PAVEMENT CRACKING:

CAUSES AND SOME PREVENTIVE MEASURES

by

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The pavement cracking problem was investigated with respect to deep seated movements and also from the standpoint of "shrinkage cracking" due to the inherent properties of road building materials. In regard to the latter part of the report, some laboratory tests involving migration of lime into dried and cracked specimens were performed. Results indicated that the use of the proper per cent solids in lime slurry, recycling of treatment and reconsolidation all contribute greatly to the effectiveness of lime's outstanding ability to reduce volume change and increase strength in an unmixed cracked clay.

An early phase of this report establishes a theory that the degree of shrinkage cracking may be a function of certain strength characteristics of base and paving materials. The data given in the report for truly flexible base materials shows the compressive strengths of such materials to be many times greater than are their respective tensile strengths.

A shear diagram classification chart is presented which divides all soil materials, with or without stabilization, into three groups, two of which are susceptible to "shrinkage cracking" and one which is not.

A chart is presented which shows the relation of compressive strength to the ratio of compressive to tensile strength for materials with widely varying characteristics. A line is drawn on this chart which appears to separate materials believed to be highly susceptible to "shrinkage cracking" from those which are less susceptible.

> Recommendations are made relative to the use of water and lime to prevent deep seated swelling, adequate thicknesses of base to support loads, wide shoulders to prevent cracking of clay subgrades and stabilized materials which have relatively high ratios of compressive to tensile strengths. The use of high grade flexible base materials containing low amounts of fine silt are recommended for use in construction of flexible pavements in areas susceptible to frost damage.

THE CAUSES

There are many factors which contribute to the occurrence of cracks in pavements, some of which are summarized as follows:

1. Excessive load deflections including resilience deflections.
2. Subsidence, consolidation, slides, etc.
3. Swell-shrink conditions of the subgrade.
4. Shrinkage cracking of the base and/or pavement due to causes other than deflections or movements of subgrade.
5. Frost and/or freeze-thaw action.
6. Brittleness of pavement due to aging and/or absence of traffic. This includes the use of hard asphalts and their hardening due to oxidation. Type of mix and/or construction procedures may contribute to oxidation.
7. Thermal expansion and contraction.

It is intended that this report will concentrate primarily upon item Nos. 3 and 4 listed above, which includes volume change of subgrade and "shrinkage cracking" caused by the inherent properties of the materials in the base and pavement. This does not mean that these items are the only important factors involved because cracking may be caused by any one or a combination of the above items. The Author has written reports indicating how triaxial tests can be used to help prevent excessive load deflections (1, 2, 3, 4, 5)* and other reports have been presented in an attempt to provide for less

*References at end of report.

detriment from shrink-swell conditions, 6, 7 and 8. Data given in these reports will not be repeated here except to state that control of subgrade moisture content at time of evaporation cut off, use of thick flexible base or stabilized layers consisting of wide blanket sections have done much to reduce pavement cracking but we have not eliminated the problem. Perhaps the damage from freeze-thaw or frost (cause 5) to some flexible base materials can be about as devastating in certain portions of Texas as any other type of cracking. This is particularly true when many freeze-thaw cycles occur during wet winter months.

In areas adversely affected the pavement cracks into blocks and base material fines pump from beneath the surfacing. An investigation involving properties of minus No. 40 materials (too numerous to include in this report) from roads noted to have various degrees of frost susceptibility indicates the following:

1. That the maximum amount of minus No. 200 in the total material often is not indicative of freeze-thaw susceptibility. It has been found to be of considerable value when expressed as a percentage of the minus No. 40 material.

2. That the per cent minus 0.005 mm. material expressed as a percentage of the minus No. 40 material also shows promise of correlating with performance as affected by freeze damage susceptibility unless surfacings consist of surface treatment applications.

3. Although no one single requirement placed in specifications is going to solve this problem, it now appears that in cases where surfacings are to consist of Premix or HMAC, the minus No. 40 portion of unstabilized base materials produced in frost or freeze damage areas of Texas should not contain more than 25% minus 0.005 mm. sizes nor more than 55 per cent passing the No. 200 sieve.

Base materials used in such areas should be durable, have high triaxial strengths and should contain small amounts of fine size particles. Unless a thorough understanding of P.V.R. "Potential Vertical Rise" exists, it is difficult to know when "Deep Treatments" are necessary to prevent cracking. Considerable success has been obtained by ponding and sealing to where P.V.R. does not exceed ½-inch, but the ponding method is so time-consuming that it has not proven to be very popular for highway construction.

CLASSES OF MATERIAL AND THEIR SUSCEPTIBILITY TO CRACKING

In order to form a general concept about soil materials, the Mohr diagram classification chart, Fig. 1 (see AASHO T 212) is used as a means for dividing all soil materials into the following three groups:

1. Group I generally includes triaxial classes 4, 5 and 6. This group consists of yielding materials in which the application of increasing increments of normal stresses will not be accompanied by corresponding equal increments of shearing strength.

2. Group II also consists of yielding materials but they differ from Group I in that increasing increments of normal stresses are accompanied by increments of shearing strength which are greater than the increments of normal stress applied. Generally this group includes the base and subbase materials generally used in highway work which give little trouble from "shrinkage cracking".

3. Group III materials consist of cemented materials which have sufficient cohesion to form slabs. This group also has high shearing strengths but slab cracking occurs due to lack of sufficient cohesion

to resist shrinkage and/or load stress. For instance, Sowers and Vesic[9] have reported tensile stresses up to 60 psi. at the bottom of soil-cement slabs. Not very many stabilized soil slabs could be expected to be strong enough to resist such stresses. One example might be portland cement concrete continuously reinforced to reduce severity of cracking due to additional cohesive strength being supplied by the steel. This group of slab forming materials can be expected to be more or less susceptible to shrinkage cracking regardless of quality of subgrade. It appeared that tensile and compressive stresses and/or strengths might be pertinent to the problem.

