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COMMISSIONER OF HIGHWAYS

COMMONWEALTH OF KENTUCKY
DEPARTMENT OF HIGHWAYS
FRANKFORT, KENTUCKY 40601

April 17, 1972

ADDRESS REPLY TO:
DEPARTMENT OF HIGHWAYS
DIVISION OF RESEARCH
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P.3.2

MEMORANDUM TO: J. R. Harbison
State Highway Engineer
Chairman, Research Committee

SUBJECT: "Expansive Limestone Aggregate in a Concrete Pavement;" I 65, Warren-Simpson Counties; KYP-56; HPR-1(7), Part III-A.

One of the first, challenging problems I encountered in research -- now 25 years ago -- involved a concrete pavement which had deteriorated because of unsound limestone, coarse aggregate. The pavement was on US 60 between Olive Hill and Grayson -- now overlaid. The problem was traced to specific ledges of impure limestone in the source quarry; the nature of the rock was defined. In the early 1950's, similar deterioration and similar aggregate was discovered on KY 32 (Morehead-Flemingsburg). Unfortunately, no specific safeguards emerged. Quality tests in specifications then would not have detected this type of rock. The rock weathered badly but slowly when exposed. It was resolved to rely upon observations of outdrops, existing exposures, and proven performance records to avoid this type of unsoundness in the future. Years elapsed, and eventually history repeated itself. I 65, south of Bowling Green (built in 1965), was the unwary recurrence.

Blowups are now rare events; the third, et seq., and the discovery of surface cracking made a research-type investigation necessary. Our studies began in 1967. It would be presumptive to say that we expected to find much similarity with the Olive Hill episode.

About 1960, a succession of reports from various states and Canada began to speak of alkali-reactivity of carbonate rocks (dolomitic limestones). Whereas alkalies are present in slightly more than trace quantities in portland cement, the rock at Olive Hill decomposed in air (in the absence of alkalies -- that is, neglecting atmospheric substances). Some reports implied no relationship between clay impurities and expansion. I had also learned from sedimentary petrology that co-deposited clay impurities interferred with crystal growth and thereby limited the size of calcite and dolomite crystals to the most minute dimensions. Metamorphosed rocks (larger crystal, little impurities) were not expansive. Perhaps I was too skeptical. It has since been recognized by others that 10- to 12-percent $MgCO_3$ and the most minute crystallization are telltale attributes of expansive rock. Meanwhile, some 10 miles of 4-lane pavement on I 65 was "poisoned" by these insidious rocks.

Late in 1967, the PCA obtained samples of ledgerock from the Hoover quarry and, in 1968, reported identification of expansive ledges. We independently confirmed their findings altogether.

I believe you will find the narratives here and in the report interesting but somewhat demanding of additional safeguards in aggregate quality requirements. I recommend the adoption of ASTM C 586 as a source-approval measure -- to which the Division of Materials may make recourse when ledgerock is suspected as being expansive. It would seem unnecessarily burdensome to perform this type of test on all source quarries and ledges not exhibiting the telltale attributes herein described. The permissive limits of expansion depend upon the duration of the test. This detail may be resolved in conference with the Division of Materials.

The condition of the pavement will, indeed, worsen with passing of time. No blowups occurred during the past year; however, additional relief joints may be needed from time-to-time. The surface

cracking may eventually mature into extensive spalling. Bituminous overlays are the customary remedy.

When the surface cracking was first discovered, it seemed to be associated with the wire mesh reinforcement -- which had been vibrated into the fresh concrete. There was a striking similarity between the crack configuration and the mesh. Cores extracted from the pavement showed the cracks directly above and in line with the wire strands. At one point (Core 6, in the report), three small dimples -- looking very much like finger impressions -- caused suspicion that someone detected the crack before the concrete hardened. And so, I cannot dismiss the possibility that the depression of the mesh induced at least a "latent" crack configuration. On the other hand, classical cases of expansive aggregate reported thus far have invariably exhibited cracking very similar to that observed on I 65 -- perhaps the I 65 cracking is somewhat more orderly.

For obvious reasons, there was much concern about the welfare of the pavement after the cracking became evident. In fact, on June 6, 1968, three clear-type coatings (protective) containing fluorescent materials -- for tracing depths of penetration -- were applied in the vicinity of Stations 432+07 to 432+43 (SB). Cores were extracted a few days later; the coatings went into the cracks to their full depth but did not fill them. After considering costs and skid-resistance, I recommended against any form of coating. This antithetical notion stands somewhat indefensible in comparison to the idea of preventive maintenance. Even so, my intuitive inclination was to wait and see how badly the pavement might suffer (if at all) in time. Nothing has happened in the interim to convince me that a coating would have served a useful purpose.

I believe even now that the pavement is in a wait-and-see condition. We should proceed, however, to safeguard future construction, as I have recommended.

Respectfully submitted,

Jas. H. Havens
Director of Research

JHH:dw
Attachments
cc's: Research Committee

Research Report
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EXPANSIVE LIMESTONE AGGREGATE IN A CONCRETE PAVEMENT

I 65-1(13)13, MP 114-263-E
Warren - Simpson Counties

I 65-1(17)6, MP 107-175-C
Simpson County

KYP-56
HPR-1(7), Part III-A

by

Jas. H. Havens
Director

and

Assaf Rahal
Research Engineer

Division of Research
DEPARTMENT OF HIGHWAYS
Commonwealth of Kentucky

April 1972

INTRODUCTION

Recurrent blowups and surface cracking are common symptoms of distress in concrete pavements. Premature appearance of distress symptoms is alarming because the materials used in the concrete become suspect. Criteria for design, quality of materials, and construction are necessarily re-evaluated. Indeed, a dutiful effort to discover the cause(s) and to provide future safeguards is reasonably expected.

The analysis of causative factors besetting I 65-1(13)13 was complicated by an intuitive notion that blowups and surface cracks might be separate and independent problems. The crack pattern resembled the configuration of the wire mesh -- which was vibrated into position after the concrete was spread and screeded. The blowups are, as the evidence presented here will show, attributable to expansive forces arising from a limestone aggregate which now has been identified with specific ledges in the source quarry. The nature of this aggregate was such that its deleterious or expansive character would not have been detected by the specification tests and routine safeguards then in effect. However, insights extending beyond specification requirements surely would have made the ledges suspect had they been brought to bear in this instance.

This report includes a relevant history of the project and results from the several investigative tests undertaken.

PROJECT HISTORY

The affected pavement begins at Station 696+75 on Project I 65-1(17)6 which, at Station 735+65.4 northward (Simpson-Warren County Line), equates to Station 0+00, the beginning of I 65-1(13)13, and continues northward in Warren County to Station 450+00 (in the US 231 interchange) -- a distance of 9.26 miles. The Drakes Creek bridges are between Stations 702+00 and 704+25 in Simpson County. Figure 1 is a strip map on which pertinent chronological events are noted. Paving began June 10, 1965, at Station 450+00 and proceeded southward in the northbound lanes. No cracking or blowups have been found north of Station 450+00 nor south of Station 696+75 in Simpson County. The contractor was the W. L. Harper Construction Company. Harper's work extended southward through Station 322+00 (I 65-1(17)6) at the Franklin interchange and to Station 118+31.02 (I 65-1(16)2) at the US 31-W interchange south of Franklin. All paving was completed in 1965. Station 696+75 has been superscripted with an asterisk to identify the point at which the contractor relocated his concrete batching plant and changed source of water. It has already been mentioned that from this point southward the cracking appears to diminish. At Milepost 11.4 (1.5 miles south of Drakes Creek), only short transverse cracks are perceptible.

The first blowup occurred May 9, 1966 -- at Station 319+10 (southbound). The second occurred on or about June 14, 1966 -- at Station 224+00 (northbound). Apparently, the crack pattern was not discovered until the summer of 1967. Installation of a regular series of relief joints was begun in the spring of 1968 but was later discontinued. Blowups occurred while sawing relief joints. No blowups occurred during 1969, but two occurred in 1970 -- one was a repeat occurrence; none occurred during 1971. Additional relief joints were installed near the Drakes Creek bridges -- where the pressure of the pavement apparently sheared the abutment wall of the bridge.

A photographic summary of the problems besetting the project is presented in Figures 2 through 12.

EARLY INVESTIGATIONS

The eventual magnitude of the problem was not apparent or forseeable when the first blowup occurred. Swiss hammer readings were taken on both sides of the affected joint. The situation is diagrammed in Figure 13. The slab on the south side of the joint was quite strong; concrete on the north side was extremely weak (1200 to 2000 psi) but increased in strength farther from the joint. Two cores were extracted near the joint (Figure 13), and these tested 2,324 and 3,451 psi. About 25 feet of the weak concrete was removed in making repairs. Inasmuch as expansion-type joints are not used in Kentucky (except at approaches to bridges), and whereas estimates indicate that thermal stresses in the order of 2,000 to 3,000 psi may arise, it seemed logical that failure occurred -- because of weakness. The second blowup was rationalized in the same way; both were at construction joints. At one time, it was determined from records that the weak concrete occurred at the continuation of paving from or abutting to a previous terminus. Similar weaknesses were expected throughout the projects. In anticipation of this, a Swiss hammer survey was made (October 10-20, 1967) from Station 696+75 to 450+00. A summary is given in Table 1. Three sites (Stations 123+50, 184+00 and 195+00 in the northbound lanes) appeared to be weak, but they have not blown up. Otherwise, the hammer indicated strengths ranging from 4200 to 5600 psi.

**8 ft. of pavement blew up (May 16, 1968) while removing filler material from relief joint.

***Occurred April 25, 1968 while removing concrete from sawed relief joint; inside lane repaired.

****Slab pushed up onto filler in relief joint (8-ft. length of concrete replaced (Summer 1968)

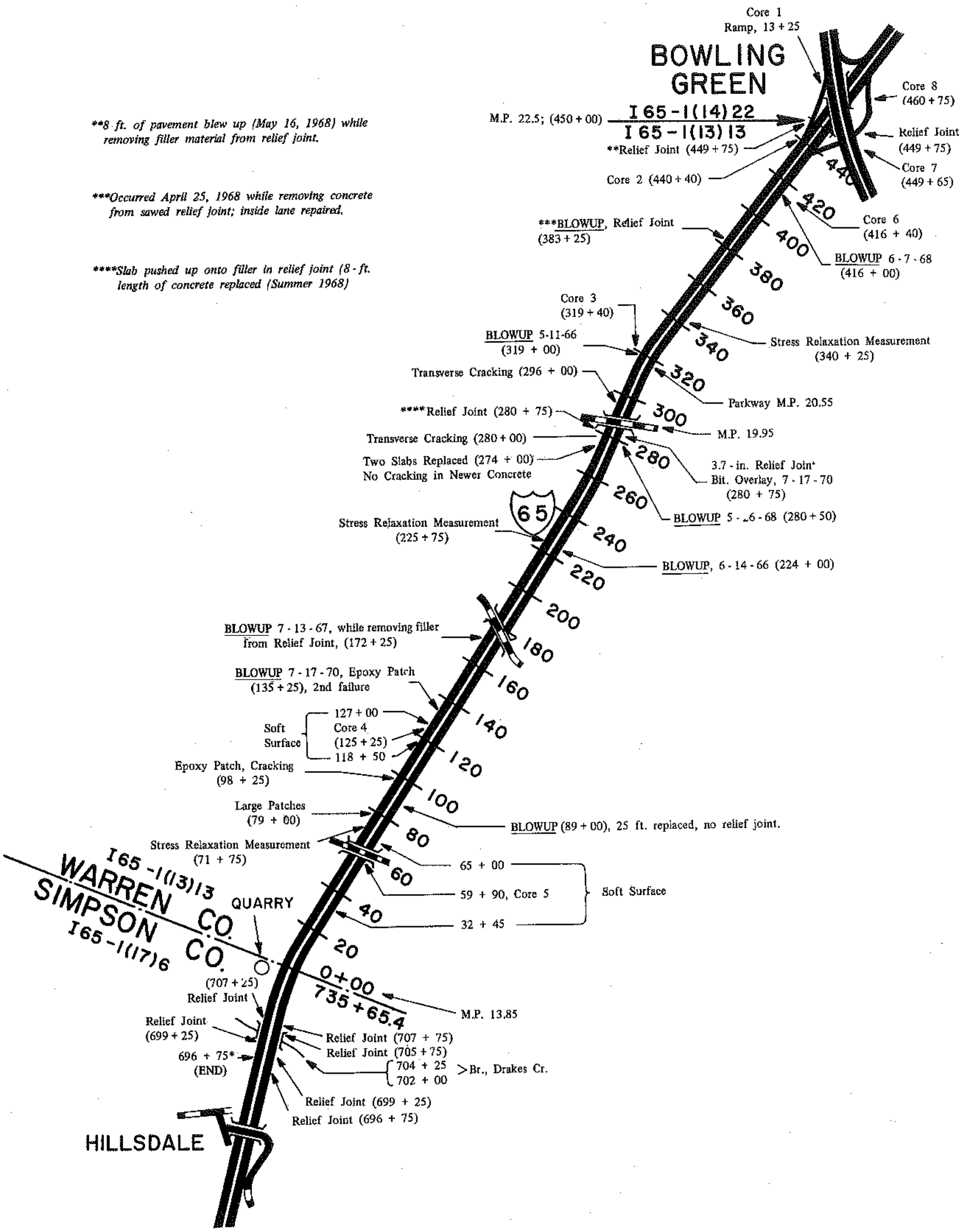


Figure 1. Location of Significant Sites and Events.

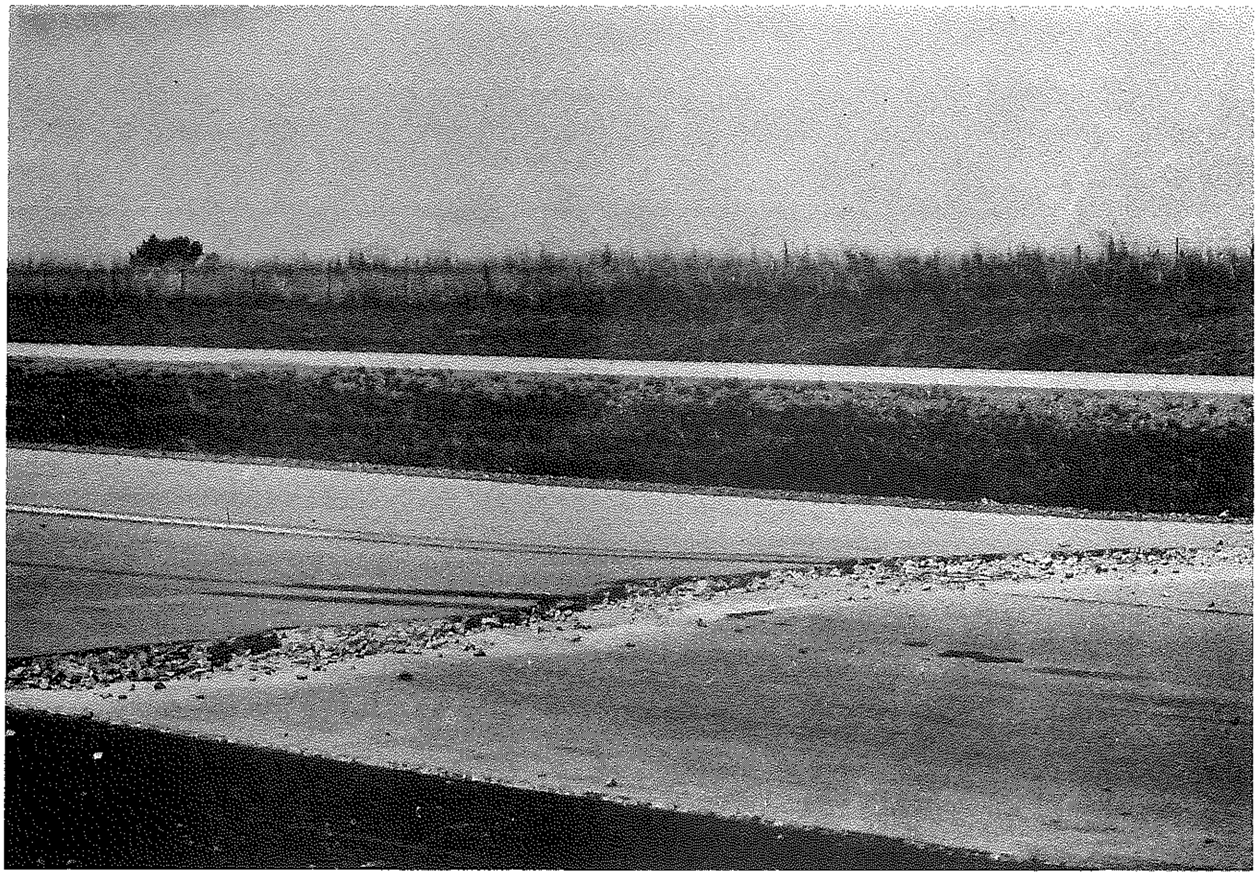


Figure 2. I 65, Second Blowup, Station 224+00 NB; June 14, 1966.

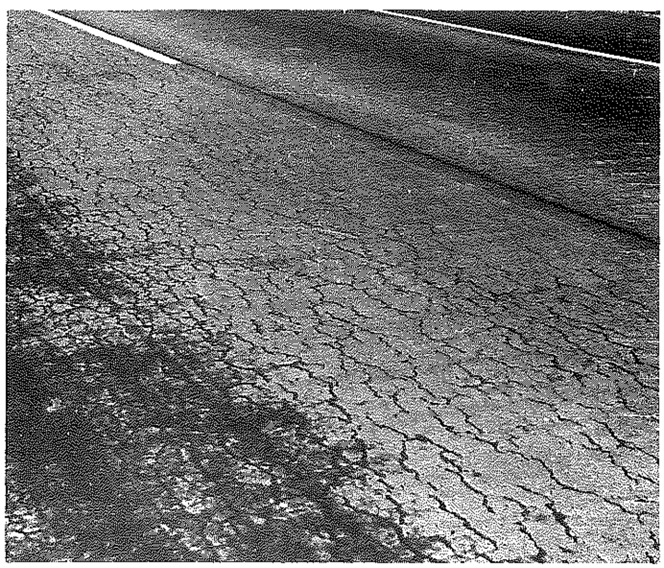


Figure 3. I 65, Crack Pattern Made Obvious by Moisture after a Rain; October 1967.

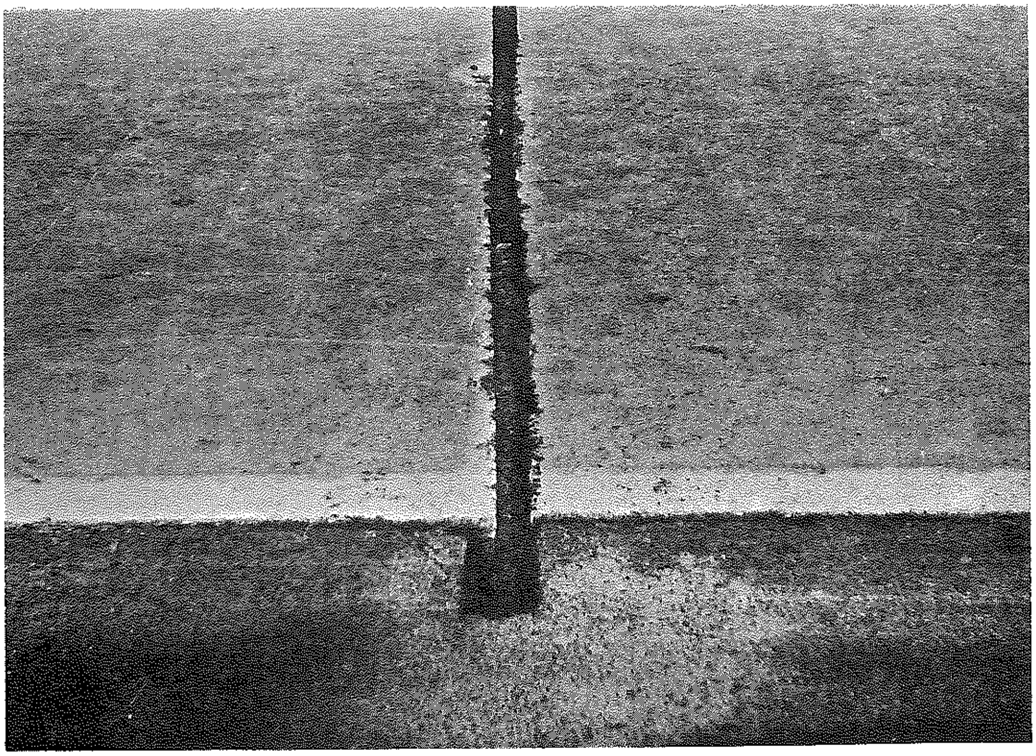


Figure 4. Closure of Relief Joint, Station 449+75 NB; July 15, 1968.

Figure 5. Closure of Relief Joint, Station 707+75 NB; July 15, 1968.

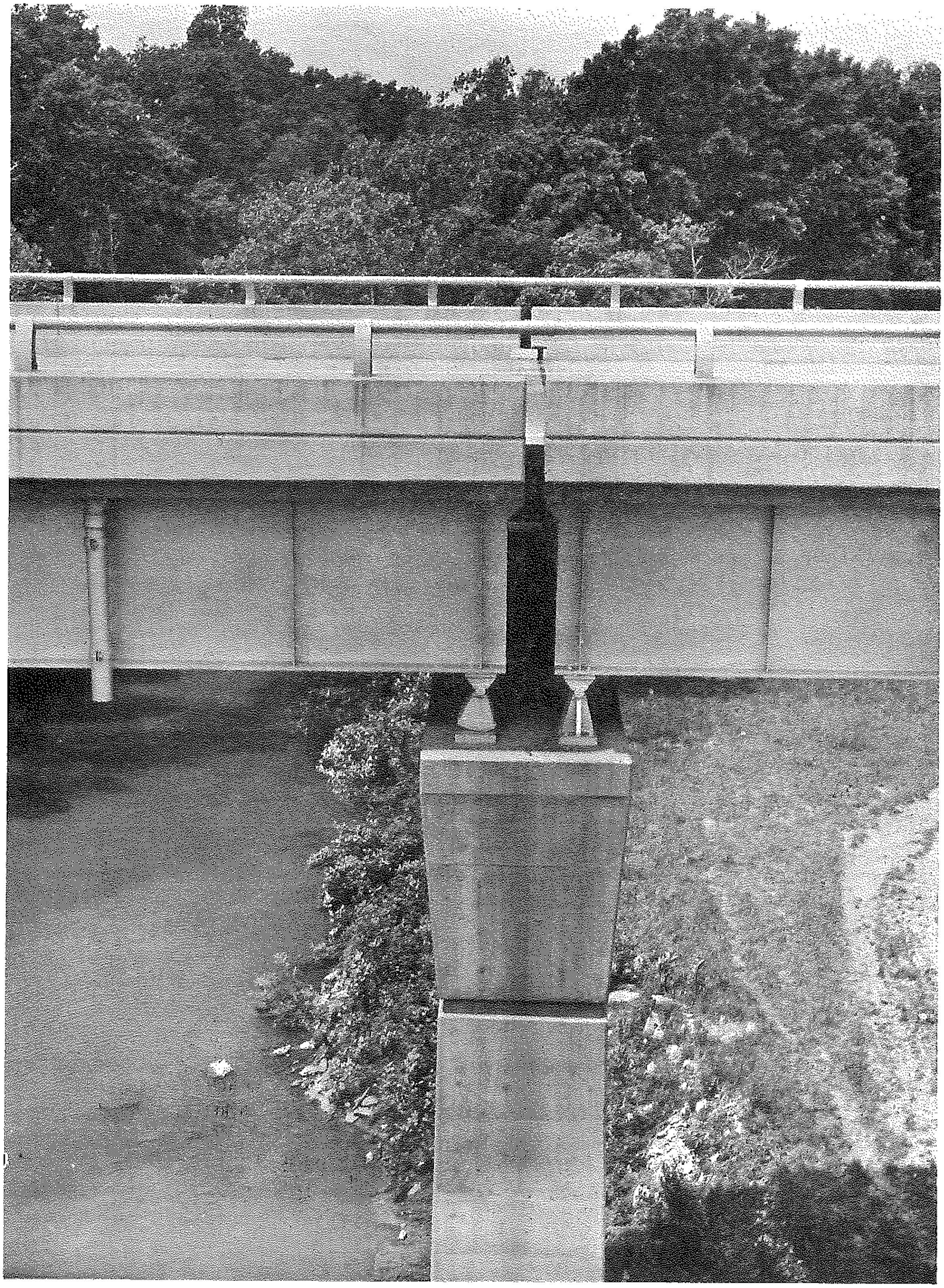


Figure 6. Bridge over Drakes Creek, I 65-1(8), Showing Northward Thrust of South Abutment and Closure at Center Pier. South abutment wall sheared, subsequently repaired; damage attributed in large part to expansion of pavement south of bridge.

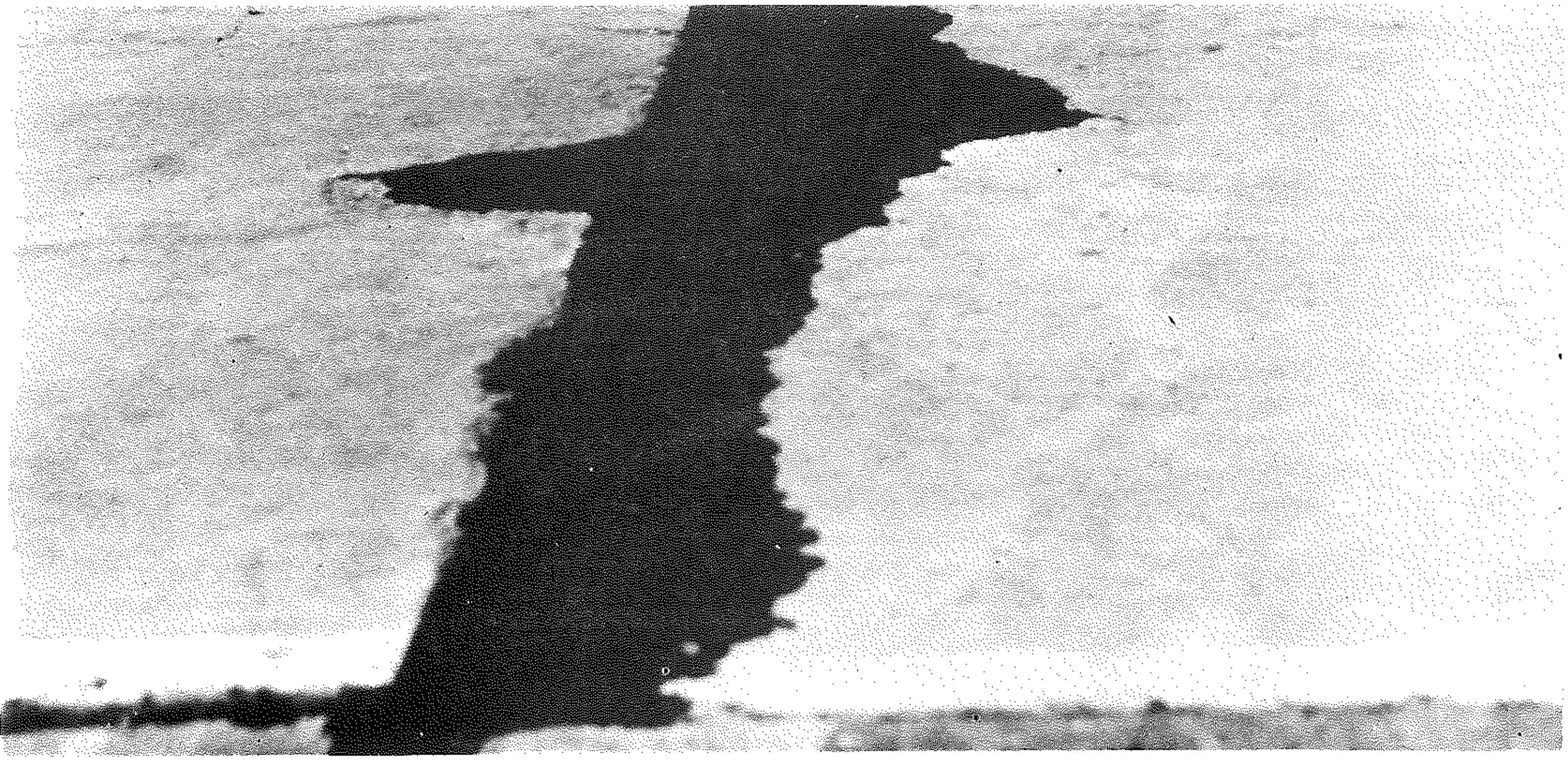


Figure 7. Relief Joint Sawed, Filled, then Wedged, Station 280+75 NB; May 23, 1968.

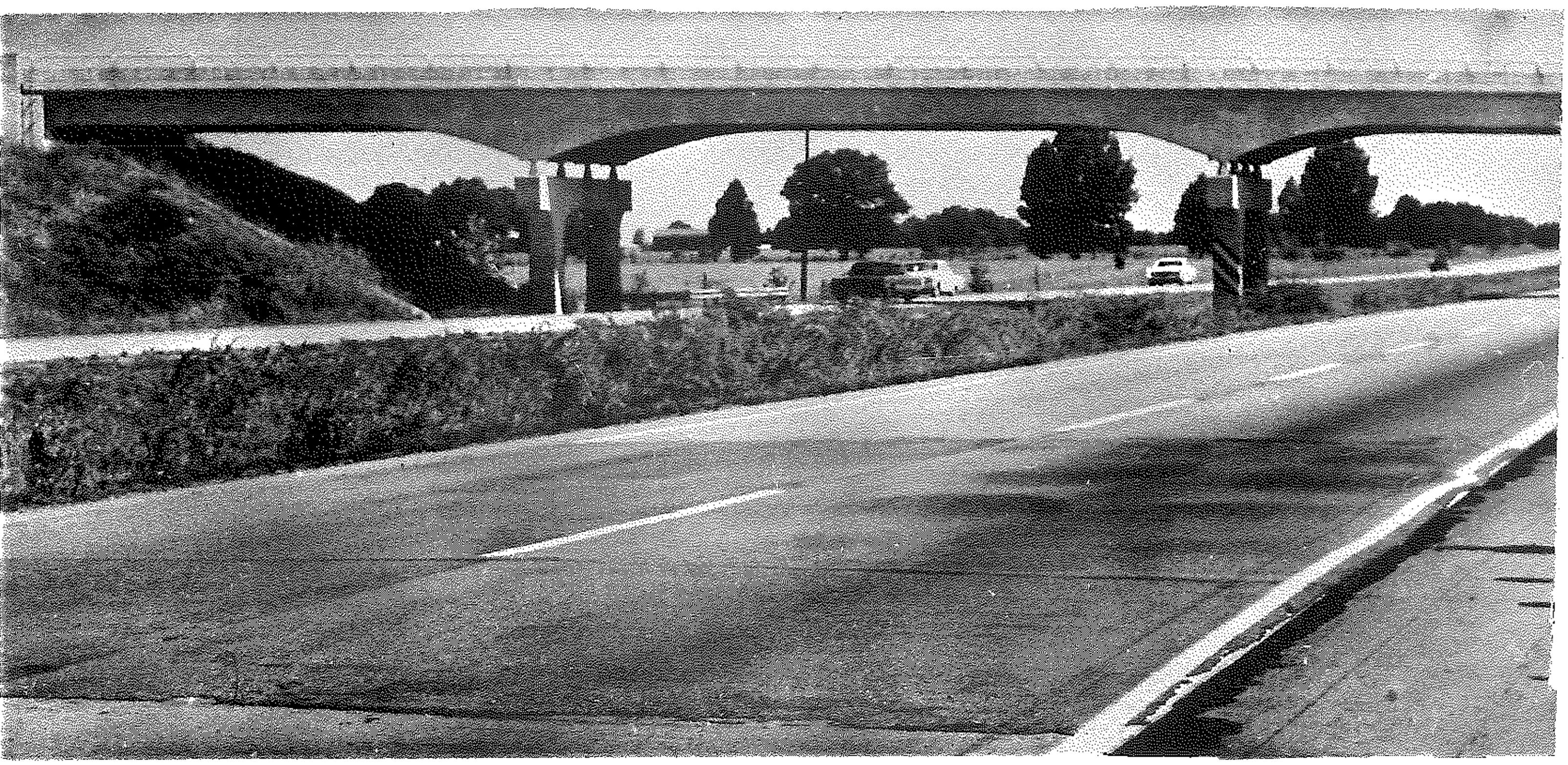


Figure 8. Same Site as above with Bituminous Patch, Station 280+75 NB; July 17, 1970.

