10 Monitoring, inspection, maintenance and repair

CHAPTER 10 CONTENTS

10 Monitoring, inspection, maintenance and repair

Chapter 10 discusses maintenance and monitoring considerations for all stages of the project life cycle including design.

Key inputs from other chapters

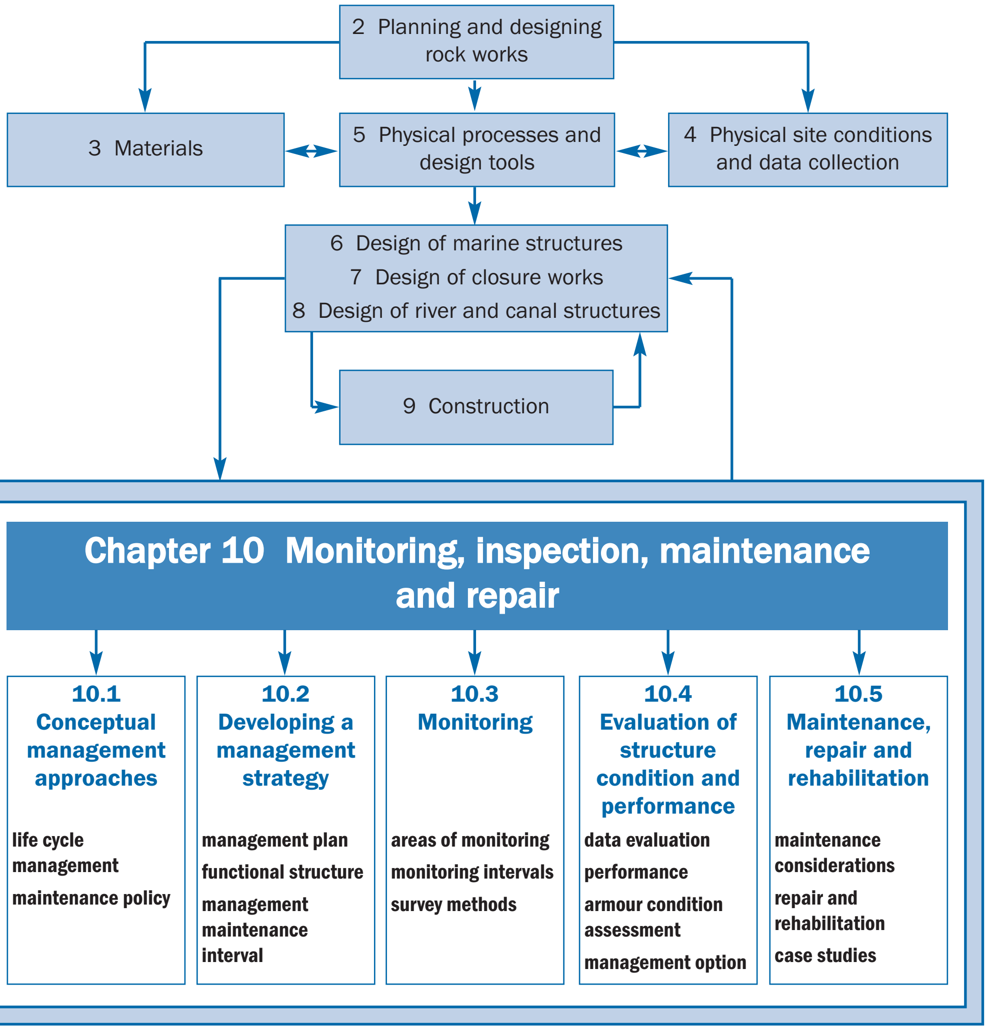
- Chapter 2 ⇨ **project requirements**
- Chapter 3 ⇨ **material properties**
- Chapter 4 ⇨ **physical site conditions**
- Chapter 6, 7 and 8 ⇨ **structure design**
- Chapter 9 ⇨ **construction methods and constraints**

Key outputs to other chapters

- **maintenance considerations** ⇨ Chapters 6, 7 and 8

NOTE: The project process is **iterative**. The reader should **revisit Chapter 2** throughout the project life cycle for a reminder of important issues.

This flow chart shows where to find information in the chapter and how it links to other chapters. Use it in combination with the contents page and the index to navigate the manual.



Substantial elements of this chapter are based on edited original text and practices described in Part VI Chapter 8 of the *Coastal engineering manual* [*CEM*] Engineer Manual 1110-2-1100 USACE (2003).

10.1 CONCEPTUAL MANAGEMENT APPROACHES

Most rock and concrete armoured structures need ongoing maintenance to ensure they continue to perform acceptably. This chapter introduces conceptual management approaches, structure monitoring, evaluation of condition and performance, repair and rehabilitation guidelines, and structure modifications. Guidance related to specific repair and rehabilitation issues is included, but in many cases design guidance for new construction is also applicable to the design of repairs (see Chapters 6, 7 and 8). Intervention may require measures for improving, extending, replacing, repairing and/or maintaining the structure.

The concepts behind structure maintenance are not difficult (USACE, 2003), but it can be difficult to determine:

- the management strategy (see Section 10.2)
- what and how to monitor (see Section 10.3)
- how to evaluate the monitoring data (see Section 10.4)
- how to assess the economic benefits of the possible responses (see Section 10.4)
- whether or not to undertake preventative or corrective action (see Section 10.4)
- how to implement repairs (see Section 10.5).

Although the same conceptual management approach is equally valid for large- and small-scale structures, in both the fluvial and coastal environments, many of the more complex techniques are of greatest relevance to large structures. The techniques discussed are generic and equally applicable to large and small structures, but a sense of proportion and risk should be applied when considering both monitoring and maintenance.

10.1.1 Life cycle management

Chapter 2 discusses the principles of life cycle management, including design methods that allow for high performance and minimal maintenance, and for low capital cost with regular maintenance (see Section 2.4.6). These design methods are implemented through the life cycle management process. This process combines and balances life cycle costs of monitoring (see Section 2.4.1), maintenance and performance within a management framework.

10.1.1.1 Service levels

Service levels are defined at the design stage by setting design conditions for the system, for individual rock structures and for structure elements. The standards may differ for each of these. For example, the structure may be designed to limit overtopping in a 1:200-year return period event, whereas the armour may only be able to withstand a 1:10-year storm without sustaining significant damage, perhaps because only limited armourstone size is available.

Functional performance is generally the most important factor and is measured by variables such as permissible overtopping or sediment control. Low-cost structures that have been designed to below-optimum stability or durability standards will demand intensive monitoring and maintenance if service levels are to be kept up (see Section 2.4). This is often the case in developing countries, where rigorous design standards may not be adopted, where high-quality or adequate-sized materials may not be available, or where suitable construction plant cannot be obtained. Conventional or high-performance designs do not permit damage under the stated design conditions and so may need little maintenance.

Many assumptions are made during the design and construction process. By developing in-service management programmes, the infrastructure manager can decide on the required service level of the complete structure and entire system. Both the initial design conditions and any changes arising from physical, economic and social drivers during the life of the structure are considered at this stage. The financial consequences of management and maintenance for the structure life cycle should also be taken into account.

10.1.1.2 *Reducing performance levels*

The destructive mechanisms of wave, tidal and fluvial action on armoured structures are manifested through changes in the profile of the structure and alterations to the size and shape of its component parts. Failure modes with a low probability of occurrence may cause immediate damage, such as catastrophic failure during a major storm. Other failure modes – such as scour at the toe of river or coastal structures – have a greater likelihood but occur gradually and without immediately affecting the structure's functionality. The armour layers gradually deteriorate as armourstone settles, individual stones are displaced, abraded, fractured or even dissolved. Degradation of armoured structures such as those covered in Chapter 7 is generally similar to degradation of coastal and river structures described below.

Degradation of armoured coastal structures

A review of 265 coastal projects in the United States (Pope, 1992) noted that 77 per cent were more than 50 years old and about 40 per cent of the breakwaters and jetties originated in the 19th century. This means that most of the structures were designed and built before the introduction of even rudimentary design guidance and armour stability criteria. In many cases the structures have survived well beyond their intended service life because they have been well maintained or were over-designed initially. A similar situation undoubtedly applies in most developed countries (USACE, 2003). In developing countries lacking rigorous design methodologies, structure life may be highly variable, with either shorter or (more rarely) longer life cycles.

Major failures by storm action are easily identified. In contrast, gradual degradation often goes undetected, because the structure continues to function as originally intended even in its diminished condition. If left uncorrected, however, this continuing deterioration can lead to partial or complete structure failure (USACE, 2003). Such damage often remains unquantified until major rehabilitation is needed or a significant failure has occurred. Structure ageing may be caused by settlement, scour, solution, loss of slope toe support, partial slope failure, loss of core or backfill material, and/or loss of armour units (USACE, 2003). Unit ageing is defined as deterioration of individual components that could eventually affect the structure's function (Pope, 1992). Examples of unit ageing include breakage of concrete armour units, fracturing or abrasion of armourstone, and concrete spalling. Because structure ageing is a slow process, and the severity of deterioration may be hidden from casual inspection, rehabilitation is often given a low priority if the structure is still functioning at an acceptable level. Neglecting necessary repairs to save money creates the risk that a far more expensive (and possibly urgent) repair will become essential later (USACE, 2003). Any quantitative analysis of the structure must be able to identify these different responses.

Degradation of rock structures in rivers and canals

Revetments and groynes along rivers, and canal bank protection, experience a range of damage mechanisms (Section 8.2.6.1). Where there is sufficient moisture and fine sediment has become trapped between the stones, plants (especially willow) may begin to grow in the armour layer. As the roots and trunks expand, the structure is damaged. During storms old trees may collapse and weaken the armour. Near groyne heads, armour can slide into growing scour holes, as the side slopes steepen. General riverbed degradation can have a similar effect. Ship collision may cause damage. For example on the intensively trafficked

River Waal (Netherlands) a ship collides with, and causes significant damage to, groyne armour once every decade, on average. There is a high incidence of damage at transition points between armour layer and bank, arising from outflanking erosion and undermining. Small armourstone pieces are sometimes moved by recreational users, such as fishermen. Bed protection around hydraulic structures can be degraded by extreme scour during a high flood. Vessels with a small keel clearance can cause damage when they pass over bed protection by generating high return flow velocities or extreme turbulent fluctuations in the screw race. Anchoring above armourstone bed protection often results in local damage.

Armourstone can be damaged when ice is frozen to the armourstone near the water level. As the water drops, large forces are exerted on the armourstone by the weight of the adhered ice. This can cause the armour layer to slide from above to below the water level. Individual stones may also become caught up in large ice flows and drift away. In lake bank protection, an ice sheet can push the stones up the slope of a revetment. In general, these phenomena are not included in the deterministic design process. Ship and wind waves often cause erosion near the water level, where the erosion develops the profile. Specific measures might be required to deal with waves from high-powered recreational craft. Ships that are manoeuvring, meeting or overtaking in narrow canal sections may accidentally collide with the bank protection. In sharp bends and in harbours the screw race of ships can attack the bank, eroding the protection layer.

10.1.2 Maintenance policy

Maintenance activities should be based on a management strategy developed at the design stage for the whole-life performance and costs of the hydraulic structure. The conceptual framework linking the design, maintenance and the risk of failure is the minimal lifetime cost:

$$\text{Minimise } \{I + \mathrm{PV}(M) + \mathrm{PV}(R) + \mathrm{PV}(P_F\, C_F)\} \tag{10.1}$$

where:

I	= investment in the structure	R	= cost of repair or replacement
PV	= present value (see Equation 2.2)	P_F	= probability of failure
M	= cost of monitoring	C_F	= cost involved with failure.

Methods are needed to evaluate the condition of the structure (see Section 10.3) so that the owner can:

- assess damage caused by particular events
- predict the future working life of the structure
- plan maintenance or rehabilitation expenditure.

The extent of monitoring required will be determined by the selected management strategy. Depending on individual structure considerations, the following management strategies may be selected:

- **failure-based maintenance** – repair is undertaken only if the structure or part of it has failed. This type of management is advisable only if the consequences of failure (risk) are very limited
- **periodic maintenance** – assumes that the structural condition deteriorates according to a known function of time. Repair is due after a certain time has elapsed
- **use-based maintenance** – suggests that the structural condition deteriorates as a known function of the number of times the structure is used. Usage has to be monitored and repair is due after a certain number of cycles of structure operation
- **load-based maintenance** – attributes the structural deterioration to heavy loading (eg storms). Loading has to be monitored and repair is due after a certain number of heavy loadings have occurred

- **condition-based maintenance** – depends on the inspection of the physical condition of the structure. If this condition seems no longer adequate, then repair is necessary.

In the above definitions all the strategies except failure-based maintenance are preventative, so monitoring is essential. Therefore the management policy developed should consider:

- access for monitoring and maintenance
- the owner's likely engineering and financial resources for executing practical monitoring
- repair activities and interpretation of structure performance.

From this approach the choice of the management strategy depends on:

- predictability of the structural deterioration
- cost of inspection and monitoring – including the engineering and financial resources required for carrying out both the required monitoring and appraisal
- availability of methods to measure the physical condition accurately
- cost of repair – including mobilisation, plant, labour and materials needed for repair, access for monitoring and maintenance
- consequences of failure (risk) – including safety requirements, damage to infrastructure.

10.2 DEVELOPING A MANAGEMENT STRATEGY

10.2.1 The management plan

A clear management strategy is needed throughout the structure life to maintain functionality and to ensure that the structure or system satisfies the expectations of the end-users in a cost-effective manner. Such management plans should allow for continuous learning, adjustment and refinement (see Figure 10.1). In many countries the designer is responsible for preparing the outline of the management plan, which is passed to the client when construction is complete. This may include aspects of safety, risk, operation and maintenance. Development of a management strategy should be followed at the design/construction stage by production of a management manual. Although this can be very simple, particularly where the structure has been designed to minimise maintenance, it needs to contain basic guidance on techniques and criteria for the main elements of the management plan. It will also set out the interrelationships between the various activities involved.

The management plan should enable the manager to:

- maximise performance of the structure or system at minimal cost
- identify which maintenance and monitoring measures are required and with what purpose
- programme maintenance and monitoring measures
- determine the current service level and condition of all structures within the system
- make adjustments to reflect changes in policy, budgets and priorities
- record knowledge and experiences to prevent loss of information.

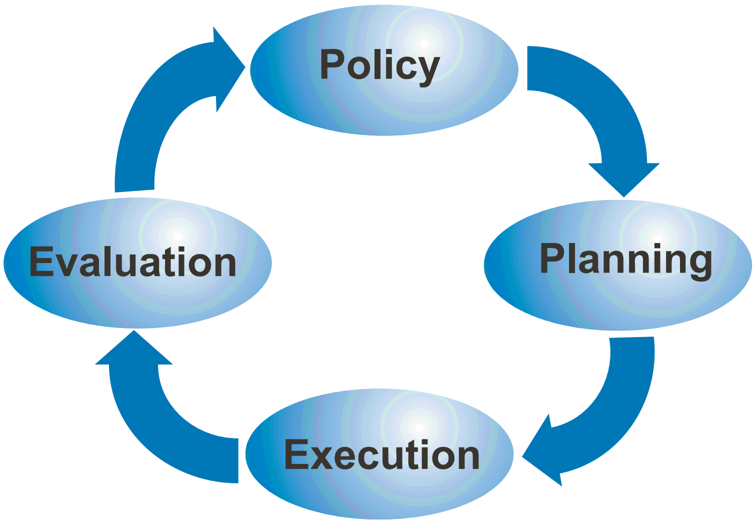


Figure 10.1 *The process of functional management*

Management plans include static and dynamic elements (see Table 10.1).

Table 10.1 *Contents of a management plan*

Static part (typically adjusted every five years)
Introduction
Description of the infrastructure system and its division into managerial units
National policy
Regional policy and formulation of target situations and functional requirements
Management philosophy
Dynamic part (adjusted annually)
Evaluation of the past year: input, output and outcome, measures that were taken and the planned effects that were achieved
Planning for the next five years: measures and costs
Outline long-term planning

The static elements form the strategic management framework. It sets out the general management objectives reflecting the structure or system role and policy requirements that are unlikely to alter frequently. The service level is defined by specifying performance parameter limits for each structure function, eg stability, scour, overtopping. These are usually, but not always, defined at the design stage (see Chapters 6, 7 and 8). The static part of the plan is reviewed periodically – perhaps every five years – to reflect changes in policy, user requirements or unexpected structure or system performance.

The dynamic elements of the management plan provide the outline of operational measures needed to manage the system, including costs. Usually it is adjusted annually on the basis of system performance derived from up-to-date monitoring data, which quantifies system performance relative to the required standard of service. Monitoring identifies whether the maintenance situation is stable, improving or deteriorating.

It is essential to review the management plan periodically to evaluate the performance of the structure and the effectiveness of the management approach; modifications should be made if required. Table 10.2 provides 10 steps of functional structure management that should deliver an efficient management plan.

Table 10.2 *The 10 steps of functional management in a structure management plan*

Functional subdivision of the infrastructure system		**Static part (adjusted five-yearly)**
	1	Describe the infrastructure system
	2	Determine the functions applying to the system
	3	Divide each part of the infrastructure system into managerial units with uniform functions and determine the required service level for each function of the construction
	4	Determine for each function the requirements that will enable it to reach this target level of service
		Dynamic part (adjusted annually)
	5	Compare the present functional condition of the construction with the target condition. If functional demands are not being fulfilled, plan suitable construction, reconstruction or improvement measures to ensure that the structure will meet functional demands
Maintaining the target situation	6	Perform a systems analysis of the structure(s) and determine the critical elements for each function
	7	Formulate parameters for inspection that are representative of the (functional) condition of the critical elements
	8	Determine the intervention level for the critical elements
	9	Determine the maintenance and inspection strategy
	10	Formulate the maintenance and inspection plan including a scheme of costs and a programme of work

A structure management plan should include the following essential elements (Vrijling *et al*, 1995):

- periodic inspection of the structure and monitoring of environmental conditions and structure response
- evaluation of inspection and monitoring data to assess the structure's physical condition (including deterioration) and its performance relative both to the design specifications and to predetermined standards such as service level and planned lifetime. These standards may vary through the lifetime of the structure, for example because of:
  - trends in water level, wave climate, river flows
  - unexpected structural response
  - economic developments
  - unforeseen change in the functions assigned to the structure
  - change in the criteria used to define the acceptable maintenance level
  - economic optimisation of social costs (failure costs and/or costs arising from damage) and the owner or manager of infrastructure cost (both investment costs and also management, maintenance and monitoring costs)
- formulation of an appropriate response based on evaluation results. Possible responses are:
  - take no action (no problems identified or problems are minor)
  - rehabilitate all or part of the structure
  - repair all or part of the structure
  - repair or replace those components of a structure that have a lifetime estimated to be less than the overall structure, or a localised area that has failed evaluation.