RATIO OF COMPRESSIVE TO TENSILE STRENGTHS

In order to contribute to the subject of "shrinkage cracking" of pavements, it became necessary for the Author to conceive of theoretical as well as practical aspects of the problem. By use of the Mohr diagram of shearing stresses, certain theoretical concepts appear to be logical. For instance: When materials are capable of forming slabs (strengths sufficient to resist applied stresses) it is necessary that <u>sufficient cohesion exist</u> in order to resist tensile cracking. It is shown graphically in Fig. 2 how a given amount of cohesion may be maintained by varying compressive and tensile strength relations or ratios in which

$$P_1 T_1 = PT$$

Where:

P = Original compressive strength

T = Original tensile strength

P_1 = New or increased compressive strength

T_1 = New or decreased tensile strength

From this standpoint it seemed that the ratio of compressive to tensile strength, hereafter referred to as CTR, would be an interesting tool for use in analyzing this problem provided we could investigate the susceptibility to shrinkage cracking of a wide variety of materials. It was soon discovered that materials with widely varying strengths would have the same CTR ratios such as steel and soil, however, they had widely different compressive strengths so it was decided to separate such materials by plotting compressive strengths against the ratio of compressive to tensile strengths as shown in Fig. 3.

CTR RATIOS FOR A WIDE VARIETY OF MATERIALS

Fig. 3 shows some correlation between compressive strength and the ratio of compressive strength to tensile strength for a number of materials having a wide variety of physical characteristics such as: steel, portland cement concrete, epoxy-sand admixtures, soil cement mixtures, soil-lime mixtures, raw flexible base materials, a gumbo clay soil and a sand soil plus OA-90 asphalt mixture. A line is drawn on the chart which tends to separate the mixtures on the left which are susceptible to shrinkage cracking from those on the right which are less susceptible to shrinkage cracking. Tensile strengths were determined by use of cohesiometer (converted to psi.) for all materials except steel, P.C. concrete and epoxy-sand admixtures which were tested in tension. The data shown on this chart indicate that steel and soil have similar CTR values but widely different compressive strengths. It may also be noted that the sand admixtures containing 4, 6, 8 and 10 per cent cement indicate that six per cent may have been about the maximum of cement that should be used before shrinkage cracking becomes even more critical. The high CTR values obtained from cores and beams consisting of sand-shell-cement perhaps explains why these mixtures exhibit low amounts of shrinkage cracking in the field. Details of materials tested in this laboratory are given in Tables I through V inclusive. References relative to data obtained elsewhere are shown on Fig. 3.

Caliche lime materials which were notable for shrinkage cracking were also investigated with respect to moist curing 3 days before compacting. It may be noted that this procedure almost doubled the CTR value and very little compressive strength was lost due to delayed compaction.

Results of CTR values obtained by testing ten good crushed stone and/or caliche flexible base materials are shown in Fig. 3 which vary from 25 to 80. These findings are consistent with the theory that CTR points for materials exhibiting small tendency toward shrinkage cracking should fall to the right of the sloping line.

Compression-tension ratios for asphaltic concrete will fall to the left of the sloping line in Fig. 3 unless tests are run at low temperatures. Results of tests run at low temperatures on one mixture of sand plus OA-90 asphalt is shown on Fig. 3. Tests run on identical mixture at 140°F plotted on the left side of the sloping line. It is doubtful if shrinkage cracking of asphaltic mixtures occurs at such elevated temperatures. Additional testing along these lines is indicated, however, if tests could be run at pertinent temperatures, there is a possibility that the suggested minimum CTR values suggested in Fig. 3 might have application to bituminous mixtures.

MIGRATION OF LIME

Presently we hear a great deal about migration, pressure injection and deep mixing of lime for the purpose of improving volume change and strength characteristics of soils in Louisiana, Oklahoma and Texas. Although field results indicate some success from use of these methods, it would seem that some laboratory scale tests might be of some assistance in evaluating proposed treatments.

In an attempt to determine the effects of lime slurry migration, some experiments using specimens consisting of "Black Gumbo" soil were subjected to the following techniques:

1. Specimens 6-in. high by 6-in. diameter were molded in three layers at optimum moisture for 5.3 ft.lbs./cu.in. compactive effort (10 lb. ram, 18 in. drop).
2. Dried in a 140^{o}F oven 96 hours.
3. Lime slurry consisting of various percentages of solids is pulled downward through the cracks by use of vacuum pump (see Fig. 4) and recirculated through the specimen until cracks are sealed, thereby preventing further circulation of the slurry. At this time measurements are taken so as to calculate per cent volumetric swell.
4. Specimens are sealed in cells and moist cured for seven days.
5. Specimens are subjected to 20 days of capillarity either as is or after being reconsolidated to their original molding density.
6. After 20 days capillarity, specimens are measured for volume change and strength characteristics. In some instances the drying and lime slurry migration procedures are repeated before subjecting to capillarity.

Some specimens taken from step 3 were cut in half as shown in Fig. 4 and sprayed with phenolphthalein to show the effect of lime migration on the pH of the soil. See bands formed in Fig. 5. Results from a number of other tests are shown in Fig. 6 where per cent dry lime solids in slurry is plotted as abscissa and per cent volumetric swell and compressive strength are plotted as ordinates. The per cent volumetric swell shown is based on the dry volume of the raw soil specimen. Calculating volumetric change on the basis of oven dried volumes is perhaps much more severe than may be expected from field conditions. It is used in this case merely for comparative purposes and it may be noted in Fig. 6 that the extremely high volume change condition of the dry clay specimens was drastically reduced by the use of lime slurry in lieu of tap water. It is also of interest to note that recycling of the drying and slurry treatment procedure helps decrease swelling probably because greater amounts of lime were deposited in the specimen. The amount of lime deposited in specimens ranged from 0.9% to 2.1% for one cycle and 2.6% to 3.7% for two cycles. The amount of lime deposited and the resulting plasticity index indicated that the optimum per cent of solids in the lime slurry for this type of treatment appeared to be between 15 and 20 per cent.

Strength curves in Fig. 6 show that unconsolidated specimens subjected to 20 days capillarity had unconfined compressive strengths which were increased three to six fold; with reconsolidation to original molding densities, strengths were increased from four to eight fold. The ability of lime to improve the quality of soils without mixing is believed to be worthy of note.

Although the module consisting of small specimens representing a fairly long section of roadway is not ideal, it is believed that the results of these experiments strongly indicate that lime pressure injection and "Deep Mixing" of lime into jointed clays of the semi-arid regions had definite possibilities of reducing volume change and increasing subgrade support.

