

TEXAS HIGHWAY DEPARTMENT

Austin 26, Texas  
November 15, 1948

ROAD DESIGN CIRCULAR NO. 11-48

SUBJECT: FREEZE DAMAGE IN FLEXIBLE PAVEMENTS

TO: ALL DISTRICT AND RESIDENT ENGINEERS

Gentlemen:

Attached for your information is copy of a report on "Freeze Damage in Flexible Pavements".

This report is a compilation of findings from investigation of the roads that were reported by the District Engineers as sections of highways where major freeze damage had been experienced during the winters of 1946-47 and 1947-48.

We wish to express appreciation for the cooperation and assistance of the District and Resident Engineers, and to compliment the personnel of the maintenance forces for the excellent repairs that were rapidly made to these damaged sections of our highways.

Sincerely yours



D. C. Greer  
State Highway Engineer

TEXAS HIGHWAY DEPARTMENT

ROAD DESIGN DIVISION

FREEZE DAMAGE IN FLEXIBLE PAVEMENTS

by

H. P. Carothers, Road Design Division

In February, 1948, the Road Design Division requested the District Engineers to report sections of highways where major freeze damage had been experienced during the winters of 1946-47 and 1947-48.

Of 18 Districts offering comments with the freeze damage reports, 10 indicated inferior base materials, with Districts 1 and 18 indicating the chalks, District 6 the gypsums, and District 15 the porous, low shrinkage ratio bases. 5 Districts pointed out the fact that heavy loads are causing much of the damage. Five Districts indicated that single asphalt surfaces leaked. Six Districts in the Arid to Sub-Humid Regions pointed out dry weather cracks and raveling of dry-brittle asphalt. Two Districts indicated the old heavy triple successful and three Districts indicated need for thicker surface. Two Districts reported unsound concrete pavement. Two Districts reported freeze damage in bases that meet the present specifications. Districts 4 and 5 reported that the bases over the fine sandy subgrades tended to freeze more quickly and more severely.

We have investigated most of the reported sections as to cause of the freeze damage and this report may be considered as a compilation of the findings of the Districts plus correlation by the writer with the general data available on materials subject to freeze damage and with geologic and climatic conditions of Texas.

There is little evidence of widespread freeze damage to concrete pavements, however, damage was reported for three unsound concrete pavement projects; namely, (1) U.S. Highway 75 near Conroe, (2) U.S. Highway 80 East of Marshall, and (3) U.S. Highway 67 between Mt. Pleasant and Omaha. A preliminary examination under microscope by the Bureau of Economic Geology shows formation of crystals in the concrete, which indicate unsound concrete possibly subject to freeze damage. They are map-cracked and being very seriously damaged by overload. Further investigation will be made on these pavements in connection with a future report on unsound concrete.

No evidence of "frost boils" in the subgrade of concrete pavements has been reported or noted in this investigation. The meager data available on frost penetration in Texas indicates that the freeze is not deep enough to cause such "frost boils". The "pumping joints" in concrete pavements noted would probably have occurred just the same from a long cold rainy spell.

All serious freeze damage occurred in the flexible bases, except that in the Panhandle it was reported and evidence indicates that freeze damage may also penetrate to the subgrade, under the edges of the thin, frozen flexible bases.

The attached map of Texas (Map No. 1) shows the sections of highways reported with serious freeze damage, the average annual frost penetration, the climatic regions, the 500' and 3000' and part of 2000' elevation contour lines, and the major geologic divisions which have a direct relation to base materials subject to severe freeze damage. The data available on frost penetration is limited and somewhat conflicting.

Map No. 1 shows data available from two sources; namely, from a small map of the U.S. published by CAA and from a small map published by the U.S. Weather Bureau. However, this data fits quite well with the depth and location of the freeze damage reported. From all available data, we have drawn a dash line on Map No. 1, north of which it is recommended that design include specific prevention of freeze damage in flexible pavements.

Map No. 2 shows the "Climatic Regions of Texas" and the average annual rainfall.

Map No. 3 shows the "Average Annual Snowfall of Texas" and a few typical figures showing total snowfall for the winters of 1946-47 and 1947-48. Districts 5, 6 and 22 reported more freeze damage for the winter of 1946-47 than for 1947-48. It will be noted that for the stations reported the snowfall was well above average in these Districts for '46-'47, and below average in Districts 5 and 6 for the winter of '47-'48. Thus, in the north half of the Sub-humid to Arid Regions, snowfall appears to be the major source of moisture for freeze damage.

Map No. 4 shows the average dates of the first and last killing frosts. It has been reported repeatedly and there is ample evidence showing that asphalt surface treatments and cold laid asphaltic concretes should not be placed out of season. Without proper curing, there are too many that are porous and thus assist in feeding excess moisture into the base - resulting in freeze damage.

Map No. 5 shows approximate seasons for placement of asphalt surfacing. The recommended seasons start a few days after the last killing frost in the spring, and ends from 30 to 45 days before the first killing frost in the fall. These dates are for surface treatments and cold laid asphaltic concrete. It appears that hot mix-hot laid asphaltic concrete may be placed about 30 days later than the dates shown.

The damaged projects reported appear to range from 100% freeze damage to 100% overload and moisture damage; however, of about 300 sections of roads reported, only about a dozen appear to have trouble not clearly including freeze damage. Therefore, there is ample evidence that our flexible bases must be designed to resist freezing as well as overstress in the presence of moisture. Design to resist freezing will have to be applied particularly north of the general location of the 1" average annual frost penetration line.

It is desired to discuss the aspects of freeze damage under two general heads; namely, (1) the prevailing concepts of flexible base material and (2) the physical causes of freeze damage.

1. The prevailing erroneous concepts we have of flexible base which admit materials subject to freeze damage are:

(a) The prevailing concept that base materials may be controlled by soil constants of the binder (which after rolling and weathering should not exceed 10 to 30 percent of the base) without any control on the durability of the aggregate in the base - which should constitute the main body of the pavement. This is, of course, a very old fashioned concept of flexible base. Excess, porous fines are particularly susceptible to freeze damage.

(b) The usual concept of adequate base and specifications therefor in Texas were derived from observation and tests on pavements from just one year to about seven years old. It takes longer to prove that a pavement is good. The caliche specifications were set up under much lighter traffic conditions than prevail today

and were based on roads too young to classify and at least half or two-thirds of the bases called caliche are actually other types of material. The last two winters have merely placed the cold finger on the fact that too many of our flexible pavements are built of poor base materials. A revised concept of specifications for good base material is certainly needed.

(c) The general inclination is to place the blame for cracking and failure of flexible pavements on the asphalt or aggregate in the wearing surface. When a building cracks, the first thing to check is the foundation. The same thing applies to an asphalt wearing surface that is cracking - check the base for excessive deflections or for excessive shrinkage and swelling from moisture changes and freeze and thaw.

(d) The desire to produce a very smooth base that will allow placement of a very thin surface treatment, results in many cases in a laitance of fines between the aggregate of the base and the asphalt surface. This laitance of fines results in poor bond between the base and surfacing and in addition is susceptible to freeze damage.

(e) In many cases we process and test the base material in a rather gentle manner; whereas, in rolling, weathering and stressing under traffic, the material is really treated rough - which may result in having a base that is in no way similar to what we expected from the test results. After rough treatment, an excess of fines, susceptible to freeze damage may be produced. We should, at least, test the base material after rolling.

(f) In addition to the present maximum P.I. requirements of 12 to 15 and soil binder of 40 to 50% being very definitely too high, there are many materials that naturally when pit run just barely pass these requirements. It is recommended that selection of base source and any necessary cleaning, screening or processing be made to at least try to produce dense, well graded base material with low P.I. and just enough cohesion to allow proper setting up of the base.

(g) Due to infrequency of severe freezes in most of Texas, we are inclined to overlook design against freeze damage to pavements. However, the data on average annual frost penetration indicate that base design over a considerable part of the State must include consideration of freeze prevention. Also, in the north part of the Panhandle, there is a possibility that some consideration will have to be given to freeze prevention in the upper part of subgrades under thin flexible bases.

## 2. The physical causes of severe freeze damage to flexible pavements are:

(a) When frost or freezing penetrates a material with the voids over 91% filled with water (pure water expands approximately 9% on freezing) freeze damage should occur. If a base is saturated at 20% moisture, it will then swell 1.8% upon freezing - if no additional moisture is drawn in by capillarity during freezing or alternate freeze and thaw from below or the side of the pavement, or by leakage through the surface. However, this amount of swell does not account for the amounts of heave (1/2" to 3") reported for bases frozen not over 5" in depth. Therefore, ice lenses must have formed to cause the most serious freeze damage. Where the binder has a high and rapid capillarity, the growth of the ice crystals to form ice lenses can easily cause the excessive heave or swell. Silt is the most susceptible to this type of freeze damage.

(b) A soil of low cohesion with high capillary power is most likely to cause frost damage. Soils of high cohesion and high capillarity are limited, by the element of time, in the amount of water they can transmit. In such soils the water must

be available from leaking, porous surfaces, seepage, or high water table at time freeze starts. Such soil binder in bases tend to fail from overload and moisture damage instead of freeze damage. In fact, freezing in tight clay soil or binder that is not saturated tends to draw moisture from adjacent binder or soil and may result in the appearance of shrinkage cracks - similar to drying shrinkage cracks.

(c) Single asphalt surface treatments, particularly without excellent curing, show the usual tendency to leaking or being porous - thus causing rapid freeze damage in bases susceptible to freeze damage.

(d) The placing of asphalt surface treatments and cold mix asphaltic concretes too late in the fall to allow proper curing and sealing may provide a porous, leaking surface that can cause rapid freeze damage the first winter after placement.

(e) Inadequate road mixing of high P.I. flexible base materials with fine or silty sand, in order to reduce the P.I. to meet present base specifications, often results in balls and streaks of fine sand and silt to rapidly feed water into the base - with resulting freeze damage.

(f) Hygroscopic salts and alkalies of the type that do not lower the freezing point appreciably, when present in the base or added in the sprinkling water helps to provide excess moisture with resulting freeze damage.

(g) The excessive swell and accumulation of moisture during freeze, results in a drastic loss of strength and modulus of elasticity in the base during the thaw and until the base dries out and resets. Traffic then breaks up the brittle asphalt surface. Graph No. 1 shows the estimated deflections within base materials for various values of modulus of elasticity. It will be noted that as the modulus of elasticity drops below 15,000, the deflections in the base rapidly increase. Therefore, it is recommended that bases be designed to maintain at least a minimum modulus of elasticity of 15,000 under all conditions. The loss in strength of the base results in rutting and corrugating. The approximate minimum cohesion and friction that should be maintained under all conditions in the base is shown in Road Design Circular No. 19-47, "Corrugating Flexible Pavements".

The base sources for most of the sections of roads reported with severe freeze damage can readily be divided as being located in the following general geologic groups or formations:

1. Austin, Gober and Pecan Gap Chalks (Upper Cretaceous)

These chalks outcrop about Austin, Waco, Dallas, Sherman and Paris. The freeze damaged roads (approximately 21 sections) wherein these chalks were used as base are indicated as (1) on the attached map.

The material consists of impure chalky limestone, and shaley limestone, interstratified with beds of softer marl.

The material is highly absorbent, (the best layers of chalk show 10 to 15% absorption) and readily weathers and breaks down under traffic to form an excess of low cohesion - highly capillary silty-clay binder. In road cuts, the alternating layers of the formation feed seepage water by capillarity into the absorbent base, which results in typical rapid and more frequent freeze damage in the cuts. The material is not strong enough to provide a good base - this results in cracked, failing surfaces, which leak surface water to the base where freezing accelerates the failure.

These chalks are located in the Humid Region of Texas and where the average annual frost penetration is about 3" to 6". Some of the worst freeze damage was experienced with these bases.

The Paris, Dallas, Waco, and Austin Districts have indicated that these chalks are unfit for base material. Our survey clearly shows agreement that the materials identified as Austin, Gober and Pecan Gap Chalks on the Geology map of Texas are not suitable for base material.

Typical tests on this material are as follows: LL = 30 to 43; P.I. = 13 to 17; L.S. = 6 to 8; S.R. = 1.7 to 1.9; (Silty clay - in this case low cohesion and highly absorptive), L.A. Abrasion = 55.

Best of Austin Chalk Produced as cleaned crushed Stone (under Item 215-A) compared with good Standard Base.

