in the rehabilitation areas. One boring at the foreshore, close to the toe line of the sea wall front slope, and one boring to be made close to the toe line of the land side slope of the sea wall. Further a laboratory testing programme was carried out by the contractor in accordance with Appendix 2.2 D of the Technical Specifications.

The average distances between the locations of the in situ soil investigations (boreholes) carried out by the Contractor under the contract of the Rehabilitation Works in the five areas are approximately:

Area B:	800 m
Area C:	600 m
Area G:	500 m
Area H, I:	600 m

The distances between the borehole locations per area is such that only global information can be obtained on the variation in the soil stratigraphy in longitudinal direction. To exclude practically all insecurities in the subsoil variation the distance required between the boreholes ought to have been in the order of the dike width. Even then deviation areas smaller than the dike width may be not be noticed. However, such small areas will hardly influence the degree of stability of the dike.

The distances chosen between the boreholes, as mentioned in the Specifications, can be considered as reasonable to start with, keeping in mind that in case unexpected variations are detected it is common practice to decide for additional surveys.

In accordance with the Tender Dossier the Contractor carried out the soil investigation programme. The reports of these geotechnical investigations were submitted per Area, see GSEC (1995-1). A summarized description of the field and laboratory investigations, as well as of the findings, is given hereafter.

Field Investigation

The number of boreholes carried out per area amounted to 2 or 3 depending on the longitudinal length of the area. It should be noted that, contrary to the Specifications of the original Tender Dossier, PEU on advice of SRKN'gineering had decided to carry out borings at the landside toe of the sea defence only. So no borings were made at the foreshore toe.

The borehole depth reached was determined by the level at which refusal was encountered due to the presence at such level of very stiff to hard clay. Samples were taken. Standard penetration tests and in situ shear vane tests were performed. All at the levels as specified by the Supervisor. It should be noted that, in the areas G, H, and I the in situ vane tests were not performed at the time of the field investigation because of absence of a field vane in Guyana. The tests were performed later on. Groundwater levels were determined in the boreholes directly and 24 hours after drilling.

Laboratory Investigation

Tests were performed in the laboratory on disturbed as well as on undisturbed samples selected from the borings. The type of the tests and the procedure of testing were in accordance with the specifications. The samples for testing and the consolidation pressures applied for the triaxial tests and the consolidation tests were specified or mandated by the Supervisor.

Soil description

The soil investigation reports indicate that the Guyana Coastal Plain which lies near sea level, is underlain by clays of the Demerara Clay and Coropina Formation. The young Demerara Clays are soft. The underlying Coropina clay is firm to hard. Practically all borings confirm the presence of both the Clays at the investigated locations. The Coropina clay was not encountered at one location (boring 2F2).

There is a considerable variation reported in thickness of the soft Demerara Clay layer found in the Rehabilitation Areas. Within the Areas the variation is less great, except for Area F. The encountered thicknesses and bottom levels of the Demerara Clays are given in the following table, which is a summary taken from the reports.

<i>Area</i>	<i>Boring</i>	<i>thickness</i> m	<i>surface elevation</i> m GD	<i>bottom elevation</i> m GD
B	1B1	7.4	16.46	
	9.06			
	1B2	7	15.85	8.85
C	1C1	7.2	15.85	8.65
G	3G1	9.5	17.36	7.86
	3G2	14	16.70	2.7
	3G3	12.5	18	5.5
F	2F1	8	16.63	8.63
	2F2	17	15.85	1.15
I	3I1	9.5	14.1	4.6
	3I2	7	15.5	8.5

Geotechnical calculations

Following the submission of the factual reports on the geotechnical investigations the Contractor submitted reports presenting the results of geotechnical analyses of the proposed sea defence embankment per Area, see GSEC (1995-2). In these reports the Contractor presents a summary of the soil parameters per distinct soil layer that were determined from the geotechnical investigation results of that specific Area, and per distinct embankment material, which he used as input for the calculation of settlements and stability.

The results of settlement analyses were presented. The total settlements were predicted for the loading imposed by the embankment for a design life of 30 years.

For the stability analysis a computer programme named PCSTABL5 was applied to calculate the embankment stability using an internationally accepted calculation method, based on circular sliding planes. In the conclusions a minimum factor of safety is presented for the embankment at the seaside slope only.

For all Areas the minimum factors of safety calculated were in excess of 1.5. This number, according to the contractor, being the required factor of safety for static load conditions for this very embankment. The calculated factors of safety for the five areas are all around 2.15, except one which gave 1.67. The Contractors conclusion was that these factors of safety satisfy the criterion for a safe embankment design. The reports (GSEC, 1995-2) were accepted by the Supervisor.

The expert's opinion on the value of the soil investigations and reports on geotechnical analyses is written hereafter.

2.3.3 Evaluation of soil investigations reports

As the failures in the Rehabilitation Area B are the result of the instability of the seaside slope of the embankment we will focus on the stability aspects of the soil investigations performed in Area B. There is a direct relation between the speed of settlement and the degree of stability, but settlement is taken into account later.

The soil investigation (in situ test equipment, laboratory test equipment, techniques and methods applied) was performed in accordance with the requirements. The results of the various laboratory tests performed on samples and the results of in situ tests in the Demerara Clays indicate that this top clay layer is of such quality, that it is essential to verify the influence of this soil layer on the degree of stability of the rehabilitated sea defence, in the construction stage as well as in the final stage.

It should be noted, that only for Areas B and C, 7 Consolidated Undrained Triaxial Compression Tests with pore pressure measurement have been carried out in accordance with the Specifications. However, the strength parameters, to be applied for effective stress analysis, have not been determined. The magnitudes were not presented in the reports and, therefore, no effective stress stability calculations have been performed.

Determination of the undrained shear strength in soil investigations can be done by measuring in the field with the shear vane apparatus, or in the laboratory performing Undrained Triaxial Tests. The latter, being the most adequate test, was not given in the Specifications, and, therefore, has not been carried out. The laboratory Torvane tests should be considered as classification tests. They are inappropriate to apply in calculations. This leaves the results of the shear vane tests only.

The shear vane test is generally considered to be a reasonable accurate test in very soft clays, but one should preferably be experienced in operating the apparatus. One should be aware of the effect of the torsion in the shaft on the results. The interpretation and determination of the magnitude of the strength parameter should be done with care. Whether this has been the case can not be determined. In the stability analysis done by the Contractor the shear vane test results have been applied as input.

Three shear tests per borehole were done in Area B in the softest clay layer described as "very soft to soft clay". The first test was done at 1.5 m depth in existing embankment material, and as such, gave relatively high values. One can determine that the representativity of the few shear vane values for the about 5 m thick very soft clay layer is then very low.

The failures were both situated in the Rehabilitation Area B. According to the Report on Stability Analyses for Areas B and C (GSEC, 1995-2), submitted before the time of the failures, which is based on the soil investigation results as reported in the Report on Geotechnical Investigations Area B and C (GSEC, 1995-1), the cross sections chosen for stability analysis are the cross sections where the boreholes have been made. In the report there is no indication that the *decisive* cross sections in the areas were chosen for calculation. Only the *borehole sections* have been subject of calculation. However, cross section 117 of boring 1B2 happens to be a decisive section with respect to the load of embankment fill.

2.3.4 Evaluation of the reports on stability analyses

In order to be able to calculate the stability of the raised embankment one should make a calculation model which shows the decisive cross section in the relevant rehabilitation area, the layers encountered in the investigation, and the phreatic line or potential lines, if any. To these layers the soil parameters are to be added that will follow from the laboratory test results and the interpretation by the consulting engineer with respect to the strength analysis. The greater the number of tests per layer, the more accurate the parameters will be and the more reliable the calculation result. The calculation method can be either the total stress analysis or the effective stress analysis. The stability analysis applied by the Contractor is the total stress analysis in which undrained shear parameters have to be applied.

In the reports on the stability analyses (GSEC, 1995-2) no information is given of the method of determining the soil stratigraphy and the soil parameters on the basis of the soil investigation results. No schematized model is presented showing the dimensions of the typical cross section taken for calculation. The boundary conditions required for calculation, such as the position of the phreatic line, high tide and/or sudden drawdown conditions, the edges and the bottom of the soil model, are not presented. The contractor submitted reports in which, on the basis of unverifiable calculations, he concluded that the embankment designs in the Rehabilitation areas were safe. These reports were accepted.

The conclusion is that the stability reports presented are incomplete and unsuitable to draw conclusions from with respect to the degree of safety against sliding.

2.3.5 The effect of only one borehole per cross-section

In geotechnical engineering there is no general standard for determining the number of survey locations in a cross section. The number will depend on the judgement of the consulting engineer unless there are local standards or procedures. In that case the requirements will determine the number.

Due to a contradiction in the Tender Dossier on the number of boreholes to be performed under the Contract it was discussed whether it was essential to carry out two boreholes per cross-section, see Section 2.2.4. The reason to perform boreholes at the inner side only has been given in personal communication with Mr. P.A.D. Allsopp, soil expert of SRKN'gineering. Mr. Allsopp has performed soil investigations between 1950 and 1970, at inner as well as at seaward locations. During these investigations he found that the subsoils present at the inner and the seaward side of the seawall were similar. Based on this experience he advised to perform soil investigations only at the inner side of the seawall. He also gave the opinion that the soils present at the shore side of the embankment have the same origin as the soils at the landside, as during time sea attack has resulted in withdrawal of the shore line.

We have no reason to doubt these experiences, although it would have been better to sink at least at one or two locations a borehole at the seaward side of the seawall for verification.

2.3.6 Additional soil investigations after the Henrietta slip

Five additional borings were performed in the failure section of the embankment at Henrietta. Three in the crest of which 2 in the failed embankment body itself and two at the toe of the slip. The in situ soil tests and laboratory tests done, revealed a subsoil that, according to the report GSEC (1996), was in the crest as well as in the toe much softer and had lower strength properties than the clay encountered in borehole 1B2.

The GSEC (1996) report concluded that apparently the soils were softer at this location than could be expected from the earlier investigations. Again it is concluded that this report is incomplete and unsuitable to draw conclusions from with respect to the degree of safety against sliding as there are insufficient data on the stability presented.