Rehabilitation implies corrective action that addresses problems before the structure's functionality is significantly degraded (USACE, 2003). Replacing broken concrete armour units or filling scour holes might be considered structure rehabilitation. Rehabilitation can also be thought of as preventative maintenance.

There are two types of preventative maintenance.

1 **Condition-based maintenance** – rehabilitation based on the observed condition of the structure.

2 **Periodic maintenance** – rehabilitation after a prescribed time period or when a particular loading level is exceeded.

Repair implies that damage has occurred and structure functionality is significantly reduced. Rebuilding a slumped armoured slope, resetting breakwater crown blocks and backfilling eroded fill could be considered structure repair. Repair can also be thought of as corrective maintenance. In many situations it is difficult to distinguish between repair and rehabilitation.

Because of the wide variety of structures and the varied environments in which they are sited it is difficult to develop a generic structure management plan (USACE, 2003). Perhaps the best source of guidance is past experience of maintaining similar structures. As well as repair and rehabilitation, a third response that might arise during maintenance is modification of a structure despite a lack of visible damage or deterioration. Monitoring might reveal that the structure is not performing as expected, or the goals of the project might have changed or expanded, necessitating structural additions or modification. Examples include raising the breakwater crest elevation to reduce overtopping, modifying groyne length to address downdrift erosion problems, and altering structures to control sediment transport or scour.

10.2.2 Optimising the maintenance interval

Conducting maintenance before it is necessary commits the manager to higher costs but provides more security about the structure's performance. Intervening too late will lead to higher costs, because more extensive repairs will be necessary, or damage arising from failure of the system may result in social losses to the end-users. Determination of the optimal time of intervention in the ageing process should be based upon costs of repairs and functionality. The intervention year represents the point in time when the risk of loss of functionality is unacceptable. The intervention level and year (see Figures 10.2 and 10.3) are determined through life cycle costing, which aims to minimise costs over the complete life cycle of the structure, allowing for design, construction, maintenance and removal.

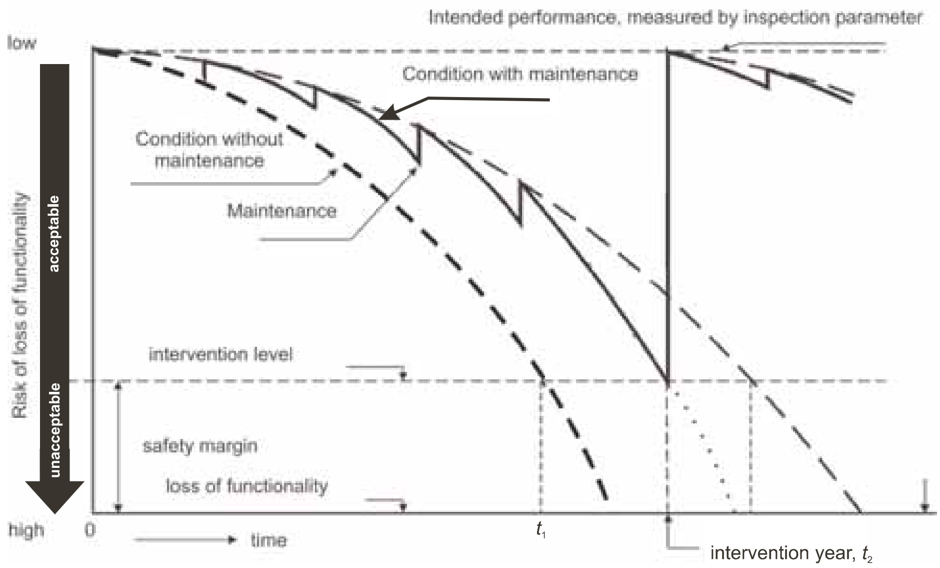


Figure 10.2 *Ageing of a structure*

Figure 10.2 shows the relationship between the ageing of a structure and the risk of loss of functionality. The intervention level should be set above the level of loss of functionality. The chosen safety margin depends on the costs of failure and the rate of loss of quality. Figure 10.3 demonstrates that costs include both maintenance costs and social costs arising from failure. At $t = 0$ the structure is completed and the risk of loss of functionality is very low and therefore acceptable. As it ages, the risk of loss of functionality increases. Maintenance will reduce this risk and allows the year of intervention to be shifted from t_1 to t_2, for example (see Figure 10.2). It may be preferable to take no further measures and to let the structure deteriorate until the intervention level is reached and the risk of loss of functionality becomes unacceptable. The social costs may remain constant over time, but often they vary as a result of developments in infrastructure landward of the structure (see Figure 10.3a). Maintenance costs increase with time, because the structure requires more maintenance as it ages, which drives up the cost of bringing the element back to its original condition. Figure 10.3 shows in schematic form how the intervention year can be determined. In Figure 10.3b the maintenance costs are capitalised (corrected for inflation and indexed) and plotted against the maintenance interval. The capitalised maintenance costs decrease as the maintenance interval increases. The risk of failure increases when the maintenance interval increases. The solid line in Figure 10.3b shows the sum of the capitalised maintenance costs and the risk of failure. The minimum of this sum shows the economic optimum for intervention, ie the intervention year. The matching value of the inspection parameter, ie the intervention level, can now be determined in Figure 10.2.

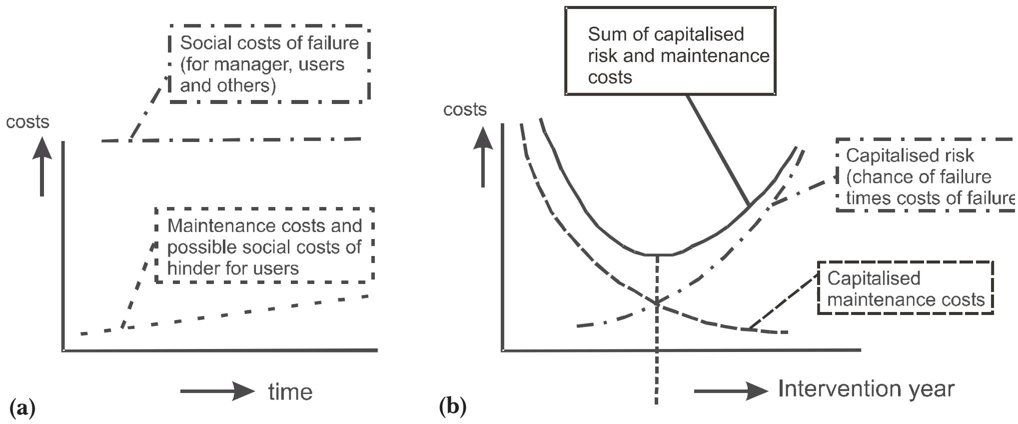


Figure 10.3 *Determination of intervention year*

10.3 MONITORING

10.3.1 Introduction and overview

Project monitoring is an integral part of life cycle management. A regular structural and environmental forcing monitoring programme enables structures to be evaluated for safety, condition and functionality. This process also allows for timely planning of repair and replacement activities and can provide an adequate understanding of failure mechanisms and damage trends. The performance of a structure is assessed by comparing measures of its condition and performance at a number of points in time. Such a monitoring programme should, ideally, be designed at the time of the structure design (see Chapters 6, 7 and 8), but this is often not the case. Techniques used should be repeatable when following a clearly defined specification as well as tolerant of slight operator or procedural variations. Interpretation procedures should enable unambiguous comparison with previous surveys. In the later stages of the structure life it is inevitable that the interpretation will be in the hands of staff unfamiliar with many of the original design assumptions (USACE, 2003).

Major failures arising from storm action are easily identified. Without monitoring, small changes may go undetected and they may ultimately result in the failure of armour layers or in unacceptably large settlements. Quantitative description of the condition of the structure needs to relate to the potential failure modes, focusing on those that have been identified as the most likely (see Section 2.3.1), and should be able to identify these different responses. This requires an understanding of the failure modes and deterioration mechanisms of individual structure components, as well as of the structure as a whole. Monitoring also has to be able to identify the environmental forces driving the responses.

It is equally important to understand the physical signs of impending failure associated with each damage mode. For example, loss of pieces of armourstone from a slope or armour unit breakage may be a precursor to slope failure. The monitoring plan should outline pre-failure symptoms and, if possible, indicate how to quantify the changes. Some identified failure modes may give no warning of impending collapse. In these cases, monitoring will not help. Past experience with similar structures will help in establishing which elements to monitor. The strategy set by the management model (see Section 10.1.2) is used to design the monitoring programme. It presents the following options:

- failure-based monitoring
- periodic monitoring
- use-based monitoring
- load-based monitoring
- condition-based monitoring.

10.3.2 Monitoring plan considerations

Monitoring principally measures:

- functional performance
- structural condition
- environmental loading conditions
- the structure's impact on the local environment.

The monitoring strategies outlined in Section 10.3.1 are used to develop the programme in context with these aims. Similar basic planning guidelines (see Section 10.3.2.1) apply to each approach, but refinements to the programme are reflected in its composition (see Sections 10.3.2.2 and 10.3.2.3) and, more particularly, in the monitoring interval (see Section 10.3.3), which clearly links back to the monitoring strategy (see Section 10.3.1).

10.3.2.1 Guidelines for developing a monitoring programme

When developing a monitoring programme:

- identify monitoring objectives and assess every component suggested for the monitoring programme relative to these. Only include monitoring elements that support the goals
- review the project planning and design information to identify the physical processes that affect the structure. Rank these in order of importance with respect to the monitoring goals. This step is often difficult because of uncertainties about the interaction between structure elements and the environmental loadings
- determine parameters of significance to the physical processes, eg wave heights, flow rates
- determine methods of measurement for each significant parameter. Selection of the appropriate instrument or technique depends on factors such as accuracy, reliability, robustness, expense, availability, and installation or servicing requirements
- plan to gather sufficient structure baseline data to provide the basis for meaningful interpretation of measurements and observations. For example, if the cross-sectional profile of a structure is to be monitored, it is necessary to establish the profile relative to known control points at the start of the monitoring period. The as-built drawings often serve as part of the baseline survey information for structure condition monitoring. It is recommended that as-built drawings based on after-construction surveys be prepared, but in their absence the original design drawings may have to serve as baseline information
- develop a plan to obtain pre-construction bathymetry and measurements of the physical parameters that are likely to be affected by the structure.

10.3.2.2 *Structural condition monitoring*

Structural condition monitoring concerns the condition of the fabric of the structure and its foundations. Condition monitoring provides the information necessary to make an updated evaluation of the structure integrity, either periodically or after extreme events, so that the appropriate maintenance action can be carried out. The complexity and scope of monitoring can vary widely. Structure condition monitoring always involves visual inspection and in some cases includes measurements to evaluate the current structure condition relative to the baseline condition. Changes can occur frequently during construction and in the first year or two after a project is completed. During this period, there can be dynamic adjustments such as structure settlement, armour units nesting and bathymetry change. After initial structure adjustment most significant changes occur during storm events. The monitoring plan should provide enough flexibility in scheduling to accommodate the irregularity of severe storms.

Details of the measures of structural condition, together with appropriate survey techniques, are given in Tables 10.3 and 10.4. The variable defining the resistance should ideally be measured directly (eg D_{n50}). This is often difficult to achieve and an alternative is chosen, such as the average armourstone size, crest level of the structure or overall geometry. Visual surveys are carried out, with the aid of photographs, to record the overall condition of the structure; they include observation of any obvious stone movements, changes in profile etc. This type of survey is very subjective and is of only limited use in a detailed quantitative evaluation of a structure. However, if fixed reference points can be established on the structure and checked at the time of the survey, visual surveys carried out by experienced personnel can form the basis of a suitable monitoring programme.

Table 10.3 *Measurement of the general condition of a rock structure*

Aspect of structure condition	Survey technique
Level I: Location Two to 10 points on the structure measured in relation to a well-established grid and datum levels. Surplus points are advised to provide redundancy if major movements are possible, without loss of survey quality	Conventional survey techniques (any settlement markers should be installed during construction)
Level II: Geometry Outer surface description, related to Level I survey points	Conventional survey, using profiling techniques. As recommended in Appendix A1 Model construction specification, but with wider-spaced profiles (say 20–30 m) For underwater surveying, bathymetric techniques may give useful information
Level III: Composition Position and attitude of each piece of armourstone, including unstable pieces. Position and size of major voids and exposures of core or underlayer	Armourstone degradation inspection techniques Comparative photography Photogrammetry For underwater surveying, side-scan sonar techniques may be used in co-ordination with diver surveys to identify features on sonar traces
Level IV: Element composition Shape and size of armourstone, including any fractures	Armourstone degradation inspection techniques

Note

Level III and Level IV armourstone degradation inspection techniques are difficult to implement on wide gradings and/or gradings with D_{15} less than about 0.3 m. However, the techniques work extremely well on large narrow graded armourstone as specified in Section 3.4.3.

Table 10.4 *Measurements of the structural condition of armoured structures*

Aspects of structure condition measured	Output from comparison of structure condition at a number of points in time
Visual	Settlement of foundation Change in alignment
Geometry, profiles	Consolidation of structure Comparing slope profiles allows overall armour layer damage parameter (S_d) to be determined Scour damage
Profiles plus details	Loss or movement of armour stones Overall sliding of armour layers, if this has occurred Voids requiring emergency planned repair
Profiles, details and special variables	Rounding of stones and loss of material, enabling revised evaluation of D_{n50}; with the design wave climate, or measured wave climate, or revised design wave climate from wave measurements, allows re-evaluation of armour layer stability parameter $H_s/(\Delta D_{n50})$, using equations presented in Section 5.2.2 Comparison with design and measured damage parameters, S_d, is also possible

Chapter 2 summarises a wide range of failure modes for structures in both fluvial and coastal environments. Design methods to prevent these failure modes are discussed in Chapters 5, 6, 7 and 8. Failure modes vary both according to the type of environment and structure, and with the role of the structure. The relative significance of each failure mode will be unique to each structure type. Different damage indicators and associated damage limits, used for evaluation of structure damage, will be relevant for each combination of failure modes and structure type. Table 10.5 provides indicative guidance on the range of failure modes and damage indicators and includes cross-references to sections of the manual that give design guidance for evaluating each failure mode.

Table 10.5 *Failure modes and damage indicators*

	Failure mode	Damage indicator	Typical damage limit allowed before failure	Manual section
	Breakwater and revetment armour layer damage	Eroded area, A_e, and damage level, $S_d = A_e/D_{n50}^2$	S_d = 8–17 (slope angle-dependent)	5.2.2.2
	Berm breakwater profile change or recession of berm	Berm recession and profile change (mobility), N_s	N_s = 1.5–2.7 for static structures Reshaping occurs for N_s > 2.7	5.2.2.1, 5.2.2.6
	Concrete armour unit breakage (double layer)	Percentage of broken units	2–15 per cent (armour unit-dependent)	5.2.2.3
	Concrete armour unit breakage (double layer)	Stability number, N_s	3–4 (armour unit-dependent)	5.2.2.3
	Ice sheet damage	Crushing load Bending load Rubbling load	See Figure 5.112 Equation 5.242 Equation 5.243	5.2.4
	Ice interaction with slopes and breakwaters	Edge failure Global active failure Total sliding failure	Equation 5.245 See Figure 5.116 Equation 5.246	5.2.4
	Toe flattening	Reduction in slope angle	20–50 per cent	5.2.2.9
	Single-layer concrete armour unit	Percentage of broken units	0–5 per cent	5.2.2.3
	Toe protection to sloping face	Stability number N_s	3–6 (toe depth-dependent)	5.2.2.9
	Dropping of falling apron toes	Cross-sectional change		6.1, 6.3
Geotechnical failure modes	Bank failure by leaching of fines	Armour settlement	Filter-dependent	5.4.3.6
Geotechnical failure modes	Settlement	Bearing capacity Slip surface potential	Bed-rock type dependent	5.4.3.7, 5.4.3.3
Geotechnical failure modes	Differential settlement of breakwaters and revetments	Crest elevation and slope angle	Subsoil-dependent	5.4.3.5, 5.4.3.7
Geotechnical failure modes	Localised settlement of waterway embankment built on soft soils	Crest elevation	Subsoil-dependent	5.4.3.5, 5.4.3.7
Geotechnical failure modes	Closure dams and reservoir dams settlement	Freeboard	Foundation dependent of ground	5.4.3.5, 5.4.3.7
Geotechnical failure modes	Settlement of river and canal structures, including dikes and bank protection	Freeboard	Subsoil-dependent	5.4.3.5, 5.4.3.7
Geotechnical failure modes	Circular slip surface caused by hydraulic loading	Lateral and vertical movements of crest and slope		5.4.3.2
Geotechnical failure modes	Sliding of slope protection along shallow, straight slip-surface	Lateral and vertical movements of crest and slope		5.4.3.2

10.3.2.3 *Structure performance monitoring*

Structure performance or function monitoring consists of observations and measurements to evaluate the structure's performance relative to the design objectives, environmental conditions and expected design performance. Typically, performance monitoring programmes are implemented early in a structure's life, with a short (less than five-year) duration relative to the structure's design life (USACE, 2003). Some performance monitoring plans are one-time, comprehensive post-construction efforts spanning several months of continuous data collection and analyses. Other monitoring plans consist of repetitive data-collection episodes spanning several years, perhaps augmented by continuous recording of environmental parameters such as wave and wind data. For unusual structures or situations where longer data records are needed to reduce uncertainty, the duration may be longer.

Common reasons for monitoring structure performance are given below (USACE, 2003).

To provide a basis for improving the attainment of project goals

The uncertainties in coastal and fluvial engineering design may result in a structure that is not performing as well as originally anticipated. Before corrective actions can be taken, monitoring is needed to determine why the structure's performance is below expectation. For example, if wave action in a harbour exceeds design criteria, it is necessary to determine the incident wave conditions (forcing) and the mechanisms (refraction, diffraction, transmission) that cause unacceptable structure behaviour, such as damage and overtopping.

To verify and improve design procedures

Design guidance is often based on systematic laboratory testing combined with practical experience gained from earlier projects. However, most coastal and fluvial structures are unique in some way – in their exposure to waves and currents, the construction materials available, combined functions, or in existing structure features. Consequently, the generic design guidance may not be entirely applicable for a specific structure. Designs often include many assumptions and the maintenance phase provides an opportunity to validate or refine these assumptions. Performance monitoring will verify whether the design is functioning as intended, and it will also provide data that can be used to improve existing design procedures or extend the design guidance over a wider range of applications. Often baseline data is lacking and models are not calibrated, leading to design uncertainty.

To validate construction and repair methods

Construction techniques for a specific project are influenced by the availability of suitable equipment, contractor experience, environmental exposure, and whether construction is carried out from land-based or floating plant. Limited guidance exists on designing repairs to deteriorated structures (PIANC, 1998, 2004). Engineers' practical experience can be very important. Performance monitoring may be needed to validate the procedures and to spot problems before serious damage can occur in these situations. For example, monitoring might be needed to evaluate the impacts of repairing a rock-armoured rubble mound structure with concrete armour units or providing scour protection to a bridge.