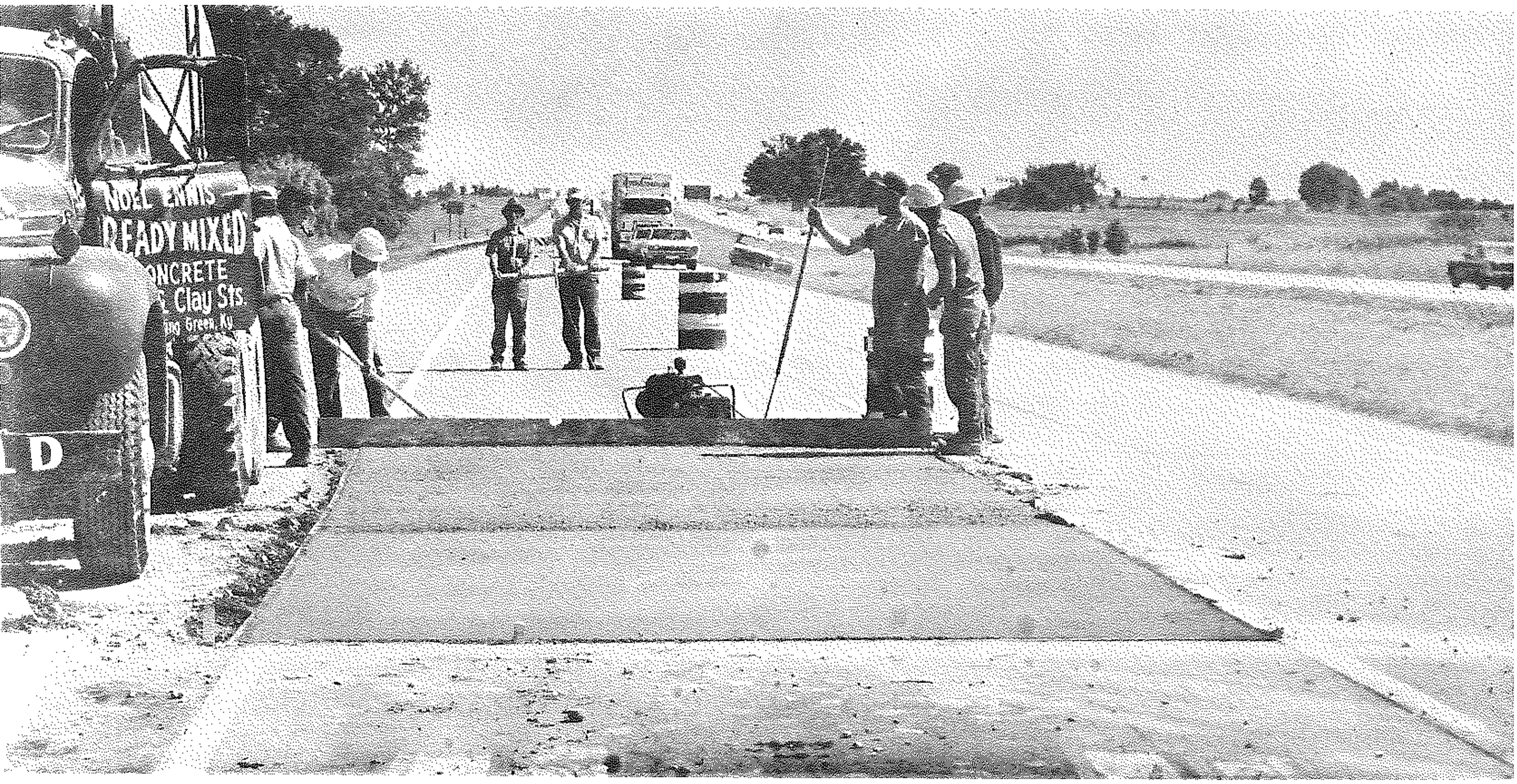


Figure 9. Station 383+50 SB; July 17, 1970. Inside lane was repaired previously. Relief joint is provided on the near edge of the patch. Note cracking in foreground.

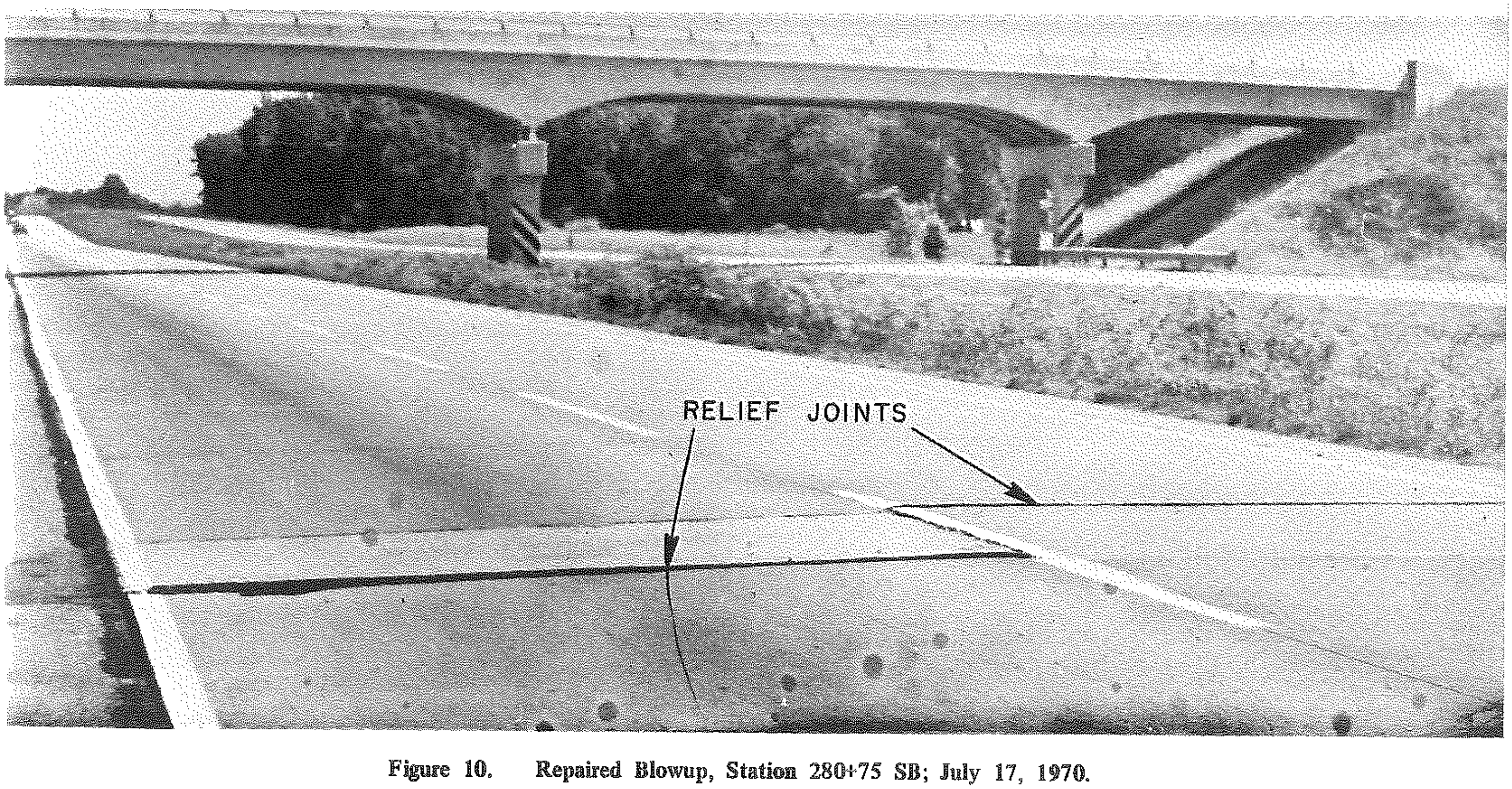


Figure 10. Repaired Blowup, Station 280+75 SB; July 17, 1970.

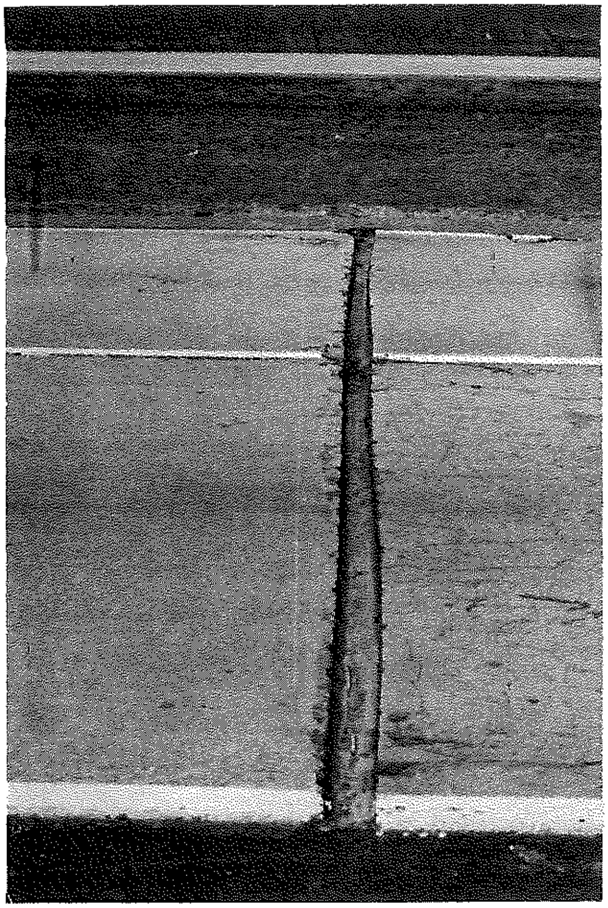
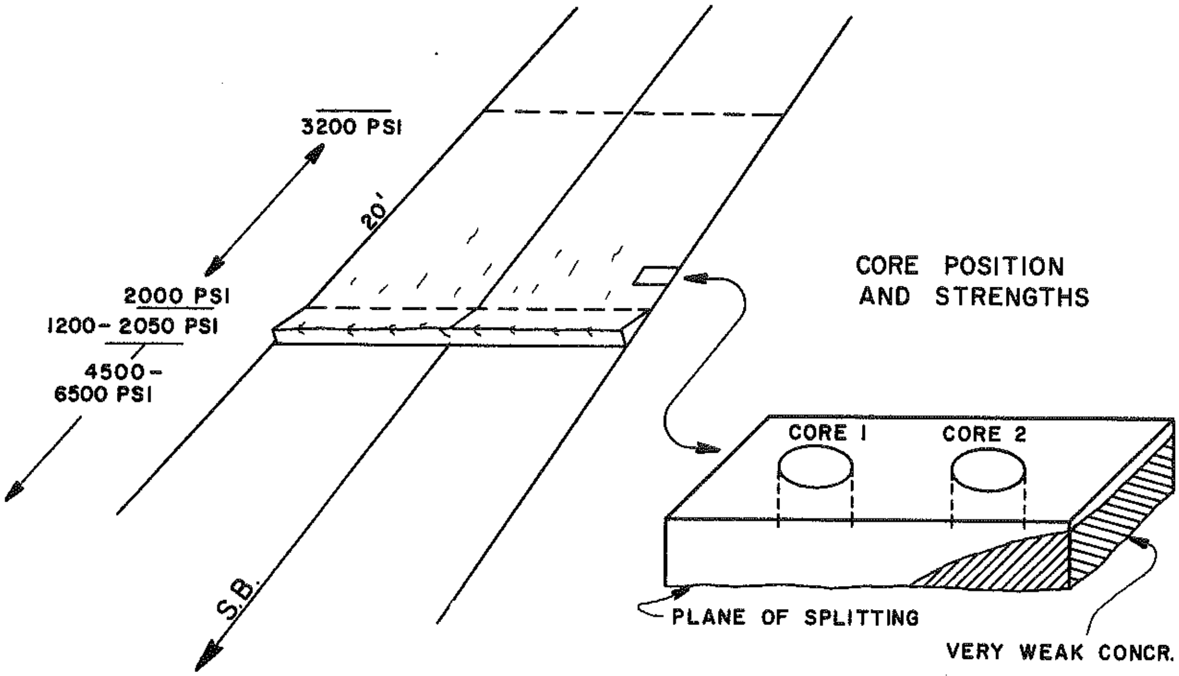


Figure 11. Under-filled Relief Joint, MP 21; March 7, 1972.

Figure 12. Rubberized Asphalt Filler Extruded from Relief Joint, near Drakes Creek, Due to Closure; March 7, 1972.



CORE	LENGTH	DIAMETER	LOAD	COMPR. STR. (CORRECTED)
1	6.40 ins.	4-3/16 ins.	49,000 lbs.	3,451 psi
2	6.25 ins.	4-3/16 ins.	33,000 lbs.	2,324 psi

Figure 13. Swiss Hammer Rebound Readings and Indicated Strengths; First Blowup, Station 319+00 SB.

TABLE 1. SUMMARY AND LIST OF DEFECTS OBSERVED DURING SWISS HAMMER SURVEY; OCTOBER 10-20, 1967

SOUTHBOUND LANES

15 unpatched spalls
10 areas where steel and steel rust were showing
20 areas where trash molded irregularities in the surfaces

STATION	
705+32	One square foot of hollow concrete
722+50 to 733+00	Many patched spalls (epoxy)
3+00 to 5+00	No cracking
5+25	Soft surface
21+50 to 25+00	Soft surface
78+40 to 79+00	Large patches
118+50 to 127+00	Soft surface
129+50	Soft surface
206+50 to 215+00	Small scale spots in outer lane
215+00 to 235+00	Small scale spots in both lanes
273+00 273+50 274+00	Sections replaced during construction
280+00 to 285+50	Transverse hairline cracking, 4' -6' long
293+00 to 296+00	Transverse hairline cracking, 4' -6' long
311+00 to 319+00	Rough surface
319+00 to 319+20	Section replaced during construction
437+00 to 444+00	Scaling in outer lane
444+00 to 450+00	Scaling in both lanes

NORTHBOUND LANES

10 unpatched spalls
15 areas where steel and steel rust were showing
25 areas where trash molded irregularities in the surface

STATION	
728+50 to 729+00	Soft surface
32+45 to 65+00	Soft surface
88+75 to 89+00	Replaced section
123+50	Construction joint, soft surface one side of joint
184+00	Construction joint, soft surface one side of joint
195+00	Construction joint, soft surface one side of joint
224+00 to 224+50	Replaced section two different pours
224+30	Crack in outer lane
291+50 to 303+50	Scaling, small pop-outs
415+00	Spall, hollow beneath surface
416+00 to 423+00	Scaling in outer lane
416+00 to 424+50	Grainy surface
444+50 to 450+00	Severe map cracking

CORES

Eight cores were drilled on May 22, 1968. Their locations and descriptions follow:

Core 1. Obtained from SB ramp to I 65 at US 231 interchange, Station 13+25, 4 feet from right shoulder. No cracking of pavement was noted on this ramp. Mortar matrix of core was hard.

Core 2. Obtained from SB lane, Station 440+40, 3 feet from left shoulder. Cracking varied in width from hairline to 1/32 inch. Mortar matrix was slightly softer and slightly darker than Core 1. Depth of cracks varied from 1/16 to 1/2 inch.

Core 3. Obtained from SB lane, Station 319+40, 9 feet from right shoulder. Popouts and 1/32 inch cracks were noted; this location was near a blowup at Station 319+00. Mortar matrix was soft and slightly darker than Core 1. Crack depth was 7/8 inch. Appearance of core suggests a low compressive strength. Top surface was rough.

Core 4. Obtained from SB lane, Station 125+25, 5 feet from right shoulder. Cracks up to 1/16 inch in width were noted. Mortar matrix was hard, and crack depth was 2 inches. Color was similar to Core 1.

Core 5. Obtained from NB lane, Station 59+90, 9 feet from right shoulder. Cracking varied from hairline up to 1/16 inch in width. Mortar matrix was slightly softer than Core 1. Color was similar to Core 1. Depth of cracks varied from 1/2 to 1 inch.

Core 6. Obtained from NB lane, Station 416+40, 5 feet from right shoulder. Surface texture was rough, cracks up to 1/16 inch in width were noted. Color was similar to that of Core 1, but the mortar matrix was slightly softer. Depth of cracking was 1-1/2 inches.

Core 7. Obtained from NB lane, Station 449+65, 4 feet from left shoulder. Closely spaced cracks up to 1/16 inch in width were noted at this section. Color of core was similar to that of Core 1, and the matrix was hard. Depth of cracking varied from 1-1/4 to 2 inches.

Core 8. Obtained from NB lane, Station 450+75, 4 feet from left shoulder. Core obtained from project north of I 65-1(13)13. No cracking of pavement noted. Matrix was hard, and color of core was lighter than cores obtained from I 65-1(13)13.

These cores were kept to illustrate the nature of the cracking. No tests were performed on them. Two were forced apart along the crack to determine (visually) the possibility of deeper, latent, or formative crack fronts. Each one split in a nearly horizontal direction and exposed fresh surfaces in the horizontal plane. Core 6 had "dimples" in the surface; these were in line with a crack; they appeared very much like finger prints.

Core 1 was obtained from the southbound ramp from US 231. The ramps and tapers were paved during 1965 but some time after the mainline paving was completed. At the time of coring, cracking was not apparent on either the southbound or northbound ramps. In March 1972, cracking was adjudged to be as extensive as on the mainline pavement -- but not as prominently revealed.

Core 8 was taken immediately north of the affected pavement. This project was paved in 1966. *(Some of these cores will be subjected to weathering on the roof deck of the Research Division Laboratory).* Cores 5, 3, and 6 are shown in Figure 14.

MEASUREMENTS OF PAVEMENT MOVEMENT

During the Fall of 1967, index lines were painted across the joint between the edge of the concrete pavement and the bituminous shoulder. Differential movement would, thereby, be readily visible. These reference points were surveyed on May 23, 1968, and on July 8, 1968. The only movements of any significant magnitude occurred at relief joints. Closures in the order of 1 inch (from each direction) were noted. The intervening pavement remained passive. The record of these measurements is presented in Table 2.

Figure 14. Cores Showing Cracks at Surface and Depth. Cracks have been wetted. Note indentation in surface of core at right; these appear to be finger imprints and suggest that someone recognized the crack before the concrete hardened. Note too that this crack is directly over a strand of steel (blackened).

TABLE 2. MEASUREMENTS OF PAVEMENT MOVEMENT

LANE	STATION	REFERENCE	MOVEMENT (inches) 5-23-68*	7-8-68**	DIRECTION OF MOVEMENT
SB	450+00	Painted Line	0.1	0.1	S
SB	449+75	4.4-inch Relief Joint	0.3	0.2	S
		(Filled with Cold Patch)	1.1	1.2	N
SB	425+00	Painted Line	0.05	0.0	S
SB	400+00	Painted Line	0.1	(Obliterated)	N
SB	383+25	4.3-inch Relief Joint	0.9	0.9	S
		(Filled with Cold Patch)			
		(Repaired Blowup)	0.9	0.9	N
SB	375+00	Painted Line	0.0	0.0	
SB	350+00	Painted Line	0.0	0.0	
SB	325+00	Painted Line	0.0	0.1	N
SB	319+00	Blowup (5-11-66)			
SB	300+00	Painted Line	0.0		
SB	280+75	4.4-inch Relief Joint	1.0	1.1	S
		(Filled with Cold Patch)	1.1	1.3	N
SB	275+00	Painted Line	0.0	0.0	
SB	250+00	Painted Line	0.1	0.1	N
SB	225+00	Painted Line	0.1	0.2	S
SB	200+00	Painted Line	0.05	0.0	N
SB	175+00	Painted Line	0.2	0.5	S
SB	172+25	4.2-inch Relief Joint	0.9		S
		(Filled with Cold Patch)			
		(Blowup)	1.0		N
SB	150+00	Painted Line	0.05	0.0	N
SB	125+00	Painted Line	0.0	0.0	
SB	100+00	Painted Line	0.1	0.1	N
SB	75+00	Painted Line	0.05	0.0	S
SB	50+00	Painted Line	0.0	0.0	
SB	25+00	Painted Line	0.0	0.0	
SB	0+00 = 735+65	Painted Line	0.05	0.0	N
SB	710+00	Painted Line	0.15	0.2	S
SB	707+25	4.3-inch Relief Joint		0.0	
		(Filled with Cold Patch)		0.6	S
SB	706+00	Painted Line	0.0	0.0	
SB	705+50	Painted Line	0.0	0.0	
SB	704+50	Bridge ED at End	2.0" Opening		
SB	703+25	Bridge ED at Center	2.6" Opening		
SB	702+00	End Bridge, No ED			
SB	699+25	4.2-inch Relief Joint	0.1	0.2	S
		(Filled with Cold Patch)	0.5	0.6	N
NB	696+75	Relief Joint		0.2	S
		(Filled with Cold Patch)		0.7	N
NB	699+25	4.0-inch Relief Joint	0.3	0.4	S
		(Filled with Cold Patch)	0.2	0.3	N
NB	702+00	End Bridge, No ED			
NB	703+25	Bridge ED at Center	1.7" Opening		
NB	704+50	Bridge ED at End	0.2" Opening		
NB	705+50	Painted Line	0.0	0.0	
NB	705+75	3.5-inch Relief Joint		1.5	S
		(Filled with Cold Patch)		1.0	N
NB	706+00	Painted Line	0.2	0.0	S
NB	707+75	3.4-inch Relief Joint	0.8	1.0	S
		(Filled with Foamed			
		Urethane)	0.1	0.5	N
NB	710+00	Painted Line	0.3	0.5	S
NB	735+64 = 0+00	Painted Line	0.0	0.0	
NB	25+00	Painted Line	0.05	0.0	S
NB	50+00	Painted Line	0.0	0.0	
NB	75+00	Painted Line	0.0	0.0	
NB	89+00	Blowup			
NB	100+00	Painted Line	0.0	0.0	
NB	125+00	Painted Line	0.0	0.0	
NB	150+00	Painted Line	0.0	0.0	
NB	175+00	Painted Line	0.0	0.0	
NB	200+00	Painted Line	0.0	0.0	
NB	224+00	Blowup (7-13-66)			
NB	225+00	Painted Line	0.1	0.0	S
NB	250+00	Painted Line	0.05	0.5	N
NB	280+50	Blowup			
NB	280+75	3.7-inch Relief Joint	1.1	0.0	N
		(Filled with Cold Patch)	1.3	0.5	S
NB	300+00	Painted Line	0.0	0.0	
NB	325+00	Painted Line	0.0	0.0	
NB	350+00	Painted Line	0.05	0.0	N
NB	375+00	Painted Line	0.0	0.0	
NB	400+00	Painted Line	0.0	0.0	
NB	415+00	Blowup			
NB	425+00	Painted Line	0.0	0.0	
NB	449+75	4.1-inch Relief Joint	0.7	1.6	N
			0.3	0.1	S
NB	450+00	Painted Line	0.1	0.0	S

* 75°F, Dry
** 75°F, Rain

INTERNAL STRESS MEASUREMENTS

In 1968, between August 15 and October 22, an attempt was made to assess the state of stress in the problem pavement and comparatively with respect to a normal pavement. Inasmuch as stress is not measurable directly, a unique method of measuring strain change accompanying relief of stress (relief of restraint) by freeing a small prism of concrete from the interior region of a slab was employed. Brass studs were glued to the concrete - at a nominal gage length of 10 inches. Readings were taken on the gage points with a Whittemore Fulcrum Strain Gage; then full-depth saw-cuts were made as shown in Figure 15. After the prism was freed, gage measurements were made until expansion ceased. Proportionality between stress and strain was assumed; E was assumed also. The strains and converted stresses are shown in Table 3. Six sites were chosen in the problem pavement and seven were chosen in a normal pavement (four were southward of the project and three were north of US 231). Two temperature regimes (representing different seasons) were included. At Station 497+75 and northward (north of US 231 interchange), the concrete was found to be in tension when sawed. One site south of Drakes Creek (Station 628+75, SB) was under slight tension. All other sites were in compression. These data are shown in Table 3.

As a matter of interest, others have attempted to assess the stress in problem pavements -- but in a different and more difficult way. In Ontario *(1)*, a few feet of concrete was removed; index marks indicated the elongation or closure of the pavement ends into the gap; hydraulic jacks were inserted; and the force necessary to restore the pavement to its original state of compression was taken as the static stress in the pavement. Attempts to transfer the compressive forces to jacks -- and to maintain a passive state -- were not successful.

AUTOCLAVE EXPANSION

Perchance, two supernumerary cylinders (Identified as Nos. 26 and 34) had been saved from the construction control series and held in storage at Bowling Green. Upon request, they were obtained for study (February 1968). Two 2 x 2 x 10-inch prisms were sawed from each cylinder; gage points were set in the ends; the specimens were then autoclaved (3 hours, 295 psi). After cooling and soaking, the cycle was repeated. The resulting expansions after each cycle are shown in Figure 16. The amount of expansion, if any, which occurred before the specimens were received is not known.

As a basis for judging the significance of the expansion shown (e.g. 0.18 percent, 5 cycles), assume that the specimens had been restrained or that a restoring force is applied to recompress the specimens to their original length. Let $E = 5 \times 10^6$ psi; the restoring stress would be:

$$\sigma = 5 \times 10^6 \times 1.8 \times 10^{-3} = 9{,}000 \text{ psi.}$$

Thus, the theoretical restoring stress would have exceeded the compressive strength of the concrete -- and rupture would have occurred.

FREEZE-THAW TESTING AND EXPANSION OF LABORATORY SPECIMENS OF CONCRETE

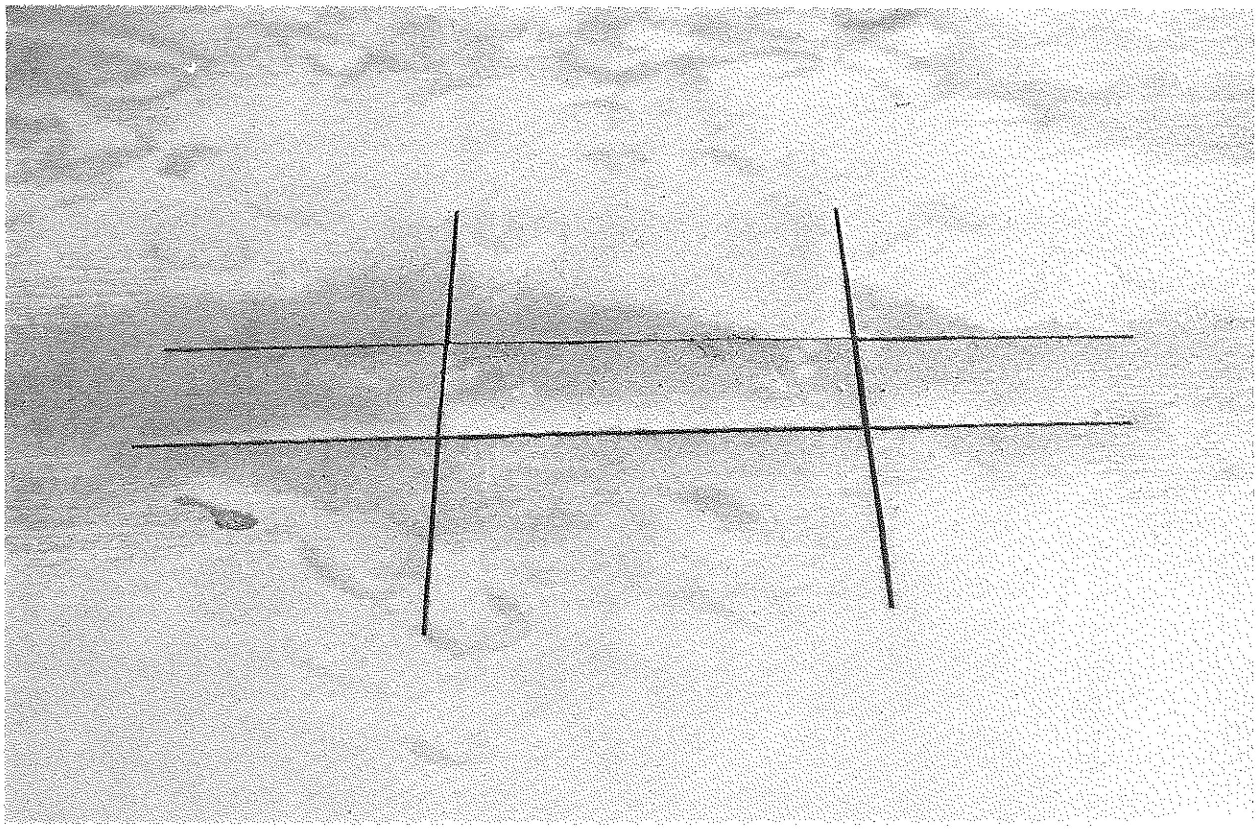
A composite sample of the Hoover aggregate was obtained in the latter part of 1967 and made into concrete specimens for freeze-thaw testing. Ledgerocks from the quarry were crushed in a laboratory mill and proportioned approximately according to occurrence. It is not known how well the resulting aggregate simulated the supply to the paving project. A reference aggregate (Tyrone and Oregon, Central Rock Company, Lexington) was employed in a companion series of concrete specimens.

