CHAPTER 48

THE EFFECT OF SEEPAGE ON THE STABILITY OF SEA WALL

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INTRODUCTION

Vertical bulkheads, or retaining walls of the sheet-pile type, a often used as sea walls at locations not subjected to continuous or severe wave action. Many miles of this type sea wall have been con structed along the Florida coast and coastline of the United States ar have given satisfactory service. However, the failures of vertical s walls which continue to occur during mild storms indicate that the d sign procedures available, or actually used, may be inadequate.

The stability of vertical sea walls placed in cohesionless soil pends upon the relations between forces which tend to overturn the v and those which provide a restraining moment. Static forces on the wall are produced by the soil and water pressure of the backfill whi tend to overturn the wall, by water and passive soil pressures on th seaward side of the wall, and by anchor loads. Dynamic forces are also applied to the wall by direct wave action and by the forces deve oped in the soil masses due to seepage flow. The soil rebound afte wave impact on the wall increases the soil pressure of the backfill . requires the development of temporarily larger passive soil and an loads for continued wall stability. Seepage forces reduce the passi pressure that can be developed on the seaward side of the wall and thereby threaten wall stability.

The stability of a sea wall thus depends directly upon the cap ity of the soil to develop sufficient passive pressure at regions of designed restraint. Any factor which reduces the available passive soil resistance of loaded regions causes a reduction in the stability the wall.

Conditions such as waves overtopping the sea wall, rain wate falling behind the wall, or the accumulation of water run-off from higher ground may result in complete or partial saturation of the b fill and cause a water level differential between the opposite sides the wall. This head difference results in a seepage flow through tl backfill and under the wall. The vertical component of the attenda seepage pressures causes a change in effective soil density and a responding change in soil pressures, such that the stability of the is changed under seepage conditions.

THE EFFECT OF SEEPAGE ON THE STABILITY OF SEA WALLS

Instability of sea walls may also develop in a progressive fashion due to extensive scour at the wall face with the resulting decrease in passive earth pressure resistance. Scour may occur due to high water velocity alone, or it may occur at lower water velocity if the effective density of the cohesionless soil is reduced by upward seepage flow. A uniform rate of upward flow in front of the wall results from seepage through the backfill and under the wall. In addition, a transient upward seepage flow is developed near the wall face due to differential water pressures on the sea bed caused by wave action. The combined effect of these two seepage flows increases the probability of scour near the face of the sea wall.

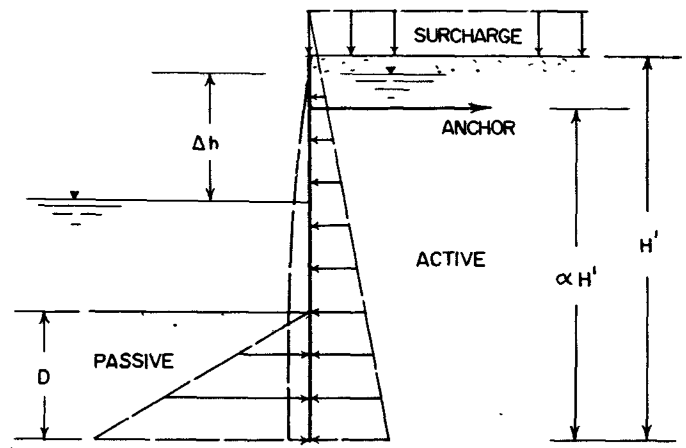
One object of this paper was to determine quantitatively the influence of seepage through the backfill on the factor of safety against wall rotation about the anchor point. The graphical flow net procedure was used to compute the additional loads, caused by seepage flow, which act on the sheet-pile walls. The results of these computations were incorporated into diagrams which permit a rapid computation of these additional wall loads and the resulting changes in the factor of safety against wall rotation. These diagrams include a sufficient range of the variables involved to be useful as design aids.

The second object of this paper was to evaluate the potential effects of seepage on the important problem of scour in front of the wall. It is demonstrated in this paper that an upward seepage gradient at the surface of the soil in front of the wall can be a major factor influencing the potential scour of this zone. Such a vertical gradient is developed when backfill seepage occurs. Furthermore, this steady gradient can be strongly reinforced by a transient gradient developed from wave action in front of the wall. The equations and diagrams presented in this paper permit evaluation of the contribution of these seepage effects to scour at the face of the wall. Small depths of scour cause appreciable changes in the factor of safety against wall rotation about the anchor point.

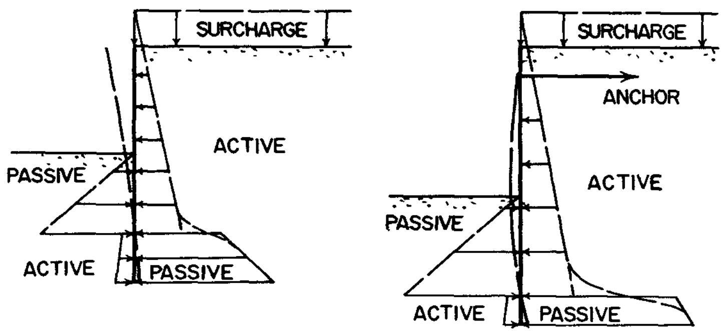
REVIEW OF LOADS ACTING ON VERTICAL SEA WALLS

The use of the classical earth pressure theories permits a simplified evaluation of the magnitude and distribution of active and passive earth pressures along the height of a vertical sheet-pile wall. These pressure distributions are illustrated in Fig. 1 as they occur along cantilever and anchored bulkheads.

The classical pressure distributions have generally been used as the basis for the design of bulkheads and sheet-pile walls although it is known that important modifications of the passive pressure distribution, in particular, may result from wall flexibility. Methods for including



(A) Anchored sea wall with free earth support.



(B) Cantilever sheet-pile sea wall (C) Anchored sea wall with fixed earth support