2.4 Evaluation of armour rock

The tender documents specify 100-600 kg rock to be used as armour layer. This rock will withstand wave attack up to 1.1 m. The specifications give:

- not more than 2% less than 60 kg
- not more than 10% less than 100 kg
- mean weight between 200 kg and 400 kg
- not more than 30% larger than 600 kg
- not more than 2% larger than 1000 kg

The weight of the rock is decisive for stability under wave attack and, therefore, for the lifetime of the structure and the expectation of damage developments. Under high wave attack (during spring tides) the waves may displace the rock around the high water level to the toe of the structure, creating holes in the armour layer around the high water area. Therefore, the rock of the armour layer was checked. At Area I two locations were checked, one location 50 m eastward and one location 50 m westwards of the sluice. At both locations about 65 stones were measured.

The three dimensions of each rock were measured. These are the length, l , the width, w , and the thickness, t . According to the CUR/CIRIA-Manual (1991) and also according to Delft Hydraulics' experience on other projects, the mass of a rock can be calculated from its three dimensions and its mass density, md , as follows:

$$M = a.l.w.t.md \quad (1)$$

The coefficient "a" is in average 0.55, but may vary between 0.3-0.7, see CIRIA/CUR (1991). In this case 0.6 was taken in order to be a little on the safe side with respect to the estimation of the existing rock grading. The mass density of the rock is 2630 kg/m^3 , see GSEC (1995-3). Based on the measured dimensions of the rock the mass was calculated. The grading at each location was determined and given in Figure 2.4.1. Also the required grading with its specified limits are given in this Figure.

It is clear that the armour layer has not been built according to the specifications. Around 15% is smaller than 60 kg, where only 2% is allowed, and about 30-35% is smaller than 100 kg, where only up to 10% is allowed. The mean weight is smaller than 200 kg. The dash-dot line represents the average of the two gradings and can be described as a grading 60-400 kg with a mean mass of 150 kg. This is about a factor two lower than specified. The existing armour layer is close to the 150 kg layer, which was also considered by DHV (May 1993, Annex 2). The effects of an armour layer of 150 kg is clearly stated in the DHV-document (page 30). Some damage can be expected in the next ten years, especially in areas where the design wave height (recurrence interval of 10 years) is estimated at 1.1 m. This is the case for sections B and F.

2.5 Evaluation of sea defence design

2.5.1 General remarks on riprap design

According to the objectives of this investigation the appropriateness of the sea defence design for the shoreline existing in Guyana has to be evaluated. The DHV report (May 1993, Annex 2, p 38-40) gives already a comparison with other designs in Guyana. We agree with this comparison and support the conclusions with respect to crest height, slope of 1:3 and armour rock of 300 kg instead of 150 kg.

The riprap design is a fairly new design in Guyana. In the past a lot of vertical sea walls have been constructed. Of course a vertical sea wall may also give a good solution. The choice between various alternative designs has to be made on the basis of costs, estimation of life time and damages, and possibilities with respect to maintenance and repair. The riprap design has some advantages, which have also been described by DHV. Repair is fairly easy; a possible erosion of the foreshore will be taken into account by the 5 m long toe (falling apron); all materials, except for the geotextile, are available in Guyana. The main disadvantage may be that local companies, contractors as well as consultants, have not yet enough experience with this design. This will change of course if more sea defences are constructed as a riprap design.

The slope of a riprap design depends on the wave height, availability of heavy armour rock and, especially in Guyana, on soil conditions.

The steepest slope possible is a 1:1.5 slope. It requires a (very) heavy rock grading. The more gentle the slope the smaller the armour rock can be. But also the amount of material, sandfill, geotextile, filter layer and armour rock, will increase. As long as soil conditions do not play a role, the availability of heavy rock and the costs of the rock will determine the slope and rock size.

2.5.2 Evaluation of 1:3 slope

In Guyana soft soils are presents and, therefore, in most cases the choice of slope will be related to geotechnical stability. Variations in soil layers along the longitudinal direction may require local adaptations in the design. But in general the choice of slope depends on the loading on the soil. More specifically:

- the difference in height between the crest and the foreshore level. The larger this difference, the higher the loading and the more gentle the slope has to be.
- the total amount of material per m run. The more weight is put on the soil, the gentler the slope has to be.

With respect to the present design a slope of 1:3 was chosen. This is according to other designs. Nedeco (1962) gives also gentler designs with slopes of 1:6 and even with a berm, but the foreshore level taken in that study was fairly low. This means that the difference in crest height - foreshore level there was also large: 22 ft or 6.7 m. The maximum difference in the constructed design was 4.9 m, see section 2.2.1. As the structure in the present design is (much) smaller than the Nedeco design a 1:3 slope can be regarded as suitable. This is also proven by the constructed work as only two sections showed a slip failure.

With the experience of the slip failures it can also be concluded that a slope 1:3 with crest level - foreshore level differences of almost 5 m, and large sandfills, may become unstable if circumstances are present that decrease the stability. These circumstances may be locally weaker soils, long and heavy rainfall, use and vibration of sheet piles, etc.

In areas where the designed cross-section is much smaller, for example where the foreshore is much higher, and where the crest level - foreshore level is lower than say 3.5 m, a steeper slope of 1:2.5 or even 1:2 may be feasible. In general, however, a slope of 1:3 will give a good design in Guyana.

2.5.3 Geotechnical stability of a slope 1:3

In Section 2.3.4 it was concluded that the stability check done by the Contractor was based on incomplete modelling of the cross section and unreliable parameters. In order to get more insight in the actual state of stability at the time of the failures Delft Geotechnics has modelled cross-section 117 in the Area B (Figure 2.5.1). The soil layers are composed from the boring 1B2 description. The soil parameters of the original soil layers have been taken from the laboratory test results submitted by the contractor (GSEC, 1995-1). The soil parameters belonging to the embankment fill material and the rip rap are assumptions. While determining the parameters the condition of the layers right before the slide, dry, wet, or saturated has

been taken into account. The shear strength values from the in situ tests have been corrected in accordance with the Bjerrum method. The position of the phreatic surface in this model has been assumed. It follows the surface of the saturated soils during the low tide period.

Figure 2.5.2 shows the soil parameters chosen for input in the computer programme. The calculation has been done using the undrained condition (total stress analysis) which enables determining the degree of stability of the sea defence (cross-section 117) at the moment before the failure. The most unfavourable loading condition, being the low tide period right after a high tide, has been analyzed.

The calculation method applied is the Modified Bishop method of slices (circular sliding planes). The computer programme used is MSTAB (Micro computer STABILITY), developed by Delft Geotechnics in co-operation with the Water Defences Department of the Ministry of Transport and Public Works of the Netherlands. The result of the total stress stability analysis is presented on the Figure 2.5.2. The sliding plane is given. The calculated minimum stability factor $F_{min} = 1.20$. This stability factor is considerably smaller than the value 1.68 mentioned in the report (GSEC, 1995-2).

In order to determine the effect of the much lower shear strengths measured near the seaside toe in the Henrietta site after the failure, the calculation has been done applying the lower C_u -values, measured in the soft clay layers near this toe. The corrected C_u values decreased to 12 kPa in the very soft clay, see the table in Figure 2.5.3. The result is that the minimum stability factor decreased to 0.62, which means slip failure.

Further the stability of the sea defence has been analyzed using effective stress parameters and taking into account the rate of dissipation of the pore pressures at the moment before the failure. The rate of dissipation has been derived from the consolidation parameters, determined in the laboratory (GSEC, 1995-1). See also Section 2.5.4 on settlement. The stability model and the parameters for the embankment fill material and the rip rap are the same as used in the total stress analysis. The effective stress parameters for the cohesive materials are estimates. Those for the soft clay have been taken from the Nedeco (1972).

effective cohesion	: $C' = 3.5 \text{ kN/m}^2$
effective friction angle	: $\phi' = 13.5 \text{ degrees}$

The position of the phreatic surface in this model has been assumed. It follows the surface of the saturated soils. Figure 5.2.4 shows the soil parameters chosen for input in the computer programme. The calculation has been done using the drained condition (effective stress analysis) which enables determining the degree of stability of the sea defence (cross-section 117) at the moment before the failure taking into account the rate of dissipation in the compressible layers. The most unfavourable loading condition, being the low tide period right after a high tide, has been analyzed.

The amount of dissipation of the pore water is assumed on the basis of the settlement calculations (Section 2.5.4). Based on a construction time of about 2 weeks, about 25 % of dissipation is assumed for the soft clay layers. For the old embankment materials 100 % of dissipation is assumed.

The result of the effective stress stability analysis is presented on the Figure 2.5.4. The sliding plane is given. The calculated minimum stability factor $F_{min} = 0.95$. This stability factor indicates that at the 7th of July 1996 at low tide the sea defence at the Henrietta site was unstable.

Questions about the value of the soil investigation results, both before and after the failure, will remain. But based on these values one can conclude from the above mentioned calculation results, that the degree of stability of the critical cross-sections in Area B during construction was close to 1.0. The calculation with the low values obtained from the soil investigations after the Henrietta slip failure shows clearly that weaker soil can lead to slip failure. And the probability of occurrence of such a slip failure was increased by the removal of the sheet piles by vibration.

2.5.4 Settlement calculation

In the reports on the geotechnical investigation for the rehabilitation areas the laboratory test results of the consolidation tests are presented. The input parameters and the results of the calculations done by the Contractor to predict the settlement of the sea defence crest in the areas are presented in the reports on the geotechnical analyses (GSEC, 1995-2). The predicted total settlements for the crest were between 0.02 m (minimum) and 0.41 m (maximum), taking into account an assumed life time of 30 years.

As in the respective reports no details were presented of the calculation model, layer structure, fill levels, phreatic surface, etc. as well as of the calculation method applied, it is not possible to rate these results at their true value. In order to be able to verify whether the predicted magnitudes are realistic Delft Geotechnics has made settlement calculations.

The settlement calculation model is based on the same cross-section 117 and the input for the layer structure (the settlement model) and soil parameters was copied from the input for the stability calculations. The settlement parameters have been taken from the laboratory test results of the samples taken from boring 1B2 (GSEC, 1995-1). The settlement parameters are given in Figure 2.5.5.

In order to be able to predict the settlements of the complete embankment a number of verticals were chosen. In each vertical the total settlement was calculated. The calculation method is the Bjerrum method, described in the Dutch Geotechnical Standard (NEN 6744). The computer programme used is MZET, (Micro computer ZETting = settlement), developed by Delft Geotechnics in co-operation with the Water Defences Department of the Ministry of Transport and Public Works of the Netherlands. The sea defence profile including the respective layers is presented in Figure 2.5.5.

According to the results of the calculation the maximum total settlement, about 0.8-1 m will occur near the toe of the original embankment where the highest fill is present. The calculated total settlement of the top of the inner edge of the rip rap at the crest will be about 0.6-0.8 m.

