Acceptable Risk Levels in Hydraulic Engineering Projects in the UK

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Abstract
Hydraulic structures providing flood and coastal protection facilitate economic development as well as living areas and often converge with other infrastructure such as railways. There is need for a general guidance for analysing risks of hydraulic structures. This paper analyses three hydraulic engineering projects in the United Kingdom with respect to the issue of accepted risk levels.

Keywords: Flood and coastal protection, acceptable risk, stakeholders, policy

1. Introduction

1.1. Background on paper

This paper is based on the findings made during a secondment to HR Wallingford in which the principal author based his Masters Thesis. The material gained was part of a two year research programme led by HR Wallingford as part of a collaborative project under the UK Department of Environment, Transport and the Regions (DETR) Construction Industry Directorate Partners in Innovation programme. Parts of this paper are also included in the final report produced by HR Wallingford for DETR.

Hydraulic structures providing flood and coastal protection facilitate economic development as well as living areas and often converge with other infrastructure such as railways. There is need for a general guidance for analysing risks of hydraulic structures, so those policy makers, clients and users are more likely to be able to get better value for money by:

- reducing the cost of over-design[1] arising from over-conservatism[1]
- reducing unexpected additional whole life cost arising from under-design[1] or inadequate appreciation of risk
- reducing the risks to health, safety and the environment arising from under-design[1]

[1] It is perhaps noteworthy when discussing issues of weaknesses and redundancy to highlight the fact that many engineers use terms such as "over designed", "under designed" and "conservative" as a shorthand description of the margins for uncertainty from the adoption of particular design approaches or methodologies.

The design of a hydraulic structure generally involves the interaction of three design disciplines, hydraulics, structures and geotechnics. The communication of risk between these three disciplines is not transparent and therefore, the risk of some elements being under-designed[1] compared to some that may be over-designed[1] is something that designers need to consider more carefully. The old analogy of the strength of the chain being that of its weakest link is very useful here, where there is little benefit in building excessive redundancy into one element if another element is likely to fail beforehand. The solution to the problem is to consider more common risk based approaches that can ensure that excessive redundancy is reduced without compromising structural safety levels.

Stakeholders also influence the detailed design of hydraulic structures by forcing the designer to make compromises. In some cases, the relevant people are not contacted in the early stages of the design project resulting in late changes to meet the needs of a particular stakeholder, resulting again generally in an inefficient design or at least a design that does not fulfill all of the projects objectives.

If through more detailed consideration of risks and their acceptability, the engineering community can make even a 5% saving in the whole life costs of fluvial, coastal and port engineering projects then the monetary benefit would be between £50 - £100 million in the UK alone. The resulting benefit to projects spread globally where UK designers are working could be in the order of £250 million. Therefore, an understanding of the risks and how to best manage them is an integral part of producing more efficient designs for clients and the general public alike.

1.2. Objectives

In this paper, three detailed case studies have been investigated as a policy analysis in order to:

- gain an understanding of the decision making process during the design of hydraulic structures
- gain an understanding of the influence that stakeholders have on setting acceptable risk levels of hydraulic structures
- determine a risk-based approach involving all three design disciplines.

1.3. Analysis Methodology

Data for each case study was obtained through interviews with the consultants and clients involved with the individual projects. Further information was then collected using key references for each case study such as feasibility studies and design reports. These reports are mentioned separately to the references, as they are not generally accessible to the public.

Generally, the case study analysis was split into three main sections:

- influence analysis of stakeholders on the final design
- decision aspects regarding setting acceptable risk levels during the design process at each stage
- acceptable risk issues regarding the interaction between design elements.

2. Case study 1

2.1. Background

The site is located on the East Coast of England just north of the Thames Estuary. The area covered by this case study extends for 3.8 km. The coastal defences protect an area with five main attributes against flooding by the sea (DAR, 1998):

- residential properties – there are approximately 2,000 properties at risk of flooding
- commercial interests – established fishing industry which takes place close to the shore
- tourism – as a sea-side resort, tourism is an important economic activity
- environment – the area is a Site of Special Scientific Interest (SSSI), a Special Protection Area (SPA) and a candidate Special Area of Conversation (cSAC).
- heritage – there is evidence of Palaeolithic human occupation at the site.