The treatment of certain specimens was varied in an attempt to force expansion. Some were soaked, boiled, and subjected to freeze-and-thaw. One set of specimens was carried to 1232 cycles of freeze-and-thaw without suffering significant loss in sonic (natural) frequency. The accompanying expansions are shown in Figures 17 and 18. The expansion of concrete made with Central Rock aggregate far exceeded that of concrete made with aggregate from the Hoover Quarry.

The durability factors (ASTM C 29, 300 Cycles) were:

Hoover Series, 6.0% air = 98
Central Rock Series, 5.5% air = 80

The treatment of a second set of specimens was somewhat more varied and involved cyclic boiling. As seen in Figures 17 and 18, an increase in expansion was achieved in both concretes. For all practical purposes, the two concretes in this set behaved similarly.



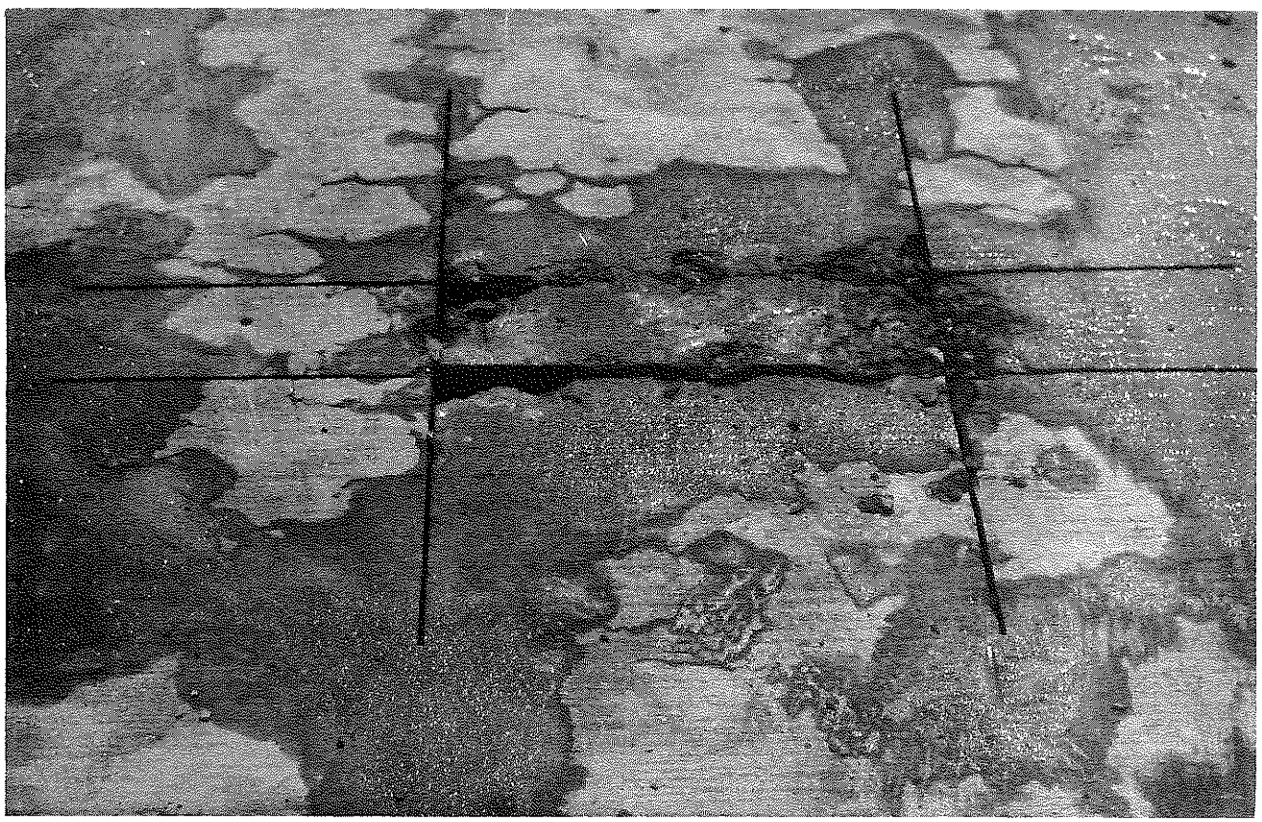


Figure 15. Gage Points Set in Pavement, Measured, then Block Sawed Free. Change in length between gage points -- due to relaxation -- was used to compute stress. Note cracking in lower photo (Station 226+00 SB).

TABLE 3. PAVEMENT STRESSES

PROJECT	STATION	DIRECTION	DATE OF INITIAL MEASUREMENT	DATE SAWED	TEMPERATURE (°F)	DATE OF FINAL MEASUREMENT	TEMPERATURE (°F)	ΔL (inch)	STRESS CHANGE (psi)	STRESS CHANGE @ 115°F
I 65-1(13)13	266+10	SB	8-15-68	NOT	79	9-11-68	82	0.0000	0	908
	266+10	SB	8-15-68	NOT	79	10-22-68	63	0.0011	550	1980
	225+75	SB	8-15-68	8-15-68	76	8-15-68	106	0.0020	1000	1247
	225+75	SB	8-15-68	8-15-68	76	9-11-68	82	0.0042	2100	3008
	225+75	SB	8-15-68	8-15-68	76	10-22-68	63	0.0054	2700	4130
	72+25	SB	8-19-68	NOT	107	9-11-68	81	0.0013	650	1585
	72+25	SB	8-19-68	NOT	107	10-22-68	64	0.0020	1000	2402
	71+25	SB	8-19-68	8-19-68	107	8-19-68	110	0.0022	1100	1237
	71+25	SB	8-19-68	8-19-68	107	9-11-68	81	0.0056	2800	3735
	71+25	SB	8-19-68	8-19-68	107	10-22-68	64	0.0062	3100	4502
	340+25	NB	8-19-68	9-10-68	112	9-10-68	85	0.0050	2500	3325
	340+25	NB	8-19-68	9-10-68	112	9-10-68	75	0.0058	2900	4000
	340+25	NB	8-19-68	9-10-68	112	9-11-68	85	0.0060	3000	3825
	340+25	NB	8-19-68	9-10-68	112	10-22-68	70	0.0065	3250	4487
	340+75	NB	8-19-68	NOT	112	9-10-68	85	0.0056	2800	3625
	340+75	NB	8-19-68	NOT	112	9-11-68	85	0.0056	2800	3625
	340+75	NB	8-19-68	NOT	112	10-22-68	70	0.0067	3350	4587
I 65-1(17)6	629+25	SB	8-19-68	NOT	112	9-11-68	81	0.0019	950	1885
	629+25	SB	8-19-68	NOT	112	10-22-68	65	0.0031	1550	2925
	628+75	SB	8-19-68	8-19-68	112	8-19-68	121	-0.0001	-50	-215
	628+75	SB	8-19-68	8-19-68	112	9-11-68	81	0.0028	1400	2335
	628+75	SB	8-19-68	8-19-68	112	10-22-68	65	0.0033	1650	3025
	590+25	SB	9-10-68	NOT	75	9-11-68	82	-0.0003	-150	757
	590+25	SB	9-10-68	NOT	75	10-22-68	64	0.0003	150	1552
	589+75	SB	9-10-68	9-10-68	75	9-10-68	70	0.0003	150	1387
	589+75	SB	9-10-68	9-10-68	75	9-11-68	82	0.0019	950	1857
	589+75	SB	9-10-68	9-10-68	75	10-22-68	64	0.0045	2250	3652
I 65-1(14)22	497+75	NB	9-11-68	9-11-68	61	9-11-68	72	-0.0009	-450	732
	497+75	NB	9-11-68	9-11-68	61	9-11-68	88	-0.0008	-400	342
	497+75	NB	9-11-68	9-11-68	61	10-22-68	75	-0.0010	-500	600
	529+75	NB	9-11-68	9-11-68	68	9-11-68	85	-0.0005	-250	575
	529+75	NB	9-11-68	9-11-68	68	9-11-68	88	-0.0004	-200	542
	529+75	NB	9-11-68	9-11-68	68	10-22-68	77	0.0001	50	995
	530+25	NB	9-11-68	NOT	68	9-11-68	88	-0.0006	-300	442
	530+25	NB	9-11-68	NOT	68	10-22-68	72	-0.0006	-300	882

Thermal Stress = $E_c \alpha_c \Delta T$

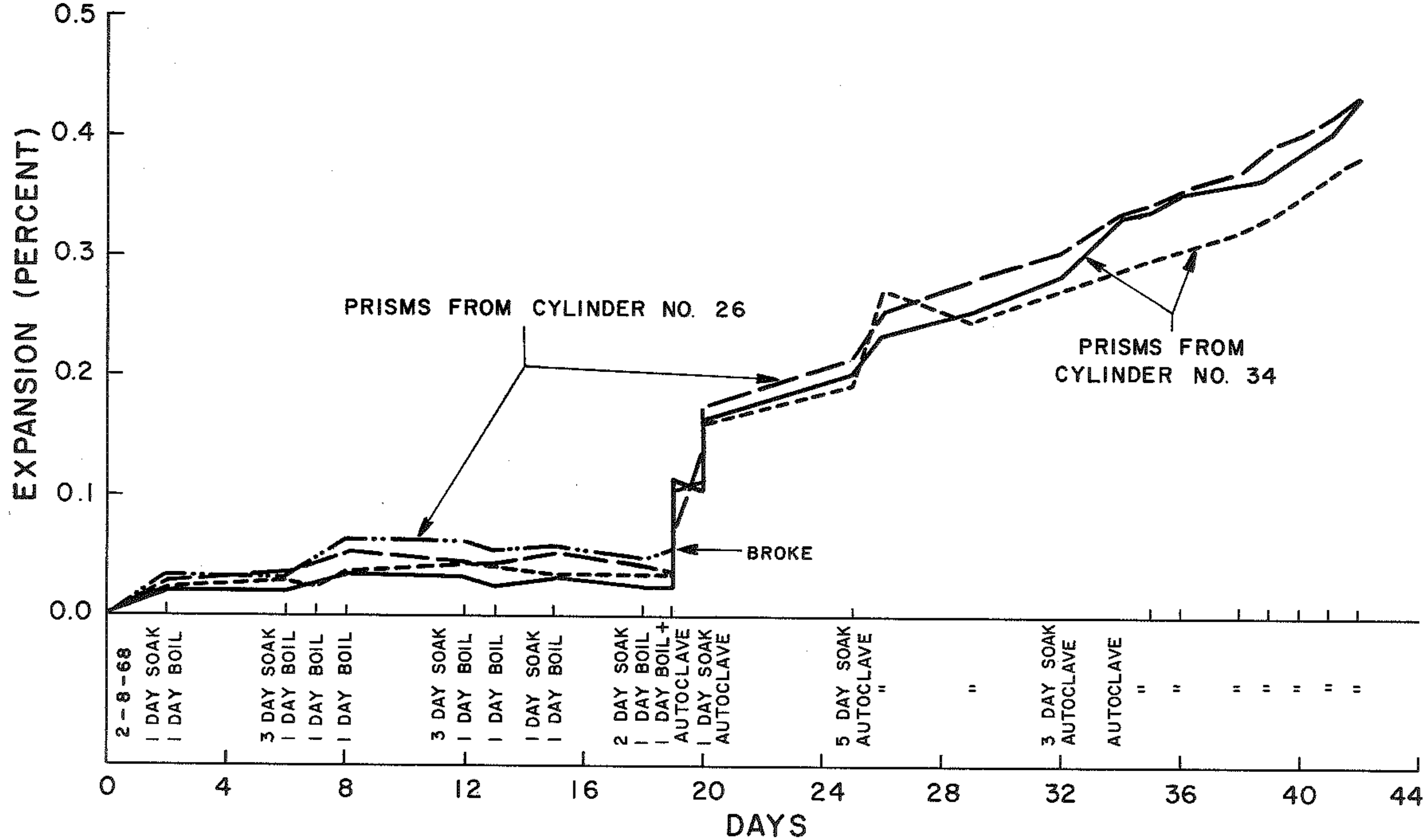


Figure 16. Expansion Induced in Prisms of Concrete Sawed from Cylinders No. 26 and No. 34 Saved from Construction Control Series.

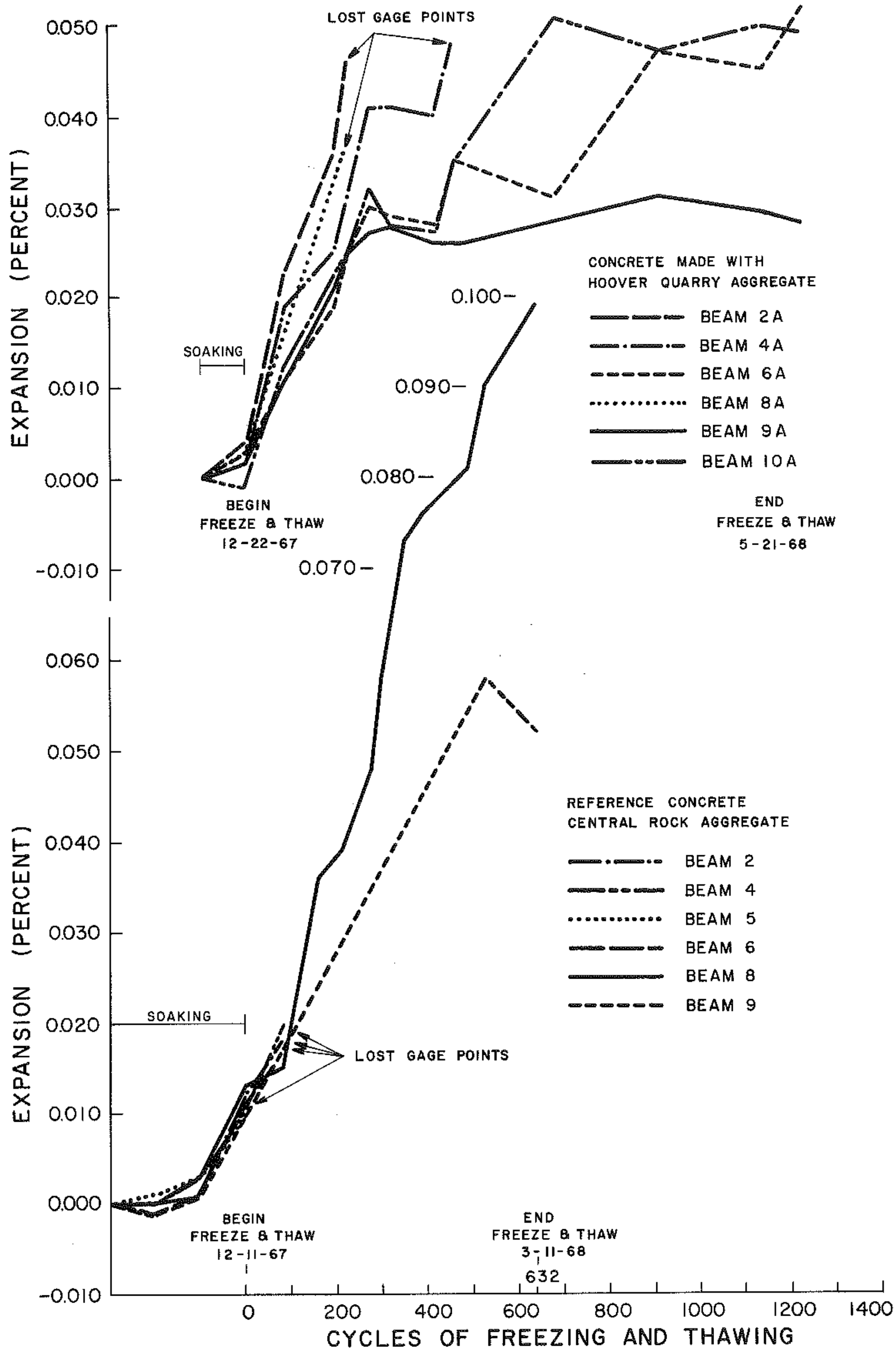


Figure 17. Expansion of Concrete Made with Coarse Aggregate from Hoover Quarry Compared to Reference Concrete; Freezing and Thawing.

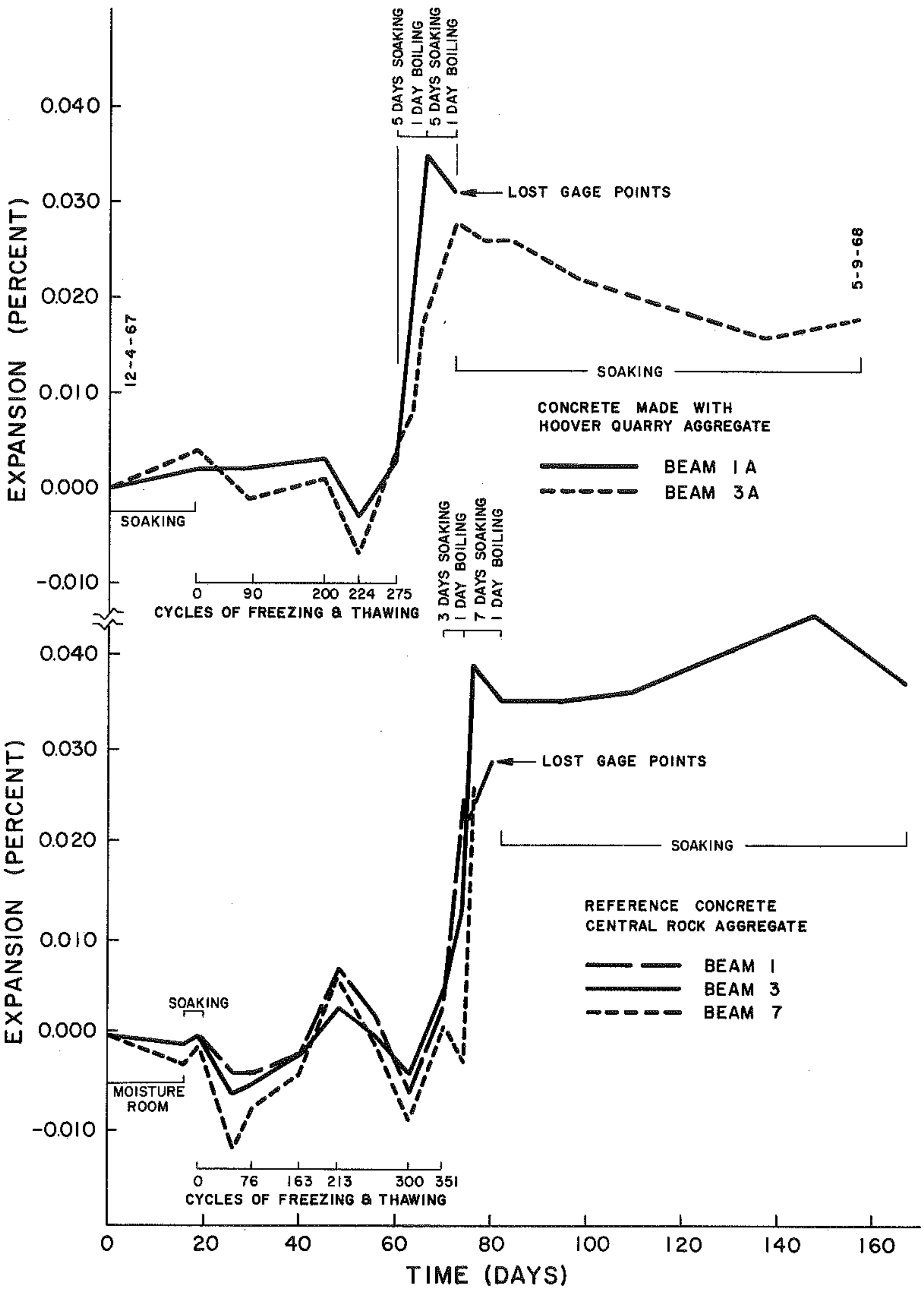


Figure 18. Companion Specimens of Concretes (Made with Hoover Quarry Aggregate and Central Rock Aggregate) Subjected to Boiling and Soaking after Having Undergone Regular Series of Freeze-Thaw Cycles.

EXPANSION OF AGGREGATE

Thirteen ledges in the source quarry were sampled during the winter of 1968-69. Specimens 3.5 x 0.7 x 0.7 inches, tapered at each end, were cut from sound chunks of ledgerock; these prisms were measured with precise calipers and immersed in a 1 N NaOH solution. This test procedure is described in ASTM C586-69 (Test for Potential Alkali Reactivity of Carbonate Rocks for Concrete Aggregates (Rock Cylinder Method)) (See APPENDIX A). The test began April 4, 1969, and length measurements were made periodically until August 3, 1970 -- a total of 479 days. Figure 19 shows, successfully, the length changes which resulted.

These tests confirmed the findings reported earlier by the Portland Cement Association (December 19, 1967) and fulfills the recommendations offered. A copy of the PCA report is included herewith (APPENDIX B).

Ledges No. 4 and No. 5 are expansive -- to a surprising extent. Expansions in the order of 0.7 percent are considered sufficient to cause disruption of concrete. The free expansion measured exceeds the critical value almost threefold. Whereas this discovery suffices to explain the occurrences of blowups and the disquieting stresses, it does not necessarily explain the surface cracking. More analytical attention is given elsewhere to the cracking phenomenon.

Explanations of the nature of the rock and the expansive processes are provided elsewhere in this report. The reaction has been called "de-dolomitization" *(2)*. Certain dolomitic limestones containing occluded clays lose $MgCO_3$ through dissolution in the presence of alkalies (NaOH and KOH); in concrete, the alkalies are contributed by the cement (0.45 percent by weight of cement, in this instance). The chemical balance expressions remain uncertain. The clays, of course, are the expansive bodies. As a rule, shalely, dolomitic limestones are suspect. These same limestones, but not exclusively these, tend to weather or decompose upon exposure -- even in the absence of alkalies. Indeed, observations at outcrops offer clues of the nature of quarry ledges. Chemical analyses (especially $MgCO_3$ and Insoluble Residue) as reported customarily by the Division of Materials provide a sufficient basis for selective testing in accordance with ASTM C586.

Figure 20 shows a typical specimen prepared for the expansion test. Figure 21 shows the measuring apparatus. Figure 22 is a side view of Specimen 5B after having undergone considerable expansion. Figure 23 shows an end view of the same specimen.

ROOF EXPOSURE

Companion specimens (whole chunks) of ledgerock from the Hoover quarry were placed on the roof deck of the laboratory while the cut prisms were immersed in an alkali solution. These specimens were about 8 inches in size. The deterioration is shown in Figures 24, 25, and 26.

SOURCE AND GEOLOGY OF LIMESTONE

Access to the quarry is from US 31-W, on KY 1199 (junction just south of Warren-Simpson County line), eastward, 4.5 miles. The quarry was operated by Hoover, Inc. The Division of Materials' Quarry Log is included here as APPENDIX C. The quarry site is just west of I 65 and between the Warren-Simpson County line and West Fork of Drakes Creek (Figure 27). It is not in the highest ground where according to GQ-277 (see Figure 27) the Ste. Genevieve formations could occur. The Ste. Genevieve, however, may comprise the higher elevations in the quarry. A portion of the quarry face is shown in Figure 28. It is the writer's belief that only the uppermost ledges of the quarry could possibly be Ste. Genevieve; however, Branson *(3)* apparently did not acknowledge the presence of Ste. Genevieve, but lumped the entire quarry into the St. Louis. The upper ledges (1 through 5) appear lighter in color -- which is said to be characteristic of the upper St. Louis.

Ledges No. 4 and No. 5 have now been shown to be highly expansive and to slake and deteriorate upon exposure to weather. Ledges 4 and 5 are non-fossiliferous, are extremely fine-grained (apparently precipitated), and contain about 15 percent $MgCO_3$ and about 9 percent acid-insolubles. Ledge No. 1 is similarly composed; but, for reasons not yet apparent, specimens did not exhibit severe expansion.

The quarrying pattern or sequence at the Hoover site remains vague. One might surmise that the percentage of aggregate contributed by Ledges 4 and 5 was higher during early operations and stockpiling if the quarry was deepened sequentially.

In Indiana, Hadley *(4)* found expansive ledges in the Ste. Genevieve (Levias) (equated to L. Ohara) and below in the top and bottom of the St. Louis formation. The Ste. Genevieve is a major source of aggregate in Kentucky. It has been or is being quarried at Smithland, Fredonia, Princeton, Hopkinsville, Franklin, Bowling Green, Brandenburg, Mt. Vernon, Morehead, and Olive Hill. The St. Louis has not been quarried extensively in Kentucky *(5, 6, 7, 8, 9, 10)*.

Branson *(3)* mentions a quarry in the St. Louis at Prices Mill, in the southwestern corner of Simpson County, which was used to construct abutments for a bridge (built in 1958, KY 591 over Red River). No other local history of St. Louis limestone usage has been uncovered. A cursory review of quarries in Kentucky which have been identified geologically indicates that

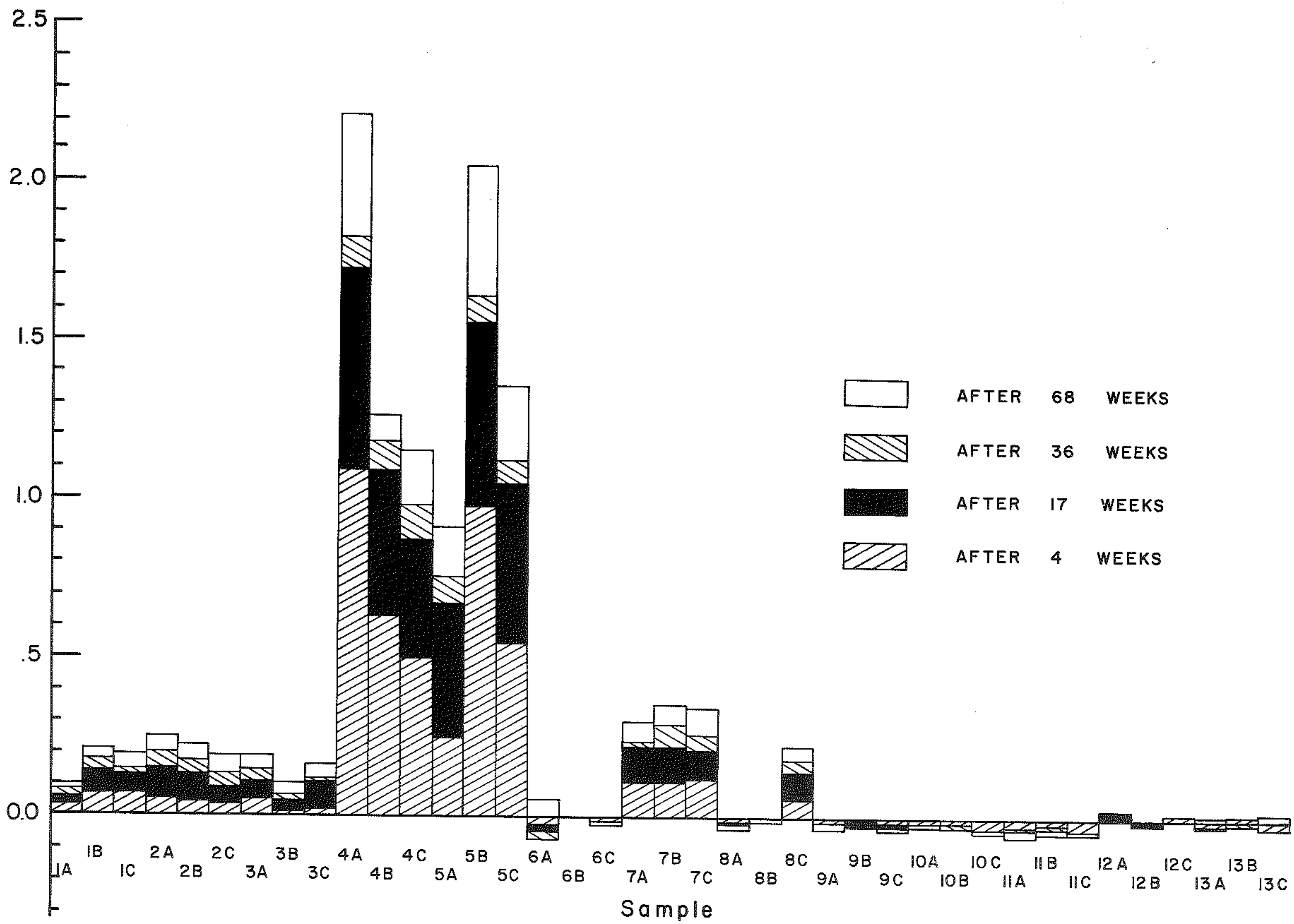


Figure 19. Expansion of Rock Specimens from Hoover Quarry.

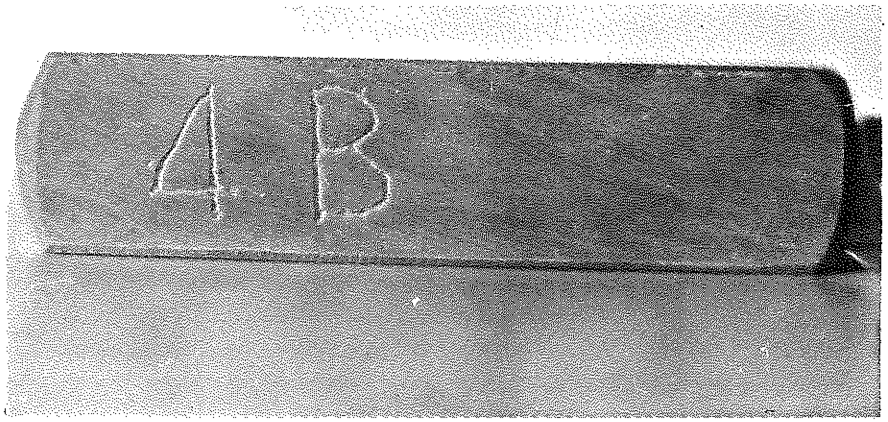


Figure 20. Rock Specimen, 0.7 x 0.7 x 3.5 Inches, Tapered at Each End.

