ON OFFSHORE SCOUR AND SCOUR PROTECTION

with the Ekofisk site as an example

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NORGES TEKNISK - NATURVITENSKAPELIGE FORSKNINGSRÅD
(ROYAL NORWEGIAN COUNCIL FOR SCIENTIFIC AND INDUSTRIAL RESEARCH)

VASSDRAGS- OG HAVNELABORATORIET
VED NORGES TEKNISKE HØGSKOLE — TRONDHEIM TILSLUTTET SINTEF
RIVER AND HARBOUR LABORATORY
AT THE TECHNICAL UNIVERSITY OF NORWAY
— a SINTEF associate Laboratory —

Utført: Sep - Dec 1973

Oppdrag nr.: 601 088
The physics of scour is briefly examined both for unidirectional currents and for waves. The feeble experience from ocean structures is cited.

For the Ekofisk site estimates show bottom velocities to grossly exceed existing stability criteria. If the site is swept by a current with a persistent direction scour is likely to occur within a distance of one tank radius from the tank.
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1. INTRODUCTION

One of the items that has to be considered when designing a sea structure resting on a sandy bottom is local scour and scour protection. Scour holes at a structure may affect the overall stability of the structure as well as the stresses in the structure itself. It is, therefore, of importance to know if scour may occur and to take possible measures for scour protection.

The structures that one has in mind in the following are gravity structures, typical examples of which are shown in Fig. 1.

![Fig. 1](image_url)

2. GENERAL ON SCOUR

2.1. Unidirectional flow

The following is a very short review of some of the fundamental aspects of sand motion in flowing water. Emphasis is placed on initial motion and local scour.
Movement of sand particles in flowing water occurs when the flow velocity is above a certain critical velocity. On a plane sand bed there may be motion of the particles without scour. In this case the amount of sand moved into an area is the same as the amount of sand moved out of the area. The sand bed is said to be dynamically stable.

If we place a structure in a stable sand bed, the flow pattern will be changed. If there was no sand motion before, or if the bed was dynamically stable, increased water velocities in the vicinity of the structure may cause the sand to move more intensely in certain areas, thus causing scour holes at the structure with depths determined by a new dynamically stable configuration.

The concept of shear force from flowing water is demonstrated in Fig. 2.

A two-dimensional uniform canal flow is assumed. The forces acting on the water element are the weight \( W_1 = \gamma \cdot d \cdot dx \), the integrated water pressure \( p \), and the integrated shear force \( \tau \) at the bottom:
\[ \frac{1}{2} \gamma \cdot d^2 + \gamma \cdot d \cdot \text{dx} \cdot \sin \alpha = \frac{1}{2} \gamma d^2 + \tau \cdot \text{dx} \]

or

\[ \tau = \gamma \cdot d \cdot \sin \alpha . \]

Since \( \alpha \) usually is small,

\[ \sin \alpha = \tan \alpha = I, \]

the slope of the canal.

If we apply the Chezy formula for the flow in the canal, we have

\[ V = C\sqrt{d \cdot I}, \]

where

- \( C \) = Chezy's coefficient
- \( V \) = average velocity.

We get then

\[ \tau = \gamma \cdot \frac{v^2}{C^2}. \]

The expression \( V_* = \sqrt{\frac{\tau}{\rho}} \), where \( \rho \) = water density, is designated "shear velocity".

The flow velocity varies from the bottom and up as indicated in Fig. 3.
For turbulent flow with a free surface, the velocity distribution with height above the bottom is

\[ \frac{V}{V_s} = \frac{1}{\kappa} \ln \frac{V}{k_v} + B_s \]

where \( \kappa = 0.4 \) (von Karman's constant).

The variation of \( B_s \) vs. \( \frac{V_k k_v}{V} \) (Reynolds number) is shown in Fig. 4.

The start of sand motion is governed by the grain size and the shear velocity. The critical shear velocity can roughly be found by use of Shields diagram, Fig. 5:
To find the critical shear velocity from this diagram, a trial and error procedure must be used. Enclosure No. 1 from [3] gives the critical shear velocity vs. grain size distribution.

