Engineering review on the final closure of Saemangeum Dike

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Summary

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HR Wallingford has completed an engineering review on the Final Closure of Saemangeum Dike. This final report contains detailed matters which deserve consideration by KARICO, but the following general conclusions may be made:

1. Much of the work that has been carried out by KARICO and RRI is of excellent quality and only deserves some small comments. However, there are a small number of issues that do require serious attention.

2. Numerical model results were not available to provide mid-gap velocities for all days of the closure operation. An interpolation routine was used to derive the missing values for the purposes of estimating stable stone sizes. This interpolation routine appears to be satisfactory, but has been difficult to estimate the velocities in the final few days of the closure operation (final days of Phase3). This is because the interpolation routine becomes an extrapolation process for these few days and because changes in flow distribution will occur as the flow area in the gaps reduces to below the flow area of the Garyeok and Sinsi sluices.

3. Scour either side of the existing bed protection will remain a problem and will become worse as velocities increase during the final phases of closure. We have considered the processes taking place and recommend that the bed protection be extended by a further 50 metres either side of the dike centre-line

4. When estimating stable stone weights, the increases from estimated mid gap velocities to peak velocity, for example at the progressing ends of the closure bunds, has not been taken into account. We have applied appropriate speed up factors varying between 5% to 14% to allow for this, but the presence of flow asymmetry means that these increases may be exceeded. We have also allowed for high turbulence, which will be particularly evident in the vortex streets emanating from the ends of the dikes.

5. We make recommendations for increases to the stone weights and/or proportions of gabions to take account of these larger velocities. These changes are significant, requiring more heavy stone (up to 6t in weight) and higher proportions of gabions. In some cases modifications to the existing sill and bed protection will be necessary. Making appropriate modifications will require serious attention by KARICO in the following respects:

    i. To ensure that appropriate stability criteria have been adopted for all materials to be used. RRI have carried out very useful physical modelling, but not all material weights and combinations of gabions for bed protection, sill and closure bund were covered by this work. We have attempted to fill the gaps in understanding by the use of published stability formulae, but further physical modelling to confirm our results would be advisable.

Summary continued

ii. To ensure that the financial and physical resources necessary to support these design and construction changes are put in place.

6. On balance, we prefer the April-May closure period over the March-April period. This is for two reasons:
(a) the flow velocities are generally slightly lower in the gaps, and, although the differences are not usually enough to affect the choice of stone size, the lower velocities give a greater margin of safety.
(b) the probability of wave overtopping reduces from about 1-2% to about 0.5%
However, should KARICO have overriding construction or other reasons to adopt the earlier period and can accept these risk increases the earlier period can be used satisfactorily.

7. To the extent that information has been provided to us, procedures for construction appear to be satisfactory

8. The problem of water leakage through the (extended) bed protection layer after final closure has been completed is significant. A strategy involving carefully timed pumping of gravel and sand into the closure bund and bed protection layer is recommended to solve this problem. This approach could also allow the filter on the back of the closure bund to be omitted if desired.

Contents

Contents continued

Tables

Contents continued

Figures

Contents continued

1. Introduction

By a contract with the Rural Research Institute (RRI) of KARICO dated June 2005, HR Wallingford undertook to carry out an engineering review on the Final Closure of Saemangeum Dike.

The Saemangeum project comprises the construction of 33 km of sea dikes which will enclose an area of reclaimed tidal flats of $282.4km^2$ and a desalinised reservoir of $117.6km^2$. During the closure of the dikes, very high flows will develop through the gaps. Studies previously carried out at RRI and at HR Wallingford anticipate that these flows could exceed 6 m/s. The purpose of the consultancy services is to review the hydraulic boundary conditions, construction schedule and the stone stability for the final closure of Saemangeum dike and, based on the study and judgment by proper tools and measures, to provide findings and recommendations which are practically acceptable for the final closure.

The completion of the studies by experts at HR Wallingford is marked by the issue of this final report (Version 3.0).

This final report has benefited from:

- a review by KARICO and by team members from the Netherlands, Prof Henk Jan Verhagen and Mr Hans van Duivendijk.
- Joint meetings with KARICO held in Wallingford UK during the last week of August 2005, which included a risk workshop.
- Meetings with KARICO held at RRI, Ansan, Korea in the second week of October 2005.

This report is structured as follows:

- Chapter 2 presents an introduction to the Saemangeum project based on information supplied to HR Wallingford by KARICO
- Chapter 3 presents a review and evaluation of the correctness of the hydraulic parameters used by KARICO based on a comparison with previous studies by HR Wallingford and other researchers. It also presents an assessment of hydraulic conditions on days during the closure process for which no numerical or physical modelling has been carried out. This assessment is based on an interpolation procedure based on the variation in the driving tidal water level gradients.
- Chapter 4 discusses the bed scour processes around the closure gaps in some detail.
- Chapter 5 presents a review of stone and gabion sizing and stability during final closure. Calculations are carried out using both internationally accepted stability formulae and these are compared with the physical modelling results of KARICO. Recommendations for design changes to the stone weights used for the bed protection, sill and closure dam are presented, allowing for the particular tidal currents to which the relevant part is exposed.
- Chapter 6 presents a review and evaluation of the applicability of the planned schedule and construction procedures for final closure, based on the information supplied to HR Wallingford by KARICO
- Chapter 7 presents a risk analyses for the final closure process, based on both a risk register and also fault and event trees

- Chapter 8 deals with a separate issue of the stability of filters within the permanent sea dike after the final closure of using the rock and gabion closure bund.
- Chapter 9 presents some key conclusions and findings for consideration by KARICO.

2. The Saemangeum project

2.1 GENERAL DESCRIPTION OF SAEMANGEUM PROJECT

The west coast of the Korean peninsular displays a frequently indented shoreline with a gently sloping sea bottom. The tidal range is so high that it reaches approximately 6 m in spring tide at the Saemangeum site. These favourable geographic and hydraulic conditions permitted the Korean Government to initiate several tideland reclamation projects along the coastline. The Korean Agricultural and Rural Infrastructure Corporation (KARICO) and the Ministry of Agriculture and Forestry (MAF) of the Republic of Korea launched a large-scale tideland reclamation project, the so-called Saemangeum Project, in 1991. Dike construction works separating the land from the sea were completed about 90% by length at the end of 2004. Only 2.7 km out of total 33 km remains to be completed to achieve final closure.

The Project site is located at the mid-west coast of Korean peninsular, approximately 200 km south from Seoul. The Project covers a total area of 401 km^2 which, after its completion, will be composed of 283 km^2 of reclaimed tidal flats and a desalinated reservoir of area 118 km^2. The major sea dikes required to enclose this huge area of the Saemangeum estuary also include two drainage sluices and navigation locks.

The watersheds of the Saemangeum reservoir total 3,319 km^2 and contain the basins of two major rivers, the Mangyeong and Dongjin, which flow into the reservoir, meandering through the plains. Water depths along the sea dikes vary from 4 m to 27 m below MSL (Mean Sea Level). Deep tidal channels have developed at three regions: south of Sinsi island; east of Yami island; and between Duri and Bukgaryeok islands. The thickness of fine sand deposits on the sea bed reaches to 20 to 30 m.

Some of the major engineering works that form part of the project include the following:

a) Sea dikes

total length	: 33.0 km
crest elevation	: EL.8.5 – EL.11.0 m above MSL
typical height	: 22.0 m
bottom width	: 290.0 m at typical section

b) Reservoir

NWL (Normal Water Level)	: EL.(-)1.5 m (as currently adopted by KARICO)
DWL (Dead Water Level)	: EL.(-)6.5 m
storage capacity	: 535.million m^3
available storage	: 355 million m^3
water surface area at NWL	: 96.7 km^2

c) Drainage sluices

Name of sluices	Sinsi	Garyeok
Location	east of Sinsi island	Bukgaryeok island
Gate - sill elevation - gate type - dimension - max. discharge	 EL.(-)6.50 m radial gate 30m x 15m x 10ea 8,812 m^3/s	 EL.(-)6.50 m radial gate 30m x 15m x 8ea 7,050 m^3/s
Navigation Lock - bottom elevation - dimension - gate type	 EL.(-)6.50 m 16m x 15m x 65m mitre gate	 EL.(-)6.50 m 4m x 15m x 30m mitre gate
Fishway		

2.2 SEA DIKES AND SLUICES

The Saemangeum project involves the offshore construction of sea dikes of a total length of 33 km. These dikes include access roads and two large discharge sluices as shown in Figure 2.1. A typical seaward cross-section of a sea dike is shown in Figure 2.2. The dikes connecting the islands scattered around the bay are Dike No. 1 to Dike No. 4, with locations as follows:

Dike No.1 (length 4.7 km) connects Daehang-ri to Garyeok Island.
Dike No.2 (length 9.9 km) connects Garyeok Island to Sinsi Island.
Dike No.3 (length 2.7 km) connects Sinsi Island to Yami Island.
Dike No.4 (length 11.4 km) connects Yami Island to Bieung Island.

As of March 2005, the status of the construction works of the dikes and sluices is as follows:

- Dike No.1, Dike No.3 and Dike No.4 have been closed;
- Construction of the Garyeok sluices has been completed and the sluices are now operational;
- Two gaps remain open in Dike No.2;
  - Gap No.1: 1,600 m (St. No.18 – No.34)
  - Gap No.2: 1,100 m (St. No.86 – No.97)
- The Sinsi sluices are still under construction. KARICO have advised that that they will be completed by the end of 2005 and will be operational (including complete removal of the surrounding cofferdam) well in advance of the earliest date for the final closure works of March 2006.

Survey control along the sea dikes is based on a series of stations every 100m, each dike starting with St. No.0. Thus for example Station 9 would be 900 m from St. No. 0.

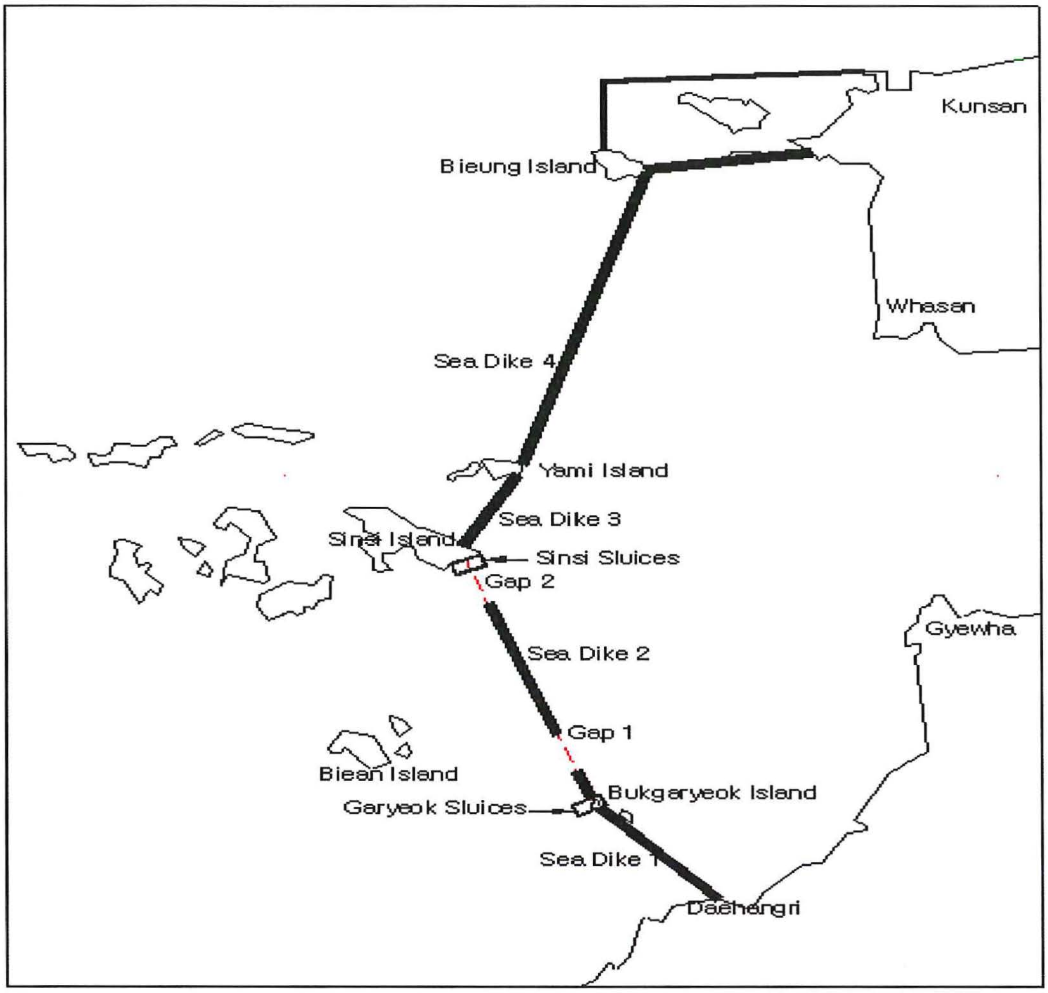

Figure 2.1 Alignment of Saemangeum Sea Dikes and Construction Stages as of March 2005

(thick lines = construction completed, dashed red lines = gaps remaining)

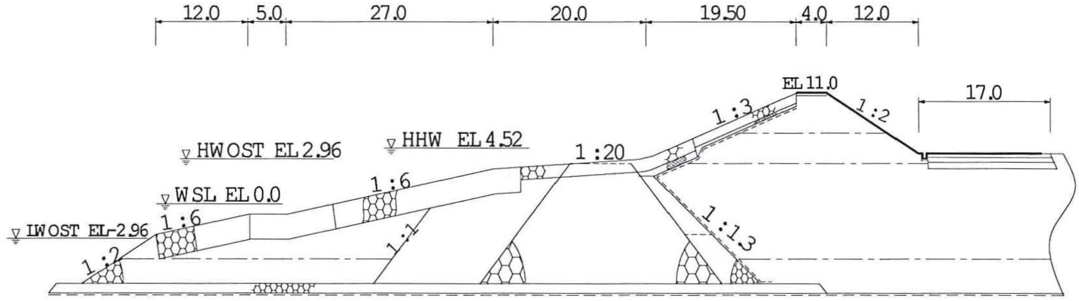

Figure 2.2 Typical seaward cross-section of Dikes

2.3 CLOSURE WORKS FOR SEA DIKES

As mentioned above, one of the major construction works includes the sea dike construction enclosing the Saemangeum estuary and incorporating two drainage sluices and navigation locks. The Saemangeum dike is the longest one ever constructed in Korea.

The final closure of the two remaining gaps in Dike No. 2 seen in Figure 2.1 will take place on both sides of each gap. During the period of closure, extremely high currents will be developed since a large amount of water flows in and out through the narrow gaps. Hydraulic model studies carried at Hydraulics Laboratory, RRI reveals that it is 6.5 m/s or more at the final stage of closing.

For more than 40 years, the Korea Agricultural and Rural Infrastructure Corporation (KARICO) have implemented the tideland reclamation projects along the west coastline of Korea for securing new agricultural land and water resources since early sixties. The final closure work of several projects had confronted dangerous situations due to lack of resistance to high speed of flow at the very final stage. In 1994, for example, the final closure of Sihwa Dike had been damaged and washed away at the last minutes, leaving a big gully scoured into the sea bed.

For the success of the final closure without a failure, many aspects have to be carefully considered such as closing sequences and construction periods, optimal weight and quantity of dumping rocks, sea bed scouring, stability of the bottom protection, etc. It is also essential to know the correct hydraulic boundary conditions to be able to evaluate and ensure good performance of the construction works.

Overall and detailed scale model tests have been performed for about 10 years and are still running at Rural Research Institute (RRI) in order to provide technical support for the Saemangeum dike construction works. The overall model is used to reproduce hydraulic conditions in the course of the final closing and the detailed one is used to predict the bed erosion pattern qualitatively due to the strong current around the gaps. Many field campaigns have also been made to acquire measured data for model verification and to facilitate construction site control. Numerical modelling of the flow conditions has also been carried out with KARICO's Delft 3D model.

The number of the closure gaps and the closure methods were decided based upon the following elements: closure gap dimensions; cross-sectional area of the individual gap; schedule of closure works for the final gaps; and tidal wave propagation through the final gaps. The method of closure influences decisions about the gap dimensions, but the nature of the gaps to be closed also influences the method. In other words, the two are interrelated. Therefore, the decision on how many gaps there are and what method should be adopted is critical to the success and cost of the project.

Based on numerous studies, KARICO has provisionally decided on the number of gaps and the method for final closure. The two remaining gaps shown in Figure 2.1 will be simultaneously closed: Gap 1 in the southern channel of Dike No. 2; Gap 2 in the northern channel of Dike No. 2. The exact dates for the closure process have yet be decided, but are expected to be over about a month during the period March to April or April to May of 2006. The method to be adopted involves a combination of vertical followed by horizontal closure:

- About 6 months before the final closure, pre-stored quarry stones and rocks are placed on top of the low dam by floating equipment such as stone dumping vessels, thereby implementing a sill construction to a desired level.
- The final closure is then built out horizontally by tipping on top of this sill using large size stone. Construction is achieved by a combination of end tipping using dump trucks and some marine plant placing side and end protection to the developing dike

The gaps expected to exist immediately prior to the final (horizontal) closure are expected to be as follows:

- Gap No.1: sill elevation -10 m below MSL; gap width 1,600 m
- Gap No.2: sill elevation -16 m below MSL; gap width 1,100 m

Both Garyeok and Sinsi sluices will be operated at the time of final closure.

HR Wallingford was requested to consider alternative timings for the final closure process as described in the following table:

			Gap1	Gap2
Stage	**Dates – earlier option**	**Dates – later option**	**Width (end of day)**	**Width (end of day)**
Start of Waiting Period No 1	30/03/2006	28/04/2006	1300	660
	31/03/2006	29/04/2006	1300	660
	01/04/2006	30/04/2006	1300	660
End WP No. 1	02/04/2006	01/05/2006	1300	660
Start of Final Closure Stage No 2	03/04/2006	02/05/2006	1230	628.19
	04/04/2006	03/05/2006	1160	596.38
	05/04/2006	04/05/2006	1090	564.56
	06/04/2006	05/05/2006	1020	532.74
	07/04/2006	06/05/2006	950	500.92
	08/04/2006	07/05/2006	880	469.1
	09/04/2006	08/05/2006	810	437.28
	10/04/2006	09/05/2006	740	405.46
	11/04/2006	10/05/2006	670	373.64
	12/04/2006	11/05/2006	600	341.82
End Stage No. 2	13/04/2006	12/05/2006	530	310
Start of Waiting Period No 2	14/04/2006	13/05/2006	530	310
	15/04/2006	14/05/2006	530	310
End of WP No.2	16/04/2006	15/05/2006	530	310
Start of Final Closure Stage No 3	17/04/2006	16/05/2006	463.75	271.25
	18/04/2006	17/05/2006	397.5	232.5
	19/04/2006	18/05/2006	331.25	193.75
	20/04/2006	19/05/2006	265	155
	21/04/2006	20/05/2006	198.75	116.25
	22/04/2006	21/05/2006	132.5	77.5
	23/04/2006	22/05/2006	66.25	38.75
End	24/04/2006	23/05/2006	0	0

Figure 2.3 provided by KARICO explains the various stages in the closure process in further detail. The waiting periods have been selected to coincide with the highest spring tides when currents will be at their maximum and successful placing of stone into these currents will be difficult.

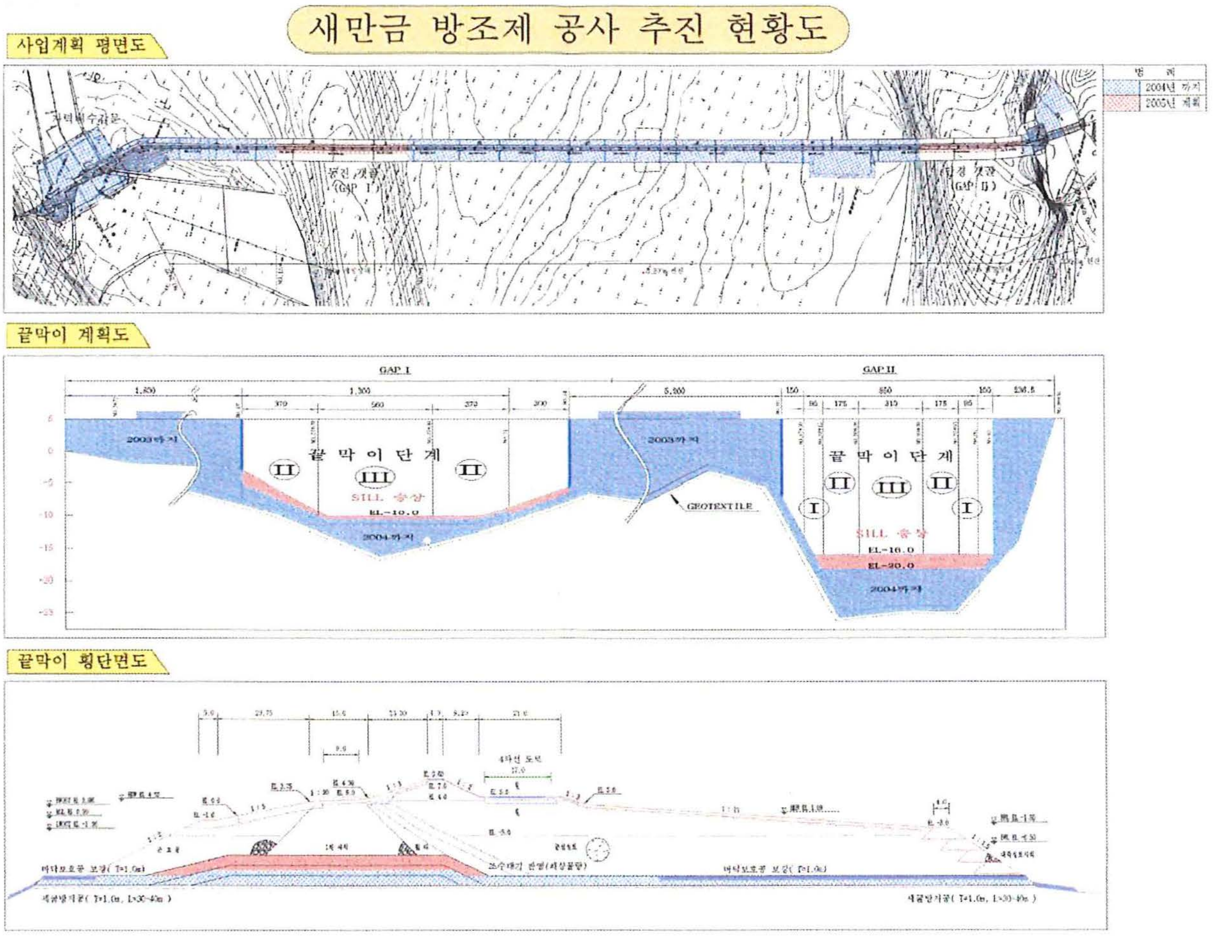

Figure 2.3 Arrangements for final closure as provided by KARICO

2.4 MANAGING BED SCOUR

The preparations for the final horizontal closure have more or less followed those originally envisaged. The only exception relates to the bed scour that has occurred either side of the openings. Bed protection was placed in advance of the gap narrowing to a variable width the maximum dimension of which was about 180m either side of the centreline of the dike.

Since placing of this bed protection at levels of the order of 15 to 20 m below MSL scour has occurred which in places has taken the bed level down to -50m below MSL, which is approximately the level of the underlying soft rock. The measured side slopes of the scour adjacent to the original bed protection are now almost at the natural angle of repose of the bed sediment at about 1:2 or even slightly steeper in places.

KARICO decided to instruct the contractor to place an additional strip of bed protection over the edges of the existing protection. The additional width of protection placed either side of the dike centre line was 30 m for Gap1 and 40m for Gap 2, increasing the total width of protection either side of the centre line to between 183m and 197m for Gap 1 and to between 218m and 232m for Gap 2. As a result, the bed protection now extends about halfway down the steep eroded face of the bed which had been generated either side of the originally placed bed protection.

The scouring processes and the need for further management of these is discussed in Chapter 4.

2.5 PROPOSED CLOSURE WORK PROCEDURES OF CONTRACTOR (HYUNDAI)

2.5.1 Basic infill of gap using land-based plant

Given the dimensions of the project, the following target rates of infill can be calculated:

Item	**Gap 1**	**Gap 2**
Cross-sectional area of closure bund	360 m^2	630 m^2
Additional equivalent cross sectional area to take account of turning bays[1]	36 m^2	50 m^2
Total equivalent cross-sectional area of closure bund	396 m^2	680 m^2
Required rate of progress per day (highest average - Stage No 2 & Stage No 3)	70 m/day	38.75m/day
Required rate of volumetric fill	27,720 m^3/day	26,350 m^3/day

KARICO's contractor, Hyundai, has advised that for each of Gaps Nos. 1 and 2:

- A total of 264 vehicles will be available per gap during the final construction stage
- These vehicles will be deployed on the pre-existing bunds either side of the gap
- Of these vehicles about 150 vehicles per gap will be small manoeuvrable dump trucks of capacity 15 tonnes (assumed able to deliver 7.5 tonnes filled bund volume.) These trucks will be deployed equally both sides of the gap.
- Construction will take place for 22 hours out of 24, the remaining 2 hours per day being allocated for work force shift changes
- On average one dump truck is expected to be discharging its load every 30 seconds on both sides of the gap.
- Assuming no delays, the total time it will take for one dump truck to load, travel to the discharge point, return and be ready to load again should be only 15 minutes.
- The loading points where the material is stored are only 1.5 to 2.0 km away from the ends of the bunds.
- The majority of the rock material is already quarried and in stockpiles ready for use.
- Gabions will be placed by end tipping from trucks in a similar way to that envisaged for the rock fill.
- Of the anticipated quantities of gabions to be placed with the rock boulders, some 50 to 60% will be placed by trucks on the progressing end face of the closure bund. The remainder of the gabions will be placed by marine plant (see below.)

[1] To facilitate truck queuing. and turning, the closure bunds will be widened approximately every 50 metres by a turning bay of width 8 metres and length 15 metres

- Backhoe machinery will be constantly available at the end of the dikes to assist in pushing into its final position the material dumped by the trucks.
- The contractor expects that the dumping efficiency (percentage of material placed that is retained on the ends of the bund) to be about 80%. However, the contractor advised that he had sufficient capacity to place faster than this should the dumping efficiency be smaller than anticipated.

KARICO have provided photographic evidence (see for example Figures 2.4) illustrating the kind of arrangement envisaged for the end of the dikes, based on the successful closure procedures adopted for the final closure of the gap in Dike No 3 in 2003.

Figure 2.4 Construction plant completing final closure of Saemangeum Dike No 4

2.5.2 Supporting operations by marine plant

Marine plant will be available to place the remaining gabions. Hyundai have advised that two kinds of equipment will be deployed:

- Bottom opening barges
- Side push barges.

This equipment will be deployed to place gabions to prevent the exposed sides of the progressing bund from being eroded. Gabions will be placed on the down-stream side of the bund at all times, whether the tide is in flood or ebb.

3. Review and evaluation of correctness of hydraulic parameters used by KARICO

3.1 REVIEW OF HYDRAULIC PARAMETERS PROVIDED BY KARICO

Prof Choi (2005) comments that the peak tidal current derived from his model of stage D of the closure process (gaps 1 and 2 both 500m) was 7.0 m/s which is in good agreement with the results of KARICOH/HR Wallingford of 6.98 m/s. This gives confidence that the models are consistent with each other in the closure up to at least this phase.

As the gap sizes subsequently reduce the requirements made on such flow models become more extreme as very fine resolution is required to simulate the current profile in the gap, the flows along the walls etc. The work by HR Wallingford (2002) found that for 100m gaps and for 50m gaps the currents were mainly somewhat stronger on ebb than flood and stronger in gap 1 than gap 2. However such assessments with all models are affected by the above mentioned problems of resolution.

All of the simulations show currents which are in the subcritical flow regime because of the water depth in the gaps. Should the depths in the gaps be reduced during closure then the possibility of critical flow occurring would exist.

Examining HR Wallingford flow results for 560/310m and 100m gaps (Figures 3.1 and 3.2) it can be seen that the water level inside the basin starts to rise when the water level in the sea is greater and continues to rise slowly until the water level in the sea falls enough for the water levels to equalise. The water level inside the basin then slowly falls. This means that at the time of highest flows through the gap the water level inside the basin is intermediate between the highest and lowest value.

In the figures it can be seen that in order to balance flows in and out of the Saemangeum tidal basin (with the peak inflow near high water and the peak outflow near low water), the peak ebb current is stronger than the peak flood current. The water levels inside the basin at the times of peak ebb and flood tide are about +0.2m MSL. There is an offset of the water level above MSL in order to provide the larger ebb current speed compared with flood tide current.