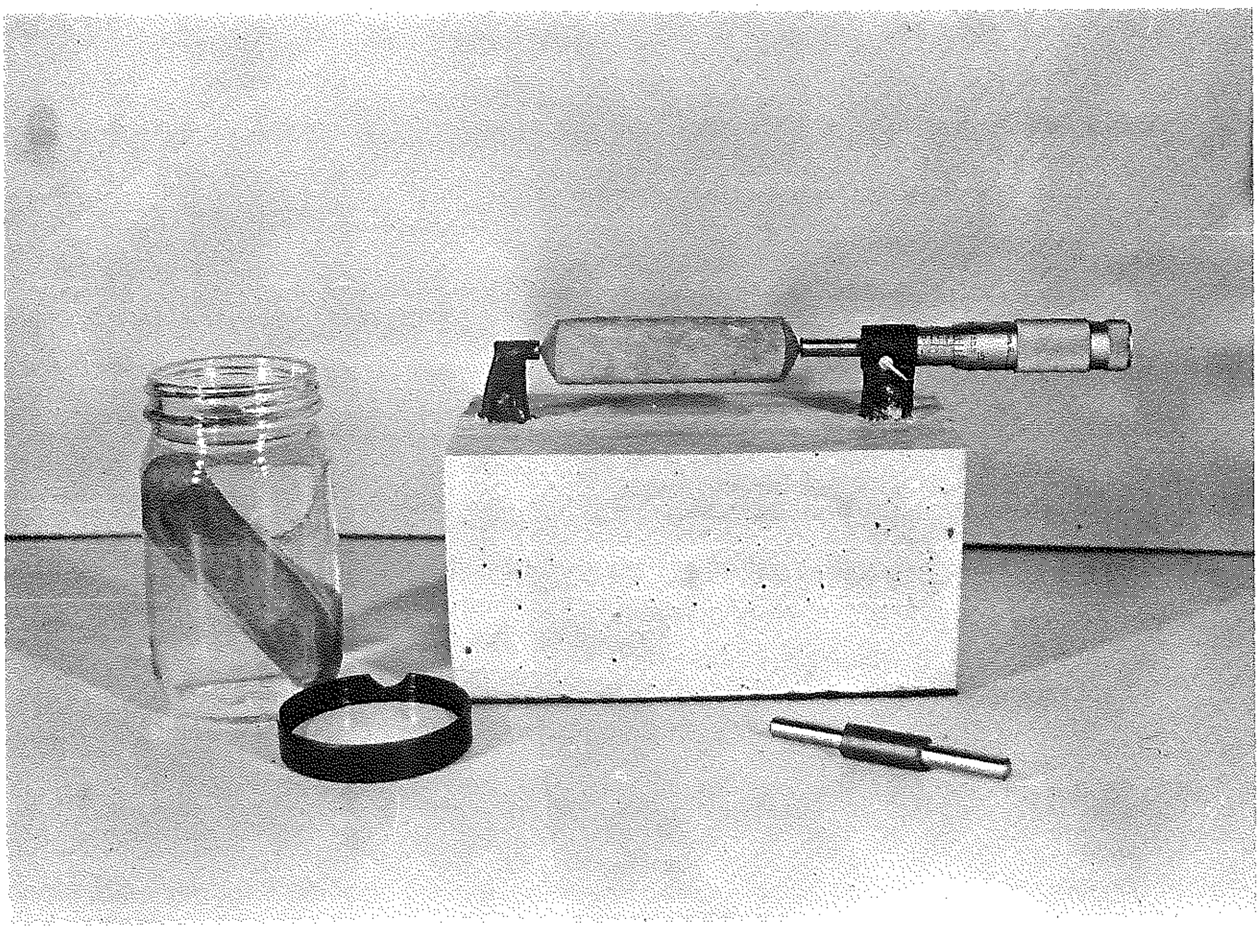


Figure 21. Micrometer Caliper for Measuring Length Change. Photo also shows reference bar and rock specimen in sodium hydroxide solution (in jar).

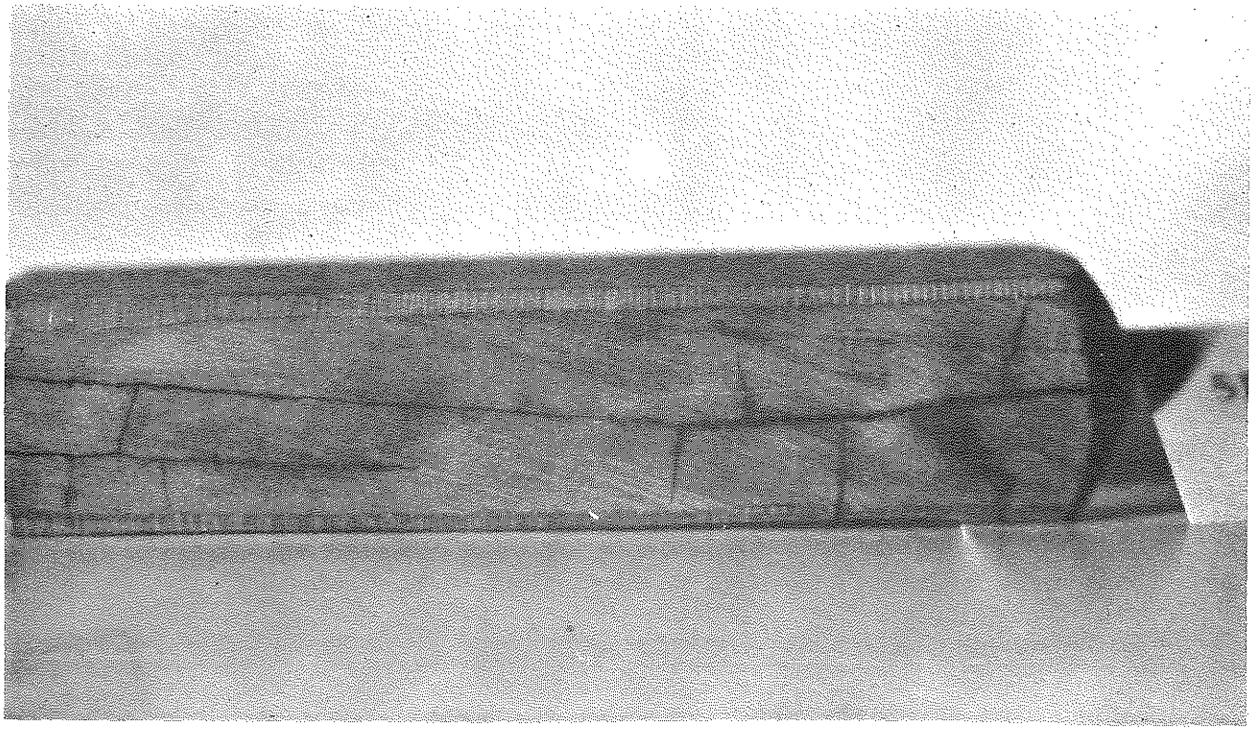


Figure 22. Specimen 5B after 478 Days in Sodium Hydroxide Solution.

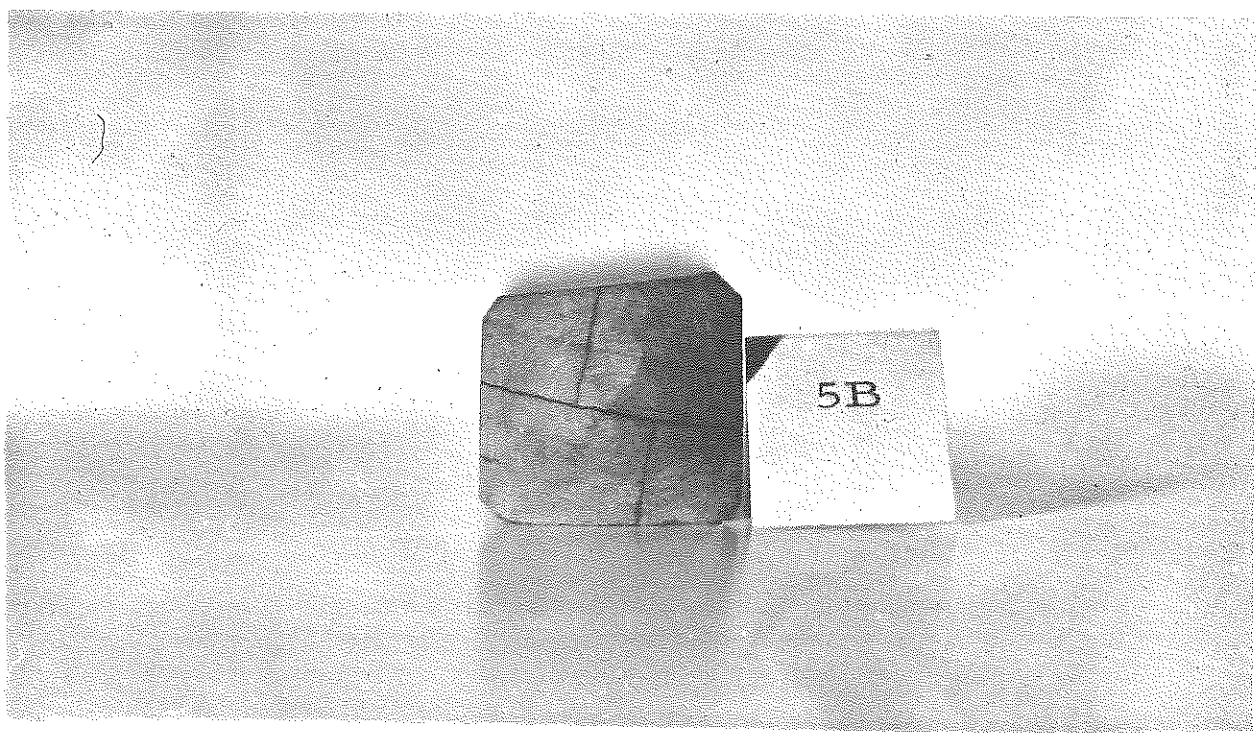


Figure 23. End View of Specimen 5B.

Figure 24. Specimens of Aggregate Exposed to Weather on Roof Deck of Laboratory since October 10, 1968; January 10, 1971.

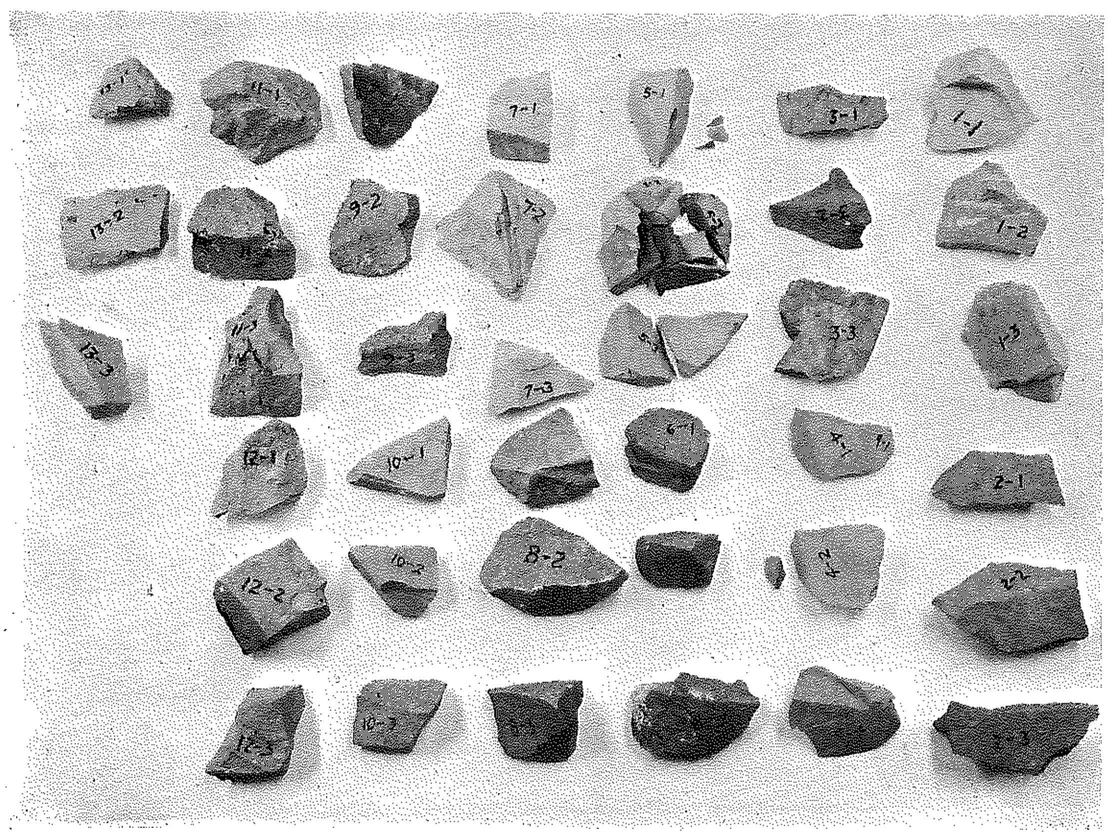


Figure 25. Same as Figure 24; November 17, 1971.

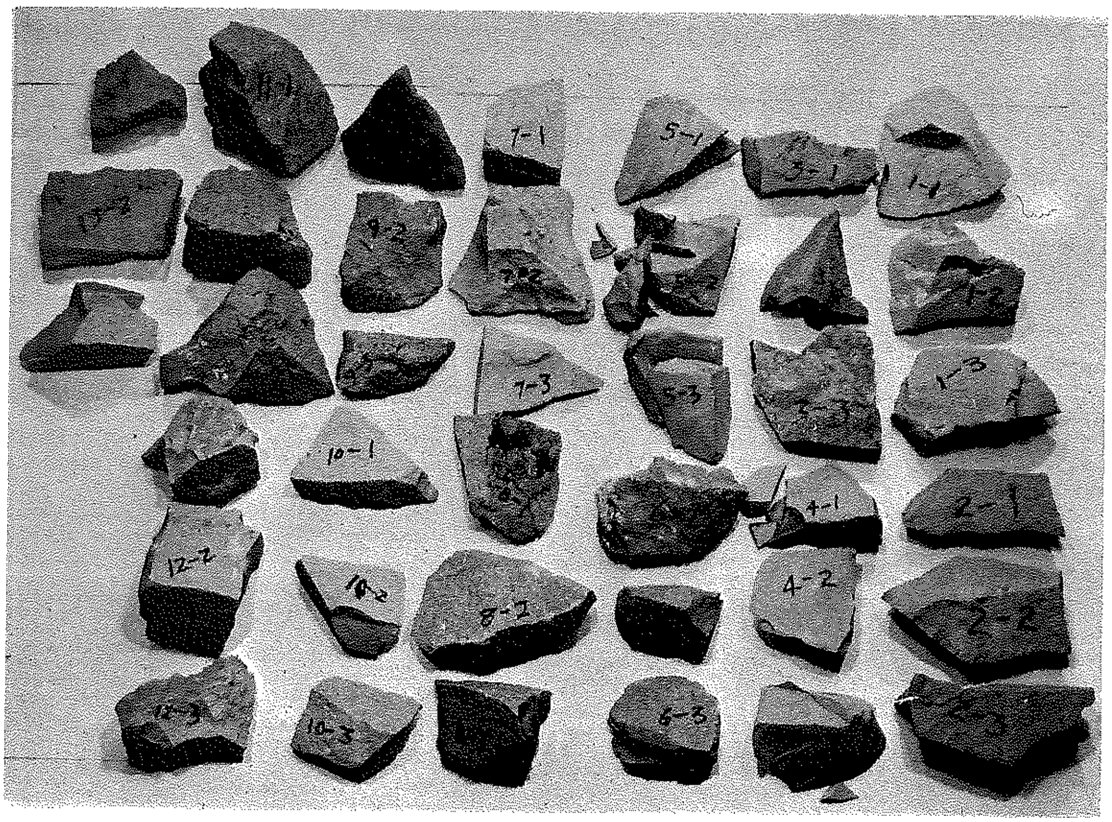
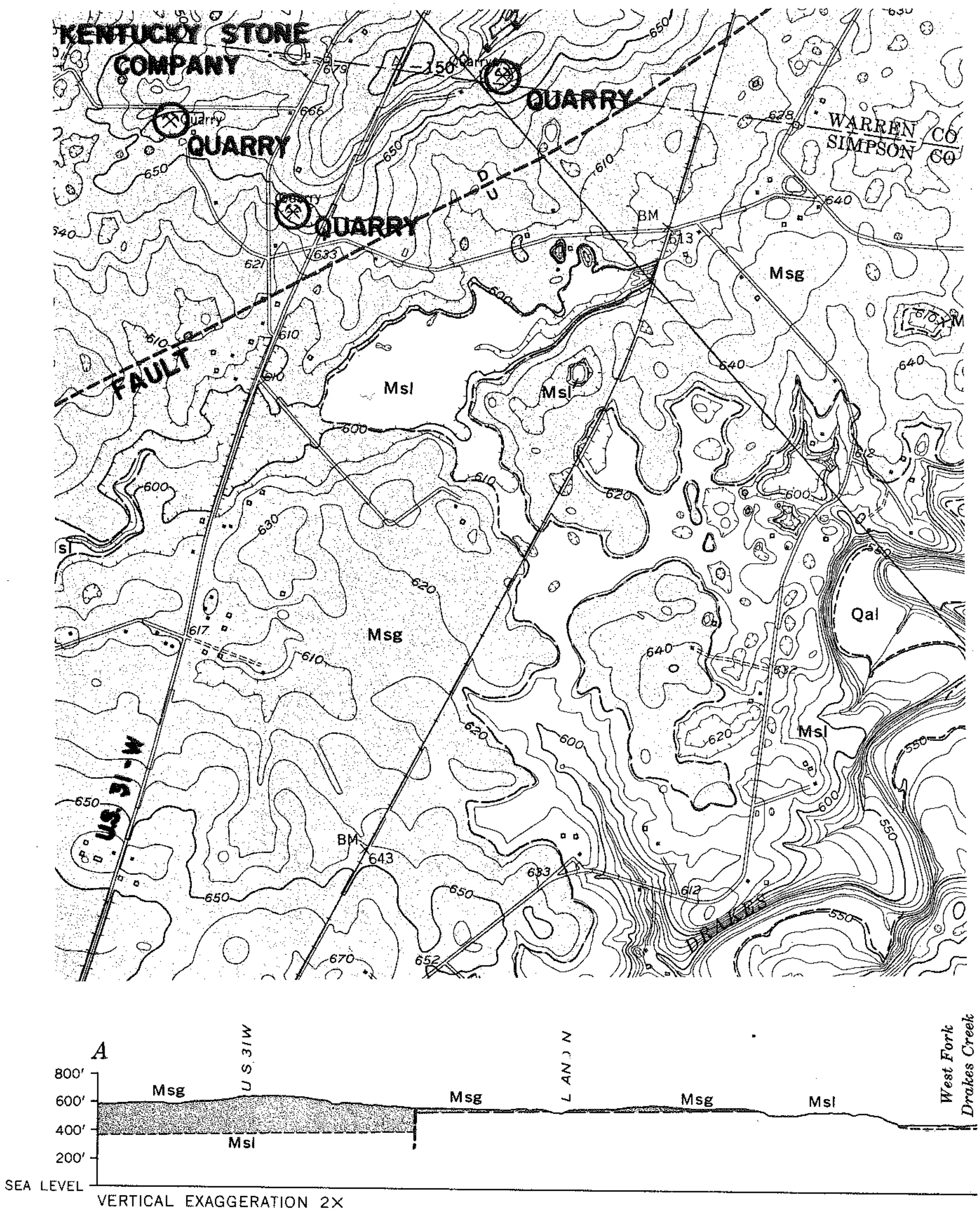


Figure 26. Same as Figure 24; March 21, 1972.



WOODBURN QUADRANGLE

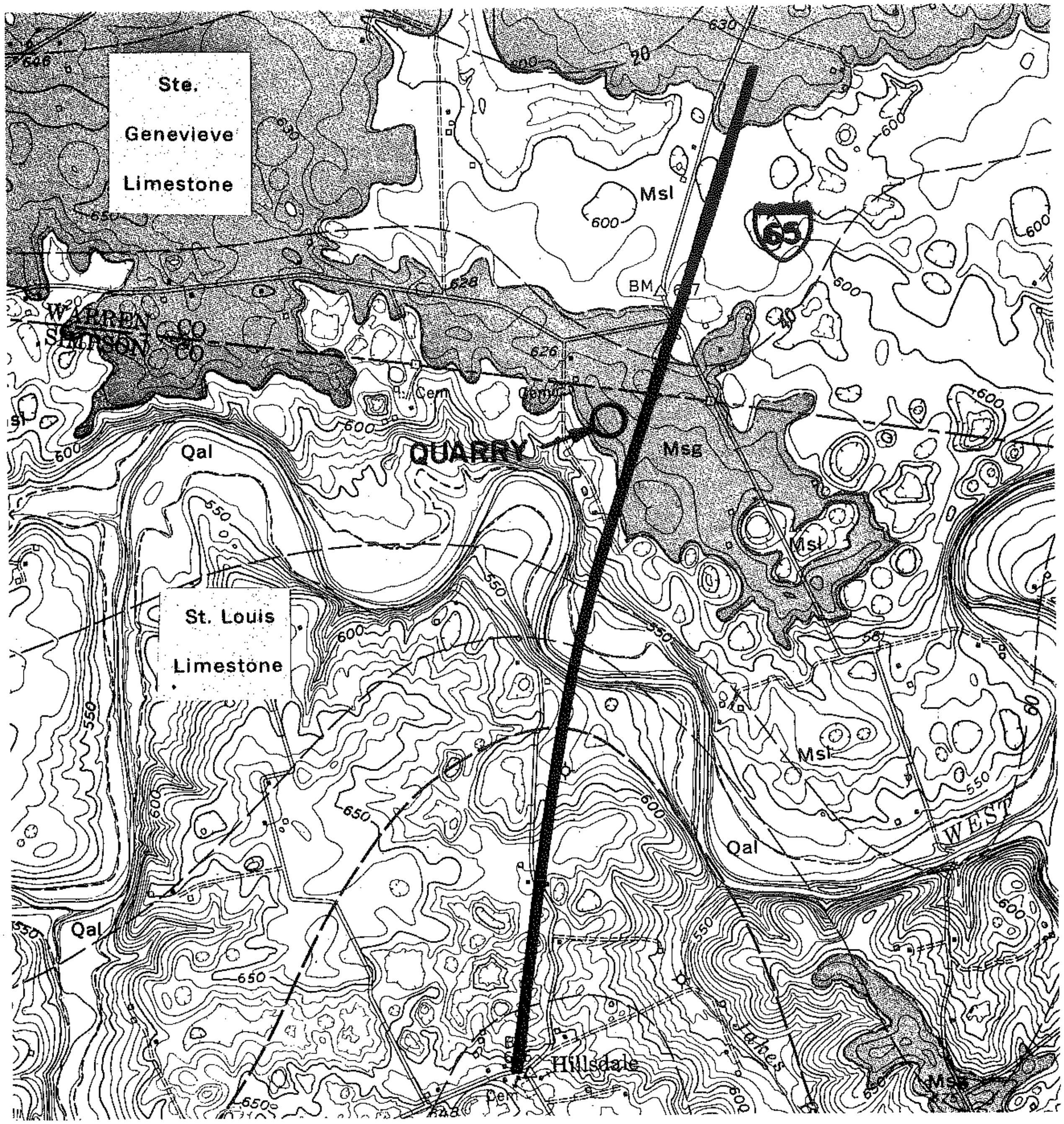


Figure 27. Portion of GQ-280 and GQ-277. Hoover Quarry is located west of I 65 and north of West Drakes Creek, in Simpson County. Note fault line crossing US 31 W and passing between quarries just northward there and the I 65 quarry. Likelihood of contiguity between these quarries seems remote (see cross section).

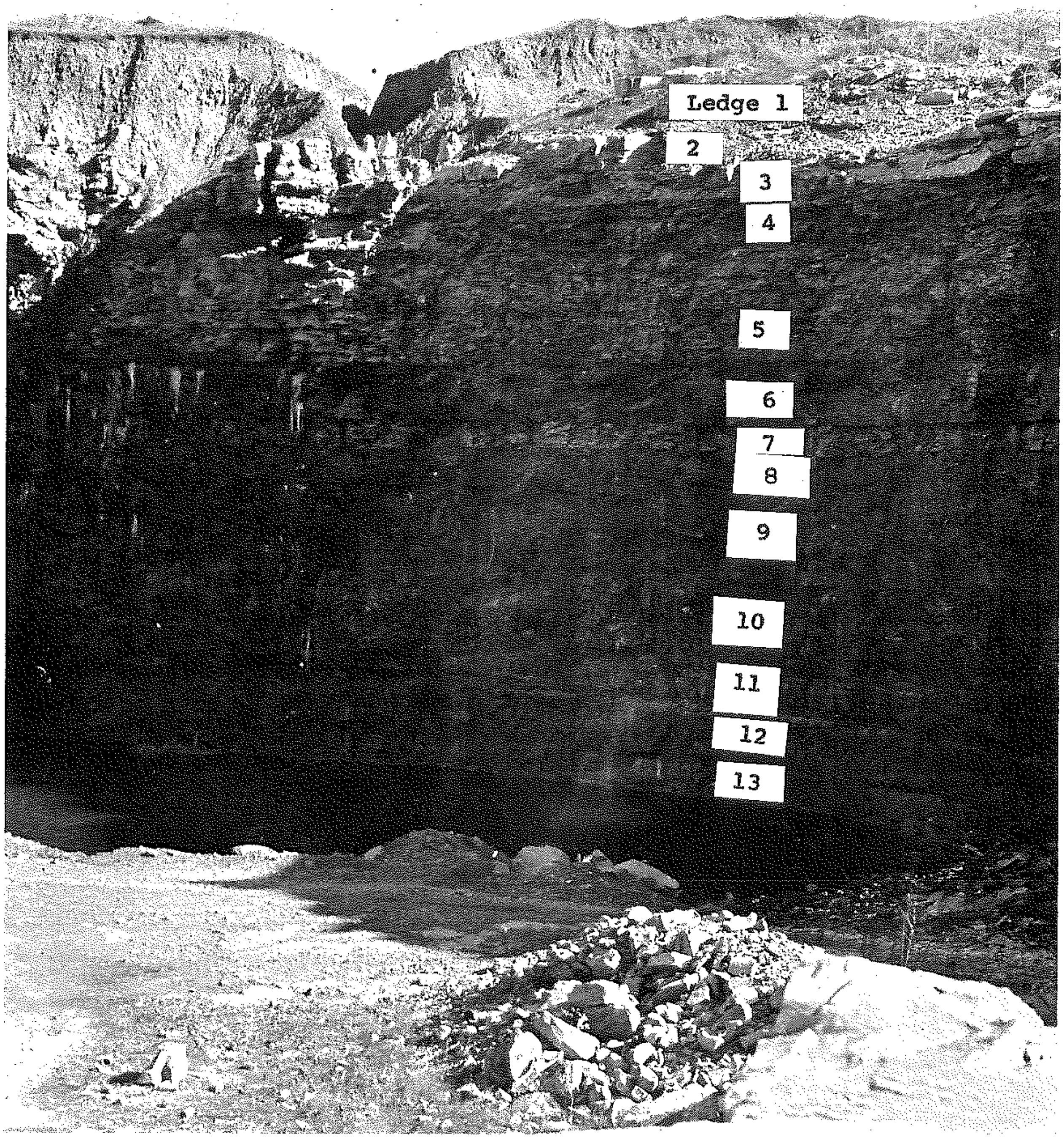


Figure 28. Hoover Quarry, Aggregate Source for I 65-1(13)13, Simpson County; January 28, 1971.

the St. Louis is a relatively minor source of aggregate in comparison to the Ste. Genevieve. Quarries which penetrate the St. Louis occur in Casey, Pulaski, Powell and Menifee Counties. It appears that a quarry in Casey County is wholly within the St. Louis. Commentaries *(9)* allude to the superior quality of the Ste. Genevieve. Several quarries penetrate to the St. Louis, but it remains there as the floor.

Note 1. *The more complete section of Ste. Genevieve should be found in the quarries west of the fault line, and west of US 31-W.*

Note 2. *Farther west, the upper member of the St. Louis has been assigned to the Ste. Genevieve. This is shown on new geological quadrangle maps for Christian and part of Trigg Counties. Westward from there, it is mapped as upper St. Louis (9).*

Note 3. *The upper limits of the Ste. Genevieve and succeeding formations are also problemmatical from the standpoint of identification (11).*

MICROSCOPY OF THIN SECTIONS

No distinctly oolitic beds were found in the Hoover Quarry. Ledges 1, 4, and 5 are fine-grained and non-fossiliferous. Only Ledges 4 and 5 are very expansive. Ledge 6 is composed mostly of smaller foraminifers (0.4 mm in diameter). *(To a field observer, using only a hand lens, these may be falsely identified as oolites).* Ledge 11 is somewhat similar to Ledge 6 but somewhat more metamorphosed and fragmented. There are indications of Porifera and Hydrozoa. Ledges 2 and 3 contain many detrital fossil fragments dispersed in a fine-grained matrix.

It appears evident that co-deposition of clays with carbonates (possibly precipitated) attended the formation of Ledges 1, 4, and 5. There are no indications of recrystallization (growth) of the carbonates.

Photomicrographs of thin sections representative of the respective ledges mentioned above are shown in Figure 29.

Figure 30 shows a northward view of I 65 from south of Drakes Creek. The quarry is to the left of I 65, in the background, beyond the Drakes Creek bridges. There is cracking in the pavement in the foreground; ledgerock exposed in the cut is similar to rock in the quarry. A specimen of rock taken at the point indicated by an arrow in Figure 30 was prepared in a thin section and photographed (Figure 31). The specimen is similar to Ledges 1, 4, and 5. Judging by particle size, it appears to be equivalent to Ledge 1. A more definite identification was not made. The geologic map (Figure 27) shows only St. Louis formations on the south side of Drakes Creek -- that is in the line of I 65.

On March 7, 1972, remnant specimens of aggregate particles were extracted from "popout sockets" in the pavement in the vicinity of Milepost 22 (NB). The advanced stage of cracking at the site is shown in Figure 32. Photomicrograph shown in Figure 33 definitely associates the particles with Ledges 1, 4, or 5 in the quarry.

The Prices Mill site was visited March 7, 1972. There were no indications of unsound aggregate in the bridge. The quarry is beside the road nearby. A photograph of the quarry is shown in Figure 34. Photomicrographs of thin sections are shown in Figure 35.

OTHER CASE HISTORIES

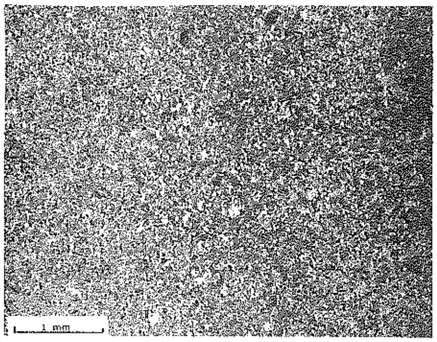
Lest I 65 be presumed a new revelation, an archival account of other indisposed pavements follows.