In order to get insight in the rate of settlement during time the Delft Geotechnics computer programme HYDRO has been used to calculate the degree of consolidation at requested points of time as well as the hydrodynamic time lag. The consolidation parameters, C_v -values (see Figure 2.5.5), have been taken from the laboratory test results of the samples taken from boring 1B2 (GSEC, 1995-1).

The results show that the theoretical end (99%) of consolidation is after 26 months. From the results it can also be derived that at cross section 117, at this moment mid-October 1996, a little bit more than 50% of the total settlement will have occurred. About 75-80% will have occurred after one year. In view of the above mentioned total settlement of 0.6-0.8 m this means, that the crest at the inner edge of the rip rap protection will have settled about 0.3-0.4 m and will, after mid-October 1996, still settle in the order of another 0.3-0.4 m.

It is therefore recommended to check as soon as possible the position of the crest as well as the top of the rip rap protection by levelling according to Georgetown Datum in order to establish whether the crest level is still in agreement with the design requirements.

3 Causes of the slip failures

After having reviewed all data on the rehabilitated embankments we conclude that the failures of the slopes at the Henrietta and the Anna Regina site are due to the generation of excess pore pressures in the underlying clay, which led to an inadmissible decrease of the shear resistance of the clay. There has been a number of factors that had an adverse effect on the actual shear resistance of the underlying soils in the area of the sliding plane. These factors will be discussed below.

- The capacity of the existing subsoil to bear the loads imposed by raising the embankment depends on the speed or rate of construction in height. If raising embankments on clayey soils one should consider the risk for instabilities due to the existence of excess pore pressures, especially in soft clays. In this case the raising of the total embankment up to 4.9 m high at once, should be considered high. With the tide of around 3 m twice per day, however, there is no other way than to construct directly to the full height. It should be noted that the construction rate *in height* is important, not the construction rate in the longitudinal direction.
- The slip failures occurred in areas with the most critical condition with respect to loading: the amount of sandfill was maximum (between 27-29 m³/m) and also the difference in crest level and foreshore level (4.9 m). This was not the only reason for the slip failure as other critical sections remained stable.
- During the construction of whole Area B there was extreme rainfall. This undoubtedly has had an adverse effect on the workability on the soft soil surfaces as well as on the quality of the construction materials applied in that period. The amount of saturation of the clays in the old embankment as well as in the top of the soils on which had to be worked will have been relatively high. At the same time the position of the phreatic line will be extremely high under saturated conditions of the clay. These conditions were present during the whole construction period and are not the only reason for failure as most of Area B has not failed.
- The slip failure areas were the only areas where the cofferdam flooded. The reason for flooding was mainly that the soils supporting the sheet piles were not stable enough. For sections 81-90 this was during spring tide. In sections 130-144 sheet piles deformed and settled under their own weight. Also a second sheet pile had to be placed in sections 130-144 to prevent the actual construction from wave action. The deformation and flooding of the cofferdams gives an indication that at these areas the soil may be weaker than in other areas.
- The flooding of the areas does not necessarily lead to worse working conditions. It is similar to working tidally.
- The slip failure areas coincide with the areas between the cofferdams. It is possible that the cofferdams had an effect on the occurrence of the slip failures. Sheet piles were placed and pulled by a vibrating hammer. The sheet piles in sections 81-90 were removed on 29 May. The slip failure occurred on 2 June after a spring tide of 3.15 m GD. Half of the sheet piles at sections 130-144 were removed when the first cracks were observed immediately after removal. The slip failure occurred one day later. Pulling

by vibration, just in front of the toe (and of potential slip circle) has decreased the resistance of the soil. The pulling of the sheet piles is one of the causes for the slip failures. It is not the only reason, as slip failures happened in two areas only.

- It is the *combination* of circumstances has caused the slip failures. These circumstances are:
 - the heavy rainfall during the whole construction period
 - the areas with possibly weaker soil, indicated by the flooding and deformation of the cofferdams
 - the removal of the sheet piles by means of vibration
 - the presence of critical cross-sections with respect to loading (amount of fill and difference between crest level and foreshore level).
- The high safety factors on stability as given in GSEC (1995-2) have given the impression that slip failures would not occur, that a slope of 1:3 was completely safe. This may have led to a lack of awareness that certain circumstances would be able to decrease the stability in such a way that slip failures were possible. Awareness could have led to paying more attention to critical cross-sections, locally soft soil conditions, etc.

4 Recommendations on repair work

4.1 Temporary measures

Since the slip failures a number of months have passed. Due to dissipation of pore water the strength of the clay will have increased. The speed of this strength increase depends on the permeability of the clay in relation with the load imposed on it. The amount of dissipation can be determined from the time settlement relation predicted from laboratory test results and from settlement monitoring data. Good predictions and monitoring data of settlement are not available.

The contractor took temporary measures directly after the slip failures. He constructed an earth embankment up to about 18 m GD. The seaward slope until the slided armour rock was protected by light rock of 10-60 kg. These measures were taken to prevent heavy overtopping during high tides and to prevent breaches and floodings. Till now this has worked satisfactorily.

The measures taken by the contractor were of course temporary and permanent measures have to be taken. With respect to safety the permanent measures should be taken in the near future. From the geotechnical point of view, however, it is better to wait as long as possible. The structure will become more stable with time. The critical part of the sea defence is mainly the slipped area. The area directly landwards of the slip circle is more stable (that is the part that has not failed). If permanent measures are concentrated on the landward side of the crest, mainly outside the slip area, these measures can be taken directly.

The present situation is more or less safe and extra temporary measures are not directly required. Permanent measures, however, should not wait too long and it is recommended to perform the permanent measures, as proposed furtheron, within half a year.

4.2 Permanent measures

The armour slope in the slipped areas is now much gentler than the 1:3 constructed slope. The upper part is now much lower than in the original situation. A repair to the original height and slope would mean a large increase of weight on the slipped area. The slipped area could possibly slip again. It is, therefore, recommended *not* to reconstruct the sea defence according to the original design.

The best solution is to leave the slipped area as it is, to make advantage of the gentle slope, and to take measures at the more stable landward side of the slip.

Figure 4.1.1 gives the measured profiles of sections 84 and 89 (SRKN'gineering/DSCL, 21 June 1996). Section 84 is in the middle of the Henrietta slip failure and section 89 is close to the side. The average slope in section 84 is now 1:6.0 and in section 89 1:4.1. This is much more gentle than the original design of 1:3. A gentle slope gives less wave run-up than a steep slope. It is therefore possible to make a lower sea defence than in the original design.

The crest height of the sea defence depends on a lot of parameters, mentioned in DHV (May, 1993). One of them is the wave run-up, which was calculated from formulae in TAW (1985).

5 Recommendations for future riprap works in Guyana

5.1 General recommendations

The construction of about 6 km of sea defence with a riprap design was one of the first works of this kind in Guyana. The evaluation of this work, unfortunately initiated by two slip failures, leads to recommendations for future riprap works in Guyana. A number of recommendations are given in this Section. More comprehensive recommendations on soil investigations and geotechnical analyses are given separately in Sections 5.2 and 5.3.

1. Normally, the analyses of soil investigations and the subsequent conclusions on settlement and stability are performed by a consultant or supervisor. It is recommended to include geotechnical experience in the team of the supervisor (PEU), in order give guidance to the work of the consultant (to be described in the tender documents).
2. Critical cross-sections should be determined along the total length of the structure. These critical cross-sections have the largest sand and rock fill and the largest difference in crest height-foreshore level. Geotechnical analyses should consider these critical cross-sections and not only the cross-sections at locations of boreholes.
3. Rehabilitation works of sea defences by means of riprap revetments give extra loads on soft clay layers. These loads should be minimised at critical cross-sections with respect to possible slip failures. It is recommended that for critical cross-sections the amount of fill (sand and rock) is distributed *over* the existing structure as much as possible and that construction *on the foreshore* is limited. This may lead to difficulties with respect to available space at the landward side, but it is an effective solution to reduce the risk of slip failures.
4. Settlement of the crest of the structure should be measured during and a few months after construction. This enables verification of the settlement predictions.
5. Sheet piles and cofferdams may locally weaken and disturb the soil, certainly if a vibration method is used during removal. It is recommended that the sheet piles are placed at least 10 m from the new toe of the structure. Flooding of cofferdams or deformation may indicate the presence of soft soils that are not able to support the sheet piles. It is recommended in that case to remove the sheet piles immediately and to construct the revetment tidally. One should also be aware that in such sections the probability of slip failures is larger and that all circumstances that might increase this probability should be avoided.
6. If very soft to soft clays are expected to be present at the site a good way to obtain initial information about the degree of softness of the top layer is the following way. One can take a 4 to 6 m long steel bar, thickness 6 to 10 mm, and push the bar into the soft soil by hand force as deep as possible. The degree of resistance to penetration as well as the depth of penetration can be a measure for the softness or firmness of the clay. This method is quick and cheap and may, if done at many locations, give a fair impression of the softest areas in the alignment.
7. The Guyana coast has retreated several hundreds of meters in the last decades at some locations. It is recommended to investigate existing data on what has been there in the past. One simple method may be to ask old people who live in that area about the history. Weak spots may be detected in this way.

8. The size of the armour rock determines its behaviour in the future. Too small rock will give more damage and maintenance than expected. Therefore, the supervision on the rock size during construction is important. An easy way is measurement of the three length dimensions of 50-100 stones at certain locations. Section 2.4 and Formula (1) gives the method to calculate the weight of each rock. The grading curve can then be established and compared with the specifications. The form factor "a" in Formula (1) can be established at the start of the works by measuring 50-100 rock on the slope and then weighing each rock individually.
9. A mid term evaluation of rehabilitation works by external and independent experts may indicate weak procedures in design, supervision and construction. In cases where experience with actual riprap design and construction is limited, such a mid term evaluation is recommended.

5.2 Recommendations on soil investigation programme

In order to be able to predict the settlements as well as to check the degree of stability of the proposed sea defence rehabilitation a soil investigation should be carried out. It is hardly possible to standardize the extent of the soil investigation programme required for sea defence construction in Guyana. When analysing the results from Nedeco (1972) on the soil investigations carried out along the Guyanan coast (see also Section 2.3.1) one may decide to choose for one survey location per 500 m. If the variation in survey results between the locations is unexpectedly great it is to the judgement of the geotechnical engineer to decide to perform additional surveys.

