MASON - de VERTEUIL GEOTECHNICAL SERVICES

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October 16, 1986

Franklin County Engineering Department
970 Dublin Road
Columbus, Ohio 43215

Attention: Mr. Ralph Crabb

Re: Slope Stability Evaluation
Roadway Embankment
Smothers Road at Hoover Reservoir
Franklin County, Ohio
P.O. No. 15506
M V Job No. 86-114

Gentlemen:

In accordance with Agreement No. 7-86, between Franklin County and Mason - de Verteuil Geotechnical Services, dated July 2, 1986, and as authorized by Franklin County Purchase Order No. 15506 dated September 9, 1986 we have completed the reference slope stability evaluation. Submitted, herewith is our report of the evaluation.

We wish to take this opportunity to thank all concerned with utilizing our services. If there are any questions regarding the information contained in this report, or if it is considered we can be of further assistance, please do not hesitate to call.

Respectfully submitted,

MASON - de VERTEUIL GEOTECHNICAL SERVICES

Balraj Varma
Civil Engineer, BSCE

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MASON-deVERTEUIL GEOTECHNICAL SERVICES, 6035 HUNTLEY ROAD, COLUMBUS, OHIO 43229, 614/888-0576

REPORT

OF

SLOPE STABILITY EVALUATION

SMOTHERS ROAD AT HOOVER RESERVOIR

FRANKLIN COUNTY, OHIO

For

FRANKLIN COUNTY ENGINEERING DEPARTMENT
960 Dublin Road
Columbus, Ohio 43215

Prepared by

MASON - de VERTEUIL GEOTECHNICAL SERVICES
A division of Dodson-Lindblom Associates, Inc.
6035 Huntley Road
Columbus, Ohio 43229

October 16, 1986
M V Job No. 86-114

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APPENDICES

1. INTRODUCTION

Around October 14, 1985, slope movements were observed on the Smothers Road embankment at Hoover Reservoir. By mid-November, 1985, these movements had accelerated and were explicitly observed at two locations i.e. between Stations 141+00 and 142+00 on the south side and between Stations 147+50 and 149+50 on the north side.

Information provided by the City of Columbus, Division of Water indicated that the pool level, during the summer and early fall of 1985, had been lowered by approximately 18.5 feet below normal pool, the lowest the pool level had been in several years. This situation was thought to have created a "sudden draw down" condition, resulting in excess pore pressures and unbalanced seepage forces in the embankment, great enough to cause the instability in the slopes.

The purpose of this study has been to:

i) Procure borings along representative sections, where movements were observed, and obtain both split spoon "drive", and shelby tube "undisturbed" samples from the borings.

ii) Perform a laboratory testing program to evaluate parameters representing the material in the embankment and foundation materials.

iii) Perform stability analyses using the computer program STABL2.

iv) From the results of the field investigation, laboratory tests and analyses, deduce the extent and probable causes of the observed movements.

v) Propose measures to control the movement along these slopes.

This report presents details of the investigation conducted and findings thereof, together with recommendations for slope correction.

2. FIELD INVESTIGATION

a. Field Reconnaissance

A field reconnaissance was made by an engineer from Mason - de Verteuil Geotechnical Services in November, 1985, to identify the lateral boundaries of the slides that had already occurred and also to recognize possible areas of further instability, if any.

Primarily, slope failures were observed at two locations on either side of the bridge on Smothers Road. These were located approximately between Stations 141+00 and 142+00 on the southern face of the embankment, west of the bridge, and between Stations 147+50 and 149+50 on the northern face of the embankment, east of the bridge. At that time no clear evidence of any further movement was observed.

At the time the above movements were observed, pool level records kept by the City of Columbus, Division of Water, disclosed the water level of the reservoir pool to be around elevation 874 feet (18.5 feet below normal pool). Subsequently, following a rise in the reservoir level, further movements stopped and the embankment appeared to stabilize.

However, on September 25, 1986, following a report of further movements in the slopes from the Franklin County Engineering Department, the site was revisited by the engineers of Mason - de Verteuil Geotechnical Services. It was observed that the slides at both locations appeared to have extended deeper into the embankment and also stretched longitudinally. The pool level at this time was reported to be around elevation 881 feet.

