Appendix D Dam Stability Analyses

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To: Klamath River Renewal Coporation

CC:

Technical Memorandum

Subject: Definite Plan for the Lower Klamath Project
Analysis of Stability of J.C. Boyle and Iron Gate Dams During Reservoir Drawdown

INTRODUCTION

AECOM prepared this technical memorandum in support of the design for the removal of the Iron Gate Dam and J.C. Boyle Dam, which are located on the Klamath River in northern California and southern Oregon, respectively. The purpose of this technical memorandum is to review existing geotechnical data related to the Iron Gate and J.C. Boyle embankments, characterize the materials in the embankments, and evaluate the stability of the upstream slopes of the embankments under various conditions of rapid drawdown of the reservoirs prior to dam removal.

Iron Gate Dam is a 189-foot high zoned earthfill embankment, as measured from the crest to the rock foundation. The crest of the dam is at El. 2343[1] feet. The crest of the dam is 20 feet wide, and the dam is approximately 740 feet long. The embankment upstream slopes are 2:1 (H:V) above El. 2328 feet, 2.5:1 from El. 2328 feet to 2300 feet, and 3H:1V below El. 2300 feet. The downstream slopes are 1.75:1 above El. 2323 feet and 2:1 below El. 2323 feet. The dam also features a 29-foot wide bench and a 10-foot wide bench at El. 2275 feet on the upstream side and downstream side, respectively. The dam consists of a central impervious clay core, an upstream and a downstream compacted pervious shell with filter zones and a downstream drain. A 10-foot thick layer of riprap protects the upstream slope of the dam against erosion. A 5-foot thick riprap layer is present on the downstream slope. In 2003, the dam crest was raised 5 feet from El. 2338 feet to 2343 feet by over-steepening the upstream and downstream slopes. To provide additional freeboard, a sheet pile was installed upstream of the dam centerline that extends five (5) feet above the dam crest to an El. of 2348 feet.

J.C. Boyle Dam consists of two portions: an earthfill embankment on the right side and a concrete spillway and gravity section on the left side. This technical memorandum evaluates the earthfill embankment portion of the dam. The earthfill embankment is a 68-foot high zoned earthfill embankment. The crest of the dam is at El. 3800 feet. The crest of the embankment is 15 feet wide and approximately 413 feet long. The upstream slopes are 2.5:1 (H:V) above El. 3780 feet and 3H:1V below El. 3780 feet. The downstream slopes are 2.5:1. The downstream slope also includes a 16-foot wide bench at El. 3768 feet. The internal zoning of the dam consists of a central impervious clay core, an upstream and a downstream compacted pervious shell consisting of sand and gravels. A filter blanket underlies the downstream shell. Erosion protection of the upstream slope is provided by a 3-foot thick riprap layer above El. 3680 feet. A 2-foot thick riprap layer below El. 3768 feet protects the downstream slope against erosion due to elevated tailwater.

EXISITNG DATA REVIEW

[1] All elevations in this memorandum are in the original datum unless otherwise indicated.

A review of existing available pertinent information for Iron Gate Dam and J.C. Boyle Dam were performed as part of this study to judge whether additional geotechnical investigation would have to be conducted for evaluating the dams for the rapid drawdown conditions. The reviewed information included design drawings, laboratory testing data for the borrow source materials, construction history, specifications, previous stability analyses, and post construction subsurface investigation. The results from the review indicate the followings:

- Representative analysis cross sections can be developed at the maximum section using the design drawings for both the Iron Gate Dam and the J.C. Boyle Dam.
- A reasonable material characterization of embankment materials, in particular the core and shell materials, can be developed using the information in the construction history, drawings, and specifications for the two dams. The source of materials, loose lift thickness and compaction efforts were discussed in those documents (California Oregon Power Company, 1960a and Unknown Publisher, Unknown Date). The results from a post-construction subsurface investigation conducted for J.C. Boyle Dam in 1994 (Black and Veatch, 1998) provide additional information for shell material characterization.
- Material properties necessary for performing slope stability and seepage analyses can be reasonably developed using the reviewed information. The reviewed information included laboratory shear strength and permeability tests conducted on the borrow source materials (California Oregon Power Company, 1960b and Unknown Date) and previous rapid drawdown analyses performed by others (Bechtel, 1968, Department of Water Resources, 1986, Black and Veatch, 1998, and PanGEO, 1998) .

The existing information for both dams are deemed sufficient to perform rapid drawdown analyses with targeted sensitivity analysis to address uncertainties associated with material properties as discussed later in this memorandum.

MATERIAL CHARACTERIZATION

Iron Gate Dam

Iron Gate Dam, which was built in 1961, is a zoned earth and rock fill dam. The dam consists of six (6) main zones: an upstream pervious shell (Zone I), a downstream pervious shell (Zone II), a central impervious core (Zone III), a transition (Zone IA) upstream of the core, a downstream chimney two-stage filter (Zone IV and Zone IVA) and drain (Zone V), and a downstream blanket filter (Zone IV) and drain (Zone V). The analysis section for rapid drawdown stability is the maximum cross section as shown on Figure 1.

The shell materials mainly consist of locally borrowed, pervious talus rock and gravel placed in 3-foot loose lifts, moisture conditioned, and compacted with four (4) passes of 72-inch vibratory roller (PanGEO, 2006). The weight of the roller was not indicated in the documents reviewed. The impervious core mainly consists of high plasticity clay from a local borrow source. The core material was placed in 8-inch loose lifts and compacted to not less than 95% of the maximum dry density as determined by ASTM D698 (California Oregon Power Company , 1960a and PanGEO, 2006). The upstream transition zone consists of graded talus rock and is approximately 20 feet in thickness. The downstream chimney and blanket filters consist of fine sand to gravel and were constructed in three (3) vertical layers (California Oregon Power Company, 1960a). Based on the design drawings, the thicknesses of the chimney and blanket filters are 20 feet and 5 feet, respectively. The downstream chimney and blanket drains consist of selected talus, gravel, or other excavations that is essentially free of materials smaller than the #100 sieve (California Oregon Power Company, 1960a). The dam was founded on basalt that is generally hard, blocky, heavily jointed, and moderately weathered (DSOD, 1986).

Iron Gate Dam Material Properties

The shear strength parameters of shell and core are very important for the rapid drawdown analysis. Shear strength parameters for the core material were developed mainly based on results from isotropic consolidated undrained triaxial tests (TX-ICU) conducted on samples obtained from borrow sources during borrow source evaluation (California Oregon Power Company, 1960b). The results of the triaxial tests are included in Attachment A. However, no laboratory shear strength tests are available for the shell and other embankment materials. Therefore, shear strength parameters for these materials were selected based on available information such as the type of construction, parameters used in previous analyses, and published data (NAVFAC, 1986 and EPRI, 1990). As mentioned above, the shell materials consist of talus rock and gravel, which were compacted during placement. Based on the published data, the effective friction angle for compacted gravelly materials would be greater than 37 degrees. For this rapid drawdown analysis, the shell materials were conservatively assigned an effective friction angle of 35 degrees. In addition, transition zone, chimney filter and drain, and blanket filter and drain were compacted during placement. Therefore, these materials were also assigned an effective friction angle of 35 degrees. The bedrock is modeled as impenetrable in the slope stability model. Table 1 summarizes these engineering parameters (best estimate parameters) used in the slope stability analyses.

The unit weights for different embankment zones were selected based on the laboratory tests conducted on the samples collected from proposed borrow areas, compaction test results on samples collected during dam construction, previous analyses (DWR, 1986 and PanGEO, 2006), and published data (NAVFAC, 1986 and EPRI, 1990).

The permeability values for the core and shell materials were selected based on the results from the falling head permeability tests performed on samples from the core and shell material borrow sources during borrow source evaluation. The results of the falling head permeability tests are included in Attachment B. Permeability values of the filter, chimney drain, the blanket drain, the riprap, and the random fill were estimated based on the characteristics of the materials, published data, and engineering judgment. The permeability parameters were selected conservatively based on typical ranges (Holtz and Kovacs, 1981), which is included in Attachment C. Table 1 summarizes permeability parameters used in the seepage analysis.

Anistropic ratios (k_h/k_v) typically range from 1 to 4 for uniform soil deposits without significant interbedding or stratification but can be higher for soil deposits with significant stratification. An anisotropic ratio of 10 for the core is selected considering the nature of the materials and its placement method. For the shell and random fill, an anisotropic ratio of 2 was selected as typical anisotropic ratios for similar materials range from 1 to 2. Anisotropic ratio for the filter/drain and riprap is selected to be 1 as the materials are expected to drain freely in both directions.

Table 1. Material Properties Used for the Analyses of Iron Gate Dam

Material	Unit Weight (pcf)	Effective Stress		Total Stress		Horizontal Permeability, k_h (cm/s)[1,3]	k_h/k_v
		Cohesion, c' (psf)	Friction Angle, ϕ' (°)[1,2]	Cohesion, c (psf)	Friction Angle, ϕ (°)		
Core	130	0	22	300	16	1.00E-07	10
Shell	135	0	35	-	-	8.00E-03	2
Filter/ Drain/ Transition Zones	135	0	35	-	-	1.00E-02	1
Riprap	135	0	35	-	-	1.00E-02	1
Random Fill	135	0	25	-	-	8.00E-03	2

Note:
1. The parameter that was used for sensitivity analyses is provided in parenthesis.
2. For compacted sand and gravel materials, the friction angles are typically greater than 34 degrees (NAVFAC, 1986 and EPRI, 1990).
3. For clean coarse materials, permeability ranges from 10^{-3} cm/s to 1 cm/s per Holtz and Kovacs (1981).

J.C. Boyle Dam

The earthfill embankment of the J.C. Boyle Dam is a zoned earth fill dam built in 1958. The dam consists of two (2) major zones: a central impervious clay core (Zone 1) and the upstream and downstream pervious shells (Zone 2). A filter blanket with thickness of 12 inches was placed between the Zone 2 materials and its foundation for the whole downstream area. An 18-inch thick gravel drain zone was also installed over part of the downstream foundation. A waste rock fill was placed at the downstream toe of the dam. Ripraps are placed on both the upstream and downstream sides of the dam. For analysis purpose, the gravel drain is modeled as part of the filter blanket. The rapid drawdown analyses were performed on maximum cross section of J.C. Boyle Dam, which is shown on Figure 2.

The impervious clay core is constructed of selected clay materials, which are described as rust colored sandy clay with some pea gravel. The shell materials were constructed of a mixture of well graded gravel with sand and well graded sand. Based on the specifications, the embankment materials were to be constructed in 8-inch loose lift and compacted with a minimum of twelve (12) passes of sheepfoot rollers to obtain a minimum of 95% of the dry density which correspond to the optimum moisture content of the materials placed The filter blanket is approximately 12 inches thick and consists of well graded sandy gravel. The waste rock fill was constructed of gravel placed under water without compaction. Specific information regarding size and compaction effort is not available for the upstream and downstream ripraps and the gravel drain. The dam is mostly founded on basalt with the exception of the right abutment, which is founded on satisfactory overburden (Bechtel, 1968).

J.C. Boyle Dam Material Properties

The effective shear strength parameters for the core material are developed based on the results of direct shear tests performed on samples from core borrow sources during borrow source evaluation. The results show that the effective friction angle is greater than that of Iron Gate Dam's core. This is consistent with the material descriptions which suggest that the core in J.C. Boyle Dam consists of lower plasticity clay and pea gravel. The results of the direct shear test are included in Attachment D. The total stress shear strength parameters are not available from the direct shear tests. For the purpose of rapid drawdown slope stability analysis, those parameters were conservatively assumed the same as those of the Iron Gate Dam core. No laboratory shear strength data are available for the other embankment materials. Previous slope stability analyses performed by others selected the shear strength parameters based on the SPT blow count data (Black and Veatch, 1998). Review of available data suggests that the shell materials consist of up to 50% of gravel. The shear strength parameters that were previously selected did not account for the presence of high gravel percentage in the shell material. Considering the high gravel content, the borrow source, and how the shell material was placed and compacted, for the purpose of the rapid drawdown analysis a friction angle of 34 degrees (the previous analysis used a friction angle of 37 degrees) was assumed. The strength parameters of the riprap are conservatively assumed to be the same as the shell materials as the anticipated effect from the riprap on the overall stability performance is not significant due to its relative thickness to the shell. The bedrock is modeled as impenetrable in the slope stability model. Table 2 summarizes the best estimate engineering parameters used in slope stability analyses.

As no total strength parameters are available for the core materials, a sensitivity analysis is performed on the strength parameters for the core materials. Total cohesion of 100 psf and total friction angle of 12 degrees were conservatively selected considering very soft soil conditions for this sensitivity analysis. This sensitivity analysis also considers a lower effective friction angle of 19.4 degrees for the core materials, which was selected based on the lowest values from the direct shear tests. As the core is relatively thin compared to the shell, it is anticipated that reducing the strength parameters for the core materials will not significantly impact the analysis results. Table 2 includes the engineering parameters used in the sensitivity analysis in parenthesis.

Compaction tests performed on the samples from the core and shell borrow sources during borrow source evaluation were used as the basis for unit weight of the materials. The results of the compaction tests are included in Attachment E. The selection of the unit weight used in the rapid drawdown analysis is based on the compaction test results, published data (NAVFAC, 1986 and EPRI, 1990), and previous analyses. Table 2 summarizes the unit weights used in the slope stability analysis.

Falling head permeability tests performed on samples from the core borrow sources during borrow source evaluation were used as the basis for permeability values of the core material. The results of the permeability test are included in Attachment F. Permeability values for the shell materials and filter blankets are estimated based on results of the grain size analysis using the Kozemy-Carmen permeability correlations, characteristics of the materials, published data, and engineering judgement. The permeability of the riprap is assumed to be the same as the shell materials, whereas the permeability of the wasterock fill is assumed to be the same as the shell. Table 2 summarizes the best estimate engineering properties used in the seepage analyses.

Similar to Iron Gate Dam, anisotropic ratios of 10 and 2 are selected for the core and shell materials with the exception of riprap, respectively. An anisotropic ratio of 1 is selected for the ripraps.

In addition, a set of sensitivity analysis was performed based on typical permeability ranges for gravel and sand materials (Holtz and Kovacs, 1981). This set of sensitivity analysis conservatively assumes the lower permeability values within the typical ranges for the shell, riprap, filter blanket, and waste rock fill. Table 2 includes the engineering parameters used in the sensitivity analysis in parenthesis.

Table 2. Material Properties Used for the Analyses of J.C. Boyle Dam

Material	Unit Weight (pcf)	Effective Stress		Total Stress		Horizontal Permeability, k_h (cm/s)[1,3]	k_h/k_v
		Cohesion, c' (psf)	Friction Angle, ϕ' (°)[1,2]	Cohesion, c (psf)[1]	Friction Angle, ϕ (°)[1]		
Core	120	0	27 (19)	300 (100)	16 (12)	1.71E-04	10
Shell	130	0	34	-	-	6.62E-01 (4.00E-03)	2
Upstream Riprap	140	0	34	-	-	1.04E-00 (4.00E-03)	1
Downstream Riprap	140	0	34	-	-	1.04E-00 (4.00E-03)	1
Filter Blanket	125	0	35	-	-	1.04E-00 (4.00E-03)	2
Waste Rock Fill	145	0	40	-	-	6.62E-01 (4.00E-03)	2

Note:
1. The parameter that was used for sensitivity analyses is provided in parenthesis.
2. For compacted sand and gravel materials, the friction angles are typically greater than 34 degrees (NAVFAC, 1986 and EPRI, 1990).
3. For clean coarse materials, permeability ranges from 10^{-3} cm/s to 1 cm/s per Holtz and Kovacs (1981).

PREVIOUS SLOPE STABILITY ANALYSIS PERFORMED BY OTHERS

Iron Gate Dam

After the construction of the Iron Gate dam, stability analyses of the dam were originally performed by the Division of Safety of Dams (DSOD) in 1962 (DWR, 1986). The slope stability analyses were performed for static, rapid drawdown, and pseudo-static loading conditions with assumed effective friction angles of 30 and 17 degrees with no cohesion for the shell and core, respectively. A minimum factor of safety of 1.67 was calculated for the rapid drawdown conditions. Bechtel Corporation analyzed stability of the embankment in 1968 using effective friction angles of 35 degrees for the shell and 22 degrees for the core. The rapid drawdown analysis performed as part of Bechtel's analyses calculated a minimum factor of safety of 1.99 (DWR, 1986). In 1986, DSOD reanalyzed the dam by assigning an effective friction angle of 35 degrees for the shell zones and drained zones, and calculated a minimum factor of safety of 2.00 for rapid drawdown. These stability evaluations were then updated in 1995 and 2004 to account for the then planned dam raises (Section 8 of STID, 2015). The existing dam incorporates the sheet-pile raised crest, and has an effective crest elevation of 2348.0 feet.

As the latest stability analysis, PanGEO performed the preliminary assessment of the stability of upstream slope under rapid drawdown conditions and presented the results in a technical memorandum (PanGEO, 2008).

J.C. Boyle Dam

Based on available information, two (2) rapid drawdown analyses were performed in 1968 and 1996 (Bechtel, 1968 and Black and Veatch, 1996). The 1968 analysis assumed a very conservative strength for the shell materials, in which the shear strength of the shell materials was assumed to be the same as the shear strength of the core materials (effective friction angle of 26 degrees). The phreatic surface used in the analysis was derived by a flow net analysis, which considered partial pore dissipation within the shell materials. The rapid drawdown analysis resulted in a factor of safety of 1.03. In 1994, three (3) borings were drilled on the downstream side of the dam to collect additional subsurface information for better material characterization for the shell materials. Based on the results of this subsurface investigation, the 1996 analysis assumed a higher shear strength for the shell material (effective friction angle of 37 degrees). No additional seepage analysis was performed, and the phreatic surface from the 1968 analysis was assumed in the 1996 analysis. The rapid drawdown analysis resulted in a factor of 1.88.

CURRENT RAPID DRAWDOWN ANALYSIS

Sudden or rapid drawdown is the most critical condition controlling the lowering of the reservoir prior to dam removal because deep slides in the upstream slope of the dam during the drawdown could lead to dam failure. Rapid drawdown reduces the total stress on the upstream face and lowers the head driving seepage through the embankment. The shear stresses within the upstream slope increase which may lead to instability. In principle, the stability of the upstream slope can be evaluated using either total stress (undrained) or effective stress (drained) strength parameters. The rapid drawdown analysis approach used for this Project involves the following steps:

1. Develop analysis sections and material properties,
2. Establish a base case by performing conventional rapid drawdown stability analysis under instantaneous drawdown for two scenarios that provide the upper and lower bound for stability of the dams during rapid drawdown:
    a. The first scenario (least conservative bound) assumes full pore pressure dissipation within the pervious shell after drawdown from the steady state condition.
    b. The second scenario (most conservative bound) assumes no pore pressure dissipation within the pervious shell from after drawdown from the steady state condition.
3. Perform transient drawdown analysis for various drawdown rates:
    a. Seepage analysis to determine the location of the phreatic surface at different time steps during reservoir drawdown
    b. Slope stability analysis for each corresponding phreatic surface during reservoir drawdown.
4. Additional sensitivity analyses, if needed.

SEEP/W (Geo-Studio, 2016) presents a method for using uncoupled transient seepage analysis along with limit equilibrium to evaluate the stability of slopes affected by changing hydraulic boundary conditions such as the conditions during rapid drawdown. The latest version of the USBR Embankment Dam design standards (2011) recommends using the effective stress approach with pore pressures from uncoupled transient seepage analysis to analyze stability following rapid drawdown. For these reasons, a transient analysis was considered as listed above. Because the shells of the dams are constructed of pervious materials rapid drawdown of the reservoir level behind the dams will result in concurrent (but slower) lowering of the phreatic surface (groundwater level) in the upstream shell of the dams. To account for this, transient seepage analyses are required. The computer programs SEEP/W and SLOPE/W (Geo-Studio, 2016) were utilized for the seepage and slope stability. SEEP/W is a two-dimensional, finite element analysis software program that has the capability to analyze both steady-state and transient seepage conditions. Slope/W is used to perform limit equilibrium slope stability analyses. Slope/W uses the phreatic surface developed in SEEP/W as input to the stability analysis. The limit equilibrium slope stability calculations use Spencer’s method, which satisfies both moment and force equilibrium simultaneously.

Acceptance Criterion

According to the Engineering Manual (EM-110-2-1902) of United States Army Corps of Engineers (USACE), the factor of safety for the rapid drawdown analyses of the upstream slope of the dam should be greater than the range of 1.1 to 1.3. Given, the importance of safety to both workers on site and the public downstream of the dams, the minimum rapid drawdown factor of safety for transient seepage analyses is selected to be 1.3.

Analysis Results

Rapid drawdown slope stability analyses were performed to calculate the minimum factors of safety for the following five (5) scenarios as described below:

1. Instantaneous drawdown from steady state condition with full pore pressure dissipation in the shell materials (least conservative bound).
2. Instantaneous drawdown from steady state condition with no pore pressure dissipation in the shell materials (most conservative bound).
3. Slow drawdown rate (3 ft/day for Iron Gate Dam and 2 ft/day for J.C. Boyle Dam)
4. Intermediate drawdown rate (6 ft/day for Iron Gate Dam and 5 ft/day for J.C. Boyle Dam)
5. Rapid drawdown rate (10 ft/day for Iron Gate Dam and 10 ft/day for J.C. Boyle Dam)

For Iron Gate Dam, the reservoir was drawn down from El. 2328 feet to El. 2202 feet. For J.C. Boyle Dam, the reservoir was drawn down from El. 3793 feet to El. 3762 feet. The results of the rapid drawdown slope stability analyses for Iron Gate Dam are summarized in Table 3. Table 3 also includes the results of the sensitivity analyses, which consider the potential lower bound strength for the shell materials. The results of rapid drawdown slope stability analyses for J.C. Boyle Dam are summarized in Table 4. Table 4 also includes the results of the sensitivity analyses, which consider the lower bounds for both the core strength and the shell permeability. The analysis results for the best estimate parameters are also shown on Figures 3 through 7 for Iron Gate Dam, and on Figures 8 through 12 for J.C. Boyle Dam. It should be noted that the plotted phreatic surfaces shown on the figures for the transient rapid drawdown analyses correspond to the phreatic surfaces at the specific time when the calculated factors of safety are minimum.

Table 3. Rapid Drawdown Slope Stability Analysis Results for Iron Gate Dam

Scenario	Factors of Safety for Best Estimate Parameters	
	Mid-Slope	Full-Slope
1. Instantaneous drawdown, full pore pressure dissipation	1.91	2.02
2. Instantaneous drawdown, no pore pressure dissipation within upstream shell	1.42	1.46
3. Slow drawdown rate (3 ft/day)	1.51	1.77
4. Intermediate drawdown rate (6 ft/day)	1.49	1.74
5. Rapid drawdown rate (10 ft/day)	1.48	1.70

Table 4. Rapid Drawdown Slope Stability Analysis Results for J.C. Boyle Dam

Scenario	Factor of Safety for Best Estimate for Core Strength		Factor of Safety from Sensitivity Analyses Using Potential Lower Bound Strength for Core	
	Mid-Slope	Full-Slope	Mid-Slope	Full-Slope
1. Instantaneous drawdown, full pore pressure dissipation	2.06 (2.06)	1.86 (1.86)	1.97 (1.97)	1.85 (1.85)
2. Instantaneous drawdown, no pore pressure dissipation within upstream shell	1.11 (1.12)	1.18 (1.18)	1.10 (1.10)	1.18 (1.18)
3. Slow drawdown rate (2 ft/day)	1.77 (1.76)	1.84 (1.74)	1.70 (1.70)	1.83 (1.73)
4. Intermediate drawdown rate (5 ft/day)	1.78 (1.76)	1.85 (1.66)	1.70 (1.69)	1.83 (1.66)

5. Rapid drawdown rate (10 ft/day)	1.78 (1.72)	1.85 (1.61)	1.75 (1.69)	1.82 (1.61)

Note: The values in parenthesis refer to the results of the sensitivity analysis using the lower permeability for the shell materials.

Conclusions

Rapid drawdown analysis results for the Iron Gate Dam and J.C. Boyle Dam indicate that the calculated factors of safety are greater than the selected minimum factor of safety of 1.3 for all cases analyzed except some cases instantaneous drawdown without any pore pressure dissipations for the J.C. Boyle Dam. However, in these cases, the minimum factors of safety are still within the range recommended by USACE. In addition, it should be noted that these cases conservatively assume no pore pressure dissipation within the upstream shell. Based on the analyses, reservoir drawdown could be as high as 10 feet/day. However, we recommend that reservoir drawdown be 5 feet/day, except as noted for J,C. Boyle Dam below.

It is our understanding that the demolition of J.C. Boyle Dam includes removal of concrete stoplogs within two diversion culverts. The removal of the concrete stoplogs (likely by blasting) will result in drawdown of approximately 10 feet for the first culvert and 8 feet for the second culvert within less than 24 hours. Although we conclude that the J.C. Boyle Dam will perform satisfactorily under these rapid drawdown conditions, we recommend a hold period of one week be implemented between removal of the stoplogs from the first culvert until the stoplogs from the second culvert are removed to allow for pore pressure dissipation.

The analysis results indicated that no slope instability would result during reservoir drawdown. However, there is a potential for shallow slumping along the upstream embankment slopes due to the potential strength loss of surficial materials during the drawdown. Therefore, we recommend frequent visual inspection during the reservoir drawdown process. If any shallow slumping is observed, riprap can be placed to provide additional resistance.

It is recommended that instrumentation should be installed to monitor the upstream slopes during reservoir drawdown for dam removal. The types of recommended instrumentation include survey monuments, inclinometers, and piezometers. Daily readings are recommended to closely monitor if there are any unanticipated slope movements or pore pressure accumulation. It is also recommended that the instrumentation be installed the year prior to reservoir drawdown. The piezometers would be monitored during reservoir drawdown to confirm that the transient phreatic surfacewithin the upstream shell of the dam falls as the reservoir elevation drops.

Limitations

AECOM represents that our services were conducted in a manner consistent with the standard of care ordinarily applied as the state of practice in the profession within the limits prescribed by our client. No other warranties, either expressed or implied, are included or intended in this technical memorandum.

Background information and other data have been furnished to AECOM by Pacific Corp and/or third parties, which AECOM has used in preparing this technical memorandum. AECOM has relied on this information as furnished, and is neither responsible for nor has confirmed the accuracy of this information.

The analyses and results presented in this report are for the current study only and should not be extended or used for any other purposes.

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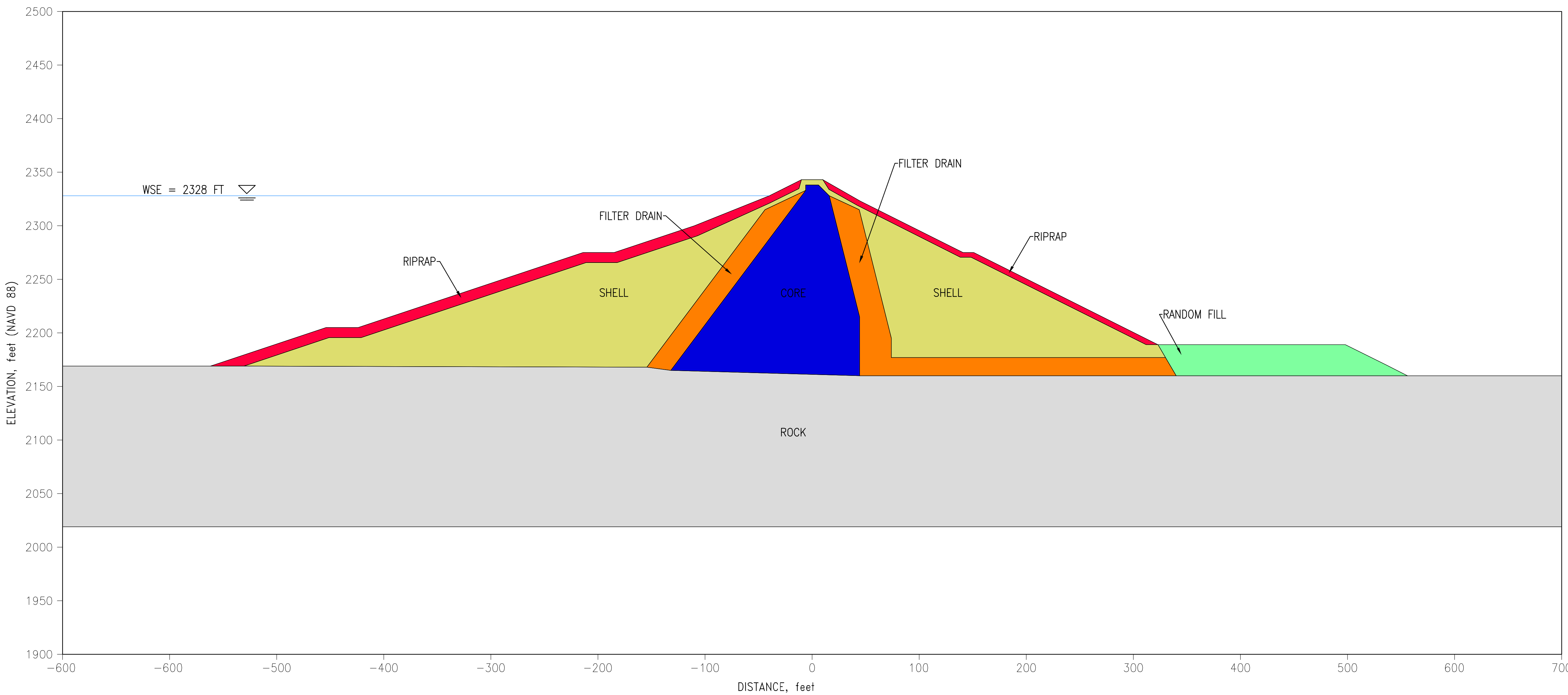
Figures



PROJECT NUMBER 60537920	PREPARED BY KANAX KANAGALINGAM	KLAMATH RIVER DAM REMOVAL	ANALYSIS MODEL GEOMETRY IRON GATE DAM	FIGURE 1
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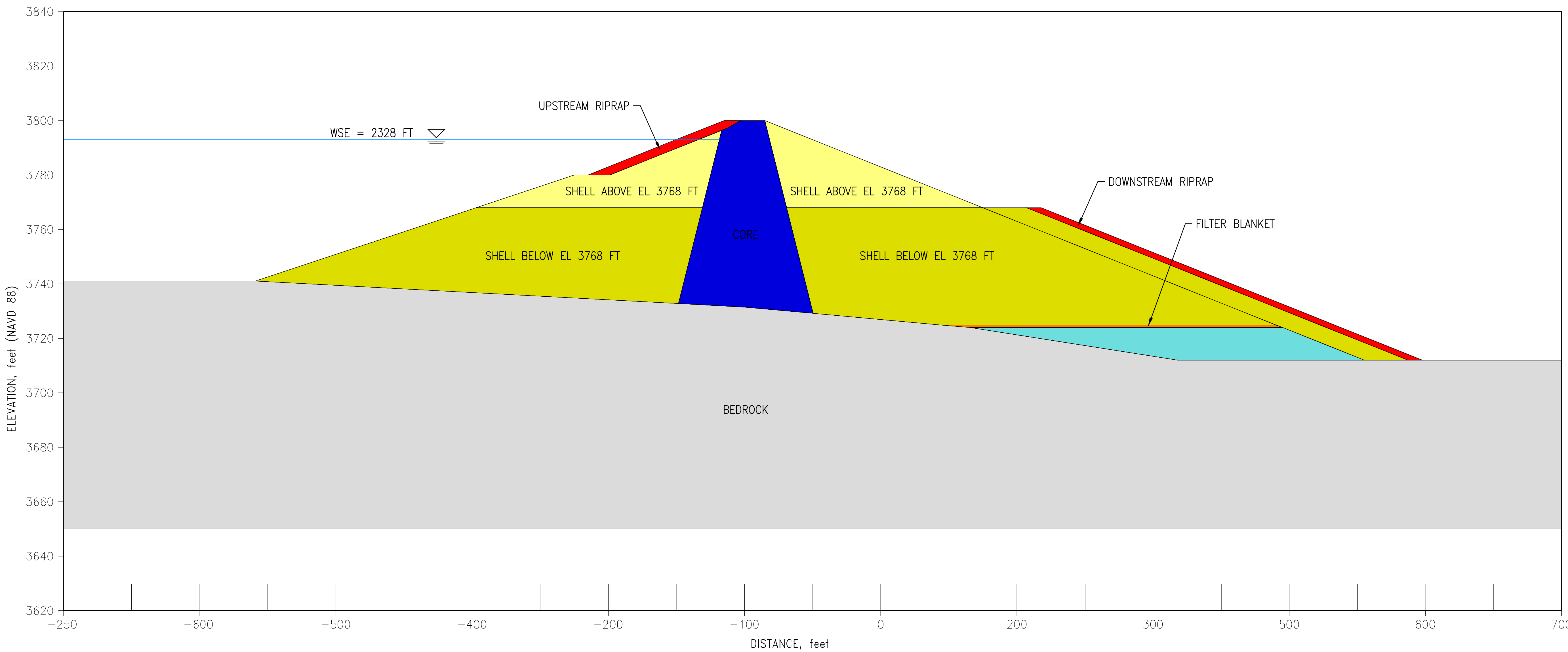

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PROJECT NUMBER 60537920	PREPARED BY BENJAMIN CHOY	KLAMATH RIVER DAM REMOVAL	ANALYSIS MODEL GEOMETRY JC BOYLE DAM	FIGURE 2
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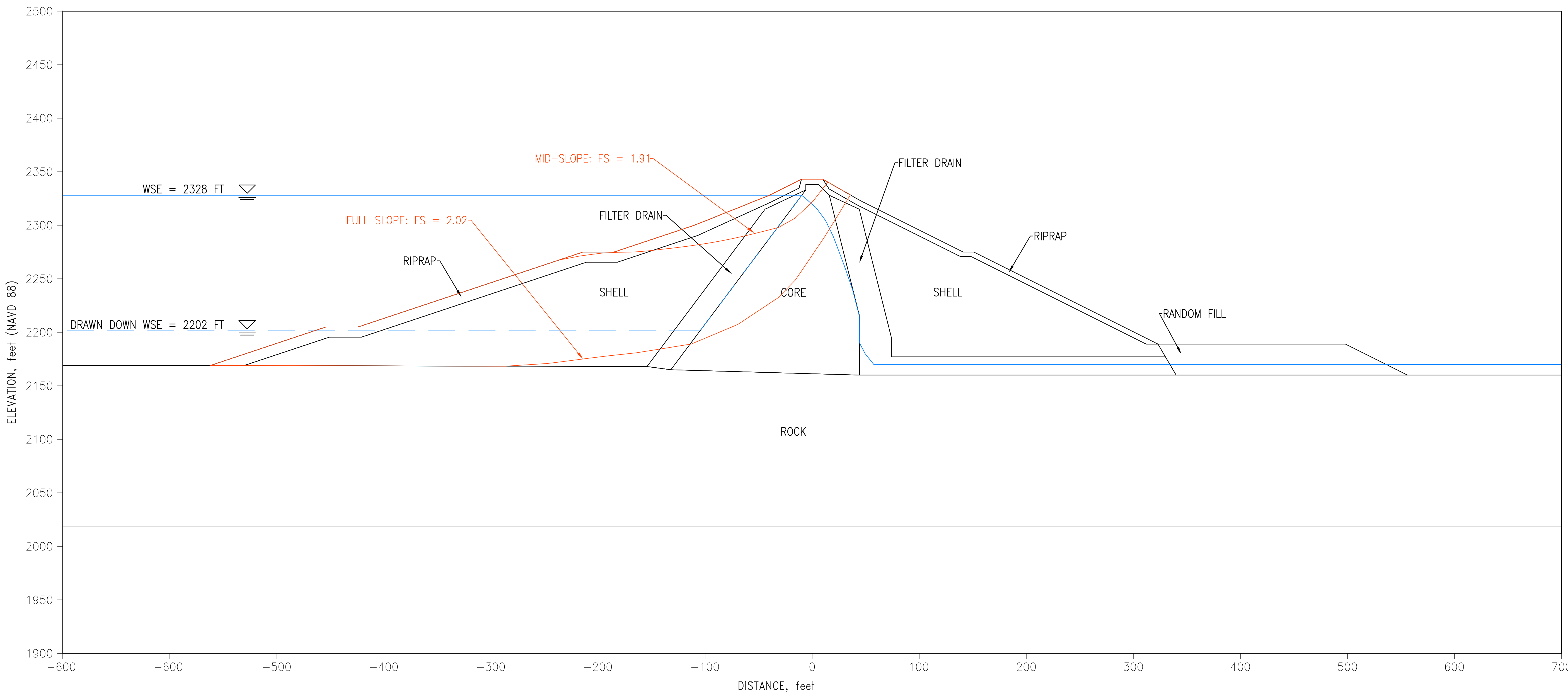

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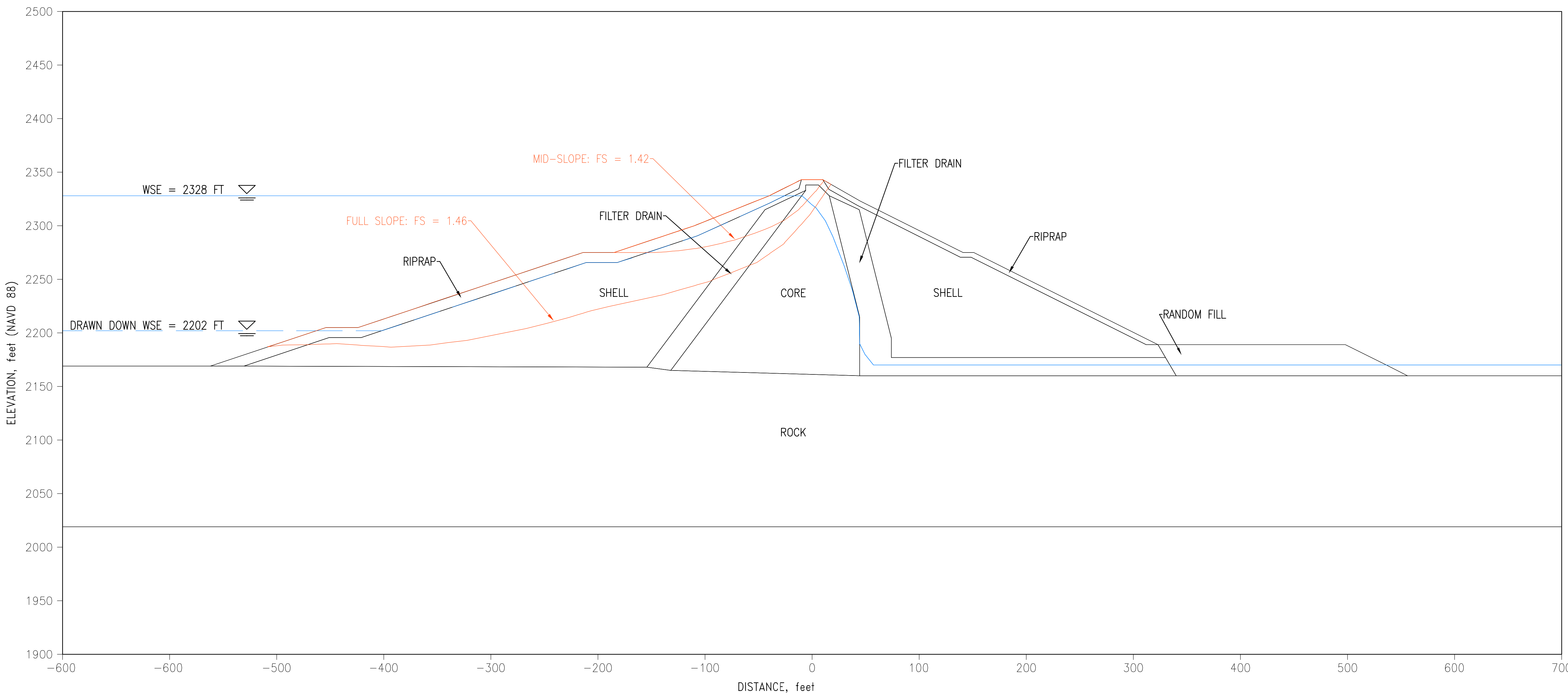
PROJECT NUMBER	PREPARED BY	KLAMATH RIVER DAM REMOVAL	RAPID DRAWDOWN STABILITY ANALYSIS IRON GATE DAM INSTANTANEOUS DRAWDOWN SCENARIO FULL PORE PRESSURE DISSIPATION	FIGURE
60537920	KANAX KANAGALINGAM			3
REVISION	CHECKED BY			
0	BENJAMIN CHOY			



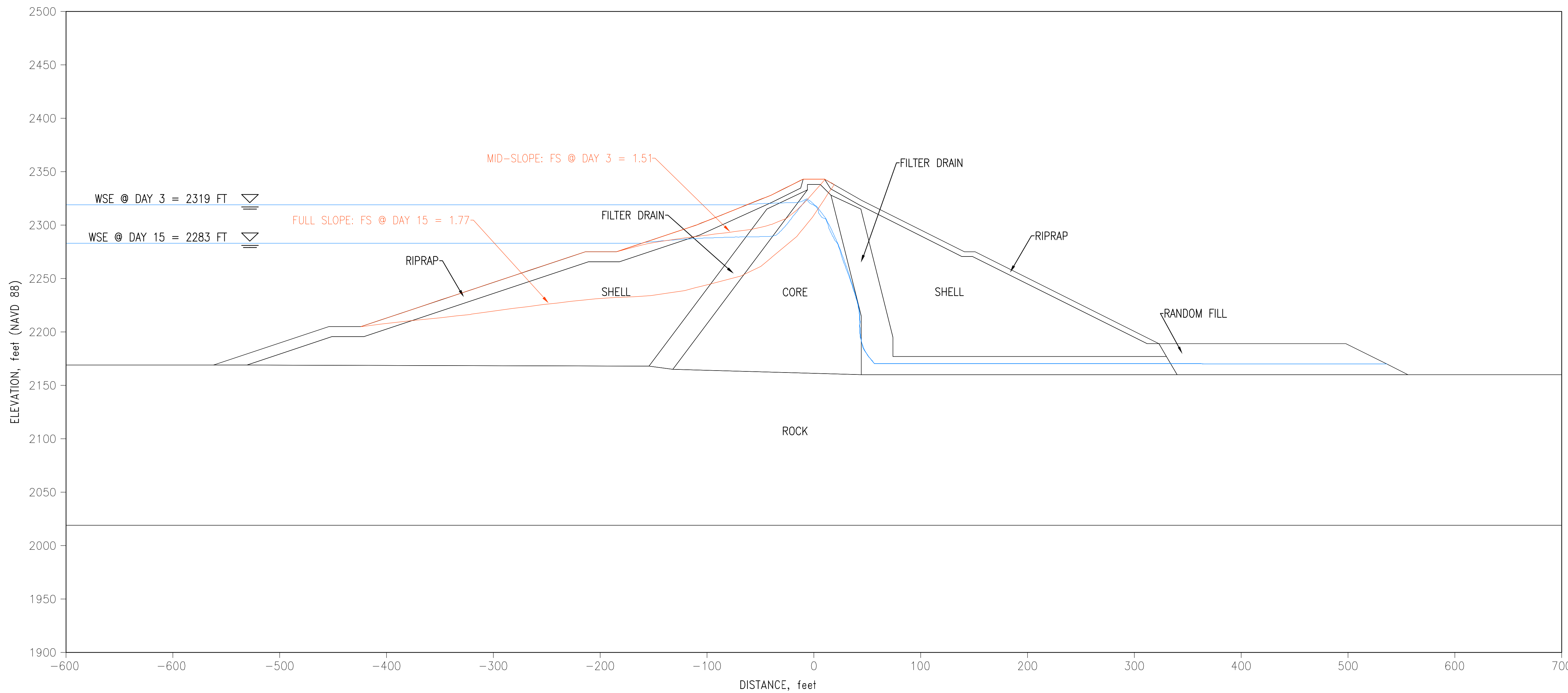
PLOT BY: BENJAMIN.CHOY − Sep 07, 2017 − 2:19:24pm
DRAWING: K:\400−Technical\430 Engineering\3.2 Tech Assess\5. Embank Stability\Iron Gate\Figures\Figure 6.dwg