During a storm event in 1953 the seawall breached resulting in a large area being flooded with the loss of 35 lives and making 600 people homeless.

In the 1980's the situation had become unacceptable and a Fishtail Breakwater Scheme was constructed creating three groyne bays. The layout of structures, however once constructed, did not provide a uniform level of protection along the frontage with pinch points (i.e. reduced beach width and lowering of beach levels in front of the seawall) forming within the groyne bays. The lengths of these pinch points extended over 60% of the frontage, causing the following problems:

- The overall residual life of the structure if a 'do nothing' approach was adopted would be approximately 15 years due to high exposure of the toe of the seawall with reducing clay levels.
- With lower beach levels, overtopping with unacceptable flooding occurs on an annual basis.
- The standard of protection against breaching was assessed to be 1 in 10 years, which means that there was a significant risk of extensive flooding to the area. The consequences of a breach are likely to be very severe, like in 1953.

In 1998, a consultant carried out a feasibility study and recommended the current spacing of the groynes were too wide to sustain a healthy beach. The solution selected, therefore, was to construct additional fishtailed groynes combined with beach replenishment.

2.2. Stakeholders

Through a written consultation process, the designer was able to build up a picture of what was acceptable and start to quantify risk levels where appropriate. Table 1 given below summarises the boundaries of acceptable risk set out by the different stakeholders.

Table 1 Acceptable risk matrix for the Sea Defences

Stakeholder	Unacceptable	Tolerable	Broadly acceptable
English Nature	Any impact on designated areas, especially a local bird breeding area Hard engineering solution over any archaeological SSSI	Do nothing Do nothing on the beach	Improvement to the beach to maintain local bird breeding area Soft engineering solution that has minimal impact on archaeological find
English Heritage	Any damage to the Martello Towers	Reduce risk to Martello Towers	No damage to Martello Towers
Local District Council and the local community	Reduction of beach quality and amenity value area A high risk of extensive flooding	No further deterioration of the beach No reduction in standard of flood defence	Improvement of beach A high standard of flood defence
Crown Estate	Damage to the beach	No damage	Improved management of the beach
Anglian water, EA, DC	Quality bathing water reduced	Do nothing	Quality bathing water improved
Anglian Water	Outfalls damaged by any works	Do nothing	No damage to outfalls
Environment Agency	Standard of defence < 1:200 years	Standard of defence 1:200 years	Stand of defence > 1:200 years
MAFF	Restriction to fishing Benefit cost ratio less than 1 over 50 years	Benefit cost ratio between 1 and 2 over 50 years.	Benefit cost ratio greater than 2 over 50 years

Late on in the design process, English Nature established the value of the ancient river bed system with Palaeolithic remains that would underlie the proposed fishtail groynes in one of the bays. This established value deemed the original design to be unacceptable and an alternative detached breakwater was proposed and accepted.

The Designer then moved onto setting out the scheme objectives, which were set out by discussions with stakeholders. The main design criteria used were:

- Design life
  The design life of the scheme was taken to equal the economic life of the scheme as stated by MAFF (MAFF, 1993), which was 50 years.

- Standard of Protection
  MAFF publish in their Project Appraisal Guidance Notes (MAFF, 1993) a set of indicative standards for flood and coast protection. The Designer assessed that the hinterland was a medium density urban area, which indicates the indicative standard of protection should have a return period of 150 years.

2.3 Risk issues regarding the interaction between different design elements

2.3.1 Failure mechanisms

As the fault tree below (Vrijling, 1981) (figure 1) shows, there are a number failure mechanisms for each section.

