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W.G. Weber

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Introduction

A freeway was built across Candlestick Cove on the west side of the San Francisco Bay just south of the San Francisco city line in San Mateo County. The portion of the freeway covered in this report is the open water fill between Candlestick Point and Sierra Point. The water in this area is about five feet deep. See Fig. No. 1.

The open water fill was constructed under five earthwork contracts, the first two being experimental units. A final contract for final grading and paving then completed the freeway. A total of 5,032,000 cubic yards of fill was used in constructing the open water fill at a total cost of $4,870,600 for the completed freeway.

The soil formation in the open water area of Candlestick Cove consists of unconsolidated sediments of recent geological age, the upper layer of which is a very soft mud with its surface at about elevation -5. The bottom of the soft mud varies from elevation -40 to -80. This mud is underlain by a somewhat stiffer material consisting variously of clayey sand, sandy clay, or clay. Bedrock varied in this area from elevation -110 to -220.

The fill as built consisted of two parts. A working table was placed in the uncompacted state. A smaller compacted prism was placed on the working table and is referred to as the roadway prism. The mud that was displaced by the fill is referred to as the displaced mud. The mud that was pushed up above the original mud line is referred to as the mud wave. See Fig. No. 2. For a detail discussion see Part I of Appendix.

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STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

CONSTRUCTION AND SPECIAL STUDIES
on the
CANDLESTICK COVE OPEN WATER FILL
IV-Sm-68-E

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State of California
Department of Public Works
Division of Highways
Materials and Research Department

December 23, 1959

IV-SM-68-E
Candlestick Cove
Lab Auth 2013-R-21

Mr. F. N. Hveem
Materials & Research Engr.
Division of Highways
Sacramento, California

Dear Sir:

Submitted for your consideration is:

A REPORT

on

CONSTRUCTION AND SPECIAL STUDIES

on the

CANDLESTICK COVE OPEN WATER FILL

IV-SM-68-E

Study made by .....Foundation Section
Under general direction of.... A. W. Root
Work supervised by..... T. W. Smith
Report prepared by..... W. G. Weber

Very truly yours,

A. W. Root
Supv. Mtls. & Research Engr.

Attach.

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Introduction

A freeway was built across Candlestick Cove on the west side of the San Francisco Bay just south of the San Francisco city line in San Mateo County. The portion of the freeway covered in this report is the open water fill between Candlestick Point and Sierra Point. The water in this area is about five feet deep. See Fig. No. 1.

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The fill as built consisted of two parts. A working table was placed in the uncompacted state. A smaller compacted prism was placed on the working table and is referred to as the roadway prism. The mud that was displaced by the fill is referred to as the displaced mud. The mud that was pushed up above the original mud line is referred to as the mud wave. See Fig. No. 2. For a detail discussion see Part I of Appendix.

Construction of the Working Table

The working table was constructed by end dumping methods. This consisted of placing the fill material as close to the edge of the fill as practical and using a bulldozer to push the dirt over the side. Sections of the fill would then fail and the contractor would then fill in the failed area restoring it to the desired grade. The nose of the fill was maintained in a wedge shape pushing the mud laterally. The displaced mud then formed large mud waves on each side of the fill.

The shape of the nose was varied as the working table was constructed. A long slender extension of the nose along centerline resulted in large mud displacement on centerline and small mud displacement at the sides. A blunt nose resulted in a saw tooth type of displacement. In all cases where a wedge shaped nose was maintained at about 45 degrees with centerline a satisfactory working table was obtained. Whenever a

wedge shaped nose was not used erratic and undesirable results generally occurred.

Stability calculations indicated that if only minor mud displacement occurred a width of 400 feet would be required for the working platform to support the roadway prism. When it was seen that considerable mud displacement was occurring it was decided to try reducing the width of the working table. The width of the working table was reduced in steps to 200 feet. When the wedge shaped nose was used the mud was split to a distance of 100 feet from centerline, and increasing the width past 100 feet resulted in lateral failures of the edge of the fill. The 200 foot width of working table proved to be the best practical width. There were no shear failures due to the inability of the working table to support the roadway prism.

The minimum rate of fill placement was 10,000 cubic yards per day on all except the first unit, which was 5,000 cubic yards per day. The rate of fill placement determined the rate of advance of the fill and sufficient fill placement was required to keep the fill above high tide when the failures occurred.

During the first two units constructed both bottom dump and tail dump equipment was used. The bottom dump equipment dumped their loads near the edge of the working table and the fill was pushed over the side with a cat. Where the bottom dump equipment was used the fill would often be advanced over the mud and failure would result on the backside of the mud wave. The tail dump trucks dumped their loads directly over the edge of the fill and a cat was only occasionally used. The tail dump trucks operation tended to keep the mud moving away from the fill and maintaining a trough between the fill and the mud wave. On all remaining units only tail dump equipment was used.

As the working table displaced the mud, large mud waves were formed. To aid in reducing the height to which these mud waves formed mud blasting was used. The mud blasting was done by shoving 50-pound charges of explosive into the mud around the nose of the fill until the charges were at about elevation -25. When the charges were exploded the fill shook violently and settled a small amount. The blasting of the mud wave was of great value in keeping the mud "liquefied" and thus preventing a large build-up of the crests of the mud wave.

During the construction of the open water fill it was necessary to make two closures, that is, advance a newly constructed fill to an existing fill that had been completed previously. The existing mud waves were reduced by means of mud blasting and dredging. The new fill was then advanced to the existing fill. A stable fill was thus obtained at the

points of closure. A detailed discussion of the construction of the working table is included in Part II of the Appendix.

Construction of the Roadway Prism

As the working table was completed the roadway prism was placed upon it. The roadway prism consisted of a ten-foot high fill placed in eight inch compacted lifts, 130 feet wide on the top with 2:1 side slopes. Profile grade of the roadway prism was +20 as constructed. In the final grading operations cuts and fills were made to bring the profile grade to +18, with a two-foot structural section specified.

During the final work on the open water fill there was a surplus of dirt. This fill was placed on the berm so as to make the berm level with the completed roadway. The fill in the disposal area is in an uncompacted condition. A detailed discussion of the construction of the roadway prism is included in Part II of the Appendix.

Failures During Construction of the Working Table

As the nose was advanced from the stable portion of the working table a crack would develop near the stable portion. Regardless of how far the fill was advanced past this crack, movement would continue to occur at this location until the nose had become stable. Then the cracking would advance to the point where the fill was again unstable. Studies were made on the failures by measuring the surface movements and running shear circles. From these measurements two types of movement were found to occur: (1) (Shear type) a rotary movement where the fill would have a large drop at the crack and small or no movement at the edge of the fill, See Fig. 3; (2) (Squeeze type) the drop would be uniform over the failed area with occasionally a small horizontal movement occurring, See Fig. 4. The normal type of failure was where the fill would fail in a rotational type of movement, the maximum drop being at the crack and small movement occurring at the edge of the fill. The mud was forced up about 20 to 40 feet from the fill and appeared to move in a vertical direction. The surface of the crest of the mud wave had a very rough appearance. Occasionally a vertical face appeared on the side of the mud wave away from the fill. It appeared that these failures followed a circular rotational movement, and are designated as shear type failures. The results of shear circle analysis of these failures indicated an average shearing strength of the soft mud required for stability was 155 pounds per square foot, with a range of from 70 to 210 pounds per square foot.

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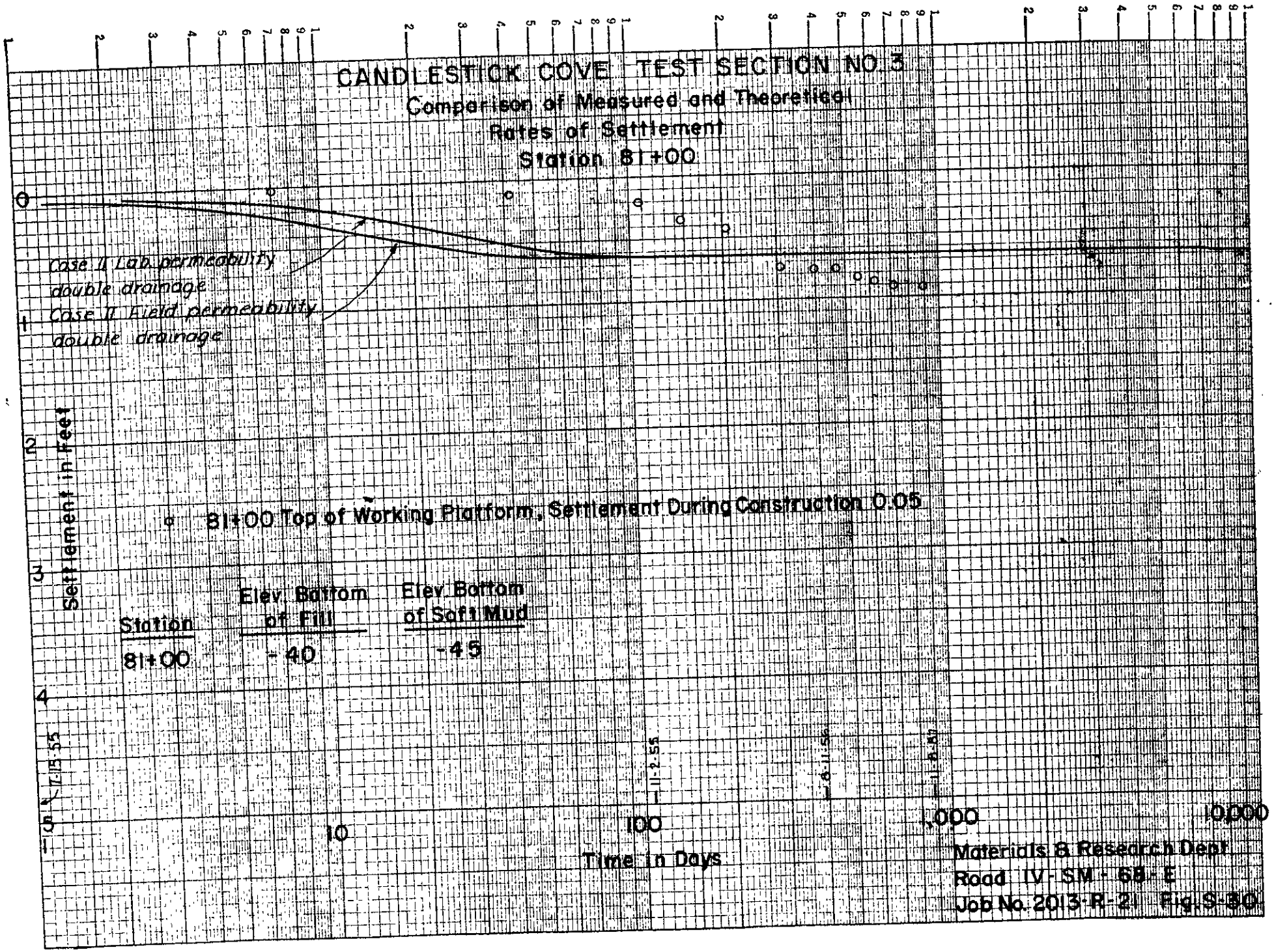


control the evenness of the mud wave built up around the nose, and appeared to control the shape of the cross-section of the fill as determined by borings.

The desirable rate of placing fill is that rate which will maintain the top of the working table at the desired elevation and still slowly advance the fill. The 10,000 cubic yards per day minimum used was sufficient to maintain the working table at the desired elevation and also to advance the nose of the fill.

The type of fill used had an apparent affect upon the starting of a failure. Once a failure had started the type of fill did not appear to have any affect upon the rate at which failure occurred.

The elevation of the top of the working table is related to the driving force in displacing the soft bay mud. It was found that to keep the mud wave moving the top of the working table had to be three to five feet above the crest of the mud wave. To avoid walking over the crest of the mud wave it was necessary to increase the elevation of the top of the working table where high mud waves were built up.

The uncontrollable class included the effect of the tide, strength of the soft bay mud and the extent of the mud wave.

The strength of the soft bay mud was reasonably uniform in the area of the open water fill and exerted a constant affect upon the fill failures.

The tide is of a variable nature at Candlestick Cove, from a maximum of elevation +8 and a minimum of elevation -2. It appears that the weight of the water, at high tide, acted to resist failure of the fill by supporting the mud wave. At low tide the mud wave failed decreasing its support of the fill and generally causing the fill to drop. The tide drop that resulted in fill movement was about three to five feet.

The mud wave was formed as the fill displaced the soft mud. The mud waves that were formed extended 500 to 700 feet from centerline. These mud waves had a crest 20 to 40 feet from the top edge of the fill. The mud wave crest varied from elevation +5 to +16. The back of the crest would slope on about a 5 per cent grade to elevation +2 and then extend out at about a 2 per cent grade.

The mud wave acted as a berm or support for the fill, greatly increasing its resistance to failure. In the completed fill section, with the roadway prism in place, stability analysis indicated that the mud wave contributed about 30 per cent of the resisting force.

The tide had considerable affect upon the height of the crest of the mud wave. As the tide stage rose the crest of the mud wave would build up as the failure of the fill occurred. As the tide stage decreased the mud would start failing within itself and gradually tend to flow away from the fill.

The extent of the mud wave was controlled by the amount of displaced mud. In areas where high mud displacement occurred the mud wave would extend up to 700 feet from centerline.

The strength of the mud wave was determined by means of borings. The shearing strength of the soft mud on the crest of the mud wave was found to vary from 0.05 to 0.3 ton per square foot depending upon the type of failure which had produced the mud wave. This strength was approximately uniform to a depth of elevation -20 and then the strength increased as prior to construction.

It thus appears that three items controlled the size of the mud wave. The quantity of the displaced mud determined the extent of the mud wave. The tides and strength of the soft mud determined the height of the crest of the mud wave. Mud blasting was used to reduce the strength of the soft mud in the mud wave and reduce the elevation of the crest of the mud wave. Dredging behind the mud wave was used at the southern closure where two mud waves from the highway fills and the mud wave from the realignment of the Southern Pacific railroad tracks produced an enormous amount of mud wave.

As construction progressed it was learned how to vary the above items so as to obtain maximum mud displacement. This was done by varying the controllable factors affecting the mud displacement in such a manner as to counteract the uncontrollable factors. A record of the amount of yardage placed at each Station was kept so as to estimate the amount of mud displaced. The major factor affecting the failures, and as a result affecting the mud displacement, was the heights of the crest of the mud wave. The height of the crest of the mud wave was partially determined by the tide which could not be controlled. The strength of the mud in the mud wave was therefore reduced by mud blasting so that the mud wave would tend to flow away from the fill. During periods of high tide and/or high crests on the mud wave the elevation of the top of the working table was raised. The rate of advance of the nose of the fill was maintained between ten and thirty feet per day to keep the mud moving uniformly. This method of controlling the mud displacement was successful in obtaining eighty per cent or greater mud displacement under the roadway prism.

The object of the working table was to enable the construction of a stable roadway prism. There were two factors to be considered; the stability of the roadway prism and settlement of the roadway prism. Both of these factors are dependent upon the amount of soft mud displaced. With mud displacement below elevation -20 no evidence of failures occurred when the roadway prism was placed. With variable mud remaining under the fill differential settlement of the roadway prism could be expected. The simplest solution was to displace as much of the soft mud as possible. The details of the displacement of the soft mud is discussed in Part III of the Appendix.

Variations in Strength of the Soft Bay Mud During and After Construction

Samples of the soft bay mud were taken from under the fill at four locations at various times before, during, and after construction. The variation of strength, moisture, and compressibility of the soft mud were determined.

The samples obtained prior to construction indicated that the shearing strength of the soft mud increased 14 pounds per square foot per foot of depth with zero strength at elevation -5. This strength was uniform for the soft bay mud throughout Candlestick Cove area.

The stability analysis of the failures indicated a shearing strength of 70 to 210 pounds per square inch for the shear type failures and 200 to 300 pounds per square inch for the squeeze type failures. Samples obtained from the mud waves, at various locations, soon after the fill became stable indicated shearing strengths ranging from 50 to 200 pounds per square foot. The higher strengths were obtained where shear failures had occurred and the lower strengths where squeeze failures had occurred.

The borings through the working table, made immediately after completion of the working table, indicated strength about 200 pounds per square foot immediately below the fill with the strength increasing with depth. The indications are that the strength of the soft mud was greatly reduced by remolding as the failures occurred. The shearing strength of the laboratory remolded samples of the soft mud varied from 50 to 100 pounds per square foot.

The borings made at various times after completion of the fill indicate a gradual increasing strength with time. Figure No. 5 shows the results obtained at two locations. The time required for the soft bay mud to regain its original strength appears to depend upon the degree of remolding that occurred in the soft mud, and varied from one to two years. As further consolidation has occurred the shearing strength of

the soft mud increased and is now exceeding its original strength by 100 pounds per square foot or more. This increase in strength of the soft mud will improve the stability of the fill. The strength of the soft bay mud is discussed in detail in Part IV of the Appendix.

Variations in Moisture Content of the Soft Bay Mud During and After Construction

The moisture content of the soft bay mud varied from 90 per cent at elevation -5 to about 60 per cent at elevation -60. The moistures were scattered in a randum manner over a range of 20 per cent with the moistures tending to decrease with depth as the only trend. Moistures taken after construction show a smaller range of moisture contents and the moistures are grouped along the lower limits of the original moisture content spread. This would indicate that a small, an average of about 5 per cent, decrease in moisture content has occurred. The theoretical decrease in moisture content under full consolidation is about ten to fifteen per cent. As the point to point variation in moisture content is of this magnitude it will be extremely difficult to determine an accurate decrease in moisture content by means of borings. The moisture content of the soft bay mud is discussed in detail in Part IV of the Appendix.

Consolidation Tests on the Soft Bay Mud

Consolidation tests were run on samples from all series of borings made. The samples prior to construction indicated that the soft mud was fully consolidated under its loading. These curves indicated a permeability of 5×10^{-5} feet per hour and a coefficient of consolidation of 5.8×10^{-3} square feet per hour. The samples obtained after the construction of the working table indicated a coefficient of consolidation of 3×10^{-3} square feet per hour and a permeability of 1.1×10^{-5} feet per hour. This is probably due to the remolding of the soft mud during the fill failures. The consolidation tests are discussed in detail in Part IV of the Appendix.

Permeabilities of the Soft Bay Mud

The permeabilities of the soft bay mud were determined by two methods. The two methods were from the consolidation test and using the piezometers as rising head permeameters.

The average permeability obtained from the consolidation tests after construction of the working table was 1.1×10^{-5} feet per hour. The load increment used was the one immediately larger than the original overburden at the depth the sample was obtained. It is felt that this figure represents the vertical permeability of the soil.

The permeabilities indicated by the piezometers average 2.2×10^{-5} feet per hour. This permeability is felt to represent the horizontal permeability of the soft bay mud. The indications are that the horizontal permeability is twice the vertical permeability, which would indicate a fairly homogeneous soil. The details of the permeability studies are included in Part IV of the Appendix.

Theoretical Settlements

There are three soil layers with varying degrees of compressibility at Candlestick Cove underlying the roadway prism. The settlement of the roadway prism is a function of the rates of settlement of all three layers. The layers are the uncompacted fill, the soft bay mud remaining below the fill and the stiff bay mud underlying the soft bay mud.

It was not possible to obtain "undisturbed" samples of the uncompacted fill; therefore, no theoretical ultimate or rates of settlements could be calculated. Two general types of uncompacted fill was used, rocky borrow and soil type borrow. Both types of borrow are pervious and would be expected to consolidate rapidly.

Theoretical rates of settlement of the soft bay mud were calculated for numerous locations. The settlement was calculated using the consolidation tests from the samples obtained after completion of the working table, the permeabilities obtained from the piezometers, and assuming double drainage. A few typical theoretical settlement curves are shown in Figure No. 6. The theoretical settlement indicates that the rate of settlement will be approximately uniform regardless of the thickness of the soft mud remaining below the fill. When the calculations for very large mud displacement indicate that the settlement is completed, the curves for small mud displacement indicate that settlement will continue at these locations with the same slope to the semi-log time settlement curves until their ultimate settlement has been reached. The differential settlement would thus be small until the consolidation is completed for the least thickness of soft bay mud and then the thicker mud locations would continue to settle resulting in differential settlement.

Theoretical settlements of the stiff bay mud were calculated assuming double drainage even though the stiff bay mud is underlain by the Franciscan formations. The reason for this is that extensive sand layers or pockets exist in this stiff mud layer, and they probably provide some drainage. The calculated ultimate settlements of this layer are rather large, being from two to four feet. The estimated theoretical rate of settlement is very slow, with an estimated one to two hundred years being required for its completion. The theoretical settlements are discussed in detail in Part V of the Appendix.

Measured Settlements

As placement of the working table progressed, settlement platforms were placed on centerline at the surface of the fill at each station. These settlement platforms will measure the total settlement of the roadway prism. In the second unit five settlement platforms were placed at the bottom of the uncompacted fill. The difference in settlement between the surface settlement platforms and the settlement platforms at the bottom of the uncompacted fill will measure the consolidation within the uncompacted fill. To check the settlement of the stiff bay mud layer four settlement platforms were installed at the surface of the stiff mud layer.

The total settlement curves for the first, second and fourth units are very similar. The slope of the log time settlement curves is about the same and indicate that only small differential settlement between any two stations is now occurring. The total settlement to date for both units is between three and five feet. The slope of the log time settlement curves increased during the placing of the roadway prism. After completion of the roadway prism the slope of the log time settlement tended to decrease for a short period of time and then slowly increased and is now continuing along a straight line. The mud displacement in these units varies widely, from small to total mud displacement.

The total settlement curves for the third unit indicate that small settlement occurred during the placing of the roadway prism. The slope of the log time settlement curve is a constant during construction of the roadway prism and for about six months after construction. About six months after construction the slope of the log time settlement curve sharply increases for about three months, generally resulting in about one-half foot of settlement. Where almost total mud displacement occurs, less than ten± feet of soft mud remaining below the fill, the curves tend to return to the original slope on the semi-log plot. Where small mud displacement occurred, more than ten± feet of soft mud remaining below the fill, the settlement tends to continue along the increased slope on the log time settlement curve. The above statements are general only with exceptions to be found. The slope of the log time settlement curves after construction of the roadway prism is not uniform in the third unit. The total settlement to date varies from one-half to four feet. Although a large mud displacement was obtained throughout most of this unit the settlements are not as consistent as in the other units.

In the fifth unit the settlement platforms were not installed until after completion of the roadway prism. In the fifth unit there are insufficient data to date to indicate any trend.

The settlement due to the uncompacted fill has varied widely. The consolidation of the uncompacted fill is generally small and appears to be completed within a year after completion of the roadway prism. The rock fill appears to consolidate about 0.1 foot and the soil fill about one foot under the loading of the roadway prism.

Settlements of the surface of the stiff bay mud were only measured at four locations, and because of this very limited data only general trends can be noted. The slope of the measured settlement curves was small after completion of the roadway prism and is now increasing. The present settlement of the surface of the stiff bay mud has exceeded the estimated settlement that was estimated to take twenty years to occur. This accelerated settlement is probably due to the existence of the sand beds or pockets throughout this layer. The measured settlements are discussed in detail in Part V of the Appendix.

Remarks on Settlement of the Roadway Prism

As the roadway prism is a compacted fill of less than ten feet thickness and under no appreciable loading, only minor consolidation would be expected within this roadway prism. The settlement of the surfaced roadway and the roadway prism are assumed to be the same, and the settlement of the roadway prism will indicate the settlement of the surfaced roadway. The settlement of the roadway prism is dependent upon the three layers underlying it.

Consolidation of the uncompacted fill progressed rapidly during the placing of the roadway prism and now appears to be essentially completed. The consolidation of the soft bay mud started after completion of the roadway prism and the slope of the log time settlement curve is large. The stiff bay mud surface settled a small amount during construction and for a short period after completion of the roadway prism and is now settling at an increased slope on the semi-log plots. Settlement of the roadway prism can be predicted only if all three of these layers are considered.

Where only a small mud displacement occurred the principle layer governing the settlement of the roadway prism after construction is the soft mud layer. The settlement due to consolidation of the stiff bay mud is small but due to its increasing slope on the semi-log time settlement plots may indicate an increasing slope on the semi-log plots for settlement of the roadway prism.

Where total or nearly total mud displacement occurred the principle layer governing the rate of settlement will be the stiff mud layer. The assumption is made that the uncompacted fill is no longer consolidating. At two locations

in the third unit, Stations 69 and 81, there are settlement platforms at the surface of the stiff bay mud and at the surface of the uncompacted fill where total mud displacement occurred. During 1957 the settlements of these platforms indicated that the surface of the uncompacted fill was closely following the settlement due to the stiff bay mud.

With mud displacements between these two extremes an irregular settlement pattern occurs. The settlement of the roadway prism is dependent upon both the soft bay mud and the stiff bay mud.

Considering all of the factors it appears that continual settlement of the roadway prism at a constant slope on the semi-log plots can be expected. Differential settlements will probably be small in the next few years; then they can be expected to increase. The settlements will continue at some locations for the next fifty years or more, with ultimate settlement from one to eight feet above those which existed at the time of paving.

Excess Hydrostatic Pressure Readings

Piezometers were installed in the first and second units. The piezometers were of the non-metallic type developed by Professor A. Casagrande for use at Logan Airport. In the first unit all piezometers failed during or shortly after placing of the roadway prism. The piezometers in the second unit are all functioning at the present time.

In the first unit two piezometers were placed in the soft mud remaining below the fill on centerline at Stations 8+00 and 14+00. The excess hydrostatic pressure indicated in the soft bay mud was equal to or greater than the increase in loading by the fill and decreased 0.05 to 0.10 tons per square foot in the sixty-day period prior to the start of the roadway prism. During the placing of the roadway prism the excess hydrostatic pressure increased in an amount equal to the increase in loading due to the roadway prism. In the three-month period after placing the roadway prism the excess hydrostatic pressure decreased 0.3 ton per square foot. By this time both piezometers on centerline had ceased to function. At Station 14+00 two piezometers were placed in the soft mud approximately below the shoulder line of the roadway prism and at the edge of the fill. The increase in excess hydrostatic pressure in the soft mud below the shoulder line of the roadway prism after placing the roadway prism was about sixty per cent of the increase in loading on centerline. The increase in excess hydrostatic pressure at the edge of the working table after placing the roadway prism was about twenty per cent of the increase in loading on centerline. Two piezometers were placed in the weathered shale and in the clayey sand underlying the soft bay mud. These piezometers indicated a slight rise in excess

hydrostatic pressure during the placing of the roadway prism and then decreased to 0.1 to 0.2 tons per square foot pressure after completion of the roadway prism. This data confirmed the belief that drainage was being provided at the bottom of the soft bay mud layer.

The piezometers in the second unit were laid out using the results of the first unit as a guide, so as to obtain a half cross-section of the excess hydrostatic pressure for every three hundred feet of fill length. A total of four cross-sections were installed using a total of twenty piezometers. Two additional piezometers were installed in the sand to clayey sand layer between the stiff bay mud and the soft bay mud.

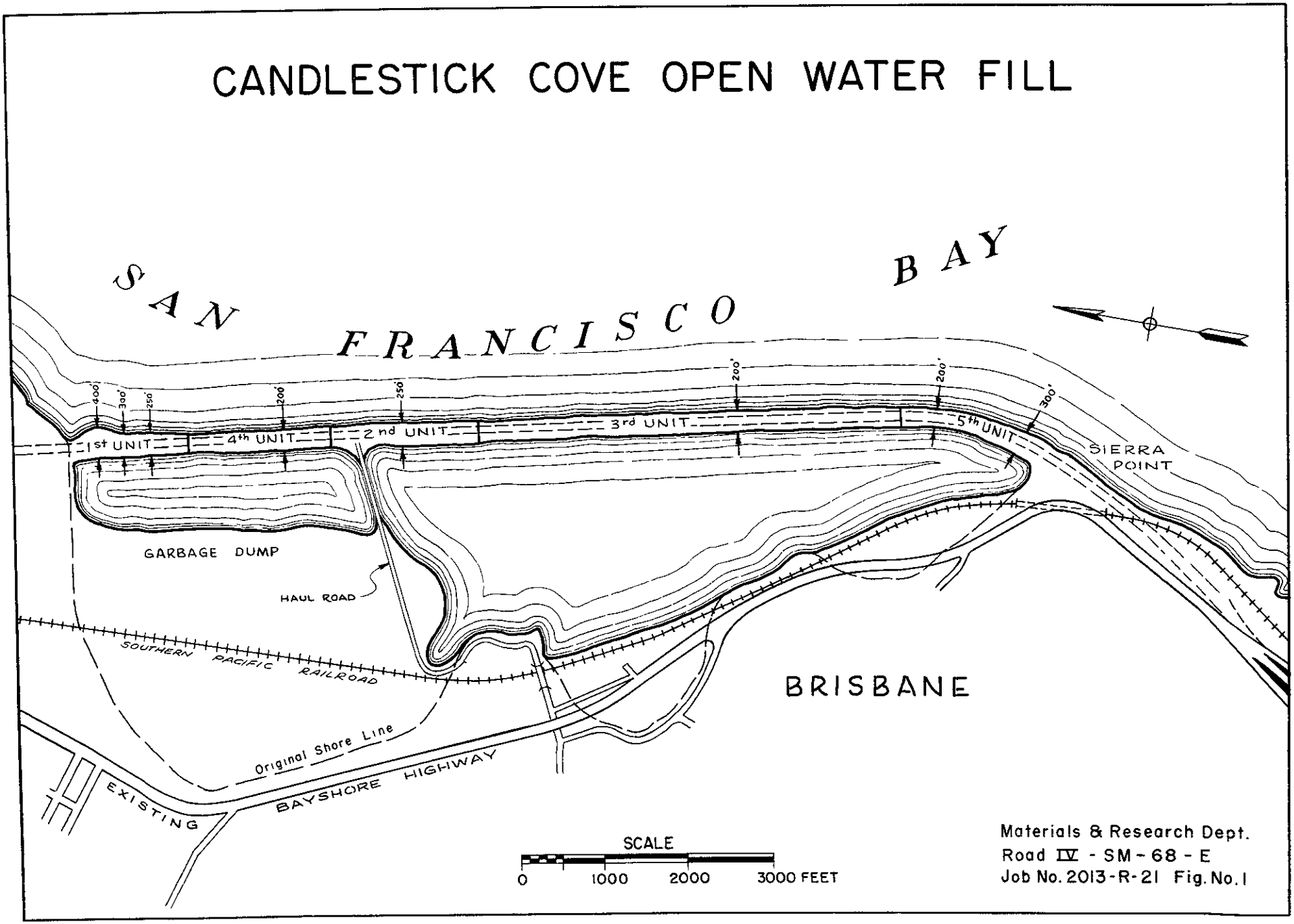
During the period between installing the piezometers in the soft bay mud and the placing of the roadway prism a slight decrease, less than 0.1 ton per square foot, in excess hydrostatic pressure occurred. During the placing of the roadway prism the loading was increased 0.6 ton per square foot. This increase in weight was reflected by a 0.5 to 0.6 tons per square foot rise in the centerline excess hydrostatic pressure and small, 0.1 to 0.2 ton per square foot rise 75 to 80 feet from centerline, but with no noticeable effect under the mud wave. After completion of the roadway prism the excess hydrostatic pressure at centerline decreased slowly from about two tons per square foot to about one ton per square foot at the present time. The excess hydrostatic pressure outside the toe of the roadway prism, 75 to 80 feet from centerline, decreased at a slower rate on the semi-log plots than at centerline. The decrease in excess hydrostatic pressure below the mud wave has been very small, 0.1 to 0.2 ton per square foot, and has been very erratic due to tidal effects. The two piezometers placed in the sand to clayey sand layer between the soft bay mud and stiff bay mud indicate that a constant excess hydrostatic pressure of about 0.2 ton per square foot exist in this layer.

The piezometers often indicated pressures that were in excess of the loading due to the fill above the piezometers. The loading at various points within one hundred feet of the piezometers were investigated, and it was found that the piezometers tended to indicate the maximum pressure within this distance. The result was a fairly uniform excess hydrostatic pressure that is not reflecting the extremes of the soft mud displacement.

The excess hydrostatic pressure measurements indicate that a rapid decrease in pressure occurred in areas of large mud displacement until the pressures were about equal to the pressures below adjacent smaller mud displacement areas. Then excess hydrostatic pressures in the two areas decreased at about the same rate. This effect appears to exist over

an area of two to three hundred feet and is not reflected in the rate of consolidation, as measured by the vertical movements. It appears that two actions may be occurring that are affecting the settlement and excess hydrostatic measurements. The excess hydrostatic pressures are being transferred in a horizontal direction up to two hundred feet and the fill is acting as an arch or bridge for a distance of two to three hundred feet in the locations of small mud displacement. The excess hydrostatic pressure results are discussed in detail in Part VI of the Appendix.

CANDLESTICK COVE OPEN WATER FILL



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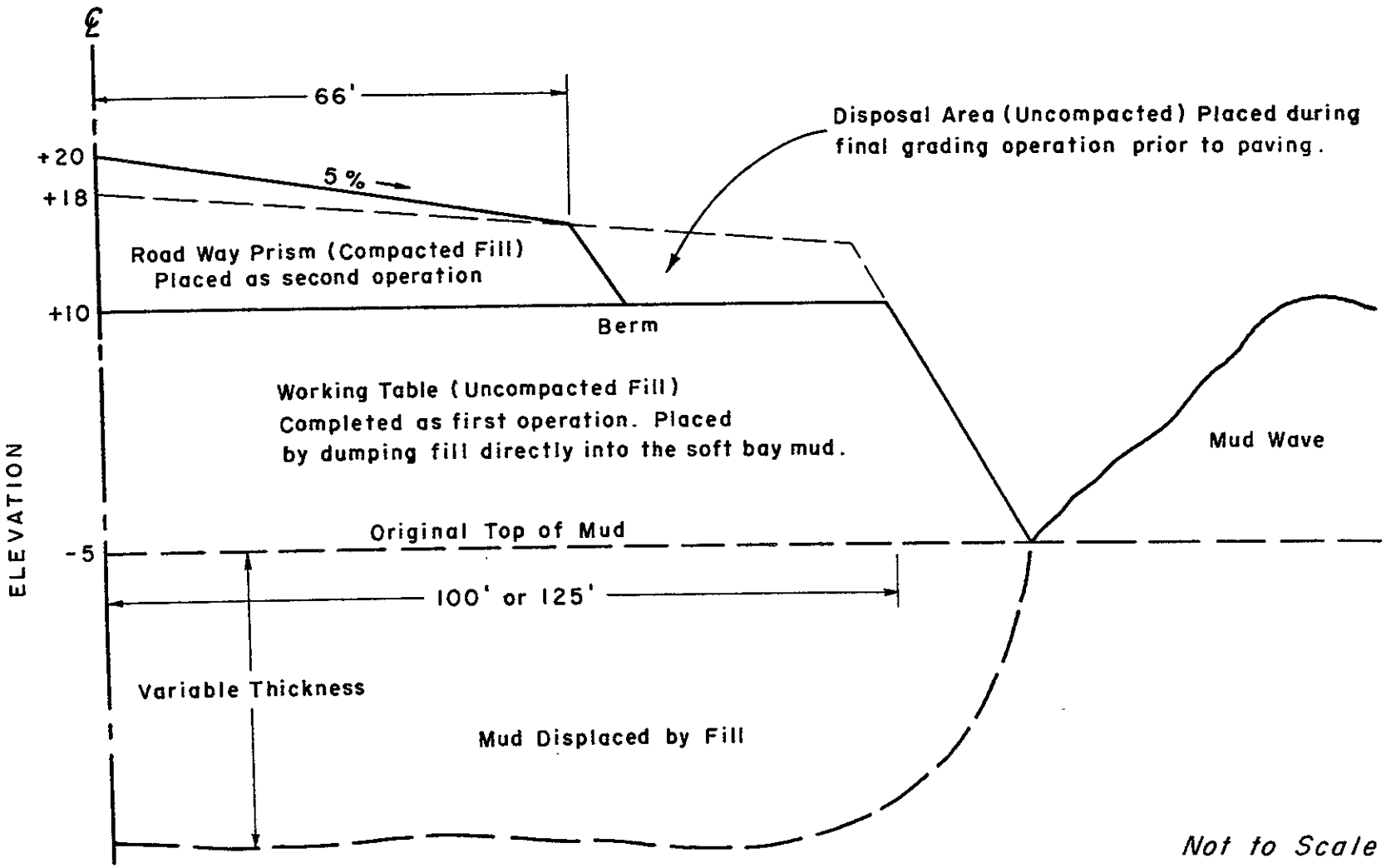
Road IV - SM - 68 - E
Job No. 2013-R-21 Fig. No. 1

CANDLESTICK COVE
Typical Cross Section
Half Section - Symmetrical About ℄



Not to Scale

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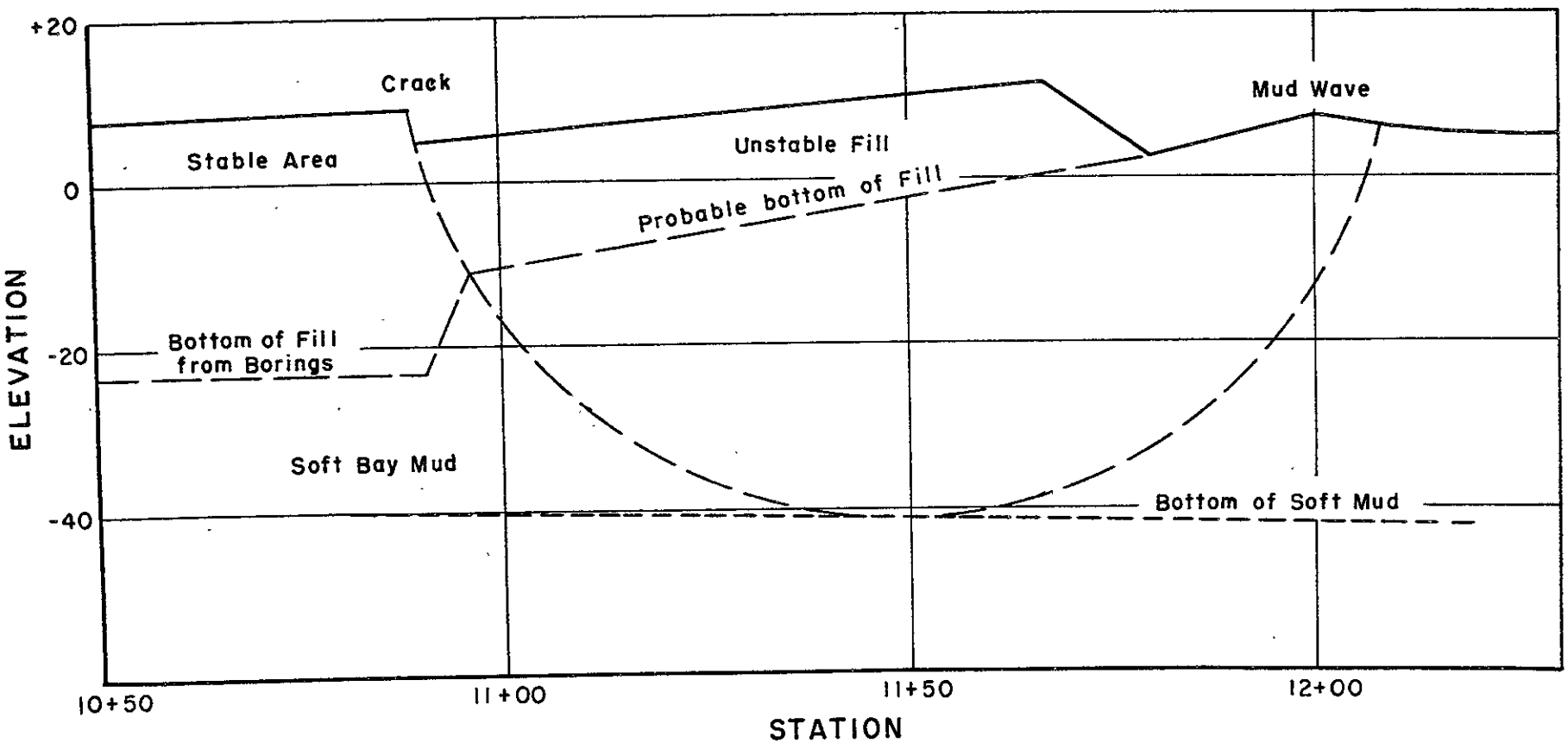
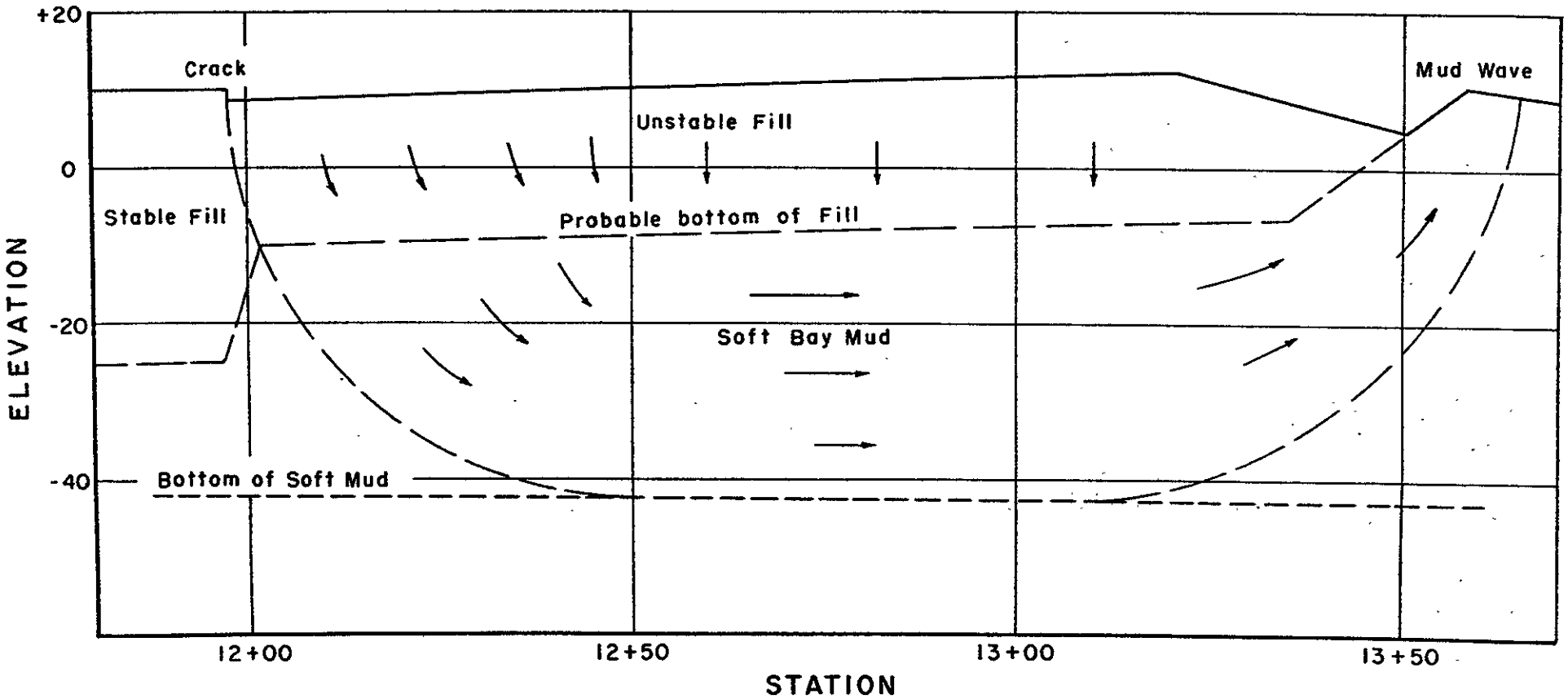
SHEAR TYPE FAILURE
Failure on center line over the weekend of May 10, 1952
Profile Along Centerline



Road IV - SM - 68 - E
Candlestick Cove
Fig. No. 3

SQUEEZE TYPE FAILURE

Profile Along Centerline



Road IV - SM - 68 - E
Candlestick Cove
Fig. No. 4

VARIATION OF SHEARING STRENGTH WITH DEPTH

Candlestick Cove

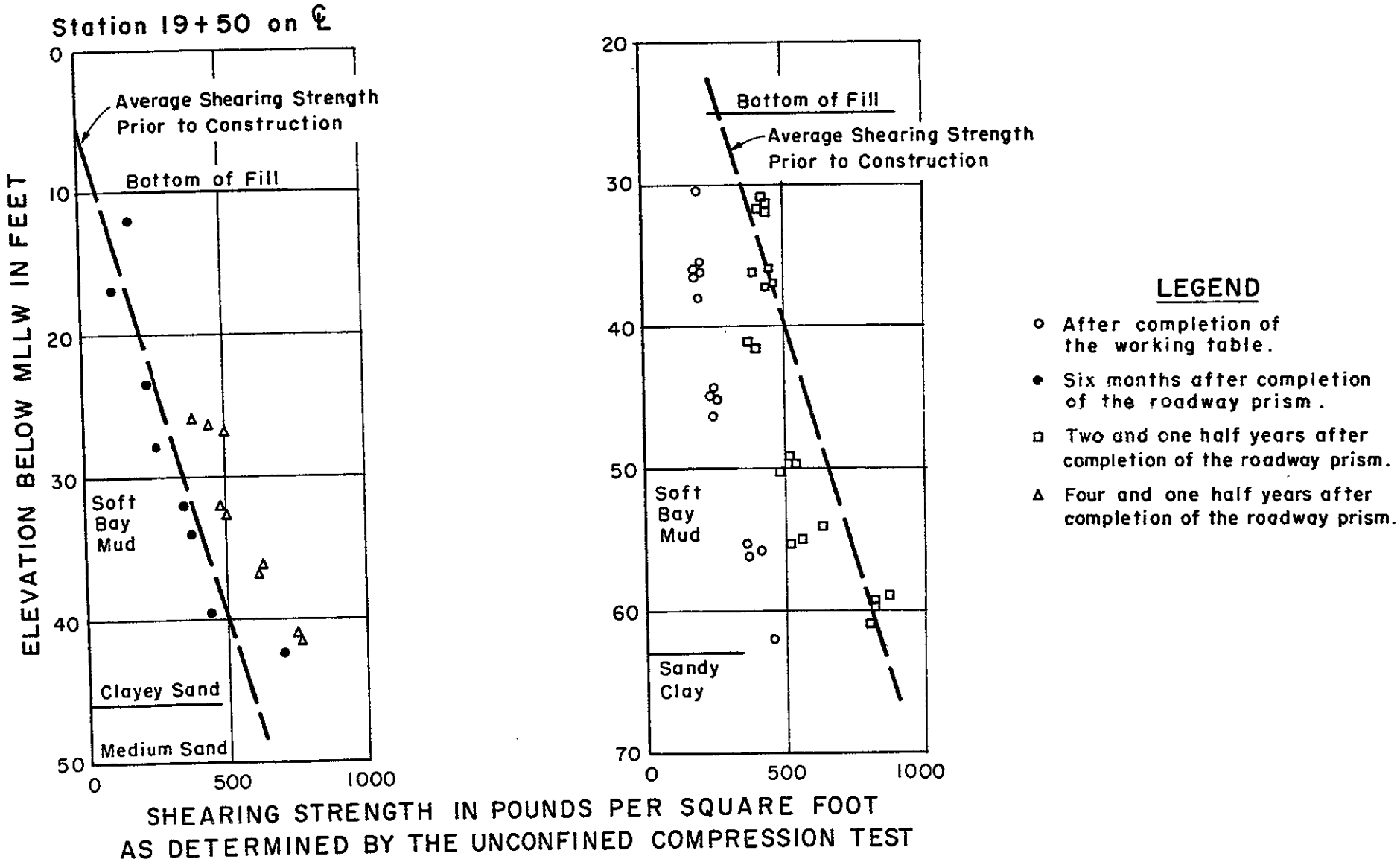


Fig. No. 5

THEORETICAL RATES OF SETTLEMENT

Candlestick Cove

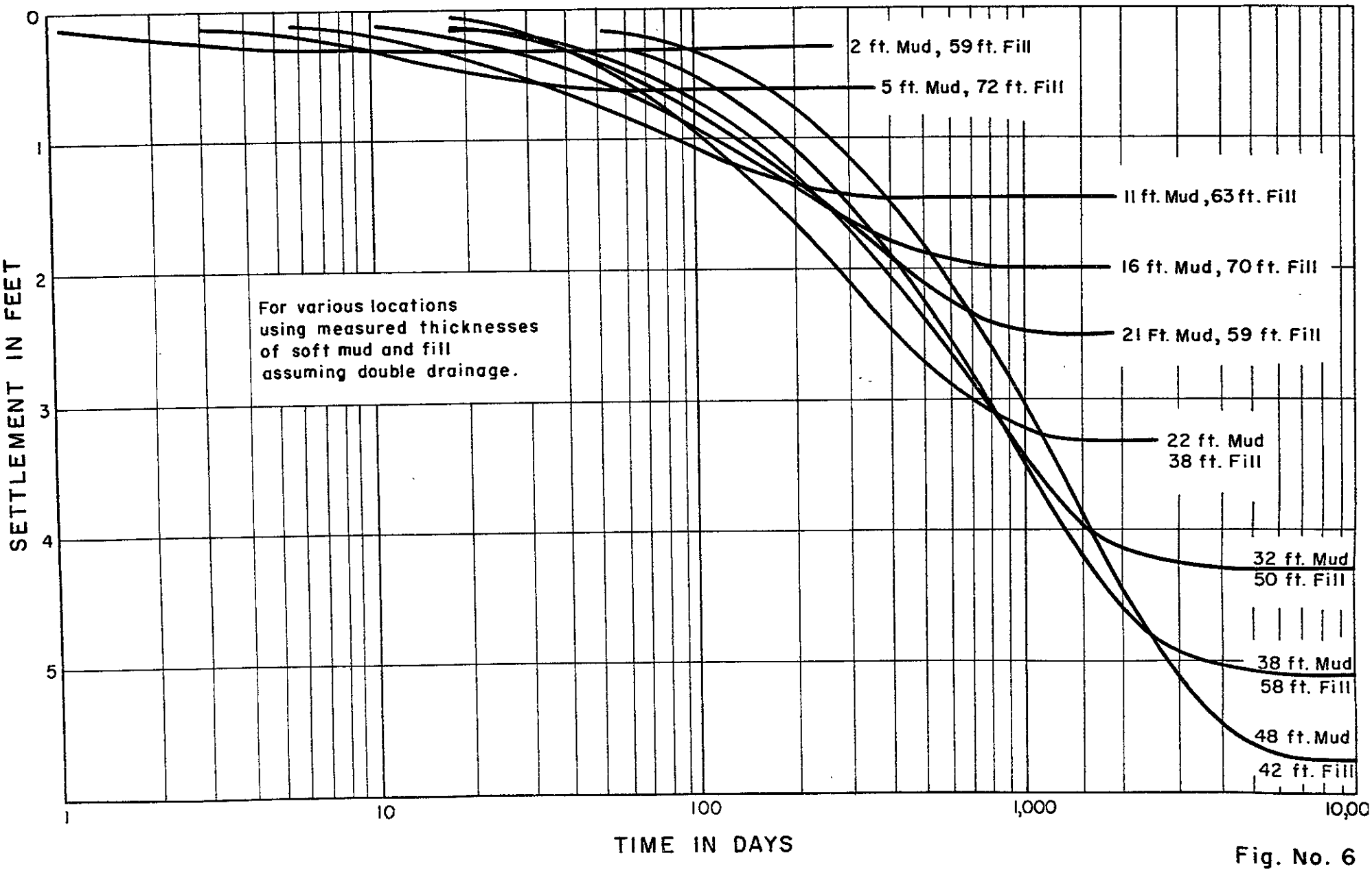
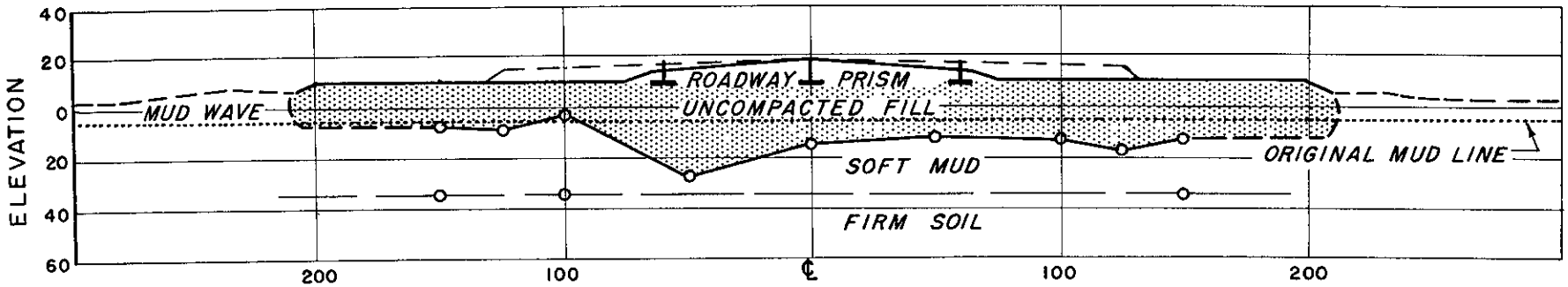


Fig. No. 6

CANDLESTICK COVE OPEN WATER FILL, 1st UNIT

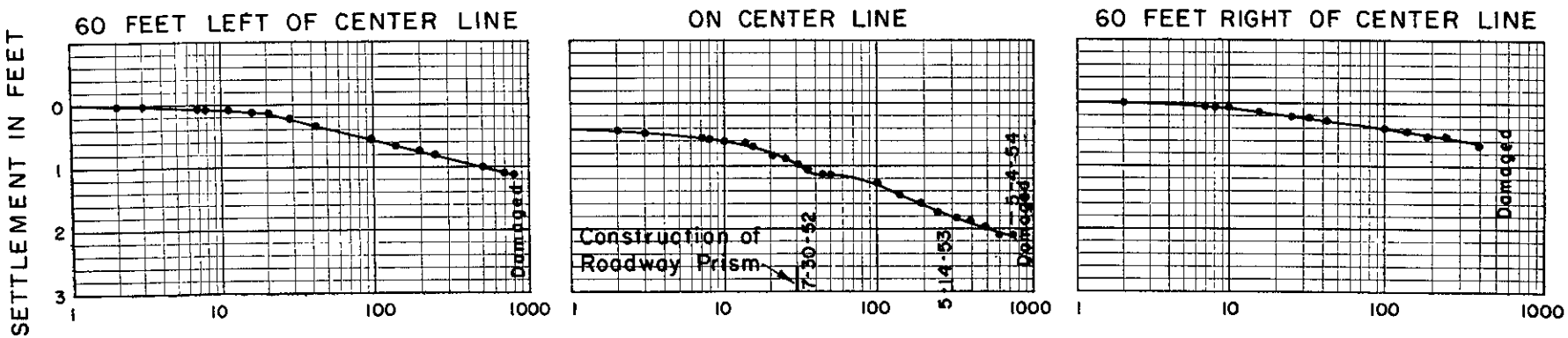
400 FEET WIDE



CROSS-SECTION AT STATION 8+00

o - Soil change determined by means of boring.

⊥ - Settlement Platform

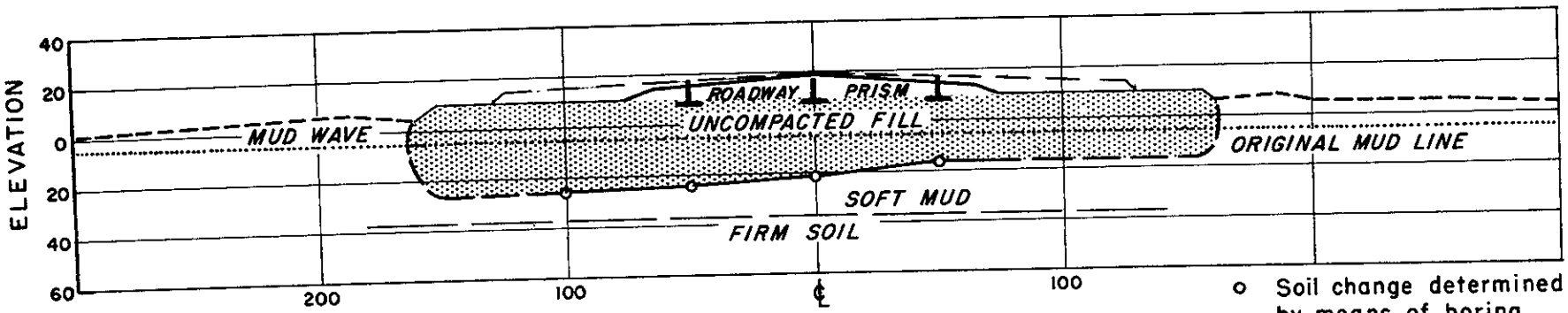


TIME IN DAYS AFTER START OF ROADWAY PRISM

SETTLEMENTS AT STATION 8+00

CANDLESTICK COVE OPEN WATER FILL, 1st UNIT

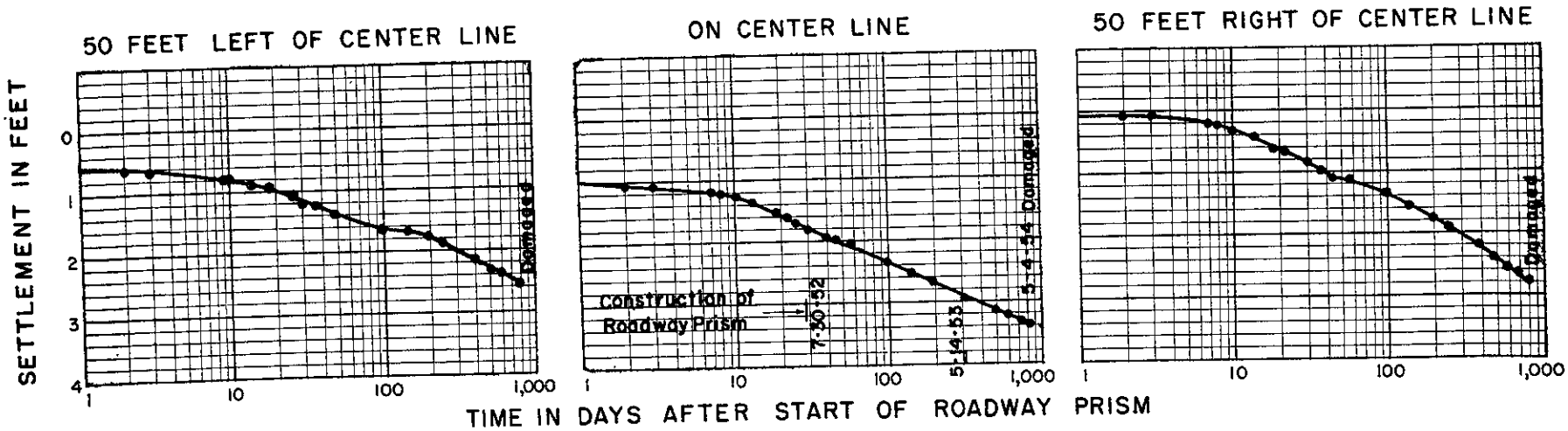
300 FEET WIDE



CROSS-SECTION AT STATION 11+00

o Soil change determined by means of boring.

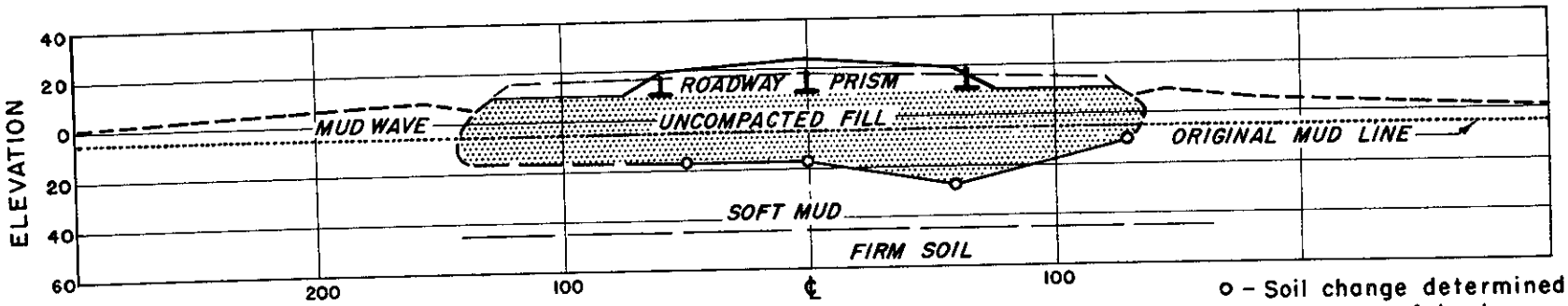
⊥ Settlement Platform



SETTLEMENTS AT STATION 11+00

CANDLESTICK COVE OPEN WATER FILL, 1st UNIT

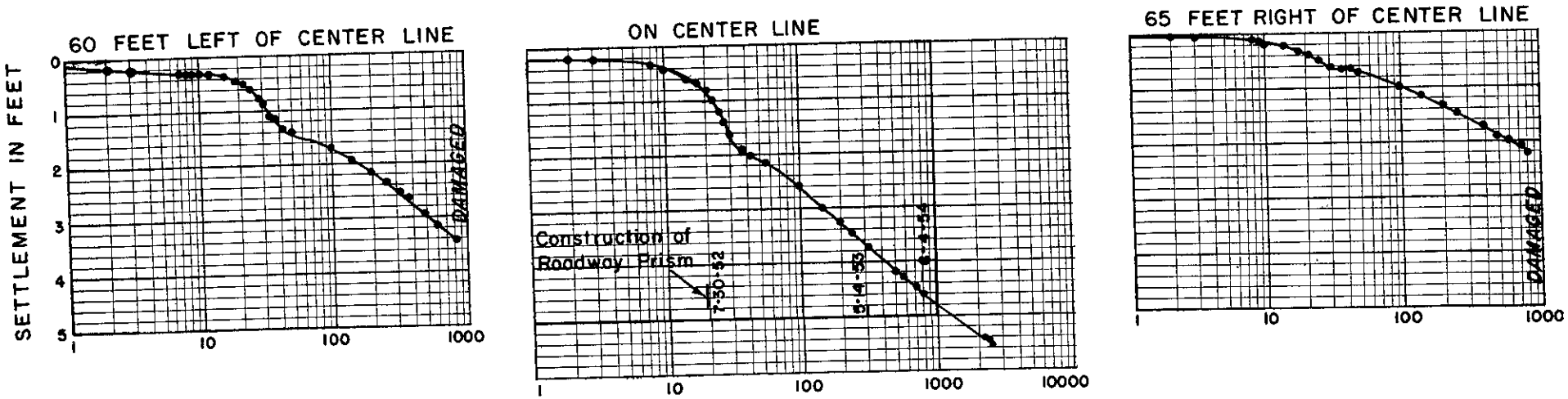
250 FEET WIDE



CROSS-SECTION AT STATION 14+00

o - Soil change determined by means of boring.

⊥ - Settlement Platform

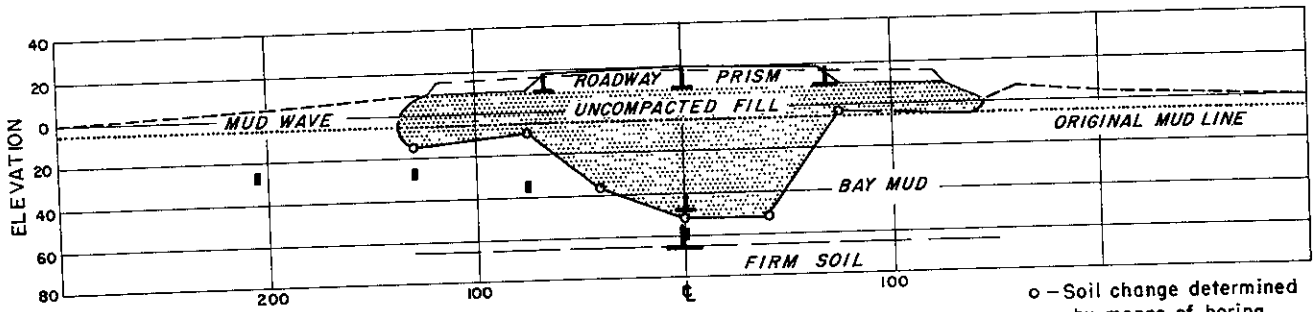


TIME IN DAYS AFTER START OF ROADWAY PRISM

SETTLEMENTS AT STATION 14+00

CANDLESTICK COVE OPEN WATER FILL, 2nd UNIT

250 FEET WIDE

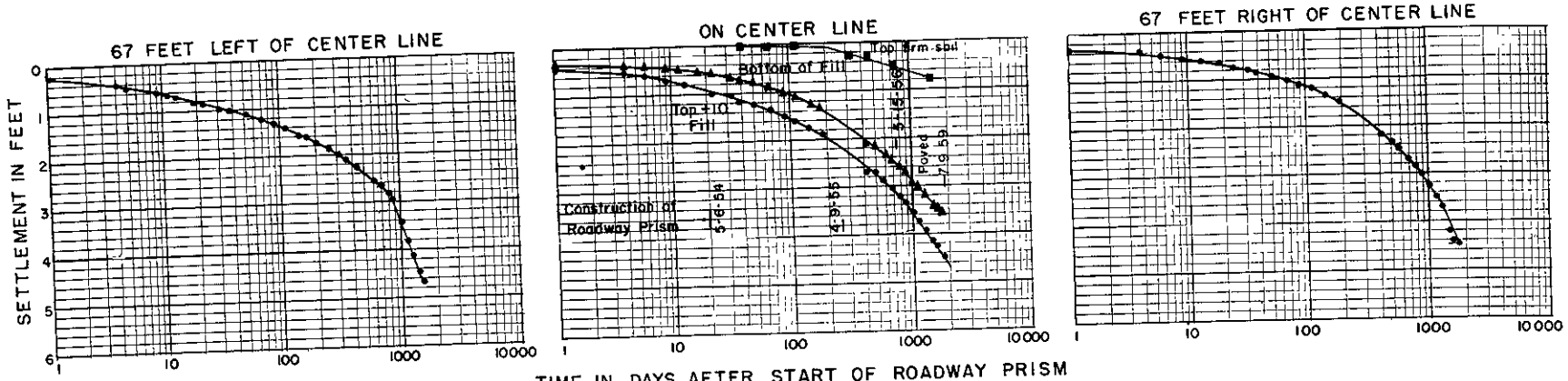


CROSS-SECTION AT STATION 44+00

o – Soil change determined by means of boring.

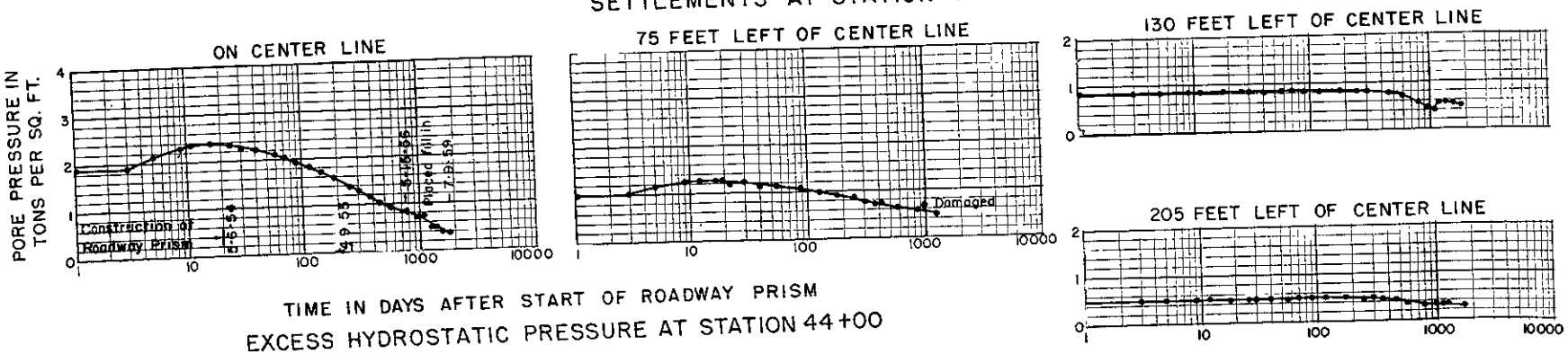
⊥ – Settlement Platform

▮ – Piezometer



TIME IN DAYS AFTER START OF ROADWAY PRISM

SETTLEMENTS AT STATION 44+00

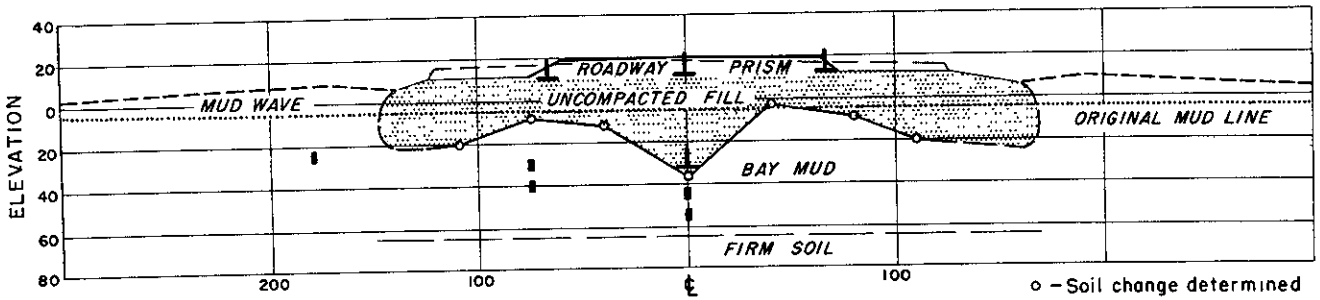


TIME IN DAYS AFTER START OF ROADWAY PRISM

EXCESS HYDROSTATIC PRESSURE AT STATION 44+00

CANDLESTICK COVE OPEN WATER FILL, 2nd UNIT

250 FEET WIDE

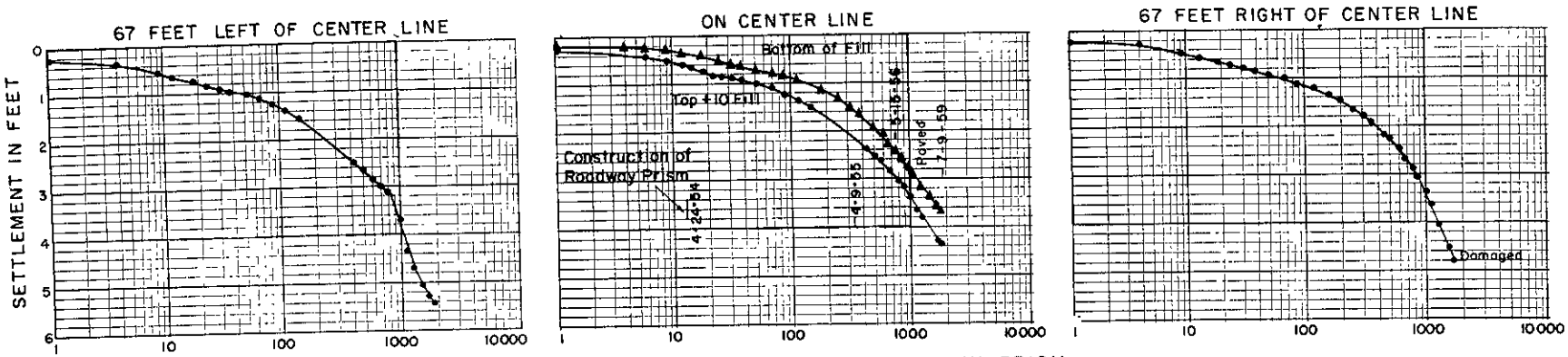


CROSS-SECTION AT STATION 47 + 00

o – Soil change determined by means of boring.

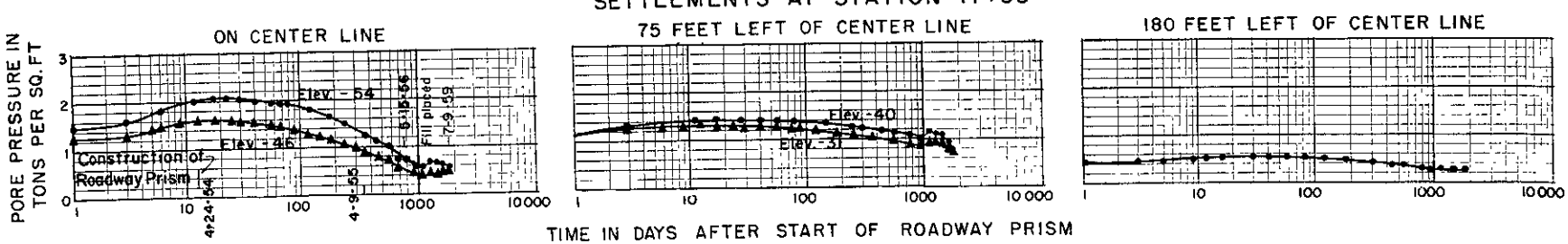
⊥ – Settlement Platform

▮ – Piezometer



TIME IN DAYS AFTER START OF ROADWAY PRISM

SETTLEMENTS AT STATION 47+00

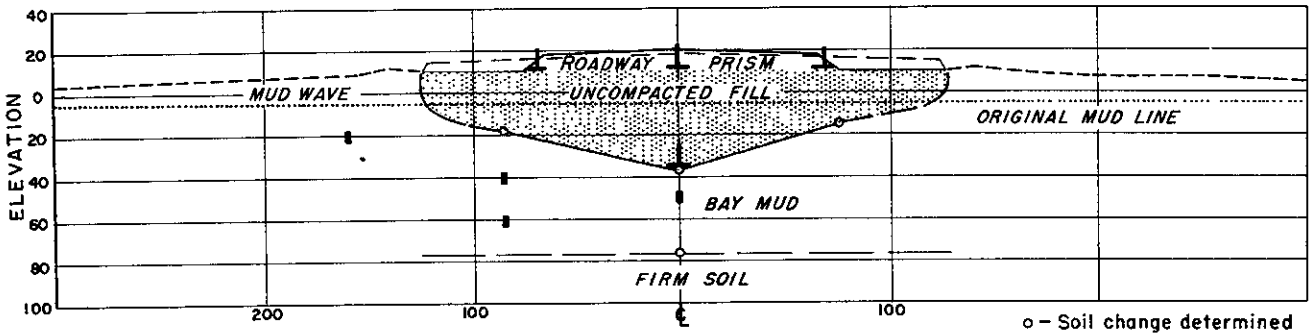


TIME IN DAYS AFTER START OF ROADWAY PRISM

EXCESS HYDROSTATIC PRESSURE AT STATION 47+00

CANDLESTICK COVE OPEN WATER FILL, 2nd UNIT

250 FEET WIDE

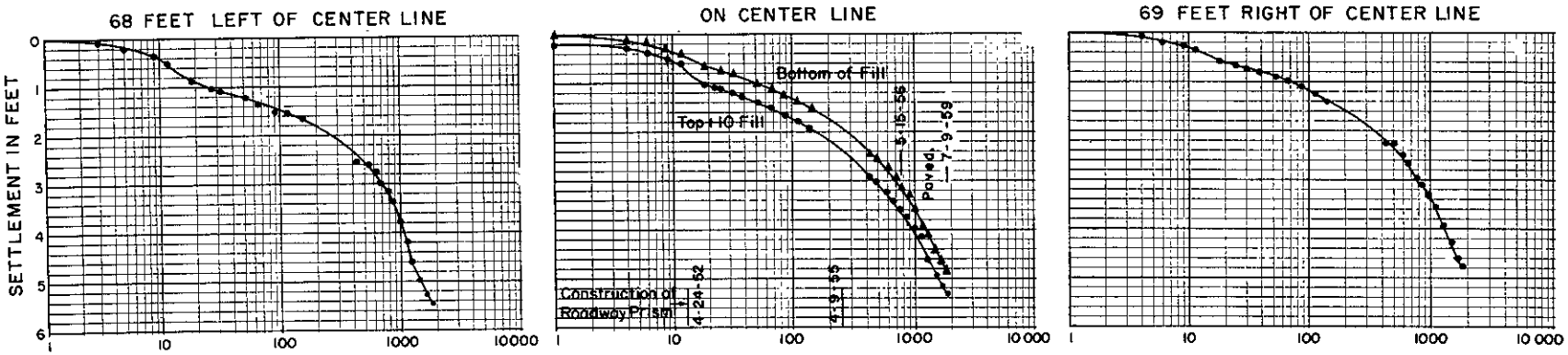


CROSS-SECTION AT STATION 50+00

o – Soil change determined by means of boring.

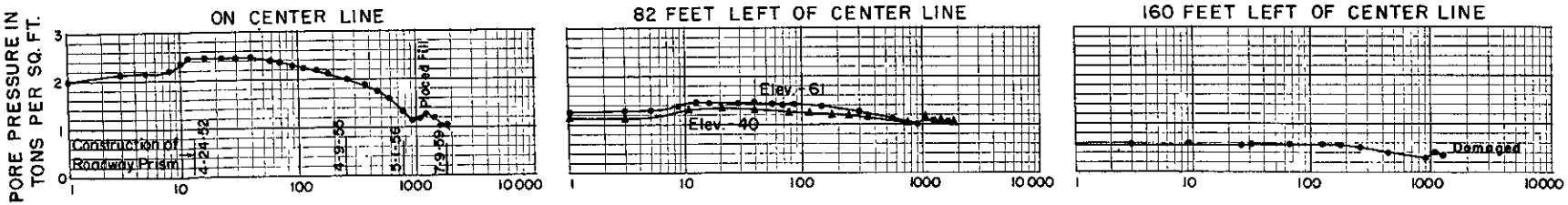
⊥ – Settlement Platform

▮ – Piezometer



TIME IN DAYS AFTER START OF ROADWAY PRISM

SETTLEMENTS AT STATION 50+00

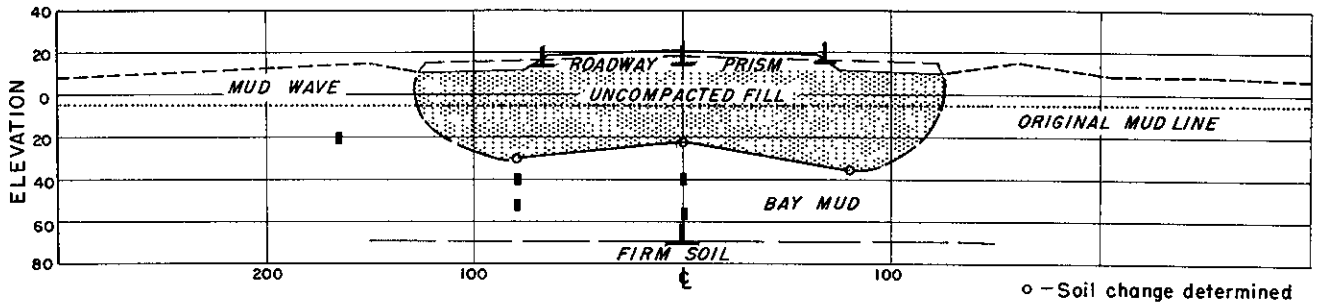


TIME IN DAYS AFTER START OF ROADWAY PRISM

EXCESS HYDROSTATIC PRESSURE AT STATION 50+00

CANDLESTICK COVE OPEN WATER FILL, 2nd UNIT

250 FEET WIDE

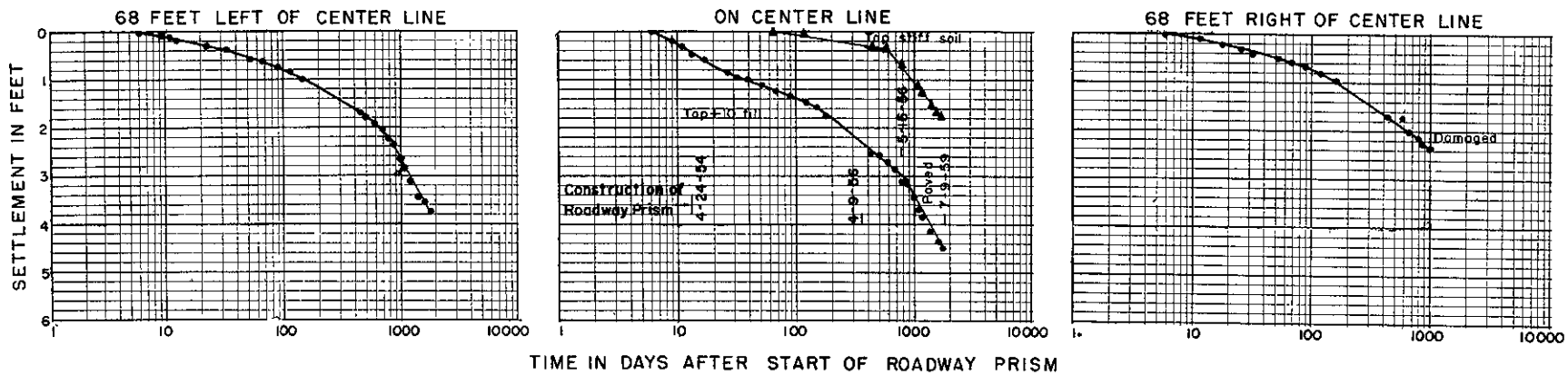


CROSS-SECTION AT STATION 53+00

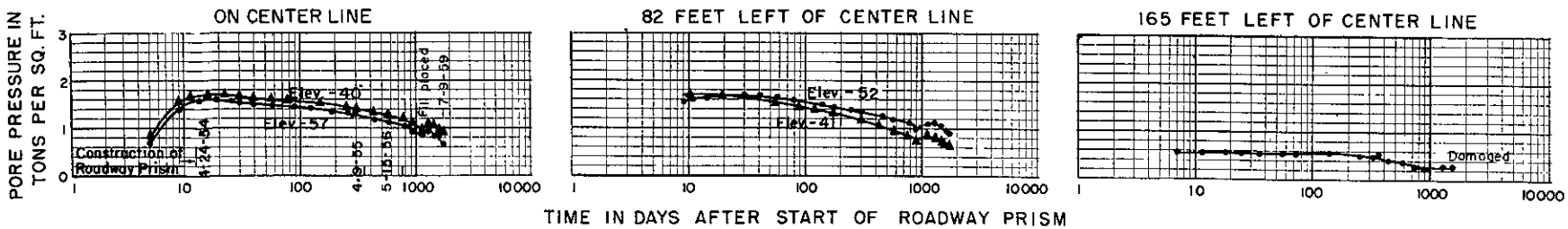
o – Soil change determined by means of boring

⊥ – Settlement Platform

▮ – Piezometer



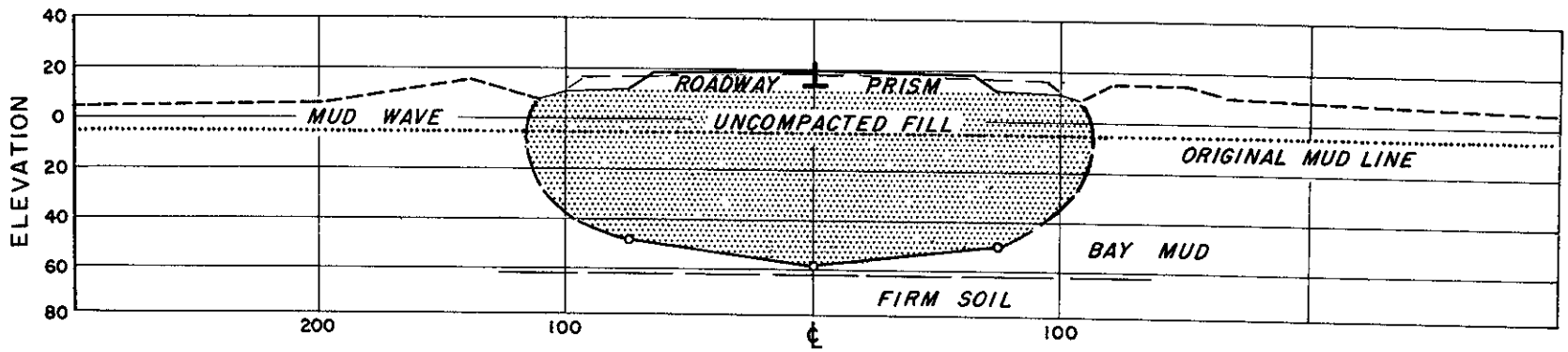
SETTLEMENTS AT STATION 53+00



EXCESS HYDROSTATIC PRESSURE AT STATION 53+00

CANDLESTICK COVE OPEN WATER FILL, 3rd UNIT

200 FEET WIDE

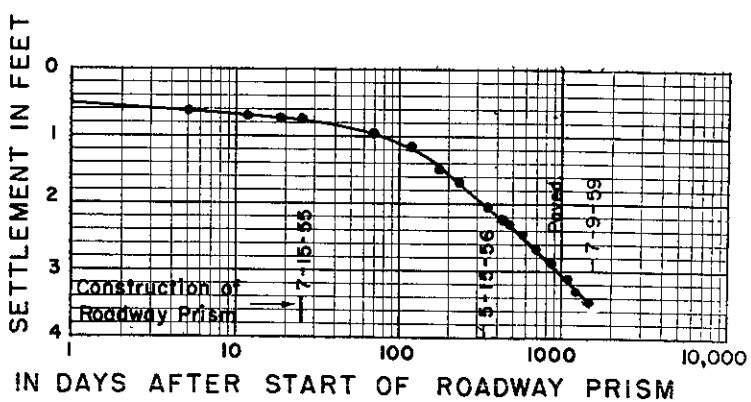


CROSS-SECTION AT STATION 61+00

o -Soil change determined by means of boring.

⊥ -Settlement Platform

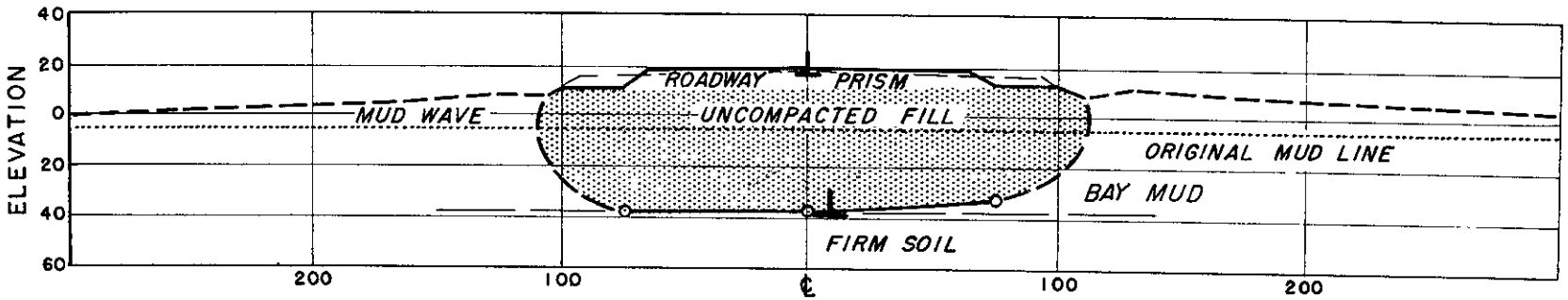
ON CENTER LINE



SETTLEMENT AT STATION 61+00

CANDLESTICK COVE OPEN WATER FILL, 3rd UNIT

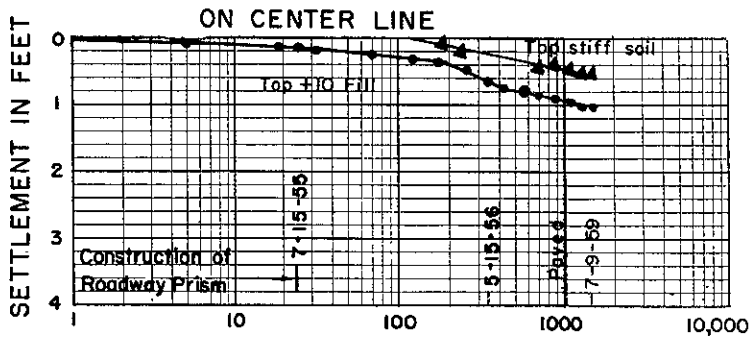
200 FEET WIDE



CROSS-SECTION AT STATION 71+00

o - Soil change determined by means of boring.

⊥ - Settlement Platform

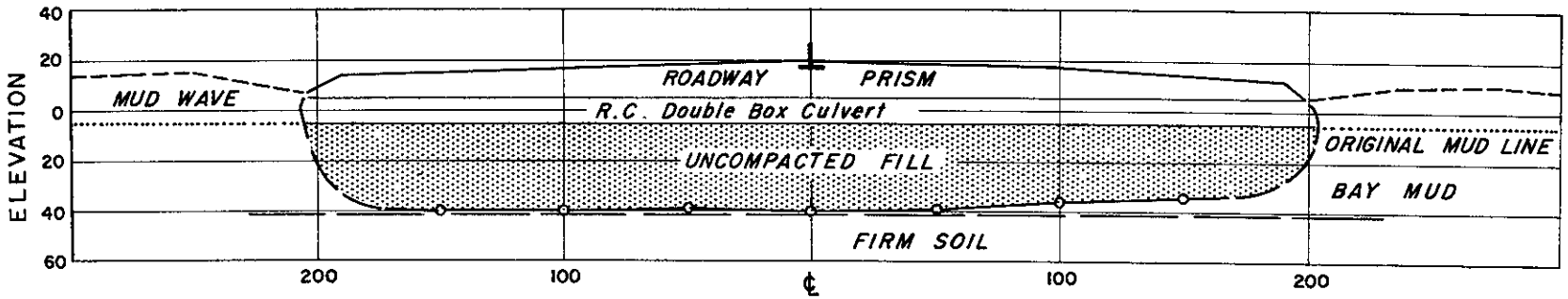


TIME IN DAYS AFTER START OF ROADWAY PRISM

SETTLEMENT AT STATION 71+00

CANDLESTICK COVE OPEN WATER FILL, 3rd UNIT

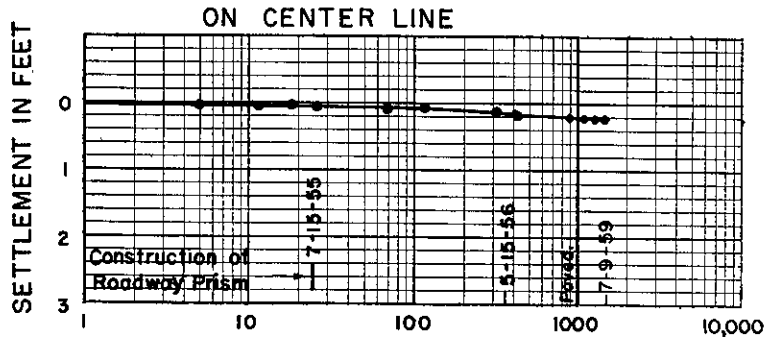
380 FOOT WIDE SECTION AT EQUALIZER STRUCTURE



CROSS-SECTION AT STATION 79+00

o - Soil change determined by means of boring.

⊥ - Settlement Platform

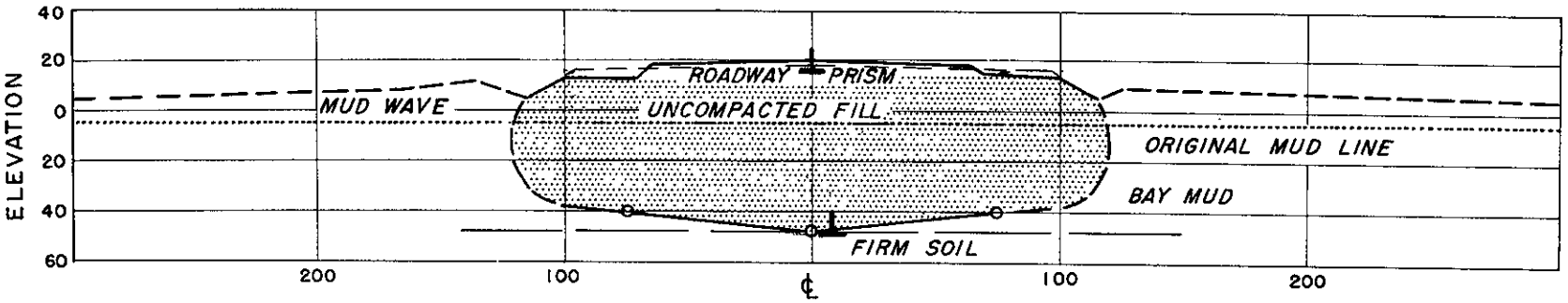


TIME IN DAYS AFTER START OF ROADWAY PRISM

SETTLEMENT AT STATION 79+00

CANDLESTICK COVE OPEN WATER FILL, 3rd UNIT

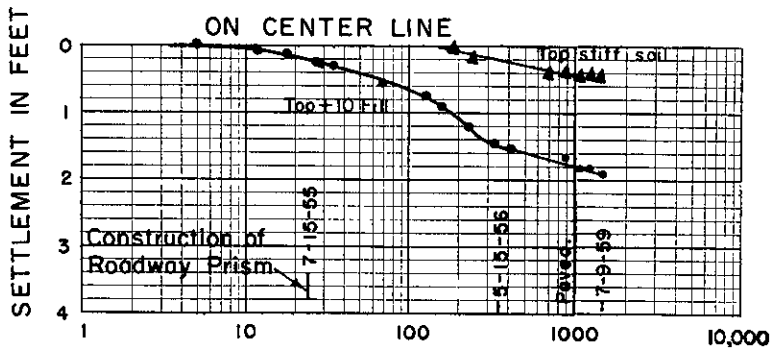
200 FEET WIDE



CROSS-SECTION AT STATION 98+00

o – Soil change determined by means of boring.

⊥ – Settlement Platform



SETTLEMENT AT STATION 98+00

CANDLESTICK COVE REPORT - PART I

Introduction

This report covers the open water section between Candlestick Point and Sierra Point, Stations 6+00 to 122+00. See Figure No. 1 for location of project.

Candlestick Cove lies on the west side of the San Francisco Bay just south of the San Francisco city line in San Mateo county. The cove is bound on the north by Hunter's Point and on the south by Sierra Point. A short distance south of Hunter's Point is Candlestick Point. A freeway was constructed from 3rd Street to the tip of Candlestick Point and southerly to the tip of Sierra Point. That portion of the freeway between Candlestick Point and Sierra Point is across open water about five feet deep at low tide.

About one-half mile west of the freeway the Southern Pacific Railroad follows the old shore line. To the west of the railroad lies a thickly built up industrial and then residential area that extends into the hills to the west of Candlestick Cove. Between the railroad and the new freeway is the San Francisco garbage dump.

The first contract, 52-4TC24-F, was let in January 1952 to construct an experimental fill south from Candlestick Point. Under this contract 484,793 cubic yards of roadway excavation was removed from Candlestick Point and placed in the open water fill between Stations 6+00 and 20+00. The object of this contract was to determine the feasibility of constructing an open water fill by end dumping methods. This was successfully accomplished for the conditions at the first unit. During 1953 and 1954, 69,000 cubic yards of excess soil was placed on the nose of this fill from nearby contract 52-4TC31-F, extending the nose to Station 22+50.

The 2nd unit was let under contract 53-4TC43-F, in June 1953 to construct an experimental fill in the portion of the cove where the greatest thickness of soft mud existed. This fill was constructed between Stations 37+00 and 58+00 using 938,492 cubic yards of imported borrow. To reach this portion of the road it was necessary to construct a haul road from the garbage dump to the freeway area. The 2nd unit was designated an experimental test section so that the construction technique could be studied, and if necessary, altered in the area of greater soft mud thickness. The fill material for this contract was obtained from a borrow site on the Crocker estate, west of the existing highway.

The third unit was built under contract 55-4TC21-F, let in August 1954, for construction of the open water fill from Station 58+00 to 110+00. The 1,953,492 cubic yards of

imported borrow for this contract was obtained from the Crocker estate. This contract was let as a normal construction contract.

The fourth unit was built under contract 54-4TC60-F let in May 1954 and contract 55-4TC56-F let in March 1955. The fourth unit extended from Station 22+50 to 37+00 and was constructed as a normal construction contract. The contract No. 54-4TC60-F was primarily for the construction of the freeway from 3rd Street to the open water fill. There was an excess of 212,000 cubic yards of roadway excavation from the 3rd Street cut that was placed in the 4th unit. Under contract 55-4TC56-F 459,328 cubic yards of fill was removed from Candlestick Point and placed in the open water fill. Surplus excavation, 20,000 cubic yards, from contract 54-4TC8-F was also placed in the 4th unit.

The fifth unit was let under contract 56-4TC49-F, in November 1955. This contract was principally for the grading work around Sierra Point and the Sierra Point overhead, but included that portion of the open water fill between Sta. 110+00 and 122+00. The fill material for the open water portion of the contract came from the Sierra Point cut and the Crocker estate for a total of 895,000 cubic yards in the open water portion of the fill.

The sixth contract was for the placing of the structural section and paving from 3rd Street to the existing freeway south of Sierra Point. This work was performed under contract No. 57-4TC39-F let in October 1956. Under this contract the open water fill was brought to a profile grade of elevation +18, and the excess fill material placed in the berm area.

A total of 5,032,000 cubic yards of fill was used in constructing the open water fill at a total cost of $4,870,600 for completed freeway. The yardages and costs are summarized in Table No. 1.

Prior to construction extensive soils studies were made. On March 8, 1950, a joint study by the Bridge Department and the Materials and Research Department of the E_3 line was reported. The E_3 line is west of the constructed freeway and the open water section of the roadway was about 2000 feet shorter than the E_n line; however, extensive structures were involved. On July 26, 1950, a joint study by the Bridge Department and the Materials and Research Department of the E_n line was reported. The E_n line is the line along which the the open water fill was actually constructed.

The soil formation in the open water area of Candlestick Cove consists of unconsolidated sediments of recent geological age, the upper layer of which is a very soft mud with its surface at about elevation -5. (Referred to U.S.C. & G.S. datum M.L.L.W.) The elevation of the bottom of this soft layer ranges from approximately -40 to -80. This soft mud

is underlain generally by a somewhat stiffer material consisting variously of clayey sand, sandy clay or clay. Bed rock varied in this area from elevation -110 to -220.

The moisture content of the upper layer of soft mud varied from approximately 90 percent at elevation -5 to approximately 60 percent at elevation -60. The wet density of the soil varied from 92 pounds per cubic foot at elevation -5 to 104 pounds per cubic foot at elevation -60. The average liquid limit of the soil was 56 and the average plasticity index 30. The clay content of the soil averaged 50 percent, with the remainder principally silt. The shearing strength of the soft mud increased about 14 pounds per square foot for each foot of depth. The consolidation tests indicated an average coefficient of consolidation of 0.006 square feet per hour. This soft bay mud contained numerous pockets or beds of shells.

The soft mud is underlain principally by a dark blue gray or brown, moist, moderately stiff to stiff silty clay. This soil is highly stratified with numerous pockets or layers of sand and has almost no shells. The unconfined compressive strength of clay samples from this soil was over one ton per square foot. The consolidation tests indicated that this soil had been preconsolidated under 5 or more tons per square foot loading.

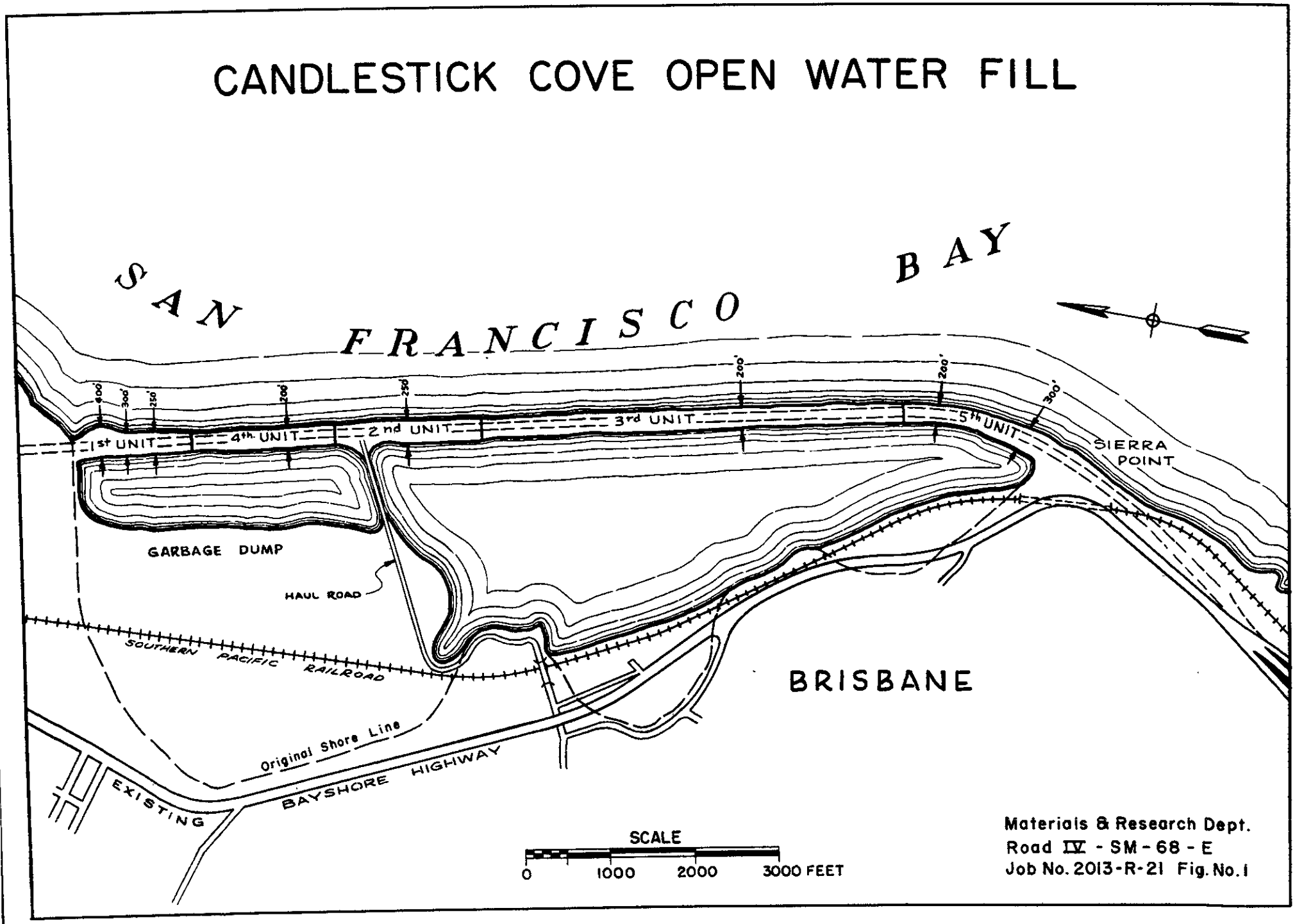
The fill as built consisted of two main parts. See Fig. No. 2. First, a working table was placed in the uncompacted state. After a stable working table was obtained a smaller compacted prism was placed on the working table, and is referred to as the roadway prism. The area between the edge of the roadway prism and the edge of the working table is referred to as the berm. In the final grading operations the roadway prism was cut down to subgrade elevation, two feet below finish pavement grade and the material thus removed was placed in the disposal area on the berms, as shown on Fig. 2. The mud that was displaced by the fill is referred to as the displaced mud. The mud that was pushed up above the original mud line is referred to as the mud wave. That portion of the fill that was being advanced is referred to as the nose of the fill.

TABLE NO. 1

ESTIMATE OF COST AND YARDAGE IN OPEN WATER FILL TO COMPLETION

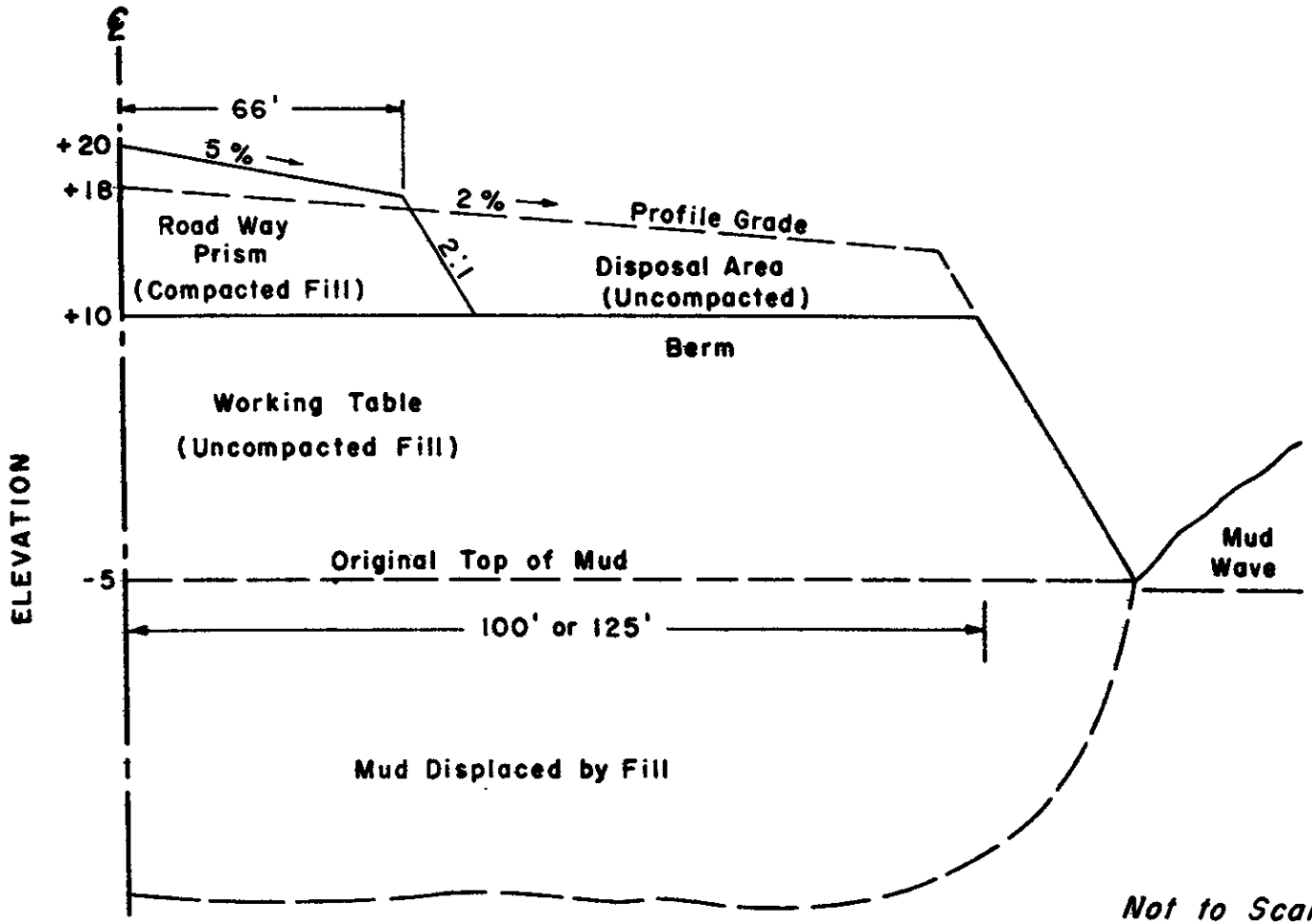
Contract	Overwater Stationing E Line	Total Cubic Yards in Over-Water Fill	Cost of Overwater Fill
52-4TC24-F	6+00 to 20+00	484,793	$ 161,060
52-4TC31-F	20+00 to 23+00	69,000	123,350
53-4TC43-F	37+00 to 58+00	938,492	863,330
54-4TC60-F	23+00 to 37+00	212,000	118,370
54-4TC8-F	23+00 to 37+00	20,000	26,300
55-4TC21-F	58+00 to 110+00	1,953,492	1,636,000
55-4TC56-F	23+00 to 37+00	459,328	379,230
56-4TC49-F Dredging, shooting, borings	110+00 to 122+00	895,000	783,000
57-4TC39-F	6+00 to 122+00	Paving	780,000
TOTALS		5,032,105	$4,870,640

CANDLESTICK COVE OPEN WATER FILL



Materials & Research Dept.
Road IV - SM - 68 - E
Job No. 2013-R-21 Fig. No. 1

CANDLESTICK COVE
Typical Cross Section
Half Section - Symmetrical About ℄



Not to Scale

Materials & Research Dept.
Road IV - SM - 68 - E
Job No. 2013-R-21 Fig. No.2

CANDLESTICK COVE REPORT - PART II

Construction

Methods of Construction

Procedures Used in Constructing the Working Table

The working table was constructed by end dumping methods. This consisted of placing the fill material as close to the edge of the fill as practical and using a bulldozer to push the dirt over the side. Sections of the working table would then fail and drop. The contractor would then bring the failed area back to grade and attempt to maintain the fill at the required grade. When the failed area had become stable the fill was advanced until another section failed. It was attempted to keep the nose in a wedge shape during these operations. As the work progressed it was learned to some degree how to control these large failures by an operation of controlled mud displacement. When the failures of the fill occurred large quantities of mud were pushed up from under the fill in the form of a mud wave. This mud wave had a crest at about 30 to 40 feet from the edge of the fill with a gentle sloping back going away from the fill for about 500 feet.

Shape of Nose

As it was desired to split the mud and push it to each side of the fill a wedge shaped nose was used. On the first and second units variations of the wedge shaped nose were used. At the start of the fill from Candlestick Point a long narrow nose was advanced into the mud. See unit No. 1 profile and cross-section sheets and Photo No. 1. This nose was advanced to Station 10± with a width of 150 feet at the shore line. Practically total displacement of the soft mud occurred with extremely high mud waves being formed. As the fill failed it was difficult to keep the working table above high tide. The fill was then widened to the desired 400 foot width. It was found that it was extremely difficult to prevent the working table from riding over the previously formed mud wave. The widening of the fill on a long front tended to result in a variable displacement of the mud. A relatively blunt nose of 50 to 75 foot width was then advanced to about Station 14± with a small mud displacement occurring. The wedge shaped nose was then generally maintained, at about a forty-five degree angle with centerline, for the remainder of the unit. See Photo No. 2. This appeared to split the mud and cause it to move in both directions in a satisfactory manner.

On the second unit a narrow, 30 foot wide, haul road was constructed from the garbage dump to the freeway area. Due to the narrow width of this fill a very blunt nose was advanced, and almost complete mud displacement occurred. In

making approximately a right angle turn in the southerly direction from the haul road to the open water fill a blunt U-shaped nose was used. By the time that Station 42± was reached a wedge shaped nose was established. The contractor then started using two types of hauling equipment, tail dump and bottom dump. The tail dump was used to advance the nose along centerline, and the bottom dump to build the sides. See profile and cross-section sheets for the second unit. This resulted in the center 50 to 60 feet of the nose being advanced 50 to 100 feet past that which would occur with a wedge shaped nose. This system of operations proved extremely undesirable and was discontinued when Station 50± was reached, at which time the tail dump trucks were used to establish and advance a nose northward and the bottom dump trucks used to advance a nose southward. The wedge shaped nose was then used in both directions.

On the third and fifth units wedge shaped noses were maintained at all times. See profile and cross-section sheets for the third and fifth units, and Photo No. 4.

On the fourth unit a wedge shaped nose was advanced southward from the first unit to Station 27± at which time the right side was advanced to the second unit. This joining of the first and second units with a fill was done to facilitate other work and resulted in an undesirable condition. By use of several extra ordinary construction methods, that will be discussed later, a fairly satisfactory working table was constructed.

In all cases where a wedge shaped nose was maintained at all times a satisfactory working table was obtained. Whenever a wedge shaped nose was not used erratic and undesirable results often occurred.

Widths of Working Tables

Stability calculations indicated that if only minor mud displacement occurred a width of 400 feet would be required for the working platform to support the roadway prism. When it was seen that considerable mud displacement was occurring, on the first unit, it was decided to try reducing the width of the working table. In the first unit the following widths were constructed, 400 feet to Station 10+00, 300 feet to Station 12+00, and 250 feet to Station 20±. See Photo No. 2. After completion of the roadway prism, without failures, at planned profile grade and a five-foot surcharge for 300 feet on the 250-foot wide section it was felt that the width could be further reduced. In the second unit a width of working table of 250 feet was used from Stations 38± to 53± and a width of 200 feet from Stations 53± to 58±. The 200-foot width of working table proved satisfactory and was used on all remaining units.

The width of fill proved to be an important item. The yardage per station for the 200 and 250-foot widths of fill was about the same. It appeared that as the fill nose was advanced the soft mud was readily displaced to a distance of about 100 feet each side of centerline. When a width wider than 100 feet from centerline was used occasionally large sections of the fill tended to break off and float laterally away from the fill. Photo No.11 shows this condition at Station 43+ on the right side. At other times the fill being placed past 100 feet from centerline encountered a large mud wave and, the fill sometimes would tend to start riding over this mud wave. See Photo No. 13. For the conditions in the open water fill the 200-foot width of working table proved to be the best practical width. There were no failures during construction due to the inability of the working table to support the roadway prism.

Daily Minimum Yardage

The special provisions for the first unit required a minimum of 5,000 cubic yards of fill to be placed in the open water fill each working day. With the large displacements that occurred this rate was insufficient to keep the working table above high tide. On all of the remaining units the minimum rate of fill placement was 10,000 cubic yards per day, and the rate varied from 10,000 to about 14,000 cubic yards per day with an average placement of about 11,500 cubic yards per day.

The rate of fill placement determined how fast the fill could be advanced. As will be discussed later, the unstable portion of the fill failed at low tide and was generally stable at high tide. If the rate of advance of the fill nose was rapid at high tide the soft mud tended to be trapped under the fill. At low tide it was necessary to keep up with the failures or a portion of the fill would break off and float away. A condition of this nature is shown on Sheet 3 of the profile and cross-section for the second unit at Station 55 to 56, and in Photo No. 12. For uniform mud displacement in constructing a fill of this nature a variable rate of placing fill would naturally be desirable; however, from a practical consideration only a minimum value for the rate of fill placement could be specified. The minimum of 10,000 cubic yards per day appeared satisfactory.

Placing of the Fill

On the first unit the contractor used tractor-drawn pans and rubber-tired pulls to haul the dirt to the working platform. The loads were then dumped near the edge of the stable section of the working table and pushed over the sides with a cat. This resulted in high loading of the edges of the fill which was thought to be desirable. However, it was found that

usually the edges or unstable portion of the fill was not being loaded during failures as the pulls could not unload directly in the failed area. At a rate of placement of 5,000 cubic yards per day it was also impossible to keep the fill at a constant elevation.

On the second unit the contractor used tail dump trucks on the haul road. The tail dump trucks dumped their loads directly over the edge of the fill and a cat was only occasionally used. The fill on the haul road tended to settle into the mud with only a few sudden large cracks occurring. As the working area on the open water fill increased the contractor started adding bottom dump trucks to his hauling equipment. The result was that where the bottom dump equipment was used the fill would often be advanced over the mud wave and failure would result on the back side of the mud wave. Where the tail dump equipment was used the loads were dumped directly over the side of the fill and tended to keep the mud wave moving away from the fill and maintaining a trough. See Photos Nos. 14 and 15. After much discussion on the action of the two types of equipment it was decided to use the tail dump equipment to advance the nose of the fill north of the haul road and to continue advancing the south nose with the bottom dump equipment. The tail dump equipment resulted in almost total mud displacement, while the use of bottom dump equipment resulted in about 50 percent of the mud being displaced. It was then decided to attempt to obtain total mud displacement to reduce the settlement after construction, so on all remaining units the special provisions specified that tail dump equipment only was to be used.

The elevation of the top of the working table on the first unit was maintained at elevation +8. During the winter the high tides flooded the working table, causing severe erosion of the roadway prism. The completed elevation of the working table on all of the remaining units was then maintained at elevation +10.

Where large areas dropped rapidly, lifts of 10 feet or more were placed in the failed areas to being them back to grade. It was desirable to avoid these large and rapid failures if possible, as it was felt that an abrupt change in depth of mud displacement was occurring at the location of the crack. See Photo No. 16 and 17. Towards the end of the second unit an attempt was made to change the way the fill failed by sloping the surface of the working table upward towards the edges in the unstable area. The edges of the fill, in unstable areas, varied from elevation +12 to +18. The result was that the working table tended to lay into place with only minor cracking and a fairly uniform mud displacement was obtained. This practice was continued on all remaining units, with the maximum elevation of the edge of the fill reaching +22. After the working table had stabilized

it was then cut to about elevation +10 and the dirt pushed ahead with cats.

Mud Blasting

As the fill displaced the mud, large mud waves were formed. The crest of the mud wave often reached an elevation of +13 and greater, or three feet above the working table. Near the end of the second unit 50-pound charges of explosives were shoved into the mud around the nose of the fill. See Photo No. 18. The tips of the charges were at elevation -25. When the charges were exploded the fill shook violently and settled about one-half foot. As the nose of the fill was advanced the mud wave flowed rather freely and the crest would drop below elevation +10. The mud wave was blasted on all remaining contracts, usually on the Monday of each week.

The blasting of the mud wave was of great value in keeping the mud "liquefied." This prevented a large build up of the crests of the mud waves, and doubtless aided in a uniform mud displacement. Mud blasting was extensively used on the fourth unit after the right side was placed and the left side was being advanced. The mud wave on the right side was shot every working day and no large build up of mud occurred.

From Station 78+00 to 80+00 it was necessary to widen the fill to 370 feet for two reinforced concrete box culverts. See profile and cross-sections for the third unit, Sheet No. 3. To accomplish this the fill was advanced past Station 80+00 in the normal manner. The mud wave was then blasted daily and the fill laterally advanced on a 200-foot front parallel to centerline. The mud blasting was successful in keeping the crest of the mud wave below elevation +15, and the mud flowed out from the fill for a large distance. See Photos No. 6 and 7. The elevation of the working table was maintained at about elevation +15 in this area to aid in moving the large mud wave formed.

At Station 37+50 to 39+50 the fill was widened for a reinforced concrete box culvert. Daily shooting of the mud wave was used as the fill was widened. In order to obtain maximum displacement of the existing fill in the area of this reinforced concrete box culver 44 borings were made through the fill and charges placed in the soft mud below the existing fill. The fill surface was then raised to elevation +20 and the charges below the fill exploded. A subsidence of about two feet occurred after exploding the these charges. This proved to be an expensive method of displacing the soft mud, with small results.

Closures

During the construction of the open water fill it was necessary to make two closures, that is advance a newly constructed fill to an existing fill that had been completed for

a period of time. The first closure was made at Station 37± where the fourth unit was joined to the second unit. The fill was advanced to Station 27± with the wedge shaped nose and then the right side was advanced to the north nose of the second unit. This was done to facilitate construction of the reinforced concrete box culvert at Station 38±. The advance of the right side resulted in a large mud wave along centerline. The left side was advanced on an angle to centerline and the mud wave was blasted daily. This kept the mud wave moving, but a smaller mud displacement occurred on the left side of centerline than on the right side. See profile and cross-sections, Sheets for the fourth unit. See Photo No. 6.

The other closure was just north of Sierra Point in the fifth unit. The excavated material from the cut at Sierra Point was used in the fill south of the point, on the railroad realignment around Sierra Point, and to advance the open water fill at elevation +20 northward from Sierra Point to Station 117±. Care was taken in advancing this fill so that there was no unstable fill remaining at this location when it was completed. This avoided previous difficulty where an unstable portion of the north nose of the second unit was allowed to set over one year and difficulty was encountered in starting movement when closure was made. The previously placed railroad fill developed a very large mud wave that flowed into the open water fill area. See Photo No. 8. It was decided to attempt dredging of the mud when the northern nose was at Station 114±, as a mud wave of exceptional size had developed. Suction dredging operations were started about 500 feet left of Station 116±. After dredging for several days the dredge was able to advance to 400 feet left of centerline. The mud was flowing from the railroad fill through the opening between the open water fills to the dredge, a distance of about 1000 feet. After two weeks of dredging an estimated 100,000 cubic yards of mud, with the dredge unable to advance closer to the fill area, the dredging operations were discontinued. The mud waves that had previously been built up to a crest elevation of +20 or greater, were below elevation +10 when dredging was completed. The right side of the north nose was then advanced to the south nose and the closure widened to the left of centerline. See profile and cross-sections for the fifth unit. Daily mud blasting was used as the fill was widened to the left of centerline. Almost complete mud displacement was obtained at this closure except to the left side at Station 117±. An extremely large mud wave was built up just north of Sierra Point.

Roadway Prism

As the working table was completed for each unit the roadway prism was placed upon it. The roadway prism consisted of a ten-foot high fill placed in 8 inch compacted lifts, 130 feet wide on the top with 2:1 side slopes. The roadway prism was the only compacted fill placed. Profile

grade of the roadway prism was +20 as constructed. In the final grading operations cut and fills were made to bring the profile grade to +18, with a two-foot structural section specified.

Disposal Area

During the final work on the open water fill there was a surplus of dirt. There was about 100,000 cubic yards from a slide in the pit, 50,000 cubic yards from an overloaded area south of Sierra Point, and fill removed from the roadway prism during final grading operations. This fill was placed on the berm so as to make the berm level with the completed roadway. The fill in the disposal area is in an uncompacted condition.

CANDLESTICK COVE REPORT - PART III

Results of Construction Studies

Failures

As the nose was advanced from the stable portion of the fill a crack would develop near the stable portion. Regardless of how far the fill, in the first and second units, was advanced past this crack, movement would continue to occur at the same location until the nose had become stable. Then the cracking would advance to the point where the fill was again unstable. It was not unusual for the crack to appear at the same location for as many as five or more consecutive days. At many locations, on the first and second units, a study of these failures were made. Hubs were placed in the fill and horizontal and vertical movements measured. From these measurements three types of movement were found to occur: 1. (Shear Type) A rotary movement where the fill would have a large drop at the crack and small or no movement at the edge of the fill, 2. (Squeeze Type) The drop would be uniform over the failed area with occasionally a small horizontal movement occurring, and 3. (Laying the Fill Into Place) A large drop at the edge of the fill with little or no drop occurring at the crack. It was noted that the only time the third type occurred on the first and second units was just before a cracking pattern became stable and the cracking advanced to a new location. On the remaining units this was the predominant type of failure. No special attempts were made to analyze this third type as it seldom occurred on the first and second units. On the third, fourth, and fifth units this type of failure predominated and is further discussed in the section on controlled mud displacement.

Hubs were placed in the fill, in both profile and cross-section, at various times in the first and second units. The vertical and horizontal movements of these hubs were measured as the fill failed. Figures F1, F4, F7, F10, and F13 are representative examples of the data obtained. The extent of the mud waves were also measured during the failures. Figures F1, F3, & F6 show representative shear failures and Figures F9 & F12 representative squeeze failures.

The failures were then analyzed to determine the average strength of the soft mud at the time of failure. Several shear circles were run using the cracking as one point on the shear circle and the approximate outer crest of the mud wave as the other control point of the circle. The thickness of the fill was estimated from records of the amount that the fill had dropped and the yardage placed in the failed areas. It was assumed that the fill contributed no strength towards

its stability at the location of the crack. The circles requiring the maximum average strength of the soft mud for the above conditions are shown in Figures No. F2, F5, F8, F11, and F14. Using the same assumptions as above, except that the location of the failure was ignored, a series of shear circles were run. The circle requiring the maximum shearing strength of the soft mud, or critical shear circle, are shown in Figure Nos. F3, F6, F9, F12, and F15.

The normal type of failure in the first and second units was where the fill would fail in a rotation type of movement, the maximum drop being at the crack and small movement occurring at the edge of the fill. See Photo 16. The mud was forced up about 20 to 40 feet from the fill and appeared to move in a vertical direction. The mud had a very rough appearance, See Photos No. 13, 18. Occasionally a vertical face appeared on the side of the mud wave away from the fill, see Photo No. 9. It appeared that these failures followed a circular rotational movement, and are designated as shear type failures.

For the shear type failures the average shearing strength of the soft mud required for stability was 155 pounds per square foot, with a range of from 70 to 210 pounds per square foot. The shear circles tended to extend to the bottom of the soft mud, indicating that the entire depth of the soft mud was being reworked. The centers of the shear circles were all about at the same elevation as the top of the fill. The indications are that the mud was reworked, with a resultant decrease in strength, over its entire thickness.

Shear circles were also run, for the same sections, without consideration for the location of the cracking to determine the required strength that would have been indicated in a stability analysis. The minimum centers were again at about the same elevation as the top of the fill. As the circles were increased in radius, increasing the depth of the soft mud included in the circle, the strength required for stability increased approximately 5 pounds per square foot for each foot of depth of the shear circle. One set of shear circles were run to a depth of elevation -100 without a maximum occurring. The maximum required strength was therefore taken at the depth of the bottom of the soft mud. These shear circles, for the shear type failure indicated required strengths 50 to 150 pounds per square foot in excess of the strength indicated by the analysis of the actual failure. It would appear that the reworking of the soft mud as the failure progressed resulted in keeping the fill unstable and if not influenced by other factors would result in nearly total displacement of the soft mud by the fill.

On the few occasions that the fill was rapidly advanced with small mud displacement an unusual type of failure occurred that has been referred to as a squeeze type failure.

Generally some factor such as an extremely large build-up of the mud wave, a period of high tides, or an extended period where the nose of the fill was not advanced, caused this condition to occur. The failed portion of the fill would crack in all directions and the settlement would occur fairly uniformly. The failed area tended to settle at a slow, steady rate. The mud was pushed out from under the fill at an angle and had a smooth appearance. See Photos No. 20 & 21. A high mud wave was never built up with this type of failure.

In attempting to analyze this type of failure it was noted that the shear circle always extended a considerable depth past the bottom of the soft mud. As the stiff mud had a shearing strength in excess of 2000 pounds per square foot it was not felt that this soil was failing. In analyzing this type of failure it was assumed that a sliding action occurred at the bottom of the soft mud. Figures No. F11, and F13 show analysis of typical squeeze type failures.

Analysis of the squeeze type failures indicated shearing strengths of from 200 to 300 pounds per square foot for the soft mud. They were in very good agreement with the strengths indicated by the critical shear circles, Figures F12 and F15.

The rate at which the squeeze type failures occurred appeared to be related to the amount of mud displaced. When the amount of displaced mud was small the drop each day was small, one to two feet. As the thickness of fill was increased, by bringing the failed area back to grade, the drop each day would increase. The maximum drop for this type of failure was about five feet in one day. When the soft mud was displaced to elevation -20 to -30 the movement would suddenly stop, with the fill becoming stable.

There were many factors which affected the fill failures. The tide was probably the most important. Often the fill was stable at high tide and unstable at low tide. The mud wave acted as a counter balance in the failures and therefore the size of the mud wave influenced the failures. The elevation at which the top of the working platform was maintained greatly influenced the failures by affecting the driving force. The type of fill used in the working platform appeared to have little affect on the failures, probably due to the continuance of a crack at the same location as the fill continued to fail. The strength of the native soil was uniform throughout the open water fill and any minor variations in strength did not appear to influence the failures.

The profile grade of the roadway prism was constructed to elevation +20. This was two feet above the desired profile grade of elevation +18 to check the stability of the completed fill and to allow for settlement as the remaining units were built. At a few locations where small, less than -20, mud displacement occurred it was noted that plastic

flow may have been occurring. No shear type failures occurred. The stability of the completed fill at profile grade of +18, appears to be above unity. One failure has occurred to date, at Station 110± where the fill increases in height to the Sierra Point overhead.

On the first unit a surcharge to elevation +22.8 was placed between Station 13+00 to 16+00. A shear type failure occurred at Station 15+75 where the mud had been displaced to elevation -25. A plastic failure occurred around Station 14± where a mud displacement to elevation -15 existed. From this data it appears that a mud displacement to at least elevation -25 is desirable for stability.

Displaced Mud

Borings were made, as the working table was constructed, to determine the depth to which the mud was being displaced. These borings were made with a four-inch churn rig. The borings were made at each station on centerline and about 75 feet right and left of centerline. From the boring data profile and cross-sections were drawn. See attached profile and cross-section sheets. The above borings were made to locate the bottom of the fill and no samples were taken.

The first, second, and fourth units, from Station 6+00 to 56+00 have a great variation in the amount of soft bay mud displaced by the working platform. There are many locations where very small mud displacement changes to total mud displacement in a distance of 100 feet. This abrupt change in the thickness of the working table was believed to be detrimental to the completed roadway.

As it was found, in the first and second units, to be impossible to control the amount of mud displaced; on all other units it was decided to attempt total mud displacement. On the fourth unit there were three contractors working at one time making it difficult to obtain total mud displacement. However, over 50 percent mud displacement was obtained at all locations in this unit.

The third and fifth units had near total mud displacement throughout their length, Station 56+00 to 122+00. This total mud displacement was obtained by the use of the studies of the factors affecting the mud displacement.

The items that affect the displacement of the soft mud, by the fill, can be placed in two classes; items that can be controlled by man, and the items that cannot be controlled by man.

The controllable class include the shape of the nose, rate of placing fill, type of fill material, and the elevation to which the working table is carried.

The uncontrollable class include the affect of the tide, strength of the bay mud and the extent of the mud wave. The depth of the soft bay mud determines the maximum extent of the mud displacement.

The use of a wedge shaped nose effectively split the mud to about 100 feet from centerline. When a blunt shaped nose was used the mud was pushed ahead of the fill and an extensive mud wave was built up. Where a long narrow nose was used a large mud wave was built up about 50 feet from centerline. Other odd shaped noses used would tend to build up large mud waves at various locations around the nose. The shape of the nose tended to control the evenness of the mud wave built up around the nose, and appeared to control the shape of the cross-section of the fill as determined by borings.

The desirable rate of placing fill is that rate which will maintain the top of the working table at the desired elevation and still slowly advance the fill. At no time was the rate of placing fill too rapid, only too slow a rate was observed. Using a slow rate, the embankment material was used to maintain the working table at the desired elevation without advancing the nose. This would result in a large mud wave ahead of the fill that may have tended to increase in strength from having set without moving for several days.

The type of fill had an apparent affect upon the starting of a failure. Once a failure had started the type of fill did not appear to have any affect upon the rate at which failure occurred.

The elevation of the top of the working table is related to the driving force in displacing the mud. The tide limited the elevation of the working table to above elevation +10. It was found that to keep the mud moving the top of the working table had to be three to five feet above the crest of the mud wave. As the elevation of the top of the working table is related to the driving force in the fill failures, the depth of the displacement of the soft mud was greatly affected by the elevation of the working table. To avoid walking over the crest of the mud wave it was necessary at a few locations to increase the elevation of the top of the working table.

At low tide the fill would tend to fail and at high tide the fill would tend to be stable. This resulted in small mud displacement during periods of high tide and large mud displacement during periods of low tide. During high tides the fill could be kept moving by raising the elevation of the top of the working table. At low tides it was not possible to lower the elevation of the working table below +10 due to the necessity of keeping the working table above the mud wave.

The strength of the soft bay mud was reasonably uniform in the area of the open water fill. There was some feeling that the extensive shell beds, in the soft mud, may have increased the strength of the soft mud; however, no factual data was obtained to substantiate this.

The mud wave acted as a resisting force in opposing the fill failures. When large mud waves were built up small mud displacements occurred and where small mud waves existed large mud displacements tended to occur.

In the first and second units it was hoped to obtain a reasonably uniform mud displacement without totally displacing the mud. This proved to be impossible and often resulted in an abrupt change in the depth to which the soft mud was displaced by the fill. In the remaining units it was decided that it would be practical to attempt to obtain near total mud displacement.

The object of the working table was to enable the construction of a stable roadway prism. There are two factors to be considered; the stability in relation to shear and plastic failures, and uniform settlement. Both of these factors are dependent upon the amount of soft mud displaced. With a very small mud displacement, about elevation -15, a danger of a shear failure occurring is rather large. Where a medium displacement, to elevation -20, occurred it has been noted that signs existed that plastic flow may have been occurring. With mud displacement below elevation -20 no evidence of failure existed. For stability the soft mud should be displaced to a depth greater than elevation -20. Considering settlement of the roadway prism any uniform thickness of mud remaining below the fill would appear to be satisfactory. The minimum settlement should occur with total mud displacement. As the bottom of the soft bay mud varied from elevation -40 to -80 a uniform mud displacement for stability would mean a variable amount of soft mud remaining under the fill, with great variations in settlement. The simplest solution was to displace as much of the soft mud as possible.

A term has frequently been used to describe the action of this fill, that is "floating" the fill. This term would indicate either of two items. The fill displaced a weight of mud equal to its own weight, that is the soft bay mud had no shear strength. This definitely did not occur. The fill displaced a small amount of the soft mud and tended to stay on top of the soft mud. This condition was found impossible to obtain. For the above reasons the term "floating" is being avoided in this report.

Mud Waves

In the first, second, and third units the mud wave was cross-sectioned as the working table was completed. A mud

wave was developed that extended 500 to 700 feet from centerline. These mud waves had a crest 20 to 40 feet from the top edge of the fill. The mud wave crest varied from elevation +5 to +16. The back of the crest would slope on about a 5 percent grade to an elevation of about +2, and then extend out at about a 2 percent grade. The cross-sections of the mud waves are shown on the profile and cross-section sheets.

Figure M.W. 1 shows the profile of the mud wave ahead of the nose of the fill on March 24, 1952. This profile was taken just after a sudden drop of about 3 feet in the nose and shows the typical block of mud lifted up. This block of mud was 16 feet thick and was lifted up 3½ feet. By the next day this block of mud had failed within itself and had disappeared with the mud wave crest at about elevation +5. These masses of mud that were lifted up when the fill failed were of fairly high strength; however, after they had failed within themselves the mud became fluid and appeared to flow. It appears that the fill, in failing on a shear plane, forced mud up from below the fill. This resulted in a slight decrease in strength due to remolding. After the mud was forced to the surface it lacked the necessary strength to maintain the vertical faces, and then formed a second wave about 20 feet behind the crest, again losing strength through reworking. This process would continue until the mud almost flowed. The type of mud wave described above occurred when the fill failure was of the shear type and also on what was previously menioned as the type three failure.

Stability analyses were made of the back slope of the mud wave. These circles showed that a strength of 0.09 tons per square foot was required to maintain a grade of five percent, and a strength of 0.05 tons per square foot was required to maintain a grade of two percent. The shear circles indicated that the mud was failing to a depth of 10 feet. These shear circles agreed with the visual observations, where the mud wave was observed to form mud waves with crests 10 to 20 feet apart.

In the squeeze type of failure the mud appeared to be squeed out from under the fill, i.e., as the mud would be squeezed between the fingers when a hand is closed over a ball of mud. The mud was generally squeezed out from under the fill at an angle of about 30 degrees and had a large reduction in strength. The crest of the mud wave did not build up to a high elevation, generally being between elevation +5 and +8. The mud would start flowing in a semi-liquid condition a few feet behind the crest of the mud wave and often carried large rocks and stones with it.

The mud wave acted as a berm or support for the fill, greatly increasing resistance to failure. In the completed fill section, with the roadway prism in place, stability analysis indicate that the mud wave contributed about 30 percent of the resisting force. The mud wave was equivalent to

increasing the width of the uncompacted fill by 50 feet on each side of centerline.

At various times in the first and second units, 2 x 4 inch soil samples were obtained from the upper five feet of the mud wave. In the section where the mud was flowing on a 2 or 3 percent grade, furthest portion of the mud wave from the fill, the mud was too soft to support itself in the sampler. Samples of the mud from the squeeze type failures taken near the fill, being sampled within an hour after being forced out from below the fill, indicated an average unconfined compressive strength of 0.05 tons per square foot with a range of 0.02 to 0.08 tons per square foot for the upper three to five feet in depth. A total of twelve samples from three different failures being obtained in the above studies. A total of fifteen samples of the soft bay mud were obtained from four different shear type failures within an hour after the mud had been pushed up. These samples indicated an unconfined compressive strength of from 0.09 to 0.3 tons per square foot for the soft mud with averages at the four locations of 0.27, 0.17, 0.13, and 0.11 tons per square foot, with all samples from each location being ± 0.05 of the average. All of the above samples were obtained using a modified California sampler with a shelby tube cutting point. The sampler was pushed to the desired depth closed and then the plug raised. The sampler was then pushed ahead one foot and the sample in the Shelby tube being the only sample saved for testing. The strength tests indicate that extensive remolding occurred in the squeeze type failures and minor remolding occurred in the shear type failures.

The tide had considerable affect upon the height of the crest of the mud wave. As the tide stage rose the crest of the mud wave would build up as the failure of the fill occurred. As the tide stage decreased the mud would start failing within itself and gradually tend to flow away from the fill. From measurements taken at various times on the second unit it was established that a two to three-foot tide drop would start the mud wave to failing.

After the mud had remained above high tide about two or three days it became firm enough that a man could walk on it without great difficulty. About one to two weeks was required in the zone where alternate wetting and drying occurred before a man could easily walk on the mud. Digging into the mud it was found that only the top inch had greatly increased in strength and the underlying mud was still extremely soft.

After the second unit working table had become stable to Station 49+00 four borings were made in the mud wave to determine the strength of the mud in depth. These borings were made at Station 44 and 47 on both the left and right sides of the fill. The borings were made at the crest of the mud wave to a depth of 45 feet. The unconfined compressive strength increased from 0.1 ton per square foot at the surface

to 0.25 tons per square foot at a depth of 45 feet. The moisture contents and unit weights are about the same as indicated by the original borings prior to construction. Summaries of the test data are included in the section Borings During Construction.

Mud blasting was used on the third, fourth and fifth units. The charges were placed around the nose of the fill to a depth of thirty feet or more below the top of the mud wave. The charges were placed in the trough formed by the fill and the mud waves. When the charges were exploded there was a noticeable softening of the mud and the mud would slowly start to flow away from the fill. The mud blasting was helpful in reducing the strength of the mud, and preventing small mud displacements where a large mud wave had been built up, an extended shutdown period had occurred, and where closures were made between two fills.

Tides

The tides at Candlestick Cove varied from elevation +8 to -2 with a M.L.L.W. as zero. The tide cycles were of about 12 hours duration. During storms, waves about two feet high would develop. This controlled the minimum elevation of +10 to which the working table was constructed.

As a mud wave was formed around the fill, of impervious soil, the water table in the fill area never dropped below elevation +4.5 to +5.0. That is when the tide was below elevation +5 the water was ponded inside the mud wave. As the tide rose above elevation +5 the water table 75 feet from centerline lagged about one-quarter hour behind the tide and the water table on centerline lagged about one hour behind the tide. The result was that at tides below elevation +5 the stability of the fill decreased rapidly, and at tides above elevation +5 the stability of the fill increased slightly. In all stability analysis the water table was taken as at elevation +5 in the fill area and the tide as at elevation 0.

Piezometers were placed at three general locations in the soft mud; on centerline, 75 feet left of centerline, and in the mud wave on the bay side of the crest. The tide had a radical affect upon the excess hydrostatic readings. Hourly readings were taken of tide, water table, and excess hydrostatic pressure over several tide cycles. The readings on centerline decreased slightly as the water table on centerline raised, the pressure rise being equal to the weight of the water, and remained constant while the tide was below elevation +5. The excess hydrostatic pressure 75 feet left of centerline showed an increase as the tide dropped below elevation +5 and decreased as the tide rose above elevation +5. The excess hydrostatic pressure behind the crest of mud wave was directly proportioned to the tide. Water table tubes were placed in the mud wave, and it was found that the water table in the mud was at the mud surface when the mud was

above the tide. Therefore the water table, or zero reference, for the excess hydrostatic pressure outside the fill area is taken at the mud surface when the mud is above the tide and at the tide elevation when the tide is above the mud surface. For the excess hydrostatic pressures in the mud below the fill the water tables in the fill are used as the zero reference. To simplify the piezometer readings and eliminate the tide affect all excess hydrostatic pressure readings for long-time studies are taken at a tide stage of +3 to +4. Table No. I shows an example of the type of readings obtained at Station 44+00 on October 13 and 14, 1954. Station 44+00 contained two extra piezometers to obtain a more complete pressure cross-section and is used as an example for this reason. All other stations showed similar readings when studies were made at their locations.

There was some feeling that the fill was rising and falling with the tide. After completion of the second unit precision levels were run over a complete tide cycle on the settlement platforms on centerline and 75 feet right and left of centerline. These measurements showed that the fill remained at a constant elevation ($\pm$0.002 feet) during the tide change.

It appears that the weight of the water, at high tide, acted to resist failure of the fill by supporting the mud wave. At low tide the mud wave failed decreasing its support of the fill and generally causing the fill to drop. At several times during the construction of the second unit the tide drop that occurred before fill failure occurred was noted. The tide drop that resulted in fill movement was about three to five feet. Several times a difference of only two feet existed between high and low tide. If the fill was stable at high tide the two-foot tide drop did not result in a fill movement.

The tides were of a variable nature at Candlestick Cove, that is at times the difference between high and low tide was a magnitude of three feet for several days, and at other times over an eight foot tide change would occur. During periods when the tide was between elevation +4 and +7 for several days the fill would be rapidly advanced, if the working table was maintained at elevation +10, with a small mud displacement. When the tide would then drop to elevation zero $\pm$ a squeeze type failure often resulted. In the second unit a study was made of the mud displacement and the tide that existed when the fill was placed. It was found that generally a high tide existed when a section of fill had a small mud displacement on centerline. To avoid the shallow mud displacement during periods of high tide the elevation of the top of the working table was varied in the latter part of the second unit. It was found that by keeping the elevation of the nose about five to six feet above the tide stage the fill could be kept moving during periods of high tide. How-

ever, an extremely large mud wave was then built up near the fill. Later mud blasting was used to liquefy the mud and aid it to flow away from the fill.

Types of Soil

The imported borrow used in the uncompacted fill consisted of two principle types of material. One was a soil that varied from a sand clay to a silty clay that was from the weathered zone of the pits. This material generally had a high cohesion and a low angle of friction. The density of this soil was about 105 pounds per cubic foot in the uncompacted fill. The other general type of borrow was unweathered hard blue sandstone or unweathered hard blue gray shale. The unweathered rock varied from a few inches in size to about ten feet in size. This material formed a fairly compact fill and no unit weight tests could be obtained. This material had a high angle of friction and small cohesion. The hard sandstone was unaffected by the sea water and only a small amount of the shale disintegrated under the action of the sea water.

The borrow material had some affect upon the failures. Where the rock type of borrow had been placed there appeared to be greater resistance to the start of a new failure. After a cracking pattern had developed there was no apparent difference in the rate or type of failure between the two types of fill material used.

Where the soil type of borrow had been used in the fill it was noted that the water tables on centerline had about twice the time lag with tide changes as the rock fill had. This is probably due to the more impervious nature of the soil fill.

The native soil was a soft gray silty clay as previously described. This soil had numerous shell layers several feet in thickness. There was no evidence that these shell layers affected the failures or mud displacement.

The soft bay mud that had been pushed up above the tide tended to dry out and form a thin crust. There was some feeling that the mud may be increasing in strength due to thixotropic effect. The borings at Stations 44+ and 47+, that were made in the mud wave, did not indicate that the soft mud had greatly increased in strength in a period of about one month or more. There was doubtlessly some thixotropic affect in the soft bay mud, however, it apparently is small and of doubtful importance.

Observations During the Placing of the Roadway Prism

The portion of the uncompacted fill outside of the roadway prism area is the berm area. In the first and second units hubs were placed 100 and 125 feet right and left of centerline at each Station to observe any surface movement

of the berm during the placing of the roadway prism. The settlement of these hubs were not accelerated by the placing of the roadway prism. There was small, about 0.1 to 0.2 foot, horizontal movement of these hubs away from centerline.

Cracking was noted, about 85 feet from centerline, in the berm area during the placing of the roadway prism. See Figure No. 22. Occasionally these cracks opened up as much as one-half of an inch and were generally about one-quarter of an inch opening. These cracks were parallel to centerline and continuous for the length of the fill, occasionally branching off and ran to the side of the berm. It appears that these cracks were tension cracks due to the consolidation of the uncompacted fill under the loading of the roadway prism.

During the placing of the roadway prism, on the first unit, it was noted a large, 0.5 to 0.8 foot, consolidation of the uncompacted fill appeared to be occurring. On the second unit five settlement platforms were placed near the bottom of the fill at locations where platforms were located on the surface of the uncompacted fill. These settlement platforms indicated that 0.2 to 0.3 foot consolidation of the uncompacted fill occurred during the placing of the roadway prism and another 0.1 to 0.2 foot occurred within a few months of the completion of the roadway prism. The indications in the first and second unit are that the consolidation of the uncompacted fill was completed in six months or less. The data on the above settlement platforms are included in the section on settlement studies.

In the first unit the uncompacted fill consisted of almost entirely soil type borrow from Candlestick Point. The second unit uncompacted fill consisted of a mixture of soil and rock type borrow from the Crocker estate borrow site. The third unit consisted of almost completely rock type borrow from the Crocker estate borrow site. The consolidation of the uncompacted fill by the roadway prism varied widely in the three units; being an average of about 0.7 in the first unit, about 0.4 in the second unit, and 0.2 in the third unit. The type of fill greatly influenced the amount of consolidation in the uncompacted fill.

Settlement platforms were placed under the shoulder line at selected stations on the first and second units. These settlement platforms, at shoulder line, indicate that the settlement was uniform under the roadway prism where uniform mud displacement had been obtained. Where differences of mud displacement were large, between centerline and the edge of the roadway prism, differential settlement of 0.4 foot occurred. The settlement cross-sections indicate the desirability of obtaining uniform mud displacement. All settlement curves are included in the section on settlement studies.

On the first and second units the mud wave was carefully observed during the placing of the roadway prism. At five locations there are indications that mud was being forced out from under the fill, as follows: Station 13± to 15± on the left side, Station 15± to 17± on both sides, Station 41± on the right side, Station 44± on the left side, and Station 51± on the right side. A shear type failure with major cracking occurred at Station 16± during the placing of the overload. At all other locations the fill showed no signs of cracking or other visible indications of failure, although higher than normal settlement occurred in these areas. The depth to which the mud was displaced 75 feet from centerline at the above Stations is: Station 14+00 on the left side, elevation -17, Station 41+00 on the left side elevation -10, and Station 51+00 on the right side elevation -15. There are other locations where the mud was displaced to elevation -10 to -15 that did not have any indications of failure occurring when the roadway prism was placed. It is felt that such factors as the size of the mud wave, strength of the soft mud, and strength of the uncompacted fill may account for this. The indications are that the fill is at marginal stability when the soft mud is displaced less than elevation -20.

In the first and second units piezometers were placed in the soft mud remaining under the fill on centerline. The rise in excess hydrostatic pressure on centerline equalled the loading applied by the roadway prism. This would indicate that no primary consolidation of the soft mud occurred during the placing of the roadway prism. In the second unit piezometers were also placed about 75 feet left of centerline in the soft mud remaining below the fill. The excess hydrostatic pressure 75 feet from centerline rose about one-half of the loading that occurred on centerline due to the roadway prism. Using the theories of pressure distribution developed by Leo Jurgenson, the rise in excess hydrostatic pressure in the soft mud, due to the roadway prism, should have been only one quarter of the pressure rise on centerline. In the second unit piezometers were also placed below the mud wave, and these piezometers indicated very small, less than 0.05 tons per square foot, rise in excess hydrostatic pressure when the roadway prism was being placed. It appears that no increase in the rate of consolidation of the soft mud occurred during the placing of the roadway prism.

Controlled Mud Displacement

After the completion of the second unit it was decided to attempt to displace at least ninety percent of the soft mud under the roadway prism area. It was planned to do this by varying the factors affecting the mud displacement that could be controlled in such a manner so as to counteract the uncontrollable factors.

A daily load count, from the number of truck loads taken to the fill, was maintained along with the approimate locations at which the borrow was placed. A record of the approximate yardage of fill between any two stations was then known. The borings prior to construction indicated the depth of the soft mud from which a desired cubic yard of fill per station could be determined. This data would then indicate when the minimum desired mud displacement had occurred. Borings were later made to check the mud displacement.

The largest factor affecting the failures, and as a result affecting the mud displacement, was the size of the mud wave. The size of the mud wave was partially determined by the tide, which could not be controlled. The strength of the soft mud was therefore reduced by mud blasting, sometimes called liquefying the mud, so that the mud would tend to flow away from the fill. The mud wave was blasted, at least once a week, with 50-pound charges of explosive place around the nose of the fill. The mud would then start moving away from the fill within a few hours, depending on the tide stage, with a reduction of the size of the mud wave near the fill.

During periods of high tide and/or large mud wave build up the elevation of the top of the working table was raised. This was done by sloping the top of the unstable portion of the fill towards the edges starting with elevation +10± at the stable portion. The edge of the fill reached a maximum of elevation +22. This resulted in a grade as large as 10 percent on the nose with an average grade of 3 to 5 percent on the nose. The action of increasing the driving force near the edge of the fill was to increase the rate of drop near the edge. The large shear or rotational movements that had been previously occurring seldom occurred on the third, fourth, and fifth units. Instead the fill tended to lay into place, that is the maximum drop occurred at the nose and tapered to small or no drop at the stable portion. The cracking was over the entire failed area with no major cracking occurring. There was no tendency for the edges of the fill to break off and separate from the main fill as had previously occurred with the shear type failures. It is felt that abrupt changes in mud displacement were avoided with this laying in of the fill action. A few shear type failures occurred on the third, fourth, and fifth units at extremely low tides, below elevation +1. However, the shear type failures were readily controlled again after the period of low tides was over.

The rate of advance of the nose was maintained between ten and thirty feet per day on the third and fifth units. This avoided the condition where the nose would remain at the same location while a rapid failure was occurring as often happened on the first and second units. This appeared to aid in keeping the mud moving and with mud blasting prevented the large build-up of the mud wave. It was found

that the stable portion of this fill advanced at about this same rate and only about 100 feet on the end of the nose was unstable. The rate of placing fill, about 12,000 cubic yards per day was sufficient to maintain this rate of advance.

This method of controlling the mud displacement worked very well. Nearly uniform mud displacement was obtained under this system of operations. See profile and cross-sections for the third, fourth and fifth units. The few locations where nonuniform mud displacement occurred was due to some avoidable condition. For example, the night foreman advanced a long narrow nose past Station 96+00, with the typical vee type mud displacement occurring.

Typical Variation of Water Table and Excess Hydrostatic Pressure with Tide

Station 44± on October 13 and 14, 1954

Time of Day	Tide Stage	Water Tables			Excess Hydrostatic Pressures				
		On ℄	75 ft. Left	150 ft. Left	On ℄	75 ft. Left	130 ft. Left	185 ft. Left	320 ft. Left
		Elevations			Tons per square foot				
October 13th									
10:00 a.m.	6.4	5.9	6.2	6.9	1.516	0.871	0.745	0.431	0.167
Noon	7.9	7.5	7.8	7.9	1.527	0.865	0.737	0.434	0.162
2:00 p.m.	7.1	7.2	7.1	7.1	1.533	0.857	0.741	0.440	0.151
4:00	3.2	5.8	5.7	7.0	1.558	0.941	0.779	0.574	0.267
5:00	+0.1	5.7	5.8	7.0	1.583	0.993	0.790	0.637	0.337
6:30	-0.2	5.7	5.7	7.0	1.587	1.007	0.799	0.656	0.341
8:00	-0.5	5.8	5.7	6.9	1.589	1.013	0.807	0.663	0.358
10:00	+0.3	5.7	5.7	6.9	1.585	1.003	0.811	0.642	0.341
Midnight	4.8	5.8	5.7	6.9	1.571	0.957	0.780	0.519	0.264
October 14th									
2:00 a.m.	5.5	5.8	5.8	6.8	1.577	0.960	0.757	0.501	0.211
5:00	4.9	5.8	5.7	6.8	1.571	0.941	0.777	0.527	0.221
7:30	3.0	5.7	5.8	6.8	1.567	0.971	0.769	0.571	0.262
9:00	3.7	5.8	5.7	6.8	1.541	0.939	0.773	0.511	0.273
10:30	7.5	7.0	7.2	7.5	1.528	0.870	0.741	0.437	0.168
1:00 p.m.	7.5	7.5	7.5	7.5	1.526	0.860	0.747	0.444	0.153
3:00	3.0	6.0	5.9	7.0	1.562	0.968	0.775	0.569	0.264
6:00	1.0	5.8	5.8	6.9	1.579	0.985	0.784	0.623	0.321

CANDLESTICK COVE REPORT - PART IV

Sample Borings and Tests During Construction

Location and Purpose

After completion of the first unit four borings were made through the fill and the underlying soft mud sampled. This sampling work was done in January 1953 at Stations 11+00, 14+00, 16+00, and 19+50. These borings were made about six months after completion of the roadway prism in the first unit.

During the second unit four sample borings were made as the working table was completed and before the start of the roadway prism. These borings were made two to four weeks after the working table had become stable at their location. The borings were made at Station 45+20, 47+00, 49+00, and 51+00. The soft bay mud remaining below the fill was sampled.

In February 1954 the mud wave, both right and left of the fill, was sampled to a depth of 45 feet. These four sample borings in the mud wave were made at Stations 44± and 47±.

During January 1957 four sample borings were made through the fill and samples of the soft bay mud underlying the completed fill obtained. These borings were made in the first and second units at Stations 14+00, 19+50, 45+00, and 49+00.

The borings in January 1953 in the first unit were made to obtain data on the strength and compressibility of the soft bay mud. As these samples were made six months after the completion of the roadway prism the strength was altered from the strength immediately after construction. In the second unit it was decided to make borings as the fill progressed to study the soil characteristics immediately after the working table was completed. These borings in the second unit were generally made about two weeks after the working table was completed in that area. It was desired to check the effects of the remolding of the soft bay mud by the fill failures. Four borings were also made of the mud wave to study its variation in strength with depth. After completion of grading operations and before paving a set of four borings were made to study the changes in strength and moisture that had occurred over a period of several years since the fill was completed. These borings were made at the locations previously sampled.

Summaries of all test data obtained during construction are attached.

Moisture Content of the Soft Bay Mud

The moisture content of the soil was plotted versus elevation for each location of the borings, see Figures D1 to D4. The scatter of the moisture contents of the individual tests are rather large, about 20 percent for any elevation and indicate the variation of the point to point moisture content of soil. The moisture content prior to construction appears to decrease with depth. The samples after completion of the working table indicate the range of moisture content is the same as prior to construction. The borings made two to four years after completion of the roadway prism show a smaller range of moisture contents and the moistures are grouped along the lower limits of the original moisture content spread. This would indicate that a small, an average of about 5 percent, decrease in moisture content has occurred.

The theoretical decrease in moisture content under full consolidation is about ten to fifteen percent. As the point to point variation in moisture content is of this magnitude it will be extremely difficult to determine an accurate decrease in moisture content by means of borings. The development of an in-place method of moisture determination of soils is necessary before small moisture changes can be accurately measured.

Variation of Shearing Strength of the Soft Bay Mud

Plots of shearing strength versus elevation were prepared at the location of each boring. The shearing strength prior to construction was determined by the average of all tests and is shown in Figure No. D5 and as a dashed line on the plots in Figure Nos. D6 to D9. The original strength was zero at the surface of the mud and increased in a linear relationship to 500 pounds per square foot at elevation -40. The strengths were grouped along this line with the maximum variation being about $\pm$100 pounds per square foot.

The samples obtained from the borings made six months after the completion of the first unit, Figures No. D6 and D7, indicate approximately the same strength as originally existed. This indication that no major change had occurred in the strength of the soft bay mud was difficult to understand. It had been expected that the reworking of the soft bay mud by the fill failures would have resulted in a decrease in strength. As a result of these tests it was decided to sample the soft bay mud as soon as possible after the completion of the working table in the second unit.

The borings in the second unit made immediately after completion of the working table, see Figures D8 and D9, indicated a major decrease in the strength of the soft bay mud had occurred. The shearing strengths were fairly uniform with depth and varied from about 200 pounds per square foot

immediately below the fill to 400 pounds per square foot at the bottom of the soft mud layer. Remolded samples of the soft bay mud indicated that the remolded strength was about 50 to 100 pounds per square foot. The indications are that the soft bay mud was partially remolded over its entire depth by the fill failures with greater remolding at the top of the mud layer than at the bottom. After the fill has set a few months without failure it appears, from the borings in the first unit, that the soft bay mud regains its original strength.

The strengths obtained from the samples from the borings made in the second unit during January 1957, two and one-half years after completion of the roadway prism, are shown in Figures No. D8 and D9. The shearing strengths have increased and were about the same as the original strengths prior to construction, and about twice the strengths obtained shortly after completion of the working table. There appears to be a slight tendency for the shearing strengths to be slightly higher at the top and bottom of the layer than in the center, as would be expected for double drainage in the soft bay mud layer.

The strengths obtained from the samples from the borings made in January 1957 in the first unit, four and one-half years after construction of the roadway prism, are shown in Figures No. D6 and D7. The shearing strength is generally 100 to 200 pounds per square foot above the shearing strength indicated by the borings prior to construction. This indicates an increase in strength is continuing to occur as the consolidation process continues.

The samples from the borings during construction indicate that the strength of the soft bay mud was reduced to about one-half its original strength by the reworking of the soft bay mud during the failures. This is in agreement with the analysis of the failures during construction. The soft bay mud appears to have regained this lost strength within a year or so after completion of the working table. The soft bay mud is now slowly increasing in strength and generally has a strength equal to or greater than its original strength.

Moistures and Strengths in the Mud Waves

The moisture content of the mud wave versus elevation is shown in Figure No. D10. The moistures are uniform to elevation -30, being from 70 to 80 percent at Station 47+00 and 65 to 85 percent at Station 44+00. This is in agreement with the range, 65 to 80 percent, of moistures indicated by the borings prior to construction.

The variation of strength in the mud waves with elevation is shown in Figure No. D11. The strength at Station 44+00 on the left and 47+00 on the right is between 50 and 100 pounds per square foot above elevation -10 and then

follows the strength existing prior to construction. At Station 44+00 on the right and 47+00 on the left the strength is between 100 and 200 pounds per square foot above elevation -20 and then tends to follow the strength prior to construction. The mud displacement is similar at all locations and would not account for the differences in strengths noted. It would appear that reworking of the soft bay mud is variable and does not produce a uniform strength mud wave. It would appear that the soft bay mud below elevation -20 was not being reworked by the shear type failures that occurred in this area.

Results of the Triaxial Shear Tests

A series of triaxial shear tests were run on the soft bay mud from the boring at Station 45+00 on centerline in the second unit. This boring was made a short time after the working table became stable and represents the remolded soil. The results of these tests are shown in Figures D12 to D14. Two types of tests were run, a quick test, and a consolidated quick test.

The quick undrained tests indicated an angle of friction of zero for the soft bay mud. The cohesion varied with the depth of the sample, and appeared to be greatly influenced by the quantity of remolding of the soft bay mud.

The consolidated quick tests indicated an angle of friction of about 13 degrees. The indicated strengths, when consolidation is completed, will vary with loading or mud displacement. With a soft mud displacement to elevation -25 a shearing strength of about 550 pounds per square foot would be indicated at the top of the layer by the consolidated triaxial tests under complete consolidation. As the loading is increased the shearing strength under complete consolidation will be greater, and may be expected to be about 900 pounds per square foot at the bottom of the soft bay mud layer, elevation -70.

A comparison of the variation of shearing strength with moisture was made using the triaxial test data. See Figure No. D15. The sample No. 1, from near the top of the soft bay mud layer, shows a large increase in strength with decreasing water content. It is believed that this soil has been extensively remolded by the fill failures. Sample No. 3, from near the bottom of the soft bay mud layer, indicates a small increase in shearing strength with a decrease in moisture content. It is believed that this soil has been only slightly remolded in the fill failures. The difference between the variation of shearing strength and moisture content appears to be a function of the remolding of the soft bay mud. Only limited data exists in this study and other variables may be influencing the results. If the remolding is affecting the strength-moisture relationship, as indicated above, the remolded soil may be stronger than the unremolded soil when consolidation is completed.

A limited number of quick drained triaxial tests were run. See Figure D13. The indications are that a slight increase in strength occurs in the drained test over the undrained test. The small and limited increase in strength in the drained tests, less than 0.1 ton per square foot, may be due to slight variations in the soil, or slight drainage occurring during testing.