PROJECT NUMBER	PREPARED BY
60537920	KANAX KANAGALINGAM
REVISION	CHECKED BY
0	BENJAMIN CHOY

KLAMATH RIVER DAM REMOVAL

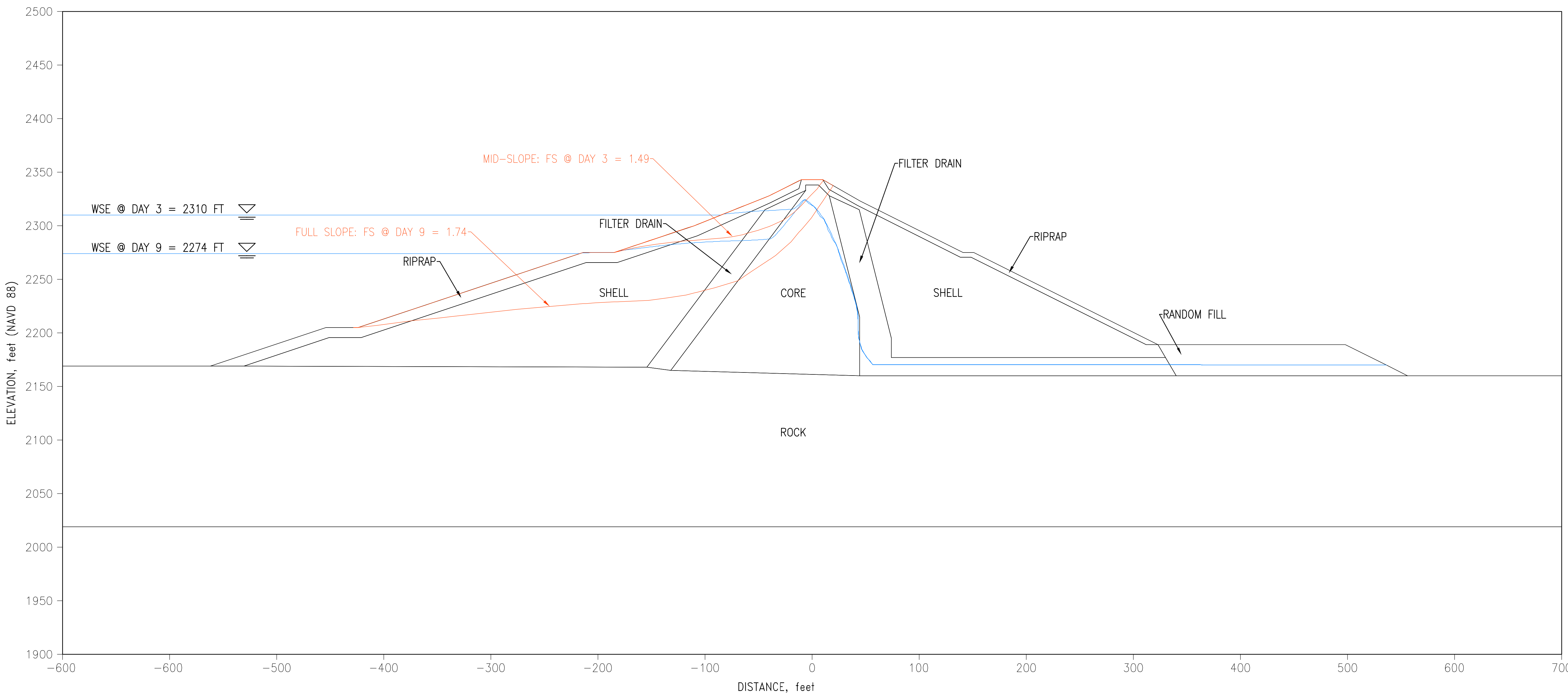
RAPID DRAWDOWN STABILITY ANALYSIS
IRON GATE DAM
INSTANTANEOUS DRAWDOWN SCENARIO
NO PORE PRESSURE DISSIPATION

FIGURE
4



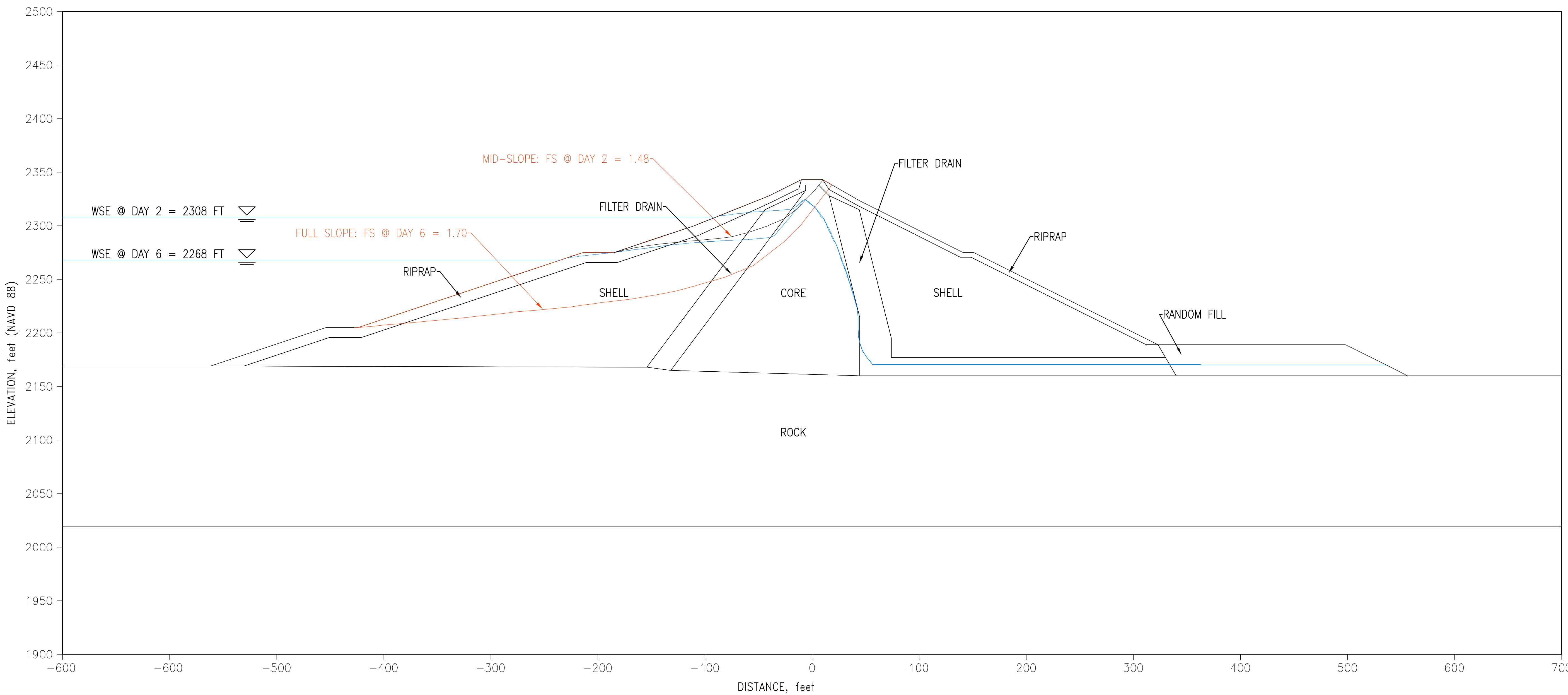
PROJECT NUMBER 60537920	PREPARED BY KANAX KANAGALINGAM	KLAMATH RIVER DAM REMOVAL	RAPID DRAWDOWN STABILITY ANALYSIS IRON GATE DAM SLOW DRAWDOWN SCENARIO 3 FT/DAY DRAWDOWN RATE	FIGURE 5
REVISION 0	CHECKED BY BENJAMIN CHOY			

PLOT BY: BENJAMIN.CHOY – Sep 07, 2017 – 2:22:05pm
DRAWING: K:\400-Technical\430 Engineering\3.2 Tech Assess\5. Embank Stability\Iron Gate\Figures\Figure 7.dwg



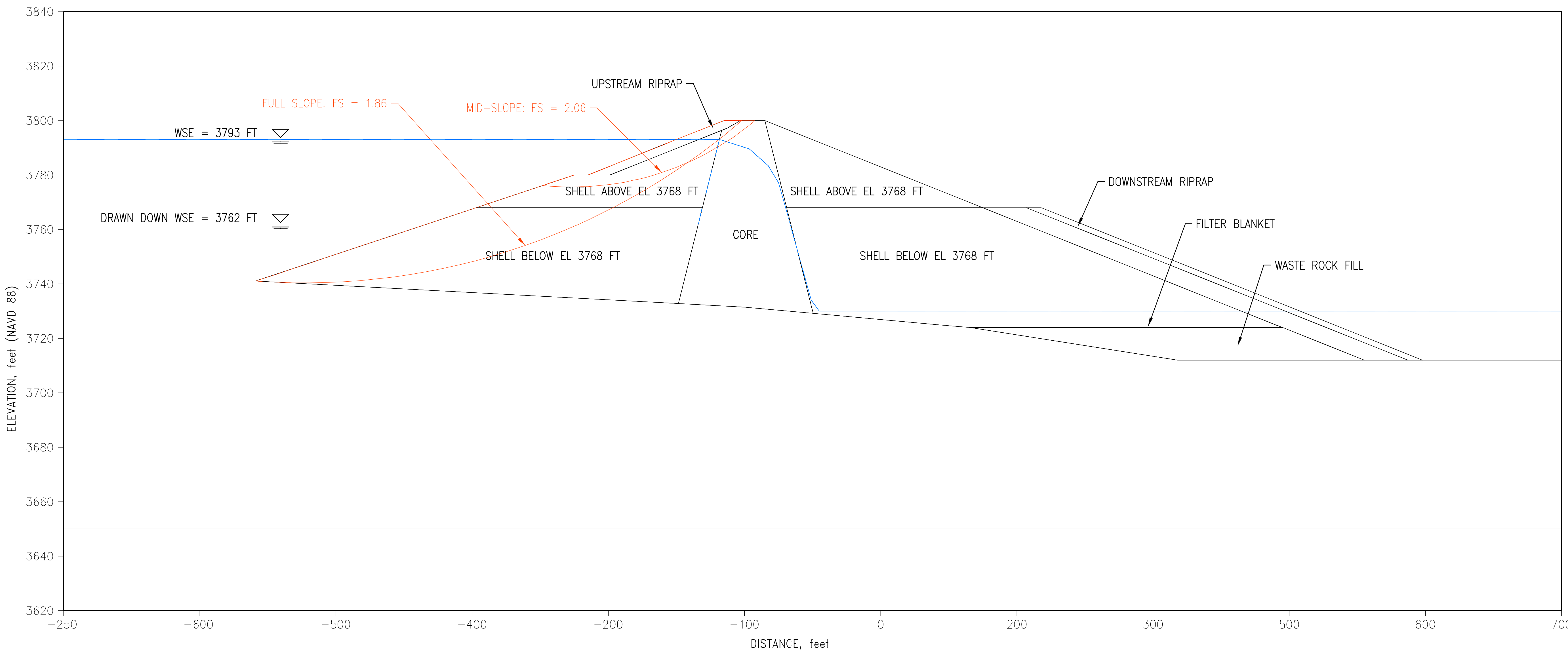
PROJECT NUMBER	PREPARED BY	KLAMATH RIVER DAM REMOVAL	RAPID DRAWDOWN STABILITY ANALYSIS IRON GATE DAM SLOW DRAWDOWN SCENARIO 6 FT/DAY DRAWDOWN RATE	FIGURE
60537920	KANAX KANAGALINGAM			6
REVISION	CHECKED BY			
0	BENJAMIN CHOY			

PLOT BY: BENJAMIN.CHOY − Sep 07, 2017 − 2:26:05pm
DRAWING: K:\400−Technical\430 Engineering\3.2 Tech Assess\5. Embank Stability\Iron Gate\Figures\Figure 8.dwg



PLOT BY: BENJAMIN.CHOY – Sep 07, 2017 – 2:29:50pm
DRAWING: K:\400-Technical\430 Engineering\3.2 Tech Assess\5. Embank Stability\Iron Gate\Figures\Figure 9.dwg

PROJECT NUMBER 60537920	PREPARED BY KANAX KANAGALINGAM	KLAMATH RIVER DAM REMOVAL	RAPID DRAWDOWN STABILITY ANALYSIS IRON GATE DAM SLOW DRAWDOWN SCENARIO 10 FT/DAY DRAWDOWN RATE	FIGURE 7
REVISION 0	CHECKED BY BENJAMIN CHOY			



PROJECT NUMBER	PREPARED BY
60537920	BENJAMIN CHOY
REVISION	CHECKED BY
0	KANAX KANAGALINGAM

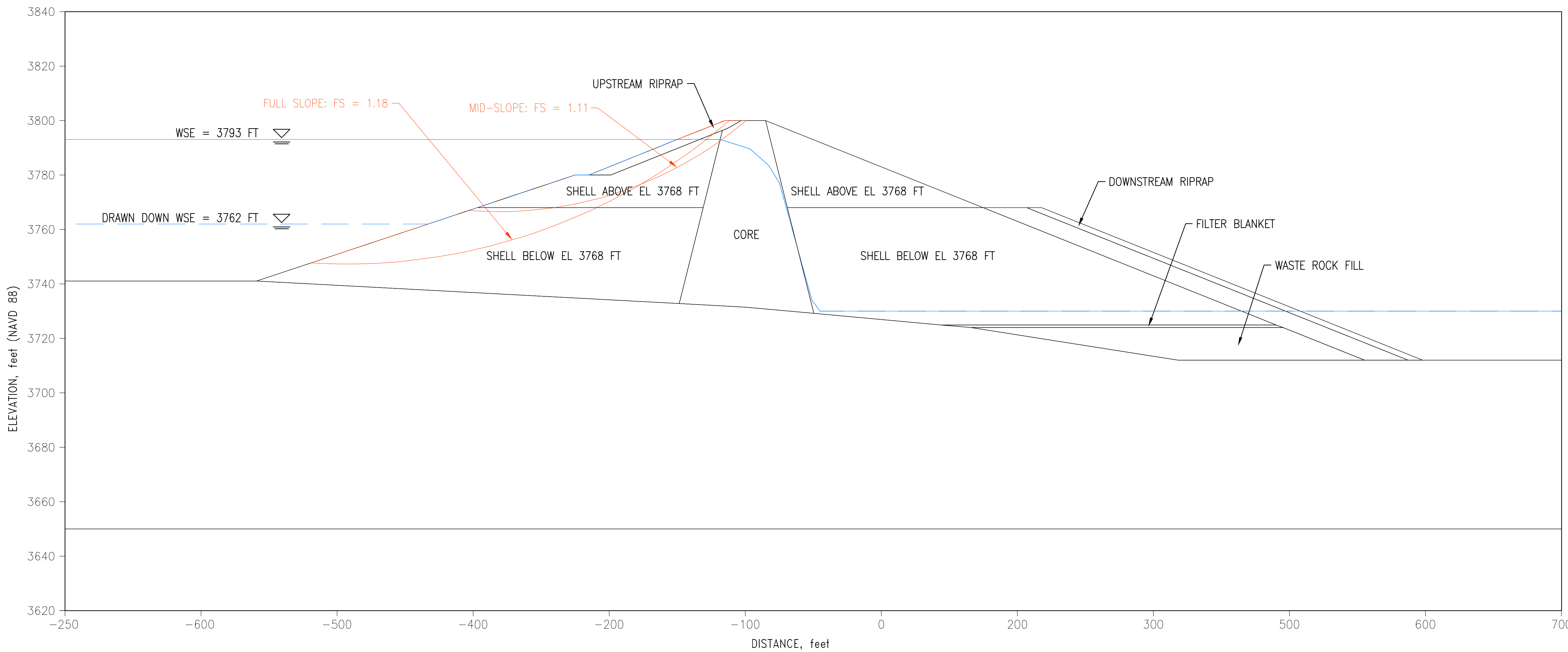
KLAMATH RIVER DAM REMOVAL

RAPID DRAWDOWN STABILITY ANALYSIS
JC BOYLE DAM
INSTANTANEOUS DRAWDOWN SCENARIO
FULL PORE PRESSURE DISSIPATION

FIGURE

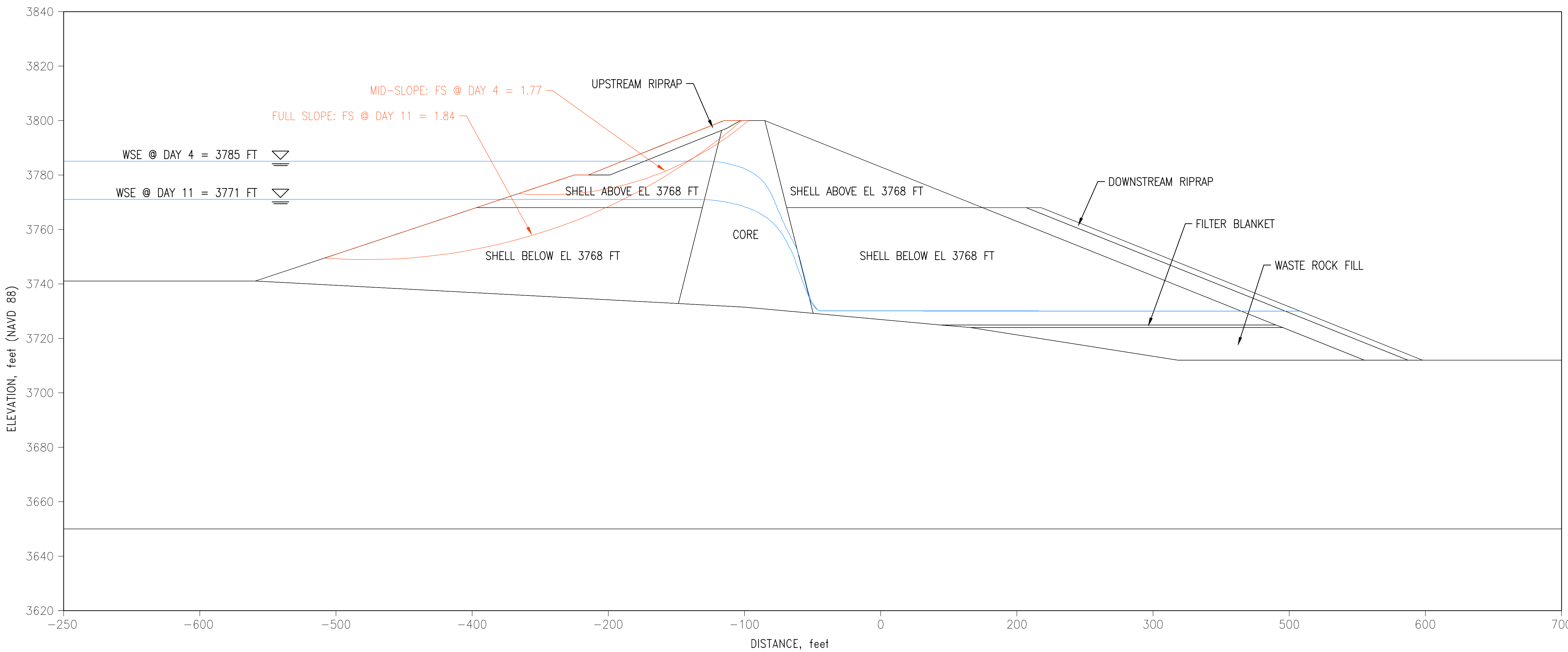
8

PLOT BY: BENJAMIN.CHOY – Sep 11, 2017 – 10:26:44am
DRAWING: K:\400–Technical\430 Engineering\3.2 Tech Assess\5. Embank Stability\JC Boyle\Figures\Figure 10.dwg



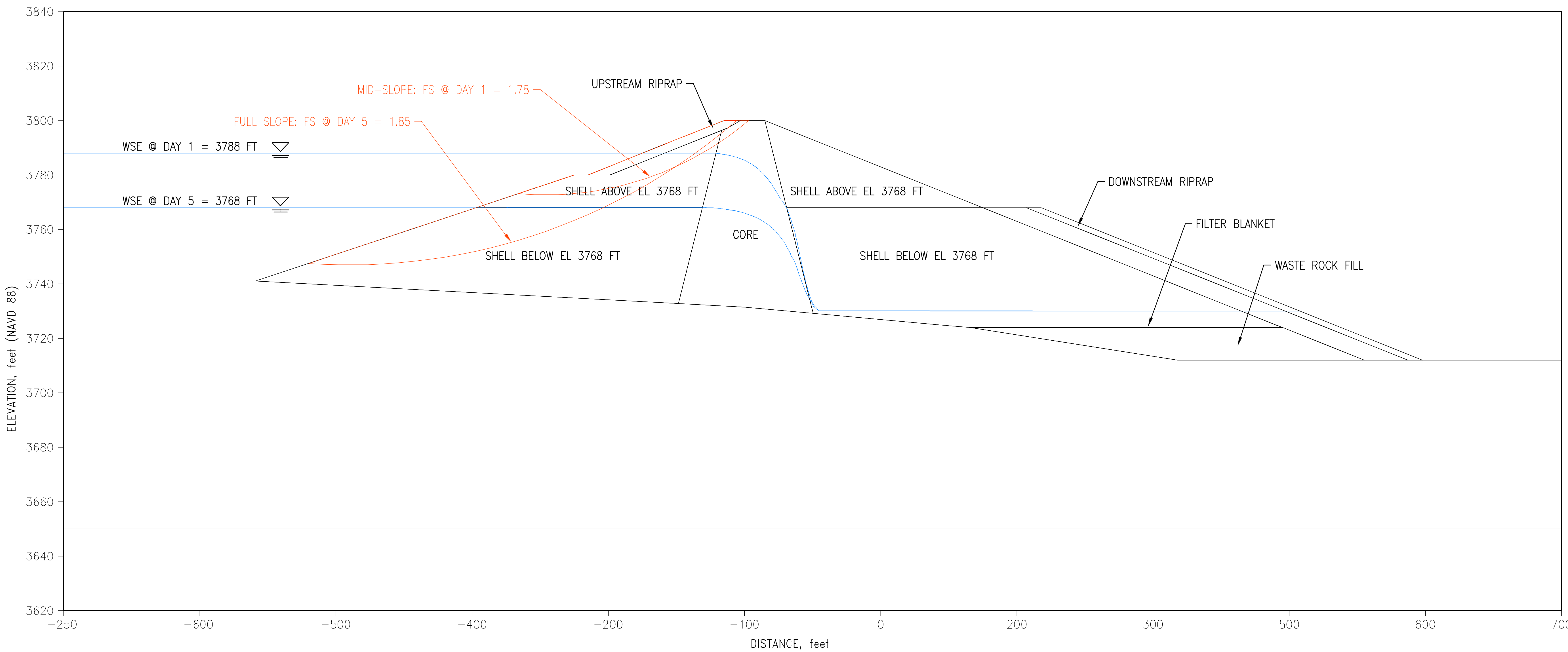
PLOT BY: BENJAMIN.CHOY - Sep 12, 2017 - 4:29:57pm
DRAWING: K:\400-Technical\430 Engineering\3.2 Tech Assess\5. Embank Stability\JC Boyle\Figures\Figure 11.dwg

PROJECT NUMBER 60537920	PREPARED BY BENJAMIN CHOY	KLAMATH RIVER DAM REMOVAL	RAPID DRAWDOWN STABILITY ANALYSIS JC BOYLE DAM INSTANTANEOUS DRAWDOWN SCENARIO NO PORE PRESSURE DISSIPATION	FIGURE 9
REVISION 0	CHECKED BY KANAX KANAGALINGAM			



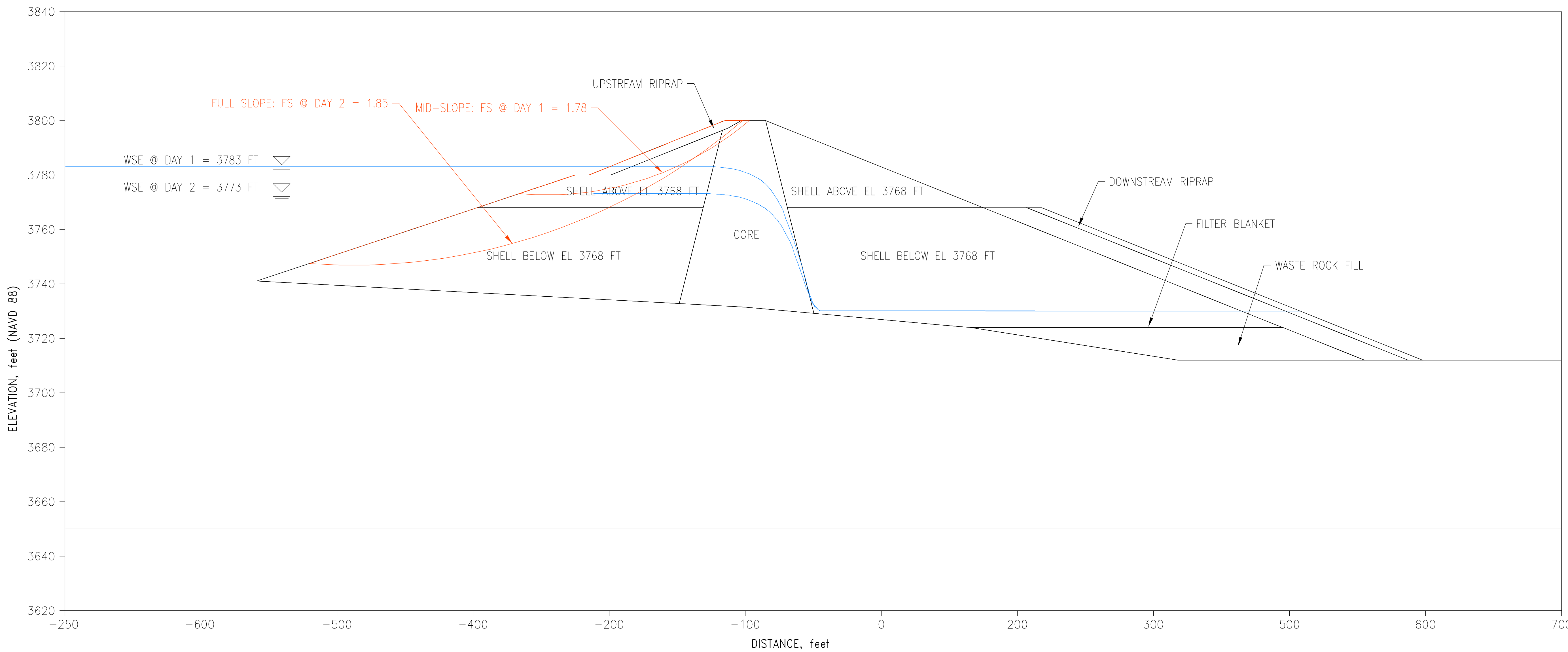
PLOT BY: BENJAMIN.CHOY – Sep 11, 2017 – 10:47:21am
DRAWING: K:\400-Technical\430 Engineering\3.2 Tech Assess\5. Embank Stability\JC Boyle\Figures\Figure 12.dwg

PROJECT NUMBER 60537920	PREPARED BY BENJAMIN CHOY	KLAMATH RIVER DAM REMOVAL	RAPID DRAWDOWN STABILITY ANALYSIS JC BOYLE DAM SLOW DRAWDOWN SCENARIO 2 FT/DAY DRAWDOWN RATE	FIGURE 10
REVISION 0	CHECKED BY KANAX KANAGALINGAM			



PLOT BY: BENJAMIN.CHOY – Sep 11, 2017 – 10:53:07am
DRAWING: K:\400-Technical\430 Engineering\3.2 Tech Assess\5. Embank Stability\JC Boyle\Figures\Figure 13.dwg

PROJECT NUMBER	PREPARED BY	KLAMATH RIVER DAM REMOVAL	RAPID DRAWDOWN STABILITY ANALYSIS JC BOYLE DAM SLOW DRAWDOWN SCENARIO 5 FT/DAY DRAWDOWN RATE	FIGURE
60537920	BENJAMIN CHOY			11
REVISION	CHECKED BY			
0	KANAX KANAGALINGAM			



PROJECT NUMBER 60537920	PREPARED BY BENJAMIN CHOY	KLAMATH RIVER DAM REMOVAL	RAPID DRAWDOWN STABILITY ANALYSIS JC BOYLE DAM SLOW DRAWDOWN SCENARIO 10 FT/DAY DRAWDOWN RATE	FIGURE 12
REVISION 0	CHECKED BY KANAX KANAGALINGAM			

PLOT BY: BENJAMIN.CHOY – Sep 11, 2017 – 10:59:18am
DRAWING: K:\400-Technical\430 Engineering\3.2 Tech Assess\5. Embank Stability\JC Boyle\Figures\Figure 14.dwg

Attachment A Triaxial Test Results

CABLE: HANK

ABBOT A. HANKS, INC.
ESTABLISHED 1866
1300 SANSOME STREET · SAN FRANCISCO 11, CALIFORNIA · EXBROOK 7-2464

Engineers
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Spectrographers
Soils and Foundations
Consulting · Testing · Inspecting

File No. 1732.1
Lab. No. 46938

May 11, 1960

Mr. W. L. Warren
Assistant Chief Engineer
The California Oregon Power Company
216 West Main Street
Medford, Oregon

Re: Iron Gate Dam
Soil Samples

Dear Sir:

Enclosed are the findings from tests performed on soil samples marked Hole No. 1, which is the only sample for which all tests are complete. Tests of remaining samples are in various stages of completion.

As you may recall from your recent visit, there appeared to be a possibility that samples from Holes 2 and 3 had been mislabelled. It now appears that all samples marked Holes 2 and 3 are nearly identical, and we are performing further tests to distinguish between them. It is quite possible that these soils are exceptionally sensitive to seasoning period, owing to the porous nature of the parent rock, and that test results, particularly optimum moisture content, are influenced by the length of seasoning period. We have completed triaxial shear and consolidation tests on the sample labelled Hole No. 2, but are not yet certain that the samples were compacted at optimum moisture content and maximum density.

We shall advise you of results of our identification tests, and shall forward sets of test data as they are completed.

Very truly yours,

ABBOT A. HANKS, INC.

L O Long

L. O. Long

LOL:hms
Encls.
Reports to:
3-The California Oregon Power Company

Iron Gate Dam
Klamath River
File No. 1732.1

Abbot A. Hanks, Inc.
Lab. No. 46938
May 10, 1960

TEST RESULTS

Hole No. 1.
Specific Gravity: 2.77.

Triaxial Shear Test

	Sample A	Sample B	Sample C
Chamber Pressure, psi	15	50	80
Unit Dry Weight at Compaction, lb/ft^3	103.3	103.6	104.3
Moisture Content at Compaction, %	21.3	22.1	20.8
Unit Dry Weight at Test, lb/ft^3	103.0	108.6	110.5
Moisture Content at Test, %	23.7	21.6	20.4
Degree of Saturation at Test, %	97	100+	100
Maximum Deviator Stress, psi	36	60	77
Pore Pressure at Max. Deviator Stress, psi	4	9	23

Permeability Test
(Constant Head Test)

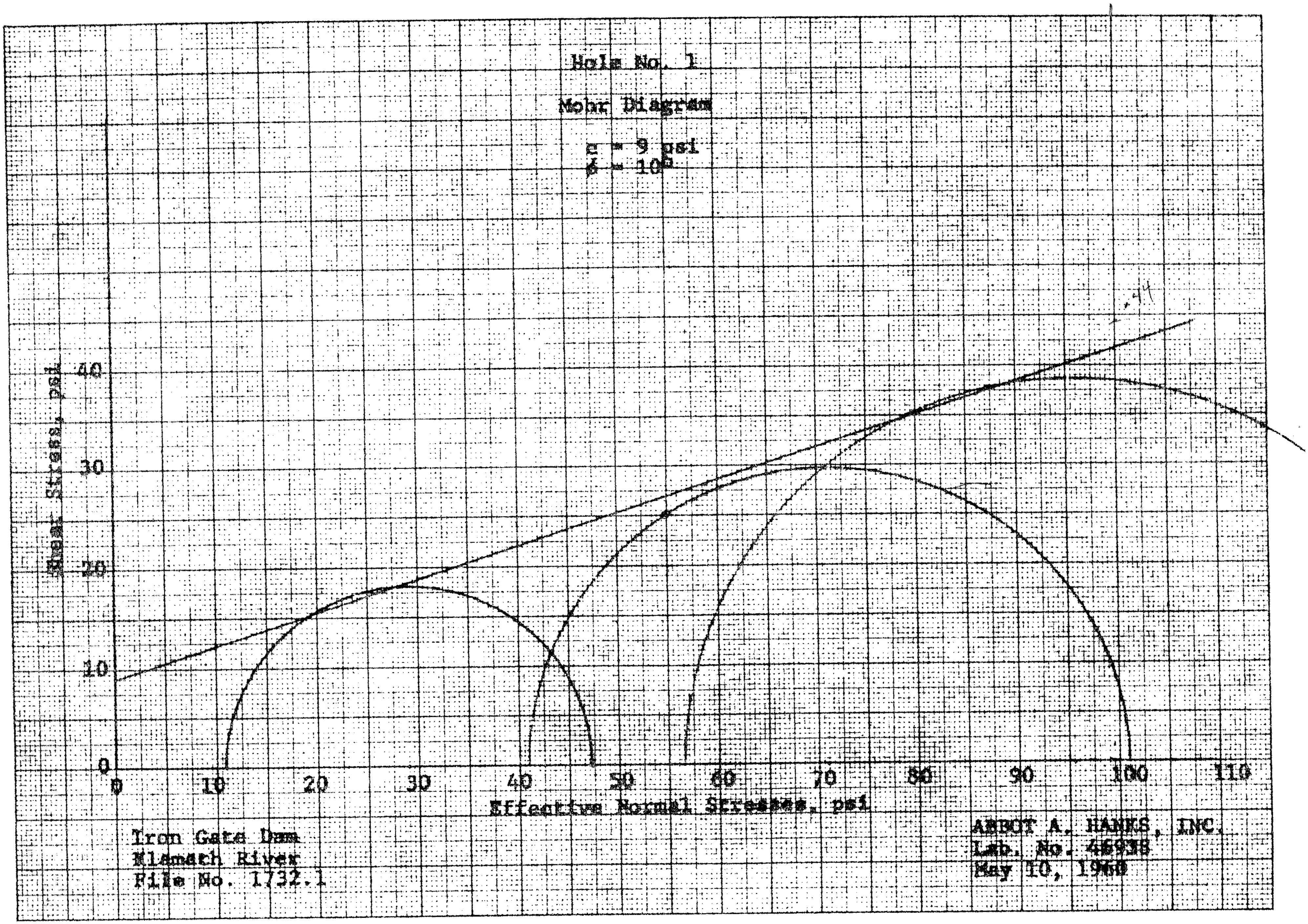
Unit Dry Weight at Compaction, lb/ft	100.6
Moisture Content at Compaction, %	23.6
Moisture Content at Test, %	24.4
Degree of Saturation at Test, %	95
Permeability coefficient, ft per yr	Less than .01
" " , cm/sec	Less than 10^{-8}

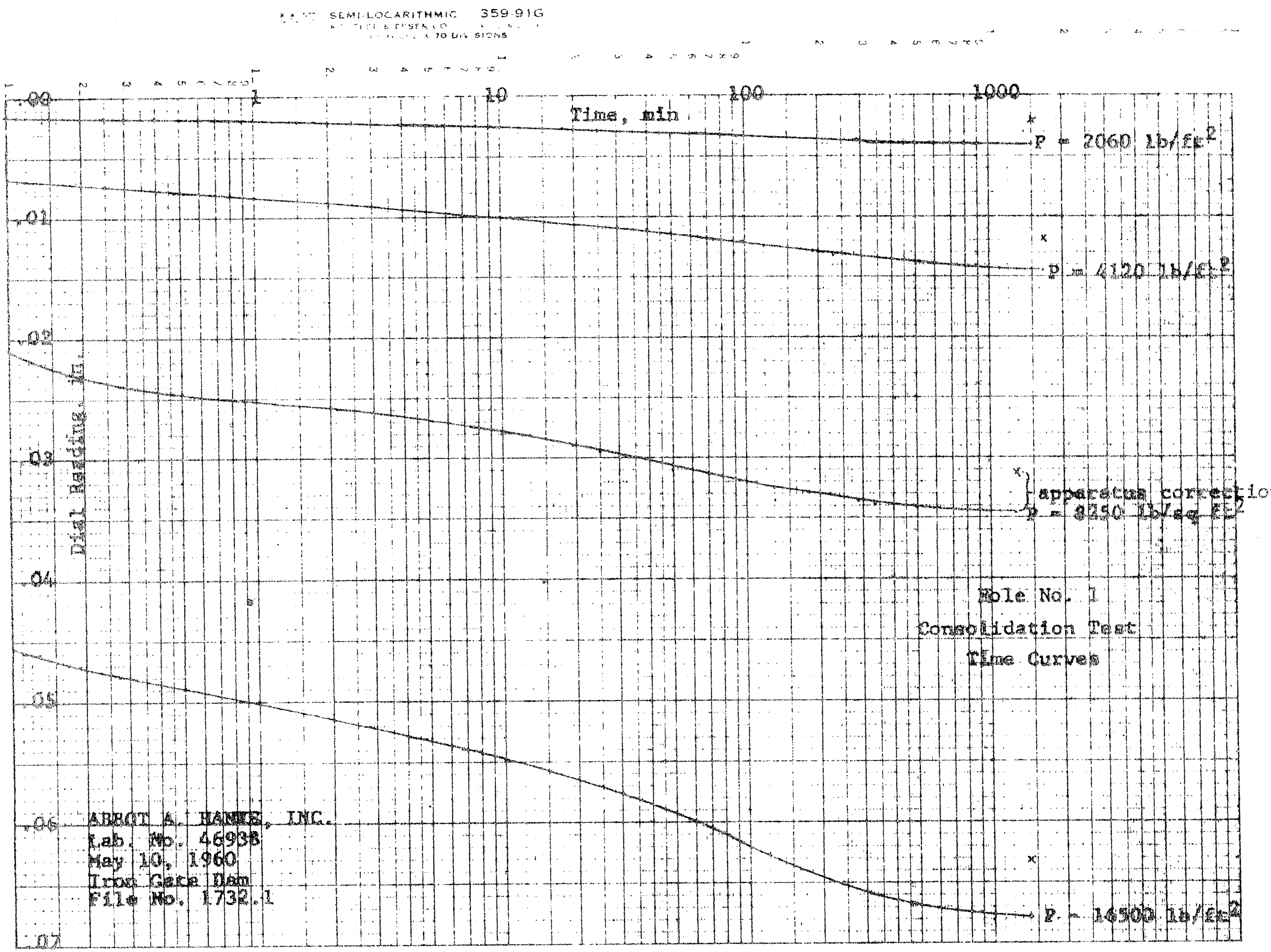
~~[illegible]~~ ~~[illegible]~~

~~optimum moisture — 22% — 22.7~~

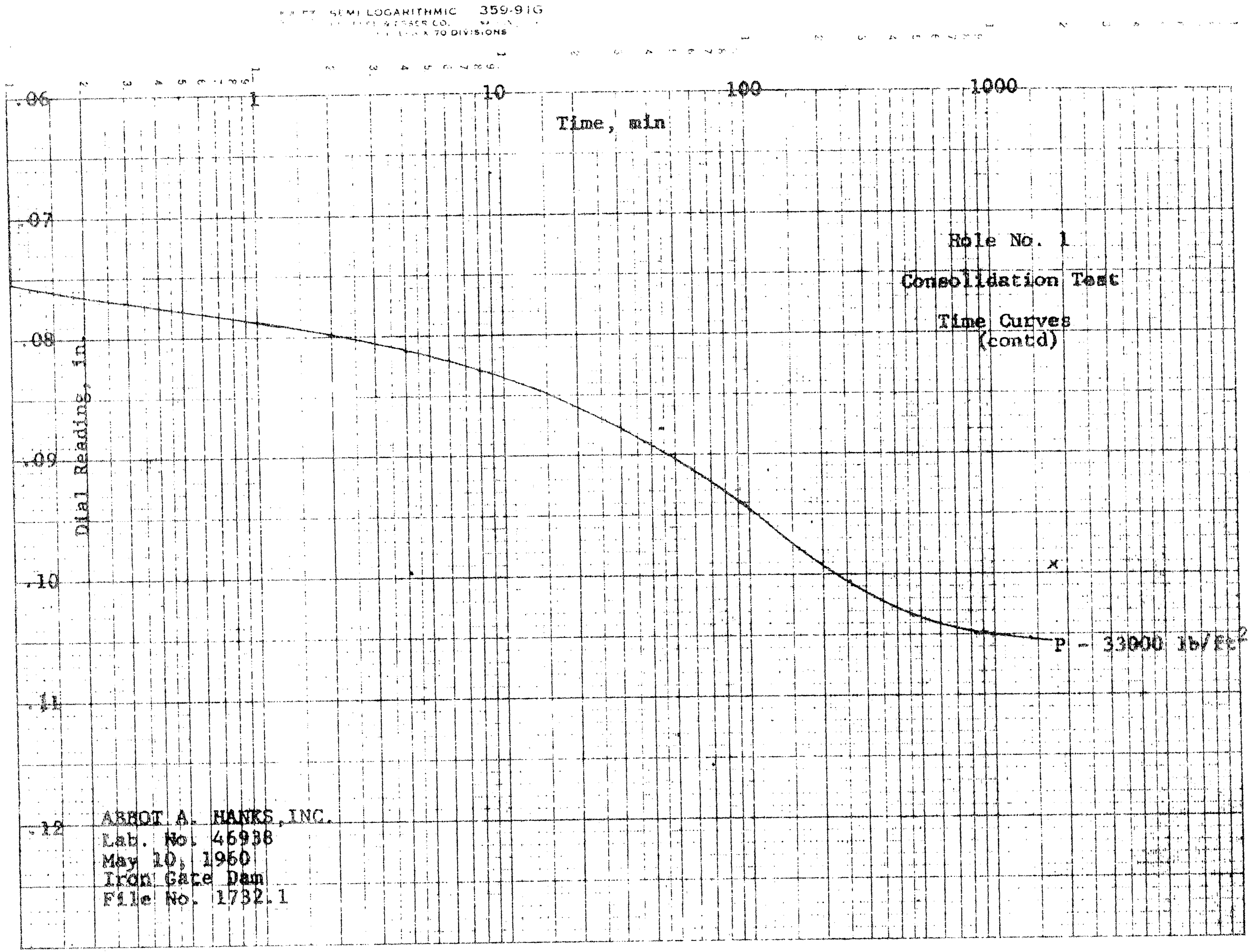
~~max dry density — 102.8 lb/cu ft. — 102~~

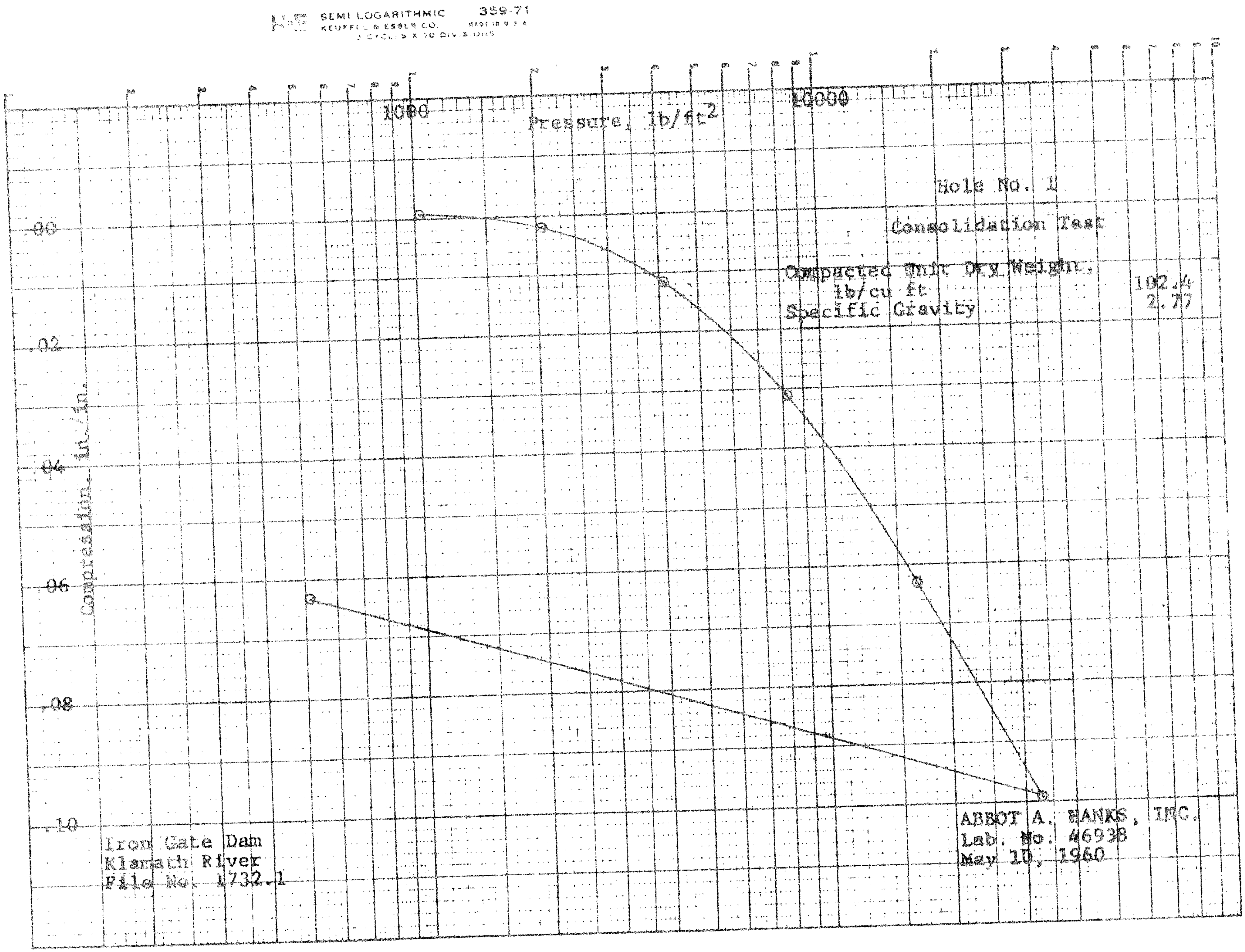
K+E 10 X 10 TO THE CM KEUFFEL & ESSER CO. 359-14G MADE IN U.S.A.