CONCLUSIONS

The results of this investigation indicate the following conclusions to be justified:

1. The data given in this report indicate that the relation of compressive strength to compression-tensile strength ratio have an influence upon shrinkage cracking of pavements.

2. The use of pressure lime injection and/or deep mixing appear to be effective for treating some jointed or cracked clays in semi-arid to arid regions for purposes of preventing excessive volume change and strength loss of subgrade soils. The feasibility of treating soils in this manner will depend upon unit costs which are not available at this time.

3. Optimum per cent of solids in lime slurry for injection purposes appears to be between 15 and 20 per cent.

4. Recycling of drying and lime migration procedures reduces swelling and improves subgrade support values.

5. Reconsolidation of lime injected soils increases supporting power of some subgrade soils greatly.

RECOMMENDATIONS

In order to prevent some of the potential hazards associated with pavement cracking, the following proposals are offered:

1. That adequate thicknesses of surfacings, bases and subbases be used so as to support the traffic loads for the life of the pavement desired.

2. That potential vertical rise be kept as low as possible, perhaps below ½-inch, and this may mean moisture control of clay subgrade prior and subsequent to subgrade rolling operations. This will probably involve ponding areas where layers are capable of producing considerable amounts of volume change and covering subgrade with suitable layer capable of retarding evaporation.

3. That swell-shrink conditions of clay subgrades be controlled by use of shoulders consisting of granular and/or stabilized soils which are wide enough to control moisture fluctuations.

4. That the soil binder portion of flexible base materials to be used in frost susceptible areas of Texas should not contain more than 25 per cent minus 0.005 mm. material.

5. That when feasible all base and pavement layers should be constructed out of materials whose ratio of compressive to tensile strength varies from a minimum of 11 for 1500 psi. compressive strength material to a minimum of 22 for 40 psi. material.

6. That highway research sections, especially in cuts, involving marls, jointed clays, etc. which have high swell potential be treated with several cycles of lime injection prior to paving. If fairly large differential movements due to volume change of soils are anticipated, especially such as

at grade points, fence lines, old road crossings, etc., it is recommended that serious consideration be given to use of the Oklahoma Deep Mixing process of lime treatment of subgrades.

ACKNOWLEDGMENTS

The writer is indebted to many who have contributed and encouraged the development of this report. 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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(listed in order of citation)

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7. "The Relation of Laboratory Testing to Design for Pavements and Structures on Expansive Soils." Presented at First Annual Soil Mechanics Conference, Colorado Section A.S.C.E. Colorado School of Mines, April 23, 1959. Published in C.S.M. Quarterly.

8. "Remedial Procedures Used in the Reduction of Detrimental Effects of Swelling Soils." WASHO June 1965, and Texas A&M September 1965. Publication undetermined as yet.

9. Sowers, G.F., and A.B. Vesic, "Vertical Stresses in Subgrades Beneath Statically Loaded Flexible Pavements", Bulletin 342, Highway Research Board, 1962.

Note: References 1 through 8 authored by Chester McDowell.
Other references for sources of data are shown on Fig. 5.