Consolidation Tests

Consolidation tests were run on samples from all series of borings made. Representative pressure void ratio and time consolidation curves from the borings prior to construction are shown in Figures No. D16 to D23. The soft bay mud samples show a curve under low loadings and then a straight line, virgin curve, under the higher loadings. In calculating the estimated ultimate settlements prior to construction the slope of the virgin curve was used. Representative time consolidation curves are shown in Figures D18 and D19 for the soft bay mud. These curves indicate a permeability of about 5×10^{-5} feet per hour and a coefficient of consolidation of about 5.8×10^{-3} square feet per hour. Representative pressure void ratio curves for the stiff bay mud are shown in Figures No. D20 and D21. The stiff bay mud layer is a very hetrogeneous mixture of a silty clay with silt and sandy layers. The pressure void ratio curves reflect this mixture of soils in this layer with many slopes to the virgin curves. Typical time consolidation curves of the stiff bay mud layer are shown in Figure No. D22 and D23. The hetrogeneous nature of this layer is also reflected in the time-consolidation curves.

The settlement curves for one typical cross-section studied during design are shown in Figures No. D24 and D25. The indications were that the soft mud would consolidate at a relatively uniform rate for various thicknesses of mud underlying the fill. When the thinner mud layers had completed their consolidation the thicker mud layers would continue to consolidate and differential settlement would result. The stiff mud layer settlements indicated a uniform rate of settlement would occur, for various thicknesses of stiff mud, in the first fifty years and that the magnitude of the settlement would be small, about 0.6 feet in 20 years.

Representative pressure void ratio curves and time-consolidation durves from the borings made six months after completion of the roadway prism in the first unit are shown in Figures No. D26 and D27. The slope of the virgin curve increased slightly from the slope indicated by the samples prior to construction and the calculated permeability from the time consolidation curves decreased to about 1.35×10^{5} feet per hour. This was apparently due to the remolding of the soft bay mud by the fill failures. All theoretical ultimate settlement calculations, in the section on settlements, made for the first unit are based upon the average curve shown in Figure No. D26. The rates of settlement were calculated using a

permeability of 1.35×10^{-5} feet per hour obtained from these time-consolidation curves.

Representative pressure void ratio and time-consolidation curves for the borings in the second unit made immediately after completion of the working table are shown in Figures No. D28 to D33. The slope of the virgin curve is slightly steeper for these samples than for the samples prior to construction. The calculated permeability from the time-consolidation curves had also decreased to 1.1×10^{-5} feet per hour. This is probably due to the remolding of the soil during the fill failures. The estimated ultimate settlements in the second and third units were calculated using the pressure void ratio curve shown in Figure No. D31.

The consolidation test data from the borings made at the time of paving are shown in Figures No. D34 to D39.

The preconsolidation pressure was determined for each consolidation test. The preconsolidation pressures prior to construction are compared to the overburden pressures in Figure No. D40. The preconsolidation pressures were all within ± 0.25 tons per square foot of the overburden pressures. These pressures indicate that the soft mud was consolidated under the overburden pressures prior to construction.

The preconsolidation pressures during various stages of construction are shown in Figures No. D41 to D45 at the various locations of the borings. The borings made in the second unit immediately after completion of the working table indicate that little or no change in the preconsolidation pressure occurred during the remolding by the fill failures. This is in agreement with the excess hydrostatic pressure readings. The samples from the borings in the first unit six months after completion of the roadway prism indicate a slight increase in preconsolidation pressure had occurred. This slight increase in preconsolidation pressure is of a magnitude of 0.25 tons per square foot and is in agreement with the decrease in excess hydrostatic pressure. The preconsolidation pressures obtained from the samples taken two and one-half years after completion of the roadway prism in the second unit indicate a definite increase in intergranular pressure has occurred. It appears that the greatest increase in intergranular pressure has occurred at the top and bottom of the soft clay layer and the smallest increase occurred in the center of the layer. The preconsolidation pressures indicated that about thirty percent of the total consolidation has occurred at Station 45$\pm$ and 49$\pm$ at the time the borings were made. The sample from the borings in the first unit four and one-half years after completion of the roadway prism indicate a considerable increase in intergranular pressure has occurred. The indications are that about 50 percent consolidation of the soft bay mud had occurred at the time the borings were made. The intergranular pressure

is about equal to the present total pressure at the top and bottom of the soft mud layer. This indicates that double drainage of the soft mud layer is occurring.

Permeabilities of the Soft Bay Mud Layer

The permeabilities of the soft bay mud layer were determined by two methods. The permeability determined from the consolidation tests is referred to as the lab permeability. The piezometers were used as rising head permeameters and this permeability is referred to as the field permeability. The flow of water in the consolidation test is in the vertical direction and indicates the vertical permeability. The piezometers have principally a horizontal flow of water from the soil and indicate the horizontal permeability.

The average permeability obtained from the consolidation tests in the second unit immediately after construction of the working table was 1.1×10^{-5} feet per hour. The load increment used was the one immediately larger than the original overburden pressure at the depth the sample was obtained. It is felt that this figure represents the vertical permeability of the soil.

The permeabilities indicated by the piezometers are shown in Table No. III, and average 2.2×10^{-5} feet per hour. This permeability is felt to represent the horizontal permeability of the soft bay mud. The indications are that the horizontal permeability is twice the vertical permeability, which would indicate a fairly uniform soil condition.

TABLE NO. III

Permeabilities as Determined with the Piezometers

Piezometer Number	Permeability Feet Per Hour
1	1.3×10^{-5}
2	1.3 "
3	4.3 "
4	5.0 "
6	1.7 "
7	1.1 "
8	1.4 "
12	2.1 "
13	1.7 "
14	2.0 "
16	3.4 "
17	2.1 "
18	2.4 "
19	1.4 "

Settlements

Installations

As placement of the fill progressed settlement platforms were placed on centerline at the surface of the working table at each station. These platforms were placed shortly after the uncompacted fill had reached a stable condition. In the first and second units settlement platforms were also placed at eight locations, about 65 feet left and right of centerline, under the roadway shoulder to obtain settlement cross-sections. The above platforms were of the riser type with one foot square bases and one-half inch pipe risers through the fill.

On the first and second units, surface settlement points were placed 100 and 125 feet right and left of centerline. These settlement points consisted of iron pins driven about 15 inches into the fill. These points were for measurement of surface movement of the berm during placement of the roadway prism. Both vertical and horizontal movement of these settlement points were recorded. A few months after completion of the roadway prism readings were discontinued and these settlement points abandoned.

After completion of the first unit it was evident that considerable consolidation had occurred in the uncompacted fill under the loading of the roadway prism. In the second unit five settlement platforms were placed at the bottom of the uncompacted fill. This was accomplished by drilling a four inch hole to the desired depth. The platform consisted of four metal gate hinges operated by a screw. The platform was placed in the boring using one-half inch pipe for the riser and one and one-half inch pipe for the casing. The riser was then rotated, turning the screw and forcing the hinges into the soil. The casing was then raised two feet to allow for movement between the riser and its casing.

To check the settlement of the stiff bay mud layer four settlement platforms were installed in the second and third units. This was accomplished by driving three inch pipe through the fill and soft mud to about ten feet above the stiff bay mud. A two inch pipe was then placed inside the three inch pipe and driven five to ten feet into the stiff bay mud. The two inch pipe was then filled with concrete mortar and capped. During the construction of the fill these settlement platforms were used as bench marks as there was relatively small movement of these settlement platforms, and it was a long distance to bench marks on stable ground.

When the final grading and paving operations were going on only one settlement platform for each five stations was maintained, the others being destroyed. Continuous readings were thereafter maintained on 20 per cent of the platforms measuring settlement of the surface of the working table and all of the platforms measuring the settlement of the bottom of the fill and the surface of the stiff bay mud. The previously destroyed platforms were replaced, after grading operations, by concrete monuments at the surface of the roadway prism.

The settlement curves for all settlement platforms are attached under "Settlement Plots." These plots include records of measured settlement of the soft bay mud, the roadway prism, the underlying stiff mud, and theoretical comparisons. For convenience the settlement curves are grouped according to units. The time scale for the settlement curves starts at the start of placement of the roadway prism. The original settlement shown, at the time that the roadway prism was started, is the settlement that occurred between this date and the date that the settlement platform was installed. For reference the elevation of the bottom of the soft mud and the bottom of the fill is shown in the lower left hand corner.

The settlement platforms at the surface of the uncompacted fill that were destroyed during the final grading and paving operations have this noted on the settlement plots. The estimated settlements are shown by a dashed line during the period that the settlements were not obtainable. This dashed line is an estimated settlement only and is based upon the slope of the settlement curve prior to the platform being destroyed. As many of these curves are not a straight line, but have changes in slope, there may be some error in this estimated settlement. For studies in this report, only the platforms with complete measured settlements will be used.

The settlement of the stiff mud layer is shown on separate plots. The elevation of the bottom of the stiff mud layer is also included on these plots.

Soil Layers

There are three soil layers with varying degrees of compressibility at Candlestick Cove underlying the roadway prism. The settlement platforms on the surface of the uncompacted fill measures the settlement due to all three layers. In order to obtain the settlement due to any one layer, intermediate platforms are required. See Figure No. S-39. The settlement platform at the bottom of the uncompacted fill measures the settlement in the soft bay mud and the stiff bay mud layers. The difference in settlement between the settlement platforms at the surface and the bottom of the uncompacted fill will give the settlement resulting from the uncompacted fill. The settle-

ment platforms set in the top of the stiff bay mud will give the settlement resulting from the stiff bay mud layer. As limited intermediate platforms were installed only indications or trends can be measured.

The settlement of the roadway prism is a function of the rates of settlement of all three layers. The consolidation of the various layers for the second unit are shown on Figures No. S-19 to S-24. The consolidation of the uncompacted fill is generally small and appears to be completed within a year after completion of the roadway prism. The rock fill appears to consolidate about 0.1 foot and the soil fill about one foot under the loading of the roadway prism. The consolidation of the stiff bay mud layer during construction of the roadway prism was small. The slope of the log time settlement curves for the stiff bay mud is now increasing and at some locations may be the principle consolidation now occurring; see Figures No. S-35 to S-38. The remaining settlement of the roadway prism represents the consolidation of the soft bay mud.

As previously discussed the amount of soft bay mud remaining under the uncompacted fill varies from zero to about 50 feet in thickness. This will result in a wide variation in the amount of consolidation within the soft bay mud layer, and should be the principle cause of differential settlement of the roadway prism.

Measured Settlement of the Roadway Prism

In the first unit the centerline settlement of the surface of the working table during the placing of the roadway prism was 0.5 to 1.2 feet. This resulted in an increased slope on the log time settlement curves while the roadway prism was being placed compared to the slope prior to the placing of the roadway prism. After completion of the roadway prism the slope of the log time settlement curve decreased and then increased. The settlement is now continuing along a straight line on the semi-log plots. There is no indication that the settlement is completed at any location on the first unit. The rate of settlement is fairly uniform along centerline and only minor differential settlement, less than 0.2 foot, occurred in a two-year period between any two adjacent stations.

In the second unit the centerline settlement of the working table during the placing of the roadway prism was from 0.2 to 0.8 foot. This resulted in an increased slope on the log time settlement curves while the roadway prism was being placed compared to the slope prior to construction of the roadway prism. After completion of the roadway prism the slope of the log time settlement curve tended to decrease for a short period of time and then slowly increased. The slope of the log time settlement curve appears to be still slowly increasing and appears to be approaching a straight line.

The total settlement curves for the first and second units are very similar. They both indicate that about the same rate of settlement is now occurring with only small differential settlement between any two adjacent stations. The total settlement to date for both units is between three and five feet. The slope of the log time settlement curves increased during the placing of the roadway prism in both units. The mud displacement in both units varies widely, from small to total mud displacement. The nearly uniform amount and slope of the log time settlement curves is explained by the theoretical studies and will be presented later.

In the first and second units settlement platforms were placed 65 feet right and left of centerline. Combined with the centerline platform, a settlement cross-section of the roadway prism is obtained. These settlement platforms, at shoulder line, indicate that the settlement was uniform during the placing of the roadway prism where uniform mud displacement was obtained. Where differences of mud displacement was large, between centerline and the shoulder of the roadway prism, differential settlements up to 0.5 foot occurred. After construction of the roadway prism the rate of settlement of the three platforms forming a cross-section was very close to uniform. The platforms were within 0.1± foot of equal settlement for the period of two to three years after construction of the roadway prism.

On the paving contract the surplus soil was placed in the disposal area on the berm. As a result the settlement platforms under the shoulder line settled one-half to one foot more than the centerline settlement platform during the same period of time. The settlement due to the placing of fill in the disposal area appeared to be dependent upon the amount of displaced mud. At several locations where the soft mud, 75 feet from centerline, was displaced to less than elevation -15 cracking of the roadway prism occurred parallel to centerline. There has been insufficient time since the placing of this fill to know if the settlement cross-sections will resume the previously uniform rate of settlement.

The total settlement curves for the third unit indicate that small settlement occurred during the placing of the roadway prism. The slope of the log time settlement curve prior to construction of the roadway prism is unknown in about one-half of this unit. The slope of the log time settlement curve is a constant during construction of the roadway prism and for about six months after construction. About six months after construction the slope of the log time settlement curve sharply increases for about three months, generally resulting in about one-half foot of settlement. Where almost total mud displacement occurs, less than ten ± feet of soft mud remaining below the fill, the curves tend to return to

the original slope on the semi-log plot. Where small mud displacement occurred, more than ten ± feet of soft mud remaining below the fill, the settlement tends to continue along the same slope on the log time settlement curve. The above statements are general only with exceptions to be found. The slope of the log time settlement curves after construction of the roadway prism is not uniform in the third unit. The total settlement to date varies from one-half to four feet. The settlement platforms indicate, generally where total mud displacement occurred, that at some locations the settlement is now continuing at a very slow rate while at other locations, generally where small mud displacement occurred, the settlements are occurring at a rapid rate. Although a large mud displacement was obtained throughout most of this unit the settlements are not as consistent as in the first and second units.

On the fourth and fifth units the settlement platforms were not installed until after completion of the roadway prism. In the fifth unit there is insufficient data to date to indicate any trend. In the fourth unit the total settlement is following a straight line on the semi-log plot at about the same rate as the first and second units. The total settlement to date varies from one to two feet with no signs that the settlement is complete. The mud displacement varies widely in this unit, varying from about one-half the mud to total displacement; however, the entire unit has about the same slope on the semi-log time settlement plots.

Theoretical and Measured Rates of Consolidation of the Soft Bay Mud

Theoretical rates of settlement of the soft bay mud were calculated for all stations on the first and second units and at selected locations in the third unit. The rates of settlement of the soft bay mud were calculated using the two available permeabilities, as determined from the consolidation test and from the piezometers. Double drainage is assumed into the sand layer overlying the stiff clay and into the uncompacted fill. Figure No. S-8 shows the theoretical rates of settlements for various locations. The theoretical settlement curves indicate that the settlement will be approximately uniform regardless of the thickness of soft mud remaining below the fill. When the curves for very large mud displacement indicate that the settlement is completed, the curves for small mud displacement indicate that settlement will continue at these locations with the same slope to the semi-log time settlement curves until their ultimate settlement has been reached. The differential settlement would thus be small until the consolidation is completed for the least thickness of soft bay mud and then the thicker mud locations

would continue to settle resulting in differential settlement. These curves in Figure No. S-8 are for the soft bay mud only, and ignore consolidation of the uncompacted fill and the stiff bay mud.

A comparison between the settlement platforms measuring the total settlement of the roadway prism and the theoretical settlement due to the soft mud are shown in Figures S-1 to S-18 and S-25 to S-34. The theoretical settlements are shown for both the laboratory and field permeabilities, and assume double drainage of the soft clay layer. The settlement rates are for the case II as described on page 237, 1948 edition of Fundamentals of Soil Mechanics, by Donald Taylor. The excess hydrostatic pressure data indicate that case II generally applies at Candlestick Cove. The time scale starts at the completion of the roadway prism. The quantity of measured settlement prior to the completion of the roadway prism is noted for each settlement platform. Three assumptions are made in comparing the measured settlements and theoretical settlements: (a) that no consolidation of the soft mud occurred in the time interval between the placing of the working platform and the placing of the roadway prism, (b) that consolidation of the uncompacted fill is completed by the time the roadway fill was completed, and (c) consolidation of the stiff bay mud can be ignored. None of the above assumptions is completely valid. The uncompacted fill and stiff bay mud consolidation will be discussed separately. There was some consolidation of the soft bay mud prior to the placing of the roadway prism. The piezometers indicate that this was a very small amount, probably less than one-half of a foot, depending on the time between completion of the working platform and the placing of the roadway prism. The settlement platforms are in agreement and indicate that less than 0.3 feet of settlement occurred prior to the start of the roadway prism. The consolidation of the soft mud during the placing of the roadway prism was assumed to be small enough to be ignored, and this was confirmed by the rise in excess hydrostatic pressure equal to the weight of the fill placed. The settlement platforms showed considerable settlement, but it was assumed that this primarily represented consolidation of the uncompacted fill and any plastic flow of the soft mud that may have occurred.

The measured settlements indicate that the roadway prism is settling at approximately the rate which is indicated by the theoretical curves for the soft bay mud. However, where the theoretical curves indicate that the consolidation process should be completed for the soft bay mud, the measured settlements continue along the same straight line on the semi-log plots and are now larger than the estimated ultimate settlement of the soft bay mud at several locations. It was

realized that the bottom of the fill was only located at a small point within one foot ± accuracy, and that the bottom of the soft mud is subject to considerable inaccuracy. The settlement platforms include settlement that may be due to plastic flow which would increase the quantity of recorded settlement.

In the second unit data are available to determine the settlement due to each of three soil layers below the roadway prism. See Figures S-19 to S-24. The indications are, in the second unit, that from one to one and one-half foot of the total settlement after completion of the roadway prism is due to consolidation of the uncompacted fill and the stiff bay mud. The measured settlement using the settlement platforms at the top and bottom of the soft mud, is shown on the Figures S-19 to S-24 along with the theoretical rate of settlement. The measured settlements of the soft bay mud indicate that the rate of settlement of the soft mud is proceeding at a rate equal to or less than that indicated by the theoretical studies. At Station 50+00 and 53+00 there are indications that the consolidation of the soft mud may be nearing completion. At several locations adjacent to near total mud displacement, Stations 45, 50 and 53, the settlement is occurring slower than the theoretical indicates.

The excess hydrostatic pressure readings were used to calculate the amount of settlement at various times. These settlement estimates are based upon the calculated ultimate settlement of the soft bay mud layer and will reflect any error in this value. The shape of the excess hydrostatic pressure curves was assumed to follow the theoretical shape with depth, with the pressure readings determining the location of the curve. The percentage of the area under the excess hydrostatic pressure curve to the total pressure curve is the percentage settlement that had occurred on the date of the readings. The excess hydrostatic pressure readings indicate that the soft mud is following the theoretical rate of consolidation.

Theoretical Rates of Consolidation of the Uncompacted Fill

It was not possible to obtain in place samples of uncompacted fill; therefore, no theoretical ultimate or rates of settlements could be calculated. It was hoped that the data from the second unit would establish a relationship between the thickness of the uncompacted fill and the settlement due to this layer. These data are very erratic and no such relationship is apparent. It is apparent that the type of fill used is an important factor. Records were also kept of the general type of fill used at each location; however, the data are general in nature, and it appears more specific

records would be required to determine a relationship. The uncompacted fill where rocky borrow was used consistantly indicated smaller settlement than where soil type borrow was used, regardless of the thickness of uncompacted fill.

The measured settlement of the uncompacted fill is shown in Figures No. S-19 and S-24. The settlement due to the uncompacted fill has varied widely. The settlement which has occurred since completion of the roadway prism varies from 0.1 to 0.4 foot and is either completed or continusing at such a slow rate as to be beyond the limits of error of measurement.

In June 1954 and July 1957 the outer casings were jacked up on the settlement platforms at the bottom of the fill to provide additional clearance between the settlement platform and the bottom of the casing. The elevation of the top of the settlement platform riser was noted before and after the casing was raised with no movement noted. There is no indication that the platform is following the movement of the roadway prism due to friction against the riser at the slip joint or pushing against the platform.

Theoretical Rates of Consolidation of the Stiff Bay Mud

Settlements of the surface of the stiff bay mud was measured at only four locations. Because of the very limited data only general trends can be noted. The theoretical settlements were calculated assuming double drainage even though the stiff bay mud is underlain by the Franciscan formations. The reason for this assumption is that extensive sand layers or pockets exist in this stiff mud layer, and they probably provide some drainage.

The ultimate settlements of this layer are rather large, being from two to five feet. The estimated theoretical rate of settlement is very slow, with an estimated one to two hundred years being required for its completion. The theoretical rate of settlement of the stiff bay mud up to 10,000 days is shown in Figures No. S-35 to S-38.

The measured settlements for the stiff bay mud are also shown on Figures No. S-35 to S-38. The slope of the measured settlement curve of the stiff bay mud was small after completion of the roadway prism and is now increasing and now indicates this settlement may be completed in about fifty years. This increased slope of the settlement curve is probably due to the existence of the sand beds located throughout this layer.

A comparison of the rates of settlement of the surface of the uncompacted fill and the stiff bay mud are shown in Table No. IV. The body of the table represents the settlement due to the uncompacted fill and the soft bay mud remaining under the fill after completion of the roadway prism. Evidence from the

second unit is that the rocky fill used in the third unit will show very small settlement and may be of a magnitude of about 0.1 to 0.4 foot. The settlement platforms with continuous readings within 500 feet of the settlement platform in the top of the stiff mud only were used in this comparison. The settlement platforms in the stiff mud are at Stations 55±, 69±, and 81±. At Stations 71, 81, 83, and 86 the borings indicated five feet or less of soft mud remains under the fill and Table No. IV indicates from 0.4 to 1.1 feet of settlement has occurred in the uncompacted fill and the soft bay mud. This quantity of settlement has remained reasonably constant from 200 days after start of the roadway prism to the present time. The indication is that the settlement of the roadway prism is resulting from consolidation of the stiff bay mud at the above locations. At Stations 61, 66, and 74 from 11 to 22 feet of soft mud remains below the fill. At Station 61, 66, and 74 there has been between 1.6 and 2.6 feet of settlement within the uncompacted fill and soft bay mud since completion of the roadway prism. This appears to be a reasonable amount of settlement due to these two layers. There is no indication that this settlement due to the soft bay mud and uncompacted fill is completed at the present time.

During July 1957 the three inch casing used in the platforms at the top of the stiff mud layer were jacked up about one-half foot. The inner pipe was checked with a level and no change in elevation noted. It appears that the casing is not dragging down the inner pipe.

Remarks on Settlement of the Roadway Prism

As the roadway prism is a compacted fill of less than ten feet thickness and under no appreciable loading, only minor consolidation would be expected within this roadway prism. The settlement of the surfaced roadway and the roadway prism are assumed to be the same, and the settlement of the roadway prism will indicate the settlement of the surfaced roadway. The settlement of the roadway prism is dependent upon the three layers underlying it.

Consolidation of the uncompacted fill progressed rapidly during the placing of the roadway prism and now appears to be essentially completed. The consolidation of the soft bay mud started after completion of the roadway prism and the slope of the log time settlement curve is large. The stiff bay mud surface settled a small amount during construction and for a short period after completion of the roadway prism and is now settling at an increased slope on the semi-log plots. Settlement of the roadway prism can be predicted only if all three of these layers are considered.

Where only a small mud displacement occurred the principle layer governing the settlement of the roadway prism after construction is the soft mud layer. The settlement due to consolidation of the stiff bay mud is small but due to its increasing slope on the semi-log time settlement plots may indicate an increasing slope on the semi-log plots for settlement of the roadway prism.

Where total or nearly total mud displacement occurred the principle layer governing the rate of settlement will be the stiff mud layer. The assumption is made that the uncompacted fill is no longer consolidating. At two locations in the third unit, Stations 69 and 81, there are settlement platforms at the surface of the stiff bay mud and at the surface of the uncompacted fill where total mud displacement occurred. During 1957 the settlements of these platforms indicated that the surface of the uncompacted fill was closely following the settlement due to the stiff bay mud.

With mud displacements between these two extremes an irregular settlement pattern occurs. The settlement of the roadway prism is dependent upon both the soft bay mud and the stiff bay mud.

Considering all of the factors it appears that continual settlement of the roadway prism at a constant slope on the semi-log plots can be expected. Differential settlements will probably be small in the next few years; then they can be expected to increase. The settlements will continue at some locations for the next fifty years or more, with ultimate settlement from one to eight feet above those which existed at the time of paving.

TABLE NO. IV

Comparison of the difference in settlement of the surface of the uncompacted fill and surface of the stiff bay mud with soft bay mud displacement; third unit at Candlestick Cove.

Location of Settlement Platform at Surface of Stiff Mud	Location of Settlement Platform at Surface of Uncompacted Fill	Elevation		Soft Mud Settlement in Feet At _____ Days*					
		Bottom of Fill	Bottom Soft Mud	100	150	200	400	700	1000
55+00	56+00	-25	-57	0.25		0.30	0.65	1.70	1.95
69+00	61+00	-54	-75	0.90		1.10	1.30	1.50	1.65
69+00	66+00	-18	-40		1.70	1.95	2.30	2.50	2.65
69+00	71+00	-36	-37		0.35	0.40	0.45	0.45	0.45
69+00	74+00	-43	-54			0.90	1.30	1.60	1.60
81+00	81+00	-40	-45			0.50	0.50	0.50	0.50
81+00	83+00	-39	-41			0.60	0.60	0.60	0.60
81+00	86+00	-45	-45			1.00	1.05	1.10	1.15

NOTE: Difference in settlements recorded to the nearest 0.05 foot.

*As determined by the difference in settlement as measured by the settlement platforms at the top of the uncompacted fill and the top of the stiff mud.

CANDLESTICK COVE REPORT - PART VI

Excess Hydrostatic Pressure Readings

Installations

In the first unit four piezometers were placed in the soft mud underlying the uncompacted fill. One piezometer was placed on centerline at Station 8+00, one on centerline at Station 14+00, and one 60 feet right of centerline at Station 14+00, and one 130 feet right of centerline at Station 14+00. One piezometer was placed in the shale underlying the soft bay mud at Station 9+00 on centerline. A piezometer was also placed in the sand between the soft bay mud and the stiff bay mud at Station 11+00, 100 feet left of centerline. The piezometers were installed using a one and one-half inch pipe with a plugged end driven through the fill and into the soft mud to the desired depth. A one and one-half inch diameter porous stone one foot long was then pushed into the soft mud so that the top of the porous stone was just below the bottom of the pipe. A plastic tubing had been previously attached to the porous stone and extended through the pipe. The space between the plastic tubing and the pipe was sealed with compacted bentonite. Due to the high excess hydrostatic pressures in the mud a gage had to be connected to the top of the plastic tubing. When the roadway prism was placed a one quarter inch copper tubing was used to place the gage outside the roadway prism so that it would be out of the contractors work area. See Figure No. P-1. Unfortunately, all of the piezometers in this unit became inoperative shortly after placement of the roadway prism. The plastic tubing broke near the lower end of the pipe, and electrolytic action destroyed the copper line and fittings.

In the second unit twenty-two piezometers were installed and all are operating at the present time. A two-inch galvanized pipe with a plug on the lower end was driven through the fill so that the lower end was five to ten feet above the desired location of the piezometer. The porous stone with the attached plastic tubing was then pushed past the lower end of the pipe to the desired depth using hollow rods and a thin metal shield around the porous stone to protect it. During these operations the soft mud flowed into the pipe several feet and it was felt that with this mud under such pressure a seal was forming between the soft bay mud and the exposed plastic tubing. The two inch pipe was extended through the roadway prism during the placing of this compacted fill and a gage assembly was used as in the first unit. A regular bentonite seal was used between the two inch pipe and the plastic tubing. See Figure No. P-1. After completion of grading operations the installations on centerline were lowered and the gages placed in eight inch casing flush with the surface of the ground, so that cars could pass over them. The pipes were left about five feet above the ground level in the piezometer installations left of centerline and boxes were attached to the pipe to house the gages.

Excess Hydrostatic Pressure Readings in the First Unit

The piezometers, on centerline, in the soft mud underlying the uncompacted fill in the first unit indicated that little consolidation occurred during the period between completion of the working table and completion of the roadway prism. The excess hydrostatic pressure decreased 0.05 to 0.10 tons per square foot in the 60-day period, prior to the start of the roadway prism. At the start of the roadway prism the excess hydrostatic pressure reading at Station 8+00 was 0.70 tons per square foot under a loading of 0.75 tons per square foot. When the roadway prism was completed an excess hydrostatic pressure of 1.30 tons per square foot was recorded under a loading of 1.35 tons per square foot. The piezometer readings indicate that very little consolidation occurred at Station 8+00 prior to completion of the roadway prism. The excess hydrostatic pressure at centerline of Station 14+00 was 1.00 tons per square foot prior to construction of the roadway prism under a loading by the fill at Station 14+00 of 0.70 tons per square foot. The mud displacement at Stations 13+00 and 15+00 was larger than at Station 14+00 and evidently produced an effective loading of about one ton per square foot at Sta. 14+00. At the completion of the roadway prism the excess hydrostatic pressure was 1.95 tons per square foot with a fill loading of 1.65 tons per square foot. The piezometer indicates that at Station 14+00 there was no appreciable consolidation of the soft mud prior to completion of the roadway prism.

Shortly after completion of the roadway prism the piezometer at Station 14+00 on centerline failed, leaving only the piezometer at Station 8+00 to record the centerline excess hydrostatic pressure. The piezometer at Station 8+00 functioned for three months after completion of the roadway prism before failing. During this three-month period the excess hydrostatic pressure dropped to 1.0 tons per square foot and followed approximately a straight line on a semi-log plot.

A piezometer 60 feet right of centerline at Station 14+00 approximately below the shoulder line of the roadway prism was one of the more successful installations. The excess hydrostatic pressure at the start of the roadway prism was 1.35 tons per square foot under a fill loading of 0.90 tons per square foot. At the completion of the roadway prism an excess hydrostatic pressure of 1.80 tons per square foot was recorded under a fill loading of 1.65 tons per square foot. The indications are that the entire increase in the centerline fill loading was not being measured by this piezometer. The excess hydrostatic pressure dropped to 1.05 tons per square foot six months after completion of the roadway prism, and tended to follow a straight line on the semi-log plot.

A piezometer 130 feet right of centerline at the edge of the working table at Station 14+00 was read for a time. This installation was made just prior to the placing of the roadway prism. The

recorded excess hydrostatic pressure at the start of the roadway prism was 0.65 tons per square foot and 0.80 tons per square foot at the end of the placing of the roadway prism. This increase in excess hydrostatic pressure due to loading at a considerable distance from the piezometer appears to be similar to the high pressure observed at Station 14+00 on centerline. This piezometer failed about a month after completion of the roadway prism after indicating a slight decrease in excess hydrostatic pressure.

The piezometer, placed in the weathered shale, at Station 9+00 on centerline indicated an excess hydrostatic pressure of 0.10 tons per square foot. During the placing of the roadway prism the excess hydrostatic pressure rose to 0.15 tons per square foot and then dropped in two weeks to 0.10 tons per square foot. This indicates that the weathered shale probably is providing drainage for the overlying soft bay mud.

The piezometer in the clayey sand underlying the soft bay mud 100 feet left of Station 11+00 indicated an excess-hydrostatic pressure of 0.4 ton per square foot prior to placing the roadway prism. During the placing of the roadway prism the excess hydrostatic pressure rose to 0.9 ton per square foot. After completion of the roadway prism the excess hydrostatic pressure slowly decreased to 0.2 ton per square foot in a six months period, at which time the piezometer failed. The excess hydrostatic pressure readings indicate that this sandy layer is probably providing drainage for the soft bay mud.

Layout of the piezometers in the second unit

The piezometers in the second unit were laid out using the results of the first unit as a guide, so as to obtain a half cross-section of the excess hydrostatic pressure for every three hundred feet of fill length. Due to the contractors truck traffic pattern the cross-section was was made left of centerline. Where abrupt changes of mud displacement occurred on centerline additional piezometers were installed. The following pattern was followed in each cross-section;; one group of piezometers on centerline, one group of piezometers 75 to 80 feet left of centerline, and one piezometer at about the back crest of the mud wave. At Station 44+00 additional piezometers were placed 130 feet and 320 feet left of centerline to develop a more complete cross-section. Each group of piezometers consisted of one or more piezometers at various elevations depending upon the amount of soft mud remaining under the fill.

A total of four cross-sections were installed using a total of twenty piezometers. Two additional piezometers were installed in the sand to clayey sand layer between the stiff bay mud and the soft bay mud.

Excess Hydrostatic Pressure Variation with Time

Plots of excess hydrostatic pressure versus the logarithm of time were prepared for each piezometer. Figures No. P-2 to P-10. The time scale starts with the beginning of the placing of the roadway prism. Prior to the placing of the roadway prism a slight, less than 0.1 ton per square foot, decrease in excess hydrostatic pressure occurred. During the placing of the roadway prism the load was increased 0.6 ton per square foot. This increase in weight was reflected by a 0.5 to 0.6 tons per square foot rise in the centerline excess hydrostatic pressure and small, 0.1 to 0.2 ton per square foot rise 75 to 80 feet from centerline, but with no noticeable effect under the mud wave.

After completion of the roadway prism the excess hydrostatic pressure at centerline decreased slowly from about two tons per square foot to about one ton per square foot at the present time. The rate of decrease on the semi-log plot of excess hydrostatic pressure at centerline was fairly rapid for a short time after completion of the roadway prism and is now proceeding at a slower rate. During the final grading and paving operations, of the second unit, the fill was raised one to two feet to bring the roadway back to desired grade. The centerline piezometers reflected this change in loading. There has been insufficient time since paving to establish a rate of decrease on the semi-log plot of excess hydrostatic pressure after completion of paving. From the data available between completion of the roadway prism and the time of paving it appears that three to twenty-five years will be required, at the various locations, for completion of the consolidation of the soft bay mud.

The piezometers installed outside the toe of the roadway prism, 75 to 80 feet left of centerline, showed less than 0.2 ton per square foot increase in excess hydrostatic pressure as the roadway prism was placed. This represents about thirty percent of the loading at centerline. This would indicate that the uncompacted fill is not spreading the load of the roadway prism and agrees with the cracking of the berm observed during construction of the roadway prism. The excess-hydrostatic pressure outside the roadway prism is decreasing at a slower rate on the semi-log plots than at centerline. During placing of the roadway excavation in the disposal area the excess hydrostatic pressure rose about 0.2 ton per square foot under an increased loading of 0.25 ton per square foot.

The excess hydrostatic pressure below the mud wave is small, about 0.5 ton per square foot. The placing of the roadway prism and roadway excavation in the disposal area had no noticeable effect upon this excess hydrostatic pressure. The decrease in excess hydrostatic pressure below the mud wave has been very small, 0.1 to 0.2 ton per square foot, and has been very erratic due to tidal effects. The piezometers are subject to damage during winter storms and are not expected to remain operational more than a few years. The loading of the soft bay mud due to the mud wave is about 0.2 ton per square foot or less than one-half of the excess hydrostatic pressure measured by the piezometers.

The two piezometers placed in the sand to clayey sand layer between the soft bay mud and the stiff bay mud indicate that about 0.2 ton per square foot excess hydrostatic pressure exists in this layer. With this small constant pressure it is felt that this sandy layer is providing bottom drainage for the soft bay mud layer.

Variation of Excess Hydrostatic Pressure

The excess hydrostatic pressures in profile and cross-section are shown in Figures No. P-11 to P-16. It may be noted that the piezometers often indicate pressures that are in excess of the loading due to the fill above the piezometers. The loading at various points within one hundred feet of the piezometers were investigated and it was found that the piezometers tended to indicate the pressure within this distance. See Table No. V. The indications are that the excess hydrostatic pressures are being affected by the maximum mud displacement within a radius of about 100 feet. The effect that this has is to result in a fairly uniform excess hydrostatic pressure under the fill that is not reflecting the extremes of the soft mud displacement. The effect that this will have on the rate of settlement is uncertain. However, it appears reasonable that the rate of consolidation on the semi-log plots of the soft bay mud in areas of small mud displacement may be slower than might have been expected until the excess hydrostatic pressures below the areas of large mud displacement have decreased to values compatible with the loading of the small mud displacement areas.

Table No. V

Comparison of Excess Hydrostatic Pressures
and Increase in Loading Due to the Fill
(Readings on April 5, 1954)

Piezometer No.	Excess hydrostatic pressure Tons/sq. ft.	Fill loading in tons/sq.ft. Above Piezometer	Fill loading in tons/sq.ft. Adjacent to Piezometer
1	1.80	1.55	1.90
2	1.60	1.00	1.55
3	1.05	0.65	1.55
4	0.80	0.75	1.55
5	0.50	0.20	0.75
6	1.25	1.25	Less than 1.25
7	1.55	1.25	Less than 1.25
8	1.15	0.65	1.25
9	1.20	0.65	1.25
10	0.45	0.25	0.65

This effect of loading at a considerable distance from the piezometer would be expected of a highly stratified soil. Samples that have been taken of the soft bay mud have shown no visible

stratification. The shell layers are probably more pervious than the soft bay mud; however, they are not extensive, and none were located by borings, under the second unit. It is surprising that the roadway prism did not show a greater effect upon the piezometers 75 to 80 feet left of centerline when this apparent horizontal transmission of excess hydrostatic pressure is considered.

Profile and Cross-sections of the Excess-Hydrostatic Pressures.

The centerline profile of the excess hydrostatic pressure is shown in Figure No. P-16. To eliminate the effect of the variation of excess hydrostatic pressures with depth the maximum pressures were calculated using theoretical equations. It was assumed that both top and bottom drainage of the soft bay mud occurred--that is, drainage occurred into the uncompacted fill and into the sand to clayey sand. From the water table variation with tide it is evident that the uncompacted fill is relatively pervious. Piezometers No. 21 & 22, in the sand to clayey sand layer indicate a very small excess hydrostatic pressure exists and the permeability tests indicate a pervious soil. The value of the consolidation ratio ($U_z = 1-U/_{Ui}$) was determined from the excess hydrostatic pressure. Knowing the consolidation ratio and the location of the piezometer in regard to thickness of the layer (defined as Z/H) the time factor was calculated using equation 10.28 page 233, Fundamentals of Soil Mechanics by D. W. Taylor. Assuming the time factor as constant the consolidation ratio was calculated for various values of Z/H and the curves plotted on Figures P-11 to P-15. Where two piezometers were available both excess hydrostatic readings were used in the solution of equation 10.28.

The profile of the excess hydrostatic pressure on centerline at the completion of the roadway prism does not reflect the increase in loading of the soft bay mud due to the fill. At nearly all of the locations of the piezometers the excess hydrostatic pressure was greater than the total loading of the fill at these points. Action of the fill, as a three-dimensional body, is believed responsible for the high pressures recorded by the piezometers. As the consolidation process has occurred the maximum excess hydrostatic pressure has decreased at a variable rate. At Station 44 the maximum excess hydrostatic pressure has dropped from 2.4 to 0.8 tons per square foot while at Station 45 the pressure has dropped from 2.0 to 0.9 tons per square foot. In examining the rate of decrease in maximum excess hydrostatic pressure it may be noted that the pressure at Station 44 dropped rapidly until the pressures at Stations 44 and 45 were about equal and have since been decreasing at about the same rate. Station 44 has greater mud displacement than in the area of Station 45. The displacement of the soft mud in the area of Station 47 is fairly uniform. At this location the maximum excess hydrostatic pressure has decreased from 2.1 to 0.6 tons per square foot at a nearly constant slope on the semi-log plot. At Station 50 a small decrease in the maximum excess hydrostatic pressure has occurred, dropping from 2.4 to 1.6 tons per square foot. The almost complete displacement of soft mud at Station 49

is apparently affecting the excess hydrostatic pressure at Station 50. At Station 53 the displacement of the soft mud is slightly larger, about 10 feet, on three sides. The excess hydrostatic pressure was close to the loading due to the fill at the completion of the roadway prism. The maximum excess hydrostatic pressure at Station 53 has been decreasing at a nearly uniform slope on the semi-log plot from 1.6 to 0.9 tons per square foot.

The measured settlement curve for the soft bay mud, Figure No. S-20, for Station 44 indicate that complete theoretical consolidation of the soft bay mud layer has now occurred. This consolidation occurred at approximately the calculated theoretical rate. At Station 45, see Figure No. S-21, the slope of measured settlement curve on the semi-log plot of the soft bay mud was less than the theoretical slope and now appears to be increasing. It appears that the slope of settlement curve was close to the same for Stations 44 and 45 until the maximum excess hydrostatic pressures with depth became close to the same value. The slopes of settlement curves on the semi-log plot then diverged with Station 45 increasing its slope. This is probably due to existence of 15 feet of soft mud at Station 44 and 35 feet of soft mud at Station 45. The settlement as determined by the excess hydrostatic pressure followed closely the calculated theoretical and measured slopes of the settlement curves at Station 44 while at Station 45 the settlement as determined by the excess hydrostatic pressure followed the calculated theoretical slope very closely but does not agree with the measured settlement curve. At Station 47 the measured settlement curve, see Figure No. S-22, is proceeding at a smaller slope than expected from the calculated theoretical settlement curve. The slope of settlement curve as determined by the excess hydrostatic pressures are closely following the calculated theoretical slope. It would be expected that the slope of the measured settlement curve at Station 47 would follow the calculated theoretical slope very closely due to the relatively uniform mud displacement in the area of Station 47, however, this is not occurring. At Station 50+00 the slope of the measured settlement curve is at a smaller slope than indicated by the calculated theoretical curve and the slope of settlement curve indicated by the excess hydrostatic pressures closely follows the calculated theoretical settlement curve. At Station 49 the soft mud was displaced to a greater depth than at Station 50 which is a similar condition to Station 44 and 45, and similar results were obtained. At Station 53 the slope of the measured settlement curve is at a small slope when compared to the calculated theoretical slope and the excess hydrostatic pressures indicate a slope close to the theoretical slope. The conditions at Station 53 are similar to the conditions at Station 47 and similar results were obtained.

A study of the excess hydrostatic pressures and the measured settlement of the soft bay mud indicate that a bridging action may be occurring in the fill. A bridgingaction in the fill between

Stations 44 and 45 is not surprising; however, for this bridging action to extend several hundred feet is difficult to understand. The indications are that where large soft bay mud displacement by the uncompacted fill was obtained the consolidation of the soft bay mud is closely following the calculated theoretical rate. At locations where small soft bay mud displacement by the uncompacted fill was obtained the excess hydrostatic pressure is closely following the calculated theoretical rate of consolidation; however, the measured vertical movement is occurring at a slower rate than the theoretical. No satisfactory explanation for this effect is known at this time. It is possible, but not likely, that the estimated ultimate settlements of the soft bay mud are grossly in error and are affecting the theoretical rates. The results of the above conditions are that the consolidation of the soft bay mud is resulting in settlement at an unexpected uniform rate and small differential settlement has occurred to date. As to what will happen in the future, no predictions can be made at the present time from the excess hydrostatic pressure readings.

The half cross-sections of the excess hydrostatic pressure are shown in Figures No. P-11 to P-15. The top portions show the variations of excess hydrostatic pressures with depth at the locations of the piezometers. The lower portion of the figures are a plot of the maximum excess-hydrostatic pressure at various distances from centerline.

At Station 44+00 the greatest displacement of the soft mud occurred on centerline. The excess hydrostatic pressure indicates that a lateral gradient exists to over 300 feet from centerline. As the excess hydrostatic pressure has decreased there has been a tendency for the pressure to equalize under the fill. At Station 47+00 the half cross-sections shows that a very small mud displacement occurred 75 feet left of centerline. On centerline there has been a rapid decrease in excess hydrostatic pressure while at 75 feet left of centerline a very small decrease of excess hydrostatic pressure has occurred. The excess hydrostatic pressure on centerline is now 0.6 ton per square foot and 75 feet left the pressure is now 1.1 tons per square foot. This is similar to what has occurred on centerline adjacent to large displacement areas. At Station 50+00 about one-half of the mud was displaced on centerline and the mud displacement decreased left of centerline. A substantial lateral pressure gradient exists to the left of centerline and has only slightly decreased. At Station 53+00 a ten-foot greater mud displacement is indicated 75 feet left of centerline than on centerline, with 30 to 40 feet of mud remaining at this cross-section. The excess hydrostatic cross-section indicated relatively high and uniform pressure under the fill and rapid decrease beyond the edge of the fill.

The half cross-sections of the excess hydrostatic pressures are in close agreement with the centerline excess hydrostatic pressures. These readings show a definite transfer of excess pressure in a horizontal direction for distances as great as 200 to 300 feet. A high concentration of pressure at one point may influence the pressure at other points where comparatively small pressure would normally be expected. When this three-dimensional effect is considered, the measured excess hydrostatic pressures do not appear unreasonable.

The excess hydrostatic pressure measurements indicate that a rapid decrease in pressure occurred in areas of large mud displacement until the pressures were about equal to the pressures below adjacent smaller mud displacement areas. Then the two areas tended to have the excess hydrostatic pressures decrease at about the same rate. This effect appears to exist over an area of two to three hundred feet and is not reflected in the rate of consolidation, as measured by the vertical movements. It appears that two actions may be occurring that are affecting the settlement and excess hydrostatic measurements. The excess hydrostatic pressures are being transferred in a horizontal direction up to two hundred feet and the fill is acting as an arch or bridge for a distance of 200 to 300 feet in the locations of small mud displacement.

CANDLESTICK COVE - PART VII

Results and Conclusions

Stability

The primary object of the two test sections at Candlestick Cove was to determine how to construct a stable fill across the open water section of this road. It was found that a 200 foot wide working table would successfully support the roadway prism when the soft mud was displaced to elevation -20 or greater. Attempts to obtain uniform mud displacement were unsuccessful so a maximum mud displacement was used. This resulted in a fill with a variable thickness that had a varying amount of soft mud remaining below the fill.

As the nose of the working table was advanced failures would occur. From a study of these failures two type of movement were found to occur: 1. (Shear Type) A rotary movement where the fill would have a large drop at the crack and small or no movement at the edge of the fill, 2. (Squeeze Type) The drop would be uniform over the failed area with occasionally a small horizontal movement occurring. The shear type failure was the normal type of failure in the first and second units. Shear circle analysis for the shear type failure indicated that the soft mud had an average shearing strength of 155 pounds per square foot. On the few occasions that the fill was rapidly advanced with small mud displacement the squeeze type failure occurred. Stability analysis of the squeeze type failure indicated shearing strengths of 200 to 300 pounds per square foot for the soft mud.

The borings prior to construction indicated shearing strengths of about 200 pounds per square foot at an elevation of -20 and increased with depth. In the shear type failures it appears that considerable remolding occurred along the shear circles reducing the strength of the soft mud. In the squeeze type failures the soft mud a short distance below the fill was moved horizontally with large remolding and vertical movement of the fill ceased when the strength of the mud approached 200 pounds per square foot. When the soft mud was displaced to elevation -20 to -30 the movement would cease, in the squeeze type failure, with the fill becoming stable. A modified shear type failure was associated with controlled placing of the fill. This type of failure predominated in the third, fourth and fifth units.

A study of the failures resulting from displacing the mud and from boring data indicated the strength of the soft mud was reduced to about one-half its original strength when the working table was completed. Within one or two years the soft bay mud regained its original strength and is continuing to

increase as consolidation occurs. This increase in strength of the soft mud will increase the stability of the fill.

A ten-foot high compacted fill was placed on the working table and constituted the roadway prism. The placing of the roadway prism did not result in any shear failures. At five locations there were indications that a plastic flow failure occurred where the soft mud had been displaced to less than elevation -20. At all locations where the soft mud had been displaced to below elevation -20 the working table successfully supported the roadway prism. All indications are that a stable fill has been successfully constructed. With the increasing strength of the soft mud as consolidation occurs the stability of the fill will increase.

Settlements

The settlement of the roadway prism is dependent upon the three layers underlying it. These soil layers have varying degrees of compressibility, and are of varying thickness. The roadway prism is underlain by the uncompacted fill. Below the uncompacted fill is the soft bay mud, varying from zero to about 50 feet in thickness. A stiff bay mud layer underlies the soft mud.

The uncompacted fill consolidated rapidly during the placing of the roadway prism, and it now appears the consolidation is complete.

The consolidation of the soft bay mud was small during the placing of the roadway prism and then after construction of the roadway prism increased its slope on the semi-log plots. This layer will be the principle cause of differential settlement due to the varying thickness of this soft mud layer. The excess hydrostatic pressures indicate that bridging action may exist in areas of small mud displacement adjacent to an area of large mud displacement. The calculated theoretical rates of settlement, for various depths of soft mud displacement, indicate that the rate of settlement will be approximately uniform regardless of the thickness of soft mud remaining below the fill. When the settlement for very large mud displacement is completed, the settlement of the areas of small mud displacement will continue at their same rate. This would then produce differential settlement. Due to the bridging action of the fill the settlements have been fairly uniform in areas of varying mud displacement.

Where total or nearly total mud displacement occurred the principle layer governing the rate of settlement will be the stiff mud layer. The stiff mud layer appears to be increasing its rate of consolidation on the semi-log plots at this time. As the stiff mud layer underlies the entire area of the open water fill only minor differential settlement should result from this layer.

Continual settlement of the roadway prism at a constant rate on the smi-log plots can be expected, and the rate may be expected to vary from station to station. The differential settlements will probably be small in the three to five years after paving, and then they may be expected to increase rapidly. The settlements will continue for the next fifty years, with ultimate settlements to vary from one to eight feet in excess of that which existed at time of paving.

Construction

The effects of many variables upon construction techniques were studied. These studies were primarily upon the construction of the working table, or uncompacted fill. The working table was constructed by end dumping methods. This consisted of placing the fill material as close to the edge of the fill as practical and using a bulldozer to push the dirt over the side. Sections of the working table would then fail and drop. The contractor would then bring the failed area back to grade and attempt to maintain the fill at the required grade. These failures resulted in a displacement of the soft bay mud by the fill and a resulting mud wave.

The items that affect the displacement of the soft mud, by the fill, can be placed in two classes: items that can be controlled by man, and the items that cannot be controlled by man. The controllable class include the shape of the nose, rate of placing fill, type of fill material, and the elevation to which the working table is carried. The uncontrollable class include the affect of the tide, strength of the bay mud, and the extent of the mud wave.

The use of a wedge shaped nose, at about forty-five degrees to centerline, effectively split the mud to about 100 feet from centerline. Odd shaped noses used tended to build up mud waves at various locations around the nose. The shape of the nose tended to control the shape of the cross-section of the bottom of the fill as determined by borings.

The desired rate of placing fill is that rate which will maintain the top of the working table at the desired elevation and still slowly advance the nose of the fill. The minimum requirement of 10,000 cubic yards per day appeared satisfactory.

The type of fill had an apparent affect upon the starting of a failure. Once a failure had started the type of fill did not appear to have any affect upon the rate at which failure occurred.

The elevation of the top of the working table is related to the driving force in displacing the mud. The tide limited the elevation of the working table to above elevation +10. During periods when it was desired to increase the driving force the elevation of the working table was raised at the edge of the fill, the maximum elevation being +22. To keep the mud moving it was found that the top of the working table had to be three to five feet above the crest of the mud wave.

At low tide the fill would tend to fail and at high tide the fill would tend to be stable. The weight of the water, at high tide, acted to resist failure of the fill by supporting the mud wave. The tide drop that resulted in fill movement was about three to five feet.

The strength of the soft bay mud was reasonably uniform in the area of the open water fill.

The mud wave acted as a berm or support for the fill, greatly increasing its resistance to failure. In the completed fill section, with the roadway prism in place, stability analysis indicated that the mud wave contributed about 30 per cent of the resisting force. The tide had considerable affect upon the height of the crest of the mud wave. As the tide stage rose the crest of the mud wave would build up as the failure of the fill occurred. As the tide stage decreased the mud would start failing within itself and tend to flow away from the fill. The quantity of displaced mud determined the extent of the mud wave and the tides along with the strength of the mud in the mud waves determined the height of the crest of the mud waves.

Mud blasting was used to prevent large build-up of the crests of the mud waves. The blasting of the mud wave was of great value in "liquefying" the mud or reducing the strength of the soft mud. This enabled some degree of control of the resisting force to fill failures by the mud wave.

As construction progressed it was learned how to vary the above items so as to obtain maximum mud displacement. This was done by varying the controllable factors affecting the mud displacement in such a manner so as to counteract the uncontrollable factors. A record of the yardage placed at each station was kept so as to estimate the amount of mud displaced. The major factor affecting the failures, and as a result affecting the mud displacement, was the height of the crest of the mud wave. The height of the crest of the mud wave was partially determined by the tide, which could not be controlled. The strength of the soft mud was therefore reduced by mud blasting so that the mud wave would tend to flow away from the fill. During period of high tide and/or high crests on the mud wave the elevation of the top of the working table was raised. The rate of advance of the nose of the fill was maintained between ten and thirty feet per day to keep the mud moving. This method of controlling the mud displacement was successful in obtaining ninety per cent or greater mud displacement under the roadway prism.

ATTACHMENTS TO REPORT

Photographs

Plan, Profile and Cross-Sections

Failure Studies

Plots of Test Results

Settlement Plots

Excess Hydrostatic Pressures

Summary of Tests on Soil Samples

PHOTO NO. 1. Taken April 2, 1952, from top of cut at Candlestick Point. General view of the long, narrow nose used at the start of the 1st unit.

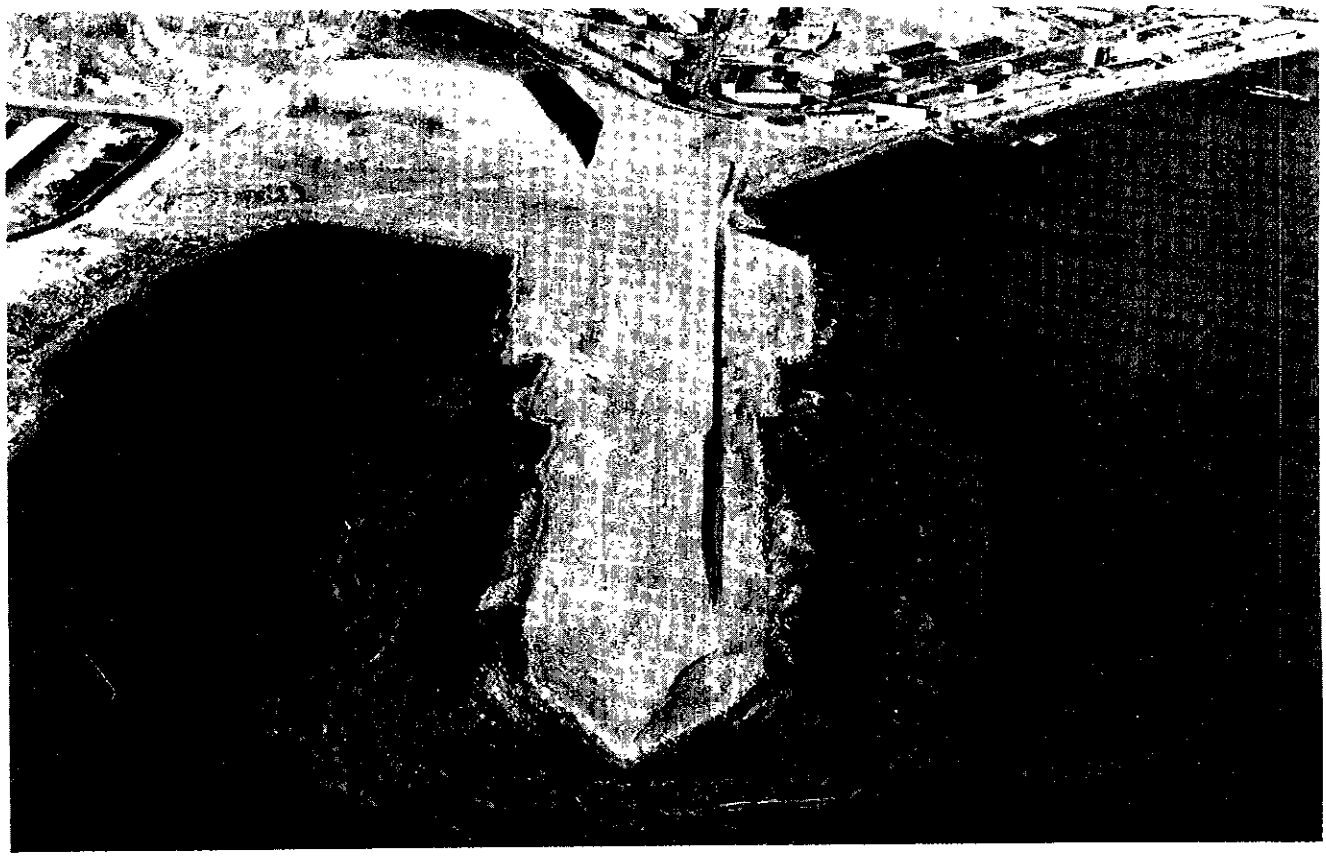


PHOTO NO. 2. Aerophotograph of the 1st unit taken October 30, 1952. Note variable widths of working table used, shape of the nose, surcharged area, and extent of mud wave.

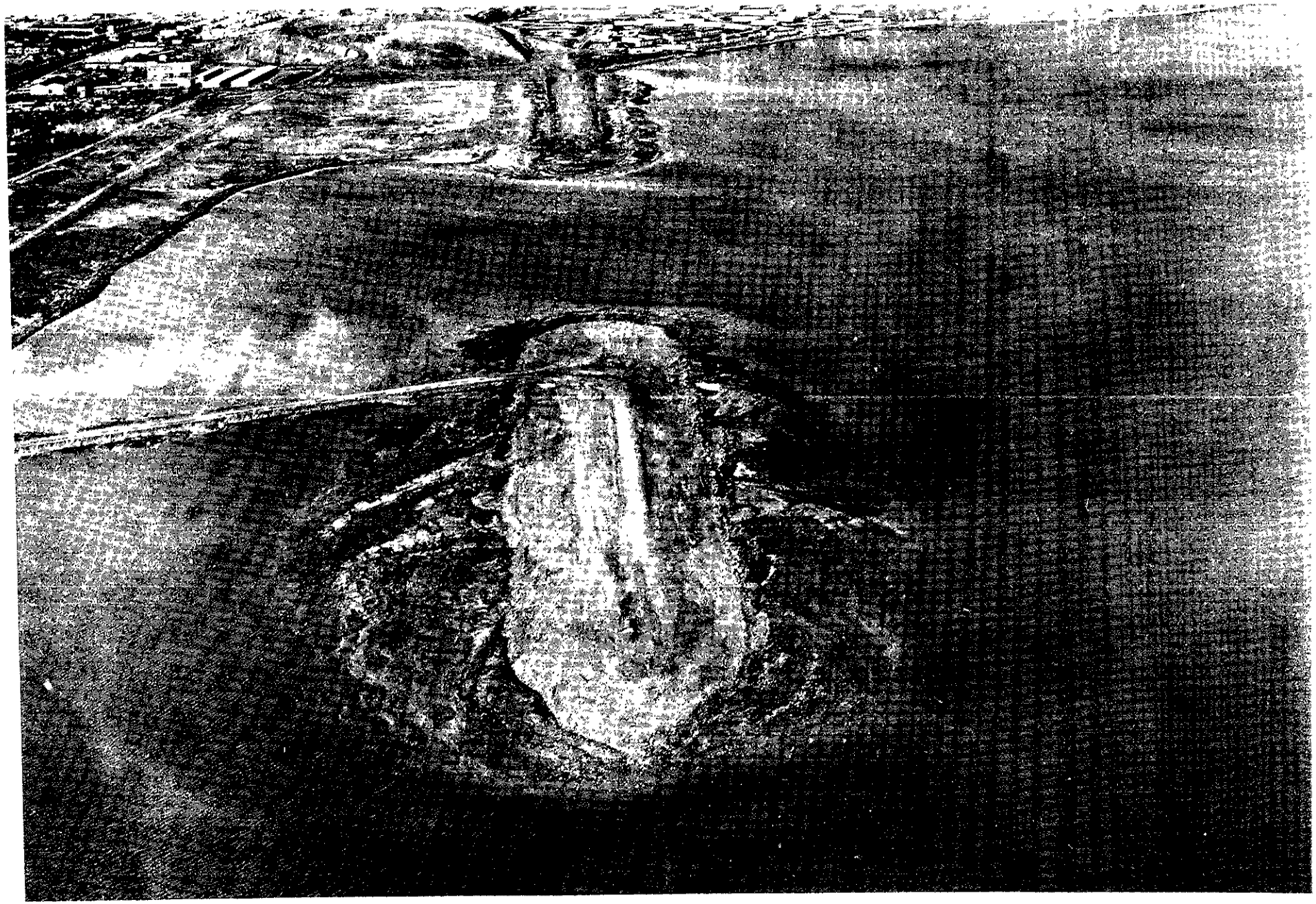


PHOTO NO. 3. Aerophotograph taken March 11, 1954, looking north showing the 1st and 2nd units completed. Note the garbage dump approaching the 1st unit, the haul road for the 2nd unit.

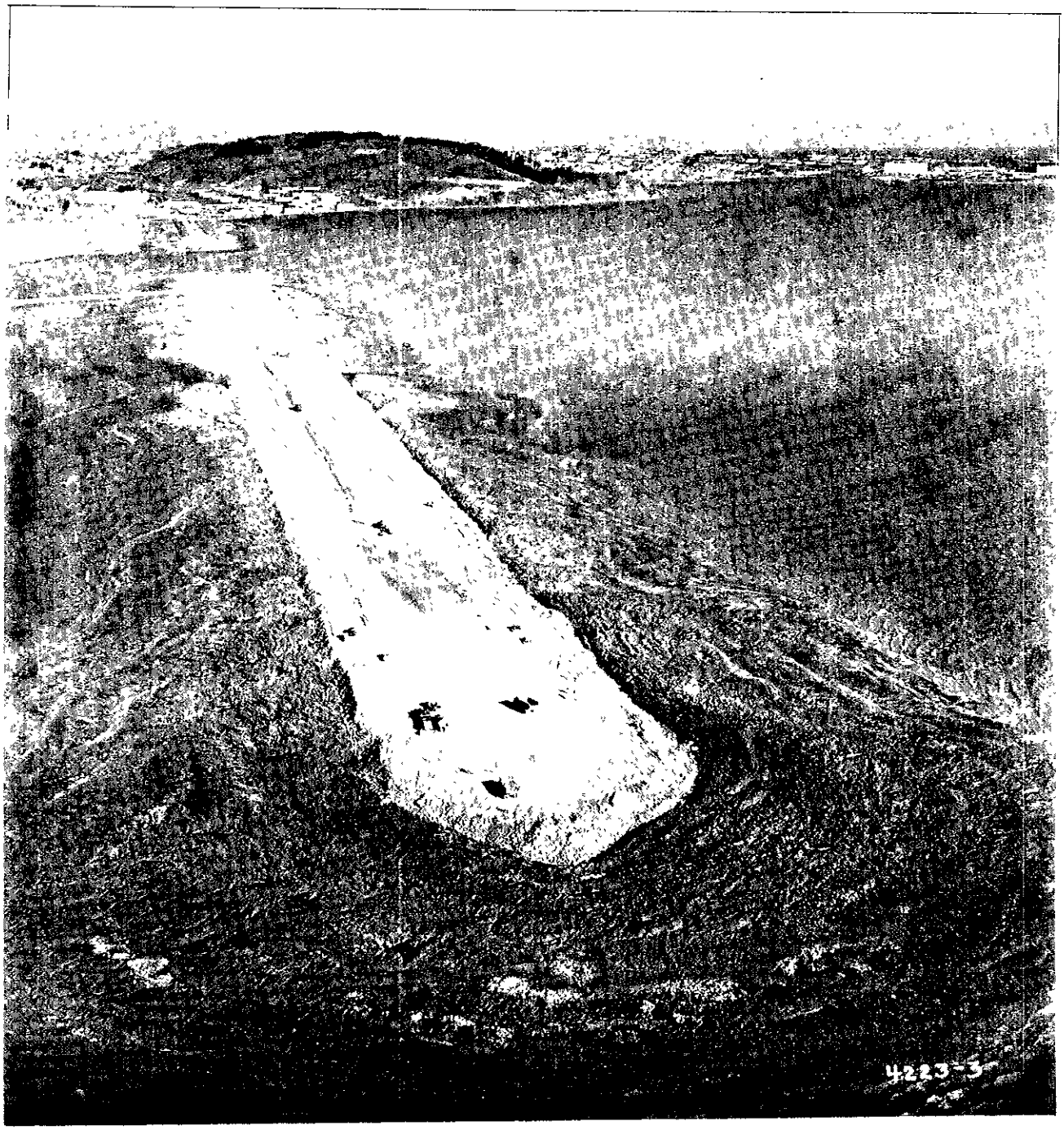


PHOTO NO. 4. Aerophotograph taken Feb. 10, 1955, looking north. Nose of the 3rd unit at Station 77+ and being advanced southward. Note the way that the mud wave is failing within itself and shape of the nose. Tide at about elevation +1.

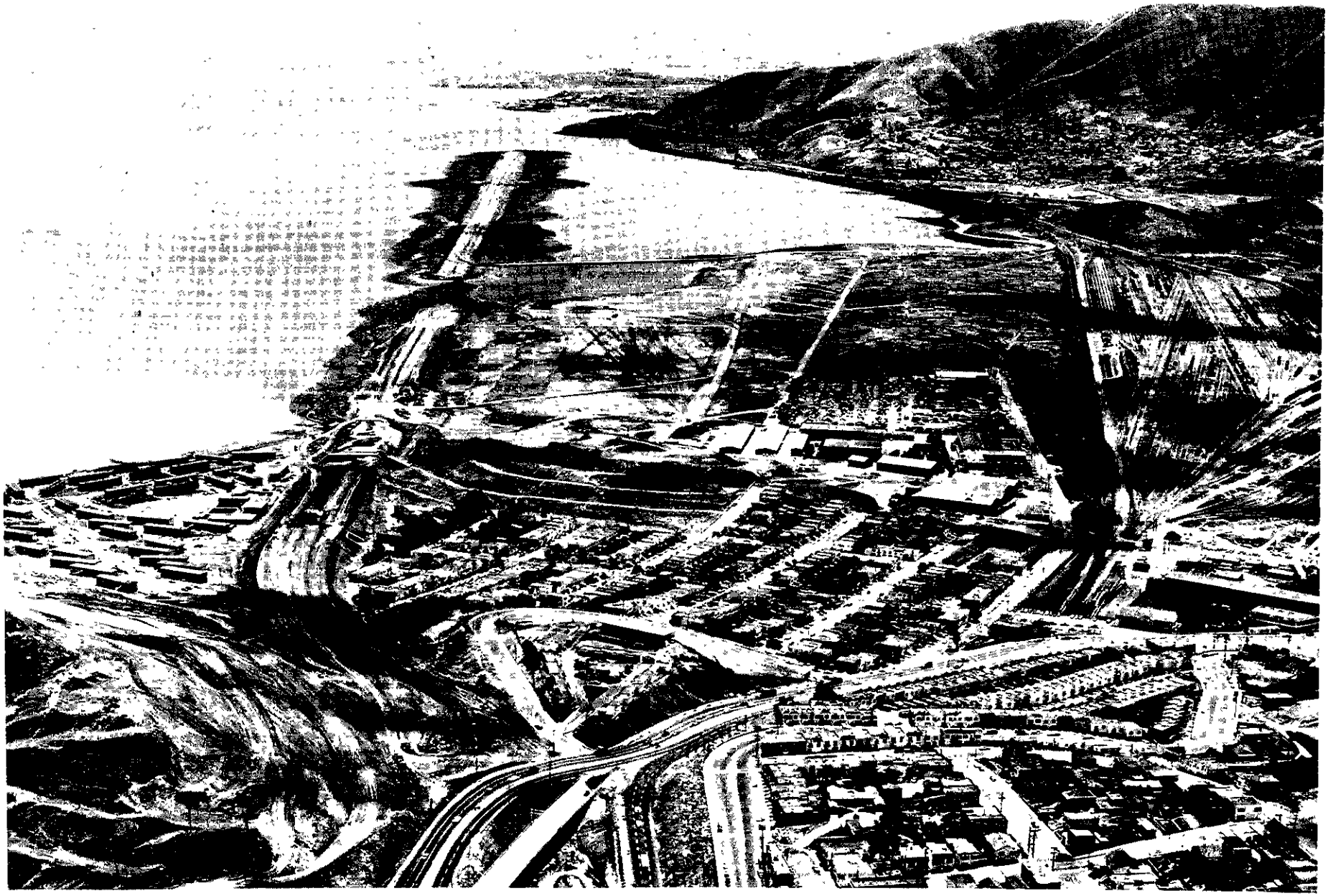


PHOTO NO. 5. Aerophotograph taken June 14, 1955, looking south. Fourth and third units under construction. Note closure had been made on fourth unit with a narrow nose on the right side. Fill being widened to the left.

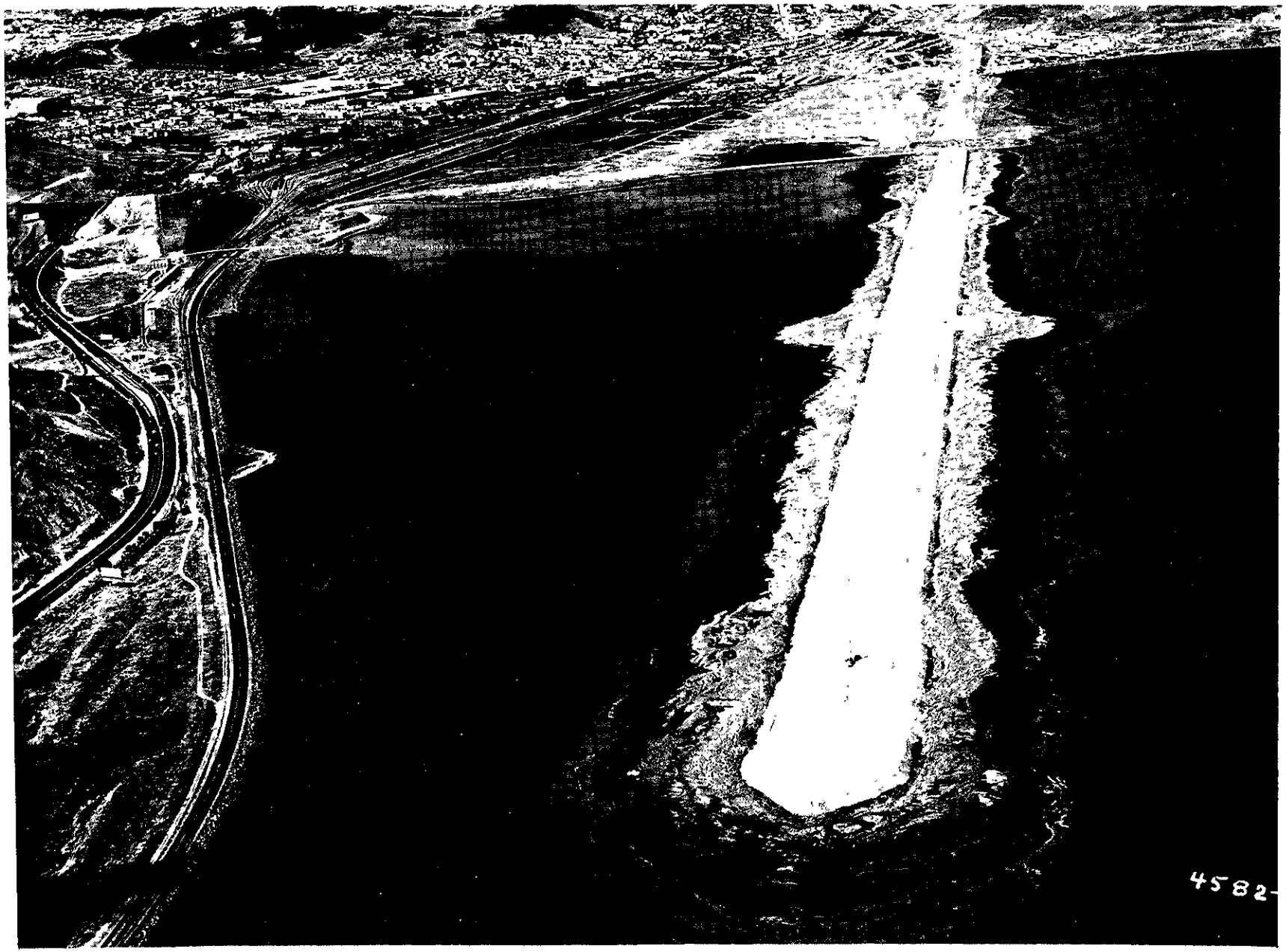


PHOTO NO. 6. Aerophotograph taken December, 1955, looking north. Third and fourth units completed. Note how the garbage dump is approaching the haul road and is against the open water fill, the widened sections for the equalizer structure, and the shape of the south nose. Tide at elevation +3.

PHOTO NO. 7. Aerophotograph taken Feb. 1, 1956, looking north. Grading operation at Sierra Point, start of fill at lower left for the railroad realignment, working on equalizer structure. Note lift on railroad fill to bring dropped area back to grader, portion of railroad fill starting to float away laterally. Tide at elevation +6.

PHOTO NO. 8. Aerophotograph taken June 19, 1956, looking north. Showing railroad fill completed, open water fill approaching closure, Sierra Point cut completed. Note large mud wave from railroad fill. Tide at elevation +5.

PHOTO No. 9. Aerophotograph taken February, 1957, looking north. Open water fill completed and grading operations in progress. Note large mud wave at south end of fill, fill placed in disposal area, suction dredge pumped at point marked X. Tide at elevation +3.

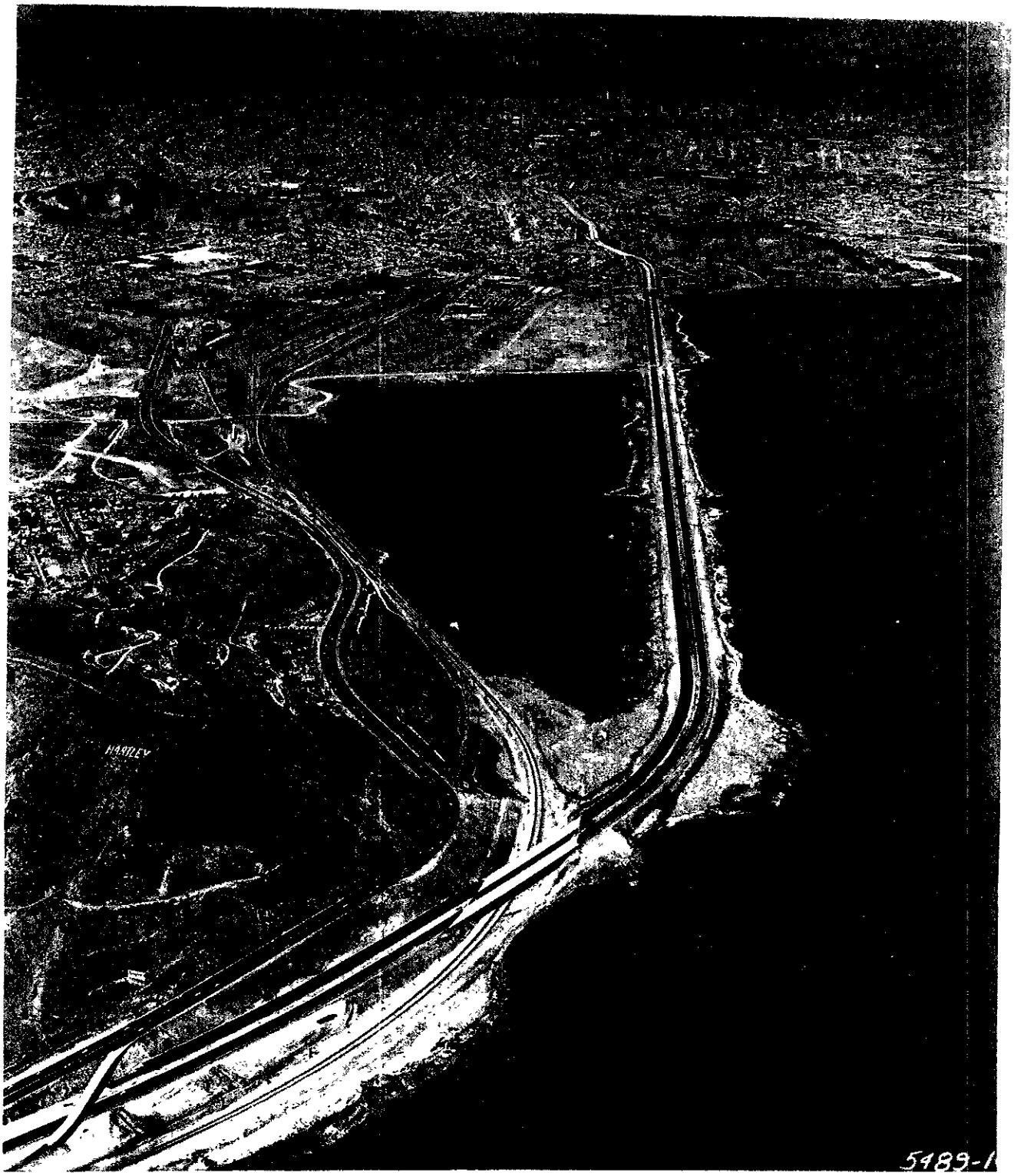


PHOTO NO. 10. Aerophotograph taken June 25, 1957, looking north. Paving of southbound lanes completed and work under way on northbound lanes. Note extent of garbage dump, mud waves. Tide at elevation +8.

PHOTO NO. 11. Photo taken Jan. 4, 1954, from the nose looking back at fill. Showing large amount (estimated at 5,000 cubic yards) of fill floating laterally away from the main fill. About Station 43± on the right side.

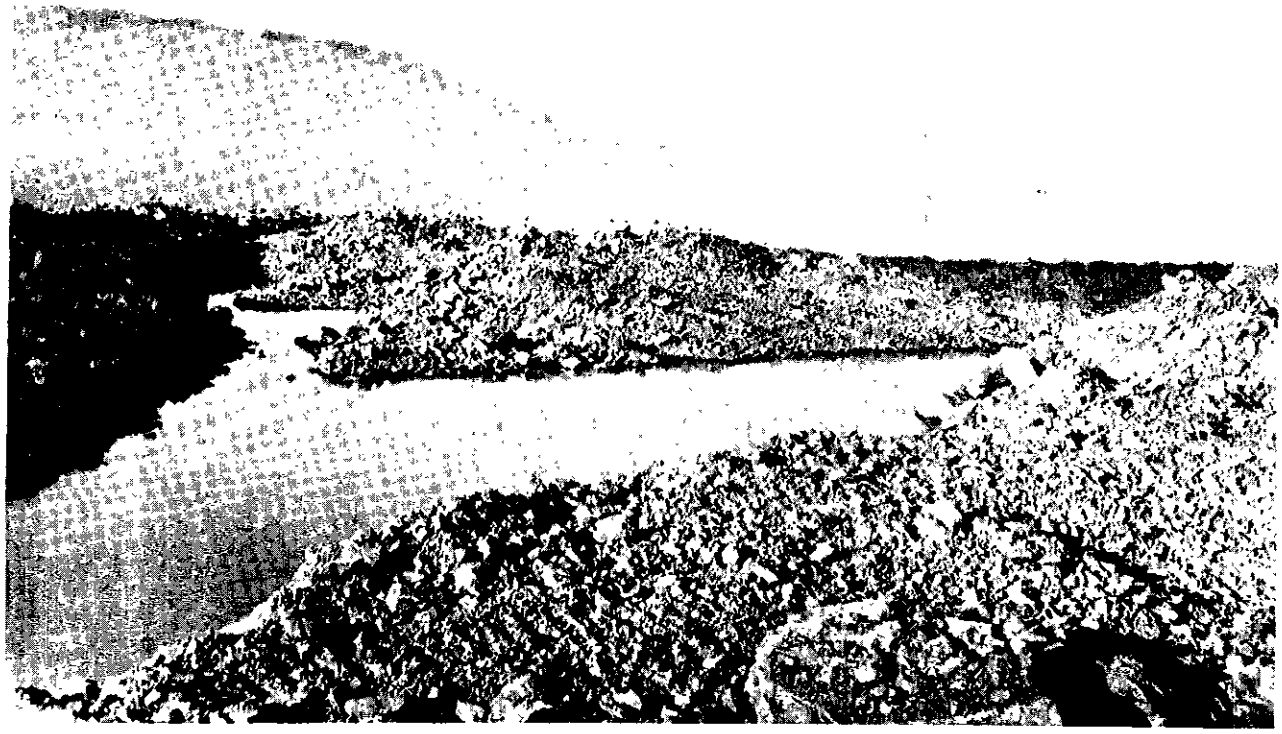


PHOTO NO. 12. Photo taken March 25, 1954. Nose breaking off at Station 55± and floating ahead of the fill.

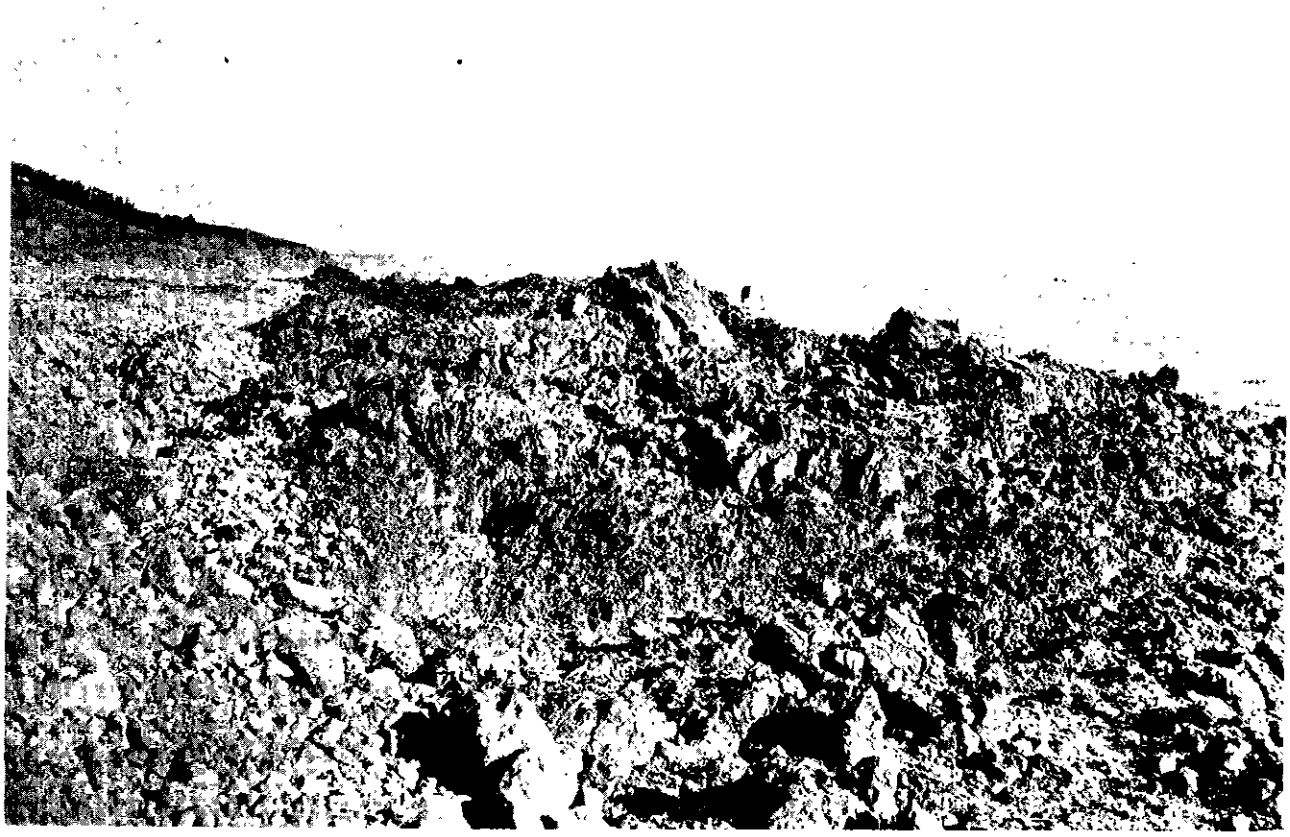


PHOTO NO. 13. Taken Dec. 30, 1953, at Station 43± on left side. Showing tendency of the fill to ride over the mud wave. Fill width is 250 feet, elevation of fill +10, elevation of mud wave is +13.5. Note the appearance of the mud wave surface. This type of mud wave is common to shear type failures.

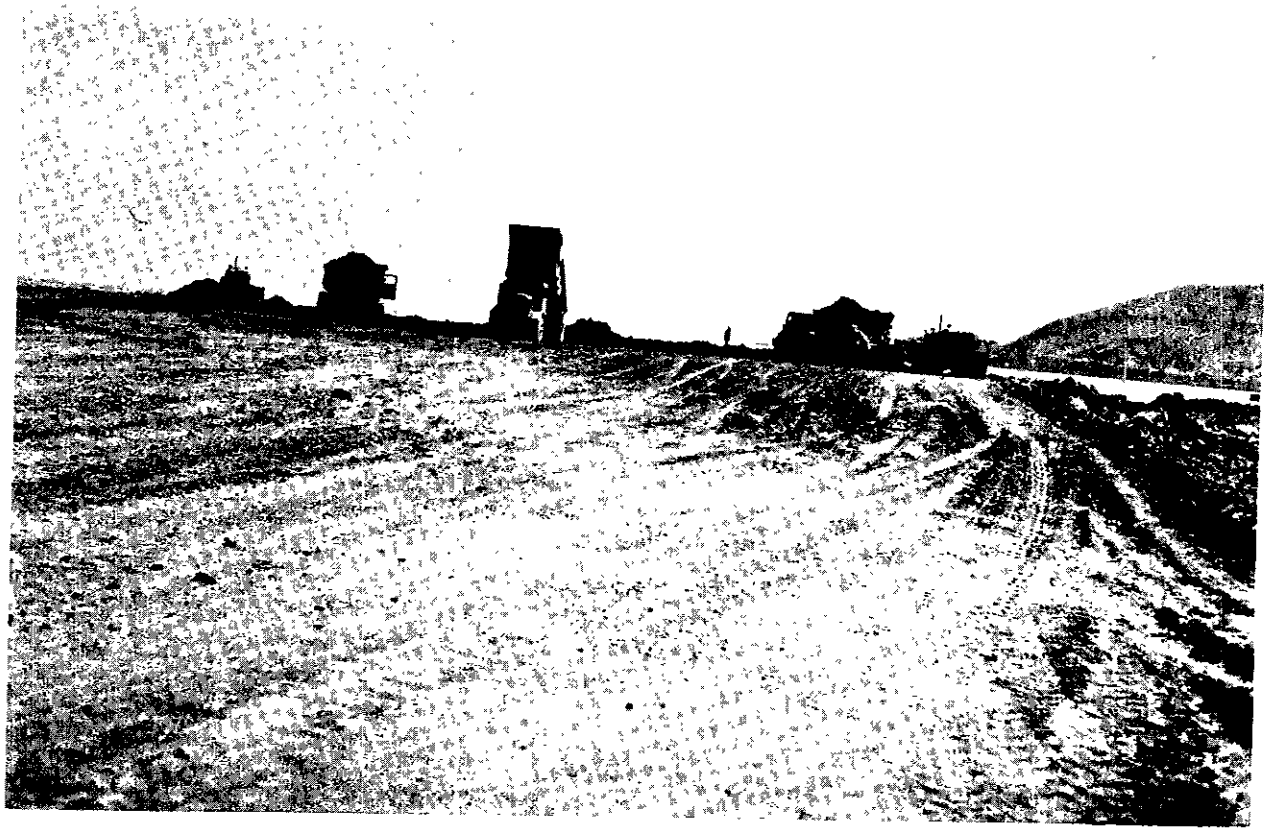


PHOTO NO. 14. Taken March 25, 1954. Showing bottom and tail dump equipment working on the nose. The cat to the left is about on centerline.

PHOTO NO. 15. Taken December 30, 1953, showing how tail dump equipment placed their loads at the edge of the fill. Note long narrow nose in advance of main fill. Tide at elevation +6.

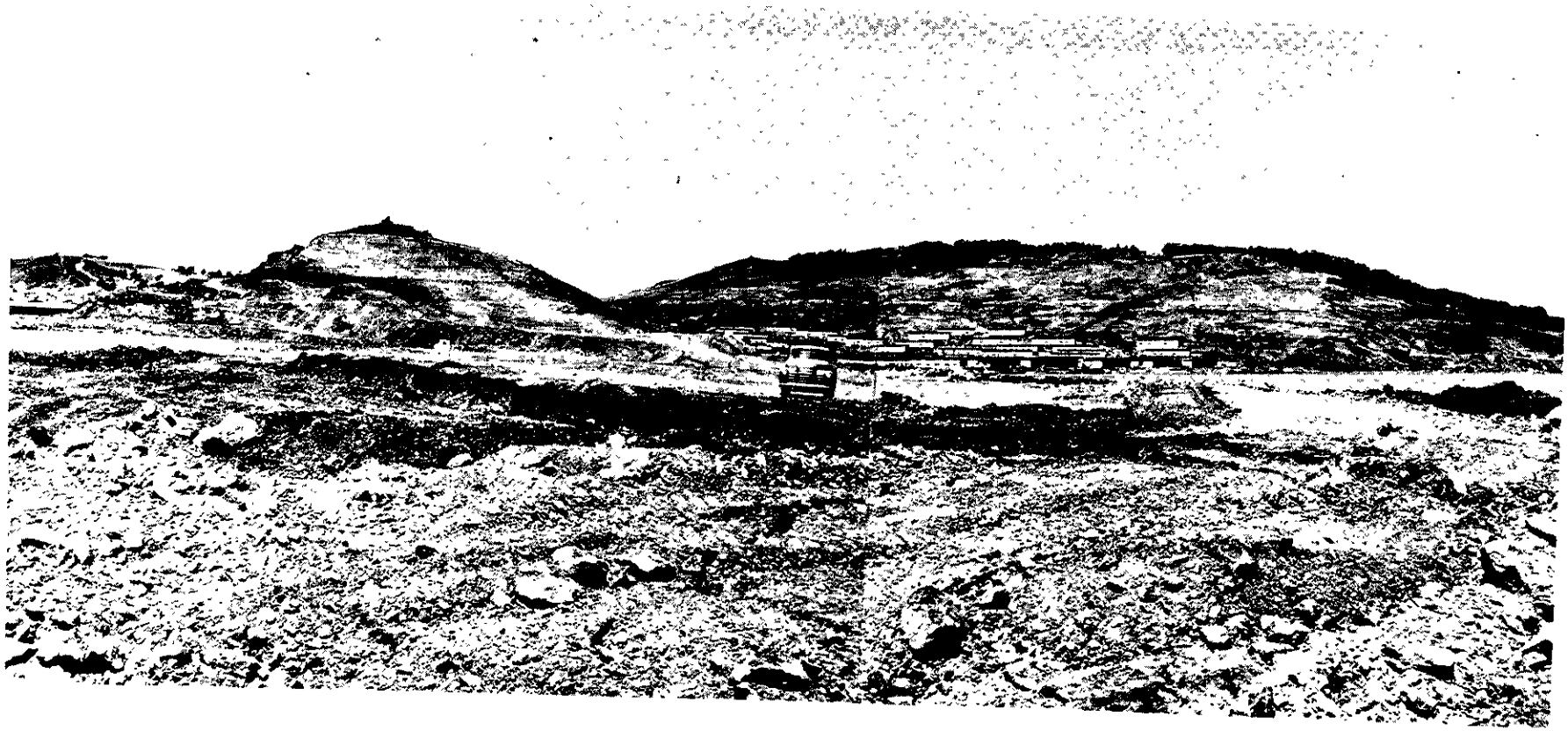


PHOTO NO. 16. Drop of about seven feet on the nose of the first unit looking north. Note how right side of fill has moved much more than the left side, and how the contractor cut the fill on the right side to get down into failed area. The pile of fill on the left side is how the bottom dump trucks left their loads. Note how the failed area slopes upward from the crack to the nose.

PHOTO NO. 17. Taken March 25, 1954; cracking at Station 54+. Drop of about 10 feet on centerline. Note rocky nature of the fill, how the bottom dump trucks had placed their load.

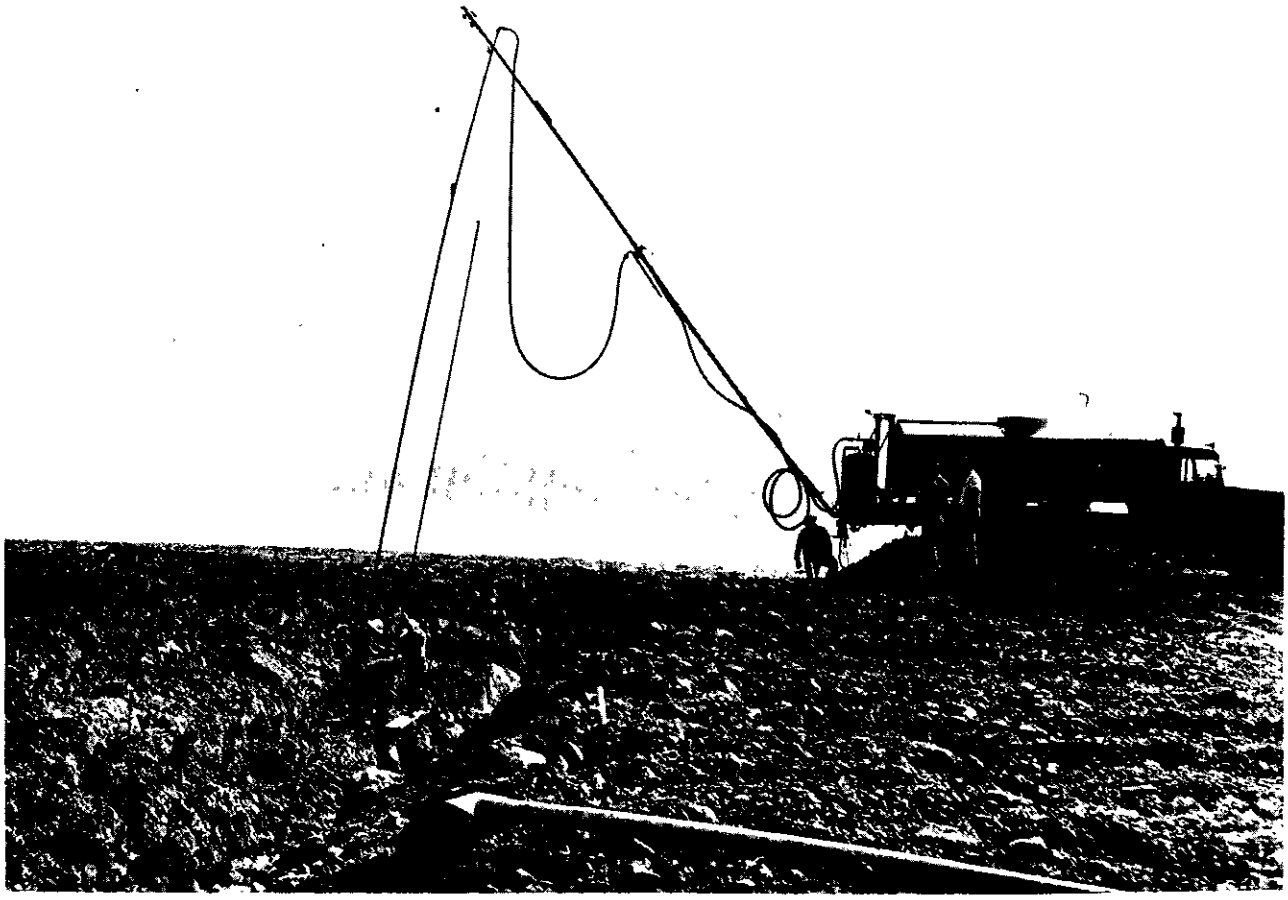


PHOTO NO. 18. Placing charges in the mud wave. Pipe with the hose attached is to jet a hole in the mud. The pole the men are handling is to push the charge into the hole. A charge made up ready for placing is on the foreground.

PHOTO NO. 19. Taken Feb. 24, 1954, from the haul road looking north. Note mud wave build up at Station 39±. The mud was pushed up during fill movement and would then break off and flow away from fill. The near vertical face is six feet high and the crest is almost flat for 30 feet. This is a typical shear type failure.

PHOTO NO. 20. Taken April 29, 1952. Mud wave to the left at Station 8±. Note the appearance of the mud wave in a typical squeeze type failure. Note the rock being carried over the mud wave by the movement or flow of the mud.

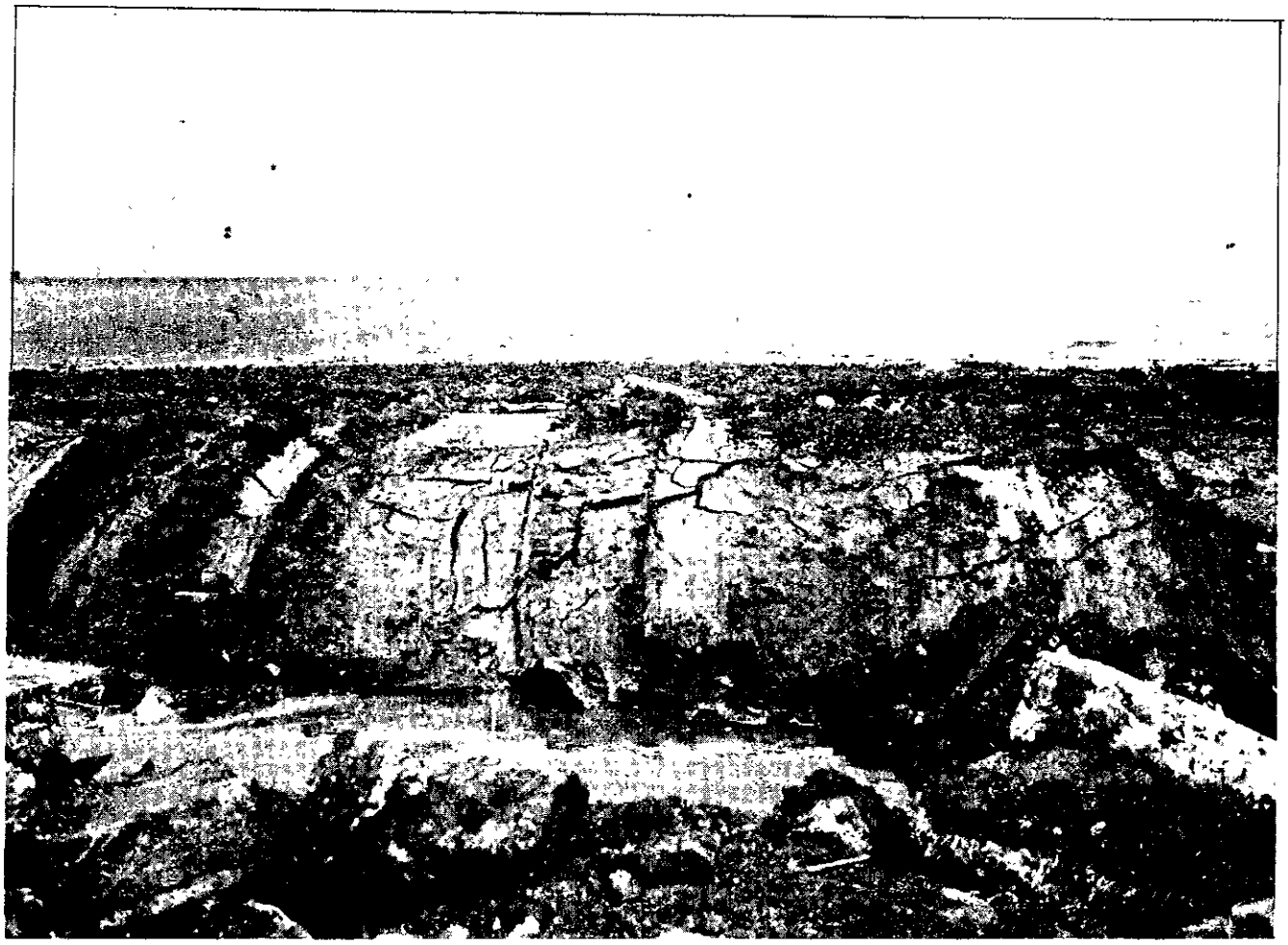


PHOTO NO. 21. Taken April 29, 1952. Mud wave in a typical squeeze type failure. Note the smooth appearance of the mud wave. Photo No. 20 was taken in a.m. when the block on concrete was at the crest of the mud wave. Photo No. 21 was taken in p.m. when concrete block had moved to backside of mud wave.

PHOTO NO. 22. Taken April 3, 1954, showing the typical cracking parallel to centerline during the construction of the roadway prism.

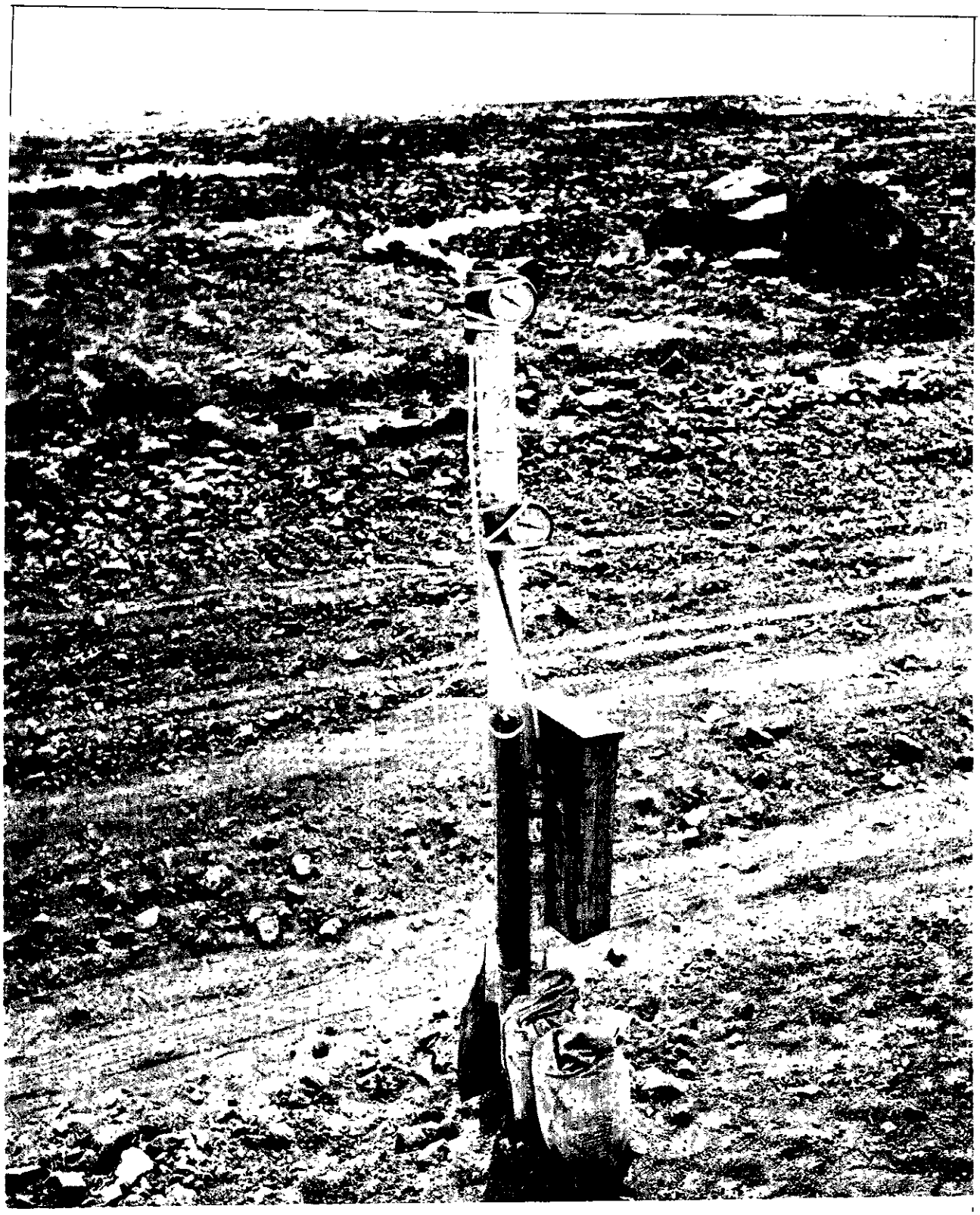


PHOTO NO. 23. Piezometer set up to run field permeability test. Gages were later placed in box attached to the pipes.

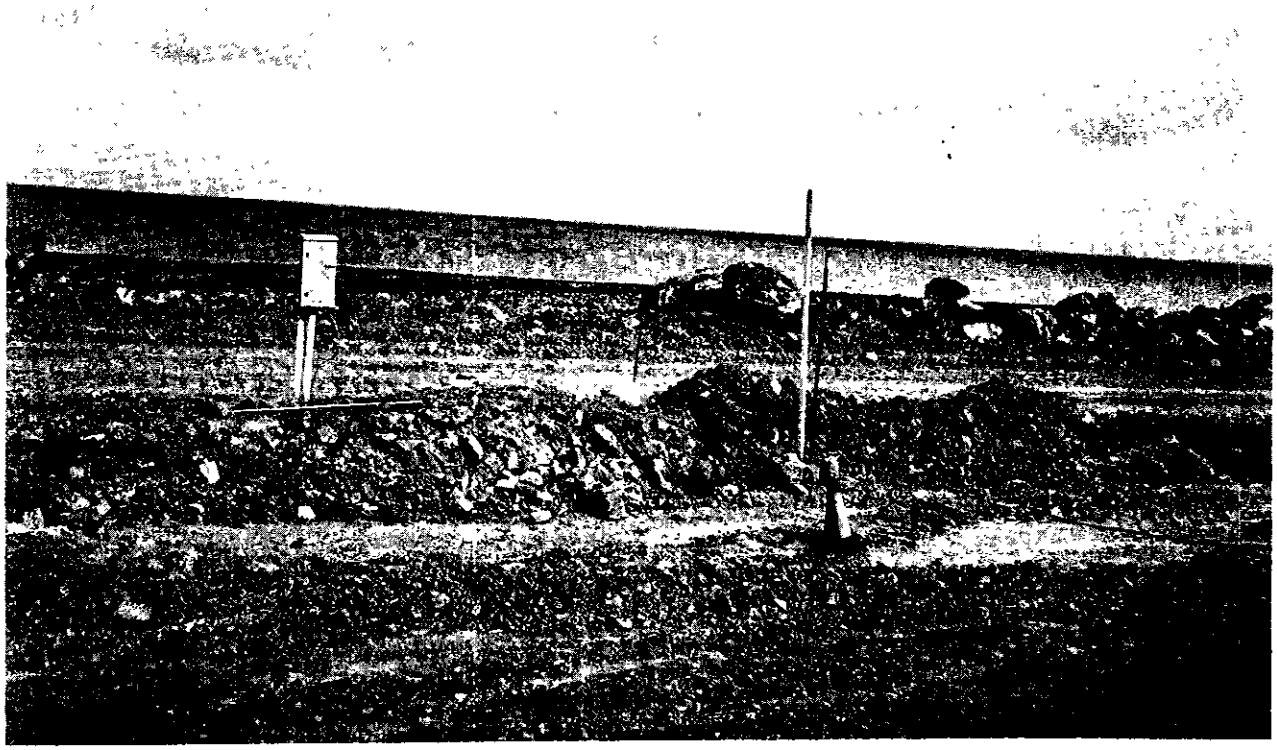


PHOTO NO. 24. Piezometer installation completed. The gages are in the box attached to the pipes. A riser type settlement platform is on the right side of the photo.

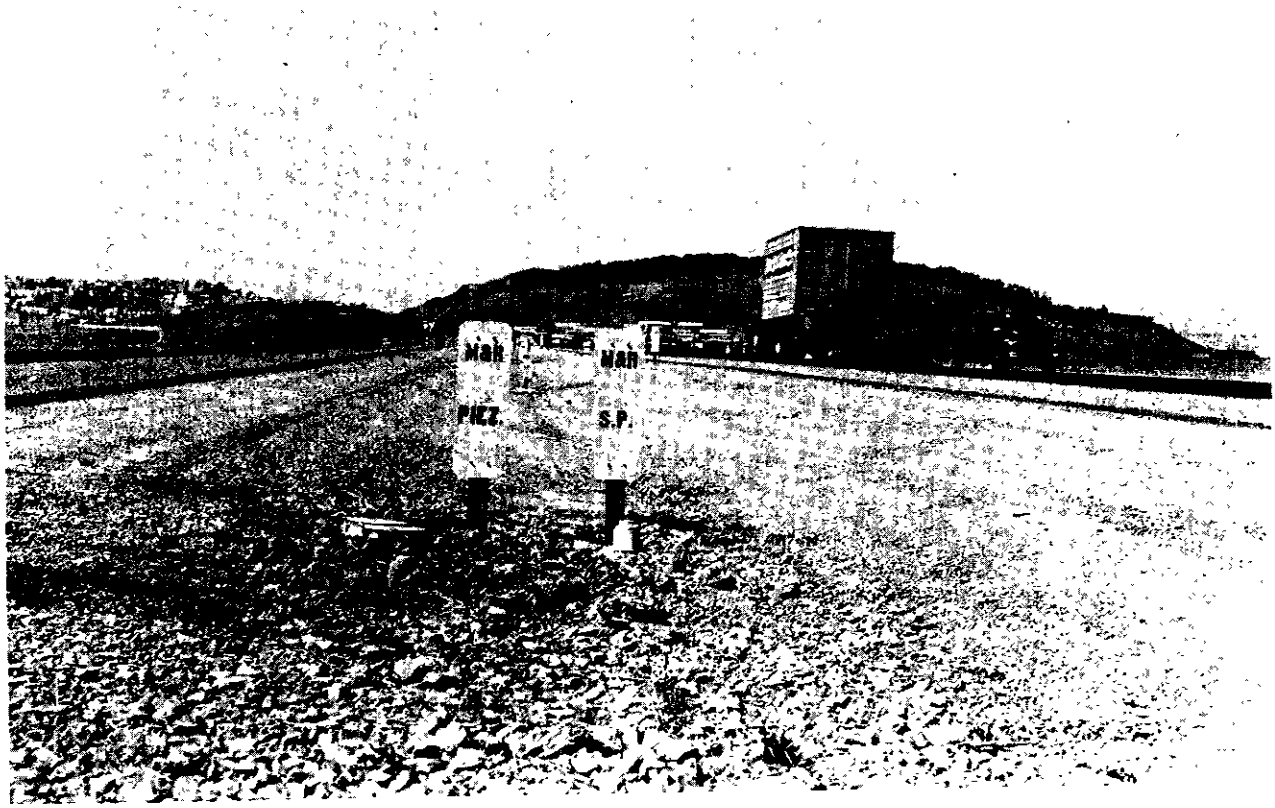


PHOTO NO. 25. A completed settlement and piezometer installation on centerline at completion of the work.

PLAN, PROFILE, AND CROSS-SECTIONS

Unit	Stations	No. of Sheets
First	6+00 to 18+00	2
Second	37+00 to 57+00	3
Third	57+00 to 106+00	7
Fourth	15+00 to 40+00	4
Fifth	107+00 to 120+00	2

FAILURE STUDIES

Subject	Plates
Movement of Hubs	F-1, F-4, F-7, F-10, F-13
Shear Circle for Actual Failure	F-2, F-5, F-8, F-11, F-14
Critical Shear Circles	F-3, F-6, F-9, F-12, F-15
Mud Wave Profiles	MW-1, MW-2

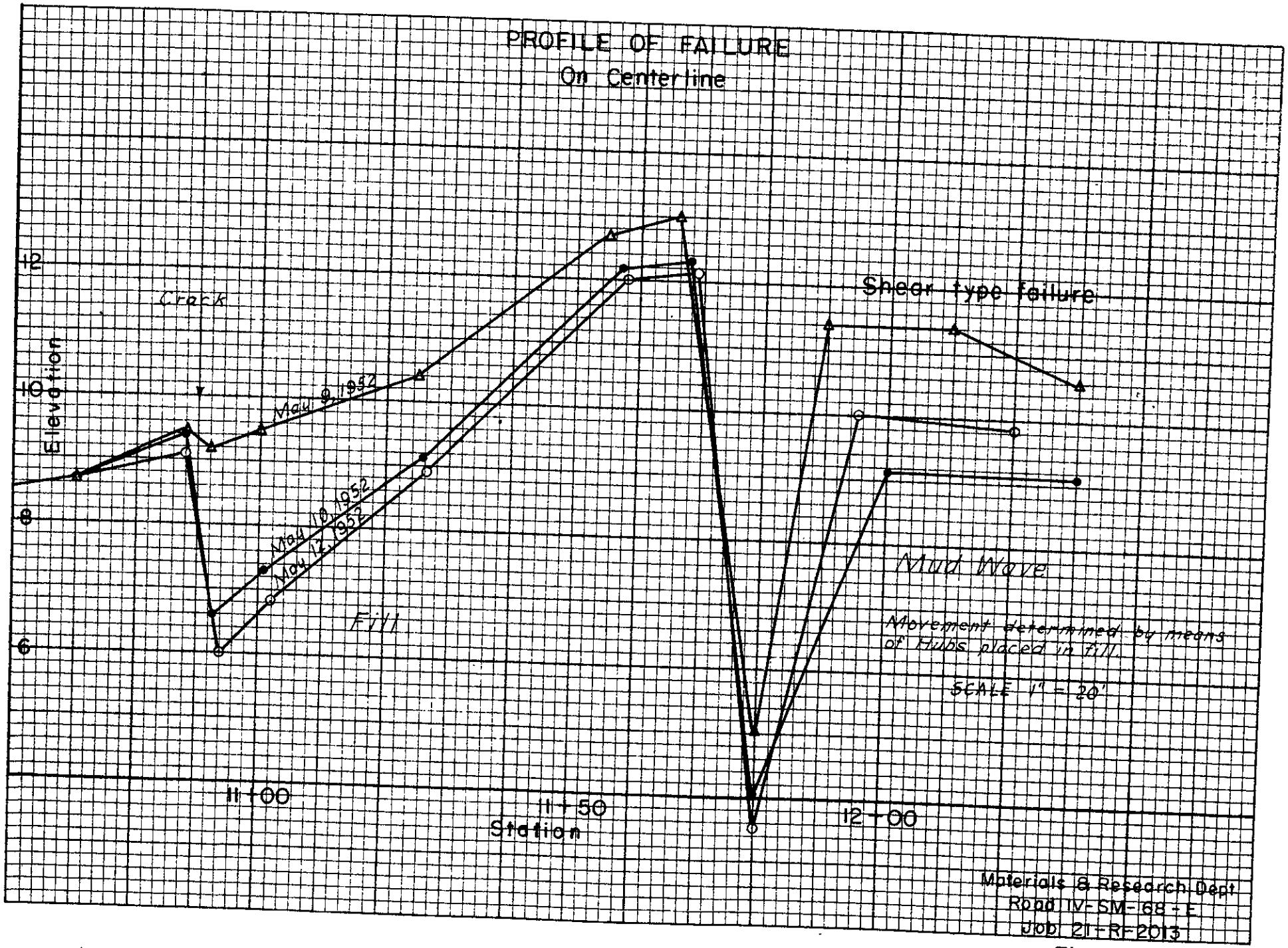

PROFILE OF FAILURE
On Centerline
Crack
May 9, 1952
May 10, 1952
May 12, 1952
Fill
Shear type failure
Mud Wave
Movement determined by means of Hubs placed in fill
SCALE 1" = 20'
Elevation
12
10
8
6
11+00
11+50
12+00
Station
Materials & Research Dept
Road IV-SM-68-E
Job 21-RF2013

Fig F1

NO. 340J -10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH
EUGENE DIETZGEN CO.
MADE IN U. S. A.

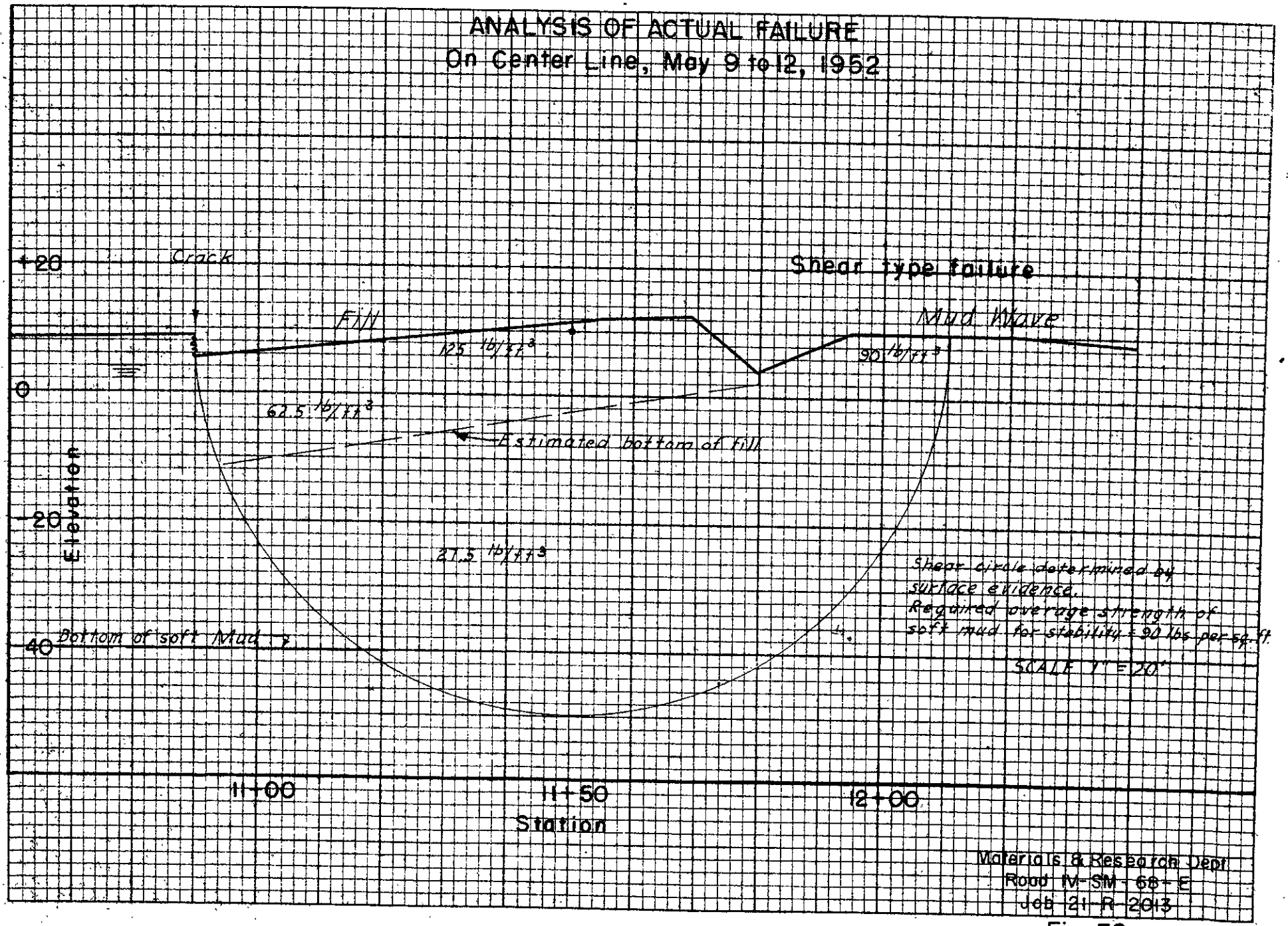
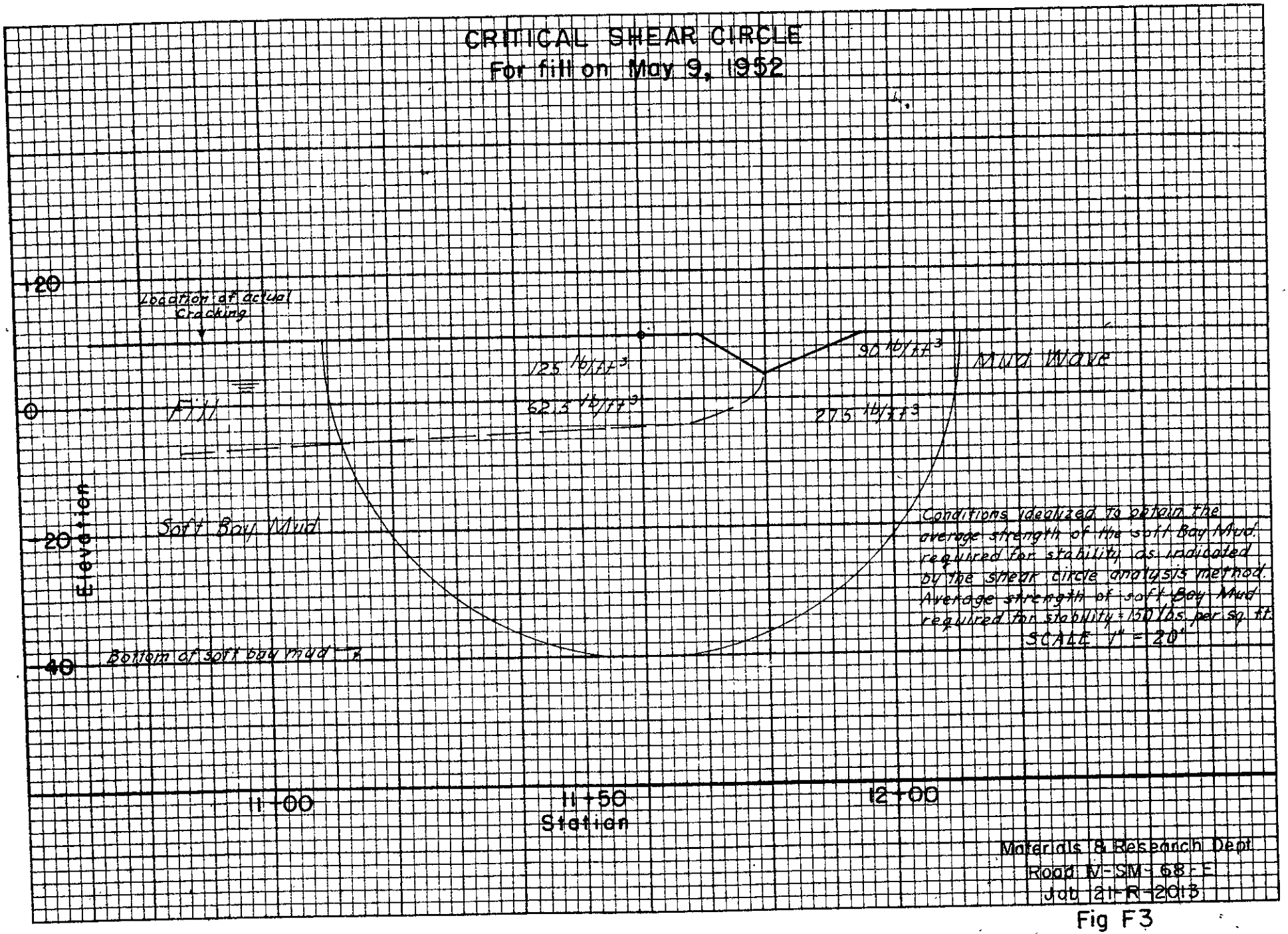


Fig F2

NO. 340\ -10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH
EUGENE DIETZGEN CO
MADE IN U. S. A.

CRITICAL SHEAR CIRCLE
For fill on May 9, 1952



Materials & Research Dept.
Road IV-SM-68-E
Job 21-R-2013

Fig F3

NO. 340 10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH
EUGENE DIETZGEN CO.
MADE IN U. S. A.

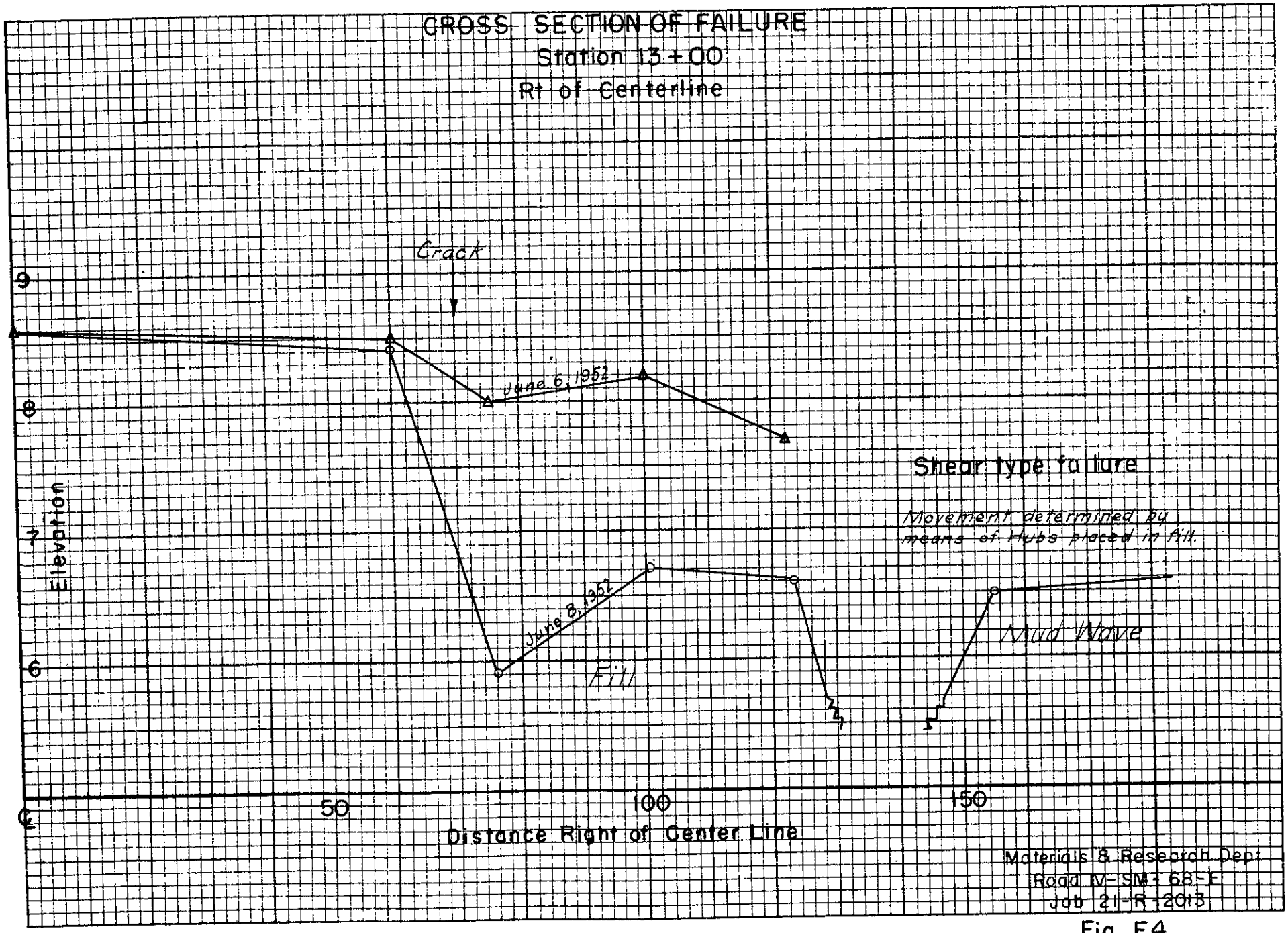


Fig F4

NO. 340\ -10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH
EUGENE DIETZGEN CO.
MADE IN U. S. A.

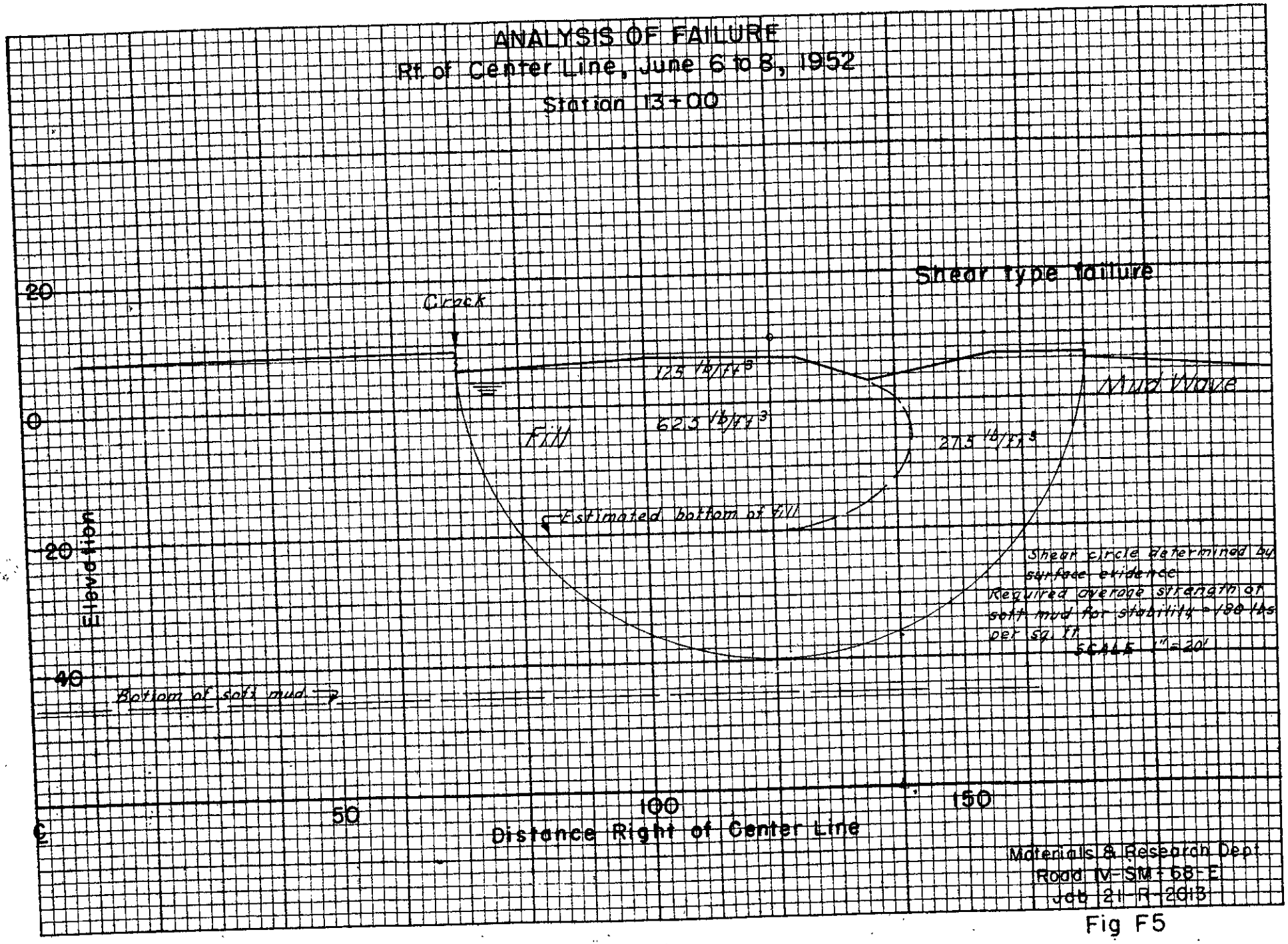
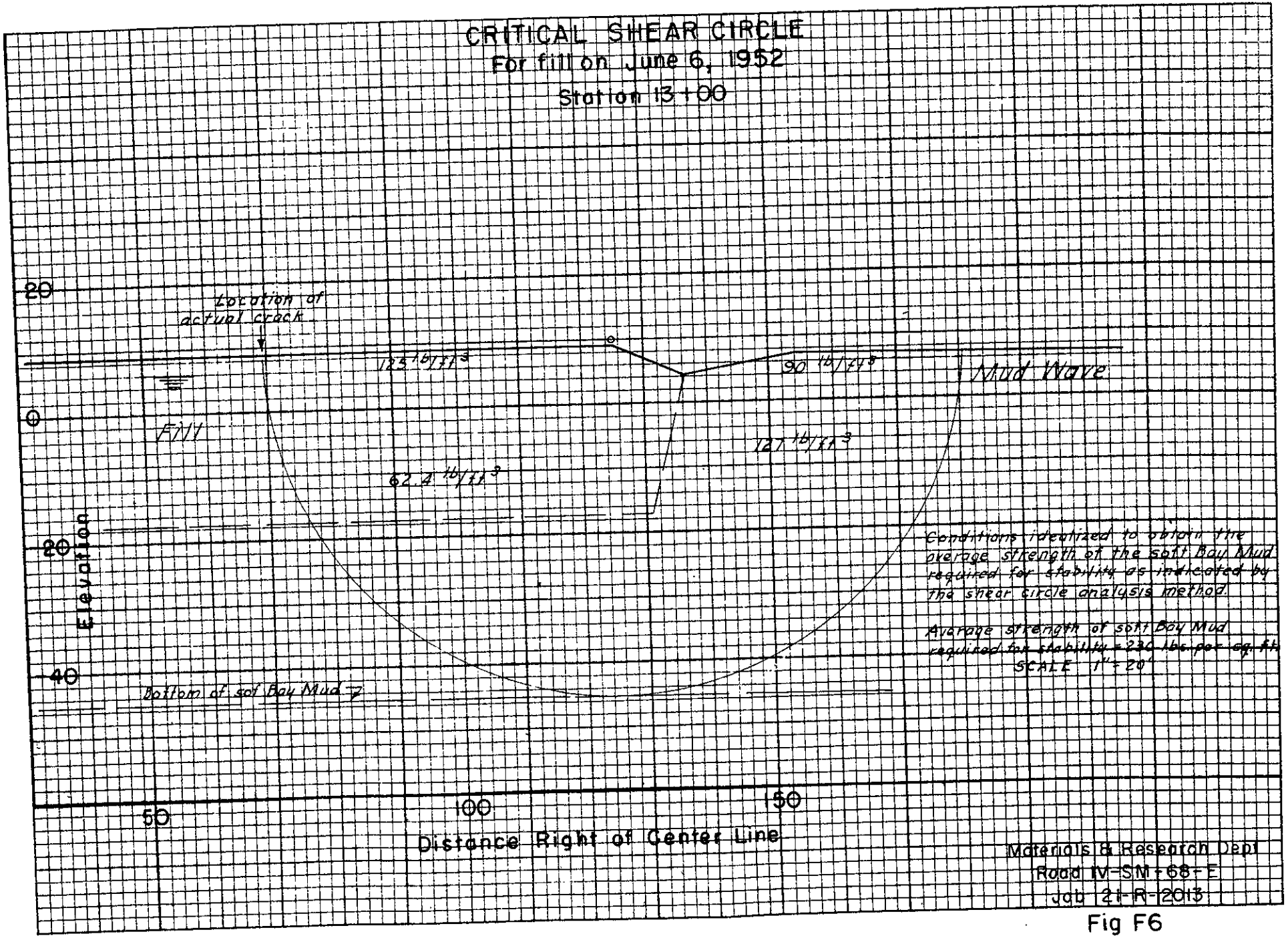


Fig F5

NO. 340-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH
EUGENE DIETZGEN CO.
MADE IN U. S. A.

CRITICAL SHEAR CIRCLE
For fill on June 6, 1952
Station 13+00



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Road IV-SM-68-E
Job 21-R-2013

Fig F6

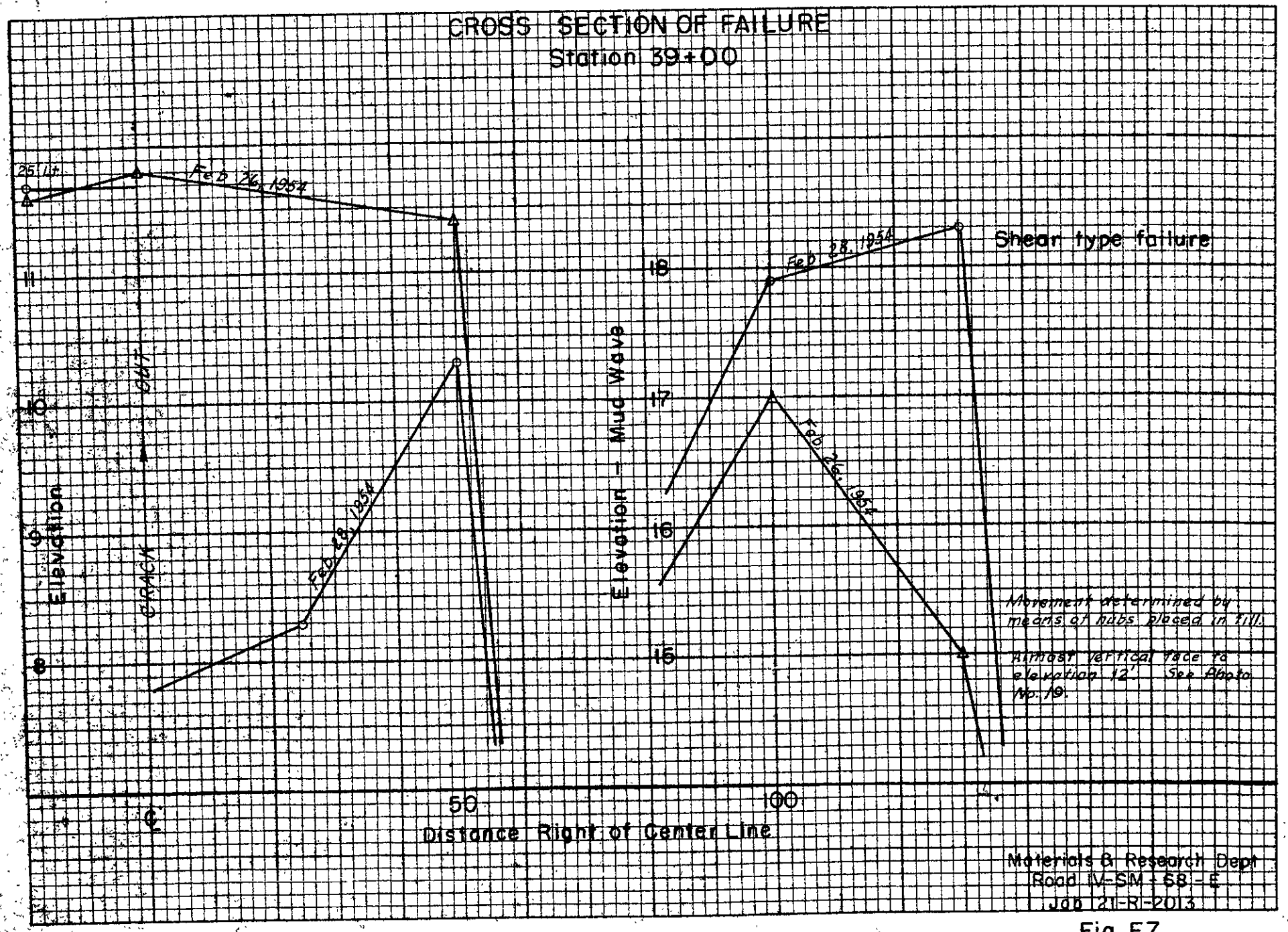

CROSS SECTION OF FAILURE
Station 39+00
25.14
Feb 26, 1954
Feb 28, 1954
CRACK
SLIP
Elevation
11
10
9
8
Feb 28, 1954
Feb 26, 1954
Elevation - Mud Wave
18
17
16
15
Shear type failure
Movement determined by means of hubs placed in fill.
Almost vertical face to elevation 12'. See Photo No. 19.
50
100
Distance Right of Center Line
Materials & Research Dept
Road IV-SM-68-E
Job 21-R-2013

Fig F7

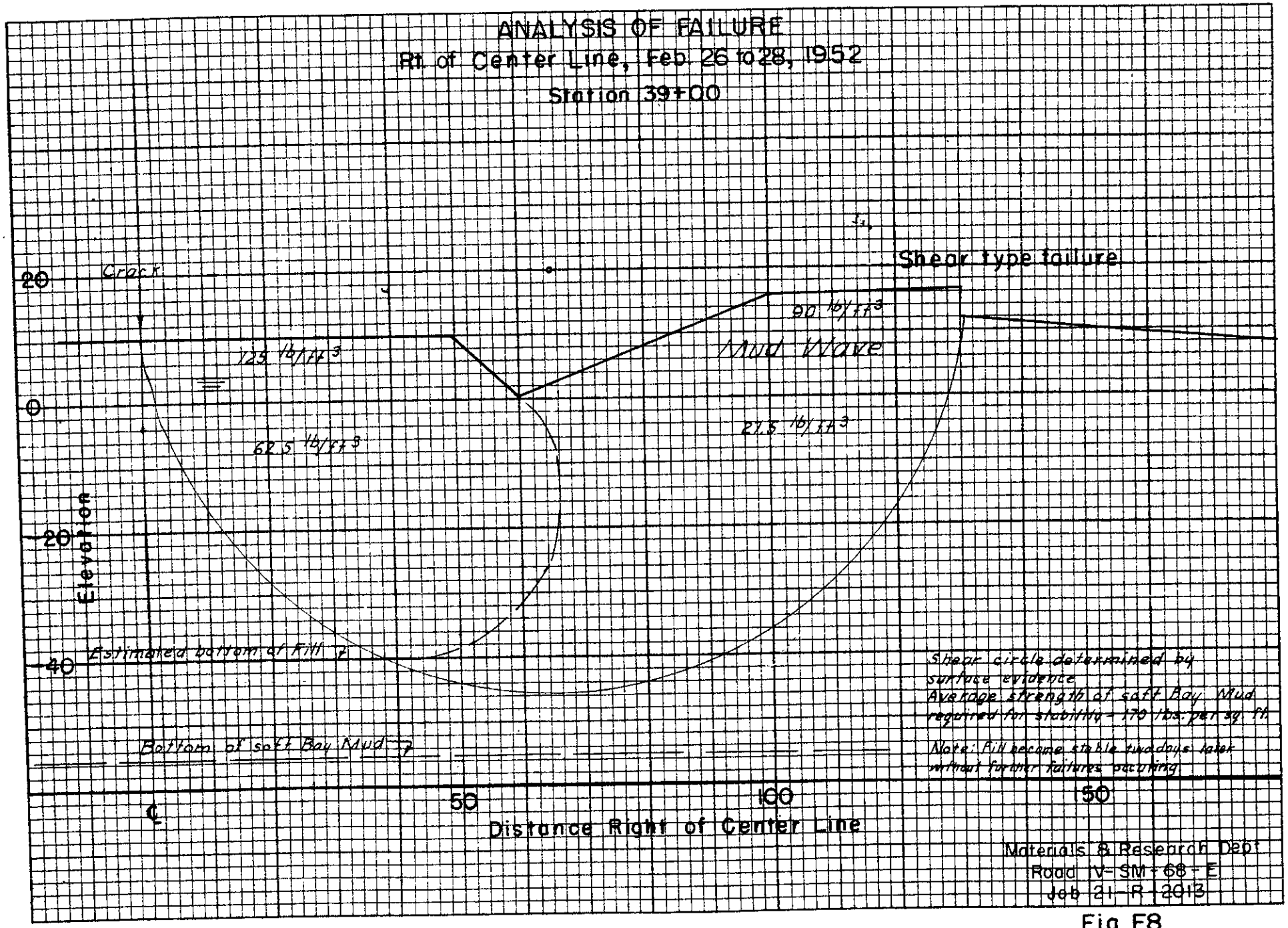

ANALYSIS OF FAILURE
Rt. of Center Line, Feb. 26 to 28, 1952
Station 39+00
Shear type failure
Crack
20
0
-20
-40
Elevation
125 lb/ft³
62.5 lb/ft³
90 lb/ft³
Mud Wave
27.5 lb/ft³
Estimated bottom of Fill
Bottom of soft Bay Mud
Shear circle determined by surface evidence
Average strength of soft Bay Mud required for stability = 170 lbs. per sq. ft.
Note: Fill became stable two days later without further failures occurring
50
100
150
℄
Distance Right of Center Line
Materials & Research Dept.
Road IV-SM-68-E
Job 21-R-2013

Fig F8

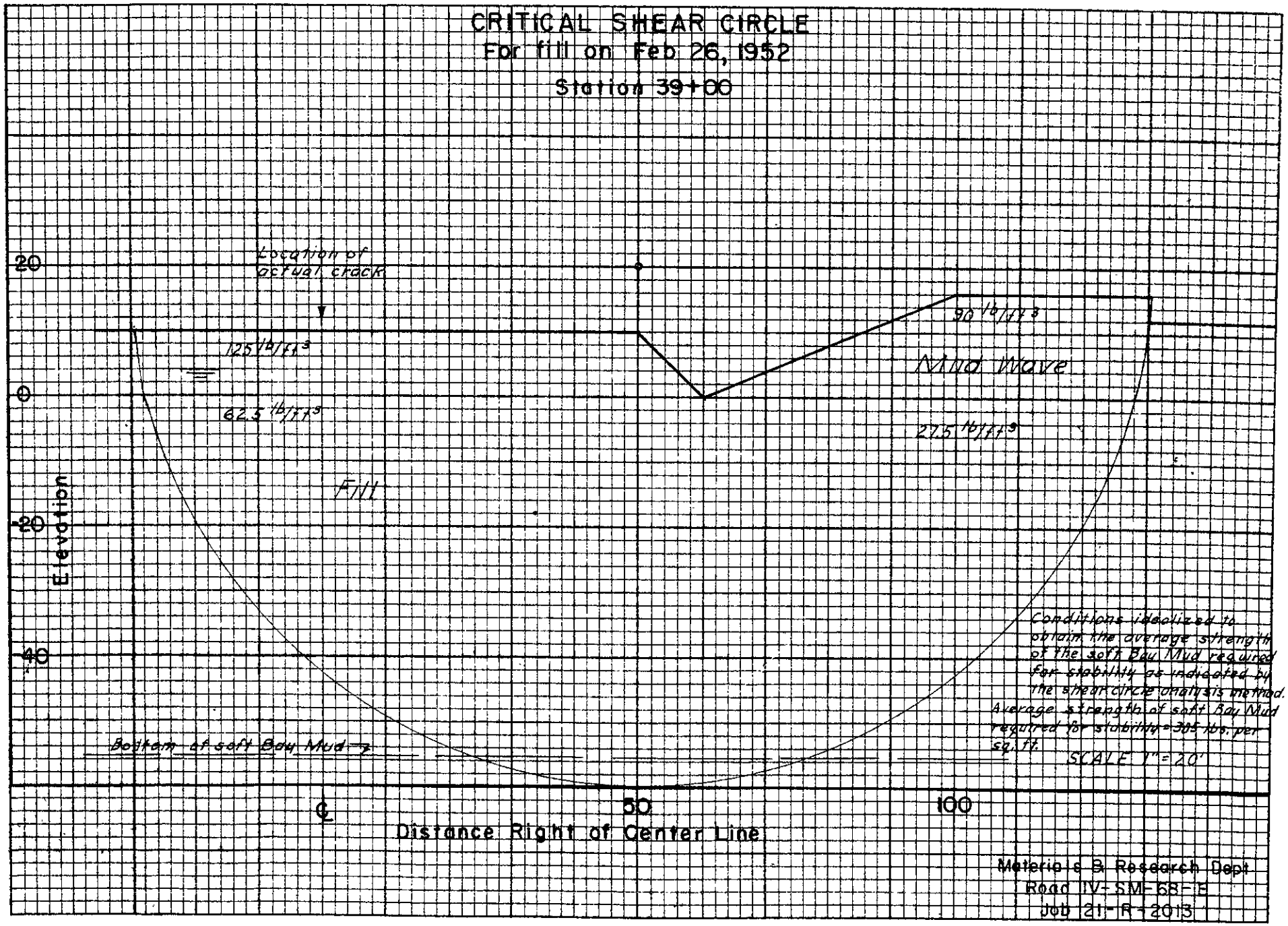

CRITICAL SHEAR CIRCLE
For fill on Feb 26, 1952
Station 39+00
Location of actual crack
125 lb/ft³
62.5 lb/ft³
Fill
90 lb/ft³
Mud Wave
27.5 lb/ft³
Bottom of soft Bay Mud
Conditions idealized to obtain the average strength of the soft Bay Mud required for stability as indicated by the shear circle analysis method.
Average strength of soft Bay Mud required for stability = 385 lbs. per sq. ft.
SCALE 1" = 20'
20
0
-20
-40
Elevation
℄
50
100
Distance Right of Center Line
Materials & Research Dept
Road IV-SM-68-E
Job 21-R-2013

Fig F9

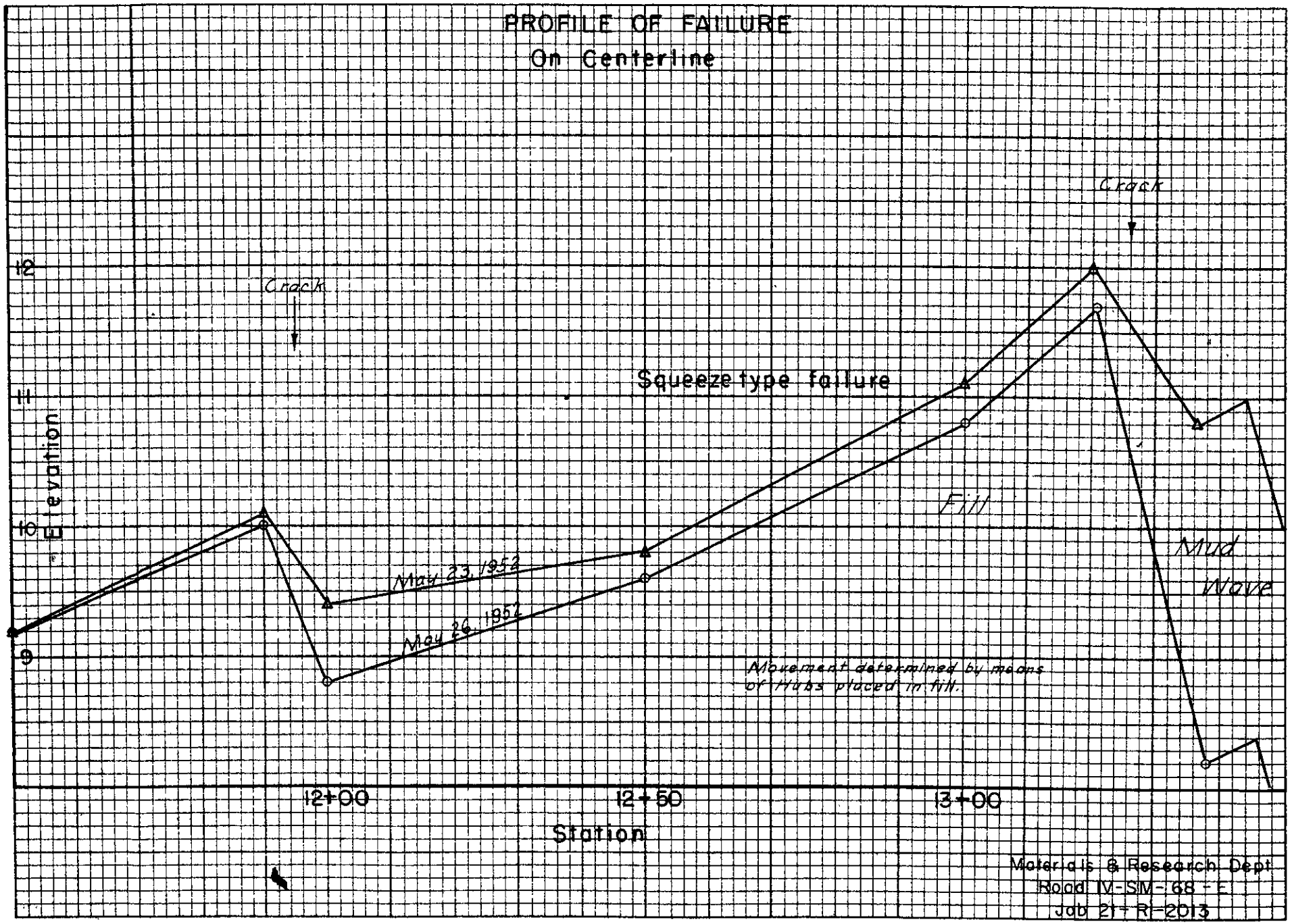

PROFILE OF FAILURE
On Centerline
Crack
Crack
Squeeze type failure
Fill
Mud Wave
May 23, 1952
May 26, 1952
Movement determined by means of Hubs placed in fill.
Elevation
12
11
10
9
12+00
12+50
13+00
Station
Materials & Research Dept.
Road IV-SM-68-E
Job 21-R-2013

Fig F10

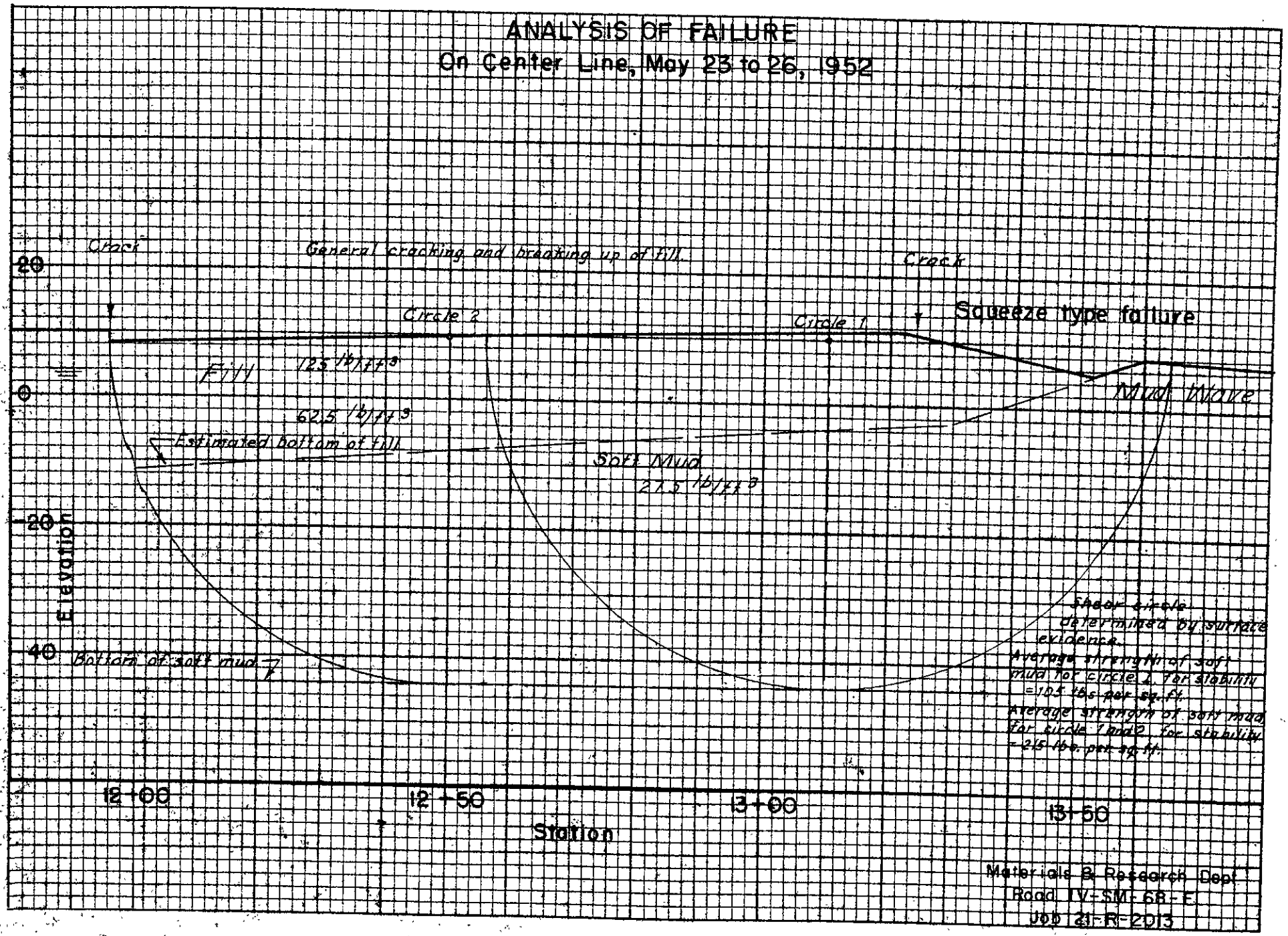

ANALYSIS OF FAILURE
On Center Line, May 23 to 26, 1952
Crack
General cracking and breaking up of fill
Crack
Circle 2
Circle 1
Squeeze type failure
Fill
125 lb/ft³
62.5 lb/ft³
Estimated bottom of fill
Soft Mud
21.5 lb/ft³
Mud Wave
Bottom of soft mud
Shear circle determined by surface evidence.
Average strength of soft mud for circle 1, for stability = 105 lbs per sq. ft.
Average strength of soft mud for circle 1 and 2, for stability = 215 lbs. per sq. ft.
20
0
-20
-40
Elevation
12+00
12+50
13+00
13+50
Station
Materials & Research Dept.
Road IV-SM-68-E
Job 21-R-2013

Fig F11

NO. 340\ -10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH
EUGENE DIETZGEN CO
MADE IN U. S. A.