The geometry of the slides as observed and measured are presented on the Site Plan of Appendix A and Cross Sections in Appendix C.

b. Preliminary Information

Based on original construction drawings, construction drawings for repairs performed in 1979, pile driving data for piles driven for the bridge in 1954, report of investigation - Movement of Substructure Elements, by Mason and Ray, Inc., March 1978, and records of pool elevation, City of Columbus, Divison of Water, the following information was gathered:

i) Normal pool elevation for the Hoover Reservoir is 893.00 feet.

ii) Lateral movement of the forward abutment has been documented previously, necessitating repairs.

iii) The embankment fill is comprised primarily of fine grained, cohesive, silty clay soils with varying percentages of sand and gravel. The fill materials were obtained from borrow areas within the present pool.

iv) The natural soil underlying the embankment fill is largely glacial till for approximately 15 feet, lying over weathered shale bedrock.

v) Based on records kept by the City of Columbus, Division of Water, the pool elevation has been fluctuating at a fairly rapid rate since 1955 and has reached a value of approximately 875 feet on many occasions prior to November, 1985. The lowest value recorded was 868.7 feet, in February, 1964.

vi) At the time movement was first observed, i.e., October 14, 1985, the reservoir elevation was 877.90 feet which subsequently fell to 874.5 feet on November 10, 1985 when further movements were observed, i.e. a drop of 18.5 feet in five months.

c. Subsurface Investigation

Following the field investigation, boring locations were established at those sections of the slides where maximum movements were observed to have occurred. Borings B-1, B-2 and B-3 were located at Station 148+40 feet,on the northern face of the embankment.

Boring B-1 was located at the top of the embankment, Boring B-2 in the middle, and Boring B-3 near the bottom. Borings B-4 and B-5 were located 50 feet due east and west on either side of Boring B-2 and at the same lateral distance from the edge of the road as Boring B-2.

Borings B-7 and B-8 were located at Station 141+39.4 feet at the southern face. Boring B-6 was not procured because of access problems and overhead power lines at the top of the slope in this area.

All the borings, were drilled between June 25, 1986 and August 8, 1986.

Boring B-1 was made by means of a truck mounted drill rig. All other borings were drilled from a barge as their locations fell within the reservoir pool. Borings were advanced through soil using 3¼-inch I.D. hollow stem augers. The embankment material was sampled continuously down to the rock line or bottom of boring. The soil samples were obtained by driving a 2-inch O.D. split-barrel sampler in accordance with ASTM D1586 (Standard Penetration Test). At locations of borings B-1 and B-5, a second hole was drilled adjacent to the borings and a 3-inch O.D. Shelby tube was pushed in order to obtain "undisturbed" samples.

All split-barrel soil samples were examined in the field, and representative portions were preserved in air-tight jars. The Shelby tube samples were preserved in the tubes by first cleaning the ends and then sealing them with wax.

In the laboratory all samples were examined and visually classified by a soils engineer. Boring logs have been prepared on the basis of the driller's field record of drilling and sampling, and the soil engineer's examination and visual classification of samples procured. The boring logs are presented in Appendix A of the report.

The location of the borings are shown on the Site Plan in Appendix A of the report.

3. LABORATORY TESTING

The natural moisture content of all cohesive soil samples was determined, and Atterberg limits and strength tests were performed on representative soil samples.

A total of three (3) consolidated - undrained triaxial compression tests, with pore pressure measurements, were performed on representative "undisturbed" samples of the embankment material, in order to determine the total and effective stress parameters. In these tests the samples were saturated using a back pressure.

One (1) direct shear test was performed and a residual strength value of the sample was determined by first shearing the sample to the practical limits of the machine to determine the peak strength and then returning the specimen to its original position and re-shearing it.

The results of the laboratory testing is presented in the Summary of Laboratory Test Data, Triaxial Test Results and Direct Shear Test Results in Appendix B of the report.

4. FINDINGS OF BORINGS

The following generalized soil profiles were encountered by the borings.

A. North slope, Station 147+50 feet to Station 149+50 feet.

Embankment

The embankment material primarily consisted of stiff to very stiff brown and gray silty clay (CL),with varying percentages of sand and gravel. Zones of very soft to soft silty clay of thicknesses 1.5 feet to 4 feet were encountered at several levels between elevation 874 feet to elevation 845 feet. These layers at specific boring locations may indicate probable locations of failure planes. Some zones of hard silty clay were found within the embankment, but these zones tended to appear closer to the centerline of the embankment. Several layers of organic contaminated soils were also encountered in the embankment fill.

Foundation

The natural soil foundation material was encountered at levels ranging from elevation 843 feet to elevation 850 feet. The materials consisted primarily of stiff to hard, silty clay (CL), containing varying percentages of sand and gravel, and zones of more silty material. A layer of dense to very dense sand and gravel was also encountered in two of the five borings.