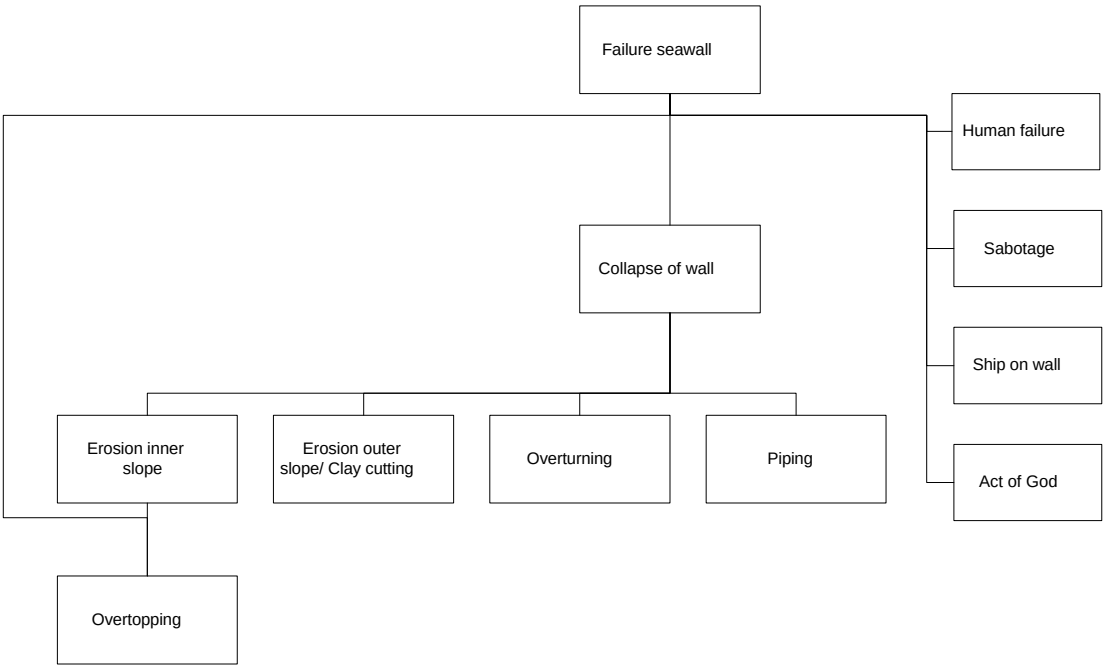

Figure 1 Fault tree.

The solution addresses the two failure mechanisms, of which the risks an order of magnitude higher than the others:

- Excessive overtopping that causes failure of the rear face protection of the seawall (which occurred in 1953).
- Instability at toe of seawall due to exposure of the clay underlayer.

Elements and their interaction

The elements of the sea defence system are:

- **A beach control structure in the form of a groyne or breakwater**
  The beach control structure is meant to reduce the wave induced loading on the beach. It is designed to last over the entire design life without significant maintenance. The risk level of the breakwater failing is supposed to be constant during the design life.

- **A beach**
  The beach is meant to reduce the loading of waves and currents on the clay layer as well as the loading of wave attack on the seawall that causes overtopping. The beach is designed to deteriorate slowly due to the longshore transport capacity of the current, and to be replenished every ten years.

- **A concrete seawall**
  The seawall is the most essential part of the defence system. When the seawall fails, the system fails, or when the seawall works, the system works. Initially, the strength of the seawall is not increased. The probability of the seawall failing varies in time because the strength of the beach that which rules the transfer function of the loading, varies in time.

2.3.2 Systems response on design conditions

To illustrate the interaction between the different design elements we considered the possible response of the system if the design conditions occurred, with respect to the main failure mechanisms of exposure of the clay underlayer and overtopping.

Clay cutting

The large beach renourishment in combination with groynes protects adequately against the failure mechanism of clay cutting, because the clay layer is no longer exposed.

Overtopping due to swell waves

The design conditions given by the consultant and the crest level of the seawall are given in table 2 below. The design loading event selected was the 150 year return period.

Table 2 Design conditions Bay 2 (DAR, 1998)

Return Period (years)	**Design condition**			
	Waves		**Water Level (m OD)**	**Crest level seawall (m OD)**
	H_s (m)	**T_z (s)**		
50	0.9	7.7	3.91	+4.6
100	0.9	7.7	4.09	
150	**0.9**	**7.7**	**4.20**	
200	0.9	7.7	4.28	

Swell waves have a relatively long wave period, this causes them to break later than other waves with a shorter wave period (Allsop et Durand, 1998). If we apply the theory of LeMehaute (LeMehaute, 1976), the water depth on which the waves will start to break is 1.02m.