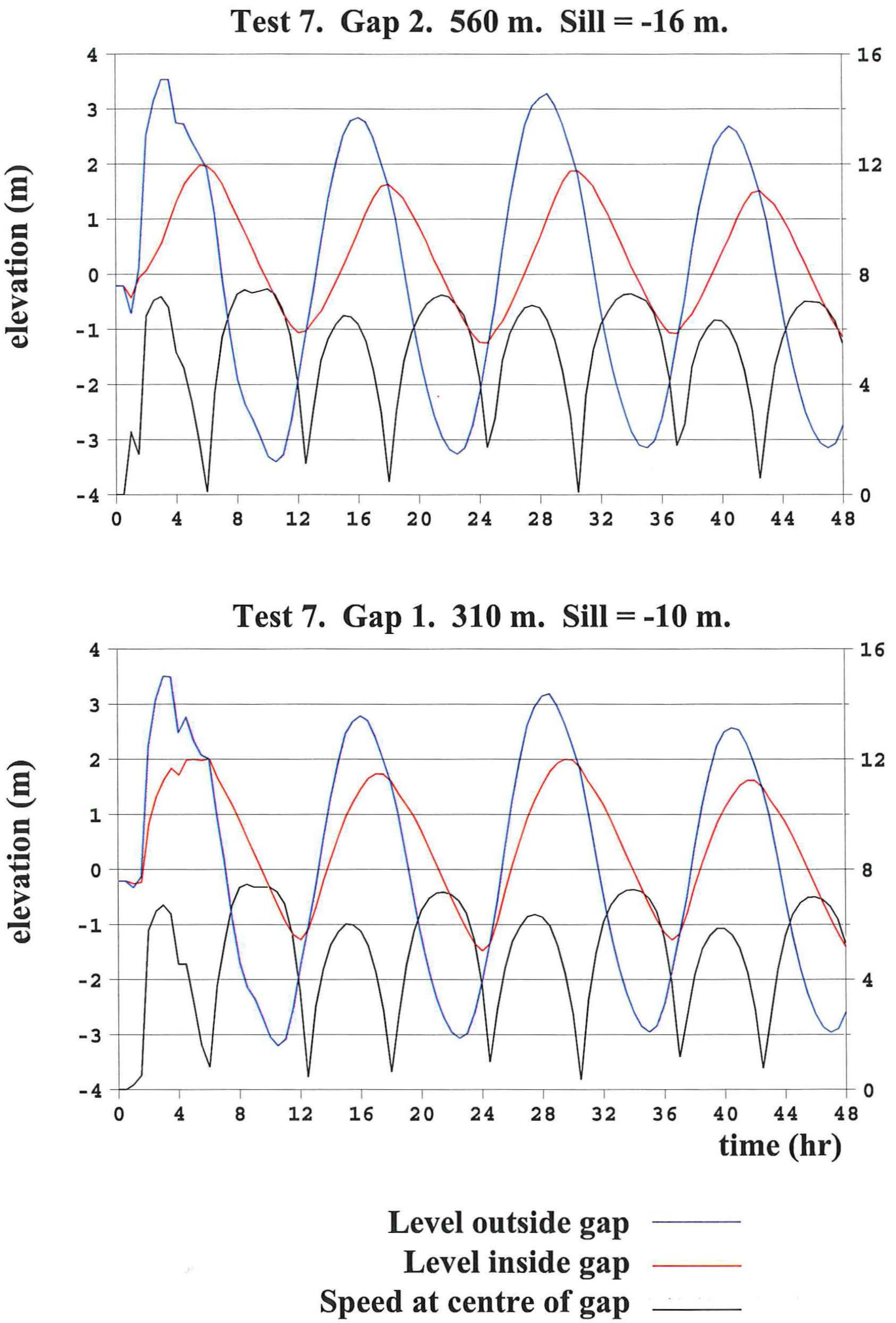

Figure 3.1 Speed at centre of gaps 1 and 2 and water level inside and outside of each gap for HRW Test 7

(Spring tide, Gap 1 = 560m, Gap 2 = 310m)

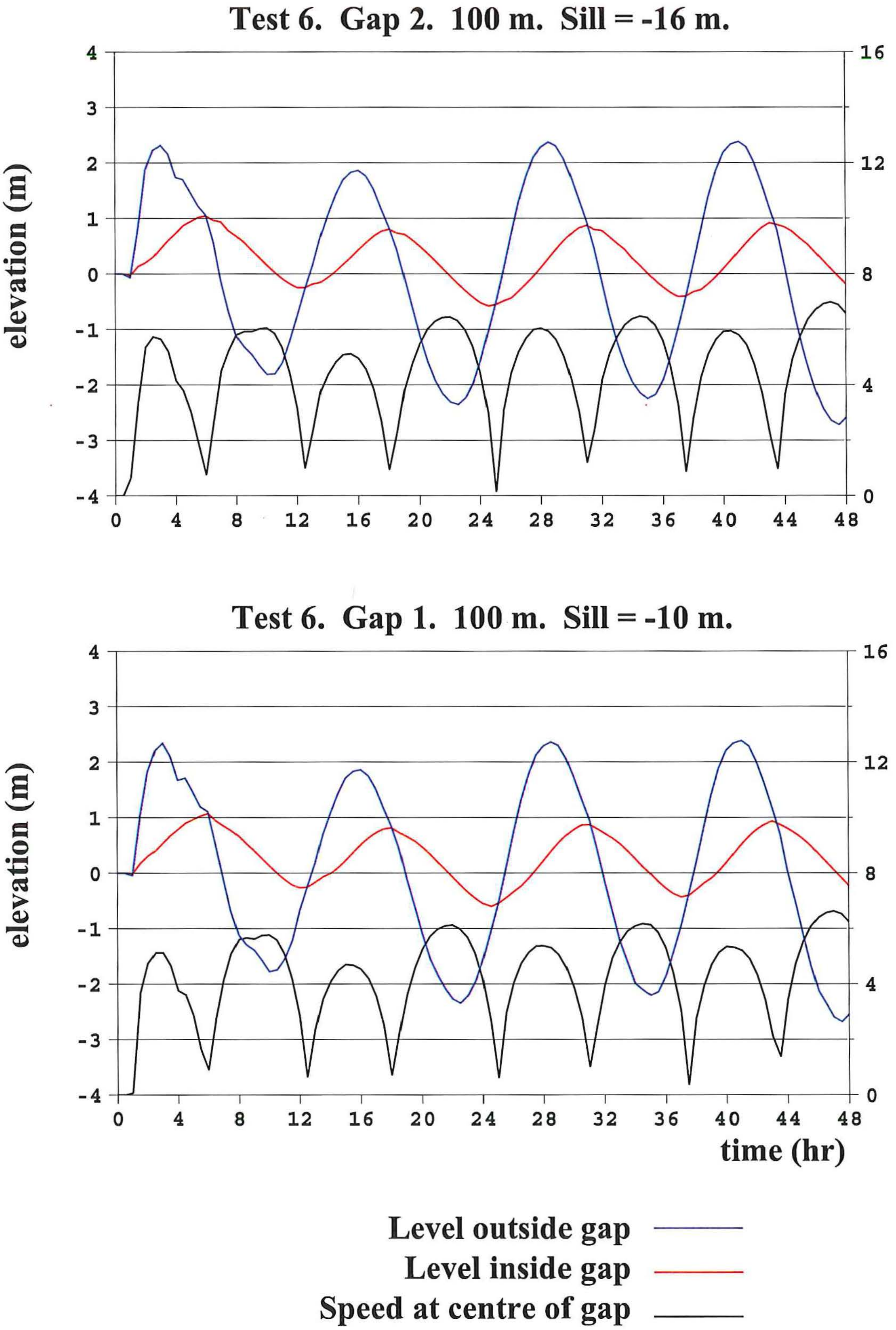

Figure 3.2 Speed at centre of gaps 1 and 2 and water level inside and outside of each gap for HRW Test 6

(Neap tide, Gap 1 = 100m, Gap 2 = 100m)

3.1.1 Comparison with formulae in CUR (1995) Rock Manual

The numerical model results of HR Wallingford, KARICO and Prof Choi are the best available approximation to the current speeds expected to be experienced in the gaps during closure. However for this kind of model problems of resolution of fine details of the flow pattern are experienced because the model also has to simulate the entire flow within and outside the 30km long basin as well as flows in the gaps that may be 50m wide or less. The flows include important details down to scales of only a few metres.

For this reason it is appropriate to consider what is known of empirical relationships that have been found to give adequate approximations to gap flows for situations such as this. The value of these empirical formulae depend upon the extent that physical modelling supports the formula and to the extent that the physical situation to be simulated is adequately similar to the physical models used to validate the empirical formulae used.

The Rock Manual gives various formulae and also some physical model results for particular parameter combinations. Unfortunately the situation at Saemangeum during final closure is complex for the following reasons:

> In the large basin water is exchanged with the sea outside both via the gaps being closed and also via the sets of sluices. The use of sluices means that even immediately after final closure there will still be a tidal range inside the basin as long as the sluices are open.
>
> The sills of the gaps are made in the flow direction, approximately 40m. As a result the sills do not function entirely like classical weirs as there is extra energy dissipation due to the strong tidal flow along the length of the sill.

This means that the particular gaps at Saemangeum do not correspond very closely to physical model relationships described in the Rock Manual. The general formula used for the average current speed across the gap is

$U_{av} = \mu \sqrt{(2g \text{ x level difference})}$

The multiplication factor μ is a coefficient that has to take account of all aspects of the geometry of the gap, width, shape, and sill breadth.

To investigate whether the model conforms to such a formula the results of the HR model for three stages of closure were tested for obedience to this formula. It was found (Figure 3.3) that the formula approximately agreed with the model but that for a wide gap the coefficient μ was about 0.7 and it grew as the gap narrowed up to a maximum value of about 1.1. These results are as might be expected although the manual suggests that generally a coefficient μ of the order of 0.9 is appropriate. In the context of horizontal closure it also suggests that values of μ rising to 1.3 or so may be possible as closure proceeds.

The reason we believe that the model is giving rather lower coefficient values (and consequently rather lower velocities in the gaps) than the formula would suggest (with a different choice of coefficient) is because the sills in the gaps are extensive and energy is lost in a current of about 5-6 m/s crossing a width of 40m of rocky sill. The total energy loss by dissipation on the sill becomes a less significant part as the hydraulic loss increases and this is why the coefficient rises during the closure process.

The agreement between the model results and the Rock Manual formula provides confidence in the model results, particularly as the reason for the low value of μ can be explained.

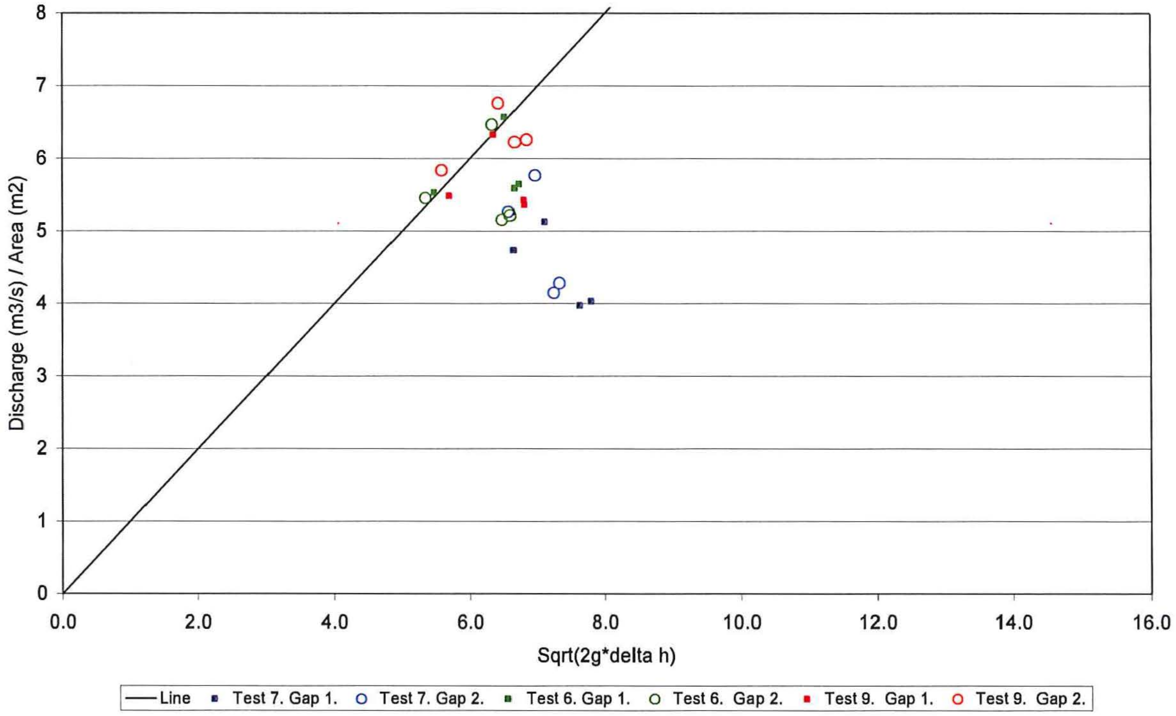

Figure 3.3 Relationship between gap centre speed and water level difference across gap, tests 6, 7 and 9

Notes:
Test 6 Neap tide gap 1 100m, gap 2 100m
Test 7 Spring tide gap 1 560m, gap 2 310m
Test 9 Neap tide gap 1 50m, gap 2 50m
Angled line represents x = y (analytical solution with coefficient = 1)

3.1.2 Comparison with work by Prof. Byung Ho Choi

Professor Choi states that the difference of tidal elevation between the inside and the outside of the barrier gradually increases as construction progress and therefore the velocities through the gaps will be greater than just those due to tidal currents due to the water elevation gradient from the inside to the outside of the barrier. However as stated above the water level inside the basin at the time of peak flood and ebb is already close to mean sea level so further reduction of the range inside of the basin caused by narrowing the gaps will not affect the current greatly.

A further effect during final closure is that the sluices, one of which is situated close to each gap, become important when their area of opening exceeds the combined size of the two gap openings. This occurs at about the time that the two gaps are 150m wide.

The change to the tidal range inside the basin is therefore not extreme during the last two days of the closure process.

3.1.3 *Results of KARICO Delft 3D model for 8th April 2005 compared with ADCP measurements.*

The KARICO Delft 3D model has been run to simulate flows on the 8th April 2005 when ADCP cross sections through the centre of the two gaps were surveyed to establish the peak flows at the centres of the two gaps.

The observed current at 12:50 at Gap 1 is shown as 4.58 m/s with a depth averaged value of 4.47 m/s. Shortly afterward the peak was 4.74 m/s with depth averaged value of 4.14 m/s. It seems that the peak in the water column may be 14 % more than the depth averaged value although this was exceptional. The model predicted velocity at the centre of Gap 1 at this time is about 4.46 m/s.

In Gap 2 the observed ebb tide current had a peak value of 5.32 m/s at 18:23 with a depth averaged value of 5.2 m/s, shortly followed by a peak of 5.57 m/s with a depth averaged value of 5.1 m/s. The model predicted a peak velocity of 5.24 m/s but at the time of the observations this had decreased to 3.9 m/s. Whether a larger peak current existed earlier than at the observed current time is not clear.

Interpreting these comparisons, it appears that more data and model/data comparisons are needed to be more confident about the accuracy of the KARICO Delft3D model. The data show that near to features of the bed topography it is possible for the local current to exceed the depth averaged current by 12% or so and this factor should be applied to any depth averaged currents if the largest point current is required. The HR Wallingford (2002) physical modelling work also showed that at the edges of the gaps the current is expected to exceed the mid gap value by up to 15%.

3.2 ESTIMATING CURRENT VELOCITIES ON DAYS ON WHICH NO NUMERICAL MODELLING IS AVAILABLE

In order to carry out a comprehensive assessment of stone stability (see Chapter 4 of this report), it was necessary to prepare a detailed schedule of maximum currents for all days during the final closure process

Results are available from KARICO modelling of the mid-gap, peak, depth-averaged current for the final closure during the following periods

1. Waiting period 1 when the widths of gaps 1 and 2 are 1300m and 660m. This phase corresponds to spring tides when the currents are too strong to continue closure.

2. Waiting period 2 when the gap widths are 530m and 310m respectively. Again this period is one of spring tides.

3. Four days before final closure when the width of Gap 1 is 265m and of Gap 2 is 155m. At this stage and from then until final closure the tides are neap tides. Consequently the mid-gap currents at this phase of closure are found to be less than those during the waiting stage 2 despite the gaps being half as wide as before. This comes about because of the smaller neap tide which gives rise to lower currents. If this phase of closure remained in place into the following spring tide periods the currents in the gaps would be very large.

As we have only the modelled currents for these periods and not for every tide of the closure period an interpolation procedure has been used to approximate the results it is expected that the model would give if it had been run for every tide of the closure sequence.

The interpolation method was based on supposing that the peak current speed during a tide would depend largely on the maximum rate of fall or rise of the tidal water level, corresponding to peak ebb and flood currents respectively.

When the model results were subjected to this assumption the results were found to fit well for the first waiting period. During the second waiting period it was found that there was a clear relationship between the maximum rate of rise or fall of the tide and the peak current but the relationship was different compared with that for the first waiting period. Within the range of values found there was an approximately linear relationship, this is shown for the Waiting Period 1, Waiting Period 2 and for gap widths equivalent to four days before final closure in Figures 3.4, 3.5 and 3.6 respectively. The equivalent relationships for other gap widths were then interpolated between those represented by the modelling results encapsulated in these three Figures.

It should be noted that, after the modelled result with widths of 265m and 155m (represented by Figure 3.3), the final three days are not an interpolation of the modelled results but an extrapolation. Different methods to extrapolate the relationship were tried and a satisfactory one chosen. Nevertheless such a procedure is too simplified as it does not take explicit account of the increasing role of the sluices in maintaining a tidal range inside the basin that would otherwise be smaller and would result in larger gap currents.

The tables of interpolated/extrapolated mid-gap current speeds are given in Tables 2.1 and 2.2. The features include a reduction of the current between waiting periods as neap tides are experienced and a rise after waiting period 2 as the gap is closing more rapidly than the tide range reduces. For the last few days there is less confidence in the extrapolation but as the gap becomes very narrow it is seen that an increase of the current is expected.

The final column in these tables reflects the increase from mid-gap velocities to peak velocities by a factor growing from 5% for a gap width of 1600m to 13% for gaps of 100m or less as found in the HR Wallingford (2002) physical modelling studies. We would point out, as we did in our 2002 report, that these tests were for specific situations and without any flow asymmetry. Some researchers have found local velocity increases of the order of 20%.

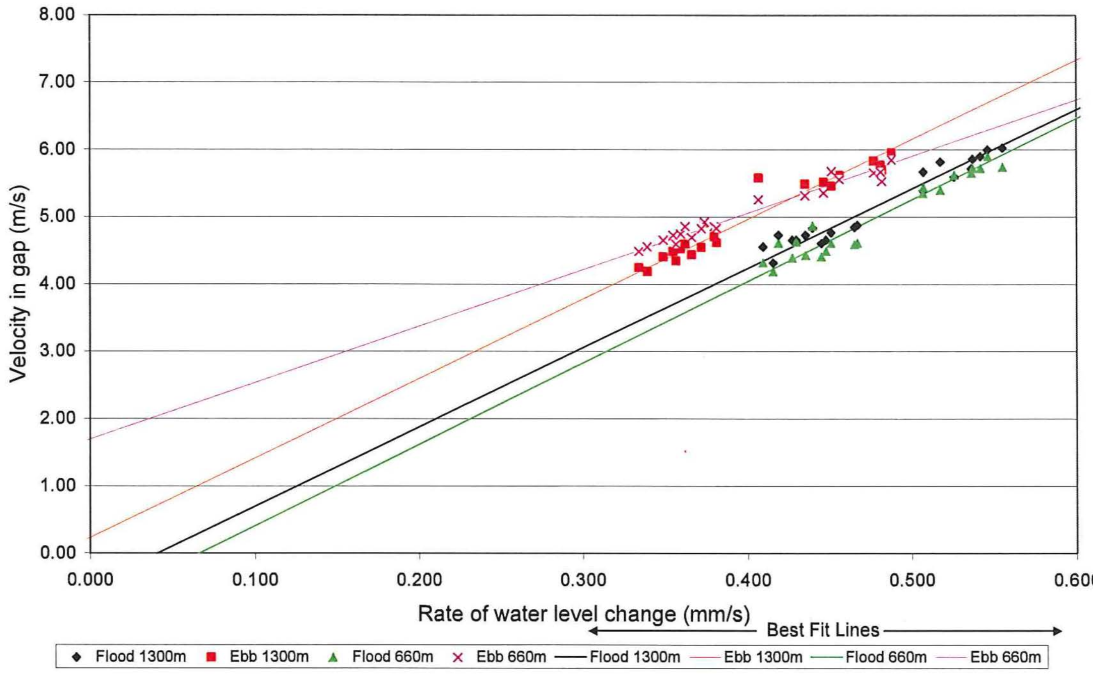

Figure 3.4 Relationship between rate of change of sea water level and modelled gap centre peak velocity. Waiting period 1.

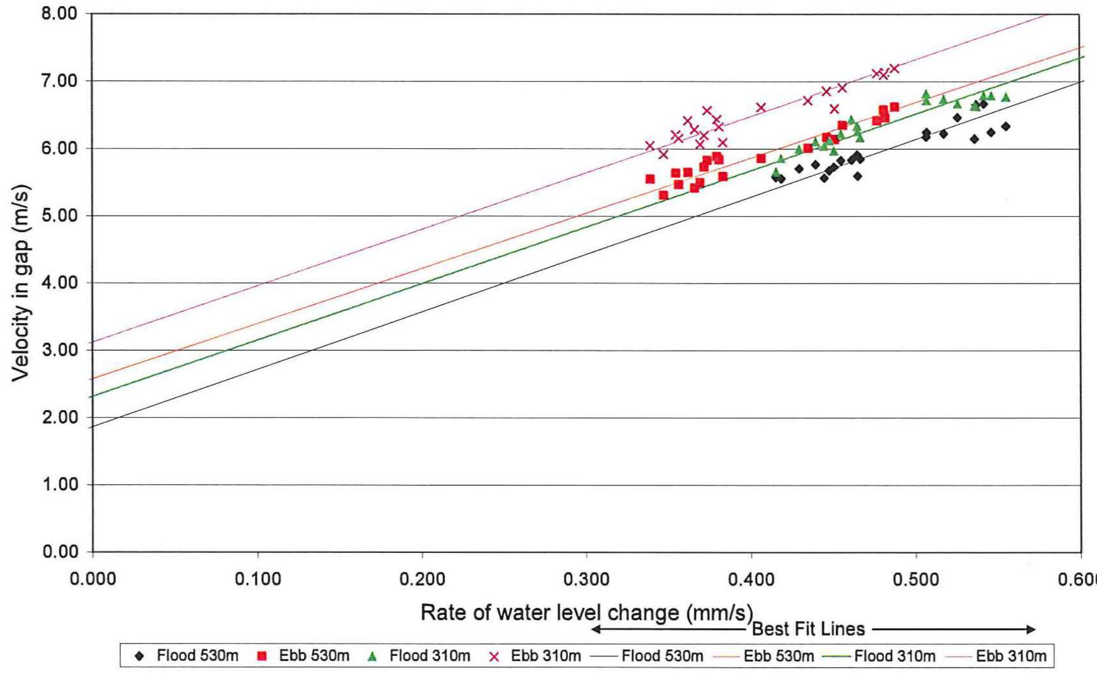

Figure 3.5 Relationship between rate of change of sea water level and modelled gap centre peak velocity. Waiting period 2.

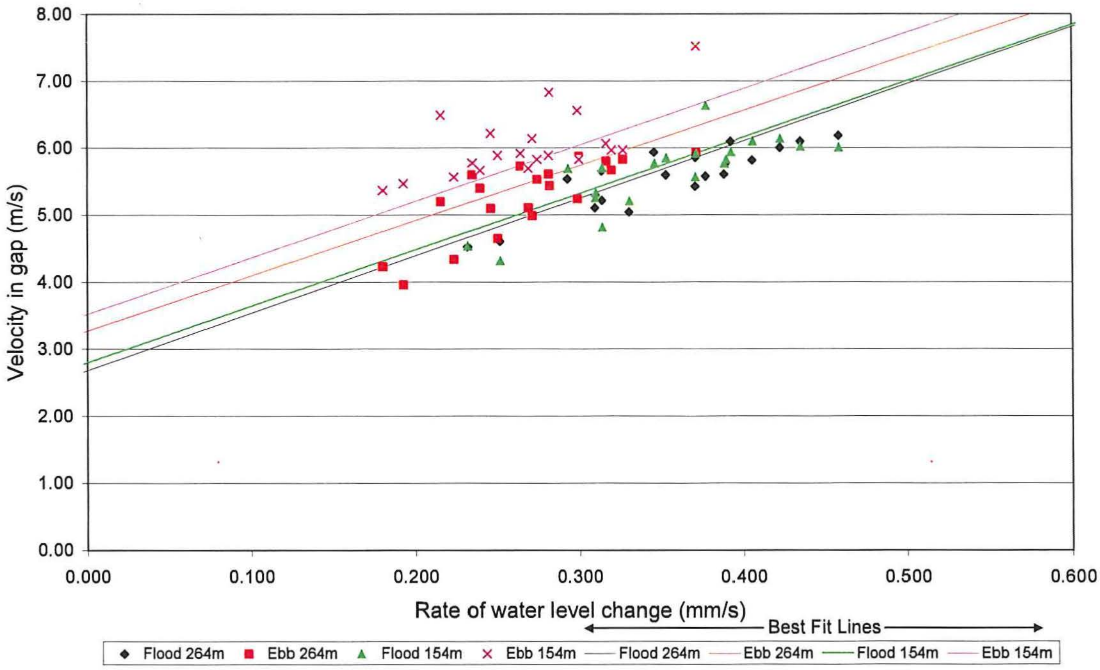

Figure 3.6 Relationship between rate of change of sea water level and modelled gap centre peak velocity. Layout 4 days before final closure.

Table 3.1 Interpolated and estimated velocities - Early Closure

Phase	Date	Gap width (m)	Mid-gap flow velocity (m/s)	Estimated peak velocity (m/s)
GAP I				
Waiting Period 1	30/03/2006	1300	5.70	6.08
	31/03/2006	1300	5.90	6.29
	01/04/2006	1300	5.86	6.25
	02/04/2006	1300	5.52	5.88
Final closure Phase II	03/04/2006	1230	5.02	5.37
	04/04/2006	1160	4.36	4.68
	05/04/2006	1090	3.63	3.91
	06/04/2006	1020	3.14	3.39
	07/04/2006	950	3.06	3.32
	08/04/2006	880	3.53	3.84
	09/04/2006	810	4.09	4.47
	10/04/2006	740	4.55	4.99
	11/04/2006	670	4.89	5.38
	12/04/2006	600	5.15	5.68
	13/04/2006	530	5.37	5.94

Waiting Period 2	14/04/2006	530	5.64	6.24
	15/04/2006	530	5.73	6.34
	16/04/2006	530	5.85	6.48
Final closure Phase III	17/04/2006	464	6.10	6.77
	18/04/2006	398	6.19	6.89
	19/04/2006	331	6.10	6.81
	20/04/2006	265	5.86	6.57
	21/04/2006	199	5.57	6.27
	22/04/2006	133	5.55	6.27
	23/04/2006	66	5.97	6.76
	24/04/2006	0	6.55	7.44
GAP II				
Waiting Period 1	30/03/2006	660	5.62	6.18
	31/03/2006	660	5.72	6.29
	01/04/2006	660	5.72	6.29
	02/04/2006	660	5.36	5.89
Final closure Phase II	03/04/2006	628	4.92	5.42
	04/04/2006	596	4.33	4.78
	05/04/2006	565	3.90	4.31
	06/04/2006	533	3.74	4.14
	07/04/2006	501	3.67	4.06
	08/04/2006	469	4.07	4.52
	09/04/2006	437	4.58	5.10
	10/04/2006	405	5.04	5.61
	11/04/2006	374	5.40	6.03
	12/04/2006	342	5.70	6.37
	13/04/2006	310	5.97	6.68
Waiting Period 2	14/04/2006	310	6.21	6.95
	15/04/2006	310	6.30	7.05
	16/04/2006	310	6.33	7.08
Final closure Phase III	17/04/2006	271	6.40	7.17
	18/04/2006	233	6.40	7.19
	19/04/2006	194	6.23	7.01
	20/04/2006	155	5.92	6.67
	21/04/2006	116	5.64	6.36
	22/04/2006	78	5.70	6.44
	23/04/2006	39	6.05	6.85
	24/04/2006	0	6.56	7.45

Table 3.2 Interpolated and estimated velocities - Later Closure

Phase	Date	Gap width (m)	Mid-gap flow velocity (m/s)	Estimated peak velocity (m/s)
GAP I				
Waiting Period 1	28/04/2006	1300	5.67	6.04
	29/04/2006	1300	5.82	6.20
	30/04/2006	1300	5.58	5.95
	01/05/2006	1300	5.16	5.50
Final closure Phase II	02/05/2006	1230	4.75	5.08
	03/05/2006	1160	4.23	4.55
	04/05/2006	1090	3.68	3.96
	05/05/2006	1020	3.27	3.53
	06/05/2006	950	3.33	3.61
	07/0 5/2006	880	3.61	3.93
	08/05/2006	810	4.00	4.37
	09/05/2006	740	4.38	4.80
	10/05/2006	670	4.70	5.17
	11/05/2006	600	5.04	5.56
	12/05/2006	530	5.36	5.93
Waiting Period 2	13/05/2006	530	5.58	6.18
	14/05/2006	530	5.57	6.17
	15/05/2006	530	5.60	6.20
Final closure Phase III	16/05/2006	464	6.12	6.80
	17/05/2006	398	6.26	6.97
	18/05/2006	331	6.24	6.98
	19/05/2006	265	6.10	6.84
	20/05/2006	199	5.93	6.67
	21/05/2006	133	6.11	6.89
	22/05/2006	66-	6.41	7.26
	23/05/2006	0	6.80	7.72
GAP II				
Waiting Period 1	28/04/2006	660	5.44	5.98
	29/04/2006	660	5.40	5.94
	30/04/2006	660	5.35	5.89
	01/05/2006	660	4.99	5.49
Final closure Phase II	02/05/2006	628	4.64	5.12
	03/05/2006	596	4.22	4.66
	04/05/2006	565	3.97	4.39
	05/05/2006	533	3.84	4.25
	06/05/2006	501	3.89	4.32
	07/05/2006	469	4.14	4.60
	08/05/2006	437	4.50	5.01
	09/05/2006	405	4.88	5.44

	10/05/2006	374	5.22	5.82
	11/05/2006	342	5.51	6.16
	12/05/2006	310	5.79	6.47
Waiting Period 2	13/05/2006	310	5.93	6.63
	14/05/2006	310	6.16	6.89
	15/05/2006	310	6.29	7.04
Final closure Phase III	16/05/2006	271	6.36	7.13
	17/05/2006	233	6.36	7.15
	18/05/2006	194	6.21	6.99
	19/05/2006	155	5.94	6.69
	20/05/2006	116	5.72	6.46
	21/05/2006	78	5.74	6.50
	22/05/2006	39	5.89	6.67
	23/05/2006	0	6.12	6.95

4. Review of scouring processes and need for extended bed protection

4.1 OVERVIEW OF THE SITUATION WITH RESPECT TO SCOURING

Bathymetric surveys taken of the seabed at Gaps 1 and 2 since October 2003 (2003.10) show the development of scour holes on each side of the bed protection mat and presently completed sills. The sill height at Gap 2 is higher than at Gap 1. Generally the scoured depths at Gap 2 are deeper than at Gap 1 in all comparative surveys; 2003.10, 2004.10, 2005.04 and 2005.06. The difference in bed levels experienced relates to the pre-existing shape and depth of channels either side of the dike alignment and the hydraulic conditions locally to each gap. The tidal flow field and interaction with the gaps has been modelled computationally by HR Wallingford and the results presented in their report EX4640. These results are useful in providing an interpretation of the driving forces for the scour that has developed at these two locations.

