Field Behavior of Retained Earth Structure

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SYNOPSIS This paper describes a case study of a Retained Earth system constructed for the on- and off-ramps of a grade separation structure in Hayward, California. Field strain gage readings of reinforcing meshes were recorded at two instrumented sites. The results were closely examined and analyzed to assess the current design procedure. Based upon the information gathered, it was concluded that the field performance behavior of the system seems to justify the current general design procedure. However, it was also noted that the design of earth reinforced structures is complicated due to the interaction between the reinforcing elements and the surrounding soil; therefore, field instrumentation, performance behavior documentation and analysis are vitally important to ensure safe and economic design.

INTRODUCTION

In the past 15 years the advancement in soil improvement technology has been phenomenal. Among the most significant achievements is the development and application of tensile reinforcing elements for soil stabilization and improvement. A variety of tensile reinforcement have been developed, ranging from original metal strips pioneered by Vidal, to more recent products of steel wire mesh, geotextile sheets, and geogrids. While each of these products has proved to be suitable for different applications, still there is a need for documented field performance data which can be used for evaluating the adequacy of the many proposed design methodologies virtually untested in the field. The following is to report on a case study of the Retained Earth* which has been used primarily as an earth retention structure. Retained Earth is an earth reinforced system in which individual steel bar mesh (reinforcing mesh) units forming the reinforcing elements are attached at one end to concrete facing panels while the other end is free. Except for the difference in the concrete facing panel design, the system is similar to the mechanically stabilized embankment system described by Forsyth (1978). The field data reported in this paper is used to examine the field behavior mechanism of the Retained Earth system.

CURRENT DESIGN PROCEDURE

Limit design methods are currently used in the stability analysis of Retained Earth walls. As in the analysis of other types of earth reinforced systems (Juran et al., 1978, Brown et al., 1979), both the external and internal stability of a wall are examined. In the external stability analysis, conventional methods are used to check the bearing of the foundation soil and the safety of the wall against sliding. In order to ensure the internal stability, two criteria must be satisfied for a Retained Earth wall, i.e., the tensile stress in the longitudinal bars, σ_t, should not exceed the allowable stress, f_a, of the steel; and the total horizontal force, T, supported by each mesh unit should not exceed the pullout resistance, P_r. Equations for evaluating T, σ_t and P_r are presented in the following.

As noted in the following discussion, the backfill is generally restricted to a cohesionless material which can be characterized by a unit weight γ and an angle of internal friction ϕ. The backfill is strengthened by uniformly distributed reinforcing mesh units spaced at S_x horizontally and δ_v vertically. Each mesh unit of width b has a number of longitudinal bars connected by cross bars; both the longitudinal and cross bars have diameter d. Assuming a vertical stress σ_v and an active earth pressure condition, k_a, near the face of the wall, the total horizontal force, T, at depth h is given by:

$$T = k_a \sigma_v S_x S_v \tag{1}$$

For a wall of height H, the vertical stress σ_v at any depth is calculated as follows:

$$\sigma_v = \frac{(\sigma_v)_{max}}{H} \cdot h \tag{2}$$

where $(\sigma_v)_{max}$ is the vertical stress at the base of the wall calculated by considering the total equilibrium of the Retained Earth mass in association with the Meyerhoff approach (1953). In this approach the total vertical force is assumed to be uniformly distributed over a reduced base area due to the eccentricity of the applied loads.

Using Eq. (1) above, the tensile stress, σ_t, can then be expressed as follows:

$$\sigma_t = \frac{4k_a \sigma_v S_x S_v}{n\pi d^2} \tag{3}$$

In the current design procedure, the pullout resistance, P_r, is assumed to develop primarily through the bearing of cross bars against the backfill material. An empirical factor A_c, known as the anchorage factor, is used in calculating P_r, i.e.,

$$P_r = A_c \gamma h d b N \tag{4}$$

Values of A_c are determined from laboratory pullout tests (Al-Yassin, 1980). N is the number of cross bars in the length of mesh extending beyond the assumed failure surface; a Rankine failure surface is used in the analysis.

DESCRIPTION OF INSTRUMENTED WALL

It is clear from the above discussion that a better understanding of the behavioral mechanism of the Retained Earth system is

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necessary. Two sections of a full scale wall constructed in Hayward, California were instrumented. The wall ranges in height from 4 ft. to 20 ft. and has an area of 4,000 sq. ft. It supports a 2:1 sloping fill which extends a distance of approximately 50 ft. into the back of the wall.