K&E SEMI-LOGARITHMIC 359-91G
Time, min
Dial Reading, in.
P = 2060 lb/ft²
P = 4120 lb/ft²
apparatus correction
P = 8250 lb/sq ft²
P = 16500 lb/ft²
Hole No. 1
Consolidation Test
Time Curves
ABBOT A. HANKS, INC.
Lab. No. 46938
May 10, 1960
Iron Gate Dam
File No. 1732.1


SEMI-LOGARITHMIC 359-91G
70 DIVISIONS
Time, min
1
10
100
1000
Dial Reading, in.
.06
.07
.08
.09
.10
.11
.12
Role No. 1
Consolidation Test
Time Curves
(contd)
P - 33000 lb/ft^2
ABBOT A. HANKS, INC.
Lab. No. 46938
May 10, 1960
Iron Gate Dam
File No. 1732.1

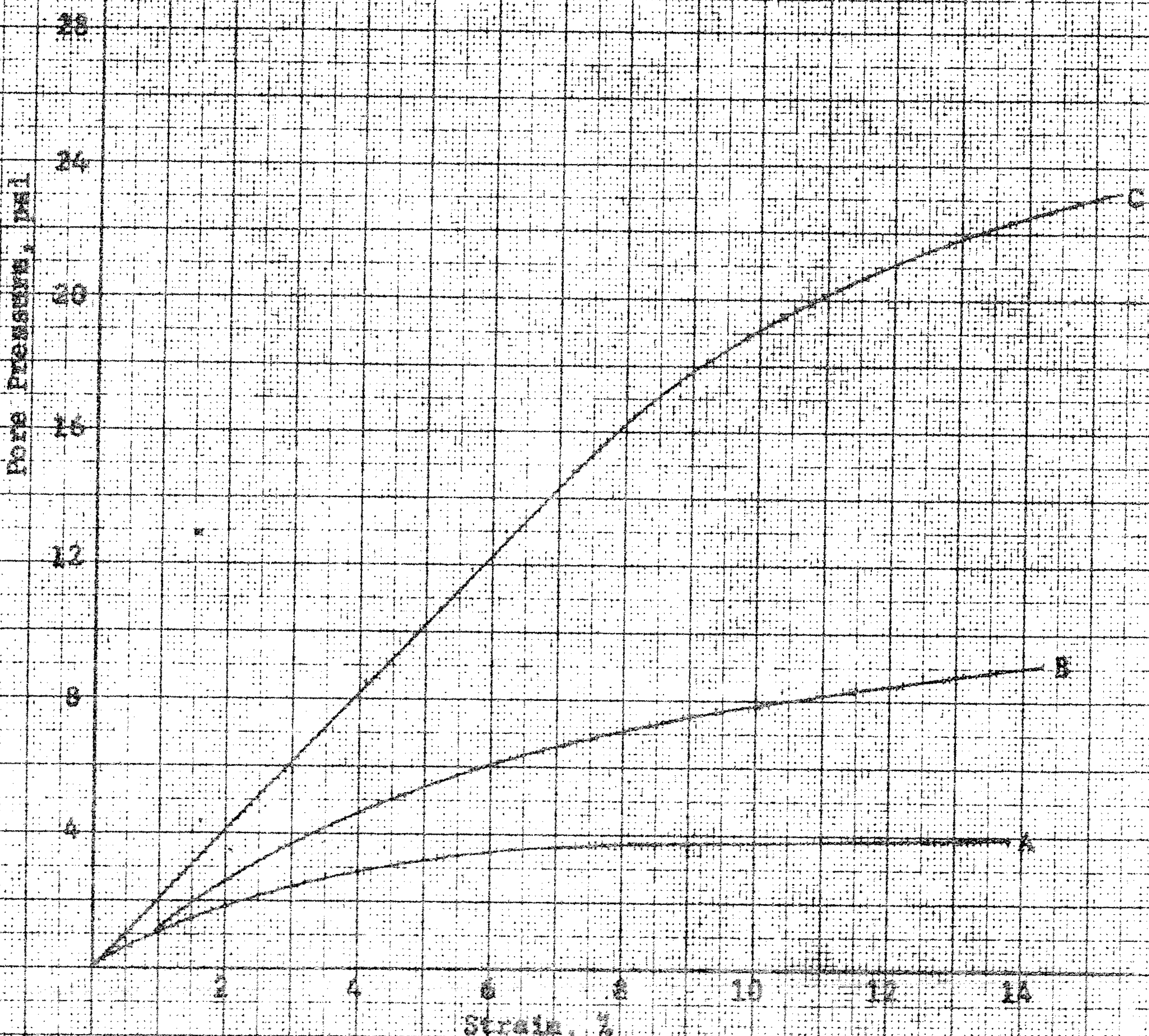


SEMI LOGARITHMIC 359-71
KEUFFEL & ESSER CO.
3 CYCLES X 70 DIVISIONS
1000
10000
Pressure, lb/ft²
Hole No. 1
Consolidation Test
Compacted Unit Dry Weight, lb/cu ft 102.4
Specific Gravity 2.77
Compression, in./in.
.00
.02
.04
.06
.08
.10
Iron Gate Dam
Klamath River
File No. 1732.1
ABBOT A. HANKS, INC.
Lab. No. 46938
May 10, 1960

Hole No. 1

Triaxial Shear Test

Pore Pressure-Strain Relationships



Iron Gate Dam
Klamath River
File No. 1732.1

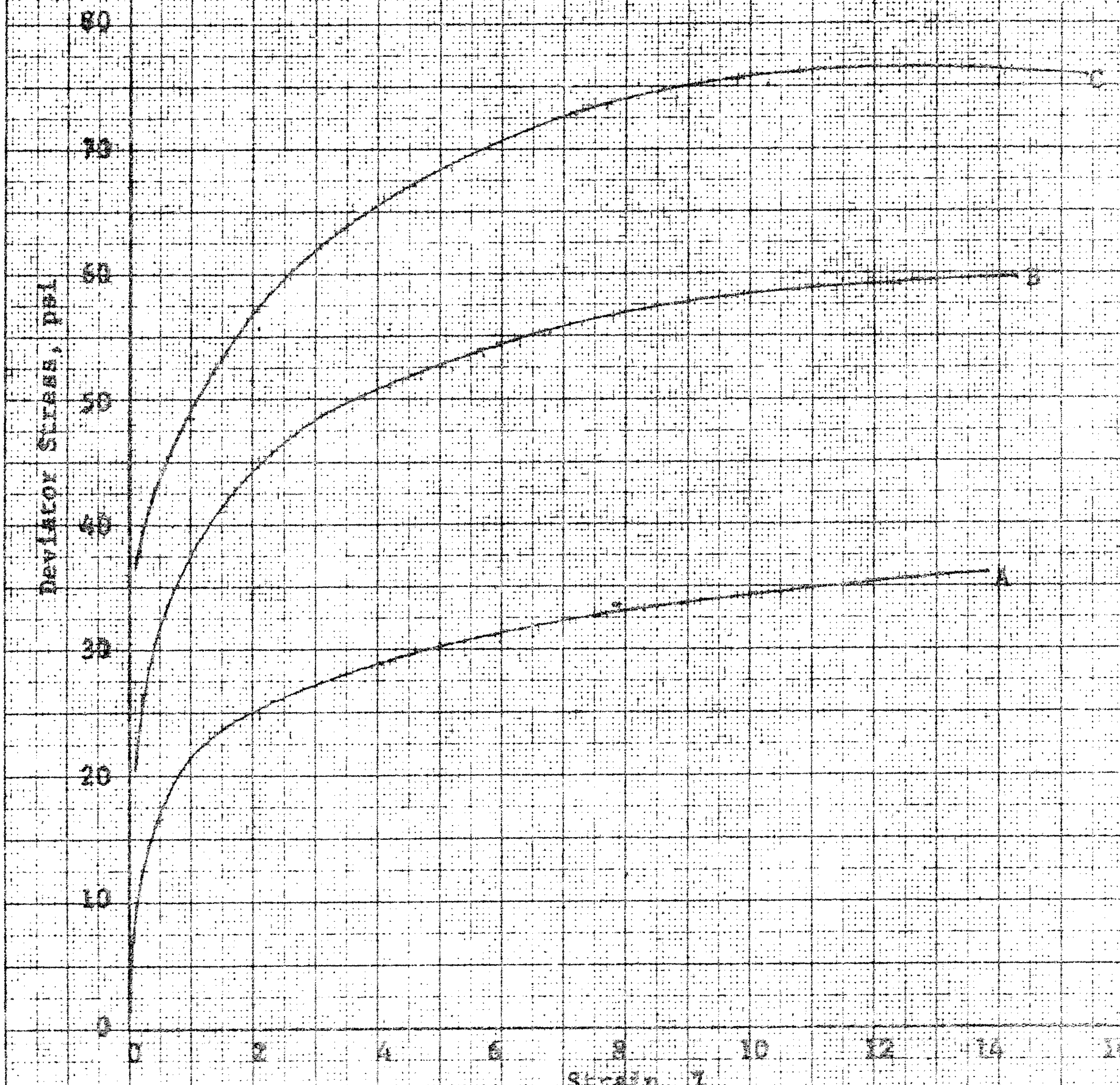
ABBOT A. HANKS, INC.
Lab. No. 46938
May 10, 1960

K&E 10 X 10 TO THE CM. 359-14G
KEUFFEL & ESSER CO. MADE IN U.S.A.

Hole No. 1

Triaxial Shear Test

Stress-Strain Relationships



Iron Gate Dam
Klamath River
File No. 1732.1

ABBOT A. HANKS, INC.
Lab. No. 46938
May 10, 1960

BRANCH LABORATORY 1086 MARTIN AVENUE • SANTA CLARA • CHERRY 8 3262
BRANCH OFFICE 10 DE LUCA PLACE • SAN RAFAEL • GLENWOOD 4 8650

CABLE HANX

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ESTABLISHED 1866
1300 SANSOME STREET • SAN FRANCISCO 11, CALIFORNIA • EXBROOK 7 2464

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Soils and Foundations
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File No. 1732.1
Lab. No. 46938

June 29, 1960

Mr. W. L. Warren
Assistant Chief Engineer
The California Oregon Power Company
216 West Main Street
Medford, Oregon

Re: Iron Gate Dam
Soil Samples

Dear Mr. Warren:

Enclosed are results of triaxial tests that were performed on Sample No. 2 before it was noted that this sample required an exceptionally small compactive effort relative to the other samples submitted.

We are also enclosing miscellaneous test results not shown on previously submitted reports.

If you require additional tests of Sample No. 2, we should have a complete new sample of about 100 lb.

Very truly yours,

ABBOT A. HANKS, INC.

L. O. Long

L. O. Long

LOL:hms
Encls.
Reports to:
15-The California Oregon Power Company

Iron Gate Dam
Klamath River
File No. 1732.1

Abbot A. Hanks, Inc.
Lab. No. 46938
July 1, 1960

TEST RESULTS

Hole No. 2.
Specific Gravity: 2.77.

Triaxial Shear Test

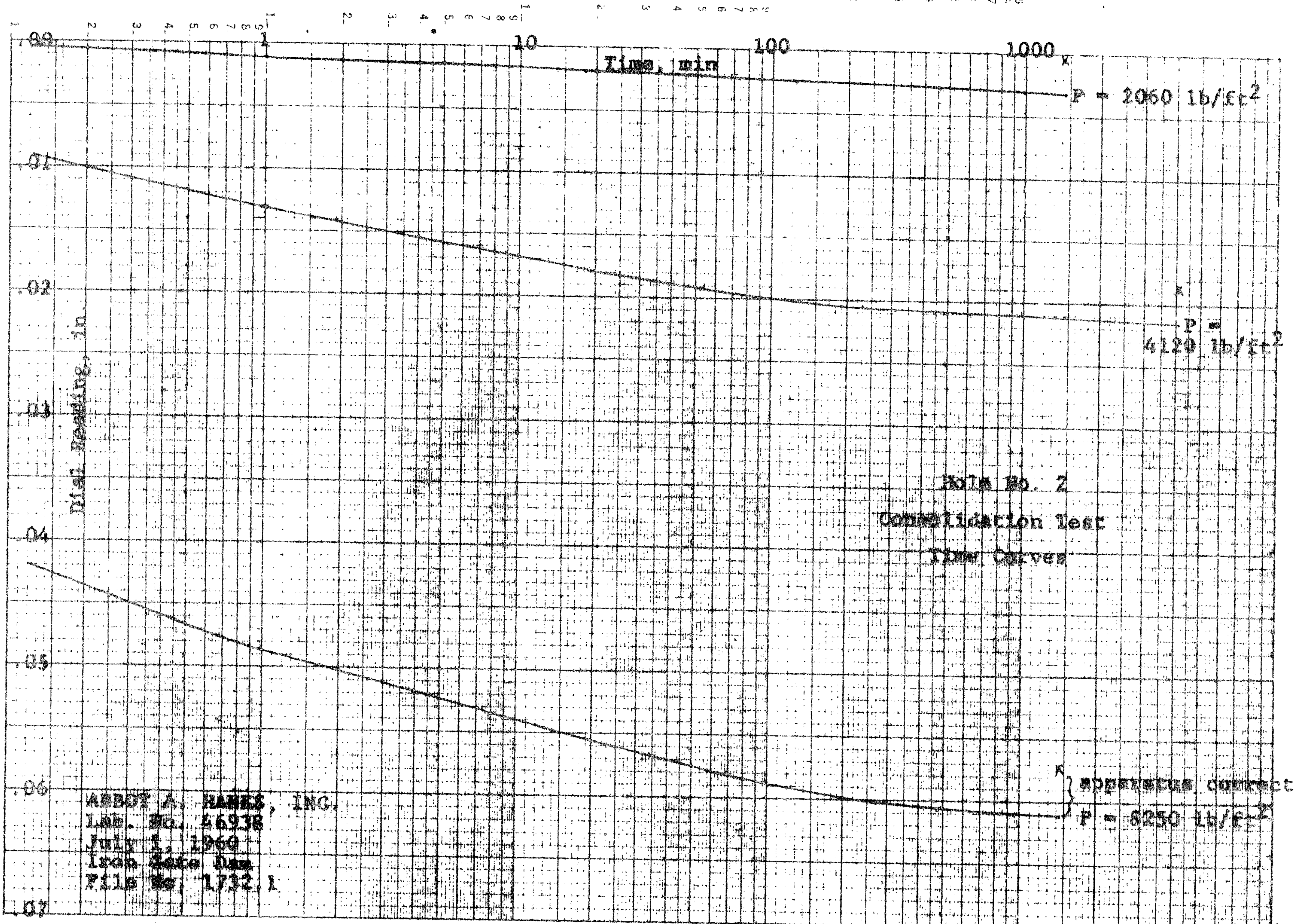
	Sample A	Sample B	Sample C	Sample D
Chamber Pressure, psi	15	50	50	80
Unit Dry Weight at Compaction, lb/ft^3	98.0	95.1	98.8	99.3
Moisture Content at Compaction, %	21	21	21	21
Unit Dry Weight at Test, lb/ft^3	99.4	109.0	106.3	100.2
Moisture Content at Test, lb/ft^3	24.4	22.3	21.6	20.3
Degree of Saturation at Test, %	93	100+	97	93
Maximum Deviator Stress, psi	19	40	45	69
Pore Pressure at Maximum Deviator Stress, psi	8	26	24	15

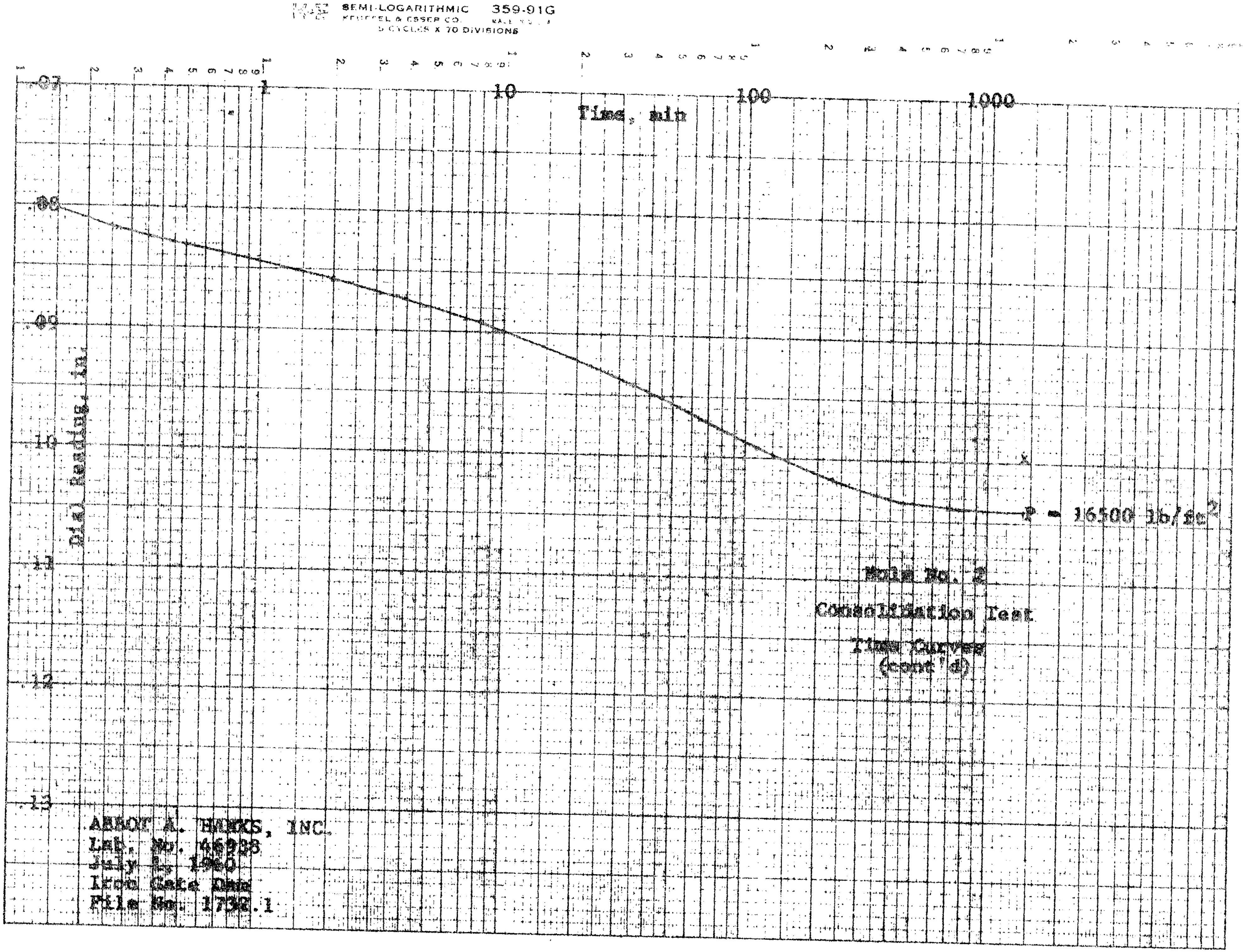
~~Report — letter~~

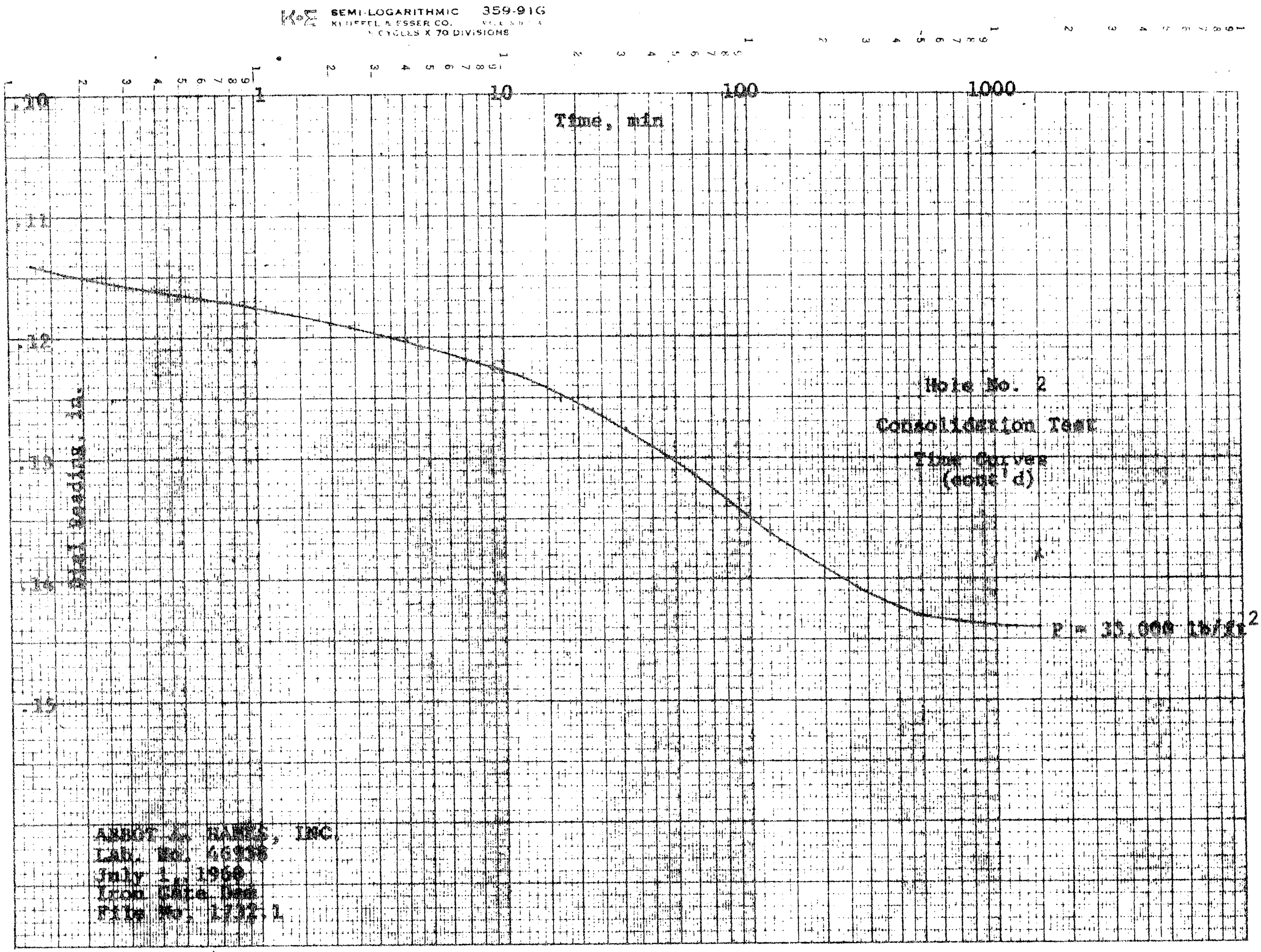
~~Optimum moisture 21 21~~

~~max dry density 102.8 103~~

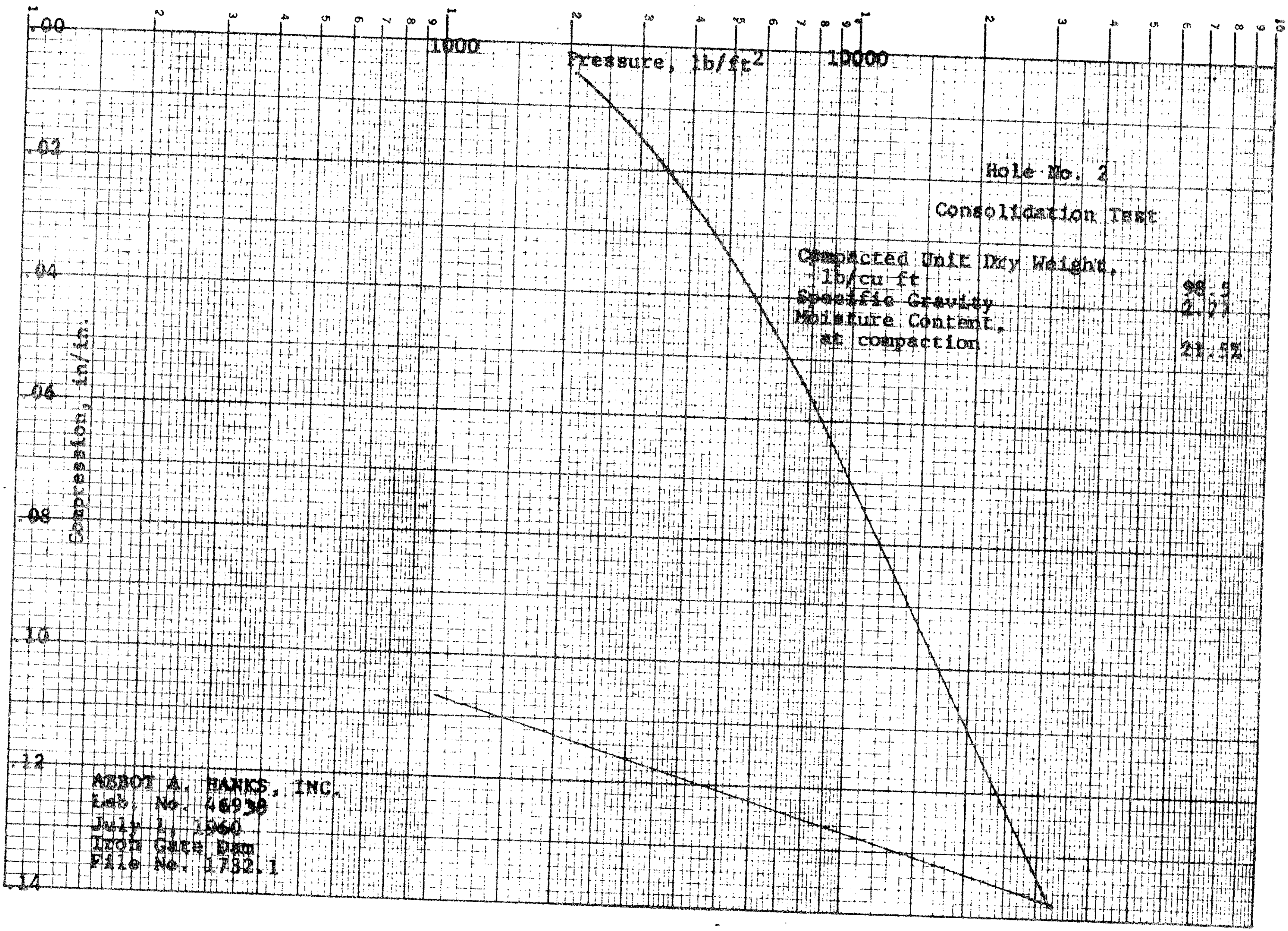
K&E SEMI-LOGARITHMIC 359-91G
KEUFFEL & ESSER CO.
3 CYCLES X 70 DIVISIONS




SEMI-LOGARITHMIC 359-91G
KEUFFEL & ESSER CO.
5 CYCLES X 70 DIVISIONS
Time, min
1
10
100
1000
Dial Reading, in.
.07
.08
.09
.10
.11
.12
.13
P = 16500 lb/ft²
Hole No. 2
Consolidation Test
Time Curves
(cont'd)
ABBOT A. HANKS, INC.
Lab. No. 46988
July 8, 1960
Iron Gate Dam
File No. 1732.1


K+E SEMI-LOGARITHMIC 359-91G
KEUFFEL & ESSER CO.
5 CYCLES X 70 DIVISIONS
Time, min
1
10
100
1000
Dial Reading, in.
.10
.11
.12
.13
.14
.15
Hole No. 2
Consolidation Test
Time Curves
(cont'd)
P = 35,000 lb/ft²
ABBOT A. HANKS, INC.
Lab. No. 46938
July 1, 1960
Iron Gate Dam
File No. 1732.1

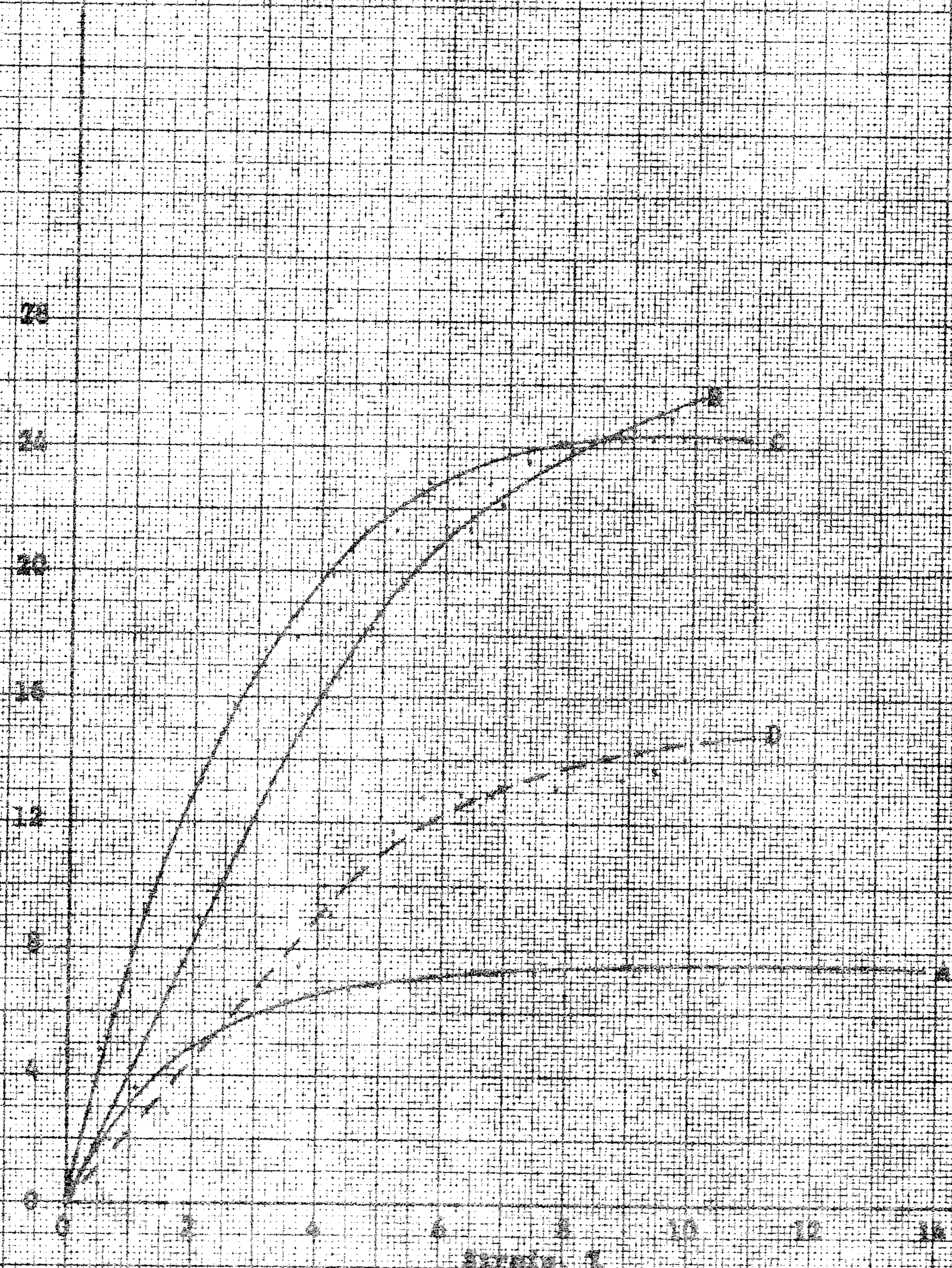
K+E SEMI-LOGARITHMIC 359-71
KEUFFEL & ESSER CO. MADE IN U.S.A.
3 CYCLES X 70 DIVISIONS



Hole No. 1

Triaxial Shear Test

Pore Pressure-Strain Relationships



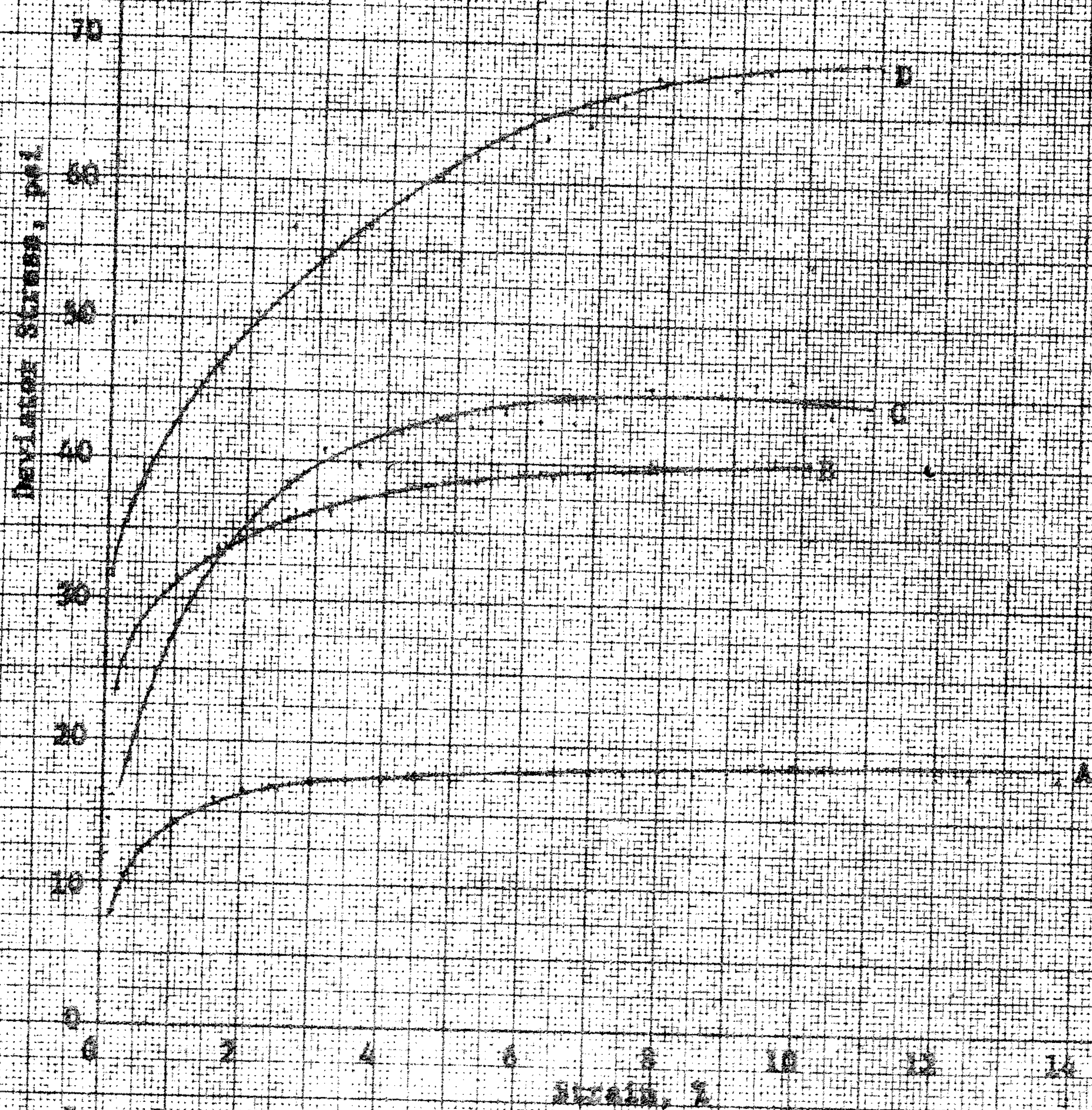
Iron Gate Dam
Klamath River
File No. 1732.1

Abbot A. Hanks, Inc.
Lab. No. 15629
July 1, 1965

Hole No. 2

Triaxial Shear Test

Stress-Strain Relationships

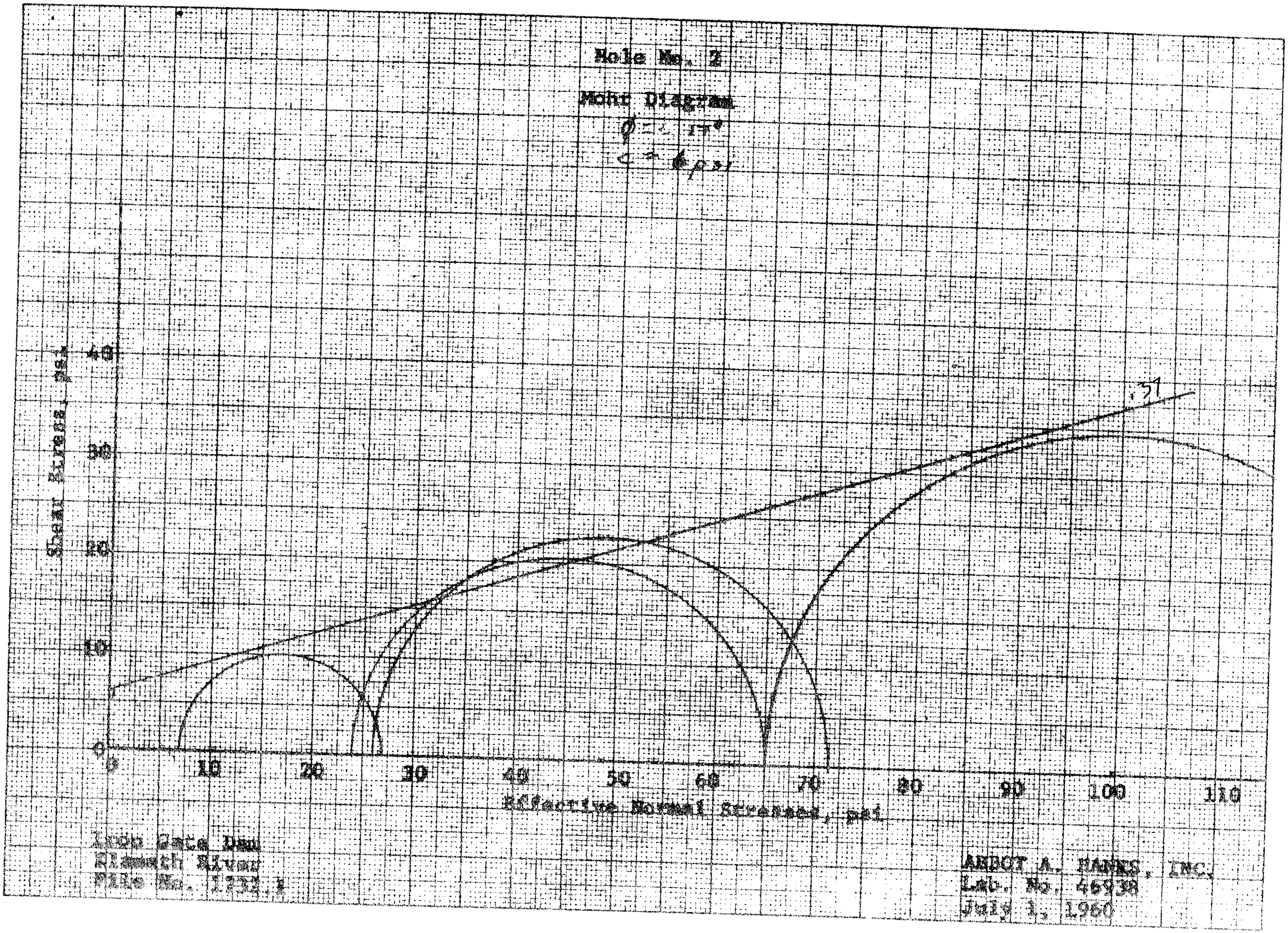


Iron Gate Dam
Klamath River
File No. 1732.1

ABBOT A. HANKS, INC.
Lab. No. 46938
July 1, 1960

K+E 10 X 10 TO THE CM. 359-14G KEUFFEL & ESSER CO. MADE IN U.S.A.

