LOCAL SCOUR NEAR STRUCTURES

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1 Introduction

Local scour is herein considered to be the lowering of the bed in the direct vicinity of a structure due to local accelerations and decelerations of the near-bed velocities and the associated turbulence (vortices) leading to an increase of the local sand transport capacity. Once a scour hole is formed, flow separation will take at the edge of the hole and a mixing layer will develop increasing the turbulence intensities and stimulating further scour of the bed (self-intensifying process). Excessive scour close to the structure may ultimately lead to instability/failure of the structure.

Herein, scour by currents, waves and combined waves and currents is considered. The scour is generally referred to as clear water scour if the ambient bed-shear stress is smaller than that for initiation of motion and to as live-bed scour otherwise. The EXCEL-file **SCOUR.xls** can be used for determination of scour depth and length estimates (Van Rijn, 2006, 2012).

Bed scour problems near walls and breakwaters generally occur near the outer toe of the trunk section of the structure and near the tip of the structure and is predominantly related to the height of spilling and plunging breaking waves during storm events, but wave reflection (and standing wave patterns) may also be important for (nearly) vertical structures. Since, the breaking wave height is depth-limited (roughly between 0.5 h for an almost flat bottom and 1 h for a steep bottom), it is most logic to assume that the maximum scour depth is related to the water depth near the toe/tip of the structure.

Various mitigating measures are available to reduce or prevent local scour processes, such as: bottom/bank protection by means of rip-rap material (stones) dumped on geotextile filter material, by flexible matts or matrasses filled with gravel/sand, by sand bags, by artificial matts, by concrete slabs and by grout injections.

Reviews of bed scour near structures are given by **Powell (1987), Kraus (1988), Fowler (1992), Kraus and McDougal (1996), Herbich (1991), Silvester (1991), Oumeraci (1994 a,b), Hoffmans and Verheij (1997), Whitehouse (1998), Sumer et al. (2001) and Sumer and Fredsøe (2002).**

2 Scour downstream of sills, weirs and barrages

Two-dimensional vertical scour downstream of a structure such as a weir or a barrage in a unidirectional current (see Figure 2.1) has been studied by many researchers (see **Hoffmans and Verheij, 1997**). The maximum scour depth in the equilibrium situation as well as the development in time of the scour depth have been studied.

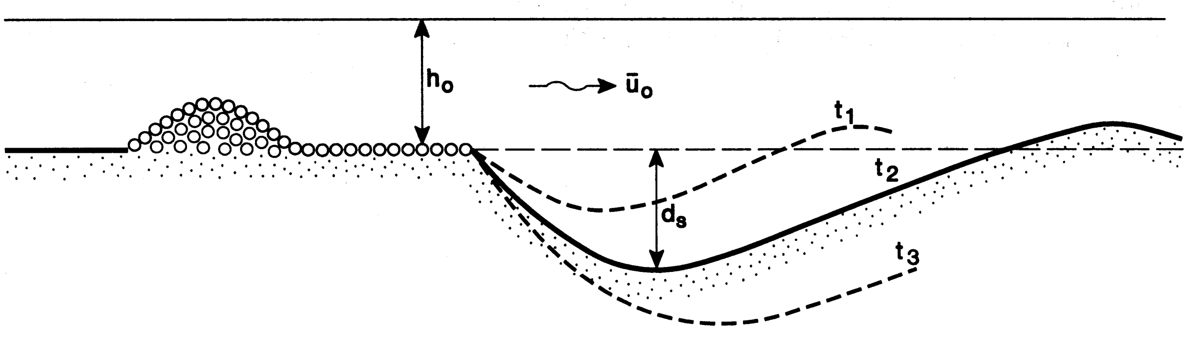

Figure 2.1 *Two-dimensional scour downstream of structure*

Delft Hydraulics (Breusers, 1967) studied the time-dependent behaviour of scour holes (in sandy beds) related to closure works in tidal channels.

Based on experimental research in flumes, the time-dependent development of the scour depth in clear water flows was found to be:

$$\mathbf{d_s(t)/h_0=(t/T_s)^{0.38}} \quad (2.1)$$

with: $d_s(t)$ = maximum scour depth at time t below original bed, see Fig. 2.1, h_0=upstream water depth, T_s= time (in hours) at which $d_s = h_0$.

Equation (2.1) is not valid close to the equilibrium situation.
A more general expression is: $d_s(t)/d_{s,max}=1-\exp(-t/T_s)^p$ with p=calibration coefficient.
The time-scale T_s (in hours) was found to be:

$$\mathbf{T_s = 330(s-1)^{1.7} (h_0)^2/(\alpha U_0-U_{cr})^{4.3}} \quad (2.2)$$

with: U_0 = depth-averaged velocity just upstream (x = 0) of scour hole,U_{cr}=critical depth-averaged velocity (initiation of motion), s = specific density (ρ_s/ρ_w),α= coefficient depending on flow and turbulence structure at the upstream end of scour hole (α= 1.7 for two-dimensional flow without structure, α= 3 for very violent three-dimensional flow, **Van der Meulen and Vinjé, 1975**).

The α-factor is related to the relative turbulence intensity $r_0=\sigma_u/U$ directly upstream of the scour hole (σ_u= standard deviation of local velocity field). For hydraulic rough flow it was found that $\alpha=1.5+5r_0$. The value of r_0 depends on the type of structure and the length of the bed protection downstream of the structure. If this length is larger than $30h_o$, additional turbulence produced by the structure has decayed and the r_0-value for uniform flow without a structure can be taken, yielding: r_0=0.1 to 0.15.
Generally-accepted formulae for the maximum scour depth in the equilibrium situation are not available. A rough estimate can be obtained from **(Dietz, 1969; Schoppman, 1972)**:

$$\mathbf{d_{s,max}/h_0 = (\alpha_d U_0 - U_{cr})/U_{cr}} \quad (2.3)$$

with: $\alpha_d = 1+3r_0$.

Usually, the river bed downstream of a weir or barrage is protected over a certain distance to reduce the maximum scour depth which is strongly dependent on the α-factor (α decreases with distance due to the decay of turbulence). The bed protection length generally is of the order of 10 to 20 h_0. The surface of the protection layer should be as rough as possible to reduce the near-bed velocities and hence scour rates.
The maximum scour depth will be reduced, if there is a supply of sediment from the upstream river section (or from the flood and ebb direction in tidal flow). In the case of tidal flow the current velocity can be schematized by an effective current velocity $U_{max,eff}=0.9U_{max,\ mean\ tide}$ to represent the velocity variation over the daily cycle and the neap-spring cycle. The bottom slope at the beginning of the scour hole may become quite steep; slopes of 1 to 2 and 1 to 3 have been observed for r_0= 0.2 to 0.4. Undermining of the bed protection at this location should be prevented. Model studies are recommended for complicated geometries.

Scour data observed near the storm surge barrier in the Eastern Scheldt, The Netherlands show scour depths of $d_{s,max}$=0.4 to 1 h_o **(Hoffmans and Verheij, 1997)**. The observed scour depths are considerably smaller than those predicted by Eq. (2.3), because sediments supplied by the bidirectional tidal flow are partly trapped in the scour hole (reduction of scour depth due to upstream supply). This latter effect is not taken into account by Equation (2.3).

3 Scour near seawalls

3.1 Review of scour data

Seawalls are generally built on receding shorelines to protect the mainland against retreat and inundation. They are not built to maintain the beach (if present) in front of the seawall. Often, the recession of adjacent shorelines is continued and even accelerated by the interaction of the seawall with the morphological system. **Pilkey and Wright (1988)** distinguished between passive erosion and active erosion; the former being the natural erosion before construction of the wall (ultimately resulting in a more exposed position of the seawall) and the latter being the additional erosion caused by the presence of the wall.

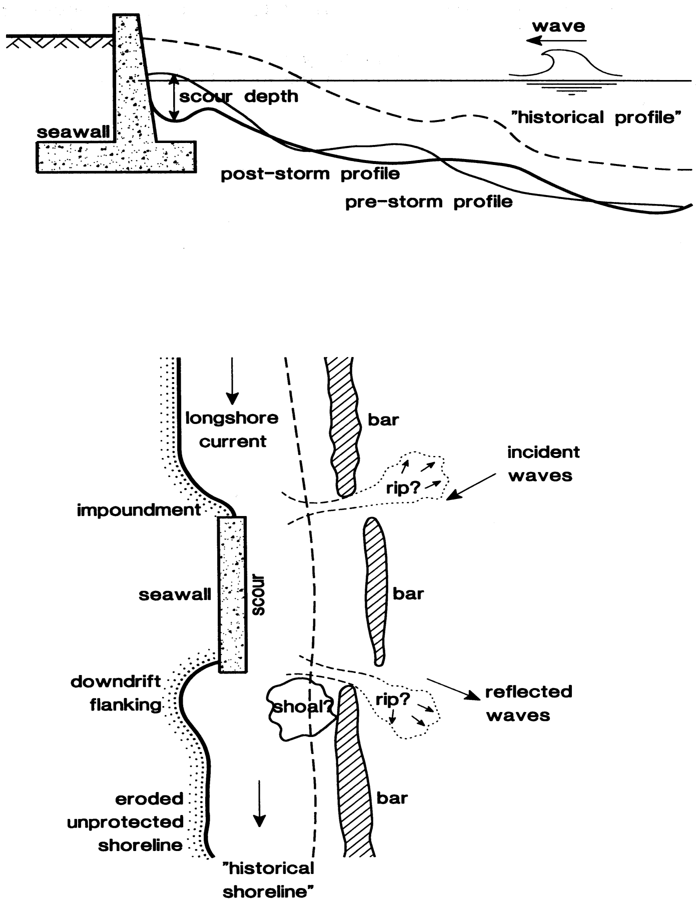

Figure 3.1 *The effects of seawalls on the beach*
Top: scour at toe of wall
Bottom: scour at end of wall

Scour near seawalls can be classified as (see Figure 3.1):

- scour at the toe of the wall; the maximium scour depth ($d_{s,max}$) is the depth below the position of the original sand surface (before the presence of the structure);
- scour of dune and beach on both ends of the wall (lee-side scour) resulting in a more exposed position of the wall and consequent narrowing of the beach in front of the wall by accelerating longshore currents around the protruding wall.

Seawalls contribute to erosion and scour by the following processes:

- interaction of incident and reflected waves and associated wave-induced drift velocities above the sand bed near the stucture;
- enhancement of offshore-directed transport by waves breaking at or near the wall (generation of undertow and stirring of sediment);
- blocking (partly) of the updrift longshore transport in case of a protruding seawall; longshore currents in front of a protruding wall are accelerated resulting in bed erosion and general lowering/steepening of bed (and hence more intensive wave attack); increased turbulence and circulations generated at the downdrift end of the wall lead to scour and retreat of the shoreline;
- impoundment of sediment behind the wall, which would otherwise be released to the littoral drift system.

A review of the effects of seawalls on the beach has been given by **Kraus (1988)** and by **Kraus and McDougal (1996)**. Their main findings are:

Based on laboratory studies

- the primary force of wave action alone does not lead to severe toe scour; the scour depth increases strongly when currents are present;
- the maximum scour depth is approximately equal to 0.5 to 1 times the significant wave height in deeper water ($d_{s,max}/H_{sig,0}$= 0.5 to 1) for an unbarred bottom profile **(Fowler, 1992)**;
- an inclined wall produces less scour than a vertical wall;
- scour is reduced if the seawall is situated at the most landward position (not protruding in the surf zone);
- scour patterns due to (partial) standing waves in front of seawall depend on the mode of sediment transport: bed-load transport or dominant suspended load transport;
- beach recovery and reduction of scour depth during fairweather conditions is possible;
- formation of bar-trough system in front of seawall is not necessarily disturbed;
- increased beach erosion at downdrift end of seawall (lee-side erosion) may occur; the alongshore erosion length for an isolated seawall was found to be about L_e= 0.7 L_{wall}; the maximum shoreline retreat at the end of the wall was $y_{s,max}$= 0.1 L_{wall} with L_{wall}= alongshore length of seawall **(Komar and McDougall, 1988)**;

Based on field studies

- the impact of a seawall on a beach is a long-term phenomenon (decades); short-term observations do not give proper results; long-term scour in front of wall may be more serious than short-term scour due to storm event; scour trough may be filled rather quickly after storm event;
- quantitative data of toe scour depths are hardly available; some scattered data suggest values of $d_{s,max}/h_{toe}$=0.5 to 1, but other data show no scour at all **(Griggs et al., 1990, 1994)**;
- the maximum scour depth at the toe is mostly assumed to be equal to the significant wave height at the edge of the surf zone (deeper water) during a storm event ($d_{s,max}/H_{sig,storm}$= 1) for an unbarred bottom profile; this will give a rather conservative estimate for a barred profile and for less exposed seawalls at the backbeach;
- maximum scour is expected to occur when the water level is highest (peak surge level), because the higher water level can support larger waves;
- the additional scour in front of a seawall is approximately equal to the amount of sediment behind the wall that would erode in the absence of a seawall **(Dean, 1986)**; this principle is difficult to apply, because it requires information of beach profiles without a seawall before and after a storm event;
- seawalls in the backshore with a beach in front give better performance than those without a beach; the impact of the wall is strongly dependent on its position with respect to the low water line; erosion is minimum if the seawall is built as far landward as possible (landward of level of maximum run-up during storm event); erosion is maximum if the seawall is built at a location seaward of the low water line so that waves will reflect and or break against the wall;
- reflective vertical or near-vertical seawalls cause relatively large scour depths at the toe; scour was found to be minimum in front of a dissipative rubble-mound seawall; reflection itself is not found to be a great contributor to scour in front of seawalls;

- erosion of berm and beach in front of seawall (located at the backshore) is of the same order of magnitude as that of adjacent beaches, but the erosion process proceeds faster if waves overtopping the beach/berm can reflect or break against the wall; narrow, steep beaches in front of seawalls are often severely eroded during storm events;
- rip currents enhance scour in front of wall; accelerating longshore currents around a protruding seawall enhance scour of the bed;
- the widths of dry beaches in front of natural shorelines (South and North Carolina and New Yersey, USA) were found to be consistently wider than those in front of hard structures; the higher the degree of stabilization, the narrower the beach **(Pilkey and Wright, 1988)**;
- beach recovery in front of a seawall after a storm event proceeds in a similar way or somewhat slower than for a natural beach; the overall recovery often is partial for a narrow and steep beach;
- longshore bar-trough system in front of a wall need not to be destroyed and can develop in much the same way as at beaches without a wall;
- beach erosion at downdrift end of wall (lee-side erosion) is often increased.