Fig. 1 - Classical concepts of earth pressure dis-

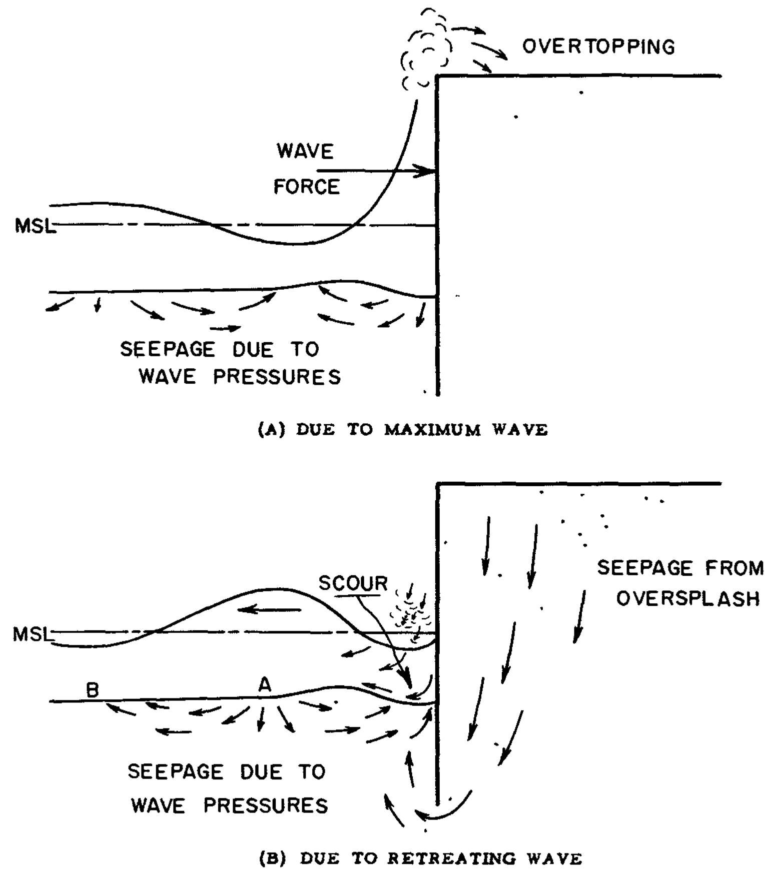


Fig. 2 - Effects of wave on vertical sea wall.
(A) Due to maximum wave
(B) Due to retreating wave

the effect of wall flexibility into design procedures have been presented by Terzaghi (1954), based on the results of model tests by Rowe (1952) and Tschebotarioff (1949).

ACTIVE EARTH PRESSURE

When a wall moves outward relative to the soil mass it confines, the soil mass produces active earth pressure on the wall. For cohesionless backfills bearing against the rear face of sea walls, the active earth pressure, P_A, at any depth, z, in soil of effective unit weight γ is given by the expression:

$$p_A = K_A \gamma z \tag{1}$$

in which K_A is the coefficient of active earth pressure, as computed from Coulomb's Equation (Taylor, 1948).

PASSIVE EARTH PRESSURE

The expression for the limiting passive earth pressure which can be developed at any depth, z, as a wall is moved into cohesionless soil is,

$$p_P = K_p \gamma z \tag{2}$$

in which K_p is computed from the Coulomb equation for passive earth pressure coefficient (Taylor, 1948). Design procedures often are based on the assumption that the angle of wall friction is zero, since this provides a conservative design. Table I gives values of the active and passive coefficients of earth pressure, K_A and K_p, for the conditions corresponding to zero angle of friction between the backfill and a vertical wall, and for which the surface of the backfill is horizontal. Terzaghi (1954) has given values of K_A and K_p, based upon values of angle of wall friction obtained from test data, which may also be used for design purposes.

Passive earth pressure provides direct restraint to the embedded portion of cantilever and anchored sheet pile sea walls, and provides indirect restraint to the upper end of anchored sea walls through the anchor system. Figure 1 shows the zones of soil developing passive resistances to motions of the wall.

TABLE I

Cohesion-less Soil	Angle of Internal Friction ϕ	Coefficient of Active Earth Pressure K_A	Coefficient of Passive Earth Pressure K_p
Dense	38	0.24	4.2
Medium	34	0.28	3.5
Loose	30	0.33	3.0

EFFECT OF STATIC WATER PRESSURE

Whenever static water pressure exists in soil adjacent to a sea wall, it causes an increase of pressure on the wall by an amount of 64 lb. per ft.^{2} for each foot of water depth, and at the same time causes a reduction of the earth pressure on the wall. The active and passive earth pressures are decreased because the submerged unit weight of the material now causes the horizontal soil force on the wall and $\gamma' = \gamma - \gamma_w$, must be used in Eqs. 1 and 2 in place of the total unit weight (γ).

When the backfill is placed hydraulically behind the sea wall, it possible for the total load on the wall to exceed the design load. Unde these conditions, water pressure loads the wall over its entire height addition to the active pressure exerted by the submerged backfill. Su a construction procedure amounts to one type of overload test of the structure and may constitute the greatest static load the wall must sustain.

EFFECTS OF WAVE ACTION

In addition to static loads, a sea wall must resist the attack of waves during storms. Vertical sea walls should not be used at locations subjected to violent, breaking waves because of the large impac forces which may develop, but are often used where moderate wave action may occur. Even moderate wave action contributes dynamic loads directly to a sea wall and the surrounding soil. In addition ter porarily induced water motion in the soil may cause significant chang in earth pressures.

Figure 2 illustrates the factors, described by Bruun (1953) con tributed by wave action, which may have important effects upon the stability of vertical sea walls. Waves acting directly against the wal produce pulsations of horizontal load which are resisted by soil forc developed as the wall moves. Occasional high waves overtop the wal and dump water onto the surface of the backfill. If the backfill material is pervious, this water finds its way back to sea level by percolating through the backfill and under the bottom of the sea wall. Finally, as the wave is reflected from the wall, water rushes down the face of the wall and produces scour of the bottom material near the face of the sea wall. The process of scour near the face of the sea wall is assisted by the upward hydraulic gradient developed in the pervious bottom material by the wave pressures on the bottom and by seepage through the backfill.

Saturation of the backfill can also occur due to direct rain water, or accumulation of rain water runoff, as well as by wave oversplash. Accumulation of water behind the wall causes an increased outward pressure and at the same time reduces passive soil resistance as a result of seepage pressures. A detailed discussion of the methods for evaluating the effects of seepage pressures on the design of sea walls is given in a following section of the paper.

EVALUATION OF SEEPAGE EFFECTS

THE SEEPAGE FLOW NET

The convenient graphical flow net construction for handling seepage problems was developed by Forscheimer (1930) and has been used extensively for many years. The flow net is established by trial sketching. Thus, it is an approximate procedure; but a flow net accurate enough for engineering purposes can be made rapidly after some practice. In addition to the numerical information obtained from such a diagram, the flow net also gives an over-all picture of the flow conditions in the region considered.

The flow net represents a steady state, two dimensional flow condition in which Darcy's Law is assumed valid. It consists of two sets of lines, flow lines and lines of equal total head. If the soil is isotropic with respect to permeability, then these lines everywhere intersect each other at right angles. For sketching and computational convenience, the flow net is generally drawn with a square as the basic element of the net and with an equal rate of flow between any two adjacent flow lines in the net. With the flow net drawn, the rate of seepage flow = q, the hydraulic gradient = i, and the water pressure = p_w, may be computed at any point within the net.