To examine operational and maintenance procedures

Many coastal and fluvial structures need procedures for their post-construction operation, and periodic structure maintenance is usually required. Performance monitoring is useful for evaluating the efficiencies and economy associated with these procedures. For example, if navigation channel maintenance includes placement of beach-quality sand on downdrift beaches, monitoring could be established to determine the best location for sand recharge and to prevent sand re-entering the channel.

10.3.2.4 Environmental monitoring

Environmental monitoring concerns the external loading on the structure and the effect the structure has on the local environment, such as a beach or river bed. Table 10.6 gives details of environmental conditions or loadings, together with appropriate monitoring techniques. The monitoring methods selected should relate to the potential failure modes for the structure in question and, in particular, to those which have been identified as the most likely (see Section 2.3.1).

Table 10.6 *Measurements of environmental conditions or loadings*

Environmental condition or loading	Measurement
Water level	Tide board, visually inspected Data from nearest local tide recording stations Use of surface elevation monitor (step gauge or resistivity gauge) recordings, if available
Wave climate	Seabed pressure meter (robust and cheap) Surface elevation monitor mounted on robust support (eg pile or triangulated scaffold tube arrangement) Wave-rider buoy or similar (will be expensive to maintain for long periods) Hindcasting analysis for storm events using wind records
Wind climate	Standard anemograph device (depending on correlation between wind and wave direction, this may be a useful way of assessing directionality of wave climate)
Wave run-up	Parallel steel wire resistivity gauge (survival is likely to be a problem)
Wave transmission (for breakwaters)	Wave gauge at rear of breakwater
Mound pore pressures	Piezometers installed within mound with automatic recording facility
Bathymetry and beach topography	Below high water, standard bathymetric techniques are possible Above low water, conventional land-survey techniques may be used or photogrammetry from aerial photography Land-based photography of waterline from fixed positions gives useful evaluation of low to high water beachform
Stress in foundation	Pressure pads
Pore pressure in foundation	Piezometers (simple standpipe or, for continuous measurements, vibrating-wire electronic recording devices may be used)

10.3.2.5 Data considerations

Three key considerations generally apply to data: accuracy, quality and quantity.

1 **Data accuracy** evaluates how close the value of a recorded piece of information is to the true value at the time of observation. Data accuracy relates directly to the means of measuring or observing the physical process. As an extreme example, visual estimates of wave height and period are much less accurate than similar estimates obtained using wave gauges.

2 **Data quality** includes site-specific factors as well as other influences such as instrument calibration. High-quality, accurate instrumentation is necessary for quality data. Data quality also requires correct sampling rates of the parameters. For example, sampling waves at a rate of 1 Hz may not adequately resolve short waves.

3 **Data quantity** can influence cost. For some measurements, well-established guidelines exist that detail the necessary data quantity for success. Uncertainty exists for some variables particularly concerning the measurement duration necessary. A realistic evaluation of data quantity will need to balance multiple factors such as cost, importance of the data, instrument reliability and natural variations.

A conceptual model for consideration of appropriate methods to capture and share data between stakeholders (Dyer and Millard, 2002) presents five principles (see Figure 10.4) that can be applied effectively to the management of rock structures.

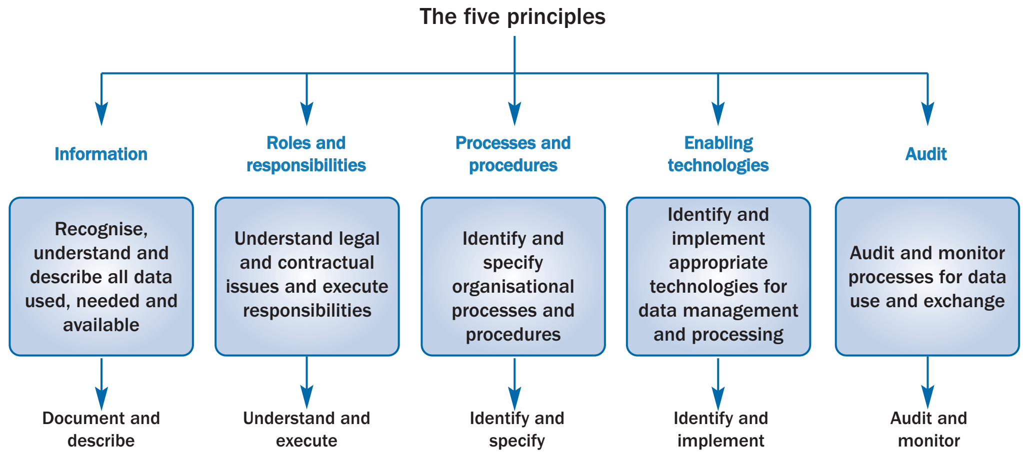


Figure 10.4 *The five principles of data management (Dyer and Millard, 2002)*

10.3.3 Intervals between monitoring

The intervals between monitoring should be predetermined by the risk associated with particular failure mechanisms, structural elements, foundation conditions, exposure conditions and design criteria. This reflects the structure's reducing resistance to failure as it degrades with time and the approaching need to carry out repair (De Quelerij and Van Hijum, 1990). Following initial settlement and packing soon after construction, rock structures generally become more stable. Most changes occur during the major storm periods or events of high river flow. Changes may be minor for some years, unless the structure has been designed to allow for some damage during frequently occurring events. Many types of armour deterioration are gradual (see Section 3.6.2). Monitoring intervals should be appropriate to the rate of degradation and damage arising and fit one of the models given in Section 3.6.5. As the armourstone degrades or environmental conditions become more severe, the rock structure may change more rapidly. This overall behaviour may be surpassed by a very severe storm event.

The timing of monitoring events will vary for different mechanisms. For practical reasons it may be necessary to combine events in order to optimise the number of inspections. Not all surveys will be at the same level of detail: owners tend to make regular brief inspections as well as less frequent but more detailed surveys. Past experience with similar structures should help determine appropriate intervals between repetitive monitoring tasks. If monitoring indicates some aspect of the structure is performing better than anticipated, then future monitoring of that aspect may be made less frequently. The important point is that monitoring plans should allow flexibility in scheduling repetitive monitoring elements to react to evolving circumstances.

Periodic inspection

Inspections are made at regular intervals identified at the design stage. The rational minimum interval, based on the changing of the seasons, is six or 12 months. River and canal banks are typically inspected following winter or wet season periods. Annual surveys ensure that survey staff remain familiar with the structure and maintain continuity of data (USACE, 2003). Intervals of up to several years may be chosen if the deterioration process is mainly a function of time and is well known (eg settlement). Inspection should be planned to occur some time before the condition is predicted to reach a certain minimum value, based on previous performance. Generally, the tasks within a condition-monitoring plan tend to be evenly spaced in time over the structure service life. Some tasks may be more frequent for several years immediately after construction to confirm the structure is reacting as intended. Additional inspections should be made after all major storms, perhaps whenever the storm wave heights have exceeded 75 per cent of the design value or the river flow equivalent. The threshold value should be set in relation to the design conditions and associated return periods and the damage response characteristics of the structure.

Ideally, monitoring of the environmental loading conditions should be continuous. A complete qualitative record should be kept, which should include logs of the weather, tidal levels, river discharges etc.

Unless special circumstances exist, it is recommended that the submerged elements of a rock structure are fully inspected at least every five years and after extreme storm events, periods of high flow or sustained freezing in rivers. In addition, annual monitoring of the upper sections may indicate possible problems on the submerged section of a structure that need further investigation. Instrumentation can also be introduced at the construction stage into structures that are partially permanently submerged, to allow certain performance aspects (eg foundation settlement) to be monitored in parallel with the more frequent inspections of the upper structure.

Use-based inspection

This approach may be appropriate if the deterioration depends mainly on the usage or the loading, the cumulative use or the cumulative loading should form the basis for inspection. Inspection is due after a specified number of events, for example storms (eg revetments: load = flood level + flow or wave height). Scour phenomena in river engineering seem very suitable for a load-based inspection scheme, leading to bathymetric surveys after a number of floods. This method is also suitable for determining monitoring frequency on low-cost structures that have been designed to allow for regular damage and maintenance.

Condition-based monitoring

Monitoring can be instigated by visual surveys, carried out by experienced personnel, to form the basis of a decision to perform a suitable in-depth monitoring programme. An incremental approach may be used to develop the programme on the basis of increasing knowledge of a previously insufficiently understood deterioration process. After the first few years, satisfactory performance may indicate that adequate monitoring will still be achieved if the detailed surveys are less frequent, for example, reducing from every 12 months to every 24 months. The frequency of structure condition monitoring adopted during the life of the structure depends principally on the following factors:

- location of structure
- type of construction
- design risk levels
- exposure conditions
- foundation conditions.

Typical monitoring intervals for a range of inspection types are given in Table 10.7.

Table 10.7 *Typical planned monitoring intervals (based on USACE, 2003)*

Inspection type	Time interval or basis
Location of whole structure (geotechnical-driven change)	Six months visual to 12 months survey
Geometry of the structure	12 months
Position of individual pieces of armourstone	12 months
Shape, size, condition of armourstone	12 months individual stones
Foundations, scour etc	Six months visual to 12 months survey
Submerged diving or multi-beam inspection	Five years or following damage to upper layers
Visual inspection of older structure	Condition-based
After major storm events or extended frozen periods in rivers	Event-based
Visual inspections only when personnel are in the region for other purposes and time permits	Resource-based
Visual inspections only after local users report a problem, eg fallen trees in rivers	Event-based

10.3.4 Surveys of structure sections above the water level

Survey methods considered in this section include conventional topographic surveying techniques and visual evaluations – particularly of the state of the armour layer – made by the surveyor of surface-emergent sections of the structure (those sections that are either temporarily or permanently above water level). These methods all require access on to and over the structure, which may be difficult. Generally, structure condition measurements focus on physical changes of the structure and its foundation. Examples include repeated elevation surveys of selected structure cross-sections to quantify settlement or loss of armourstone pieces, and *in situ* testing of materials undergoing deterioration. Most measurements require baseline data for comparison, and sequential measurements help to assess the rate of change for the monitored structure. Some of the techniques described are suitable only for large structures. Visual and photographic methods are generally the most appropriate techniques for small structures and are particularly useful for river and canal structures.

10.3.4.1 Visual surveys

Visual inspection of structural components above the water level can be accomplished by:

- walking on the structure (with care)
- viewing across a river or canal with binoculars
- viewing from adjacent land
- viewing from a boat or an aircraft.

The effectiveness of visual inspection depends heavily on the scale of the structure, having an understanding of the symptoms of deterioration, and quantification of the changes that have occurred since the previous inspection. Broken armour units and displaced stones are obvious signs of potential trouble. Visual inspections are necessarily subjective, so experience is paramount in recognising likely problems. It is often hard to ensure consistency of evaluation, and detailed notes and photographic records should be kept to act as guidance for, and ensure continuity of, monitoring standards in future evaluations. Thought should be given to the most appropriate recording method. Field notes and rough sketches should be translated and expanded shortly after the inspection.

Structures are hazardous areas on which to operate. Personnel conducting visual inspections should be familiar with working near water and intertidal working where appropriate, and should be well informed on the wave, tidal and flow conditions anticipated during the survey period. Periods of low summer water levels in rivers or spring tides will allow access to lower levels, but in many locations the water level can change rapidly. Access to the armour face is a common problem. The intertidal zone is frequently covered in weed and/or algal growth, and movement over this area requires great care, particularly where there are large, smooth armour units, which may offer large voids into which the surveyor could fall.

A baseline set of information is required for the structure and its environment at the time of construction and during the defects liability period. This information should be stored in a format that allows it to be retrieved in future years for comparison purposes. Construction-phase data are also required as a check against the assumptions and details established at the design stage. Information recorded should include basic geometric survey data of profiles and records of any failures of the rock elements during the construction period. The contractor should document the basic monitoring information and hand it over to the owner or designer by the end of the defects liability (or guarantee) period.

10.3.4.2 Armour layer and armourstone degradation

Simple, inexpensive techniques can be used during the on-site inspection, including:

- counting broken armour units
- spray paint marking of cracks or suspected displacements
- using a tape to measure distances between established points on the structure
- identifying the elevation of selected locations using a level
- repeated photo-documentation from the same vantage point (Pope, 1992).

The condition, and hence performance, of the armour layers depends upon the size, shape and surface texture of the armour units. Visual evaluation methods to identify stones known to be susceptible to degradation have been identified (Allsop *et al*, 1985; Poole *et al*, 1983). The monitoring procedure is intended to identify progressive armour layer damage as given by:

- cavities – defined as a void that could be filled by a piece of armourstone of design size
- fractured armourstone – includes all cases where the stone has broken *in situ*
- sub-size armourstone – all stones smaller than the specified lower limit
- unstable armourstone – stones that move under wave or current action.

In each location, an area containing at least 100 pieces of armourstone is marked out. Each survey area should run from the crest to as low a level as practical. Typically, the survey area should be at least 5 m wide. The survey area should be able to be moved in subsequent years, eg either side of a profile line. The first task is to count the total number of armour stones falling within the survey area. It is convenient to count only those in the uppermost layer. Each example falling into the above categories should be recorded.

The state of interlock of the armour layer may be assessed by two methods.

1. Unstable armour may be defined as an armourstone piece that is under reduced restraint from adjacent units and which can be moved easily by storm waves. Such armourstone is often characterised by edges that have been rounded or abraded by rocking movements. On new structures, initial placement may have allowed the armourstone to move freely. Such a condition is usually temporary, as unstable stones tend either to stabilise or be removed completely from the armour layers by wave action.
2. An alternative parameter is the co-ordination number. This is defined as the average number of stones in contact with each stone in the sample in the same layer. Its assessment is somewhat laborious, and may be subject to variation between surveyors. The co-ordination number is a function of the grading width, D_{85}/D_{15}, and it is probably only practical for narrow-graded armourstone rather than rip-rap. While the data

derived is of value in understanding armour layer integrity, the method is difficult to implement and is not advised for structures that have been in service for some time, where access across the armour layer may be unsafe. For the main damage categories identified above, the number of armour units may be expressed as a proportion or percentage of the total counted.

These evaluation methods have been used at UK sites and the results are summarised in Table 10.8. Progressive damage can be determined to evaluate degradation of the armourlayer layers. Although they are valuable in situations where armour damage is a problem, these techniques are time-consuming and rely on safe access across the structure.

Table 10.8 *Summary of survey data at three UK revetment sites*

Survey location (date)	No of armourstone pieces in sample	Fractures (%)	Cavities (%)	Sub-size (%)	Unstable (%)	Total damage (%)	Co-ordination number
Stornoway (10/85)	693	1.4	1.7	2.2	3.0	8.3	4.6
Stornoway Area 1 (11/89)	100	4.5	3.5	14.0	2.0	24	3.8
Stornoway Area 2 (11/89)	106	1.9	15.1	15.1	3.8	22	4.0
Herne Bay (10/86)	828	0.2	2.2	0.0	1.7	4.1	
Herne Bay (2/87)	860	0.6	2.4	0.0	0.7	3.7	–
Port Talbot (7/83)	2712	1.1	9.4	1.5	0.8	12.8	4.2

10.3.4.3 *Photographic methods*

Photographs provide a cheap, permanent record of a structure's condition at a given time, and so are often used for monitoring purposes. Techniques may include complex photogrammetric methods and/or simple record photographs taken during visual inspections.

Comparative photography

The most used, and basic, photographic survey technique is comparative photography (Boxes 10.1, 10.2 and 10.3). Photographs of the same view are repeated on each survey and the images compared to detect differences. The value and accuracy of such methods is controlled by the location from which the picture is taken, the field of view and the precision with which images are matched on each subsequent survey. Records must therefore be made of the location of the point from which the photograph is taken, the distance and orientation of the object from that point, and the focal length of the lens used. This method is ideal for use on narrow rivers or canals where images of the structure can be taken from the opposite bank.

Box 10.1 *Comparative photography from low-level elevated platform*

It is relatively easy to obtain photographs of a structure that is fully surface-emergent at some stage of the tide or flow cycle. The most useful angle of view of a structure is normal to the slope; this necessitates photography from an elevated platform, which is not straightforward, but has been adopted with some success at Table Bay harbour breakwater (Kluger, 1988). The photographer was suspended from a crane hook in a cradle and the position and orientation of the cradle was controlled by light lines that led diagonally back to the crown wall. Predetermined camera positions can be controlled with position-fixing equipment. The configuration used (by Kluger) to generate stereo photographs is summarised in Table 10.9.

Table 10.9 *Camera set-up for comparative photography*

Camera elevation above MSL	21.5 m
Stereo cover area (each pair)	24 m × 17 m
View angle of camera (both planes)	70°
Separation in stereo photographs	6.0 m
Overlap of stereo pairs	70 per cent
Horizontal distance of camera from wall	7.5 m
Camera distance perpendicular to slope	18 m

Box 10.2 *Comparative photography from model aircraft and balloons*

Low-level aerial surveys (20–30 m elevation) can be conducted from either model aircraft or tethered balloons, which can provide high-resolution images from above the structure that are suitable for analysis of armour integrity. The photographs contain sufficient detail to allow identification of slope irregularities, stone displacements, interlock and stone shape. Locating damage requires registration of the images relative to ground control points (see targets in Figure 10.5). This relatively low-cost technique is ideal for small structures such as revetments or rock groynes. It also has the benefit of being very safe, avoiding the need for surveyors to climb across potentially slippery structures with large voids.

Figure 10.5 *Low-level aerial photography (courtesy New Forest District Council)*

Photography from the water requires a more complex procedure. Kluger (1983) suggests photographing a breakwater in sections from a boat at a distance so that individual armour units can be identified easily (Box 10.3). The whole of the surface-emergent section of the structure can be covered in a series of photographs. The accuracy of this method depends largely on the accuracy of the position fixing, of both the section of structure photographed and the position from which the photograph is taken. Kluger's method does not incorporate sophisticated positioning equipment but relies on a relatively simple alignment system; GPS control would enhance this technique. It is equally applicable to river monitoring from the opposite bank.

Box 10.3 *Comparative photography from boats*

Kluger's method relies on two pairs of shore-parallel and shore-normal beacons at each profile, providing a fixed reference line parallel to the line of the breakwater. The principle of *leading lights* is followed, by keeping the two pairs of beacons aligned (Figure 10.6). The same procedure can be conducted without visual markers using predefined GPS-controlled line files and targets within GPS survey software.