The surge level with a return period of 150 years is +4.20m OD (DAR, 1998), so if the beach is below +3.18m OD, the waves will not break before they impact on the beach or seawall. If the waves are not broken, they will reach the seawall in tact and reflect. The waves will fully reflect, so the wave height doubles (Schiereck, 2000). The crest level of the reflected waves will be approximately 4.20mOD + H_s = +5.1m OD. This would result in half a meter green water overtopping.

If this event occurred, then structural strength is controlled by the standard of the rear face protection. If the design of the rear face was not considered then the risk level may have been higher than perceived. This example stresses the importance of considering the

interaction of all design elements, such that the rear face protection of the seawall should communicate well with the level of the beach and the position of the breakwater. The final solution selected by the consultant was to construct a beach that was sufficient to break the waves onto the beach rather then strengthen the seawall.

2.4 Case study conclusions

- The risk was quantified in accordance with the procedures as set out by MAFF in their Project Appraisal Guidance Notes. The requirement to get grant aid is a design life of 50 years and the recommended indicative annual probability of failure of 1:150 years for a Medium Density Urban Area turned out to be the optimal probability of failure.

- Environmental pressure has caused late changes in the design. The most remarkable change in the design due to stakeholders interests was the breakwater. The original designed scheme comprised a fishtail breakwater. English Heritage discovered that an ancient river bed system with valuable Palaeolithic remains underlies that designed fishtail breakwater. The design of an offshore breakwater was the result.

- The interaction between the different design elements is indeed very important. The breakwaters control the beach, the beach and the breakwaters control the transfer function of the wave loading on the seawall. The seawall has to be able to resist the design event consisting of a water level accompanied by waves with wave period T_z and wave height $H = H(H_i,T_z,\text{beach},\text{breakwaters})$ with a certain exceedanceprobability. If the beach drops under a certain level and the design conditions would occur, the crest (crest is crest + rear face) of the seawall has to be able to withstand overtopping water of waves with a crest height half a meter higher than the crest level of the seawall.

3. Case study 2

3.1. Background

The next case is the Castle Cove cliff stabilisation on the Isle of Wight.
Castle Cove is situated in the 12 km long unstable landslide complex, the Undercliff.

The problem was that coastal erosion of toe material of the slope increased further instability of the slope. Not only locally, but the stability of the whole Undercliff area is effected, so major failures were to be expected.