3.2 Wave-related scour near toe of seawall

The basic shape of a toe scour hole **(Steetzel, 1988)** is shown in Figure 3.2. The proper determination of the water depth at the toe (h_{toe}) of the structure may give problems in field conditions.

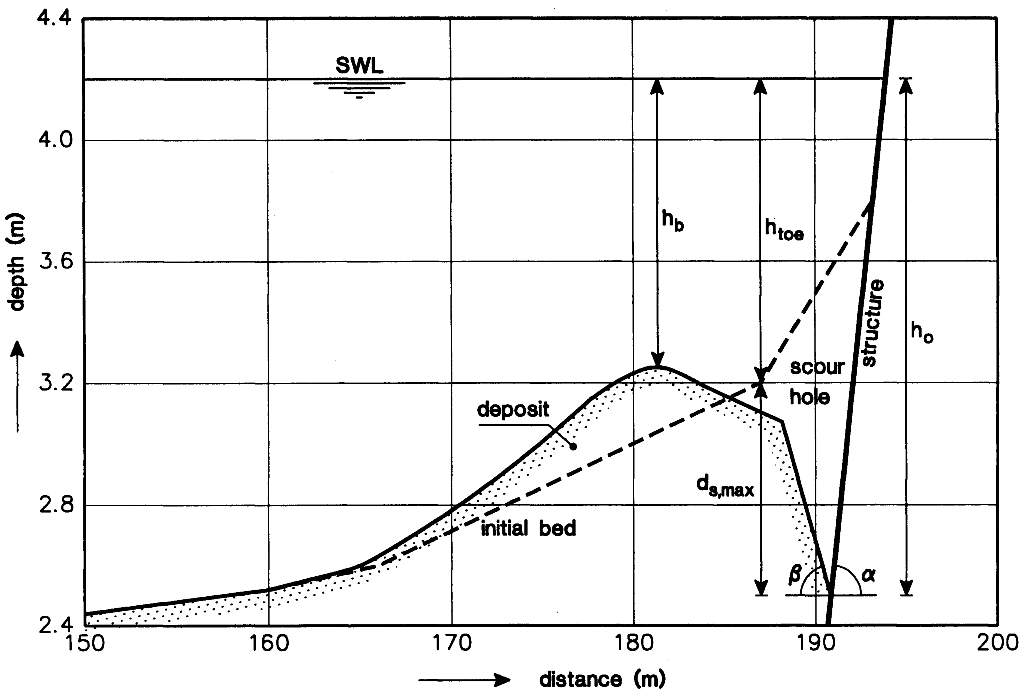

Figure 3.2 *Basic shape of scour hole near toe of seawall (Steetzel, 1988)*

According to the **Shore Protection Manual (1984),** the scour depth is given by the following simple rule:

$$\mathbf{d_{s,max} = H} \tag{3.1}$$

with H= height of maximum unbroken wave at toe of structure.

Many researchers have conducted two-dimensional movable-bed laboratory tests to determine the toe scour of wall-type breakwaters (see **Kraus, 1988**).
Hereafter, some examples of laboratory experiments are given.

Herbich et al. (1965) performed two-dimensional movable-bed tests in a laboratory flume with regular non-breaking waves (period of about 1.5 s) on walls made of plexiglas. The slope angle (α) of the wall was varied in the range of 15^o to 90^o (90^o= vertical). The bed material consisted of sand with a median diameter of 0.483 mm.

The most important results are, as follows:

- slope angle of 15^o: wave reflection was less than 20% and the equilibrium scour depth below the natural bed ($d_{s,max}$) was about $d_{s,max}/H$= 0.4 to 0.45 with H= incident wave height;
- slope angle of 30^o to 90^o: wave reflection was larger than 40% and the equilibrium scour depth was about $d_{s,max}/H$= 0.5 to 0.6;
- primary scour was observed under the nodes of the wave envelope;
- (partial) standing waves were observed to give patterns of alternating scour and deposition in front of the wall;
- more reflective conditions resulted in an increase of the scour depth,
- scouring always occurred within a distance of 1/4 L (L= wave length) from the toe of the structure.

Steetzel (1988) analyzed toe scour near structures both in field conditions and in small-scale and large-scale laboratory experiments. His findings are:

- the scour depth is strongly related to the incident wave conditions, surge levels, beach slope and water depth near the toe;
- the maximum water depth including the scour depth was found to be h_0/h_b=1.7 to 1.8 (see Figure 3.2) with h_0= maximum water depth in scour hole, h_b= minimum water depth above bar deposit; this is roughly equivalent with $d_{s,max}/h_{toe}$= 0.75 (see Figure 3.2);
- the maximum value of the landward slope of the scour hole was between 1 to 3 (tanβ=0.33) and 1 to 5 (tanβ=0.2);
- the shape of the scour hole is related to the steepness of the seawall; the maximum scour depth is closer to the wall for a steeper slope of the wall.

Fowler (1992) analyzed laboratory test results and proposed an empirical method to determine the scour depth at the toe of vertical walls. Based on this approach, the maximum scour depth roughly is:

$$
\begin{aligned}
&\mathbf{d_{s,max}/H_{s,0}= 0.6} && \mathbf{for\ h_{toe}/L_0= 0.005} \\
&\mathbf{d_{s,max}/H_{s,0}= 0.8} && \mathbf{for\ h_{toe}/L_0= 0.02} \\
&\mathbf{d_{s,max}/H_{s,0}= 1.0} && \mathbf{for\ h_{toe}/L_0= 0.04}
\end{aligned}
\qquad (3.2)
$$

with: $H_{s,0}$= significant wave height in deep water, L_0= wave length in deep water, h_{toe}= water depth at toe of structure.

The scour depth increases with decreasing wave length, because shorter waves tend to break against or in front of the wall. Breaking waves produce a larger scour depth.

Kraus and McDougal (1996) reported about scour at the toe of a seawall due to breaking waves in large-scale tests conducted in the USA. Two-dimensional tests were conducted in a large-scale flume (Supertank at the Hinsdale Wave Research Laboratory, Oregon State, USA). The beach material consisted of uniform 0.22 mm-sand. The significant offshore wave heights ranged between 0.4 and 1.0 m and periods between 3 and 8 s. The vertical wall was placed at the end of the beach. A remarkable result was that the bed profiles in front of the wall did not show a large scour trench. A rather small scour trench was created at the toe of the wall, but the influence was highly localized in the immediate vicinity of the wall. The maximum scour depth was about 0.3 m after 10,000 waves in a (original) water depth of about h= 0.5 m. Thus, $d_{s,max}$= 0.6 h. Scouring of the bed was not observed outside a distance of 5 times the initial water depth at the toe. Reflection was found to be a relatively unimportant parameter in the scouring process.

4 Scour near toe of wall-type breakwaters

Wall-type breakwaters are structures perpendicular or oblique to the shoreline; the seaward end section of the breakwater may run more or less parallel to the shoreline. Breakwaters may also be built as submerged structures parallel to the shoreline. Generally, the bed surface in front of a breakwater is relatively flat, whereas the bed (beach) in front of a seawall is relatively steep. For waves approaching normal to the structure, the scouring process is similar to that near a seawall.

The basic processes are:

- interaction of incident and reflected waves, yielding wave-induced drift velocities above the sand bed near the structure (relatively slow process);
- interaction of waves breaking in front of the structure and associated return currents (undertow) above the sand bed (relatively rapid process);
- seaward-directed currents generated along the structure in case of oblique (breaking) waves.

Irie and Nadaoka (1984) studied scour by reflecting non-breaking waves in two- and three-dimensional laboratory models with various sediments (sand of 0.2 mm and 0.33 mm; light-weight coal material of 0.33 mm). Their results are:

- deposition at the nodal locations and scour at the antinodal locations (N-type scour); this will occur when the bed-load transport is dominant because wave-induced drift velocities (under partial or full standing waves) near the bed cause the bed-load grains to move toward the nodes of the standing waves (see Figure 4.1);
- scour at the nodal locations (L-type scour) and deposition at the antinodal locations; this will occur when the suspended load transport is dominant due to the presence of drift velocities (above the wave boundary layer) in the direction from nodes to antinodes (see Figure 4.1); vortices generated in the scour hole enhance the movement of sediment to the nodes on both sides of the scour hole.

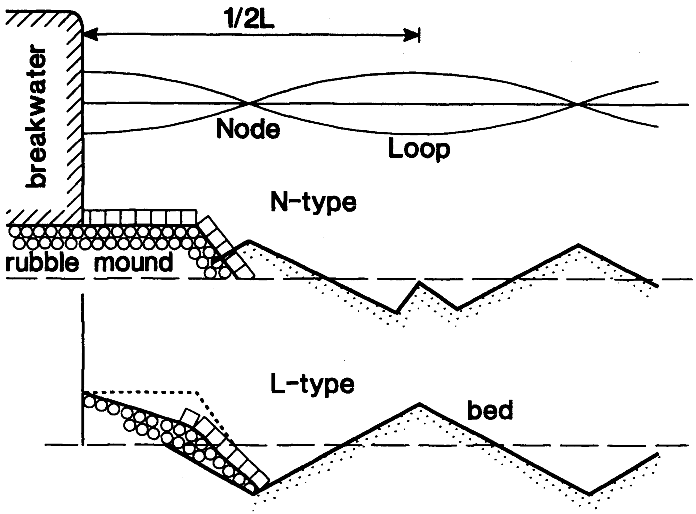

Figure 4.1 *Scour by standing waves*
Top: N-type scour for dominant bed-load transport conditions
Bottom: L-type scour for dominant suspended transport conditions

L-type scour under suspended load conditions during storm events is most critical for the stability of the structure, because the scour hole develops close to the toe of the structure. This type of scour was found to be dominant for $U_{max}/w_s>10$ with U_{max}= near-bed peak orbital velocity and w_s= fall velocity of sediment. Toe protection should have a length equal to about $0.25L_w$.

A two-dimensional wave flume test with regular waves of 0.12 m (period of 1.4 s) in a depth of 0.3 m over a fine sand bed of 0.06 mm resulted in a scour hole with a maximum depth of $d_{s,max}/h$= 0.25 with h= depth at the toe.

Three-dimensional tests with irregular waves at 30° degrees (to a line normal to the breakwater) over a sand bed of 0.13 mm showed near-bed drift velocities parallel to the breakwater in the direction of the shoreline and scour at the nodal locations close to the toe in the case of dominant suspended load transport. Scour was found to be largest near the tip of the breakwater.

Table 4.1 shows scour depth values at the toe of detached vertical breakwaters given by **Sumer and Fredsøe (2000)** and by **Sumer et al. (2001)**. Regular and irregular waves were generated in a 2D wave flume with a sand bed (0.2 mm-sand) and a water depth of 0.3 m. Bed-load transport without much suspension was observed in the tests. The toe scour data are in agreement with those of **Xie (1981)** for a breakwater with a vertical wall. The scour depths of Table 4.1 show an increasing trend with increasing wave length. This trend is opposite to the data of Fowler (1992). The scour depth strongly decreases with decreasing side slope angle of the breakwater.

The scour depth was somewhat smaller in tests with irregular waves than in tests with regular waves. Deposition was observed at the location of the nodal points in front of the structure. The data of Table 4.1 in the bed-load transport regime are representative for normal daily wave conditions. The scour depth in the suspended transport regime are representative for storm events. These latter scour depths are roughly 20% to 40% larger than those in the bed-load transport regime. Toe protection against scour should have a length equal to about $0.25L_w$.