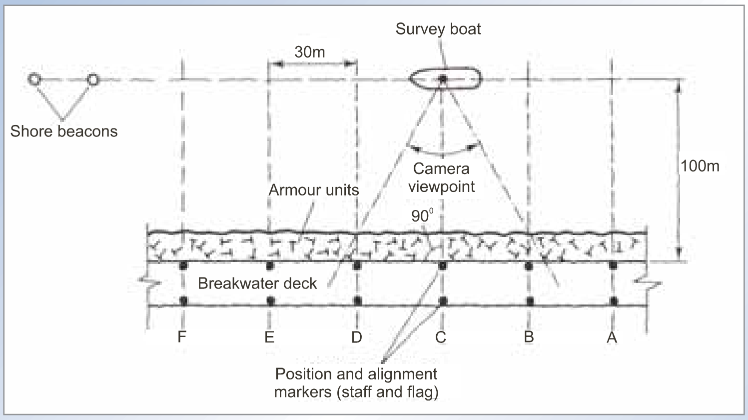


Figure 10.6

Taking comparative photographs from a boat

Overlapping photographs offer the possibility of three-dimensional viewing under a stereoscope. Where possible, surveys should be carried out at low-water spring tides or during periods of low river flow, to ensure maximum coverage of the emergent section of the structure. This is particularly important because the intertidal zone is the area most susceptible to damage under wave conditions. The survey needs to be well documented so that it may be repeated precisely. For initial comparison of consecutive surveys, photographs of corresponding structure sections are enlarged to the same scale – 1:250 is usually suitable. Using digital image analysis, photographs are overlaid and examined to detect any major changes to the armouring.

Aerial visual inspection of coastal structures is an option that has several advantages. Aircraft provide easier access to structures at remote sites and those that are not attached to shore. They also allow the inspector to witness the structure's performance during wave conditions that would be unsafe for a walk-over inspection. Several individual structures along a stretch of coastline can be inspected from the air within a single mobilisation. Still photographs and video can be taken to augment the inspection notes. Images can be compared with previous photographs to identify changes. A disadvantage of aerial inspections is that only significant changes can be identified, whereas subtle changes and signs of deterioration may go undetected, even after inspection of enlarged aerial photographs.

Photogrammetry

Photogrammetric techniques are particularly appropriate for profiling rubble mound structure cross-sections and monitoring movement of armour units on exposed structures (Kendall, 1989; Hughes *et al*, 1995a, 1995b). The technique of photogrammetry is summarised in Section 4.1.1.1.

To date, the detailed methods outlined above have been applied principally to (large) breakwaters armoured with concrete units. However, they are also applicable to armourstone. High-quality, low-level stereo photographs of the structure are used with the ground survey information to establish a computer-based stereo-model, which is a true 3D representation of the study area. Repeat flights over a structure using the same control reference points facilitate comparisons from which can be extracted information such as armourstone movement and profile changes above water level. Examples of photogrammetry for rubble mound structures can be found in Cialone (1984), Kendall (1989) and Hughes *et al* (1995a, b). Estimates of the accuracy of photogrammetry for breakwater analysis have been given by Gerbert and Clausner (1985) and Nale (1983). Periodic photogrammetric mapping permits detection of initial or progressive failure along any visible portion of a structure. Photographs provide a permanent record of the structure. Photogrammetric analysis of aerial photographs is discussed in Box 10.4.

Box 10.4 *Photogrammetric analysis of aerial photography on rock structures*

Permanent photo-identifiable control points are required on or near the structure. Precise co-ordinate positions of the control points are established using conventional ground surveying techniques; these are used in the photogrammetry analysis to correct for aircraft motion, to determine the camera position and orientation relative to ground features, and to compensate for the earth's curvature. At least five or six evenly distributed control points are needed in each photographic stereo pair to remove geometric distortions. Control co-ordinates on the structure should be checked periodically from nearby geodetic and vertical control benchmarks, since they may be subject to settlement. Large circles divided into alternate black and white quadrants, painted at intervals of 100–200 m along the crest of the structure, are ideal. Figure 10.7 shows the typical layout and style of ground control markings.

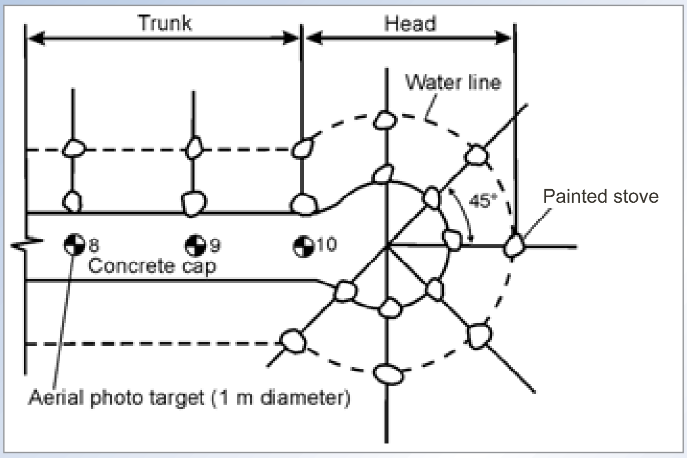


Figure 10.7

Armour survey scheme

Lighting conditions for aerial photography are usually best between 10 am and 2 pm, to minimise shadows, but tidal elevation is of overriding importance. Surveys should coincide with low water spring tides so that exposure of the structure is maximised. The time of year is also relevant, since shadow length varies with the position of the sun. Aerial photography is impractical in low visibility caused by cloud or rain. Low-level flights are desirable to ensure the highest-definition photography. The limit for light aircraft is about 130 km/h at 180 m; this provides photographs at intervals of about 110 m at a contact scale of 1:1200. Air traffic control may limit the flying elevation at some locations. An accurately calibrated FMC cartographic camera is necessary to achieve good results. The forward overlap of the photographs should be at least 60 per cent.

Advantages of aerial photography are that the whole of the visible area of the structure can be recorded in one flight, horizontal movements of armouring are easily defined and the area close to still water level can be monitored without risk. Also, photographs can present data on precise locations of the magnitude and directions of movements in a way that is easily interpreted at any time after the survey. Photogrammetry may be used to quantify movement of individual armour units or stones and/or to describe the outer surface of the structure.

All such surveys require survey control points to be identified. Such control points can be brass discs set into the breakwater crown wall, or steel pins cast into armour units to allow precise identification of their position and level. Anti-fouling and epoxy paints can be used to form targets on units. Painted targets wear away, but repainting at each survey may suffice for frequent surveys. Kinematic GPS enables points to be revisited on each survey without visual reference points, but visual control provides more robust data quality. For photogrammetric control, a survey baseline is usually set along the crest of the structure, by reference to land-based control points; this can be defined by fixed targets on the structure, generally painted at each profile line. Targets must be large enough to be visible in the photographs.

10.3.4.4 *Land-based topographic surveys*

The measurement of the location of the structure in relation to an external reference system requires conventional engineering survey equipment such as levels, total station or kinematic GPS. The generic techniques used for such surveys are consistent with standard topographic survey techniques. Techniques of specific relevance to rock structures include:

- the use and interpretation of measurements
- resolution levels
- control and reference points
- setting out and relocation of profile lines.

Topographic survey methods may be used to define the position of reference points, spot heights on main structural elements, control points used for photogrammetric measurements, and points along the profile lines – see Section 4.1.1.1. Levelling surveys can be used to determine settlement or other displacement of the crown wall and/or armouring. Repeated profile surveys should also locate and quantify areas of armour stone displacement, local settlement and toe erosion or accretion (see Box 10.5), where access is safe and practical.

Profile surveys

The results of profile surveys may be used to calculate areas of erosion, and hence damage levels, S_d (see Sections 5.2.1.2, 5.2.2.2), comparable with methods used in hydraulic model tests. This application of monitoring data suggests that profile survey methods provide the best form of evaluation relative to design conditions. Profile or spot height surveys may also be used to generate contoured plans of the structure. Areas where levels fall below or exceed design values may then be identified. Data may also be used to prepare plans showing areas of settlement or displacement of armour. An example drawn from work by Weymouth and Magoon (1969) is shown in Figure 10.8.

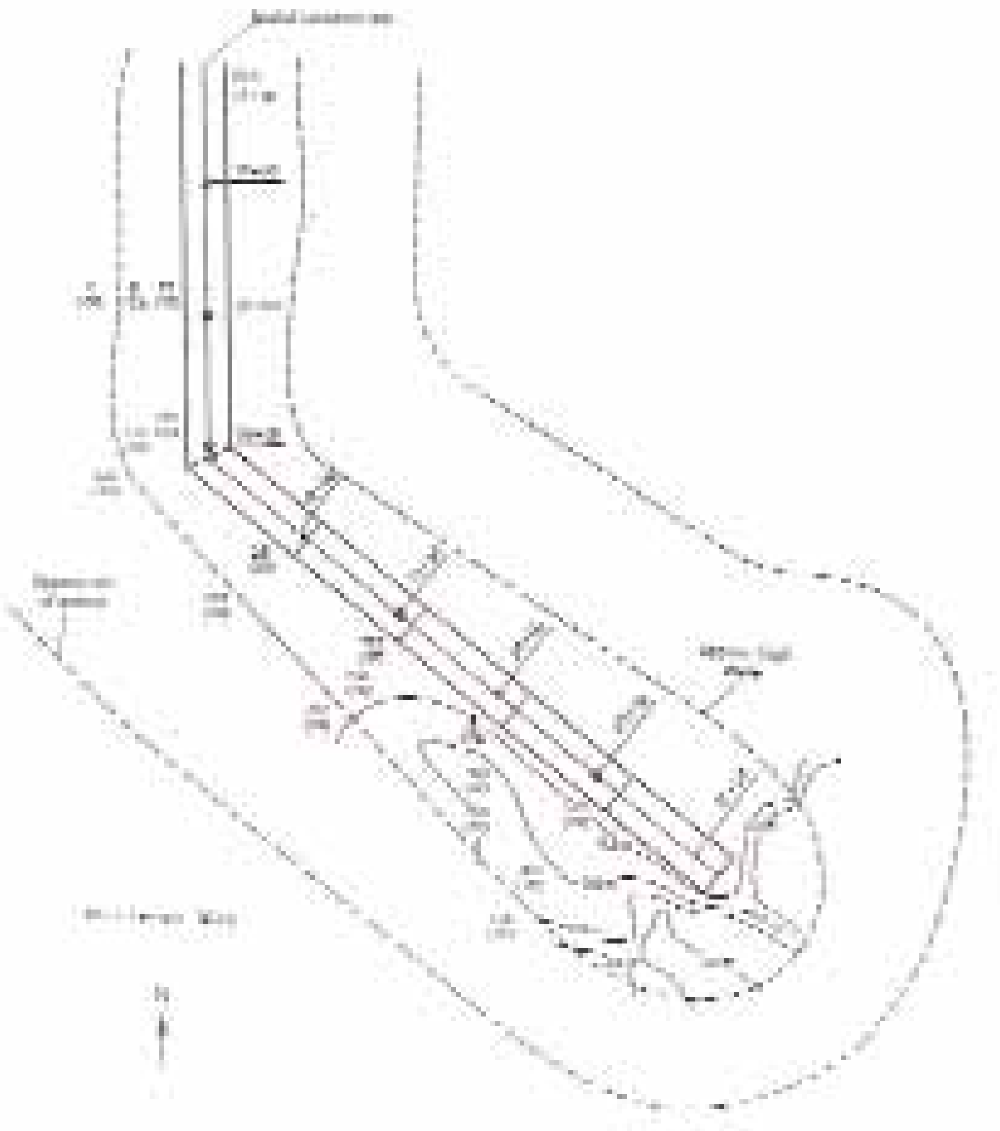


Figure 10.8 *Contours of equal settlement or displacement of armourstone on a breakwater (Weymouth and Magoon, 1969)*

Profile surveys can be conducted using the following procedure.

1 Determine the profile interval along the structure according to the resolution needed, the structure complexity and the resources available. Typical monitoring surveys have adopted profile intervals between 5 m and 300 m, depending on structure complexity. Typically profiles are needed at 10–20 m to define changes adequately.

2 Set out profiles perpendicular to the crest, or setting-out line, using simple sighting aids or GPS.

3 Take profile measurements. Generally these should be made at intervals of less than D_{n50}, which enables the results to be compared with design damage derived from hydraulic model tests (Bradbury and Allsop, 1989). A fixed sampling interval along the profile line is desirable for interpretation, although this is difficult to enforce unless RTK GPS is used in *stakeout* mode. It may be a reasonable compromise to ask the surveyor to take coordinates on the centre of each piece of armourstone along the profile line.

4 Compare profile lines from different surveys. Interpolation is needed between survey points. A profile analysis method using a cubic curve-fitting technique (as used in hydraulic model tests (Bradbury and Allsop, 1989)) can be applied to field data for structure evaluation.

5 On large structures, and where access is difficult, the survey staff may be replaced by a heavy plumbing rod or tube marked off in height intervals. This can be suspended from a crane. At the foot of this *staff* a spherical cage replaces the spherical foot (see Section 9.9.8.1). Provided that the crane reach is adequate, the staff can be used down much of the structure face. The plan position and elevation of the survey point is confirmed by theodolite or GPS.

Box 10.5 *Case study condition monitoring of Hurst Spit revetment*

Condition monitoring of a coastal revetment at Hurst Spit, Hampshire, UK (see Figure 10.9), provided an opportunity to examine the performance of a low-cost structure under extreme conditions, using damage evaluation methods comparable to those now employed in physical models for the design of structures. The structure had been constructed as an emergency measure in 1963 without conventional design input and before any such guidance was available. Nominal armourstone grading was 2–4 t, placed at a slope angle of 1:3. The armour layer was constructed unconventionally in a single layer, placed directly on core material. This type of emergency construction is common in older structures. This low-cost structure was frequently damaged and required regular maintenance.

Figure 10.9 *Hurst Spit armourstone revetment in December 1989, after storm damage (courtesy A P Bradbury)*

Box 10.5 *Case study condition monitoring of Hurst revetment (contd)*

Given the expectations of frequent damage, a maintenance programme was developed to include regular structure condition monitoring. This included visual inspection of armour layer and armourstone degradation (see Section 10.3.4.2) and regular profile surveys (see Section 10.3.4.4) of the revetment – both annual and post-storm (see Figure 10.10). These have formed the basis for maintenance planning. Profiling of the structure was conducted by levelling a series of profiles across the structure with points on the profiles at 0.5 D_{n50} intervals. Severe damage to the crest and armoured slopes and exposure of the core material occurred in a storm at an extreme water level in 1989. Profile data was put in context with environmental loading conditions using wave data from a local buoy (H_s = 2.9 m, T_m = 9 s) and a local tide gauge.

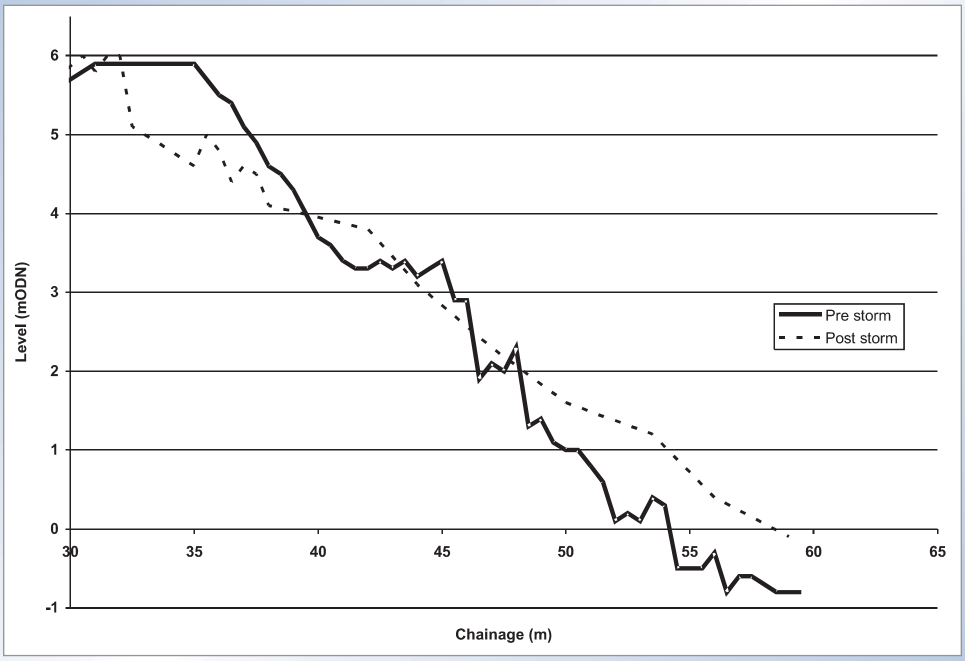


Figure 10.10 *Comparison of pre- and post-storm structure profiles showing damage (S_d = 7)*

Subsequent analysis of the data demonstrated that the armourstone was significantly under-size for the slope (had the structure been designed for *no damage*), the crest detailing was poor and the risk of damage to armour layer was high in events with a return period of one year. Following economic evaluation the structure was redesigned for a *no damage* situation and constructed using a conventional two-layer structure of 3–6 t armourstone placed at a slope angle of 1:3. Continued monitoring of the new structure, built in 1996, has shown no damage, despite exposure to events with H_s = 4.1 m and T_m = 9 s.

Recent developments in the use of land-based laser scanning technology have been trialled (see Box 10.6) to assess their value for monitoring structure condition (see Figure 10.11). Laser scanning is discussed in Section 4.1.1.1. Preliminary findings suggest that this technology could be an extremely efficient method of assessing large-scale structures, overcoming many safety issues. Centimetre-level precision can be achieved. Detailed profiles and surface models can be generated and compared in digital terrain models to identify precisely the location and volume of change to structures when extracted from a 3D point cloud of data.

Box 10.6 *Application of land-based laser scanning to surveys of coastal structures*

3D laser scanning is discussed in Section 4.1.1.1. Laser scanning provides rapid acquisition of 3D data and high-definition digital photography – see Figure 10.11. Fixed markers linked to a GPS control network provide control such that the data can be used to generate a digital terrain model (DTM) of the structure. Rectified orthophotography can be produced by combining the DTM with the 2D photography. The generation of orthophotography is particularly suited to vertical or near-vertical surfaces and is therefore useful for the presentation of faces of sea defence structures. DTMs can also be exploited further, to allow the generation of thematic maps showing colour-coded structure views according to structure levels, or change-detection maps showing areas of erosion and accretion as derived from repeat surveys.