US 60, OLIVE HILL-GRAYSON, 1926

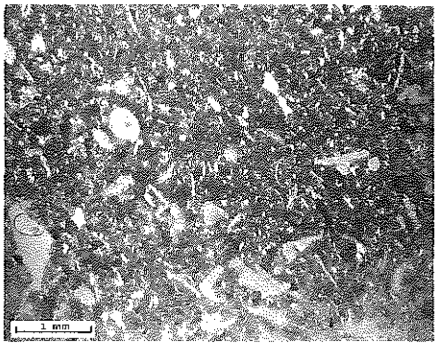
US 60 (5.04 miles, FA-13B), vicinity of Tygarts' Creek, eastward toward Grayson was paved in 1926. About 14 years later, surface cracking and popouts appeared. Figure 36 shows a portion of the pavement as it appeared in 1948. Popouts were caused by a chalky limestone which was distinctly associated with two ledges in the source quarry. The quarry was located just east of Olive Hill (visible rightwardly from present US 60). It was operated by the Codell Construction Company and by Central Rock Company. After World War II, this and three other quarries were studied intensely *(12, 13, 14, 15)*. Rock quality requirements were not well established when this pavement was constructed; however, the quarry was abandoned sometime around 1940 because of difficulties in meeting specifications. In the course of the study (1948), it was observed that rock fragments from the offending ledges had chalked and deteriorated while laying on the floor of the quarry (Figure 37).

The offending ledges were identified then *(7, 8)* as Gasper -- overlaying a thin, shale remnant of Ohara. Identification remains questionable. Here, as well as in the Hoover Quarry in Simpson County, the offending ledges do not contain fossils. Indeed, this is a mark of similarity -- but not the only one. They are microscopically similar; both are very fine grained and dolomitic.

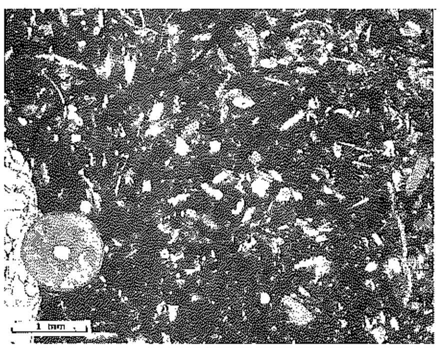
Platycrinus penicillus defines the upper limit of the Ste. Genevieve; *Talarocrinus* defines the Chester series (paloi, Renault). Apparently these key fossils occur out of order in some places; and so the upper portion of the Ste. Genevieve is sometimes confused. At Bowling Green, the "Gasper" has been equated to Girken; but at the I 65 quarry, only the St. Louis and (or) the lower Ste. Genevieve is exposed. If the expansive ledges in the



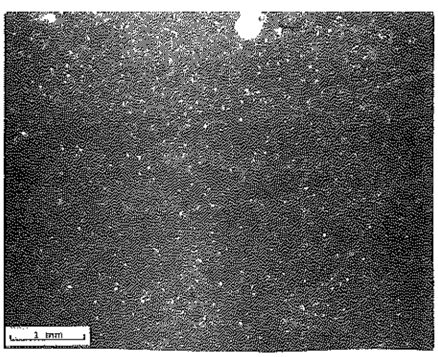
Ledge 1



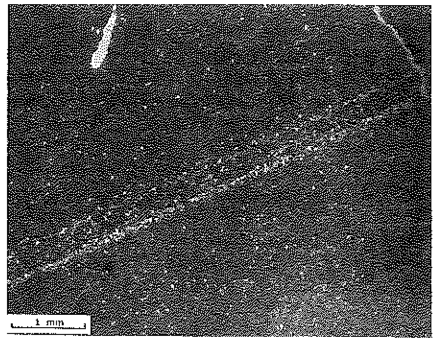
Ledge 2



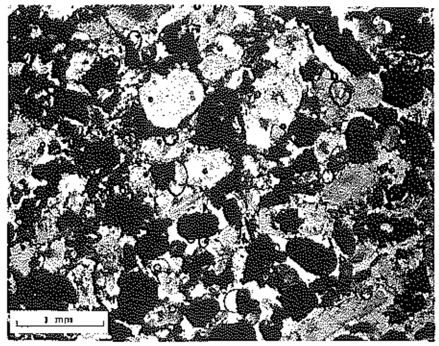
Ledge 3



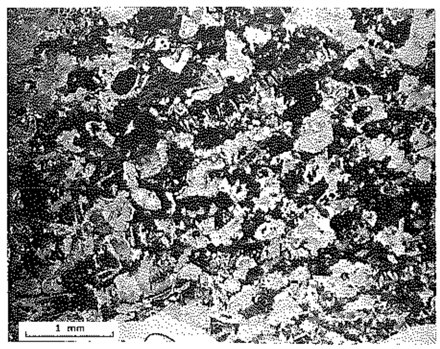
Ledge 4



Ledge 5



Ledge 6



Ledge 11

Figure 29. Photomicrographs of Thin Sections, Hoover Quarry; I 65 from US 231 Southward to 0.74 Mile South of Warren-Simpson County Line.

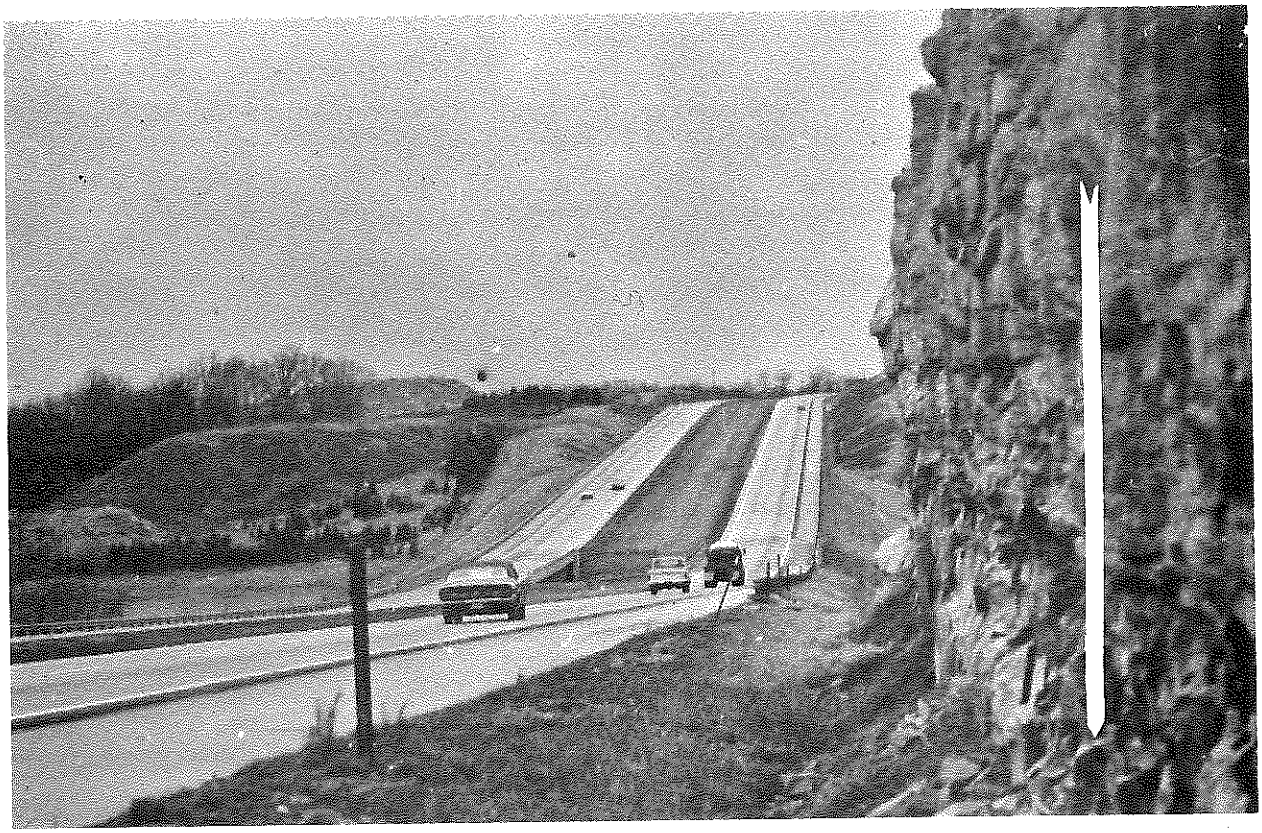


Figure 30. Northward View of Hoover Quarry, from I 65 South of Drakes Creek; Arrow Indicates Ledge Where Specimen Was Taken (Photomicrograph shown in Figure 31); March 9, 1972.

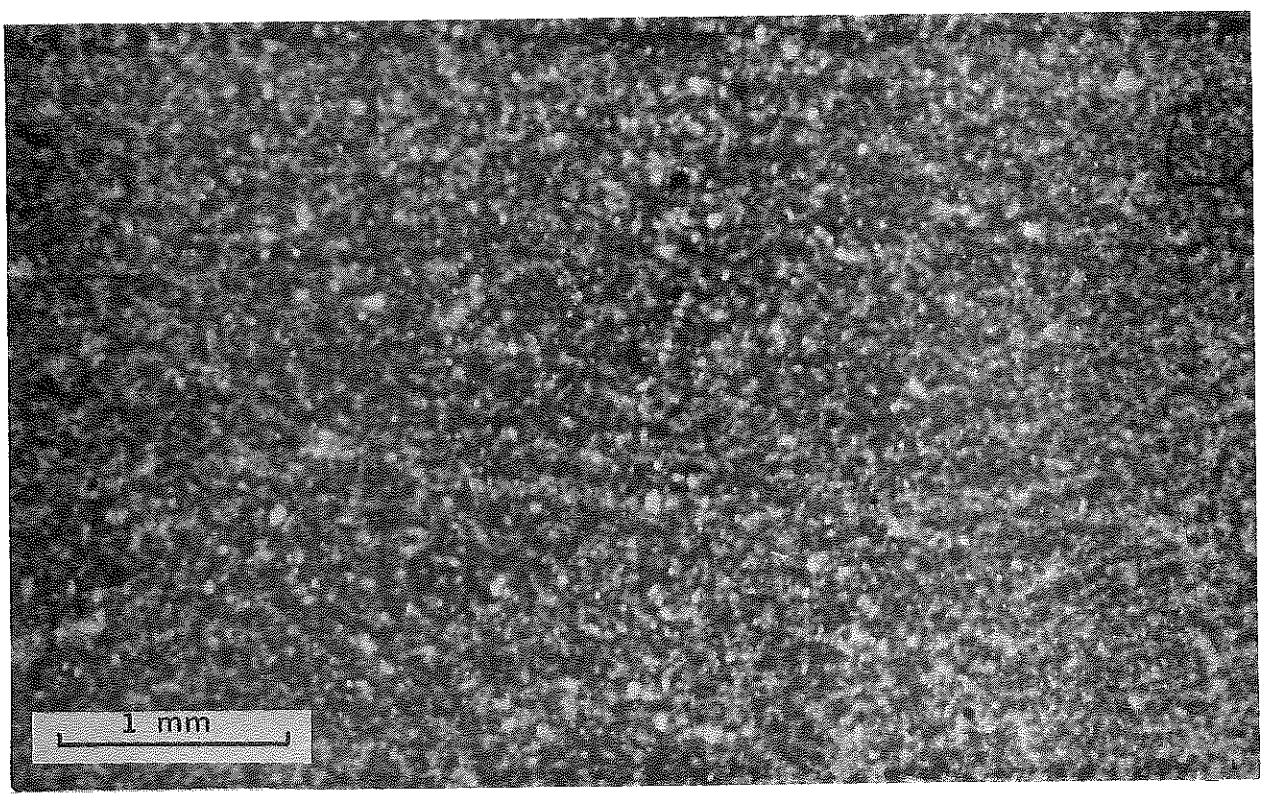


Figure 31. Photomicrograph of Thin Section; Specimen taken from Roadside (See Figure 30).

Figure 32. Advanced Cracking, I 65, MP 22, NB; March 7, 1972.

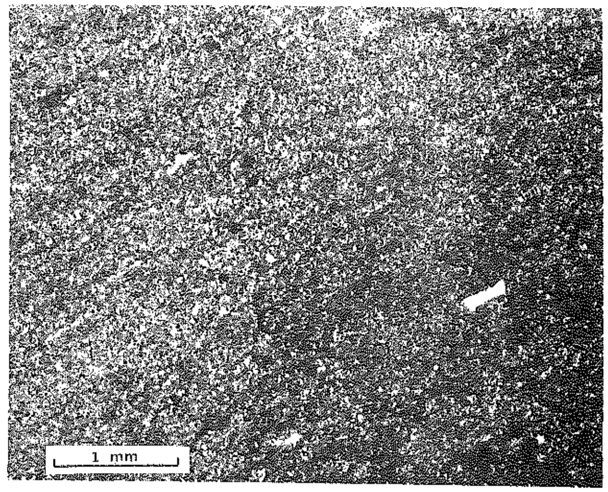


Figure 33. Photomicrograph of Thin Section; Specimens Obtained from "Popout Sockets" in Pavement Section Shown in Figure 32; March 7, 1972.

Figure 34. Quarry at Prices Mill, Southwest Simpson County; March 7, 1972.

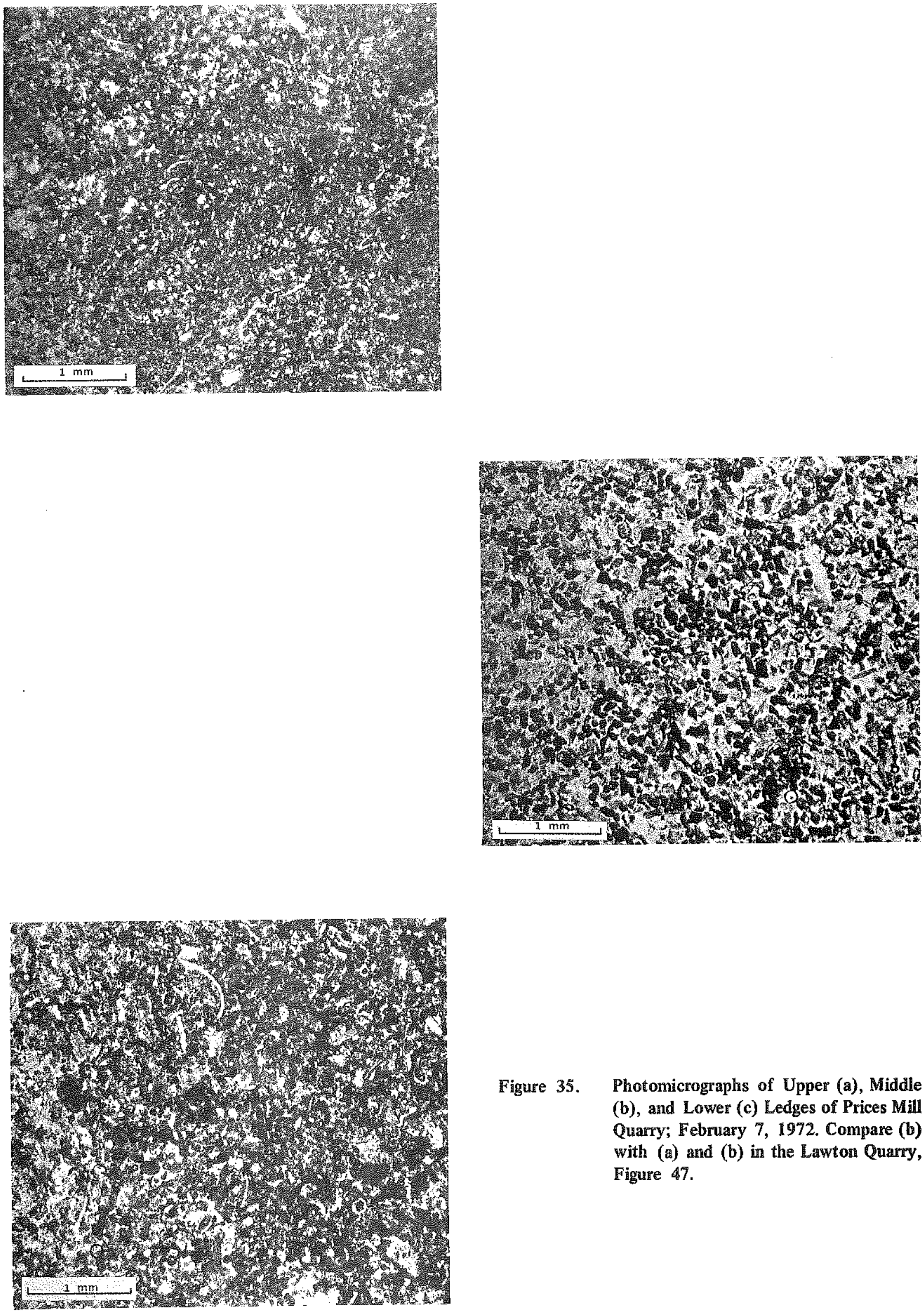


Figure 35. Photomicrographs of Upper (a), Middle (b), and Lower (c) Ledges of Prices Mill Quarry; February 7, 1972. Compare (b) with (a) and (b) in the Lawton Quarry, Figure 47.

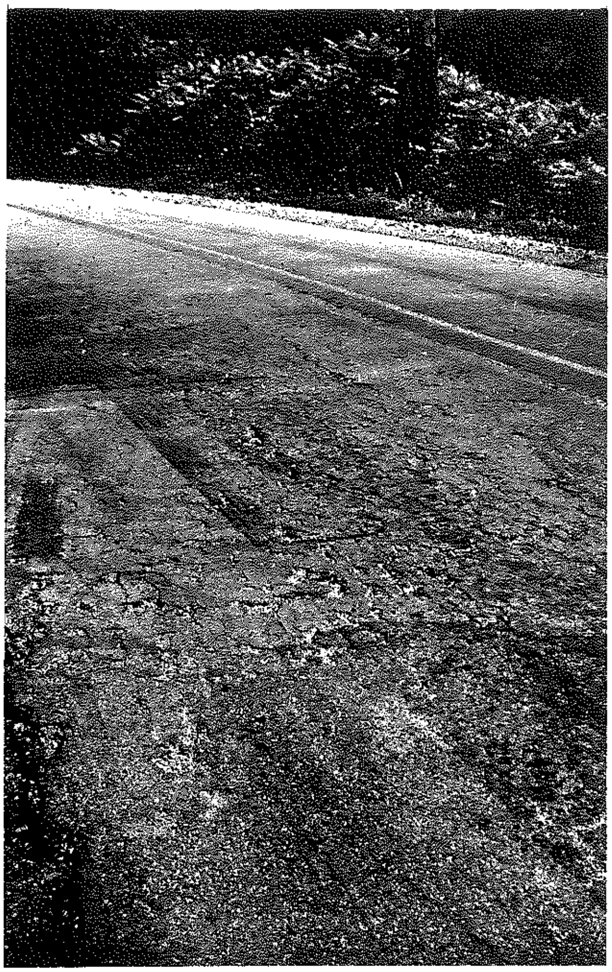


Figure 36. **Section of US 60 between Olive Hill and Grayson; 1948** ***(14).***

Figure 37. **Weathered Rock Found in Floor of Olive Hill Quarry in 1948** ***(14).***

Hoover Quarry were stratigraphically or depositionally identical to the expansive ledges in the quarry at Olive Hill, the Ste. Geneveive at Olive Hill would necessarily encompass higher ledges formerly identified as Ohara and Gasper. Also, if Ledges 1 through 5 in the Hoover quarry are indeed Ste. Genevieve instead of St. Louis, the expansive ledges would be at the base of the Ste. Genevieve -- which seems altogether inconsistent. The other possibility is that the Olive Hill Quarry may be largely St. Louis. On the basis of microfossils only, this seems entirely possible. No attempt was made in this study to reconcile macrofossils.

Coincidentally, the only expansive rock found in the Ste. Genevieve in Indiana by Hadley *(4)*, was in the Levias, which there and in western Kentucky refers to the upper strata of the Ste. Geneveive. The fresh rock mentioned by Hadley appeared quite sound. It was fine grained, dolomitic, and contained about 11 percent insoluble residue (mostly illite clay).

A 1948 photograph of the quarry is shown in Figure 38. To further illustrate the in situ weathering and decomposition of this rock, a 1972 photograph of the quarry face is shown in Figure 39.

Thin sections made of this quarry in 1947 were reexamined, and representative photomicrographs are shown in Figure 40. The uppermost ledge (65 feet) contains distinct "smaller forams" and either "larger forams" fragments or Porifera fragments.

Surely, the most distinctive bed in this quarry is the well-developed oolite which occurs briefly at the 63-foot level, near the top of the exposure. The succeeding two feet revert to more fragmented fossil remnants, some sand grains, and an abundance of smaller foraminifers. The 63-foot level is somewhat distinctive; it is not oolithic, but contains many smaller and microforaminifers. A similar ledge was recognized in 1947 in the 64-78-foot zone of a quarry in Mt. Vernon *(13)*. Below the 62-foot level is about 4 feet of a somewhat granular, brownish dolomite. From there through the 51-foot level consists of rather dense, fragmented, microfossil remnants. The section is somewhat similar to Ledges 2 and 3 in the Hoover Quarry. Then, there is about 1 foot of fine-grained, nonfossil-bearing rock underlain by 3 feet of sparsely fossiliferous, fine-grained material. The crystallization is indistinguishable at low magnification. From 43 feet to 34 feet (downward), the rock is fine grained and void of "forams." This rock weathers white and is the unsound section of the quarry. From 33 to 29 feet, the rock seems more resistant; but the 28- and 27-foot levels become fine grained again; and, from the 28-foot level to the 20-foot level, the rock weathers like the 43- through 34-foot zone. Below 24 feet there are 2 feet of seemingly cherty, brownish microfossils dispersed in a clayey matrix. From this point downward, "forams" re-appear; at some levels, they appear almost as distinctive as in Ledge 6 of the Hoover Quarry. The 5-foot level then becomes distinctly like the 55-foot level. Others have described these lower sections as "sandy" limestone. The whole section seems to fit recent descriptions of a near-by exposure by Weir *(16)*.

KY 32, MOREHEAD TO FLEMING COUNTY LINE

Figure 41 shows a blowup on KY 32 in 1950. Many structures on this road (Morehead - Flemingsburg) deteriorated because of so-called chalky aggregates which doubtlessly are similar to the I 65 aggregate *(17)*. A series of photos are reproduced here. The road was built in 1933. The source has been identified as the Tygart Limestone Company, of Lawton, in Carter County. A quarry was in operation at Lawton in 1949 and in 1943. *(Highway Materials Research Laboratory, Lab Project No. 8; Investigation of Aggregates from State Quarries, May 21, 1946, Research Division File A-1-2)*. However, the exact identification of the source quarry site has not been confirmed. A recent photograph of the Lawton quarry (mine) is included here as Figure 46. Photomicrographs of thin sections of specimens of rock from ledges indicated by letters are shown in Figure 47. The ceiling ledge and part of the supporting ledge appear to be similar to the 62-foot level in the Olive Hill quarry; the d-level is similar to the 49-foot level at Olive Hill. It seems, on the basis of these observations, that the unsound ledge in the Olive Hill quarry, if present, lies below the floor level in this quarry.

US 60 BRIDGE OVER TYGARTS CREEK (East of Olive Hill)

Although no specific investigation has been made in regard to the aggregate used in the Tygarts Creek Bridge, deterioration of concrete handrails has been apparent for many years. The source of aggregate has not been identified; but it is likely that it was supplied from the Olive Hill quarry. A recent photograph (Figure 48) is included here as a matter of record.

Figure 38. Olive Hill Quarry in 1948 *(14)*.

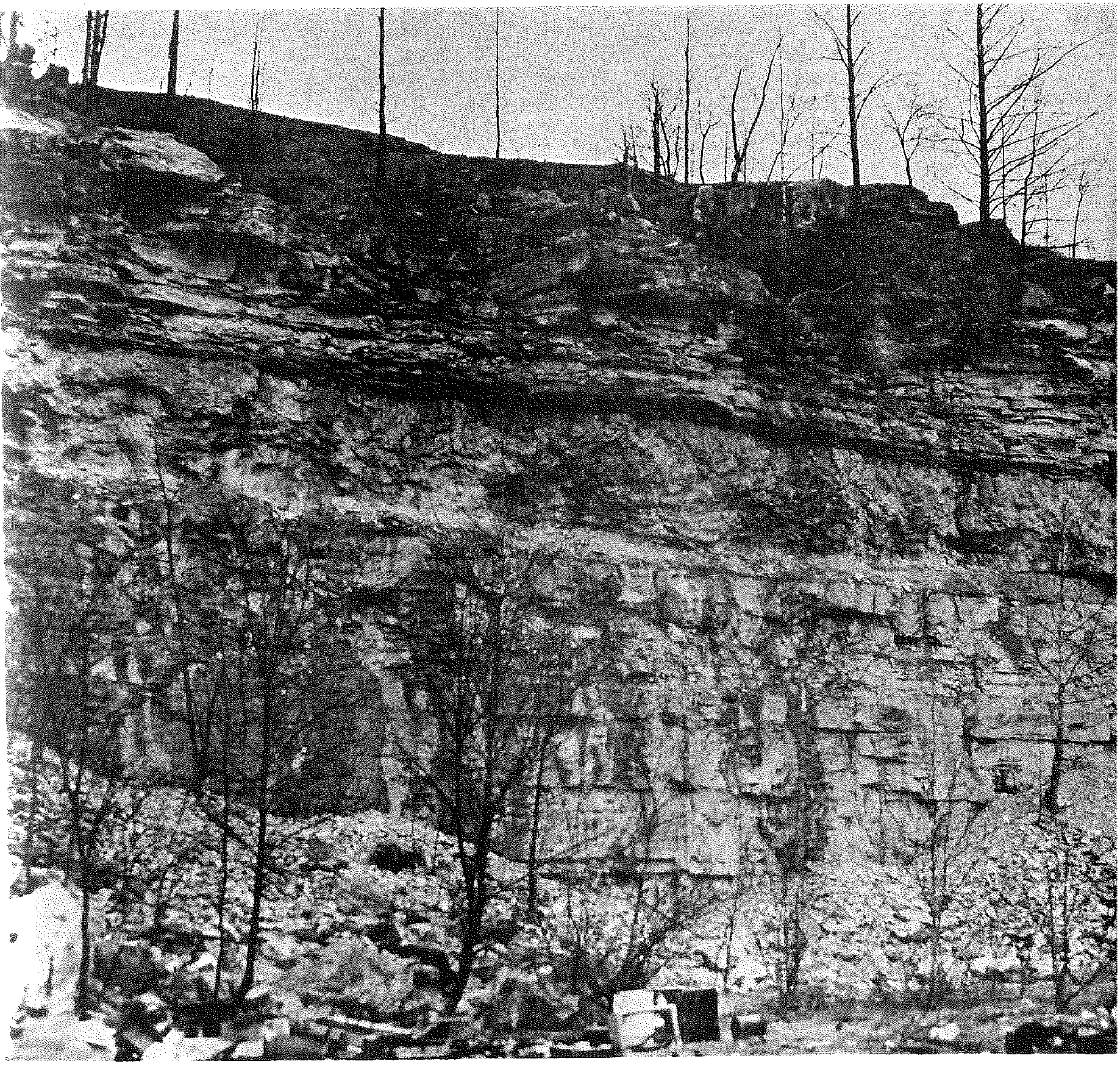
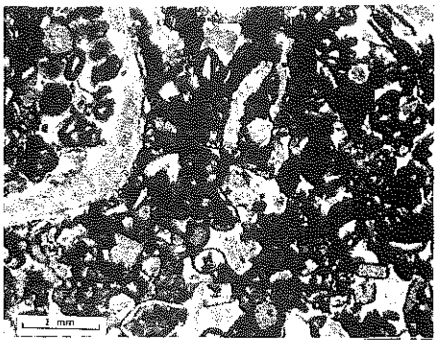
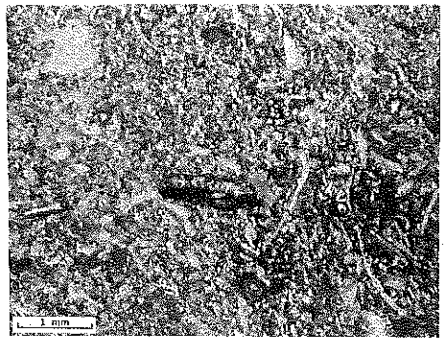


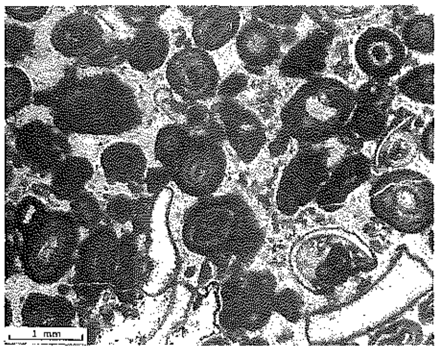
Figure 39. Olive Hill Quarry. Weathered ledges are at mid-elevation and are separated by a thin layer of more resistant rock; January 1972.



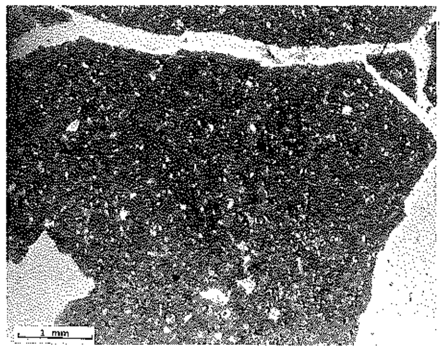
65 ft.



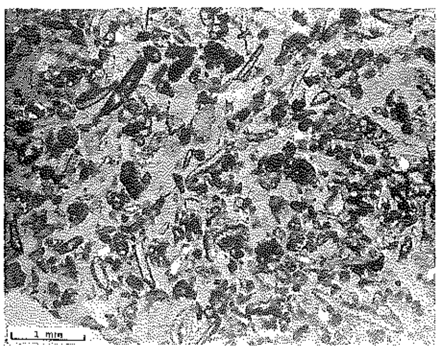
55 ft. (55 thru 51 ft.)



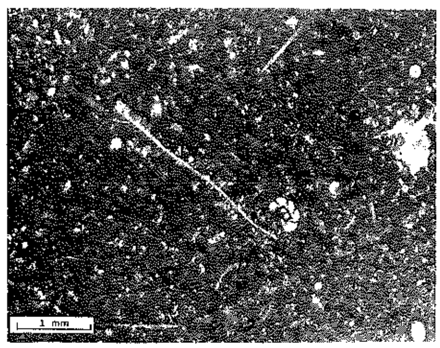
63 ft.



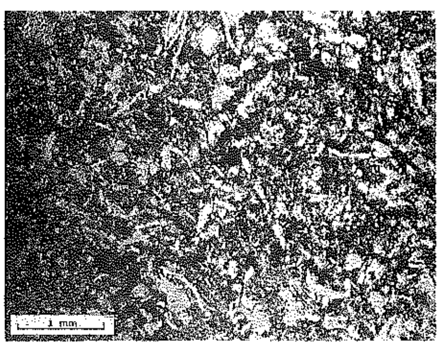
50 ft. (Like 42 thru 34 ft.)