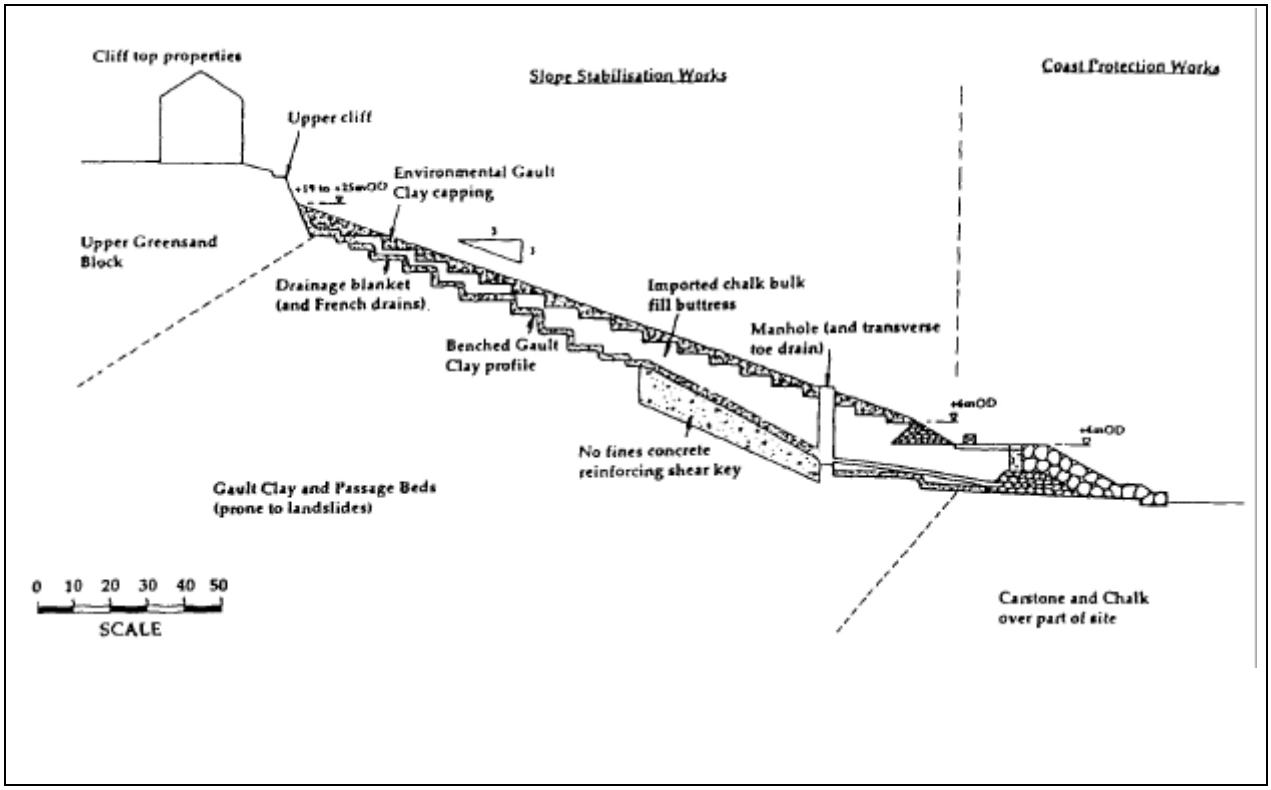

Figure 3 Typical section through slope works (ER, 1994)

Figure 3 is a cross section of the scheme as built now, the slope was unstable but is now stabilised and further coastal erosion is prevented by means of this rock armoured toe revetment.

The solution consist of two elements:

- rock armoured toe revetment (Hydraulic and Structural discipline)
- slope stabilisation (Geotechnical discipline).

3.2. Stakeholders

There were a number of other interested parties with the main ones being summarised in table 3.

Table 3 Main stakeholders and their areas of interest

Stakeholder	Area of interest
South Wight Borough Council	Local coastal authority responsible for coastal protection.
Ministry of Agriculture, Fisheries & Food	Providing grant aid to the scheme. Interested in value for money and protection of the environment.
English Nature	Interested in protecting the rare flora and fauna that exists on the coastal slopes. The site is recognised as being Special Area of Conservation, proposed Site of Special Scientific Interest (SSSI) and an Area of Outstanding Natural Beauty (AONB).
Local community	Interested in boosting the local amenity, the re-opening of coastal paths and improving emergency access to properties.

South Wight Borough Council (SWBC), the coastal authority for the area was the main driver of the project with the focus on protecting properties along the cliff top.

The design procedures as set out by MAFF (MAFF, 1993) were followed in order for the scheme to be rewarded with grant aid, resulting in the following design parameters:

- design life: 50 years
- indicative standard of protection: return period of 100 years.

English Nature as a MAFF statutory consultee and the local authority required that on top of the slope one metre depth of natural Gault clay was placed to preserve original flora and special fauna.

3.3. Design

The design process can be summarised by the following diagram (Figure 4).

