STRESSES AND DEFORMATIONS IN JAIL GULCH EMBANKMENT

Jerry C. Chang and Raymond A. Forsyth,
California Division of Highways

Presented is a comprehensive case history of stress and deformation measured by field instrumentation in a 200-ft high Jail Gulch highway embankment. Theoretical analyses in which the finite-element method was used were conducted to predict stresses and deformations within the embankment. Theoretical equations were developed for calculating the nonlinear, stress-dependent tangent modulus of elasticity of soil; parameters obtained from triaxial compression tests on soil samples were used. When a constant value of Poisson's ratio was used, a Poisson's ratio of 0.3 was found to yield results best agreeing with the field data. The stresses and deformations calculated by the finite-element method of analysis agreed reasonably well with the field-measured data.

•THE California Division of Highways has conducted field performance studies to determine the stresses and deformations in several highway embankments over 200 ft in height. This paper presents the results of the field studies and theoretical analysis by finite-element method for the Jail Gulch embankment. A more detailed report of this research project has been presented by Chang et al. (2).

The Jail Gulch embankment was constructed on I-5 about 8 miles north of Yreka, California, during June through December 1968. The maximum height of the embankment is about 200 ft at the centerline of the roadway. The side slopes of the embankment are 1.5 to 1 normal and about 1.6 to 1 on the instrumented section due to a slight skew.

The embankment material consists of hard, metamorphic rock (greenstone) excavated from adjacent cuts. It is very coarse-graded with only a small percentage of fines. Geologic exploration in the nearby area indicated that there was a very thin overburden of about 3 to 4 ft over foundation rock consisting of weathered and broken greenstone to a depth of about 20 ft and underlain by hard hornblende andesite.

The test section of the Jail Gulch embankment is instrumented to measure horizontal movement, vertical settlement, and soil stresses at four levels.

FIELD MEASUREMENTS

Horizontal Movement

The horizontal movements were measured by horizontal movement platforms and turn pots placed within the fill and surveys on surface monuments. The most significant horizontal movement for all movement platforms and surface monuments occurred in the period from the beginning to about 100 days after completion of the embankment. Figure 1 shows the contours of the horizontal displacements at 15 months after completion of construction. The largest movements occurred near the midheight of the embankment. Because of the unsymmetrical configuration of the embankment, the contour of zero movement occurred to the right of the centerline.

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Vertical Movement

Vertical movements were measured by settlement platforms and surveys on surface monuments. Contours of vertical settlements 15 months after completion of construction are shown in Figure 2. The magnitude of settlement at any point at each level depends on the height of fill above that level and the depth between the instrumentation level and the foundation rock surface. A maximum settlement of 1.53 ft was measured at the B-instrumentation level on centerline on completion of the embankment. The settlements at this point were 1.66 ft and 1.74 ft respectively at 100 days and 15 months after completion of construction. It was expected that the maximum settlement would occur near C-level where the midheight of the embankment is located. The depth of overburden including weathered rocks above the foundation rock was assumed to be approximately 20 ft based on explorations at adjacent areas, and it was anticipated that the settlement, because of compression of the foundation overburden, would be small. The fact that the maximum settlement occurred at the B-level indicates a possibility that the actual depth of overburden and weathered rock was greater than anticipated, resulting in larger settlement due to compression in the foundation overburden. No borings were made to investigate the actual depth of overburden at the test section. Because there are no settlement data measured at the foundation boundary, the contours were connected by dashed lines based on estimates. Settlement platforms should be installed in the foundation overburden in future research projects.

Soil Stresses

Soil pressure cell groups were embedded in the embankment to measure vertical, horizontal, and inclined stresses. Although the measured soil pressures generally conform to the increase in embankment overburden pressure, the magnitude of measured pressure was found to vary somewhat with the type of bedding material. Generally, the cells embedded in sand indicated more consistent measurements compared to those embedded in clay or random fill material.