<u>Description of Test</u>	<u>Austin Chalk</u>	<u>Good Standard Base</u>
<u>Aggregate</u>		
L.A. Abrasion	55	Max. 50% Loss
Soundness	Not Sound	Max. 15% Loss
<u>Binder</u>		
LL	32	Max. 30
PI	14	Max. 9
LS	6.5	Max. 5
SR	1.78	Min. 1.8
% S.B.	30	10-30%
PRA Class	A-2-4	A-1, A-2 or A-3
<u>Combined Compacted Materials</u>		
Dry Unit Wt.	112	Min. 130 lb/cu.ft.
*Void Ratio	0.48	Max. 0.3
<u>Strength Test</u>		
Cohesion	3 lb/sq.in.	Min. 8 to 16 lb/sq.in.
Friction	45°	Min. 35°
Modulus of Elasticity	7,000	Min. 15,000

\*Void Ratio - ratio of volume of voids to volume of solids.

Graph No. 2 shows the triaxial compression test made at 5 and 20 pounds lateral pressure on the above Austin Chalk base material.

The unsound aggregate degrades to increase soil binder and soil constants. The low unit weight with high porosity makes the material subject to freeze damage. From the strength test showing low cohesion, it is estimated that 10" to 12" of better flexible pavement will be needed on this Austin Chalk for heavy traffic. Such depth of base and surface as required on the chalk subbase for strength design will also protect the material from freezing.

In University of Texas Publication No. 4246, "Building Stones of Central Texas", Austin Chalk with a sp.gr. of 1.96 and absorption of 11.6% shows severe damage in freeze-thaw tests. ←

## 2. Ogallala Caliche (Panhandle High Plains)

The Ogallala Formation covers the high plains of the panhandle of Texas. It is located in the Sub-Humid Region. It covers most of Districts 4 and 5 and part of District 6. The freeze damaged roads (approximately 45 sections) wherein these caliches were used as base are indicated as (2) on the attached map.

The formation is a broad alluvial out-wash plain consisting of gravels, grits, gray sands, buff sands, clays and silty clays, volcanic ash and caliche. The surface largely consists of fine sands, silts and silty clays, much reworked by winds which results in a loess-like soil. The fine sandy loams and loose sands (largely covering the south half of the Ogallala Formation) are eolian or wind deposits. The clay loams weather from the silty clays of the Ogallala. The caliche has formed in the soil by accumulation of calcium carbonate by evaporation of seepage and percolating water. The caliche is hard to soft and usually porous. This porous, soft caliche along with the included fine sand, silt, and silty clay makes it highly susceptible to freeze damage. It has been reported that some of the harder caliche in the "Cap Rock" is fresh water limestone, formed in old lakes.

Most of the Ogallala Caliche bases are placed with too much porous, silty fine sand and silty clay binder or rolling, weathering and stress produce excess binder from the soft, porous caliche. The susceptibility to freeze damage increases directly with the amount of such binder (when in excess of voids in the aggregate) and decreases with increase in unit weight of the total base. The usual minimum of 130 to 135 pounds per cubic foot (for aggregate of 2.65 specific gravity) probably can be reduced for good caliche base since one specific gravity test on hard caliche shows about 2.4.

In 1941 it was reported that all the freeze damage occurred in bases with PI over 15 and soil binder over 50. Since then the additional freeze-thaw cycles and increase in amount and weight of traffic has caused much freeze damage in bases showing lower soil constants and unit weight of base under about 120 lbs.

The shrinkage ratio of the soil binder in these freeze damaged bases range from 1.57 to 1.78. Findings to date indicate that much of the serious freeze damage bases have soil binders with SR of less than 1.7 or 1.8. Low shrinkage ratio is just another indication that the material is porous. For instance a SR of  $1.57 \times 62.5 = 98$  lbs. per cu. ft. (weight of the binder at shrinkage limit). Usually the compacted unit weight of such soil is little more than the above. A good sand clay binder should weigh over 120 lbs/cu. ft. The aggregate combined with binder should weigh over 130 lbs/cu.ft. These figures are set up for materials of 2.65 to 2.75 specific gravity.

The typical behavior of these porous, high binder, soft caliche bases is to show excessive transverse cracking and map cracking throughout the pavement surface and failure of the pavement working in from the edge. The Ogallala Formation is in the dry half of the Sub-Humid Region, with average frost penetration of about 2" to over 6". The maximum penetration in the north part has been reported at from 10" to 18".

The transverse cracking and map cracking is from drying shrinkage of the base with excessive binder, probably helped from similar shrinkage from freezing when base is rather dry. Also, the excess silty binder should "spring" under the heavy wheel loads and help crack the asphalt when it is cold and brittle.

In the Sub-Humid climate, the best source of moisture is from moisture at the edge, particularly snow piled up on shoulders after removal from surface. The high capillarity of the base rapidly draws this moisture into the outer edge of pavement - causing the most severe freeze damage. The next prolific source of moisture is from the cohesionless sands in the subgrade - which can feed moisture rapidly to the high capillary base. However, it appears that any base material particularly susceptible to freeze damage will eventually get enough water to suffer severe freeze damage.

The first step in freeze prevention in the Ogallala Caliche should be to locate pits where there is the most of the harder caliche, limit depth of pit to the harder parts or including considerable percentage of harder nodules. The usual silt, fine sand and silty clay and soft caliche should be scalped or screened out, then the harder material crushed and rescreened if necessary to hold the binder below 30% after rolling. Investigations should be made as to revised test methods and requirements to predict ultimate binder content and freeze resistance of the harder caliche - such as longer soaking periods - rougher treatment of material in determining binder, and reasonable limits of the L.A. Abrasion test and soundness test.

The moisture content of the base shows large variations with the seasons and changing weather; therefore, if the binder more than fills the aggregate voids, shrinkage cracks should be expected in the surface. Naturally, freeze damage results. The shrinkage and swell of the total base material is of more importance than that of the binder alone.

There is some evidence that the calcium available flocculates the clay in such way as to cause more silt size pores. Several gradation tests on these caliche bases show considerable increase in colloidal sizes when the material is deflocculated. Also soil constants on one base shows LL = 39, PI = 17 and LS = 5 by the usual test procedure. But with long soaking time plus deflocculating agent (sodium silicate) and with soil constants run as the material dried back the following results were obtained: LL = 46, PI = 24, and LS = 11.1. Such a comparison on a good gravel base with PI of 6 did not show any change in the soil constants of the binder.

It appears that more use may be made of the available gravels in the upper part of the base.

Although thicker surfaces provide additional protection and slow up the freeze damage, it is recommended that the basic trouble be corrected before considerable money is spent in thicker asphalt surfaces.

The quick removal of snow from the shoulders, sealing of shoulders, nominal increases in asphalt surface thickness may be considered as fighting a delaying action - but cannot replace the need for reasonably sound and stable base of minimum adequate depth.

The attached Graph No. 3, shows typical grading analysis on Ogallala caliche bases that failed from freeze damage before 1941 (S.B. + 50 and PI + 15) and on caliche bases that were good in 1941 but have since failed (S.B. 33 to 50 and PI 10 to 15). Note that the binders with shrinkage ratio of less than 1.7 fail fast and those with S.R. between 1.7 and 1.9 are slower to fail.

Strength and freeze-thaw tests have been run on the following recently used caliche base materials:



Moore County - State Highway 354 (Ogallala Caliche)

Tests on Crusher Run

LL = 31  
PI = 7  
LS = 4.1  
SR = 1.63  
SB = 17%

Tests after compaction of sample

LL = 29  
PI = 8  
LS = 5.1  
SR = 1.71  
SB = 30%

Strength Test on Total Base

Cohesion = 14 lbs/sq.in.  
Friction = 45°  
Modulus of Elasticity = 12,800

After a severe freeze-thaw cycle with capillary moisture available, the material showed little change in strength (cohesion and friction) but the modulus of elasticity was reduced from 12,800 to 9,200 - indicating a little more "spring" or deflection within the base under load. There was a little increase in volume during freeze, but no ice lenses appeared and most of the swell was lost on thawing. A consolidation load during thaw would probably reset the material completely.

The increase in binder from 17% to 30% with laboratory compaction indicates that there is too much soft material included in the aggregate, since job compaction usually produces more binder than laboratory compaction. During rolling on the job, an excess of fines was produced. After investigation of the material source and with consideration of the above tests and observations, it appears that about 25% of silty sand and soft caliche can be screened or scalped out before crushing and a more durable, denser base may be produced.

Carson County - State Highway 136 (Ogallala Caliche)

Tests on Crusher Run

LL = 31  
PI = 9  
LS = 4.7  
SR = 1.65  
SB = 28%

Tests after compaction of sample

LL = 30  
PI = 9  
LS = 4.6  
SR = 1.69  
SB = 35%

Strength Test on Total Base

Cohesion = 8 lbs/sq.in.  
Friction = 48°  
Modulus of Elasticity (E) = 9,200

On freezing, there was definite formation of ice lenses and the sample showed a 5% volume increase. On thaw, the net volume increase with respect to volume before freezing was 2.8% and the loss in strength was about 40%. This is indication that 35% of the porous binder is too much. The softer material can be screened out and the harder material used to produce good, sound base.

3. Lower Cretaceous Limestones:

The freeze damaged roads wherein the base materials are from these formations are indicated as (3) on the attached map.

The Lower Cretaceous, consisting of the Washita, Fredericksburg, and Trinity groups, outcrops over a large portion of central and west Texas - north and west of a line from Del Rio, San Antonio, Austin, Waco, Ft. Worth and Denton. It is in all climatic regions (arid and semi-arid, sub-humid and humid) and where average frost penetration varies from 0" to 6".

Many of the Cretaceous soft limestones and chinks in the Humid Region have been placed erroneously under the Caliche specification. Caliche does not tend to form in this region, and to date we have not found enough caliche in the Humid Region to warrant the use of the specification. Proper crushed stone or gravel specifications are recommended for use. West of the division between the Humid and Sub-Humid Regions, the chances for development of deeper and harder caliche gradually increases with decrease in average rainfall and with increase in average evaporation. The caliche develops in the deeply weathered soils and weathered rocks of the parent materials and in talus slopes, alluvial soils and old high terraces. Generally, caliche pits will have to be shallow, and the excess soils in which the caliche has formed screened out so the hardest material is used to produce base.

We wish to amend our previous optimistic report on the Edwards Limestones (Fredericksburg group). Much of the concentrated freeze damage north and north-west of San Antonio must be blamed on the Edwards. A check of the geology reports show that there is considerable pulverulent Edwards limestone in this area. Webster defines pulverulent as "consisting of, or reducible to, fine powder."

The Pulverulent Edwards can be readily identified by visual inspection; however, there are six tests as follows that identify it as poor base material and particularly susceptible to freeze damage:

Comparison of Pulverulent Edwards Limestone with good Standard Base

<u>Description of Test</u>	<u>Pulverulent Edwards Limestone</u>	<u>Good Standard Base</u>
<u>Aggregate:</u>		
L.A. Abrasion	80%	Max. 50%
Soundness	Failed 1st Cycle	Max. 15% for 5 Cycles
<u>Soil Binder:</u>		
LL	41	Max. 30
PI	6	Max. 9
% Binder (-40M)	32	10-30
SR	1.43	Min. 1.8
PRA Class	A-5	A-1, A-2 or A-3
*Dust Ratio	0.74	Max. 0.5

\* Dust Ratio - ratio of % passing 200 mesh to % passing 40 mesh screen.

The indications of susceptibility to freeze damage are as follows:

- (1) Unusually high loss in the abrasion test (turns to silt size powder under roller and traffic).
- (2) Very rapid failure in the soundness test (turns to powder under freeze-thaw).
- (3) Unusually high L.L. in relation to low P.I. (high capillarity with low cohesion).

- (4) Unusually low shrinkage ratio ( $1.43 \times 62.5 = 89.4$  lbs/cu.ft.)  
(low unit weight)
- (5) The FRA class A-5 identifies the binder as high in silt.
- (6) The high dust ratio with low P.I. shows too much silt in relation to the fine sand.

Graph No. 4 shows the gradation of total base and gradation of soil binder in the above pulverulent Edwards base that had severe freeze damage. The typical gradation of a good base material is shown for comparison.

The Pulverulent Edwards is unfit for base.

The usual range in soil constants of this pulverulent material is as follows:

LL = 25 to 41

PI = 4 to 8

SR = 1.4 to 1.65

SE = 40 to 50% (Crusher run)

(Note: This easily meets the present caliche specifications)

However, the geology reports show a wide range of various types of limestone in the Edwards. Good rock for building stone, flexible base, concrete aggregate, asphalt surface treatments and asphaltic concrete is being produced from the Edwards. The limestone produced by Servtex Materials Company is from the Edwards.

The first step to take in producing good base from the Edwards is to check the various strata by the Los Angeles Abrasion Test and the Sodium Sulphate Soundness Test. (See AASHO specification and test requirements for crushed stone base).