Bedrock

All borings except Boring B-1 were drilled to the approximate bedrock level,which ranged from elevation 824 feet to elevation 829 feet. Though no coring was done into the bedrock, the material appeared to be weathered, soft, carbonaceous shale. The material type and respective elevations for probable bedrock observed, are in agreement with the elevations shown in the original construction drawings referred to earlier in this report.

B. South slope Station 141+00 to 143+00

Embankment

The material encountered at this location was similar in content and nature to that encountered by the borings drilled between Stations 147+50 and 149+50. However, no soft layers were encountered at this location.

Foundation

The natural soil foundation material was encountered at levels ranging from elevation 852 feet to elevation 855 feet. The material consisted of stiff to very stiff, silty clay with varying percentages of sand and gravel. Some zones of hard silty clay were also encountered, along with pockets and seams of fine sand and silt. Underlying the above deposits was a layer of medium dense to very dense sand and gravel, with varying percentages of silt. Borings were terminated within this layer of materials since the standard penetration test blow counts obtained during drilling, were consistently above 50 blows per foot.

Bedrock

Borings were not advanced to bedrock level as very high standard penetration test blow counts were obtained within the foundation material itself.

The elevation of bedrock assumed for purposes of analysis has been obtained from the original construction drawings.

For more detailed information as to the findings of the borings, please refer to the Boring Logs in Appendix A of the report.

5. SHEAR STRENGTH RESULTS AND STABILITY ANALYSES

Shear strength parameters, for the portion of the embankment that had not failed, were determined from the consolidated-undrained triaxial compression test results. The residual shear strength of the failed portion of the embankment was determined from the direct shear test results. Also a probable lower limit for the residual parameters (i.e. $c' = 0$, $\phi' = 24°$) was assumed, based on data presented in References 2 and 3.

Strength parameters for the natural soil foundation materials were assumed, based upon interpretation of the boring data and from parameters considered for the foundation material in the report entitled, Hoover Dam Safety Upgrading, by Burgess & Niple, Limited, February, 1983. (Reference 4).

The strength parameters for the rip rap and the proposed rock fill slope correction, were assumed based on typical values used by the U.S. Army Corps of Engineers. (Reference 5).

The strength parameters used in the analyses are presented in Tables, No. 1A and 1B, together with the factors of safety obtained for the various loading conditions analyzed.

Stability analyses were performed on critical sections, in the area where maximum slope movement was observed, on both the north and south slopes, using the STABL2 computer program. This slope stability program permits the analysis of circular as well as non-circular failure surfaces. The program analyzes the slope and determines the ten most critical failure surfaces, unless the analysis of a particular failure surface is specified.

The earth embankment was analyzed under loading conditions of (1) normal pool, elevation 893 feet, and (2) sudden drawdown from elevation 893 feet to 878, 874, 847 or 850 feet. The sudden drawdown elevations of 878 feet and 874 feet were chosen as these were the pool levels recorded when the slope movement was observed. The sudden drawdown elevations of 847 and 850 feet were chosen as representing the worst cases of sudden drawdown arising out of emptying of the reservoir.

The slopes were analyzed assuming (1) the pre-slide profile, (2) the failed profile, and (3) the failed profile with the rock fill correction. The residual strength parameters obtained from the direct shear test appear to be somewhat high as compared with typical values obtained for similar soils as reported in references 2 and 3. For this reason case (2) and case (3) were analyzed using both, the residual shear strength parameters obtained from the direct shear test and, as a worst case, the probable lower limit assumed from information reported in references 2 and 3.

The results of the stability analyses for the two slide areas are summarized in the following Tables, No. 1A and 1B.

A graphical presentation of the computed failure surfaces, together with the estimated actual failure surface determined from field measurements and from the boring data are presented in Appendix C. The computer printout showing the factor of safety for the various loading conditions analyzed, is presented in Appendix D of the report.

