Interrelationship of Load, Volume Change, and Layer Thicknesses of Soils to the Behavior of Engineering Structures

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In the preparation of this report an attempt has been made to comprehend most of the variables necessary to formulate an engineering approach to the vertical movement of the earth's surface caused by the shrinking and swelling properties of soils. Calculation of such movements by the proposed methods establishes data which are valuable for use in analyzing long-time performance or behavior of engineering structures. It will also be shown that the behavior of pavements, whose failures are not attributable to the use of inferior quality material or to the depths of material, can be explained on the basis of volume change. In order to establish a rational approach to the problem, this report sets forth the following:

1. The relation of volumetric swell to linear swell.
2. The relation of moisture change to volume change for soils having various shrinkage characteristics.
3. Relation of moisture contents to the *LL* of clays taken from subgrades underneath old pavements.
4. The relation of load to the volume change (swell) of soils involving behavior of both sublayers and overburden strata.
5. Calculation of potential vertical rise for layered systems.
6. Relation of P.I. and thickness of soil layer to potential vertical rise.
7. Relation of potential vertical rise to behavior of concrete pavements and structures.

It is suggested that the use of potential vertical rise calculations (also referred to as P.V.R.) will make it possible to determine the following:

1. Whether pavements will develop irregular riding due to movements not attributable to the thickness of materials or the use of materials of inadequate quality.
2. Whether or not culverts, buildings, and other lightweight structures will rise excessively.
3. When it is desirable and feasible to suspend floor slabs from foundation shafts or to utilize subgrade soils as the supporting medium.

● IT is not uncommon to find pavements and other lightweight structures which have moved excessively even though they are underlaid by very firm soils. After observing and testing soils from many such cases, it occurred to the author that movements were due primarily to volume change caused largely by fluctuation of moisture contents. If this is true, and most engineers and scientists seem to agree that it is, then it seems that some procedures and techniques, at least for interpretative purposes, are in order. It is the intent of this report to point out some of the aspects of the problem and to offer some methods for its solution.

At the outset it was necessary to determine the relation of volumetric swell to linear or vertical swell. Figure 1 shows the relation of these two types of swell obtained by measurement of a number of triaxial specimens which

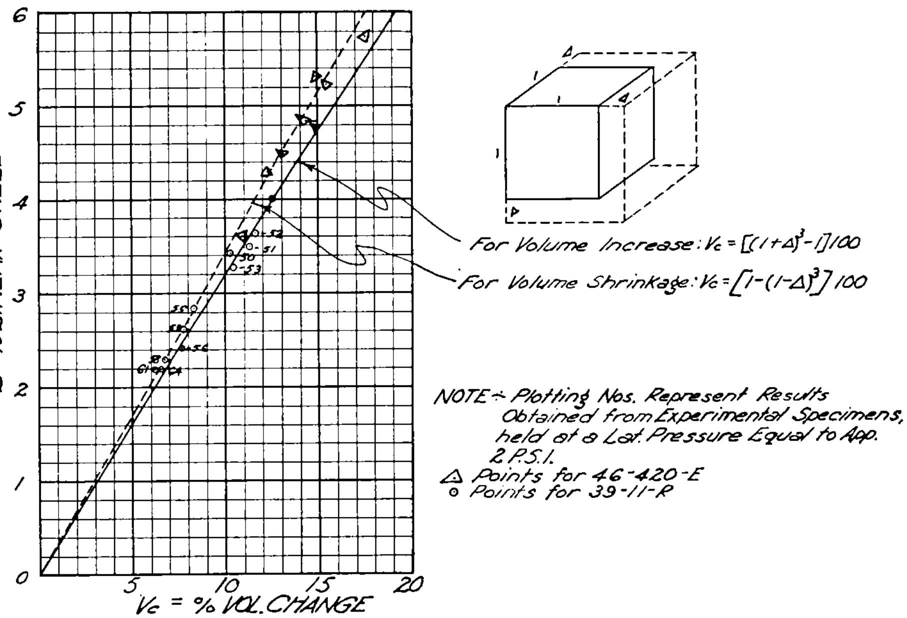

Figure 1. Relation of volumetric to linear swell.

were held at a lateral pressure of approximately 2 psi during absorption.

It may be noted on this chart that the usual formulae for converting volume change of a cube to linear change straddle the experimental data. It is realized that such laboratory data, in the absence of actual field measurement, are incomplete and also that the data shown in Figure 1 were not sufficiently complete within themselves to analyze recorded field movements where only moisture contents and soil constants are known. Therefore, Figure 2 was prepared which shows the relation of volume change to moisture change for soils having various shrinkage factors. Table 1 shows data taken in a heavy clay area of Navarro County where soil constants and moisture contents were determined. Elevations were also obtained on bench marks set in the ground at various levels (*1*). By plotting moisture ranges in Figure 2, percent volume change was obtained graphically by differences and recorded in Table 1. Positive values are shown in cases where moisture increased and negative values are recorded if moisture contents decreased. Volume change was converted into linear change by use of Figure 1. For this conversion the shrinkage curve was used when moisture was lost and the volume increase curve when moisture contents gained. Knowing the thicknesses of layers, the vertical linear movement was calculated. It is interesting to note that the total estimated rise of the ground during the period involved was 0.208 foot. Precise levels run on bench marks set at various levels indicated the upper bench mark (the bottom of which was set at the 1-foot level) actually rose 0.235 foot. This is a surprisingly close check to the value estimated. It may be that vertical volume change of soils in deep cuts and under certain types of structures will be greater than that shown, due to restraint of side walls causing most of the swell to move vertically; however, the portion of soil moving upward must overcome its own shearing strength before this can happen. Cuts in marls and clays usually heave more than other sections because of their low moisture contents.

In order to pursue this problem further, it was necessary to obtain a means of estimating moisture fluctuations. The points plotted in Figure 3 show the moisture contents found immediately beneath old pavements, throughout various parts of Texas, which were placed on various types of soils containing no aggregate. The line represented by the equation $0.47LL + 2$ corresponds to the maximum

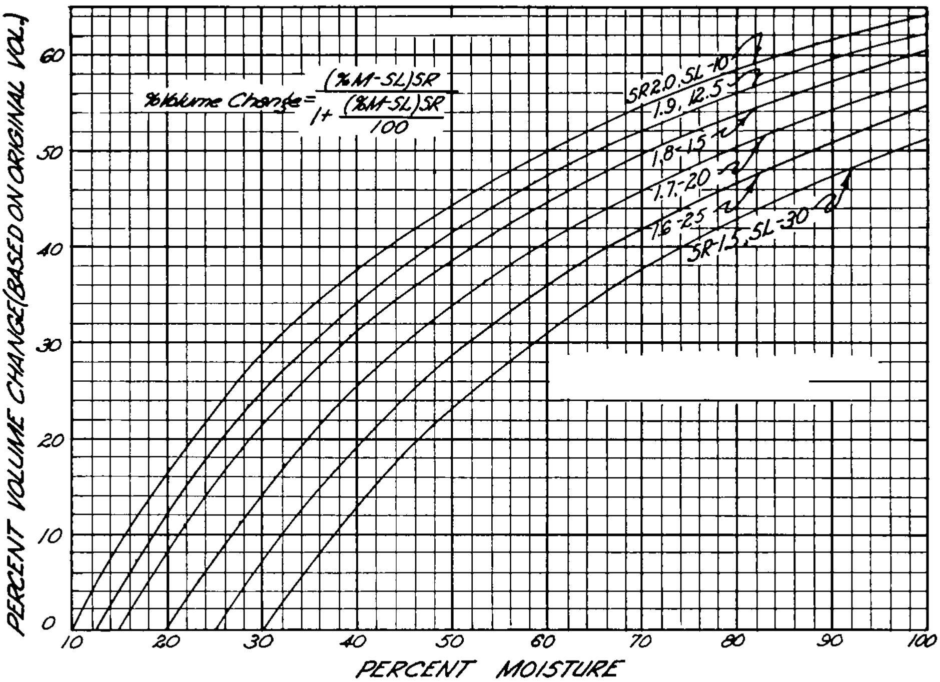

Figure 2. Relation of moisture change to free (i.e., neglecting expansion pressures and surcharge loads) volume change.

TABLE 1
MOISTURE CONTENTS AND ELEVATIONS AT 50 FOOT RT. STATION 497 + 50, HIGHWAY 14, NAVARRO COUNTY

Depth Below Surface	Summer 1934 Elevation 420.5	Summer 1934 Moisture	Spring 1936 Elevation 420.735	Spring 1936 Moisture	Volume Change	Linear Change	Vertical Linear Movement
		per-cent		*per-cent*	*percent*	*percent*	*feet*
0-1		15		10	−9	−3.1	−0.031
1-3		20		34	+16	+5.1	+0.102
3-5		21		28	+9	+2.9	+0.058
5-7		22		27	+7	+2.3	+0.046
7-9		22		25	+4	+1.3	+0.026
9-11		22		24	+3	+1.0	+0.020
11-13		24		25	+1	+0.4	+0.008
13-15		27		27	0	0	0
15-17		27		25	−2	−0.7	−0.014
17-18		29		27	−2	−0.7	−0.007
Total							+0.208

Note: BM at 20-foot depth did not move during this period; therefore moisture below 18-foot level must have been constant during this period.

capillary absorption obtained by laboratory tests on specimens molded at optimum. During or shortly after rainy seasons some broken or poorly drained pavements show greater moisture contents than $0.47LL + 2$, but this usually applies only to a thin layer near the bottom of pavement. The line represented by the expression $(LL - 15)\ 0.4$ indicates the driest condition to be expected under old pavements. Therefore, it is possible that moisture contents of subgrades at shallow depths may fluctuate between these limits. These limits of variation in moisture change do not occur at greater depths because surcharge restraint affects absorption. Another notable exception where some of the absorption variation previously mentioned does not produce swell is in those cases where the shrinkage limits are higher than respective values of $0.4(LL - 15)$. The minimum moisture contents from which swelling clays usually expand are represented by the line expressed as $0.2LL + 9$.

