COASTAL

ENGINEERING

1: COASTAL ENGINEERING

General Introduction and Definitions

1.1 Coastal Engineering

Coastal engineering is the study of the processes ongoing at the shoreline and construction within the coastal zone. The field involves aspects of near shore oceanography, marine geology, and civil engineering, often directed at combating erosion of coasts or providing navigational access.

1.2 Coastal Zone

Coastal zone is the interface where the land meets the ocean encompassing shore line environment as well as adjacent coastal waters. Its components can include river deltas, coastal planes, wet lands, beaches, reefs, mangrove forest, lagoons and other coastal features. The coastal zone is divided into four subzones as show in figure (1-1) as follow:

- **Coast** The coast is a narrow zone where the land meets the sea. It is constantly changing due to the effect of land, marine and air processes.

- **Shore or a Shoreline** is the fringe of land at the edge of a large body of water, such as an ocean, sea, or lake.

- **Shoreface** is the zone where offshore generated waves interact with the upward sloping seabed, in front of the shoreline. The width of sandy shorefaces typically ranges between a few hundred meters and a few kilometers and the average slope between 1/20 and 1/200

- **Continental shelf** is a portion of a continent that is submerged under an area of relatively shallow water known as a shelf sea. The shelf surrounding an island is known as an insular shelf.

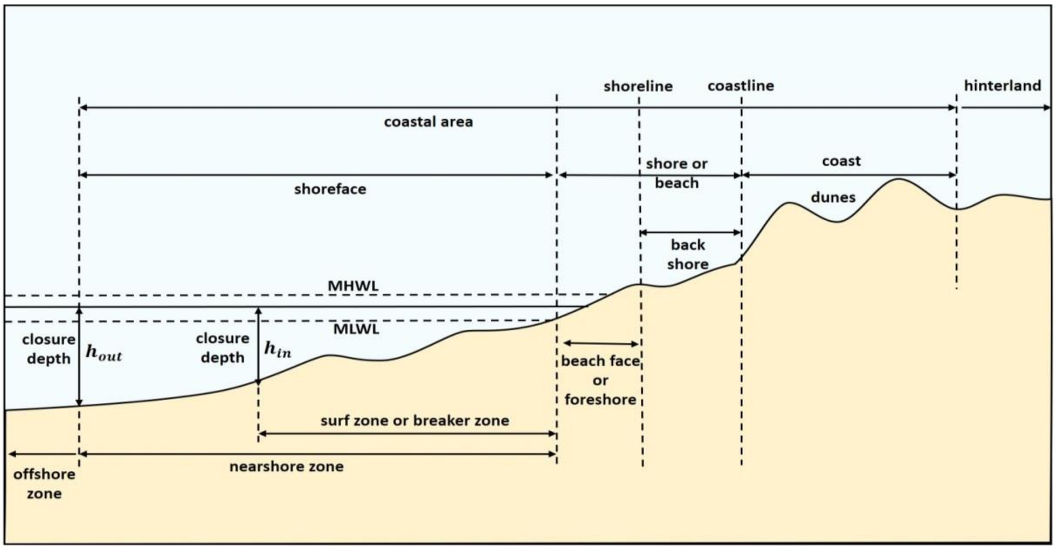


Figure 1-1 Coastal Zones

1.3 Coastal Regulation Zone

Coastal Stretches of creeks, bays, seas, rivers and backwaters that are affected by the tidal actions of up to 500 meters from the High Tide Line and the land between the Low Tide Line and the High Tide line are Coastal Regulation Zones (CRZ). As per the notification, coastal areas are of four categories as CRZ-1, CRZ-2, CRZ-3 and CRZ-4.

CRZ-1: These are ecologically sensitive areas which are essential in maintaining the ecosystem of the coasts as show in figure (1-2). These include national parks/marine parks, sanctuaries, reserve forests, wildlife habitats, mangroves and corals/coral reefs. These areas are situated between the high and low tide lines.

CRZ-2: The areas that have already developed up till the shoreline of the coast are included in this zone. Construction of unauthorized structures is prohibited in this zone.

CRZ-3: Rural and urban localities that are relatively undisturbed and do not belong to the first two categories are included under CRZ-3. Only specific activities related to agriculture or some public facilities are allowed in this zone. It includes areas within municipal limits or in legally designated urban areas that are not substantially built-up.

CRZ-4: These areas include the coastal stretches in Lakshadweep, the Andaman and Nicobar Islands and some other small islands, except those termed as CRZ-I, CRZ-II, or CRZ-III. These areas reside in the aquatic region up to the territorial limits. Activities such as fishing and other allied services are permitted in this zone. Releasing solid waste is prohibited on such land.

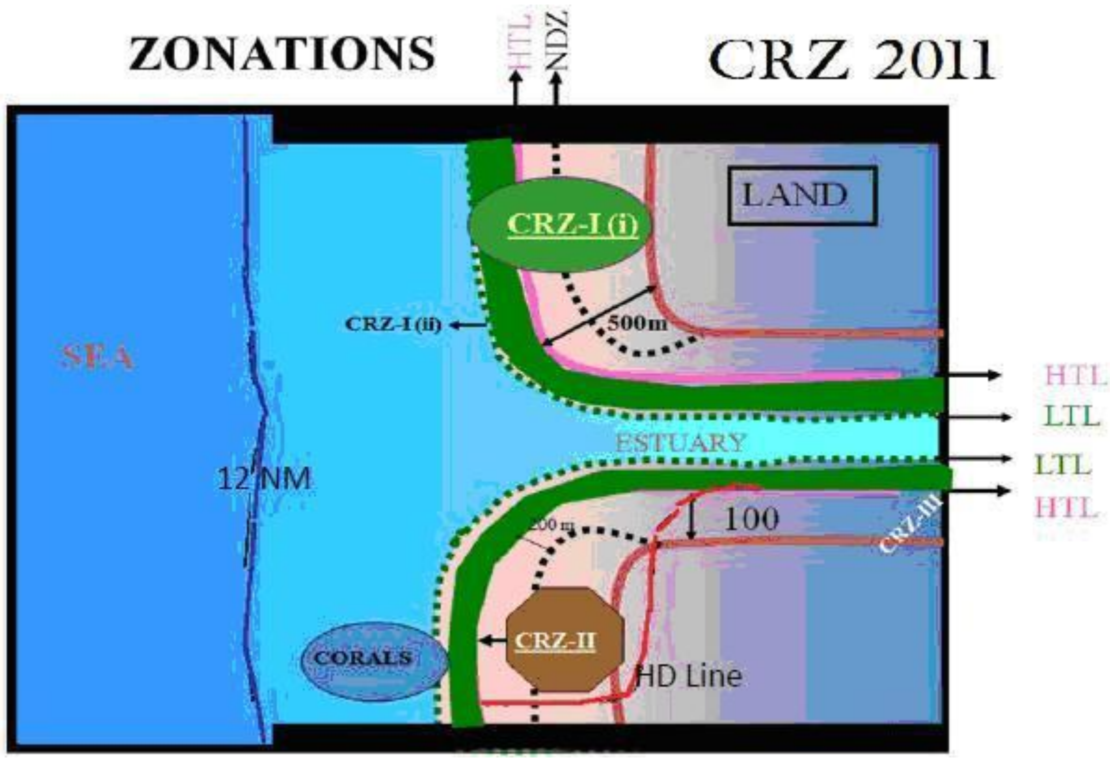


Figure 1-2 Coastal Regulation Zones

1.4 Coastal Type

The coastal type function of the coastal system Coastlines comprises the natural boundary zone between the land and the ocean. Their natural features depend on the type of rocks exposed along the coastline, the action of natural processes and the work of vegetation and animals. The intensity of natural processes formed their origin - either as erosional or depositional features. The geological composition of a coastal region determines the stability of the soil, as well as the degree of rocky materials and their breakdown and removal.

1.4.1 Cliff Coast

Cliff coast can be classified as hard coast as it was formed from resistant materials such as sedimentary or volcanic rocks. This type of coast typically has a short shore platform that is usually exposed during low tide. Natural erosion is attributable to slope instability, weathering and wave action and leads to regression of the shoreline. As illustrated in Figure (1-3), extreme wave conditions such as storm waves and tsunamis will have a less erosive effect on this type of coast; traces of tsunami wave height can be found on cliffs as a trim line where trees or shrubs on the cliff had been erased.

Figure (1-3) a Cliff Coast

Figure (1-3) b Cliff Coast

1.4.2 Clayey Bank Coast

This type of coast can be classified as a “semi-hard” coast, consisting of cohesive soils; it is common on estuarine coastlines and often has nearly vertical banks ranging from one to five meter in height. The rate of erosion is relatively high compared to the hard coast because it is composed of weaker and less resistant material. Erosion is mostly due to coastal processes, weathering and loss of vegetation cover (ARC, 2000). For extreme events such storms and tsunami, as illustrated in Figure (1-4), vegetation cover plays a significant role in protecting the coast from flooding and inundation by reducing wave height and energy and decelerating tsunami flow speed; hence, erosive forces and inundation distance are decreased.

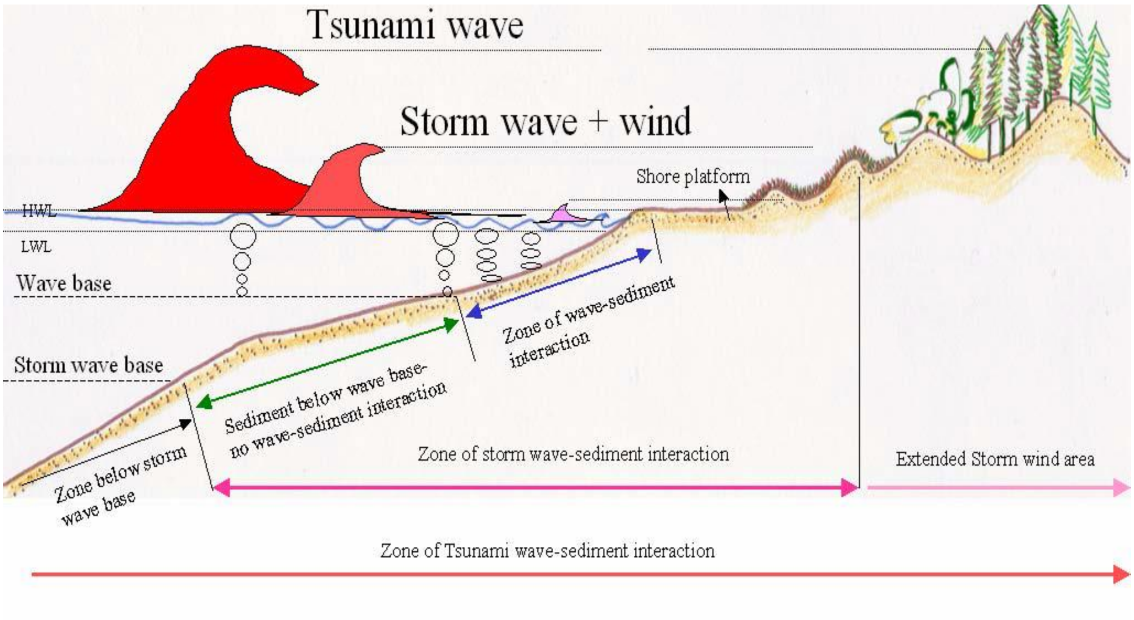


Figure (1-4) Clayey bank type coast

1.4.3 Intertidal/Muddy Coast

This type of coast is characterized by fine-grained sedimentary deposits, predominantly silt and clay that come from rivers; it can be classified as a "soft" coast. It has a broad gentle seaward slope, known as an intertidal mud flat where mangrove forest, saltmarshes, shrubs and other trees are found. Most erosion is generated by river damming that reduces sediment supply, diminishes vegetation cover (usually mangroves and saltmarshes) and exposes vegetation roots by lowering the mud flat Figure (1-5) that leads to their final collapse. During storms, healthy and dense vegetation/coastal forest and trees can serve as barriers and reduce storm wave height, as well as affording some protection to the area behind them. In the case of a tsunami, coastal forest and trees can decrease wave height and tsunami flow speed to some extent if the forest is dense and wide enough. Both extreme events can cause severe erosion and scouring on the coast and at the river mouth.

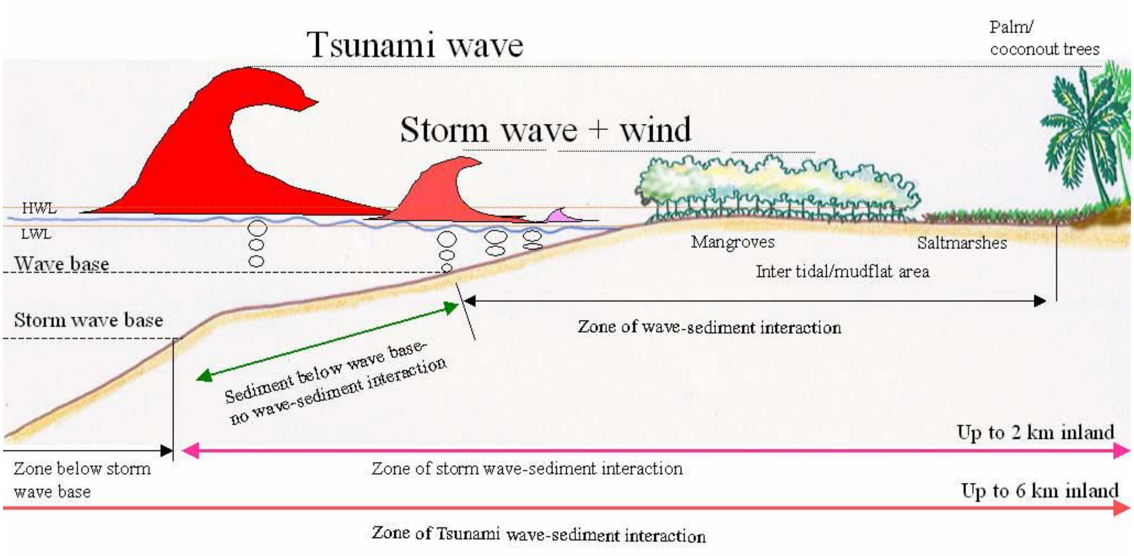


Figure (1-5) Intertidal/muddy coast

1.4.4 Sand Dune Coast

This type of coast consists of unconsolidated material, mainly sand, some pebbles and shells; it can be classified as a soft coast. It has a gentle seaward slope - known as dissipative beaches that have broad fine sand and gradually steep slopes at the backshore/foredunes. Its profile depends on wave form and energy and wind direction; hence, profiles can be adjusted to provide the most efficient means of dissipating incoming wave energy. This type of coast experiences short-term fluctuation or cyclic erosion - accretion and long-term assessment is needed to identify erosion as a problem here. Often accretion and dune rebuilding take much longer than erosional events and the beach has insufficient time to rebuild before the next erosive event occurs. Erosional features are a lowered beach face slope and the absence of a nearshore bar, berm and erosional scarps along the foredune. Generally, erosion is a problem when the sand dunes completely lose their vegetation cover that traps wind-borne sediment during rebuilding, improves slope stability and consolidates the sand. During extreme events such as storms and tsunamis Figure (1-6), this type of coast can act as a barrier for the area behind the dunes. Sand dunes and their vegetation cover are the best natural protective measures against coastal flooding and tsunami inundation.