Table No. 1-A

STABILITY ANALYSES ON NORTH SLOPE (Station 148+40 feet)

No.	Loading Condition		Strength parameters used in analysis: Rip-Rap	Embankment: Peak	Embankment: Residual	Foundation	Corrective Rock Fill	Minimum Factor of Safety
	Pre-slide Profile							
1.	Normal pool - 893 feet	γ (pcf)	115	132		130		1.42
2.	Sudden drawdown - 878 feet	ϕ' (°)	38	31		34		1.04
3.	Sudden drawdown - 874 feet	c'(psf)	0	100		0		0.99
	Failed Profile							
4.	Normal pool - 893 feet	γ (pcf)	115	132	130	130		1.36
5.	Sudden drawdown - 881 feet	ϕ' (°)	38	31	30	34		1.03
		c'(psf)	0	100	0	0		
6.	Normal pool - 893 feet	γ (pcf)	115	132	130	130		1.05
7.	Sudden drawdown - 881 feet	ϕ' (°)	38	31	24	34		0.80
		c'(psf)	0	100	0	0		
	Corrected Profile - 3:1 slope							
8.	Normal pool - 893 feet	γ (pcf)	115	132	130	130	115	1.73
9.	Sudden drawdown - 862 feet	ϕ' (°)	38	31	30	34	38	1.24
10.	Sudden drawdown - 847 feet	c'(psf)	0	100	0	0	0	1.49
11.	Normal pool - 893 feet	γ (pcf)	115	132	130	130	115	1.40
12.	Sudden drawdown - 862 feet	ϕ' (°)	38	31	24	34	38	1.00
13.	Sudden drawdown - 847 feet	c'(psf)	0	100	0	0	0	1.24
	Corrected Profile - Berm with 3:1 slope							
14.	Normal pool - 893 feet	γ (pcf)	115	132	130	130	115	1.87
15.	Sudden drawdown - 870 feet	0'(°)	38	31	30	34	38	1.38
16.	Sudden drawdown - 847 feet	c'(psf)	0	100	0	0	0	1.65
17.	Normal pool - 893 feet	γ (pcf)	115	132	130	130	115	1.53
18.	Sudden drawdown - 870 feet	0'(°)	38	31	24	34	38	1.13
19.	Sudden drawdown - 847 feet	c'(psf)	0	100	0	0	0	1.39

Table No. 1-B

STABILITY ANALYSES ON SOUTH SLOPE (Station 141+39 feet)

No.	Loading Condition	Strength parameters used in analysis						Minimum Factor of Safety
			Rip-Rap	Embankment		Foundation	Corrective Rock Fill	
				Peak	Residual			
	Pre-slide Profile							
1.	Normal pool - 893 feet	γ (pcf)	115	132		135		1.34
2.	Sudden drawdown - 878 feet	ϕ' (°)	38	31		40		1.02
3.	Sudden drawdown - 874 feet	c'(psf)	0	100		0		0.97
	Failed Profile							
4.	Normal pool - 893 feet	γ (pcf)	115	132	130	135		1.52
5.	Sudden drawdown - 881 feet	ϕ' (°)	38	31	30	40		1.1
		c'(psf)	0	100	0	0		
6.	Normal pool - 893 feet	γ (pcf)	115	132	130	135		1.18
7.	Sudden drawdown - 881 feet	ϕ' (°)	38	31	24	40		0.85
		c'(psf)	0	100	0	0		
	Corrected Profile 3:1 slope							
8.	Normal pool - 893 feet	γ (pcf)	115	132	130	135	115	1.76
9.	Sudden drawdown - 871 feet	ϕ' (°)	38	31	30	40	38	1.42
10.	Sudden drawdown - 850 feet	c'(psf)	0	100	0	0	0	1.70
11.	Normal pool - 893 feet	γ (pcf)	115	132	130	135	115	1.44
12.	Sudden drawdown - 871 feet	ϕ' (°)	38	31	24	40	38	1.16
13.	Sudden drawdown - 850 feet	c'(psf)	0	100	0	0	0	1.42
	Corrected Profile - Berm with 3:1 slope							
14.	Normal pool - 893 feet	γ (pcf)	115	132	130	135	115	1.81
15.	Sudden drawdown - 871 feet	0'(°)	38	31	30	40	38	1.43
16.	Sudden drawdown - 850 feet	c'(psf)	0	100	0	0	0	1.73
17.	Normal pool - 893 feet	γ (pcf)	115	132	130	135	115	1.49
18.	Sudden drawdown - 871 feet	0'(°)	38	31	24	40	38	1.17
19.	Sudden drawdown - 850 feet	c'(psf)	0	100	0	0	0	1.45

6. CONCLUSIONS

Based on the results of the investigation and stability analyses performed, the cause of the slope movement is considered to be due to the repetitive "sudden drawdown" conditions the embankment has been subjected to over the past 30 years, culminating in the sudden drawdown conditions experienced in 1985.