The choice of the survey type can be: boreholes with or without Standard Penetration Tests, Dutch Cone Penetration Tests, as well as laboratory testing on recovered samples. It would be advantageous to have Dutch CPT's carried out for a number of reasons:

- the CPT results supply valuable data on the degree of softness and firmness of the soils. A continuous picture of the soil resistance and local friction is presented.
- the CPT can be carried out in a short period of time, compared to the making of a borehole to the same depth. This means that CPT's can be performed also at the seaside of the embankment during the low tide period, thus allowing the engineer to get informed on the properties of the soils offshore.
- consequently the cost of performing CPT's will be lower than of making boreholes.

Still a certain number of boreholes should be performed in order to relate the CPT results to soil descriptions. An appropriate number is: 1 borehole per 5 CPT's.

5.3 Recommendations on geotechnical stability calculations

The stability of the designed sea defence with respect to sliding along deep slip surfaces should be checked. The stability check constitutes the final phase of the design of the sea defence. It is recommended that the degree of stability should be checked by assuming circular slips. The modified Bishop method of slices is a simple, reliable and internationally accepted method for this purpose. There are two determinant conditions to be distinguished:

- the determinant condition for the inner slope occurs at flood level (spring tide), if a steady groundwater flow through the embankment and the subsoil is assumed.
- the determinant condition for the outer slope occurs during draw down of sea water after spring tide when there is a retardation in the adaption of pore water pressure in the embankment.

For a general approach to the related pore water pressure pattern in the sea defence and in the subsoil, the following assumptions can be adopted for a clay embankment on clayey subsoil:

- linear pore water pressures and effective stress distributions occur along vertical cross sections
- the effective stresses in the subsoil result from the surcharge and a hydrostatic pore pressure distribution
- the effective stresses in the clay embankment equal the effective stresses at flood level (considered to be stationary).

When assessing the degree of stability the following aspects are to be considered:

- the loads on the existing embankment
- the strength properties of the soil materials
- the geometry of the structure
- the mathematical model for checking the stability

As in practice load and strength properties (from soil investigations) are variables a sufficiently large safety margin should be chosen such, that the probability of failure of the structure is reduced to an acceptable level with respect to risk of lives and costs. In general the magnitude of this safety factor is largely based on practical experience.

Safety factors internationally accepted for the stability of water retaining structures like sea defences are:

- for the inner slope during the execution of the work: $n = 1.3$
- for the inner slope after the execution (final stage): $n = 1.5$
- for the outer slope during the execution of the work: $n = 1.2$
- for the outer slope after the execution (final stage): $n = 1.3$

It is recommended to use these factors of safety as minimum factors for the stability when analysing the degree of stability of a cross-section drawn up on the basis of a sound soil investigation programme.

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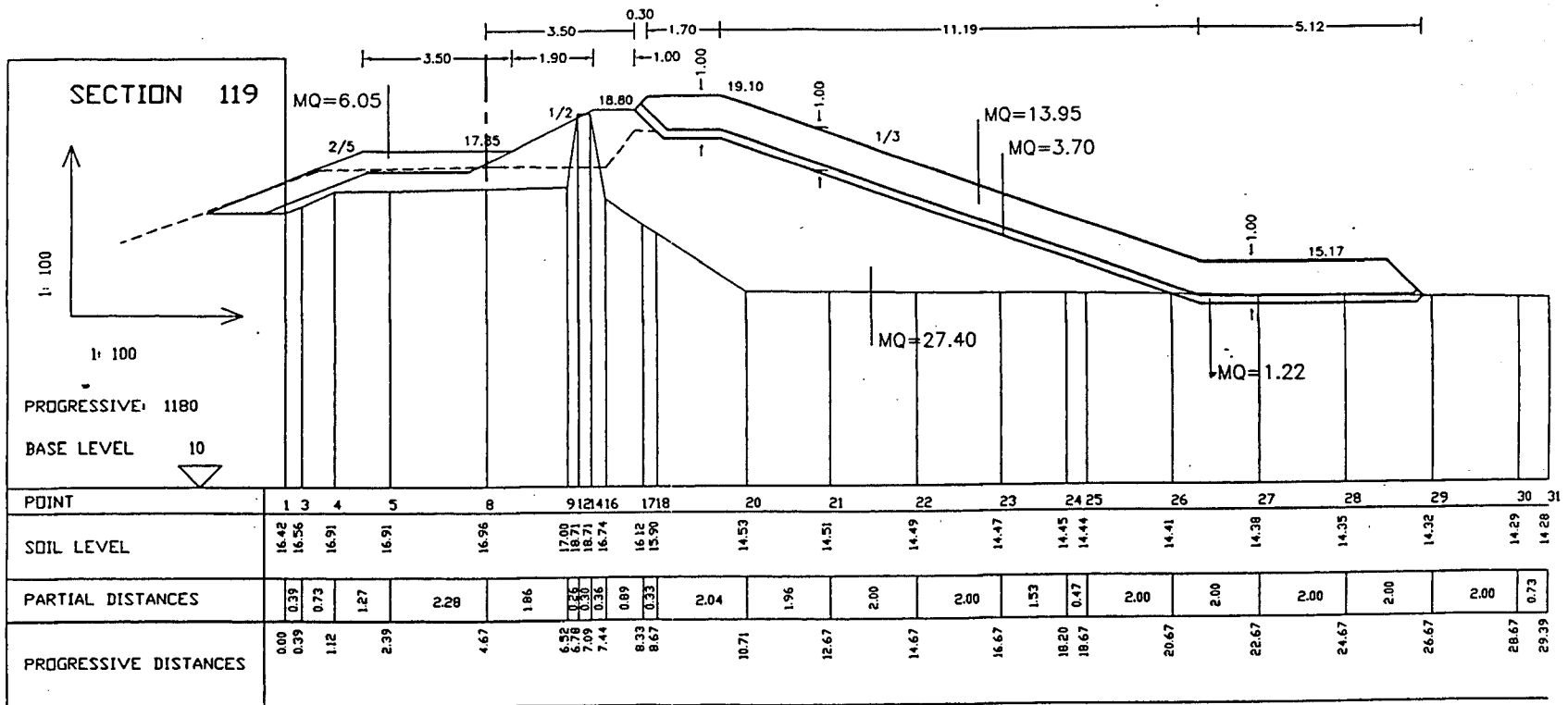
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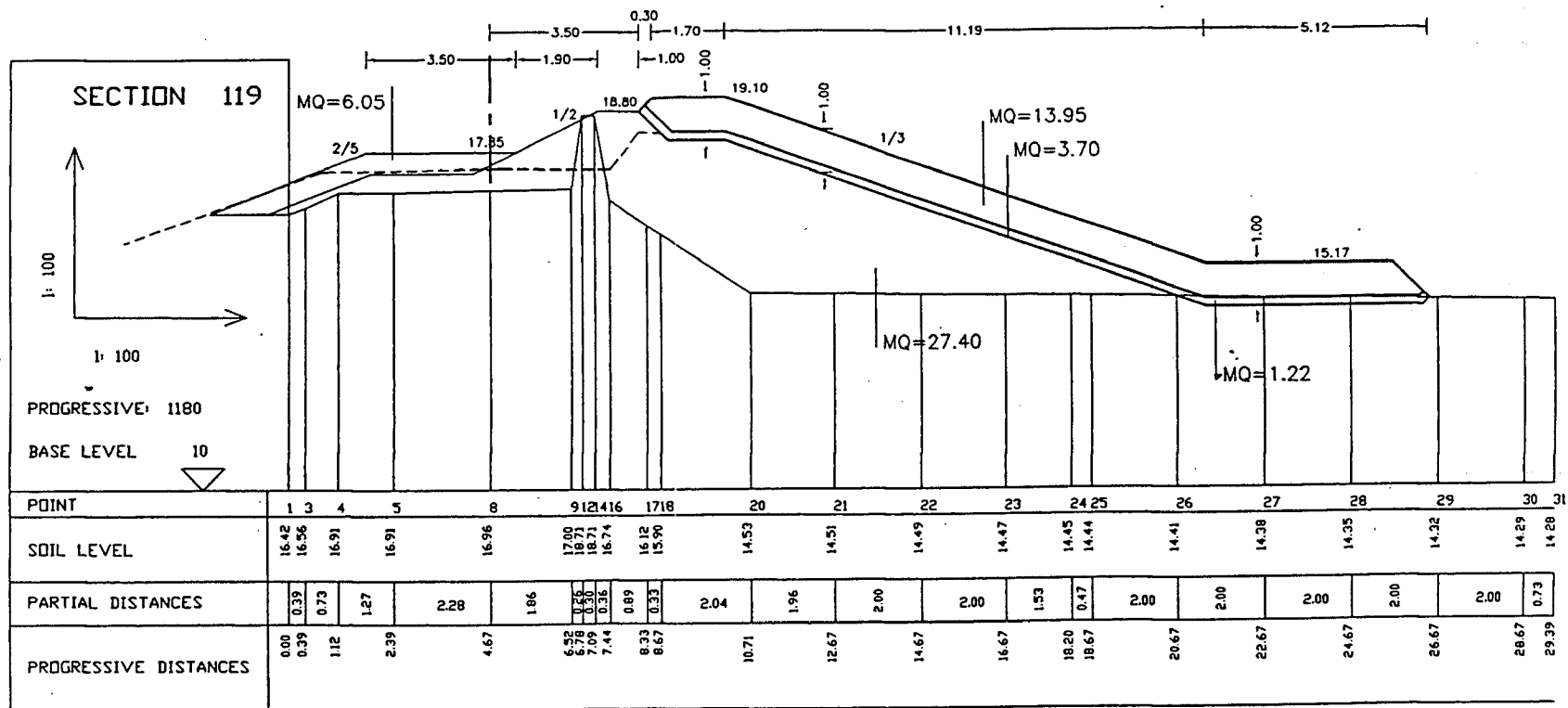
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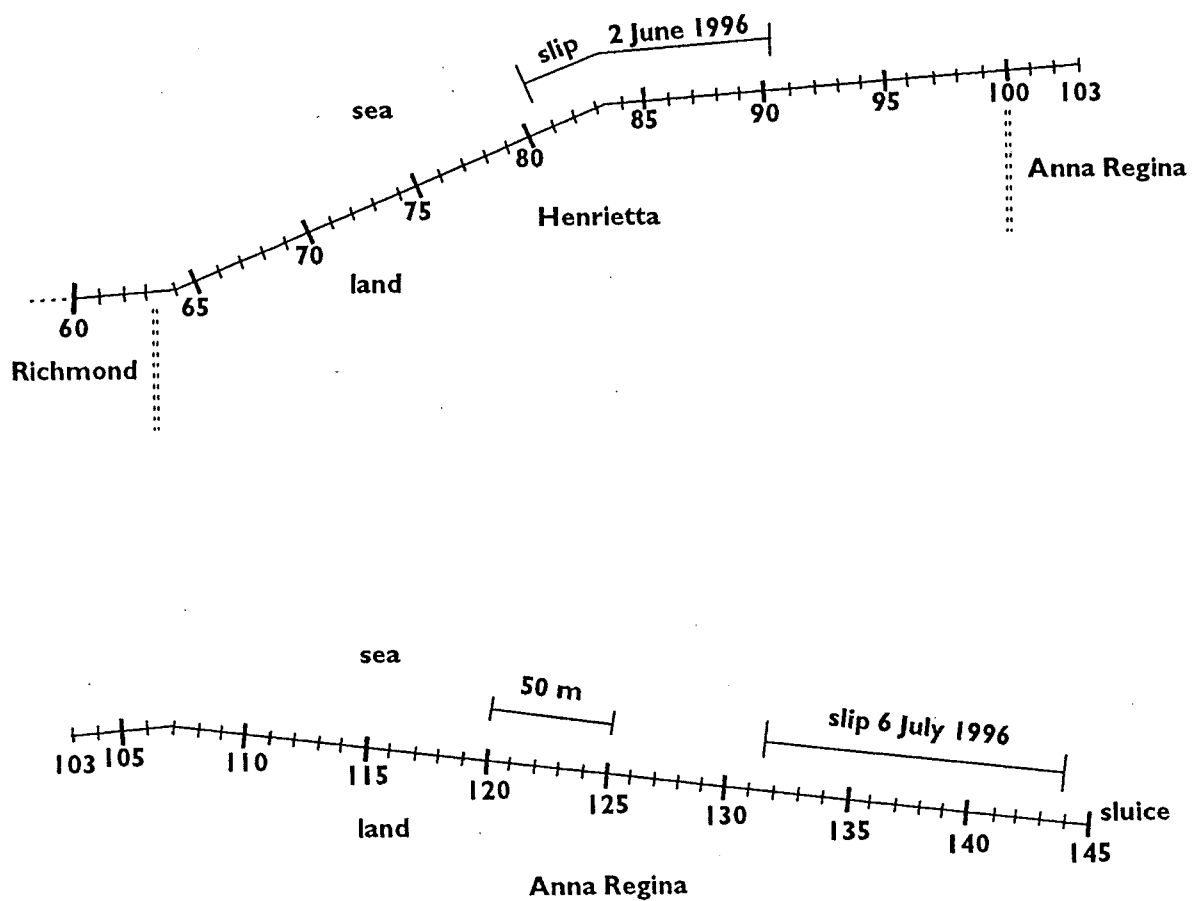
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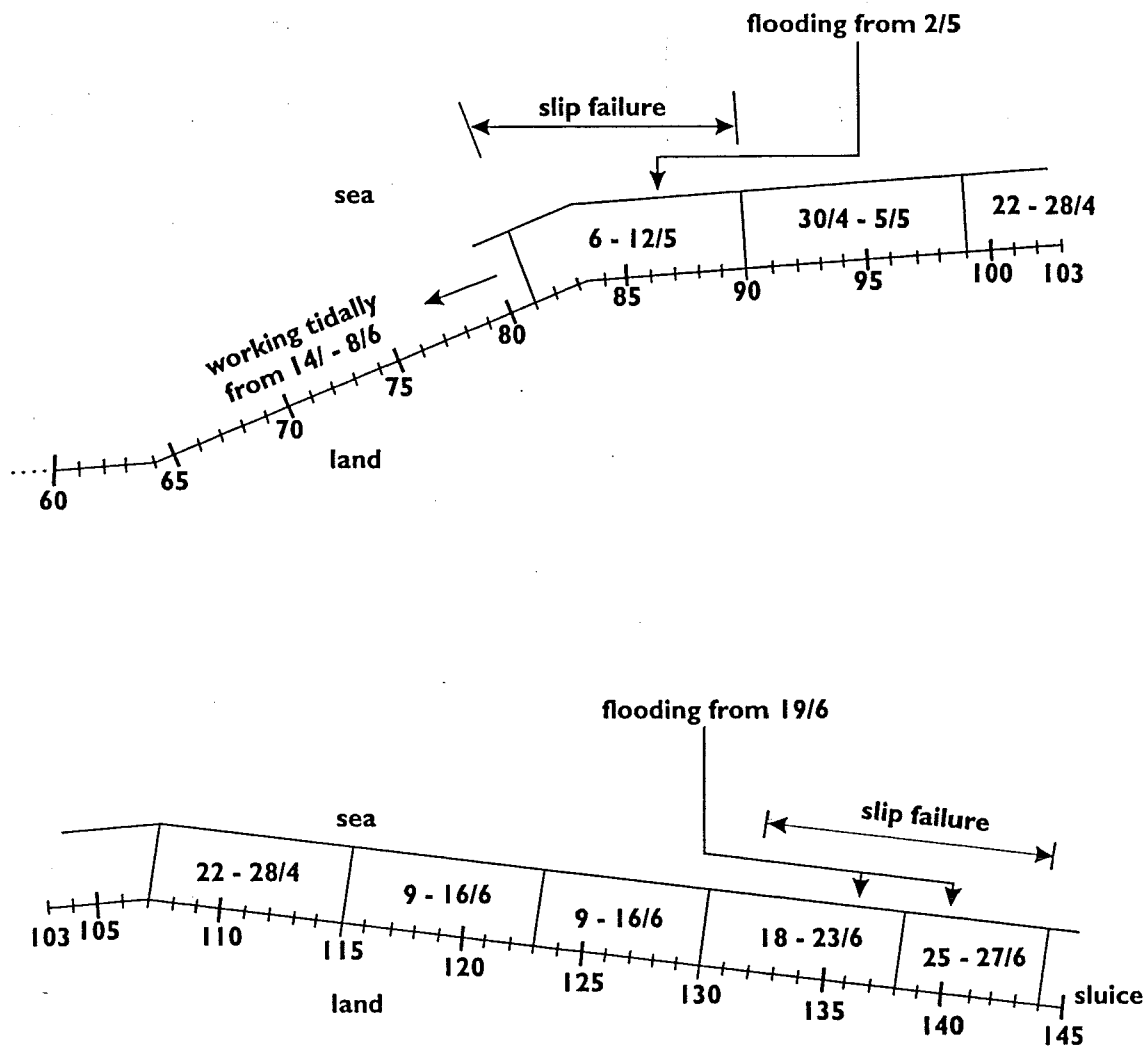