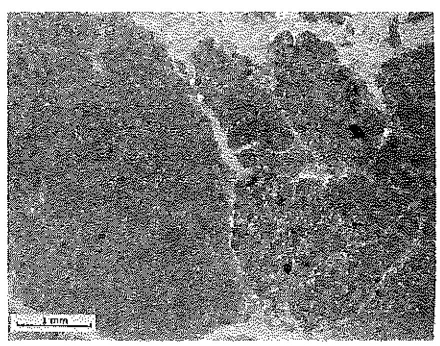
62 ft.



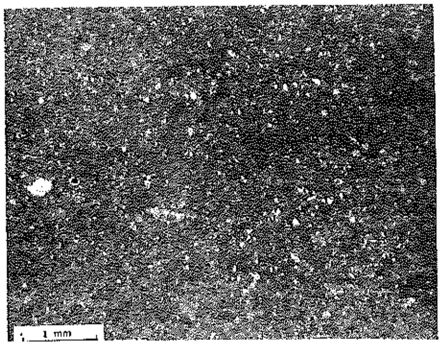
49 ft. (49 thru 43 ft.)



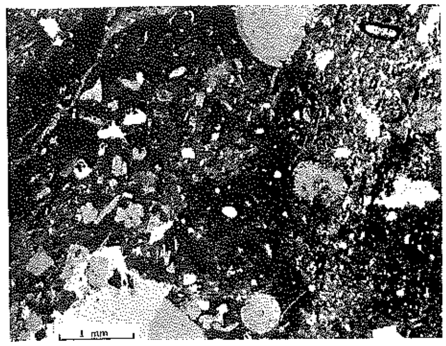
56 ft. (61 thru 56 feet)



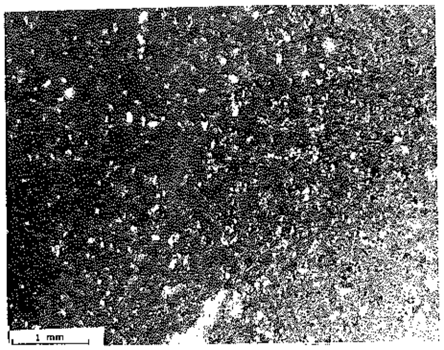
40 ft. (42 thru 34 ft.)



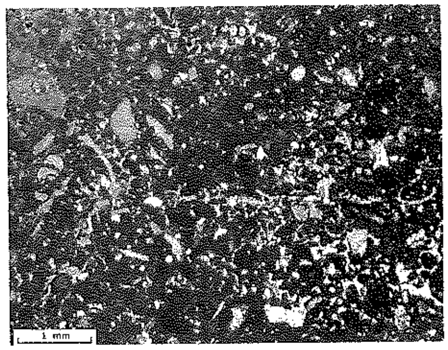
33 ft. (34 thru 29 ft)



20 ft.

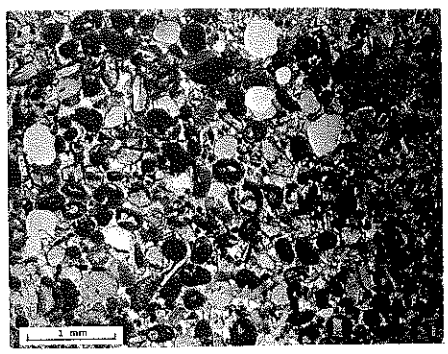


28 ft. (28 thru 27 ft.)

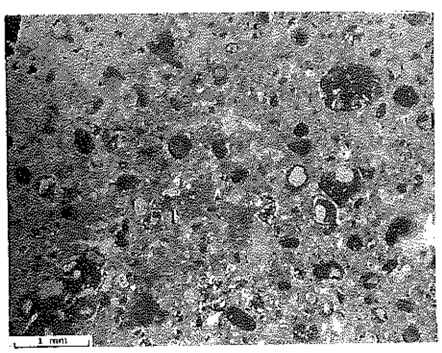


13 ft. (19 thru 8 ft.)

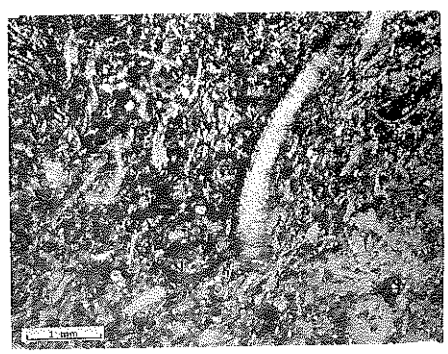
25 ft. (26 thru 24 ft.)



8 ft. (19 thru 8 ft.)



22 ft. (23 thru 21 ft.)



5 ft. (7 thru 1 ft.)

Figure 40. **Photomicrographs of Thin Sections, Old Olive Hill Quarry. Rapid weathering zones are from 42 through 34 feet and from 28 through 20 feet.**

Figure 41. Blowup on KY 32; June 1950.

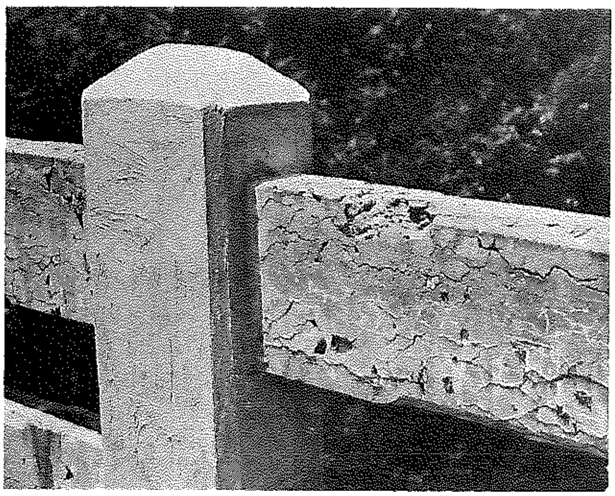


Figure 42. Decaying Safety Rail on Bridge.

Figure 43. Decaying Culvert Headwall.

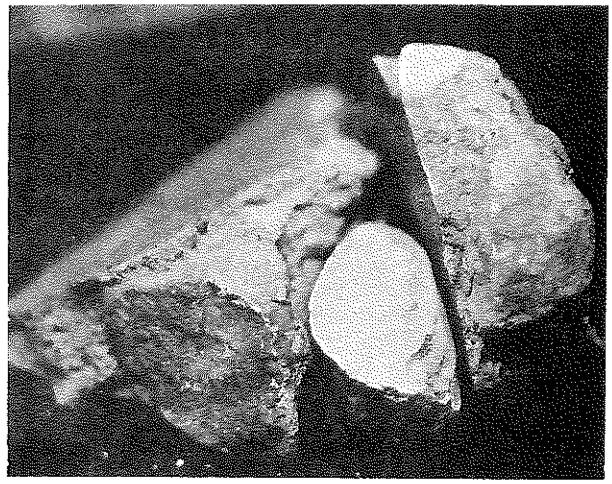


Figure 44. Chaulking Aggregate.

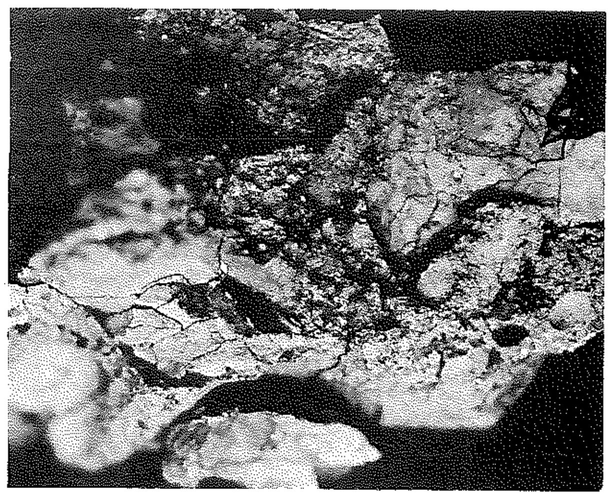


Figure 45. Internal Cracking.

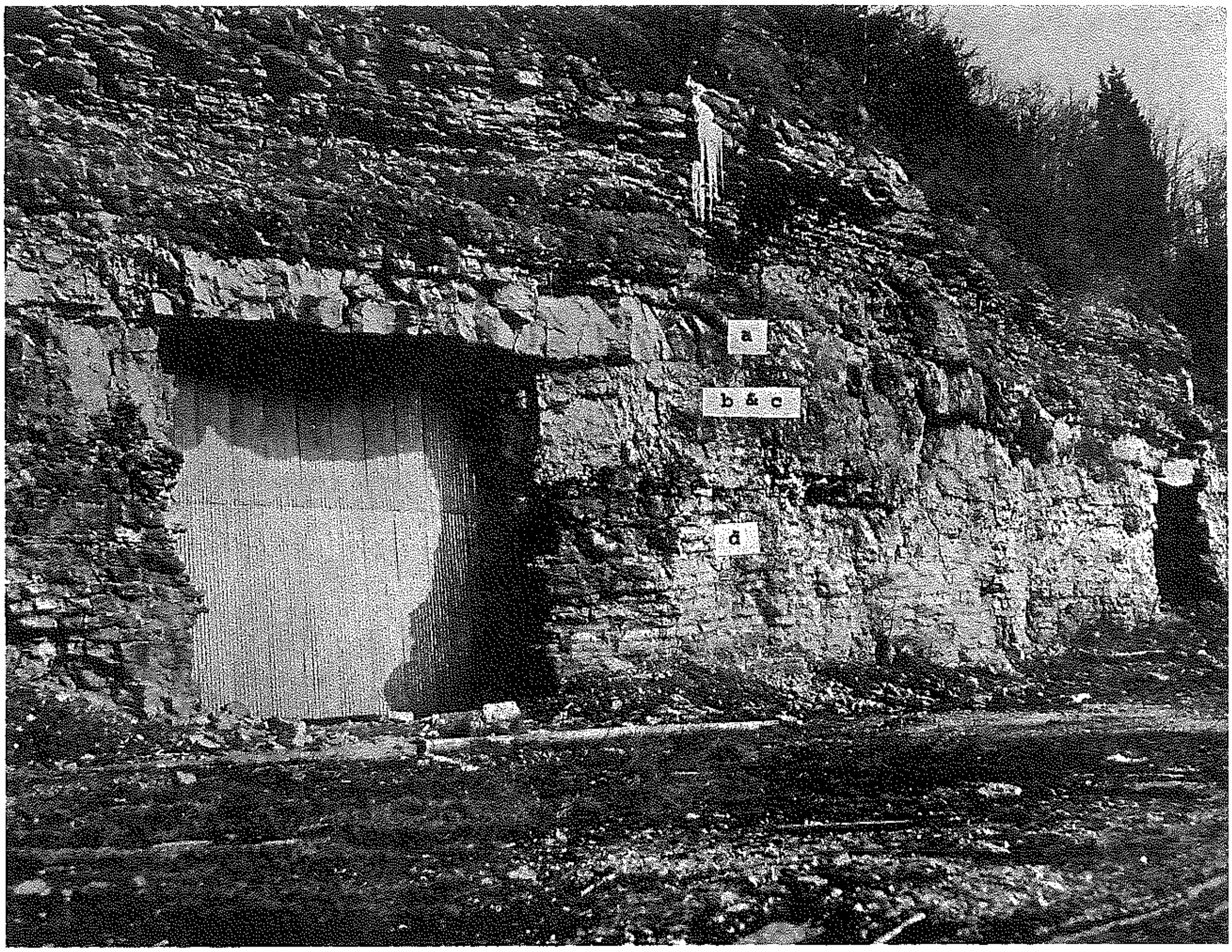


Figure 46. Old Tygart Limestone Company's Quarry and Mine at Lawton, Kentucky. Mine is now used to grow mushrooms. Proximity to Olive Hill quarry and poor performance of this aggregate on KY 32 invited investigation; February 10, 1972.

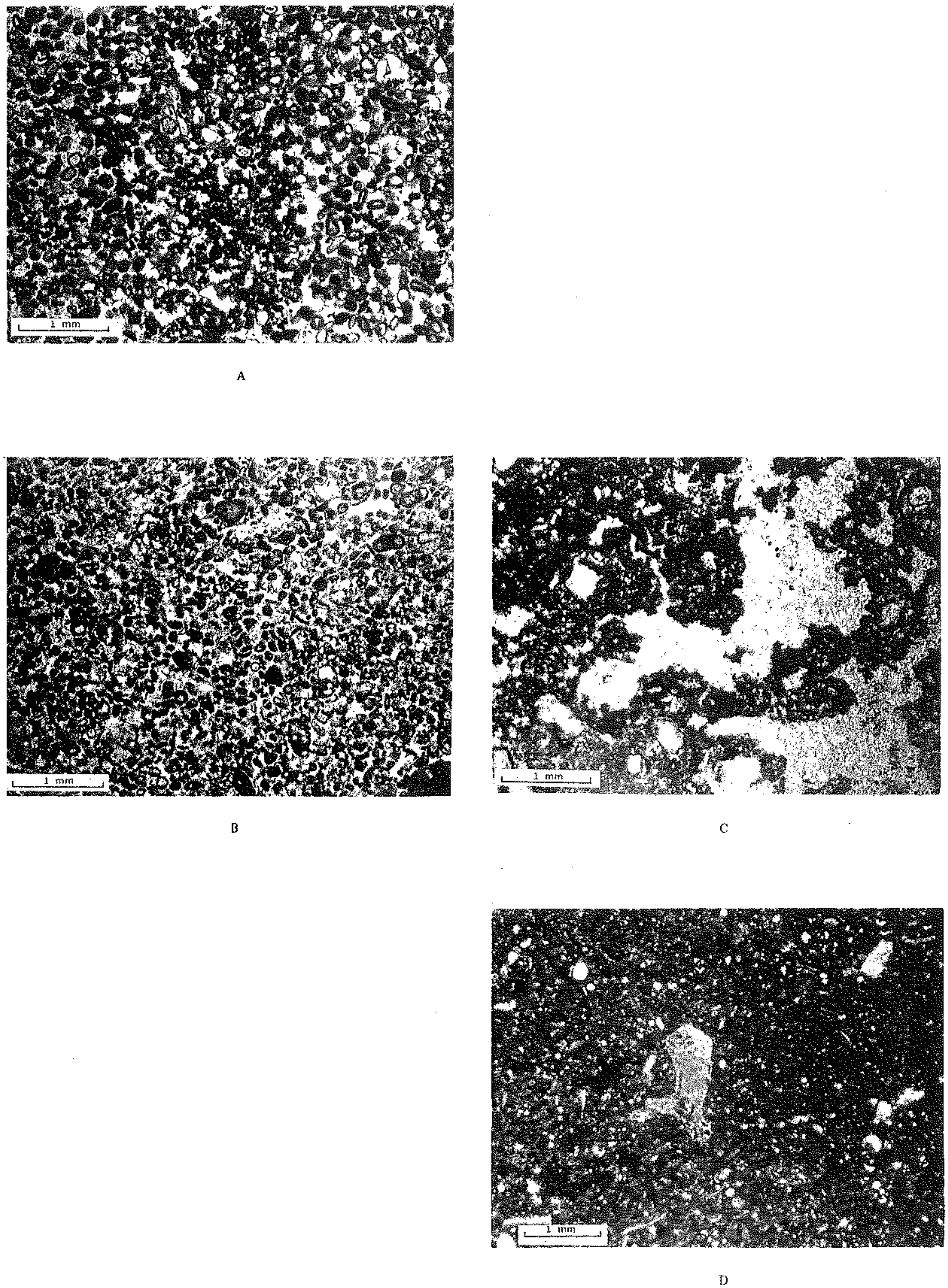


Figure 47. Photomicrographs of Rock Specimens from Lawton Quarry and Mine. The uppermost photograph (a) represents the ceiling ledge shown in Figure 46. The supporting ledge is shown in (b). A more crystallized portion of the same specimen is shown in (c). A specimen taken about mid-elevation of the portal is shown in (d).

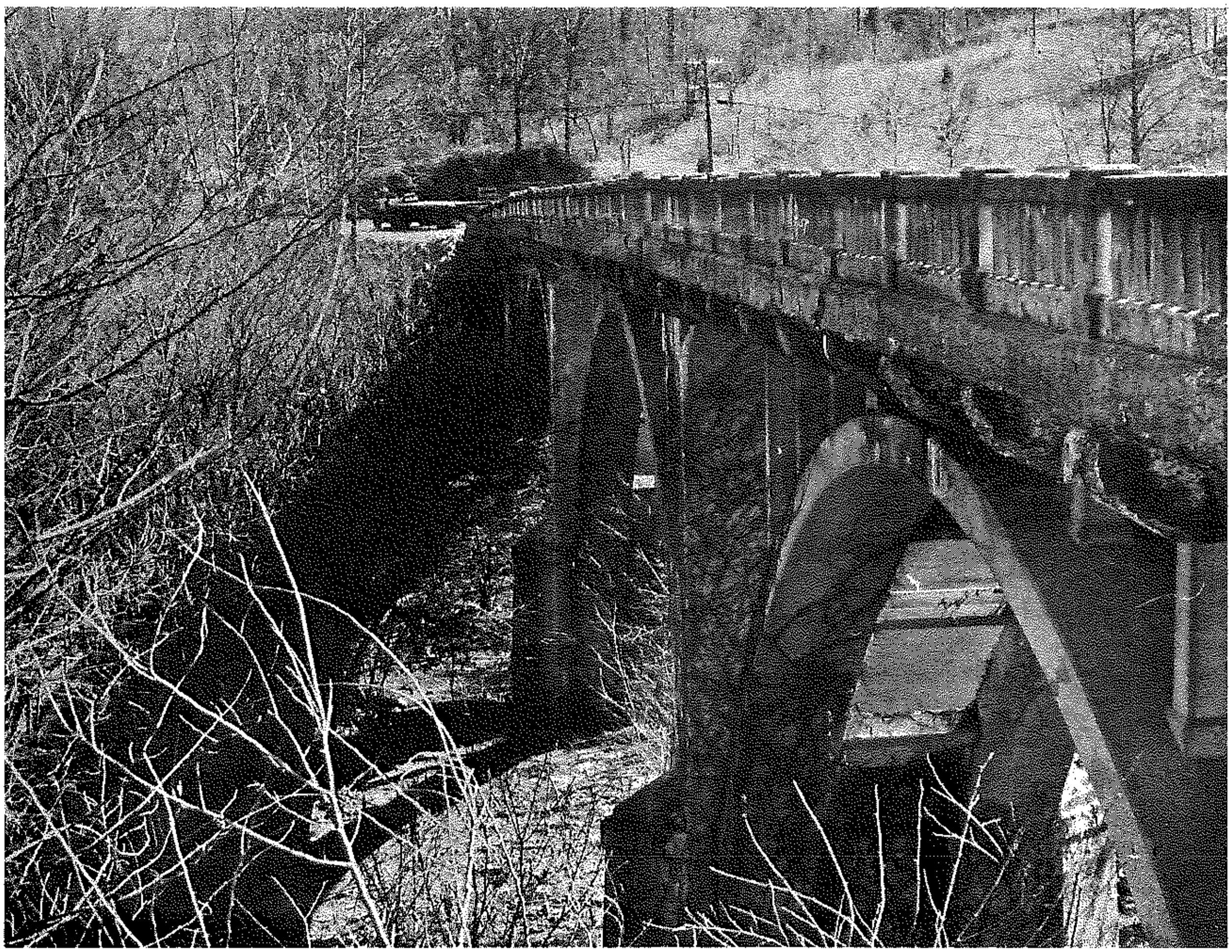


Figure 48. Tygart's Creek Bridge, US 60, East of Olive Hill; Built about 1927; March 1972.

US 31 W, FRANKLIN-TENNESSEE LINE, 1949

During the investigation of cracking on the I 65 project, a similar type of cracking was discovered on US 31 W between Franklin and the Tennessee line. The cracking is shown in Figure 49. Cores were extracted to determine the depth of the cracks. Three cores are shown in Figure 50. This pavement was constructed in 1949 *(18)*. It was an experimental design (no contraction joints, no wire mesh, expansion joints used at the end of each day of paving). Crack development was monitored for many years. The pavement carried heavy truck traffic until I 65 was opened to the Tennessee line. No satisfactory explanation for this cracking can be offered. Considerable edge deflection and some "pumping" could be seen when a heavy truck was passing. No cracks were detectable in the bottoms of the cores.

On the assumption that the cracks were not caused by fatigue, some attention was directed to aggregate source and to the possibility of expansive aggregate having been involved.

In 1949, the Harris and Crowden Quarry, 6 miles north of Franklin, in Simpson County, was operating in the Ste. Genevieve and or Gasper *(5)*. In 1958, the Southern Stone Company was operating a quarry, in the Ste. Genevieve, 6 miles north of Franklin and just west of US 31 W *(10)*. Kentucky Stone Company is now operating at or near this site. The stone used in paving the US 31 W project *(18)* is believed to have come from the Southern quarry. A 1958 quarry report, including chemical analysis, is included here as APPENDIX D. An analysis (circa 1950) of the nearby Harris and Crowden Quarry is presented below.

LEVEL	ZONE	INSOLUBLE RESIDUE	R_2O_3	$CaCO_3$	$MgCO_3$
38-41	1				
32-38	2	4.70%	0.84%	94.01%	0.26%
28-32	3	9.20%	2.24%	79.00%	7.52%
23-28	4	2.94%	0.96%	92.95%	2.39%
20 1/2-23	5				
15-20 1/2	6	6.64%	0.68%	92.34%	0.09%
10 1/2-15	7	1.62%	0.86%	95.59%	1.56%
8-10 1/2	8				
0-8	9	2.50%	0.72%	95.00%	1.62%

Zones (ledges) 6, 10, and 11 in the Southern quarry and Zone 3 in the Harris and Crowden Quarry appear suspect in relationship to the Hoover Quarry and the amount of $MgCO_3$ and insolubles.

Note: *A fault line runs between this area and the site of the Hoover Quarry. The displacement may exceed 100 feet. The quarries in the vicinity of US 31 W are evidently much higher in the Ste. Genevieve than the Hoover Quarry. The possibility of the same ledges having been quarried on both sides of the fault seems remote (See Figure 25). There seems to be a greater likelihood of continuity between these quarries and the Olive Hill quarry than between these quarries and the Hoover Quarry. Unfortunately, this situation was not discovered until late in the study, and the quarries west of 31 W were not sampled and analyzed.*

The abandoned quarry was visited March 7, 1972. No severely weathered ledges were noticeable. A photograph is shown in Figure 51.

DISCUSSIONS

SURFACE CRACKING

The pattern of cracking witnessed on I 65 is similar to cracking attributed to dolomitic, clay-bearing, limestone aggregates. The cracking mechanism is not understood. Cracks are shallow and appear to be caused by concrete shrinkage (due to drying). This seems somewhat contrary to the basic mechanisms -- which are expansion and swelling. Whereas overstresses arising from expansion are expected to be fatiguing and to eventually cause cracking, it is difficult to understand why the cracks do not extend through the full depth of the slabs. In a report of a study made in Arkansas *(19)*, it was noted that all blowups were associated with map cracking -- but cracking did not always result in a blowup. Cracks have not been detected in the bottom surface of the slabs on I 65.

It may be that cracks are somehow limited to the drying and wetting zone - that is, the uppermost 2 inches of the slab. In other words, the lower body of the slab never undergoes severe drying shrinkage.

BLOWUPS: INCIDENCE, CAUSES AND MECHANISMS

Blowups occur in pavements which in all other respects appear to be sound. They occur in pavements which have and do not have expansion joints in them. Statewide, in 1968, 14 blowups other than those on I 65, were reported. They occur mostly in long, straight sections of road. They tend to occur during the heat of the day (after 1 p.m.) and after a rain -- that is, when the pavement is in a tepid or nearly steaming condition.

Figure 49. US 31 W North of Tennessee Line; Non-Reinforced, 8-inch Slab, No Joints; Built Experimentally in 1949; Carried Heavy Traffic until I 65 Opened; Source of Aggregate Unknown; June 6, 1968.

Figure 50. Cores from US 31 W North of Tennessee Line. Surface cracks are delineated by wetting. The aggregate is presumed to be from the Ste. Genevieve formations north of Franklin. Pavement was cured with a lacquer-type membrane.

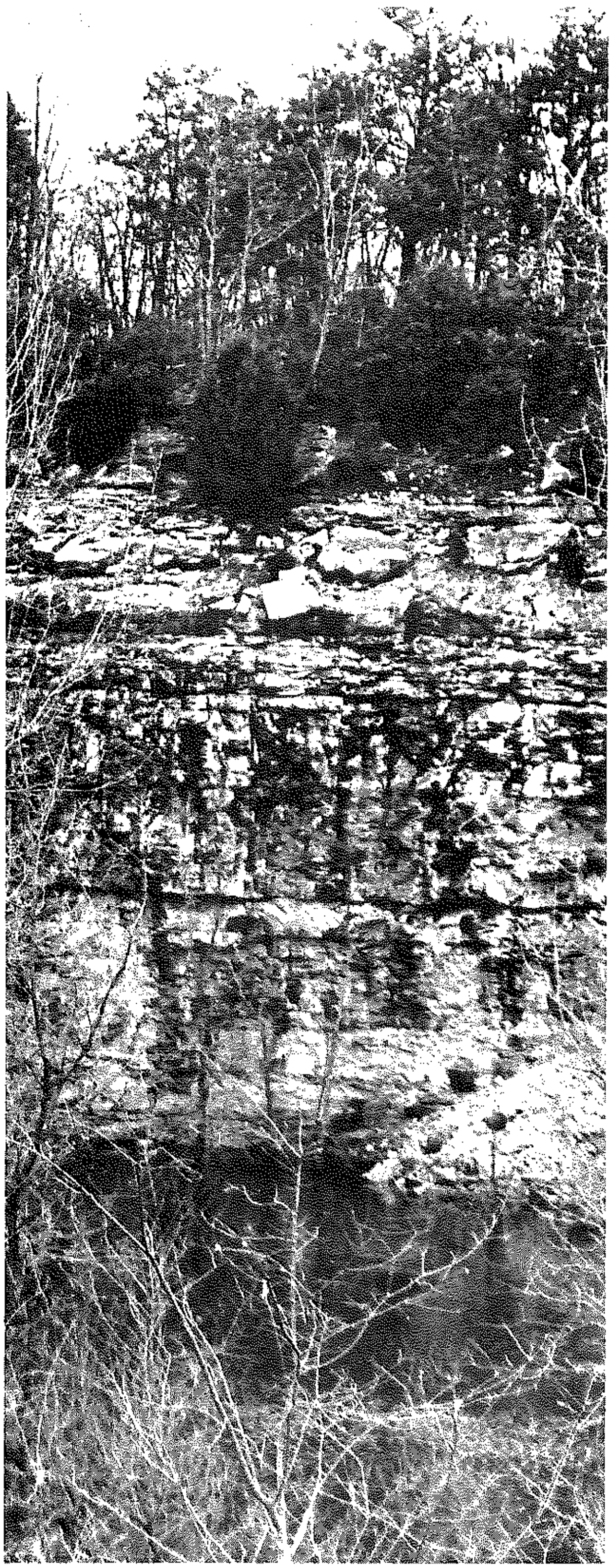


Figure 51. **Abandoned Quarry West of US 31 W, Six Miles North of Franklin; near Kentucky Stone Company's Quarry Now in Operation; March 7, 1972.**

The writer has heard eye-witness reports of lightning striking and shattering concrete pavements. Hensley *(19)* found fragments of concrete 95 feet away from a blowup. The highest incidence of blowups appears to be about 25 per mile *(20)*.

For a violent or explosive blowup to occur simply means that the expansive stresses become as great as the compressive strength of the concrete. For instance, a mile of pavement if tilted into a vertical position would exert a bearing pressure of about 5,000 psi. The acceleration in $F = Wa/g$ equates to coefficient of friction -- i.e. $f = a/g$; a is the acceleration in the direction considered; when $f = 1$, $a = g$. Therefore, if the friction of this mile of pavement laying horizontally on the earth were 1 -- usually assumed to be between 1.0 and 1.5 -- it would take as much force to push or pull the pavement horizontally as it would to lift it straight up.

Considering variability in concrete strength -- say 3,500 to 6,500 psi -- the zone of influence can extend from a half to more than a mile in each direction from the midpoint. Such would be the case if there were free ends. The compressive stresses at the ends would be zero. If the ends abutted other pavements exerting counter stresses, the stress would tend to become uniform throughout the length of the pavement. It would tend to fail at its weakest point.

It is difficult to calculate temperature stress rise in pavements because it is natural for concrete to shrink during hardening and to crack. Thermal stresses begin to develop after sufficient expansion has occurred to close all cracks and joints and to bring all surfaces into bearing. Cursory calculations indicate potential stresses in the order of 3,500 psi at 150° F. These stress cycles are unlikely to be fatiguing if they do not exceed 50 percent of the compressive strength of the concrete. When they reach 90 percent, a few cycles may cause failure. On the basis of this bit of logic, strong concrete is much more resistant toward blowups than weak concrete -- all other factors being equal.

Pavements built without expansion joints depend on this very principle.

MECHANISMS OF TIME EXPANSION AND SWELLING OF ROCK

A familiar analogy is the swelling of soaked wood. Swelling of clays is probably a better analogy here. In wood, the mechanism is definitely osmotic; in clays, the action is more abstractly termed electro-chemical -- which may involve some osmotic pressures. However, drying and resorption (absorption of water from humid air) is known to cause shrinking and swelling. Capillary forces accompanying re-wetting can cause explosive disintegration. Swelling pressures can be enormous. It is not unusual for clays to appear as impurities in sedimentary rocks such as limestones. In fact, the amount of impurities co-deposited varied from time to time. Shale partings and seams of clays and shales resulted. In some instances, the slimes became varved. During succeeding ages, consolidation and recrystallization proceeded toward a more orderly arrangement. Clays, not readily soluble, could not be expelled and so became entrapped. However, their presence interferred with the reformation and inter-growth of calcite and dolomite crystals. Some dolomite crystals encapsulated clay bodies; but, for the most part, both calcite and dolomite crystals remained small and discrete. Even so, there was sufficient interaction to achieve lithification. The cementing material is generally microcrystalline carbonates.

It is unlikely that these occlusions of clay have ever been deprived of equilibrium moisture -- desiccated or air-dried -- or subjected to cyclic wetting-and-drying. However, like shales, these impure limestones tend to air slake or otherwise deteriorate upon exposure to weather.

Others have suggested that the clay bodies are completely encapsulated by dolomite crystals and that subsequent exposure to alkalies (such as sodium and potassium originating from portland cement) causes dissolution of the dolomite -- thus freeing the clays to expand.