The Retained Earth backfill material consisted of a gravelly sand with the following properties: density, γ = 122 pcf, uniformity coefficient, C_u = 42.7, and an angle of internal friction ϕ = 40.6°.

The reinforcing mesh was shop fabricated from cold drawn steel wire with a yield stress f_y = 65,000 psi. Each mesh consisted of 5 W11 longitudinal bars spaced at 6 in. on center and connected by W11 cross bars spaced at 2 ft. on center. Two mesh lengths of 14 ft. (Site 2) and 16 ft. (Site 1) were used.

Figure 1 shows the wall elevation and the location of the instrumented sections referred to as Site 1 and Site 2 respectively. Alternate layers of the reinforcing mesh were mounted with strain gages to measure the tensile strain in the longitudinal bars. A schematic representation of the strain gage layout is shown in Figure 2.

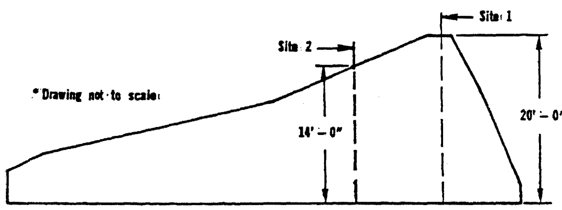

Figure 1. Elevation of Retained Earth Wall

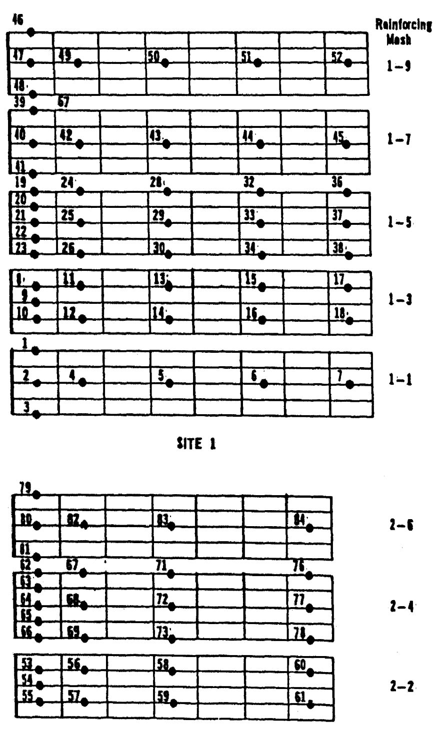

Figure 2. Strain Gage Layout

Field measurements were taken from the start and continu for a period of approximately one year after the wa construction was completed. However, the discussion present in this paper will be limited to the results taken at the end the wall construction, prior to the placement of sloping fi

FIELD MEASUREMENTS

Field strain gage readings were recorded and converted to tensi stresses, the distributions of tensile stress along longitudin bars were plotted with depth as shown in Figures 3 and 4 f Sites 1 and 2 respectively. It is important to point out tl data in its original form has large scatters; this is particular true for Site 2 readings. Data points shown on Figure 3 a average readings taken at the end of wall construction f Site 1. Curves are drawn to represent the adjusted field tensi stresses with depth which will be used later as the basis f discussion and comparison. The Site 2 readings were much mo erratic; a considerable amount of judgment was exercised establishing the adjusted curves. The following consideratio were given:

1) The data points shown in Figure 4 are the highest fie values recorded at each of the locations indicated.

2) Except for the curve representing the relationship at o foot from the face of the wall, all other curves a approximated, using data obtained from mesh 2-4, and tl general trend shown in Figure 3.

Based upon the information presented in Figures 3 and 4, it c be said that: (1) the tensile stresses are generally higher ne the face of the wall, decreasing to smaller values as th approach the free end of the mesh; and (2) the tensile stress seem to increase with increasing depth of fill.

