PHYSICAL STUDIES of PENNSYLVANIAN SHALE in a HIGH FILL

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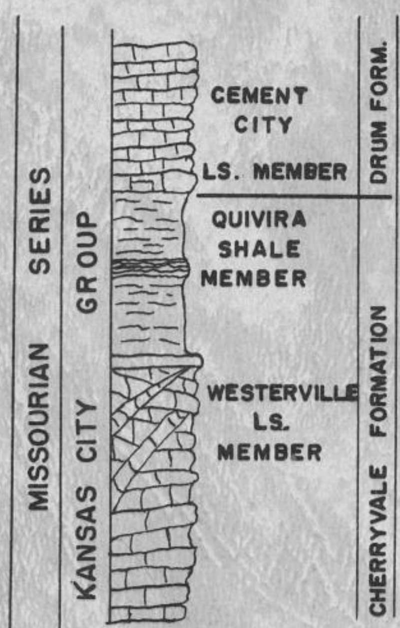

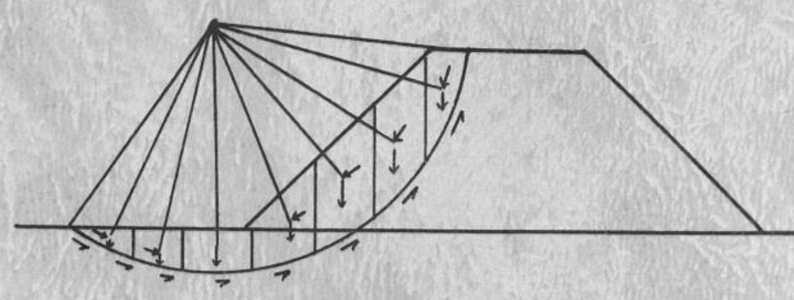
EMBANKMENT STABILITY ANALYSIS

MISSOURI STATE HIGHWAY COMMISSION

DIVISION of MATERIALS

GEOLOGY & SOILS SECTION

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Presented at the 1958 Meeting of the

American Association of State Highway Officials

December 5, 1958, San Francisco, California

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SYNOPSIS

Building Missouri highways to Interstate standards involves the design and construction of deeper cuts and higher fills than have heretofore been considered. Several such projects will be built in the Kansas City area, where the underlying formations are interbedded Pennsylvanian limestones and shales, both of which will be involved in construction. Tests were made on these shales to determine weathering, swell, settlement, and shear strength characteristics. Suggestions are made to incorporate these test results, with other information available, into design.

INTRODUCTION

In Missouri, the current requirements of straight alignment and gentle grades demand deep cuts and high fills which exceed the scope of local experience in design and construction. Many of these are in areas where Pennsylvanian Shales predominate and proposed cuts and fills may range up to 100 feet depth and height. Natural exposures give clues to weathering characteristics, natural slopes, and other information that is valuable for cut slope design. For the design of the shale fills, previous experience stops at about 25 feet height. Lack of experience on higher fills made necessary studies to give design recommendations to prevent or reduce settlement within the fills, control subgrade and slope swell, and insure safety against sliding. These studies included boring and topographic studies, limited local experience, review of the literature, and laboratory tests to determine weathering, volume change, and strength properties of the disturbed shales. This problem was brought into sharp focus when the initial soil survey for the Interstate Circumferential Route around Kansas City Missouri encountered certain Pennsylvanian Shales.

The columnar section for the shales encountered on this route is shown in Fig. 1(1)*. On future locations, undoubtedly other Pennsylvanian formations will be encountered. Then additional tests, using this study as a guide, will be made to the extent necessary to give design criteria.

These shales are described by Howe (2) as:

"<u>Chanute Formation</u>

The Chanute formation is underlain by the Cement City member of the Drum formation in this area. It includes, from the base upward, 1) light gray, silty shale ranging in thickness from less than 1 foot to as much as 3 or 4 feet, 2) silty gray and/or maroon clay and silt-stone ranging in thickness from only a foot or so to as much as 10 feet or more, and 3) overlying shale, commonly calcareous with scattered

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* Numbers in parenthesis refer to Bibliography at end of paper.

fossils, and with maximum thickness of about 10 feet. Logged thickness of the Chanute in the area indicate a range in thickness from 12 to 22 feet, the differences usually involving the presence or absence of unit 1, and variation in thickness of unit 2.

Drum formation (not described)

Cherryvale Formation

Quivira member

The Quivira member includes beds above the Westerville member of the Cherryvale formation and below the Cement City member of the Drum formation. Thickness and other characteristics of the member are complicated by distribution and thickness characteristics of the upper part of the underlying Westerville member. The Quivira includes, from the base upward, 1) argillaceous to sandy shale ranging in thickness from less than 1 foot to at least 15 feet (this shale may actually contain lenses of sandstone and locally may be maroon in color), 2) Light-colored clay (actually an underclay with coal horizon at top) with uniform thickness of about 1 foot, and 3) shale, including dark gray to black, sub-fissile portions at the base and in the middle part, with total thickness of from 4 to 6 feet. Where the underlying upper Westerville is at or near maximum thickness (about 20 feet) the Quivira includes units 2 and 3 only, and is quite uniform in thickness.

Westerville member (not described)

Wea member

The Wea member occurs between the Block limestone member below and the Westerville above. It is characteristically dark- to medium-gray, argillaceous, and very sparsely fossiliferous. In this area (1 to 2 square miles) the thickness of this member varies from less than 15 to over 30 feet, but probably averages about 20 feet. Variation in thickness does not appear to be associated with significant changes in the lithology of the Wea itself."

In this paper, Quivira #1 refers to (3) above and Quivira #2 to (2) above.

For the high degree of performance expected in an urban area, an inspection of cores, exposed slopes, and the behavior of shallow fills indicated the best solution would be to waste the shales. This, however, is impossible since there is no place to dump the surplus excavation, and little available borrow for embankment except more shale.

LABORATORY TESTING

With no experience records to serve as criteria, it was necessary to study and test the shales encountered. This paper describes these studies and their incorporation into design. The tests described are normal routine laboratory tests (for example, 90% density and optimum moisture means a sample compacted with normal laboratory control technique and not with the refinement or repetitions of basic research). A.S.T.M. (3) and A.A.S.H.O. (4) and other accepted investigative procedures (5), (6), (7), were used. Values that appeared to be anomalies or errors were usually checked.

These shales had been observed to weather rapidly, to appear to have low strength in the weathered state, and to be expansive in subgrades (8). The only test data available were index properties:

Shale	Liquid Limit	Plastic Index	Maximum* Density	Optimum* Moisture
Quivira 1	41.0	19.5	106.6	20.5
Quivira 2	48.0	25.3	106.6	18.6
Chanute	53.3	26.1	103.6	21.0
Wea	41.9	18.7	107.9	18.8

Solution of the problem of shale fill stability requires test data concerning the following, each of which will be described and discussed separately.

1. Soundness and weathering
2. Volume change
3. Consolidation
4. Structural stability

SOUNDNESS AND WEATHERING: To determine whether any of these shales would retain their inherent shape and strength characteristics if they were not thoroughly broken down before inclusion in the embankment, a variation of the magnesium sulfate soundness test as suggested by Welch (9) was used to subject the three shales to accelerated testing. The Chanute shale could not be prepared to the specified size (100 gm. cubes) because of laminations. The small pieces were tested to observe disintegration, if any. All three shales showed complete failure on the first cycle, indicating that weathering would be severe.

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* In this report, for brevity MD is used for maximum density and OM is used for optimum moisture.

It was desired to develop a preparation method which would most nearly simulate natural field weathering. The only available materials for comparison purposes were composite samples from the weathered surfaces of shales of the same three formations. While it was realized that these would not entirely represent the weathered condition of the considered shales when used in high fills, they were still the best available criteria for comparison.

The various methods of preparation for the data in Table I are listed below:

Weathered in Field - Tests on the samples obtained by gathering a composite of the thin weathered surfaces.

CNL(Tests) - Samples submitted from the district for initial testing. Three cycles - wetting and drying. Then ground to pass #4 sieve. The values on density and moisture from this series were used for compaction of the other test specimens.

A1 - 5 cycles - wetting and drying. Then ground to pass #4 sieve.

A2A - 1 cycle - wetting-freezing-thawing-drying. Then ground to pass #4 sieve.

A2B - 5 cycles - wetting-freezing-thawing-drying. Then ground to pass #4 sieve.

A3A - Dry. Then ground to pass #4 sieve.

A3B - Dry. Then ground to pass #10 sieve.

A3C - Dry. Then ground to pass #40 sieve.

A3D - Dry. Then ground to pass #200 sieve.

A4 - Dry grinding to as fine as burr mill would grind.

As shown in the table, the fact that these shales are soft and readily broken (in the laboratory, at least) made almost any method of preparation satisfactory.

VOLUME CHANGE: A preliminary estimate of the swelling properties of these shales was made by the "Free Swell" test as used by the Bureau of Reclamation (10), giving average values of 70% for Quivira #1, 95% for Quivira #2, 90% for Chanute, and 68% for Wea. These values, considered along with shrinkage limit and shrinkage ratio values from Table I, indicated that further tests were needed.

Figure 2 shows the results of the swell tests (Reference #2, p. 280) conducted with the standard minimum restraining load of porous stone and loading piston. Figure 3 shows the swell data for specimens compacted to 90% MD at OM but loaded to the equivalent of the weight of pavement and base plus increments of fill up to 15 feet.

Examination of these data, with the proper interpolation, leads to the following indications concerning their swelling properties:

1. Vertical swell will be negligible under loading greater than 10 feet of fill plus pavement and base.

2. Objectionable swell will occur if the shales are compacted to high densities unless restrained. This swell tendency will be particularly severe if compaction is on the dry side of optimum moisture. For Chanute shale at 100% MD and OM, computations interpolated from Figs. 2 and 3 indicate a swell of about 7" in the upper 10 foot layer with the top 3 feet contributing approximately half of the swell.

3. From a consideration of the swelling properties alone it would be advisable to build the top ten feet of each fill with 90% density and the highest workable moisture. The outer faces of each side slope would come under this same consideration.

4. Figures 2 and 3 indicate that, for any specified moisture-density relationship, swelling will be largely restrained at depths below 10 feet by the load of the overlying shale fill.

CONSOLIDATION: Consolidation tests were desirable to estimate magnitude of settlement within the fill proper and to consider pore pressure effects on stability of higher fills. Preliminary tests were run on each shale in floating ring consolidometers on 3/4 inch thick by 2-1/2 inch diameter specimens under three conditions.

Condition 1: Samples were statically compacted to nominal 90% standard density at optimum moisture, then sealed in humid condition and loaded in a normal cycle of 1/8, 1/4, 1/2, 1, 2, 4, and 6 kips/sq.ft. taking routine time - consolidation data. Then the seal was slit and water was added to saturate the bottom porous stone and the sample was allowed to take on water by capillarity to equilibrium (Plotted as a dotted line in Figs. 5, 9, 15, and 17.)

Condition 2: Samples were statically compacted to nominal 90% density at optimum moisture, the consolidometer was sealed in humid condition, loaded in one increment to 6 kips/sq.ft. and deformation was measured after consolidation ceased. Then the seal was slit and the bottom stone was saturated and the sample was allowed to take on water by capillarity.

Condition 3: Samples were statically compacted to nominal 90% standard density at optimum moisture and conventional "submerged, restrained swell" consolidation tests were made with double increment loads to 32 kips/sq.ft.

Numerous fills built of these shales will be 50 to 60 feet in height, giving an average weight on the bottom layer of about 6 kips/sq.ft. For this load, these results were obtained:

		Type Shale			
	Condition	Quiv. 1	Quiv. 2	Wea	Chanute
Total change in void ratio before capillary wetting	1	.242	.186	.183	.221
	2	.234	.201	.177	.211
	3	-	-	-	-
Total change in void ratio after capillary wetting	1	.244	.190	.183	.224
	2	.235	.204	.179	.212
	3	.200	.225	.190	.241

These pilot tests indicated that:

1. Samples compacted to 90% MD at OM and loaded before inundating as in Condition 1 compressed almost instantaneously, but at higher loadings the time required followed a somewhat normal time-consolidation curve. For these shales, compacted at nominal 90% maximum density at optimum moisture, this change usually occurred at from 1 to 2 kips/sq.ft. loading, indicating that the early volume change was caused by compression of gases. As soon as volumes were reduced to the point where compaction moisture was near saturation moisture, normal consolidation behavior occurred.

2. No additional consolidation can be attributed to introducing capillary water for Conditions 1 and 2. Probably the very small change would have occurred as creep under continued loading whether or not capillarity had been permitted.

3. The difference in consolidation for the three conditions was small enough to warrant selection of Condition 3, the conventional tests, as standard procedure. All three methods might have been desirable but available time and the training of personnel made conventional tests more desirable.

Conventional "submerged, restrained swell" tests were then run on all four shales at these density-moisture relations:

1. 85% Density
   a. Optimum moisture minus 5%
   b. Optimum moisture
   c. Computed saturation
2. 90% Density
   a. Optimum moisture minus 5%
   b. Optimum moisture
   c. Computed saturation
3. 95% Density
   a. Optimum moisture minus 5%
   b. Optimum moisture
   c. Computed saturation
4. 100% Density
   a. Optimum moisture minus 5%
   b. Optimum moisture
   c. Computed saturation

Figures 4 to 19 are conventional void ratio, e, vs. log of pressure, p, curves of test results. Figures 20 to 23 are plots of the worst anticipated settlement in any 10' layer for the height of fill above that layer. The total settlement would be the summation of the settlement in all 10 foot layers.

For instance, comparing Quivira #1 in a fill 65 feet high compacted to 90% MD at OM with the same height fill compacted to 100% MD at OM, these data are tabulated from Figure 21:

Layer	Depth to Midpoint of Layer	Settlement @ 90% MD and OM	@ 100% MD and OM
65-55	60	1.16	.18
55-45	50	1.06	.14
45-35	40	.96	.12
35-25	30	.86	.08
25-15	20	.58	.06
15- 5	10	.28	.04
		4.90	0.62

While the consolidation properties of these shales are distinct enough to require individual consideration there are characteristics that are somewhat similar as:

1. Lower densities give much higher consolidation.

2. For a given density, a low compacting moisture content causes greater settlement.

3. The compacting moisture content is less critical at higher densities than low within the range OM-5 to saturation.

4. For fills of considerable height, higher densities will be required if excessive settlement is to be avoided.

5. While c_v values for remolded soils at less than saturation are somewhat erratic those computed for these shales indicate a trend toward slower rates of consolidation with increased amount of consolidation. The coefficient of consolidation, c_v, values are plotted in solid symbols on the void ratio, e, vs. log of pressure, p, curves, Figs. 4 - 19. Also, plots of deformation vs. time indicate considerable time would be required to complete settlement.

6. For these remolded specimens, the curves exhibited a distinct break in curvature such as is found in preconsolidated clays. The curvature break is caused by a tendency to swell at initial loadings. This tendency to swell was actually restrained in test procedure by loading and this portion of the curve is shown by dashed lines on the e-log p curves.

7. The consolidation tests show additional swell characteristics of the shales. The e-log p curves indicate large rebounds. For 100% densities the shales consolidated measureably under 32 kip/sq.ft. loading but on removal of load, expanded to more than molding volume. Fig. 24 shows these 100% MD, OM-5 specimens after rebound.

STRUCTURAL STABILITY: Many of the fills built of these shales are in areas of valuable real estate. Structural stability is a necessity in any location, however, in urban areas minimum slopes and right-of-way place an added incentive on maximum shearing strength. Shear tests were made at different moisture-density relations to determine this maximum strength (providing volume change properties are not adversely affected). The field compaction and placement should also utilize in the embankments, excavation from the rock ledges (Fig. 1), to achieve best internal drainage and maximum stability.

Shear tests were of two types:

1. Unconfined compression tests were run on statically compacted specimens, 1.4 x 2.8 inches at OM-5, optimum and computed saturation moisture contents for 85, 90, 95 and 100% of maximum density. Four individually molded specimens were

tested for each value and reruns made for any erratic results. These data are plotted as average curves in Figs. 25 - 28 and tabulated in Table II showing trends of cohesive strength. The Wea and Chanute shales had a tendency to crumble at the OM-5% condition giving lower strengths.

2. Direct shear tests were run on 2-1/2 inch diameter by 1 inch thick statically compacted specimens after soaking and consolidating under normal loads of 850, 1650, 3450, and 6925 pounds/sq.ft. The rate of shearing deformation was .001 in/sec. These data are plotted in Figs. 29 - 32.

In a complex fill, shear strength is merely one good step toward solution for stability. Stability of foundation soil, slope of natural ground surface, pore pressures (11), predicted future moisture conditions based on Missouri conditions and measurements (12), seepage into ends of fills, heterogeneity of fill mass, cracking of shoulders or slopes with subsequent wetting and hydrostatic head, erosion, vibration in industrial areas, partial inundation and drawdown, depth to water table, must all be considered. Thus, in soil structures approaching the magnitude of moderate sized earth dams, individual analysis becomes a necessity.

From cross section, boring and laboratory data, and working assumptions, a method of solution is selected to give accuracy with minimum computations.

The solutions most commonly applied are: Taylor's chart (13), Swedish method of slices (14) (solutions are run out on electronic computor if data fits computer program (15), "sliding block", stress at a point (16), (17), or combinations.

TYPICAL PROBLEM

A typical problem, somewhat simplified for brief presentation, encountered on FAI-8, Kansas City Circumferential Route, using topographic profile features, boring data, test data, and necessary assumptions for solution is:

1. Profile and topography:
   a. Fill of maximum height of 65 feet.
   b. Valley 1500 feet from cut section to cut section.
   c. Fairly uniform natural slopes. (No bluffs at edge of valley).
   d. 10 x 10' box culvert, (No bridge ends).
   e. Natural ground reasonably level under higher portion of fill.
   f. Fill 108' wide at top.
   g. Seepage from shale beds in hills.
2. Boring data indicate:
   a. Firm foundation soil.
   b. Excavation going into fill will be 70% Wea and Quivira shale and 30% rock.
   c. High water table.
3. Test data
   a. Values for this report apply.
4. Assumptions
   a. Inundation and sudden drawdown will not occur.
   b. Average values for Quivira and Wea shales permissible.
   c. Some control of rock placement possible.
   d. Estimate 30% of settlement will occur rapidly (by time fill is completed).

e. The interior lower part of the fill will be essentially saturated.

f. Pore water pressure increases will offset shear strength gains from consolidation, therefore, unconfined compression values will be used.

g. In stability analysis, no mathematical consideration is made for rock, i.e., entire fill is computed as if shale.

h. Apparent factor of safety in excess of 1.5 required for urban character of area.

i. Without restraint, side slopes of shale will expand, reducing soil strength and stability.

j. Erosion of the weathered shales must be controlled.

Applying test data, it is estimated that a 65' fill built of Quivira and Wea shale at 90% MD and OM will settle as:

Total settlement	4.25±
Less 30% rock	1.28
	2.97
Less 30% during construction	.89
Residual after construction	2.08

while at 100% MD and OM

Total settlement	0.62
Less 30% rock	.18
	0.44
Less 30% during construction	.13
Residual	0.31

For a reach of 1500' with no bridge ends, 0.31 ft. of settlement should be unnoticed. Thus 100% MD is recommended.

Stability: Using unconfined compression values, from Table II, applying stability charts (10), a maximum height of fill of 40 feet for 2:1 side slopes is indicated at 100% MD, compacted on the dry side of optimum (to alleviate pore pressures). Over 40 feet, flatter side slopes are indicated and for a 65' fill 3:1 side slopes or equivalent toe berms are indicated.