Cross-section of Area B, section 119

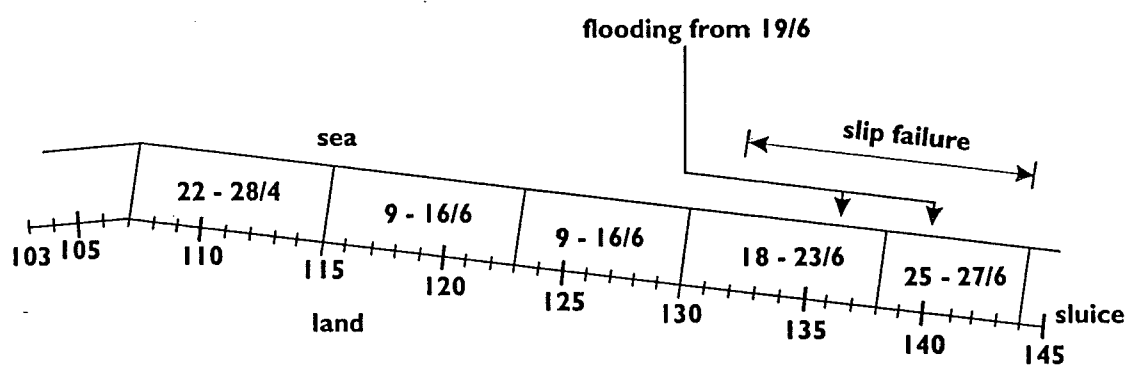
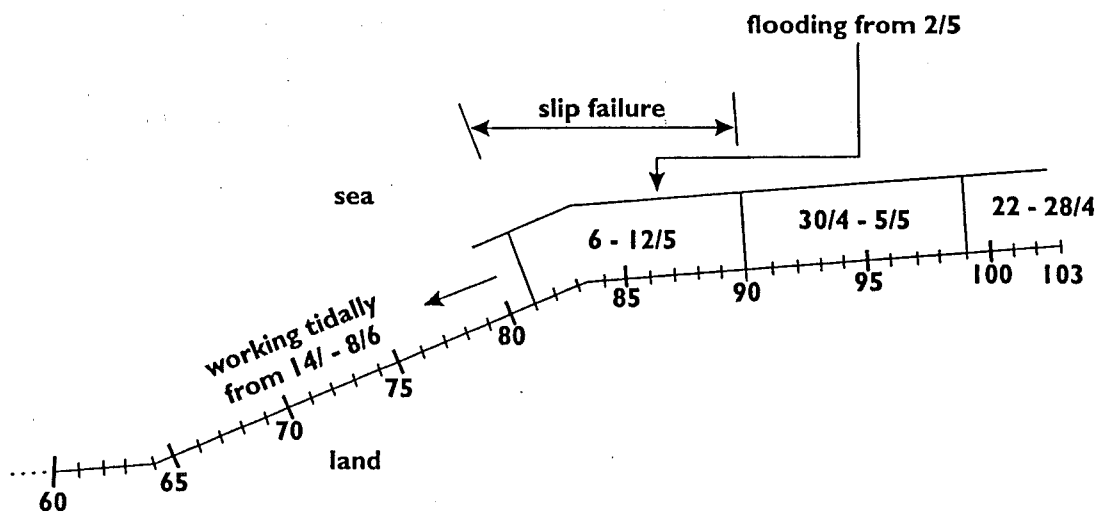