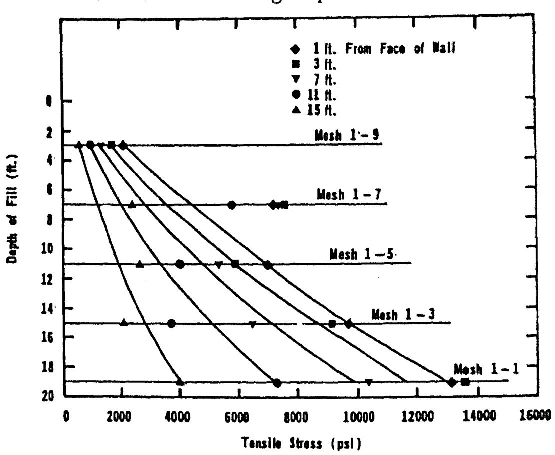

Figure 3. Tensile Stress in Reinforcing Mesh (Site 1)

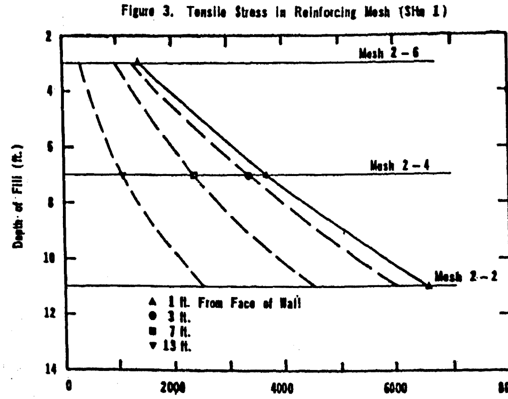

Figure 4. Tensile Stress in Reinforcing Mesh (Site 2)

COMPARISON OF THEORETICAL PREDICTIONS AND FIELD RESULTS

Tensile stress values were calculated using Eq. (3). These values are shown as dotted lines in Figures 5 and 6. In comparison with the adjusted field values, it can be seen that the calculated stresses are generally lower than the maximum field values. The higher field values could have been caused by the effects of compaction and other construction operations which cannot be accounted for in the analysis. Additional field measurements (not presented here) taken one year after the wall construction was completed show a drop in maximum tensile stresses in the meshes despite the additional load resulting from the sloping fill. This indicates that stress relaxation and redistribution might have taken place within the Retained Earth system. It can, therefore, be speculated that had the wall not been subjected to additional loading (from the sloping fill), the maximum tensile stresses may have dropped to even lower values than those measured at the end of construction, which may agree better with values predicted by Eq. (3). Laboratory pullout tests of wire mesh by Chang et al. (1977) have shown that redistribution of shear resistance takes place along the length of a mesh at relatively higher loading levels. Observation from this study seems to indicate that the same phenomenon also takes place in the field under long-term loadings.

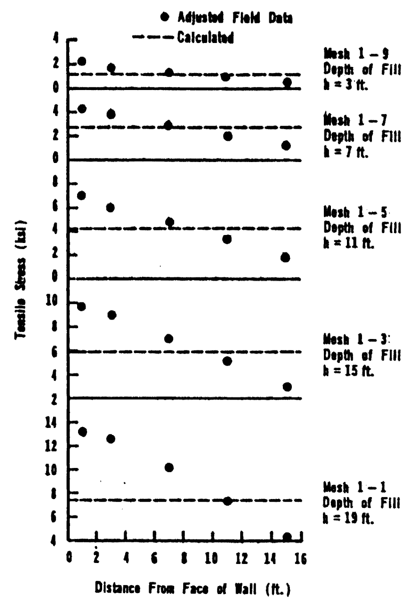

Figure 5. Tensile Stress Distribution Along Reinforcing Mesh (Site 1)

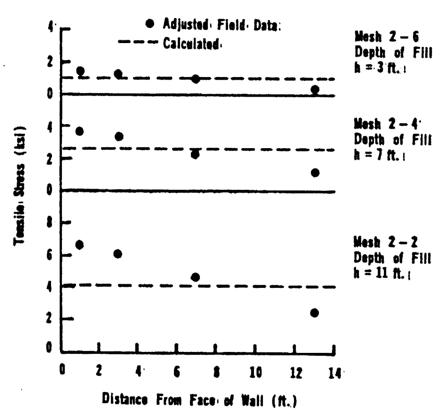

Figure 6. Tensile Stress Distribution Along Reinforcing Mesh (Site 2)

The pullout resistance was calculated using Eq. (4) and compared with the total horizontal force supported by each instrumented mesh layer as shown in Table 1. Values of the total horizontal force were calculated by multiplying the adjusted maximum tensile stress for each mesh by the cross-sectional area of steel per mesh (i.e., 0.55 in.^{2}). It can be seen from Table 1 that the ratio P_r/T is much greater than 1, thus ruling out the pullout failure mode for the given loading and geometric conditions of the systems studied.