2.2. Waves

If there is wave action, the particle velocity distribution is different from the distribution in flowing water. The water particle motions due to waves are as shown in Fig. 5.

The water particles move in nearly closed orbits; but they have a slight forward motion (mass transport velocity).

It is to be expected that the shear stress due to waves is different from that due to unidirectional flow. The particle velocities close to the bed and shear velocities due to waves have not been as thoroughly investigated as for the case of unidirectional steady current. Riedel, Kamphuis and Brebner [4] measured \( \tau_o \) and estimated values for the friction factor \( f_w \) used in the following expression for the shear stress due to waves:

\[
\tau_o = \frac{\rho}{2} f_w U_b^2
\]
\[ \rho = \text{density of the water} \]
\[ U_b = \text{water particle velocity at the bottom due to waves} \]

From this expression it follows that the shear velocity is

\[ V_* = \sqrt{\frac{\tau_0}{\rho}} = \sqrt{\frac{f_w}{2}} \cdot U_b \]

Enclosure No. 2 shows the results of Riedel et al. [4].

2.3. Current and waves

Bijker [5] has to some extent treated sediment transport due to waves and currents. But the flow field close to the bottom is not very well known and this case will not be considered further at this time.

2.4. Sediments in suspension

It is known that when the water velocity is above a certain value, some of the bottom material is thrown into suspension. It is of course difficult to state an exact value for the start of the material being suspended. Engelund [6] gives this criterion for the start of the suspension:

\[ \frac{W}{V_*} = 1 \]

where \( W \) is the settling velocity of the material. This velocity is dependent on the specific weight of the sand, the diameter of the grains, the "roundness" of the grains, and the kinematic viscosity of the water. Fig. 6 from [1] gives the settling velocity of spheres.

Bijker et al. [3] refers to Shinohara and Tsubaki [7] who state that for unidirectional steady flow the amount of material moved
in suspension is larger than the material moved as bed load when $\frac{V_s}{W} > 1.7$. A similar criteria for waves is not existing, but for the time being it is assumed that approximately the same criteria applies.

Fig. 6.

Chan, Baird and Round [13] are concerned with the effect of oscillatory liquid motion on a bed of dense particles. The intention with their study was to investigate the transition between a stationary bed regime and a complete suspension of particles in the liquid.

The transition from a dune-forming bed to a bed in which the surface particles were in motion through the oscillation cycle was found to occur when

$$\frac{a \omega^{1/2}}{\left(\gamma g\right)^{0.5} D^{0.1} \nu^{0.2}} = 6.6$$
where

\[ a = \text{liquid particle amplitude} \]
\[ D = \text{bed material diameter} \]
\[ \omega = \text{angular frequency} \]

The criteria for the transition of the material in total suspension seems somewhat unrealistic in relation to a prototype situation and is therefore not mentioned here.

2.5. Scour at ocean structures

The following is taken from the introduction of Reference [8]:

"The purpose of the study, the results of which are summarized in this report, is to define:

"The occurrence and effects of scour and fill in the vicinity of ocean bottom structures and foundations. The study shall consist of a summary and an appraisal of available information within the present state of the art and a compilation of methods which may be usable for predicting the occurrence and effects of scour and fill in the vicinity of ocean bottom structures and foundations".

"It may be stated at the onset that no case of scour and deposition in the deep waters could be found in a special library of about 10,000 reports and papers on ocean engineering and oceanography which was sufficiently complete to permit a significant analysis. There are instances in which a flow was observed, but not resulting in scour or deposition of sediment. In other cases the sediment pattern was described, but the flow that had caused it was unknown. One is therefore entirely dependent on experience obtained in the laboratory or under non-
ocean conditions in predicting the relationships between flows and the resulting scour and deposition of ocean sediment".

Numerous papers have been written on scour due to current at bridge piers etc. Reference [9] also describes some model results on local scour due to waves and current, but none are directly relevant to a large volume ocean structure.

Reference [10] gives some interesting results of prototype scour at a framed platform in about 13 m of water, where the bottom material is a mixture of fine sand and silt. Fig. 7 shows how the scour due to waves and current occurred.