Flow net construction and analysis can be modified to handle more complicated conditions such as cases where the permeability of the soil in the horizontal and vertical directions are quite different, transient flow problems, some three-dimensional flow problems, and flow systems through layers of different permeabilities. Since a detailed discussion of the development and use of flow nets, as well as treatments of

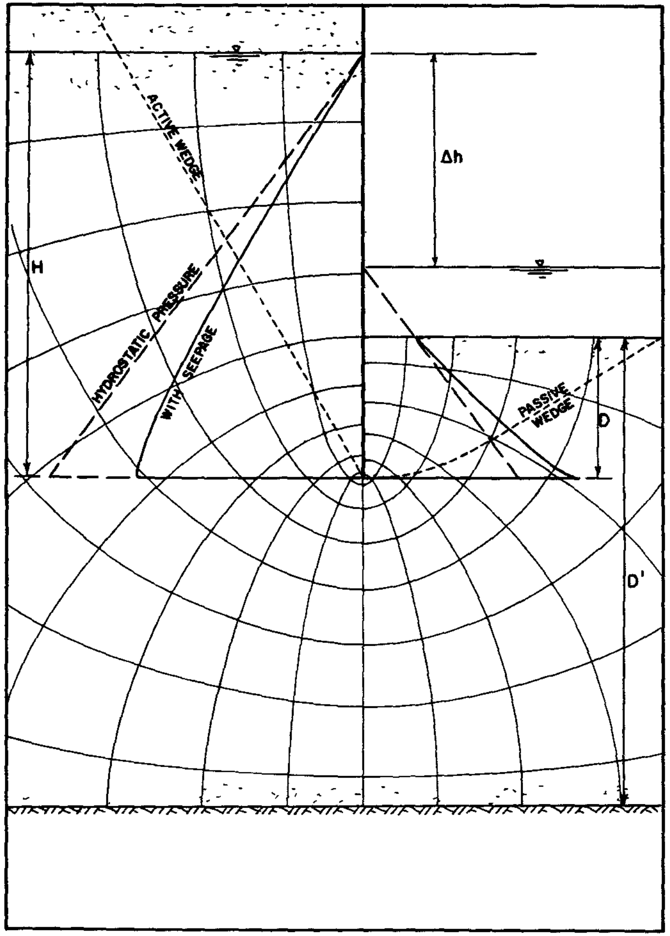
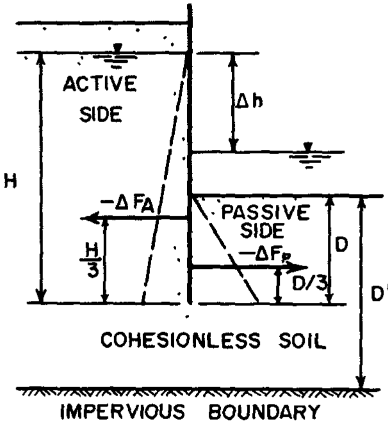


Fig. 3 - Flow net for seepage through the



HOW TO USE THIS FIGURE

1. Compute no-seepage soil and water pressures on active and passive sides of wall by any chosen method.
2. Determine (D/D'). If within 0.1 - 0.7 this figure may be used.
3. Determine (H/D), K_A and K_p values.
4. Use charts below to determine "A" and "P" values.
5. Compute:

$$\Delta F_A/_{\text{unit width}} = -(D\gamma_w \Delta h)A$$
$$\Delta F_p/_{\text{unit width}} = -(H\gamma_w \Delta h)P$$

6. Algebriac sum of above (1) and (5) forces equals the forces on the wall with seepage effects included.

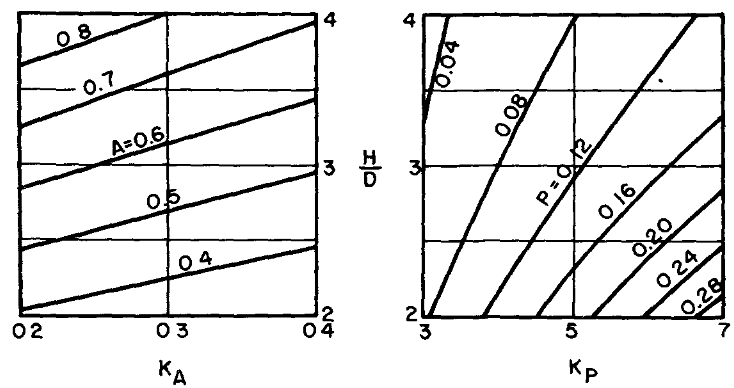


Fig. 4 - Computation of seepage correction forces.

special problems, was given by Casagrande (1937) and by Terzaghi (1943), and is available in recent books on soil mechanics, it will not be repeated here.

EFFECTS OF SEEPAGE ON THE PRESSURE DISTRIBUTION AROUND VERTICAL WALLS

As shown by Eqs. 1 and 2, the active and passive soil pressures are directly related to the unit weight of the soil. The effective unit weight of the soil changes with the development of seepage flow and the associated seepage pressures exerted on the soil. Downward seepage flow behind the wall increases the effective unit weight of the soil and thus increases the active pressure pushing the wall seaward. Upward flow in front of the wall decreases the unit weight of this soil and thus reduces the passive soil resistance to any outward movement of the toe of the wall.

The above effect can be calculated from the formula,

$$\Delta\gamma' = i_v \gamma_w \tag{3}$$

where $\Delta\gamma'$ is the change in the submerged unit weight of the soil,

γ_w is the unit weight of water,

and i_v is the vertical component of the hydraulic gradient.

Thus, in order to obtain the average change in effective unit weight due to seepage flow, it is only necessary to obtain the average vertical component of the hydraulic gradient in either the active or passive failure wedges, and to multiply this by the unit weight of water. Using the flow net of Fig. 3, the average vertical components of hydraulic gradient have been determined for the active and passive earth failure wedges indicated in Fig. 3. These gradients then change the effective unit weight of the soil by the following amounts:

$$\begin{aligned} \Delta\gamma'_{ACTIVE} &= (i_v)_{ACT}\,\gamma_w = +0.21(64) = +13\ \text{lb/ft}^3 \\ \Delta\gamma'_{PASSIVE} &= (i_v)_{PASS}\,\gamma_w = -0.30(64) = -19\ \text{lb/ft}^3 \end{aligned} \tag{4}$$

Since a typical value for the submerged unit weight of a sand is 60 lbs. ft.^{3}, it may be seen that seepage can cause appreciable earth pressure changes in the direction of wall instability.

On the other hand, seepage flow also has an effect which increase the wall stability. The water pressure has a more favorable distributic against the wall when seepage flow exists than the hydrostatic distribution for the same water levels, as shown on Fig. 3. It may be seen thε the effect is one of increased stability of the wall due to reduction of water pressures on the active side, and additional water pressure on tl passive side.