The pre-existing seabed bathymetry provides steering of the flow such that the approach direction of flow is not at right-angles to the gap. For Gap 2 – Figure 4.1 (Figure 2.19 from EX 4640) shows the peak flood flow vectors at a slight angle anticlockwise for Gap 2 whereas Gap 1 has a more straightforward approach. In Figure 4.2 the Ebb flow at Gap 2 approaches at an angle, passes through the gap and leaves at a smaller angle. Again at Gap 1 the approach is more normal to the dike.

The relationship with the scoured bed topography, reviewed from the engineering drawings provided by KARICO, is discussed here. Both Gap 1 and Gap 2 have scoured topographies that are at an angle to the gap and are not symmetric in plan-shape. For example, on the seawards side the scoured area is deepest at the north side and trends at an angle away from the gap. The scour profile data for 2005.04 was investigated to see whether there was any systematic pattern of deeper scour on the seawards or landwards side of the gaps. The data for both Gaps 1 and 2 at 300m from the dike centreline showed no such systematic pattern.

4.2 HYDRAULIC AND SEDIMENT TRANSPORT CONSIDERATIONS

The scour will have been caused by the following hydraulic processes:

- Flow acceleration at the gaps due to the reduced cross-sectional area at the gaps;
- Flow turbulence generated from the dike walls and the bed protection mat and sill; and,
- Transition effects in flow and sediment transport at the discontinuity between protected and non-protected areas of seabed

Over the natural seabed the tidal flow processes leading to sediment transport are related to the time averaged flow properties and the turbulence in the marine boundary layer. There may also be the effect of wind waves in stirring the sediment at the bed and making it available for transport by the currents. In the area of the works there is an appreciable acceleration of the mean flow speed caused by flow constriction and it will be expected that the absolute levels of turbulence in the flow will increase. Both these factors will increase the ability of the flow to transport sediment and lead to locally enhanced sediment transport and scouring.

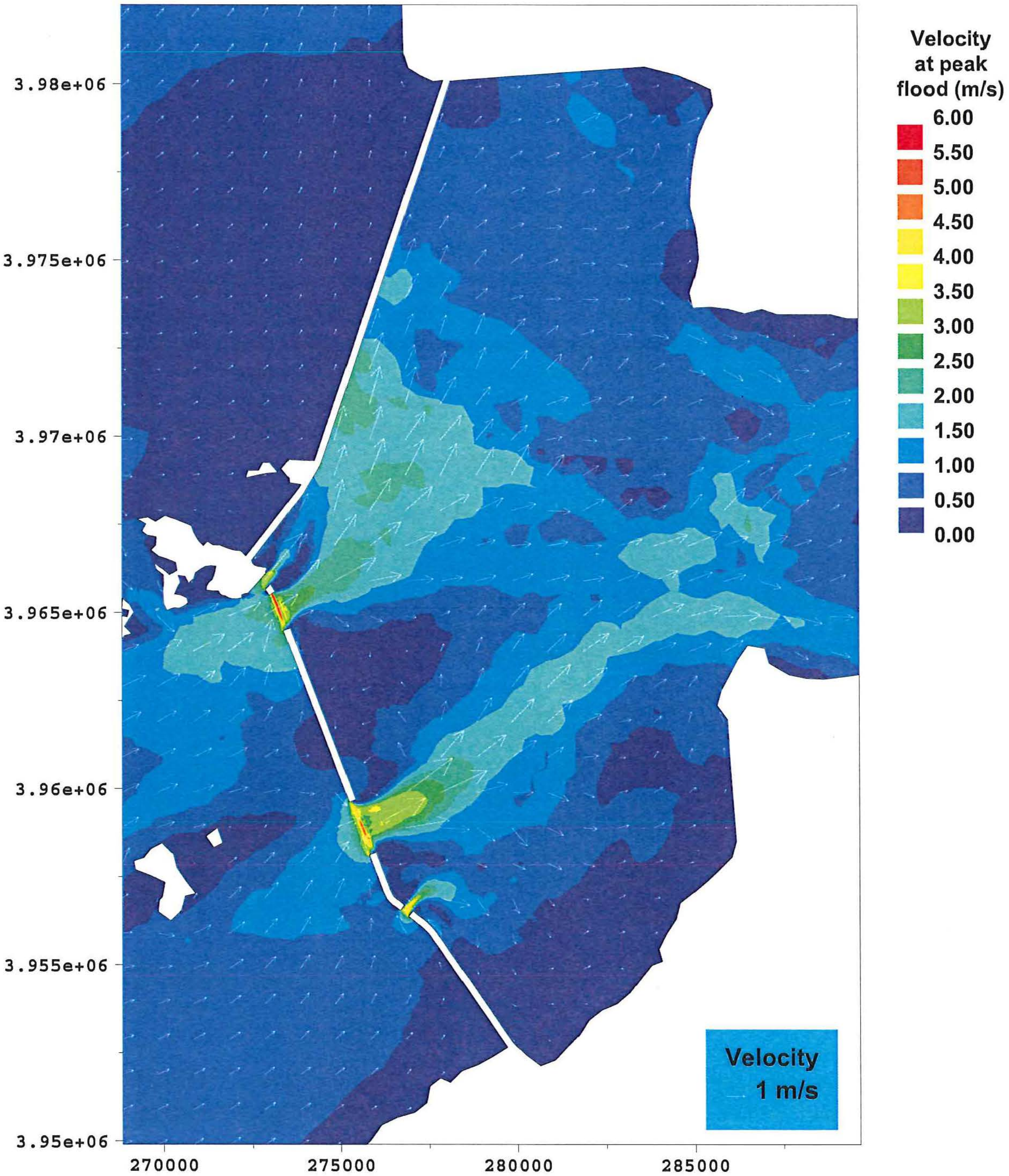

Figure 4.1 Current velocity vectors at peak flood, Test 1

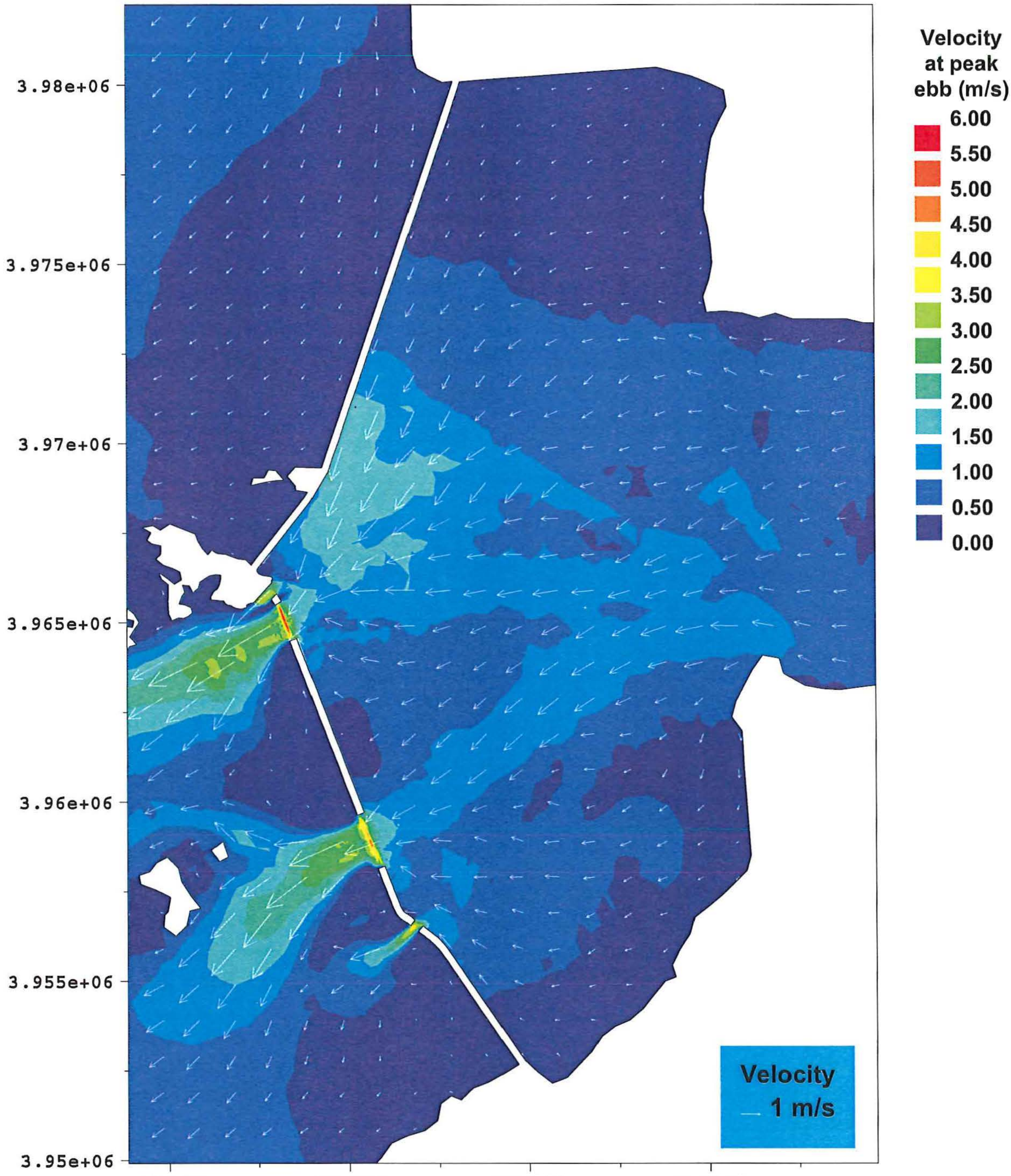

Figure 4.2 Current velocity vectors at peak ebb, Test 1

In a vertical plane (see Figure 4.3), the processes can be considered in 2-dimensions as follows. The flow approaches the trapezoidal cross-section of the dike gap over the open seabed which adjacent to the toe of the area covered by the bed protection mat has a slope in the range 1:8 to 1:3 (7 to 18.5 degrees). In the deepest scoured areas it can be steeper at 1:2 (26.5 degrees) and locally it can be as steep as 1:1.5 (33.7 degrees). The flow accelerates as it passes up the slope and into the gap, where the highest velocities are experienced, extending to the downstream side where the mean flow speed is noticeably faster for at least 500m. At the transition between the shallower and steeper sloping sections of the bed there will be high levels of turbulence due to the disturbance of the boundary layer. This will lead to a high local potential for sediment transport which can maintain the bed slope at this location, if the sediment transport is the same everywhere on the slope. The flow turbulence and intermittent flow separation on the downstream slope will lead also to enhanced sediment transport potential at that location. The more turbulent flow has the potential to erode sediment from the bed upstream and downstream and, in conjunction with the accelerated flow in the dike gap, to carry suspended sediment through and away from the gap.

As the flow speed varies through the tide the detailed 2-dimensional flow pattern will change. On the downstream slope in the early stages of the tide the flow will have a tendency to separate at the sections with steeper slopes, producing a recirculation with upstream directed flow at the bed. As the flow speed increases to the maximum in the tide this will tend to produce less intermittent QUERY separation on the downstream slope with high levels of turbulence.

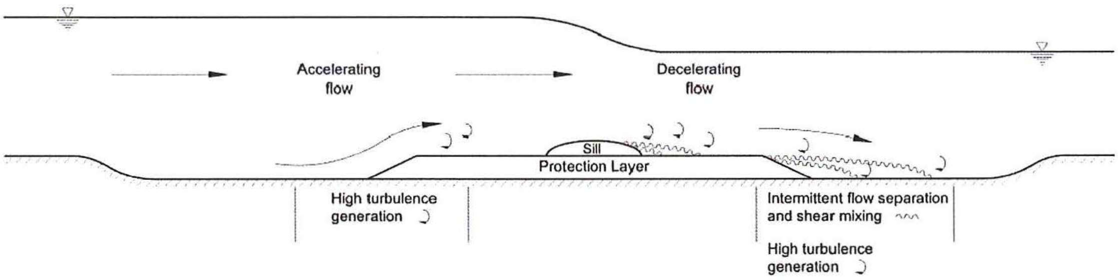

Figure 4.3 Schematic representation of vertical structure of flow field through closure gap.

In plan view (see Figure 4.4) the flow passes towards the gap and accelerates locally through the gap generating a zone of faster flowing water on the downstream side of the gap. The flow interaction with the side walls will generate shear layers at either side of the gap leading to vortex action with locally increased turbulence intensities.

It is expected that the deeper areas of scour measured at the toe of the bed protection mat on the seawards and landwards sides of the gap at both north and south ends of the gap are related to the shear layers generated from each side of the gap on flood and ebb flow directions.

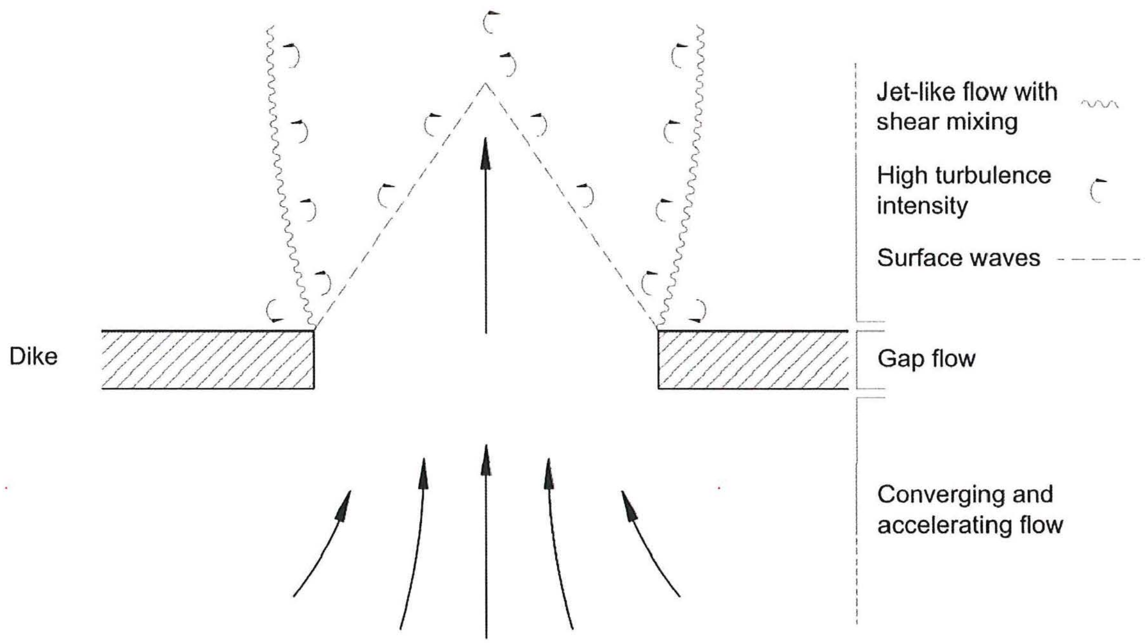

Figure 4.4 Schematic representation of horizontal structure of flow field through closure gap at peak tide.

4.3 PRESENT AND FUTURE SCOUR

If we assume that the scoured bed topography in the vicinity of the gap is presently (2005.04 survey) in equilibrium with the present flow regime then the question is whether the bed topography can expect to undergo further change during the final stages of the closure operation. This will depend on how the flow velocity changes through the final stages of closure and information on this is presented below.

The modelling results in EX4640 enable some interpretation of the change in hydraulic conditions as the gaps are closed. At Gap 1 we have taken the 1300m length gap (Figure 4.5) as the baseline against which to judge changes as the gap is closed. The 800m, 500m and 200m gaps (Figures 4.6, 4.7 and 4.8) produce faster flow in the zone downstream of the gap. The results in EX4640 on neap tides with the 100m and 50m gaps produce slower flows than those obtained on spring tides. A similar pattern of results is obtained for flood tide on Gap 2, using the 850m gap as the baseline (Figure 4.5).

In terms of ebb tide flow, similar increases in flow speed are predicted for both Gap 1 and Gap 2. It is only in the final stages of closure with the 100m and 50m gaps on neap tides that the flow speeds return to levels similar to that predicted for the baseline.

Because the flow modelling results for the reduced width gaps show that in the intermediate stages of closure the flow speed is increased, it is expected that the increase in mean flow speed will be associated with increased levels of turbulence. The gap closure works can lead to flow velocities that are greater than those presently experienced. Both factors will provide the potential for further scour either side of the bed protection. Another factor to be considered is the way in which the two shear layers generated from the sides of the gap will move together as the gap is closed; the moving zone of influence of the shear layers can lead to deepening of the bed either side of the bed protection mat similar to that presently experienced at the north and south locations in the existing bathymetry. The location of the increased scouring potential will be adjacent to the chainage positions of the gaps as they are closed and will vary with the tidal conditions experienced.

It is assumed that the sediment regime in the vicinity of the gaps will remain unchanged but if, for any reason, a sediment starved regime develops there will be less sediment being transported into the scour holes to maintain the previously obtained equilibrium. This might lead to a deepening of the scour holes unless they are constrained by the rock strata underlying the sedimentary deposits.

4.4 ENGINEERING RECOMMENDATION

The stability of the edges of the bed protection bordering the deep scour holes is a key factor for the whole closure project.

It is therefore recommended that KARICO should extend the existing bed protection by a further 50 metres either side of the centre line. However, it is probably only necessary to do this over the lengths open during final closure Phases II and III. The protection may of course be subject to some settlement as it will not be practical to place a geotextile before dumping of the stone. However, the additional stone will reduce the risk of scour and flow slides and act as a supporting berm.

The size gradation of the stone to be used for the additional bed protection is explained in Chapter 5.

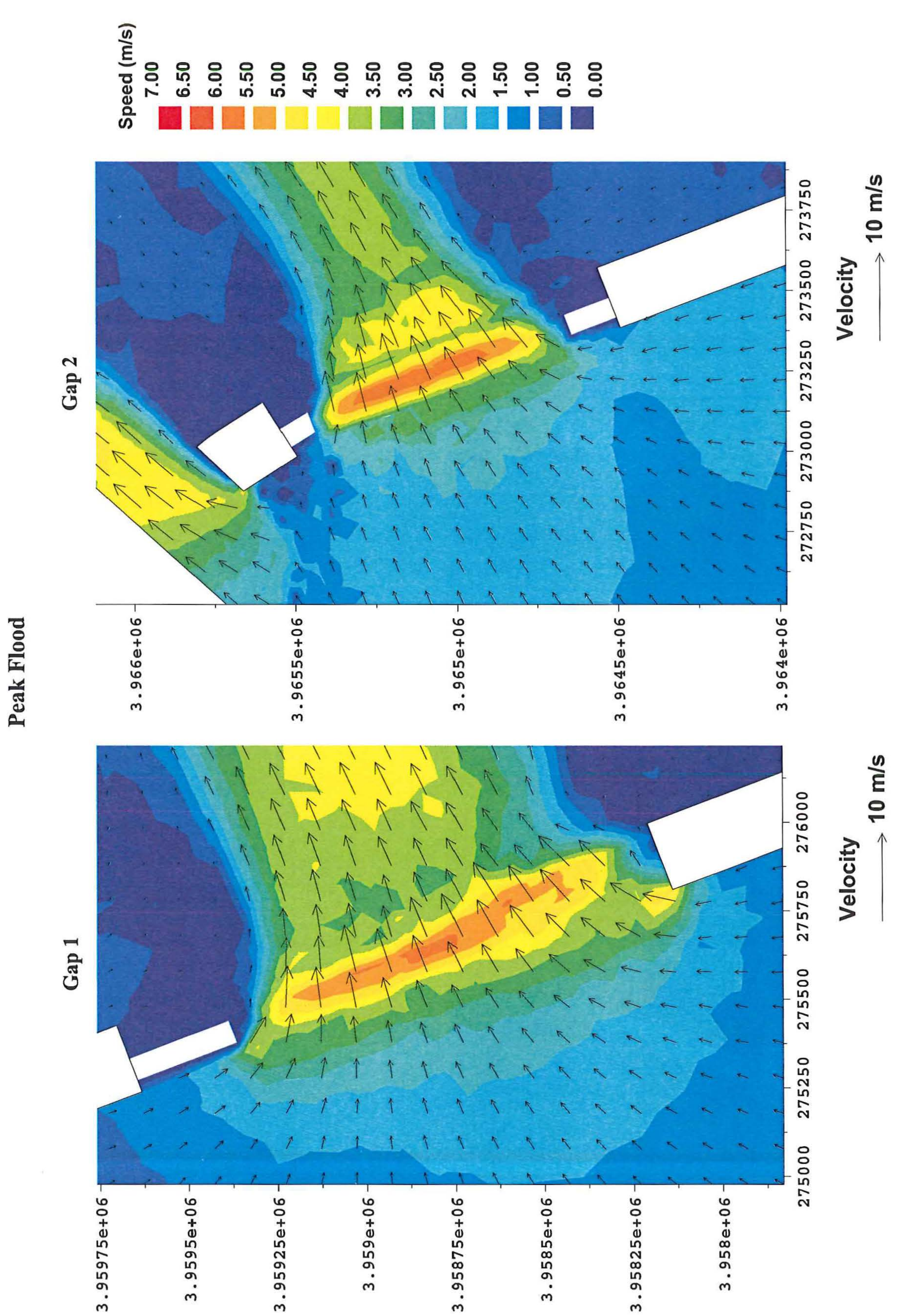

Figure 4.5 Current velocity vectors at peak flood. Spring tide. Test 3. Gap 1 : 1300 m. Gap 2 : 850 m

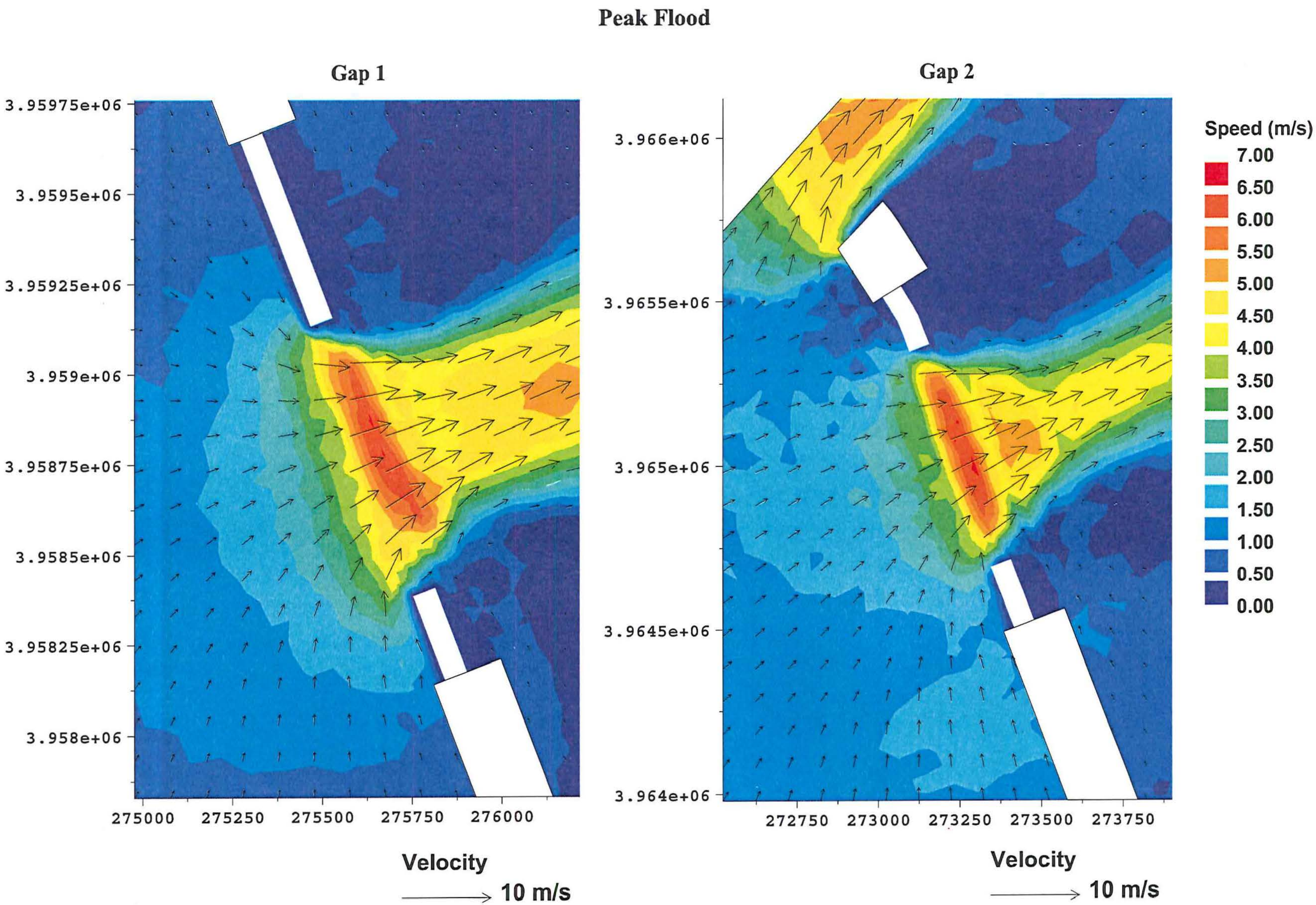

Figure 4.6 Current velocity vectors at peak flood. Spring tide. Test 4. Gap 1 : 800 m. Gap 2 : 700 m

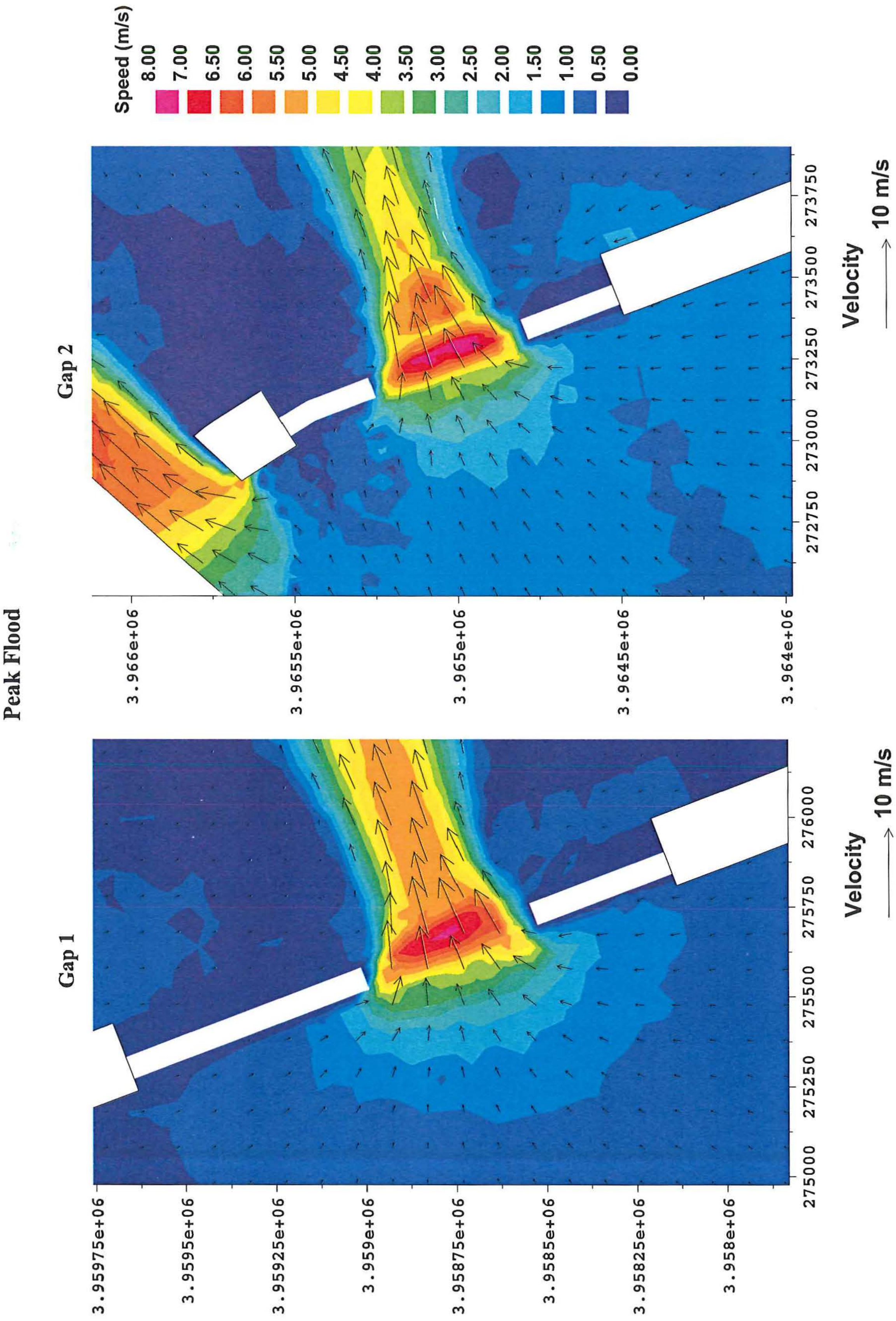

Figure 4.7 Current velocity vectors at peak flood. Spring tide. Test 5. Gap 1 : 500 m. Gap 2 : 500 m