It is seen that it is not local scour around each pile that occurred in this case, but a saucer-like-depression under the whole structure. Without any special viewpoint taken regarding the reason for this erosion, the following is quoted from [11], which deals with the same structure as [10]:

---

![Fig. 7](image-url)
"The more common type of erosion which causes individual scour holes can be visualized as being due to the local increase in velocity around the piers with consequent removal of the material under the constant surging of the induced current. In the new type of erosion, these local steady concentrations of current are too feeble to attack, but large transitory currents induced in the bed by the varying pressures under the great waves become big enough to throw bed material into suspension. Turbulence generated by the piles prevents it from settling until after it has been carried away from the structure. For those to whom the existence of flows through the bed seem unreal, we would point out that flow in a porous medium respond to unsteady conditions with a velocity of the order of that of sound waves, in other words practically instantaneous".

Data on scour of large ocean structures in deep water are understandably not available, since such structures are non-existing. The first problem is to estimate from general knowledge if scour possibly will occur and where this is likely to happen.

3. THE EKOFISK SITE.

3.1. Estimates of the bottom velocities in an irregular wave field

The water particle velocities (maximum) at the bottom due to regular waves are given by:

\[ U = \frac{\pi H}{T} \frac{1}{\sinh kd} \] (1. order wave theory)
An estimate of the distribution of peak velocities in irregular waves, design storm conditions, are made according to the following concept, which is often used in dealing with randomly varying signals.

Wave power spectrum $S_H(f)$

Transfer function $\frac{U_b}{H}$

Velocity power spectrum

$E_{U_b}(f) = \left(\frac{U_b}{H}\right)^2 \cdot S_H(f)$

The relation between the velocity power spectrum and the velocities is approximately:

$$\sqrt{U_b^2} = 2.83 \left(\int E_U(f) \, df\right)^{\frac{1}{2}}$$
It is further assumed that the velocity peaks are distributed according to a Rayleigh distribution:

\[ F(U_b) = 1 - \exp \left( -\frac{U_b^2}{U_b^2} \right) \]

A wave power spectrum which is assumed to be valid (approximately Ekofisk design waves) is shown in Enclosure No. 4. The spectrum has a significant wave height \( H_{1/3} \) of 12 m. In the following a wave period of 15 sec. is assumed for the Ekofisk site.

The transfer function of the water wave particle velocity at 70 m depth is shown in Enclosure No. 5 and the corresponding power spectrum of the water wave particle velocity in Enclosure No. 6. The spectrum has a significant particle velocity in Enclosure No. 6. The spectrum has a significant particle velocity of 1.54 m/sec. Enclosure No. 7 shows the estimated distribution of the particle velocity \( U_b \) in a free wave. The diagram shows also a distribution of the particle velocity \( U_p \) where \( U_p \) is the sum of the water wave particle velocity and a current of 0.40 m/sec. at the bottom.

In this connection it is also of interest to know the order of magnitude of the shear velocities due to waves (see chapter 2.2.):

\[ V_x = \sqrt{\frac{f_w}{2}} \cdot U_b \]

Based on the results in [4], a friction factor \( f_w \) of 3.4 \( \cdot 10^{-3} \) was calculated for the Ekofisk bottom sand \( (D_{50} = 0.15 \text{ mm}) \).

The transfer function of the shear velocities at a depth of 70 m and the corresponding power spectra are shown in Enclosure No. 8 and 9 respectively. The significant shear velocity was estimated to 6.06 cm/sec.
The distribution of $V_\star$ is shown in Enclosure No. 7. The critical shear velocity is given in the same diagram, indicating motion of the bed material at a relatively low wave particle velocity. The estimate of $V_\star$ critical is based on the diagram in Enclosure No. 1.

Diver experience from the Ekofisk area (personal communication, I. FOSS, DnV) suggests that motion does occur during storms. Following a storm the bearing capacity of the top sand layer is reduced so that the diver's feet sink in some 10 cm, while the normal imprint is insignificant.

BRATTELAND and BRUUN (1973) observed movement at the Ekofisk site of a fluorescent tracer with grain size distribution does the actual distribution for the local sediments. Fig. 8 shows the result of a sampling on May 3rd 1973 of a tracer released March 15th.