Type	**Fine sand (suspended transport mode) based on Xie (1981)**	**Coarse sand (bed load transport mode) based on Sumer and Fredsøe (2000)**
Vertical wall	$d_{s,max}/H_{rms}$=1.0 for h/L_w=0.08 $d_{s,max}/H_{rms}$=0.7 for h/L_w=0.10 $d_{s,max}/H_{rms}$=0.35 for h/L_w=0.15	$d_{s,max}/H_{rms}$=0.8 for h/L_w=0.08 $d_{s,max}/H_{rms}$=0.5 for h/L_w=0.10 $d_{s,max}/H_{rms}$=0.25 for h/L_w=0.15
Rubble mound Slope angle=40° (1 to 1.2)	not tested	$d_{s,max}/H_{rms}$=0.35 for h/L_w=0.08 $d_{s,max}/H_{rms}$=0.30 for h/L_w=0.10 $d_{s,max}/H_{rms}$=0.15 for h/L_w=0.15
Rubble mound Slope angle=30° (1 to 1.75)	not tested	$d_{s,max}/H_{rms}$=0.15 for h/L_w=0.08 $d_{s,max}/H_{rms}$=0.10 for h/L_w=0.10 $d_{s,max}/H_{rms}$=0.05 for h/L_w=0.15

Table 4.1 *Scour depths at toe of breakwater (h= water depth in front of wall, but outside scour zone, H_{rms}=root-mean-square wave height in front of wall, outside of scour zone; L_w= wave length based on peak period in front of wall, outside scour zone; slope angle= angle of side slope with horizontal bottom)*

Field data of toe scour generally include the combined effect of currents and waves on the scouring process. Field results are given below.

Sato et al. (1968) studied toe scour near the vertical breakwater of Kashima Port and the east port of Niigata, Japan (see Fig. 4.2). Tidal currents are relatively small. The maximum scour depth near the breakwater of Kashima port was found to be 3 m, measured two weeks after a storm event. The maximum significant wave height was found to be 3 m at a depth of 12 m. Thus, the maximum scour depth is of the same order as the deep water wave height.
In terms of the initial water depth at the toe of the structure, the following values can be derived from their data (h_{toe}= initial water depth prior to construction):

$$\begin{aligned} d_{s,max}/h_{toe} &= 1.5 && \text{for } h_{toe}<2\text{ m,} \\ d_{s,max}/h_{toe} &= 0.5 && \text{for } h_{toe}=4\text{ m,} \\ d_{s,max}/h_{toe} &= 0.3 && \text{for } h_{toe}=7\text{ m,} \\ d_{s,max}/h_{toe} &= 0.1 && \text{for } h_{toe}=9\text{ m.} \end{aligned} \tag{4.1}$$

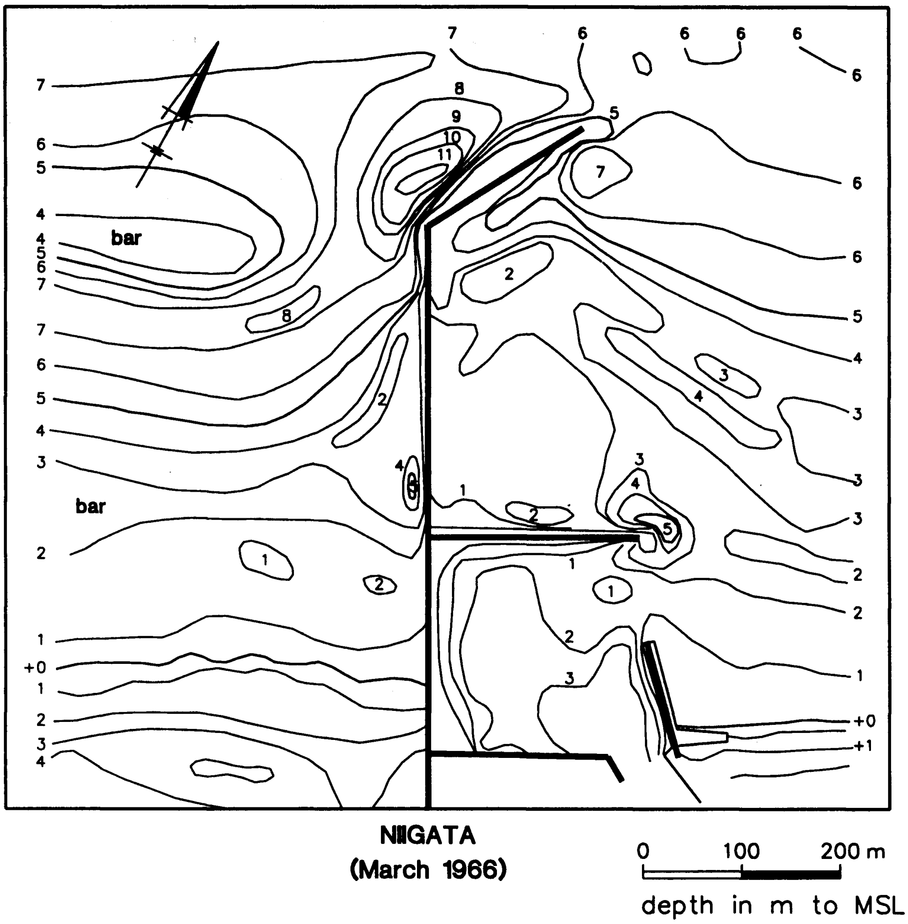

Figure 4.2 *Scour near breakwater of east port of Niigata, Japan (Sato et al., 1968)*

Scour was found to be maximum:

- in the zone where the breakwater crosses the longshore bar (see Figure 4.2),
- near the junction point (of different alignment angles) where seaward return currents are converging;
- near the tip of the breakwater due to relatively large gradients of wave energy and turbulence intensities.

The scour between the breakwater and the longshore bar at 4 m below MSL (see Figure 4.2) is of the order of the initial water depth ($d_{s,max}/h_{toe,initial}=1$). This relatively large value was believed to be related to the presence of seaward-directed rip currents, generated along the structure.

Yokoyama et al. (2002) have analysed field data and applied a numerical model to evaluate the scour depth near the toe of wall-type structures. From their graphs the following values can be obtained:

$$\begin{aligned} &\mathbf{d_{s,max}/H_s = 0.2} && \mathbf{for\ H_s/h_{toe} = 0.33} \\ &\mathbf{d_{s,max}/H_s = 0.6} && \mathbf{for\ H_s/h_{toe} = 0.5} \\ &\mathbf{d_{s,max}/H_s = 1.0} && \mathbf{for\ H_s/h_{toe} = 0.67} \\ &\mathbf{d_{s,max}/H_s = 1.5} && \mathbf{for\ H_s/h_{toe} = 1.0} \end{aligned} \qquad (4.2)$$

5 Scour near toe of rubble-type breakwaters

Wave reflection tests on breakwaters of different armour units in flumes **(Losada and Gimenez, 1981)** show that the reflection coefficients can be as large as 70% for rubble mound or Dolos elements with $\tan\alpha/(H/L_o)^{0.5}$ between 5 and 10.

Irie et al (1986) conducted three-dimensional tests in a laboratory basin with oblique regular and irregular waves on a rubble-mound breakwater. The bed material was uniform 0.14 mm-sand. The maximum scour depth was attained after 30,000 waves (10 hours) and found to be $d_{s,max}/h_{toe}$= 1. The scour depth was maximum within a distance of 1/2 L from the toe of the breakwater.

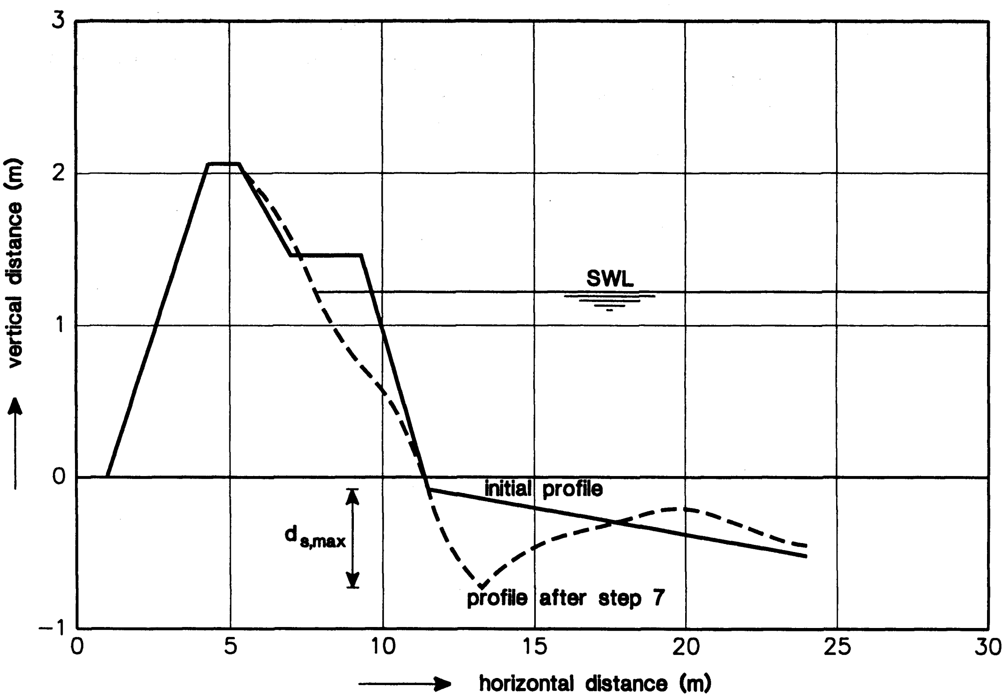

Figure 5.1 *Bed level profile at initial time and after step 7 for large-scale tests at Delft Hydraulics (1985)*

Delft Hydraulics (1985) reported about two-dimensional large-scale laboratory tests on a rubble-mound breakwater related to the design of the breakwater of St. George Harbor, Alaska. The bed (slope of 1 to 30) consisted of rather uniform 0.225 mm-sand. The breakwater consisted of a rubble-mound structure with a berm (berm width=2.5 m, outer slope of 1 to 1.5, crest about 0.3 m above MSL, see Figure 5.1). The design storm was represented in 8 steps (duration of 30 to 45 minutes) of different wave heights and periods, as given in Table 9.5.1. The water depth at the toe of the breakwater was 1.2 m. The relative wave height at the toe varied between 0.6 and 0.9. The maximum scour depth after step 7 was found to be $d_{s,max}/h$= 0.5 with h= water depth at toe.

Sumer and Fredsøe (2000) and **Sumer et al. (2001)** present results of toe scour in front of rubble-mound breakwaters based on tests in a 2D wave flume, see Table 4.1. The scour depth is significantly smaller than that near the toe of a vertical breakwater. Toe protection against scour should have a length equal to about $0.25L_w$.

Katayama et al. (1974) studied short-term scour at the toe and near the tip of an offshore breakwater (on the Niigata coast of Japan), which was temporarily submerged due to settlement and scour. The Niigata coast is exposed to severe wave action in winter season. The tidal range varies between 0.5 m and 1 m. The offshore breakwater was initially built as a partially submerged breakwater with a crest height of 1.1 m above low water (water depth of 4 m below low water level). The structure was heavily damaged due to scour beneath the structure and the crest height was raised to 3 m above LW.

The maximum scour depth was determined from the settlement of iron rings (free movable) along poles placed in the bed; the rings move downward if the bed is scoured. This technique has been used because it gives the maximum scour depth, not affected by post-storm deposition in the scour hole. Two situations were studied: crest height at 1 m above LW and crest height at -2 m below LW (damaged submerged structure).
The results are:

- crest height at -2 m below LW,
  - maximum scour depth of 4 m in water depth of about 4 m ($d_{s,max}/h_{toe}$=1) at the seaward side of the submerged structure; the maximum scour depth occurred at a distance of about 20 m from the toe of the structure; scour was negligible at a distance of 70 m from the toe;
  - maximum scour depth of 2.5 m at landward side of the structure due to wave overtopping and overplunging;
- crest height at 1 m above LW,
  - maximum scour depth of 2 m in water depth of about 4 m ($d_{s,max}/h_{toe}$=0.5) at the seaward side of the structure; the maximum scour depth occurred at a distance of about 20 m from the toe of the structure;
  - maximum scour depth of 0.8 m landward of breakwater due to longshore currents generated by water level variations.

Thus, the scour near a submerged breakwater is considerably larger than that near a breakwater with its crest level above LW. This is caused by wave overtopping and overplunging.

Ichikawa (1967), Silvester (1991) and Uda and Noguchi (1993) present data of short-term (2 to 3 years) scour near breakwaters in micro-tidal regimes for some Japanese ports.
Based on the data, the following rough scour ranges are given:

$d_{s,max}/h_{toe}$= 1 to 0.5 for vertical caisson-type structures in depths of 5 to 10 m, (5.1)
$d_{s,max}/h_{toe}$= 0.5 to 0.2 for vertical caisson-type structures in depths of 10 to 30 m,
$d_{s,max}/h_{toe}$= 0.3 to 0.2 for breakwaters with armour units in depths of 10 to 20 m.

6 Scour near tip of breakwaters and groynes

Scour near the tip of breakwaters can be classified as *current-dominated* scour or *wave-dominated* scour. Scour is considerably enhanced, if tide-, wind- and wave-induced longshore currents with velocities exceeding 0.5 m/s are present. Wave-related scour generally is dominant in micro-tidal conditions.