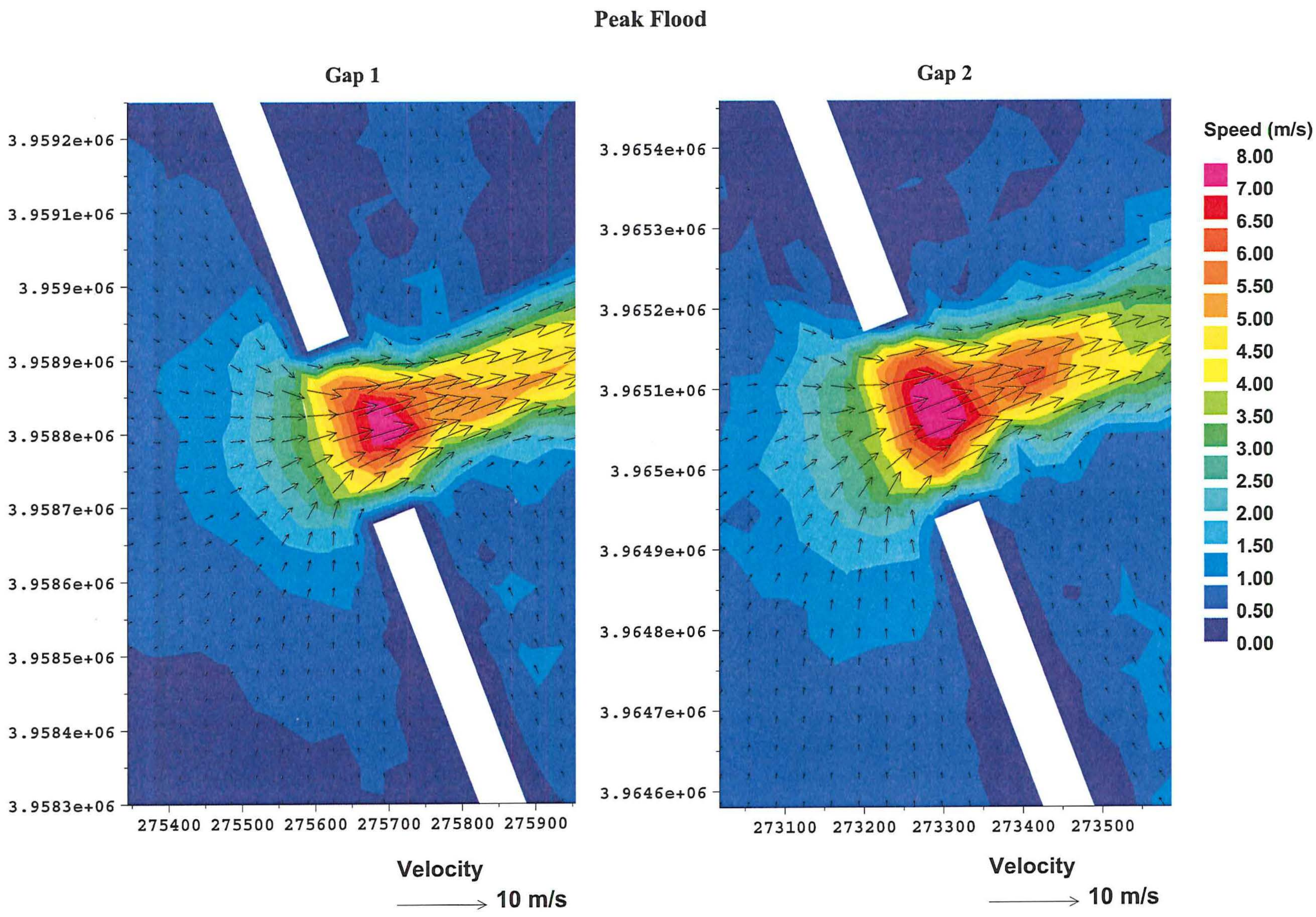

Figure 4.8 Current velocity vectors at peak flood. Spring tide. Test 8. Gap 1 : 200 m. Gap 2 : 200 m

5. Review of stone and gabion sizing and stability during final closure

5.1 INTRODUCTION

5.1.1 Objectives

The objectives of this work item are to:

- review the design processes of determining stone sizes for each of the three works (bed protection, sill and primary dam);
- evaluate the stability of stones/gabions against currents at Gap No. 1 and Gap No. 2 during final closure;
- provide advice on the preferred closure date from a hydraulic view point (Early Closure with a start in late March 2006 or Later Closure with a start in late April 2006).

5.1.2 Data sources

The calculations carried out to determine the stability of rock materials during the closure of Gaps No. 1 and No. 2 used information contained in the following documents:

- Report EX4640 "Computational and Physical Modelling on Saemangeum Closure Works", HR Wallingford, September 2002;
- Drawing (not numbered) showing plan view of bed protection within the gaps, including information on stone sizes;
- Drawing (not numbered) showing bathymetry, cross-sections through gaps and cross-section of dike;
- Drawing (not numbered) showing cross-sections of dike through Gaps I and II, including information on stone sizes;
- Tables (untitled) of final closure dates, associated tide levels and gap widths;
- KARICO "Table 5" with stable velocities for a range of stone/gabion sizes during placement;
- KARICO "Table 6" with proposed stone/gabion sizes for Gaps No. 1 and 2;
- KARICO "Table 4.6" with stable velocities for stone/gabions when settled, for the bed protection case;
- KARICO "Table 4.5" with stable velocities for stone/gabions when settled, for the sill case;
- KARICO "Table 4.8" with stable velocities for stone/gabions when settled, for the dike/dam face case;
- Information provided on 12th October 2005 on bottom protection (numbered page 12)
- Information provided on 12th October 2005 on sill protection (numbered page 21)

From the above information, Table 5.1 was produced summarising the material currently specified at the three types of location (bed protection, sill and dike) and, where available from tests carried out at RRI, the corresponding flow velocities at which the materials are stable. For simplicity, these flow velocities were termed here "stable velocities". Table 5.1 includes stable velocities for settled material and during placement.

Table 5.1 Material currently specified & stable velocities from RRI physical model tests

<table>
<tr><th>Location</th><th colspan="4">Material currently specified
&
Stable velocities</th></tr>
<tr><td rowspan="2">Bed protection</td><td>Gap No. 1</td><td colspan="3">Between Chainages18+00 and 22+00 and between 27+00 and 31+00 (i.e Phase II)
0.5 – 1.0t (90%) rock
2t (10%) gabions
~ 4.7m/s*
< 5.06m/s**
Between Chainages 22+00 and 27+00 (i.e. Phase III)
0.5-2.5t (90%) rock
2t (10%) gabions
~5.5m/s*
NA</td></tr>
<tr><td>Gap No. 2</td><td colspan="3">Between Chainages 87+50 and 95+50 (i.e. comprises Phases II and III)
0.5-2.5t (90%) rock
2t (10%) gabions
~5.5m/s*
NA</td></tr>
<tr><td rowspan="2">Sill</td><td>Gap No. 1</td><td>Between Chainages 18+00 and 20+50 and between Chainages 28+50 and 31+00
0.5-2.5t (90%) rock
3t (10%) gabions
NA
NA</td><td>Between Chainages 20+50 and 22+00 and between Chainages 27+00 and 28+50
2.5-3.0t (80%) rock
3t (20%) gabions
~6.7m/s*
<5.89m/s**</td><td>Between Chainages 22+00 and 27+00 (Phase III)
3.0-5.5t (50%) rock
3t (50%) gabions
>6.7m/s
NA</td></tr>
<tr><td>Gap No. 2</td><td>Between chainages 95+50 and 96+00 (Phase I)
0.5-4.0t (80%) rock
3t (20%) gabions
NA
NA</td><td>Between Chainages 87+50 and 89+00 and between Chainages 94+00 and 95+50
4.0–5.0t (70%) rock
3t (30%) gabions
NA
NA</td><td>Between Chainages 89+00 and 94+00
4.0–5.0t (50%) rock
3t (50%) gabions
NA
NA</td></tr>
<tr><td rowspan="3">Dike</td><td></td><td>Phase I</td><td>Phase II</td><td>Phase III</td></tr>
<tr><td>Gap No. 1</td><td>1.5-3.0t (70%) rock
3t (30%) gabions
~7.2m/s*
6.15m/s**</td><td>3.0-5.0 (80%) rock
3t (20%) gabions
NA
5.89m/s**</td><td>3.0-5.0 (70%) rock
3t (30%) gabions
NA
6.15m/s**</td></tr>
<tr><td>Gap No. 2</td><td>1.5-3.0t (60%) rock
3t (40%) gabions
~7.2m/s*
NA</td><td>3.0-6.0 (80%) rock
3t (20%) gabions
NA
6.31m/s**</td><td>3.0-6.0 (50%) rock
3t (50%) gabions
NA
7.18m/s**</td></tr>
</table>

* Stable velocities determined from RRI experimental work for settled material

** Stable velocities determined from RRI experimental work for stone during placement

NA Not available

Information on predicted flow velocities through the gaps (at mid-gap) was complemented by further values of depth-averaged velocity in mid-gap interpolated by HR Wallingford (see Chapter 3) which covered all the dates considered in the Early and Later Closure periods. For calculations of stone/gabion stability, where the flood and ebb flow velocities differed, the maximum value was taken to ensure conservative estimates.

With regard to the assessment of the stability of the bed protection, the use of mid-gap velocities was considered inappropriate since the bed is at the gaps locally protected by the sill. The bed protection mat extends beyond this location to areas where flow velocities are likely to be smaller. An approximate assessment was made of how distance from the centreline of the dike could affect flow velocities by interrogating the HR Wallingford numerical model at distances of 90m and 250m from the centreline of dike for Gap No.1 and at 60m and 250m from centreline of dike for Gap No.2. At distances of 90m and 60m the bed is essentially flat whereas at 250m surveys have shown that deep scouring has occurred. At these locations it is expected that the flow velocities are significantly reduced when compared with mid-gap velocities due to the effects of lateral expansion and increased water depth. Values of the ratio of local velocity/mid-gap velocity were obtained for conditions similar to those associated with Waiting Periods 1 and 2 for each of the two gaps. It was found that for the flat part of the bed protection, for both Gaps No.1 and No.2, the ratios were 80% and 90% for Waiting periods 1 and 2 respectively. On the basis that there is an increase in the ratios between these two periods, it was decided to adopt 80% for Waiting period 1, 85% for Phase II, 90% for Waiting period 2 and 100% for Phase III. For the case of the edge of the mat, which has experienced scour (see Section 2.4), the ratios obtained suggested using a ratio of 80% for both gaps (all phases) – see Section 5.7.2 for recommendations on protection of the edges of the mat.

The calculations of flow velocity during the closure phases also enabled some conclusions with regard to the estimation of tidal water levels associated with the various closure dates, which were required for the application of certain stone stability equations. It was found from the HR Wallingford numerical model that the tidal levels varied with the gap width and location (i.e. inside the basin and through the gap – see Section 3.1) as well as with the nature of the tide (i.e. neap or flood). The calculations of stone size are not particularly sensitive to water depth and so the following general assumptions were made for water elevations:

Gap width of 1300m to 1000m: water elevation of -3m MSL
Gap width of 1000m to 300m: water elevation of -2m MSL
Gap width of <300m: water elevation of -1m MSL.

5.2 GENERAL CONSIDERATIONS ON DETERMINATION OF STONE STABILITY

5.2.1 Calculation approach

Calculations of stone and gabion stability under current attack are normally carried out using empirical equations which can provide a relatively wide range of results for similar data sets. Because of this variation in results, it is good practice, where possible, to apply more than one predictive equation and use engineering judgement to assess the results. This variability between equations can be attributed to the following factor: most stability equations were developed from laboratory tests to reproduce particularly well a specific phenomenon within a specified range of flow conditions (e.g. turbulence or river currents) and may not take account of other factors such as water depth variation, which can be significant in tidal situations. In the present case there is an additional factor that contributes to uncertainty: the specification of materials consisting of a mixture of stone and sack gabions. Most stability equations were developed for riprap (i.e. dumped graded stone) and, although some equations can be applied to gabions, these are few and applicable to either box or gabion mattresses and not sack gabions (or mixtures consisting of sack gabions and rock).

In the present case, there are three situations to consider (or types of work/location):

- bed protection stability
- sill stability
- dike closure, or dam face stability.

At the above locations different calculation methods are required, which are described in the following sections. The calculations were carried out to determine the flow velocities that the currently specified stone/gabion mixtures can withstand (termed stable velocities). This was carried out at both gaps (Gap No. 1 and Gap No. 2) as they have different widths, sill levels, and corresponding water depths and flow velocities. The calculations were repeated for the two closure periods being considered: Early Closure (starting 30 March and finishing on 24 April 2006) and Later Closure (starting on 28 April and finishing on 23 May 2006).

5.2.2 Assumptions

The following assumptions were made:

- Stone/gabion density $\rho_s = 2650$ kg/m^3
- Seawater density $\rho_w = 1025$ kg/m^3

As mentioned in Section 5.1.2, some approximations were made regarding the water elevations and depths at the gaps above sill level, which are summarised in Table 5.2:

Table 5.2 Approximate water elevations and depths assumed for calculations of stone stability

Gap width approx. (m)	Water elevation (mMSL)	Water depth (m)	
		Gap No. 1	Gap No. 2
1300-1000	-3	7	NA
1000-300	-2	8	14
<300	-1	9	15

NA – Not applicable

5.3 STABILITY OF EXISTING BED PROTECTION DURING CLOSURE

5.3.1 Determination of representative stone size

The bed protection in place at Gaps Nos. 1 and 2 is shown in Table 5.1. There are two different stone mixes specified:

- 0.5 – 1.0t (90%) rock plus 2t (10%) gabions
- 0.5-2.5t (90%) rock plus 2t (10%) gabions

Given that the bed protection material includes combinations of graded rock and gabions, it is necessary to determine a representative stone size that can be used in the calculations of stability of the bed protection during the closure process. On this basis, Pilarczyk's equation (Equation 2 in Section 5.3.2), one of the most widely applied stability equations, was used to determine the nominal stone size that would be stable under flow velocities of about 5m/s and 5.5m/s respectively for the two mixtures. This approach gave a representative stone size of D_n=0.8m for the 0.5–1.0t (90%) rock plus 2t (10%) gabions mixture and 0.9m for the 0.5-2.5t (90%) rock plus 2t (10%) gabions mixture.

5.3.2 *Stability formulae used*

Once the representative stone size was determined, four different stability equations due to Izbash & Khaldre (1970), Pilarczyk (1990), Escarameia & May (1992) and Maynord (1993) were used to calculate the stable velocities during closure. The form of the last three equations used may be found in Escarameia (1998). For these calculations it was assumed that the bed protection in place was approximately horizontal and therefore a correction for the destabilising effect of placing stone on a slope was not considered (note that Escarameia & May's equation intrinsically takes this effect into account for slopes as steep as 1V:2H). The stability of the bed material on a slope was estimated during the assessment of stability of the edges of the mat, as described in Section 2 and presented in the recommendations given in Section 5.7.

The stability equations used are presented below:

Izbash & Khaldre (1970)

$$D_{s50} = C\,(U_b^2)/\,[g(s-1)K_s] \qquad (1)$$

or

$$U_d = 1.25\left(\frac{1.25 g \Delta D_n K_s}{C}\right)^{0.5}$$

where

U_d	depth-averaged velocity
D_{s50}	diameter of equivalent sphere $D_{s50}=1.13D_{50}$ $D_n=0.9D_s/1.13=0.8D_s$
C	numerical coefficient: C=0.35 low turbulence; C=0.68 partially developed turbulent boundary layer. In the present case a value of 0.68 was used
s	relative density of stone (Δ= s-1)
U_b	velocity near the bed; U_d is assumed to be approximately equal to $1.25U_b$
K_s	Slope factor
g	acceleration due to gravity.

Pilarczyk (1990)

$$D_n = (\Phi/\Delta)\, K_T K_h K_s^{-1}\, (0.035/\Psi_{cr})\, (U_d^2/2g) \qquad (2)$$

Table 5.3 Calculated stable velocities for bed protection (flat area) - Early Closure

Phase	Date	Gap width (m)	Mean flow velocity at bed protection mat (m/s)	Peak velocity at bed protection mat (m/s)	Averaged stable velocity for bed protection (m/s)
GAP No. 1					
Waiting Period 1	30/03/2006	1300	4.56	4.86	5.0
	31/03/2006	1300	4.72	5.03	5.0
	01/04/2006	1300	4.69	5.00	5.0
	02/04/2006	1300	4.42	4.71	5.0
Final closure Phase II	03/04/2006	1230	4.27	4.57	5.0
	04/04/2006	1160	3.71	3.98	5.0
	05/04/2006	1090	3.08	3.32	5.0
	06/04/2006	1020	2.67	2.89	5.0
	07/04/2006	950	2.60	2.82	5.0
	08/04/2006	880	3.00	3.27	5.0
	09/04/2006	810	3.48	3.80	5.0
	10/04/2006	740	3.87	4.24	5.0
	11/04/2006	670	4.16	4.57	5.0
	12/04/2006	600	4.38	4.83	5.0
	13/04/2006	530	4.56	5.05	5.0
Waiting Period 2	14/04/2006	530	5.08	5.62	5.0
	15/04/2006	530	5.16	5.71	5.0
	16/04/2006	530	5.26	5.83	5.0
Final closure Phase III	17/04/2006	463.75	6.10	6.77	5.6
	18/04/2006	397.5	6.19	6.89	5.6
	19/04/2006	331.25	6.10	6.81	5.6
	20/04/2006	265	5.86	6.57	5.6
	21/04/2006	198.75	5.57	6.27	5.6
	22/04/2006	132.5	5.55	6.27	5.6
	23/04/2006	66.25	5.97	6.76	5.6
	24/04/2006	0	6.55	7.44	5.6
GAP No. 2					
Waiting Period 1	30/03/2006	660	4.50	4.95	5.7
	31/03/2006	660	4.58	5.04	5.7
	01/04/2006	660	4.58	5.04	5.7
	02/04/2006	660	4.29	4.72	5.7

Final closure Phase II	03/04/2006	628	4.18	4.61	5.7
	04/04/2006	596	3.68	4.06	5.7
	05/04/2006	565	3.32	3.66	5.7
	06/04/2006	533	3.18	3.52	5.7
	07/04/2006	501	3.12	3.46	5.7
	08/04/2006	469	3.46	3.84	5.7
	09/04/2006	437	3.89	4.33	5.7
	10/04/2006	405	4.28	4.77	5.7
	11/04/2006	374	4.59	5.12	5.7
	12/04/2006	342	4.84	5.41	5.7
	13/04/2006	310	5.07	5.68	5.7
Waiting Period 2	14/04/2006	310	5.59	6.25	5.7
	15/04/2006	310	5.67	6.34	5.7
	16/04/2006	310	5.70	6.37	5.7
Final closure Phase III	17/04/2006	271	6.40	7.17	5.7
	18/04/2006	233	6.40	7.19	5.7
	19/04/2006	194	6.23	7.01	5.7
	20/04/2006	155	5.92	6.67	5.7
	21/04/2006	116	5.64	6.36	5.7
	22/04/2006	78	5.70	6.44	5.7
	23/04/2006	39	6.05	6.85	5.7
	24/04/2006	0	6.56	7.45	5.7

Table 5.4 Calculated stable velocities for bed protection (flat area) - Later Closure

Phase	Date	Gap width (m)	Mean flow velocity at bed protection mat (m/s)	Peak velocity at bed protection mat (m/s)	Averaged stable velocity for bed protection (m/s)
GAP No. 1					
Waiting Period 1	28/04/2006	1300	4.54	4.84	5.0
	29/04/2006	1300	4.66	4.97	5.0
	30/04/2006	1300	4.46	4.76	5.0
	01/05/2006	1300	4.13	4.40	5.0
Final closure Phase II	02/05/2006	1230	4.04	4.32	5.0

	03/05/2006	1160	3.60	3.86	5.0
	04/05/2006	1090	3.13	3.37	5.0
	05/05/2006	1020	2.78	3.01	5.0
	06/05/2006	950	2.83	3.07	5.0
	07/05/2006	880	3.07	3.34	5.0
	08/05/2006	810	3.40	3.71	5.0
	09/05/2006	740	3.72	4.08	5.0
	10/05/2006	670	4.00	4.39	5.0
	11/05/2006	600	4.28	4.73	5.0
	12/05/2006	530	4.56	5.04	5.0
Waiting Period 2	13/05/2006	530	5.02	5.56	5.0
	14/05/2006	530	5.01	5.55	5.0
	15/05/2006	530	5.04	5.58	5.0
Final closure Phase III	16/05/2006	463.75	6.12	6.80	5.6
	17/05/2006	397.5	6.26	6.97	5.6
	18/05/2006	331.25	6.24	6.98	5.6
	19/05/2006	265	6.10	6.84	5.6
	20/05/2006	198.75	5.93	6.67	5.6
	21/05/2006	132.5	6.11	6.89	5.6
	22/05/2006	66.25	6.41	7.26	5.6
	23/05/2006	0	6.80	7.72	5.6
GAP No. 2					
Waiting Period 1	28/04/2006	660	4.35	4.79	5.7
	29/04/2006	660	4.32	4.75	5.7
	30/04/2006	660	4.28	4.71	5.7
	01/05/2006	660	3.99	4.39	5.7
Final closure Phase II	02/05/2006	628	3.94	4.35	5.7
	03/05/2006	596	3.59	3.96	5.7
	04/05/2006	565	3.37	3.73	5.7
	05/05/2006	533	3.26	3.61	5.7
	06/05/2006	501	3.31	3.67	5.7
	07/05/2006	469	3.52	3.91	5.7
	08/05/2006	437	3.83	4.25	5.7
	09/05/2006	405	4.15	4.62	5.7
	10/05/2006	374	4.44	4.95	5.7
	11/05/2006	342	4.68	5.23	5.7
	12/05/2006	310	4.92	5.51	5.7
Waiting Period 2	13/05/2006	310	5.34	5.97	5.7
	14/05/2006	310	5.54	6.20	5.7
	15/05/2006	310	5.66	6.33	5.7

Final closure Phase III	16/05/2006	271	6.36	7.13	5.7
	17/05/2006	233	6.36	7.15	5.7
	18/05/2006	194	6.21	6.99	5.7
	19/05/2006	155	5.94	6.69	5.7
	20/05/2006	116	5.72	6.46	5.7
	21/05/2006	78	5.74	6.50	5.7
	22/05/2006	39	5.89	6.67	5.7
	23/05/2006	0	6.12	6.95	5.7

5.4 STABILITY OF SILL DURING CLOSURE

5.4.1 Determination of representative stone size

The materials specified for the sills at Gaps Nos. 1 and 2 are given in Table 5.1.

At Gap No. 1 the material is specified as consisting of three different mixtures:

- 0.5-2.5t (90%) rock plus 3t (10%) gabions
- 2.5–3.0t (80%) rock plus 3t (20%) gabions
- 3.0–5.5t (50%) rock plus 3t (50%) gabions.

For the application of the stability formulae, it is necessary to estimate representative stone sizes. From Table 5.1 there is indication that the above mixtures can withstand flow velocities of the order of 6.5m/s. Representative stone sizes were estimated such that there was consistency between these estimates and sizes determined for other parts of the dike where more information was available. The representative stone sizes used were: 0.92m, 0.95m and 1.3m for the three mixtures in the order described above.

With regard to Gap No. 2, the material specified for the sill consists of the following mixtures:

- 0.5-4.0t (80%) rock plus 3t (20%) gabions
- 4.0–5.0t (70%) rock plus 3t (30%) gabions
- 4.0–5.0t (50%) rock plus 3t (50%) gabions

As can be seen from Table 5.1, there is no information available on the velocities that these mixtures in Gap No. 2 are likely to withstand. Physical model tests conducted at HR Wallingford and described in EX 4640 showed that a mixture of 80% 5t rock plus 20% 3t gabions could withstand a flow velocity of approximately 8m/s. This indicates that the value of flow velocity for which mixtures consisting of 4-5t rock and 3t gabions are likely to be stable will be smaller than 8m/s. For the application of the stability equations, the estimation of representative stone sizes was made in order to achieve consistency with previous estimates and led to the use of: 0.9m, 1.1m and 1.2m for the three mixtures in the order described above.

5.4.2 Stability formulae used

For the calculations of stability at the sill two formulae were used: Izbash's equation for partially developed turbulent boundary layer and the discharge criterion recommended in CUR, 1995 (the other criterion recommended in CUR, the overtopping-height criterion, was not used as there is some evidence from laboratory work that the discharge criterion may be more accurate).

Izbash's equation was presented in Section 5.3.2 - see Equation (1).

The **discharge criterion** is given as follows:

$$\frac{q}{g^{0.5}(\Delta D_n)^{1.5}} = 1.99952 + 1.06866\frac{h_d}{\Delta D_n} + 0.13884(\frac{h_d}{\Delta D_n})^2 - 0.00234(\frac{h_d}{\Delta D_n})^3 - 0.00002(\frac{h_d}{\Delta D_n})^4 \quad (5)$$

or, taking q as approximately equal to $U_d h_d$, gives

$$U_d = g^{0.5}(\Delta D_n)^{1.5}\{1.99952 + 1.06866\frac{h_d}{\Delta D_n} + 0.13884(\frac{h_d}{\Delta D_n})^2 - 0.00234(\frac{h_d}{\Delta D_n})^3 - 0.00002(\frac{h_d}{\Delta D_n})^4\}$$

where

q	specific discharge per metre of gap
U_d	depth-averaged velocity
g	acceleration due to gravity
Δ	density of rock relative to water
D_n	nominal diameter of rock
h_d	downstream water level relative to sill height.

5.4.3 *Results*

The results obtained from application of the Izbash equation and the discharge-criterion described in Section 5.4.2 are presented in Tables 5.5 and 5.6 for the Early and Later Closures respectively. These tables also include predicted flow velocities through the gaps corresponding to mid gap conditions and also peak velocities which represent an increase of up to 14% over the mid-gap velocities (see Chapter 3). For comparison of the average stable stone velocities with predicted flow velocities through the gaps it was decided to consider the peak velocities rather than the mid-gap velocity values. The reason for this was that the sill will be exposed to the increased velocities at the edges of the dam and the additional turbulence will have a destabilising effect on the sill material. With this approach, it can be seen from Tables 5.5 and 5.6 that, although the predictions using Izbash's equation and the Discharge Criterion are different, they both indicate that the materials of the sill are likely to be stable at both gaps and for both the Early and the Later Closure dates. The differences in the results from the two equations used are attributed to Izbash's equation not taking into account the effect of water depth over the sill, as can be appreciated by the differences in the results obtained with the two formulae for Gaps No. 1 and No. 2. Discussion of these results is presented in Section 5.6.1.