The soil stresses measured at completion of the fill are given in Table 1. These data indicate that the vertical stresses measured at PCG-1, -4, and -7 are generally in reasonably good agreement with the embankment overburden pressure with a difference of only 5 to 10 percent. The embankment overburden pressures were computed assuming a constant unit weight of 140 lb/ft^3 for the fill. The measured lateral horizontal stresses vary from 10 to 60 percent of embankment overburden. The longitudinal horizontal stresses and the 45-deg inclined stresses are within 10 to 80 percent of embankment overburden except at PCG-2 where 125 percent of embankment overburden was recorded. The major and minor principal stresses and the maximum shear stresses were also calculated from the measured stresses (Table 1). These calculated stresses appear to be of the proper order of magnitude.

FINITE-ELEMENT ANALYSIS

A computer program using finite-element method of analysis was developed for this project. This program permits the use of nonhomogeneous and nonlinear material properties in the evaluation of stresses and deformations in an embankment. It also permits an incremental construction analysis to simulate more closely the placement of successive layers of embankment materials during construction. A finite-element analysis of such nature has been used previously (3, 5, 9).

A finite-element analysis requires evaluation of the elastic constants of the embankment material. One method of calculating the nonlinear tangent modulus of soils was proposed by Duncan (4) using initial tangent modulus, shear strength parameters, and principal stresses at failure.

Chang et al. (2) proposed a basic equation for calculating the tangent modulus, E_t, of soils as

$$E_t = E_i \left[1 - \frac{\gamma H \sin \phi \, (1 - \sin \phi)}{2c \cos \phi + 2q \sin \phi}\right]^2 \qquad (1)$$

Figure 1. Contours of horizontal displacement.

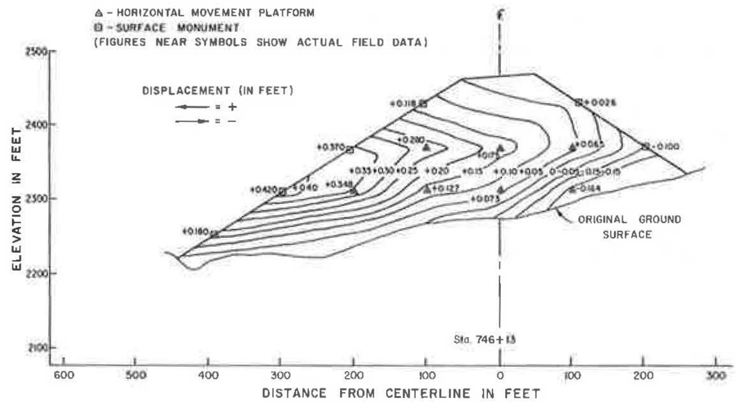


Figure 2. Contours of settlement.

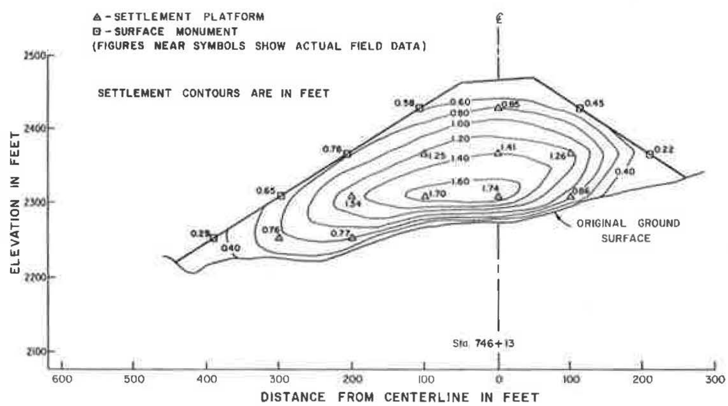


Table 1. Summary of soil stress measured at completion of fill.

