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Technical
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Engineering and Design
STABILITY OF GRAVITY WALLS
VERTICAL SHEAR

1. Purpose

This engineer technical letter (ETL) provides guidance for incorporating and calculating a shear force acting along the backs of gravity earth retaining walls within the procedures for analyzing the stability of navigation structures, including lock walls, and approach walls. This shear force is also referred to as a "downdrag" force or "drag" force. The simplified procedure described in this ETL for calculating the magnitude of shear force is restricted to concrete gravity earth retaining walls founded on rock.

2. Applicability

This ETL applies to all HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities having responsibilities for the design and construction of civil works projects.

3. References

References are included in Appendix A.

4. Background

a. Calculation of the stability of gravity walls.

(1) A common procedure used for designing new gravity walls and for evaluating the safety of existing gravity walls is the conventional equilibrium method of analysis. The conventional equilibrium method involves assumptions regarding the loading and resisting forces that act on the structures. In most cases of massive retaining walls constructed on rock foundations, movements of the wall and backfill are not sufficient to fully mobilize the shear resistance of the soil.

(2) Past practice has been to assign at-rest lateral earth pressures against the back of the gravity wall and set the interface friction between the wall and the backfill equal to zero. Zero interface friction along the back of the wall corresponds to a zero shear force along the back of the wall. In addition, boundary water pressures were assigned along the back, front, and base of the wall for navigation structures. With all forces and their points of action on the free body diagram of the wall defined, wall stability was checked against the recommended criteria (EM 1110-2-1605, EM 1110-2-2502, EM 1110-2-2602, ETL 1110-2-22, ETL 1110-2-256, ETL 1110-2-310).

b. Experience with existing gravity walls. Past practice has been to use the same stability criteria for designing new gravity earth retaining structures and for reviewing the margin of safety available for existing structures. Several existing structures, although not meeting the referenced stability criteria (EM 1110-2-1605, EM 1110-2-2502, EM 1110-2-2602, ETL 1110-2-22, ETL 1110-2-256, ETL 1110-2-310), have performed satisfactorily for many years. It may not be necessary to improve a structure’s stability to satisfy the referenced criteria when the remaining life of the structure is relatively short or when there are no indications of any stability problem. A research investigation, performed as part of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Program (Ebeling, Duncan, and Clough 1990; Filz and Duncan 1992), was undertaken to study the stability of existing concrete structures. The results of the initial REMR research, and the experience from subsequent research programs (Ebeling, Duncan, and Clough 1990; Filz and Duncan 1992), are described in Sections 4*c* and 4*d* of this ETL.

c. Shear force - finite element analyses.

(1) To develop an improved understanding of the interaction between gravity walls, their foundations, and their backfills, an investigation using finite element analyses was conducted (Ebeling et al. 1992; Ebeling, Duncan, and Clough 1990). The analyses demonstrated that the backfill settles relative to the wall and develops downward shear loads on the wall. Some examples are given in Figure 1, which shows the results of finite element analyses of four walls founded on rock and retaining dry backfill. In Figure 1, the magnitude of the vertical shear force F_v is expressed in terms of a vertical shear coefficient K_v, which is related to the shear force on the vertical plane through the heel of a wall by the following equation:

$$F_v = K_v \cdot \frac{1}{2} \gamma_t H^2 \qquad (1)$$

where

F_v = shear force on the vertical plane through the heel of the wall (force per unit length of wall)

K_v = vertical shear coefficient (dimensionless)

γ_t = total unit weight of backfill

H = wall height

(2) Analyses indicated that the gravity walls would move only a very small amount during placement of the toe fills and backfills. As a result, the earth pressures on the backs and the fronts of the walls are close to those that exist at rest. Even so, settlement of the backfill relative to the wall as it is placed behind the wall is sufficient to generate a significant amount of shear force on the wall. Values of K_v range from 0.09 to 0.21 for the four cases shown in Figure 1.