Samples were taken at 5 and 10 ft distances from the tracer source. It may be noted that tracers have mainly moved towards northwest and towards south, the concentration decreasing with distance from the tracer source.
3.2. Transport of bed material.

If it is assumed that the sand grains are spherical, the settling velocity of the grains \( \text{D = 0.15 mm} \) is about 1.5 cm/s. It is seen that even without the structure being present the ratio \( \frac{V}{w} \) will exceed 1.7 from a relatively low wave particle velocity. The ratio of 1.7 was assumed as the ratio when the suspended load is larger than the bed load[3].

The ratio between the wave particle excursion amplitude at a depth of 70 m and the amplitude estimated by the criteria given on page 7 is:

\[
\frac{a(d=70m)}{a_{\text{criteria}}} = 0.8
\]

where \( a_{d=70} \) is the significant wave excursion amplitude at the bottom. This ratio indicates a high motion intensity at the bottom (chapt. 2.4.).

Bottom shear stresses when the structure is present can not be calculated according to the results of [4] because the flow field is different from that during the tests described in [4].

Silvester[12] deals with sediment motion beyond the breaker zone. He has developed the following Table:

<table>
<thead>
<tr>
<th>Author</th>
<th>Equation</th>
<th>( T = 12 \text{ sec.} )</th>
<th>( T = 14.5 \text{ sec.} )</th>
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<tr>
<td>Carsnels</td>
<td>5</td>
<td>5.7</td>
<td>3.6</td>
</tr>
<tr>
<td>( N_s = 3.2 )</td>
<td>6</td>
<td>5.5</td>
<td>2.9</td>
</tr>
<tr>
<td>Bagnold</td>
<td>3</td>
<td>8.2</td>
<td>4.3</td>
</tr>
<tr>
<td>( \phi = 7.45 )</td>
<td>4</td>
<td>4.65</td>
<td>2.45</td>
</tr>
<tr>
<td>Minohar</td>
<td>3</td>
<td>8.2</td>
<td>4.3</td>
</tr>
<tr>
<td>Vincent</td>
<td>4</td>
<td>4.65</td>
<td>2.45</td>
</tr>
</tbody>
</table>
From the work of Silvester we take the following conclusion:

"The depth at which the ocean bed can be disturbed by waves was thought to be very limited when computed on the basis of mean shear stress. As replication of the water particle motion has approached that of natural conditions, this apparent reach of the waves has increased significantly. From the discussion presented herein, it would appear that the whole Continental Shelf is the stage for the sediment drama of oceanic swell".

The wave motion causes a back and forth oscillatory motion of the material when it is in bedload. This may not cause any significant scour, although the waves may cause a net motion in one direction due to the mass transport velocity. More serious may be the waves' apparent ability to throw the sand material into suspension. The current and the mass transport velocity of the waves may then move the suspended material away from the structure and thus scour holes may occur.

The ability to get the bed materials into suspension increases when the turbulence activity increases. The perforated wall-system of the EKOFISK TANK is designed to convert wave energy into turbulence, but to what extent this local turbulence picks up bed material is not known.

However, simply by perturbing the free flow field of the waves and the current, the tank has an important impact on the nearby bed. This question is dealt with in the next sections.
3.3. The bottom velocity field around a vertical, semi-submerged cylindrical tank in regular waves.

By reflection and diffraction of the incident waves, the tank will impose a new wave regime on its surroundings. The superposition of incident and reflected waves in front of the tank, and of crossing wave trains in the rear, cause higher than incident waves to occur several wavelengths from the structure.

To get an impression of the scouring potential of this new wave field, we have computed the new velocity field at the bottom.