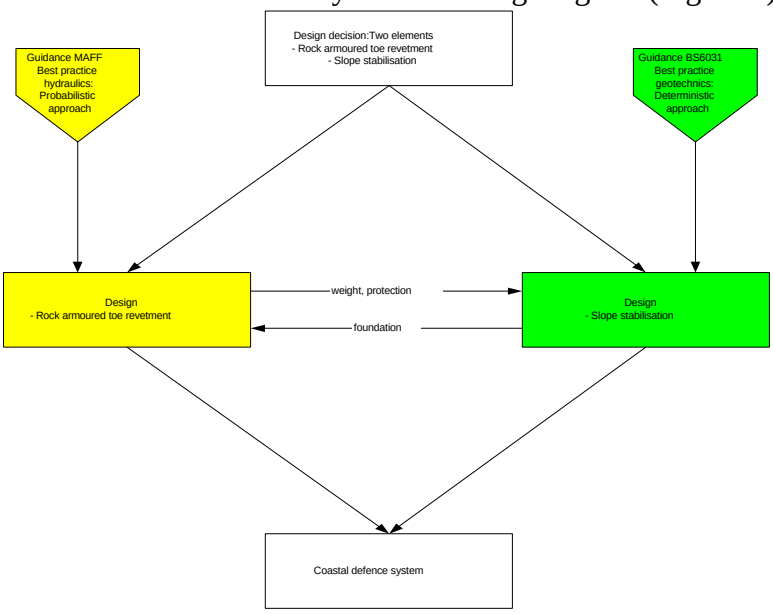

Figure 4 Simplified version of Interaction Diagram

The rock armoured toe revetment was designed according to best practice hydraulics with probabilistic methods. The design event was a storm event with a probability of occurrence of 1 in 100 years.

The slope stabilisation was design according to best practice geotechnics, resulting in a Factor of Safety of 1.3 (BS 6031, 1981).

3.4. Acceptable Risk issues

The system can fail due to failure of rock revetment (sea conditions) and through slope instability (rainfall). The conditions causing these two types of failure can be assumed to be independent.

$$P(\,failure) = P(I) + P(II) - P(I \cap II) \tag{1}$$

with

P(I)	is probability of scheme failing through sea conditions
P(II)	is probability of scheme failing through a heavy rainfall event
P(I and II)	is joint probability of a scheme events I and II occurring together

If we regard the product P(I)P(II) to be negligible then:

$$P(\,failure) = P(I) + P(II) - P(I)P(II) \cong P(I) + P(II) \tag{2}$$

The risk levels of these two elements are expressed in different units though. The risk level of the slope stabilisation has to be expressed in the same unit as the rock armoured toe revetment in order to make a calculation of the risk level of the scheme.

Through discussions with Steve Fort of High Point Rendel and Jim Hall of Bristol University we came to the method of expressing the risk level of the slope in an annual probability of failure (Hall et al, 2000). The data used for this analysis combines historical records of landslides against rainfall specifically for the Isle of Wight.

The annual probability failure of the slope due rainfall occurrence of a certain rainfall intensity is multiplied by the conditional probability of failure of the slope (Lee et al, 1998).

$$P(II) = \sum P(\text{rainfall intensity R}) \times P(II \mid \text{rainfall intensity R}) \tag{3}$$

Table 4 Calculation of PII, without project

4ERA	P(4AER)	P(II | 4AER)
<410	0.8	0.0
410-540	0.16	0.2
540-640	0.03	0.6
>640	0.01	1.0
Annual probability of II: 0.06		

Table 5 Calculation of PII, with project

4ERA	P(4AER)	P(II | 4AER)
<410	0.8	0.0
410-540	0.16	0.0
540-640	0.03	0.0
>640	0.01	0.1
Annual probability of II: 0.001		

The results are:

- without project the annual probability of failure of the slope is 0.06 per year.
- with project the annual probability of failure of the slope is 0.001 per year.

The rock revetment is designed with probabilistic methods to resist a 1:100 year storm event. We will assume that it fails when a bigger storm event occurs. Therefore we take P(I) = 1/100 per year.

If we fill in these values for P(I) and P(II) in equation (2) we find an annual probability of failure.

$$P(\,failure) = 0.01 + 0.001 = 0.011 \tag{4}$$

3.5 Case study conclusions

- The design procedure was followed as set out by the funder, MAFF.

- English Nature with the requirement of the installation of a Gault Clay layer had the most influence on the final outcome of the design.

- The acceptable risk levels of the rock armoured toe revetment (hydraulic discipline) and the slope stabilisation (geotechnical discipline) are expressed in different units. It is worthwhile doing more research on how to express a factor of safety of 1.3 used in geotechnical design into an annual probability of failure used in hydraulic design.