(3) Parametric studies demonstrated that the most important factors influencing the value of K_v for concrete gravity walls on rock foundations are the depth of the backfill, the stiffness of the backfill, the inclination of the back of the wall, and the number of steps in the back of the wall. The following trends were observed:

(a) For low walls, the value of K_v increases with increasing wall height because more backfill compression occurs due to self-weight of the backfill. The resulting increase in differential movement between

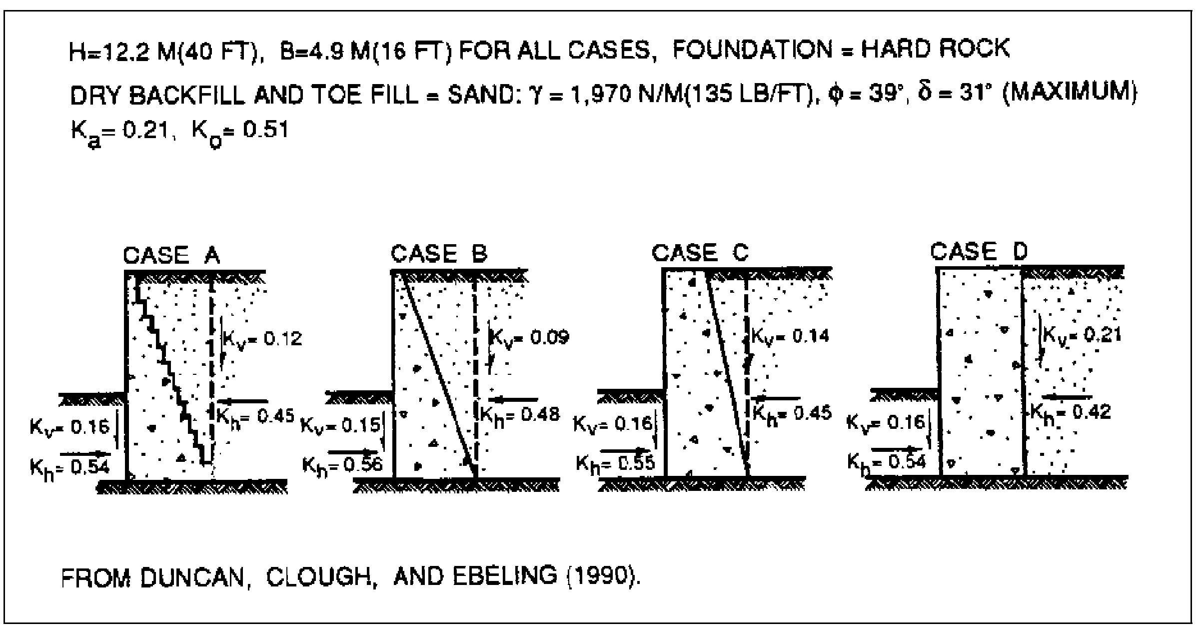


Figure 1. Results of finite element analyses of four walls founded on rock retaining dry backfill

backfill and wall causes a greater portion of the interface strength to be mobilized. This process approaches a limiting condition for high walls as the interface strength becomes fully mobilized over most of the wall-backfill contact area.

(b) As the stiffness of the backfill increases, backfill compression decreases, and the wall height necessary to mobilize the full interface strength increases. For low walls with vertical back sides, the value of K_v decreases as the backfill stiffness increases.

(c) The value of K_v decreases as the back side of the wall becomes inclined away from the backfill and towards the front of the wall.

(d) The value of K_v is greater for a wall with a stepped back side than for a wall with a smooth back side at the same average slope.

d. Shear force - instrumented field and model wall measurements.

(1) Shear loads have been reported for several instrumented walls (Duncan, Clough, and Ebeling 1990), including a lock wall 30.2 m in height and founded on rock (Hilmer 1986). Measurements at the lock wall are reported over a 6-year period. Mobilized interface friction at the lock wall fluctuates seasonally and with changes in water level inside the locks. However, the data indicate that the shear force is persistent over the 6-year period, and does not decay with time. According to a conservative interpretation of the data, the minimum value of K_v during the 6-year period is about 0.18.

(2) In a recent research program conducted at Virginia Polytechnic Institute and State University (Filz and Duncan 1992), both the horizontal earth pressure force and the vertical shear force along the vertical back side of a 2.1-m- (7-ft-) high rigid retaining wall were measured. The research program included 16 tests using compacted fine sand (Unified Soil Classification SP) and compacted non-plastic silty sand (SM) as backfill. Measured values of K_v ranged from 0.11 to 0.23 (Table 8.9 in Filz and Duncan (1992)). The more compressible backfills exhibited higher K_v values. The compacted backfills were left in place for periods ranging from 1 to 14 days after completion of backfilling. Values of K_v tended to increase with time.

5. Procedures for Calculating the Vertical Shear Force

Two procedures for computing the magnitudes of shear loads along the backs of gravity walls are described in this section: a simplified procedure, and a complete soil-structure interaction analysis procedure using the finite element method. These procedures are intended only as guidelines and are not intended to replace judgment by the engineers responsible for the project.

a. Simplified procedure.