The net effect of the simultaneous two changes in pressure distri bution must be evaluated when studying the effects of seepage on the stability of a wall. Figure 4 was prepared in order to permit a rapid estimate of the change in horizontal forces on a sheet-pile type wall wl the hydrostatic pressure condition is changed by seepage flow under th wall. The seepage force correction method presented in Fig. 4 is bas on the following assumptions:

1. The wall is placed in a homogeneous, isotropic, cohesic less soil which overlies an impervious layer.

2. All changes in soil and water pressures due to seepage effects are assumed to vary linearly with depth, which permits the two pressure change effects to be incorporated into one ΔF computation.

3. The "A" and "P" charts are only valid with wall peneti tion ratios (D/D') between 0.1 and 0.7. Within this range it has been determined that neglecting the individual D/D' ratio involv a maximum error of less than 10%.

Any errors involved in the use of assumptions 2 and 3 are probε minor compared with the potential errors in assumption 1; soils placε by man or nature in horizontal strata are not likely to be homogeneou and isotropic. Therefore, in many instances the use of the seepage force correction procedure suggested herein must be considered as a preliminary computation to determine if the pressure changes due to potential seepage flow are significant in the wall design.

SEEPAGE RESULTING FROM WAVE ACTION

Surface water waves occurring in a finite depth of water produc underwater pressures which can be measured, or may be estimated l use of an appropriate wave theory. The pressure at the sea bottom ı be expressed as,

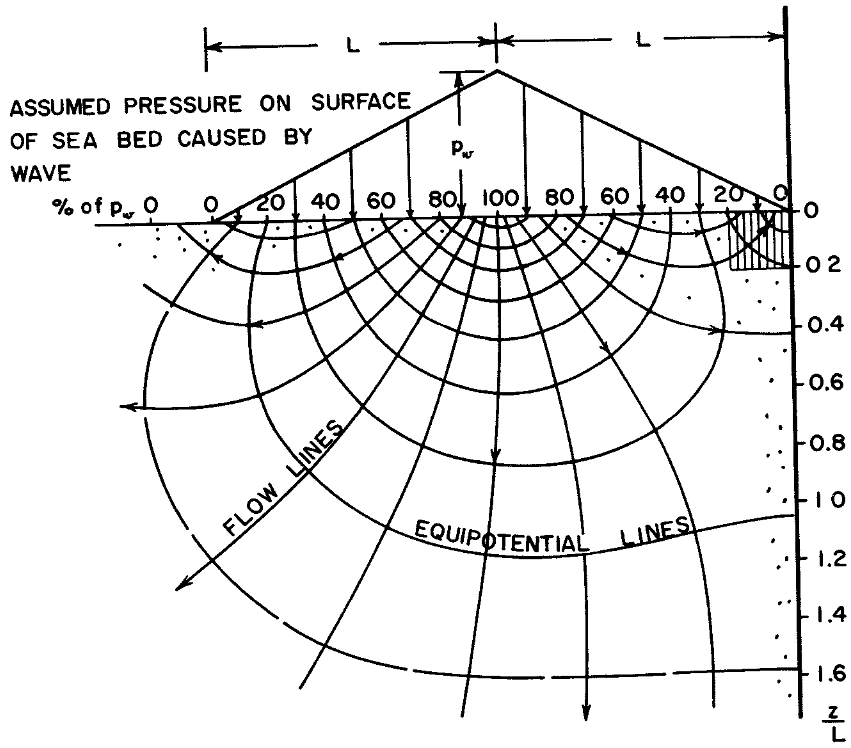
$$p_w = K \gamma_w \,\psi(x,t) \tag{5}$$

where y (x, t) represents the elevation of the wave surface measured from the still water level, γ_w is the density of water, and K represents a "sub-surface pressure response factor."

At any instant of time a difference in pressure exists between two points separated by a distance x along the sea bottom. If the bed material is permeable this pressure difference on its surface will cause seepage flow. In his study of the damping effect on gravity waves contributed by permeable sea bed material, Putnam (1949) considered that this seepage caused by gravity waves is governed by Darcy's law for steady flow. Recently, Reid and Kajiura (1957), also investigated the effect of a permeable sea bed on the damping of gravity waves by treating the problem as a two-layer, coupled system. They included the effects of acceleration of flow in the permeable layer, but found the effects of acceleration to be negligible for practical cases.

In the immediate vicinity of a sea wall the seepage caused by the differential wave pressures along the bottom is considerably affected by the presence of the wall. As the wave runs into a sea wall, the water height at the wall reaches at least two times the unobstructed wave height and produces a corresponding increase in pressure on the bottom. As the water falls along the wall to develop a retreating wave, a trough is formed adjacent to the sea wall. Figure 2 (a) and (b) illustrate the seepage flow in a permeable sea bed resulting from pressures developed by these two conditions of wave motion at the sea wall face. The impermeable boundary formed by penetration of the sea wall into the permeable material will force seepage flow to become vertical at the wall face as indicated in Fig. 2 (b). For the wave position as indicated in Fig. 2 (b), the upward seepage forces near the wall face due to the wave will reinforce the upward seepage forces developed from oversplash. At the point A, the wave seepage forces are down and in this region they will tend to counteract the oversplash effects.

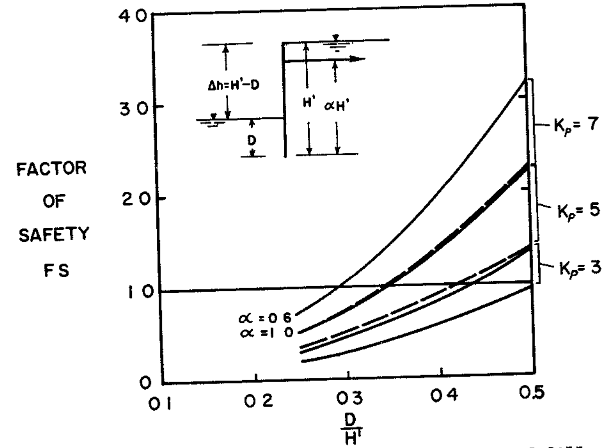
In order to evaluate the importance of the seepage due to wave action near a sea wall, a triangular distribution of pressure on the sea bottom was assumed to represent the transient pressure beneath a retreating wave. By considering this pressure distribution as static at a particular instant of time, the steady state seepage flow was established by means of the "Relaxation" procedure, and the flow net shown in Fig. 5 was obtained. The hydraulic gradient which causes upward flow at the face of the sea wall depends directly upon the hydraulic gradient along the sea bottom, $p_w/\gamma_w L$. The maximum value of hydraulic gradient at the face of the sea wall and at the sand surface is, $i_{max} \approx 1.6\, p_w/\gamma_w L$, and the average value over an area 0.2L deep and extending 0.2L from the wall face is, $i_{ave} \approx 0.66\, p_w/\gamma_w L$. The values of hydraulic gradient at other points beneath the wave can be obtained from Fig. 5.