One can only speculate as to why the failures occurred at these two specific locations and not elsewhere along the embankment, or as to why the failures occurred in 1985, when similar drawdown conditions had occurred several times prior to 1985.

This can probably be explained as resulting from the occurrence of successive cycles of instability due to drawdown, followed by stabilization due to rise in the reservoir pool level. The pre-slide minimum factor of safety obtained for conditions of sudden drawdown to elevation 878 feet and 874 feet, is close to unity. Hence, the failure plane may have progressively built up as a result of repetition of such cycles, causing a reduction in the shear strength of the soil with each cycle. The sudden drawdown conditions experienced in October 1985, would have therefore been enough to generate sufficient unbalanced pore pressures, enabling the progressively building failure surface to extend further, thus exceeding the available shear strength of the soil, causing the slide.

It is considered that in time, other areas of the embankment subjected to repeated sudden drawdown conditions may also fail.

The factors of safety obtained using residual strength parameters of $c' = 0$ psf and $\phi' = 30°$ for the failed profile with the normal pool at elevation 893 feet are 1.36 and 1.52 for the north and south slopes, respectively. This would explain why the slopes stabilized once the pool level was raised in November of 1985 and why they continued to move once the pool was lowered again to around elevation 881 feet in September 1986. The minimum factor of safety obtained for sudden drawdown to elevation 881 feet using the same residual strength parameters are 1.03 and 1.1 for the north and south slopes, respectively.

It would be pertinent at this stage to point out that, in practice, the factor of safety cannot be obtained with an accuracy greater than about ±10%. Hence, the above factors of safety for sudden drawdown to 881 feet indicate critical conditions.

The above analyses on the failed profile were repeated using the probable lower limits of the residual strength parameters, i.e. $c' = 0$ psf and $\phi' = 24°$. The minimum factors of safety obtained for normal pool at elevation 893 feet are 1.05 and 1.18, and for sudden drawdown to elevation 881 feet are 0.8 and 0.85, for the north and south slopes respectively. These indicate that for the probable worst case, the failed slopes are presently only marginally stable with the normal pool at elevation 893 feet and are unstable with the pool lowered to elevation 881 feet.

Presently the bridge abutments do not appear to have been affected by the observed slope movements. However, it is worthwhile noting that movement of the abutments has occurred in the past. A report of the investigation of these movements by Mason and Ray, Inc. (reference 6), states that the movements were primarily a result of reduced sliding resistance of the embankment soil. Thus, even though the presently observed slides do not appear to have extended deep enough into the embankment to affect the

For the corrective treatment using the rockfill berms, the most critical cases were found to be, "sudden drawdown" to elevation 870 feet and 871 feet for the north and south slopes respectively. The minimum factors of safety obtained for these slopes using assumed probable lower limit residual parameters were 1.13 and 1.17 respectively.

The actual field residual stength parameters are expected to be higher than the assumed probable lower limit values. Hence, it is felt that the actual factors of safety for the two slopes will be higher than the values stated above.

The rockfill berm configuration will require a smaller quantity of rockfill material than the uniform 3:1 slope correction. This factor coupled with the higher factors of safety obtained, make the rockfill berm configuration a better alternative.

(WAS NOT DONE)

It is also proposed that the rockfill correction be applied to the slope in front of the abutments. Although no slope failure is presently visible in front of the abutments, the possibility of a slope failure occurring in the future exists, which may jeopardize the abutments. The construction of the rockfill correction in front of the abutments at this time, would minimize the risk of this occurring.

Relative to the rockfill, the following is recommended:

1. The corrective treatment should be applied to the slope in front of each abutment and should extend all the way around to station 149+50 on the north slope and to station 140+50 on the south slope.

2. The rockfill should comprise of a free draining material similar to the existing rock rip rap.

3. The rockfill should be placed in horizontal layers starting at the bottom of the embankment. The bottom part of the rockfill should first be placed under water to the proposed slope over the entire length where such correction is required. Only on completion of this stage should rockfill correction for the upper part of the embankment be placed.

It should be noted that other areas of the embankment having an arrangement similar to those where failures have occurred, have been subjected to similar drawdown conditions. It is therefore possible that other areas may also exhibit movement in the future. In light of the above, consideration may also be given to providing a stabilizing rockfill at the bottom of such other areas, as a preventative measure.

Several other configurations for the corrective rockfill have been analyzed. Should you desire these analyses, we would be happy to provide them.