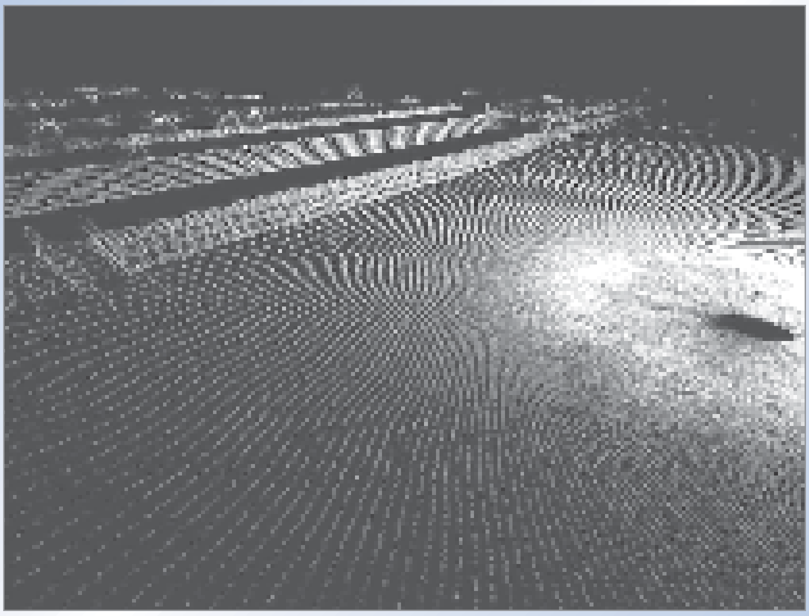


Figure 10.11 *Land-based scanning laser of structure showing 3D point cloud data (courtesy Halcrow)*

10.3.5 Underwater surveys

Considerable damage may occur beneath the water level, particularly immediately below it, where wave impacts and damage are often greatest. The toe is susceptible to damage and is an area from which damage can spread rapidly. To analyse the performance of continually immersed sections, there is a need for techniques that can identify both large- and small-scale damage. Although quantifying underwater changes to coastal and fluvial structures is difficult, it is an important part of monitoring structure condition (USACE, 2003).

A limited but safe method for measuring underwater profiles of sloping structures is to make soundings of the underwater portions by means of a crane or hydraulic machine situated on the structure crest or river bank. Horizontal and vertical positions can be established using kinematic GPS or with an EDM target (see Figure 10.12). This method relies on the availability and capability of the crane and also on access to the structure crest.

Figure 10.12 *Profiling the submerged section of a breakwater by soundings from a crane (courtesy David Bowie Photographic)*

10.3.5.1 Single- and multi-beam bathymetric surveys

Acoustic sounding equipment such as the single-beam echosounder has been relied upon for some time to determine the shape of submerged structures and the adjacent sea bed. Surveys using this equipment provide information about variations in depth to the sea bed or structures on it. Alone they cannot give enough information about the submerged portion of the structure to allow damage to be assessed, but by combining bathymetric surveys with other techniques it is possible to build up a more complete description of its condition. Bathymetric charts are compiled using three components of data:

- location in the horizontal plane
- depth of soundings
- water level at the time of sounding.

Position fixing is now typically carried out using kinematic or differential GPS. Soundings may be made with single- or multi-beam echosounders. Normally, single-beam soundings are made along parallel sounding lines across the survey area, being run as closely as possible to right angles to the depth contours. The interval between single-beam sounding lines varies according to the accuracy required, but line spacing should not normally exceed 10 m for structure surveys. Single-beam bathymetric surveys are restricted by safety considerations when working close to a structure.

Displaced armourstone may present a survey hazard and is often a problem at sites with a narrow tidal range where it is difficult to survey the section of structure that lies immediately below low water. Wave action reduces the quality of records from acoustic sounding equipment and can cause large offsets to the bottom traces; this may prevent surveying over relatively steep sections of the structure. Waves may also restrict where and how the vessel can be positioned. It is possible to filter wave noise from the traces, although line detail will be lost. Records collected on calm days show more detail.

High-resolution, high-frequency, narrow multi-beam signals provide the most accurate measurements and are most suitable for measuring breakwater profiles. The best solution to date is a downward and side-looking single-transducer multi-beam sonar system (Prickett, 1996). The instrument is mounted on a vessel with the sonar head positioned to transmit on a plane perpendicular to the vessel's heading. The sonar transmits between 60 and 100 sonar beams on radials spaced at 1.5°, giving total swath coverage of 90–150°. By tilting the sonar head, the instrument can provide data for mapping almost the entire underwater portion of a sloping rubble mound structure from just below the sea surface to the structure toe (Box 10.7), often enabling the full structure survey to be completed in a single track. Data must be synchronised with simultaneous readings of vessel position, heading and motion. The final analysed product is a spatially rectified map of the submerged structure condition.

Although it is difficult to identify displacement of individual armourstone, any slope irregularities caused by construction or subsequent damage are easily spotted on the map (USACE, 2003). Variations such as slope changes, depressions, other irregularities and the line of the toe can usually be detected. While pattern-placed or regular armour can be easily located, randomly orientated units are more difficult to assess (Tomlinson *et al*, 2001). Individual armour units are not easily identified. Relative accuracy of various acoustic sounding methods is discussed in detail by Rotterdam PWED *et al* (2001).

10.3.5.2 Side-scan sonar

Side-scan sonar records are obtained by towing an instrument from a vessel running parallel to the structure. The records can be interpreted to give general information of underwater structure condition, particularly near the bed. The main advantage of side-scan sonar is the coverage and the speed of surveying (USACE, 2003). Specialist skills are needed to interpret the record. Side-scan is useful to identify structure portions that need to be examined in detail by divers. Additional information and operating rules of thumb are provided by Kucharski and Clausner (1989, 1990) and Morang *et al* (1997).

The signals transmitted by the transducers from side-scan sonar systems are directed laterally by two side-surveying beams. The recorder initiates a signal that is reflected back and appears as a darkened area. The more reflective the object illuminated by the signal, the darker the record. Shadow areas are shown according to the angle of the signal and the size of the object. Various shades of grey indicate changes in texture and relief. The image projected on to the sonograph is not a true representation of the slope scanned so needs correction for distortions.

Wave conditions and the speed of the boat can affect the quality of results. Boat speeds of less than 1 m/s are required to identify features of about 1 m size. Similarly, the frequency of the transducer signal is most important. Transducers with a frequency of 500 kHz may be capable of resolving variations in size of armour units, although they cannot identify the precise location of individual units. The electronics are located in a housing, known as a *fish*, which is towed behind the survey vessel. The resolution of a sonograph also depends to a large extent upon the beam width and method of towing the fish. If the fish is towed low in the water, offshore from the structure, the line of the toe may be well defined on the sonograph. If it is towed close to water level and viewing down the slope, shadowing effects are accentuated, allowing high areas and depressions to be identified. Similarly, steep zones on a structure may reduce the definition of other parts of the structure. Smearing of the sonograph may occur if the fish is allowed to yaw while being towed. If the armourstones are laid in a regular manner, gaps, irregularities and placement trends may be identified on the sonar traces that will allow divers to locate areas requiring detailed inspection (see Section 10.3.5.5).

Box 10.7 *Multi-beam inspection of Peterhead Breakwaters*

Peterhead Breakwaters, in north-east Scotland, lie in 12–20 m water. One is 870 m, the other 460 m long. The structures are vertical masonry walls on a rubble mound foundation. Significant wave heights reach 8 m in the 1:50-year storm and regular damage occurs to the foundation mound, although this does not generally threaten overall stability.

Diver surveys of the structures used to be conducted once or twice a year, each of which could take more than three months to complete. Now they are complemented by remotely operated vehicle (ROV) and side-scan surveys and multi-beam bathymetry. Side-scan surveys have provided useful data, which are combined with data from swath bathymetry surveys undertaken in 2000. The latter provided full coverage of the structure in a few days (Tomlinson *et al*, 2001). The system has the potential to identify armourstone movements and breakage, and is of value in identifying scour, sedimentation and settlement. The imagery (Figure 10.13) provides a good overview of the integrity of the structures. Each part of the structure has been geo-referenced, with heights resolved to better than 0.2 m. Individual blocks with 1 m sides have been located within 0.3 m horizontally. Data plotted in GIS enables surveys to be directly compared, individual unit movements to be detected and the condition of the blockwork walls to be appraised. With GIS, data can be managed effectively and all inspection types can be combined.

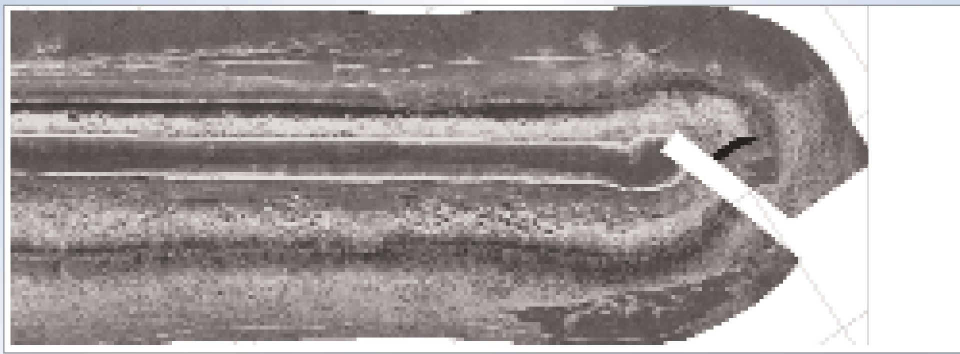


Figure 10.13 *Multi-beam image of Peterhead Breakwater (courtesy EMU Environmental)*

Good agreement has been achieved between datasets. Data are used to direct highly localised diver inspections of areas showing signs of movement and provides an affordable and repeatable methodology for underwater inspection.

10.3.5.3 Airborne remote sensing of submerged structures

Airborne LIDAR (light detection and ranging) is a laser scanning technique that can be employed on both the underwater and above-water portions of sloping structures (Parsons and Lillycrop, 1988). Such a survey is not usually conducted with the sole purpose of examining structures but is an added benefit that occurs during the survey of a much larger area (typically several square kilometres are flown in a single mobilisation). The spatial distribution of data is insufficient to recognise smaller irregularities in the armour layer, such as individual movements. However, larger problems in the armour slope and details of adjacent scour holes are easy to see. Applications of this technology are limited to sites with good through-the-water visibility because LIDAR penetration through water is limited to 2 × Secchi depth.

10.3.5.4 Sub-bottom surveys

Sub-bottom surveys are undertaken to identify problems in connection with foundations but, after construction, can only be used alongside the structure. For example, subsidence resulting from sub-surface faulting may be detected by shallow seismic survey work. A surface-towed sub-bottom profiler may be employed to identify the structure of the sub-surface environment. This instrument is of use in determining the depth of bedrock and type of surface cover and can also identify the location of faults at depth below the bed. Indication of such features may be of importance if extensions or repairs to a structure have been proposed. Such surveys are best carried out at the design stage to identify faults and are only occasionally of value for maintenance monitoring.

10.3.5.5 *Underwater visual inspections*

At many sites it is difficult, if not impossible, to make a visual inspection of underwater structure condition. Where conditions permit, divers are used to survey small, submerged sections of structures, often in areas where irregularities have been detected by multi-beam surveys. Inspections require professional divers who also understand the signs of structure damage and deterioration. Diving inspection methods are limited by the range and angle of underwater visibility, which is generally poor and reduces with depth and concentration of material in suspension. The angle of view often restricts the diver's vision to no more than 3 m width of the structure, making it hard to identify large irregularities in structure profile.

Weed on or around the armouring can also limit observations. Visibility needs to be such that the diver can see enough of the slope to recognise missing, damaged or displaced armour and slope discontinuities. Even in the best of conditions, diver surveys produce subjective information and only sparse spatial detail, while coverage is unlikely to exceed 50 m^2 per day. The stability of the toe (and the toe trench, if applicable) may be examined and small-scale movements in this area identified, providing confirmatory evidence or details of damage that has been identified with other techniques such as multi-beam surveys. In some circumstances, it may be possible to use video cameras lowered into the water or mounted on remotely controlled vehicles to inspect underwater portions of a structure.

A detailed underwater survey is labour-intensive and expensive. A typical survey procedure for the examination of a breakwater would require a diving team made up of three engineers – a diver, a standby diver and a supervisor. The diver often works from a boat, attached by a safety line or communication rope, and is also linked to a floating marker buoy with a separate line. Inspection of the armour slope is carried out by ascending and descending, on a compass bearing at fixed chainage points, marked at predetermined positions along the crest of the structure. The spacing of these points would typically be at 5 m centres, assuming that visibility is sufficient to see 2.5 m or more.

On observing a fault or point of interest on the structure (ie location of voids, broken armourstones or exposure of core), the diver maintains position and communicates to the shore party, which records the position of a buoy that the diver holds directly over the point of interest. Position-fixing of the buoy is typically by GPS. The level of the subject of interest can be measured by a helium pressure-depth gauge on the diver's wrist to an accuracy of between 0.1–0.3 m. Alternatively, the line to the floating marker buoy can be calibrated for direct depth reading. A tide graph and times of measurements are required to relate the depth to the datum used. This method should identify a position within 1 m or 2 m radius in plan position and +0.3 m in level, which should be sufficient to pinpoint an individual armour unit at a later date.

All diving operations should be carried out in accordance with local and national safety standards. Weather conditions also impose restrictions. Diving surveys are best carried out at neap tides; even then, underwater inspections can often only take place during the slack water. This allows safer investigation of the intertidal zone, which is potentially the most vulnerable part of the structure. Further information on diver inspections is given in Thomas (1985).

10.4 EVALUATION OF STRUCTURE CONDITION AND PERFORMANCE

Appraisal of monitoring data is used to determine the timing and extent of maintenance requirements. Procedures are required to determine:

- how to evaluate the monitoring data
- whether or not to undertake preventative or corrective action
- how to assess the economic benefits of the possible responses and the implications of doing nothing
- how to determine the extent of required maintenance works
- when to increase the monitoring interval.

10.4.1 Evaluation of monitoring data

Visual inspections are subjective (see Section 10.3.4.1) and one observer's overall evaluation of structure condition may differ substantially from another's opinion. Nevertheless, this system is still of considerable value (USACE, 2003). By comparison, the various topographic and bathymetric survey techniques (see Section 10.3.4.4) can produce numerical output that can be linked directly with design techniques (see Section 5.2) and provide a meaningful measure of damage, damage progression and risk of failure (Melby, 1999). For further details on damage progression, see Section 5.2.2.2. This section gives guidelines for evaluating in a consistent way the physical condition and functional performance of armoured structures. These procedures provide a meaningful evaluation, by quantifying inspection observations and surveys relative to condition and performance criteria, and also enable better tracking of structure condition over time, as discussed by Oliver *et al* (1998). The condition and performance rating system (USACE, 2003) places the emphasis on the question "how well is the structure functioning?" rather than on "what is the physical condition relative to the as-built structure?". This focus recognises that armoured structures have some level of deterioration tolerance before they suffer significant loss of functionality, so condition alone is not sufficient justification for rehabilitation. The evaluation system should offer a means of continually evaluating design and performance criteria. Typical aspects of evaluation are listed in Table 10.10, which identifies changing condition at a range of spatial scales.

Table 10.10 *Outputs from comparison of measures of the state of rock structures over time*

Aspect of structure state measured	Structure state at a number of points in time
Level I: Location	Settlement of foundation; change in alignment
Level II: Geometry	Consolidation of structure; comparison of slope profiles enables overall armour damage parameter, S_d (see Chapter 5), to be determined; scour damage
Level III: Composition	Loss or movement of armour stones; overall sliding of armour layers if this has occurred; voids requiring emergency or planned repair
Level IV: Element composition	Rounding of armour stones and loss of material, enabling revised evaluation of D_{n50} with the design wave climate, or measured wave climate, or revised design wave climate from wave measurements; this allows re-evaluation of armour stability parameter $H_s/(\Delta D_{n50})$ using equations given in Section 5.2.2. Comparison with design and measured damage parameter, S_d, is also possible

Functional performance and structure condition need to be examined together. The process involves several steps, including:

- definition of performance requirements related to design conditions
- measurement of damage progression
- loss of functionality
- prediction of future performance loss
- planning the timing and development of repairs.

10.4.2 Performance evaluation

The review of structure maintenance requirements should be condition-based and should relate current performance to functional design requirements. There are five key questions to be answered.

1. Is the structure's hydraulic performance and stability adequate?
2. Does the structure meet health and safety requirements?
3. Does the structure have an adverse impact on the local system, eg sediment transport?
4. Is the structure's performance declining?
5. Is performance likely to be reduced to an unacceptable level before the next repair or inspection opportunity?

Detailed advice is provided on a range of conceptual approaches to performance evaluation. For example, the performance-based asset management (PAMS) system (HR Wallingford, 2003) is directed at flood risk reduction, whereby management of assets is not only dictated by structural condition (ie some form of improved condition grade) but is also based on its function, reliability and criticality in terms of its contribution to risk and risk reduction (see Figure 10.14). In the longer term, it will also provide a means of identifying a preferred set of management interventions appropriate to achieve a particular outcome in terms of risk reduction or investment profile. Oliver *et al* (1998) present a similar model for coastal structures using condition indices and Melby (1999) provides methods for prioritisation of maintenance on large groups of structures under the same management.

This section focuses on the application of these broad principles to the evaluation of armour damage and degradation. To conduct a performance evaluation the procedures described in Table 10.11 should be followed.

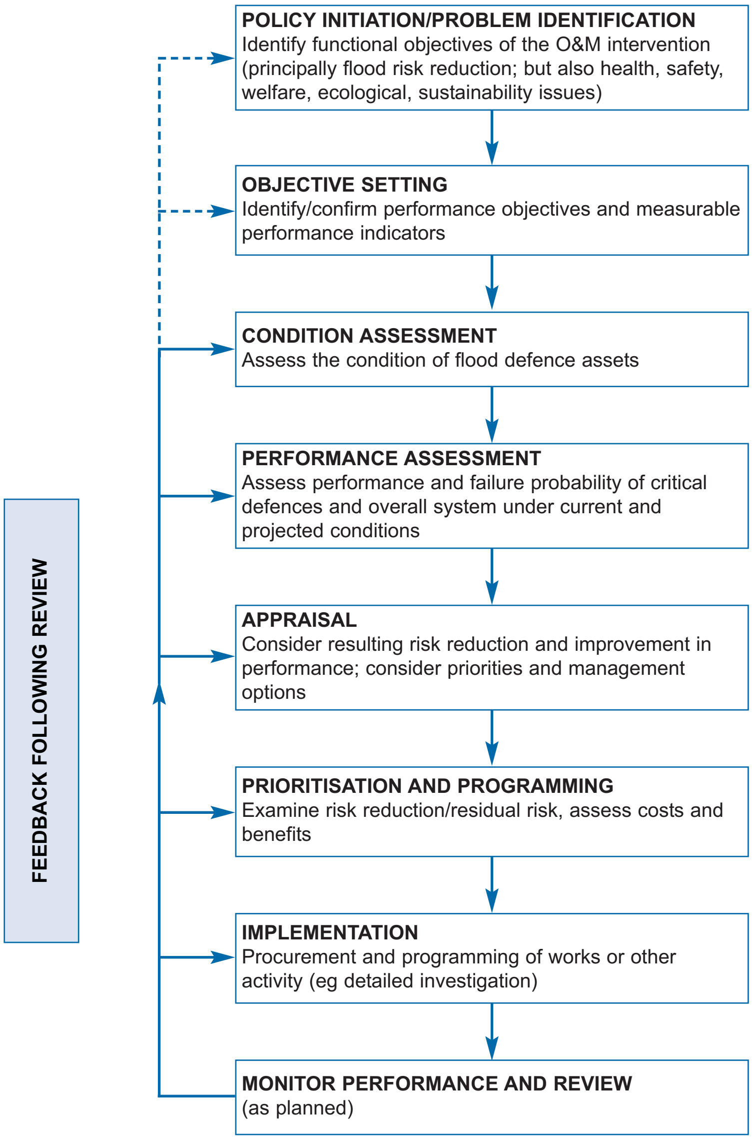


Figure 10.14 *Logical framework for operational and maintenance activities as used by UK Environment Agency (Posford Haskoning, 2002)*

Table 10.11 *Maintaining a structure on the basis of functional performance (based on USACE, 2003)*

Step	Description
Steps 1–3 are initial (once only) or conducted when functional requirements alter	
1	Determine structure performance requirement for each function, and structure element, relative to design requirements (Table 10.12)
2	Establish functional performance criteria (Table 10.12)
3	Establish structural requirements – identify allowable damage before repair is required
Steps 4–6 are repeated as necessary	
4	a Inspect structure b Produce structural evaluation
5	Produce functional evaluation
6	a Review structural requirements relative to defined performance requirements b Estimate minimum time before maintenance is required c Prioritise maintenance

The performance level on a degrading structure is initially difficult to define but should be linked to the appropriate design limits for that structure, using procedures outlined in Chapters 5, 6, 7 and 8 (see Table 10.12). Performance requirements will vary according to structure function. For example, a reduction in crest elevation of a coastal revetment designed to protect against flooding, caused by armour slope damage, may result in potentially unacceptable flooding. A similar reduction in crest elevation of a low-cost rock groyne, designed to control sediment transport rates, is likely to have less significant (or less immediate) implications. In each case, the acceptable threshold limits should be determined by reference to design techniques and the life cycle cost approach (see Section 2.4); this should be linked to a risk assessment of continued structural damage.