(1) Inclusion of a shear force on a vertical plane through the heel of the wall, as shown in Figure 2, can be computed by use of the following equation:

$$F_v = K_v \cdot \left[\frac{1}{2} \gamma_t (D_1)^2 + \gamma_t (D_1 D_2) + \frac{1}{2} \gamma_b (D_2)^2 \right] \qquad (2)$$

where

D_1 = thickness of backfill above the hydrostatic water table

D_2 = thickness of submerged backfill above the base of the wall

γ_b = buoyant unit weight of submerged backfill, $\gamma_t - \gamma_w$

γ_w = unit weight of water

(2) As indicated in Figure 2, the total height of the backfill against the wall is the sum of the thicknesses D_1 and D_2:

$$H = D_1 + D_2 \qquad (3)$$

(3) Equation 2 requires a value for K_v. In the simplified procedure, the value of K_v is obtained from Figures 3 through 5 and Equation 4:

$$K_v = (1 - C_\theta C_s) \cdot (K_v)_{vert} \qquad (4)$$

where

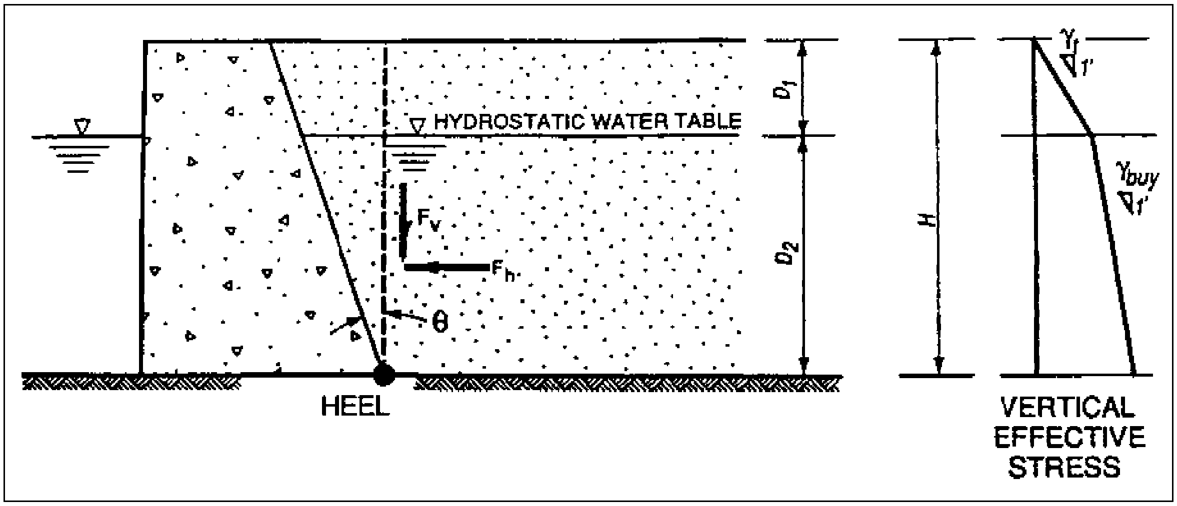


Figure 2. Resultant earth pressure forces on a vertical plane through the heel of a gravity wall founded on rock

