Review on sediment scour, transport, and deposit in coastal environments, including under tsunami forcing

Jørgen Fredsøe, Techn. Univ. Denmark
At which location are the flow velocity largest?

Carrier et al 2003: at the moving shoreline. Max inshore direction: initial waveform depression. Offshore directed if initial wave have a dominant elevation characteristic.
Soil properties?? Sand in the sea – and in the beach. Brought further onshore: ???
I: Run up: onshore sediment transport

II: Sedimentation

III: Draw down: offshore sediment transport
Flow velocities

River flow:

- lower reaches: flood: 2.5-4 m/s, Sand
- Upper reaches, Mountain streams: 4-6 m/s, stones
Rivers: Engelund-Hansen: $\theta$ smaller than 2-3
But coarser fractions can be mobilized
Waves

D=40 m  T=15 sec  H=20m (e.g. the North Sea):

U(max)=3.7 m/s  \( \theta \) about 5. (thin boundary layer).
Tsunami: T=13 min, Amplitude=0.75 m in 2000 m waterdepth

\[ \xi = \frac{S}{\sqrt{H_0 / L_\infty}} \]

- Slope: 1/15: \( \xi = 53 \), Vertical run-up=3.5 m, U(max)=0.42 m/sec
- S=1/120: \( \xi = 6.6 \), run-up=9.9 m, U=9.5 m/sec
Observed or estimated flow velocities

- Hokkaido 1993: 10-18 m/s (Tsutsumi et al, ASCE, 2000) (evaluated by considering deformation of railway tracks)
Morphology on land:

Flat

Dikes

Dunes
Planform view

longshore variations in dune crest level etc
2004 Indian Ocean Tsunami

- Incised Erosional Channels
  (30 m long & 1 m deep)
What special is there about a Tsunami from a hydrodynamic/sediment-transport point of view, compared to coastal or river environment.

- Magnitude of flow velocity
- Duration (transient problem)
- The magnitude of run-up
- Flow reversal
- Groundwater flow
- Sediment properties. (fine sediment + large V)
What to focus on?

1. Basic concepts of sand transport mechanisms

2. Boundary layers (bed friction) and turbulence in waves.

3. Net transport in a Tsunami

4. Scour in waves
PART 1: Non-cohesive sediment transport modeling
Some simple physical considerations on transport of bed load, suspended load and sheet flow
\[ \tau, U, \sqrt{u'}^2 \]
\[ F_d = \frac{1}{2} \rho c_D \left[ \alpha U_f - U_B \right]^2 \frac{\pi}{4} d^2 \]

\[ F_s = W \mu_d = \frac{\pi}{6} \rho g(s - 1) d^3 \mu_d \]
bedload transport.mov
\[ F_d = \frac{1}{2} \rho c_D \left[ \alpha U_f - U_B \right]^2 \frac{\pi}{4} d^2 \]

\[ F_s = W \mu_d = \frac{\pi}{6} \rho g (s - 1) d^3 \mu_d \]

\[ \frac{U_b}{U_f} = \alpha U_f \left[ 1 - 0.7 \sqrt{\frac{\theta_c}{\theta}} \right] \]
Tau is transferred to the grains as drag, so $\tau$ decreases to the critical

$$\tau_b = \tau_c + nF_D$$

$$\theta = \frac{\tau_b}{\rho g (s-1)d}$$

$$\theta = \theta_c + \frac{\pi}{6} \mu_d \rho$$
$$q_b = \frac{\pi}{6} d^3 \frac{p}{d^2} U_b$$

$$\Phi_b = \frac{q_b}{\sqrt{(s-1)gd^3}} = 8(\theta - \theta_c)^{3/2}$$
\[ C = C_b \left( \frac{D-z}{z} \frac{b}{D-b} \right)^Z \]

\[ Z = \frac{w}{\kappa U_f} \]
Suspended sediment

- Requirement for a bed particle to go into suspension

\[
\frac{w_s}{U_f} \leq 0.8 - 1.0
\]
Settling = diffusion
mixing length = l, vertical velocity fluctuation = \Lambda

\[ c_w + \frac{1}{2} v \ell \frac{dc}{dy} = 0 \]
Problems

Bed concentration

Influence of coherent structures in the turbulence
Experiments on a gravity-free dispersion of large solid spheres in a Newtonian fluid under shear