6.1 Wave-dominated scour near tip of vertical wall-type breakwater

Sumer and Fredsøe (1997) studied wave-dominated scour near the tip of a vertical wall-type (rounded tip) breakwater in laboratory conditions.

Based on flow visualization measurements, the scouring mechanisms were found to be:

- generation of vortices (see Fig. 6.1) in the lee-side zone of the wall for KC= 1 to 12; vortices are not generated for KC<1; KC=$U_{max}T/B$= Keulegan-Carpenter number, U_{max}= peak orbital near-bed velocity, T= wave period and B= width of wall;
- generation of lee-side vortices and horse-shoe vortices for KC>12; horse-shoe vortices are vortices generated near the bed in front of and along the tip of the wall due to rotation of the approaching flow; in field conditions the KC-number is of the order of 1 and therefore horse-shoe vortices are not of practical relevance.

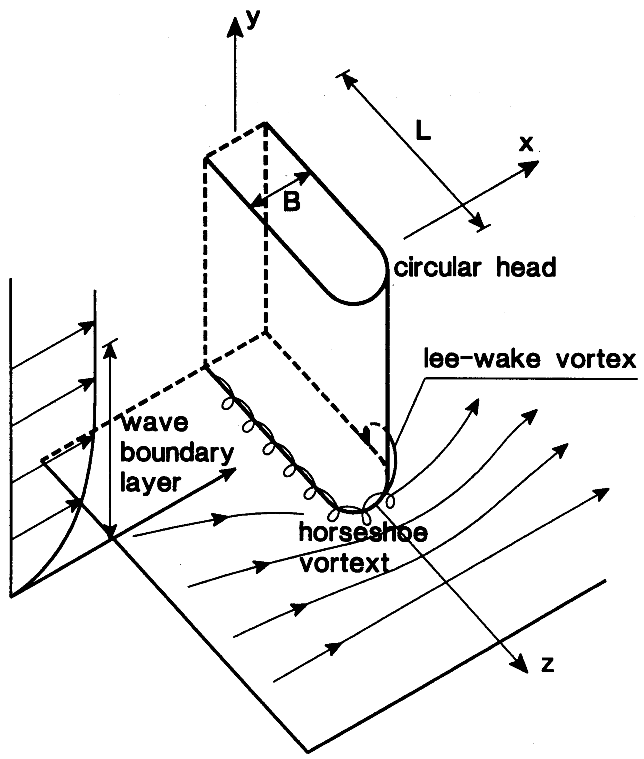

Figure 6.1 *Vortex patterns near tip of wall-type breakwater*

Scour tests over a movable bed of 0.17 mm-sand were conducted in a depth of 0.4 m with regular non-breaking waves (periods between 1 and 4 s). The width of the structure was B=0.14 and 0.40 m. Hence, the width-depth ratios were B/h=0.35 and 1. The observed maximum scour depths ($d_{s,max}/B$) for normal incident waves (90^o) were found to be related to the KC-number, see Table 6.1. The maximum scour depth was attained after about 1000 waves. The results are only valid for a vertical breakwater with a maximum width equal to the water depth (B/h=1).
The scour was maximum at the location of the tip (in the middle of the tip, see Fig. 6.2) of the breakwater. The observed scour length L_s (normal to wall) is also given in Table 6.1.

The maximum scour depth roughly increased by a factor 2 for a straight wall tip (sharp edge) in stead of a rounded tip.
The maximum scour depth increased by about 20% for oblique incident waves.
The maximum scour increased considerably, when the waves were superimposed on a following current (U_c). For example, KC= 2 and $U_c/(U_c+U_{max})$= 0.5 resulted in $d_{s,max}/B$= 1.

NON-BREAKING WAVES		
SCOUR DEPTH $d_{s,max}/B$	SCOUR LENGTH L_s/B	KC-number
0.02	0.5	1
0.1	1.5	2
0.2	2.5	4
0.3	3.5	7
0.4	-	10

Table 6.1 *Scour depth for normal incident non-breaking regular waves over a sand bed of 0.17 mm in a laboratory flume (Sumer and Fredsøe, 1997)*

The scour can be eliminated by means of a protection layer on the bed. The length L normal to the structure should be about L/B= 2 for KC= 2. In that case the maximum scour depth is reduced by a factor 3. In case of L/B=1, the maximum scour depth is reduced by about 30%

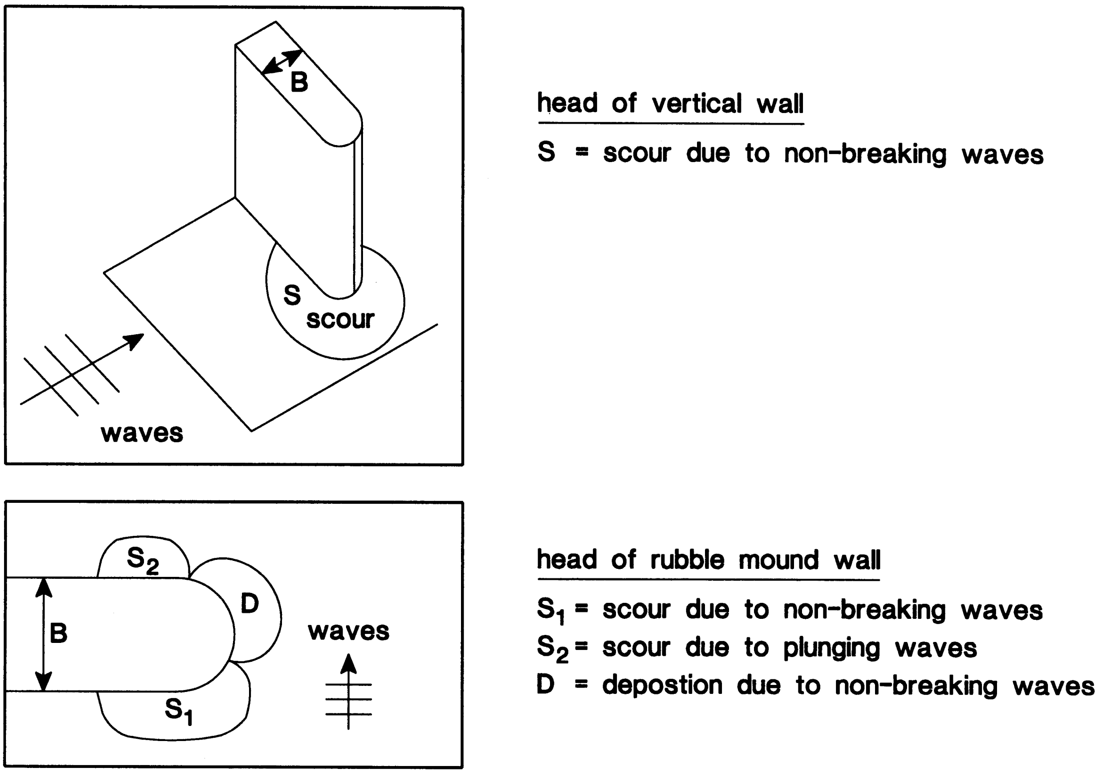

Figure 6.2 *Scour and deposition locations near vertical and rubble-mound breakwaters*
Top: Vertical wall
Bottom: Rubble-mound breakwater

6.2 Wave-dominated scour near tip of rubble-mound breakwater

Fredsøe and Sumer (1997) studied wave-dominated scour near the tip of a rubble-mound breakwater in laboratory conditions. The basic scouring mechanisms were found to be:

- non-breaking waves; wave-induced steady streaming near the bed due to non-uniformity of the wave boundary layer and contraction of flow upstream and around the tip of the breakwater (see Fig. 6.2);
- breaking waves; relatively high waves (H_s/h= 0.5 to 1 depending on bottom slope of foreshore) arriving near the toe of the breakwater may break locally by plunging on the sloping part of the tip; a three-dimensional jet is generated, attacking the sand bed in the lee of the sloping breakwater tip resulting in lee-side scour at the junction between the tip and the trunk section, see Fig. 6.2.

Scour tests over a movable bed of 0.19 mm-sand were conducted in a depth of 0.4 m with irregular (non) breaking waves (periods between 2 and 6 s). The relative wave heights were in the range H_s/h= 0.4 to 0.5. The slope of the breakwater was 1 to 1.5. The bottom width of the breakwaer was about 2.25 m at the sand bed level (width-depth ratio of B/h=5.6).

The observed maximum scour depth ($d_{s,max}/B$) for normal incident **non-breaking** waves (90^o) was found to be related to the KC-number, see Table 6.2. The maximum scour depth was attained after 20,000 waves. The scour was maximum at a short distance upwave of the tip of the breakwater. The observed scour length L_s was about L_s/B=1 normal to the structure and about 1.5 parallel to the structure, see Fig. 6.2.

The observed maximum scour depth ($d_{s,max}/H_s$) for normal incident **breaking** waves (90^o) was found to be related to the parameter $T_p(gH_s)^{0.5}/h$, see Table 6.2. The maximum scour depth was attained after 20,000 waves. The scour was maximum in the lee-side zone of the tip of the breakwater. The observed scour length L_s was about L_s/H_s=2 to 3 normal to the structure and about 5 to 10 parallel to the structure, see Fig. 6.2.

Based on the laboratory data of Fredsøe and Sumer, the maximum scour depth for breaking wave conditions is:

$$d_{s,max}/h_{toe}=0.25 \text{ to } 0.5 \quad \text{for } H_s/h=0.5\text{-}1.0 \qquad (6.1)$$

The scour depth decreased by factor 2 when the slope of the structure was decreased from 45° to 30°.
Scour can be eliminated by means of a protection layer. The length of the protection layer should be about L/B=0.5 (normal to structure) for KC=0.4 and L/B=1 for KC=1. In that case the maximum scour depth is reduced by a factor 3. In case of L/B=0.3, the maximum scour depth is reduced by a factor 2.

NON-BREAKING WAVES		PLUNGING BREAKING WAVES	
SCOUR DEPTH $d_{s,max}/B$	KC-number	SCOUR DEPTH $d_{s,max}/H_S$	$T_p(gH_s)^{0.5}/h$
0.01	0.1	0.1	4
0.02	0.2	0.2	8
0.04	0.5	0.5	14

Table 6.2 *Scour depth for normal incident (non) breaking irregular waves over a sand bed of 0.19 mm in a laboratory flume (Fredsøe and Sumer, 1997)*

Fredsøe and Sumer (1997) also present some scour depth values of rubble-mound breakwaters in field conditions in the USA (based on data of **Lillycrop and Hughes, 1993**), see Table 6.3.
Based on this dataset, the maximum scour depth is about:

$$d_{s,max}/h=0.4\text{-}0.5 \qquad \text{for } H_s/h= 0.8\text{-}0.9 \qquad (6.2)$$

Field data of scour near the tip of rubble-mound breakwaters in Japan show:

$$d_{s,max}/h_{toe}= 0.3 \text{ to } 0.2 \quad \text{for depths between 10 and 20 m} \qquad (6.3)$$

Katayama et al. (1974) present information of scour near the tip of an offshore breakwater on the Niigata coast in Japan. Scour depths between 2 and 4 m in water depth of about 4 m were observed (based on soundings made after the stormy season). Thus:

$$d_{s,max}/h_{toe}= 0.5 \text{ to } 1 \qquad \text{for depths smaller than 4 m} \qquad (6.4)$$

LOCATION	TYPE	DEPTH AT TOE h (m)	WAVE HEIGHT H_s (m)	PEAK PERIOD T_p (s)	MAX. SCOUR DEPTH $d_{s,max}$ (m)	
					front-side	lee-side
Morro Bay California	slope 1:2 base B=76 m	6	5.3	10-15	-	3
Cattaraugus Harbour New York	slope 1:2 base B=50 m	3	2.4	8.3	0.6	1.2

Table 6.3 *Maximum scour depth of sand and gravel bed near the tip of rubble-mound breakwaters due to wave motion (Fredsøe-Sumer, 1997)*

6.3 Current-dominated scour near tip of rubble-mound breakwaters and groynes

Scour near the tip of a breakwater or groyne (normal or slightly oblique to the bank or shore) is considerably enhanced, if wind-, wave- and tide-induced longshore currents with velocities exceeding 0.5 m/s are present.

The key scouring mechanisms are:
- flow contraction near tip increasing with the protrusion length of the groyne/breakwater (Fig. 6.3);
- large-scale vortices generated at the tip of the groyne/breakwater increasing the transport capacity of the flow.

The sediments are mobilized by the near-bed velocities and by the stirring action of the waves (if present) and carried away by the currents, but currents alone are also capable of mobilizing the sediments. Laboratory experiments for combined wave-current scour near coastal inlet structures have been performed by **Hughes and Kamphuis (1996)** and by **Sumer and Fredsøe (1997)**.

The latter give some values for scour depth along the tip of a round vertical wall breakwater:

$$\begin{aligned} &\mathbf{d_{s,max} = 0.2B} \quad &&\mathbf{for\ U_c/(U_c+U_m)=0.2\ and\ KC=2} \\ &\mathbf{d_{s,max} = 0.7B} \quad &&\mathbf{for\ U_c/(U_c+U_m)=0.2\ and\ KC=7} \end{aligned} \tag{6.5}$$

$$\begin{aligned} &\mathbf{d_{s,max} = 0.7B} \quad &&\mathbf{for\ U_c/(U_c+U_m)=0.4\ and\ KC=2} \\ &\mathbf{d_{s,max} = 1.5B} \quad &&\mathbf{for\ U_c/(U_c+U_m)=0.4\ and\ KC=7} \end{aligned} \tag{6.6}$$

with U_c= depth-averaged current velocity and U_m= peak orbital velocity near bed.