The velocity potential around a vertical, cylindrical tank fixed to the bottom and extending through the surface due to a regular incident wave is, according to 1. order wave theory, given by

$$\phi = \frac{gH}{2\sigma} \frac{\cosh k(z+d)}{\cosh kd} e^{-igt} \sum_{m=0}^{\infty} \varepsilon_m(i) \cos(m\theta)$$

$$\cdot \left[ e^{i(\frac{\pi}{2} - \gamma_m)} \sin\gamma_m H_m^{(1)}(kr) - J_m(kr) \right]$$

Here $g$ is the acceleration of gravity, $H$ is wave height, $\sigma$ is frequency, $k$ is wave number, $d$ is mean water depth, $z$ is vertical coordinate, $r$ is radial coordinate, $\theta$ is angular coordinate measured anti-clockwise from the positive $x$-axis and $t$ is time. The parameter $\varepsilon_m$ is 1 for $m = 0$ else 2, and $J_m(kr)$ and $H_m^{(1)}(kr)$ are Bessel and Hankel functions respectively, of first kind and $m$-th order.

Based on this formula velocity amplitudes in radial and tangential direction are calculated for $\theta = 0, \frac{\pi}{2}$ and $\pi$, and $r = 1, 2, 3$ and 4 radial distances from the center of the tank, in all cases for $z = -d$ corresponding to points
on the bottom. The values calculated are approximate in the sense that the series expansion is terminated after \( m = 8 \), and \( t \) is given 10 discrete values within a half-period, and the numerically largest value is chosen as the amplitude value.

The results are presented in Enclosures 10-17 for depths ranging from 50 m to 200 m. All velocities are calculated relative to the velocity amplitude along the bottom in the incident wave.

3.4. The steady flow field near a cylinder.

The increased oscillatory motion due to the perturbation of the wave field by the tank increases the suspension of bed particles. Provided the steady current is not slowed down by the tank, the transport of sediments out of the perturbed area should increase. At the same time the transport of sediments into this area has not changed, and so scour will result.

The steady potential flow field near a cylinder is given by

\[
\phi = u(r + \frac{a^2}{r}) \cos \theta
\]

\( u \) - velocity at \( r = \infty \)
\( a \) - radius of cylinder
\( r, \theta \) - polar coordinates

The velocity potential \( \phi \) gives the radial and tangential velocities

\[
u_r = u(1 - \frac{a^2}{r^2}) \cos \theta \\
u_\theta = -u(1 + \frac{a^2}{r^2}) \sin \theta
\]
and a total velocity

$$|\mathbf{v}|^2 = u_2^2 + u_\theta^2 = u^2(1 - \frac{2a^2}{r^2} \cos 2\theta + \frac{a^4}{4})$$

We want to find the area where the perturbed velocity $\mathbf{v}$ exceeds the velocity $u$ without the cylinder. The limiting condition

$$|\mathbf{v}| = u$$

gives

$$\cos 2\theta = \frac{1}{2} \frac{a^2}{r^2}$$

At the cylinder $r = a$ and $\theta = 30^\circ$ so reduced velocities are found in a rather wide pear-shaped area upstream (and for the potential field, downstream) of the obstruction. (Enclosure No. 18).

The maximum velocity occurs at the side of the tank and is about twice the undisturbed velocity for the steady current and also for the longer waves.

Substantial velocity increases will prevail at considerable distances from the tank (within 1 diameter).
3.5. Pore pressure

As described on page 9-10 the wave action also induces an upward pressure force on the bottom particles, due to the pressure gradient set up within the soil.

We have estimated these gradients for North Sea conditions. Enclosures No. 19 and 20 show pore water pressure profiles for fine sand and silt, respectively, for a 15 second wave of height 24 metres. These pressures are directly proportional to the wave height, so a 12 m wave would give 50% of the values for a 24 m wave.

The estimates are based on the assumption that the bed material is nondeformable, and that on the other hand the water is compressible. This gives the following equation for the wave pressure transmission

\[ \frac{\partial^2 p}{\partial x^2} + \frac{\partial^2 p}{\partial y^2} = \frac{n_v \gamma}{kE} \cdot \frac{\partial p}{\partial x} \]