K•E 10 X 10 TO THE CM. 359-14G
KEUFFEL & ESSER CO. MADE IN U.S.A.



BRANCH LABORATORY 1086 MARTIN AVENUE • SANTA CLARA • CHERRY 8-5262
BRANCH OFFICE 10 DE LUCA PLACE • SAN RAFAEL • GLENWOOD 4-8650

CABLE, HANK

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File No. 1732.1
Lab. No. 46938

June 3, 1960

Mr. W. L. Warren
Assistant Chief Engineer
The California Oregon Power Company
216 West Main Street
Medford, Oregon

Re: Iron Gate Dam
Soil Samples

Dear Mr. Warren:

Enclosed are the findings from tests performed on soil samples marked Hole No. 3.

Very truly yours,

ABBOT A. HANKS, INC.

Donald W Radbruch

Donald W. Radbruch

hms
Encls.
Reports to:
3-The California Oregon Power Company

Iron Gate Dam
Klamath River
File No. 1732.1

Abbot A. Hanks, Inc.
Lab. No. 46938
June 3, 1960

TEST RESULTS

Hole No. 3
Specific Gravity: 2.76

Triaxial Shear Test

	Samples A	Samples B	Samples C
Chamber Pressure, psi	15	50	80
Unit Dry Weight at Compaction, lb/ft^3	104.4	104.5	103.5
Moisture Content at Compaction, %	21.9	22.0	22.1
Unit Dry Weight at Test, lb/ft^3	105.3	107.5	109.2
Moisture Content at Test, lb/ft^3	24.0	22.4	23.5
Degree of Saturation at Test, %	100+	100+	100+
Maximum Deviator Stress, psi	34	59	79
Pore Pressure at Max. Deviator Stress, psi	2	5	2

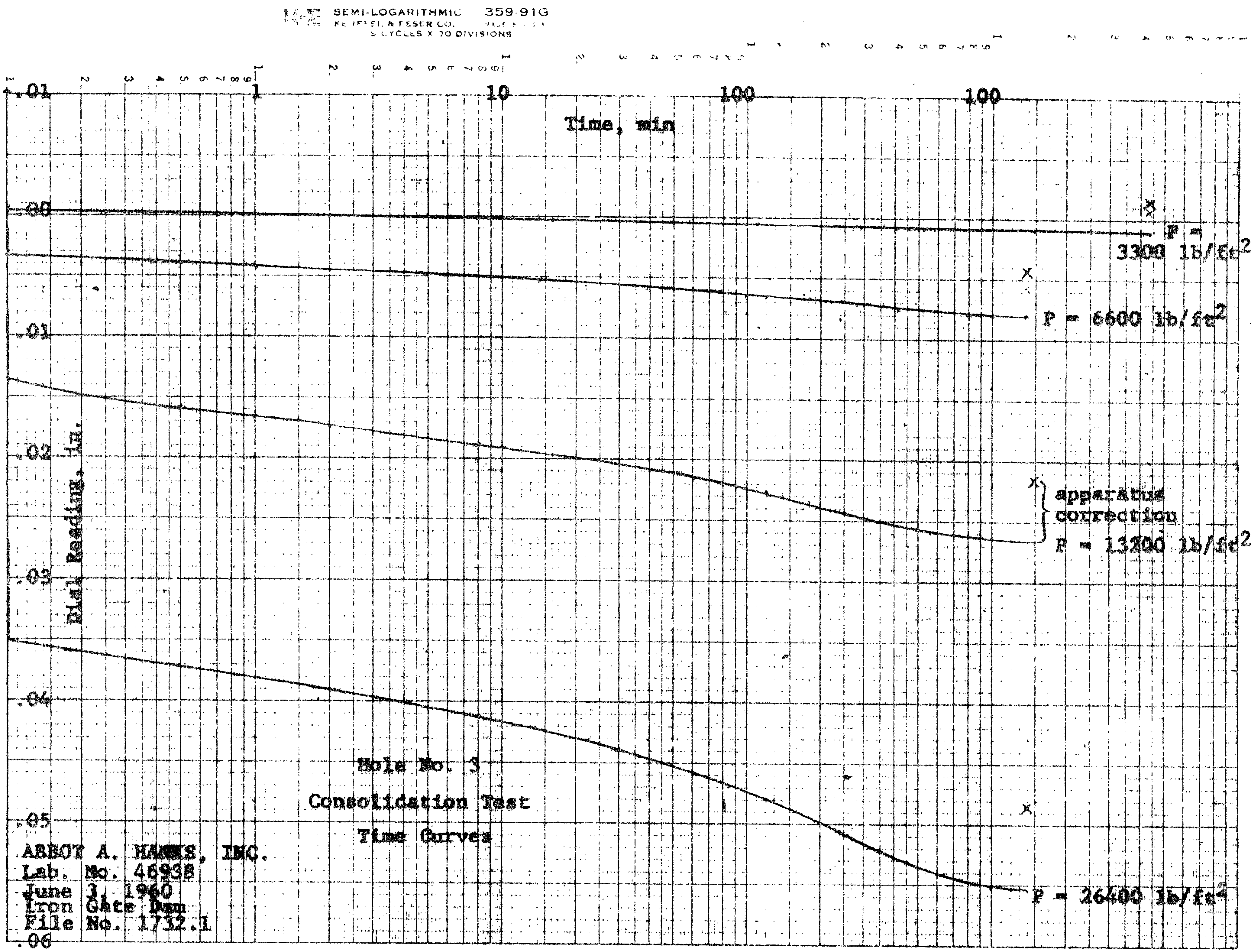
Permeability Test
(Constant Head Test)

Unit Dry Weight at Compaction, lb/ft^3	106.5
Moisture Content at Compaction, %	22.0
Moisture Content at Test, %	23.1
Degree of Saturation at Test, %	100+
Permeability Coefficient, ft per yr	Less than .01
" " , cm/sec	Less than 10^{-8}

~~Report~~ ~~Filter~~

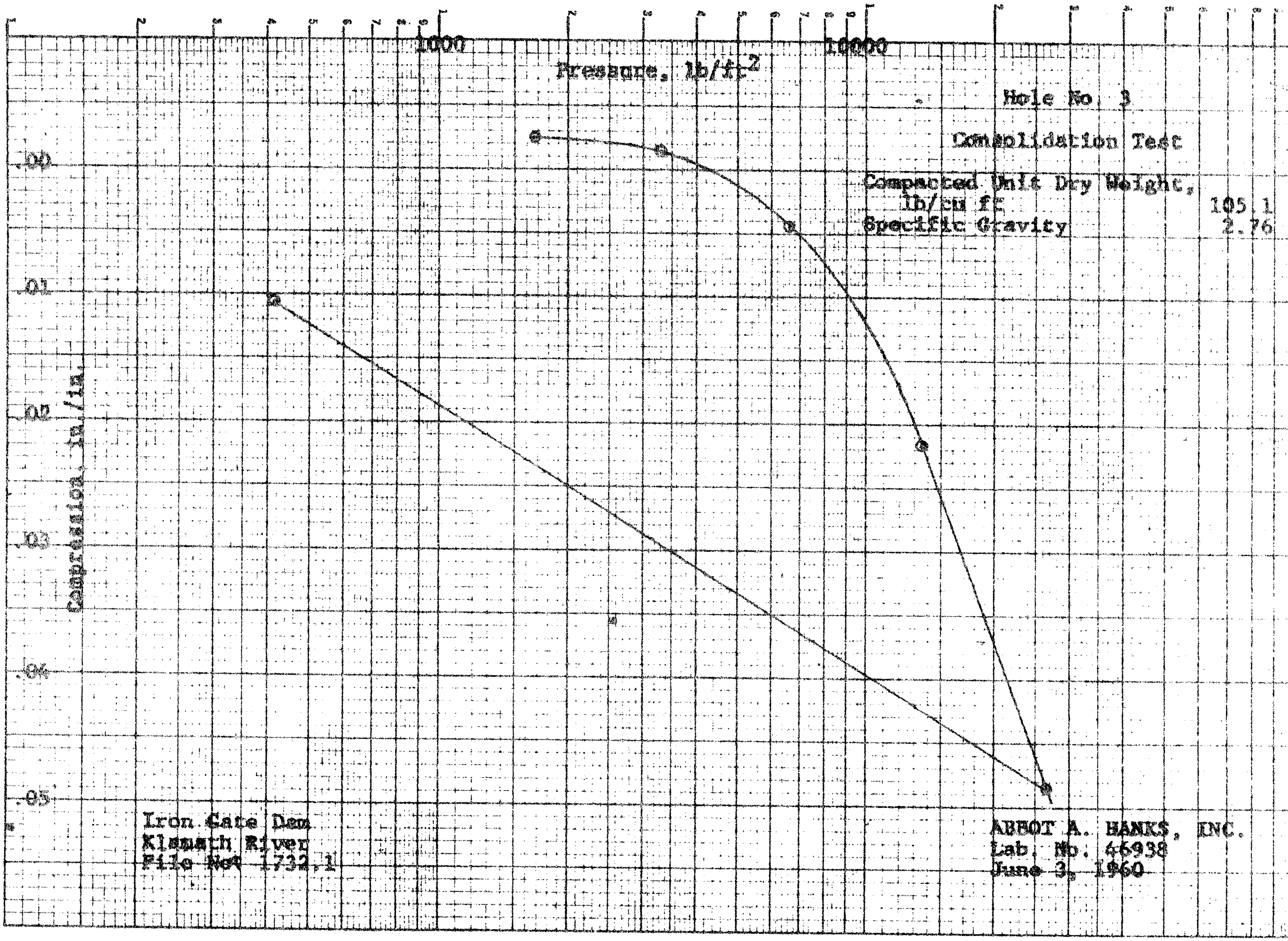
~~Optimum moisture~~ ~~15.3~~ ~~15.5~~

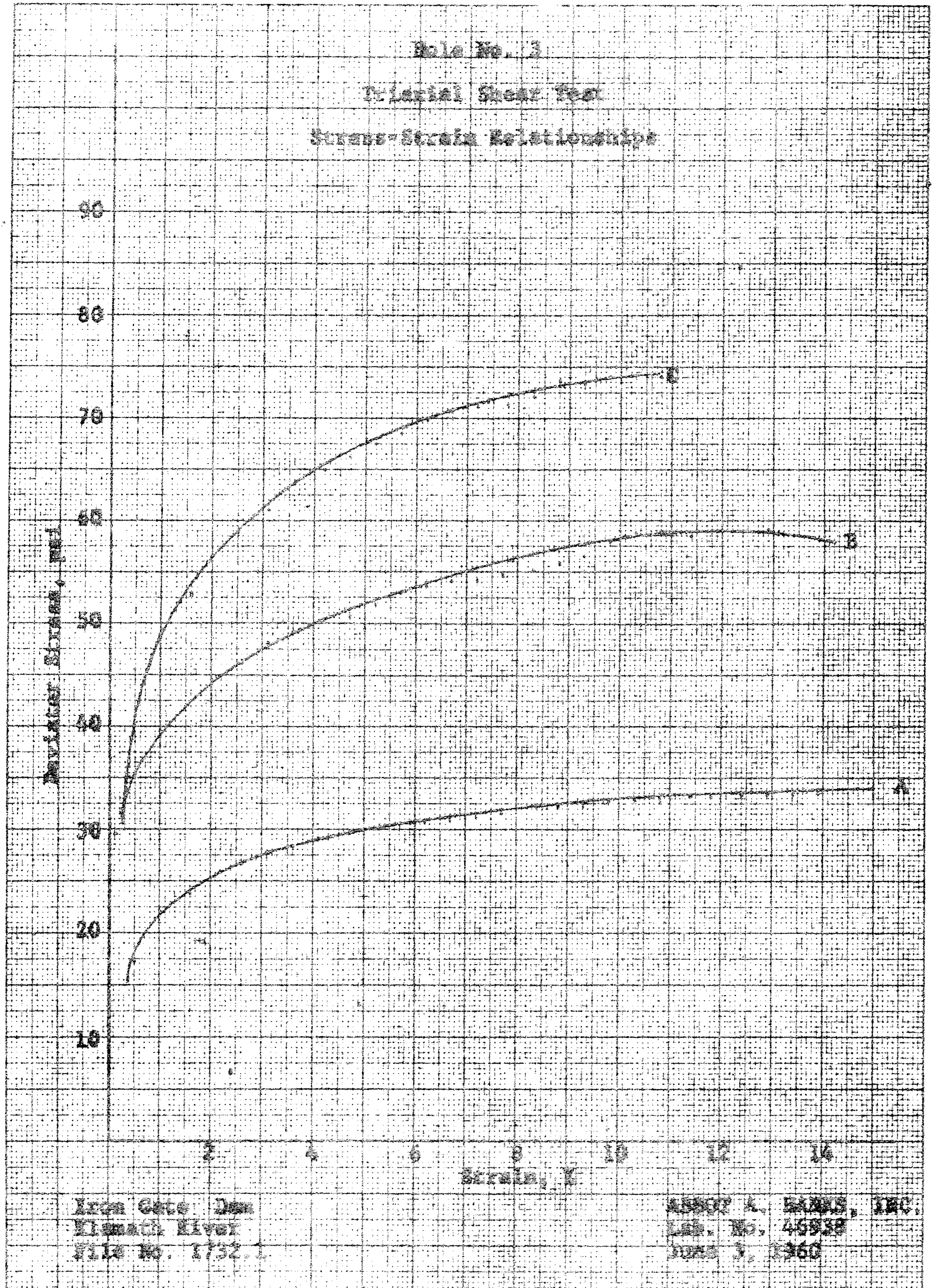
~~Max. Dry density~~ ~~111.6~~ ~~111.5~~



SEMI-LOGARITHMIC 359-91G
KEUFFEL & ESSER CO.
5 CYCLES X 70 DIVISIONS
Time, min
.01
1
10
100
100
Dial Reading, in.
.00
.01
.02
.03
.04
.05
.06
P = 3300 lb/ft2
P = 6600 lb/ft2
apparatus correction
P = 13200 lb/ft2
P = 26400 lb/ft2
Hole No. 3
Consolidation Test
Time Curves
ABBOT A. HANKS, INC.
Lab. No. 46938
June 3, 1960
Iron Gate Dam
File No. 1732.1

K-E SEMI-LOGARITHMIC 359-71
KEUFFEL & ESSER CO. MADE IN U.S.A.
2 CYCLES X 70 DIVISIONS



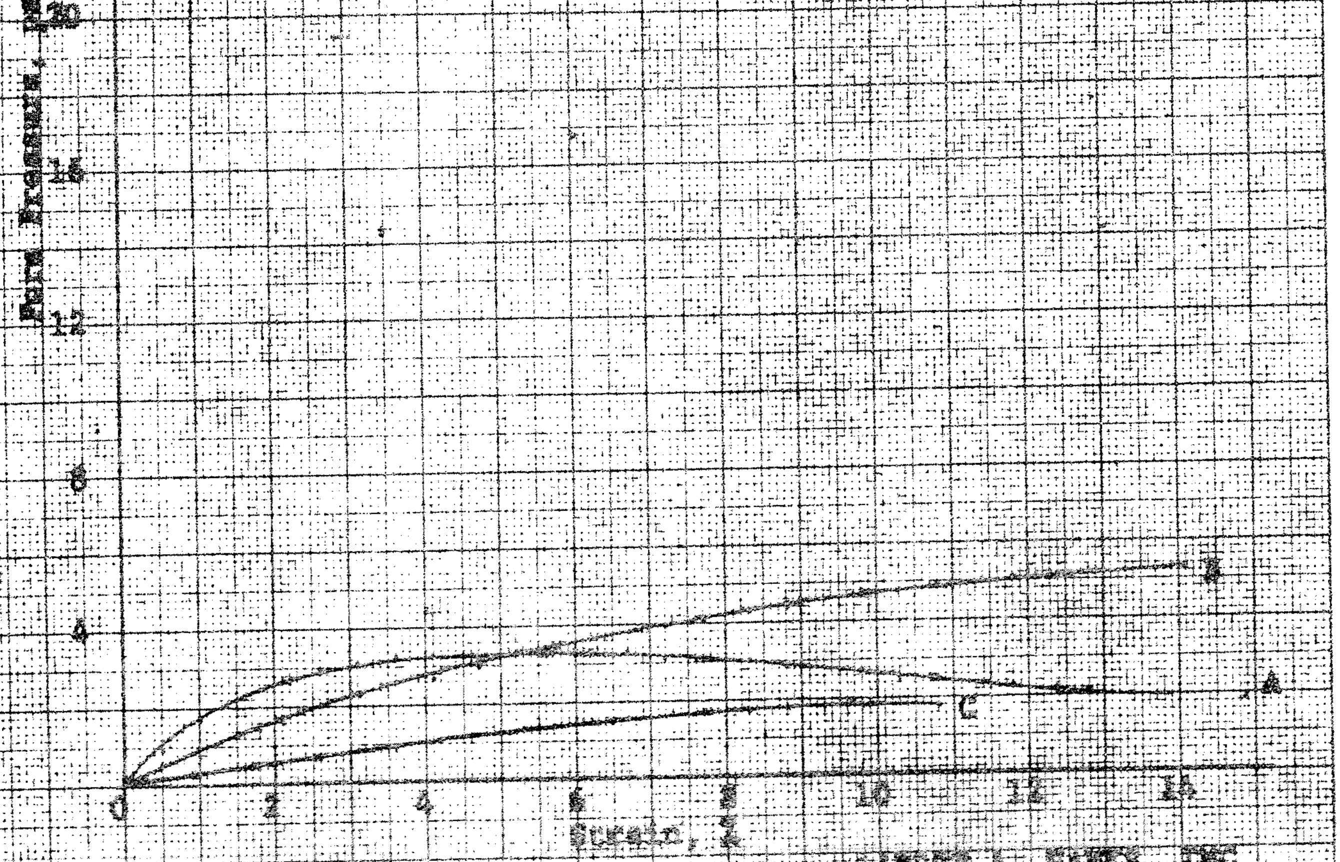


K+E 10 X 10 TO THE CM. 359-14G
KEUFFEL & ESSER CO. MADE IN U.S.A.

Hole No. 3

Triaxial Shear Test

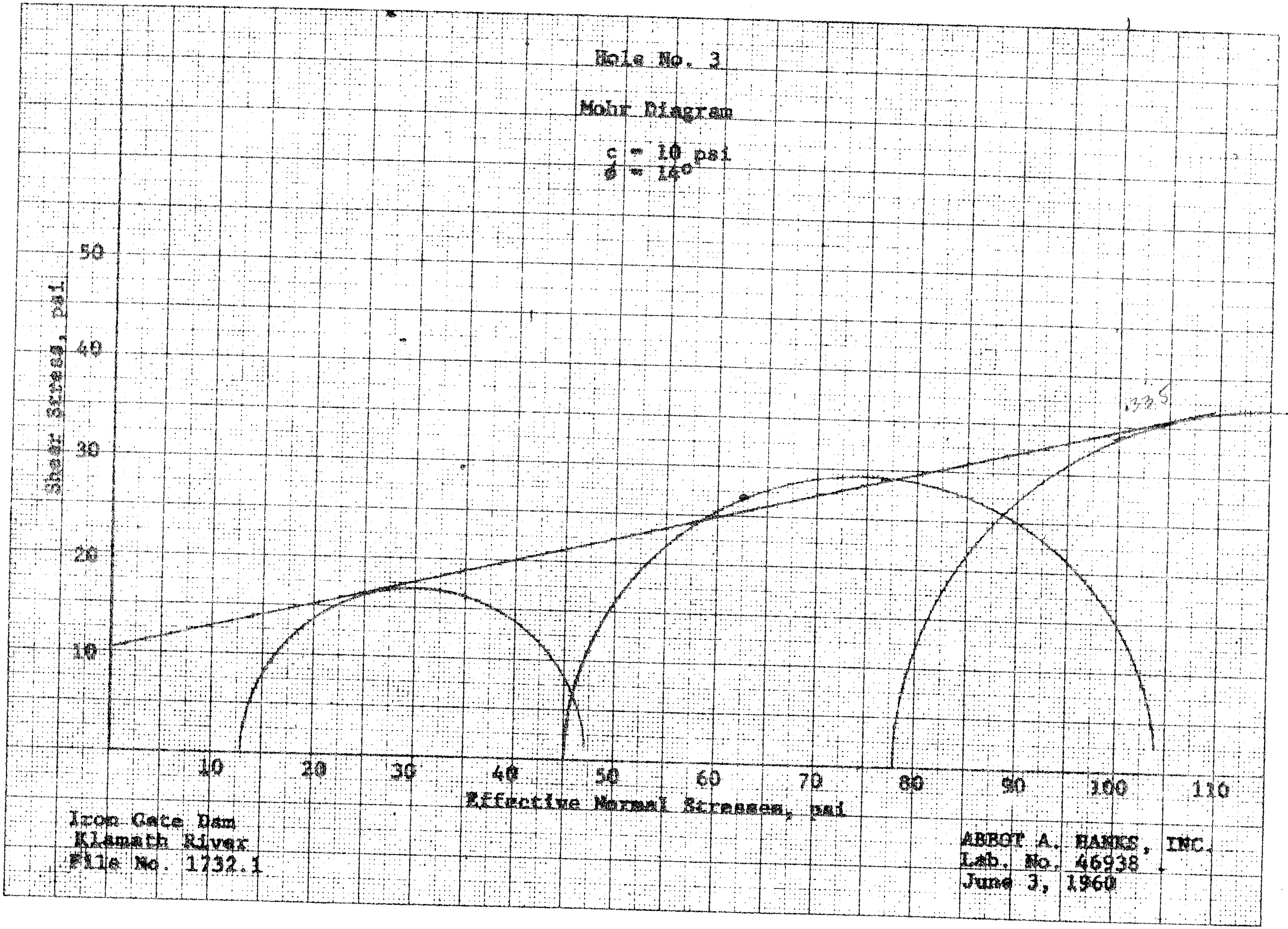
Pore Pressure-Strain Relationships



Iron Gate Dam
Klamath River
File No. 1732.1

[illegible] A. [illegible], INC.
Lab. No. [illegible]
June 2, 1960

K&E 10 X 10 TO THE CM. 359-14G
KEUFFEL & ESSER CO. MADE IN U.S.A.



BRANCH LABORATORY 1056 MARTIN AVENUE • SANTA CLARA • CHERRY 8-5262
BRANCH OFFICE 10 DE LUCA PLACE • SAN RAFAEL • GLENWOOD 4-8650

CABLE HANX

ABBOT A. HANKS, INC.

ESTABLISHED 1866

1300 SANSOME STREET • SAN FRANCISCO 11 CALIFORNIA • EXBROOK 7-2464

Engineers
Assayers
Chemists
Metallurgists
Spectrographers
Soils and Foundations
Consulting Testing Inspecting

File No. 1732.1
Lab. No. 46938

June 9, 1960

Mr. W. L. Warren
Assistant Chief Engineer
The California Oregon Power Company
216 West Main Street
Medford, Oregon

Re: Iron Gate Dam
Soil Samples

Dear Mr. Warren:

Enclosed are the findings from tests performed on soil samples marked Hole No. 4.

You will note that the permeability coefficient of the first sample is 3000 times the permeability coefficient of the second sample. We attribute this large difference to the differences in both density and moisture content at compaction. The second sample, compacted at 16% moisture content, appeared to be somewhat over optimum moisture.

Very truly yours,

ABBOT A. HANKS, INC.

Donald W Radbruch

Donald W. Radbruch

LOL:hms
Encls.
Reports to:
3-The California Oregon Power Company

Iron Gate Dam
Klamath River
File No. 1732.1

Abbot A. Hanks, Inc.
Lab. No. 46938
June 8, 1960

TEST RESULTS

Hole No. 4
Specific Gravity: 2.77

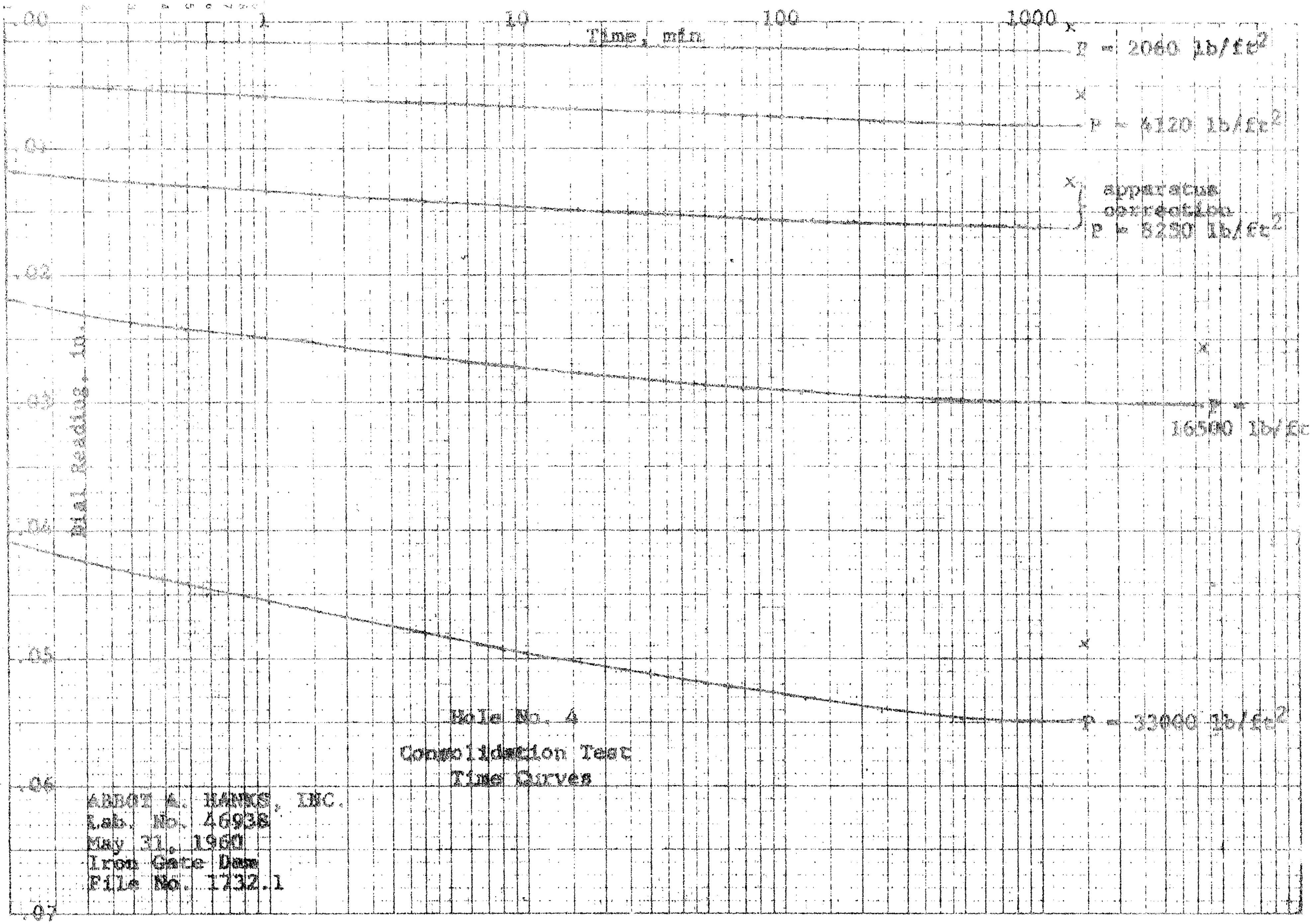
Triaxial Shear Test

	Sample A	Sample B	Sample C
Chamber Pressure, psi	15	50	80
Unit Dry Weight at Compaction, lb/ft^3	112.8	112.3	116.5
Moisture Content at Compaction, %	13.8	13.6	15.4
Unit Dry Weight at Test, lb/ft^3	112.8	114.2	119.4
Moisture Content at Test, lb/ft^3	16.5	17.6	16.0
Degree of Saturation at Test, %	87	96	100
Maximum Deviator Stress, psi	34	85	152
Pore Pressure at Max. Deviator Stress, psi	3	18	9

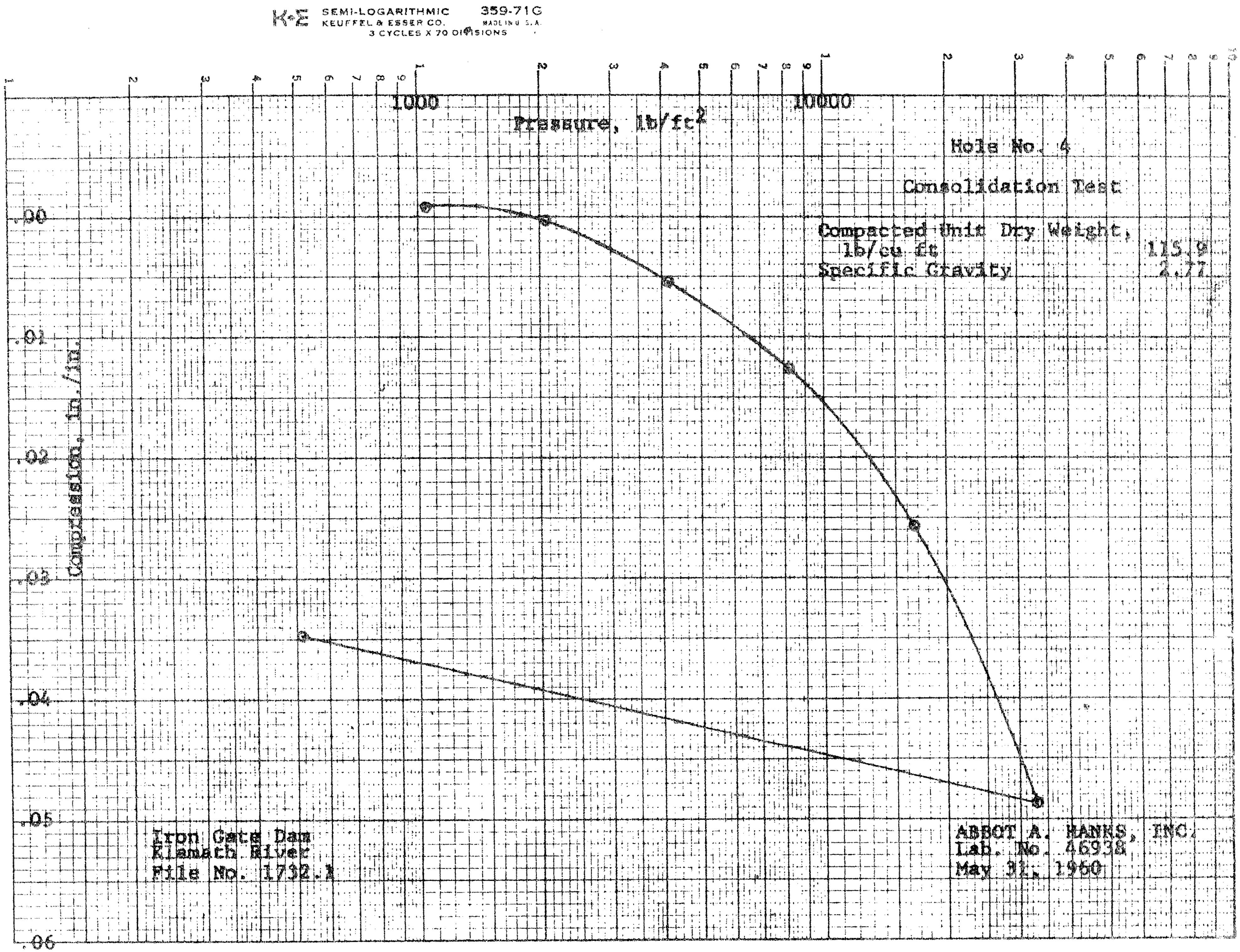
Permeability Tests
(Constant Head Tests)

Unit Dry Weight at Compaction, lb/ft^3	113.3
Moisture Content at Compaction, %	14.3
Moisture Content at Test, %	20.4
Degree of Saturation at Test, %	100+
Permeability Coefficient, ft per yr	30-40
" " , cm/sec	$3\text{-}4 \times 10^{-5}$
Unit Dry Weight at Compaction, lb/ft^3	116.2
Moisture Content at Compaction, %	16.0
Moisture Content at Test, %	17.0
Degree of Saturation at Test, %	97
Permeability Coefficient, ft per yr	.01-.04
" " , cm/sec	$1\text{-}4 \times 10^{-8}$

	Report	Letter
~~Optimum moisture~~	~~14.5~~	~~14-15~~
~~max dry density~~	115.9	115



Time, min
1
10
100
1000
Dial Reading, in.
.00
.01
.02
.03
.04
.05
.06
.07
p = 2060 lb/ft²
p = 4120 lb/ft²
apparatus correction
p = 8250 lb/ft²
p = 16500 lb/ft²
p = 33000 lb/ft²
Hole No. 4
Consolidation Test
Time Curves
ABBOT A. HANKS, INC.
Lab. No. 46938
May 31, 1960
Iron Gate Dam
File No. 1732.1

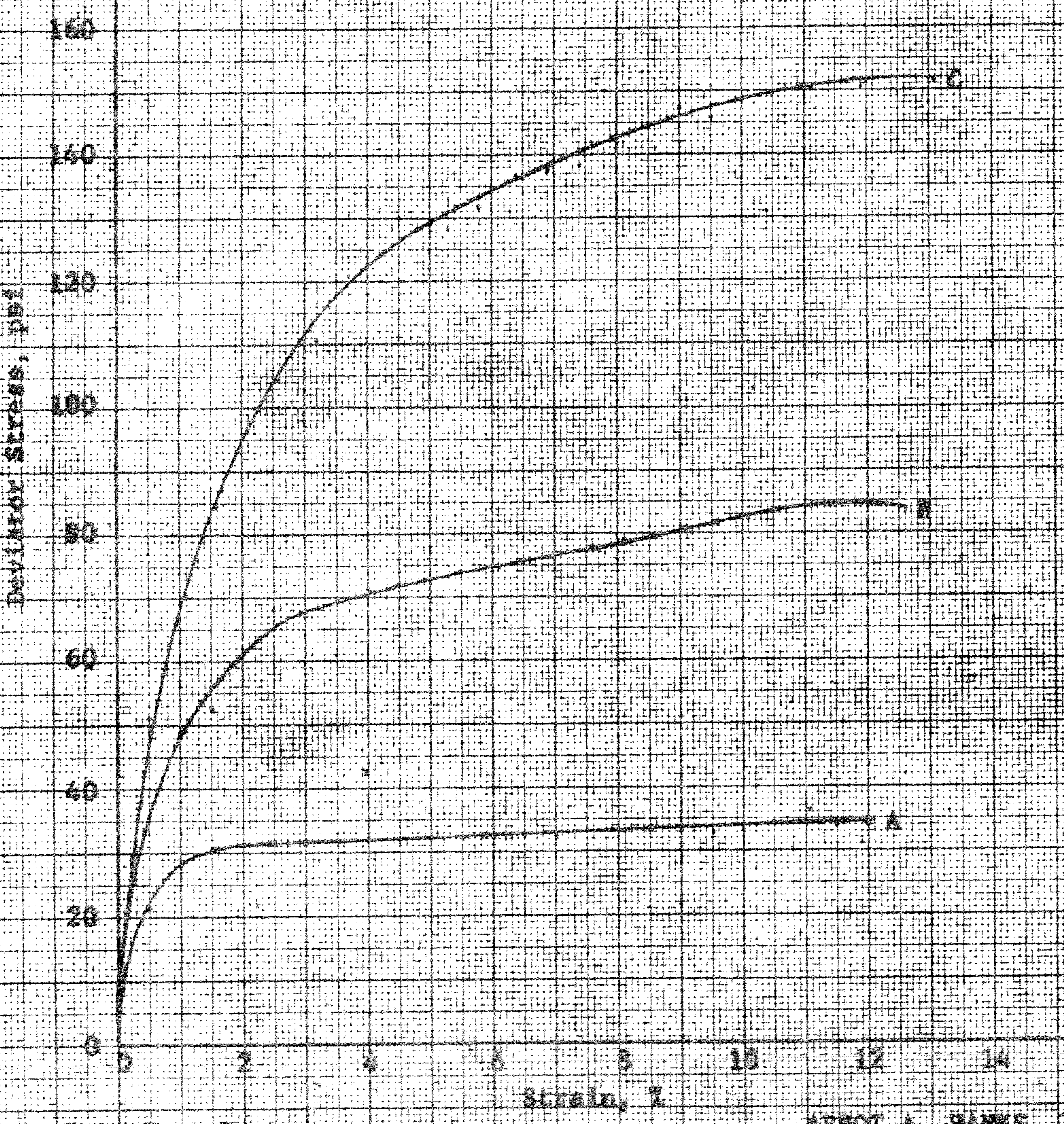


K·E SEMI-LOGARITHMIC 359-71G
KEUFFEL & ESSER CO. MADE IN U.S.A.
3 CYCLES X 70 DIVISIONS
1000
10000
Pressure, lb/ft^2
Compression, in./in.
.00
.01
.02
.03
.04
.05
.06
Hole No. 4
Consolidation Test
Compacted Unit Dry Weight, lb/cu ft 115.9
Specific Gravity 2.77
Iron Gate Dam
Klamath River
File No. 1732.1
ABBOT A. HANKS, INC.
Lab. No. 46938
May 31, 1960

Hole No. 4

Triaxial Shear Test

Stress-Strain Relationships



Iron Gate Dam
Klamath River
File No. 1732.1

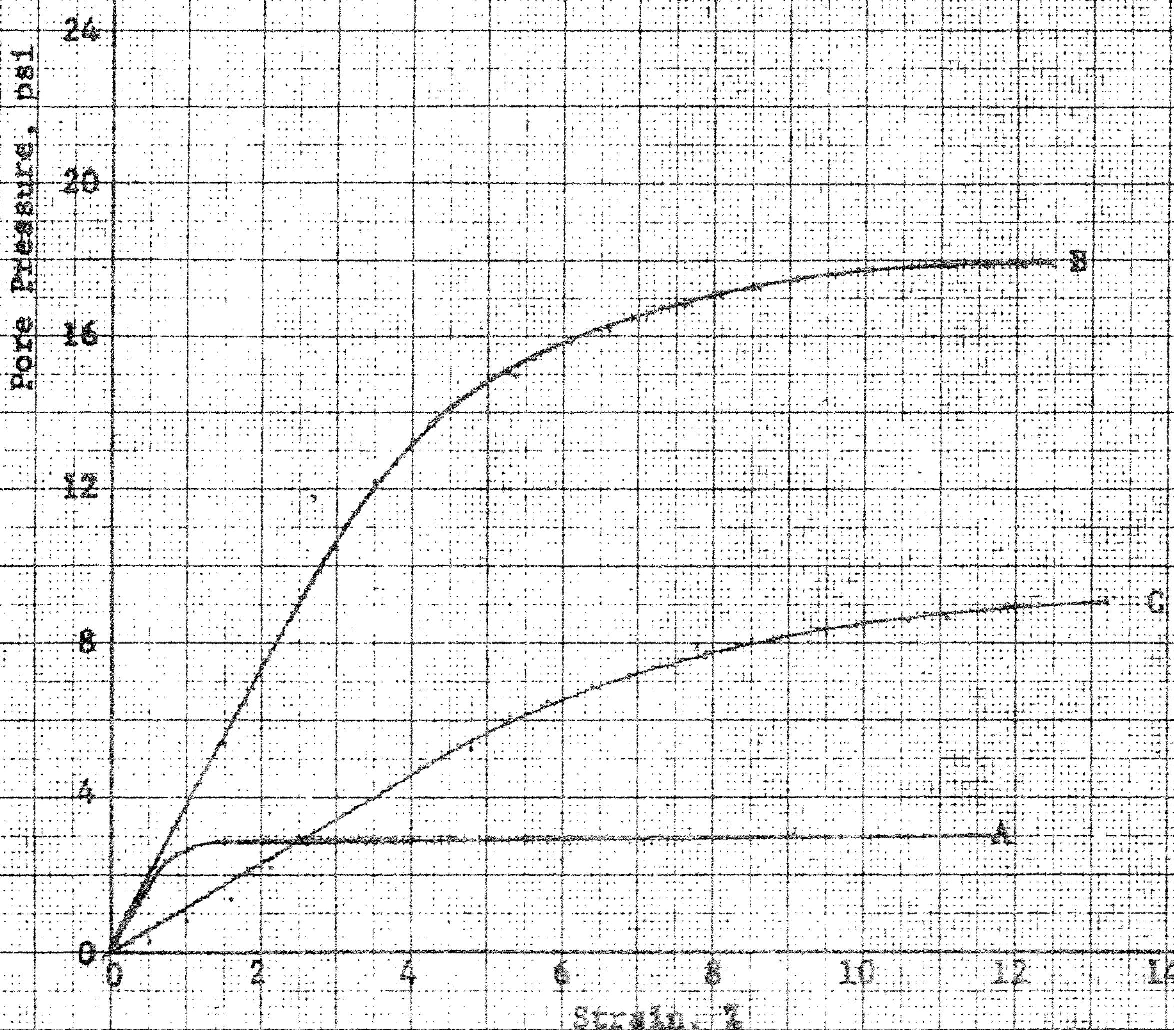
ABBOT A. HANKS, INC.
Lab. No. 46938
May 31, 1960

K&E 10 X 10 TO THE CM. 359-14G
KEUFFEL & ESSER CO. MADE IN U.S.A.