Since it is rather obvious that restraint affects the movement of soils, it became necessary to establish a set of swell-pressure curves as shown in Figure 4. The points plotted were obtained by measuring the expansive pressures of ½-inch height by 4-inch diameter specimens by use of the Standard Harvard Consoli-

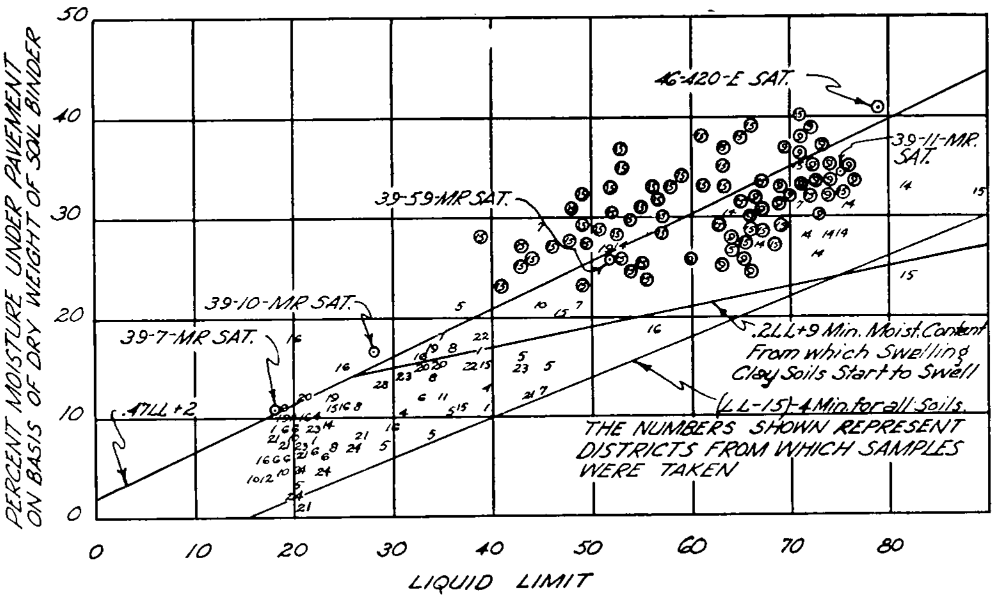

Figure 3. Moisture data for subgrade soils under pavement. Samples contained from 90 to 100 percent soil binder.

dometer equipment. Expansive pressures were obtained on specimens molded to various moisture-density conditions. A family of swell-pressure curves has been established in Figure 4 which is identified on the chart by its volume-change values at no restraint condition. The range of moisture content fluctuation of from $0.2LL + 9$ to $0.47LL + 2$ in conjunction with Figure 2 is useful in identifying the volume change family member curve. To illustrate how this works and also to determine how well the method checks field conditions, the following example involving the previously mentioned Navarro County bench marks is presented:

The subsoil existing at the 25-foot level has an $LL = 80$ and an $SL = 9$. If this soil were at the surface of the ground under a pavement, its moisture content would vary from as low as $0.2(80) + 9 = 25$ percent to as high as $0.47 \times 80 + 2 = 39$ percent. From Figure 2 the volume change would be 38 percent for 39 percent moisture—25 percent for 25 percent moisture = 13 percent. If this free swelling condition is restrained by 1½-psi surcharge (weight of *B.M.* and 1 foot of soil), percent volume swell will be equal to 11 (see Figure 4). By interpolation from the family of curves in Figure 4, it may be observed that a load of 27 psi would be required to prevent a soil of this type from swelling. The hydrostatic load from a column of soil 25 feet deep weighing 125 pounds/cubic foot would be approximately equivalent to 23 psi. It is interesting to note

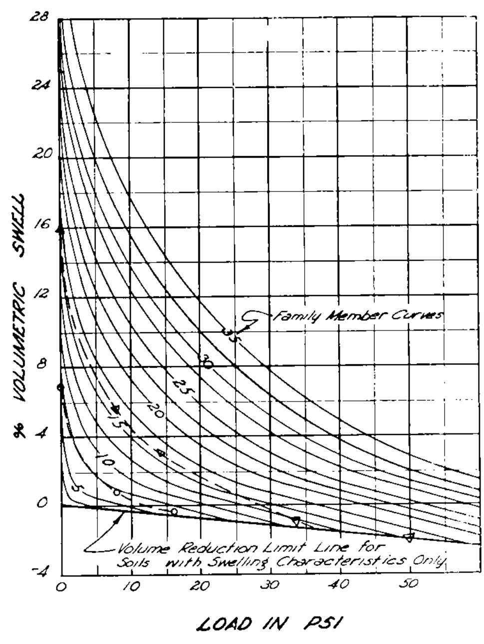

Figure 4. Relation of load to the volume change of swelling clay soil.

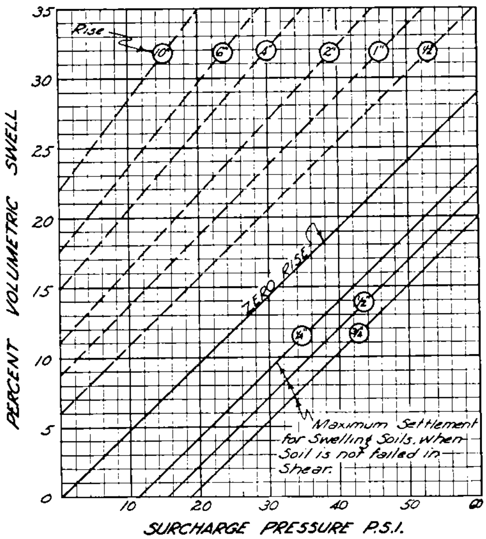

Figure 5. Potential vertical rise for underlying layers of subsoil 60 feet or more thick.

that level readings made over a period of two years on bench marks set at various levels up to 25 feet in depth showed that only the *B.M.* at the 25-foot level was free from vertical movement. The 25-foot-level *B.M.* might have moved if levels had been run over a greater period of time.

Figures 4 and 5 show clearly that some loadings on soils of certain volume-change characteristics create conditions capable of producing settlement. Such loadings are not ordinarily encountered in placing lightweight structures on clay soils which are saturated in excess of $0.48LL + 2$. In dealing with light loads on such weak, "mucky" soils, or with firmer clay soils to carry heavy loads, consideration should be given to several of the following aspects which are not covered in this report.

1. Shearing strengths of soils required to prevent shear failures or plastic flow.

2. Consolidation. The standard methods in common use are recommended.

Tables 2 through 6 show the computations (based on interpolations from Figure 4) of potential vertical rise for thick layers of clay soils having volume changes of 35, 25, 22.5, 15, and 7.5, respectively. Data from these tables are plotted in Figure 5 pertaining to thick layers of subsoil. Data from these tables

TABLE 2

CONVERSION OF VOLUME CHANGE TO POTENTIAL VERTICAL RISE WHEN AN 18-INCH PAVEMENT OVERLIES A CLAY HAVING 35 PERCENT VOLUME SWELL IN AN UNLOADED CONDITION