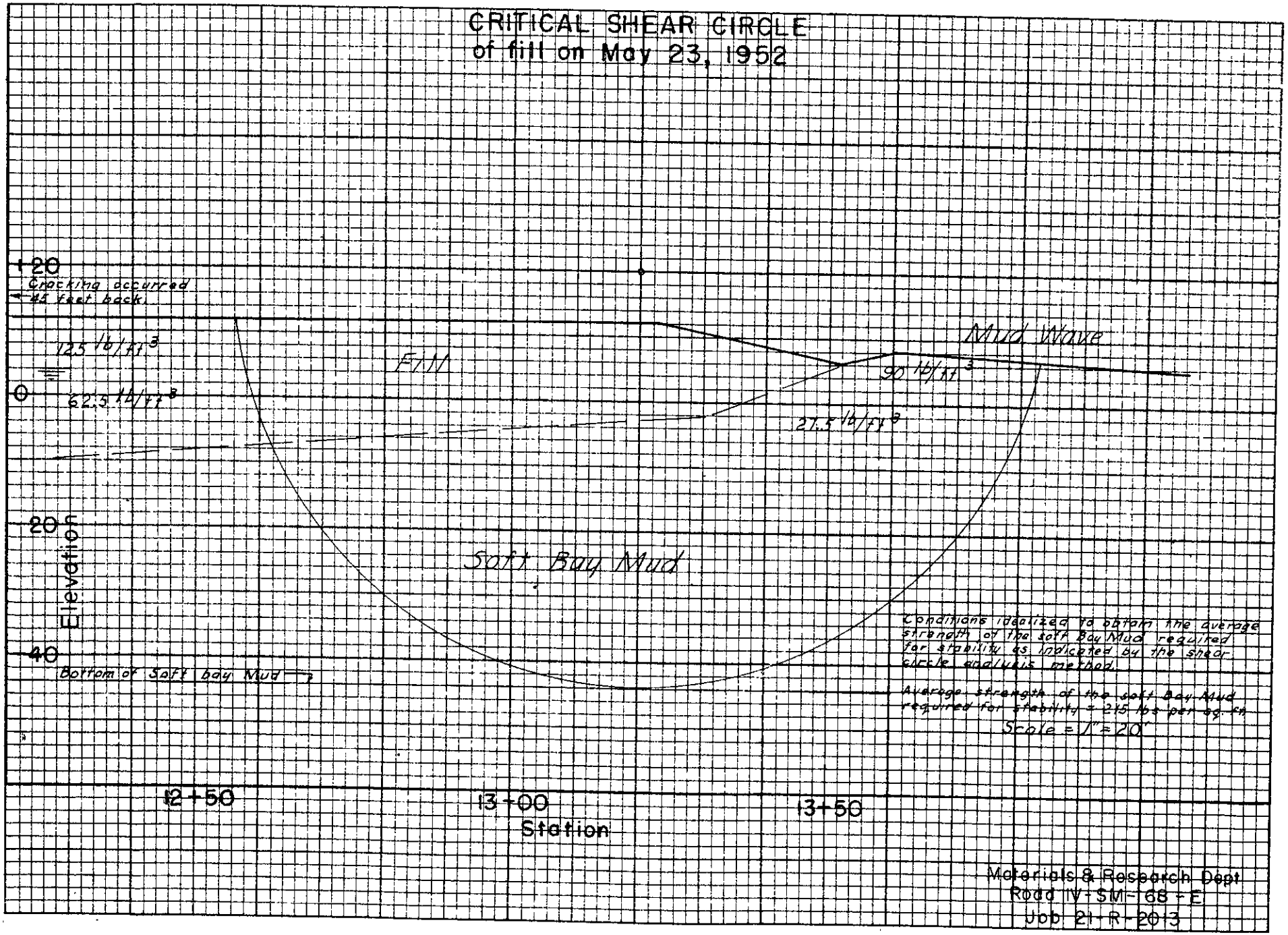


Fig F12

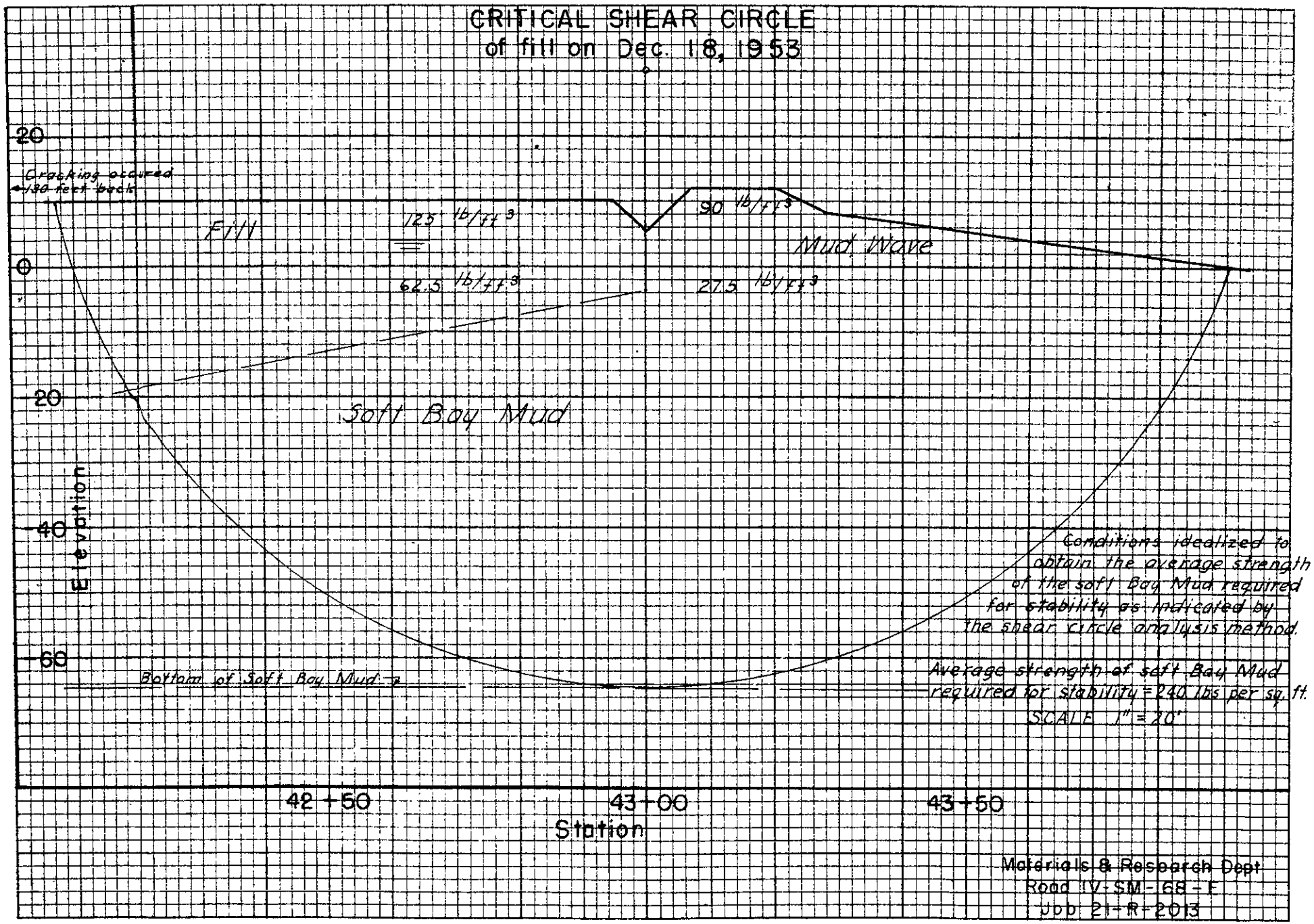

CRITICAL SHEAR CIRCLE
of fill on Dec. 18, 1953
20
0
20
40
60
Elevation
Cracking occured
130 feet back
Fill
125 lb/ft³
90 lb/ft³
Mud Wave
62.5 lb/ft³
27.5 lb/ft³
Soft Bay Mud
Bottom of Soft Bay Mud
Conditions idealized to
obtain the average strength
of the soft Bay Mud required
for stability as indicated by
the shear circle analysis method.
Average strength of soft Bay Mud
required for stability = 240 lbs per sq. ft.
SCALE 1" = 20'
42+50
43+00
43+50
Station
Materials & Research Dept
Road IV-SM-68-F
Job 21-R-2013

Fig F15

EUGENE DIETZGEN CO.
MADE IN U. S. A.

NO. 340 -10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

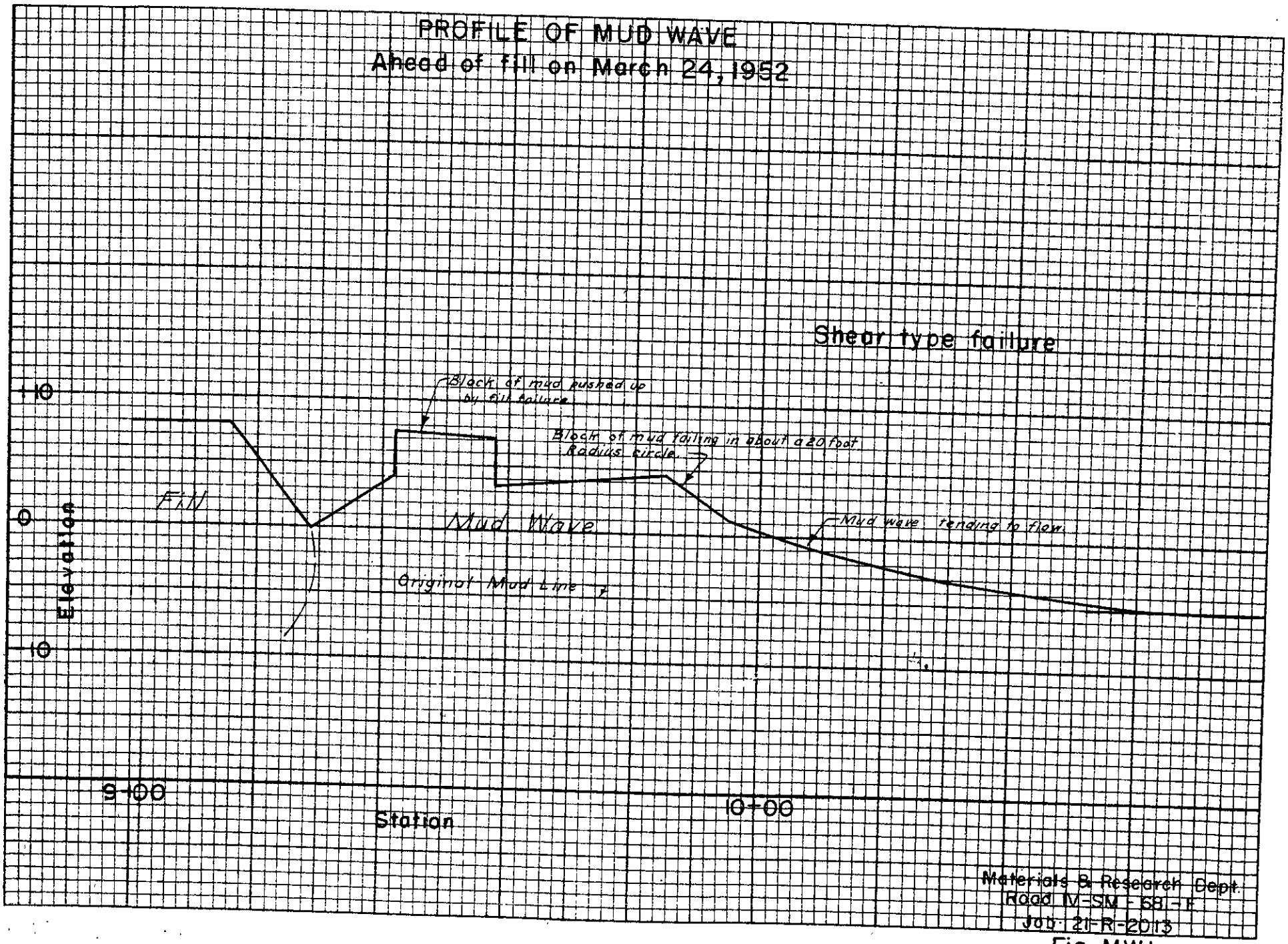


Fig M.W.1

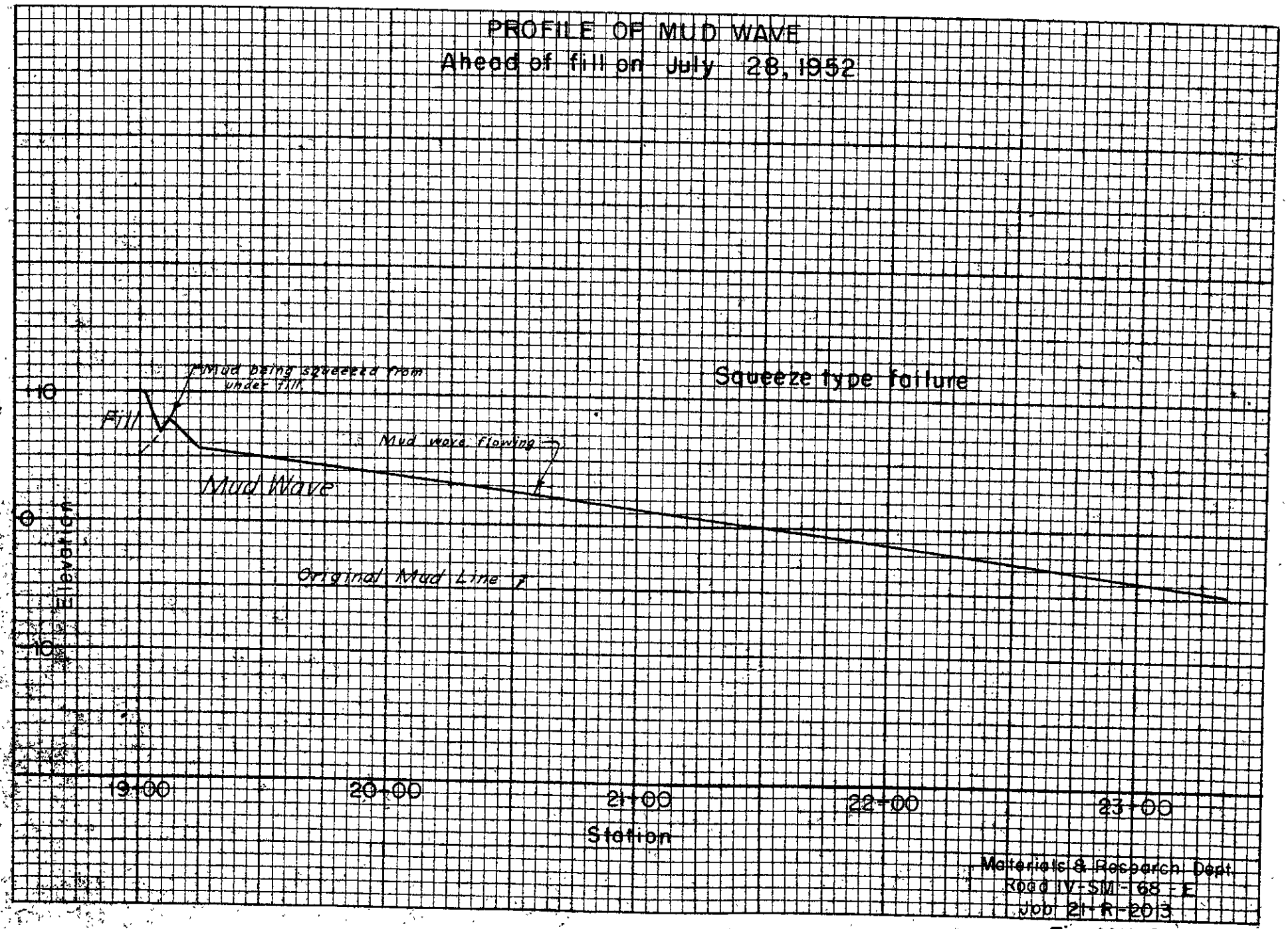

PROFILE OF MUD WAVE
Ahead of fill on July 28, 1952
Mud being squeezed from under fill
Squeeze type failure
Fill
Mud wave flowing
Mud Wave
Original Mud Line
+10
0
-10
Elevation
19+00
20+00
21+00
22+00
23+00
Station
Materials & Research Dept.
Road IV-SM-68-E
Job 21-R-2013

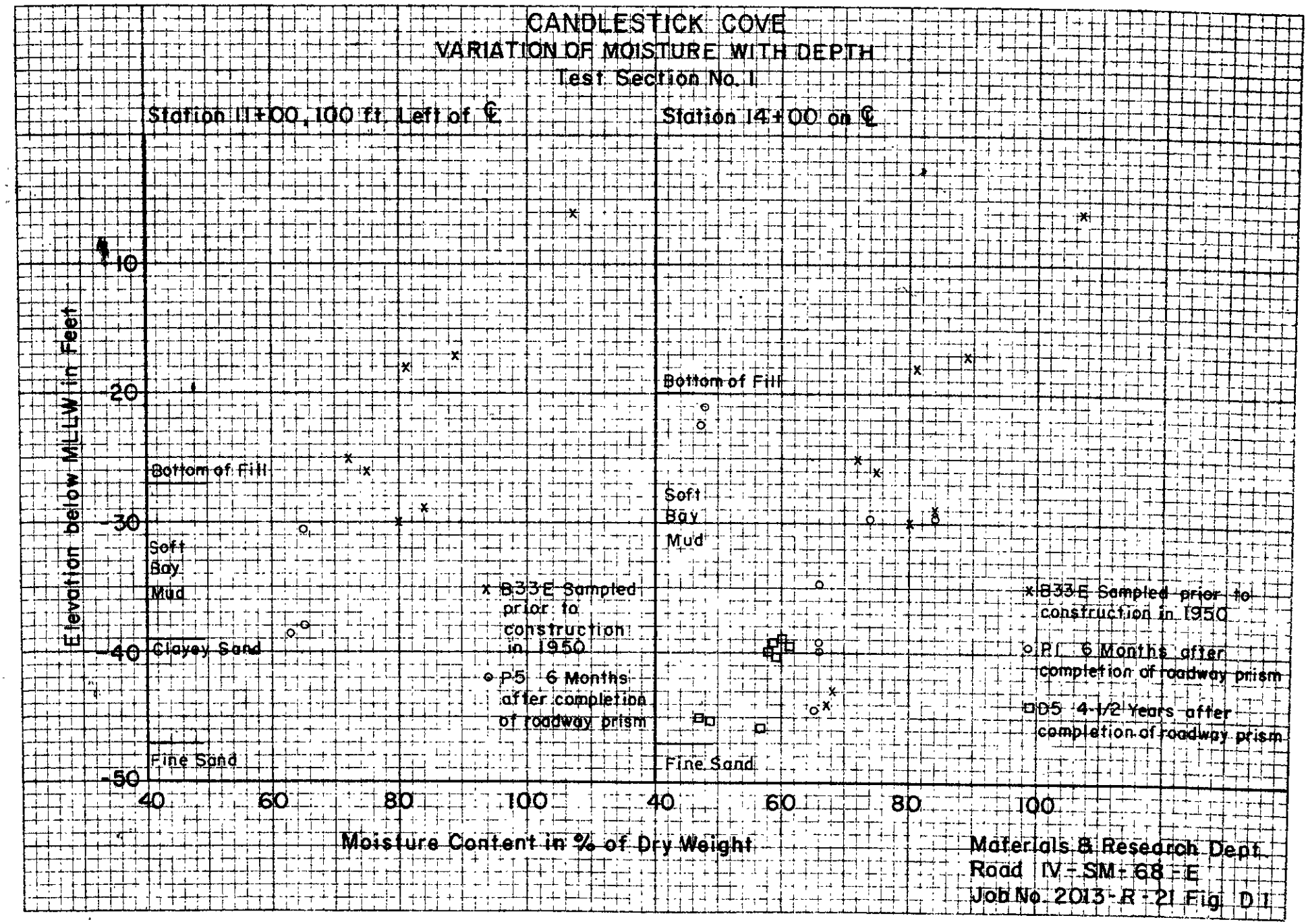
Fig M.W.2

PLOTS OF THE TEST RESULTS

Description	Plates
Moisture Comparisons	D-1, D-2, D-3, D-4, D-10
Strength Comparisons	D-5, D-6, D-7, D-8, D-9, D-11
Triaxial Test Plots	D-12, D-13, D-14, D-15
Consolidation Test Plots	
Preliminary Test Borings	D-16, D-17, D-18, D-19, D-20, D-21, D-22, D-23
First Unit - Borings, about one year after completion	D-26, D-27
Second Unit - Borings, about one month after completion	D-28, D-29, D-30, D-31, D-32, D-33
First Unit - Borings, about four and one-half years after completion	D-34, D-37
Second Unit - Borings, about two and one-half years after completion	D-35, D-36, D-38, D-39
Preconsolidation Pressures	D-40, D-41, D-42, D-43, D-44, D-45

NO 340 -10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

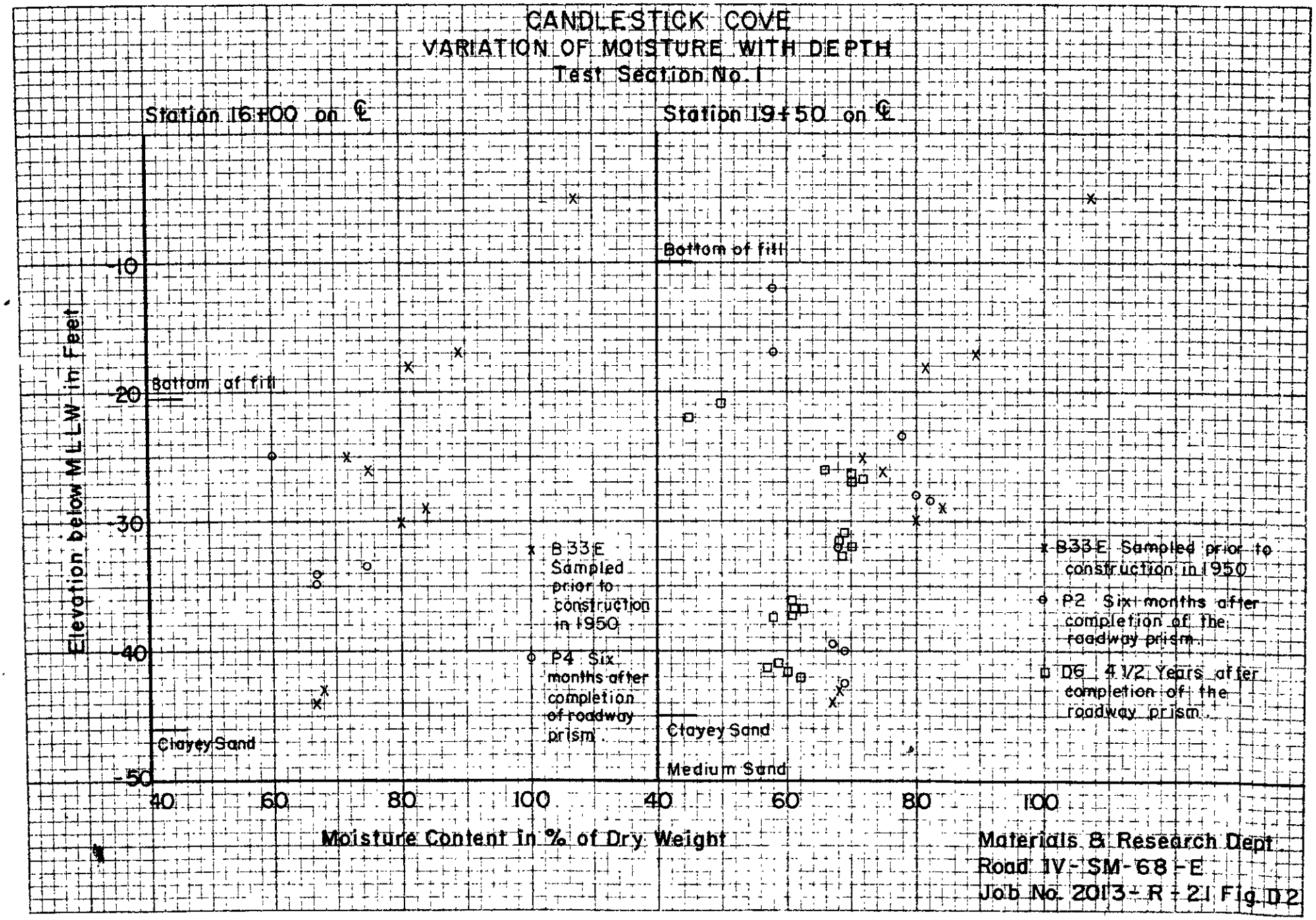
EUGENE DIETZGEN CO
MADE IN U. S. A.



Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. D1

NO 340 -10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

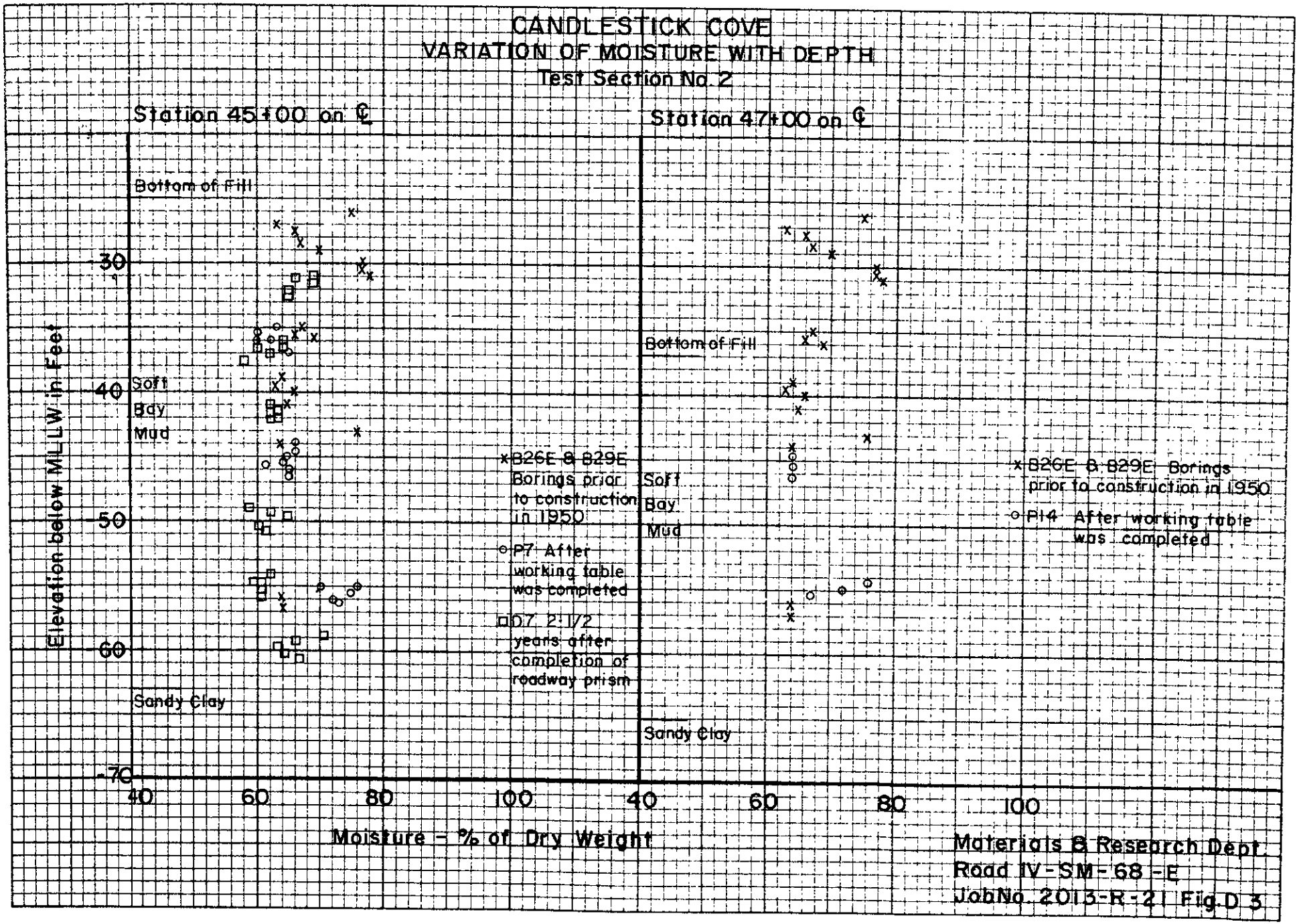
EUGENE DIETZGEN CO.
MADE IN U S A.



Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. D2

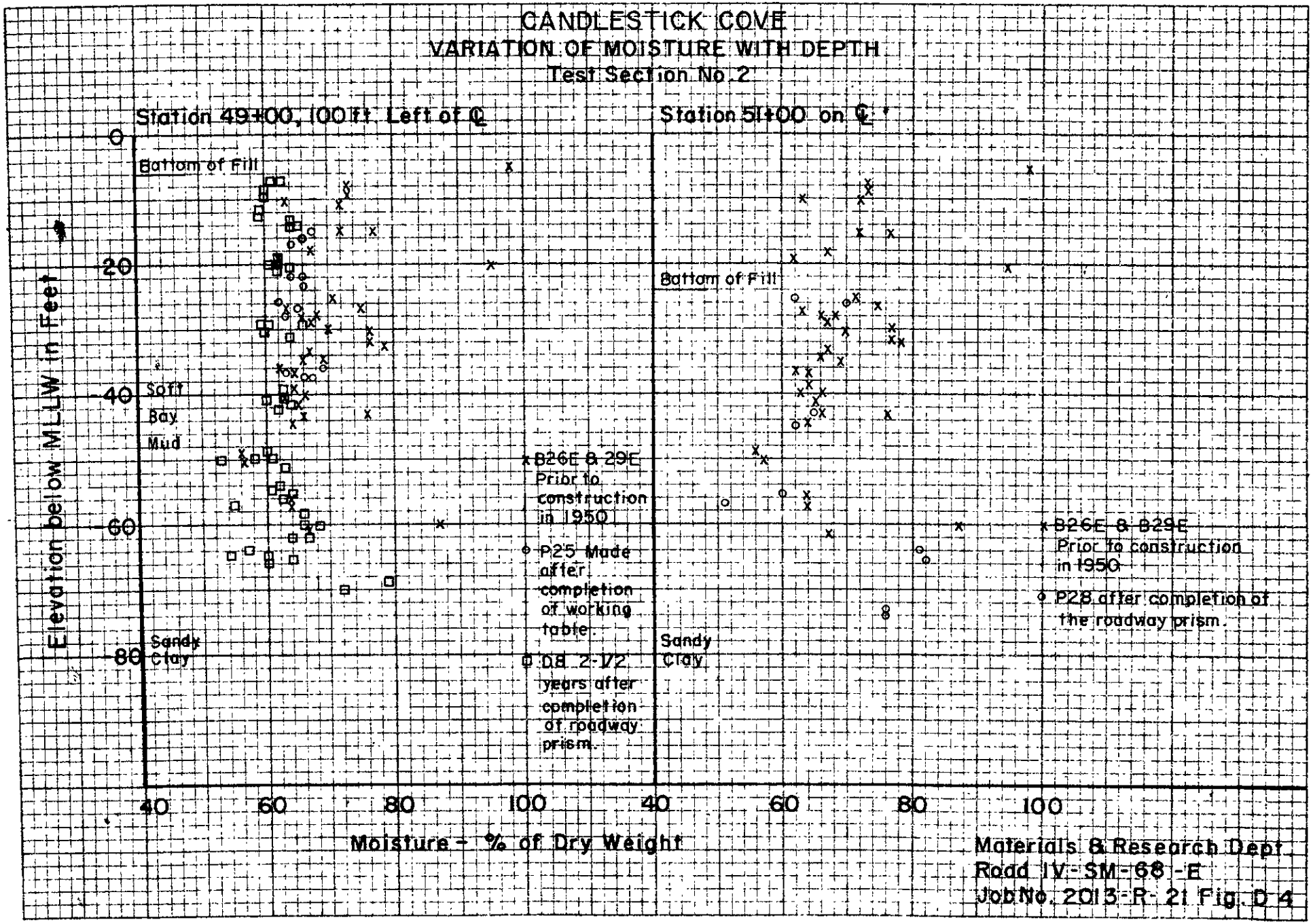
NO 340, -10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

EUGENE DIETZGEN CO
MADE IN U S A.

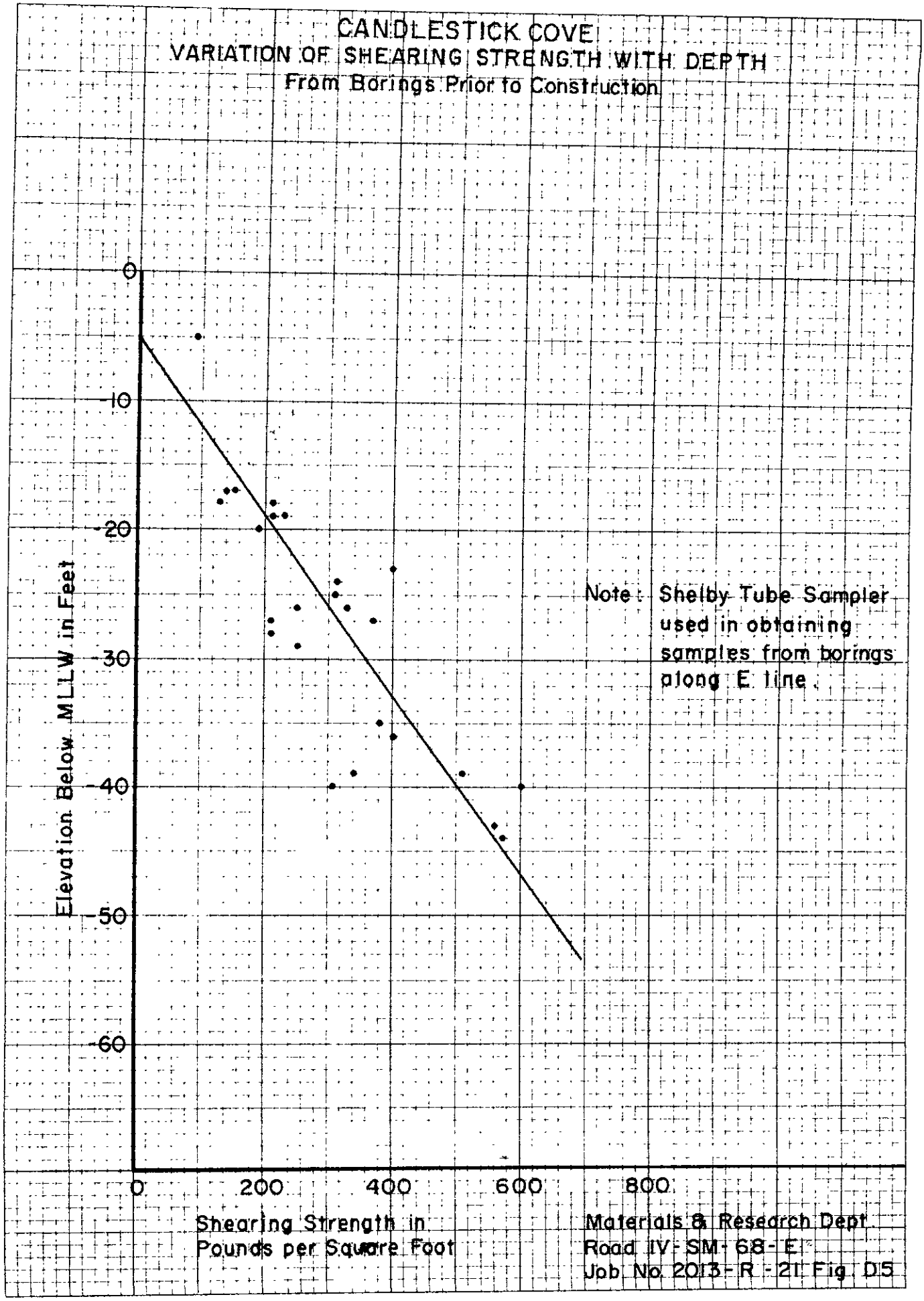


NO 340 -10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

EUGENE DIETZGEN CO
MADE IN U. S. A.



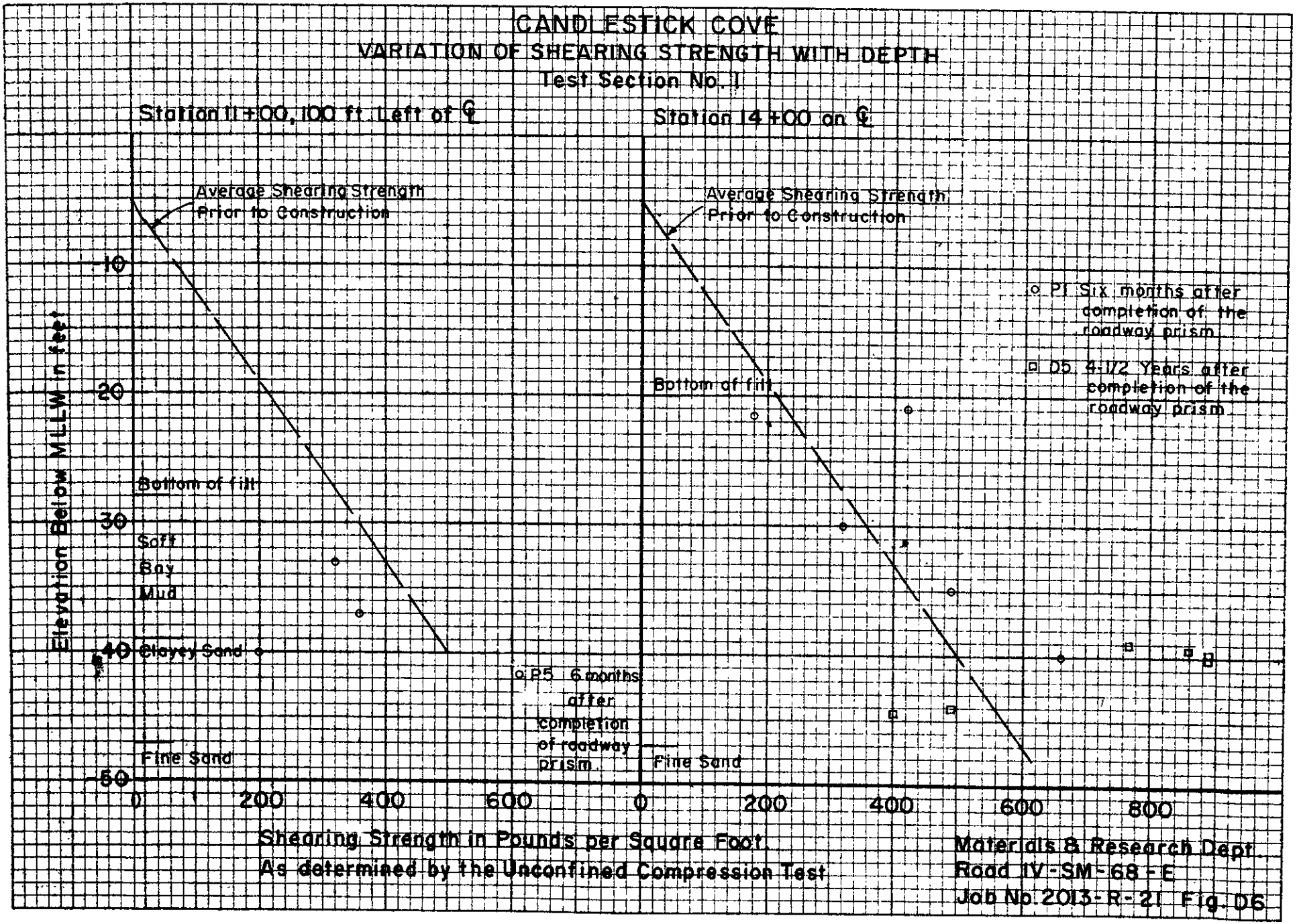
Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. D 4



EUGENE DIETZGEN CO.
NO. 340R-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

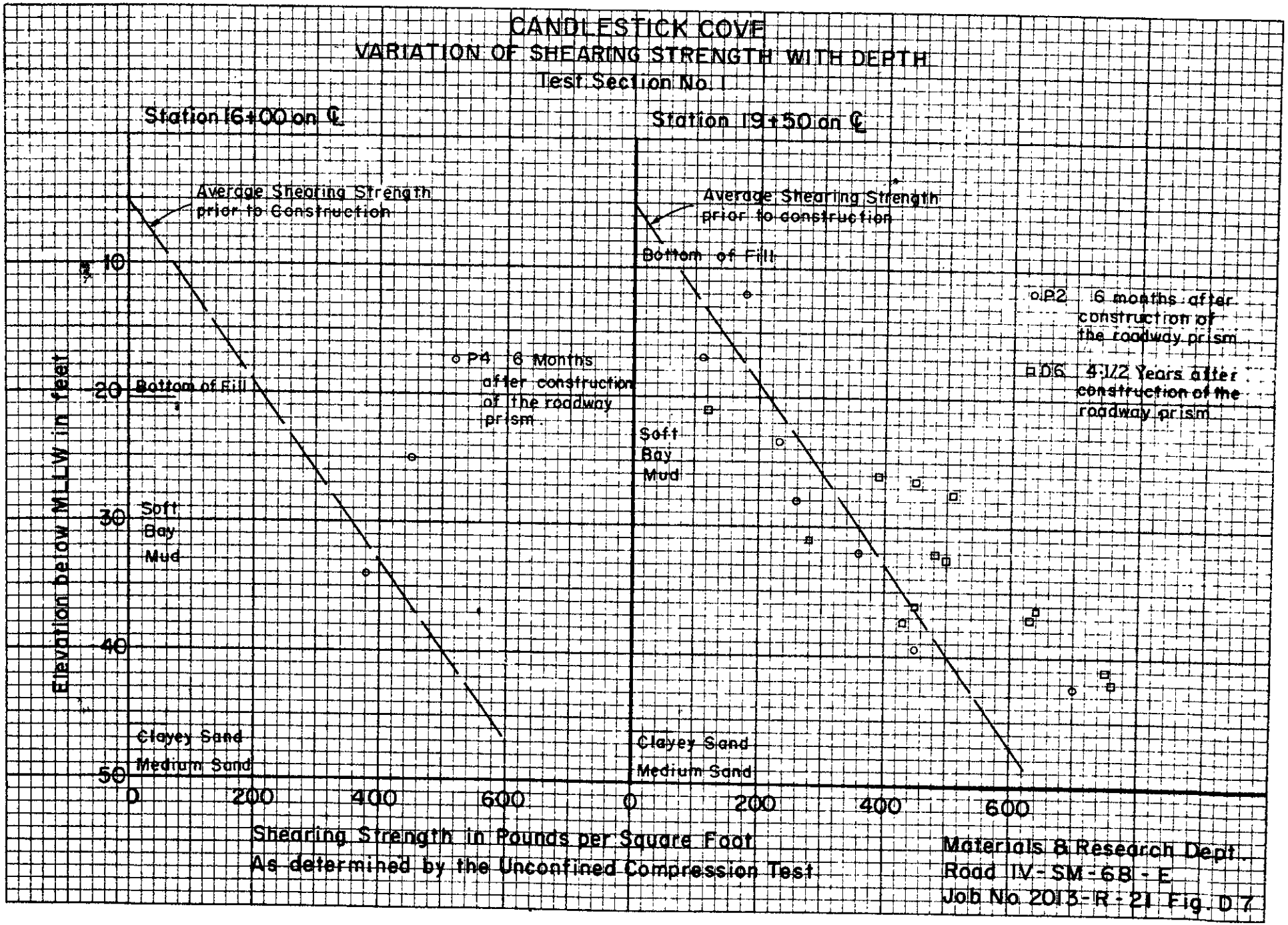
NO 340, -10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

EUGENE DIETZGEN CO.
MADE IN U. S. A.



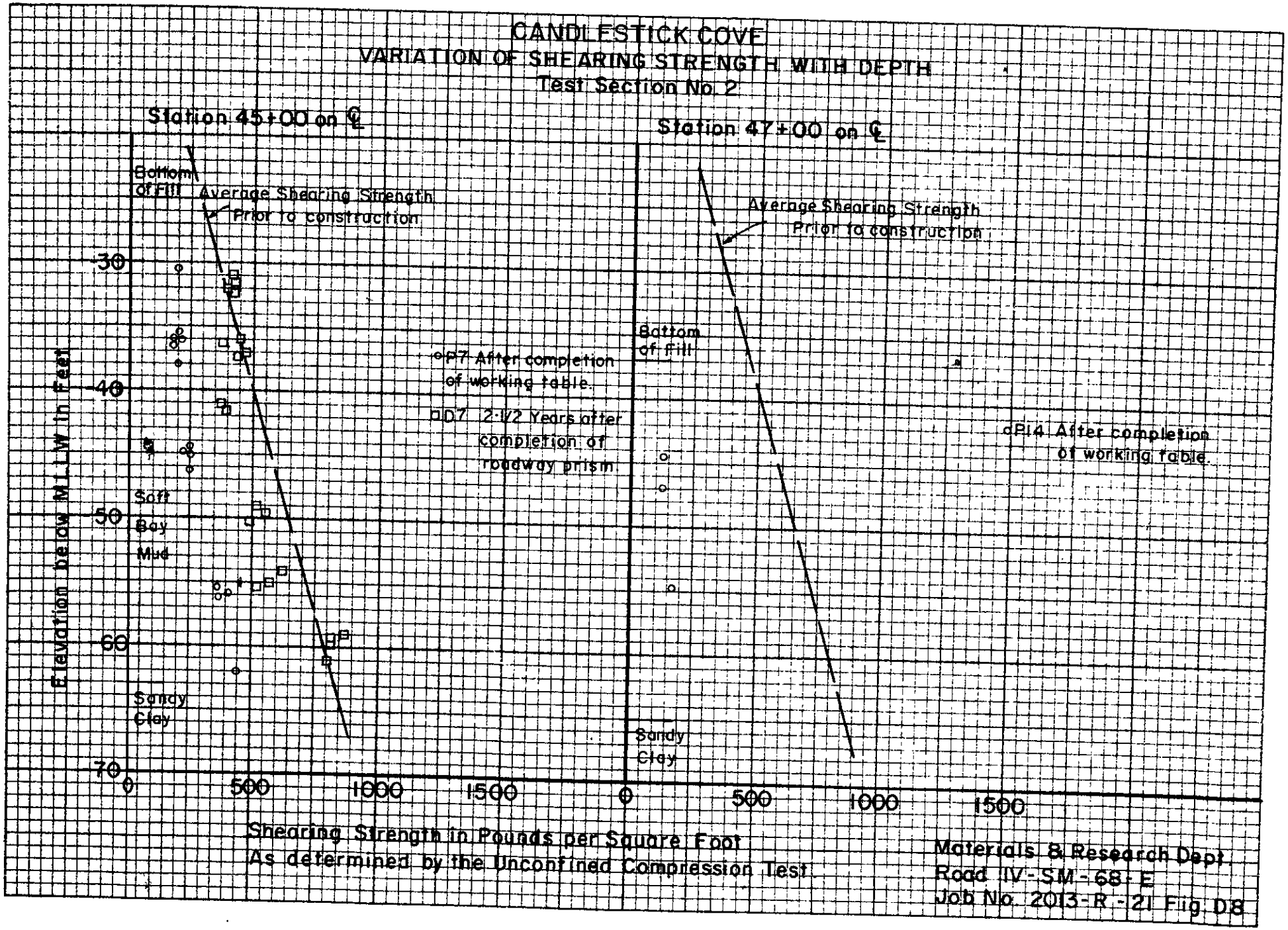
NO 340/ -10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

EUGENE DIETZGEN CO.
MADE IN U. S. A.



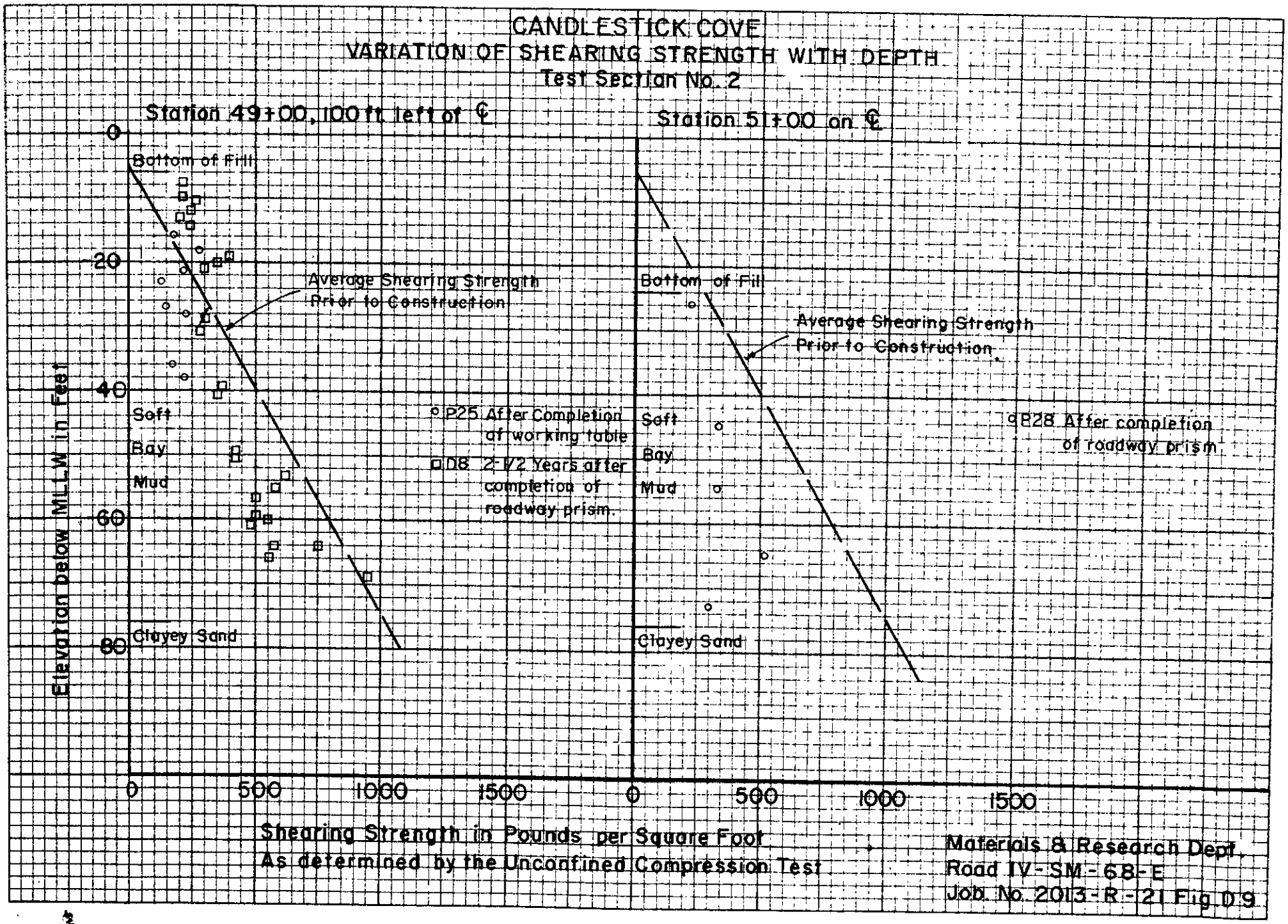
NO. 340/ -10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

EUGENE DIETZGEN CO.
MADE IN U. S. A.



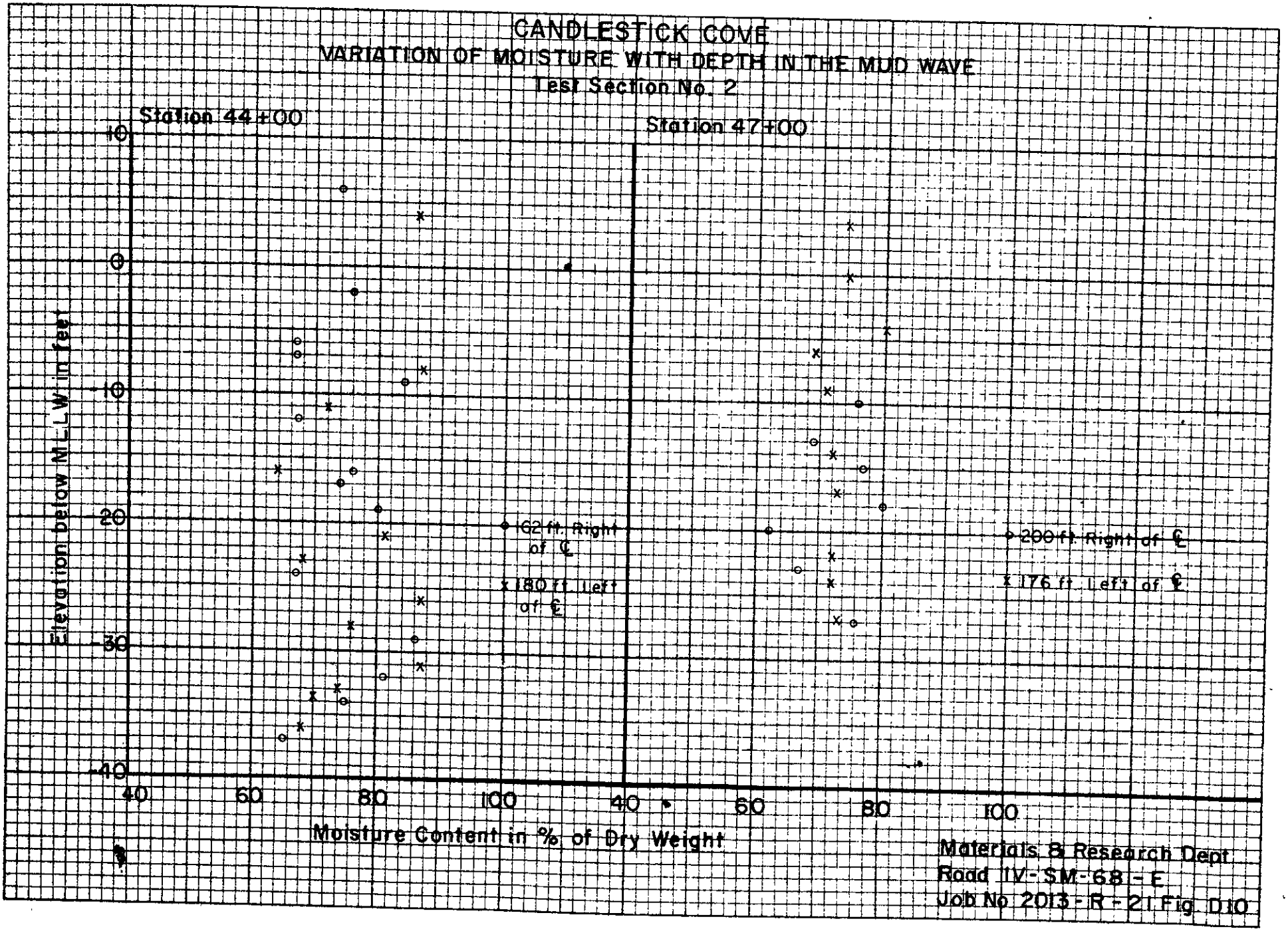
NO 340, -10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

EUGENE DIETZGEN CO
MADE IN U. S. A.



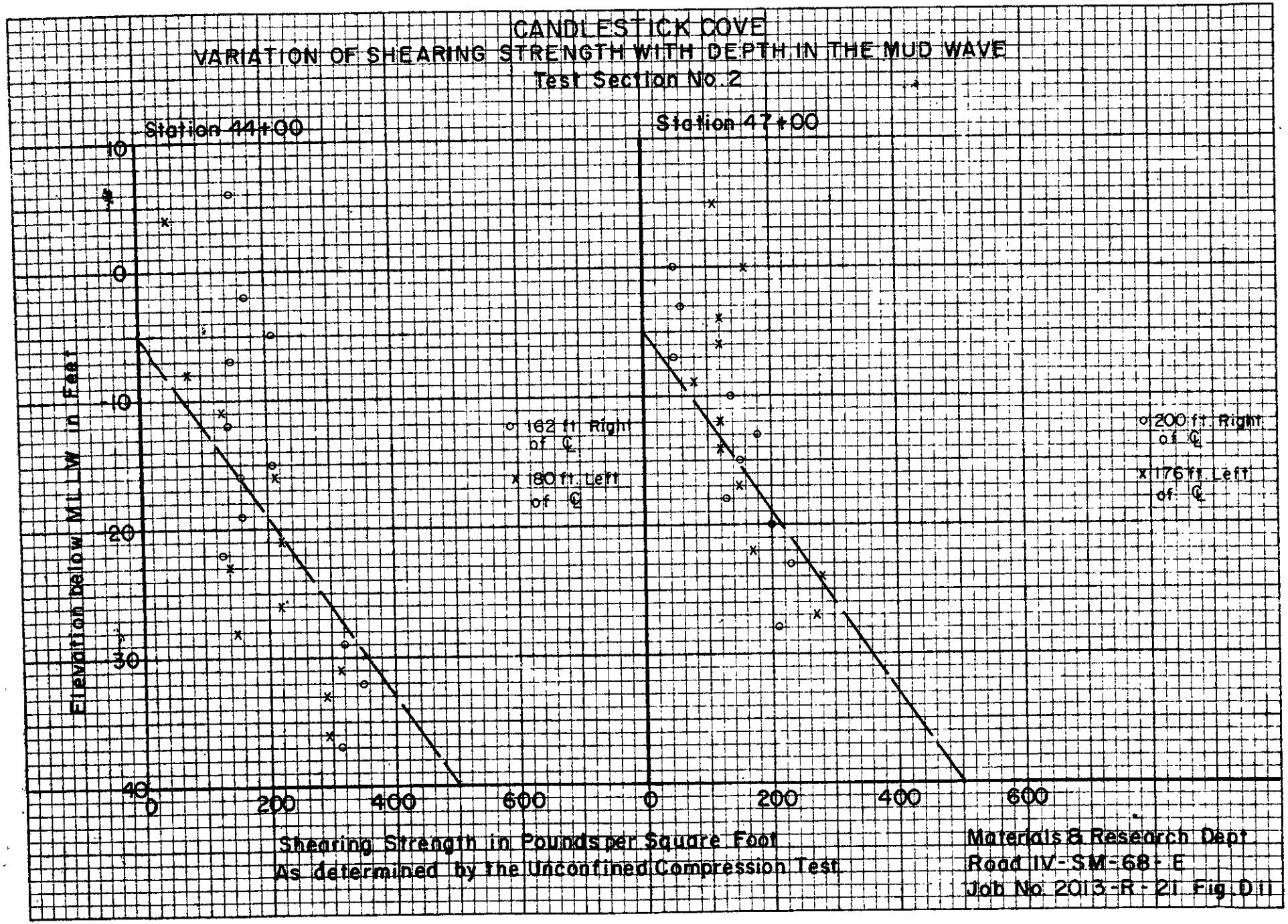
NO 340/ -10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

EUGENE DIETZGEN CO
MADE IN U. S. A.

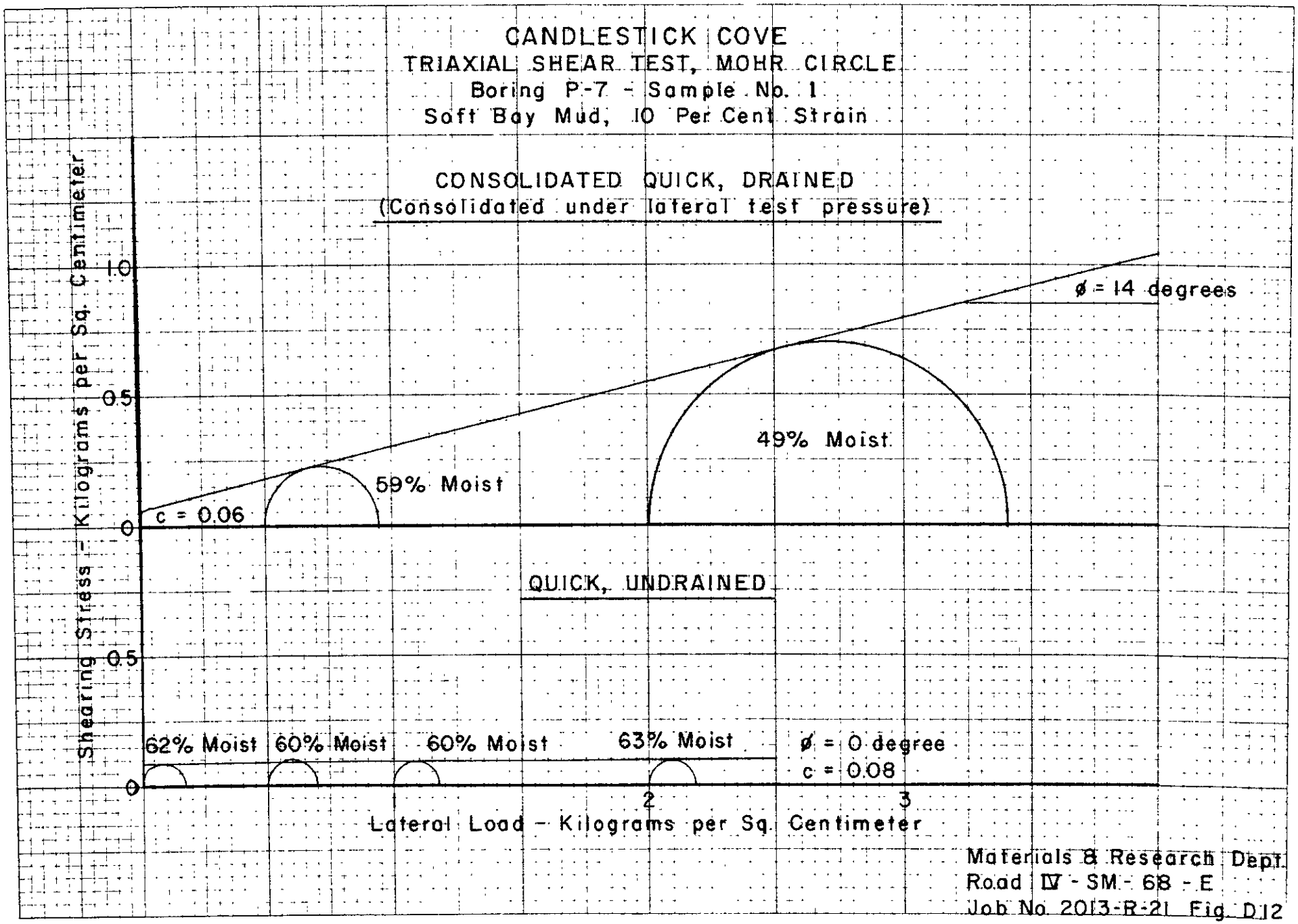


NO. 340/ -10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

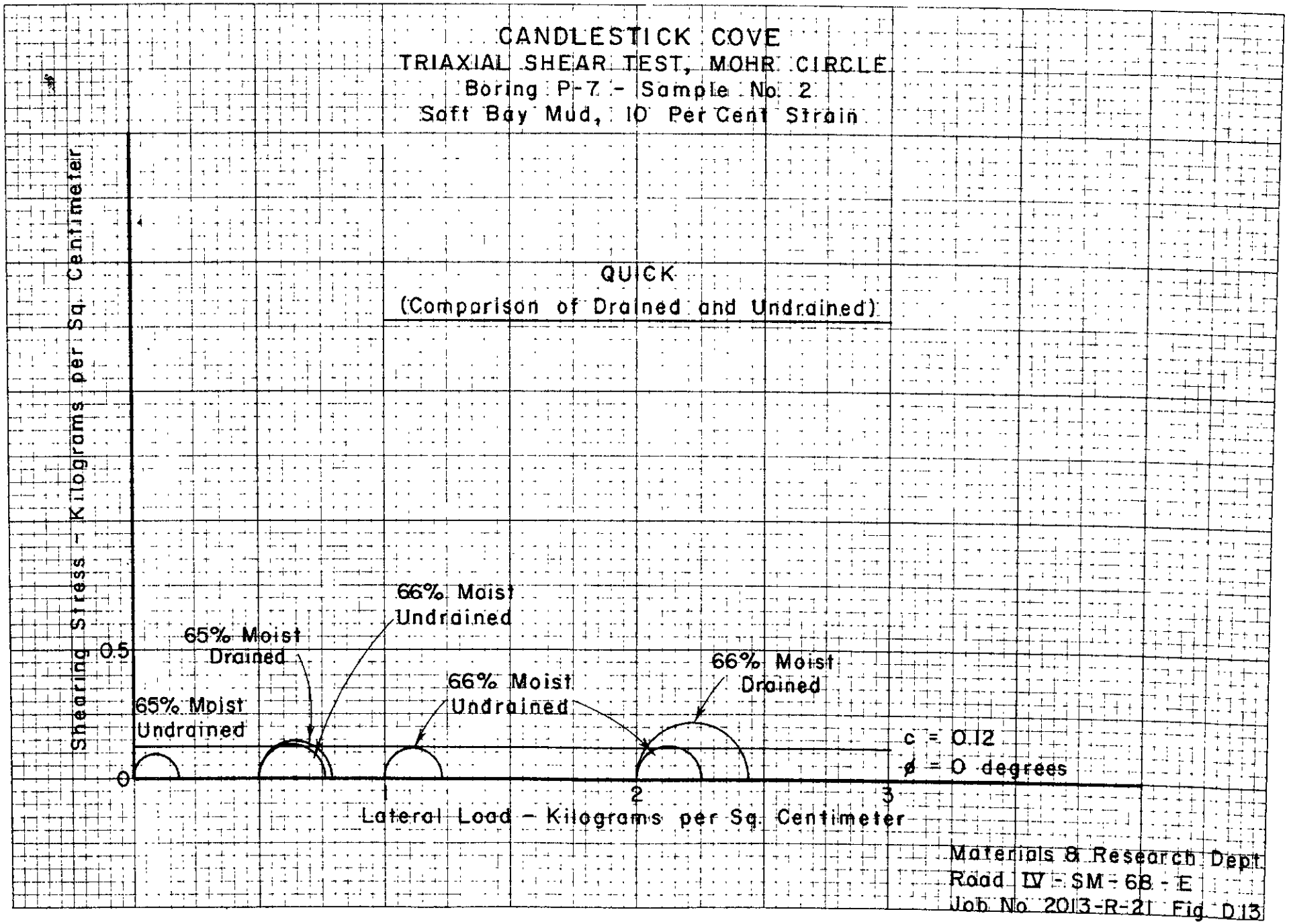
EUGENE DIETZGEN CO.
MADE IN U. S. A.



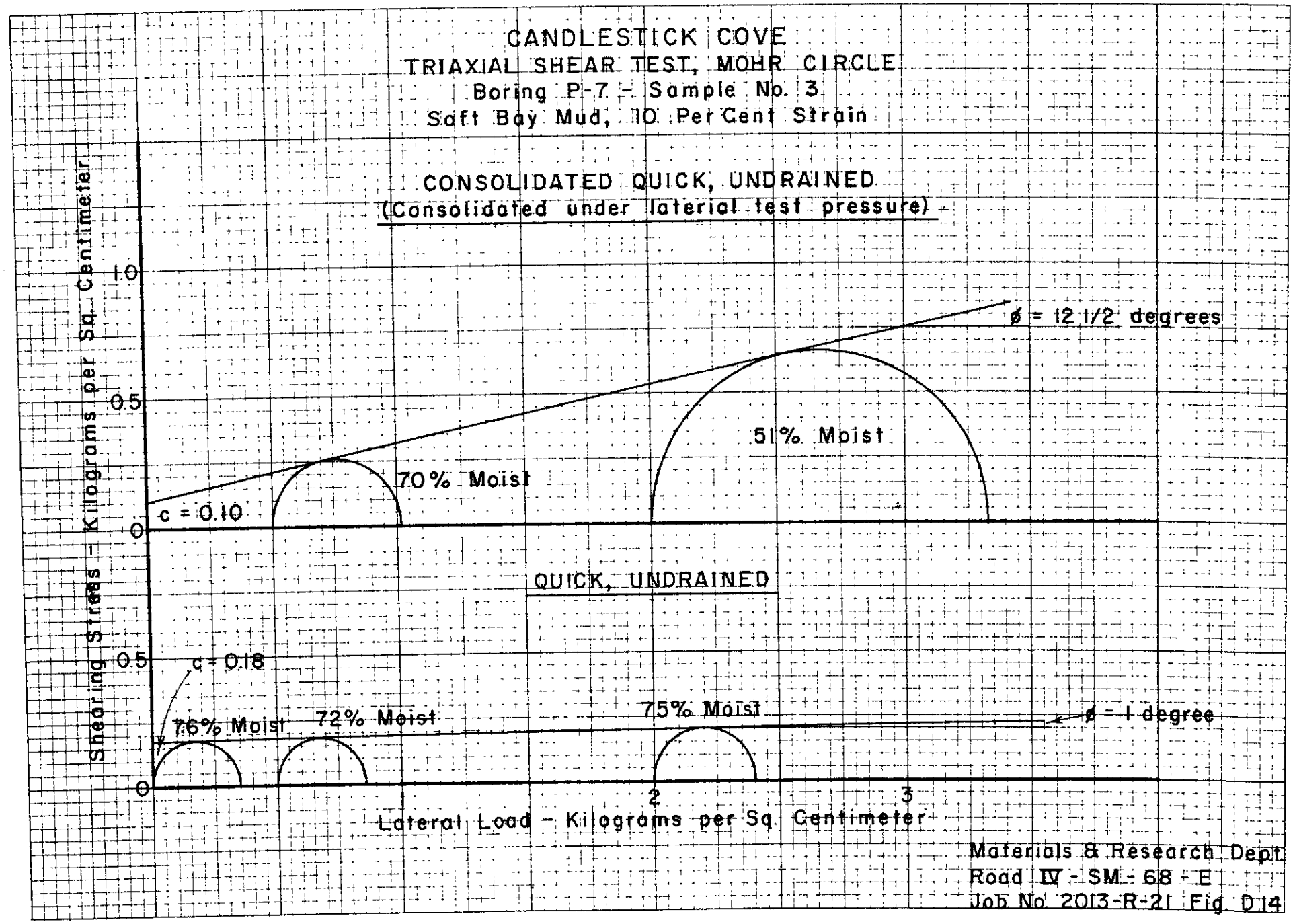
NO 340R 10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH
EUGENE DIETZGEN CO.



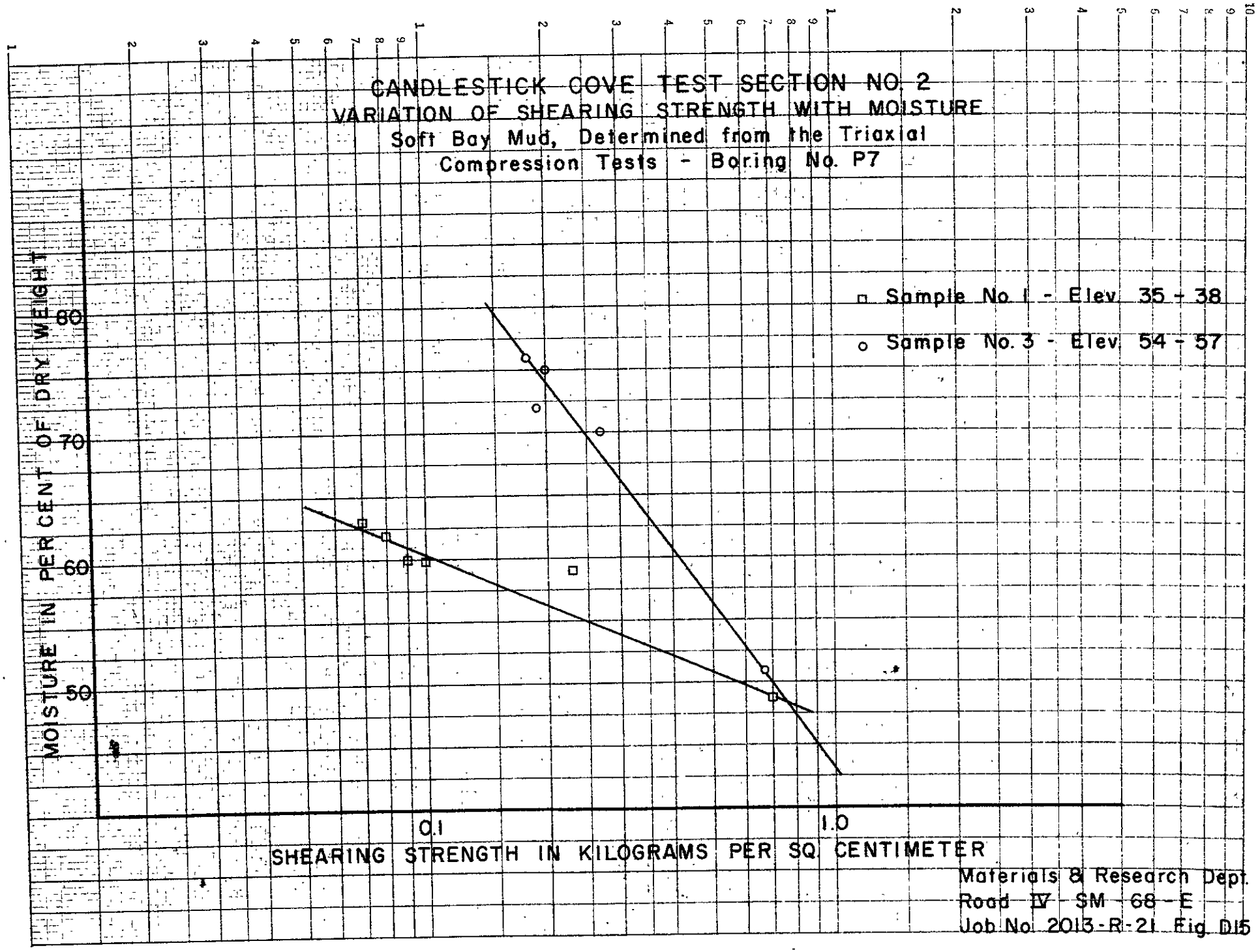
NO. 340R 10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH
EUGENE DIETZGEN CO.
MADE IN U.S.A.



NO. 340R-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH
EUGENE DIETZGEN CO
MADE IN U.S.A.

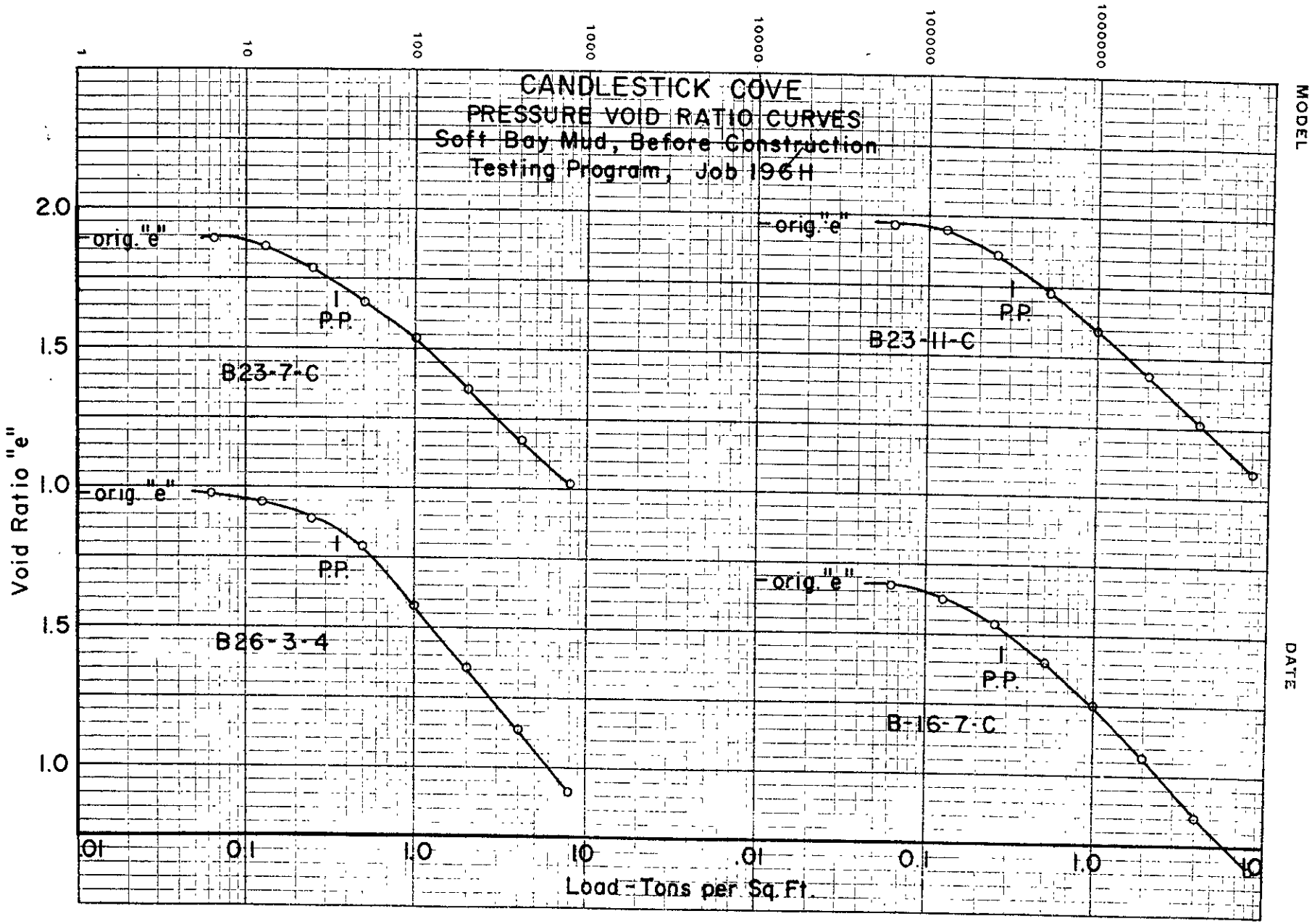


359-73 KEUFFEL & ESSER CO.
Semi-Logarithmic, 3 Cycles X 10 to the ½ inch.
MADE IN U.S.A.



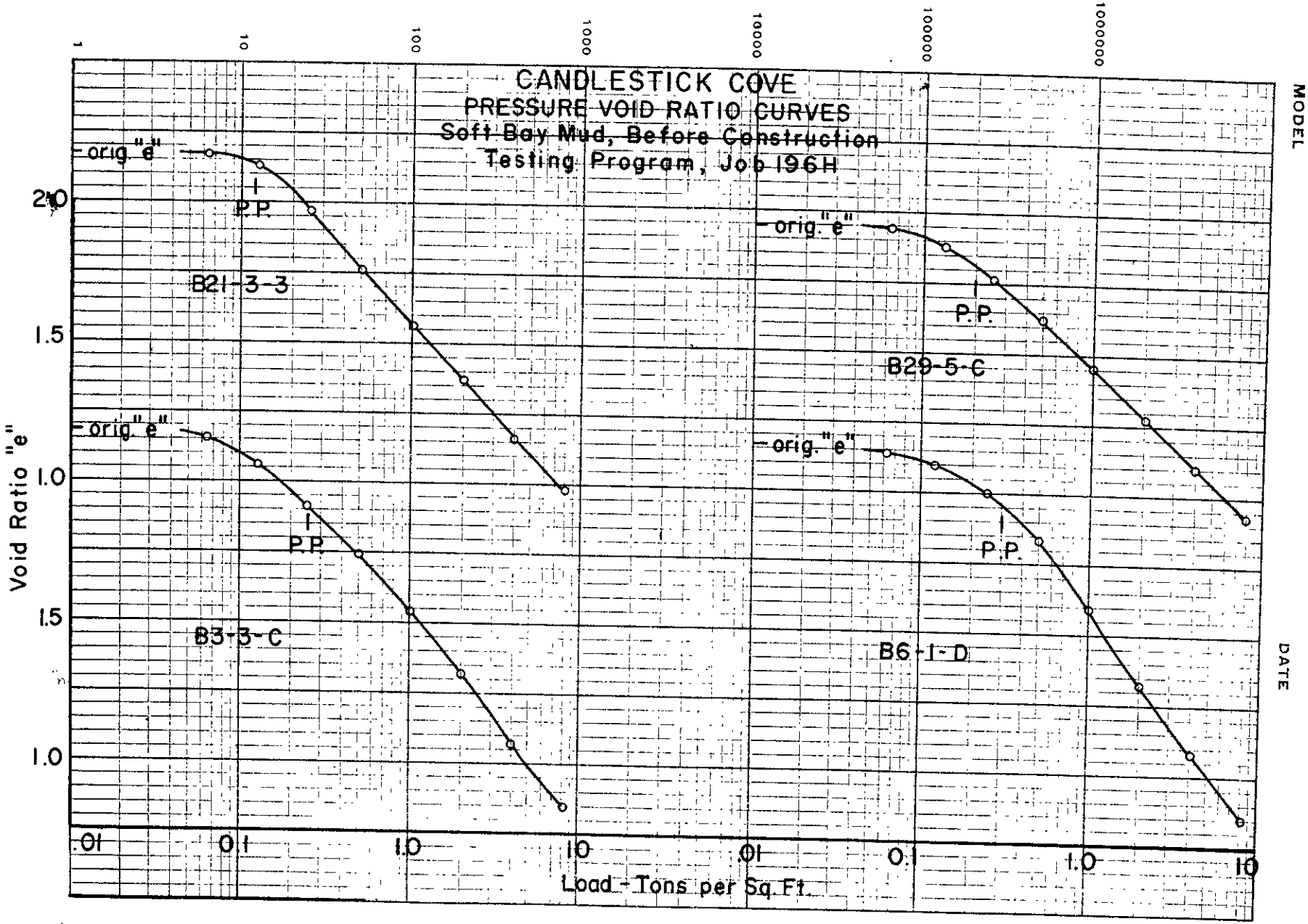
Materials & Research Dept.
Road IV - SM - 68 - E
Job No. 2013-R-21 Fig. D15

359 96 KEUFFEL & ESSER CO
Semi-Logarithmic, 7 Cycles X 5 to the ½ inch.
MADE IN U. S. A

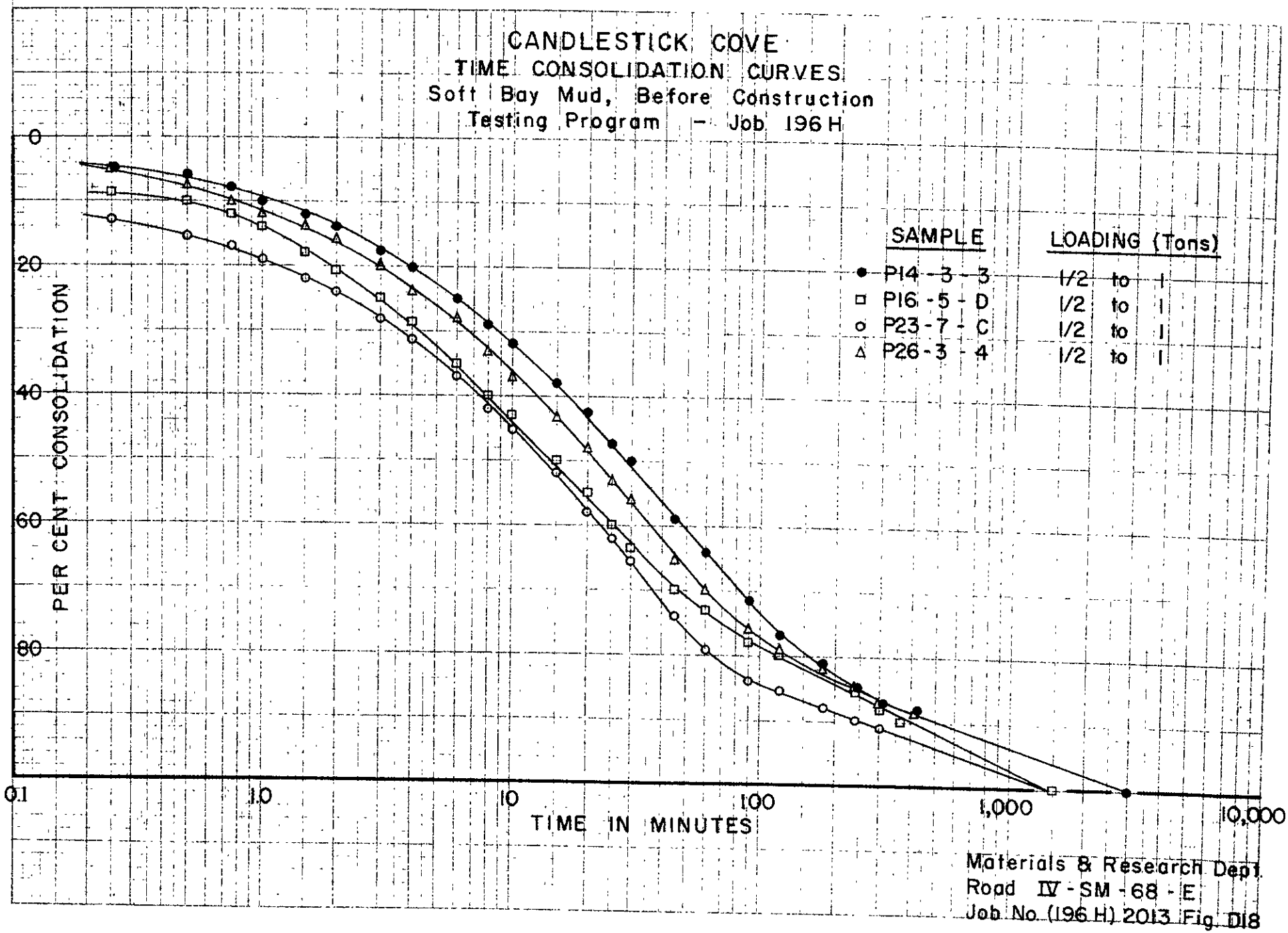


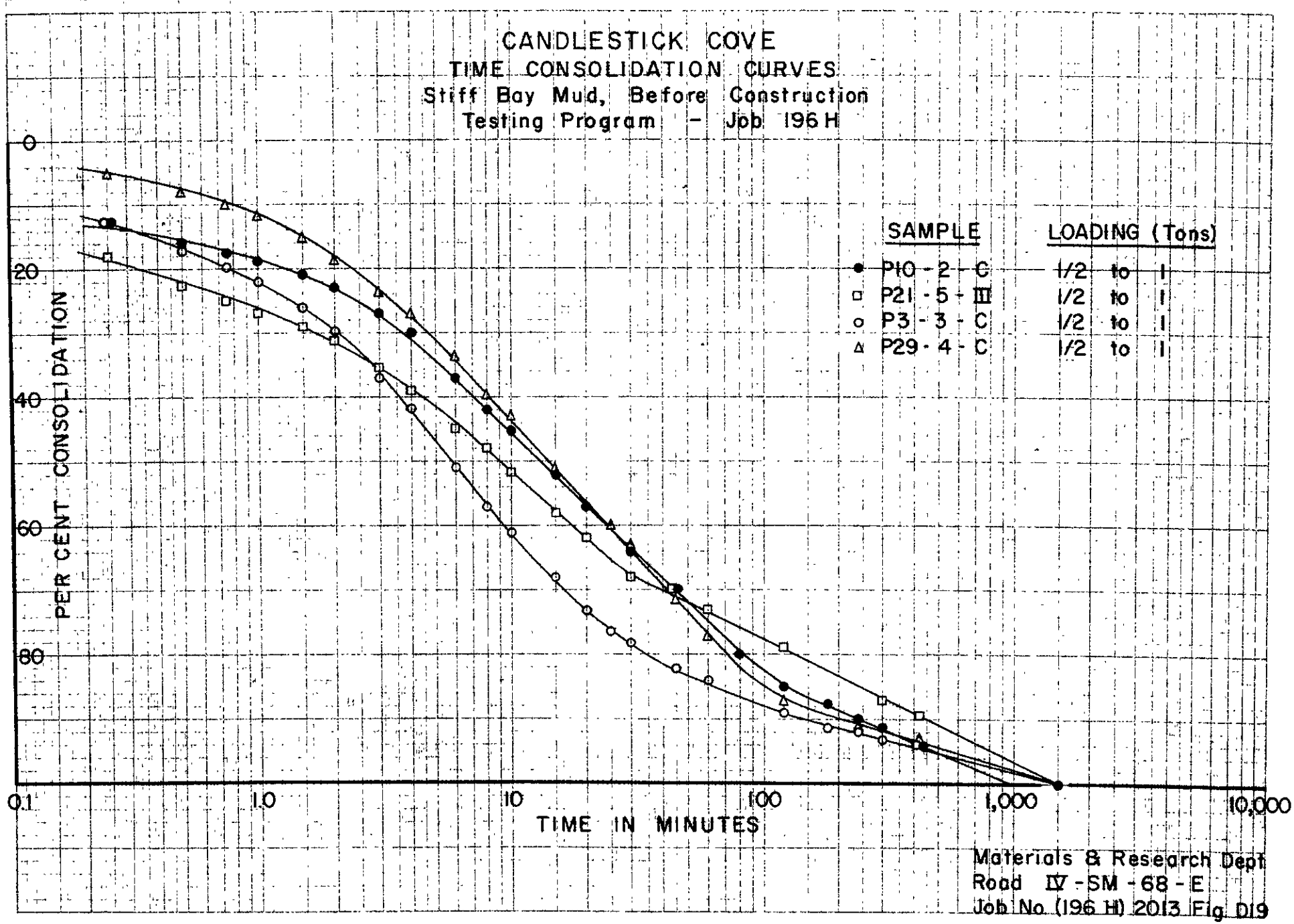
Materials & Research Dept.
Road IV-SM-68-E
Job No.(196H) 2013 Fig. D 16

359-96 KEUFFEL & ESSER CO
Semi-Logarithmic, 7 Cycles X 5 to the ½ inch
MADE IN U S A

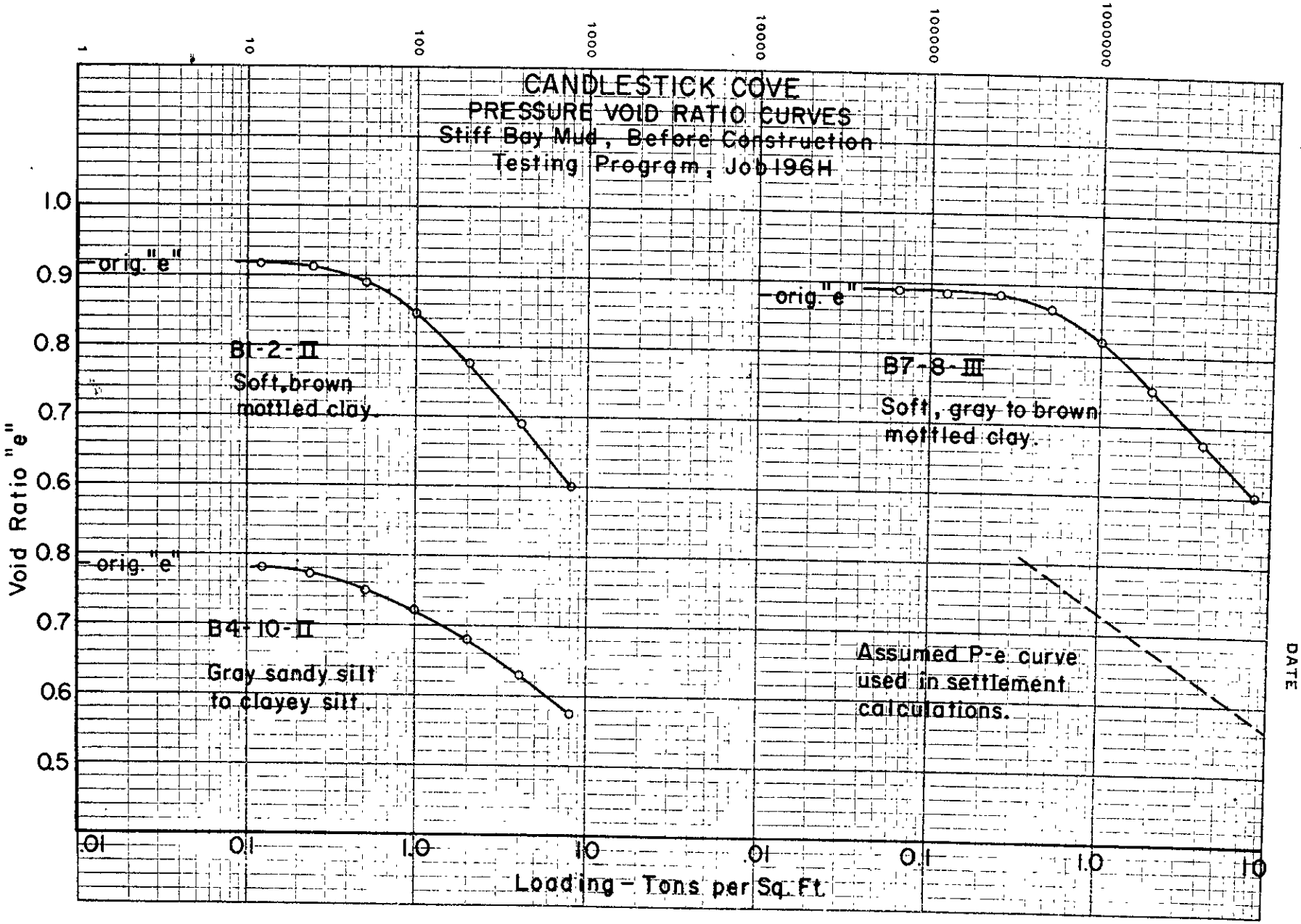


Materials & Research Dept.
Road IV - SM - 68 - E
Job No. (196H) 2013 Fig. D 17


CANDLESTICK COVE
TIME CONSOLIDATION CURVES
Soft Bay Mud, Before Construction
Testing Program - Job 196 H
SAMPLE
LOADING (Tons)
P14 - 3 - 3 1/2 to 1
P16 - 5 - D 1/2 to 1
P23 - 7 - C 1/2 to 1
P26 - 3 - 4 1/2 to 1
PER CENT CONSOLIDATION
0
20
40
60
80
0.1
1.0
10
100
1,000
10,000
TIME IN MINUTES
Materials & Research Dept.
Road IV - SM - 68 - E
Job No. (196 H) 2013 Fig. D18


CANDLESTICK COVE
TIME CONSOLIDATION CURVES
Stiff Bay Mud, Before Construction
Testing Program - Job 196 H
SAMPLE
LOADING (Tons)
P10 - 2 - C 1/2 to 1
P21 - 5 - III 1/2 to 1
P3 - 3 - C 1/2 to 1
P29 - 4 - C 1/2 to 1
PER CENT CONSOLIDATION
0
20
40
60
80
0.1
1.0
10
100
1,000
10,000
TIME IN MINUTES
Materials & Research Dept
Road IV - SM - 68 - E
Job No. (196 H) 2013 Fig. D19

359-96 KEUFFEL & ESSER CO.
Semi-Logarithmic. 7 Cycles X 5 to the ½ inch
MADE IN U. S. A.

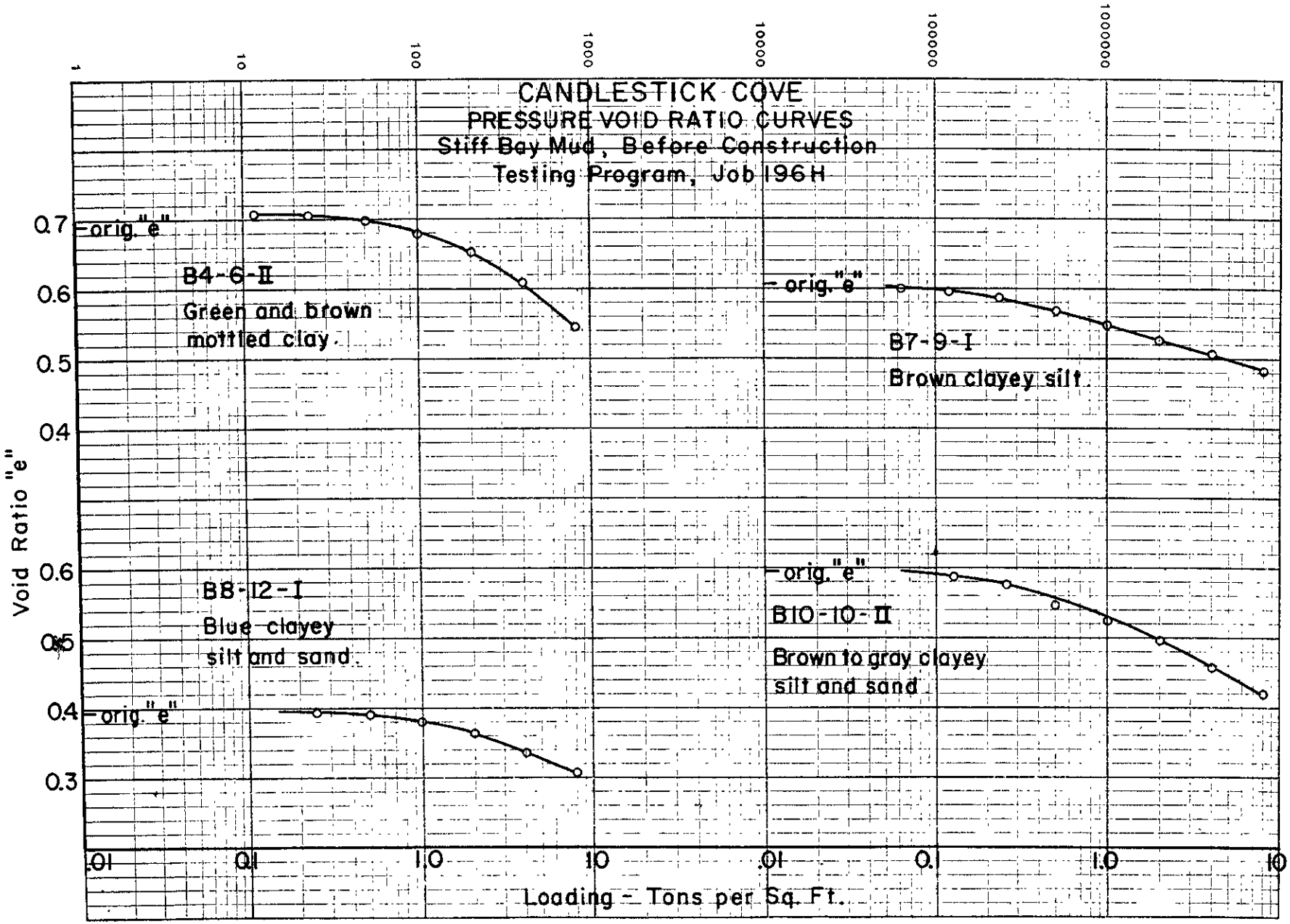


MODEL

DATE

Materials & Research Dept.
Road IV - SM - 68 - E
Job No. (196H) 2013 Fig. D 20

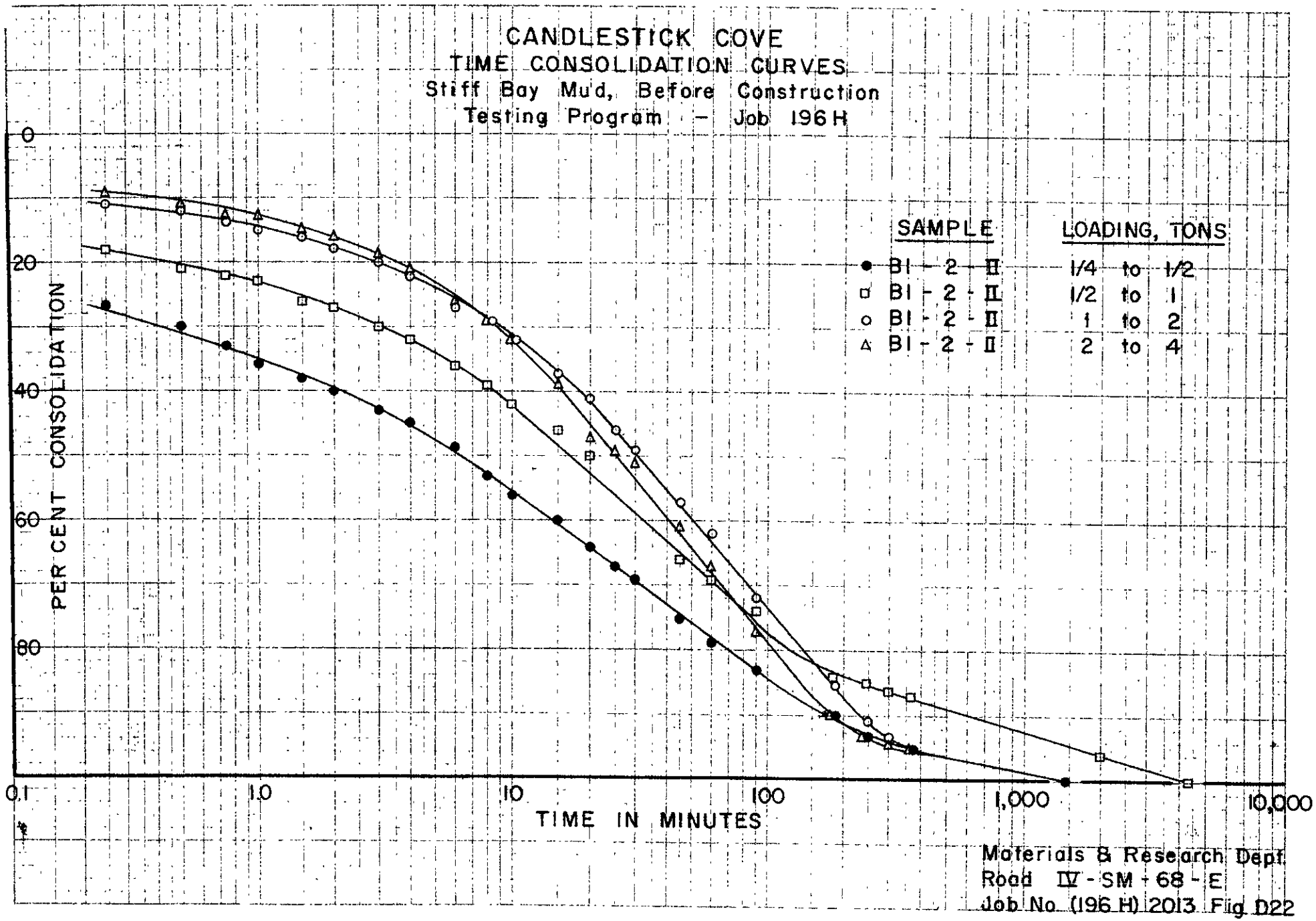
359-96 KEUFFEL & ESSER CO
Semi-Logarithmic, 7 Cycles X 5 to the ½ inch
MADE IN U S A.



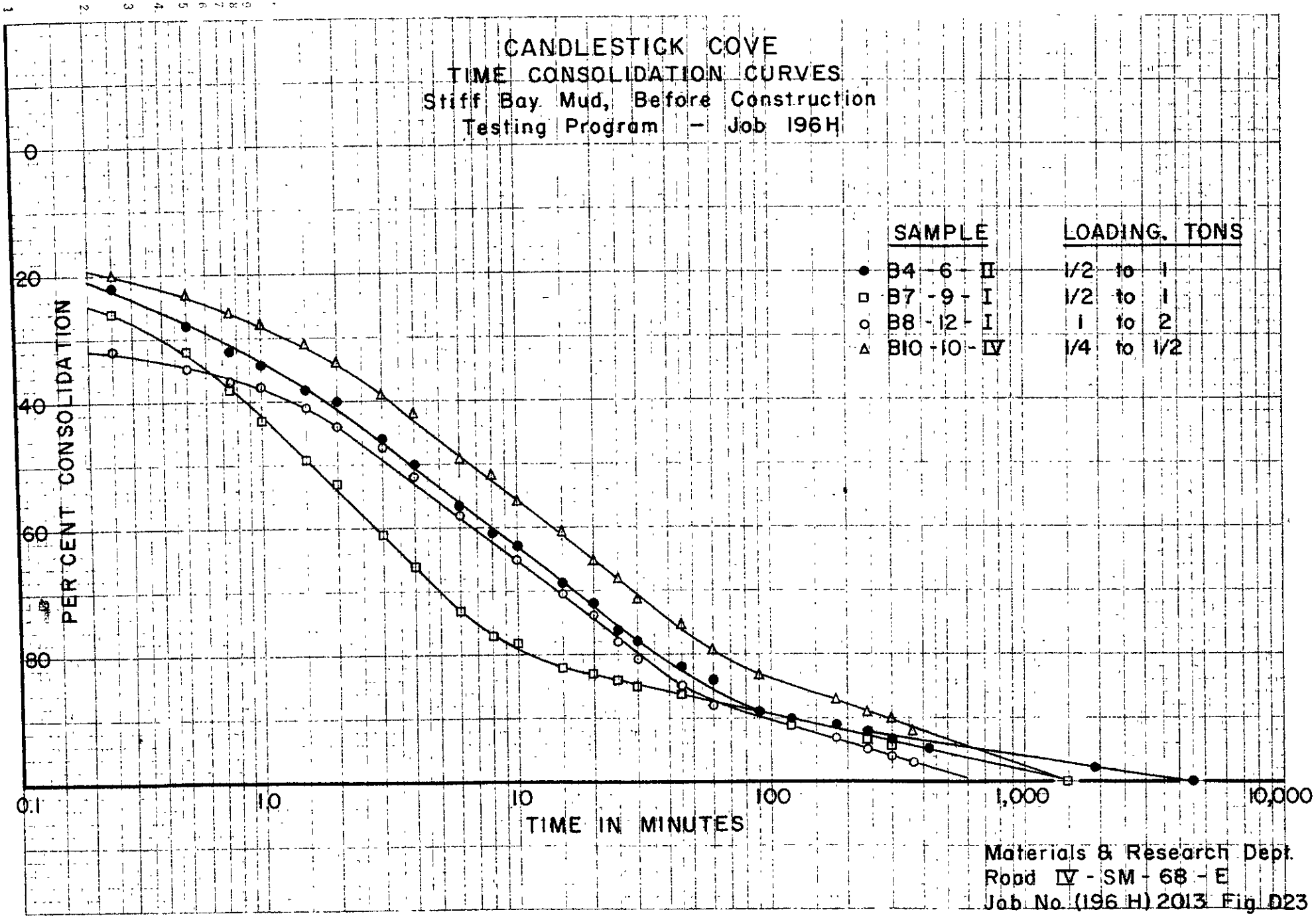
MODEL

DATE

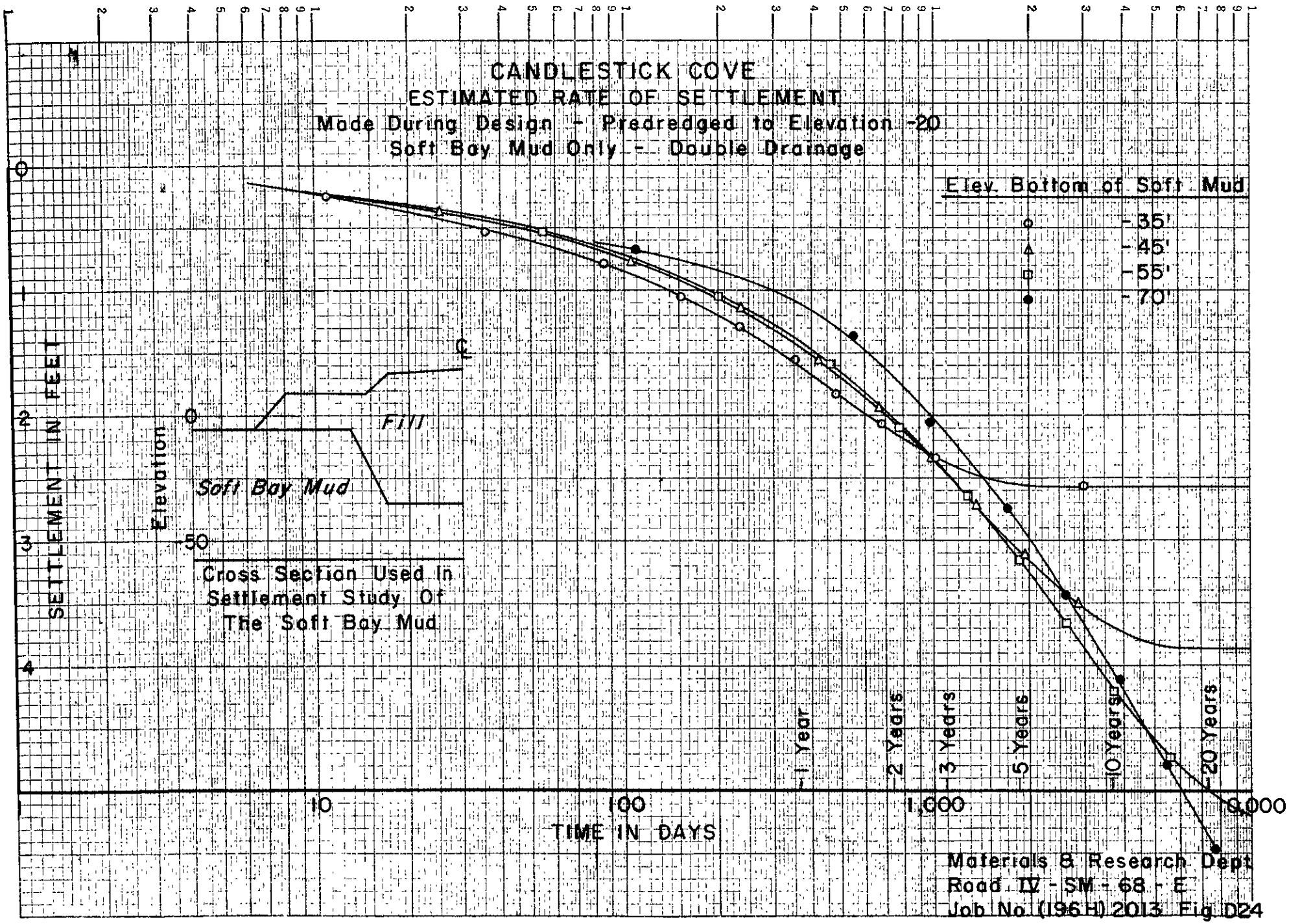
Materials & Research Dept.
Road IV - SM - 68 - E
Job No. (196H)2013 Fig.D 21



Materials & Research Dept.
Road IV - SM - 68 - E
Job No. (196 H) 2013 Fig. D22

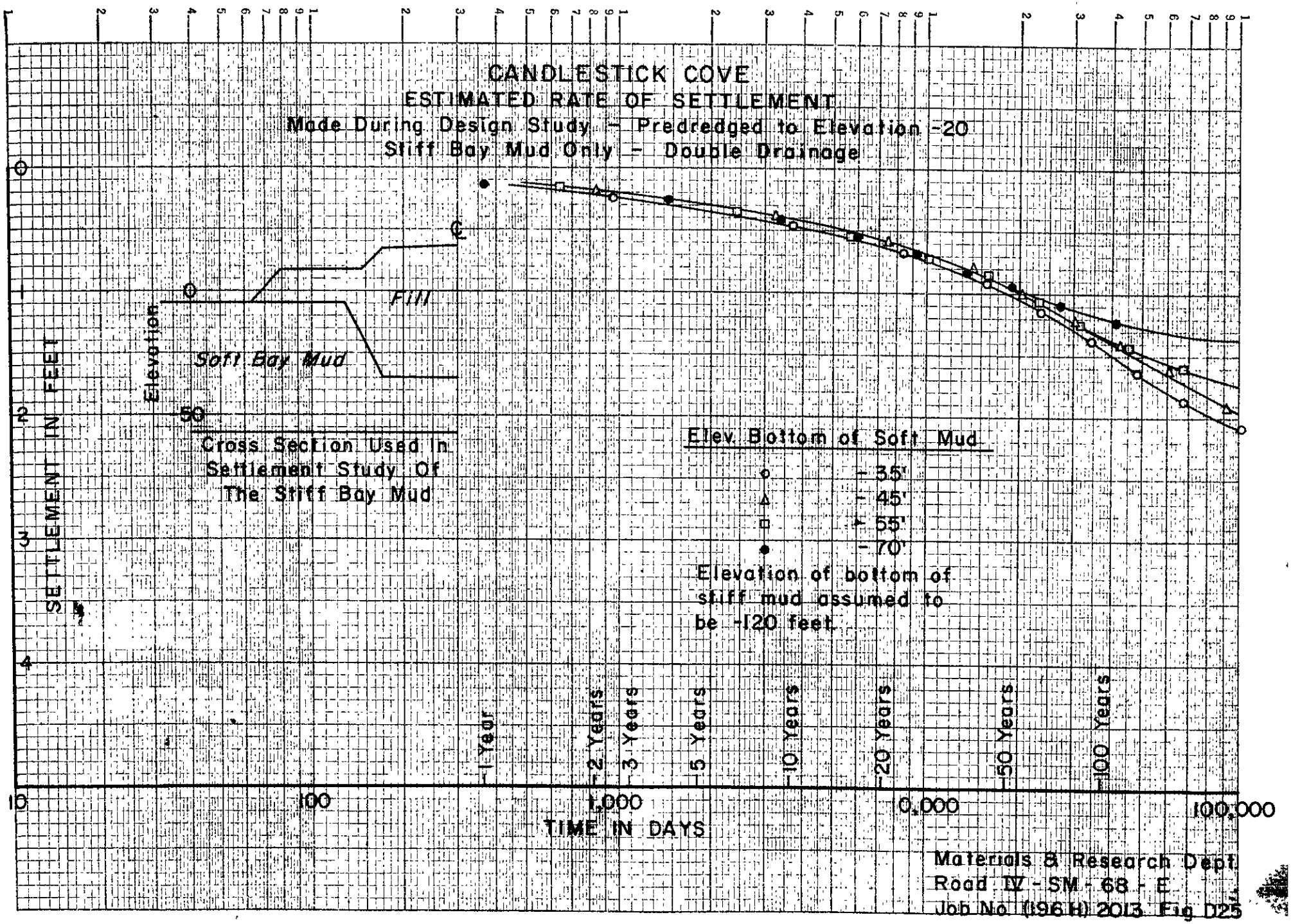

CANDLESTICK COVE
TIME CONSOLIDATION CURVES
Stiff Bay Mud, Before Construction
Testing Program - Job 196H
SAMPLE
LOADING, TONS
B4 - 6 - II 1/2 to 1
B7 - 9 - I 1/2 to 1
B8 - 12 - I 1 to 2
B10 - 10 - IV 1/4 to 1/2
PER CENT CONSOLIDATION
0
20
40
60
80
0.1
1.0
10
100
1,000
10,000
TIME IN MINUTES
Materials & Research Dept.
Road IV - SM - 68 - E
Job No. (196 H) 2013 Fig D23

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



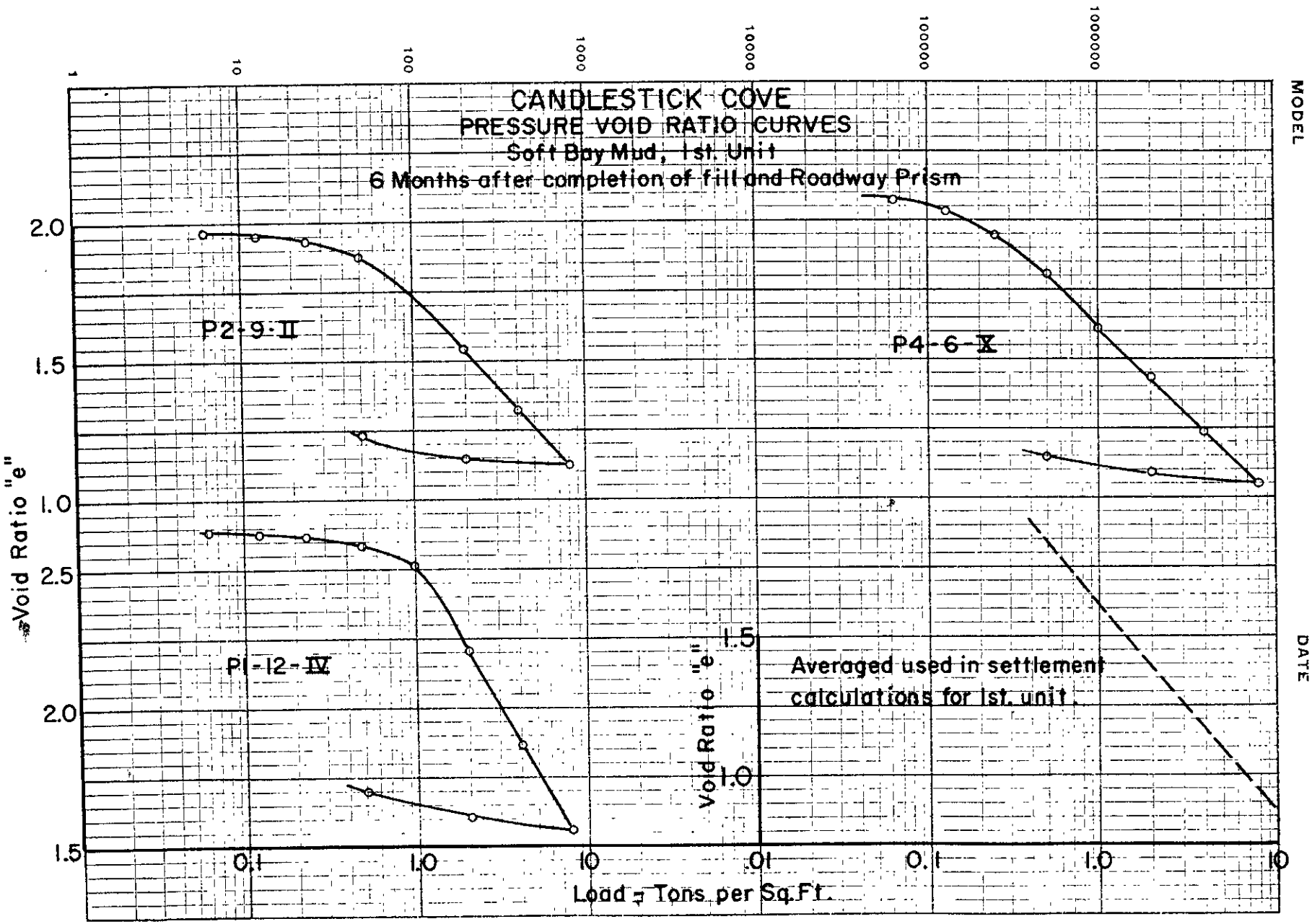
Materials & Research Dept.
Road IV - SM - 68 - E
Job No. (196H) 2013 Fig. D24

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

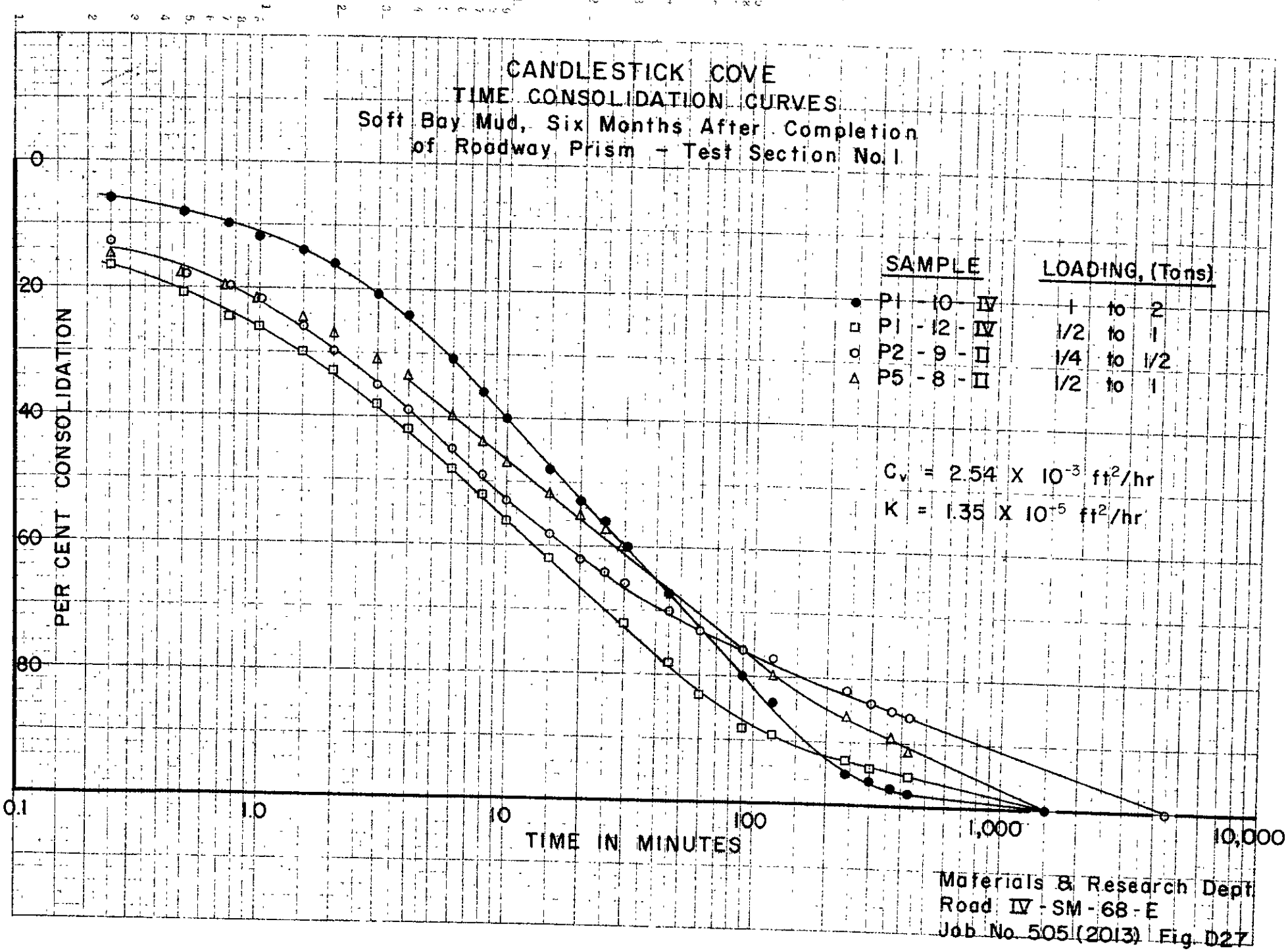


Materials & Research Dept.
Road IV - SM - 68 - E
Job No (1961) 2013 Fig D25

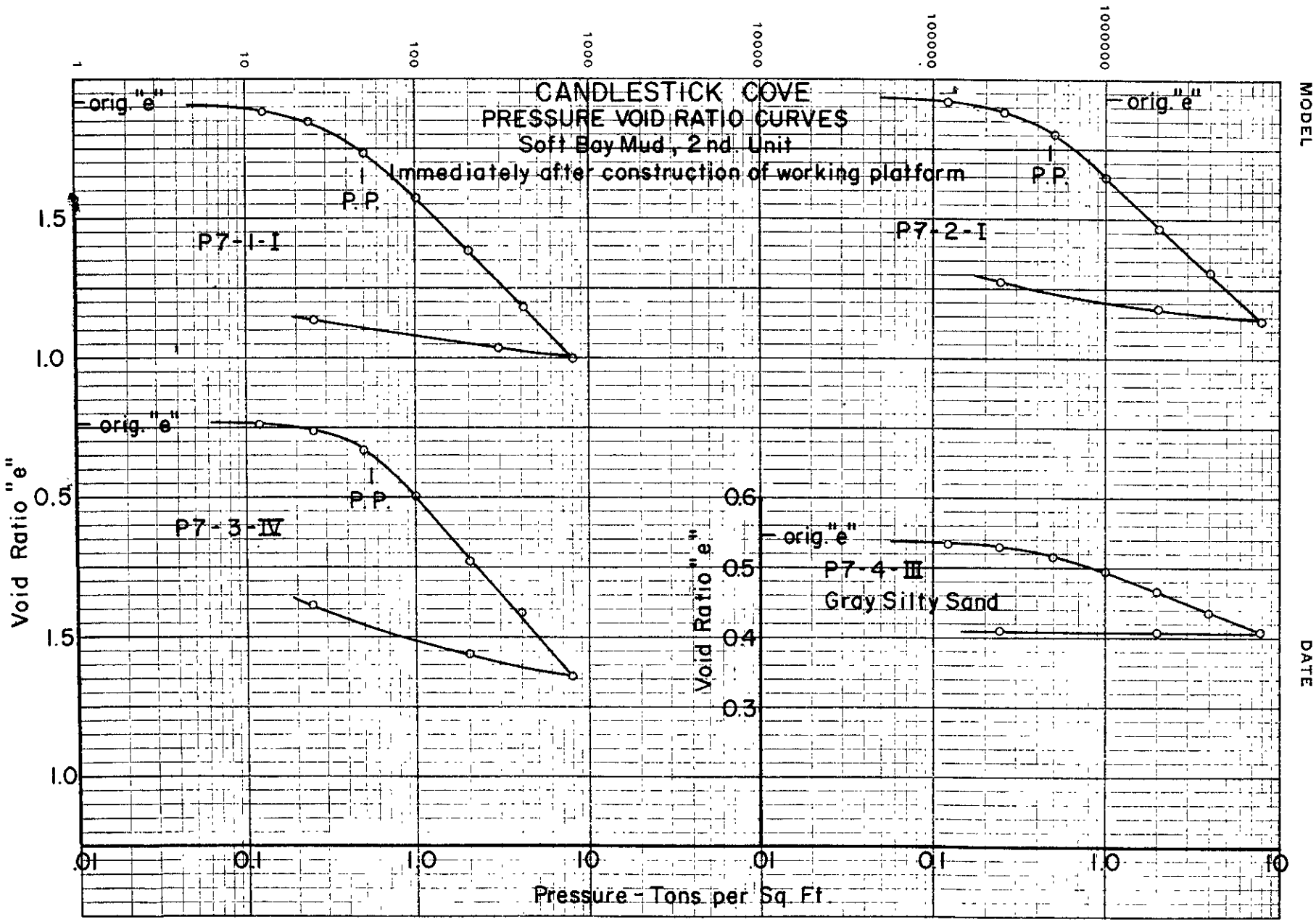
359-96 KEUFFEL & ESSER CO
Semi-Logarithmic, 7 Cycles X 5 to the ½ inch.
MADE IN U. S. A.



Materials & Research Dept.
Road IV- SM- 68- E
Job No. (505) 2013 Fig. D26

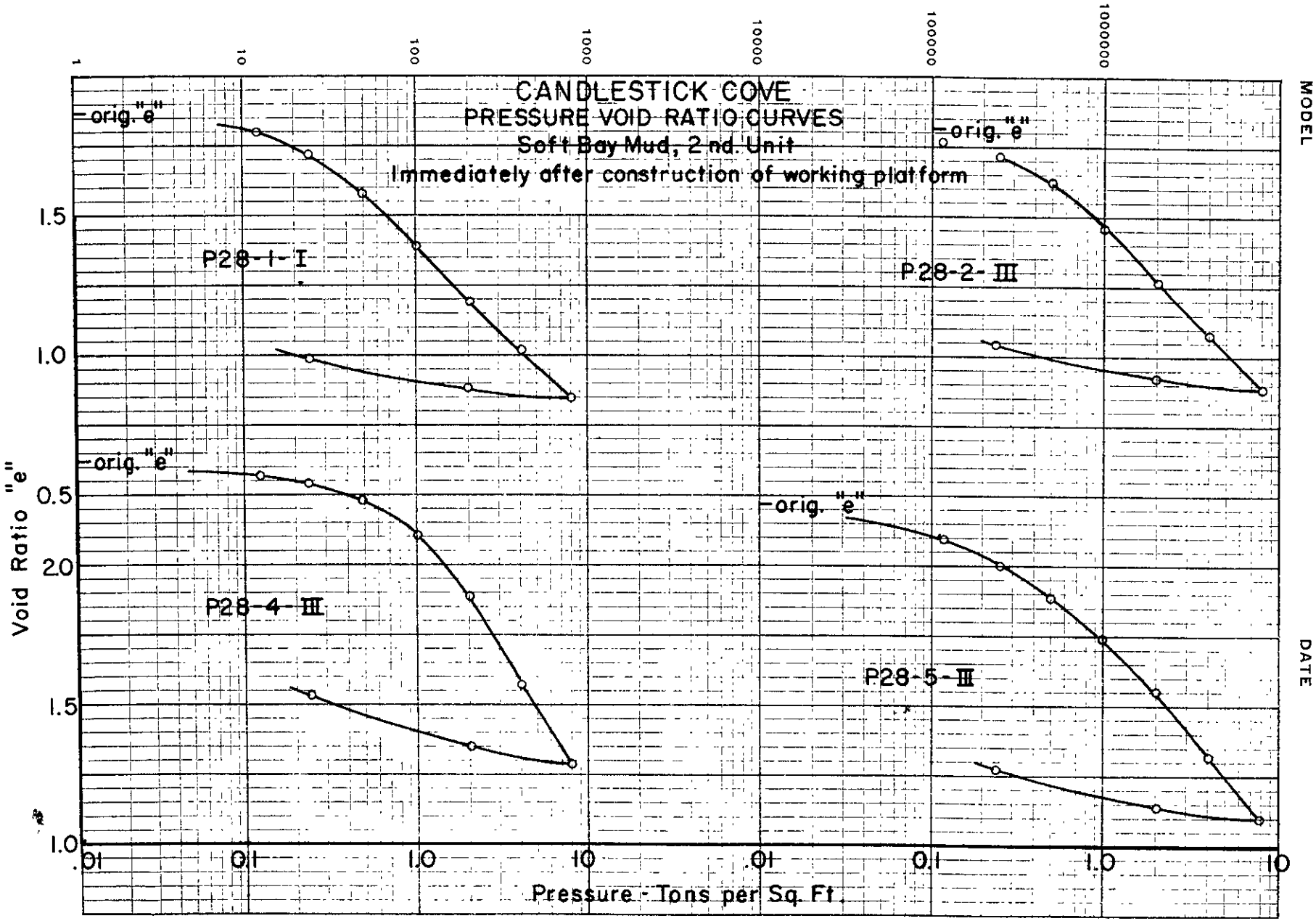

CANDLESTICK COVE
TIME CONSOLIDATION CURVES
Soft Bay Mud, Six Months After Completion
of Roadway Prism - Test Section No. 1
SAMPLE
LOADING, (Tons)
P1 - 10 - IV 1 to 2
P1 - 12 - IV 1/2 to 1
P2 - 9 - II 1/4 to 1/2
P5 - 8 - II 1/2 to 1
C_v = 2.54 X 10^{-3} ft^2/hr
K = 1.35 X 10^{-5} ft^2/hr
PER CENT CONSOLIDATION
0
20
40
60
80
0.1
1.0
10
100
1,000
10,000
TIME IN MINUTES
Materials & Research Dept
Road IV - SM - 68 - E
Job No. 505 (2013) Fig. D27

359-96 KEUFFEL & ESSER CO
Semi-Logarithmic, 7 Cycles X 5 to the ½ inch.
MADE IN U S A.



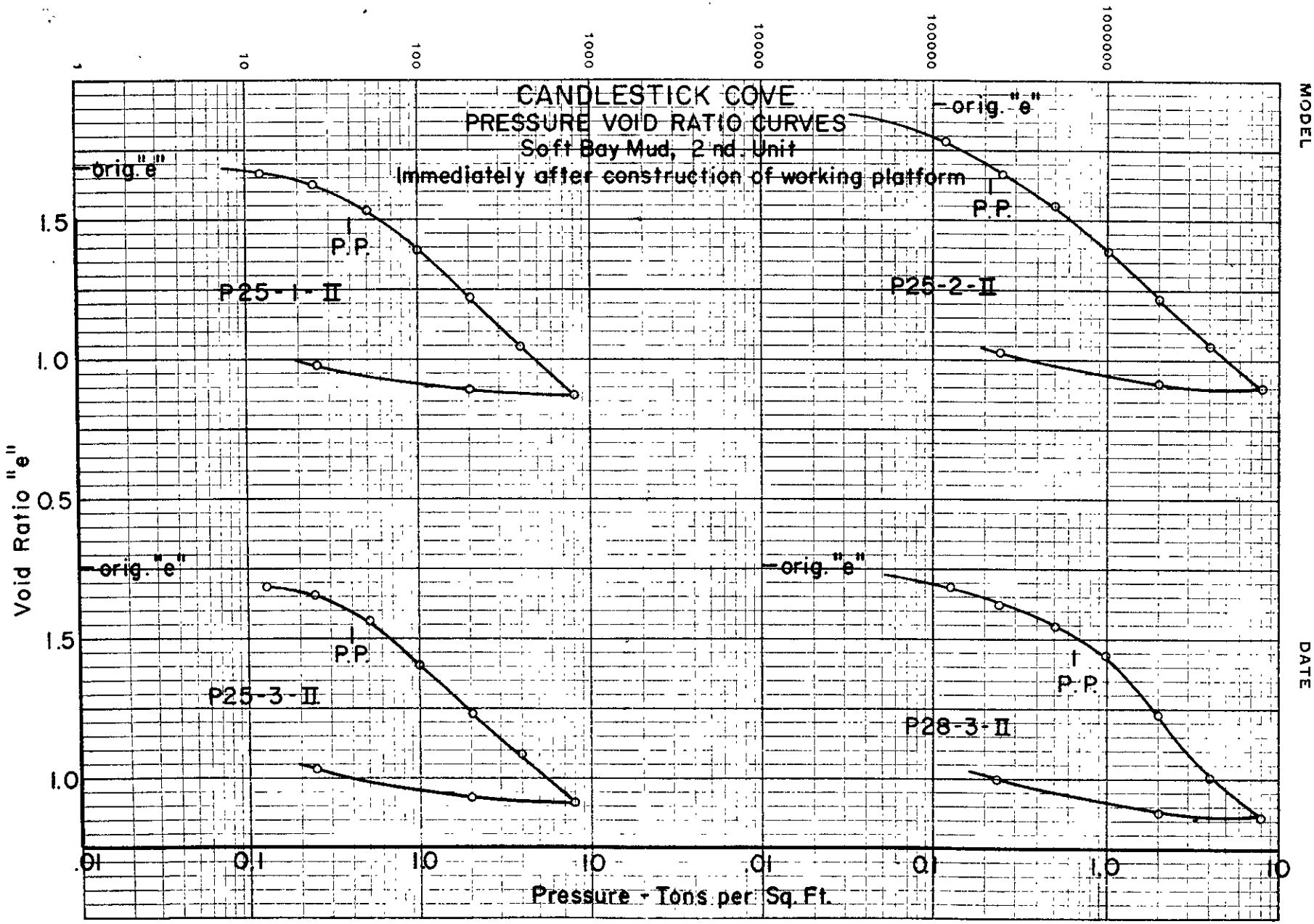
Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. D 28

359-96 KEUFFEL & ESSER CO
Semi-Logarithmic 7 Cycles X 5 to the ½ inch
MADE IN U. S. A



Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. D 29

359-96 KEUFFEL & ESSER CO.
Semi-Logarithmic, 7 Cycles X 5 to the ½ inch
MADE IN U. S. A.

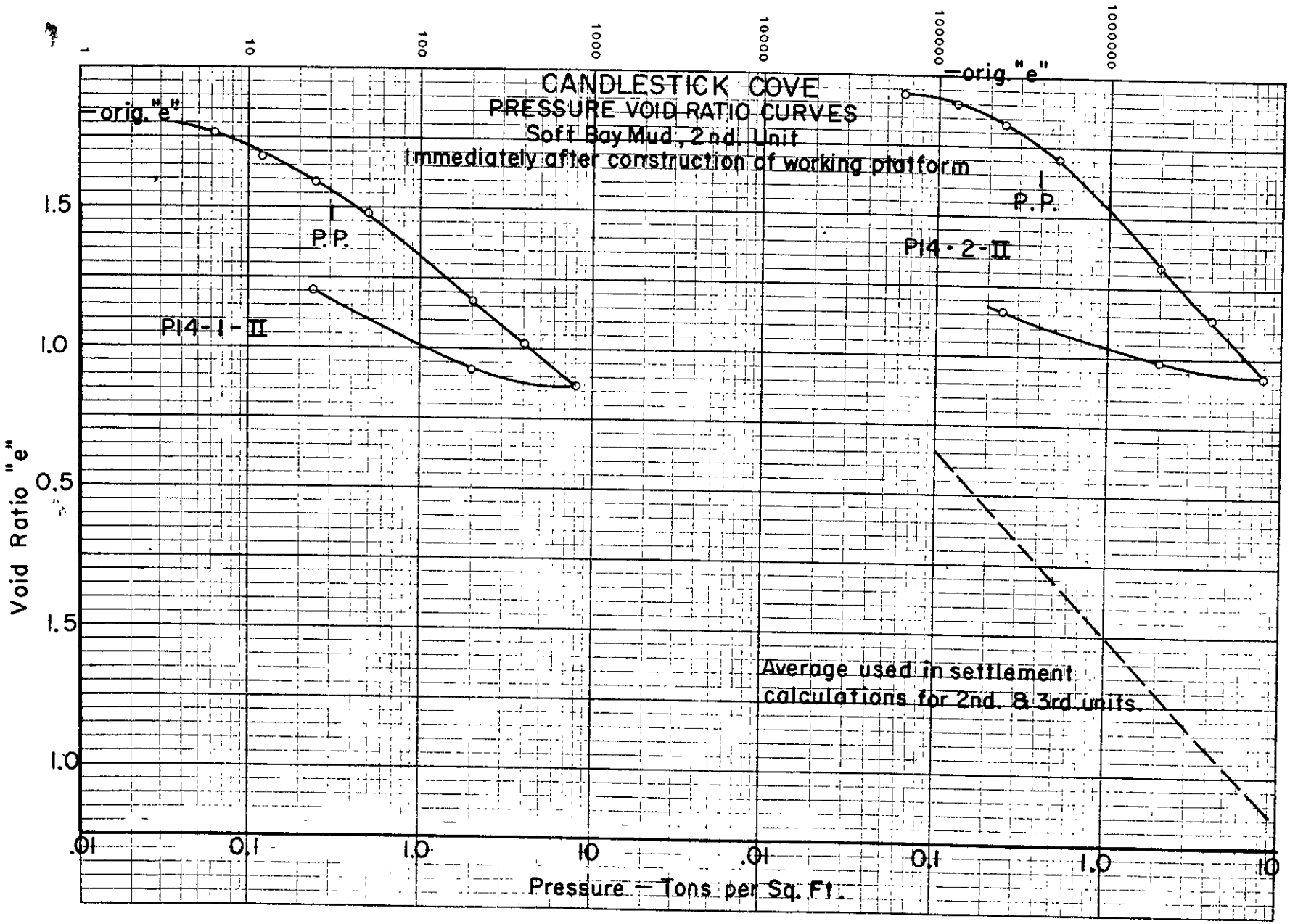


MODEL

DATE

Materials & Research Dept.
Road IV - SM-68-E
Job No. 2013-R-21 Fig. D30

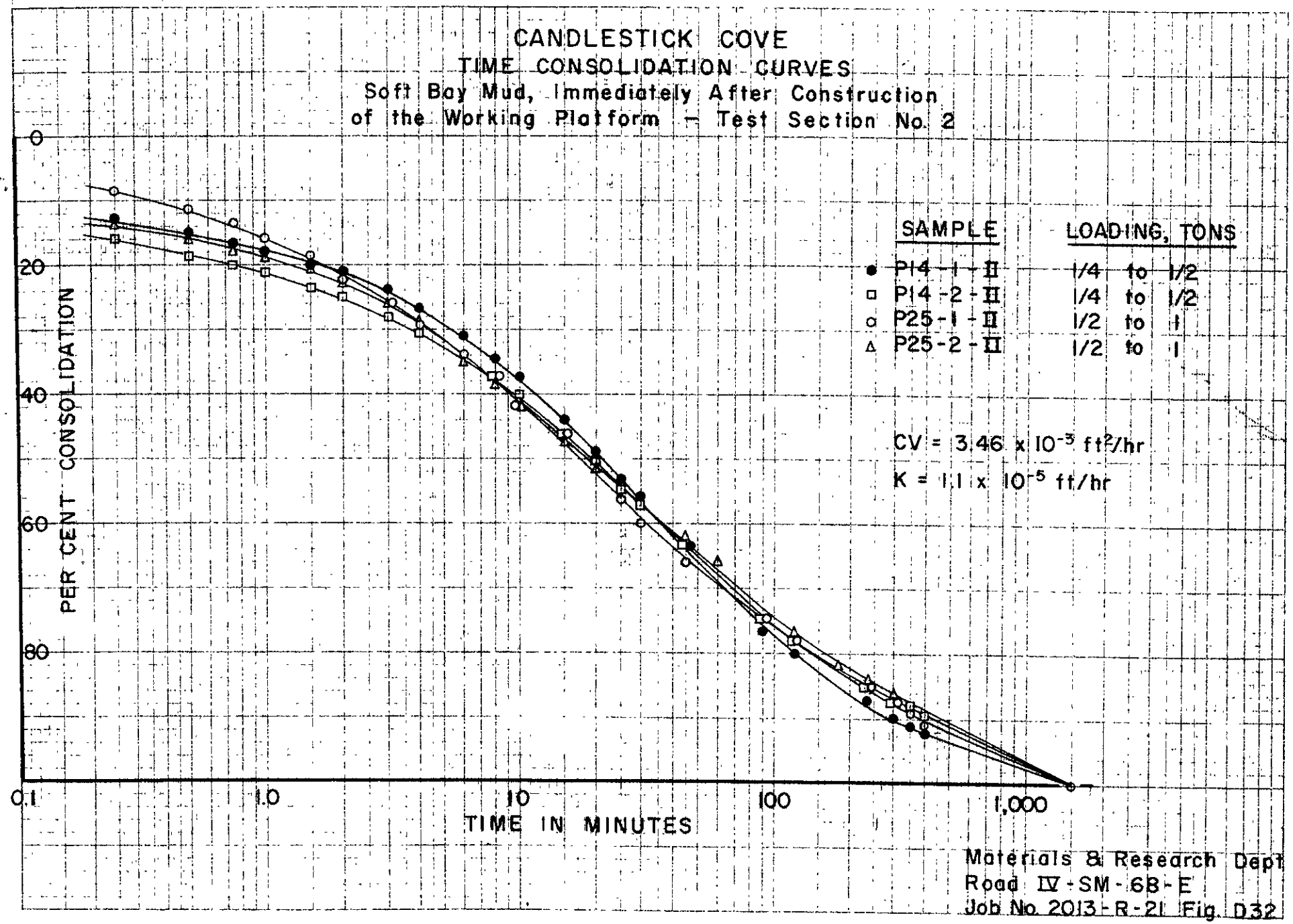
359-96 KEUFFEL & ESSER CO
Semi-Logarithmic, 7 Cycles X 5 to the ½ inch
MADE IN U. S. A.

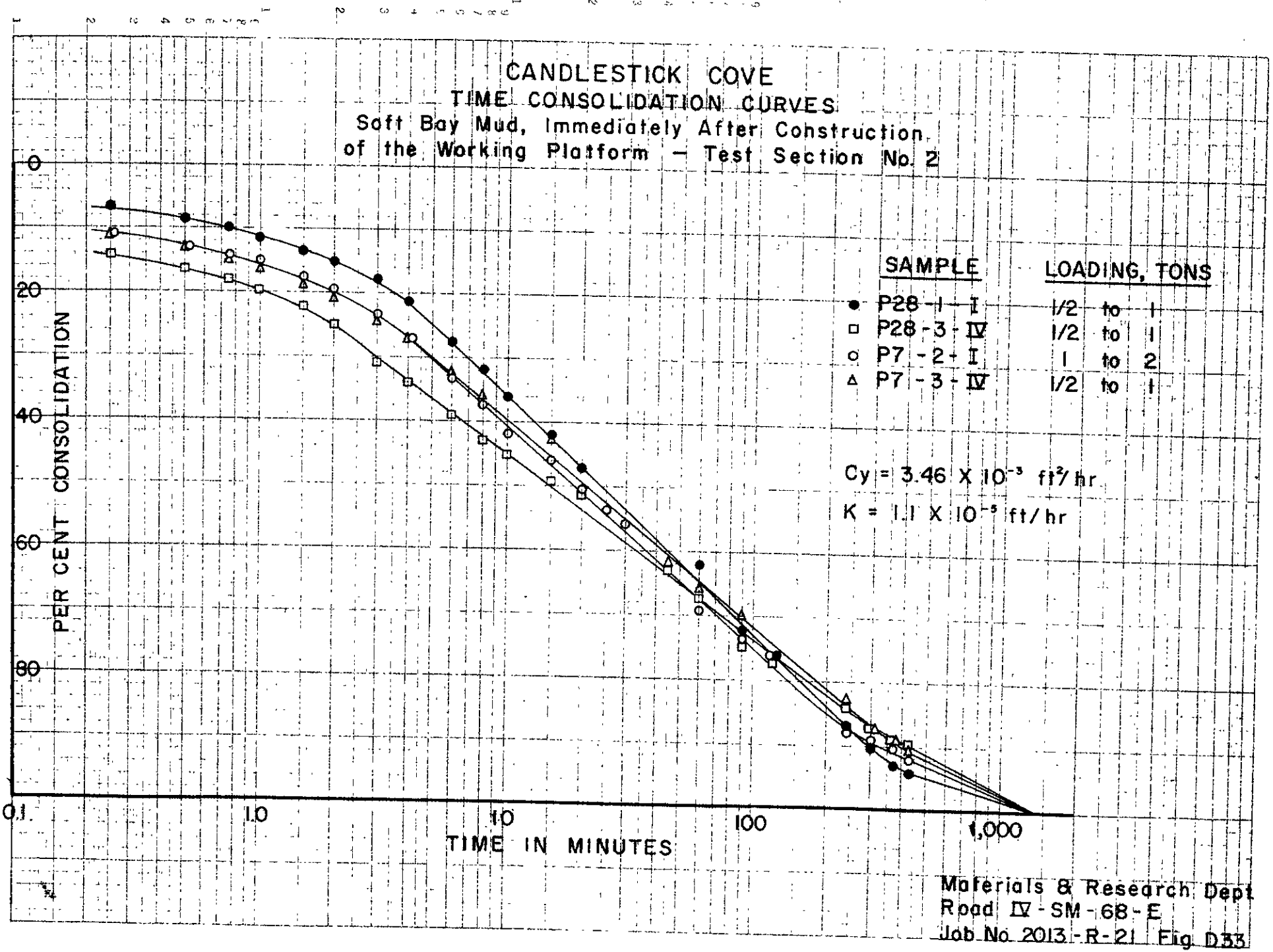


MODEL

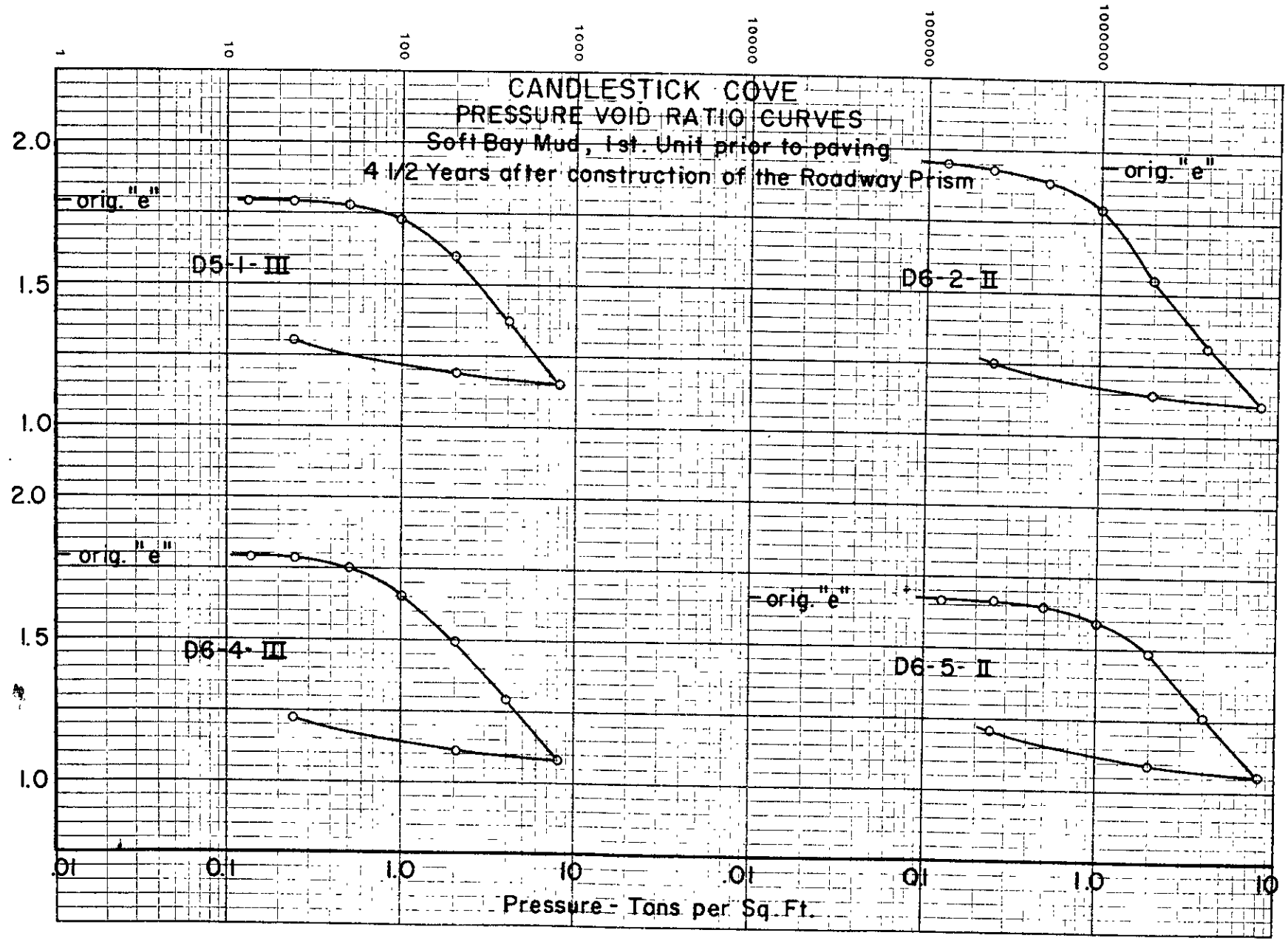
DATE

Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. D 31


CANDLESTICK COVE
TIME CONSOLIDATION CURVES
Soft Bay Mud, Immediately After Construction
of the Working Platform — Test Section No. 2
SAMPLE
LOADING, TONS
P14-1-II 1/4 to 1/2
P14-2-II 1/4 to 1/2
P25-1-II 1/2 to 1
P25-2-II 1/2 to 1
CV = 3.46 x 10^-3 ft²/hr
K = 1.1 x 10^-5 ft/hr
PER CENT CONSOLIDATION
0
20
40
60
80
0.1
1.0
10
100
1,000
TIME IN MINUTES
Materials & Research Dept
Road IV-SM-68-E
Job No. 2013-R-21 Fig. D32


CANDLESTICK COVE
TIME CONSOLIDATION CURVES
Soft Bay Mud, Immediately After Construction of the Working Platform — Test Section No. 2
SAMPLE
LOADING, TONS
P28-1-I 1/2 to 1
P28-3-IV 1/2 to 1
P7-2-I 1 to 2
P7-3-IV 1/2 to 1
Cy = 3.46 X 10⁻³ ft²/hr
K = 1.1 X 10⁻⁵ ft/hr
PER CENT CONSOLIDATION
0
20
40
60
80
0.1
1.0
10
100
1,000
TIME IN MINUTES
Materials & Research Dept
Road IV-SM-68-E
Job No. 2013-R-21 Fig. D33

359-96 KEUFFEL & ESSER CO
Semi Logarithmic, 7 Cycles X 5 to the ½ inch
MADE IN U. S. A.

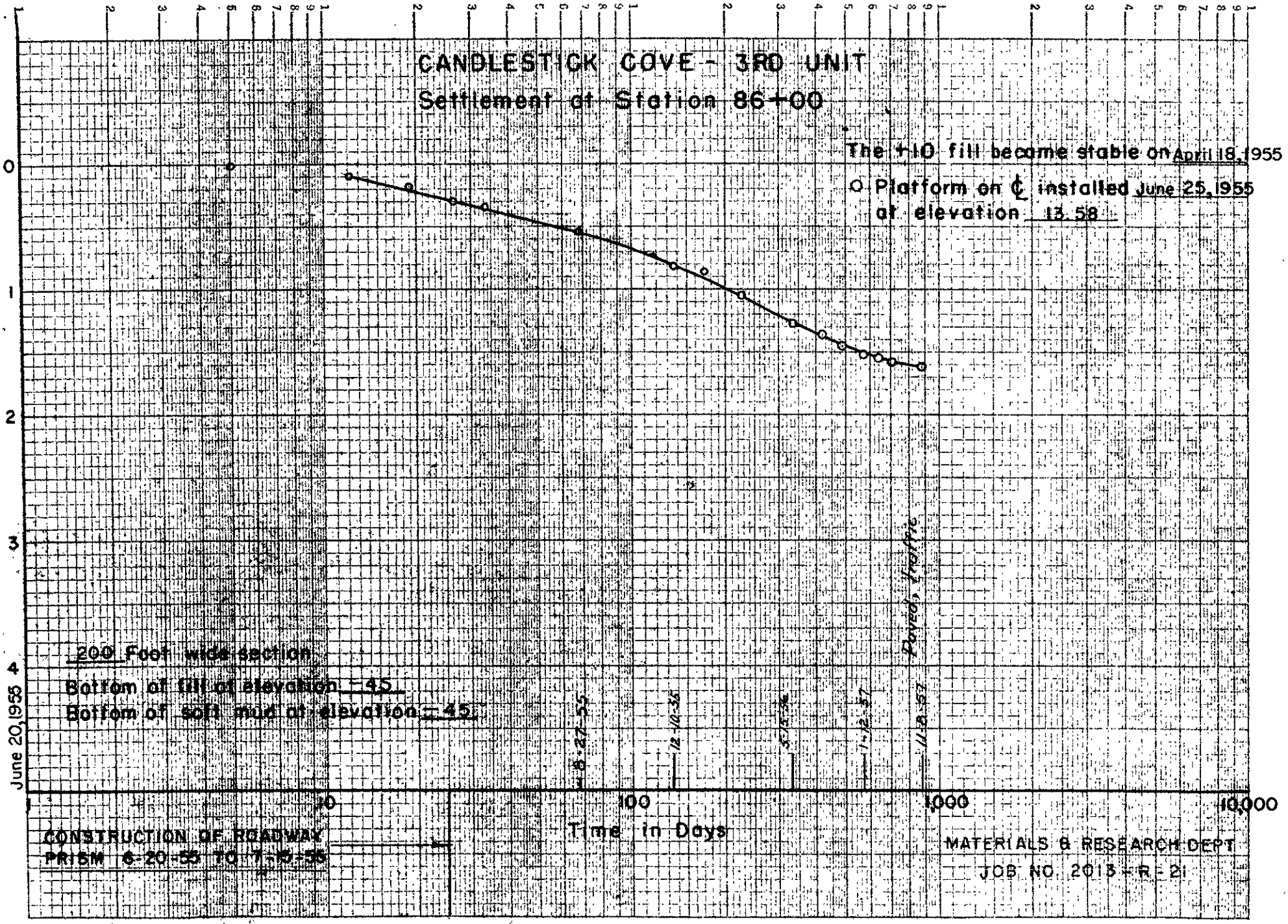


MODEL

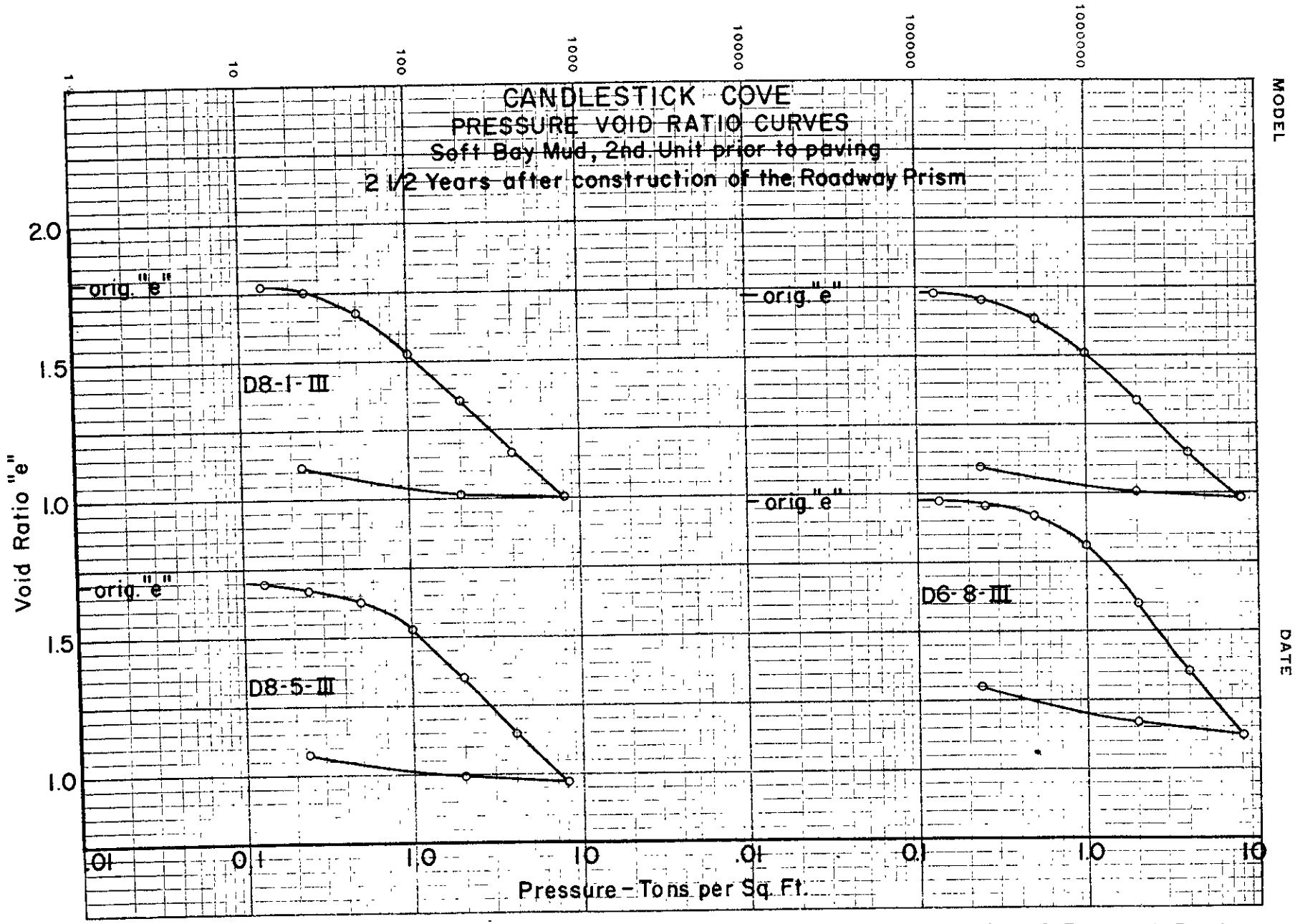
DATE

Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. D34

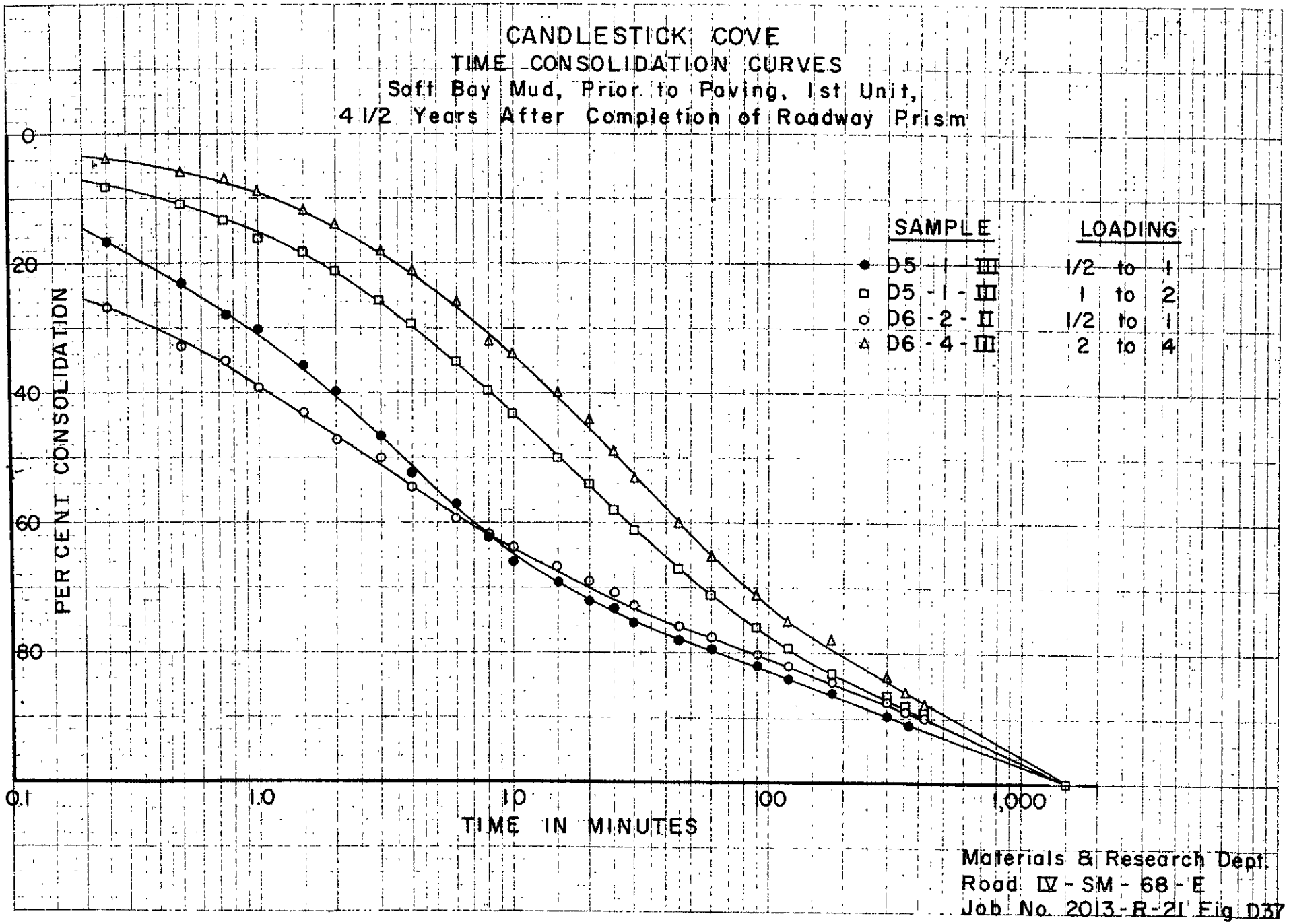
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



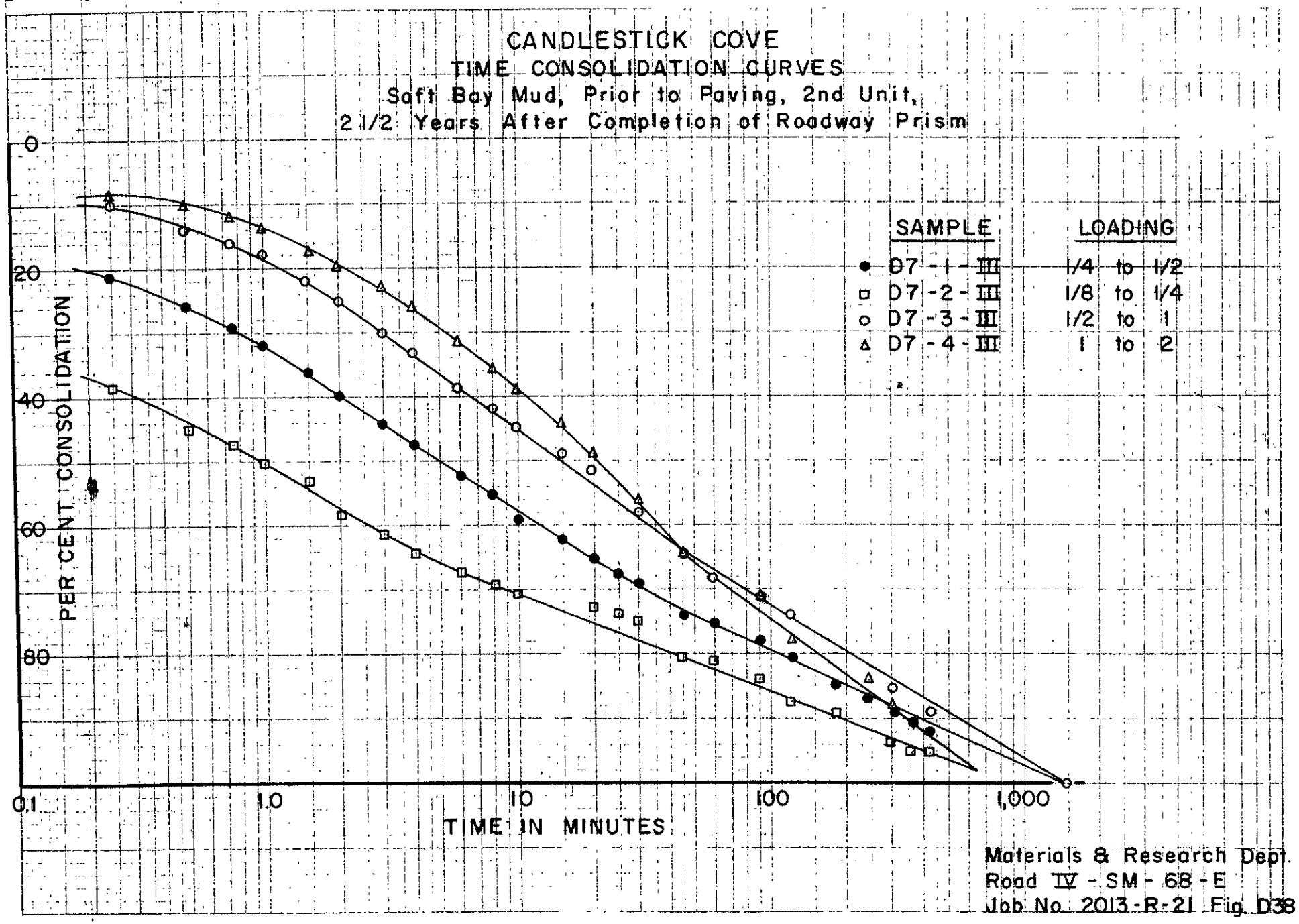
359 96 KEUFFEL & ESSER CO
Semi-Logarithmic, 7 Cycles X 5 to the ½ inch.
MADE IN U. S. A.