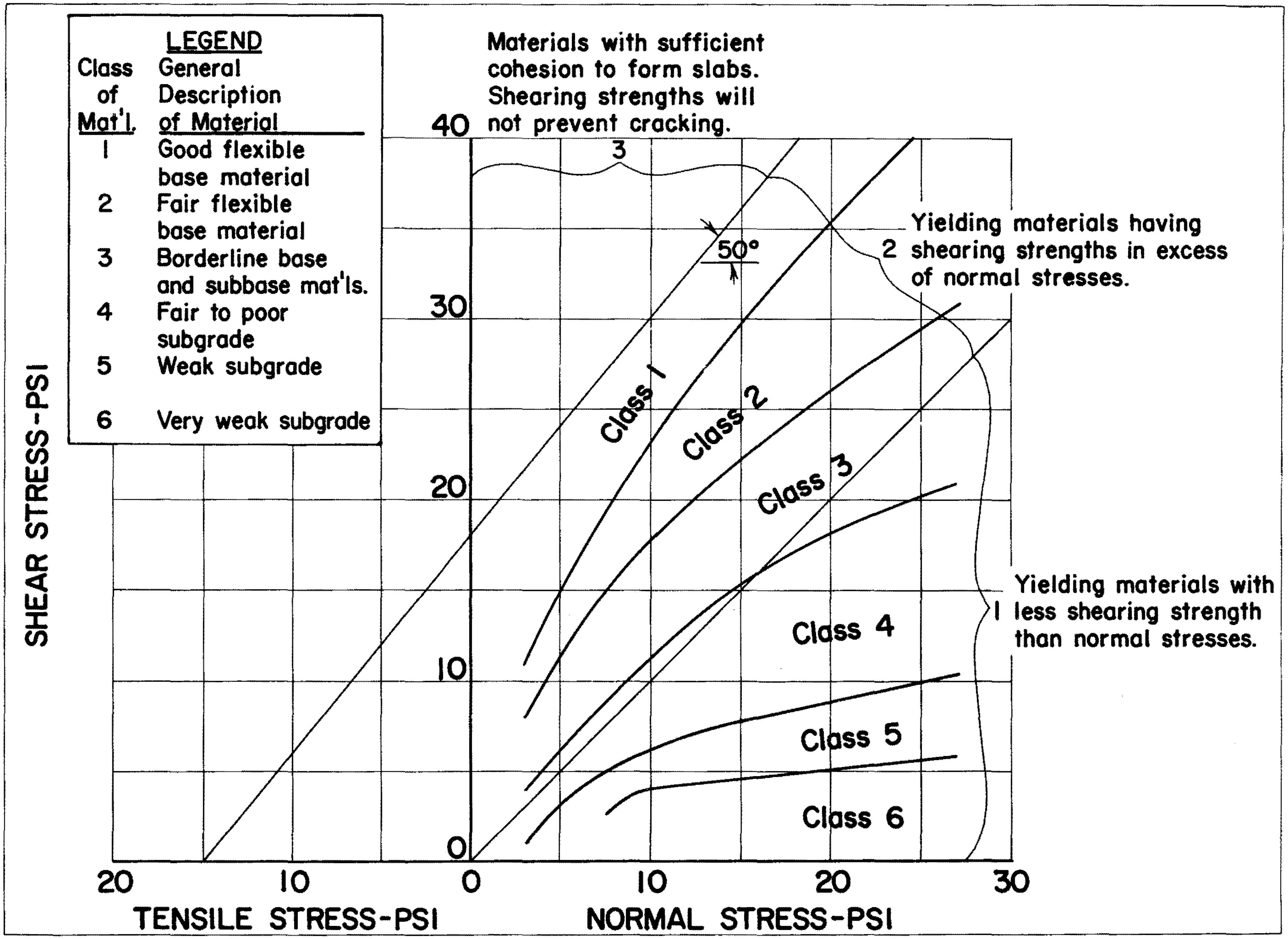

LEGEND
Class of Mat'l. | General Description of Material
1 Good flexible base material
2 Fair flexible base material
3 Borderline base and subbase mat'ls.
4 Fair to poor subgrade
5 Weak subgrade
6 Very weak subgrade
Materials with sufficient cohesion to form slabs. Shearing strengths will not prevent cracking.
3
50°
2 Yielding materials having shearing strengths in excess of normal stresses.
1 Yielding materials with less shearing strength than normal stresses.
Class 1
Class 2
Class 3
Class 4
Class 5
Class 6
SHEAR STRESS-PSI
40
30
20
10
0
20
10
0
10
20
30
TENSILE STRESS-PSI
NORMAL STRESS-PSI

Fig. 1

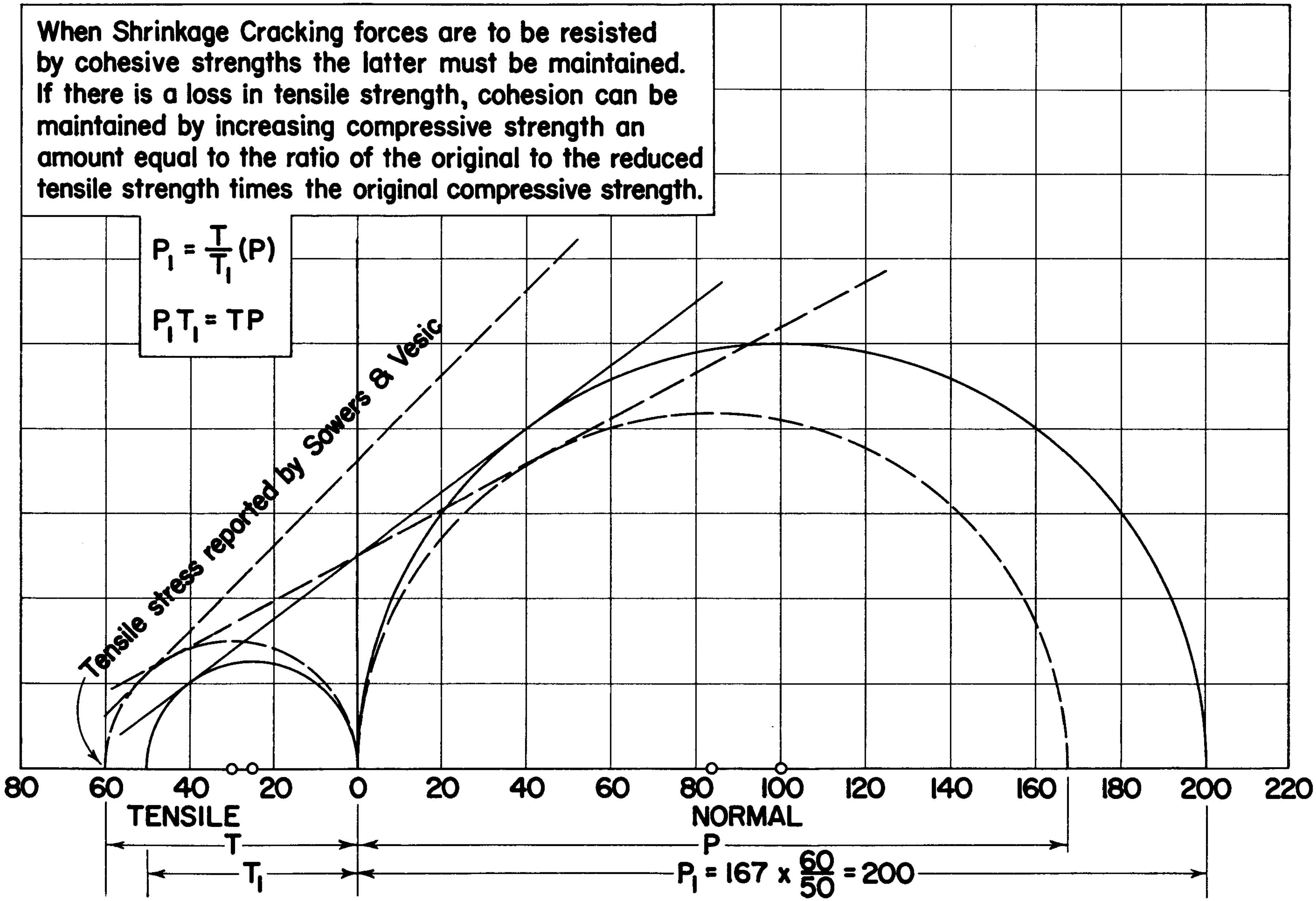

When Shrinkage Cracking forces are to be resisted by cohesive strengths the latter must be maintained. If there is a loss in tensile strength, cohesion can be maintained by increasing compressive strength an amount equal to the ratio of the original to the reduced tensile strength times the original compressive strength.
$P_1 = \frac{T}{T_1}(P)$
$P_1 T_1 = TP$
Tensile stress reported by Sowers & Vesic
80 60 40 20 0 20 40 60 80 100 120 140 160 180 200 220
TENSILE
NORMAL
T
T_1
P
$P_1 = 167 \times \frac{60}{50} = 200$