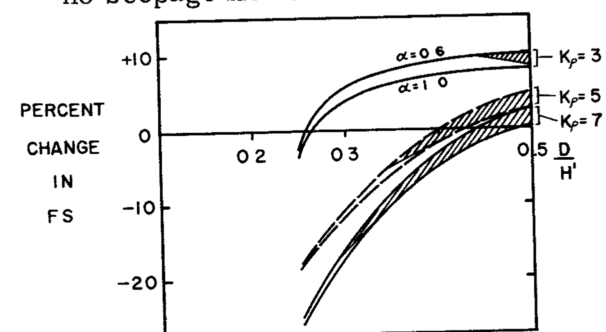
MAXIMUM EXIT HYDRAULIC GRADIENT – AT WALL FACE, $i_{max} = 1.6 \frac{p_w}{\gamma_w L}$

AVERAGE EXIT HYDRAULIC GRADIENT – SHADED AREA, $i_{av} = 0.66 \frac{p_w}{\gamma_w L}$

Fig. 5 - Flow net in sea bed caused by transient wave pres-



(A) Factor of safety - saturated backfill and no seepage flow.



(B) Percent change in factor of safety due to seepage flow.

Fig. 6 - Change in factor of safety of sheet-pile wall - due to seepage.

Upward forces near the sea wall face resulting from wave seepage pressures reduce the effective unit weight of the sea bed material by an amount equal to $i_v \gamma_w$. Figure 5 permits an evaluation of the hydraulic gradient in the sea bed in terms of the wave pressure gradient on the sea bottom, resulting from any given surface wave shape. Thus an estimate can be obtained of the contribution toward scour of the sea bed material which is produced by wave seepage pressures.

The effects of acceleration of flow due to the time rate of change of pressure distribution on the surface of the sea bed were neglected in this study. However, it might be anticipated that the upward hydraulic gradients near the sea wall face would be increased somewhat by accelerative flow.

CHANGES IN WALL STABILITY DUE TO SEEPAGE

In order to evaluate the effect of seepage on the stability of a vertical sheet-pile type sea wall, consider an anchored wall with free earth support as illustrated in Fig. 1 (a). The factor of safety with regard to rotation of the wall as a rigid body about the anchor point will be used as the criterion for evaluating the stability of the wall.

In order to simplify the equations for the factor of safety, it was assumed that the submerged unit weight of the soil, γ', is equal to the unit weight of water, γ_w. The factor of safety with and without seepage flow was determined for a head difference, Δh, as shown on Fig. 1 (a), assumed to be equal to its maximum value of H' - D. For the condition of hydrostatic pressure of soil and water acting on the wall, the factor of safety against rotation about the anchor point is,

$$F.S. = \frac{(D/H')^2 K_P (3\alpha - D/H')}{(1 + K_A)(3\alpha - 1) - (D/H')^2 (3\alpha - D/H')} \qquad (6)$$

in which D is the depth of pile penetration, H' is the total pile length, and α H' represents the distance from the tip of the pile to the anchor point.

Equation 6 involves three geometrical variables, D, H', and α, and two quantities, K_A and K_p, which depend primarily upon the angle of internal friction of the cohesionless soil. The coefficient of active earth pressure, K_A, has a value of 0.3 for loose, clean sand, and a slightly lower value for sand in a more dense condition. In order to reduce the number of variables involved, $K_A = 0.3$ was used in Eq. 6. Then a diagram was prepared using the dimensionless ratio D/H' as abscissa and factor of safety, F.S., as ordinate. Values of α of 0.6 and 1.0 and K_p of 3, 5, and 7 were used as parameters to prepare the families of curves shown on Fig. 6 (a). Thus for a wall having particu lar geometrical ratios D/H' and α, the factor of safety will depend up‹ the allowable value of the coefficient of passive earth pressure, K_p. B definition, a factor of safety of 1.0 or greater is required for stability of the wall, consequently the curves shown on Fig. 6 (a) which extend below F.S. = 1.0 represent unstable conditions.

When seepage occurs, still with Δh maintained as the maximum value of H' - D, a change in the factor of safety of the wall occurs, or the factor of safety under seepage conditions is given as

$$F.S. = \frac{(3\alpha - D/H')\left[(D/H')^2 K_P - 2P(1 - D/H')\right]}{(3\alpha - 1)\left[1 + K_A - 2A\left\{\frac{D}{H'} - \left(\frac{D}{H'}\right)^2\right\}\right] - \left(\frac{D}{H'}\right)^2\left(3\alpha - \frac{D}{H'}\right)} \qquad (7)$$

Then the change in factor of safety as a result of the seepage flow can be determined from,

$$\Delta F.S. = \frac{\text{Eq. 6} - \text{Eq. 7}}{\text{Eq. 6}} \qquad (8)$$

Figure 6 (b) shows the percent change in factor of safety due to seepa flow. The shaded portions of the diagrams represent the portion of practical significance, for which the wall is stable under conditions o complete backfill saturation and no seepage flow. When seepage flow occurs, the factor of safety may be decreased or increased, however the effect is generally less than 10 percent different from that for the flow condition.

From this simplified treatment of the stability of sheet-pile typ sea walls it is evident that if such a wall is designed to withstand full water pressure difference and retain an adequate factor of safety und these conditions, that the additional effects of seepage flow will prod only small changes in the factor of safety.

EFFECTS OF CHANGES IN D/H'

From Fig. 6 (a) it is seen that the factor of safety depends ma edly upon the ratio D/H', when α and K_p are maintained constant. I order to study the relative importance of seepage compared to chang of D/H' as would occur as a result of decreasing D by scour, a spec example was chosen.

Figure 7 (a) shows the dimensions and soil characteristics chosen to represent approximately a sheet-pile type sea wall which failed during a mild storm. When the backfill is completely saturated, and the water level on the outside of the wall is just at the sand surface, the unbalanced water head, Δh, is 8.5'. For this condition and for no seepage flow, the factor of safety against rotation about the anchor point is 1.62. When seepage flow occurs the factor of safety is reduced to 1.57, or a reduction of about 3 percent. Consequently, the effect of seepage flow alone is insignificant.

When the depth of embedment, D, is varied, the effect on the factor of safety is as shown in Fig. 7 (b). The wall becomes unstable when D is reduced just slightly more than one foot, for the condition including seepage flow, and for slightly less than 1.5' for no flow. This magnitude of scour has been observed at the face of sea walls that failed, and it is probable that backfill saturation and toe scour were important factors in the failures.

The horizontal forces developed during seepage flow thus appear to be of small importance compared to changes in depth of embedment as each contributes to a reduction in the factor of safety of sheet-pile type sea walls with free earth support.

EFFECT OF SEEPAGE ON SCOUR AT WALL FACE

HYDRAULIC GRADIENTS AND SCOUR VELOCITY

The preceding section has illustrated the importance of relatively small reductions in the depth of wall embedment upon the factor of safety. The depth of material which restrains the toe of the wall against outward motion is reduced when scour occurs in this region; scour is particularly important when it occurs adjacent to the wall face. Thus it becomes necessary to estimate the effects contributed by seepage flow toward increasing the probability of scour at the face of the wall.