Hadley *(2)* proposed the following reaction (attributed originally to T. E. Stanton, cf, **Proceedings**, ASCE, December 1940):

$$CaMg(CO_3)_2 + 2\ M\ OH \rightarrow M_2CO_3 + Mg(OH)_2 + CaCO_3$$

in which M = Na, K, or Li. Apparently the $CaMg(CO_3)_2$ (dolomite) does decay in the presence of alkalies; but the difficulty with the reaction lies in the fact that it attributes the expansion entirely to the formation of $Mg(OH)_2$ (brucite) which cannot be found in any appreciable quantities in the rock specimens after expansion has occurred. The $Mg(OH)_2$ product, if formed, would indeed have a greater specific volume than the reactants. Nevertheless, it has been dubbed "the de-dolomitization" reaction and has won general acceptance elsewhere. Invariably, however, the most expansive rocks have contained about 10 percent $MgCO_3$ and about 10 percent clay and have had a very fine-grained structure.

Gillot *(21)*, in Canada, has been able to show swelling of "booked" clay minerals in this type of limerock by use of the scanning electron microscope. In Canada (Ontario), the problem rocks are known widely as the Kingston limestones and are Ordovician in origin (there, the Black River Group). It is interesting to note that the bedrock in the Bluegrass of Kentucky is Ordovician also.

It has been demonstrated here that impure limestones (Olive Hill and Franklin quarries) disintegrate in the absence of alkalies.

Finally, it is suggested that a year or more of outdoor exposure of ledgerock samples may disclose unsoundness not otherwise detectable by ordinary rock quality tests. An exposure field for ledgerock samples and (or) samples of standard aggregates used on significant projects would, in time, produce helpful information toward understanding the nature of unsoundness and undoubtedly lead to improvements in rock quality requirements and source approvals *(22)*.

Much yet could be related here about susceptibility of rock to damage by wetting and drying and freezing and thawing. Materials which have great, internal, surface area or which are clayey or clay-bound -- such as shales and shaley limestones -- decompose during soaking or cyclic wetting and drying. The rate of decomposition varies with the characteristics of the specific material. Exterior exposure will surely reveal this type of unsoundness; however, the process may be accelerated by forced wetting-and-drying apparatus.

Previous mention has been made of the fact that insidious expansion may escape detection unless a test such as ASTM C 586 or outside exposure is brought into use.

> Let the stone be got out two years before, in summer but not in winter, and let it lie in exposed places. Those stones which in this time are damaged by weather are to be used in foundations. Those which are not faulty are tested by Nature and can endure when used in building above ground.
>
> - Vitruvius, Roman Architect & Engineer, circa 25 B.C.

POROSITY: ROCK QUALITY AND DURABILITY CONTROL

Susceptibility to freeze-thaw damage is somewhat more predictable. In 1938, Cantrill and Campbell *(23)* associated poor performance of concrete pavements in Kentucky with absorption of coarse aggregates (cherts and gravels). Absorptions in excess of 3 percent were adjudged to be critical. Limestones were characteristically much less absorptive, and superior performance of concrete pavements containing limestone coarse aggregates was recognized at that time. In 1964 *(24)*, Scott and Laughlin demonstrated a high probability (nearly 100 percent, 4 cycles, submerged in chilled mercury) of rupture of aggregate particles when the saturated absorption exceeded 4 percent by weight. Further research on the mechanisms of damage in concrete was reported summarily in 1970 *(25)*. These studies clearly indicated that absorption (more especially absorptive capacity or porosity as determined from volume of absorbed water) is a measure of durability (resistance to freezing and thawing) and is, therefore, a quality-control property -- which, unfortunately, is too often not utilized fully in specifications. When absorption is determined from composite samples (containing many particles as in the case of gravels and stockpiled aggregates), the result is a weighted average and does not analytically reveal the nature of the constituent particles -- that is to say, a significant proportion of the particles could be highly absorptive while that of the composite sample might be low. Actually, it may be the percentage of damage-susceptible particles which renders the whole aggregate undesirable. Heavy medium separation is a means of sorting out the more absorptive particles. A Jolly balance enables one to weigh relatively small particles in air and in water and to, thereby, determine absorptions of small aggregates, particle by particle. Ledgerock specimens do not involve so much tedium.

Currently, tests on ledgerock specimens from quarries are being used for source approvals. Wear and sodium sulphate soundness tests are the principal controls. It is suggested here that the sodium sulphate test could be supplanted altogether by absorption and specific gravity tests -- which are customarily and routinely performed and reported. For instance, a relatively pure limestone having an SSD specific gravity of 2.55 (an absorption of 7.74 percent by volume) is definitely considered to be unsound. To enable implementation of these ideas into quality control procedures, a nomograph is provided (Figure 52) which, by interpolation, permits one to determine absorption by volume (porosity) -- knowing the SSD specific gravity and mineralogical composition. There, it is possible to adjust for heavier and lighter minerals if the composition is known and to determine porosity knowing only the SSD specific gravity. Other helpful formulas follow.

1. If only the SSD specific gravity and chemical composition are known (c.f., Quarry Log), compute the average combined specific gravity of solids from:

   a. Sp. G. (Avg. comb.) = 100[(%$CaCO_3$/2.68) + (%$MgCO_3$/3.02) + (% Insol./ 264)]

   b. Porosity (% by vol.) = [Sp.G. (Avg. comb.) - SSD Bulk Sp.G. x 100] / Sp. G.(Avg. comb.) - 1]

   c. Absorption (% by wt.) = (%Porosity x 62.4/ 100)/[SSD Bulk SP.G. x 624 - (Porosity x 62.4/100].

2. When SSD Bulk specific gravity and OD Bulk specific gravity are known:

   a. Porosity (% by vol.) = (SSD Bulk S.G. - OD Bulk S.G.) x 100

   b. Absorption (% by wt.) = 100 (SSD Bulk Sp.G. - OD Bulk Sp.G.)/OD Bulk Sp.G.

3. When OD Bulk S.G. and Absorption are known:

   Porosity(% by vol.) = OD Bulk Sp. G. x %Absorption)/62.4.

4. When SSD Bulk Sp.G. and Absorption are known:

   Porosity (% by vol.) = SSD Bulk Sp. G. x % Absorption/ [1 + (% Absorption/100)].

The probability of the aggregate fracturing in 4 cycles of quick freezing is shown in Figures 53 and 54. A probability limit of 50 percent or less might be considered a reasonable goal. This would be less than half the limit of absorption established by Cantrill and Campbell *(16)*. Porosity (volume of absorbed water) is a purer parameter than absorption (percent by weight). Disqualifying ledges would necessarily be eliminated by selective quarrying.

Figures 53 and 54 invite further conjecture. Cursory analysis of weather records indicate that there are about 32 cycles of freezing and thawing per year in this area; but for the sake of this conjecture, assume a 25 percent probability of 4, saturated, freezing cycles occurring in any year. The probability of damage in any year is: 25 percent x the probability of damage occuring in 4 cycles. Further conjecture permits estimation of life expectancy. This may be defined as: 100/probability of damage in any year. The resulting life expectancies are shown superimposed on Figures 53 and 54.

The question then arises: How much or what percentage of such particles is admissable in a composite aggregate? First, consider near-surface particles only -- that is, the likelihood of popouts or surface spalling occurring. To complete the exercise, assume a continuous distribution (statistical) of aggregate particles having a porosity of 1.2 percent and particles having a porosity of 4.1 percent. Overlaying this cumulative frequency line on Figure 54 permits interpolation of: 1) sample porosity, 2) probable life (8 years for particles having a porosity of 4.1 percent), and 3) the percentage of offending particles (near the surface and throughout). Other distributions, if known, would be amenable to analysis; but the analysis becomes infinitely more

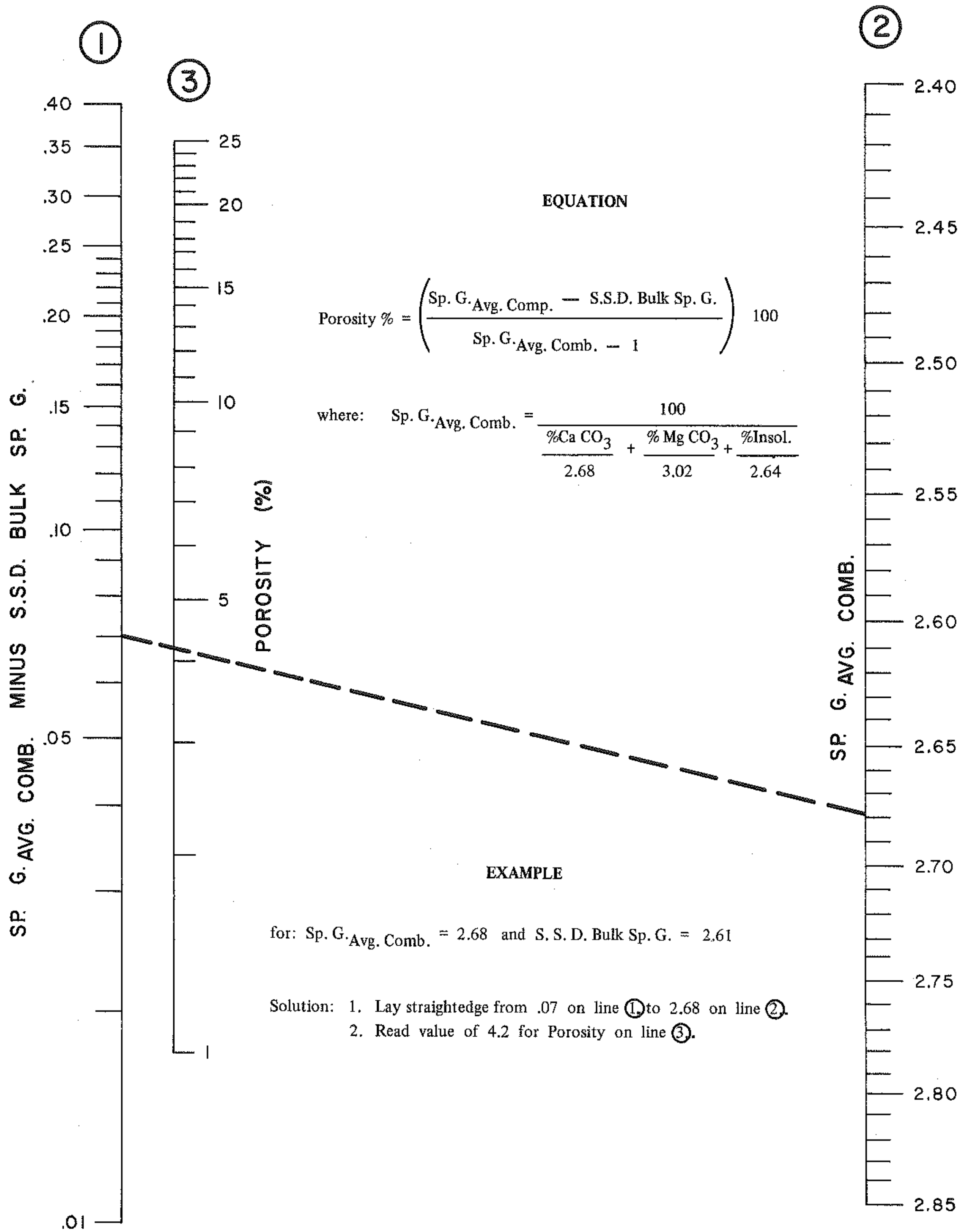


Figure 52. Nomograph for Interpolating Porosity when Two Specific Gravities are Known.

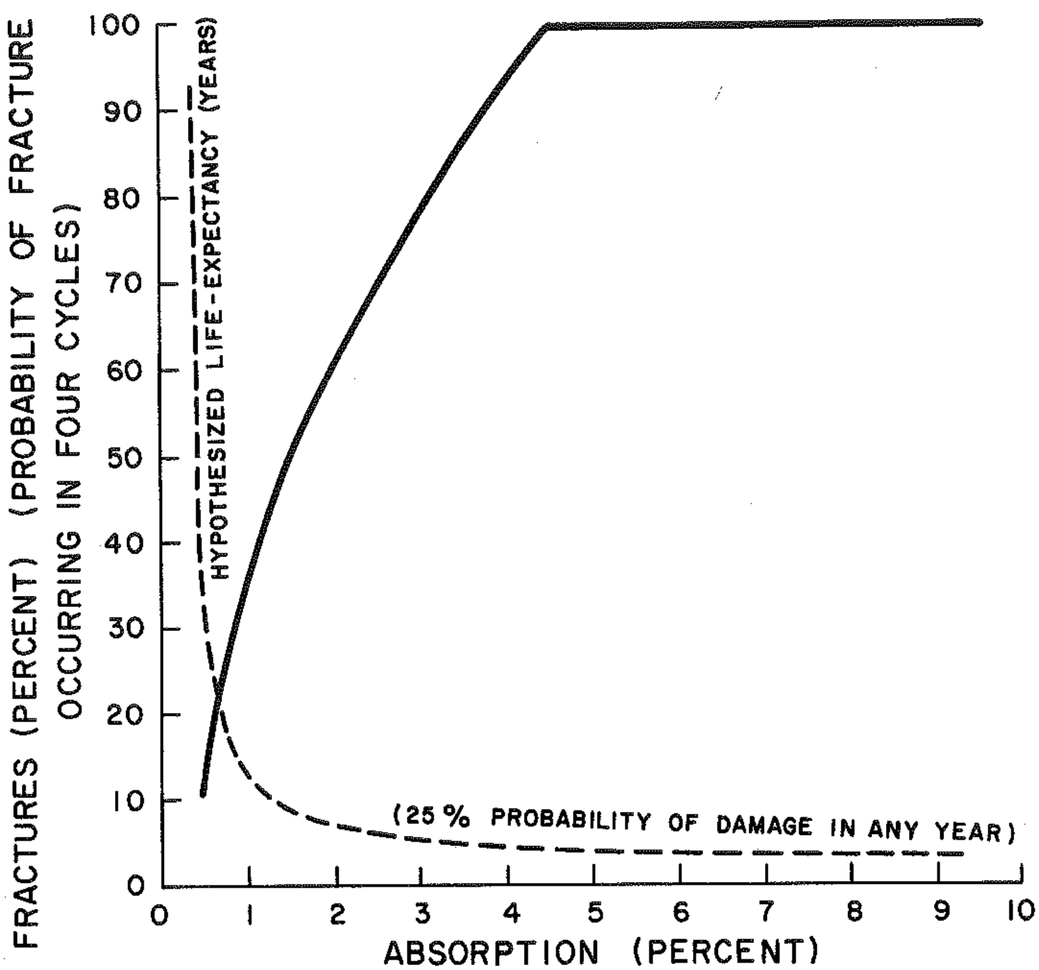


Figure 53. Relationship between Absorption and Percentage of Fractured Particles after Exposure to Four Freeze-Thaw Cycles (also Shows Hypothesized Life Expectancy).

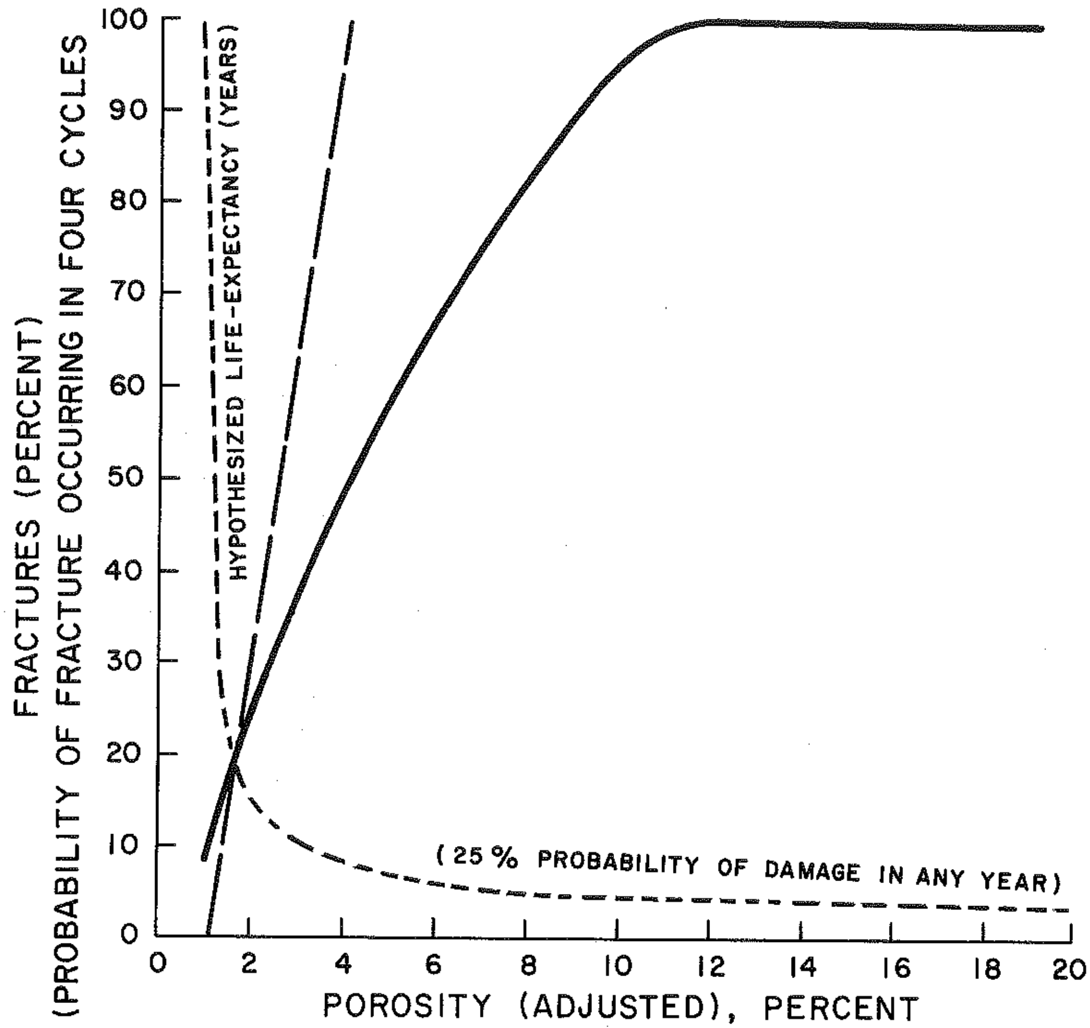


Figure 54. Relationship between Porosity (Adjusted Value) and Percentage of Fractured Particles after Exposure to Four Cycles of Freeze-and-Thaw. Hypothesized life expectancy is shown by the short dashed curve (read years on ordinate). The long dashed line shows a continuous distribution of particles having a porosity of 1.2% and 4.1%, respectively. The percentage of particles having 4.1% porosity is read on the ordinate. The sample porosity is read on the abcissa. The percentage of particles having short life expectancy is, here, the same as the percentage of particles having a porosity of 4.1%.

complicated. At least 30, randomly chosen particles would suffice to establish a statistically valid distribution. When the sample porosity is small, the likelihood of highly absorptive particles being present is also small. A sample porosity of not greater than 2 percent would seem highly reassuring.

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APPENDIX A

ASTM C 586-69

Standard Method of Test for

POTENTIAL ALKALI REACTIVITY OF CARBONATE ROCKS FOR CONCRETE AGGREGATES (ROCK CYLINDER METHOD)[1]

This Standard is issued under the fixed designation C 586; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval.

1. Scope

1.1 This method covers determination of the expansive characteristics of carbonate rocks while immersed in a solution of sodium hydroxide (NaOH) at room temperature. The observable length changes occurring during such immersion indicates the general level of reactivity of rocks and whether tests should be made to determine the effect of aggregate prepared from the rocks upon the volume change in concrete.

NOTE 1—Alkalies participating in the expansive reactions usually are derived from cement; under certain circumstances they may be derived from other constituents of concrete or from external sources. Two types of alkali reactivity of aggregates are recognized: (*1*) an alkali-silica reaction involving certain siliceous rocks, minerals, and artificial glasses and (*2*) an alkali carbonate reaction involving dolomite in certain calcitic dolomites and dolomitic limestones. This method is not recommended as a means to detect the first of these reaction types.

1.2 The test is intended as a research screening method rather than for specification enforcement. It is intended to supplement data from field service records, petrographic examinations, and tests of aggregate in concrete.

2. Summary of Method

2.1 Small rock cylinders are continuously immersed in a solution of NaOH. The length change of each specimen is periodically determined.

3. Significance

3.1 The method is intended to give a relatively rapid indication of the potential expansive reactivity of certain carbonate rocks that may be used as concrete aggregates. The method has been successfully used in (*1*) research and (*2*) preliminary screening of aggregate sources to indicate the presence of material with a potential for deleterious expansions when used in concrete.

4. Apparatus and Reagents

4.1 *1 N Sodium Hydroxide Solution*—Dissolve 40 ± 1 g of reagent grade sodium hydroxide (NaOH) in distilled water, dilute to 1 liter and store in a polyethylene bottle (Note 2).

4.2 *Sawing, Drilling, and Grinding Equipment*, suitable for preparing test specimens of the dimensions given in Section 7. This will require one or more rock saws, depending upon the size of the original sample, a drill press equipped with a small diamond core barrel for removing the cylindrical core, and a lap, grinder, or suitable modified lathe for shaping the ends of the specimens.

4.3 *Storage Bottles*, approximately 50 to 100-ml capacity with caps and openings of sufficient size to facilitate removal of specimens.

NOTE 2—In selection of the bottle, care should be taken to assure that the solution will not be modified by reaction with the material composing the container, including pigments or other additives

[1] This method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.02.02 on Chemical Reactions of Aggregates in Concrete.
Current edition effective Oct. 3, 1969. Originally issued 1966. Replaces C 586 - 66 T.

or by transpiration of phases through the walls of the container. Bottles with wall thickness not less than 0.020 in. (0.50 mm) and composed of high density polyethylene meeting the requirements of ASTM Specification D 1248, for Polyethylene Plastics Molding and Extrusion Materials,[2] for materials of Type [1], Class A, are suitable.

4.4 *Length Comparator*, of such design as to provide, permit, or include the following characteristics (Note 3):

4.4.1 A positive means of contact with the conical ends of the specimen to ensure reproducible measurement of length (Note 4),

4.4.2 A high-grade barrel or dial micrometer graduated to read in 0.0001-in. (0.0025-mm) units, and accurate within 0.0001 in. (0.0025 mm) in any 0.0010-in. (0.025-mm) range and within 0.0002 (0.0050 mm) in any 0.0100-in. (0.25-mm) range (Note 5),

4.4.3 A sufficient range to allow for small differences among gage lengths of various specimens (Note 6),

4.4.4 A standard or reference for checking the measuring device at regular intervals (Notes 7 and 8), and

4.4.5 A convenient and rapid means for measurement of specimens.

NOTE 3—One type of comparator which has been found satisfactory is shown in Fig. 1.

NOTE 4—A variety of contact points have been used successfully. Care should be exercised to ensure that when using specimens with conical ends as described in 6.3, contact is made on the end along a circle which is concentric about the long axis of the specimen. If the measuring device is a barrel micrometer, it shall have a ratchet stop to produce a constant contact pressure on the specimen.

NOTE 5—It is recommended that the measuring device be calibrated throughout its range to determine both periodic and cumulative errors for proper correction of observed data.

NOTE 6—If care is taken in the fabrication of the specimens, a measuring device with a travel of not less than 0.3 in. (7.5 mm) provides ample range in the instrument.

NOTE 7—The bar that serves as a reference for the length comparator shall have an over-all length of 1.38 ± 0.08 in. (35 ± 2 mm). The bar shall be fused silica or a steel alloy having a coefficient of thermal expansion not greater than 1.0×10^{-6}/deg C. Each end shall be machined to the same shape as that of the rock specimens. If a steel alloy is used, it shall be heat treated, hardened, and then polished. The reference bar shall be placed in the instrument in the same position each time a length measurement is made.

NOTE 8—The micrometer setting of the measuring device shall be checked by use of the reference bar at least at the beginning and end of the readings made within a half day when the apparatus is kept in a room maintained at constant temperature. It shall be checked more often when kept in a room where temperature is not constant.

5. Sampling

5.1 This method is applicable to a sample of rock secured in accordance with the applicable requirements of ASTM Methods D 75, Sampling Stone, Slag, Gravel, Sand, and Stone Block for Use as Highway Materials.[3]

5.2 Sampling should be under the direction of an individual capable of distinguishing differences in lithology, and the sample of rock should be taken to represent only the particular lithology under consideration bearing in mind the limitations and significance of this method as stated in 3.1. Each sample of rock should be in one piece of sufficient size for preparing the necessary test specimens.

5.3 One test specimen will sufficiently represent the sample of rock unless shale seams or other discontinuities are present or the bedding is not discernible. In these cases, it may be desirable to prepare and test 3 mutually perpendicular specimens. Of such three specimens that one showing the greatest length change after 28 days of immersion in alkali solution as determined in 7.7 shall be used. The remaining two may then be discarded.

6. Test Specimens

6.1 Test specimens shall be in the form of right circular cylinders with conical ends.

NOTE 9—In recommending the specimens described in 6.1, it is not intended to exclude the use of other test specimens for special purposes or under special conditions. If square prismatic specimens are used the over-all dimensions of length and distance between parallel faces should approximate the corresponding dimensions for the test specimen described in 6.2.

6.2 The specimen shall have an over-all length of 1.38 ± 0.20 in. (35 ± 5 mm) and a diameter of 0.35 ± 0.04 in. (9 ± 1 mm).

NOTE 10—Care shall be exercised in the preparation of the specimens to avoid alteration of the cylindrical surface by polishing or with materials which will affect the rate of entry of alkali solution into the rock.

6.3 The included angle of the conical ends shall be approximately 120 deg.

NOTE 11—Flat end faces may be used in place of

[2] *Annual Book of ASTM Standards*, Part 26.
[3] *Annual Book of ASTM Standards*, Part 10.

conical ends provided precautions are taken to ensure that the end faces are parallel to each other and perpendicular to the major axis of the specimen and providing that the requirements set forth in 4.4 are met.

7. Procedure

7.1 Place a position mark on the specimen to permit placing the specimen in the comparator in the same position during subsequent measurements.

7.2 Measure the length of the test specimen.

7.3 Immerse the specimen in distilled water at room temperature.

7.4 At intervals, remove the specimen, blot to remove excess surface water and measure until the change in length during a 24-h water immersion period does not exceed 0.02 percent as calculated in 8.1. The length when this condition is obtained is taken as the reference length.

NOTE 12—The reference length is usually achieved after 1 to 4 days of immersion.

7.5 Immerse the water-saturated specimens in a bottle containing a minimum of 35 ml of 1*N* NaOH solution per specimen at room temperature and seal. Immerse no more than two specimens in a bottle.

7.6 Measure the length of the specimens after 7, 14, 21, and 28 days of immersion in NaOH solution and at 4-week intervals thereafter. If the tests continue beyond 1 year, make measurements at 12-week intervals.

7.7 When measurements are made, remove the specimen from the bottle, rinse with distilled water, blot to remove excess surface water and determine its length in the same position as during the initial measurement.

7.8 After measurement, immediately return the specimen to the bottle and reseal.

7.9 Replace the solution every 6 months during the testing period.

8. Calculations

8.1 Calculate the length change to the nearest 0.01 percent of the reference length as follows:

$$\Delta l = [(l_1 - l_o)/l_o] \times 100$$

where:

Δl = length change at test age, percent,

l_1 = length in in. (mm) at test age, and

l_o = reference length after equilibrium in water, as outlined in 7.4.

9. Interpretation of Results

9.1 Since the expansion caused by reactions between cement alkalies and carbonate aggregates is sensitive to subtle changes in aggregate lithology, the results of measurements should be interpreted with full recognition of the variables which would affect the results obtained. The acceptance or rejection of aggregate sources based solely on the results of this test is not recommended since, in commercial production, expansive and nonexpansive materials may occur in close proximity and the securing of samples adequately representative of the variability of the production source is a difficult task and requires the efforts of an individual trained to distinguish differences in lithology. The test procedure is intended as a research or screening method rather than for specification enforcement. It is intended to supplement data from field service records, petrographic examinations, and tests of aggregate in concrete.

9.2 The relationship of the test results to the behavior of large quantities of rock from a given source will depend upon the degree to which the petrographic and chemical properties of the rock vary within the source.

9.3 Research results have indicated that the expansive behavior of aggregate in concrete is qualitatively predicted by the results of the rock cylinder test. Quantitative prediction of the expansion of concrete containing reactive aggregate depends upon (*1*) the degree of aggregate reactivity, (*2*) the amount of reactive constituent, (*3*) the alkali content of the cement, and (*4*) the environment. Appreciable expansion should indicate the need for further testing. In the light of current knowledge, it appears that expansions in excess of 0.10 percent are indicative of chemical reaction and should warrant additional testing, preferably in concrete. Usually expansive tendencies are evident after 28 days of immersion in alkali, however, exceptions to this have been noted. Deleterious expansion of concrete appears to depend upon the magnitude and rate of aggregate expansion and the time at which it begins. However, quantitative predictions of concrete expansion in service solely from results of this test method are not yet possible.

10. Report

10.1 The report shall include the following:

10.1.1 Identification number,

10.1.2 Type and source of rock,

10.1.3 Specimen shape and dimensions if other than right circular cylinder,

10.1.4 Length change in percent to the nearest 0.01 percent at each time of measurement. Where no times of measurement are specifically requested, data should be presented for at least the following ages: 1, 4, 8, and 16 weeks and the age at the final measurement,

10.1.5 Significant features revealed by examination of specimen during and after storage in alkali solution, such as cracking, warping, splitting, etc., and

10.1.6 Other significant information as deemed necessary, such as petrographic and chemical analyses.

11. Precision and Accuracy

11.1 If the results of replicate specimens measured by the same operator and which presumably represent the same material, differ by more than 0.10 percentage point for expansions less than 1.0 percent, it is highly probable that the specimens represent rocks that are significantly different in chemical composition, texture, or both.

11.2 The single-operator, single-comparator, single-specimen precision has been found to be ±0.02 percentage point (3S) as defined in ASTM Recommended Practice E 177, for Use of the Terms Precision and Accuracy as Applied to Measurement of a Property of a Material.[9]

11.3 The multi-operator, single-comparator, single-specimen precision has been found to be ±0.03 percentage point (3S) as defined in Recommended Practice E 177.

11.4 The multi-operator, multi-comparator, single-specimen precision has been found to be ±0.05 percentage point (3S) as defined in Recommended Practice E 177.

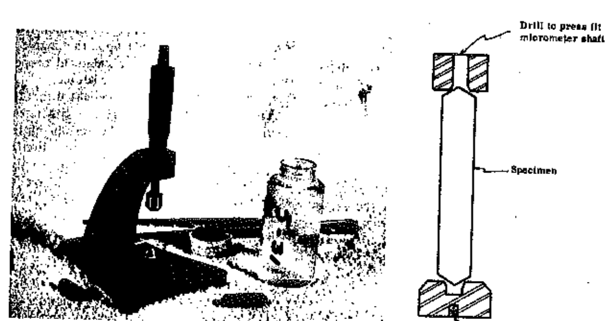

FIG. 1 A Typical Length Comparator.