Fig. 2

RELATION OF COMPRESSIVE STRENGTH TO CTR

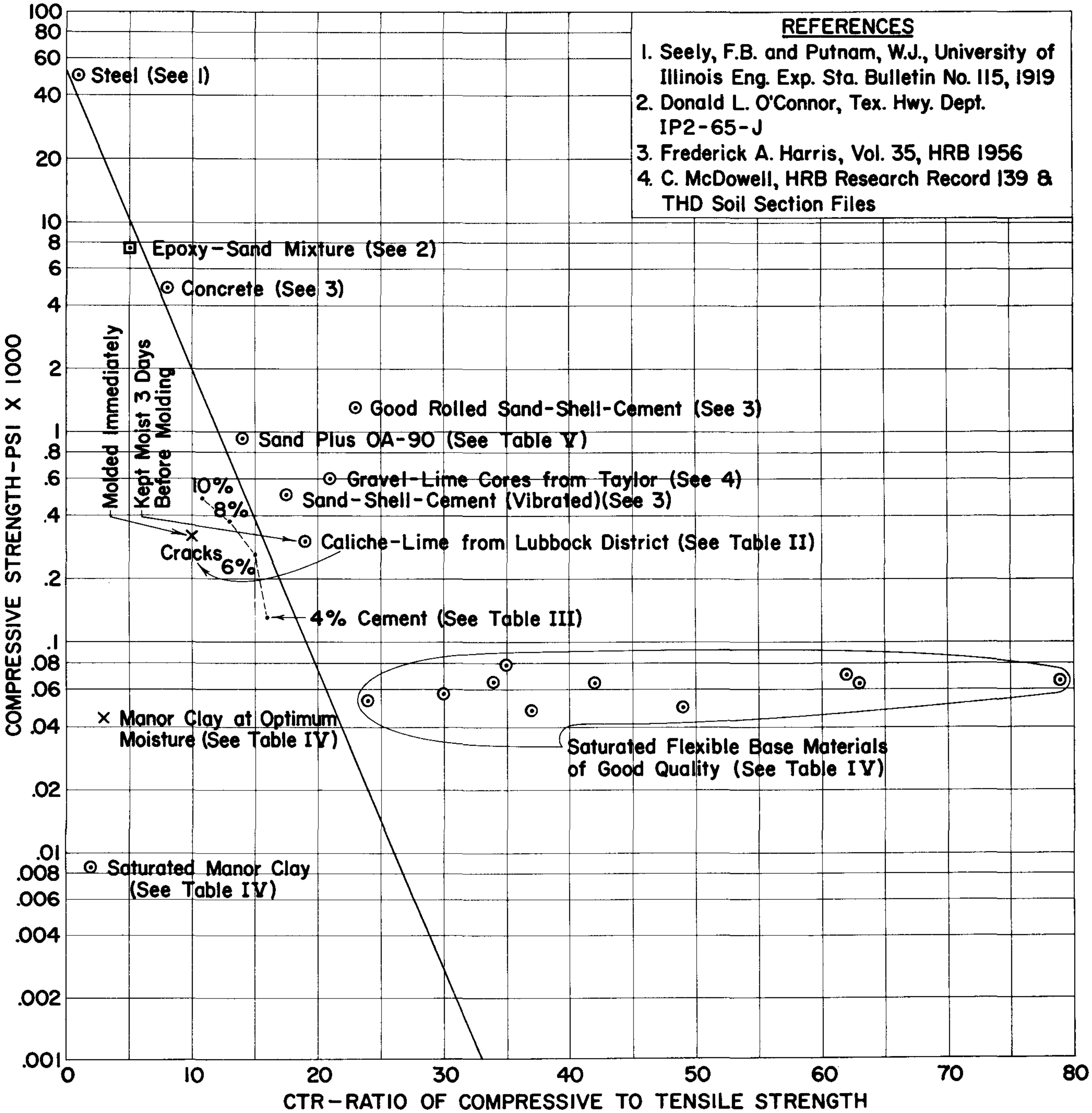


Fig. 3

LIME MIGRATION APPARATUS

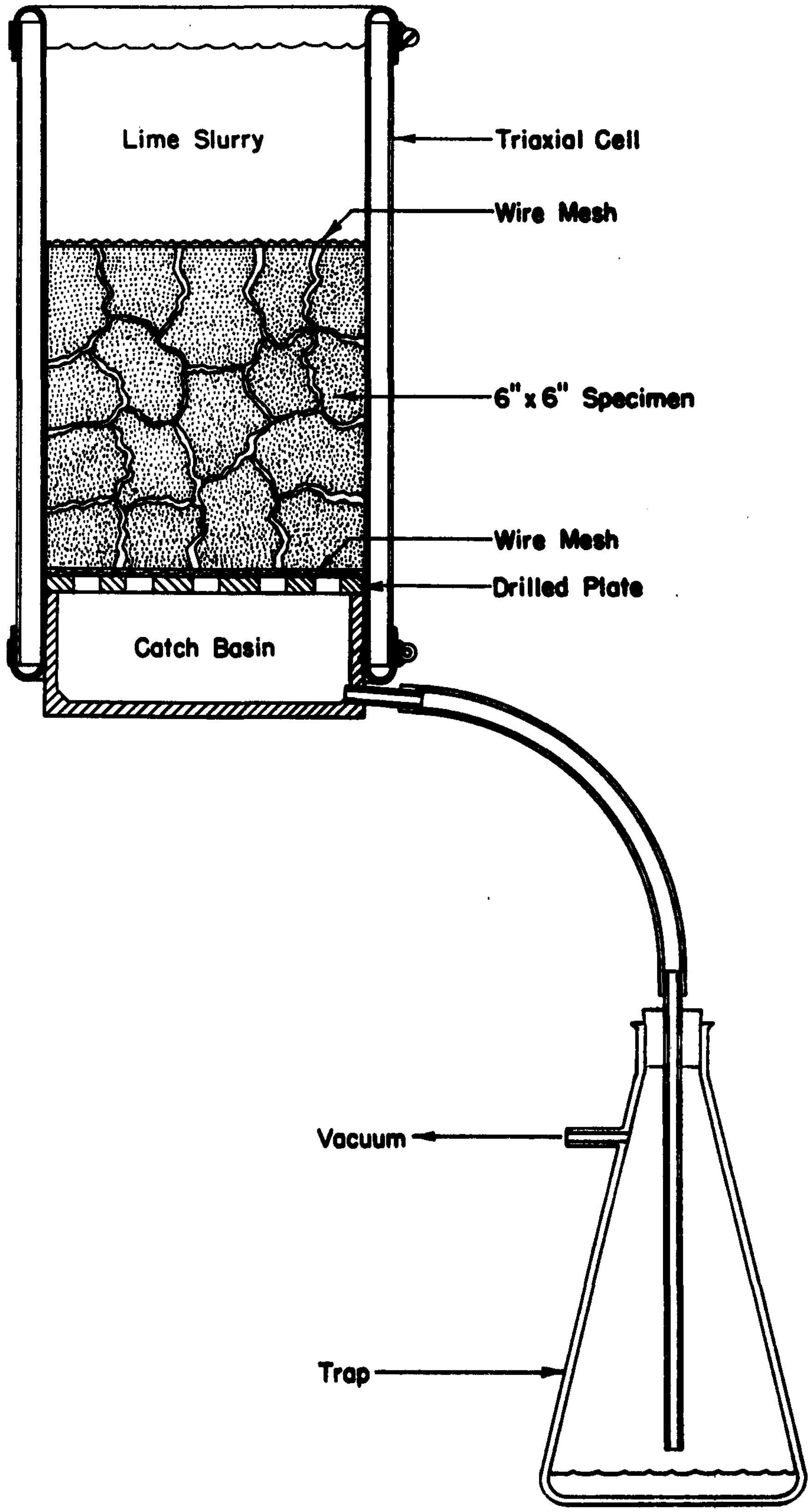
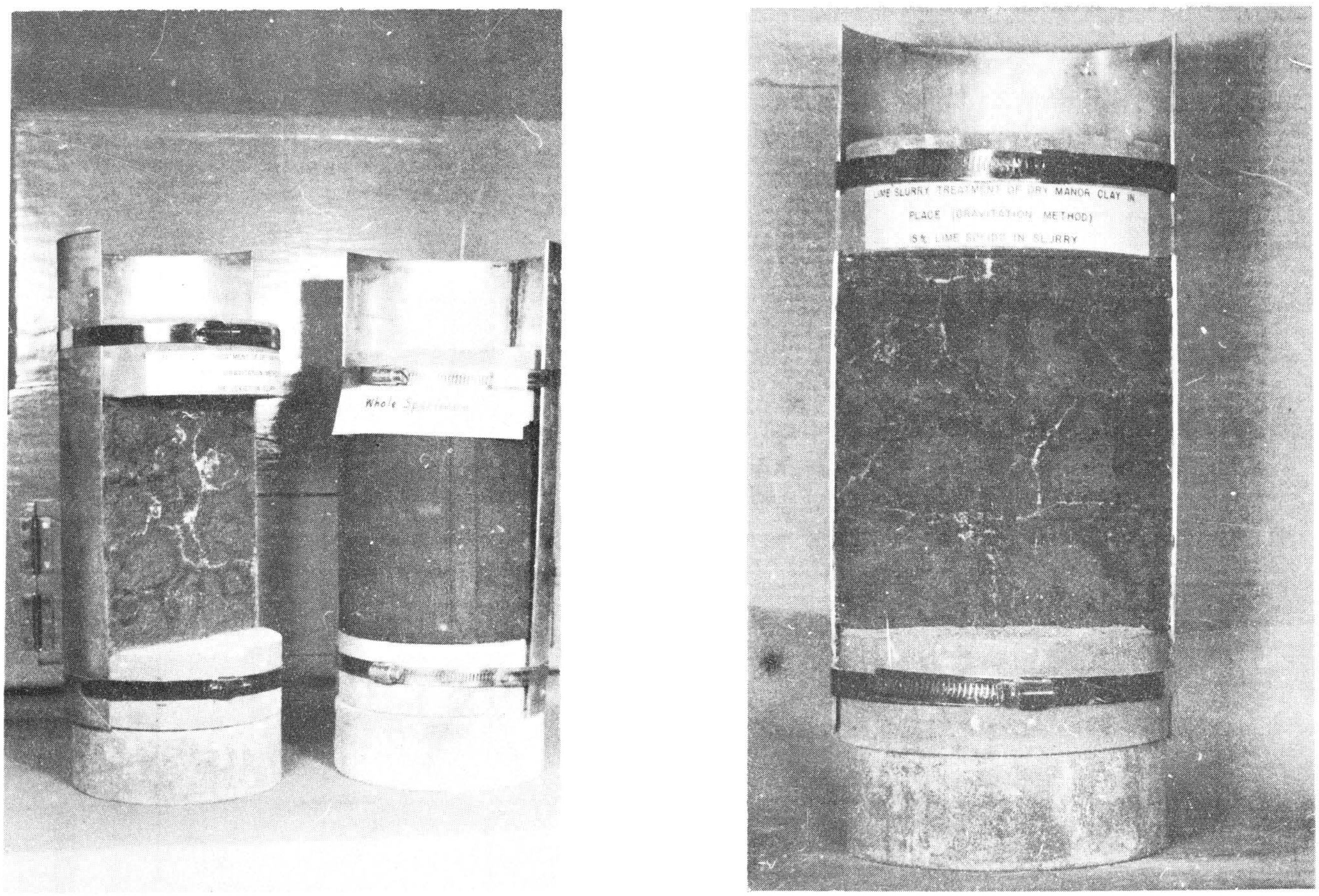


Fig. 4

HALF SECTION OF TREATED SPECIMEN SHOWING
LIME MIGRATION BETWEEN SPECIMEN CRACKS



Specimens clamped in metal half cylinders to facilitate slicing in half. Close-up at right shows surface of bisected lime treated specimen after spraying with phenolphthalein. Note that darkly shaded areas are much wider than the white streaks of lime, indicating that the pH of a considerable portion of the sample has been altered.