Hole No. 4

Triaxial Shear Test

Pore Pressure-Strain Relationships

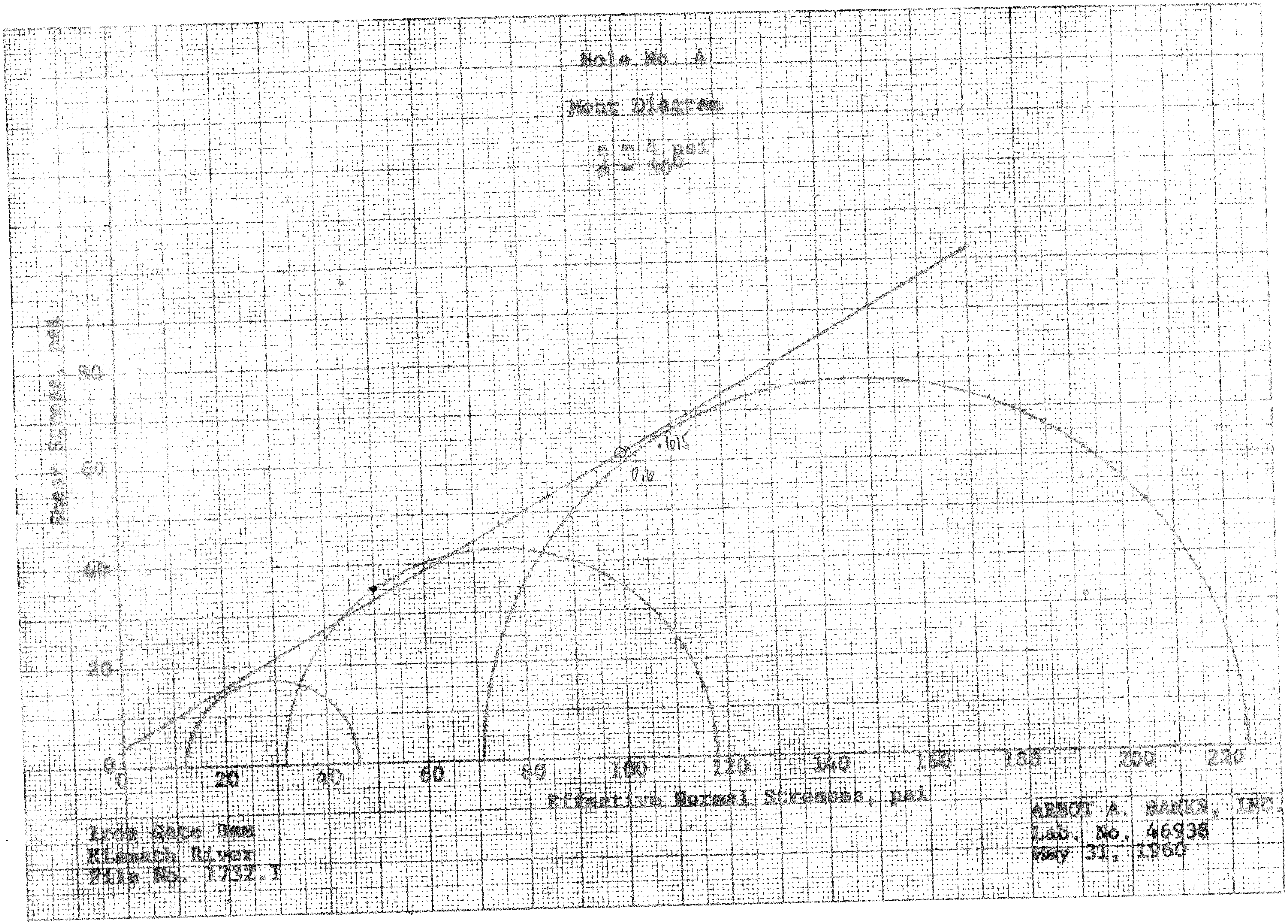


Iron Gate Dam
Klamath River
File No. 1732.1

ABBOT A. HANKS, INC.
Lab. No. 46938
May 31, 1960

10 X 10 TO THE CM. KEUFFEL & ESSER CO. 359-14G MADE IN U.S.A.

K&E 10 X 10 TO THE CM. 359-14G
KEUFFEL & ESSER CO. MADE IN U.S.A.



BRANCH LABORATORY 1085 MARTIN AVENUE · SANTA CLARA · [illegible] · CABLE HANK
BRANCH OFFICE 10 DE LUCA PLACE · SAN RAFAEL · [illegible] 4 [illegible]

ABBOT A. HANKS, INC.

ESTABLISHED [illegible]

1300 SANSOME STREET · SAN FRANCISCO 11 CALIFORNIA · EXBROOK 2 2464

Engineers
Assayers
Chemists
Metallurgists
Spectrographers
Soils and Foundations
Consulting · Testing · Inspecting

File No. 1732.1
Lab. No. 46938

June 9, 1960

Mr. W. L. Warren
Assistant Chief Engineer
The California Oregon Power Company
216 West Main Street
Medford, Oregon

Re: Iron Gate Dam
Soil Samples

Dear Mr. Warren:

Enclosed are the findings from tests performed on soil samples marked Hole No. 7.

Very truly yours,

ABBOT A. HANKS, INC.

Donald W Radbruch

Donald W. Radbruch

hms
Encls.
Reports to:
3-The California Oregon Power Company

Iron Gate Dam
Klamath River
File No. 1732.1

Abbot A. Hanks, Inc.
Lab. No. 46938
June 9, 1960

TEST RESULTS

Hole No. 7
Specific Gravity: 2.74

Triaxial Shear Test

	Samples			
	A	B	C	D
Chamber Pressure, psi	15	50	80	80
Unit Dry Weight at Compaction, lb/ft^3	109.1	110.0	110.0	109.3
Moisture Content at Compaction, %	17.6	17.3	19.0	19.3
Unit Dry Weight at Test, lb/ft^3	106.7	111.5	114.7	114.8
Moisture Content at Test, lb/ft^3	22.6	19.3	18.7	19.8
Degree of Saturation at Test, %	100+	100	100	100+
Maximum Deviator Stress, psi	22	67	77	79
Pore Pressure at Max. Deviator Stress, psi	6	12	23	17

Permeability Test
(Constant Head Test)

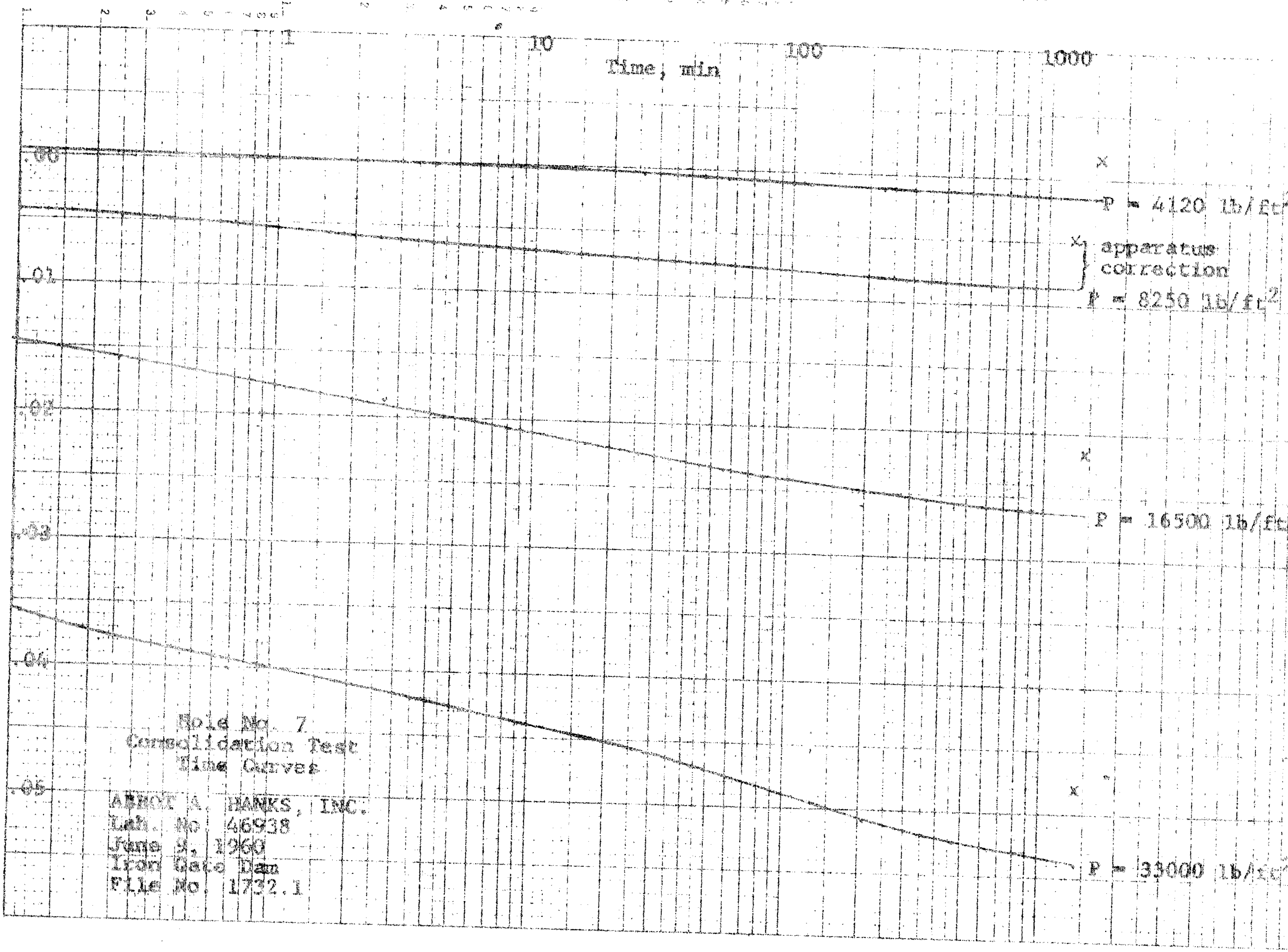
Unit Dry Weight at Compaction, lb/ft^3	109.3
Moisture Content at Compaction, %	17.8
Moisture Content at Test, %	19.7
Degree of Saturation at Test, %	96
Permeability Coefficient, ft per yr	Less than .01
" " , cm/sec	Less than 10^{-8}

Report
~~Letter~~ ~~a~~ ~~letter~~

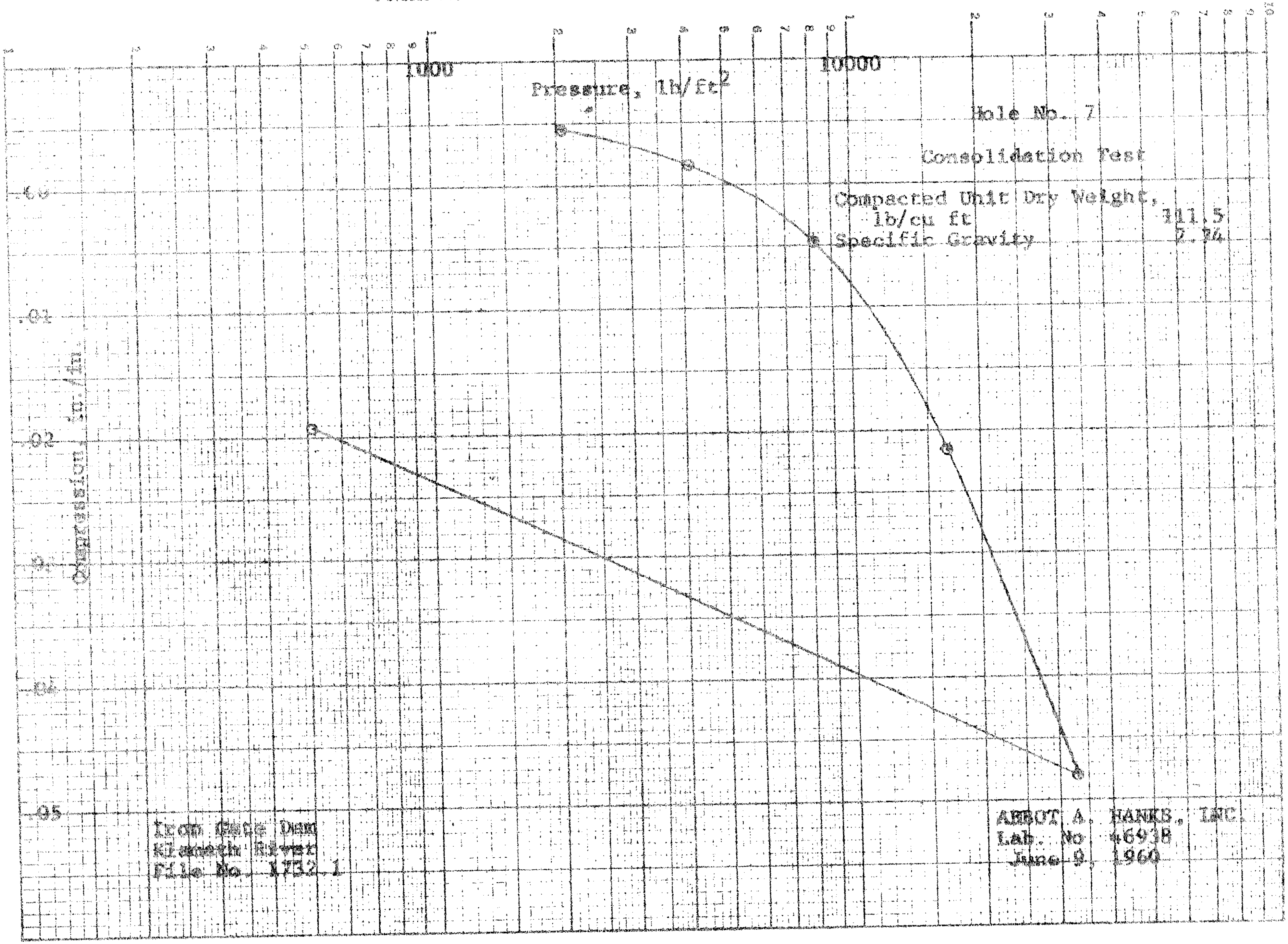
~~Optimum moisture~~ ——— ~~15.2~~ ——— 15

Max. ~~Dry density~~ ——— 117.6 ——— 116.0

SEMI-LOGARITHMIC 359-91C
5 CYCLES X 70 DIVISIONS



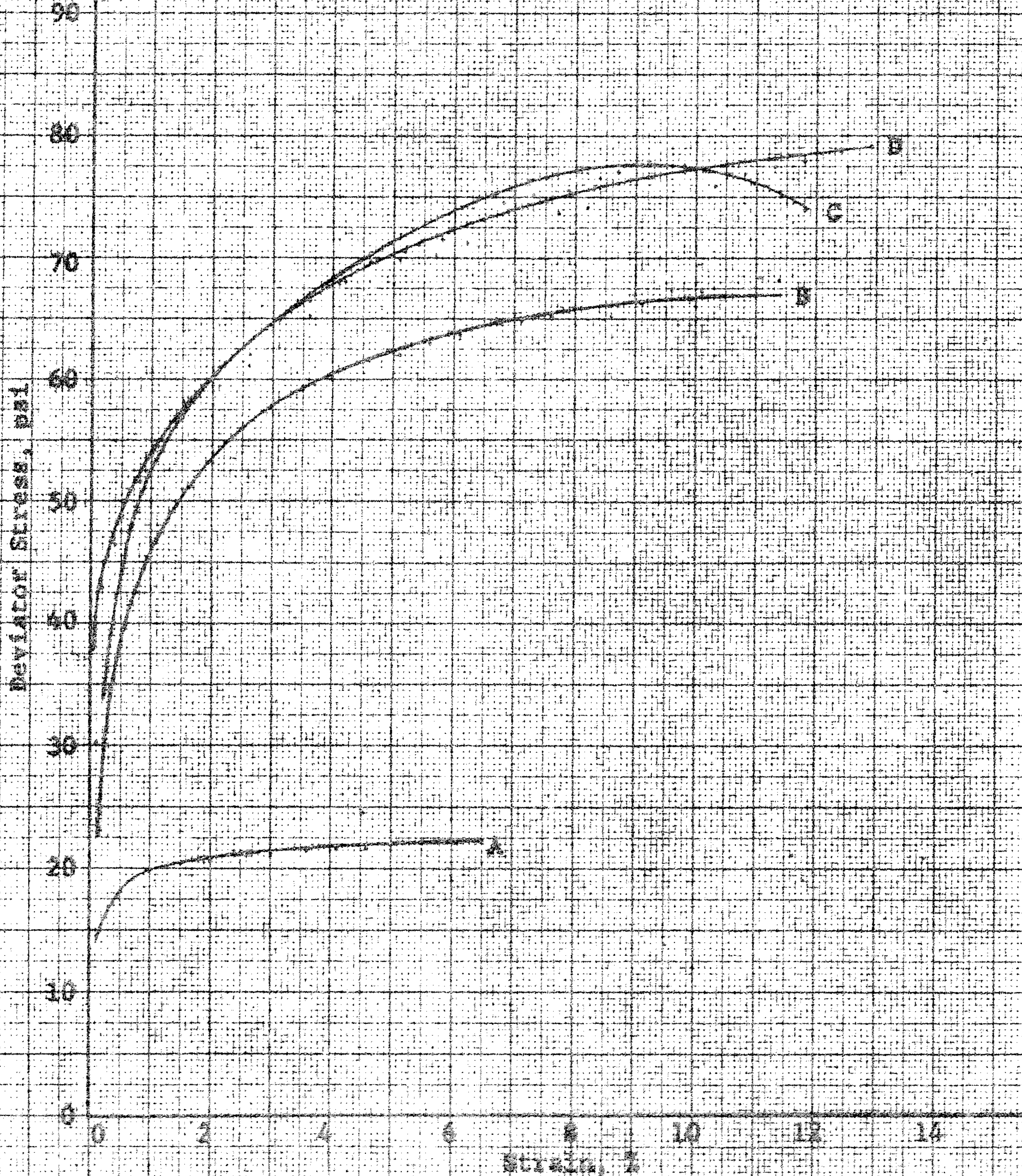
SEMI-LOGARITHMIC 359-71
KEUFFEL & ESSER CO.
3 CYCLES X 70 DIVISIONS



Hole No. 7

Triaxial Shear Test

Stress-Strain Relationships



Iron Gate Dam
Klamath River
File No. 1732.1

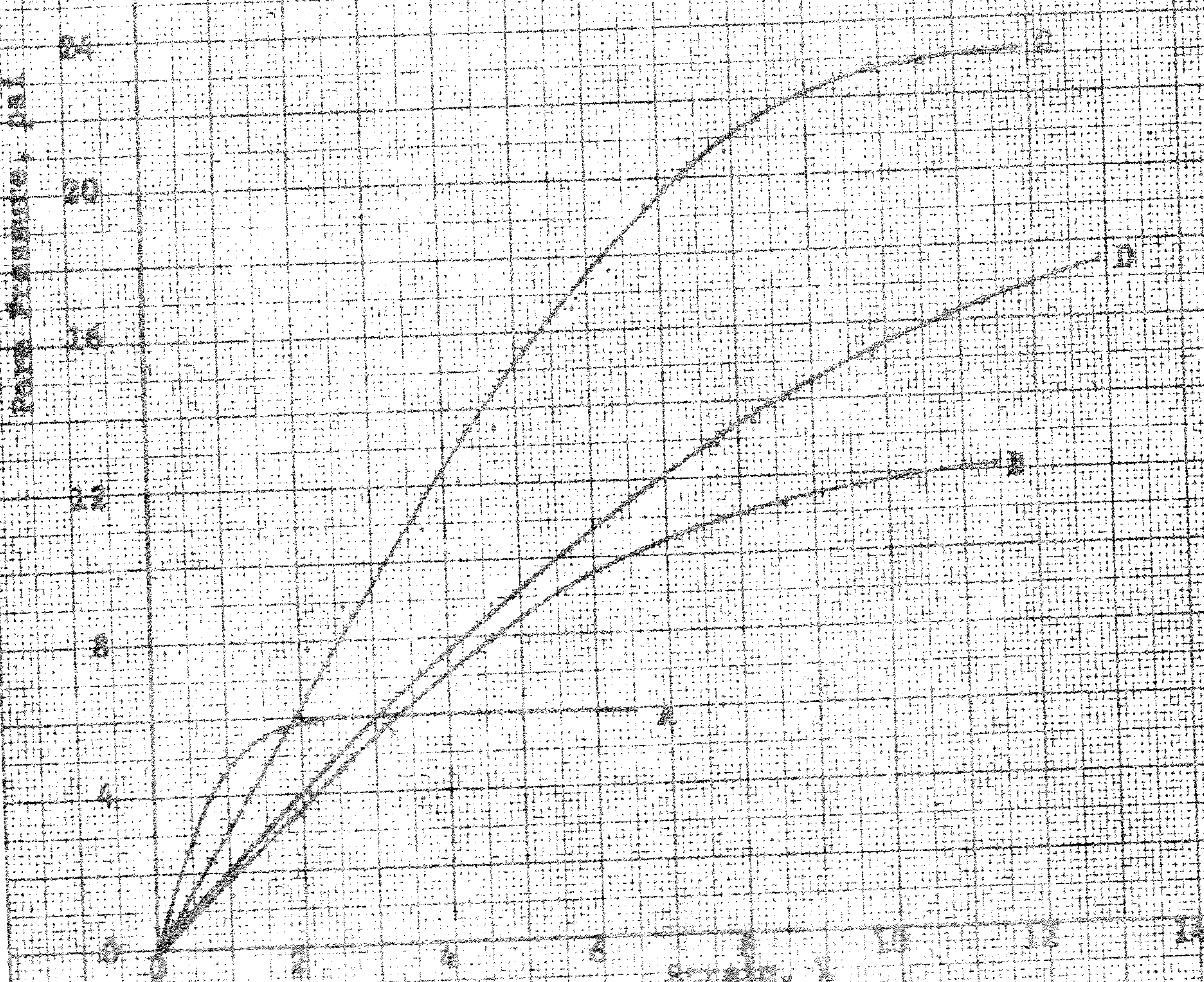
ABBOT A. HANKS, INC.
Lab. No. 46838
June 9, 1960

K&E 10 X 10 TO THE CM. 359-14G
KEUFFEL & ESSER CO. MADE IN U.S.A.

Hole No. 7

Triaxial Shear Test

Pore Pressure-Strain Relationships

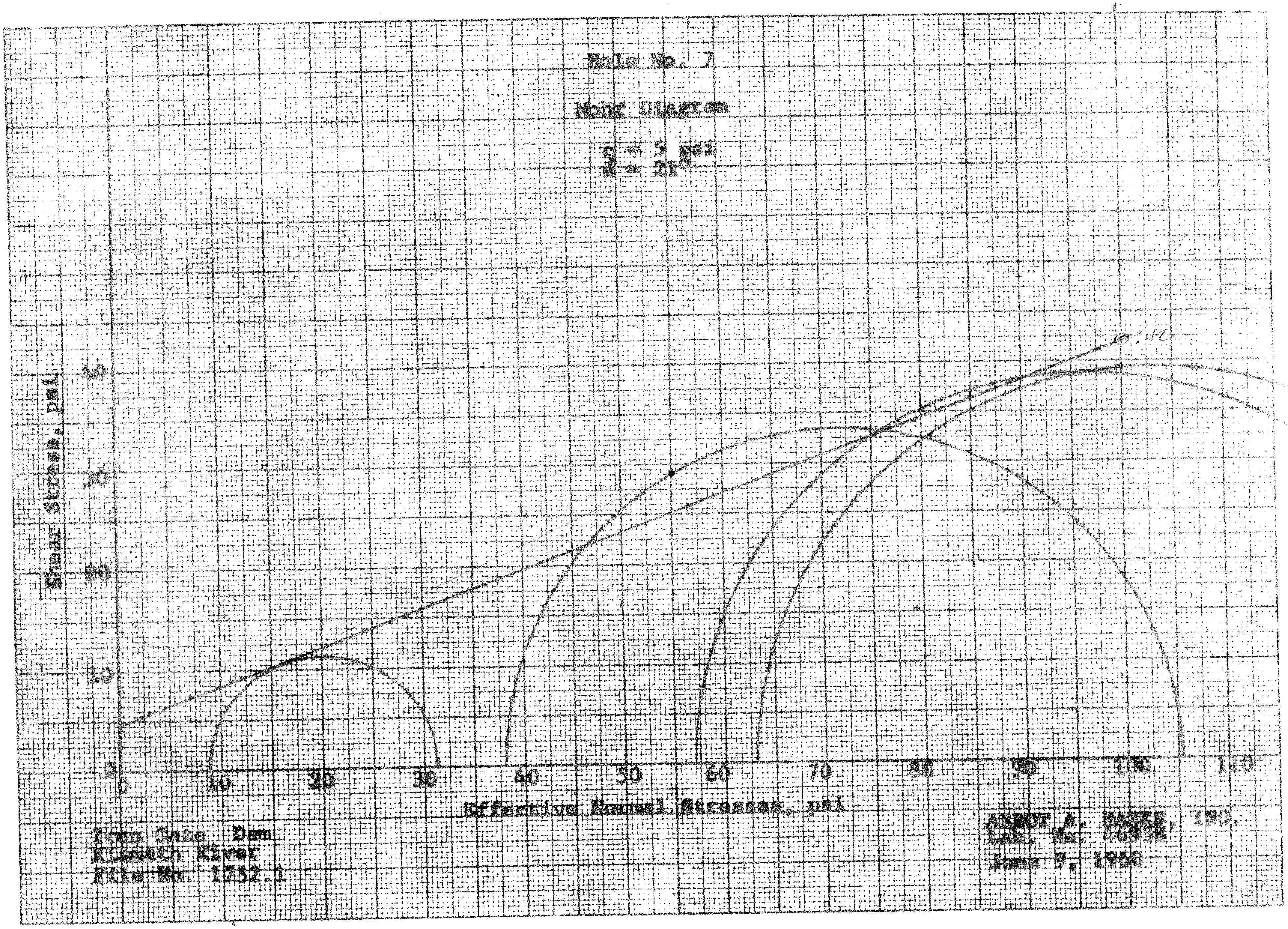


Iron Gate Dam
Klamath River
File No. 1732.1

[illegible]
[illegible]
June 9, 1960

10 X 10 TO THE CM. 359-14G
KEUFFEL & ESSER CO. MADE IN U.S.A.

K+E 10 X 10 TO THE CM. 359-14G
KEUFFEL & ESSER CO. MADE IN U.S.A.



BRANCH LABORATORY 1080 MARTIN AVENUE · SANTA CLARA · CHERRY 3-7761
BRANCH OFFICE 10 DE LUCA PLACE · SAN RAFAEL · GLENWOOD 4-8850

ABBOT A. HANKS, INC.

1300 SANSOME STREET · SAN FRANCISCO 11, CALIFORNIA · EXBROOK 7-2464

Engineers
Assayers
Chemists
Metallurgists
Spectrographers
Soils and Foundations
Consulting · Testing · Inspecting

File No. 1732.1
Lab. No. 46938

May 24, 1960

Mr. W. L. Warren
Assistant Chief Engineer
The California Oregon Power Company
216 West Main Street
Medford, Oregon

Re: Iron Gate Dam
Soil Samples

Dear Mr. Warren:

Enclosed are the findings from tests performed on soil samples marked Hole No. 8.

Very truly yours,

ABBOT A. HANKS, INC.

Donald W Radbruch

Donald W. Radbruch

hms
Encls.
Reports to:
3-The California Oregon Power Company

Iron Gate Dam
Klamath River
File No. 1732.1

Abbot A. Hanks, Inc.
Lab. No. 46938
May 18, 1960

TEST RESULTS

Hole No. 8
Specific Gravity: 2.75

Triaxial Shear Test

	Sample A	Sample B	Sample C
Chamber Pressure, psi	15	50	80
Unit Dry Weight at Compaction, lb/ft^3	98.6	99.7	98.9
Moisture Content at Compaction, %	19.9	19.9	20.1
Unit Dry Weight at Test, lb/ft^3	95.4	100.6	102.9
Moisture Content at Test, lb/ft^3	28.4	26.1	25.0
Degree of Saturation at Test, %	98	100+	100+
Maximum Deviator Stress, psi	21	48	66
Pore Pressure at Max. Deviator Stress, psi	5	13	30

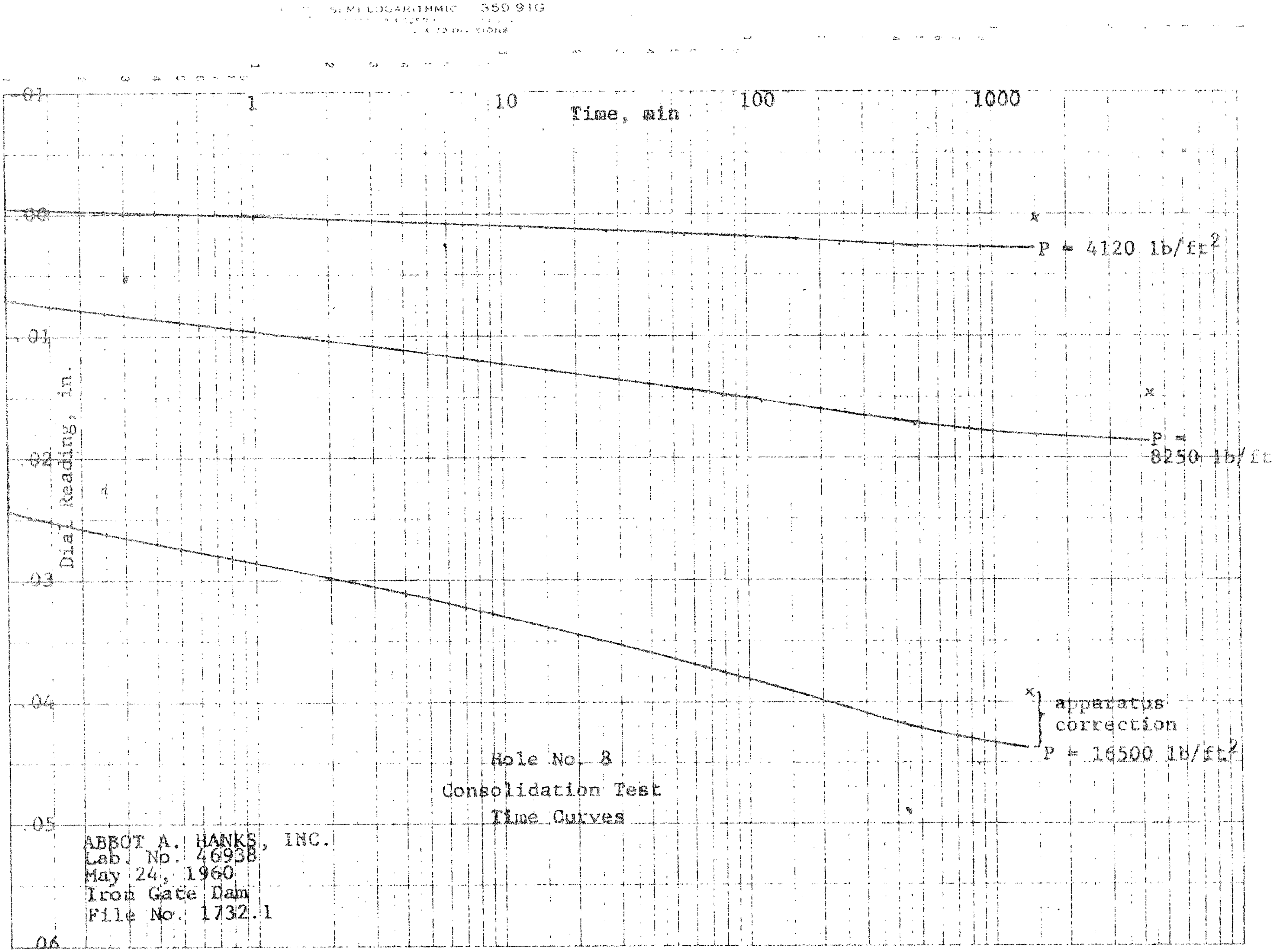
Permeability Test
(Constant Head Test)

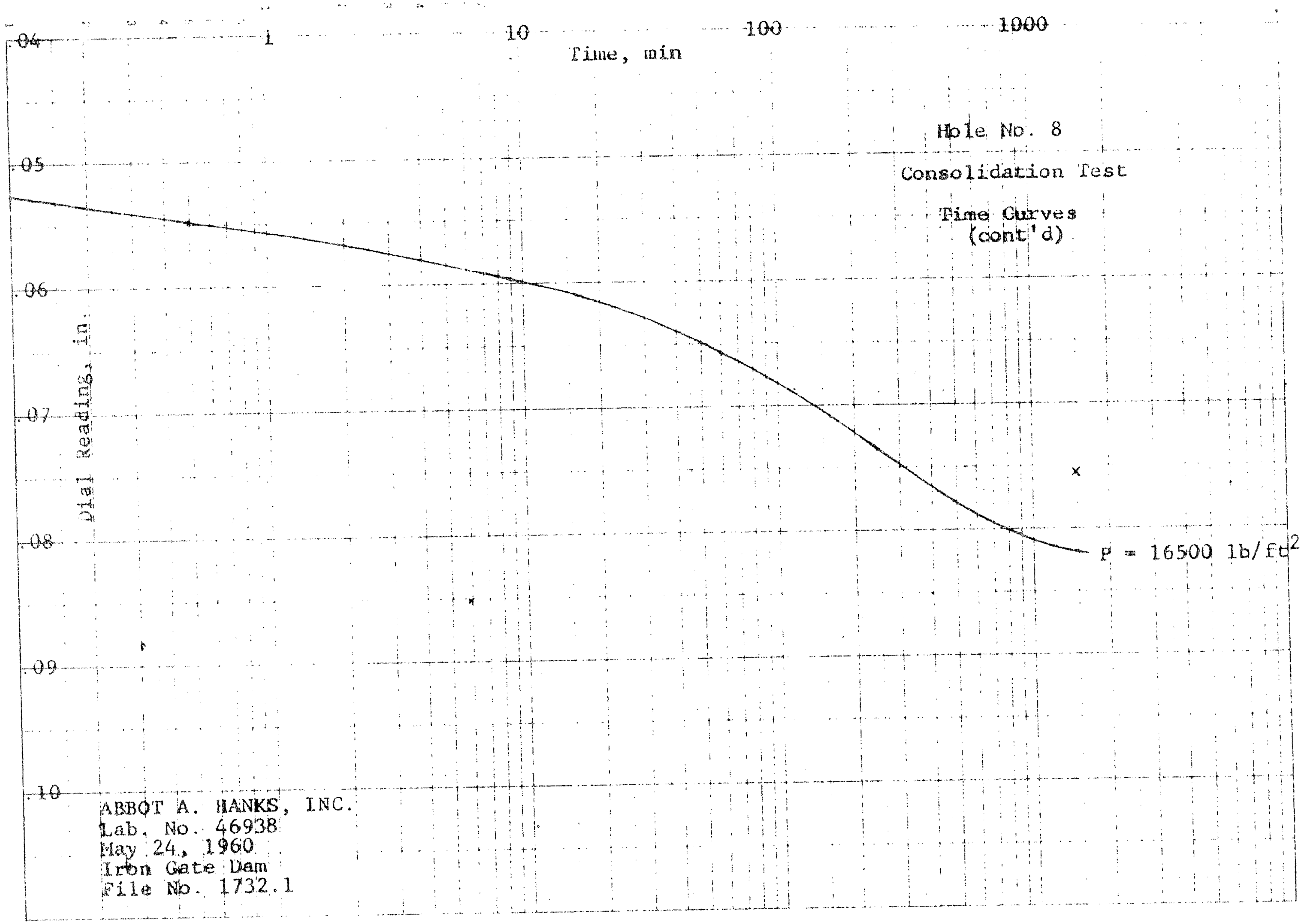
Unit Dry Weight at Compaction, lb/ft^3	100.8
Moisture Content at Compaction, %	21.1
Moisture Content at Test, %	25.4
Degree of Saturation at Test, %	100
Permeability Coefficient, ft per yr	Less than .01
" " , cm/sec	Less than 10-8

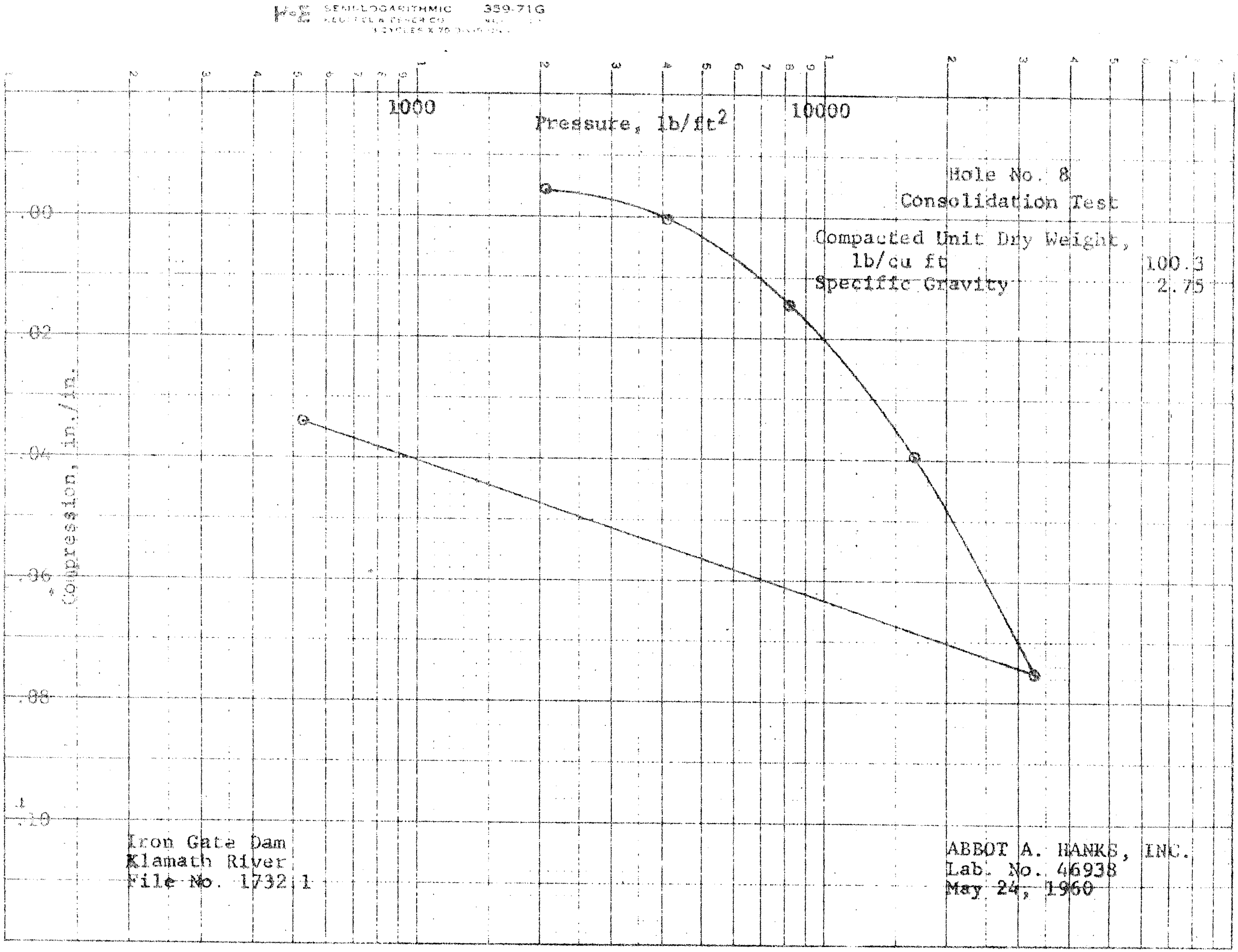
~~Report letter~~

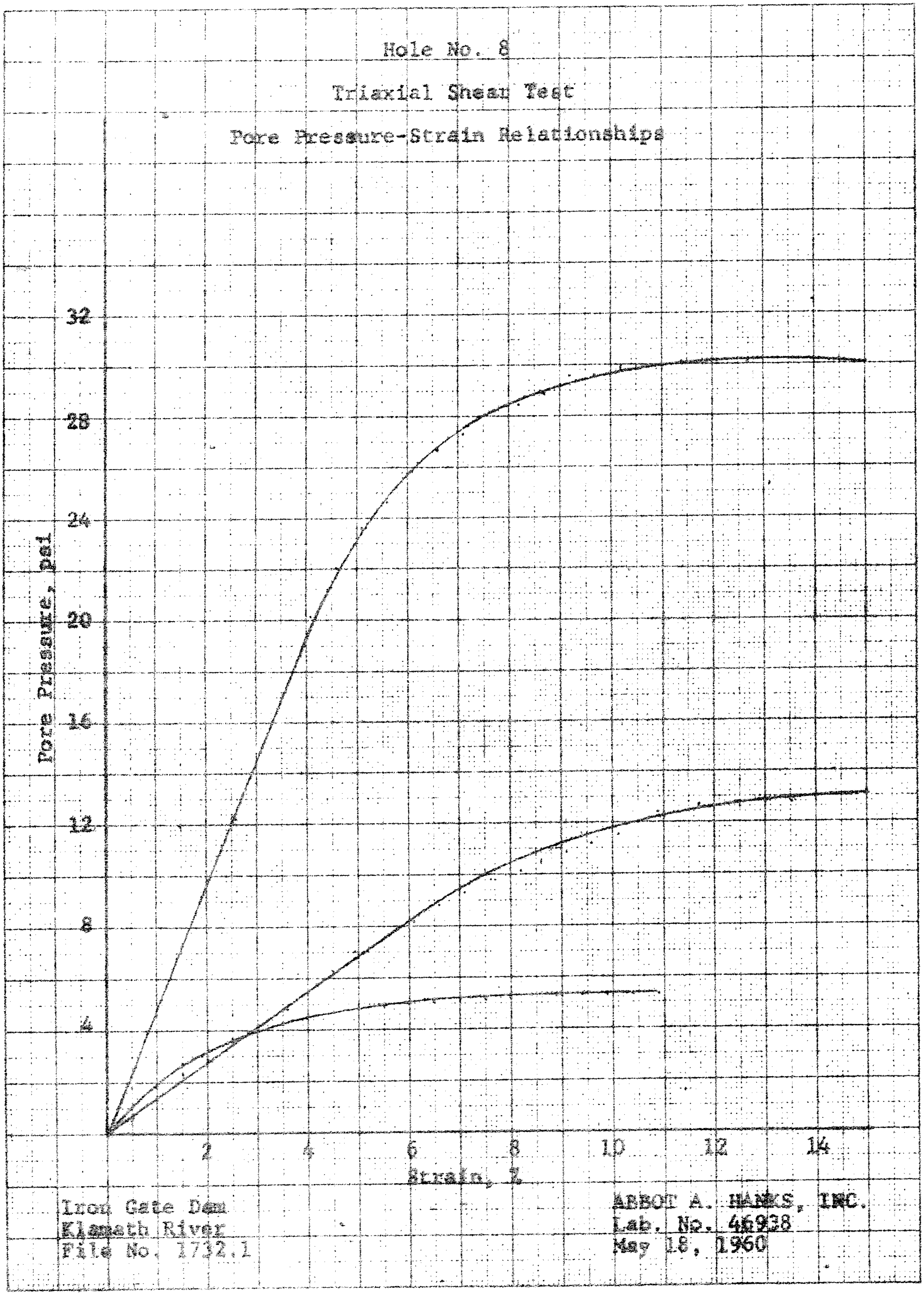
~~optimum moisture~~ 17.2 17.8

~~max. dry density~~ ~~107.8 100.0~~


SEMI-LOGARITHMIC 359-91G
Time, min
1
10
100
1000
Dial Reading, in.
.00
.01
.02
.03
.04
.05
.06
P = 4120 lb/ft²
P = 8250 lb/ft
apparatus correction
P = 16500 lb/ft²
Hole No. 8
Consolidation Test
Time Curves
ABBOT A. HANKS, INC.
Lab. No. 46938
May 24, 1960
Iron Gate Dam
File No. 1732.1


Hole No. 8
Consolidation Test
Time Curves
(cont'd)
Time, min
1
10
100
1000
Dial Reading, in.
.04
.05
.06
.07
.08
.09
.10
p = 16500 lb/ft²
ABBOT A. HANKS, INC.
Lab. No. 46938
May 24, 1960
Iron Gate Dam
File No. 1732.1


K&E SEMI-LOGARITHMIC 359-71G
1000
10000
Pressure, lb/ft²
Compression, in./in.
.00
.02
.04
.06
.08
.10
Hole No. 8
Consolidation Test
Compacted Unit Dry Weight, lb/cu ft 100.3
Specific Gravity 2.75
Iron Gate Dam
Klamath River
File No. 1732.1
ABBOT A. HANKS, INC.
Lab. No. 46938
May 24, 1960