Designation	Orientation	Stress Location PCG-1	PCG-2	PCG-3	PCG-4	PCG-5	PCG-6	PCG-7	PCG-8	PCG-9
σ_a	Vertical	58[a] (110)	110[a] (105)	20[a] (40)	100[a] (90)			50[a] (90)	42[a] (45)	35[a] (65)
σ_b	Lateral horizontal	18[a] (30)	35[a] (32)	30[a] (60)	35[a] (32)			0[a] (0)	60[a] (60)	6[a] (10)
σ_c	Longitudinal horizontal	42 (80)	53 (50)	30 (55)	33 (30)			10 (20)	40 (40)	10 (20)
σ_d	Laterally inclined 45 deg	17[a] (30)	45[a] (40)	16[a] (30)	24[a] (20)	36 (25)	90 (80)	16[a] (30)	43[a] (45)	40[a] (75)
σ_e	Laterally inclined 45 deg		132 (125)		60 (55)				40 (40)	
σ_f	Longitudinally inclined 45 deg				60 (55)					
σ_g	Longitudinally inclined 45 deg				30 (25)					
σ_1	Major principal	67 (127)	119 (114)	35 (70)	122 (110)			52 (93)	63 (68)	45 (84)
σ_2	Minor principal	9 (17)	26 (25)	15 (30)	13 (12)			2 (4)	39 (42)	-4 (7)
τ_{max}	Maximum shear	25 (48)	47 (45)	10 (20)	55 (50)			25 (45)	12 (13)	25 (46)

Note: Numbers without parentheses give soil stresses in psi; numbers within parentheses give soil stress in percentage of embankment pressure directly above each pressure cell group.

[a]Used to compute the major and minor principal stresses and the maximum shear stresses.

where

E_i = initial tangent modulus in pounds per square foot,
c, ϕ = shear strength parameters determined from ultimate strength criterion (c is expressed with unit in pounds per square foot),
γ = density of soil in pounds per cubic foot,
H = height of embankment overburden in feet,
N_ϕ = $\tan^2 (45 \text{ deg} + \phi)$, and
$q = \frac{\gamma H}{N_\phi} - \frac{2c}{(N_\phi)}$.

The initial tangent modulus can be expressed in a linear form as suggested by Scheidig (11)

$$E_i = A + B\sigma_3 \tag{2}$$

and by Janbu (8) in an exponential form,

$$E_i = K(\sigma_3)^n \tag{3}$$

Based on the finding by Jaky (7) and Brooker and Ireland (1), the principal stress σ_3 can be expressed as

$$\left.\begin{aligned} \sigma_3 &= (1 - \sin \phi)\sigma_1 \\ \sigma_3 &= (1 - \sin \phi)\gamma H \end{aligned}\right\} \tag{4}$$

where σ_1 is assumed to be equal to the vertical embankment overburden pressure. The constants A, B, K, and n are determined from arithmetic and log-log plots using the values of E_i and the confining pressures, σ_3, obtained from triaxial compression tests. The Poisson's ratio was also calculated by using the stress-strain and volume change data obtained from triaxial compression tests of the Jail Gulch material and by using the equation proposed by Duncan (4).

$$\mu = \frac{\Delta\xi_v - \Delta\xi_a}{2\Delta\xi_a} \tag{5}$$

where

μ = Poisson's ratio,
$\Delta\xi_a$ = incremental axial strain, and
$\Delta\xi_v$ = incremental volumetric strain.

The relation between the calculated values of Poisson's ratio and the corresponding deviator stress is shown in Figure 3. This figure indicates that, for a given confining pressure, the Poisson's ratio increases approximately linearly with deviator stress. All values of μ fell within a range of 0.05 to 0.5 regardless of the magnitude of confining pressure. Poisson's ratio values of 0.25, 0.3, and 0.4, which are near the midpoints of the test data shown in Figure 3, were selected for use in the finite-element analysis.

In the finite-element method of analysis, the cross section of a solid mass is divided into a finite number of elements connected at nodal points. The finite-element mesh model for the Jail Gulch embankment is shown in Figure 4. Based on the exploration data of the surrounding area, an overburden of 20 ft in depth was assumed in the foundation. Boundary limits of zero deformation were assumed at 100 ft in the foundation rock and 100 ft and 150 ft away from the right and left toes of the embankment respectively. The embankment was divided into nine horizontal layers to simulate incremental construction. Incremental construction analysis involves calculation of stresses and deformations in a successive superposition procedure.

The foundation rock and overburden were assumed to be weightless. The modulus of elasticity of the foundation rock was estimated to be 4.4×10^8 lb/ft^2 based on Fairhurst (6). The modulus is the dynamically determined in situ modulus for andesite rock.