Fig. 5

LIME SLURRY TREATMENT OF MANOR CLAY (GRAVITATION METHOD)

Percent Volume Swell and Unconfined Compressive Strength in Relation to Percent Lime Solids in Lime Slurry

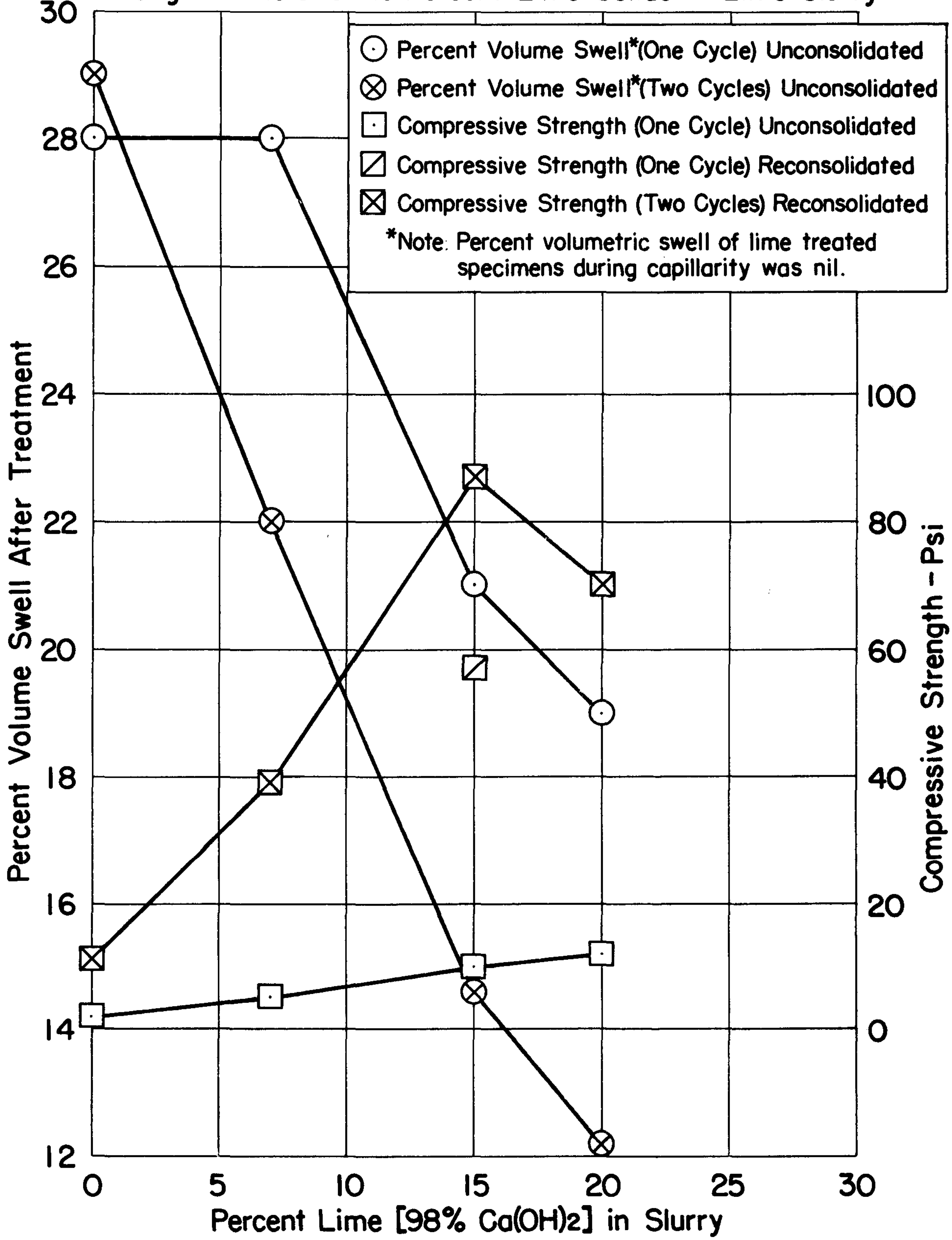


Fig. 6

TABLE I

TABLE SHOWING SOIL CONSTANTS AND GRADATION OF FLEXIBLE BASES AND SOILS

Lab. No.	LL	PI	SL	LS	SR	Soil Binder	W B M % Loss
63-282-R	21	7	14	4.4	1.92	17	33
64-459-R	19	5	14	3.7	1.93	21	32
65-67-R	30	9	20	5.0	1.68	29	37
65-68-R	28	11	17	6.3	1.78	20	32
65-100-R	24	9	15	4.7	1.89	14	19
66-48-R	24	5	16	4.4	1.79	24	33
66-49-R	22	6	15	3.7	1.86	20	35
66-50-R	28	6	21	3.5	1.57	22	34
66-169-R	33	8	24	4.4	1.60	19	35
66-171-R	21	6	16	3.3	1.86	21	32
62-375-E	21	3	16	2.4	1.66	97	
64-526-R	29	14	18	5.9	1.74	42	
66-248-R	25	5	21	2.3	1.69	96	
Manor Clay	70	41	11	20.0	1.93	100	