Table 5.5 Calculated stable velocities for sill stability - Early Closure

Phase	Date	Gap width (m)	Mid-gap flow velocity (m/s)	Peak velocity (m/s)	Stable velocity for sill Izbash (m/s)	Stable velocity for sill Discharge Criterion (m/s)
GAP No. 1						
Waiting Period 1	30/03/2006	1300	5.7	6.08	6.4	7.9
	31/03/2006	1300	5.9	6.29	6.4	7.9
	01/04/2006	1300	5.86	6.25	6.4	7.9
	02/04/2006	1300	5.52	5.89	6.4	7.9
Final closure Phase II	03/04/2006	1230	5.02	5.37	6.4	7.9
	04/04/2006	1160	4.36	4.68	6.4	7.9
	05/04/2006	1090	3.63	3.91	6.4	7.9
	06/04/2006	1020	3.14	3.40	6.4	7.9
	07/04/2006	950	3.06	3.32	6.4	8.0
	08/04/2006	880	3.53	3.84	6.4	8.0
	09/04/2006	810	4.09	4.47	6.4	8.0
	10/04/2006	740	4.55	4.99	6.5	8.1
	11/04/2006	670	4.89	5.38	6.5	8.1
	12/04/2006	600	5.15	5.68	6.5	8.1
	13/04/2006	530	5.37	5.95	6.5	8.1
Waiting Period 2	14/04/2006	530	5.64	6.24	7.6	9.4
	15/04/2006	530	5.73	6.34	7.6	9.4
	16/04/2006	530	5.85	6.48	7.6	9.4
Final closure Phase III	17/04/2006	463.75	6.10	6.77	7.6	9.4
	18/04/2006	397.5	6.19	6.89	7.6	9.4
	19/04/2006	331.25	6.10	6.81	7.6	9.4
	20/04/2006	265	5.86	6.57	7.6	9.4
	21/04/2006	198.75	5.57	6.27	7.6	9.4
	22/04/2006	132.5	5.55	6.27	7.6	9.4
	23/04/2006	66.25	5.97	6.76	7.6	9.4
	24/04/2006	0	6.55	7.44	7.6	9.4
GAP No. 2						
Waiting Period 1	30/03/2006	660	5.62	6.18	7.0	9.4
	31/03/2006	660	5.72	6.29	7.0	9.4
	01/04/2006	660	5.72	6.29	7.0	9.4
	02/04/2006	660	5.36	5.90	7.0	9.4
Final closure Phase II	03/04/2006	628	4.92	5.42	7.0	9.4
	04/04/2006	596	4.33	4.78	7.0	9.4
	05/04/2006	565	3.9	4.31	7.0	9.4

	06/04/2006	533	3.74	4.14	7.0	9.4
	07/04/2006	501	3.67	4.07	7.3	9.6
	08/04/2006	469	4.07	4.52	7.3	9.6
	09/04/2006	437	4.58	5.09	7.3	9.6
	10/04/2006	405	5.04	5.61	7.3	9.6
	11/04/2006	374	5.4	6.02	7.3	9.6
	12/04/2006	342	5.7	6.37	7.3	9.6
	13/04/2006	310	5.97	6.68	7.3	9.6
Waiting Period 2	14/04/2006	310	6.21	6.95	7.3	9.6
	15/04/2006	310	6.3	7.05	7.3	9.6
	16/04/2006	310	6.33	7.08	7.3	9.6
Final closure Phase III	17/04/2006	271	6.40	7.17	7.3	9.8
	18/04/2006	233	6.40	7.19	7.3	9.8
	19/04/2006	194	6.23	7.01	7.3	9.8
	20/04/2006	155	5.92	6.67	7.3	9.8
	21/04/2006	116	5.64	6.36	7.3	9.8
	22/04/2006	78	5.70	6.44	7.3	9.8
	23/04/2006	39	6.05	6.85	7.3	9.8
	24/04/2006	0	6.56	7.45	7.3	9.8

Table 5.6 Calculated stable velocities for sill stability - Later Closure

Phase	Date	Gap width (m)	Mid-gap flow velocity (m/s)	Peak velocity (m/s)	Stable velocity for sill Izbash (m/s)	Stable velocity for sill Discharge Criterion (m/s)
GAP No. 1						
Waiting Period 1	28/04/2006	1300	5.67	6.05	6.4	7.9
	29/04/2006	1300	5.82	6.21	6.4	7.9
	30/04/2006	1300	5.58	5.95	6.4	7.9
	01/05/2006	1300	5.16	5.50	6.4	7.9
Final closure Phase II	02/05/2006	1230	4.75	5.08	6.4	7.9
	03/05/2006	1160	4.23	4.54	6.4	7.9
	04/05/2006	1090	3.68	3.97	6.4	7.9
	05/05/2006	1020	3.27	3.54	6.4	7.9
	06/05/2006	950	3.33	3.61	6.4	8.0
	07/05/2006	880	3.61	3.93	6.4	8.0
	08/05/2006	810	4	4.37	6.4	8.0
	09/05/2006	740	4.38	4.80	6.5	8.1
	10/05/2006	670	4.7	5.17	6.5	8.1
	11/05/2006	600	5.04	5.56	6.5	8.1
	12/05/2006	530	5.36	5.93	6.5	8.1

Waiting Period 2	13/05/2006	530	5.58	6.18	7.6	9.4
	14/05/2006	530	5.57	6.17	7.6	9.4
	15/05/2006	530	5.6	6.20	7.6	9.4
Final closure Phase III	16/05/2006	463.75	6.12	6.80	7.6	9.4
	17/05/2006	397.5	6.26	6.97	7.6	9.4
	18/05/2006	331.25	6.24	6.98	7.6	9.4
	19/05/2006	265	6.10	6.84	7.6	9.4
	20/05/2006	198.75	5.93	6.67	7.6	9.4
	21/05/2006	132.5	6.11	6.89	7.6	9.4
	22/05/2006	66.25	6.41	7.26	7.6	9.4
	23/05/2006	0	6.80	7.72	7.6	9.4
GAP No.						
Waiting Period 1	28/04/2006	660	5.44	5.99	7.0	9.4
	29/04/2006	660	5.4	5.94	7.0	9.4
	30/04/2006	660	5.35	5.89	7.0	9.4
	01/05/2006	660	4.99	5.49	7.0	9.4
Final closure Phase II	02/05/2006	628	4.64	5.11	7.0	9.4
	03/05/2006	596	4.22	4.66	7.0	9.4
	04/05/2006	565	3.97	4.39	7.0	9.4
	05/05/2006	533	3.84	4.25	7.0	9.4
	06/05/2006	501	3.89	4.31	7.3	9.6
	07/05/2006	469	4.14	4.60	7.3	9.6
	08/05/2006	437	4.5	5.00	7.3	9.6
	09/05/2006	405	4.88	5.44	7.3	9.6
	10/05/2006	374	5.22	5.82	7.3	9.6
	11/05/2006	342	5.51	6.16	7.3	9.6
	12/05/2006	310	5.79	6.48	7.3	9.6
Waiting Period 2	13/05/2006	310	5.93	6.64	7.3	9.6
	14/05/2006	310	6.16	6.89	7.3	9.6
	15/05/2006	310	6.29	7.04	7.3	9.6
Final closure Phase III	16/05/2006	271	6.36	7.13	7.3	9.8
	17/05/2006	233	6.36	7.15	7.3	9.8
	18/05/2006	194	6.21	6.99	7.3	9.8
	19/05/2006	155	5.94	6.69	7.3	9.8
	20/05/2006	116	5.72	6.46	7.3	9.8
	21/05/2006	78	5.74	6.50	7.3	9.8
	22/05/2006	39	5.89	6.67	7.3	9.8
	23/05/2006	0	6.12	6.95	7.3	9.8

5.5 STABILITY OF DIKE DURING CLOSURE

5.5.1 Determination of representative stone size

As can be seen in Table 5.1, different rock/gabion mixtures are currently proposed for the three phases of the vertical closure of the dike and for each of the two gaps. Representative stone sizes were calculated using Pilarczyk's equation (Equation 2). In order to apply this equation it is necessary to include a value of water depth, which was determined so that velocity values similar to those obtained experimentally by RRI for the various stone/gabions envisaged for the dike closure would be achieved. The nominal water depth used in the calculation of the representative stone sizes was 5m, and the values of stone size are shown in Table 5.7.

Table 5.7 Material currently specified and representative stone sizes for calculations of stability during vertical closure of dike

Location	Material currently specified & representative stone size (m)			
Dike		Phase I	Phase II	Phase III
	Gap I	1.5-3.0t (70%) rock 3t (30%) gabions 0.83-1.0	3.0-5.0 (80%) rock 3t (20%) gabions 1.0-1.24	3.0-5.0 (70%) rock 3t (30%) gabions 1.0-1.24
	Gap II	1.5-3.0t (60%) rock 3t (40%) gabions 0.83-1.0	3.0-6.0 (80%) rock 3t (20%) gabions 1.0-1.3	3.0-6.0 (50%) rock 3t (50%) gabions 1.0-1.3

5.5.2 Stability formulae used

For the calculations of stability of the dike during closure, the approach due to Naylor (1976) and further developed by Akkerman (1986) was used, which provides the velocity at which the materials at the face of a dike subjected to currents are stable:

$$U = \log\left(3\frac{h_d}{D_n}\right)(F\,\Delta\,g\,D_n)^{0.5} \tag{6}$$

where

U	mean flow velocity in the gap
h_d	control water depth, taken here as the average water depth through the gap
D_n	nominal stone size
g	acceleration due to gravity
Δ	density of rock relative to water
F	factor to account for extreme roughness, defined as $F = 0.8 \exp[1.174/(h_d/1.5D_n)]$

F takes the value of 1 for $h_d/(1.5D_n)>5.2$
and 2.7 for $h_d/(1.5D_n)<1.0$

5.5.3 Results

The analysis of stone stability during the closure of the dike at the two gaps needs to take into account the three planned phases: Phase I, Phase II and Phase III.

Phase I refers to gap widths larger than 1300m for Gap No. 1 and 660m for Gap No. 2 for which no data on flow velocities or water levels was available. However, it is possible to infer some values from those predicted for Phase II as an approximate check. At the start of Phase II peak flow velocities can reach 6.3m/s. For Gap No. 1 the reduction in cross-sectional area from Phase I to Phase II is 23% and for a fixed flow rate this equates to a decrease in mean flow velocity for Phase I of 23% to 4.85m/s. For Gap No. 2 the reduction in cross-sectional area is about 29%, which corresponds to an estimated flow velocity through the gap in Phase I of 4.5m/s. A comparison of these values with the RRI experimentally determined stable velocities for the two gaps (see Table 5.1) indicates that during Phase I the currently specified materials are adequate.

With regard to Phases II and III, the results of the calculations are presented in Tables 5.8 and 5.9 for the Early and Later Closures, respectively. Given the critical conditions that the closing of the dike entails, the peak velocities through the gaps were used for comparison purposes. The dates associated with exceedance of the stable velocities are indicated in the Tables in red. Discussion of the results is presented in Section 5.6.1.

Table 5.8 Calculated stable velocities for dike stability - Early Closure

Phase	Date	Gap width (m)	Mid-gap flow velocity (m/s)	Peak velocity (m/s)	Stable velocity for dike face (m/s)
GAP No. 1					
Waiting Period 1	30/03/2006	1300	5.7	6.08	5.3
	31/03/2006	1300	5.9	6.29	5.3
	01/04/2006	1300	5.86	6.25	5.3
	02/04/2006	1300	5.52	5.89	5.3
Final closure Phase II	03/04/2006	1230	5.02	5.37	5.3
	04/04/2006	1160	4.36	4.68	5.3
	05/04/2006	1090	3.63	3.91	5.3
	06/04/2006	1020	3.14	3.40	5.3
	07/04/2006	950	3.06	3.32	5.4
	08/04/2006	880	3.53	3.84	5.4
	09/04/2006	810	4.09	4.47	5.4
	10/04/2006	740	4.55	4.99	5.4
	11/04/2006	670	4.89	5.38	5.4
	12/04/2006	600	5.15	5.68	5.4
	13/04/2006	530	5.37	5.95	5.5
Waiting Period 2	14/04/2006	530	5.64	6.24	5.5
	15/04/2006	530	5.73	6.34	5.5
	16/04/2006	530	5.85	6.48	5.5

Final closure Phase III	17/04/2006	463.75	6.10	6.77	5.5
	18/04/2006	397.5	6.19	6.89	5.5
	19/04/2006	331.25	6.10	6.81	5.5
	20/04/2006	265	5.86	6.57	5.7
	21/04/2006	198.75	5.57	6.27	5.7
	22/04/2006	132.5	5.55	6.27	5.7
	23/04/2006	66.25	5.97	6.76	5.7
	24/04/2006	0	6.55	7.44	5.7
GAP No. 2					
Waiting Period 1	30/03/2006	660	5.62	6.18	6.4
	31/03/2006	660	5.72	6.29	6.4
	01/04/2006	660	5.72	6.29	6.4
	02/04/2006	660	5.36	5.90	6.4
Final closure Phase II	03/04/2006	628	4.92	5.42	6.4
	04/04/2006	596	4.33	4.78	6.4
	05/04/2006	565	3.9	4.31	6.4
	06/04/2006	533	3.74	4.14	6.4
	07/04/2006	501	3.67	4.07	6.4
	08/04/2006	469	4.07	4.52	6.4
	09/04/2006	437	4.58	5.09	6.4
	10/04/2006	405	5.04	5.61	6.4
	11/04/2006	374	5.4	6.02	6.4
	12/04/2006	342	5.7	6.37	6.4
	13/04/2006	310	5.97	6.68	6.8
Waiting Period 2	14/04/2006	310	6.21	6.95	6.8
	15/04/2006	310	6.3	7.05	6.8
	16/04/2006	310	6.33	7.08	6.8
Final closure Phase III	17/04/2006	271	6.40	7.17	6.9
	18/04/2006	233	6.40	7.19	6.9
	19/04/2006	194	6.23	7.01	6.9
	20/04/2006	155	5.92	6.67	6.9
	21/04/2006	116	5.64	6.36	6.9
	22/04/2006	78	5.70	6.44	6.9
	23/04/2006	39	6.05	6.85	6.9
	24/04/2006	0	6.56	7.45	6.9

Table 5.9 Calculated stable velocities for dike stability - Later Closure

Phase	Date	Gap width (m)	Mid-gap flow velocity (m/s)	Peak velocity (m/s)	Stable velocity for dike face (m/s)
GAP No. 1					
Waiting Period 1	28/04/2006	1300	5.67	6.05	5.3
	29/04/2006	1300	5.82	6.21	5.3
	30/04/2006	1300	5.58	5.95	5.3
	01/05/2006	1300	5.16	5.50	5.3
Final closure Phase II	02/05/2006	1230	4.75	5.08	5.3
	03/05/2006	1160	4.23	4.54	5.3
	04/05/2006	1090	3.68	3.97	5.3
	05/05/2006	1020	3.27	3.54	5.3
	06/05/2006	950	3.33	3.61	5.4
	07/05/2006	880	3.61	3.93	5.4
	08/05/2006	810	4	4.37	5.4
	09/05/2006	740	4.38	4.80	5.4
	10/05/2006	670	4.7	5.17	5.4
	11/05/2006	600	5.04	5.56	5.4
	12/05/2006	530	5.36	5.93	5.5
Waiting Period 2	13/05/2006	530	5.58	6.18	5.5
	14/05/2006	530	5.57	6.17	5.5
	15/05/2006	530	5.6	6.20	5.5
Final closure Phase III	16/05/2006	463.75	6.12	6.80	5.5
	17/05/2006	397.5	6.26	6.97	5.5
	18/05/2006	331.25	6.24	6.98	5.5
	19/05/2006	265	6.10	6.84	5.7
	20/05/2006	198.75	5.93	6.67	5.7
	21/05/2006	132.5	6.11	6.89	5.7
	22/05/2006	66.25	6.41	7.26	5.7
	23/05/2006	0	6.80	7.72	5.7
GAP No. 2					
Waiting Period 1	28/04/2006	660	5.44	5.99	6.4
	29/04/2006	660	5.4	5.94	6.4
	30/04/2006	660	5.35	5.89	6.4
	01/05/2006	660	4.99	5.49	6.4
Final closure Phase II	02/05/2006	628	4.64	5.11	6.4
	03/05/2006	596	4.22	4.66	6.4
	04/05/2006	565	3.97	4.39	6.4
	05/05/2006	533	3.84	4.25	6.4
	06/05/2006	501	3.89	4.31	6.4
	07/05/2006	469	4.14	4.60	6.4
	08/05/2006	437	4.5	5.00	6.4
	09/05/2006	405	4.88	5.44	6.4

	10/05/2006	374	5.22	5.82	6.4
	11/05/2006	342	5.51	6.16	6.4
	12/05/2006	310	5.79	6.48	6.8
Waiting Period 2	13/05/2006	310	5.93	6.64	6.8
	14/05/2006	310	6.16	6.89	6.8
	15/05/2006	310	6.29	7.04	6.8
Final closure Phase III	16/05/2006	271	6.36	7.13	6.9
	17/05/2006	233	6.36	7.15	6.9
	18/05/2006	194	6.21	6.99	6.9
	19/05/2006	155	5.94	6.69	6.9
	20/05/2006	116	5.72	6.46	6.9
	21/05/2006	78	5.74	6.50	6.9
	22/05/2006	39	5.89	6.67	6.9
	23/05/2006	0	6.12	6.95	6.9

5.6 CONCLUSIONS

Conclusions regarding the final design of the stone during closure can be drawn from two different approaches:

1) by comparing the predicted flow velocities through the gaps with the calculated stable velocities using empirical equations and described in the previous sections.

2) by comparing the predicted flow velocities through the gaps with the velocities at which various combinations of stone and gabions were found to be stable through the experimental work carried out at RRI (and summarised in Table 5.1);

5.6.1 Conclusions based on calculations using empirical formulae

Stability of the bed protection

The following conclusions can be drawn from Tables 5.3 and 5.4.

<u>Early Closure and Later Closure:</u>
Gap No. 1
- The stone/gabion mixture currently specified (see Table 5.1) is at the limit of stability during Waiting period 1 and end of Final Closure Phase II, and is unlikely to provide suitable protection during Waiting period 2 and Final Closure Phase III.

Gap No. 2
- The stone/gabion mixture currently specified (see Table 5.1) is unlikely to be stable during Waiting period 2 and Final Closure Phase III.

From Tables 5.3 and 5.4 it can be seen that, particularly from the end of Phase II, the stone specified for protection of the bed is likely to become unstable. The discussion in Section 2 highlights the potential for further instability of the bed protection mat at the edges and strongly points out to the need for upgrading of this material.

Stability of the sill

The following conclusions can be drawn from Tables 5.5 and 5.6.

Early Closure and Later Closure:

Gap No. 1

- The two formulae used gave quite different results: Izbash suggested stable velocities for the currently specified materials of between of 6.4 and 7.6m/s whereas the discharge criterion gave values between 7.9 and 9.4m/s. Neither of these values are predicted to be exceeded by peak velocities.

Gap No. 2

- The two formulae used gave quite different results: Izbash suggested stable velocities for the currently specified materials as 7.0 to 7.3m/s whereas the discharge criterion gave values of 9.4 to 9.8m/s. Neither of these calculated values is predicted to be exceeded by mid gap or peak velocities.

Stability of dike

The following conclusions can be drawn from Tables 5.8 and 5.9.

Early Closure

Gap No. 1

- The stone/gabion mixture currently specified for Phase I (see Table 5.1) appears to be adequate
- The stone/gabion mixtures currently specified are considered inadequate for the Waiting periods, Final Closure Phase III and part of Final Closure Phase II.

Gap No. 2

- The stone/gabion mixtures currently specified for Phase I and Phase II and Waiting Period 1 (see Table 5.1) appear to be adequate
- The stone/gabion mixture currently specified is considered inadequate for Waiting Period 2 and at the limit during Final Closure Phase III.

Later Closure

Gap No. 1

- The stone/gabion mixture currently specified for Phase I (see Table 5.1) appears to be adequate
- The stone/gabion mixtures currently specified are considered inadequate for the Waiting periods, Final Closure Phase III and the latter part of Final Closure Phase II.

Gap No. 2

- The stone/gabion mixtures currently specified for Phase I and Phase II and Waiting Period 1 (see Table 5.1) appear to be adequate
- The stone/gabion mixture currently specified is considered to be at the limit during Waiting Period 2 and during Final Closure Phase III.

The Later Closure option is marginally preferable to the Early Closure option.

The above conclusions are summarised in Tables 5.10 and 5.11, which provide a general way of appreciating the adequacy or otherwise of the proposed materials. In these tables red indicates that calculations using empirical equations suggest that the proposed materials are inadequate, whereas green suggests no appreciable concern and orange indicates limit of stability or instability during part of the period.

	20/04/2006	265	5.86	6.57	~5.5 NA
	21/04/2006	198.75	5.57	6.27	~5.5 NA
	22/04/2006	132.5	5.55	6.27	~5.5 NA
	23/04/2006	66.25	5.97	6.76	~5.5 NA
	24/04/2006	0	6.55	7.44	~5.5 NA
GAP No.2					
Waiting Period 1	30/03/2006	660	4.50	4.95	~5.5 NA
	31/03/2006	660	4.58	5.04	~5.5 NA
	01/04/2006	660	4.58	5.04	~5.5 NA
	02/04/2006	660	4.29	4.72	~5.5 NA
Final closure Phase II	03/04/2006	628	4.18	4.61	~5.5 NA
	04/04/2006	596	3.68	4.06	~5.5 NA
	05/04/2006	565	3.32	3.66	~5.5 NA
	06/04/2006	533	3.18	3.52	~5.5 NA
	07/04/2006	501	3.12	3.46	~5.5 NA
	08/04/2006	469	3.46	3.84	~5.5 NA
	09/04/2006	437	3.89	4.33	~5.5 NA
	10/04/2006	405	4.28	4.77	~5.5 NA
	11/04/2006	374	4.59	5.12	~5.5 NA
	12/04/2006	342	4.84	5.41	~5.5 NA
	13/04/2006	310	5.07	5.68	~5.5 NA
Waiting Period 2	14/04/2006	310	5.59	6.25	~5.5 NA
	15/04/2006	310	5.67	6.34	~5.5 NA
	16/04/2006	310	5.70	6.37	~5.5 NA
Final closure Phase III	17/04/2006	271	6.40	7.17	~5.5 NA
	18/04/2006	233	6.40	7.19	~5.5 NA
	19/04/2006	194	6.23	7.01	~5.5 NA

	20/04/2006	155	5.92	6.67	~5.5 NA
	21/04/2006	116	5.64	6.36	~5.5 NA
	22/04/2006	78	5.70	6.44	~5.5 NA
	23/04/2006	39	6.05	6.85	~5.5 NA
	24/04/2006	0	6.56	7.45	~5.5 NA

NA - Not Available

Table 5.13 Comparison of predicted flow velocities with stable velocities determined experimentally by RRI for Bed Protection – Later Closure

Phase	**Date**	**Gap width (m)**	**Mean flow velocity (m/s)**	**Peak velocity (m/s)**	**Stable velocities from RRI tests (m/s)**
					Bed protection (when settled) (being placed)
GAP No. 1					
Waiting Period 1	28/04/2006	1300	4.54	4.84	~4.7 < 5.06
	29/04/2006	1300	4.66	4.97	~4.7 < 5.06
	30/04/2006	1300	4.46	4.76	~4.7 < 5.06
	01/05/2006	1300	4.13	4.40	~4.7 < 5.06
Final closure Phase II	02/05/2006	1230	4.04	4.32	~4.7 < 5.06
	03/05/2006	1160	3.60	3.86	~4.7 < 5.06
	04/05/2006	1090	3.13	3.37	~4.7 < 5.06
	05/05/2006	1020	2.78	3.01	~4.7 < 5.06
	06/05/2006	950	2.83	3.07	~4.7 < 5.06
	07/05/2006	880	3.07	3.34	~4.7 < 5.06
	08/05/2006	810	3.40	3.71	~4.7 < 5.06
	09/05/2006	740	3.72	4.08	~4.7 < 5.06
	10/05/2006	670	4.00	4.39	~4.7 < 5.06
	11/05/2006	600	4.28	4.73	~4.7 < 5.06
	12/05/2006	530	4.56	5.04	~4.7 < 5.06

Waiting Period 2	13/05/2006	530	5.02	5.56	~4.7 < 5.06
	14/05/2006	530	5.01	5.55	~4.7 < 5.06
	15/05/2006	530	5.04	5.58	~4.7 < 5.06
Final closure Phase III	16/05/2006	463.75	6.12	6.80	~5.5 NA
	17/05/2006	397.5	6.26	6.97	~5.5 NA
	18/05/2006	331.25	6.24	6.98	~5.5 NA
	19/05/2006	265	6.10	6.84	~5.5 NA
	20/05/2006	198.75	5.93	6.67	~5.5 NA
	21/05/2006	132.5	6.11	6.89	~5.5 NA
	22/05/2006	66.25	6.41	7.26	~5.5 NA
	23/05/2006	0	6.80	7.72	~5.5 NA
Gap No.2					
Waiting Period 1	28/04/2006	660	4.35	4.79	~5.5 NA
	29/04/2006	660	4.32	4.75	~5.5 NA
	30/04/2006	660	4.28	4.71	~5.5 NA
	01/05/2006	660	3.99	4.39	~5.5 NA
Final closure Phase II	02/05/2006	628	3.94	4.35	~5.5 NA
	03/05/2006	596	3.59	3.96	~5.5 NA
	04/05/2006	565	3.37	3.73	~5.5 NA
	05/05/2006	533	3.26	3.61	~5.5 NA
	06/05/2006	501	3.31	3.67	~5.5 NA
	07/05/2006	469	3.52	3.91	~5.5 NA
	08/05/2006	437	3.83	4.25	~5.5 NA
	09/05/2006	405	4.15	4.62	~5.5 NA
	10/05/2006	374	4.44	4.95	~5.5 NA
	11/05/2006	342	4.68	5.23	~5.5 NA
	12/05/2006	310	4.92	5.51	~5.5 NA

Waiting Period 2	13/05/2006	310	5.34	5.97	~5.5 NA
	14/05/2006	310	5.54	6.20	~5.5 NA
	15/05/2006	310	5.66	6.33	~5.5 NA
Final closure Phase III	16/05/2006	271	6.36	7.13	~5.5 NA
	17/05/2006	233	6.36	7.15	~5.5 NA
	18/05/2006	194	6.21	6.99	~5.5 NA
	19/05/2006	155	5.94	6.69	~5.5 NA
	20/05/2006	116	5.72	6.46	~5.5 NA
	21/05/2006	78	5.74	6.50	~5.5 NA
	22/05/2006	39	5.89	6.67	~5.5 NA
	23/05/2006	0	6.12	6.95	~5.5 NA

NA - Not Available

Table 5.14 Comparison of predicted flow velocities with stable velocities determined experimentally by RRI for Sill - Early Closure

Phase	Date	Gap width (m)	Mid-gap flow velocity (m/s)	Peak velocity (m/s)	Stable velocities from RRI tests (m/s)
					Sill (when settled) (being placed)
GAP No. 1					
Waiting Period 1	30/03/2006	1300	5.7	6.08	NA NA
	31/03/2006	1300	5.9	6.29	NA NA NA
	01/04/2006	1300	5.86	6.25	NA NA
	02/04/2006	1300	5.52	5.89	NA NA
Final closure Phase II	03/04/2006	1230	5.02	5.37	NA NA
	04/04/2006	1160	4.36	4.68	NA NA
	05/04/2006	1090	3.63	3.91	NA NA
	06/04/2006	1020	3.14	3.40	NA NA

	07/04/2006	950	3.06	3.32	NA NA
	08/04/2006	880	3.53	3.84	NA NA
	09/04/2006	810	4.09	4.47	NA NA
	10/04/2006	740	4.55	4.99	~6.7 < 5.89
	11/04/2006	670	4.89	5.38	~6.7 < 5.89
	12/04/2006	600	5.15	5.68	~6.7 < 5.89
	13/04/2006	530	5.37	5.95	~6.7 < 5.89
Waiting Period 2	14/04/2006	530	5.62	6.24	>6.7 NA
	15/04/2006	530	5.73	6.34	>6.7 NA
	16/04/2006	530	5.85	6.48	>6.7 NA
Final closure Phase III	17/04/2006	463.75	6.10	6.77	>6.7 NA
	18/04/2006	397.5	6.19	6.89	>6.7 NA
	19/04/2006	331.25	6.10	6.81	>6.7 NA
	20/04/2006	265	5.86	6.57	>6.7 NA
	21/04/2006	198.75	5.57	6.27	>6.7 NA
	22/04/2006	132.5	5.55	6.27	>6.7 NA
	23/04/2006	66.25	5.97	6.76	>6.7 NA
	24/04/2006	0	6.55	7.44	>6.7 NA
Gap No. 2					
Waiting Period 1	30/03/2006	660	5.62	6.18	NA NA
	31/03/2006	660	5.72	6.29	NA NA
	01/04/2006	660	5.72	6.29	NA NA
	02/04/2006	660	5.36	5.90	NA NA
Final closure Phase II	03/04/2006	628	4.92	5.42	NA NA
	04/04/2006	596	4.33	4.78	NA NA
	05/04/2006	565	3.9	4.31	NA NA
	06/04/2006	533	3.74	4.14	NA NA

	07/04/2006	501	3.67	4.07	NA NA
	08/04/2006	469	4.07	4.52	NA NA
	09/04/2006	437	4.58	5.09	NA NA
	10/04/2006	405	5.04	5.61	NA NA
	11/04/2006	374	5.4	6.02	NA NA
	12/04/2006	342	5.7	6.37	NA NA
	13/04/2006	310	5.97	6.68	NA NA
Waiting Period 2	14/04/2006	310	6.21	6.95	NA NA
	15/04/2006	310	6.3	7.05	NA NA
	16/04/2006	310	6.33	7.08	NA NA
Final closure Phase III	17/04/2006	271	6.40	7.17	NA NA
	18/04/2006	233	6.40	7.19	NA NA
	19/04/2006	194	6.23	7.01	NA NA
	20/04/2006	155	5.92	6.67	NA NA
	21/04/2006	116	5.64	6.36	NA NA
	22/04/2006	78	5.70	6.44	NA NA
	23/04/2006	39	6.05	6.85	NA NA
	24/04/2006	0	6.56	7.45	NA NA