The Edwards Limestone is reported in "The Geology of Texas", University of Texas Bulletin No. 3232, as diverse in lithology. The following are some of the various types of limestone:

- (1) Hard crystalline limestone
- (2) Whitish nodular limestone
- (3) Soft, marly limestone
- (4) Pulverulent limestone
- (5) Pure white limestone, hard to friable
- (6) Yellow silt stone
- (7) Dense, ringing, fine-grained limestone - good building stone
- (8) Shell limestone

In University of Texas Bulletin 4246, "Building Stones of Central Texas", one sample of Edwards with sp.gr. of 2.5 and percent absorption of 2.55 shows excellent resistance to freeze-thaw, but another Edwards limestone with sp.gr. of 2.14 and absorption of 8.43% shows severe damage in the freeze-thaw test.

On typical samples selected by the Bureau of Economic Geology, the Edwards limestone in the Belton-Waco area show ranges of L.A. Abrasion test from 39% to 84% loss and soundness test of 0% loss to 82% loss in five cycles. These tests should be used to help secure the better limestone from this formation. Preliminary tests on percent absorption can be made, then particular care taken to check the soundness of rocks with absorption of over 3%.

The Comanche Peak (nodular) Limestone was reported previously as being particularly susceptible to freeze damage. It appears that there is little chance to produce good base from this formation. First, look for better limestone in the Edwards, just above the Comanche Peak.

The attached graph No. 5 shows results of abrasion and soundness on Edwards and Comanche Peak Limestones. These samples were selected by the Bureau of Economic Geology from a current cooperative project for investigation of limestones in the Waco Area. It will be noted that all typical samples of the nodular Comanche Peak Limestone are unsound, with % loss of 72 to 100 in the AASHTO soundness test. This concurs with our impression of the nodular Comanche Peak Limestone where used as base.

Walnut Clay Formation (Fredericksburg group). The Walnut Clay formation, basal part of the Fredericksburg Group, consists of yellow clays, flaggy limestones with thin layers of clay, marly limestones, chalky nodular limestone, some medium hard limestone suitable for building stone, and a widely persistent, consolidated, and mappable shell aggregate.

The shell aggregate has been rather widely used as base. Much of this shell aggregate has just plain black mud in the shells, somewhat similar to recent shell produced along the coast- but generally it is cemented just enough to make washing difficult. This mud makes the base quite susceptible to freeze damage as well as load damage. Most of this Walnut shell aggregate might be satisfactory for sub-base, but so far we have been unable to find where it has produced really satisfactory base material.

The Washita limestones have much nodular limestone and limestone with marly layers between. However, where the harder and sounder limestones have been selected and the marl and shale properly cleaned out, satisfactory bases have been produced.

The Trinity Group consists of packsand, sandstone, and alternating layers of hard limestone with marl or marly limestone (weathers to form terraced or "Staircase" topography). This formation weathers rapidly - resulting in many talus slopes and alluvial consisting of sand, silt and clay with some calcareous material mixed therein. This material has been used in many places for base placed under the caliche specifications. It is light, porous, and produces an excess of binder. This calcareous dirt is the cause of much of the freeze damage in the Trinity area and is unfit for base material.

The alternating layers of the Trinity Group produce seepage. Additional and rapid freeze damage through the seepy cuts is particularly noticeable.

In University of Texas Bulletin No. 4246, "Building Stones of Central Texas", two tests on Glen Rose Limestone (Trinity Group) show a sp.gr. of 2.0 and percent absorption of 12 to 15%. These samples of rock failed the freeze-thaw test. Care should be taken to investigate these porous limestones before using in base.

#### 4. IRON ORE GRAVELS:

Freeze damage reported in iron ore bases is shown as (4) on Map No. 1.

Reference is made to Road Design Circular No. 17-47, "Relation of Geology to Iron Ore Base Material", concerning the location of the East Texas Iron Ores. The regular iron ore base material is located between the 1" to 3" average frost penetration line and all below the 500' contour line. The length of freezing temperature

drops rapidly below the general location of the 500' contour. This and the inherent density (usual SR of 1.8 to 1.95) of just a fair iron ore base combine to prevent freeze damage being a serious problem in this area.

Under traffic and weathering the iron ore gravels do not tend to degrade into increasing amounts of weak, porous binder like the other types of base materials that are particularly susceptible to freeze damage. Many of the short sections of freeze damage reported in the iron ore bases appear to be more subgrade failures than typical freeze damage. The evident freeze damage sections are clearly due to excessive clay or fine, silty sands, with most of the damage in cuts close to seepage or high water table. Usually the excessive clay can easily be prevented by not cutting too deep in the pits. The excessive silty sand binder often located in thin cover over the iron ore gravels can be eliminated by either proper stripping or by processing the material. Most of the freeze damage occurs when soil binder is in excess of 40%; however, it occurs where the soil binder is over 30 or 35%.

As reported in "Road Design Circular No. 17-47", the trend is to increase available base material by crushing and processing the iron ore. A series of strength and freeze-thaw, wet-dry tests should be run on a typical range of iron ore bases to help establish the minimum requirements for both the iron ore gravel and crushed iron ore base material.

The usual requirement of minimum 130 to 135 pounds per cubic foot compacted dry density for gravel can not be applied to iron ore since the specific gravity of some of the aggregate may run as high as 5. For instance, one job of Willis Iron Ore Gravel shows dry compacted field densities of 133 to 176 pounds per cubic foot. 165 pounds is the maximum for solid material of specific gravity of the usual 2.65. It is suggested that any study of or requirements on gradation in this iron ore material be based on percent by volume instead of the usual percent by weight - thus the variable specific gravity will not confuse the issue on fundamentals of good, dense gradation.

##### 5. PERMIAN GYPSUM AND SALTS, POROUS DOLOMITES AND SHALE

The Permian System consists of limestones, sandstones, dolomites, red shale ("Red Beds"), beds of gypsum at the surface (weathered from anhydrite) and various salts underground. The anhydrite and salts were formed by chemical precipitation and complete evaporation of sea water. The salts are usually closely associated with the gyps. Thus the alluvials from the Permian have much redeposited gypsite and salts (alkali flats).

The gypsite looks like caliche but in places consists of 100% gypsum crystals, which are hygroscopic. Flexible bases made of this material are particularly subject to freeze damage.

Gravels with excessive amounts of gyp or the hygroscopic salts naturally draw excessive moisture and assist the freezing action. Where base materials are sprinkled with the alkali water they puff up as evaporation forms salt crystals. This action along with the additional porosity and hygroscopic action certainly expedites frost action.

To prevent such freeze damage, in the permian or Alluvials from the gyp and salt portions of the Permian, do not mistake the gypsum (hydrous calcium sulfate,  $\text{Ca-SO}_4\text{-2H}_2\text{O}$ ) for caliche base material (calcium carbonate,  $\text{Ca CO}_3$ ).

The Pecos District has found that where the salt water is not satisfactory for watering stock, it is not satisfactory for use in sprinkling flexible base.

The Childress District has experienced considerable freeze damage in the porous dolomites and soft sandstone with layers of gray shale. The soundness of the rocks will have to be tested and the shale will have to be cleaned out before successful bases can be made of this material. It appears that a considerable amount of the dolomite can be found sufficiently sound for base.

The freeze damaged roads that may be associated with the gyms, salts, and dolomites are indicated as (5) on the attached map. There is very definite evidence that these same gyms and associated salts may be blamed for unsound concrete pavements, bridges, culverts, right of way markers and curbs in the area.

## 6. GRAVEL BASES

Freeze damage reported in gravel bases is shown as (6) on Map No. 1.

Gravel bases have shown a surprising resistance to severe freeze damage. They have to be extremely poorly graded and usually with unusual amounts of silty binder or to have the assistance of the Permian gyms and salts to show considerable freeze damage.

In comparison with the various unsound materials that are particularly subject to freeze damage, most of the gravel is naturally sound or it could not have been transported and redeposited as gravel. However, overload and moisture damage show a definite need for more processing and better control of gravel bases.

For instance, a very poorly graded gravel with 59% binder did not show freeze damage and was placed on the same project with a base of Pulverulent Edwards with 32% soil binder which did show severe freeze damage. The gradation of the soil binders (calculations based on 100% soil binder) are practically the same, but note the following comparison of soil constants:

### Freeze damaged Pulverulent Edwards

LL	PI	FME	SL	SR	% SB
41	6	40	33	1.43	32

### Gravel base - No freeze damage

30	11	24	17	1.82	59
----	----	----	----	------	----

The 41 LL with 1.43 shrinkage ratio of the freeze damaged section shows high porosity of the silt size; whereas the 30 LL with 1.82 shrinkage ratio of the more freeze resistant material indicates that it will be much slower in taking up water and will not store as much for freezing.

Of course, the above does not prove that gravel base with 59% soil binder is good - since the project was less than one year old and has very light traffic, both in volume and weight. This comparison does serve to point out the type of base material that is particularly susceptible to rapid and severe freeze damage. The widely different soil constants for the soil binders with similar gradation probably indicates difference in shape, texture, and chemical composition of the soil particles. This is

just another check on the long established fact that design and control of bases can not be based on gradation only, but we should give more attention to control of gradation than at present.

The attached graph No. 6 shows the gradation and soil constants of three types of freeze damaged bases in comparison with the gradation and soil constants of a very good gravel base that is resistant to freeze damage as well as load failure. This graph indicates that for good base, it is desirable to produce less than 30% soil binder, with LL less than 25, P.I. less than 6, shrinkage ratio (SR) greater than 1.8 and with a dust ratio (ratio of % passing 200 mesh to % passing 40 mesh sieve) of less than 0.5. Except for the use of the shrinkage ratio, this data merely indicates agreement with many published articles on specifications for good base material.

#### 7. STABILIZATION:

The roads with some type of stabilization that were reported with freeze damage are indicated as (7) on the attached map. This group includes all types of stabilization, such as road mix of sand to reduce high P.I. binder, soil-asphalt, soil-cement, and soil-lime.

The type most susceptible to quick freeze damage is that with poor road mix of fine, silty sand with high P.I. binder in order to reduce the P.I. Inadequate road mix leaves sand lenses and clay balls, wherein the fine sand rapidly feeds water to the clay. This causes rapid freeze damage as well as load and moisture damage. Here a thorough lab mix indicates ideal P.I. is secured, but the results on the road are quite different. There is enough evidence to set this type of freeze damage as a definite pattern.

There are enough freeze damage sections of the soil-asphalt, soil-cement, and soil-lime stabilization types to clearly indicate that design and control should definitely include a durability test, if the material is to be used as base.

It has been noted that with some types of soils, the admixture of 3% lime reduces the shrinkage ratio and unit weight of the compacted material. Also, some tests have shown that the admixture of oil with some soils may increase the permeability several times. With such materials, there may be a temporary delay in failures, but the long time result is liable to be very poor. The design and construction of these various stabilizations for use as base should not be governed alone by lab strength or punch tests, but should include consideration of the actual job mixture obtained and should include consideration and testing of durability or resistance to freeze-thaw and wet-dry cycles.

#### 8. MISCELLANEOUS FORMATIONS

Woodbine Sand: The Woodbine group consists of sand, sandy clay, clay, soft to hard ferruginous sandstone and ironstone concretions. These basal sands of Upper Cretaceous outcrop in Hill, Johnson, Tarrant, Denton, Cooke and Grayson Counties.

The hard ferruginous sandstone and ironstone concretions are usually limited in amount, but the freeze damaged bases consist mostly of the soft ferruginous sandstone which under rolling and traffic readily reduces to sandy soil binder (about 75% binder)

The sands and sandy clays have films, lenses and layers of clay which help provide ample seepage in cuts to supply water to the base for severe freeze damage. Also, it is easy to get excess clay and clay balls in the base material.

The soft sandstone can not be expected to produce a good base. Of course the hard sandstone and ironstone concretions can be screened out to produce base - but the hard material is rather limited and infrequent. Investigations can be made to check possibilities of breaking down and then stabilizing the soft sandstone for light traffic roads.

Tehuacana Limestone: This limestone is in the Kinciad Formation. (See Pages 537 and 538 of U.T. Bulletin No. 3232, "Geology of Texas"), and outcrops in Falls, Limestone, Navarro, Kaufman and Hunt Counties. Similar limestone is also named Rocky Cedar Creek and Lone Oak limestone lentils.

This limestone consists of hard to medium -hard fossiliferous limestone including highly fragmental shell material cemented by calcite. The limestone alternates with thin layers of sand (sometimes slightly cemented) or at other places with thin layers of marl.

Typical moderate freeze damage in Tehuacana Limestone base material where the limestone has thin layers of sand is on U.S. Highway 84 in Limestone County from McLennan County Line to Mexia.