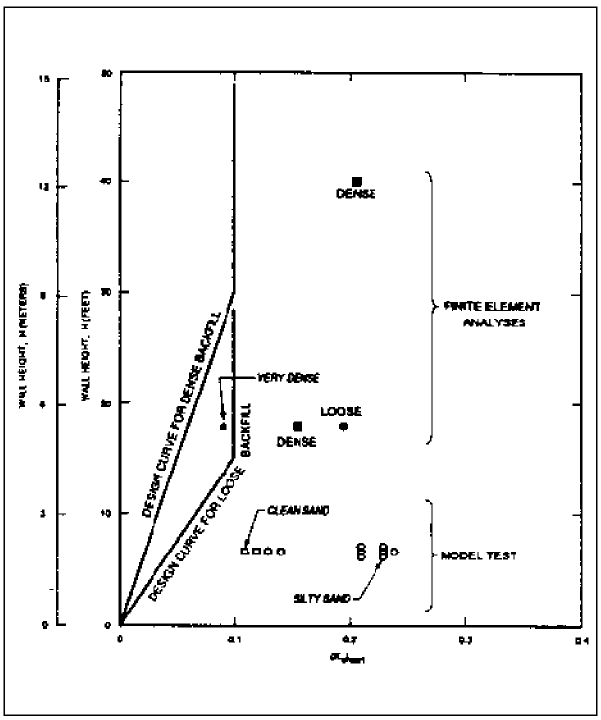


Figure 3. Values of $(K_v)_{vert}$ for design of gravity walls founded on rock

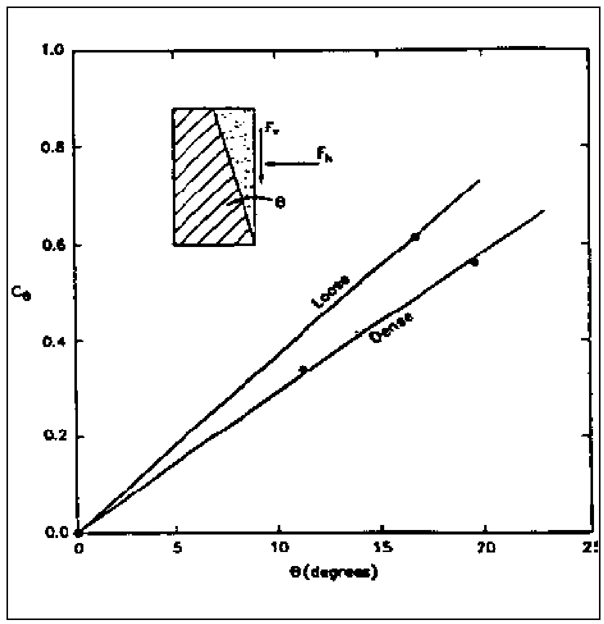


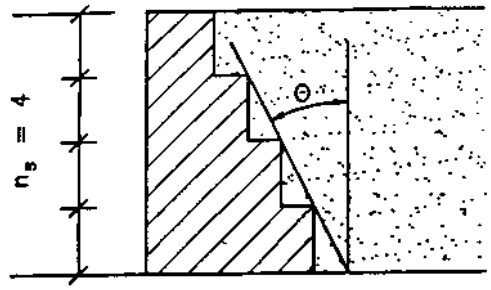
Figure 4. Values of the correction factor C_θ for inclination of the back side of gravity walls founded on rock

C_θ = correction factor to account for inclination of the back side of the wall

C_s = correction factor to account for steps in the back side of the wall

$(K_v)_{vert}$ = value of K_v for a wall with a vertical back side

(4) Figure 3 shows that the value of $(K_v)_{vert}$ for design increases with increasing wall height until a limiting value of 0.1 is reached. This limiting value for design is well below the actual limiting value of $(K_v)_{vert}$ indicated by measurements and analyses. It was selected conservatively so that the change to previous design procedures (i.e., K_v = 0) would not be large. Even with this conservative selection of the limiting value of $(K_v)_{vert}$, significant economies can



n_s	C_s
2	0.16
3	0.28
4	0.38
5	0.46
8	0.64
10	0.71
16	0.83
32	0.95
Planar	1.00

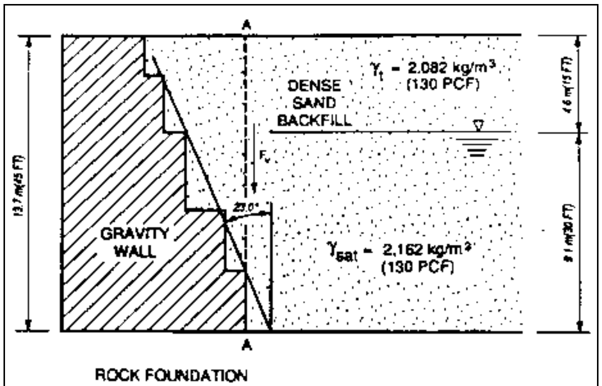
Figure 5. Definition sketch and values of the correction factor C_s for gravity walls founded on rock with stepped back sides

be obtained by including the vertical shear force in design.

(5) Figure 3 also shows that the limiting value of $(K_v)_{vert}$ develops at lower wall heights for walls with loose backfill than for walls with dense backfill.

(6) Figures 4 and 5 show the values of the correction factors C_θ and C_s, respectively, that are to be applied.