Load	Av. Load	Percent Swell		Depth of Layer	Vertical Movement
		Volume average	Linear		
psi	*psi*			*feet*	*inches*
0		35			
1.5-2.5	2	27.5	8.3	1 × 1.15 = 1.15	8.3% × 1.15 × 12 = 1.15
2.5-5	3.75	24	7.3	2.5 × 1.15 = 2.87	7.3% × 2.87 × 12 = 2.51
5-7.5	6.25	20.5	6.3	2.5 × 1.15 = 2.87	6.3% × 2.87 × 12 = 2.17
7.5-10	8.75	18.5	5.8	2.5 × 1.15 = 2.87	5.8% × 2.87 × 12 = 2.00
10-12.5	11.25	16.5	5.2	2.5 × 1.15 = 2.87	5.2% × 2.87 × 12 = 1.79
12.5-15	13.75	14.7	4.7	2.5 × 1.15 = 2.87	4.7% × 2.87 × 12 = 1.62
15-17.5	16.25	13.5	4.3	2.5 × 1.15 = 2.87	4.3% × 2.87 × 12 = 1.48
17.5-20	18.75	12.0	3.8	2.5 × 1.15 = 2.87	3.8% × 2.87 × 12 = 1.31
20-22.5	21.25	11.0	3.5	2.5 × 1.15 = 2.87	3.5% × 2.87 × 12 = 1.20
22.5-25	23.75	9.9	3.2	2.5 × 1.15 = 2.87	3.2% × 2.87 × 12 = 1.10
25-27.5	26.25	8.9	2.8	2.5 × 1.15 = 2.87	2.8% × 2.87 × 12 = 0.96
27.5-30	28.75	8.0	2.6	2.5 × 1.15 = 2.87	2.6% × 2.87 × 12 = 0.89
30-32.5	31.25	7.3	2.4	2.5 × 1.15 = 2.87	2.4% × 2.87 × 12 = 0.83
32.5-35	33.75	6.5	2.1	2.5 × 1.15 = 2.87	2.1% × 2.87 × 12 = 0.72
35-37.5	36.25	5.7	1.9	2.5 × 1.15 = 2.87	1.9% × 2.87 × 12 = 0.65
37.5-40	38.75	5.1	1.7	2.5 × 1.15 = 2.87	1.7% × 2.87 × 12 = 0.58
40-42.5	41.25	4.5	1.5	2.5 × 1.15 = 2.87	1.5% × 2.87 × 12 = 0.52
42.5-45	43.75	4.0	1.3	2.5 × 1.15 = 2.87	1.3% × 2.87 × 12 = 0.45
45-47.5	46.25	3.5	1.1	2.5 × 1.15 = 2.87	1.1% × 2.87 × 12 = 0.38
47.5-50	48.75	3.0	1.0	2.5 × 1.15 = 2.87	1.0% × 2.87 × 12 = 0.35
50-52.5	51.25	2.6	0.9	2.5 × 1.15 = 2.87	0.9% × 2.87 × 12 = 0.31
52.5-55	53.75	2.3	0.7	2.5 × 1.15 = 2.87	0.7% × 2.87 × 12 = 0.24
55-57.5	56.25	2.0	0.6	2.5 × 1.15 = 2.87	0.6% × 2.87 × 12 = 0.20
57.5-60	58.75	1.7	0.5	2.5 × 1.15 = 2.87	0.5% × 2.87 × 12 = 0.17
60-62.5	61.25	1.4	0.5	2.5 × 1.15 = 2.87	0.5% × 2.87 × 12 = 0.17
62.5-65	63.75	1.1	0.4	2.5 × 1.15 = 2.87	0.4% × 2.87 × 12 = 0.14
65-67.5	66.25	0.8	0.3	2.5 × 1.15 = 2.87	0.3% × 2.87 × 12 = 0.10
67.5-70	68.75	0.6	0.2	2.5 × 1.15 = 2.87	0.2% × 2.87 × 12 = 0.07
70-73	71.50	0.3	0.1	3 × 1.15 = 3.45	0.1% × 3.45 = 0.03
				Total depth = 82.09	Total P.V.R. due to swell = 24.09

TABLE 3

CONVERSION OF VOLUME CHANGE TO POTENTIAL VERTICAL RISE WHEN AN 18-INCH PAVEMENT OVERLIES A CLAY HAVING 25 PERCENT VOLUME SWELL IN AN UNLOADED CONDITION

Load	Average Load	Percent Swell		Depth of Layer	Vertical Movement
		Volume average	Linear		
psi	*psi*			*feet*	*inches*
0		25		0	
1.5–2.5	2	18.2	5.7	1.15 × 1 = 1.15	5.7% × 1.15 × 12 = 0.79
2.5–5	3.75	15.5	4.9	1.15 × 2.5 = 2.87	4.9% × 2.87 × 12 = 1.69
5–7.5	6.25	13.0	4.18	1.15 × 2.5 = 2.87	4.18% × 34.4 = 1.44
7.5–10	8.75	11.1	3.55	1.15 × 2.5 = 2.87	3.55% × 34.4 = 1.22
10–12.5	11.25	9.5	3.06	1.15 × 2.5 = 2.87	3.06% × 34.4 = 1.05
12.5–15	13.75	8.3	2.66	1.15 × 2.5 = 2.87	2.66% × 34.4 = 0.91
15–17.5	16.25	7.2	2.36	1.15 × 2.5 = 2.87	2.36% × 34.4 = 0.81
17.5–20	18.75	6.2	2.05	1.15 × 2.5 = 2.87	2.05% × 34.4 = 0.70
20–22.5	21.25	5.4	1.75	1.15 × 2.5 = 2.87	1.75% × 34.4 = 0.60
22.5–25	23.75	4.6	1.5	1.15 × 2.5 = 2.87	1.5% × 34.4 = 0.52
25–27.5	26.25	3.8	1.2	1.15 × 2.5 = 2.87	1.2% × 34.4 = 0.41
27.5–30	28.75	3.3	1.1	1.15 × 2.5 = 2.87	1.1% × 34.4 = 0.38
30–32.5	31.25	2.7	0.86	1.15 × 2.5 = 2.87	0.86% × 34.4 = 0.30
32.5–35	33.75	2.3	0.75	1.15 × 2.5 = 2.87	0.75% × 2.87 × 12 = 0.26
35–37.5	36.25	1.8	0.55	1.15 × 2.5 = 2.87	0.55% × 34.4 = 0.19
37.5–40	38.75	1.6	0.5	1.15 × 2.5 = 2.87	0.5% × 34.4 = 0.17
40–42.5	41.25	1.2	0.36	1.15 × 2.5 = 2.87	0.36% × 34.4 = 0.12
42.5–45	43.75	0.9	0.26	1.15 × 2.5 = 2.87	0.26% × 34.4 = 0.09
45–47.5	46.25	0.6	0.2	1.15 × 2.5 = 2.87	0.2% × 34.4 = 0.07
47.5–50	48.75	0.4	0.15	1.15 × 2.5 = 2.87	0.15% × 34.4 = 0.05
50–52.5	51.25	0.0	0.0	1.15 × 2.5 = 2.87	0% × 34.4 = 0.00
				Total depth = 58.55	Total P.V.R. due to swell = 11.77

TABLE 4

CONVERSION OF VOLUME CHANGE TO POTENTIAL VERTICAL RISE WHEN AN 18-INCH PAVEMENT OVERLIES A CLAY HAVING 22.5 PERCENT VOLUME SWELL IN AN UNLOADED CONDITION

Load	Average Load	Percent Swell		Depth of Layer	Vertical Movement
		Volume average	Linear		
psi	*psi*			*feet*	*inches*
0		22.5			
1.5–2.5	2	16.3	5.15	1.15	5.15% × 1.15 × 12 = 0.71
2.5–5	3.75	14.0	4.47	2.87	4.47% × 2.87 × 12 = 1.54
5–7.5	6.25	11.5	3.7	2.87	3.7% × 34.4 = 1.27
7.5–10	8.75	9.5	3.1	2.87	3.1% × 34.4 = 1.07
10–12.5	11.25	8.1	2.62	2.87	2.62% × 34.4 = 0.90
12.5–15	13.75	6.8	2.2	2.87	2.2% × 34.4 = 0.76
15–17.5	16.25	5.7	1.85	2.87	1.85% × 34.4 = 0.64
17.5–20	18.75	5.0	1.6	2.87	1.6% × 34.4 = 0.55
20–22.5	21.25	4.2	1.35	2.87	1.35% × 34.4 = 0.46
22.5–25	23.75	3.5	1.15	2.87	1.15% × 34.4 = 0.40
25–27.5	26.25	2.9	0.95	2.87	0.95% × 34.4 = 0.33
27.5–30	28.75	2.4	0.8	2.87	0.8% × 34.4 = 0.28
30–32.5	31.25	1.9	0.6	2.87	0.6% × 34.4 = 0.21
32.5–35	33.75	1.6	0.5	2.87	0.5% × 34.4 = 0.17
35–37.5	36.25	1.3	0.4	2.87	0.4% × 34.4 = 0.14
37.5–40	38.75	0.9	0.3	2.87	0.3% × 34.4 = 0.10
40–42.5	41.25	0.6	0.2	2.87	0.2% × 34.4 = 0.07
42.5–45	43.75	0.4	0.15	2.87	0.15% × 34.4 = 0.05
45–47.5	46.25	0.2	0.05	2.87	0.05% × 34.4 = 0.02
Total depth				52.81	Total P.V.R. due to swell = 9.67

are also used to plot curves in Figure 6 which pertains to potential vertical rise of a 33-foot layer of clay subgrade. Figures 7, 8, 9, and 10 are also plotted from data shown in Tables 2 through 6. Figures 7*A* and *B*, 8*B*, 9*B*, and 10*B* and *C* show the effects of surcharge weight such as might be found from heavy structures or layers of inactive overburden upon volumetric swell.