4. Case study 3

4.1. Background

The next case is a seawall, situated on the south coast of England. The wall is owned by a private company responsible for the railway infrastructure in the UK. The wall also provides protection for an urban area. Figure 5 gives a typical cross section of the seawall. The wall was built in 1845, carries two railway lines, which the owner maintains and rents out to that train operating companies. The management and maintenance of the seawall has been an engineering challenge right through its lifetime. Even during the construction phase, considerable problems were experienced in 1846 with a breach in the wall.

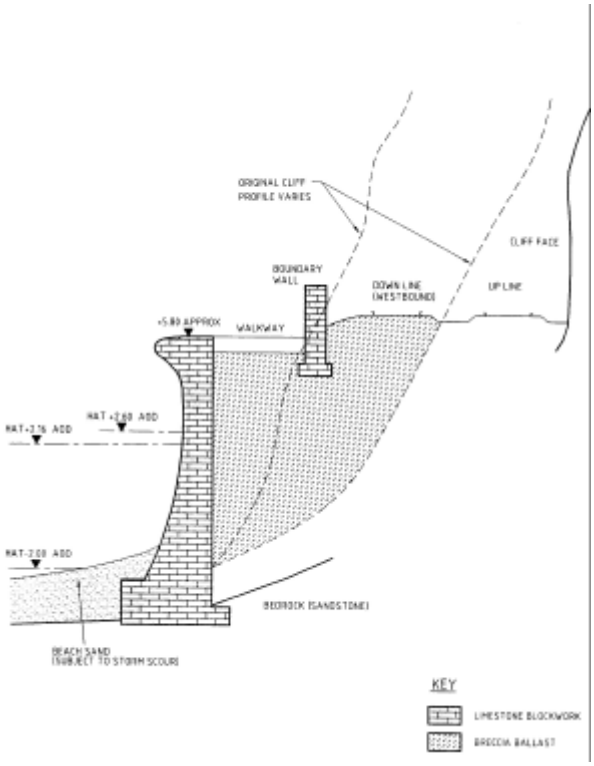

Figure 5 Typical cross section of the seawall (FS, 1998)

In 1997, after a century and a half of reactive repairs, the owner commenced work on providing a large scale concrete toe to sections of the wall perceived to be most at risk.

The problems of the wall are the following:

- reduction in beach levels causes undermining of the wall and fill material being sucked out through holes creating voids in the wall
- overtopping causes the railway line to be taken out of use few times per year.

Downtime is very expensive because of the high penalties due to contractual agreements with train operating companies.

The solutions adopted were:

- detection of voids and grouting them
- construction of a new stepped toe
- new facing works at train station
- masonry repairs to the face of the wall
- concrete spraying at the toe of the wall as an emergency measure.

4.2. Stakeholders

The reactions of the consulted bodies are summarised in terms of boundaries of acceptable risk in table 6.

Table 6 Acceptable risk levels set out by different parties

Party	Unacceptable	Tolerable	Broadly acceptable
Railtrack	NPV/outlay <= 1 No financial benefit	NPV/outlay > 1, Financial benefit to Railtrack	NPV/outlay >> 1 Improved financial benefit to Railtrack
Countryside commission	Damage to coast path	Keep status quo	Enhancement
South Hams Coast & Countryside Service	Damage to coast path	Keep status quo	Enhancement
English Nature	Construction on or damage to SSSI	Keep status quo	Enhancement
DETR	Negative impact on navigation	Mitigation against negative impacts	Keep status quo
MAFF	Negative environmental impact on fisheries.	Would provide grant aid only if a significant negative impact to the national economy.	Keep status quo
Dawlish Town Council	Negative impact on tourism	Keep status quo	Enhancement
Teignbridge District Council	Negative impact on tourism	Keep status quo	Enhancement
Environment Agency	Increase risk of flooding	Keep status quo	Keep status quo
Crown Estate	Negative impact on the beach	Keep status quo	Keep status quo

As a private company, the owner found it difficult to obtain any financial support from other stakeholders. As a consequence, the final design was very much focused on the owner´s needs. If external funding had come forward then the final design may have been significantly different but still meet the objectives of the owner and made improvements to the local area. An example is MAFF, who are responsible for coast protection in England and Wales. Funding is not made direct to individuals or companies as it is felt that they should not benefit from taxpayer money.