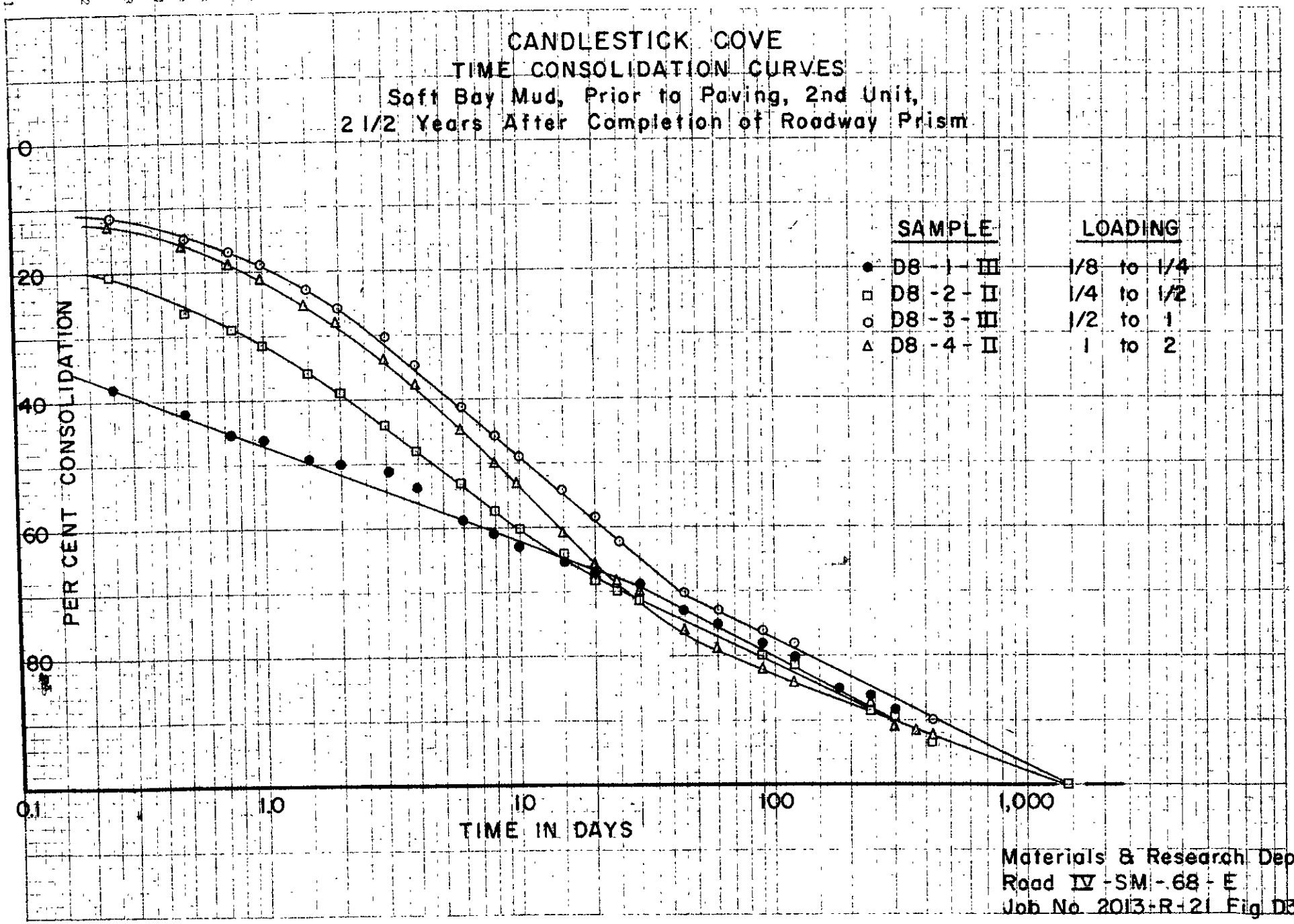
Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. D36



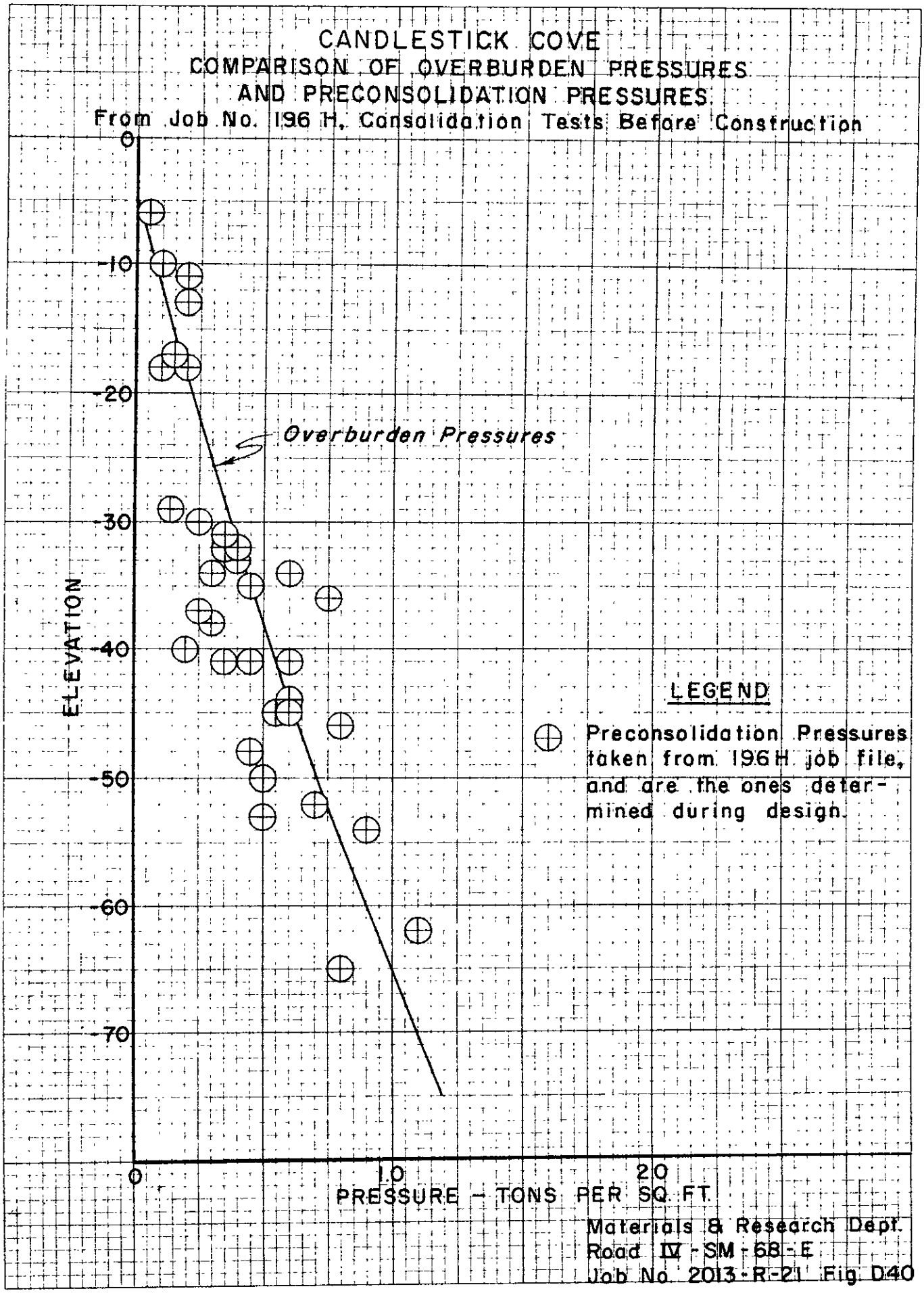
Materials & Research Dept.
Road IV - SM - 68 - E
Job No. 2013-R-21 Fig D37



Materials & Research Dept.
Road IV - SM - 68 - E
Job No. 2013-R-21 Fig. D38

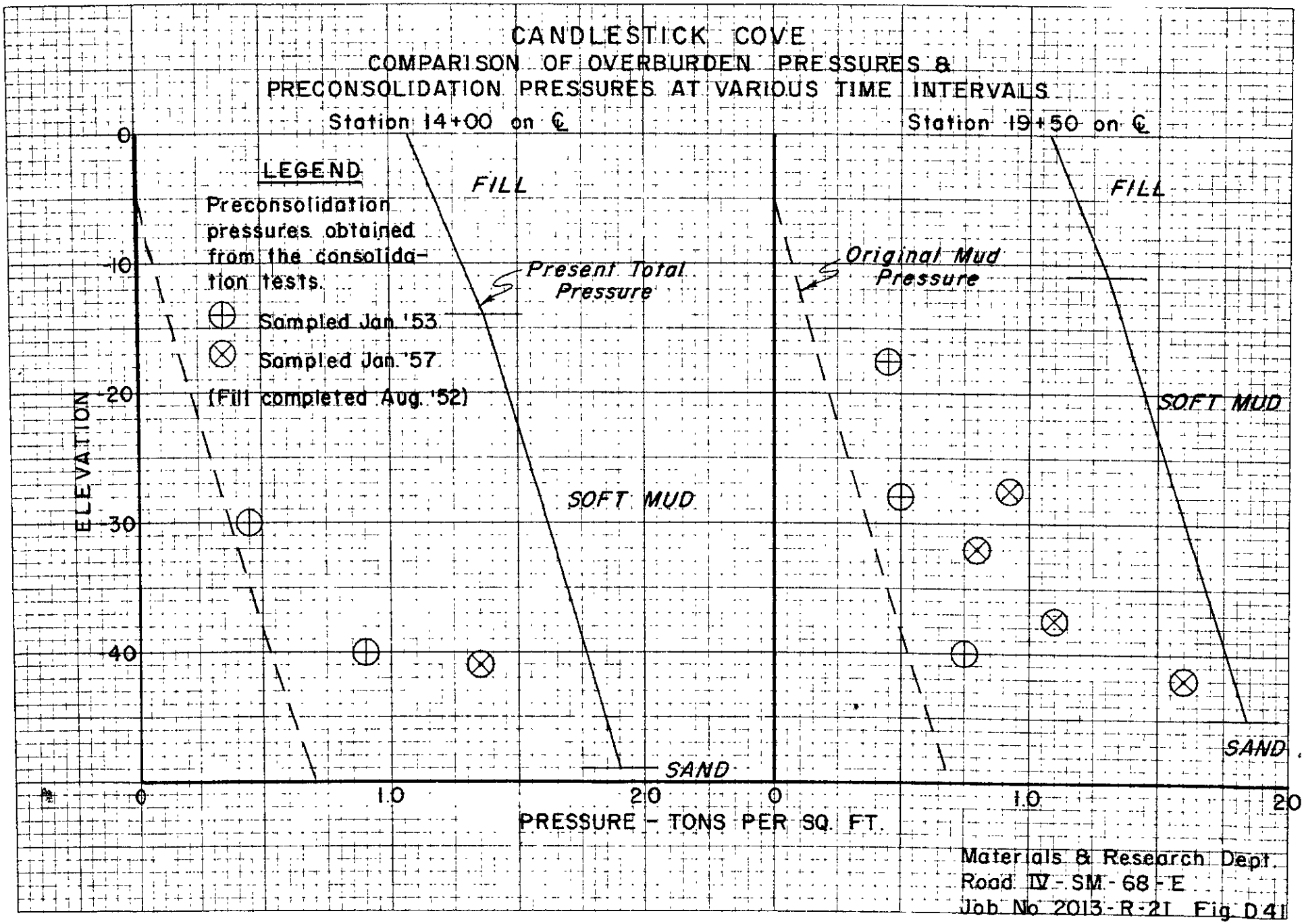


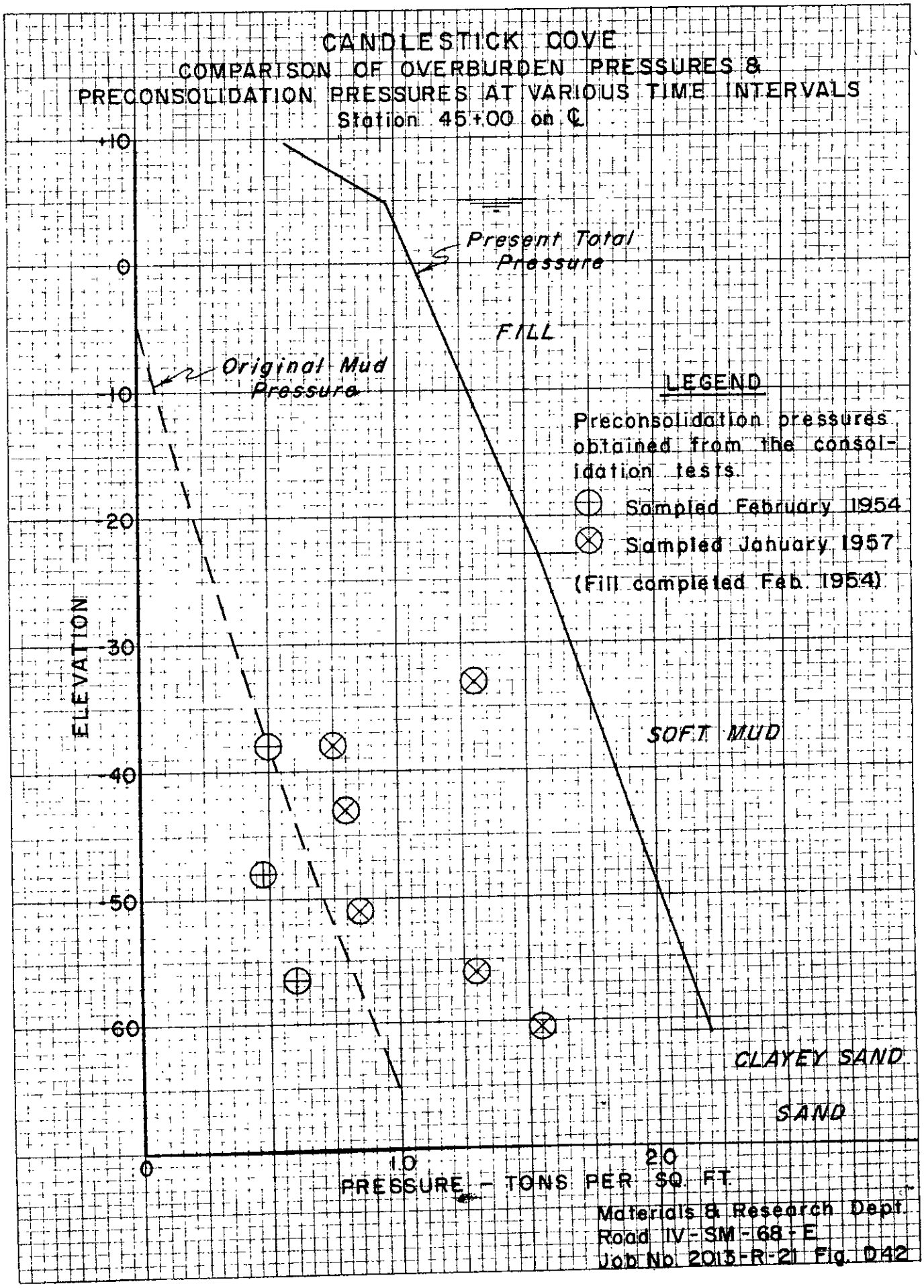
Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. D39



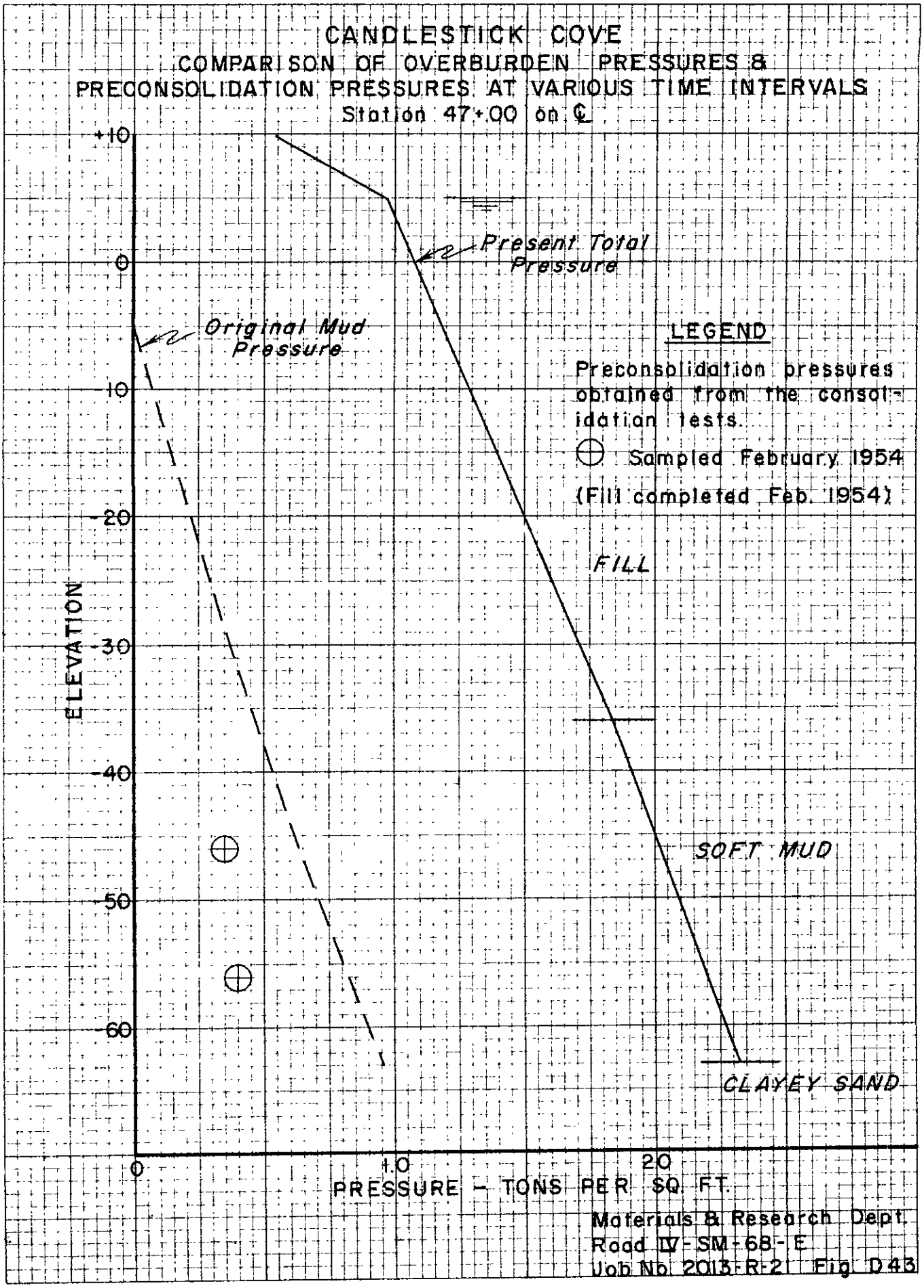
NO. 340R-10 DIETZGEN GRAPH PAPER 10 X 10 PER INCH
EUGENE DIETZGEN CO. MADE IN U.S.A.

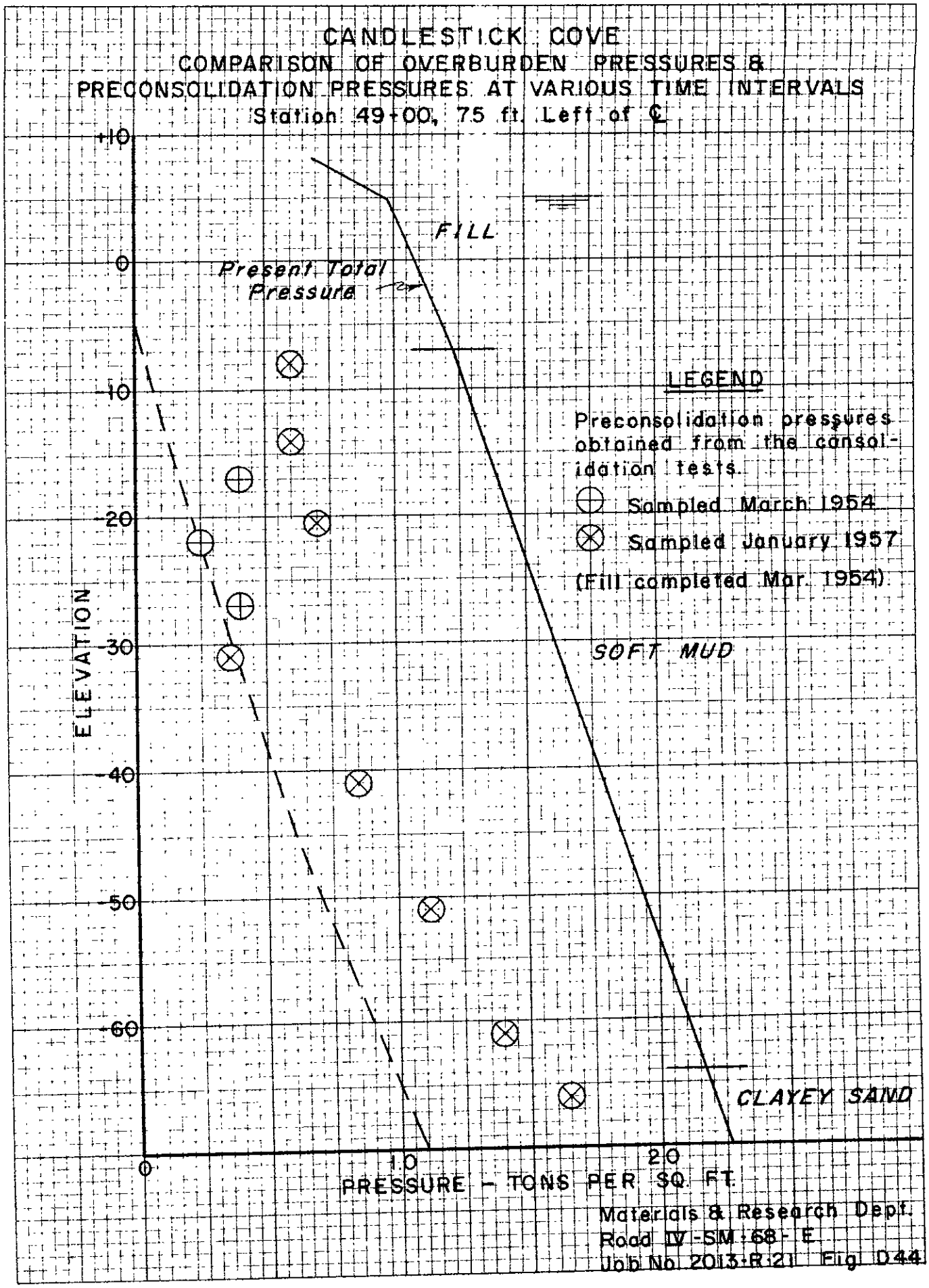
NO 340R-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH
EUGENE DIETZGEN CO
MADE IN U.S.A.

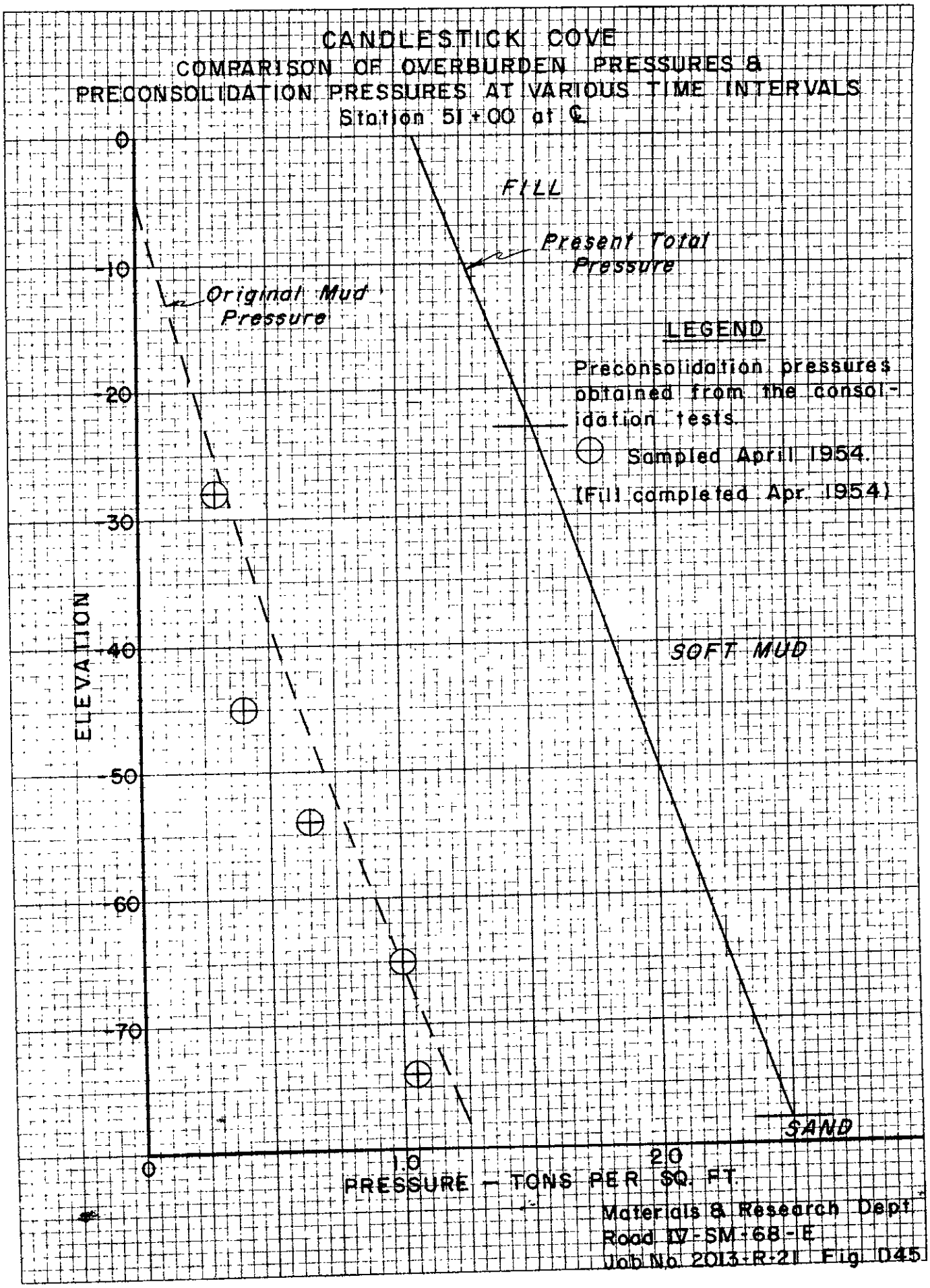




EUGENE DIETZGEN CO
NO 340R-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH


CANDLESTICK COVE
COMPARISON OF OVERBURDEN PRESSURES &
PRECONSOLIDATION PRESSURES AT VARIOUS TIME INTERVALS
Station 47+00 on ℄
+10
0
-10
-20
-30
-40
-50
-60
ELEVATION
Present Total Pressure
Original Mud Pressure
LEGEND
Preconsolidation pressures obtained from the consolidation tests.
Sampled February 1954
(Fill completed Feb. 1954)
FILL
SOFT MUD
CLAYEY SAND
0
1.0
2.0
PRESSURE – TONS PER SQ. FT.
Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-2 Fig. D43


CANDLESTICK COVE
COMPARISON OF OVERBURDEN PRESSURES &
PRECONSOLIDATION PRESSURES AT VARIOUS TIME INTERVALS
Station 49+00, 75 ft. Left of ℄
+10
0
-10
-20
-30
-40
-50
-60
ELEVATION
FILL
Present Total Pressure
SOFT MUD
CLAYEY SAND
LEGEND
Preconsolidation pressures obtained from the consolidation tests.
Sampled March 1954
Sampled January 1957
(Fill completed Mar. 1954)
0
1.0
2.0
PRESSURE - TONS PER SQ. FT.
Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. D44

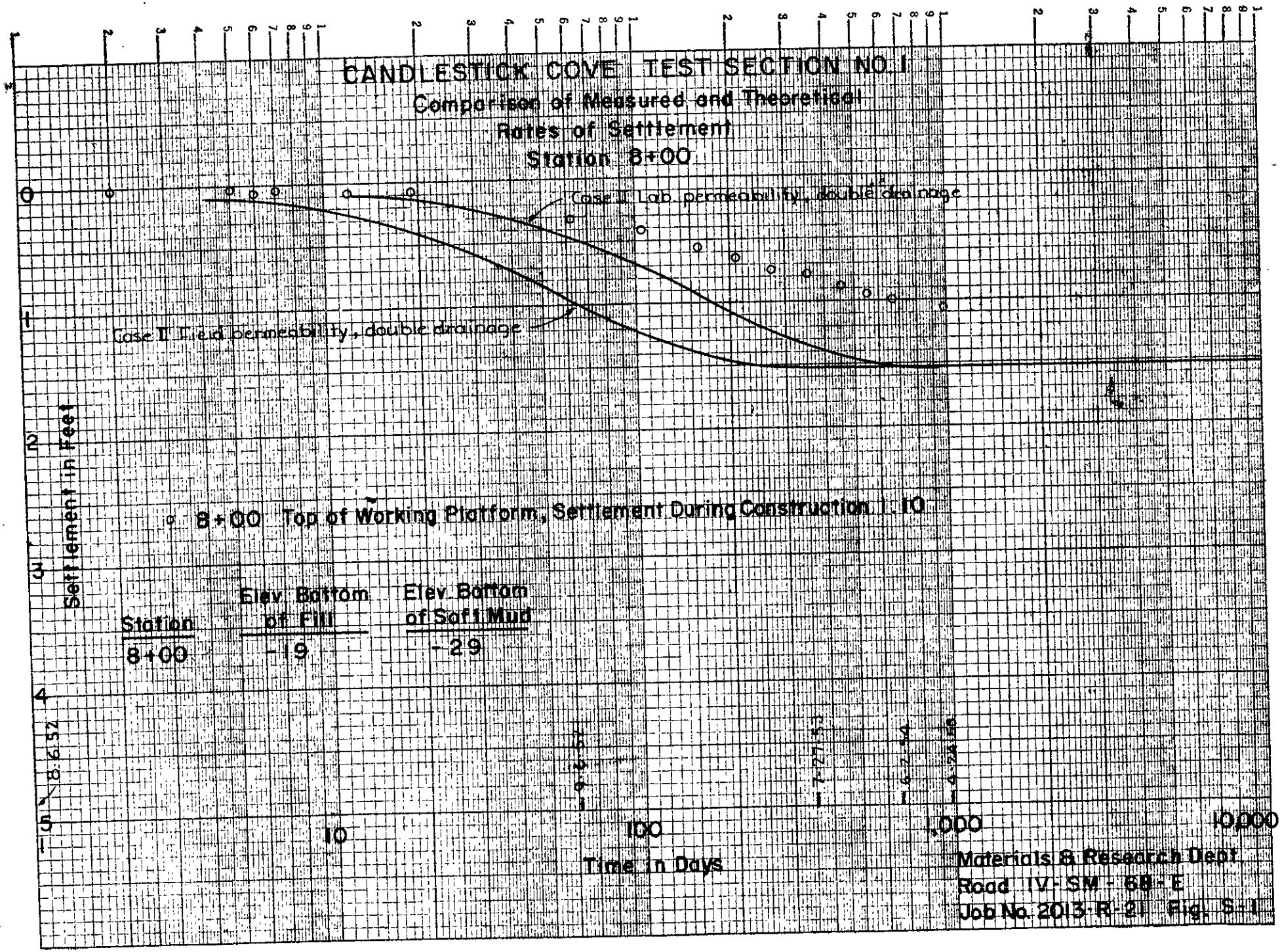


EUGENE DIETZGEN CO.
NO. 340R-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

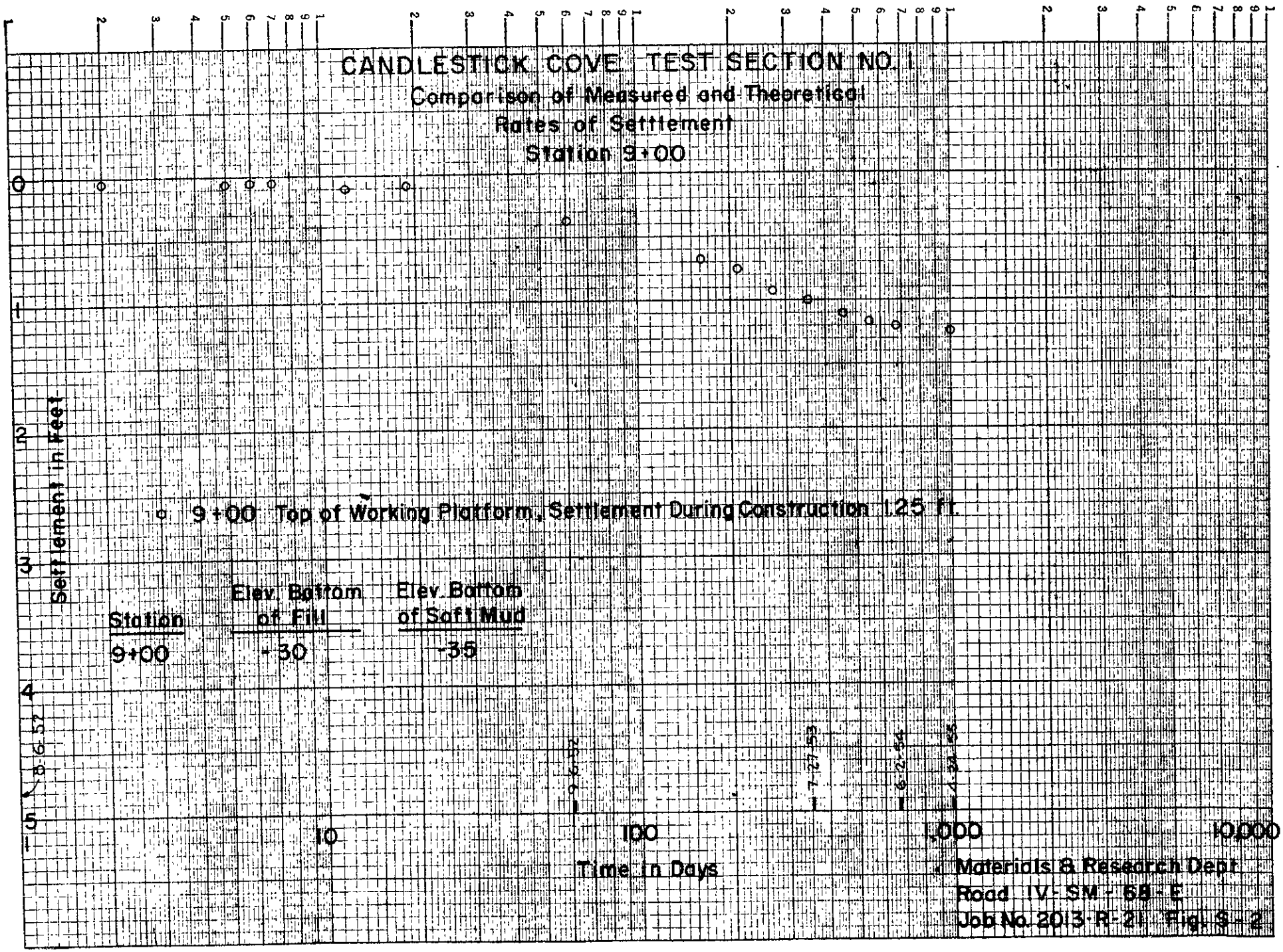
SETTLEMENT PLOTS

Item	Plate
Comparison of Measured and Theoretical Settlements, for the Soft Bay Mud	S-1 to S-18, S-25 to S-34
Settlements of the Various Layers Underlying the Roadway Prism	S-19 to S-24, S-39
Comparison of Measured and Theoretical Settlements, for the Stiff Bay Mud	S-35 to S-38
Measured Settlements of the Roadway Prism	
First Unit	S-40 to S-60
Second Unit	S-61 to S-87
Third Unit	S-88 to S-137
Fourth Unit	S-138 to S-158
Fifth Unit	S-159 to S-172
Settlement of the Stiff Mud Layer	S-173 to S-176

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

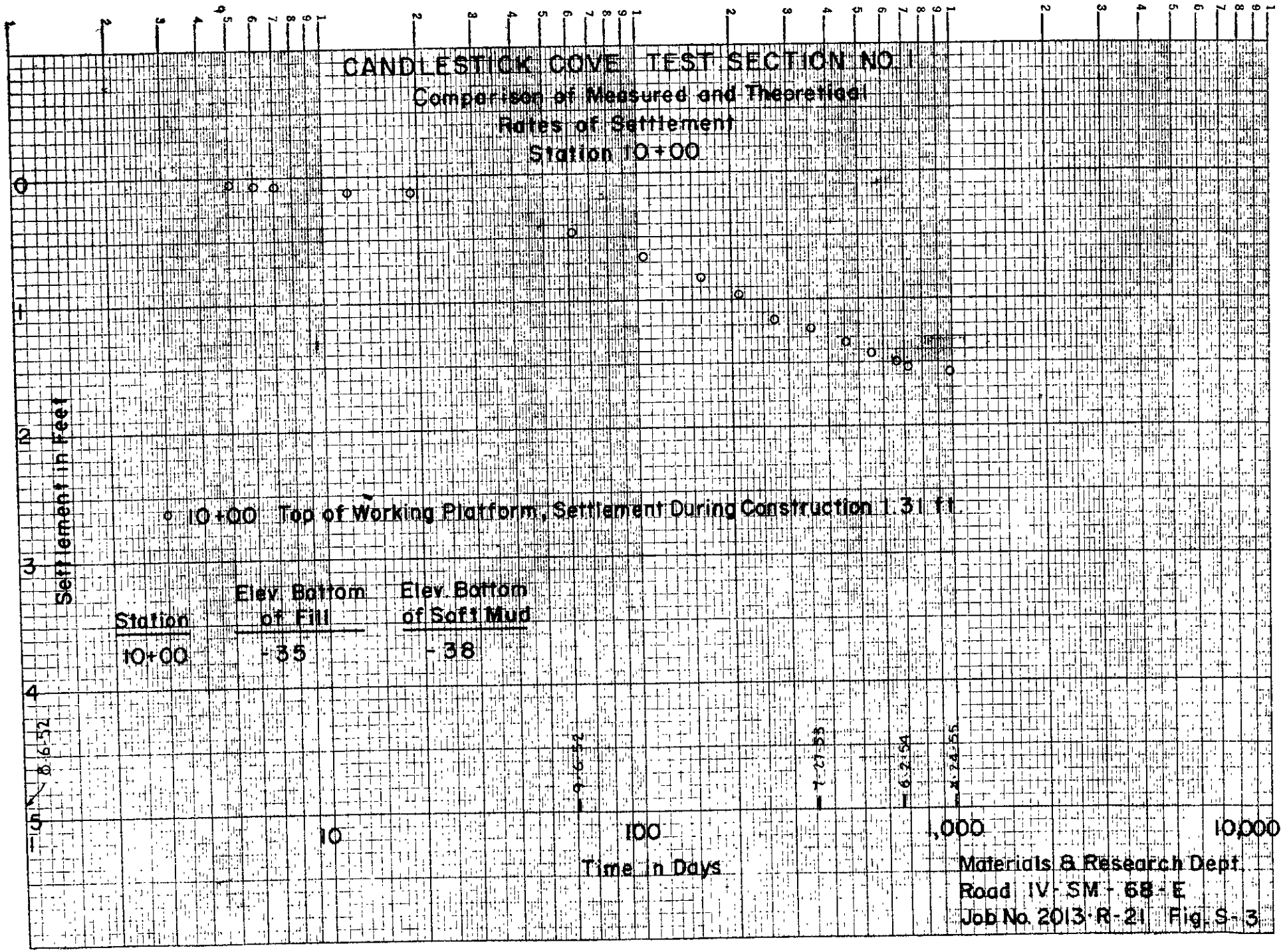


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

CANDLESTICK COVE TEST SECTION NO. 1

Comparison of Measured and Theoretical Rates of Settlement

Station 10+00



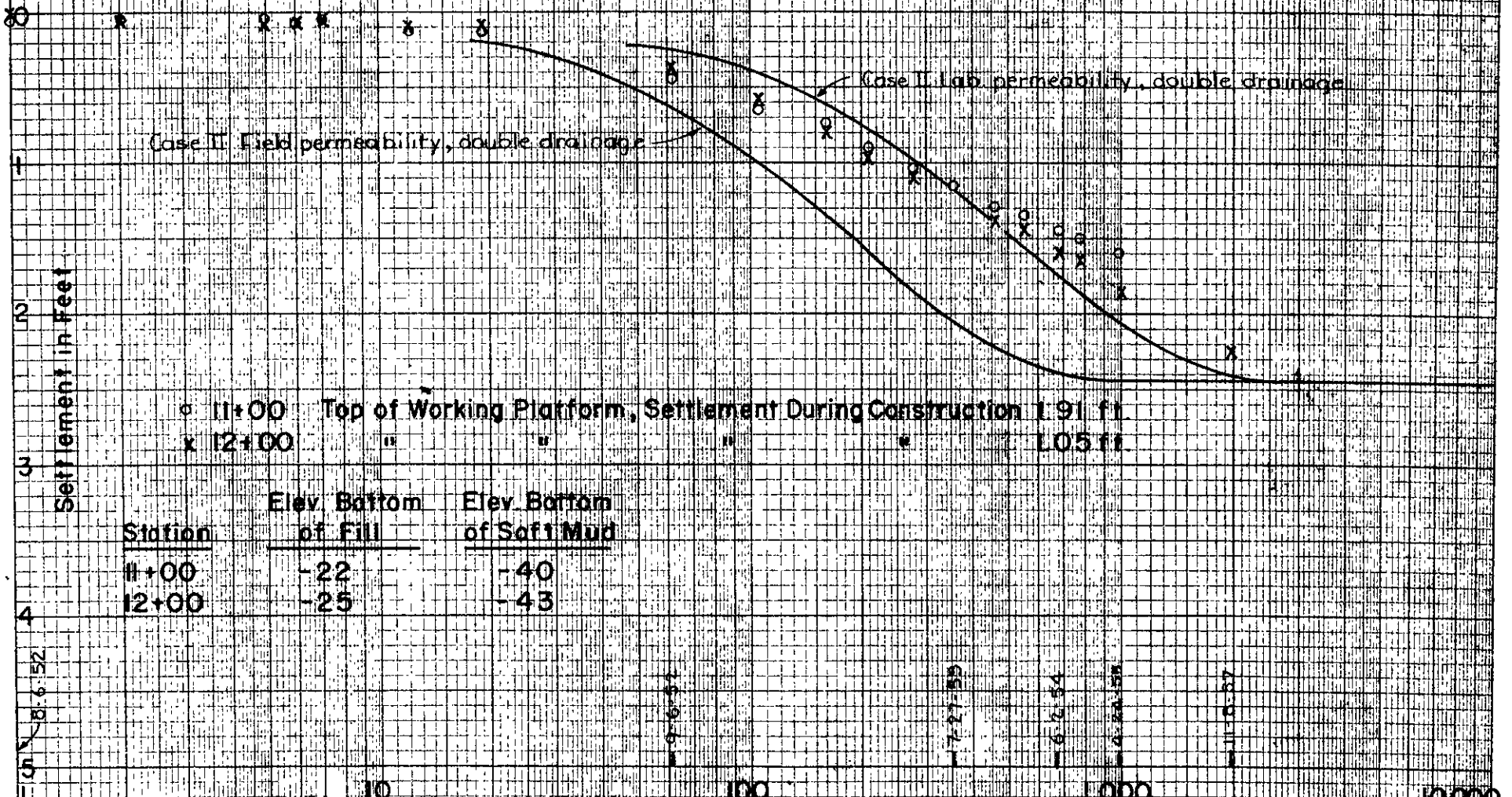
Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. S-3

K+E SEMI LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

CANDLESTICK COVE TEST SECTION NO. 1

Comparison of Measured and Theoretical Rates of Settlement

Station 11+00 & 12+00

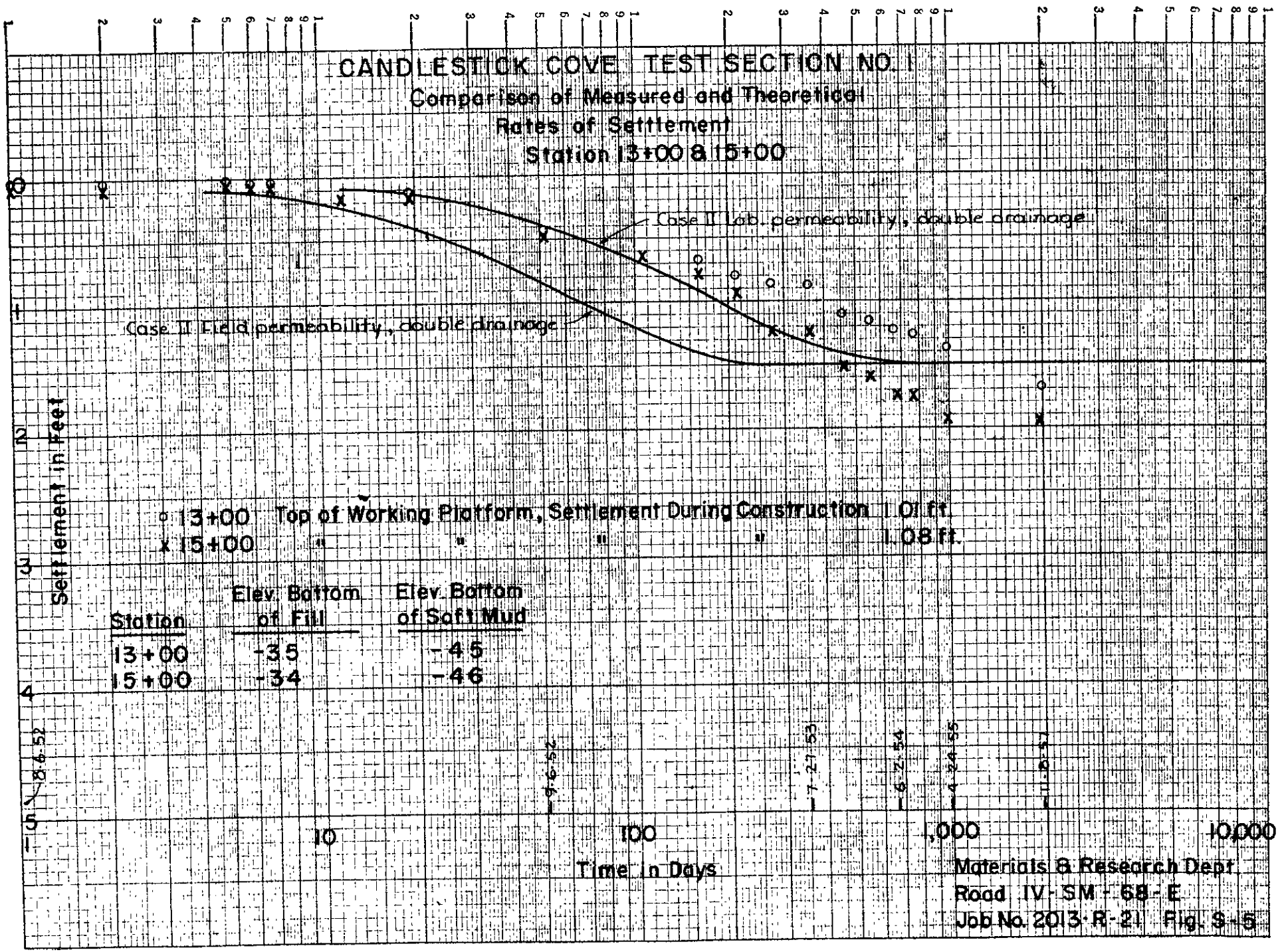


o 11+00 Top of Working Platform, Settlement During Construction 1.91 ft.
x 12+00 " " " " 1.05 ft.

Station	Elev. Bottom of Fill	Elev. Bottom of Soft Mud
11+00	-22	-40
12+00	-25	-43

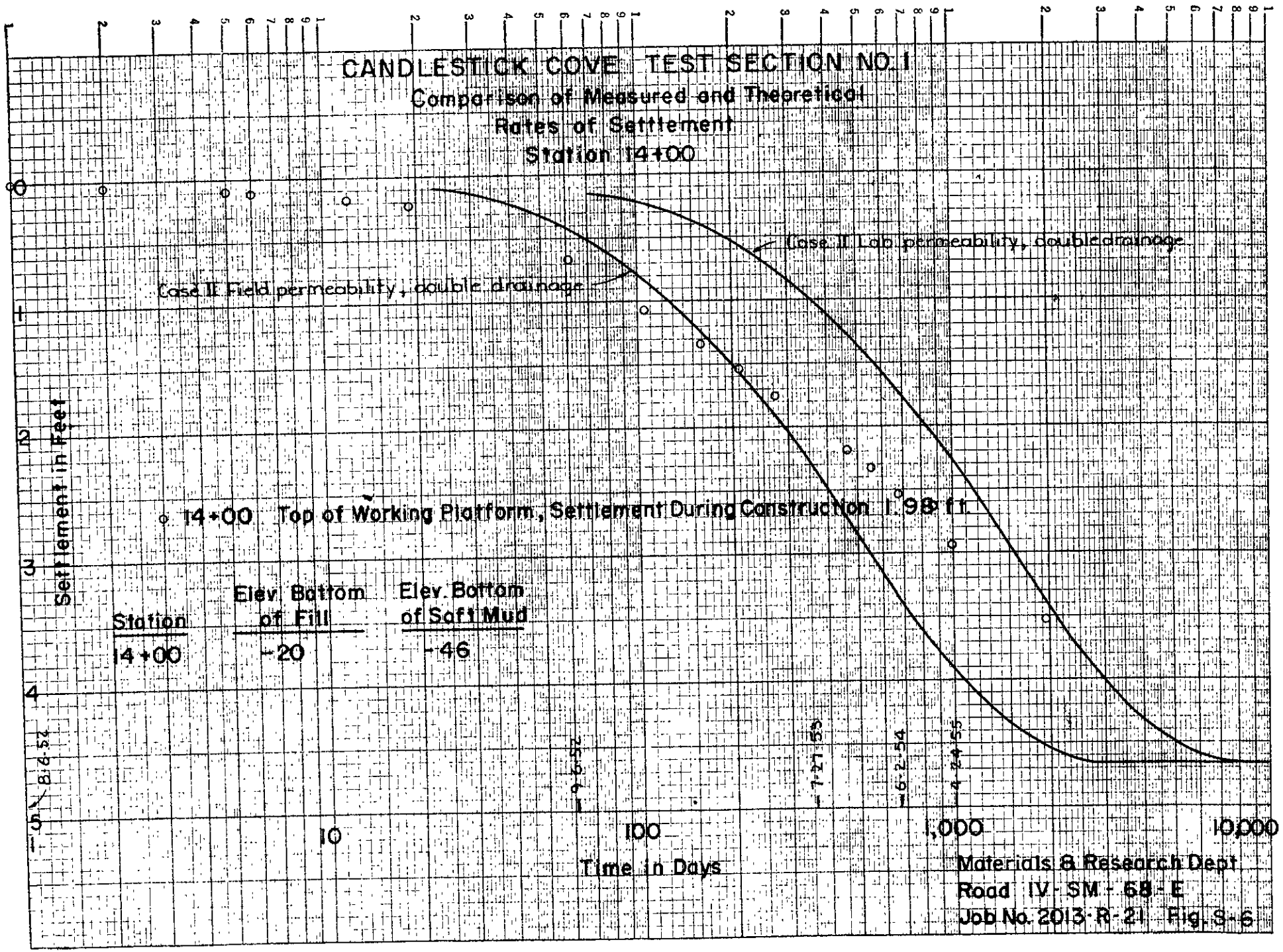
Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. S-4

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

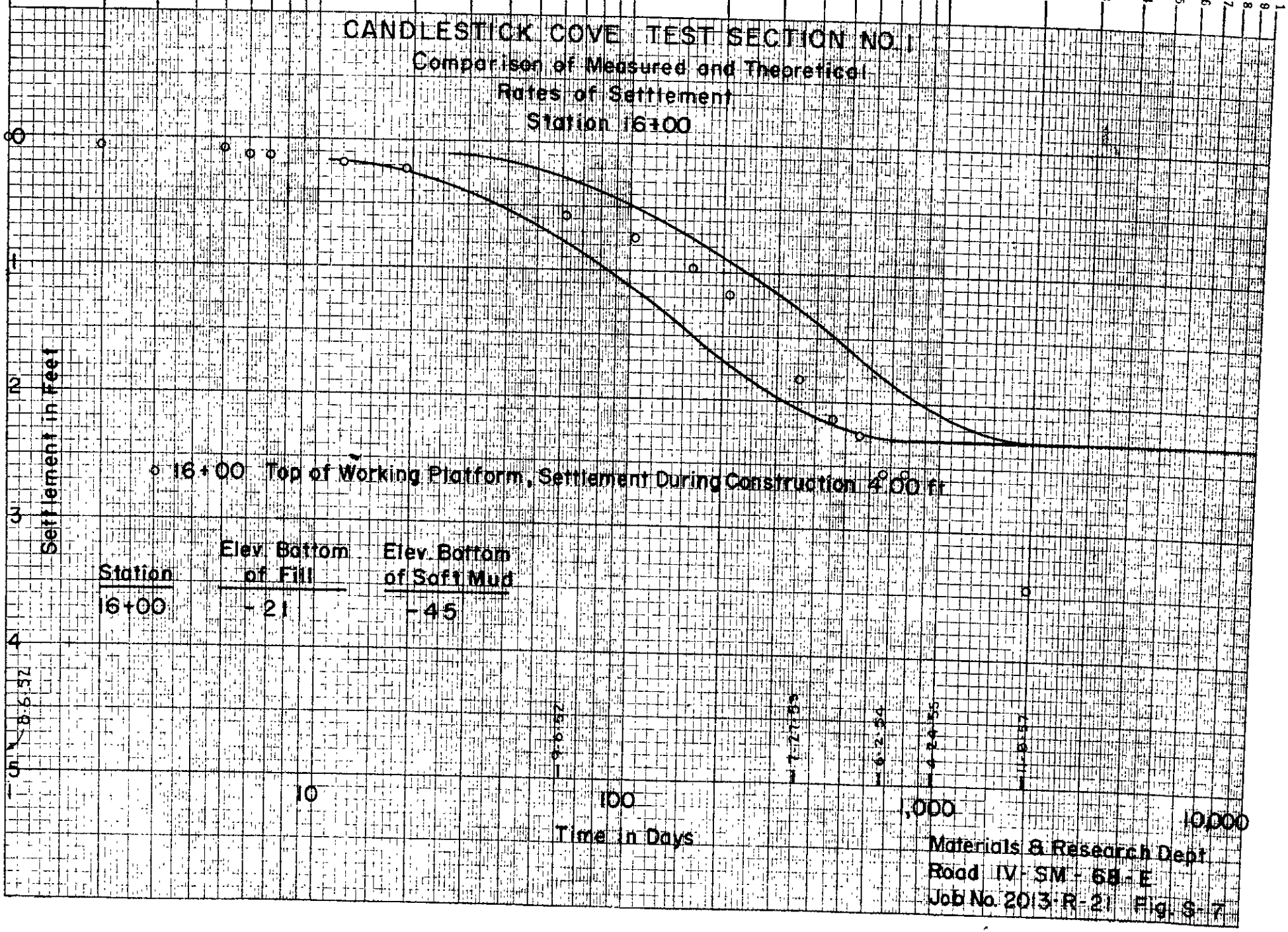


Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. S-5

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

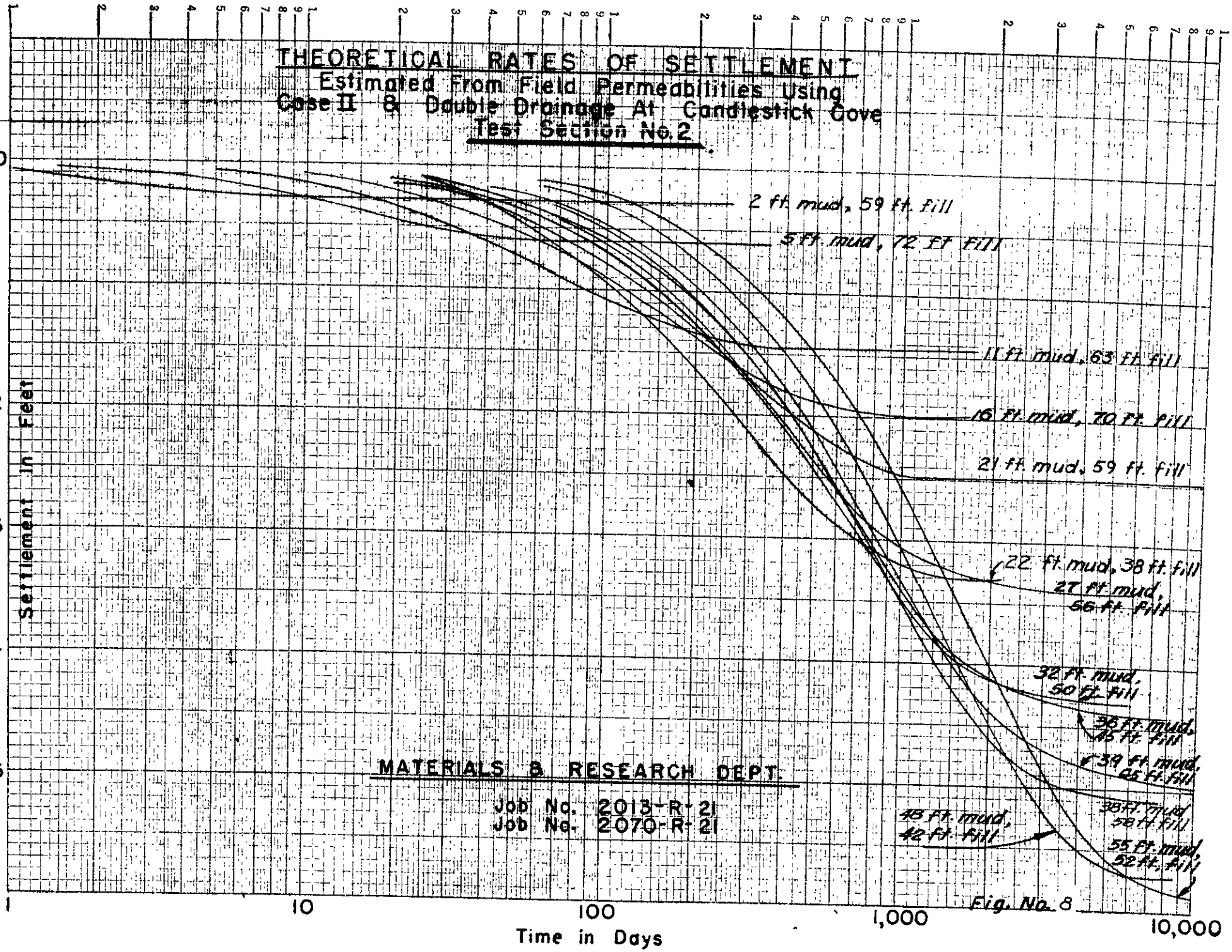
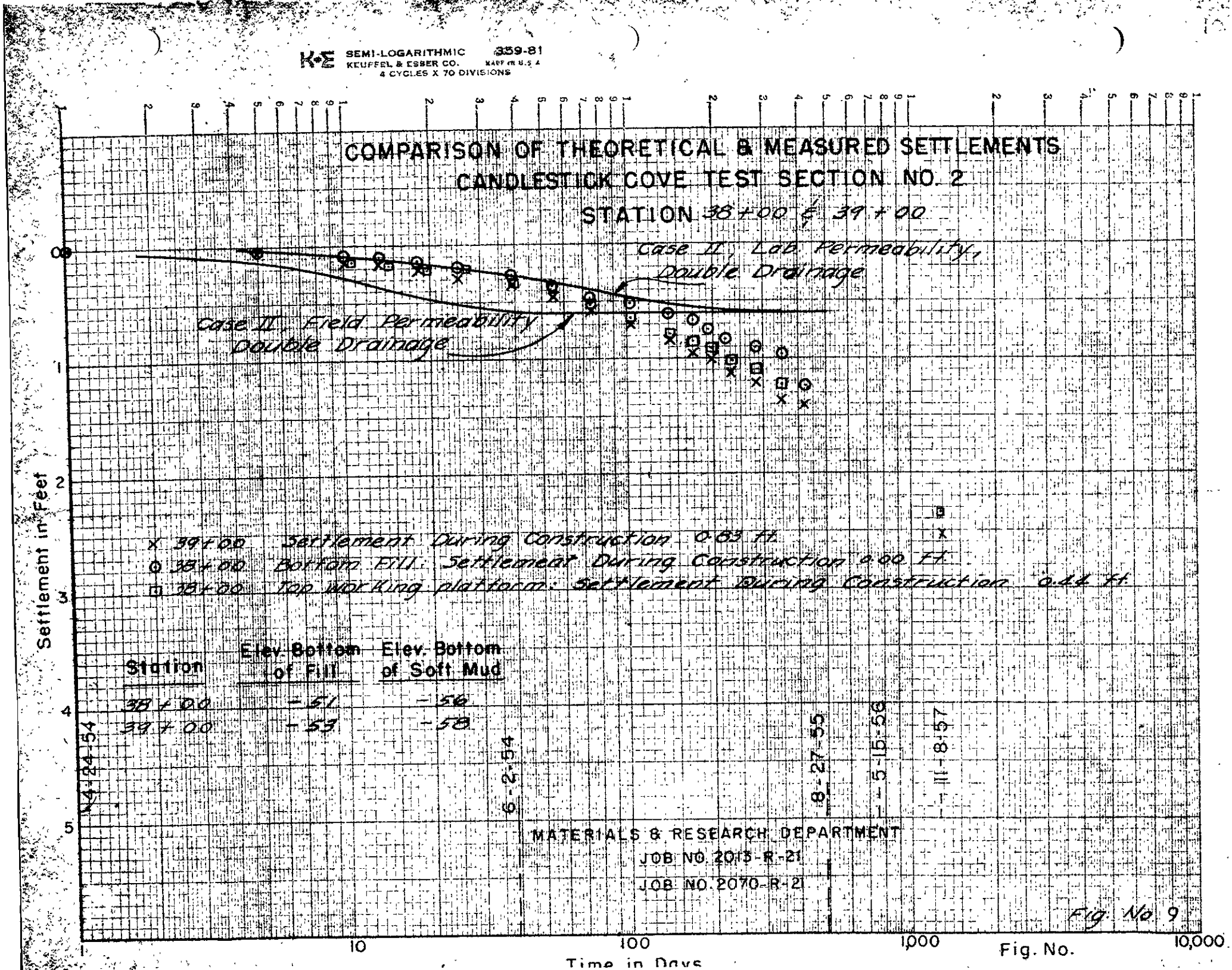
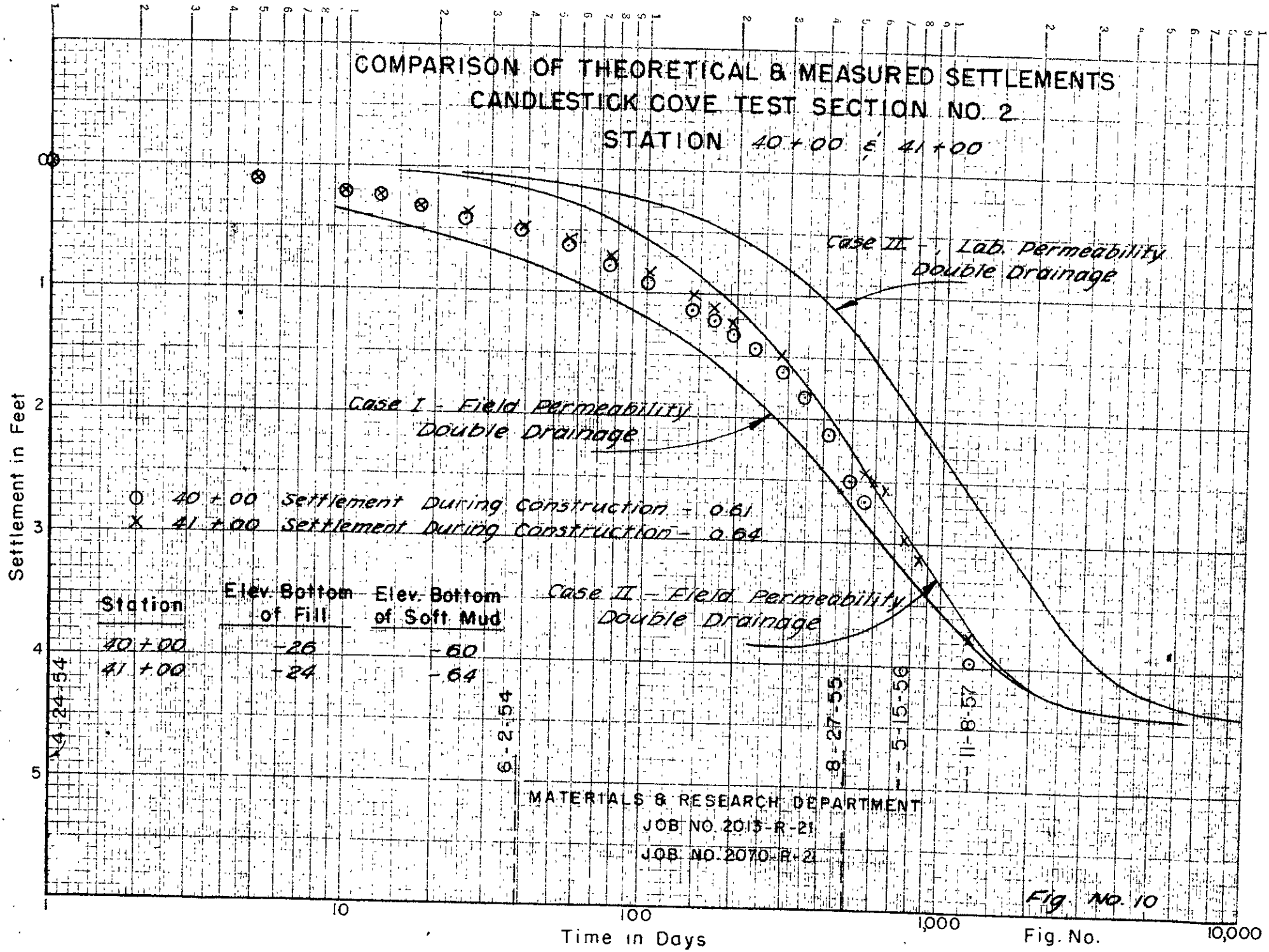


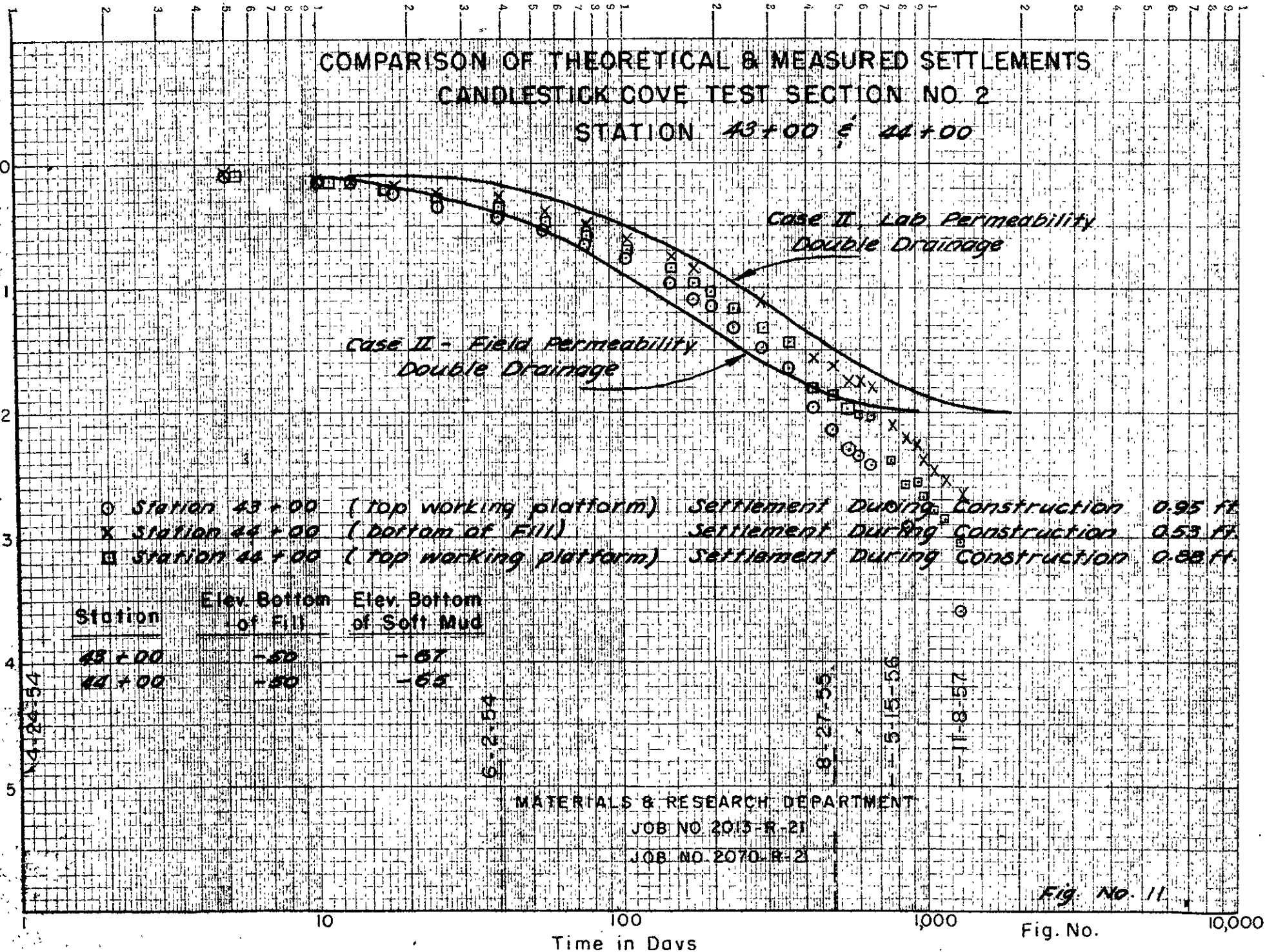
Fig. No. 8


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
COMPARISON OF THEORETICAL & MEASURED SETTLEMENTS
CANDLESTICK COVE TEST SECTION NO. 2
STATION 38+00 & 39+00
Case II, Lab Permeability, Double Drainage
Case II, Field Permeability Double Drainage
× 39+00 Settlement During Construction 0.83 ft.
⊙ 38+00 Bottom Fill. Settlement During Construction 0.00 ft.
⊡ 38+00 Top Working platform. Settlement During Construction 0.44 ft.
Station
Elev. Bottom of Fill
Elev. Bottom of Soft Mud
38+00 -51 -56
39+00 -53 -58
4-24-54
6-2-54
8-27-55
5-15-56
11-8-57
Settlement in Feet
Time in Days
MATERIALS & RESEARCH DEPARTMENT
JOB NO. 2013-R-21
JOB NO. 2070-R-21
Fig. No. 9

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO.
4 CYCLES X 70 DIVISIONS



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

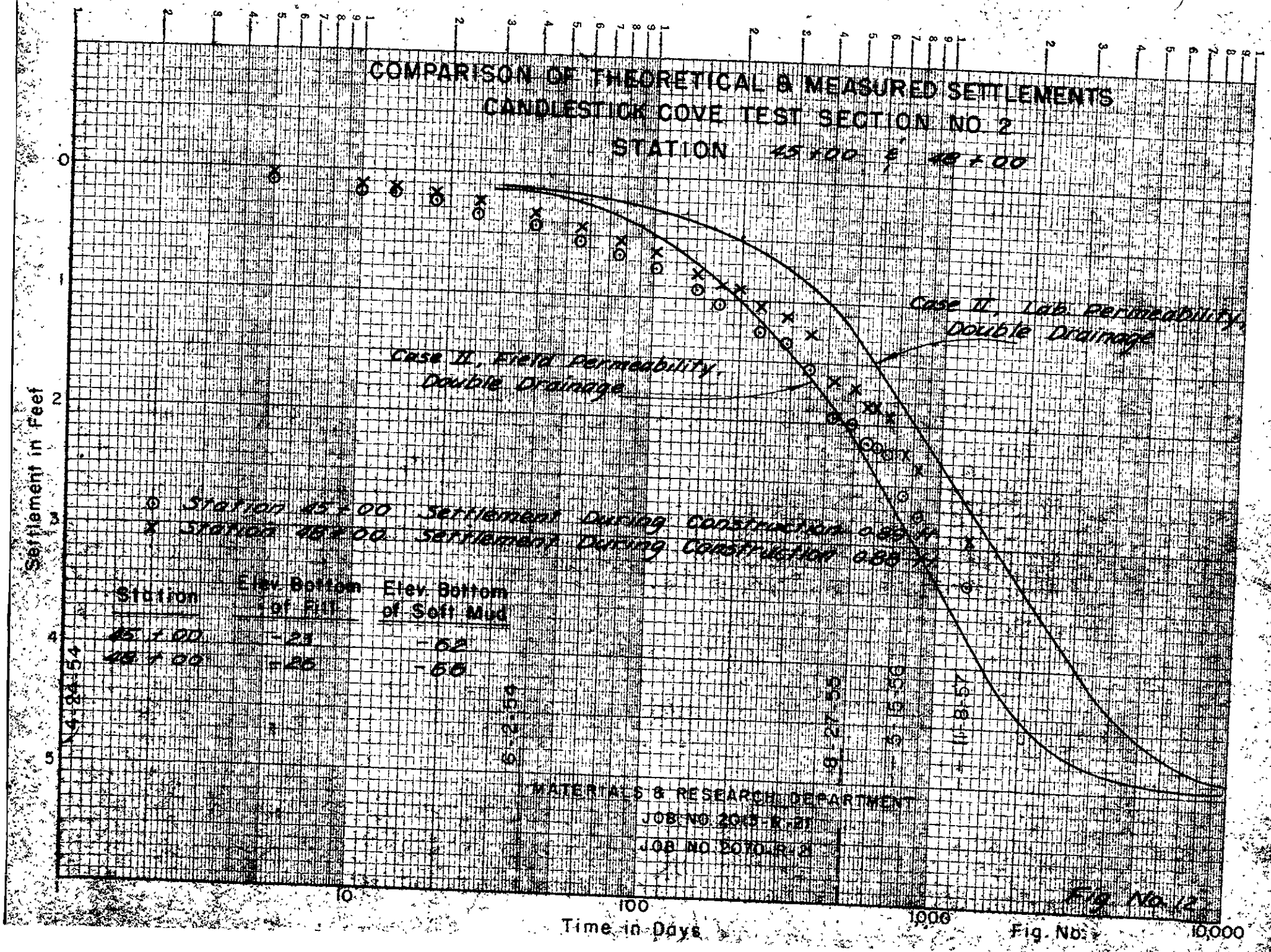
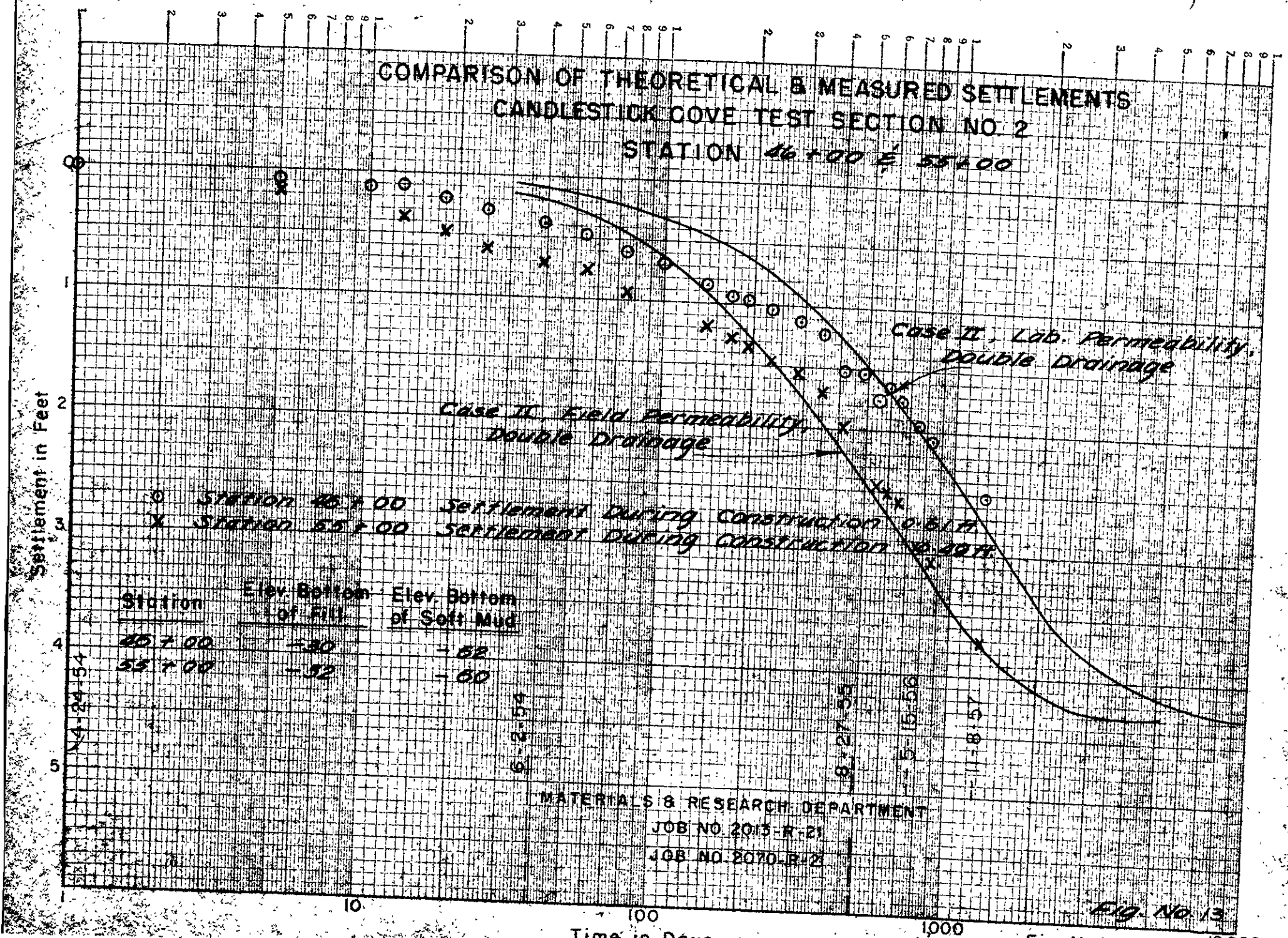
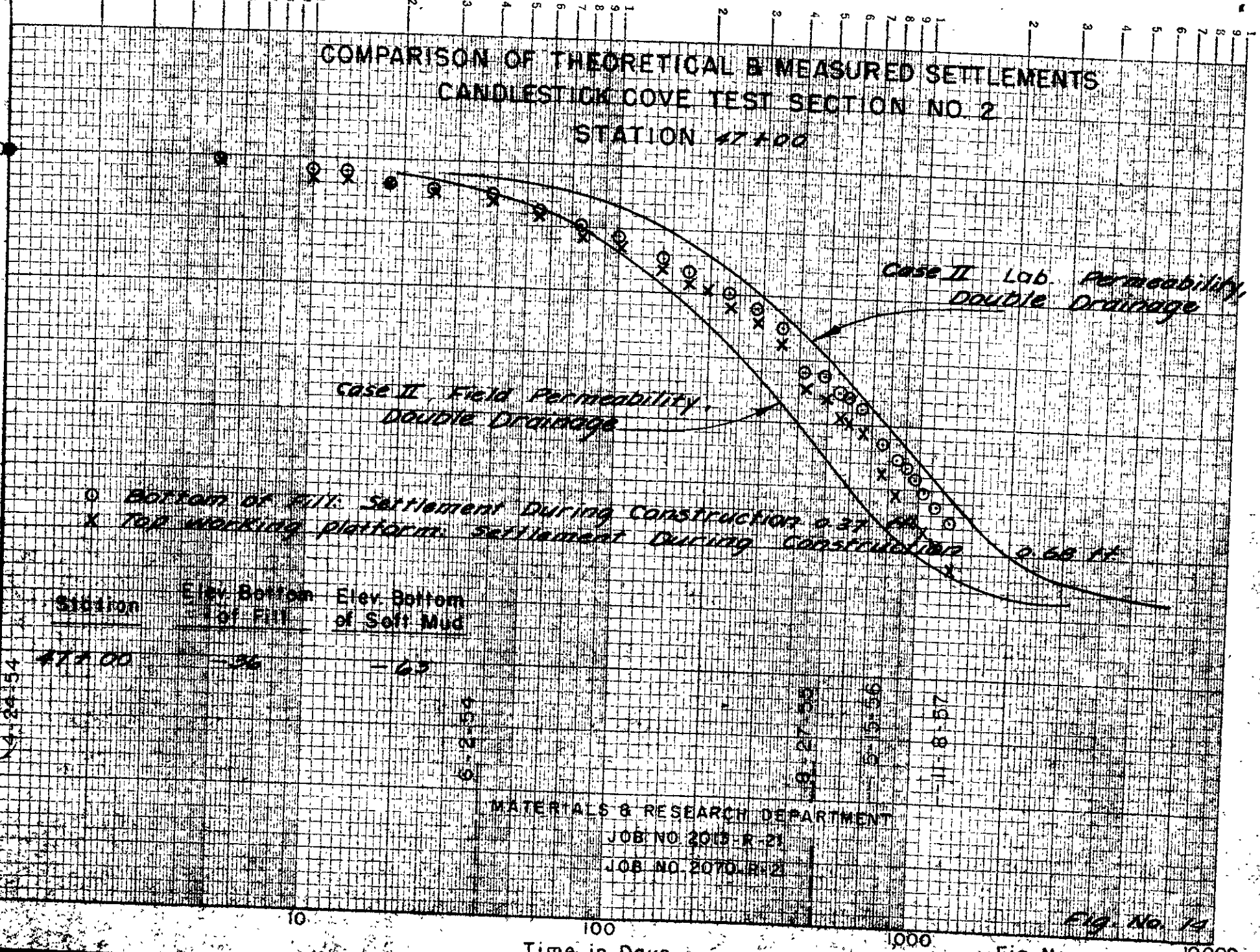


Fig. No.

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

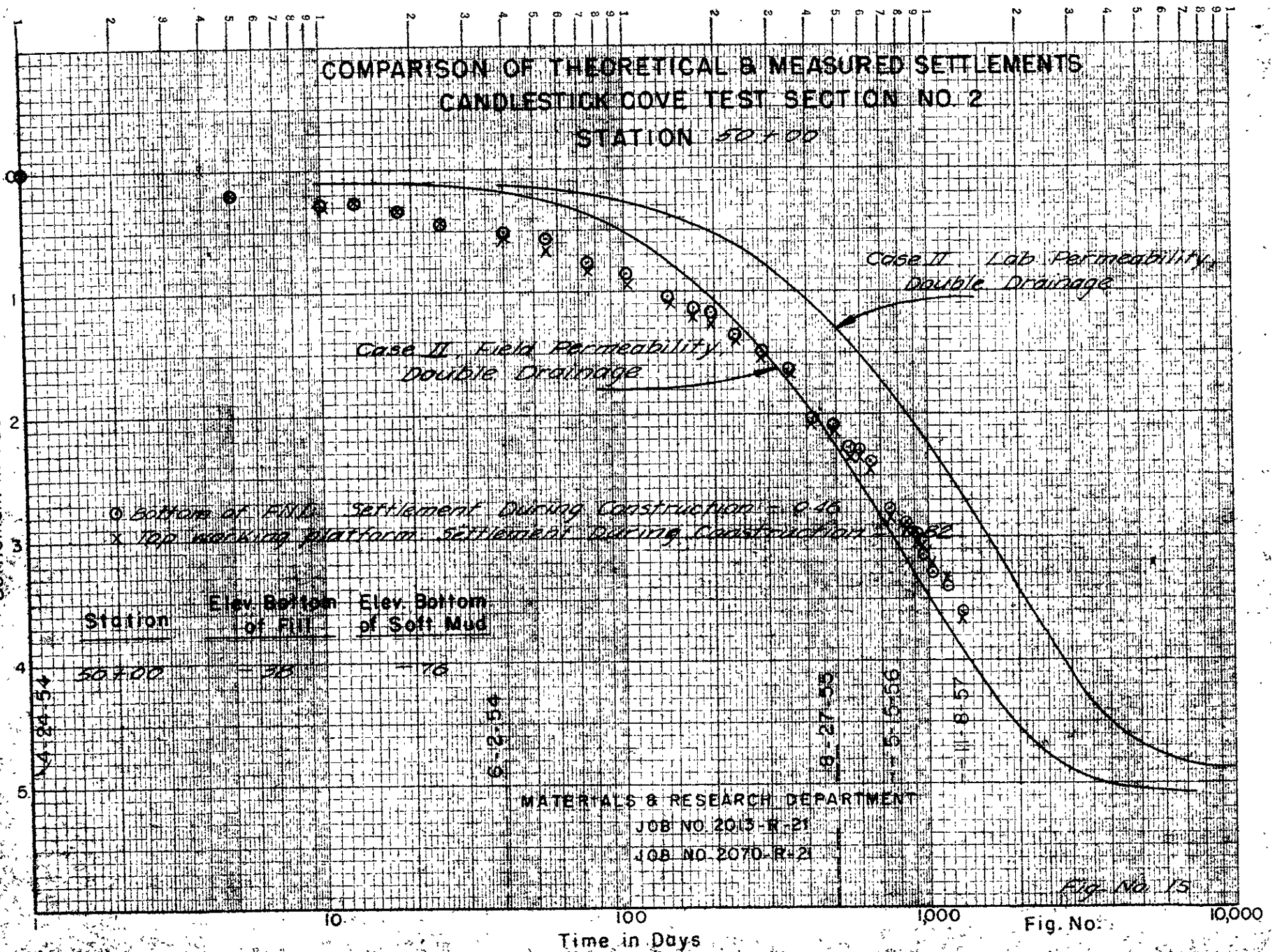
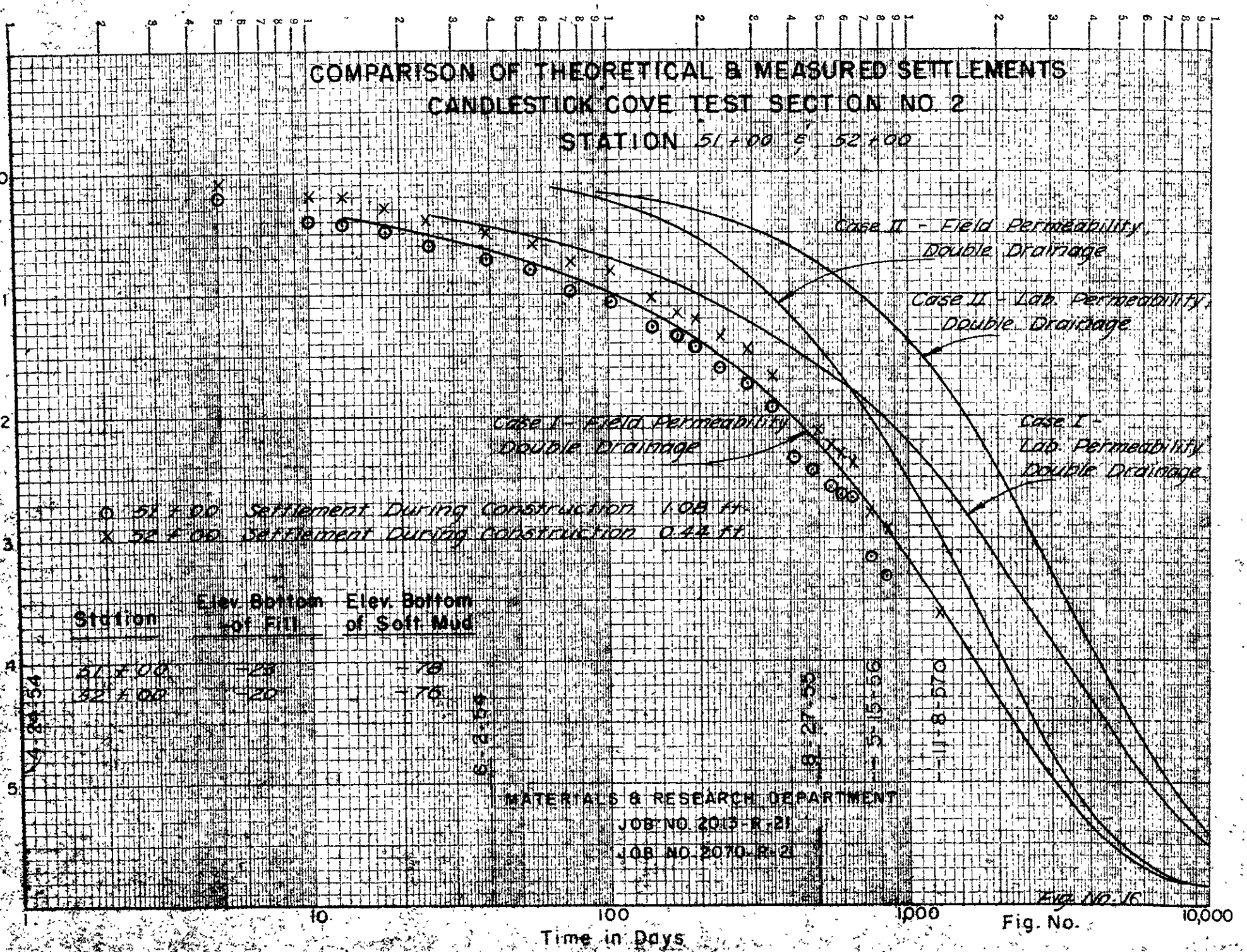


Fig. No.

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

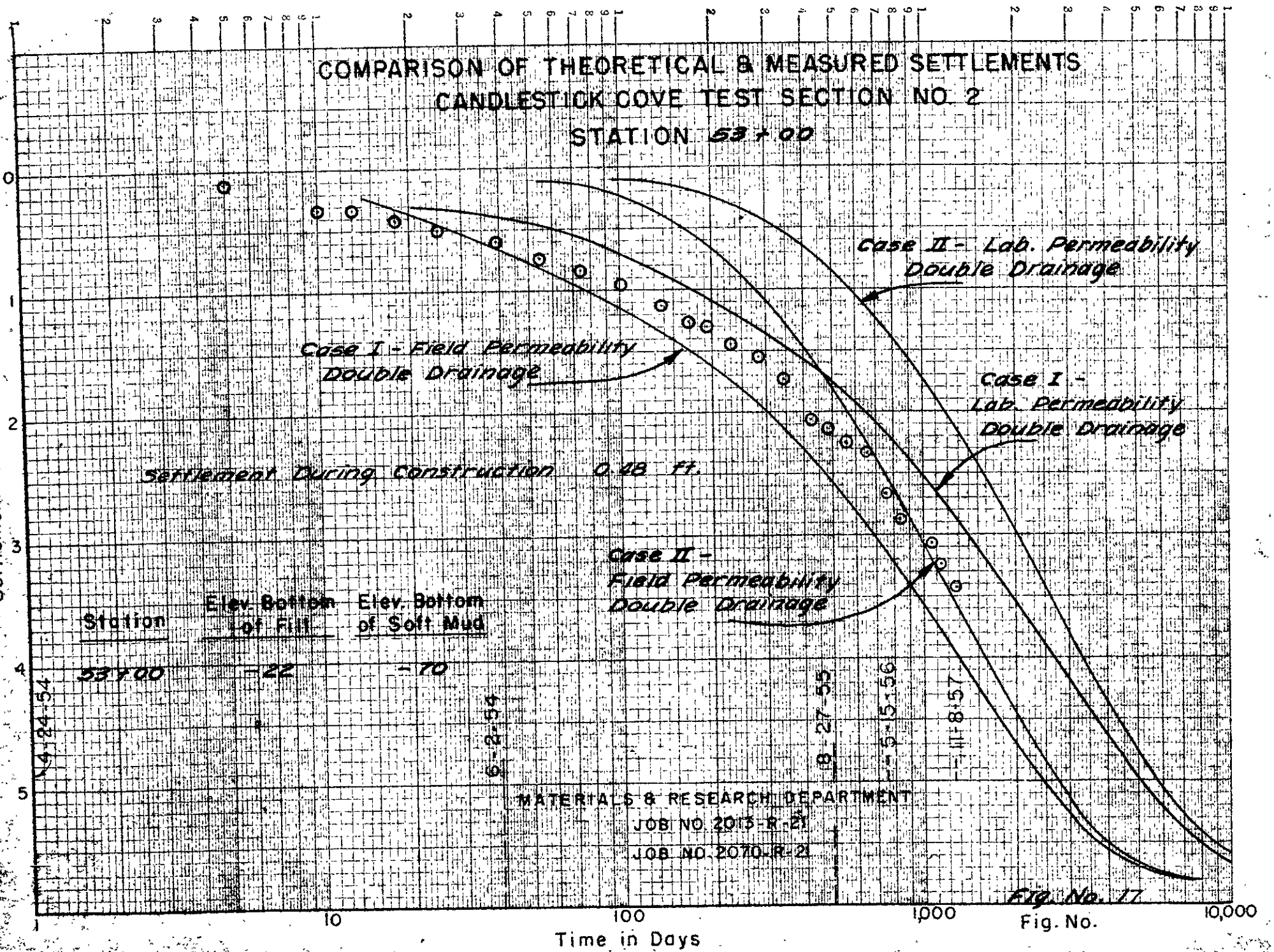
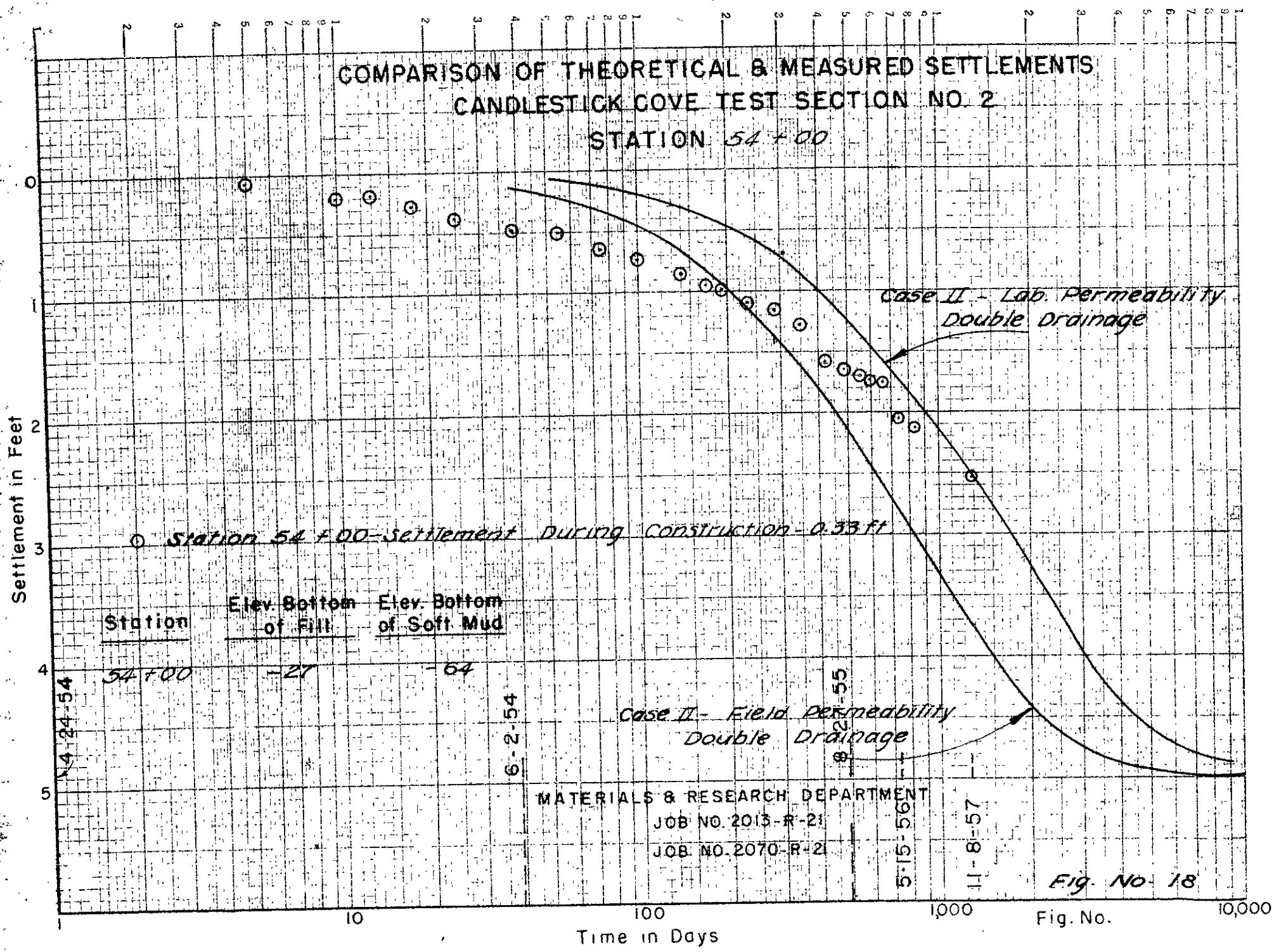


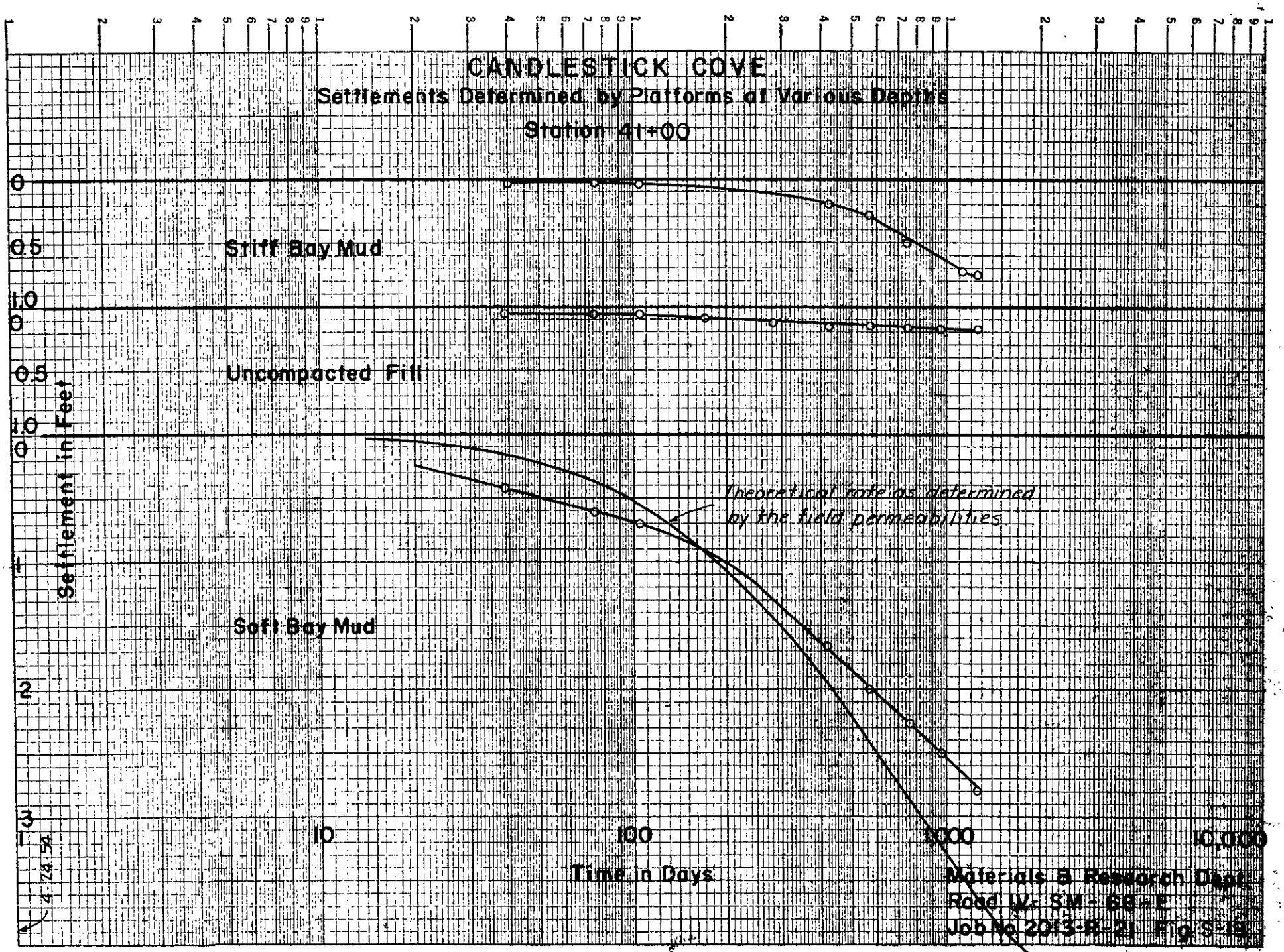
Fig. No. 17

Fig. No.

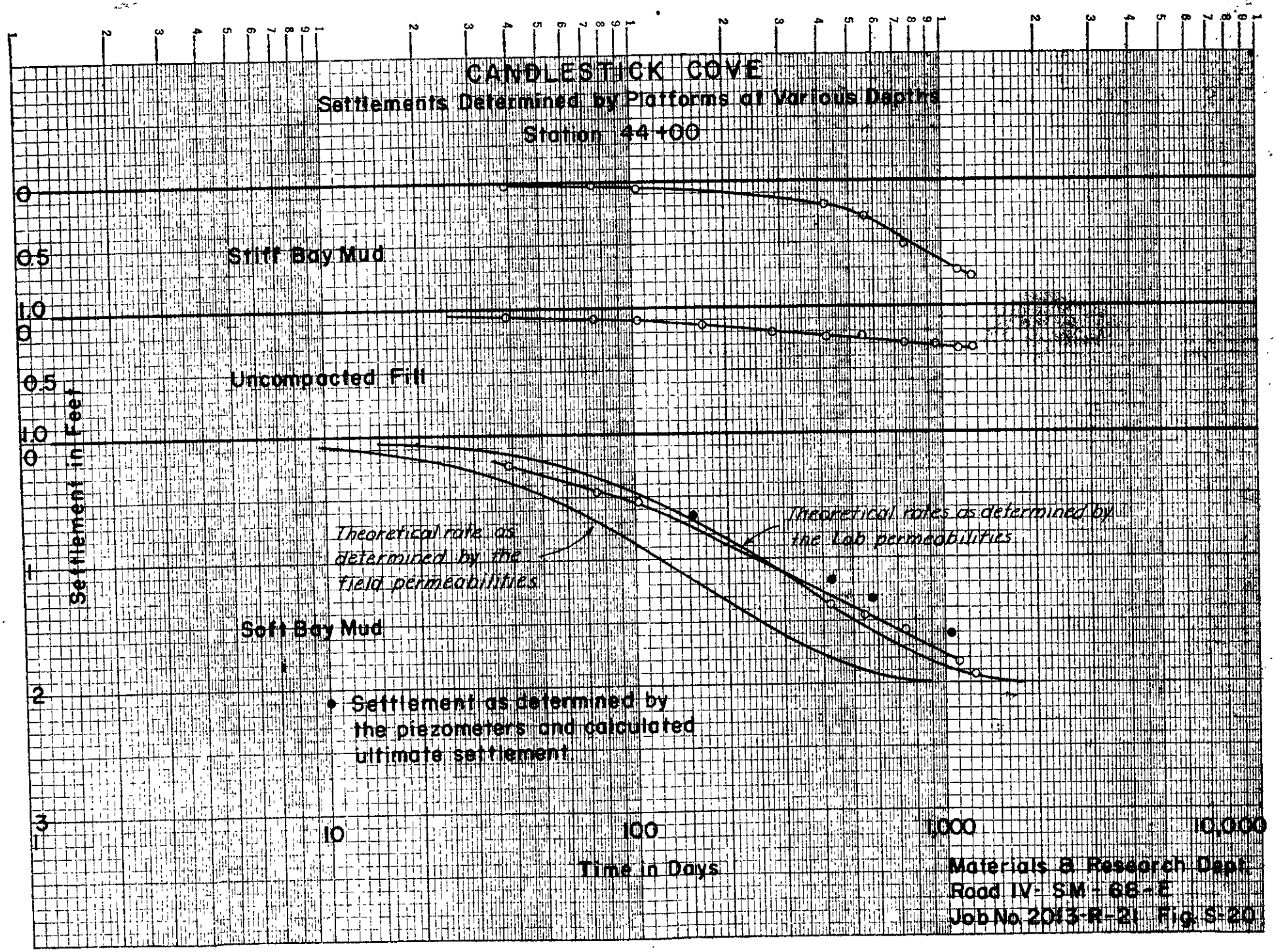
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



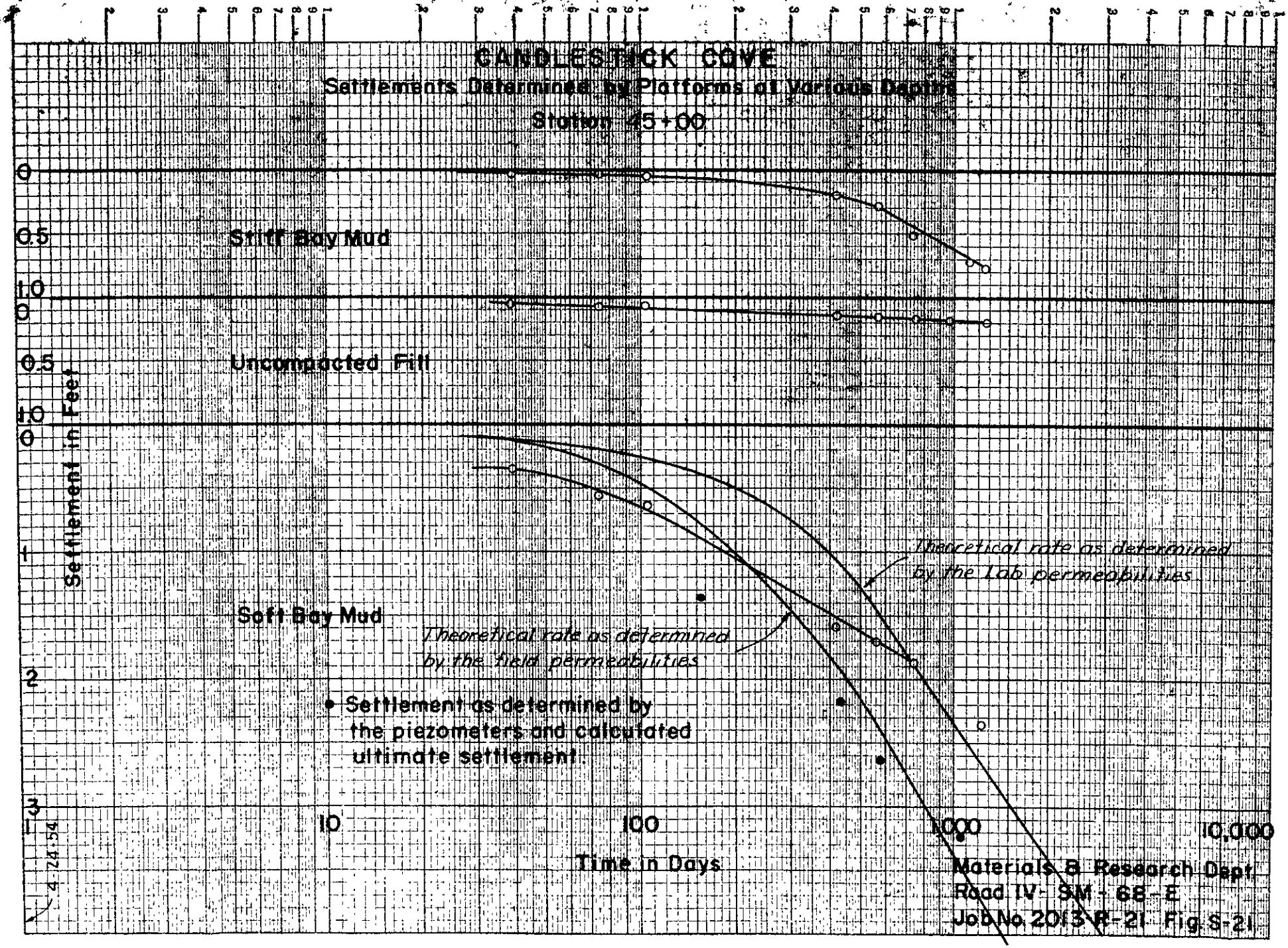
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



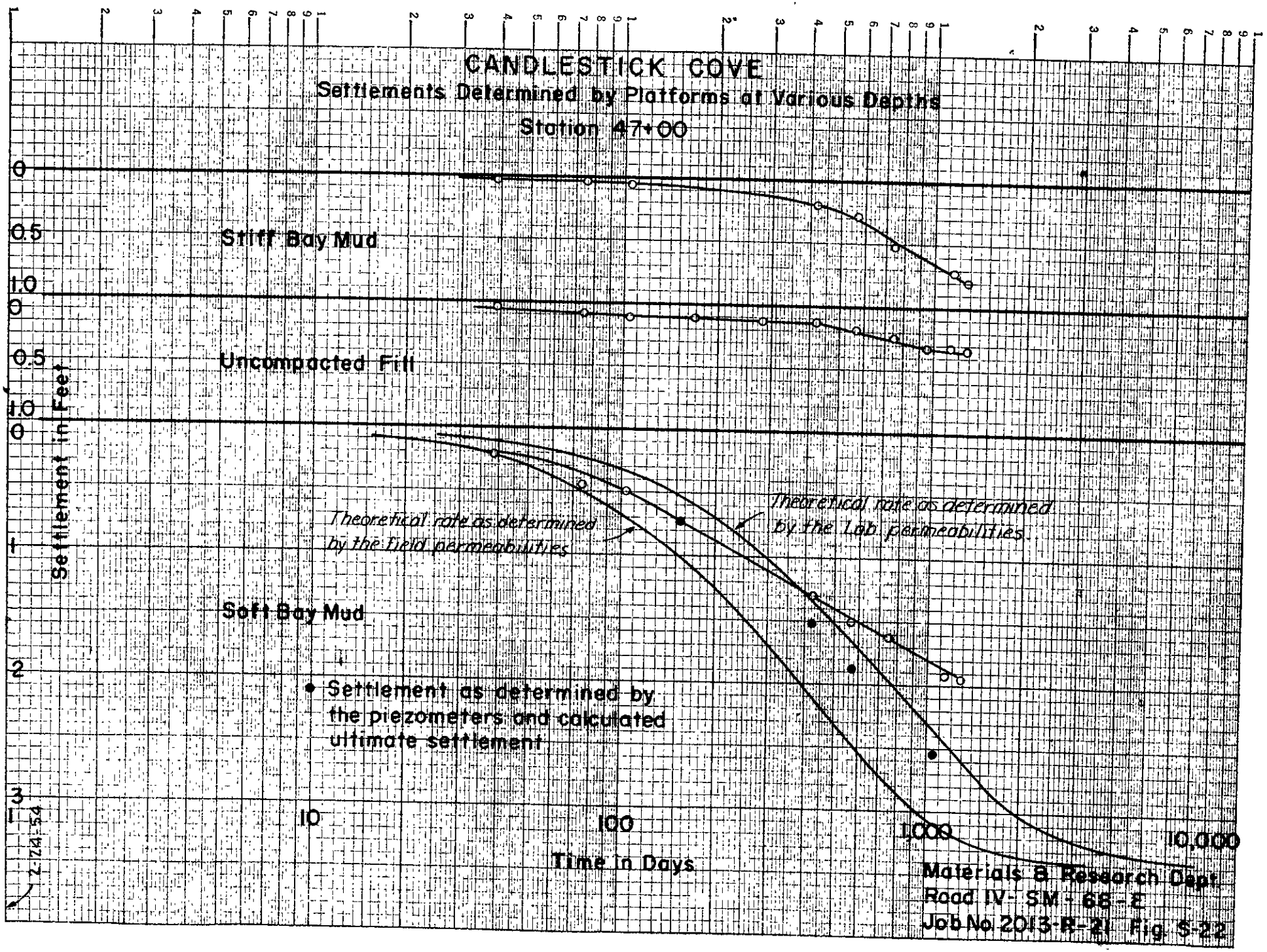
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



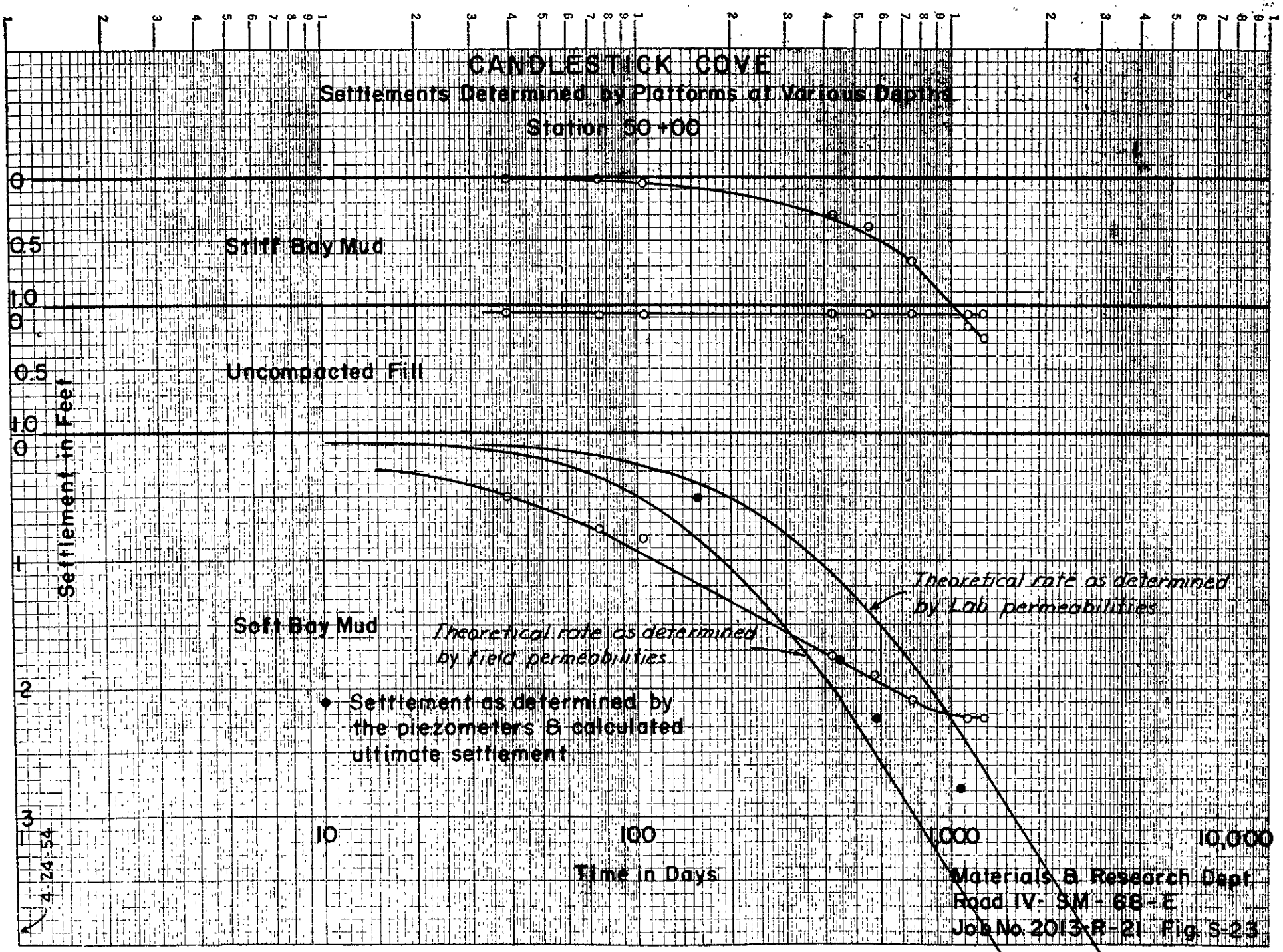
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



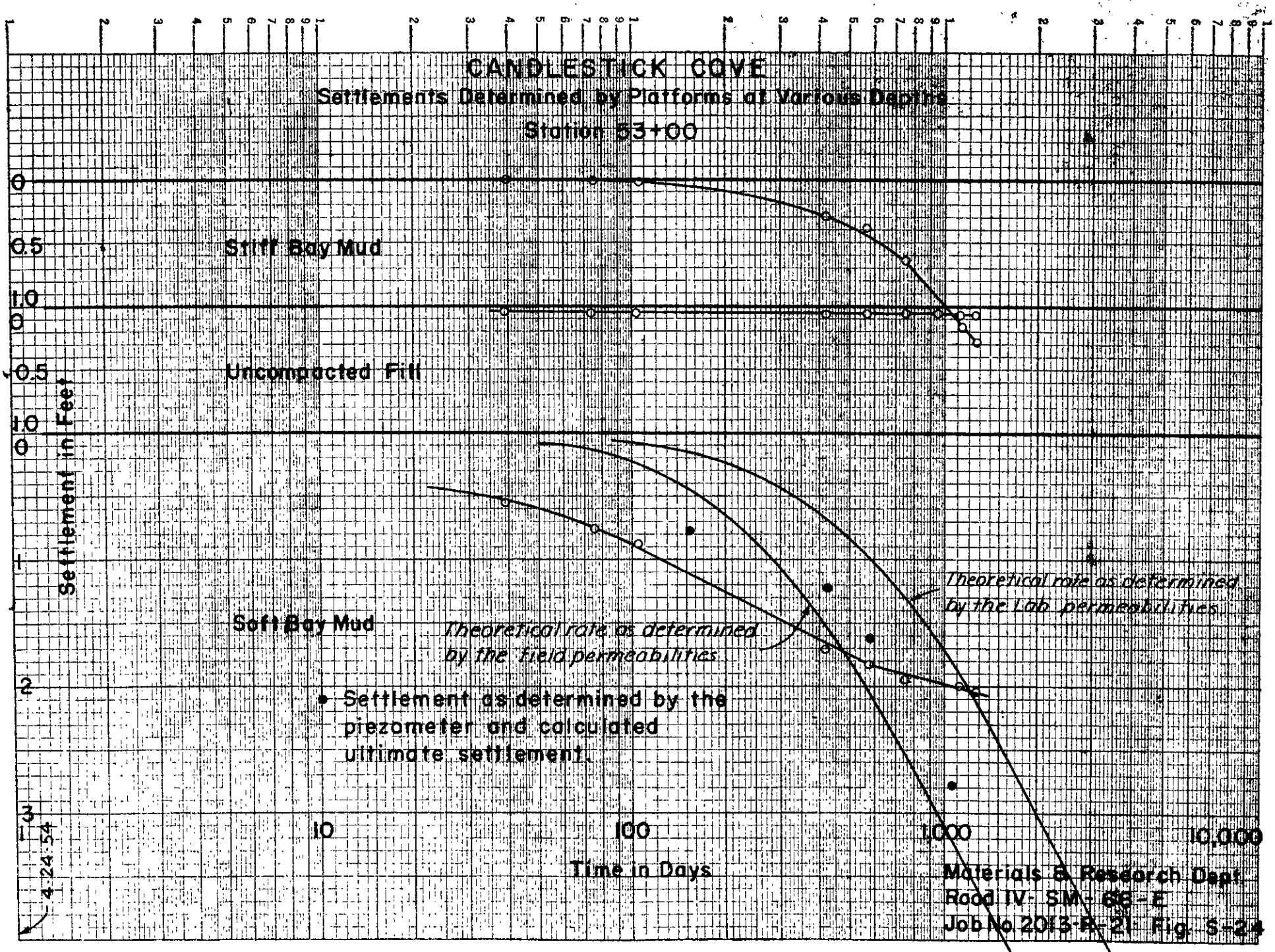
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

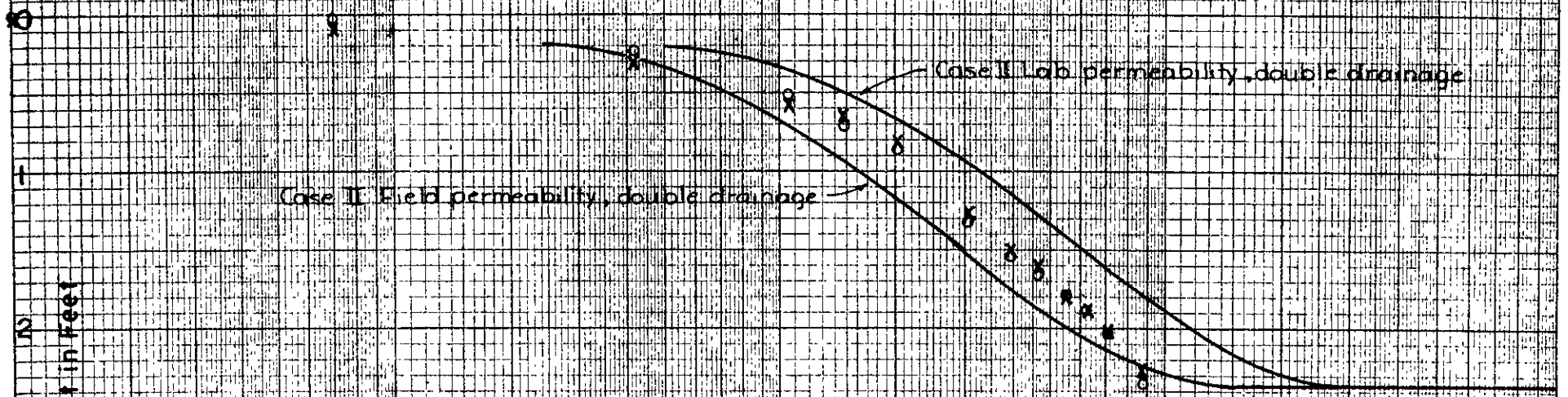


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

CANDLESTICK COVE TEST SECTION NO. 3

Comparison of Measured and Theoretical Rates of Settlement

Station 56+00



x 56+00 Top of Working Platform, Settlement During Construction 1.80 ft.

• 56+00 Bottom of Fill " " 1.4 ft.

Station	Elev. Bottom of Fill	Elev. Bottom of Soft Mud
56+00	-25	-57

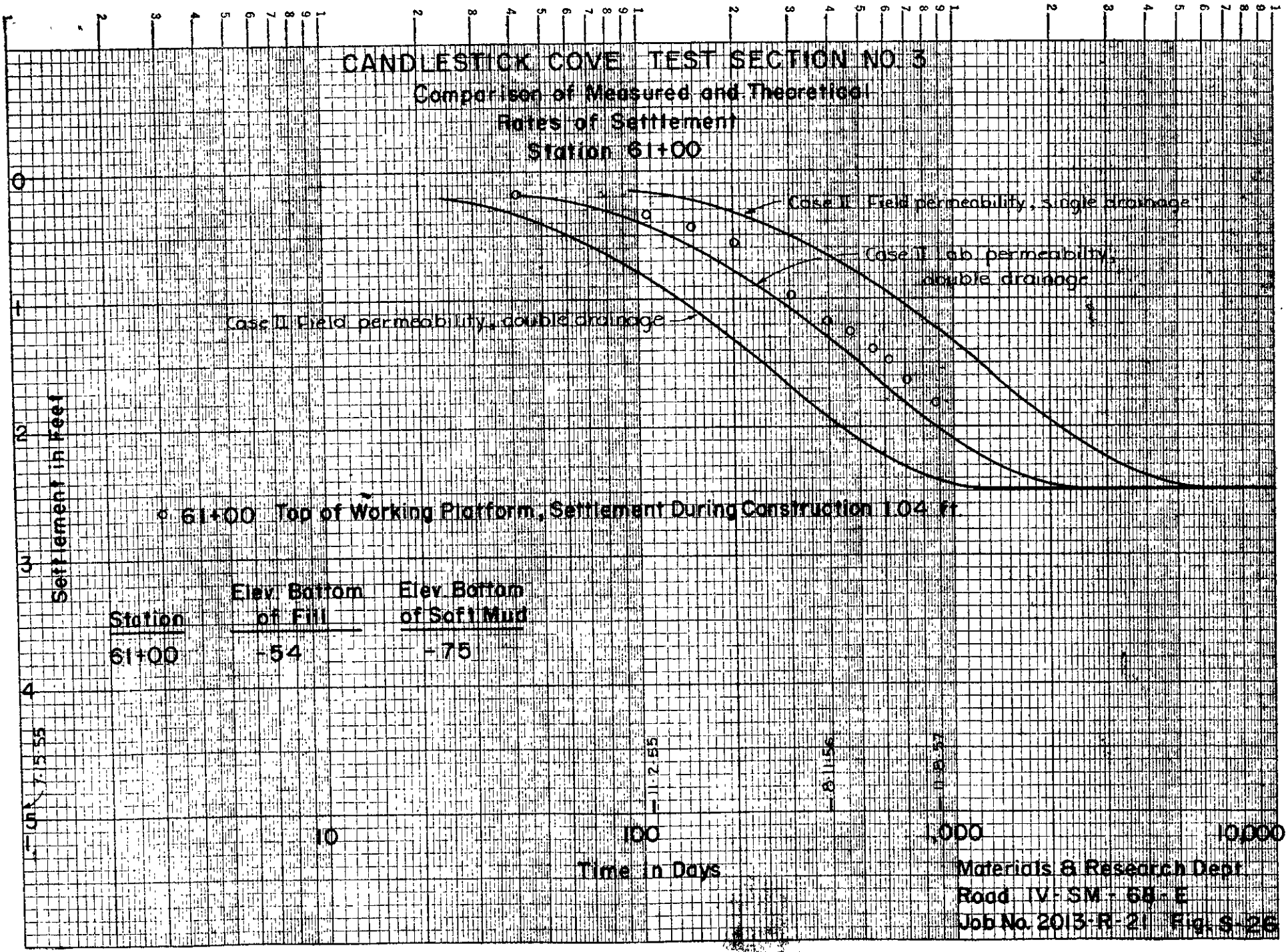
Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. S-25

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

CANDLESTICK COVE TEST SECTION NO. 3

Comparison of Measured and Theoretical Rates of Settlement

Station 61+00



Station	Elev. Bottom of Fill	Elev. Bottom of Soft Mud
61+00	-54	-75

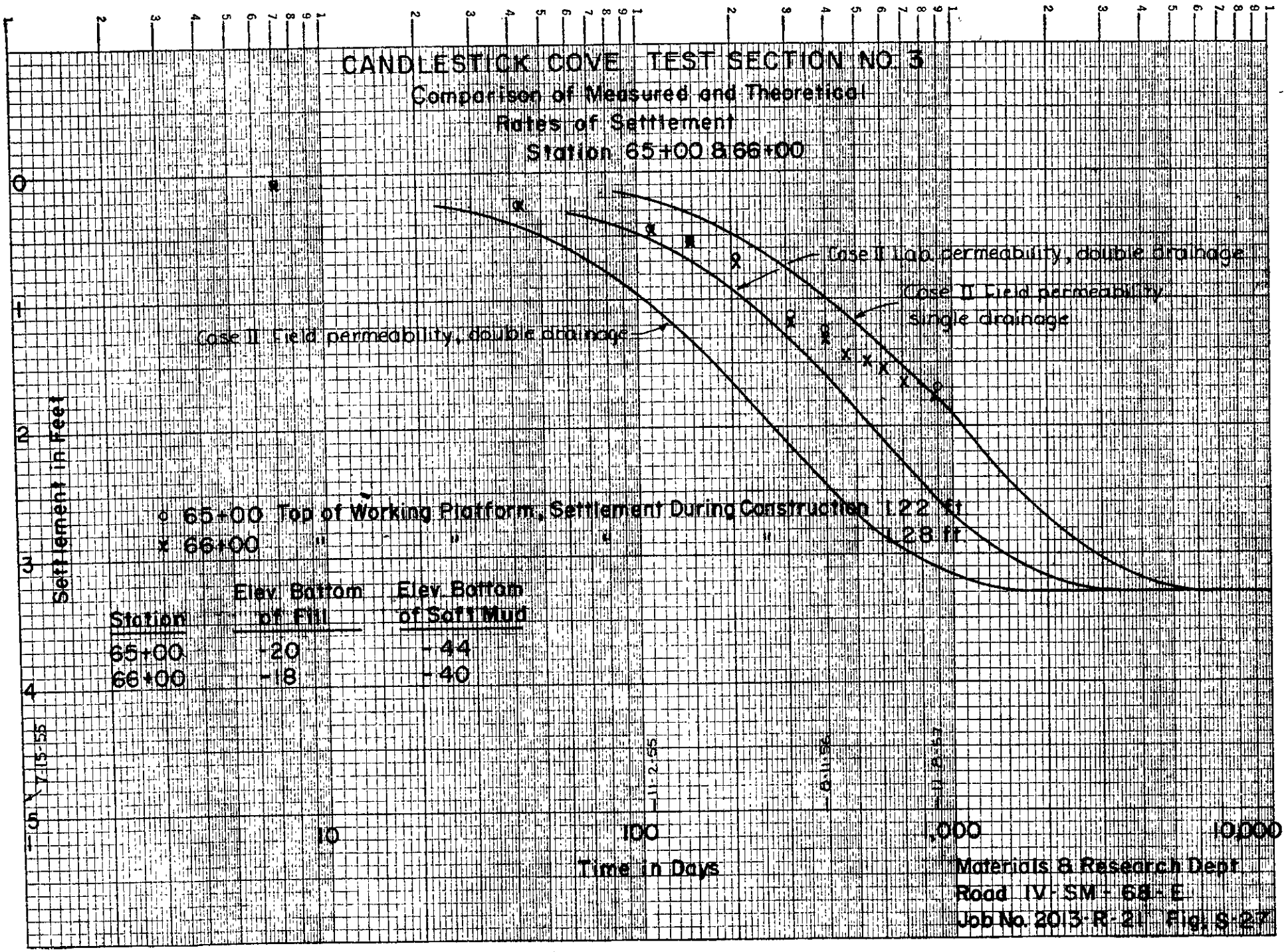
Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. S-26

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

CANDLESTICK COVE TEST SECTION NO. 3

Comparison of Measured and Theoretical Rates of Settlement

Station 65+00 & 66+00



○ 65+00 Top of Working Platform, Settlement During Construction 1.22 ft.
× 66+00 " " " " 1.28 ft.

Station	Elev. Bottom of Fill	Elev. Bottom of Soft Mud
65+00	-20	-44
66+00	-18	-40

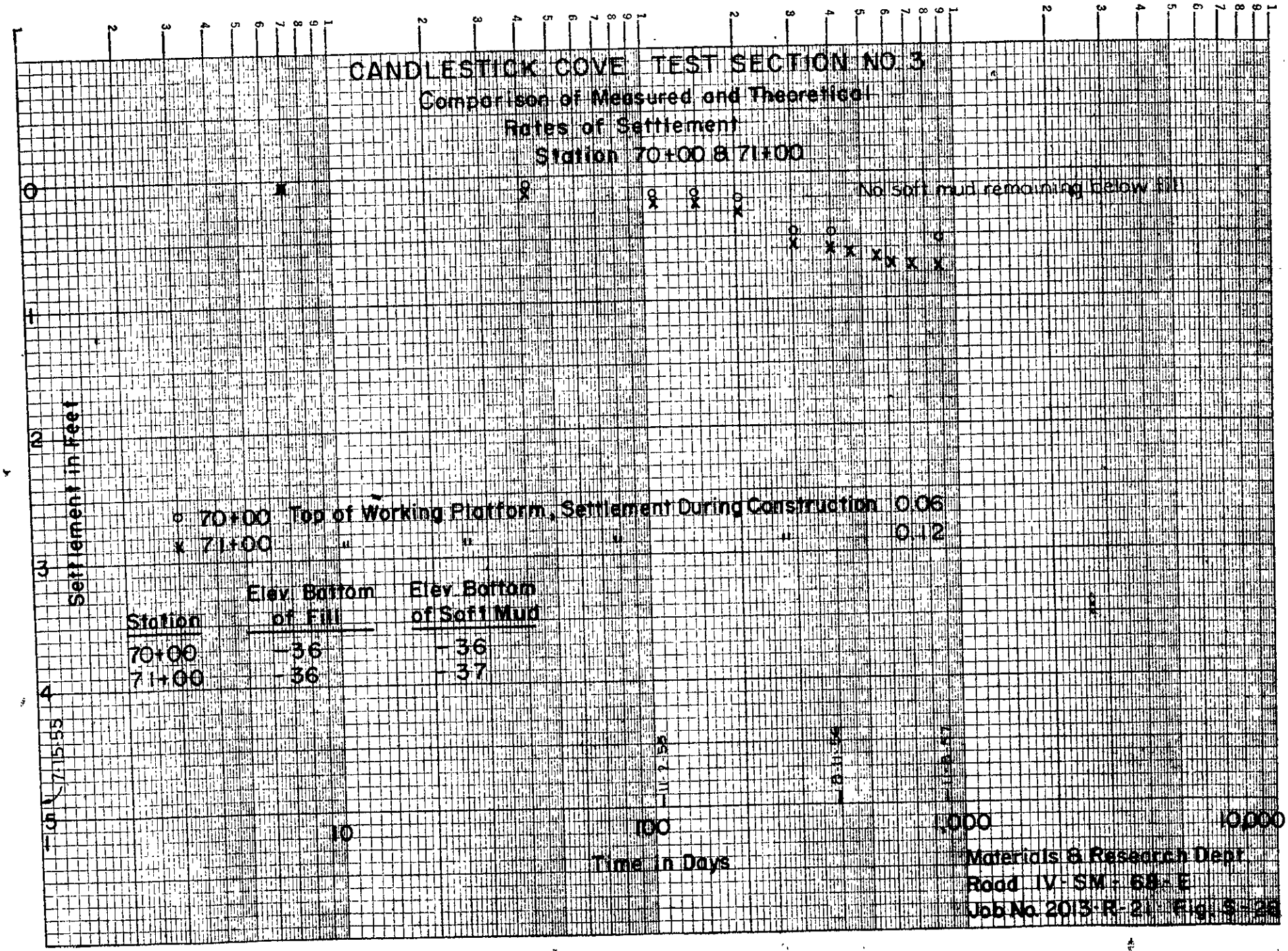
Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. S-27

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

CANDLESTICK COVE TEST SECTION NO. 3

Comparison of Measured and Theoretical Rates of Settlement

Station 70+00 & 71+00



o 70+00 Top of Working Platform, Settlement During Construction 0.06
x 71+00 " " " " 0.12

Station	Elev. Bottom of Fill	Elev. Bottom of Soft Mud
70+00	-36	-36
71+00	-36	-37

Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. S-26

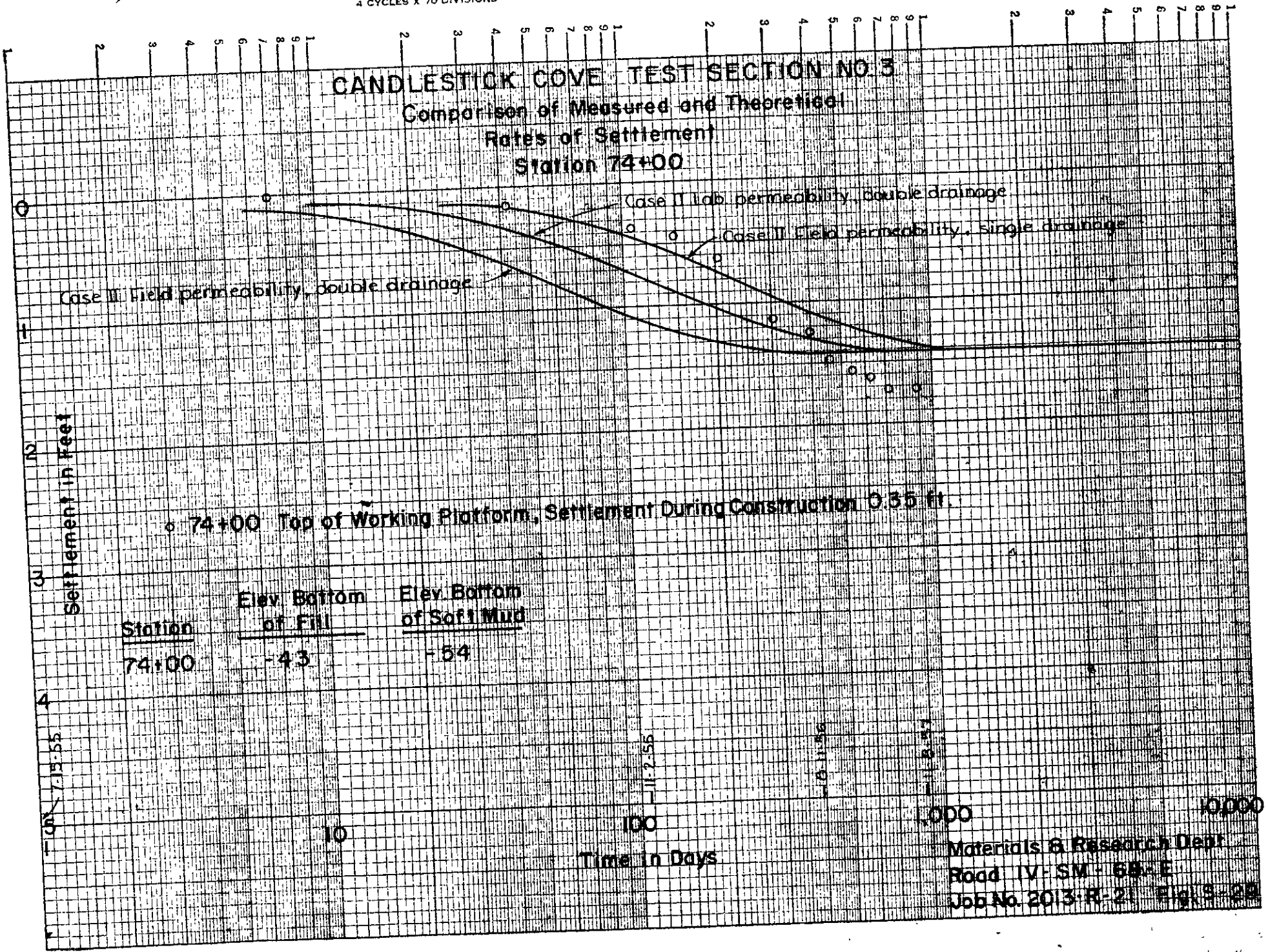
7-15-55

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

CANDLESTICK COVE TEST SECTION NO. 3

Comparison of Measured and Theoretical Rates of Settlement

Station 74+00

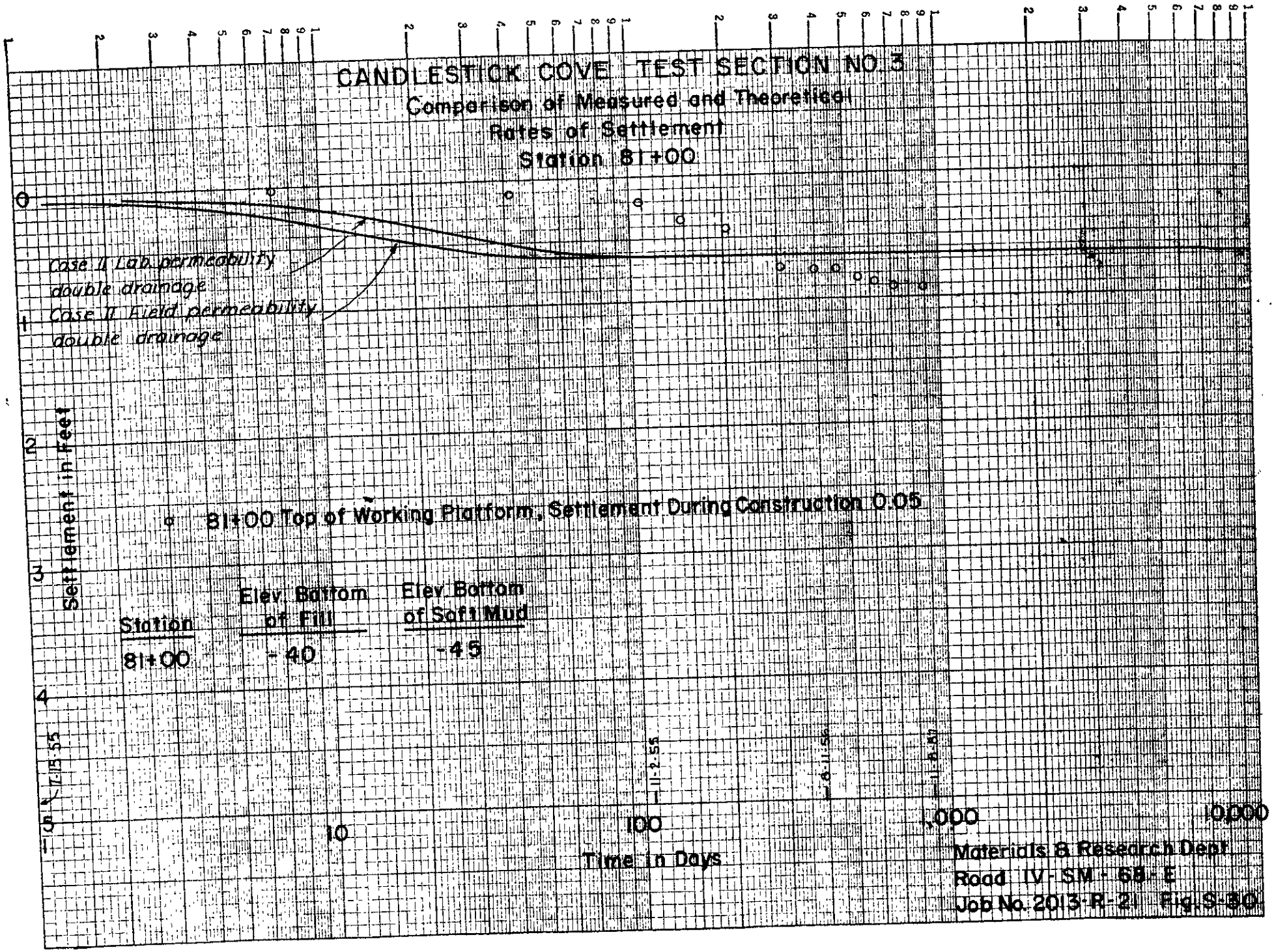


o 74+00 Top of Working Platform, Settlement During Construction 0.35 ft.

Station	Elev. Bottom of Fill	Elev. Bottom of Soft Mud
74+00	-43	-54

Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. S-28

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

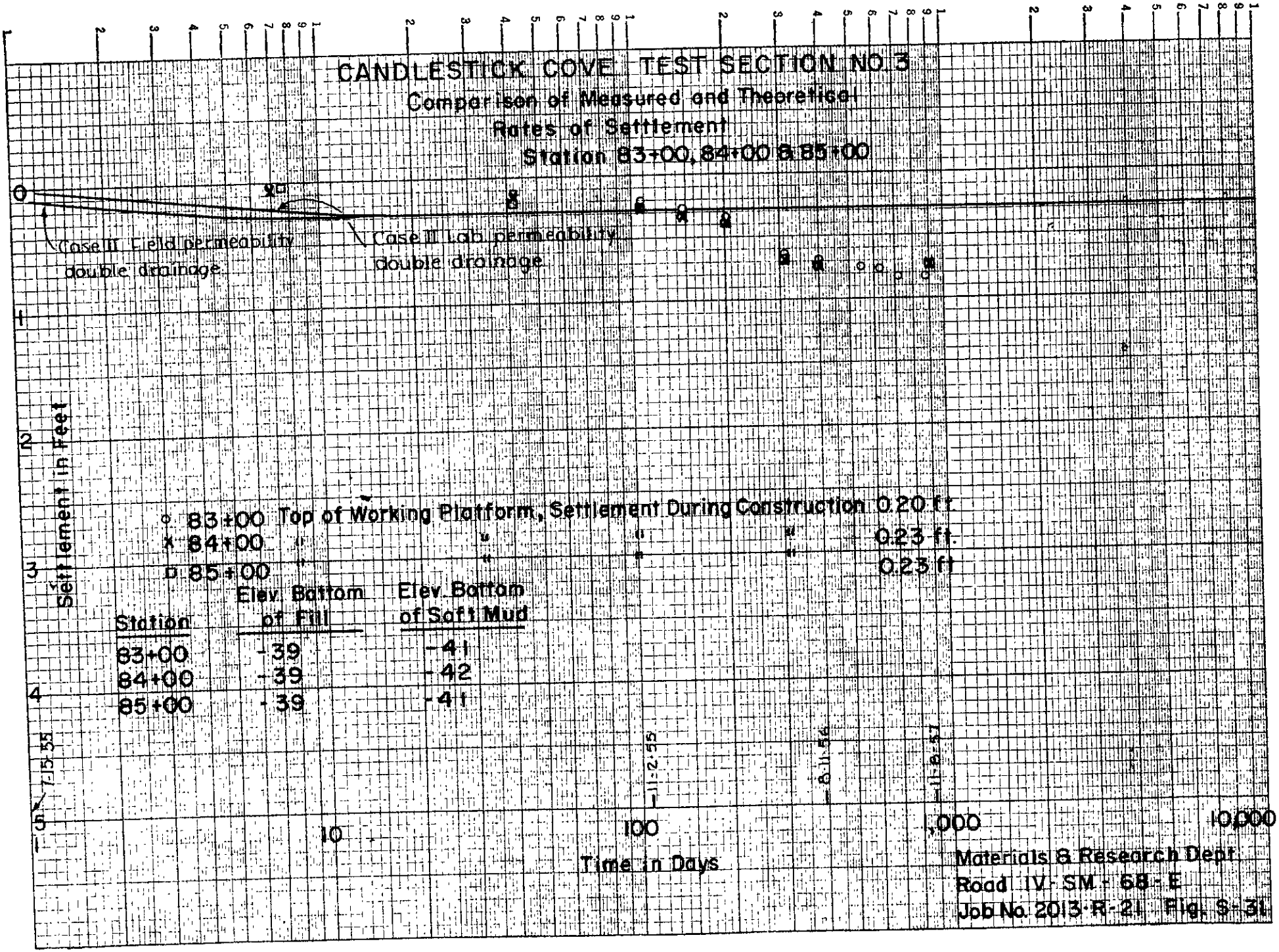


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

CANDLESTICK COVE TEST SECTION NO. 3

Comparison of Measured and Theoretical Rates of Settlement

Station 83+00, 84+00 & 85+00

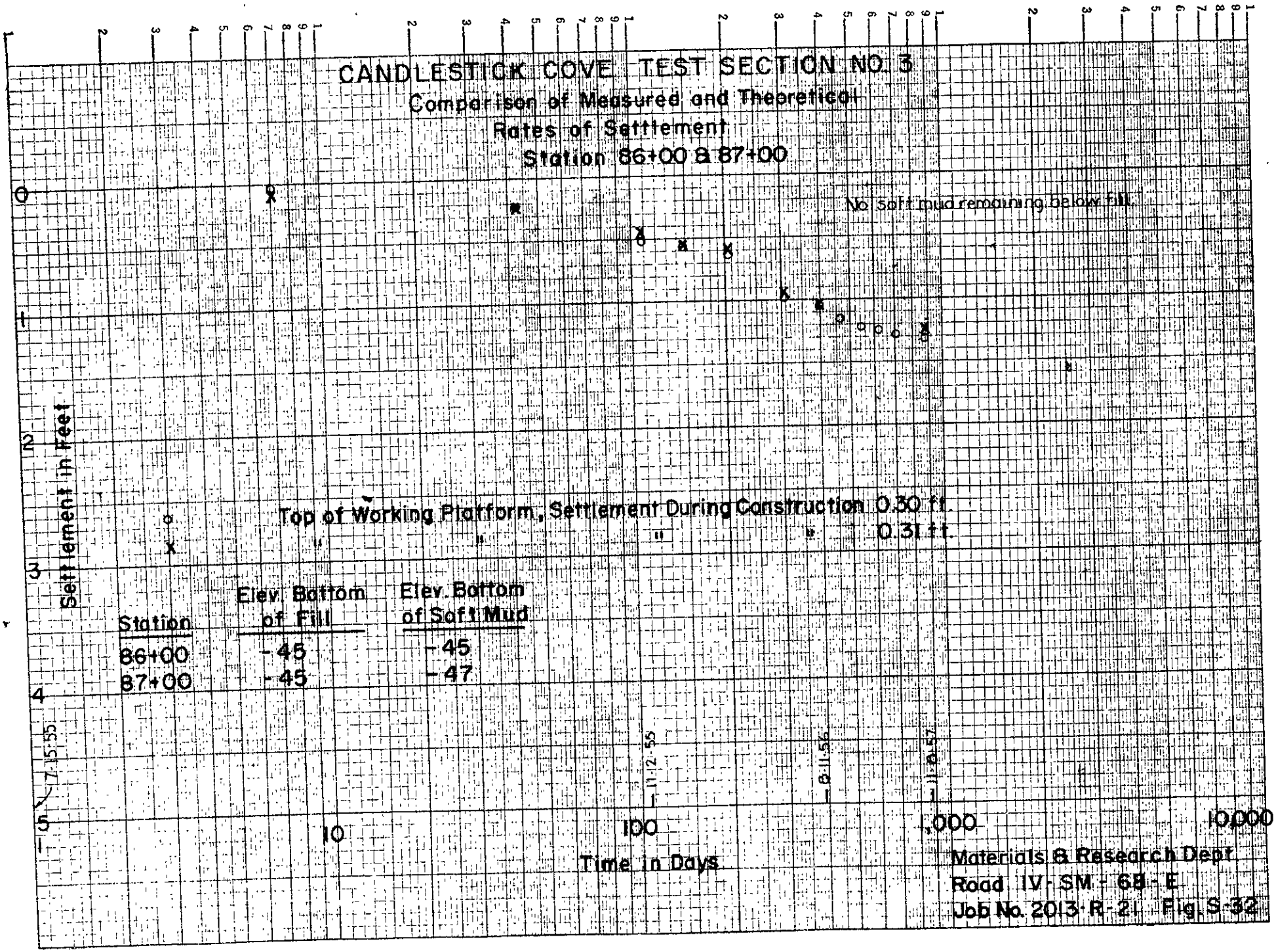


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

CANDLESTICK COVE TEST SECTION NO. 3

Comparison of Measured and Theoretical Rates of Settlement

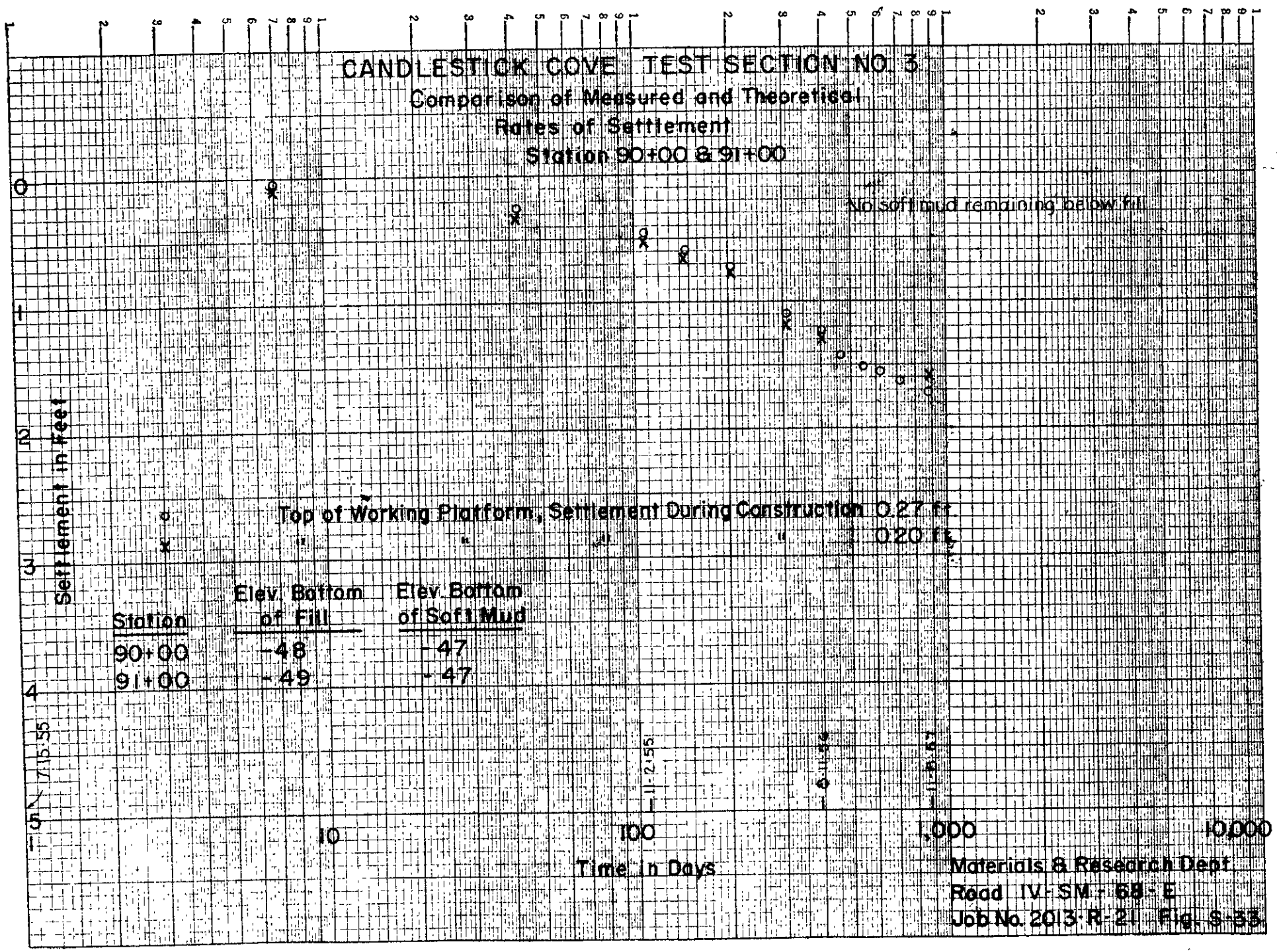
Station 86+00 & 87+00



Station	Elev. Bottom of Fill	Elev. Bottom of Soft Mud
86+00	-45	-45
87+00	-45	-47

Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. S-32

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

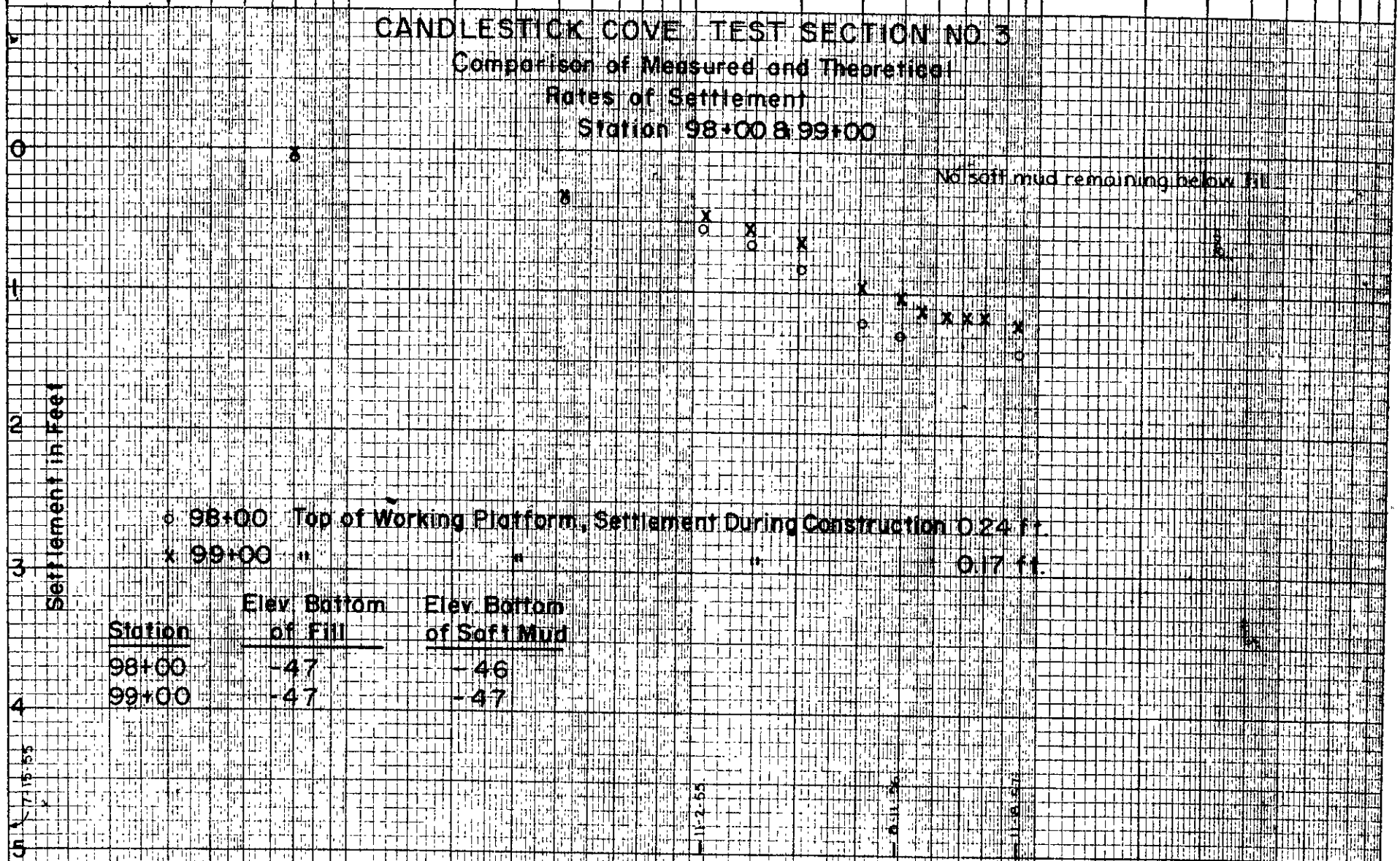


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

CANDLESTICK COVE TEST SECTION NO. 3

Comparison of Measured and Theoretical Rates of Settlement

Station 98+00 & 99+00

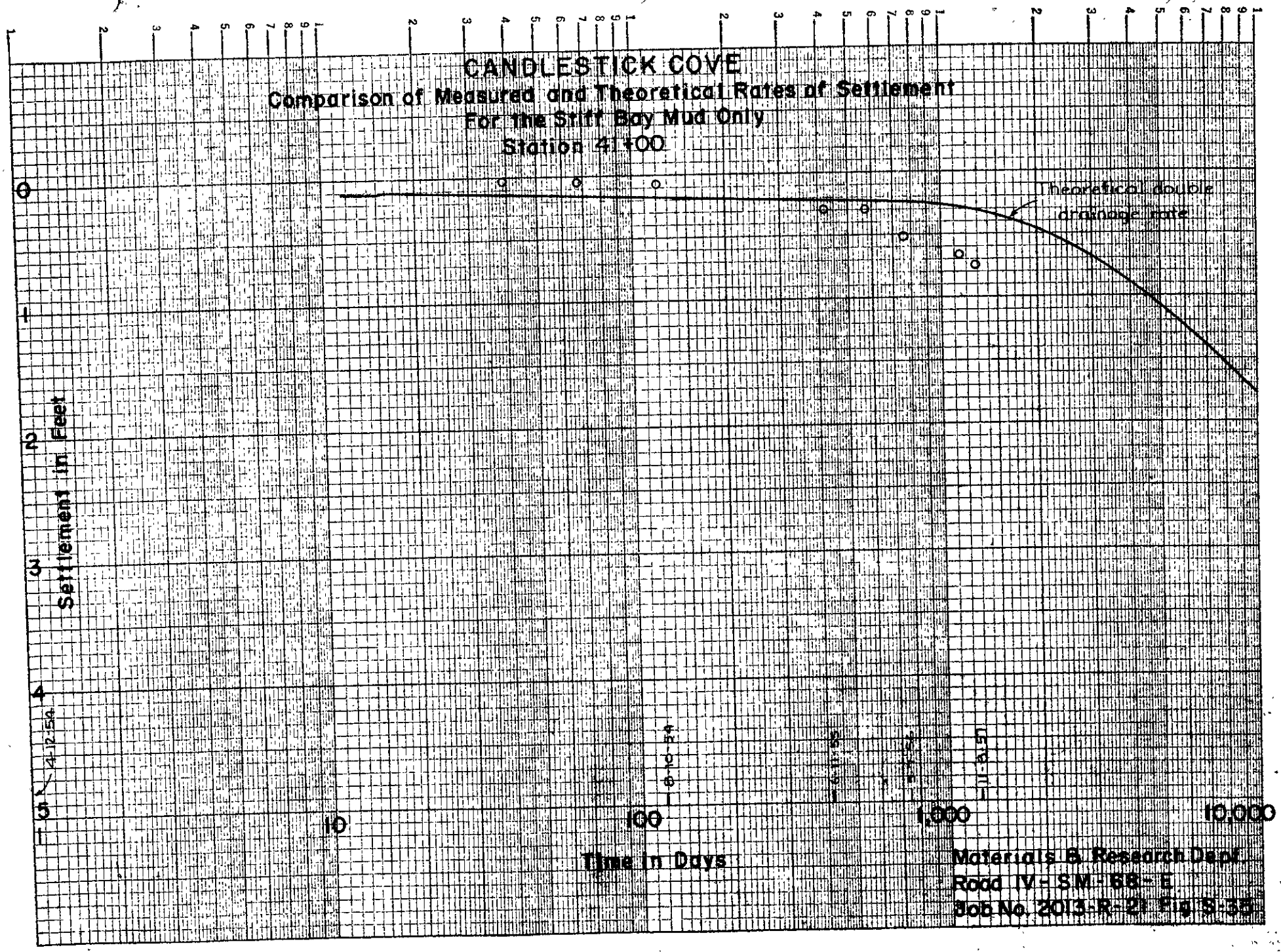


o 98+00 Top of Working Platform, Settlement During Construction 0.24 ft.