The modulus of the foundation overburden was assumed to be the same as that obtained from the JG-1 sample material. The nonlinear tangent moduli were calculated in accordance with the embankment load corresponding to the assumed sequence of incremental construction. As mentioned previously, Poisson's ratios of 0.25, 0.30, and 0.40 were introduced separately in the analysis. However, it was found that the magnitude of the maximum displacement, calculated from the trial analyses using tangent modulus values computed from the linear expression for initial tangent modulus and a Poisson's ratio of 0.30, was in best agreement with the maximum measured value of the field displacement. This Poisson's ratio value of 0.3 is consistent with the finding by Richart, Hall, and Woods (10) for soils of this type. The results of the finite-element analysis are presented as follows.

Vertical Displacements

The contours of the calculated settlements from finite-element analysis are shown in Figure 5. The calculated maximum settlement was 1.94 ft, which compares reasonably well with the largest measured settlement of 1.74 ft at 15 months after completion of the embankment. However, the distribution pattern of the calculated settlement does not compare as favorably with that measured (Fig. 2). The maximum field settlement occurred near the bottom third of the embankment, whereas the calculated maximum settlement was located near the midpoint.

The discrepancies are thought to be attributed largely to initial settlement. The actual initial settlement that occurred below any settlement platform level was undetected in the construction process because the fill was placed to the planned elevation. However, in the computer analysis, the calculated settlement includes the initial settlements of each layer of fill placed due to the increase in gravity load of subsequently placed layers. Thus, the calculated results indicate a greater value of settlement and a higher elevation of the maximum settlement to occur in the embankment.

Horizontal Displacements

The contours of calculated theoretical horizontal displacements are shown in Figure 6. A comparison of this figure with the measured horizontal movements in Figure 1 shows that the calculated values are generally larger than those measured near the slopes of the embankment, particularly on the left side. Good agreements are seen in surface horizontal movements and the position of zero horizontal movement contour.

Stresses

The magnitude of calculated vertical stresses agrees reasonably closely with the embankment overburden pressure. Contours of the calculated vertical stresses are shown in Figure 7. The measured vertical stresses are also shown in this figure with the magnitude indicated in parentheses.

Contours of the calculated lateral stresses are shown in Figure 8 along with field-measured stresses. The agreement between measured and calculated lateral stresses is generally good except at PCG-8.

Generally, the results of the finite-element analysis of stresses and deformations correspond quite closely with observed behavior in both the magnitude and pattern of movements. Some factors that could influence the accuracies of the analysis results are believed to be as follows:

1. A two-dimensional finite-element analysis was made. The Jail Gulch embankment is constructed across a V-shaped canyon where three-dimensional analysis would have been more appropriate. Some longitudinal strains were detected by turn pot 13 along the longitudinal axis of the embankment, which indicates the possible error due to the two-dimensional assumption.

Figure 3. Variation of Poisson's ratio with deviator stress and confining pressure.

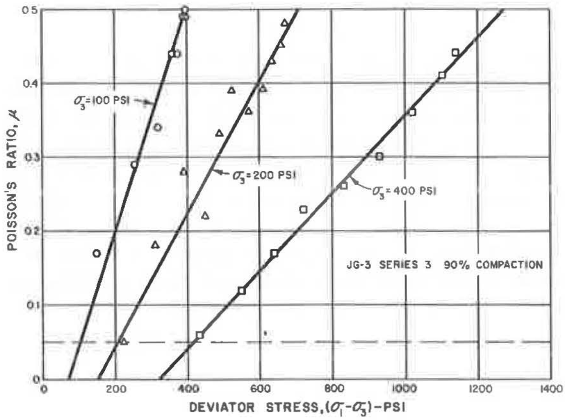


Figure 4. Finite-element mesh for Jail Gulch embankment (station 746+13).