Table 10.12 *Typical functional and structural categories (based on Oliver et al, 1997)*

Functional performance area	Functional requirement categories	Structure type	Structural damage categories	Impacts of structure degradation on performance based on design requirements
Harbour	Harbour navigation Harbour use	Breakwater	Breach Core exposure Loss of armour units or stones	Increased overtopping (5.1.1.3) or wave transmission (5.1.1.4) Increased wave disturbance – unacceptable wave activity for vessels
River bank	Erosion control Navigation	Revetment	Core exposure Loss of armourstone Crest damage	Bank vulnerable to erosion under high flow Navigation hazard
Navigation channel	Entrance use Channel use	Breakwater Dike	Loss of contact and interlock of armourstone Loss of armourstone	Navigation hazard
Sediment management	Longshore transport Sedimentation Bed protection Shoreline erosion control	Coastal and river groynes, breakwaters	Armour quality defects Core exposure Loss of armourstone	Structure failure leading to increase in sediment transport rate Changes in current flow patterns Increase in shoreline erosion, beach lowering
River structures	Scour protection	Closure works Bridge piers	Core exposure Loss of armourstone	Undermining of structure and collapse Toe erosion and destabilisation Impacts of temporary structures on closure projects
Coastal erosion and flooding	Erosion control Flood control	Revetment	Core exposure Armourstone loss Crest damage Loss of contact and interlock of armourstone	Increased frequency and extent of flooding Coastal erosion and property damage

10.4.2.1 *Subdivision of structures into structural and length elements*

A structure's function can vary over different portions of its length (USACE, 2003). Division of a structure into manageable maintenance lengths is based primarily on changes in construction characteristics. Examples include changes in type of construction, type or size of armour, changes in cross-sectional dimensions or geometry, and rehabilitated sections. Final subdivisions are based on length where function and construction are uniform over long distances. Generally, each final division length should be between 60 m and 150 m, with the head section always being considered as a separate element. A systematic numbering scheme enables the structure to be assessed in small manageable segments with similar performance requirements.

10.4.2.2 Establishing functional performance criteria

Once structure functions have been determined for each structure element, the next step is to determine the expected performance level for each rating category. These criteria must be based on how well the structure could perform relative to storm events when in perfect physical condition. The design storm is the largest storm (or most adverse combination of storm conditions) that the structure is intended to withstand while maintaining full functional performance requirements. Design conditions may include wave height, direction and period; water level; storm duration; flow rate; and combinations of these factors. The design storm is usually designated by the frequency or probability of occurrence. Performance criteria may be defined relative to varying combinations of conditions with assigned probabilities of occurrence. Project history, public input and analysis may be required to identify these dimensions – this is not an exact science and some engineering judgement is necessary to produce reasonable estimates. The following procedures should be observed:

- review the structure history
- check whether design limits have been changed or if they need to be changed based on past observations or functional requirements
- calculate acceptable design limits for each function, usually referenced to design events with defined probabilities of occurrence
- determine the sensitivity of structure performance and stability for a range of event combinations
- define acceptable performance limits for each event and each function.

10.4.3 Armour condition assessment

The most suitable way to quantify armour damage is to calculate the clearly defined damage parameter, S_d (see Sections 5.2.1.2 and 5.2.2.2). This should be done by comparing measured profiles of conditions – such as the physical condition, alignment and cross-sectional dimensions – of an existing structure to the expected conditions of a similar new-build structure built according to good practice and with good-quality materials. The comparison process is demonstrated in Figure 10.15, based on the modified damage measurement procedure defined by Melby and Kobayashi (1998, 2000). The procedure provides an enhanced description of damage, using the same profile data, to determine dimensionless descriptors that describe the thickness of armour layer, depth of erosion and slope length of erosion. The most significant addition is the inclusion of cover depth above the underlayer. The acceptable limits of S_d depend mainly on the slope angle of the structure. For a double armour layer of thickness $2\,k_t\,D_{n50}$ the values in Table 5.23 can be used to define:

- start of damage; $S_d = 2$, corresponding to *no damage*
- intermediate damage
- failure, corresponding to reshaping of the armour layer such that the filter layer under a $2\,k_t\,D_{n50}$ thick armour layer is visible.

Although a damage level of $S_d = 2–3$ is often used for design purposes, in some cases it might be feasible to apply higher damage levels of $S_d = 4–5$. This depends on the desired life cycle of the structure. Under most circumstances repair would not normally be required until damage has at least reached the intermediate damage category (see Table 5.23). This quantitative assessment technique can be used in parallel with more subjective visual inspections. Because rubble mound structures tolerate a degree of damage before loss of functionality, structural damage does not automatically equate to loss of function (USACE, 2003). The structural requirements are established by determining what minimum structure cross-sectional dimensions, crest elevation and level of structural integrity are needed to meet the functional performance requirements. Initial efforts to determine these structural dimensions can be aided by estimating the impact on structure functionality if the element under study were to be completely destroyed.

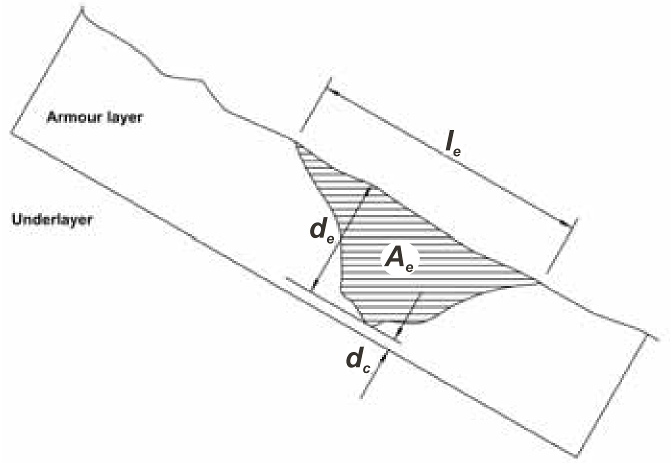


Figure 10.15 *Sketch of breakwater profile with definition of damage parameters (Melby, 1999)*

Damage progression is likely to continue throughout a structure's life and an equilibrium profile does not appear to develop before failure. Melby (1999) presents a method for predicting progressive armour damage. This can be used either to analyse short-term residual life, following a single event, or to simulate life cycle changes. Damage can be predicted by applying stability formulae for design conditions, or a range of life cycle conditions, relative to the current state (see Section 5.2.2.2).

The recommended procedure for structural evaluation of armour layers is outlined below:

- define design condition combinations (H_s, T_m, SWL)
- define the failure condition, as a defined value of S_d, for major rehabilitation to be required
- define the critical damage condition, as a defined value of S_d, for repair maintenance, based on the functional evaluation, for each structure element
- define the significance of failure mode for each element to the overall performance of the structure
- define and map outlines of structure evaluation segments, for example:
  - a crest
  - b roundhead
  - c trunk
  - d toe
- measure cross-section profiles along the length of the structure (see Section 10.3.4.4)
- compare measured profiles with the theoretical thickness, as-built or design profile
- compare measured profiles with the previous survey(s) to analyse damage progression
- calculate damage, S_d, for each profile using design methods (see Section 5.2.2.2)
- determine the variability of S_d and remaining cover depth over the damaged length
- map the results and identify zones of damage
- identify zones that have reached the allowable damage threshold and plan maintenance
- plot damage trends over the course of surveys and relate these to loading conditions
- project damage trends forward on the basis of trends and probability of storm events using either Melby's progressive damage predictors or the method of Van der Meer (see Section 5.2.2.2)
- estimate when damage is likely to impact on performance
- assess when the underlayer is likely to be exposed
- estimate the residual life of the structure for each damage pattern and design condition
- determine the probability of reaching the intervention threshold before the next scheduled survey.

10.4.3.1 Standard damage descriptions

To assist with the numerical evaluation of armour condition, some standard subjective descriptions derived from visual inspections are presented both below and in Table 10.13 (USACE, 2003).

Breach/loss of crest elevation

A *breach* is a depression (or gap) in the crest of a rubble mound structure that extends to or below the bottom of the armour layer. It is caused by armour displacement. To be defined as a *breach* the gap must extend across the full width of the crest. Loss of crest elevation is primarily caused by settlement either of the structure or of the foundation, both of which result in a reduced structure height.

Core (or underlayer) exposure/core loss

When the underlayer or the core are clearly visible through gaps between the primary armour stones this is termed *core exposure*. Core loss occurs when underlayer or core is removed from the structure by waves passing through openings or gaps in the armour layer. Movement and separation of armourstones often result in the exposure of the underlayer or core material.

Armourstone loss

- *Displacement* is most likely to occur near the still water line where dynamic wave and uplift forces are greatest. Localised loss of armourstone (up to four or five stones in length) is typically like a pocket in the armour layer at the waterline where the displaced stones have moved downslope to the toe of the structure. (If the area extends further than four or five armour stones, use the rating for *slope defects* given below.)
- *Settling* may take place along or transverse to the slope. Causes include consolidation or settlement of underlayer stone, the core or foundation soils.
- *Bridging* is a form of armourstone loss that may apply to the side slopes or the crest of a rubble mound structure. It occurs when the underlying layers settle but the top armour layer remains in position at or near its original elevation. This leaves a *bridge* over the resulting cavity much like an arch.

Loss of armourstone contact or interlock

Armourstone *contact* is the edge-to-edge, edge-to-surface, or surface-to-surface contact between adjacent armour units, particularly large armour stones. *Interlock* refers to physical containment by adjacent armour units. Certain types of concrete armour units are designed to permit part of one unit to nest with its neighbours. In this arrangement, one or more additional units would have to move significantly to free any given unit from the matrix. Any special armour unit placement should be stated in the inspection notes.

Armourstone quality defects

This rating category deals with structural damage to the armour units or pieces of armourstone. It is not a rating of potential armour durability, but rather a reflection of how much damage or deterioration has already occurred. Four kinds of armourstone quality defects are defined below.

- *Rounding* of armour stones, rip-rap, or concrete armour units with angular edges is caused by cyclic small movements or by abrasion that wear edges into smoother, rounded contours. This reduces the overall stability of the armour layer by decreasing the effectiveness of edge-to-edge or edge-to-surface contact between units and making it easier for them to move.
- *Spalling* is the loss of material from the surface of the armour unit. Spalling can be caused by mechanical impacts between units, stress concentrations at edges or points of

armour units, deterioration of both rock and concrete by chemical reactions in seawater, freeze-thaw cycles, ice abrasion, or other causes.

- *Cracking* is defined by visible fractures in the surface of either armourstones or concrete armour units. Cracks may be superficial or may penetrate deep into the body of the armour unit. Cracking is potentially most serious in slender concrete armour units.
- *Fracturing* occurs where cracks progress to the stage that the armour unit breaks into at least two major pieces. Fracturing has serious consequences for armour layer stability and brings a risk of imminent and catastrophic failure.

Slope defects

When loss of armour units or settlement occurs over a large enough area to change the shape or angle of the side slope this constitutes a slope defect. Slope defects occur when many adjacent armour units (or underlayer stones) appear to settle or slide as if they are a single mass. There are two forms of slope defect.

- *Slope steepening* is a localised process where the sloping surface appears steeper than originally designed or constructed. Steepening is evidence of a failure in progress on the slope of a rubble mound structure.
- *Sliding* is a general loss of the armour layer directly down the slope. Unlike slope steepening, this problem is usually caused by more serious failures at the toe of the structure. Slope failure can be caused by severe toe scour, such as can occur at a tidal inlet with strong currents, or by failure within weak, cohesive soils when soil shear strength is exceeded.

Table 10.13 *Descriptive rating guidance for armour loss (based on USACE, 2003)*

Structural rating (typical associated maintenance)	Description
Minor or no damage (no action)	At most, slight movement of armourstones in a few isolated spots Movement has left a depression no larger than ¼ of one armour stone (or unit) diameter Armourstone movement has caused some waviness along the slope surface with depressions less than ¾ of the armour layer thickness Any bridging is over a void less than ½ of the armourstone diameter. Underlayer may be seen in places, but no material has been lost
Moderate damage (repair)	Some loss of armourstone in spots, leaving voids or depressions about the size of an armour unit; units surrounding the void may be rocking or gradually moving out of place Underlayer or core might be seen at these spots, but armourstone position still prevents loss of this material Bridging to a diameter of an armour stone may be visible in several places Armour units have been lost or displaced in some portions of the reach length Voids are just large enough to allow loss of underlayer
Major damage (rehabilitation)	Armour units have been fully displaced or lost Voids are large enough to allow underlayer and core loss with ease Armour units have been fully displaced or lost Loss of underlayer material is evident Armour units are gone or fully displaced. Structure is unravelling

10.4.4 Management options

Having completed the evaluations, structure managers should decide to opt for one or more of the maintenance actions (see Section 10.5). The options may be briefly listed as:

- do no repair or replacement work and await next planned monitoring report

- do no repair or replacement work but instigate additional future monitoring of structure state and/or environmental conditions
- carry out further detailed inspection before making a decision
- undertake temporary or *emergency* repair or replacement works
- undertake permanent repair or replacement works
- instigate development of a new (rehabilitated or replacement) structure
- instigate abandonment or removal of the structure.

Usual indications that a structure needs some type of repair or rehabilitation are summarised in USACE (2003) as when:

- damage has occurred from storms or other events such as vessel impacts or earthquakes
- periodic condition inspections indicate progressive deterioration to the point where functionality is jeopardised
- performance monitoring indicates the structure is not functioning as planned
- the structure is suffering chronic damage from underestimation of design loads
- the intended structure function is modified to provide new or enhanced service that was not originally in the design.

The decisions made in response to a monitoring report should be set against the performance and failure criteria that have been established at the design stage and with relation to the life cycle management policy (see Section 2.4.1). Performance and failure criteria can change as technical understanding develops and as the requirements for the function of a structure alter as a result of changes in use or safety standards. Data from the armour condition evaluation and the functional evaluation are used together with the project life cycle plan to produce recommendations for maintenance or additional inspection. Outputs from the armour damage evaluation and the functional requirements will provide a matrix of performance thresholds, measured damage and, possibly, prediction of further damage. One of the hardest questions to answer is, "When should a structure or coastal project be repaired or rehabilitated?". This, of course, depends on what functions are served by the project and how critical the structure is relative to other structures in need of repair. If any of the minimum performance thresholds, linked to life cycle cost plans, have been exceeded, then maintenance should be conducted. The following procedures are included in the decision-making process:

- review the life cycle management plan and amend if appropriate
- review previously established functional performance criteria and amend if appropriate
- compare armour condition evaluation results with the performance thresholds:
  - determine risk of failure (probability)
  - determine probability of risk of damage to infrastructure
  - define acceptable risks (Section 2.3.3.2)
- compare damage evaluation with performance trigger levels (Section 10.4.2.2) to identify the requirement for one of the following:
  - modified monitoring frequency
  - maintenance repair
  - rehabilitation
- predict useful residual life
- if trigger levels have been exceeded, plan maintenance works:
  - use damage evaluation to identify location and extent of required repairs
  - examine possible damage repair methods (Section 10.5)
- review the costs of required maintenance:
  - compare costs with available budget and revise works programme or budget if necessary

  - review the implications of deferring maintenance if the budget restricts repairs
  - revise advanced budget planning
- prioritise the maintenance programme:
  - critical repairs must be completed immediately
  - intervene where risk of failure or further damage is highest
  - intervene on those elements of structures that are likely to be damaged first
  - structures serving vital functions have a high likelihood of being repaired quickly, whereas less critical structures may continue in a damaged or deteriorated condition for many years until funding is available
  - less critical repairs, where loss of functionality is not great and additional damage is unlikely to occur, can be planned to suit budget requirements.

10.5 MAINTENANCE, REPAIR AND REHABILITATION

10.5.1 General maintenance considerations

Generic maintenance guidance on structure repair and rehabilitation is less well developed than for new project construction. This is because damage or deterioration is often localised and specific to the particular structure, which may require innovative approaches. This section provides generic guidance applicable to all rock structures.

10.5.1.1 Changes to design conditions

The actual design parameters (waves, water levels, storm frequency) are usually unchanged from the time of original construction to the time when repairs are needed. There may be exceptions where exposure to the wave climate has been altered (by construction of an offshore breakwater, for example) or where bathymetry has changed (eg growth of an ebb-shoal bar, profile deepening or steepening, toe erosion, sea level change). More reliable estimates of the design parameters may be available than for original construction or during the previous repair/rehabilitation. For example, several years of wave measurements may enhance wave climate statistics. These are often influenced by data for major storms thought to have caused damage.

Designers of repairs and rehabilitations should draw on whatever available knowledge exists about past project performance, including performance of similar structures. Monitoring data collected before damage occurs can be crucial to understanding why the structure was damaged and how to prevent a recurrence. New regulations or environmental restrictions may now apply that did not exist at the time of original construction. Consequently, it may not be feasible to repair or rehabilitate the structure with the same construction methods or materials. Similarly, design standards may have changed or may have been implemented for certain types of structures or structural components. For older structures, the original environmental forcing design criteria may be less significant, because the environmental forcing was probably not well characterised.