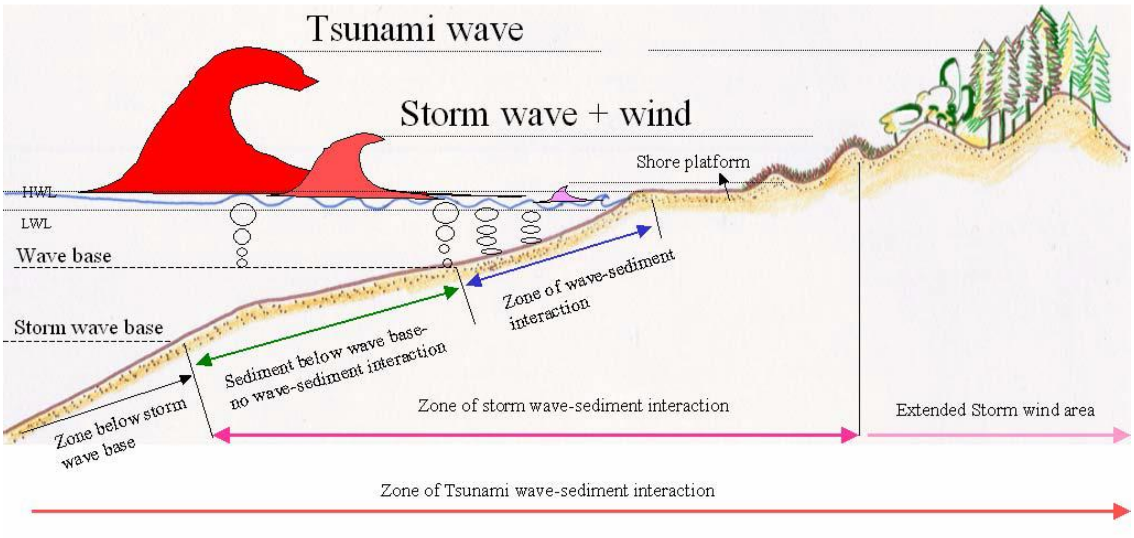


Figure (1-6) Sand dune

1.4.5 Sandy Coast

This type of coast consists of unconsolidated material - mainly sand from rivers and eroded headlands, broken coral branches (coralline sand) and shells from the fringing reefs. It can be classified as a soft coast with reef protection offshore. The beach slope varies from gentle to steep slopes depending on the intensity of natural forces (mainly waves) acting on them. Coconut trees, waru (Hibiscus tiliaceus), Casuarina catappa, pandanus, pine trees and other beach woodland trees are common here. Most erosion is caused by loss of the following:

1. The protective function of the coastal habitat, especially coral reefs (where they are found) that protect the coast from wave action.
2. Coastal trees that protect the coast from strong winds. During extreme events.

Figure (1-7), healthy coral reefs and trees protect coasts to some extent by reducing wave height and energy as well as severe coastal erosion.

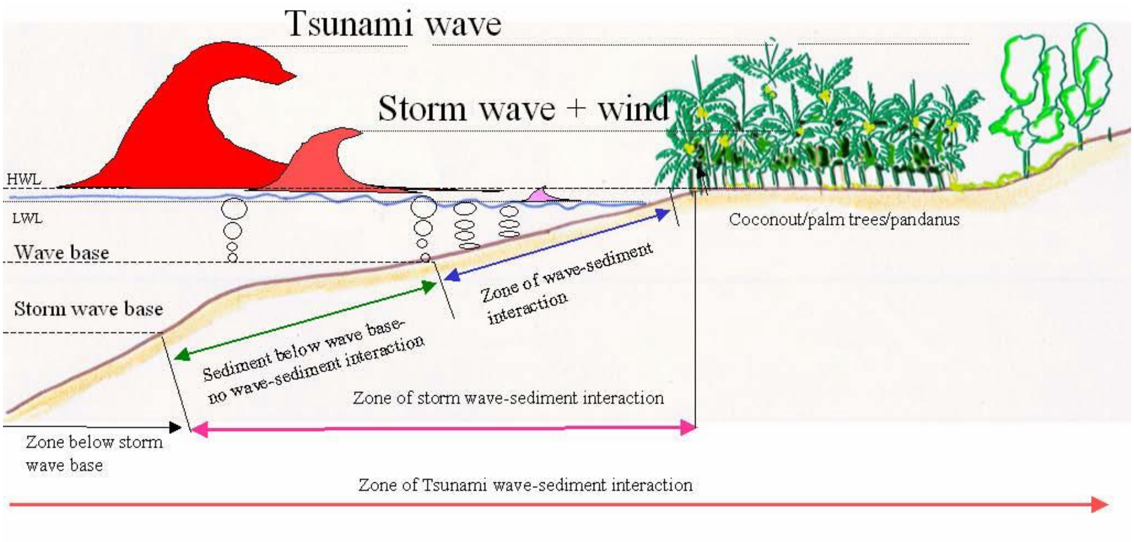


Figure (1-7) Sandy coast

2: Coastal Erosion: Extent and Causes

Coastal erosion and accretion are natural processes; however, they have become anomalous and widespread in the coastal zone of Asia and other countries in the Indian Ocean owing to combinations of various natural forces, population growth and unmanaged economic development along the coast, within river catchments and offshore.

2.1 The Causes of Coastal Erosion

Coastal erosion and accretion are complex processes that need to be investigated from the angles of sediment motion under wind, wave and tidal current action; beach dynamics within a sediment/littoral cell; and human activities along the coast, within river catchments and watersheds and offshore, both at spatial and temporal scales. In terms of temporal scales, the issue of sea-level rise is complex and produces a range of environmental problems. As the sea level rises, the water depth increases and the wave base becomes deeper; waves reaching the coast have more energy and therefore can erode and transport greater quantities of sediment. Thus, the coast starts to adjust to the new sea level to maintain a dynamic equilibrium.

Figure (2-1) lists the processes of coastal erosion and accretion, as well as natural factors and human activities.

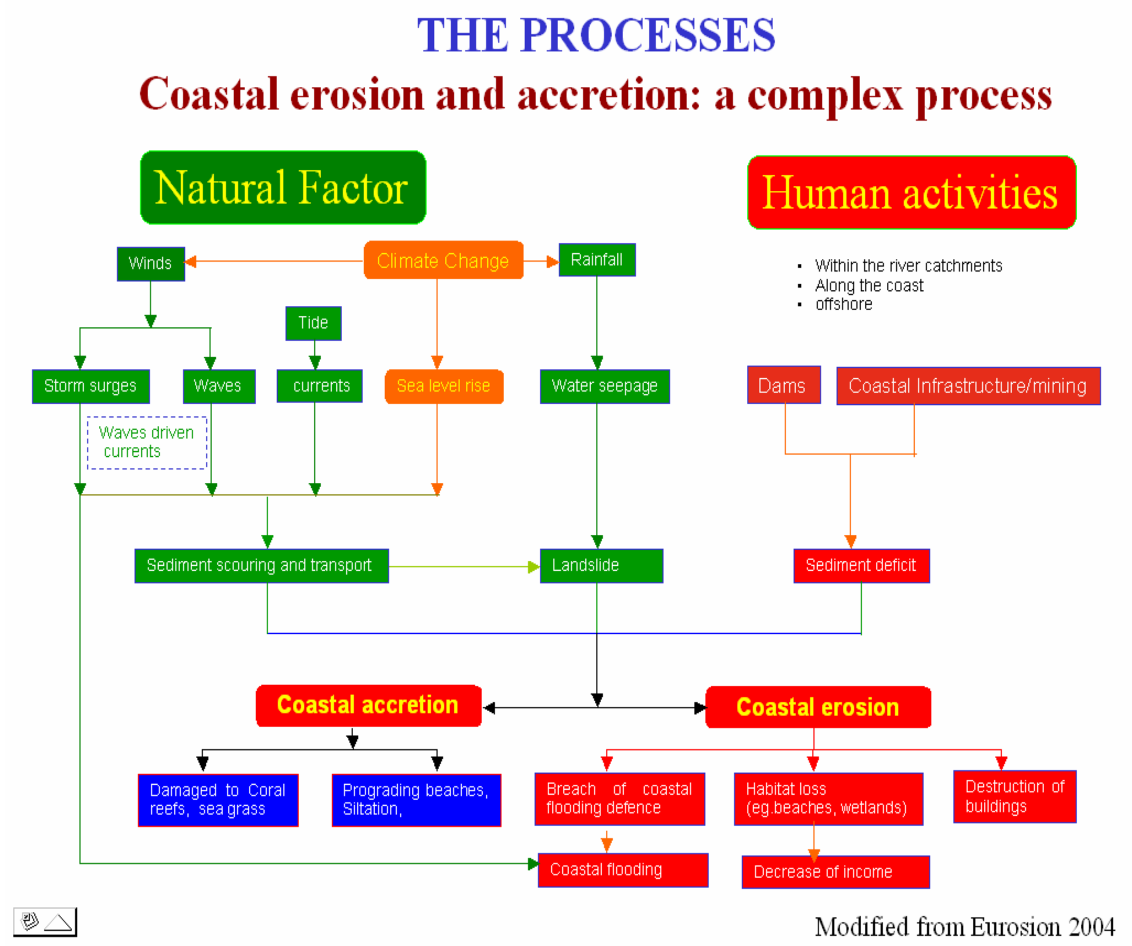


Figure (2-1) the complex processes of coastal erosion and accretion

The key physical parameters that need to be understood to identify coastal erosion as a problem in the coastal zone are:

2.1.1 Coastal Geomorphology

It's mean by coastline type and sensitivity to coastal processes.

2.1.2 Wind

The main force in wave generation; under the right environmental conditions, wind may transfer sediment from the beach environment landward on all open coastlines.

2.1.3 Waves

They are the most important forces for sediment erosion and transport to the coastal zone. They introduce energy to the coast and also a series of currents that move sediment along the shore (longshore drift) and normal to the shore (cross-shore transport). It is important to understand the movement of wave forms as well as water particles and their interaction with seabed material; also how the waves determine whether the coasts are erosive or accretional.

2.1.4 Tides

They are influential in beach morphodynamics. They modulate wave action, controlling energy arriving on the coast and drive groundwater fluctuation and tidal currents.

The interaction of groundwater with tides in the coastal forest environment is crucial in understanding why coastal forest clearance causes intensive coastal erosion in particular environments.

2.1.5 Vegetation

It's important for improving slope stability, consolidating sediments and providing some shoreline protection.

Equally significant human activities that must be considered over the range of spatial and time scales are:

2.1.6 Activities along the Coast

Building houses via land reclamation or within sand dune areas and port/harbor development has a long-term impact on shoreline change; protective seawalls lead to erosion at the end of the structures, generate beach scouring at the toe of seawall and shorten the beach face. This can occur in the short term (less than five years) or the long term (more than five years). Other structures such as groynes and jetties typically cause erosion down-drift of the structure within a short period of time (between five and ten years). Removal of dune vegetation and mangroves will expose low energy shorelines to increased energy and reduced sediment stability, causing erosion within five to ten years

2.1.7 Activities within River Catchments/Watersheds

Dam construction and river diversion cause reduction of sediment supply to the coast that contributes to coastal erosion. The effects of dam and river diversion in terms of coastal erosion are not straightforward.

2.1.8 Onshore and Offshore Activities

Sand and coral mining and dredging may affect coastal processes in various ways such as contributing to sediment deficit in the coastal system and modifying water depth that leads to altered wave refraction and longshore drift. The impact of these activities will be obvious within a short period of time (one to ten years).

Understanding the key processes of coastal dynamics and how the coasts function both in spatial and temporal time scales (short and long term), as well as human activities along the coast, within the river watershed and offshore is essential for managing coastal erosion because it may occur without reason. A quantitative understanding of changes in spatial and short- and long-term time scales is indispensable for the establishment of rational policies to regulate development in the coastal zone (NRC, 1990).

3: COASTAL PROTECTION

3.1 The Protective Function of Coastal Systems

Coastal areas with natural protective features can reestablish themselves after natural traumas or long-term changes such as sea-level rise. The protective features of the coastal system vary (Figure 4.8). The role of coral reefs in coastal protection has been studied for some time and recent efforts have focused on the role of coastal vegetation, especially mangrove forest and saltmarshes in this context.

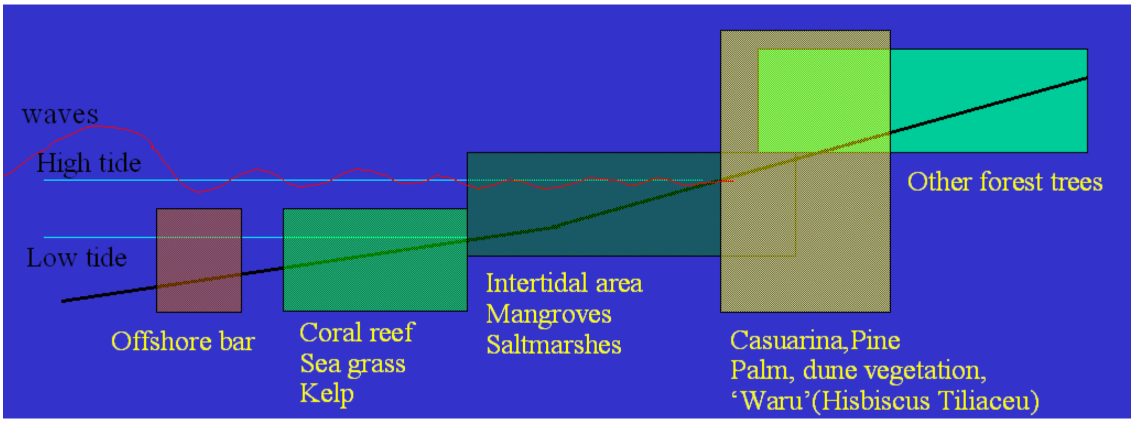


Figure 3.1 the natural protective features of coastal systems

3.2 Managing the Coastal Erosion Problem

Assessing coastal erosion can be done by visual observation and through discussions with inhabitants to ascertain its degree and when it started. Common visual indicators to identify erosion problems are summarized in Table 4.2. However, determining the causes of coastal erosion and which coastal protection options should be used requires a comprehensive study of coastal processes that work on a regional scale (not only on sites) through every season. Options for combating coastal erosion are traditionally twofold, namely hard structural/engineering options and soft structural/engineering options. These solutions have at least two hydraulic functions to control waves and littoral sediment transport in applying the solutions, their underlying principles should be well-understood, otherwise they will fail. A combination of hard and soft options has become more popular recently for optimum results because they have weaknesses when used singlely. Many schemes have failed and resulted in environmental and socio-economic problems owing to improper design, construction and maintenance, and were often only implemented locally in specific places or at regional or jurisdictional boundaries, rather than at system boundaries that reflect natural processes.

Table 3-1 common visual indicators for identifying erosion problems

All coastlines	Cliff and platform (hard coast)	Clayey banks and muddy coast (semi-hard coast)	Sandy coast (soft coast)
Object (e.g. fence, shed or tree) which falls into the sea	Very steep cliff faces	Tree angle	Stable backdune vegetation in the active zone
Presence of existing coastal erosion management works (particularly poor condition of structures)	Shore platforms	Non-vegetated clayey banks	Damaged vegetation in the active zone (exposed roots)
	Sea caves, notches, stacks	Slumping slopes	Erosion scarps
	Debris at toe of cliff	Dislodged vegetation in the coastal area	Discontinuous vegetation cover on foredunes
	Tree angle at the top of the cliff	Erosion scarps	Irregular foredune crest, blow outs
			Very steep dune formation

3.2.1 Hard Structural/Engineering Options

Hard structural/engineering options use structures constructed on the beach (seawalls, groynes, breakwaters/artificial headlands) or further offshore (offshore breakwaters). These options influence coastal processes to stop or reduce the rate of coastal erosion.