From laboratory studies, such as those presented by Ippen and Verma (1953), it has been shown that scour is a complex phenomena, even in a controlled laboratory flume. The scouring action of waves reflected from a vertical wall, with appreciable air and soil contained in the turbulent water represents an even more complex problem. However, the contribution of vertical seepage toward increased scour can be estimated by considering only its effect on the unit weight of the soil particles.

Figure 8 shows a surface cohesionless soil particle being subjected to potential scour by water moving across the particle with velocity = V. The forces acting on this particle are as follows:

F_D = drag force = $B\ V^{3/2} \approx B_1 V^2$ (Ippen and Verma, 1953)

F_L = lift force = $C\ V^2$ (Ippen and Verma, 1953)

W' = effective weight of the particle

F_f = maximum friction force = $(W' - F_L) \tan \phi$; where ϕ is the friction angle and includes both the true friction and particle interlocking effects.

An upward hydraulic gradient, i_v, through the soil bed on which the soil particle is resting reduces the effective weight of the soil particle thus reducing the friction force.

The maximum non-scour velocity, V, is attained when $F_D = F_f$, and $i_v = 0$, and may be expressed as:

$$B_1 V^2 = (W'_{i_v=0} - CV^2) \tan \phi \tag{9}$$

When upward seepage is occurring, ($i_v \neq 0$), W' is reduced and the maximum non-scour velocity is reduced to V'. Equation 9 then becomes:

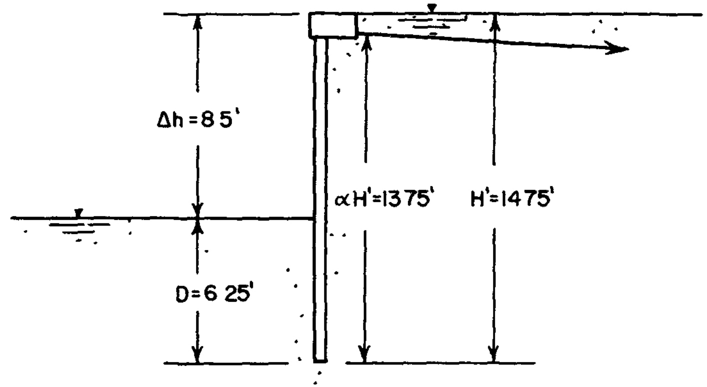
$$B_1 V'^2 = (W'_{i_v \neq 0} - CV'^2) \tan \phi \tag{10}$$

Dividing equation 10 by 9, and simplifying, gives:

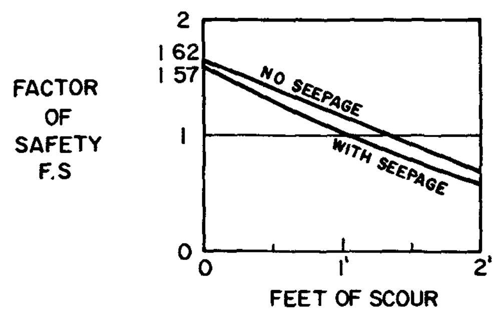
$$\frac{V'}{V} = \left[\frac{W'_{i_v \neq 0}}{W'_{i_v = 0}}\right]^{\frac{1}{2}} \tag{11}$$

The ratio V'/V represents the factor by which the horizontal velocity has to be reduced to prevent scour after an upward hydraulic gradien has developed. This ratio will be called the "scour velocity reductio factor" and be given the symbol R.

Any percent reduction in the effective weight of each soil partic results in a similar reduction in the intergranular pressures within t



(A) Dimensions of example sea wall (saturated backfill, and $K_A = 0.3$, $K_p = 7$)



(B) Variation of F. S. with seepage and scour

Fig. 7 - Example of the effect of seepage and scour on the stability of a sheet-pile type sea wall.

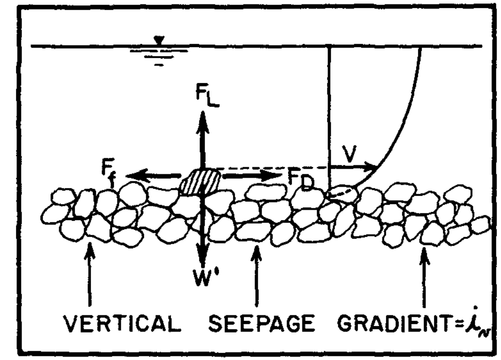


Fig. 8 - Forces on sand particle subjected to scouring action.

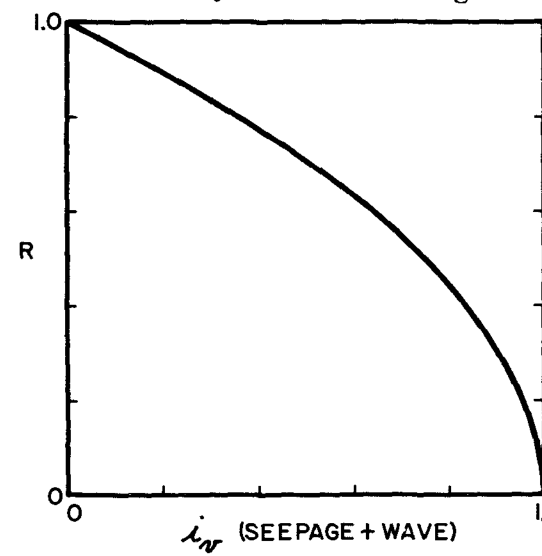


Fig. 9 - Scour velocity reduction factor vs. vertical hydraulic gradient.

soil mass. An expression for ratio of vertical intergranular pressure, $\bar{p}$, and therefore also of individual particle weight, W', at any depth z in a submerged soil mass, before and after upward seepage flow is:

$$\frac{\bar{p}_{i_v \neq 0}}{\bar{p}_{i_v = 0}} = \frac{W'_{i_v \neq 0}}{W'_{i_v = 0}} = \frac{z\left[\left(\frac{G-1}{1+e}\right)\gamma_w - i_v \gamma_w\right]}{z\left(\frac{G-1}{1+e}\right)\gamma_w} = \left[1 - i_v\left(\frac{1+e}{G-1}\right)\right] \quad (12)$$

The specific gravity values, G, of the three most common sand grain minerals are quartz = 2.66, calcite = 2.72, and feldspar = 2.56. A common range of void ratio values, e, for uniform sand is 0.55 to 0.7 Therefore, unless G and e are materially different from the above, equation 12 may be written with sufficient accuracy as:

$$\frac{W'_{i_v \neq 0}}{W'_{i_v = 0}} \approx (1 - i_v) \quad (13)$$

From equations 11 and 12, one can obtain:

$$\frac{V'}{V} = R = \left[1 - i_v\left(\frac{1+e}{G-1}\right)\right]^{\frac{1}{2}} \quad (14)$$

Or, from equations 11 and 13 one can obtain the more approximate fo

$$\frac{V'}{V} = R = (1 - i_v)^{\frac{1}{2}} \quad (15)$$

Equation 15 has been used to prepare a graph of the relationsh between the scour velocity reduction factor, R, plotted against the ve tical hydraulic gradient, i_v. This graph is presented in Fig. 9. At instant of time, i_v represents the summation of the gradients due to steady seepage from the backfill and the transient seepage condition to wave action.