A typical test on the crusher-run (before placing on road) material indicates excellent base as judged by present specifications. This test shows 64% on 1/4" screen and 76% on 40 mesh. The soil binder (-40 mesh) tested as follows:

LL	PI	FME	CME	LS	SL	SR	% SR
27	6	21	10	20	3.5	1.71	24

The base material pit shows a top hard, dense layer of limestone with most of the pit consisting of moderate hard, shelly limestone alternating with thin layers of slightly cemented sand and the rock underlain with sand. The following detailed tests have been run to help determine the cause of the freeze damage:

Material	Los Angeles Abrasion % Loss	Specific Gravity	Absorption	Soundness Test % Loss	Remarks on Soundness
Hard Top Layer of Limestone	40%	2.55	1.9%	25%	Largely surface disintegration
Medium, Hard, Shelly Limestone (most of pit)	46%	2.49	3.6%	43%	Disintegration due to unsound matrix

The following soil constants were determined on the four types of material included in the pit:



Material	LL	PI	FME	SL	L.S.	S.R.	% Soil Binder
Hard Top Layer of Limestone (Binder from crusher-run)	26	7	22	21	2.6	1.71	22
Medium Hard Shelly Limestone (most of Pit) (binder from L.A. Abrasion Test)	24	7	20	19	3.0	1.79	36
Alternating thin layers of <u>slightly</u> cemented sand	31	10	24	22	4.1	1.61	95
Underlying Sand	32	9	25	26	3.1	1.54	93

Both sands show about 70% between the 100 and 200 mesh sieve (fine sand). Also, both the limestones tend to produce 10% to 15% fine sand between the 100 to 200 mesh. The sum of all, including too much of the underlying sand as binder, has produced an excess of low shrinkage ratio, uniform fine sand to draw excess moisture with resulting free damage. The percent soil binder varied from 15% to 50% during construction. This is considered as the primary cause of the early freeze damage. Although the A.A.S.H.O. limits % loss in the soundness test to 15%, we may be able to secure good base from the hard limestone with up to 25% loss, but most of the limestone (medium hard shelly shows a loss of 43%. This is, no doubt, a contributing factor in the freeze damage. It will be noted that both types of limestone pass the maximum 50% L.A. Abrasion requirement.

The Tehuacana Limestone has a rather confused history in performance. It has been reported as good to poor for concrete, railroad ballast, and base material. A material that is hard and that passes the L.A. Abrasion test, but is about half unsound can be expected to have such history.

Certainly the interstratified sand or marl should be cleaned out and the limestone tested for soundness before use as good base material. When the material fails the sulphate soundness test, final checks can be made with actual freeze-thaw, wet-dry tests before final approval or disapproval of the material.

#### SUMMARY OF CONCLUSIONS

1. Most of the freeze damage appears to be in poor base material, particularly those with an excess of porous binder (FRA Class A-4 and A-5).
2. Single asphalt surface treatments show a definite tendency to be porous and do not generally provide sufficient protection to base materials that are subject to freeze damage.
3. An excess of fines left on top of base during finishing operations is susceptible to freeze damage.
4. Inadequate road mixtures of fine silty sand to reduce P.I. often result in lenses of sand to rapidly feed water into the base, resulting in rapid freeze damage.

5. The higher the per-cent binder (above the amount needed to fill the voids in the aggregate) and the more porous the binder (particularly of the silt size) the more susceptible the base is to freeze damage.

6. The closer the base is to water table or seepage, the more severe and quicker is the freeze damage.

7. Freeze damage occurs quicker and with more severity in porous bases that are placed on fine sandy and silty subgrades (that can feed more water faster to the base) than on tighter sand-clay or clay subgrades.

8. More attention should be given to the old established principal of providing sound, durable aggregate in the base.

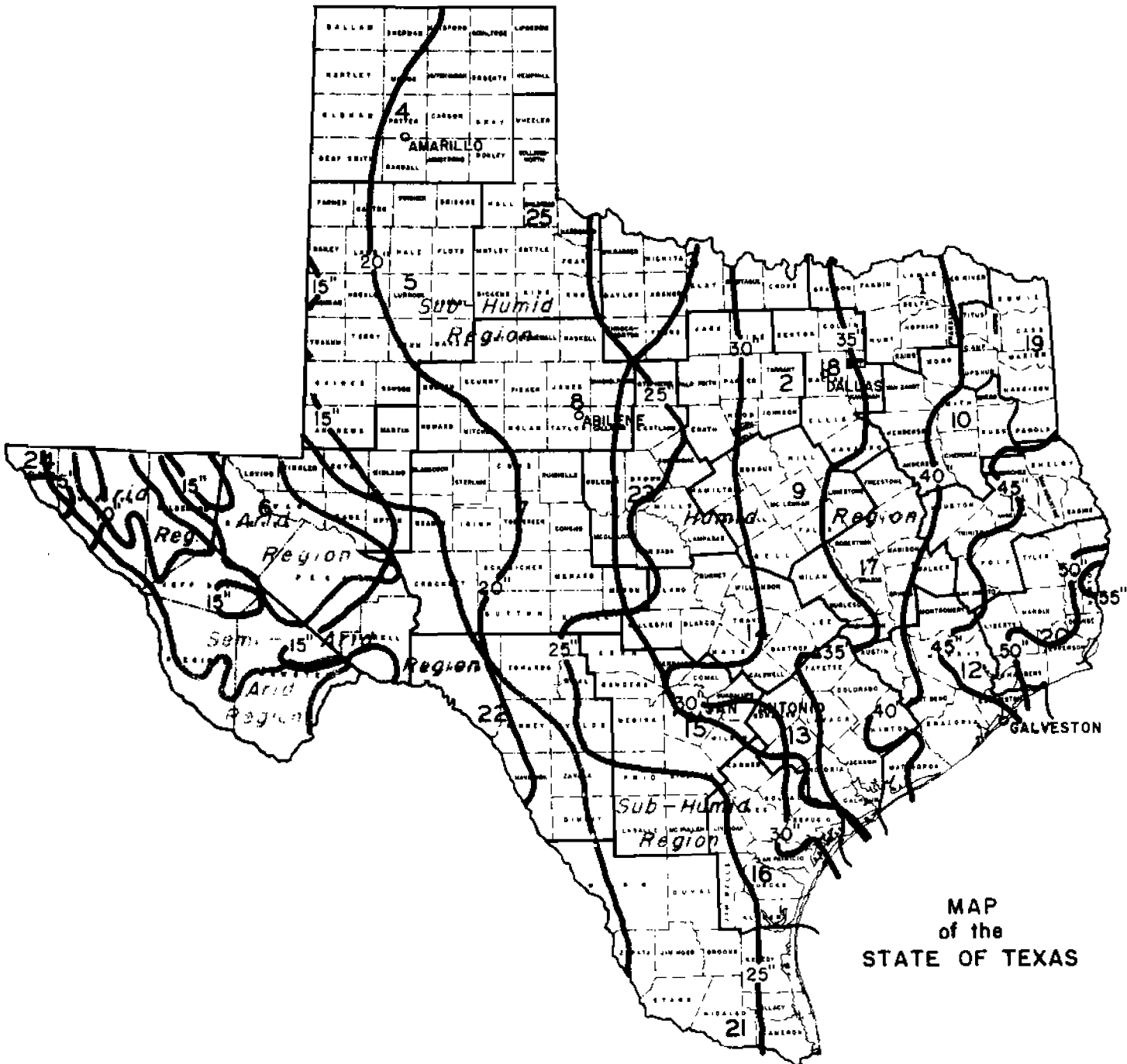
9. More attention should be given to the old established principal of providing a dense gradation in the base material.

10. Many materials that are poor base material as pit-run or crusher-run can be made fair to excellent base materials by proper cleaning or screening and processing to meet good standards.

11. More District Laboratories should be reopened and extensive investigations made to make maximum use of the local materials available.

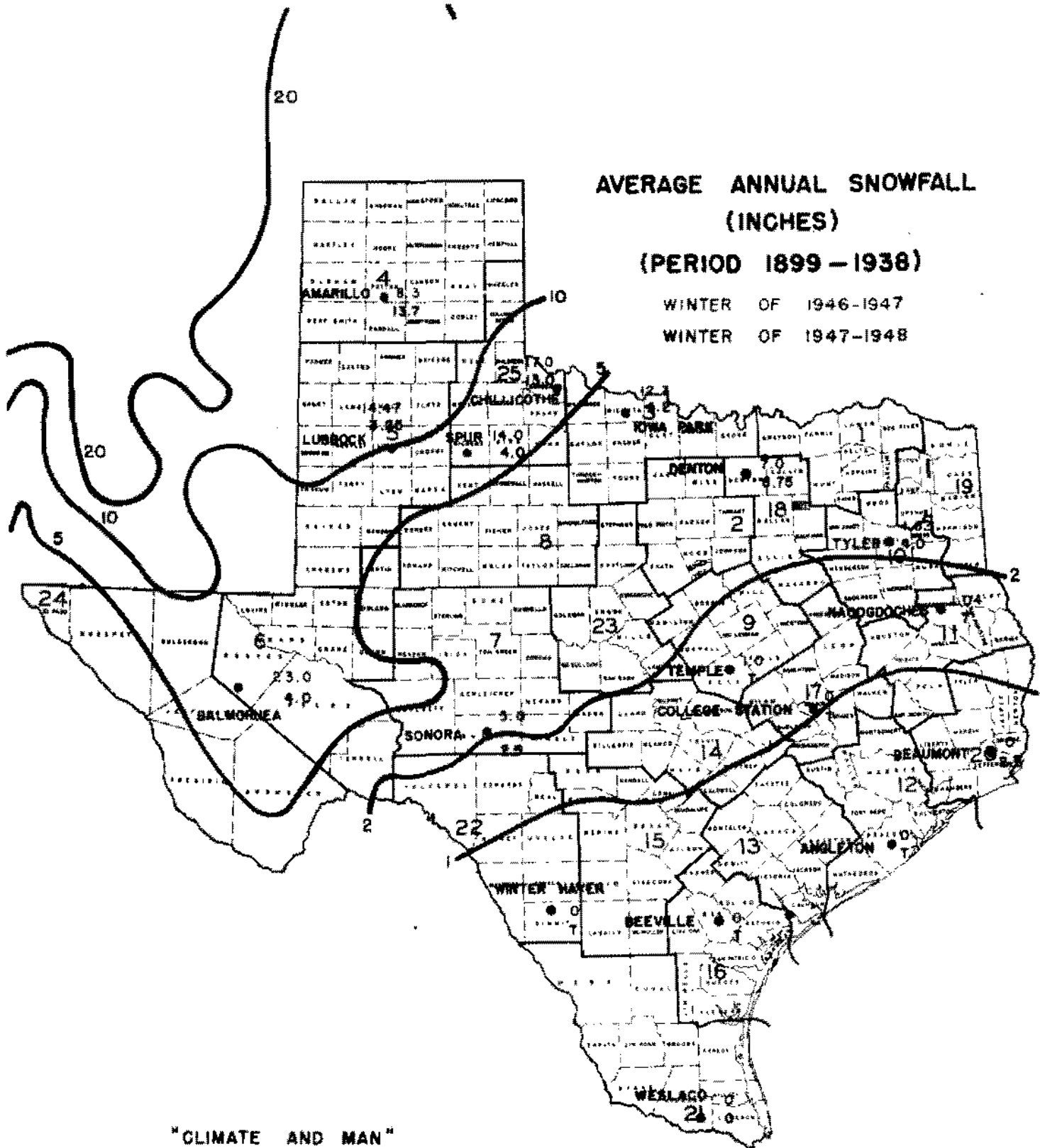
12. The sound fundamentals for good base materials should be applied more and more, instead of leaning on the crutch of just making the P.I. of the soil binder pass the specifications.