3.2.1.1 Groynes

A coastal structure constructed perpendicular to the coastline from the shore into the sea to trap longshore sediment transport or control longshore currents. This type of structure is easy to construct from a variety of materials such as wood, rock or bamboo and is normally used on sandy coasts as show in figure (3.2).

Figure 3.2 groynes wall

Groynes have the following disadvantages:

- Induces local scour at the toes of the structures.
- Causes erosion downdrift; requires regular maintenance.
- Typically more than one structure is required.

3.2.1.2 Seawall

A seawall is a structure constructed parallel to the coastline that shelters the shore from wave action. This structure has many different designs; figure (3.3) shows different type of seawall.

Figure 3.3 seawall

Seawall can be used to protect a cliff from wave attack and improve slope stability and it can also dissipate wave energy on sandy coasts. The disadvantages of this structure are:

- It creates wave reflections and promotes sediment transport offshore.
- Scour occurs at the toes of eroded beaches.
- It does not promote beach stability.
- It should be constructed along the whole coastline; if not, erosion will occur on the adjacent coastline.

3.2.1.3 Offshore Breakwater

Figure (3.4) shows an offshore breakwater is a structure that parallels the shore (in the nearshore zone) and serves as a wave absorber. It reduces wave energy in its lies and creates a salient behind the structure that influences longshore transport of sediment. More recently, most offshore breakwaters have been of the submerged type; they become multipurpose artificial reefs where fish habitats develop and enhance surf breaking for water sport activities. These structures are appropriate for all coastlines.

Figure 3.4 Offshore Breakwater

Their disadvantages are:

- They are large structures and relatively difficult to build.
- They need special design.
- The structure is vulnerable to strong wave action.

3.2.2 Soft Structural/Engineering Options

Soft structural/engineering options aim to dissipate wave energy by mirroring natural forces and maintaining the natural topography of the coast. They include beach nourishment/feeding, dune building, revegetation and other non-structural management options.

3.2.2.1 Beach Nourishment

The aim of beach nourishment is to create a wider beach by artificially increasing the quantity of sediment on a beach experiencing sediment loss, improving the amenity and recreational value of the coast and replicating the way that natural beaches dissipate wave energy.

Offshore sediment can be sourced and is typically obtained from dredging operations; landward sources are an alternative, but are not as effective as their marine equivalents because the sediment has not been subject to marine sorting. Figure (3.6) shows the Beach Nourishment process.

Figure 3.5 Artificial Headland

This method requires regular maintenance with a constant source of sediment and is unlikely to be economical in severe wave climates or where sediment transport is rapid. It has been used in conjunction with hard structural/engineering options, i.e. offshore breakwaters, headlands and groynes to improve efficiency.

3.2.2.2 Dune Building/Reconstruction

Sand dunes are unique among other coastal landforms as they are formed by wind rather than moving waters; they represent a store of sand above the landward limits of normal high tides where their vegetation is not dependent on the inundation of seawater for stability as show in figure (3.6). They provide an ideal coastal defense system; vegetation is vital for the survival of dunes because their root systems bind sediment and facilitate the build-up of dune sediment via wind baffle. During a storm, waves can reach the dune front and draw the sand onto the beach to form a storm beach profile; in normal seasons the wind blows the sand back to the dunes. In dune building or reconstruction, sand fences and mesh matting in combination with vegetation planting have successfully regenerated dunes via sediment entrapment and vegetation colonization. The vegetation used should be governed by species already present, such as marram, sand couch grass and Lyme grass.

Figure 3.6 dune Building/reconstruction

4: Two-Dimensional Wave Equations and Wave Characteristics

4.1 Waves

Ocean waves are undulations of the water's surface resulting from the transfer of energy. The disturbance is propagated by the interactions of disturbing (e.g. wind) and restoring (e.g. gravity) forces.

The energy in most ocean waves originates from the wind blowing across the water's surface. Large tsunami or seismic sea waves are generated by earthquakes, volcanic eruptions or large marine landslides. On the other hand, Tides, largest of all ocean waves result from the combined gravitational force exerted on the oceans by the sun and the moon figure (4-1) shows the wave formation.

When wind blows over water it exerts a drag on water surface and water by virtue of its fluidity gets disturbed giving rise to waves. Such waves are referred as wind waves or storm waves. Waves are usually defined by their height, length and period.

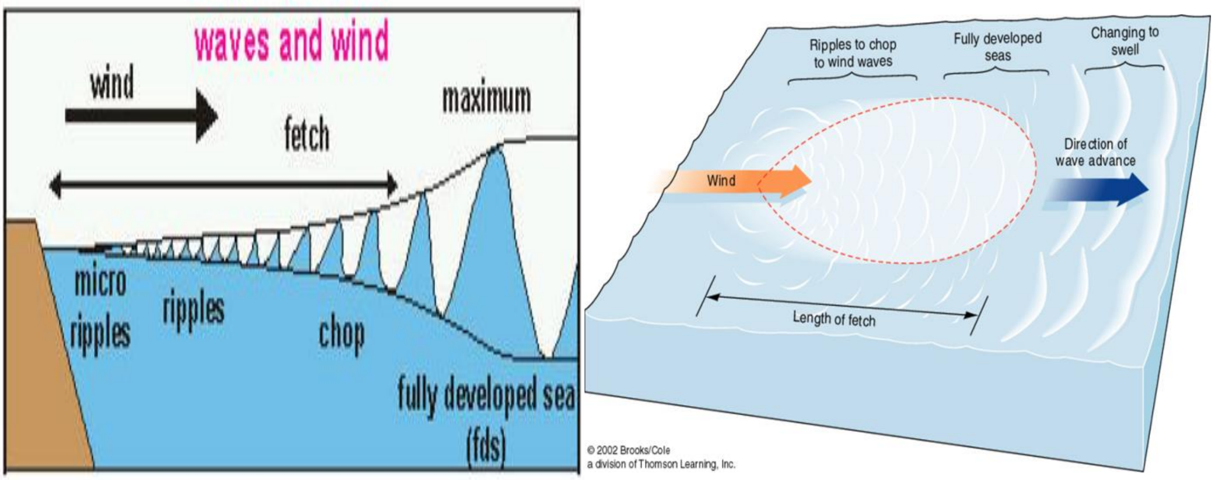


Figure (4-1) waves

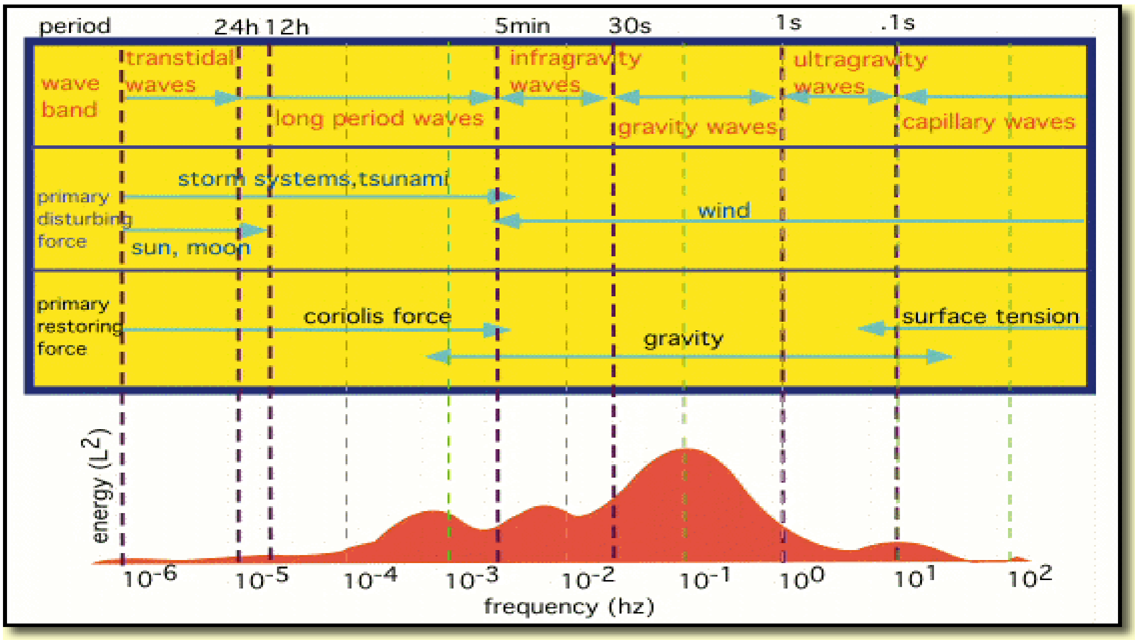
4.2 Tides

Tides are the rise and fall of sea levels caused by the combined effects of the gravitational forces exerted by the moon and the sun, and the rotation of the Earth figure (4-2) show fall and rise of sea level.

Figure (4-2) waves

4.3 Classification of Waves

The spectrum of ocean surface wave shown below categorizes waves according to wave period, or frequency. The period is the length of time it takes for an entire wave to pass a point. Frequency is the inverse of the period (1/T). Ocean waves with the longest periods are tidal wave produced by the gravitational forces exerted on the Earth by the Moon and Sun. Tides move sediment perpendicular to the shore and controls the daily movement of the surf up and down the foreshore. Wind generates the ocean waves we sea breaking in the surf zone. Breaking waves produce the longshore currents that transport sediment parallel to the shore.



Approximate distribution of ocean surface wave energy (after Kinsman 1965)

4.3 Wave Theories

Wave theories are approximate to the reality are as follow:

1. Linear or Airy's (or sinusoidal or small amplitude) wave theory.
2. Non-linear (Stoke finite amplitude) wave theory.
3. Other theories such as (Non-linear shallow water wave theory, KdV and Boussinesq, Conidal wave theory, Stream function theory and Fourier approximation)

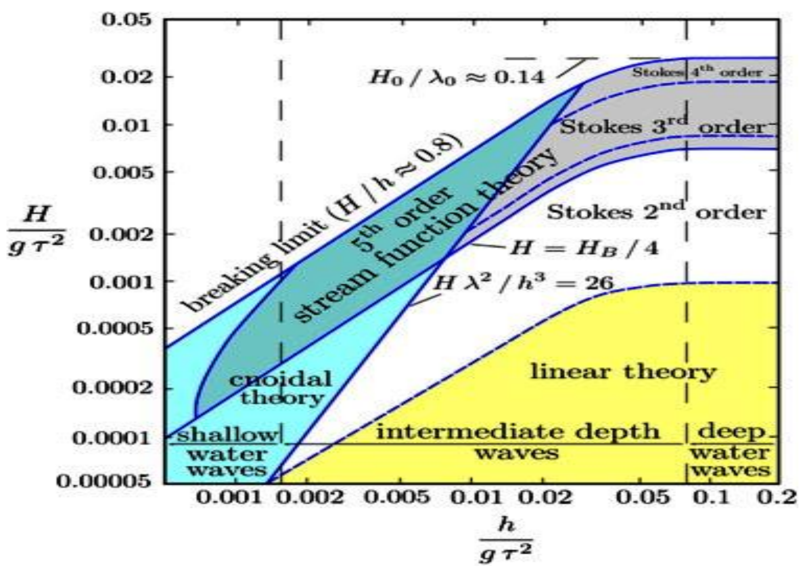


Figure (4-4) ranges of applicability of various wave theories

4.3.1 The assumptions of wave theories

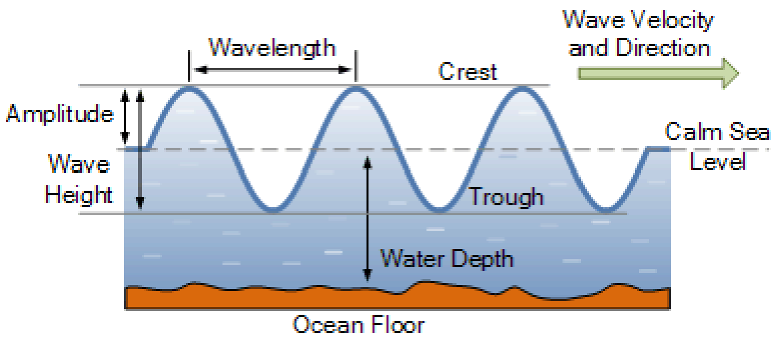
- The waves have regular profiles
- The flow is two dimensional
- The wave propagation is unidirectional
- The fluid is ideal (i.e.) inviscid, incompressible and irrotational
- The sea bed is impermeable and horizontal

4.3.2 State the assumptions to be made in linear wave theory.

- Fluid is homogeneous and incompressible. Therefore the density is constant.
- Surface tension can be neglected.
- Pressure at free surface is uniform and constant.
- The fluid is ideal.
- The particular wave being considered does not interact with any other wave motion.
- The bed is horizontal, fixed, impermeable boundary which implies that the vertical velocity at the bed is zero.
- The wave amplitude is small and invariant in time and space.

4.4 Wave Amplitude, Height, Length, and Wave Period

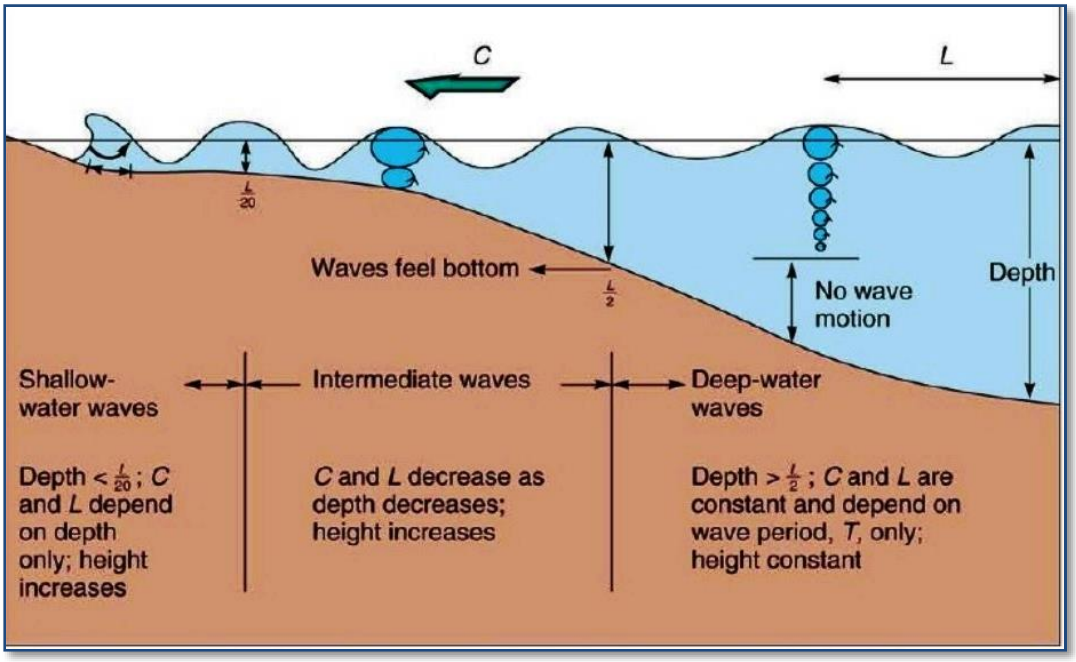
The maximum vertical displacement of the sea surface from still water level (half the wave height). The vertical distance from the top of the crest to the bottom of the trough is called wave height. The horizontal distance between the successive crest is called wave length. The time between successive crest passing a given point is called wave period.