EVALUATION OF EXIT GRADIENT DUE TO BACKFILL SEEPAGE

The flow nets constructed for the purpose of evaluating the ho zontal forces on the wall due to seepage also provide values of the e hydraulic gradient at the soil surface. For this study, flow nets we constructed for a D/H range from 0.25 to 0.50, and for a D/D' ran of 0.1 to 0.7. By expressing the exit gradient as

$$i_e = S\left(\frac{\Delta h}{D}\right) \tag{16}$$

the extreme values of S were found to be 0.21 and 0.28 for values of the parameters D/D' = 0.7, D/H = 0.25, and D/D' = 0.1, D/H = 0.5, respectively. Values of "S" obtained from data given by McNamee (1949) for the same range of geometrical parameters were found to be 0.22 and 0.30, respectively. In comparing the results of eleven flow nets with McNamee's results, the deviations in individual values of exit hydraulic gradient, i_e, varied from zero to 7 percent. This indicates that the graphical flow nets were uniformly accurate.

Since the values of S were found to have a fairly small variation, it is suggested that an average value of 0.25 be used in Eq. 16, or that the exit hydraulic gradient caused by seepage through the backfill be taken as,

$$i_e = \frac{1}{4}\left(\frac{\Delta h}{D}\right) \tag{17}$$

The use of S = 1/4 involves a maximum error of 17% for the cases studied, but this error is no doubt much smaller than those arising from the differences between actual soil conditions and the homogeneous, isotropic, conditions assumed as a basis for the flow net construction. The use of S = 1/4 is a slight refinement of the upper limiting value of S = 1/3 which was suggested by Terzaghi (1954).

EXAMPLE

The example of Fig. 7 (a) is here continued to investigate the importance of seepage gradients on potential scour in front of this wall. Consider the case of this wall during a storm, when rainfall and oversplash have completely saturated the backfill and waves are striking and being reflected by the wall.

For the maximum value of Δh of H' - D used with Eq. 17, the exit hydraulic gradient due to seepage through the backfill for the example, Fig. 7 (a), is

$$i_e = \frac{H'-D}{4D} = 0.34 \tag{18}$$

In addition to the steady seepage, wave pressures on the surface of the sea bed produce a maximum value of vertical hydraulic gradient at the face of the wall of,

$$i_{max} = 1.6 \frac{p_w}{\gamma_w L} \tag{19}$$

and an average value over a distance 0.2L deep and 0.2L away from tl wall face, of,

$$i_{ave} = 0.66 \frac{p_w}{\gamma_w L} \tag{20}$$

For a value of $p_w/\gamma_w L = 0.4$, the maximum and average value of vertical hydraulic gradient due to wave pressures are,

$$\begin{aligned} i_{max} &= 0.64 \\ i_{ave} &= 0.26 \end{aligned} \tag{21}$$

These temporary gradients are added to the steady value due to backf seepage to give

$$\begin{aligned} (i_v)_{TOTAL_{MAX}} &= 0.98 \\ (i_v)_{TOTAL_{AVE}} &= 0.60 \end{aligned} \tag{22}$$

When entering Fig. 9 with the above results, it may be seen tha the scour velocity reduction factor, R, is almost zero for the maximı gradient immediately adjacent to the wall, and is about 0.6 for the av age condition over the 0.2L distance from the wall. Thus, the backfi seepage and wave conditions assumed in this example are certain to result in scour immediately adjacent to the wall, and there is a great increased likelihood of scour for a distance of at least 0.2L from the wall.

METHODS OF ELIMINATING OR MINIMIZING SCOUR

Scour at the face of a vertical sheet-pile type sea wall may cau a marked reduction in factor of safety of the wall, as was demonstra in the study of the example shown on Fig. 7. Therefore, precaution should be taken to minimize the possibilities of scour at the most cri cal regions.

Since upward seepage forces contribute to the probability of sc methods of preventing or controlling seepage flow through the backfil should be incorporated into the design of the wall. Paving the surfac of the backfill is one obvious method of preventing water from enteri

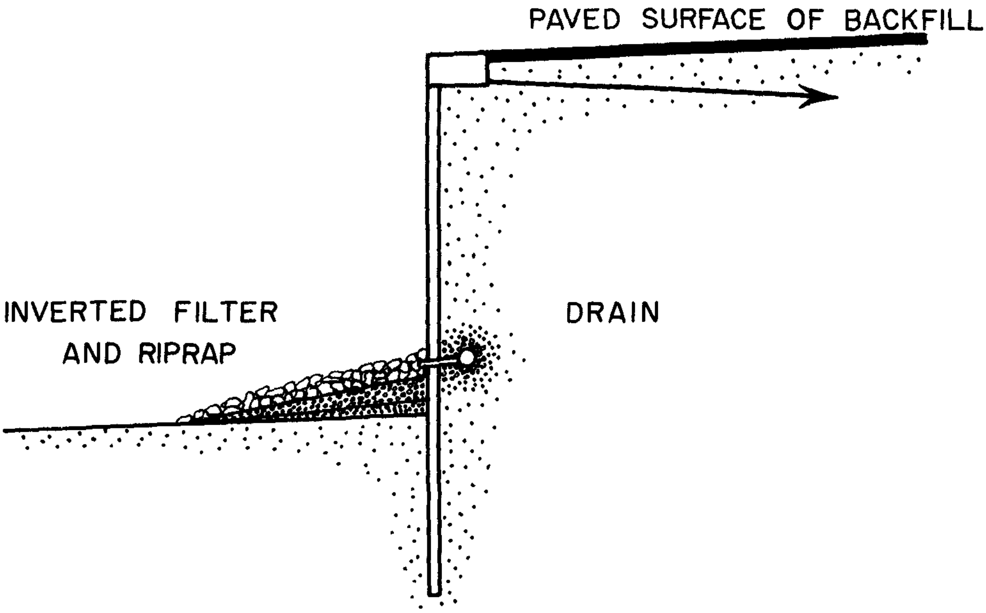


Fig. 10 - Methods of minimizing effects of seepage.