A similar check was made assuming full consolidation of fill using direct shear values. With the same assumptions a factor of safety of 2.+ is achieved.

CONCLUSIONS

The conclusions to be made from analytic solutions, from limited experience, and practices of others are:

1. Each location of a high fill should be examined closely and considered individually.

2. Construction procedures should be set up as uniformly as possible. A conservative procedure generally applicable is preferable to numerous individual non-uniform procedures which may more nearly approach theoretical correctness.

3. Heavy rollers, spike or wedge tooth, wetting and drying, and combinations of both should be used to break these shales down initially.

4. Rock and soil mantle material should be handled by selective grading and cross hauling to give:

   a. A well shattered rock blanket under high fills.

   b. No open rock embankment in the center of fills.

   c. Better material at the fill edges to reduce swell, cracking, and erosion.

d. A 5-foot layer of select soil at the top of fills for subgrade material. If suitable soil is not available, lime, cement or other treatment of the shales should be considered.

e. A 2-foot layer of select or treated soil in cuts. The cut is less serious than fill as the shale is less disturbed.

5. As initially stated, this program is an effort to extrapolate limited experience. Research studies will be undertaken during construction and through the service period of the fills to ascertain:

a. The extent and degree of actual weathering inside the fills.

b. Magnitude of settlement of different layers within the fill.

c. Moisture changes within the fills.

d. Pore pressure measurements in the fill proper.

e. Comparison of tests of the materials compacted by construction equipment against laboratory tests.

6. This approach will be beneficial when extensive studies are made of other large, well defined, soil or shale groups.

7. As a result of the data obtained from these tests, it will not be desirable to use 85% MD on future tests.

8. For the shales tested, the test results were not particularly affected by these laboratory methods of artificial weathering.

9. These shales are highly susceptible to swell. Hence the shales should be covered by rock or select soil to restrain undesirable volume change with subsequent subgrade heave and loss of stability.

10. At equal densities, these shales generally exhibit greater consolidation at low molding moisture contents. This is less true at higher densities. The amount of rebound appears somewhat constant at all molding moistures for a given density.

11. There is a large increase in unconfined compressive strength with increase in density.

12. The direct shear results are not particularly affected by initial density and molding moisture.

13. Triaxial shear tests with pore pressure measurements would have been desirable but the time required places such tests outside the scope of routine control testing.

14. While better embankment material would be desirable, satisfactory fill can be built by the selective placing of excavation, control of density and moisture, and good construction practices.

BIBLIOGRAPHY

1. Greene, F. C. and Howe, W. B., "Geologic Section of Rocks Exposed in the Kansas City Area.", Information Circular No. 8, Missouri Division of Geology and Water Resources.
2. Howe, W. B., Division of Geological Survey and Water Resources, Personal Communication, 11-17-58.
3. "Procedures for Testing Soils", American Society of Testing Materials, April, 1958.

4. "Highway Materials, Part II", 7th Edition, American Association of State Highway Officials, 1955.

5. Lambe, T. W., "Soil Testing for Engineers", John Wiley and Sons, 1951.

6. Dawson, R. F., "Laboratory Manual in Soil Mechanics", Pitman Publishing Co., 1949.

7. Casagrande, A. and Fadum, R. E., "Notes on Soil Testing for Engineering Purposes". Soil Mechanics Series No. 8, Harvard University, 1940.

8. Jenkins, G. Jr., "Soil Survey Report, Route IN-8, Project I-522(1)", Missouri State Highway Department, 1958.

9. Welch, J. D., "Rock Weathering Classification of Excavation Slopes", Proceedings, American Society of Civil Engineers, 1955.

10. Holtz, W. G. and Gibbs, H. J., "Engineering Properties of Expansive Clays", Proceedings, American Society of Civil Engineers, 1954.

11. ------, "Earth Manual", Bureau of Reclamation, 1957.

12. Guinnee, J. W. and Thomas, C. E., "Subgrade Moisture Conditions Under an Experimental Pavement", Bulletin 111, Highway Research Board, 1955

13. Taylor, D. W., "Fundamentals of Soil Mechanics", John Wiley and Sons, 1948.

14. Tschebotarioff, G. P., "Soil Mechanics, Foundations, and Earth Structures", McGraw Hill Book Co., 1951.

15. LeClerc, R. V. and Hansen, R. J., "Computer Solution of the Swedish Slip Circle Method of Analysis for Embankment Stability". Paper presented at 37th Annual Meeting of the Highway Research Board, 1958.

16. Barber, E. S., "Use of Direct Shear Tests in Highway Design", American Society of Testing Materials, 1952.

17. Spangler, M. G., "Soil Engineering", International Textbook Company, 1951.

Table I
WEATHERING TESTS

QUIVIRA #1 Specific Gravity - 2.71

Weathering Tests	O.M.	M.D.	L.L.	P.L.	P.I.	S.L.	S.R.	-#200	% Silt	% Clay	Approx. % Colloids	H.R.B. Classification & Group Index
Weathered in Field	16.4	107.7	44.3	21.3	23.0	17.8	1.77	97.6	52.0	43.0	20.0	A-7-6(14)
CNL(Tests)	20.5	106.6	41.0	21.5	19.5	15.5	1.85	99.4	43.5	47.5	20.0	A-7-6(12)
A1	19.2	106.7	42.6	21.2	21.4	16.5	1.84	97.8	51.0	44.0	22.0	A-7-6(13)
A2A	19.9	107.0	42.2	21.0	21.2	14.5	1.87	97.6	50.0	45.0	21.0	A-7-6(13)
A2B	19.0	108.6	42.7	23.2	19.5	15.4	1.85	97.8	48.0	47.0	20.0	A-7-6(12)
A3A	19.0	107.2	42.4	21.5	20.9	16.5	1.84	97.0	52.5	41.5	21.0	A-7-6(13)
A3B	19.0	107.6	41.7	20.7	21.0	16.8	1.81	98.0	51.0	44.0	21.0	A-7-6(12)
A3C	18.4	106.9	40.6	21.5	19.1	15.1	1.85	98.2	50.0	45.0	19.0	A-7-6(12)
A3D	19.3	107.2	40.3	23.6	16.7	14.2	1.90	98.4	50.0	44.0	17.0	A-7-6(11)
A4	18.8	107.3	42.2	20.5	21.7	17.4	1.82	98.4	50.5	45.0	22.0	A-7-6(13)

QUIVIRA #2 Specific Gravity - 2.71

Weathering Tests	O.M.	M.D.	L.L.	P.L.	P.I.	S.L.	S.R.	-#200	% Silt	% Clay	Approx. % Colloids	H.R.B. Classification & Group Index
Weathered in Field	No Sample											
CNL(Tests)	18.6	106.6	48.0	22.7	25.3	14.5	1.94	95.0	37.0	55.0	25.0	A-7-6(16)
A1	16.7	110.6	40.7	22.9	17.8	17.5	1.77	98.6	58.0	38.0	18.0	A-7-6(11)
A2A	18.3	108.4	39.1	21.0	18.1	17.4	1.75	97.6	55.0	39.0	18.0	A-6 (11)
A2B	18.5	107.9	39.4	21.0	18.4	16.2	1.79	93.2	49.0	43.0	18.0	A-6 (11)
A3A	16.8	110.7	40.1	21.9	18.2	23.4	1.80	84.4	48.0	29.0	18.0	A-7-6(11)
A3B	18.0	108.4	38.2	22.5	15.7	15.8	1.92	90.0	45.0	39.0	15.0	A-6 (10)
A3C	18.5	108.2	38.9	24.4	14.5	16.0	1.80	91.0	45.0	39.0	14.0	A-6 (10)
A3D	16.4	110.6	38.9	22.2	16.7	16.1	1.80	98.6	54.0	37.0	16.0	A-6 (11)
A4	17.0	109.6	39.6	18.2	21.4	15.9	1.82	97.6	55.0	34.0	22.0	A-7-6(12)

WEA Specific Gravity - 2.72

Weathering Tests	O.M.	M.D.	L.L.	P.L.	P.I.	S.L.	S.R.	-#200	% Silt	% Clay	Approx. % Colloids	H.R.B. Classification & Group Index
Weathered in Field	16.0	109.2	46.2	25.1	21.1	20.7	1.75	95.4	42.0	52.0	20.0	A-7-6(14)
CNL(Tests)	18.8	107.9	41.9	23.2	18.7	11.8	1.98	97.6	44.0	50.0	19.0	A-7-6(12)
A1	16.8	109.6	46.1	24.0	22.1	19.7	1.70	97.6	47.0	47.0	20.0	A-7-6(14)
A2A	15.8	110.6	43.1	26.3	16.8	19.8	1.75	96.8	44.0	49.0	17.0	A-7-6(11)
A2B	17.0	109.1	40.8	24.6	16.2	18.9	1.75	97.6	43.0	52.0	16.0	A-7-6(11)
A3A	16.0	113.8	51.6	25.3	26.3	15.4	1.83	94.2	47.0	42.0	20.0	A-7-6(17)
A3B	16.0	112.0	40.8	23.0	17.8	19.5	1.70	96.6	50.0	43.0	17.0	A-7-6(11)
A3C	16.0	113.2	40.7	24.8	15.9	19.1	1.75	95.4	49.0	40.0	16.0	A-7-6(10)
A3D	16.1	111.9	38.8	22.7	16.1	16.5	1.79	96.2	51.0	38.0	16.0	A-6 (10)
A4	18.0	109.5	38.4	23.6	14.8	18.2	1.77	95.0	48.0	40.0	15.0	A-6 (10)

CHANUTE Specific Gravity - 2.70

Weathering Tests	O.M.	M.D.	L.L.	P.L.	P.I.	S.L.	S.R.	-#200	% Silt	% Clay	Approx. % Colloids	H.R.B. Classification & Group Index
Weathered in Field	16.5	108.8	48.6	24.9	23.7	15.8	1.85	90.8	44.0	43.0	20.0	A-7-6(15)
CNL(Tests)	21.0	103.6	53.3	27.2	26.1	11.3	1.91	97.2	42.0	51.0	26.0	A-7-6(15)
A1	20.5	105.6	58.5	25.5	33.0	16.8	1.81	97.2	39.0	56.0	33.0	A-7-6(20)
A2A	19.5	105.7	54.6	25.1	29.5	14.5	1.89	96.6	38.0	56.0	30.0	A-7-6(19)
A2B	18.7	106.2	55.9	28.3	27.6	16.1	1.85	97.0	35.0	60.0	28.0	A-7-6(18)
A3A	17.0	107.8	43.0	24.3	18.7	18.1	1.77	95.6	39.0	52.0	19.0	A-7-6(12)
A3B	19.4	107.7	55.7	25.9	29.8	17.2	1.84	96.0	42.0	50.0	25.0	A-7-6(19)
A3C	19.5	106.6	50.4	27.8	22.6	16.1	1.83	97.6	37.0	56.0	23.0	A-7-6(15)
A3D	19.8	106.0	50.1	25.8	24.3	14.9	1.84	97.2	40.0	54.0	24.0	A-7-6(16)
A4	19.3	105.4	58.7	25.6	33.1	17.2	1.81	98.6	37.0	58.0	33.0	A-7-6(20)

Table II

UNCONFINED COMPRESSION TEST

Shale	Molding Moisture	Density 85	90	95	100
		Stress - P/2A-lbs./sq.ft.			
Chanute	OM-5	950	1350	2075	3350
	OM	1100	1500	2225	3150
	Sat.	600	1075	1900	3000
Quivira 1	OM-5	1450	2350	3650	5500
	OM	800	1550	2450	3500
	Sat.	600	1050	1650	3400*
Quivira 2	OM-5	1000	1650	2650	4350
	OM	900	1400	1950	3500
	Sat.	400	950	1250	2950
Wea	OM-5	725	1075	1700	3150
	OM	1150	1450	2000	3200
	Sat.	300	650	1350	2450

* 99% M.D.