CLIMATIC REGIONS OF TEXAS  
AVERAGE ANNUAL PRECIPITATION



MAP  
of the  
STATE OF TEXAS

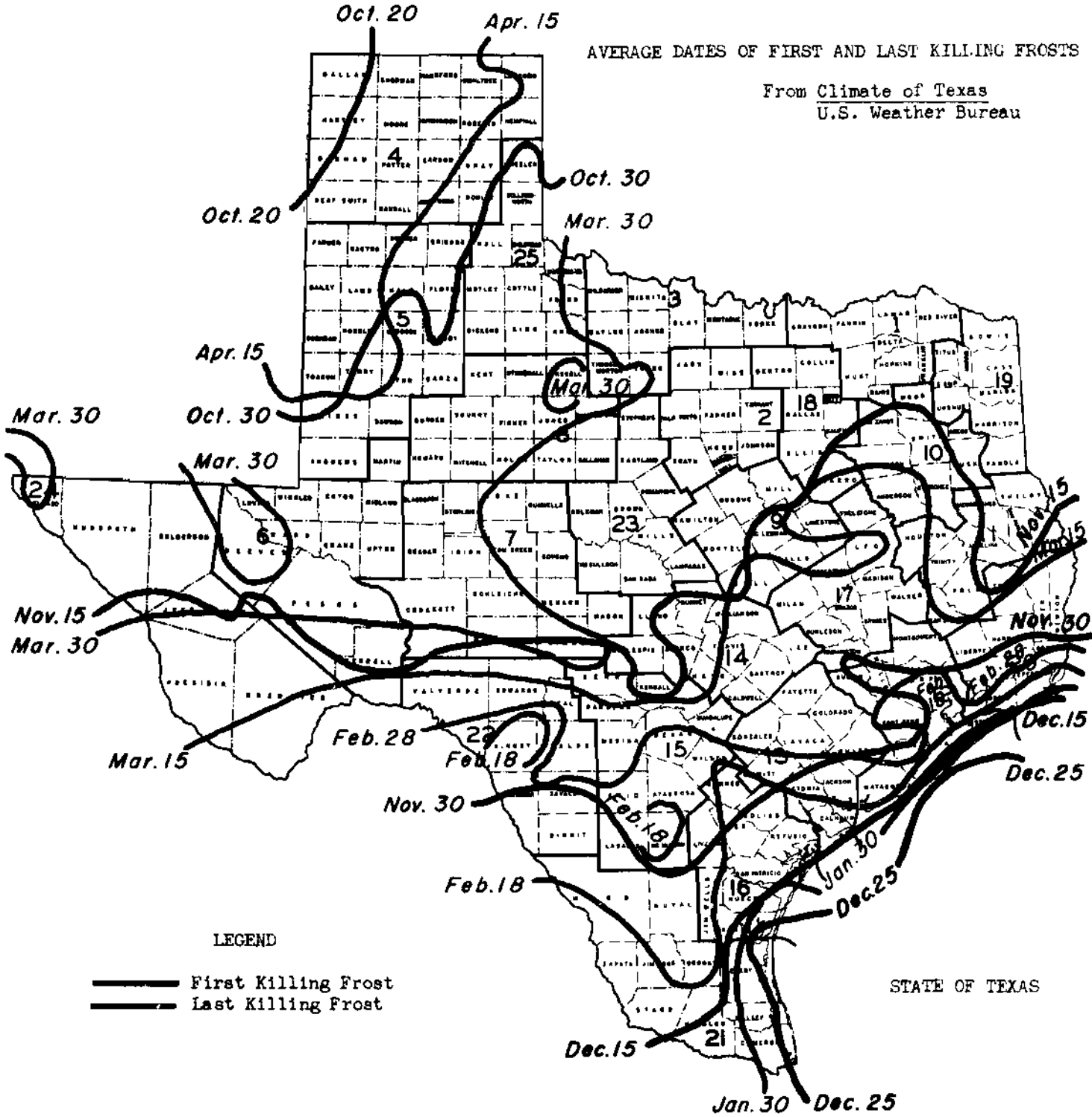
"THE SOILS OF TEXAS"  
BULLETIN NO. 431  
TEXAS AGRICULTURAL EXPERIMENT STATION



"CLIMATE AND MAN"  
YEARBOOK OF AGRICULTURE  
1941  
U.S. DEPARTMENT OF AGRICULTURE  
WASHINGTON, D.C.

AVERAGE DATES OF FIRST AND LAST KILLING FROSTS

From Climate of Texas  
U.S. Weather Bureau



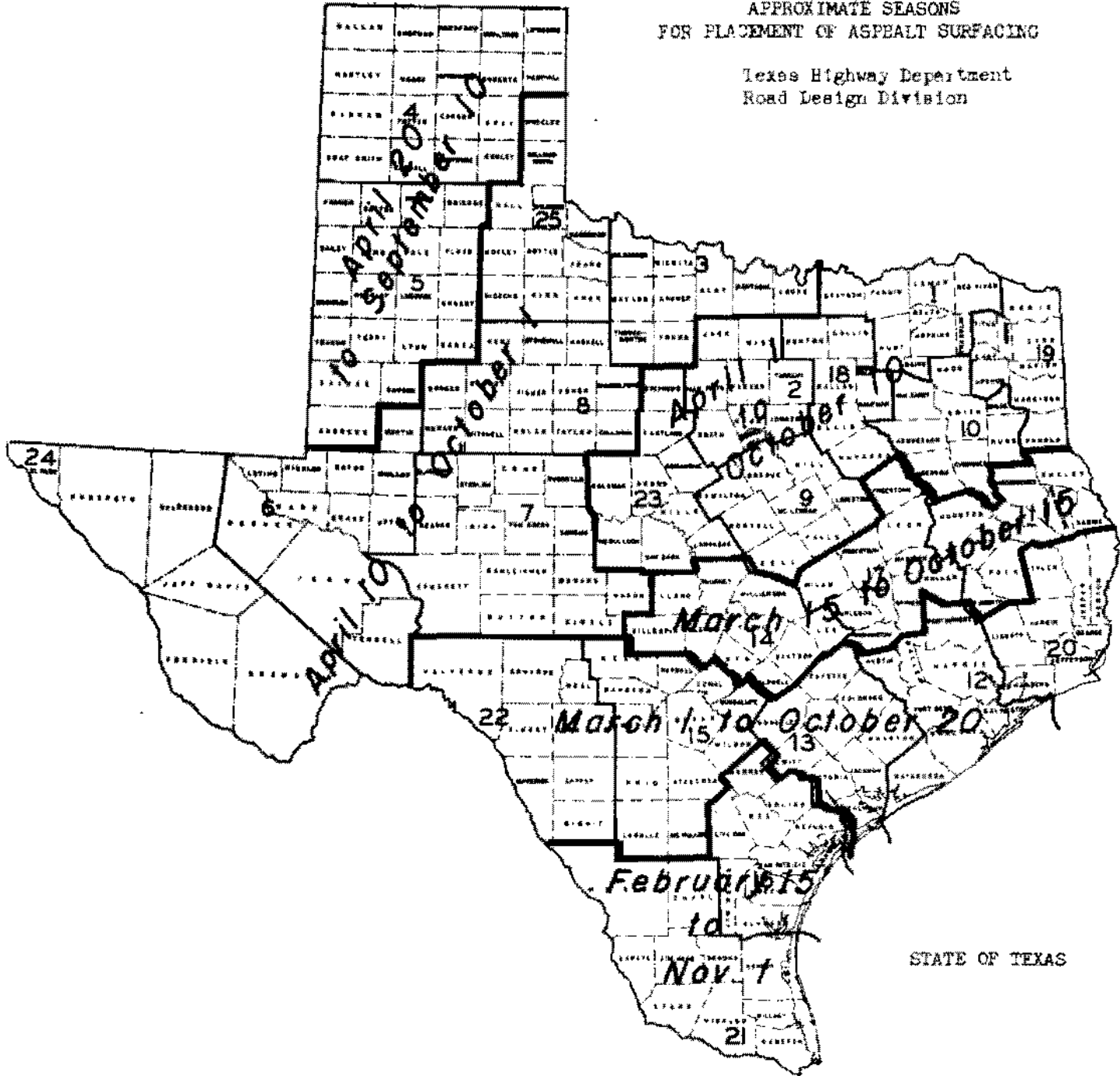
LEGEND

- First Killing Frost
- Last Killing Frost

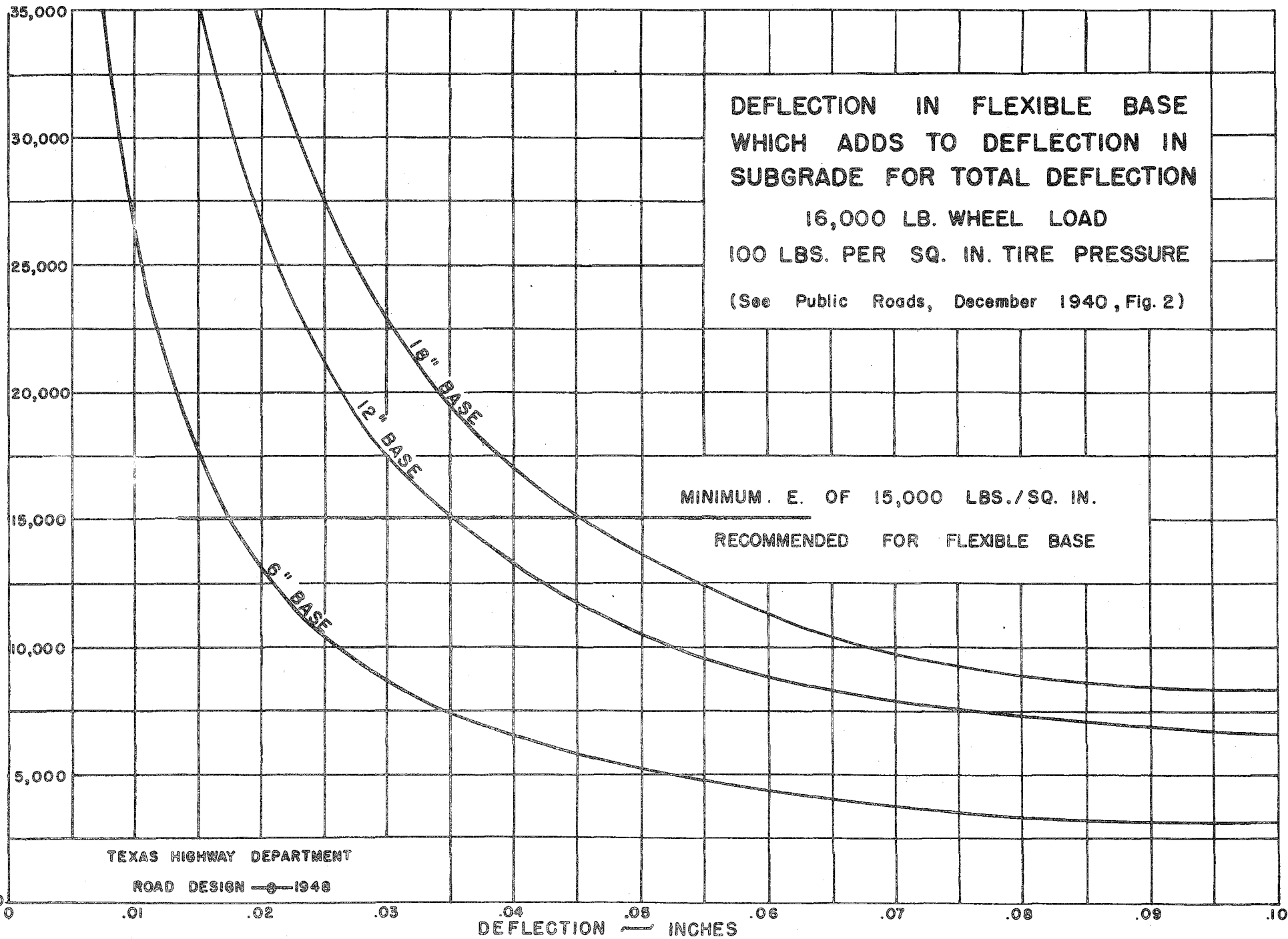
STATE OF TEXAS

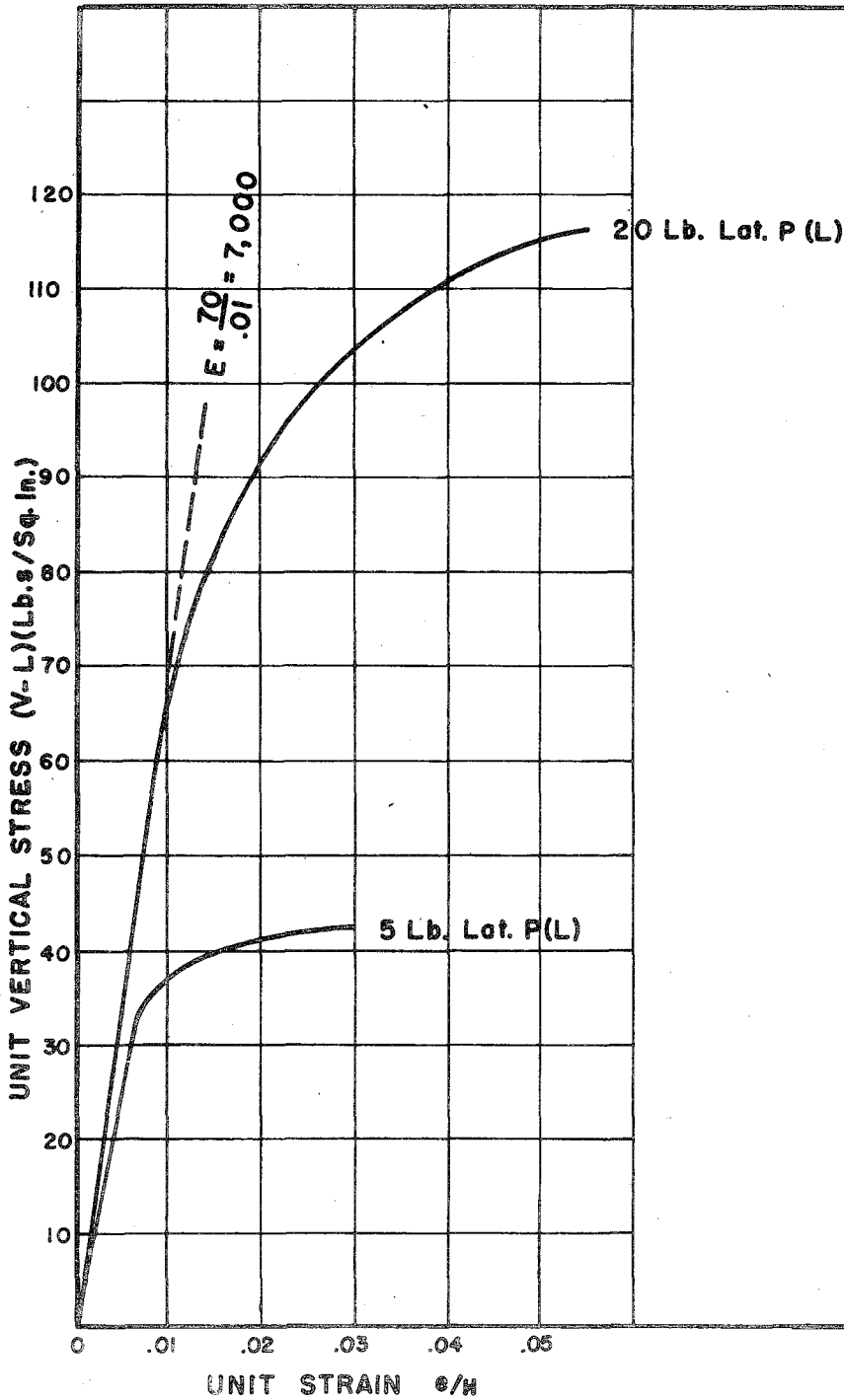
APPROXIMATE SEASONS  
FOR PLACEMENT OF ASPHALT SURFACING