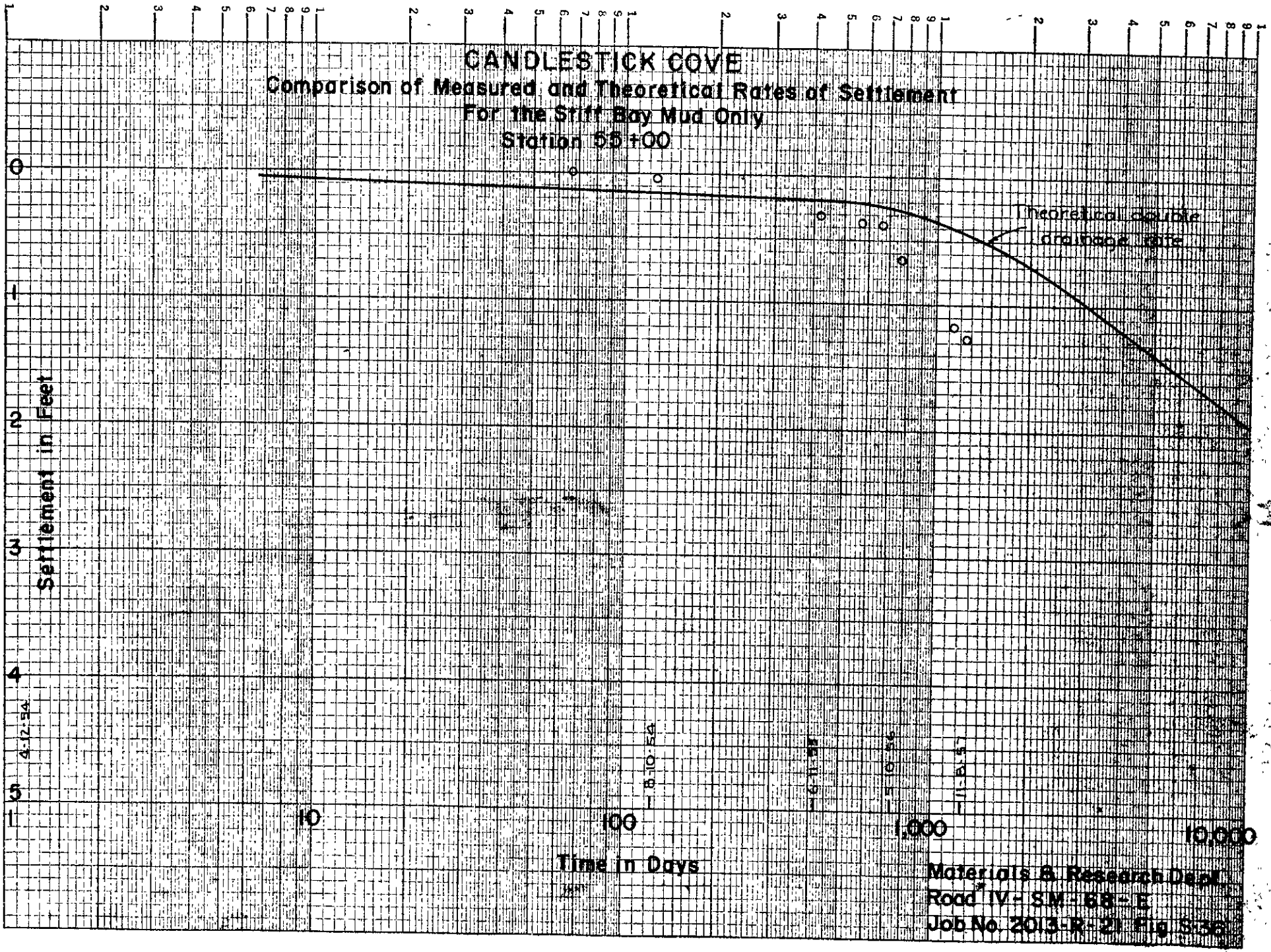
x 99+00 " " " 0.17 ft.

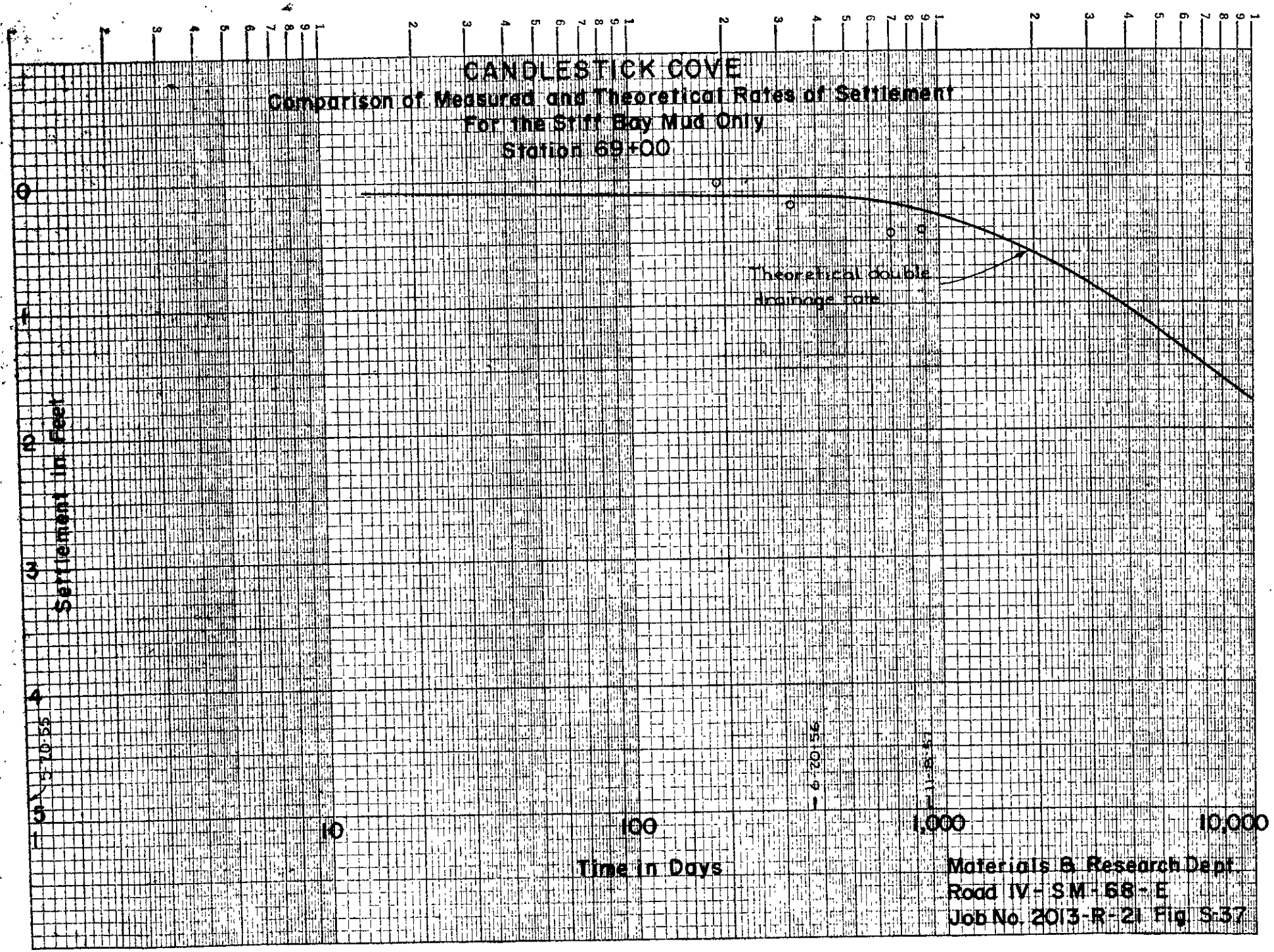
Station	Elev. Bottom of Fill	Elev. Bottom of Soft Mud
98+00	-47	-46
99+00	-47	-47

Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. S-34

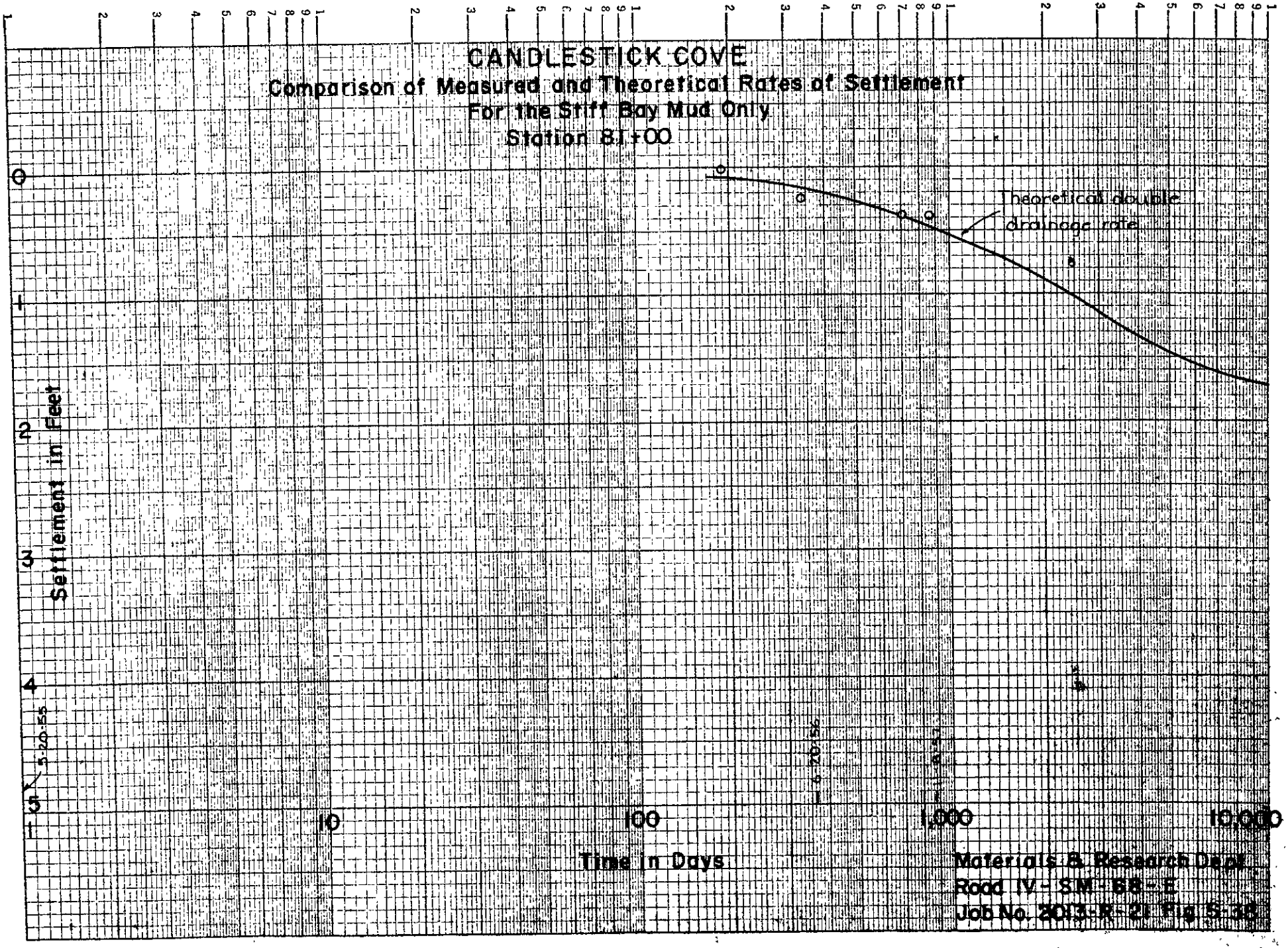

CANDLESTICK COVE
Comparison of Measured and Theoretical Rates of Settlement
For the Stiff Bay Mud Only
Station 41+00
Theoretical double drainage rate
Settlement in Feet
Time in Days
0
1
2
3
4
5
10
100
1,000
10,000
4-12-54
6-10-54
9-11-55
7-8-57
5-9-11-57
Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig S-35

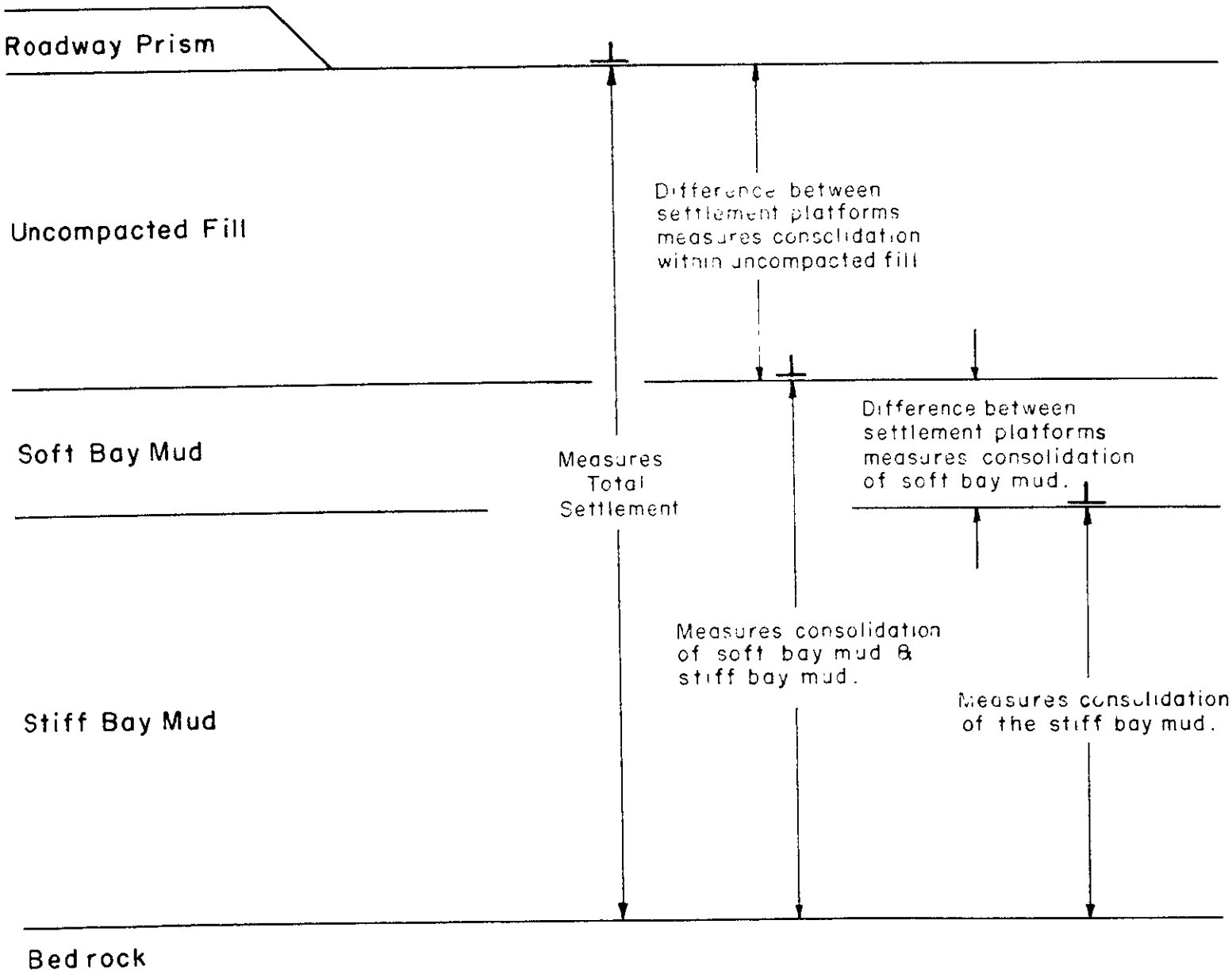
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



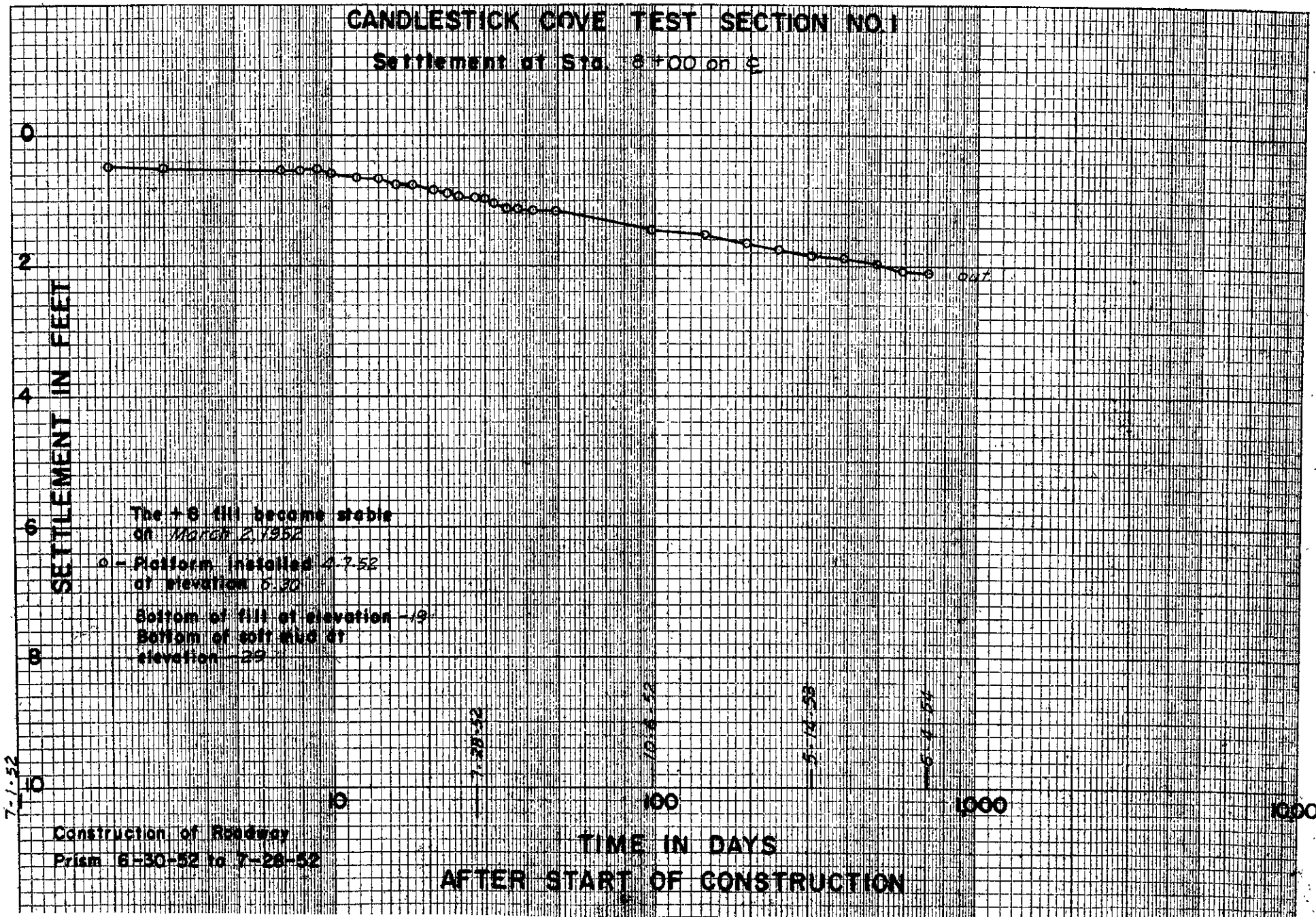

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE
Comparison of Measured and Theoretical Rates of Settlement
For the Stiff Bay Mud Only
Station 69+00
Theoretical double drainage rate
Settlement in Feet
Time in Days
10
100
1,000
10,000
Materials & Research Dept.
Road IV-SM-68-E
Job No. 2013-R-21 Fig. S-37

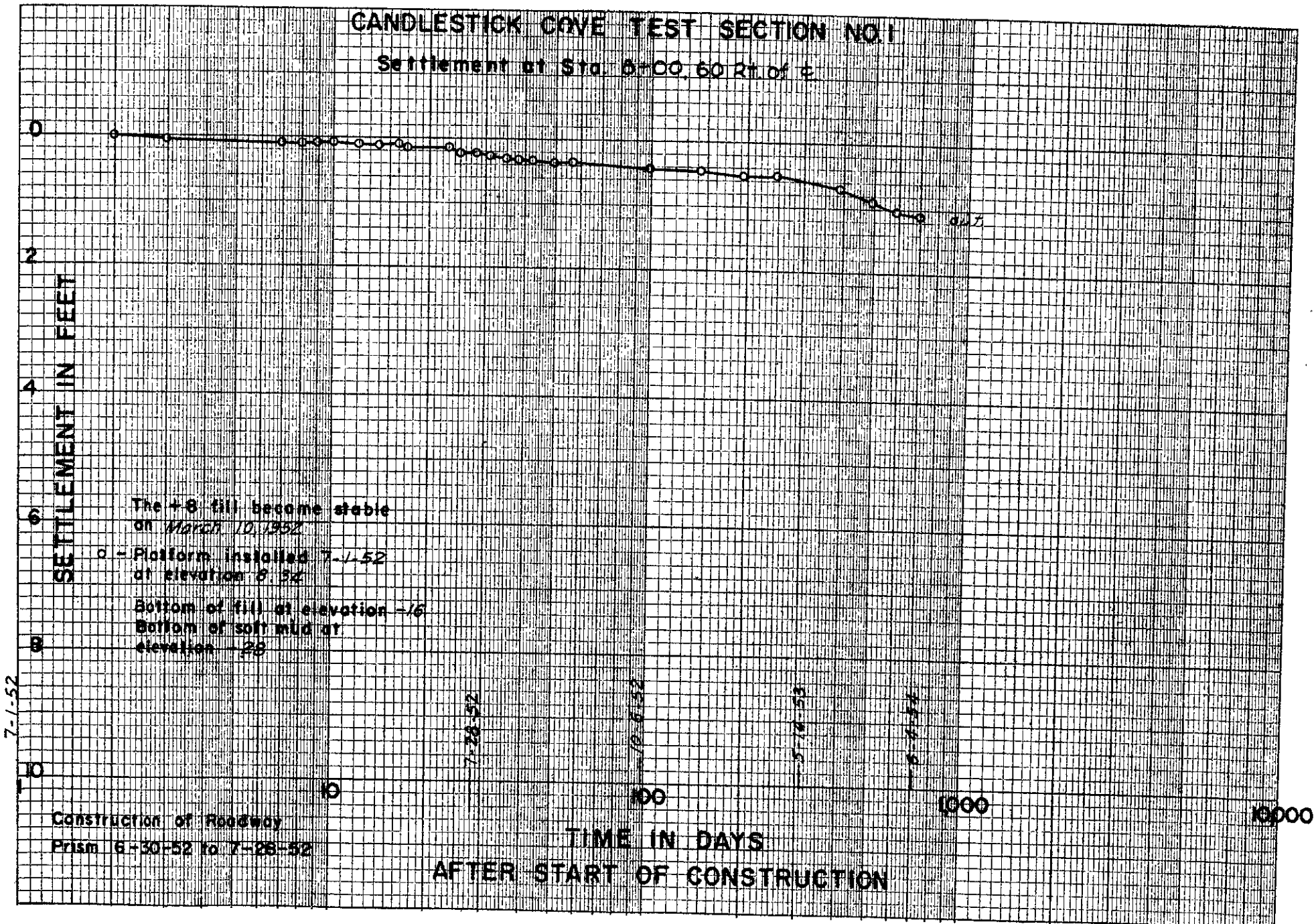
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

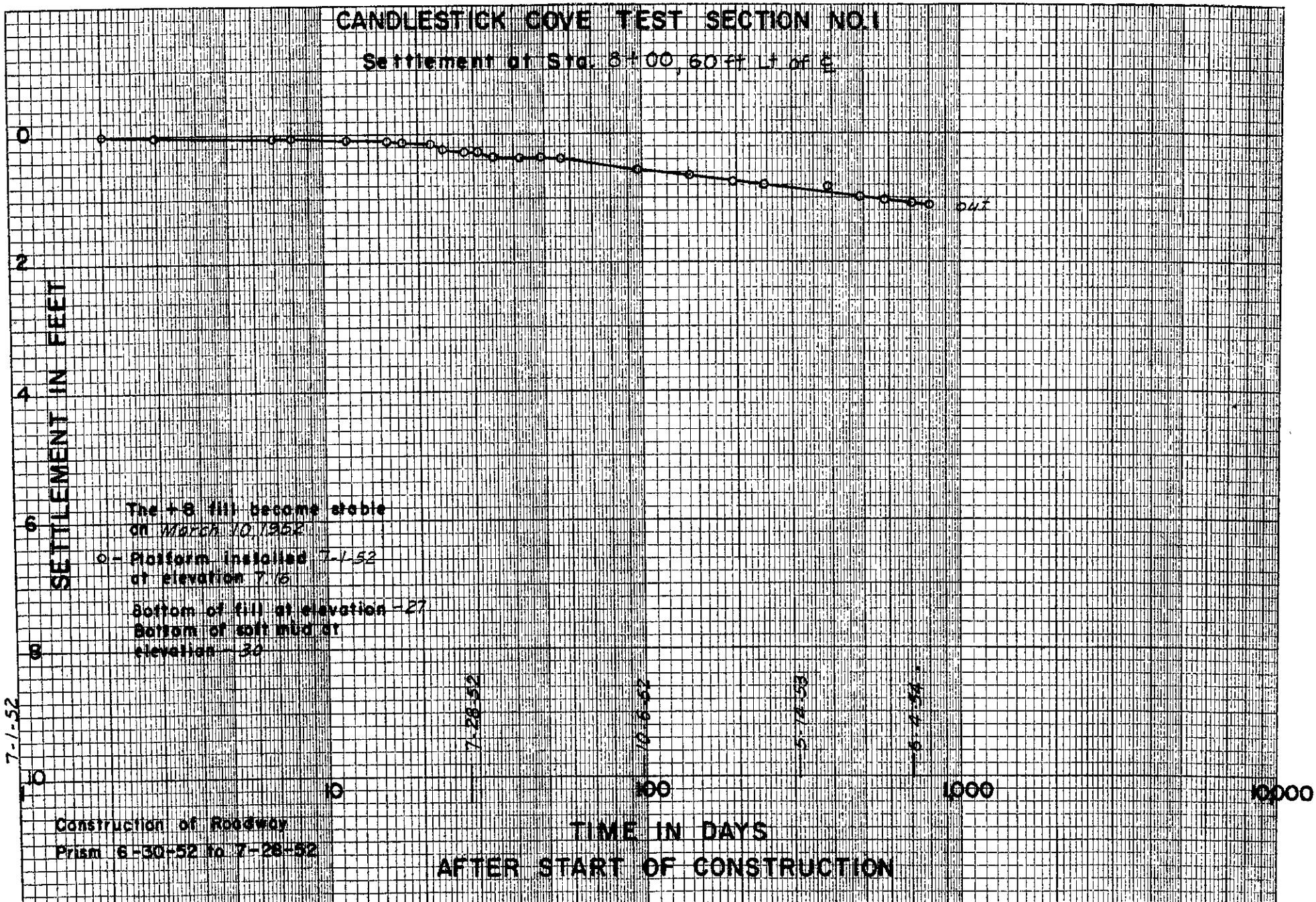


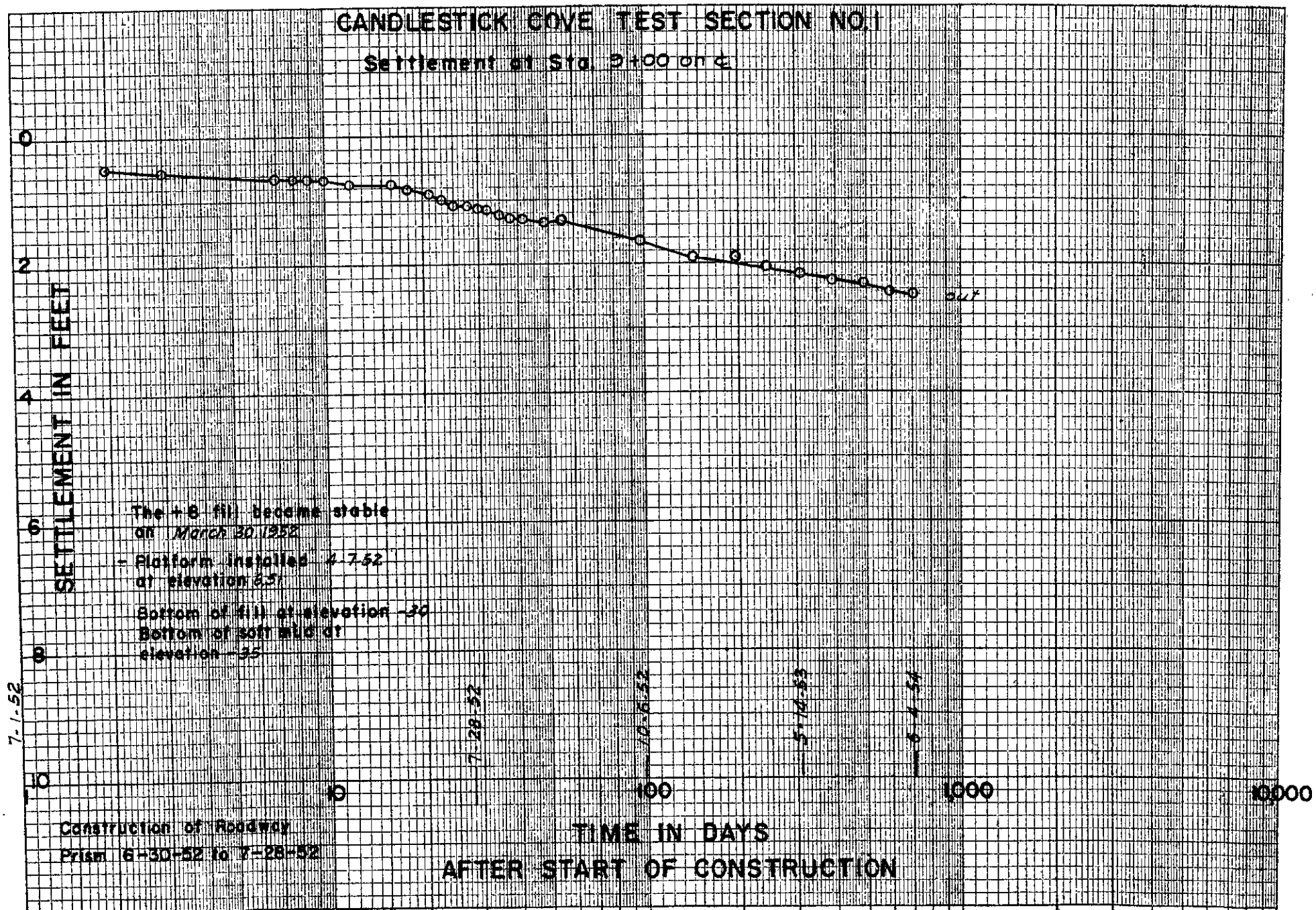


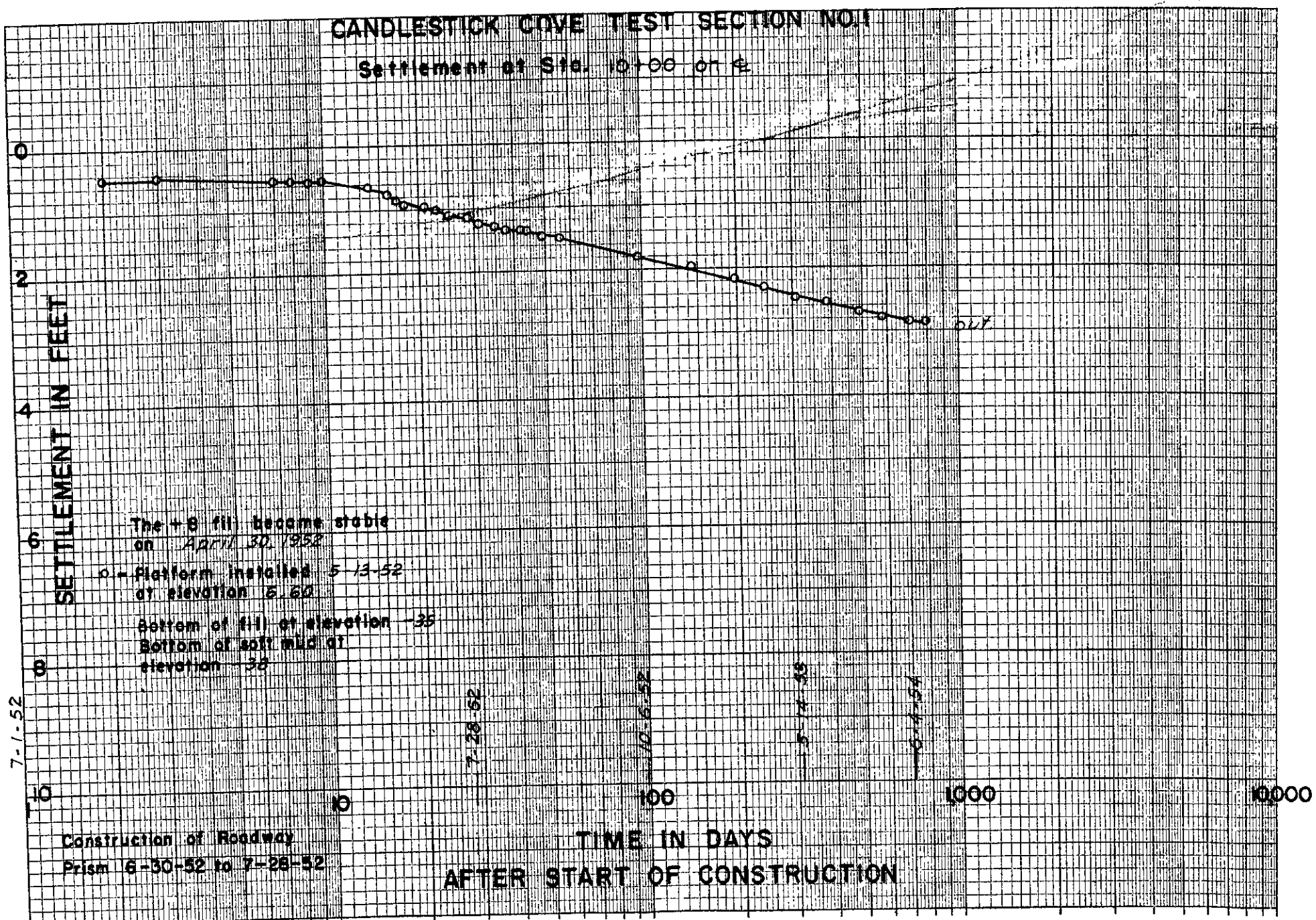
Materials & Research Dept.
Road IV - SM - 58 - E
Job No. 2013 - R - 21 Fig. S-39

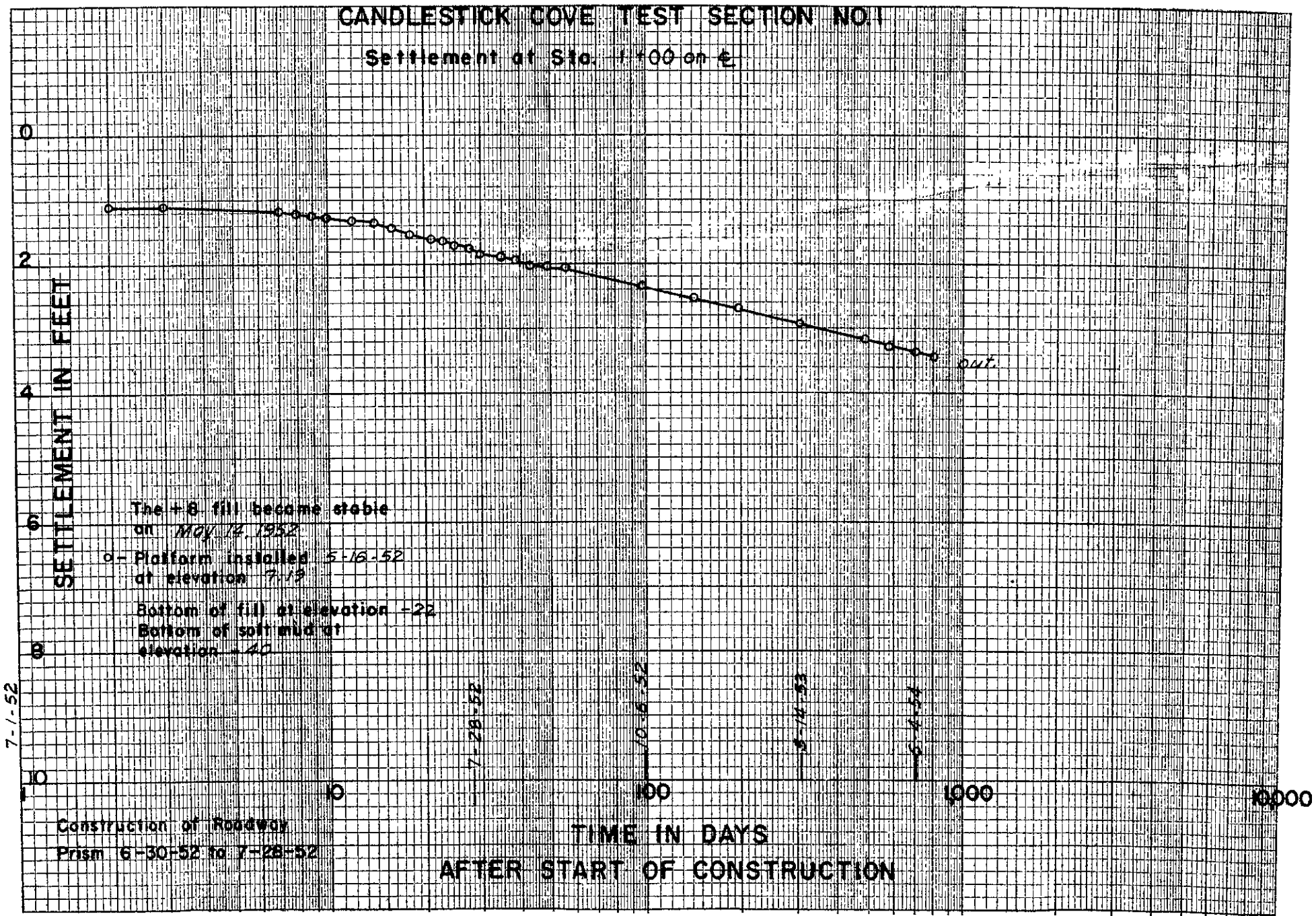

CANDLESTICK COVE TEST SECTION NO. 1
Settlement at Sta. 8+00 on ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
out
The +8 fill became stable on March 2, 1952
o – Platform installed 4-7-52 at elevation 5.30
Bottom of fill at elevation -19
Bottom of soft mud at elevation -29
7-1-52
7-28-52
10-8-52
5-12-53
6-4-54
10
100
1000
10000
Construction of Roadway Prism 6-30-52 to 7-28-52
TIME IN DAYS
AFTER START OF CONSTRUCTION

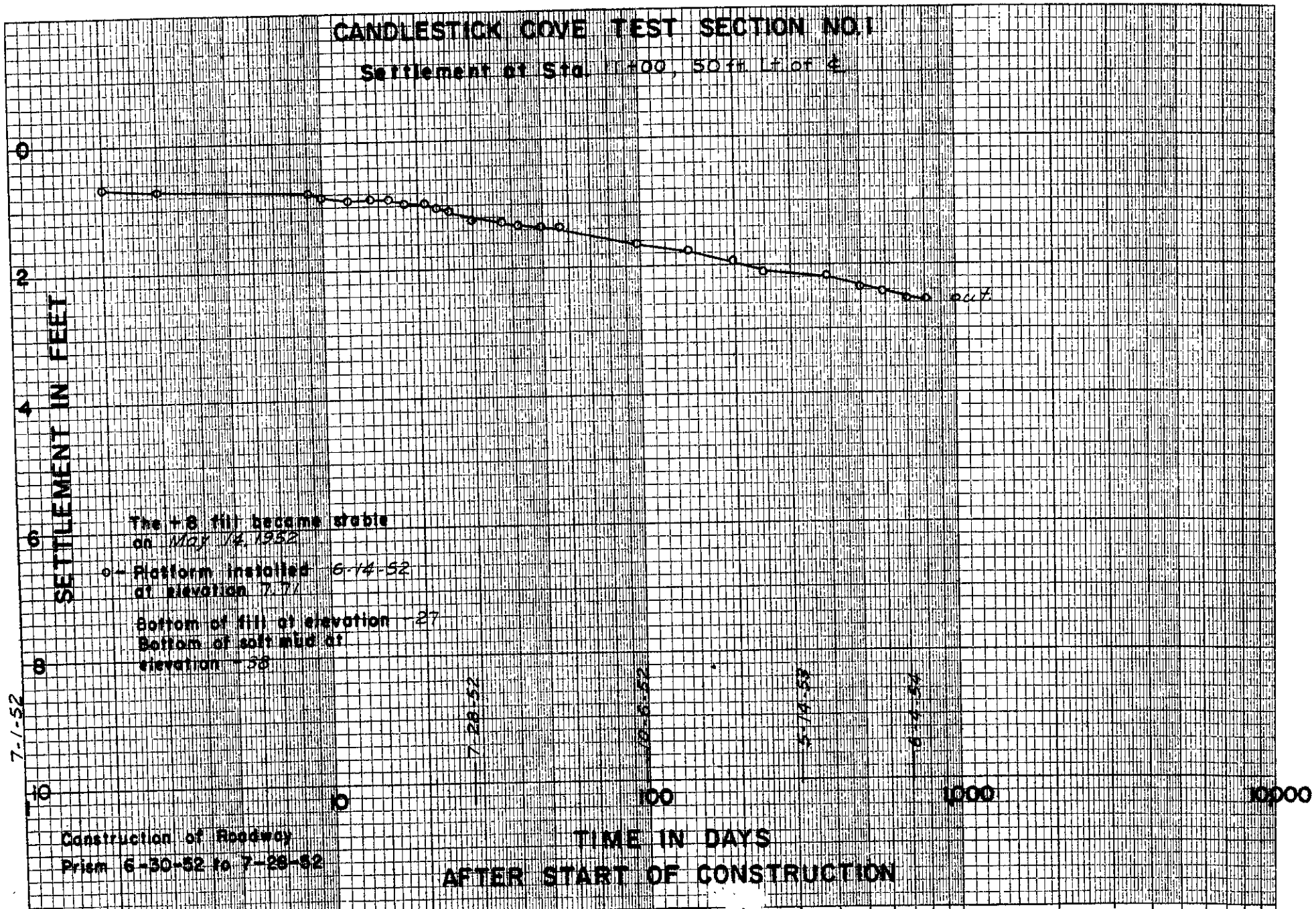

CANDLESTICK COVE TEST SECTION NO. 1
Settlement at Sta. 6+00, 60 Rt. of ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
The +8 fill became stable on March 10, 1952
o - Platform installed 7-1-52 at elevation 8.32
Bottom of fill at elevation -16
Bottom of soft mud at elevation -28
7-1-52
7-28-52
10-6-52
5-14-53
6-4-54
Construction of Roadway Prism 6-30-52 to 7-28-52
1
10
100
1000
10,000
TIME IN DAYS
AFTER START OF CONSTRUCTION

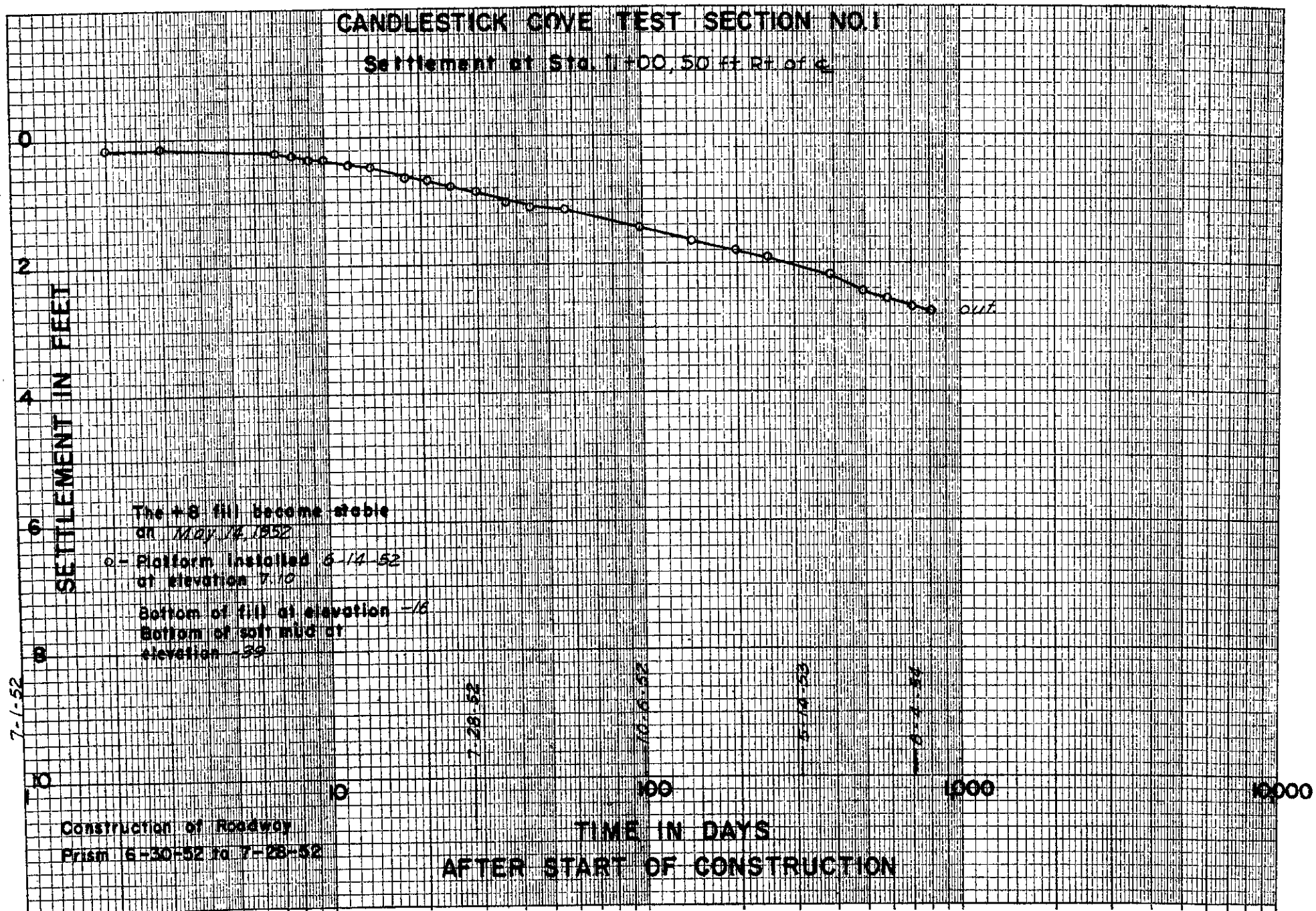

CANDLESTICK COVE TEST SECTION NO. 1
Settlement at Sta. 8+00, 60 ft. Lt. of ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
out
The +8 fill became stable on March 10, 1952
o - Platform installed 7-1-52 at elevation 7.16
Bottom of fill at elevation -27
Bottom of soft mud at elevation -30
7-1-52
7-28-52
10-6-52
5-14-53
6-4-54
1
10
100
1000
10000
TIME IN DAYS
AFTER START OF CONSTRUCTION
Construction of Roadway Prism 6-30-52 to 7-28-52

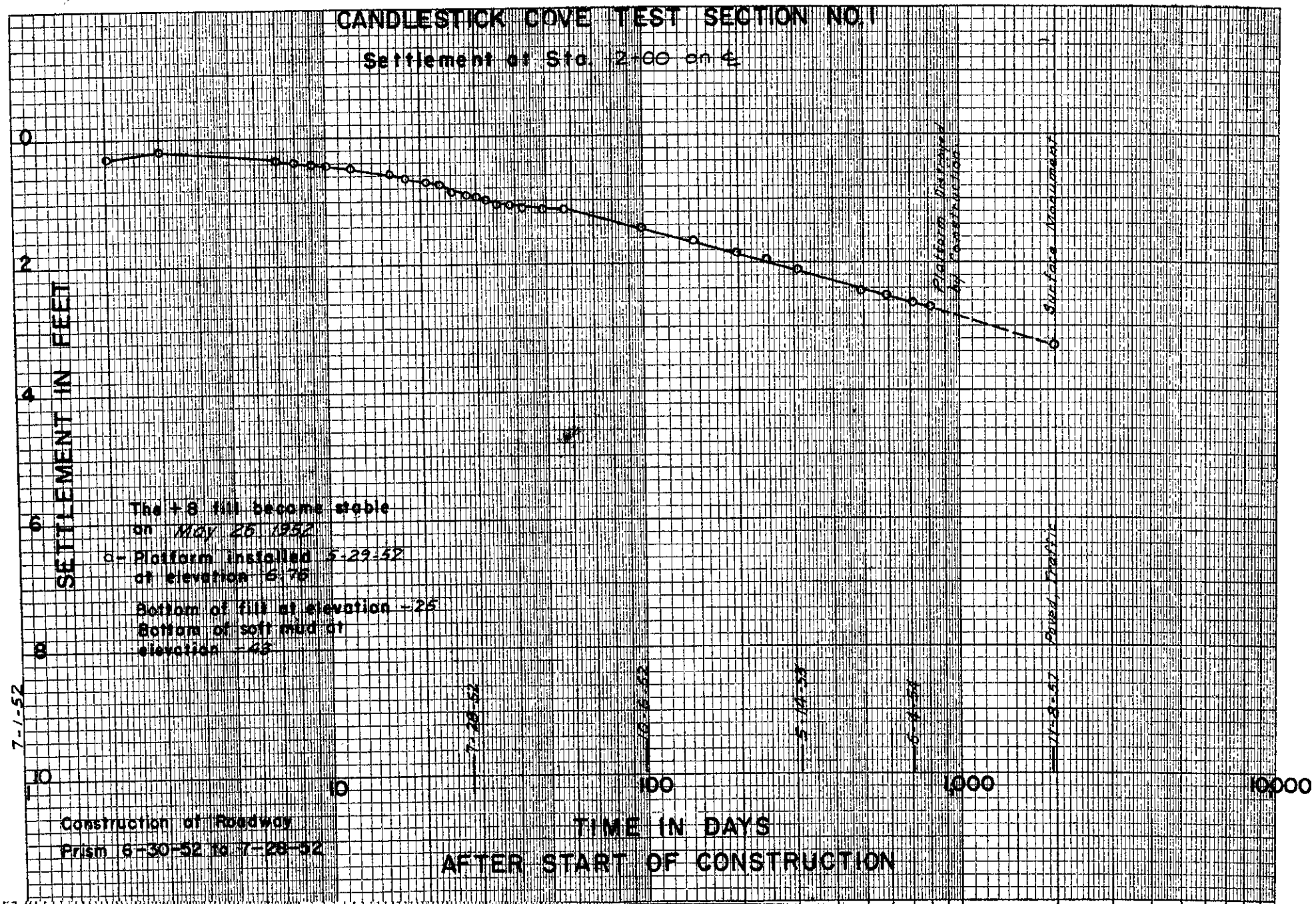

CANDLESTICK COVE TEST SECTION NO. 1
Settlement at Sta. 3+00 on ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
The +8 fill became stable on March 30, 1952
Platform installed 4-7-52 at elevation 8.5'
Bottom of fill at elevation -30
Bottom of soft mud at elevation -35
7-1-52
7-28-52
10-6-52
5-14-53
6-4-54
10
100
1000
10,000
Construction of Roadway Prism 6-30-52 to 7-28-52
TIME IN DAYS
AFTER START OF CONSTRUCTION

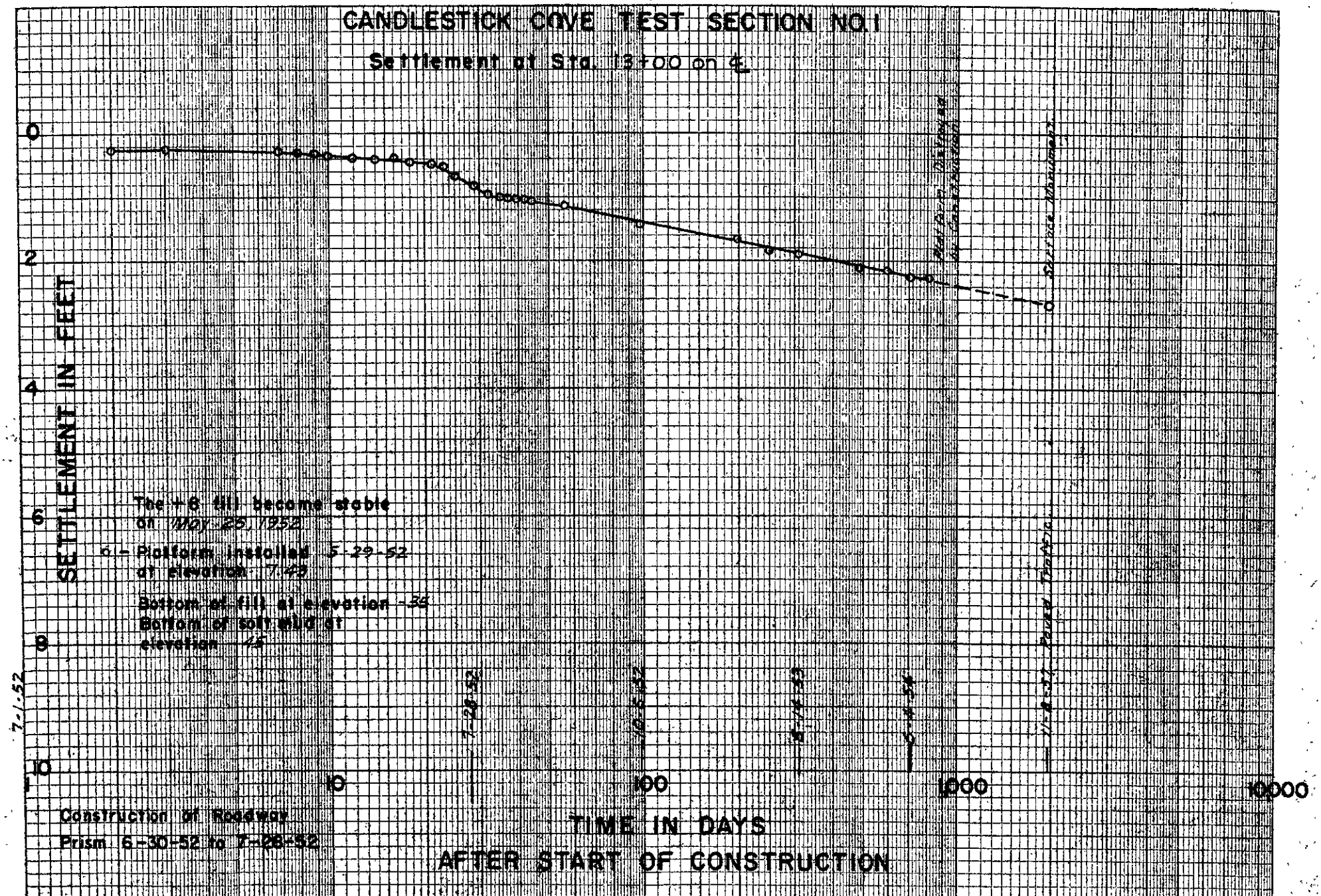

CANDLESTICK COVE TEST SECTION NO.1
Settlement at Sta. 10+00 on ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
The + 8 fill became stable on April 30, 1952
o - Platform installed 5-13-52 at elevation 5.60
Bottom of fill at elevation -35
Bottom of soft mud at elevation -38
7-1-52
7-28-52
10-6-52
5-14-53
10-4-54
out
10
100
1000
10000
TIME IN DAYS
AFTER START OF CONSTRUCTION
Construction of Roadway Prism 6-30-52 to 7-28-52

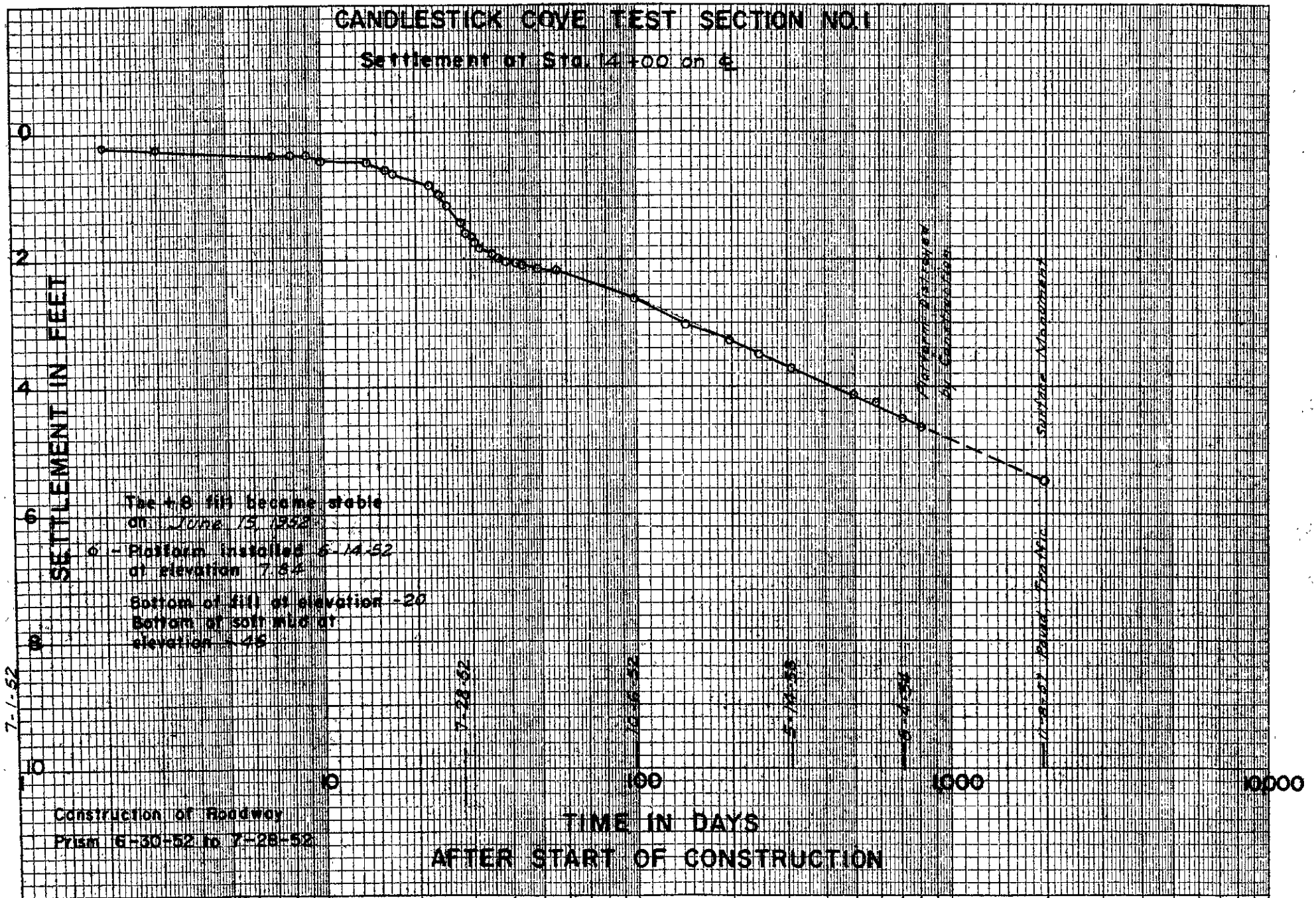

CANDLESTICK COVE TEST SECTION NO. 1
Settlement at Sta. 11+00 on ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
1
10
100
1000
10,000
TIME IN DAYS
AFTER START OF CONSTRUCTION
out.
The +8 fill became stable on May 14, 1952
o – Platform installed 5-16-52 at elevation 7.13
Bottom of fill at elevation -22
Bottom of soft mud at elevation -40
7-29-52
10-6-52
5-14-53
6-4-54
7-1-52
Construction of Roadway Prism 6-30-52 to 7-28-52

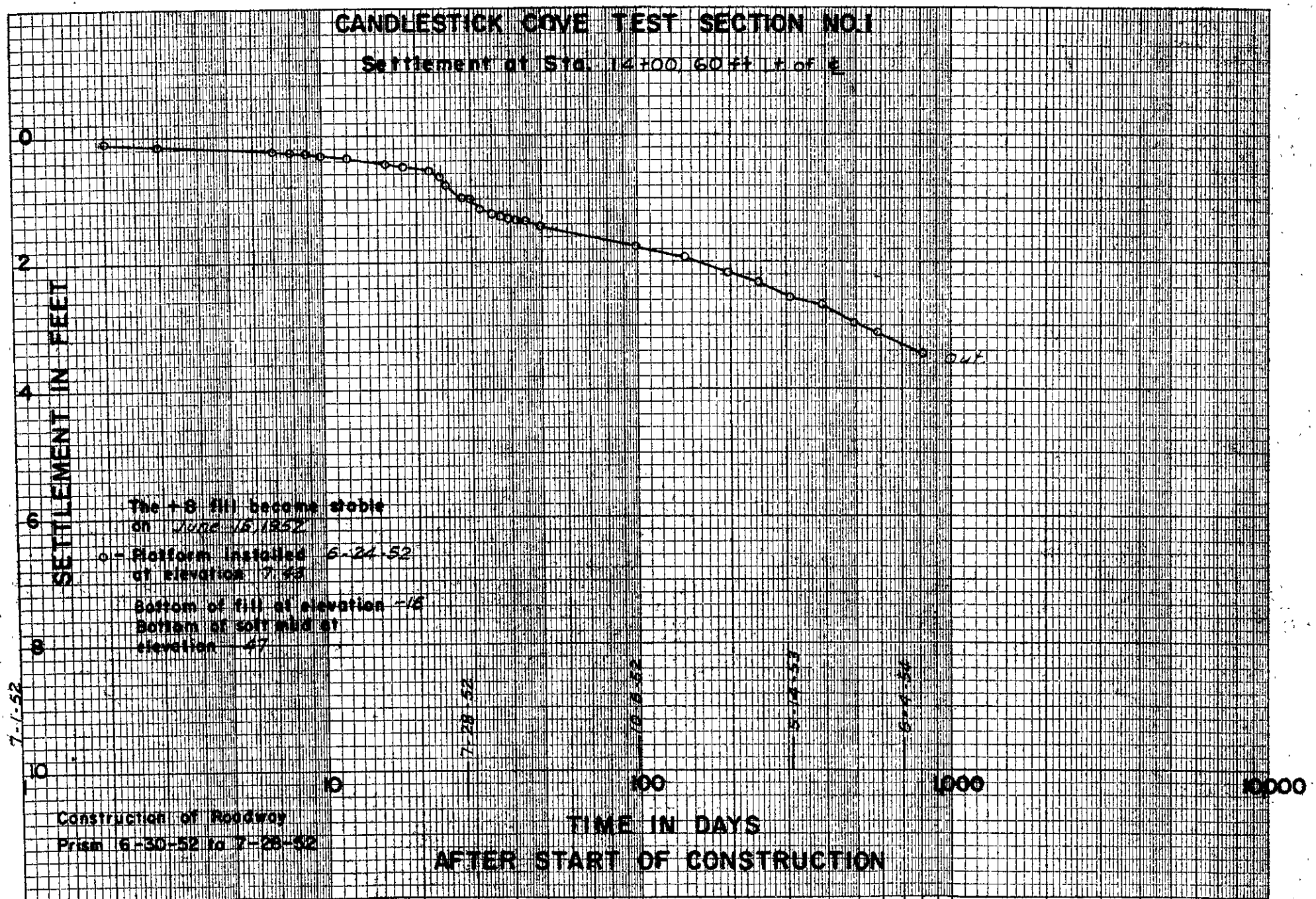

CANDLESTICK COVE TEST SECTION NO. 1
Settlement at Sta. 11+00, 50 ft. Lt. of ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
The +8 fill became stable on May 14, 1952
o – Platform installed 6-14-52 at elevation 7.71
Bottom of fill at elevation -27
Bottom of soft mud at elevation -38
7-1-52
7-28-52
10-6-52
3-14-53
6-4-54
10
100
1000
10,000
Construction of Roadway Prism 6-30-52 to 7-28-52
TIME IN DAYS
AFTER START OF CONSTRUCTION

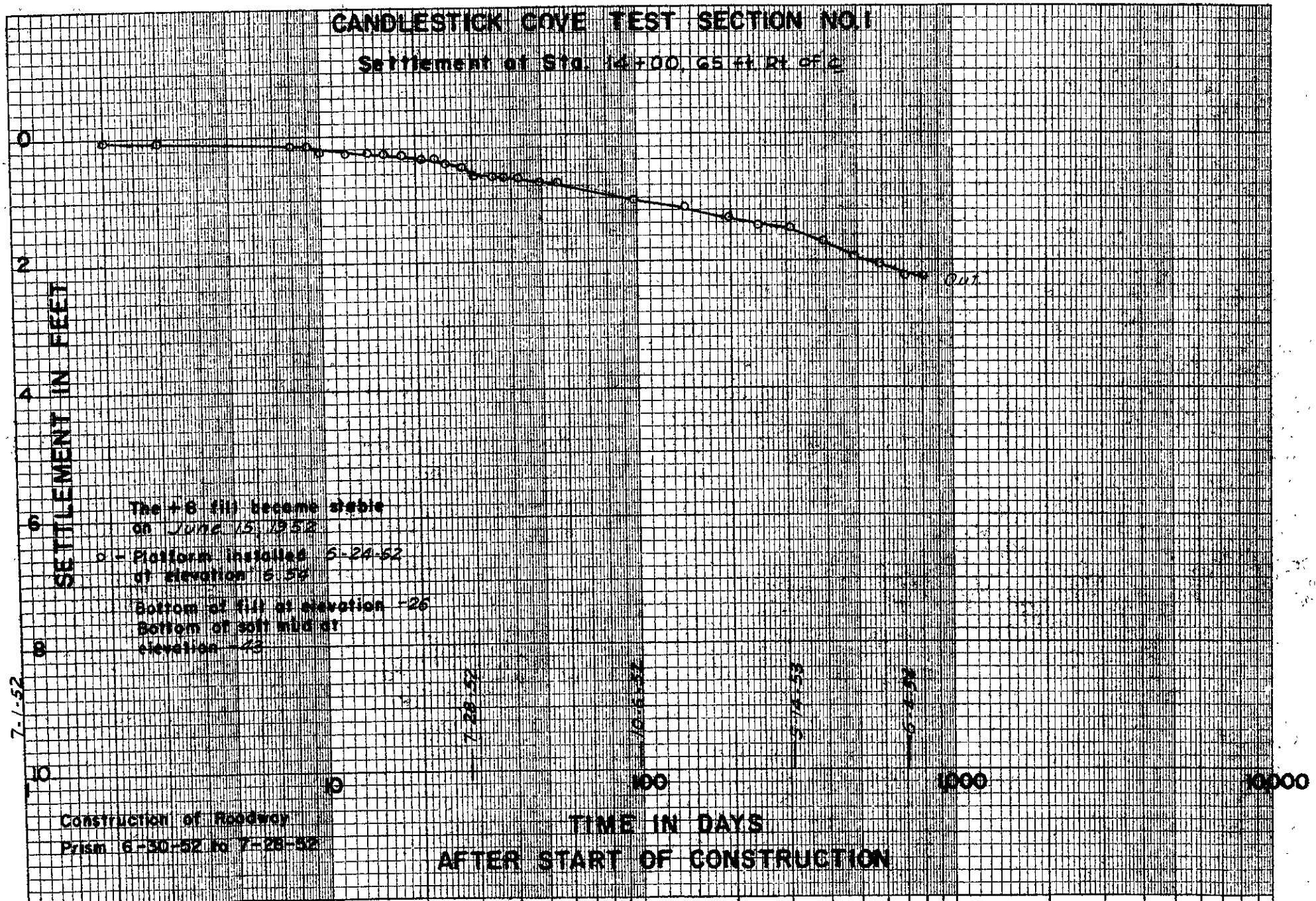

CANDLESTICK COVE TEST SECTION NO. 1
Settlement at Sta. 11+00, 50 ft. Rt. of ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
out
The + 8 fill became stable on May 14, 1952
o - Platform installed 6-14-52 at elevation 7.10
Bottom of fill at elevation -16
Bottom of soft mud at elevation -39
7-1-52
7-28-52
10-6-52
5-14-53
6-1-54
10
100
1000
10000
Construction of Roadway Prism 6-30-52 to 7-28-52
TIME IN DAYS
AFTER START OF CONSTRUCTION

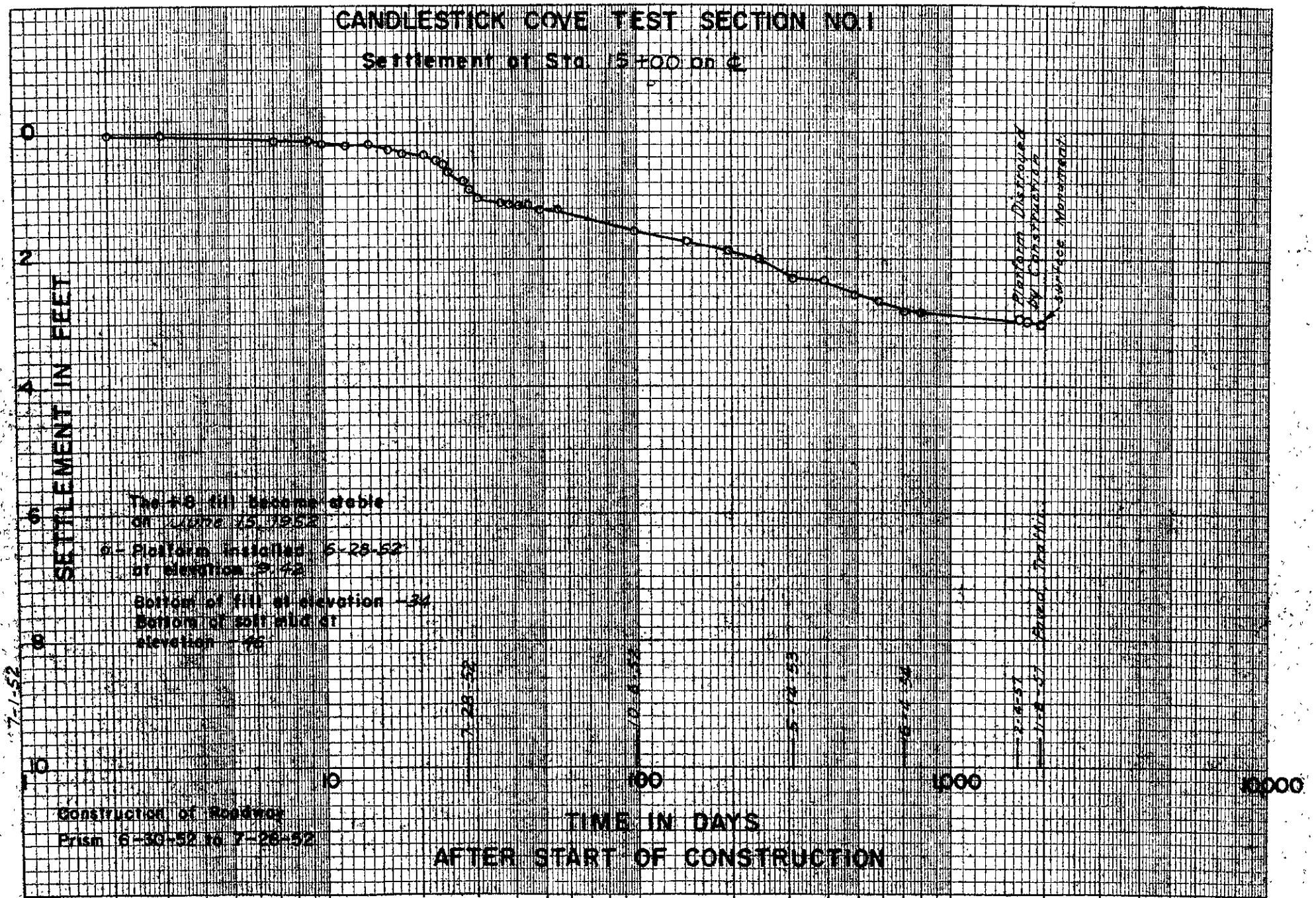

CANDLESTICK COVE TEST SECTION NO. 1
Settlement at Sta. 2+00 on ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
The +8 fill became stable on May 25 1952
a - Platform installed 5-29-52 at elevation 6.75
Bottom of fill at elevation -25
Bottom of soft mud at elevation -43
7-1-52
7-28-52
10-6-52
5-14-53
6-8-54
11-8-57
Paved Traffic
Surface Monument
Platform Destroyed by reconstruction
10
100
1000
10000
TIME IN DAYS
AFTER START OF CONSTRUCTION
Construction of Roadway Prism 6-30-52 to 7-28-52

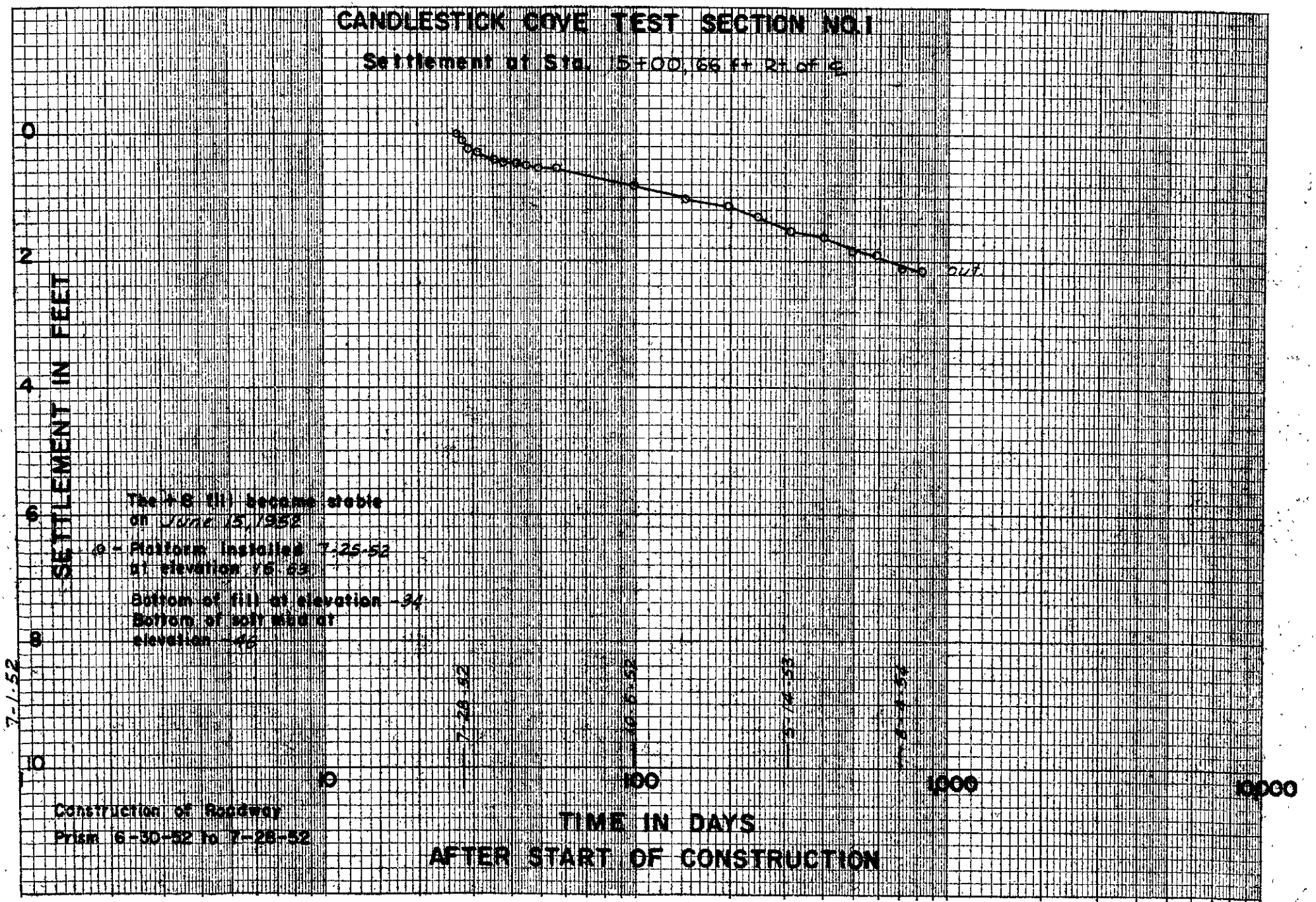

CANDLESTICK COVE TEST SECTION NO. 1
Settlement at Sta. 13+00 on ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
The +6 fill became stable on May 25, 1952
6 – Platform installed 5-29-52 at elevation 7.42
Bottom of fill at elevation -34
Bottom of soft mud at elevation -45
7-1-52
7-28-52
10-5-52
11-8-52
Paved Traffic
Surface Measurements
Road Bed Placed by Contractor
1
10
100
1000
10000
Construction of Roadway Prism 6-30-52 to 7-28-52
TIME IN DAYS
AFTER START OF CONSTRUCTION

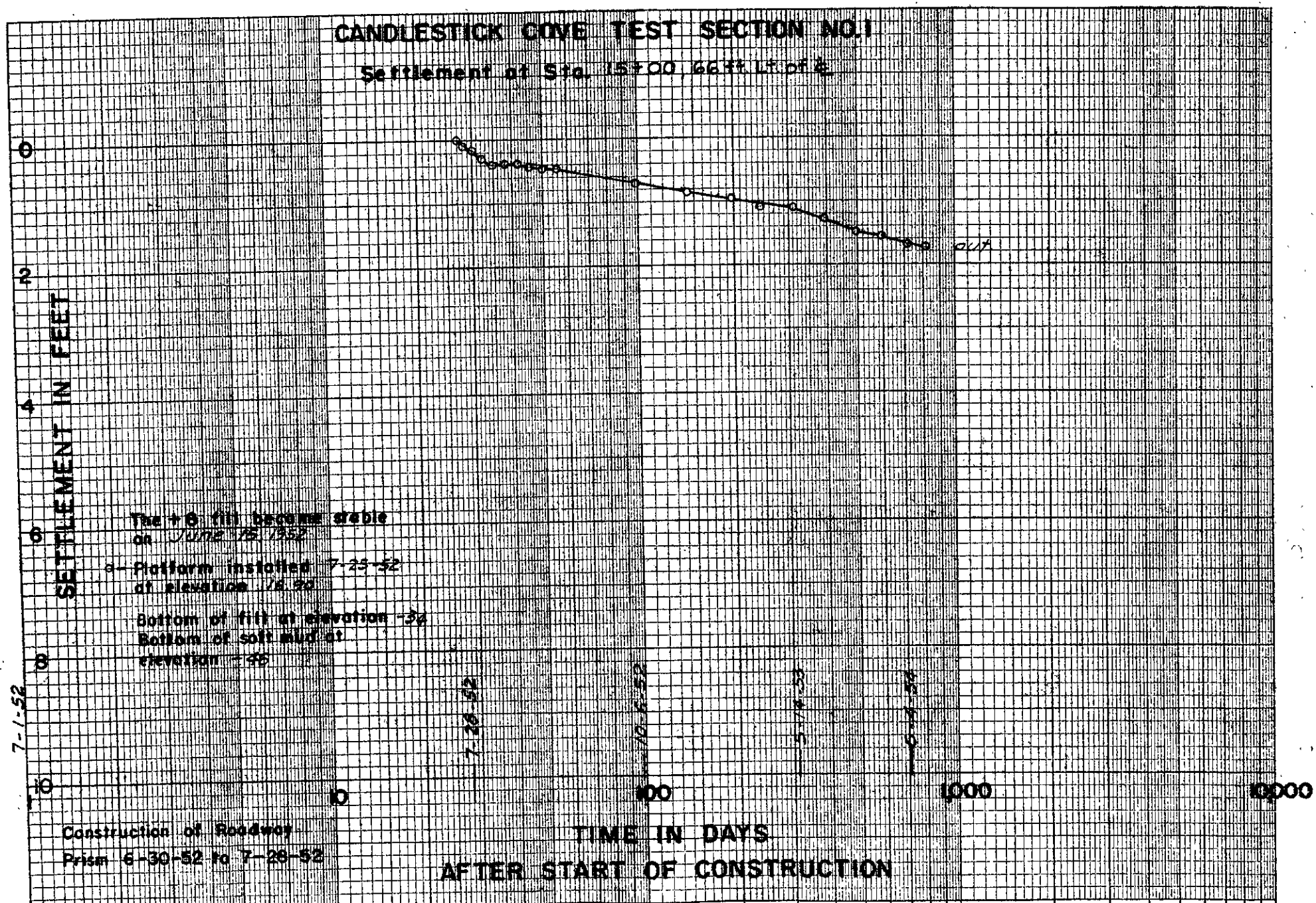

CANDLESTICK COVE TEST SECTION NO.1
Settlement at Sta. 14+00 on ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
0
100
1000
10000
TIME IN DAYS
AFTER START OF CONSTRUCTION
The + 6 fill became stable on June 15, 1952
6 - Platform installed 6-14-52 at elevation 7.82
Bottom of fill at elevation -20
Bottom of soft mud at elevation -4.45
Construction of Roadway Prism 6-30-52 to 7-28-52
7-1-52
7-28-52
10-16-52
5-14-53
6-4-54
Platform Installed by Contractor
Surface Movement
U.P.R.R. Road Profile

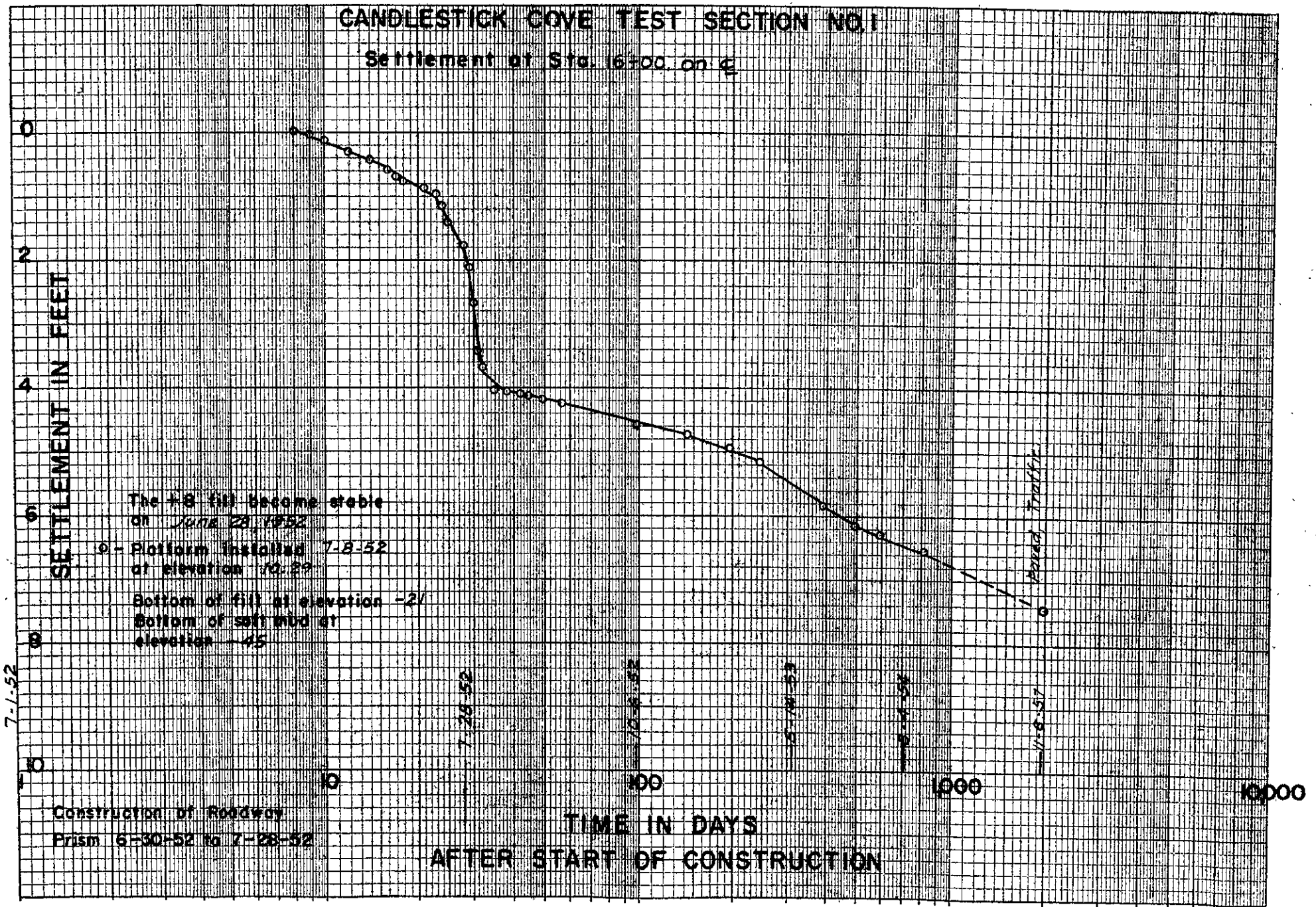

CANDLESTICK COVE TEST SECTION NO.1
Settlement at Sta. 14+00, 60 ft. lt. of ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
The +8 fill became stable on June 16 1952
o - Platform Installed 6-24-52 at elevation 7.43
Bottom of fill at elevation -16
Bottom of soft mud at elevation -47
7-1-52
7-29-52
10-6-52
5-14-53
6-4-54
Out
1
10
100
1000
10000
TIME IN DAYS
AFTER START OF CONSTRUCTION
Construction of Roadway Prism 6-30-52 to 7-28-52

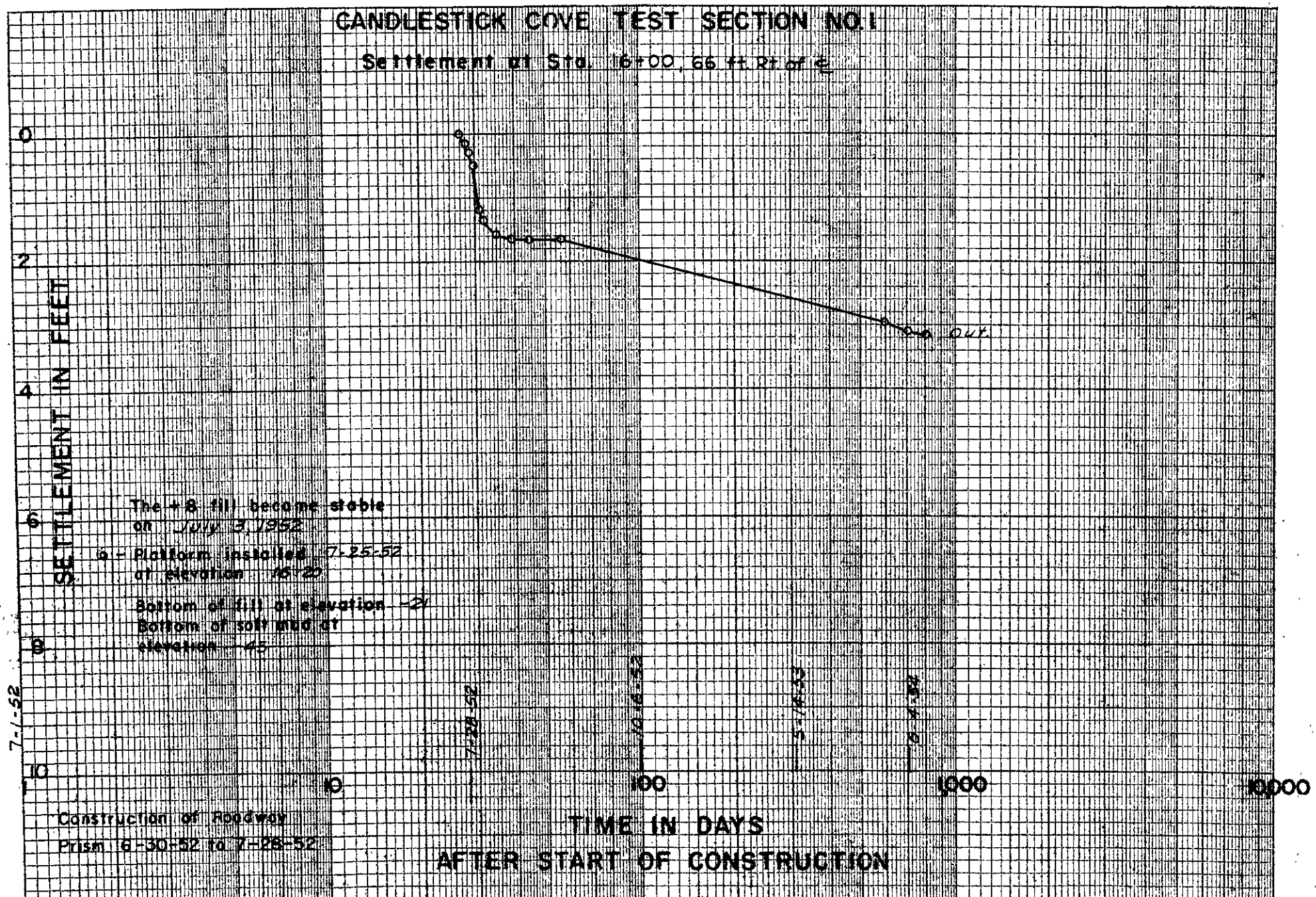

CANDLESTICK COVE TEST SECTION NO. 1
Settlement at Sta. 14+00, 65 ft. Rt. of ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
Out
The +8 fill became stable on June 15, 1952
o - Platform installed 5-24-52 at elevation 6.52
Bottom of fill at elevation -26
Bottom of soft mud at elevation -43
7-1-52
7-28-52
10-8-52
3-14-53
6-4-53
10
100
1000
10000
Construction of Roadway Prism 6-30-52 to 7-28-52
TIME IN DAYS
AFTER START OF CONSTRUCTION

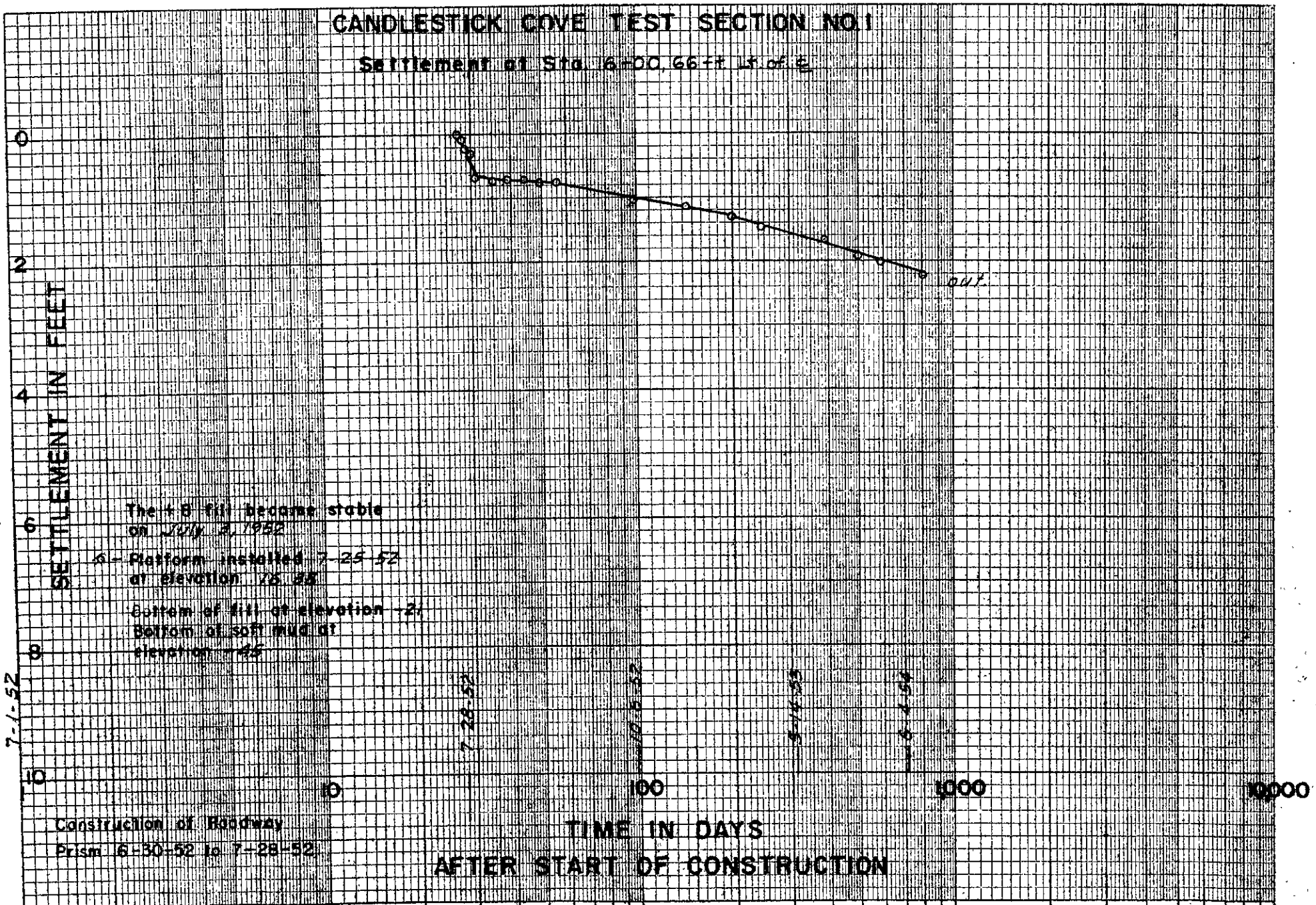

CANDLESTICK COVE TEST SECTION NO. 1
Settlement at Sta. 15+00 on ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
The +8 fill became stable on August 15, 1952
Platform installed 6-28-52 at elevation 9.42
Bottom of fill at elevation -34
Bottom of soft mud at elevation -46
7-1-52
7-28-52
10-4-52
5-14-53
6-4-54
2-4-57
1-4-57
Platform Destroyed by Construction
Surface Monument
Construction of Roadway Prism 6-30-52 to 7-28-52
10
100
1000
10000
TIME IN DAYS
AFTER START OF CONSTRUCTION

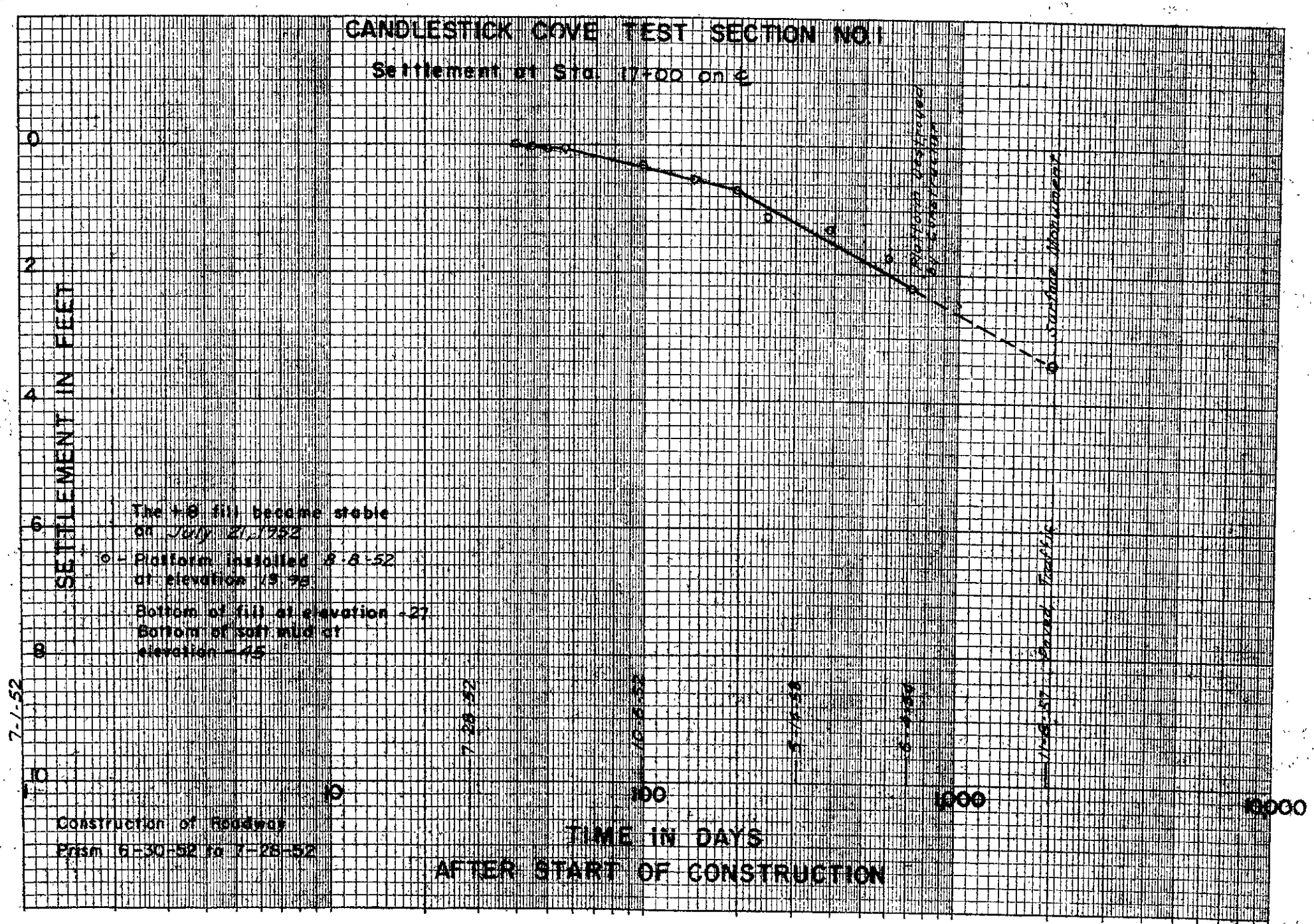

CANDLESTICK COVE TEST SECTION NO.1
Settlement at Sta. 15+00, 66 ft. Rt. of ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
10
100
1000
10,000
TIME IN DAYS
AFTER START OF CONSTRUCTION
7-1-52
7-28-52
10-5-52
3-14-53
6-14-53
out.
Toe + 8 fill became stable on June 15, 1952
Platform installed 7-25-52 at elevation 16.63
Bottom of fill at elevation -34
Bottom of soft mud at elevation -46
Construction of Roadway Prism 6-30-52 to 7-28-52


CANDLESTICK COVE TEST SECTION NO. 1
Settlement at Sta. 15+00, 66 ft. Lt. of ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
out
The + 8 fill became stable on JUNE 15, 1952
Platform installed 7-25-52 at elevation 16.90
Bottom of fill at elevation -34
Bottom of soft mud at elevation -46
7-1-52
7-26-52
10-6-52
5-19-53
9-19-53
1
10
100
1000
10000
Construction of Roadway Prism 6-30-52 to 7-26-52
TIME IN DAYS
AFTER START OF CONSTRUCTION

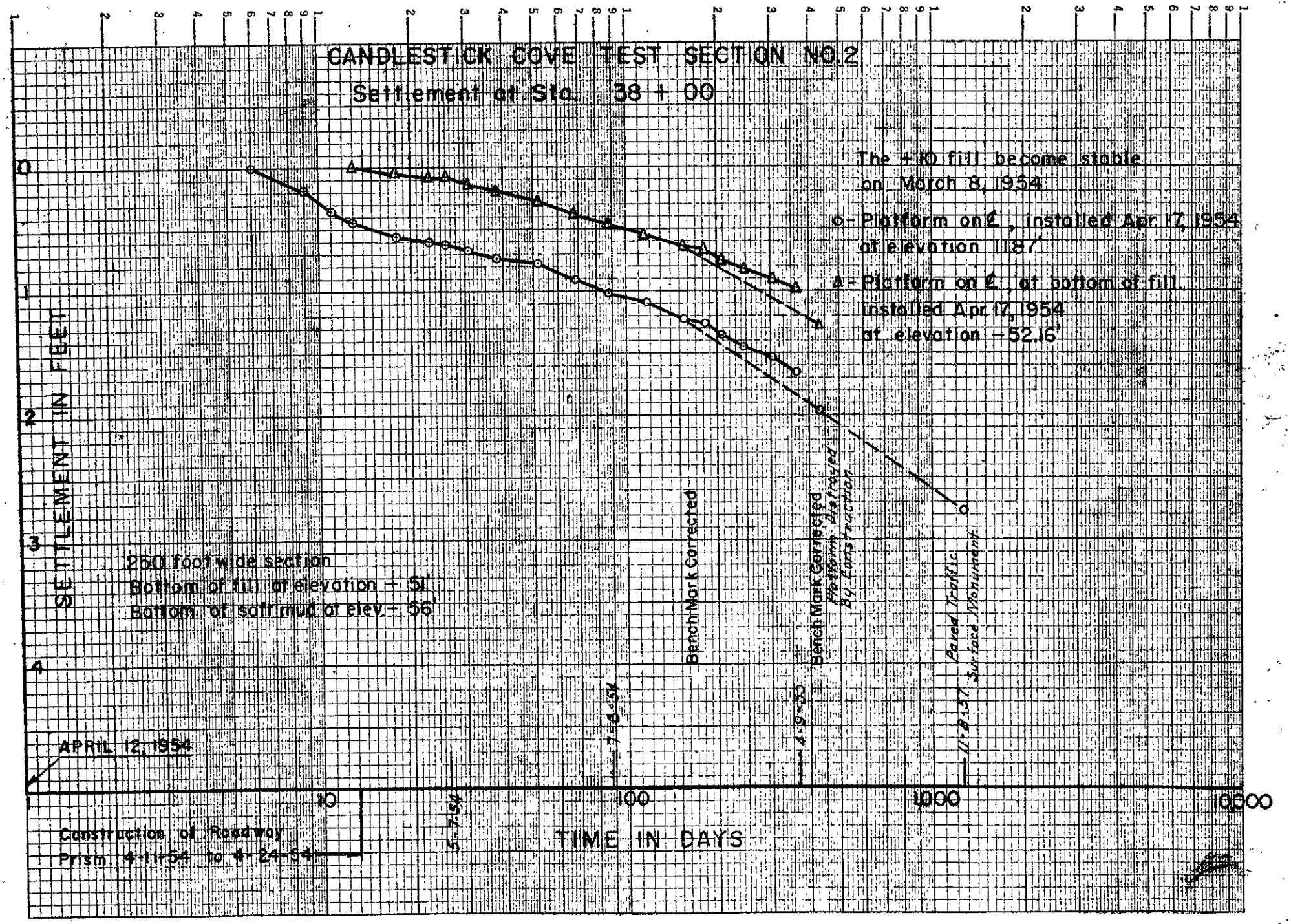

CANDLESTICK COVE TEST SECTION NO. 1
Settlement at Sta. 16+00 on ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
The +8 fill become stable on June 28, 1952
⊙ - Platform Installed 7-8-52 at elevation 10.22
Bottom of fill at elevation -21
Bottom of soft mud at elevation -45
7-1-52
7-29-52
10-6-52
5-14-53
6-14-54
11-8-57
Road Traffic
10
100
1000
10000
Construction of Roadway Prism 6-30-52 to 7-28-52
TIME IN DAYS
AFTER START OF CONSTRUCTION


CANDLESTICK COVE TEST SECTION NO. 1
Settlement at Sta. 16+00, 66 ft. Rt. of ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
The + 8 fill became stable on July 3, 1952
Platform installed 7-25-52 at elevation 16-20
Bottom of fill at elevation -21
Bottom of soft mud at elevation -45
7-1-52
7-28-52
10-8-52
5-14-53
8-4-53
out
Construction of Roadway Prism 6-30-52 to 7-28-52
10
100
1000
10000
TIME IN DAYS
AFTER START OF CONSTRUCTION

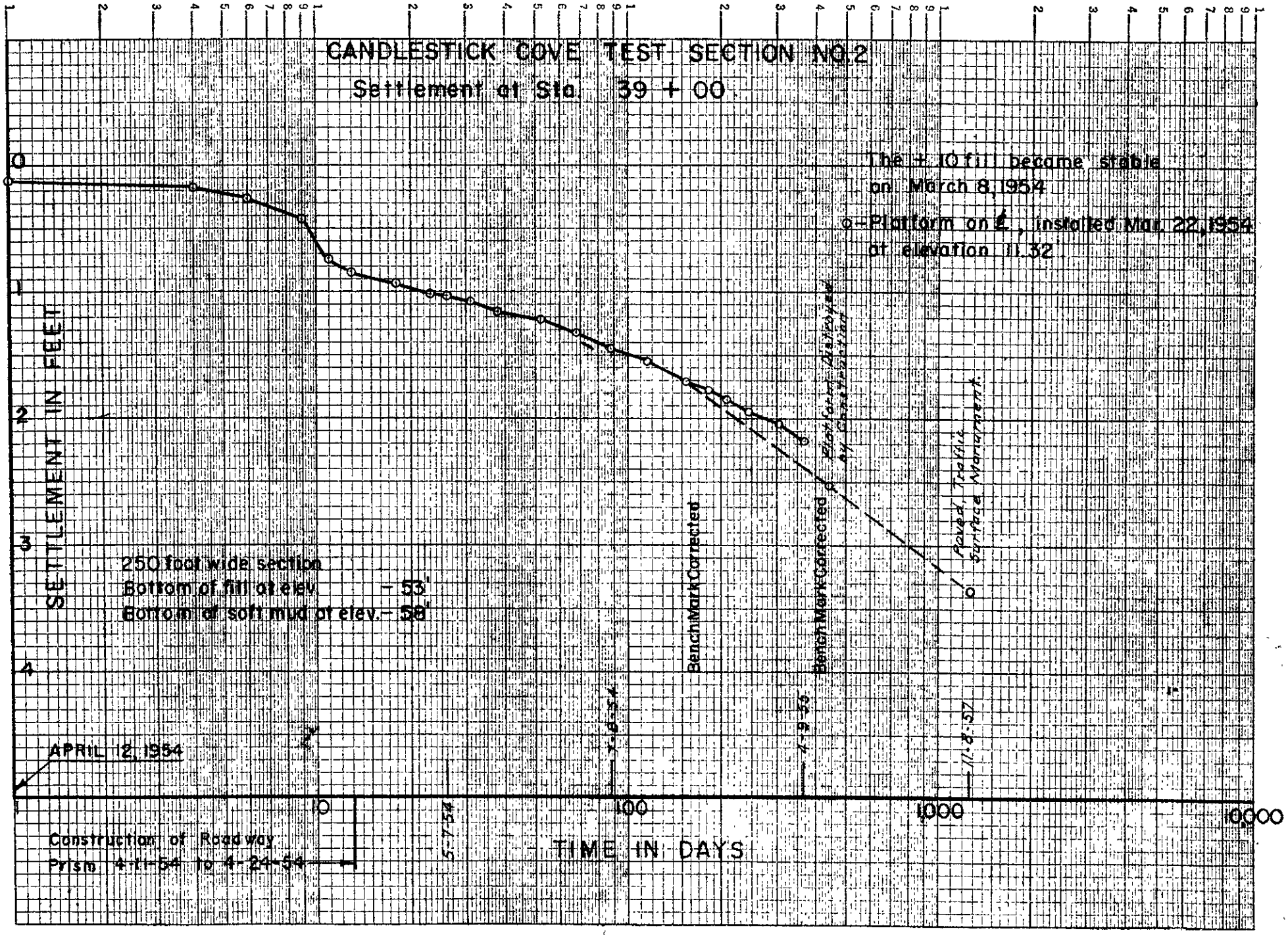

CANDLESTICK COVE TEST SECTION NO. 1
Settlement at Sta. 6+00, 66 ft Lt of ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
OUT
The 4 ft fill became stable on July 2, 1952
6 - Platform installed 7-25-52 at elevation 16.25
Bottom of fill at elevation -2
Bottom of soft mud at elevation -4.5
7-1-52
7-28-52
10
100
1000
10000
Construction of Roadway Prism 6-30-52 to 7-28-52
TIME IN DAYS
AFTER START OF CONSTRUCTION


CANDLESTICK COVE TEST SECTION NO. 1
Settlement at Sta. 17+00 on ℄
SETTLEMENT IN FEET
0
2
4
6
8
10
The 1-8 fill became stable on July 27, 1952
Platform installed 8-8-52 at elevation 19.78
Bottom of fill at elevation -27
Bottom of soft mud at elevation -45
7-1-52
7-28-52
10-6-52
1-14-54
6-4-54
11-8-57
Platform destroyed by construction
Surface Monument
Road Traffic
10
100
1000
10000
Construction of Roadway
Prism 6-30-52 to 7-28-52
TIME IN DAYS
AFTER START OF CONSTRUCTION

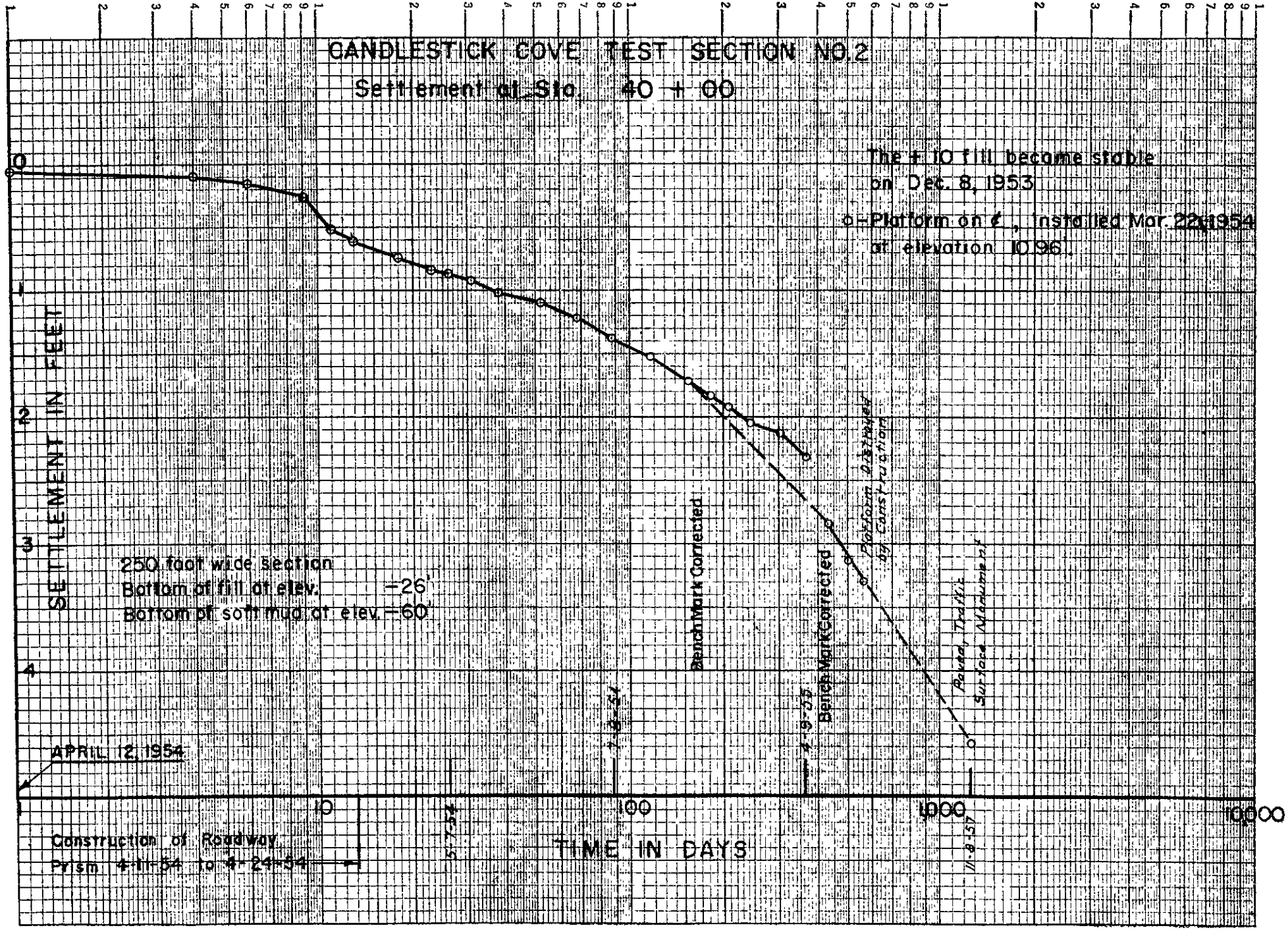
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 10 to the inch.
5th lines accented.
MADE IN U. S. A.



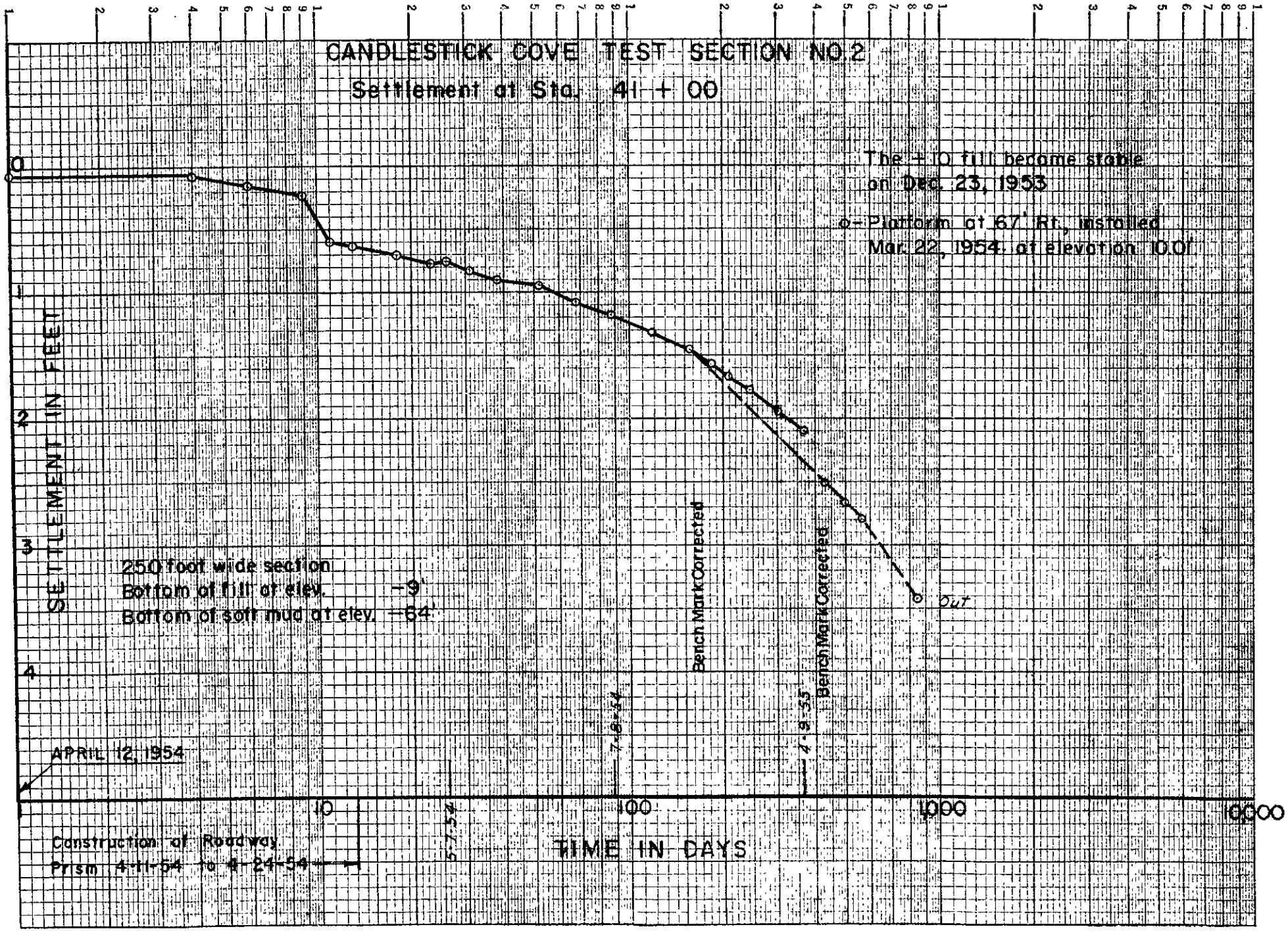
359-51 KEUFFEL & ESSER CO
Semi-Logarithmic, 4 Cycles x 10 to the inch.
5th lines accented.
MADE IN U. S. A.



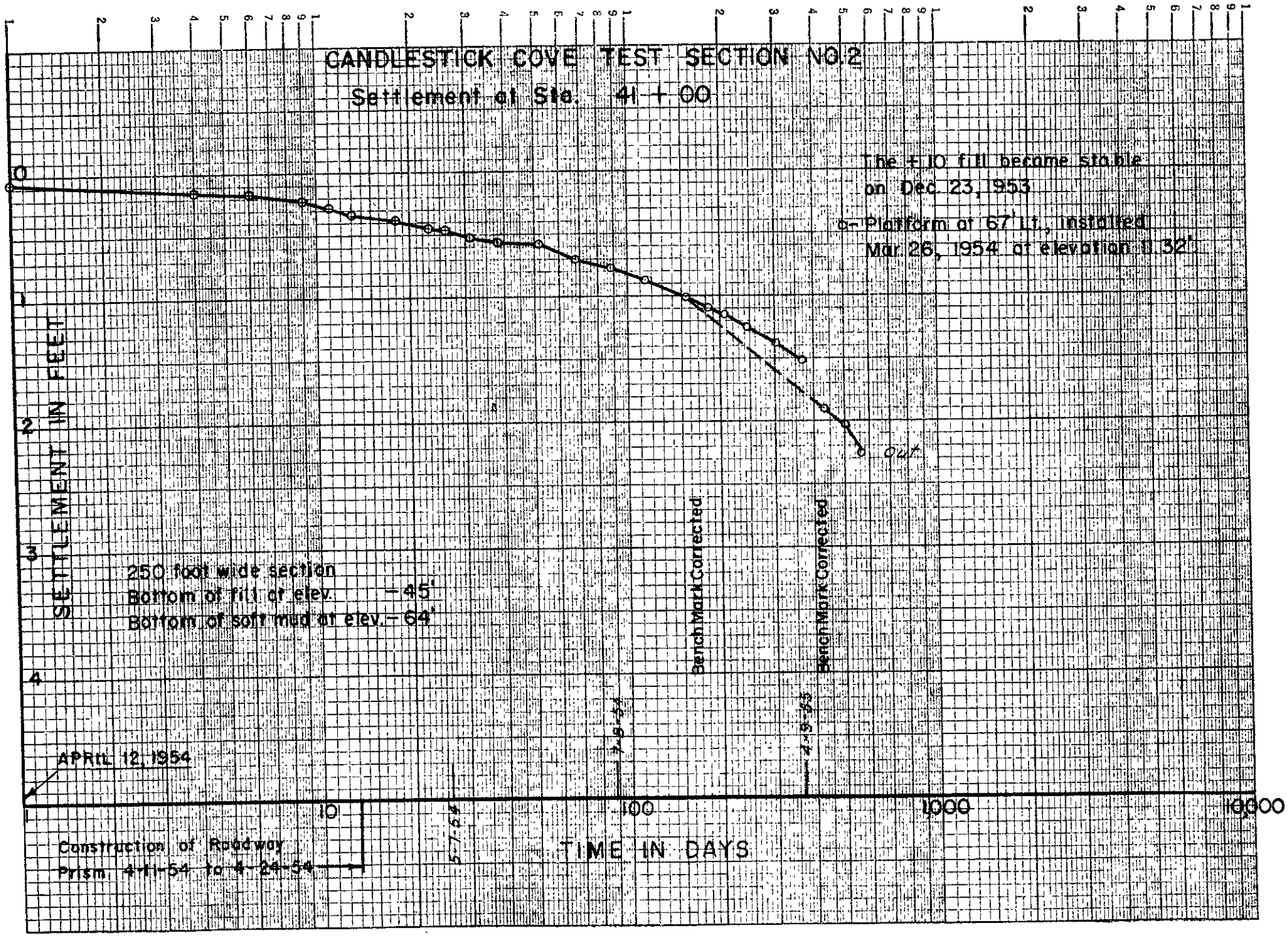
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 10 to the inch
5th lines accented.
MADE IN U.S.A.

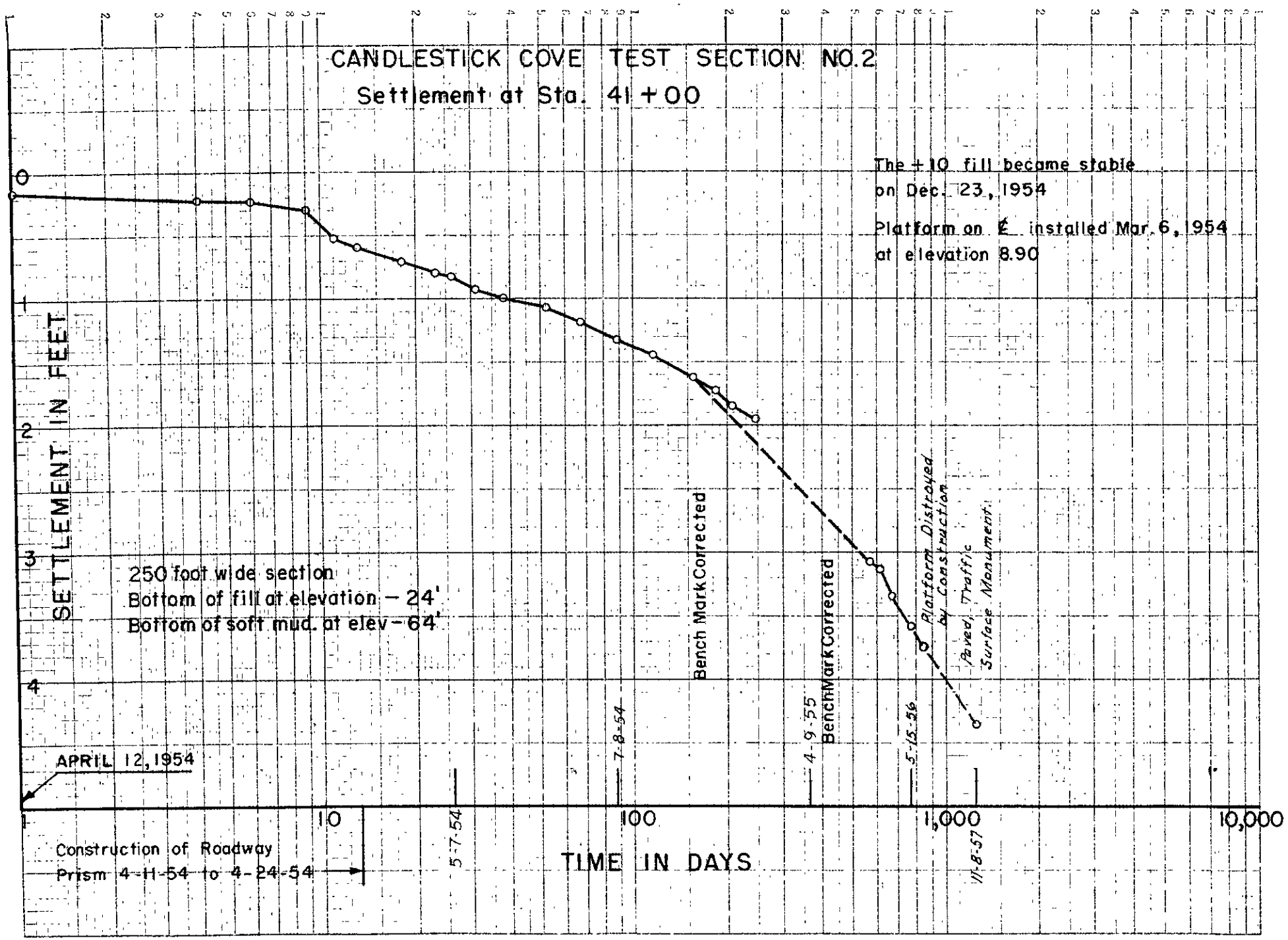


359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles x 10 to the inch
5th lines accented
MADE IN U.S.A.

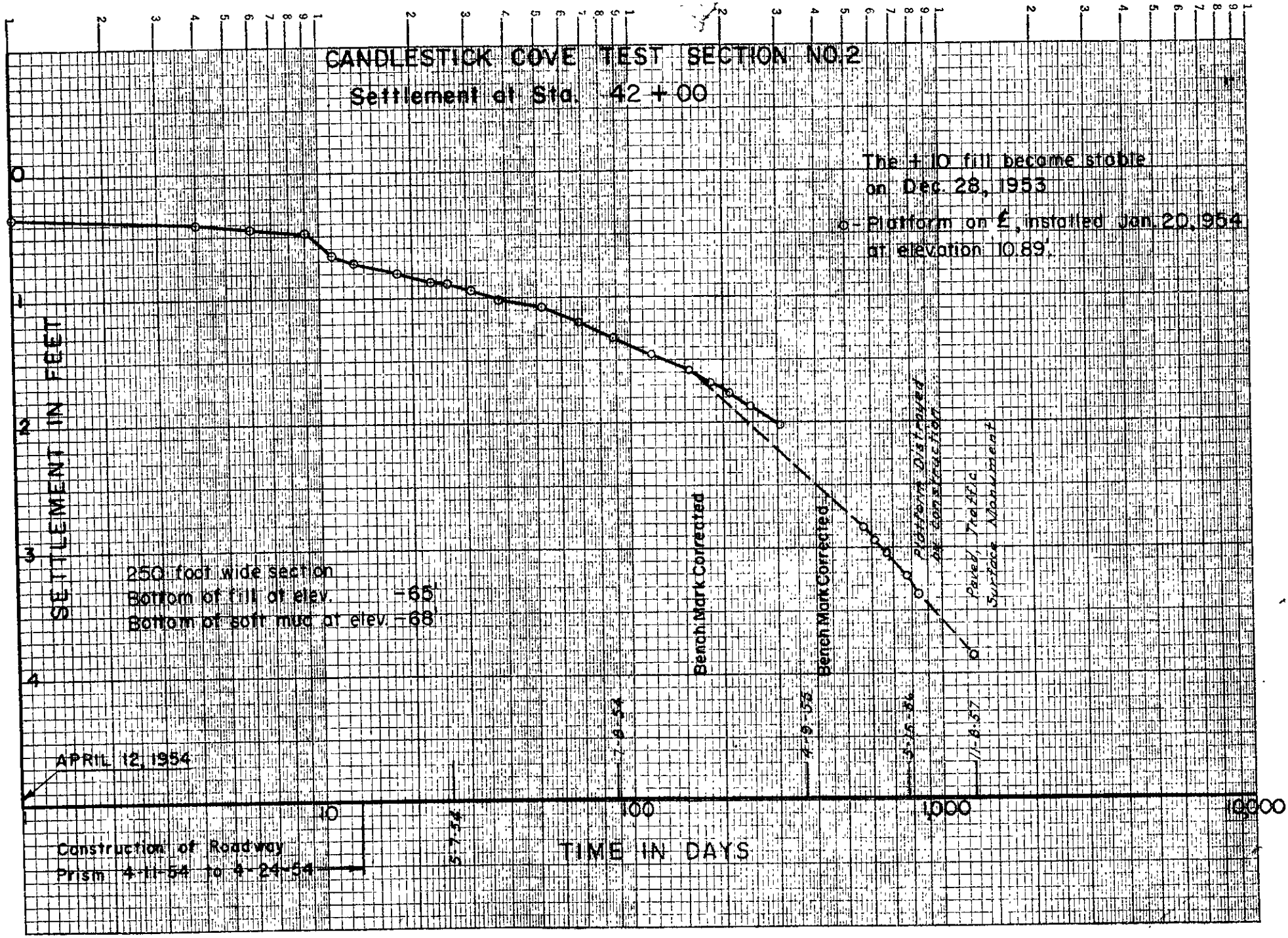


359-81 KEUFFEL & ESSER CO
Semi-Logarithmic, 4 Cycles × 10 to the inch
with lines accented
MADE IN U.S.A.

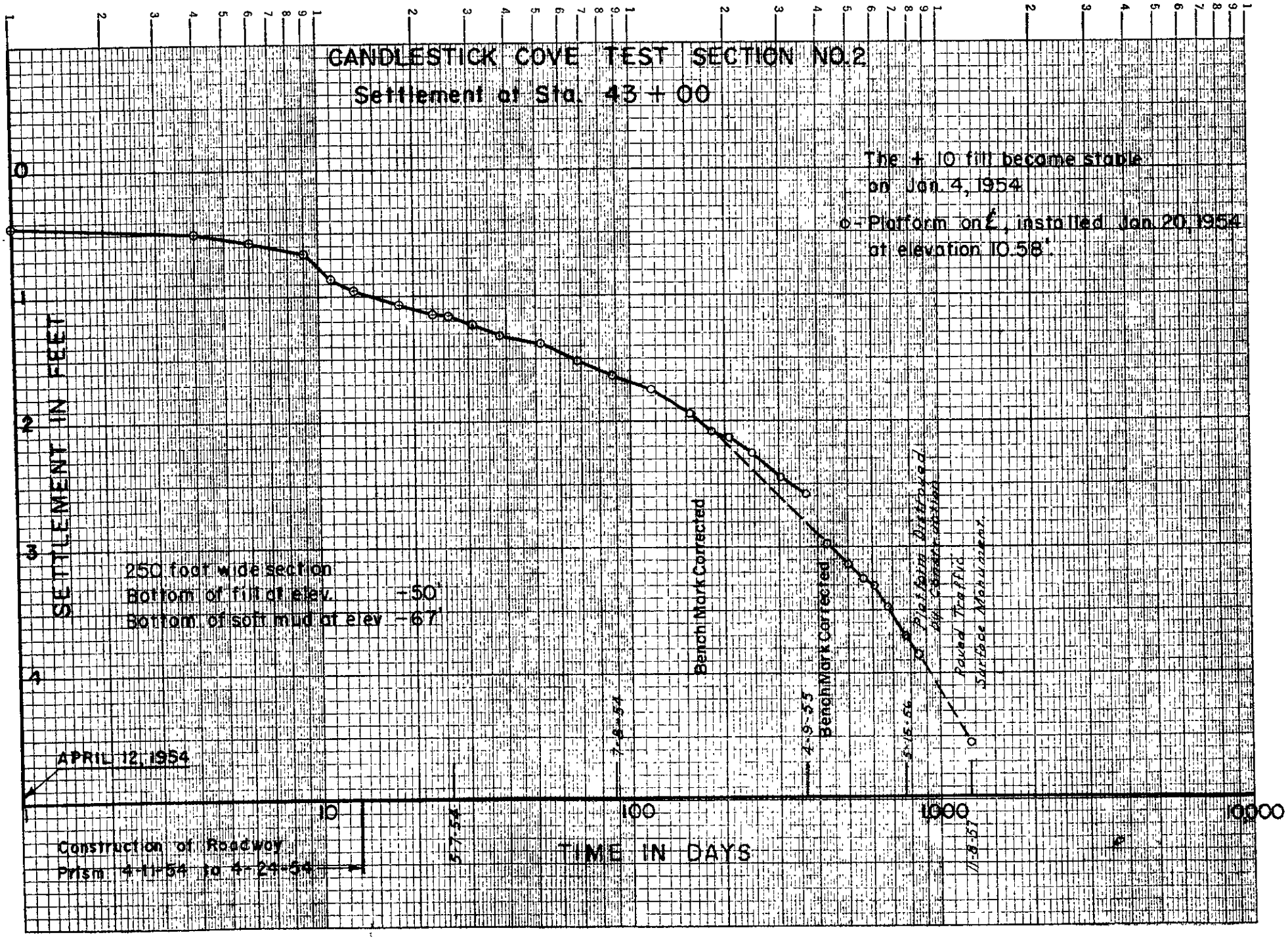



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE TEST SECTION NO.2
Settlement at Sta. 41 + 00
The +10 fill became stable on Dec. 23, 1954
Platform on ℄ installed Mar. 6, 1954 at elevation 8.90
SETTLEMENT IN FEET
0
1
2
3
4
250 foot wide section
Bottom of fill at elevation – 24'
Bottom of soft mud. at elev – 64'
APRIL 12, 1954
Bench Mark Corrected
7-8-54
4-9-55
Bench Mark Corrected
5-15-56
Platform Distroyed by Construction
Paved Traffic Surface Monument
11-8-57
1
10
100
1,000
10,000
5-7-54
TIME IN DAYS
Construction of Roadway Prism 4-11-54 to 4-24-54

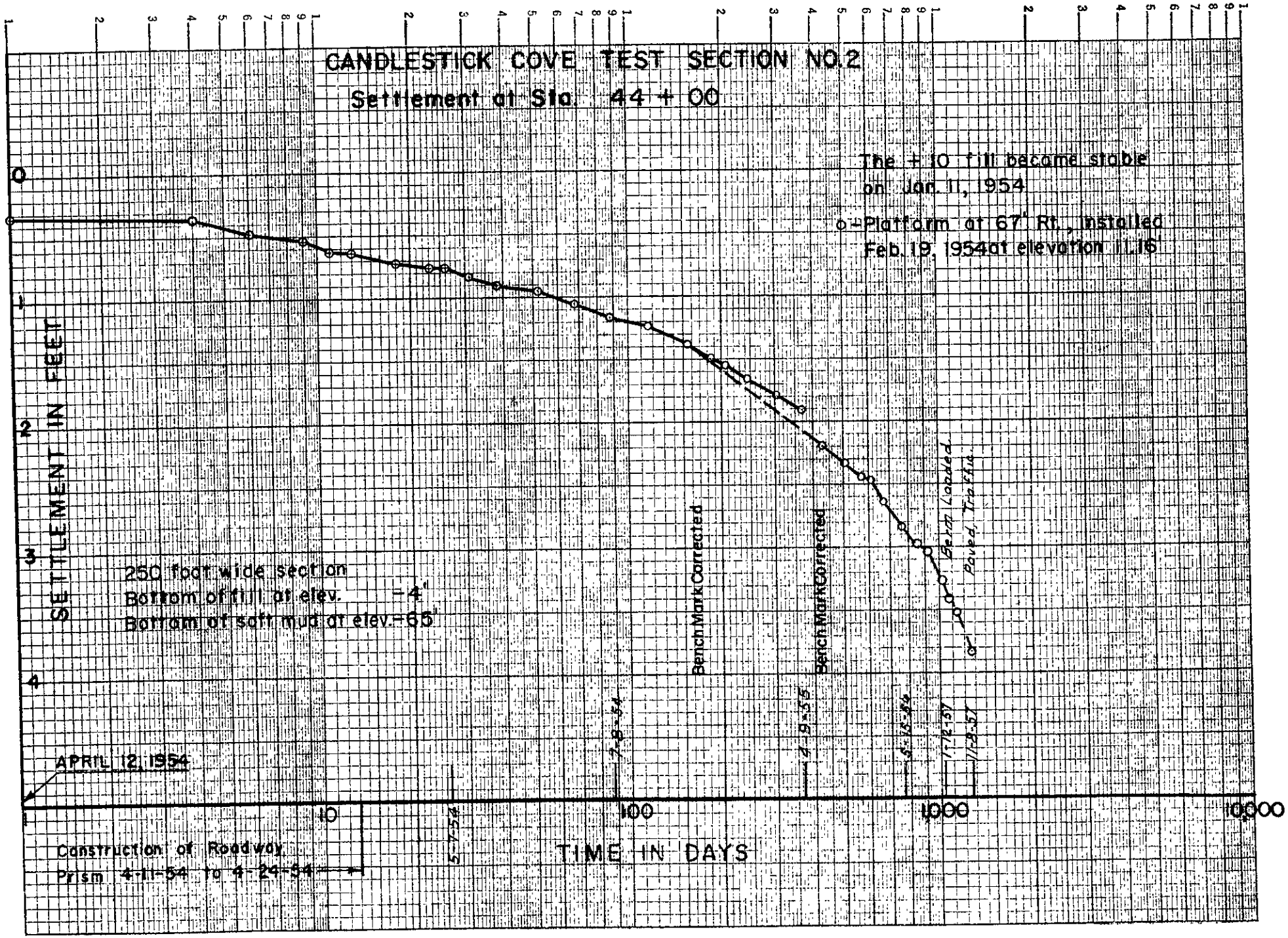
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 10 to the inch.
5th lines accented.
MADE IN U.S.A.



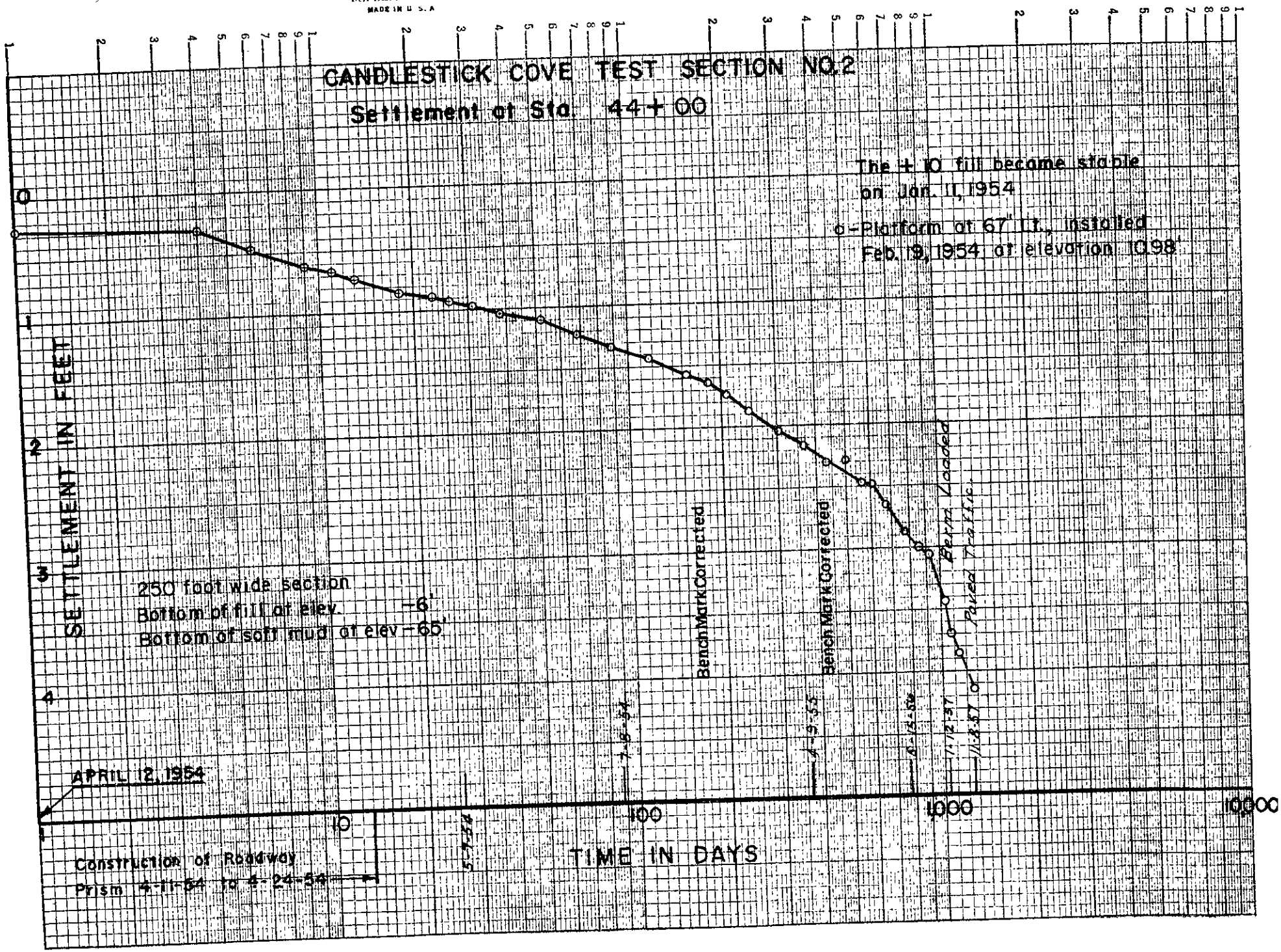
359-81 KEUFFEL & ESSER CO
Semi Logarithmic, 4 Cycles × 10 to the inch
5th lines accented.
MADE IN U.S.A.



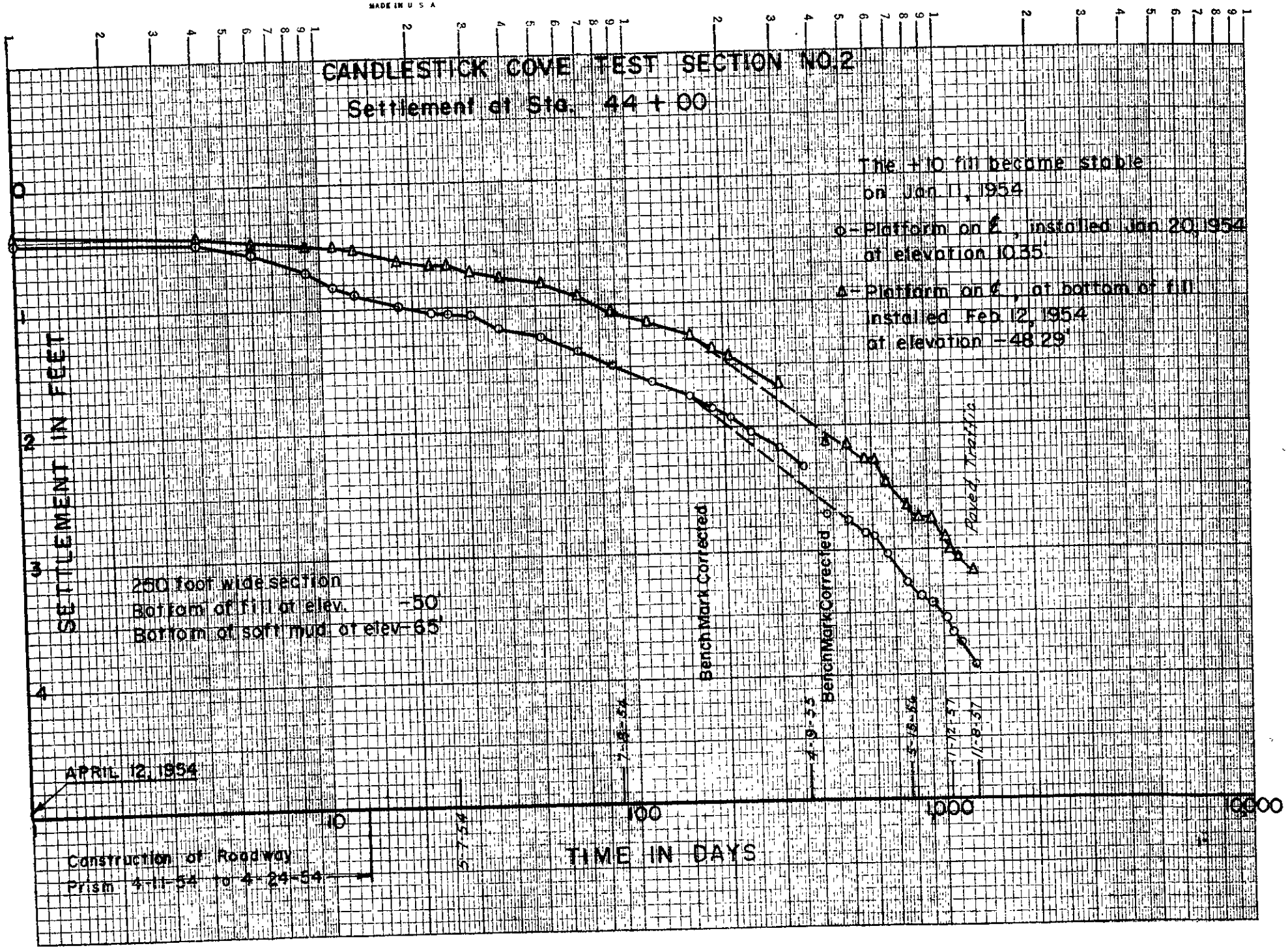
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 1a to the inch.
5th lines accented.
MADE IN U. S. A.



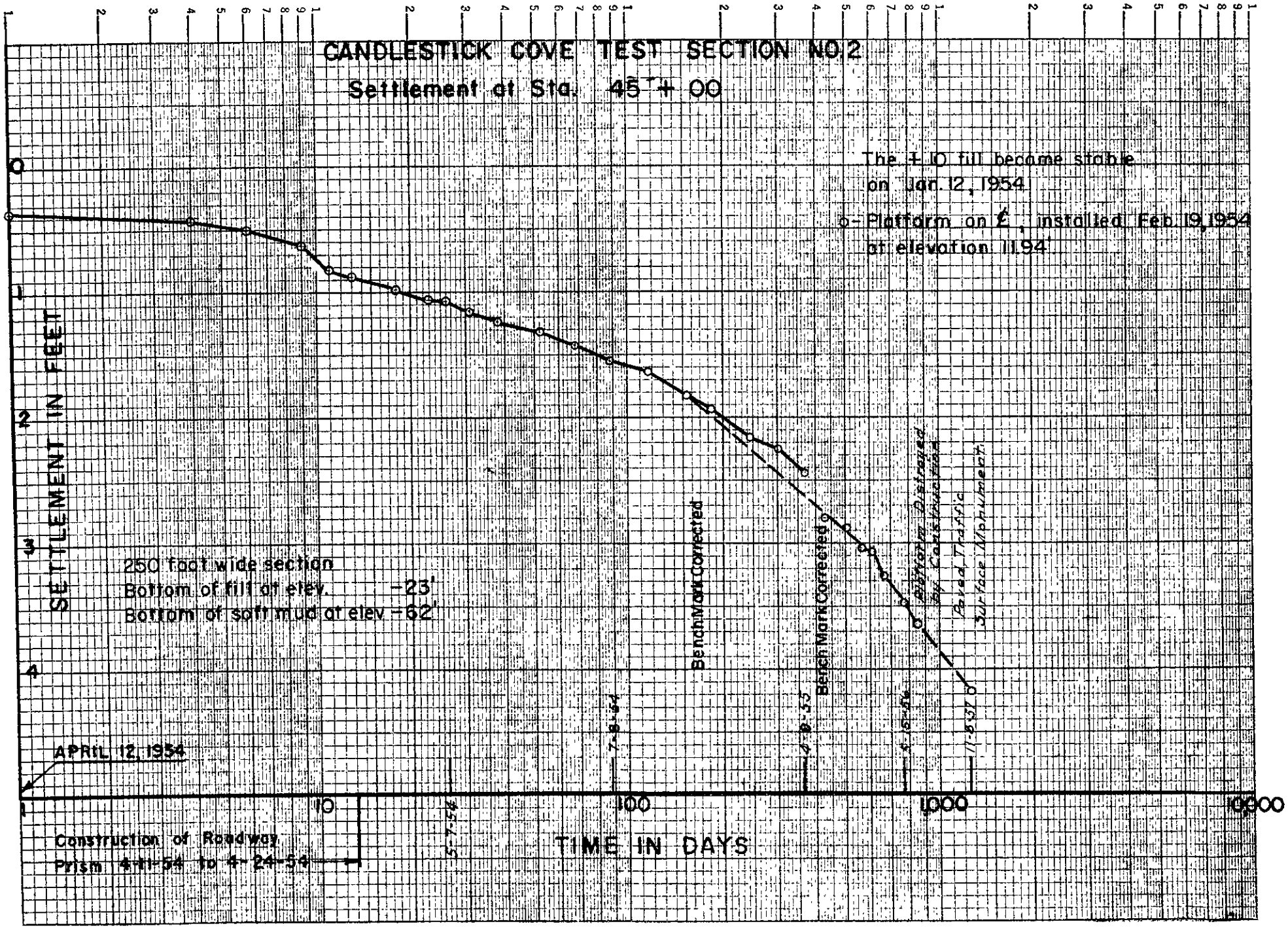
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles x 10 to the inch
5th lines accented
MADE IN U.S.A.



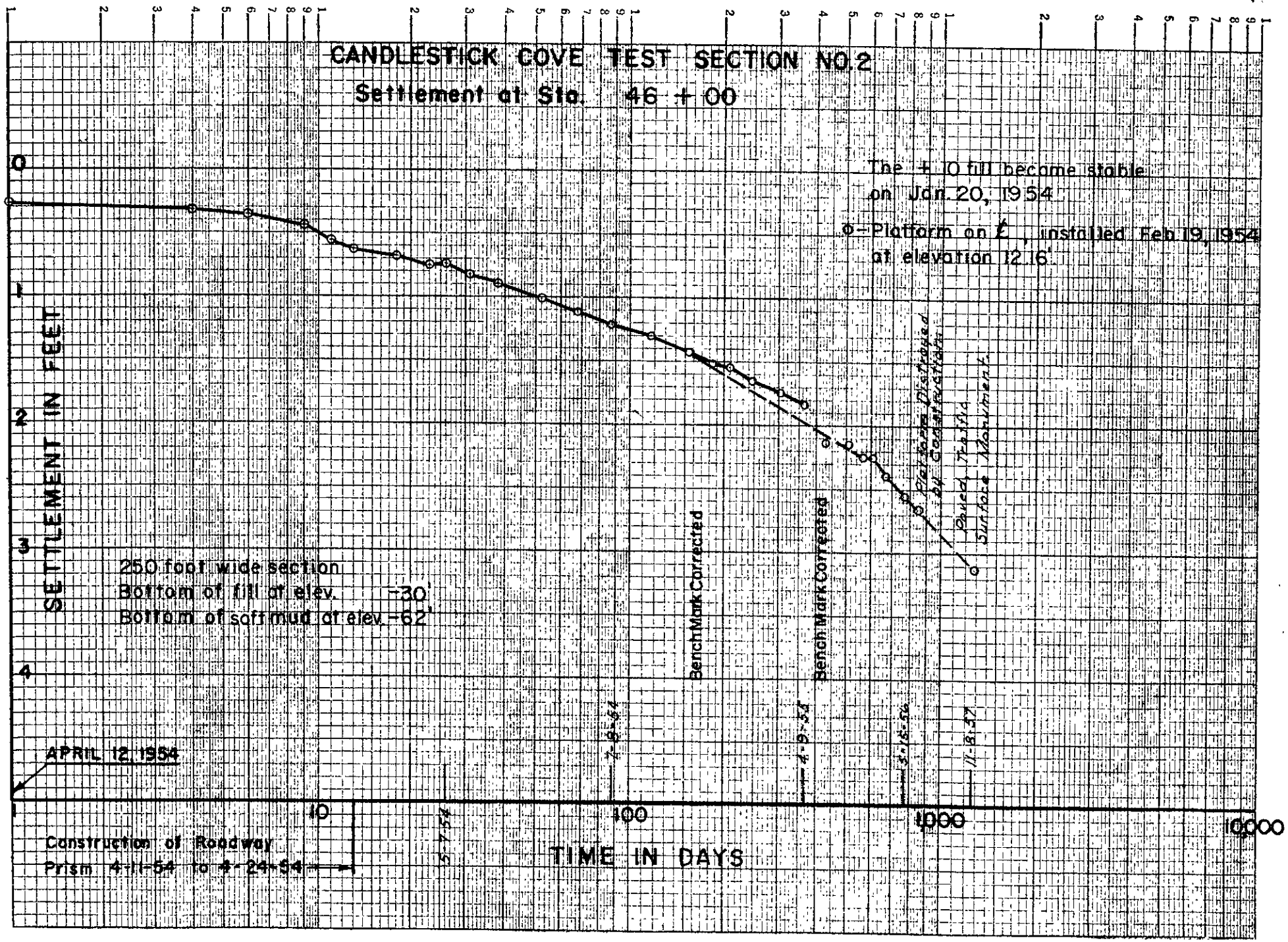
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 10 to the inch.
5th lines accented
MADE IN U.S.A.



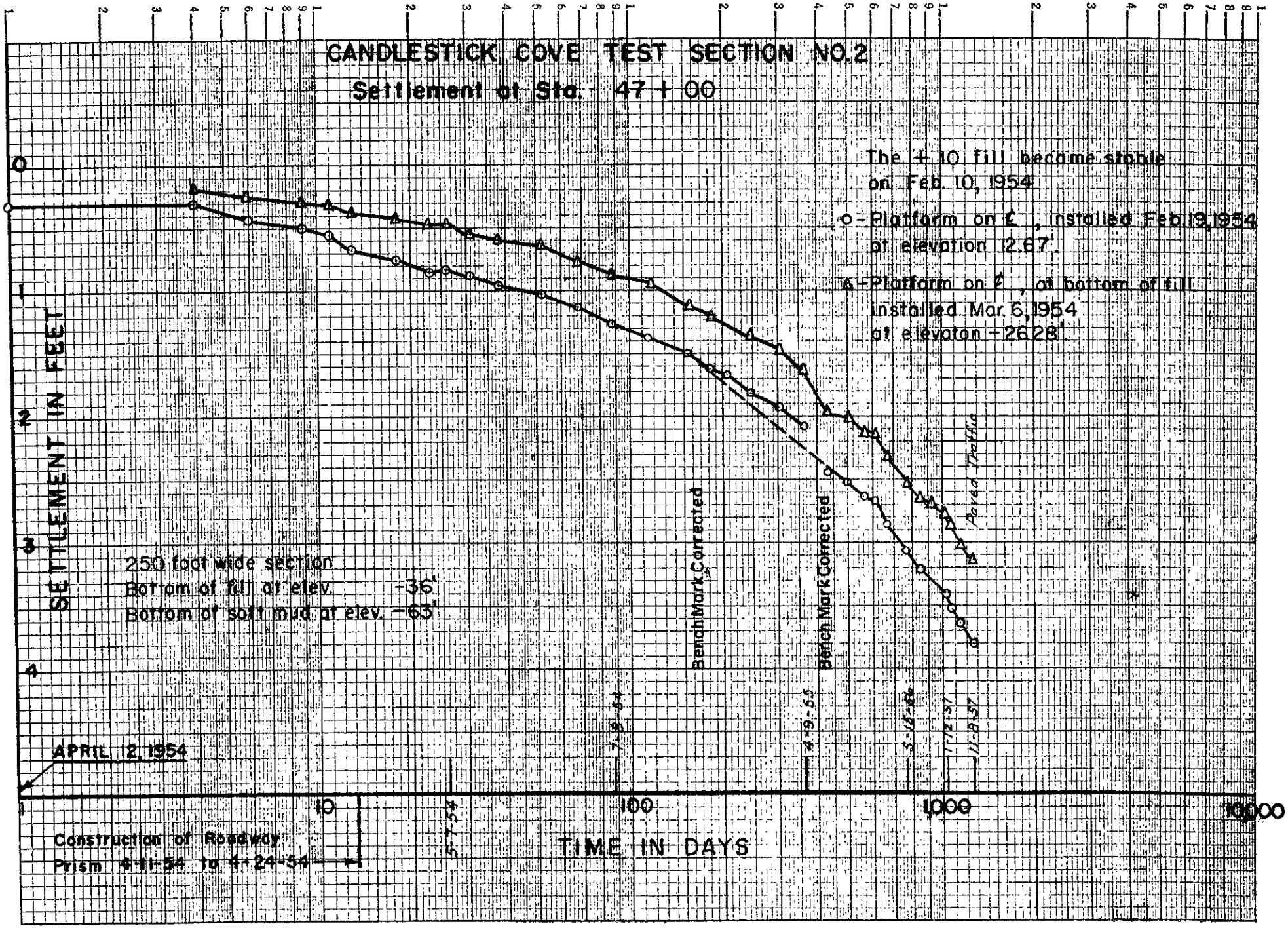
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 10 to the inch
5th lines accented.
MADE IN U.S.A.