The detailed design was based largely on intuitive engineering judgement, rather than detailed modelling or design codes. The designers largely depended on looking at the performance of the materials and structures already in-situ and making a decision based on observation.

4.3. Acceptable risk issues

The coastal defence system comprised the following co-operating elements:

- beach reduces wave attack on wall, and prevents fill material being sucked out from underneath the seawall
- seawall keeps fill material in place and reduces overtopping
- track support, or fill, needs to be protected.

Using recent failure data provided by the consultant, we can make an estimate of the potential risk level. In 1996 there was a breach necessitating closure of the two tracks for

10 days. This covers nearly all the penalty costs and that these costs are in the order of hundreds of thousands of pounds (Sky News, 2000).

Breach in wall comes down mainly to the toe being exposed. The owner's plan is to implement a stepped concrete toe right along the frontage, to be constructed in phases.

An alternative was to raise and maintain beach levels significantly. This would prevent the toe from being exposed and unstable, track support from being sucked out and overtopping.

In this case the consultant chose to maintain the wall, and renovate a little, rather then building a new structure or upgrade the wall. The reasons for this are:

- possible objections stakeholders
- design methods: try and see. The way they design is to try a solution on a section of the wall and then decide, which is not applicable for a beach
- low management structure policy

4.4 Case study conclusions

- The definition of risk involved with a coastal defence set by a private company are tailored to its own needs, although it does need to consider other stakeholders. There is little conflict as long as the status quo is maintained or enhancements are made. Risk is expressed in train delay rather than in terms of assets or human life.

- The private company has sole responsibility for defining the acceptable level of risk.

- If other stakeholders, such as the local authority had been more active and government policy had allowed grant funding to go to a private company. Then the accepted design may have been different (e.g. raise the level of the beach along the frontage).

5. Overall conclusions of paper

1 Stakeholders can have a crucial impact on the design of coastal defence schemes. This influence is best managed by early consultation process. Even when all stakeholders are involved in the decision and design process at an early stage, it is still possible that late changes in the design are required due to any number of reasons.

2 The design procedures including the setting of acceptable risk levels are dictated by the Funder of the project. Decision making in the private sector therefore results in the setting of different acceptable risk levels than in the case of public funded projects. In different countries the acceptable risk levels for use in hydraulic design differ significantly. The inclusion of societal risk by the funder (government) in the setting of acceptable risk levels for use in hydraulic design does have a major impact, the acceptable probability of failure would be much lower.

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Disclaimer

This paper is a contribution to research generally and it would be imprudent for third parties to rely on it in specific applications without first checking its suitability. HR Wallingford and the Technical University of Delft accept no liability for loss or damage suffered by client or third parties.

References

1. Ministry of Agriculture, Fisheries and Food, 1993. Flood and Coastal Defence Project Appraisal Guidance Notes.
2. Vrijling, J.K, 1981. Whitstable central area sea defences. Public inquiry. Witness No.5
3. Allsop N.W.H., Durand N., 1998. Influence of steep seabed slopes on breaking waves for structure design.
4. LeMehaute, 1976. Validity of wave theories.
5. Schiereck, Ir. G.J., Jan 2000. Introduction to Bed, Bank, and Shore protection).
6. British Standard BS 6031:1981. Code of practice for Earthworks.
7. Hall J.W. Lee E.M. Meadowcroft I.C. September 2000. Risk-based benefit assessment of coastal cliff protection. Proceedings of the Institution of Civil Engineers. Water and Maritime Engineering. Volume 142 Issue 3. ISSN 0965 0946.
8. Lee E.M. Moore R. McInnes R., 1998. Assessment of the probability of landslide reactivation: Isle of Wight Undercliff, UK. 8th International IAEG Congress. Balkema, Rotterdam, ISBN 9054109904.
9. Sky News, November 13th 2000. Business News from Sky News.

Reports used

1. Detailed Appraisal Report, Case study , 1998.
2. Engineers Report, Case study 2, 1994.
3. Feasibility Study, Case study 3, 1998.