These values do not represent the equilibrium values as the laboratory tests were only done for a relatively short time period (sand bed layer was not thick enough). As the maximum width of the structure in the model tests was about equal to the water depth, the maximum scour depth can also be related to the water depth yielding values in the range **of** $d_{s,max}$ = 0.2 to 1.5 h for $U_c/(U_c+U_m)$=0.2 to 0.4 and KC=2 to 7. The equilibrium values may be 50% larger.

The flow near a round vertical wall (groyne) in a steady current is characterized by the curvature of the streamlines resulting in a spiral type motion like flow in a river bend. The maximum velocity occurs near the tip of the groyne. The length L_1 over which the flow field is disturbed in the contracted cross-section is approximately equal to the length of the groyne ($L_1 \approx L$), if the total river width is larger than twice the groyne length (Figure 6.3).

Based on analysis of field data for unidirectional flow in rivers, the following scour depth expression has been proposed **(Hoffmans and Verheij, 1997)**:

$$\mathbf{d_{s,max} = \alpha\ [q_o/(1\text{-}m)]^{2/3} - h_1} \tag{6.7a}$$

with:

$d_{s,max}$	=	maximum scour depth near head of structure,
h_1	=	mean water depth of contracted section before scour,
q_o	=	discharge per unit width upstream of contracted section (in m^2/s),
m	=	L/B= blocking coefficient,
B	=	channel width,
α	=	coefficient depending on geometry (≈1 to 2 for straight channel and groyne normal to bank).

Lacey (1930) proposed a formula for the prediction of the maximum scour depth around abutment-type structures, as follows (see Rahman and Haque, 2003):

$$\mathbf{d_{s,max} = 0.47h_1K\,[Q/(f\,h_1^3)]^{1/3} - h_1} \qquad (6.7b)$$

with:

$d_{s,max}$ = maximum scour depth near head of structure,
h_1 = mean water depth of contracted section before scour,
Q = regime discharge (in m^3/s),
f = $56(d_{50})^{0.5}$= sediment factor,
d_{50} = sediment diameter (in m),
K = coefficient depending on geometry (≈2 for rounded head to 4 for steep sloping head).

Rahman and Haque (2003) taking the structure length into account, modified Equation (6.7b) into:

$$\mathbf{d_{s,max} = 0.47h_1\,M^{1/3}[1+1.5L/h_1]^{1/3} - h_1} \qquad (6.7c)$$

with:

$d_{s,max}$ = maximum scour depth near head of structure,
h_1 = mean water depth of contracted section before scour,
M = $Q/(fh_1^3)$= discharge coefficient,
f = $56(d_{50})^{0.5}$= sediment factor,
d_{50} = sediment diameter (in m).

Rahman and Haque (2003) also presented field data of scour depths near abutment-type structures along the Jamuna river in Bangladesh. The relative scour depth values ($d_{s,max}/h_1$) are in the range of 0.5 to 2 for a length scale of about L/h_1=7 to 12 and about 1 for L/h_1=40. This latter value is significantly overpredicted by Equations (6.7b and 6.7c).

Coleman et al. (2003) proposed for vertical wall bridge abutments of varying lengths the following expression:

$$\mathbf{d_{s,max} = K_{yL}\,K_d\,K_s\,K_\theta\,K_G\,K_I} \qquad (6.7d)$$

with:
U= depth-averaged approach velocity;
U_{cr}= critical depth-averaged velocity; h=approach water depth;
K_{yL}= factor related to abutment size=10h for h/L=0.04,
$K_{yL}=2(h_1L)^{0.5}$ for $0.04\leq h_1/L\leq 1$, $K_{yL}=2L$ for $h_1/L>1$;
K_d = sediment size factor= 1 for $L/d_{50}>25$,
K_s = foundation type factor=1 for vertical wall;
K_θ = alignment factor=1 for 90 degrees (normal to bank),
K_θ = 0.95 for 45 degrees,
K_θ = 1.1 for 150 degrees;
K_G = river channel factor=1 for rectangular channels;
K_I= flow intensity factor=U/U_{cr}; K_I=1 for $U/U_{cr}>1$.

Another method is to assume that the cross-sectional area of the contracted section ultimately will be equal to that without the groyne (see Figure 6.3). This means that the scoured area (A_s) will be equal to the area blocked by the groyne. Thus: $A_s = h_1L$.

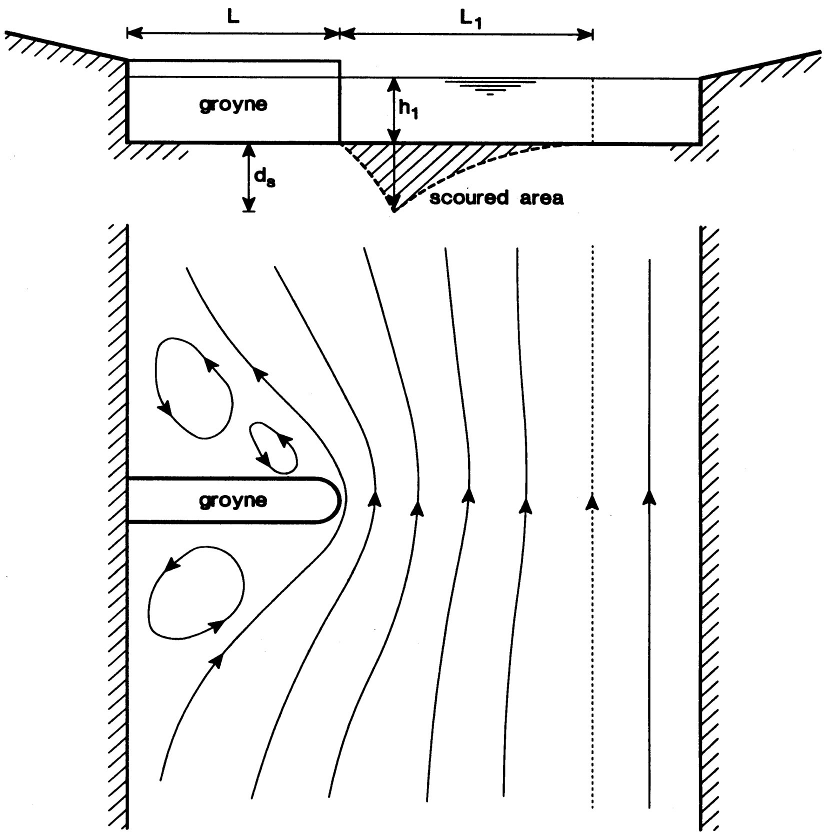

Figure 6.3 *Flow pattern and scour near a groyne*

Assuming that A_s=1/3($d_{s,max}L_1$) for a long groyne (L>10 h_1) and L_1=L, it follows that:

$$d_{s,max}/h_1 = 3 \qquad \text{for } L>10\ h_1 \tag{6.8}$$

This is in good agreement with values observed by **Richardson et al. (1988)**, who found for rock dikes (with L/h_1>25) in the Mississippi river: $d_{s,max}/h_1 \leq 4$.
The expression $d_{s,max}/h_1=3$ is valid for a relatively long groyne ($L/h_1 \geq 10$) resulting in a significant increase of the velocities in the contracted section. The channel bed is assumed to be composed of sandy material and the approach velocity is assumed to be larger than the critical velocity for initiation of motion (U/U_{cr}>1). Armouring which may occur in course bed material, will result in reduced scour depths.
The scour depth near a short groyne will be considerably smaller. The maximum scour depth is:

$$d_{s,max}/h_1 = 0.5 \text{ to } 1.5 \qquad \text{for } L = 1 \text{ to } 3\ h_1 \tag{6.9}$$

The shape of the groyne will also affect the scour depth. Scour is maximum near a vertical wall (rectangular cross-section). The scour depth may be reduced with about 30% in case of a rock-type groyne with a trapezoïdal cross-section or with a rounded tip.

Kothyari and Ranga Raju (2001) discuss the scour around spur dikes and bridge abutments in alluvial rivers. The horse-shoe vortex and associated downflow are found to be the prime agents causing scour similar to scour around bridge piers (see Figures 7.1 and 6.1). They defined an analogous circular pier which has such a size that the scour around it is the same as that around the given abutment or spur dike under similar hydraulic conditions.

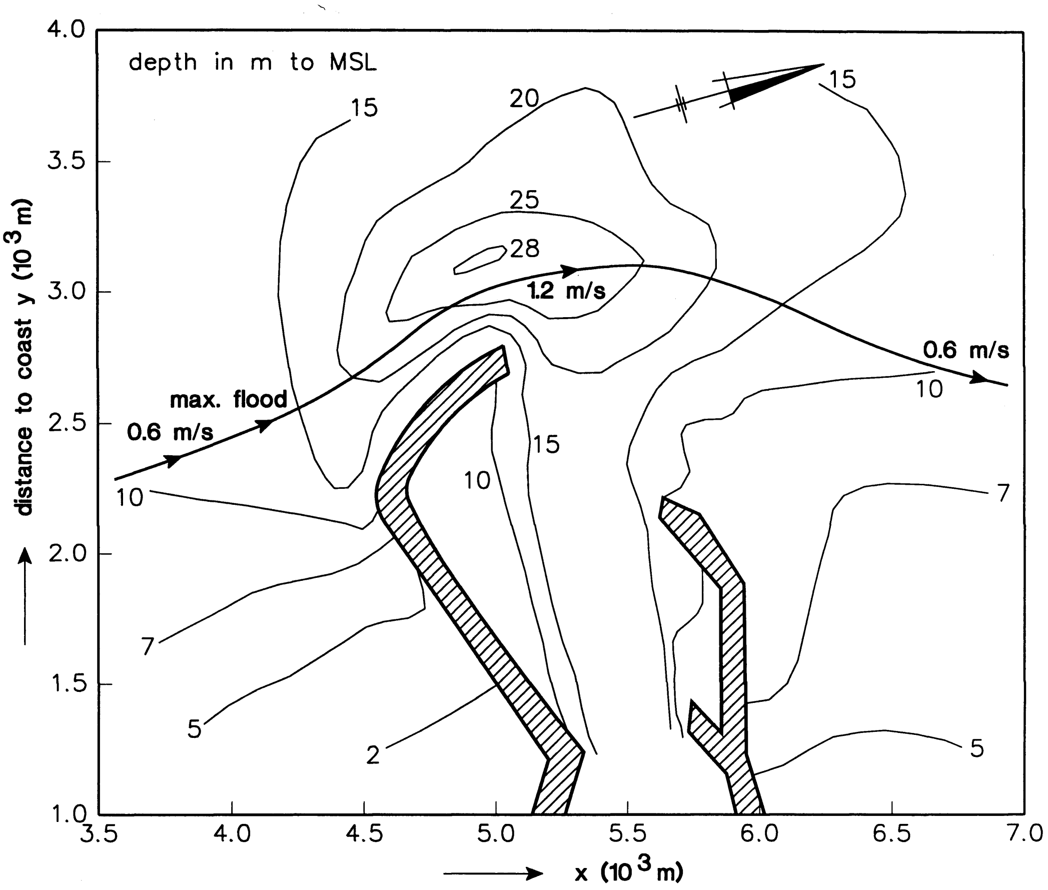

Figure 6.4 *Scour (in 1986) near tip of breakwater of IJmuiden on meso-tidal North Sea coast, The Netherlands (Delft Hydraulics, 1988)*

Delft Hydraulics (1988) reported about a large scour hole, which was observed near the tip of the breakwater of IJmuiden harbour (approach to Port of Amsterdam), The Netherlands. The breakwater is built normal to the shore over about 2 km; the end section is situated at an angle of 60^o to the shoreline over about 0.5 km. The bed consists of sand with d_{50} of 0.2 to 0.3 mm. The tide is meso-tidal; the maximum tidal current velocity in the original undisturbed situation was about 0.6 to 0.7 m/s during flood, which increased to about 1.2 m/s after construction of the breakwaters. The wind waves are oblique to the breakwater; swell is not of significant importance.
The maximum scour depth near the tip of the breakwater was found to be about 15 m below the original sea bed; the original water depth below MSL was about 15 m, see Figure 6.4.
Thus, the maximum scour depth is as large as the original water depth at the toe of the structure:

$$\mathbf{d_{s,max}/h_{toe}= 1} \qquad \textbf{for original depth of 15 m} \qquad (6.10)$$

Rijkswaterstaat (1996) reported about a deep scour hole near a long groyne (Eierland dam; length of 800 m), which was built (in May-July 1995) normal to the North Sea coast of one of the West Frisian barrier islands of The Netherlands to protect the tip of the island against erosion by the tidal currents passing the inlet on the eastern side of the groyne, see Figure 6.5. The bed consists of sand with d_{50} of about 0.3 mm. The original water depth at the toe of the groyne was about 4 m below MSL. The maximum current velocities (during flood) in the original situation were about 0.7 m/s, which increased to 1.2 m/s after construction of the dam based on flow computations. The sand bed near the tip of the groyne was scoured away to a depth of 13 m below the original bed in a period of 9 months. Figure 6.5 shows a plan view of the scour hole after 9 months with respect to June 1, 1995 and Figure 6.5 also shows cross-sections in the axis of the groyne at various times. The maximum scour depth is 13 m below the original bed (4 m below MSL). The width of the deepest section is about 150 m. The steepest slope close to the toe is about 1 to 1.5 and is protected by layers of stones. The maximum slope below -10 m is 1 to 2.5.