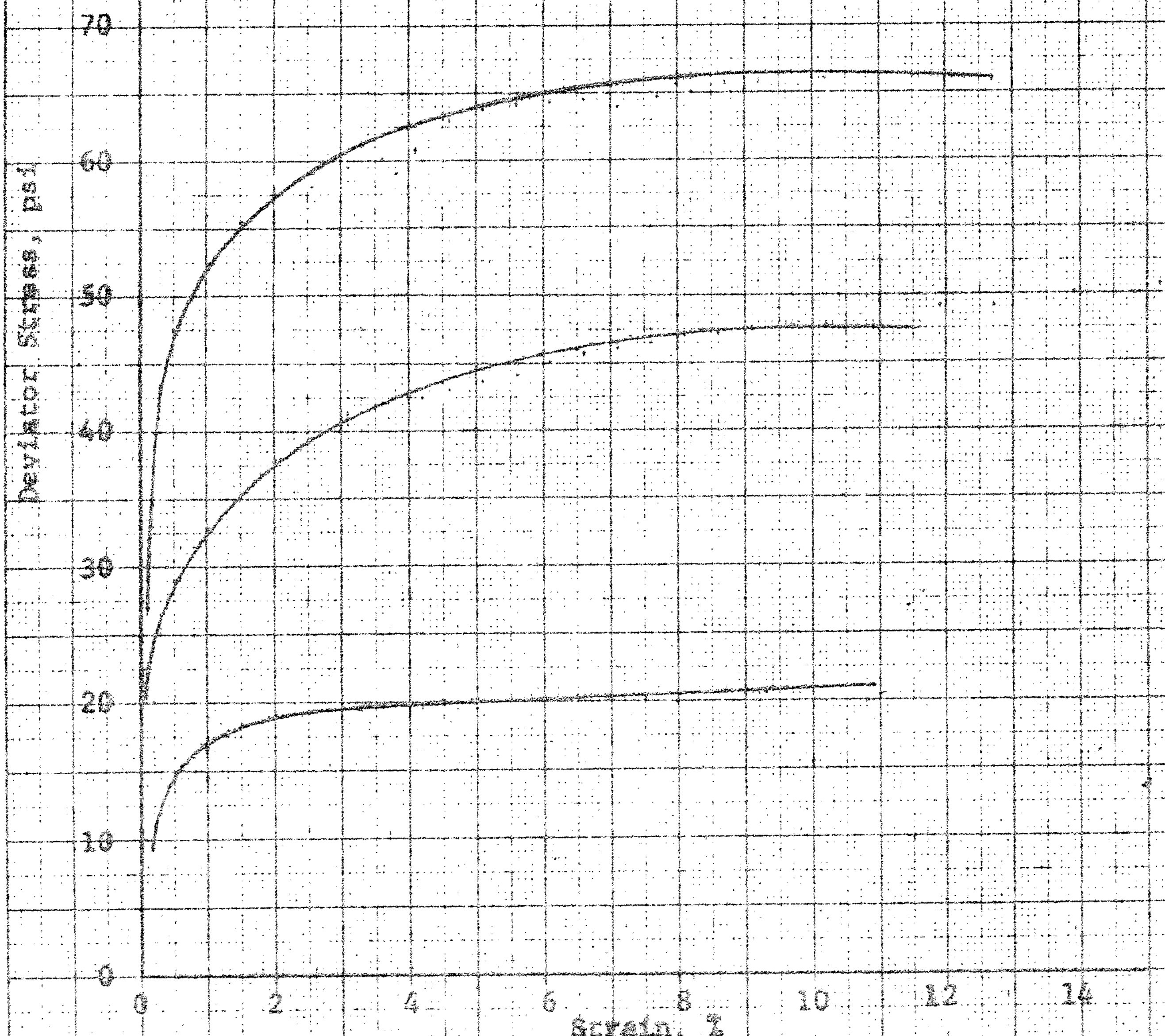


10 X 10 TO THE CM. 359-14G
KEUFFEL & ESSER CO.

Hole No. 8

Triaxial Shear Test

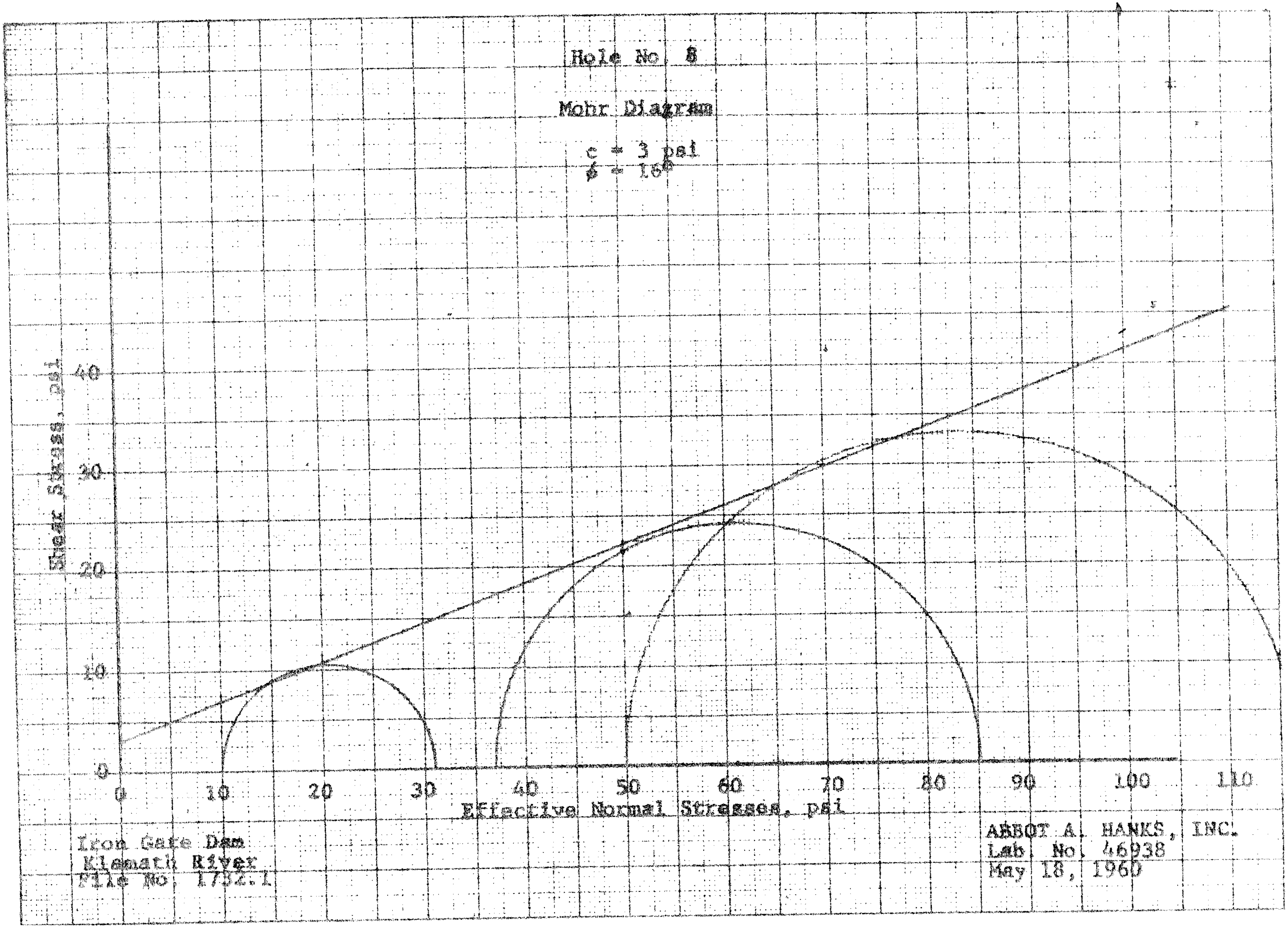
Stress-Strain Relationships



Iron Gate Dam
Klamath River
File No. 1732.1

ABBOT A. HANKS, INC.
Lab. No. 46938
May 18, 1960

10 X 10 TO THE CM. 359-14G



BRANCH LABORATORY 1086 MARTIN AVENUE • SANTA CLARA • CHERRY 8 5262
BRANCH OFFICE 10 DE LUCA PLACE • SAN RAFAEL • GLENWOOD 4 8550
CABLE HANX

ABBOT A. HANKS, INC.
ESTABLISHED 1866
1300 SANSOME STREET • SAN FRANCISCO 11 CALIFORNIA • EXBROOK 7 2464

Engineers
Assayers
Chemists
Metallurgists
Spectrographers
Soils and Foundations
Consulting · Testing · Inspecting

File No. 1732.1
Lab. No. 46938

May 19, 1960

Mr. W. L. Warren
Assistant Chief Engineer
The California Oregon Power Company
216 West Main Street
Medford, Oregon

Re: Iron Gate Dam
Soil Samples

Dear Sir:

Enclosed are the findings from tests performed on soil samples marked Hole No. 11.

Very truly yours,

ABBOT A. HANKS, INC.

Donald W Radbruch

Donald W. Radbruch

hms
Encls.
Reports to:
3-The California Oregon Power Company

Iron Gate Dam
Klamath River
Rile No. 1732.1

Abbot A.H anks, Inc.
Lab. No. 46938
May 19, 1960

TEST RESULTS

Hole No. 11
Specific Gravity: 2.75

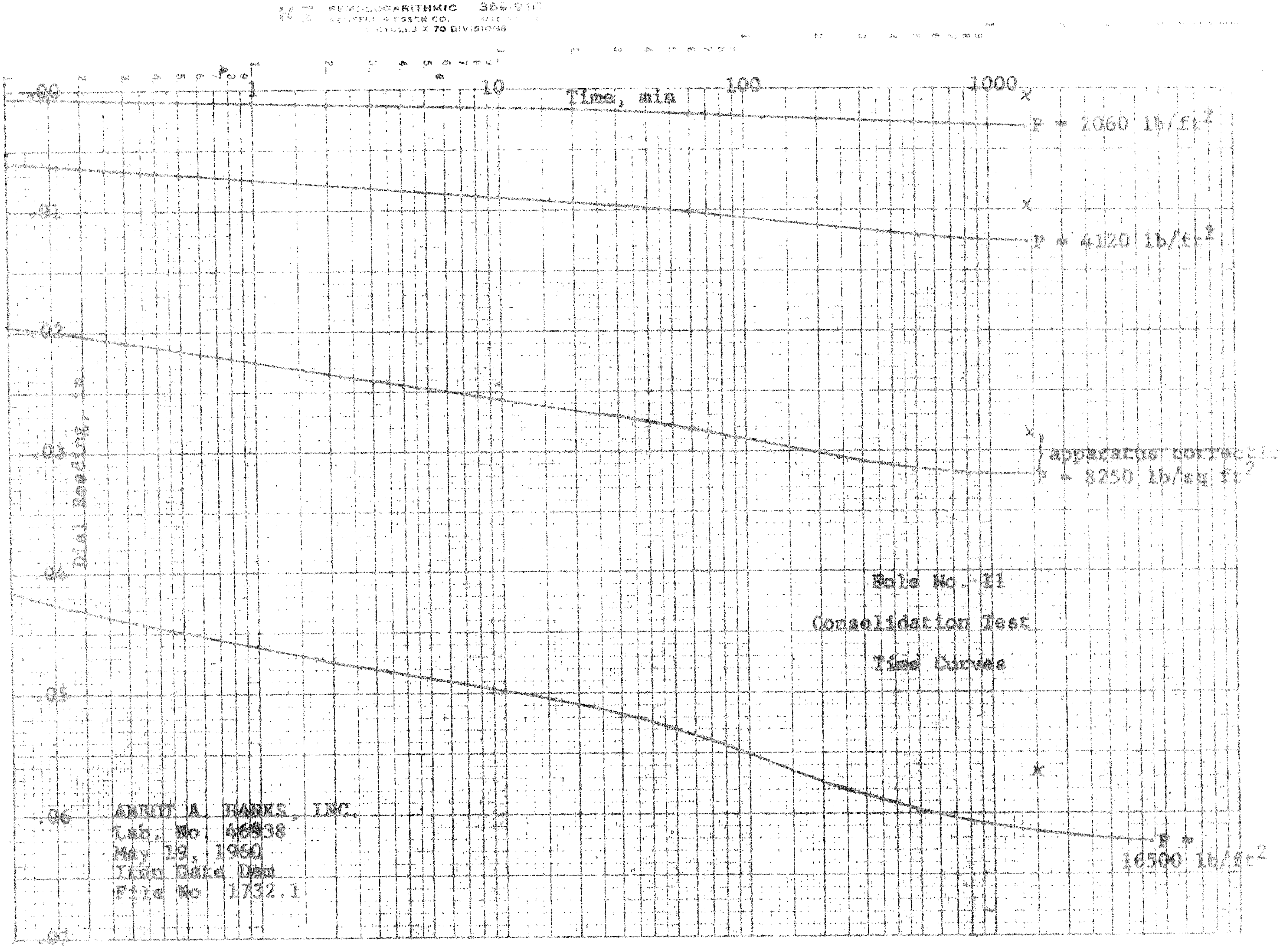
Triaxial Shear Test

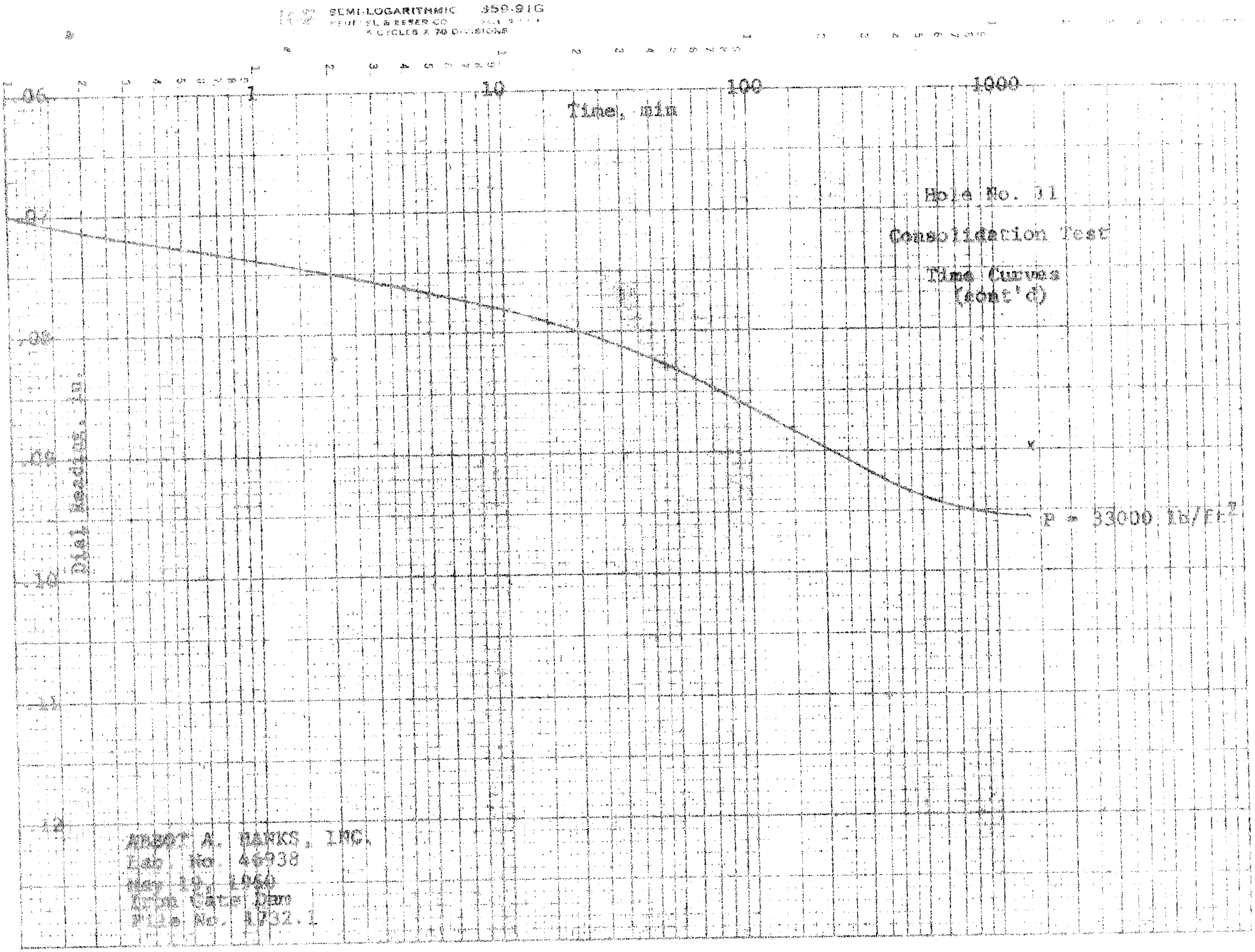
	Sample A	Sample B	Sample C
Chamber Pressure, psi	15	50	80
Unit Dry Weight at Compaction, lb/ft^3	101.5	101.7	102.1
Moisture Content at Compaction, %	22.2	21.7	22.0
Unit Dry Weight at Test, lb/ft^3	102.3	105.1	107.6
Moisture Content at Test, lb/ft^3	25.0	22.9	21.7
Degree of Saturation at Test, %	100	100	100
Maximum Deviator Stress, psi	21	55	73
Pore Pressure at Max. Deviator Stress, psi	5	7	22

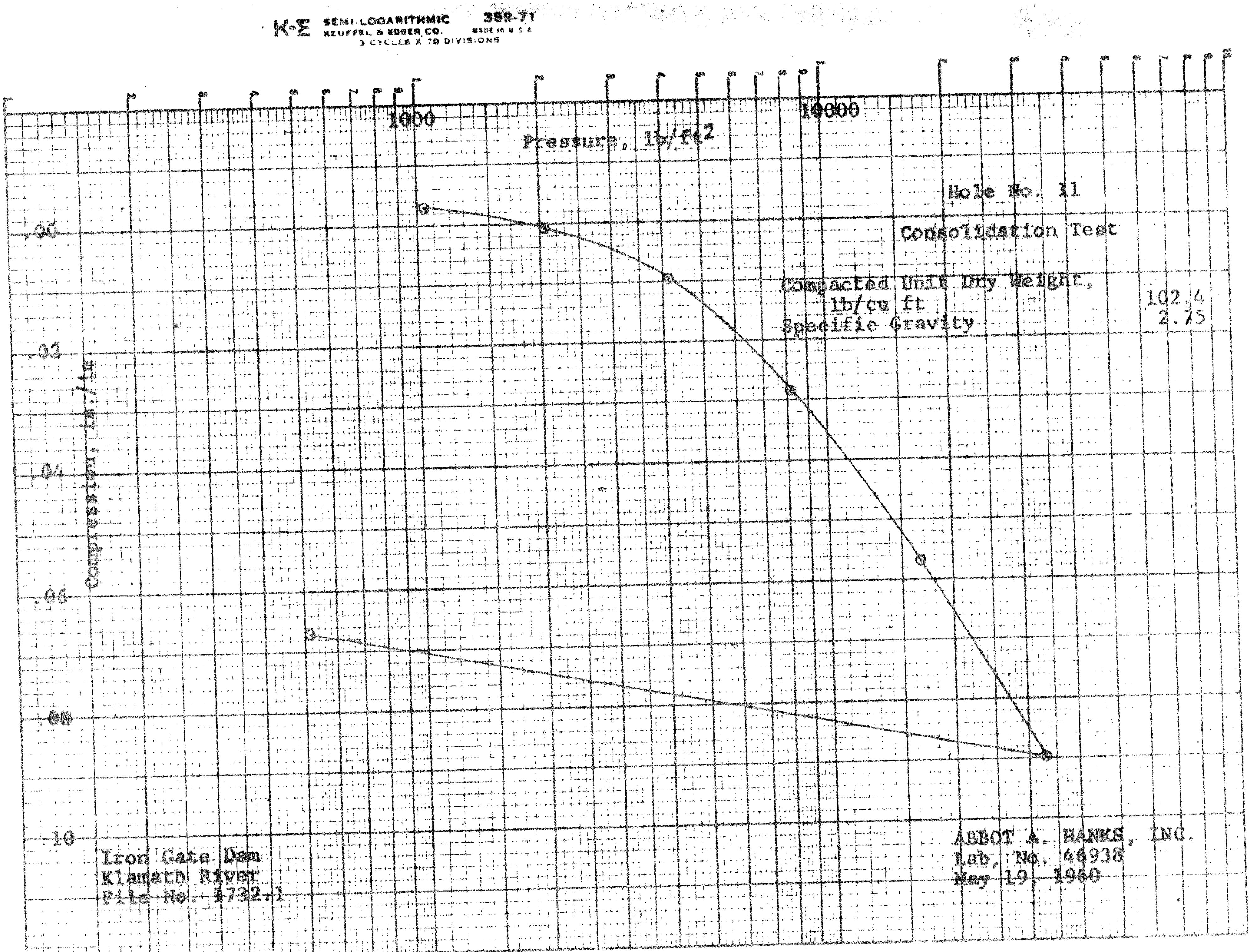
Permeability Test
(Constant Head Test)

Dry Density at Compaction, lb/ft^3	101.0
Moisture Content at Compaction, %	22.4
Permeability Coefficient, ft per yr	Less than .01
" " , cm/sec	Less than 10^{-8}

	~~[illegible]~~	~~[illegible]~~
~~Opt. min moisture~~	~~12.4~~	~~[illegible]~~
~~max dry density~~	~~[illegible]~~	~~[illegible]~~


SEMI-LOGARITHMIC 359-91C
KEUFFEL & ESSER CO.
CYCLES X 70 DIVISIONS
Time, min
10
100
1000
Dial Reading, in
.00
.01
.02
.03
.04
.05
.06
.07
P = 2060 lb/ft^2
P = 4120 lb/ft^2
apparatus correctio
P = 8250 lb/sq ft^2
P = 16500 lb/ft^2
Hole No. 21
Consolidation Test
Time Curves
HANKS, INC.
Lab. No
May 19, 1960
Dam
File No 1732.1

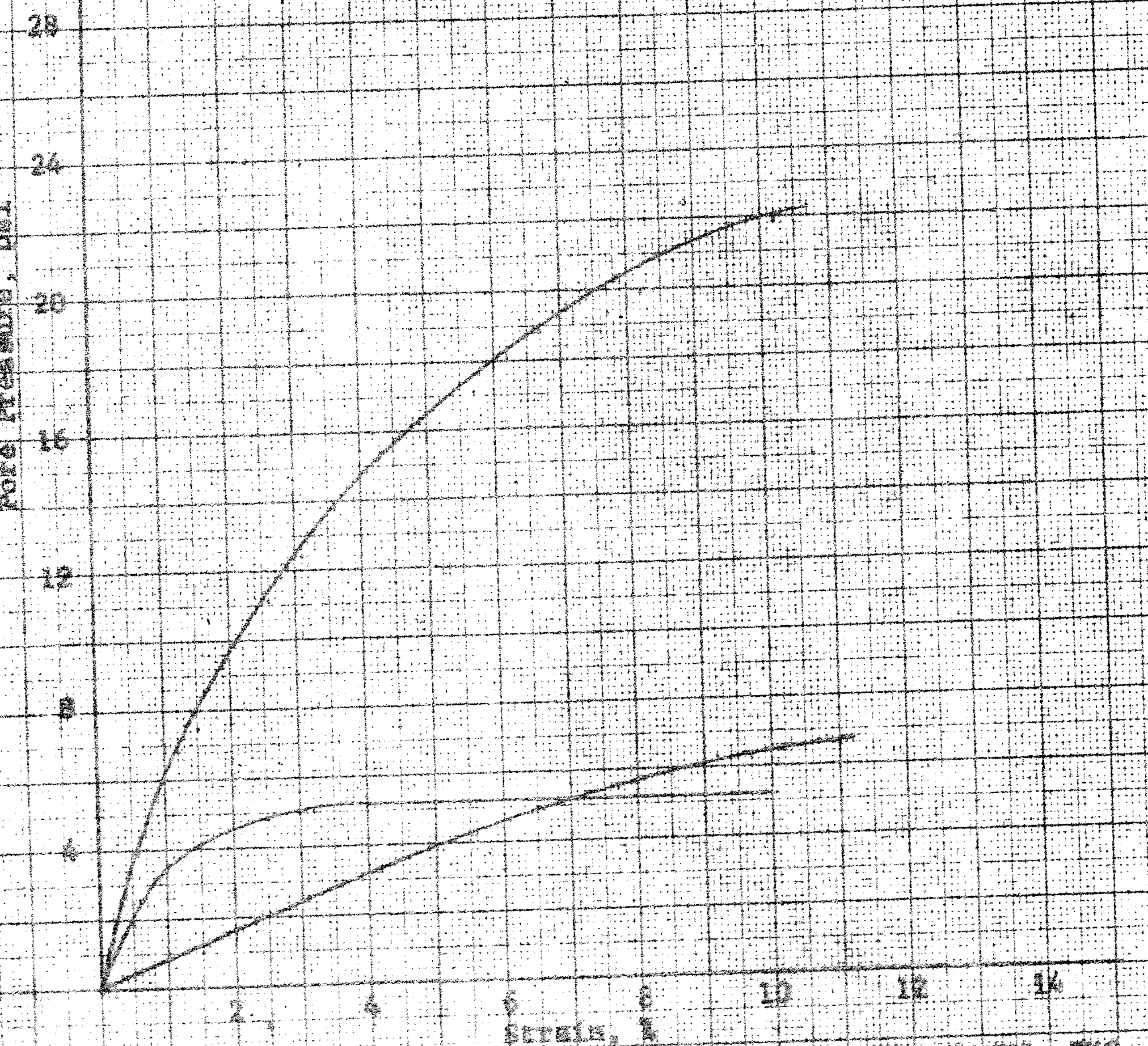

SEMI-LOGARITHMIC 359-91G
KEUFFEL & ESSER CO.
5 CYCLES X 70 DIVISIONS
Time, min
1
10
100
1000
Dial Reading, in.
.06
.07
.08
.09
.10
.11
.12
Hole No. 11
Consolidation Test
Time Curves
(cont'd)
P = 33000 lb/ft²
A. HANKS, INC.
Lab. No. 46938
File No. 1732.1


K&E SEMI-LOGARITHMIC 359-71
KEUFFEL & ESSER CO. MADE IN U.S.A.
3 CYCLES X 70 DIVISIONS
1000
10000
Pressure, lb/ft2
Compression, in./in.
.00
.02
.04
.06
.08
10
Hole No. 11
Consolidation Test
Compacted Unit Dry Weight, lb/cu ft 102.4
Specific Gravity 2.75
Iron Gate Dam
Klamath River
File No. 1732.1
ABBOT A. HANKS, INC.
Lab. No. 46938
May 19, 1960

Hole No. 11

Triaxial Shear Test

Pore Pressure-Strain Relationships



Iron Gate Dam
Klamath River
File No. 1732.1

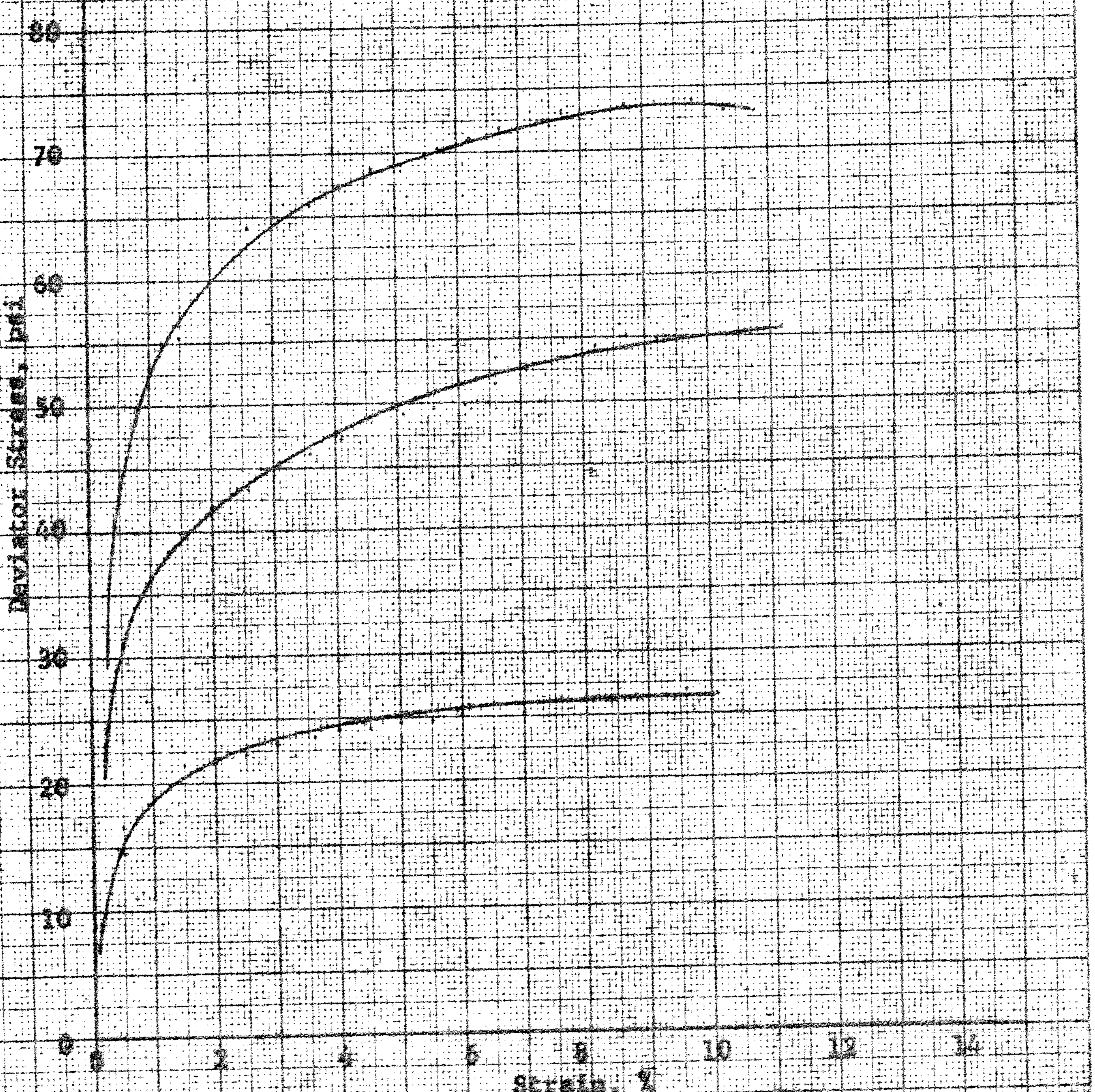
ABBOT A. HANKS, INC.
Lab. No. 46936
May 19, 1960

K+E 10 X 10 TO THE CM. 359-14G
KEUFFEL & ESSER CO. MADE IN U.S.A.

Hole No. 11

Triaxial Shear Test

Stress-Strain Relationships

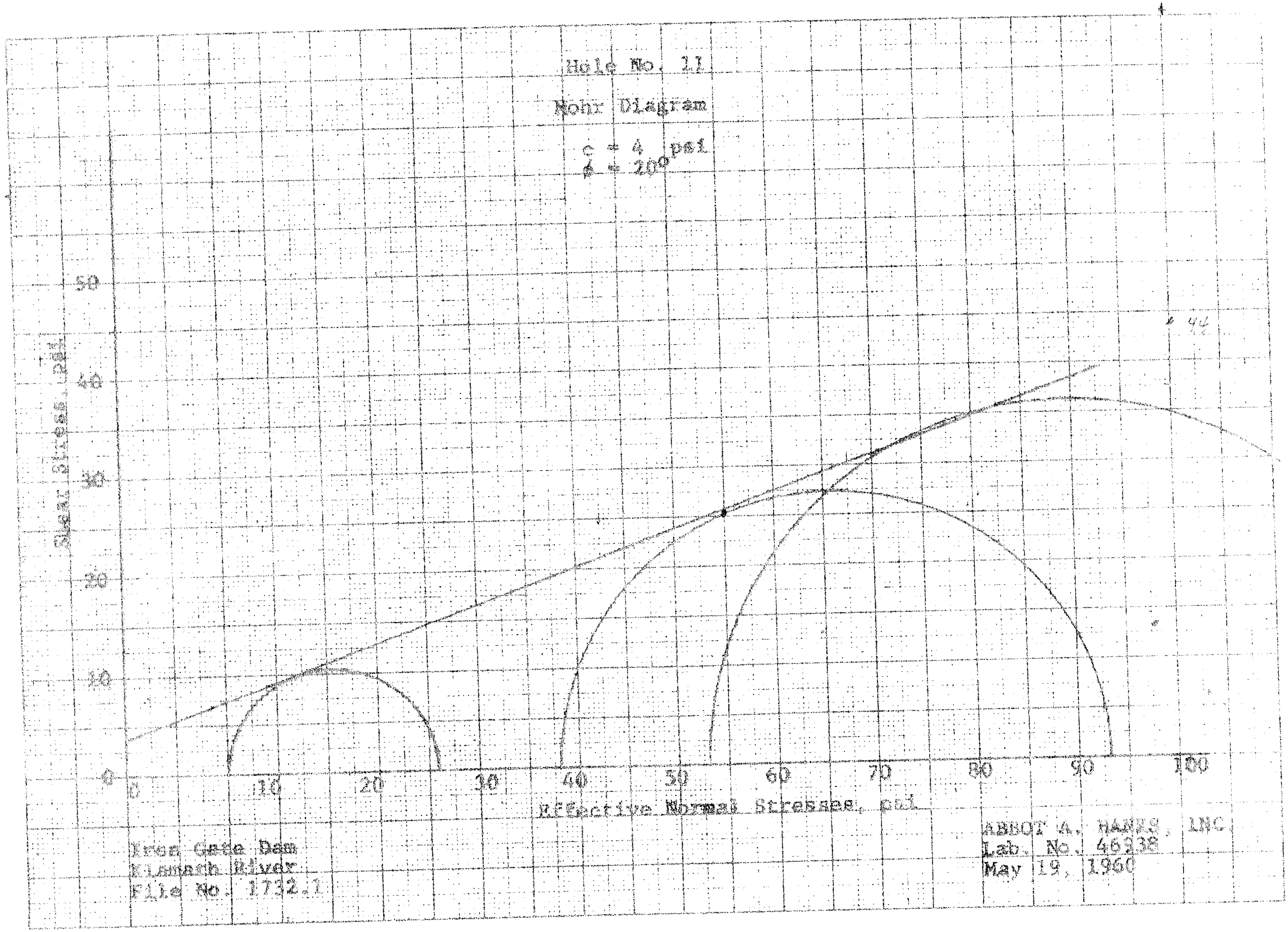


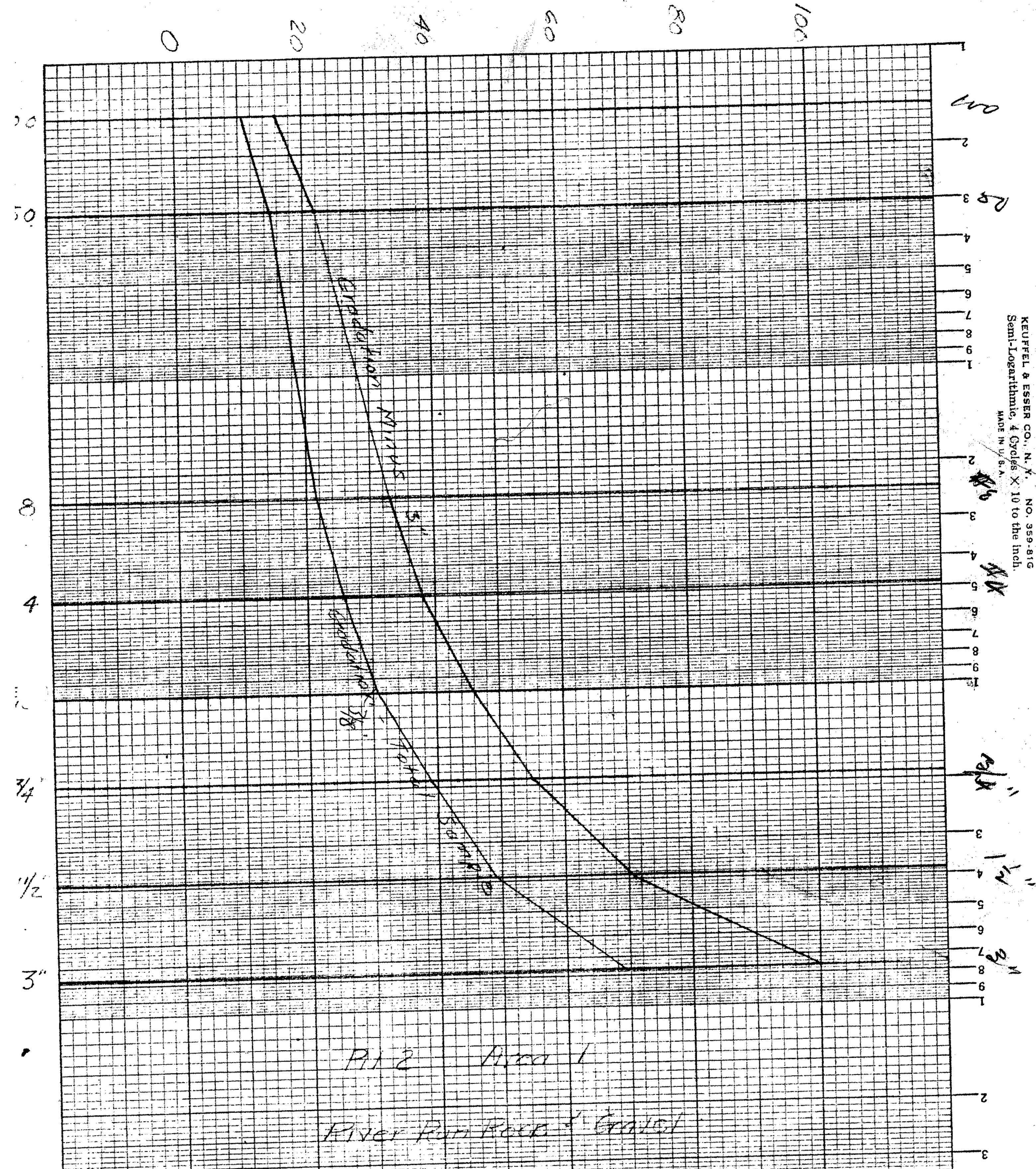
Iron Gate Dam
Klamath River
File No. 1732.1

ABBOT A. HANKS, INC.
Lab. No. 46938
May 18, 1960

K+E 10 X 10 TO THE CM. 359-14G
KEUFFEL & ESSER CO. MADE IN U.S.A.

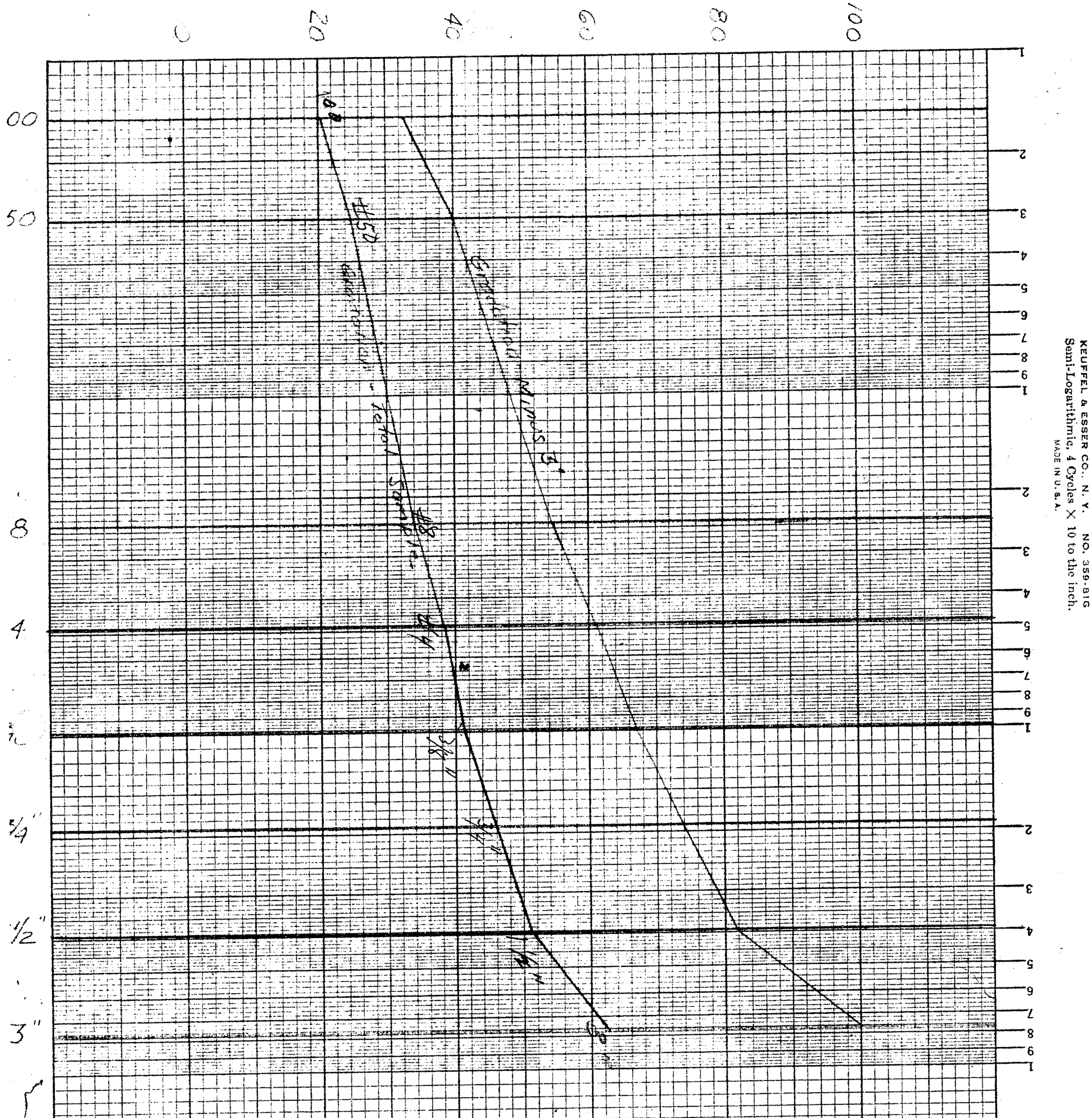
K&E 10 X 10 TO THE CM. 359-14G
KEUFFEL & ESSER CO.





THE CALIFORNIA OREGON POWER COMPANY
IRON GATE DAM
GRAIN SIZE CURVE

NOTE: Curves obtained by dry shaking sample through nest of sieves.