Figure 5 indicates that a pavement having a surcharge weight of 1½ psi constructed on a thick layer of clay soil has a potential vertical

TABLE 5

CONVERSION OF VOLUME CHANGE TO POTENTIAL VERTICAL RISE WHEN AN 18-INCH PAVEMENT OVERLIES A CLAY HAVING 15 PERCENT VOLUME SWELL IN AN UNLOADED CONDITION

Load	Average Load	Percent Swell		Depth of Layer	Vertical Movement
		Volume average	Linear		
psi	*psi*			*feet*	*inches*
0		15			
1.5–2.5	2	9.1	2.9	1 × 1.15 = 1.15	2.9% × 1.15 × 12 = 0.40
2.5–5	3.75	7.5	2.4	2.5 × 1.15 = 2.87	2.4% × 2.87 × 12 = 0.82
5–7.5	6.25	5.5	1.8	2.5 × 1.15 = 2.87	1.8% × 34.4 = 0.62
7.5–10	8.75	4.5	1.5	2.5 × 1.15 = 2.87	1.5% × 34.4 = 0.52
10–12.5	11.25	3.5	1.1	2.5 × 1.15 = 2.87	1.1% × 34.4 = 0.38
12.5–15	13.75	2.6	0.8	2.5 × 1.15 = 2.87	0.8% × 34.4 = 0.28
15–17.5	16.25	2.0	0.6	2.5 × 1.15 = 2.87	0.6% × 34.4 = 0.21
17.5–20	18.75	1.5	0.5	2.5 × 1.15 = 2.87	0.5% × 34.4 = 0.17
20–22.5	21.25	1.0	0.3	2.5 × 1.15 = 2.87	0.3% × 34.4 = 0.10
22.5–25	23.75	0.8	0.25	2.5 × 1.15 = 2.87	0.25% × 34.4 = 0.09
25–27.5	26.25	0.5	0.2	2.5 × 1.15 = 2.87	0.2% × 34.4 = 0.07
27.5–31	29.25	0.2	0.1	3.5 × 1.15 = 4.03	0.1% × 4.03 × 12 = 0.05
				Total depth = 33.88	Total P.V.R. due to swell = 3.71

TABLE 6

CONVERSION OF VOLUME CHANGE TO POTENTIAL VERTICAL RISE WHEN AN 18-INCH PAVEMENT OVERLIES A CLAY HAVING 7.5 PERCENT VOLUME SWELL IN AN UNLOADED CONDITION

Load	Average Load	Percent Swell		Depth of Layer	Vertical Movement
		Volume average	Linear		
psi	*psi*			*feet*	*inches*
0		7.5		0	0
1.5–2.5	2	3.5	1.1	1 × 1.15 = 1.15	1.1% × 1.15 × 12 = 0.15
2.5–5	3.75	2.4	0.8	2.5 × 1.15 = 2.87	0.8% × 34.4 = 0.29
5–7.5	6.25	1.4	0.45	2.5 × 1.15 = 2.87	0.45% × 34.4 = 0.15
7.5–10	8.75	0.8	0.25	2.5 × 1.15 = 2.87	0.25% × 34.4 = 0.09
10–12.5	11.25	0.5	0.15	2.5 × 1.15 = 2.87	0.15% × 34.4 = 0.05
12.5–15	13.75	0.3	0.10	2.5 × 1.15 = 2.87	0.1% × 34.4 = 0.03
15–16	15.5	0.1	0.05	1.0 × 1.15 = 1.15	0.05% × 13.8 = 0.01
				Total depth = 16.65	Total P.V.R. due to swell = 0.77

rise of ½ inch if the clay has 6½ percent volumetric swell. Figure 11 shows that this average amount of swell correlates with a plasticity index of 33. Figure 12 shows that thick layers of clay subsoils in Guadalupe County having a P.I. above 33 were unsatisfactory subgrade for 6-inch concrete pavement regardless of whether or not sub-base materials were used. Therefore, it is most likely that soil layers having a vertical potential rise of ½ inch or more are destructive to pavements or light structures.

In a similar manner it can be shown from Figures 6, 8, 9, and 10 that thin layers of clays having a vertical potential rise of ½ inch can have amounts of volumetric swell far in excess of 6½ percent.

Figure 11 indicates that the stability of soils with plasticity indexes below an average of 11 should be little affected by volume change and that soils having plasticity indexes of 33 would have an average of 6½ percent volumetric swell. The soils having a P.I. of 33 swelling from drier and wetter conditions would have 9½ and 4½ percent volumetric swells, respectively.

Data for theoretical points used in plotting curves shown in Figure 11 are summarized as follows:

Lab. No.	*LL*	PI	*SL*	Abs. Moisture Condition 0.47 *LL* + 2		Opt. Condition of Soils		Dry Conditions for Swelling Soils 0.2 *LL* + 9		Effective Percent Volumetric Swell	
				%*M*.	% vol. swelling*	%*M*.	% vol. swelling	% *M*.	% vol. swelling*	From opt. to abs. moisture	From dry moisture conditions to abs. moisture
39-59-MR	52	31	11	26	22	22	17	19	13	22 − 17 = 5	22 − 13 = 9
39-11-MR	74	45	12	37	32	30	25	24	19	32 − 25 = 7	32 − 19 = 13
46-420-E	79	55	8	39	40	31	31	25	24	40 − 31 = 9	40 − 24 = 16

* Taken from Figure 2.

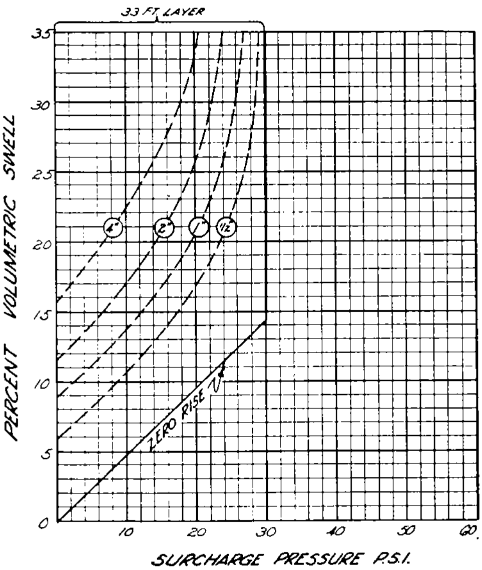

Figure 6. Potential vertical rise for underlying layers of soil 33 feet thick.

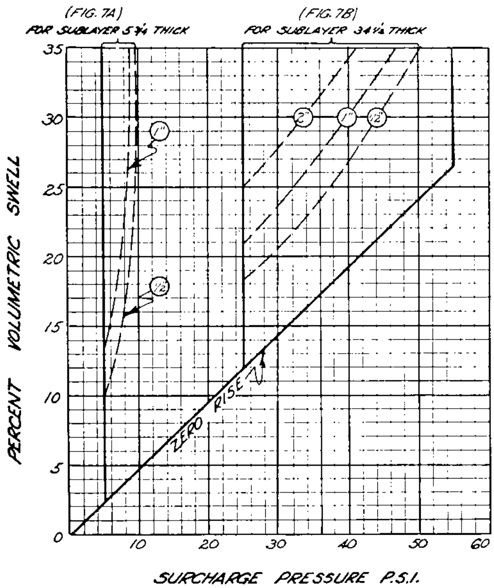

Figure 7. Potential vertical rise for some subsoil layers.

It was the writer's privilege to observe the performance of the previously mentioned experimental project on U. S. 90 in Guadalupe County (*2*) over the period 1933 to 1948 at which time modifications in sections dictated recovering all of the old concrete pavement with asphaltic concrete. Just before resurfacing the entire project, a condition survey of the 38 sections was made and results of that survey, together with minimum and maximum plasticity indexes of the top 36 inches of a deep subgrade soil, are shown in Figure 12. The sections classified as "poor" were patched extensively and those called "fair" were patched somewhat. Most sections classified as "good" were cracked but had good riding characteristics and were not covered with A.C. or patched. Figure 12 indicates that in general most sections having any subgrade with a P.I. in excess of 33 or an average percent volume swell of 6½ or greater (see Figure 11) did not perform well, the only exceptions being sections 9, 10, 11, and 12 which were 10-8-10 slabs. Sections 28 and 31 were also 8-inch slabs which were covered with asphaltic concrete. All other sections consisted of 9-6-9 concrete pavement slabs. Many other jobs have given similar performance where subsoil layers are thick. If slabs thicker than 6 inches are placed on thin layers of soil, the maximum allowable P.I. probably could be somewhat higher. A few of the many examples

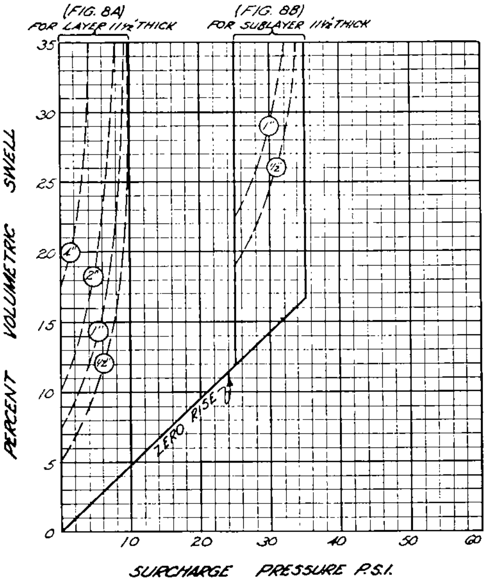

Figure 8. Potential vertical rise for some subsoil layers.