By R. A. Bagnold, F.R.S.
Figure 1. a, Cross-section of equidistant grain arrangement (three-dimensional). (Grains of alternate layers displaced in z-direction.) b, Two-dimensional sketch of possible statistically preferred grain arrangement (non-equidistant) which might allow of dispersive pressure proportional to shear stress, in a viscous fluid.
\[ \mu' = \mu (1 - \frac{\phi}{\phi_m})^{2.5\phi_m} \]

\[ \phi = \text{solid fraction, } = 0.64 \]
\[
\sigma = 0.013 s \left( \lambda d \frac{dU}{dz} \right)^2
\]

\[
c = \frac{0.65}{(1 + 1/\lambda)^3}
\]
Bed concentration of suspended sediment

\[ \frac{\tau_s}{\rho} = 0.013s \left( \lambda d \frac{dU}{dz} \right)^2 \]

\[ \tau_b = \tau_c + nF_D + \tau_s \]

\[ \tau_s = \frac{0.013}{\kappa^2} s\Theta\lambda_b^2 \]
Figure 7.16 Bed concentration $c_b$ versus $\theta'$, assuming $\theta_c = 0.05$, $s = 2.65$, and $\mu_d = 0.50$. 
FIG. 7(a). Comparison of Einstein's, Smith and McLean's, and Engelund and Fredsoe's Formulations to Values of $c_b$ Determined; and (b) Comparison of van Rijn's Relation for $c_b$ to Experimental Values
Sheet flow

• Wilson 1966:”The moving solid particles appeared generally to be travelling in a dense layer immediately above the bed supported by intergranular collisions rather than by fluid turbulence”
FIG. 4. Sequence Illustrating Transition from "No Suspension" Sheet-Flow Regime to "Suspension" Sheet-Flow Regime for Tests Referred to in Fig. 3(b): (a) $\theta = 1.2$; (b) $\theta = 1.6$; (c) $\theta = 2.0$; (d) $\theta = 3.0$; (e) $\theta = 4.0$
Break: Sheet flow movie
Fig. 1. Wilson's experimental data, Ref. [2].
Collisions between particles create a stress field (a particle pressure \( P \) or \( \sigma \))

- Jenkins (1987) kinetic theory for rapid grain flow
- Jenkins and Hanes (JFM 1998): turbulent velocity fluctuations neglected
- Hsu, Jenkins and Liu (Proc Royal Soc 2004): incl description of turbulence
A simple sheet flow model (Engelund 1981)

- Based on Bagnolds expression
- Assumes the particles to move with max concentration in a layer of considerable thickness - compared to the grain diameter
Fig. 2. Distribution of sediment concentration $c$ and dispersive stress $\sigma$. 
\[(\rho_s - \rho_f)cg + \frac{d\sigma}{dz} = 0\]

or

\[d\left(\frac{\sigma}{\rho}\right) = -c(s - 1)g\]
\[
\frac{\sigma_0}{\rho} = c_0 (s - 1) g L
\]
\[
\frac{dU}{dz} = \frac{U_f}{\kappa z},
\]
\[
z = d
\]
\[
\frac{\sigma_0}{\rho} = 0.013s \left( \lambda_0 d \frac{dU}{dz} \right)^2
\]
\[
= 0.013s \left( \lambda_0 d \frac{U_f}{\kappa d} \right)^2 \approx 1.3s U_f^2
\]
\[ d^p = c^0 \Gamma \cap B \]

\[ U_B = KU_f \]

\[ K = 10.5 \]

\[ c_0 = .33 \]
\[
\frac{L}{d} = 1.3 s \theta \\
\Phi_b = 4.5 s \theta^{3/2}
\]
s = 2.670: eq3
s = 1.138: eq4

\[ \Phi = 12(\theta - \theta_c)^{3/2} \]

\[ \Phi = 5(\theta - \theta_c)^{3/2} \]
Figure 5. Comparison of sheet layer thickness $\delta_s$. ○, results from the present model; ×, visual observations by Sumer et al.