NA – Not available

Table 5.15 Comparison of predicted flow velocities with stable velocities determined experimentally by RRI for Sill – Later Closure

Phase	Date	Gap width (m)	Mid-gap flow velocity (m/s)	Peak velocity (m/s)	Stable velocities from RRI tests (m/s)
					Sill (when settled) (being placed)
GAP No. 1					
Waiting Period 1	28/04/2006	1300	5.67	6.05	NA NA
	29/04/2006	1300	5.82	6.21	NA NA NA

	30/04/2006	1300	5.58	5.95	NA NA
	01/05/2006	1300	5.16	5.50	NA NA
Final closure Phase II	02/05/2006	1230	4.75	5.08	NA NA
	03/05/2006	1160	4.23	4.54	NA NA
	04/05/2006	1090	3.68	3.97	NA NA
	05/05/2006	1020	3.27	3.54	NA NA
	06/05/2006	950	3.33	3.61	NA NA
	07/05/2006	880	3.61	3.93	NA NA
	08/05/2006	810	4	4.37	NA NA
	09/05/2006	740	4.38	4.80	~6.7 < 5.89
	10/05/2006	670	4.7	5.17	~6.7 < 5.89
	11/05/2006	600	5.04	5.56	~6.7 < 5.89
	12/05/2006	530	5.36	5.93	~6.7 < 5.89
Waiting Period 2	13/05/2006	530	5.58	6.18	>6.7 NA
	14/05/2006	530	5.57	6.17	>6.7 NA
	15/05/2006	530	5.6	6.20	>6.7 NA
Final closure Phase III	16/05/2006	463.75	6.12	6.80	>6.7 NA
	17/05/2006	397.5	6.26	6.97	>6.7 NA
	18/05/2006	331.25	6.24	6.98	>6.7 NA
	19/05/2006	265	6.10	6.84	>6.7 NA
	20/05/2006	198.75	5.93	6.67	>6.7 NA
	21/05/2006	132.5	6.11	6.89	>6.7 NA
	22/05/2006	66.25	6.41	7.26	>6.7 NA
	23/05/2006	0	6.80	7.72	>6.7 NA
GAP No. 2					
Waiting Period 1	28/04/2006	660	5.44	5.99	NA NA
	29/04/2006	660	5.4	5.94	NA NA

	30/04/2006	660	5.35	5.89	NA NA
	01/05/2006	660	4.99	5.49	NA NA
Final closure Phase II	02/05/2006	628	4.64	5.11	NA NA
	03/05/2006	596	4.22	4.66	NA NA
	04/05/2006	565	3.97	4.39	NA NA
	05/05/2006	533	3.84	4.25	NA NA
	06/05/2006	501	3.89	4.31	NA NA
	07/05/2006	469	4.14	4.60	NA NA
	08/05/2006	437	4.5	5	NA NA
	09/05/2006	405	4.88	5.44	NA NA
	10/05/2006	374	5.22	5.82	NA NA
	11/05/2006	342	5.51	6.16	NA NA
	12/05/2006	310	5.79	6.48	NA NA
Waiting Period 2	13/05/2006	310	5.93	6.64	NA NA
	14/05/2006	310	6.16	6.89	NA NA
	15/05/2006	310	6.29	7.04	NA NA
Final closure Phase III	16/05/2006	271	6.36	7.13	NA NA
	17/05/2006	233	6.36	7.15	NA NA
	18/05/2006	194	6.21	6.99	NA NA
	19/05/2006	155	5.94	6.69	NA NA
	20/05/2006	116	5.72	6.46	NA NA
	21/05/2006	78	5.74	6.50	NA NA
	22/05/2006	39	5.89	6.67	NA NA
	23/05/2006	0	6.12	6.95	NA NA

NA – Not available

Table 5.16 Comparison of predicted flow velocities with stable velocities determined experimentally by RRI for Dike - Early Closure

Phase	Date	Gap width (m)	Mid-gap flow velocity (m/s)	Peak velocity (m/s)	Stable velocities from RRI tests (m/s) Dike (when settled) (being placed)
GAP No. 1					
Waiting Period 1	30/03/2006	1300	5.7	6.08	NA NA
	31/03/2006	1300	5.9	6.29	NA NA
	01/04/2006	1300	5.86	6.25	NA NA
	02/04/2006	1300	5.52	5.89	NA NA
Final closure Phase II	03/04/2006	1230	5.02	5.37	NA 5.89
	04/04/2006	1160	4.36	4.68	NA 5.89
	05/04/2006	1090	3.63	3.91	NA 5.89
	06/04/2006	1020	3.14	3.40	NA 5.89
	07/04/2006	950	3.06	3.32	NA 5.89
	08/04/2006	880	3.53	3.84	NA 5.89
	09/04/2006	810	4.09	4.47	NA 5.89
	10/04/2006	740	4.55	4.99	NA 5.89
	11/04/2006	670	4.89	5.38	NA 5.89
	12/04/2006	600	5.15	5.68	NA 5.89
	13/04/2006	530	5.37	5.95	NA 5.89
Waiting Period 2	14/04/2006	530	5.62	6.24	NA NA
	15/04/2006	530	5.73	6.34	NA NA
	16/04/2006	530	5.85	6.48	NA NA
Final closure Phase III	17/04/2006	463.75	6.10	6.77	NA 6.15
	18/04/2006	397.5	6.19	6.89	NA 6.15
	19/04/2006	331.25	6.10	6.81	NA 6.15

	20/04/2006	265	5.86	6.57	NA 6.15
	21/04/2006	198.75	5.57	6.27	NA 6.15
	22/04/2006	132.5	5.55	6.27	NA 6.15
	23/04/2006	66.25	5.97	6.76	NA 6.15
	24/04/2006	0	6.55	7.44	NA 6.15
Gap No. 2					
Waiting Period 1	30/03/2006	660	5.62	6.18	~7.2 NA
	31/03/2006	660	5.72	6.29	~7.2 NA
	01/04/2006	660	5.72	6.29	~7.2 NA
	02/04/2006	660	5.36	5.90	~7.2 NA
Final closure Phase II	03/04/2006	628	4.92	5.42	NA 6.31
	04/04/2006	596	4.33	4.78	NA 6.31
	05/04/2006	565	3.9	4.31	NA 6.31
	06/04/2006	533	3.74	4.14	NA 6.31
	07/04/2006	501	3.67	4.07	NA 6.31
	08/04/2006	469	4.07	4.52	NA 6.31
	09/04/2006	437	4.58	5.09	NA 6.31
	10/04/2006	405	5.04	5.61	NA 6.31
	11/04/2006	374	5.4	6.02	NA 6.31
	12/04/2006	342	5.7	6.37	NA 6.31
	13/04/2006	310	5.97	6.68	NA 6.31
Waiting Period 2	14/04/2006	310	6.21	6.95	NA NA
	15/04/2006	310	6.3	7.05	NA NA
	16/04/2006	310	6.33	7.08	NA NA
Final closure Phase III	17/04/2006	271	6.40	7.17	NA 7.18
	18/04/2006	233	6.40	7.19	NA 7.18
	19/04/2006	194	6.23	7.01	NA 7.18

	20/04/2006	155	5.92	6.67	NA 7.18
	21/04/2006	116	5.64	6.36	NA 7.18
	22/04/2006	78	5.70	6.44	NA 7.18
	23/04/2006	39	6.05	6.85	NA 7.18
	24/04/2006	0	6.56	7.45	NA 7.18

NA – Not available

Table 5.17 Comparison of predicted flow velocities with stable velocities determined experimentally by RRI for Dike – Later Closure

Phase	Date	Gap width (m)	Mid-gap flow velocity (m/s)	Peak velocity (m/s)	Stable velocities from RRI tests (m/s) Dike (when settled) (being placed)
GAP No. 1					
Waiting Period 1	28/04/2006	1300	5.67	6.05	NA NA
	29/04/2006	1300	5.82	6.21	NA NA
	30/04/2006	1300	5.58	5.95	NA NA
	01/05/2006	1300	5.16	5.50	NA NA
Final closure Phase II	02/05/2006	1230	4.75	5.08	NA 5.89
	03/05/2006	1160	4.23	4.54	NA 5.89
	04/05/2006	1090	3.68	3.97	NA 5.89
	05/05/2006	1020	3.27	3.54	NA 5.89
	06/05/2006	950	3.33	3.61	NA 5.89
	07/05/2006	880	3.61	3.93	NA 5.89
	08/05/2006	810	4	4.37	NA 5.89
	09/05/2006	740	4.38	4.80	NA 5.89
	10/05/2006	670	4.7	5.17	NA 5.89
	11/05/2006	600	5.04	5.56	NA 5.89
	12/05/2006	530	5.36	5.93	NA 5.89
Waiting Period 2	13/05/2006	530	5.58	6.18	NA NA

	14/05/2006	530	5.57	6.17	NA NA
	15/05/2006	530	5.6	6.20	NA NA
Final closure Phase III	16/05/2006	463.75	6.12	6.80	NA 6.15
	17/05/2006	397.5	6.26	6.97	NA 6.15
	18/05/2006	331.25	6.24	6.98	NA 6.15
	19/05/2006	265	6.10	6.84	NA 6.15
	20/05/2006	198.75	5.93	6.67	NA 6.15
	21/05/2006	132.5	6.11	6.89	NA 6.15
	22/05/2006	66.25	6.41	7.26	NA 6.15
	23/05/2006	0	6.80	7.72	NA 6.15
GAP No. 2					
Waiting Period 1	28/04/2006	660	5.44	5.99	~7.2 NA
	29/04/2006	660	5.4	5.94	~7.2 NA
	30/04/2006	660	5.35	5.89	~7.2 NA
	01/05/2006	660	4.99	5.49	~7.2 NA
Final closure Phase II	02/05/2006	628	4.64	5.11	NA 6.31
	03/05/2006	596	4.22	4.66	NA 6.31
	04/05/2006	565	3.97	4.39	NA 6.31
	05/05/2006	533	3.84	4.25	NA 6.31
	06/05/2006	501	3.89	4.31	NA 6.31
	07/05/2006	469	4.14	4.60	NA 6.31
	08/05/2006	437	4.5	5	NA 6.31
	09/05/2006	405	4.88	5.44	NA 6.31
	10/05/2006	374	5.22	5.82	NA 6.31
	11/05/2006	342	5.51	6.16	NA 6.31
	12/05/2006	310	5.79	6.48	NA 6.31
Waiting Period 2	13/05/2006	310	5.93	6.64	NA NA

	14/05/2006	310	6.16	6.89	NA NA
	15/05/2006	310	6.29	7.04	NA NA
Final closure Phase III	16/05/2006	271	6.36	7.13	NA 7.18
	17/05/2006	233	6.36	7.15	NA 7.18
	18/05/2006	194	6.21	6.99	NA 7.18
	19/05/2006	155	5.94	6.69	NA 7.18
	20/05/2006	116	5.72	6.46	NA 7.18
	21/05/2006	78	5.74	6.50	NA 7.18
	22/05/2006	39	5.89	6.67	NA 7.18
	23/05/2006	0	6.12	6.95	NA 7.18

NA – Not available

Bed protection

From Tables 5.12 and 5.13 it is apparent that there is considerable uncertainty regarding the actual flow resistance capability of the rock/gabion mixtures used for the bed protection, particularly for the situation when the mixtures are being placed on the bed. Using also information from RRI on other mixtures it can be inferred that the existing bed protection may be at the limit of stability for flow conditions associated with Waiting Period 1 and be unstable during Waiting Period 2 and Phase III. Recent surveys show no evidence of scour developing in the protected area apart from the edges of the mat and this is substantiated by experimental work carried out at HR Wallingford which suggested stable velocities of the order of 6 to 6.5m/s. However, as discussed in Chapter 4, during the closure phases there is potential for increased turbulence to be generated at the edges of the bed protection mat which may instigate further scour. The existing bed protection is considered to be at the limit of stability during Waiting period 1, is expected to be stable during most of Final Closure Phase II but to require upgrading for the latter part of Phase II, Waiting Period 2 and Phase III.

Sill

It is expected that the sill would have been built by the start of Waiting period 1 and therefore information in Tables 5.14 and 5.15 regarding stable velocities when the material is settled will apply. From these tables, for Gap No. 1, comparison of the peak velocities and stable velocities for the sill material indicates that the proposed sill materials are likely to be stable during Phase II and Waiting Period 2 (no information is available for Waiting Period 1) but that during Phase III they will be unstable particularly for the Later Closure option. No information is available regarding the materials used for Gap No.2.

Dike

From Tables 5.16 and 5.17, it can be seen that most of the data available on the stability of the dike refers to the placing conditions rather than the settled state. Placing conditions are generally more severe and therefore these provide sufficient information for conclusions to be drawn. During Phase I the specified mixture is likely to be stable. For Gap No.1, there is no information regarding Waiting period 1 and the specified stone mixtures are likely not to be stable towards the end of Phase II, Waiting period 2 and under Phase III conditions. For Gap No.2 conditions at the end of Phase II and Waiting period 2 are likely to generate instability.

The above conclusions are summarised in Tables 5.18 and 5.19, which provide a general way of appreciating the adequacy or otherwise of the proposed materials. In these tables, red indicates that RRI tests suggest that the proposed materials are inadequate, whereas green suggests no appreciable concern and orange indicates limit of stability or instability during part of the period (blank areas indicate no information).

Table 5.18 Summary of conclusions based on test results by RRI – Gap No.1

Phase	Early Closure			Later Closure		
	Bed	Sill	Dike	Bed	Sill	Dike
Phase I						
Waiting Period 1						
Final Closure Phase II						
Waiting Period 2						
Final Closure Phase III						

Table 5.19 Summary of conclusions based on test results by RRI – Gap No.2

Phase	Early Closure			Later Closure		
	Bed	Sill	Dike	Bed	Sill	Dike
Phase I						
Waiting Period 1						
Final Closure Phase II						
Waiting Period 2						
Final Closure Phase III						

5.7 RECOMMENDATIONS

5.7.1 Basis for the recommendations

The previous sections described various methods used for the determination of stable velocities and indicated that two different approaches for the elaboration of conclusions could be used: the empirical equations and the experimental information obtained by RRI. Given that the information from RRI offers specific data for the mixtures of rock and gabions that are envisaged to be used, and that the application of the empirical formulae required certain assumptions to be made to determine representative stone sizes for these mixtures, it is considered that the recommendations should be based primarily on the RRI information. This is supported by the fact that conclusions using the empirical equations, although different in places, generally agreed well with those drawn from the RRI results (compare for example Tables 5.10 and 5.18). However, the RRI information is not complete and it is useful to supplement it with conclusions drawn from the empirical equations. This is depicted in Tables 5.20 and 5.21, where the gaps shown in Tables 5.17 and 5.18 were filled by information from Tables 5.10 and 5.11.

Table 5.20 Summary of conclusions – Gap No. 1

Phase	Early Closure			Later Closure		
	Bed	Sill	Dike	Bed	Sill	Dike
Phase I						
Waiting Period 1						
Final Closure Phase II						
Waiting Period 2						
Final Closure Phase III						

Table 5.21 Summary of conclusions – Gap No. 2

Phase	Early Closure			Later Closure		
	Bed	Sill	Dike	Bed	Sill	Dike
Phase I						
Waiting Period 1						
Final Closure Phase II						
Waiting Period 2						
Final Closure Phase III						

Recommendations for the bed protection, sill and closure dike are based on the assumption that the peak velocities generated at the ends of the dike openings represent a 13% increase for gap widths of 100m, as found in the HR Wallingford (2002) physical modelling studies, reducing to 5% for gap widths of 1600m. A linear variation was assumed between the two values. The adoption of a variable factor for determination of the peak velocities was requested by KARICO during meetings held at HR Wallingford on 30 August 2005. We would point out, as we did in our 2002 report that the HR Wallingford tests were for a specific situation and without any flow asymmetry. Some researchers have found local velocity increases of the order of 20%. Stable stone weights are a function of the 6^{th} power of velocity and hence, should these larger velocity increases be present, our calculations and recommendations would not be valid and larger stone sizes or increased proportions of gabions would be necessary.

HR Wallingford was requested to provide recommendations for stable stone/gabion mixtures based on periods of 48 hours. For the development of these recommendations, the maximum predicted peak velocities during each of the two-day periods were used.

It can be seen from tables 5.20 and 5.21 that the currently specified stone/gabion mixtures (summarised in Table 5.1) are not suitable for all the types of location and periods of closure considered in this assessment, and therefore, recommendations are made in this section to achieve stability. RRI had provided information on stability of several rock/gabion mixtures. Based on this information, for each of these stone mixtures (e.g. 3-5t rock, or 3-6t rock), the percentage of gabions was plotted against the flow velocities that the mixture is able to withstand (see Figures 5.1 to 5.4) and equations were obtained by linear regression that enabled extrapolations for higher gabion percentages, if needed. In all cases, the relationship between percentage gabion and stable flow velocity was very linear and high regression coefficients (of the order of 0.99) were obtained. It should be noted, however, that the stability of mixtures with very high percentages of gabions may not be adequately represented by these equations: within the team's experience, observations have indicated that in mixtures of rock/gabions with high proportions of sack gabions these tend to roll, thus producing a less stable mixture than expected. For this reason, when deriving the recommendations, it was assumed that 50% would be the maximum percentage of gabions in the mixtures. On this basis, it was possible to suggest stable stone sizes for the various works, as described below in Sections 5.7.2 to 5.7.4.

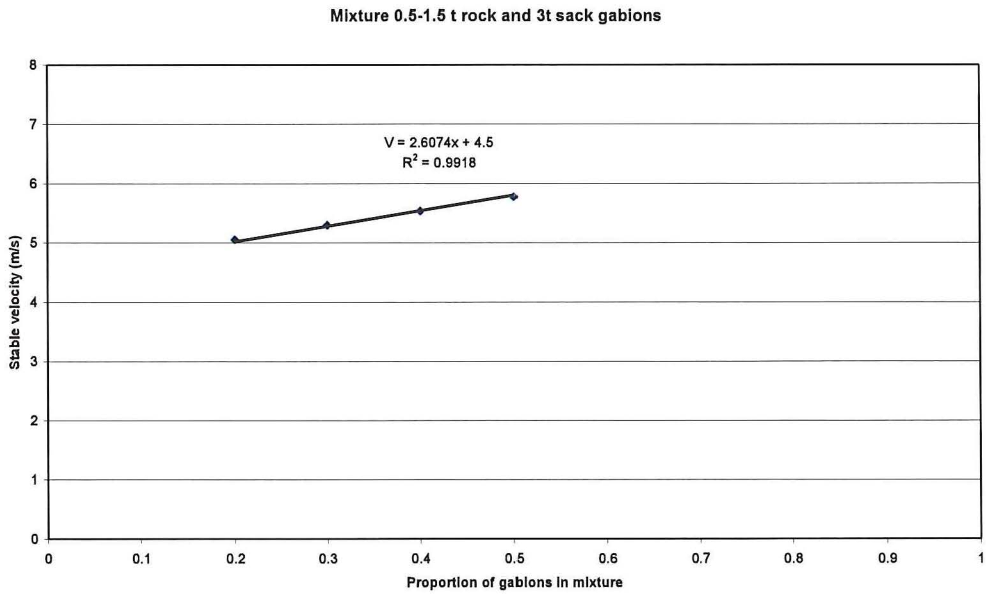

Figure 5.1 Relationship between stable velocities for mixture 0.5-1.5t rock and 3t gabions and proportion of gabions in mixture

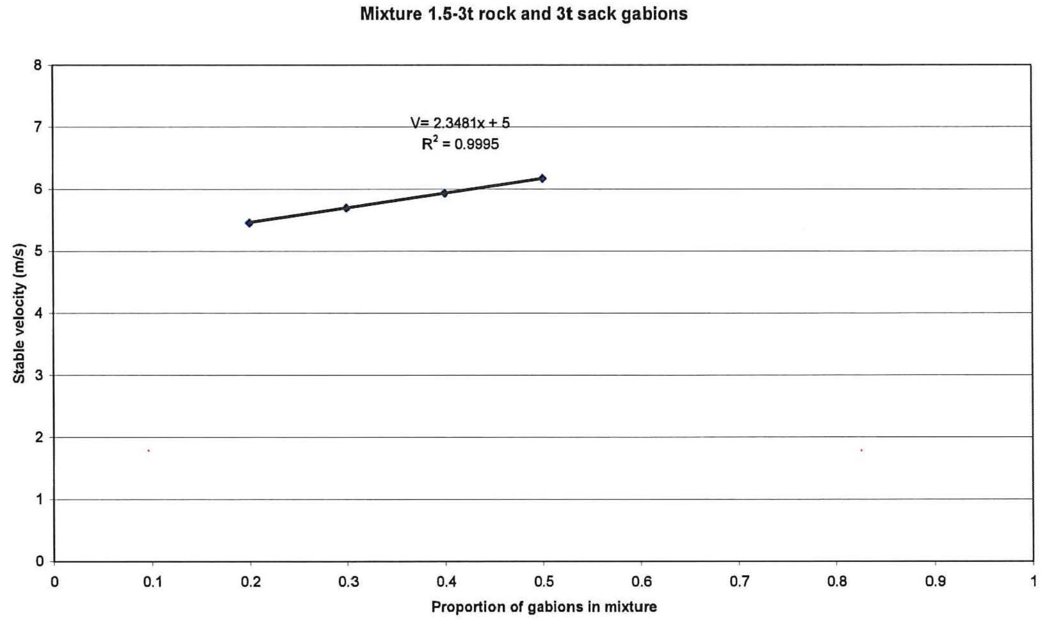

Figure 5.2 Relationship between stable velocities for mixture 1.5-3.0t rock and 3t gabions and proportion of gabions in mixture

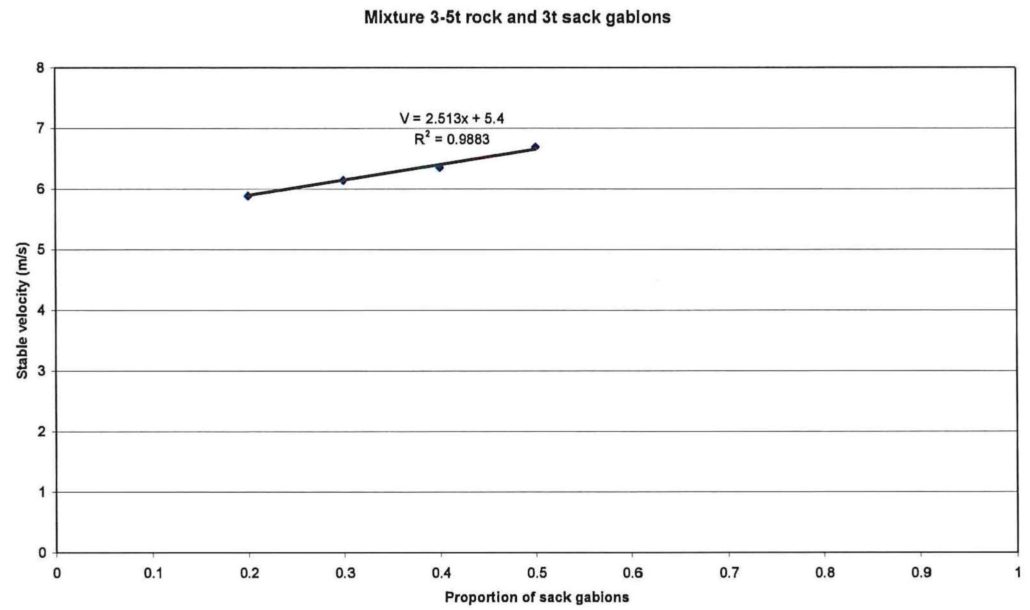

Figure 5.3 Relationship between stable velocities for mixture 3.0-5.0t rock and 3t gabions and proportion of gabions in mixture

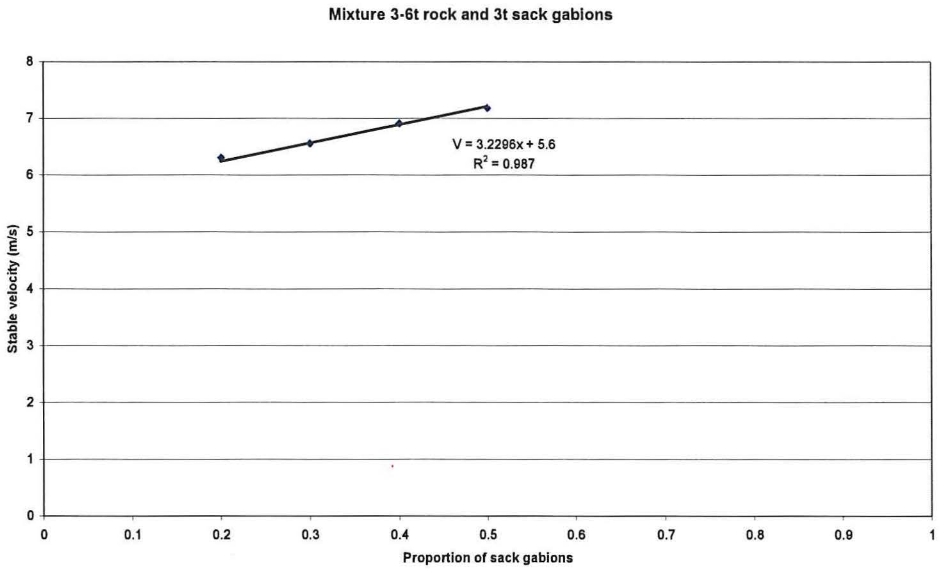

Figure 5.4 Relationship between stable velocities for mixture 3.0-6.0t rock and 3t gabions and proportion of gabions in mixture

It will be noted that, in some cases, the recommendations indicate the need to use 3-6t rock plus 3t gabions. It is appreciated that the specification of large quantities of stone of this size has practical and economical implications but we would want to stress the importance of ensuring safe design which will later be reflected in safe construction.

The assessment carried out did not indicate a strong advantage in adopting either the Early or the Later Closure options but the Later option was marginally more advantageous for Gap No.2.

5.7.2 Recommendations for the bed protection

Ensuring adequate protection of the bed during the closure stages involves consideration of two distinct aspects: the stability of the bulk of the bed protection mat which is approximately level, and the stability of the mat at the edges, where deep scour has occurred in the past and further scour is predicted (see Chapter 4).

With regard to the bulk of the mat (flat area), recommendations need to take account of the relationship between the following two parameters: date and location. Tables 5.20 and 5.21 indicate that the currently specified mixture will need to be upgraded from the end of Phase II. A decision regarding upgrading during Phase II will be dictated by economic reasons as the bed material is expected to be at the limit of stability and some movement may be acceptable. Tables 5.22 and 5.23 (for the Earlier and Later Closure options, respectively) present the sizes of the stone/gabion mixtures that are expected to provide stable protection based on two-day periods. During Phase III, large stone of 3-6t (50%) plus 3t gabions (50%) will be required, which implies that it only needs to be placed over the central sections of Gaps No. 1 and 2.