Here \( x \) and \( y \) are horizontal and vertical coordinates resp., \( n_v \) is relative pore volume, \( \gamma \) is specific weight of water, \( k \) is permeability of bed material and \( E \) is the modulus of elasticity of water. The solution to this equation for a bottom layer of thickness \( d \) when a regular pressure wave of amplitude \( p_o \), length \( L \) and period \( T \) has been acting on the bottom surface for a sufficiently long time is given by

\[ p(x,y,t) = \text{Re} \left[ p_o \frac{\cosh \mu(y+d)}{\cosh \mu d} \cdot e^{i2\pi \frac{x-x}{L-T}} \right] \]

with the vertical "wave number" \( \mu \) given by

\[ |\mu| = \left[ \left( \frac{2\pi}{L} \right)^4 + \left( \frac{2\pi}{T} \right)^4 \cdot \frac{n_v \gamma}{kE} \right]^{1/4} \]
\[ \text{Arg}(\mu) = \frac{1}{2} \arctg \left( \frac{\eta \gamma}{kE} \cdot \frac{L^2}{2\pi T} \right) \]

The bed also is assumed to satisfy the condition of isotropic flow.

The pore pressure gradient is highest at the mud line. For a 24 m wave a maximum hydraulic gradient of about 0.3 is obtained for fine sand and about 10 times higher for silt.

Liquefaction is assumed to result when the pressure gradient exceeds unity.

In the vicinity of the tank the pore pressures must also be influenced by the oscillating wave load transmitted from the tank to the sea bed. While this effect is likely to be confined to the close proximity of the tank perimeter, it may yield high lift forces on the bed particles.

3.6. Model tests on the EKOFISK oil storage tank

During the tests on the EKOFISK storage tank at RHL an unsuccessful attempt was made to measure the wave pressures under the tank when the tank was resting on fine sand with \( D_{50} = 0.076 \) (approximately). The scale of these tests was 1:100. It is of interest to look at the scour pattern at the tank and to estimate to what extent the model tests can represent prototype conditions.
Fig. 9 shows the bottom condition after approximately 1 hour test run with waves of height $H_m = 24$ cm ($H_{prot} = 24$ m) and periods $T_m = 1.5$ sec. ($T_{prot} = 15$ sec.). Since these tests were not concerned with scour, no exact record was made of bed movements.

Shear velocity in the model. To determine the surface roughness $k_s$ ENGELUND [14] suggests:

$$k_s = 2.5 \, D_f$$

where $D_f$ is the fall-diameter.

In accordance with [14] $D_f$ is estimated as a function of the median grain-diameter $D$. 
This gives:

\[ k_s = 0.225 \text{ mm} \]
\[ a = 6.27 \text{ cm} \]
\[ u_b = 26.3 \text{ cm/sec.} \]
\[ \frac{a}{k_s} = 280 \]
\[ Re = \frac{au_b}{v} = 1.3 \cdot 10^3 \]

Enclosure 2 shows that \( f_w \) is lying in the rough turbulent flow regime where \( f_w \) only is dependent on the ratio \( \frac{a}{k_s} \). This functional relation is shown in Enclosure 3, with a friction factor \( f_w \) of \( 0.07^{-3} \) which gives

Shear velocity \( v_x = 4.9 \text{ cm/sec} \)

Critical shear velocity \( v_{xcr} = 1.2 \text{ cm/sec} \)

Shear velocity in the prototype. The following prototype data are used:

- Wave height: \( H = 24 \text{ m} \)
- Wave period: \( T = 15 \text{ sec.} \)
- Water depth: \( d = 70 \text{ m} \)
- Bed material: \( D = 0.15 \text{ mm} \)

Calculation

\[ a = 6.27 \text{ m} \]
\[ u_b = 2.63 \text{ m/sec} \]

\[ \frac{a}{k_s} > 6000 \text{ i.e. Smooth flow regime (Enclosure 2)} \]

\[ Re = 1.3 \cdot 10^6 \]
The conclusion which may be drawn is: When a prototype bed material is used in a model, it is not possible to estimate the prototype scour situation on the basis of the model data. This sort of model scour investigation is in general not directly applicable for the prototype situation.

3.7. Preliminary estimates of the size of the stones in a stone blanket as scour protection

The best example known to us of a test of a material suitable for scour protection in waves was reported in Hydraulic Research 1968 (Appendix 1). This material was a layer of shingle. The tests were made in an oscillating water tunnel.

It is seen that the diagram of Appendix 1 does not cover a depth of 70 m. However, since the water particle velocity governs the size of the protection stones, the diagram may still be used.