Figure 6. Non-dimensional total sediment transport rate $\Psi = \frac{q_s}{d\sqrt{(s - 1)gd}}$ with respect to the Shields parameter. ○, Results from the present model; $\Psi = 20.0(\theta - 0.05)^{1.6}$.
Sediment mixtures
\[ F_d = \frac{1}{2} \rho c_D \left[ \alpha U_f - U_B \right]^2 \frac{\pi}{4} d^2 \]

\[ F_s = W \mu_d = \frac{\pi}{6} \rho g (s - 1) d^3 \mu_d \]
Bed forms: not in sheet flow

Flow direction: bottom right to top left corner
Conclusions:

• Sediment transport in a tsunami will usually be as sheet flow and in suspension.
• Models are available for the sheet flow in current
• Coarse sediment will be transported as bedload, but might be easier mobilized due to presence of fine sediment
• The transition from the sheet flow layer to the suspended sediment is not fully described, and the bottom boundary condition for the suspended sediment needs clarification
• Transport of mixtures needs to be studied
PART 2: SEDIMENT TRANSPORT IN WAVES

• Needed for sediment transport: friction, level of turbulence
Potential flow and Wave boundary layer
Flow velocities in the wbl
Turbulence level in wbl
Friction factor
Boundary layer thickness

\[ f_w = \frac{1}{2} \rho U_{10}^2 \]
Vertical distribution of sediment

\[
\frac{\partial c}{\partial t} = \frac{\partial}{\partial y} \left( wc + \varepsilon \frac{\partial c}{\partial y} \right)
\]

\[c_{\text{bed}} = c_b(\theta)\]
Distribution of suspended sediment: comparison with lab measurements
Distribution of suspended sediment: comparison with field measurements
Sediment in broken waves
Distribution of sediment in a spilling breaker
Lin and Liu, 1998:

\text{VOF (RIPPLE)} + \text{k-ε} \text{ Normalized turbulence intensity}

\textit{Breaking waves in the surf zone}
On or off shore transport

Waves:
Asymmetry in nearbed velocities
Drift (time averaged)
Streaming (time averaged)
Percolation
Wave breaking: Undertow (time-averaged)
Wave asymmetry

- Larger flow velocities onshore than offshore
- Sed tr proportional to $U^{**3}$
Ribberink 2006: wave asymmetry

Fig. 7. Measured net transport rates (symbols) and predicted net transport rates with PSM (lines) for asymmetric waves as a function of $U_{\text{rms}}$ (sheet flow regime). The solid line and black symbols refer to 3 medium sands ($0.21$, $0.32$, $0.46$ mm), the dashed lines and open symbols refer to 2 fine sands ($0.13$, $0.15$ mm).
Adaptation length – or time
Percolation
Web search

- Tsunami AND turbulence
- Boundary layer under a solitary wave
Net transport in a Tsunami (erosive or depositeive)

1: the very much idealized case
Run up

\[ U = 5 \text{m/s} : \]
\[ \delta = 1 \text{m after } 60 \text{ sec.} \]
Draw down: boundary layer much thicker

Result: larger friction on- than off-shore (?)
Fig. 6. Variation of $f_0$ and $f_b$ spanning the high tide cycle at Gleaeden Beach, OR. Cross-shore set-up level as obtained from leading edge time series. Friction coefficients as estimated from comparisons between actual and ballistic swash trajectories. The dotted line is a second-order fit through the backwash data only.
Larger flow velocities during draw down than in the Run-up.
Flow in the sand