10.5.1.2 Basic principles of repair planning

Every repair or rehabilitation is unique. However, the general guidelines listed below apply to many structures.

Review the original design criteria, plans and specifications and identify the designer's main considerations. As-built drawings are especially important, because they document what was actually constructed, and identify any changes dictated by local conditions that were not identified during design.

Determine the cause of the problem. It may be obvious, such as a major storm, but sometimes the cause will not be so easily determined. For example, armour units might be lost through extreme events, by breakage of units into smaller pieces, or by slumping of the entire armour layer. Monitoring data will often provide valuable information. Examine the failure modes for the structure type and determine how the structure sustained damage. Remember that damage or failure may have been caused by a combination of circumstances rather than any single factor. If the true cause of damage is not identified, there is a risk of future damage occurring in the same manner. The structure's past history, as captured through condition monitoring, will provide details of repairs or modifications completed and the various repair methods that have been used.

If damage can be attributed to a single storm or series of storms, **estimate the severity of the events** by using available data and observations. Accurate estimates are critical in designing a repair that will withstand future events of similar strength.

Investigate the present structure relative to the as-built plans and locate discrepancies. This may help isolate problem areas as well as identify regions where future problems might develop.

Devise a solution for the problem. If possible, propose several solutions and prepare cost estimates for each. Costs associated with testing or optimising the final design should be included. For large repairs or rehabilitation, physical modelling will be a small fraction of the total cost, and the modelling should more than pay for itself by helping to optimise the design. For smaller structures, potential cost savings may not justify extensive laboratory testing.

Design a repair that solves the problem without extensive modification. If a structure has to be extensively modified to achieve functionality then it may have been poorly designed initially and perhaps should be completely reappraised by the designer. It is generally uneconomic to under-design a structure in such a way that the underlayer or core will be exposed when damage occurs. In this situation repairs are similar to a major rehabilitation and consequently are expensive. Economic maintenance procedures are gradually confined to repairs to the armour layer. In cases where low-grade, or sub-size material has been used in construction, expectations of damage and subsequent regular repair may be integral to the design and management process. The main considerations then become availability of (and access to) materials and plant.

10.5.1.3 *Reuse and supply of materials*

There are three options for armour supply for maintenance.

1 Reuse of existing armourstone.

2 Use of new armourstone.

3 Improved stability of present armourstone.

Since armourstone is reusable, repair work may only involve retrieving dislodged stones and placing them back into the face of the structure, where access is possible. The repair will need to ensure good interlocking, to reduce the possibility of damage recurring. In other situations new armourstone will be required for repairs. If provision for suitable access has been made at the design stage, and if it is economic, the armourstone can be imported as required. However, it is very expensive to import small quantities of additional armourstone, particularly if the source is remote. If it is apparent at the design and construction stage that access for haulage trucks will be difficult or impossible after construction, consideration should be given to stockpiling spare material at the site as part of the main initial construction operation (see Chapters 6, 7, 8 and 9). This armourstone can be placed in stockpiles, buried beneath fine material or used to mark access roads. Occasionally armourstone can be stored under water, eg close to the neck of a breakwater. Availability of materials and construction plant need to be taken into account. For example, a local quarry that produced the original armourstone may not be in operation and there might not be any other local suppliers of adequate armourstone. This may mean that concrete armour units have to be used instead of quarried rock.

10.5.1.4 Access and plant

Access for repair plant to, along and around the structure for repairs is crucial and should be considered when detailing or dimensioning new rock structures.

- Access for initial construction is often gained along the core or underlayer of a partly completed structure or by purpose-built and expensive temporary works. Once construction is complete these forms of access will no longer be available for repair works, which reduces the options for equipment and site access for repairs. For example, the original structure might have been constructed from an access road on top of the structure crest, whereas the repair might have to be accomplished from floating plant.
- Since previous work on the structure took place, development of the surrounding area may have significantly altered access to the site for construction and storing materials. This will influence the design by limiting construction sequence options.
- Concrete crown walls are sometimes included, partly to make provision for future maintenance. However, they are only of value in this regard if it can be guaranteed that they will not be damaged by storms.

Land-based equipment commonly used for repair of armour layers is listed in Table 10.14 together with comments on suitability and potential access constraints (see also Sections 9.3.2 and 9.3.3). Handling attachments are discussed in Table 10.15. For repair work, the *positive pick-up* and placing capability of a fixed-arm grapple often makes this the favoured attachment. This is because the grapple tends to allow more rapid working by enabling stones to be picked up easily from a stockpile or from a position already in the structure. It can also be used to push stones into position. Specialist floating plant may be required for maintenance of some structures; the techniques are similar to those described in Sections 9.3.4 and 9.3.5.

Table 10.14 *Construction equipment for land-based repair of armourstone layers*

Method	Access
Tracked hydraulic excavator	Suitable on soft beaches; also used to track over heavy armourstone on crest of structure with aid of an experienced banksman
Wheeled hydraulic excavator	Only suitable where hard access is available, the stones are relatively small and short reach required
Crawler crane	Suitable for remote areas of a structure where hard access is available above water
Jack-up on pontoon with crane or excavator	Suitable for sites that do not dry and where access is not available along the structure

1 2 3 4 5 6 7 8 9 10

Table 10.15 *Construction handling attachments for land based repair of armourstone layers*

Handling attachment	Comment
Bucket	Positive pick-up and placing; limitations on placing and moving; tendency to drop stones; better with armourstone < 2 t
Orange-peel grab	Non-positive pick-up and placing; difficult to pick up individual stones from face
Grapple grab	As above, for handling. Often the favoured attachment for placement of armourstone; attachment slower on crane than excavator
Buckets and skips	Applicable to areas where importing heavy armourstone is difficult; limited control
Chains and lifting eyes	Suitable where armourstone does not have to be moved far across the structure

10.5.2 Repair and rehabilitation of rock-armoured structures

Maintenance is considered in two main categories (USACE, 2003):

- **repair** – making good portions of a structure that have been damaged by waves, winds, currents, surges, impacts or seismic activity
- **rehabilitation or strengthening** – renovation of deteriorated structure components to original condition or upgrading the structure to withstand greater design loads.

Repairs of rubble mound structures generally consist of rebuilding the rock structure or replacing the armourstone with new material. In some cases repair can be achieved with concrete or asphalt grout. The following considerations distinguish the design of repairs to rubble mound structures from the design of new structures:

- repairs are made to an existing rubble mound structure that may have been damaged by storm action, resulting in either a lower crest or shallower slopes than originally built
- original armour may be mixed with underlayer armourstone
- changing armour slope to suit design parameters is difficult
- embedding and securing a new armour slope toe is more difficult than new construction
- transitions between the repair section and the existing undamaged slope must be accomplished without creating weaknesses in the armour layer
- repairs to armour slopes may involve mixing of sizes and types (overlaying armourstone with concrete armour units, for example)
- to begin repairs, it is usually necessary to remove part or all of a damaged armour slope and in some instances broken armour units must be removed. This will temporarily expose the underlayer, and during the repair it may be necessary to remove material from the site or stockpile it for reuse
- spot repairs to isolated damage on armour slopes require substantial mobilisation of equipment, and might have to be postponed in the absence of economical methods
- concrete and bitumen injection may be used for repairs, particularly if the existing armourstone size is considered to be too small for stability and larger armourstone is not readily available.

Depending on the extent of damage or deterioration on a rubble mound structure, repair options range from minor re-dressing of the primary armour layer to complete replacement of the structure. Pope (1992) listed the common options for repairing rubble mound structures shown in Table 10.16.

Table 10.16 *Options for repairing rubble mound structures (based on Pope, 1992)*

Problem area	Options (coastal structures)	Options (river and canal structures)
Slope and crest repair	• Chinking (adjustment of the armour surface by plugging with smaller material), resurfacing • Addition of armourstone • Layer reconstruction • Raising the crest • Burial of existing armourstone	• Chinking, resurfacing, addition of similar or dissimilar armourstone to fill holes above water • Screw race erosion has to be repaired by dumping a larger stone size on top of the eroded bank line • Cut bushes and trees regularly • Small erosion not repaired but the permitted maximum ship speed is lowered
Toe and foundation repair	• Reconstruction of the toe • Scour apron • Addition of a berm or toe trench	• New fascine mattress loaded with coarse stones or light armourstone
Core repair or void sealing	• Precast concrete blocks • Filter cloth (geotextile) • Grout	• Filter and subsoil reconstruction
Failure damage	• Replacement of original structure • Complete removal of existing structure	• Replace original structure • Entirely remove existing structure

10.5.3 Repair

10.5.3.1 Armour layer

There are four general categories of armour layer repair:

- spot replacement of broken or dislodged armour stones or concrete armour units
- overlaying existing armour layers
- replacing armour layers
- rebuilding the structure.

Design guidance for armour layer repairs is sparse and common sense rules of thumb are often applied, taking into account the unique aspects of each repair. Past repair experience on the same or similar structures provides valuable design input (USACE, 2003).

10.5.3.2 Spot or localised replacement of broken or dislodged armourstone or concrete armour units

When the primary armour layer has been damaged through displacement of individual armour units, and when the percentage of displaced units is below 5 per cent, it is often possible to repair the armour layer by replacing dislodged units with units of similar type and size (Groeneveld *et al*, 1985). It is acceptable practice to reuse displaced stones or units supplemented with new ones, provided that the old units are still sound and have not been broken into smaller pieces. If damage occurred as a result of forcing conditions that were similar to the design event this may be acceptable. If damage was caused by frequently occurring storm events, then repair with similar units may not provide adequate long-term protection (USACE, 2003).

Localised or *spot* replacement of dislodged armour stones or units is the least expensive of the repair options because it requires less time on site, material costs are lower and less rehandling of existing armour is needed; however, costs per unit replaced are very high because of the high mobilisation costs. Individual armour stones or units are generally dislodged in the vicinity of the still water line and repairs can be achieved using plant perched on the structure crest, if accessible, or by hydraulic machines that can gain access to the structure slope. Tracking of plant across large armourstone is difficult and is often limited to armourstone with an upper limit of about 6 t. Movements should be made with care, as excavator tracks may easily be damaged. For structures where crest access is unavailable or where building temporary construction roads is too expensive, repairs can be made from floating plant, but this is extremely expensive to mobilise unless plant is available locally.

Before repairs are started, loose armour stones or units should be removed from the damaged area. Voids above the still water level (SWL) are then filled with new armourstone. In practice, many units may need to be unpicked before a single armour stone can be replaced, making the process highly labour-intensive. Voids on either side of the SWL can be filled with new armourstone, and adjacent undisturbed armour stones reoriented to provide better interlocking. Local circumstances may dictate removal of a V-shaped section of the upper armour layer to facilitate such a repair while maintaining interlock, necessitating extensive removal of armour on a long slope. Alternatively, existing stones from the armour layer above the void can be used to fill the void, thereby progressively moving the void upslope to the crest, where new armour stones can be added (Ward and Markle, 1984).

This method helps ensure there is good contact between armour stones in the repaired area and in the slope above the repair. The main difficulties are obtaining armour stones of the right size and shape to repair the void, and maintaining good interlock when relaying the removed armour stones. This method also eliminates multiple handling of armourstone and the need to stockpile existing armour stones. Achieving armour interlock is critical for stability of the spot repair. If an existing damaged armour layer has good interlock in the undamaged portions, reseating or shifting those armour stones during spot repairs may introduce new weaknesses in the armour layer and reduce the armour stability. Similarly, to achieve a stable repair the new armour stones placed on the structure must be well seated to ensure maximum contact with surrounding stones (USACE, 2003).

Underwater spot repairs on slopes are extremely difficult to conduct. It is usually impractical to unpick the armour layer and rebuild to the design profile. Instead, armourstone is usually placed directly into and around the void area and interlock is achieved by making minor readjustment to the surrounding armourstone and adding more material. This often results in a localised variation to the design profile and in continued vulnerability. Spot repairs that require mobilisation of floating plant are very expensive and they are usually conducted only when there is considerable risk to structure stability or functionality.

The stability of interlocking concrete armour units is based on the interlock between adjacent units (USACE, 2003). Groeneveld *et al* (1985) recommend that, when repairing concrete armour unit layers, both damaged and undamaged units should be removed from the repair location all the way up the slope to the crest, and then replaced with undamaged units. This guarantees proper interlocking throughout the armoured slope. Turk and Melby (1997) suggest two methods for repairing concrete armour slopes:

- the **spot repair method**, which is used to repair a small cluster of broken armour units. Broken units are removed from the slope and replaced with new units. Because there is little handling of adjacent undamaged units, care must be taken to achieve good interlocking of the new units with the existing ones. Concrete armour units cannot generally be repaired and should be replaced.
- the **"*V-notch*" method** of repair, which is more extensive because armour is removed from the point of damage up the slope in a V-shape that widens as it approaches the structure crest. The notch is then filled in using either all new units or a combination of new and original armour units.

10.5.3.3 *Toes and berms*

The most effective way to repair a submerged berm crest is to add new material on top of the existing berm, which allows the berm to be restored to the design cross-section. If damage is minor and thought to be caused by exceedance of the design condition, the new material can be the same size or slightly larger than that placed originally. If the damage is severe, it may be necessary to redesign the berm using larger armour stones. It may be possible to place the new berm on the remnants of the damaged one, but this may require removal of some or all of the existing berm material. Scattered material may have to be removed if it interferes with navigation or other activities. Existing structures can be upgraded by adding a berm to reduce wave energy and increase stability of the primary armour layer. A berm can also decrease wave run-up and overtopping on structures that are not functioning as intended. Case histories of stability problems with armoured toes, as well as detailed design and repair guidance, are reported by Markle (1986). Toe stability is a major issue and often relates either to inadequate sizing and placement of armourstone at the toe, or to undermining of the toe berm by scour.

Toe instability caused by waves and currents scouring the bed adjacent to the structure toe is problematic. Additional toe material can be placed to rebuild a toe profile that has been degraded by materials falling into the scour hole, or a scour blanket can be constructed to protect the toe from damage by scour. A third, more expensive, solution is to excavate and reconstruct the structure toe. If this course is adopted, care must be taken not to initiate slope failure of the main armour layer during repair. It may be advantageous to reconstruct the damaged toe berm to a larger size than originally designed in order to reduce run-up and overtopping and to increase armour stability.

Some designs include provisions for scour protection, usually in the form of a rock apron extending some distance from the structure toe. Scour protection is more often added to a structure after monitoring has revealed that scour holes or trenches have been formed by currents, waves or a combination of both. For example, where scour has undermined the structure toe berm, the repair plan should include some type of scour protection to prevent recurrence. Prediction of scour is difficult, with past experience being the best gauge. There is little difference between designing scour protection as part of a structure repair or rehabilitation and designing the protection for a new structure. After scour has occurred, one of the main decisions is whether or not to fill in the scour hole before placing a protective armourstone blanket. If the scour hole is close to the structure toe and has relatively steep side slopes, there is a risk of the toe falling into the scour hole, either by armour slope failure or slip-circle failure. This presents a difficult design decision because no guidance exists for armour slope instability relative to scour hole side slopes adjacent to the structure toe. In addition, deep scour holes indicate strong local currents – filling in these scour holes may substantially increase currents and cause scour in adjacent, unprotected portions of the bed.

10.5.3.4 *Void sealing*

Rubble mound structures have a degree of permeability that varies significantly with cross-section design. This permeability absorbs wave energy, reduces wave run-up and overtopping, decreases wave reflection and generally enhances armour layer stability. Where rock structures are placed within active littoral zones sand can flow through permeable rubble mounds and deposit in shoals on the lee side of the structure. Permeability of newly constructed rubble mound structures may decrease slightly as the structure settles and stones shift into the voids. Conversely, rubble mound permeability can increase over time if smaller core material is washed out of the structure by wave action or if portions of the armour layer suffer damage. Where structure permeability causes problems it may be possible to decrease it by *void sealing*, in which grout or a sealant is injected into the structure to fill the voids (USACE, 2003). Using techniques developed in civil and mining engineering, it is possible to fill interior voids of up to 1 m in diameter but, lacking long-term field experience in the aquatic environment, the longevity of grouts and sealants placed in rock structures is unknown.

10.5.4 Major rehabilitation and strengthening

10.5.4.1 Overlaying damaged armour layers

If widespread armour layer damage occurs that causes large sections of armour to be displaced or to slump, it may be feasible to repair the structure by adding an overlay consisting of similar or dissimilar units. Overlays can also be used to increase crest elevation to reduce overtopping or to flatten the armour slope for improved stability. Constructing an overlay is expensive because of the quantity of new armour required but is less expensive than replacing the armour layer completely.

The structure cross-section can be degraded when armour stones are dislodged, underlayer materials are lost, or toe failure and slumping of the slope armour occurs. In some cases the resulting structure cross-section will form at a lower crest elevation and with shallower side slopes than originally built. Overlay design requires an understanding of the cause of damage. If it results from armourstone instability, the overlay will need to consist of armour stones (or units) capable of resisting the wave loading; this implies overlaying armourstone that is larger than the original armour or placed at a flatter slope. Where sufficiently large stone is unavailable, concrete armour units are the only option (see Section 3.12 and Section 6.1).

Armourstone overlays need to be checked as described below.

Single-layer armourstone overlays

There are no established stability coefficients that can be used in stability formulae for single-layer armourstone overlays placed on existing structures. It is inappropriate to use published stability coefficients intended either for two-layer armour layers or for single-layer new construction. Physical model tests should be used to optimise a stable one-layer overlay design. Wolf (1989) gives an overview of armour construction using a single layer of armourstone and provides general guidance on armourstone placement and stability, although this is not the same as an overlay.

Two-layer armourstone overlays

Most stability coefficients for armourstone are based on two-layer design, and these coefficients should be adequate for two-layer overlays provided sufficient care is taken to create a stable interface with the underlying existing armour slope. For large structures, physical model tests are warranted.

Overlays using dissimilar armour units

Typically, this refers to overlaying an existing rock-armoured structure with an armour layer composed of concrete armour units. The aim is to improve stability through better interlocking, greater mass (cubes), or a combination of both. Experiences based on the use of dissimilar armour units for repairs are reported by Carver (1989). In all cases design of the overlay was based on design guidance for new construction, evaluation of model tests of similar structures, site-specific model tests, engineering judgement or prototype experience.

Armour interface with existing armour

During placement, care is needed to maximise interlocking between the new armour layer and the existing armour layer beneath. Typically, the profile of the underlying armourstone will be irregular. In some places the new armour stones will be resting directly on existing armour, but elsewhere additional underlayer amourstone may be needed to restore the existing slope to a uniform grade. Construction of underwater portions of armour slopes is always difficult, but this difficulty is compounded when the existing slope is irregular. Care must be taken to ensure the underwater portion of the armour overlay is reasonably uniform and free of gaps.