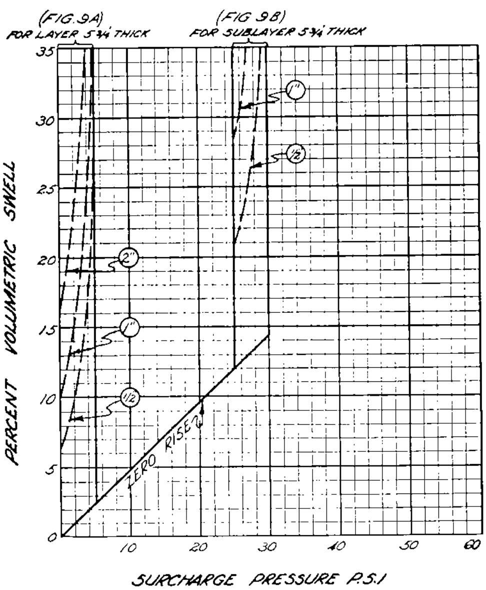

Figure 9. Potential vertical rise for some subsoil layers.

of 6-inch slabs which have performed well on thin clay layers with rocky substrata may be found on U. S. 67 south of Waxahachie, Texas; U. S. 81 near Jarrell, Texas, and U. S. 84 east of McGregor, Texas. These heavy traffic roads served well for many years on subgrades at

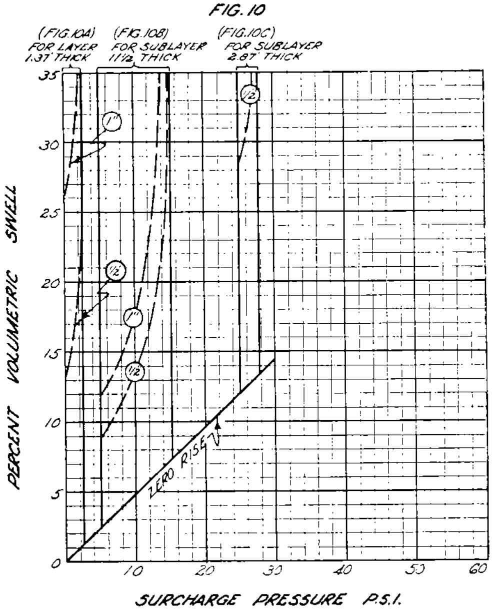

Figure 10. Potential vertical rise for several subsoil layers.

many places with P.I. ranging from 38 to 45. Moisture control during and subsequent to construction can also be very helpful in reducing movements particularly of clay soils with P.I. in excess of 30. In order to obtain maximum subsequent stability, the control of density for all soils is a well-known prerequisite. The author expressed his views on this subject in two reports to the Highway Research Board (*3, 4*).

Many 9-6-9 sections of concrete pavement in Texas built on deep strata of low volume change sand-clays have served satisfactorily for many years. Heavy loads have caused more cracks to appear than desired but generally they are still valuable pavements. Such pavements built on sand-clays underlaid by clay have not served nearly so well because P.V.R. values are in excess of ½ inch.

Over 20 years ago, the author served as water subgrade inspector on concrete paving projects in Shelby, Williamson, Lee, and Brazos Counties. A great deal of test data from these jobs was recorded and has been studied from time to time in conjunction with the service behavior of these projects. Results of these observations have been in line with the interpretations pertaining to potential

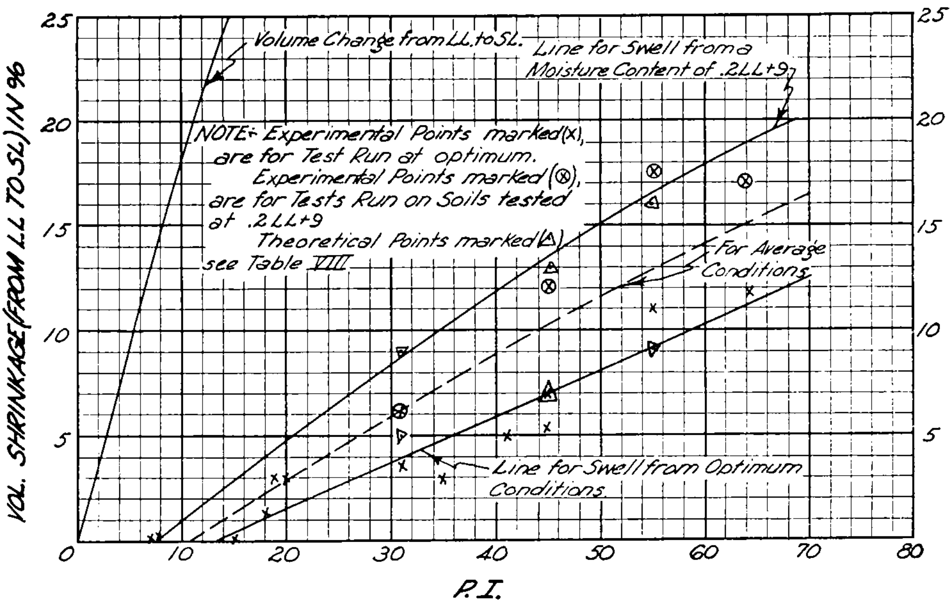

Figure 11. Interrelationship of P.I. and volume change. (Specimens subjected to swell under Ave. of 1 P.S.I. surcharge.)

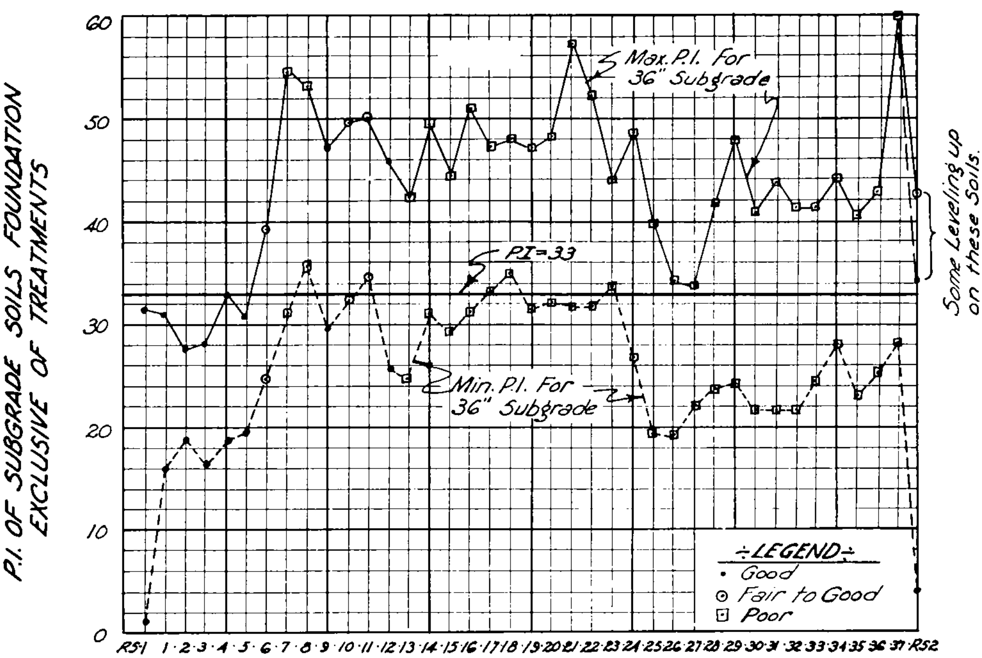

Figure 12. Relation of P.I. of subgrade of Guadalupe County concrete pavement sections.

vertical rise as described in this report. Additional data showing correlation between potential vertical rise and performance of structures are shown in this report under "Examples Showing Use of Potential Vertical Rise."

DETERMINATION OF POTENTIAL VERTICAL RISE

The following is a suggested outline for determination of potential vertical rise (P.V.R.).

1. *Thicknesses of Layers.* Determine thicknesses of soil layers, especially clay layers existing below structure. In the case of massive clay layers, the maximum depth to investigate will depend upon the position and amount of load proposed and the expansive characteristics of the clay (P.I., moisture content, and density). Usually, depths of 30 feet are sufficient for most situations.

2. *Expansion Characteristics of Soils.* Determine expansion characteristics of soil by use of one of the following three methods:

(a) The preferred way is to obtain undisturbed cores and subject them to capillary absorption and measure volume change as outlined in test procedure THD-80 of "Soil Testing Procedures" Nov. 1953, Texas Highway Department [or see (*5*)].

(b) Expansion characteristics from undisturbed cores after absorption also may be determined by use of the Harvard consolidation type of equipment provided specimens are trimmed to layers not to exceed ½ inch in height.

(c) By use of Figure 11, expansion characteristics of disturbed clay soils also may be estimated from their respective plasticity indexes provided portions of sample retained on the No. 40 sieve are considered to be inert on some proportional basis. Disturbed soils also may be remolded at desired or anticipated moisture-density conditions and tested in accordance with steps 2(a) or 2(b).

3. *Identification of Swell Curve Family Member.* After expansion characteristics have been obtained at some convenient surcharge load in accordance with step 2, the data may be entered in Figure 4 to identify the member of the swell family of curves to which the soil belongs. When this is done, it is possible to determine graphically the potential volume changes under almost any condition of loading from structure and/or surcharge.

4. *Loadings.* Calculate the unit loads from structure and soil of both the top and bottom of each clay layer involved. For this purpose, use of unit weight of structure plus hydrostatic pressures of soils will be preferable. The use of hydrostatic pressures in soils in this case is justified because if swell produces a rise, it must lift the weight of soil and water above the point which is swelling.

5. *Estimation of Volume Change.* Enter loadings found in step 4 on member of swell curve family found in step 3 in Figure 4. A summary of the difference of the range of volume change between the top and bottom of clay layer represents the total potential vertical rise only in terms of percent volumetric swell. Since the swell pressure relations are in the form of curves, increments of the above range showing percent volumetric swell should be tabulated as shown in Tables 2 to 6.

6. *Conversion of Volumetric Swell into Linear Swell.* For this purpose convert volume change values obtained in step 5 to linear volume change by use of Figure 1 and tabulate.

7. *Calculation of Potential Vertical Rise.* Multiply each thickness of layer by its respective percent linear swell. See Tables 2 to 6. A total of all increments is equal to the potential vertical rise.