• Weak seapage
• Liquefaction
• Fluidization
Austin et al 2006: Gravel beach
Seepage

Ground water flow pattern in the beach during falling sea level.
Figure 1. Schematics of soil fluidization in a granular bed in stages of (a) unfluidized and (b) fluidized responses (Huang, 1996).
Seepage

Simona Francalanci
University of Florence, Italy

Master Class
Lisboa 5 September 2006
Experimental set-up

The system was a sediment-recirculating, water-feed flume.

- Sand layer
- Fine filter
- Perforated sheet
- Seepage box
- Valve
- Flow meter
- 6 cm
- 110 cm
- Bed flume
- Incoming Pipeline
**Theoretical background**

A ground-water flow induces a non-hydrostatic pressure distribution along the vertical.

Darcy's law

\[
v_s = -K \frac{\partial h_p}{\partial z}
\]

\[
\frac{dh_p}{dz} = \frac{d}{dz} \left( \frac{p}{\rho g} + z \right) = -\frac{v_s}{K}
\]

- \( \frac{dp}{dz} = -\rho g \) - Hydrostatic pressure
- Archimedean buoyant force
- \( \frac{dp}{dz} = \rho g \left(-1 - \frac{v_s}{K}\right) < -\rho g \) - Non-hydrostatic pressure
- Increased buoyant force
- \( \frac{dp}{dz} > -\rho g \) - Non-hydrostatic pressure
- Decreased buoyant force

**General expression for the pressure distribution**

\[
\frac{1}{\rho g} \frac{dp}{dz} = -1 \frac{v_s}{K}
\]

Vertical pressure gradient \( dp/dz \) near the bed and buoyant force \( F_b \) acting on a bed particle.
Effect of an upward seepage flow on bed dynamics

Short-term experiment

Time 10 min - Velocity 20x

Run 2-3

Side panoramic view

Velocity 5x

Run 2-3
Results – the total shear stress

The evaluation of the shear stress at the bed is done through the momentum integral equations in the case of a boundary seepage, using the water surface slope, the bed slope, the seepage velocity and other flow parameters, depth-averaged velocity and water depth.

\[ \tau_b = \rho u^2 - \rho g h \sin \phi_b - \rho g h \left( \frac{dh}{dx} \cos \phi_b - \frac{\rho U^2}{g h} \right) - 2 \beta \rho U v_s \]

Chen & Chiew (1998a)

Two combined effects on Shields parameter

\[ \tau^* = \frac{\tau_b}{\left( \rho_s - \rho \left(1 + \frac{v_s}{K}\right) \right) g D} \]

Reduction of the shear stress with upward seepage

- experiments
- Chen & Chiew (1998a)
Theoretical and numerical model

Numerical results of short term experiments (10 min) with upward seepage flow, reduction of shear stress and no-correction in the Shields parameter

$$\tau^* = \frac{\tau_b}{(\rho_s - \rho)gD}$$

Under-estimation of local scour depth
Results - On the incipient motion condition with upward seepage

The critical condition for the onset of motion is affected by the upward seepage: the critical shear stress required for a particle to move over the porous bed will decrease in the presence of an upward seepage (Cheng & Chiew, 1999).

\[
\left( \frac{U_c}{U_{oc}} \right)^2 = 1 - \left( \frac{v_s}{v_{sc}} \right)^m
\]

\[
\frac{\tau_c}{\tau_{oc}} = 1 - \left( \frac{v_s}{v_{sc}} \right)^m
\]
Seepage "destroy" the velocity profile
Liquefaction: reshaping the grain skeleton
Fig. 6. Typical wave-induced unfluidized and fluidized pore pressure responses in Sand II ($d_{50}=0.092$ mm) of (a) Test series 3, (b) Test series 8 and (c) in silt (Foda and Tsang, 1994).
Fig. 8. Behaviour of the accumulated pore pressure for large times. $h/L = 0.145$, $d/L = 0.059$, $H/d = 0.54$ for $H = 9.1$ cm, and 0.75 for $H = 12.8$ cm, and $c_vT/d^2 = 7.1 \times 10^{-4}$.
Momentary liquefaction