It should also be noted that the assumptions used in Eq. 3, i.e., an active earth pressure condition and a Meyerhoff vertical pressure distribution, may not be adequate; a discussion of these assumptions is presented in the following section.

TABLE 1

Comparison Between Pullout Resistance and Total Horizontal Force

Site	Reinforcing Mesh	Depth of Fill (ft.)	T (kips)	P_r (kips)	P_r/T
1	1-9	3	1.2	4.6	3.8
	1-7	7	2.3	12.8	5.6
	1-5	11	3.9	20.3	5.2
	1-3	15	5.4	32.0	5.9
	1-1	19	7.3	46.4	6.4
2	2-6	3	0.8	5.6	7.0
	2-4	7	2.1	12.8	6.1
	2-2	11	3.6	23.5	6.5

ANALYSIS OF FIELD RESULTS

As mentioned earlier, the tensile stress varies along the reinforcing mesh from a maximum near the face of the wall to a minimum near the free end. This indicates that a similar variation in vertical stress should also exist along the reinforcing mesh. The Meyerhoff distribution used to calculate the vertical pressure in the current design procedure is not capable of modeling the variation. This assumption, which may be adequate for design purposes and has shown to be applicable in reinforced earth analysis (Shen et al., 1976), does not accurately model the behavior of the Retained Earth system. The field measurements seems to indicate that the variation of vertical pressure may be approximated by a trapezoidal distribution where the maximum stress $(\sigma_v)_{max}$ occurs at the face of the wall, and the minimum stress $(\sigma_v)_{min}$ occurs at the free end of the reinforcing mesh.

In addition, the active earth pressure coefficient is used in the current design procedure to calculate the tensile stress in the reinforcement. There is no data currently available regarding the state of stress within the Retained Earth system. The assumption of a k_a condition is open to questions. In the following discussion the active earth pressure coefficient (k_a = 0.22) was replaced by a value of 0.3; this represents an average value of the active and at rest earth pressure coefficients for the Retained Earth backfill. Using this new value for the earth pressure coefficient, the validity of the trapezoidal vertical pressure distribution was verified and the field and theoretical values were compared. The following is a brief description of the analysis:

1) The adjusted tensile stress values at 1 ft. and 15 ft. from the face of the wall were randomly chosen to calculate the

corresponding vertical stresses for mesh 1-1 at Site 1. These values were then used to calculate $(\sigma_v)_{max}$ and $(\sigma_v)_{min}$ for the trapezoidal distribution at depth of fill h = 19 ft.

2) Assuming that vertical stresses are directly proportional to the depth of fill, values of $(\sigma_v)_{max}$ and $(\sigma_v)_{min}$ at the base of the wall (h = 20 ft.) for Site 1 were calculated; these values are 3.93 ksf and 1.06 ksf respectively. The ratio of $(\sigma_v)_{max}/(\sigma_v)_{min}$ is approximately 4.

3) Using the vertical stress distribution established in Step 2 above for Site 1, the total vertical force at the base of the wall was calculated to be 40 kips. This is in good agreement with the weight of fill (39 kips) at the base level.

4) Vertical stresses at 1, 3, 7, 11 and 15 ft. from the face of the wall were calculated for h = 20 ft. Corresponding values for h = 3, 7, 11, 15 and 19 ft. were then calculated.

5) Tensile stresses in the reinforcing mesh were calculated using Eq. (3) and the vertical stresses obtained in Step 4.

Both the calculated and the adjusted field tensile stress values are tabulated in Table 2. It is reasonable to say that the comparisons are quite good.

TABLE 2

Comparison of Calculated and Measured Tensile Stresses (Site 1)

Reinforcing Mesh	Depth of Fill (ft.)	Distance From Face of Wall (ft.)				
		1	3	7	11	15
1-9	3	2.1*	1.9	1.5	1.1	0.7
		2.1**	1.7	1.3	1.0	0.5
1-7	7	4.9*	4.4	3.5	2.6	1.6
		4.2**	3.7	2.9	2.0	1.1
1-5	11	7.7*	6.9	5.5	4.0	2.5
		7.0**	6.0	4.8	3.4	1.8
1-3	15	10.5*	9.5	7.5	5.5	3.5
		9.7**	9.0	7.0	5.1	2.9
1-5	19	13.3*	12.8	9.5	6.9	4.2
		13.2**	12.6	10.2	7.3	4.2

*Calculated tensile stress in ksi.
**Adjusted field tensile stress in ksi.