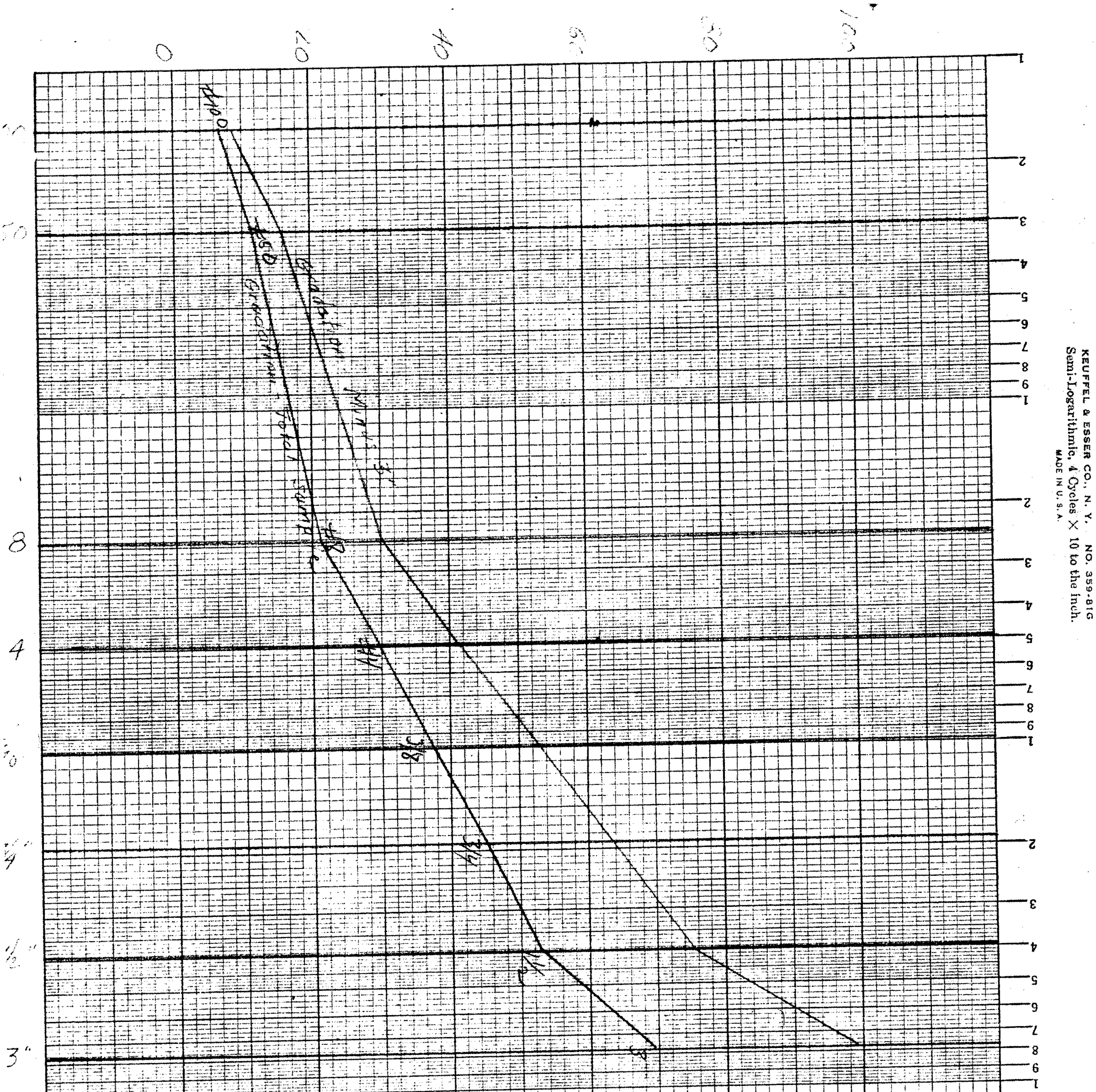


Pit No 3 Area No 1

River Run Rock & Gravel

THE CALIFORNIA OREGON POWER COMPANY
IRON GATE DAM
GRAIN SIZE CURVE

NOTE: Curves obtained by dry shaking sample through nest of sieves.

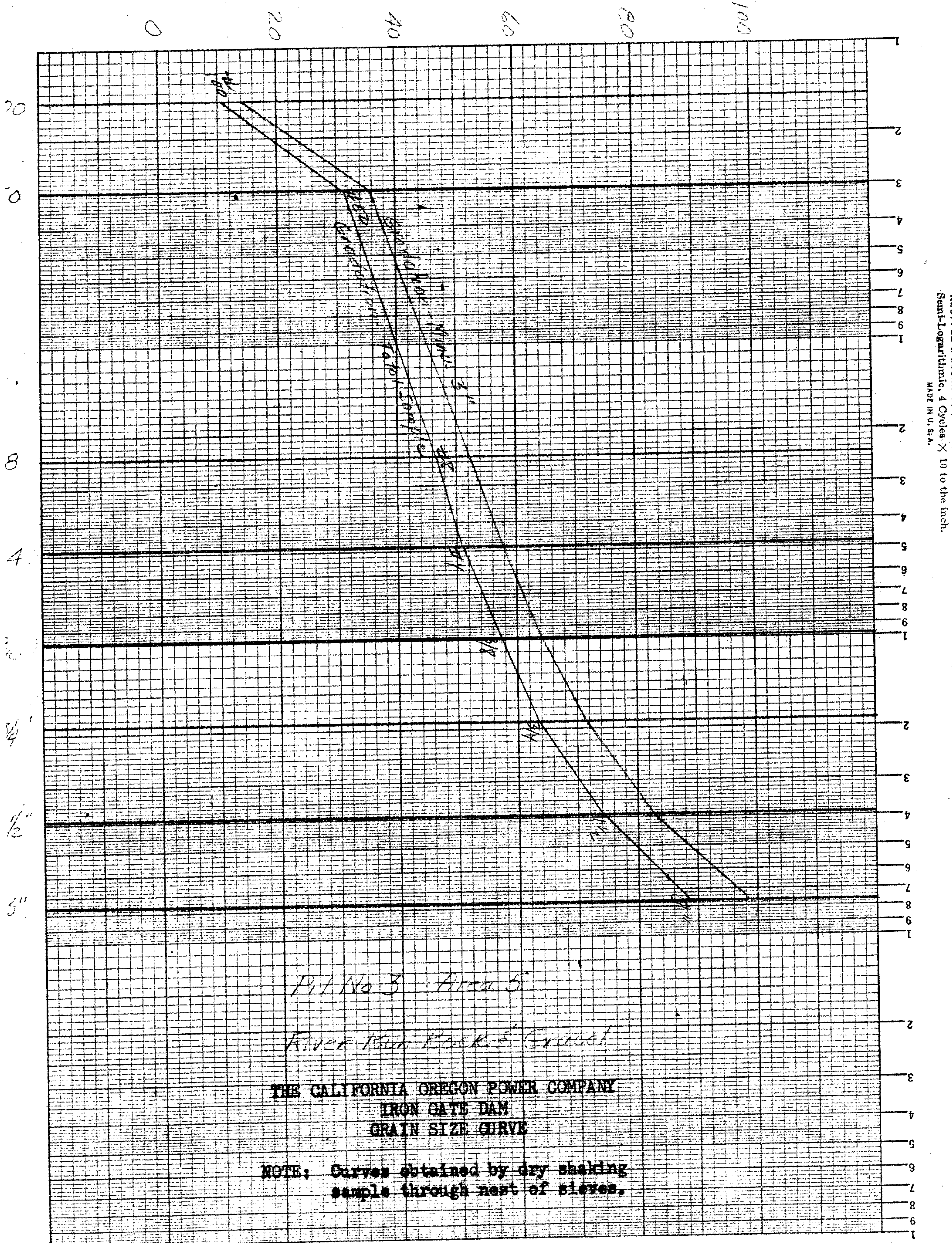


Pit No 2 Area 5

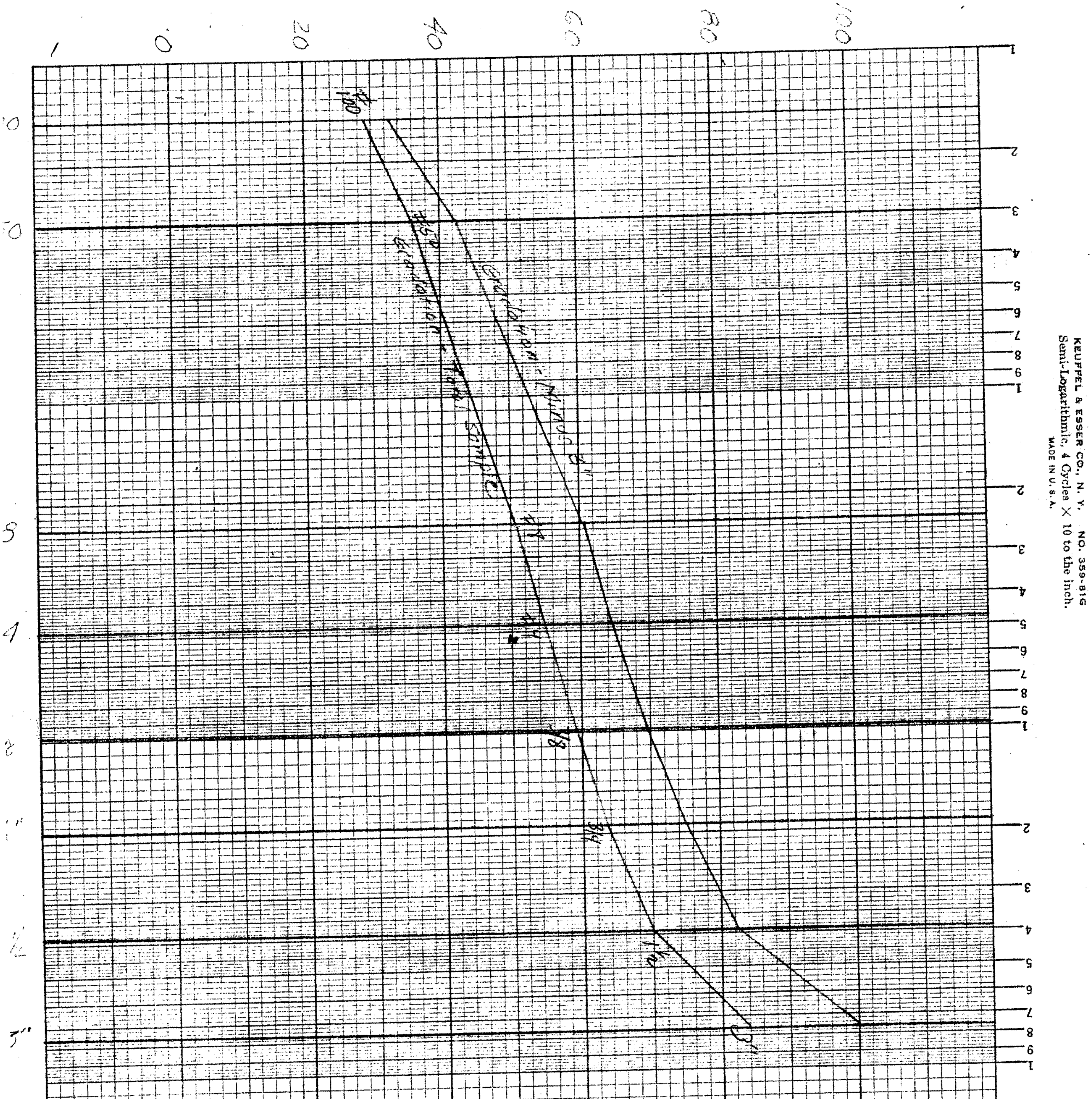
River Run Gravel

THE CALIFORNIA OREGON POWER COMPANY
IRON GATE DAM
GRAIN SIZE CURVE

NOTE; Curves obtained by dry shaking sample through nest of sieves.



KEUFFEL & ESSER CO., N. Y. NO. 359-81G
Semi-Logarithmic, 4 Cycles X 10 to the inch.
MADE IN U. S. A.



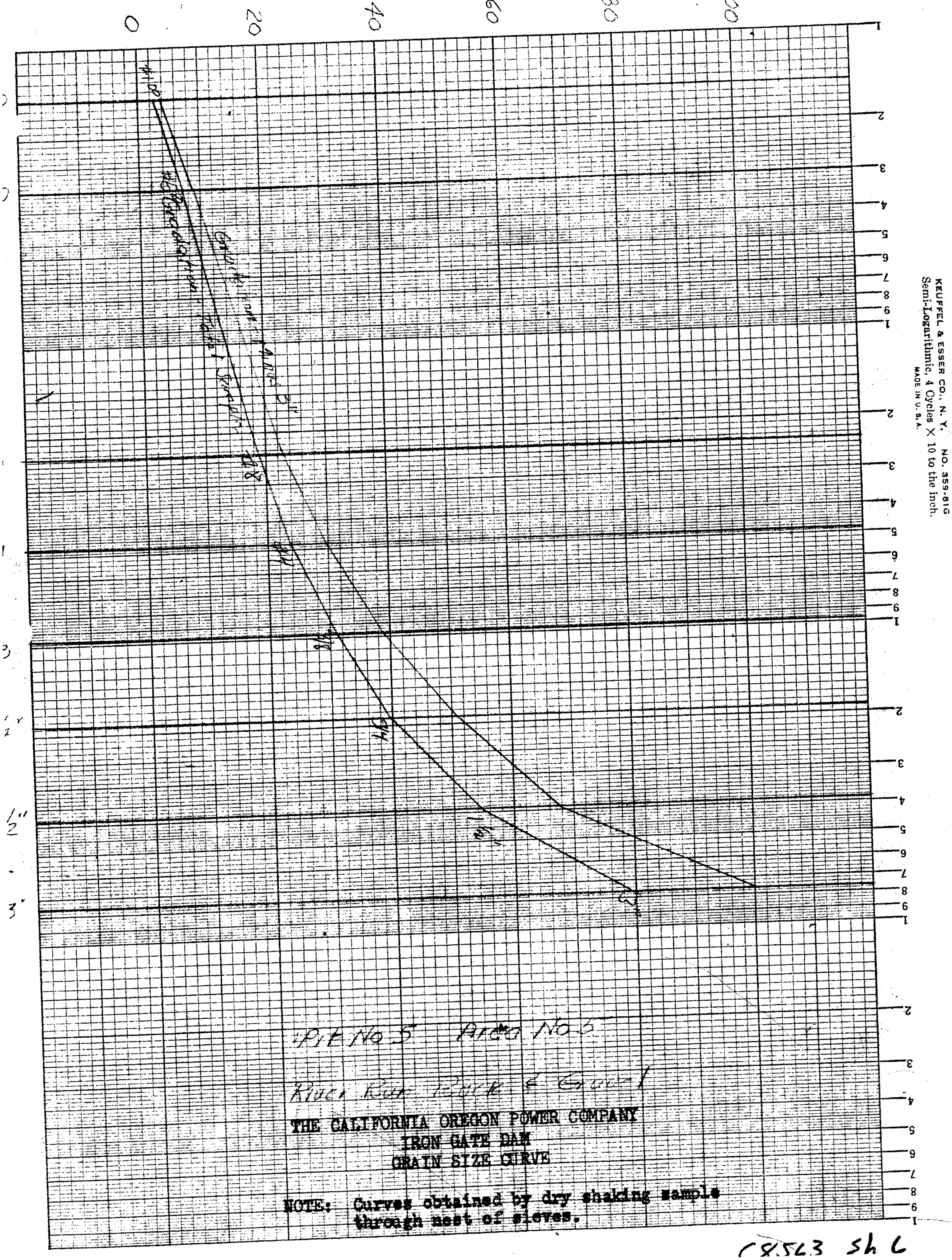
Pit No 4 Area 5

River Run Rock & Gravel

THE CALIFORNIA OREGON POWER COMPANY
IRON GATE DAM
GRAIN SIZE CURVE

NOTE: Curves obtained by dry shaking sample through nest of sieves.

C 9523 545


0
20
40
60
80
100
#10
Gradation Curve for Area 5
Grade limit for Aggregate 3"
#8
#4
3/8
3/4
1 1/2
3"
1"
2"
Pit No 5 Area No 5
River Run Rock & Gravel
THE CALIFORNIA OREGON POWER COMPANY
IRON GATE DAM
GRAIN SIZE CURVE
NOTE: Curves obtained by dry shaking sample through nest of sieves.
KEUFFEL & ESSER CO., N. Y. NO. 359-81G
Semi-Logarithmic, 4 Cycles X 10 to the inch.
MADE IN U.S.A.

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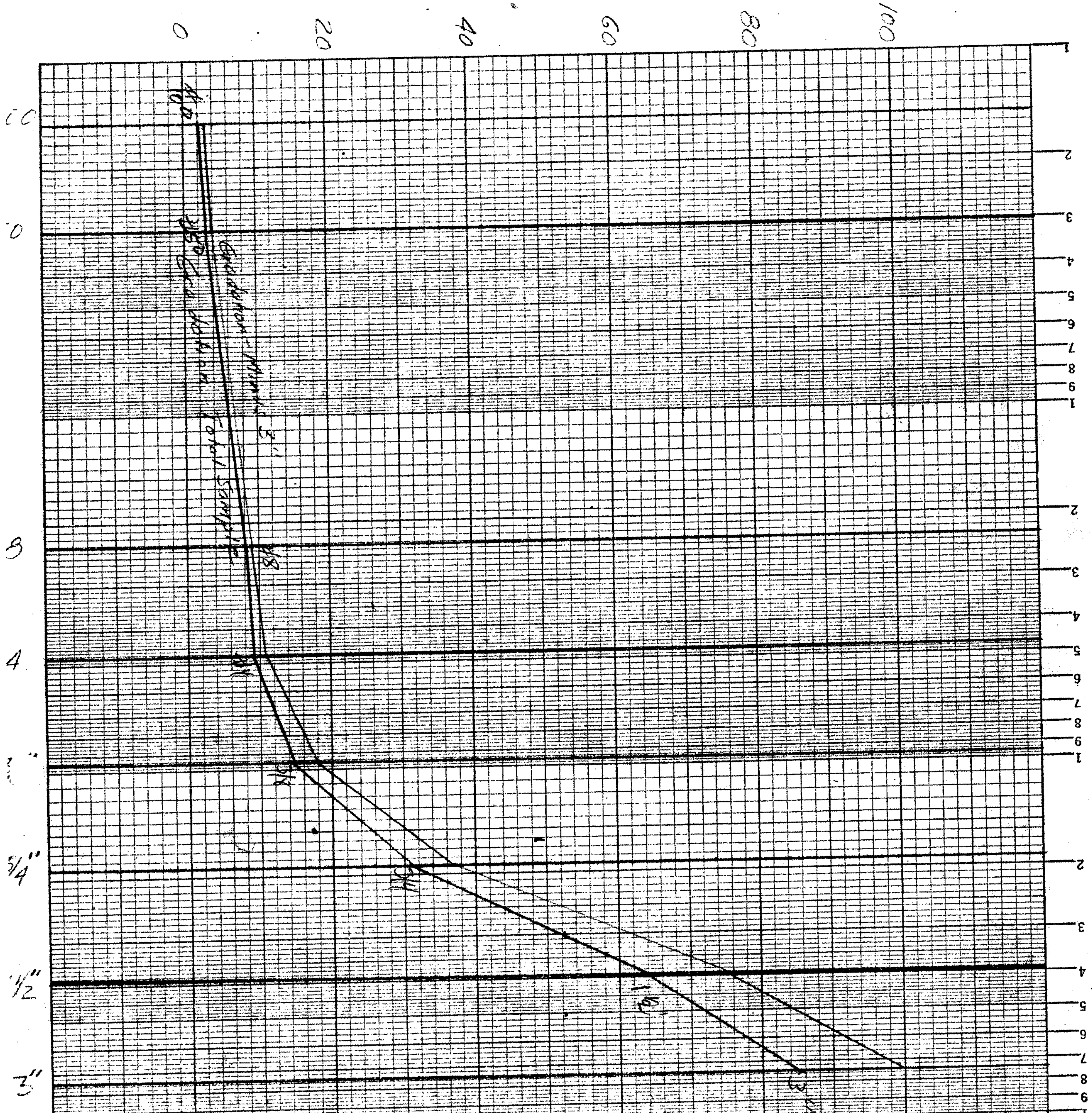
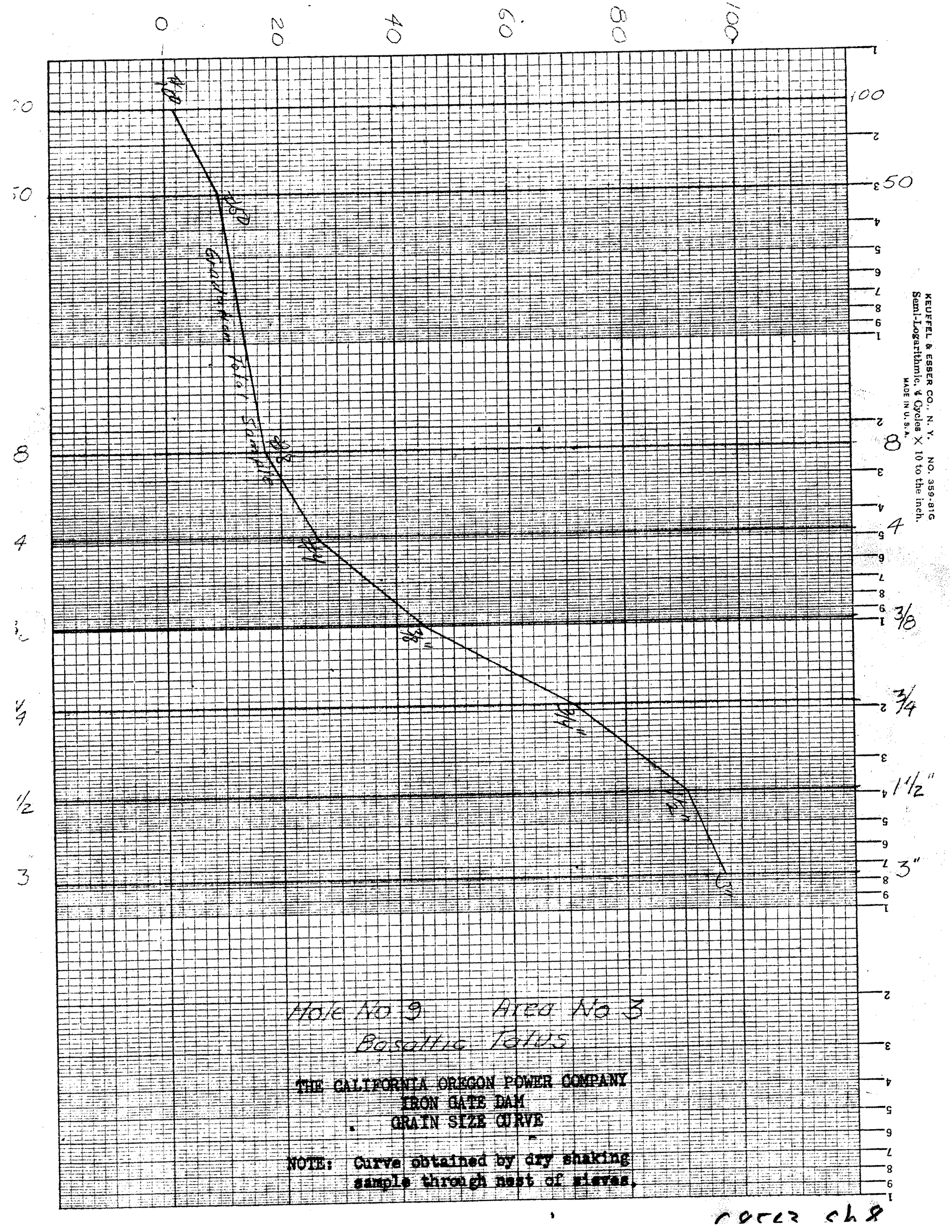


Pit 1 Dodson Area
Basaltic Talus

THE CALIFORNIA OREGON POWER COMPANY
IRON GATE DAM
GRAIN SIZE CURVE

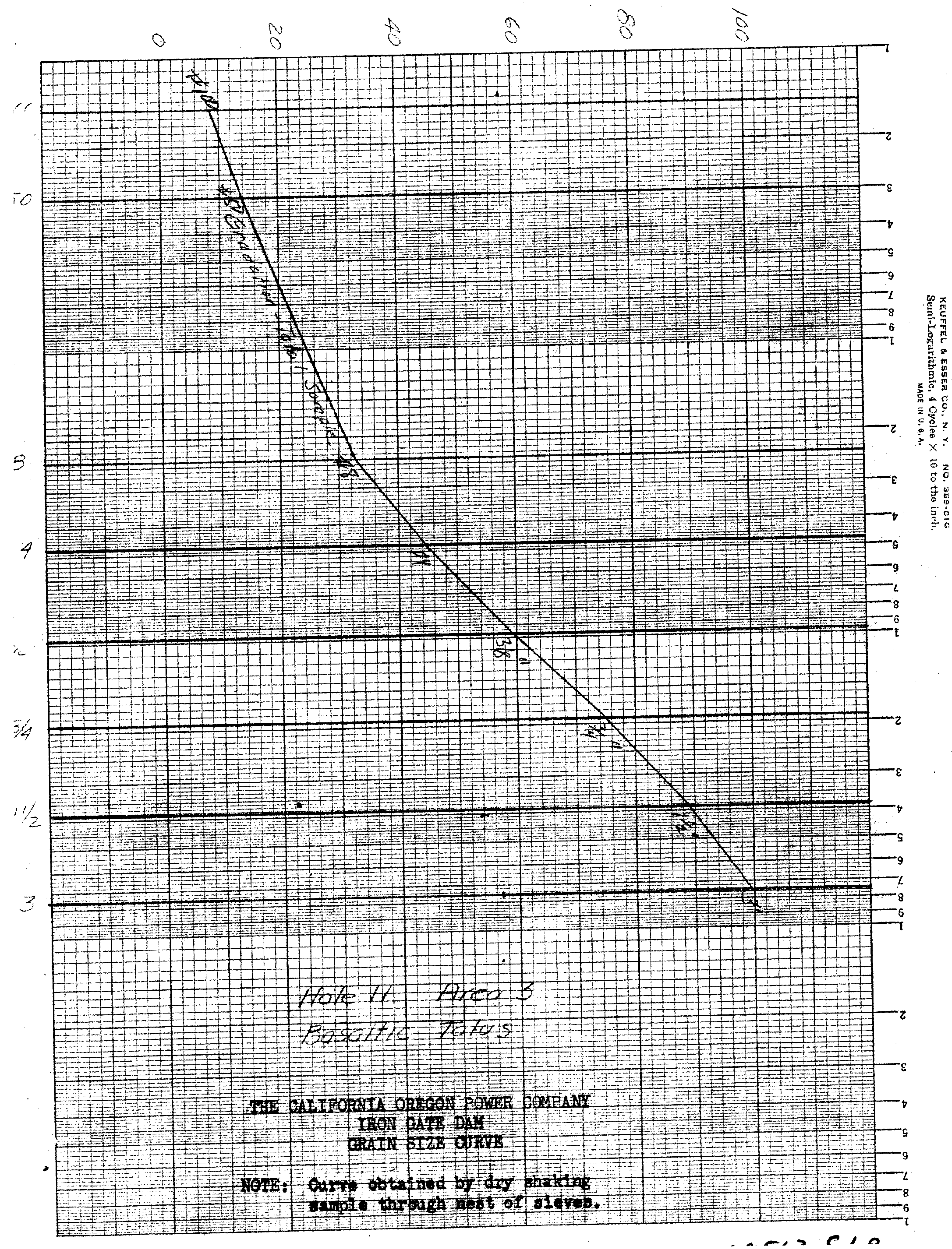
NOTE: Curves obtained by dry shaking sample through nest of sieves.

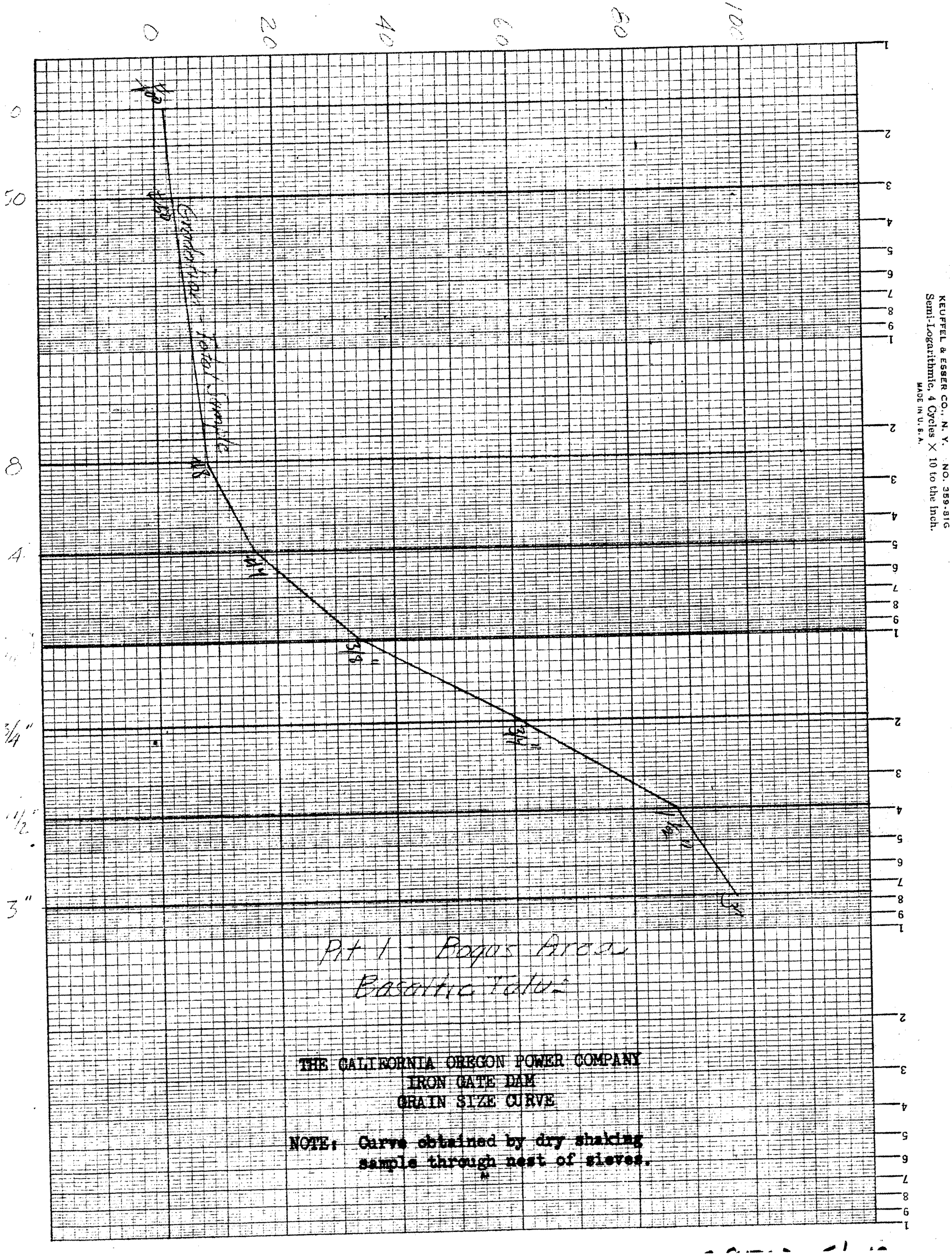
KEUFFEL & ESSER CO., N. Y. NO. 359-81G
Semi-Logarithmic, 4 Cycles X 10 to the inch.
MADE IN U.S.A.



THE CALIFORNIA OREGON POWER COMPANY
IRON GATE DAM
GRAIN SIZE CURVE

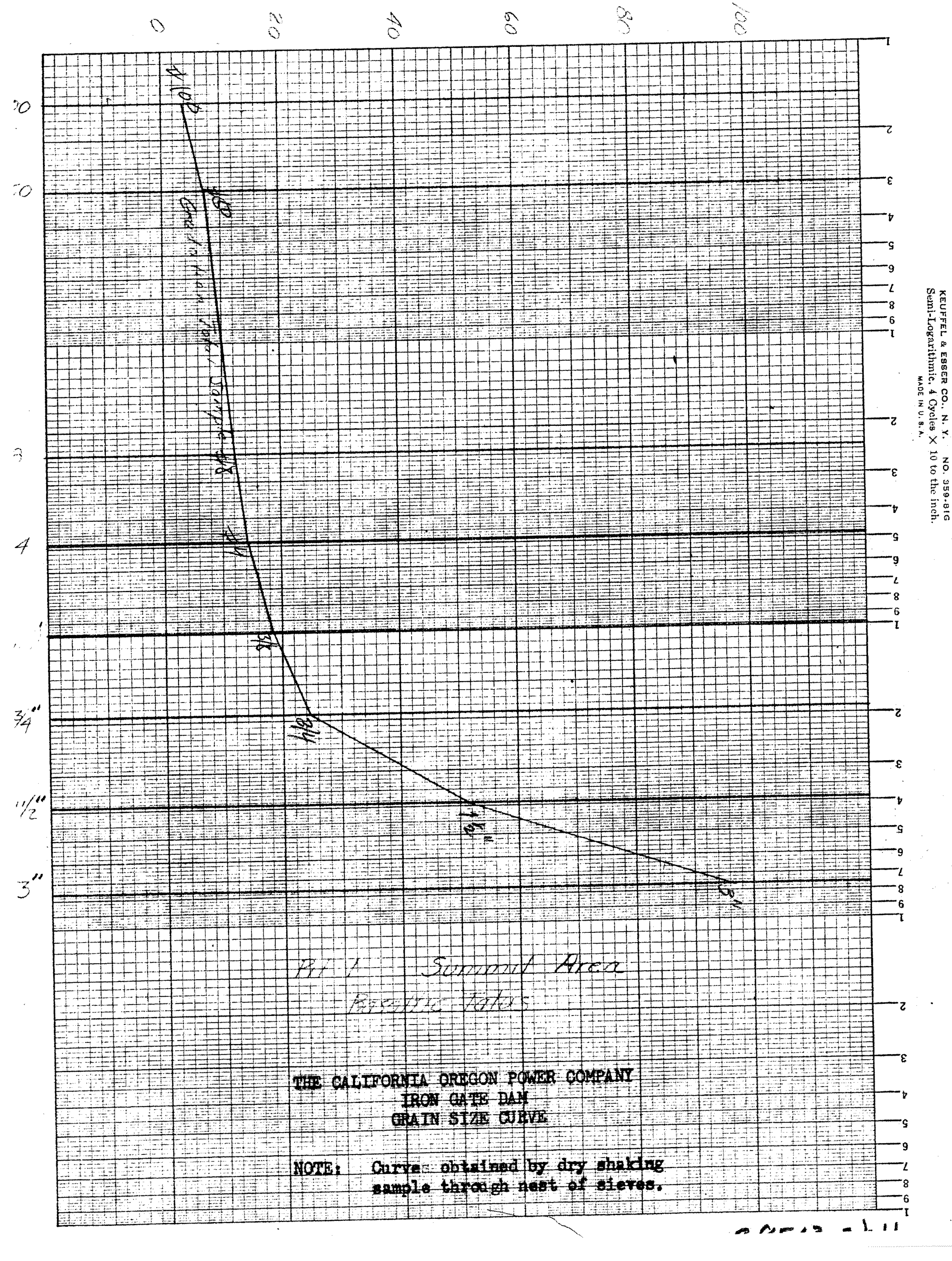
NOTE: Curve obtained by dry shaking sample through nest of sieves.

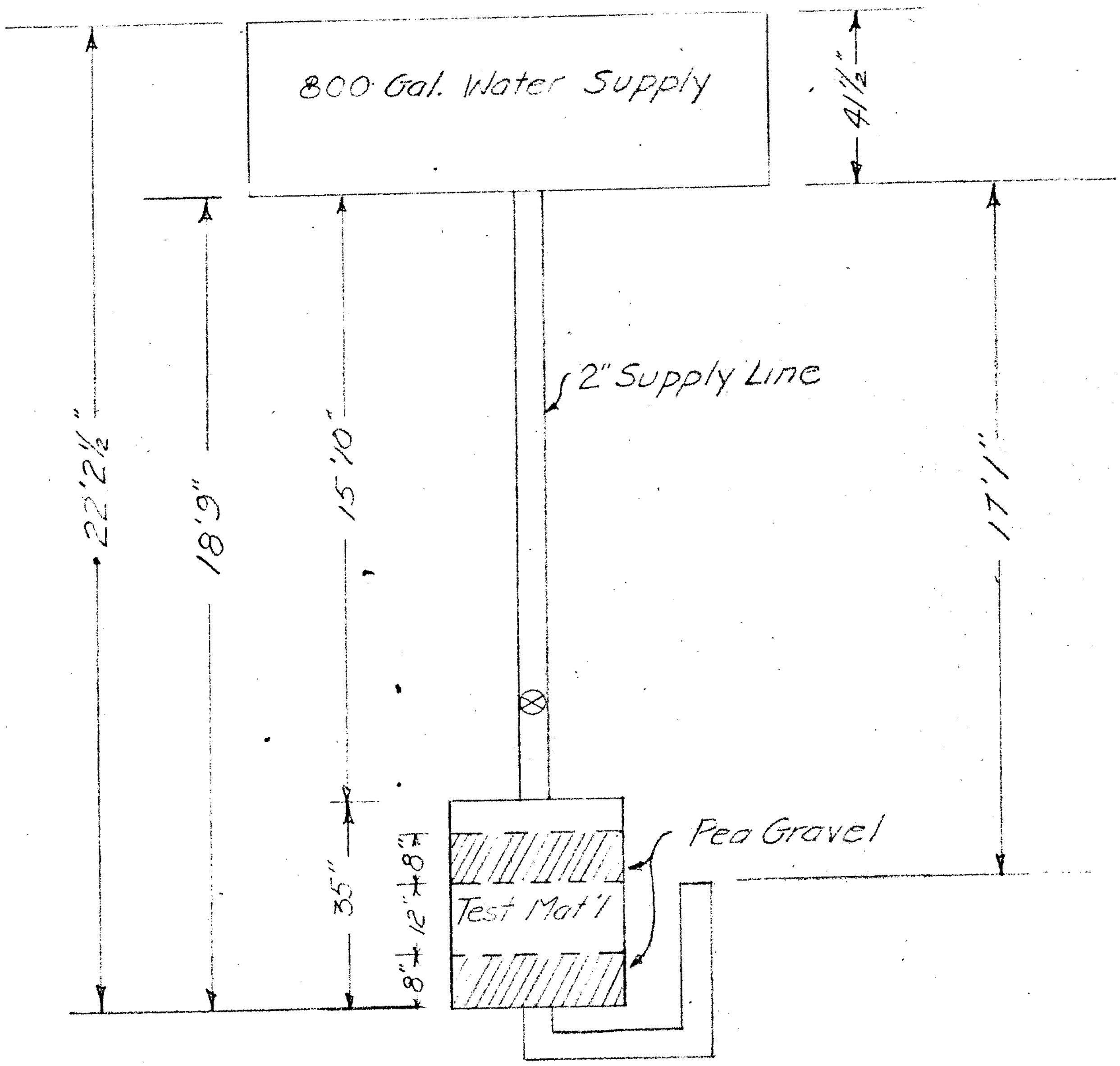

0
20
40
60
80
100
#10
#50 Gradation - Talus Sample
#8
#4
3/8"
3/4"
1 1/2"
3"
Hole 11 Area 3
Basaltic Talus
THE CALIFORNIA OREGON POWER COMPANY
IRON GATE DAM
GRAIN SIZE CURVE
NOTE: Curve obtained by dry shaking sample through nest of sieves.
KEUFFEL & ESSER CO., N. Y. NO. 359-81G
Semi-Logarithmic, 4 Cycles X 10 to the inch.
MADE IN U.S.A.



THE CALIFORNIA OREGON POWER COMPANY
IRON GATE DAM
GRAIN SIZE CURVE

NOTE: Curve obtained by dry shaking sample through nest of sieves.


0
20
40
60
80
100
3/4"
1 1/2"
3"
4
#100
#50
#8
#4
3/8
3/4
1 1/2"
3"
Gradation Total Sample
Pit 1 Summit Area
Basalt Talus
THE CALIFORNIA OREGON POWER COMPANY
IRON GATE DAM
GRAIN SIZE CURVE
NOTE: Curves obtained by dry shaking sample through nest of sieves.
KEUFFEL & ESSER CO., N. Y. NO. 359-81G
Semi-Logarithmic, 4 Cycles X 10 to the inch.
MADE IN U.S.A.



Permeability Test Method
No Scale
Iron Gate Project

DRAWN BY		J. O.
TRACED BY		J. O. REQ.
CHECKED BY		MAP NO.
APPROVED BY	SEC. | TWP. | RANGE	AREA
SCALE	SCHOOL DIST. | ROAD DIST.	RURAL ☐ URBAN ☐
DATE 7-20-60	THE CALIFORNIA OREGON POWER COMPANY	C 8560

BRANCH LABORATORY 1086 MARTIN AVENUE • SANTA CLARA • CHERRY 8-5262 CABLE HANX
BRANCH OFFICE 61 JORDAN STREET • SAN RAFAEL • GLENWOOD 4-8650

ABBOT A. HANKS, INC.
ESTABLISHED 1866
1300 SANSOME STREET • SAN FRANCISCO 11, CALIFORNIA • EXBROOK 7-2464

Engineers
Assayers
Chemists
Metallurgists
Spectrographers
Soils and Foundations
Consulting · Testing · Inspecting

File No. 1732.2
Lab. No. 52348

August 8, 1961

The California Oregon Power Company
Iron Gate Project
Post Office Box 201
Hornbrook, California

Re: Iron Gate Dam - P. O. #39636
Soil Tests, Sample 16+00 300' L

Gentlemen:

Based on tests of four specimens compacted in the range of 81 to 82 lb per cu ft, it appears that the intergranular strength factors of the above sample in a consolidated shear test with pore pressures measured are as follows:

Friction angle	11½ - 15½ degrees
Cohesion	450 - 800 lb/sq ft.

This soil is a highly plastic, impervious clay. Consolidation was extremely slow, requiring 10 days to complete the consolidation and saturation of each 2 in. diameter by 4 in. length specimen. An extremely long seasoning period was also necessary to attain uniformity of moisture content prior to compaction of specimens at the specified moisture content.

We are proceeding with tests of Sample 12+00 400' L, and will submit details of all tests upon completion.

Very truly yours,

ABBOT A. HANKS, INC.

L. O. Long

L. O. Long

LOL:hms
Reports to:
3-Iron Gate Project, Hornbrook, Calif.
1-The California Oregon Power Company, Medford, Ore.

BRANCH LABORATORY: 1086 MARTIN AVENUE • SANTA CLARA • CHERRY 8-5262
BRANCH OFFICE: 61 JORDAN STREET • SAN RAFAEL • GLENWOOD 4-8650

CABLE: HANX

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Engineers
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Spectrographers
Soils and Foundations
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File No. 1732.2
Lab. No. 52348, 52871

October 20, 1961

Mr. M. L. Warren
Assistant Chief Engineer
The California Oregon Power Co.
216 Main Street
Medford, Oregon

Re: Iron Gate Project Samples

Dear Mr. Warren:

Attached are the findings from triaxial shear tests performed on soil samples marked "12+00, 400'L", and "16+00, 300'L".. The triaxial tests were performed in the same manner as described in our letter of June 29, 1960.

Complete saturation was not attained in the tests because, when confined under the higher lateral pressures, the specimens were virtually impermeable, and complete saturation could not be attained even by application of a high vacuum on the top of the specimens and a small positive pressure on the base.