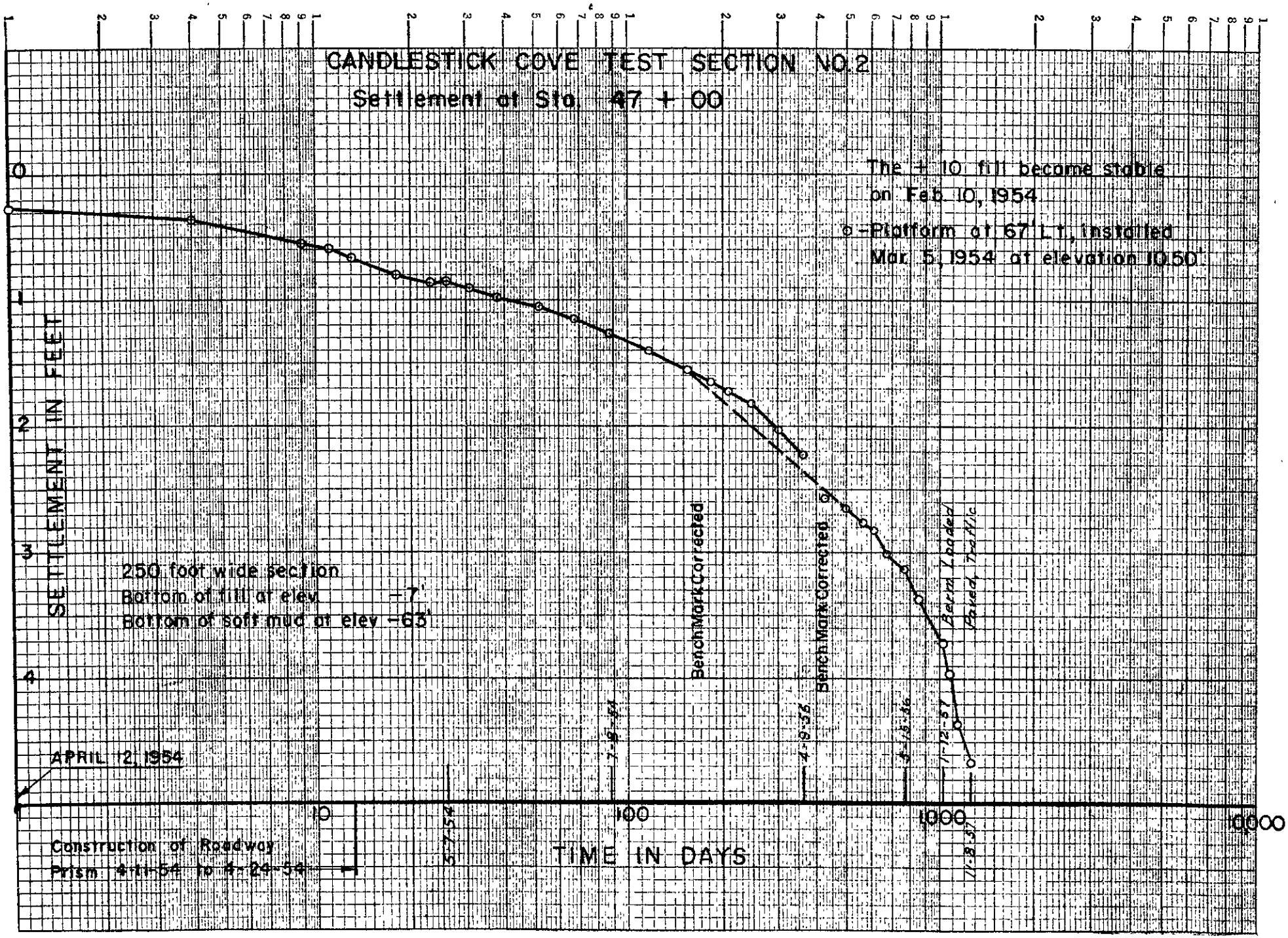
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles × 10 to the inch.
5th lines accented.
MADE IN U.S.A.



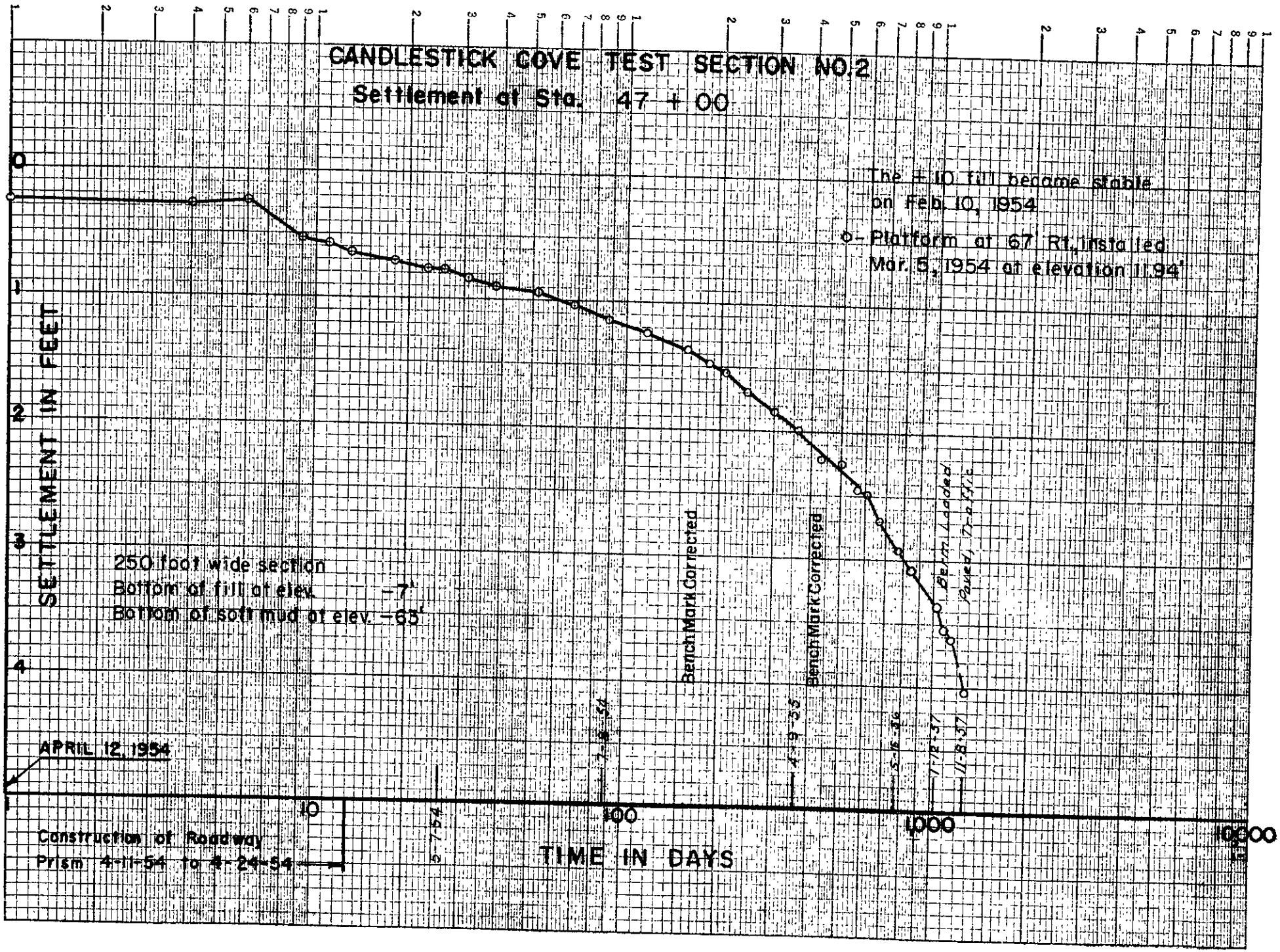
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 10 to the inch,
5th lines accented.
MADE IN U.S.A.



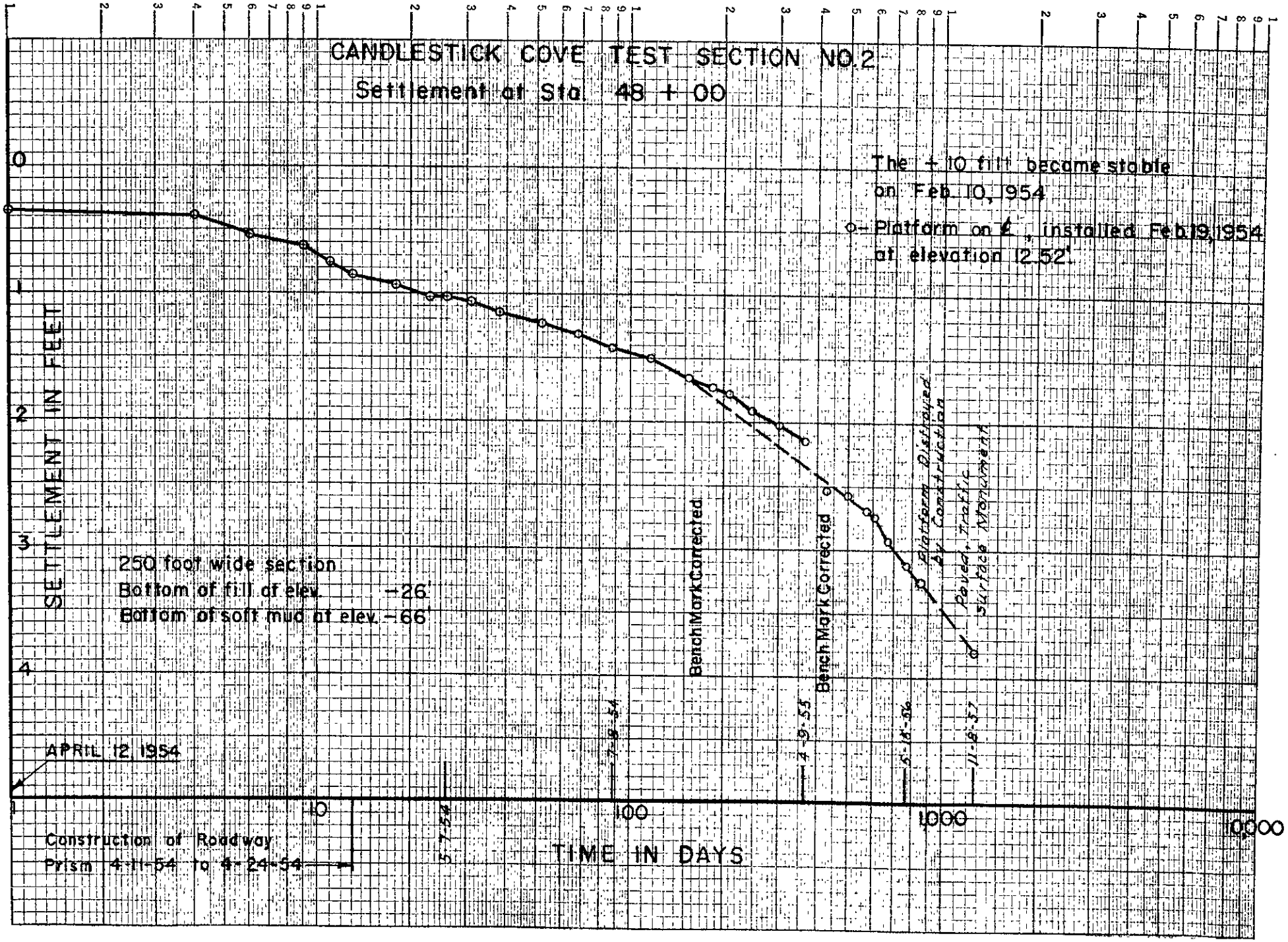
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles × 10 to the inch.
5th lines accented.
MADE IN U. S. A.



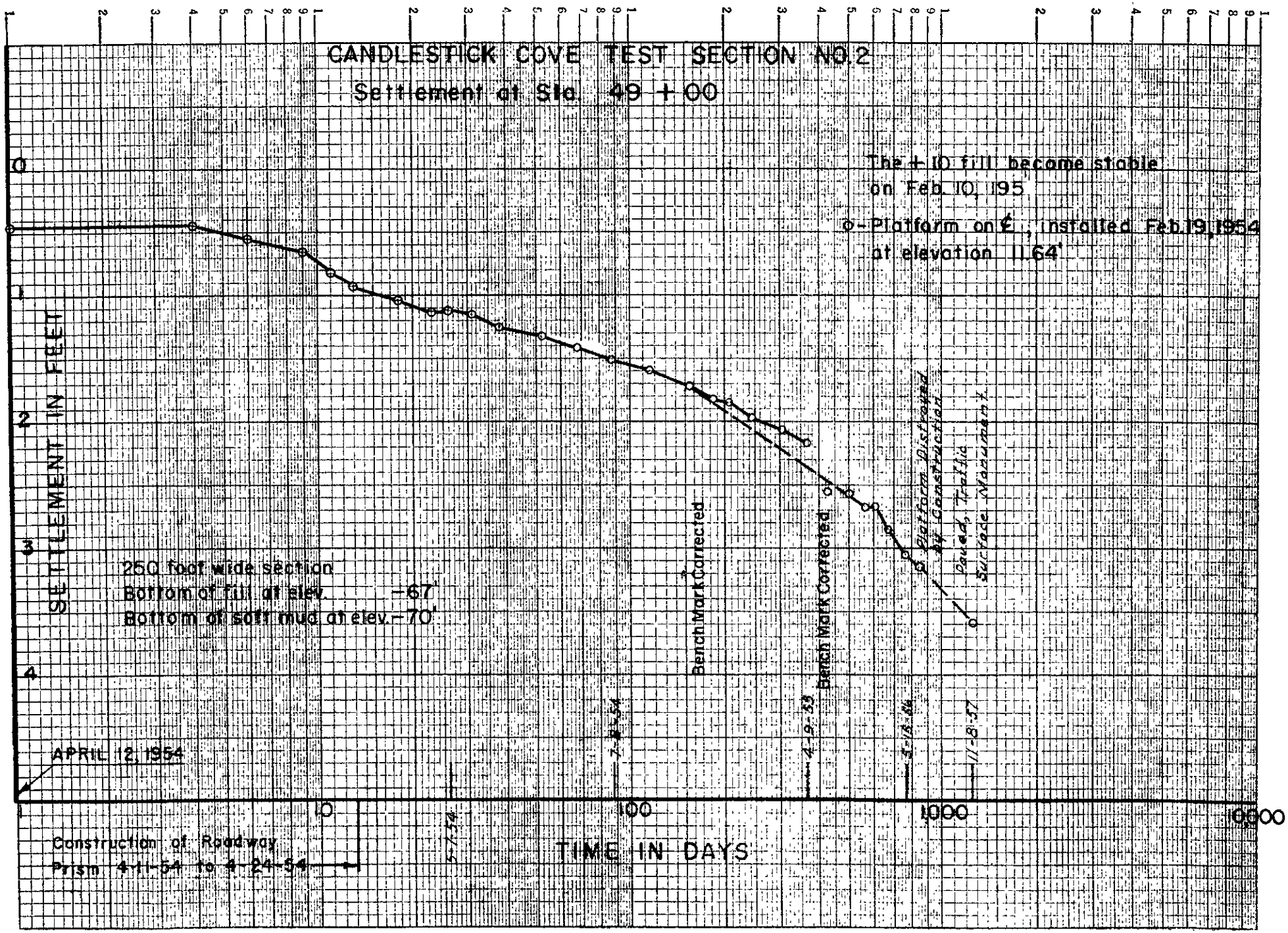
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 10 to the inch,
5th lines accented.
MADE IN U.S.A.



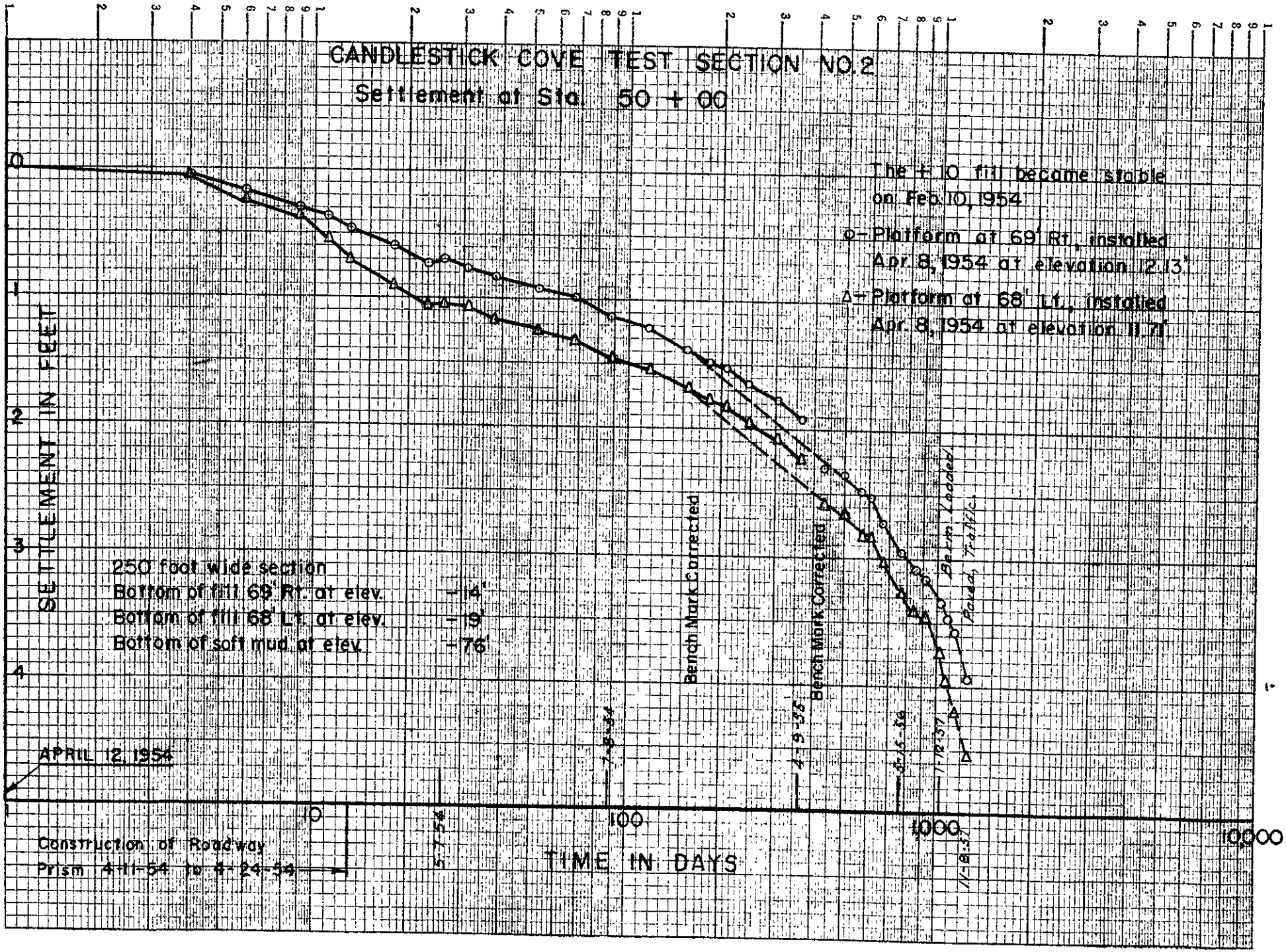
359-81 KEUFFEL & ESSER CO
Semi-Logarithmic, 4 Cycles x 10 to the inch
5th lines accented
MADE IN U.S.A.



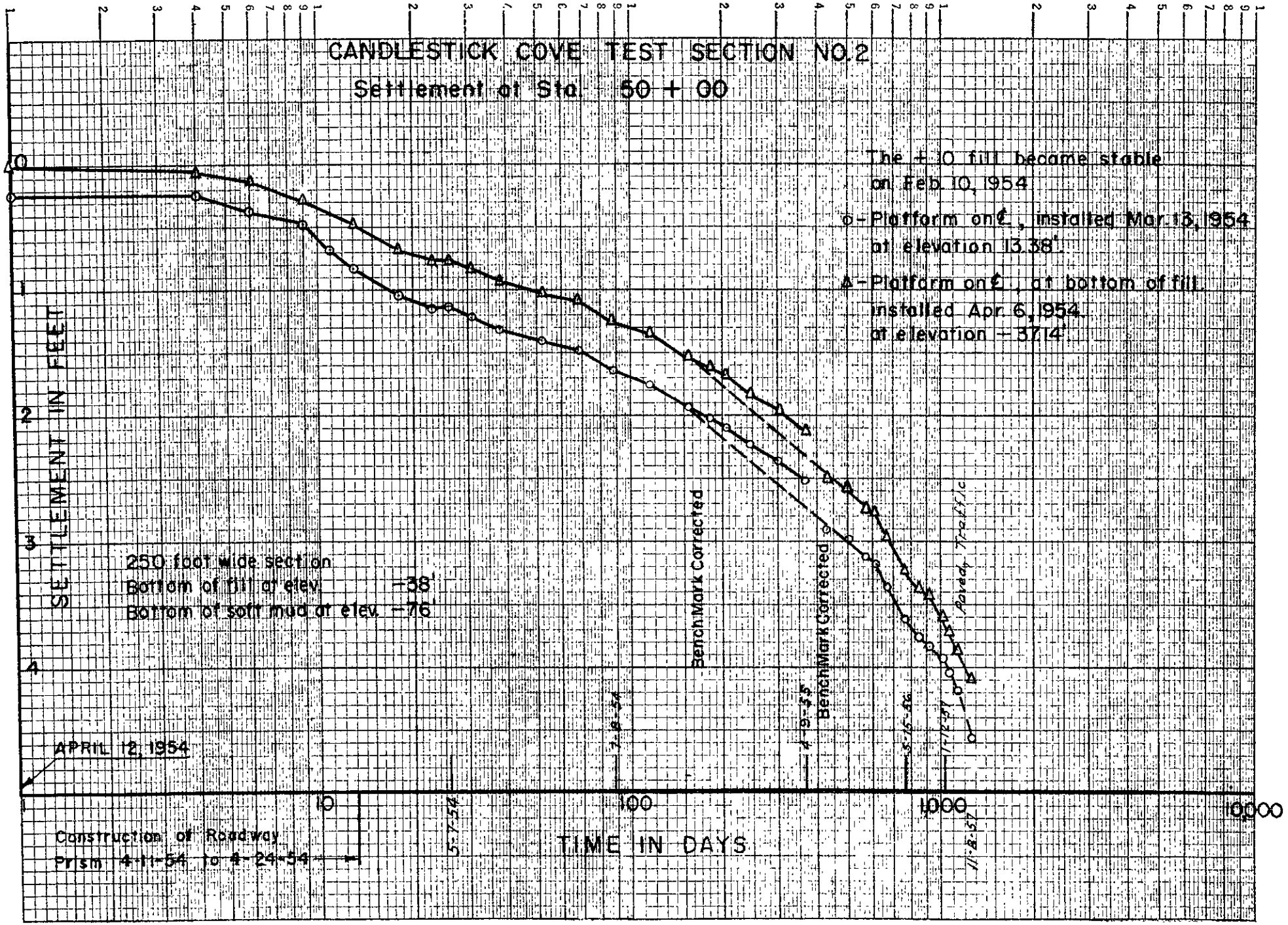
359-61 KEUFFEL & ESSER CO
Semi-Logarithmic, 4 Cycles x 10 to the inch
5th lines accented
MADE IN U.S.A.



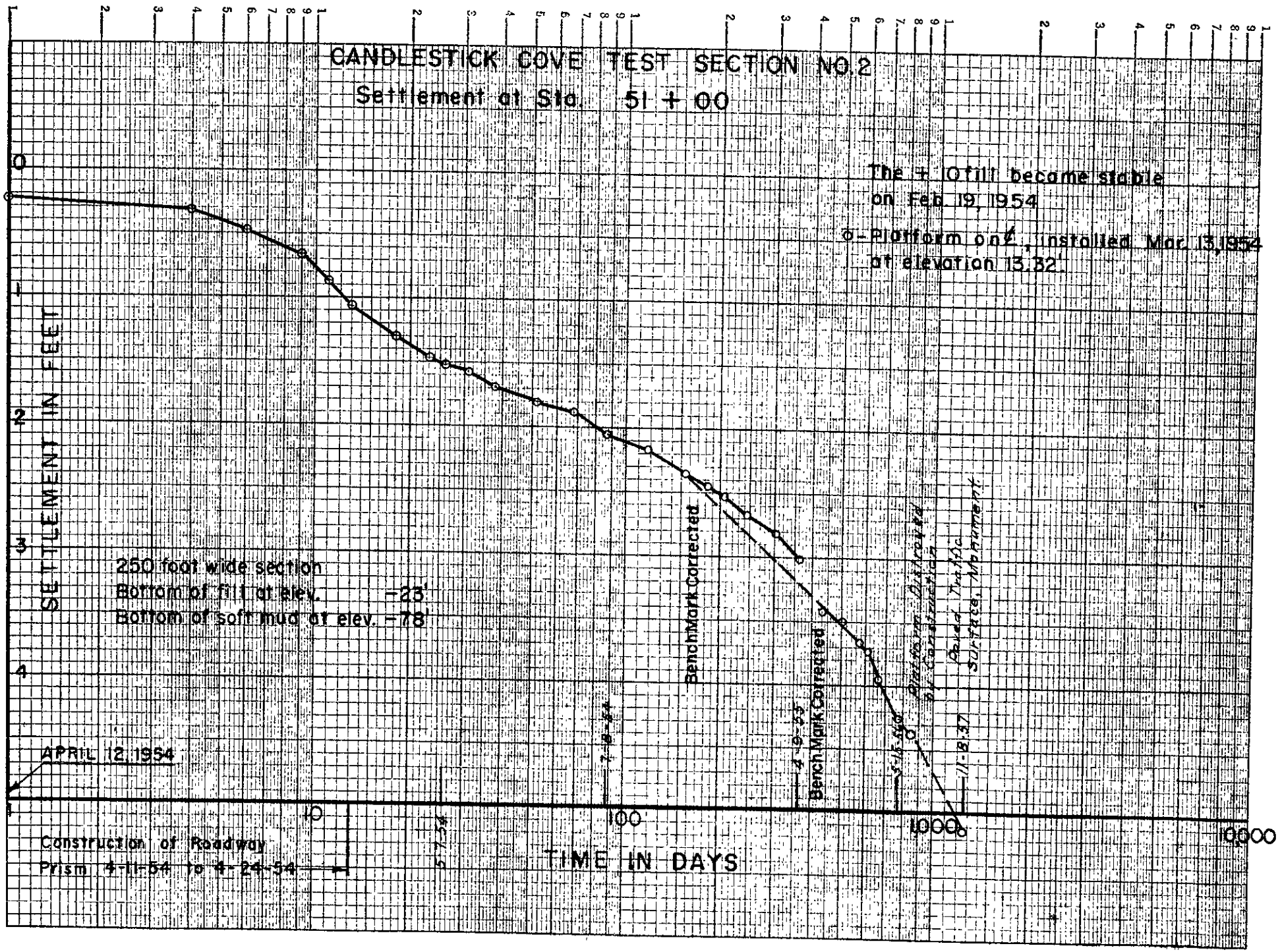
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles × 10 to the inch.
5th lines accented.
MADE IN U.S.A.



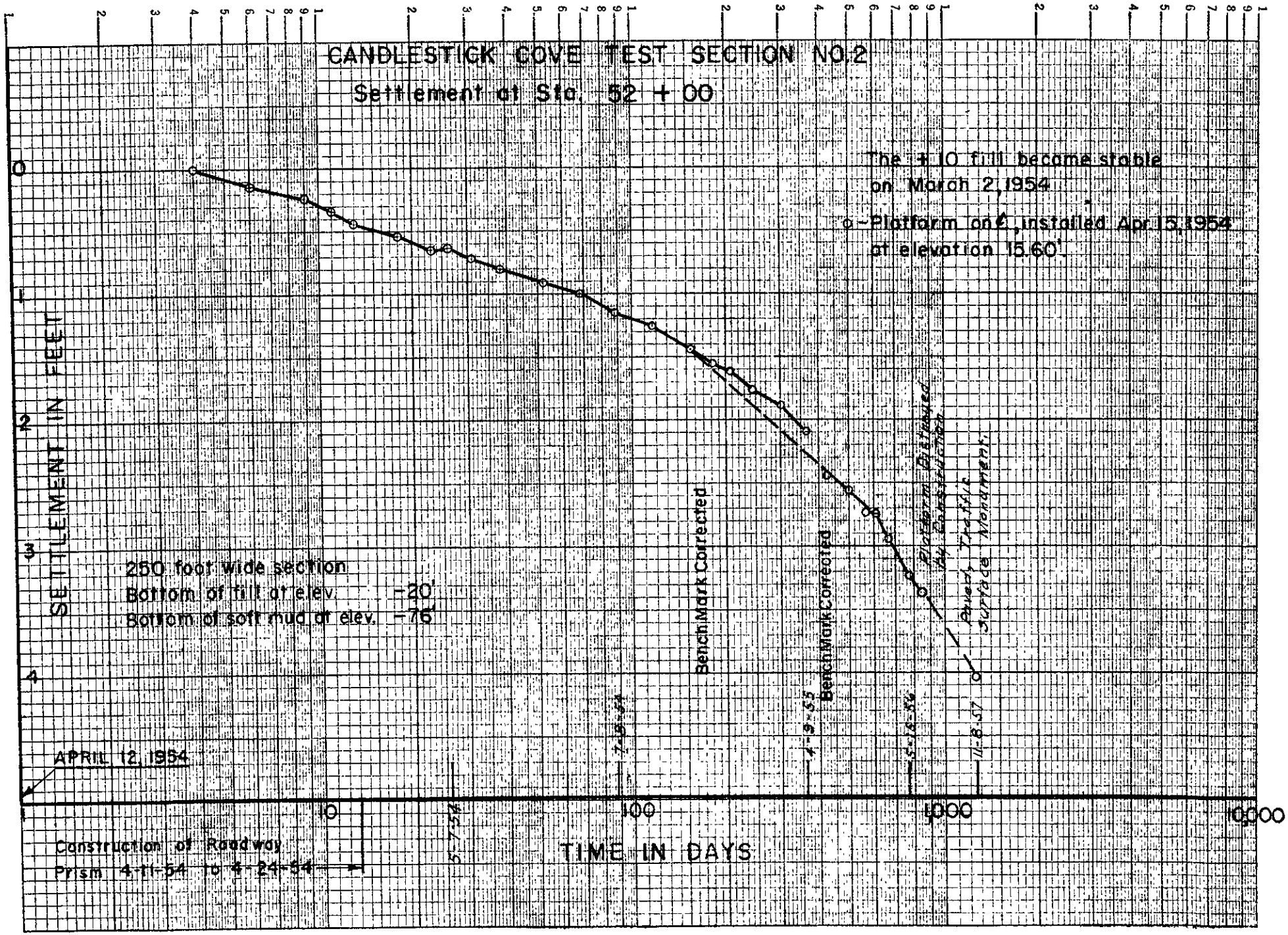
359-31 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles × 10 to the inch
5th lines accented.
MADE IN U. S. A.



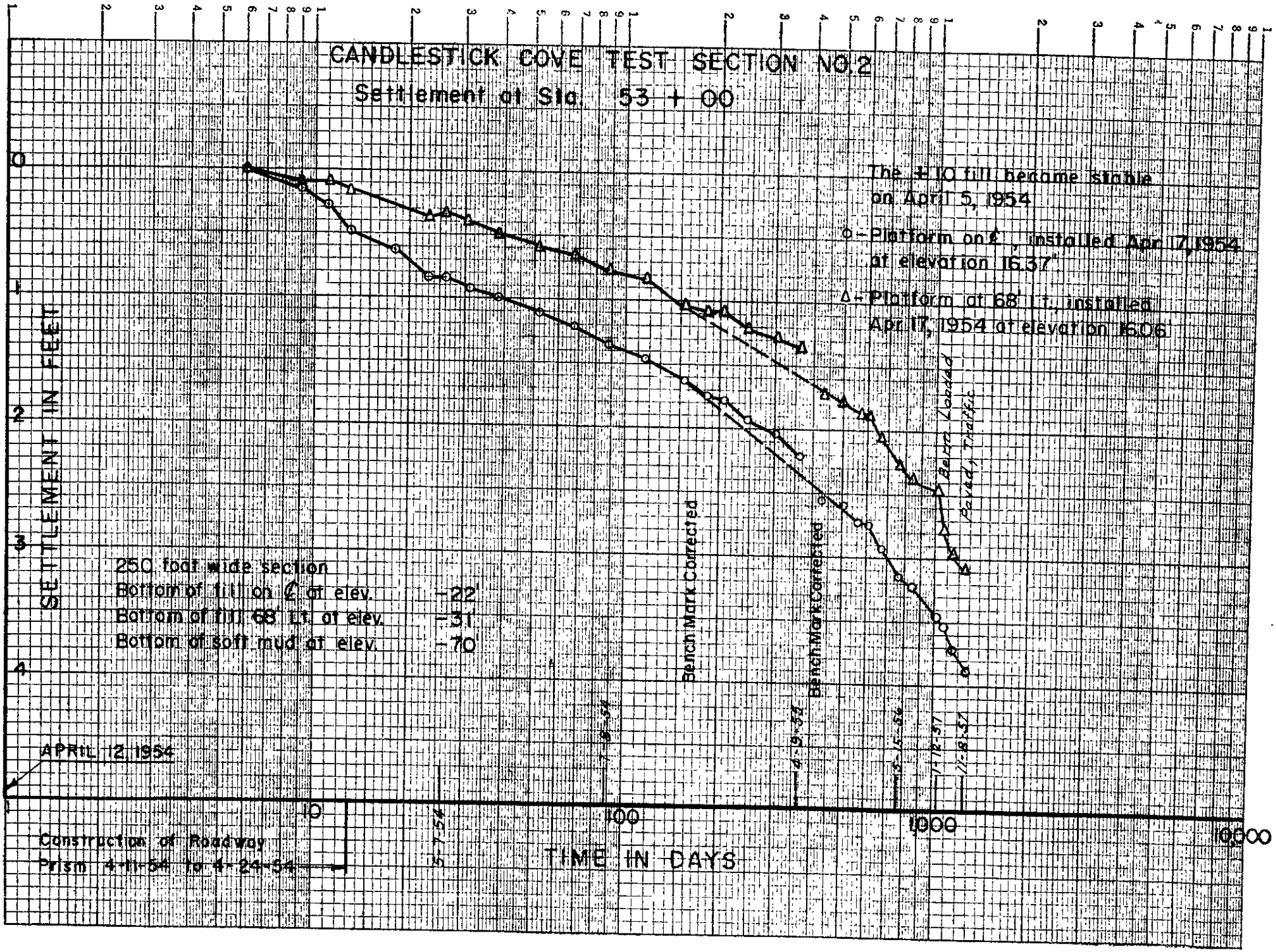
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles × 10 to the inch
5th lines accented.
MADE IN U.S.A.



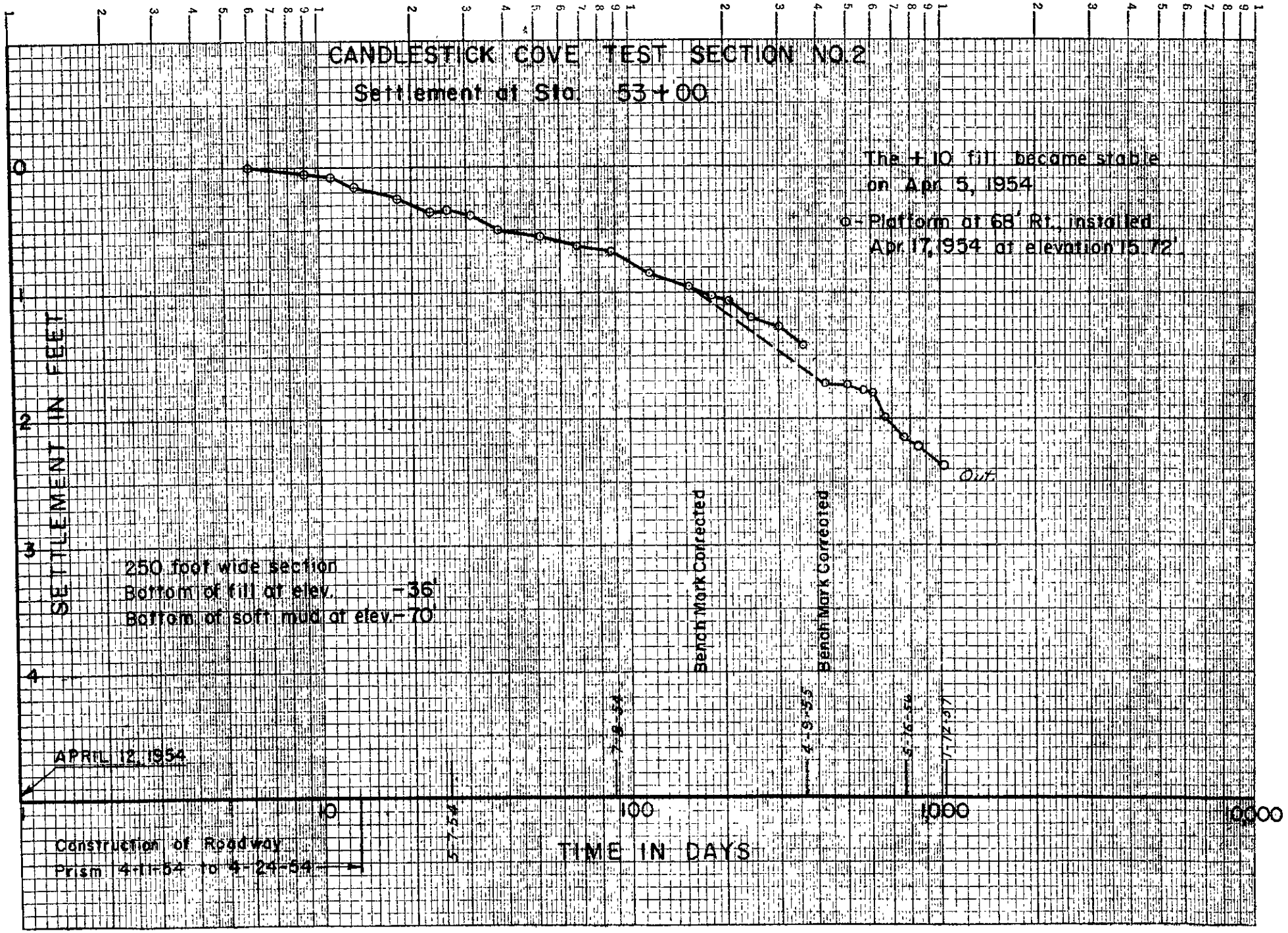
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles x 10 to the inch.
5th lines accented.
MADE IN U.S.A.



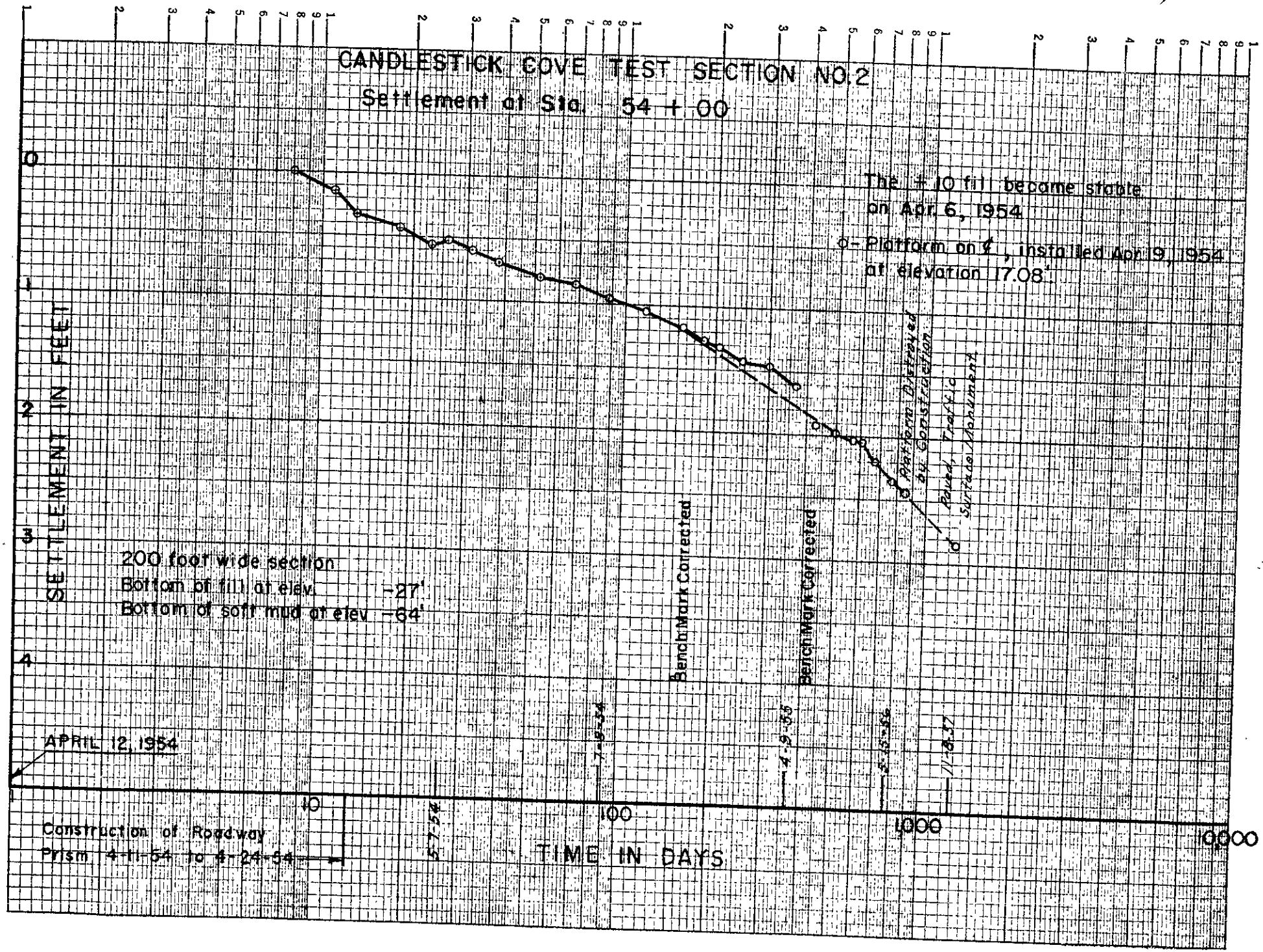
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles × 10 to the inch
with lines accented.
MADE IN U.S.A.



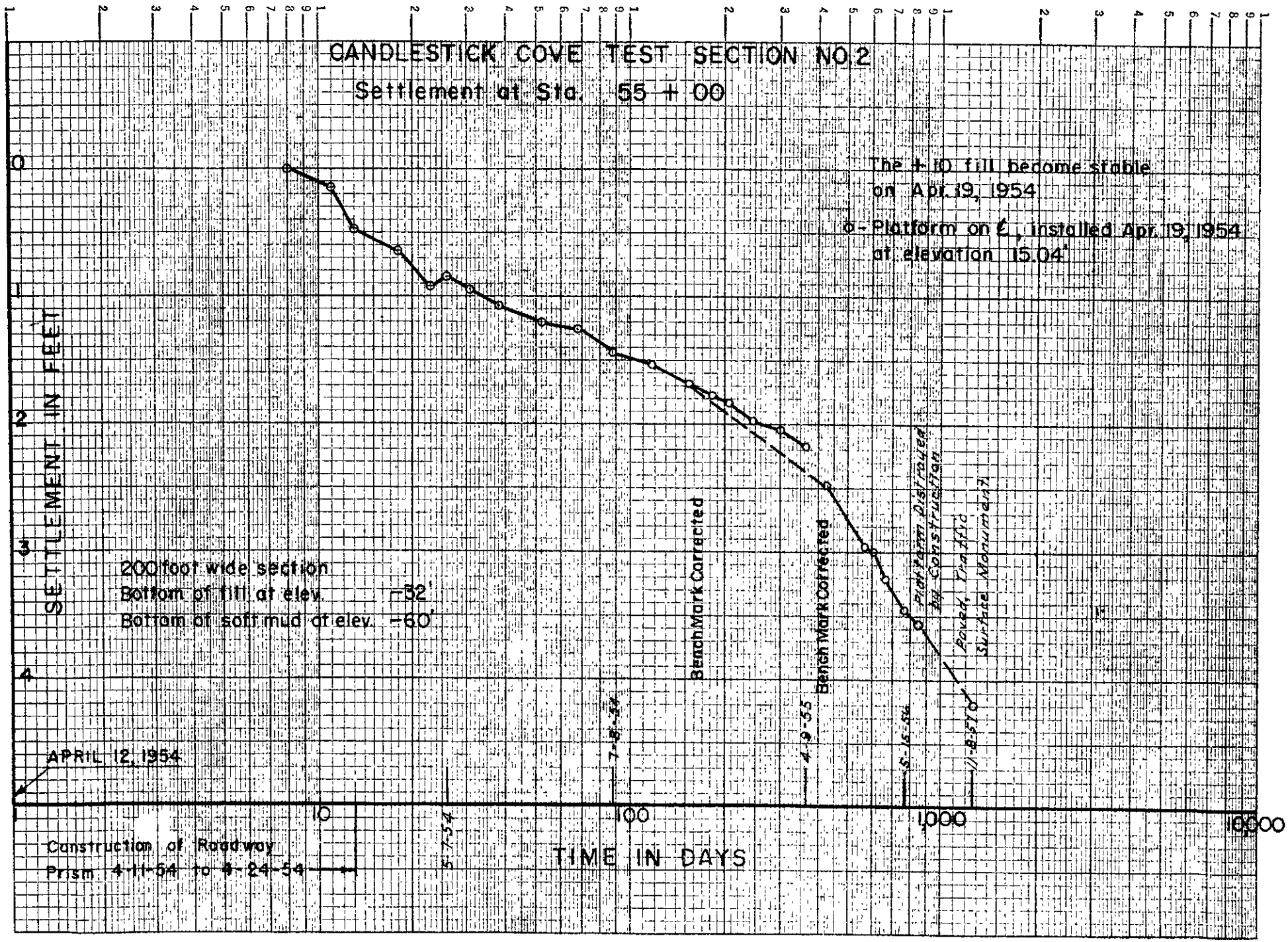
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 10 to the inch
5th lines accented.
MADE IN U. S. A.



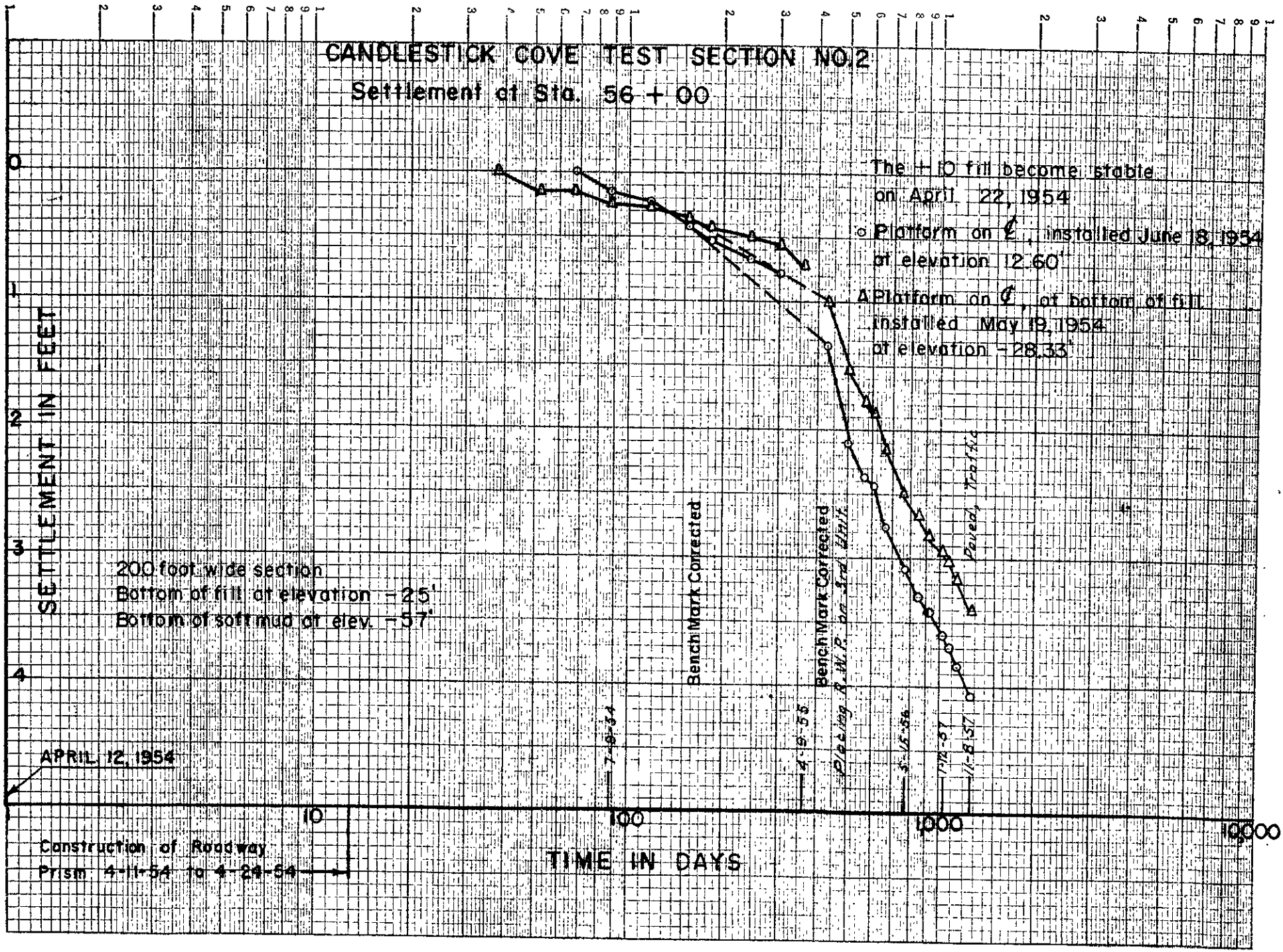
359-81 KEUFFEL & ESSER CO
Semi-Logarithmic, 4 Cycles x 10 to the inch
5th lines accented
MADE IN U.S.A.

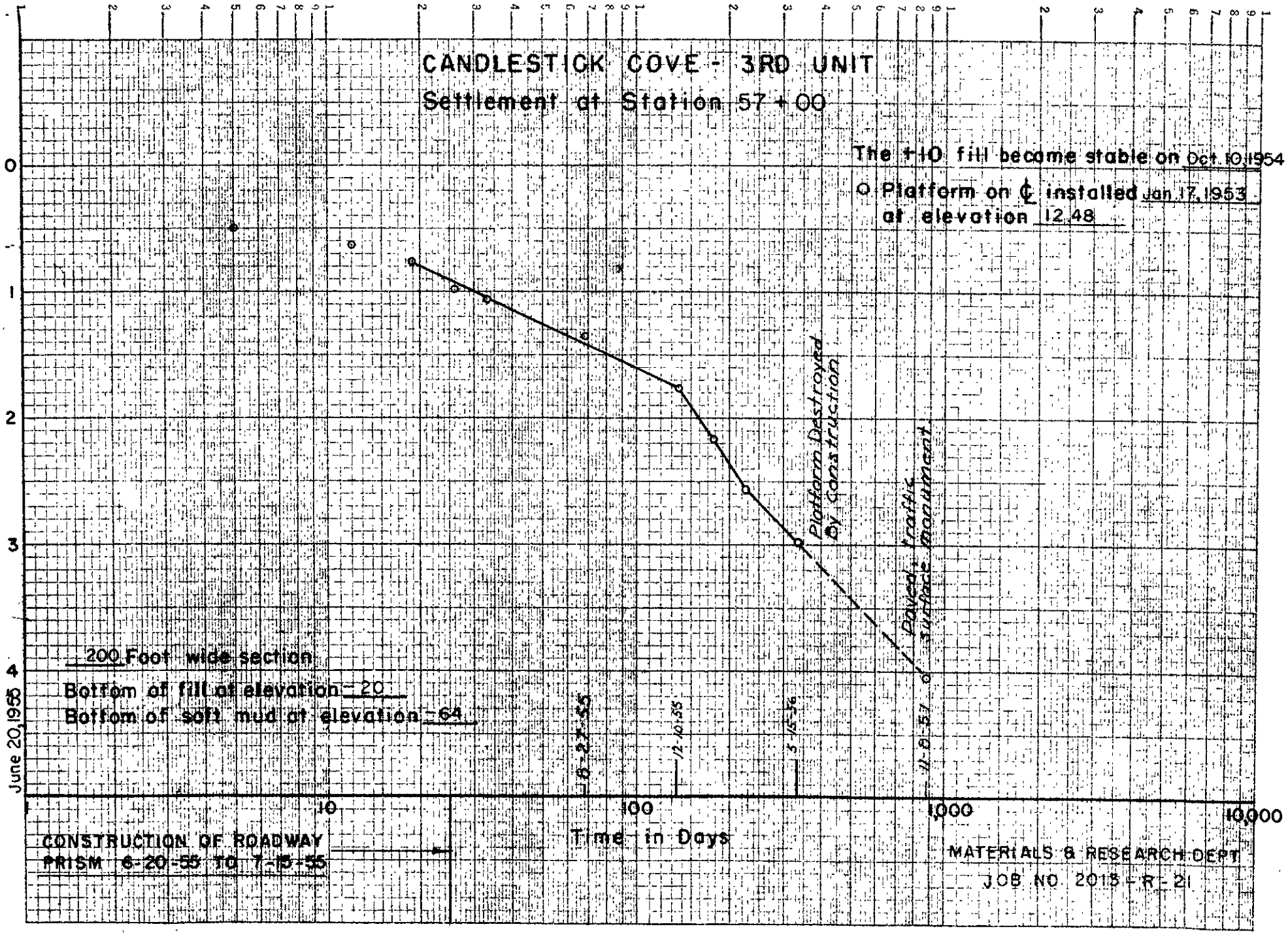


359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 10 to the inch.
5th lines accented.
MADE IN U.S.A.

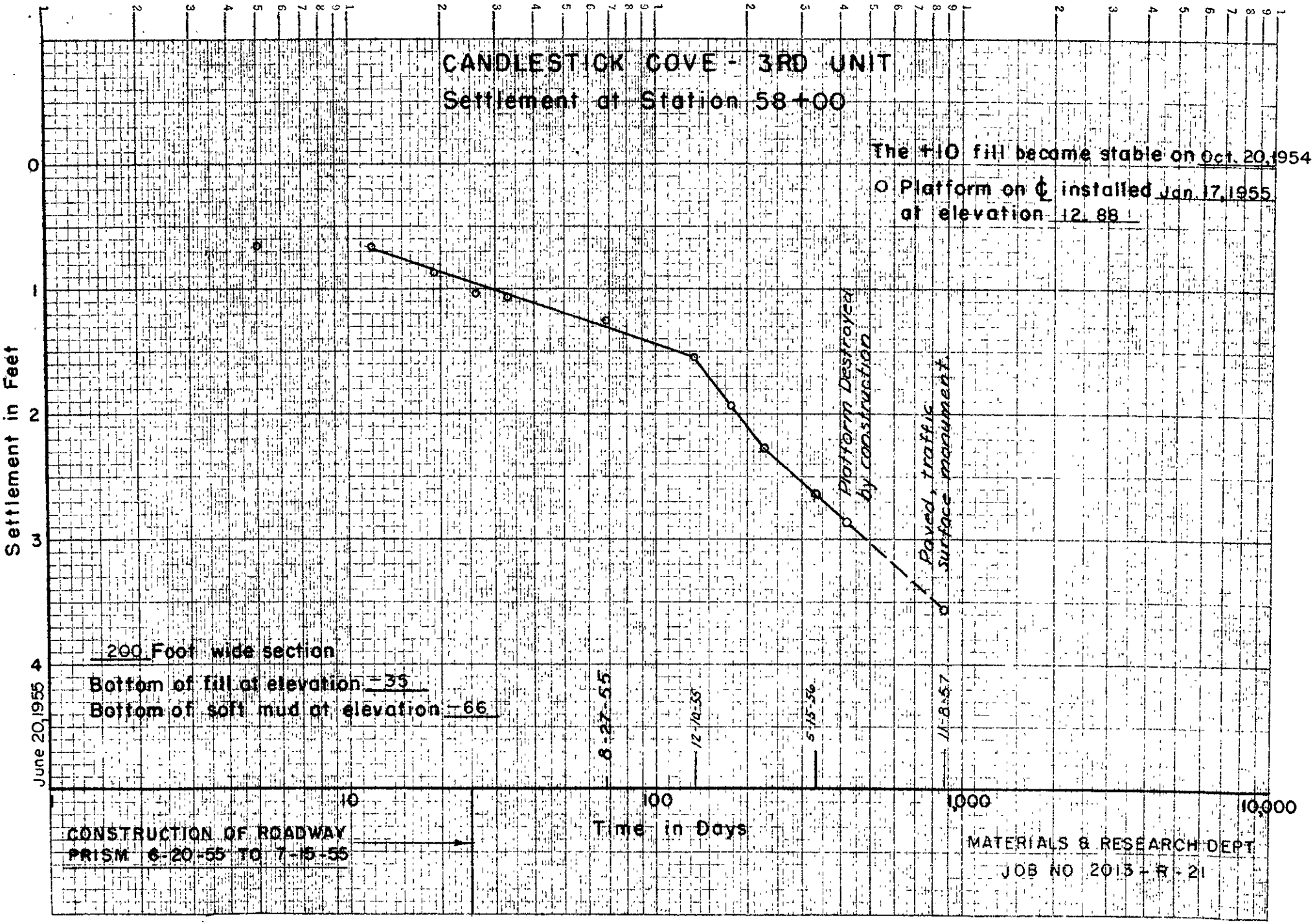


359-81 KEUFFEL & ESSER CO
Semi-Logarithmic, 4 Cycles X 10 to the inch
5th lines accented
MADE IN U.S.A.

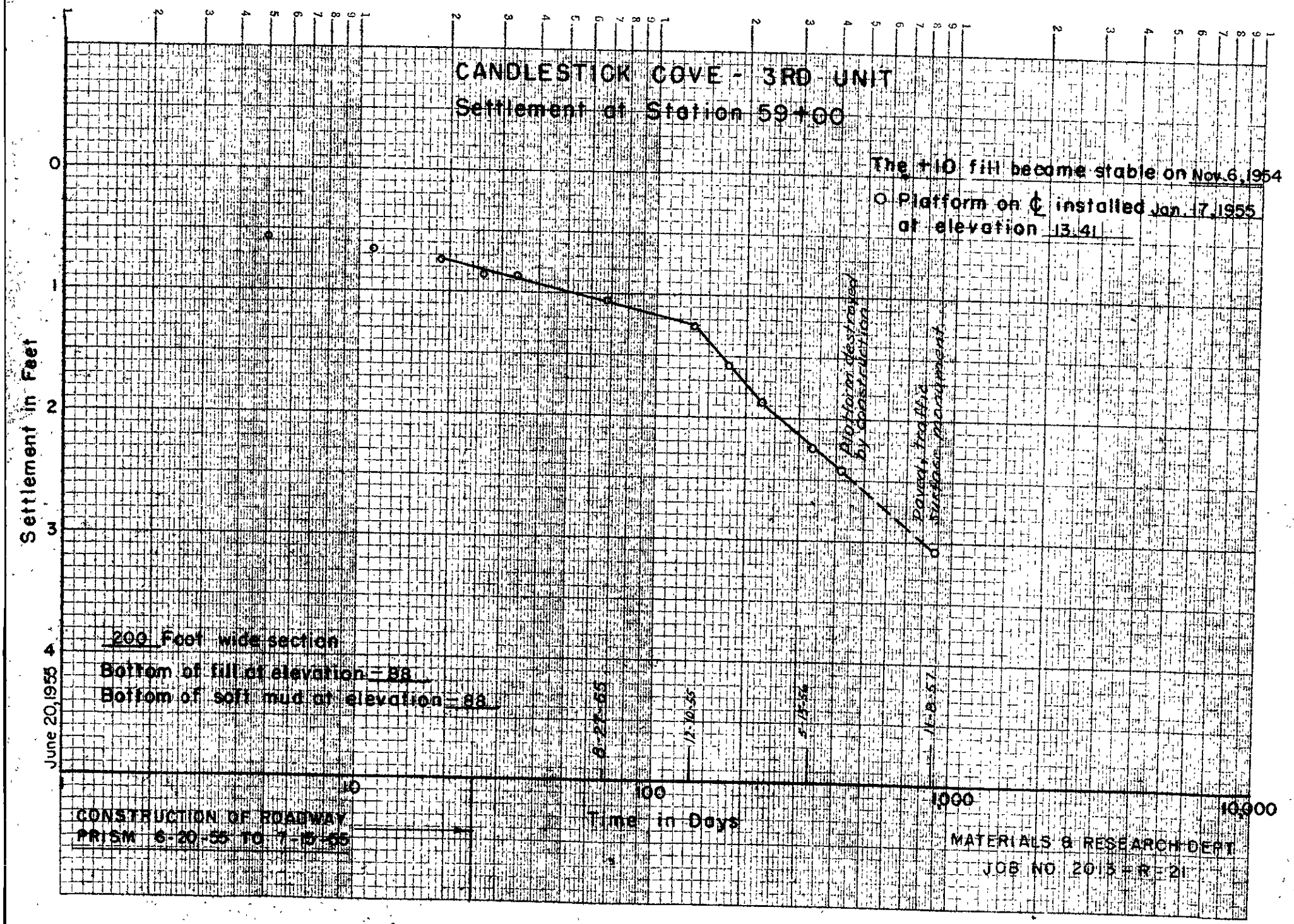



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 57 + 00
The +10 fill became stable on Oct. 10, 1954
O Platform on ℄ installed Jan. 17, 1953
at elevation 12.48
Settlement in Feet
0
1
2
3
4
June 20, 1955
Platform Destroyed by Construction
Paved traffic surface maintenance
200 Foot wide section
Bottom of fill at elevation -20
Bottom of soft mud at elevation -64
8-27-55
12-10-55
3-15-56
11-8-57
10
100
1,000
10,000
Time in Days
CONSTRUCTION OF ROADWAY PRISM 6-20-55 TO 7-15-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2015-R-21

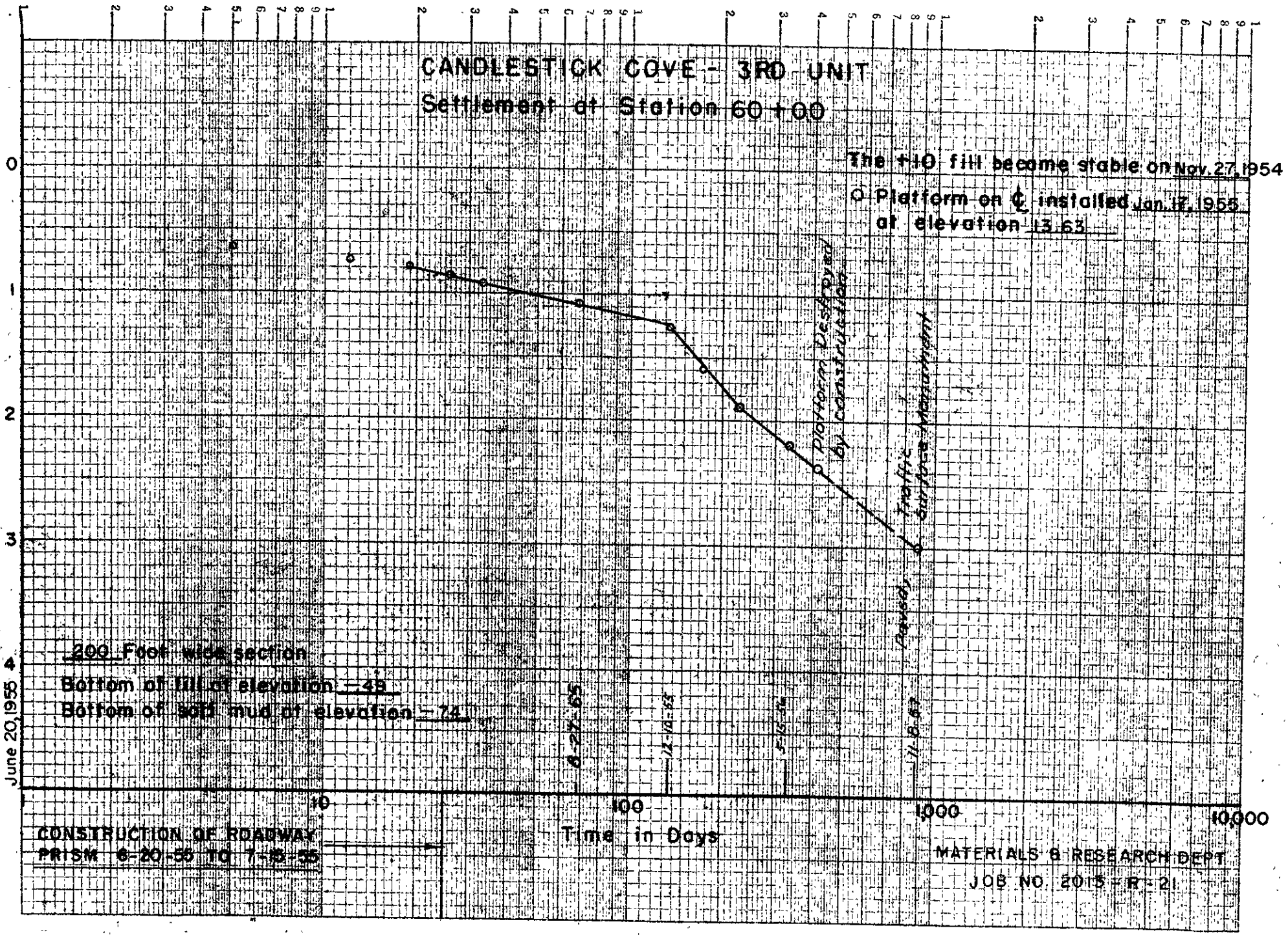
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



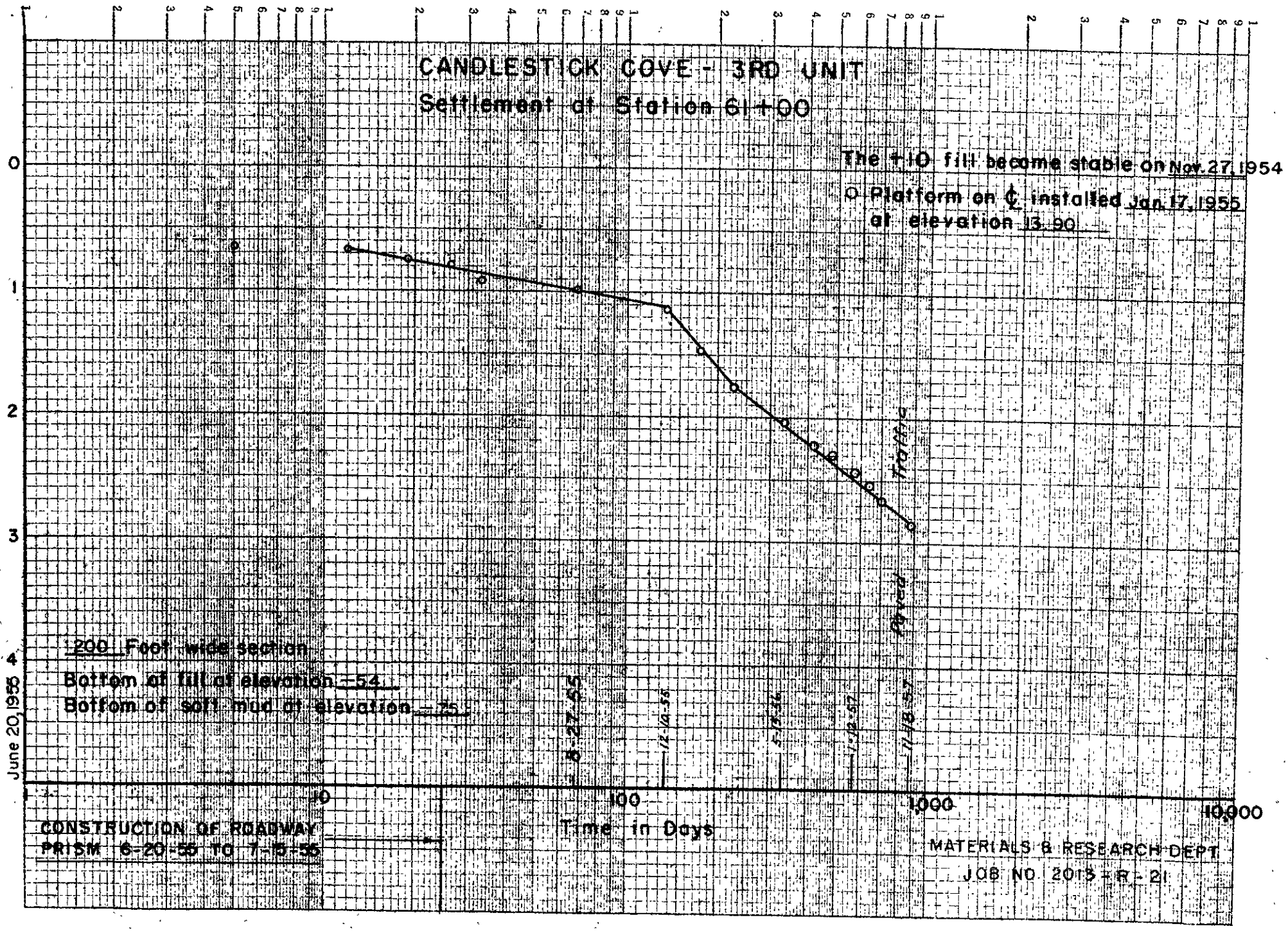
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

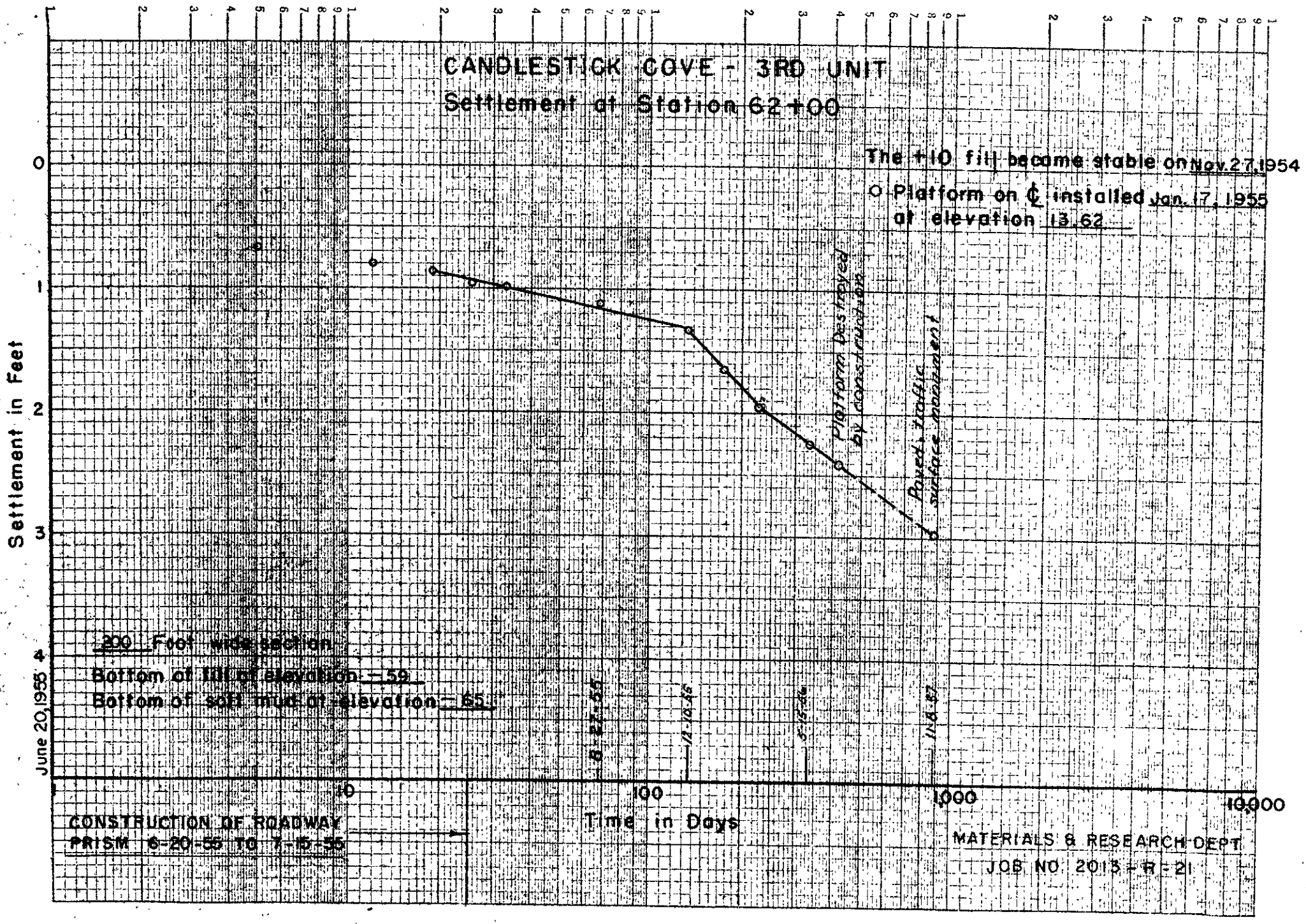


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

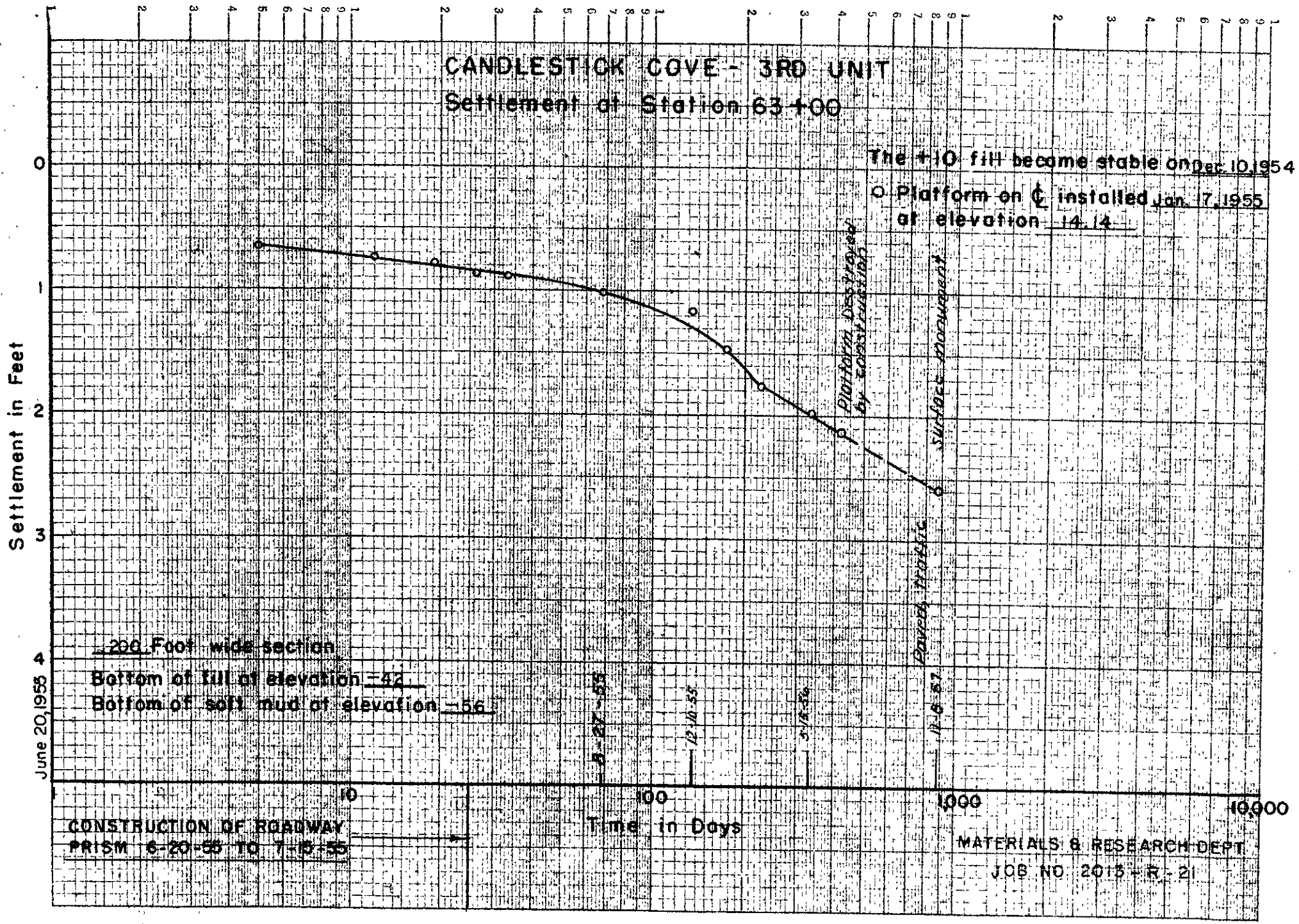


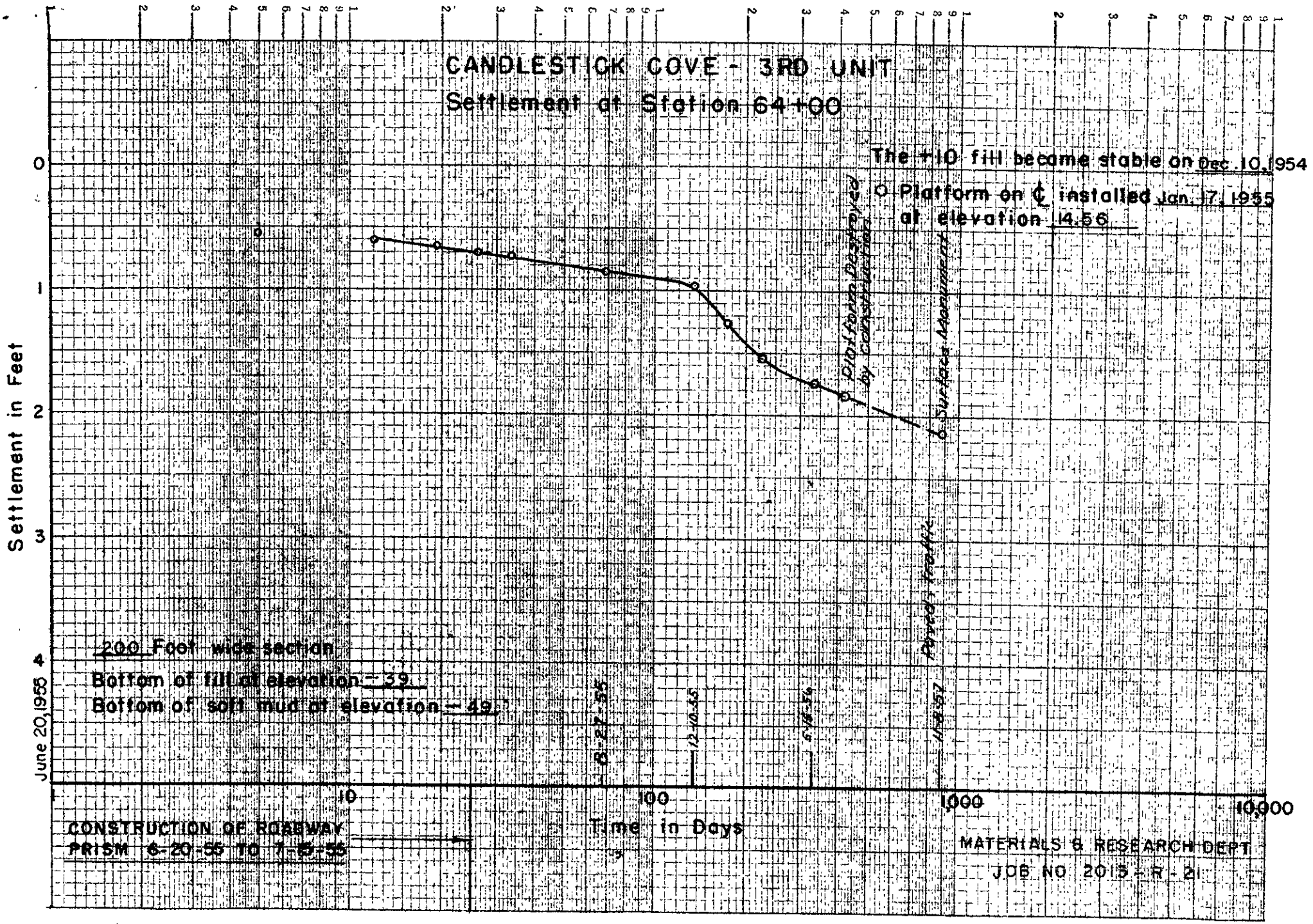
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

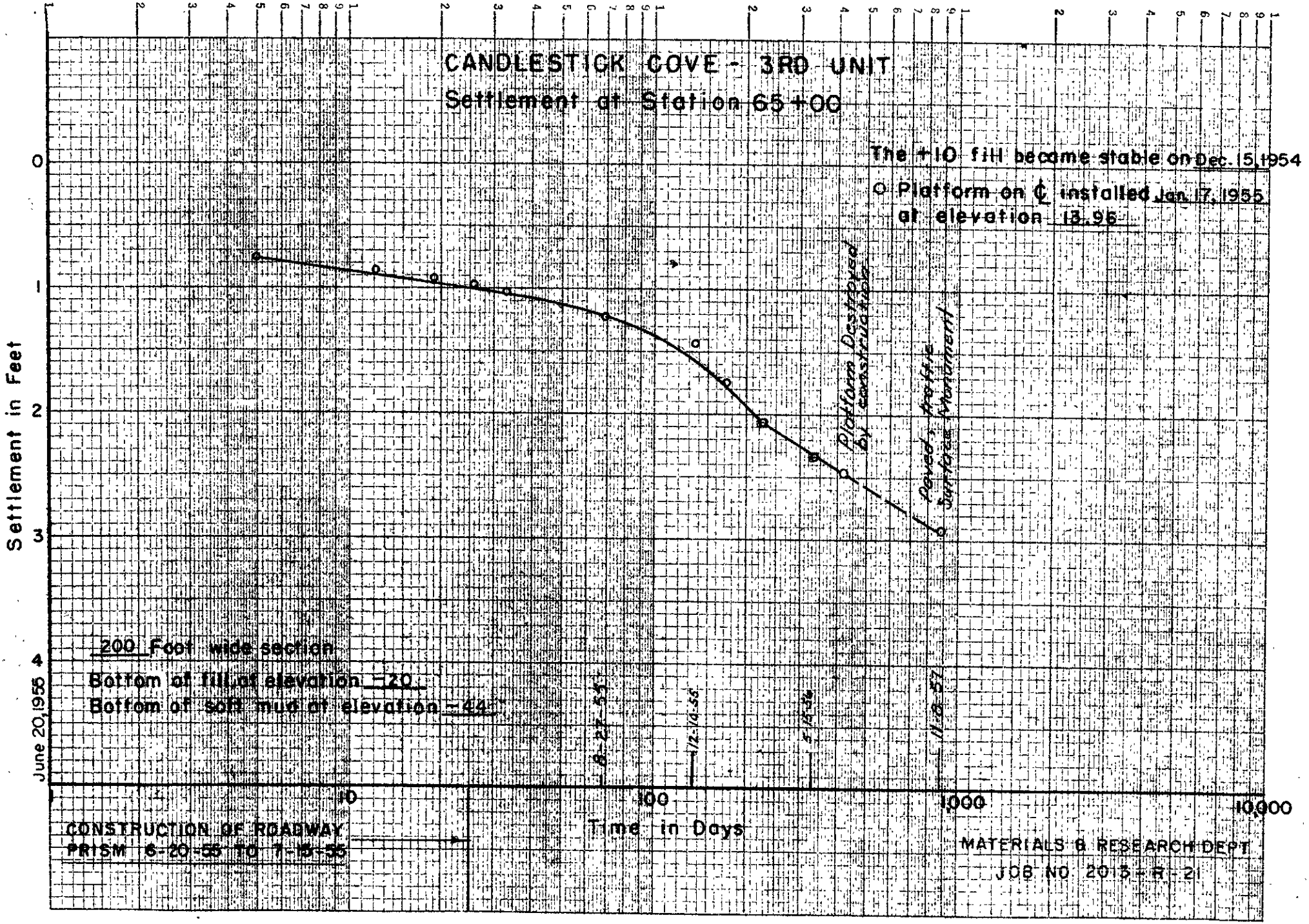



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 62+00
The +10 fill became stable on Nov. 27, 1954
O Platform on ℄ installed Jan. 17, 1955
at elevation 13.62
Platform Destroyed by construction
Paved Traffic surface placed
200 Feet wide section
Bottom of fill at elevation -59
Bottom of soft mud at elevation -65
CONSTRUCTION OF ROADWAY PRISM 6-20-55 TO 7-15-55
Settlement in Feet
Time in Days
June 20, 1955
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

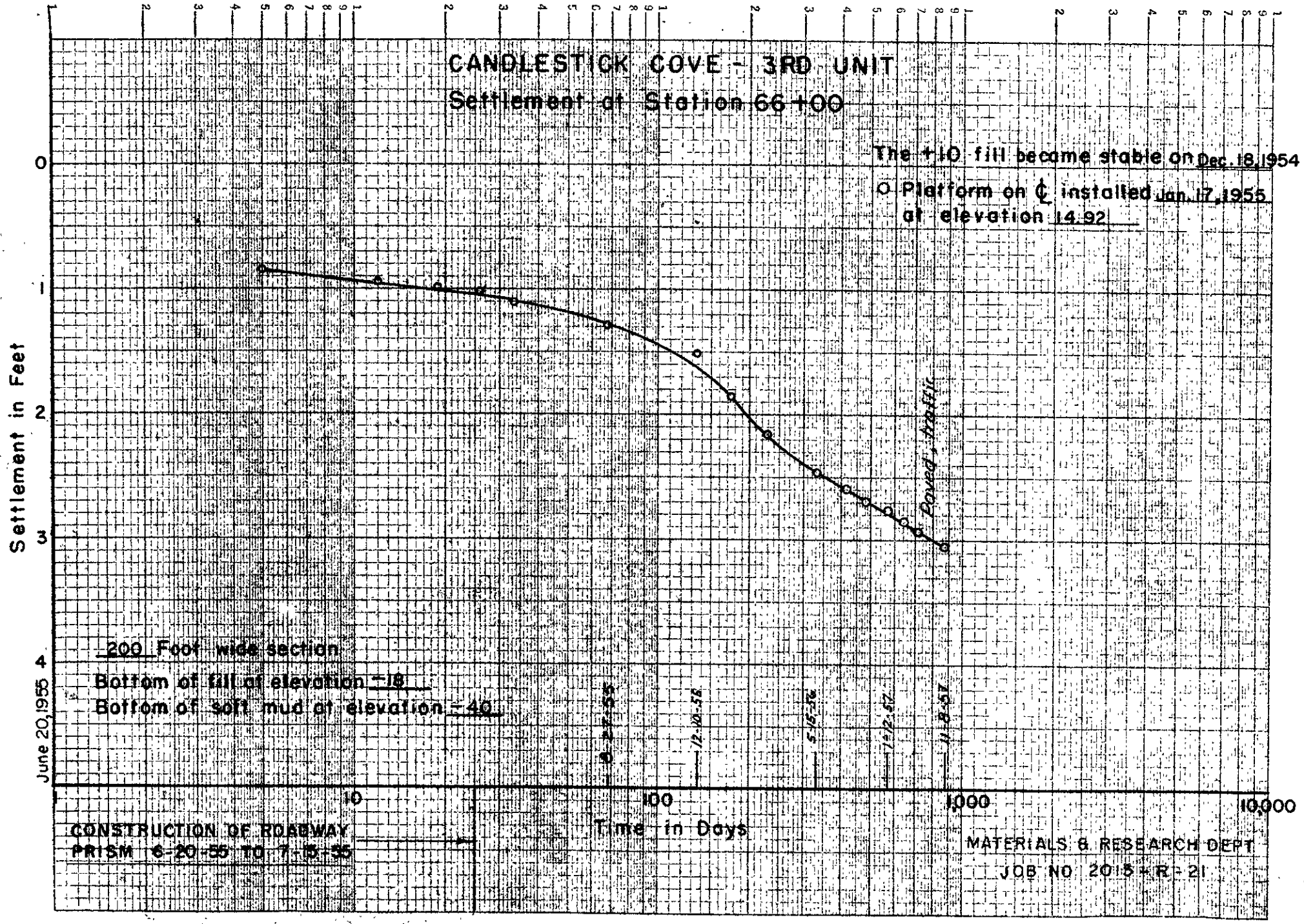
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



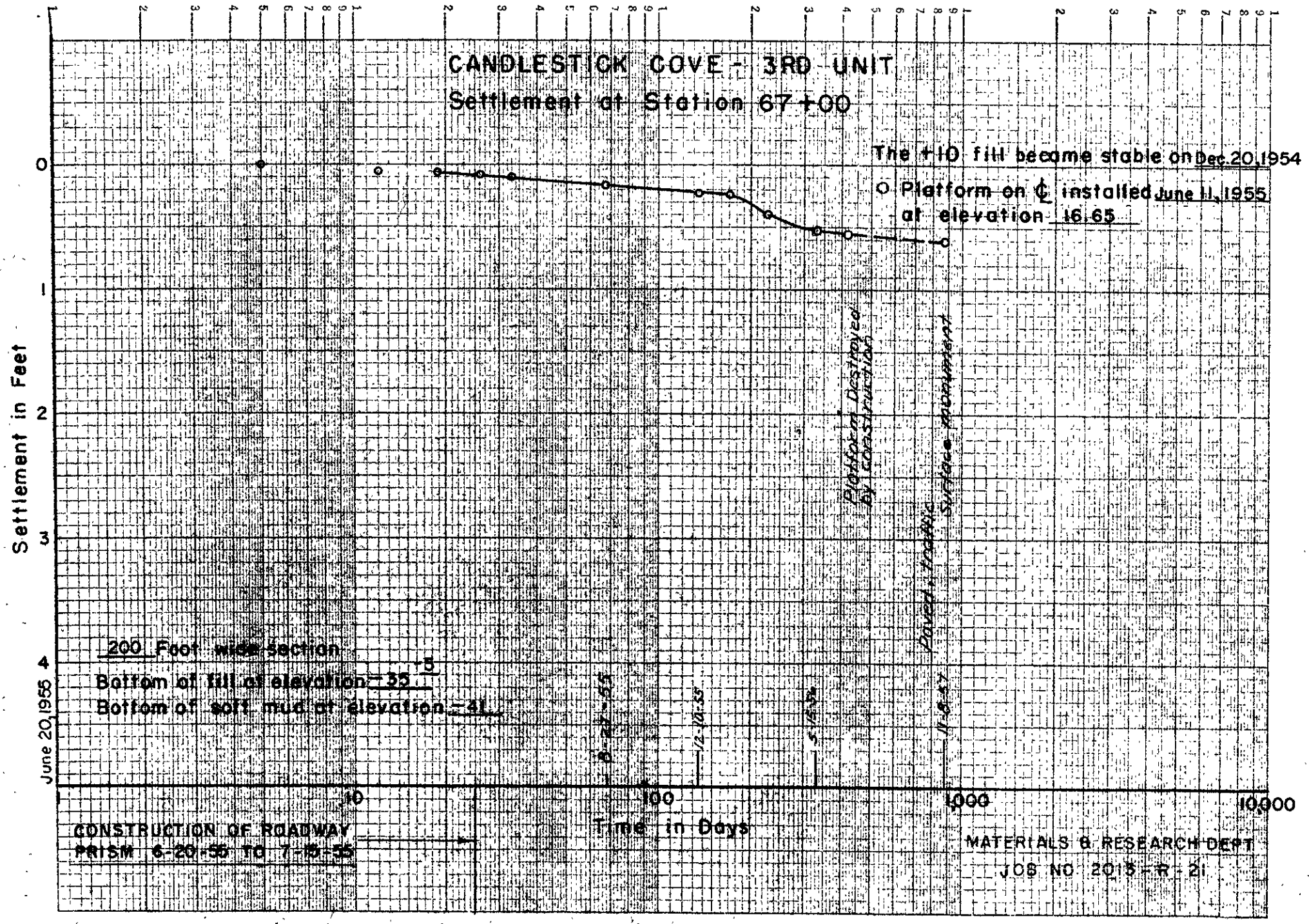

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 64+00
The +10 fill became stable on Dec. 10, 1954
Platform on ℄ installed Jan. 17, 1955
at elevation 14.56
Platform Destroyed by construction
Surface Movement
Paved Traffic
10-8-57
12-10-55
8-27-55
5-16-55
200 Foot wide section
Bottom of fill at elevation -39
Bottom of soft mud at elevation -49
June 20, 1955
Settlement in Feet
0
1
2
3
4
10
100
1000
10,000
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 6-20-55 TO 7-5-55
MATERIALS & RESEARCH DEPT
JOB NO 2015-R-21


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 65+00
The +10 fill became stable on Dec. 15, 1954
Platform on ℄ installed Jan. 17, 1955
at elevation 13.95
Platform Destroyed by construction
Paved Traffic Surface Movement
200 Foot wide section
Bottom of fill at elevation -20
Bottom of soft mud at elevation -44
8-27-55
12-14-55
5-12-56
11-8-57
CONSTRUCTION OF ROADWAY PRISM 6-20-55 TO 7-15-55
Settlement in Feet
June 20, 1955
Time in Days
MATERIALS & RESEARCH DEPT
JOB NO. 2013-R-21

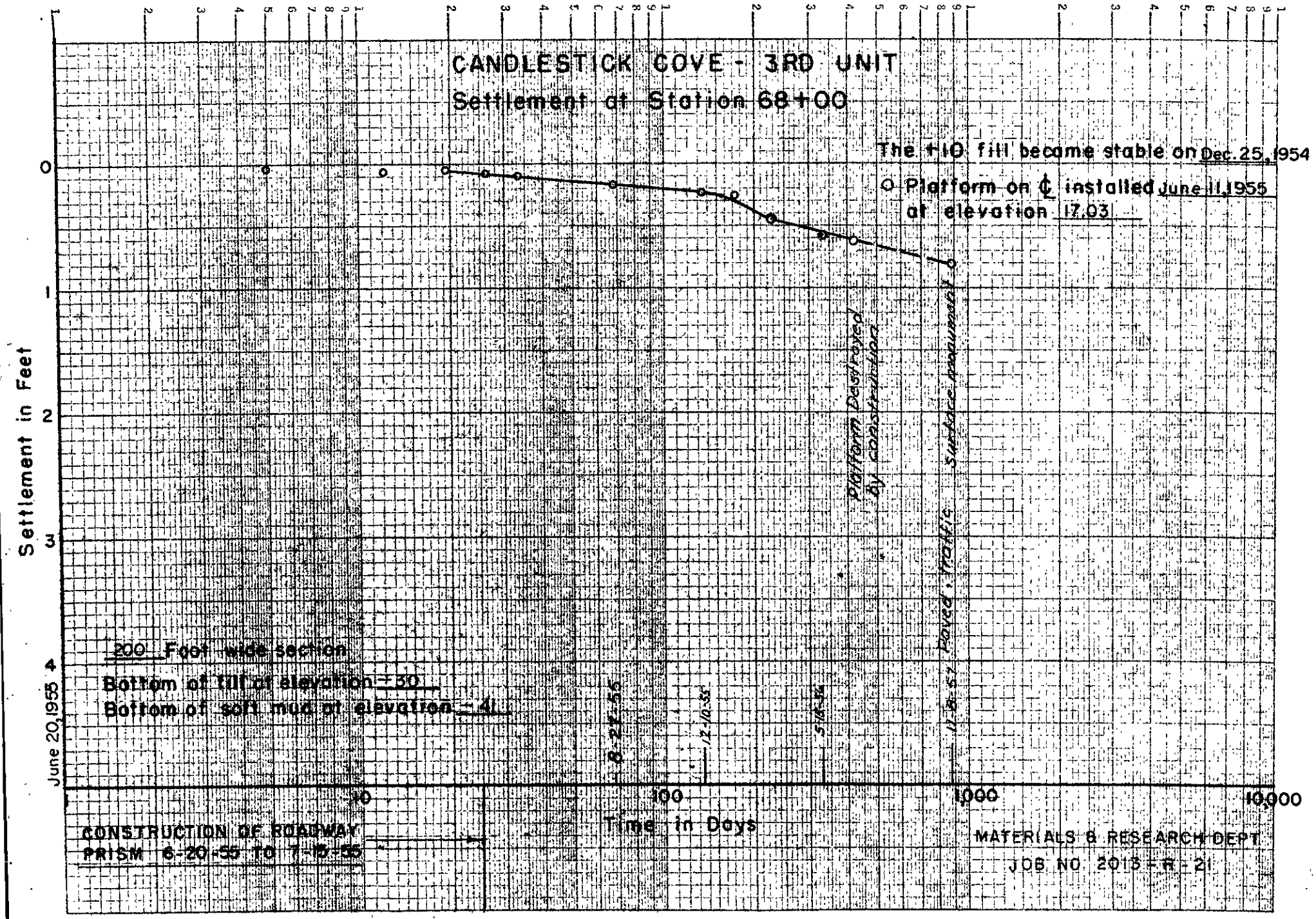
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



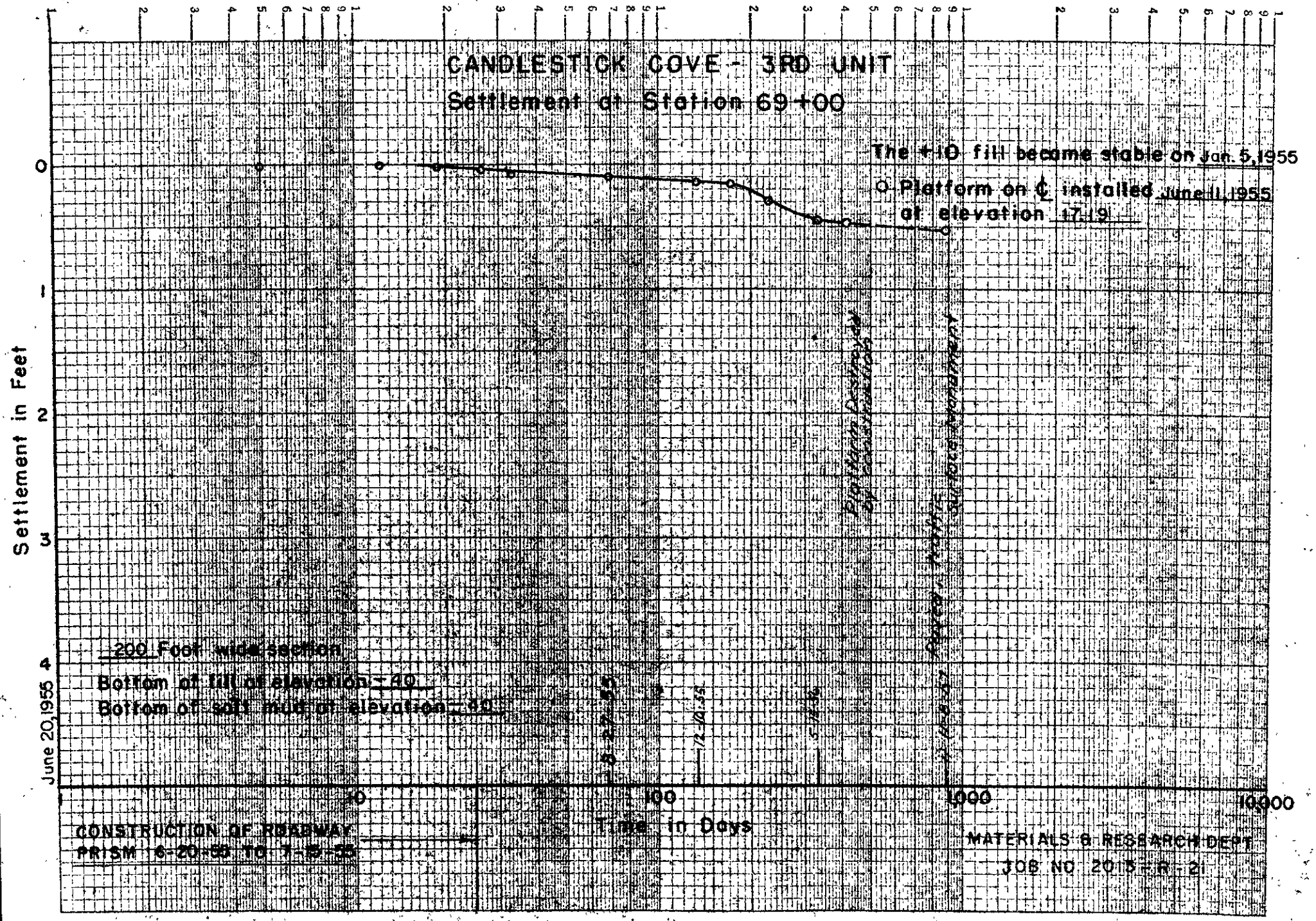
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



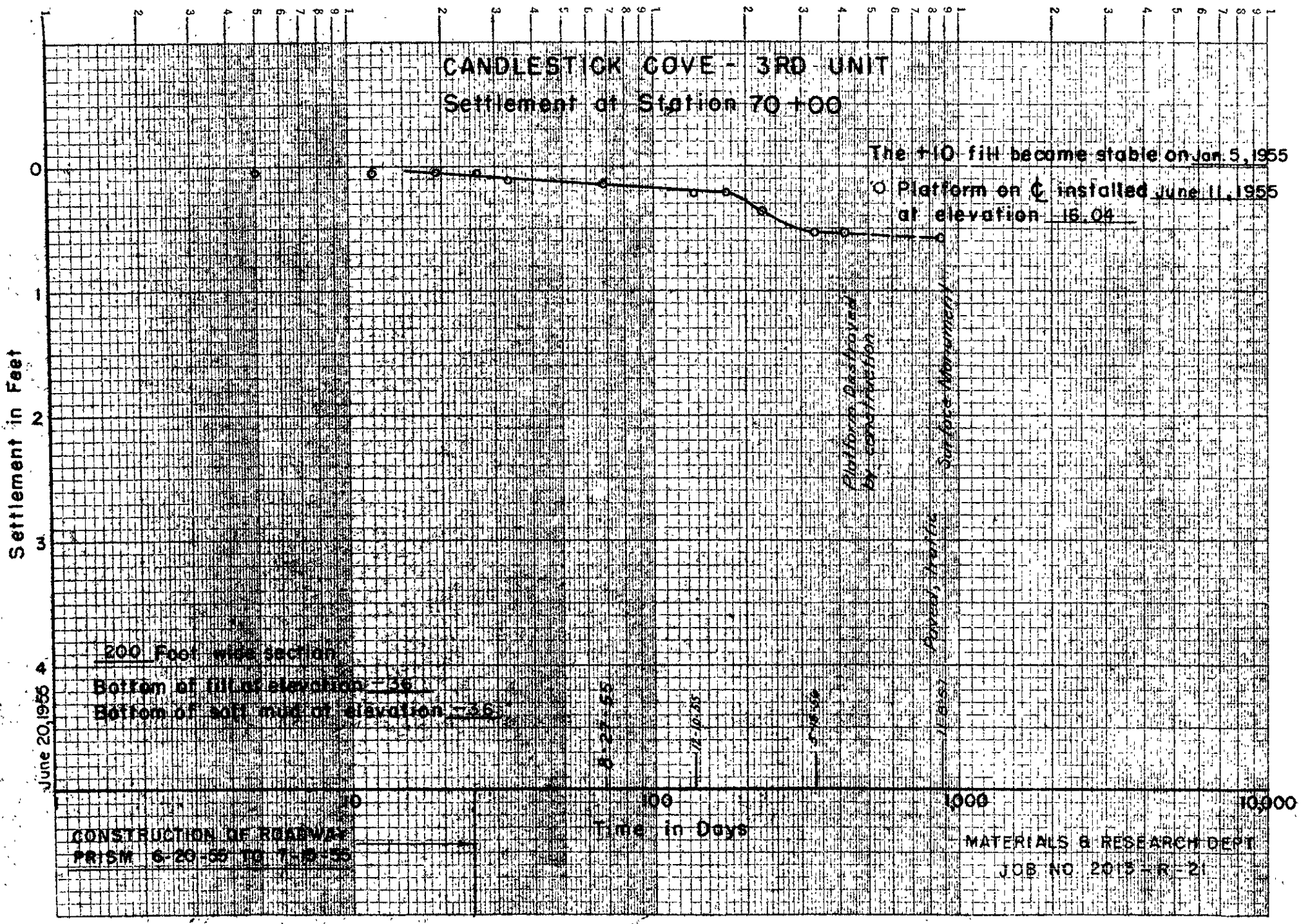
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

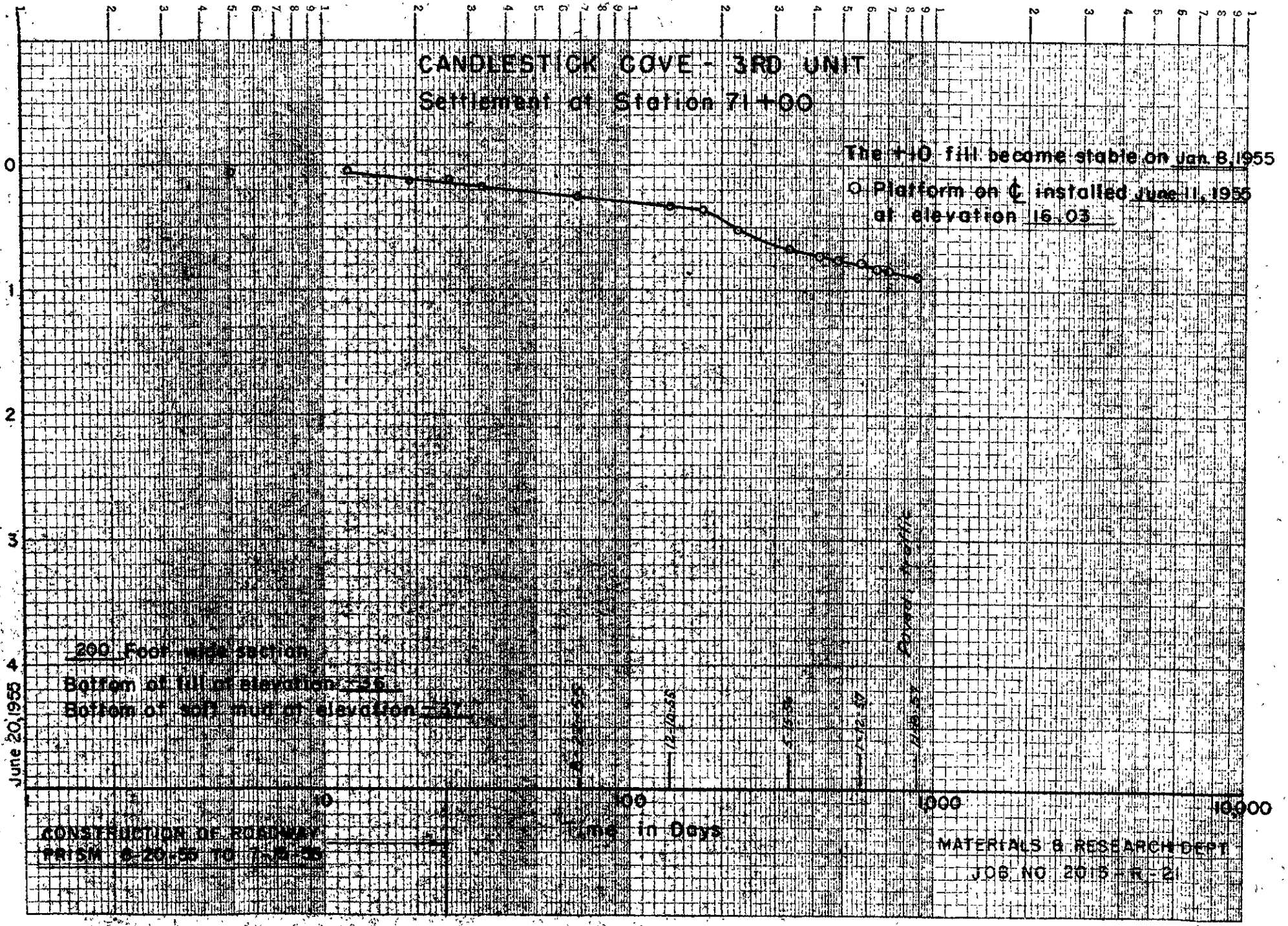


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

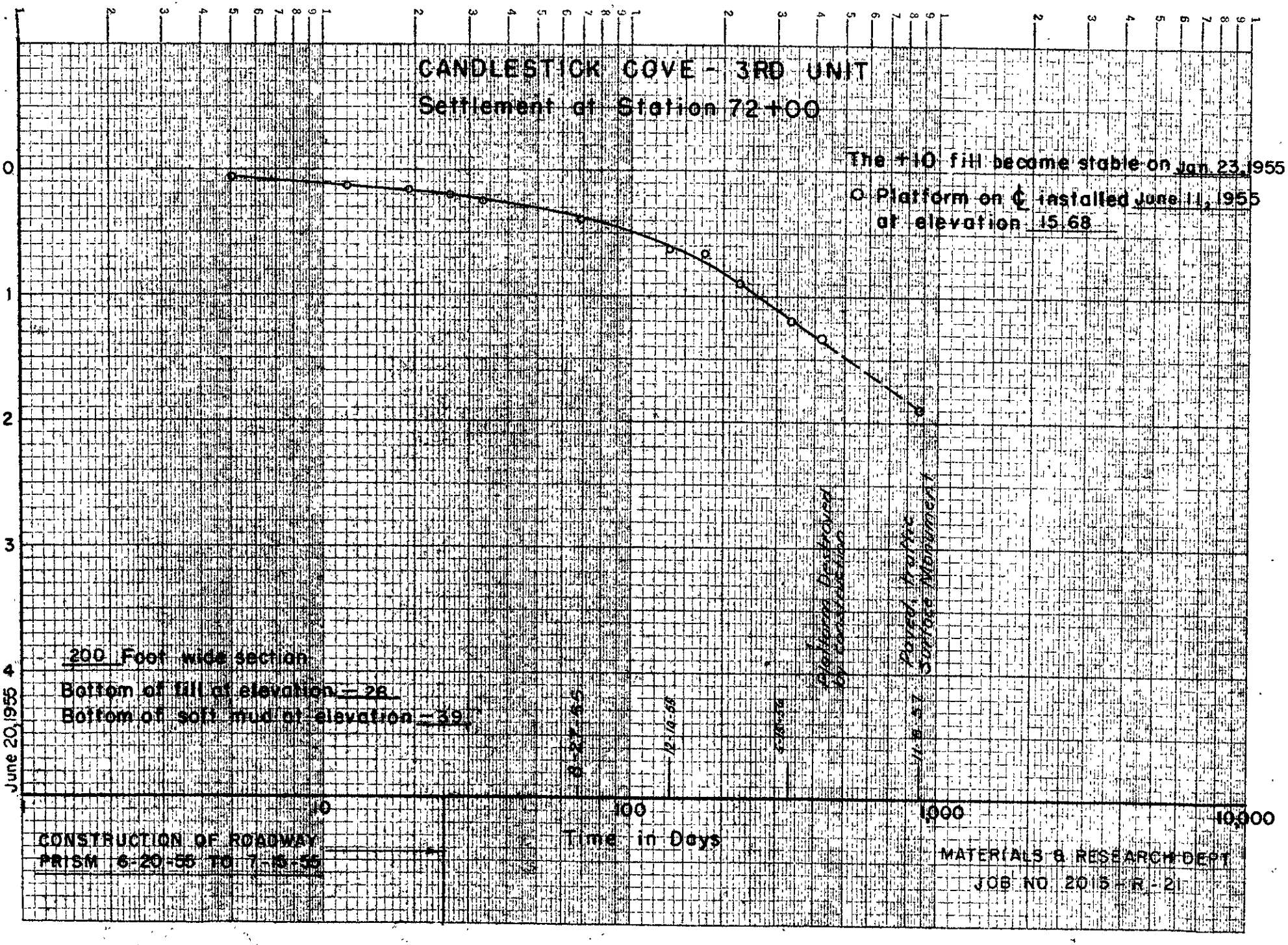


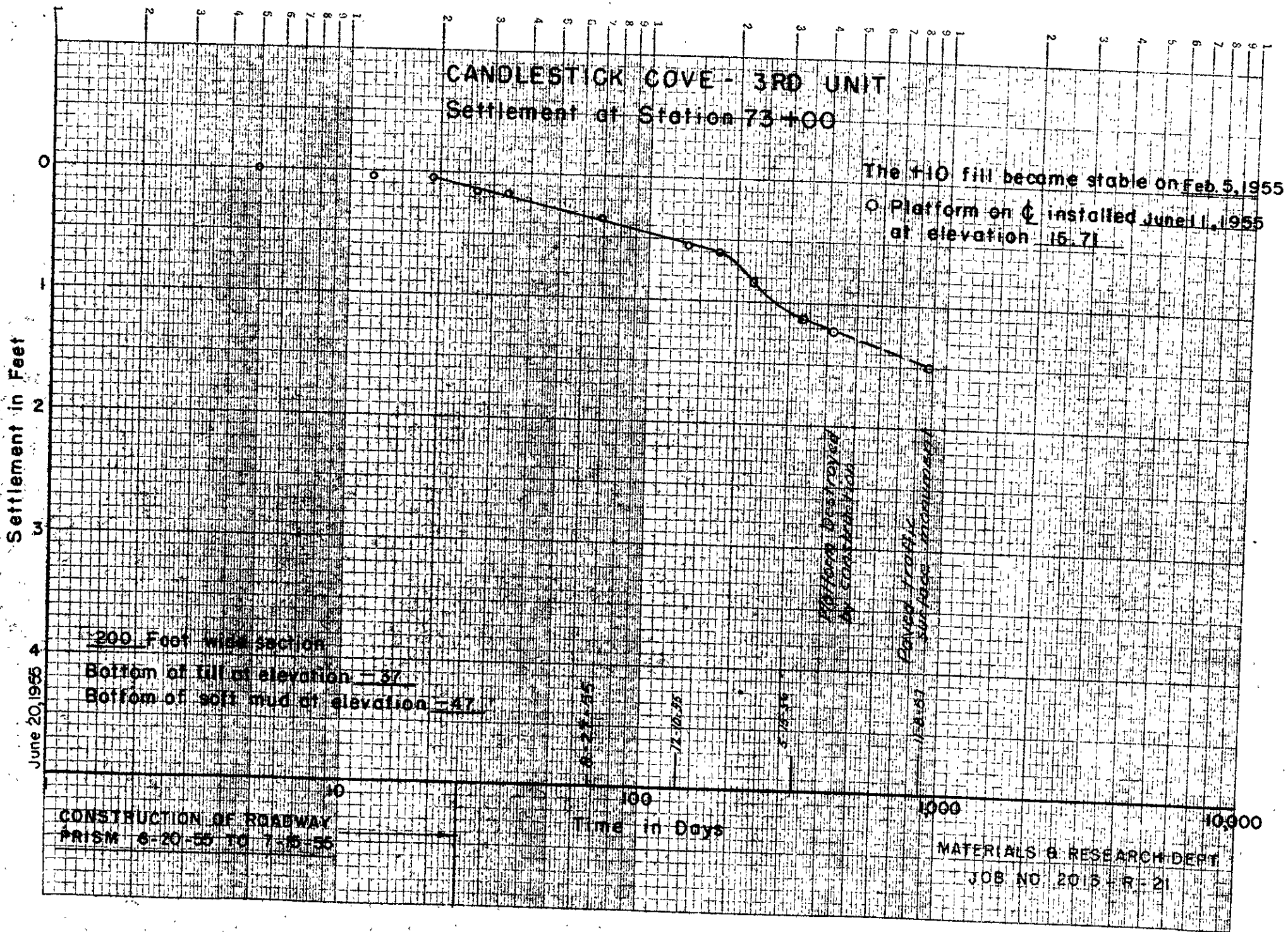
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



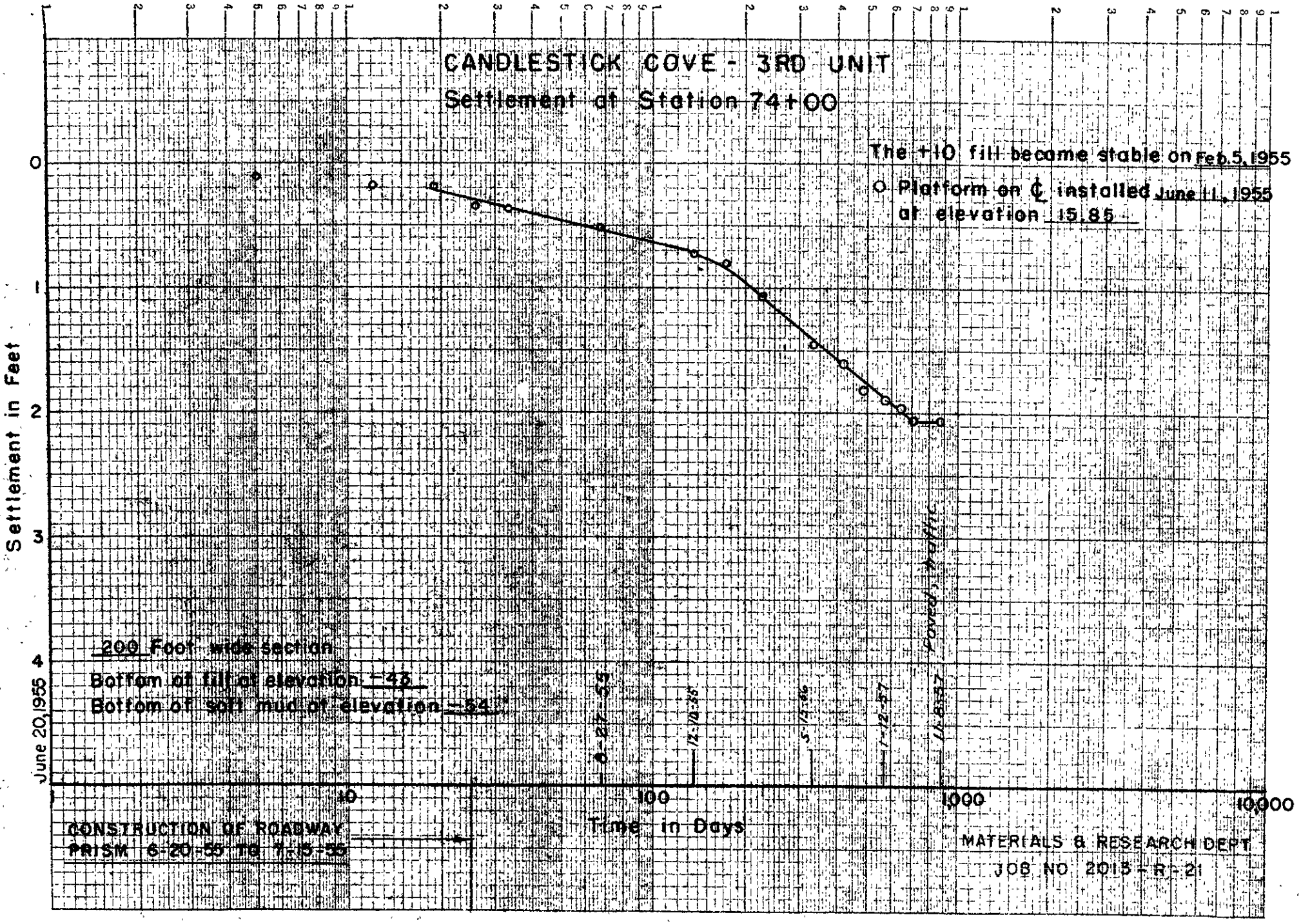

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 71+00
The +10 fill became stable on Jan. 8, 1955
Platform on ℄ installed June 11, 1955
at elevation 16.03
Settlement in Feet
0
1
2
3
4
June 20, 1955
200 Foot wide section
Bottom of fill at elevation +3.5
Bottom of soft mud at elevation -3.7
12-10-55
4-25-56
12-12-57
Point of traffic
1
10
100
1000
10,000
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 6-20-55 TO 7-8-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2015-R-21

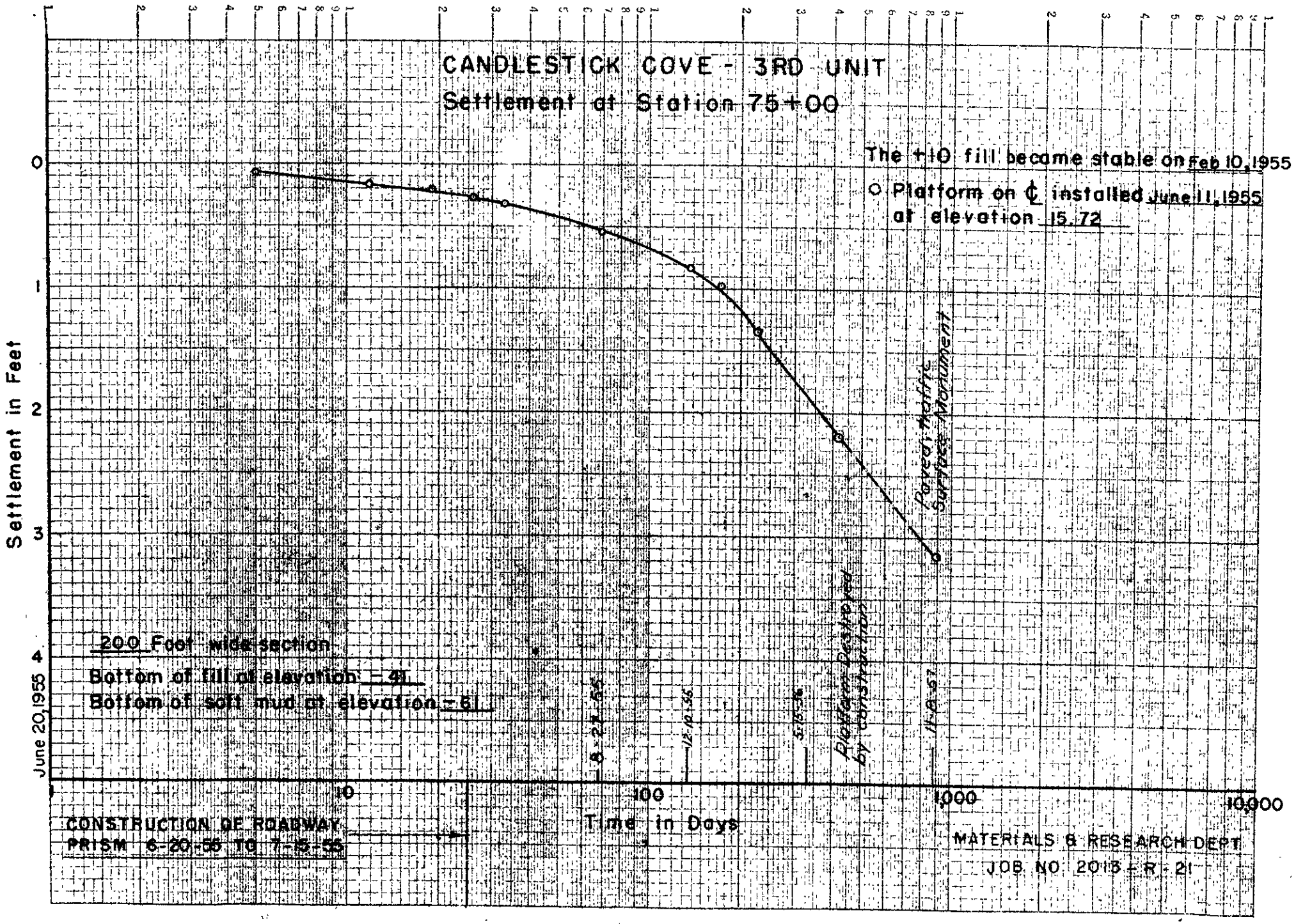
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



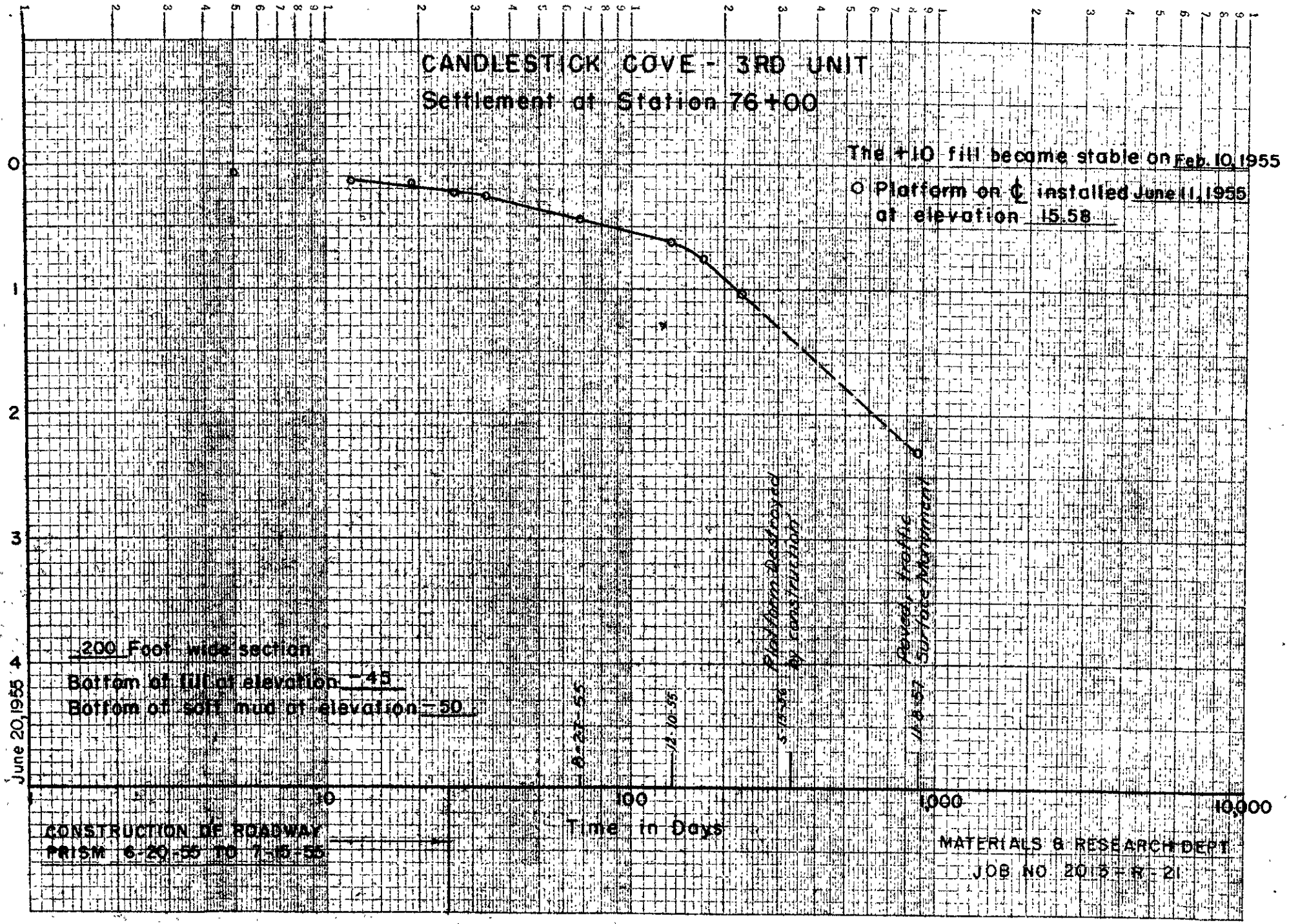

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 73+00
The +10 fill became stable on Feb. 5, 1955
○ Platform on ℄ installed June 11, 1955
at elevation 15.71
Platform destroyed by construction
Paved traffic surface installed
200 Foot wide section
Bottom of fill at elevation -37
Bottom of soft mud at elevation -47
CONSTRUCTION OF ROADWAY PRISM 6-20-55 TO 7-15-55
Settlement in Feet
Time in Days
June 20, 1955
MATERIALS & RESEARCH DEPT.
JOB NO. 2015-R-21

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

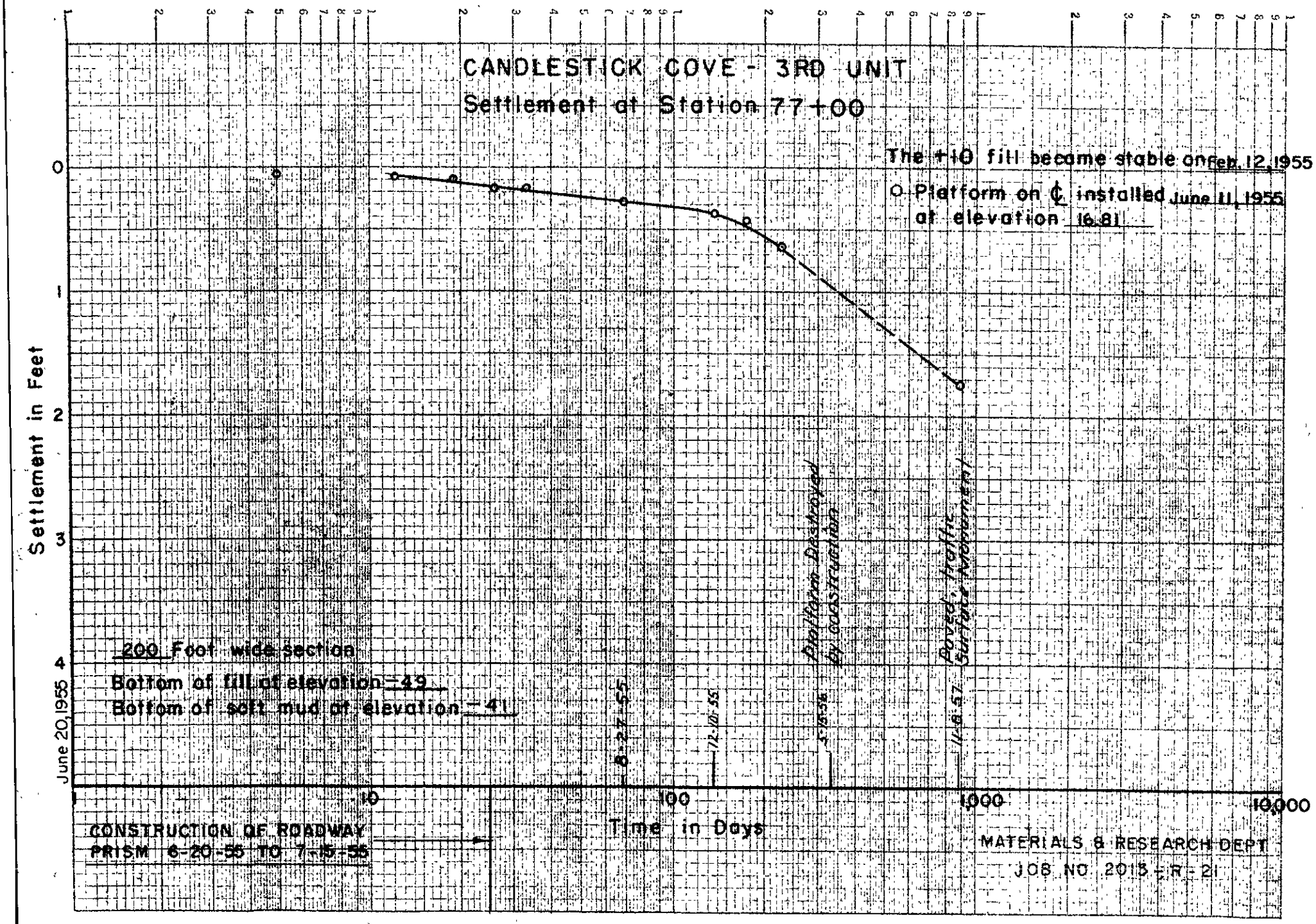



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 75+00
The +10 fill became stable on Feb 10, 1955
Platform on ℄ installed June 11, 1955 at elevation 15.72
Settlement in Feet
0
1
2
3
4
June 20, 1955
200 Foot wide section
Bottom of fill at elevation -4
Bottom of soft mud at elevation -6
8-27-55
12-10-55
5-15-56
Platform Removed by construction
11-8-57
Paved Traffic Surface & Monument
10
100
1000
10,000
Time in Days
CONSTRUCTION OF ROADWAY PRISM 6-20-55 TO 7-15-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

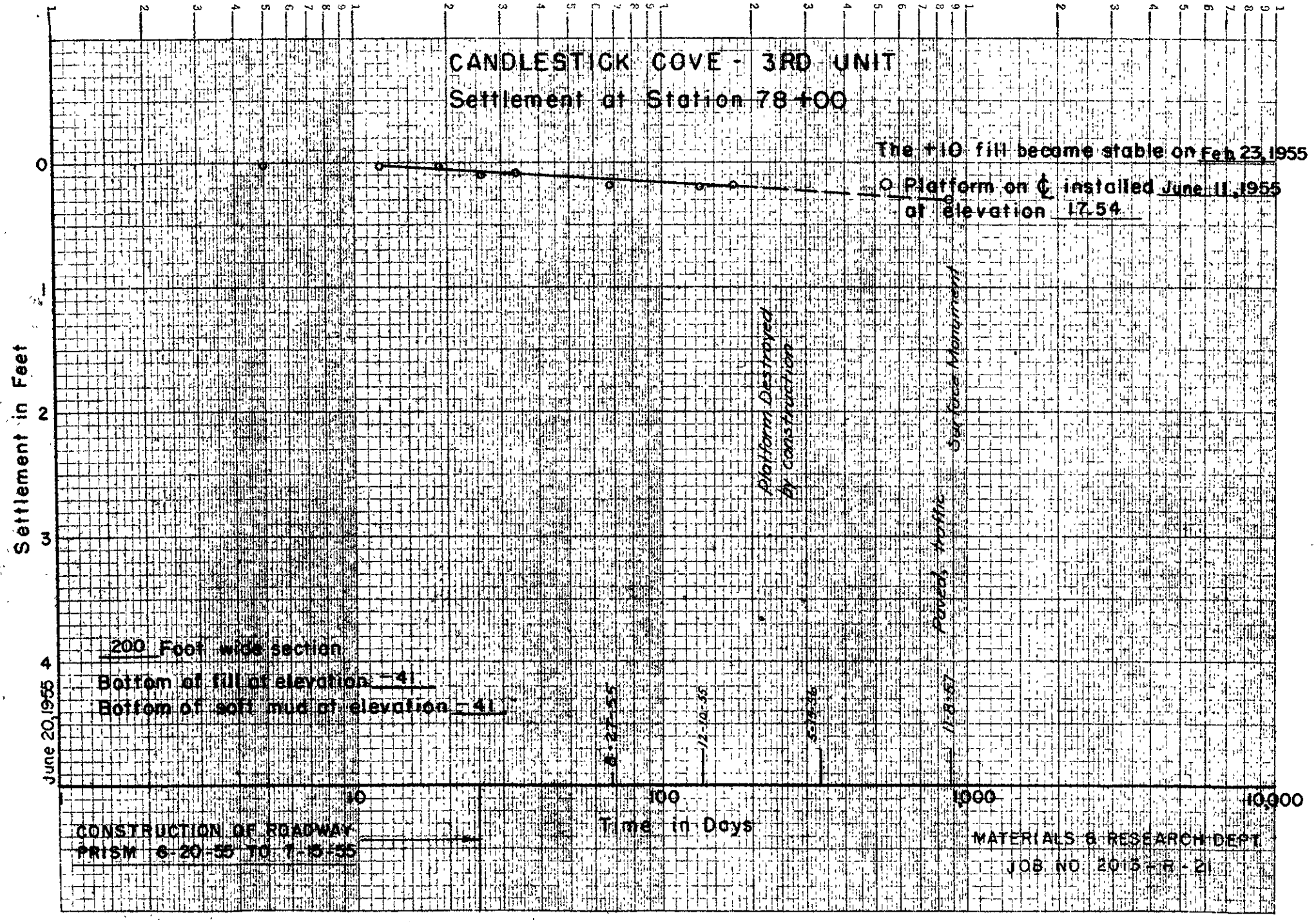
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

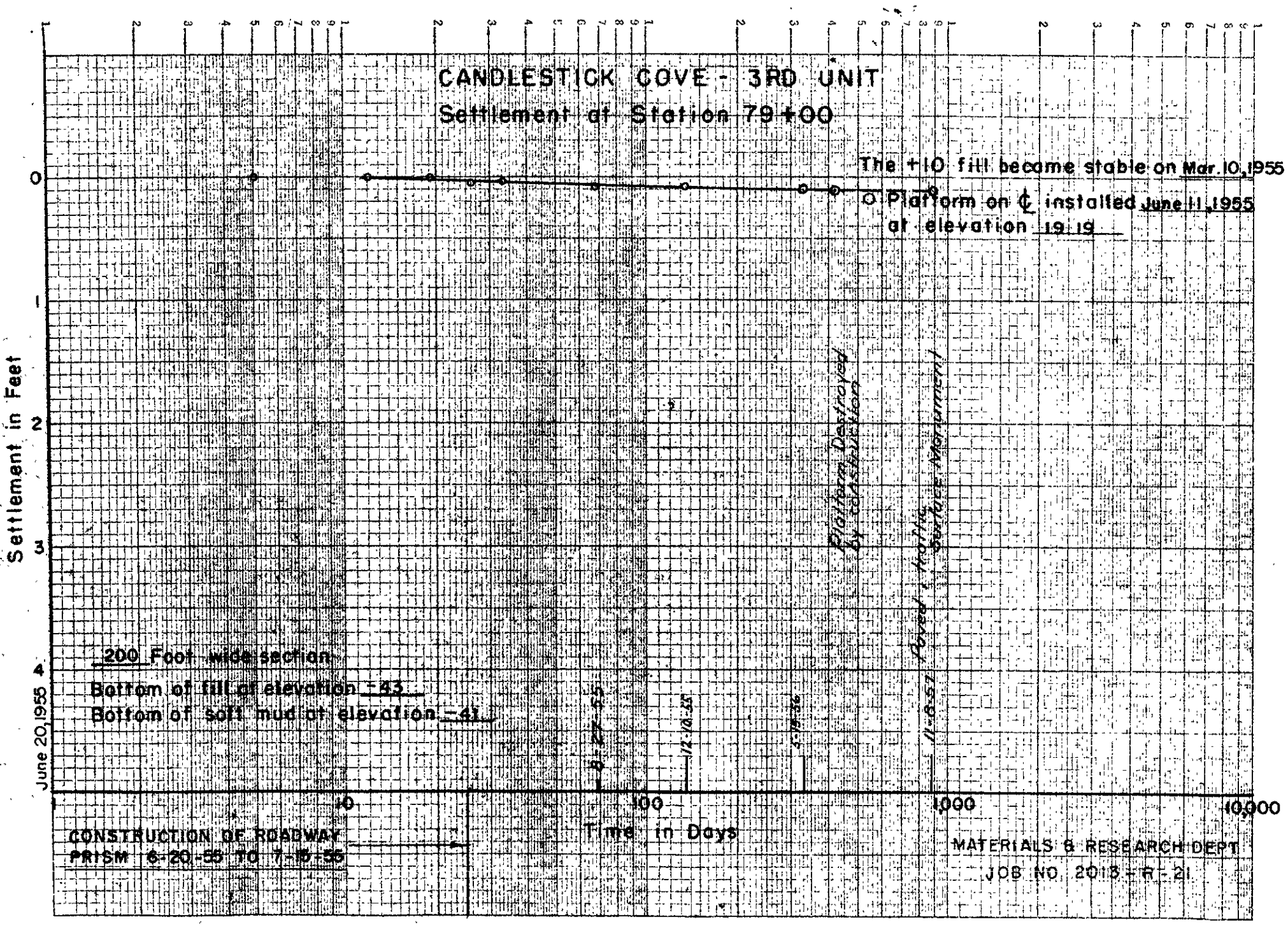


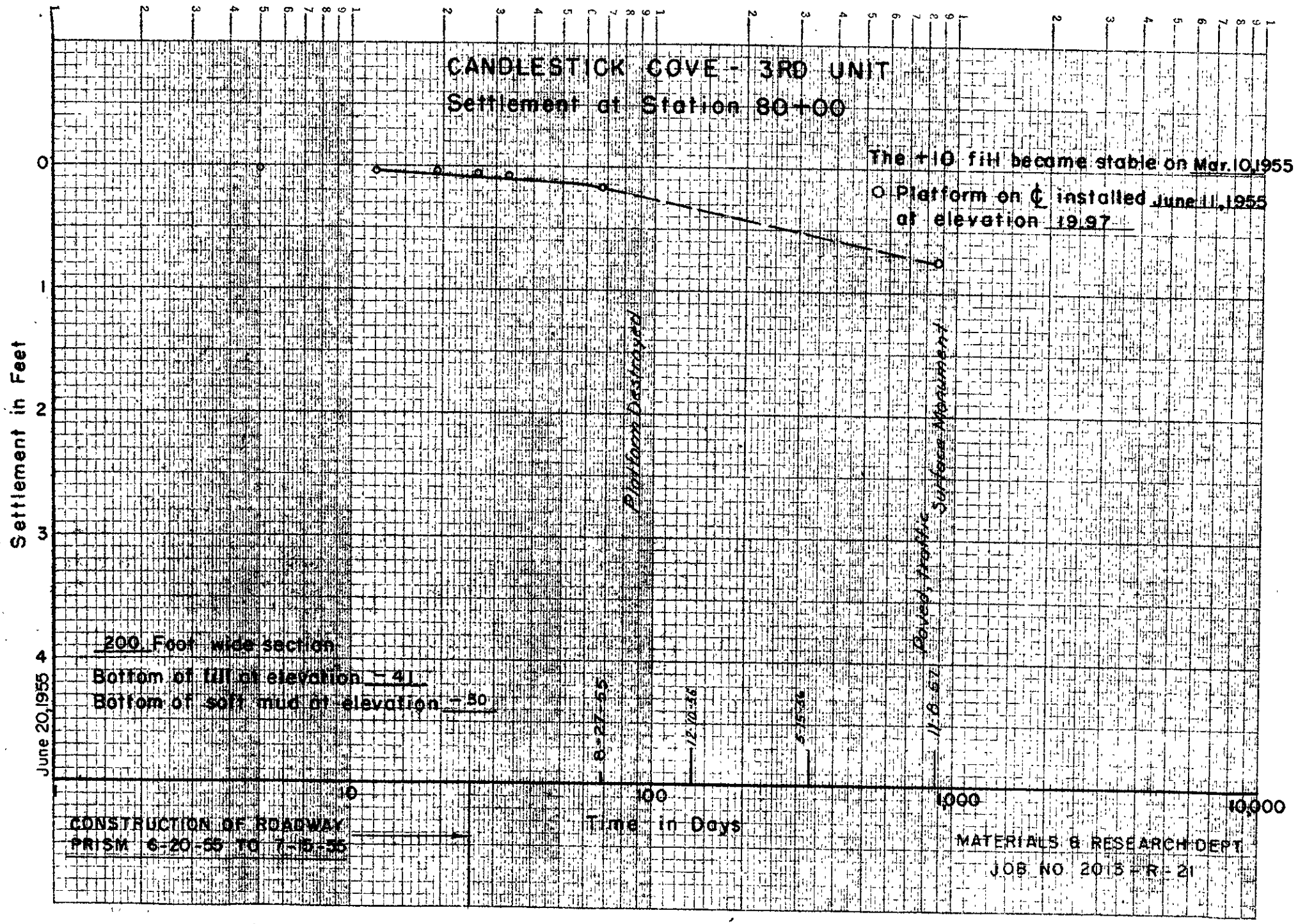
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



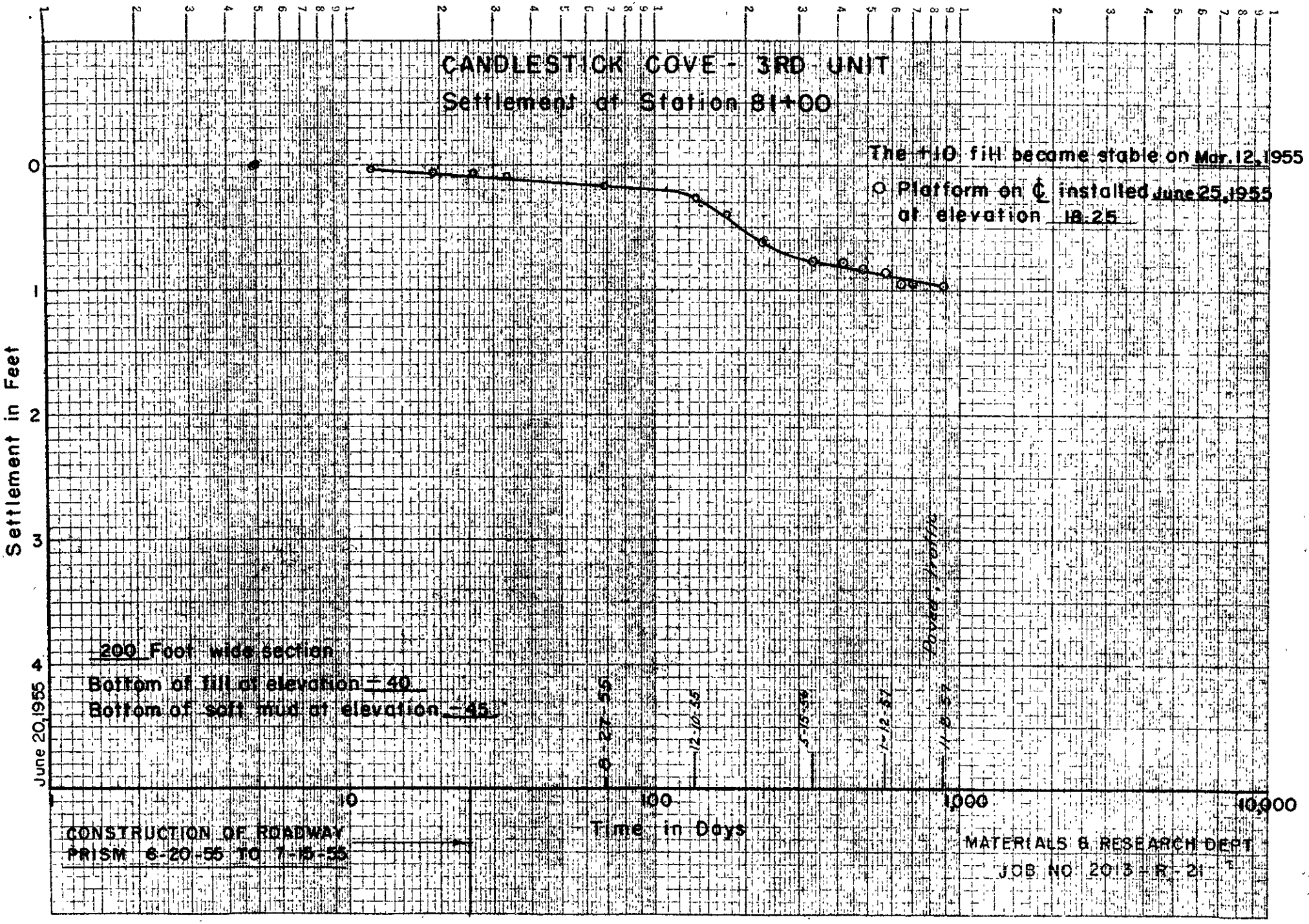
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



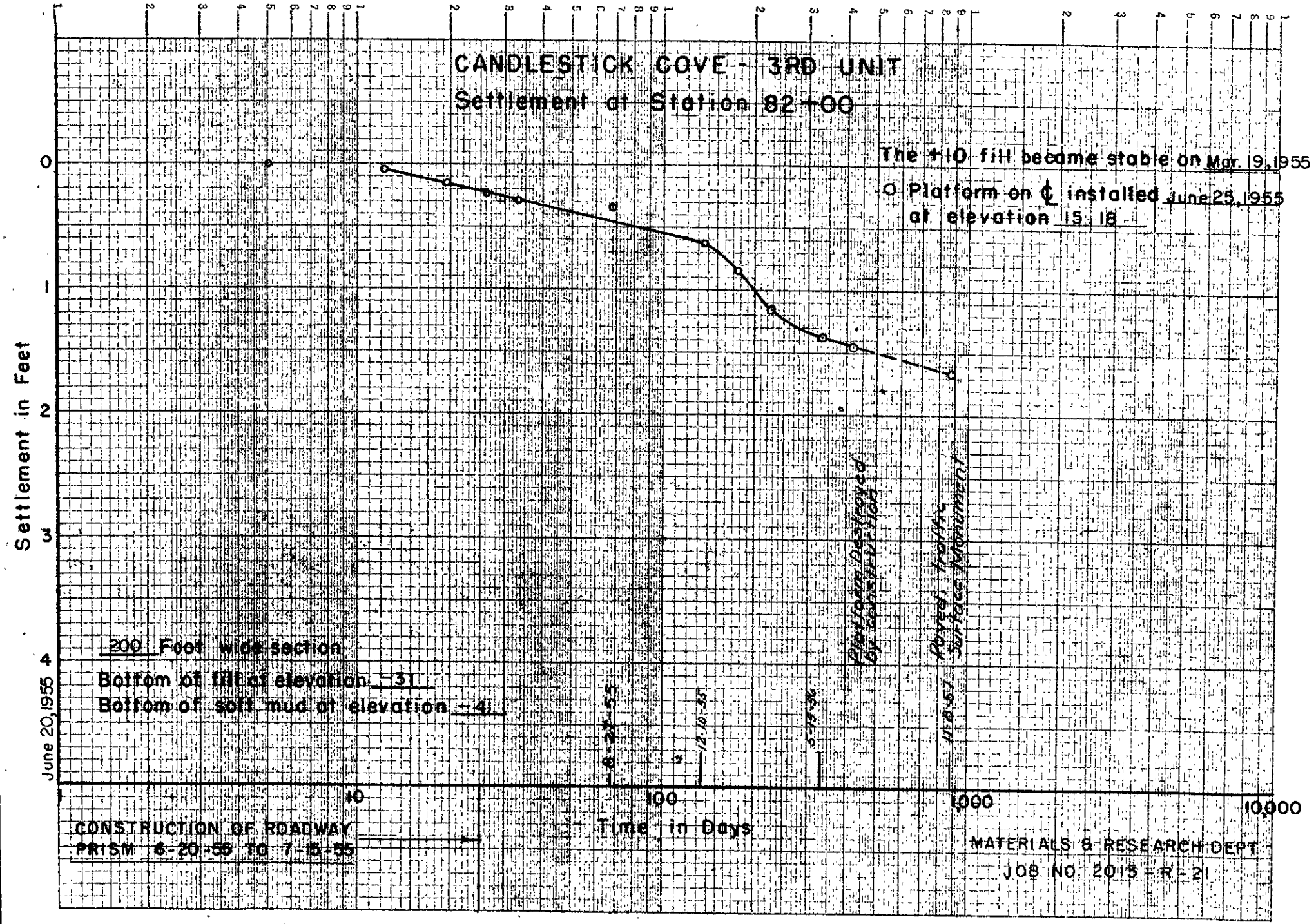

K+E SEMI LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 79+00
The +10 fill became stable on Mar. 10, 1955
Platform on ℄ installed June 11, 1955
at elevation 19.19
Platform Destroyed by construction
Road Paving
Surface Monument
200 Foot wide section
Bottom of fill at elevation -43
Bottom of soft mud at elevation -41
6-27-55
12-10-55
5-19-56
11-8-57
Settlement in Feet
0
1
2
3
4
June 20, 1955
1
10
100
1000
10,000
Time in Days
CONSTRUCTION OF ROADWAY PRISM 6-20-55 TO 7-15-55
MATERIALS & RESEARCH DEPT
JOB NO. 2013-R-21


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 80+00
The +10 fill became stable on Mar. 10, 1955
O Platform on ℄ installed June 11, 1955
at elevation 19.97
Platform Destroyed
Paved, Traffic Surface Movement
8-27-55
12-10-56
5-15-56
11-8-57
200 Foot wide section
Bottom of fill at elevation -41
Bottom of soft mud at elevation -50
CONSTRUCTION OF ROADWAY
PRISM 6-20-55 TO 7-5-55
Settlement in Feet
June 20, 1955
Time in Days
MATERIALS & RESEARCH DEPT.
JOB NO. 2015-R-21

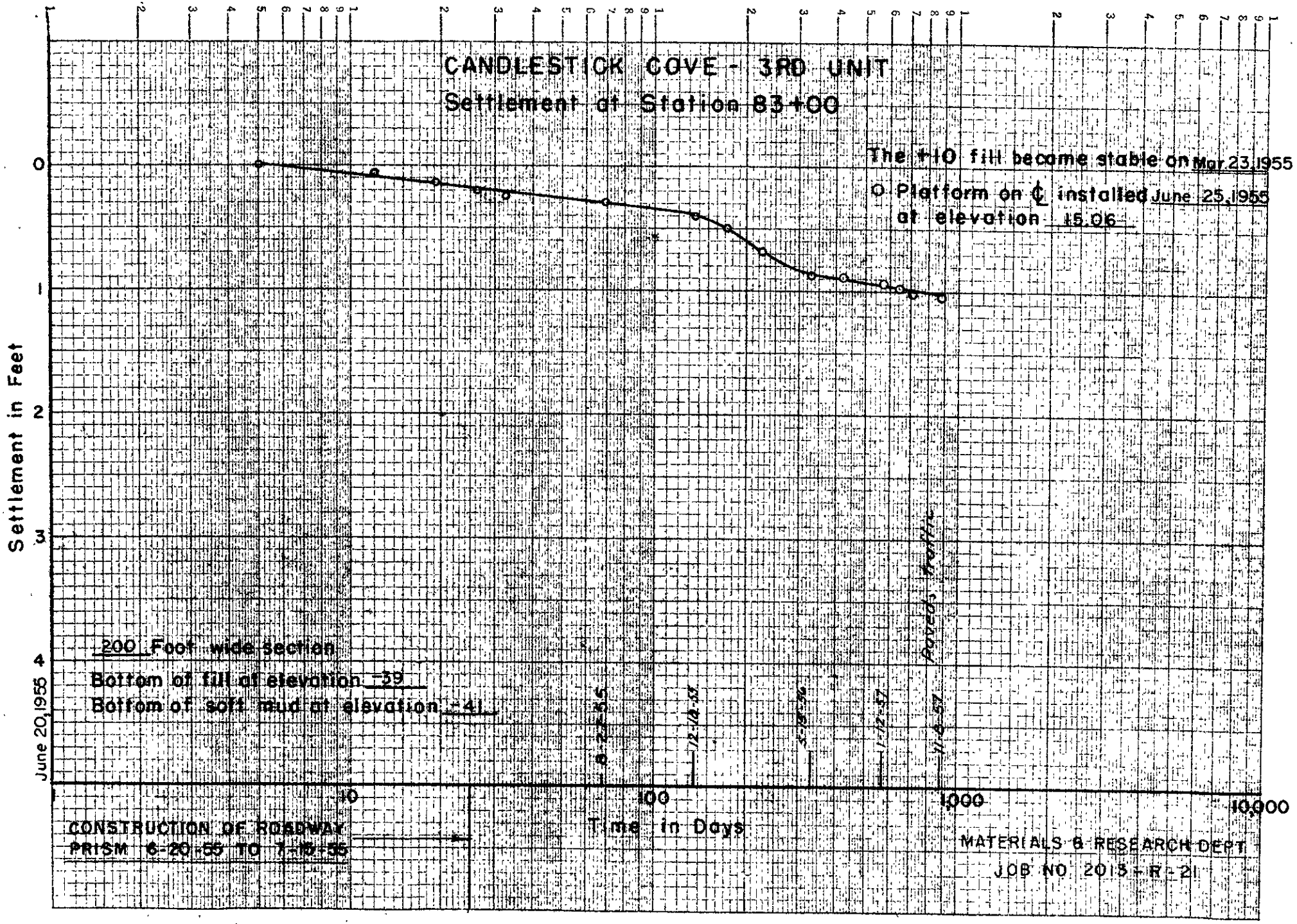
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



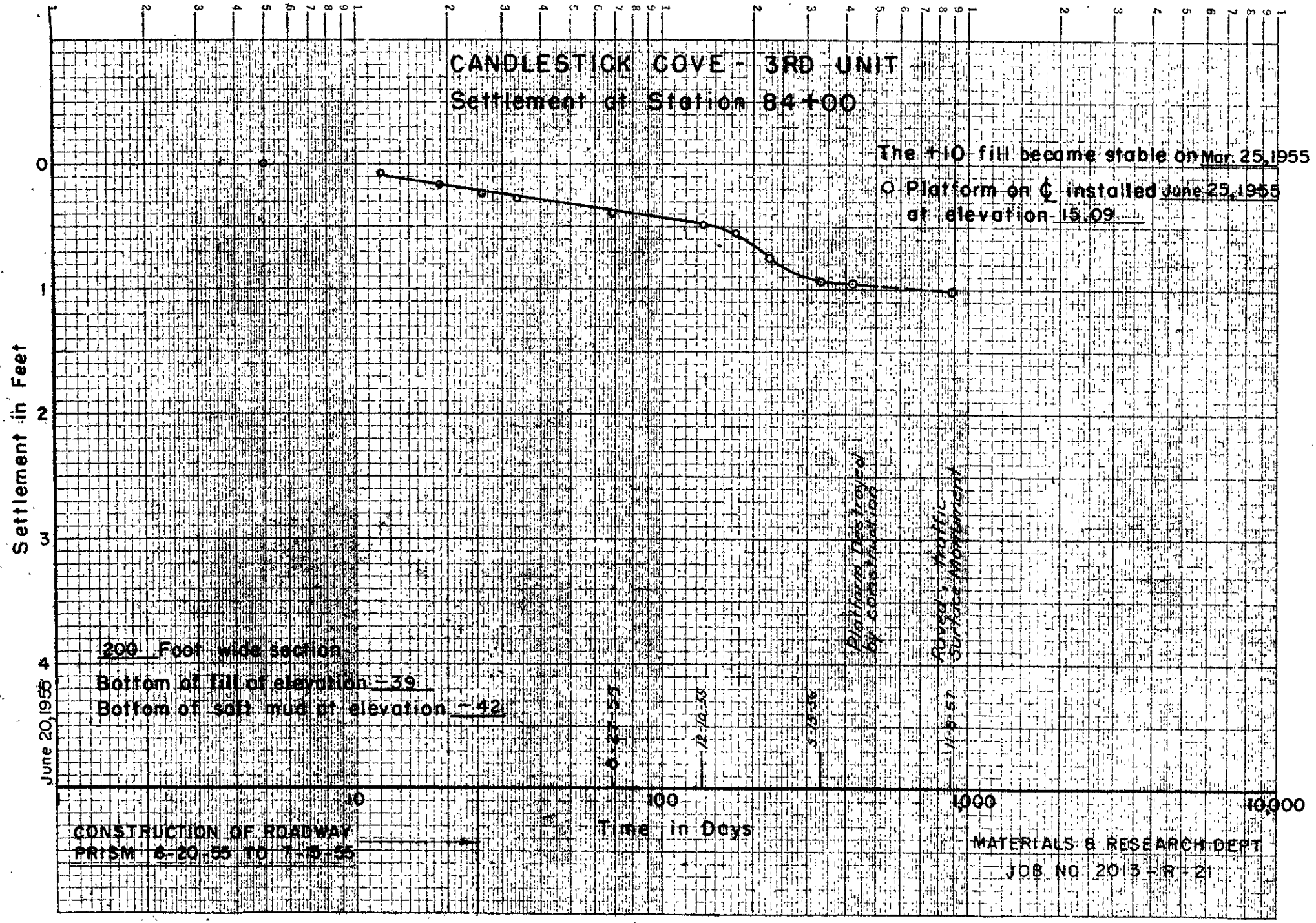
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