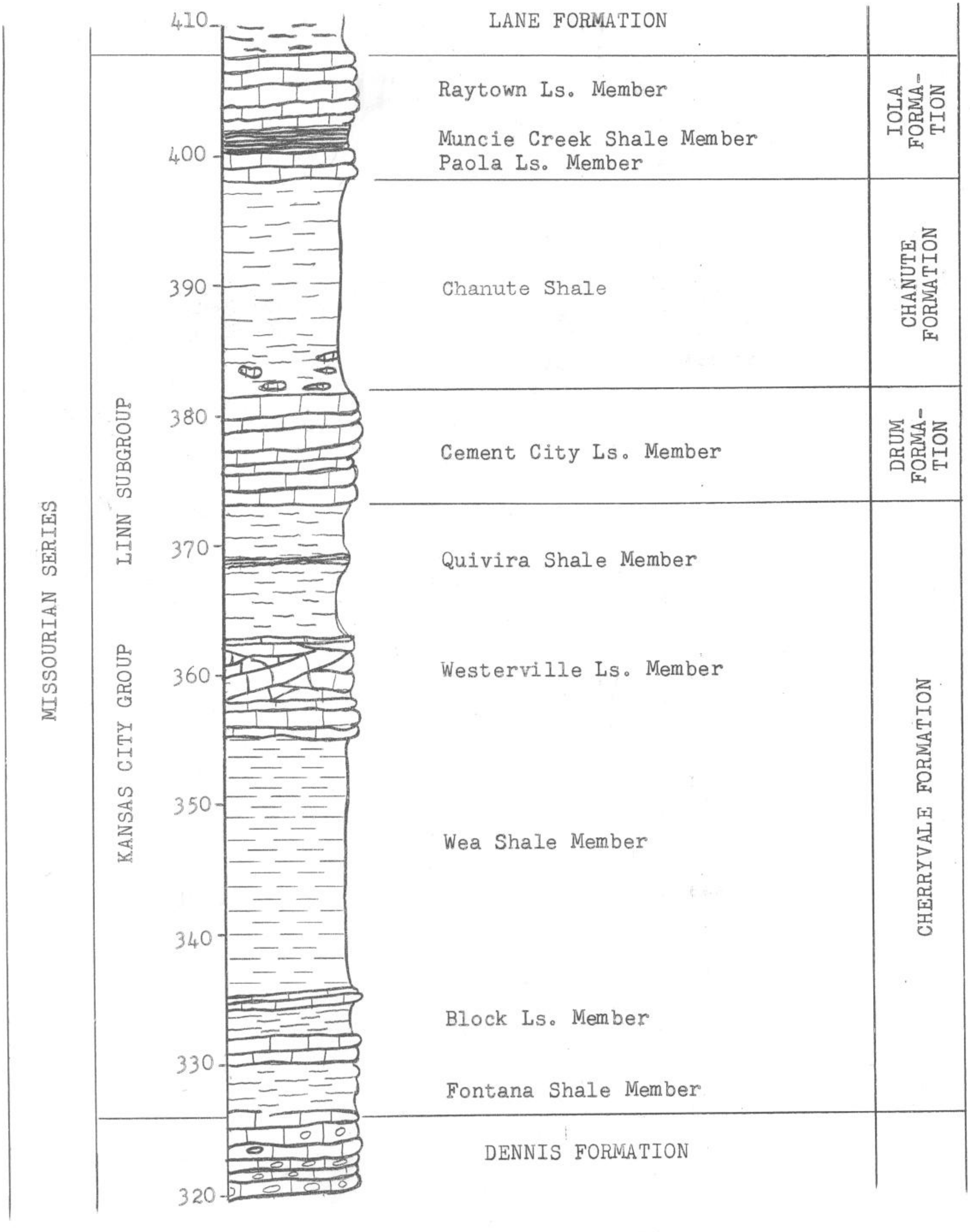


Figure No. 1 Partial Columnar Section of Kansas City Area

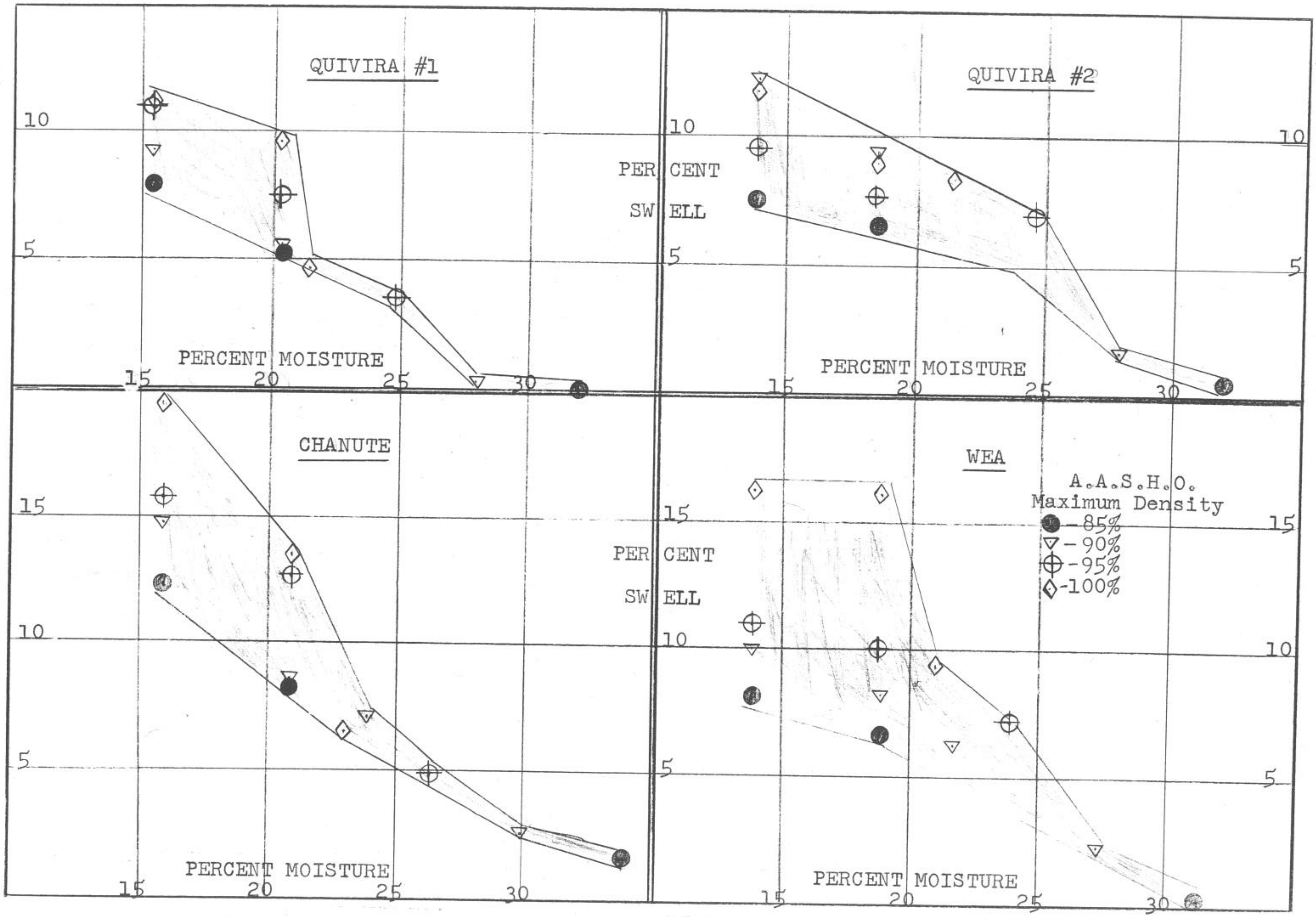


Figure No. 2 Molded Moisture Vs. Percent Swell

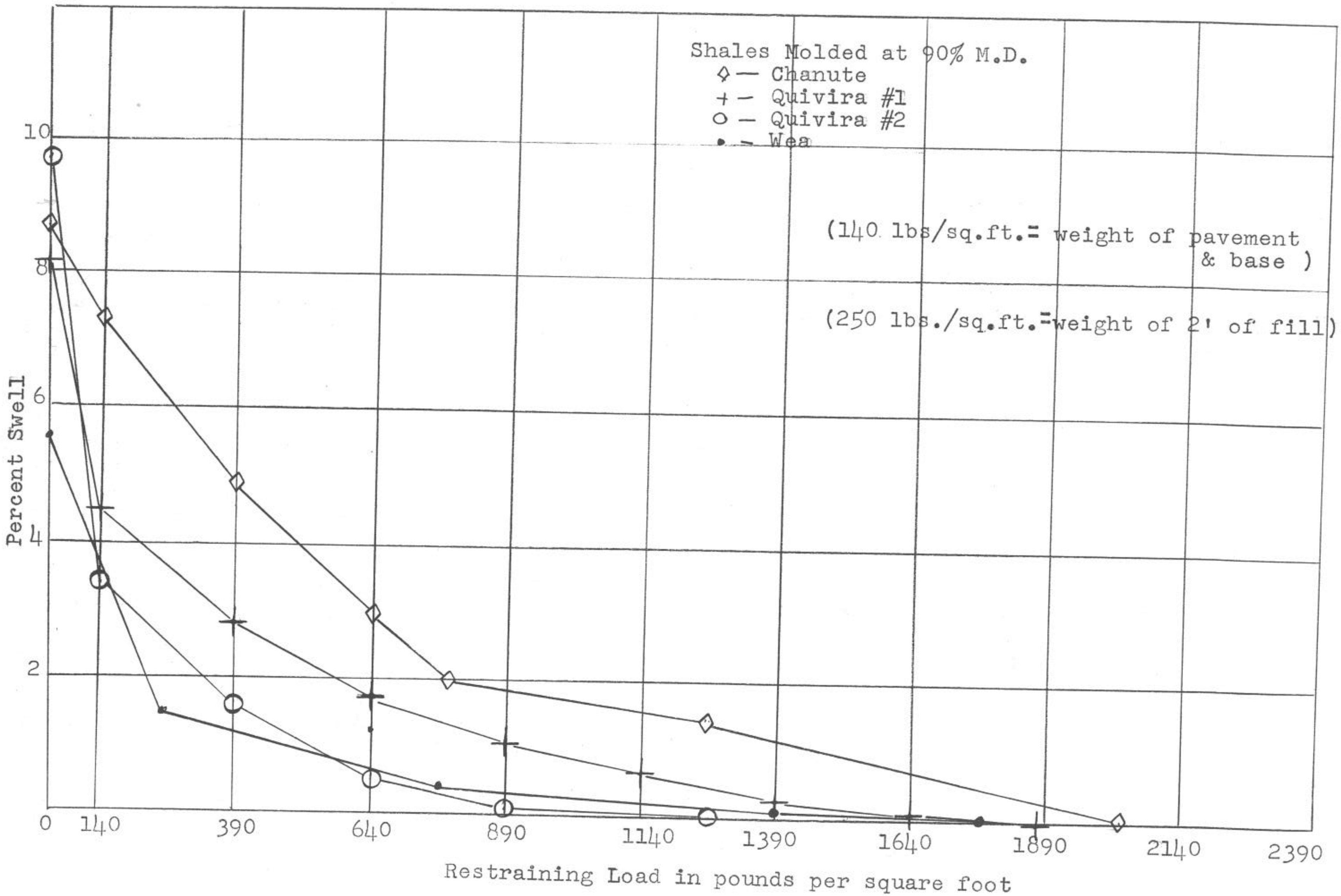


Figure No. 3 Percent Swell Vs Restraining Load

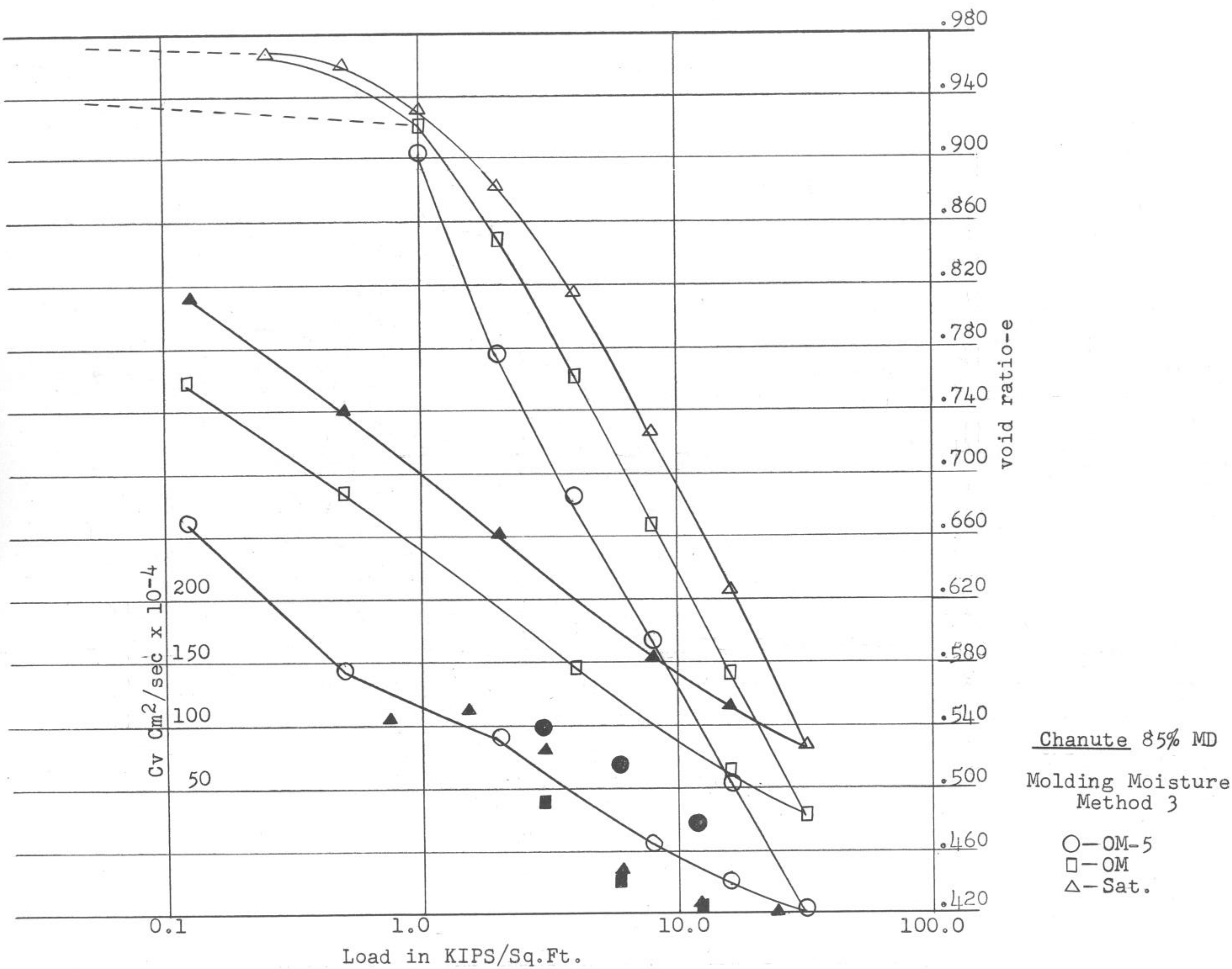


Figure No. 4 Consolidation Curve Chanute Shale at 85%M.D.

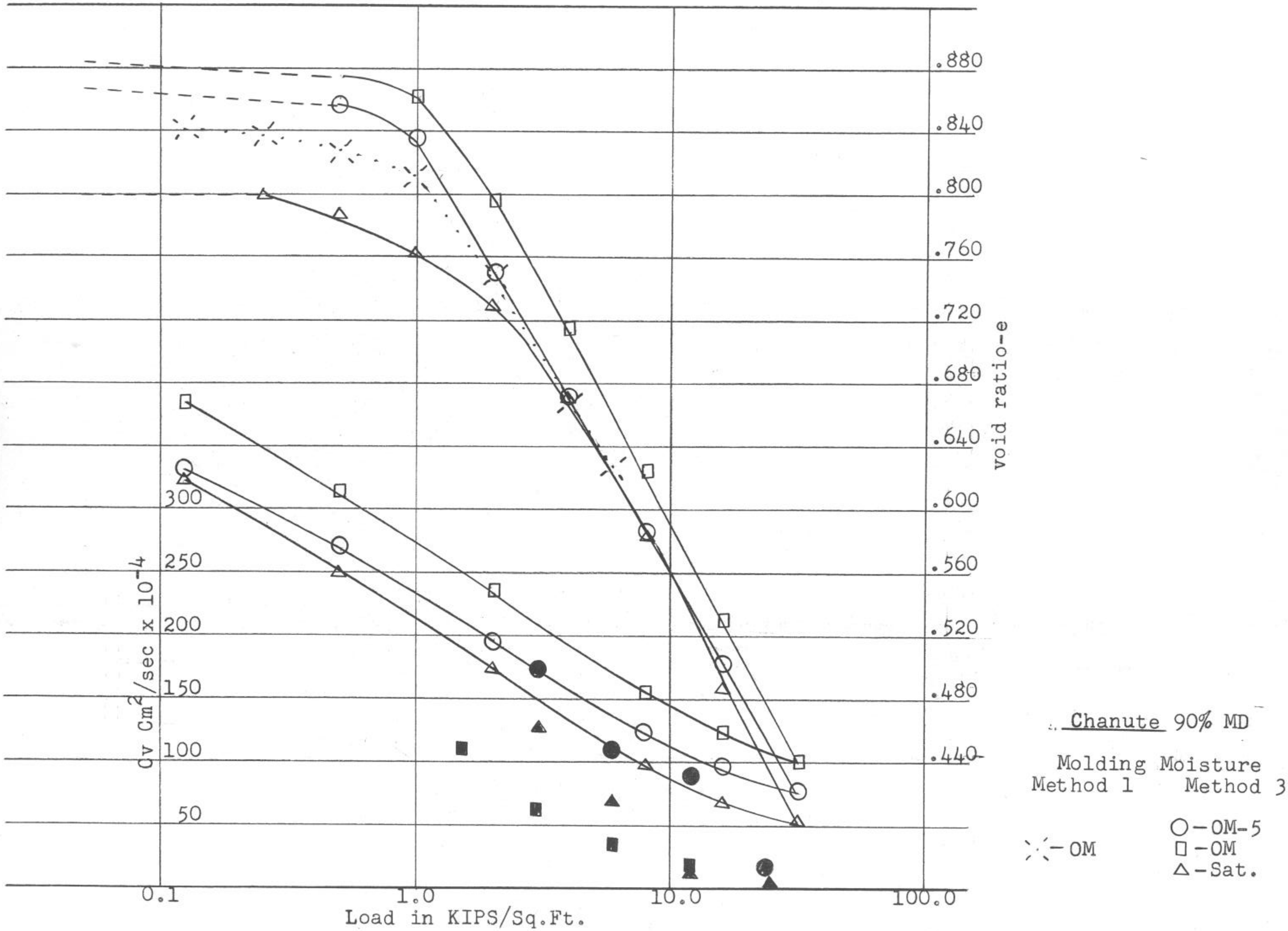


Figure No. 5 Consolidation Curve Chanute Shale at 90% M.D.

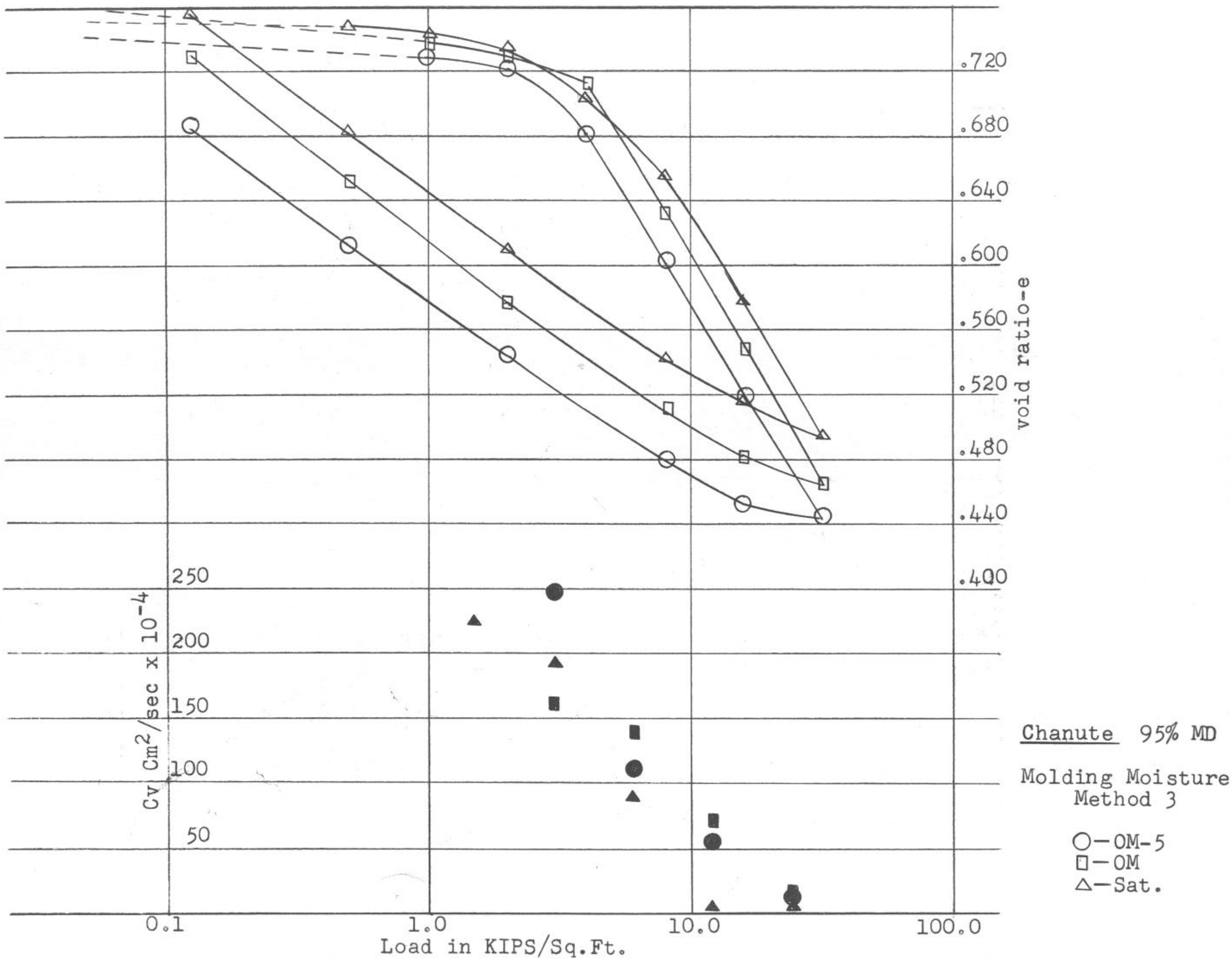


Figure No. 6 Consolidation Curve Chanute Shale at 95%M.D.

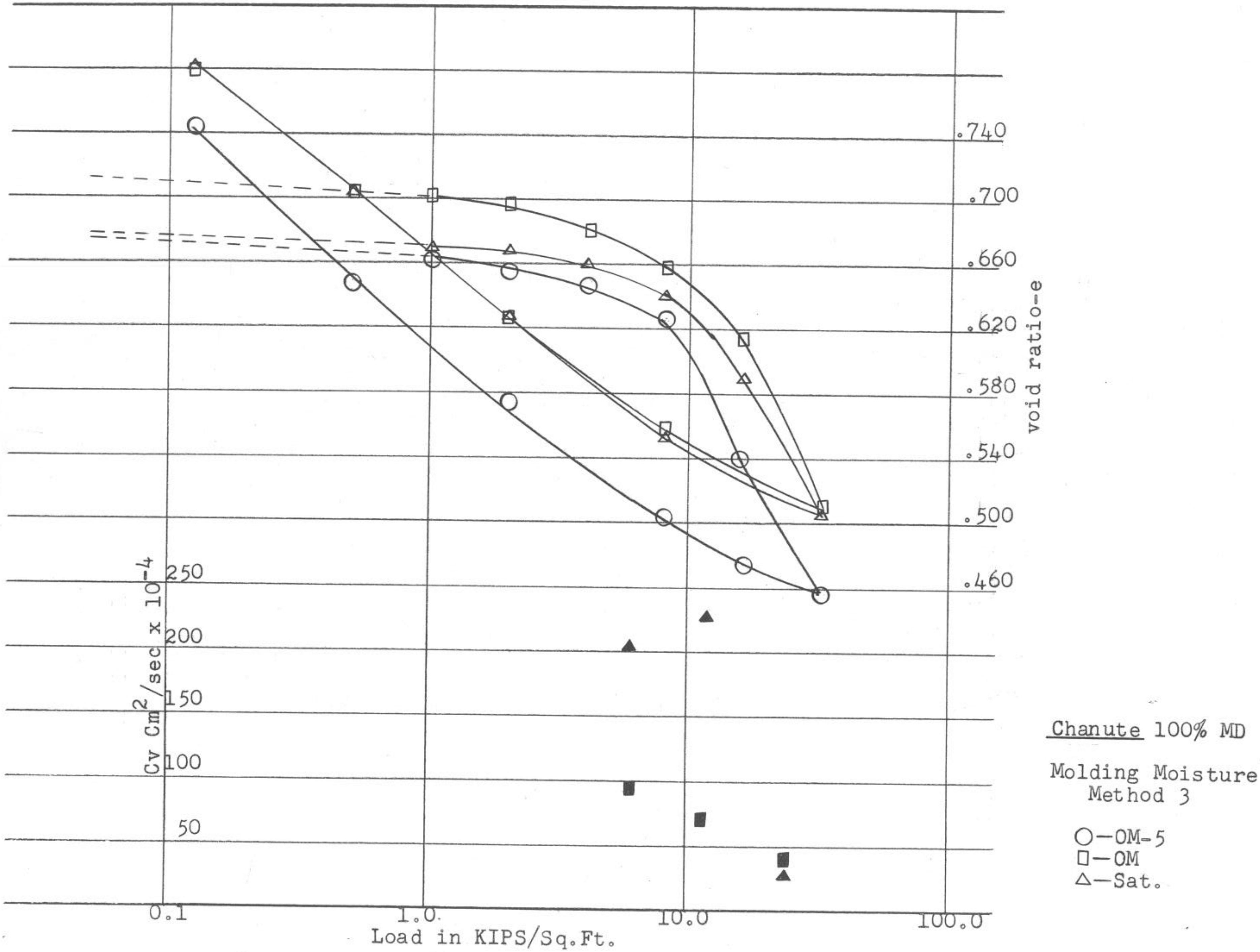


Figure No. 7 Consolidation Curve Chanute Shale at 100% M.D.