You will note that we did not submit data for a specimen of sample 16+00, 300'L at 80 psi chamber pressure. The data for this specimen was not consistent with the remainder of the test data, and it appears likely that there was leakage of the membrane during the test. If you feel that a repetition

MW

of the test at 80 psi would serve a useful purpose, we should be pleased to repeat the test.

We should be pleased to discuss any questions in connection with these tests.

Very truly yours,

ABBOT A. HANKS, INC.

L. O. Long

L. O. Long

LOL:hms
Encls.
Reports to:
3-The California Oregon Power Co.

Iron Gate Project
The California Oregon Power Company
File No. 1732.2

Abbot A.Hanks, Inc.
Lab. No. 52348
October 17, 1961

TABLE NO. I

Sample: 16 + 00 300'L.
Soil Type: Dark yellow-brown clay.

Sample	A	B	C	D
Chamber Pressure, psi	15.5	15	50	50
Unit Dry Weight at Compaction, lb/ft^3	81.3	81.2	82.7	82.5
Moisture Content at Compaction, %	25.0	24.6	24.2	23.7
Unit Dry Weight at Test, lb/ft^3	76.8	74.7	84.7	84.7
Moisture Content at Test, %	42.3	36.8	--	35.3
Degree of Saturation at Test, %	94	80	--	97
Maximum Deviator Stress, psi	15	18	43	35
Pore Pressure at Maximum Deviator Stress, psi	6	4	1	2

Iron Gate Project
The California Oregon Power Company
File No. 1732.2

Abbot A. Hanks, Inc.
Lab. No. 52871
October 17, 1961

TABLE NO. II

Sample: 12 + 00, 400'L.
Soil Type: Very dusky red clay.

Sample	A	B	C
Chamber Pressure, psi	15	50	80
Unit Dry Weight at Compaction, lb/ft^3	88.6	86.7	86.9
Moisture Content at Compaction, %	24.5	23.5	24.5
Unit Dry Weight at Test, lb/ft^3	81.6	89.2	91.0
Moisture Content at Test, %	36.3	30.7	27.2
Degree of Saturation at Test, %	89	94	86
Maximum Deviator Stress, psi	16	48	81
Pore Pressure at Maximum Deviator Stress, psi	1	4	1

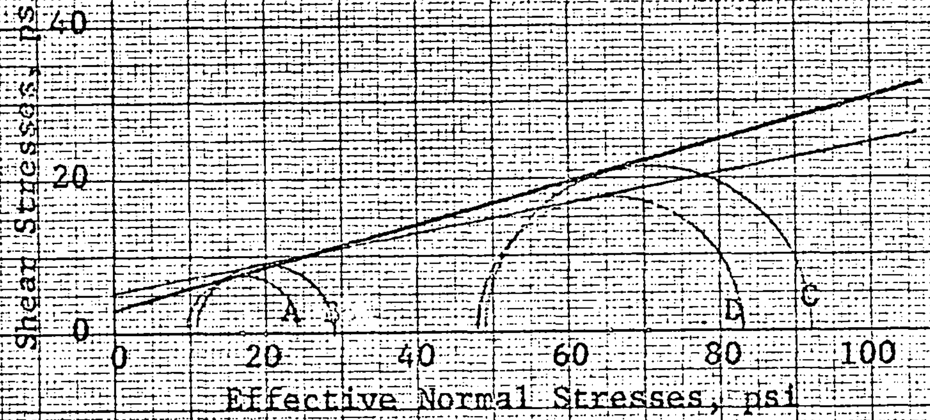
hms

K+E 10 X 10 TO THE CM. 359-14G
KEUFFEL & ESSER CO.

Sample 16+00, 300'L

MOHR DIAGRAM

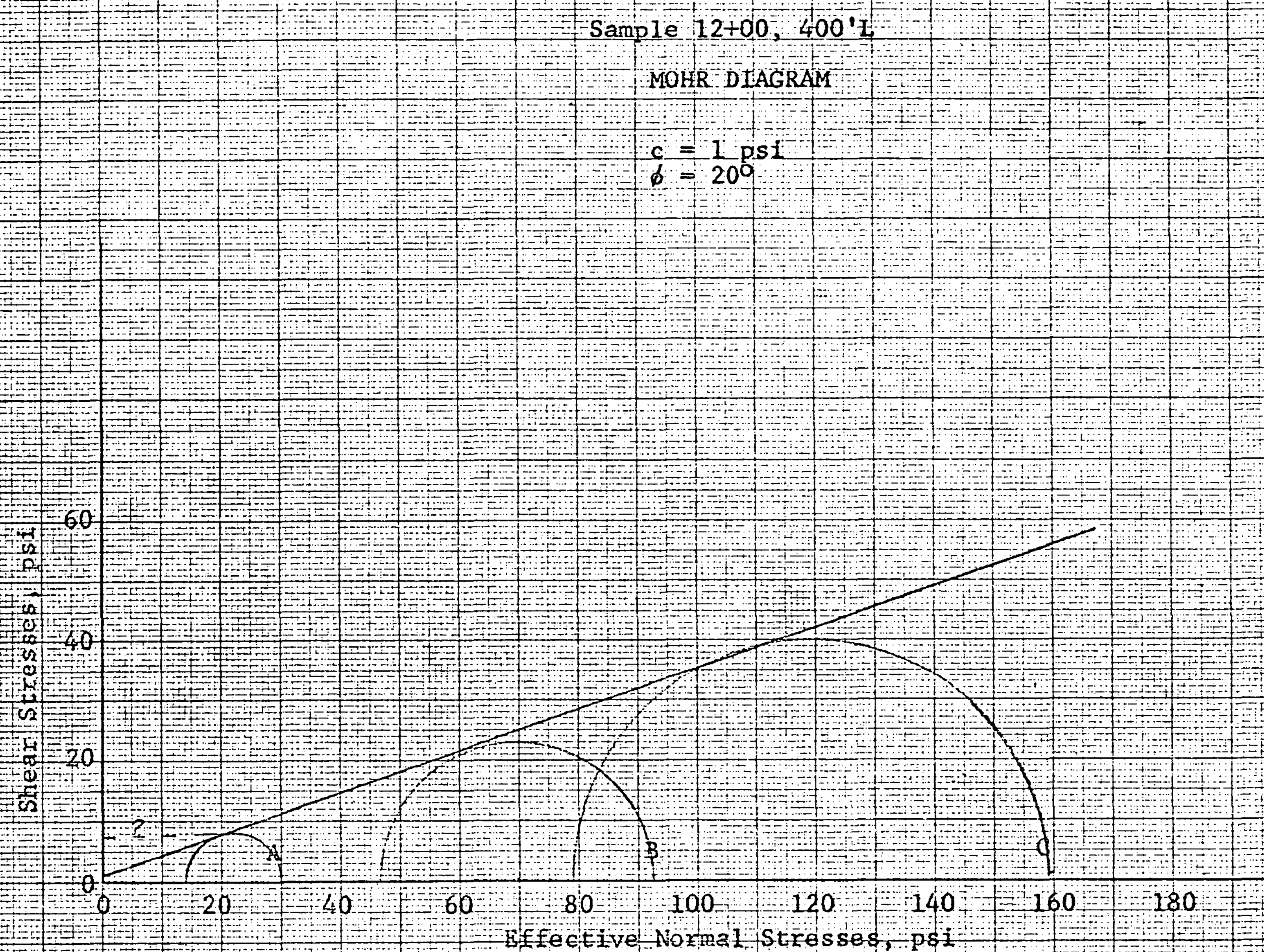
c = 3-5½ psi
ϕ = 11-15°



Iron Gate Dam
File No. 1732.2

ABBOT A. HANKS, INC.
Lab. No. 52348

K+E 10 X 10 TO THE CM. 359-14G
KEUFFEL & ESSER CO.



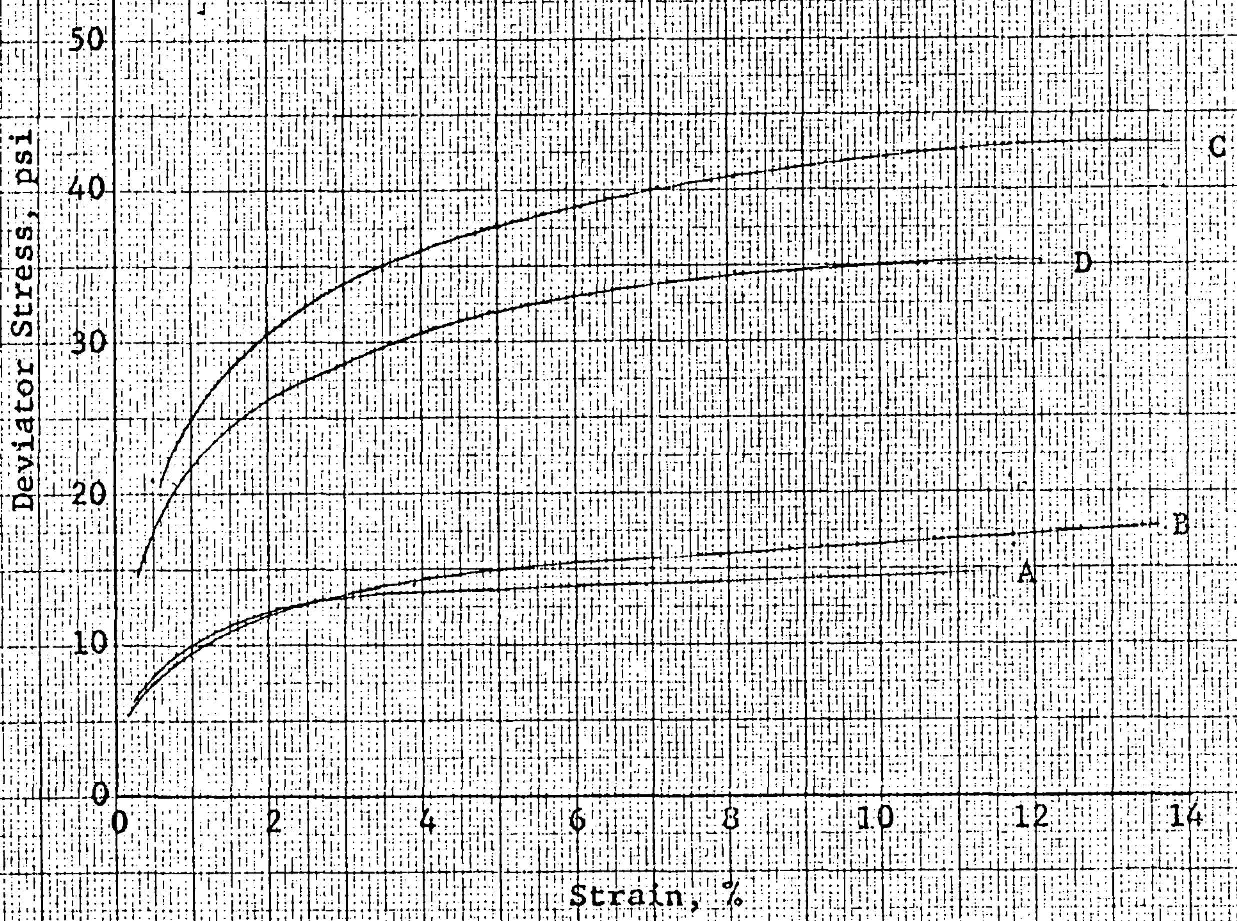
Iron Gate Dam
File No. 1732.2

ABBOT A. HANKS, INC.
Lab. No. 52871

Sample 16+00, 300'L

TRIAXIAL SHEAR TEST

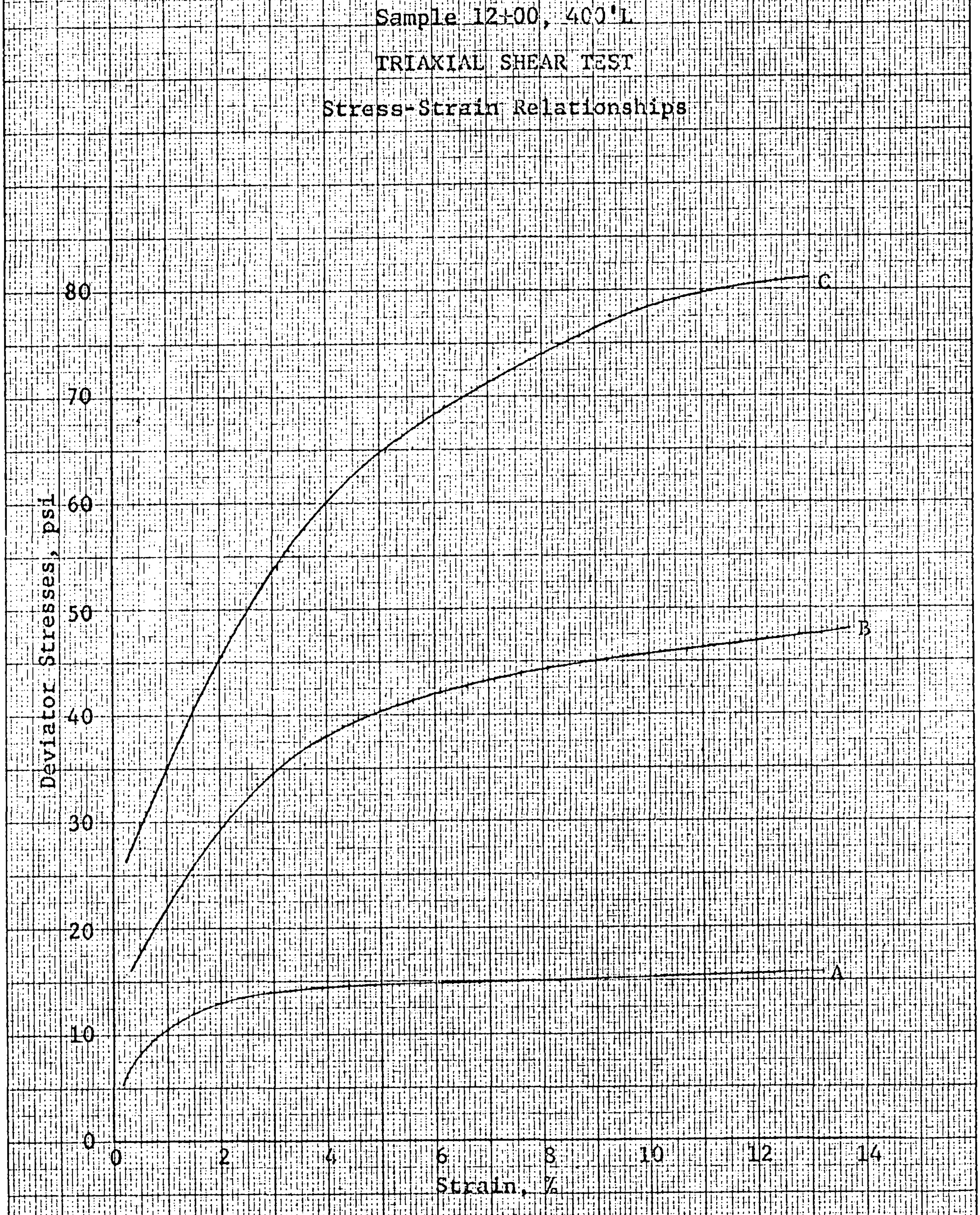
Stress-Strain Relationships



Iron Gate Dam
File No. 1732.2

ABBOT A. HANKS, INC.
Lab. No. 52348

10 X 10 TO THE CM 359-14G KEUFFEL & ESSER CO.



Iron Gate Dam
File No. 1732.2

ABBOT A. HANKS, INC.
Lab. No. 52871

10 X 10 TO THE CM. 359-14G
KEUFFEL & ESSER CO.

Attachment B Falling Head Permeability Tests

Undrained, consolidated triaxial shear tests with pore pressure measurements, consolidation tests, and constant head permeability tests were performed on representative samples of material for each hole by Abbot A. Hanks Laboratory in San Francisco. Before each test the material was compacted to near optimum moisture and maximum dry density. The results of these tests are shown in Abbot A. Hanks report as Plate IV. For convenient reference, the permeability coefficient, the angle of internal friction and cohesion for the material from each hole are tabulated below:

HOLE NO.	COHESION p.s.i.	psf	ANGLE OF INTERNAL FRICTION	PERMEABILITY FT/YEAR Less than	COEFFICIENT CM/SEC Less than
1.	9	1300	10°	.01	10-8
2.	6	860	17°	-	-
3.	10	1440	14°	.01	10-8
4.	4	580	30°	.01 - .04	1-4 x 10-8
7.	5	720	21°	.01	10-8
8.	3	430	16°	.01	10-8
11.	4	580	20°	.01	10-8

844 psf 7) 128 (18° av

The quantities of the impervious materials available in Areas "A" and "5", are estimated to be 264,000 cubic yards. The quantities in Area "1" are estimated to be 57,000 dubic yards. However, most of Area "1" will be inundated by backwater from the construction of the cofferdam so, in order to utilize this material, it will be necessary that it be stockpiled, which does not seem practical.

2. Pervious Shell Materials

Two types of pervious materials were investigated:

A. Gravels in the flood plain of the river in Areas "1" and "5"

B. Talus deposits of basaltic rock in Area "3", Bogus Creek Area, Summit Area and Dodson Area.

Sample	% Passing #100 Sieve	% Moisture	Dry Density Lbs/cu ft	Wet Density Lbs/cu ft	Permeability Coefficient Ft/Day
Area "1"					
Pit 2, 1-14' Depth	15	8	119.5	129.2	11.1
Pit 3, 3-12' "	21	7	128.5	138.2	17.6
Area "5"					
Pit 2, 2-16' Depth	9	9	129.5	141.5	12.6
Pit 3, 2-12' "	14	9	137.5	151.0	.59
Pit 4, 4-20' "	33	11	105.0	118.0	8.94
Pit 5, 4-16' "	3	9	125.0	137.0	3.95

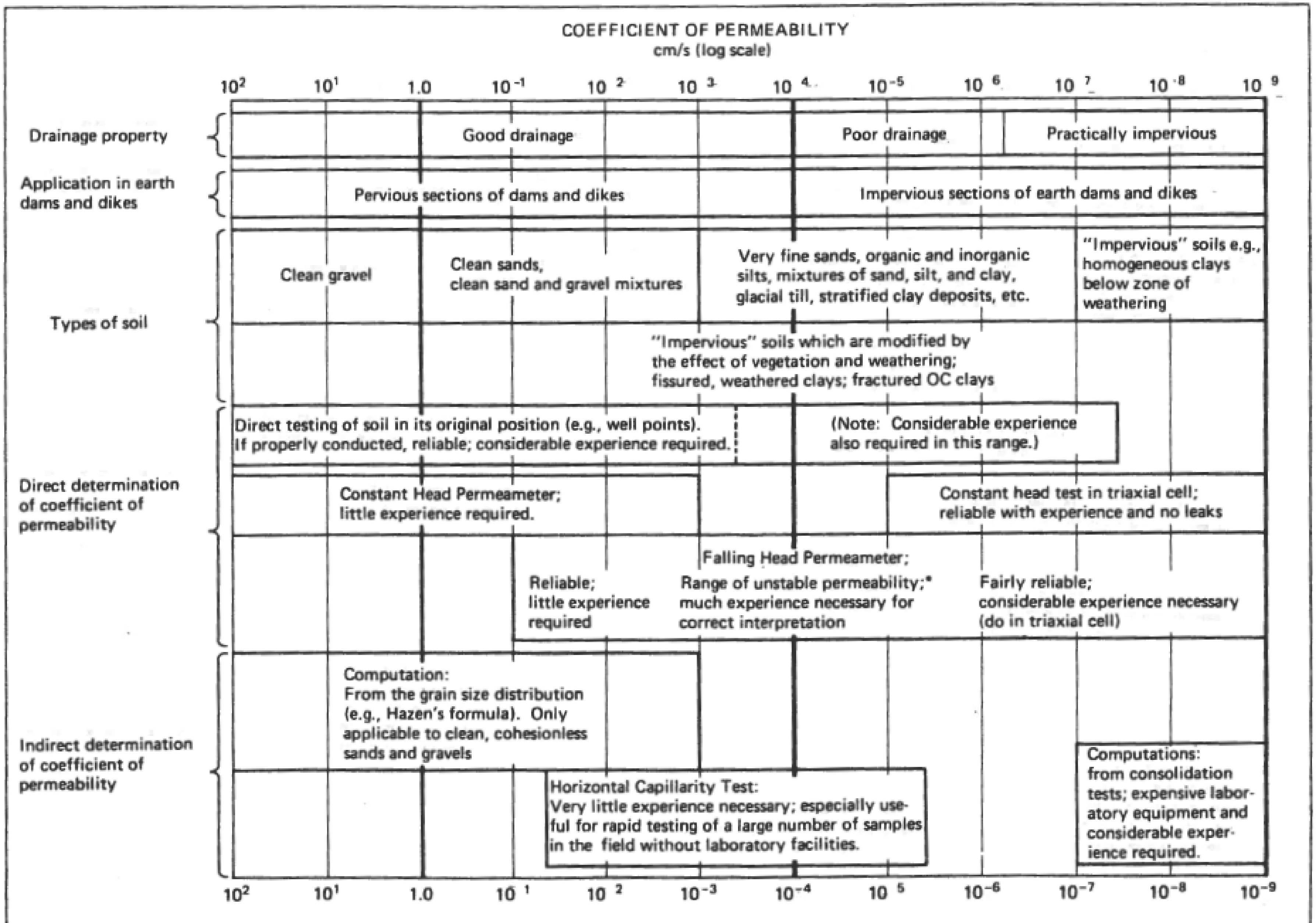
The estimated quantity of gravel materials in Area "1" is 90,000 cubic yards, which can be obtained without stockpiling the impervious material overlining the gravels, and in Area "5", is 381,000 cubic yards, making a total of 471,000 cubic yards.

Talus deposits.

The talus material was tested in a manner similar to that described in the section headed "Gravels". No settlement was noted after the permeability test on any of the samples of talus material. The results of the grain size distribution tests are shown on Drawing C-8563, Sheets 7 through 11, and attached hereto as Plate V. The results of the permeability tests are tabulated below:

Sample	% Passing #100 Sieve	% Moisture	Dry Density Lbs/cu ft	Wet Density Lbs/cu ft	Permeability Coefficient Ft/Day
Area "3"					
Hole 9, 0-12' Depth	7	8	125.2	135.5	21.2
Hole 11, 0-6' "	9	13	116.5	134.0	5.45
Dodson Area					
Pit 1, 0-10' Depth	2	7	118.5	127.2	19.2
Bogus Area					
Pit 1, 10' Depth	8	8	113.3	124.0	10.5
Pit 1, 0-9' "	2	9	112.2	122.5	25.9
Summit Area					
Pit 1, 0-7' Depth	4	9	117.5	128.5	9.65

Attachment C Typical Permeability Ranges



*Due to migration of fines, channels, and air in voids.

Attachment D Direct Shear Test Results

Direct shear tests were performed on each sample by Pittsburgh Laboratories. Results were unobtainable on Type 3 and Type 4 because of shear ring binding and mechanical interlocking of coarse sand particles. It is felt however that the shearing resistance of Types 3 and 4 is very similar to Types 2 and 3 and therefore may be used in design. The shear resistance of Types 1 and 2 are as follows:

Type No.	1	2	Avg.
Cohesion (Tons/sq/ft/)	.37	1.64	1.00
Angle of Internal Friction (Ø)	34.4°	19.4°	26.9°

A stability analysis was made using approximate methods, the above average test values and the section shown on Plate V. This analysis indicates that a factor of safety of approximately 2.25 will be obtained.

The volume of the material available as determined from the drill holes is as follows:

Types 1, 2 and 3 - 96,000 cubic yards
" 4 - - - - - - - 55,000 " "

Field observation indicates that additional material is available on the borders of the areas drilled which is similar to the material tested.

CONCLUSIONS:

It is concluded that the earthen embankment of Big Bend Dam may be constructed of the materials which have been analyzed in this report to the approximate typical section as shown on Plate V, and to specifications attached hereto entitled "Earthwork Specifications for Big Bend Dam" and labelled "Appendix 1".

The impervious core of the embankment, Zone 1, should be constructed of the materials classified in this report as Types 1, 2 and 3. The moisture content of these materials is about 25% and the maximum dry density between 85 and 95 lbs per cubic foot. The "yardstick" for construction should be set for this section at 25% moisture and 90 lbs per cubic foot. As construction progress is made, the "yardstick" may be varied to more closely conform to field results.

Zone 2 of the embankment, the semi-pervious section, should be constructed of the material classified as Type 4. The optimum moisture content of this material is about 18% with a dry density, including the gravel, of 128 lbs per cubic foot. Again, as field results are obtained, this "yardstick" may be varied.

Attachment E Compaction Test Results

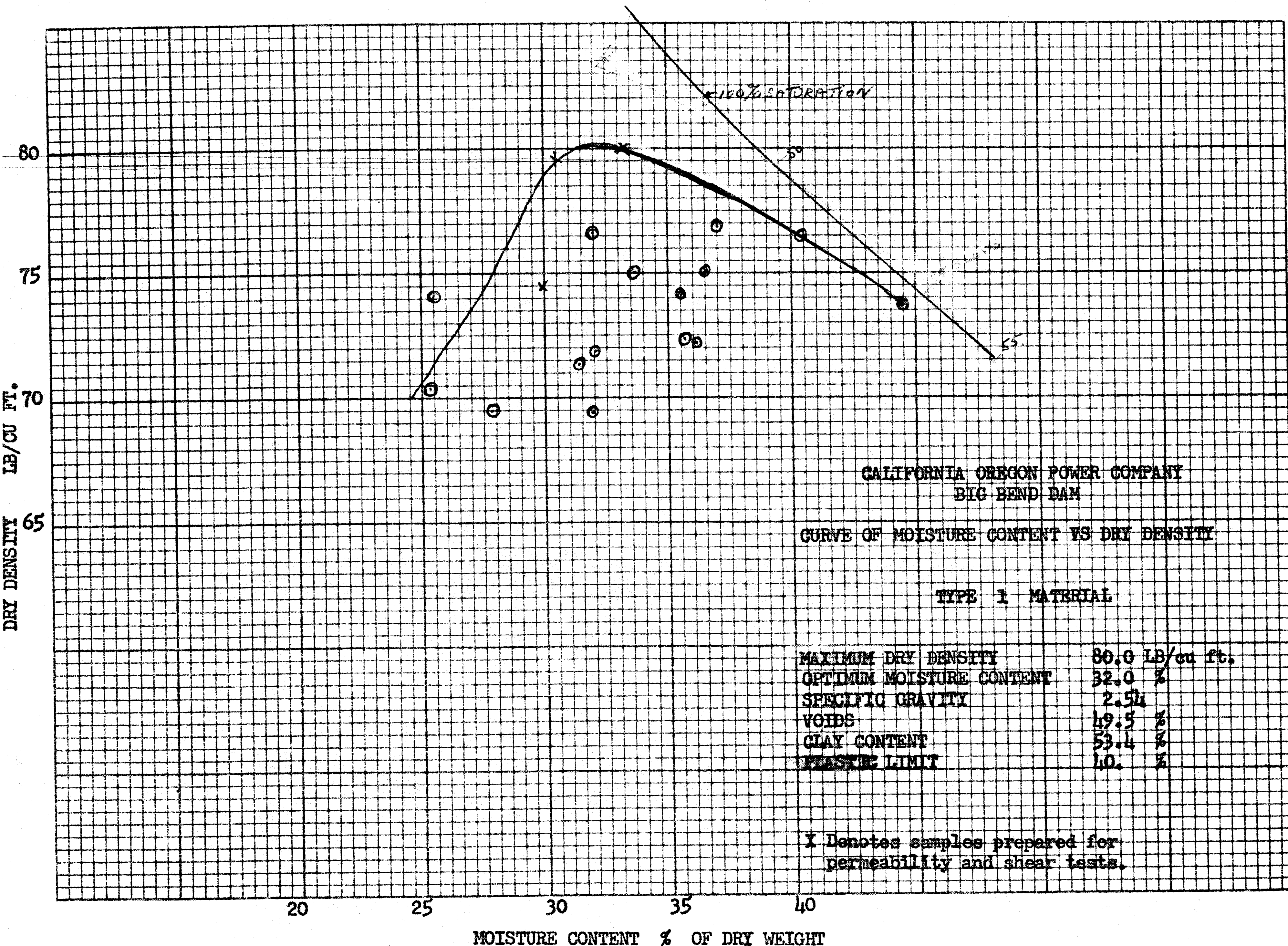
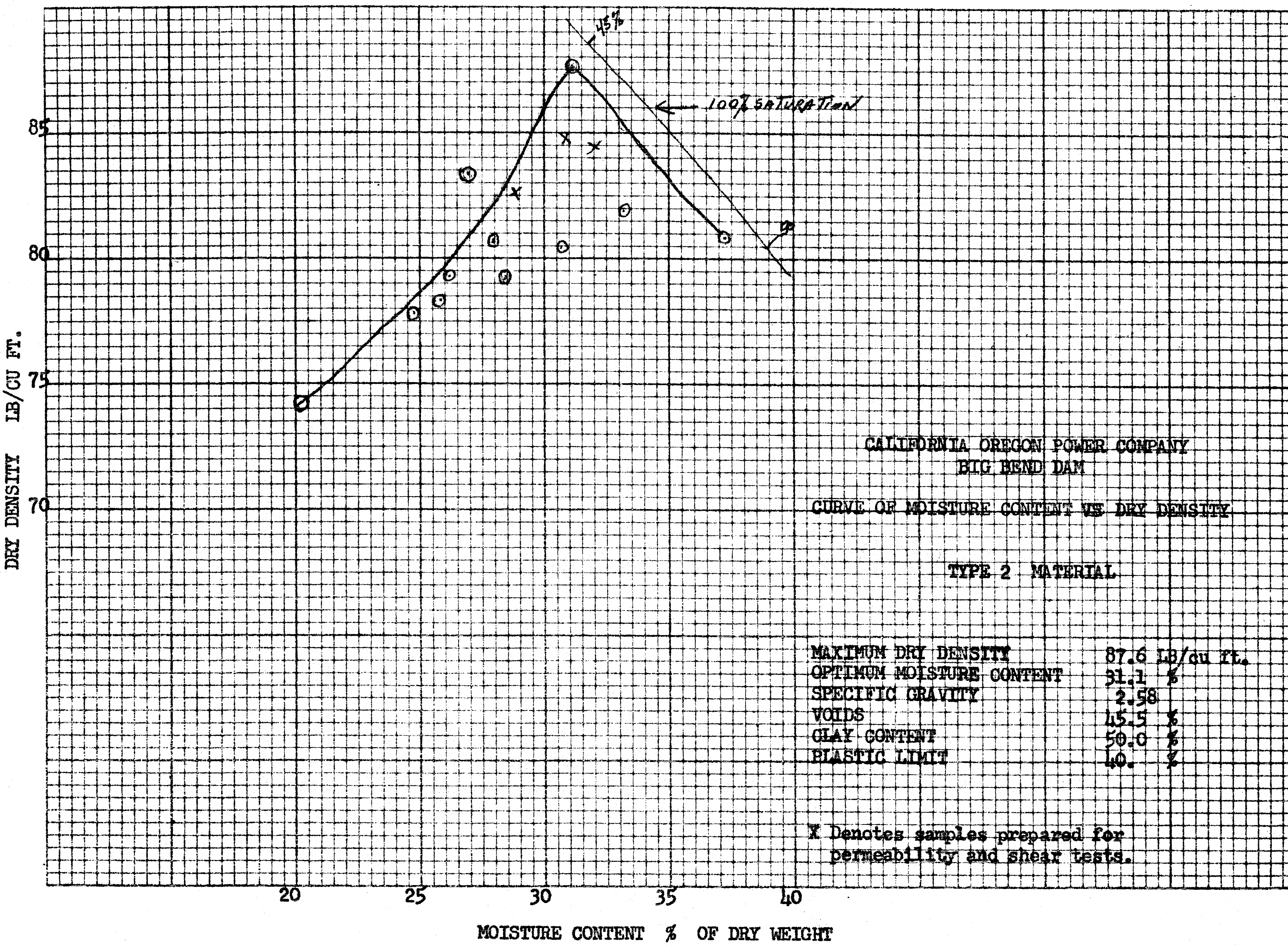


PLATE III - Sheet No. 1



CALIFORNIA OREGON POWER COMPANY
BIG BEND DAM

CURVE OF MOISTURE CONTENT VS DRY DENSITY

TYPE 2 MATERIAL

MAXIMUM DRY DENSITY	87.6 LB/cu ft.
OPTIMUM MOISTURE CONTENT	31.1 %
SPECIFIC GRAVITY	2.58
VOIDS	45.5 %
CLAY CONTENT	50.0 %
PLASTIC LIMIT	40. %

X Denotes samples prepared for permeability and shear tests.

No. 911-10 10 x 10 to 1" The A. Lietz Co., San Francisco Made in U.S.A.

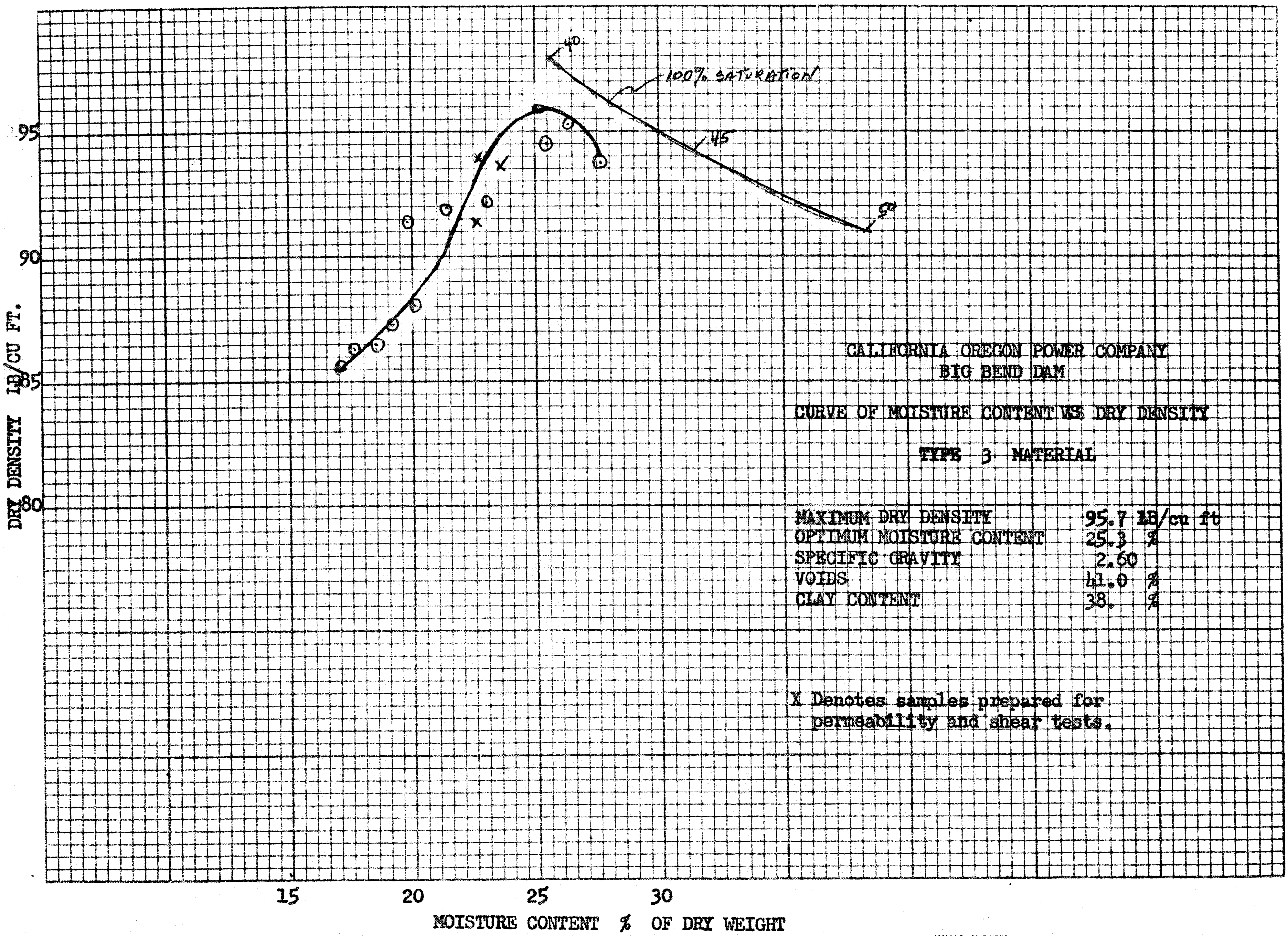
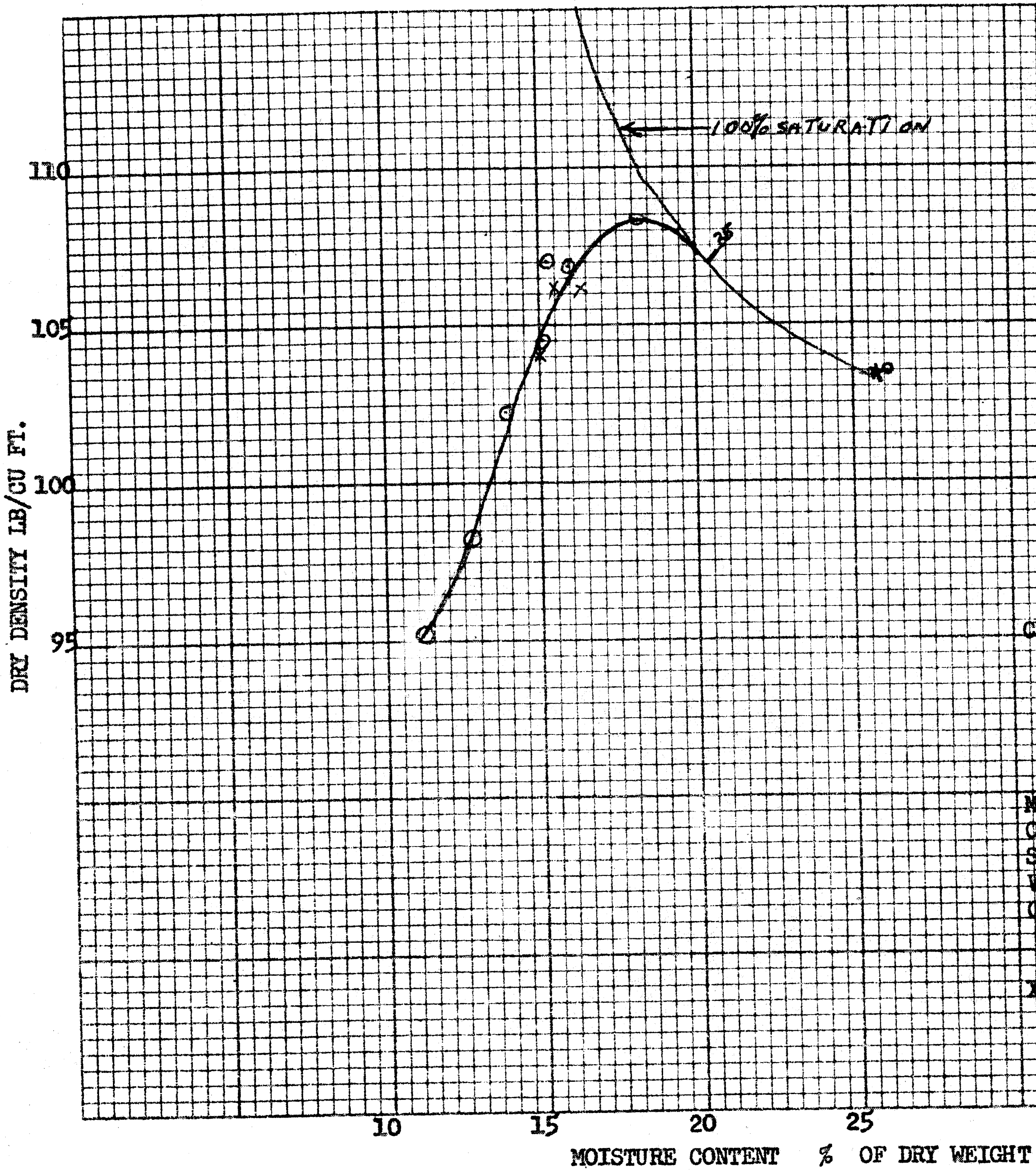


PLATE III - Sheet No. 3

No.811-10 10 x 10 to 1" The A. Lietz Co., San Francisco Made in U.S.A.



CALIFORNIA OREGON POWER COMPANY
BIG BEND DAM

CURVE OF MOISTURE CONTENT VS DRY DENSITY

TYPE 4 MATERIAL

MAXIMUM DRY DENSITY	108.2 lb/cu ft.
OPTIMUM MOISTURE CONTENT	18.0 %
SPECIFIC GRAVITY	2.63
VOIDS	34.0 %
CLAY CONTENT	18. %

X Denotes samples prepared for permeability and shear tests.

No.911-10 10 x 10 to 1"
The A. Lietz Co., San Francisco
Made in U.S.A.

Attachment F Permeability Test Results

	In Place Moisture	Clay Content (Percent)	Specific Gravity of Particles	% Finer than #4 Sieve
Avg.	19.7	35	2.64	51
Max.	26.9	48	2.72	63
Min.	14.5	24	2.58	39

Number of Samples - 14.

Note: Material passing the No. 4 sieve was not plastic. Could not roll 1/8" diameter thread due to sand grains. Specific gravity of the rock (larger than #4 sieve) = 2.64.

The curves showing the moisture content dry density relation of the material finer than the No. 4 sieve are shown on Plate III, Sheet #4. At an optimum moisture content of 18%, the dry density was 108.2 lb/cu ft. with 34% voids. When the material passing the No. 4 sieve and the larger material are combined, the theoretical density can be computed as follows:

$$Drs = \frac{Ds\ Dr}{PD_s + (1-P)\ D_r}$$

Where Drs = Theoretical density of combination
Ds = Density of soil
Dr = Density of rock
P = Percentage of rock (expressed as decimal)

$$Drs = \frac{(108.2)\ (62.4)\ (2.64)}{(\ .49)\ \ (108.2) + (\ .51)\ (165)} = 130 \text{ lb/cu ft.}$$

The above value is a theoretical quantity - which normally cannot be obtained in practice due to the interferance to compaction by the rock. A more practical equation is as follows:

$$Drs = \ (1-P)\ D_s + 0.9\ P\ Dr$$

$$Drs = \ (1-.49)\ 108.2 + 0.9\ \ (\ .49)\ (165) = 128 \text{ lb/cu ft.}$$

II. Qualitative capillarity tests were performed on each of the four types of material. The test consisted of placing a 4" diameter by 4" high compacted sample of the material in a pan of water. In each case the sample of material became saturated in approximately 4 hours. After 48 hours, no sloughing or breakdown of the sample had taken place.

Permeability tests by the falling head method were performed on each type of material by Pittsburgh Testing Laboratories. The permeability was determined to be as follows:

Type 1	-	0.0000187	centimeters per second
" 2	-	0.0000155	" " "
" 3	-	0.000226	" " "
" 4	-	0.00000383	" " "

These co-efficients of permeability fall within the impervious sections of earth dams or dikes as classified by Casagrande and Fadum, Harvard University. The permeability test on Type No. 4 material was performed on the material passing the No. 4 sieve. While this material seems to have the smallest permeability, the gravel content is so high (50%) as to make the overall material questionable as to watertightness.