Table 5.22 Recommended stone/gabion mixtures for Bed Protection (flat area) - Early Closure

Phase	GAP No. 1		GAP No. 2	
	Required stable vel. (m/s)	**Required stone/gabion mix**	**Required stable vel. (m/s)**	**Required stone/gabion mix**
Waiting Period 1				
30/03/2006 31/03/2006	5.0	0.5-1t (90%) rock* 2t (10%) gabions	5.0	0.5-2.5t (90%) rock* 2t (10%) gabions
01/04/2006 02/04/2006	5.0	0.5-1t (90%) rock* 2t (10%) gabions	5.0	0.5-2.5t (90%) rock* 2t (10%) gabions
Phase II				
03/04/2006 04/04/2006	4.6	0.5-1t (90%) rock* 2t (10%) gabions	4.6	0.5-2.5t (90%) rock* 2t (10%) gabions
05/04/2006 06/04/2006	3.3	0.5-1t (90%) rock* 2t (10%) gabions	3.7	0.5-2.5t (90%) rock* 2t (10%) gabions
07/04/2006 08/04/2006	3.3	0.5-1t (90%) rock* 2t (10%) gabions	3.8	0.5-2.5t (90%) rock* 2t (10%) gabions
09/04/2006 10/04/2006	4.2	0.5-1t (90%) rock* 2t (10%) gabions	4.8	0.5-2.5t (90%) rock* 2t (10%) gabions
11/04/2006 12/04/2006	4.8	0.5-1t (90%) rock* 2t (10%) gabions	5.4	0.5-2.5t (90%) rock* 2t (10%) gabions
13/04/2006	5.0	0.5-1t (90%) rock* 2t (10%) gabions	5.7	0.5-2.5t (90%) rock* 2t (10%) gabions
Waiting period 2				
14/04/2006 15/04/2006	5.7	(1)	6.3	(3)
16/04/2006	5.8	(1)	6.4	(3)
Phase III				
17/04/2006 18/04/2006	6.9	(4)	7.2	(4)
19/04/2006 20/04/2006	6.8	(4)	7.0	(4)
21/04/2006 22/04/2006	6.3	(3); for practical reasons use (4)	6.4	(3); for practical reasons use (4)
23/04/2006 24/04/2006	7.4	>(4)	7.4	>(4)

* As specified (see Table 5.1)

Notes:

Stone/gabion mixes	Stable velocity (m/s)	
(1) 0.5-1.5t rock (50%) + 3t gabions (50%)	5.8	(Fig. 5.1)
(1a) 0.5-1.5t rock (80%) + 3t gabions (20%)	5.0	(Fig. 5.1)
(1b) 0.5-1.5t rock (90%) + 3t gabions (10%)	4.8	(Fig. 5.1)
(2) 1.5-3t rock (50%) + 3t gabions (50%)	6.2	(Fig 5.2)
(3) 3-5t rock (50%) + 3t gabions (50%)	6.7	(Fig 5.3)
(4) 3-6t rock (50%) + 3t gabions (50%)	7.2	(Fig 5.4)

Table 5.23 Recommended stone/gabion mixtures for Bed Protection (flat area) - Later Closure

Phase	GAP No. 1		GAP No. 2	
	Required stable vel. (m/s)	**Required stone/gabion mix**	**Required stable vel. (m/s)**	**Required stone/gabion mix**
Waiting Period 1				
28/04/2006 29/04/2006	5.0	0.5-1t (90%) rock* 2t (10%) gabions	4.8	0.5-2.5t (90%) rock* 2t (10%) gabions
30/04/2006 01/05/2006	4.8	0.5-1t (90%) rock* 2t (10%) gabions	4.7	0.5-2.5t (90%) rock* 2t (10%) gabions
Phase II				
02/05/2006 03/05/2006	4.3	0.5-1t (90%) rock* 2t (10%) gabions	4.4	0.5-2.5t (90%) rock* 2t (10%) gabions
04/05/2006 05/05/2006	3.4	0.5-1t (90%) rock* 2t (10%) gabions	3.7	0.5-2.5t (90%) rock* 2t (10%) gabions
06/05/2006 07/05/2006	3.3	0.5-1t (90%) rock* 2t (10%) gabions	3.9	0.5-2.5t (90%) rock* 2t (10%) gabions
08/05/2006 09/05/2006	4.1	0.5-1t (90%) rock* 2t (10%) gabions	4.6	0.5-2.5t (90%) rock* 2t (10%) gabions
10/05/2006 11/05/2006	4.7	0.5-1t (90%) rock* 2t (10%) gabions	5.2	0.5-2.5t (90%) rock* 2t (10%) gabions
12/04/2006	5.0	0.5-1t (90%) rock* 2t (10%) gabions	5.5	0.5-2.5t (90%) rock* 2t (10%) gabions
Waiting period 2				
13/05/2006 14/05/2006	5.6	(1)	6.2	(2)
15/05/2006	5.6	(1)	6.3	(3)
Phase III				
16/05/2006 17/05/2006	7.0	(4)	7.2	(4)
18/05/2006 19/05/2006	7.0	(4)	7.0	(4)
20/05/2006 21/05/2006	6.9	(4)	6.5	(3); for practical reasons use (4)
22/05/2006 23/05/2006	7.7	>(4)	7.0	(4)

* As specified (see Table 5.1)

Notes:

Stone/gabion mixes	Stable velocity (m/s)	
(1) 0.5-1.5t rock (50%) + 3t gabions (50%)	5.8	(Fig. 5.1)
(1a) 0.5-1.5t rock (80%) + 3t gabions (20%)	5.0	(Fig. 5.1)
(1b) 0.5-1.5t rock (90%) + 3t gabions (10%)	4.8	(Fig. 5.1)

(2) 1.5-3t rock (50%) + 3t gabions (50%)	6.2	(Fig 5.2)
(3) 3-5t rock (50%) + 3t gabions (50%)	6.7	(Fig 5.3)
(4) 3-6t rock (50%) + 3t gabions (50%)	7.2	(Fig 5.4)

With regard to protection of the edges of the mat, calculations were carried out using the empirical equations described in Section 5.3.2 in which the destabilising effect of the slope at the edges of the mat was taken into account. For these calculations an approximate method was applied which assumed that flow velocities at the edge of the mat would be about 80% of the peak velocities at the gaps, a reduction caused by the distance from the dike centreline as well as the increase in water depth at the scour hole formed at the edge of the mat. According to recent surveys, the slope of the scoured edges of the mats ranges between 1:8 and 1:1.5. For the purpose of these calculations, a typical slope of 1:2 was considered and water depths of 20m and 30m were assumed for Gap No.1 and Gap No.2 respectively. The calculations were carried out for the Early Closure option but conclusions would be similar for the Later option. With these assumptions, the following recommendations can be made in order to try to prevent further erosion of the edges of the existing bed protection:

Gap No.1
The edges of the bed protection mat would need to be reinforced by the addition of stone with D_n=0.95m (or 2.3t in weight) to achieve the required stability from Waiting period 1 to end of Phase II. However, in order to withstand the flow conditions occurring during Phase III, it is recommended to use a larger rock size of with D_n= 1.25m (or 4.6t in weight).

Gap No.2
The edges of the bed protection mat should be reinforced by the addition of stone with D_n=1.25m (or 4.6t in weight). This additional protection will be required from the latter parts of Phase II.

5.7.3 Recommendations for the sill

For Gap No. 1, as can be seen from Tables 5.20 and 5.21, the specified stone/gabion mixtures appear to be stable up to Phase III, when an upgrade will be required to ensure stability. In order to prevent instability during Phase III, it is recommended to use 3-6t rock (50%) and 3t gabions (50%) in any further work carried out on the sill between the present time and Phase III.

For Gap No. 2 the specified stone/gabion mixtures are likely to be stable during the closure works.

5.7.4 Recommendations for the dike

For Phase I, the assessment of the data indicated that the specified stone mixture (1.5-3.0t (70%) rock plus 3t (30%) gabions for Gap No. 1 and 1.5-3.0t (60%) rock plus 3t (40%) gabions) for Gap No. 2 is adequate.

For the Waiting periods and Phases II and III, the assessment based on two-day periods is summarised in Tables 5.24 and 5.25, where examples are given of stone/gabion mixtures that should provide stability.

Table 5.24 Recommended stone/gabion mixtures for Dike - Early Closure

Phase	GAP No. 1		GAP No. 2	
	Required stable vel. (m/s)	Required stone/gabion mix	Required stable vel. (m/s)	Required stone/gabion mix
Waiting Period 1				
30/03/2006 31/03/2006	6.3	(3)	6.3	(3) or as specified
01/04/2006 02/04/2006	6.3	(3)	6.3	(3) or as specified
Phase II				
03/04/2006 04/04/2006	5.4	(1)	5.4	(1)
05/04/2006 06/04/2006	3.9	(1b)	4.3	(1b)
07/04/2006 08/04/2006	3.8	(1b)	4.5	(1b)
09/04/2006 10/04/2006	5.0	(1a)	5.6	(1)
11/04/2006 12/04/2006	5.7	(1)	6.4	(3)
13/04/2006	5.9	(2)	6.7	(3)
Waiting period 2				
14/04/2006 15/04/2006	6.3	(3)	7.0	(3)
16/04/2006	6.5	(3)	7.1	(3)
Phase III				
17/04/2006 18/04/2006	6.9	(4)	7.2	(4)*
19/04/2006 20/04/2006	6.8	(4)	7.0	(4)*
21/04/2006 22/04/2006	6.3	(3); for practical reasons use (4)	6.4	(3); for practical reasons use (4)*
23/04/2006 24/04/2006	7.4	(4)	7.4	(4)*

* As specified (see Table 5.1)

Notes:

Stone/gabion mixes	Stable velocity (m/s)	
(1) 0.5-1.5t rock (50%) + 3t gabions (50%)	5.8	(Fig. 5.1)
(1a) 0.5-1.5t rock (80%) + 3t gabions (20%)	5.0	(Fig. 5.1)
(1b) 0.5-1.5t rock (90%) + 3t gabions (10%)	4.8	(Fig. 5.1)
(2) 1.5-3t rock (50%) + 3t gabions (50%)	6.2	(Fig 5.2)
(3) 3-5t rock (50%) + 3t gabions (50%)	6.7	(Fig 5.3)
(4) 3-6t rock (50%) + 3t gabions (50%)	7.2	(Fig 5.4)

Table 5.25 Recommended stone/gabion mixtures for Dike - Later Closure

Phase	GAP No. 1		GAP No. 2	
	Required stable vel. (m/s)	Required stone/gabion mix	Required stable vel. (m/s)	Required stone/gabion mix
Waiting Period 1				
28/04/2006 29/04/2006	6.2	(2)	6.0	(2)
30/04/2006 01/05/2006	5.9	(2)	5.9	(2)
Phase II				
02/05/2006 03/05/2006	5.1	(1)	5.1	(1)
04/05/2006 05/05/2006	4.0	(1b)	4.4	(1b)
06/05/2006 07/05/2006	3.9	(1b)	4.6	(1b)
08/05/2006 09/05/2006	4.8	(1b)	5.4	(1)
10/05/2006 11/05/2006	5.6	(1)	6.2	(2)
12/04/2006	5.9	(2)	6.5	(3)
Waiting period 2				
13/05/2006 14/05/2006	6.2	(2)	6.9	(4)
15/05/2006	6.2	(2)	7.0	(4)
Phase III				
16/05/2006 17/05/2006	7.0	(4)	7.2	(4)*
18/05/2006 19/05/2006	7.0	>(4)	7.0	(4)*
20/05/2006 21/05/2006	6.9	(4)	6.5	(3); for practical reasons use (4)*
22/05/2006 23/05/2006	7.7	>(4)	7.0	(4)*

* As specified (see Table 5.1)

Notes:

Stone/gabion mixes	Stable velocity (m/s)	
(1) 0.5-1.5t rock (50%) + 3t gabions (50%)	5.8	(Fig. 5.1)
(1a) 0.5-1.5t rock (80%) + 3t gabions (20%)	5.0	(Fig. 5.1)
(1b) 0.5-1.5t rock (90%) + 3t gabions (10%)	4.8	(Fig. 5.1)
(2) 1.5-3t rock (50%) + 3t gabions (50%)	6.2	(Fig 5.2)
(3) 3-5t rock (50%) + 3t gabions (50%)	6.7	(Fig 5.3)
(4) 3-6t rock (50%) + 3t gabions (50%)	7.2	(Fig 5.4).

6. Review and evaluation of the applicability of the planned schedule for final closure

6.1 ASSESSMENT OF CONTRACTOR'S CAPACITY TO ACHIEVE RATES OF PLACING OF MATERIAL

The contractor's capacity to place material for the horizontal closure at an adequate rate can be assessed from the data furnished by the contractor and summarised in Section 2.5 above. The following points may be made about the rate of progress by evaluating the anticipated use of the land-based plant:

1. Based on deliveries every 30 seconds on both sides of the gap, we can say that a total of 2x2 = 4 deliveries would be made per minute. Given that the contractor has advised that the truck cycle time is estimated as 15 minutes, then a total of 4x15 = 60 trucks would be required. However, the contractor has advised that in fact some 150 vehicles will be deployed per gap. If 120 of these vehicles were operational at any time (allowing 20% outage for refuelling/maintenance), we can therefore conclude that the cycle time per vehicle will increase on average to about 30 minutes. This is much more satisfactory as it means that vehicles will tend to be queuing at both the loading and discharge locations and this will ensure a steady rate of progress.

2. One truck discharging every 30 seconds is an extremely rapid rate of progress and the contractor, Hyundai should be asked to demonstrate to KARICO that this rate of progress can be achieved, including allowing adequate time and space for manoeuvring of trucks. It is also essential to ensure that adequate human safety precautions are maintained at all times whilst working at this extremely rapid rate, especially during hours of darkness at night.

3. Based on deliveries every 30 seconds on both sides of the gap, an estimate can be made of the adequacy of the rate of filling that can be achieved. We have assumed in our calculations that one load discharged from a 15t truck will fill 7.5m^3 of closure bund. This density is probably slightly conservative, but it allows for the actual quantity per truck to vary a little below the maximum.

4. The **rate of filling** that could be achieved, based on 4 truck loads per minute, is 30m^3 per minute or 39,600 m^3 per day (allowing for 22 hours per day working). This compares with the required maximum rate of progress of 27,720 m^3/day. Comparing these two figures implies that the **placing efficiency** could be as low as about 70%. The contractor has estimated based on his experience that the placing efficiency might be of the order of 80% (i.e. 20% losses). Thus the available capacity means that this rate of loss could increase by a further 50% and adequate progress still be maintained.

All these calculations of rates of progress exclude the additional input of gabions to the sides of the progressing closure bund provided by the marine plant.

6.2 GABIONS FOR USE IN CONSTRUCTION

Having established that the rate of placing of materials is feasible, the other key feature of the final closure operation is to ensure that the placed stone is sufficiently stable. We will not repeat here our concerns expressed in the Chapter 5 about the stability of the stone – Chapter 5 makes it clear that significant increases to stone sizes and/or proportions of gabions will be necessary in addition to the increases already recommended by RRI. We have two main concerns in regard to the gabions.

1. It is important that the gabions as constructed are as similar as possible to those tested in the physical modelling at RRI which validated their use. Photographs and videos do not really prove that the full size gabions are similar to the model gabions in terms of their construction and quality (shape, compactness, grading, strength of wire netting, deformation due to currents). KARICO must therefore invest considerable effort with the Contractor to ensure the gabions are of appropriate quality.

2. Considerable efforts will be needed over the next few months to ensure that the required additional gabions have been manufactured in time for the closure period.

7. *Risk analysis for final closure*

7.1 INTRODUCTION

KARICO have identified as a key risk the failure of the final closure of gaps 1 and 2 by a major collapse during final closure, involving subsequent erosion of top layers and/or under layers and cause deep sea bed scour that could possibly reach the bed rock level. Refilling of the scour hole and rebuilding of the seabed protection and sill and/or temporary rock dam would entail large costs and significant project delay, preventing completion of closure within working season 2006. However a dam breach of a completed section of the temporary closing dam could also cause subsequent erosion of dam material and result in deep sea bed scour.

Apart from the event of a major collapse during final closure of gaps 1 and 2, also inadequate progress during final closure would cause project delay, potentially preventing completion of closure within the 2006 working season. Inadequate progress, i.e. slower than expected progress of gap closure, may include temporary widening of the gap due to failure of the head of the temporary rock dam due to high flow velocities. Construction issues, however, such as inadequate rock delivery logistics or social, political or judicial issues could also cause inadequate progress. Such events could result in minor as well as major project delays, possibly dependent on how the closure process is modified to increase progress.

The risk of the 2 events described above has been analysed using two approaches:

- Preparation of a risk register, using the PPP-COM tool
- Development of fault trees

7.2 DEVELOPMENT OF A RISK REGISTER FOR THE CLOSURE PROJECT

7.2.1 Explanation of the workshop process

The workshop held at the end of August at HR Wallingford was structured around the generic risk assessment and management steps set out in CIRIA Risk Com, using the PPP-COM tool. This tool, developed by HR Wallingford, uses five simple steps as shown in Table 7.1.

Table 7.1 Generic risk management steps

Step	Description
1. Defining the assessment	Setting the assessment objectives, focus the process, understand the bigger picture
2. Identifying the hazards	Listing what can go wrong
3. Assessing the risks	Assessing the likelihood and consequences of the risks
4. Setting up the response/action plan	Planning control actions
5. Carry out the response/action plan	Responding to the assessment

The workshop agenda followed this sequence:

- Welcome and introductions
- Step 1: Defining the assessment

  - Discussion of aims and objectives of the workshop
- Step 2: Hazard/risk identification
  - To be grouped in various risk areas
- Step 3: Risk assessment
  - Identification of the probability/frequency of occurrence and the likely impact
  - Prioritisation of risks
- Step 4: Risk mitigation strategies
  - Identification of likely strategies, their likely effectiveness and practicability.

Step 1: Defining the assessment
The first step was to define the scope of the risk assessment workshop.

Step 2: Identifying the risks
The second step was to identify the risks. HR Wallingford presented a list of risks prepared prior to the workshop. No additional items were added to this draft risk register during the workshop, but some descriptions were modified.

Step 3: Assessing the risks
<u>Method</u>
The objective of the risk assessment was to decide by considering the likelihood and consequence of the risk, which risks need managing.

In qualitative risk assessment scales are used for likelihood and consequence so that management efforts can be focused on the most important risks. This tool uses a simple approach to assessing risks:

Risk rating = Likelihood x Consequence

- *Likelihood* is sometimes referred to as *Probability.*
- *Impact* is another term that is commonly used in this context and means the wider unintended *consequence* of an event.

The Probability is the chance of the hazard/opportunity occurring whereas the Impact is the effect the hazard/opportunity has if it was to occur.

<u>Assessment scales</u>

In assessing likelihood the following figures have been held in mind:
Low likelihood: 0 – 30% probability that an event occurs
Medium likelihood: 30 – 70% probability that an event occurs
High likelihood: 70 – 100% probability that an event occurs

In assessing consequence the following financial figures have been held in mind:
Low consequence: of the order of £0.4 million
Medium consequence: of the order of £4 million
High consequence: of the order of £40 million

These figures related to the estimated monthly construction costs at the Saemangeum site of £40 million per month and the estimated time for the final closure of about a month.

Table 7.2 Summary of risk assessment categories used in the assessment

		Likelihood		
		Low	**Medium**	**High**
Consequence	**Low**			
	Medium			
	High			

Low risk:		Medium risk:		High risk:	

During the workshop some modifications were made to the assessments of probability and consequence, as they were listed in the draft risk register prepared prior to the workshop.

Step 4: Mitigation measures

The next step was to determine appropriate mitigation measures, focusing on the risks with high risk ratings. The workshop served to identify a portfolio of risk mitigation measures.

Step 5: Carrying out the response plan

This step is perhaps the most important, as without effective action, the risks identified and assessed will not be managed better in the future.

Step 4 - Setting up the response/action plan

Task:
Risk assessment of the closure process

Update:
26/09/2005 14:48

Assessment by:
ICC

CIRIA
sharing knowledge
building best practice

Item No	Area	Risk summary	Likely start of risk period	Likely end of risk period	Likelihood	Consequence	Risk rating	Control strategy	Control strategy cost (High, Med, Low)	Owner	Action by when	Residual Likelihood	Residual Consequence	Residual risk rating	Risk control strategy in place yet ? (Y/N)	Risk control strategy reference
1	Political	Strikes/labour relations			L	M	L	Maintain good labour relations	Nil	KARICO		L	M	L	N	
2	Political	Outbreak of hostilities			L	M	L	None specific	Nil	KARICO		L	M	L	N	
3	Political	Environmental body objects/injunctions			L	H	M	Monitor situation	L	KARICO		L	H	M	N	
4	Political	Environmental protestors			L	L	L	Maintain high site security	L	KARICO		L	L	L	N	
5									Nil	KARICO					N	
6	Technical	Change design			H	L	M	Address early	L	KARICO		M	L	L	N	
7	Natural	Behind schedule due to weather			L	L	L	Maintain addequate project float	L	KARICO		L	L	L	N	
8	Natural	Weather on the day			L	L	L	Monitor weather forecasts	L	KARICO		L	L	L	N	
9	Human	Human error in planning			L	M	L	Provide adequate checks	L	KARICO		L	M	L	N	
10	Human	Human error during closure			L	M	L	Provide adequate checks	L	KARICO		L	M	L	N	
11	Human	Inadequate quality control			L	M	L	Provide adequate checks	L	KARICO		L	M	L	N	
12	Technical - Hydrodynamics	Input conditions			L	M	L	Continue to monitor using real time measurements	Nil	KARICO		L	M	L	N	
13	Technical - Hydrodynamics	Modelling approach and calibration			M	M	M	Undertake and incorporate recommendations of the independent peer reviews	L	KARICO		L	M	L	N	
14	Technical - Hydrodynamics	Application of modelling outputs			M	M	M	Undertake and incorporate recommendations of the independent peer reviews	L	KARICO		L	M	L	N	
15	Technical - Sill	Erosion of stone			M	M	M	Undertake and incorporate recommendations of the independent peer reviews	L	KARICO		L	M	L	N	
16	Technical - Sill	Geotechnical failure			L	M	L	Undertake and incorporate recommendations of the independent peer reviews	L	KARICO		L	M	L	N	
17	Technical - Bed protection	Stone stability			H	M	H	Undertake and incorporate recommendations of the independent peer reviews	L	KARICO		M	M	M	N	
18	Technical - Bed protection	Scour induced slides			M	H	H	Undertake and incorporate recommendations of the independent peer reviews	L	KARICO		L	H	M	N	
19	Technical - Bed protection	Earthquake			L	M	L	None specific	Nil	KARICO		L	M	L	N	
20	Technical - Closure dam	Not making progress (Erosion of the head)			L	H	M	Have ability to place at faster rate and increase effective stone weight if required	L	KARICO		L	H	M	N	
21	Technical - Closure dam	Not making progress (Logistics - Sea based equipment failure)			L	L	L	Ensure maintenance and monitor forecasts	L	KARICO		L	L	L	N	
22	Technical - Closure dam	Not making progress (Logistics - Land based equipment failure)			L	L	L	Ensure maintenance and monitor forecasts	L	KARICO		L	L	L	N	
23	Technical - Closure dam	Not making progress (Logistics - Equipment collision)			L	L	L	Ensure good site control	L	KARICO		L	L	L	N	
24	Technical - Closure dam	Not making progress (Logistics - Material supply failure)			L	H	M	Ensure adquate reserves	L	KARICO		L	H	M	N	
25									Nil	KARICO					N	
26	Technical - Closure dam	Flow through the structure causing blow out of downstream face			L	M	L	Undertake independent peer reviews and monitor situation	L	KARICO		L	M	L	N	
27	Technical - Closure dam	Breaching of closure dam by wave action			L	M	L	Check stone size/Undertake and incorporate recommendations of the independent peer reviews	L	KARICO		L	M	L	N	
28	Health and Safety	Equipment collisions			M	L	L	Ensure good site control	L	KARICO		M	L	L	N	
29	Health and Safety	Accidents due to working at night			L	L	L	Ensure good site control	L	KARICO		L	L	L	N	
30	Health and Safety	Accidents due to working during poor conditions			L	L	L	Ensure good site control	L	KARICO		L	L	L	N	
31	Health and Safety	Accidents due to tiredness			L	L	L	Ensure good site control	L	KARICO		L	L	L	N	

Figure 7.1 Finalized risk register prepared during the risk workshop at HR Wallingford

7.3 FAULT TREES FOR THE INITIATION OF THE KEY FAILURE MODES

Various failure modes of the sills or adjacent sea bed protection or failure modes of the trunk of the dam can ultimately result in a major collapse and large scour hole. Many logical relations between these failure modes exist. A fault tree has been developed that shows these relations for the key failure modes. The presented fault tree does not intend to fully analyze all failure modes, but rather intend to illustrate how failure modes can be analyzed by developing fault trees to various levels of detail. Not all failure modes have been analyzed to the same level of detail. Also the fault trees do not address the likelihood of the various possible events. The fault tree is presented in Appendix A.

Recommendations from fault tree analysis

<u>Major collapse during final closure</u>

The following control and mitigating measures for risk on major collapse are recommended for consideration:

- Use a calibrated wave and flow model and weather and tidal forecast system to predict peak hydraulic loads, possibly including exceedence of design conditions.
- Monitor flow velocity at the gaps during final closure and compare with design conditions.
- Monitor start of erosion of stones at the gaps (Survey during slack water).

  In case of unexpected high flow velocities or start of erosion the following mitigating measures can be taken:
- Reinforce sill and sea bed protection with heavier stone (during slack water). This measure would require stockpile of heavier stone and will cause some project delay.
- Coordinate progress at gaps 1 and 2, to minimise flow velocities or stone erosion at the gap which is at the highest risk of failure.
- Temporarily increase the width of a gap which is at high risk by removing rock from the heads of the temporary rock dam. This measure would benefit from equipment to be on stand-by such as a crane on the dam or a crane barge or backhoe dredger.

7.4 ASSESSMENT OF STABILITY OF TEMPORARY ROCK DAM UNDER WAVE ATTACK AND INFLUENCE ON TIMING OF CLOSURE

The temporary rock dam will be exposed to wave attack during final closure and some period of time after final closure. Stability of the temporary rock dam during this time has been assessed, using the Van der Meer formulae for stability of rock under wave attack (refer CIRIA 1991). Wave data associated with various return periods from HRW report EX 3668 has been adopted for stability calculations.

The results are presented in the Tables below and indicate that only a 100 year storm at gap 1 will result in some damage to the temporary rock dam. A damage number of S = 5.5 indicates slight reshaping of the seaward slope that will not require repair during the final closure operation.

Gap 1

Dam material: 3 – 5 tonne
Average weight 4 tonne
Wave conditions have been taken from HRW report EX 3668 Table 2.5 (output point 1)

	H_s (m)	T_m (s)	Damage number S (-)
R = 1 year	2.1	7.2	No damage
R = 10 year	2.9	8.1	No damage
R = 100 year	3.8	8.9	5.5

Gap 2

Dam material 3 – 6 tonne
Average weight 4.5 tonne
Wave conditions have been taken from HRW report EX 3668 Table 2.7 (output point 3)

	H_s	T_m	Damage number S (-)
R = 1 year	1.5	5.9	No damage
R = 10 year	1.7	6.3	No damage
R = 100 year	2.1	7.2	No damage

The probability that a 100 year storm will occur during the time that the temporary dam is exposed to waves is dependent on the duration of exposure and is indicated in the Table below:

	3 months	6 months	9 months	12 months
R = 100 year	0.25%	0.50%	0.75%	1.0%

7.4.1 Assessment of start data alternatives of the final closure operation in view of monthly wave climate variability

Currently two different start dates for final closure are being considered:

- An early start, entailing the greater part of the work being carried out in April 2006
- A late start, entailing the greater part of the work being carried out in May 2006.

It has been assessed whether monthly wave climate variability affects the relative suitability of the above options.

Two key risk items related to wave height have been identified that affect the final closure operation:

- Damage to the temporary dam while it is not yet incorporated in the final dam structure and is exposed to wave attack
- Wave overtopping over temporary dam hindering construction activities on the dam during final closure (e.g. operation of dump trucks)

Monthly offshore wave data has been reviewed in order to assess the options. The following data has been made available:

- RRI wave data: Waverider data at Maldo station from February to July.
- KORDI data: Average monthly wave heights and periods at various grid points off the western coast of South-Korea.

Damage to the temporary dam

Both the RRI and KORDI wave data indicate a slightly more benign wave climate in May than in April. However, significant damage to the temporary dam due to wave attack storms with large return periods of at least 100 years. It has been assessed therefore that there is no significant advantage for final closure in May rather than April in order to limit the risk of damage to the structures due to wave attack.

Wave overtopping

Guidance on acceptable overtopping levels for construction operations on the temporary rock dam is provided by CIRIA (1991), indicating that a threshold average overtopping discharge for safe operations is in the range of 0.01 – 0.1 l/m/s.

The probability of wave overtopping over the temporary rock dam has been assessed for both April and May, at both gaps 1 and 2. Overtopping has been calculated for various combinations of wave height and water level.

The exceedance probabilities of various wave conditions during April and May have been adopted from wave rider data at Maldo, using linear interpolation.

The near shore wave conditions near gaps 1 and 2 associated with these offshore wave conditions have been taken from HR Wallingford report EX 3668, Tables 2.5 and 2.7. Additional near shore wave conditions have been estimated by interpolating in these Tables. It is noted that wave transformation from offshore to nearshore is affected by the offshore wave direction. The offshore and associated nearshore wave conditions presented in Tables 2.5 and 2.7 are part of the "summer population" (April – August), which is dominated by south-westerly waves. The Maldo wave data does not include wave direction. However, as April and May are "summer" months it is assumed that wave transformation from offshore to near shore indicated in Tables 2.5 and 2.7 is representative for April and May.

A typical tidal curve near the closure dam has been adopted from HR Wallingford report EX 4640, Figure 2.5. The probabilities of exceedance for 3 arbitrary tidal levels (-2.0m MSL, 0m MSL and +2m MSL) have been estimated from this curve.

Wave overtopping has been calculated using Owen's formula (CIRIA 1991):

Crest level: +5.0m MSL
Slope 1:1
Roughness coefficient: 0.5

Wave overtopping calculation results are presented in the Tables below.