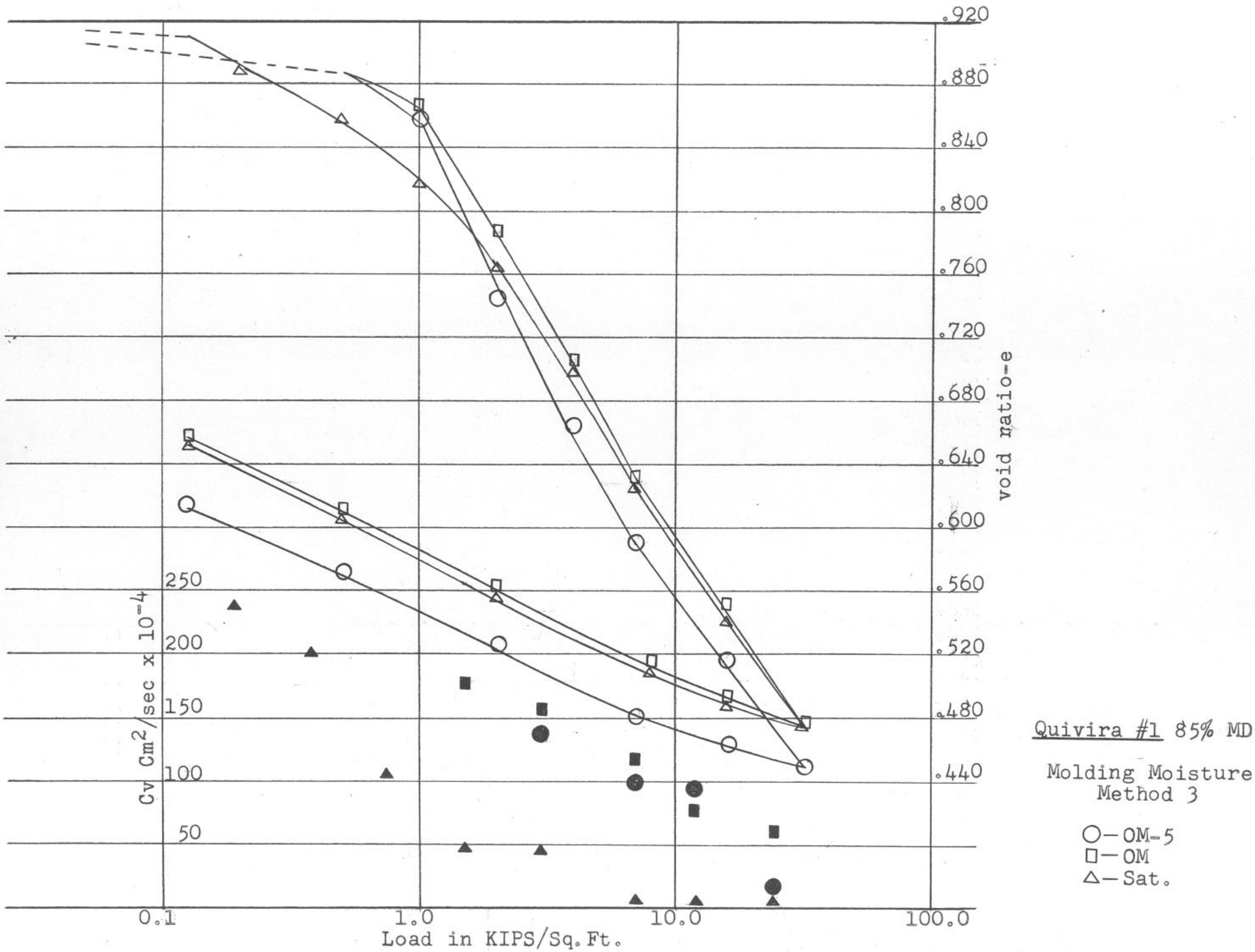


Figure No. 8 Consolidation Curve Quivira #1 at 85% M.D.

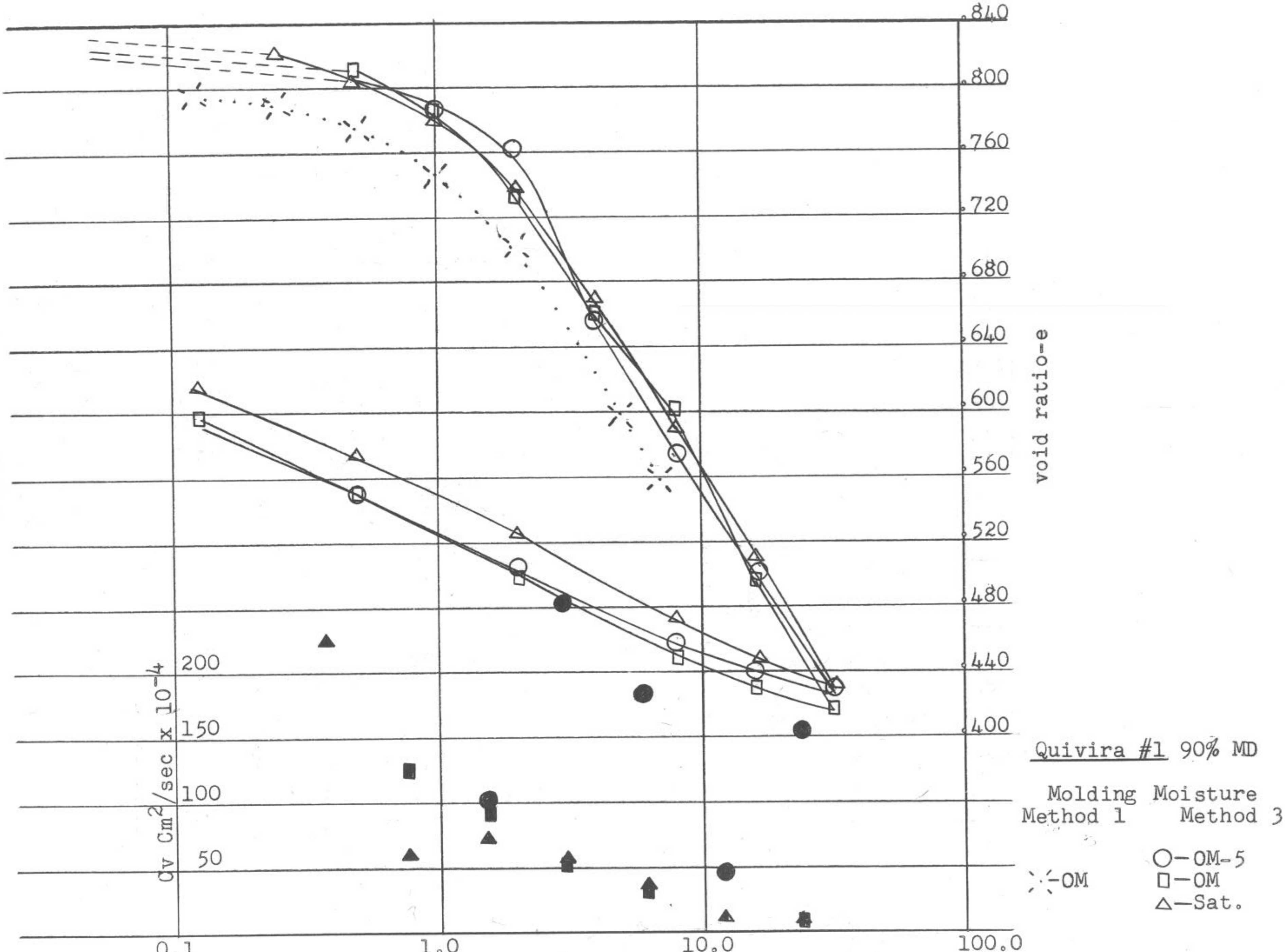


Figure No. 9 Consolidation Curve Quivira #1 at 90% M.D.

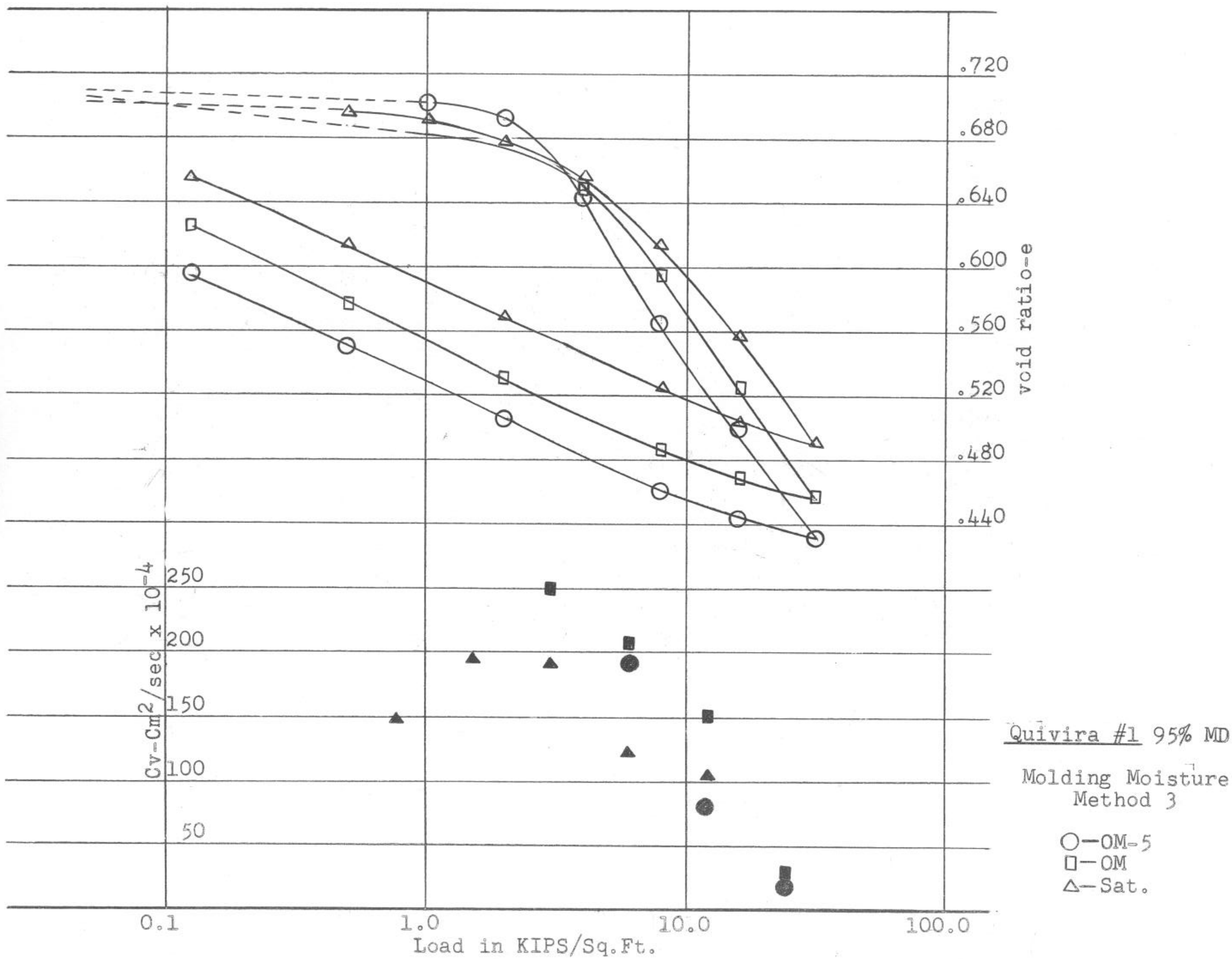


Figure No 10 Consolidation Curve Quivira #1 Shale at 95%M.D.

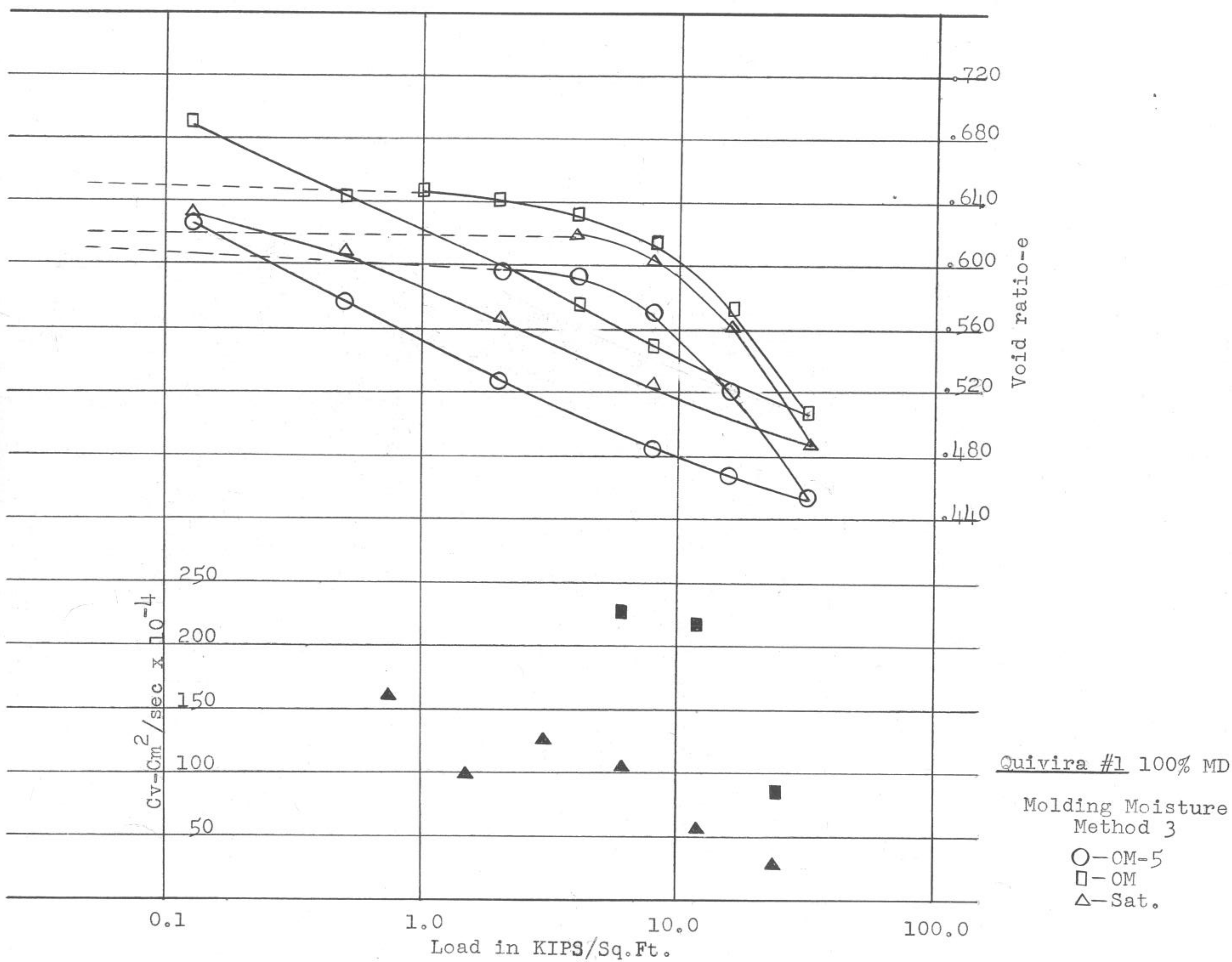


Figure No. 11 Consolidation Curve Quivira #1 Shale at 100%M.D.

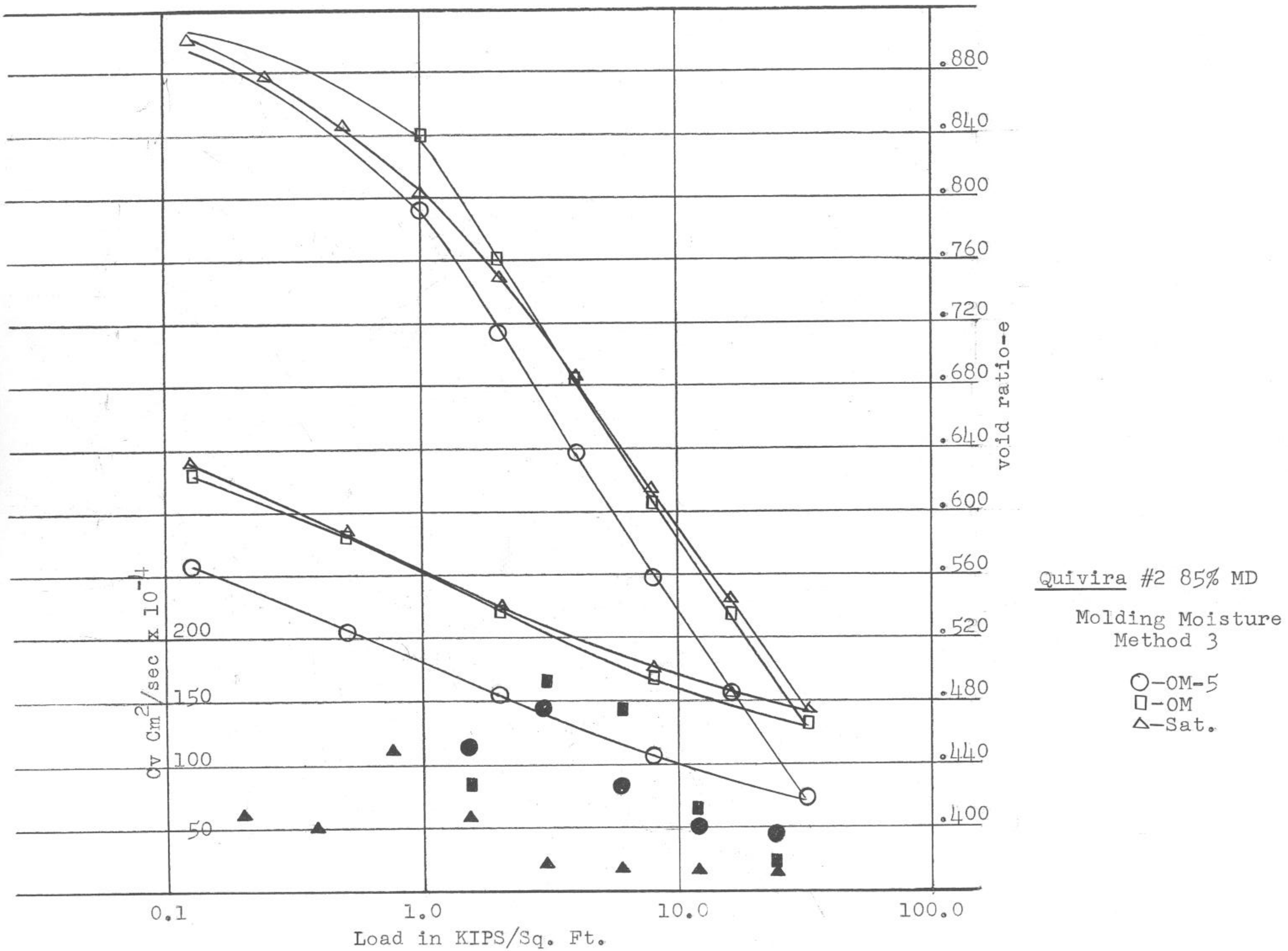


Figure No. 12 Consolidation Curve Quivira #2 Shale at 85%M.D.