Figure 10.2: Typical distributions of pore pressure (in excess of hydrostatic pressure) during the passage of wave trough. (a) Saturated soil. (b) Unsaturated soil.
Is it a bore?? –or a fast rising tide- or a surge?? more sediment on-than off-shore
\[ \xi = \frac{S}{\sqrt{H_0/L_\infty}} \]

Fig. 1. Plot of the non-dimensional (a) run-up and (b) maximum horizontal velocity as a function of \( \xi \) for various \( a_0/h_0 \).
Petti & Longo, Coastal Eng 2001

Fig. 10. Frame sequences at the breaking for tests $T = 2$ s and $T = 3$ s (time step $2/25$ s).
Effect of external generated turbulence on sediment-transport (Sumer et al. 2003)
<table>
<thead>
<tr>
<th>Symbol</th>
<th>(\frac{\sqrt{u'^2}}{U_{fb}})</th>
<th>Flow</th>
<th>Authors</th>
</tr>
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<tr>
<td>•</td>
<td>1.7</td>
<td>Undisturbed</td>
<td></td>
</tr>
<tr>
<td>□</td>
<td>1.7</td>
<td>Present</td>
<td></td>
</tr>
<tr>
<td>○</td>
<td>1.7 - 2.0</td>
<td>With turbulence generator;</td>
<td></td>
</tr>
<tr>
<td>×</td>
<td>2.0 - 2.25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>△</td>
<td>2.25 - 2.5</td>
<td>Long grid</td>
<td></td>
</tr>
<tr>
<td>+</td>
<td>2.5 - 2.75</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M</td>
<td>Undisturbed</td>
<td>Meyer-Peter and Muller (1948)</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>Undisturbed</td>
<td>Engelund and Fredsøe (1972)</td>
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</tbody>
</table>
Scour

Part 1: SCOUR IN DUNES:
Important for a Tsunami: timescale
Dunes
b. Looking east, at breach cut through the beach berm.

Figure 1. Mecox Pond, breached the morning of February 14, 1998, (photographs by N. Kraus, afternoon of February 14).
Breach 1d

BarrierBreach_1D.avi
Need for research

• Wave boundary layers in runup and drawdown, and under breaking waves
  • Impact of sheet flow on near bed turbulence – and the bed condition for suspended sediment in the sheet flow regime
  • Transport of mixtures in the sheet flow
Part 2

• Scour around structures in a Tsunami-like environment
roads, bridges, buildings, etc.
- Culverts
- Bridge abutments
Types of scour (Melville and Coleman)

Figure 1.1. The types of scour that can occur at a bridge.
Degradation

Figure 4.8. Degradation exacerbated by sand extraction for Republic of China.

Figure 9.15. Example of the use of check dams to control degradation.
Figure 1.3. Failure of Waipaoa River Rail Bridge (situated near Gisborne, New Zealand) during Cyclone Bola, March 1988, due to channel shift at the bend (flow from the top to the bottom of the figure).

(Melville and Coleman)
Figure 1.5. Floating debris accumulation at a bridge pier, bridge on Tauwhareparae Road over Mangahei River, New Zealand, Cyclone Bola, 1988.
The wood increases the effective width of the pier and decreases the effective clear span for the flow. In addition it spoils the smooth lines of the pier.

Figure 6.35. Local scour depth variation with quantity of floating debris.
Figure 1.10. Bridge failure mechanisms.