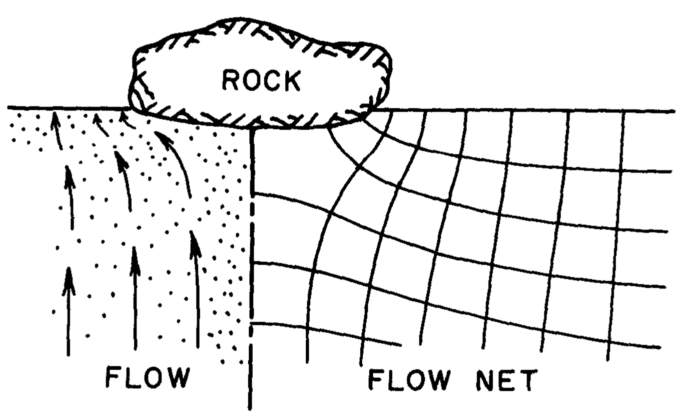


Fig. 11 - Effect of impermeable body on vertical seepage flow.

the backfill by oversplash or rain run-off, and has been found effective in increasing the wall stability. Drains are also desirable to intercept water which leaks through cracks in the paving, or to equalize the unbalanced water pressure developed during a rapid drawdown of the me sea level. By intercepting the water flow, the drains prevent flow beneath the toe of the wall and do not allow vertical hydraulic gradients t develop in the zone of passive soil pressure. Figure 10 illustrates an effective location for a drain system; the drain is low enough that larg unbalanced water heads cannot develop.

Upward seepage forces on the sea bed can be counteracted by pr viding static weight near the wall face. Riprap or large rocks must b used as cover for this loaded region to maintain the load at the proper location, even during severe action by longshore currents and breakir waves. Criteria established for breakwater design, such as those given by Hedar (1953), may be used to select the proper size of cover material. However, it is not satisfactory to place large rocks direct upon beach material, since the finer material may be carried through the voids in the larger material by water flow.

Figure 11 illustrates the disturbance of vertical flow by an impermeable object resting on the surface of the permeable bed. The flow of water must detour around the obstacle, thereby crowding the flow lines together and increasing the hydraulic gradient near the boundary of the rock. In this region the effective weight of the bed material has been reduced by the upward flow. Erosion will occur beneath the edges of the rock as a result of horizontal water velocities caused by waves acting on the bed material. Progressive undermini of the edges of the rock will eventually cause it to sink into the sand and become ineffective as scour protection.

More than forty years ago it was recognized that a permeable surcharge placed over the region of upward seepage flow would prev the occurrence of "piping" beneath structures due to seepage flow through permeable foundations. Terzaghi has made extensive use of such a permeable surcharge constructed in the form of an "inverted filter," in which the layers of cohesionless materials increased in grain size toward the top. He also established relations for the gra size variations of successive layers. These rules for grain size va ations were later modified slightly as a result of extensive tests by Waterways Experiment Station, Vicksburg, Mississippi, and the rec mended design procedures are summarized in the paper by Posey (1

Filters should also be placed around collector pipes in the bac fill drain system to prevent the backfill material from flowing out through the drains. A saturated cohesionless backfill will flow reac through relatively small holes or cracks in a retaining wall and can cause a cave-in of the backfill surface. Several cave-in type failures have been reported by Gebhard (1949) which were caused by sand flowing through small holes or cracks in the structure. When a cave-in type failure of a sea wall backfill occurs during a storm, the soil resistance to wave forces is eliminated at this location and the wall is knocked over, landward, by repeated wave impacts. Thus the stability of the sea wall against this type of failure depends upon maintaining continuity of the backfill.

CONCLUSIONS

The effects of seepage flow on the stability of vertical sheet-pile walls were considered in this study. Stability was evaluated in terms of the factor of safety against rigid body rotation of the wall about the point of anchor attachment.

Seepage flow through the backfill and under the wall causes horizontal forces on the wall as a result of the changes in water and soil pressure distributions from those corresponding to the hydrostatic condition. The net effect on the factor of safety produced by these changes in water and soil pressures was found to be unimportant for the cases studied.

A study of the importance of the geometrical parameters of a sea wall demonstrated that the factor of safety changes significantly with small changes in the embedded depth of the wall. Removal of material at the outer face of the wall by scour changes the embedded length of the sea wall and in this way scour may change the factor of safety of the wall appreciably.

One effect of seepage through the backfill is to reduce the effective density of the cohesionless material in front of the sea wall. This steady state reduction in soil density is reinforced by a transient effect resulting from seepage flow induced by pressure gradients developed along the sea bottom by water waves. A diagram is included which illustrates the relation between the exit hydraulic gradient due to seepage flow, which determines the effective soil density, and the reduction in value of horizontal water velocity required to produce scour.

A brief discussion is also included of methods for reducing or eliminating scour of material at the face of the sea wall.

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APPENDIX

LIST OF SYMBOLS

A = coefficient used to calculate ΔF_A
B = constant
C = constant
D = depth of penetration of sheet pile wall (ft.)
D' = depth from dredge level to first impervious soil layer (ft.)
e = void ratio of soil
F_D = drag force due to water flowing across part surface soil particle (lbs.)
F_f = friction force resisting scour movement of soil particle (lbs.)
F_L = lift force due to water flowing across part surface soil particle (lbs.)
F.S. = factor of safety
ΔF_A = net change in force on active side of wall, due to backfill seepage effects (lbs.)
ΔF_P = net change in force on passive side of wall, due to backfill seepage effects (lbs)
G = specific gravity of soil solids
H = height from tip of wall to water level in backfill, but not to exceed H' (ft.)
H' = total height of wall backfill, from embedded tip of sheet piles (ft.)
Δh = height difference between water level behind and in front of wall (ft.)
i = hydraulic gradient (ft. /ft.)
i_e = vertical exit gradient, immediately seaward of wall, due to backfill seepage
i_v = vertical component of hydraulic gradient
K = sub-surface pressure response factor
K_A = coefficient of active earth pressure
K_P = coefficient of passive earth pressure
L = assumed distance between points of maximum and 0 wave pressure on a horizontal sea bed (f)
P = coefficient used to calculate ΔF_P
p_A = active earth pressure (lbs. /ft.^{2})
p_P = passive earth pressure (lbs /ft.^{2})
p_w = water pressure (lbs. /ft.^{2})
q = rate of seepage flow (ft.^{2}/sec.)
R = scour velocity reduction factor V'/V
S = coefficient used to determine i_e
t = time
V = maximum water velocity across top of soil particle, without scour movement of particle (ft. /sec.)
V' = value of V when upward seepage is occurring (ft./sec)
W' = effective submerged weight of individual soil particle (lbs.)
x = distance along the sea bottom (ft.)
z = depth from soil surface (ft.)
α = ratio of height to anchor point/total backfill height
γ = unit weight of soil (lbs. /ft.^{3})
γ' = submerged unit weight of soil (lbs. /ft.^{3})
γ_w = unit weight of water (lbs. /ft.^{3})
ϕ = angle of internal friction of soil