EXAMPLES SHOWING USE OF POTENTIAL VERTICAL RISE

When architects began preparation of plans for Saint Paul Lutheran Church Parish Hall, they asked the writer if floors should be suspended from foundation or be supported by the subgrade soil. Being able to utilize subgrade support would result in considerable savings. An investigation of the site revealed that over half of the building was on a gravelly, sandy soil underlaid with chalk. The rear half of the building was located on a 3-foot thick layer of black clay having a plasticity index of 33 and underlaid by a thick stratum of very low-volume change chalk. By the use of Figures 4 and 11 (V_s = 6.5), the following tabulation was prepared (A).

Inasmuch as the above movements did not seem excessive and since the concrete floor of the Parish Hall was to be overlaid with a hardwood floor at the conclusion of construction, it was decided that the floor did not have to

(A)

Load	Average Load	Percent Swell		Depth of Layer	Potential Vertical Rise
		Average volume	Average linear		
psi	*psi*			*feet*	*inches*
0	0	8			
1.0*	1.0	6.5			
1.0*–2.5	1.75	4.0	1.3	1.5 × 1.15 = 1.72	1.72 × 1.3% × 12 = 0.27
2.5 –3.6	3.0	2.5	0.8	1.1 × 1.15 = 1.27	1.27 × 0.8% × 12 = 0.12
				Total depth = 2.99	Total P.V.R. = 0.39

* Weight of floor and sand cushion.

(B)

Load	Average Load	Average volume	Average linear	Depth of Layer	Potential Vertical Rise
psi	*psi*			*feet*	*inches*
0	0	11.0			
1*	1.0	9.5			
1.0–2.5	1.75	7.0	2.3	1.5 × 1.15 = 1.72	1.72 × 2.3% × 12 = 0.47
2.5–3.6	3.0	5.5	1.8	1.1 × 1.15 = 1.27	1.27 × 1.8% × 12 = 0.29
				Total depth = 2.99	Total P.V.R. = 0.76

* Weight of floor and sand cushion.

be suspended from foundation shafts if the subgrade was ponded.

Unfortunately, the subgrade became very dry and the contractor did not pond it with water. Therefore the problem was recalculated using the upper curve in Figure 11 for dry conditions as in (B).

It is interesting to note that at the conclusion of construction when the time came to place the hardwood floors, the floor sills in the rear of the building had to be reduced ¾ of an inch in thickness in order that a level floor could be constructed. After four years' service there are no other signs of movement. Ponding was employed in the construction of two adjacent buildings, neither of which has shown any signs of movement in a period of three years.

The Casis Community Center of West Austin, which was constructed in 1953, afforded another opportunity for the author to utilize the elements of this report as a basis upon which to make recommendations to the owner and contractor. The building (see Figure 13) with plate-glass fronts was to be built on a 15-foot-thick gravel bed which was underlaid with a thick layer of yellow clay out of the Del Rio geological formation having a P.I. of 45. The following computations were made (C).

(C)

Load	Average Load	Percent Swell		Depth of Layer	Potential Vertical Rise
		Average volume	Average linear		
psi	*psi*			*feet*	*inches*
0					
1*					
14†					
14–17.5	15.75	1.4	0.4	3.5 × 1.15 = 4.0	3.5 × 0.4% × 12 = 0.17
17.5–20	18.75	1.0	0.3	2.5 × 1.15 = 2.87	2.87 × 0.3% × 12 = 0.10
20–22.5	21.25	0.8	0.25	2.5 × 1.15 = 2.87	2.87 × 0.25% × 12 = 0.08
22.5–25	23.75	0.4	0.1	2.5 × 1.15 = 2.87	2.87 × 0.10% × 12 = 0.03
				Total depth active below gravel = 12.61	Total P.V.R. = 0.38

From Figure 11, % volume swell @ 1 # surcharge for average soil conditions = 10.
* Weight of floor and sand cushion
† Weight of floor and sand cushion plus weight of 15-foot gravel layer.

Since this project appeared to have nearly a ½-inch potential vertical rise, all of the steps for reducing volume swell suggested in this report were carried out and the concrete floor slab and sand cushion were placed on top of the gravel strata. After two years' service, only a few small cracks, probably caused by shrinkage of overhead concrete beams, have appeared in the rear corners of the buildings; otherwise, the buildings are performing perfectly.

Foundation footings and basement floor of

Figure 13. Parking area used to prevent drying of subsoil.

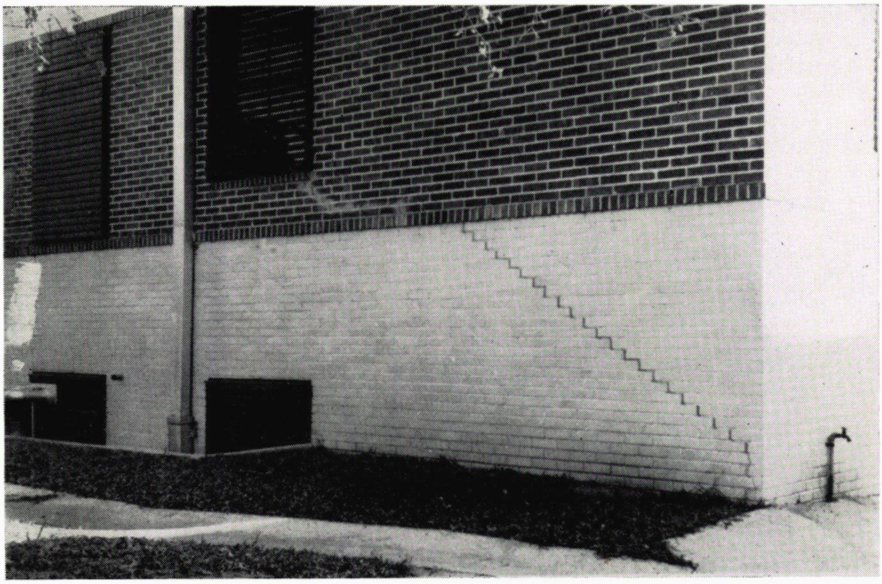

Figure 14. Showing cracks in T.H.D. Reproduction building.

the Texas Highway Department Reproduction Building were placed directly on a yellow clay soil out of the Eagle Ford geological formation having a P.I. of 60. Within two years' time, serious damage to floors, overhead beams, and brick walls was evident. Estimates of potential vertical rise, not tabulated in this report but calculated in the manner proposed, indicate that the P.V.R. for a light floor would be between 3 and 4 inches. There is evidence of this much movement (see Figures 14 and 15).

Figures 16 and 17 show evidence of distortion of a multiple box culvert over Mayhard Creek on U. S. 183, Travis County, south of Austin. The subsoil consisted of yellow clay from the Kemp Marl geological formation, having a P.I. between 30 and 40. This culvert was constructed after four, consecutive dry years. On the basis of dry condition curve in Figure 11, a P.V.R. of approximately 1½ inch should have been anticipated. It appears that ends of the multiple box culvert remained relatively free from movement, but the expansion span in the center of the structure probably moved upward because it acted as a hinge for the bridge.

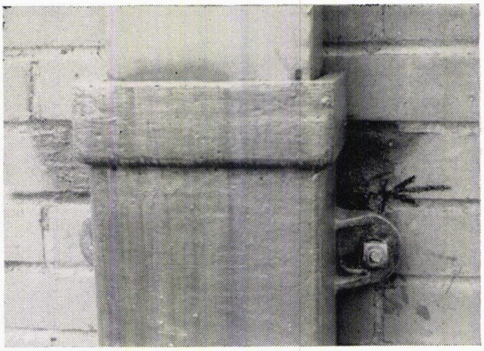

Figure 15. Showing drain spout bolt sheared from vertical movement of T.H.D. Reproduction building.

Engineers from the Portland Cement Association (*6*) ponded (for a long period of time) a small area of a dry bulb of the Wilson top soil over the Taylor formation in McLennan County on U. S. 81, north of Waco. The following tabulation lists calculations of potential rise for the soils at a locatioı on U. S. 81, 5.76 miles north of Waco. Since the soils were taken from an unpaved dry area, it was assumed that the soils were dry enough to

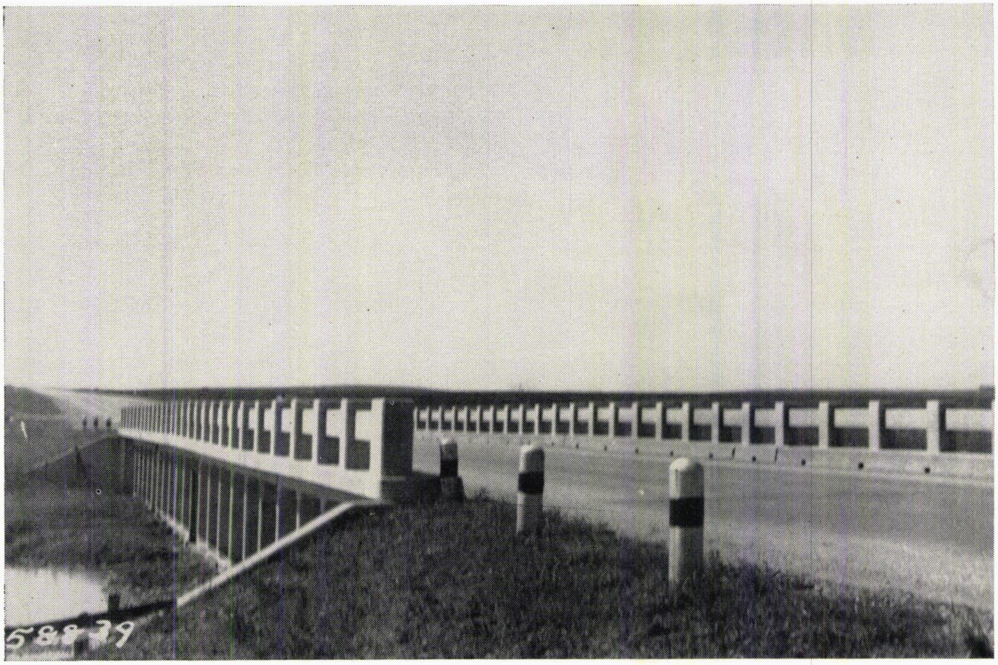

Figure 16. Showing multiple box culvert on U. S. 183 over Mayhard Creek. Note vertical rise at midstream.