(a) Pier undermining (complete undermining or due to a loss of friction length)

(b) Boulder impact

(c) Debris rafting/impact
(a) Noncohesive - Flow Slides

- Failure surface
- Fluvial erosion of lower bank material

(b) Cohesive

- Tension crack
- Shallow rotational slip
- Deep rotational slip
- Plane slip
Figure 3.1. Spatial scales appropriate to the different types of scour.
Scour in a current: Bridge scour:
\[ d_s = K_{yb} K_I K_d K_s K_\theta K_G K_T \]

yb = depth-size
I = flow intensity
d = sediment size
s = shape
\( \theta \) = pier alignment
G = channel geometry
T = time
Scour depth variation with velocity

Figure 6.7. Local scour depth variation with flow intensity.
Figure 3.25: Effect of sediment gradation. Data by Baker (1986). $d_{50} = 0.6$ mm.
From Melville and Surtherland (2000).
<table>
<thead>
<tr>
<th>Foundation Type</th>
<th>Class</th>
<th>B/y</th>
<th>Local Scour Dependence</th>
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<td>Pier</td>
<td>Narrow</td>
<td>b/y&lt;0.7</td>
<td>d_8 ∝ b</td>
</tr>
<tr>
<td></td>
<td>Intermediate width</td>
<td>0.7&lt;b/y&lt;5</td>
<td>d_8 ∝ (by)^0.5</td>
</tr>
<tr>
<td></td>
<td>Wide</td>
<td>b/y&gt;5</td>
<td>d_8 ∝ y</td>
</tr>
<tr>
<td>Abutment</td>
<td>Short</td>
<td>L/y&lt;1</td>
<td>d_8 ∝ L</td>
</tr>
<tr>
<td></td>
<td>Intermediate length</td>
<td>1&lt;L/y&lt;25</td>
<td>d_8 ∝ (Ly)^0.5</td>
</tr>
<tr>
<td></td>
<td>Long</td>
<td>L/y&gt;25</td>
<td>d_8 ∝ y</td>
</tr>
</tbody>
</table>

**Variation with shallowness**
Figure 6.10. Local scour depth variation with sediment coarseness.
Figure 6.17. Local scour depth variation with pier alignment.
For \( y/D \leq 6 \), multiply \( t \) by:

\[ 0.64 \left( \frac{y}{D} \right)^{0.25} \]

Figure 6.32. Plot of (6.14) for equilibrium time \((y/D > 6)\).
• Scour movie by Roulund
Stone-protection
• foredrag\Copy of Riprap1.avi
Sacrificial piles: deflects the high velocity flow
Vanes: inducing secondary currents which interfere with the horseshoe vortex

Figure 9.26. The use of Iowa vanes as a pier scour countermeasure.
Collar: Shields sediment bed from the downflow and the horseshoe vortex
Unprotected pile, where vortex is well-defined before shedding

Pile with splitter-plate, where vortex grows initially and finally breaks down
- Wires of diameters 2 cm and 3 cm
- Thread angles $\alpha$ for piles having diameters 4 cm and 6 cm were 65° and 60°

Threaded pile (helical wires or cables wrapped spirally on the pile)
Marine environment
Scour around a cube under wave action

(Univ. Sydney)
Difference in scour mechanism in between current and waves:

Flow attack from two directions

KC-dependence (maybe no vortex shedding)
Wave scour depends on KC

\[ KC = \frac{2\pi a}{D} \]
Tsunami scour

Does the flow last long enough time so the scour will develops fully?

Time-scale!
Additional problems:

- liquefaction: reshaping the grain-skeleton
- Momentary fluidization: the effective stresses between the grains disappear due to groundwater flow.
Two very idealized cases:

- The vertical cylinder (pile)
- The horizontal cylinder (pipeline, seewage pipe etc).
Pressure variation in a current

Pipeline, placed on the ground
- Onset of scour: Seepage flow
Figure 2.6: Sequence of flow pictures over one wave cycle. Sumer et al. (2001 a).
Onset of Wave scour depends on KC

\[ KC = \frac{2\pi a}{D} \]
Tunnel erosion (piping)

Figure 2.10: Definition sketch of approach flow.