Since the analysis performed for Site 1 gave good agreement between the calculated and the measured values, it was interesting to see if the same approach could be used to predict the field behavior of Site 2. This was done first by establishing the trapezoidal vertical pressure distribution based upon the total weight of the wall and a $(\sigma_v)_{max}/(\sigma_v)_{min} = 4$. The tensile stresses according to Eq. (3) were calculated for meshes 2-2, 2-4 and 2-6. These values are tabulated together with the corresponding adjusted field values in Table 3. Except for the values in mesh 2-6, the comparisons are rather favorable.

TABLE 3

Comparison of Calculated and Measured Tensile Stresses (Site 2)

Reinforcing Mesh	Depth of Fill (ft.)	Distance From Face of Wall (			
		1	3	7	1
2-6	3	2.1*	1.8	1.4	[illegible]
		1.4**	1.3	1.0	[illegible]
2-4	7	4.9*	4.3	3.2	[illegible]
		3.7**	3.4	2.4	[illegible]
2-2	11	7.6*	6.7	5.0	[illegible]
		6.6**	6.0	4.6	[illegible]

*Calculated tensile stress in ksi.
**Adjusted field tensile stress in ksi.

SUMMARY AND CONCLUSIONS

This paper describes a case study of a Retained Earth syste constructed for the on- and off-ramps of a grade separati structure in Hayward, California. Field strain gage readings the reinforcing meshes were taken at two instrumented site The results were closely examined and analyzed to assess t current design procedure. Based upon the information gathere the following tentative conclusions may be stated:

1) The field performance behavior of the system seems justify the current general design procedure.

2) The reinforcing meshes are capable of developing mu larger pullout resistance than strips; therefore, the pullo failure mode is not likely to control the design.

3) For the cases studied, the vertical pressure distribution the base of the wall can be better approximated by trapezoidal distribution with $(\sigma_v)_{max}/(\sigma_v)_{min} = 4$.

4) The average lateral earth pressure coefficient for granul fill of 0.3 may be used for Retained Earth analysis.

One should also realize from this and similar studies that th design of reinforced earth structures is complicated due large to the interaction between the reinforcing elements and th surrounding soil. Factors such as the rigidity of the systen the geometry of the structure and the backfill, the boundar and compressibility of the foundation play important roles determining the stresses developed in the reinforced earth mas Unless all the factors are thoroughly examined and their effec understood, it is difficult to present a generalized desig procedure. The complexity of the problem can be furthe compounded by relaxation, load transfer and redistribution withi the reinforced soil, which cannot be properly included in lim analyses. We believe field instrumentation, performanc behavior documentation and analysis are vitally important an much needed for the safe and economic design of eart reinforced structures.

ACKNOWLEDGMENT

The authors wish to express their appreciation to VS Corporation for supplying the field data presented in this pape

EFERENCES

l-Yassin, Z. (1980), "Pullout Test Results vs. Retained Earth," a VSL Corporation Report.

·own, I. E. W. and T. A. Rochester (1979), "Reinforced Earth-Technical Guidance Provided by the Department of Transport, England," International Conference on Soil Reinforcement: Reinforced Earth and Other Techniques, Vol. II, Paris, 423-430.

ıang, J. C., J. B. Hannon and R. A. Forsyth (1977), "Pull Resistance and Interaction of Earthwork Reinforcement and Soil," research report, Transportation Laboratory, CALTRANS.

ɔrsyth, R. A. (1978), "Alternative Earth Reinforcements," Proc. ASCE Symposium on Earth Reinforcement, Pittsburgh, 358-370.

ıran, I and F. Schlosser (1978), "Theoretical Analysis of Failure in Reinforced Earth Structures," Proc. ASCE Symposium on Earth Reinforcement, Pittsburgh, 528-555.

eyerhoh, G. G. (1953), "The Bearing Capacity of Foundations Under Eccentric and Inclined Loads," Proc. 3rd Int. Conf. Soil Mech. Fdn. Engng., Vol. I, 440-445.

ıen, C. K., K. M. Romstad and L. R. Herrmann (1976), "Integrated Study of Reinforced Earth - II: Behavior and Design," Vol. 102, No. GT6, ASCE.