PERCENT RETAINED ON

<table>
<tr><th></th><th colspan="12">Square Mesh Sieves</th><th colspan="3">Grain Diam.</th><th></th><th></th></tr>
<tr><th>Lab.</th><th colspan="6">Opening in Inches</th><th colspan="6">Sieve Numbers</th><th colspan="3">In Millimeters</th><th>Spec.</th><th></th></tr>
<tr><th>No.</th><th>1-3/4</th><th>1¼</th><th>7/8</th><th>5/8</th><th>3/8</th><th>4</th><th>10</th><th>20</th><th>40</th><th>60</th><th>100</th><th>200</th><th>.05</th><th>.005</th><th>.001</th><th>Grav.</th><th>Material</th></tr>
<tr><td>63-282-R</td><td>0</td><td>9</td><td>28</td><td>40</td><td>57</td><td>70</td><td>76</td><td>80</td><td>83</td><td>85</td><td>87</td><td>89</td><td>90</td><td>93</td><td>98</td><td>2.70</td><td>Flexible Base</td></tr>
<tr><td>64-459-R</td><td>0</td><td>6</td><td>20</td><td>33</td><td>46</td><td>59</td><td>68</td><td>75</td><td>79</td><td>81</td><td>83</td><td>85</td><td>87</td><td>95</td><td>98</td><td>2.72</td><td>" "</td></tr>
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<tr><td>66-49-R</td><td>0</td><td>3</td><td>15</td><td>26</td><td>40</td><td>55</td><td>67</td><td>76</td><td>80</td><td>81</td><td>82</td><td>83</td><td>87</td><td>94</td><td>98</td><td>2.63</td><td>" "</td></tr>
<tr><td>66-50-R</td><td>0</td><td>5</td><td>17</td><td>28</td><td>42</td><td>56</td><td>68</td><td>74</td><td>78</td><td>82</td><td>86</td><td>90</td><td>96</td><td>97</td><td>99</td><td>2.64</td><td>" "</td></tr>
<tr><td>66-169-R</td><td>0</td><td>4</td><td>14</td><td>26</td><td>41</td><td>54</td><td>69</td><td>77</td><td>81</td><td>83</td><td>86</td><td>88</td><td>89</td><td>97</td><td>98</td><td>2.71</td><td>" "</td></tr>
<tr><td>66-171-R</td><td>0</td><td>3</td><td>13</td><td>24</td><td>37</td><td>53</td><td>64</td><td>74</td><td>79</td><td>82</td><td>84</td><td>86</td><td>87</td><td>93</td><td>97</td><td>2.74</td><td>" "</td></tr>
<tr><td>62-375-E</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>0</td><td>3</td><td>13</td><td>47</td><td>81</td><td>87</td><td>94</td><td>96</td><td>2.64</td><td>Sand</td></tr>
<tr><td>64-526-R</td><td>0</td><td>10</td><td>22</td><td>29</td><td>34</td><td>42</td><td>50</td><td>55</td><td>58</td><td>61</td><td>70</td><td>80</td><td>81</td><td>89</td><td>94</td><td>2.63</td><td>Flexible Base</td></tr>
<tr><td>66-248-R</td><td></td><td></td><td></td><td></td><td></td><td></td><td>0</td><td>1</td><td>4</td><td>25</td><td>68</td><td>77</td><td>79</td><td>84</td><td>87</td><td>2.67</td><td>Subgrade Soil</td></tr>
<tr><td>Manor Clay</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>0</td><td>1</td><td>4</td><td>8</td><td>9</td><td>45</td><td>59</td><td>2.71</td><td>Subgrade Soil</td></tr>
</table>

TABLE II

COMPRESSION AND COHESIOMETER TEST RATIO

SAMPLE NO. 64-526-R	PERCENT STABILIZER (Lime)	CURING TIME (Days)	COMPRESSIVE STRENGTH (PSI)	DRY DENSITY P.C.F.	COHESIOMETER VALUE (PSI)	RATIO (CTR)	AVERAGE RATIO (CTR)
4	2	21	336.9	120.7		10.8	10.8
1	2	21		121.2	31.1		
6	2	21	345.7	120.6		10.8	
2	2	21		121.3	32.0		
*21	2	21	298.2	118.8		19.0	19.1
* 8	2	21		119.5	15.7		
*25	2	21	318.4	118.9		19.2	
*13	2	21		120.1	16.6		
11	4	21	316.5	119.2		9.6	10.2
3	4	21		120.0	33.0		
12	4	21	330.7	119.4		10.7	
4	4	21		119.8	31.0		
*29	4	21	311.1	117.5		14.5	14.0
* 9	4	21		119.0	21.5		
*30	4	21	312.0	117.5		13.4	
*10	4	21		118.3	23.2		
18	6	21	320.0	117.5		14.3	14.2
5	6	21		119.9	22.4		
19	6	21	341.0	117.5		14.1	
6	6	21		119.7	24.1		
*35	6	21	284.5	116.8		12.7	12.8
*11	6	21		117.4	22.4		
*37	6	21	273.8	116.7		12.8	
*12	6	21		118.1	21.4		

*(Moist. cured 3 days before molding)

Specimens are 6" x 8" molded with equipment described in AASHO T 212 and using a compactive effort of 50 ram blows per layer. (Ten pound segment of a circle hammer dropping 18")

TABLE III

COMPRESSION AND COHESIOMETER TEST RATIO

SAMPLE NO. 62-375-E	PERCENT STABILIZER (Cement)	CURING TIME (Days)	COMPRESSIVE STRENGTH (PSI)	DRY DENSITY P.C.F.	COHESIOMETER VALUE (PSI)	RATIO (CTR)
	4	7	122.0	105.2	7.5	16.3
	6	7	259.0	107.2	18.2	14.2
	8	7	387.0	108.7	29.9	12.9
	10	7	477.0	110.2	44.0	10.8

Specimens are 6" x 8" molded with equipment described in AASHO T 212 and using a compactive effort of 25 ram blows per layer. (Ten pound segment of a circle hammer dropping 18")

TABLE IV

COMPRESSION AND COHESIOMETER TESTS
FOR MANOR CLAY AND FLEXIBLE BASE MATERIALS

LAB. NO. OF SOIL	DRY DENSITY (P.C.F.)	COHESIOMETER VALUE (PSI)**	UNCONFINED COMPRESSIVE STRENGTH (PSI)*	CTR RATIO Nearest Whole No.
Manor Clay Saturated	92.2	4.2	8.5	2
Manor Clay @ Optimum	94.0	14.4	43.8	3
63-282-R	135.6	1.0	62.5	63
64-459-R	139.9	1.3	47.7	37
65-67-R	135.4	1.9	56.4	30
65-68-R	137.6	1.9	64.5	34
65-100-R	149.7	1.0	49.2	49
66-48-R	143.7	1.5	63.2	42
66-49-R	138.1	2.2	52.1	24
66-50-R	125.9	0.8	63.5	79
66-169-R	131.1	1.1	67.9	62
66-171-R	136.4	2.2	76.2	35

*Unconfined compressive strengths obtained from specimens molded and tested according to AASHO T 212.

**Cohesiometer specimens molded by gyratory compactor to comparable moistures and densities in all specimens except Manor Clay.

TABLE V

COMPRESSION AND COHESIOMETER TESTS
FOR HOT MIX ASPHALTIC MATERIALS

LAB. NO. OF SOIL	DRY DENSITY (P.C.F.)	COHESIOMETER VALUE (PSI)	UNCONFINED COMPRESSIVE STRENGTH (PSI)	CTR RATIO (Nearest Whole No.)
66-248-R	136.1	65.3	937.5	14
+ 8½% OA-90		*	**	

*Average of 6 tests @ 74^{o}F

**Tested @ 40^{o}F