Figure 2.11: Tunnel erosion below a pipeline.
Scour development, current

Figure 2.13: Scour development. Times in minutes. $\theta = 0.098$. Current. Mao (1986).
Figure 2.21: Lee-wake effect.
Final scour depth

Tsunami??

\[ S/D = 0.1 \sqrt{KC} \]
\[ T^* = \theta^{-5/3} / 50 \]

\[ T^* = \frac{\sqrt{g(s-1)d^3}}{D^3} T \]

Figure 2.43: Time scale. All data (current/wave). Live bed \((\theta > \theta_{cr})\). Fredsøe et al. (1992).
Example: Diameter=30 cm
grain size=.5 mm
Waterdepth=5 m

• Flow velocity=.6 m/s T=1.4 hour
• Waves: waveperiod=10 sec H=2m
depth=10m: T=5min
• Flow velocity=6m/s T= 3 sec.
Vertical structure: the single pile

(a) Scour hole at a vertical circular pile under waves

(b) For $6 < KC < 100$, vortex shedding is the dominant mechanism for scour at piles under waves
Figure 3.37: Equilibrium scour depth. Effect of cross-sectional shape. Live bed ($\theta > \theta_{cr}$). Sumer et al. (1993).
Current

\[ T^* = \theta^{-2.2} h / D / 2000 \]

\[ T^* = \sqrt{g(s-1)d^3} / D^2 T \]

Waves

\[ T^* = 10^{-6} \left( \frac{KC}{\theta} \right)^3 \]
• D=3m d=.5 mm V=.6 m/s h=5 m : T=50 hours
• V instead 6 m/s : T=8 sec,
Threaded pile (helical wires or cables wrapped spirally on the pile)

- Wires of diameters 2 cm and 3 cm
- Thread angles $\alpha$ for piles having diameters 4 cm and 6 cm were $65^\circ$ and $60^\circ$
• Experimental results of unprotected piles correspond closely with the curve proposed by Sumer et al. (1992)
• Reduction of scour depth for protected piles is prominent

Variation of nondimensional scour depth $S/D$ with $KC$ number for unprotected and protected piles under waves
Fluidization

Fig. 1. Schematics of soil fluidization in a granular bed in stages of (a) unfluidized and (b) fluidized responses (Huang, 1996).
Fig. 5. Typical (a) wave records and wave-induced unfluidized pore pressure responses in Sand I ($d_{50}=0.134$ mm) at depths of $d=$ (b) - 6 cm, (c) - 10 cm and (d) - 30 cm (Test 1-2: $H=10.3$ cm, $T=1.49$ s).
Sand

gravel

Figure 9. Wave height (blue line), and pore pressure heads at 10 cm (cyan), 20 cm (yellow) and 30 cm (pink) depth, measured for Case 1: (a) at the front of the cylinder, (b) at the back of the cylinder.

Figure 12. Wave height (blue line), and pore pressure heads at 10 cm (cyan), 20 cm (yellow) and 30 cm (pink) depth, measured for Case II: (a) at the front of the cylinder, (b) at the back of the cylinder.
Figure 7. Scour depth as a function of time for Case I. Crosses, at the front of the cylinder; squares, at the side; circles, at the back. The arrow indicates the time of flow reversal, 6 s after wave impact.

Figure 11. Scour depth as a function of time for Case II. Crosses, at the front of the cylinder; squares, at the side; circles, at the back. The arrow indicates the time of flow reversal, 5 s after wave impact.
Need for research

- Is scour different in the sheet flow regime
- Scour in super-critical regime
- Timescale for scour at large Shields parameters
- Scour around structures with large horizontal dimensions compared to waterdepth