Texas Highway Department  
Road Design Division



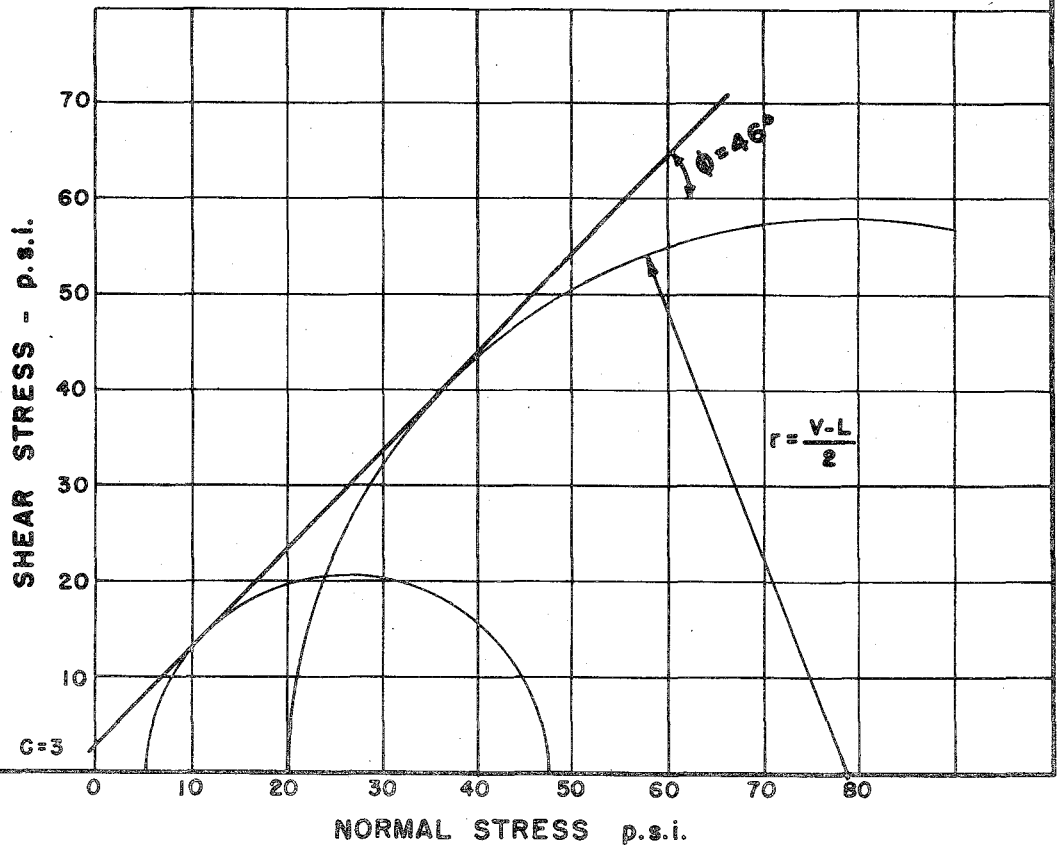
Graph No. 1  
E - MODULUS OF ELASTICITY OF BASE (LBS./SQ. IN.)





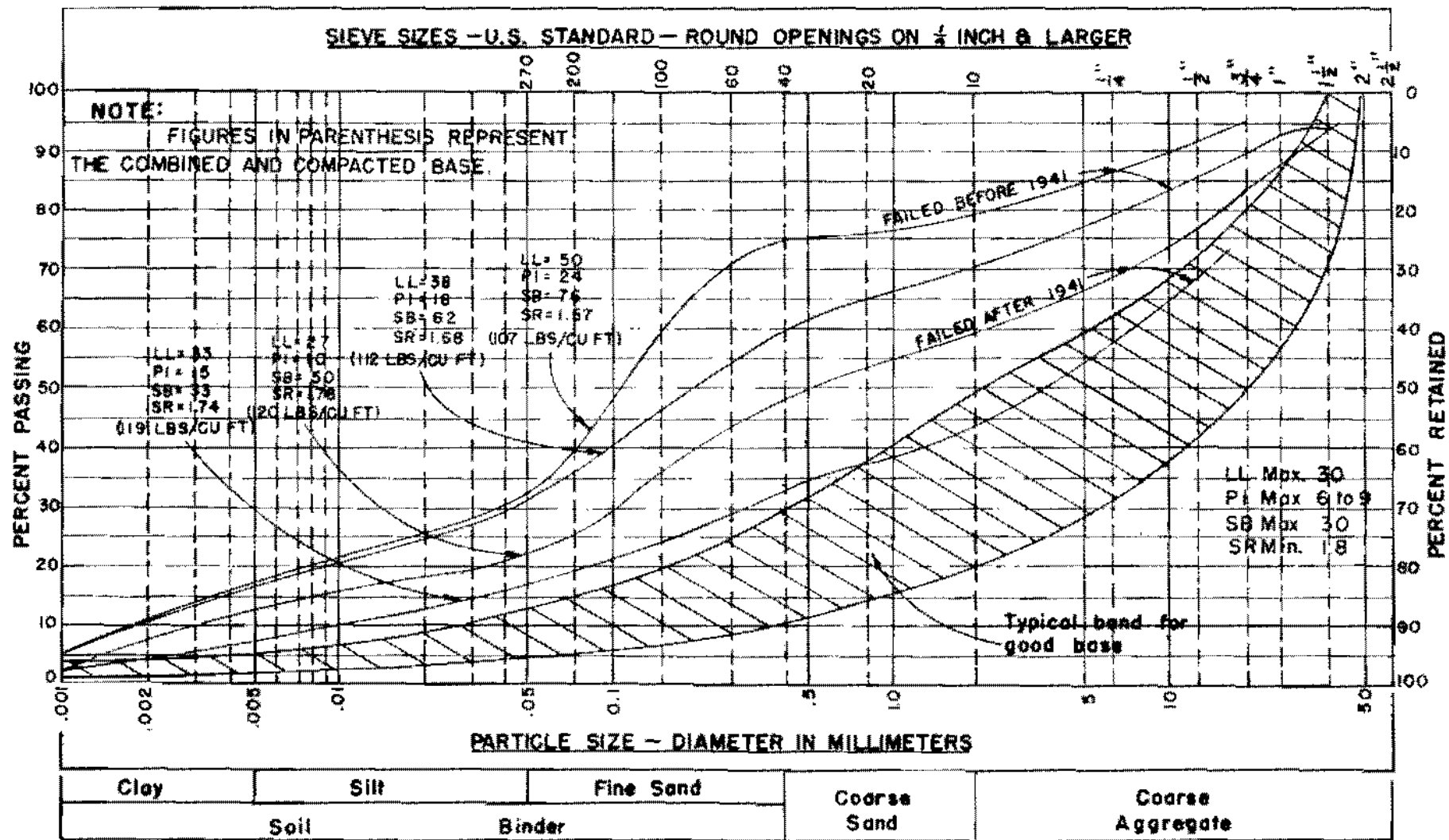
# TRIAXIAL TESTS ON CLEANED AND CRUSHED AUSTIN CHALK

Cohesion = 3 Lb.s/Sq. In.  
 Friction =  $46^\circ$   
 Modulus of Elasticity (E) = 7,000





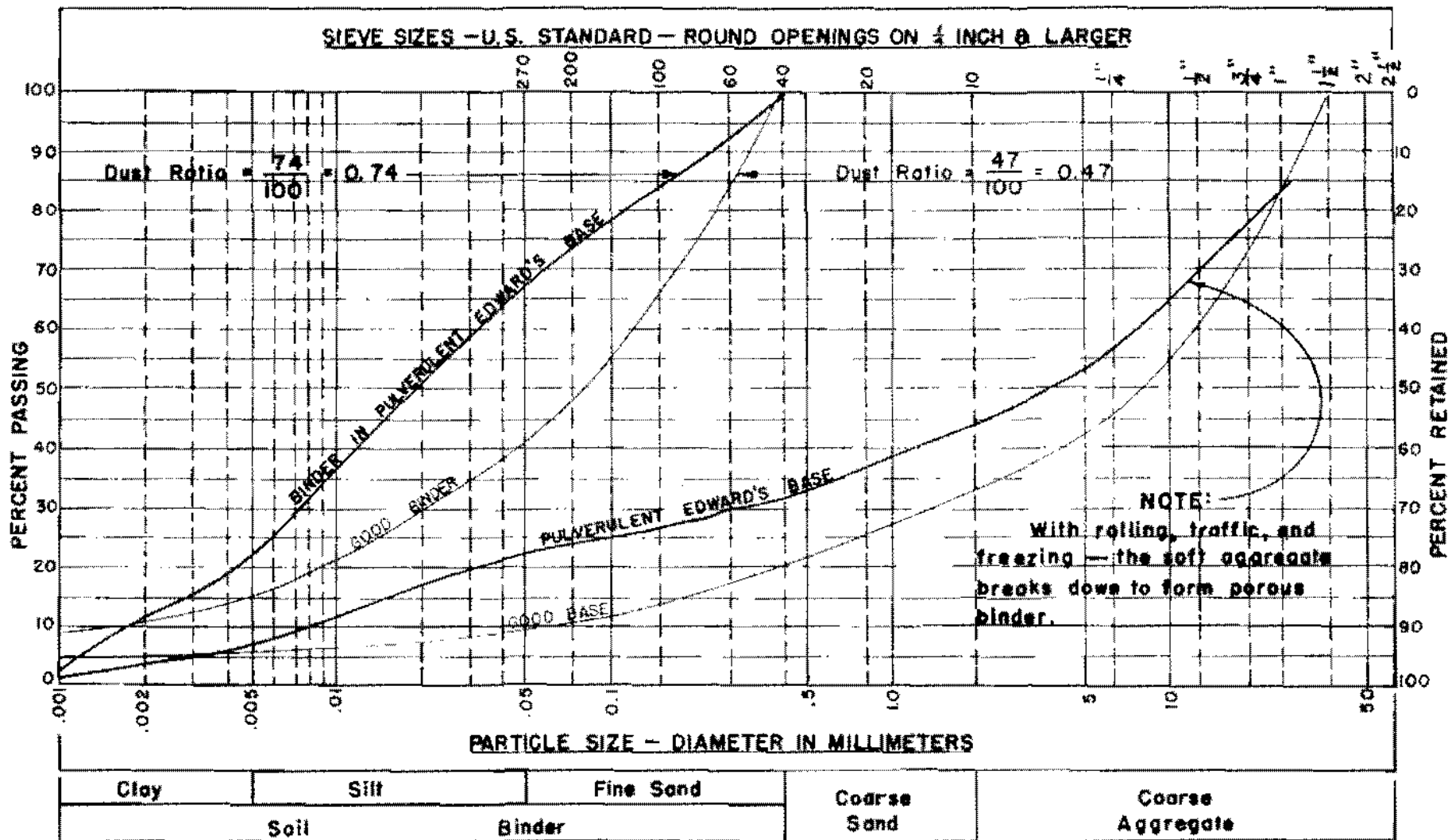
# CUMULATIVE MECHANICAL ANALYSIS OGALALLA CALICHE