The maximum water particle motion due to a regular wave at the bottom is given by

$$u = \frac{\pi H}{T} \frac{1}{\sinh \frac{2\pi d}{L}}$$
We now assume

- Wave height: 12 m
- Wave period: 15 sec.

giving \( u_{b_o} = 1.3 \text{ m/s} \).

Near the tank we assume

\[ u_{b_o} + u_o = 2 \cdot 1.3 + 0.4 = 3.0 \text{ m/s} \]

where \( u_o \) is the steady current.

This gives:

\[ \sinh \left( \frac{2\pi d}{L} \right) = \frac{\pi \cdot H}{T} \cdot \frac{1}{u_b} = \frac{\pi \cdot 12}{15} \cdot \frac{1}{3} = 0.84 \]

From tables

\[ \frac{d}{L_o} = 0.078 \]

\[ d = 0.078 \cdot L_o \]

where \( L_o = \frac{g}{2\pi} \cdot T^2 \) (deep water wave length)

For \( T = 15 \text{ sec.} \), \( L_o \approx 346 \text{ m} \) which gives

\[ d = 346 \cdot 0.078 \approx 27 \text{ m} \]

From the diagram in Appendix 1 it is seen that the required diameter of the shingle is \( D \approx 15 \text{ cm} \).
It must be noted that the flow around a large volume ocean structure is different from that of the tests on which the diagram has been based.

The diagrams in Enclosure 10 to 14 indicate that the tangential velocity component decreases with the distance from the tank.

Over a distance of about one "tank-radius" out from the tank wall the tangential velocity component exceeds the undisturbed bottom particle velocity. This means that the width of the layer probably will have to be 40-50 m wide.

It is stressed that further consideration and testing are needed before a final design of such a scour protection is made.

4. CONCLUSIONS

Offshore scour is the result of a combined action of waves and current. The waves provide the major destabilizing flow forces on the bed particles, while the current carries them away.

In the North Sea the wave action at the bottom is sufficient to move loosely deposited sand everywhere except, perhaps, in the deepest parts (d > 250 m).

The presence of a structure agitates the wave action and increases the bottom velocities in the general vicinity of the structure.
A cylindrical tank will slow down a steady current in a 90° sector facing upstream, and accelerate it over the remaining upstream halfspace. The highest velocities are twice the undisturbed current speed and occur where the sides of the tank are parallel to the undisturbed current.

A persistent current is likely to carve a scour pattern near a structure, and the intensity of wave action is likely to determine the depth of the scour holes.

However, if the current veers sufficiently, a redistribution of the sediments will occur and the deepest holes will probably be filled in.

The duration of a current from the same corner therefore seems to be the governing parameter.

If complete scour protection is desired for a cylindrical tank, it is necessary to cover the bed to a distance of at least one tank radius with a stone blanket. The required stone size depends on the location and can only be very approximately estimated at present.

Adequate protection may be provided by a narrower stone blanket, limiting the scour to a shallow ring at a safe distance from the tank. However, at present no method exists for predicting such a partial scour protection.
LIST OF REFERENCES


Appendix 1

From Hydraulic Research 1968.


Threshold of movement of shingle subjected to wave action

Experimental studies of the threshold of movement of granular beds under wave action have in the past had to be limited to the finer grain sizes. The Pulsating Water Tunnel, described in Hydraulics Research 1966, p. 82, has now made it possible to determine the necessary wave conditions for the initiation of movement of shingle sizes.

At the beginning of the study it was thought to be desirable to define the initiation of movement in some precise way rather than to rely on a purely visual assessment. A method, analogous to the zero transport approach, was carried out for a number of test runs by initially marking a band across the bed and subsequently noting the number and position of particles which had moved out of the marked area. This type of test was carried out for various wave amplitudes, at set wave periods, to determine the positions within the range of particle movement which were consistently in agreement with the calculated values. Thus, having calibrated the observer, all subsequent determinations were carried out visually.