REFERENCES

1. U.S. Army Corps of Engineers, "Engineering and Design, Stability of Earth and Rock-fill Dams", Engineer Manual EM1110-2-1902, April 1970,

2. George A. Hall "The Development of Design Criteria for Soil slopes on West Virginia Highways", Ph.D Thesis 1974.

3. J. M. Duncan "Methods of Analyzing the stability of Natural Slopes", 17th Annual Ohio River Valley Soil Seminar", October 1986.

4. City of Columbus Division of Water "Hoover Dam Safety Upgrading Capital Improvement Project Number 982-244". February 1983.

5. U. S. Army Corps of Engineers, Pittsburgh District. "Grays Landing Lock and Dam Monongahela River, Design Memorandum No. 3", July 1986.

6. Mason and Ray, Inc., "Movement of Substructure Elements, Franklin-Delaware County Line Road Bridge over Hoover Reservoir, Report of Investigation" submitted to Franklin County Engineering Department, March 1978.

APPENDIX A

SITE (BORING) PLAN

LEGEND - BORING LOG TERMINOLOGY

BORING LOGS - 7 BORINGS

bridge abutments, the possibility of some slope movements in front of the abutment, having already occurred or possibly occurring at a later date, cannot be ruled out. Both the bridge abutments are founded on piles, which have been terminated mostly in the embankment fill itself, while few of the piles extend a few feet into original soil, as reported by Mason and Ray, Inc. (reference 6). The stability analyses performed, indicate that the failure surface could extend to the original foundation soil level. Hence, if further movements were to occur in the slope in front of the abutments, they would result in a movement of the abutments.

7. SLOPE CORRECTION

Several methods of slope correction have been considered, namely:

1. Removal of the failed area of the slope and reconstructing the embankment with flatter slopes. In order to do this, the reservoir would have to be emptied.

2. Construction of a retaining wall at the top of the slope to retain the roadway and allowing the slope to continue to move below the wall until it reaches a state of equilibrium. However, economically, the feasibility of this treatment is considered to be questionable.

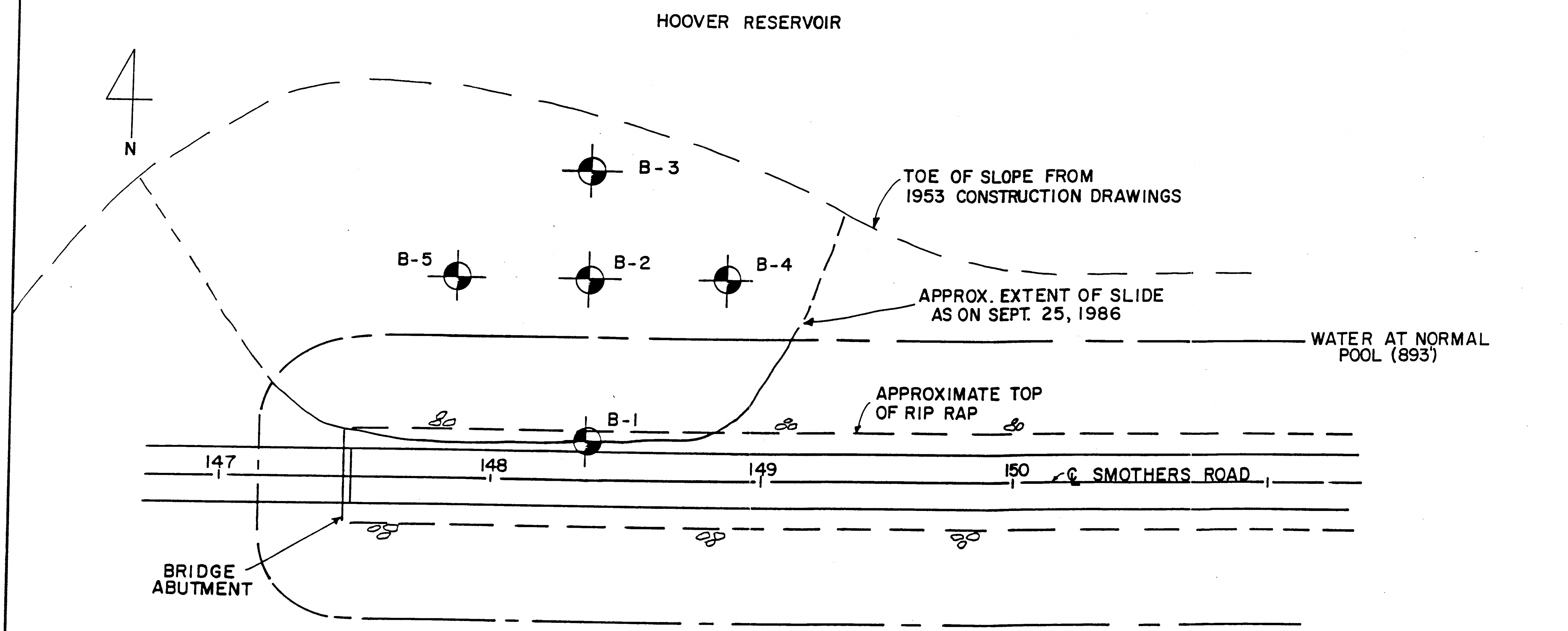
3. Uniformly flattening the existing failed slope from top to bottom, using a rock fill to a slope of approximately 3 horizontal to 1 vertical (3:1), as shown in Appendix C.