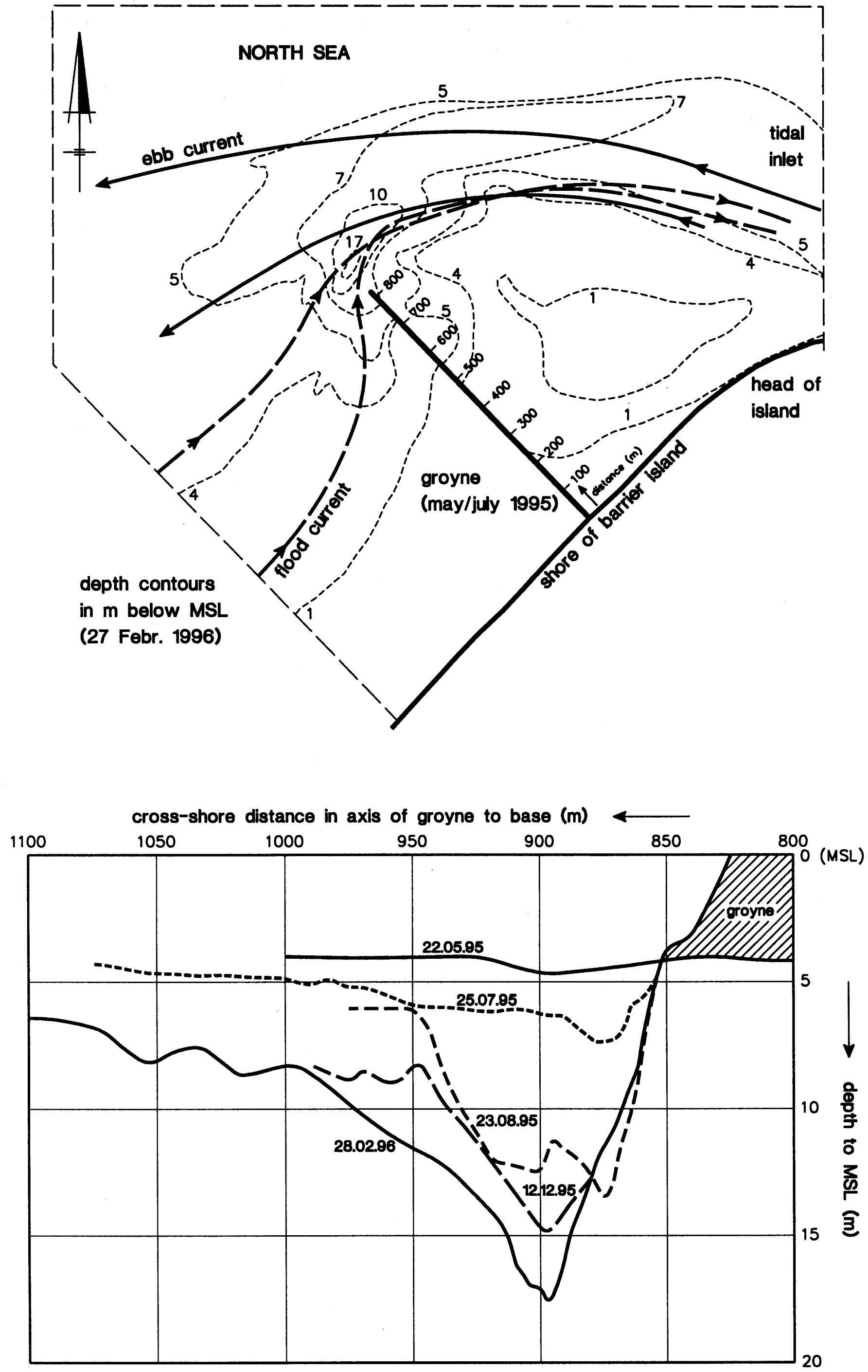

Figure 6.5 *Scour near tip of Eierland groyne on meso-tidal North Sea coast of barrier island, The Netherlands*
Top: Plan view
Bottom: Cross-sections in axis of groyne

Thus, the ratio of the scour depth and the original water depth at the toe is:

$$d_{s,max}/h_{toe} = 3 \qquad \textbf{for original depth of 4 m} \qquad (6.11)$$

Based on all available data, the ratio of the scour depth and original water depth (below MSL) at the toe roughly varies, as follows:

$$\begin{aligned} d_{s,max}/h_{toe} &= 4 \text{ to } 2 && \textbf{for depths < 4 m,} \\ d_{s,max}/h_{toe} &= 2 \text{ to } 1 && \textbf{for depths = 4-10 m,} \\ d_{s,max}/h_{toe} &= 1 \text{ to } 0.5 && \textbf{for depths > 10 m.} \end{aligned} \qquad (6.12)$$

In current-dominated conditions the scour area can have large-scale dimensions. The slopes of the scour holes near the structure may be quite steep locally, which may lead to soil sliding due to grain-shear failure and liquefaction endangering the foundation of the structure. This should be prevented by construction of relatively large and flexible bottom protections (dumping of stone layers) over a length (normal to the structure) of 2 to 3 times the undisturbed water depth (L= 2 to 3h). Regular monitoring should be performed (after storms). Liquefaction can easily occur in loosely-packed sand layers (bore hole information and penetration resistance).

7 Scour near vertical pipes, piles and piers

Generally, a distinction is being made between clear water scour and mobile-bed scour. The former is related to conditions with no upstream sediment transport ($U<U_{cr}$ with U=depth-averaged velocity); the latter is related to conditions with sediment transport ($U>U_{cr}$).
Literature reviews have been given by **Breusers et al. (1977), Melville (1988), Melville-Sutherland (1988), Kothyari et al. (1992), Melville (1997)**, **Lim (1997)** and **Melville and Coleman (2000)**.

7.1 Current-related scour near vertical pipes and piles

The scouring process around vertical piles (bridge piers) is dominated by the following effects:
- local disturbance of the flow field (local scour);
- local reduction of cross-section (constriction of the flow due to the presence of the structure; contraction scour); $h_1= b_o h_o/b_1$ with h_1= mean depth of cross-section in contraction zone, b_1=effective flow width of cross-section in contraction zone, b_o= upstream flow width, h_o= upstream mean flow depth).

Other general scour effects which can be important are:
- general degradation effects (downstream of weirs, reservoir dams, etc);
- bend scour; deeper part of cross-section in outer bend area (variability in river planform); depth in bend may be 2 to 3 times larger than the mean depth of the cross-section;
- confluence scour; deeper parts of cross-section downstream of confluence;
- thalweg variations (deepest point of coss-section may shift in lateral direction);
- bed-form variations.

Coleman and Melville (2001) propose to determine the total scour depth near the foundation of a bridge pier on the basis of superposition of general scour and local scour at the foundation. They discuss the failure of bridges in New Zealand due to excessive scour at the piers. The Bulls Road bridge failure in 1973 during an annual flood event with a discharge of 675 m^3/s (not an extreme event; maximum recorded value is 3800 m^3/s) can be attributed to a combination of general scour arising from gravel mining and local pier scour. The local scour was enhanced by: (i) the obliqueness of the flow to the pier, (ii) the flow constriction caused by the piling up of debris behind old timber piers immediately downstream of the bridge and (iii) the presence of fine sand substrata exposed during the scouring process and accelerating the scouring process. The maximum depth of scour measured below the armoured bed level adjacent to the collapsed pier was about 12 m.

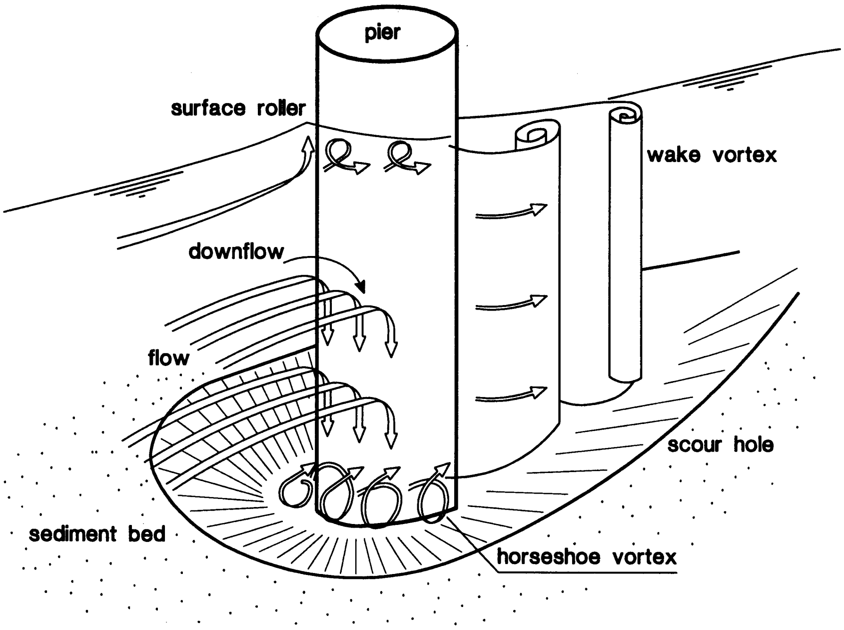

Figure 7.1 *Flow pattern and scour near pipe (Melville, 1988)*

The flow pattern around a cylindrical pipe is characterized by (see Figure 7.1):
- water surface roller in front of pipe;
- downflow in front of pipe;
- vortex-shedding in separation zone;
- wake flow downstream of pipe;
- generation of horseshoe-vortices in scourhole.

Based on analysis of field and flume data, **Breusers et al. (1977)** have found for a single pipe in uniform bed material:

$$\mathbf{d_{s,max}/D = \alpha_1\, \alpha_2\, \alpha_3\, \alpha_4} \qquad (7.1)$$

with:

$d_{s,max}$	=	maximum scour depth below original bed,	
D	=	width of pipe or pile cap (connecting several piles) normal to flow; D=diameter for circular pipe,	
α_1	=	coefficient related to U/U_{cr},	
α_2	=	coefficient related to h/D,	
α_3	=	coefficient related to shape of pipe,	
α_4	=	coefficient related to angle of attacking flow,	
U	=	depth-averaged flow velocity upstream of pipe,	
U_{cr}	=	critical depth-averaged flow velocity (upstream),	
h	=	flow depth (upstream),	
α_1	=	0	for $U/U_{cr}< 0.5$ (no upstream transport),
α_1	=	2(U/Ucr - 0.5)	for $U/U_{cr}= 0.5$ to 1.0 (no upstream transport),
α_1	=	1	for $U/U_{cr}\geq 1$,
α_2	=	2 tanh(h/D)	yielding $\alpha_2= 2$ for $h/D \geq 3$,
α_2	=	1.5	for $h/D < 1$,

α_3	=	1	for circular pipes,
α_3	=	0.75	for streamlined pipes,
α_3	=	1.3	for rectangular pipes,
α_4	=	1	for flow normal to pipe,
α_4	=	1.3	for flow under angle of 15^O and length-width ratio of 4,
α_4	=	2	for flow under angle of 15^o and length-width ratio of 8.

Often the piers of a bridge are connected by a pile cap under water (just above bed level). In that case the width of the pile cap should be taken to estimate the D-parameter. During flood events with relatively large water depths and oblique approaching flow (worst case scenario), the maximum scour will be of the order of $d_{s,max}$ = 4 to 5 D. If a pile cap (say width of 1.5 m) is present, the maximum local scour close to the pile cap can easily go up to values of 5 to 7 m. The piling up of debris at the bridge during flood events should be explicitly taken into account!

When bed forms are present, an extra foundation depth equal to 0.5 times the maximum dune height to be expected, should be taken into account.

The length of the scour hole is about 1D (D = diameter of pipe) upstream of the pipe and about 5D downstream of the pipe. The width of the scour hole is about 2D on each side of the pipe.
The time scale of the scouring process (time at which $d_{s,max}$= D) depends primarily on the approach velocity, the sediment size and the width of the pipe.

A group of pipes yields a larger scour depth (factor 1.5 to 2) when the pipes are spaced closely (spacing<5 to 10D). As the spacings between the piles decrease, a point is reached at which a cluster of piles would act as a single pile with a greater effective diameter.