(7) An example application of the simplified procedure is shown in Figure 6. It can be noted that the steps in the back side of the wall in Figure 6 are not uniform. An average slope consistent with the



$H = 13.7$ m(45 FT) $\Rightarrow (K_v)_{vert} = 0.1$ (FIG. 3)

$\theta = 0.4$ RAD(23 DEG) $\Rightarrow C_\theta = 0.67$ (FIG. 4)

$n_s = 5 \Rightarrow C_s = 0.46$ (FIG. 5)

$$K_v = (1 - C_\theta C_s)(K_v)_{vert}$$
$$= (1 - 0.67(0.46))(0.1)$$
$$= 0.07$$

$$F_v = K_v(1/2\gamma_t D_1^2 + \gamma_t D_1 D_2 + 1/2\gamma_b D_2^2)$$
$$= 0.07(1/2(130)(15)^2 + 130(15)(30) + 1/2(135 - 62.4)(30)^2)$$
$$= 106{,}535 \text{ N/m}(7{,}300 \text{ LBS/FT})$$

Figure 6. Example calculation of the vertical shear force for a gravity wall founded on rock

definition sketch in Figure 5 is used to obtain the value of the correction factor C_θ from Figure 4.

(8) The vertical shear force determined using the simplified procedure can be incorporated in conventional equilibrium calculations. The results should be checked against the recommended criteria (EM 1110-2-1605, EM 1110-2-2502, EM 1110-2-2602, ETL 1110-2-22, ETL 1110-2-256, ETL 1110-2-310). When a toe fill of significant height exists, a vertical shear force at the toe should be included in the equilibrium calculations if a vertical shear force was applied to the back side of the wall. Neglecting the vertical shear force at the toe could result in unconservative estimates of the base contact area and the maximum bearing pressure on the foundation.

(9) Use of the simplified procedure to obtain a vertical shear force for stability calculations is restricted to gravity earth retaining walls that satisfy the following criteria:

(a) The vertical displacements within the foundation during construction of the wall and backfilling are negligible when compared with the vertical settlement within the backfill due to self-weight. Gravity-walls founded on competent rock foundations satisfy this criterion.

(b) The backfill soil does not creep. Compacted soils classified as SW, SP, GW, and GP according to the Unified Soil Classification System (American Society for Testing and Materials 1990) do not experience significant creep movements. The simplified procedure is also applicable to select SM backfills with nonplastic fines that do not creep.

(c) No special features that reduce or eliminate interface friction exist along the interface between the back of the wall and the backfill. Examples of special features that would reduce interface friction include bituminous coatings and synthetic barriers with low interface friction values.

(d) The interface between the back side of the wall and the backfill is capable of developing interface friction values of $\delta > 0.7\phi$, where ϕ is the effective angle of internal friction for the soil comprising the backfill. This is satisfied by SW, SP, GW, and GP backfills compacted against concrete walls. It is also satisfied by SM backfills with nonplastic fines compacted against concrete walls.

(e) The water table within the backfill is hydrostatic. If the variation of pore water pressure is not hydrostatic, the values of D_1 and D_2 in Equation 3 should be selected to represent the average conditions in the backfill.

b. Soil-structure interaction analysis.

(1) A complete soil-structure interaction analysis (Ebeling 1990) for computing shear loads along the backs of gravity walls can be accomplished using a finite element program such as SOILSTRUCT (Ebeling, Peters, and Clough 1992). Unlike conventional equilibrium procedures (Section 4*a*), an SSI analysis does not require the use of predetermined pressure distributions between the soil and the wall. Instead, it allows for development of these pressures through soil-structure interaction by simulating the staged construction that occurs. The computer program SOILSTRUCT can model the nonlinear stress-strain behavior of the soil, and allow for relative movement between the soil and the structure by incorporating interface elements in the mesh.

(2) Soil-structure interaction analyses are also especially useful for analyzing retaining structures founded on either soils or compressible rock foundations. Differential settlements within the foundation affect the magnitude of the shear force that the backfill exerts on the wall. The SSI analysis procedure has been successfully used for a wide variety of problems, including the Port Allen and Old River locks (Clough and Duncan 1969) and, more recently, the lock at Red River Lock and Dam No. 1 (Ebeling et al. 1993).

(3) A soil-structure interaction analysis is recommended for those structures for which the simplified procedure is not applicable (Section 5*a*) or for those cases in which a more precise evaluation of the shear force is required. Soil-structure interaction analyses are recommended for U-frame locks, retaining structures founded on soils, and structures with complicated geometry.

FOR THE DIRECTOR OF CIVIL WORKS:

1 Appendix

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