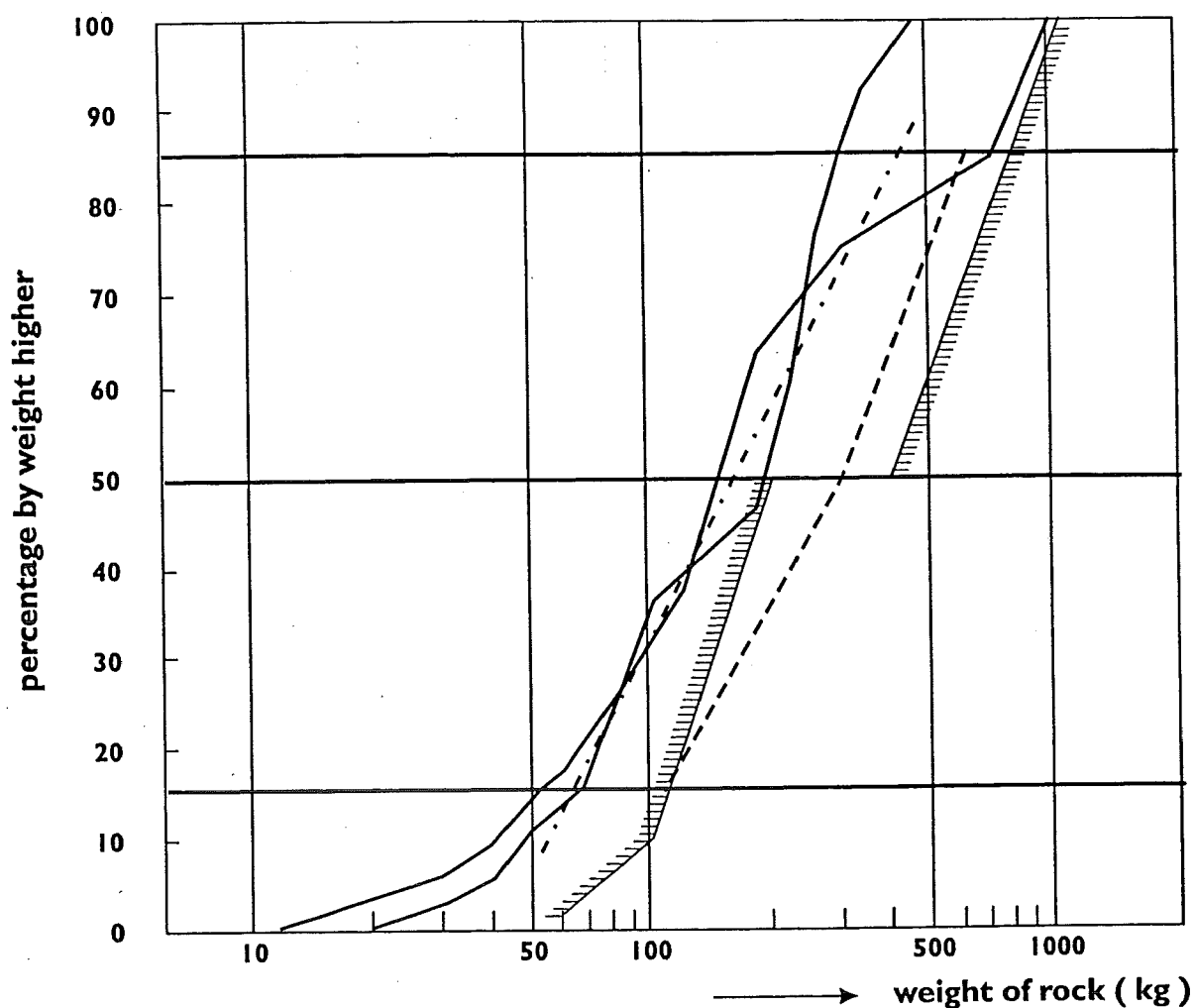
Plan view of area B from sections
60 - 145 with locations of slip failures



Location cofferdams and progress of work



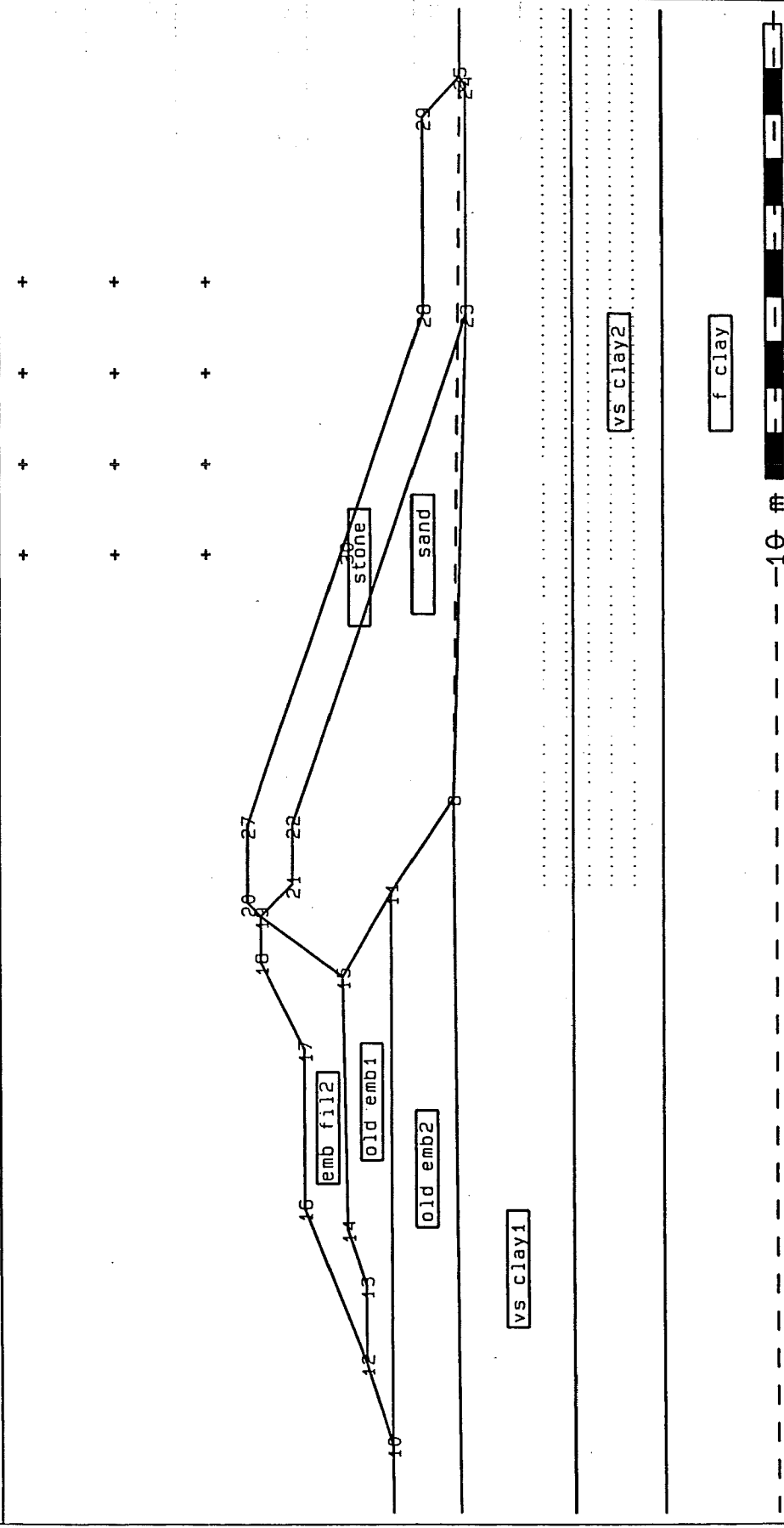
Location cofferdams and progress of work



- measured grading
- - - average measured grading : 60 - 400 kg
with mean weight 150 kg
- . - . required grading : 100 - 600 kg
with mean weight 300 kg
- /// limits for grading
according to specs.

Grading curves armour rock at site I

CHECK	INPUT	Bishop

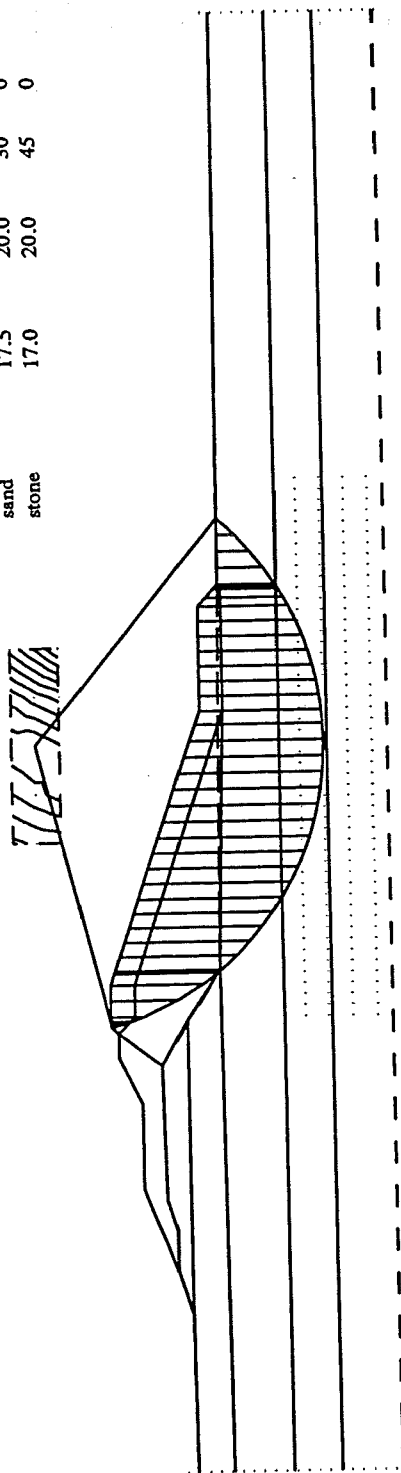


boring 1b2

FIG. 2.5.1

CRITICAL SLIP CIRCLE : Measured Cu-value

soil type	rho-dry	rho-wt	phi	Cu
f clay	19.4	19.4	0	19.32
vs clay2	20.2	20.2	0	15.19
vs clay1	15.6	15.6	0	17.92
old emb2	17.0	18.0	0	10.5
old emb1	17.0	18.0	0	14.0
emb fil2	16.0	17.0	0	14.4
sand	17.5	20.0	30	0
stone	17.0	20.0	45	0



20 m

Xm = 20.00 m	R = 12.00 m	ISO 1 = 1.199
Ym = 22.00 m	Fmin = 1.199	ISO 11 = 1.809

cs 117 corrected cu

boring 1b2

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MSTAB [6.1.2]