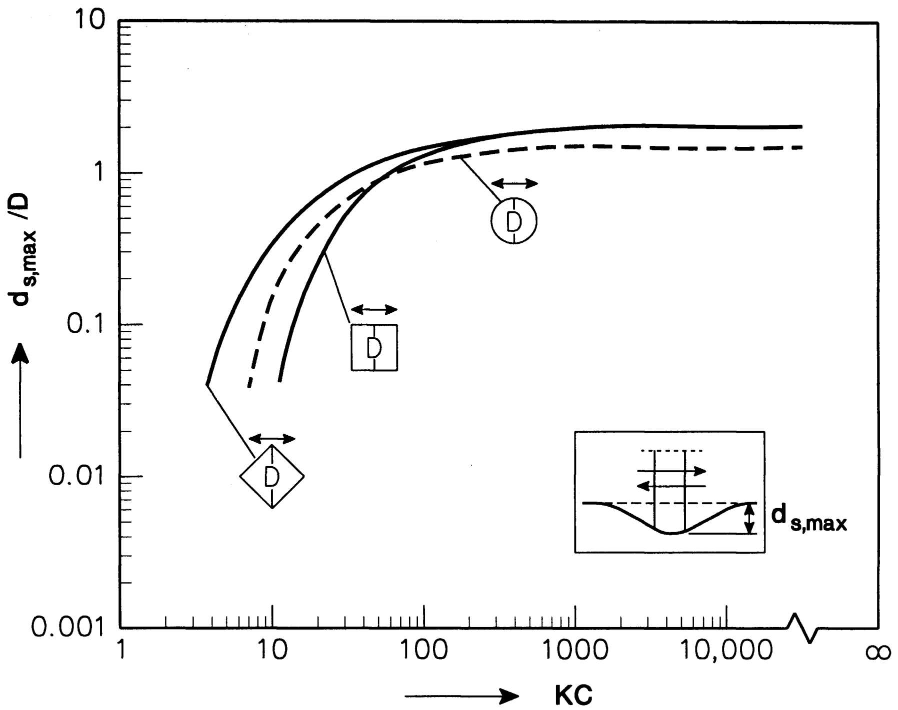

Figure 7.2 *Wave-related scour near vertical piles (Sumer et al., 1993)*

7.2 Wave-related scour near vertical pipes and piles

The near-bed flow around the pile generates horseshoe vortices generated at the upstream side of the pile and at the lee-side of the pile. The horseshoe vortices are insignificant if the wave boundary layer is thin (KC<10).
Based on experimental data for regular waves, **Sumer et al. (1992)** have found for small circular piles with diameter D (see also Fig. 7.2):

$$
\begin{aligned}
&\mathbf{d_{s,max}/D= 0.01} && \textbf{for KC<5} \\
&\mathbf{d_{s,max}/D= 0.1} && \textbf{for KC=10} \\
&\mathbf{d_{s,max}/D= 0.5} && \textbf{for KC=20} \\
&\mathbf{d_{s,max}/D= 1.0} && \textbf{for KC=100} \\
&\mathbf{d_{s,max}/D= 1.3} && \textbf{for KC=1000}
\end{aligned} \qquad (7.2)
$$

The length of the scour hole with respect to the pile axis roughly is: L/D= 5 to 10.

Sumer et al. (1993) have tested piles with a square cross-section placed at different angles to the incident waves. The results are given in Figure 7.2.

Hotta and Mauri (1976) studied scour depths of piles in the surf zone of Ajigaura beach, Japan. The maximum scour depth was found to be $d_{s,max}/D$= 1 to 1.5 and the maximum scour length with respect to the pipe axis was L/D= 7 to 10.

Sumer et al. (2001) state that wave-scour results from Figure 7.2 are also valid in shallow depth with non-breaking waves on a sloping profile (1 to 20). A pile landward of the breakerline is strongly affected by the position of the breaker bar. Scour depth will be relatively large in the trough zone of the bar.

Sumer and Fredsøe (2001) studied the scour near large circular cylinders under regular waves. The water depth was about 0.4 m. The cylinder diameters were D=0.54, 1.0 and 1.53 m. Rigid-bed and movable-bed experiments were performed. Detailed velocity measurements were carried out to determine the local flow field around the cylinder. The movable-bed experiments (0.2 mm sand) were done to determine the maximum scour depth. Based on the velocity measurements, it is concluded that wave stirring in combination with wave-induced streaming are responsible for the scouring process. When a large vertical cylinder is subjected to a progressive wave, a complicated wave field is generated consisting of the incident waves, reflected waves and diffracted waves. A near-bed 3D steady streaming occurs in the vicinity of the cylinder. The streaming is directed toward (in wave direction) the cylinder in the region in front of the cylinder; the streaming is outward and opposite (to the wave direction) in the region adjacent to the cylinder. The maximum streaming is about 25% of the peak orbital velocity (undisturbed) near the bed. The scour depth increases with increasing KC-number and increasing D/L_w value. The maximum scour depth is about 0.05 D for a KC-number of about 1 and D/L_w of about 0.15 with L_w= wave length.

Sumer and Fredsøe (1998) studied the wave-induced scour around a group of vertical piles. Various configurations were tested. The water depth was 0.4 m above a sand bed (0.2 mm). The diameters of the single piles were D=32 to 90 mm. Their conclusions are:

- the smaller the pile spacing, the larger the interference between the piles; the pile group behaves as a single body for very small spacings G/D<0.1, with G= gap size between piles, D= pile diameter; the interference disappears for G/D>1 to 3, depending on pile arrangement;
- two-pile group: the scour depth increases by a factor of 3 for the side-by-side arrangement (G/D=0.4 and KC=13); the scour depth decreases by a factor of 2 for the in-line (tandem) arrangement (G/D=0.4 and KC=13); the angle of attack has a substantial effect on scour depth;
- three-pile group: the scour depth increases by about 30% for the side-by-side arrangement compared with the two-pile side-by-side arrangement; the scour depth for the in-line arrangement is the same as that for a two-pile group;

- four-pile square group; the scour depth decreases by a factor of 3 for KC=13 compared with the scour around a single pile; the scour depth increases by a factor of 3 for KC=37 compared with the scour around a single pile;
- given the pile spacing (G/D), the scour depth is governed by the KC-number; the larger the KC-number, the larger the scour depth.

7.3 Wave and current-related scour near vertical pipes and piles

De Bruyn (1988) studied the scour process near a pipe in current and wave conditions. The bed material was sand with d_{50} = 0.2 mm. The water depth (laboratory) was 0.3 m. The depth-averaged velocity upstream of the pipe was 0.4 m/s (mobile bed, $U/U_{cr} > 1$). The maximum scour depth was found to be:

$$\mathbf{d_{s,max}/D = \alpha} \tag{7.3}$$

with:
α = 1.3 for a current alone,
α = 1 for current and non-breaking waves,
α = 1.9 for current and breaking waves.

The length of the scour hole was 3D upstream and 5D downstream of the pipe for a current alone. For combined current and waves the scour length upstream was 4D and 6D downstream of the pipe.
Eadie and Herbich (1986) found α=1.2 for a current alone and α= 1.4 for irregular non-breaking waves plus current with H_s/h= 0.15 and U= 0.15 m/s over a fine sand bed.
Rance (1980) studied scour near large-diameter piles with $D>0.1L_w$ (L_w=wave length) by waves and currents and found α=0.04 to 0.07 for circular and hexagonal piles and α=0.13 to 0.2 for square piles. The scour length was about 1D.
Sumer and Fredsøe (2001) studied the scour around a vertical pile in a sand bed (0.16 mm) with irregular non-breaking waves in combination with a current (U_c). The water depth was 0.4 m. The depth-averaged current velocities (U_c) were varied in the range between 0.1 and 0.5 m/s. The diameters of the single piles were D=30 to 90 mm. They showed that the empirical expressions relating the scour depth to the KC-number in the case of regular-waves alone can also be used for the case of irregular waves alone, provided that the KC-number is computed as $KC=U_m/(D\ f_p)$ with $U_m=1.41\sigma_u$= peak value of near-bed orbital velocity , f_p=peak wave frequency ($1/T_p$), σ_u=root-mean-square value of the near-bed orbital velocity. The maximum scour depth in conditions with a current alone was in the range between $d_{s,max}/D$= 1.2 to 2. The observed maximum scour depths in relation to the KC-number and velocity ratio are given in Table 7.1.

	$U_c/(U_c+U_m)$=0 waves alone	**$U_c/(U_c+U_m)$=0.3**	**$U_c/(U_c+U_m)$=0.5**	**$U_c/(U_c+U_m)$=0.7**	**$U_c/(U_c+U_m)$=1.0 current alone**
KC=26	$d_{s,max}/D$=0.8	$d_{s,max}/D$=1.3	$d_{s,max}/D$=1.5	$d_{s,max}/D$=1.6	$d_{s,max}/D$=1.2 to 2.0
KC=8	$d_{s,max}/D$=0.1	$d_{s,max}/D$=0.3	$d_{s,max}/D$=0.9	$d_{s,max}/D$=1.3	$d_{s,max}/D$=1.2 to 2.0
KC=4	$d_{s,max}/D$=0.06	$d_{s,max}/D$=0.1	$d_{s,max}/D$=0.6	$d_{s,max}/D$=1.0	$d_{s,max}/D$=1.2 to 2.0

Table 7.1 *Scour depth in combined wave-current conditions*

The data values show that for small KC-numbers a slight increase of the depth-averaged current velocity (U_c) results in a significant increase of the scour depth. The scour depth approaches its steady-current value for a velocity ratio larger than about 0.7. The scour depth is practically independent of the angle between the wave and current direction; the scour depth was about the same for an angle of 0 and 90 degrees.
Usually, the bed near a pipe has to be protected by a layer of stones (rip-rap) on a filter layer or matt to prevent erosion of fine sediments through the protection layer of stones. The protection layer should be placed below the lowest bed level to prevent the creation of extra obstruction. The design velocity should be taken 2 times the average approach velocity to account for the local increase of the velocity near the pipe. Model tests are recommended for complicated situations.

8 Scour near horizontal pipes

8.1 Current-related scour

Scour near and under a pipeline is caused by changes of the local flow field due to the presence of the pipeline, see Fig. 8.1 and Fig. 8.2. Where there is a local increase in the transport capacity, erosion will take place. Sedimentation will take place where the transport capacity decreases. Usually, the velocity under the pipe will increase when there is a small local gap between the pipe and the sea bed. This will initiate and intensify the erosion process.
Experiments have shown that erosion will always take place if a pipeline is placed on an erodible seabed, and when there is transport of sediment upstream of the pipeline. The processes causing onset of scour will be briefly described hereafter.
The mechanisms can be divided into three groups:

- *flow induced pressure differences*
  In the case with flow perpendicular to the pipeline axis, there is a pressure difference between the upstream and the downstream part of the pipeline. This difference ΔP is normally written as $\Delta P = \rho\, C_p\, (U^2/2g)$ with U= the undisturbed near-bed velocity; the pressure coefficient is approximately $C_p=1$ in steady current and $C_p=3$ for waves; due to these pressure differences, ground water flow can take place and the sediment may be carried away.
- *vortices near the pipeline*
  Three types of vortices are observed near the pipeline, see Figure 8.1. The vortices can transport the sediment away; suspended as well as bed transport can occur. Vortex A and vortex C move the sand particles away from the pipe area, while vortex B moves the sand particles toward the pipe.
- *imperfections in the seabed near the pipeline*
  Variations/imperfections of the bed near the pipeline or of the pipeline itself may result in the presence of gaps between the pipeline and the bed and hence to flow under the pipeline.

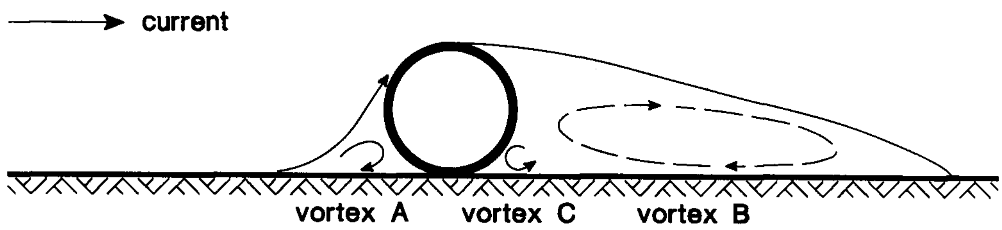

Figure 8.1 *Vortices near the pipeline in unidirectional flow*

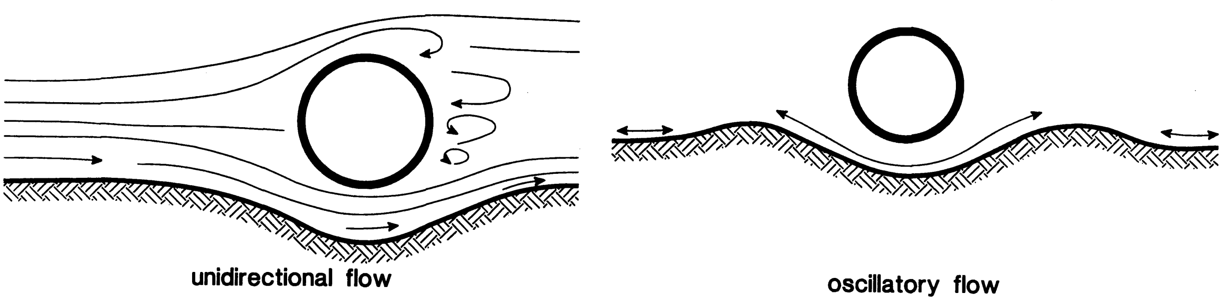

Figure 8.2 *Scour in unidirectional and oscillatory flow*

The development of scour in a current is governed by the velocity below the pipeline, the downstream wake and the vortex shedding downstream of the pipeline. From experimental results it appears that the near-bed velocity below the pipeline decreases with the depth of scour and increases with the width of scour.
If a pipeline is placed on a plane seabed in a current, a downstream wake will be developed. The length of this downstream wake is approximately six times the pipeline diameter.
If the pipeline is partly buried, the length of the downstream wake decreases. An analogy can be made with flow past a step.