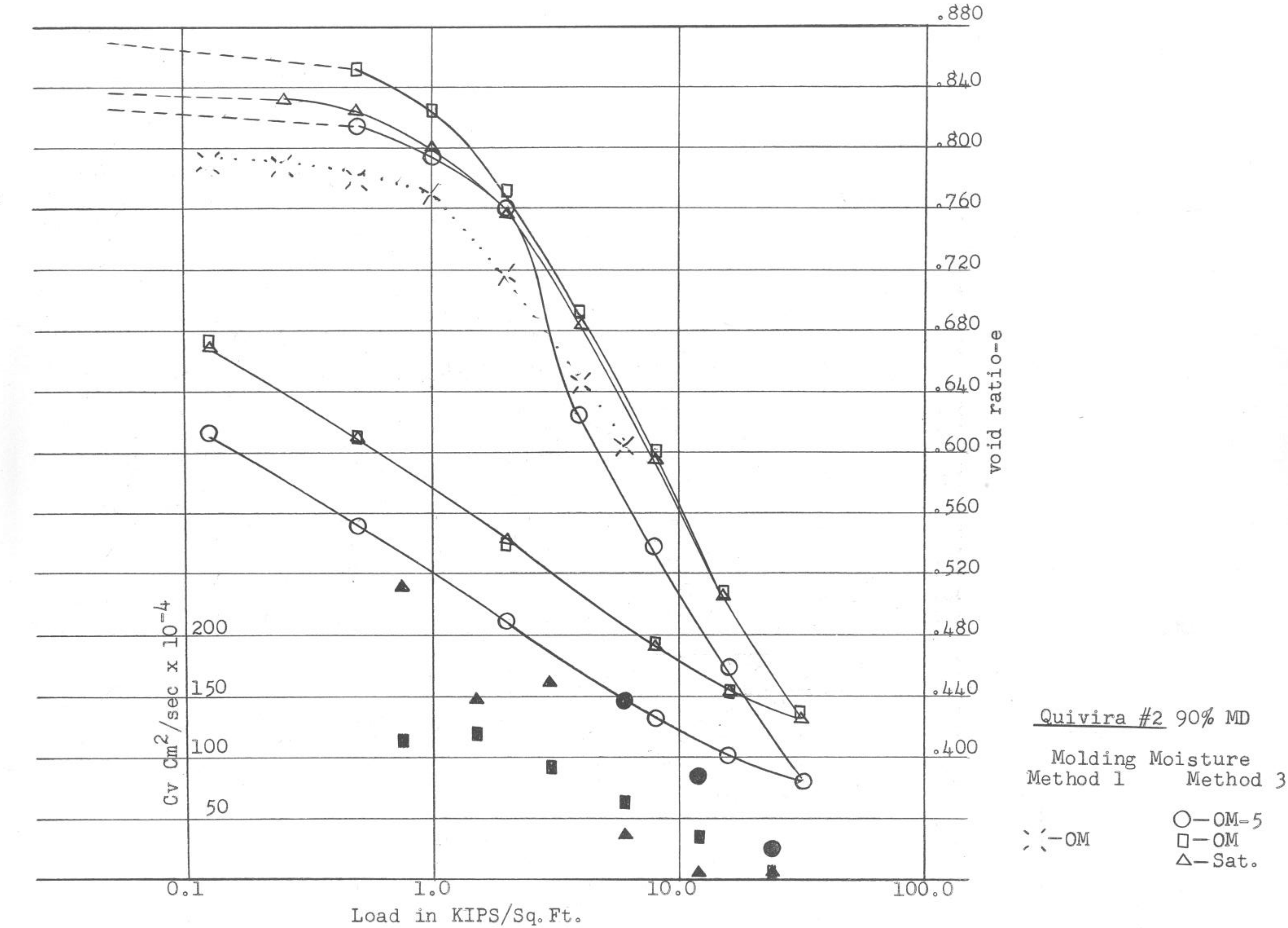


Figure No. 13 Consolidation Curve Quivira #2 Shale at 90%M.D.

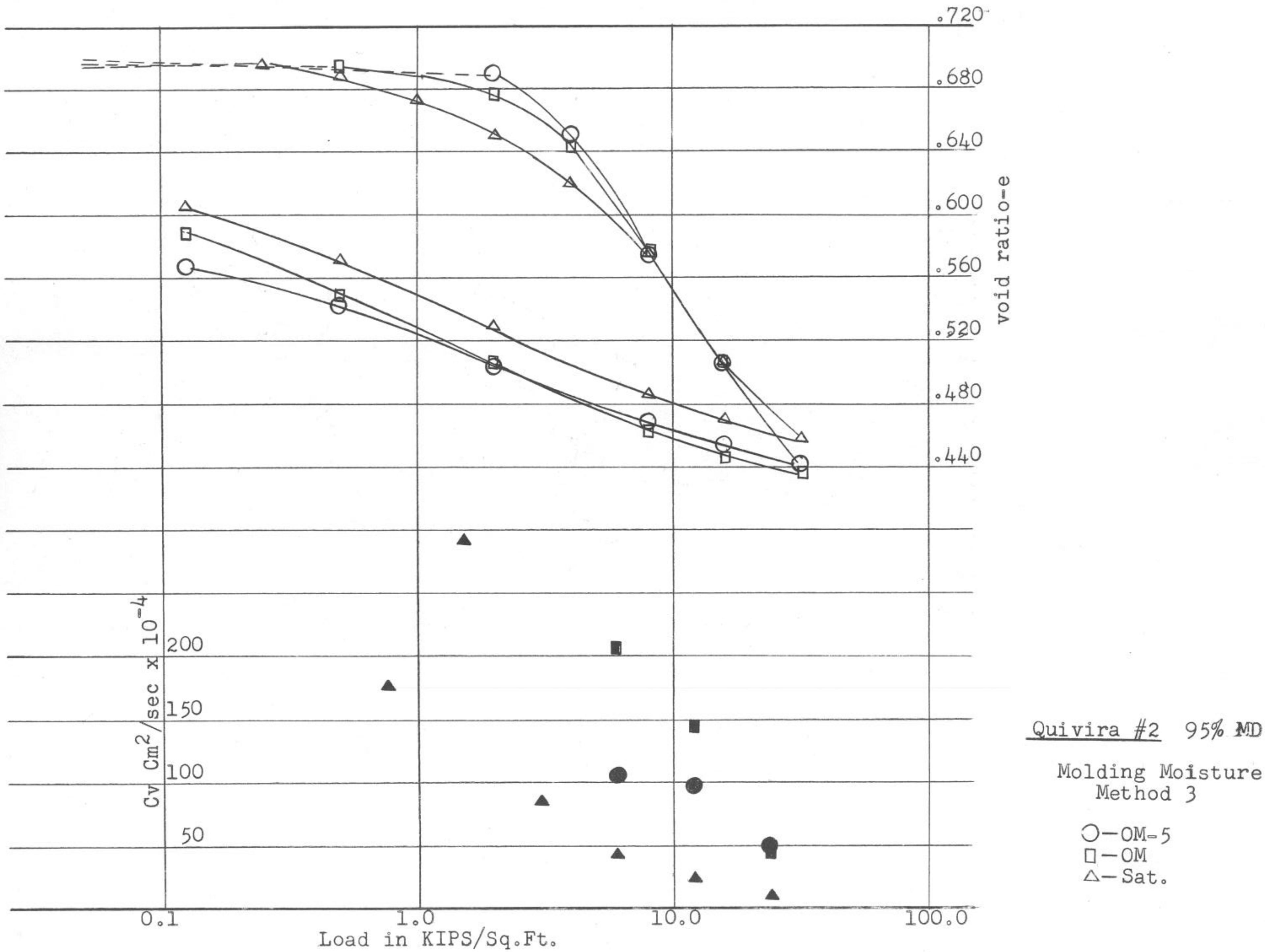


Figure No. 14 Consolidation Curve Quivira #2 Shale at 95%M.D.

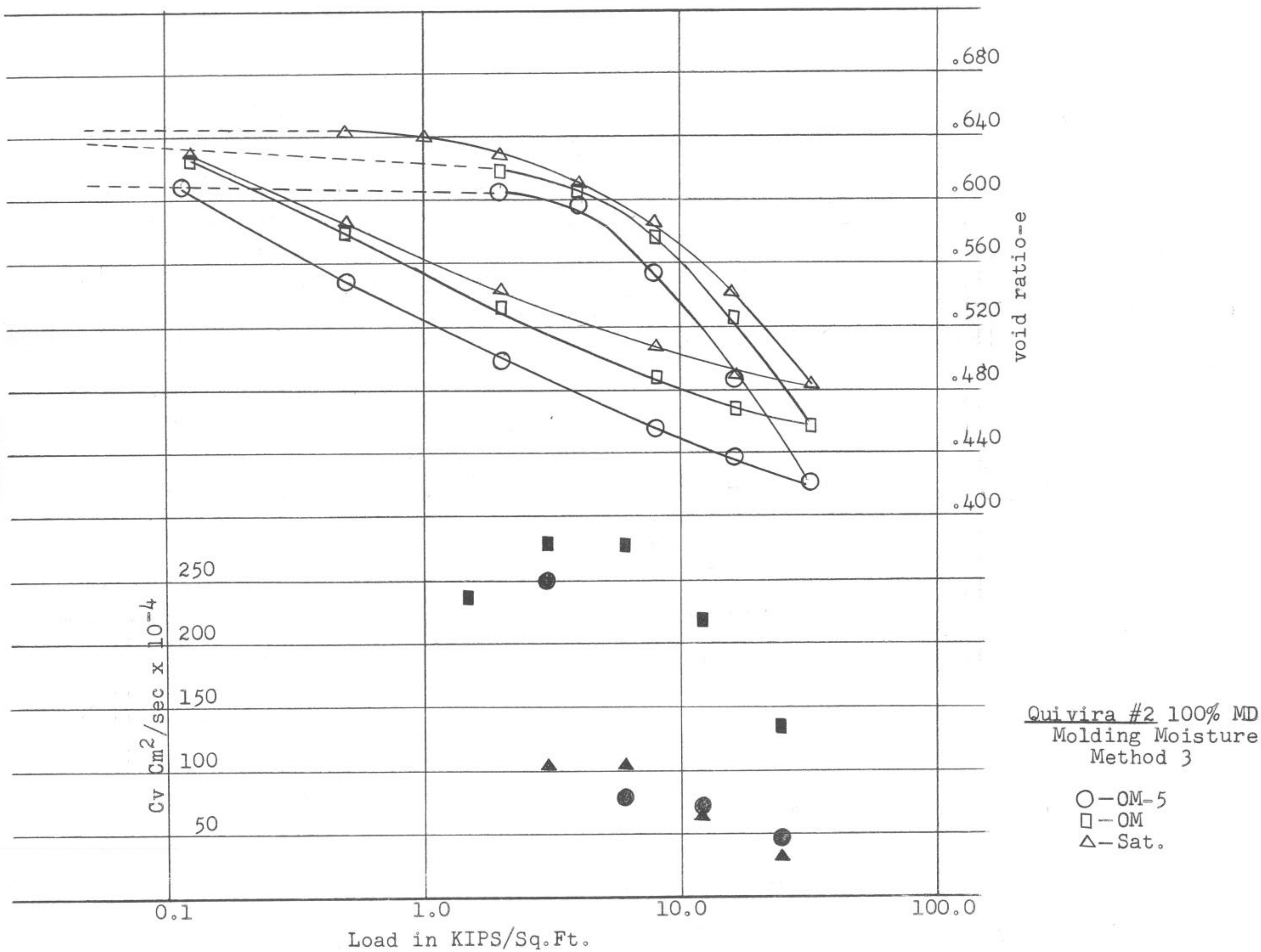


Figure No. 15 Consolidation Curve Quivira #2 Shale at 100%M.D.

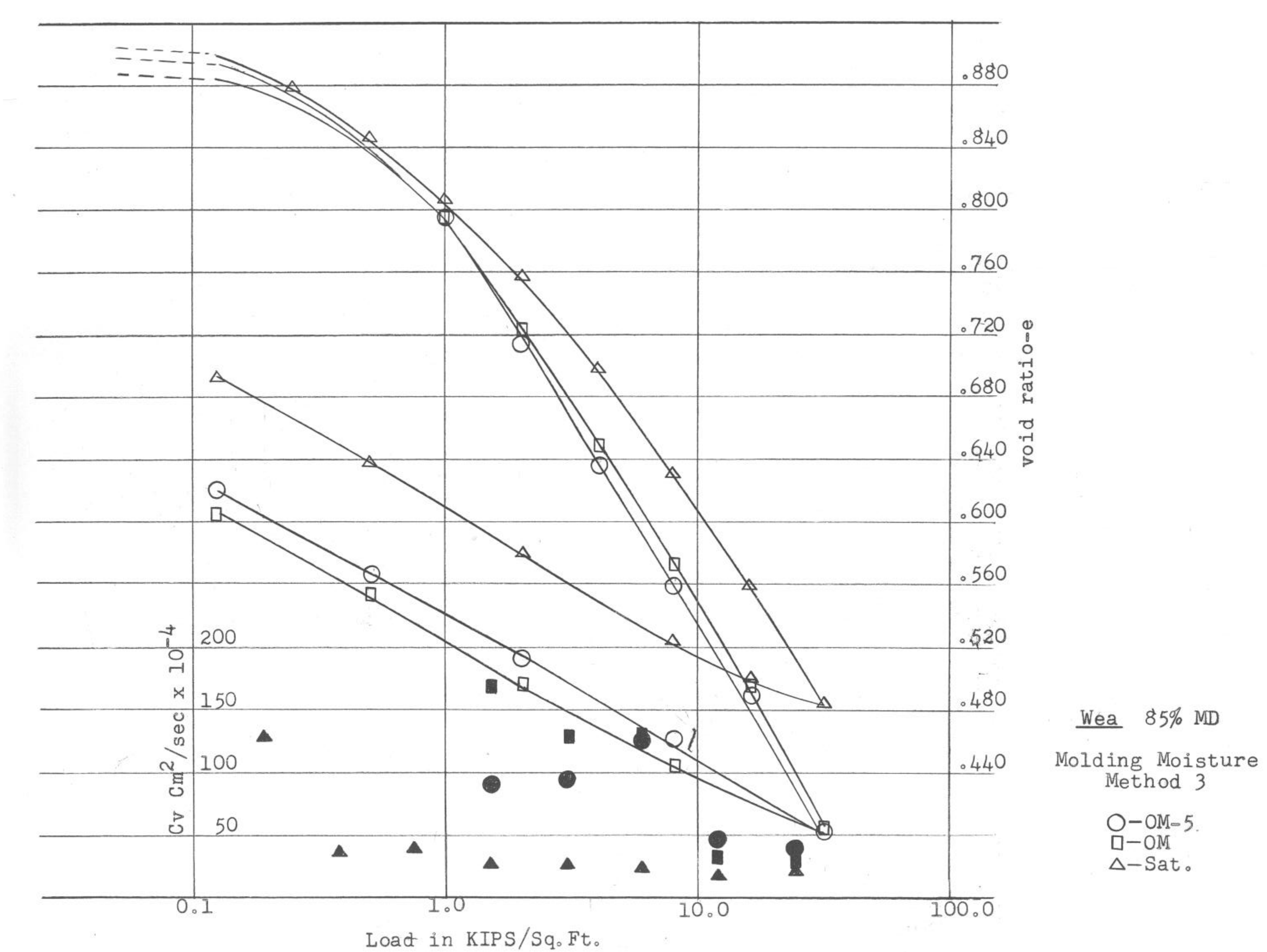


Figure No. 16 Consolidation Curve Wea Shale at 85%M.D.

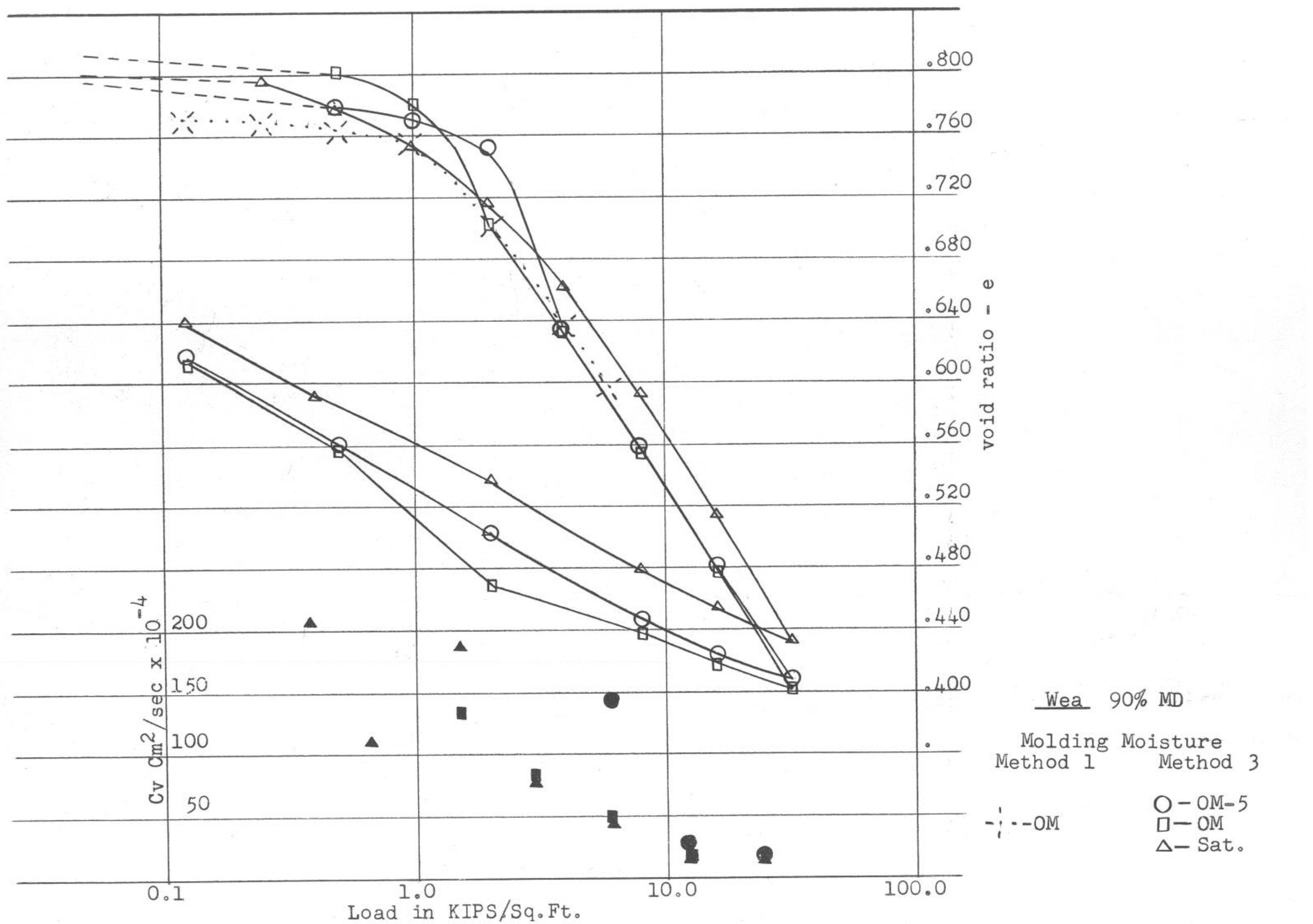


Figure No. 17 Consolidation Curve Wea Shale at 90%M.D.