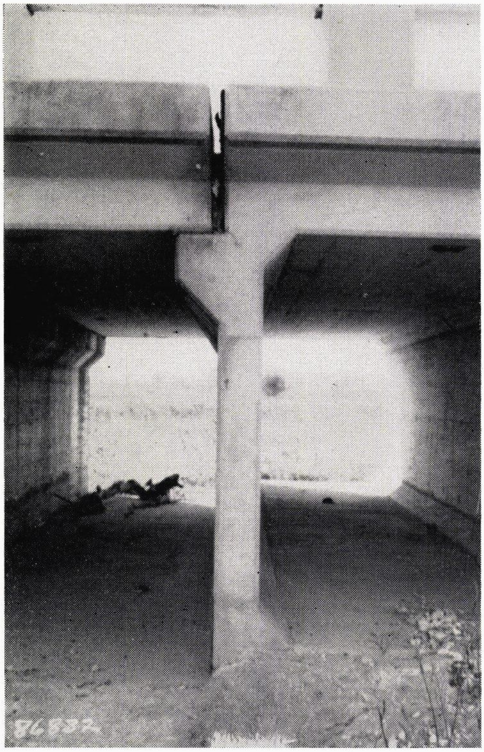

Figure 17. Showing opening of expansion joint at midstream of M.B.C. on Mayhard Creek.

justify using the upper line for P.I. versus swell in Figure 11 for the driest condition. Other sampling locations in the same vicinity indicated that the P.I. of subsoils below the 10-foot level was in the vicinity of 45. The following calculations were made (D).

The actual rise measured in the field from this operation was approximately 4 inches which is in close agreement with the above calculations.

A bridge consisting of 19 piers resting on spread footings was constructed in 1941–42 on U. S. Highway 183 over the Elm Fork of the Trinity River in Dallas County. Consolidation tests were run and size of footings designed so that not more than 1 inch of settlement would occur. Levels have been run at several different times over a period of time from 1941 to 1950 which indicate some of the piers rose instead of settling. Enough soil data were taken near Pier 18, which rose approximately ½ inch, to apply to the theory of this report. This pier was constructed on a 9′ by 9′ footing which was resting on a thick layer of clay soil having an LL = 65, P. I. = 40, and a moisture content of 21 percent.

From Figure 11, volume swell is 12 percent for P.I. = 40 and 1 # surcharge at dry condition of $0.2LL + 9$. Figure 4 shows family curve to be the 16 percent volume swell line (E).

Eighteen other piers were constructed on above project on various soils varying in P.I. from 25 to 50 at various moisture contents. Some piers settled to as much as 1½ inches. Others rose to as much as 1½ inches. A period of two to five years was required for completion of rise for this particular set of piers. Unfortunately, the soil and moisture data are

(D)

P.I.	Load	Average Load	Percent Swell		Depth of Layer	Potential Vertical Rise
			Average volume	Linear		
	psi	*psi*			*feet*	*inches*
20	0–2.5	1.25	5.0	1.6	2.5 × 1.15 = 2.87	1.6% × 2.87 × 12 = 0.55
20	2.5–5	3.75	2.5	0.8	2.5 × 1.15 = 2.87	0.8% × 2.87 × 12 = 0.28
20	5–7	6	1.5	0.5	2 × 1.15 = 2.30	0.5% × 2.3 × 12 = 0.14
30	7–10	8	3.5	1.1	3 × 1.15 = 3.45	1.1% × 3.45 × 12 = 0.46
45	10–12.5	11.25	5.0	1.6	2.5 × 1.15 = 2.87	1.6% × 2.87 × 12 = 0.55
45	12.5–15	13.75	4.0	1.3	2.5 × 1.15 = 2.87	1.3% × 2.87 × 12 = 0.45
45	15–17.5	16.25	3.4	1.1	2.5 × 1.15 = 2.87	1.1% × 2.87 × 12 = 0.38
45	17.5–20	18.75	2.5	0.8	2.5 × 1.15 = 2.87	0.8% × 2.87 × 12 = 0.28
45	20–22.5	21.25	2.0	0.6	2.5 × 1.15 = 2.87	0.6% × 2.87 × 12 = 0.21
45	22.5–25	23.75	1.6	0.5	2.5 × 1.15 = 2.87	0.5% × 2.87 × 12 = 0.17
45	25–27.5	26.25	1.2	0.4	2.5 × 1.15 = 2.87	0.4% × 2.87 × 12 = 0.14
45	27.5–30	28.75	0.8	0.3	2.5 × 1.15 = 2.87	0.3% × 2.87 × 12 = 0.10
45	30–32.5	31.25	0.6	0.2	2.5 × 1.15 = 2.87	0.2% × 2.87 × 12 = 0.07
45	32.5–35	33.75	0.4	0.1	2.5 × 1.15 = 2.87	0.1% × 2.87 × 12 = 0.03
45	35–37.5	36.25	0.2	0.1	2.5 × 1.15 = 2.87	0.1% × 2.87 × 12 = 0.03

Total potential vertical rise, 3.84.

(E)

Load	Average Load	Percent Swell		Depth of Layer	Potential Vertical Rise
		Average volume	Average linear		
psi	*psi*			*feet*	*inches*
0	0	16.0			
1		12.0			
20*					
20–22.5	21.25	1.3	0.4	2.5 × 1.15 = 2.87	0.4% × 2.87 × 12 = 0.13
22.5–25	23.75	0.8	0.25	2.5 × 1.15 = 2.87	0.25% × 2.87 × 12 = 0.09
25–27.5	26.25	0.7	0.2	2.5 × 1.15 = 2.87	0.2% × 2.87 × 12 = 0.07
27.5–32.5	30.0	0.3	0.1	5 × 1.15 = 5.75	0.1% × 5.75 × 12 = 0.07
				Total depth active below footing = 14.36	Total P.V.R. = 0.36

* Dead load of super structure, pier, overburden soil, and footing.

not complete enough to apply to calculation of movements at other piers.

Since it is a tedious and laborious job to carry out calculations like those shown in the various tabulations in this report, the author has developed a slide rule which enables engineers, architects, contractors, inspectors, etc., regardless of whether or not they are familiar with soils engineering, to rapidly perform the necessary calculations for P.V.R.

STEPS FOR REDUCING DETRIMENTAL EFFECTS OF SWELLING SOILS

When potential vertical rise calculations are ½ inch or less, most lightweight structures and pavements designed in accordance with good practice can be expected to perform satisfactorily. For flexible pavements to be designed properly, utmost care must be taken to prevent wheel load stresses from exceeding strengths of soils [see Texas Triaxial Method (*5*)].

In cases where potential vertical rise calculations are in excess of ½ inch, the following steps may be taken to reduce the detrimental effects of swelling soils:

1. Reduce swelling pressures by ponding the area in question for two or three weeks prior to placing of foundation or paving. In such cases, holes for foundations should be opened up after ponding but precautions should be taken to prevent drying. In the case of earth fills, moisture-density relations during and subsequent to construction should be controlled. Subsequent loss of moisture may be prevented by application of asphalt membranes, paving of parking lots, or placing layers of granular materials shortly after ponding or rolling.
2. Use steel reinforcing in both top and bottom portions of foundation grade beams.
3. Use a heavy gauge of reinforcing steel in foundation shafts and isolate walls of shafts from clays.
4. Leave an air space between bottom of foundation grade beams and the surface of the ground.
5. If possible, suspend floors from foundation shafts. If this is not possible, the floor should be separated from foundation grade beams by insertion of expansion joints.

CONCLUSIONS

The foregoing report makes it possible to utilize swell tests and soil constants in determining:

1. Whether or not a problem is one of shrink-swell or of settlement due to flow and consolidation.
2. Whether pavements on swelling clays will develop irregular riding surfaces from causes not attributable to the use of inadequate thicknesses or poor pavements, base and/or surfacings.
3. Whether or not lightweight structures, such as light buildings, box culverts, etc., will move excessively.
4. Whether or not to suspend floor slabs from foundation shafts or let the existing soil subgrade support them.

It is believed that this report enlarges upon the significance and usefulness of swell tests and the soil constants extensively.

The determination and use of potential vertical rise (P.V.R.) should be as commonplace as the performance and use of identification, consolidation, and stability tests. This simple term, P.V.R., summarizes the effects of all the following important variables:

1. Type of soil.
2. Density.
3. Moisture fluctuations.
4. Thickness and significance of sublayer.
5. Effect of surcharge load from structure and/or overburden.