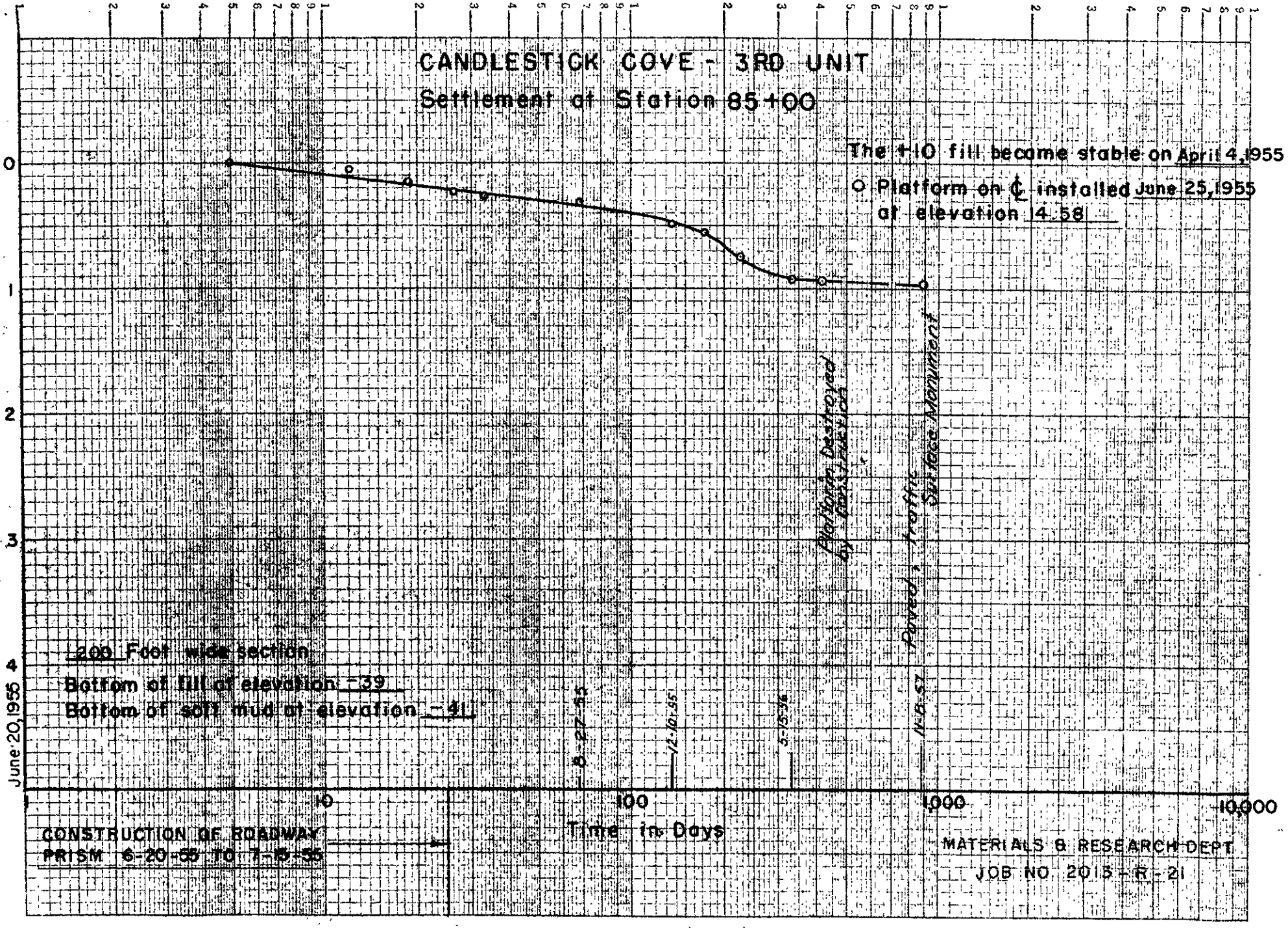
K+E SEMI LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



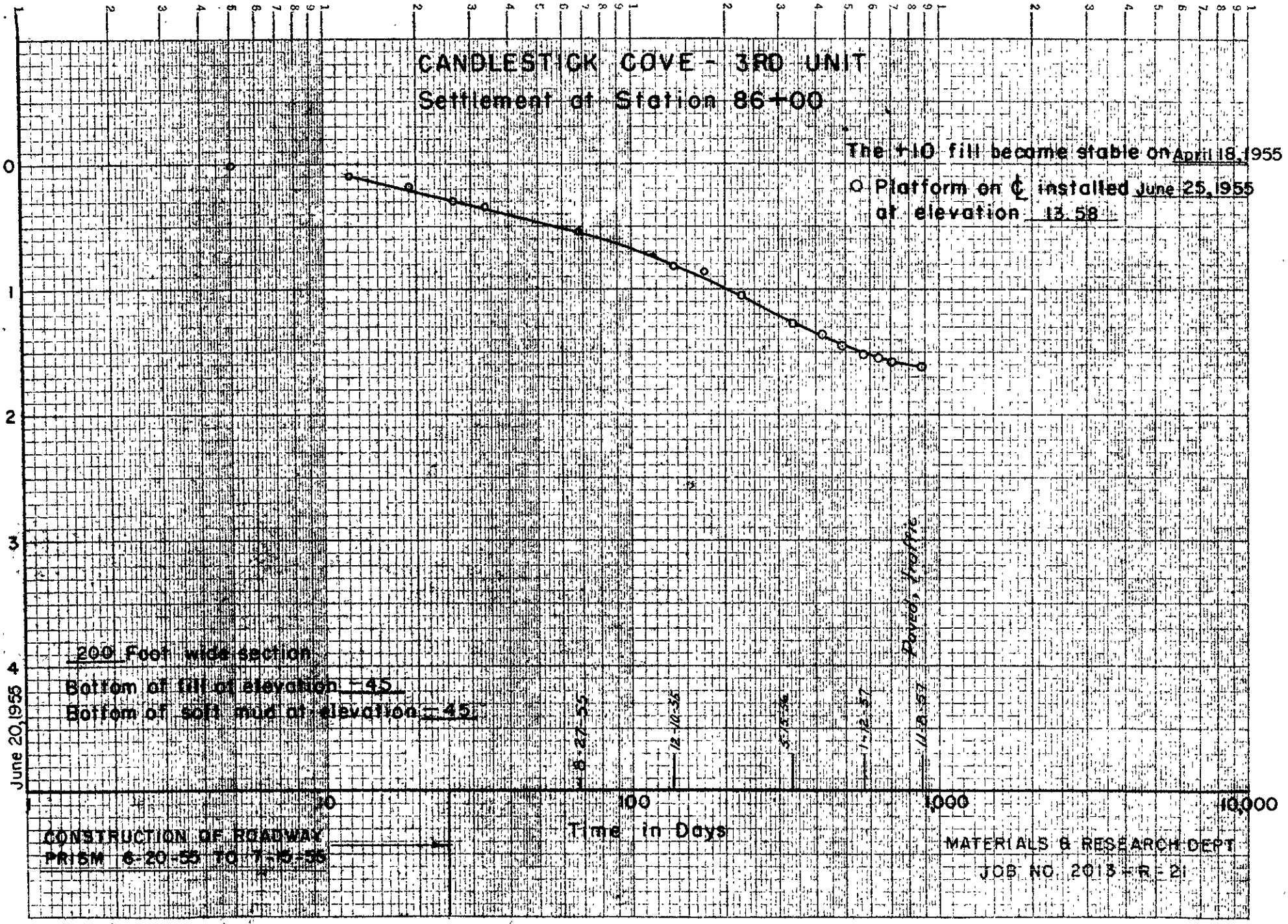
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

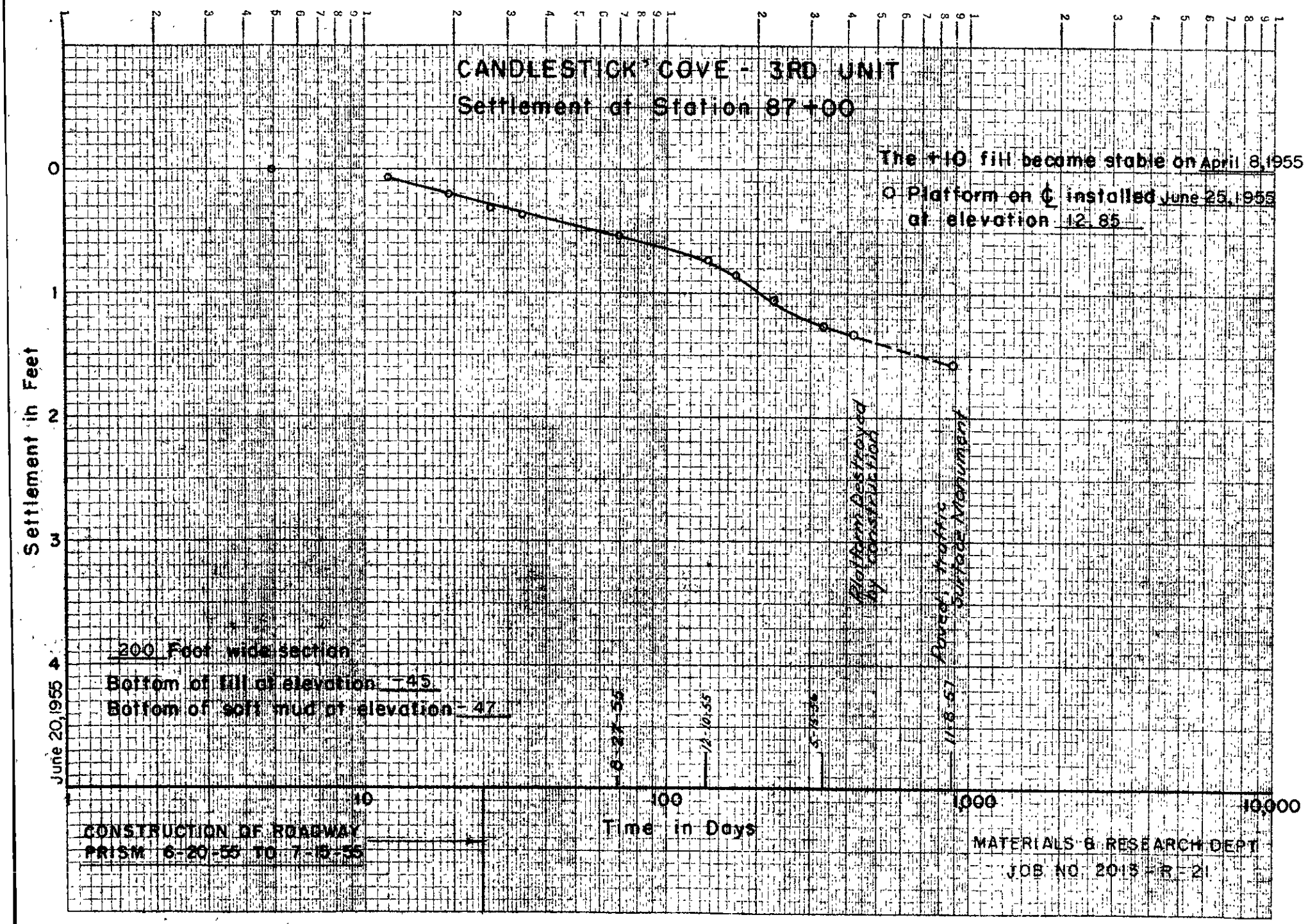


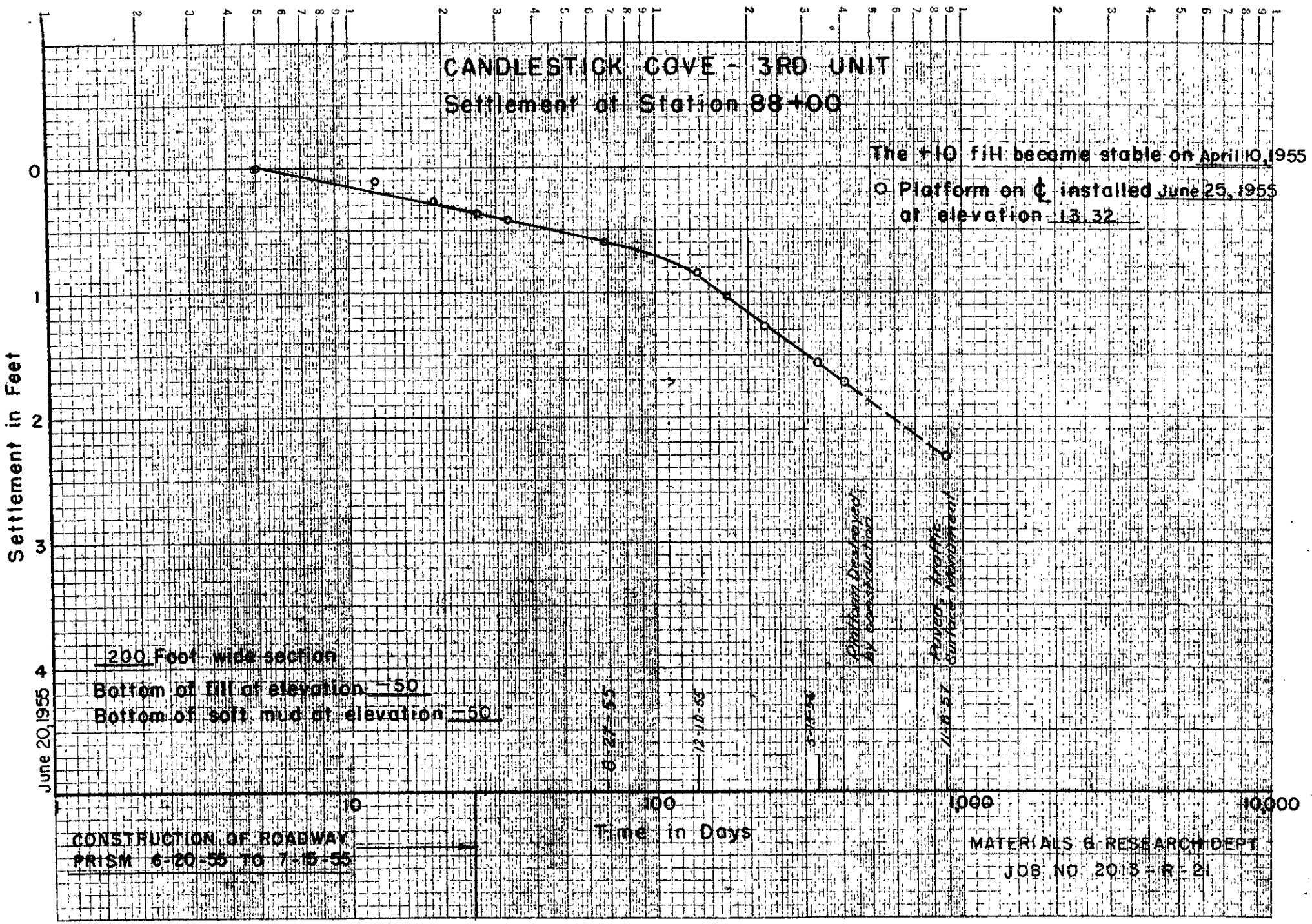
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



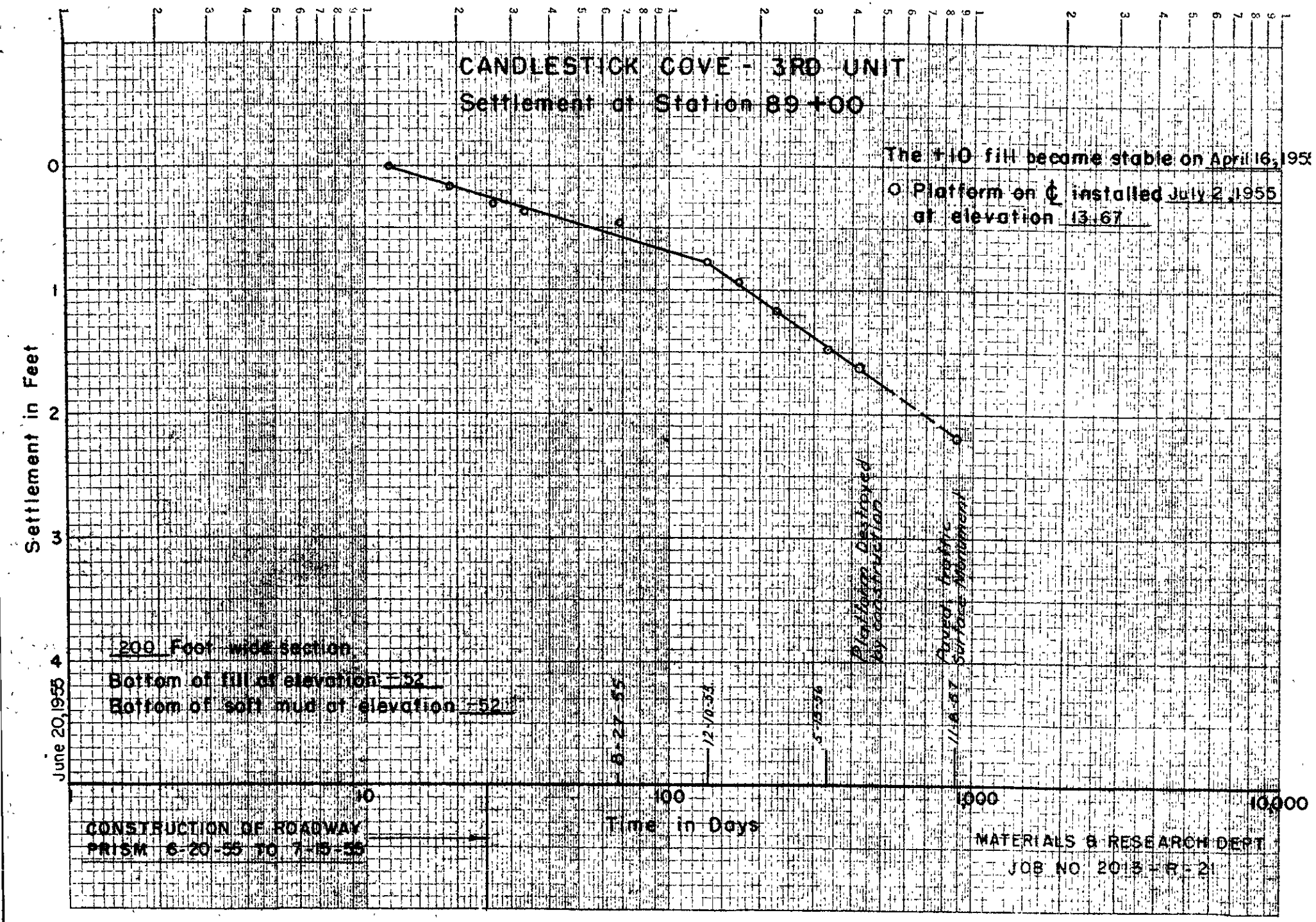
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



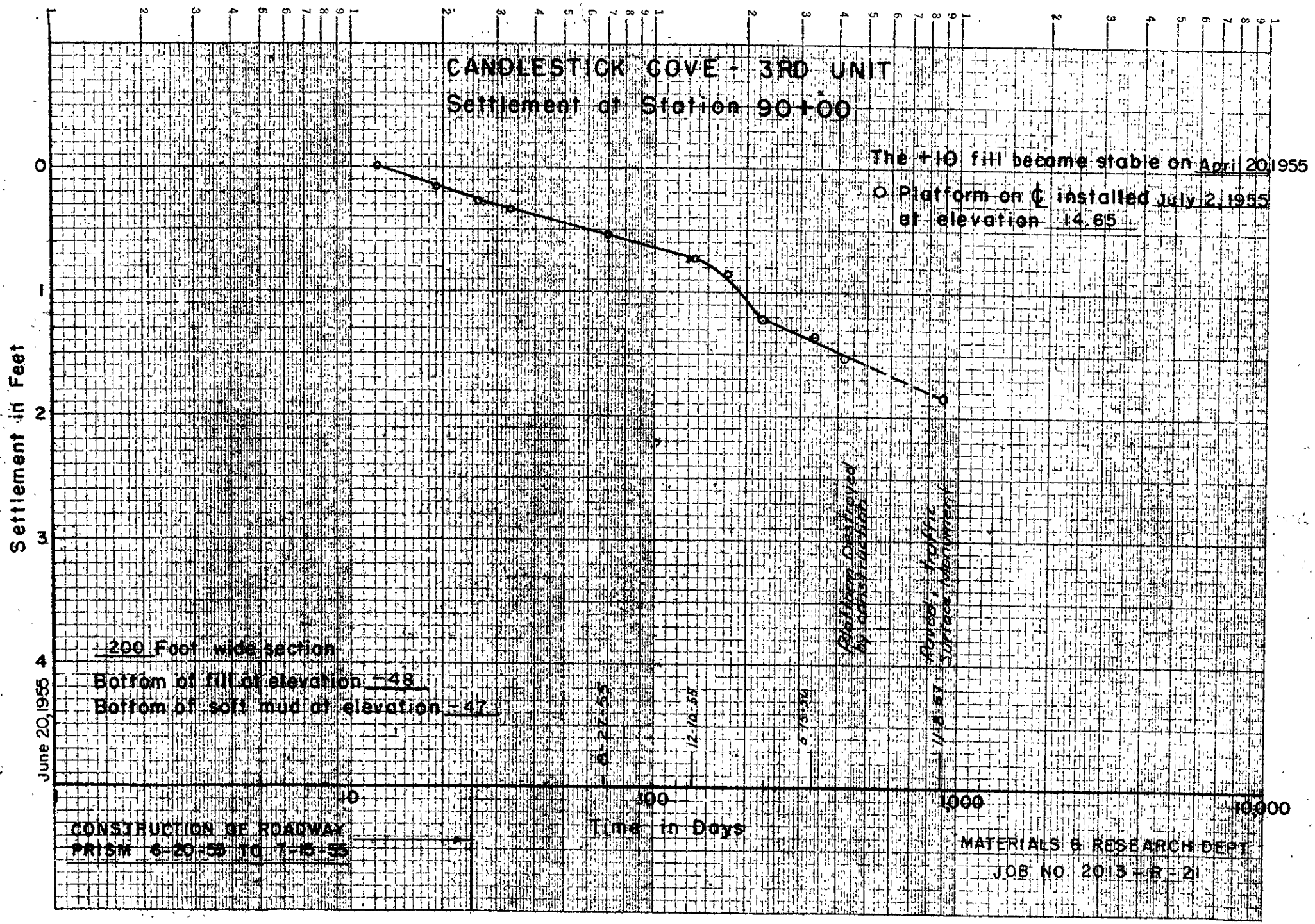

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 87+00
The +10 fill became stable on April 8, 1955
Platform on ℄ installed June 25, 1955
at elevation 12.85
Platform Destroyed by Construction
Posed Traffic Surface Monument
200 Foot wide section
Bottom of fill at elevation -45
Bottom of soft mud at elevation -47
8-27-55
12-10-55
5-15-56
11-8-57
Settlement in Feet
June 20, 1955
Time in Days
CONSTRUCTION OF ROADWAY PRISM 6-20-55 TO 7-15-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 88+00
The +10 fill became stable on April 10, 1955
Platform on ℄ installed June 25, 1955
at elevation 13.32
Settlement in Feet
0
1
2
3
4
June 20, 1955
200 Foot wide section
Bottom of fill of elevation -50
Bottom of soft mud at elevation -50
Platform Destroyed by construction
Paved Traffic Surface Nonuniform
6-27-55
12-10-55
3-15-56
11-8-56
1
10
100
1000
10,000
Time in Days
CONSTRUCTION OF ROADWAY PRISM 6-20-55 TO 7-15-55
MATERIALS & RESEARCH DEPT
JOB NO. 2013-R-21

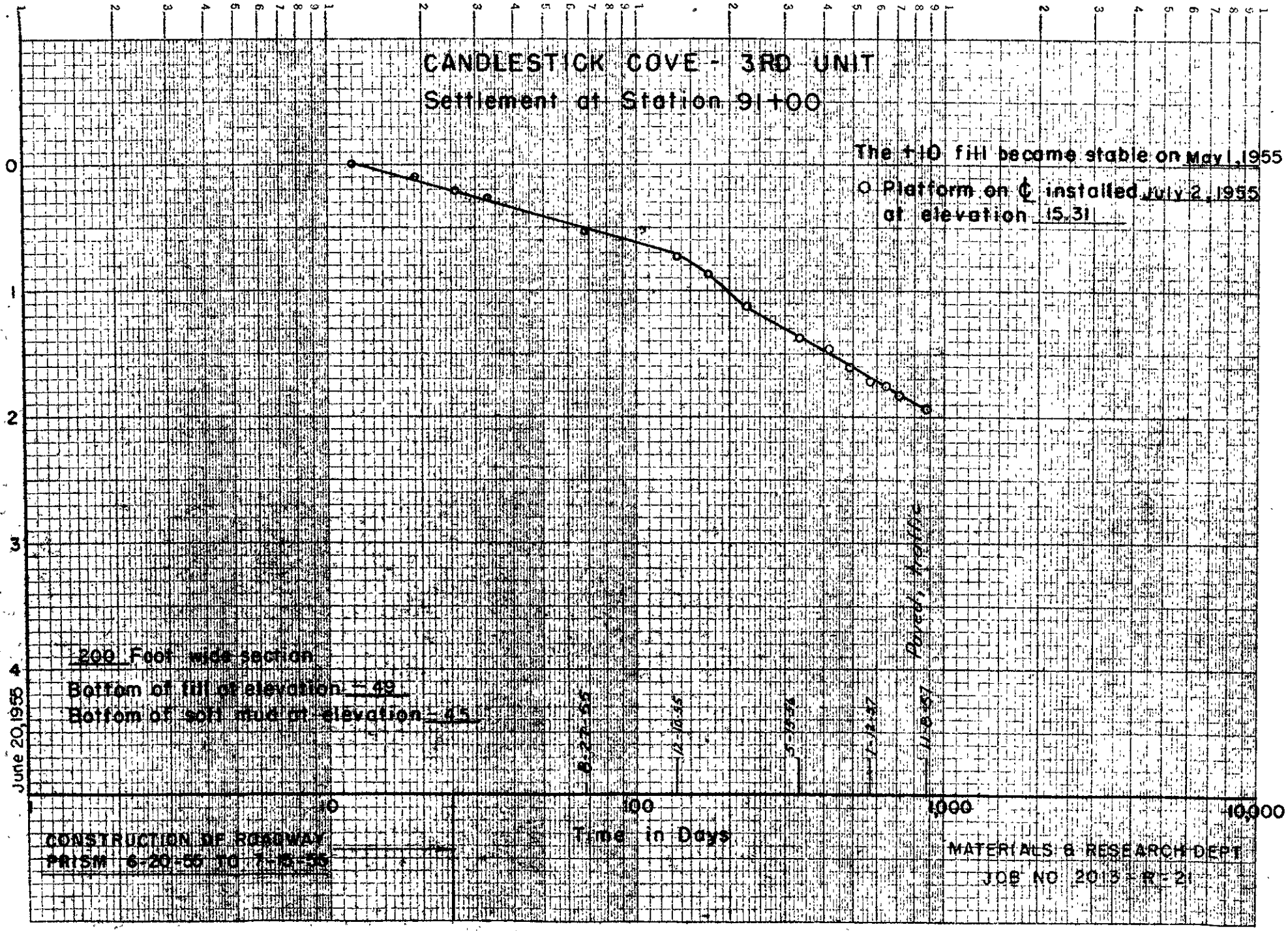
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



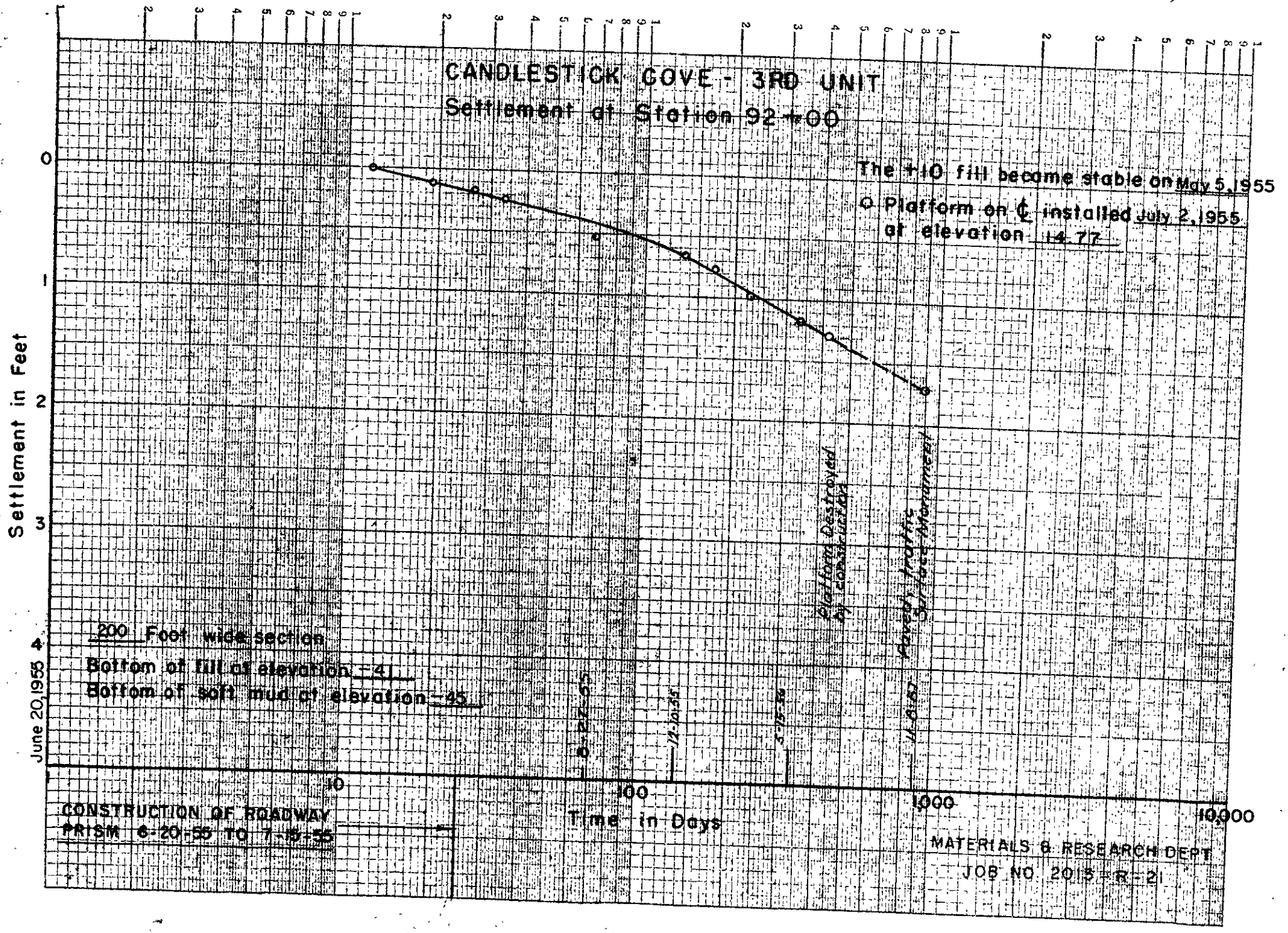
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



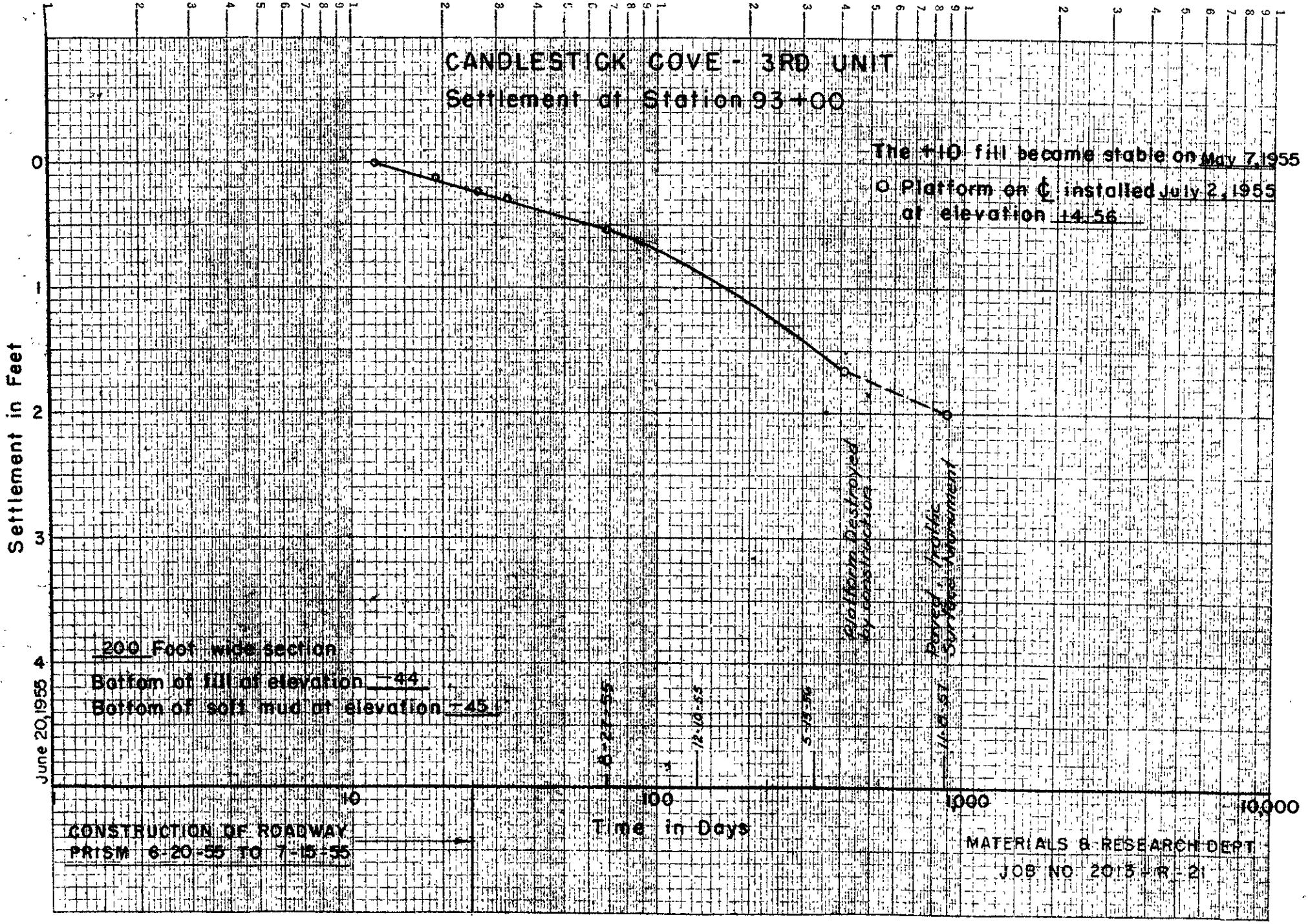
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

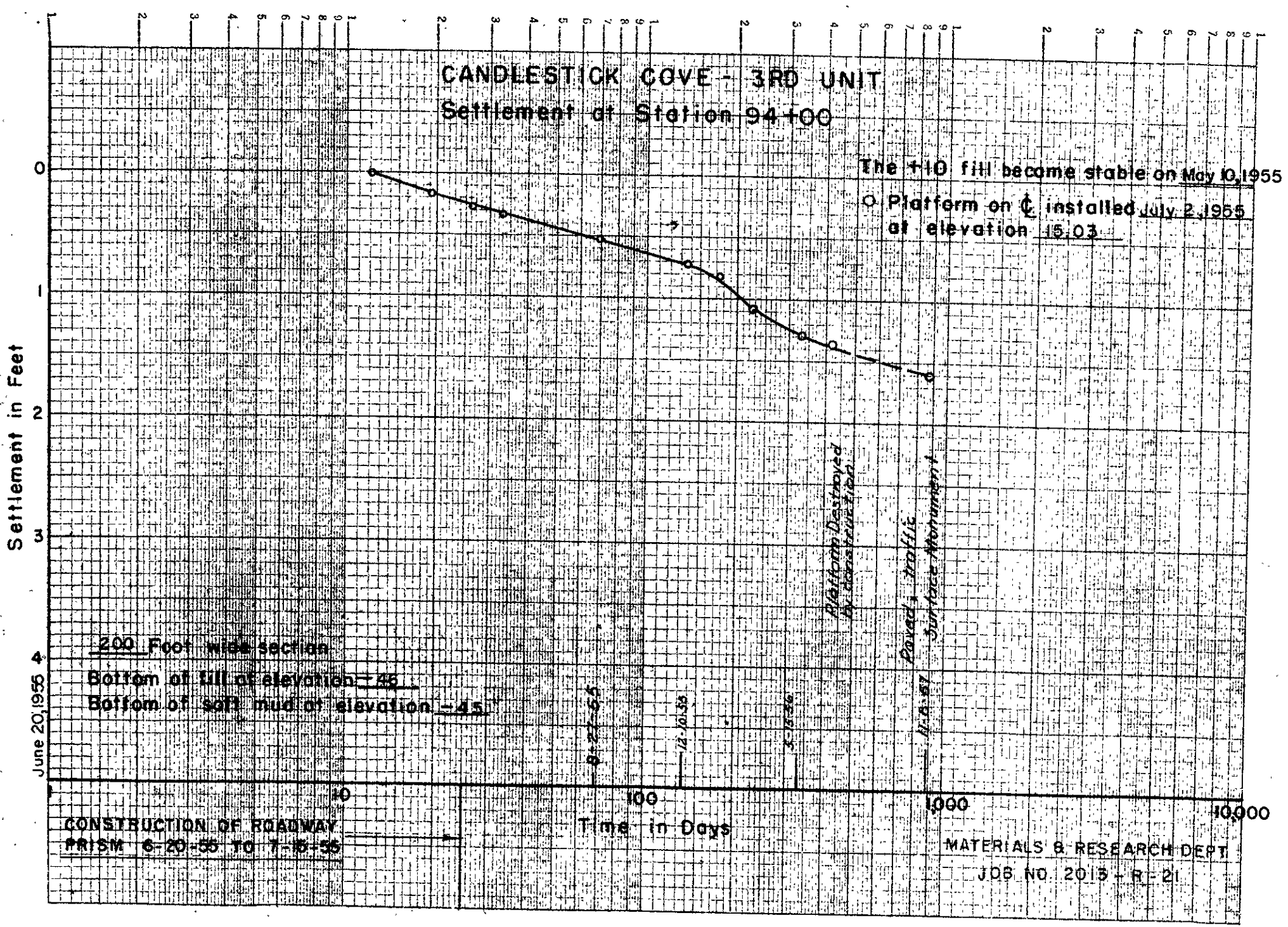


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

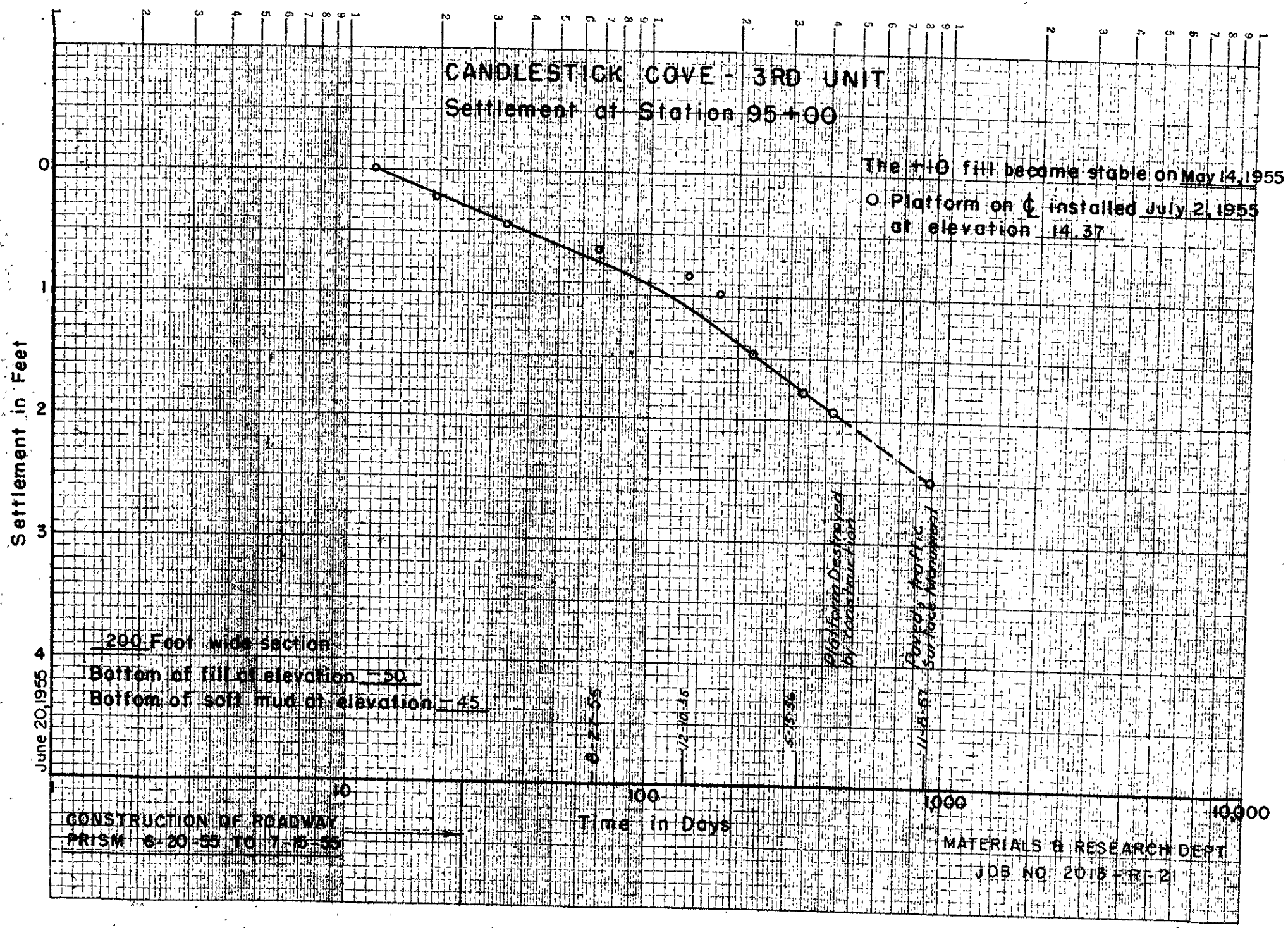


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

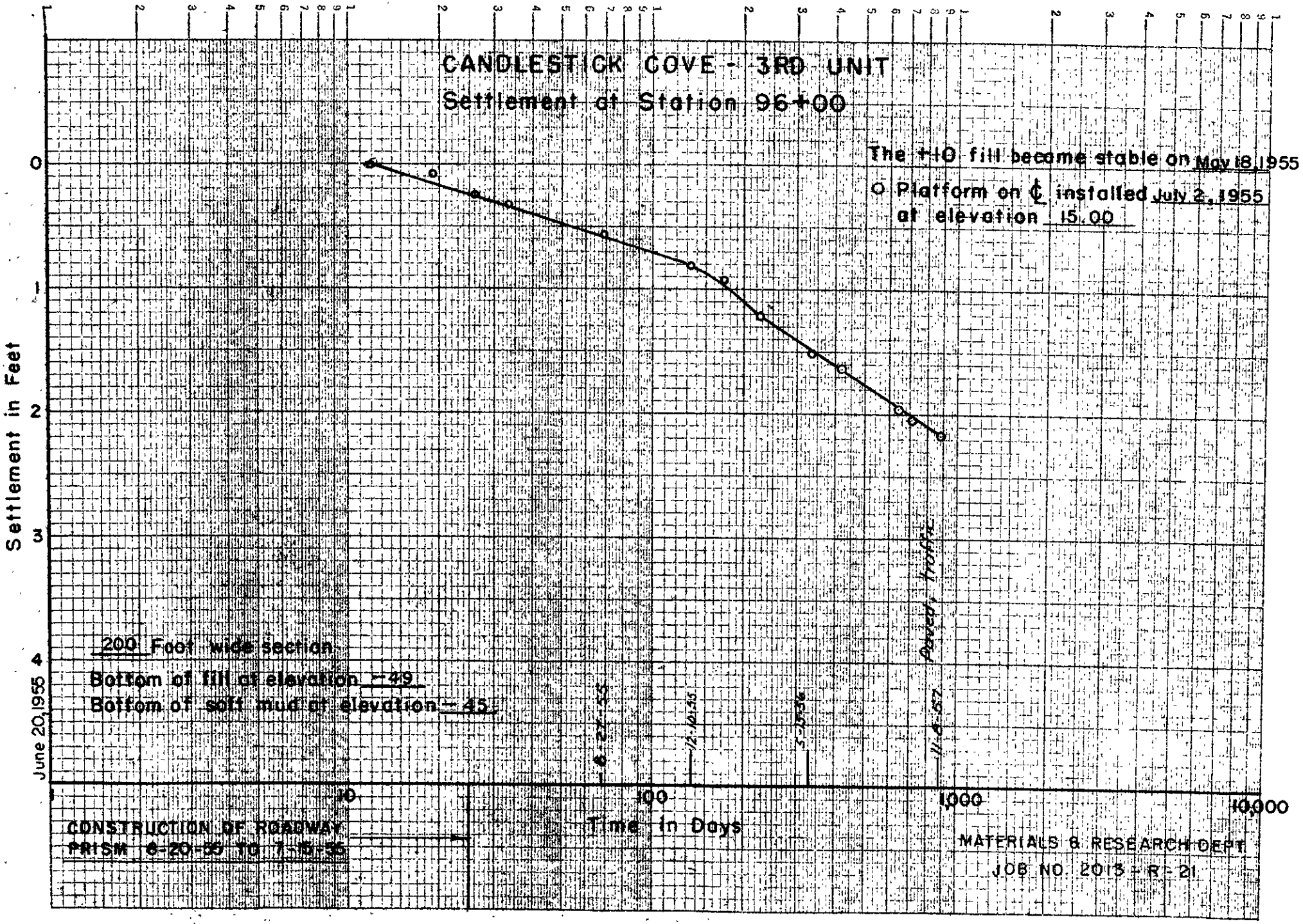


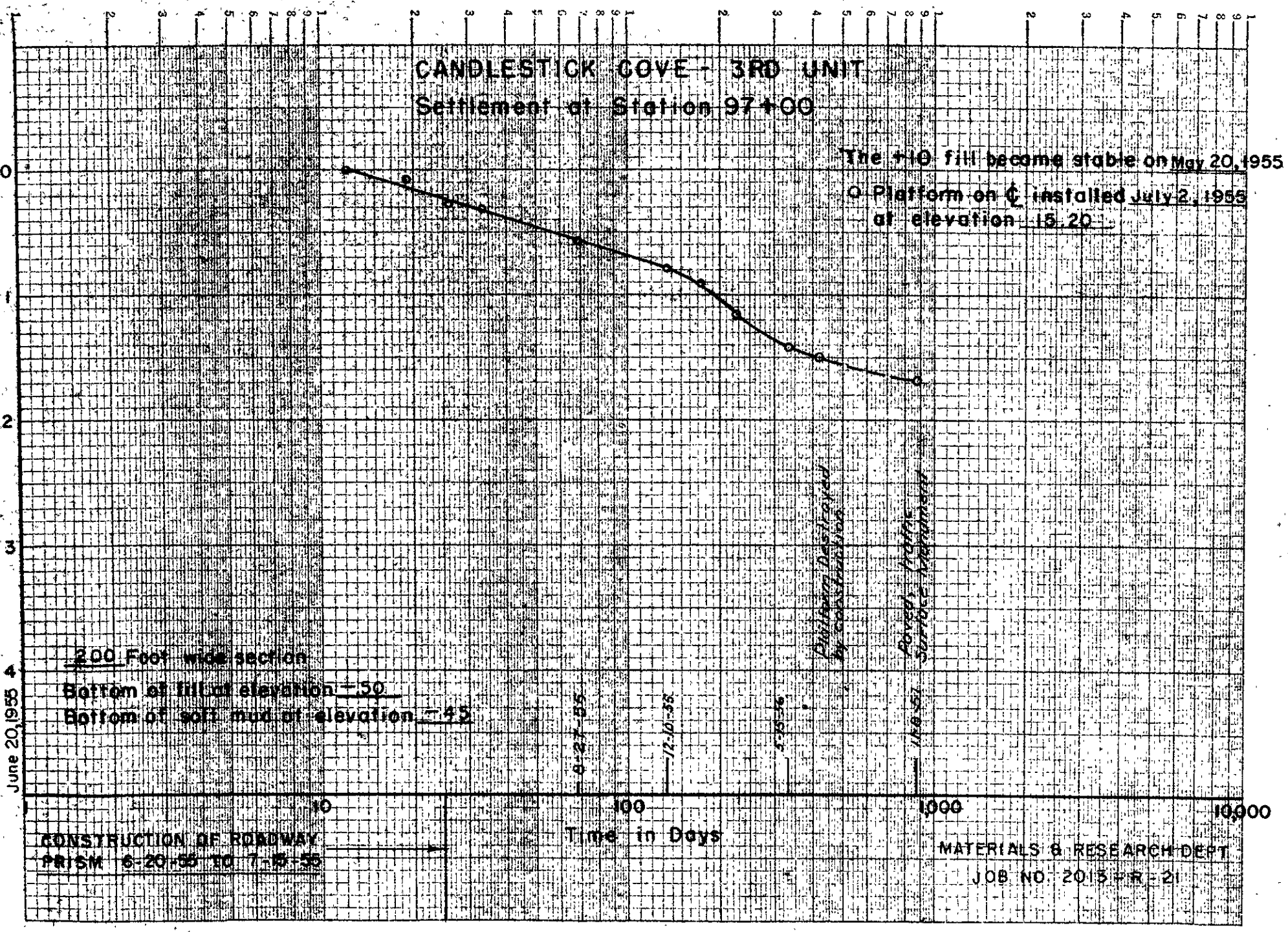

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 94+00
The +10 fill become stable on May 10, 1955
Platform on ℄ installed July 2, 1955
at elevation 15.03
Platform Destroyed By construction
Paved, traffic
Surface Monument
200 Foot wide section
Bottom of fill at elevation -46
Bottom of soft mud at elevation -45
June 20, 1955
9-27-55
12-10-55
3-15-56
11-8-57
Settlement in Feet
0
1
2
3
4
10
100
1000
10,000
Time in Days
CONSTRUCTION OF ROADWAY PRISM 6-20-55 TO 7-15-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

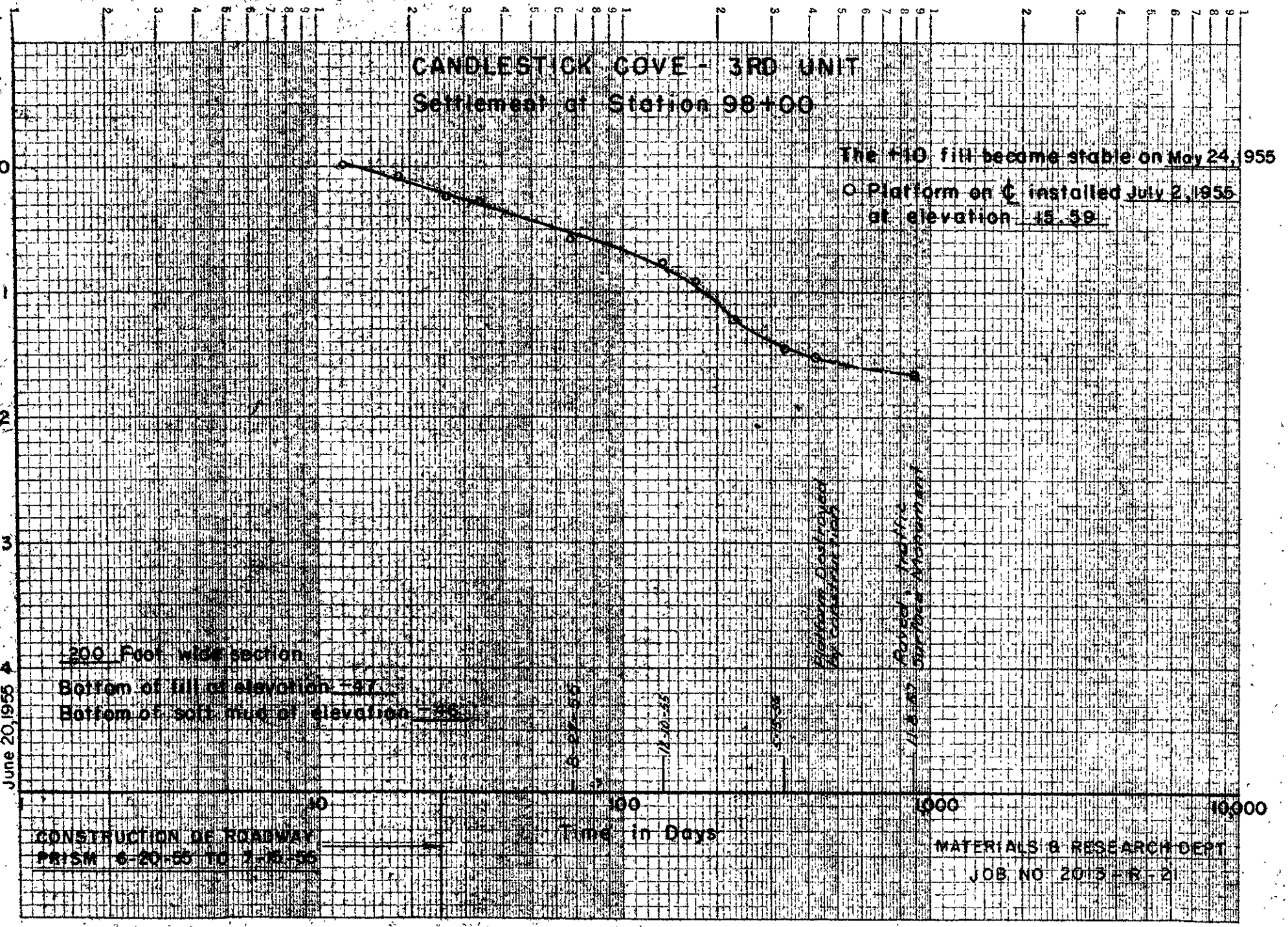
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

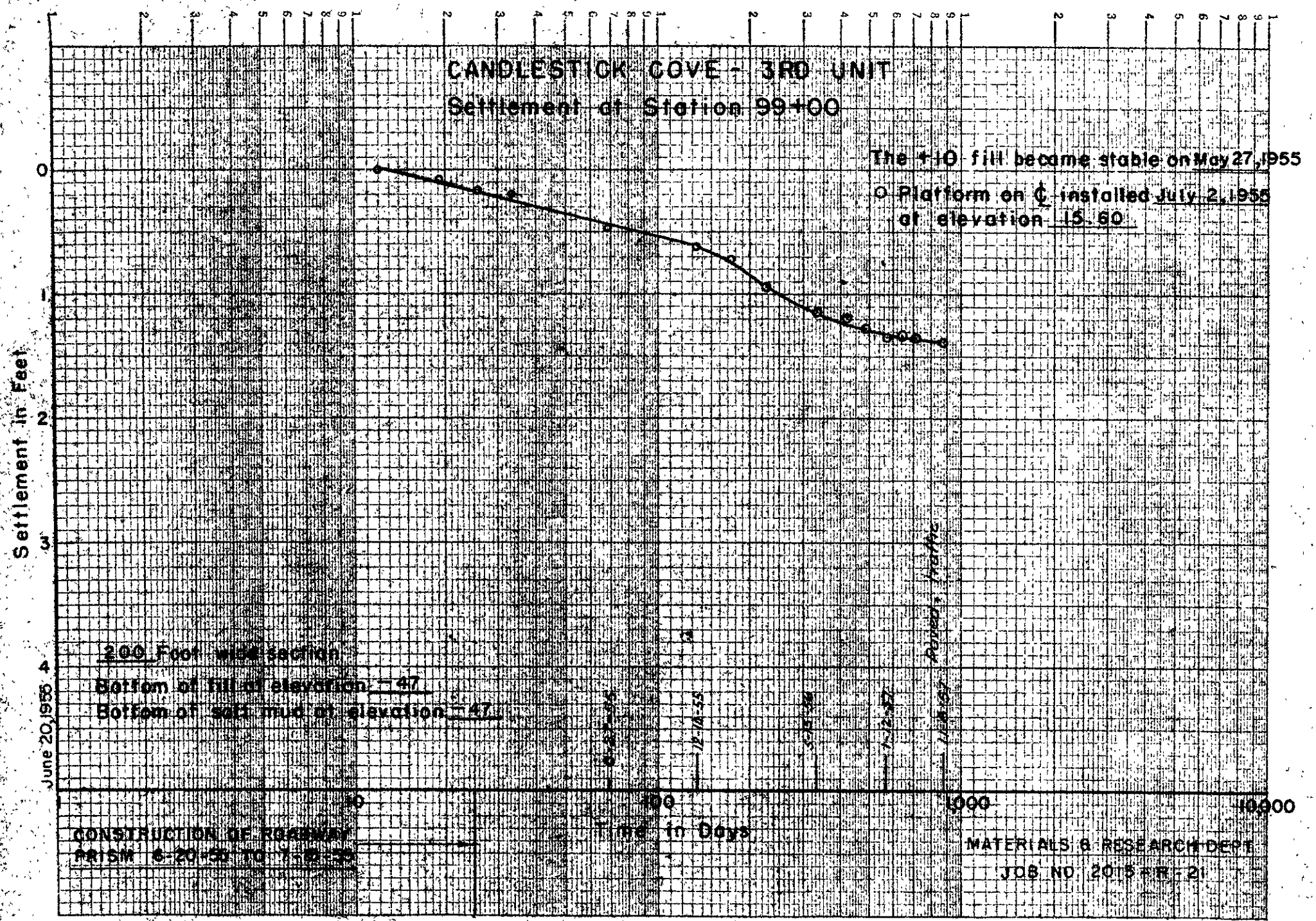


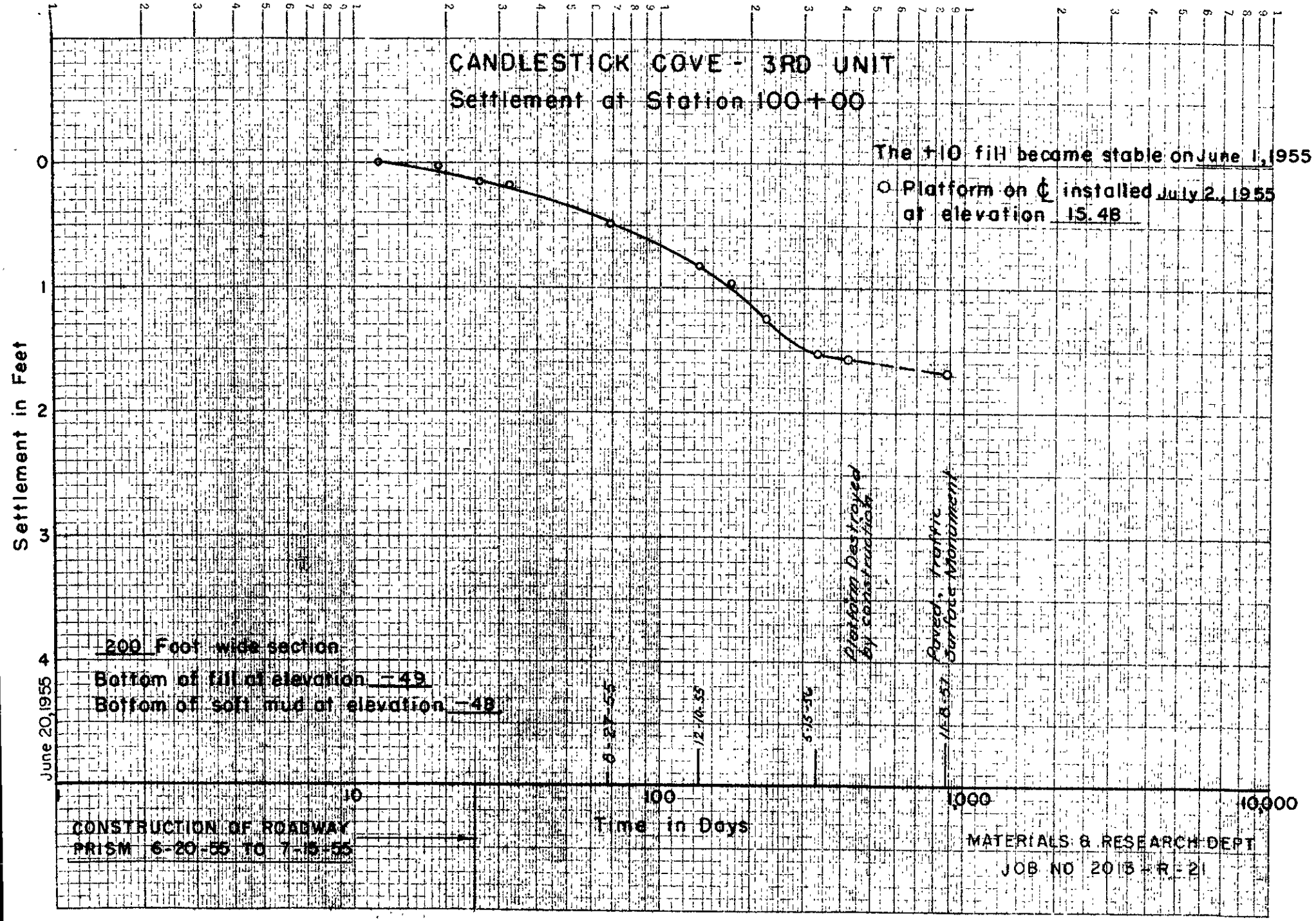
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

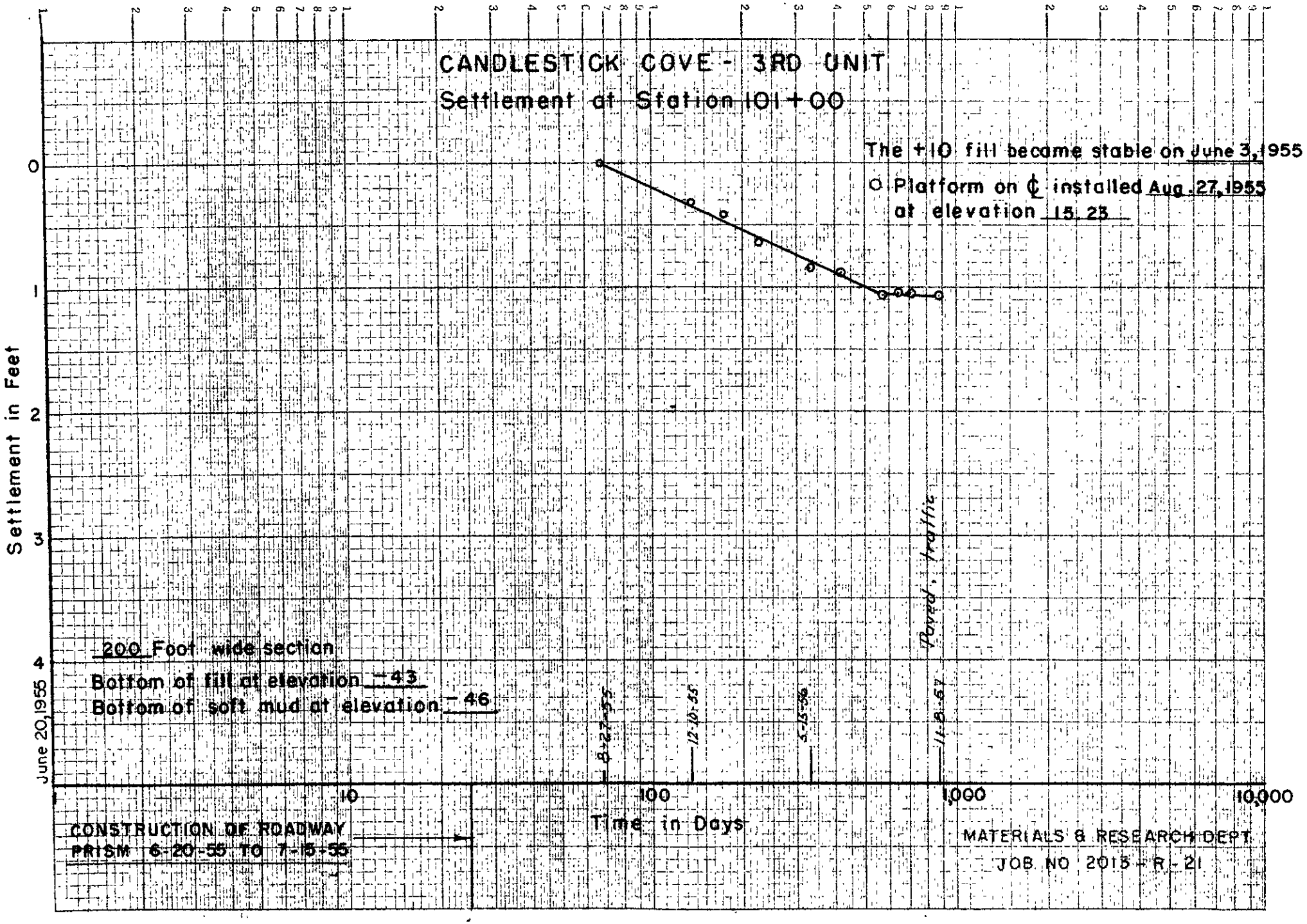



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 97+00
The +10 fill became stable on May 20, 1955
O Platform on ℄ installed July 2, 1955
at elevation 15.20
Settlement in Feet
0
1
2
3
4
June 20, 1955
200 Foot wide section
Bottom of fill at elevation -.50
Bottom of soft mud at elevation -4.5
8-27-55
12-6-55
5-5-56
Pavement Destroyed by construction
Paved Traffic Surface Mainland
1-8-57
1
10
100
1000
10,000
Time in Days
CONSTRUCTION OF ROADWAY PRISM 6-20-55 TO 7-15-55
MATERIALS & RESEARCH DEPT
JOB NO. 2015-R-21

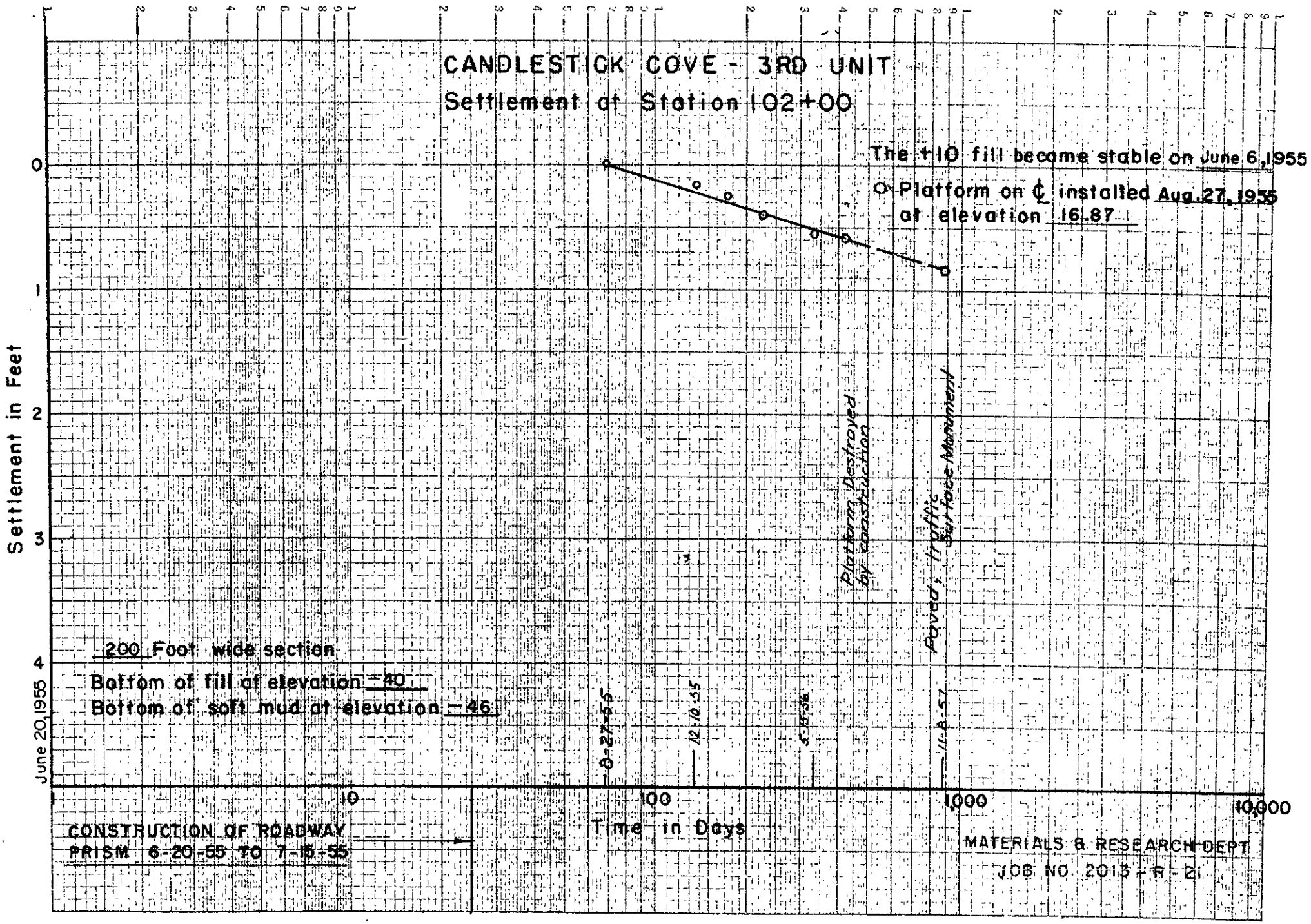

K+E SEMI-LOGARITHMIC 359-31
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 98+00
The +10 fill become stable on May 24, 1955
Platform on ℄ installed July 2, 1955
at elevation 15.59
Settlement in Feet
0
1
2
3
4
June 20, 1955
200 Foot wide section
Bottom of fill at elevation -37
Bottom of soft mud at elevation -55
Platform Destroyed by collapsed fills
Paved Traffic Surface Movement
10
100
1000
10,000
Time in Days
CONSTRUCTION OF ROADWAY PRISM 6-20-55 TO 7-15-55
MATERIALS & RESEARCH DEPT
JOB NO 20-3-R-21


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 99+00
The +10 fill became stable on May 27, 1955
O Platform on ℄ installed July 2, 1955
at elevation 15.60
Settlement in Feet
0
1
2
3
4
June 20, 1955
200 Foot wide section
Bottom of fill at elevation -47
Bottom of soft mud at elevation -47
12-14-55
Heavy traffic
1
10
100
1000
10,000
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 6-20-55 TO 7-22-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2015-R-21

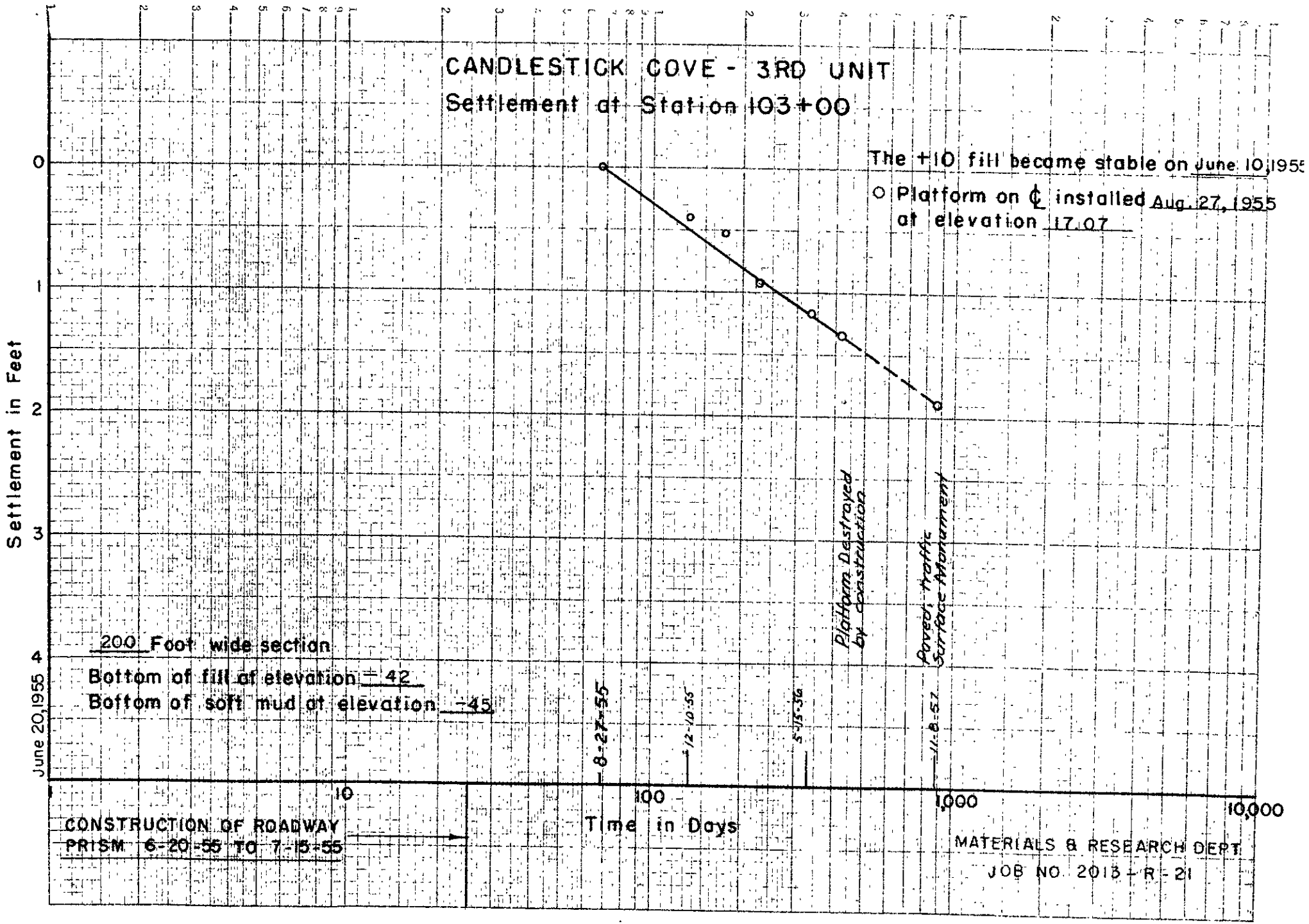

K+E SEMI LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 100+00
The +10 fill became stable on June 1, 1955
Platform on ℄ installed July 2, 1955
at elevation 15.48
Settlement in Feet
0
1
2
3
4
June 20, 1955
200 Foot wide section
Bottom of fill at elevation -49
Bottom of soft mud at elevation -49
Platform Destroyed By construction
Paved, Traffic
Surface Monument
8-27-55
12-16-55
5-15-56
10
100
1000
10,000
Time in Days
CONSTRUCTION OF ROADWAY PRISM 6-20-55 TO 7-15-55
MATERIALS & RESEARCH DEPT
JOB NO 2015-R-21


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 3RD UNIT
Settlement at Station 101+00
The +10 fill became stable on June 3, 1955
Platform on ℄ installed Aug. 27, 1955
at elevation 15.23
Settlement in Feet
0
1
2
3
4
200 Foot wide section
Bottom of fill at elevation -43
Bottom of soft mud at elevation -46
June 20, 1955
8-27-55
12-10-55
5-15-56
11-8-57
Paved, Traffic
1
10
100
1,000
10,000
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 6-20-55 TO 7-15-55
MATERIALS & RESEARCH DEPT.
JOB NO 2015-R-21

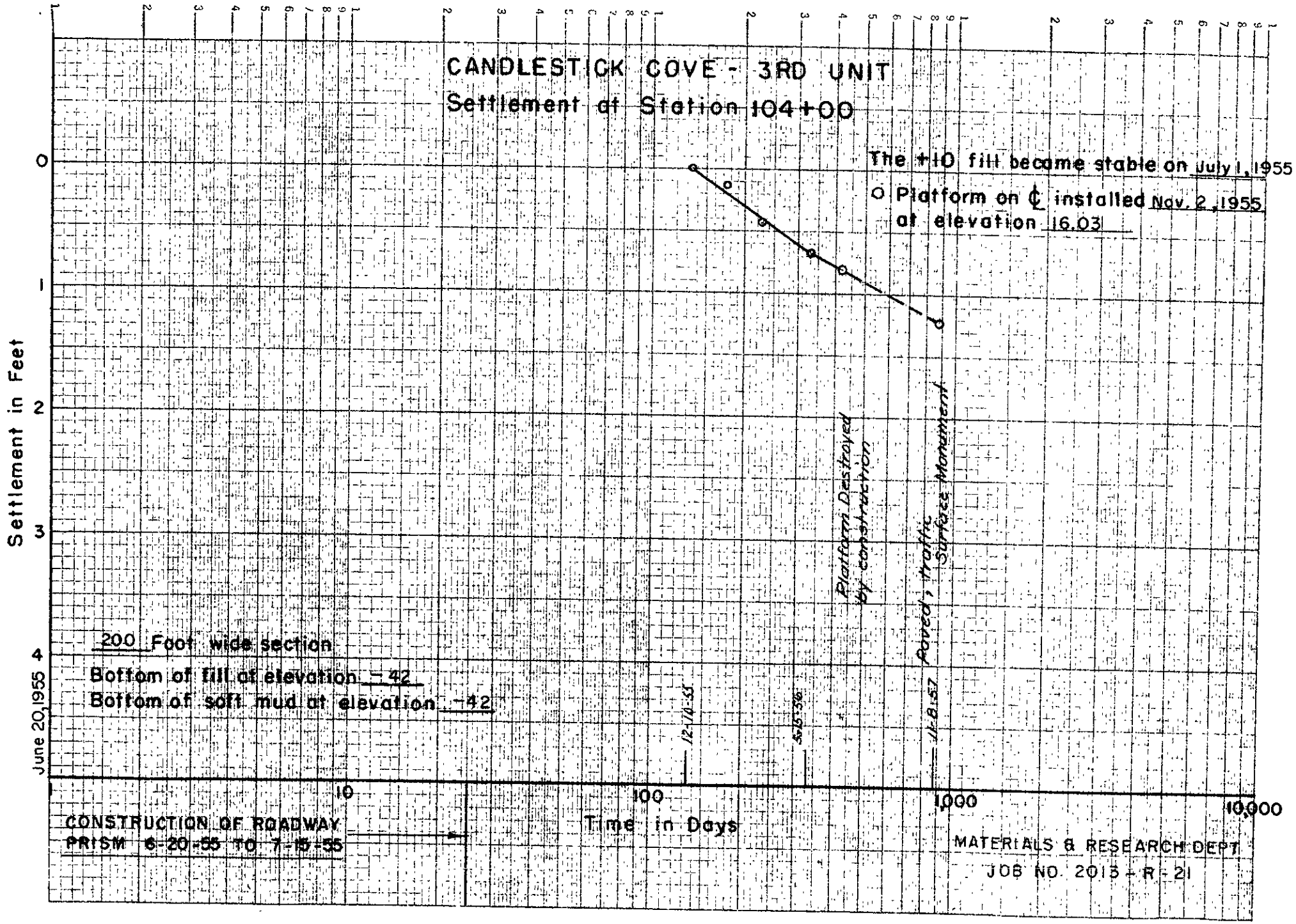
K+E SEMI-LOGARITHMIC 359-31
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



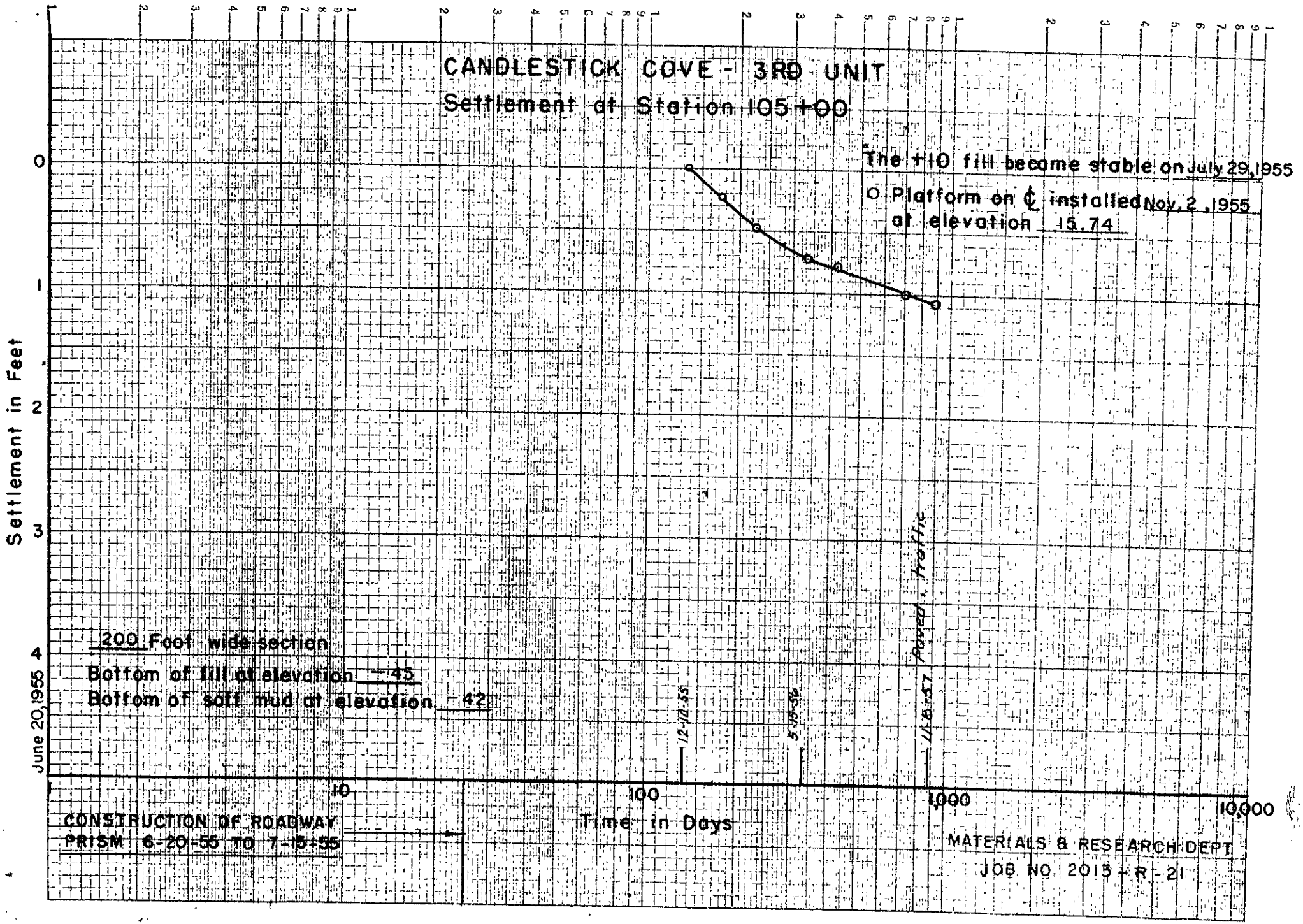
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



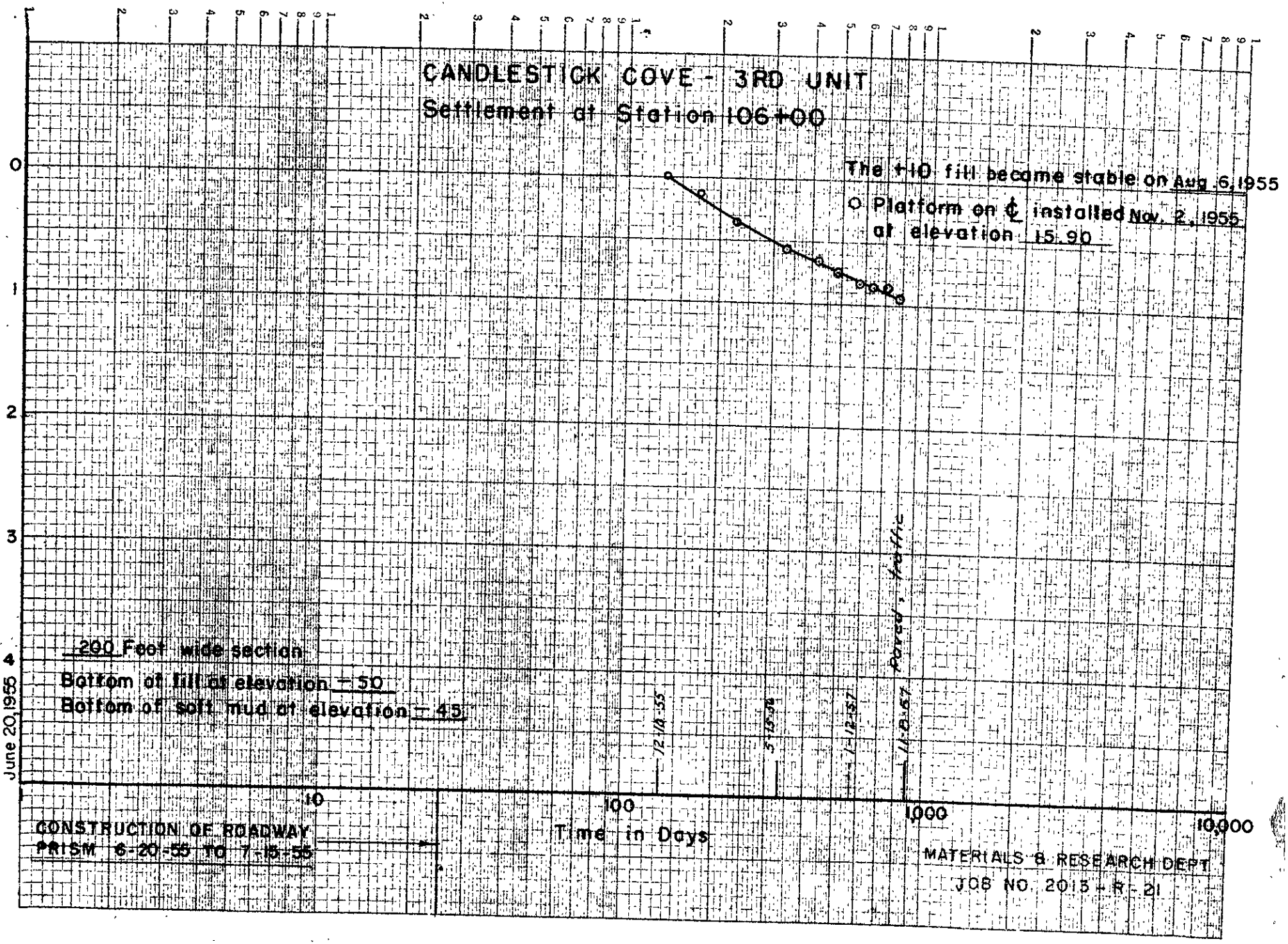
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

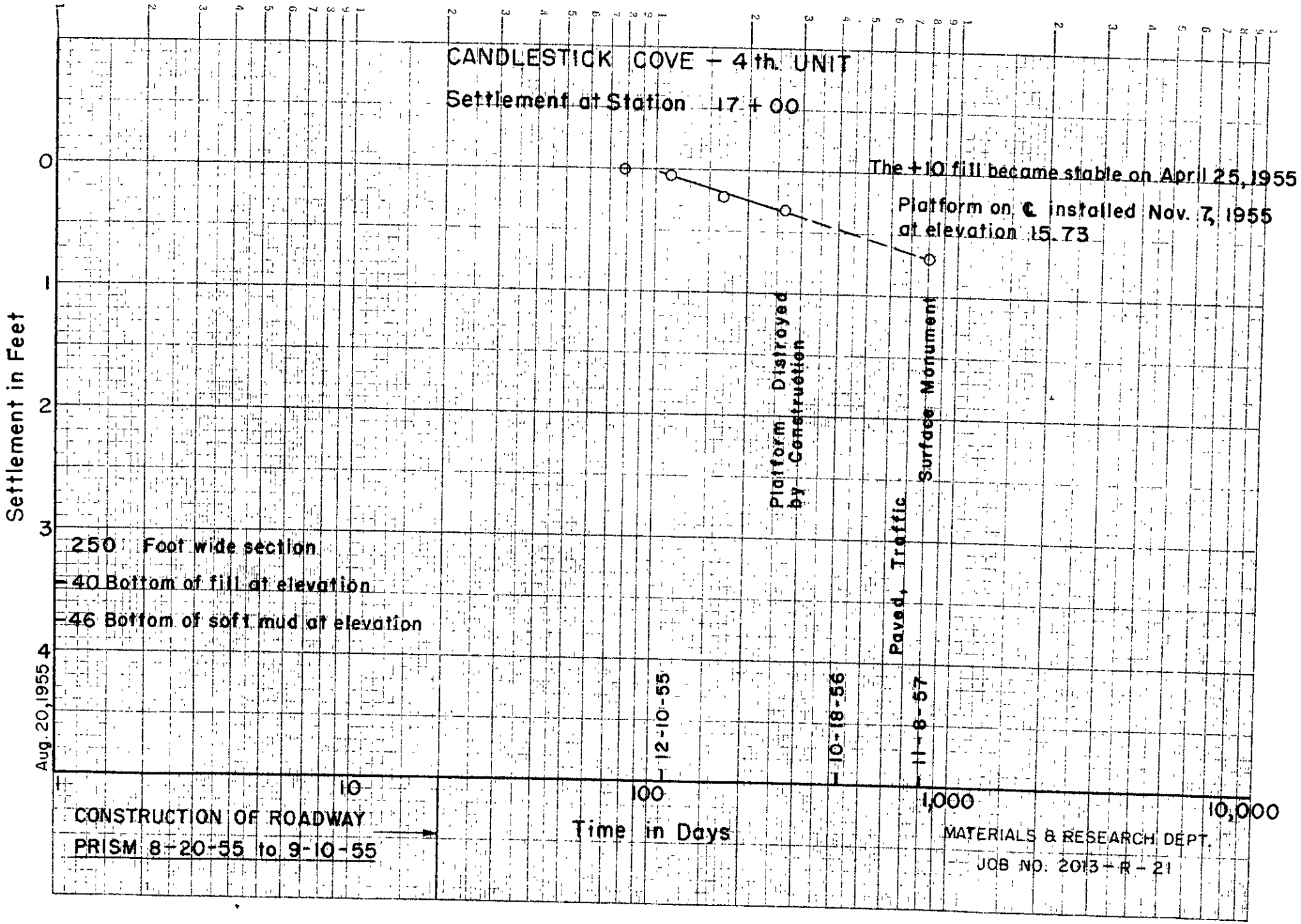


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

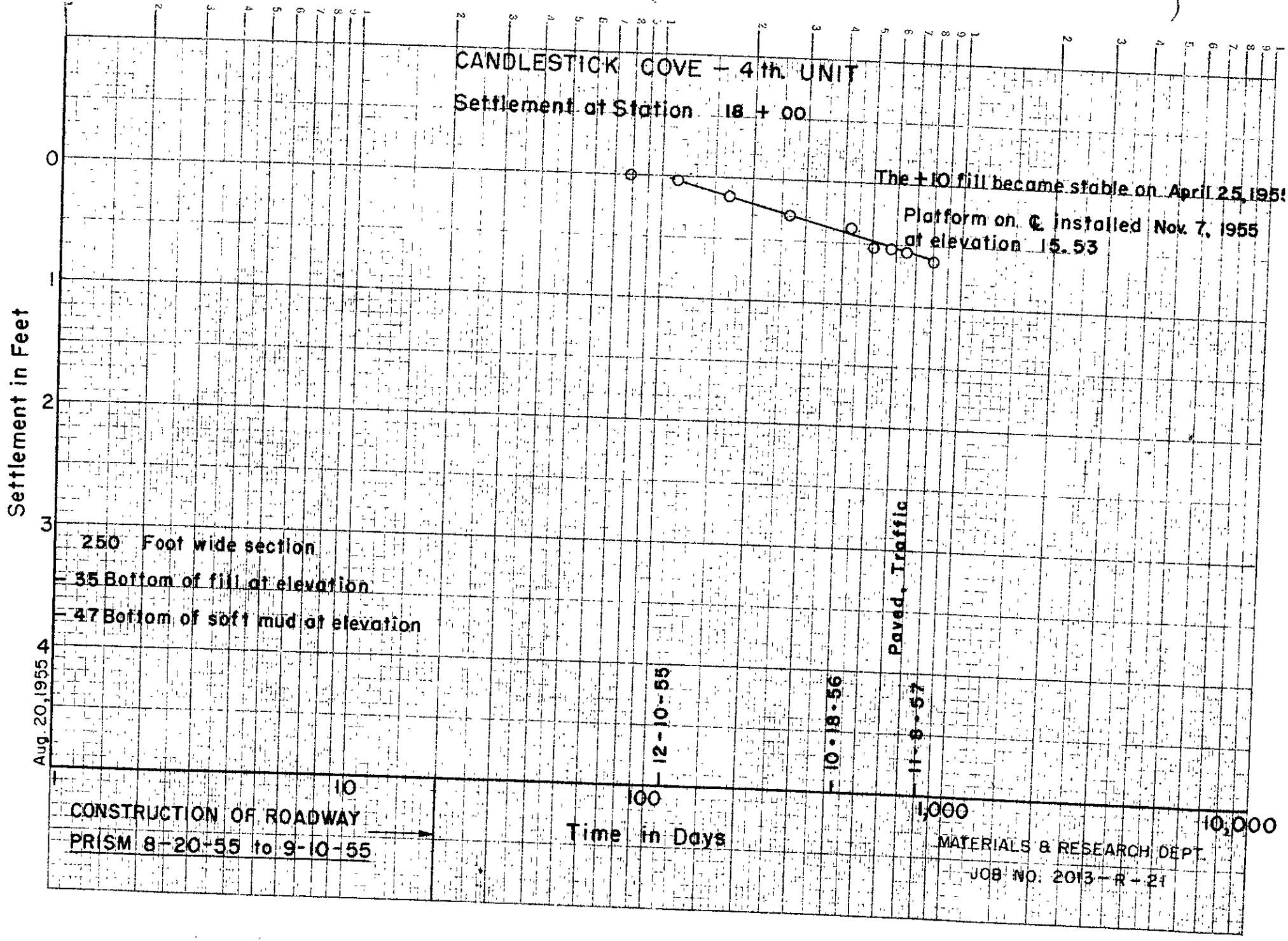


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

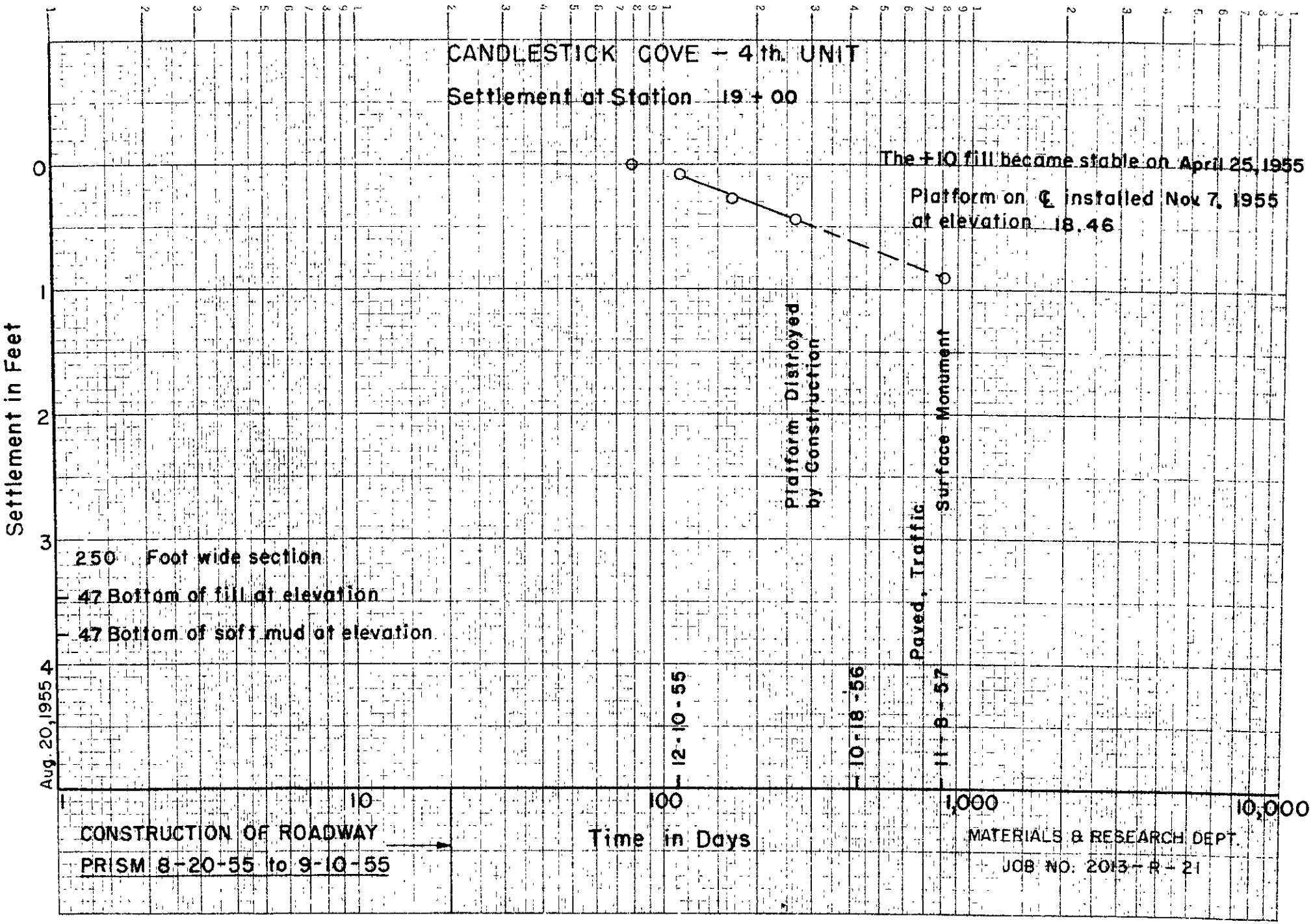



SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO.
CANDLESTICK COVE – 4th. UNIT
Settlement at Station 17 + 00
The +10 fill became stable on April 25, 1955
Platform on ℄ installed Nov. 7, 1955
at elevation 15.73
Platform Distroyed by Construction
Paved, Traffic Surface Monument
250 Foot wide section
-40 Bottom of fill at elevation
-46 Bottom of soft mud at elevation
Settlement in Feet
0
1
2
3
4
Aug. 20, 1955
12-10-55
10-18-56
11-8-57
1
10
100
1,000
10,000
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 8-20-55 to 9-10-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

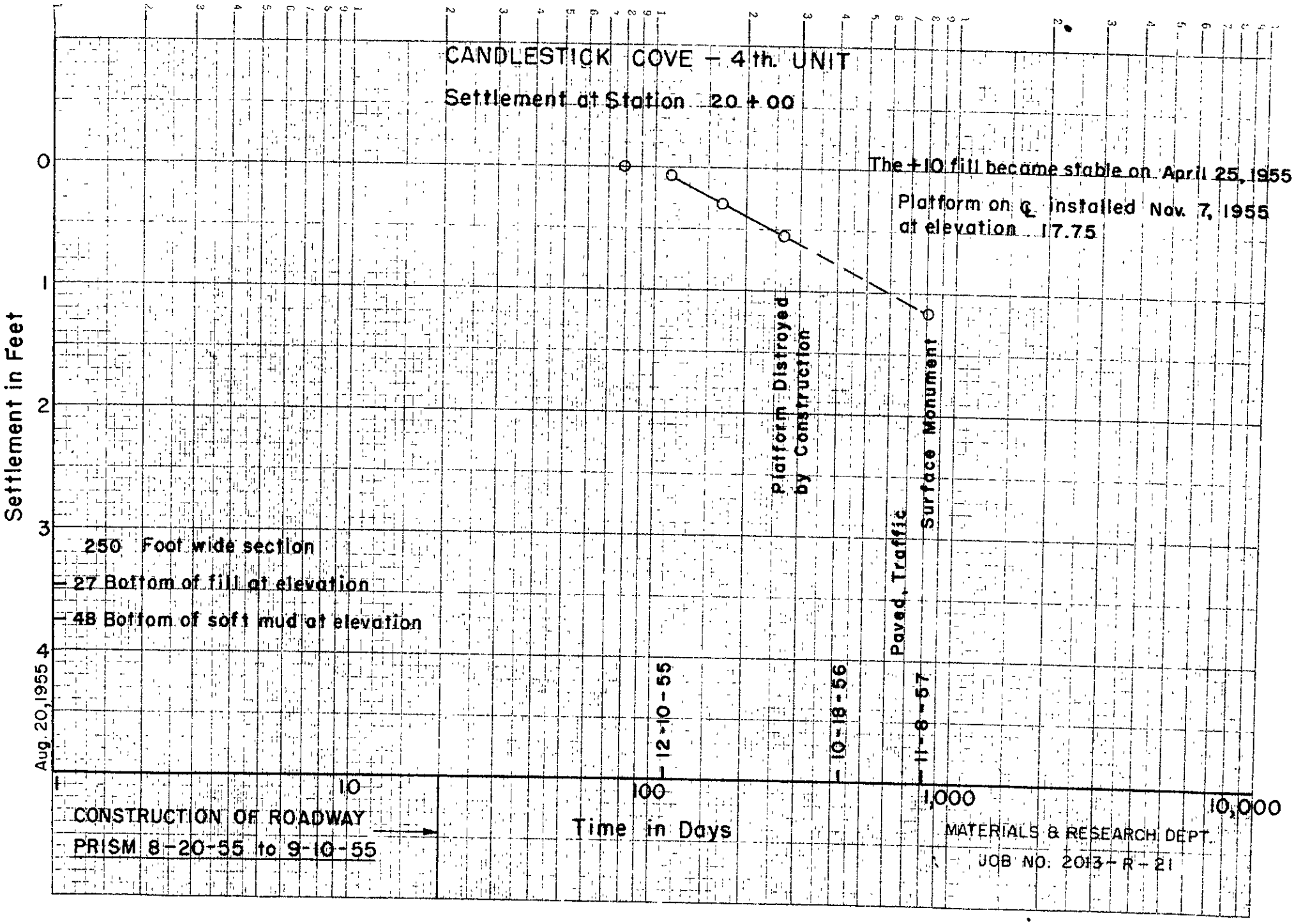
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
2 CYCLES X 70 DIVISIONS



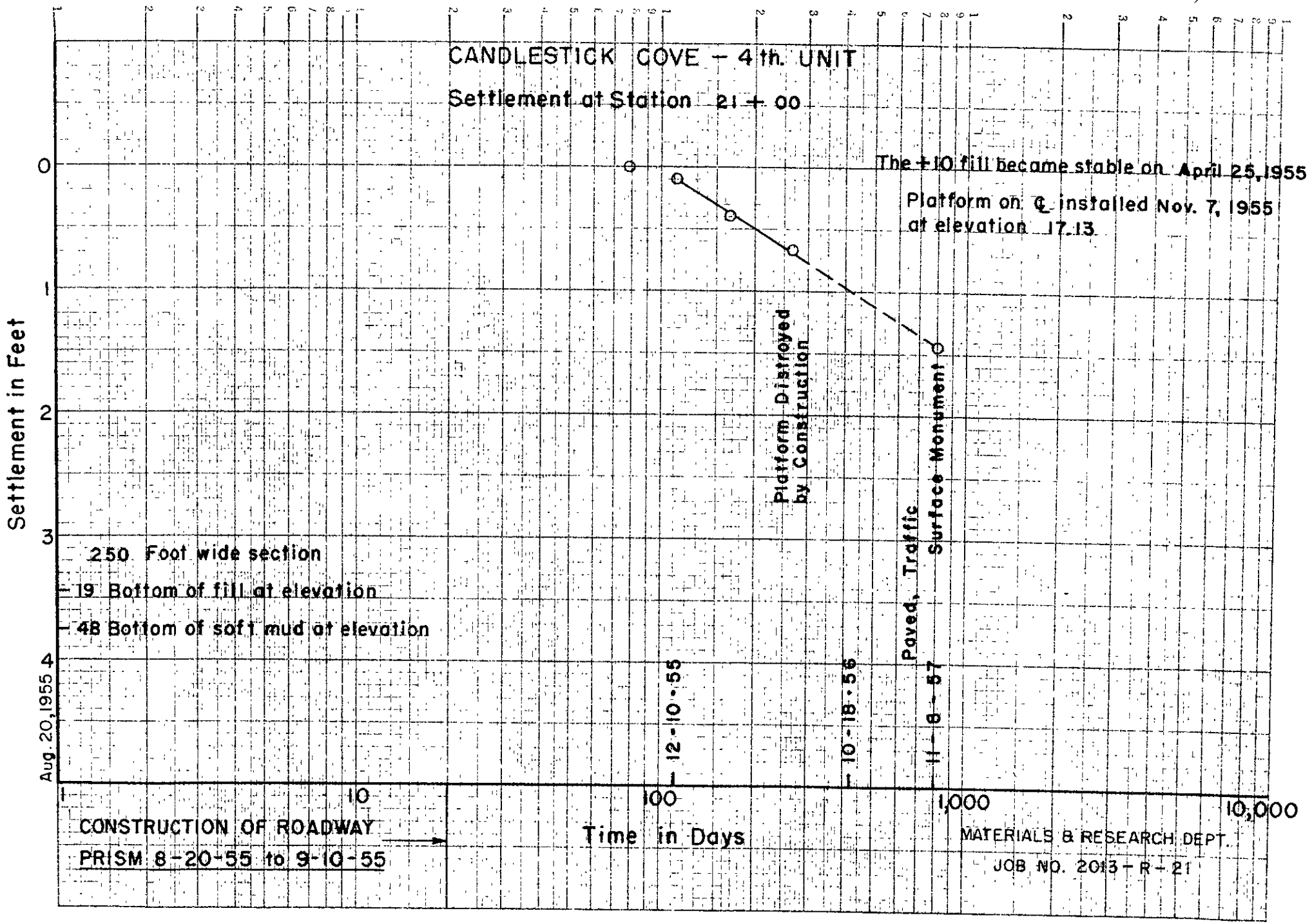
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

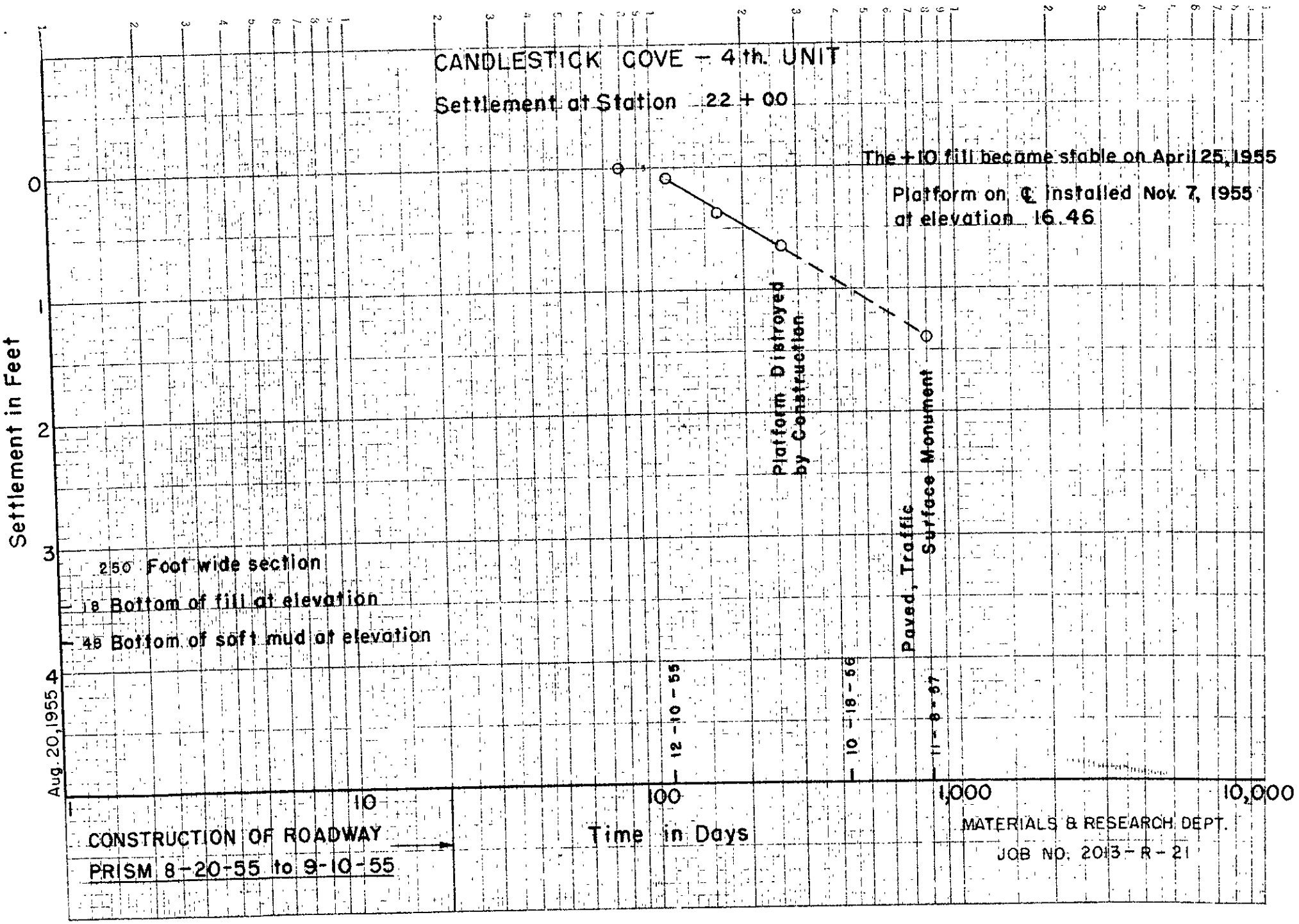


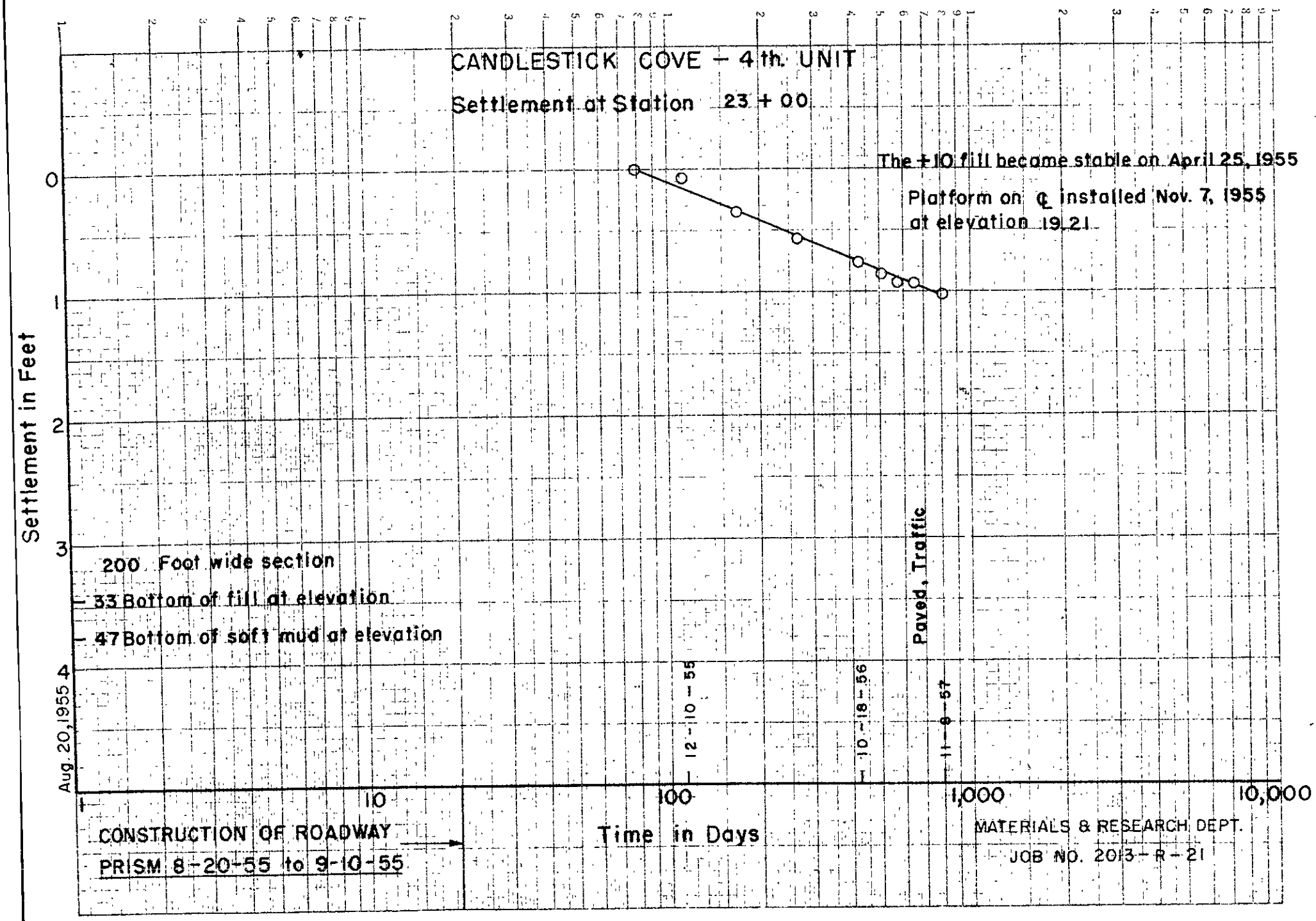
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
1 CYCLE X 70 DIVISIONS

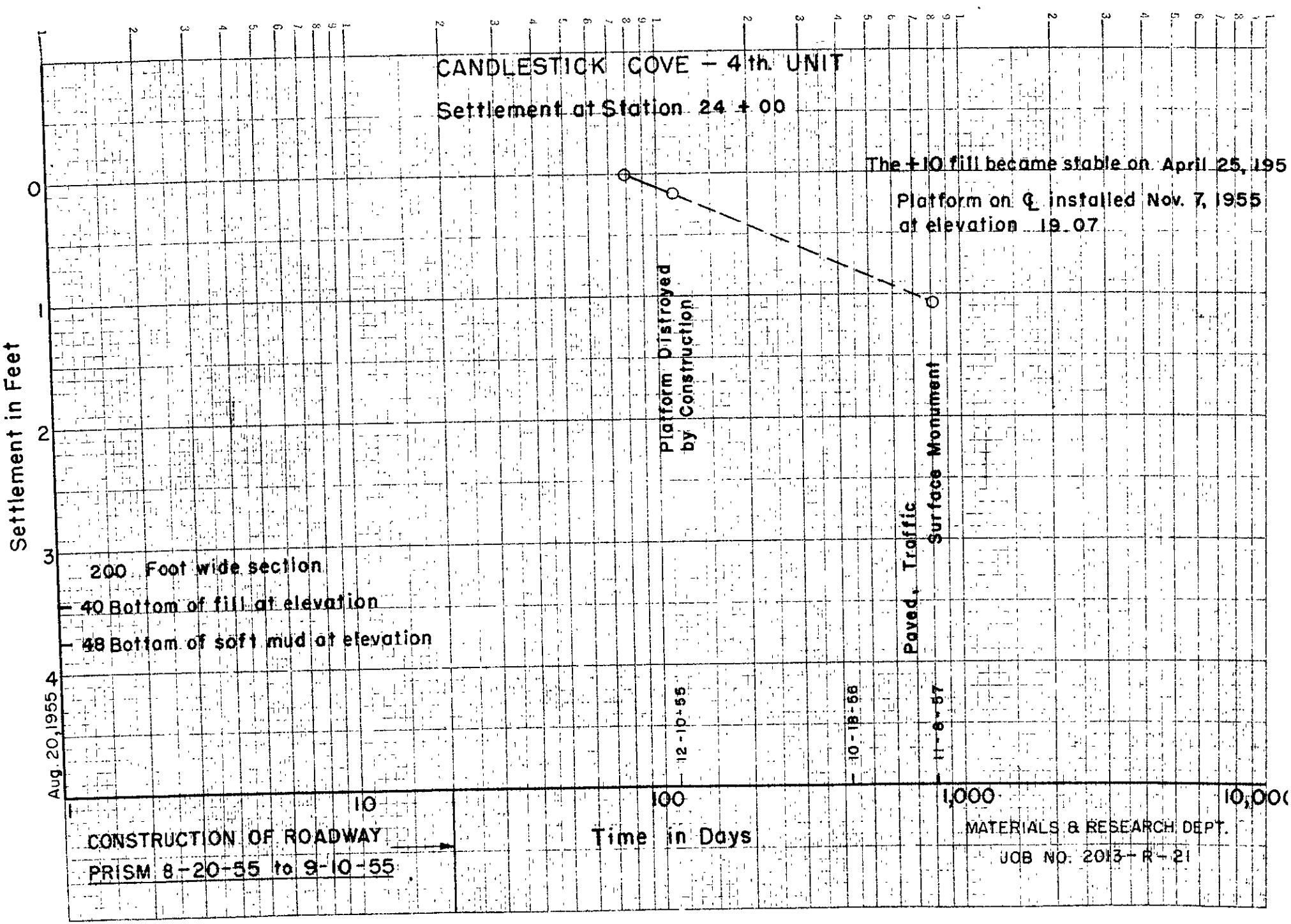


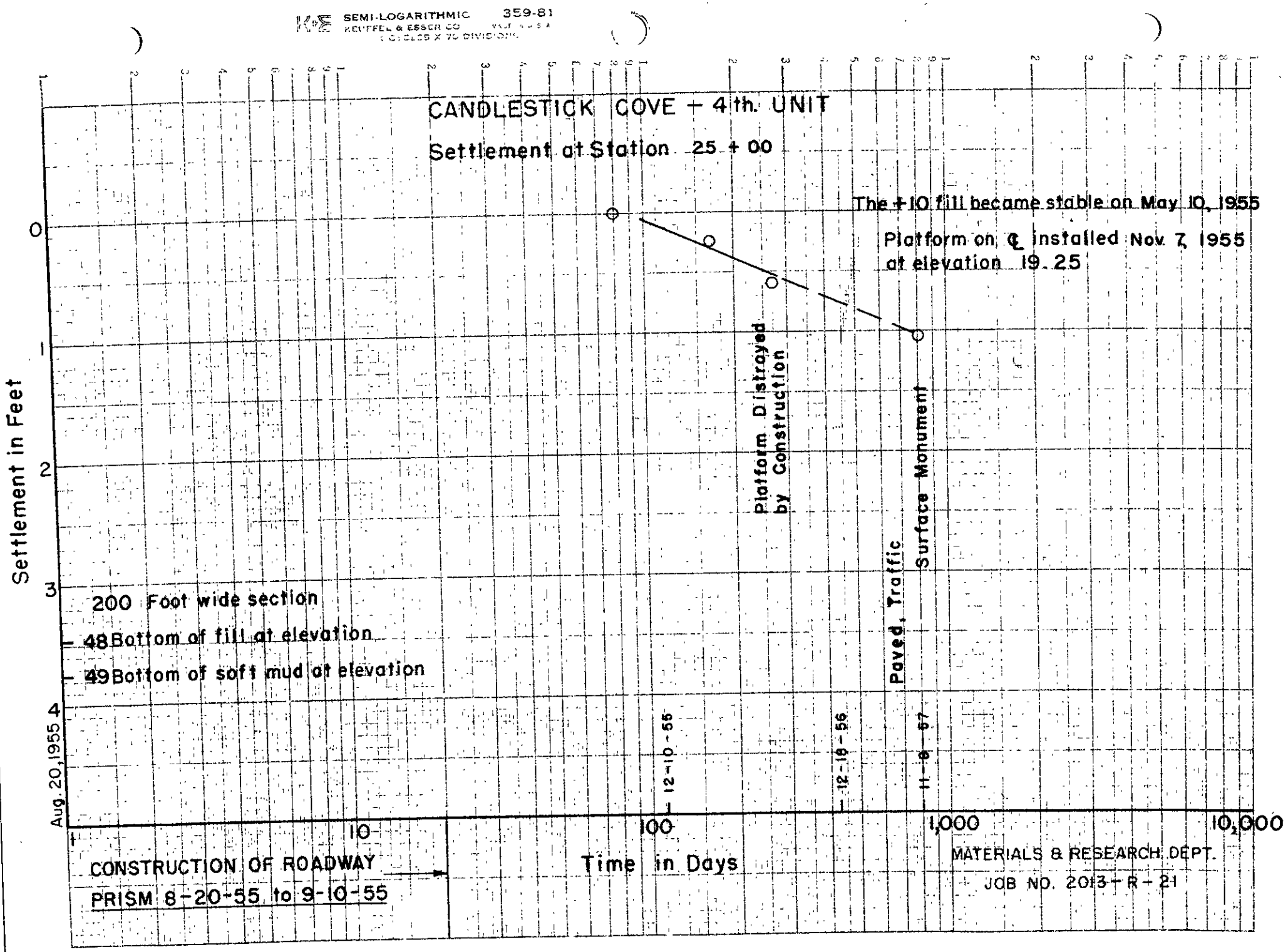
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



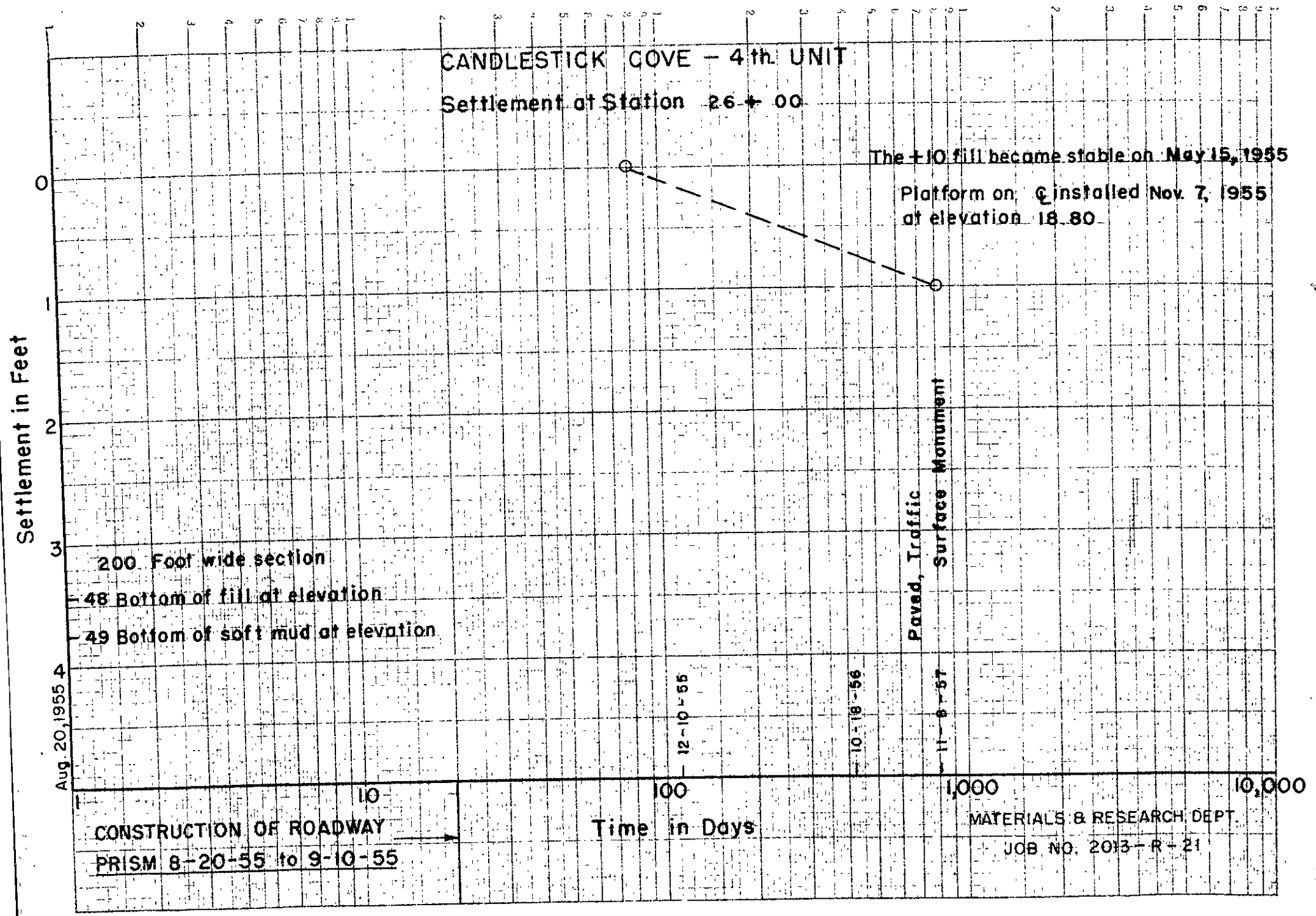

SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO.
4 CYC. 65 X 70 DIVISIONS
CANDLESTICK COVE – 4th. UNIT
Settlement at Station 22 + 00
The +10 fill became stable on April 25, 1955
Platform on ℄ installed Nov. 7, 1955
at elevation 16.46
Platform Distroyed by Construction
Paved, Traffic Surface Monument
12-10-55
10-18-56
11-8-57
250 Foot wide section
18 Bottom of fill at elevation
48 Bottom of soft mud at elevation
Settlement in Feet
0
1
2
3
4
Aug. 20, 1955
10
100
1,000
10,000
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 8-20-55 to 9-10-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

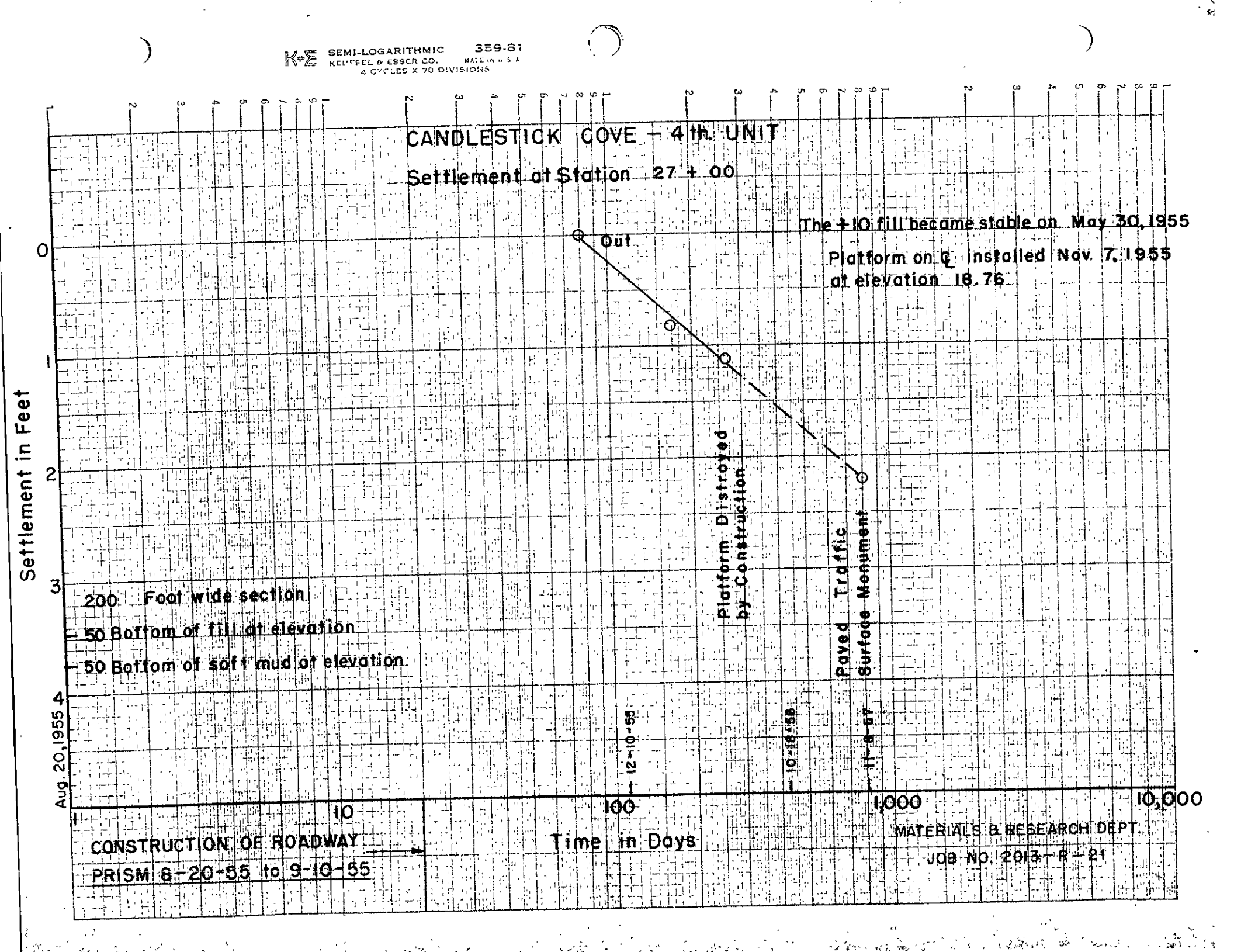

SEMI-LOGARITHMIC 359 81
KEUFFEL & ESSER CO.
3 CYCLES X 70 DIVISIONS
CANDLESTICK COVE – 4th. UNIT
Settlement at Station 23 + 00
The +10 fill became stable on April 25, 1955
Platform on ℄ installed Nov. 7, 1955
at elevation 19.21
Settlement in Feet
0
1
2
3
4
Aug. 20, 1955
200 Foot wide section
33 Bottom of fill at elevation
47 Bottom of soft mud at elevation
12-10-55
10-18-56
11-9-57
Paved, Traffic
1
10
100
1,000
10,000
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 8-20-55 to 9-10-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

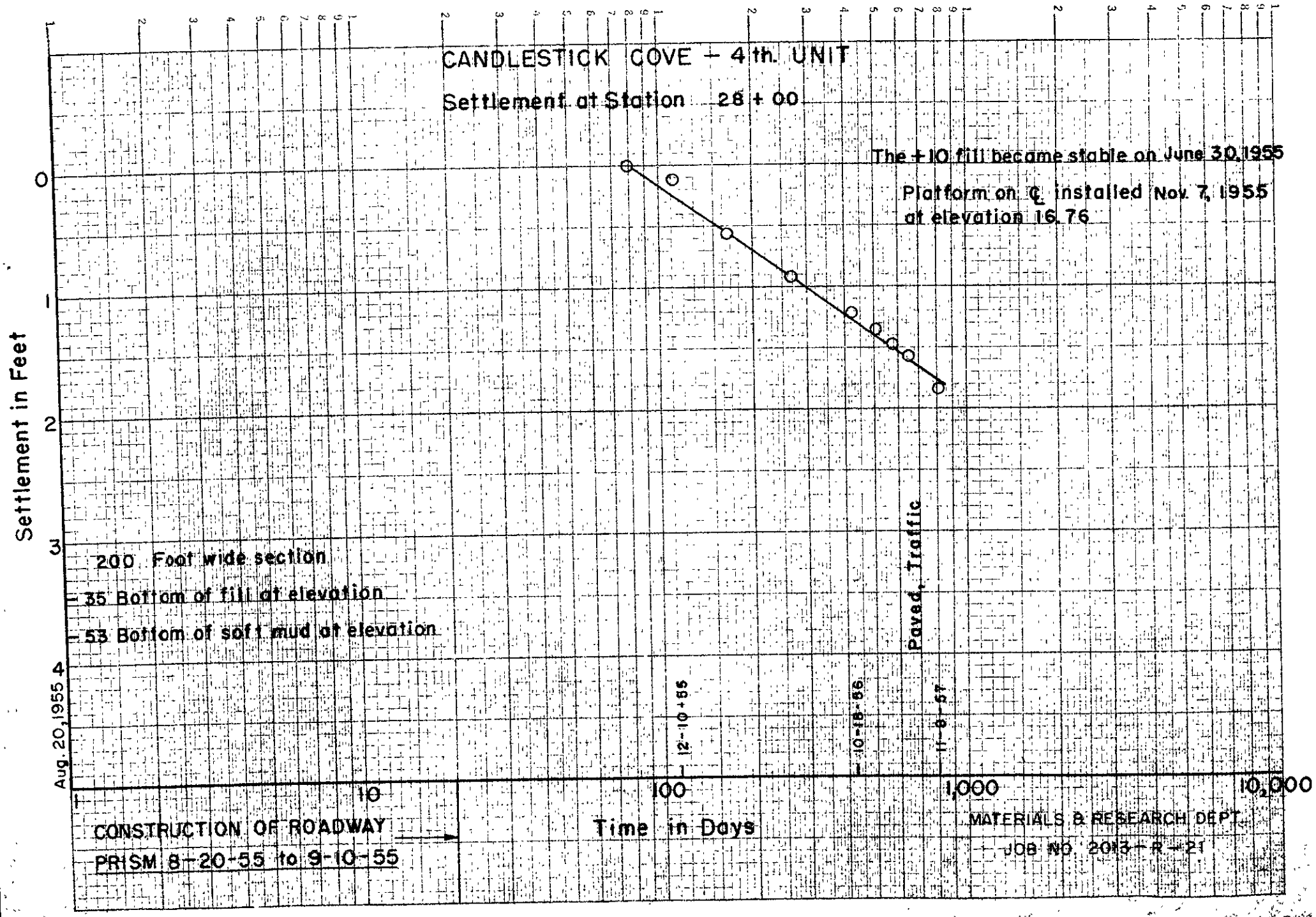

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
2 CYCLES X 70 DIVISIONS
CANDLESTICK COVE – 4th UNIT
Settlement at Station 24 + 00
The +10 fill became stable on April 25, 195
Platform on ℄ installed Nov. 7, 1955
at elevation 19.07
Platform Distroyed by Construction
Paved Traffic
Surface Monument
12-10-55
10-18-56
11-8-57
200 Foot wide section
40 Bottom of fill at elevation
48 Bottom of soft mud at elevation
Settlement in Feet
Aug. 20, 1955
0
1
2
3
4
1
10
100
1,000
10,000
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 8-20-55 to 9-10-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

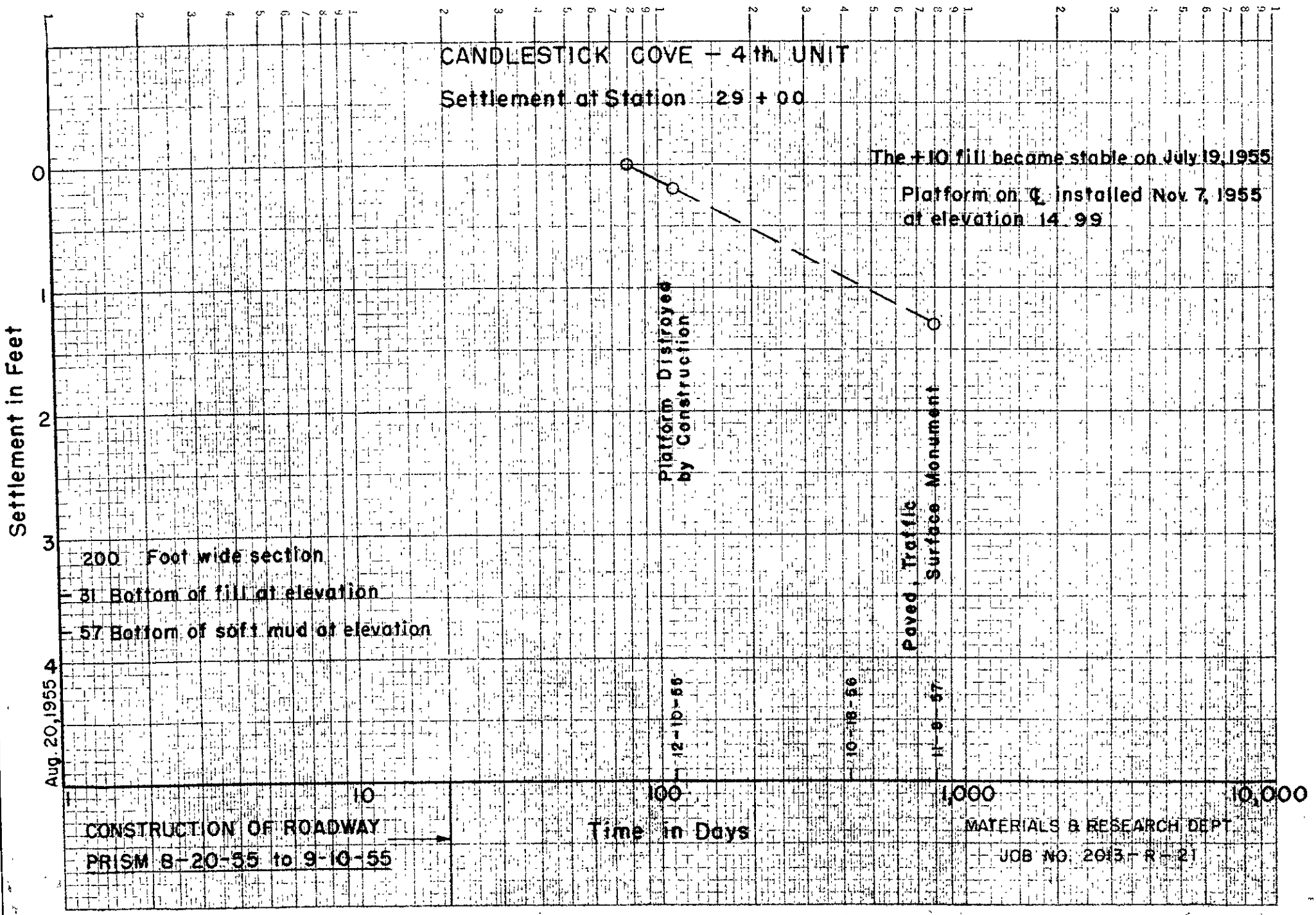

SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO.
CANDLESTICK COVE – 4th. UNIT
Settlement at Station 25 + 00
The +10 fill became stable on May 10, 1955
Platform on ℄ installed Nov. 7, 1955
at elevation 19.25
Platform Distroyed by Construction
Paved, Traffic
Surface Monument
12-10-55
12-19-56
11-8-57
200 Foot wide section
48 Bottom of fill at elevation
49 Bottom of soft mud at elevation
Settlement in Feet
0
1
2
3
4
Aug. 20, 1955
1
10
100
1,000
10,000
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 8-20-55 to 9-10-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

K+E SEMI-LOGARITHMIC 359-61
KEUFFEL & ESSER CO.
3 CYCLES X 70 DIVISIONS

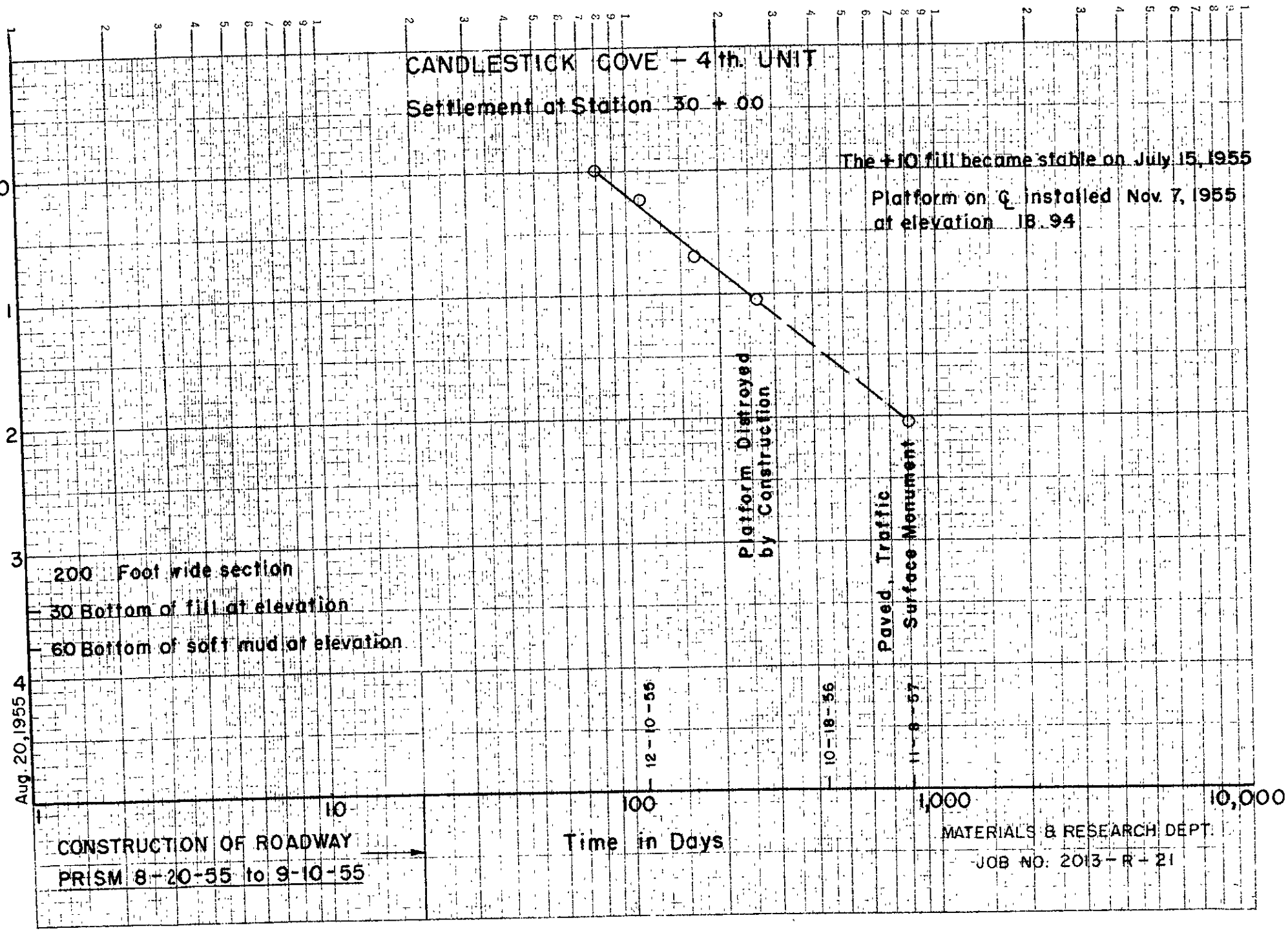



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE – 4th UNIT
Settlement at Station 27 + 00
The +10 fill became stable on May 30, 1955
Platform on ℄ installed Nov. 7, 1955
at elevation 18.76
Out
Platform Distroyed by Construction
Paved Traffic Surface Monument
12-10-55
10-18-55
11-8-57
200 Foot wide section
50 Bottom of fill at elevation
50 Bottom of soft mud at elevation
Settlement in Feet
0
1
2
3
4
Aug. 20, 1955
1
10
100
1,000
10,000
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 8-20-55 to 9-10-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

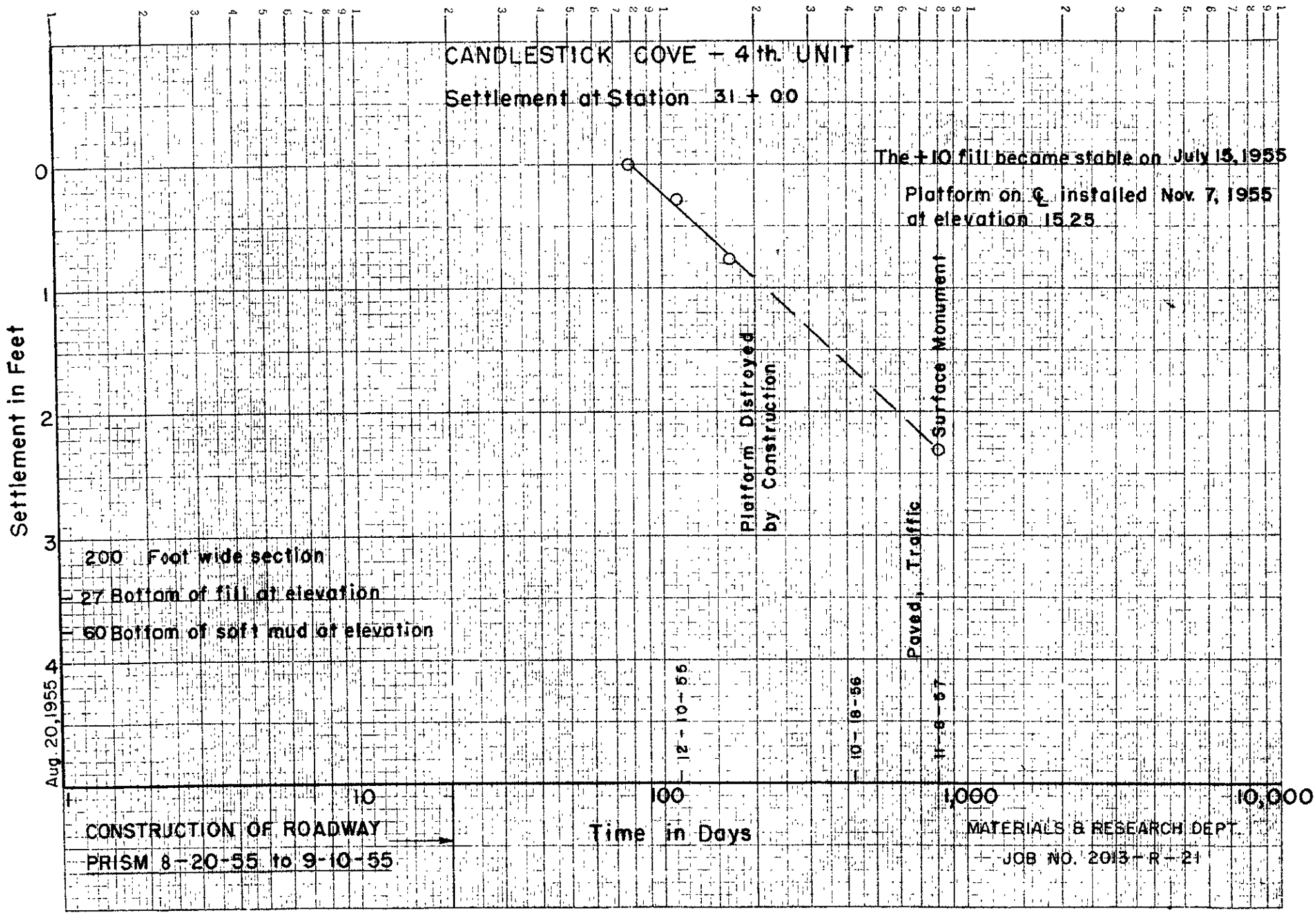

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE – 4th. UNIT
Settlement at Station 28 + 00
The +10 fill became stable on June 30, 1955
Platform on ℄ installed Nov. 7, 1955
at elevation 16.76
Settlement in Feet
0
1
2
3
4
Aug. 20, 1955
200 Foot wide section
35 Bottom of fill at elevation
53 Bottom of soft mud at elevation
12-10-55
10-18-56
11-8-57
Paved, Traffic
10
100
1,000
10,000
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 8-20-55 to 9-10-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21


SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO.
CANDLESTICK COVE – 4th. UNIT
Settlement at Station 29 + 00
The +10 fill became stable on July 19, 1955
Platform on ℄ installed Nov. 7, 1955
at elevation 14.99
Platform Distroyed by Construction
Paved Traffic Surface Monument
12-10-56
10-18-56
11-8-57
200 Foot wide section
-31 Bottom of fill at elevation
-57 Bottom of soft mud at elevation
Settlement in Feet
0
1
2
3
4
Aug. 20, 1955
10
100
1,000
10,000
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 8-20-55 to 9-10-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

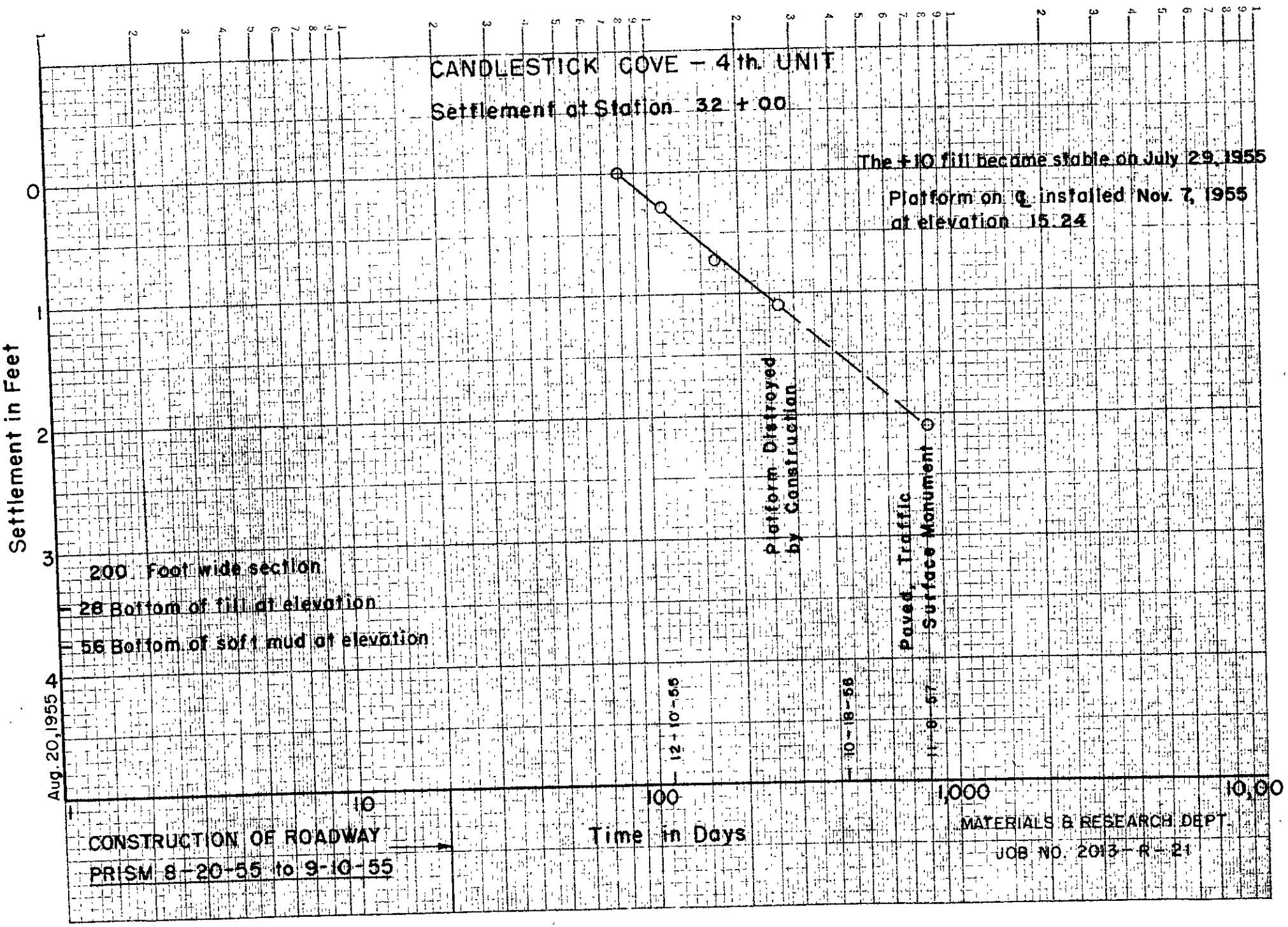
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

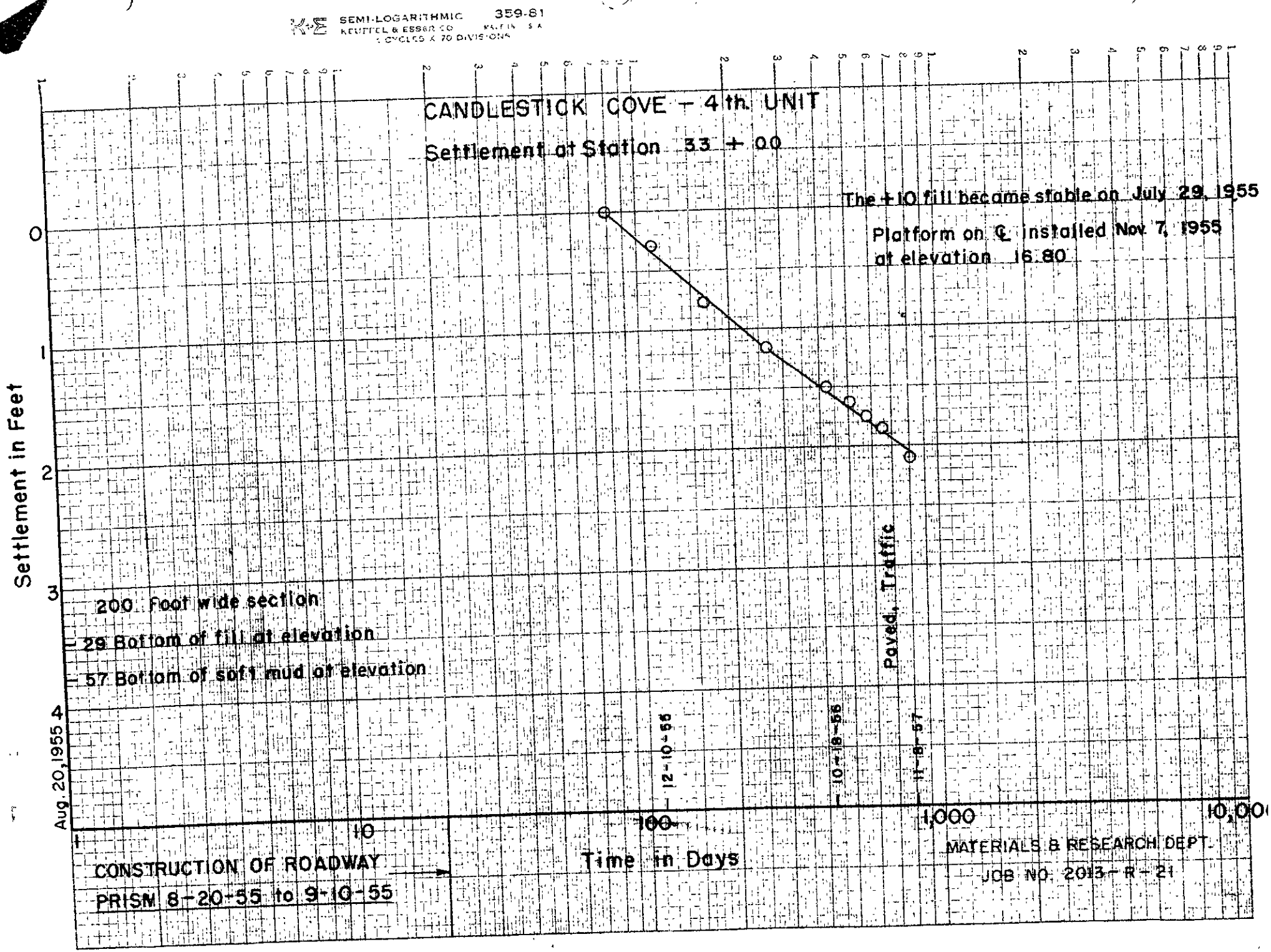


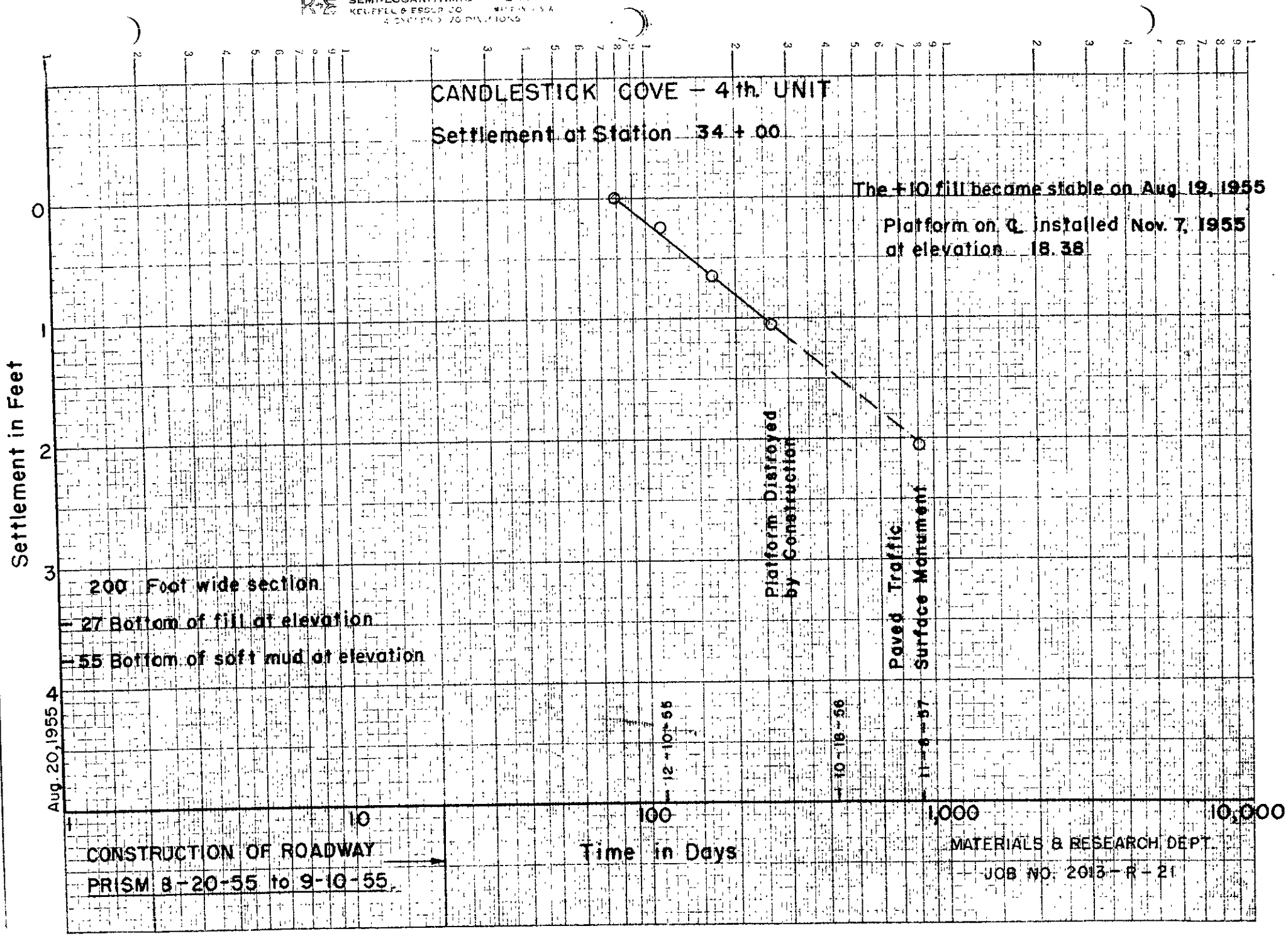
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO.
4 CYCLES X 70 DIVISIONS