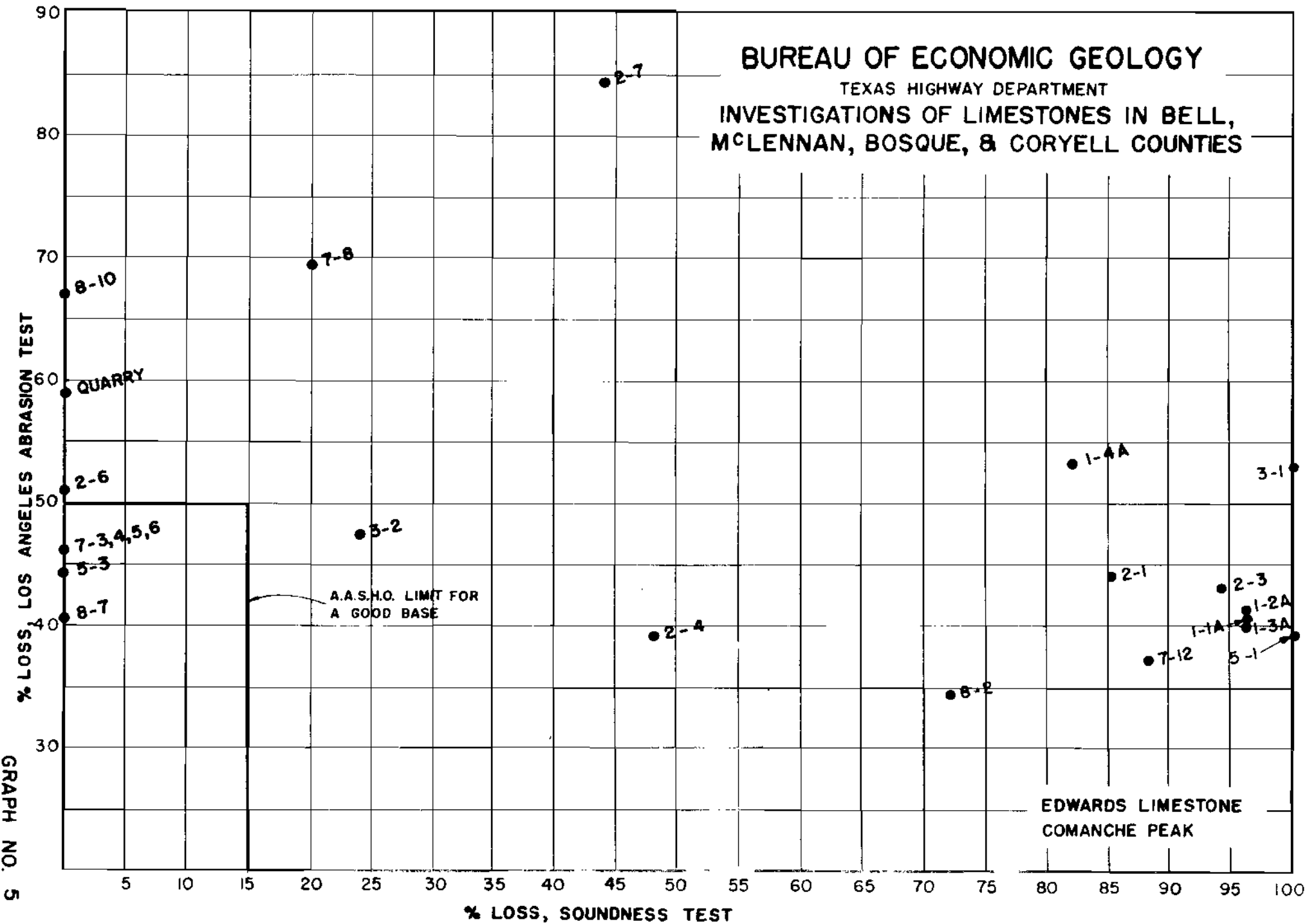
# CUMULATIVE MECHANICAL ANALYSIS

## PULVERULENT EDWARDS

LL = 41 , PI = 6 , % SB = 32 , SR = 1.43 X 62.5 = 89.4 %/CU. FT.



BUREAU OF ECONOMIC GEOLOGY  
 TEXAS HIGHWAY DEPARTMENT  
 INVESTIGATIONS OF LIMESTONES IN BELL,  
 MCLENNAN, BOSQUE, & CORYELL COUNTIES

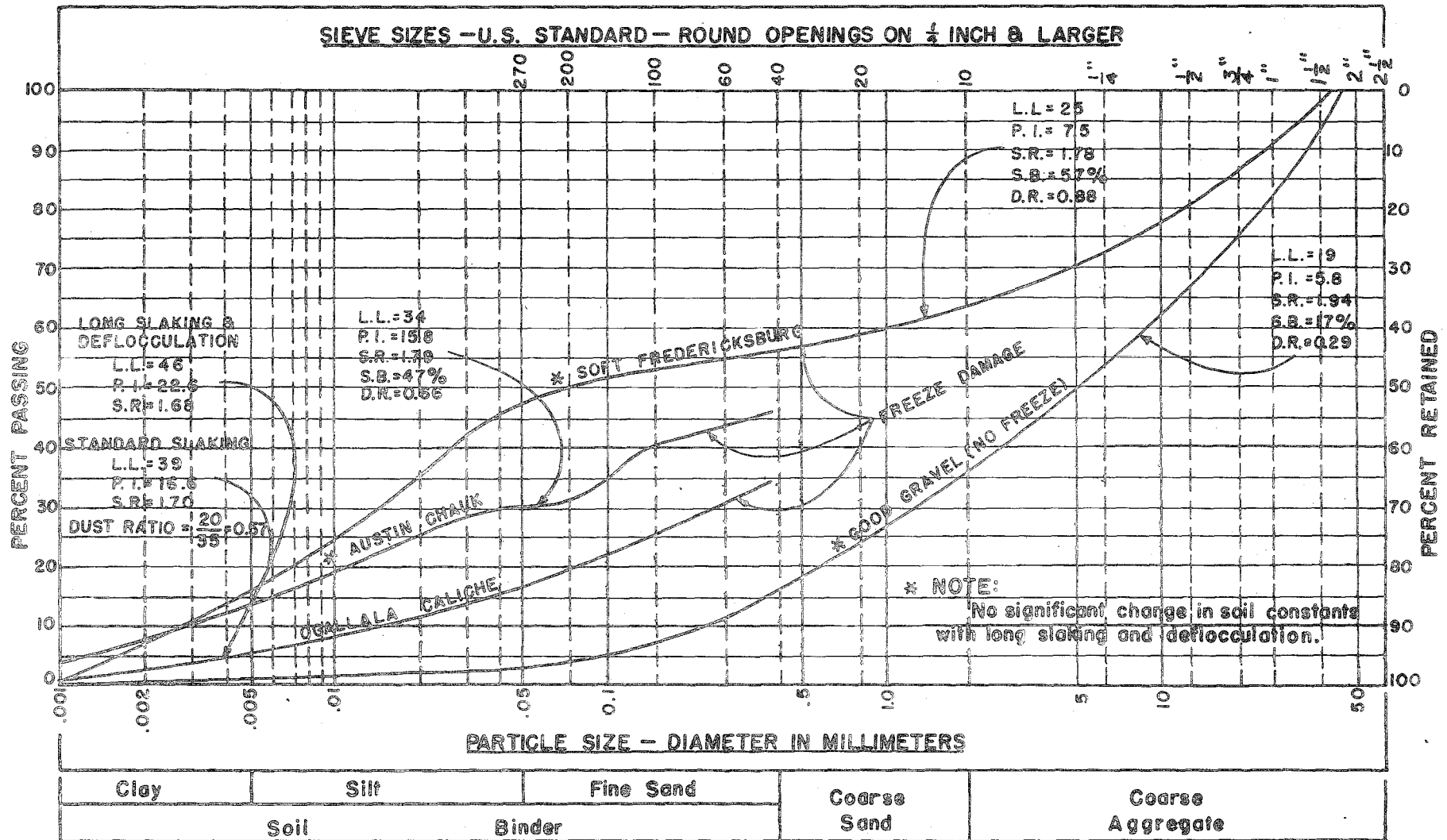


EDWARDS LIMESTONE  
 COMANCHE PEAK

GRAPH NO. 5

CUMULATIVE MECHANICAL ANALYSIS

GRADATION OF FREEZE DAMAGED BASES



TEXAS HIGHWAY DEPARTMENT

ROAD DESIGN DIVISION

1948

PICTURES OF FREEZE DAMAGED PAVEMENTS  
(Winters of 1946-47 & 1947-48)

Pictures by: R. M. Stene and  
Districts 1, 18 & 15



1. GOBER CHALK

Fannin County, Highway 121, Bonham  
to Trenton - Freeze damage to  
whiterock base with double asphalt  
surface treatment.



2. AUSTIN CHALK

Grayson County - Highway  
289, Dorchester to  
Highway 82.

Severe freeze damage to  
poor Austin chalk base -  
This particular part of  
formation in Grayson  
County has been reported  
as being more marly and  
shaley than usual.  
LL=40, PI=19, %SB= 32.



### 3. AUSTIN CHALK

Dallas County - U.S. Highway  
77, South of Dallas

This is temporary surfacing  
used several years on sub-  
base of chalk. This material  
is from best part of Austin  
chalk formation - should show  
clearly that this formation  
cannot be used as source for  
good base material.



### 4. AUSTIN CHALK

Pit used on U.S. 77  
South of Dallas.  
Appears to be about  
best material that  
can be found in this  
formation. See  
Picture No. 3 for  
view of pavement.



5. AUSTIN CHALK

Dallas County - U.S.  
Highway 67, Southwest  
of Dallas.

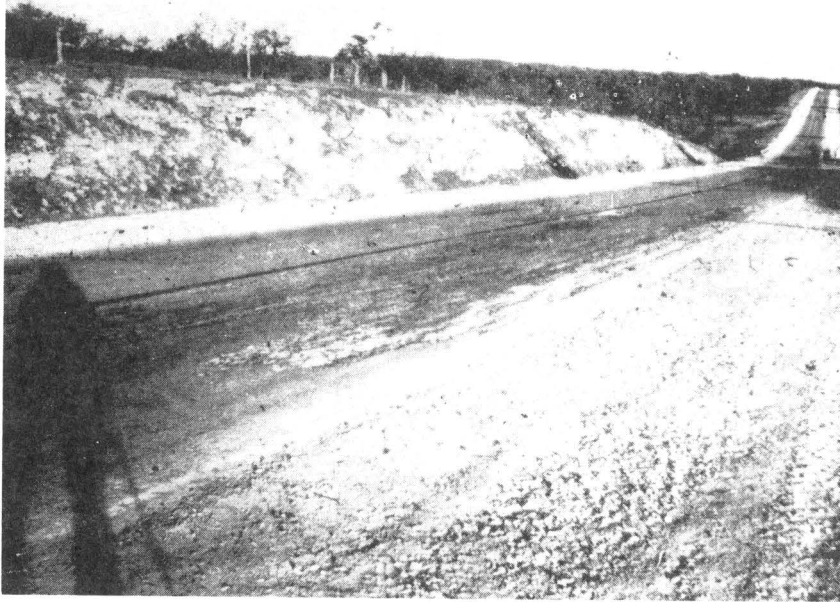
There is a very definite  
pattern of more frequent  
and more severe freeze  
damage in cuts where base  
can secure moisture by  
capillarity from high  
water table or seepage.



6. AUSTIN CHALK

Note the typical  
shaley or marly  
structure in the  
lower part of this  
pit of Austin Chalk  
used on U.S. 287,  
Ellis County, from  
Waxahachie to  
Midlothian.





7. SOFT FREDERICKSBURG  
LIMESTONE

Gillespie County, U.S. 87,  
north of Fredericksburg.  
This section of road  
reported to have about  
35% binder with PI of 6.  
Freeze damage considered  
due to soft, porous,  
limestone base.