ACKNOWLEDGMENTS

The writer is indebted to the many people who have contributed to and encouraged the development of this report. A few of these people or organizations have been mentioned in the text; however, it should be mentioned that the work of the members of the Soils Section and other members of the Materials and Tests Division of the Texas Highway Department, under the able guidance of Mr. A. W. Eatman, has been a major factor in making this report possible.

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DISCUSSION

Raymond C. Herner, *Chief, Airport Division, CAA Technical Development and Evaluation Center*—We are indebted to Mr. McDowell for an able and interesting discussion of problems associated with volume change in soils. While all of us recognize the existence of such problems, we find little in the literature to help solve them. By combining theory with long years of experience in dealing with critical soils, Mr. McDowell now suggests a relatively simple and practical approach to the normal design problem.

Universal application of this method requires the ability to forecast soil moisture fluctuation over a wide range of climatic and environmental influences. Mr. McDowell relates these fluctuations to the soil constants, particularly the liquid limit, and furnishes field data to support these relationships. It would appear desirable to supplement these data with similar information from other sources before accepting the suggested relationship for universal use. This is intended only as a word of caution, not as a criticism of this excellent and worthwhile paper.

Several years ago the writer had occasion to make a limited investigation of soil moistures under pavement and found, like Mr. McDowell, that the best correlation was with the liquid limits of the soils involved. Other investigators, however, have reported their most consistent correlation with the plastic limit or with optimum moisture, and usually have not even reported liquid limits of the soil. The proper committee of the Highway Research Board could make a real contribution by encouraging some research agency to gather together the soil-moisture data available from various sources and to attempt some universal method of correlation. It also might set up suggested standards for future investigators so that test data will be obtained on a common basis.

F. L. D. Wooltorton, *Planning Engineer, Public Works Department, Kenya Colony, Nairobi, Africa*—Mr. McDowell's paper is not only of the greatest interest but also is most opportune. The phenomenon discussed is undoubtedly responsible for one of the most prevalent of the yet-to-be-thoroughly-examined causes of road pavement failures. It is particularly active under roads constructed over clay soils subject to desiccation and is especially common in tropical regions. Its effects are sometimes most blatently obvious under even temperate climates after those rare droughts occasionally experienced in such areas.

It is probably far more responsible for road failures in temperate regions than is at present realized because desiccation, productive of later distortion, may occur in localized areas due to an enforced temporary change in soil climate not in any way connected with any climatic drought.

Mr. McDowell's approach to the solution of this problem of differential heaving is particularly interesting, as it runs parallel to that adopted in volume-change and pavement-distortion studies made by Wooltorton[1].

Before commenting on the paper, it is desired to refer to some of the results obtained during a study in pavement distortion the writer has been carrying out during the past three years.

A short section of a dual carriageway (pavement 21 in. thick and bituminized, carriageways 30 ft wide to a hanging-lie and separated by a 10-ft wide grassed median) constructed in East Africa over a so-called blackcotton soil (characteristics similar to those of sample 46-420-E) was precision leveled soon after construction and periodically releveled afterwards.

Data not yet completely plotted indicated that (*a*) Points on the center lines had moved sinusoidally with time, varying by as much as 0.75 in. above to 2 in. below their original level; (*b*) Edge movements were from 2 in. above to 3 in. below initial levels; and (*c*) A pivoting effect occurred whereby the curbs oscillated about a point some 7 ft in from each edge. The relative over-all movement about the pivot points (which also moved) reached maximum values of about 2¾ in. This up and down movement relative to the pivot points had already lead to evidence of stripping along the longitudinal pivot lines. The pivoting effect was more severe along the lower and drained carriageway edges than along the higher and undrained edges.

Relation of Volumetric to Linear Swell

It is not clear whether the data used for Figure 1 were obtained for disturbed samples in which the clay particles would probably form a random pattern (as in fills) or for undisturbed samples when the micelles might be oriented (as in cuts).

Should the material be fill, or contain clay of spherical or aggregated microstructure, the apparent assumption that linear changes are equal in the horizontal and vertical directions is likely true; but should the material be in cut and of a single-grained laminar microstructure, there is some evidence[1] that the two values may have a very different ratio.

Relation of Moisture Content to Volume Change

The curves of Figure 3 are of interest in view of the world-wide desire to be able to estimate the equilibrium or the maximum moisture content under a pavement and in view of the up-to-now little stressed desirability of knowing the likely minimum moisture content as well.

The latter is of value in estimating the important design consideration of whether incipient slaking is likely to occur. The author's endeavor to determine the minimum moisture content is the first seen in print.

England has advocated the suction-pressure method of determining the equilibrium value, whereas Wooltorton[1] has suggested it can be more easily obtained from density-moisture content change curves or, if a quick estimate obtained from density-moisture content change curves or, if a quick estimate is required, from the field moisture equivalent—an approach which can be supported on a pF basis. The approximate value would be 0.75 F.M.E. for more stable A-6 clays varying to 1.00 F.M.E. for those less stable A-7 clays for which the minimum moisture content may approach the appropriate shrinkage limit.

It should be kept in mind, when considering data of the nature of McDowell's results for volume changes, that volume change and moisture intake depend not only on the "soil constants" but also on density, any desiccation, and on the structure of the soil material.

When no external energy is applied, it appears[1] that the volume change per unit weight of dry soil, or the dry density change per unit volume increases as the moisture content decreases below optimum or as the percentage compaction at optimum decreases (i.e., as the percentage of air voids increases) and there is artificial desiccation, until, more generally, a maximum value is reached.

Minimum values appear to occur for minimum air voids content as in a sample prepared for shrinkage test. This explains the value of "ponding" referred to by McDowell as a measure for reducing detrimental volume change.

Structure, whether mechanically imparted, as in the compaction of fills, or whether built up by chemical and natural physical processes during aging, as in cut subgrades, affects volume change but these effects are too intricate to be perused here.

The inferences from considerations of such variables not normally studied are:

(*a*) Graphs of the nature of Figure 3 must invariably indicate appreciable point scatter.

(*b*) Graphs of the nature of Figure 2 may only be strictly applicable to fully saturated remoulded soil material or material acting as such, as some friable-when-dry A-7 clays.

Another cohesive soil variable which may be important in this problem in its general application, but which has not yet been considered by any known person studying such problems, is the change in the value of the in situ shrinkage limit with change in density and change in soil structure. An undisturbed soil, as in cut, may have a very much higher *S.L.* than that determined by accustomed test. For disturbed soils, as fills compacted under a given compaction effort, the active shrinkage limit appears to increase as the percentage compaction at optimum increases, reaching a maximum and approaching the test value for a percentage compaction slightly in excess of 100 percent. Despite its high shrinkage limit, an undisturbed soil may be much more resistant to

[1] "The Scientific Basis of Road Design", F. L. D. Wooltorton, Ed. Arnold, London ,1954.

desiccation forces than a disturbed soil because of its chemical water-resisting structure.

Swelling Pressure and Vertical Rise

As the various estimates of vertical rise have apparently been based on the use of Figures 1 and 4, it is not clear whether the approach used is applicable without modification to profiles which normally act as desiccated soils and generally contain appreciable air under all natural conditions of moisture content.

CHESTER McDOWELL, *Closure*—Additional studies of fluctuations of moisture contents certainly are in order. We attempted to correlate moisture absorption for non-granular soils with various of the soil constants and reached the same conclusion mentioned by Mr. Herner; namely, that the liquid limit correlated with moisture absorption more satisfactorily than any of the other soil constants.

In the case of coarse granular soils the *LL* is not nearly as indicative of moisture absorption as we would like for it to be; therefore, we are finding some indications that some selected optimum moisture contents are related to absorbed moistures. Under conditions contributing to high absorption, very stable base materials probably will absorb lesser amounts than optimum, intermediate good soils will absorb amounts about equal to optimum moisture, and plastic clays will absorb from 4 to 10 percent in excess of their respective optimum moistures. Therefore, it is suggested that a study of optimum moistures relation to absorption be made in connection with the project suggested by Mr. Herner.

Mr. Woolterton's comments and experiences are extremely interesting and are of much value to this subject. Points shown in Fig. 1 were obtained by testing specimen molded of disturbed soils; however, six out of seven examples showing correlation of theoretical movement to actual movement involved some undisturbed desiccated clays. Therefore, it is believed the proposed method is applicable to both disturbed and undisturbed soils. However, the question of particle orientation may be of greater importance than previously considered. It is believed that desiccated soils have a slower rate of absorption than most other soils but given enough time they will swell too. One of the reasons for using the term "Potential Vertical Rise" is because some such soils absorb moisture so slowly—cases of damage to structures due to high volume change soils in the deserts of Arizona and California have been frequently reported in the literature.

The author's principal concern about the swelling properties of desiccated soils is that the upper curve in Fig. 11 is probably too low for use in all cases. In such cases percent volumetric swell probably should be obtained from swell tests run on undisturbed samples. When soils are compacted to a low density and on the dry side of optimum moisture, they will shrink to a density which is less than that found at the standard shrinkage limit. When soils with such low densities absorb moisture, they will swell less than denser samples but they will have such low load-carrying capacities that they may be susceptible to flow and/or consolidation. It has been found that specimens compacted at or above optimum moisture will shrink to a density approaching that found at the standard shrinkage limit but none measured by us have exceeded it.