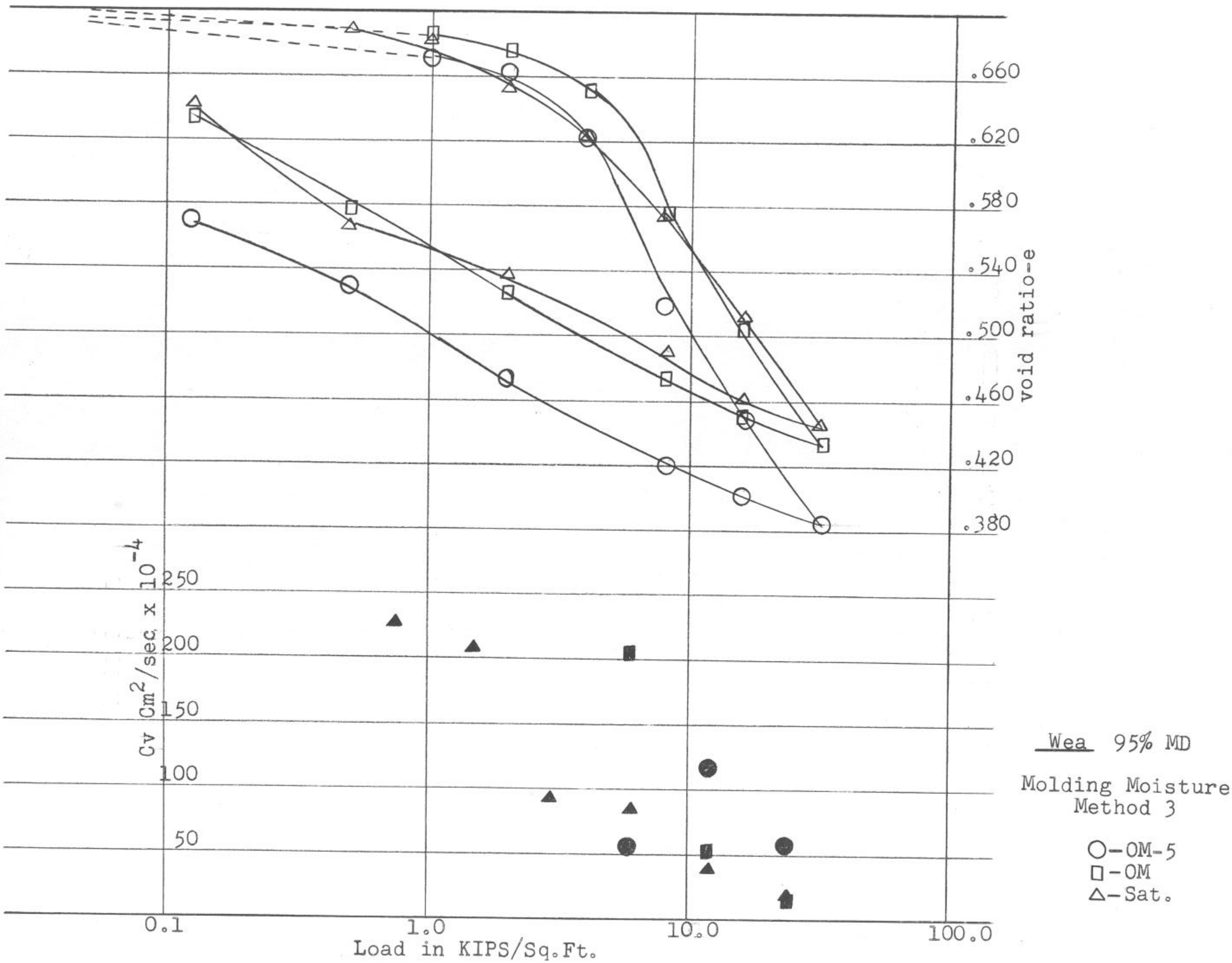


Figure No. 18 Consolidation Curve WEA Shale at 95% M.D.

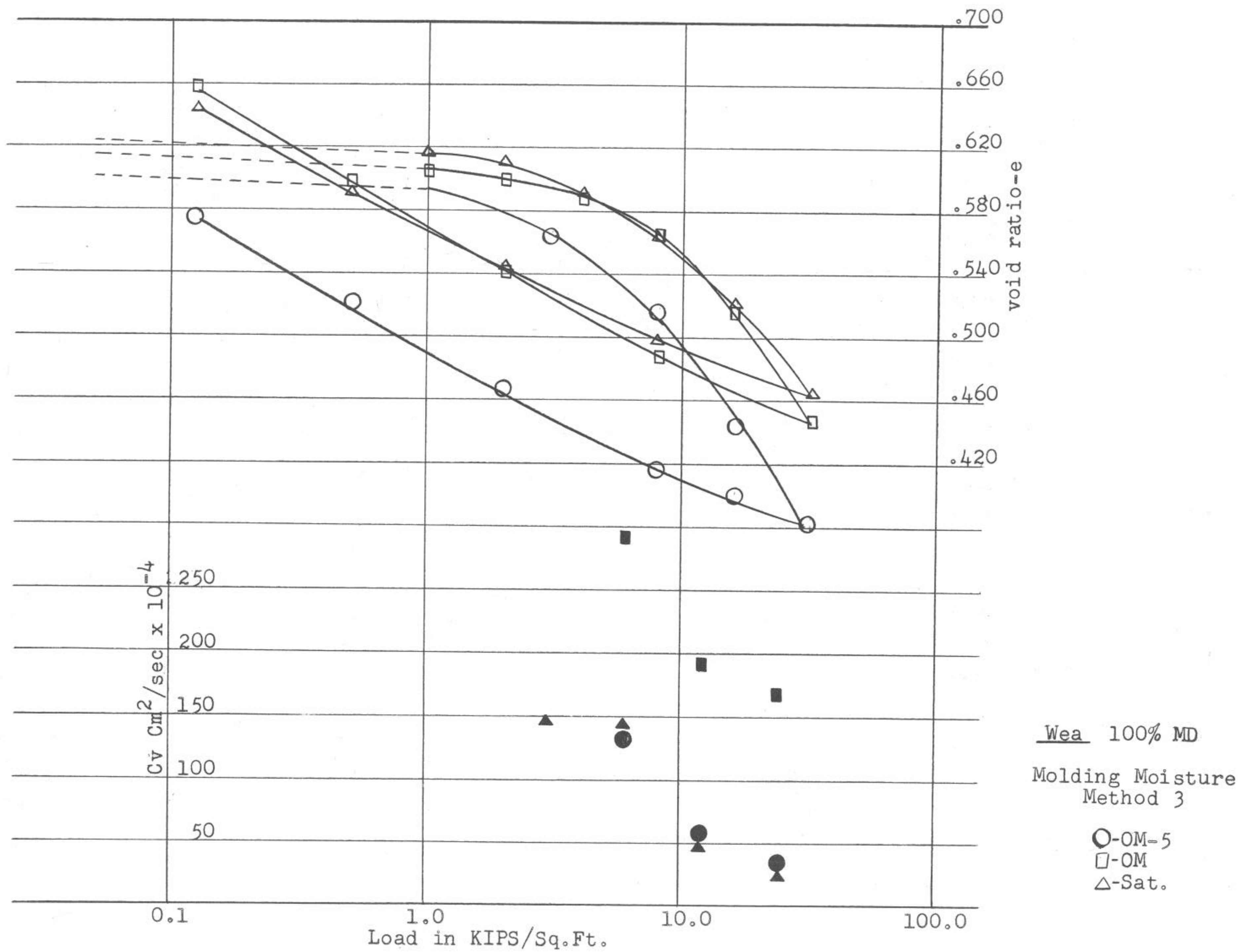


Figure No. 19 Consolidation Curve Wea Shale at 100% M.D.