In cases with a small gap below the pipeline (less than 0.3 times the diameter) no vortex shedding occurs.
If the current approaches the pipeline axis at a certain angle, the length of the downstream wake decreases. For flow angles below 30° no vortex shedding occurs.

Kjeldsen et al. (1973) performed flume experiments with pipelines resting on the bed. Based on dimensional analysis, they found (U = mean flow velocity, $d_{s,max}$= scour depth below bottom of pipe, D = diameter of the pipe, h = water depth, d_{50} = mean grain diameter, g = gravity) that the maximum scour depth can be expressed as:

$$\mathbf{d_{s,max} = 0.97\ D^{0.8}\ (U^2/2g)^{0.2}} \tag{8.1}$$

This formula should not be applied for conditions which do not represent the test conditions. For example, the formula can give erroneous results in the clear water case, i.e. where no sediment transportation takes place far from the pipeline.

All measurements of the scour development under a fixed pipeline in a current perpendicular to the pipe axis show that the maximum scour depth is obtained when the pipe is placed on the original seabed and the maximum scour depth (below the bottom of the pipe) is approximately one diameter.

Even, if there is no moving sediment upstream the pipeline (i.e. clear water case with small Shields parameters) scour may take place under the pipeline.

The bed-shear stress increases with the near-bed velocity. So even, if the far-field Shields parameter is less than the critical value of 0.05, the value near the pipe can be larger than 0.05, and erosion will take place.

The maximum scour depth will be highly dependent on the far-field Shields parameter. The maximum scour depth in the clear water case (no upstream sediment transport) is always observed to be smaller than that in case with active sediment transport.

The scour profile in unidirectional currents is characterized by a steep upstream slope and a more gentle downstream slope. In tidal flow the scour profile is symmetrical.

8.2 Wave-related scour near horizontal pipes

In flow with small Keulegan-Carpenter KC numbers, (KC is defined as KC = U_m T/D where U_m is the amplitude of the near-bed orbital velocity, T = wave period, and D = diameter of the pipeline), the downstream wake will not be fully developed. For low KC values (KC < 6) no downstream wake will be developed and the flow field can be described by potential theory. This theory predicts relatively high velocities below the pipeline.

In comparison with the development of scour in stationary flow, different mechanisms are present in the wave-induced scour processes. The time scale for the scour development and the maximum scour depth can change significantly. For example, scour depths of two times the pipe diameter are observed in the case of waves alone, while scour depths are less than approximately one pipe diameter in the case of currents alone.

In unidirectional flow the scour hole is formed by the combined effect of upstream erosion due to increasing velocities under the pipeline and downstream erosion due to turbulent velocities in the wake zone behind the pipe. The downstream erosion zone is wider and has a more gentle slope than the upstream erosion zone.

In oscillatory flow the upstream and downstream effects are reversed every half cycle of the wave motion, yielding a larger erosion zone.
This explanation is valid when the wave motion is sufficiently long, so that lee-wake induced erosion can be effective in each half period of the wave motion. This will be the case for wave motion with a large Keulegan-Carpenter number (KC > 300 with $KC=U_mT/D$, where U_m=maximum undisturbed near-bed orbital velocity).

Sumer and Fredsøe (1990) have given a simple empirical formula that expresses the maximum scour depth $d_{s,max}$ under the pipe with diameter D:

$$\mathbf{d_{s,max}/D = 0.1\ (KC)^{0.5}} \tag{8.2a}$$

The pipe is assumed to rest on the bed and to remain in that position (no vertical lowering of pipe). The stage at which the scour breaks out is the onset of the scouring process. The onset of scour is primarily caused by piping (groundwater flow). The scour depth was found to be sensitive for the presence of a gap (height e) between the bottom of the pipe and the bed surface. This gap is often related to the presence or development of free spans. The e-parameter was varied. Scour did also occur for embedded pipes (negative e-values). The scour depth was maximum for e between e=0 (pipe resting on bed) and e=-0.5 D (pipe buried over half its diameter). If the pipe is partly buried, the scour depth is given with respect to the bottom of the pipe. The scour depth decreases for increasing (positive) values of e/D, because the pipe is further away from the bed. The scour depth was found to be zero ($d_{s,max}/D= 0$) for e/D= 1 at KC< 10; for e/D= 1-3 at KC= 10-30 and for e/D= 3-5 at KC= 30-1000.
The roughness of the pipe was not found to have a significant effect on the scouring process.
The length of the scour hole (centerline to end of scour hole) can be estimated from (**Sumer and Fredsøe, 2002)**:

$$\mathbf{L_{s,max}/D = 0.35\ (KC)^{0.65}} \tag{8.2b}$$

According to **Myrhaug and Rue (2003),** the scour characteristics of horizontal piles in random waves should be based on $H_{1/10}$ rather than on H_{rms} or $H_{1/3}$.

Cevik and Yüksel (1999) studied the scour under horizontal pipelines at a sloping bed (1 to 5 and 1 to 10). The pipeline was parallel to the shoreline. The scour depth on a sloping bottom is found to be about two to three times larger than that on a horizontal bottom for the same incident wave conditions.

Sumer et al. (2001) studied the onset of scour in steady currents and in regular waves and the self-burial of pipelines. The water depths were about 0.3 m. The bed consisted of sand with d_{50} of 0.18 mm and 1.25 mm. The pipe diameters were D=10 and 5 cm. The onset of scour is defined as the stage when the bed is washed away underneath the pipe. This situation is basically related to the seepage flow in the sand beneath the pipeline, which is driven by pressure differences between the upstream (up wave) and downstream (downwave) sides of the pipeline.

Various modes of self-burial of the pipe may occur: (i) scour, sagging, backfilling and eventually self-burial of the pipeline between the span shoulders and (ii) sagging of the pipeline at the span shoulders due to general shear failure of the soil or failure of the soil supporting the pipeline due to liquefaction.
After the scour breaks out underneath the pipeline at certain locations, it will popagate along the length of the pipeline. A 3D-scour pattern will develop in which the scour holes are interrupted by stretches of soil (known as span shoulders, see Figure 8.3), where the pipeline obtains its support. As the process continues, the length of the free span will be larger and larger at the expense of the span shoulder. More and more weight of the pipe will be exerted on the soil over a shorter and shorter length of the span shoulder. The soil will fail when the bearing capacity of the soil is exceeded (general shear failure or liquefaction). As the sand at

the span shoulder fails progressively, the pipeline sinks into the sand and, at the same time, it sinks into the scour hole on both sides of the span shoulder. The scour process comes to an end when the pipeline reaches the bottom of the scour holes. At this moment the scour depth will be fairly close to that obtained for a fixed pipeline originally in contact with the bed. This scour depth is given by Eq. (8.2) for waves alone and by Eq. (8.1) for steady currents. Subsequently the space between the pipe and the scour hole is gradually backfilled with sand and the length of the span shoulder begins to increase due to backfilling process. When this process is completed, the pipeline is buried. The burial depth will be approximately equal to the scour depth.
A pipeline will be fully buried (e/D= -1) for KC larger than about 100. The self-burial depth may reach values as large as e/D= -3 for very large KC-numbers (say 1000), representing tidal flow. In the case of a steady current the self-burial depth will be about e/D= -0.7.

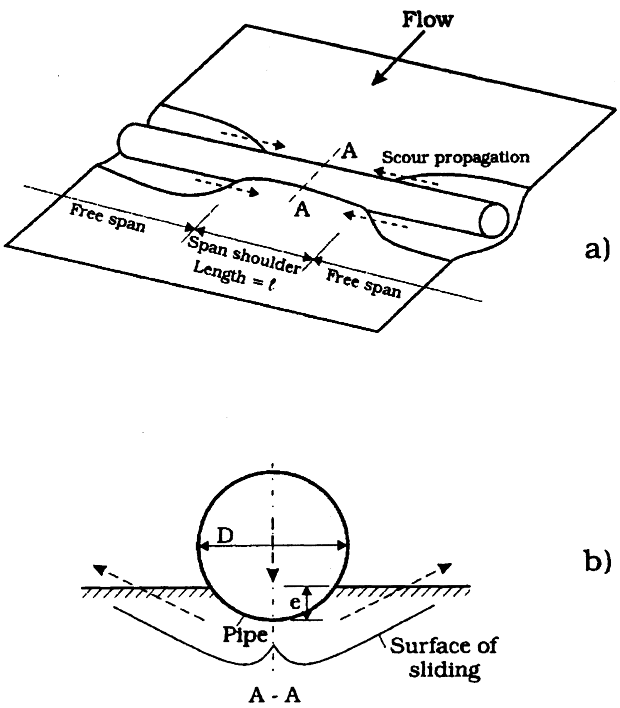

Figure 8.3 *Pipeline resting at span shoulder (scour along pipeline creating free spans)*

8.3 Wave and current-related scour near horizontal pipes

In combined wave and current motion it is advised to use the wave-related scour data, if the wave motion is dominant. The current-related scour data should be taken, if the current motion is dominant. The relative strength of both types of motions can be determined from the wave/current ratio $\alpha = U_m/U_c$ where U_c is the current velocity at ½ to 1 pipe diameter above the bed and U_m is the amplitude of the near-bed oscillatory velocity.

9 Scour near gravity structures

The construction of a large gravity structure (platform) on the seabed may lead to considerable near-field and far-field scour (dishpan scour). The plan shape of these massive structures may vary from circular to square and hexagonal with dimensions of the order of 100 m. Local scour may occur near corner points or near individual legs (if present), but there may also be a general degradation of the bed over distances equal to several times the horizontal dimension of the structure. Square-type structures suffer the greatest scour, particularly at the corners where vortices are formed by currents and waves. An estimate of the degradation can be obtained by assuming that the scoured area will be equal to the flow area blocked by the structure. This requires information of the area of flow contraction, which can be obtained from a mathematical model.

10 Summary of scour near seawalls and breakwaters

The maximum scour depth at the toe of a structure strongly depends on the relative wave height H_s/h in front of the structure. This latter value is related to the bottom slope of the foreshore. Reasonable values are H_s/h= 0.5 for breaking waves over a relatively flat bottom slope of 1 to 100 **(Nelson, 1983)** increasing to H_s/h= 1 for plunging breaking waves over a steep bottom slope of 1 to 30. These latter values were observed in the large-scale flume tests for St. George Harbour **(Delft Hydraulics, 1985)**. The field data for breakwaters in the USA also showed H_s/h-values close to 1.

The maximum scour depth near seawalls and breakwaters can be summarized by the following (conservative) values:

10.1 Wave-related scour near toe of seawalls and wall-type breakwaters

$d_{s,max}/h_{toe}$= 1.5 to 1	for depths < 2 m	(10.1)
$d_{s,max}/h_{toe}$= 1 to 0.7	for depths = 2 to 4 m	
$d_{s,max}/h_{toe}$= 0.7 to 0.5	for depths = 4 to 10 m	
$d_{s,max}/h_{toe}$= 0.5 to 0.3	for depths = 10 to 20 m	

An inclined wall produces less scour than a vertical wall.
Seawalls in the backshore with a beach in front of it show less toe scour than more exposed seawalls.
The scour depth strongly increases when a current is generated near the structure (undertow).

10.2 Wave-related toe scour near rubble-mound breakwaters

$d_{s,max}/h_{toe}$= 1 to 0.5	for depths < 4 m	(10.2)
$d_{s,max}/h_{toe}$= 0.5 to 0.3	for depths = 4 to 10 m	
$d_{s,max}/h_{toe}$= 0.3	for depths = 10 to 20 m	

10.3 Wave-related scour near (round) tip of vertical wall-type breakwater

$d_{s,max}/B$= 0.02 to 0.4 for KC= 1 to 10 (10.3)

with: B= width of wall

Scour is maximum at the mid-tip position.
Scour increases by a factor 2 for a wall with a sharp tip in stead of a round tip.

10.4 Wave-related scour near tip of rubble-mound breakwater

$d_{s,max}/h_{toe}$= 1 to 0.5	for depths < 4 m	(10.4)
$d_{s,max}/h_{toe}$= 0.5 to 0.3	for depths = 4 to 10 m	
$d_{s,max}/h_{toe}$= 0.3	for depths = 10 to 20 m	

Scour is maximum in the lee-side zone of the tip of the breakwater during conditions with breaking waves.
Scour decreases with decreasing slope of the structure (factor 2 for 30^o in stead of 45^o).

10.5 Current-related scour near tip of rubble-mound breakwater

$d_{s,max}/h_{toe}$= 4 to 2	for depths < 4 m	(10.5)
$d_{s,max}/h_{toe}$= 2 to 1	for depths = 4 to 10 m	
$d_{s,max}/h_{toe}$= 1 to 0.5	for depths > 10 m	

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