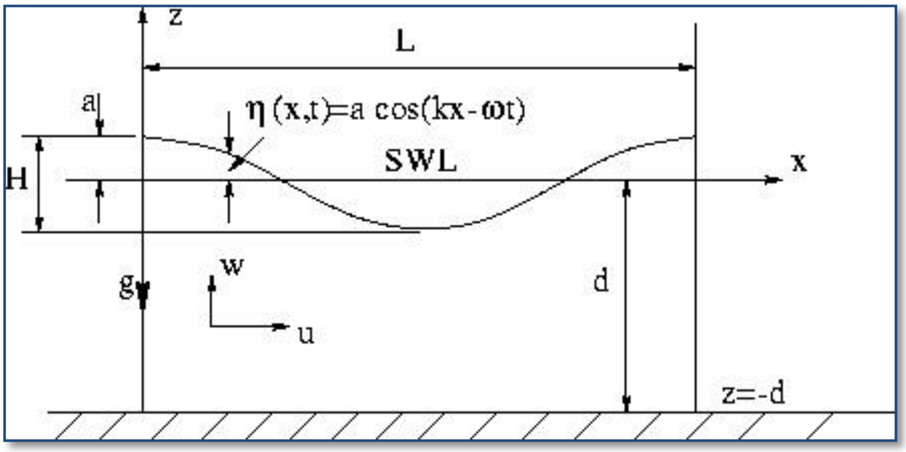


4.5 Deep, Shallow Water and Wave Celerity

Wave can be classified according to the relative wave (***d/L***). there are two mean type of relative depth as follow:

- **Deep water**: when the relative depth, ***d/L***, is greater than 0.5 and, ***tanh (2πd/L)*** ≈1. The phase speed of the waves is hardly influenced by depth (this is the case for most wind waves on the sea and ocean surface).
- **Shallow water**: another extreme condition may be found when the relative depth, ***d/L***, is less than 0.005, ***tanh (2πd/L)*** **≈*2πd/L*.** The phase speed of the waves is hardly influenced by depth (this is the case for most wind waves on the sea and ocean surface).





The wave celerity is as follow:

$$C_\circ = \sqrt{\frac{gL}{2\pi} tanh(\frac{2\pi d}{L})} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(4-1)}$$

$$C_s = \frac{gT}{2\pi} \tanh(\frac{2\pi d}{L}) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(4-2)}$$

$$L = \frac{gT^2}{2\pi} tanh(\frac{2\pi d}{L}) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(4-3)}$$

Where:

C_o = wave celerity in deep water

C_s = wave celerity in shallow water

g = acceleration due to gravity

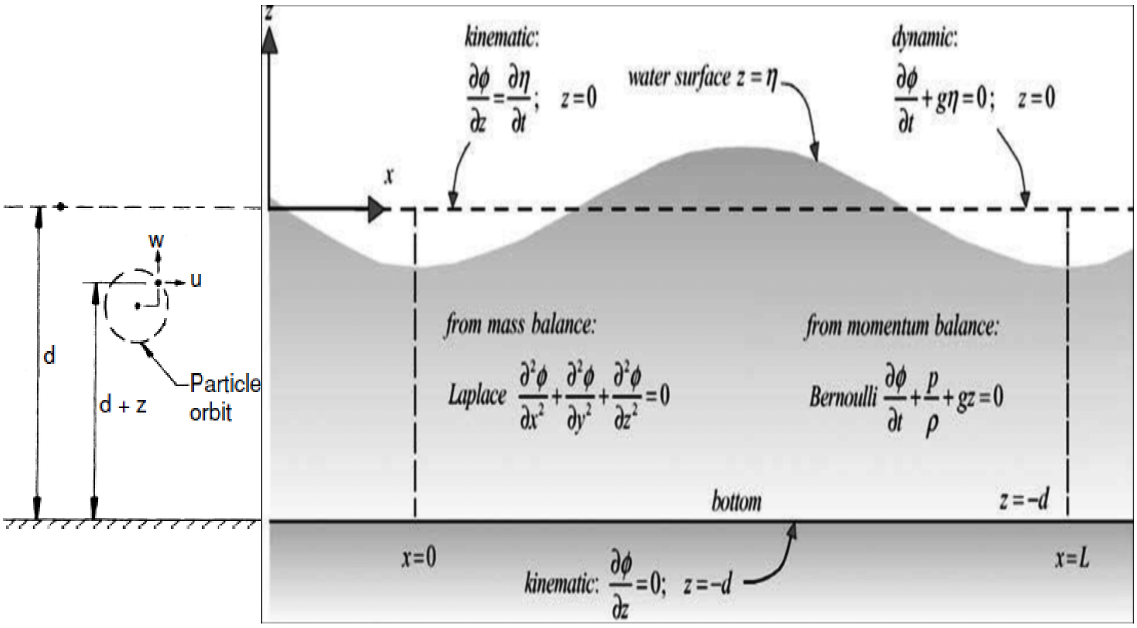
T = wave period

Fenton (1990) has given an approximate relationship from which the wave length can be directly calculated from available knowledge of wave period and depth of water as follow:

$$L = \frac{gT^2}{2\pi}\left\{tanh\ (\frac{\omega^2 d}{g})^{\frac{3}{4}}\right\}^{\frac{2}{3}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(4-4)}$$

4.6 Velocity potential function and stream function.

Employing the Laplace equation, we can derive the velocity potential for the small-amplitude wave. The most useful form of this velocity potential is



$$\emptyset = \frac{H}{2}\frac{g\cosh(k\ (d+z))}{\omega\cosh(kd)}\sin(kx-\omega t) \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(4-5)}$$

Where:

k is the wave number $k = \frac{2\pi}{L}$

ω is wave angular frequency $\omega = \frac{2\pi}{T}$ $\quad and\ \ \omega^2 = gk\tanh(kd)$

4.7 Wave Kinematics and Pressure

The water particle velocity and acceleration as well as the pressure Weld in a wave are all needed to determine wave-induced forces on various types of coastal structures.

- **Wave Kinematics**

The horizontal and vertical components of water particle velocity (u and v, respectively) can be determined from the velocity potential where

$$u = \frac{\partial \emptyset}{\partial x} \quad and \quad v = \frac{\partial \emptyset}{\partial y} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(4-6)}$$

The particle velocity is

$$u = \frac{\pi H}{T} \frac{\cosh k(d+z)}{\sinh kd} \cos(kx - \omega t) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(4-7)}$$

$$v = \frac{\pi H}{T} \frac{\sinh k(d+z)}{\sinh kd} \sin(kx - \omega t) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(4-8)}$$

Similarly the acceleration is

$$a_x = \frac{\pi^2 H}{T^2} \frac{\cosh k(d+z)}{\sinh kd} \sin(kx - \omega t) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(4-9)}$$

$$a_y = \frac{\pi^2 H}{T^2} \frac{\sinh k(d+z)}{\sinh kd} \cos(kx - \omega t) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(4-10)}$$

- **Pressure Field**

The pressure Weld in a wave

$$P = -\rho g z + \frac{\rho g H}{2} \left[\frac{\cosh k(d+z)}{\cosh kd}\right] \cos(kx - \omega t) \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(4-11)}$$

The first term on the right gives the normal hydrostatic pressure variation and the second term is the dynamic pressure variation owing to the wave-induced particle acceleration.

4.8 Wave energy

The total energy, E of a wave system is the sum of its kinetic energy, E_k and potential energy, E_p. According to linear wave theory, the total wave energy in one wave length per unit crest width is given by:

$$E = E_k + E_p = \rho\ g\ H^2\ L/8 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots$$

References

1- Young C. Kim ***Design of coastal engineering and sea defenses***, series on coastal ocean engineering, vol2.

2- Dominic Reeve, Andrew Chadwick ***Coastal engineering: processes, theory and design practice***, Spon press, 2004.

3- Robert M. Sorensen ***Basic coastal engineering*** , third edition, 2006

WATER HAMMER

Table of Contents

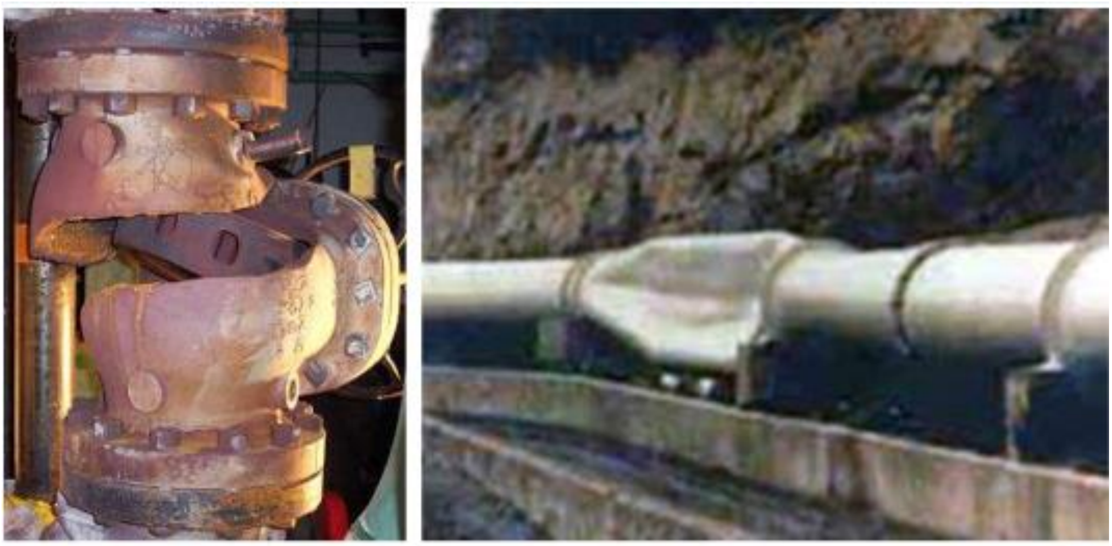
<u>(1)Water Hammer Basics</u>

Water hammer, *Pressure Surge or hydraulic shock, is a term for the destructive pressure increase and accompanying shock wave that takes place in pipeline or piping systems when the flow rate of liquid changes suddenly*. A large vapor pocket is formed because of this within the pipeline; and when this vapor pocket collapses, a highly powerful, bidirectional pressure wave propagates away from the origin at a very very high speed. In extreme situations, the magnitude of this shock wave can reach up to 100 times the closure velocity of the pocket. In the actual scenario, This huge force can force the pump shell to rupture.

The most well-known causes of water hammer are sudden valve closure and other forms of flow restriction. In cases where one of these is the cause, pump operators can easily eliminate water hammer by following proper valve closure procedures.

(2) Figures for Pipes Broken From Water Hammer

Figure(1)



Figure(2)

Figure(3)

(3)Flashing VS. Water Hammer

Flashing is a different kind of pressure spike event. Flashing occurs in steam systems where steam condensate (liquid water) has accumulated within the piping system. This liquid water can suddenly convert from a liquid to a steam with a subsequent volumetric expansion factor of 400-600 times. Flashing needs to be dealt with in totally different ways. While it’s equally important to control, for purposes of this article, we confine our discussions to liquid mediums and water hammer noises only.

(4)Difference between Water Hammer and Cavitation

Sometimes, a water hammer or surge is confused with pump cavitation, but both are two different phenomena. However, cavitation can easily increase the potential risk of water hammer. In normal practice, cavitation itself is not always so destructive. The main difference between the two is that Cavitation involves the implosion of many small, localized vapor pockets whereas the water hammer involves the collapse of a single large vapor pocket. Cavitation damage builds up over time before causing a part to fail; conversely, a single instance of water hammer may cause catastrophic damage.

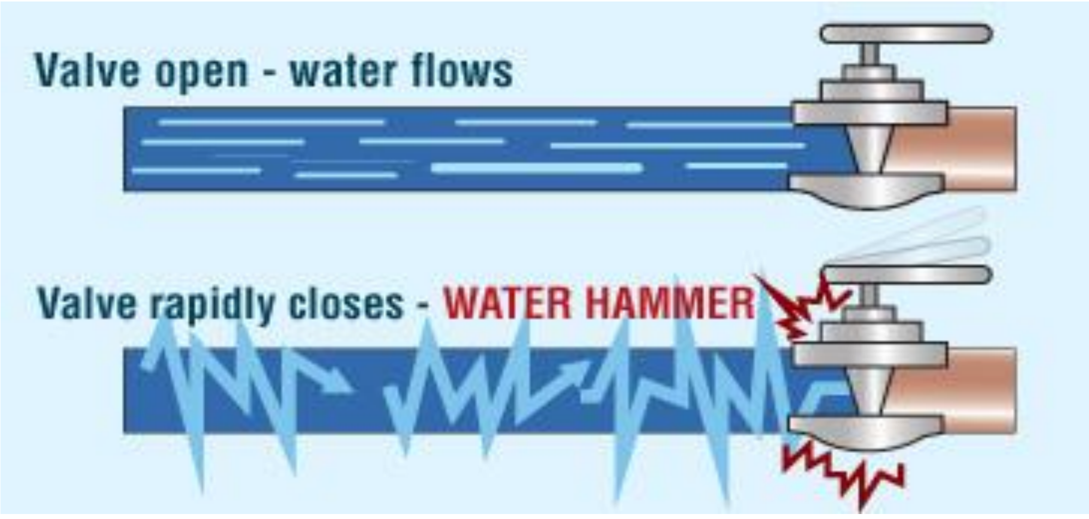
Extreme cavitation can occasionally result in a water hammer. For example, when cavitation is not severe enough to stop operation entirely but severe enough to impede pumping, a vapor pocket is expected to form in the pipeline. When pumping resumes, the pocket collapses, thereby causing a water hammer.

(5) What Can Cause Water Hammer?

Sudden flow restriction due to valve closure or pump trip, load shedding, etc. are the most popular reasons behind water hammer phenomena. If any of the above is the actual cause, the pump operators can easily reduce or even eliminate the possibility of water hammer by using proper valve closure procedures.

However, the pumping environment can also be one of the reasons for its creation. For example, In the phosphate industry pressure surge in pit pump applications are the most common. In such cases, during operation, the suction pipe can become blocked, not during valve closure. While quick filling of long pipelines Surge can occur where the flowing fluid and the static fluid meets.

The most well-known causes of water hammer are sudden valve closure and other forms of flow restriction. In cases where one of these is the cause, pump operators can easily eliminate water hammer by following proper valve closure procedures.



Figure(4)

The most well-known causes of water hammer are sudden valve closure and other forms of flow restriction. In cases where one of these is the cause, pump operators can easily eliminate water hammer by following proper valve closure procedures.

(6) Basic Methods of Mechanism of Water Hammer events

water hammer event or hydraulic transient results when the velocity of flow changes in a pipeline. Water hammer is the transmission of pressure waves along pipelines resulting from a change in flow velocity. When the steady flow of an elastic fluid in a pipe is disturbed (for example, opening or closing a valve in a pipeline) the effect is not felt immediately at other points (2) in the pipeline. The effect is transmitted along the pipeline at a finite velocity called the wave speed of the fluid. Typical causes of water hammer include the adjustment of a valve in a piping system, starting or stopping of a pump, and load rejection of a turbine in a hydro-electric power plant. Water hammer in systems is becoming increasingly important as technology advances, larger equipment is constructed, and higher speeds are employed for pumps and turbines. Possible outcomes of water hammer events include dangerously high pressures, excessive noise, fatigue, pitting due to cavitation, disruption of normal control of circuits, and the destructive resonant vibrations associated with the inherent period of certain systems of pipes. The objective of water hammer analysis is to calculate the pressures and velocities dur-ing an unsteady-state mode of operation. The analysis of unsteady flow is much more complex than for steady flow. Another independent variable, that of time, enters and the resulting equations are partial differential equations rather than ordinary differential equations. The so-lution of the resulting hyperbolic partial differential equations by the method of characteristics is well suited to the speed and accuracy of digital computers.