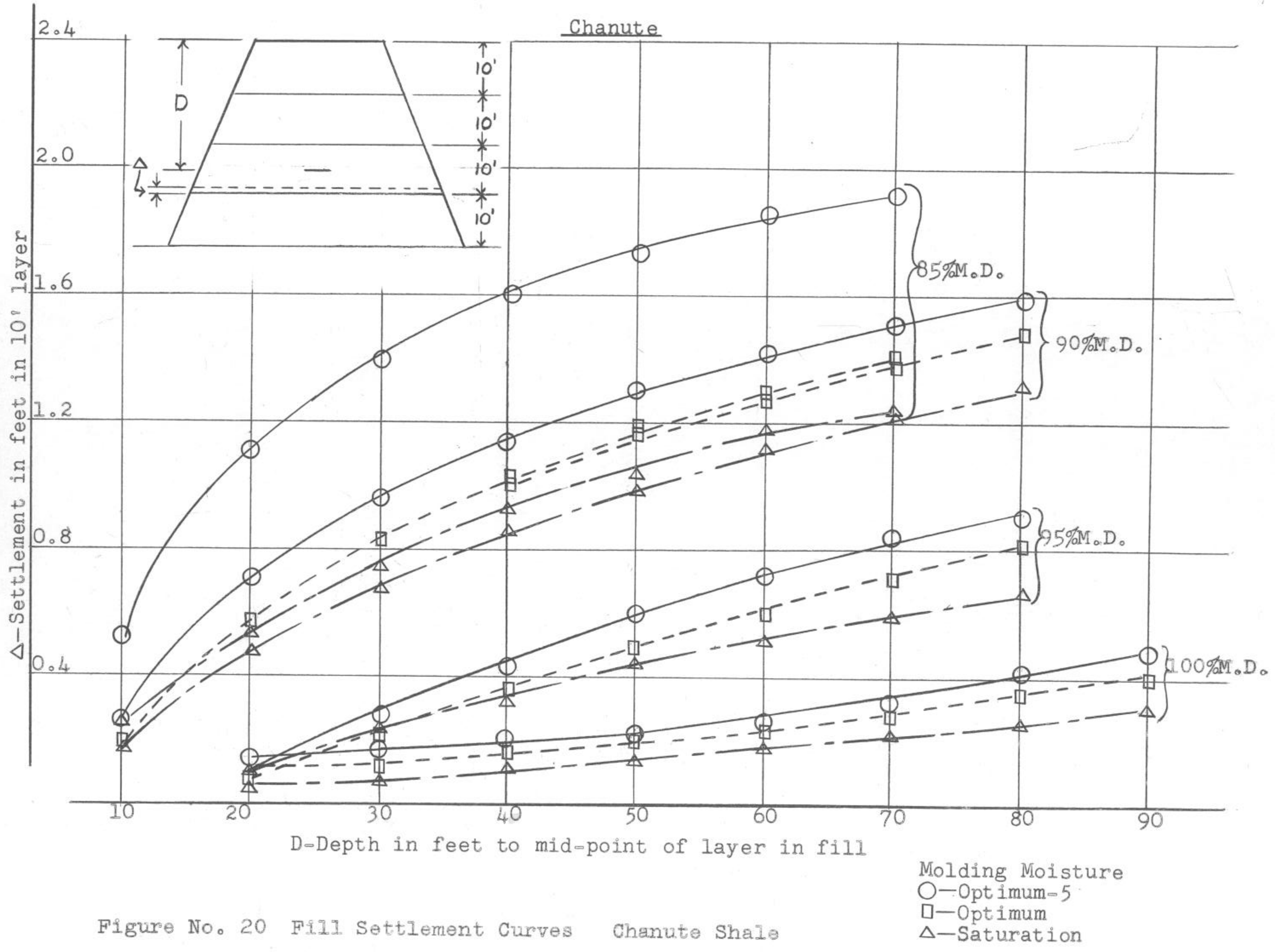


Figure No. 20 Fill Settlement Curves Chanute Shale

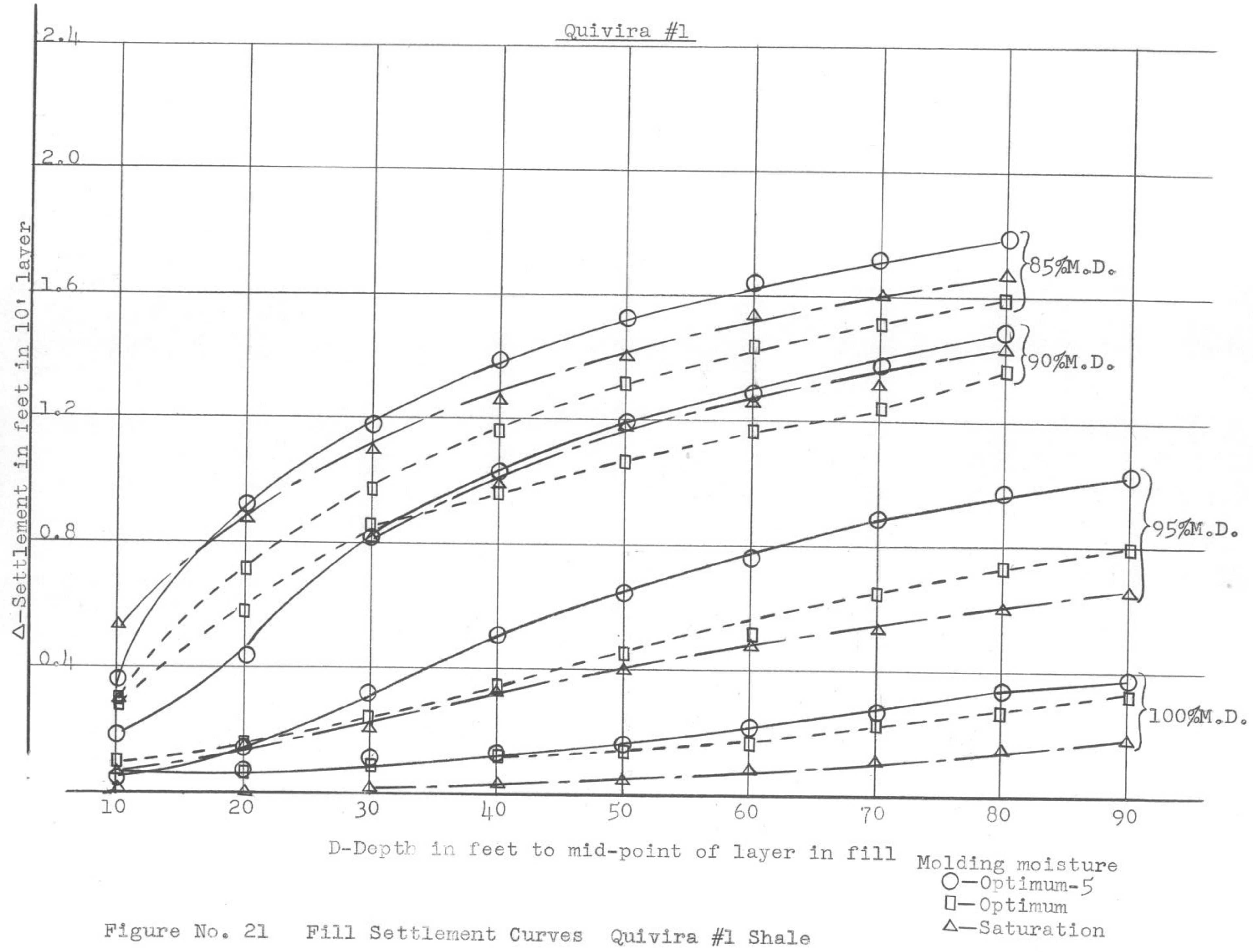


Figure No. 21 Fill Settlement Curves Quivira #1 Shale

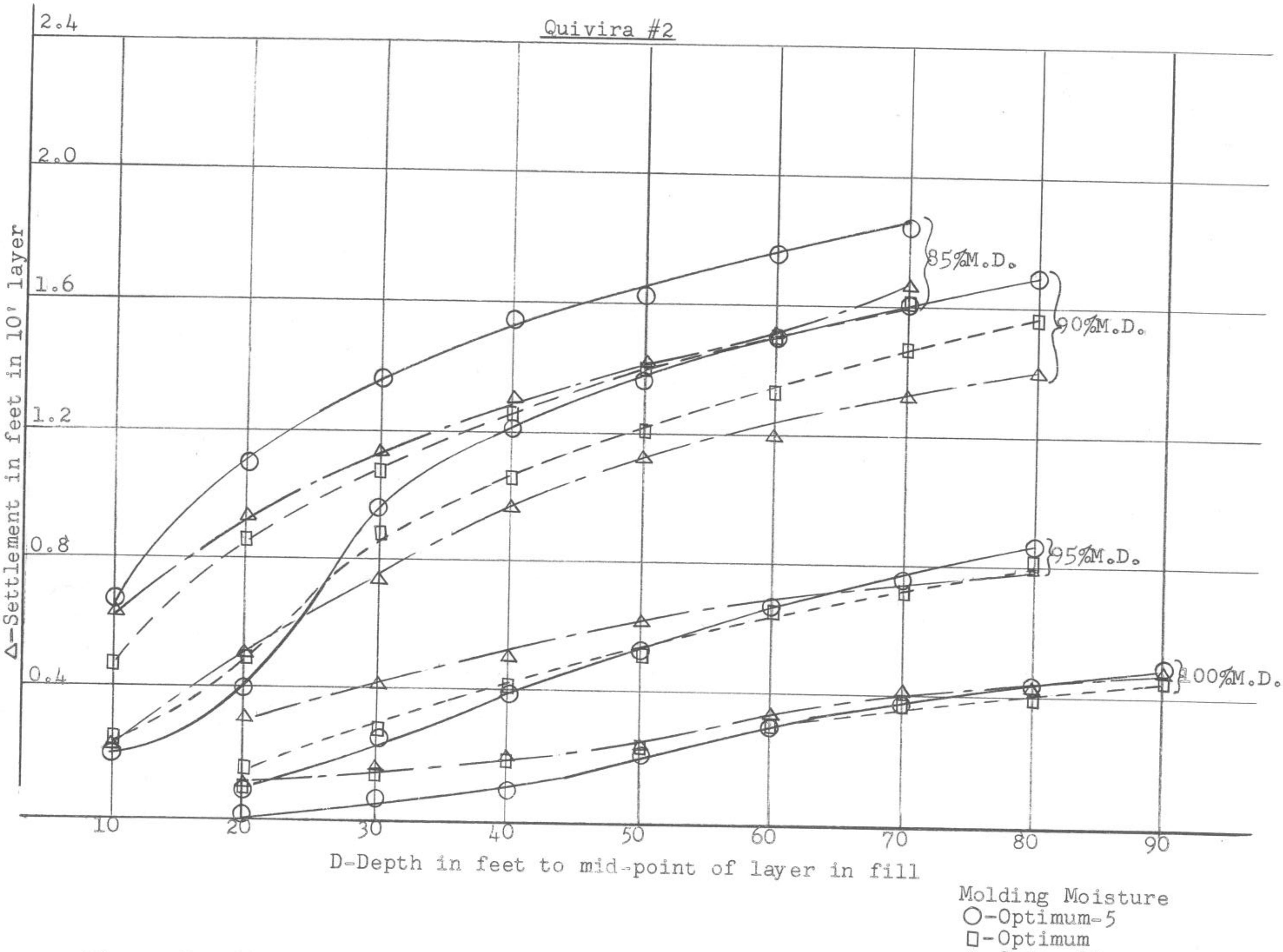


Figure No. 22 Fill Settlement Curves Quivira #2 Shale

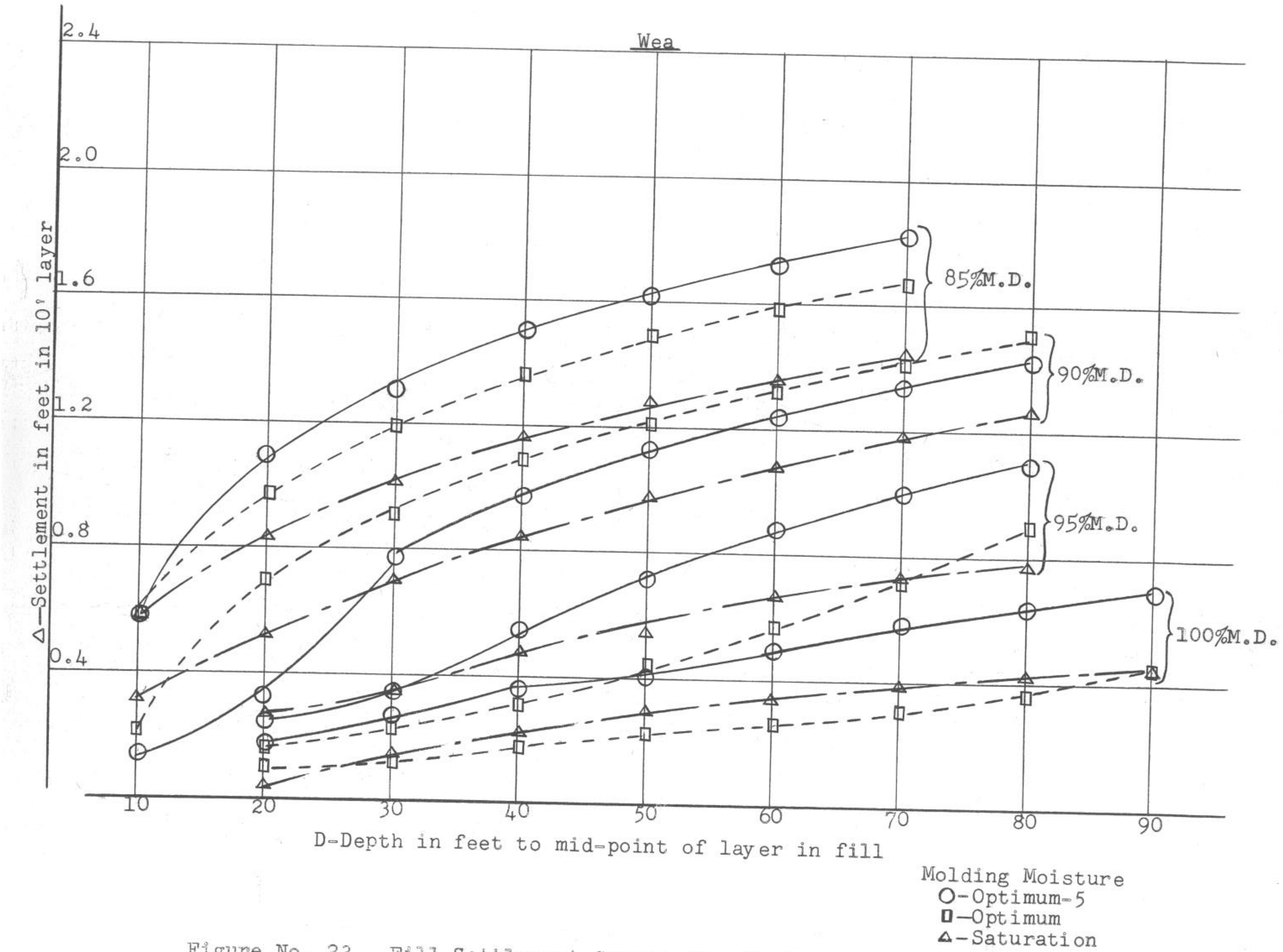


Figure No. 23 Fill Settlement Curves Wea Shale

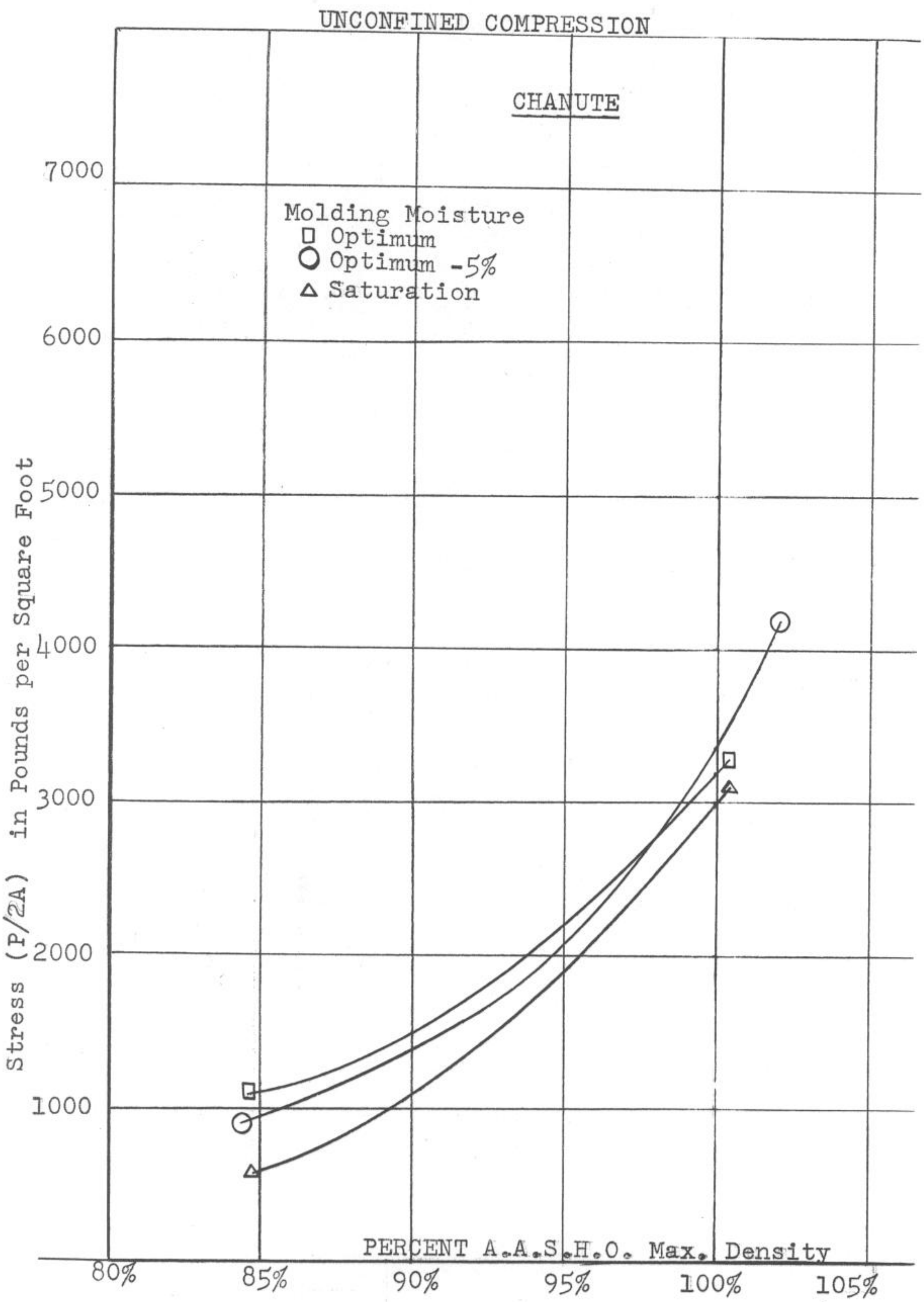


Figure No. 25 Stress(P/2A) Vs. Percent A.A.S.H.O. Maximum Density

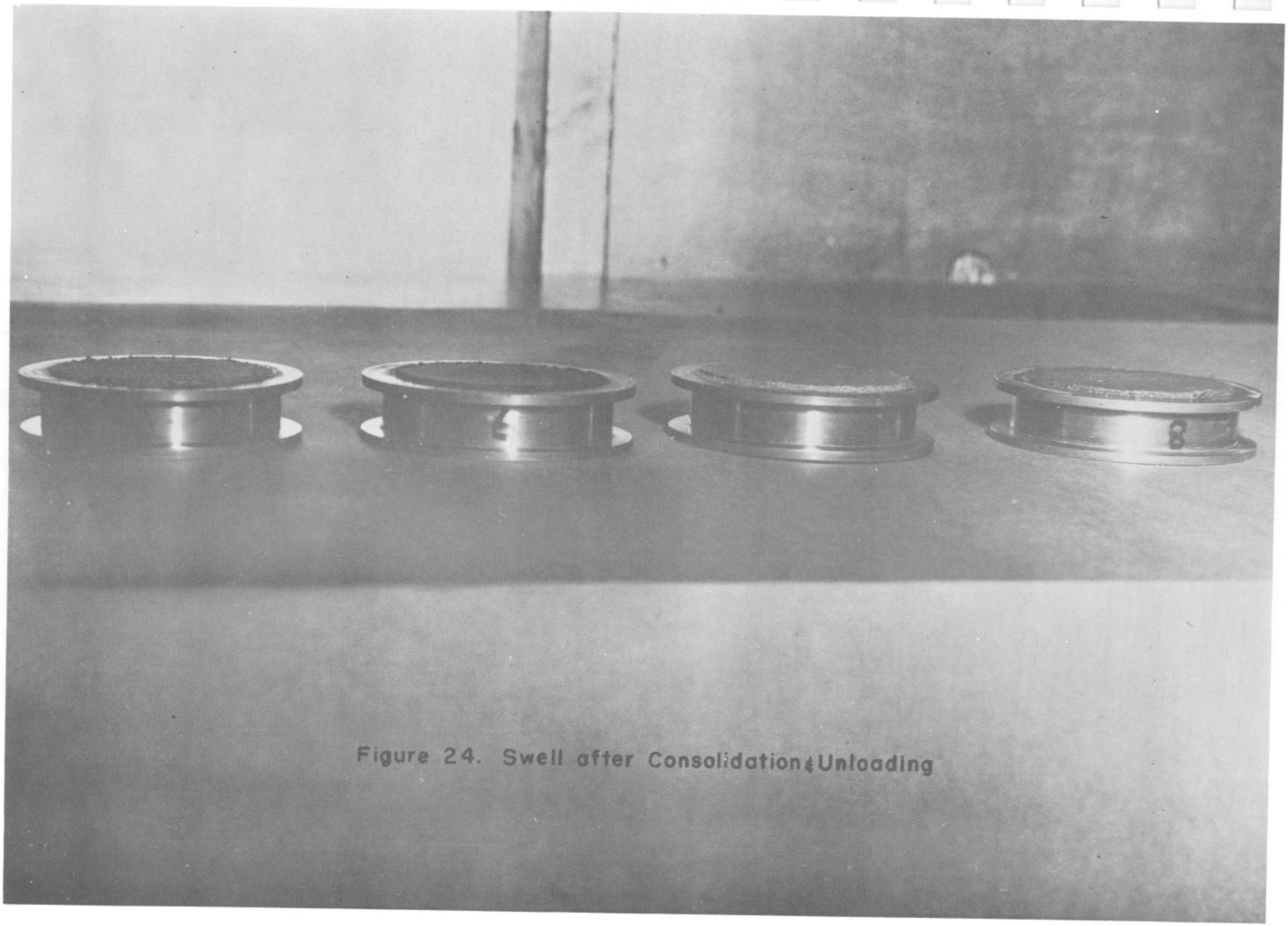


Figure 24. Swell after Consolidation & Unloading

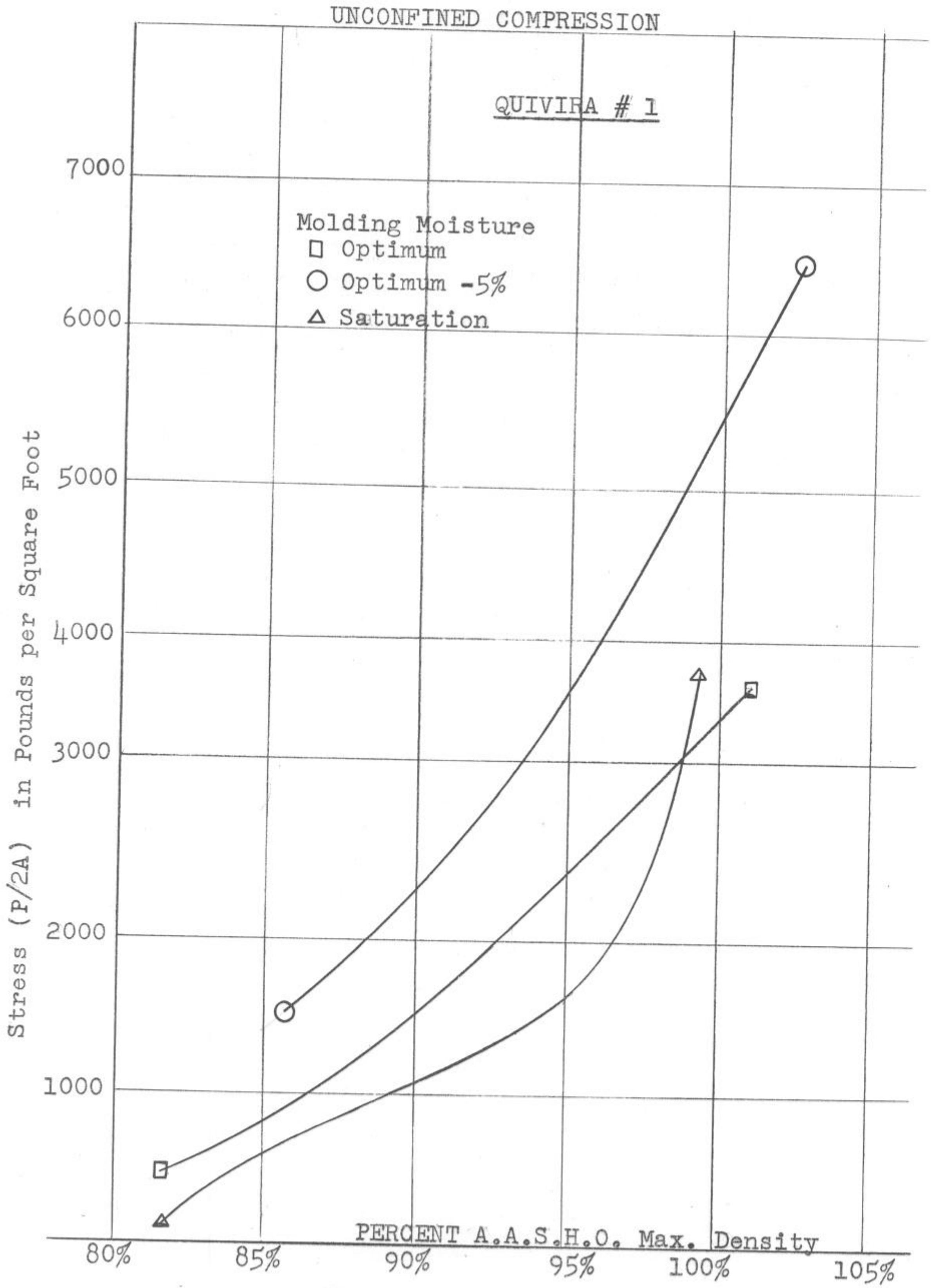


Figure No.26 Stress(P/2A) Vs. Percent A.A.S.H.O. Maximum Density

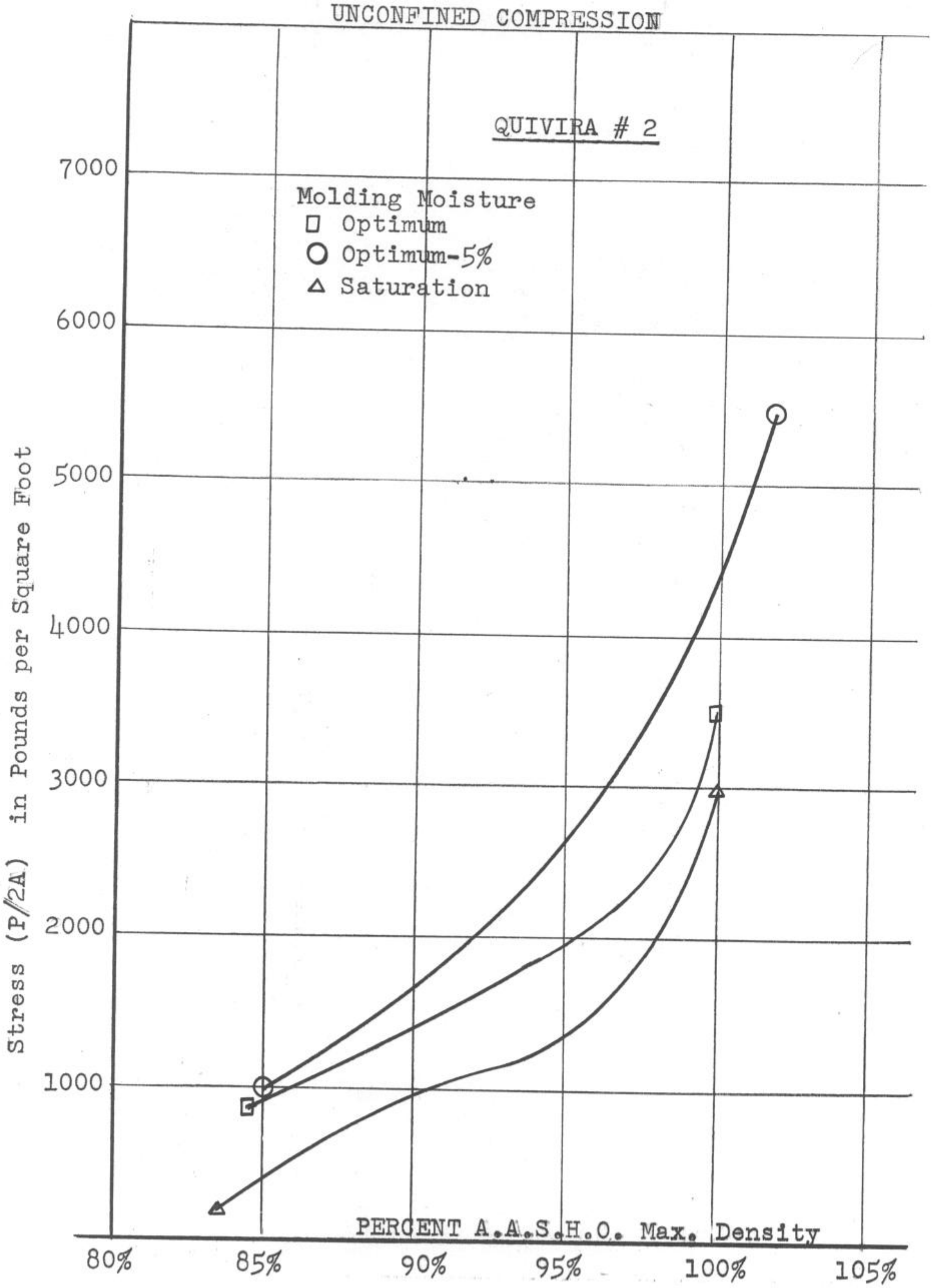


Figure No. 27 Stress(P/2A) Vs. Percent A.A.S.H.O. Maximum Density

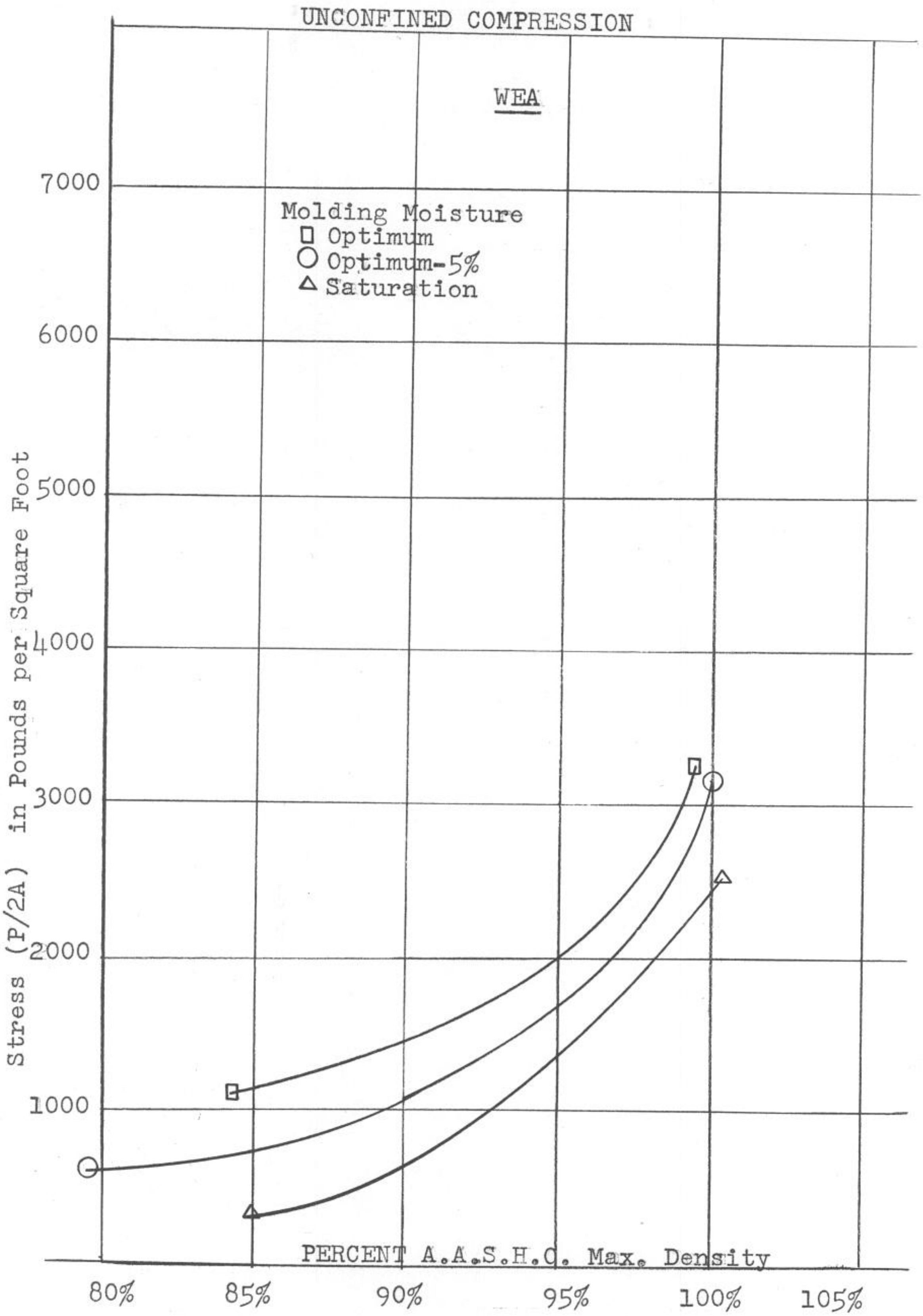
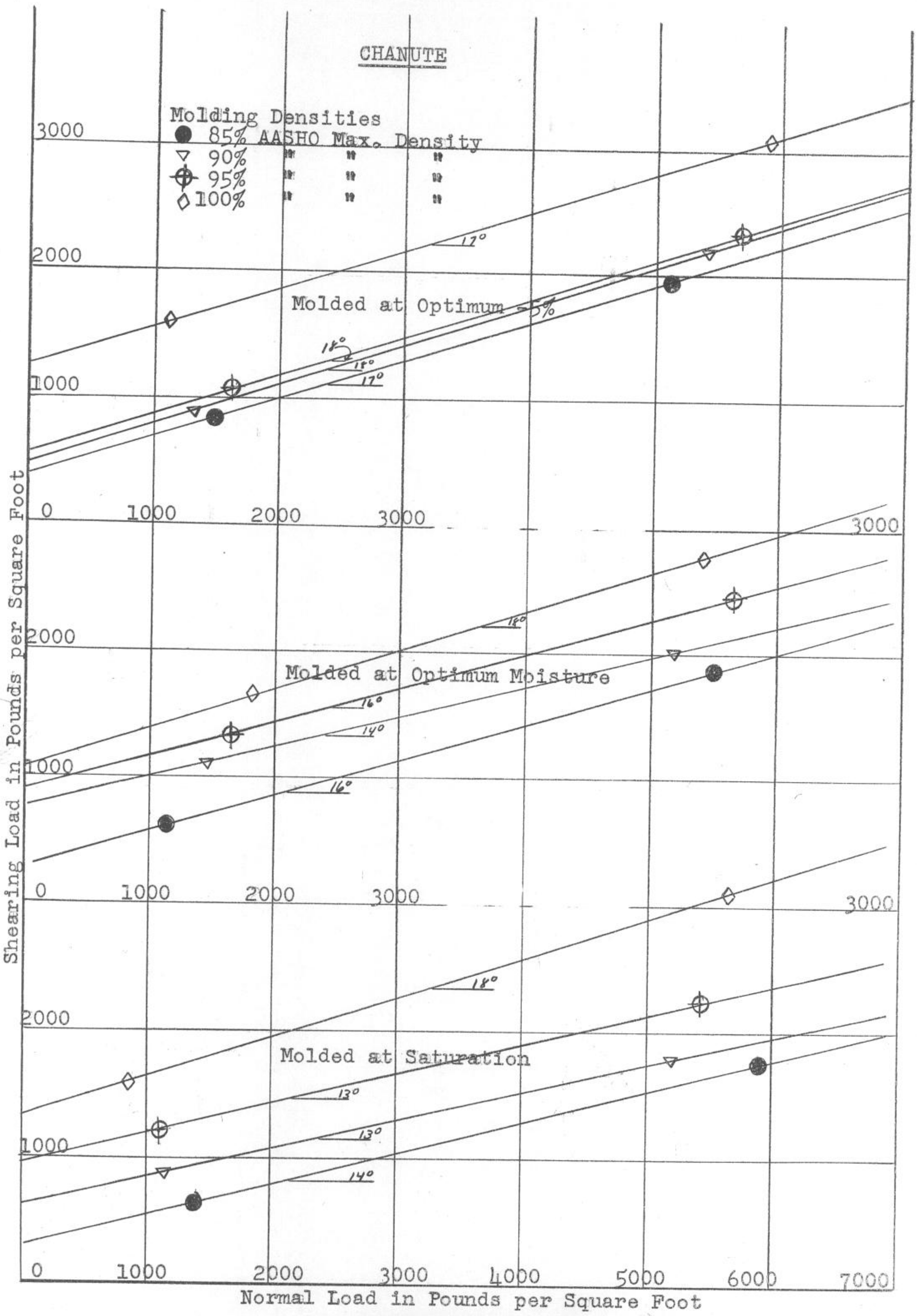
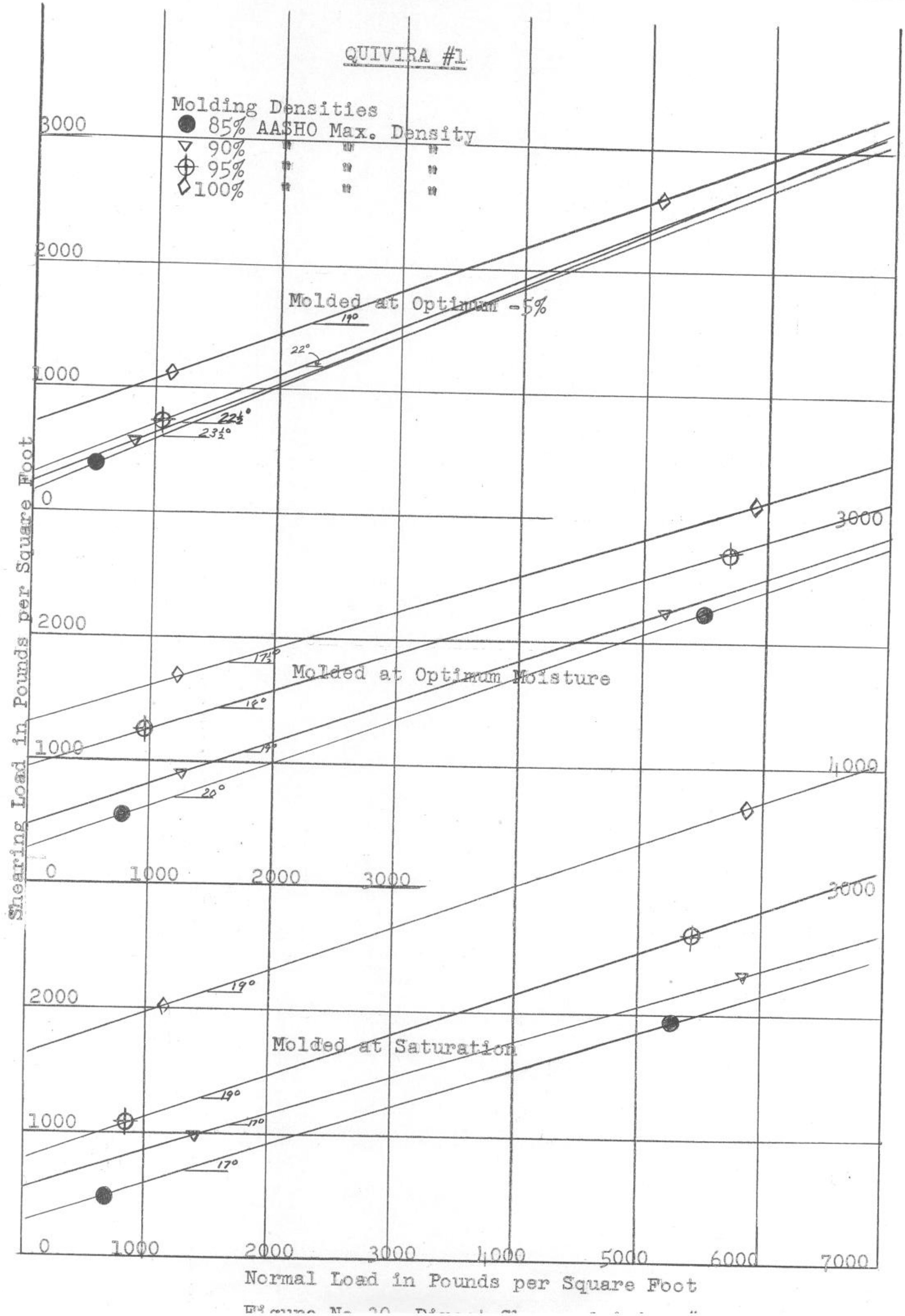
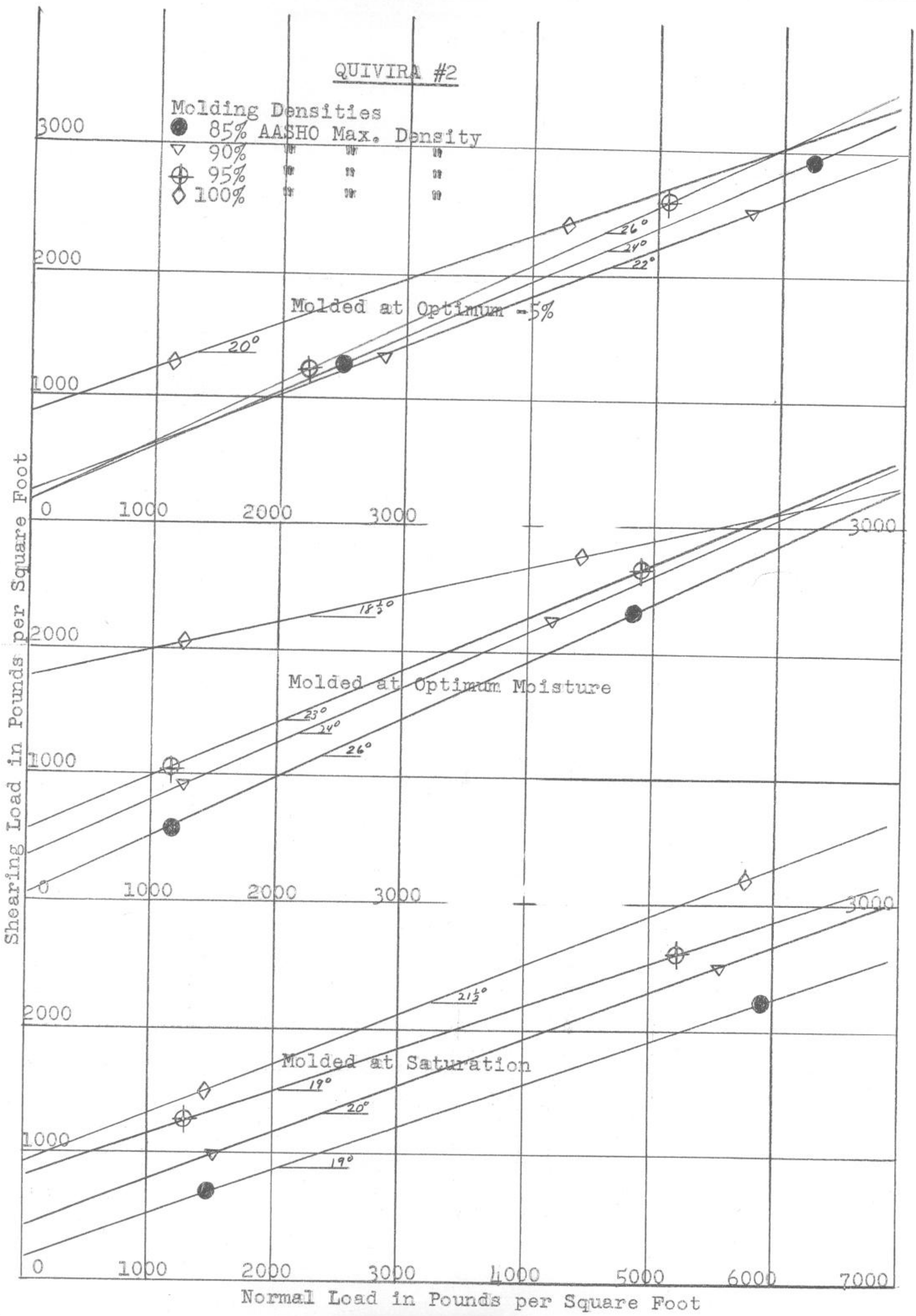


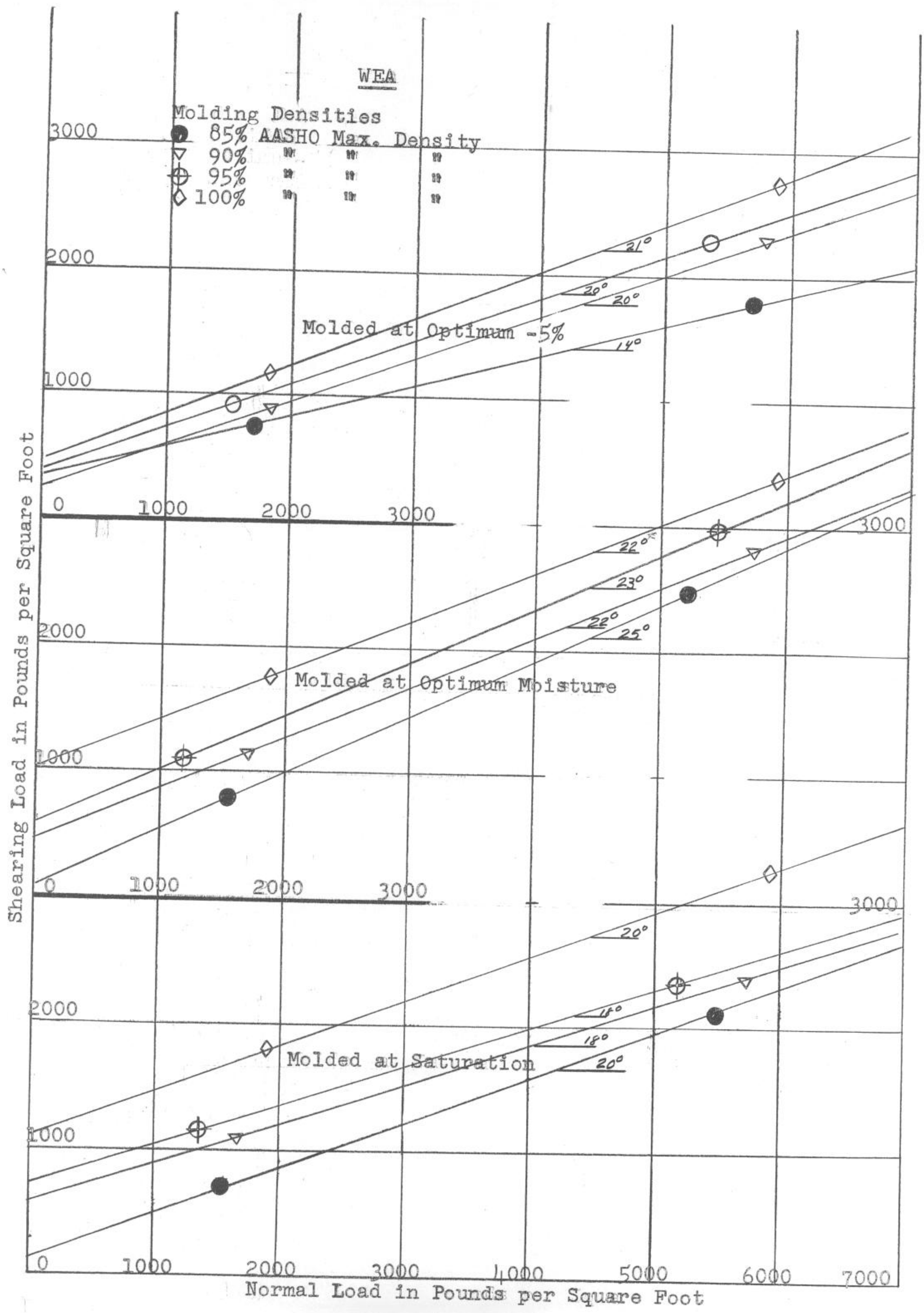
Figure No. 28 Stress(P/2A) Vs. Percent A.A.S.H.O. Maximum Density


CHANUTE
Molding Densities
85% AASHO Max. Density
90% " " "
95% " " "
100% " " "
Molded at Optimum -5%