Lee-side crest armour units

If heavy overtopping of the new overlay or transmission through the structure is expected, care should be taken to securely key-in to the existing structure the lee-side crest armour units, as if these are lost the crest could unravel.

Overlay toe

The new overlay toe should be securely positioned and adequately protected, which may require construction of a new toe berm or excavation of a toe trench for a shallow-water structure. Difficulties arise where dislodged armour stones litter the toe area. Some of these displaced armour stones may need to be removed before work starts on construction of the new toe.

Construction methods

Constructing an overlay is similar to new construction in that armourstone placement begins at the toe and proceeds upslope. Where the existing slope is irregular, extra effort is needed to achieve good interlocking between the overlay and existing armour. In some cases it may be necessary to remove or relocate existing armour units.

Geotechnical stability

It is essential to make allowance for additional loads that were not considered during the original design.

Hydrodynamic stability

Higher structures might incur higher loads.

10.5.4.2 Replacement of armour layer

A more expensive alternative to constructing an overlay is to entirely replace or rebuild the armour layer over a portion of the structure. Where the original armourstone has proven to be inadequate either structurally or functionally the armour layer may have to be replaced. Replacement is also a possibility where there are excessive broken armour stones or units, undersized materials, excessive wave overtopping or excessive wave transmission. Rebuilding the armour layer is only advisable when it can be determined that damage was caused by something other than armour instability, such as faulty construction or seismic events.

Although very expensive, armour layer replacement or rebuilding is justified if the cost is projected to be less than future maintenance costs (based on past performance). Replacement armour layers should be designed using the guidance available for new construction (see Chapters 6, 7 and 8). The existing underlayer on which the new armour will be placed should be of a standard that will prevent loss of underlayer through voids in the primary armour. Construction of replacement armour layers requires removal of all existing armour units and their substitution by new armour units to the revised design. Construction typically begins at the toe and works up the slope.

Disposing of the original armour entails substantial expense. If possible, the old armour stones or units should be recycled to minimise handling costs. One option might be to place damaged and undersized armour stones or units at the toe of the structure, to create an elevated berm that serves as a base for the new armour layer. Broken pieces of armourstone or units (especially rounded concrete pieces) should generally not be used to fill in as underlayer material.

A construction plan should be devised for removing the existing armourstone over a section of the structure, then replacing the armour units starting with the toe and working upslope. Costs can be controlled through efficient stockpiling of armour units and minimising rehandling of armour. Armour layer rebuilding or replacing is similar to new construction, except that the core and underlayer already exist. It may be necessary to replace, add to or adjust portions of the underlayer to accept the new armour layer. The underlayer should be checked for correct thickness and compaction. If the new armour layer consists of much larger armour units, the armourstone grading in the underlayer may have to be increased to avoid loss of material through the voids. The slope of the new primary armour layer can be reduced, by placing additional underlayer armourstone on the existing slope.

10.5.4.3 Reconstruction of rock structures

Structures that sustain catastrophic damage, where the integrity of the structure has been lost, or where repair can only be realised through a major redesign, will need to be entirely rebuilt. If the structure function requires reconstruction at the same location, it may be necessary to bury or completely remove the existing structure. Burial of an existing structure will probably result in a larger cross-section (Pope, 1992). In cases where the same functionality can be achieved by building a new structure adjacent to the damaged one, it may be possible to abandon the old structure. Design of replacement structures follows the same guidance as new construction.

If the new structure is to be placed over the remnants of the existing one, special attention is needed to prepare the existing structure to serve as the rubble base of the new construction. This may involve removing material, preparing a new toe and placing new bedding material. Removing materials from the old structure is a major expense and consideration should be given to the possible reuse of the material in the new construction.

10.5.5 Armoured structure repair case histories

Repairs to rock- and concrete-armoured structures are widespread, but only limited documentation of maintenance activities is available worldwide. Lessons learned from previous work on the same or similar structures are a valuable source of ideas and methodologies that can be drawn on when a structure needs repair or maintenance. Nevertheless, each structure is unique in its location, exposure, construction and intended functionality, so it is important not to adopt a repair procedure without first carefully evaluating all these aspects. Inevitably those structures suffering high or regular damage are better documented in this respect.

Repair case histories describing various approaches are summarised in Boxes 10.8, 10.9, 10.10 and 10.11.

Box 10.8 *Life cycle maintenance and management of low-cost structures*

Rock groynes were constructed in 1992 to replace life-expired timber groynes at the base of an unconsolidated sandy clay cliff in Christchurch Bay, UK (see Figures 10.16 and 10.17). To minimise the cost while maintaining the existing groyne positions and spacing, alternating long (80 m) and short (60 m) rock groynes were used to retain the required width of shingle beach. The groynes were designed to make use of locally available Purbeck limestone (of 1–7 t), which is readily available and relatively inexpensive, but does deteriorate in exposed coastal conditions. The crest width was designed to allow access for construction plant (armourstone is delivered by trucks, which reverse along the length of the groyne with smaller stones temporarily placed on the crest as kerbing for safety) and the side slopes at 1:2 are as steep as is practicable for stability. Concrete tripods were originally used to provide a stable toe, but were found to be unnecessary, so in structures constructed later selected larger armour stones were used as *toe stones*. The groynes were built directly on the thin sand beach (approximately 0.3 m thick at the head of the structures) and some material (particularly the concrete tripods) quickly settled.

Figure 10.16

Low-cost groynes at Highcliffe (courtesy A P Bradbury)

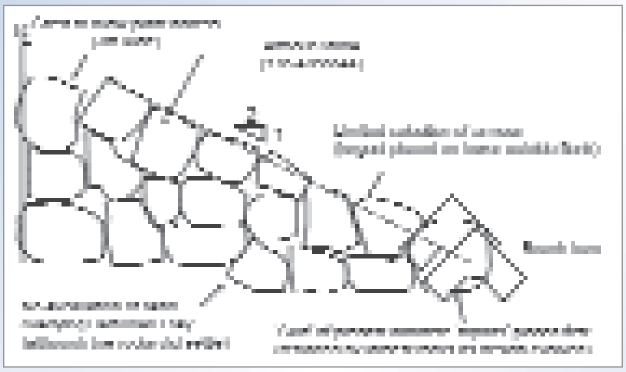


Figure 10.17

Cross-section of rock groynes at Highcliffe

The groynes are inspected every month and maintenance works (predominantly for public safety) cost approximately 0.5 per cent of the present value of capital works per year, which includes the importation of limited quantities of armourstone to compensate for deterioration. The performance is considered to be appropriate to the location and constraints. Experienced local authority engineers manage the monitoring and maintenance programme. To allow direct comparison between the costs and benefits of low-cost rock groynes with other project options, future values are discounted to present values (see Figure 10.18). UK guidance recommends that a discounted benefit cost analysis is examined over 50 years using an annual 3.5 per cent discount multiplier (Bradbury *et al*, 2004). For this, the economic evaluation should consider both capital costs and anticipated maintenance (life cycle) costs. Innovative structures demand a knowledge of the impacts and costs of relaxing conventional guidance and an understanding of the implications of simplified design. Because of the discount rates used in economic analysis, projects with lower capital cost and significant maintenance in latter years, are often of economic benefit. The example of life cycle costing and benefits of low-cost approaches demonstrates that over a 50-year design life, the low-cost non-standard rock groyne construction is expected to be more economical than conventional structures.

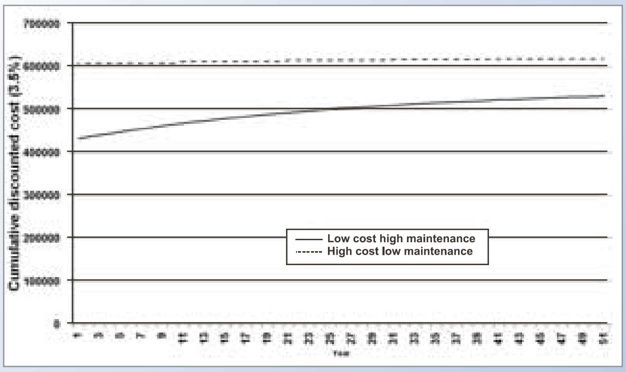


Figure 10.18

Discounted cost over project life cycle

Box 10.9 *Functional management of groynes in the River Waal*

The River Waal in the Netherlands contains about 800 groynes over a length of almost 100 km. Most of the groynes were built between 1900 and 1925, and have a mean length of 85 m, a mean height of 6 m and a mean width of 2.5 m. In many cases the upper half of the structure is of pitched stone and the lower half of armourstone. The management aim is to maintain them efficiently so that the groynes fulfil their function – providing sufficient water depth for navigation during low-discharge periods – at minimum cost. A maintenance plan describes timing, purpose and nature of maintenance measures. Systems analysis was applied to identify elements that are critical for failure (loss of functionality) and priority given to monitoring those elements. They can be selected by criteria such as the cost of maintaining the element, the cost of failure of the element and the timing and frequency of intervention. The critical elements that are predominant for failure are the groyne head and the groyne trunk (exposed to sliding due to scour) and the root of the groyne exposed to outflanking, ie development of a channel along the root of the groyne, causing it to become detached (see Figure 10.19).

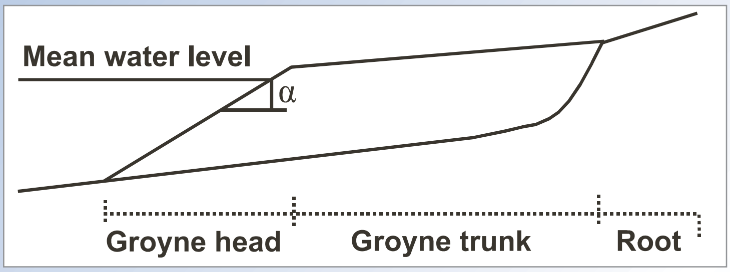


Figure 10.19 *Longitudinal section of a groyne*

The groyne head can fail through steepening of the slope, which can cause the head to slide and the channel to become shallower, thereby impeding navigation. For an objective inspection of the critical elements, parameters need to be formulated to monitor the state of repair. The condition of the head is monitored by inspections and the parameter is the slope angle, α (see Figure 10.19).

The maintenance and inspection plan contains the optimal schedule of all inspections, maintenance measures and replacements and the costs for a certain period. The inspection interval decreases as the condition of the element deteriorates, ie as the slope steepens. When the intervention level is reached the groyne head is maintained by dumping stones, or repairing or *washing in* of pitched stone (filling gaps between stones with gravel to maintain structural integrity and inter-stone friction). This maintenance is carried out at defined intervals. Table 10.17 summarises the maintenance and inspection plan and can be used to determine the required budget for a particular period.

Table 10.17 *Summary of maintenance and inspection plan*

Element	Inspection parameter	Inspection interval	Intervention level	Measure	Intervention interval
Groyne head	Slope angle	Two years (or more if no damage)	Slope angle 1:1.5 in outer bend Slope angle 1:1 in inner bend > 4 m^2 damaged	Dumping stone; repairing or washing-in of pitched stone	14 years
Groyne trunk	Slope angle	Two years (or more if no damage)	Slope angle 1:1.5 in outer bend Slope angle 1:1 in inner bend > 4 m^2 damaged	Dumping stone; repairing or washing-in of pitched stone	14 years
Root of groyne	Depth and length of outflanking channel	Depends on value of inspection parameter	Depth of channel < 1 m and length < 5 m Depth of channel > 1 m and length > 5 m	Dumping stone; washing-in of pitched stone Place timber sheet-piling, geotextile and stone	56 years

Box 10.10 *Implementation of a master plan for the maintenance of the embankment protection on the developed River Rhône*

The multi-purpose development of the River Rhône between Lyon and the Mediterranean was carried out between the late 1940s and 1980. It required the installation of rip-rap to protect river embankments from loads due to hydraulic conditions (current and wave attack) and weather conditions (freeze-thaw). These conditions led to deterioration of the rip-rap, which has required increasing levels of maintenance. The control of the maintenance and repair budget, combined with the increased security requirement of the installations, drove Compagnie Nationale du Rhône (CNR) to undertake a comprehensive diagnostic study that delivered a master plan for embankment maintenance.

The studies were organised in two phases: an evaluation of the current state of embankments, including evaluation of the risks, and a post-analysis stability modelling to locate areas at risk of instability. The master plan includes an evaluation of various areas of banks:

- areas to be reinforced immediately
- areas to be monitored
- areas without problem.

The actual size of the rip-rap protective layer that had been in place for 20–50 years was found to be inadequate for 94 of the 244 km of embankment analysed (Figure 10.20). Comparisons with design requirements, indicated problems related to currents, wind waves and ship waves. A major problem also arises from the in-service evolutionary geological properties of the rocks, such as gneiss or weatherable and frost-sensitive limestone. Both result in steady long-term degradation, and particularly fracture of the stones. This study resulted in development of an accelerated armourstone ageing test to improve diagnosis and prediction of armourstone evolution. The specifications for mechanical tests on rip-rap were refined and hydraulic calculation methods were improved with the aid of small-scale experiments including evaluation of bank protection on curves, and computation of the impacts of ship waves.

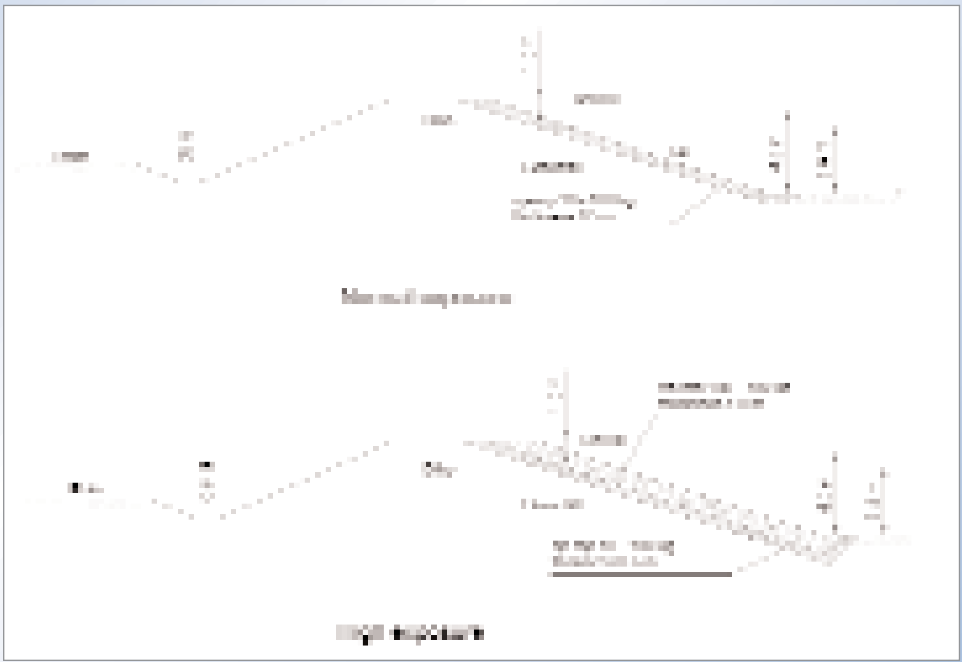


Figure 10.20 *Modifications to bank protection on the River Rhône*

1 2 3 4 5 6 7 8 9 10

Box 10.11 *Functional management of the breakwaters at IJmuiden*

The initial idea for the functional management of the breakwaters at IJmuiden (Netherlands) was to apply the 10-step system (see Table 10.2), but theory and practice do not always coincide. At the time of writing, the system analysis in step 6 has led to a demand for more research and analysis of structure behaviour before intervention levels, inspection strategy and plan can be determined. Maintenance of the structures over the past 35 years was evaluated and modified into a functional management model.

In the 1970s the breakwaters were extended to cope with increasing navigation intensity and larger ships on the navigation route. An innovative design was used for the new breakwaters, which incorporated a stone asphalt cover layer of more than 2 m on both seaward and harbour side slopes of the structure (see Figure 10.21). However, this layer is badly affected by wave attack on the breakwaters. Soon after construction large concrete cubes were placed on top of the asphalt layer to provide extra mass to prevent the asphalt from being pushed up from below by large wave pressures. The cubes were not stable under storm conditions and maintenance of the breakwaters since initial construction has been substantial. The main cost aspects were repair of the stone asphalt layer and replacement of displaced cubes. Another maintenance cost was the levelling of the crest units to form an accessible road.

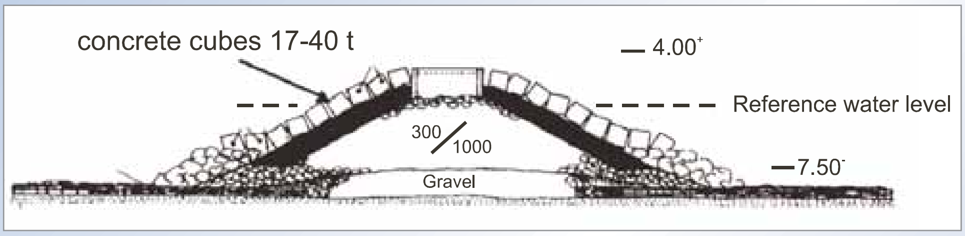


Figure 10.21 *Modification after construction: adding concrete cubes*

The following conclusions were derived:

- on a national level, the infrastructure system has the function of a main transport route
- the breakwaters should at least maintain the functions they have performed until now
- under extreme storm conditions the breakwaters act as low-crested structures and wave transmission is large. The crest should remain at its current height to maintain similar transmission behaviour
- the structure should be stable under severe storm conditions. While some damage is acceptable, the integrity of the structure should remain. Limits for acceptable damage are difficult to define
- damage to the structure will not directly influence its functioning with respect to the layout. Only a (large) breakthrough will change currents and wave transmission and will not be acceptable. Acceptable damage is established from required minimum maintenance and repair costs
- the main parts of the breakwaters to be considered in a management plan are the asphalt slopes, the cubes on these slopes and the crest element
- even 35 years after construction, the crest element has been very stable and hardly any settlement has occurred. The main problem since initial construction was the stability of the asphalt slopes and, after placement of extra cubes, the stability of these cubes. Until now the slopes and cubes have not met the target situation of a stable breakwater, at least from the point of view of limited maintenance
- the problems regarding maintenance of the breakwaters require a change of maintenance strategy. An optimal strategy must be formulated to minimise maintenance costs and fulfil the functional requirements of the breakwaters, as described above. To identify an optimal strategy the most important causes of observed damage must be identified and alternatives to prevent damage must be developed. Only then will it be possible to describe how the functions of the breakwaters can most effectively be maintained through definition of inspection parameters, intervention levels, inspection strategy and the formulation of a maintenance and inspection plan
- this conclusion also means that the development of the 10-step procedure given in Table 10.2 cannot be fulfilled until the technical research for a more stable structure has resulted in a solution. At this stage it is not possible to complete the management plan.

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