Wave overtopping at Gap 1

	Offshore wave conditions			Wave conditions near the gap		Water level (m MSL)	Time exceeded (%)	Time wave height and water level is exceeded (%)	Overtopping (l/m/s)
	Hs (m)	Tm (s)	Time exceeded (%)	Hs (m)	Tm (s)				
April	1.1	3.8	11.1	0.7	3.9	+2	28	3.1	$1.6x10^{-3}$
						0	50	5.6	$6.1x10^{-7}$
						-2	71	7.9	$2.3x10^{-10}$
	1.7	4.9	4.4	1.0	4.9	+2	28	1.2	0.15
						0	50	2.2	$0.78x10^{-3}$
						-2	71	3.1	$4.2x10^{-6}$
	2.2	5.8	1.6	1.3	5.9	+2	28	0.4	2.0
						0	50	0.8	0.043
						-2	71	1.1	$9.5x10^{-4}$
	3.4	7.0	0	1.9	7.0	+2	28	0	19
						0	50	0	1.3
						-2	71	0	$9.4x10^{-2}$
May	1.1	3.8	7.4	0.7	3.9	+2	28	2.1	$1.6x10^{-3}$
						0	50	3.7	$6.1x10^{-7}$
						-2	71	5.3	$2.3x10^{-10}$
	1.7	4.9	1.5	1.0	4.9	+2	28	0.4	0.15
						0	50	0.8	$0.78x10^{-3}$
						-2	71	1.1	$4.2x10^{-6}$
	2.2	5.8	0.2	1.3	5.9	+2	28	0.06	2.0
						0	50	0.1	0.043
						-2	71	0.14	$9.5x10^{-4}$
	3.4	7.0	0	1.9	7.0	+2	28	0	19
						0	50	0	1.3
						-2	71	0	$9.4x10^{-2}$

Wave overtopping at Gap 2

	Offshore wave conditions			Wave conditions near the gap		Water level (m MSL)	Time exceeded (%)	Time wave height and water level is exceeded (%)	Overtopping (l/m/s)
	Hs (m)	Tm (s)	Time exceeded (%)	Hs (m)	Tm (s)				
April	1.1	3.8	11.1	0.5	3.3	+2	28	3.1	8.8×10^{-6}
						0	50	5.6	1.5×10^{-10}
						-2	71	7.9	2.4×10^{-15}
	1.7	4.9	4.4	0.75	4.2	+2	28	1.2	0.62×10^{-2}
						0	50	2.2	5.3×10^{-6}
						-2	71	3.1	4.6×10^{-9}
	2.2	5.8	1.6	1.0	5.1	+2	28	0.4	0.21
						0	50	0.8	1.4×10^{-3}
						-2	71	1.1	8.9×10^{-6}
	3.4	7.0	0	1.4	5.8	+2	28	0	2.3
						0	50	0	0.05
						-2	71	0	1.3×10^{-3}
May	1.1	3.8	7.4	0.5	3.3	+2	28	2.1	8.8×10^{-6}
						0	50	3.7	1.5×10^{-10}
						-2	71	5.3	2.4×10^{-15}
	1.7	4.9	1.5	0.75	4.2	+2	28	0.4	0.62×10^{-2}
						0	50	0.8	5.3×10^{-6}
						-2	71	1.1	4.6×10^{-9}
	2.2	5.8	0.2	1.0	5.1	+2	28	0.06	0.21
						0	50	0.1	1.4×10^{-3}
						-2	71	0.14	8.9×10^{-6}
	3.4	7.0	0	1.4	5.8	+2	28	0	2.3
						0	50	0	0.05
						-2	71	0	1.3×10^{-3}

Based on above Tables and considering the wave overtopping threshold of 0.01 – 0.1 l/m/s, the following probability for unsafe working conditions on the temporary dam have been estimated:

April	Gap 1	Approx. 2% of the time
	Gap 2	Approx. 1% of the time
May	Gap 1	Approx. 0.5% of the time
	Gap 2	Approx. 0.5% of the time

Based on above Table it is concluded that wave conditions in May are slightly better than in April with respect to wave overtopping over the temporary dam and associated safety of operations. In view of the small probabilities for unsafe working conditions due to wave overtopping over the temporary dam during both April and May, however wave overtopping is not assessed to be a significant issue that would require a late closure in May.

8. Internal stability of sea-dike after final closure

HR Wallingford were asked to comment on several different aspects of the design and construction of the sea dike after the central core closure bund had been completed.

The aspects on which comments were sought were as follows:

1. The design and construction of the filter on the landward side of the closure bund between the bund and the sand fill. The design for this filter currently comprises of the following sequence:
   a. 200mm stone
   b. 14 to 200mm stone
   c. 1.5 to 76mm stone
   d. Filter mat; O_{90} size of filter mat not known

2. The risk that flow through the closure bund will create hydraulic pressures on the filter mat and make it very difficult to place and retain the filter mat or cause other difficulties with the placing of the filter materials

3. Whether there would be losses of the fine sand dredged material into the closure bund above level -4.0m MSL after placement

4. Whether there would be excessive losses of fine sand into the bottom protection layer

5. Whether there was likely to be a problem of piping through the placed dredged sand due to differential water pressures being transmitted through the permeable bed protection layer.

Aspects 4 and 5 are the most critical and are therefore addressed first in Section 8.1 Aspects 1 to 3 are subsequently examined in Section 8.2.

8.1 ACCOMMODATING BED PROTECTION LAYER WITHIN THE FINAL DESIGN

There is some concern about settlement arising from sand only migrating slowly into the bed protection layer but the main concern is that the bed protection layer provides a direct water path to allow water pressures to build up underneath the body of sand and generate piping routes through the sand creating ongoing damage to the permanent structure of the dike.

The **settlement risk** is one which is not considered to be serious if the piping problem can be resolved. If there is no route for sand to be lost by piping then suffusion of the sand into the bed protection layer will probably happen during the first few weeks of construction, any resulting settlement can be accommodated, therefore, within the construction process and before the permanent surface protection works are carried out.

Assessment of whether there is a significant **piping problem** is difficult. However, piping normally only takes place along an interface between a permeable and impermeable layer where the locally high hydraulic gradients exceed the capacity of the soil to resist motion of the particles.

the results of the RRI geotechnical investigation of the Closure at Gap no 3 in Dike 4 (described to HR Wallingford during their visit in October 2005) are informative on the issue of piping and flows. The investigation measured pressure fluctuations due to the external tide in both the bed protection layer and in the sand body above it. Measurements were taken at three positions. They indicated that the pressure fluctuations from the external tide reduced on moving landwards into the dike body from the closure bund area. By the position equivalent to the landward side of the proposed new roadway, the pressure fluctuations had become small and the net pressure head over a height of distance of 3.6 metres was less than 1 metre.

Whether or not the bed protection layer extends all the way through the dike, these results are informative and encouraging. They suggest that **liquefaction** will not occur within the sand body, as this requires the effective stress, $\sigma' = \sigma - u$ to be reduced to zero (where σ is the total submerged weight of the sand overburden and u is the applied pore water pressure from the bed protection layer).

However, the bed protection layer has recently been extended in width by 40 metres either side of the centre line and we are recommending (see Chapter 4) that it should be extended by a further 50 metres. As a result the bed protection layer will generally lie outside the footprint of the dike itself and there will thus be a direct water path through the dike. As a result, there will be a large flow of water through the bed protection layer under the sand body. This has the significant consequence that there may be considerable leakage. Using formulae developed by Escarameia and reported by Martins (1990), we have estimated that the velocity of the water through the bed protection layer might be of the order of 0.3 m/s. If there were no sand suffusion into the bed protection layer this would imply discharges of the order of $1 m^3/s$ per metre run of dike. In reality of course sand will have suffused into the layer but if full depth penetration of the sand into the bed protection layer has not been achieved, then some leakage will remain.

Solutions to the leakage problem was therefore sought which would not generate a piping or washout problem within the bed protection layer.

The HR Wallingford team did consider removal of part of the bed protection layer on the landward side of the sill and replacing it with sand or some other material. This approach was eventually rejected because the construction operation of removing this part of the bed protection layer and replacing it with sand or some other material would be extremely difficult (especially because of the presence of gabions) and hence very costly. It is conceivable to think that a long arm powerful hydraulic backhoe dredger could remove at least some of the bed protection layer over. However, in order to remove sufficient of the rocks and gabions to the full depth of perhaps 3-4 metres, it would be necessary to excavate a longitudinal trench some 10m wide, running alongside the back toe of the sill. The difficulty working at such depths would be to be sure that the full depth of the layer had been removed. It would then be necessary to pump some kind of sealant, such as asphaltic grout, into the trench.

Based on previous experience in the Netherlands with dike closures, it is instead recommended to adopt the much simpler solution of washing as much fine material (gravel and sand) as possible into the rock layers to reduce the permeability of the rock layers and thus their effectiveness in transmitting hydraulic pressures. This can be achieved by the a sequence of operations illustrated in Figures 8.1 to 8.5.

1. Figure 8.1 represents the dike immediately after final closure. The first step is to wash into the closure bund itself as much coarse gravel as possible over the are indicated in Figure 8.2. This material would need to be less than 80mm in size and for this purpose the already prepared 1.5 to 76mm stone would be ideal. It is difficult to assess the depth of penetration of this material, but we think it will be of the order of 1 to 2 metres. Hence the dosing rate will need to be of the order of 1 to 2 tonnes of gravel per square metre of surface area. (Note that the vertical dosing rate will need to be increased to allow for any slop of the rock surface.) It is important that the gravel is placed on the landward side of the closure bund on the "ebb tide"[2] and placed on the seaward side on the flood tide. Placing of the gravel should be carried out accurately using a fall pipe.
2. The next step is to place and wash the 0.1mm Saemangeum embankment sand material over the closure bund and over the majority of the bed protection areas as indicated in Figure 8.3. It is important to place this sand on the seaward side of the closure bund as well as the landward in order to ensure that as much sand as possible gets drawn into the bed protection layer. As with the gravel, it is important that the gravel is placed on the landward side of the closure bund on the "ebb tide"[2] and placed on the seaward side on the flood tide.
3. Finally the modified filters are placed as indicated in Figure 8.4 before constructing the remainder of the dike (Figure 8.5)

[2] Please note that by "placing during the ebb tide" we mean "during the time at which the water level on the landward side of the dike is higher than that on the seaward side." Similarly, by "placing on the flood tide" we mean "during the time at which the water level on the landward side of the dike is lower than that on the seaward." Note that Figures 3.1 and 3.2 show that the "ebb" and "flood" are delayed by a kind of 'phase lag' in relation to the time of the natural ebb and flood of the external tide. The actual phase lag associated with the gaps closed and just the Garyeok and Sinsi sluices being open would have to be established and taken into account in the timing of the placement of the sand.

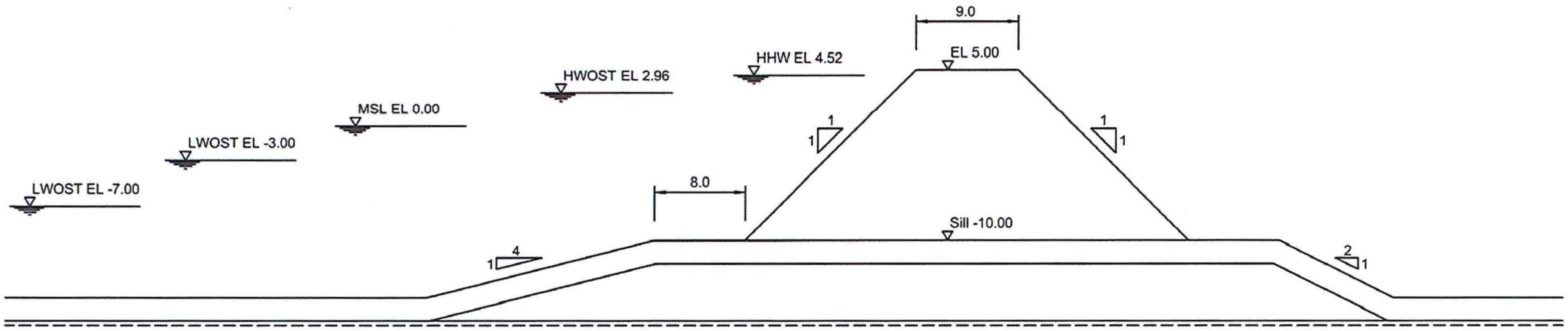

Figure 8.1 Typical Section through bund immediately after final closure (Gap1)

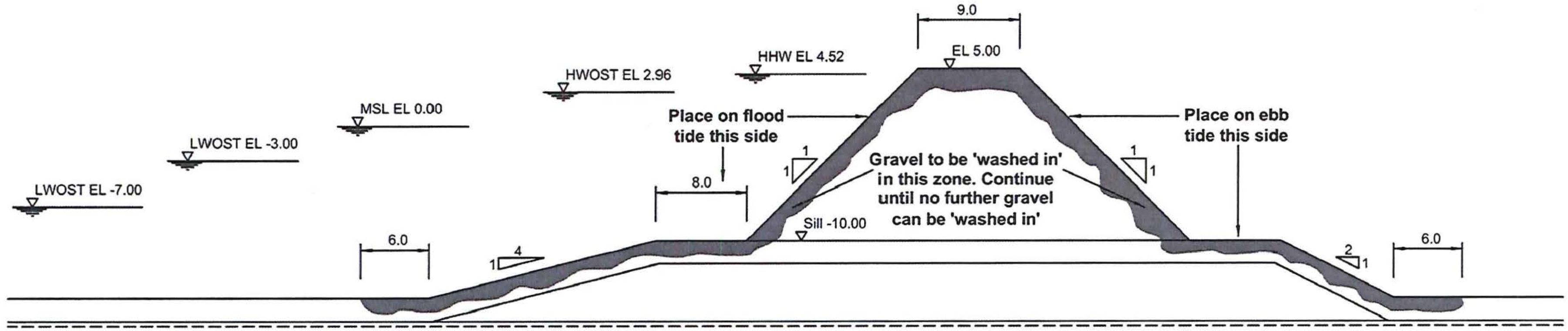

Figure 8.2 Location of placement of 1.5-76 mm gravel for washing into already placed rock and gabion

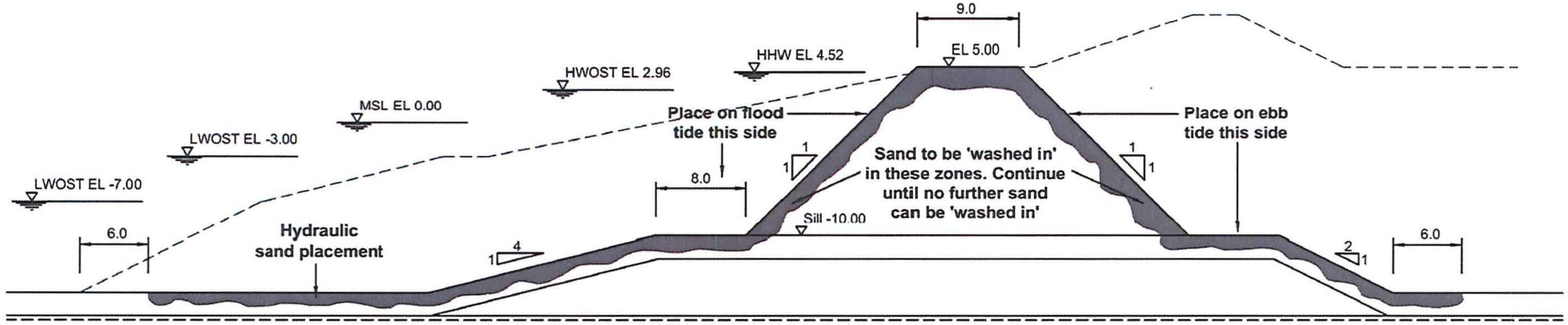

Figure 8.3 Location of sand placement after gravel placement complete

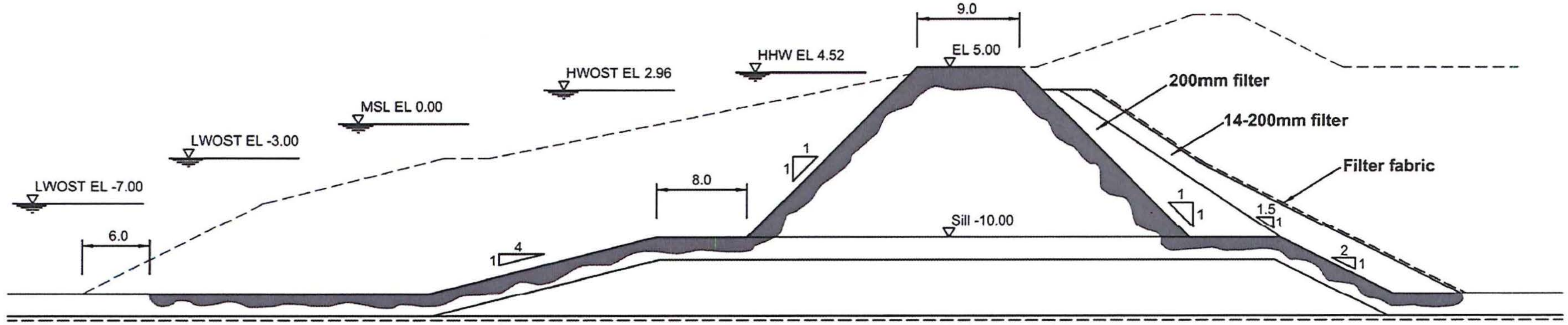

Figure 8.4 Modified filter design after washing in gravel and sand (Filter only retained to avoid unnecessary construction contract changes)

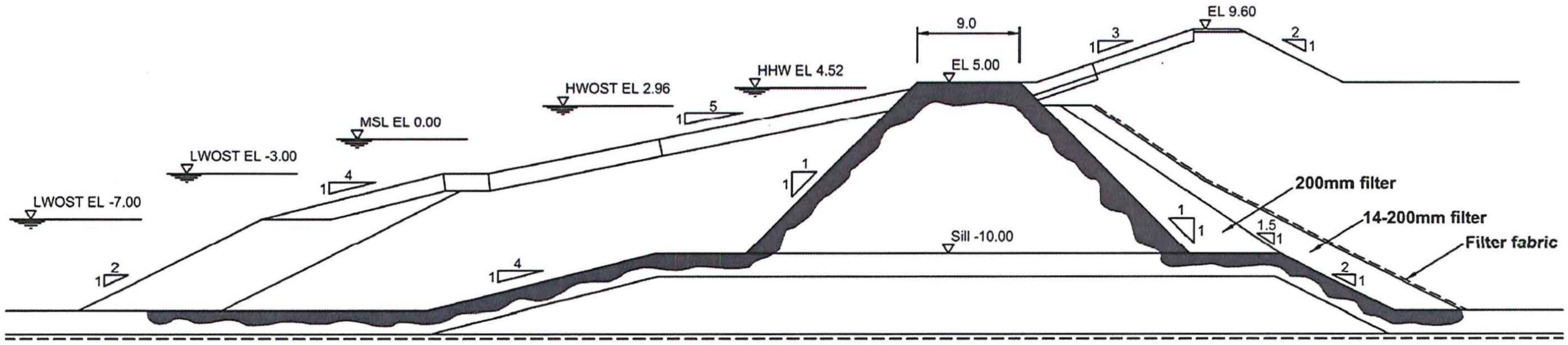

Figure 8.5 Final Modified Cross-section of dike at Gap 1

Nieuwendijk (1971) describes this type of approach in connection with the closure of the 'Brouwershavensche Gat', one of the major closures of the Delta Project. One of the gaps was closed by means of a dam of concrete cubes (Dn = 1 m) dumped from an overhead cable way. The porosity of this dam was some 43 %. The sealing of this dam is described as follows:

> "*After completion of the northern part of the closure dam a start was made with the sealing of the pores between the concrete cubes using gravel (30 to 100 mm). This material was distributed by two cranes, equipped with conveyor belts, always at the side where it would be washed between the cubes by the current. The gravel was dredged from under water stock piles in the working harbour and transported to and under the cranes with inland navigation craft. This method, which had never been practised before, turned out to be very successful: The flow through the dam sealed with gravel had been reduced sufficiently to place sand as hydraulic fill against the dam.*"

The risk of the sand and gravel being washed out of the bund by wave action has been assessed. On the front face wave action will only serve to wash the gravel and sand further into the bund, a beneficial process. On the rear face wash out is unlikely to occur. Estimates of the water particle velocities within the mound suggest that wave action able to cause motion of the sand will not penetrate further than about 20 metres horizontally into the completed dike and hence sand and gravel on the back face of the closure bund will be relatively immune to motion due to this cause.

However, the approach may have some remaining drawbacks and these should be recognised and managed.

1. The progressive washing of gravel and sand into the rock layers means that there is likely to be some settlement of the sand body. This lowering of the sand body can be repaired if necessary. However, the greatest settlement is likely to take place at the inner side of the dam, and not near the central part. As a result damage to the road will be limited. However, it is suggested to start with final road construction relatively late, and use in the first year only a temporary road over the sand body. RRI have geotechnical investigations underway at this time to assess the magnitude of the settlement

2. After completion of construction of the whole dike cross-section, it may be that leakage rates are still unacceptably high. These high leakage rates could cause unacceptable loss of the sand which has suffused into the bed protection layer and consequent subsidence of the sand body in places. The leakage rates should be reduced to avoid this risk. Dutch experience suggests that the leakage can be decreased by washing further fine dredged sand or other fine silty material into the seaward side of the dike, placing this material during the "flood" tide. This will increase the hydraulic resistance of the whole structure and in this way decrease the total groundwater flow.

8.2 DESIGN OF FILTER BETWEEN CLOSURE BUND AND SAND BODY OF DIKE

8.2.1 Conventional filter design

The filter is presently designed by KARICO as a conventional graded filter, but including a geotextile filter mat placed on the fine side of the filter. In general terms, because a filter mat operates in a stand-alone way in delivering its filtration function, the 200mm, 14-200mm and 1.5-76mm layers are not required. Therefore, assuming construction is carried out correctly ensuring good overlaps of the facbic and avoiding damage to the fabric, it would not be necessary to comply with conventional (Terzaghi-like) filter rules.

However, in practice some of the other layers would still be needed to allow for the temporary conditions and for construction reasons.

If the washing of sand and gravel into the closure bund described in Section 8.1 of this report were not to be carried out, then:

- The 200m stone layer would be required:
  - as a first regulating course to smooth out the very uneven face of the closure bund, with its mixture of gabions and rock placed during the final closure process; and
  - to reduce the flow rate through the closure bund. Approximate calculations carried out using a formula for turbulent seepage flow through rockfill structures reported by Martins (1990) suggest that the flow rate through the closure bund after final closure will be of the order of 0.7 m/s. Addition of the 200mm layer will further reduce this flow velocity to perhaps of the order of 0.3 to 0.4 m/s, as a significant part of the hydraulic head difference is placed onto the filter layer.

- The 14 to 200mm layer would be required:
  - as a second regulating course to smooth further the surface prior to placing of the filter mat; and
  - to reduce further the flow rate through the closure bund. Approximate calculations carried out using the formula reported by Martins (1990) suggest that addition of the 14 to 200mm layer will further reduce flow rates, perhaps by a further 50%.

Having placed these two filters, the filter mat could be placed. There would however be a pressure from the water flow. This pressure could be resisted by placing material on top of the filter mat in one of the ways examined by the Saemangeum project office. Of those suggested, the use of geotubes would seem to be the most straightforward way of providing sufficient weight to avoid the filter mat lifting off the surface of the underlying filter. As the geotubes would only have a temporary function, their long term durability would not be of concern.

8.2.2 Natural filter alternative

If our recommendation to wash gravel and sand into the closure bund described in Section 8.1 and Figures 8.1 to 8.5 is adopted, a conventionally-designed filter at the inner side of the rock dam is no longer considered necessary. The alternative approach of a natural filter is therefore sufficient and all three filter layers (200mm , 14-200mm and 1.5 to76mm) and the filter fabric can be omitted. As explained in the previous section wave action will have reduced to such an extent by the time it reaches the main sand body that the risk of sand washout is very small.

However, for practical reasons in Figure 8.4 and 8.5 we have retained the 200mm and 14 to 200mm material and the filter fabric in our sectional drawings, purely because the contract to place these materials is already in place. If the contract can be varied or these materials used in other ways, they can be omitted entirely. In Figures 8.4 and 8.5, the 1.5 to 76mm filter layer has been omitted, because this material is being used for the alternative purpose of washing into the closure bund (Figure 8.2.)

9. Conclusions

1. Much of the work that has been carried out by KARICO and RRI is of excellent quality and only deserves some small comments. However, there are a small number of issues that do require serious attention.

2. Numerical model results were not available to provide velocities for all days of the closure operation. An interpolation routine was used to derive the missing values for the purposes of estimating stable stone sizes. This interpolation routine appears to be satisfactory, but has been difficult to estimate the velocities in the final few days of the closure operation (final days of Phase3). This is because the interpolation routine becomes an extrapolation process for these few days and because changes in flow distribution will occur as the flow area in the gaps reduces to below the flow area of the Garyeok and Sinsi sluices.

3. Scour either side of the existing bed protection will remain a problem and will become worse as velocities increase during the final phases of closure. We have considered the processes taking place and recommend that the bed protection be extended by a further 50 metres either side of the dike centre-line

4. When estimating stable stone weights, the increases from estimated mid gap velocities to peak velocity, for example at the progressing ends of the closure bunds, has not been taken into account. We have applied appropriate speed up factors varying between 5% to 14% to allow for this, but the presence of flow asymmetry means that these increases may be exceeded. We have also allowed for high turbulence, which will be particularly evident in the vortex streets emanating from the ends of the dikes.

5. We make recommendations for increases to the stone weights and/or proportions of gabions to take account of these larger velocities. These changes are significant, requiring more heavy stone (up to 6t in weight) and higher proportions of gabions. In some cases modifications to the existing sill and bed protection will be necessary. Making appropriate modifications will require serious attention by KARICO in the following respects:

   i. To ensure that appropriate stability criteria have been adopted for all materials to be used. RRI have carried out very useful physical modelling, but not all material weights and combinations of gabions for bed protection, sill and closure bund were covered by this work. We have attempted to fill the gaps in understanding by the use of published stability formulae, but further physical modelling to confirm our results would be advisable.

   ii. To ensure that the financial and physical resources necessary to support these design and construction changes are put in place.

6. On balance, we prefer the April-May closure period over the March-April period. This is for two reasons:

   a. the flow velocities are generally slightly lower in the gaps, and, although the differences are not usually enough to affect the choice of stone size, the lower velocities give a greater margin of safety.

   b. the probability of wave overtopping reduces from about 1-2% to about 0.5%

c. However, should KARICO have overriding construction or other reasons to adopt the earlier period and can accept these risk increases the earlier period can be used satisfactorily.

7. To the extent that information has been provided to us, procedures for construction appear to be satisfactory

8. The problem of water leakage through the (extended) bed protection layer after final closure has been completed is significant. A strategy involving carefully timed pumping of gravel and sand into the closure bund and bed protection layer is recommended to solve this problem. This approach could also allow the filter on the back of the closure bund to be omitted if desired.

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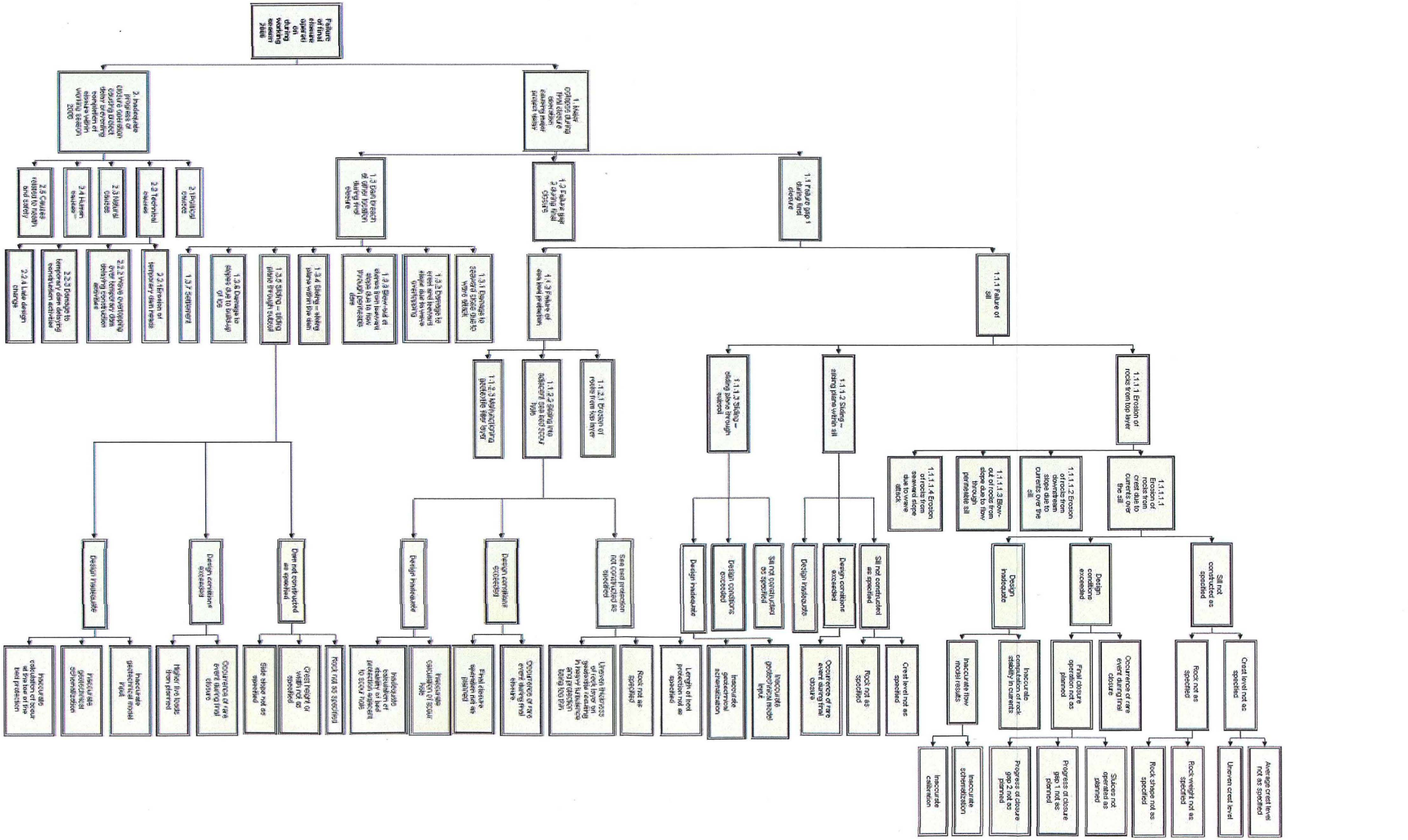
Appendix A



Fault tree of failure of final closure operation during working season 2006