17°
18°
18°
17°
Molded at Optimum Moisture
18°
16°
14°
16°
Molded at Saturation
18°
13°
13°
14°
Shearing Load in Pounds per Square Foot
Normal Load in Pounds per Square Foot
0
1000
2000
3000
4000
5000
6000
7000


QUIVIRA #1
Molding Densities
85% AASHO Max. Density
90% " " "
95% " " "
100% " " "
Molded at Optimum -5%
19°
22°
22½°
23½°
Molded at Optimum Moisture
17½°
18°
19°
20°
Molded at Saturation
19°
19°
17°
17°
Shearing Load in Pounds per Square Foot
Normal Load in Pounds per Square Foot
0
1000
2000
3000
4000
5000
6000
7000



QUIVIRA #2
Molding Densities
85% AASHO Max. Density
90% " " "
95% " " "
100% " " "
Molded at Optimum -5%
26°
24°
22°
20°
Molded at Optimum Moisture
18½°
23°
24°
26°
Molded at Saturation
21½°
19°
20°
19°
Shearing Load in Pounds per Square Foot
Normal Load in Pounds per Square Foot
0
1000
2000
3000
4000
5000
6000
7000


WEA
Molding Densities
85% AASHO Max. Density
90% " " "
95% " " "
100% " " "
3000
2000
1000
0
1000
2000
3000
21°
20°
20°
14°
Molded at Optimum -5%
22°
23°
22°
25°
Molded at Optimum Moisture
20°
18°
18°
20°
Molded at Saturation
4000
5000
6000
7000
Shearing Load in Pounds per Square Foot
Normal Load in Pounds per Square Foot