File : ANNAREGI



Stability analysis
Total stress, shear strength before failure

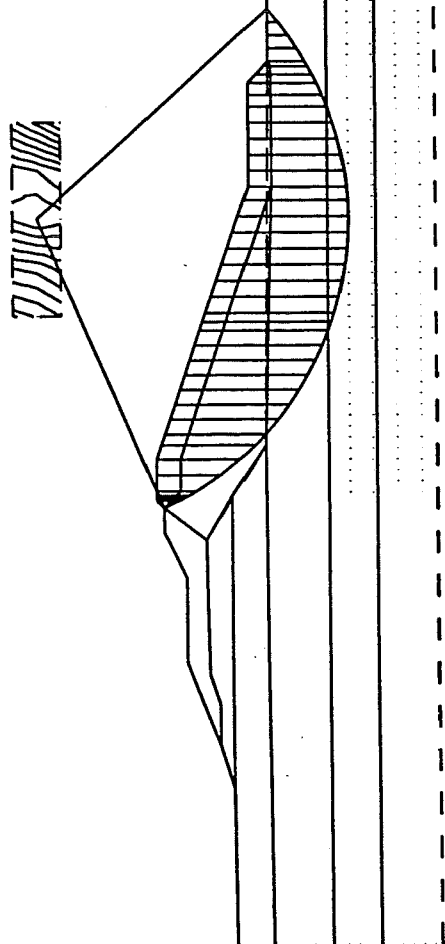
DELFT HYDRAULICS

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FIG. 2.5.2

CRITICAL SLIP CIRCLE : Measured Cu-value

soil type	rho-dry	rho-wt	phi	Cu
f clay	19.4	19.4	0	19.32
vs clay2	20.2	20.2	0	12.0
vs clay1	15.6	15.6	0	12.0
old emb2	17.0	18.0	0	10.5
old emb1	17.0	18.0	0	14.0
emb fil2	16.0	17.0	0	14.4
sand	17.5	20.0	30	0
stone	17.0	20.0	45	0



20 m

$X_m = 20.00 \text{ m}$ $R = 13.00 \text{ m}$ $Iso \ 1 = 0.620$
 $Y_m = 24.00 \text{ m}$ $F_{min} = 0.620$ $Iso \ 11 = 0.912$

cs 117 corrected cu after slide
 boring 1, 2, 3, 4, 5

DELFT GEOTECHNICS

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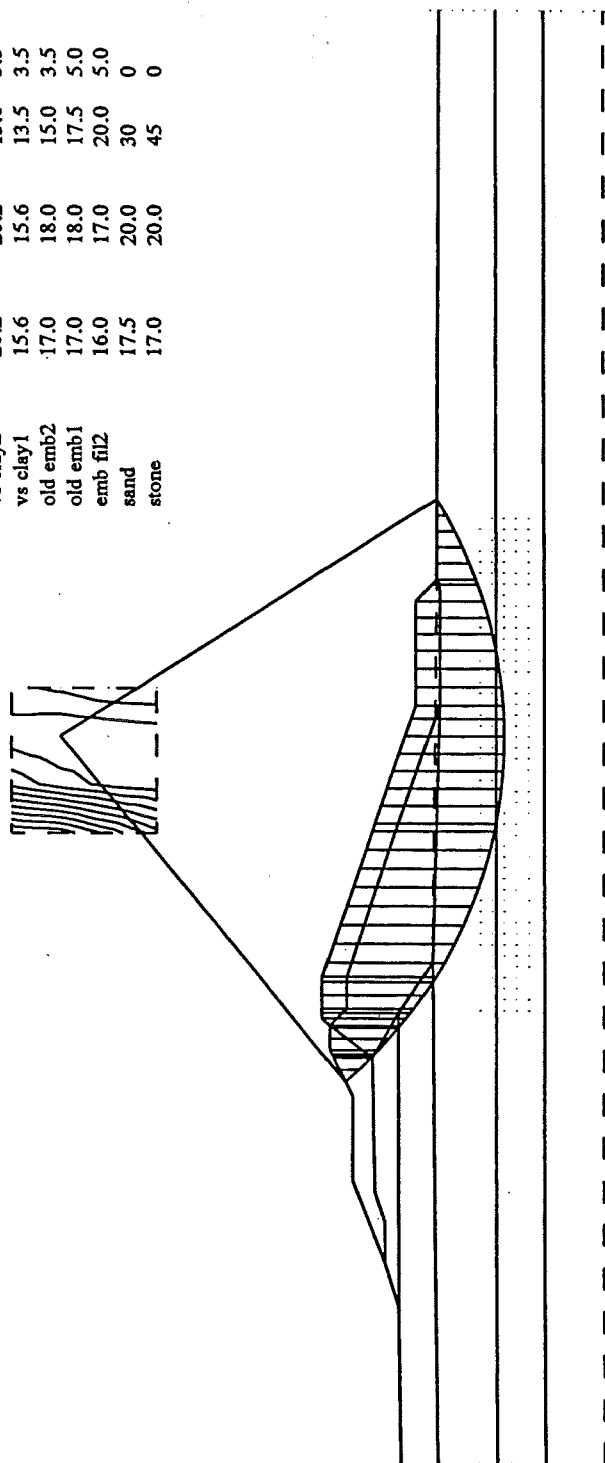
MSTAB [6.1.2]

File : ANNARE1B

Stability analysis
 Total stress, shear strength after failure

CRITICAL SLIP CIRCLE : Bishop

soil type	rho-dry	rho-wt	phi	cohesion
f clay	19.4	19.4	20.0	5.0
vs clay2	20.2	20.2	15.0	3.5
vs clay1	15.6	15.6	13.5	3.5
old emb2	17.0	18.0	15.0	3.5
old emb1	17.0	18.0	17.5	5.0
emb fil2	16.0	17.0	20.0	5.0
sand	17.5	20.0	30	0
stone	17.0	20.0	45	0



20 m

DELFT GEOTECHNICS	Xm = 20.00 m	R = 18.50 m	Iso 1 = 0.948
Lic: INTERN Cop: S1	Ym = 30.00 m	Fmin = 0.948	Iso 11 = 1.310

cs 117 EFF. STRESS constr. phase
boring 1b2

MSTAB [6.1.2]
File : ANNAREG5



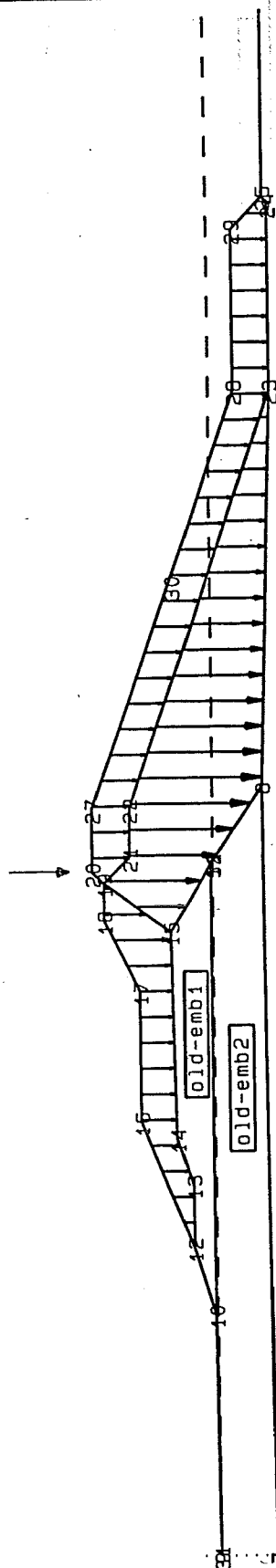
Stability analysis
Effective stress

DELFT HYDRAULICS

H 3095

FIG. 2.5.4

CHECK INPUT - GEOMETRY



vs-clay1

vs-clay2

f-clay

soil type	rho-dry	rho-wt	consolidation coefficient	compression index
old-emb1	17.0	18.0	1.0	0.060
old-emb2	17.0	18.0	1.0	0.125
vs-clay1	15.6	15.6	$1.5 \cdot 10^{-7}$	0.642
vs-clay2	20.2	20.2	$1.5 \cdot 10^{-7}$	0.642
f-clay	19.4	19.4	$3.0 \cdot 10^{-5}$	0.060

10 m

DELFT GEOTECHNICS

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MZET [2.1.1]