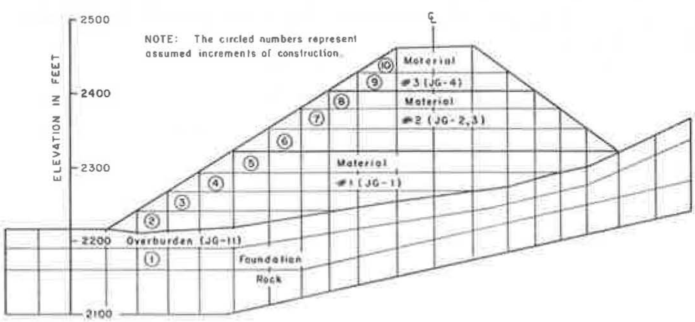


Figure 5. Contours of theoretical settlement.

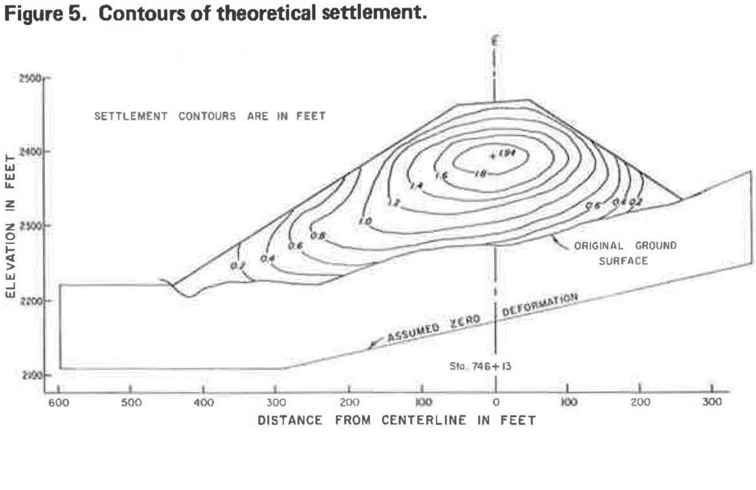


Figure 6. Contours of theoretical horizontal movement.

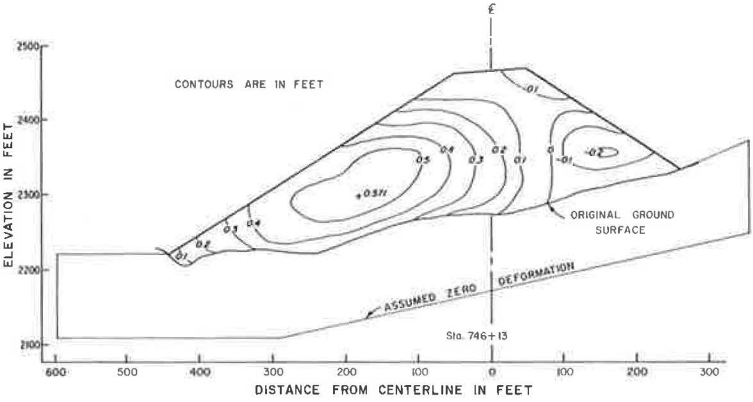


Figure 7. Contours of theoretical vertical stress.