(7)Predicting Water Hammer Pressure Spikes

It is possible to calculate the magnitude of water hammer pressure spikes based on detailed knowledge of the piping system and the media transported. The actual force of water hammer depends on the flow rate of fluid when it is stopped and the length of time over which that flow is stopped. For example, consider 100 gallons of water flowing in a 2-inch pipe at a velocity of 10 feet per second. When the flow is quickly brought to a halt by a fast-closing valve, the effect is equivalent to that of an 835-pound hammer slamming into a barrier. If the flow is stopped in less than a half second (which might be the closing speed of the valve), then a pressure spike over 100 psi greater than the system operating pressure can be generated.

The equation for calculating the potential magnitude of the spike is as follows:

$\Delta H = a/g * \Delta V$

ΔH is the change in head pressure

ΔV is the change in fluid flow velocity

a = acoustic velocity in the media

g = gravitational constant

An example is:

a = 4864 feet per second

g = 32.2 feet per second2

ΔV = 5 feet per second

ΔH would be 756 feet (328 psi)

This value is assuming instantaneous valve closure exists.

(8) How to Stop Water Hammer

The main step towards reducing the water hammer possibility is by educating the pump and valve operators or plant operators.

As it is well-known that quick valve closing is the ultimate cause for water hammer generation, so it the responsibility of the valve operators to close the valves in the correct way as mentioned in the operating manuals.

For situations where the pump suction gets blocked, plant operators must stop the pumps following the same sequence steps that they would use while a controlled shutdown is planned. In such cases, the vapor pocket will close at the minimum possible velocity this, in turn, will limit the magnitude os shock wave to a minimum.

Operators should attempt to control the suction pipe movement and position it at the pipe entrance to keep it free from blocking. However, if there is a slump in the mining or dredging face, the pipe may still become blocked. Such risks can be carefully considered before moving the suction pipe entrance, as it may cause a water hammer.

So, by identifying and understanding the causes of surge or water hammer in pumps, and by following proper operating procedures, operating manuals, vendor guidance notes, plant operators can prevent a disaster happening due to water hammer. However, the pumping environment also plays an important role and situations may require expert help.

(9) Control of Water Hammer in Pipe line Systems

Water hammer caused by the start-up, or stoppage of pumps, pump run down, and opening and closing of valves in the pipeline is manifested as high pressure fluctuations and possible column separation in the system.

Other possible effects are excessive reverse pump rotation and check valve slam. The undesirable water hammer effects may disturb overall operation of the system and damage components of the system, for example pipe rupture. Therefore several design approaches may be adopted to solve water hammer problems:

• Installation of surge control devices in the system. Table 1 shows a summary of various water hammer control devices which may be installed in the system.

• Redesign of the pipeline layout e.g. change of elevation, length or diameter of the pipeline.

• Design of a thicker pipeline or selection of a pipe material with higher strength to allow column separation in the system.

• Alteration of operational parameters e.g. reduction of velocity in the pipeline. Economic and safety factors are decisive for the type of protection against undesirable water hammer effects. A number of alternatives should be considered before final design which may include a combination of various design approaches.

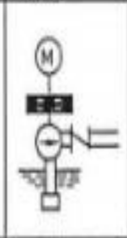
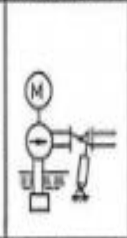
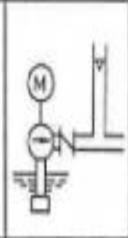
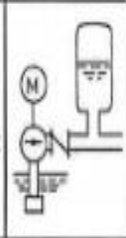
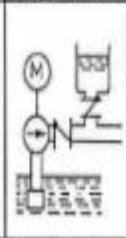
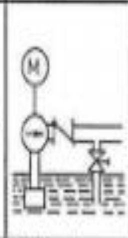
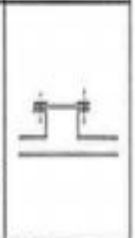
Device	Additional Inertia	Controlled valve closure	Surge tank	Air chamber	One-way surge tank	Pressure relief valve	Bypass	Air valve	Rupture disk
Schematic									
Principle of operation	Lengthens rundown time	Regulates discharge	Energy accumulator	Energy accumulator	Provides flow	Relieves pressure	Maintains flow, controls reverse flow	Air admission and release	Relieves pressure
Pipeline system/ effectiveness	<2000 m	Always useful	Very low head systems	Long pipelines, medium to high head systems	Long pipe with high points	High head systems	Low head systems, long suction line	Long pipelines with high points	High head systems
Protection against	High pressures, column separation	High pressures	High pressures and column separation	High pressures and column separation	Column separation	High pressures	Column separation	Column separation	Very high pressures
Reliability	Excellent	Moderate	Excellent	Good	Moderate	Poor	Poor	Poor	Excellent
Auxiliary equipment/ maintenance	Larger electric motor	Hydraulic control system	None	Compressor or gas bottle	Checking tank level	Regular Maintenance	In-line valve	None	Removal of water
Restarting problems	None	Check hydraulic control system	None	Check air chamber pressure	Refilling of tank	None	Check in-line valve	Remove air from pipeline	Replace rupture disk
Frequency of application	Sometimes	Very often	Rarely	Very often	Rarely	Sometimes	Sometimes	Often	Sometimes
Cost	Fairly low	Low	High	Very high	Moderate	Moderate	Low	Low	Low

Table (1) : a summary of various water hammer control devices.

(10)Value Closure Time Calculations

Water hammer is obviously a serious issue in industrial settings, such as at a wastewater plant or municipal water system. In contrast to the example above, the average bathroom faucet is usually based on a half-inch nominal line size and has a water pressure that ranges between 60-80 psi and delivers about 8-10 gallons per minute. A 6-inch line in a water treatment plant would deliver 900 gallons per minute with a velocity of 10 feet per second. A 24-inch water main could be delivering over 12,000 gallons of water per minute, enough to fill the average backyard swimming pool in less than two minutes.

The basic formula for valve closure time is: $T = 2L/a$

T = minimum time in seconds

L = length of straight pipe between the closing valve and the next elbow, tee or other change

For water at 70°F (21°C) where you have 100 feet of straight pipe:

T= 41 milliseconds minimum closure time

(11) Solutions to Water Hammer and Conclusion

There are many ways to mitigate the effects of water hammer, depending on its cause. One of the simplest methods of minimizing water hammer caused by hydraulic shock is to train and educate operators. Operators who learn the importance of opening and closing manual or actuated valves properly can take precautions to minimize the effects. This is particularly true for quarter-turn valves such as ball valves, butterfly valves and plug valves.

Conclusion Water hammer has been studied for many years. Some of the founding research dates back to the late 19th century. Research continues today. Many major universities in the United States, U.K. and the Netherlands as well as well-respected valve companies have authored articles on the comparison of various styles of check valves and their installed dynamic characteristics.

This article only scratches the surface of the subject of fluid transients by exploring some of the causes and solutions of what we commonly call water hammer. Solutions to deal with water hammer problems can be quite costly, and, as always, an ounce of prevention is worth a pound of cure. Pumps feeding into vertical lines or common headers and rapid valve closures can all be designed out of a process at the beginning. Once the piping is in place and the plant processes are underway, the challenge is to find solutions given the specific constraints. Most manufacturers of in-line silent check valves understand water hammer very well and have engineers on staff that can help. They can be the best source of knowledge when it comes to the right solution.

(10):References

1) http://campaign.ksb.com/blog/what-causes-water-hammer
2) https://www.valvemagazine.com/magazine/sections/back-to-basics/8418-water-hammer.html
3) Loofbourow R.F. Ground water and ground-water control. SME Mining Engineering Handbook. Ed. by Cummins A.B. and Given I.A. Soc. of Mining Eng. of AIME, New York, Vol. 2, Chapter 26, 55pp. (1973).
4) Lupton H.R. Graphical analysis of pressure surges in pumping systems. Journal of the Institution of Water Engineers. Vol. 7, pp. 87-156 (1953).
5) Pearsalll.S. The velocity of water hammer waves. Proceedings Institution Mechanical Engineers. Vol. 180, Part 3E, pp. 12-20 (1965-66).
6) Wylie E. B. and Streeter V. L. Fluid Transients (Republished with minor corrections by FEB Press, Ann Arbor, Michigan, 1983). McGraw-Hill, New York, 384pp. (1978). 5. Wylie E.B. The microcomputer and pipeline transients. Journal of Hydraulic Engineering. ASCE, Vol. 109, Dec. pp. 1723-1739 (1983).
7) Chaudhry M.H. Applied Hydraulic Transients. Van Nostrand Reinhold Co., New York, 2nd Ed., 521 pp. (1987).
8) Wylie E.B. Simulation of vaporous and gaseous cavitation. ASME Journal of Fluids Engineering. Vol. 106, pp. 307-311 (1984).

RIPRAP

1-Introduction

Riprap is a permanent layer of large, angular stone, cobbles, or boulders typically used to armor, stabilize, and protect the soil surface against erosion and scour in areas of concentrated flow or wave energy. Riprap, also known as rip rap, rip-rap, shot rock, rock armor, or rubble, is human-placed rock or other material used to protect shoreline structures against scour and water, wave, or ice erosion. ... Rubble from building and paving demolition is sometimes used

Riprap is typically placed along graded ditch, channel, and shoreline banks over geotextile, which prevents erosional undercutting. It can also be used with other mixed size rock to construct retention berms for sediment traps and check dams protecting high volume/velocity culvert inlets.

Rip rap & river rocks are effective in erosion prevention because they prevent the soil or sand from being cut away by the sweeping action of the water. Rip rap differs from river rock in size and material, although their end purpose of erosion prevention is the same

Riprap describes a range of rocky material placed along shorelines, bridge foundations, steep slopes, and other shoreline structures to protect from scour and erosion. Rocks used range from 4 inches to over 2 feet. The size of the rock needed on a project depends on the steepness of the slope and how fast water is moving. Riprap is a very durable, natural-looking treatment. One drawback is the potential for the rocky material to not be easily traversable by animals; filling the open spaces between the rocks with soil or smaller rocks helps to address this issue.

Pic 1: Rip rap

Pic 2: Rip rap in river

2- Purpose and function

Riprap is used to stabilize areas on a construction site with high erosive power by increasing surface roughness and slowing the velocity of runoff. Applicable areas on a site may include inlets and outlets of storm pipes and culverts, bridges, slopes drain, storm drains, and other areas where concentrated runoff may occur. Riprap is also effective for protecting and stabilizing slopes, channels, stream banks, and shorelines.

There are many applications where riprap is more desirable than other erosion control practices. Although riprap is not often considered aesthetically pleasing, it can be one of the most effective methods of erosion prevention and is particularly desirable in areas where conditions prohibit establishment of vegetation (for example, areas where velocities are too great for vegetation to withstand). Compared to other erosion control practices, riprap is relatively simple to install and maintain. However, riprap is typically more expensive to install compared to vegetation (i.e. due to equipment and handling costs) and does not provide some of the secondary benefits provided by vegetated practices (e.g., habitat enhancement).

3-Site Applicability

Riprap is useful in areas in which the powers of erosion outweigh the stabilization capacity of other erosion control practices such as vegetative control and mulching. As noted above, riprap is especially useful for armoring channel and ditch banks, lake shorelines, and for sediment trap berms and high-volume/velocity check dams. Because of the "hard" look of riprap, its higher overall cost, the growing popularity of vegetated solutions using turf reinforcement matting and other products, and the difficulty in removing it after installation, contractors should ensure that the post-construction site design specifically includes riprap before using it during construction. Riprap may be unstable on very steep slopes. For slopes steeper than 2:1, consider using materials other than riprap for erosion protection such as turf reinforcement matting over seed, open-cell articulated concrete mats, or other slope protection geogrid products/matrices.

4-Site preparation

Before laying riprap, filter material should be placed on a prepared surface in accordance with site plans unless otherwise required by the contract. The foundation surface should be relatively smooth and free of stones, sticks, or other debris. Filter material can be either granular filter or nonwoven geotextile filter, unless specified in the contract. The primary function of a filter material or filter fabric is to prevent soil from “piping” through the riprap stone.

If using granular material, spread the filter material to a minimum thickness of at least 6 inches over the prepared foundation. If using a geotextile, ensure that the fabric liner is pulled taut with no folds or creases before anchoring in place (if needed) with stables or anchor pins. To prevent water from flowing beneath fabric, overlap edges by at least a foot and a half in the downhill or downstream direction. Riprap should be placed over the geotextile fabric no later than seven (7) days after the fabric is laid. Avoid tearing, puncturing, or shifting the fabric and do not operate equipment on top of the geotextile or stones after placement. Geotextile filter material should not be used under hand placed or grouted riprap unless required by the contract.

If placing geotextile on slopes steeper than 3:1 (horizontal to vertical), anchor the geotextile in consecutive trenches on the contour at least every 15 feet. Geotextile should not be used on slopes exceeding 2:1 (horizontal to vertical).

5- Riprap types

Riprap is used to protect a slope against erosion or scour and is placed where vegetation or other methods would be ineffective or impracticable. The types of riprap that may be used are:

1) Dumped Riprap

2) Revetment Riprap

3) Grouted Riprap

4) Precast Concrete Riprap

5) Uniform Riprap

Regardless of the type of riprap used, the foundation grade is required to be stable and true for the riprap to be effective.

5-1 Dumped Riprap

Dumped riprap may consist of several different types of material. Often the riprap is waste material that is on the contract. Dumped riprap may consist of any of the following:

1) Broken concrete, masonry, or stone removed from an old structure

2) Broken pieces removed from concrete pavement, base, or monolithic brick pavement

3) Broken rock from Class X or Class Y, unclassified excavation

4) Broken rock from solid rock excavation

5) Material produced from sources outside the right-of-way. These materials are required to be coarse aggregate, class F or higher.

5-2 Revetment Riprap

Revetment riprap is the most commonly used riprap. Revetment riprap, Class 1 riprap and Class 2 riprap are required to consist of aggregate, Class F or higher. This material is required to be in accordance with Section 904.04 for gradation. The maximum dimension of an individual piece is required to not be greater than three times the minimum dimension and have a gradation as follows:

1) 100 % of the materials passes a 18 in. sieve

2) 90 to 100 % of the material passes a 12 in. sieve

3) 20 to 40 % of the material passes a 6 in. sieve

4) Not more than 10 % of the material passes a 3 in sieve

Pic 3 : Revetment Riprap

5-3 Grouted Riprap

Grouted riprap is required to have the same aggregate, preparation of slope, and method of placement as that required for Revetment riprap.

5-4 Precast Concrete Riprap

Precast concrete riprap consists of unreinforced concrete units. The nominal thickness is detailed on the plans or proposal. These units are required to be produced by an INDOT Certified Precast Concrete Producer.

5-5 Uniform Riprap

Uniform riprap is placed to produce a surface of approximate regularity with the edges having projections no more than 3 in. above the required cross section. This material is hand placed, and the gradation is required to be in accordance with Section 904.04(d).

6- Rip Rap Design

6.1 Characteristics Requiring Design

The following features of riprap revetments require consideration during design. Appendix A to the guide provides technical details.

• Length and alignment of bank protection • End treatment and top treatment of bank protection • Rock shape, size, gradation and blanket thickness • Bank slope and the vertical height (extent) of protection • Toe treatment, or protection against scour or undermining • Filter layer

Repairs of riprap revetments generally do not need to consider the first two items in detail. However, they are key issues in the successful design of works for banks that have not been previously protected, in re-design of works following their complete, or near-complete, failure, and in long extensions of existing revetments. Consultation with a professional engineer that specializes in river engineering works is advised for major repairs or new construction.