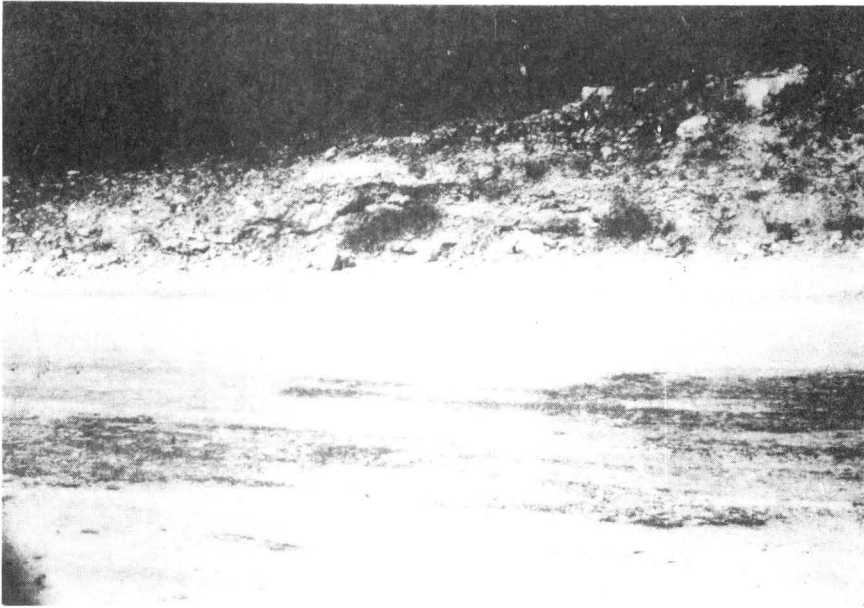


8. TRINITY CALCAREOUS CLAY

Hays County - U.S. 290

This is typical freeze  
damage on the calcareous  
clay associated with the  
Trinity Group.





9. TRINITY NODULAR LIMESTONE

Hays County - F. M. Highway 32,  
picture taken in cut in  
"Devils Backbone" ridge.

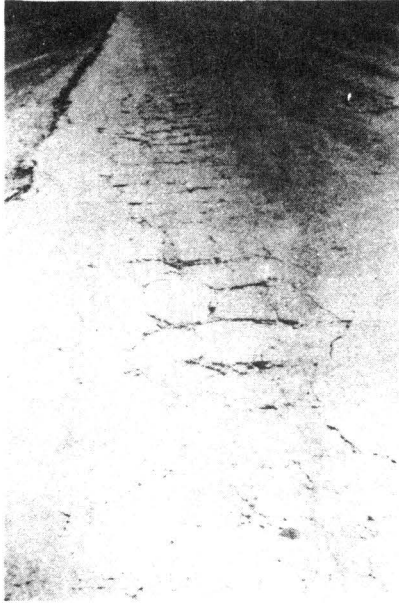
Note freeze damage in cut.  
Picture No. 10 shows detail  
of this cut.



10. TRINITY NODULAR LIMESTONE

Hays County, F. M. Highway 32,  
cut in "Devils Backbone" ridge.

Note the layer of nodular  
limestone with marly matrix  
about the center of picture.  
It appears that this marly  
limestone weathers to form  
deposits of weak calcareous  
clay associated with the  
Trinity Group.



## 11. TRINITY NODULAR LIMESTONE

Kendall County - U.S. Highway 87  
Boerne to Comfort.

This is Rock Asphalt ( $1\frac{1}{2}$ "") on high P.I. Trinity Nodular Limestone. Failure due to heavy loads on weak high P.I. base (nodular limestone with marly matrix). Typical cracking where rigid asphalt concrete is on flexible pavement with high deflections.

Tests on representative Trinity Nodular Limestone:

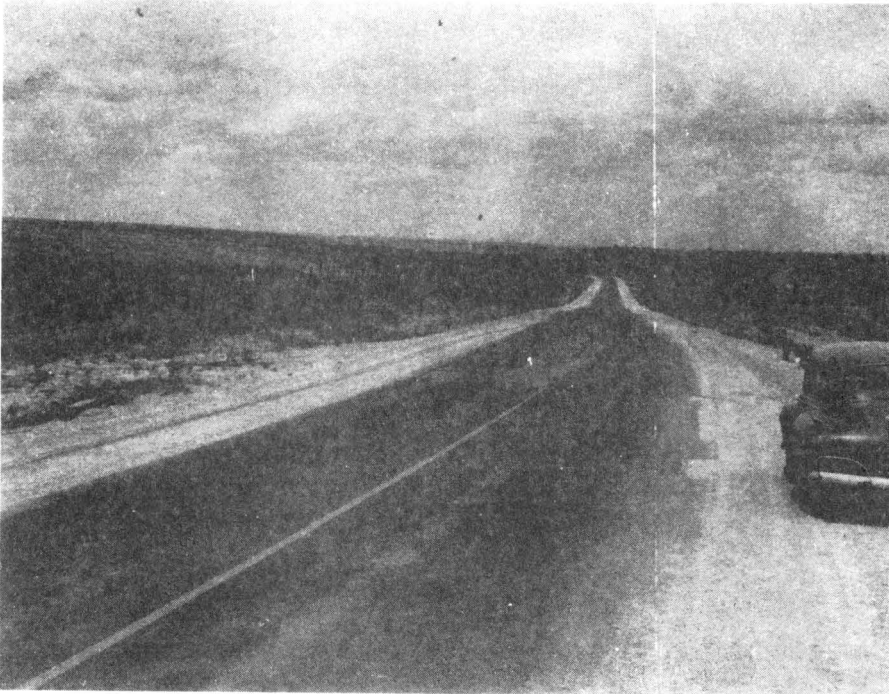
LL=33 Shrinkage Ratio = 1.92  
PI=18 %Soil Binder = 40 to 50%  
(Low LL with high SR slows up freeze damage).



## 12. TRINITY NODULAR LIMESTONE

Kendall County - U.S. Highway 87  
Boerne to Comfort.

Same road as in Picture 11. Note the white material in the cracks of the rock asphalt surface. This pavement "pumped" base material after the freeze. This appears to be half load failure on high P.I. - high soil binder base with about half freeze damage. This road, 14 years old, has had three full length seal coats and considerable base repairs and asphaltic concrete leveling.



13. COMANCHE PEAK LIMESTONE

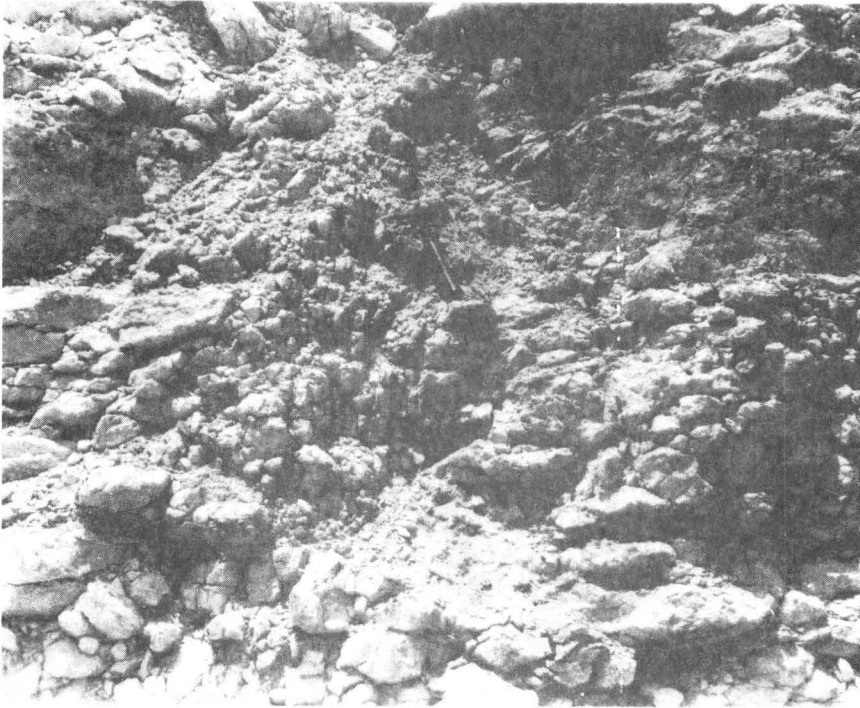
Tarrant, Parker & Johnson Counties, U.S. Highway 377 - 1 mile northeast of Cresson

This is a new flexible base with double asphalt surface treatment. According to present tests - this was considered good base, but shows rather severe freeze damage.



14. COMANCHE PEAK LIMESTONE

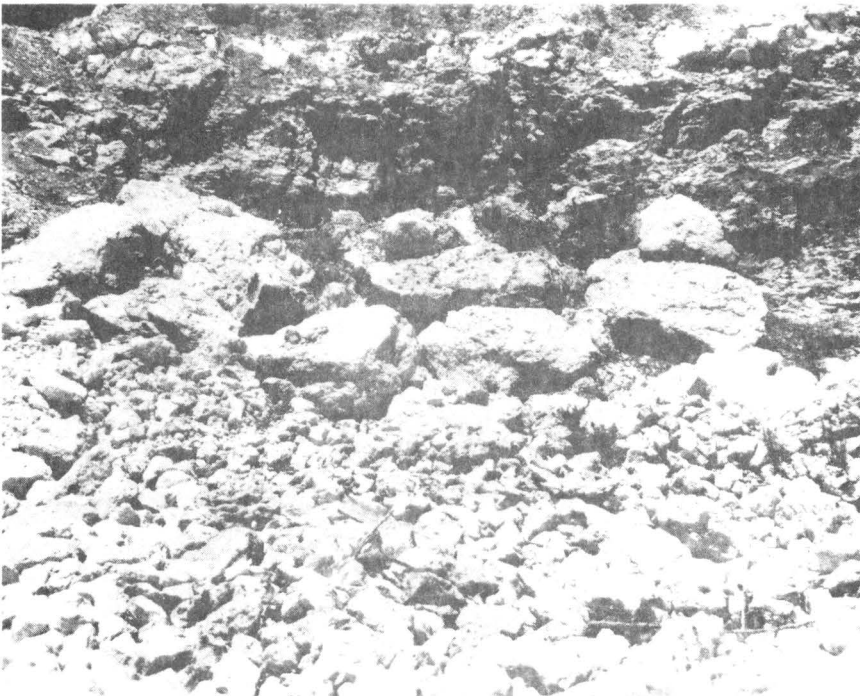
Same as picture No. 13, except located 2 miles northeast of Cresson. All soil constant tests were within usual specification requirements.



15. COMANCHE PEAK LIMESTONE

Tarrant, Parker and Johnson  
Counties - U.S. Highway 377

Pit used for base from  
Cresson to Benbrook - Freeze  
damage shown in Pictures  
13 and 14. Note the nodular  
limestone with shaley  
weathering.



16. COMANCHE PEAK LIMESTONE

U.S. Highway 377 same as  
Picture 15. The nodular  
limestone showing shaley  
weathering passed all  
present requirements. The  
weathering indicates that  
material would not pass  
soundness tests.



6" AVERAGE ANNUAL FROST PENETRATION (U.S. Weather Bureau)

6" AVERAGE ANNUAL FROST PENETRATION (U.S. Weather Bureau)

3" AVERAGE ANNUAL FROST PENETRATION (U.S. Weather Bureau)

3" AVERAGE ANNUAL FROST PENETRATION (U.S. Weather Bureau)

1" AVERAGE ANNUAL FROST PENETRATION (U.S. Weather Bureau)

NOTE: DESIGN TO PREVENT FREEZE DAMAGE IN BASES NORTH OF DASH LINE.

NOTE: DESIGN TO PREVENT FREEZE DAMAGE IN BASES NORTH OF DASH LINE.

- LEGEND**
- Ogallala formation (fine sand, silt, silty-clay, gravel, and caliche)
  - Iron ore area of the Reklaw, Queen City, Neches, Sparta, and Cook Mt. formations
  - Upper and Lower Cretaceous (limestones, chalks, marls, sands, and clays)
  - Permian (limestones, sandstones, dolomites, red shale, gypsum, and salts)
  - Pennsylvania (shale, limestone, sandstone, and conglomerate)

**BASE MATERIALS SUBJECT TO SEVERE FREEZE DAMAGE**

- ① AUSTIN, GOBER, & PECAN GAP CHALKS (Upper Cretaceous Age) Consists of porous, highly absorbent chalky or marly limestones, chalky marl, and calcareous clay.
- ② OGALLALA CALICHE (Pliocene, Pleistocene to Recent Age) Light, soft, porous, high binder, silty caliche or calcareous clay.
- ③ LOWER CRETACEOUS LIMESTONES (Washita, Fredericksburg, and Trinity Groups) Consisting of soft, porous, pulverulent limestones (mostly Edwards)...alternating layers of hard to soft limestone and marl...nodular hard to soft limestone in marly matrix...and calcareous clay in the weathered talus slopes.
- ④ IRON ORE GRAVEL with either too much clay or too much silty sand.
- ⑤ PERMIAN GYPSUM and associated salts...porous dolomite with alternating layers of sandy, silty shale.
- ⑥ GRAVEL BASES...pit-run gravel with far too much, or very poorly graded, soil binder (excess silt and clay).
- ⑦ STABILIZATION...various types of porous stabilization, including poor road mix.

**SYMBOLS**

- X Trouble not clearly including much freeze damage, but mainly due to other causes, such as overload, cold wet weather, underdesign, weak materials, very weak subgrade, etc.
- ? Unable to determine whether freeze penetrated base.
- Freeze damaged pavements reported by Districts for winters of 1946-47 and 1947-48.

**STATE OF TEXAS MAP NO. 1**