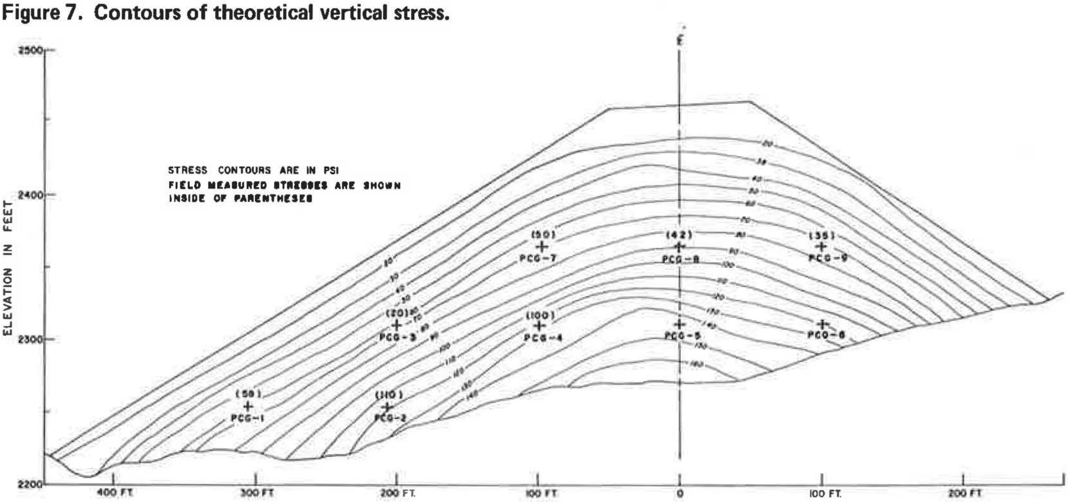
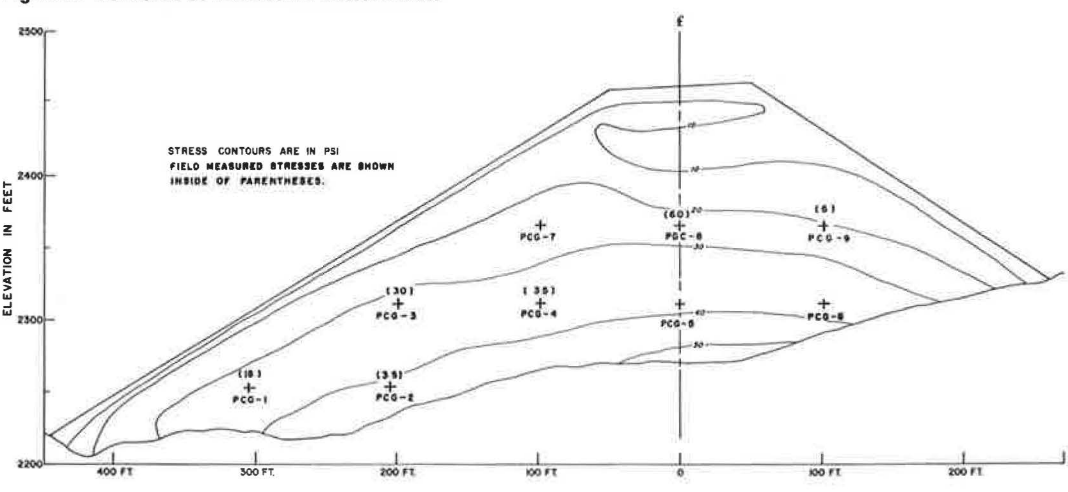


Figure 8. Contours of theoretical lateral stress.



2. The embankment material was nonuniform. The embankment was assumed to consist of only three types of soils with different elastic properties as determined from the laboratory tests on four soil samples. These did not include the large rocks prevalent in the actual embankment.

3. The equations for tangent modulus were simplified by assuming that (σ_1) ult = (σ_1) even though the theoretical maximum stresses in the embankment do not reach the plastic equilibrium condition at any point. These equations were further simplified by assuming that the embankment pressure represents the major principal stress in order to eliminate the unlimited iteration in the computing process. In addition, nonlinearity of the Poisson's ratio was not introduced into the finite-element analysis because of the limitation of available laboratory test data. A constant value of 0.3 was selected to represent the Poisson's ratio in the entire embankment material in the final analyses.

CONCLUSIONS

The study substantiates the following conclusions:

1. Most of the soil stresses measured by pressure cells in the Jail Gulch embankment responded proportionally to the embankment load. The soil stresses generally became stabilized when the fill reached its final grade directly above the point where the soil stresses were measured.
2. The measured deformations in the Jail Gulch embankment increased generally in proportion to the height of the embankment. However, these deformations continued at a significant rate until approximately 100 days after completion of fill. The deformations increased continuously through the observation period of 15 months after completion of fill, but the rate of increase became progressively smaller.
3. The theoretical calculations of stresses and deformations using two-dimensional finite-element analysis in general compare favorably with measured values. In the method of analysis used, the evaluation of the elastic constants was found to be the most important factor in the prediction of the embankment performance.
4. The successful application of finite-element method in evaluating the performance of the Jail Gulch embankment indicates that this method may be more extensively used in the future as a design tool.

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