APPENDIX B

Report of Portland Cement Association
January 30, 1968
December 19, 1967

N Days Copy

PORTLAND CEMENT ASSOCIATION
RESEARCH AND DEVELOPMENT LABORATORIES
5420 Old Orchard Road
Skokie, Illinois

COPY OF LETTER FOR

Applied Research Section

January 30, 1968

Copy - L H Havens
Research

Paul Klieger

Laboratories

On December 20, 1967, a report of our investigation of abnormal cracking in Interstate 65 in southern Kentucky was forwarded to the PCA Kentucky District Office. It was concluded from this investigation that the observed cracking was caused by expansive alkali carbonate reactivity involving a portion of the coarse aggregate. Supporting laboratory evidence was presented which indicated that unused aggregate taken from the same quarry that provided rock for the pavement showed expansions in excess of 0.10 percent after 28 days' immersion in 1N NaOH solution (ASTM C 586-66T). At that time, however, the only direct link between these findings and the conclusion that the aggregate used in the pavement was deleteriously reactive was the similarity in texture and composition of aggregate that was tested; particles taken from the pavement concrete and immersed in alkaline solution showed only minor shrinkages.

Later data are now available that indicate that coarse aggregate used in the pavement is indeed deleteriously reactive. After 8 weeks' storage in alkaline solution, particles (No. 11, Table 1 of original report) removed from the concrete show expansions in excess of 0.70 percent, thus confirming the conclusions set forth in the original report.

David Stark

DAVID STARK

D.S.*jd
9.16

From LH Kentucky
Dist - Feb 9-68

PORTLAND CEMENT ASSOCIATION

INTER-OFFICE CORRESPONDENCE

Mays Copy
From Johnston-PCA
1-5-68
Copy to J Havens
1-12-68
COPY TO D. ARNALL
1-18-68

Applied Research Section

December 19, 1967

Paul Klieger

Laboratories

OUR FILE

YOUR FILE

In a recent visit to the Laboratories, Mr. William Teske of the PCA General Office brought to our attention the development of abnormal map cracking in approximately 20 miles of Interstate 65 located in southern Kentucky. Discussion of the problem failed to reveal whether the observed cracking was the result of shrinkage or expansion; however, several blowups have been reported, which would suggest a relationship with excessive expansions. In his request for our assistance, Mr. Teske provided several concrete hand specimens taken from a blow up for laboratory examination. Later, Mr. Johnston of the PCA Kentucky District Office provided photographs of the pavement and aggregate samples obtained from the quarry that supplied material for the concrete in question. The results of the laboratory investigation are reported below.

Pavement Design and Materials

The section of Interstate 65 showing the abnormal cracking was constructed in 1964-65. Design called for uniform 10 inch thick, mesh reinforced slabs with joints spaced at 50-foot intervals. Materials used included Cosmos cement containing 0.45 percent alkali as Na_2O and coarse aggregate, 2 to 2½ inch maximum size, obtained from the Hoover quarry located in Simpson County, Kentucky. Air entrainment was also specificed.

Info from where?

Laboratory Investigation

Microscopic examination of the concrete revealed the presence of a random crack pattern in the mortar that did not provide unequivocal evidence of either shrinkage or expansion. Other microcracks were observed in gray and grayish brown, fine grained carbonate aggregate particles, some of which extended into the surrounding mortar and appeared to have resulted from excessive expansion originating in the particles. It was noted that many of these particles contained very thin but well defined dark rims that are typical of those often associated with expansive alkali-carbonate reactivity. Several of these particles also displayed fine peripheral separations at the paste interface that suggest shrinkage of the aggregate relative to the surrounding mortar.

In view of the evidence of possible alkali-carbonate reactivity and expansion, a sample of aggregate was obtained from the Hoover quarry and tested according to ASTM C 586-66T. For this purpose, the aggregate sample was separated into 10 groups based on color and megascopic textural variations. Prisms were sawed from particles in each group, stabilized in water and then immersed in 1 N NaOH for 28 days, during which period measurements were made for possible length changes. In addition, powdered samples were set aside from the same particles for measurement of insoluble residues and X-ray determination of calcite-dolomite ratios. The results of this work are given in Table 1.

These data show that two rock specimens, Nos. 9 and 10, produced expansions of 0.67 percent and 1.25 percent, which are greatly in excess of the 0.10 percent expansion regarded as sufficient to disrupt concrete. These expansions, however, were not unexpected since the calcite-dolomite ratios, insoluble residue contents and microscopical textural characteristics (Fig. 1) compare closely with those listed as optimum for excessive expansions. Two other specimens, Nos. 1 and 6, show expansions of 0.06 percent and 0.07 percent, and it is anticipated that they will eventually exceed 0.10 percent expansion since the minor mineralogical deviations generally have the effect of delaying or reducing only the rate of expansion to the 0.10 percent criterion.

Specimen No. 11 in Table 1 was taken from a piece of concrete from I 65 and is essentially identical to No. 9, as seen in the table and Fig. 2. However, no significant length change was recorded at 28 days, and considering the optimum criteria, the specimen should have shown excessive expansions. Possibly, much of its potential reactivity was consumed in the concrete, as suggested by the observed reaction rim and internal micro-cracking, thus rendering it relatively insensitive to further immersion in alkaline solutions.

Review of test data reported by the State of Kentucky, Department of Highways at the time of construction and reproduced in part as Table 2, also indicate that several ledges in the quarry may fall in the category where excessive expansions resulting from alkali reactivity would be expected. As shown by recalculated dolomite and insoluble residue contents, Nos. 34448, 34449, 34450 and 41286 are close to optimum conditions for expansion in alkali. Information on microtextural characteristics and mineralogy of the insoluble residues were not available; however, if only rock of the types listed in Table 2 was used in the concrete, Fig. 2 would indicate that the required microtexture is also present in at least one of the approved ledges.

PORTLAND CEMENT ASSOCIATION

Paul Klieger

December 19, 1967

The significance of the results reported herein lies in the fact that certain ledges in the Hoover quarry contain potentially deleteriously reactive carbonate rocks, and that this type of reactivity is a strong possibility as the cause of the abnormal map cracking observed in Interstate 65. It is thus recommended that ledge by ledge testing be done in this quarry to determine exactly where the questionable rock is located, so that it may be excluded from use as aggregate for future construction.

Check this out

David Stark

DAVID STARK

D.S.*jd

TABLE 1 - SUMMARY OF DATA ON AGGREGATE SAMPLE

BCA No.	% Dolomite of Total CO_3	% Insol. Residue	% Expansion 28 Days	Rock Description: Macroscopic	Rock Description: Microscopic**
1	22	17.9	0.06	Brown to gray, fine grained; distinct clay or shale seams.	Fine to coarse calcite with occasional clay and dolomite interspersed with thin clay seams containing quartz, pyrite and dolomite. Calcite areas very dense.
2	<10	5.4	-0.03	Brown, fossiliferous, fine grained, blocky.	Pure, very dense, fine to coarse calcite. Small quartz grains scattered throughout. Little or no clay.
3	19	1.2	-0.04	Tan to light gray, coarse grained, distinct bedding.	Very coarse, dense calcite with occasional dolomite. Very little quartz or clay. Pores large but isolated.
4	<10	7.9	-0.05	Brown, fossiliferous, fine to medium grained, blocky.	Fine to coarse, dense calcite with quartz scattered throughout. Minor clay and pyrite.
5	<10	29.0	-0.05	Brown, fossiliferous, coarse grained, distinct bedding.	Very coarse, dense calcite containing relatively large areas of quartz. Minor clay and pyrite occur in scattered fine, porous seams.
6	27	9.5	0.07	Brown to dark gray, fine grained, fossiliferous, blocky.	Fine to coarse, dense calcite with occasional dolomite. Small quartz and pyrite scattered throughout. Minor clay in small isolated areas.

(Cont'd.)

TABLE 1 - SUMMARY OF DATA ON AGGREGATE SAMPLE (Cont'd.)

PCA No.	% Dolomite of Total CO_3	% Insol. Residue	% Expansion 28 Days	Rock Description: Macroscopic	Rock Description: Microscopic**
7	52	33.4	-0.05	Light tan to gray, coarse to fine grained, irregular bedding, blocky.	Coarse mixture of calcite and dolomite crystals. Some isolated areas of either calcite or dolomite. Abundant quartz, minor clay. Relatively coarse pore structure.
8	<10	4.4	-0.07	Light gray, fine grained with pockets of coarse calcite, irregular bedding, conchoidal fracture.	Extremely dense, fine to coarse calcite. Quartz scattered throughout. Little or no pyrite or clay.
9	52	14.4	0.67***	Grayish brown, very fine grained, conchoidal fracture.	Mixture of fine calcite and dolomite. Abundant quartz, minor clay and pyrite
10	37	14.2	1.25	Dark gray to brown, fine grained faint bedding, conchoidal fracture.	Mixture of fine calcite and dolomite. Abundant clay throughout. Considerable quartz and pyrite.
11*	52	11.2	-0.03	Gray, fine grained, faint bedding.	Mixture of fine calcite and dolomite. Clay scattered throughout. Considerable quartz and pyrite.
Optimum conditions for expansion	50	10-20 primarily clay	>0.10	Variable	Finely despersed mixture of dolomite, microcrystalline calcite and clay.

* Rock specimen was taken from concrete sample and displayed reaction rim and internal cracking.

** The term quartz is used here to include all detectable forms of SiO_2, so as to avoid the use of such ambiguous terms as chert, opal, chalcedony, etc.

*** 14-day reading. Specimen was broken at 18 days.

Portland Cement Association
December 19, 1967

TABLE 2 - TABULATION OF QUARRY DATA AS REPORTED BY THE STATE OF KENTUCKY

Ledge*	Thick-ness	% Dolomite of Total CO_3**	% Insol. Residue **	Description of Ledges as Reported
34447	8'8"	24	10	Fine grained smooth gray limestone with calcite crystals in places. Conchoidal fracture. Top 2'4" is one bed-bottom 5' is thinner bedded with flint lenses throughout. Thin shale partings toward bottom.
34448	4'0"	47	10	Medium grained gray limestone with siliceous pockets (white spots crystals). Flint lenses throughout. Bedding distinct.
41289	1'0"	13	20	Medium grained gray-black banded limestone with siliceous crystals in places. Bedding irregular.
34449	3'3"	36	10	Medium to fine grained dark gray limestone. Conchoidal fracture. Bedding blocklike and distinct.
34450	5'0"	40	10	Fine grained light gray smooth limestone. Conchoidal fracture. Bedding irregular.
41285	3'0"	9	6	Medium grained dark brown slightly crystalline granular limestone thick bedded and tough.
41286	2'4"	52	8	Fine grained dark brown flaky brittle limestone. Conchoidal fracture. Distinct bedding.
41287	3'0"	8	6	Fine grained dull gray slightly crystalline fossiliferous limestone. Distinct bedding. Weathers light tan.
41288	12'6"	7	5	Medium to fine grained dark gray flaky crystalline fossiliferous limestone. Rough textured and tough. Thick bedded (Siliceous).

(Cont'd.)

TABLE 2 - TABULATION OF QUARRY DATA AS REPORTED BY THE STATE OF KENTUCKY (Cont'd.)

Ledge*	Thick-ness	% Dolomite of Total CO_3**	% Insol. Residue **	Description of Ledges as Reported
6504	8'6"	18	1	Medium grained gray brown limestone, holds moisture - Chert at bottom - Thick bed.
6505	3'4"	7	3	Medium to coarse grained dark gray limestone - Black flecks - Nodular chert in places - 2 beds.

* As indicated by Lab No.
** Recalculated from reported chemical analyses.

Portland Cement Association
December 19, 1967

Ls. No. 14 Rev. 1-3-55

QUARRY REPORT

Dept. of Highways, Div. of Materials *1. Open Face 2. Local Miss St. Louis* — State of Kentucky

County Simpson — Prop. Owner Willard Atchinson — Operator Hoover, Inc. — Date Sampled 9-29-65

Sampled By David L. Arnall, Phil Graves, Ronald Nugent, Geologist — Date Rechecked

Location Intersection US 31W and Ky. 1199. (4-1/2 Miles) East on Ky.1199 to quarry — Date Received 9-29-65

[illegible] site. — Date Reported 11-3-65

Physical Tests: Spec. Grav.	Soundness %	LA Wear %	Passed or Failed — Chemical Analysis: $CaCO_3$ $MgCO_3$ Insol. R_2O_3	Ledge: Lab No.	No.	Thickness	Description of Ledges
			STRIPPING (WASTE)			12'-25'	Red clay with bouldery ledges of limestone with inclusions of chert throughout. Chert nodules are size of baseball and larger.
			WASTE			2"	Black shale.
			STRIPPING WASTE *91.7 3.8 2.9 1.1*			2 1/2' 7'	Medium grained dark gray dense flaky limestone with heavy chert lenses throughout the bed. 1 Bed.
			WASTE			2"	Black shale
2.68	2.9	25.8	PASSED FOR ALL USES *80.1 9.9 9.1 0.8*	34447	1	8'8"	Fine grained smooth gray limestone with calcite crystals in places. Conchoidal fracture. Top 2'4" is one bed-bottom 5' is thinner bedded with flint lenses throughout. Thin shale partings toward bottom.
2.70	2.2	22.2	PASSED FOR ALL USES *70.4 19.3 9.3 0.8*	34448	2	4'	Medium grained gray limestone with siliceous pockets (white spots crystals). Flint lenses throughout. Bedding distinct.
2.65	10.9	26.1	PASSED FOR ALL USES *72.8 4.6 17.5 2.2*	41289	3	1'	Medium grained gray-black banded limestone with siliceous crystals in places. Bedding irregular.
2.71	1.7	27.8	PASSED FOR ALL USES *74.9 14.9 9.2 0.9*	34449	4	3'3"	Medium to fine grained dark gray limestone. Conchoidal fracture. Bedding blocklike and distinct.
			WASTE			1"	Black shale
2.73	13.6	28.6	PASSED FOR ALL USES *73.5 16.4 9.3 0.9*	34450	5	5'	Fine grained light gray smooth limestone. Conchoidal fracture. Bedding irregular.
			WASTE			1" - 2"	Black shale
2.71	1.0	20.7	PASSED FOR ALL USES *90.2 3.8 5.6 0.5*	41285	6	3'	Medium grained dark brown slightly crystalline granular limestone thick bedded and tough.
			Shaly limestone			2"	Shaly limestone.

Copies To: Const-Dist 3(1)
Co(1)-Arnall(4)-Mays(1)
1w

Remarks: ✓RED.....Indicates Failed Ledge & Waste(Shale,Dirt,etc)
✓GREEN...Indicates Base Course and Concrete
✓YELLOW..Indicates Traffic Bound Stone (#610's ONLY)

Respectfully Submitted
Harold Gene Mays
Director of Materials

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Lab. No. 14 Rev. 1-3-55

QUARRY REPORT

Dept. of Highways, Div. of Materials

State of Kentucky

County Simpson Prop. Owner Willard Atchinson Operator Hoover, Inc.

Sampled By Arnall, Graves, Nugent - Geologist

Location Intersection US 31W and Ky.1199.

Date Sampled 9-29-65

Date Rechecked

Date Received 9-29-65

Date Reported 11-3-65

Physical Tests			Passed or Failed / Chemical Analysis					Ledge			Description of Ledges
Spec. Grav.	Soundness %	LA Wear %		$CaCO_3$	$MgCO_3$	Insol.	R_2O_3	Lab No.	No.	Thickness	
2.71	1.6	17.1	PASSED FOR ALL USES	66.8	20.9	7.9	1.6	41286	7	2'4"	Fine grained dark brown flaky brittle limestone. Conchoidal fracture. Distinct bedding.
2.67	5.8	25.8	PASSED FOR ALL USES	90.5	3.4	5.3	0.6	41287	8	3'	Fine grained dull gray slightly crystalline fossiliferous limestone. Distinct bedding. Weathers light tan.
2.67	2.7	31.4	PASSED FOR ALL USES	90.9	2.9	4.8	0.6	41288	9	12½'	Medium to fine grained dark gray flaky crystalline fossiliferous limestone. Rough textured and tough. Thick bedded(Siliceous)
------			MAIN FLOOR 7-27-65					VERY IRREGULAR			--------
2.54	1.0	33.4	PASSED FOR ALL USES	90.4	8.2	1.2	1.1	6504	10	8'6"	Medium grained gray brown limestone, holds moisture - chert at bottom - thick bed.
2.69	1.0	32.2	PASSED FOR ALL USES	88.6	3.1	2.0	1.2	6505	11	3'4"	Medium to coarse grained dark gray limestone-black flecks - Nodular chert in places - 2 beds.
	BENCH		IRREGULAR								
2.62	2.1	30.9	PASSED FOR ALL USES	78.7	13.4	2.9	0.9	9673	12	8'6"	Medium grained gray brown crystalline thick bedded porous limestone.
2.61	2.2	27.8	PASSED FOR ALL USES	87.1	9.9	4.5	1.2	9674	13	6'6"	Medium grained gray brown thick bedded limestone.
2.67	5.6	23.7	PASSED FOR ALL USES	83.9	2.9	9.5	1.1	12673	14	1'3"	Medium grained brown to black laminated limestone, shaly at top 2".
2.65	1.2	29.1	PASSED FOR ALL USES	90.2	3.8	5.3	0.8	12674	15	7'0"	Fine to medium grained gray smooth at top limestone color and texture varies. Flinty spots.
			IRREGULAR FLOOR - WATER								
2.70	8.5	23.0	PASSED FOR ALL USES	[illegible]	6.2	10.3	0.8	21172	16	3'6"	Fine to medium grained dark gray brown limestone - Flinty - Gypsum pockets.
2.69	9.6	27.1	PASSED FOR ALL USES	77.5	14.2	6.6	0.7	21173	17	12'6"	Medium grained gray thick bedded flinty limestone. 2 beds. black specks throughout.
			LOWEST POINT								

REMARKS CONTINUED ON FOLLOWING PAGE

Copies To:

1w

Remarks:

Respectfully Submitted

Harold Gene Mays

Director of Materials

Lab. No. 14 Rev. 1-3-55

QUARRY REPORT

Dept. of Highways, Div. of Materials — State of Kentucky

County Simpson Prop. Owner Willard Atchinson Operator Hoover, Inc.

Sampled By Arnall, Graves, Nugent, Geologist

Location Intersection US 31W and Ky.1199.

Date Sampled 9-29-65
Date Rechecked
Date Received 9-29-65
Date Reported 11-3-65

PHYSICAL TESTS			PASSED OR FAILED CHEMICAL ANALYSIS				LEDGE			DESCRIPTION OF LEDGES
Spec. Grav.	Soundness %	LA Wear %	$CaCO_3$	$MgCO_3$	Insol.	R_2O_3	Lab No.	No.	Thickness	

NOTE: THE CHERT AND FLINT FROM THIS SITE HAS BEEN CHECKED FOR 2.35 GRAVITY. NONE WAS IN THE FINISHED PRODUCT AT RELEASE OF THIS LOG, HOWEVER, IT MUST BE CHECKED PERIODICALLY.

D. L. ARNALL

REMARKS:

1. The test results indicated on this Report represents the "Ledge Rock Sample" as submitted by the Geologist for Approval of Source. Any change in quality of a passing ledge - Ledge becoming Soft, Dirt Stained, etc., will be a basis for rejecting the finished product.
2. Material crushed for the Department of Highways must be from "Passing Ledges" and the finished product must meet specification requirements pertaining to gradation, shale, chert, dirt, etc.
3. Please keep this quarry report at the quarry.

Ledges 1, 2, 4, & 5... Sampled 4-19-65
Ledges 3, 6, & 7......... Sampled 5-27-65
Ledges 8 & 9........... Sampled 6-1-65
Ledges 10 & 11 .... Sampled 7-27-65
Ledges 12 & 13 .... Sampled 8-12-65
Ledges 14 & 15 .... Sampled 8-24-65
Ledges 16 & 17 .... Sampled 9-29-65

Copies To: 1w

Remarks:

Respectfully Submitted
Harold Gene Mays

APPENDIX C

Division of Materials' Quarry Report

Lab. No. 14 Rev. 1-3-55

QUARRY REPORT

Dept. of Highways, Div. of Materials

State of Kentucky

County Simpson Prop. Owner Willard Atchinson Operator Hoover, Inc.

Date Sampled 9-29-65

Date Rechecked

Sampled By David L. Arnall, Phil Graves, Ronald Nugent, Geologist

Date Received 9-29-65

Location Intersection US 31W and Ky. 1199. (4-1/2 Miles) East on Ky.1199 to quarry site.

Date Reported 11-3-65

PHYSICAL TESTS Spec. Grav.	Sound-ness %	LA Wear %	PASSED OR FAILED CHEMICAL ANALYSIS ($CaCO_3$, $MgCO_3$, Insol., R_2O_3)	LEDGE Lab No.	No.	Thickness	DESCRIPTION OF LEDGES
			STRIPPING (WASTE)			12'-25'	Red clay with bouldery ledges of limestone with inclusions of chert throughout. Chert nodules are size of baseball and larger.
			WASTE			2"	Black shale.
			STRIPPING WASTE			2 1/2' 7'	Medium grained dark gray dense flaky limestone with heavy chert lenses throughout the bed. 1 Bed.
			WASTE			2"	Black shale
2.68	2.9	25.8	PASSED FOR ALL USES	34447	1	8'8"	Fine grained smooth gray limestone with calcite crystals in places. Conchoidal fracture. Top 2'4" is one bed-bottom 5' is thinner bedded with flint lenses throughout. Thin shale partings toward bottom.
2.70	2.2	22.2	PASSED FOR ALL USES	34448	2	4'	Medium grained gray limestone with siliceous pockets (white spots crystals). Flint lenses throughout. Bedding distinct.
2.65	10.9	26.1	PASSED FOR ALL USES	41289	3	1'	Medium grained gray-black banded limestone with siliceous crystals in places. Bedding irregular.
2.71	1.7	27.8	PASSED FOR ALL USES	34449	4	3'3"	Medium to fine grained dark gray limestone. Conchoidal fracture. Bedding blocklike and distinct.
			WASTE			1"	Black shale
2.73	13.6	28.6	PASSED FOR ALL USES	34450	5	5'	Fine grained light gray smooth limestone. Conchoidal fracture. Bedding irregular.
			WASTE			1" - 2"	Black shale
2.71	1.0	20.7	PASSED FOR ALL USES	41285	6	3'	Medium grained dark brown slightly crystalline granular limestone thick bedded and tough.
			Shaly limestone			2"	Shaly limestone.

Copies To: Const-Dist-3(3)
Co(1)-Arnall(4)-Mays(1)
1w

Remarks: RED.....Indicates Failed Ledge & Waste(Shale, Dirt etc.)
GREEN...Indicates Base Course and Concrete
YELLOW..Indicates Traffic Bound Stone (#610's ONLY)

Respectfully Submitted
Harold Gene Mays
Harold Gene Mays
Director of Materials

Lab. No. 14 Rev. 1-3-55

QUARRY REPORT

Dept. of Highways, Div. of Materials

State of Kentucky

County Simpson Prop. Owner Willard Atchinson Operator Hoover, Inc.
Sampled By Arnall, Graves, Nugent - Geologist
Location Intersection US 31W and Ky.1199.

Date Sampled 9-29-65
Date Rechecked
Date Received 9-29-65
Date Reported 11-3-65

Physical Tests: Spec. Grav.	Physical Tests: Soundness %	Physical Tests: LA Wear %	Passed or Failed — Chemical Analysis ($CaCO_3$, $MgCO_3$, Insol., R_2O_3)	Ledge: Lab No.	Ledge: No.	Ledge: Thickness	Description of Ledges
2.71	1.6	17.1	PASSED FOR ALL USES	41286	7	2'4"	Fine grained dark brown flaky brittle limestone. Conchoidal fracture. Distinct bedding.
2.67	5.8	25.8	PASSED FOR ALL USES	41287	8	3'	Fine grained dull gray slightly crystalline fossiliferous limestone. Distinct bedding. Weathers light tan.
2.67	2.7	31.4	PASSED FOR ALL USES	41288	9	12½'	Medium to fine grained dark gray flaky crystalline fossiliferous limestone. Rough textured and tough. Thick bedded(Siliceous)
------			MAIN FLOOR 7-27-65	VERY IRREGULAR			-------
2.54	1.0	33.4	PASSED FOR ALL USES	6504	10	8'6"	Medium grained gray brown limestone, holds moisture - chert at bottom - thick bed.
2.69	1.0	32.2	PASSED FOR ALL USES	6505	11	3'4"	Medium to coarse grained dark gray limestone-black flecks - Nodular chert in places - 2 beds.
	BENCH		IRREGULAR				
2.62	2.1	30.9	PASSED FOR ALL USES	9673	12	8'6"	Medium grained gray brown crystalline thick bedded porous limestone.
2.61	2.2	27.8	PASSED FOR ALL USES	9674	13	6'6"	Medium grained gray brown thick bedded limestone.
2.67	5.6	23.7	PASSED FOR ALL USES	12673	14	1'3"	Medium grained brown to black laminated limestone, shaly at top 2".
2.65	1.2	29.1	PASSED FOR ALL USES	12674	15	7'0"	Fine to medium grained gray smooth at top limestone color and texture varies. Flinty spots
			IRREGULAR FLOOR - WATER				
2.70	8.5	23.0	PASSED FOR ALL USES	21172	16	3'6"	Fine to medium grained dark gray brown limestone - Flinty - Gypsum pockets.
2.69	9.6	27.1	PASSED FOR ALL USES	21173	17	12'6"	Medium grained gray thick bedded flinty limestone, 2 beds, black specks throughout.
			LOWEST POINT				
			REMARKS CONTINUED ON FOLLOWING PAGE				

REMARKS CONTINUED ON FOLLOWING PAGE

Copies To: 1w

Remarks:

Originally Signed
Harold Gene Mays
Harold Gene Mays
Director of Materials

Lab. No. 14 Rev. 1-3-55

QUARRY REPORT

Dept. of Highways, Div. of Materials

State of Kentucky

County Simpson Prop. Owner Willard Atchinson Operator Hoover, Inc.

Date Sampled 9-29-65

Date Rechecked

Sampled By Arnall, Graves, Nugent, Geologist

Date Received 9-29-65

Location Intersection US 31W and Ky. 1199.

Date Reported 11-3-65

PHYSICAL TESTS			PASSED OR FAILED CHEMICAL ANALYSIS				LEDGE			DESCRIPTION OF LEDGES
Spec. Grav.	Soundness %	LA Wear %	$CaCO_3$	$MgCO_3$	Insol.	R_2O_3	Lab No.	No.	Thickness	

NOTE: THE CHERT AND FLINT FROM THIS SITE HAS BEEN CHECKED FOR 2.35 GRAVITY. NONE WAS IN THE FINISHED PRODUCT AT RELEASE OF THIS LOG, HOWEVER, IT MUST BE CHECKED PERIODICALLY.

D. L. ARNALL

REMARKS:

1. The test results indicated on this Report represents the "Ledge Rock Sample" as submitted by the Geologist for Approval of Source. Any change in quality of a passing ledge - Ledge becoming Soft, Dirt Stained, etc., will be a basis for rejecting the finished product.
2. Material crushed for the Department of Highways must be from "Passing Ledges" and the finished product must meet specification requirements pertaining to gradation, shale, chert, dirt, etc.
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Ledges 1, 2, 4, & 5... Sampled 4-19-65
Ledges 3, 6, & 7......... Sampled 5-27-65
Ledges 8 & 9............. Sampled 6-1-65
Ledges 10 & 11 .... Sampled 7-27-65
Ledges 12 & 13 .... Sampled 8-12-65
Ledges 14 & 15 .... Sampled 8-24-65
Ledges 16 & 17 .... Sampled 9-29-65

Copies To:

lw

Remarks:

Respectfully Submitted

Harold Gene Mays

Harold Gene Mays

Director of Materials

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APPENDIX D

Quarry Report from *Kentucky Aggregates*

Division of Materials
1959

(Southern Stone Company Quarry)

COUNTY: Simpson **PROPERTY OWNER:** D. H. Billingslea **OPERATOR:** Southern Stone Company
TYPE OF QUARRY: 1. Open Face 2. Commercial
LOCATION: Franklin, Ky - 6 Miles North on US 31 W. Turn left at quarry sign and go 0.7 mile to quarry.
DATE LAST CHECKED: 18 March 1958

PHYSICAL TESTS			RESULTS	LEDGE		REMARKS	CHEMICAL ANALYSIS			
Sp Gr	Sound	%Wear		No.	Thickness		$CaCO_3$	$MgCO_3$	Insol	R_2O_3
			STRIPPING		5"-15'	Dirt, Weathered Stone				
2.70	0.7	26.7	PASSED	1	3'	Gray, Smooth	91.2	5.2	3.2	0.4
2.71	1.7	25.8	PASSED	2	4'	Gray, Flaky	86.8	7.6	5.1	0.4
			BENCH		2"-3"	Shale				
2.72	0.5	28.4	PASSED	3	9 1/2'	Brown, Flaky	91.2	6.6	1.7	0.3
2.61	72.7	43.9	FAILED	4	5'-6'	Gray, Oolitic	95.8	2.2	1.4	0.4
2.65	1.5	32.2	PASSED	5	4'-5'	Gray, Oolitic, Crystalline	94.0	3.2	2.2	0.4
			BENCH		5"	Clay and Shale				
2.72	3.0	25.8	PASSED	6	4'-6'	Brown, Crystalline	72.2	17.4	7.1	1.1
2.70	3.6	28.7	PASSED	7	3'	Brown, Crystalline Oolitic	96.3	0.6	1.9	0.5
2.69	3.0	26.1	PASSED	8	3'	Brown, Flaky, Oolitic	91.6	3.7	1.2	0.5
			WASTE			Shale				
2.70	1.3	28.2	PASSED	9	2 1/2'	Gray, Crystalline, Oolitic	91.9	5.6	1.5	0.3
			WASTE		3"-1 1/2'	Shaly Stone				
2.71	1.0	24.8	PASSED	10	6'-6 1/2'	Tan, Porous	76.6	13.9	7.9	0.8
			WASTE		1"-2"	Shale				
2.52	3.2	27.1	PASSED	11	2'-4'	Tan, Gray, Blocklike	57.4	29.9	11.1	1.4
2.70	5.9	24.4	PASSED	12	2 1/2'	Gray, Flaky	96.7	1.5	1.6	0.2
			PRESENT FLOOR							
			NOT TO BE USED		5'-6'	Gray, Soft				
			LOWEST FLOOR							