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Another possible reason for inexactitude was anticipated. It was thought that the grading or non-uniformity in size of particles being tested might influence the result and give an unwanted scatter in the results. All materials tested were therefore sieved between close limits. This in turn raised the problem of particle shape. Most naturally occurring gravels have a distinct change in shape with change in particle size. An examination of several materials showed that limestone chips came closest to retaining the same shape over a wide range of sizes. Initial experiments were therefore conducted on various fractions of limestone chips, although tests on other materials became necessary in order to extend the range of conditions. As the tests proceeded it became apparent that shape variations to be expected in shingle would not have a measurable effect on threshold values, for shapes ranging from cubes to spheres had no apparent effect on the results. Recent tests on a fully graded gravel from the North Sea have indicated that grading is relatively unimportant and the $D_{90}$ size is the appropriate dimension to characterize it.

Within the shingle range (i.e. material >2 mm diameter) the Reynolds Number was found to play no part and a unified plot of the results for materials ranging from limestone chips and glass spheres to light-weight materials such as coal and perspex cubes was obtained by plotting the parameter $a g T^2$ against the parameter $a/D$, where $a$ is the semi-orbit length at the bed, $D$ the equivalent mean sphere diameter of the particles, $T$ the period of the wave, and $g$ the effective gravity, equal to $g (\rho_s-\rho)/\rho$, where $\rho_s$ = density of particles and $\rho$ = density of fluid. The parameter $a/D$ may be considered to be the ratio of drag force divided by acceleration force. $a g T^2$  would appear at first sight to be simply the ratio of acceleration force to gravitational force. However, since acceleration forces are relatively small this interpretation seems unlikely. Rather than this, it is taken to be an expression of the fact that there must be a time dependency. The drag force must act over...
a certain finite length of time before particles can become displaced, otherwise merely rocking will occur. This contrasts with the threshold condition in uni-directional flow where the time dependency is purely arbitrary.

From the unified plot a working diagram, Fig. 5, has been prepared, based on the assumption of shingle with a s.g. of 2.65. This makes it possible to determine the wave conditions necessary to move a particular size of shingle or, conversely, given a set of wave conditions to determine the size of material that will not move. Two examples are illustrated.

Example A shows that with an 8s wave, 3.5 m high in 20 m depth of water fine shingle with a median size of 6.4 mm would be on the point of moving. In the case of example B the height of a 6s wave in 4.5 m of water necessary to move 35 mm diameter shingle would be 2 m.

![Working diagrams](image-url)

Fig. 5. THRESHOLD OF MOVEMENT OF SHINGLE SUBJECTED TO WAVE ACTION
Bed shear stress velocity $v_\text{crit}$ versus D for various materials.
FRICITION FACTOR $f_w$ VERSUS $R_e$, AFTER [4]

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FRICTION FACTOR $f_w$ VERSUS $\frac{3}{k_s}$, AFTER [4]

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ASSUMED WAVE POWER SPECTRUM, $H_{y_3} = 12.0 \, m$
TRANSFER FUNCTION - WAVES

BOTTOM PARTICLE VELOCITIES AT WATER DEPTH OF 70 m

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VELOCITY POWER SPECTRUM
AT WATER DEPTH 70 m
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ESTIMATES OF DISTRIBUTION OF PEAK WAVE OF VELOCITIES AT THE BOTTOM-WATER DEPTH 70 m

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SHEAR VELOCITY POWER SPECTRUM
AT WATER DEPTH 70m

WAVE FREQUENCY f, sec⁻¹

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DEPTH 150
DIAM: 90

H/L = 0.1

L/D 1.87  Vg 0.018
L/D 3.33  Vg 0.058
L/D 5.00  Vg 0.205
L/D 6.67  Vg 0.609
L/D 8.33  Vg 0.140

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V/W versus the tank diameter

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14

VW

L/h = ∞

V/W

L/h

- 1
- 2
- 3
- 4
- 5

Depth: 50

Diam. (m)
$V_t/V_0$ VERSUS THE TANK DIAMETER

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$V_t/V_0$ VERSUS THE TANK DIAMETER

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V/V₀ VERSUS THE TANK DIAMETER

VERSUS THE TANK

DIAMETER

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PRESSURE PROFILES IN SAND BOTTOM FOR REGULAR WAVE.
PRESSURE PROFILES IN SILT BOTTOM FOR REGULAR WAVE.

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