6.2 Background to Design

The design of riprap revetments is based on the nature of the streambank and the hydraulic characteristics of the stream at the design flood. As a result, the following background information is usually required as part of design:

• An inspection of the banks and river channel, the extent of recent erosion, potential hard points (inerodible materials along the bank), and the general behaviour of the river near the proposed bank protection site.

• For most projects, cross sections of the bank, stream channel and floodplain are required. If the stream has been previously surveyed for design of bank protection works or floodplain mapping, then these would be repeated.

• Maps, air photographs, bed and bank material descriptions, channel surveys, design briefs for floodplain mapping and previous studies for bank protection all provide valuable information on channel characteristics and behaviour (see USACE 1994).

• Hydrologic analyses, either of a local Water Survey of Canada gauge record on the stream, or a regional analysis, is often required to predict the design discharge for the stream. In British Columbia, this is almost always the 200-year instantaneous maximum discharge.

Design briefs for floodplain mapping, or previous engineering studies for bank protection works may provide a suitable design discharge.

• Hydraulic analyses, at the design or other discharges, are required to predict design water levels, and maximum average velocities and depths. A key aspect of the hydraulic analysis is to predict the maximum average velocity that occurs along the channel and, by extension, the bank where protective works are to be constructed.

6.3 Scour Depth

A key aspect of the design of riprap revetment is to adequately protect its toe from undermining by scour, where this refers to a lowering of the channel bed below some observed level.

Local scour often occurs at bends, changes in flow direction, obstructions, constrictions, sills, control structures, piers or abutments. In addition, general bed lowering may result from long-term degradation, perhaps downstream of a large dam, or from the effects of long-term gravel mining (Galay 1983). The general bed lowering is added to the local scour to predict design scour depth.

A variety of publications provide methods to predict scour depths (Hoffmans et al 1997; Breusers and Raudkivi 1991). The proposed revisions to the "Guide to Bridge Hydraulics" provide methods that are thought to be particularly applicable to British Columbia (Neill 1973).

6.4 The Size of Riprap

Riprap is used for erosion control, to prevent scour, and to minimize sediment transport in rivers and streams. A stable riprap rock size is desired. The Isbash equation computes the smallest diameter stone D having specific gravity S that if dropped in water flowing at velocity V will settle and remain stationary on the channel bed (Blevins, 2003, p. 218; NCHRP, 2006, p. 14)

$$D = \frac{V^2}{2gC^2(s-1)}$$

Our calculation will also solve the equation in reverse. In that case, riprap rock size D is entered. The computed water velocity V is the maximum water velocity at which the riprap will remain in place

The variables in the riprap sizing equation are (units shown in SI, but our calculation allows a variety of units):
C = Isbash constant. Per references below, C=0.86 for highly turbulent conditions or C=1.2 for low turbulence.
D = Median diameter of spherical stone or rock. Also known as D50 (m).
g = Acceleration due to gravity, 9.8066 m/s2.
S = Specific gravity of stone or rock. Typically varies from 2.56 to 2.92 depending on the rock material. A commonly used value is 2.65.
V = Water velocity approaching the riprap (m/s).

6.5 Rock Shape

Rocks used for riprap should be blocky and angular, with sharp clean edges and relatively flat faces. It is generally recommended that individual pieces be close to equi-dimensional, rather than elongate, although this may not always be practical. Typically, the average ratio of the long axis, a, to the thickness, c, for an individual rock should be less than 2.

USACE (1991) notes that if rounded stones are used for riprap that they should be placed on flatter slopes (not exceeding 2.5H:1V) and that the predicted or recommended median rock diameter be increased by 25%, with a comparable increase in the thickness of the revetment.

6.6 Rock Density

The density of rock used for riprap typically varies from about 2,400 kg/m3 (150 lb/ft3) to 2,800 kg/m3 (175 lb/ft3), with a density of about 2,600 kg/m3 (162.5 lb/ft3) common for the granitic or granodioritic rocks that are often quarried in British Columbia.

6.7 Determining the Required Rock Size

Rock dimensions may be successfully designed to resist failure based on local experience, empirical guidelines, or hydraulic relationships that predict stable riprap sizes, based on bank slope and stream characteristics. In this guide, the design of riprap size is based on the latter approach, which requires hydrologic and hydraulic analysis as part of the design process.

Nearly all the riprap sizing methods that are commonly used in North America are based on stream velocity, usually predicting rock size as a power of the velocity against the riprap embankment (Brown and Clyde 1989, CALTRANS 1997, Maynord et al 1989, USACE 1991). As a result, the estimated bank velocity is often the most important factor in determining rock size.

Care is recommended when calculating mean channel velocities and careful consideration is required when selecting the ratio used for converting mean velocities to bank velocities. Stream behavior resulting in direct impingement of flow on the bank, where bank velocities may be much larger than the mean velocity, is often a critical factor for riprap design.

The relative merits of the various riprap sizing equations are described in Meville and Coleman (2000). Following the practice of the proposed revisions to the "Guide to Bridge Hydraulics" (Neill 1973), the USACE (1991) method for rock sizing is described in detail in Appendix A to this guide. Other riprap sizing equations may work equally well in British Columbia.

6.8 Riprap Gradation

The riprap specifications in Section 205 of MOTH (1999) provide a suitable range of standard sizes for use in British Columbia.

Their specifications meet typical standards for graded mixtures, provide a range of specifications up to 4 tonnes nominal size and are commonly produced by quarries in British Columbia. MOTH bases their specifications on rock weight, converted to a median diameter, D50, by assuming a spherical shape for the individual pieces.

The USACE (1991) method predicts the D30 riprap diameter, which would be converted to the required median rock diameter, D50, by multiplying by

1.25. The appropriate specification is then selected from MOTH (1999) as that gradation whose median diameter is equal to or larger than the required D50 predicted by the USACE method. This approach will provide a conservative riprap gradation.

6.9 Thickness

The revetment should be thick enough to include all the rocks in the specified gradation within the layer. Oversize stones that project through the layer may contribute to failure by creating turbulence. Based on Brown and Clyde (1989), the riprap thickness normal to the slope should meet the following criteria:

• Not less than 350 mm, • Not less than 1.5 x D50; and • Not less than a D100.

The above specifications are roughly equivalent to the thicknesses recommended in MOTH (1999).

6.11 Riprap Filters

In British Columbia, where riprap is often placed on banks composed of sand and fine gravel, a filter layer is necessary to avoid loss of bank material through the riprap. The traditional filter material is gravel or crushed rock. Geotextiles are also an alternative because they may be cheaper and easier to install in certain circumstances.

6-RIP-RAP Equation Selection and Rock Sizing

Reference	Equation	Standard Format (for comparison)	Comments
Bonasoundas (1973)	$d_{r50}\ (cm) = 6 - 3.3V + 4V^2$		Equation applies to stones with $S_s = 2.65$ V = mean approach velocity (m/s)
Quazi and Peterson (1973)	$N_{sc} = 1.14\left(\frac{d_{r50}}{y}\right)^{-0.2}$	$\frac{d_{r50}}{y} = \frac{0.85}{(S-1)^{1.25}}Fr^{2.5}$	N_{sc} = critical stability number $= V^2/[g(S_s-1)d_{r50}]$ Fr = Froude number of the approach flow $= V/(gy)^{0.5}$
Breusers et al. (1977)	$V = 0.42\sqrt{2g(S_s-1)d_{r50}}$	$\frac{d_{r50}}{y} = \frac{2.83}{(S_s-1)}Fr^2$	S_s = specific gravity of riprap stones y = mean approach flow depth
Farraday and Charlton (1983)	$\frac{d_{r50}}{y} = 0.547Fr^3$	$\frac{d_{r50}}{y} = 0.547Fr^3$	
Parola et al. (1989)	$\frac{d_{r50}}{y} = \frac{C^*}{(S_s-1)}Fr^2$	$\frac{d_{r50}}{y} = \frac{C^*}{(S_s-1)}Fr^2$	C^* = coefficient for pier shape; C^* = 1.0 (rectangular), 0.61 (round-nose)
Breusers and Raudkivi (1991)	$V = 4.8(S_s-1)^{0.5}d_{r50}^{1/3}y^{1/6}$	$\frac{d_{r50}}{y} = \frac{0.278}{(S_s-1)^{1.5}}Fr^3$	
Austroads (1994)	$\frac{d_{r50}}{y} = \frac{0.58K_pK_v}{(S_s-1)}Fr^2$	$\frac{d_{r50}}{y} = \frac{0.58K_pK_v}{(S_s-1)}Fr^2$	K_p = factor for pier shape; K_p = 2.25 (round-nose), 2.89 (rectangular) K_v = velocity factor, varying from 0.81 for a pier near the bank of a straight channel to 2.89 for a pier at the outside of a bend in the main channel
Richardson and Davis (1995)	$d_{r50} = \frac{0.692(f_1 f_2 V)^2}{(S_s-1)2g}$	$\frac{d_{r50}}{y} = \frac{0.346f_1^2f_2^2}{(S_s-1)}Fr^2$	f_1 = factor for pier shape; f_1 = 1.5 (round-nose), 1.7 (rectangular) f_2 = factor ranging from 0.9 for a pier near the bank in a straight reach to 1.7 for a pier in the main current of a bend
Chiew (1995)	$d_{r50} = \frac{0.168}{\sqrt{y}}\left(\frac{V}{U_*\sqrt{(S_s-1)g}}\right)^3$	$\frac{d_{r50}}{y} = \frac{0.168}{(S_s-1)^{1.5}U_*^3}Fr^3$ $U_* = \frac{0.3}{K_dK_y}$	$K_y = 0.783\left(\frac{y}{b}\right)^{0.322} - 0.106$ $0 \le (y/b) < 3$ $K_y = 1$ $(y/b) \ge 3$ $K_d = 0.396\ln\left(\frac{b}{d_{50}}\right) - 0.034\left[\ln\left(\frac{b}{d_{50}}\right)\right]^2$ $1 \le (b/d_{50}) < 50$ $K_d = 1$ $(b/d_{50}) \ge 50$ K_y = flow depth factor K_d = sediment size factor

Table 1 : RIP-RAP Equation Selection and Rock Sizing

7-Pier Riprap

Pic4 : Pier Riprap

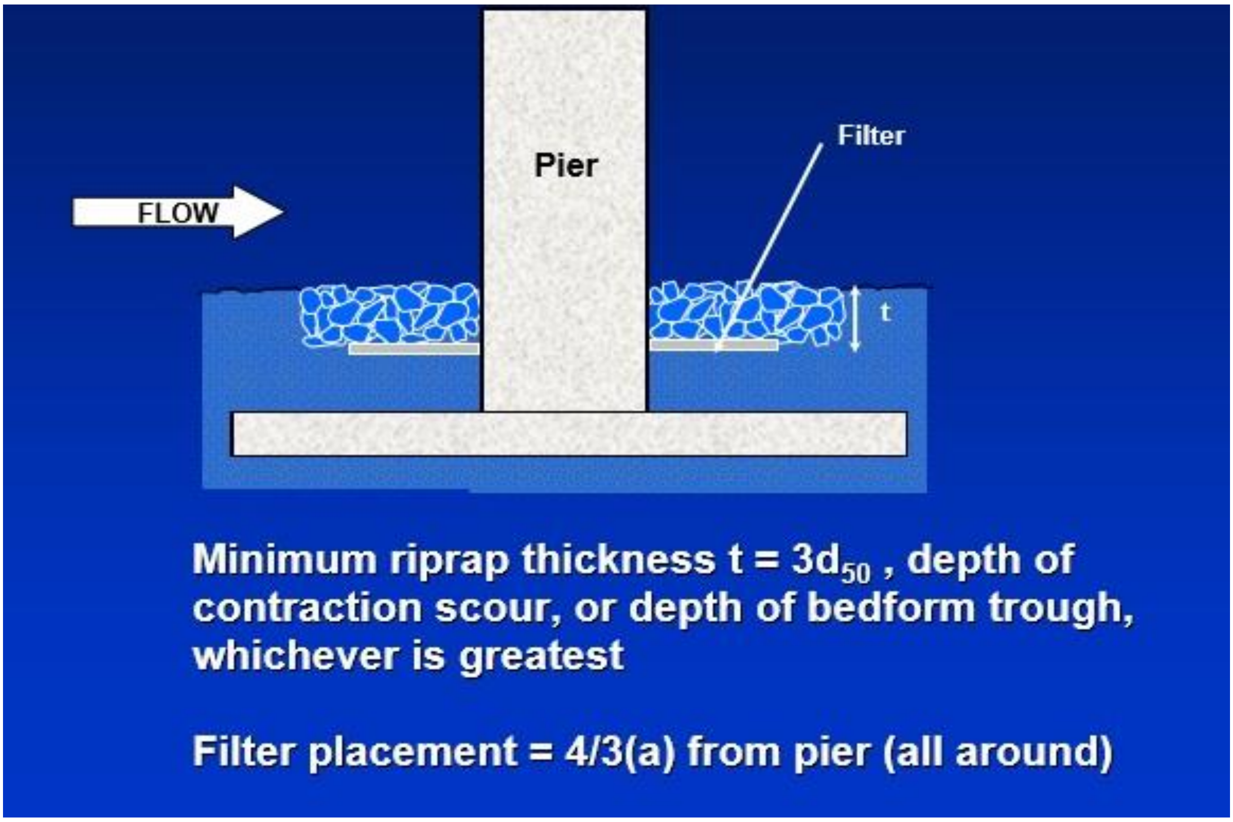


Fig 1 : Cross section of Pier Riprap

8- Abutment Riprap

Pic 5 : Abutment Riprap

Pic6 : Abutment Riprap

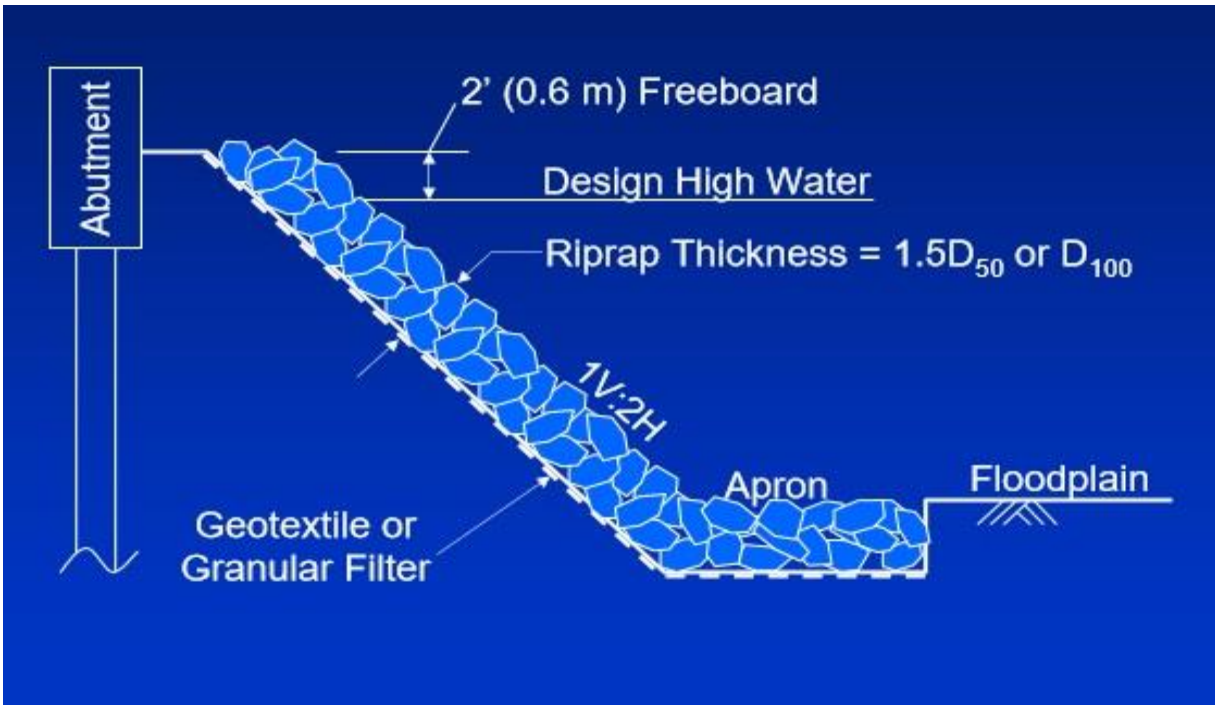


Fig 2 : Cross section of Abutment Riprap

9-Riprap Channel Sizing

Data:

8'	Bottom Width	(b)
2:1	Side Slopes	(Z)
1.0'	Flow Depth	(d)
0.06	Manning's	(n)
0.010 ft/ft	Slope	(s)
65 cfs	Discharge	(Q)

Where: A = Area

WP = Wetted Perimeter

R = Hydraulic Radius

V = Velocity

1.A = (b + Zd)(d)

A = (8’ + 2×1’)(1’)

A = 10ft2

2. WP = b + zd(Z2 + 1)1/2

WP = 8’ + 2×1’(2’2 + 1)1/2

WP = 8’ + 2’(5ft2)1/2

WP = 8’ + 4.47’

WP = 12.47

3.r = A/WP

r = 10ft2/12.47ft

r = 0.80’

4.V = 1.486/n × r2/3 × S 1/2

V = 1.486/0.06 × 0.080 2/3 ×0.10 1/2

V = 24.77 × 0.862 ×0.316

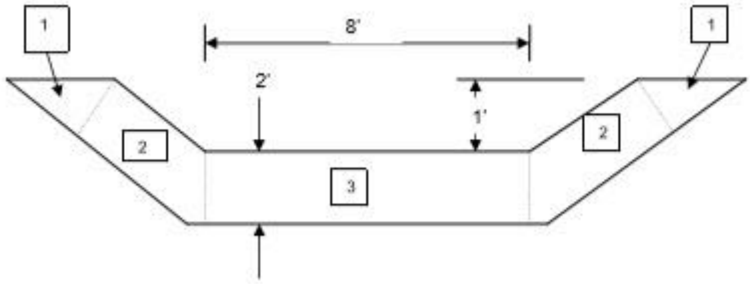
V = 6.75 ft./sec.

5.Q = AV Q = (10ft2 × 6.75ft./sec.)

Q = 67.5 cfs

Use class I riprap D50 - 8 inches, Dmax – 12 inches Thickness = 1.5 Dmax = 18 - inches (use 24 inches)

Class of Riprap	Size in Weight	Approximate Diameter	Percent of Total by Weight
0	Heavier than 33 lb Heavier than 10 lb Less than 1 lb	7 Inches 4 Inches 2 Inches	0 50 10 max
I	Heavier than 150 lb Heavier than 40 lb Less than 2 lb	12 Inches 8 Inches 3 Inches	0 50 10 max
II	Heavier than 700 lb Heavier than 200 lb Less than 20 lb	20 Inches 14 Inches 6 Inches	0 50 10 max
III	Heavier than 2000 lb Heavier than 600 lb Less than 40 lb	28 Inches 20 Inches 8 Inches	0 50 10 max



Area 1 = (((Z2 + d2) x Thickness)/2) x 2 each

Area 1 = ((2’2 + 1’2)(2’))/2 x 2 each

Area 1 = 2.23ft2 x 2 each

Area 1 = 4.46 ft2

Area 2 = (2’2 + 1’2)1/2 x 2’ x 2 each

Area 2 = (5 ft2)1/2 x 2’ x 2 each

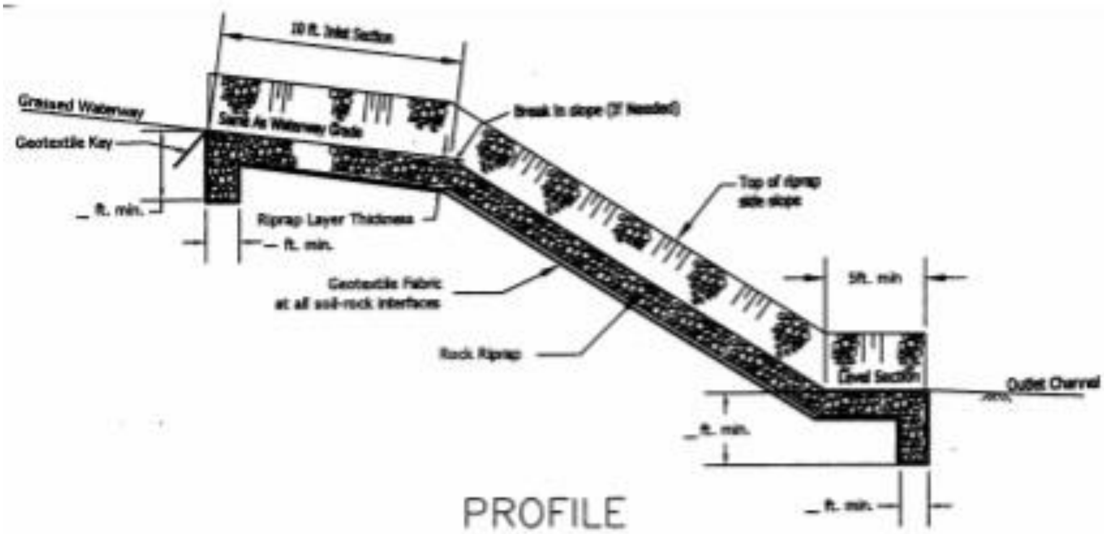
Area 2 = 8.94 ft2 Area 3 = 2’ x 3’

Area 3 = 16 ft2

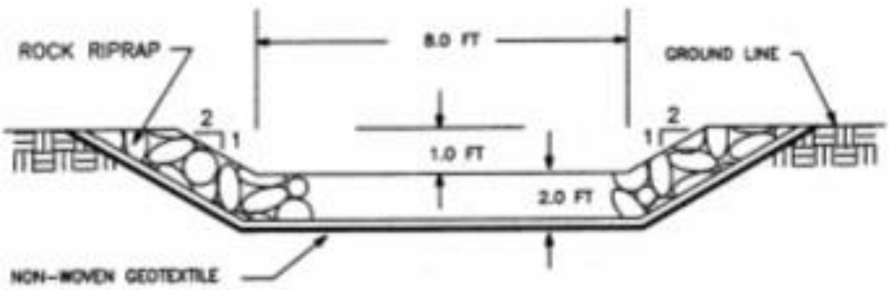
Volume123 = (Area 1 + Area 2 + Area 3)(Length) Volume123 = (4.46 ft2 + 8.94 ft2 + 16 ft2)(30 feet) Volume123 = 882 ft3 Cutoff Volume = (16’ long) x (2’ thick) x (2’ wide) x (2 each) = 128 ft3

Total Volume = 882ft3 + 128 ft3 = 1010 ft3

Convert to Tons (1010 ft3) x (1 yd3/ 27 ft3) x 1.8 ton/yd3 = 67 ton



PROFILE
(NOT TO SCALE)



DESIGN EXAMPLE
TYPICAL SECTION
(NOT TO SCALE)

10-Reference

1-Blevins, Robert D. 2003. Applied Fluid Dynamics Handbook. Krieger Publishing Co.

2-National Cooperative Highway Research Program [NCHRP]. 2006. Riprap Design Criteria, Recommended Specifications, and Quality Control. Report 568. Transportation Research Board. Retrieved from http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_568.pdf

3-Blodgett, J.C. 1986. Rock riprap design for protection of stream channels near highway structures. Volume 1 – Hydraulic characteristics of open channels. US Geological Survey Water-Resources Investigation Report 86-4127. Prepared in cooperation with the Federal Highway Administration. 60 pp.

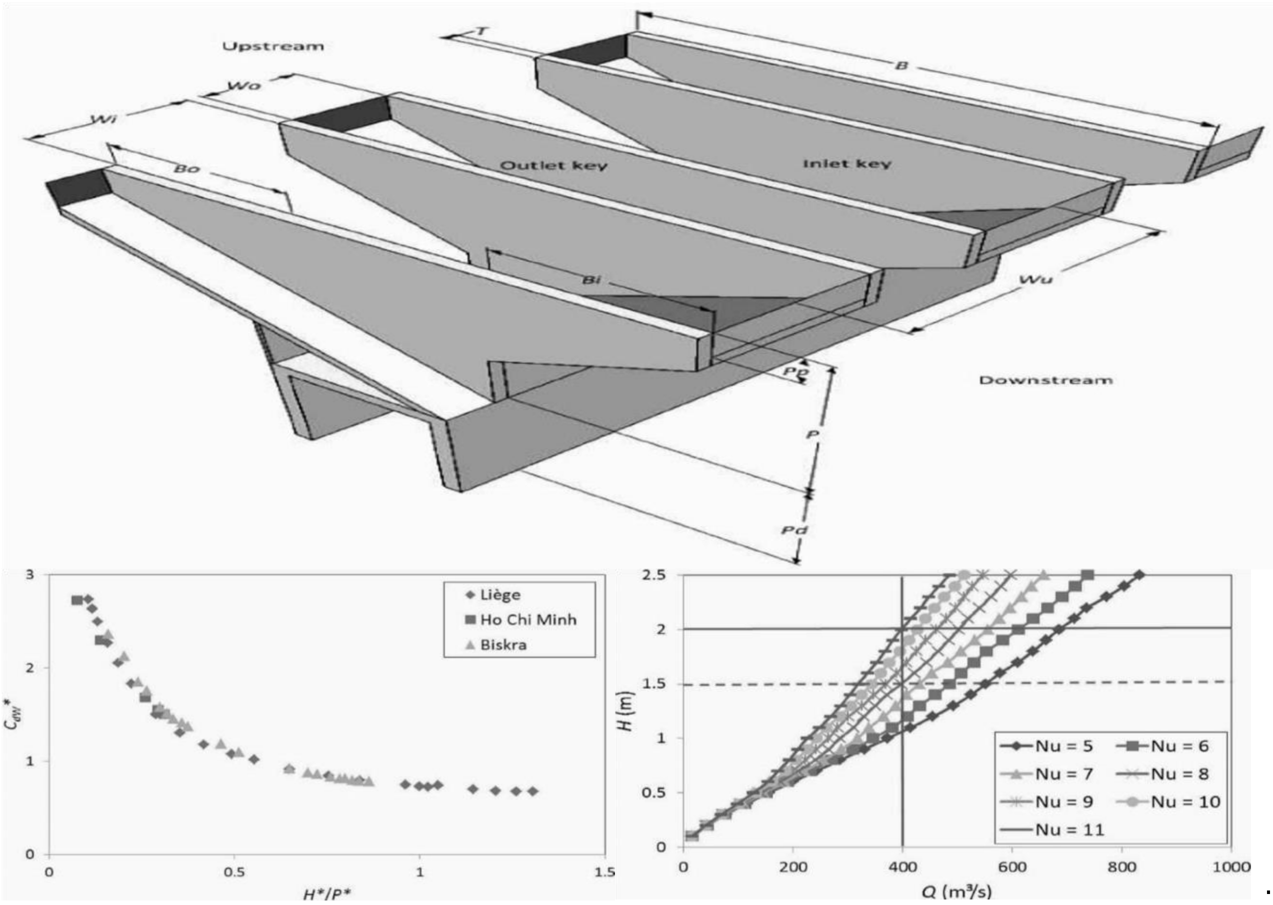
4-Breusers, H.N.C. and Raudkivi, A.J. 1991. Scouring. IAHR Hydraulic Structures Design Manual 2. A.A.Balkema, Rotterdam and Brookfield.

5-Brown, S.A. and Clyde, E.S. 1989. Design of riprap revetment. Report FHWA-IP-89-016, prepared by Sutron Corporation for U.S. Federal Highway Administration, Washington, DC.

DESIGN OF PIANO KEY WEIRS

1.1 Abstract and Figures

Piano Key Weirs (PKWs) are an alternative to linear overflow structures, increasing the unit discharge for similar heads and spillway widths. Thus, they allow to operate reservoirs with elevated supply levels, thereby providing additional storage volume. As they are relatively novel structures, few design criteria are available. Hence, physical model tests of prototypes are required. This study describes comprehensive model tests on a sectional set-up of several A-type PKWs, in which the relevant parameters were systematically varied. Considering data of former studies, a general design equation relating to the head– discharge ratio is derived and discussed. The latter is mainly a function of the approach flow head, the developed crest length, the inlet key height, and the transverse width. To extend its application range, case study model tests were analyses to provide a design approach if reservoir approach flow instead of channel flow is considered.



...project characteristics calculated for nu varying from 5 to 11 are presented in table 4. Figure 3 shows the head/ discharge curves calculated for these nu values. Many solutions exist to ensure the dam safety

...increase in the normal reservoir level enables to increase the hydropower capacity. a first optimization could thus consist in the limitation of the design head to 1.5 m, enabling to fix the normal reservoir level to 452.5 m nGF and limiting the solutions to nu values under 8 Figure 3). another optimization could concern the structural con- straints)

1.2 INTRODUCTION

The Piano Key Weir (PKW) is a particular geometry of weir associating to a labyrinth shape the use of overhangs to reduce the basis length. The PKW could thus be directly placed on a dam crest. Together with its important discharge capacity for low heads, this geometric feature makes the PKW an interesting solution for dam rehabilitation. However, its hydraulic design remains problematic, even at a preliminary stage. This paper presents a preliminary design method based on results of experimental tests. The method enables to design project models by extrapolation of characteristics of existing idealized scale models. A practical application is The Piano Key Weir (PKW) is a further development of the Labyrinth Weir. It was mainly elaborated by Hydro coop(France), in collaboration with the Laboratory of Hydraulic Developments and Environment of the University of Biskra, Algeria, and the National Laboratory of Hydraulic and Envi-ronment of Electricité de France (EDF-LNHE Chatou). Schleiss)2011(and Lempérière et al. (2011) present historical reviewson the evolution from Labyrinth Weirs to PKWs. Two advantages of PKWs as compared with Labyrinth Weirs presented to illustrate the method are. The PKW shows geometric specificities such as up- and/or downstream overhangs with variable width, inlet and outlet bottom slopes, which involve a large set of variable parameters (Figure 1). The “PKW-unit” is the smallest extent of a complete PKW composed of an entire inlet key with two side walls and half an outlet key on both sides. The main geometric parameters of a PKW are the weir height P, the number of PKW-units Nu, the lateral crest length B, the inlet and outlet widths Wi and Wo, the up- and downstream over-hang lengths Bo and Bi and the wall thickness T .Experimental studies have been and are currently carried out in different laboratories to improve the understanding of the flow behavior over PKW (Anderson and Tullis 2011, Machiels et al. 2011, Machiels 2012), and to characterize the influence of a number of geometrical parameters on the discharge capacity of the PKW (Hien et al. 2006, Ouamane and Lempérière 2006b, Le Doucen et al. 2009, Machiels et al. 2010, Machiels 2012). Based on the first experimental results, numerical models have been developed. A 3D model is used by EDF to design the prototype models which will be tested in laboratories (Luck et al. 2009, Pralong et al. 2011). A 1D model has been developed at the University of Liege to improve the design of the experimental models used for parametric studies (Erpicum et al. 2010, Erpicum et al. 2011).The first real size PKWs have been built by EDF in the last six years (Laugier 2007, Bieri et al. 2009, Laugier et al. 2009, Vermeulen et al. 2011, Laugier et al. 2012). Till now, the hydraulic design of a PKW is mainly performed on the basis of experimental knowledge and numerical models, used to design an initial geometry, which is then studied on scale models and modified step by step following the ideas of the project engineers (Ribeiro et al. 2007, Erpicum et al. 2012). By exploitation of existing experimental results, a preliminary design method for PKW has been developed and is presented in this paper. To limit the experimental studies, the design method aims at approaching as well as possible the final project model, compromise between hydraulic optima respect of project constraints and cost effective building .To illustrate the method developed in this paper, it is applied to a PKW project on a large dam.