4. Providing a rock fill slope with a berm at elevation 885 feet, the portion below this level being at a slope of 3 horizontal to 1 vertical. Specific arrangements for this type of corrective treatment for both north and south slopes are shown in the sections presented in Appendix C.

The first two methods of slope correction are however not considered to be either practically or economically feasible, whereas the last two can be applied without having to empty the reservoir.

The treatments proposed in (3) and (4) above were analyzed for "normal pool" and "sudden drawdown" conditions using both measured and assumed probable lower limit residual strength parameters. Sudden drawdown elevations giving the lowest factors of safety have also been established for each type of corrective treatment. The minimum factors of safety obtained in the analyses are presented in Tables 1A and 1B.

For the corrective treatment involving uniform flattening to a 3:1 slope using rockfill, the most critical cases were found to be "sudden drawdown" to elevation 862 feet and 871 feet for the north and south slopes respectively. The minimum factors of safety obtained, using assumed probable lower limit residual strength parameters were 1.0 and 1.16 for the north and south slopes respectively, which are slightly less than a minimum value of 1.2 generally recommended for sudden drawdown from normal pool for dam embankments (Reference 1).



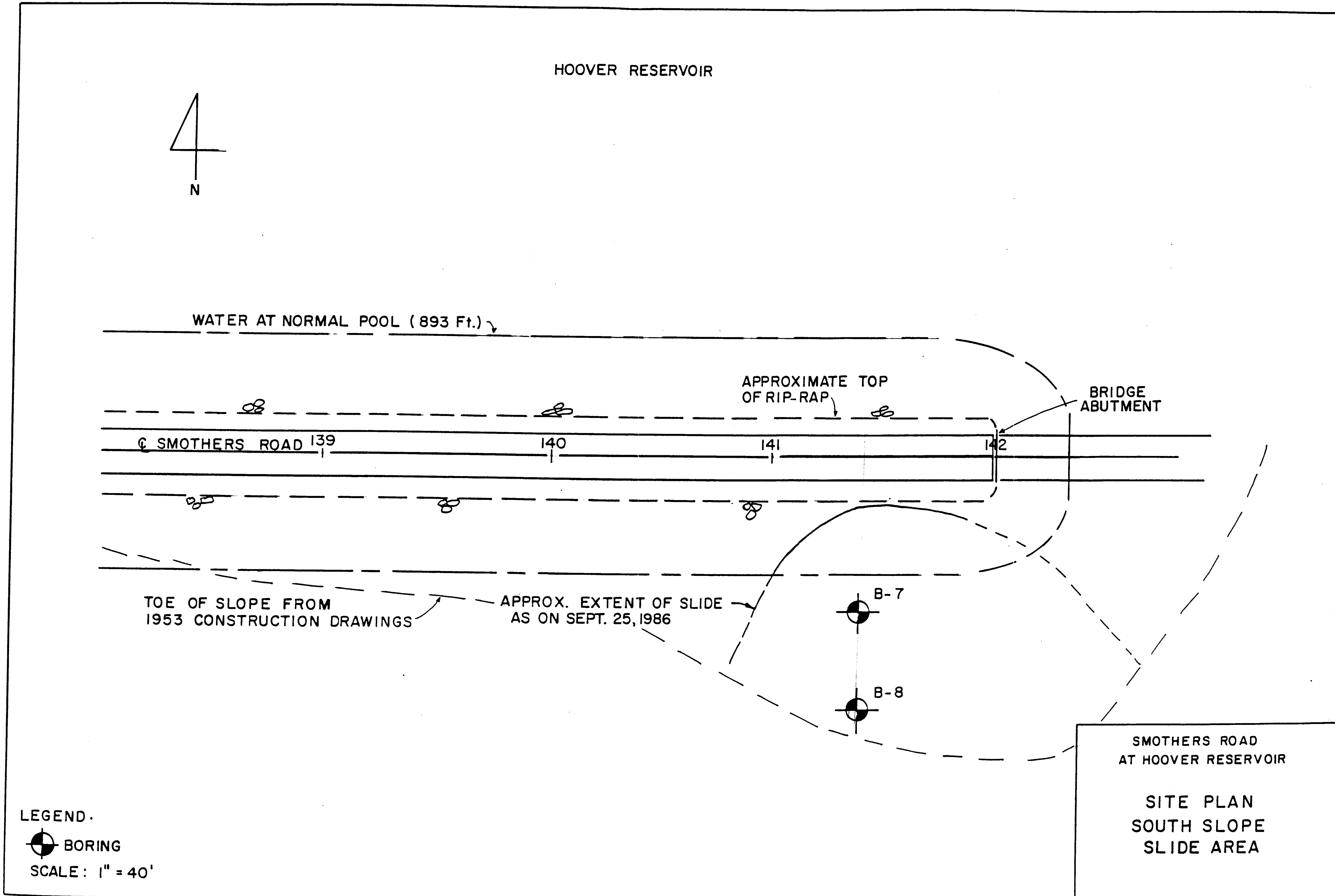
LEGEND:

BORING

SCALE: 1" = 40'

SMOTHERS ROAD
AT HOOVER RESERVOIR

SITE PLAN
NORTH SLOPE
SLIDE AREA


HOOVER RESERVOIR
N
WATER AT NORMAL POOL (893 Ft.)
APPROXIMATE TOP OF RIP-RAP
BRIDGE ABUTMENT
℄ SMOTHERS ROAD
139
140
141
142
TOE OF SLOPE FROM 1953 CONSTRUCTION DRAWINGS
APPROX. EXTENT OF SLIDE AS ON SEPT. 25, 1986
B-7
B-8
LEGEND.
BORING
SCALE: 1" = 40'
SMOTHERS ROAD AT HOOVER RESERVOIR
SITE PLAN
SOUTH SLOPE
SLIDE AREA

LEGEND - BORING LOG TERMINOLOGY

Explanation of each column, progressing from left to right.

1. Depth (in feet) - is distance below the ground surface.

2. Elevation (in feet) - is referenced to mean sea level, unless otherwise noted.

3. Standard Penetration (N) - the number of blows required to drive a 2-inch O.D., 1-3/8 inch. I.D., split-spoon sampler, using a 140 pound hammer with a 30-inch free fall, recorded for 6-inch drive increments. Standard penetration resistance is based on total number of blows required for one-foot of penetration.

4. Length of sampler drive is indicated graphically by horizontal lines across the "Standard Penetration" and "Recovery" columns.

5. Recovery from each drive is indicated numerically, in the column headed "Recovery".

6. Drive sample location is designated by the heavy vertical bar in the "Sample No., Drive" column.

7. Length of hydraulically pressed "Undisturbed" sample is indicated graphically by horizontal lines across the "Press" column.

8. Sample numbers are designated consecutively, increasing in depth.

9. Description

   a. Moisture content of cohesive soils (silts and clays) is expressed relative to plastic properties:

Term	Relative Moisture or Appearance
Dry	Powdery
Damp	Moisture content slightly below plastic limit
Moist	Moisture content above plastic limit, but below liquid limit
Wet	Moisture content above liquid limit

   b. Moisture content of cohesionless soils (sands and gravels) is described as follows:

Term	Relative Moisture of Appearance
Dry	No moisture present
Damp	Internal moisture, but none to little surface moisture
Moist	Free water on surface
Saturated	Voids filled with free water

   c. Texture is based on the Unified Classification System. Soil particle size definitions are as follows:

Description	Size	Description	Size
Boulders	Larger than 8"	Sand - Coarse	4.76 mm. to 2.00 mm.
Cobbles	8" to 3"	- Medium	2.00 mm. to 0.42 mm.
Gravel - Coarse	3" to 3/4"	- Fine	0.42 mm. to 0.074 mm.
- Fine	3/4" to 4.76 mm.	Silt	0.074 mm. to 0.005 mm.
		Clay	Smaller than 0.005 mm.