File : ANNAREGZ

-- old loads

--- new loads

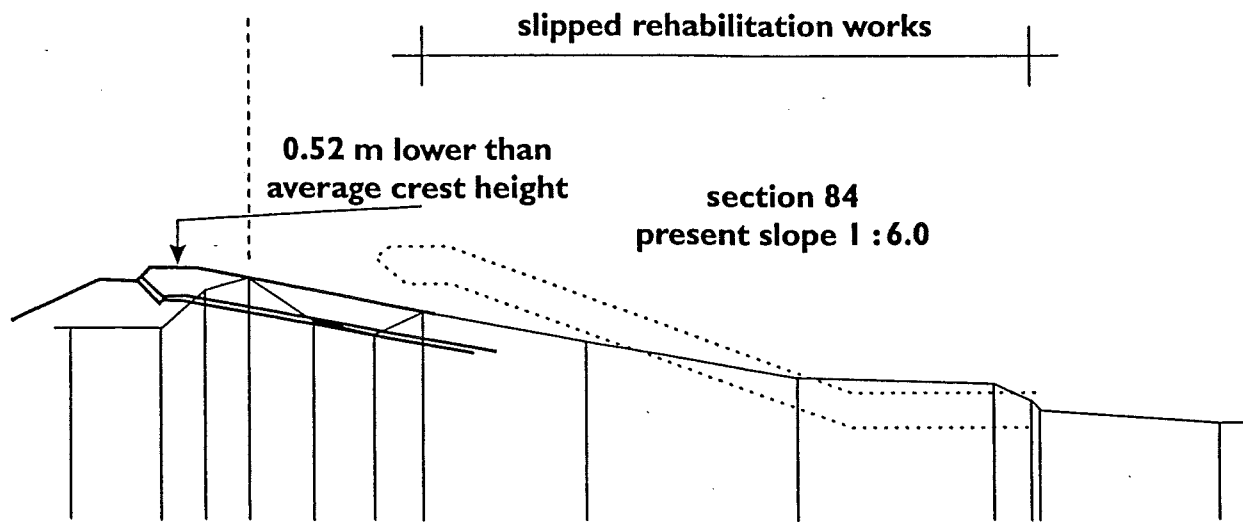
mzet

Input for settlement calculations

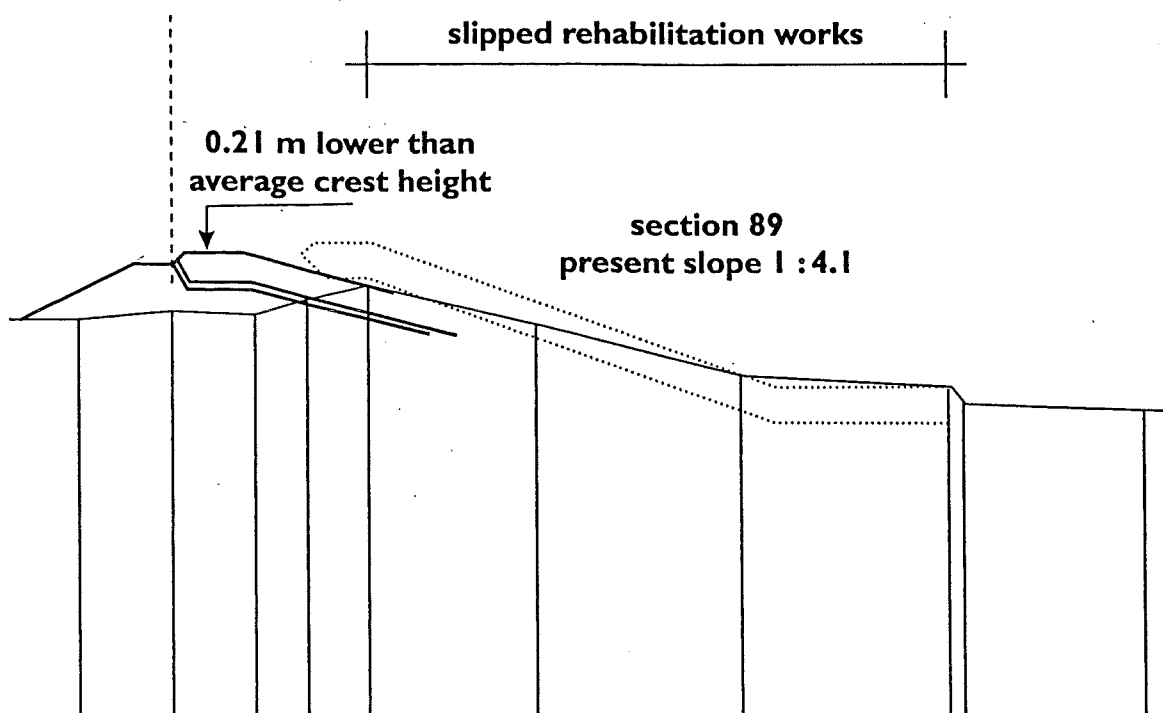
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FIG. 2.5.5



Details according to original design

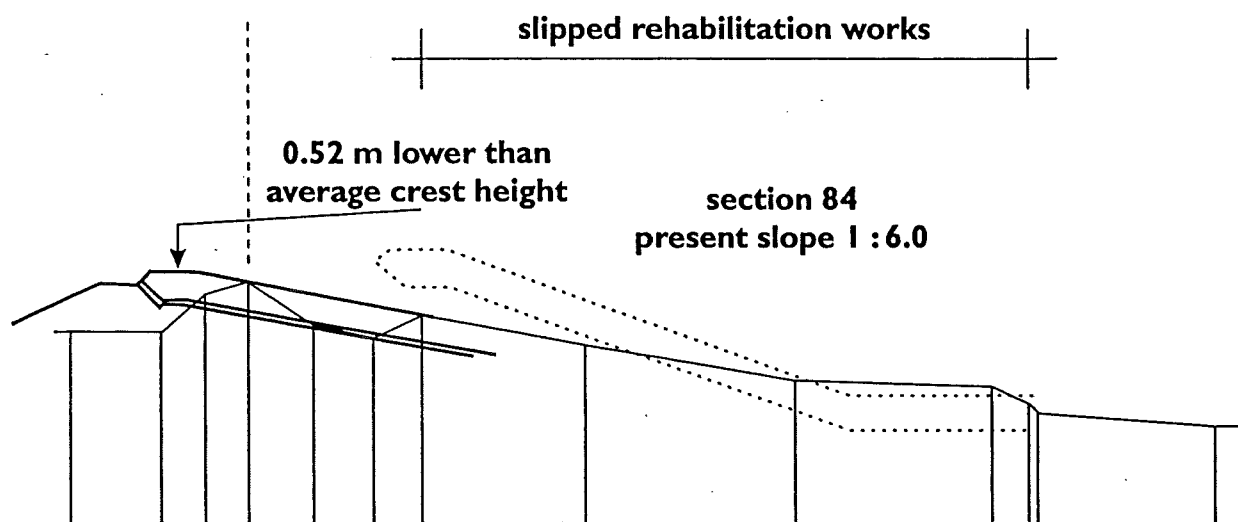


Proposed permanent repair works at sections
84 and 89 Area B

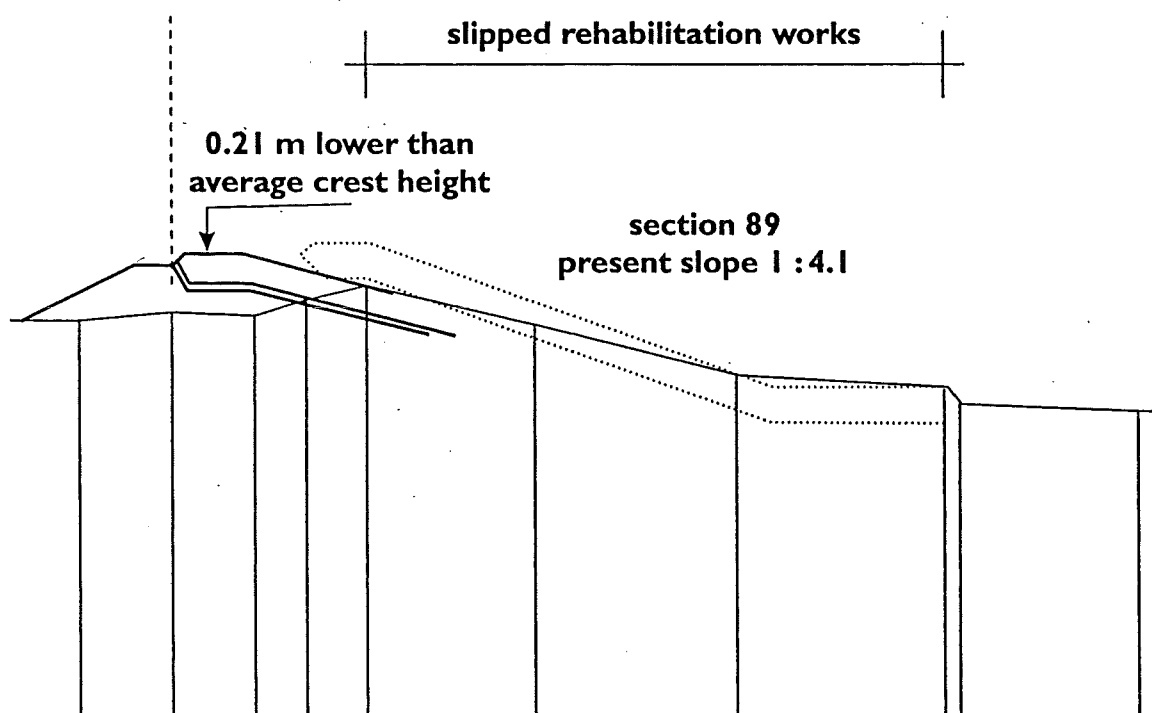
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FIG. 4.1.2



Details according to original design

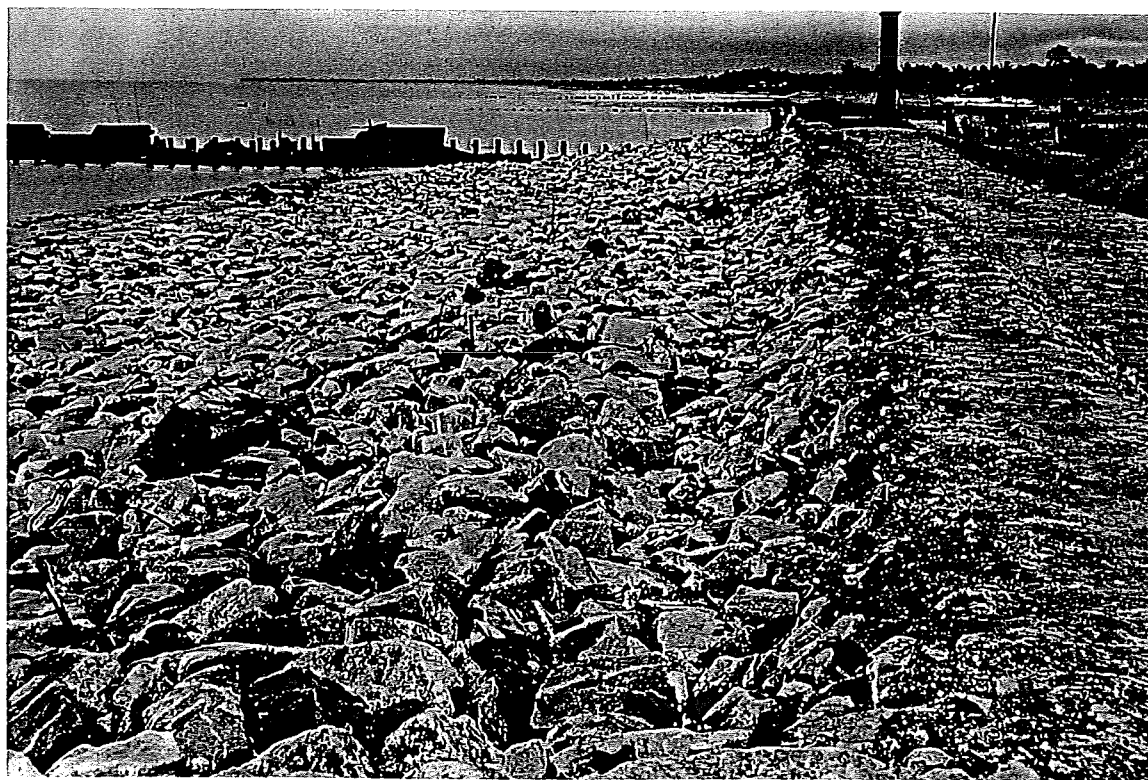


Proposed permanent repair works at sections
84 and 89 Area B

DELFT HYDRAULICS

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FIG. 4.1.2



Constructed and stable section (above)
and slip failure at Anna Regina (below)