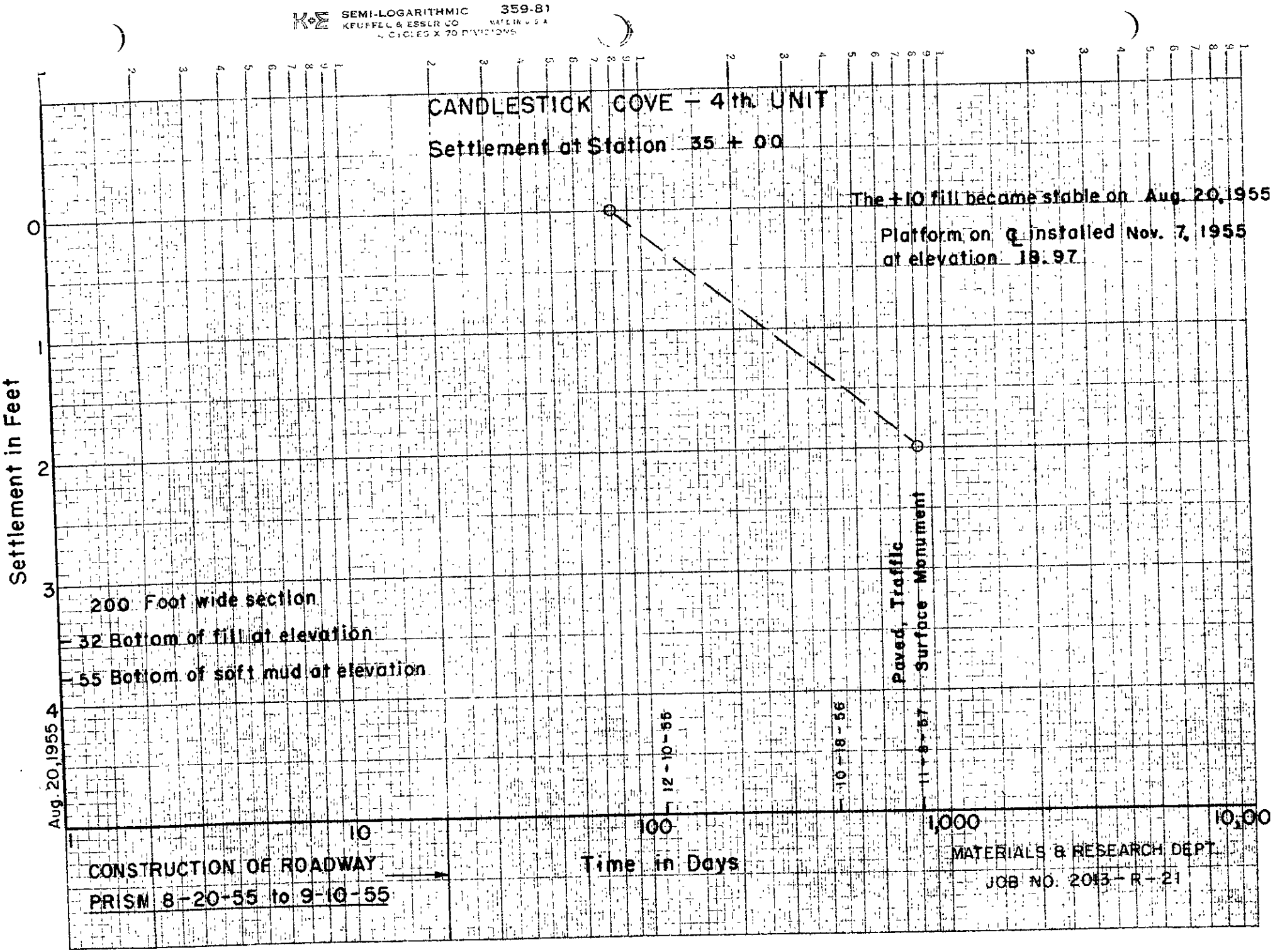


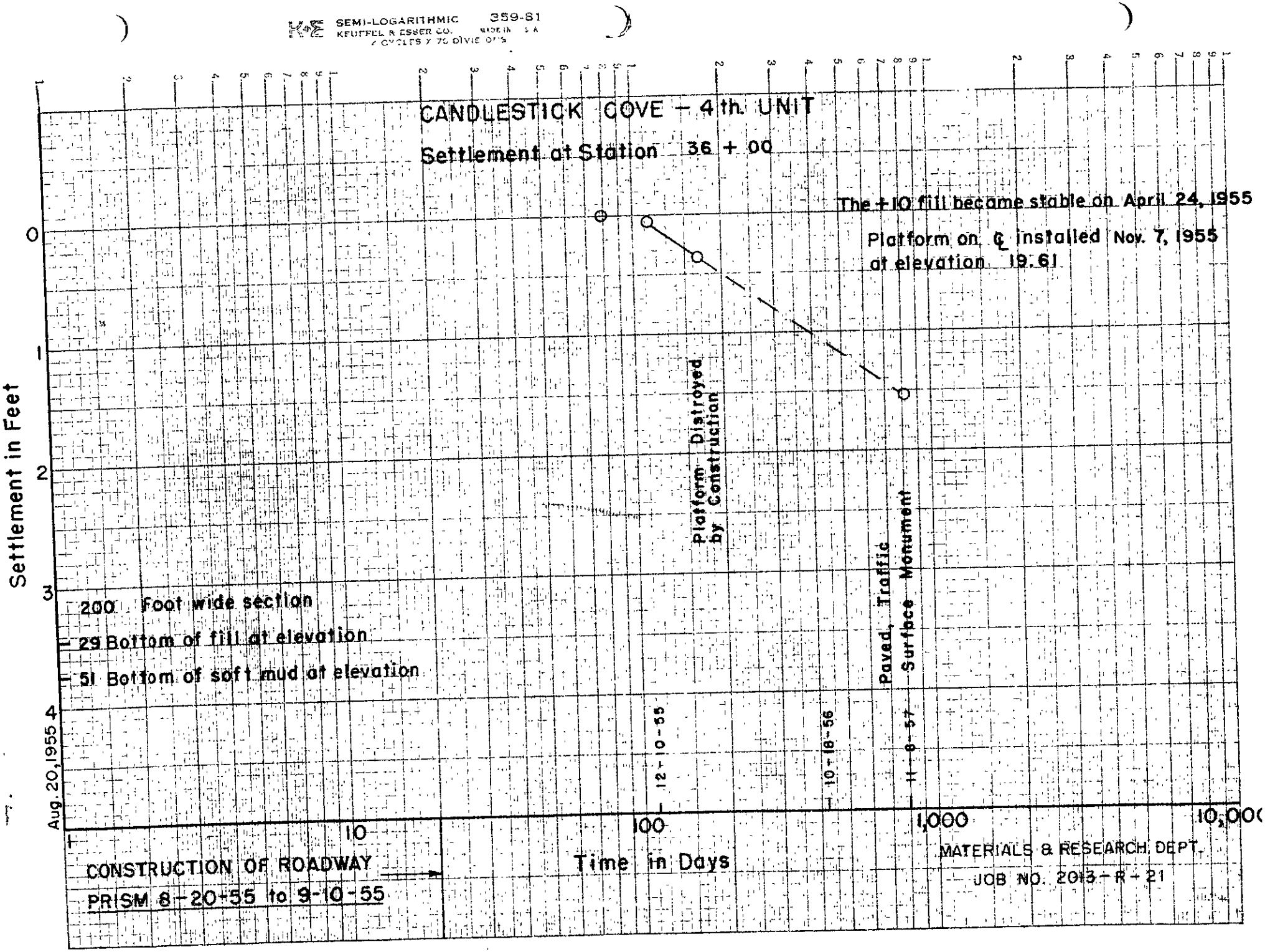
K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

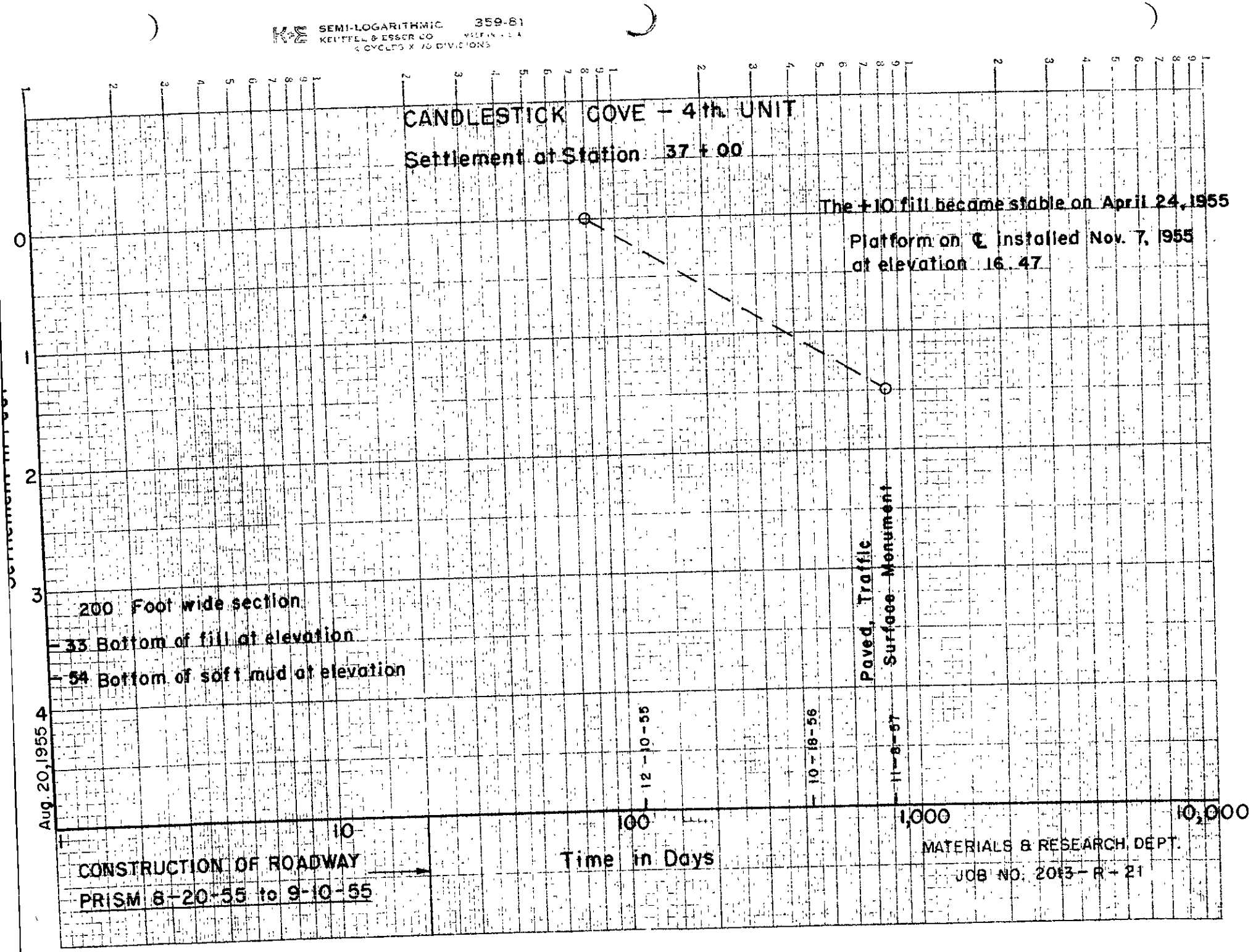


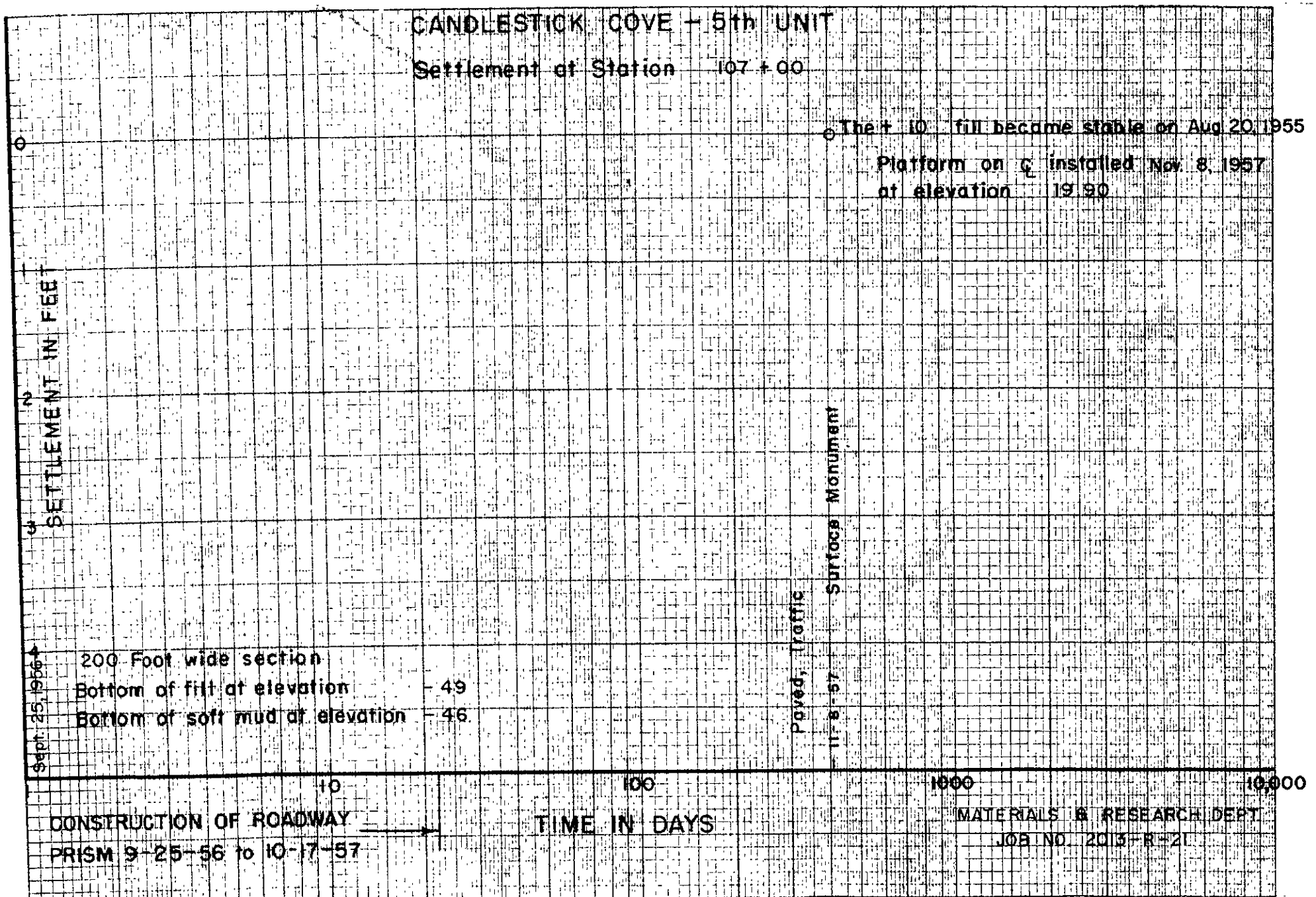

SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO.
1 CYCLES X 70 DIVISIONS
CANDLESTICK COVE - 4th UNIT
Settlement at Station 33 + 00
The +10 fill became stable on July 29, 1955
Platform on ℄ installed Nov. 7, 1955
at elevation 16.80
Settlement in Feet
0
1
2
3
4
Aug. 20, 1955
200 Foot wide section
29 Bottom of fill at elevation
57 Bottom of soft mud at elevation
Paved, Traffic
12-10-55
10-18-56
11-8-57
1
10
100
1,000
10,000
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 8-20-55 to 9-10-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

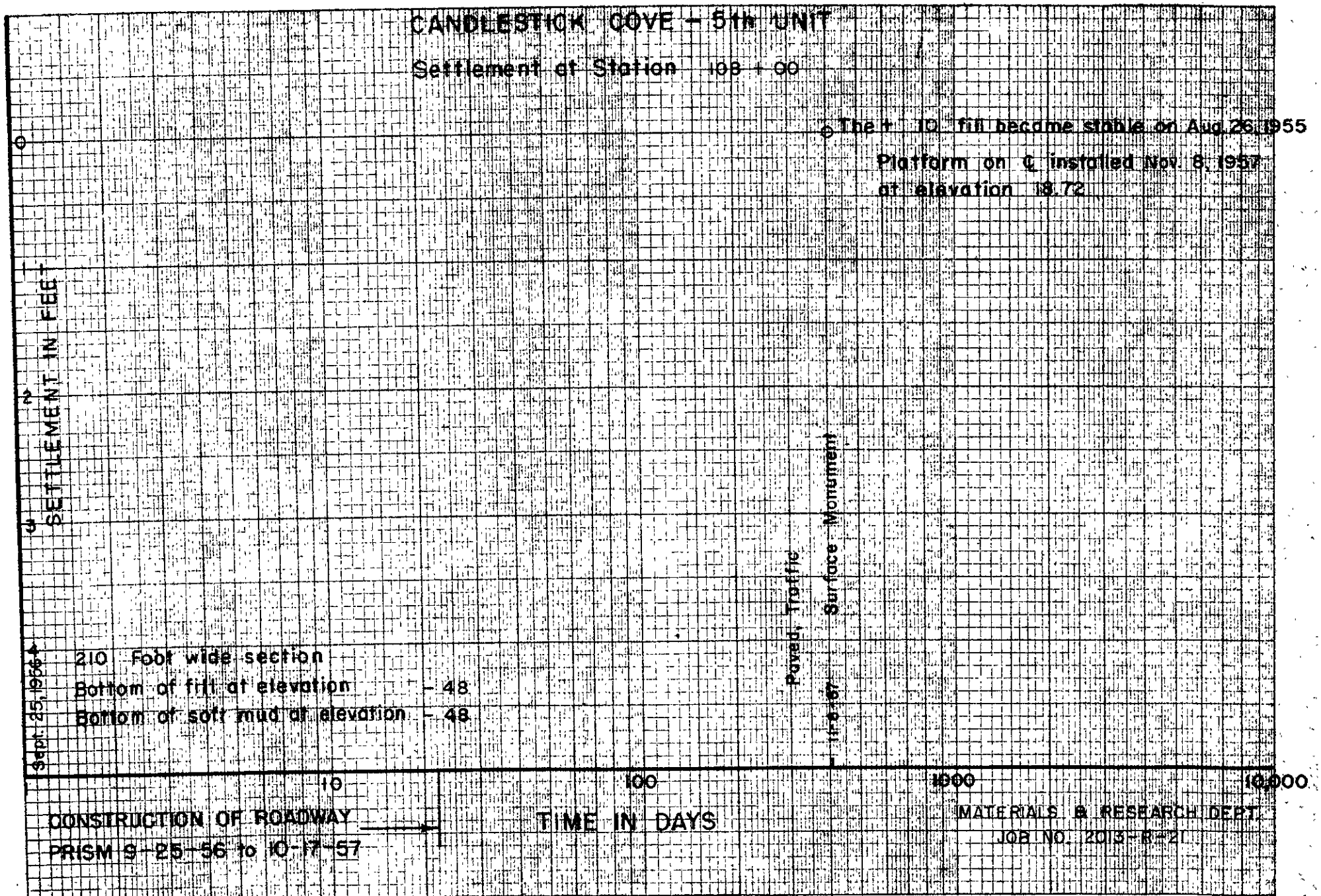

SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO.
CANDLESTICK COVE – 4th. UNIT
Settlement at Station 34 + 00
The +10 fill became stable on Aug. 19, 1955
Platform on ℄ installed Nov. 7, 1955
at elevation 18.38
Settlement in Feet
0
1
2
3
4
Aug. 20, 1955
200 Foot wide section
– 27 Bottom of fill at elevation
–55 Bottom of soft mud at elevation
Platform Distroyed by Construction
Paved Traffic
Surface Monument
12-10-55
10-18-56
11-6-57
1
10
100
1,000
10,000
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 8-20-55 to 9-10-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

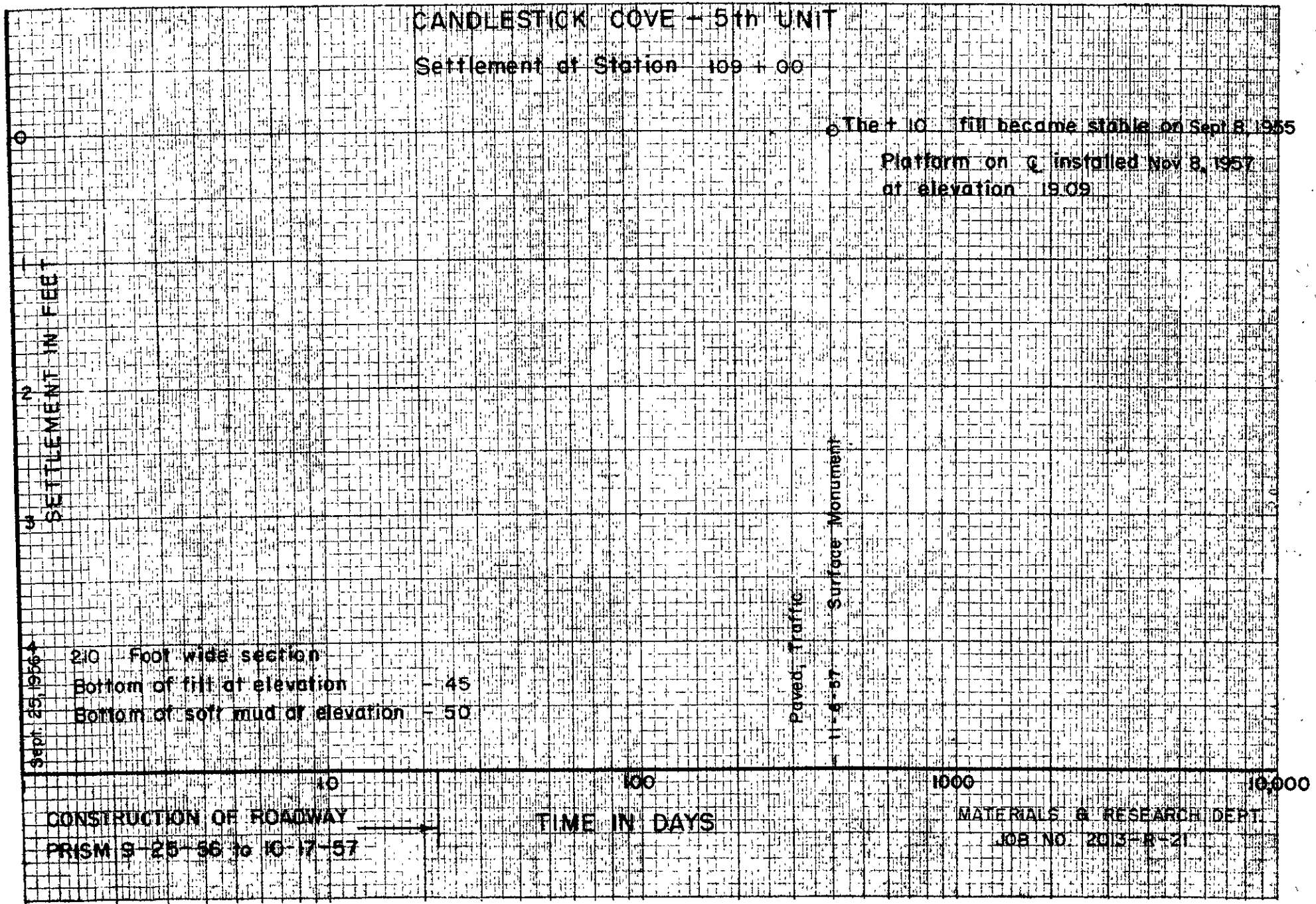

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
3 CYCLES X 70 DIVISIONS
CANDLESTICK COVE – 4th UNIT
Settlement at Station 35 + 00
The +10 fill became stable on Aug. 20, 1955
Platform on ℄ installed Nov. 7, 1955
at elevation 18.97
Settlement in Feet
0
1
2
3
4
Aug. 20, 1955
200 Foot wide section
-32 Bottom of fill at elevation
-55 Bottom of soft mud at elevation
12-10-56
10-18-56
Paved, Traffic
11-8-57
Surface Monument
1
10
100
1,000
10,00
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 8-20-55 to 9-10-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

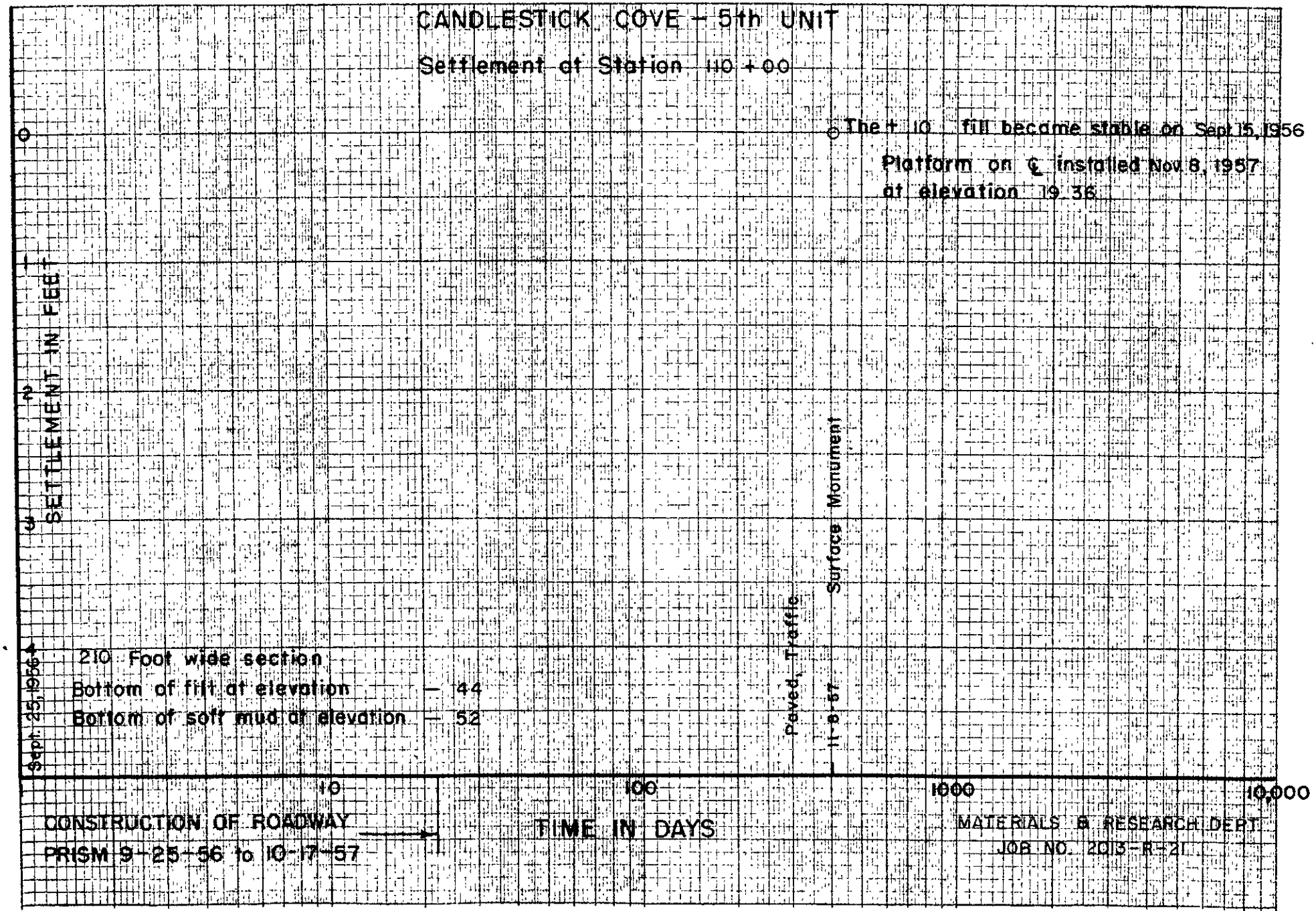

SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO.
CANDLESTICK COVE – 4th. UNIT
Settlement at Station 36 + 00
The +10 fill became stable on April 24, 1955
Platform on ℄ installed Nov. 7, 1955
at elevation 19.61
Platform Distroyed by Construction
Paved, Traffic Surface Monument
200 Foot wide section
29 Bottom of fill at elevation
51 Bottom of soft mud at elevation
Aug. 20, 1955
12-10-55
10-18-56
11-8-57
Settlement in Feet
0
1
2
3
4
1
10
100
1,000
10,000
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 8-20-55 to 9-10-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

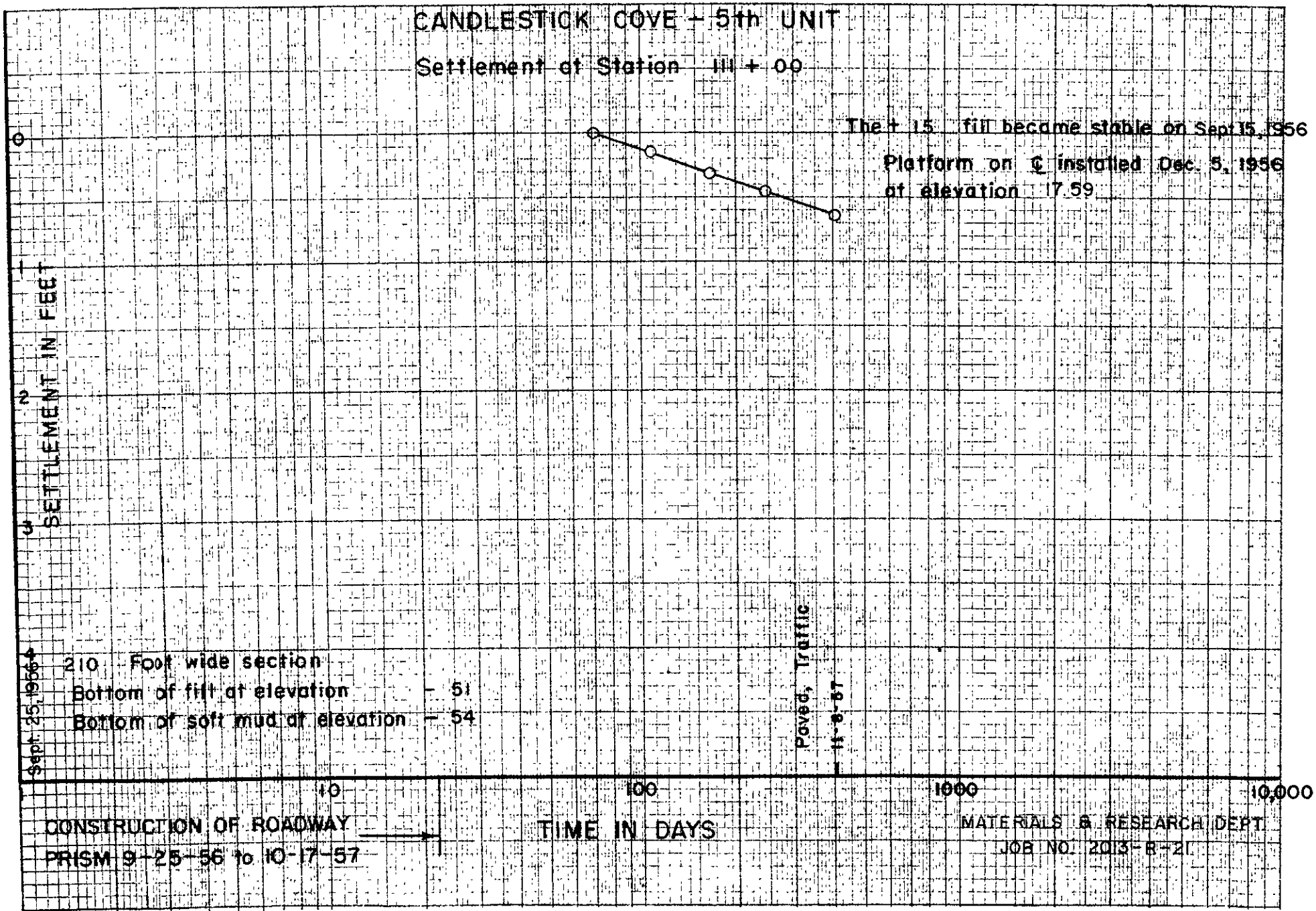

K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO.
4 CYCLES X 70 DIVISIONS
CANDLESTICK COVE – 4th UNIT
Settlement at Station 37 + 00
The +10 fill became stable on April 24, 1955
Platform on ℄ installed Nov. 7, 1955
at elevation 16.47
Paved, Traffic Surface Monument
200 Foot wide section
-33 Bottom of fill at elevation
-54 Bottom of soft mud at elevation
12-10-55
10-18-56
11-8-57
Settlement in Feet
0
1
2
3
4
Aug. 20, 1955
10
100
1,000
10,000
Time in Days
CONSTRUCTION OF ROADWAY
PRISM 8-20-55 to 9-10-55
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

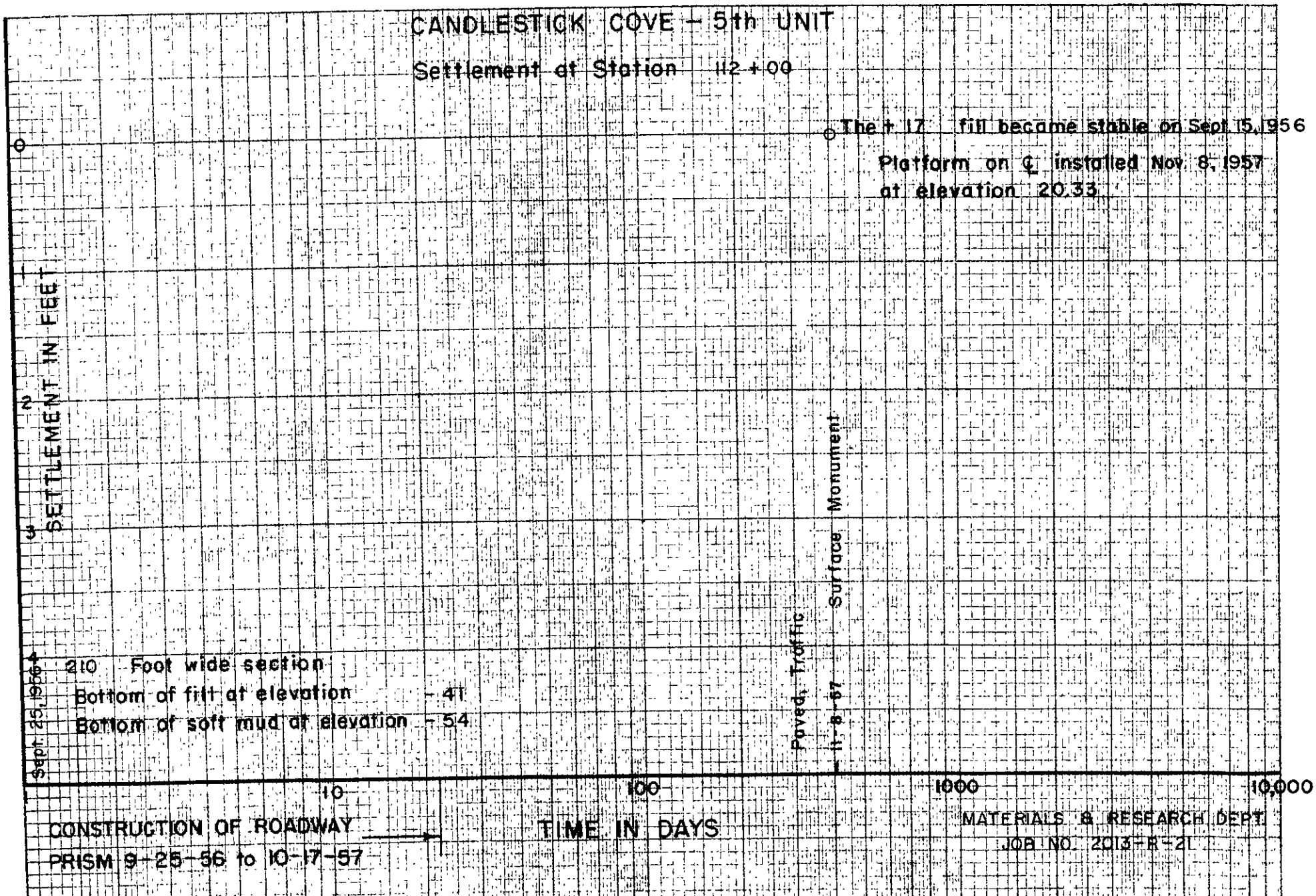

CANDLESTICK COVE – 5th UNIT
Settlement at Station 107 + 00
The + 10 fill became stable on Aug 20, 1955
Platform on ℄ installed Nov. 8, 1957
at elevation 19.90
SETTLEMENT IN FEET
0
1
2
3
4
Sept. 25, 1956
200 Foot wide section
Bottom of fill at elevation - 49
Bottom of soft mud at elevation - 46
Paved, Traffic
11-8-57
Surface Monument
10
100
1000
10,000
TIME IN DAYS
CONSTRUCTION OF ROADWAY
PRISM 9-25-56 to 10-17-57
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

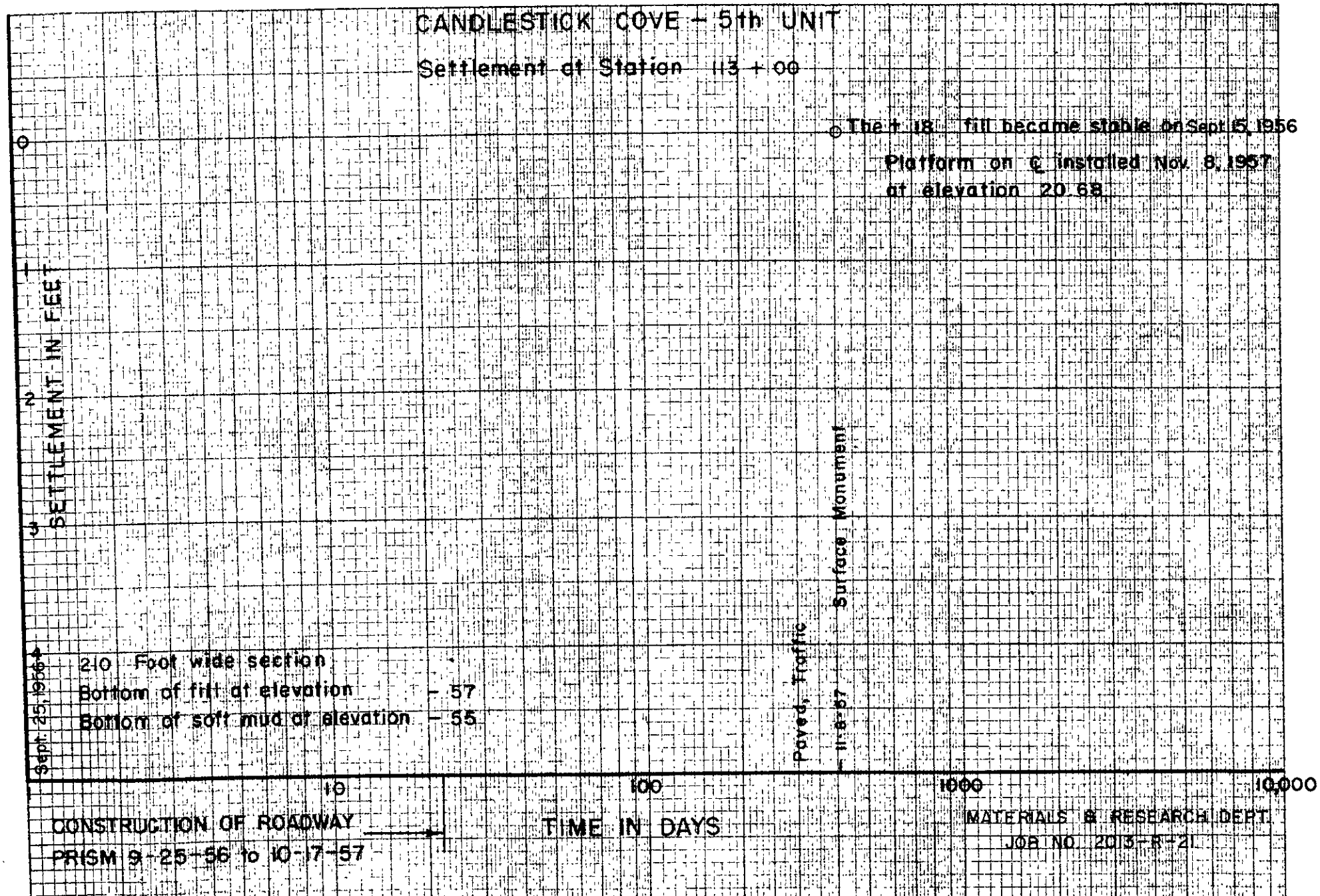

CANDLESTICK COVE – 5th UNIT
Settlement at Station 108 + 00
The + 10 fill became stable on Aug. 26, 1955
Platform on ℄ installed Nov. 8, 1957
at elevation 18.72
SETTLEMENT IN FEET
0
2
4
Sept. 25, 1956
210 Foot wide section
Bottom of fill at elevation – 48
Bottom of soft mud at elevation – 48
Paved, Traffic
11-8-67
Surface Monument
10
100
1000
10,000
TIME IN DAYS
CONSTRUCTION OF ROADWAY
PRISM 9-25-56 to 10-17-57
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

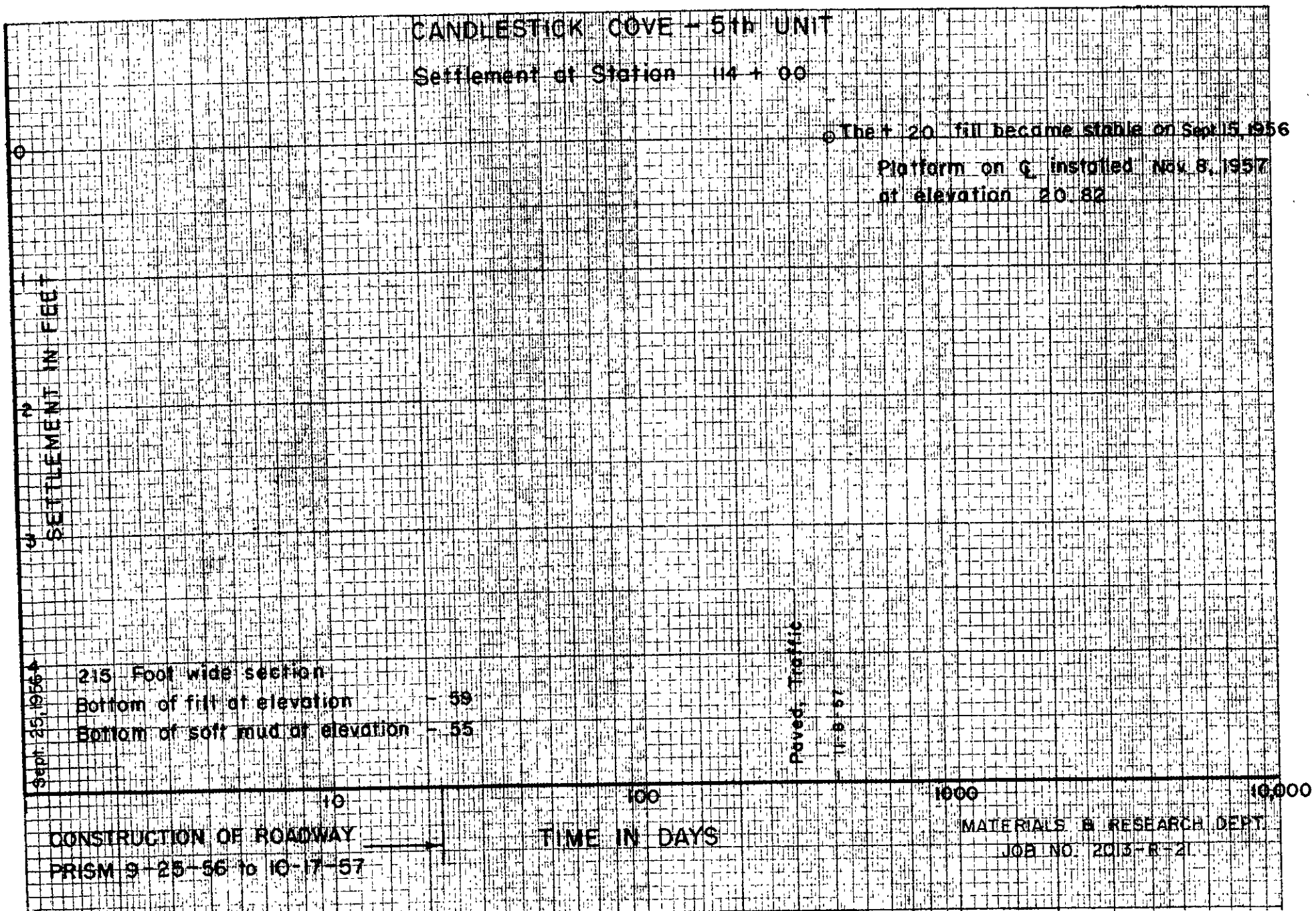

CANDLESTICK COVE – 5th UNIT
Settlement at Station 109 + 00
The + 10 fill became stable on Sept 8, 1955
Platform on ℄ installed Nov 8, 1957
at elevation 19.09
SETTLEMENT IN FEET
0
2
4
6
Sept. 26, 1956
210 Foot wide section
Bottom of fill at elevation – 45
Bottom of soft mud at elevation – 50
Paved Traffic
11-6-57 Surface Monument
10
100
1000
10,000
CONSTRUCTION OF ROADWAY
PRISM 9-25-56 to 10-17-57
TIME IN DAYS
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

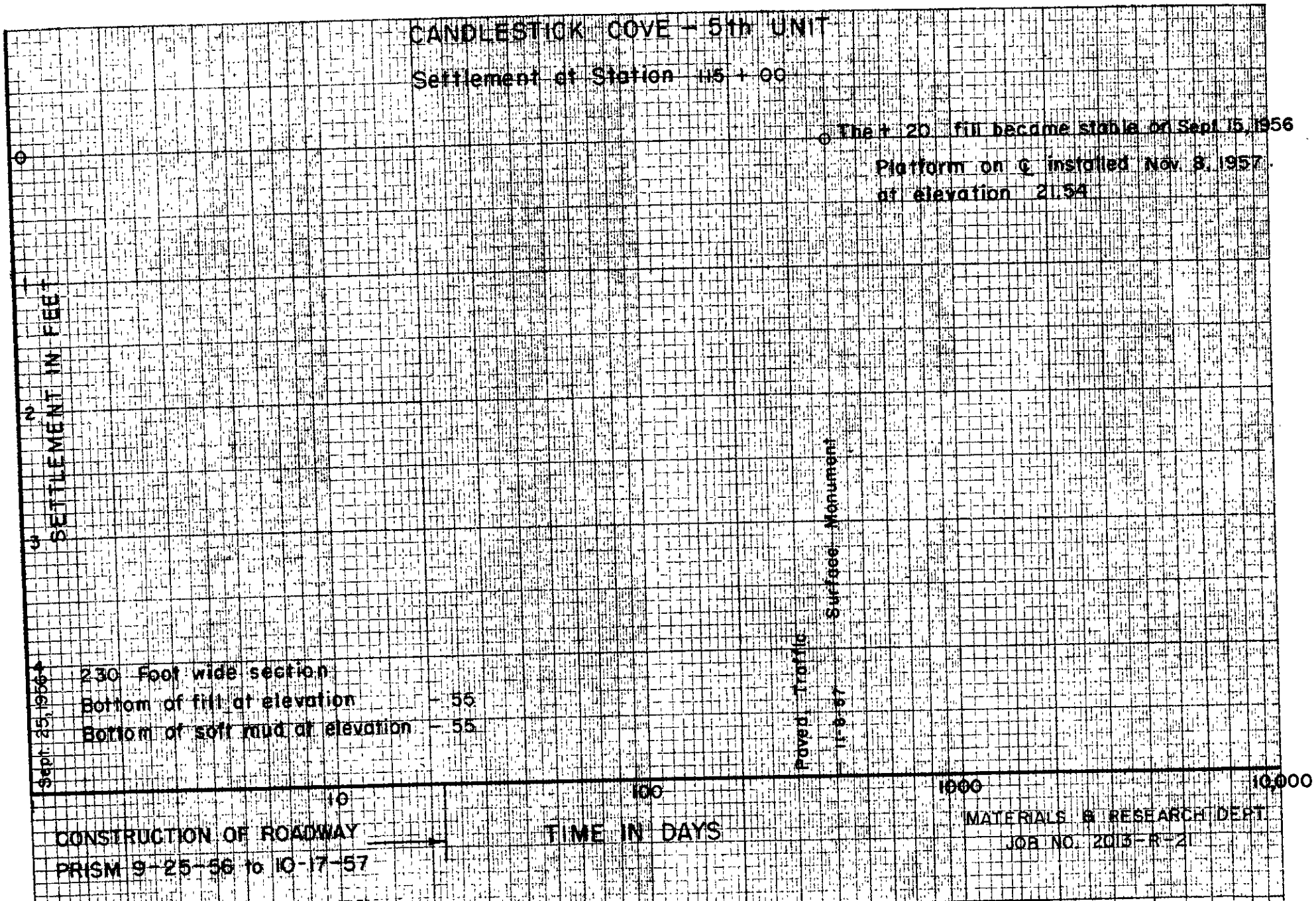

CANDLESTICK COVE – 5th UNIT
Settlement at Station 110 + 00
The + 110 fill became stable on Sept 15, 1956
Platform on ℄ installed Nov 8, 1957
at elevation 19.36
0
SETTLEMENT IN FEET
Surface Monument
Paved, Traffic
11-8-57
Sept. 25, 1956
210 Foot wide section
Bottom of fill at elevation — 44
Bottom of soft mud at elevation — 52
10
100
1000
10,000
CONSTRUCTION OF ROADWAY
PRISM 9-25-56 to 10-17-57
TIME IN DAYS
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

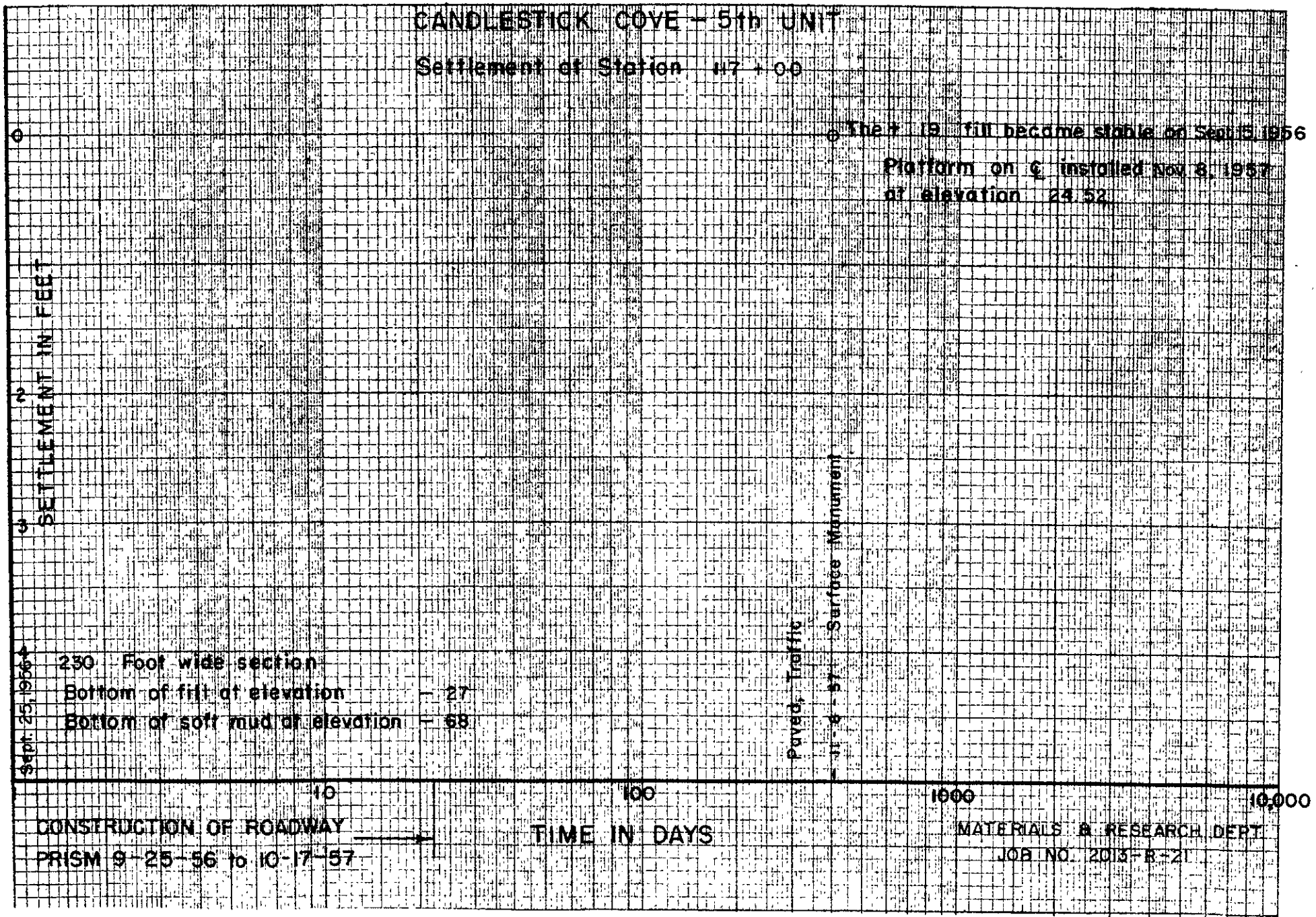

CANDLESTICK COVE – 5th UNIT
Settlement at Station 111 + 00
The + 15 fill became stable on Sept 15, 1956
Platform on ℄ installed Dec. 5, 1956
at elevation 17.59
SETTLEMENT IN FEET
0
1
2
3
Sept. 25, 1956
210 Foot wide section
Bottom of fill at elevation – 51
Bottom of soft mud at elevation – 54
Paved, Traffic
11-8-57
10
100
1000
10,000
TIME IN DAYS
CONSTRUCTION OF ROADWAY
PRISM 9-25-56 to 10-17-57
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-8-21

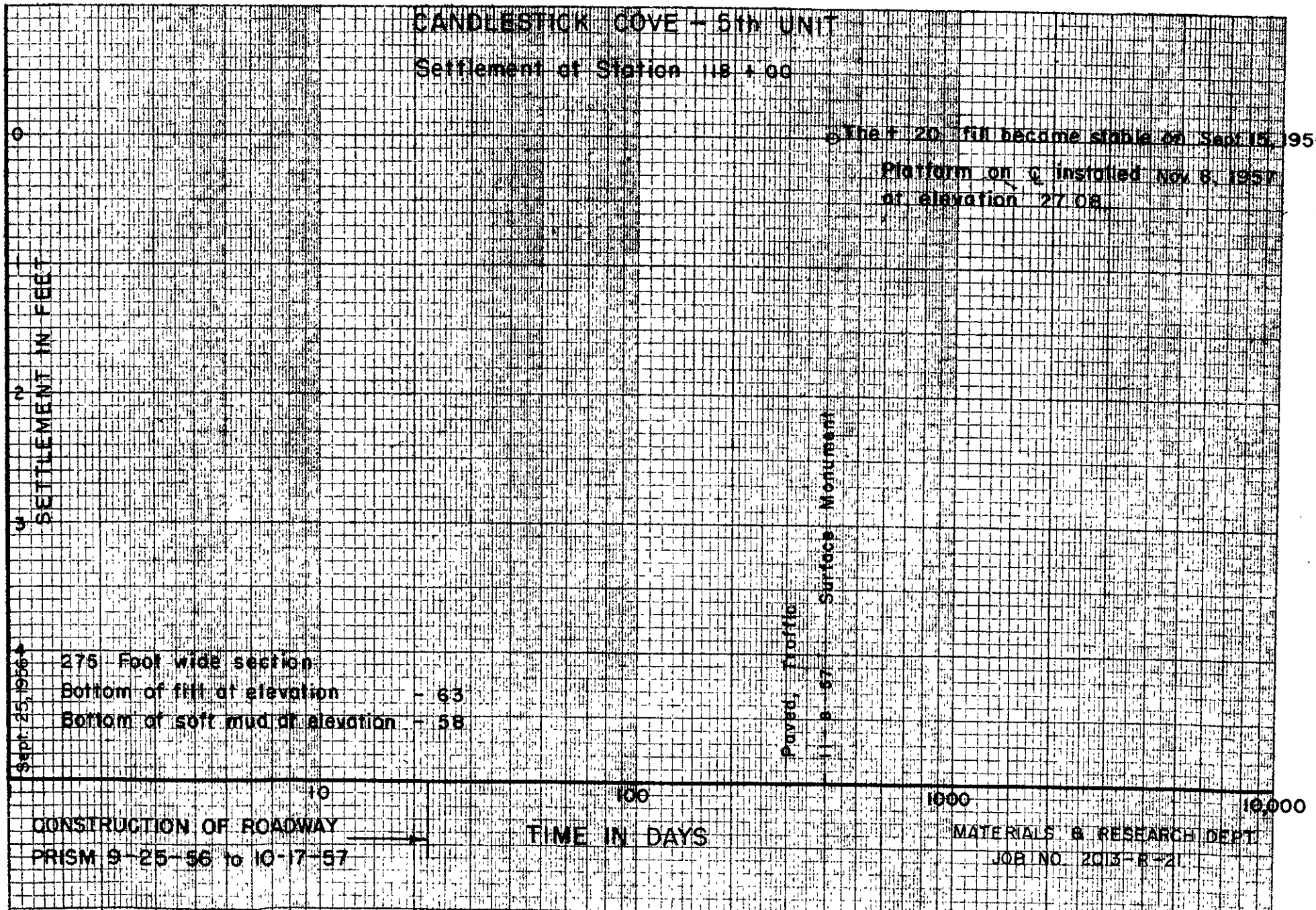

CANDLESTICK COVE – 5th UNIT
Settlement at Station 112+00
The + 17 fill became stable on Sept. 15, 1956
Platform on ℄ installed Nov. 8, 1957
at elevation 20.33
0
1
2
3
4
SETTLEMENT IN FEET
Sept. 25, 1956
210 Foot wide section
Bottom of fill at elevation -41
Bottom of soft mud at elevation -54
Paved, Traffic
11-8-57
Surface Monument
10
100
1000
10,000
CONSTRUCTION OF ROADWAY
PRISM 9-25-56 to 10-17-57
TIME IN DAYS
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

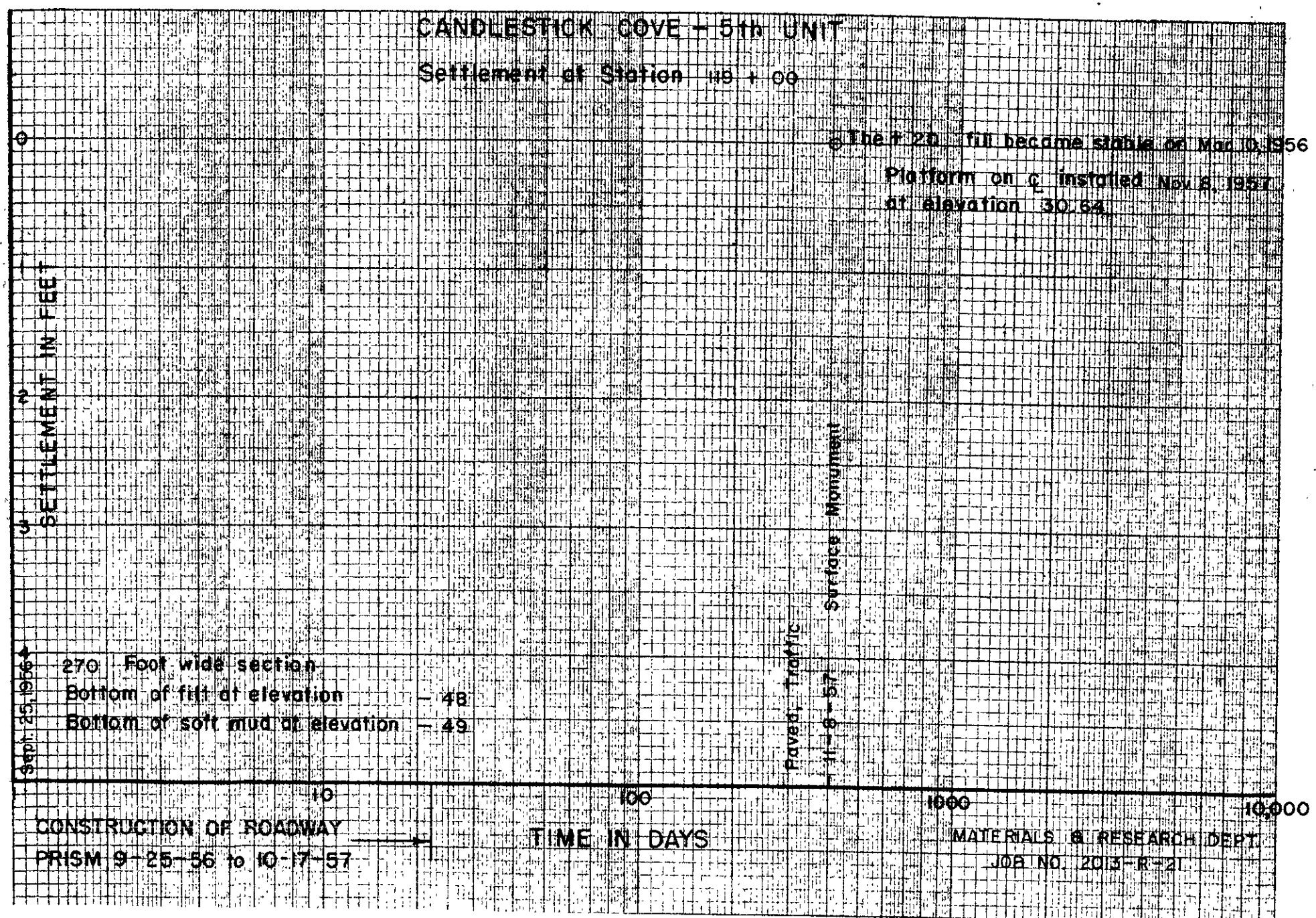

CANDLESTICK COVE - 5th UNIT
Settlement at Station 113 + 00
The + 18 fill became stable on Sept. 5, 1956
Platform on ℄ installed Nov. 8, 1957
at elevation 20.68
0
1
2
3
SETTLEMENT IN FEET
Sept. 25, 1956
210 Foot wide section
Bottom of fill at elevation - 57
Bottom of soft mud at elevation - 55
Payed, Traffic
11-8-57
Surface Monument
10
100
1000
10,000
CONSTRUCTION OF ROADWAY
PRISM 9-25-56 to 10-17-57
TIME IN DAYS
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

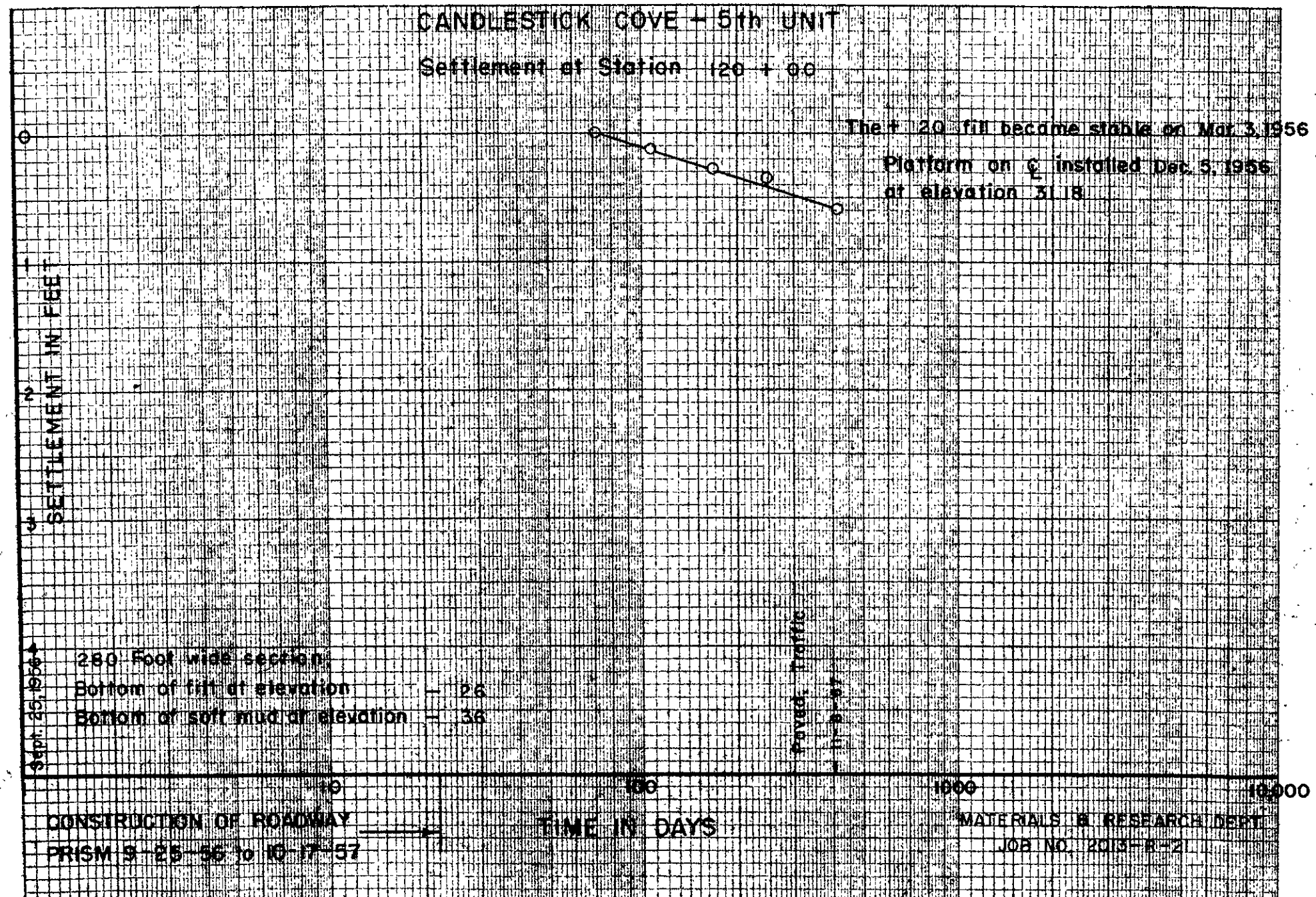

CANDLESTICK COVE - 5th UNIT
Settlement at Station 114 + 00
The + 20 fill became stable on Sept 15, 1956
Platform on ℄ installed Nov 8, 1957
at elevation 20.82
0
SETTLEMENT IN FEET
Sept. 25, 1956
215 Foot wide section
Bottom of fill at elevation - 59
Bottom of soft mud at elevation - 55
Paved, Traffic
11-8-57
10
100
1000
10,000
CONSTRUCTION OF ROADWAY
PRISM 9-25-56 to 10-17-57
TIME IN DAYS
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21


CANDLESTICK COVE - 5th UNIT
Settlement at Station 115 + 00
The + 20 fill became stable on Sept 15, 1956
Platform on ℄ installed Nov. 8, 1957
at elevation 21.54
SETTLEMENT IN FEET
0
1
2
3
Sept. 25, 1956
230 Foot wide section
Bottom of fill at elevation - 55
Bottom of soft mud at elevation - 55
Paved Traffic
11-6-57
Surface Monument
10
100
1000
10,000
TIME IN DAYS
CONSTRUCTION OF ROADWAY
PRISM 9-25-56 to 10-17-57
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

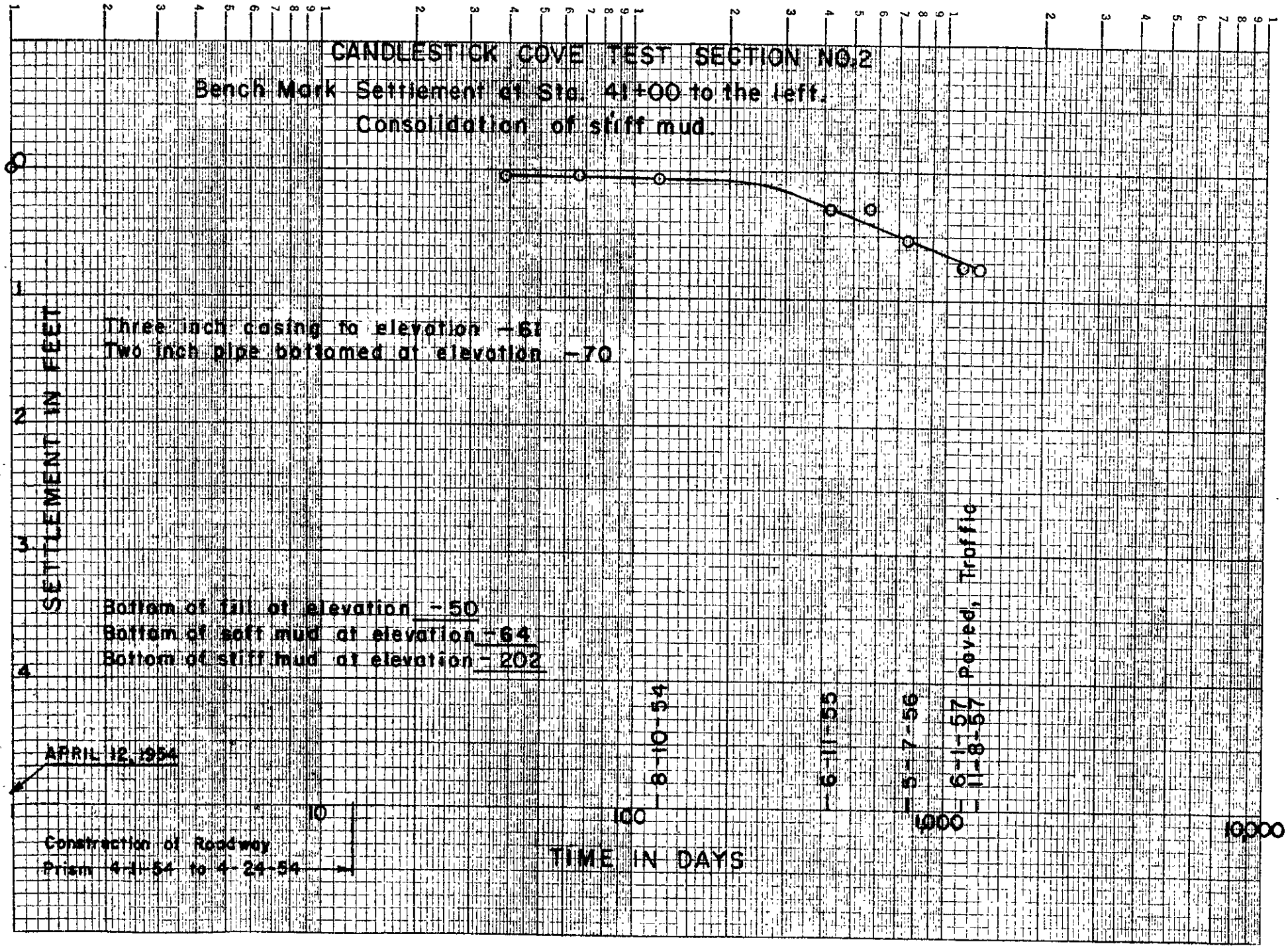

CANDLESTICK COVE – 5th UNIT
Settlement at Station 117 + 00
The 19' fill became stable on Sept. 15, 1956
Platform on ℄ installed Nov 8, 1957
at elevation 24.52
SETTLEMENT IN FEET
0
2
3
Sept. 25, 1956
230' Foot wide section
Bottom of fill at elevation – 27
Bottom of soft mud at elevation – 68
Paved, Traffic
Surface Monument
10
100
1000
10,000
CONSTRUCTION OF ROADWAY
PRISM 9-25-56 to 10-17-57
TIME IN DAYS
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21


CANDLESTICK COVE - 5th UNIT
Settlement at Station 118 + 00
0
2
4
SETTLEMENT IN FEET
The + 20' fill became stable on Sept. 15, 1956
Platform on ℄ installed Nov. 8, 1957
of elevation 27.08
Sept. 25, 1956
275 Foot wide section
Bottom of fill at elevation - 63
Bottom of soft mud at elevation - 58
Paved, Traffic
11-8-57
Surface Monument
10
100
1000
10,000
CONSTRUCTION OF ROADWAY
PRISM 9-25-56 to 10-17-57
TIME IN DAYS
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

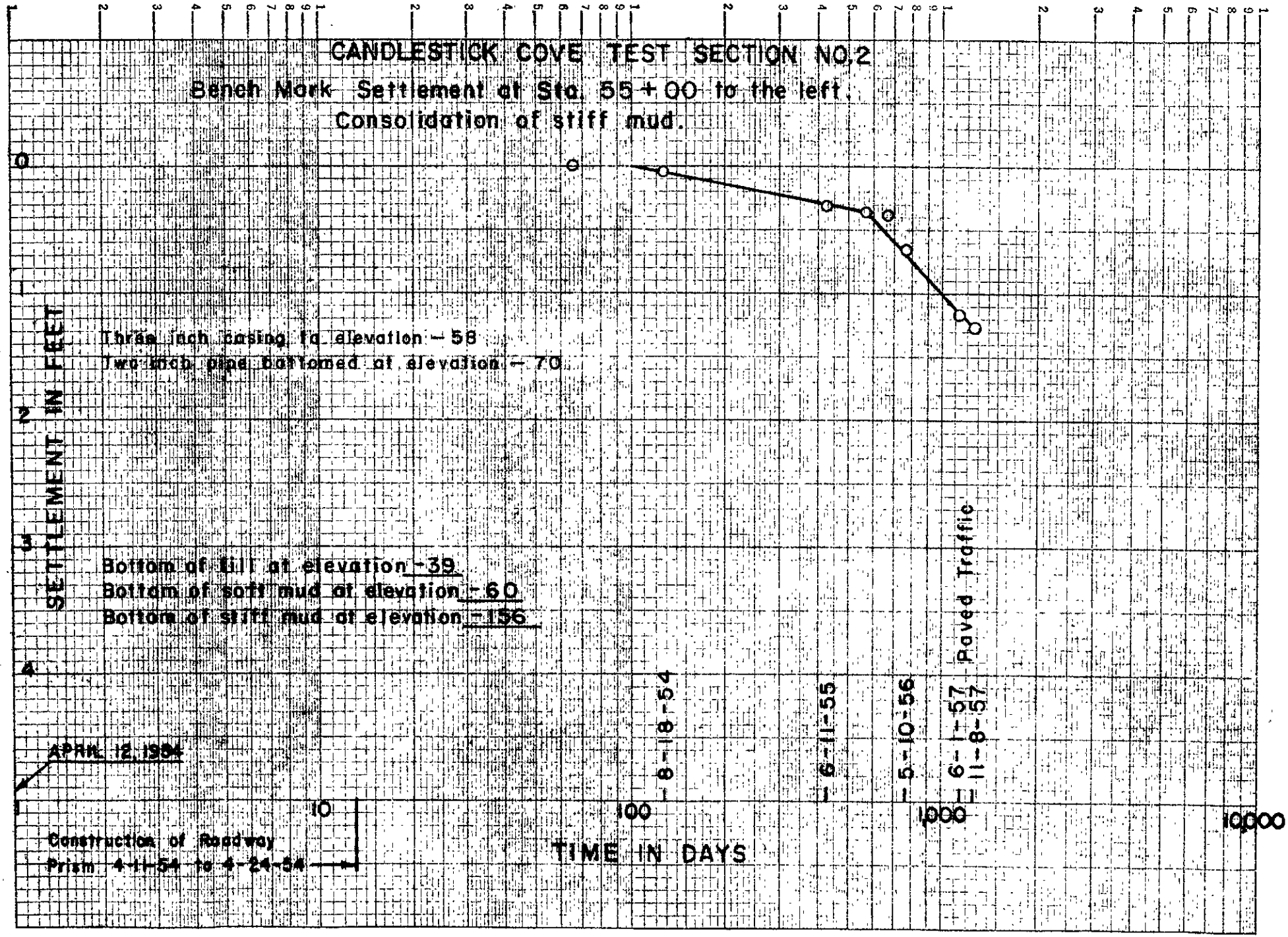

CANDLESTICK COVE – 5th UNIT
Settlement at Station 119 + 00
The # 2a fill became stable on Mar. 10, 1956
Platform on ℄ instalted Nov. 8, 1957
at elevation 30.64
0
2
3
SETTLEMENT IN FEET
Sept. 25, 1956
270 Foot wide section
Bottom of fill at elevation – 48
Bottom of soft mud at elevation – 49
Paved Traffic
11-8-57
Surface Monument
10
100
1000
10,000
CONSTRUCTION OF ROADWAY
PRISM 9-25-56 to 10-17-57
TIME IN DAYS
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21


CANDLESTICK COVE - 5th UNIT
Settlement at Station 120 + 00
The + 20 fill became stable on Mar. 3, 1956
Platform on ℄ installed Dec. 5, 1956
at elevation 31.18
SETTLEMENT IN FEET
0
1
2
3
4
Sept. 26, 1956
280 Foot wide section
Bottom of fill at elevation - 26
Bottom of soft mud at elevation - 36
Paved, Traffic
10
100
1000
10,000
CONSTRUCTION OF ROADWAY
PRISM 9-25-56 to 10-17-57
TIME IN DAYS
MATERIALS & RESEARCH DEPT.
JOB NO. 2013-R-21

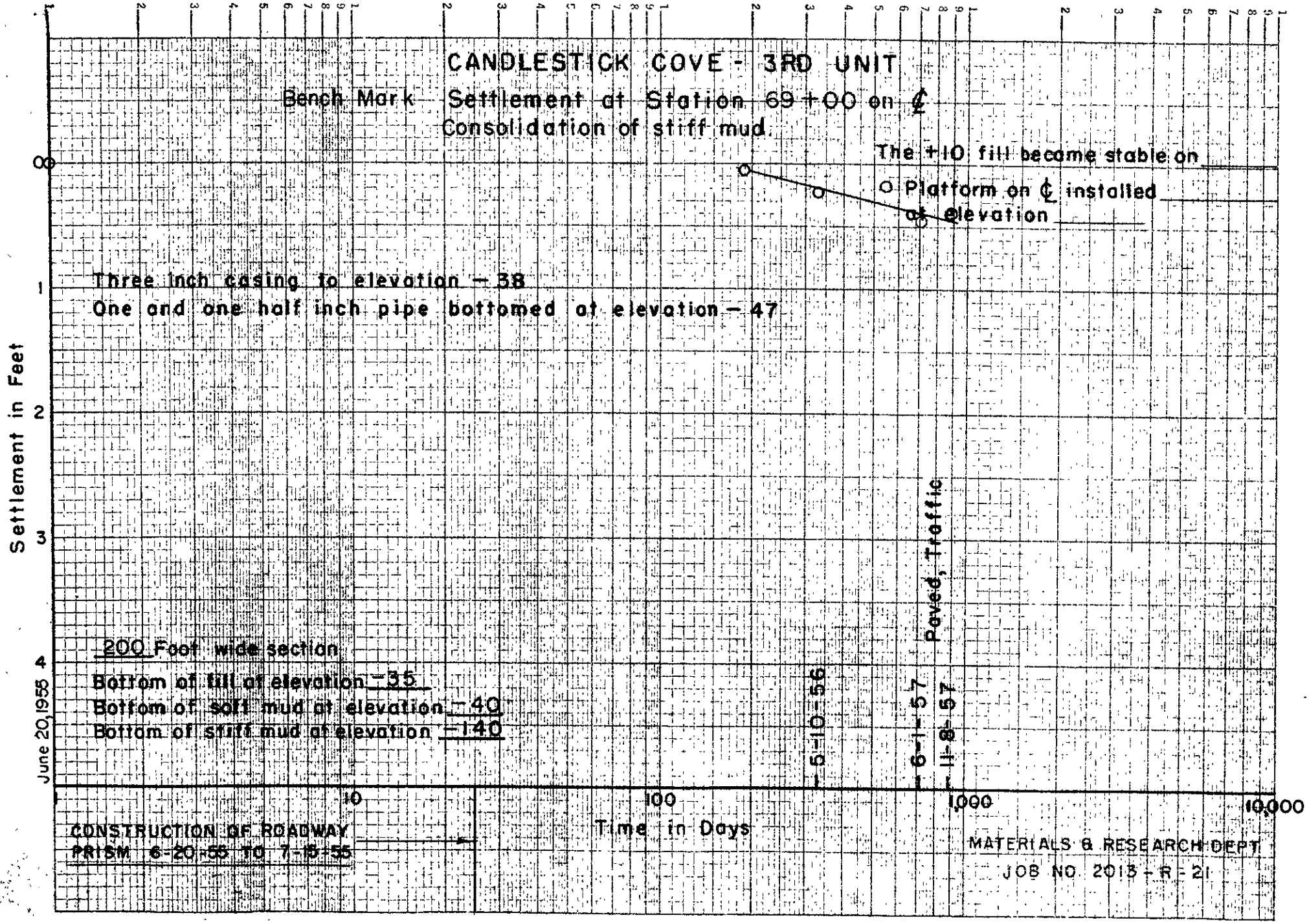
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 10 to the inch.
5th lines accented
MADE IN U.S.A.



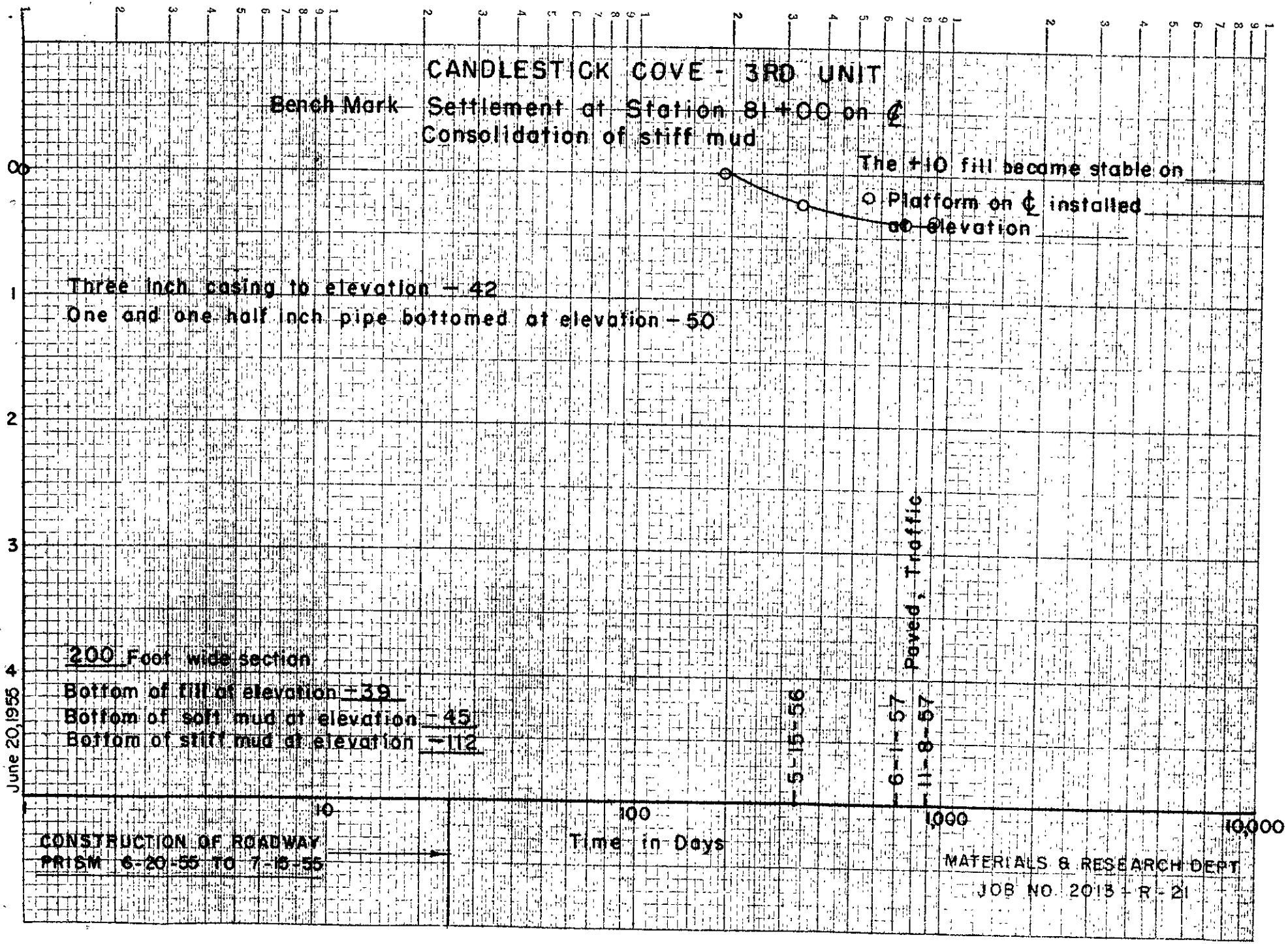
339-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 10 to the inch.
5th lines accented.
MADE IN U.S.A.



K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

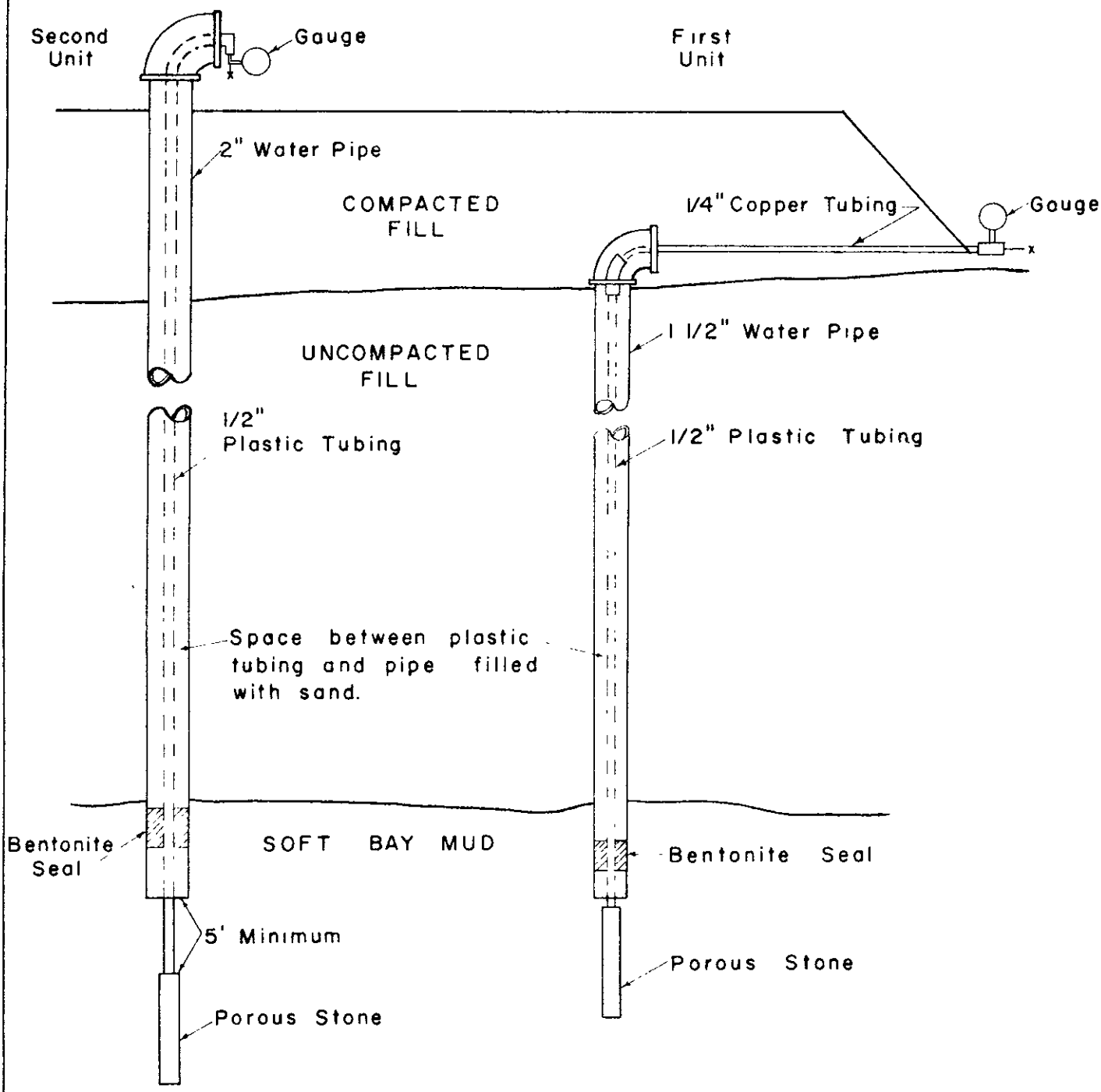


K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



EXCESS HYDROSTATIC PRESSURES

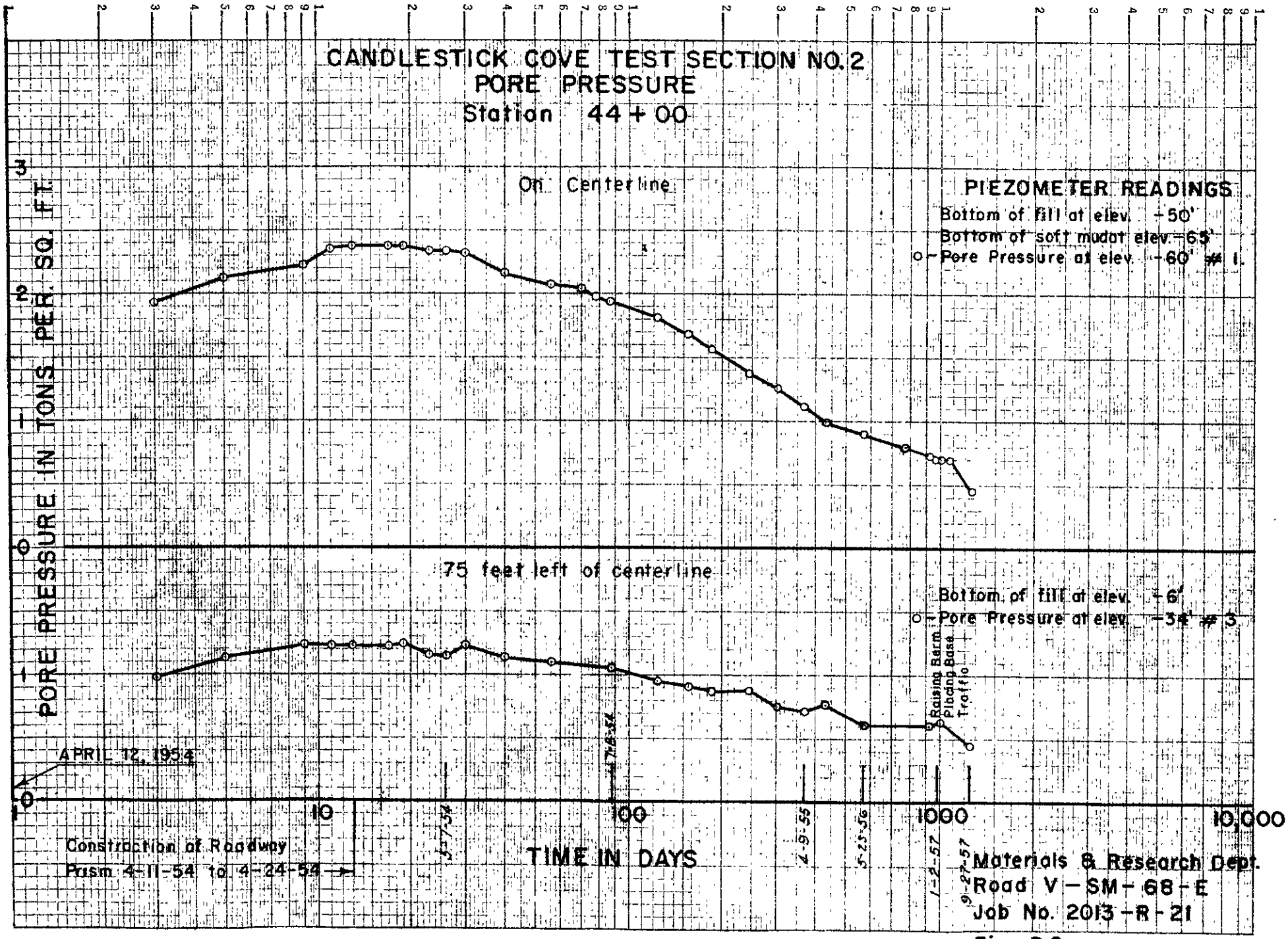
Subject	Plate
Installations, Type	P-1
Time Plots	P-2 to P-10
Variation vertically and horizontally	P-11 to P-16



No Scale

Materials & Research Department
TYPICAL PIEZOMETER INSTALLATIONS
Job No. 2013 – R – 21
Fig. No. P–1

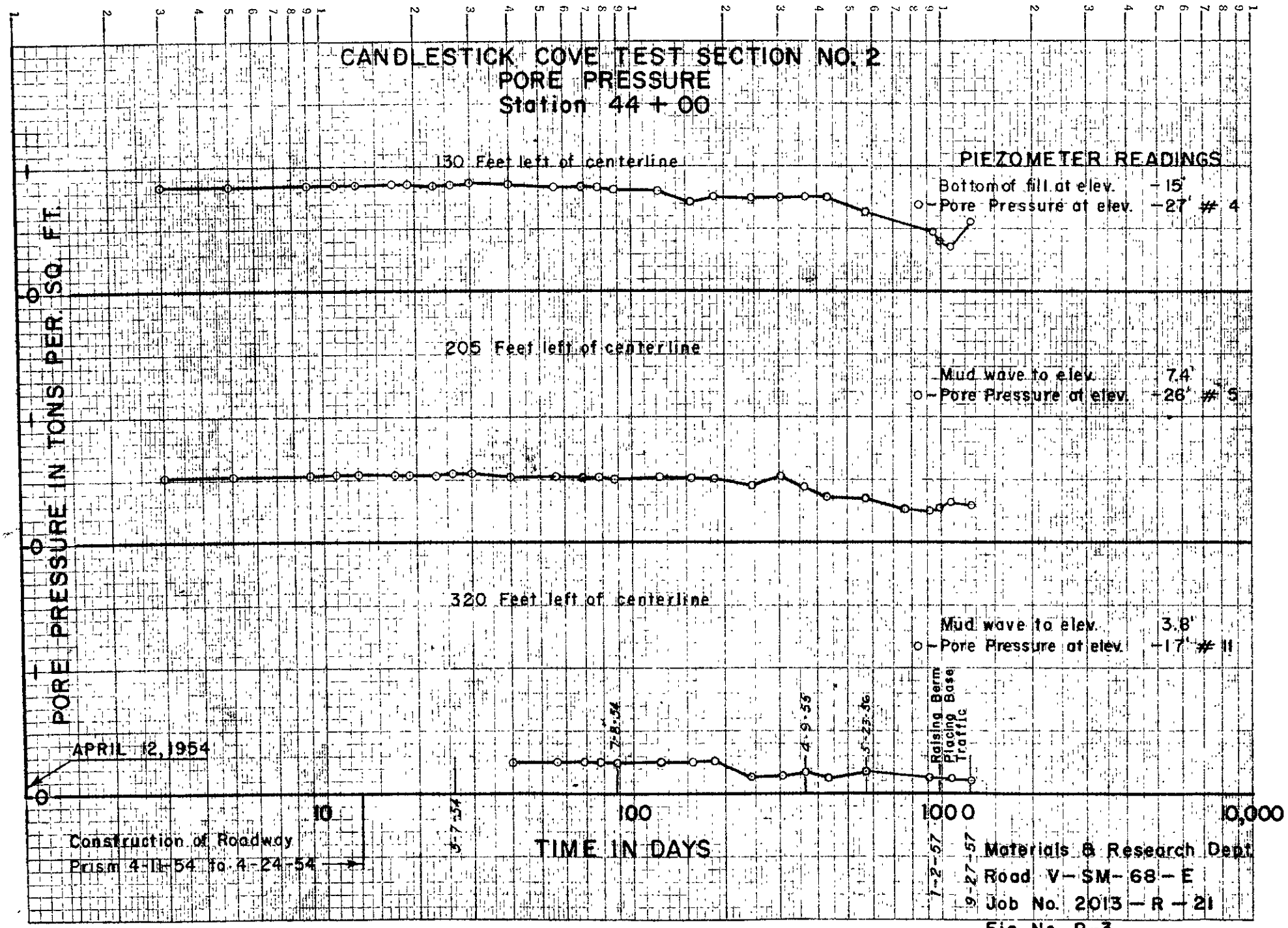
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 10 to the inch.
5th lines accented.
MADE IN U.S.A.



Materials & Research Dept.
Road V-SM-68-E
Job No. 2013-R-21

Fig. P-2

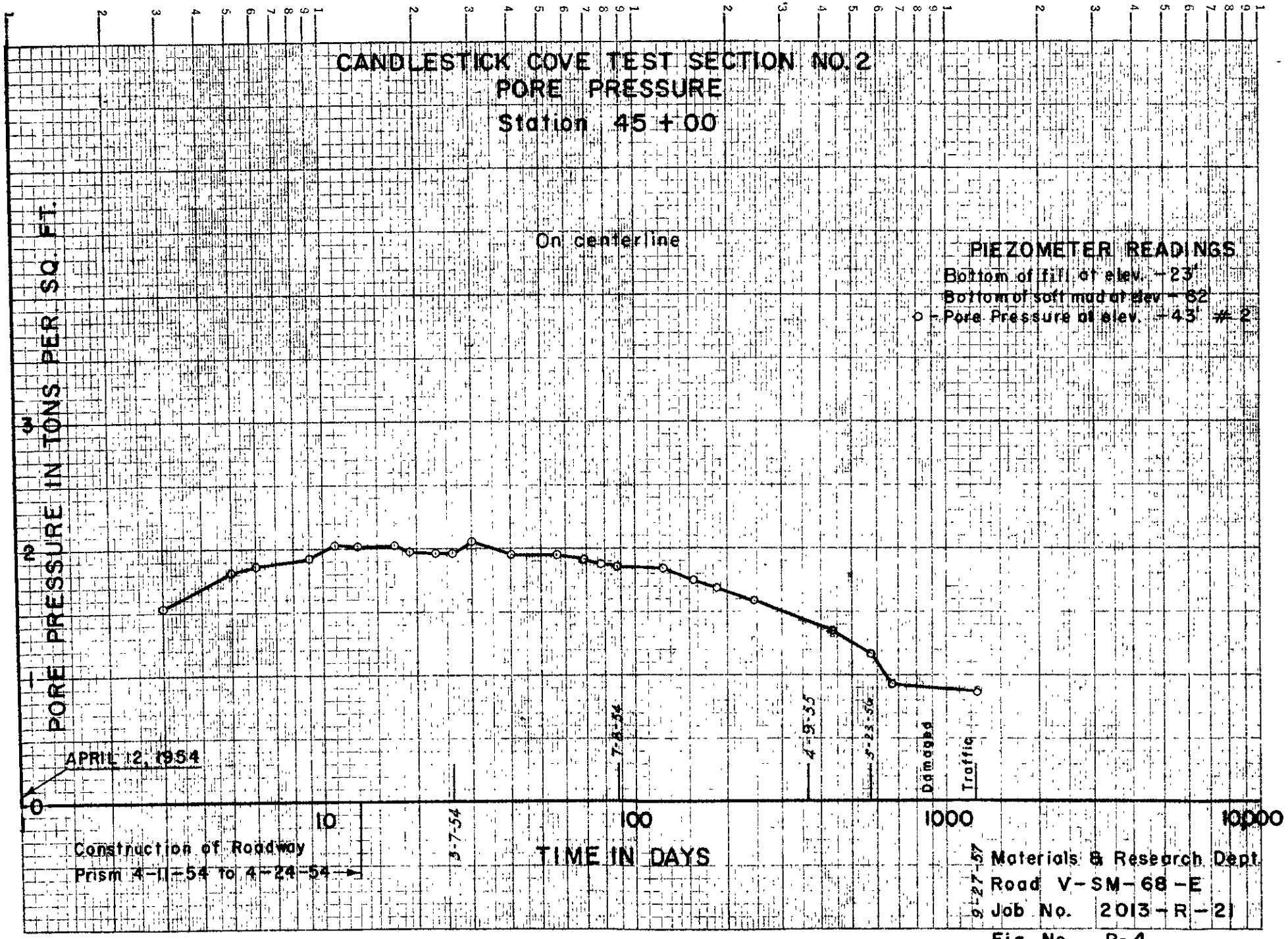
359-81 KEUFFEL & ESSER CO.
Semi Logarithmic, 4 Cycles X 10 to the inch.
5th lines accented.
MADE IN U.S.A.



Materials & Research Dept.
Road V-SM-68-E
Job No. 2013-R-21

Fig. No. P-3

359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 10 to the inch.
5th lines accented.
MADE IN U.S.A.



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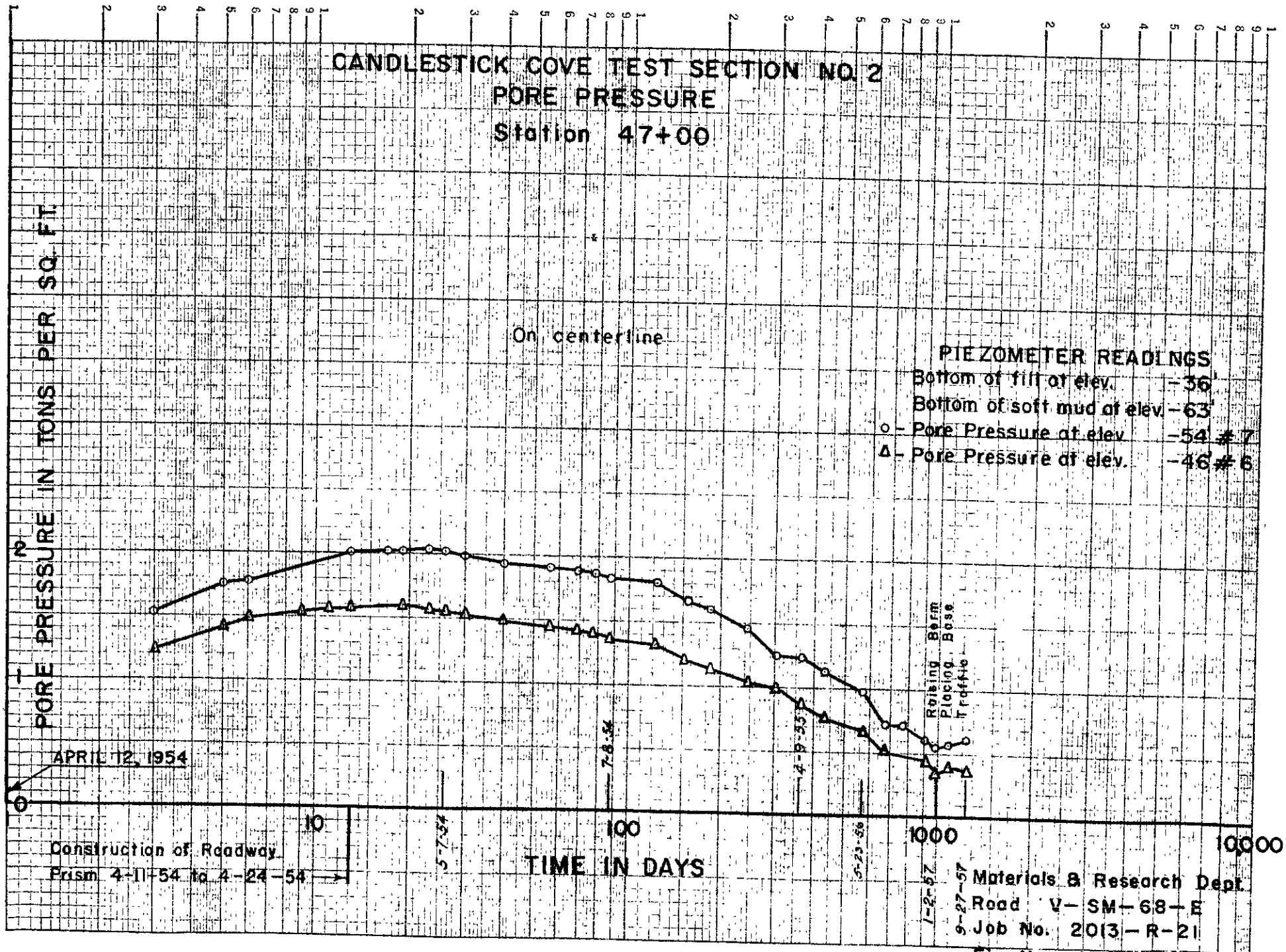
Road V-SM-68-E
Job No. 2013-R-21

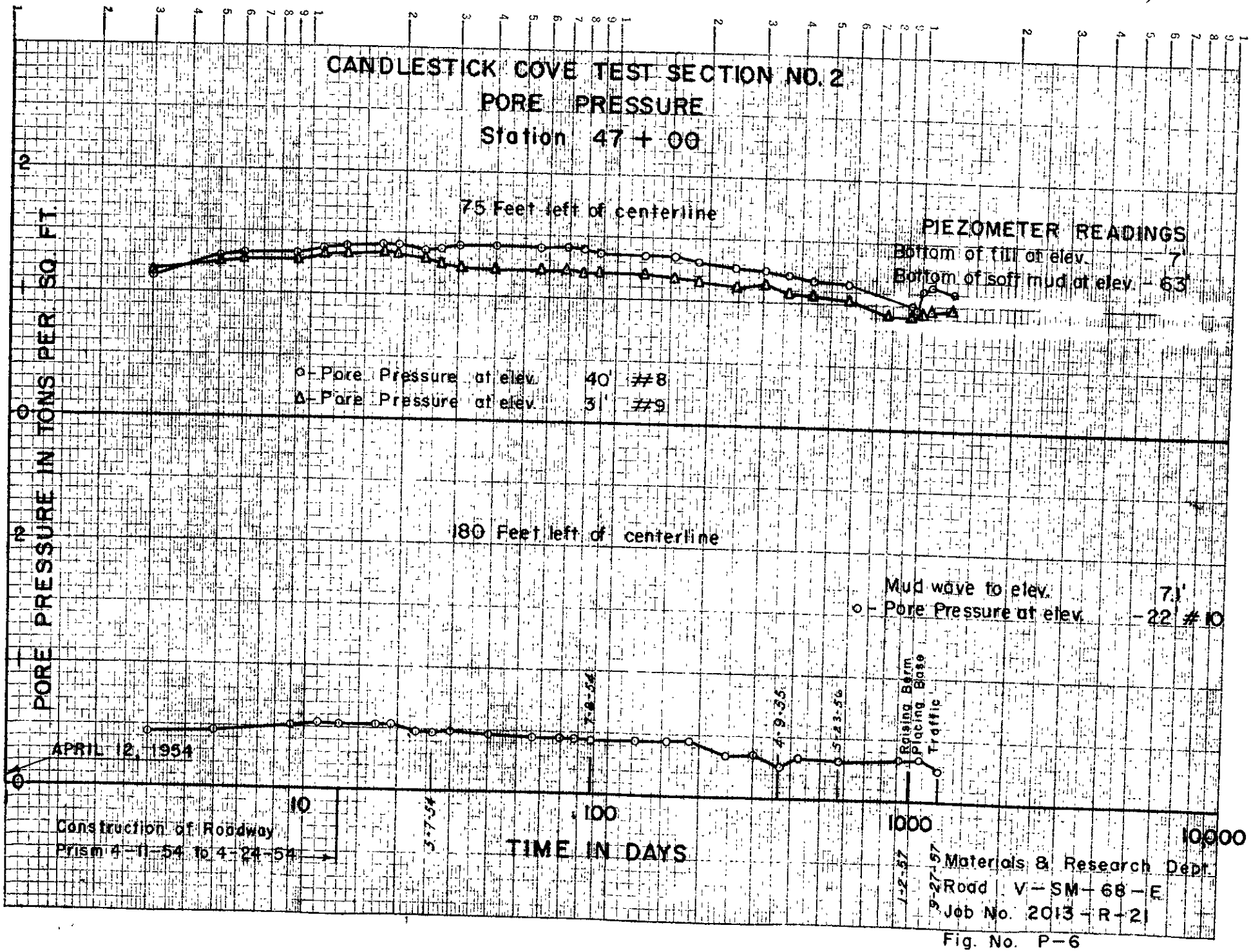
Fig. No. P-4

359-81 KEUFFEL & ESSER CO
Semi-Logarithmic, 4 Cycles X 10 to the inch.
5th lines accented.
MADE IN U.S.A.



Materials & Research Dept.
Road V-SM-68-E
Job No. 2013-R-21
Fig. No. P-5

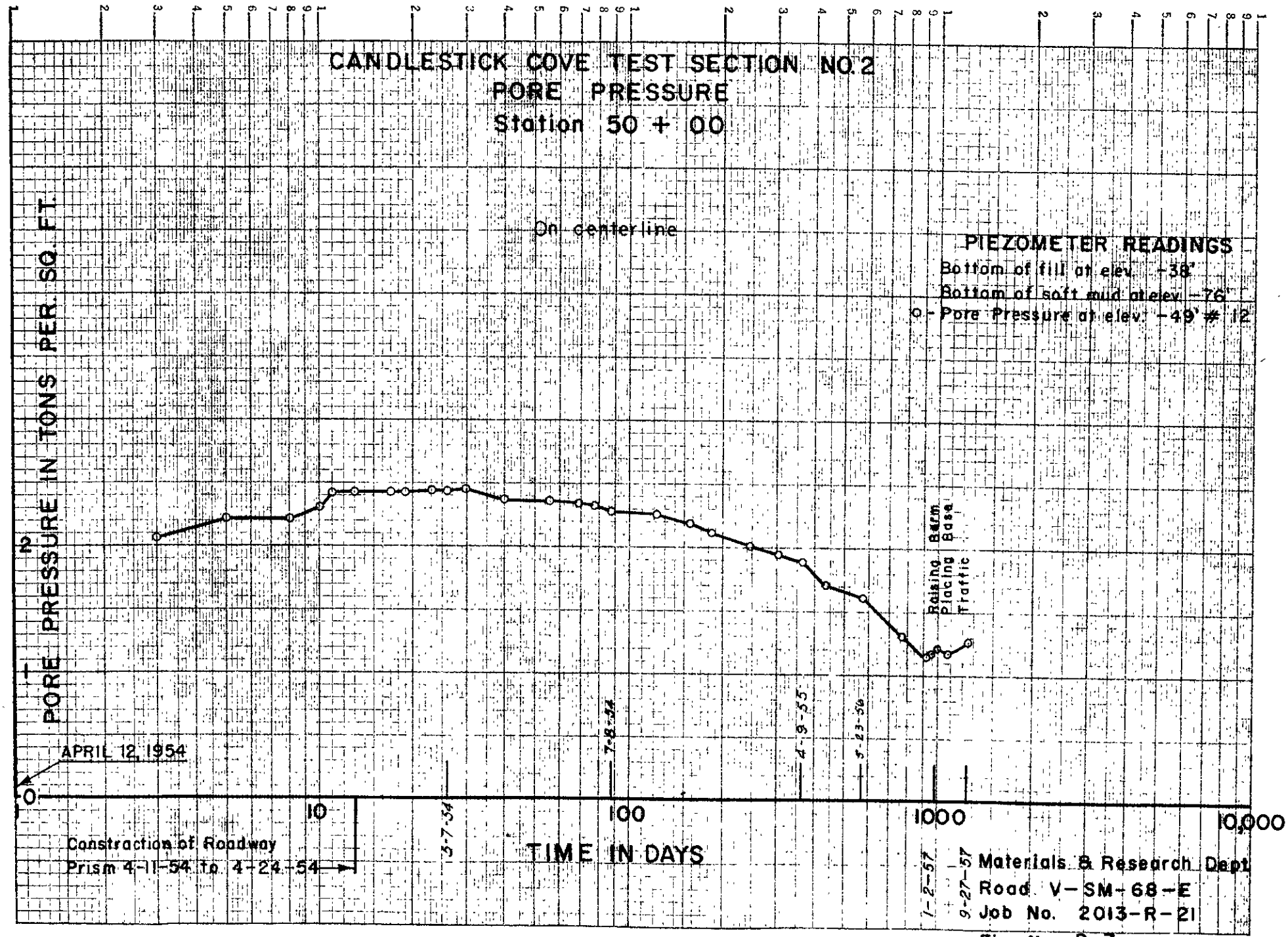
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 10 to the inch
5th lines accented.
MADE IN U.S.A.



Materials & Research Dept.
Road V-SM-68-E
Job No. 2013-R-21

Fig. No. P-6

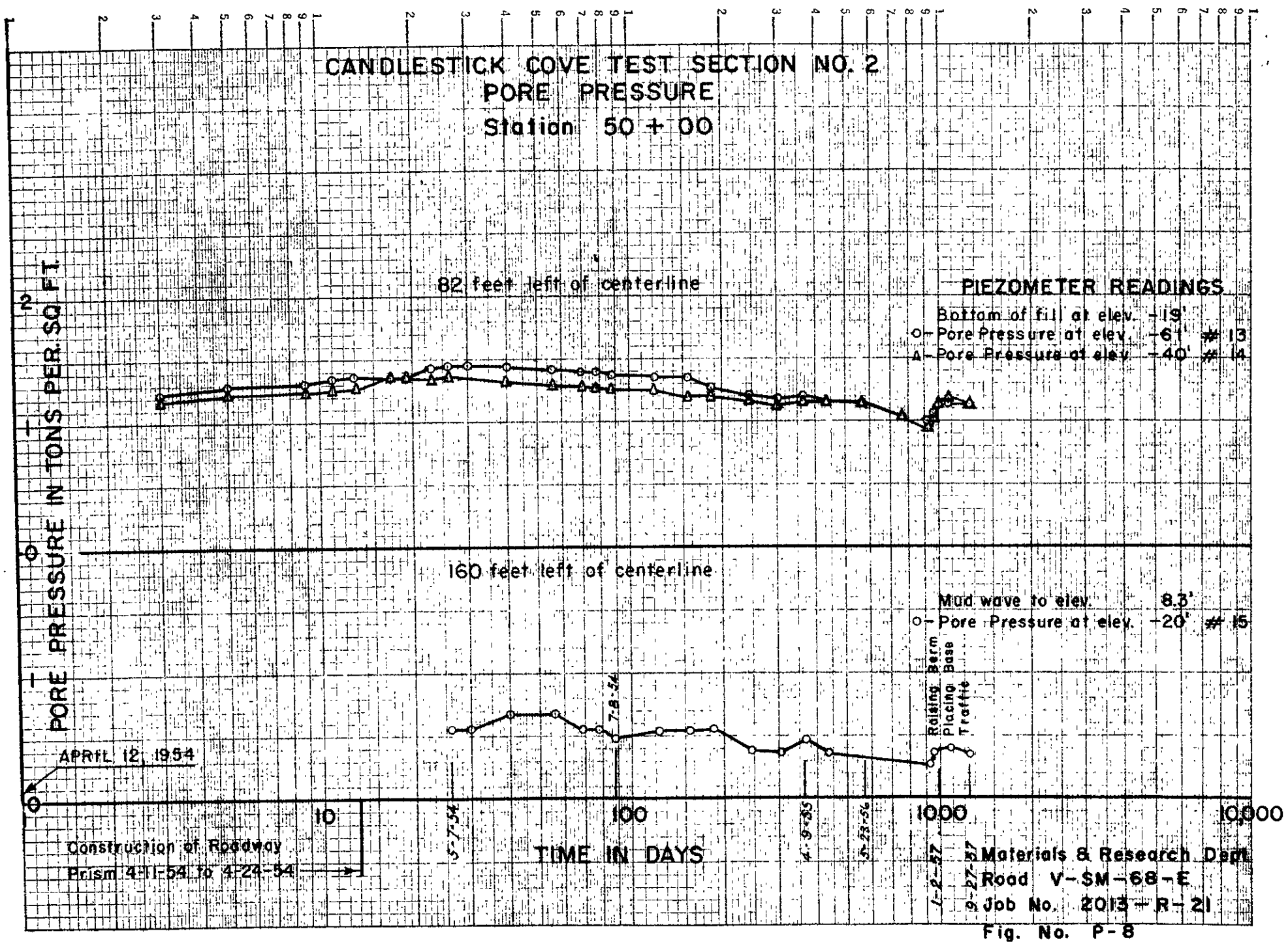
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 10 to the inch.
5th lines accented.
MADE IN U.S.A.



Materials & Research Dept
Road V-SM-68-E
Job No. 2013-R-21

Fig. No. P-7

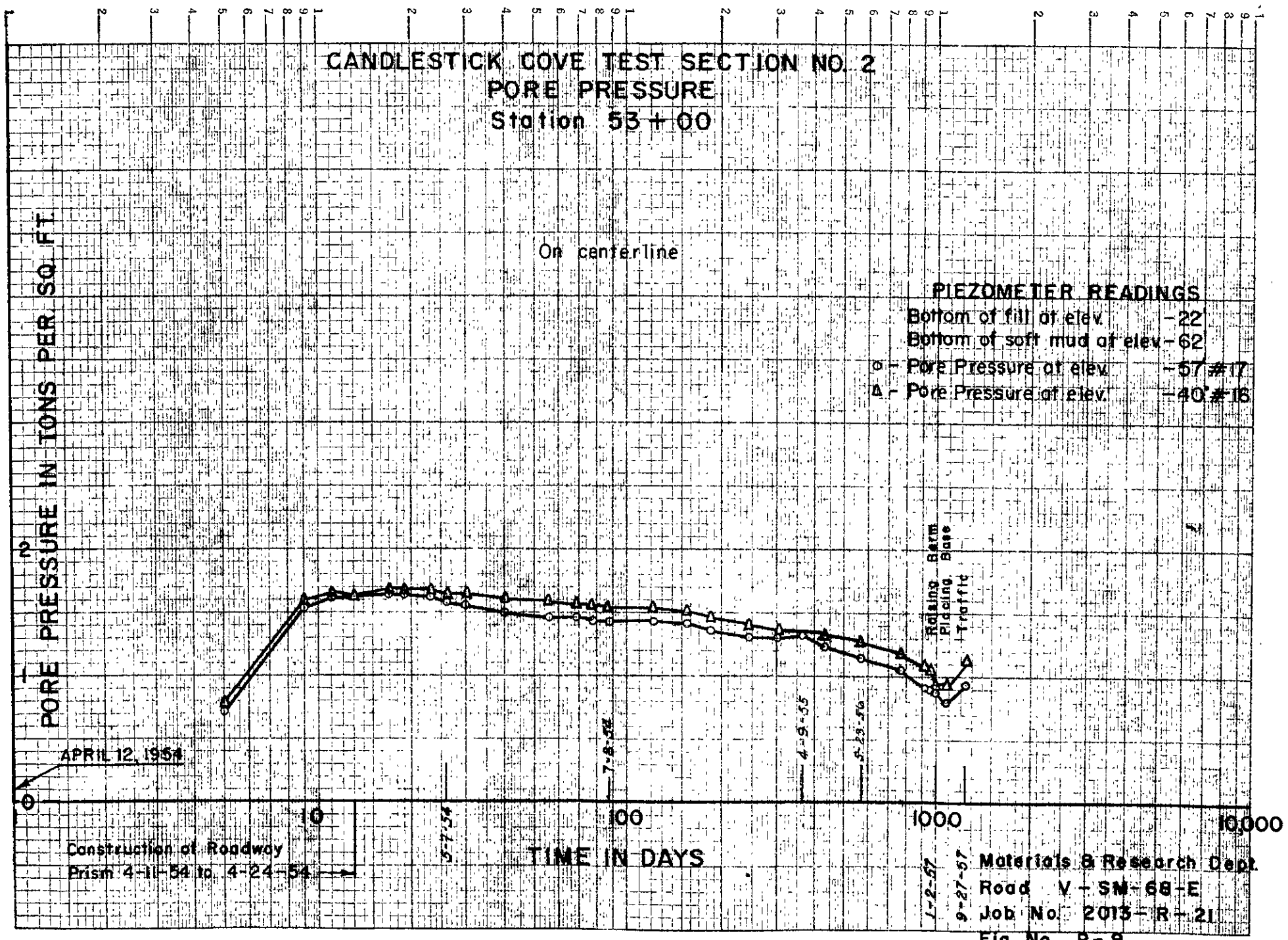
359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles × 10 to the inch.
5th lines accented.
MADE IN U.S.A.



Materials & Research Dept.
Road V-SM-68-E
Job No. 2013-R-21

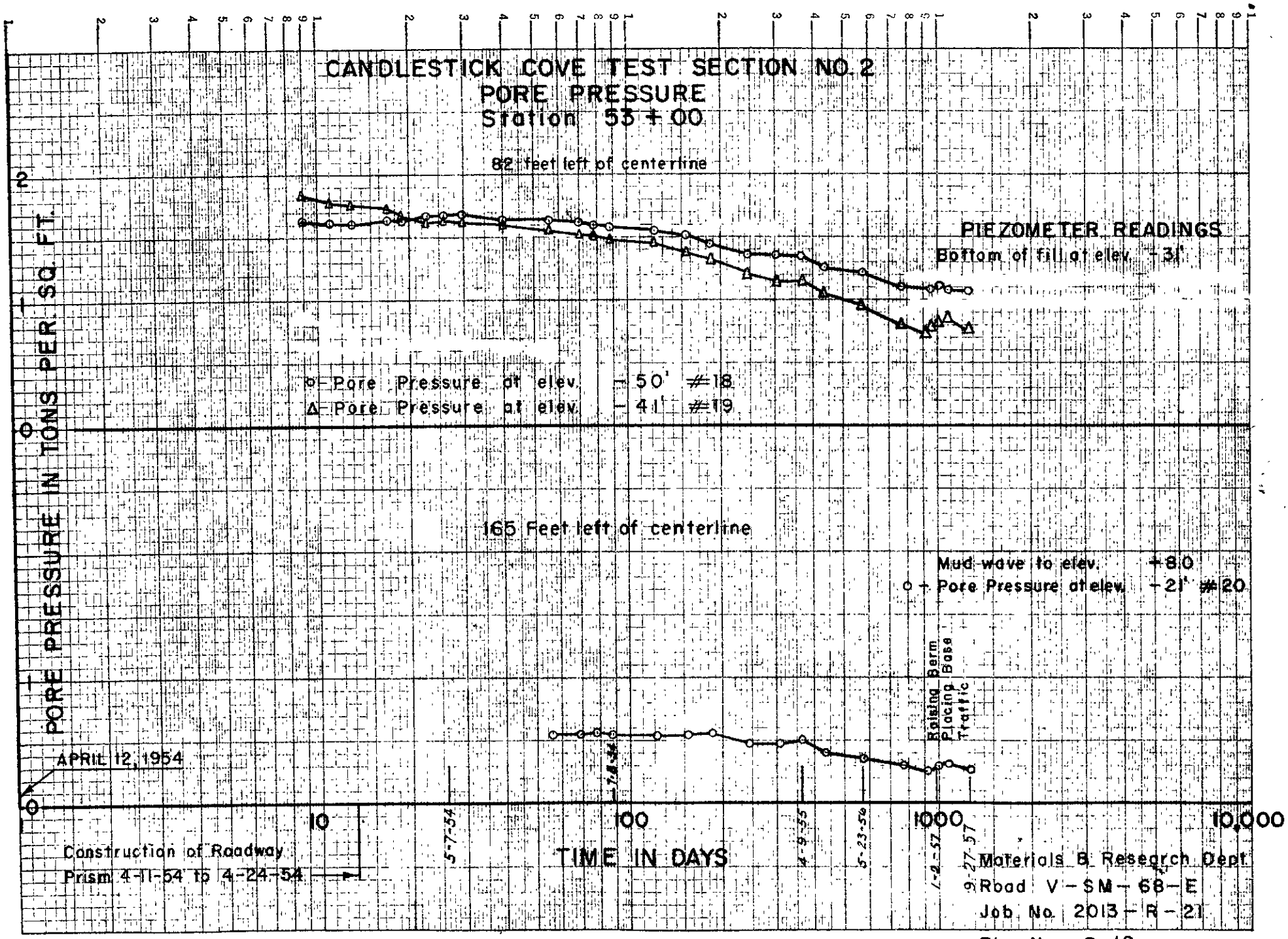
Fig. No. P-8

359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles X 10 to the inch.
5th lines accented.
MADE IN U.S.A.



Materials & Research Dept.
Road V-SM-68-E
Job No. 2013-R-21
Fig. No. P-9

359-81 KEUFFEL & ESSER CO.
Semi-Logarithmic, 4 Cycles / 10 to the inch.
5th lines accented.
MADE IN U.S.A.



Materials & Research Dept.
Road V-SM-68-E
Job No. 2013-R-21

Fig. No. P-10

SUMMARY OF TESTS ON SOIL SAMPLES

OBTAINED BY MEANS OF BORINGS

Borings P-1, P-2, P-4, P-5 made in First Unit about one year after completion of the working table.

Borings P-6, P-7, P-14, P-25, P-28 made in Second Unit within one month after completion of the working table and before placing roadway prism.

Borings D-1, D-2, D-3, D-4 made in Second Unit mud wave about two months after completion of the working table.

Borings D-5, D-6 made in the First Unit about four and one-half years after completion of the working table.

Borings D-7, D-8 made in the Second Unit about two and one-half years after completion of the working table.

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No. 505
Boring No. P1 & P2

LOCATION P1 14+03 1'LT ℄ / P2 19+50 3'RT ℄ DIST IV CO SM RTE 68 SEC E WO

DATE REPORTED

FIELD DATA			TEST RESULTS																							
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT		MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS					ATTERBERG LIMITS		CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT							DIRECT SHEAR		UNCONF COMP	REMARKS
			WET	DRY				GRAVEL	SAND	SILT	CLAY	COLL'D	LL	PI	1/8	1/4	1/2	1	2	4	8	COHESION psf	ANGLE OF INTERNAL FRICTION	COMP STRENGTH T/FT²	
P1																										
9	V	-20.2	107		48																			0.42	
	I	-22.5	104		47																			0.18	
10	IV	-29.5	94		84										0.5	2.1	15.2	10.5	19.6	28.0	34.7				
	III	-29.8	74		74																			0.32	
11	III	-34.8	100		66																			0.49	
12	IV	-39.5	101		66										0.02	0.6	1.9	5.0	12.8	22.0	29.8				
	III	-39.8	102		66																			0.66	
13	III	-44.8	100		65																			0.05	
14	II	-50.3	136		15																				
P2																									
4	III	-12.0	103		58																			0.18	
5	IV	-17.0	103		58																			0.41	
	III	-18.0	104												1.1	3.2	6.1	10.3	16.5	22.6	28.1				
6	III	-23.5	95		78																			0.23	
7	III	-28.0	98		80																			0.26	
	II	-28.3	97		82										0.9	2.8	6.3	11.8	19.4	27.8	34.6				

Borings P1 & P2 were made six months after completion of roadway prism in first unit.

SHEET NO 1 OF 23 SHEETS

FORM T-39-50
3-148-1

STATE OF CALIFORNIA
MATERIALS AND RESEARCH DEPARTMENT
DIVISION OF HIGHWAYS

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No. 505

Boring No. P2, P4 & P5

LOCATION P4 – 15+98 – 4' RT. ℄ / P5 – 11+03 – 98 LT. ℄ DIST IV CO SM RTE 68 SEC E WO

DATE REPORTED

FIELD DATA			TEST RESULTS																							
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT		MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS					ATTERBERG LIMITS		CONSOLIDATION % OF ORIGINAL HEIGHT LOAD–TONS PER SQ FT							DIRECT SHEAR		UNCONF COMP	REMARKS
			WET	DRY				GRAVEL	SAND	SILT	CLAY	COLL'D	LL	PI	1/8	1/4	1/2	1	2	4	8	COHESION psf	ANGLE OF INTERNAL FRICTION	COMP STRENGTH T/FT²	
P2																										
8	III	-32.0	101		68																			0.36	
9	III	-39.5	100		67																				
	II	-39.8	100		69										0.12	1.3	3.1	7.4	14.2	21.5	28.3			0.45	
10	III	-42.1	100		69																			0.70	
11	II	-47.2	133		18																				
P4																								0.15	
5	II	-25.0	102		60																			0.45	
6	V	-33.2	100		75										1.8	5.1	8.7	15.4	21.1	27.6	33.6				
	III	-34.0	102		67																			0.38	
	II	-34.3	102		67										1.1	8.9	11.6	16.3	23.4	31.1	36.6				
P5																									
7	II	-37.2	104		63																			0.32	
8	III	-37.0	104		65																				
	II	-37.3	104		63										0.12	1.6	3.8	8.1	14.8	22.7	30.0			0.36	
9	III	-39.8	115		39																				
10	IV	-45.0	115		36																			0.20	
																								0.04	

Borings made six months after completion of roadway prism in first unit.

SHEET NO 2 OF 23 SHEETS

FORM T-39-50
5-148-1

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No 2013-R-21
Boring No P6 & P7

LOCATION P6 - 45+07 - 1' LT. ℄ / P7 - 45+02 1' LT ℄ DIST IV CO SM RTE 68 SEC E WO

DATE REPORTED

FIELD DATA			TEST RESULTS																						
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT WET	UNIT WEIGHT LBS PER CU FT DRY	MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS GRAVEL	SAND	SILT	CLAY	COLL'D	ATTERBERG LIMITS LL	PI	CONSOLIDATION % OF ORIGINAL HEIGHT LOAD-TONS PER SQ FT 1/8	1/4	1/2	1	2	4	8	Triaxial SHEAR σ_3	$\sigma_1-\sigma_3$	UNCONF COMP COMP STRENGTH T/FT²	REMARKS Type of Test
																						Kgs. per sq. cm			
P6																									
1	I	-51	97	60	62																			0.18	
P7																									
1	VI	-35½	98	60	63																	2.0	0.20		"Q" undrained
	V	-35⅝	99	62	60																	1.0	0.19		"Q" undrained
	IV	-36⅛	99	62	60																	0.5	0.22		"Q" undrained
	III	-36⅜	99	62	62																			0.19	
	II	-36½	100		49																	2.0	1.42		"Q" drained
	I	-36⅞	100		59										1.1	2.5	6.4	11.3	18.7	25.5	31.4	0.5	0.47		"Q" drained
	A	38½	98	59	65																			0.20	
2	VI	-44½	100	60	66																	2.0	0.25		"Q" undrained
	V		100	60	66																	1.0	0.22		"Q" undrained
	IV		100	61	65																	0.5	0.26		"Q" undrained
	III		100	62	64																				
	II		97	60	61																	2.0	0.43		"Q" drained
	I		98	59	65										0.6	1.7	4.3	9.4	16.1	20.9	27.1	0.5	0.30		"Q" drained
	A	-46½	100	61	65																			0.24	

Borings P6 & P7 made within one month after working table had become stable and before placing of roadway prism. Second unit.

SHEET NO 3 OF 23 SHEETS

FORM T-39-50
5-148-1

STATE OF CALIFORNIA DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No. 2013-R-21
Boring No. P7

LOCATION 45+02 · 1' LT. ℄ E DIST. IV CO. SM RTE. 68 SEC. E WO. DATE REPORTED

FIELD DATA			TEST RESULTS																							
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT		MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS					ATTERBERG LIMITS		CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT							TRIAXIAL SHEAR		UNCONF COMP	REMARKS
			WET	DRY				GRAVEL	SAND	SILT	CLAY	COLOR	LL	PI	1/8	1/4	1/2	1	2	4	8	σ_3	$\sigma_1-\sigma_3$	COMP STRENGTH T/FT²	Type of Test
																						Kgs. per sq. cm.			
3	VI	-54.5	99		51																	2.0	1.32		
	V		98		53																	1.0	0.33		"Q" Undrained
	IV		96		70	2.82									0.1	0.6	2.9	7.9	15.2	20.9	28.0	0.5	0.54		"Q" Undrained
	III		95	54	76																			0.37	
	II		97	55	75																	2.0	0.42		"Q" Undrained.
	I	-56.5	97	57	72																	0.5	0.38		"Q" Undrained
	A		95	55	73																			0.224	
4	VI	-62.5	97		15																	2.0	1.03		"Q" Undrained
	V		120		16																			0.44	
	IV		122		13																	0.5	3.4		"Q" Drained
	III		137			2.70									0.7	1.1	1.9	3.2	4.9	7.6	9.3	1.0	1.14		"Q" Undrained
	II		134						47	34	19	7										1.0	3.03		"Q" Drained
	I	-69.5	127																			0.5	0.11		"Q" Undrained

Boring made within one month after working table had become stable and before placing of roadway prism. Second unit.

SHEET NO 4 OF 23 SHEETS

FORM T-39-50
5-148-1

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No. 2013-R-21
Boring No. P14

LOCATION 46+90 7' Lt ℄ DIST. IV CO. SM RTE. 68 SEC. E WO. DATE REPORTED

FIELD DATA			TEST RESULTS																						
SAMPLE NO	TUBE NO	DEPTH	UNIT WEIGHT LBS PER CU FT WET	UNIT WEIGHT LBS PER CU FT DRY	MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS GRAVEL	SAND	SILT	CLAY	COLOR	ATTERBERG LIMITS LL	PI	CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT 1/8	1/4	1/2	1	2	4	8	DIRECT SHEAR COHESION psf	ANGLE OF INTERNAL FRICTION	UNCONF COMP COMP STRENGTH T/FT²	REMARKS
1	V	-44½	99																						
	IV	-44½	99	62																				0.14	
	III	-45½	100																						
	II	46¾	99	62											5.2	8.5	12.4	—	23.8	28.9	34.0				
	I	-48½	99	62																				0.12	
2	VI	-54½	90																						
	V	-54½	89		76																			0.16	
	IV	-55½	91																						
	III	-55¾	94																						
	II	56½	95		72										4.0	6.6	10.3	—	23.3	29.4	35.8				
	I	-56½	97		67																				
3	IV	-64½	138																						
	III	-64½	135																						
	II	-65½	133																						
	I	-66	137		16																			2.02	

Boring made within one month after The working Table had become stable, and before placing of roadway prism. Second Unit.

SHEET NO 5 OF 23 SHEETS

FORM T-39-30
S-148-1

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No. 2013-R-21
Boring No. P 25

LOCATION 49+00 77' Lt. & E DIST. IV CO. SM RTE. 68 SEC. E WO. DATE REPORTED

FIELD DATA			TEST RESULTS																						
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT WET	UNIT WEIGHT LBS PER CU FT DRY	MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS GRAVEL	SAND	SILT	CLAY	COLOR	ATTERBERG LIMITS LL	PI	CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT ⅛	¼	½	1	2	4	8	DIRECT SHEAR COHESION psf	ANGLE OF INTERNAL FRICTION	UNCONF COMP COMP STRENGTH T/FT²	REMARKS
1	V	-15.5	100																						
	IV		98		67																			0.176	
	III		100																						
	II		100		66										3.0	4.3	7.4	13.3	19.7	26.2	32.1				
	I	-18	100		64																			0.29	
2	VI	-21	99																						
	V		99		64																			0.215	
	IV		97																						
	III		98																						
	II		97		66										5.3	8.7	13.0	18.2	24.1	29.8	35.4				
	I	-23	98		66																			0.127	
3	VI	-26	99																						
	V		99		62																			0.137	
	IV		99																						
	III		101																						
	II		99		65										2.3	3.7	7.0	13.0	19.0	24.8	31.0				
	I	-28	98		63																			0.23	
4																									
	VII	-35.5	98		69																			0.17	
	V		100		63																				
	II		99		66										1.6	3.5	7.4	13.5	19.3	25.6	31.8				
	I	-38.5	99		67																			0.22	

Boring made with in one month after working Table had become stable, and before placing of The roadway prism. Second Unit.

SHEET NO 6 OF 23 SHEETS

FORM T-39-30
S-148-1

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No. 2013-R-21
Boring No. P28

LOCATION 51+05 ON ℄ DIST. IV CO. SM RTE 68 SEC. E WO. DATE REPORTED

FIELD DATA			TEST RESULTS																						
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT WET	UNIT WEIGHT LBS PER CU FT DRY	MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS GRAVEL	GRAIN SIZE ANALYSIS SAND	GRAIN SIZE ANALYSIS SILT	GRAIN SIZE ANALYSIS CLAY	GRAIN SIZE ANALYSIS COLOR	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT 1/8	1/4	1/2	1	2	4	8	DIRECT SHEAR COHESION psf	DIRECT SHEAR ANGLE OF INTERNAL FRICTION	UNCONF COMP COMP STRENGTH T/FT²	REMARKS
I	III	-25	97																						
	II		98		62																			0.204	
	I	-28	95		70										2.5	4.9	9.9	16.9	23.1	29.8	35.5				
2	VII	-43	98																						
	VI		102																						
	V		100																						
	IV		99																						
	III		99		65										1.7	3.6	6.7	12.8	19.6	26.2	32.8				
	II		100		62																			0.328	
	I	-46	98																						
3	IX	-53	97																						
	VIII		96																						
	VII		97																						
	VI		99																						
	V		101												2.5	4.6	7.8	11.7	19.2	26.5	32.5				
	IV		100		60																			0.328	
	III		102		51																				
	II		103																						
	I	-56	99																						

Boring was made within one month after working Table had become Stable, and before placing of the roadway prism. Second Unit.

SHEET NO 7 OF 23 SHEETS

FORM T-39-50
5-148-1

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No. 2013-R-21
Boring No. P28

LOCATION 51+05 on ℄ DIST. IV CO. SM RTE. 68 SEC. E WO. DATE REPORTED

FIELD DATA			TEST RESULTS																							
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT WET	UNIT WEIGHT LBS PER CU FT DRY	MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS GRAVEL	SAND	SILT	CLAY	COLL'D	ATTERBERG LIMITS LL	PI	CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT 1/8	1/4	1/2	1	2	4	8	DIRECT SHEAR COHESION psf	ANGLE OF INTERNAL FRICTION	UNCONF COMP COMP STRENGTH T/FT²	REMARKS
4	IV	-63	94												1.5	2.4	4.3	8.1	15.2	25.1	33.8				
	III		90		81																				
	II		89		82																			0.523	
	I	-66																							
5	VII	-73	94																						
	VI		96																						
	V		91																						
	IV		91		76										3.8	7.1	10.8	15.1	21.1	28.3	35.1				
	III		91		76																			0.297	
	II		94																						
	I	-76	96																						
6	V	-78	126																						
	IV		127																						
	III		128		18																			0.354	
	II		118																						
	I	-81	121																						

Boring was made within one month after working Table had become Stable, and before placing of roadway prism. Second Unit.

SHEET NO 8 OF 23 SHEETS

FORM T-39-50
5-148-1

STATE OF CALIFORNIA DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No. 2013-R-21
Boring No. D1

LOCATION 43+85 162 Rt. ℄ DIST. IV CO. S.M. RTE. 68 SEC. E WO. DATE REPORTED

FIELD DATA			TEST RESULTS																							
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT		MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS					ATTERBERG LIMITS		CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT							DIRECT SHEAR		UNCONF COMP	REMARKS
			WET	DRY				GRAVEL	SAND	SILT	CLAY	COLL'D	LL	PI	1/8	1/4	1/2	1	2	4	8	COHESION psf	ANGLE OF INTERNAL FRICTION	COMP STRENGTH T/FT²	
1	I	+6	99		74																			0.149	
2	III	+1	97																						
	II		97																						
	I	-2	97		76																			0.17	
3	III	+4	96																						
	II		100		67																			0.21	
	I	-7	100		67																			0.15	
4	IV	-9	96		84																			0.045	
	III		97																						
	II		97																						
	I	-12	101		67																			0.14	
5	III	-14	100																						
	II		97		76																			0.21	
	I	-17	98		74																			0.16	
6	I	-19	94		80																			0.155	
7	I	-24	99		67																			0.13	

Boring made in The mud wave about two months after The working Table had become stable. In Second Unit.

SHEET NO 9 OF 23 SHEETS

FORM T-39-50
5-148-1

STATE OF CALIFORNIA DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No 2013-R-21
Boring No D1

LOCATION 43+85 162' Rt. ℄ DIST IV CO SM RTE 68 SEC E WO ..... DATE REPORTED .....

FIELD DATA			TEST RESULTS																							
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT		MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS					ATTERBERG LIMITS		CONSOLIDATION % OF ORIGINAL HEIGHT							DIRECT SHEAR		UNCONF COMP	REMARKS
			WET	DRY				GRAVEL	SAND	SILT	CLAY	COLL'D	LL	PI	LOAD—TONS PER SQ FT							COHESION psf	ANGLE OF INTERNAL FRICTION	COMP STRENGTH T/FT²	
															1/8	1/4	1/2	1	2	4	8				
8	VI	-29	94		86																			0.32	
	V		95																						
	IV		95																						
	III		95																						
	II		95																						
	I	-32	95		81																			0.35	
9	V	-34	97		75																			0.31	
	IV		95																						
	III		99																						
	II		100																						
	I	-37	101		65																			0.31	

See sheet #9

SHEET NO 10 OF 23 SHEETS

FORM T-39-50
5-148-1

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No 2013-R-21
Boring No D2

LOCATION 44+15 180' LT. E DIST IV CO SM RTE 68 SEC E WO ..... DATE REPORTED .....

FIELD DATA			TEST RESULTS																						
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT		MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS					ATTERBERG LIMITS		CONSOLIDATION % OF ORIGINAL HEIGHT							DIRECT SHEAR		UNCONF COMP	REMARKS
			WET	DRY				GRAVEL	SAND	SILT	CLAY	COLL'D	LL	PI	LOAD—TONS PER SQ FT							COHESION psf	ANGLE OF INTERNAL FRICTION	COMP STRENGTH T/FT²	
															1/8	1/4	1/2	1	2	4	8				
1	I	+4	93		86																			0.05	
3	VI	-8	94		87																				
	V		94																					0.08	
	IV		96																						
	III		98																						
	II		98																						
	I	-11	99		72																			0.13	
4	III	-13	100																						
	II		101																						
	I	-16	101		64																			0.21	
5	IV	-18	91																						
	III		95																						
	II		98																						
	I	-21	97		81																			0.22	
6	VI	-23	99		68																			0.14	
	V		99																						
	IV		100																						
	III		99																						
	II		99																						
	I	-26	95		87																			0.22	

Boring made in The mud wave about two months after the working Table had become stable. In second Unit.

SHEET NO 11 OF 23 SHEETS

FORM T-39-50
5-148-1

STATE OF CALIFORNIA DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No. 2013-R-21
Boring No. D 2

LOCATION 44+15 180' LT E DIST IV CO SM RTE 68 SEC E WO ..... DATE REPORTED .....

FIELD DATA			TEST RESULTS																						
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT WET	UNIT WEIGHT LBS PER CU FT DRY	MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS GRAVEL	SAND	SILT	CLAY	COLL'D	ATTERBERG LIMITS LL	PI	CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT 1/8	1/4	1/2	1	2	4	8	DIRECT SHEAR COHESION psf	ANGLE OF INTERNAL FRICTION	UNCONF COMP COMP STRENGTH T/FT²	REMARKS
7	VII	-28	97		76																			0.15	
	VI		99																						
	V		97																						
	IV		97																						
	III		95																						
	II		96																						
	I	-31	95		87																			0.31	
8	VII	-33	94		74																				
	VI		97		70																			0.29	
	V		99																						
	IV		97																						
	III		98																						
	II		98																						
	I	-36	99		68																			0.29	

See Sheet #11

SHEET NO 12 OF 23 SHEETS

FORM T-39-50
5-148-1

STATE OF CALIFORNIA DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No. 2013-R-21
Boring No. D3

LOCATION 47+00 176' Lt. E DIST IV CO SM RTE 68 SEC E WO ..... DATE REPORTED .....

FIELD DATA			TEST RESULTS																							
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT		MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS					ATTERBERG LIMITS		CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT							DIRECT SHEAR		UNCONF COMP	REMARKS
			WET	DRY				GRAVEL	SAND	SILT	CLAY	COLL'D	LL	PI	1/8	1/4	1/2	1	2	4	8	COHESION psf	ANGLE OF INTERNAL FRICTION	COMP STRENGTH T/FT²	
1	II	+6	96																							
	I	+4½	97		74																			0.11	
2	II	+1	96																							
	I	-3	96		74																			0.16	
3	V	-4	96		80																			0.12	
	IV		96																						
	III		99																						
	II		100																						
	I	-6½	101		69																			0.12	
4	V	-9	98		71																			0.08	
	IV		99																						
	III		96																						
	II		99																						
	I	-12	102																					0.72	
5	V	-14	98		72																			0.13	
	IV		96																						
	III		96																						
	II		98																						
	I	-17	97		73																			0.15	

Boring made in the mud wave about one month after the working table had become stable. In second unit.

SHEET NO 13 OF 23 SHEETS

FORM T-39-50
5-168-1

STATE OF CALIFORNIA DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No. 2013-R-21
Boring No. D3

LOCATION 47+00 176' Lt. E DIST III CO SM RTE 68 SEC E WO ..... DATE REPORTED .....

FIELD DATA			TEST RESULTS																						
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT		MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS					ATTERBERG LIMITS		CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT							DIRECT SHEAR		UNCONF COMP	REMARKS
			WET	DRY				GRAVEL	SAND	SILT	CLAY	COLL'D	LL	PI	1/8	1/4	1/2	1	2	4	8	COHESION psf	ANGLE OF INTERNAL FRICTION	COMP STRENGTH T/FT²	
6	II	-19	99																						
	I	-22	98		72																			0.17	
7	V	-24	98		72																			0.28	
	IV		96																						
	III		96																						
	II		98																						
	I	-27	97		73																			0.27	

See Sheet #13

SHEET NO 14 OF 23 SHEETS

FORM T-39-50
5-148-1

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No. 2013-R-21
Boring No. D4

LOCATION 47+25 200' RT. "E" DIST IV CO. S.M RTE 68 SEC. E WO ..... DATE REPORTED .....

FIELD DATA			TEST RESULTS																						
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT		MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS					ATTERBERG LIMITS		CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT							DIRECT SHEAR		UNCONF COMP	REMARKS
			WET	DRY				GRAVEL	SAND	SILT	CLAY	COLL'D	LL	PI	1/8	1/4	1/2	1	2	4	8	COHESION psf	ANGLE OF INTERNAL FRICTION	COMP STRENGTH T/FT²	
4	V		97		76																			0.14	
	IV		97																						
	III		102																						
	II		98																						
	I		98		69																			0.18	
5	V		95		77																			0.15	
	IV		95																						
	III		95																						
	II		94																						
	I		95		80																			0.13	
6	VI		101		62																			0.20	
	V		98																						
	IV		98																						
	III		98																						
	II		98																						
	I		98		67																			0.23	
7	III		96																						
	II		97																						
	I		97		76																			0.21	

Boring made in mud wave about one month after the working Table had become stable. In second Unit.

SHEET NO 15 OF 23 SHEETS

FORM T-39-50
5-148-1

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No 2013-R-21
Boring No. D5

LOCATION 14+00 on ℄ DIST. IV CO SM RTE 68 SEC. E WO. DATE REPORTED

FIELD DATA			TEST RESULTS																							
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT WET	UNIT WEIGHT LBS PER CU FT DRY	MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS GRAVEL	SAND	SILT	CLAY	COLL'D	ATTERBERG LIMITS LL	PI	CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT 1/8	1/4	1/2	1	2	4	8	DIRECT SHEAR COHESION psf	ANGLE OF INTERNAL FRICTION	UNCONF COMP COMP STRENGTH T/FT²	REMARKS
1	V	-39	103		60								64	31										0.77	
	IV		103		59			0	0	33	67	40												0.86	
	III		103		62	2.74									-0.1	—	+0.7	2.5	7.0	15.2	23.1				
	II		104		58								67	34										0.89	
	I		104		59			0	0	36	64	37												0.89	
2	VI	-44	128		22								21	3										0.49	
	V		132		21			0	79	11	10	6												0.40	
	IV		109		47								44	20											
	III		107		49	2.74									1.9	5.5	11.9	17.2	21.9	26.2	29.8				
	II		103		57			0	11	36	53	31													
	I		103										56	26										0.08	
3	III	-48	137																						
	II		141		13			0	90	8	2	1													
	I		144																						
4	II	-52	130		21			2	86	8	4	2													
	I		132																						
5	III	-57	132																						
	II		139		18			1	92	4	3	1													
	I		134																						

Boring made about four and one half years after completion of The Working Table in The first unit.

SHEET NO 16 OF 23 SHEETS

FORM T-39-50
S-148-1

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No 2013-R-21
Boring No D5 & D6

LOCATION D5-14+00 on ℄ D6-19+50 on ℄ DIST IV CO SM RTE 68 SEC E WO ..... DATE REPORTED .....

FIELD DATA			TEST RESULTS																							
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT WET	UNIT WEIGHT LBS PER CU FT DRY	MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS GRAVEL	SAND	SILT	CLAY	COLL'D	ATTERBERG LIMITS LL	PI	CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT 1/8	1/4	1/2	1	2	4	8	DIRECT SHEAR COHESION psf	ANGLE OF INTERNAL FRICTION	UNCONF COMP COMP STRENGTH T/FT²	REMARKS
6	III	-62	134																							
	II		132		20			4	92	3	1	1														
	I		133																							
D6																										
1	II	-21	104		50			Shell	15	59	41	21												0.12		
	I		112		45																					
2	V	-26	99		[illegible]																					
	IV		101		66								65	33										0.39		
	III		98		70			0	2	39	59	35												0.45		
	II		98		72	2.71									0.2	0.7	2.2	5.3	13.4	21.8	28.5					
	I		98		70								75	40										0.51		
3	V	-31	99		69			0	0	37	63	37												0.23		
	IV		99		68								75	47										0.28		
	III		99		69	2.76									0.3	0.9	2.5	6.4	14.5	21.9	29.2					
	II		98		70			0	0	38	62	38												0.48		
	I		98		69								72	38										0.50		

Boring made about four and one half years after Completion of The working Table in The first Unit.

SHEET NO 17 OF 23 SHEETS

FORM T-39-50
5-148-1

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No. 2013-R-21
Boring No. D6

LOCATION 19+50 on ℄ DIST IV CO SM RTE 68 SEC E WO ____ DATE REPORTED ____

FIELD DATA			TEST RESULTS																							
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT WET	UNIT WEIGHT LBS PER CU FT DRY	MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS GRAVEL	SAND	SILT	CLAY	COLL'D	ATTERBERG LIMITS LL	PI	CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT 1/8	1/4	1/2	1	2	4	8	DIRECT SHEAR COHESION psf	ANGLE OF INTERNAL FRICTION	UNCONF COMP COMP STRENGTH T/FT²	REMARKS
4	V	-36	102		61			0	0	44	56	33												0.45	
	IV		103		61								67	35										0.64	
	III		101		63	2.78									0.4	0.6	1.6	5.0	10.6	18.1	25.7				
	II		103		61			0	0	41	59	35												0.63	
	I		104		58								67	36										0.53	
5	IV	-41	102		59			0	0	36	64	36												0.43	
	III		102		57								64	30										0.75	
	II		103		60	2.72									←	0.1	0.6	2.6	6.8	14.8	23.0				
	I		102		62			0	0	34	66	40												0.76	
6	B	-46			76			Shells		49	51	31	63	31											
	A				27			0	74	13	13	7													
7	B	-51			25			0	93	4	3	1													
	A				19			0	94	4	2	1													
	I																								
8	B	-55			16			1	97	1	1	0													

See Sheet #17

SHEET NO 18 OF 23 SHEETS

FORM T-39-30
5-148-1

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No 2013-R-21
Boring No. D 7

LOCATION 45+00 on ℄ DIST IV CO SM RTE 68 SEC E WO ____ DATE REPORTED ____

FIELD DATA			TEST RESULTS																						
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT WET	UNIT WEIGHT LBS PER CU FT DRY	MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS GRAVEL	SAND	SILT	CLAY	COL'D	ATTERBERG LIMITS LL	PI	CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT 1/8	1/4	1/2	1	2	4	8	DIRECT SHEAR COHESION psf	ANGLE OF INTERNAL FRICTION	UNCONF COMP COMP STRENGTH T/FT²	REMARKS
1	VI	-31	99																						
	V		99		69			0	0	38	62	37												0.42	
	IV		99		66								71	40										0.43	
	III		96		69										0.3	1.1	3.0	9.6	16.3	23.9	31.0				
	II		99		65			0	0	41	59	34												0.40	
	I		100		65								73	41										0.42	
2	V	-36	102		60			0	2	47	51	30												0.44	
	IV		100		64								62	32										0.39	
	III		100		64	2.71									0.3	1.0	2.9	7.5	14.5	22.5	28.8				
	II		101		62			0	0	45	56	31												0.43	
	I		101		58								59	29										0.40	
3	VI	-41	99																						
	V		99		62			0	0	47	53	32												0.38	
	IV		100		62								53	23										0.40	
	III		101		63	2.73									0.6	1.5	3.5	7.1	13.9	21.0	27.6				
	II		101		62			0	0	43	57	35													
	I		100		63								65	36											
4	V	-49	103		59			0	2	40	58	35												0.51	
	IV		101		62								67	38											
	III		100		65										0.5	0.9	2.4	6.0	12.8	20.8	28.0				
	II		102		60			0	0	40	60	35												0.56	
	I		102		61								69	37										0.49	

Boring made about Two and one half years after completion of The working Table in The Second Unit.

SHEET NO 19 OF 23 SHEETS

FORM T-39-50
S-140-1

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No. 2013-R-21
Boring No. D7

LOCATION 45+00 on ℄ DIST. IV CO. S.M. RTE. 68 SEC. E WO. ..... DATE REPORTED .....

FIELD DATA			TEST RESULTS																							
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT WET	UNIT WEIGHT LBS PER CU FT DRY	MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS GRAVEL	SAND	SILT	CLAY	COL'D	ATTERBERG LIMITS LL	PI	CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT ⅛	¼	½	1	2	4	8	DIRECT SHEAR COHESION psf	ANGLE OF INTERNAL FRICTION	UNCONF COMP COMP STRENGTH T/FT²	REMARKS
5	VI	-54	99																							
	V		103		62			0	0	34	66	38												0.63	
	IV		101		61								72	41												
	III		101		59	2.72									0.1	0.3	1.2	3.3	9.2	18.1	25.3				
	II		101		61			0	0	38	62	32												0.57	
	I		101		61								66	36										0.52	
6	VI	-59	95																							
	V		96		71			0	0	20	80	44												0.87	
	IV		99		66								87	50										0.81	
	III		99		63										0.7	1.2	2.6	5.1	10.0	17.7	25.6				
	II		98		64			0	0	22	78	47												0.81	
	I		98		67								92	56										0.80	
7	B	-64			21			0	64	25	11	6													
	II		130		18			0	60	28	12	4												0.16	
	I		132																						
8	V	-69	135		16			0	46	31	23	9												2.66	
	IV		135		16								26	11										1.91	
	III		136		17	2.70																			
	II		137		16			0	47	33	20	11												1.47	
	I		138		16								23	8										1.14	

See Sheet #19

SHEET NO 20 OF 23 SHEETS

FORM T-39-50
S-148-1

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No 2013-R-21
Boring No D7 & D8

LOCATION D7-45+00 on ℄ / D8-49+00 77' Lt. ℄ DIST IV CO SM RTE 68 SEC E WO ..... DATE REPORTED .....

FIELD DATA			TEST RESULTS																						
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT WET	UNIT WEIGHT LBS PER CU FT DRY	MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS GRAVEL	SAND	SILT	CLAY	COLOID	ATTERBERG LIMITS LL	PI	CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT 1/8	1/4	1/2	1	2	4	8	DIRECT SHEAR COHESION psf	ANGLE OF INTERNAL FRICTION	UNCONF COMP COMP STRENGTH T/FT²	REMARKS
D7																									
9	V	-74	133		17			0	68	16	16	7												1.30	
	IV		130																						
	III		131		21	2.75										0.1	0.5	1.3	2.5	4.3	6.8				
	II		131																						
	I		131		19								25	9										1.55	
D8		-7																							
1	IV		100		61			0	0	46	54	32												0.21	
	III		100		63	2.73									0.6	1.8	4.4	9.4	16.0	22.8	29.2				
	II		103		60			0	0	46	54	32												0.21	
	I		100		60								65	33										0.27	
2	V	-12	100		59			0	0	42	58	36												0.24	
	IV		100		59								65	34										0.20	
	III		100		64			0	2	42	56	32													
	II		100		65										0.6	1.6	4.5	9.5	16.3	23.2	29.1				
	I		100		64								66	35										0.25	
3	V	-19	101		62	2.78		0	0	44	66	38												0.40	
	IV		101		61								67	36										0.36	
	III		99		62										0.4	1.4	3.8	8.0	14.5	21.4	27.7				
	II		100		64			0	0	43	57	35													
	I		101		62								66	35										0.30	

Boring D8 made about Two and one half years after completion of The working Table in The first Unit.

SHEET NO 21 OF 23 SHEETS

FORM T-39-50
5-148-1

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No. 2013-R-21
Boring No. D8

LOCATION 49+00 77' Lt. ℄ DIST. IV CO. SM RTE. 68 SEC. E WO. DATE REPORTED

FIELD DATA			TEST RESULTS																							
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT		MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS					ATTERBERG LIMITS		CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT							DIRECT SHEAR		UNCONF COMP	REMARKS
			WET	DRY				GRAVEL	SAND	SILT	CLAY	COLLOID	LL	PI	1/8	1/4	1/2	1	2	4	8	COHESION psf	ANGLE OF INTERNAL FRICTION	COMP STRENGTH T/FT²	
4	V	-29	100		66			0	0	41	59	36												0.30	
	IV		101		59								67	36											
	III		100		61			0	2	39	59	32													
	II		100		60										0.6	2.3	6.0	11.6	18.2	24.8	32.0				
	I		100		64								66	35										0.29	
5	VI	-39	100																						
	V		100		63			0	0	49	51	25													
	IV		100		63								65	33											
	III		103		60	2.75									0.3	1.0	2.8	6.2	13.3	20.6	27.4				
	II		101		64			0	0	41	59	34												0.37	
	I		100		62								65	34										0.35	
6	VI	-49	101																						
	V		104		60			0	4	38	58	34													
	IV		100		58								68	39										0.43	
	III		105		53	2.76									0.4	1.1	2.4	4.8	9.7	15.7	22.7				
	II		102		61			0	0	41	59	37													
	I		100		63								74	41										0.43	
7	VI	-54	100																						
	V		104		62			0	0	52	48	28												0.61	
	IV		104		61								71	39										0.58	
	III		100		64	2.76									0.4	1.3	5.3	9.0	16.2	24.8	32.2				
	II		100		63			0	0	27	73	42												0.48	

See Sheet #21

SHEET NO 22 OF 23 SHEETS

FORM T-39-50
5-168-1

STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

SUMMARY OF TESTS—FOUNDATION INVESTIGATION

Job No. 2013-R-21
Boring No. D8

LOCATION 49+00 77' Lt ℄ DIST IV CO SM RTE 68 SEC E WO

DATE REPORTED

FIELD DATA			TEST RESULTS																							
SAMPLE NO	TUBE NO	ELEV DEPTH	UNIT WEIGHT LBS PER CU FT WET	UNIT WEIGHT LBS PER CU FT DRY	MOIST CONT % DRY WEIGHT	SPEC GRAV	VOIDS RATIO	GRAIN SIZE ANALYSIS GRAVEL	SAND	SILT	CLAY	COLL'D	ATTERBERG LIMITS LL	PI	CONSOLIDATION % OF ORIGINAL HEIGHT LOAD—TONS PER SQ FT 1/8	1/4	1/2	1	2	4	8	DIRECT SHEAR COHESION psf	ANGLE OF INTERNAL FRICTION	UNCONF COMP COMP STRENGTH T/FT²	REMARKS
7	I		102		55								78	44										0.50	
8	V	-59	99		66			0	0	27	73	39												0.50	
	IV		99		66								76	42										0.54	
	III		99		68	2.74									0.2	0.6	2.3	5.6	12.6	21.1	28.8				
	II		98		67			0	0	23	77	42												0.48	
	I		99		64								72	38										0.42	
9	V	-69	103		57			0	0	27	73	38												0.57	
	IV		103		54								80	45										0.75	
	III		100		60	2.77									0.3	0.7	2.0	4.3	8.7	15.7	23.0				
	II		100		64			0	0	20	80	44												0.44	
	I		100		60								81	44										0.55	
10	V	-69	92		79			0	0	20	80	47												0.31	
	IV		96		72								96	53										0.95	
	III		128																						
	II		133		16			0	62	24	14	8												0.57	
	I		128																						
11	V	-74	125		22			0	8	40	52	29												1.04	
	IV		131		22								44	24										1.04	
	III		132																						
	II		133		18			0	43	28	29	14												1.08	
	I		133																						

See Sheet # 21

SHEET NO 23 OF 23 SHEETS

FORM T-39-50
5-168-1