Method for the preliminary design of Piano Key Weirs9.39 9.3

1.3-1 DESIGN METHOD

The design method is based on the project constraints (discharge, reservoir levels, available space …) and on extrapolation of existing experimental results from a refer ence scale model. By the study of the different possibilities to scale the reference models, the final design is defined depending on project engineer's interests (increase of the security level, increase of the reservoir capacity, decrease of the structure dimensions …). The elements necessary to the design method are categorized in project elements and reference model elements. The project elements are the hydraulic and geometric specificities of the project (discharge, maximal head and available width). Regarding the reference model, a release capacity curve, issued from experimental tests, is necessary as well as geometric characteristics of the tested model. Based on the project elements, different efficient designs may exist. The first step of the method aims at defining these different possibilities as a function of the number of PKW-units in the structure, by scaling of the geometric and hydraulic parameters of the reference model. The PKW-unit width Wu is defined as a function of the number of PKW-units Nu and the available width for the project W :

$$W_u = \frac{W}{N_u} \quad (1)$$

The scale of the project model x is then defined as the ratio between the widths of PKW-units on the project Wu and on the reference model Wu*

$$x = \frac{W_u}{W_u^*} \quad (2)$$

Applying this scale on the design water head H, the corresponding head on the reference model H* is

$$H^* = \frac{H}{x} \quad (3)$$

The discharge coefficient Cdw, of the Poleni equation (Eqn), usually used to characterize weirs efficiency, is a non-dimensional number. There is thus no scaling on its value and the Cdw value of the project model for the design head H is equal to the Cdw* value of the reference model at the corresponding head H*(Eqn).

$$Q = C_{dw} W . \sqrt{2gH^3} \quad (4)$$

$$C_{dw}(H) = C_{dw}^*(H^*) \quad (5)$$

inserting Eqn to and in the Poleni equation , a relation between the head and the discharge is obtained for the project model depending on the hydraulic and geometric characteristics of the reference model (Wu* and Cdw*), the project constraint (W) and the number of PKW-units on the project

$$Q = C_{dw}^{*}\ W\ .\sqrt{2gH^{3}}\quad \text{with}\ C_{dw}^{*}\ = f\left(\frac{H\,W_{u}^{*}\,.N_{u}}{W}\right) \quad (6)$$

If the discharge coefficient of the reference model is not known for the corresponding head, its value is calculated by extrapolation in-between the existing values. The accuracy of the design method is thus directly link to the experimental tests accuracy and to the number of available results for the reference model. By drawing the head/discharge curves defined by Eqn for different numbers of PKW-units and limiting these curves to head and discharge values under the design head and over the design discharge, different designs may respond to the project constraints. The second step of the method is then to optimize remaining parameters depending on the project engineer’s interests, to clearly define the final design. By application of the scale factor to the reference model dimensions X*, the project model dimensions X are completely defined (Eqn (7)), per-mitting the optimization of the final design including structural, economical or hydraulic criteria

$$X = x.X^{*} \quad (7)$$

The method can finally be summarized as the following four

steps:

1) Choose of a reference model ;
2)Scaling of the geometric and hydraulic characteristics of the reference model, corresponding with different number of PKWunits;

Isolation of the designs enabled to respond to the project

3) constraints;

4)Optimization of the design based on structural, economical nor hydraulic criteria, depending on the project engineer’s interests.

This method has been applied to the pre-design of varied PKW configurations (Machine's 2012) highlighting a good prediction of the experimental results as long as the side effects are still moderated. This is true for a large number of PKW-units or for still reservoir conditions.

APPLICATION

To show the interest of the method to optimize the design regarding the project engineer’s interests, it has been applied to the preliminary design of a PKW. Let us consider a dam project with features as summarized in Table 1. The discharge to release on the PKW at maximal reservoir level is 400 m3/s. The specific discharge is thus 11.5 m3/s/m, to be evacuated under a head of 2 m. The choice of the reference model highly influences the accuracy of the final design and has to be realized according to the project constraints and the range of available results (Michael's 2012). From the different available results from the literature (Hien et al. 2006, Le Doucen et al. 2009, Machiels et al. 2009), the model from Biskra (Ouamane and Lempérière 2006a) has been choosen as the reference model. Indeed, this model presents efficient geometric (symmetric overhangs of limited length) and hydraulic (adimensional

Table 1 : Project dam and reservoir characteristics

Available crest length	35 m
Normal reservoir level	452.0 m NGF
Maximal reservoir level	454.0 m NGF

head/discharge curve) characteristics regarding the project constraints. Furthermore, a large set of results are available for this geometry, which increases the method accuracy (Table 2). The geometric characteristics of the model are given in Table 3 and the experimental results are summarized in Figure 2 in terms of discharge coefficient Cdw of the Poleni equation function of the ratio between the water head H and the weir height P.

Considering a 5-units PKW, the width of each unit is 7 m (Eqn). It represents a scale ratio of 42.4 between the project and the reference model (Eqn). The corresponding design head on the reference model is 0.0471 m (Eqn). By interpolation in-between the experimental results, the discharge coefficient of the PKW for the design head is equal to 1.56 (Eqn). A PKW with 5 units placed on the crest of the projected dam enables to discharge 685 m3/s (Eqn). A 5-units PKW enables thus to respect the project constraints. The project characteristics calculated for Nu varying from 5 to 11 are presented in Table 4. Figure 3 shows the head/ discharge curves calculated for these Nu values. Many solutions exist to ensure the dam safety. An optimization of the design on different parameters is thus possible. An increase in the normal reservoir level enables to increase the hydropower capacity. A first optimization could thus consist in the limitation of the design head to 1.5m,

Table 2 : Hydraulic characteristics of the reference model(Biskra)

H*/P*	C_{dw}	H*/P*	C_{dw}
0.16	2.36	0.51	1.10
0.20	2.12	0.65	0.92
0.24	1.85	0.70	0.88
0.26	1.76	0.72	0.86
0.30	1.58	0.76	0.84
0.32	1.52	0.78	0.82
0.33	1.45	0.80	0.81
0.36	1.42	0.82	0.80
0.37	1.37	0.84	0.79
0.46	1.19	0.86	0.78

Table 3 : Geometric characteristics of the reference model (Biskra)

P* (m)	0.155
W_i* (m)	0.089
W_o* (m)	0.074
B_o* (m)	0.103
B_i* (m)	0.103
T* (m)	0.003
B* (m)	0.412

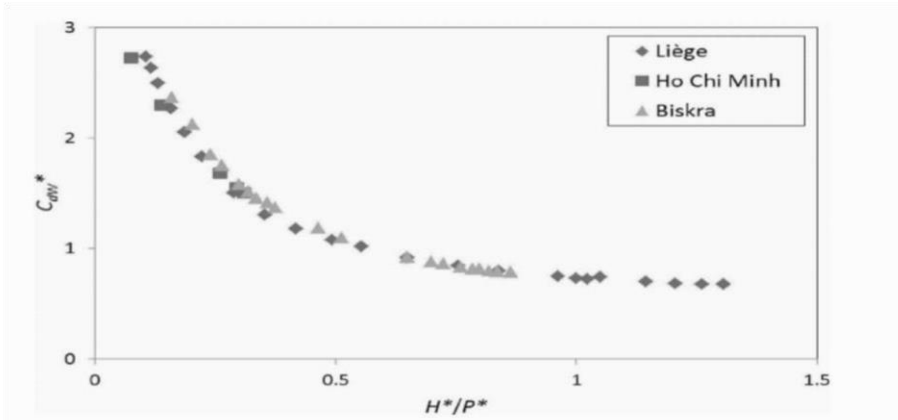


Figure 2 : Non-dimensional head/discharge curves for the available reference models

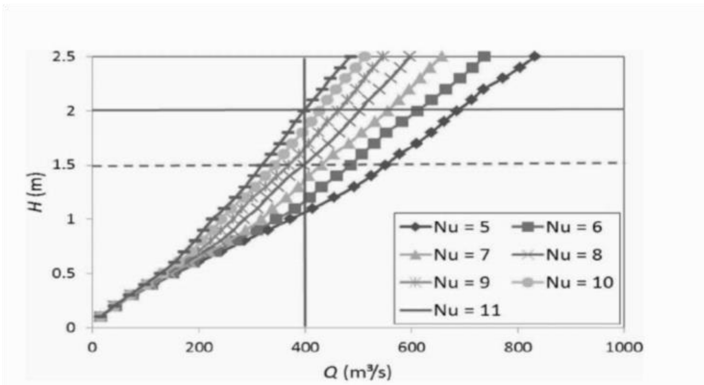


Figure 3 : Project head/discharge curves for various Nu values

Table 4 : Project characteristics for various Nu values

N_u	5	6	7	8	9	10	11
P (m)	6.6	5.5	4.7	4.1	3.7	3.3	3.0
W_i (m)	3.8	3.2	2.7	2.4	2.1	1.9	1.7
W_o (m)	3.2	2.7	2.3	2.0	1.8	1.6	1.4
$B_o = B_i$ (m)	4.4	3.6	3.1	2.7	2.4	2.2	2.0
B (m)	17.5	14.6	12.5	10.9	9.7	8.7	7.9
Volume of concrete (m^3) – T = 0.2 m	267	223	191	167	149	134	122
C_{dw} for H = 2 m	1.56	1.39	1.26	1.14	1.05	0.97	0.90
Q (m^3/s) for H = 2 m	685	611	554	502	461	427	397

enabling to fix the normal reservoir level to 452.5 m NGF and limiting the solutions to Nu values under 8 (Figure 3Another optimization could concern the structural con-straints. By limitation of the basis position of the PKW over 446.5 m NGF (50 cm over the gated weir crest), the reservoir and the existing weir stays partially usable during PKW construction. There is thus no need of diversion works and the hydroelectric power plant could still been used during the PKW construction. The weir height is thus limiting under 6 m, what involve Nu values over 6 (Table 4). The final choice between the four last solutions could be realized to minimize the necessary volume of concrete Considering a thickness of the walls of 20 cm (Table 4), the PKW design considering 7 units is the best solution Other designs may be optimum considering other criteria for optimization. For instance, a 5-units PKW ensures a higher level of security, and a 10-units PKW may suggest the use of prefabricated elements due to the smaller size of the alveoli.

5- CONCLUSION

Based on existing experimental results, a preliminary design method for PKW is proposed. In a first step, the different existing designs, enable to fit to the project constraints, are define by scaling the different geometric and hydraulic parameters of a reference model, considering different number of PKW-units. An optimal design is then clearly defined based on structural, economical or hydraulic criteria, depending on the project engineer’s interests. The application of the method highlights its interests to overcome varied projects constraints. The accuracy of the method is mainly related to the available results accuracy. The insertion in this method of new experimental results from more efficient scale models would enable to improve the design efficiency. In the future, numerical models should help in improving this preliminary design, before detailed scale model studies.

6 REFERENCES

1-Anderson R.M. and Tullis B.P. (2011) — Influence of Piano Key Weir geometry on discharge. Labyrinth and piano key weirs -PKW 2011, CRC press, London, 75-80

2-Bieri M., riBeiro M.L., BoillAT J.-L., schleiss A., lAugier F delorme F. and VillArd J.-F. (2009) — Réhabilitation de la capacité d'évacuation des crues : Intégration de « PK-Weirs»., SHF : « Dimensionnement et fonctionnement des évacuateurs de crues », Paris, France.

3- BlAnc P. and lempérière F. (2001) — Labyrinth spillways have apromising future. Hydropower & Dams. (4) 129

4-erpicum S., mAchiels O., ArchAmBeAu P., dewAls B.J. and piroTTon M. (2010) — 1D numerical approach to model the flow over a Piano Key Weir (PKW), in proceedings of SimHydro 2010, Nice, France.

5- erpicum S., mAchiels O., ArchA mBeAu P., dewAls B. and piroTTon M. (2011) — 1D numerical modeling of the flow over a Piano Key Weir, Labyrinth and piano key weirs-PKW 2011, CRC Press, London, 151-158.

6-erpicum S., mAchiels O., dewAls B., piroTTon M. and ArchAmBeAu P. (2012) — Numerical and physical hydraulic modelling of Piano Key Weirs, in proceedings of ASIA

7-mAchiels O., erpicum S., dewAls B., ArchAmBeAu P. and piroTTon M. (2011) — Experimental observation of flow characteristics over a Piano Key Weir. Journal of hydraulic research. 49 (3) 359-366.

8-mAchiels O. (2012) — Experimental study of the hydraulic behavior of Piano Key Weirs, PhD Thesis, Faculty of Applied Science, University of Liège, Belgium.

9-ouAmAne A. and lempérière F. (2003) — The piano keys weir a new cost-effective solution for spillways. Hydropower& Dams.

10- ouAmAne A. and lempérière F. (2006a) — Design of a new economic shape of weir, Dams and Reservoirs, Society and Environment in the 21st Century.

11- ouAmAne A. and lempérière F. (2006b) — Nouvelle conception de déversoir pour l'accroissement de la capacité des retenues des barrages, in proceedings of Colloque international surla protection et la préservation des ressources en eau, Bilda Algérie;

12- prAlong J., monTArros F., BlAncher B. and lAugier F. (2011— A sensitivity analysis of Piano Key Weirs geometrical parameters based on 3D numerical modeling, Labyrinth and piano key weirsPKW 2011, CRC Press, London, 133-139.

13- riBeiro M.L., AlBAlAT C., BoillAT J.-L., schleiss A.J. and lAugier F.

(2007) — Rehabilitation of St-Marc dam.

14- Experimental optimization of a piano key weir, in 32th Cong proceedings of IAHR, Venice, Italy.

15- lAugier F. (2007) — Design and construction of the first Piano Key Weir (PKW) spillway at the Goulours dam. Hydropower & Dams. (5) 94-101.

16- lAugier F., lochu A., gille C., riBeiro M.L. and BoillAT J.L.(2009) — Design and construction of a labyrinth PKW spillway at Saint-Marc dam, France. Hydropower & Dams. 16 (5) 100-107

17- lAugier F., Vermeulen J. and prAlong J. (2012) — Achievement of New Innovative Labyrinth Piano Key Weir Spillways (PKW), in proceedings of Piano Key Weir for in-stream storage and dam safety (PKWISD-2012), New Delhi, India, 25-42.

18- le doucen O., riBeiro M.L., BoillAT J.-L., schleiss A. and lAugier F. (2009) — Etude paramétrique de la capacité des PK-Weirs, in proceedings of Modèles physiques hydrauliques outils

indispensables du XXIe siècle, SHF, Lyon.